Banana Shire Council

Biloela WTP

Planning Report

June 2009

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Overview

This report outlines the investigations undertaken by City Water Technology on the Biloela WTP. Issues and outcomes from the study are summarised below.

Water Demands and Required WTP Production

Current water demands on the WTP include the combined town demands plus a 2.5 ML/d allocation to Callide power station. The water demands and WTP capacity were investigated for current conditions and for a number of future scenarios, representing the impacts on WTP demand of town bore availability and of a possible additional 1 – 2 ML/d demand from the Biloela meat works. The findings on current and ultimate required WTP production capacity are outlined in the table below.

WTP Demands and Flow Requirements Summary

Scenario	WTP Production/ Demand Conditions	WTP Treated Water Production Capacity* (ML/d)	Associated WTP Inlet Flow Rate (L/s)**
Current Typical WTP Demand	Typical 2009 WTP treated water output	3.5	-
Current WTP	Typical raw water quality	7.8 – 8.4	120 - 130
	Poor raw water quality	5.2	80
Ultimate WTP Production Demand **Scenario 1**	Ultimate town development, with town bore output available to complement WTP output	10.2	160
Ultimate WTP Production Demand **Scenario 2**	Ultimate town development, with town bores producing zero output	12.1	190
Ultimate WTP Production Demand **Scenario 3**	As for Scenario 2, plus extra meat works demand of 1-2 ML/d	13.1 – 14.1	210 - 220

*For ultimate demand scenarios, required treated water production capacity calculated using MDMM basis, as detailed in section 2.2.2. Alternative basis for capacity calculation is discussed in section 2.2.2.

**Allows for 10% water losses through the WTP process (approx. 5% average losses are expected) and maximum 20 hours WTP operation per day.

It is noted that permanent water restrictions may potentially reduce the future WTP demands for all scenarios, however the effect of potential water restrictions has not yet been quantified.

Water Quality Issues

A review of the raw water and WTP treated water quality found that:

- Callide Dam raw water is generally of good quality, with the main problems being algal blooms, taste and odour compounds and the occasional high turbidity event from significant rain water inflows into the dam;
- WTP treated water typically meets target values, but with periodic excursions. Filtered water turbidity is sometimes above target levels. Final treated water chlorine residuals are highly variable and are sometimes not adequate to maintain a residual at Callide Township (supplied directly from the clear water tank);

- Town bore water, added to the water supply downstream of the WTP, is low in turbidity but has sometimes had elevated TDS and nitrate levels;
- The combined town water has historically been variable due to the changing blends of WTP treated water and town bore water. It has sometimes had elevated TDS and nitrate. Chlorine levels after re-chlorination at the town reservoir may not always be adequate, especially at system extremities. Coliforms have been present in a number of samples, but E.Coli has not been detected.
- From modelling of corrosivity potentials, the bore water is generally likely to be corrosive. The WTP treated water is not likely to be corrosive except during dam fresh water inflows.

WTP Process, Chemical Systems and Operational Issues

A review of the WTP treatment processes and chemical dosing systems found that:

- Most of the plant components are matched well in terms of meeting the 120 L/s design flow. Under very dirty raw water conditions or the presence of algal cells, the clarifiers may limit plant throughput to 80 L/s due to poorer settling;
- In terms of increasing the WTP inflow rate above 130 L/s, the limiting components are the raw water pumps, clarifiers, filters and potentially some of the inlet components and the supernatant return pumps. The clear water tanks, sludge lagoons and flash mixing tank may be suitable to service significantly higher WTP flow rates;
- Of the chemical dosing systems, the alum and PAC systems are somewhat undersized at the current WTP rate of 120 L/s. Most of the other chemical systems would service WTP flows up to 200 L/s. The lime and WTP pre-chlorine would service WTP flows up to 240 L/s at likely maximum doses;
- A new fluoride dosing system either at the WTP or at the town reservoir is required to meet the Qld government requirement for fluoridation by 2012;
- Plant control and automation, safety and maintenance issues were also reviewed, with recommendations outlined below.

WTP Upgrade Requirements and Cost Estimates

Upgrade of the capacity of the WTP to meet future demand scenarios would require the addition of WTP Stage 2 process components as well as upgrade of other equipment to achieve the treated water outputs required by future demand scenarios. Stage 2 treatment process options considered include conventional treatment with settling (similar to Stage 1 process), and an alternative of dissolved air flotation (DAF) clarification. It is recommended that Council consider the DAF process for Stage 2 upgrades because of potential lower capital costs, particularly for a 'DAF above filters' design, and its suitability for the removal of algal cells. Design to allow bypass of the clarification stage to run in direct filtration mode is also recommended if practical to potentially achieve lower operational costs during good raw water quality.

WTP capacity upgrade options to achieve the target flow rates and associated capital cost estimates are summarised in the following table. The costs given in this table allow for the required upgrades for the raw water pumps, raw water mains, WTP inlet components and flash mixing tank, Stage 2 clarifiers and filters, filtered water well, plus the wastewater and lagoon supernatant recycle systems and chemical dosing systems. Allowances have been added for commissioning, O&M manuals, engineering, design, project management and contingency.

Capacity Upgrade Options and Cost Estimates Summary

Scenario	Design WTP Inlet Flow Rate (L/s)*	Stage 2 Clarification Option	Estimated Costs for All Capacity-related Upgrades
Ultimate WTP Production Demand **Scenario 1**	180	Settling Clarifiers	$ 2.8 million
		DAF above Filters	$ 2.6 million
Ultimate WTP Production Demand **Scenario 2**	210	Settling Clarifiers	$ 3.7 million
		DAF above Filters	$ 3.55 million
Ultimate WTP Production Demand **Scenario 3**	240	Settling Clarifiers	$ 5.0 million
		DAF above Filters	$ 4.3 million

* Design WTP inlet flow rates selected to allow modular design, with Stage 2 component capacities matching Stage 1 component capacities where practical. Note that flow rate capacities are for typical raw water quality, with down-rating of Stage 1 components potentially required in poor raw water quality conditions.

Under the higher ultimate demand scenarios, the rate of water transfer between the WTP and the town reservoirs would also need to be increased to keep up with demand. Trunk main upgrade options to increase the transfer are summarised in the following table with their associated cost estimates. Costs include the provision of a booster pump station and increase of the hydraulic capacity of the ground level inlet mixing tank, plus allowances for engineering, project management and contingency. It is noted that these costs may be avoided if an alternative hydraulic solution is found to increase the water transfer rate into town.

Treated Water Trunk Main Upgrade Options and Cost Estimates Summary

Scenario	Design WTP Inlet Flow Rate (L/s)*	Estimated Costs for Trunk Main Upgrades
Ultimate WTP Production Demand **Scenario 1**	180	$ 310,000
Ultimate WTP Production Demand **Scenario 2**	210	$ 340,000
Ultimate WTP Production Demand **Scenario 3**	240	$ 365,000

As seen above, the extent of capacity upgrades required at the Biloela WTP depends on the ultimate demand level used for design. Water efficiency improvements would potentially reduce the size and cost of the ultimate capacity upgrade requirements. It is recommended that Council discuss the demand scenarios further based on the information and costs given in this report to identify the most suitable demand basis for design of the upgraded WTP.

Various other recommended upgrade tasks additional to the capacity upgrade requirements were identified during the course of this study. Those recommended to be pursued with high priority are summarised in the following table with their associated cost estimates. For further details on these upgrades and the cost estimates, refer to Section 7 and 8 of this report.

Additional Priority Upgrade Options and Cost Estimates Summary

Upgrade Task	Summary	Estimated Cost
Short Timeframe Actions		
Algal toxins analysis	Analysis of water for algal toxins if potentially toxic species detected	$5,000
Taste and odour compound analysis	Analyse water when taste and odours are present to identify taste and odour compounds	$5,000
Chlorine residual meter	Connection of existing installed meter to SCADA system, with dial out alarm	$10,000
Clarifier access walkways	Designed to allow access to clarifier surface, to meet OH&S requirements	$45,000
Second clear water tank	Identical size to existing clear water tank	$1,000,000
Standby cationic polydadmac dosing pump	Capacity similar to existing duty pump. Purchased but not installed	$6,000
Dirty water jar testing	Jar testing simulating dirty raw water to optimise coagulation and settling during high turbidity periods	$10,000
Medium Timeframe Actions		
Upgrade PAC system	New bulk bag system sized to dose at least 60 mg/L into required WTP flow rate	$ 540,000
PAC jar testing	Jar testing of PAC products to compare their effectiveness and contact time requirements with compounds of interest	$15,000
Alternative to PAC: Ozone BAC system	Ozone generator, contact tank, BAC filters and associated equipment for 120 – 240 L/s	$ 5.8 – 7.6 million
Existing alum system screw feeder upgrade	Fit new larger feeder into existing system	$ 30,000
Alternative new liquid alum system	New system including storage tanks, dosing equipment and tanker unloading area	$ 290,000

Further recommended improvement options considered medium or low priority are listed in Section 7 of this report. It is recommended that Council pursue all of these measures as practical in future.

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1. Introduction and Objectives

Banana Shire Council engaged City Water Technology to conduct a detailed review of the Shire's water treatment plants (WTPs), reviewing the treatment plant capacity and addressing planning issues for current and future upgrade requirements. Plant capacity was then compared to future demands and other requirements determined with the assistance of Council. This planning report outlines the findings from the above investigations for the Biloela WTP and sets out options for addressing the upgrade requirements.

The objectives of the review and planning report for each WTP are to:

- Review water demands and required WTP output for present and future development;
- Review raw water quality and treated water requirements;
- Assess the capacity and effectiveness of the WTP, unit processes and chemical dosing systems;
- Identify any other operational and process control issues;
- Identify the capacity and process upgrade requirements to meet current and future demand scenarios and water quality challenges.

2. Plant Flow Rate and Demand Issues

2.1 Town Supply Water Allocations and Delivery Capacity

2.1.1 Overview of Town Water Sources

The sources of water supplied to the Biloela reticulation system are:

- Callide Dam, treated at the Biloela Water Treatment Plant (WTP); and
- Town bore fields drawing from the Callide Valley Aquifer.

The two water sources are blended in the mixing tank adjacent to the reservoirs on State Farm Rd in Biloela.

2.1.2 Callide Dam System Allocation and Capacity

The Callide Dam has in the past been kept at a level of 20% capacity, giving a storage level of 25,000 ML out of the full dam capacity of 127,000 ML. However in recent years, levels have been below 10% due to ongoing drought. The dam water is topped up via a pipeline from the Awoonga Dam, with 9,000 ML/yr (of the total 14,000 ML/yr pumped through the pipeline) delivered to Callide Dam. Any storage in the Callide Dam in excess of the 20% dam capacity level is normally released to recharge the Callide Valley aquifers.

The Callide Dam raw water is extracted for use at both the Biloela WTP and the Callide Power Station. Under current allocations, the Council can withdraw an allocated 1,200 ML/yr from the dam for town supply, plus a supplementary 400 ML/yr which CS Energy is obligated to supply into Callide Dam (via the Awoonga Pipeline), if required for town supply.

The dam water withdrawn for town supply is treated at the Biloela WTP. Water is pumped up to the WTP site by the two (duty/ standby) raw water pumps. The effective flow rates for the raw water pumps and treatment process are discussed in detail in later sections of this report.

The typical flow through the town supply main from the WTP into the mixing tank near the reservoirs on State Farm Rd is reportedly around 55 L/s. The flow to town from the WTP is adjusted at times to achieve a suitable water quality blend with the available bore water flow.

2.1.3 Callide Valley Aquifer Bores Allocation and Capacity

The Callide Valley aquifer bores are located in Biloela at two sites in town:

- Gladstone Rd: Bore Pumps 4 and 5; and
- Dakenbar-Jambin Rd: Bore Pumps 8, 9, 10 and 11

Bore water from the two bore fields is pumped via two 200mm diameter rising mains to the mixing tank near the reservoirs on State Farm Rd.

The bores draw from the Callide Valley aquifer. The estimated total safe annual yield from this aquifer is 23,000 ML/yr, however the aquifer has been over-allocated. Due to this over-allocation, each individual allocation has been subject to reductions, announced for each financial year. The Council's allocation of water from the bores for town water supply is nominally 987 ML/yr, however a significantly reduced actual allocation has been announced for recent financial years.

The output from the bores varies with the water level in the aquifer. The individual bores can reportedly supply flows in the range 12 – 30 L/s, and 13 L/s is reported to be a typical flow for each bore on its own. The reported combined maximum flow from all the bores is 45 L/s (3.25 ML/d over 20 hours), however with reduced aquifer levels in 2006, the combined flow was around 26 L/s (1.87 ML/d over 20 hours). In 1995/96, historically record low aquifer levels caused complete failure of bores 10 and 11 and significantly restricted flow of the other bores. Current (2009) dry conditions are resulting in the aquifer again reaching record low levels and Council are concerned that the bores may not be viable at all in the future if aquifer levels continue to reduce.

The town bore stations are currently (2009) undergoing a process of upgrading to install new bore pumps and fix bore casings. In some cases, bore holes have had to be redrilled and a new casing inserted due to deterioration of the original casing. Council are also investigating the possibility of alternative aquifer sources.

2.1.4 Summary of Water Source Capacity and Allocations

The annual water allocations and capacities from each water source are outlined in the following table.

Water Source Extraction Capacity and Allocations

<table>
<tr><th>Water Source</th><th>Allocation (ML/yr)</th><th>Maximum Capacity (L/s)</th></tr>
<tr><td>Callide Dam</td><td>1200</td><td rowspan="2">Depends on capacities of:
• Raw water pumps
• WTP
• Treated water mains
As discussed later in this report</td></tr>
<tr><td>CS Energy (Awoonga Dam Pipeline)</td><td>400
(supplementary allocation released into Callide Dam)</td></tr>
<tr><td>Callide Valley Aquifer Bore Fields</td><td>Nominal: 987
2004/05: 790 (80%)
2005/06: 740 (75%)
2006/07: 690 (70%)
2007/08: 640 (65%)
2008/09: 640 (65%)
2009/10: 494 (50%)</td><td>Replenished aquifer: 45 L/s
Severe drought: Limited flow</td></tr>
</table>

As seen in the table, the total annual allocation for town water supply is nominally: 2587ML/yr, however the actual allocation is generally lower than this figure due to announced reduction in bore water allocations and/or low flow rates from the bores.

2.2 Current and Future Water Demand

2.2.1 Demand for Town Water

A Water Supply Planning Report was prepared by SKM in 2006, reviewing water demand in Biloela under current conditions and projecting water demand for the ultimate development levels. An ultimate population (by 2031) in Biloela of 8000 people was assumed for the study. Town water demands were considered for the areas of:

- Biloela;
- Thangool; and

- Callide Township.

The current and future total annual demand levels determined in SKM's study for supply to Biloela, Thangool and Callide Township were:

- Current conditions: 1700 ML/yr;
- Ultimate development: 2332 ML/yr.

It is noted that the ultimate water demand is somewhat higher than the actual water allocations for recent years, therefore a new water source or significant water efficiencies would need to be developed in order for the town to develop past a population of around 8000.

An infrastructure agreement also requires Council to supply up to 2.5 ML/d treated water to the Callide Power Station. It is noted that the usage by the power station is under a separate allocation, i.e. this water usage is not included in the town water annual allocation. Flows to the power station are typically around 33 L/s, however instantaneous flows up to the maximum capped rate of 65 L/s have been recorded.

The Biloela meat works have also made recent enquiries about the use of town water, as their normal bore water source is at low levels. If Council agreed to supply them with water, it is understood that a new mains pipe to the meat works would be connected to the WTP treated water main between the WTP and the main town reservoirs. The proposed supply agreement the meat works is the provision of 1 to 2 ML/d of potable water, although this water may only be required when the meat works bore is not available.

The table below shows the expected average daily demands from the consumers of town water, with the power station included separately. The table is based on figures from the SKM planning report, plus an assumed usage of 2.5 ML/d by the Callide power station and a potential 1 – 2 ML demand from the Biloela meat works.

Current and Full Development - Average Daily Water Demands

Consumer		**Demand – Average Daily (ML/d)**	
		Current	**Full Development**
Biloela Town System:	• Biloela	4.42	6.10
	• Thangool	0.19	0.25
	Total - Biloela and Thangool	*4.61*	*6.35*
Callide Township		0.04	0.04
	Total Demand – Town Supplies	**4.7**	**6.4**
Callide Power Station		2.5	2.5
Total Demand - All Consumers		**7.2**	**8.9**
Biloela Meat Works (proposed)		1 - 2	1 - 2
Total Demand - Including meat works		**8.2 – 9.2**	**9.9 – 10.9**

2.2.2 WTP Treated Water Demand

The WTP is required to supply treated water for the following consumers:

- Town supply:
    - Biloela and Thangool (WTP water plus town bore water);
    - Callide Township (WTP water only); and
- Callide Power Station;
- Proposed future consumer - Biloela meat works.

Council have identified a number of potential future scenarios for the demand on WTP treated water. These are:

- Scenario 1: Development as per SKM modelling, with bores remaining in service to complement the WTP treated water output;
- Scenario 2: Development as per SKM modelling, but town bores unable to provide serviceable flow output due to depleted aquifer;
- Scenario 3: Development as per SKM modelling, but town bores unable to provide serviceable flow output and additional flow demand of 1 - 2 ML/d from meat works.

A fourth scenario was also identified: The impact of permanent water restrictions reducing the future town water demand to less than the SKM predicted levels. However the effect of potential water restrictions has not yet been quantified, so this scenario is not shown in the table.

The demand values under Scenarios 1 to 3 are shown in the table below. An average total town bore flow contribution of 1.87 ML/d was assumed for Scenario 1 calculations, based on the 2006 combined bore output of 26 L/s over 20 hours per day.

Peaking factors for the town demands were calculated as follows, following the Queensland DNRM (2005) approach, as also used in the SKM planning report:

- Mean Day Maximum Month (MDMM) = 1.5 x Average Day Demand
- Max Day (MD) = 1.5 x MDMM = 2.25 x Average Day Demand.

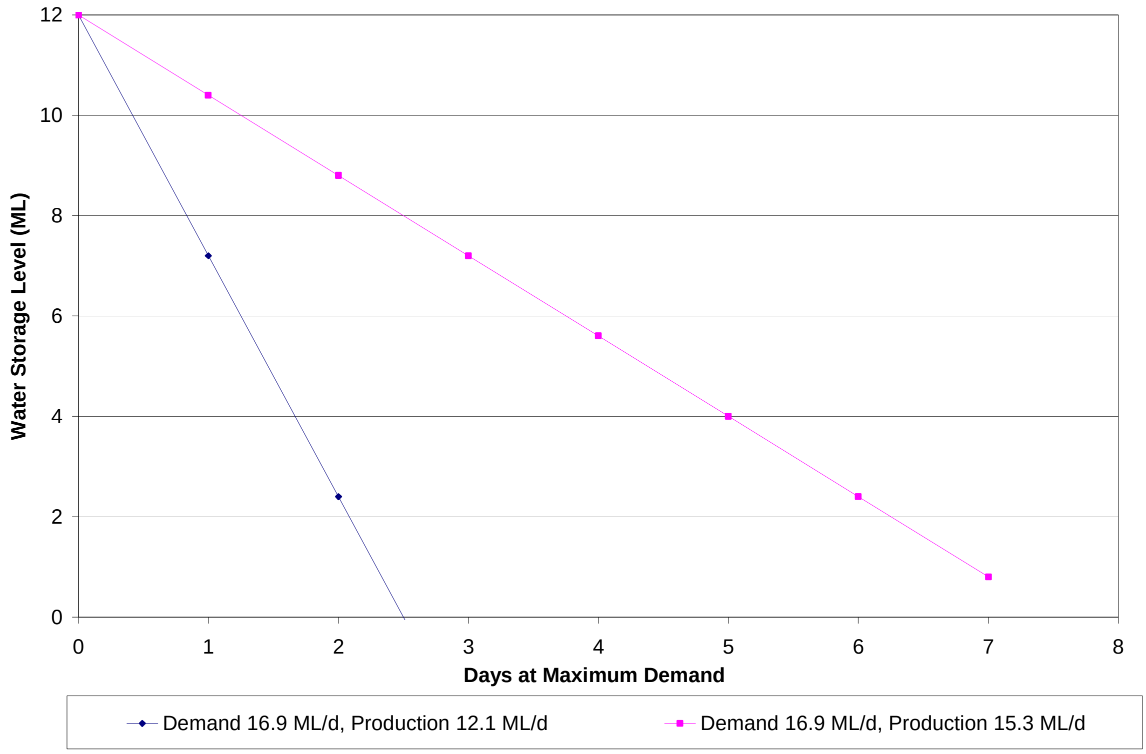
Council have advised that they wish to use the Queensland DNRM rule of thumb, also adopted by SKM (2006), to size the WTP based on producing the mean day max month (MDMM) demand over 20 hours operation per day (equivalent to 1.8 x Average Day on a 24 h/day basis). This approach has been adopted in this report, although it is noted that a more conservative approach is often used, with many WTPs in NSW designed with a capacity of 2 to 3.5 x Average Day. Alternative WTP sizing factors of 2.0 – 2.5 x Average Day for 20 hrs per day basis (equivalent to 2.4 - 3.0 x Average Day on a 24 h/day basis) have also been shown in the table below for comparison.

Full Development Demands for WTP Treated Water

Demand Condition	System Demands (ML/d)			Total WTP Treated Water Demand (ML/d)		
	Town Demand	Power Station Demand	Total Demand (Town + Power Station)	Scenario 1: Total Demands minus Bore Flow	Scenario 2: Total Demands (Town Bores Not Avail)	Scenario 3: Total Demands + Meatworks 1 – 2 ML/d
Average Day	6.4	2.5	**8.9**	7.0	8.9	9.9 - 10.9
Calculated Mean Day Max Month	9.6	2.5	**12.1**	10.2	12.1	13.1 – 14.1
Calculated Max Day	14.4	2.5	**16.9**	15.0	16.9	17.9 – 18.9
WTP Sizing: MDMM demand	9.6	2.5	12.1	**10.2**	**12.1**	**13.1 – 14.1**
Alternative WTP sizing:						
2.0 x AD town demand	*12.8*	*2.5*	*15.3*	***13.4***	***15.3***	***16.3 – 17.3***
2.5 x AD town demand	*16*	*2.5*	*18.5*	***16.6***	***18.5***	***19.5 – 20.5***

As seen in the table above, the WTP sizing basis used has a significant effect on the size of the WTP required. The MDMM basis gives a WTP output significantly less than the calculated maximum day demand, whereas the 2.5 x town demand basis gives a WTP output higher than the calculated maximum day demand.

At the calculated Scenario 2 maximum day demand of 16.9 ML/d, the effects on town storage levels for WTP output capacities of 12.1 ML/d (MDMM basis) and 15.3 ML/d (2.0 x AD basis) are compared in the graph below for reference. The trends shown in the graph are for the scenario where maximum day demand (16.9 ML/d) occurs for several days in a row, with a starting storage level of 12 ML.



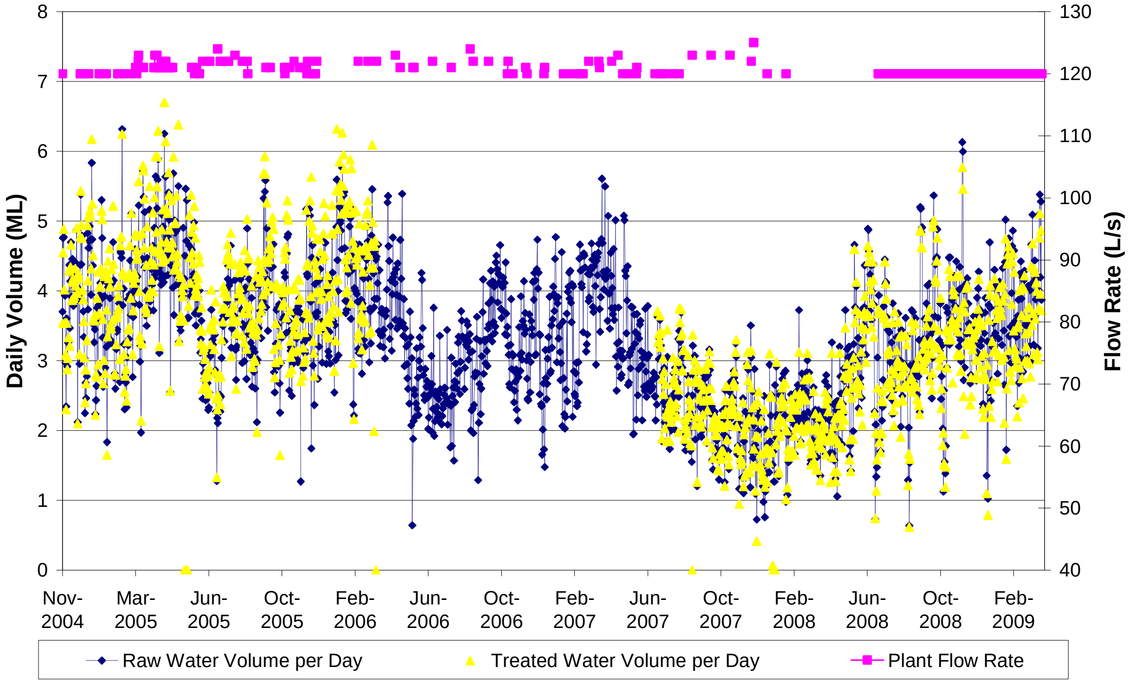
Graph of Storage Trends at Maximum Demand

The graph above shows that at the 12.1 ML/d WTP sizing, a 12 ML storage volume would be exhausted within 2.5 days at sustained maximum day demands. The more conservative 15.3 ML/d WTP sizing would be able to sustain storage levels for up to 7 days at the calculated maximum day demand. Thus the more conservative WTP sizing basis would significantly reduce the load on town storage volumes under maximum demand conditions.

2.3 Current Plant Production and Flow Rates

2.3.1 Plant Production and Flow Rate Trends

The data available electronically on WTP treated daily volumes and flow rates is shown in the graph below. Daily volumes are based on the raw and treated flow meter totalisers. It is noted that the treated water flow meter was out of service during 2006-07.



Graph of Biloela WTP Daily Volumes Treated and Flow Rates

Plant records show that, for the period graphed, raw water volumes varied between 0.6 and 6.3 ML/d. The average volume treated based on the data above is around 3.3 ML/d. This value compared well to data available in the SKM report which gave a total raw water volume of 1333 ML for the year 2004, equivalent to an average daily flow of 3.6 ML/d.

Current (2009) daily flows have averaged around 3.5 ML/d, even with the town bores contributing little flow to the town supply. It is noted that this demand level is much lower than the total demand of 7.2 ML/d calculated by SKM in 2006 for 'current' development conditions (see table above). The lower demand can be attributed to ongoing water restrictions and the power station taking less than its full allocation of 2.5 ML/d.

There was little variation in plant flow rate over the period shown in the graph, with the raw water flow rate varying from 120 to 125 L/s.

The logged plant run hours data indicates that the plant operates for between 1.4 and 14.5 hours per day, and typically around 7.4 hours per day. It is noted that there is additional potential to run the plant for more hours per day to achieve higher outputs. The maximum practical run hours is expected to be around 20 hours per day, to allow the filters to be backwashed while the plant is off line (current typical practise) and to allow for maintenance down time. In worst case demand conditions, the Biloela WTP may be able to run constantly for several days, provided that adequate performance could be maintained while backwashing the filters with the plant online and that there were no critical maintenance issues during the period.

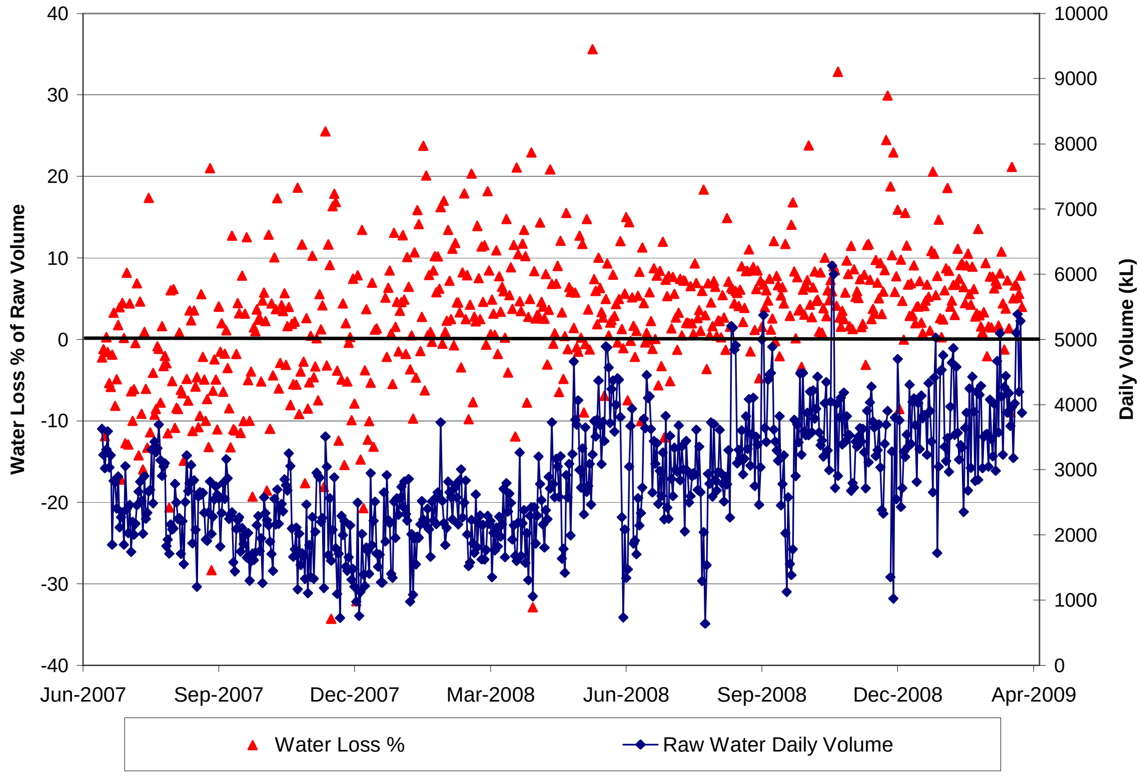
2.3.2 Flow Rate Variation Through WTP Process

The value used as the plant flow rate set point is the raw water inlet flow to the WTP. It is noted that the actual flow rate will vary through the WTP process as water flow is added and removed as follows:

- Plant inlet flow measured on raw water flow meter at inlet to plant;
- Flow added as lagoon supernatant return, just prior to flash mixer;
- Flow removed periodically from clarifiers during desludges;

- Flow removed periodically from clear water tank for filter backwashing;
- Treated water flow measured at outlet of clear water tank.

Plant data for the period 2007-09 was reviewed to look at the difference between daily raw water inflow and treated water outflow. During the prior to 2007, the treated water flow meter was known to be inaccurate and/or was out of service. The calculated 'water loss', raw water minus treated water daily volume as a percentage of the raw water inflow, is shown in the graph below. The daily flow volume is also shown.



The 'water loss' values calculated in the graph above appear to mostly range between 0 and 10%, although significant variation has been seen. Negative 'water loss' values (treated water flow was higher than the raw water flow volume) would occur due to the flow buffering by the clear water tank storage volume.

The 'water loss' values shown in the graph above were compared to the supernatant return and wastewater flows estimated later in this report under Section 4.12, i.e.:

- Estimated supernatant return flow of 5 to 8 L/s, or approximately 4 to 6 % of the raw water flow rate, but supernatant quality sometimes unsuitable for recycling;
- Estimated wastewater flows for desludge and backwashing of around 4 – 8 % of the raw water volume.

The wastewater flows above relate to total water losses of 0 to 8 %, matching the data shown in the graph. It is therefore expected that an average 'water loss' of around 5% would occur through the plant process, i.e. that the treated water output would be roughly 95% of the raw water flow under typical circumstances. If required, the figure could be refined by further examination of raw and treated water flows and consideration of the accuracy of each meter.

2.4 Potential Plant Production and Flow Rates

2.4.1 Operators' Observations

Based on available information and operator comments on historical operations, the following flow rate ranges are expected for the plant:

- Design plant flow of 120 L/s;
- Minimum practical plant flow of 60 L/s (quoted in original operating manual);
- Maximum plant flow of 130 L/s successfully run under normal raw water conditions. Operators reported a rise in sludge blanket and poor settling in the clarifiers when WTP was run at 140 L/s for 24 hours;
- Plant flow required to be down-rated to 80 L/s under high raw water solids loading (e.g. February 2003) to achieve effective settling;
- Plant flow required to be down-rated to 100 L/s under high raw water algal cells levels (e.g. September 2003).

2.4.2 Plant Flow Rate Trial May 2009

A short plant flow rate trial was undertaken in May 2009 to try to confirm the maximum rate of the existing WTP process. The maximum flow rate trialled was 138 L/s (as measured on the raw water flow meter), which was found to be the limit of the raw water pump capacity at the existing low water levels in the dam. This flow rate was trialled for around one hour after which a VSD fault caused the raw water pumps to shut down. The plant was run at a lower flow rate of 130 - 136 L/s for around 3 hours.

Raw water quality during the plant flow rate trial was good, with low turbidity and no algae or taste and odour issues. Each of the WTP filters was freshly backwashed prior to the trial and no PAC dosing was used.

The plant flow rate trial showed that at the elevated flow rates of 130 L/s to 138 L/s:

- No hydraulic limitations through the plant were noted;
- Settling performance of the clarifiers was unchanged in the short term – Some turbidity carryover was noted at initial plant startup, however the operators report that this frequently occurs at normal flow rates;
- The combined filtered water turbidity trend showed no discernable change associated with the higher flow rates.

The flow rate trial indicated that flows of up to 138 L/s could successfully be treated for short periods under ideal raw water conditions. Because of the short duration of the trial, the long term effects of the higher flow rate on clarifier sludge production and filter run times could not be assessed. It is likely that the performance of the clarifiers may deteriorate over time if excess sludge was allowed to build up in them. The filters would also be under more pressure over the long term as the headloss levels increased, and the headloss accumulation rate would be increased if PAC dosing was undertaken.

The likely maximum capacity of each plant process component is discussed further in Section 4 of this report.

2.4.3 Original Design Flow Rate and Up-Rating Plans

The existing Biloela WTP was designed as a first stage, with potential to be upgraded to greater capacity in the future. The Stage 1 design flow rate is 120 L/s. The actual hydraulic flow capacities for each plant component vary and are discussed in later sections of this report.

The main plant up-rating options identified in the original operating manual included:

- Stage 1A: Capacity increase to 160 L/s by up-rating the flow through the existing facilities clarifiers and filters (subject to successful process testing), or
- Stage 2 (option 1): Capacity increase to 240 L/s by adding Stage 2 clarifiers and filters of the same capacity as the Stage 1 components, or
- Stage 2 (option 2): Capacity increase to 240 L/s by up-rating the flow through the existing clarifiers and filters to Stage 1A levels and providing additional Stage 2 components of smaller capacity than the Stage 1 components.

It is noted that the plans including the Stage 1A upgrading option of raising the flow through the existing plant infrastructure were always provisional on the existing clarifiers and filters being able to handle the additional flow, which was to be proved in operational trials and otherwise. Plant trials have not been conducted at 160 L/s, however it is expected that this flow rate would not be able to be achieved without some modifications to the existing process components. The expected process-based capacities of the clarifiers and filters are discussed in more detail in later sections of this report.

2.4.4 Daily Water Production Capacity for Various WTP Flow Rates

The table below summarises the potential maximum daily water output for various WTP inlet flows, based on up to 20 hours WTP operation per day. Figures are shown separately for process water losses of:

- Estimated typical 5% loss (i.e. 95% of inlet flow converted to treated water); and
- Expected long term worst case 10% loss (90% of inlet flow converted to treated water).

Estimated Water Production at Various WTP Flow Rates

WTP Inlet Flow Rate (L/s)	Estimated Treated Water Output (ML/d) with 20 h/day Operation	
	5% Water Loss	10% Water Loss
80	5.5	5.2
120	8.2	7.8
130	8.9	8.4
140	9.6	9.1
150	10.3	9.7
160	10.9	10.4
170	11.6	11.0
180	12.3	11.7
190	13.0	12.3
200	13.7	13.0
210	14.4	13.6
220	15.0	14.3
230	15.7	14.9
240	16.4	15.6
250	17.1	16.2

WTP Inlet Flow Rate (L/s)	Estimated Treated Water Output (ML/d) with 20 h/day Operation	
	5% Water Loss	10% Water Loss
260	17.8	16.8
270	18.5	17.5
280	19.2	18.1
290	19.8	18.8
300	20.5	19.4
310	21.2	20.1
320	21.9	20.7

As seen in the table, the current estimated maximum WTP flow rate of 120 - 130 L/s would give a treated water output of 7.8 – 8.4 ML/d, and the poor raw water quality down-rated flow rate of 80 L/s would give a treated water output of only 5.2 – 5.8 ML/d.

Plant flow rates required to meet the future town demand scenarios are shown in the section below.

2.5 Plant Flow Rate Requirements for Future Demand Scenarios

For each of the future water demand scenarios identified earlier in this report, the table above was used to identify the nominal plant flow rates needed to meet the WTP sizing daily output. The flow rates given below allow for water losses of up to 10% through the WTP process.

WTP Flow Requirements for Forecast Demands

Flow Parameter	Scenario 1: Total Demands minus Town Bore Flow	Scenario 2: Total Demands (Town Bores Not Avail)	Scenario 3: Total Demands + Meatworks 1 – 2 ML/d
WTP Sizing based on MDMM (ML/d)	10.2	12.1	13.1 – 14.1
Required WTP Raw Water Flow Rate (L/s)	160	190	210 - 220
***Alternative** WTP Sizing based on 2.0 x AD (ML/d)*	*13.4*	*15.3*	*16.3 – 17.3*
Required WTP Raw Water Flow Rate (L/s)	*210*	*240*	*255 - 270*

For Scenario 1 and sizing based on MDMM level, the required WTP capacity of 160 L/s in the table above matches that recommended by SKM for this scenario as part of their 2006 investigations.

3. Water Quality Issues

3.1 Overview of Town Water Sources

The sources of water supplied to the Biloela reticulation system are:

- Callide Dam, treated at the Biloela Water Treatment Plant (WTP); and
- Town bore fields drawing from the Callide Valley Aquifer.

The two raw water sources are blended in the mixing tank adjacent to the reservoirs on State Farm Rd in Biloela. Chlorine is dosed to the WTP treated water and also to the combined water in the mixing tank.

Water quality issues are reviewed below for the following water types in the Biloela town supply system:

- WTP raw water from Callide Dam;
- WTP treated water;
- Town bore water; and
- Combined town water

3.2 Biloela WTP Raw and Treated Water Quality

3.2.1 Dam Source and Catchment

Raw water to Biloela WTP is sourced from Callide Dam.

The dam is filled by both:

- Local catchment runoff; and
- Flow pumped from Awoonga Dam through the Awoonga pipeline.

Callide Dam capacity, management and allocation volumes are addressed in the previous section of this report.

The dam catchment contains agricultural land, mainly cattle and horse farming. Industry such as mining and the Callide power station are located close to the catchment.

The water supplied by Callide Dam is reportedly generally of very good quality throughout the year, however algal blooms occur on occasion and taste and odour compounds are a problem.

3.2.2 Raw and Treated Water Quality Monitoring Data

Water quality data graphed below has been mainly taken from the WTP log sheets where it is regularly recorded in electronic format. Samples have also been taken for external laboratory analyses, and the available data from this source has been summarised in Appendix A of this report. Water quality results are discussed below for each relevant water quality parameter.

The ‘worst case’ dirty water quality event recorded for Biloela WTP is the February 2003 ‘fresh’ (rain runoff event) into the dam. This event resulted in elevated raw water turbidities for several days. The data for this period was not available in electronic spreadsheet form, so it has been copied below from paper records kept at the plant.

Records from Poor Raw Water Quality Event 11-14/2/03

Parameter	Units	Raw Water	Treated Water
Turbidity	NTU	116 - 153	0.022
Apparent Colour*	HU	496 - 820	5
pH	-	7.2	7.2 – 8.7
Alkalinity	mg/L as $CaCO_3$	67	49
Iron	mg/L	0.94	0.01
Manganese	mg/L	0.218	0.029
Chlorine	mg/L	-	0.5
Aluminium	mg/L	-	0.03

* True colour not measured

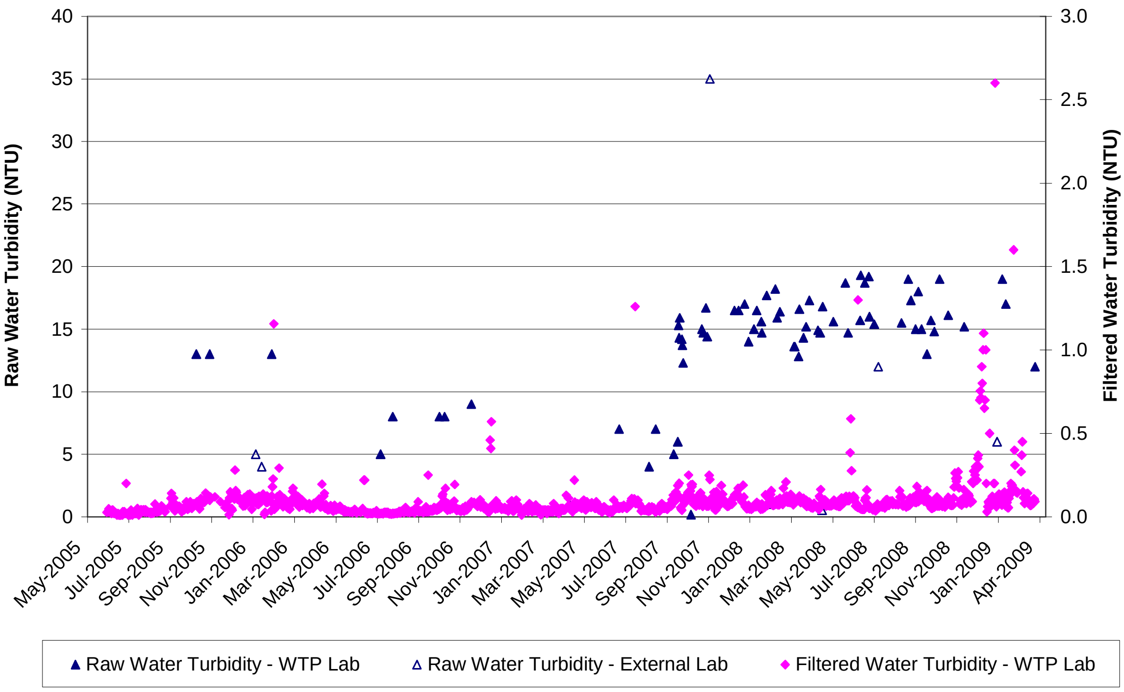
As seen in the table, the ‘fresh’ conditions brought a much higher sediment load into the plant. It appears that the typical treated water turbidity level was able to be maintained under these conditions, although it was understood that the plant flow rate was downrated to around 80 L/s under these conditions. It is understood that the type of event shown above has only occurred very rarely over the history of the WTP. Water quality records were only available for this one ‘fresh’ type event.

Apart from ‘fresh’ events, other problems occurring in the raw water include algal cells and taste and odour compounds as discussed further below. It is also noted that the typical raw dam water can be hard to treat and settle because of the low solids and high organics levels.

3.2.3 Turbidity

The graph below shows the raw and filtered water turbidity for the period 2005 to 2009, based on WTP log sheets with some extra raw water turbidity data added from external laboratory analysis results. It is noted that the graph does not include the worst case raw water turbidity event of February 2003, where turbidities up to 153 NTU were recorded.

The WTP log sheet filtered water turbidity values are recorded daily from the online turbidity meter. It is understood that comparison filtered water grab samples are taken periodically in order to check the calibration of the online meter.



Graph of Raw and Filtered Water Turbidity

The records above show that the raw water turbidity is generally in the range 5 – 20 NTU, although one external laboratory result gave a turbidity of 35 NTU. Rare fresh events may bring turbidities of 153 NTU or more.

Filtered water turbidity values recorded on the WTP log sheets are generally < 0.2 NTU, with several spikes up to approximately 1.5 NTU and a maximum recorded value of 2.6 NTU. The external laboratory analyses for treated water (shown in Appendix) also showed several values of 1 NTU, although it is expected that the treated water turbidity would be higher than the filtered water turbidity because of impurities in the post-filtration lime dose.

Overall, reasonable treated water turbidities appear to be achieved by the plant most of the time, however the ADWG level of < 1 NTU for effective disinfection was exceeded on some occasions. The ADWG turbidity level of <1 NTU is based on disinfection efficiency, as the particulates associated with turbidity can shield micro-organisms from chlorination. The ADWGs give a higher guideline level of <5 NTU as a limit for aesthetic purposes, as long as adequate disinfection can be achieved. Modern drinking water treatment plants often aim for a turbidity of < 0.3 NTU for best-case disinfection conditions and for effective removal of *Cryptosporidium* and *Giardia* cysts which are resistant to chlorine disinfection.

It is noted that the external laboratory results for raw water turbidity often differed significantly from the WTP records, the reason for which is not clear and **should be investigated by continuing to compare the external laboratory results with WTP analyses**. Further, the external laboratory data appears to limit the resolution of the turbidity measurement to 1 NTU unit. This is inappropriate for treated water turbidities in particular, which should be < 1 NTU at most times. **It would be helpful to request the external laboratory to report turbidity more accurately, i.e. to a resolution of at least 0.1 NTU.**

3.2.4 Colour

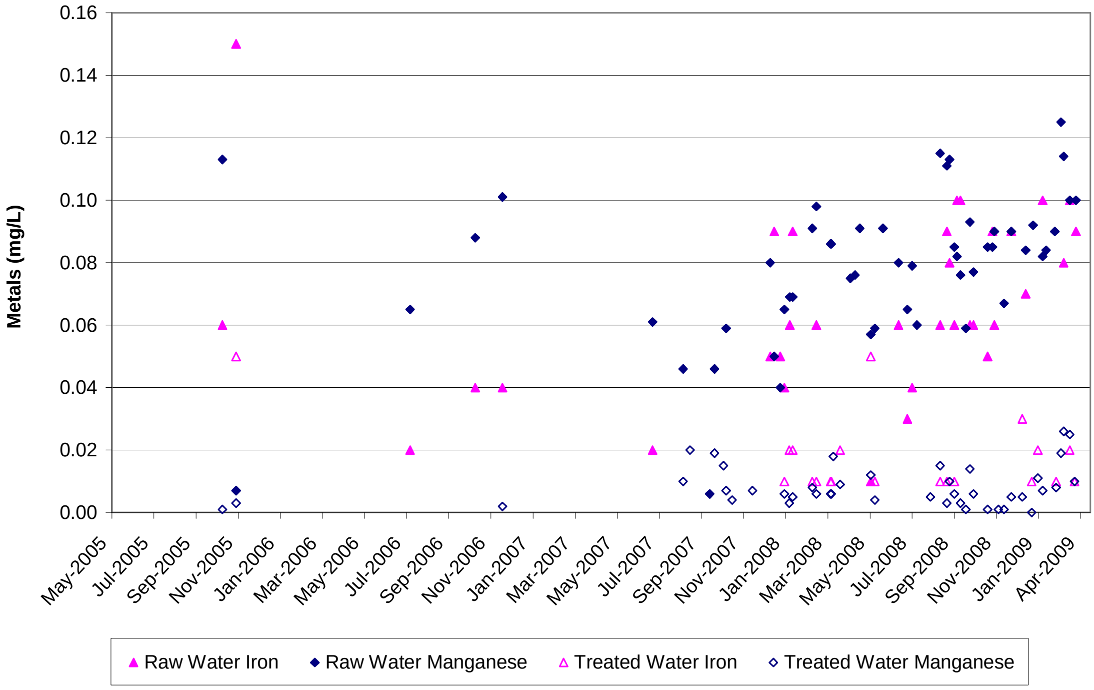
The measurement of 'true colour' requires the filtration of the sample to remove turbidity before colour analysis. True colour has been measured at the plant since 2008 and results show a raw water colour range of 4 – 15 HU with typical levels around 6 HU and treated water colour levels typically around 0 – 5 HU. The independent lab results for the period

2002-09 above indicate a raw water true colour of <1 to 7 HU and a treated water true colour of 2 HU or less.

Raw water apparent colour values measured have been significantly higher than the rue colour values. It is noted that apparent colour is not a true representation of the dissolved organics which make up the colour parameter because of the interference of turbidity.

3.2.5 Iron and Manganese

Iron and manganese levels from the WTP log sheet data are shown in the graph below. It is noted that the graph does not include the poor raw water quality event of February 2003, where iron was measured at 0.94 mg/L and manganese at 0.218 mg/L.



Graph of Raw and Treated Water Metals

As seen in the graph, raw water iron and manganese levels generally range up to 0.15 mg/L and 0.13 mg/L respectively, although rare fresh events may bring higher levels in both metals. Only total iron and manganese were reported, therefore the amount of soluble and particulate metals is not known. External laboratory data records (summarised in Appendix) showed lower maximum values for these metals.

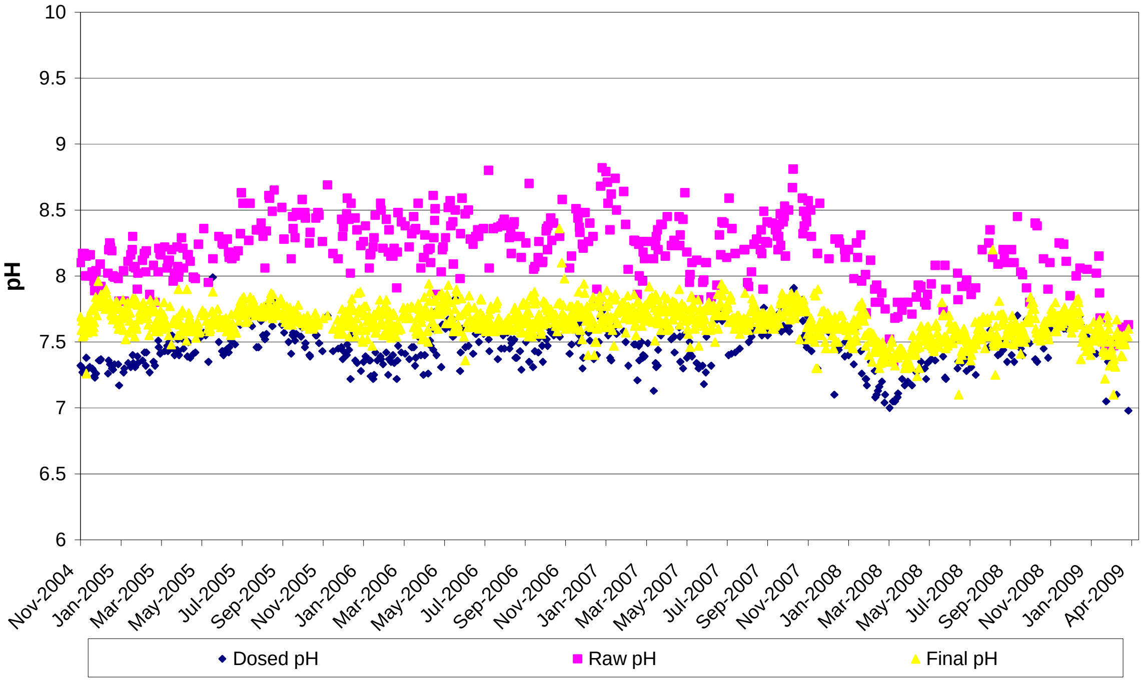
In all treated water samples reported, including the February 2003 fresh event, the iron has been removed to levels of 0.05 mg/L or less. It is likely that the iron is in particulate or an easily oxidised form, allowing it to be removed by coagulation and filtration. Treated water levels are well below the 2004 Australian Drinking Water Guidelines recommended level of 0.3 mg/L.

In all treated water samples reported, including the February 2003 fresh event, treated water manganese values were < 0.03 mg/L. It therefore appears that the plant can remove a significant amount of manganese from the raw water, either because the manganese was in particulate form and/or because it was oxidised by pre-chlorine dosing in the WTP process.

For manganese, the 2004 Australian Drinking Water Guidelines set an aesthetic guideline of 0.1 mg/L (100 µg/L) and a guideline health value of 0.5 mg/L. However it has been found that significant numbers of dirty water complaints are usually received when treated water manganese concentrations exceed 0.02 mg/L and targets as low as 0.01 mg/L have been set in some WTPs with manganese problems. **Raw water total and soluble manganese levels should continue to be measured periodically at the WTP, especially if high levels are detected.**

3.2.6 pH and Alkalinity

The raw, dosed and treated water pH is plotted in the figure below, based on measurements taken by operators. It is noted that the graph period does not include the 'fresh' event of February 2003.



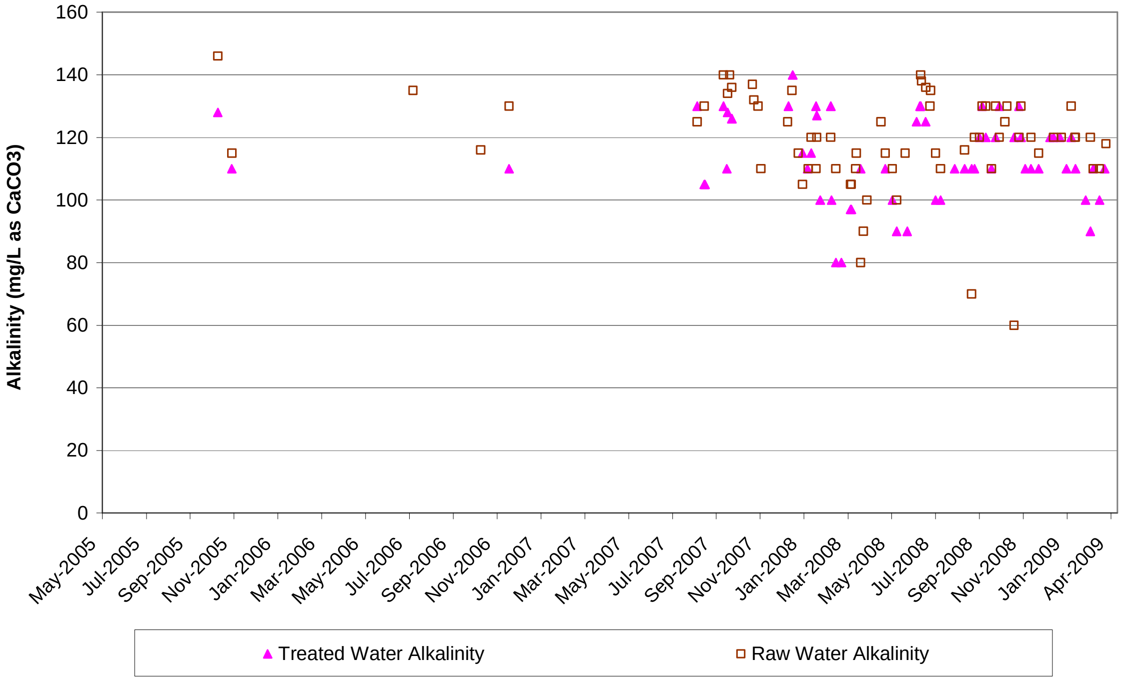
Graph of Raw, Dosed and Treated Water pHs

As seen in the graph, raw water pH has historically varied little throughout the year with pH typically in the band 8.0 – 8.5. A drop to raw water pHs <8.0 was seen in the early months of 2008, associated with several days rain in the catchment. Under the dam fresh conditions noted for February 2003, the raw water pH dropped to 7.2.

The dosed water pH has varied between pH 7.0 and 8.0 for the time period plotted. A drop to pH 7.0 is seen during early 2008, corresponding with the drop in raw water pH. For the dam fresh in February 2003, the dosed water pH dropped to 6.2 – 6.4, due to both a lower raw water pH and a much higher alum dose, which may have led to poorer coagulation.

The final pH, as shown on the graph is fairly constant, but has varied between 7.1 and 8.4. This indicates that post-filtration lime dosing is normally relatively stable and adjusted as required to meet the pH targets. For the dam fresh in February 2003, final pHs of 7.2 to 8.7 were recorded, as operations were adjusted to try to match the changing water quality.

Alkalinity levels from the WTP log sheets are shown in the following graph.



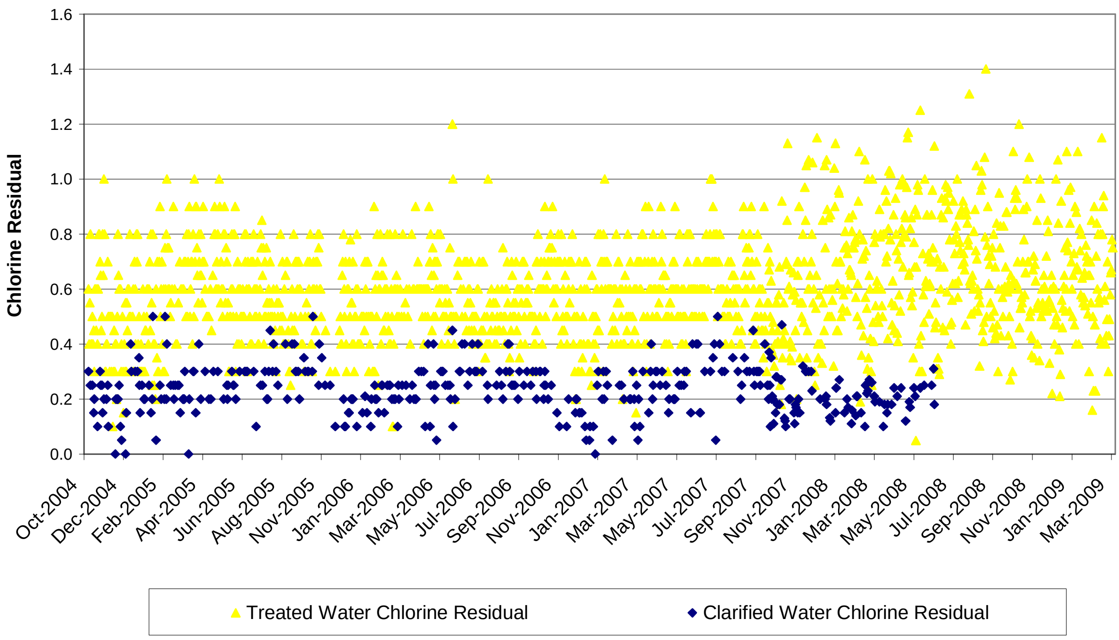
Graph of Raw and Treated Water Alkalinity

As seen in the graph, raw water alkalinity has generally been between 80 and 155 mg/L as $CaCO_3$, but dropped to 60 mg/L as $CaCO_3$ on one occasion. Treated water alkalinity has varied between 80 and 140 mg/L as $CaCO_3$, at which levels it is expected to give reasonable buffering capacity within the distribution system.

For the dam fresh in February 2003, the raw water alkalinity was around 67 and the treated water alkalinity was around 49 mg/L as $CaCO_3$.

3.2.7 WTP Chlorine Residuals

The chlorine residuals measured in the clarified water (after pre-chlorine dosing at the WTP) and in the treated water are shown in the graph below. Data for the clarified water chlorine residual was not available between July 2008 and March 2009.



Graph of Raw and Treated Water Chlorine Residuals

As seen in the graph, the clarified water chlorine residual varies between 0.1 and 0.5 mg/L. It is understood that the operators aim to achieve a small chlorine residual to ensure the chlorine demand of the raw water is met.

The treated water chlorine residual is highly variable from day to day, varying between <0.1 and 1.4 mg/L, but generally averages around 0.6 mg/L, which is in the targeted range of 0.5 – 0.6 mg/L. Low values may represent less than ideal disinfection conditions, depending on mixing conditions in the clear water tank. It is noted that there is a rechlorination station at the town reservoirs, however this does not treat the water transferred to the Callide township, therefore chlorination at the WTP is critical for adequate disinfection of the water supplied to Callide Township. **The achievement of effective chlorination for disinfection at the WTP should be further investigated.**

3.2.8 Pesticides and Herbicides

The data available on Callide dam water pesticide and herbicide analyses is shown in the table below.

Pesticide and Herbicide Analysis Results for WTP Water

Parameter	Units	Raw Water (Date not recorded)	WTP 6/11/02	WTP 6/11/02	Callide Dam Raw Water 30/04/07
OC Pesticides	µg/L	<0.10	<0.10	<0.10	<0.3
OP Pesticides	µg/L	*Not meas.*	<0.1	<0.1	<0.10
Herbicide Atrazine	µg/L	<0.02	<0.02	<0.02	<0.01
Herbicide Diuron	µg/L	<0.01	<0.01	<0.01	<0.01
Herbicide Hexazinone	µg/L	<0.10	*Not meas.*	*Not meas.*	<0.01
Herbicide Simazine	µg/L	*Not meas.*	*Not meas.*	*Not meas.*	<0.01

Parameter	Units	Raw Water (Date not recorded)	WTP 6/11/02	WTP 6/11/02	Callide Dam Raw Water 30/04/07
Herbicide Tebuthiuron	µg/L	*Not meas.*	*Not meas.*	*Not meas.*	<0.01
Herbicide Fluometuron	µg/L	<0.01	*Not meas.*	*Not meas.*	<0.01

As seen in the table above, pesticides and herbicides were below the detectable levels in all samples taken.

3.2.9 Algae and Algal Toxins

Algal blooms are reported to occur in the dam most summer seasons. Analysis of algae in the dam is performed by Sunwater, with few results obtained by Council. Results from testing in March 2006 are given below. It was noted that the raw water was previously sampled on 20th February 2006, when a level of 28,000 cells of *Cylindrospermopsis raciborskii* was reported.

WTP Raw and Treated Water Algal Analysis

Organism	Raw Water 7/3/06 (cells/mL)	Treated Water 7/3/06 (cells/mL)
CYANOPHYTES (blue-green algae)		
Nostocales		
Anabaena spp.(coiled)	32200	578
Anabaenopsis spp.	272	n.s.
Cylindrospermopsis raciborskii	6200	180
Total (Nostocales)	38700	758
Oscillatoriales		
Geitlerinema spp.	7800	n.s.
Planktolyngbya minor	4500	n.s.
Planktolyngbya subtilis	225	n.s.
Pseudanabaena limnetica	5054	68
Pseudanabaena galeata	500	n.s.
Spirulina laxissima	3800	n.s.
Unidentified Oscillatoriales	175	n.s.
Total (Oscillatoriales)	22100	68
Chroococcales		
Aphanocapsa spp.	8250	n.s.
Aphanothece spp.	200	n.s.
Chroococcus minutus	100	n.s.
Chroococcus spp.	900	n.s.
Gloeocapsa spp.	100	n.s.
Merismopedia spp.	28056	476
Microcystis cf. aeruginosa	340	130
Myxobaktron spp.	150	n.s.
Rhabdoderma spp.	25	n.s.
Snowella spp.	100	n.s.
Synechococcus spp.	150	n.s.
Unidentified Chroococcales	50	85
Total (Chroococcales)	38400	691
Total (CYANOPHYTES)	99100	1520
TOTAL CELLS PER ML	99100	1520

The potentially toxic algae species identified in the samples above are:

- Cylindrospermopsis raciborskii; and
- Microcystis cf. aeruginosa

It was noted that both these species were also detected in the WTP treated water, which is of concern as it shows that the process is not robust in the removal of algal cells.

The raw and treated water from 7th March 2006 samples was tested for the presence of Cylindrospermopsin toxin, with levels found to be below detectable limits (<0.2 µg/L). Toxin levels were not reported for the sample from 20th February, 2006 where *Cylindrospermopsis* levels were significantly higher.

The operators also reported that algal cells and taste and odour problems in the raw water in September 2003 had required significant changes to the operation of the plant, including down-rating the flow to 100 L/s.

The presence of potentially toxic blue green algae in the dam is a concern. It is understood that Sunwater, the operator of the dam, regularly tests for algae. Council should ideally **monitor algae levels closely (in conjunction with Sunwater if appropriate) and water should be analysed for toxins if significant levels of potentially toxic algae are present.**

3.2.10 Taste and Odour Compounds

Taste and odour are reported to be a problem in the raw water. This issue is addressed by dosing PAC and by adjusting the ratio of the WTP treated water vs the bore water flows (blended at the town reservoir mixing tank). The cause of the taste and odour problems has not been identified. Taste and odour compounds can be associated with algae present on the dam. Analysis of taste and odour compounds was not available.

The cause of the taste and odour problems should be further investigated to develop the most effective treatment approach. Raw water samples known to contain tastes and odours should be analysed to identify the compounds responsible.

3.2.11 Microbiological Parameters

Five microbiological analysis results for WTP water were recorded from 2003 – 2006. Coliforms were ‘not detected’ in all samples, except one which had a level of > 200 coliforms, but no detectable *E. Coli*.

Due to the agricultural landuses in the dam catchment, it is suggested that **the raw and treated water could be analysed for *Cryptosporidium* and *Giardia* on occasion to check background levels of these pathogens,** particularly after heavy rain.

3.2.12 Other WTP Water Quality Issues

Based on the external laboratory results (summarised in Appendix), the levels of TDS, hardness and other ions reported are in the typical range expected for raw water.

Corrosivity data for the WTP raw and treated water are investigated further later in this chapter of the report.

3.2.13 WTP Raw and Treated Water Quality Summary

The range and typical values for significant water quality parameters in the WTP raw and treated waters are summarised in the table below.

WTP Raw and Treated Water Quality Parameters Summary

Parameter	Units	WTP Raw Water		WTP Treated Water	
		Range	Typical	Range	Typical
Turbidity	NTU	< 1 – 153	5 - 20	< 0.1 – 2.6	0.1 or lower
True Colour	Pt-Co	< 1 - 15	6	< 1 - 5	0
pH	-	7.2 – 8.7	8.2	6.2 – 8.4	7.7
Iron (total)	mg/L	<0.01 – 0.94	0.06	<0.05	0.025
Manganese (total)	mg/L	< 0.03 – 0.22	0.08	< 0.03	0.010
Alkalinity	mg/L $CaCO_3$	60 – 155	100 - 140	49 - 140	110
Chlorine	mg/L	0.1 – 0.5	0.2	<0.1 – 1.2	0.6
E.coli	MPN/100ml	Not detected		Not detected	
Pest/ Herbicides	-	Not detected		Not detected	
Blue Green Algae	-	Potentially toxic species detected		Potentially toxic species detected	
B-G Algal Toxins	-	Not detected		Not detected	
Taste and odour compounds	-	Reported to occur		Reported to occur	

3.2.14 WTP Treated Water Quality Objectives

The treated water quality target levels recommended in the Australian Drinking Water Quality Guidelines (NHMRC, 2004) are shown in the table, with common targets based on industry experience and other water treatment plants around Australia and the current targets used at the Biloela WTP.

Treated Water Quality Targets & Guideline Values

Parameter	Units	ADWG		Common Industry Treated Water Targets	Current Biloela Treated Water Target
		Health	Aesthetic		
Turbidity	NTU	1	5	< 0.1	< 0.1
Colour	HU		15	≤ 5	≤ 5
pH			6.5 – 8.5	7.5 – 8.3	7.6
Chlorine	mg/L	5		Depends on system	0.6 – 0.8
Total Aluminium	mg/L	0.2		≤ 0.2	-
Total Manganese	mg/L		0.1	≤ 0.05	-
Total Iron	mg/L		0.3	≤ 0.3	-
Total Alkalinity	mg/L as $CaCO_3$			≥ 40	-
Total Dissolved Solids (TDS)	mg/L		< 500	< 500	-

Parameter	Units	ADWG		Common Industry Treated Water Targets	Current Biloela Treated Water Target
		Health	Aesthetic		
CCPP				-1 to -5	-
Total Trihalomethanes	mg/L	0.25		≤ 0.15	-

The targets adopted at the WTP are reasonable compared to the ADWG and industry values. It is noted from the analysis of water quality above that the targets are not always met.

3.3 Town Bores Water Quality

3.3.1 Bore Water Quality Monitoring Data

Bore water quality data was available from external laboratory analysis results. This data is summarised in the table below. Water quality parameters of note are discussed below.

General Analysis Town Bores

Parameter	Units	Bore 4&5 Gladstone Rd 17/10/05	Bore 4&5 Gladstone Rd 24/1/06	Bores 8,9,10,11 Jambin Rd 17/10/05	Bores 8,9,10,11 Jambin Rd 24/1/06	Bore 4&5 Gladstone Rd 14/11/06	ADWG Values
Turbidity	NTU	<1	<1	1	1	<1	< 1
True Colour	HU	1	<1	<1	<1	1	< 15
pH @ 21C		7.08	7.05	7.3	7.13	7.1	6.5 – 8.5
Conductivity	µs/cm	631	634	70	942	670	-
Total Diss Solids	mg/L	354	362	402	550	377	< 500
Total Diss Ions	mg/L	399	410	472	622	425	-
Total Hardness	mg/L as $CaCO_3$	191	197	199	309	205	60 – 200 'Good' 200 – 500 Increasing scaling
Alkalinity	mg/L as $CaCO_3$	128	129	172	174	131	-
Silica	mg/L	34	33	36	37	33	-
Sodium	mg/L	49	50	65	77	51	< 180
Potassium	mg/L	1.1	1.2	0.8	1.1	1.2	-
Calcium	mg/L	44	45	50	73	46	-
Magnesium	mg/L	20	21	18	31	22	-
Hydrogen	mg/L	0	0	0	0	0	-
Bicarbonate	mg/L	155	158	210	212	160	-

Parameter	Units	Bore 4&5 Gladstone Rd 17/10/05	Bore 4&5 Gladstone Rd 24/1/06	Bores 8,9,10,11 Jambin Rd 17/10/05	Bores 8,9,10,11 Jambin Rd 24/1/06	Bore 4&5 Gladstone Rd 14/11/06	ADWG Values
Carbonate	mg/L	0.1	0.1	0.3	0.2	0.1	-
Hydroxide	mg/L	0	0	0	0	0	-
Chloride	mg/L	106	111	105	186	116	< 250
Fluoride	mg/L	0.1	0.1	0.2	0.2	0.12	< 1.5
Nitrate	mg/L	0.5	0.8	3.6	10	0.7	< 50
Sulphate	mg/L	23	23	18.8	31	28	< 250
Iron	mg/L	<0.01	<0.01	<0.01	<0.01	<0.01	< 0.3
Manganese	mg/L	<0.03	<0.03	<0.03	<0.03	<0.03	< 0.1
Zinc	mg/L	0.03	0.16	0.05	0.08	0.08	< 3
Aluminium	mg/L	<0.05	<0.05	<0.05	<0.05	<0.05	< 0.2
Boron	mg/L	0.05	0.05	0.05	0.05	0.05	< 4
Copper	mg/L	<0.03	<0.03	<0.03	0.03	0.03	< 1
Free Carbon Dioxide*	mg/L	21	23	17	26	21	-

*Values for free carbon dioxide levels were calculated by CWT based on alkalinity and pH levels.

3.3.2 Turbidity and Colour

Available data shows that the turbidity in the bore water is in general low. Some values of 1 NTU have been recorded. Because of the resolution of the laboratory method for measuring turbidity, (assumed to be to 1 NTU unit), it is difficult to determine whether the ADWG limit of <1 NTU has been exceeded.

Low true colour measurements of 1 HU or less have been recorded. This is as expected for bore waters, which are typically low in organics.

3.3.3 Alkalinity, pH and Carbon Dioxide

The pH and alkalinity levels are relatively stable, based on available data. The high alkalinity levels are expected to give reasonable buffering capacity.

Free carbon dioxide levels were elevated compared to the levels calculated for the surface water samples. High levels of free carbon dioxide can contribute to the corrosivity of the water, as discussed further later in this report. It is noted that the pH of the samples above was measured after transport to the laboratory. The pH should ideally be checked as soon as the sample is taken as some of the dissolved carbon dioxide may escape during handling and transport, resulting in a rise in pH.

3.3.4 TDS and Hardness

The TDS and hardness values show that the bore water contains high levels of dissolved minerals. The ADWG values for these parameters were exceeded in one sample from the Jambin Rd bores (on 24/1/06).

The TDS value of >500 mg/L is aesthetically-based rather than based on health requirements. The ADWG ranks water with TDS levels of 80 - 500 mg/L as ‘good quality’ and unlikely to cause excessive scaling in pipes and fittings. Taste complaints are more likely if water exceeds 500 mg/L. It is noted that as the bore water is blended with WTP treated water, the TDS level is mitigated.

Hardness in water is usually due primarily to the presence of calcium and magnesium ions. Ground water supplies are generally harder than surface waters. As with TDS, no health-based guideline exists, however, the ADWG recommends that the total hardness not exceed 200 mg/L in order to prevent undesirable scaling in hot water systems. Hardness levels of 200 – 500 mg/L are considered to be in the region where the water is oversaturated and will tend to form scale on pipes and tanks. The scaling/ corrosion potential of the water types is covered further in a later section in this chapter.

It is noted that the Jambin Rd bores appear to be slightly higher in dissolved minerals than the Gladstone Rd bores.

3.3.5 Sodium and Chloride

Sodium and chloride, which are components of TDS, were also elevated in the bore water samples. Sodium was measured up to 77 mg/L, compared to the ADWG level of 180 mg/L. Chloride levels were up to up to 186 mg/L, compared to the ADWG of < 250 mg/L.

It is noted that the ADWG recommended levels are based on the prevention of undesirable taste, rather than on health requirements, although sodium levels do present a health risk for certain people suffering severe hypertension and congestive heart failure.

3.3.6 Nitrate

Nitrate levels were generally low, however a level of 10 mg/L was measured in the Jambin Rd bores. This level, although elevated, is below the ADWG guideline value of 50 mg/L as nitrate (NO_3) for bottle-fed infants and the higher level of 100 mg/L (as nitrate) for adults and children over 3 months of age.

It is noted that nitrite levels are not given. The ADWG levels for nitrite is 3 mg/L as nitrite (NO_2), however the nitrite ion is relatively unstable and easily oxidised to nitrate, and therefore not often found in well aerated or chlorinated water supplies.

Because of the elevated level noted, nitrate levels should continue to be monitored periodically. If more elevated nitrate levels are found, the water should also be measured for nitrite.

3.4 Combined Town Water Quality

3.4.1 General Town Water Quality Monitoring Data

Data available from external laboratories for the Biloela pump station combined town water quality is summarised in Appendix A of this report. The town water is a combination of the WTP treated water and the town bores water.

Because the blend of WTP treated water and the town bores water has historically been varied, the water quality results do not necessarily represent typical town water quality at any given blend. It is interesting to note that the TDS, hardness and alkalinity parameters measured on 24/1/06 (see data in Appendix A) were significantly lower than other samples although the town bore water quality for this date gave the highest values in these parameters from the Jambin Rd bores. It is assumed that the bore water made up a lower percentage of the overall town blend on this date, compared to other sampling dates.

The main water quality parameters have already been discussed for the WTP treated water and the town bores water above. The additional notes are added from an analysis of the final blended water quality shown in the external laboratory reports:

- Turbidity was generally reported at 1 NTU or lower, indicating that the water is generally below or at the ADWG limit of < 1 NTU. However one high value of 12 NTU was reported for 30/04/07. The elevated turbidity is not linked to high levels in the WTP filtered water;
- TDS and total hardness levels reported are all within ADWG levels for the combined water, except that of 26/11/07 which is just above the ADWG level;
- Nitrate levels up to 3.3 mg/L were reported.

3.4.2 Chlorine Residuals

It is noted that the treated water used for town supply is re-chlorinated after mixing with the town bore water. The rechlorination is carried out to try to meet a target of 0.4 – 0.5 mg/L chorine residual for the blended water. The operators report that chlorine levels below 0.4 mg/L as measured after rechlorination are generally insufficient to maintain suitable residuals around the reticulation system.

It is understood that chlorine levels are monitored in the reticulation system by the Health Department and Council. Data on chlorine residuals at various points in the reticulation system, taken from the SKM “Biloela Water Supply Planning Report” (2006), showed residuals of 0.2 – 0.6 mg/L within the Biloela reticulation system, however 0.0 mg/L was measured at Callide Township (supplied from WTP clear water tank) and at the Thangool airport. This limited data shows that chlorine residuals may not be adequate at times in various locations around the water distribution systems. The operators are aware that residuals can be low at Callide Township because of extended retention times in the main (due to low demand). Thangool residuals may also tend to be low because it is at the extremity of the town reticulation system.

The chlorine residuals within the system should continue to be monitored, and the chlorination residual targets at the WTP and after blending with the bore water should be reviewed in relation to the findings.

3.4.3 Microbiological Parameters

The microbiological monitoring results available for town water are shown in the table below. It is noted that recent changes to State Government legislation require Council to test for E.Coli and coliforms once a week for Biloela and Moura (towns of >5000 people) and report the results to the Department of Health.

Microbiological Parameters for Town Water

	Biloela Town PumpStation		**Biloela Council Chambers**		**Biloela Hospital**	
	Coliforms	*E.coli*	Coliforms	*E.coli*	Coliforms	*E.coli*
6/9/05	ND	ND	-	-	-	-
17/10/05	ND	ND	ND	ND	-	-
15/11/05	2	ND	29	ND	-	-
24/1/06	ND	ND	-	-	-	-
20/2/06	83	ND	>200	ND	-	-
21/1/08	1	ND	ND	ND	-	-
11/3/08	ND	ND	ND	ND	-	-
12/5/08	ND	ND	ND	ND	-	-

10/6/08	ND	ND	ND	ND	-	-
17/7/08	ND	ND	ND	ND	-	-
4/8/08	ND	ND	ND	ND	-	-
7/10/08	ND	ND	ND	ND	-	-
20/1/09	ND	ND	ND	ND	-	-
4/2/09	ND	ND	ND	ND	-	-
11/2/09	ND	ND	ND	ND	-	-
18/3/09	ND	ND	-	-	ND	ND
24/3/09	1	ND	-	-	1	ND

As shown in the table, coliforms were detected on a number of occasions, however *E.coli* has not been detected. The presence of coliforms may be associated with faecal contamination, however there are many environmental coliforms which are not of faecal origin and may be associated with pipe or sediment biofilms, ingress of soil and/or insufficient chlorination.

The ADWGs have no recommended level for coliforms, but state that *E.Coli* or thermotolerant coliforms should not be detected. These conditions have been met in all town water samples reported.

3.4.4 Town Water Blends Quality Summary

The range and typical values for significant water quality parameters in each of the water sources and the combined water feeding Biloela town are summarised in the table below.

Town Water Blends Quality Parameters Summary

Parameter	Units	WTP Treated Water		Town Bores		Combined Town Water	
		Range	Typical	Range	Typical	Range	Typical
Turbidity	NTU	< 0.1 – 2.6	0.1 or lower	<1 – 1	1	<1 – 12	< 1
True Colour	Pt-Co	< 1 - 5	0	< 1 – 1	1	< 1 – 7	1
pH	-	6.2 – 8.4	7.7	7.05 – 7.3	7.2	7.1 – 7.9	7.5
Iron (total)	mg/L	<0.05	0.025	< 0.01	< 0.01	< 0.01	< 0.01
Manganese (total)	mg/L	< 0.03	0.010	< 0.03	< 0.03	< 0.03	< 0.03
Alkalinity	mg/L $CaCO_3$	49 - 140	110	128 – 174	150	99 – 143	130
TDS	mg/L	214 – 260	230	399 – 622	420	198 - 377	320
Hardness	mg/L	122 – 140	130	191 – 309	195	112 - 208	170
E.coli	MPN/ 100 ml	Not detected		-		Not detected	
Pesticides/ Herbicides	-	Not detected		-		-	
Blue Green Algae	-	Potentially toxic species detected		-		-	

3.5 Water Corrosivity Issues

3.5.1 Problems Typically Associated with Corrosive Waters

Waters may be potentially corrosive due to various combinations of parameters such as low pH, low alkalinity, or low hardness. Problems commonly experienced in a water supply as a result of aggressive water include:

- reduced disinfection efficiency at elevated pH levels;
- pitting corrosion, high copper levels and blue water in copper pipes within buildings;
- elevated iron levels associated with iron or steel pipes;
- meringue dezincification of brass fittings at pHs of 8.5 or higher;
- high pH values throughout the reticulation due to the dissolution of various compounds from concrete and cement within the system.

These problems can lead to increased health risk to consumers and deterioration of service pipes and fittings in water supply schemes.

Water quality is considered the main contributing factor to corrosion of infrastructure in water supply systems. Other factors contributing to corrosion may include micro-organisms on pipe walls; reticulation design and layout; materials used; and water use characteristics.

3.5.2 Reported Corrosion Problems in Biloela

Council have reported problems with the deterioration concrete in the main 9ML town reservoir (ground level reservoir No.1) and the adjacent mixing tank where the bore and treated dam water sources are combined. The problems may be associated with corrosion processes. Problems noted include:

- Concrete etching in the mixing tank.
- Delamination 'blistering' on floor, deterioration, cracking and leaking on walls and corrosion of iron reinforcing exposed to water in the reservoir.

It is noted that the mixing tank receives bore water, WTP treated water and chlorine dosing. Chlorine is a strong oxidising agent but is not expected to be particularly corrosive to concrete.

It is noted the condition of the concrete in the reservoir was the subject of a report by CorPrev Corrosion Prevention Technologies in 2005. This report did not directly mention the likelihood that the problems were caused by aggressive water.

3.5.3 Corrosivity Indices

Indices which reflect the corrosion potential or "aggressiveness" of water can be modelled using water quality data. These indices are useful in estimating the likely corrosion potential of waters, although they do not necessarily apply to all types of waters. They include:

- the Calcium Carbonate Precipitation Potential (CCPP); and
- the Langelier Index.

The CCPP and Langelier Index are indicators of whether a water is likely to be aggressive or scale forming. Negative values indicate that waters are likely to be corrosive while positive values indicate the water is likely to form calcium carbonate scale.

If the CCPP is zero then the water is saturated in terms of calcium carbonate. If the CCPP is positive then the water is over-saturated and likely to precipitate a film, predominantly of $CaCO_3$, onto pipes and other water supply infrastructure in contact with the water. If the CCPP is negative then the water is under-saturated and is likely to be corrosive. Various studies have shown CCPP to be an accurate indicator of corrosiveness of concrete and cement linings.

The Langelier Index (LI) has also been found to be an accurate indicator of water scaling and hence corrosivity under most circumstances. It is the difference between the saturated pH and the water's actual pH, and is therefore on a logarithmic scale. Again a negative value indicates that the water is likely to be corrosive and a positive value shows it to be over-saturated and therefore likely to be scale forming.

3.5.4 Water Quality Targets for the Prevention of Corrosion

The water quality targets outlined in the table below are generally recommended to minimise potential corrosivity in treated waters, based on industry experience.

Typical Target Water Quality Parameters for Corrosion Control

Parameter	Units	Target	Guideline Range
pH	pH units	7.8 to 8	7.6 to 8.2
Alkalinity	mg/L as $CaCO_3$	45 to 55	> 40
Ca Hardness	mg/L as $CaCO_3$	> 40	> 40
CCPP	mg/L	- 3	- 6 to 0
Langelier Index	pH units	- 0.3	- 0.6 to 0

The pH of the water should be above 7.6 for waters leaving the WTP but should not exceed 8.3 as dezincification can occur at pHs of around 8.5 and above. At pHs above 7.0, the effectiveness of chlorine disinfection is reduced.

A free chlorine residual of around 0.2 mg/L in the extremities of the reticulation system is usually recommended to minimise the possibility of microbiologically-induced corrosion.

3.5.5 Biloela Corrosion Indicators

Corrosivity indices were modelled for the Biloela water sources and are set out in the table below, with the data used as input to calculations included. A temperature of 20 ^{o}C has been assumed for all calculations.

Corrosivity Indices for Biloela Raw and Treated Water

Parameter	Units	Typical			Worst Case Range		
		Dam	Bores	Blend	Dam	Bores	Blend
Temperature	°C	20	20	20	20	20	20
TDS	mg/L	260	400	340	214 – 260	354 – 550	264 – 350
Alkalinity	mg/L as $CaCO_3$	125	150	135	49 – 140	128 – 174	115 – 138
Calcium	mg/L as Ca	29	50	41	28 – 33	44 – 73	34 – 45
Calcium hardness	mg/L as $CaCO_3$	72.5	125	102.5	70 – 82.5	110 – 182.5	85 – 112.5
pH	-	7.95	7.1	7.5	7.2 – 8.7	7.05 – 7.3	7.3 – 7.6
Chloride	mg/L	60	110	97	47 – 70	105 – 186	69 – 100
Sulphate	mg/L	20	23	24	18.4 – 22	18.8 – 31	20 – 27.5
Free CO_2	mg/L	2.5	22	10	< 5	17 - 26	7 - 14
CCPP	**mg/L**	**3.3**	**-18.8**	**-2.0**	**-12.4 – 14**	**-25.2 – 5.6**	**-12.9 – 2.5**
LI	**-**	**0.2**	**-0.2**	**-0.1**	**-0.8 – 0.9**	**-0.3 – 0.1**	**-0.3 – 0.1**

The results of the modelling show that for typical water quality of the treated dam water the water is slightly oversaturated and may tend to be scale forming rather than corrosive. The worst case dam water ranged from significantly corrosive (associated with a fresh event in the dam) to significantly scale forming. It is likely that the dam water would tend to be scale forming rather than corrosive except for the rare events of large inflow to the dam. It is noted that Council report that there have been no known problems with scaling in the town water supply.

The bore water showed modelled values ranging from the region associated with significantly corrosive water, to the region of slightly scale-forming water. This is unusual for waters with such high alkalinity and calcium hardness values and is probably associated with the significant free carbon dioxide levels present in the water. Dissolved free carbon dioxide forms carbonic acid, which would tend to make the water more corrosive. It is possible that the significant corrosion potential of the bore water may be contributing to some of the problems with concrete deterioration in the mixing tank and main reservoir.

The combined blend of town water, based on available data, gave indices in the range indicating reasonably corrosive to mildly scale forming water. For typical values, however, it showed the parameters of alkalinity, calcium hardness, CCPP and LI within the recommended range to minimise corrosivity. The typical pH of 7.5 is just outside the recommended corrosion-related range of pH. The level of free carbon dioxide is reduced in the combined water compared to the raw water levels. The combined water is expected to be most corrosive when the bore water makes up a larger proportion of the blend.

3.5.6 Corrosion Minimisation Measures

It is noted that the problems of concrete deterioration in the reservoir are not necessarily caused by corrosive water. The type and strength of the concrete and the method of laying may also be critical factors and subject to further investigation.

However, based on the analysis above, the raw bore water is quite likely to be contributing to corrosion in the mixing tank at least, and possibly also in the reservoir. The bore water could potentially be made less corrosive by aeration before blending to remove dissolved carbon dioxide present. If required, **jar testing chould be initially undertaken to look at the effect of aeration on the water quality and on the likely corrosivity of the bore water**.

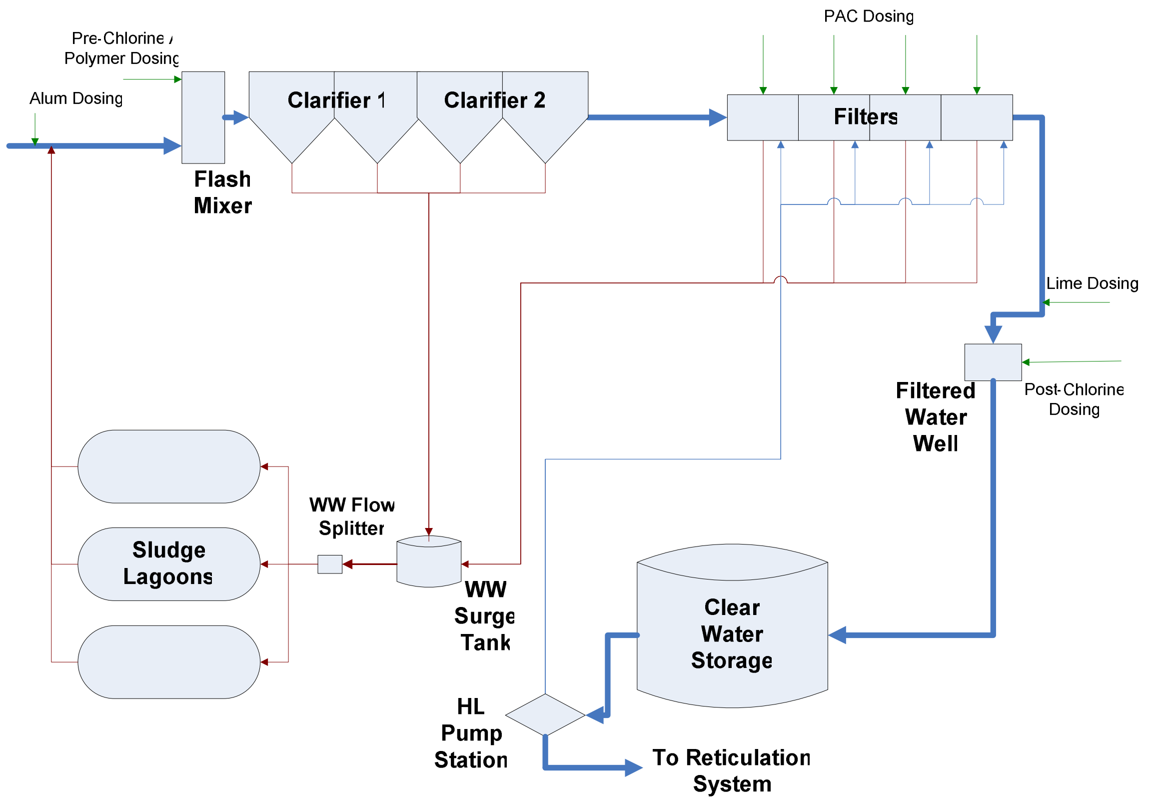
4. WTP Process Description and Capacities

4.1 Process Overview

Biloela WTP was constructed in 1983/84. The plant is a conventional treatment process, comprising the following main unit processes:

- Coagulation
- Flocculation
- Settling
- Filtration
- Disinfection
- pH adjustment

A diagram of the plant process is shown below.



4.2 Raw Water Pump Station and Chlorine Dosing

4.2.1 Raw Water Pump Station

Raw dam water is drawn from the Callide Dam manifold outlet to the raw water pump station's two centrifugal pumps. The dam intake has two possible draw-off points at different depths in the dam. The highest draw-off point is generally used, although the lower point has reportedly been used in the past when there were large numbers of blue-green algae in the top levels of the dam.

The raw water pumps are located in the raw water pump house below the dam wall. Raw water is delivered to the WTP via a 375 mm main, which is approximately 3.5 km long. The raw water pumps are VFD controlled so that the flow rate to the WTP can be altered. They are usually set to operate at about 130 L/s, with the flow throttled down to the design WTP plant flow of 120 L/sec at the WTP inlet valve.

The pumps reportedly give their rated maximum capacity of 180 L/sec at the normal dam operating water level of 20%, however the pump capacity is limited on the suction side when dam levels are low. The plant flow rate trial undertaken on 6/5/09 showed that the maximum pump output (at 100% VSD setting) with dam levels <10% was around 138 L/s. It was noted during the flow trial that the VSD faulted repeatedly when set to 100%. **This VSD problem should be further investigated and rectified to allow the VSD to be run at 100%**.

The raw water pumps are powered and controlled from the raw water pump station switchboard and VFD.

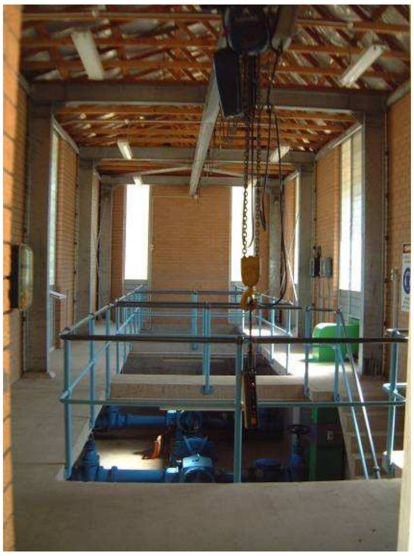


Photo of Raw Water Pump Station

The pump station and raw water mains parameters are summarised in the table below. The plant inlet hydraulic capacity is also given.

Raw Water Pumps

Component	Parameter (Units)	Design Criteria	Comments
Callide Dam Pumps	Intake type		Pump station situated below dam wall
	No. and Capacity each (L/s)	2 (duty/ standby) x 180 L/s each for full dam head. At current (2009) low dam levels, pump capacity approx. 138 L/s	Pumps have VSDs for speed adjustment and slow start
	Pump details	Axially split single stage centrifugal supplied by Thompsons Byron Jackson Motor: 200 kW TECO motor, 415V, 50 Hz, 1460 rpm	Based on SKM planning report 2006
	Mains diameter, length	375 mm, 3.5 km	

Component	Parameter (Units)	Design Criteria	Comments
	Mains maximum flow rate (ML/d, L/s)	180 L/s	Operator said had run at 180 L/s briefly to test hydraulics

4.2.2 Pump Station Pre-Chlorine Dosing

The raw water is dosed with chlorine at the raw water pump station via an on-site chlorinator. The chlorine is dosed to consume some of the raw water chlorine demand. The dose is set to be equal or less than the chlorine demand of the water, so that under normal circumstances there is only a small chlorine residual in the water by the time the water reaches the WTP inlet.

4.3 Plant Inlet

The raw water main section at the inlet to the WTP, prior to the flash mixing tank, contains the following components:

- Raw water flow meter;
- Inlet control valve;
- Coagulant and pre-coagulation dosing points;
- Sludge lagoon supernatant return point;
- An inline mixer section (just before flash mixing tank).

These components are shown in the following figure.

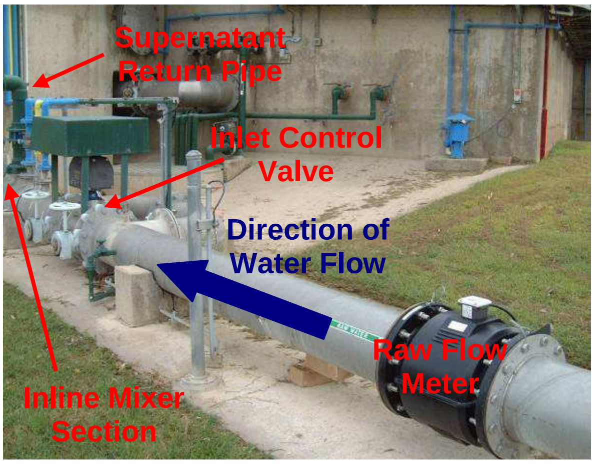


Photo of Raw Water Main at Plant Inlet

It is assumed that the capacity of the raw water main is at least 280 L/s, based on the planned Stage 2 flow rate for future upgrade options, however the condition and hydraulics of the mains pipe in its current condition should be checked as part of any plans to upgrade the plant capacity.

The exact capacity of the raw water flow meter and control valve are not known, however the original WTP operating manual notes that these components were selected to suit the Stage 1 flow of 120 L/s, therefore their suitability for higher flows should be checked, and adjustments to their ranges may be necessary.

The inline mixer is a three-baffle arrangement designed to provide mixing of chemicals and the supernatant return water just prior to the flash mixing tank. The orientation of this mixer was changed in 2007 to align the baffles sideways rather than top and bottom to minimise the buildup of sediment behind the baffle. According to the original WTP operating manual, the inline mixer was designed for the Stage 1 flow of 120 L/s, however it could reportedly be modified to suit higher flows of up to 240 L/s by drilling holes in the baffle plates.

The flow rate capacities of components on the inlet section of the main, as given in the original operating manual, are summarised in the table below.

WTP Inlet

Component	Parameter (Units)	Design Criteria	Comments
Plant Inlet Components	Raw water pipeline capacity (L/s)	280 (assumed)	Based on original intended Stage 2 capacity plus hydraulic overload
	Raw water flow meter capacity (L/s)	At least 120	Selected for Stage 1 flow (per original operating manual)
	Inlet control valve capacity (L/s)	At least 120	Selected for Stage 1 flow (per original operating manual)
	Inline Mixer capacity (L/s)	120 currently Up to 240 with modifications	Based on original operating manual. Note that inline mixer currently replaced by temporary arrangement

4.4 Pre-Filtration Chemical Dosing

4.4.1 Chemical Dosing Locations Before Flash Mixing Tank

The location of dosing points on the raw water mains before the flash mixing tank is shown in the figure below. The only dosing point currently being used is the alum dosing point.

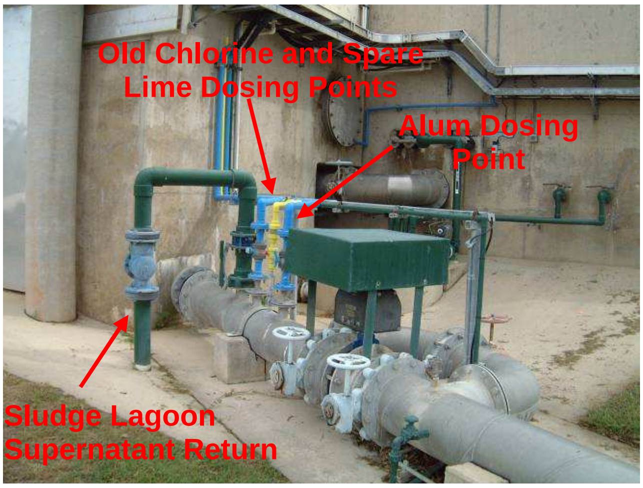


Photo of Raw Water Main Dosing Points

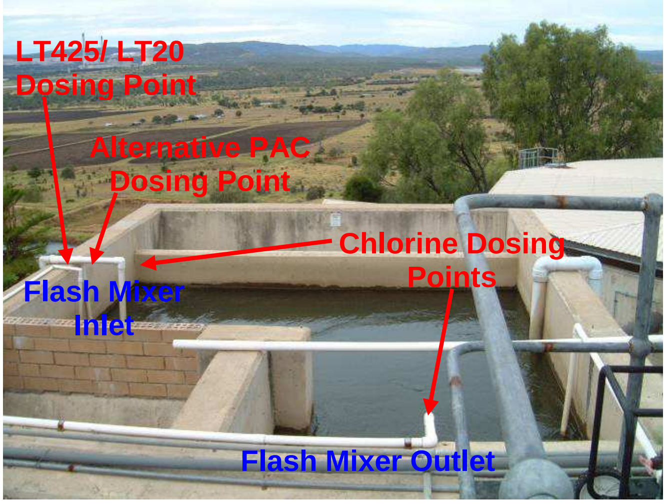


Photo of Chemical Dosing Points at Flash Mixer

4.4.2 Coagulant Dosing

The primary coagulant, alum, is dosed as a solution into the raw water pipeline before the water enters the flash mixer. Alum is dosed as a solution made up from powdered alum. Chemical doses used and the alum makeup and dosing system are addressed in the next chapter of this report.

It is noted that adequate mixing is essential for effective coagulation. Coagulant mixing is normally achieved via the inline mixer in the main just before flash mixing tank (with further hydraulic mixing in the flash mixing tank).

It is noted that the sludge lagoon supernatant return line joins the raw water main just after the alum dosing point. Ideally the supernatant would be mixed with the raw water before coagulant dosing, although the dosing point and supernatant addition are close enough that this may not be a major issue.

4.4.3 Secondary Coagulant (Cationic PolyDADMAC) Dosing

Cationic polydadmac (LT425) is dosed as a secondary coagulant. The dosing point used is at the inlet to the flash mixing tank. Chemical doses used and the cationic polydadmac dosing system are addressed in the next chapter of this report.

It is noted that the same dosing point is used for both cationic polydadmac (LT425) and polyacrylamide (LT20), with the delivery pipework from both systems joining before the dosing point in the flash mixer. **Ideally, separate pipework should be provided for the two types of polymer**, as they are likely to interfere with each other. It was reported that plant operation was less successful when LT425 and LT20 were dosed together (through the same dosing line).

4.4.4 Coagulant Aid Polyacylamide (LT20) Dosing

Coagulant aid polyacrylamide (LT20) is dosed occasionally as a coagulant aid when high turbidity raw water is experienced. The operators report that the filters have higher headloss accumulation rates when coagulant aid is dosed.

As reported above, dosing LT20 in same pipe as LT425 is likely to cause problems due to interference. These products should be separated for best effect.

It may be possible to optimise the use of polyacrylamides as coagulant aids, for example by using alternative polymer products, doses and/ or dosing points. **Jar testing or plant**

trials could be undertaken to further test the usefulness of polyacrylamides as coagulant aids.

4.4.5 Pre-Coagulation Lime Dosing

Due to typically high alkalinities of the raw water, pre-lime dosing has never been used, however there is provision for it in terms of a spare dosing ejector and dosing point on the raw water main leading to the plant. Pre-lime dosing would only be used if raw water alkalinity were particularly low and/or high alum doses were required. The lime dosing system is detailed in the next chapter of this report.

4.4.6 Pre-Filtration Chlorine Dosing

As well as the pre-chlorination carried out at the raw water pump station, further pre-filtration chlorine is also dosed at the WTP flash mixer (prior to the clarifiers).

Pre-coagulation chlorine at the WTP can be dosed at either the inlet or at the outlet of the flash mixing tank. It is normally dosed at the outlet of the flash mixing chamber. Previously, pre-chlorine could also be dosed into the raw water main near the alum dosing point, however this dosing point is not currently connected and is not used due to reported problems with the non-return valve arrangement.

The pre-filtration chlorine dose to the flash mixing tank is adjusted to give a residual of around 0.3 mg/L, measured in the clarifiers.

Chemical doses used and chlorine storage and dosing system capacities are discussed in the next chapter of this report.

The following comments are made on the existing arrangements for pre-chlorine dosing:

- Chlorine can be effective for oxidising some taste and odour compounds and some algal toxins. Chlorine can also effectively oxidise dissolved iron and manganese.
- The risks of dosing chlorine to raw water include the potential formation of by-products such as THMs. **THM levels should be monitored.**
- If algal cells are present, chlorine may lyse the cells, potentially releasing toxins from the cells into the water. Pre-chlorine dosing should not be used if large numbers of potentially toxic algae are present in the dam water.
- Pre-chlorination at the raw water pump station will give the chlorine a long contact time for oxidation, however the chlorine demand will potentially be high because of the organics in the raw water and biofilms on the raw water main.
- The second stage pre-chlorination dosing point at the outlet of the flash mixer may not allow substances oxidised by the chlorine to be effectively bound up in floc.
- If PAC is dosed to the flash mixing tank, there is the potential that it may absorb some of the chlorine residual. Pre-chlorination to the flash mixer should not be used when PAC is dosed to the flash mixer as neither process will be efficient.

4.5 Flash Mixing Tank

4.5.1 Flash Mixing Tank Parameters

Water flows from the raw water main into the bottom of the flash mixing tank. The tank provides an opportunity for hydraulic mixing and detention time after coagulant dosing, with further chemical dosing to the flash mixing tank itself (see above). Water flow out of the flash mixing tank to the clarifiers via a weir outlet.

The flash mixing tank acts as an emergency overflow for the whole plant. If more flow is received than can be processed through the plant components, the water level will build up in the filters and clarifiers until the flash mixing tank level reaches the overflow weir level. The overflow system is designed to take a flow rate of 120 L/s (according to the old operations manual). The overflow weir box discharges to the stormwater system (draining to the surrounding environment).

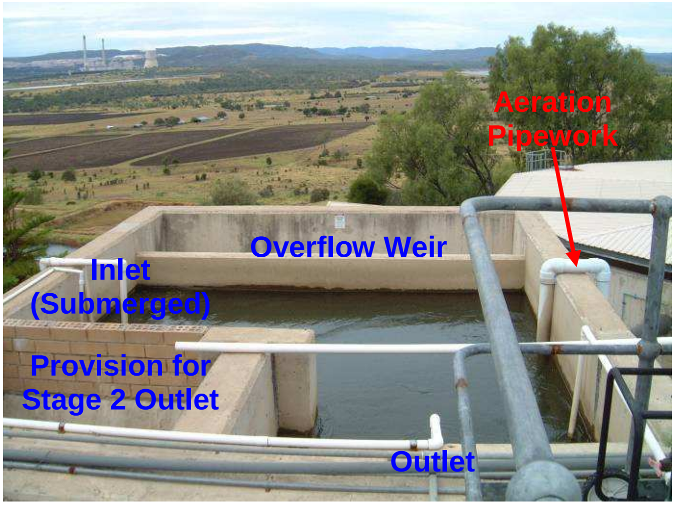


Photo of Flash Mixing Tank Components

The flash mixing tank has been sized for a detention time of 4 minutes at 120 L/s. It is noted that the actual flow through the flash mixing tank is generally higher because of the additional sludge lagoon supernatant recycle flow (say around 125 L/s, giving 3.9 minutes detention time).

The design of the flash mixing tank allows for future upgrades increasing the plant flow up to 240 L/s, plus the hydraulic over load (total flow of 280 L/s) according to the original up-rating options outlined in the original operating manual. The bricked-up outlet from the flash mixer was designed to be used as an outlet to future Stage 2 clarifiers.

According to the original operating manual, for an increase in flow rate up to 160 L/s (Stage 1A) the flash mixer weir would not need to be modified and the additional flow would simply increase the operating head over the weir. The operators report that the plant has been run briefly at 180 L/s to test the hydraulics, however this could be re-confirmed by conducting further plant trials. Modification of the flash mixing tank outlet weir (opening of the bricked-up outlet) would be required for any upgrades requiring the addition of new clarifiers.

If it was intended to use the one flash mixing tank for an upgraded plant, the effective detention time in the tank would be reduced significantly and testing to confirm the required floc formation times in the flash mixer should be undertaken.

The main criteria for the flash mixing tank are summarised below.

Flash Mixing Tank

Component	Parameter (Units)	Design Criteria	Comments
Flash Mixing Tank	Depth (m), Volume (m^3)	3.5, 29	
	Hydraulic capacity (L/s)	280	Based on original up-rating design (240 L/s plus hydraulic overload)

Component	Parameter (Units)	Design Criteria	Comments
	Detention time	4 minutes at 120 L/s 2 minutes at 240 L/s	Based on original operations manual
	Flash mixing energy	Hydraulic mixing only	

4.5.2 Flash Mixing Tank Aeration System

A standby aeration system is installed next to the flash mixing tank, which delivers air into the flash mixing chamber via a PVC pipe. This system has been used in the past as additional treatment for tastes and odours, but has reportedly not been run for the last 5-10 years except for regular start-ups to maintain the blower in working order. The operators reported that the system was not favoured as the aeration can lead to the flotation of floc, impeding settling. It was not clear whether the aeration process was beneficial or not for taste and odour reduction.

It is noted that the blower has been set up to aerate intermittently, controlled by a timer, to try to minimise the effect of floc flotation. The details of the aeration system such as the capacity of the air blower are not known.

Photo of Aeration Blower at Flash Mixing Tank

4.6 Clarifiers

4.6.1 Clarifier Overview

Flocculation occurs via hydraulic mixing in the flash mixing tank and within the sludge blanket as water flows up through the clarifiers.

There are two upflow clarifiers, each fitted with settling tubes. Water from the flash mixer is split hydraulically between the two clarifiers via symmetrical clarifier inlet pipework. Flow enters the clarifiers at the level of the bottom hoppers, discharging from the inlet downpipes towards the hoppers' bottom then being deflected upwards by a baffle system.

The discharge velocity through the downpipes is designed to keep most of the sludge in suspension.

A system of 60° settling tubes are used to catch lighter rising floc where they eventually settle out and conglomerate into larger floc at the bottom of the tubes, enhancing the settling rate. Clarified water rises past the top of the tubes, is collected by three launders in each clarifier (6 total), and passes to the filter inlet channel.

A sludge blanket is formed in the hoppers and below the tube settlers and is constantly mixed with the incoming water from the flash mixer and kept in suspension.

Photo of Clarifier Showing Settling Tubes

It was noted that the settling tubes are old and are becoming brittle, requiring careful treatment by the operators during cleaning etc. Some replacement settling tubes are stored in the filter gallery of the WTP. **Replacement of the settling tubes (as required) will need to be considered in future works programs.**

It was also noted that there is a lack of suitable walkways to access the top of the clarifiers for operational duties, as discussed further Section 6 of this report below.

The original plant operating manual outlined a number of options for up-rating the plant to a higher flow rate, one of which involved running the existing clarifiers at a rate of 160 L/s. The hydraulic capacity of the clarifier vessels is therefore understood to be at least 160 L/s. However the up-rating plan was subject to testing of the clarifier settling performance under the higher loading rates. At the higher flow rate, larger diameter discharge nozzles would need to be fitted to the clarifier down pipes which discharge dosed water into the hoppers in order to maintain a suitable discharge velocity.

The main parameters for the clarifiers are summarised below.

Clarifiers

Component	Parameter (Units)	Design Criteria	Comments
Clarification	Hydraulic capacity (L/s)	160 L/s (through both clarifiers)	Based on original up-rating plan. Modification to downpipe discharge nozzles required for plant flows <120 L/s

Component	Parameter (Units)	Design Criteria	Comments
	Clarifier type	Upflow solids contact with settling tubes	Based on original operations manual
	Surface rating (m/h)	3.84 at 120 L/s 4.16 at 130 L/s 5.12 at 160 L/s	Based on original operations manual
	Dimensions: length, width (m), Settling zone surface area (m^2)	7.5 x 7.5m, 56.25 m^2 per clarifier	Based on original operations manual
	Depth (m)	5.5 m total depth (includes 2.4 m hopper depth)	Based on original operations manual. Settling tube depth 0.53 m
	Sludge scraping	No sludge rake	
	Sludge drawoff system	Automatic desludge (taken from overflow weir at top of hoppers) Manual scour valve taken from bottom of hoppers	
	Sludge hoppers	4 hoppers per clarifier	
	Tank drainage facilities	Manual sludge bleeds take water from near the base of the hopper	Because withdrawal pipes rise up over hopper walls, gravity removal of sludge is only possible to top of hoppers. Further draining achieved with flexidrive pump
	Typical sludge removal frequency, duration (sec), volume	Automatic: 15 min off, 1.5 min on Manual scours: approx. 2 x week	Rarely adjusted. May be increased if poor raw water quality

The surface rating quoted in the original operating manual for the clarifiers at Stage 1 flows is 3.84 m/h. It is noted that the actual flow through the clarifiers at a raw water flow rate of 120 L/s would be up to 8 L/s higher because of the additional supernatant recycle stream. An effective flow rate of 130 L/s is equivalent to a surface loading rate of 4.16 m/h. Typically, plate or tube settlers are given ratings of up to 4 or 5 m/h, so it appears that the clarifiers were designed to operate at a reasonably high rate for this technology.

The operators report that settling performance in the clarifiers at a raw water inflow of 120-125 L/s appears to be reasonable under most circumstances, however when high solids loadings or algae are experienced it has been necessary to down-rate the plant to a flow of 80 - 100 L/s (surface loading rate of 2.56 – 3.2 m/h). It is suspected that the reason for poor settling at these times may have been a combination of non-optimal coagulation and limitations in the design of the clarifiers to handle high solids loads.

With typical raw water quality, the operators feel that a raw water inflow of around 130 L/s (clarifier flow rate of up to 138 L/s if supernatant recycled) is the maximum flow rate achievable through the clarifiers for effective settling, and report that the sludge blanket was seen to lift when a plant inflow of 140 L/s was trialled for 24 hours, causing excessive floc carryover. The adjustment of the clarifier downpipe nozzles which control the flow velocity of inflow into the hoppers (as mentioned in the original WTP operating manual) and/or upgrade of the settling tube design could be further investigated if it was required to run flows of > 130 L/s through the clarifiers, although based on general industry experience it is expected that the effective surface loading rate in this type of clarifier would be limited to 5 m/h (156 L/s) in most cases.

Based on the information available on the clarifier performance above, it would appear that the maximum clarifier process capacity is around 130 L/s under typical conditions and possibly as low as 80 L/s under worst case raw water quality.

For poor raw water quality conditions, the optimisation of chemical dosing, including optimisation of coagulant doses and addition of pre-coagulation lime and/ or flocculant aid if required could possibly improve settling to allow higher flow rates under these conditions. **Jar testing simulating dirty raw water quality could be used to investigate the optimum dosing regime for such waters.**

Preventing the development of biological growth on the clarifier tubes would also potentially enhance settling. This is presently achieved by regular cleaning of the tubes, but could be assisted by covering the clarifiers, as noted later in this report.

4.6.2 Clarifier Sludge Blowdown

The operators check the height of the sludge blanket by taking samples through a series of valves on the side of the clarifier. The sludge blanket level is usually adjusted, when necessary, by opening the manual scour valve located on each clarifier. The frequency and duration of sludge draw-off are adjusted occasionally.

It was noted that the sludge draw-off system has the following problems:

- The automatic sludge blowdown withdraws sludge from the sides of the hoppers (rather than the bottom) and, although this was the part of the original design (intended to retain some of the solids to build the sludge blanket), sludge can accumulate at the bottom of the hoppers and turn anoxic, making the taste and odour problems worse;
- When draining the clarifiers for periodic maintenance, the manual desludge valves can not drain the hoppers fully due to hydraulics of the draw-off line.

The sludge buildup problem associated with the drawoff point for the automatic sludge blowdown is overcome by the operators by performing manual desludges around twice a week, as the manual desludge lines withdraw sludge from closer to the bottom of the hoppers. The manual desludges are normally performed when the plant is off-line and the sludge fully settled. **The addition of automatically controlled valves on the manual desludge lines (set to desludge at regular intervals, perhaps each time the plant is off-line) could be considered to make operation of the existing clarifiers easier. For any future upgrades, the design of the new clarifiers should avoid this problem of extended periods of sludge accumulation.**

The problem with draining the clarifiers for periodic maintenance is due to the design of the manual desludge pipes, which travel from the drawoff point at the base of the hoppers, up over the top of the hoppers then down again to the drain point. When the water level in the clarifiers is low, there is not enough head to push the water through these pipes to drain the hoppers fully. The operators address this problem by pumping out the last water from the clarifier hoppers (approx 2 m depth) with a sump pump. As the arrangement of the desludge lines is not easily changed and the clarifiers only need to be drained one to two times per year, it is likely that pumping out the water is the best option for the clarifier draindown. **For any future upgrades, the design of new clarifiers should ensure suitable draining facilities.**

Photo of Sludge Testing Valves and Manual Sludge Bleed Valves

Photo of Automatic Sludge Bleed Valves

4.7 Powdered Activated Carbon (PAC) Dosing

Powdered activated carbon (PAC) is dosed under most circumstances at the WTP in order to address taste and odour problems. The PAC dosing system makes up the PAC into a slurry for transport to the dosing point. Chemical doses used and the PAC dosing system capacities are discussed in the next chapter of this report.

Photo of Normal PAC Dosing Point in Filter Inlet Channel

The PAC is normally dosed prior to filtration, at the end of each clarifier launder where clarified water empties into the filter inlet channel. One of these filter inlet channel dosing points is shown in the figure below. The continuous dosing of PAC to the filters requires a lower PAC dose compared to the raw water and gives a long contact time as the PAC stays in the filter bed, however there is a risk that the uncoagulated PAC fines may overload or pass through the filters.

As an alternative dosing point, the PAC can be dosed at the inlet to the flash mixing chamber, prior to the clarifiers. Under this arrangement the clarifiers would be expected to remove the bulk of the PAC particles, which will be bound up in the floc, leading to potentially longer filter run times. The operators reported that the flash mixing tank dosing point required a significantly higher dose of PAC compared to the filter inlet channel. This is to be expected, as the raw water will have a higher PAC demand than the clarified water and, if pre-chlorine dosing is continued when PAC is dosed, the chlorine will react with the PAC making both chemicals less effective.

A split dosing arrangement, with some PAC dosed to the flash mixer and a smaller dose to the filters, could also be suitable. The operators advised that they had trialled this arrangement during a period of 2006.

If the PAC dosing system is unavailable, PAC can also be dosed manually into the filters. This would be done by adding an appropriate amount of PAC (calculated based on the required dose and the water volume expected for each filter run) to the water above the filter at the start of each filter run. The PAC is mixed to a slurry and then added via a funnel and the dosing downpipes installed in each filter. It is noted that this manual system is similar to the manual PAC dosing arrangements used at the other Banana Shire Council WTPs.

The PAC product used is James Cumming and Sons (C&S) product No. MDW 3545CB. This is a coal based PAC with a high iodine no. of 1050. It is an appropriate choice for general algal toxin and taste and odour removal. As Biloela WTP in particular appears to have ongoing problems with taste and odours, **a range of different PAC products could be compared in jar testing to find out whether there is a product more effective for the particular taste and odour compounds of interest and to look at adsorption contact time requirements.**

If taste and odour compounds and algal toxins are unable to be satisfactorily addressed by PAC dosing, an alternative would be to include an ozone- biological activated carbon (BAC) process to remove these compounds from the filtered water. It is noted that the ozone BAC process is very effective for the removal of various organic contaminants. It would however be expensive in terms of both capital and operating costs compared to the current plant process.

4.8 Filtration and Backwashing

4.8.1 Filter Overview

Water from the clarifiers flows to the filter inlet channel via six launders (three in each clarifier). From the filter inlet channel, the flow is split evenly between the four filters by adjustable inlet weirs. Flow enters the filters via inlet plug valves. The filters work on the principal of constant flow rate, rising head filtration, where the inflow is equally split between the filters throughout the filter runs and the water level in the filter slowly builds up as the headloss across the media bed increases.

The filters contain single media filter sand beds (with gravel base). Filtered water is collected in the underdrain system below the filter media and transported to the filtered water outlet manifold. In the common filtered water well, a fixed overflow weir located slightly above the surface of the filter media beds, prevents the water level dropping below this level and ensures filters do not drain down below the top of the media beds.

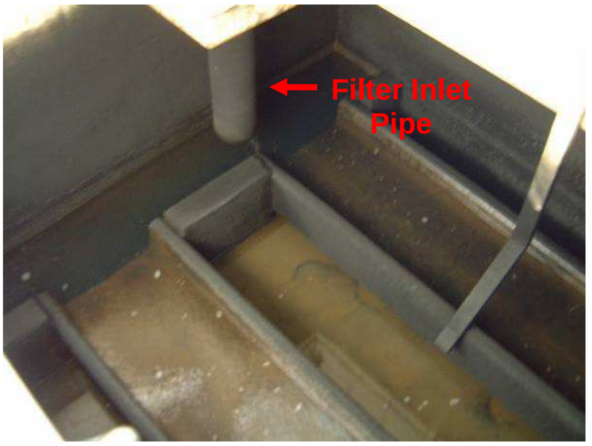


Photo of Filter Showing Inlet, Media and Backwash Troughs

The main parameters of the filtration system are summarised in the table below.

Filters

Component	Parameter (Units)	Design Criteria	Comments
Filter Beds	Hydraulic capacity (L/s)	160 L/s	Based on original up-rating plan
	Type	Gravity feed	
	Number of filters	4	
	Surface dimensions (m) and area per filter (m^2)	3.6 x 3.6 m, 12.5 m^2	Based on original operations manual
	Total Filter Area (m^2)	50 m^2	
	Filtration Rate (m/h) All filters operating One filter offline Two filters offline	 8.64 m/h at 120 L/s 9.36 m/h at 130 L/s 11.52 m/h at 160 L/s 11.52 m/h at 120 L/s 12.48 m/h at 130 L/s 17.28 m/h at 120 m/h 18.72 m/h at 130 L/s	 (i.e. 30 L/s to each filter) (i.e. 32.5 L/s to each filter) (i.e. 30 L/s to each filter) (40 L/s to each online filter) (43.3L/s to each online filter) (60 L/s to each online filter) (65 L/s to each online filter)
	Filter run times: Typical (hrs)	24 – 35 (hours online) Up to 38 hrs in winter	Noted that run times are longer when PAC not being dosed
Filter Media	Filter sand depth (mm)	900 sand Filter 4 has additional 50mm coal layer added April 2009 as a trial	Effective size of sand not known Coals: C&S brand coal UC1.7mm, ES not known
	Filter gravel depth (mm)	250	Effective size not known
Underdrains	Underdrains	ABS laterals cast into concrete. Polypropylene filter hoods (nozzles)	From original manual

Component	Parameter (Units)	Design Criteria	Comments
Filtered water well	Hydraulic capacity (L/s)	280 L/s	Based on original up-rating plan (240 L/s plus hydraulic overload)

It is understood that the grain size of the sand media is not known. **A sample of the sand filter media should be analysed for effective size and uniformity coefficient.**

Filter run times of 24 – 35 hours are reported, with longer run times achieved when PAC is not dosed. The run times are based on the time each filter is actually online (i.e. not counting hours that the plant is off). The operators report that under typical circumstances, two of the filters are backwashed each day.

It is noted that the actual flow through the filters at a raw water flow rate of 120 L/s may be up to 8 L/s higher because of the additional supernatant recycle stream. At flow rates of 120 – 130 L/s, the filtration rates of 8.64 – 9.36 m/h are reasonable for a mono-media sand filter of fine to coarse sand media. For sand of effective size up to around 1.0 mm, the maximum filtration rate is typically around 10 m/h. Coarser media (say, grain size > 1.2 mm) can take higher flow rates of up to 12 – 15 m/h. Higher flow rates may be possible, but may result in poorer filtered water quality. Filter aid polymer can sometimes be used to improve filtration performance at higher filtration rates.

For flow distributed between three filters when one is taken off-line for backwash, the filtration rate of 11.52 to 12.48 m/h is at the high end of the range usually applied to sand mono-media. **The best practise in terms of flow control through the filters would therefore be to wash the filters when the plant is off-line (the current normal practice)**. It is noted that, if washing the filters while the plant is online, ideally a minimum number of six filters would be recommended so that the flow change was not as great when one was taken off-line for backwashing.

It is also noted that having the plant start and stop during each filter run is not ideal, as the change in flow rates can cause shocks to the media which can shear floc through the media bed.

According to the original operating manual, the filter cells have been sized with a hydraulic capacity of at least 160 L/s, to allow the potential upgrading of the existing mono-media filters to a mixed media design which may be able to treat this higher flow rate (equivalent to 11.52 m/h with all filters online and 15.36 m/h with the flow distributed between three filters). A dual media has been trialled in Filter 4 during 2008-09 by adding a layer (around 50mm deep) of C&S brand filter coal (size not known, UC 1.7). The operators reported that performance of this filter was similar to the other filters, although there is a lack of instrumentation to enable a detailed comparison of the performance of the new media configuration. It is noted that the configuration trialled in Filter 4 does not follow the typical modern dual media design which would normally employ a deep layer of coal and a smaller layer of fine sand (e.g. 1000mm of coal plus 300 mm of sand).

It is uncertain whether a deep bed dual media design could be retrofitted into the existing filter beds because there appears to be little clearance between the surface of the existing 900mm deep sand layer and the backwashing troughs. Further, a deep bed dual media design may different backwashing conditions to those currently provided. Therefore the suitability of the filter cells and backwashing system for upgrade to a dual media design should be thoroughly investigated before this is accepted as a practical option to increase plant throughput and, assuming such media could be retrofitted into the filter beds, the maximum rates for the upgraded media beds should be proven by pilot testing before the design for any upgrade is finalised.

Based on the consideration of filtration rates, a maximum flow rate of 125 - 130 L/s through the filters is expected to give the best performance in terms of water quality and run times. Higher plant flow rates could possibly be applied for short periods, provided all four filters were on line.

It is noted that the filtered water well at the discharge of the combined filter outlet pipe is reportedly designed to handle up to Stage 2 normal (240 L/s) plus overload flows. The original up-rating plan allowed for new Stage 2 filters to discharge into this well.

It was noted during the CWT site visit in March 2006 that the air scour distribution appeared to be slightly uneven in Filter #2. **The condition of the underdrain system should be examined when practical**, for example if the filter media is removed for an upgrade.

4.8.2 Filter Backwashing

The purpose of backwashing is to remove the floc and other solids form the filter media at the end of a run. Air scour is often used before water washing to loosen the floc from the media grains. The air scour and water washing rates should be carefully designed and controlled to provide adequate washing without disturbing the media bed or washing filter media into the wastewater collection troughs.

At Biloela WTP, backwashing is performed manually, controlled by the operators via filter backwash consoles. The operators manually open the appropriate valves and start and stop the backwash pump and blower based on a stopwatch. **Automation of the backwash cycle would be preferred to free up the operators to do other operational tasks**.

The main trigger for backwashing is the water level in the filter. Level switches in each filter trigger an alarm indicating a backwash is required when the water level reaches the trigger level. There are no headloss meters on the filters. Operators may also perform backwashes if they notice a rise in filtered water turbidity (measured by an online meter).

It is noted that headloss trends can provide useful operational information on filters**. The retrofitting of headloss meters could be considered.**

Photo of Main Backwash Control Panel

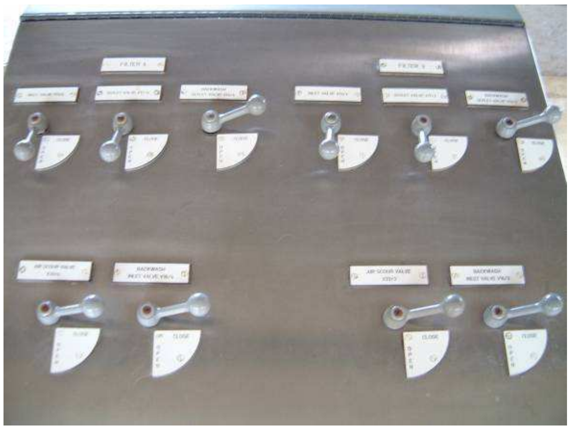


Photo of Filter Valves Control Panel

Typically, filters are backwashed when the plant is offline. Two filters are normally backwashed consecutively. The backwash phases include airscour, combined air and water wash and high rate water wash.

The main backwash related parameters are summarised below.

Filter Backwashing

Component	Parameter (Units)	Design Criteria	Comments
Backwashing Parameters	Backwash control	Manual only	
	Backwash phases	• Air only • Air plus low rate water • High rate water	
	Backwash triggers	• Level rise in filter • High combined filtered water turbidity	Headloss meters could be installed and used as backwashing triggers
	Effective Headloss (m)	Not known	
	Backwash frequency (hours)	See filter run times above	
	Air Scour blower capacity (m^3/h)	318 at outlet	Based on original operations manual. Delivery pressure 50 kPa
	Air scour rate (m/h, m^3/h), typical time (min)	25.4 m/h, 318 m^3/h, 3 min	
	Backwash pumps: No. of, capacity (m^3/h)	2 x 700 m^3/h (Duty/ standby)	Based on original operations manual. Discharge pressure 18m
	Water scour rate (m/h, m^3/h), typical time (min)	Low rate: 33 m/h, 415 m^3/h High rate: 56 m/h, 700 m^3/h Combined air/ water: 1 min Low rate: 1 min High rate: 3 – 4 min, or until clean	

The airscour rate quoted is lower than typical airscour levels used, but on observation seemed to aerate the media suitably. It is noted that there is no standby for the air scour blower. Backwashes have been performed without air scour at times when the blower was not available due to maintenance etc.

The water wash rate is a typical backwashing rate but should be reviewed further when the sand filter media size is known. Backwash water is supplied from the clear water tank by duty/ standby backwash pumps.

Photo of Airscour Blower

Photo of Backwash Pumps

4.9 Post-Filtration Chemical Dosing

4.9.1 Post-Filtration Lime Dosing

Post-filtration lime ('post-lime') is dosed to correct the pH of the water after coagulation and filtration to a level suitable for release into the distribution system. The post-lime is dosed into the common filtered water pipe just before the filtered water well. The lime dosing relies on hydraulic mixing in the filtered water well weir and pipes to the clear water tank for distribution in the water.

The post-lime dosing rate is adjusted up or down based on the measured pH in relation to the pH target. (Target values discussed in Water Quality Issues section of this report).

Chemical doses used and lime storage and dosing system capacities are discussed in the next chapter of this report.

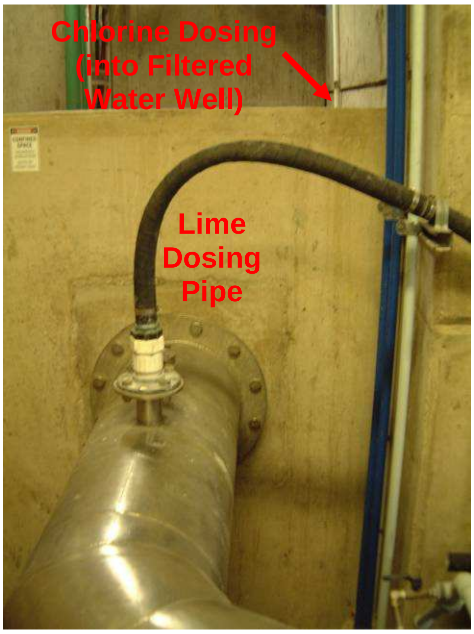


Photo of Post-Lime and Chlorine Dosing Points

4.9.2 Post-Filtration Chlorine Dosing

Post-filtration chlorine ('post-chlorine') is dosed for the purpose of disinfection, and to contribute a residual chlorine concentration to the water leaving the plant.

The post-chlorine is dosed into the weir inside the filtered water well, shortly after post-lime dosing. The post-chlorine dose is adjusted to meet the final water chlorine residual target (Target values are discussed in Water Quality Issues section of this report).

It is noted that water flowing from the WTP clear water tank to the town reservoirs is rechlorinated at the mixing tank where it is blended with the town bore water.

Chemical doses used and chlorine storage and dosing system capacities are discussed in the next chapter of this report.

4.10 Clear Water Storage and Gravity Trunk Mains

From the common filtered water well, water passes via the filter outlet pipework via gravity to the clear water storage tank located downhill from the plant.

Photo of Clear Water Storage Tank

The filtered water well, intermediate pipework and clear water tank are reportedly designed to handle up to Stage 2 (240 L/s) flows plus hydraulic overload, giving a total hydraulic capacity of 280 L/s.

From the Clear Water Tank, treated water gravitates to:

- Biloela town supply (mixing tank and ground level reservoirs at State Farm Rd;
- Callide Township; and
- Callide Power Station (into a 0.1 ML reservoir)

Flow rates to these various consumers are discussed in the first chapter of this report.

Clear Water Tank

Component	Parameter (Units)	Design Criteria	Comments
Clear Water Storage Tank	Type	Round, concrete tank	
	Diameter (m)	24 (internal diameter)	Based on original operations manual
	Depth (m)	4.8 (water depth) 4.85 (to overflow) 5.05 (overall height)	Based on original operations manual
	Total Capacity CWT (ML)	1.8 (working capacity) 2.0 (to overflow)	Based on operator advice

Component	Parameter (Units)	Design Criteria	Comments
	Detention time	4.16 hrs at 120 L/s 3.13 hrs at 160 L/s 2.50 hrs at 200 L/s 2.10 hrs at 240 L/s	Theoretical detention time if no short circuiting and full tank working capacity used
	Hydraulic capacity (L/s)	280	Based on original operations manual
Clear Water Pumps	No. and Capacity each (L/s)	No pumps. Feed to town and to power station is by gravity	
Trunk mains to town	Diameter (mm), Length (km)	300, Approx. 11 km	
	Flow capacity (L/s)	At least 70 L/s	Limited by mixing tank hydraulics

It is noted that there is additional detention time in the Biloela town supply system main and ground level town reservoirs. Callide Township, however, receives the water straight from the WTP clear water tank, therefore an adequate detention time is critical for chlorine disinfection.

The clear water tank would theoretically provide over 4 hours detention at 120 L/s if no flow short circuiting occurred. It was noted that the flow enters the clear water tank above the water surface through a vertical section of pipe and exits near the base of the tank, which should reduce short circuiting, although some short circuiting is likely to occur in any tank which does not have effective baffling. Flow patterns could be investigated in more detail by further studies.

Based on methods used by the US EPA (USEPA, 1999), assuming that the baffling effect in the tank is ‘poor to medium’, the *effective* disinfection contact time for 90% of the water passing through the tank may be only around 0.3 to 0.5 times the theoretical detention time, i.e. 1.25 – 2.1 hours at 120 L/s. Disinfection times in this range would be adequate for at least 4 log inactivation of viruses and may achieve 1 to 2 log inactivation of Giardia based on a chlorine residual of 0.6 mg/L, pH of 7.5 – 8.0 and temperature 10-20◦C (USEPA, 1999).

At increased flow rates through the clear water tank, the lower contact times would reduce the disinfection effectiveness. The actual target disinfection times depend on the log inactivation targets adopted. If required, the effective disinfection time could be increased by installing baffles in the existing tank, or by providing another tank for additional volume.

During the site visit in May 2009, the operators noted that **the clear water tank is leaking and requires significant maintenance work**. The operators are unable to take the tank off line for maintenance because there are no bypass provisions and, even if possible, bypassing this tank would not be advised as it would lead to insufficient disinfection contact time for water to Callide Township. **A second clear water tank should be provided** with valving to allow one tank to be taken off line for repairs and maintenance while the other remains online. It is understood that plans are underway (May 2009) and budget has been allocated to provide a second clear water tank equal in capacity to the existing tank. The increase in potential clear water tank storage volume would also be of benefit in terms of flow buffering to reduce the number of WTP startups per day.

The treated water flow meter is located on the gravity mains after the clear water tank. The treated water flow rate should equal the sum of the power station, Callide Township and the town supplies flow rate measurements.

Flow by gravity through the trunk mains is controlled by:

- An automatic on/ off valve
- A manual valve used to adjust flow rate.

It is understood that the flow rate is generally limited to around 70 L/s using the manual valve because of hydraulic limitations at the inlet mixing tank to the town reservoirs (see below). However the planning report by SKM (2006) also identified hydraulic limitations in the gravity flow through the trunk main and recommended that **a booster pump be fitted to the trunk main to achieve adequate flow during peak demand conditions,** including a pump bypass allowing non-peak demands to be supplied without pumping.

4.11 Reticulation and Town Bore Blending System

4.11.1 Blending of Waters and Ground Level Reservoirs

The WTP treated water is blended with the town bore water in the mixing tank on the side of the 9 ML town ground level reservoir (GLR) No.1. From the mixing tank, water flows through a bell-mouth outlet into GLR No.1 and/or GLR No.2. The 1.5 ML GLR No.2 is interconnected to GLR No.1 as additional storage.

Chlorine is dosed to the blended water in the mixing tank. The chlorination equipment is located in the High Lift (HL) Pump Station. It is noted that at one stage the addition of facilities to dose soda ash for pH correction and fluoride and the upgrade of the existing chlorination facilities was being considered, as outlined in the “Biloela Water Supply Chemical Correction Planning Report”, June 1997.

The hydraulic capacity of the mixing tank and bell-mouth draw-off is known to be limited, especially when the 1.5 ML GLR No.2 is online alone, and the tank has been known to overflow under certain conditions. The hydraulic capacity of the bell-mouth draw-off is lowest when the GLRs are full, and is roughly estimated to be around 100 L/s (total flow from WTP main and bores). **The hydraulic limitations of the mixing tank should be further reviewed.**

The town ground level reservoir parameters are summarised below.

Town Mixing and Ground Level Reservoirs

Component	Parameter (Units)	Design Criteria	Comments
Town Reticulation	Ground level reservoir No.1	Capacity 9 ML	
	Ground level reservoir No.2	Capacity 1.5 ML	
	Mixing tank	Flow Capacity approx. 100 L/s (worst case)	Tank outlet bell-mouth maximum flow rate is limited

4.11.2 Town High Lift Pumps, Elevated Reservoirs and Reticulation

Water is drawn from GLRs No.1 & 2 into the HL Pump Station at the State Farm Rd site. The HL Pump Station includes two Low Zone HL centrifugal pumps (duty/standby) and two High Zone HL centrifugal pumps (duty/standby).

The Low zone HL pumps operations are controlled by the water level in the Low Zone Elevated Reservoir at the State Farm Rd site. The High Zone HL pump operations are controlled by the water level in the High Zone Elevated Reservoir on a reserve off Earlsfield St.

Council report that all domestic services are fitted with metric positive displacement meters, with helical rotor meters for larger installations.

The town reticulation parameters are summarised below.

Town Reticulation

Component	Parameter (Units)	Design Criteria	Comments
Town Reticulation	Low Zone HL Centrifugal Pumps	Duty/Standby pumps Capacity 94 L/s	Controlled by water level in Low Zone Elevated Reservoir
	High Zone HL Centrifugal Pumps	Duty/Standby pumps Capacity 150 L/s	Controlled by water level in High Zone Elevated Reservoir
	Low Zone Elevated Reservoir	Capacity 1.36 ML	Located at State Farm Rd
	High Zone Elevated Reservoir	Capacity 1.95 ML	Located on a reserve off Earlsfield St
	Reticulation Mains	100 & 150 mm AC mains	
	Service Water Lines	20 mm polyethylene pipes	

4.12 Wastewater System

4.12.1 Estimated Wastewater Production

The following notes are made regarding wastewater production at Biloela WTP:

- Clarifier desludge volumes are reported daily, based on flow totaliser readings;
- Backwash volumes are calculated by the operators based on standard backwash flow rates and wash times;
- Each filter backwash typically produces 40 – 60 kL of wastewater (depending on wash times). Generally 2 filters are backwashed per day (depending on filter run times).

From a preliminary review of operational data from the plant and figures available from the SKM Planning report (2006) and WTP operational data, the following values were estimated for wastewater volume production and sludge lagoon supernatant return.

Estimated Current Wastewater Production Volumes

Wastewater Type	Daily Volume (kL/d)		Percentage of Raw Water Flow
	Expected Range	Typical	
Clarifier Sludge Blowdown	70 – 130	80	Approx. 2%
Waste Backwash Water	80 – 240	120	2 – 6 %
Total Wastewater	**150 - 370**	**200**	**4 – 8%**
Supernatant Return Flow	100 - 500	150	Approx. 4 - 6%. But may not be returned if quality poor

4.12.2 Wastewater System Overview

The wastewater system is designed to capture all the plant's wastewater via gravity to the wastewater collection surge tank and then from there via gravity to the sludge lagoons.

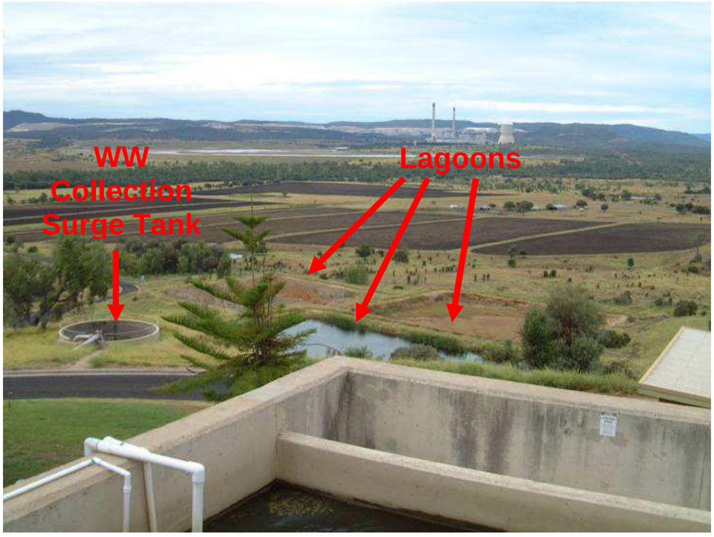


Photo Showing Wastewater System Components

The main parameters of the wastewater system are summarised below.

Wastewater System

Component	Parameter (Units)	Design Criteria	Comments
Wastewater Collection Surge Tank	Capacity (kL)	120 kL working capacity	Takes backwashes and desludges: Max capacity based on 2 x 60 kL backwashes, plus 6 x clarifier desludges delivered over 20 min
Supernatant Return System	Components	Supernatant pump well Supernatant return pumps	Pump start/ stop is controlled by level in pump well

Component	Parameter (Units)	Design Criteria	Comments
	Supernatant return pumps: No. of, Capacity (L/s)	2 (duty/ standby), each 5 – 6 L/s current capacity Original pump capacity 8 L/s, lower now due to aging of pumps	Supernatant is recycled to head of plant, just after alum dosing Flowmeter on recycle line, reported to be inaccurate
	Reasons for Halting Supernatant Return	Supernatant return may be halted when taste and odour problems, as supernatant may concentrate T&O compounds	When supernatant not recycled, lagoons are allowed to overflow over weirs into the adjoining paddock
Sludge Lagoons	No. of, Capacity total (m^3), Sludge (m^3)	3, Capacity not known	Each lagoon designed to receive 6 months sludge at Stage 2 production, i.e. >12 months at Stage 1 (per original operating manual)
	Typical maximum fill level	1 – 1.5 m depth	
	Drainage system	Concrete overflow weir with stopboards (no underdrains)	Drainage includes 'ag pipe' drainage at foot of lagoon outlet wall

4.12.3 Wastewater Collection Surge Tank

The wastewater collection surge tank has a working volume of approximately 120 m^3, designed to hold two successive backwashes over a 20 minute period in addition to 6 clarifier desludges within the same period. Wastewater is designed to enter the tank in a downwards direction in order to create turbulence and scouring of any accumulated sludge on the tank walls. Once collected, flow is steadily released to the sludge lagoons at a low rate in order to prevent disturbance of settled lagoon sludge.

The tank has been sized to meet current sludge volumes and, according to the original operating manual, would meet requirements for an upgraded plant to Stage 2 capacity as the backwash and desludge volumes would be the same except they would be twice as frequent. The volumes would need to be discharged from the tank at up to twice the rate, which can reportedly be achieved by raising the tank level control float valve to give a higher discharge head.

It is noted in the original operating manual that blanked-off pipe tees are provided in the PVC line between the wastewater collection surge tank and the lagoons to allow for the future interconnection of intermediate treatment processes between the tank and the lagoons.

4.12.4 Sludge Lagoons

The sludge lagoons receive wastewater via the wastewater splitter box which distributes flow to those lagoons online. Operation is flexible and controlled by the operators. According to the original operating manual, the lagoons have been designed to each receive 100 % of the wastewater flow for a minimum of six months at Stage 2 sludge production rates, which should give more than 12 months storage per lagoon at the present Stage 1 sludge production rates.

Photo of Lagoon Wastewater Splitter Box

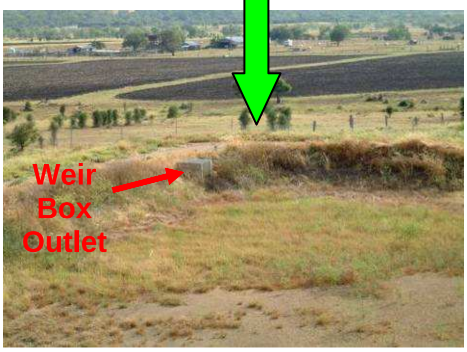
A lagoon is usually taken offline once its water level exceeds 1 – 1.5 m in depth and an offline lagoon is brought into service. Lagoon supernatant is then decanted through manual removal of stop boards located in the lagoon's weir box until the settled sludge layer has been reached. At this point, the drying phase begins. When the sludge is of suitable consistency, removal of the dried sludge is performed through mechanical excavation.

The operators report that the lagoons take several months to dry adequately but that the capacity of the lagoons appears to be more than adequate for the sludge produced at the plant. The operators could not remember any extended wet periods which had impacted on the drying in the lagoons.

The lagoons do not have underdrains, and the operators commented that the organics in the wastewater would be likely to block up a sand filter media and underdrains if they were fitted. The lagoons do provide additional drainage through ag-pipe drains at the foot of the lagoon outlet wall, which are normally plugged during lagoon filling and then opened during the drying phase via stop-plugs in the lagoon outlet wall.

Online Lagoon

Offline Lagoon (Drying)



Offline Lagoon (Standby)

4.12.5 Lagoon Supernatant Return

Supernatant from the lagoons is directed to the supernatant pump well. The supernatant pump well is reportedly designed to have adequate volume for WTP Stage 2 sludge production levels.

From the supernatant pump well, the duty supernatant recycle pump returns the collected water at a steady rate to the head of the plant where it is added to the raw water main just before the flash mixing tank. Pumping from the supernatant pump well is designed to be at a constant rate, with pumping head controlled by float valve in the pump well.

The supernatant return pipe has a flow meter fitted. This flow meter apparently does not always read accurately, possibly because of sludge solids affecting the wheel in the meter. **The supernatant flow meter should be refurbished if possible to record accurately, or replaced with a more suitable type of meter.** The graph of supernatant flow rates below has been prepared from a sample of 2009 plant records. Daily flow volumes were calculated from supernatant flow totaliser readings (noting that there may be some inaccuracy with the meter), and supernatant flow rates were calculated using the plant run hours for each period.

Graph of Sludge Lagoon Supernatant Return Flows

The calculated output of the supernatant return pumps shown in the graph above appears to vary between 3 and 8 L/s, with most values in the range 5 – 8 L/s. The original capacity of these pumps was reported as 8 L/s. The reason for the apparent flow variations has not been proved, and may be due to inaccuracies in the flow meter, varying head conditions or actual variation in the performance of the pumps. **The performance of the existing supernatant pumps should ideally be further investigated through controlled trials to confirm the actual output of the pumps.**

At 5 L/s, the pumps would take around 7 hours to pump the estimated typical supernatant return volume of 130 kL. This rate is reasonable for the current supernatant volumes, but not likely to meet requirements if an upgraded plant rate leads to significantly higher supernatant flows. It is noted from the original operating manual that the supernatant pumps were sized for Stage 1 flows only, with the installation of another equal-sized pump proposed for the Stage 2 upgrade to give three 50% duty pumps (2 duty, 1 standby). It is therefore expected that the supernatant pump capacity would need to be upgraded for plant flows greater than 120 L/s.

The quality of the supernatant is an issue, particularly when taste and odour compounds are in the raw water. The supernatant can contain concentrated levels of these compounds and is reported to cause problems even though at 5 L/s it represents around 4% of the total flow into the flash mixing tank. If the supernatant is not of suitable quality to be recycled, the lagoons are allowed to overflow onto neighbouring land. **In future, the overflow of supernatant may be restricted for environmental and/or water efficiency reasons, therefore treatment of the supernatant to remove problem compounds may need to be considered**.

4.13 Plant Components Capacity Summary

The current capacities of the main WTP unit processes were estimated, based on the review of components outlined above. These values are shown in the table below, along with notes on the main capacity-limiting factor for each unit process, and options which could be undertaken to increase the capacity of that unit process, if required.

Chemical system capacities are addressed in the next chapter of the report.

Process Components Capacity Summary

Component	Main Limiting Factor(s)	Estimated Maximum Capacity	Options to Increase Capacity (if Required)
WTP Process Components			
Raw Water Pumps	Pump condition/ capacity, dam level	180 L/s at normal dam operating level Currently 130 L/s due to low dam level	Upgrade/ add pumps. Operate dam at higher level
Plant Inlet	Inline mixer, flow meter and control valve flow ratings	At least 120 L/s	Modify/ replace limiting components
Flash Mixing Tank	Hydraulic design, required floc formation time	280 L/s hydraulic capacity, however floc formation time to be considered	Add new tank
Clarifiers	Surface loading rate for effective settling	130 L/s (typical raw water) Approx 80 L/s (worst case raw water)	Further optimise settling to increase loading rate. Add more clarifiers
Filters	Filtration rate for effective performance	125 - 130 L/s	Upgrade to dual-media (if possible). Add more filters
Clear Water Tank	Detention time	At least 120 L/s	Install baffles to increase the effective contact time. Add new tank
Town Water Delivery System			
Trunk Main to Town	Gravity flow rate	At least 70 L/s	Add booster pump

Component	Main Limiting Factor(s)	Estimated Maximum Capacity	Options to Increase Capacity (if Required)
Mixing Tank at GLR No.1	Outlet hydraulics	Approx. 100 L/s (worst case)	Change outlet arrangement
WTP Wastewater System			
W/water Collection Surge Tank	Volume	Sized for 240 L/s plant flow	Duplicate tank
Sludge Lagoons	Lagoon volume and drying time required	Sized for 240 L/s plant flow	Provide additional lagoon
Supernatant return pumps	Pump rate	Acceptable for 120L/s flow	Refurbish pumps, add new pumps

From the summary above, it appears that the plant capacity (in terms of instantaneous flow rate) is most limited by the following processes:

- Raw water pumps – limited by current low level in dam;
- Inlet components – some components may be limited to 120 L/s;
- Clarifiers – based on reported settling performance;
- Filters – based on expected effective filtration rates;
- Lagoon supernatant return pumps – may not be adequate if wastewater production significantly increased.

From the review above it can be said that most of the plant components are matched well in terms of meeting the 120 L/d design flow, with no individual component more restrictive than others under normal raw water quality conditions. Under very dirty raw water conditions or the presence of algal cells, the clarifiers may limit plant throughput to 80 L/s due to poorer settling.

A few components, including the clear water tank (depending on disinfection time requirements), the sludge lagoons and potentially the flash mixing tank (depending on floc formation time requirements) are sized for handling flows up to 240 L/s.

5. Chemical System Descriptions, Doses and Capacities

5.1 Chemical System Descriptions

5.1.1 Alum

Alum is supplied in powdered form in 1 tonne bags. The bags are unloaded by forklift into the alum chemical feed hopper in the chemicals room. The screw feeder under the feed hopper meters the powder into the solution mixing tank, where the alum mixes with water and is carried to the dosing point as a solution.

The main details for the alum makeup and dosing system are given in the table below.

Alum System

Component	Parameter (Units)	Design Criteria	Comments
Alum System	Chemical product and Strength (%)	Powdered alum (supplied by Omega)	
	Storage capacity (1 tonne bags)	Approx. 30 tonnes alum in 1 tonne bags stored in chemical loading room	At high capacity use approx. 1 tonne per week
	Number of makeup and dosing systems	1 plus common standby with lime system (appropriate auger size to be inserted in screw feeder)	
	Makeup and dosing system components	Chemical feed hopper Screw feeder Solution mixing tank Ejector	
	Hopper capacity (m^3)	2.55 m^3 (approx. 2 tonnes)	
	Bag unloading arrangement	1 tonne bags moved to special unloading pallet and raised with forklift. Bag untied and unloaded	Dust extraction system available
	Feed system arrangement	Screw feeder meters the required dose rate into solution tank. Constant dilution water flow.	
	Screw feeder capacity (kg/h)	50 kg/h	Based on original operating manual
	Solution tank capacity (L)	> 115	Original 115L rectangular tank was enlarged when high alum doses required
	Dose adjustment method	Adjustable output (i.e. speed) of screw feeder	
	Dosing mechanism	Ejector used to transport solution to dosing point	
	Ejector: No. of, Capacity each (L/h)	1, capacity not known	Ejector suction rate can be varied via suction rate-set valves

Photo of Alum Screw Feeder and Dilution Tank

The alum dose is adjusted by changing the adjustment knob of the screw feeder (by releasing the locking screw and adjusting knob between 0 – 500 units). The operators noted that a given setting may give variable speed output over time due to clutch wear and feeder cleanliness, therefore the drop rate is periodically checked by performing drop tests.

The alum system reportedly runs well with few blockages. A timer system allows the chemical stirrers and ejector to run after the plant has shut down and the screw feeder has stopped, effectively flushing the slurry tank, ejector and delivery pipework.

The screw feeders were reportedly designed to have double the expected Stage 1 capacity, in order to cater for the future possible upgrade to Stage 2 plant flowrates. The operators noted that at very high dosing rates, adequate dissolution of the powder into the carry water should be proved by testing. It is noted that spares are no longer sold for the Wallace and Tiernan alum feeders. Spare parts have been machined or adapted in the past, however **an upgrade of the screw feeder technology could be considered in future so that spare parts can be more easily obtained.**

It is noted that the alum tanks are surrounded by a low (approx. 50 mm high) bund, which is in common with the lime dilution tank. The bund is reportedly designed so any spills drain away directly to the sludge lagoons. **The bunding requirements for the dosing arrangement should be reviewed and the bund enlarged if required.**

It is noted that **a liquid alum system could be considered as an alternative** if the dosing system was upgraded in future. Council have recently (June 2009) reported difficulty in sourcing bulk bag granulated alum from local suppliers. A liquid alum system would have significantly lower manual handling requirements for delivery and unloading, although the storage of this corrosive product would require adequate bunding to prevent spills entering the environment.

5.1.2 Cationic PolyDADMAC (LT425)

Cationic polyDADMAC is dosed as a solution of the liquid product. The solution is made up in a 1000 L bulky box which acts as a mixing and dosing tank. To make up the solution, a forklift is used to lift the container in which the liquid product is supplied. With this container suspended above the mixing/ dosing tank, the required amount is released into the mixing/ dosing tank. Treated water is piped to the inlet at the top of the tank and is added manually to the required via a manual ball valve. The mixing/ dosing tank water level is viewed via the calibration tube (also used for drop tests).

It is noted that chemical unloading into the mixing/ dosing tank could possibly be made simpler and safer by the use of a suitable chemical drum pump, although the operators appear to be happy with the current arrangement.

The main details for the cationic polyDADMAC makeup and dosing system are given in the table below.

Cationic PolyDADMAC (LT425) System

Component	Parameter (Units)	Design Criteria	Comments
Cationic PolyDADMAC System	Chemical product and Strength (%)	Ciba Magnafloc LT425. Dosed as solution of liquid product	
	Batching system	Manual addition of product and dilution with treated water	
	Batching concentration (g/L)	10% v/v = 109 g/L	100 L added to 900L to make up dosing solution
	Storage space (bulky box)	1 x 1000 L bulky box	
	Dosing tank size	1000 L (converted bulky box)	
	Dosing pumps: No. of, Capacity (L/h)	1 x 26.5 L/h	Pump model Encore 100 (US Filter)

Only one dosing pump is available for dosing cationic polyDADMAC. **A standby pump should be kept available for installation in case of failure of the dosing pump**.

The pumping rate is changed by adjusting the pump stroke rate at the adjustment knob on the pump. A calibration tube is used for drop tests to check the pumping rate.

As previously noted, the cationic polyDADMAC should not be dosed in the same pipe as any other polymer (it is currently dosed together with polyacrylamide when both products are used).

Photo of Cationic Polydadmac Dosing System

It was noted that the mixing/ dosing tank is not bunded. **A bund should be provided for the mixing/ dosing tank and also for the container(s) in which the liquid product is supplied and stored on site.**

5.1.3 Polyacrylamide (LT20)

Polyacrylamide is a powdered product which must be wetted up into a solution for dosing. The powder is manually batched, with the required amount added to the mixing tank via a funnel and eductor. Dilution water is also manually added. The mixing tank contains a mechanical mixer, which is set to mix each batch for 20 – 40 minutes via a countdown timer. The mixing time is limited to 40 minutes as the operators are concerned that the polymer chains will be sheared if mixed too long.

Once the batch is fully mixed, it is transferred to the dosing tank (by gravity). Dosing is achieved by two eductors, with the dosing rate controlled by diaphragm valves. The eductor power water also acts as carry water for the dosed polyacrylamide solution. A calibration tube is used to measure the flow rate through the dosing eductor.

The main details for the polyacrylamide makeup and dosing system are given in the table below.

Polyacrylamide (LT20) System

Component	Parameter (Units)	Design Criteria	Comments
Polyacrylamide System	Chemical product and Strength (%)	Ciba Magnafloc LT20, supplied as powder	
	Storage space (bags)	Several bags can be stored in chemicals room	
	Batching system	Manual powder wetting-up eductor Mixing tank with mechanical mixer Dosing tank	Manual feed into mixing tank via eductor. Manual transfer from mixing tank to dosing tank
	Mixing tank capacity (L)	1000	
	Dosing tank capacity (L)	Approx. 1000	
	Batching concentration (g/L)	1 g/L	1 kg into 1000 L
	Dosing mechanism	Eductors, controlled with diaphragm valves	
	Dosing eductors: No. of, Capacity (L/h)	2, Approx. 216 L/h total capacity	Capacity estimated by operators. Higher flows may be possible

No significant operational problems were reported for the polyacrylamide system. It is noted that the polyacrylamide mixing and dosing tanks are not bunded, and **a suitable bund should be provided**.

Photo of Polyacrylamide Dosing Eductors

5.1.4 PAC

The PAC dosing system was reportedly designed as a temporary system and has been made up of spare components from other defunct systems. A permanent system is to be designed. **The permanent PAC system should allow for dosing up to 60 mg/L to treat algal toxins (may need to be dosed to the flash mixing tank).**

The PAC powder, supplied in 20 kg bags, is manually unloaded into the dosing hopper. A dust extraction system is provided for bag unloading activities. **Any new system should consider ways to minimise manual lifting and to avoid operator contact with PAC dust.**

PAC in the dosing hopper is metered into the dilution tank by a screw feeder. The dilution tank is mixed by a mechanical mixer to keep the PAC in suspension. The PAC solution is dosed via an ejector and carried through a flexible hose to the dosing point.

The main details for the PAC dosing system are given in the table below.

PAC System

Component	Parameter (Units)	Design Criteria	Comments
PAC System	Chemical product and strength (%)	Powdered carbon C&S brand product No. MDW 3545 CB	Coal based product
	Storage space (PAC bags)	1 pallet of 20 kg bags stored in filter gallery area of WTP building	
	Hopper capacity (kg)	50 kg	
	Bag unloading arrangement	Bags manually lifted into hopper	Dust extraction system available
	Screw feeder type	W&T model EA32-055	Installed in 2000
	Screw feeder rate (kg/h)	Small auger output (current setup) measured by operators at 600 setting gave 1 kg/h	Based on W&T manual, feeder can supply 0.85 L/h – 1416 L/h (max requires use of largest auger)
	Dosing mechanism	Ejector	

The PAC dosing rate is adjusted by changing the screw feeder VSD setting (on controls located inside the cabinet in the filter gallery). Typical VSD settings range from 250 to 600. The setting of 600 reportedly corresponds to a dosing rate of around 1 kg/h. A larger auger for the screw feeder is reportedly available on site if higher doses are required, which may be able to supply up to around 1000 kg/h, however it is not certain whether this screw feeder would be suitable for PAC (the system was originally designed for fluoride) and the turndown with the larger auger would need to be considered if it was required to increase the maximum output of the system.

It is noted that the PAC dilution tank does not have a bund. **Appropriate bunding should be provided for this tank**.

Photo of PAC Dosing System

5.1.5 Chlorine

There are chlorine systems both at the raw water pump station and at the plant. The main details for the chlorine dosing system are given in the table below.

Chlorine Systems

Component	Parameter (Units)	Design Criteria	Comments
Chlorine System at Raw Water Pump Station	Chemical product	Chlorine gas	
	Chlorine room capacity (drums)	Currently 1 duty 920 kg drum, standby 70 kg cylinder	
	Chlorinators: No. of, Capacity (kg/h)	1 x 1 kg/h	W&T S10k unit, installed 2007-08
	Load cells capacity	No load cell	
	Service water pumps: No. of, Capacity (L/s)	No pumps Pressure provided by head of dam	
Chlorine System	Chemical product	Chlorine gas	

Component	Parameter (Units)	Design Criteria	Comments
at WTP	Chlorine room capacity (drums)	Currently 1 duty 920 kg drum, standby 70 kg cylinder	
	Chlorinators: No. of, Capacity (kg/h)	1 x Pre- chlorine: 4 kg/h 1 x Post-chlorine: 4 kg/h	Upgraded in 2006. W&T S10k chlorinators (model 210s)
	Load cells capacity	Weight dial gauge for 920 kg drum Scales for 70 kg cylinder	
	Service water pumps: No. of, Capacity (L/s)	No dedicated chlorine booster pumps. Pressure provided by service water system	

An auto changeover function is provided by the vacuum regulators on both the pump station and the WTP systems.

It is noted that heating and lagging has been provided for the WTP chlorine system, presumably to maximise the feed rate out of the drum/ cylinder in cold weather.

The continued dosing of post-chlorine is critical for disinfection of the final water. It is understood that the installation of an online chlorine residual meter is planned. **The online residual meter should be connected to the SCADA system, with an associated dial out alarm to reduce the potential response time if a chlorine dosing failure should occur when the plant is unattended.**

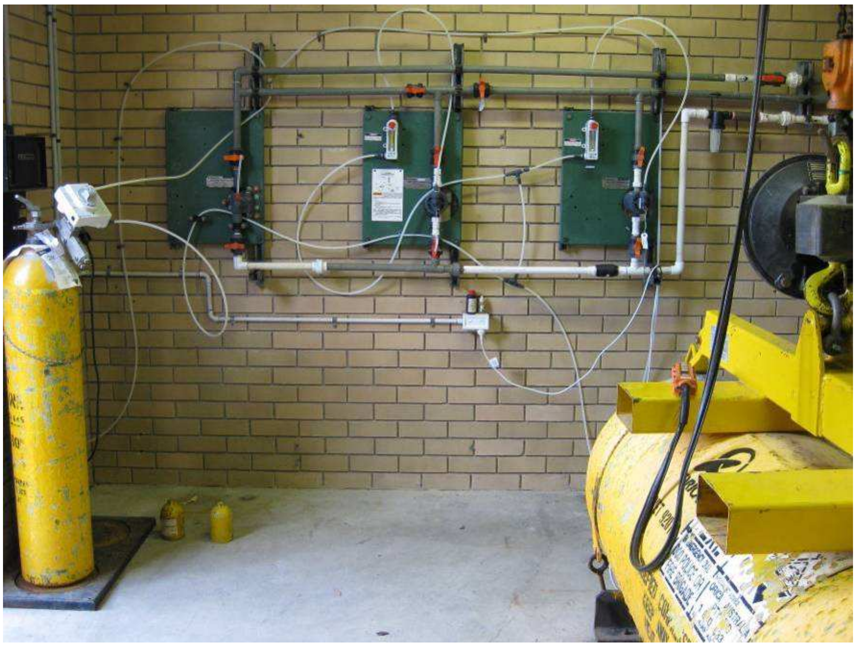


Photo of WTP Chlorine Systems

The design of the chlorine systems and the chlorine room is compared to some of the requirements of the Australian Standard for chlorine installations (AS/NZS 2927: 2001) in the table below.

Australian Standard Requirements for Chlorine Installations

Clause	Reqts Met? Raw PS	Reqts Met? WTP Systm	Comments
Cladding or lining of any indoor installation shall be incombustible. Floor shall be of concrete (Clause 3.5.1)	Y	Y	
At least one sign prominently displayed at eye level, visible when door is open. Where kept with other dangerous goods, chlorine storage area shall be clearly delineated and marked with signs (Clause 3.5.1)	Y	N	WTP: sign is partially obscured when door open
Pits, sumps and machinery wells enclosed or below the level of the chlorine installation shall have no unsealed openings into the chlorine storage or chlorinator areas or areas traversed by pipes carrying chlorine (Clause 3.5.1)	Y	Y	WTP: no pits etc. noted P/Stn: no pits etc. noted
Personnel doors shall open outwards and be fitted with devices to hold the door open (Clause 3.5.2)	Y	Y	Doors open outwards and latch supplied P/Stn: Drum room has double sliding door
A sign, indicating that the door is to be kept open whenever personnel are inside, shall be fitted outside the door and shall be visible when the door is open (Clause 3.5.2)	N	N	Appropriate signs provided, but not visible when doors are open
Natural ventilation required for areas where chlorine stored or < 2000 kg connected to withdrawal system. Natural ventilation requires at least 0.1 m^2 for each 2 m of external wall, near to floor level in opposite walls to create a cross-draught (Clause 3.5.3)	N	N	Pump Station: May need more natural ventilation. No mechanical exhaust fan WTP: Ventilation area limited, however mechanical exhaust fan mounted (manually operated)
Mechanical ventilation required for areas where > 2000 kg chlorine connected to withdrawal system (Clause 3.5.3)	n/a	n/a	Currently both sites have < 2000 kg connected to withdrawal system
Leak detectors shall be installed where chlorine is stored in tanks or where liquid chlorine is withdrawn. (Clause 4.8.1)	Y	Y	Leak detectors installed
Leak detectors shall be tested each week (Clause 4.8.1)	N	N	Detectors not regularly triggered for testing

The Raw Water Pump Station chlorine system and the Biloela town booster stations were audited by Orica in 2006. It is understood that the WTP chlorine system was not audited. Orica's review of the Raw Water Pump Station chlorine system included recommendations for the provision of:

- New safety signs and MSDS in the chlorine room
- A mechanical cylinder lifting device to reduce manual handling;

- A first aid kit onsite or in the truck;
- Weekly checking of chlorine leak detectors and alarms;
- Use of a maintenance and changeover log book; and
- Further safety training for operators.

As seen above, the installation does not meet some of the requirements of the Standards and further improvements in operation and training could be made. **Ideally, the chlorine installation should be improved to meet the Australian Standards and Orica's recommendations, as outlined above.**

5.1.6 Lime

Lime is supplied in powdered form in 20 kg bags. The bags are unloaded manually into the lime chemical feed hopper in the chemicals room. The screw feeder under the feed hopper meters the powder into the solution mixing tank, where the lime mixes with water and is carried to the dosing point as a solution.

It is noted that the lime feed hopper and the standby feed hopper (common to the lime and alum systems) are both fitted with a vibrator to reduce lime hang-ups. The vibrator works periodically, as set on an internal timer which can be adjusted if necessary (timer not usually changed).

One lime batch tank serves both the pre and post-lime dosing points via separate ejectors. It is noted, however, that pre-lime has never yet been required to be dosed. If both pre- and post-lime are used in future, the doses to each dosing point would be controlled by varying the output of each ejector.

The main details for the lime makeup and dosing system are given in the table below.

Lime System

Component	Parameter (Units)	Design Criteria	Comments
Lime System	Chemical product and strength (%)	Hydrated lime – Various suppliers, various purity	
	Number of makeup and dosing systems	1 plus common standby with alum system (appropriate auger size to be inserted in screw feeder)	
	Makeup and dosing system components	Chemical feed hopper Screw feeder Solution mixing tank Ejector	Note hopper and common standby hopper are fitted with a vibrator
	Dose adjustment method	Adjustable output (i.e. speed) of screw feeder	
	Storage capacity	Approx. 8 tonnes of lime in 20 kg bags	Based on original operating manual
	Hopper capacity (m3)	2.55 m3 (approx. 2 tonnes)	Note that standby hopper, feeder and solution tank can be used for lime or powdered alum
	Bag unloading arrangement	Bags manually lifted into hopper	Dust extraction system available

Component	Parameter (Units)	Design Criteria	Comments
	Feed system arrangement	Screw feeder meters the required dose rate into solution tank. Constant dilution water flow.	
	Screw feeder capacity (kg/h)	30 kg/h	Based on operator advice
	Solution tank capacity (L)	115 L	
	Dosing mechanism	1 pre lime, 1 post lime ejector used to transport solution to dosing points	Pre lime system never yet used
	Service water system	Service water pumps from clear water tank	

The lime dose is generally adjusted by changing the adjustment knob of the screw feeder and the lime ejector is generally kept at a constant rate (except if pre-lime were also to be dosed as noted above). The operators noted that a given feeder setting may give variable speed output over time due to clutch wear and feeder cleanliness, therefore the drop rate is periodically checked by performing drop tests.

The screw feeders were reportedly designed to have double the expected Stage 1 capacity, in order to cater for the future possible upgrade to Stage 2 plant flowrates. It is noted that spares are no longer sold for the Wallace and Tiernan feeders. Spare parts have been machined or adapted in the past, however **an upgrade of the screw feeder technology could be considered in future so that spare parts can be more easily obtained.**

Photo of Lime Screw Feeder and Dilution Tank

There are dual flexible dosing lines to the post-lime dosing point, allowing for duty/ standby lines so that one dosing line can be cleaned out while operation continues through the other line.

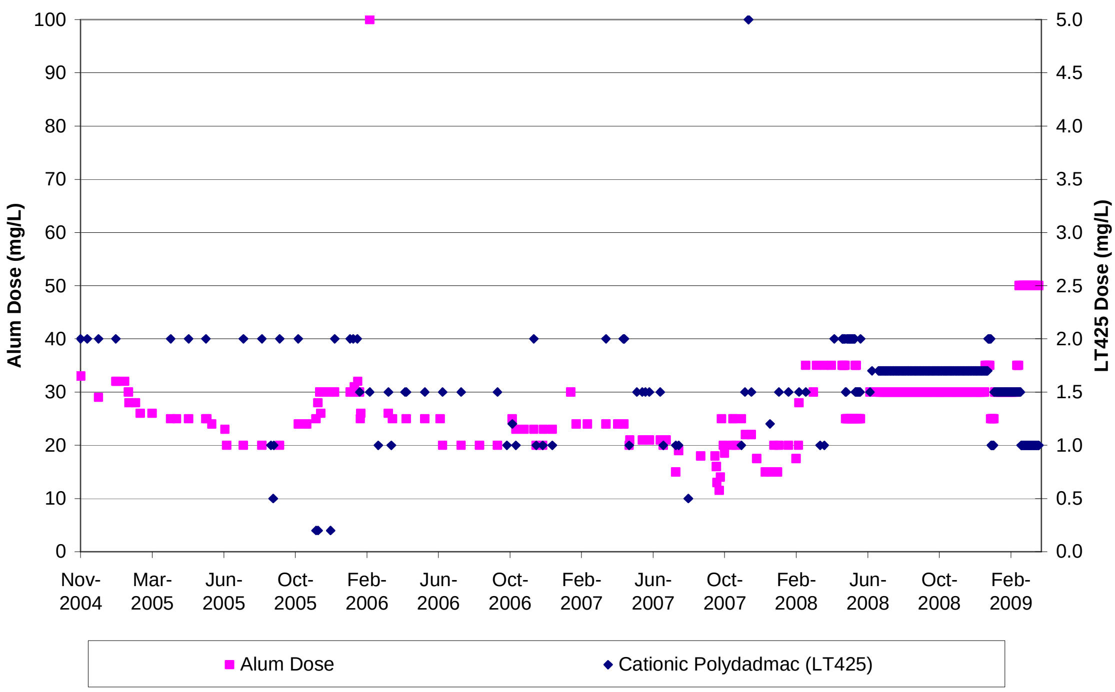
A timer system allows the chemical stirrers and ejector to continue to run after the plant has shut down and the screw feeder has stopped, effectively flushing the slurry tank, ejector and delivery pipework. Regardless of this flushing, it is reported that there is a gradual buildup of lime on the injectors. As part of general maintenance, around twice a week the operators remove the duty ejector, flush the system with water (using a bridging piece where the ejector sites) and then replace the duty ejector with a clean ejector. The used ejector is soaked for around 20 minutes in a hydrochloric acid solution to remove the lime before reuse. **More effective flushing and/or acid washing systems could be investigated if lime scale formation is seen to be a problem.**

It is noted that the lime dilution tank is surrounded by a low (approx. 50 mm high) bund, which is in common with the alum dilution tank. **The bunding requirements for the dosing arrangement should be reviewed and the bund enlarged if required.**

5.2 Chemical Doses Used

5.2.1 Alum and Cationic Polydadmac (LT425)

The doses of coagulants used, as reported in operational data for 2004 - 2009, are shown on the graph below.



Graph of Alum and Cationic Polydadmac (LT425) Doses

The data shows that the alum dose has mainly varied between 11.5 and 50 mg/L since 2004. The operators, however, reported that maximum doses of 100 to 125 mg/L could be expected and that up to 140 mg/L had been used many years ago during very poor raw water quality associated with freshes in the dam. It was noted that alum doses of 70 – 100 mg/L were reported during the February 2003 dam fresh event, and a dose of 70 mg/L was reported during an algal bloom event in September 2003. A dose of 100 mg/L was also seen in March 2006.

The cationic polydadmac dose shown in the graph varies between zero and 2 mg/L (with one reported point at 5 mg/L) and is typically around 1 to 2 mg/L. Doses up to 3 mg/L were noted in records for the algal bloom event in September 2003. It understood from the operators that the alum and polydadmac doses are re-optimised by jar testing when there is a change in raw water quality turbidity or colour.

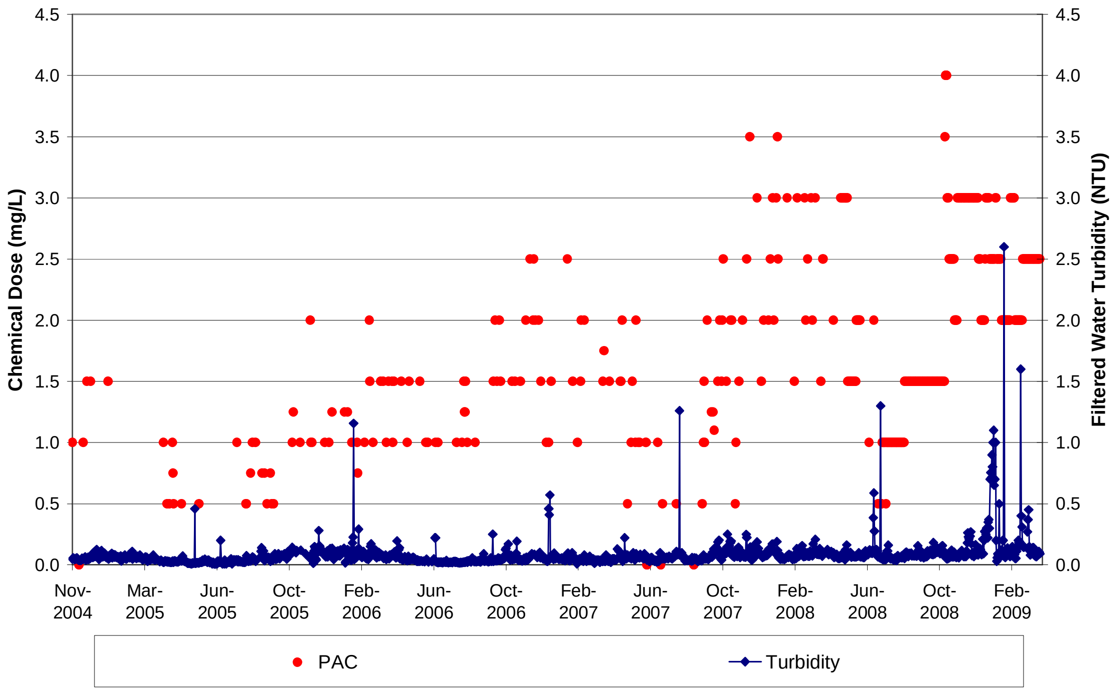
5.2.2 Polyacrylamide (LT20)

Polyacrylamide has only been dosed occasionally when high raw water turbidities have been experienced, and not at all in the data available for the period 2004 to 2006.

According to the calculation sheet used for polyacrylamide drop tests, it is expected that doses of up to 0.3mg/L may have been applied.

5.2.3 PAC

The doses of PAC used, as reported in operational data, are shown on the graph below. Filtered water turbidity is also shown in the graph.



Graph of PAC Doses Showing Filtered Water Turbidity

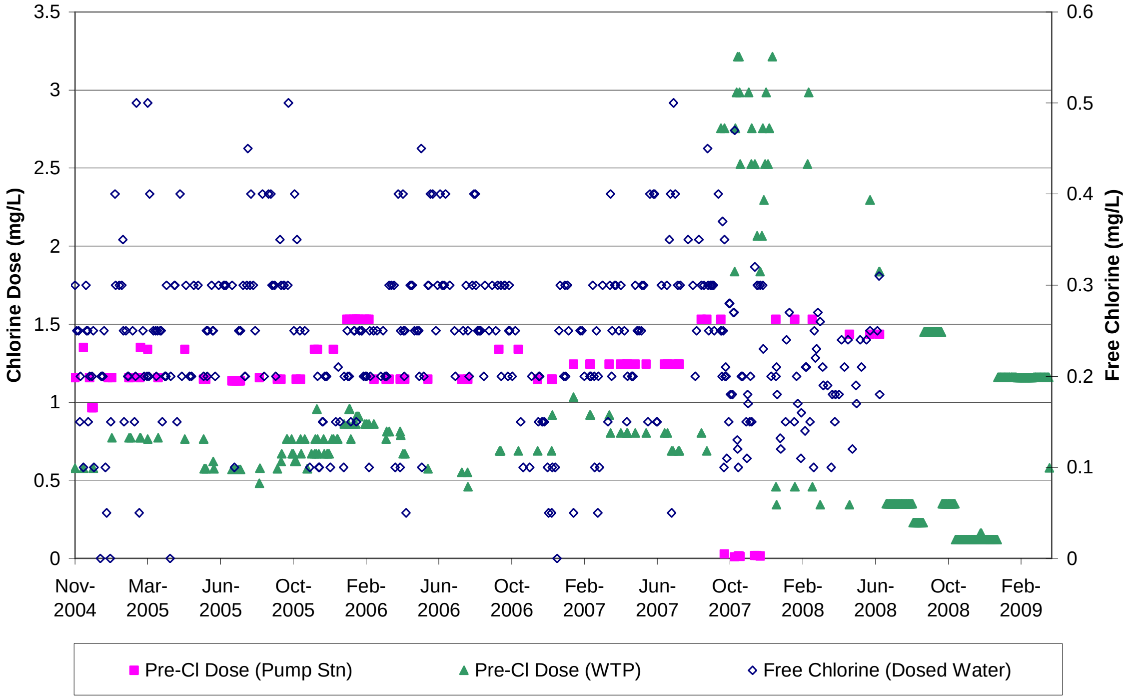
As seen in the graph, doses of zero to 4 mg/L of PAC were used. For the algal bloom event in September 2003, a PAC dose of 2.5 mg/L was recorded (into a down-rated plant flow of 100 L/s). It is noted that the elevated PAC doses have been dosed into the flash mixing tank rather than the filter inlet channel.

It is noted that, based on industry experience, a much higher dose of up to 60 mg/L PAC may be required to be dosed into the raw water in order to effectively treat algal toxins. Because of the potential for algal toxins to be present in the raw water, **an upgraded system should potentially be designed to dose PAC to 60 mg/L.**

A slight correlation between PAC dosing and filtered water turbidity is seen in the graph above, which may be may be coincidental or may be related to PAC particles affecting filtration or passing through the filter media. It should be checked whether PAC particles can be detected in the filtered water.

5.2.4 Pre-Coagulation Chlorine

The doses of pre-chlorine reported in operational data for 2004 -2009, both at the raw water pump station and the WTP flash mixing basin, are shown on the graph below. The chlorine residual achieved in the dosed water is also shown in the graph.



Graph of Pre-Chlorine Doses and Residuals

The data shown in the graph indicates the following pre-chlorine dose ranges:

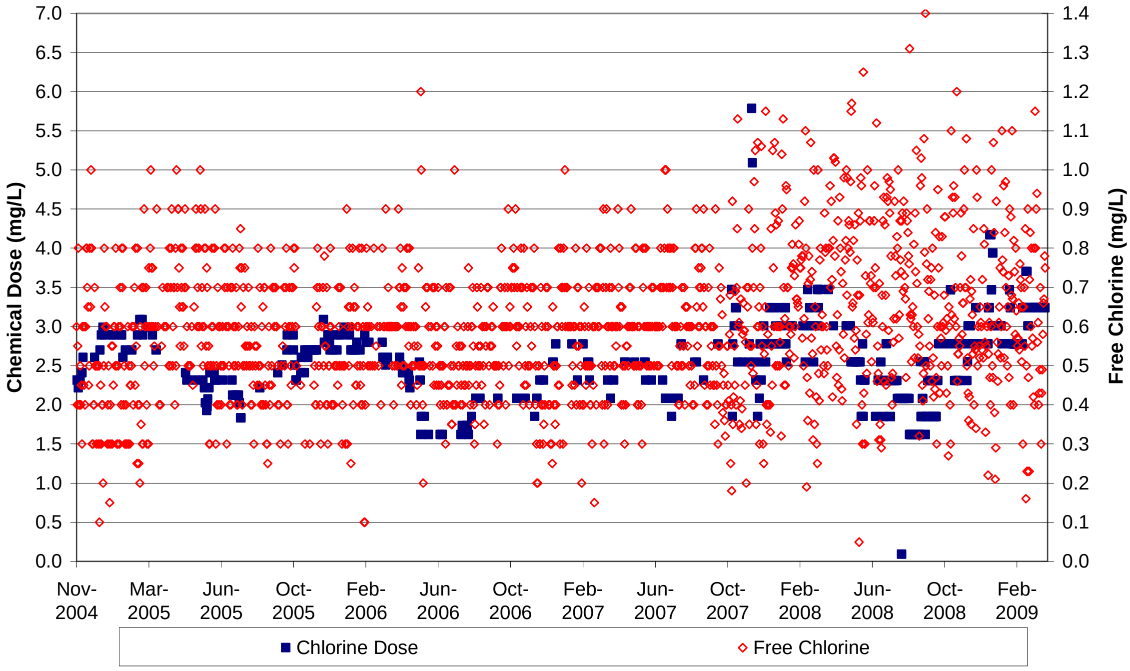
- Pump station dosing: 0.01 – 1.55 mg/L, typical 1.15 mg/L;
- WTP pre-chlorine dosing: 0.34 – 3.2 mg/L, typical 1.1 mg/L.

The chlorine residual levels reported have ranged from 0 to 0.5 mg/L and are typically around 0.25 mg/L. Based on the typical values above, it appears that the typical chlorine demand of the raw water is around 1 mg/L.

It was noted that pre-chlorine dosing was stopped during the algal bloom event in September 2003 while PAC was being dosed to the flash mixing chamber.

5.2.5 Post-Filtration Chlorine

The doses of post-chlorine reported in operational data for 2004 -2009 are shown on the graph below, along with the treated water chlorine residual measured.



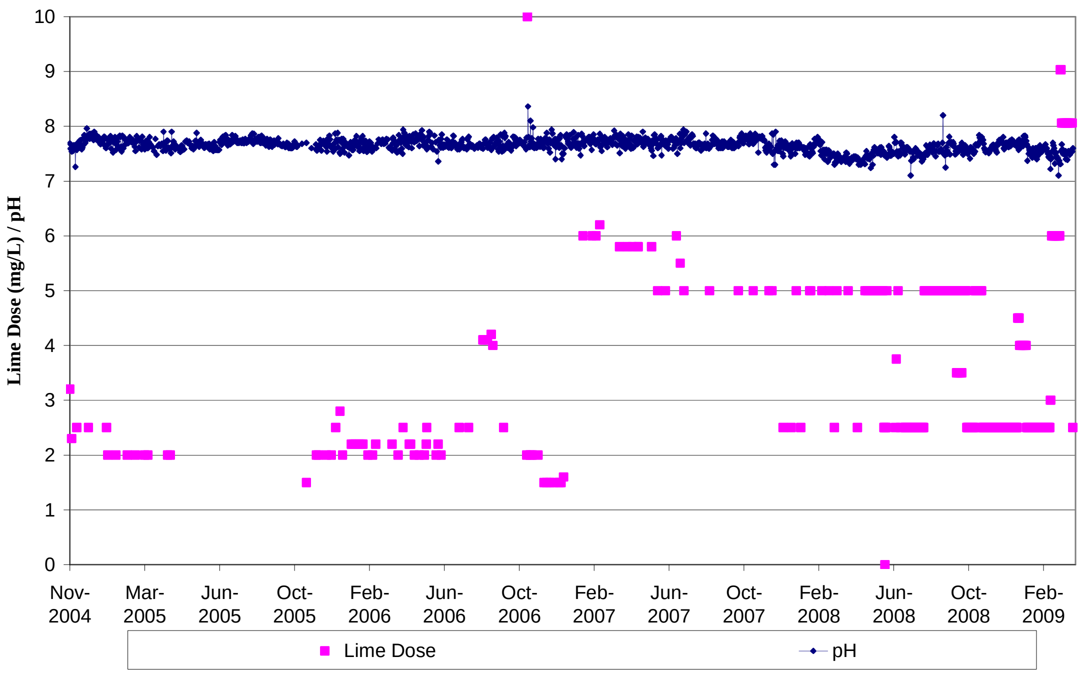
Graph of Post-Chlorine Doses and Residuals

The data in the graph shows the post-chlorine dose has ranged from 1.6 – 5.8 mg/L, with a typical dose of around 2.75 mg/L in the summer periods shown and a slightly lower level of around 2.25 mg/L in the winter periods. It is noted that the winter period of slightly lower chlorine demand also coincided with a slightly lower/ more stable filtered water turbidity, as shown on the graph for PAC above.

During the February 2003 dam fresh event, post-chlorine doses of up to 3.3 mg/L were used.

5.2.6 Post-Filtration Lime

The lime doses reported in operational data are shown on the graph below. The final water pH is also shown in the graph.



Graph of Lime Doses and Treated Water pH

The data indicates that the post-lime dose has generally varied between 1.5 and 10 mg/L, showing that only a low dose is normally required to raise the pH to the target band.

For the February 2003 dam fresh event, lime doses of 20 - 30 mg/L were recorded. These elevated lime levels were required to compensate for the pH reduction caused by the significantly higher alum doses applied.

5.3 Chemical Dose and System Capacity Summary

5.3.1 Calculated System Dosing Capacities

The current capacities of the chemical systems are shown in the table, along with options for increasing the capacity of each system. Chemical capacities in terms of 'mg/L' are shown in the following section.

Chemical Dosing Capacities

Chemical	System Parameters		Storage Usage Time		Options to Increase Capacity (if Required)
	Dosing capacty (kg/h)	Dosing Soln (g/L)	Vessel capacty (kg)	Time to Empty* (h)	
Alum	50	n/a	2000	40	Larger screw feeder/ New liquid alum system
Polydadmac (LT425)	26.5 L/h	109	1000 L	38	Stronger solution, larger pump
Polyacrylamide (LT20)	216 L/h	1	1000 L	4.6	Stronger solution, larger eductors
PAC	1	n/a	50	50	Larger screw feeder or new larger system
Raw Water P/Stn Pre-chlorine	1.83	n/a	920	500	Upgrade dosing system

Chemical	System Parameters		Storage Usage Time		Options to Increase Capacity (if Required)
	Dosing capacty (kg/h)	Dosing Soln (g/L)	Vessel capacity (kg)	Time to Empty* (h)	
WTP Pre-chlorine	4	n/a	920	230	Upgrade dosing system
Post-chlorine	4	n/a	920	230	Upgrade dosing system
Post-lime	30	n/a	2,000	67	Larger screw feeder

* At system dosing capacity (i.e. maximum rate possible for system)

5.3.2 *Chemical Dose Requirements Compared with System Capacities*

The actual chemical doses, as discussed above, are summarised in the table below and compared with the estimated system capacity at various plant flow rates (calculated from available data).

Actual Doses Used (Estimated from Available Data)

Chemical	Actual Doses Used (mg/L)		Dosing capacty (kg/h)	Capacty at 120 L/s Plant Flow (mg/L)	Capacity at 160 L/s Plant Flow (mg/L)	Capacity at 200 L/s Plant Flow (mg/L)	Capacity at 240 L/s Plant Flow (mg/L)
	Range	Typical					
Alum	20 - 125	25	50	116	87	69	58
Cationic polydadmac (LT425)	0 – 5	1.5	26.5 L/h	6.7	5	4	3.5
Polyacrylamide (LT20)	0 – 0.3	Not typically used	216 L/h	0.5	0.38	0.3	0.25
PAC	0 – 4*	2	1	2.3	1.7	1.4	1.15
Raw Water P/S Pre-chlorine	0.01 – 1.55	1.2	1	2.3	1.7	1.4	1.15
WTP Pre-chlorine	0.3 – 3.2	1.1	4	9.3	6.9	5.6	4.6
Post-chlorine	1.6 – 5.8	2.5	4	9.3	6.9	5.6	4.6
Post-lime	0 – 30	2 - 5	30	69	52	42	35

*Higher doses up to 60 mg/L expected to be required if algal toxins present

From the above comparison of actual doses with the capacity of each chemical system, the following comments are made.

For 120 L/s Flow Rate:

- The **alum screw feeder** has much larger capacity than the typical dosing range, however **would not meet the expected worst case dose at current plant flow**;
- The **capacity of the PAC system is less than the maximum PAC dose recorded.**
- All other chemical systems appear to be adequate for the doses expected.

For 160 L/s Flow Rate:

- As above for the **alum and PAC systems**;

- The capacities of the cationic polydadmac, polyacrylamide, pump station pre-chlorine and post-chlorine systems are just above the maximum doses recorded;
- All other chemical systems appear to be adequate for the doses expected.

For 200 - 240 L/s Flow Rate:

- As above for the **alum and PAC systems**;
- The **capacities of the cationic polydadmac, polyacrylamide, pump station pre-chlorine and post-chlorine systems are less than the maximum doses recorded**;
- The WTP pre-chlorine and lime systems appear to be adequate for the doses expected.

5.3.3 Addition of Fluoride Dosing Facilities

Recent decisions by the State Government require Council to implement fluoride dosing at Biloela by 2012. Fluoride dosing could be carried out either at the WTP or at the town reservoir.

Dosing at the WTP would be practical in terms of operator access for monitoring and in terms of site security. It is noted that the power station requires un-fluoridated water, so fluoride may most practically be dosed into a second clear water tank (if provided), with the power station drawing from the first clear water tank. However, if the town bores continued to be used, this water would dilute that fluoridated WTP treated water at the town reservoir.

Alternatively, dosing could be carried out at the town reservoir, although this site is not as convenient to the WTP operators and public access to the area would need to be controlled. It is noted that a fluoride dosing system was originally installed at this location, but this system has not been used for a number of years and has been partially dismantled. The advantage of dosing fluoride at the town reservoirs is that the full flow rate of WTP treated water and bore water could be dosed to the required fluoride target level. Dosing would however be complicated if un-fluoridated water was drawn off the trunk main before the reservoir and then returned to the main distribution system, as Council reportedly have the capability to do.

The location and design of a fluoride dosing facility should be further investigated in conjunction with the WTP upgrade requirements.

6. WTP Operational Issues

6.1 Plant Control and Automation Issues

6.1.1 General Observations

It was observed that the plant control system generally has a low level of automation, with no flow pacing or automatic dose adjustment and fully manual backwashing. The existing system is relatively simple, but requires greater operator input than more automated systems.

If automation improvements are made, it would be beneficial that the capacity to run systems in manual mode is still retained to give operational flexibility.

6.1.2 SCADA System

A RADTEL brand SCADA system common to all Council WTPs has been implemented over recent years. The Biloela SCADA pages show the water supply and distribution system and some parameters for the WTP.

It was noted that the SCADA system is subject to ongoing development. The SCADA system will potentially be useful in terms of adjusting setpoints, logging online data, remote plant observation/ operation and callout alarms.

It was noted that operational staff are still learning about the SCADA system. The trending of on-line information will be a particularly good tool for WTP operators to use as input for operational decisions. **Ongoing training of the operators and other staff on the various SCADA capabilities would be beneficial.**

6.1.3 Control of Plant Startup and Shutdown

The plant is automatically started and stopped based on water demand via the following control loops:

- Raw Water Pump Control: The WTP clear water tank level signal is used to generate a starts/stop signal to the raw water pumps via radio telemetry;
- Chemical Dosing Systems Control: The chemical dosing systems start when the trigger level raw water flow rate is reached and stop when the raw water flow rate drops below this trigger level. The trigger level (currently 40 - 50 L/s) can be adjusted on controls inside the mimic panel.
- Chemical Dosing Shutdown Timer: When the chlorine, lime, alum and PAC systems receive the shutdown trigger signal, the chemical feeds stop, however the transfer water continues for a time set on a timer inside the mimic panel (currently 3 – 4 min), flushing these systems and the dosing lines.

It is noted that all valves on the flow path through the WTP remain open when the plant is off. The filter inlet and outlet valves are manually operated only. This arrangement causes the level in the clarifier and filters to drain down each time the plant shuts down.

6.1.4 Control of Plant Flow Rate

The raw water pumping rate can be controlled by adjusting the raw water pump VSD (at the raw water pump control panel) and the inlet valve. However, for a given raw water pumping rate set point the flow rate can vary according to dam and pump conditions.

In the original plant design, a flow controller maintained a set level in the flash mixing tank by automatically adjusting the flow via the inlet control valve. According to the operators, this automatic level controller system never actually worked effectively, therefore in order to maintain a stable level in the flash mixing tank the inlet flow rate is currently fine tuned by *manual* adjustment of the inlet control valve.

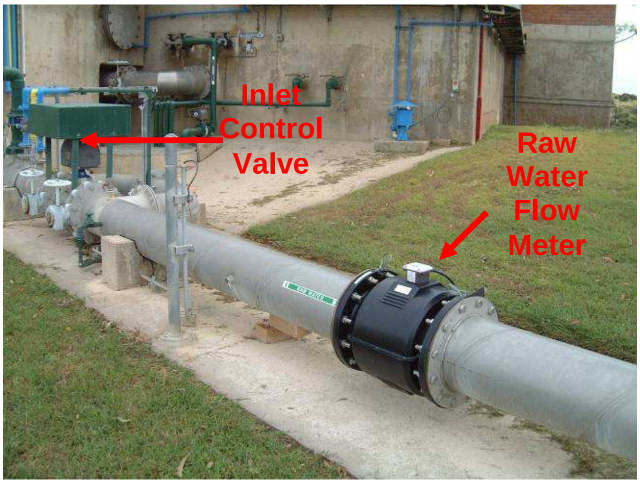


Photo of Raw Water Flow Meter and Inlet Control Valve

It is noted that none of the chemical feed systems are automatically flow paced, therefore if the plant flow rate is adjusted, the chemical systems will need to be manually adjusted to match the new flow rate.

6.1.5 Control of Flow into Town Mixing Tank and Reservoirs

Flow into the town mixing tank and ground level reservoirs (GLRs) is controlled based on the water level in the online reservoir(s):

- WTP Treated Water Flow Control: GLR water level setpoints trigger the start and stop of flow, controlled by an automatic butterfly valve on the gravity main. Note that the flow rate (gravity flow) depends on hydraulic conditions and can be throttled adjusted using a manually controlled valve on the main.
- Bore Water Flow Control: GLR water level setpoints trigger the start and stop of the groups of bore pumps. Note that bore pumping rates depend on the level in the aquifer, and individual bore flow rates can be throttled using manually controlled valves.

6.1.6 Process Impacts of Plant Startup

It is noted that clarifier and chemical dosing systems generally run more smoothly with continuous rather than start/stop operation. Flow changes due to plant start-up may disturb settling in clarifiers and/or lead to periods of over/ under dosing due to delays in chemical dosing system starts and stops. Flow changes can also put stress of filters, causing particles to shear through the media.

The operators feel that flow changes at the Biloela WTP due to starting and stopping are not a great problem in terms of process stability and water quality achieved. If the impact of start-ups is seen to be limiting further optimisation of the plant process in future, the number of start-ups per day could be potentially reduced by running the plant at a lower rate during periods of low demand. This could be achieved using the existing VSDs on the raw water pumps, however the automation of the chemical systems to make them flow paced would be required to reduce the manual changes required.

6.1.7 Filter Backwashing Automation

As discussed in the previous section on Filter Backwashing, the backwashing system could be improved in terms **of upgraded control of filter valves and provision for an automated backwash sequence** to make operation easier and reduce the chance of operator error.

6.1.8 Online Monitoring

The existing and planned online monitoring facilities for the WTP are summarised in the table below.

Online Monitoring Meters Summary

Component	Parameter (Units)	Design Criteria	Comments
Turbidity	Type	HACH 1720C	To be logged to SCADA
	Sampling location	Takeoff from filtered water pipe before filtered water well	Local display in filter viewing area
pH	Type	ABB	To be logged to SCADA
	Sampling location	Takeoff from filtered water pipe before clear water tank	Local display only
Chlorine Residual	Type	Not reported	To be logged to SCADA
	Sampling location	Final water	
Raw Water Flow	Type	ABB magflow meter	Logged to SCADA
	Sampling location	Raw water rising main prior to supernatant return point and flash mixer	Local display in filter viewing area
Supernatant Return Flow	Type	Wheel type meter	Local display only
	Sampling location	Supernatant return pipe	To be refurbished
Treated Water Flow	Type	ABB magflow meter	Logged to SCADA
	Sampling location	Main, after clear water tank	To be replaced/ refurbished

As seen in the table, flow rates, filtered water turbidity, pH and chlorine residual are monitored by on-line instruments. Most of these signals are or will be connected to the SCADA system and thus are should be able to be trended.

It is understood that the provision of an on-line instrument for measuring treated water chlorine residual is budgeted and that this instrument would also be connected to the SCADA system.

It is noted that the latest USA regulations (Enhanced Surface Water Treatment Rule) require the monitoring of turbidity on individual filters. To bring the plant in line with modern industry practise and give the operators more tools for monitoring the condition of the process, **the provision of online turbidimeters for each filter should be considered**. Sample points could potentially be provided at the drain valve tappings on the outlet of each filter.

6.1.9 SCADA Callout Alarms

There are a number of alarms which register on the mimic panel. Some of these alarms are also connected to the SCADA system. It is understood that alarms are gradually being connected to the SCADA system and the SCADA callout alarms list.

The alarms for the plant process operation which are considered critical enough to be on the SCADA callout list (when available) include:

- Filtered water turbidity high;
- Power outage;
- Treated water chlorine residual low/ high;
- Treated water pH low/ high.

6.1.10 Power Failure Protection

It is understood that there is no backup power supply for the plant in case of power failure. Considering the reasonable storage volume and the alternative bore water source for the town, short term power outages may pose a minimal risk, however **Council should consider the risks of interruption of water supply posed by longer term power failures and develop contingency plans to address such an event.**

It is noted that if the power fails, the plant stops but the process path for water remains open. There is some risk that if there is a power failure at the plant but not at the raw water pump station, raw water will continue to be pumped through the plant without chemical dosing. Ideally, **the controls of the raw water pumps should be interlocked so that the pumps will stop if the signal from the plant is lost.** Alternatively, the filter inlet valves could be set to 'fail closed' so that water could not pass through the filters when the plant was off.

6.1.11 Fire Protection

It is noted that fire fighting pumps and hose reels are provided for the event of a fire on site. The two fire fighting pumps work on a dual stage (pressure switch activation) system to boost pressure to the fire hose reels.

6.2 Safety and Environmental Issues

The following safety and environmental issues were noted during CWT's WTP inspection, however this is not intended to be a full and exhaustive OH&S audit.

6.2.1 Chemical Bunding

It is noted that the storage and/or solution tanks for the cationic polydadmac, PAC and polyacrylamide systems are not bunded. The alum and lime solution tanks share a very low common bund.

All chemical storage and batching/ dosing tanks and all chemical dosing pumps should be bunded to prevent the spillage of chemicals. Bunds should be built to conform with standards on design and materials.

6.2.2 Manual Handling and Chemical Contact

Manual handling issues identified on the plant include:

- Manual handling of 20 kg lime bags for filling lime dosing hopper;
- Manual handling of 20 kg PAC bags for filling PAC hopper.

Because chemicals are unloaded manually, there is some potential for the operators to come in contact with the chemicals such as lime and alum dust, PAC and polymer powders. It is noted that a dust extractor is provided for the lime, alum and PAC hopper systems.

Any upgrades of the chemical systems should consider ways to further minimise manual handling requirements and the risk of operator contact with chemicals.

6.2.3 Access to Clarifiers

It is noted that operator access to the clarifier supernatant collection weirs and the surface of the clarifiers for cleaning and maintenance is currently achieved with temporary access boards. These are not safe and would not meet safety regulations. **Suitable walkways with railings should be provided to allow safe access to various parts of the clarifier surfaces.** It is understood that arrangements are currently (May 2009) being made to provide these walkways.

Photo of Temporary Clarifier Access Board

6.2.4 Access to Filtered Water Well

It was noted that there is no walkway or permanent ladder to give access to the filtered water well (where post-chlorine is dosed). Operators are currently required to either lift a ladder into place or to climb onto the filtered water pipe to access the weir box. **A suitable and safe access facility should be provided for accessing this pit.**

Photo of Access to Filtered Water Well

6.2.5 Laboratory and Office Facilities

The laboratory and small office rooms provided at the WTP appear to be adequate and are air conditioned for operator comfort.

It was noted that there is no separate lunch room for the operators. From a safety viewpoint, food should not be stored or consumed in the laboratory, and there is insufficient space and no sink facilities in the office room for lunch facilities. **Ideally, a separate lunch room, with table and sink facilities, should be provided in an area where there is no risk of chemical contamination.** It is understood that works are currently (May 2009) underway to draw up plans for an enclosed lunch room within the filter viewing area.

6.3 Maintenance Issues

6.3.1 Algal Growth on Clarifiers

It was reported that algal growth can be a problem in the clarifier supernatant collection troughs and on the settling tubes. The operators prevent the algae accumulating by regularly cleaning the clarifier components. **A shade cover could be considered in order to limit growth of the algae.** This would also assist in preventing the deterioration of the tube settlers by UV light.

6.3.2 Site Subsidence

It is noted that the plant structures may have settled over time, for example the cement paths around clarifier have settled a few inches and pipework has had to be altered to account for the lower cement area. The steps to the main entrance have also slipped down requiring new tiles to be added. **The stability of the site should be checked and considered in the design and placement of any new structural components part of an upgrade of the WTP.**

7. WTP and System Upgrade Requirements

7.1 Identified Potentials for WTP Improvement

The issues identified for potential improvement of the WTP are tabulated below, with time frame and priority level noted. Where upgrades in the capacity of WTP sub-processes are required to achieve adequate performance at the *current* WTP design flow rate they are noted below. Options to increase the capacity of the overall WTP and associated systems to meet *future* demand scenarios are addressed in Section 8.2.

7.1.1 Treatment Process Improvements

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Treatment of taste and odours and algal toxins	Improve PAC dosing capacity. Ozone BAC is an alternative an option for ongoing, high risks from algal organic contaminants	4.7	Medium	High
	Compare range of different PAC products in jar testing to find out whether there is a product more effective for compounds of interest and determine adsorption contact time requirements	4.7	Medium	High
Treatment of dirty raw water	Investigate optimisation of chemical dosing to achieve settling at normal plant rates during very dirty water quality periods	4.6.1	Short	High
Treatment of poor quality lagoon supernatant	Investigate treatment of sludge lagoon supernatant to remove problem compounds so supernatant can continue to be recycled	4.12.5	Medium	Medium
Treatment of bore water to reduce corrosivity	If required, jar testing could be undertaken to look at the effect of aeration on the water quality and on the likely corrosivity of the bore water	3.5.6	Medium	Low

7.1.2 Water Quality Monitoring

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Raw dam water quality	Monitor algae levels closely (in conjunction with Sunwater's algae monitoring program if appropriate) and analyse for toxins if significant levels of potentially toxic algae are present	3.2.9	Short	High
Raw dam water quality	Investigate cause of the taste and odour problems. Analyse raw water samples known to contain tastes and odours to identify the compounds responsible	3.2.10	Short	High
Chlorine residuals	Continue to monitor chlorine residuals within the system and revise chlorination residual targets at the WTP and after blending with the bore water as appropriate	3.2.7, 3.4.2	Short	Medium

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
WTP treated water quality	Monitor THM levels in WTP treated water	4.4.6	Short	Medium
General	Request the laboratory to report turbidity more accurately, to a resolution of at least 0.1 NTU	3.2.3	Short	Low
General	Continue to check plant water quality results against laboratory results, and investigate the reason any significant discrepancies	3.2.3	Medium	Low
Raw dam water quality	Raw water total and soluble manganese levels should continue to be measured periodically at the WTP, especially if high levels are detected	3.2.5	Short	Low
Raw dam water quality	The raw and treated water could be analysed for *Cryptosporidium* and *Giardia* on occasion to check background levels of these pathogens	3.2.11	Medium	Low
Raw bore water quality	Because of the elevated level noted, nitrate levels should continue to be monitored periodically. If more elevated nitrate levels are found, the water should also be measured for nitrite	3.3.5	Short	Low

7.1.3 Online Instruments

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Post-chlorine	Connect online chlorine residual meter to the SCADA system, with an associated dial out alarm	5.1.5, 6.1.8	Short	High
Filters	Consider provision of online turbidimeters for each filter to bring plant into line with latest industry approach	6.1.8	Long	Medium
Filters	The retrofitting of headloss meters could be considered	4.8.2	Long	Medium

7.1.4 Raw Water Pump Station and Plant Inlet

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Raw water Pumps	Investigate and rectify VSD problem to allow the VSD to be run at 100% without faulting	4.2.1	Short	Medium

7.1.5 Clarifiers

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Clarifiers	Suitable walkways with railings should be provided to allow safe access to various parts of the clarifier surfaces	6.2.3	Short	High
Clarifiers	A shade cover could be considered in order to limit growth of the algae and deterioration of the tube settlers due to UV light	6.3.1	Long	Medium
Clarifiers	Replacement of the settling tubes (as required) will need to be considered in future works programs	4.6.1	Long	Low
Clarifier desludge	The addition of automatically controlled valves on the manual desludge lines (set to desludge at regular intervals perhaps each time the plant is off-line) could be considered	4.6.2	Medium	Low
Clarifier desludge	For any future upgrades, the design of the new clarifiers should avoid the problem of extended periods of sludge accumulation	4.6.2	Long	Low
Clarifier desludge	For any future upgrades, the design of new clarifiers should ensure suitable draining facilities	4.6.2	Long	Low

7.1.6 Filters

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Filters	Analyse filter sand media for effective size and uniformity coefficient	4.8.1	Short	Medium
Filters	For current system, best practise in terms of flow control through the filters is to wash the filters when the plant is off-line (the current normal practice).	4.8.1	Short	Medium
Filters	Automation of the backwash cycle would be preferred, requiring upgraded control of filter valves and programming for an automated backwash sequence	4.8.2, 6.1.6	Long	Medium
Filtered water well	A suitable and safe access facility should be provided for accessing the filtered water well	6.2.4	Medium	Medium
Filters	The condition of the underdrain systems should be examined when practical	4.8.1	Medium	Low

7.1.7 Clear Water Tank and Town Reservoirs

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Clear water tank	Provide second clear water tank to allow current tank to be taken offline for maintenance	4.10	Short	High
Clear water tank	Carry out maintenance on existing clear water tank	4.10	Medium	Medium
Trunk Mains	Provide booster pump on trunk main to increase flows to meet peak demands	4.10	Medium	Medium
Town Reservoirs	Review and address hydraulic limitations in ground level reservoirs inlet mixing tank	4.11	Medium	Medium

7.1.8 Wastewater System

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Supernatant return system	Refurbish supernatant flow meter if possible to record accurately, or replace with a more suitable type of meter	4.12.5	Medium	Medium
Supernatant return system	Investigate performance of existing supernatant pumps to confirm the actual output of the pumps	4.12.5	Medium	Medium

7.1.9 Chemical Dosing

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
PAC	Upgrade capacity of PAC system by: • Investigating use of the larger auger; or • Provision of a new system of larger capacity Note that significantly higher capacity required for algal toxin removal	5.1.4, 5.2.3, 5.3.2	Medium	High
Alum	Upgrade alum system to meet expected worst case dose at current/ future plant flow rates	5.3.1	Long	High
Alum	Upgrade screw feeder technology so that spare parts can be more easily obtained OR Replace granulated alum system with a liquid alum storage and dosing system	5.1.1	Medium	High
Cationic polydadmac	Keep standby pump available for installation in case of failure of the dosing pump	5.1.2	Short	High
Polyacry-lamide	Separate pipework should be provided for the polyacrylamide and cationic polydadmac dosing systems	4.4.3	Short	Medium

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Polyacry-lamide	Jar testing or plant trials could be undertaken to further test the usefulness of polyacrylamides as coagulant aids	4.4.4	Medium	Medium
Lime	Upgrade screw feeder technology so that spare parts can be more easily obtained	5.1.6	Medium	Medium
Alum, Lime	Review bunding requirements for the dosing arrangement and enlarge if required	5.1.1, 6.2.1	Medium	Medium
Cationic polydadmac	Provide suitable bund for the mixing/ dosing tank and container(s) in which the liquid product is supplied and stored	5.1.2, 6.2.1	Medium	Medium
Polyacry-lamide	Provide suitable bund for the mixing/ dosing tanks (if to be used again in future)	5.1.3, 6.2.1	Medium	Medium
PAC	Any new system should consider ways to minimise manual lifting and to avoid operator contact with PAC dust	5.1.4	Medium	Medium
PAC	Provide suitable bund for the PAC dilution tank	5.1.4, 6.2.1	Medium	Medium
Chlorine	Ideally, the chlorine installation should be improved to meet the Australian Standards and Orica's recommendations	5.1.5	Medium	Medium
General	Any upgrades of the chemical systems should consider ways to further minimise manual handling requirements and the risk of operator contact with chemicals	6.2.2	Long	Medium
Fluoride	Investigate location and design of a new fluoride dosing facility in conjunction with WTP upgrade requirements.	5.3.3	Medium	Medium

7.1.10 General

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
SCADA	Ongoing training of the operators and other staff on the various SCADA capabilities would be beneficial.	6.1.2	Medium	Medium
Power failure	Council should consider the risks of interruption of water supply posed by longer term power failures and develop contingency plans for such an event	6.1.10	Medium	Medium
Power failure	Provide interlock controls of the raw water pumps to stop them if the signal from the plant is lost (e.g. if power fails at plant but not at pump station)	6.1.10	Medium	Medium
General	Provide a separate lunch room, with table and sink facilities, in an area where there is no risk of chemical contamination	6.2.3	Medium	Medium

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Site stability	The stability of the site should be checked and considered in the design and placement of any new structural components part of an upgrade of the WTP	6.3.2	Medium	Medium

7.2 Upgrade Requirements to Increase Treated Water Production Capacity

7.2.1 Design Basis for Upgrade Options

Based on considerations outlined in the earlier chapters of this report, the maximum capacity of the existing Biloela WTP has been shown to be:

- 120 L/s raw water inflow under typical water quality and up to 130 L/s under ideal water quality conditions. Note that sludge lagoon supernatant recycle stream is added after the raw water flow meter, so the flocculation, clarification and filtration processes may treat up to 8 L/s more than the raw water inflow;
- 80 L/s raw water inflow under high solids load in the raw water (relatively rare water quality conditions);
- 100 L/s raw water inflow with high levels of algal cells in the raw water.

Where possible, a modular approach to capacity upgrades is considered most practical and flexible for operation. Design of new unit processes to match the size of the existing clarifiers and filters where practical would give simpler and more flexible operation. It is noted that this approach was not practical for the design of the Stage 2 clarifier for Scenario 2, which has instead been designed at 1.5 times the capacity of each existing Stage 1 clarifier (as shown in the table below).

It is noted that water demand Scenario 1, 2 and 3 raw water flow rate requirements are based on the MDMM capacity basis and allow for up to 10% water 'losses' through the WTP process, as outlined in Section 2 of this report. Alternative WTP sizings based on 2 times the average daily demand are shown in brackets in the table below for reference.

The design of WTP capacity upgrades described below is based on the following assumptions:

- Raw water pump station upgrades would need to achieve the required output capacity at the lowest dam (pump suction head) levels;
- The maximum water demand would only occur only under dry weather conditions where the capacity of the existing Stage 1 process is equivalent to at least 120 L/s as raw water inflow and 130 L/s at the clarification and filtration stages i.e. full capacity of the upgraded plant is based on the Stage 1 components achieving these design flow rates. It is noted that there is a small chance that maximum summer demands would coincide with algal blooms, in which case the maximum demand may not be met if it is required to down-rate the Stage 1 components;
- The new Stage 2 components would be sized conservatively so that they were able to achieve their design flow rate even under worst case conditions of high turbidity or algal cell contamination. It is noted that this may mean they have some 'spare' capacity during good water quality conditions;
- Existing clarifiers are limited to around 130 L/s total capacity (65 L/s each) and 80 L/s (40 L/s each) under poor raw water quality. This is a conservative approach.

Up-rating to 150 L/s or more by modification of the inlet tubes may be possible but has not yet been proven.

- Existing filters are limited to around 130 L/s total capacity (32.5 L/s each) based on the filtration rates for the existing type of media. Upgrade of the media to a higher-rate configuration may be possible but has not yet been proven practical.
- Existing sludge lagoon supernatant recycle pump capacity is 8 L/s. Note that this is subject to confirmation by further testing.

7.2.2 WTP Upgrade Requirements

The table below summarises the plant upgrades which would be required to meet the demand scenarios developed in Section 2 of this report.

WTP Upgrade Requirements for Forecast Demand Scenarios

Parameter	Existing WTP Facilities	Upgraded WTP Requirements		
		Scenario 1: Total Demands minus Town Bore Flow	Scenario 2: Total Demands (Town Bores Not Avail)	Scenario 3: Total Demands + Meatworks 1 – 2 ML/d
Demand WTP Intake Capacity (L/s)	120 – 130 maximum	160 (*Alternative sizing: 210)	190 (*Alternative sizing: 240)	200 – 220 (*Alternative sizing: 255 - 270)
Design WTP Intake Capacity (L/s): (A) Typical raw water quality (B) Poor raw water quality	**(A) 120** **(B) 80**	**180** **140**	**210** **170**	**240** **200**
Sludge Lagoon Supernatant Return (L/s)	Up to 8	12	14	16
WTP Inflow plus S/natant Return (L/s)	**128**	**192**	**224**	**256**
Raw Water Pump Station Upgrades	2 (duty/ standby) pumps, each 130 – 180L/s	Add a third pump rated at 130L/s at lowest dam level 2 duty pumps with VSDs to give 180L/s flow rate	Add a third pump rated at 130L/s at lowest dam level 2 duty pumps with VSDs to give 210L/s flow rate	Add a third pump rated at 130L/s at lowest dam level 2 duty pumps with VSDs to give 240L/s flow rate
Raw Water Main Upgrades	At least 180L/s. Possibly up to 280L/s	Upgrade not required	May need to upgrade/ duplicate main	May need to upgrade/ duplicate main
Inline Flash Mixer and Pipe	Designed	Modify/ replace inline mixer,	Modify/ replace inline mixer,	Modify/ replace inline mixer,

Parameter	Existing WTP Facilities	Upgraded WTP Requirements		
		Scenario 1: Total Demands minus Town Bore Flow	Scenario 2: Total Demands (Town Bores Not Avail)	Scenario 3: Total Demands + Meatworks 1 – 2 ML/d
Fitting Upgrades	for 120 L/s	upgrade flow meter, control valve if reqd	upgrade flow meter, control valve if reqd	upgrade flow meter, control valve if reqd
Flash Mixing Tank	Hydraulic capacity 280 L/s	See discussion below	See discussion below	See discussion below
Clarifier Upgrades Raw water quality: (A) Typical (B) Poor	2 clarifiers: (A) Up to 130L/s (65L/s each) (B) Possibly down to 80L/s (40L/s each)	Add 1 x clarifier**, capacity: (A) 65L/s or more (B) 65 L/s Capacity with 3 clarifiers: (A) 195 L/s (B) 145 L/s	Add 1 x clarifier**, capacity: (A) 97.5L/s (B) 97.5 L/s (1.5 x one existing clarifier) Capacity with 3 clarifiers: (A) 227.5 L/s (B) 177.5 L/s	Add 2 x clarifiers**, capacity: (A) 130L/s or more (65L/s each) (B) 130L/s (65L/s each) Capacity with 4 clarifiers: (A) 260 L/s (B) 210 L/s
Filter Upgrades	4 filters: Max. 130 L/s (32.5 L/s each)	Add 2 x filters, capacity 32.5 L/s each Total capacity for 6 filters: 195 L/s	Add 3 x filters, capacity 32.5L/s each Total capacity for 7 filters: 227.5 L/s	Add 4 x filters, capacity 32.5L/s each Total capacity for 8 filters: 260 L/s
Filtered Water Well Upgrades	240 L/s (subject to testing)	Connect new filters to existing well	Connect new filters to existing well	Duplication/ modification of filtered water well may be required
Clear Water Tank Upgrades	1.8 ML (4h storage at 120L/s)	Need for second clear water tank discussed separately below	Need for second clear water tank discussed separately below	Need for second clear water tank discussed separately below
Treated Water Trunk Main Upgrades	At least 70L/s	Install booster pump or upgrade pipes to achieve 140L/s transfer rate	Install booster pump to achieve 170L/s transfer rate	Install booster pump to achieve 200L/s transfer rate
Town Reservoir Inlet Mixing Tank Upgrades	Approx 100L/s worst case	Upgrade tank hydraulic capacity to 140L/s	Upgrade tank hydraulic capacity to 170L/s	Upgrade tank hydraulic capacity to 200L/s
Wastewater Surge Tank and Sludge Lagoon	Surge tank and lagoons	No upgrades likely to be required (subject	Check capacity of Surge Tank to handle additional backwash/	Check capacity of Surge Tank to handle additional backwash/ clarifier

Parameter	Existing WTP Facilities	Upgraded WTP Requirements		
		Scenario 1: Total Demands minus Town Bore Flow	**Scenario 2: Total Demands (Town Bores Not Avail)**	**Scenario 3: Total Demands + Meatworks 1 – 2 ML/d**
Upgrades		to checking)	clarifier flows	flows
Lagoon Supernatant Recycle Pump Upgrades	2 (dty/stby) pumps, each 8L/s (to be confirmed)	Add new 8L/s pump and bypass/ control valve for up to 12L/s with 2 duty, 1 standby	Add new 8L/s pump and bypass/ control valve for up to 14L/s with 2 duty, 1 standby	Add new 8L/s pump and bypass/ control valve for up to 16L/s with 2 duty, 1 standby
Chemical Dosing System Upgrades	See Section 5 of this report	Upgrade alum and PAC systems	Upgrade capacity of alum, cationic polymer, polyacrylamide, PAC systems, Raw P/S chlorine and post-chlorine systems	Upgrade capacity of alum, cationic polymer, polyacrylamide, PAC systems, Raw P/S chlorine and post-chlorine systems

*WTP sizing based on MDMM. Alternative sizing based on 2 x AD. Refer to Section 2 of this report for further explanation.

**Clarifier numbers based on settling or conventional DAF clarifiers (separate to filters). Possible clarification option of 'DAF above filters' is discussed below. The bypass of clarification (direct filtration) is also discussed below.

7.2.3 Stage 2 Clarification Bypass (Direct Filtration) Mode Option

Because of the occasional high turbidity events in the dam, and because the existing WTP is a conventional treatment process, a conventional treatment process design would be recommended for future additional stages at the Biolela WTP. However the design of a future stage could include provision for bypass of the clarification stage to run the plant as a direct filtration process during the extended periods of low solids raw water quality.

In direct filtration, coagulation chemical dosing is followed by flocculation delay time and mixing energy and then the water flows directly to the filtration stage, thus the filters are the only step for solids removal. Chemical doses are generally lower than in conventional treatment, so treatment costs are reduced, although the direct filtration process is not as robust as the conventional treatment process and there is less delay time for the operator to react if the raw water quality changes.

Design of the Stage 2 facilities to allow them to be run in direct filtration mode would include a flocculation zone which was separate from the Stage 2 clarifier(s), so that flocculation energy could be employed even if the clarifier was bypassed for direct filtration. Certain clarifier designs are more suited to this approach than others, for example the Stage 1 clarifier design would be unsuitable as flocculation is incorporated within the sludge blanket below the settling zone. A suitable filter media configuration, designed to maximise floc storage within the filters, would also be required.

7.2.4 Design of Stage 2 Inlet Facilities

The existing Flash Mixing Tank reportedly has hydraulic capacity to be used as an inlet to both Stage 1 and Stage 2 up to 240 L/s, plus allowance for supernatant recycle and hydraulic overload up to 280 L/s. However a design using higher flow rates through the tank would need to be tested to prove that the reduced contact time in this tank would be adequate for the Stage 1 and Stage 2 mixing and floc formation requirements.

If the resulting contact time was found to be suitable, the existing Flash Mixing Tank could be used as a common inlet for Stage 1 and 2 if the Stage 2 process uses a similar settling type clarifier requiring identical coagulant doses to the Stage 1 process. It is understood that the Flash Mixing Tank has been designed to achieve hydraulic flow splitting between the two stages, however this facility should be further investigated, particularly if Stage 2 is designed for a different flow rate to Stage 1.

If the Stage 2 clarification process was different to the Stage 1 process, for example if a direct filtration process (see discussion above) or a dissolved air flotation clarification process (see further discussion below) was used, a branch in the raw water main and separate coagulant dosing facilities and inlet facilities would be needed to allow different coagulant doses and mixing energies to be provided for the Stage 2 process. For Stage 2, coagulant mixing may be achieved by an inline mixer in the Stage 2 raw water inlet pipeline, followed directly by the flocculation process components, rather than providing a second flash mixing tank for Stage 2. Flow control valves would be required on the inlet pipes for Stage 1 and 2 to achieve the required split in flow between the two stages.

7.2.5 Design of Stage 2 Flocculation and Clarification Components

The Stage 2 flocculation and clarification processes could be achieved using a settling-type clarifier design, such as a sludge blanket clarifier with tube settlers similar to the existing Stage 1 clarifiers or an alternative design with separate flocculation zone with or without tube settlers. If settling-type clarifier(s) are used, they should be designed with consideration of the following points:

- The design surface loading rate should be selected based on the clarifier type and the expected raw water quality;
- The design of the new clarifiers should avoid the problem of extended periods of sludge accumulation (to improve upon the design of the existing clarifiers);
- The design of new clarifiers should ensure suitable draining facilities (to improve upon the design of the existing clarifiers);
- The design of the clarifiers should allow for the dosing of significant doses of PAC upstream of the clarifiers, with sludge removal system designed to accommodate PAC-laden sludge;
- Design incorporating a separate flocculation zone would allow possible bypass of the clarifiers to run the WTP in direct filtration mode (as described above) during good raw water quality.

As an alternative to a settling-type clarifier design, clarification could be achieved using a dissolved air floatation (DAF) process. In a DAF process, flocculated water is combined with a stream of water containing dissolved air which combines the flow with micro-bubbles causing it to float to the surface instead of settling. The floated floc is periodically removed from the water surface, either hydraulically (by raising the water level) or by mechanical skimming. The process can be configured either as:

- Conventional DAF: Flocculation tank followed by DAF tank, separate to filtration process; or
- DAF above filters: Flocculation tank followed by DAF installed in the same structures as the filter cells, with the floc floated to the surface above the filter media.

DAF clarification is particularly suitable for waters which have low turbidity and may be difficult to settle due to organics or algal cells. Effective DAF designs are also generally capable of treating water with turbidity of up to 100 - 150 NTU and may treat even higher turbidities at a down-rated surface loading rate. Thus it is expected that the process would successfully treat the typical Callide Dam water, would also treat the worst case high turbidity 'fresh' events, and may be more successful than a settling process in treating water with high numbers of algal cells. Jar testing simulating the DAF process should be used to confirm the suitability of the process for treating all types of Callide Dam water and to determine the critical design parameters.

The DAF process typically has higher surface loading rates than settling clarifiers, thus a smaller tank footprint and lower capital costs, particularly for the 'DAF above filters' option, compared to settling clarification. The process uses smaller floc than a settling process, and thus chemical doses are lower, although the overall operating costs are generally similar due to the higher power costs associated with the saturator air compressors.

The 'DAF above filters' is considered a reasonable option the Stage 2 upgrade to the Biloela WTP, with potential for lower construction costs compared to the inclusion of separate clarifier tanks. Either of the DAF process options would be suitable for design to allow bypass to run in direct filtration mode if required.

The interaction between the Stage1 and Stage 2 components would need to be considered carefully, particularly if a DAF process is adopted for Stage 2. For example, if Stage 2 incorporated a 'DAF above filters' process, the filters of Stage 1 and 2 would need to operate independently, which would have implications for sizing the filters to allow for backwashing while the plant is online. These details should be further addressed in the early concept design stage for the upgrade.

7.2.6 Design of Stage 2 Filters

The new filters to be provided as Stage 2 would most practically be designed to have the same capacity as the existing filters. The footprint of the new filters may be different to the existing filters, depending on the filter media configuration and filtration rate used to design them.

The backwashing requirements for the new filters would also ideally be similar to the existing filters to allow the existing backwashing system to be used. However if this is not possible because a different filter media design is preferred, a second backwash pump and blower system could be provided as part of the upgrades.

Ideally, future upgrades would be designed to allow backwashing to be undertaken while the plant is online, with the flow from the offline filter distributed evenly between the remaining online filters, rather than the current practice where the operator washes the filters while the plant is off. The greater the number of filters running, the less the effect of taking one filter offline for backwashing while the other filters continue to run. For the sizings given in the table above, it has been assumed that the addition of at least two more filters would allow one filter to be taken off line for backwashing without an excessive flow increase in the remaining online filters. This should be reassessed when process options for the clarifiers and flow interaction between Stage 1 and Stage 2 are known. It is noted that the maximum filtration rates can be better determined for the Stage 1 filters when the media has been analysed to determine the effective size and uniformity coefficient.

Ideally the new filters would be installed with headloss measurement (differential pressure cells) and online turbidimeters for each filter and with provision for automatic backwashing. The existing filters would also ideally be upgraded to include these features.

The design of the new filters should also allow for possible PAC dosing upstream of the filters, currently a common practice, which will add to the solids loads on the filters and decrease run times. The Stage 2 filters would also ideally be designed to be suitable for direct filtration, if required, as noted above.

7.2.7 Treated Water Transfer System Upgrade Requirements

Under higher ultimate demand scenarios, the rate of water transfer between the WTP and the town reservoirs would also need to be increased. As around 40 L/s is supplied directly from the WTP clear water tank to the Power Station, the required maximum transfer rate to town is assumed to be the required WTP design flow rate (based on the nominal demand rate) minus 40 L/s.

SKM (2006) advised that a booster pump would be required to achieve adequate flow rate through the trunk main, with the hydraulic capacity of the town reservoir inlet mixing tank also a limiting factor. Upgrades to achieve higher transfer rates using this approach are outlined in the table below. It is noted that alternative approaches, such as splitting the flow between the main ground level reservoirs and other reservoirs may also achieve increased transfer rates, however these options would require further investigation before they can be proved to be effective.

Trunk Main Upgrade Requirements for Forecast Demand Scenarios

Parameter	Existing Facilities	Upgraded WTP Requirements		
		Scenario 1: Total Demands minus Town Bore Flow	**Scenario 2: Total Demands (Town Bores Not Avail)**	**Scenario 3: Total Demands + Meatworks 1 – 2 ML/d**
Design WTP Intake Capacity (L/s):	**120**	**180**	**210**	**240**
Treated Water Trunk Main Upgrades	At least 70L/s	Install booster pump or upgrade pipes to achieve 140L/s transfer rate	Install booster pump to achieve 170L/s transfer rate	Install booster pump to achieve 200L/s transfer rate
Town Reservoir Inlet Mixing Tank Upgrades	Approx 100L/s worst case	Upgrade tank hydraulic capacity to 140L/s	Upgrade tank hydraulic capacity to 170L/s	Upgrade tank hydraulic capacity to 200L/s

7.3 Additional High Priority Upgrade Requirements

Tasks additional to the WTP capacity upgrade requirements and considered *high priority* (as identified in Section 8.1) are discussed further below. All of these tasks would be required to be carried out in the *short to medium timeframe.*

7.3.1 Monitoring and Treatment of Algal Cells

Algal blooms occur regularly in Callide Dam. The water treatment processes of coagulation, clarification and filtration can generally remove algal cells, but problems such as sludge flotation, a higher solids loading and filter clogging may occur. Release of undesirable intracellular compounds from the algal cells is another risk associated with algal blooms.

Operational approaches to address algal blooms in the dam include:

- Monitor algae levels closely (in conjunction with Sunwater's algae monitoring program if appropriate) and analyse for toxins if significant levels of potentially toxic algae are present;
- Investigate cause of taste and odour problems. Analyse raw water samples known to contain tastes and odours to identify the compounds responsible;
- Selectively extract water from a deeper level in the dam to avoid water high in algal cells;
- Avoid physical or chemical damage to the algae cells by pre-coagulation chlorination or high energy mixing, to prevent any intracellular taste and odour compounds and/ or algal toxins being released into the water;
- Optimise coagulation chemical dosing and possibly add polyacrylamide to better settle the solids in the clarifiers;
- Monitoring recycled sludge lagoon supernatant quality closely, as it is likely that algal cells will break down in the lagoons and potentially release intracellular compounds.

Depending on the threat from algal blooms and the ability to selectively withdraw water from the dam, additional treatment process options which could be considered as part of any WTP upgrade include:

- The use of dissolved air flotation (DAF) clarification process rather than settling in the new Stage 2 clarifier(s);
- Measures to address taste and odour compounds and algal toxins as outlined further below.

Another barrier to algal blooms is a destratification system within the dam, which would aim to keep the dam well mixed to reduce the advantage of blue-green algae over other algae in the dam. This approach may help to reduce the severity of algal blooms in the dam, although algal problems are influenced by various other factors which would also need to be controlled. Provision of a destratification system in the dam has not been further addressed in this report as it is considered outside the scope of the study and would require input from parties other than Council who have control of the operation of the dam.

7.3.2 Treatment of Taste and Odours and Algal Toxins

Taste and odour problems occur regularly in the Callide Dam water, although the source of this problem and the specific taste and odour compounds causing the problem have not yet been identified. Algal toxins may also occur as several toxin-producing species of blue-green algae have been found in blooms on the dam, although toxins have not yet been measured above detectable levels in limited sampling. Taste and odour compounds lead to customer complaints but may not pose a threat to health, whereas toxins are a danger to health if present at certain threshold levels.

Dissolved taste and odour compounds and algal toxins cannot be removed by simple solids removal processes. Their destruction and/ or removal requires one or more of the following processes:

- Adsorption of the taste and odour compounds and/ or toxins onto activated carbon or by biological processes as part of the ozone-biological activated carbon process;
- Oxidation to destroy (break down) the taste and odour compounds and/ or toxins (with or without subsequent removal of the resulting products).

PAC (powdered activated carbon) is currently dosed at the WTP to absorb taste and odour compounds, with operators reporting that the PAC is normally quite successful in removing the tastes and odours as measured by observation. As discussed in Section 5 of this report, PAC doses up to 4 mg/L have been applied, although the PAC dosing system is limited to lower doses at the full WTP flow rate. Typical doses of around 2 mg/L doses are commonly dosed downstream of the clarifiers, but it is necessary to dose the PAC into the Flash Mixing Tank when higher doses are applied because of the impact of the higher solids loading on filter headloss. In theory, a minimum of 15 minutes of contact time is generally sufficient for most taste and odour compounds, however the compounds MIB and geosmin may require considerably longer contact times.

PAC can also be effective for the removal of algal toxins such as cylindrospermopsin, microcystin, nodularin and saxitoxins, however to achieve high removal efficiency of algal toxins with PAC, significant doses (60 - 100 mg/L) and long contact times (at least 30 minutes and sometimes much higher) are generally required. It is also necessary to select a PAC type suitable to the particular toxin targeted. Because of the high doses required, dosing the PAC upstream of the clarifiers would be required to remove most of the PAC solids before the filters. The detention time in the existing clarifiers is around 1 hour at 120L/s flow rate. If required, further detention time could potentially be achieved by dosing into the raw water pipeline upstream of the plant, for example at the raw water pump station, although the issues of having a dosing system remote from the WTP and the risk of PAC buildup within the raw water pipeline would need to be addressed in the design of the system. The impact of elevated doses of PAC on the sludge characteristics and sludge removal from the existing clarifiers should also be investigated.

PAC adsorption will be affected by the level of background organics in the water. Oxidising chemicals may also interfere with the adsorption capacity of the PAC, thus pre-chlorine dosing should be halted when PAC is being dosed.

Chlorine oxidation, under optimum dose and pH conditions, is reported to be effective for the destruction of blue-green algal toxins cylindrospermopsin, microcystin-LR, nodularin and may also be effective for saxitoxin. It may also have some effect on taste and odour compounds although it is not considered a reliable treatment for earthy / musty odour compounds such as MIB and geosmin even at high doses and significant contact times. To maximise the effectiveness of oxidation dosing and minimise the formation of undesirable by-products, chlorine should ideally be dosed after coagulation and/or filtration when the level of organics has been reduced. Chlorine dosed to treated water for disinfection purposes is acknowledged to have an effect on the destruction of low levels of taste and odour compounds and algal toxins which have passed through the WTP process.

Operational and upgrade measures which could be used to increase the effectiveness of the WTP for taste and odour and algal toxin removal include:

- Upgrade of PAC dosing system to achieve doses of at least 60 mg/L, with dosing points prior to the clarifiers. Further investigation should confirm optimal pre-clarifier dosing point and check the impact of high PAC doses on the clarifiers;
- Jar testing to look at various PAC products and detention time requirements for the target taste and odour and algal toxin contaminants;
- Maintenance of adequate disinfection chlorine doses which will provide an additional barrier for some tastes and odours and algal toxins.

If there was perceived to be an ongoing high risk from algal toxins and/ or PAC dosing was no longer sufficient to treat taste and odour problems, and ozone plus biological activated carbon (BAC) process could be considered to remove organic compounds from the filtered water. The combination of ozone with biological activated carbon (BAC) provides both oxidation and adsorption (biological) treatment processes. The combined process is effective for the removal of a wide range of taste and odour compounds and algal toxins, as well as other organic compounds. Many WTPs around Australia have employed this process for the high level treatment or organic contaminants where the risk was perceived to be high and/or very high quality treated water was required.

7.3.3 Treatment of High Solids Raw Water

High solids raw water from the occasional dam ‘fresh’ event has reportedly been hard to settle in the existing clarifier. It is likely that not only the high turbidities but also the rate of change in the water quality would have caused operational challenges during such an event.

Operational and upgrade measures which could be used to increase the effectiveness of the WTP for treating poor quality water:

- Jar testing to look at optimisation of chemical dosing, including addition of pre-coagulation lime and/ or flocculant aid, to achieve better coagulation and settling with very dirty water;
- Re-commissioning or further testing of pre-coagulation lime dosing which may be required to compensate for lower raw water and higher alum doses during such events;
- Conservative design of the Stage 2 clarifiers to treat dirty water while maintaining design flow rates.

7.3.4 Upgrade of PAC Dosing Capacity or Ozone BAC Alternative

From a review of the chemical dosing capacities, it was seen that the existing PAC system does not have enough capacity to meet the current expected worst case dose of 4 mg/L at the WTP design flow rate of 120 L/s to address taste and odour contamination.

As an interim measure, the output of the existing, temporary PAC dosing system could be boosted to achieve at least the equivalent of, say, 4 mg/L by replacing the screw feeder only. A larger auger is reportedly available onsite, but it has not yet been determined whether this auger is suitable and whether it could achieve the full range of PAC doses currently used.

A PAC system capable of dosing at the higher doses potentially required to treat algal toxins would be substantially larger than the existing WTP PAC system. To treat 10 - 15 ML/d of water at a PAC dose of 60 mg/L would consume 500 – 1000 kg of PAC product per day. These quantities of chemical use would be more suited to a bulk bag unloading system rather than the manual unloading of 20 kg bags.

Ideally the existing PAC system would ultimately be replaced with a larger capacity permanent bulk bag system suitable for the higher doses of around 60 mg/L. A bulk bag type PAC system would require:

- PAC bulk bag storage area;
- PAC bulk bag unloading system – gantry and crane or possibly a forklift arrangement similar to the existing alum unloading system;
- Hopper of at least 1.5 t capacity;
- Screw feeder of capacity to achieve at least 60 mg/L into the design water flow;
- Suitably sized dilution water stream, dilution tank, wetting up arrangement and ejector;
- Suitable separate room or shed to house the equipment, fitted with dust extraction system and platforms/ stairs as required to access the equipment.

As an alternative to the use of PAC for removing organic contamination from the water, and ozone BAC could be considered as a very effective but high capital and operating cost option. An ozone BAC system would require the following major facilities to be added to the WTP:

- Ozone generator (may also require a power system upgrade to feed generator);
- Ozone contact tank;
- BAC filter beds and associated backwashing and monitoring equipment.

7.3.5 Upgrade of Alum Dosing Capacity

From the review of the chemical dosing capacities it was seen that the existing alum system cannot meet the expected worst case dosing requirements at the current WTP design flow rate of 120 L/s. An upgrade of the system is recommended to allow the worst case doses (125 mg/L or higher) to be achieved at full plant flow. Any system upgrade should also be designed to cater for the expected ultimate dosing requirements if the WTP capacity is increased.

An upgrade of the existing alum system would require installation of a larger sized screw feeder. The existing system is quite old but apparently performs adequately apart from the output varying over the long term due to clutch wear and chemical build-up. Council have also recently reported difficulty in sourcing bulk bag granulated alum from local suppliers. Spare parts are no longer sold for the feeding system, so an update to current technology would be recommended as part of the screw feeder upgrade.

As an alternative, the existing granular alum system could be replaced by a new liquid alum system of appropriate capacity. Liquid alum has significantly lower manual handling requirements than granular alum and the dosing system is generally simpler to operate. The liquid alum is a corrosive chemical and appropriate operator safety equipment and tank bunding would be required for this system. Liquid alum is usually delivered in a 25t bulk tanker. A suitable access road and truck turning area would be required to allow the delivery of the chemical.

It is noted that liquid alum chemical costs are lowest if the system can accept a full tanker load of 25t, as transport costs are charged per tanker load. This would require storage capacity of at least 20 kL. A volume of 20 kL would provide around 95 days storage at current typical alum dosing rates (30 mg/L into 4 ML/d), and around 14 days storage at potential maximum rates (100 mg/L into 8 ML/d) at the current WTP design flow of 120L/s. This storage volume would also be adequate if the capacity of the WTP was doubled to 240 L/s in the future.

A liquid alum system would include:

- A bunded bulk tanker delivery area;
- One or more storage tanks and bunding;
- Duty/ standby dosing pumps;
- Carrier/ dilution water systems to assist dosing.

7.3.6 Second Clear Water Tank

A second clear water tank is required to allow the existing tank to be taken off line for repairs to address leaks. The provision of a new tank of similar size to the existing tank would be recommended to simplify operation, providing the same storage volume and detention times for chlorination regardless of which individual tank is online.

A second equal-sized clear water tank would increase the total clear water storage volume at the WTP site to around 3.6 ML, equivalent to around 8 hours operation at 120 L/s. The greater storage volume would allow a larger buffer to be used to reduce the number of WTP startups per day. It would also provide a longer disinfection detention time if higher WTP flowrates are implemented.

Connections to and between the two clear water tanks should allow for either tank to be online independently, but also for the tanks to be operated in series if both tanks are online. This will simplify flow paths through the tanks and allow better control of the disinfection contact time. The design of the new tank should consider flow short circuiting issues and would ideally include inlet/ outlet design and internal baffles to minimise short circuiting to achieve the best possible effective disinfection time. Baffles may also be able to be retrofitted inside the existing clear water tank once it is able to be taken offline.

7.3.7 Standby Cationic Polydadmac Dosing Pump

Standby equipment for all coagulant chemicals should be kept in stock to enable the process to be swiftly re-established on breakdown of duty equipment. In particular it was noted that there was no standby dosing pump for cationic polydadmac kept in stock. A suitable pump should be obtained and kept on site.

7.3.8 Chlorine Residual Meter SCADA Connection

The online treated water chlorine residual meter should be connected to the SCADA system as soon as possible, to allow the provision of a dial out alarm to alert the operator if disinfection dosing fails.

7.3.9 Clarifier Access Walkways

Suitable walkways with railings are needed on the existing clarifiers to allow safe access to various parts of the clarifier for maintenance activities.

7.4 Additional Medium and Low Priority Upgrade Requirements

All operational and upgrade measures recommended to improve the WTP performance are listed in Section 8.1. Treatment process improvements of *medium or low priority* are commented on further below.

The measures considered *medium or low priority* have not all been discussed in detail or costed, however all of these measures would be useful to improve either the operability and/ or the performance of the WTP and it is recommended that Council pursue them as practical in future.

7.4.1 Treatment of Sludge Lagoon Supernatant Recycle Stream

The recycling of the sludge lagoon supernatant is beneficial in terms of water conservation and minimising the release of water from site. The quality of the sludge lagoon supernatant has at times been unsuitable for recycling back into the raw water stream due to contamination with taste and odour compounds. Although the supernatant is significantly diluted in the main water stream, taste and odour compounds in particular can still cause a taste or odour in the water at very low levels. In current practice, the recycling of the supernatant is halted when there is a contamination problem and the water overflows to waste onto the neighbouring property.

Achieving ongoing supernatant recycling when the quality of the supernatant is poor would save water and may be required if offsite discharges are limited by the regulatory bodies in future. It is noted that recycling of poor quality supernatant is only recommended where the contaminants do not pose a health risk, and supernatant recycling would *not* be recommended at any time the supernatant contained concentrated levels of algal toxins or other substances at levels which may not be removed by the WTP process and could harm the health of the consumers.

Potential options to extend the ability to recycle poorer quality supernatant are outlined below:

- Dose extra PAC to the main water stream to target the problem compounds in the supernatant;
- If possible, minimise the detention time of sludge in lagoons to reduce the resolubilisation of problem compounds;
- Add a separate treatment system for the supernatant stream e.g. Second PAC dosing facility (removal and disposal of additional PAC solids should be considered) or chlorination (may create elevated levels of byproducts such as THMs);
- Install granular activated carbon (GAC) filter beds, sacks or pits where the supernatant can contact the GAC for adsorption of problem compounds similar to PAC.

All of the options listed above have advantages and disadvantages which would need to be considered, were the continuous recycling of supernatant required in future.

7.4.2 Treatment of Bore Water to Reduce Corrosivity

A review of the potential corrosivity of the town water types showed that the bore water at times shows high potential corrosivity, possibly due to dissolved carbon dioxide. If corrosion problems are found to be ongoing, jar testing could be used to look at aeration of the bore water as a way of releasing the carbon dioxide and making the water less potentially corrosive.

8. Budget Costs for Critical Upgrade Requirements

8.1 WTP Capacity Increases

The estimated capital costs for measures required to increase the capacity of the WTP to meet the three future demand scenarios are outlined in the following tables. The costs allow for the construction of new structures, supply and installation of equipment, piping, electrical, instrumentation and control components. Allowances have also been added for site establishment, commissioning, engineering and project management and contingency. The costs do not include GST.

8.1.1 Upgrades Including Stage 2 Settling Type Clarification

The first table shows the costs for the option of a settling-type clarifier for the Stage 2 process.

Estimated Capital Costs for WTP Capacity Upgrades – Settling Clarifier Option

Item	Estimated Cost			Cost Basis
	Scenario 1	Scenario 2	Scenario 3	
Raw Water Pump Station – Add new 130L/s pump	$200,000	$200,000	$200,000	CWT estimate based on similar eqpt
Raw Water Main – Duplicate main pipe	Not reqd	$50,000	$100,000	Allowance
Inline Flash Mixer and Fittings - Upgrade to larger Capacity	$20,000	$30,000	$40,000	Allowance
Flash Mixing Tank Modifications	$10,000	$10,000	$10,000	Allowance
Construction of Stage 2 Settling Clarifiers plus separate filters	$1,500,000	$2,000,000	$2,800,000	CWT estimate based on similar eqpt
Filtered Water Well upgrade	Not reqd	Not reqd	$15,000	Allowance
Clear Water Tank and Treated Water Transfer System upgrades	Cost given separately in later section	Cost given separately in later section	Cost given separately in later section	-
Wastewater Surge Tank and Sludge Lagoons – Duplicate surge tank	Not reqd	Possibly required – Allow $15,000	Possibly required – Allow $15,000	Allowance
Lagoon Supernatant Recycle Pumps – Add new 8L/s pump	$15,000	$15,000	$15,000	Estimate
Chemical Dosing Systems - Upgrades to give same mg/L capacity for higher WTP flows	$50,000	$100,000	$100,000	Allowance

Item	Estimated Cost			Cost Basis
	Scenario 1	Scenario 2	Scenario 3	
Sub-total	**$1,795,000**	**$2,420,000**	**$3,295,000**	
Site Establishment, QA, Commissioning, O&M manual etc	$100,000	$100,000	$100,000	Allowance
Engineering & Project Management	$359,000	$484,000	$659,000	20% Allowance
Contingency	$538,500	$726,000	$988,500	30% Allowance
TOTAL	**$2,792,500**	**$3,730,000**	**$5,042,500**	

8.1.2 Upgrades Including Stage 2 DAF Clarification

The next table shows the costs for the option of the DAF above filters alternative for achievement of clarification for the Stage 2 process.

Estimated Capital Costs for WTP Capacity Upgrades – DAF Above Filters Option

Item	Estimated Cost			Cost Basis
	Scenario 1	Scenario 2	Scenario 3	
Raw Water Pump Station – Add new 130L/s pump	$200,000	$200,000	$200,000	CWT estimate based on similar eqpt
Raw Water Main – Duplicate main pipe	Not reqd	$50,000	$100,000	Allowance
Stage 2 Inlet Static Mixer, Flow meter, Flow Control Valve	$25,000	$25,000	$25,000	Allowance
Flocculation Tanks upstream of DAF	$60,000	$80,000	$100,000	Allowance
DAF Above Filters – DAF Equipment plus Filter Cells and Eqpt	$1,300,000	$1,800,000	$2,200,000	CWT estimate based on similar eqpt
Filtered Water Well upgrade	Not reqd	Not reqd	$15,000	Allowance
Clear Water Tank and Treated Water Transfer System upgrades	Cost given separately	Cost given separately	Cost given separately	-
Wastewater Surge Tank and Sludge Lagoons – Duplicate surge tank	Not reqd	Possibly required – Allow $15,000	Possibly required – Allow $15,000	Allowance
Lagoon Supernatant Recycle Pumps – Add new 8L/s pump	$15,000	$15,000	$15,000	Estimate

Item	Estimated Cost			Cost Basis
	Scenario 1	Scenario 2	Scenario 3	
Chemical Dosing Systems - Upgrades to capacity plus separate alum and polydadmac dosing point for Stage 2	$60,000	$110,000	$110,000	Allowance
Sub-total	**$1,660,000**	**$2,295,000**	**$2,780,000**	
Site Establishment, QA, Commissioning, O&M manual etc	$100,000	$100,000	$100,000	Allowance
Engineering & Project Management	$332,000	$459,000	$556,000	20% Allowance
Contingency	$498,000	$688,500	$834,000	30% Allowance
TOTAL	**$2,590,000**	**$3,542,500**	**$4,270,000**	

8.1.3 WTP Stage 2 Operating Costs

Operating costs for Stage 2 of the WTP, in terms of cost per ML water treated, are expected to be similar to the existing process (± 20%) for both the settling clarifier and the DAF conventional treatment process options.

If the plant was designed to run in direct filtration mode when raw water quality was suitable, then the operating costs may be reduced to around 80% of cost of conventional treatment when operating in this mode.

8.2 Treated Water Transfer System Capacity Increases

The estimated capital costs for measures required to increase the rate of treated transfer between the WTP and the town reservoirs to meet the three future demand scenarios are outlined in the table below.

Estimated Capital Costs for Trunk Main Capacity Upgrades

Item	Estimated Cost			Cost Basis
	Scenario 1	Scenario 2	Scenario 3	
Design WTP Intake Capacity (L/s)	**180**	**210**	**240**	
Treated Water Trunk Main – Booster pumps (duty/standby) with bypass and NRV	$210,000	$230,000	$250,000	SKM (2006) estimated $209,000 for booster pump for Scenario 1
Inlet Mixing Tank at Town Reservoirs – Modify to allow higher inflow	$30,000	$30,000	$30,000	Allowance
Sub-total	**$240,000**	**$260,000**	**$280,000**	

Item	Estimated Cost			Cost Basis
	Scenario 1	Scenario 2	Scenario 3	
Design WTP Intake Capacity (L/s)	**180**	**210**	**240**	
Engineering & Project Management	$36,000	$39,000	$42,000	15% Allowance
Contingency	$36,000	$39,000	$42,000	15% Allowance
TOTAL	**$312,000**	**$338,000**	**$364,000**	

8.3 Additional High Priority Upgrade Requirements

8.3.1 Upgrade of PAC System or Ozone BAC Alternative

The PAC system needs to be upgraded to be able to dose an adequate PAC dose at even the current WTP design rate of 120 L/s. It could potentially be upgraded by installing a larger screw feeder (to be confirmed by further investigation).

The preferred upgrade for the PAC system would allow the system to dose at least 60 mg/L into the full plant flow rate to address the potential need for algal toxin removal. The estimated capital costs for a new, permanent bulk bag PAC system of a size suitable to dose up to 60 mg/L (equivalent to around 500 – 1000 kg/day at 120 - 240 L/s WTP flow rates) are given in the table below.

Estimated Capital Costs for High Dose PAC System

Item	Estimated Cost	Cost Basis
Bulk bag unloading, storage and dosing system as described in 8.2.4 above	$400,000	CWT estimate based on similar eqpt
Site Establishment and shed to house PAC system	$30,000	
Sub-total	**$430,000**	
Commissioning, O&M manual, Design, Engineering & Project Management	$43,000	10% Allowance*
Contingency	$64,500	15% Allowance
TOTAL	**$537,500**	

*System detail design, commissioning, O&M manual etc by supplier and included in $400,000 system supply and installation cost

As an alternative to PAC dosing, an ozone BAC system could be considered if there was an ongoing risk of organic contamination. The size of each component of this system would best be determined by jar or pilot testing to determine the required ozone dose and the BAC filter empty bed contact time. Indicative costs for this option are given in the table below. As the power demand of an ozone generator is high, an upgrade of the power supply to the WTP may be required but has not been allowed for at this stage.

Estimated Capital Costs for Ozone BAC System

Item	Estimated Cost		Cost Basis
	Current WTP 120 L/s	**Possible Ultimate WTP 240 L/s**	
Ozone generator	$800,000	$1,000,000	CWT estimate based on similar eqpt
Ozone contact tank and BAC filters	$3,000,000	$4,000,000	CWT estimate based on similar eqpt
Sub-total	**$3,800,000**	**$5,000,000**	
Site Establishment, QA, Commissioning, O&M manual etc	$100,000	$100,000	Allowance
Design, Engineering & Project Management	$760,000	$1,000,000	20% Allowance
Contingency	$1,140,000	$1,500,000	30% Allowance
TOTAL	**$5,800,000**	**$7,600,000**	

8.3.2 Upgrade of Alum Dosing Capacity

The existing alum system needs to be upgraded to be able to dose the worst case expected dose even at the current WTP design rate of 120 L/s. The system could be upgraded by installing a larger screw feeder into the existing hopper. Estimated costs for this upgrade are given below, assuming that a suitable feeder can be purchased to fit in the existing hopper system.

Estimated Capital Costs for New Granular Alum Feeder

Item	Estimated Cost	Cost Basis
Screw feeder - Max capacity approx. 55 – 110 kg/h (depending on WTP flow), plus supporting equipment and instruments	$10,000	Allowance
Electrical and control installation	$10,000	Allowance
Sub-total	**$20,000**	
Design, Engineering & Project Management	$4,000	20% Allowance
Contingency	$6,000	30% Allowance
TOTAL	**$30,000**	

As an alternative, the granular alum system could be replaced with a liquid alum system, which would reduce manual handling. The estimated costs for a new liquid alum system are given in the table below. It is assumed that the storage tanks could be located outside the WTP building and the dosing pumps would be located within the WTP building, thus the costs do not allow for a new shed to house the equipment, or for roofing of the alum tank bunds. It is also assumed that the existing road and truck turning facilities at the WTP are suitable for 25t bulk tankers (this should be checked), thus the costs do not include any road modifications except for the truck unloading bund area.

Estimated Capital Costs for New Liquid Alum System

Item	Estimated Cost	Cost Basis
Storage Tanks – Say 2 x 10 kL plus tank instruments and bunding	$60,000	Allowance
Dosing System – Duty/ standby dosing pumps, max capacity approx. 90 – 190 L/h (depending on WTP flow), plus supporting equipment and instruments	$50,000	Allowance
Tanker unloading area bund	$30,000	Allowance
Electrical and control systems	$40,000	Allowance
Sub-total	**$180,000**	
Site Establishment, QA, Commissioning, O&M manual etc	$20,000	Allowance
Design, Engineering & Project Management	$36,000	20% Allowance
Contingency	$54,000	30% Allowance
TOTAL	**$290,000**	

The delivered cost per tonne of liquid alum is roughly half that of granular alum (based on comparison of indicative prices from Omega Chemicals, June 2009), however its active strength is around 0.45 times that of granular alum, so the equivalent cost per mg/L active ingredient is slightly higher for the liquid alum.

8.3.3 Other High Priority Works

Cost estimates for the other high priority upgrade works discussed in this section are outlined in the table below.

Estimated Capital Costs for Other High Priority Upgrades

Item	Estimated Cost	Cost Basis
Second clear water tank	$1,000,000	Council budget allowance
Standby cationic polydadmac dosing pump	$6,000	Allowance
Chlorine residual meter connection to SCADA system, with dial out alarm	$10,000	Allowance
Clarifier access walkways	$45,000	Subcontractor quote advised by Council
Analyse water for algal toxins if significant levels of potentially toxic species measured	$5,000	Allowance
Analyse water when taste and odours are present to identify taste and odour compounds	$5,000	Allowance

Item	Estimated Cost	Cost Basis
Jar testing of PAC products to compare their effectiveness and contact time requirements with compounds of interest	$15,000	Allowance
Jar testing simulating dirty raw water to optimise coagulation and settling during high turbidity periods	$10,000	Allowance

9. Findings and Recommendations

9.1 Findings

9.1.1 WTP Flow Rate and Demand Issues

The following parameters for the existing WTP system were determined during the course of this investigation:

- Design water inflow capacity of the current WTP is 120 L/s. Achievable flow rates vary between 80 – 130 L/s depending on raw water quality conditions;
- Water losses through the existing process average around 5% of the raw water inflow;
- Assuming 20 h/day WTP operation at maximum demand, the current WTP could produce 7.8 – 8.4 ML/d treated water (based conservatively on 10% process losses) at 120 – 130 L/s. In poor quality raw water, production may be limited to 5.2 ML/d at 80 L/s.

Current WTP treated water consumers include the various towns and the Callide power station. The Biloela meat works may become a consumer in future, subject to an agreement with Council.

Water from the WTP has historically been complemented by flows from the town bores, however it is not certain that these bores will continue to be available into the future because of record low aquifer levels.

Demand for WTP treated water was determined for the three scenarios outlined in the following table. As per Council's advice, the required WTP treated water capacity for each scenario was determined based on the MDMM value, as detailed in section 2.2.2. It was noted that more conservative bases can be used, resulting in a higher WTP design flow rate but reducing the load on water storages during extended periods of maximum demand.

WTP Demands and Flow Requirements Summary

Scenario	WTP Production/ Demand Conditions	WTP Treated Water Production Capacity (ML/d)	Associated WTP Inlet Flow Rate (L/s)**
Ultimate WTP Production Demand **Scenario 1**	Ultimate town development, with town bore output available to complement WTP output	10.2	160
Ultimate WTP Production Demand **Scenario 2**	Ultimate town development, with town bores producing zero output	12.1	190
Ultimate WTP Production Demand **Scenario 3**	As for Scenario 2, plus extra meat works demand of 1-2 ML/d	13.1 – 14.1	210 - 220

*Calculated using MDMM basis

**Allows for 10% water losses through the WTP process and maximum 20 hours WTP operation per day.

It is noted that permanent water restrictions may potentially reduce the WTP production demands for all of these scenarios, however the effect of potential water restrictions has not yet been quantified.

9.1.2 Water Quality Issues

A review of the raw water and WTP treated water quality found that:

- Callide Dam raw water is generally of good quality, with the main problems being algal blooms, taste and odour compounds and the occasional high turbidity event from significant rain water inflows into the dam;
- WTP treated water typically meets target values, but with periodic excursions. Filtered water turbidity is sometimes above target levels. Final treated water chlorine residuals are highly variable and are sometimes not adequate to maintain a residual at Callide Township (supplied directly from the clear water tank);
- Town bore water, added to the water supply downstream of the WTP, is low in turbidity but has sometimes had elevated TDS and nitrate levels;
- The combined town water has historically been variable due to the changing blends of WTP treated water and town bore water. It has sometimes had elevated TDS and nitrate. Chlorine levels after re-chlorination at the town reservoir may not always be adequate, especially at system extremities. Coliforms have been present in a number of samples, but E.Coli has not been detected.
- From modelling of corrosivity potentials, the bore water is generally likely to be corrosive. The WTP treated water is not likely to be corrosive except during dam fresh water inflows.

9.1.3 WTP Process, Chemical Systems and Operational Issues

A review of the WTP treatment processes and chemical dosing systems found that:

- Most of the plant components are matched well in terms of meeting the 120 L/d design flow. Under very dirty raw water conditions or the presence of algal cells, the clarifiers may limit plant throughput to 80 L/s due to poorer settling;
- In terms of increasing the WTP inflow rate above 130 L/s, the limiting components are the raw water pumps, clarifiers, filters and potentially some of the inlet components and the supernatant return pumps. The clear water tanks, sludge lagoons and flash mixing tank may be suitable to service significantly higher WTP flow rates;
- Of the chemical dosing systems, the alum and PAC systems are somewhat undersized at the current WTP rate of 120 L/s. Most of the other chemical systems would service WTP flows up to 200 L/s. The lime and WTP pre-chlorine would service WTP flows up to 240 L/s at likely maximum doses;
- A new fluoride dosing system either at the WTP or at the town reservoir is required to meet the Qld government requirement for fluoridation by 2012;
- Plant control and automation, safety and maintenance issues were also reviewed, with recommendations developed as outlined below.

9.2 Recommendations

9.2.1 Recommendations for Upgrades to Increase System Capacity

The recommended upgrades to increase the treated water capacity have been outlined in Section 7 of this report for the three main demand scenario options. The required upgrades include the addition of a Stage 2 process to the WTP as well as upgrades to various supporting equipment including the raw water supply system and the treated water transfer system between the WTP and the town reservoirs. Cost estimates for the various upgrade options are given in Section 8.

It is recommended that Council discuss the demand scenarios further based on the information and costs given in this report to identify the most suitable demand basis for design of the upgraded WTP. The impact of water restrictions and other potential water efficiencies on the ultimate town demands should also be considered.

For design of the future Stage 2 upgrade, it is recommended that Council consider dissolved air flotation (DAF) clarification as a process alternative to conventional settling, as a DAF option will have lower capital costs and may be more successful at treating algal cells than a settling clarifier process.

Design of the upgraded WTP to allow operation in direct filtration mode, if practical, would also be recommended to potentially reduce operational costs during the extended periods of good raw water quality.

9.2.2 Additional Recommended Upgrades

Recommended improvements additional to the WTP capacity upgrade requirements are listed in Section 7 of this report, with cost estimates for the highest priority tasks given in Section 8. All of these measures would be useful to improve either the operability and/ or the performance of the WTP and it is recommended that Council pursue them as practical in future.

Tasks which it is recommended that the Council pursue with high priority are listed below.

Short timeframe actions:

- Analyse water for algal toxins if significant levels of potentially toxic species measured;
- Analyse water when taste and odours are present to identify taste and odour compounds;
- Chlorine residual meter connection to SCADA system, with dial out alarm;
- Clarifier access walkways;
- Second clear water tank;
- Obtain standby cationic polydadmac dosing pump;
- Jar testing simulating dirty raw water to optimise coagulation and settling during high turbidity periods.

Medium timeframe actions:

- Upgrade PAC system capacity to at least 60 mg/L, or look at ozone BAC as alternative for treating organic contamination;
- Jar testing of PAC products to compare their effectiveness and contact time requirements with compounds of interest;
- Upgrade existing granular alum system or install new liquid alum dosing system.

10. References

- Department of Natural Resources and Mines (DNRM), Planning Guidelines for Water Supply and Sewerage, March 2005;
- Sinclair Knight Merz, Banana Shire Council - Biloela Water Supply Planing Report (Rev 0), 2 May 2006;
- US EPA, Disinfection Benchmarking and Profiling Guidance Manual, EPA 815-R-99-013, US EPA Office of Water, August 1999.

11. Appendices

APPENDIX A –WATER QUALITY - EXTERNAL LABORATORY ANALYSIS RESULTS

General Analysis Results for WTP Raw Water

Parameter	Units	2002		2005	2006		2007			2008			2009	ADWG
		Dam 9/9	PS 9/9	PS 17/10	PS 24/1	PS 15/1	PS 5/2	Dam 30/4	PS 26/11	PS 19/2	PS 12/5	PS 4/8	PS 29/1	
Turbidity	NTU	<1	<1	1	4	5	1	1	35	1	<1	12	6	< 1
True Colour	HU	<1	<1	1	2	6	3	7	7	3	3	<1	6	< 15
pH		7.9	7.95	7.96	7.96	8.08	7.65	7.38	7.75	7.95	7.75	7.88	7.75	6.5 – 8.5
Conductivity	μs/cm	465	495	443	401	368	379	535	388	394	669	433	437	-
Total Diss Solids	mg/L	260	260	241	217	189	198	280	200	200	360	230	220	< 500
Total Diss Ions	mg/L	330	330	310	277	257	258	343	270	259	421	288	282	-
Total Hardness	mg/L as $CaCO_3$	135	140	139	124	111	111	155	111	112	203	128	125	60 – 200 preferred
Alkalinity	mg/L as $CaCO_3$	125	125	129	113	116	100	119	117	100	135	105	113	-
Silica	mg/L	4	5	10	9	3	1	11	2	2	22	7	8	-
Sodium	mg/L	45.5	45	33	31	28	31	43	31	32	50	33	34	< 180
Potassium	mg/L	2.8	2.9	3.2	3	3.5	3.8	3.1	3.8	4.1	2.6	4.0	3.8	-
Calcium	mg/L	28	28	33	29	25	24	34	23	24	45	30	27	-
Magnesium	mg/L	16.5	16.5	14	13	12	12	17	13	13	22	13	14	-
Hydrogen	mg/L	0	0	0	0	0	0	0	0	0	0	0	0	-
Bicarbonate	mg/L	150	150	156	136	140	122	145	141	121	164	127	137	-
Carbonate	mg/L	0.7	0.8	0.9	0.9	0.9	0.2	0.3	0.5	0.6	0.6	0.5	0.5	-
Hydroxide	mg/L	0	0	0	0	0	0	0	0	0	0	0	0	-
Chloride	mg/L	70	70	48	47	41	46	80	47	48	109	54	49	< 250
Fluoride	mg/L	0.2	0.2	0.2	0.2	0.17	0.13	0.15	0.15	0.13	0.12	0.14	0.15	< 1.5
Nitrate	mg/L	<0.5	<0.5	<0.5	<0.5	<0.5	0.5	2.2	<0.5	0.9	3.0	<0.5	<0.5	< 50
Sulphate	mg/L	20	20.5	22	18.4	6.7	18.7	18.1	9.6	14.7	25	27	15.2	< 250
Iron	mg/L	0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	< 0.3

Parameter	Units	2002		2005	2006		2007			2008			2009	ADWG
		Dam 9/9	PS 9/9	PS 17/10	PS 24/1	PS 15/1	PS 5/2	Dam 30/4	PS 26/11	PS 19/2	PS 12/5	PS 4/8	PS 29/1	
Manganese	mg/L	<0.03	<0.03	<0.03	<0.03	<0.03	<0.01	<0.01	0.02	<0.01	<0.01	0.01	<0.01	< 0.1
Zinc	mg/L	0.01	0.05	<0.01	<0.01	<0.01	0.01	0.13	<0.01	0.01	<0.01	<0.01	<0.01	< 3
Aluminium	mg/L	0.09	0.05	0.11	0.14	<0.05	0.19	<0.05	<0.05	0.10	<0.05	0.06	<0.05	< 0.2
Boron	mg/L	<0.02	0.02	0.05	0.05	<0.06	0.05	0.06	0.05	0.06	0.05	0.06	0.06	< 4
Copper	mg/L	<0.03	0.04	<0.03	<0.03	<0.03	<0.03	0.05	<0.03	<0.03	<0.03	<0.03	<0.03	< 1

General Analysis Results for WTP Treated Water

Parameter	Units	WTP 9/9/02	WTP 24/1/06	ADWG
Turbidity	NTU	1	1	< 1
True Colour	HU	<1	2	< 15
pH		7.9	7.9	6.5 – 8.5
Conductivity	µs/cm	495	399	-
Total Diss Solids	mg/L	260	214	< 500
Total Diss Ions	mg/L	330	273	-
pH @ 21C Saturated	Calc for $CaCO_3$	8.05	7.92	-
Total Hardness	mg/L as $CaCO_3$	140	122	60 – 200 for 'Good quality'
Alkalinity	mg/L as $CaCO_3$	125	110	-
Silica	mg/L	4	9	-
Sodium	mg/L	45.5	30	< 180

Parameter	Units	WTP 9/9/02	WTP 24/1/06	ADWG
Potassium	mg/L	2.8	3.2	-
Calcium	mg/L	28.5	28	-
Magnesium	mg/L	16.5	13	-
Hydrogen	mg/L	0	0	-
Bicarbonate	mg/L	150	133	-
Carbonate	mg/L	1	0.5	-
Hydroxide	mg/L	0	0	-
Chloride	mg/L	70	46	< 250
Fluoride	mg/L	0.2	0.1	< 1.5
Nitrate	mg/L	<0.5	0.5	< 50
Sulphate	mg/L	20	18	< 250
Iron	mg/L	<0.01	<0.01	< 0.3
Manganese	mg/L	<0.03	<0.03	< 0.1
Zinc	mg/L	0.03	0.23	< 3
Aluminium	mg/L	0.1	0.11	< 0.2
Boron	mg/L	<0.02	0.05	< 4
Copper	mg/L	<0.03	<0.03	< 1
Free Carbon Dioxide*	mg/L	2.2	2.6	-

*Values for free carbon dioxide levels were calculated by CWT based on alkalinity and pH levels.

General Analysis Biloela Town Pump Station

Parameter	Units	2002				2005	2006	2007				2008			2009	ADWG Values
		6/8	9/9	7/10	5/11	17/10	24/1	5/2	30/4	6/8	26/11	19/2	12/5	4/8	28/1	
Turbidity	NTU	<1	<1	<1	<1	<1	1	1	12	1	1	1	<1	1	1	< 1
True Colour	HU	1	2	<1	<1	<1	2	3	7	<1	6	2	3	<1	<1	< 15
pH		7.55	7.6	7.3	7.45	7.36	7.46	7.1	7.83	7.59	7.69	7.85	7.75	7.9	7.66	6.5 – 8.5
Conductivity	µs/cm	610	620	620	630	623	482	583	399	590	684	391	669	445	460	-
Total Diss Solids	mg/L	340	340	350	350	342	264	318	201	319	377	198	360	236	235	< 500
Total Hardness	mg/L as $CaCO_3$	190	185	180	185	195	148	177	113	180	208	112	203	132	130	60 – 200 'Good quality'
Alkalinity	mg/L as $CaCO_3$	135	135	135	135	138	115	123	102	129	143	99	135	103	100	-
Silica	mg/L	20	20	21	21	21	16	17	0	19	24	3	22	9	8	-
Sodium	mg/L	52	51	53	54	46	37	45	32	47	53	32	50	34	35	< 180
Potassium	mg/L	2.2	2	2	2	2.4	2.5	2.5	3.8	2.4	1.9	4.0	2.6	3.6	3.8	-
Calcium	mg/L	41	40.5	40	41	45	34	40	24	40	46	24	45	31	29	-
Magnesium	mg/L	21	20.5	20	21	20	15	19	13	19	23	12	22	13	14	-
Hydrogen	mg/L	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-
Bicarbonate	mg/L	165	160	165	165	168	140	149	123	156	173	120	164	125	121	-
Carbonate	mg/L	0.3	0.4	0.2	0.3	0.2	0.3	0.1	0.5	0.5	0.5	0.5	0.6	0.6	0.4	-
Hydroxide	mg/L	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-
Chloride	mg/L	99	98	100	100	95	69	95	47	92	121	48	109	57	56	< 250
Fluoride	mg/L	0.6	0.6	0.6	0.7	0.1	0.2	0.13	0.15	0.12	0.11	0.13	0.12	0.14	0.14	< 1.5
Nitrate	mg/L	1.9	1.9	1.7	1.9	3.3	0.6	2.1	0.5	1.8	2.9	1.1	3	0.5	0.5	< 50
Sulphate	mg/L	25.5	23.5	27.5	25.5	26	20	24	18.1	18.9	20	14.9	25	25	28	< 250
Iron	mg/L	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	0.02	<0.01	<0.01	<0.01	< 0.3
Manganese	mg/L	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	< 0.1
Zinc	mg/L	0.03	0.05	0.05	0.05	0.08	0.04	0.07	0.01	0.02	<0.01	<0.01	<0.01	<0.01	<0.01	< 3
Aluminium	mg/L	<0.05	<0.05	0.05	<0.05	<0.05	0.05	<0.05	0.13	0.05	<0.05	0.07	<0.05	<0.05	0.10	< 0.2
Boron	mg/L	0.11	<0.02	0.07	<0.02	0.06	0.05	0.05	0.06	0.05	0.05	0.06	0.05	0.06	0.07	< 4
Copper	mg/L	0.03	<0.03	0.03	0.03	<0.03	<0.03	0.04	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	< 1
Free carbon dioxide	mg/L	7.6	6.8	13.5	9.6	12.0	8.0	19.5	3.0	6.6	5.8	2.7	4.8	2.6	4.4	-

*Values for free carbon dioxide levels were calculated by CWT based on alkalinity and pH levels.