Moura WTP

Planning Report

June 2010

Banana Shire Council
Moura WTP Planning Report

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Overview

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

WTP Capacity and Forecast Water Demands

Moura WTP's original design flow rate was 70 – 75 L/s. The plant has been well proven at raw water flow rates of 52 to 62 L/s, but has only been tested at higher flow rates during an 8 hour test at 77 L/s.

Water demands on the WTP include demand from Moura and Banana townships and have been estimated for three development timeframes, with the corresponding WTP flow rate requirements calculated as shown in the table below.

WTP Demands and Flow Requirements Summary

Flow Parameter	Timeframe 1: Intermediate Development	Timeframe 2: Ultimate Development	Timeframe 3: Maximum Ultimate Development
WTP Sizing based on MDMM (ML/d)	3.40	4.27	4.68
Required WTP Raw Water Flow Rate (L/s):			
If 10% water losses through WTP process:	52.0	65.2	71.5
If 20% water losses through WTP process:	56.7	71.1	78.0

Required treated water production capacity calculated using MDMM basis and 20 hours operation per day, as detailed in section 2.4.2. Alternative basis for capacity calculation discussed in section 2.4.2.

As shown in table, the forecast 'intermediate' and 'ultimate' development demands are within the original WTP design capacity and are likely to be achieved without major upgrades to the existing WTP. The worst case 'maximum ultimate' development demand exceeds the current WTP design capacity and would require more significant upgrades to achieve the required output.

It is noted that permanent water restrictions may potentially reduce WTP demands for all timeframes, 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:

- Raw water issues include very high turbidity and colour from river flow events, periodic high manganese levels and the presence of herbicides;
- WTP treated water typically meets target values, with periodic excursions on treated water turbidity and true colour. The chlorine residual measured after the clear water tanks is highly variable and manganese targets are also sometimes exceeded;
- Total coliforms have been present in a number of treated water samples, but *E.Coli* has not been detected;
- From modelling of corrosivity potentials, the typical treated water is likely to be only mildly corrosive except under worst case conditions.

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 sized to achieve the design flow of 70-75 L/s although performance under these loadings has not been proven over a long term test. The most limiting process components are the raw water pontoon pumps, which can only supply 52 L/s to the WTP and Clarifier 1 which is generally operated well below its design rate due to poor settling at higher flows;
- Of the chemical dosing systems, the pre- and post-chlorine dosing systems would not achieve the maximum required doses at current flow rates without modifications. The alum pump (no longer used) would not achieve the maximum required doses at WTP design flow rates of 70 – 75 L/s;
- Plant control and automation, safety and maintenance issues were also reviewed, with various recommendations identified. Generally there is a need for more online monitoring and plant automation as well as some safety upgrades.

WTP Upgrade Requirements and Cost Estimates

Recommended WTP upgrade requirements have been identified and categorised into short, medium and long timeframe actions. All high and medium priority actions are listed in the tables below with estimated budget costs where appropriate. Further details of actions and costs are given in Sections 7 and 8 of this report.

Short Timeframe Upgrade Actions

Item	Estimated Cost
High Priority	
Online meters for turbidity, chlorine and pH	Council investigating
Backwash rates and process investigation, including testing of backwashing from treated water main	Council to carry out internally, with external assistance if required
Raw water pump – installation of third pump	Council investigating
Long term (≥ 1 week) trials of 65-75 L/s WTP flow rates	Council to carry out internally, with external assistance if required
Further trials of chlorine for oxidation of manganese. Investigate pre-chlorine dosing and optimisation of filtration	
Confirm dose rates and maximum pump capacity for Nalco coagulant and polyacrylamide	
Control system upgrades to prevent filter overflows	$5,000
New backwash pump (estimated duty approx 450 m^3/h at 5m head), if required after investigation	$50,000
Pre- and post-chlorine dosing system upgrade to achieve maximum dosing rates at ≥ 52 L/s	$ 5,000
Medium Priority	
Turbidimeters for individual filters	Council investigating
Relocation of lime and chlorine dosing points to weir box or pipeline after the two filtered water streams have combined	Council to carry out internally, with assistance if required
Provision of automatic feedback control for chlorine and pH	Council investigating

Item	Estimated Cost
Additional water quality analysis	Council investigating
Spare benchtop turbidimeter	Council investigating
Standby coagulant dosing pump	$ 5,000
Filter refurbishments – 4 x new flow meters, ultrasonic level sensors and DP cells	$ 40,000
SCADA programming to allow automatic initiation of backwash on trigger levels for time, headloss, filter high water level, and filtered water turbidity	$ 10,000 Plus cost of general backwash valve control repairs/ upgrades
Chlorine system upgrade to Aust Standards – Improve ventilation, add signs, add eyewash	$ 10,000

Medium to Long Timeframe Upgrade Actions

Item	Estimated Cost
High Priority	
Stage 1 filter investigation, media and underdrains refurbishment	$ 30,000 Plus additional costs if underdrains also need replacing
Upgrade of lime and alum systems – Tank platform railings. Supply and installation 2 x manual bag unloading cabinets. Supply and installation	 $ 2,000 $ 30,000
Medium Priority	
Flow meters calibration check - Inlet and outlet (consumption) flow meters	Recalibration undertaken regularly by Council
Clarifier 1 investigation of options to improve settling.	Council to carry out internally, with assistance if required
SCADA upgrades - Operator training, review of call-out arrangements, on call lap-top, Control system improvements and extra callout alarms	
Review of wastewater issues for current disposal method	
PAC dosing system, if required	$ 300,000
Flow meters upgrade: 3 new flow meters - On inlet pipe to the Stage 2 clarifier, backwash and desludge	$ 40,000
Clarifier 1 improvements: Automation of sludge drawoff from scour valve, if required Retrofit of settling tubes to increase settling rate, if required	$ 5,000 $ 180,000
Wastewater system – Appropriate type of wastewater treatment facilities, as required	$ 200,000
Upgrade of bunding, check and upgrade of all railings	$ 10,000
Improved laboratory and office facilities	$ 30,000

Actions designated as low priority have not been listed above, but are given in Section 7.5. It is noted that some of these issues may become higher priority if conditions or concerns change in future.

As an alternative to upgrading the existing WTP, the construction of a new WTP is expected to cost around $4 to 6 million.

Recommendations

It is recommended that all upgrade requirements identified in this report be addressed as practical. In particular, the following improvement works are considered very high priority and should be carried out urgently:

- Install online turbidity and chlorine monitoring;
- Initiate regular sampling/ online monitoring of filtered water turbidity;
- Update filter control system to prevent filter overflows;
- Test and establish system for backwashing using the treated water pumps. If unsuccessful, install a new/ refurbished standby backwash pump.

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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 Moura WTP and sets out options for addressing the upgrade requirements.

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

- Review treatment requirements based on raw water quality and treated water requirements;
- Review the capacity of each plant and each unit process, identifying capacity restraints or any available excess capacity;
- Identify issues and process upgrade requirements for current and future demand scenarios;
- Identify options to achieve the required upgrades and improvements.

2. Plant Flow Rate and Demand Issues

2.1 Annual Water Allocations and Use

Moura's water is sourced 100% from the Dawson River. Current allocation from the river is 800 ML/yr. Treated water is supplied to Moura and also pumped to the Banana township.

Actual annual raw and treated water usages determined from WTP data are shown below, based on readings from the raw and treated flow meters.

Annual Water Use

Year	Annual Raw Water Use (ML/yr)	Annual Treated Water Use (ML/yr)
2004	704	720
2005	-*	-*
2006	883	768
2007	723	620
2008	-*	575
2009	738	611

* Totals for 2005 and 2008 not shown due to incomplete data.

It is noted that the estimated annual total for the calendar year 2006 exceeded the allocation of 800 ML/yr, however it is understood that official allocations are based on financial year periods rather than calendar years.

The treated water totals for 2006, 07 and 09 are around 85% of the raw water inflows, reflecting 15% water losses through the treatment process if both meters were reading accurately.

2.2 WTP Flow Rates

2.2.1 Design and Actual Flow Rates

The Moura WTP was built in two stages:

- Stage 1, built in 1970, has a design capacity of 25 L/s;
- Stage 2, built in 1977, has a design capacity of 45 - 50 L/s;

The plant's total design capacity is therefore around 70 – 75 L/s, based on the original design rates.

The actual flow rates used in the process are outlined below:

- The plant is usually run at 52 L/s because of the limited capacity of the pontoon-mounted raw water pumps (Pumps 1a and 1b). See section 4.2 for further comments on the raw water capacity.
- The plant was run at 62 L/s for extended periods in the past (around 5 - 10 years ago) and also for most of 2008, using the larger deck-mounted pump (Pump 2), with process performance reported to be reasonable.
- A flow rate of around 77 L/s was reportedly run through the plant for an eight hour period when the QNP raw water pump was being commissioned (in 1999/ 2000).

The operators report that the WTP process was maintained during this period, although a longer flow rate trial would be required to prove performance at high flows.

It is noted that the design maximum rate of 70 - 75 L/s has not been tested over a long term period. **A higher plant flow rate should be trialled for at least a week, when pumping capacity, system demand and storage allow, to test whether the existing plant can successfully treat a flow of 65 – 75 L/s over the long term.** The estimated capacities of each individual unit process are discussed in section 4 of this report.

2.2.2 Plant Production and Flow Rate Data

Available plant data for plant meter readings, inflow rates and plant run times for the period 2004 to 2009 was analysed to work out the daily water inflow and outflow volumes.

The flow data available for the water treatment process is limited to two flow meters:

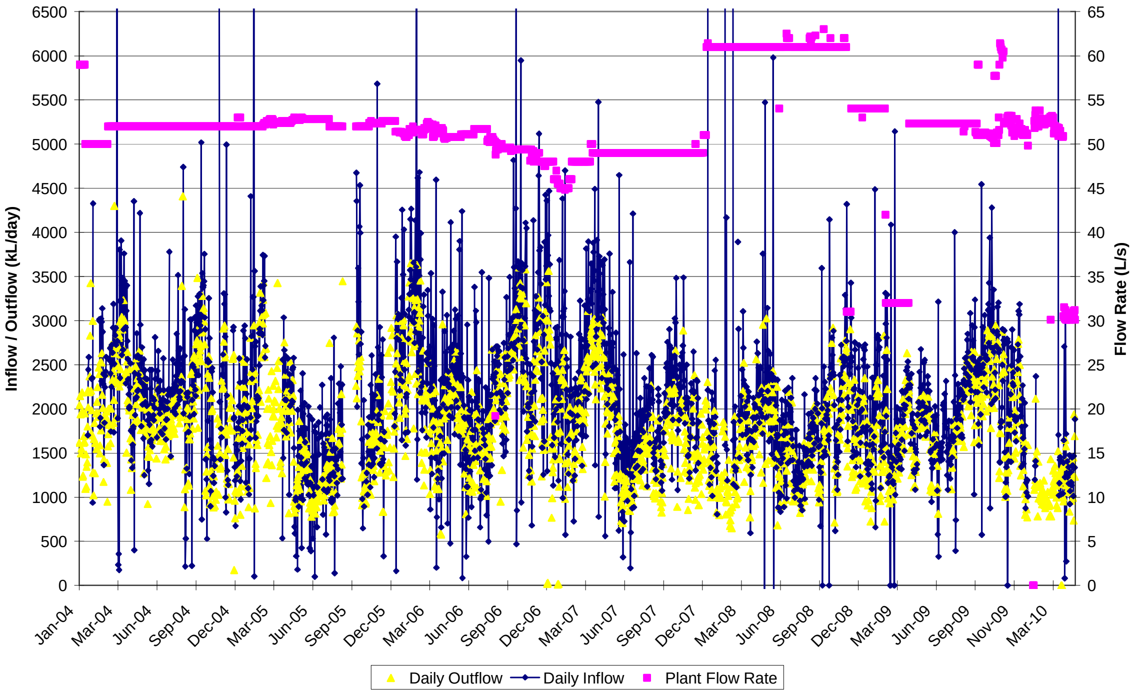
- The plant inlet flow meter – Measures flow into the plant through the raw water main;
- The 'consumption' (plant outlet) flow meter – Measures flow out of the plant through the rising main after the clear water tanks and pressure pumps.

The two flow meter measurements cannot be directly compared for any one day, because of the volume buffering capacity of the clear water tanks. The total volumes over a longer period, say a year, should however be comparable.

It was noted that the inlet flow meter appeared to be offline for a period during March 2005, and that data was missing for several other periods. The data was analysed taking this into account.

During data analysis, it was noted that the daily plant inflow volume calculated from the flow meter totaliser reading often did not match the equivalent volume calculated from the run time and flow rate records. The value based on the flow meter tended to be more erratic, and also to be slightly higher on average than the value based on the run time and flow rate. There are bound to be inaccuracies in the two different calculation methods, however the reasons for the more significant differences are not clear. **The calibration and accuracy of the inlet and outlet flow meters should be checked.**

The inflow and outflow data was used to prepare the following graph. The inflow data shown below is based on the meter totaliser readings. Daily inflow volumes were not available for periods during December 2007 to February 2008, understood to be due to the flow meter being inaccessible due to flooding.



Graph of Plant Flow Rate and Inlet and Outlet Flow Volumes

From the graph it can be seen that the plant flow rate has varied between 19 and 63 L/s, but has been mostly in the range 45 to 55 L/s. The achievable flow rate dropped at the end of 2006 and 2007 due to very low water levels at the river drawoff area. During early 2008, raw water was supplied by the largest deck mounted pump at around 63 L/s as the water level was high enough to allow this pump to be run without drawing in sediment or water rich in manganese. During early 2009 and early 2010, the raw water inlet flow dropped to 32 L/s when one pontoon mounted raw water pump failed and access for maintenance was restricted due to flooding.

The plant inflow and outflow have generally followed rough seasonal trends, with the lowest demand for the period seen in winter 2005. The highest demands for the period shown, peaking at around 3,500 kL/day, were in the summers of 2006 and 2007.

The data spikes of 5000 kL/ day and higher shown in the graph are suspected to be inaccuracies, since such high inflow volumes could only be produced with a flow rate of around 58 L/s (higher than the pontoon raw water pump capacity) over the full 24 hours. It is noted that the maximum daily inflow volume calculated based on run time and flow rate is 4500 ML/day (52 L/s x 24 hours).

Notable in the graph were a number of sudden spikes in the daily inlet volume, which were not associated with equivalent spikes in the outlet (‘consumption’) values. The operators advise that these spikes probably represent times when the plant filters have overflowed due to high headloss buildup, mostly overnight when the plant is unmanned, as there is no filter level callout alarm. At such times, because filter backwashes can only be manually triggered, the plant continues to operate with overflowing filters and reduced filtered water flow until the clear water tanks reach the ‘plant stop’ trigger level or an operator intervenes to start a backwash. Such occurrences lead to the waste of a significant amount of raw water and it is a concern that they appear to happen very regularly. **Upgrades to the callout alarms and/ or to filter operation are required urgently to stop the occurrence of these accidental filter overflows.** Filter upgrade requirements are listed in more detail in section 4.6 below.

The plant is programmed to automatically start and stop to keep the clear water tank volume within a certain range. The plant may therefore have various starts and stops over the day based on clear water tank levels. It is noted that the inlet flow volumes do not always match with the plant run hours recorded. This suggests that there is inaccuracy in the inlet flow meter and/or the plant hours meter records.

From logged plant data, the plant generally runs between 5 and 20 hours per day but has been run for 20 - 24 hours per day on several occasions. It appears that many of the occasions where the plant has operated for 20 or more hours in a day were not due to high system demand and are potentially related to plant shutdowns for cleaning/ maintenance (plant required to operate longer than normal to refill clear water tanks after shutdown), accidental filter overflows or excessive losses through the treatment system.

The operator reported that greater operator input was required when the plant was required to run for extended hours per day. It is expected that the Moura WTP could be run constantly for several days if required by high demand, provided that operators were available to closely monitor and optimise the treatment process and that there were no critical maintenance issues. The maximum practical long term daily plant run hours is expected to be around 20 hours per day, to allow for maintenance down time. This may still require more online monitoring, filter automation and more operator input than is currently available.

2.3 WTP Treated Water Production Capacity

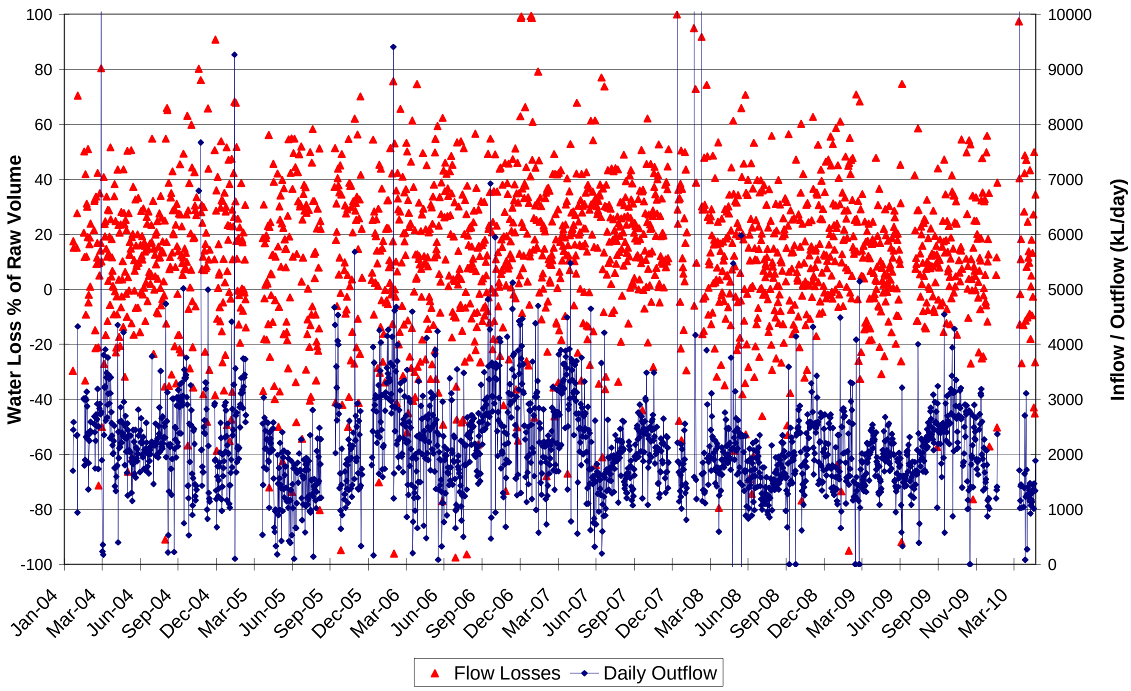
2.3.1 Flow Rate Variation Through WTP Process

The plant flow rate set point is the raw water inlet flow to the WTP. The actual flow rate will vary through the WTP process as water flow is removed as follows:

- ‘Plant flow rate’ measured on raw water flow meter at inlet to plant;
- Flow removed periodically from clarifiers during desludges;
- Flow removed periodically from clear water tank/ rising main during filter backwashing;
- Flow lost because of filter overflows etc.
- ‘Consumption’ flow measured after clear water tank and after treated water pumps

Because there is no recycling of the wastewater streams back to the head of the plant, the wastewater flows are permanently removed from production.

Plant data comparing the inflow and outflow volumes was used to prepare the following graph of water losses through the process. Data was not available for the period December 2007 to November 2008. The calculated ‘water loss’ (raw water minus treated water daily volume as a percentage of the raw water inflow) and the actual daily flow volumes are shown in the graph.



Graph of Water Losses Through Process

From the graph above it can be seen that the calculated 'water loss' varies greatly. The median water loss appears to be around 10 - 20% of the raw water volume entering the plant.

From other available flow data, it was calculated that the average daily inflow volume is around 1869 kL based on reported flow rate and plant run hours, or around 2206 kL based on totaliser readings (with the two methods of calculation varying significantly as noted above). The average daily outflow volume is around 1797 kL. Thus based on these calculations the outflow volume has been between 3 and 19% less than the raw water inflow on average.

Based on the information above, it is estimated that the treated water supplied to the town system is expected to be roughly 80 - 90% of the raw water flow under typical circumstances. Around 5 – 10% losses would normally be the target for a well operated typical conventional treatment plant. To get the most efficient use of the raw water drawn from the river, process losses at Moura WTP could potentially be reduced by preventing accidental filter overflows and by providing suitable wastewater holding and treatment units to allow recycle of treated wastewater streams back to the head of the plant.

It is difficult to identify what proportion of the losses is related to clarifier desludges, backwash water and accidental overflows or to confirm flow losses due to the lack of meters on the clarifier sludge and backwash lines. **Ideally, a flow meter should be provided on both the backwashing and the desludge lines to aid in flow loss assessment.**

2.3.2 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 current typical 20% loss (i.e. 80% of inlet flow converted to treated water); and

- Improved WTP 10% loss (90% of inlet flow converted to treated water) with wastewater recycling to the head of the works and prevention of overflows.

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	
	20% Water Loss	10% Water Loss
52	3.0	3.4
60	3.5	3.9
62	3.6	4.0
65	3.7	4.2
70	4.0	4.5
75	4.3	4.9
80	4.6	5.2
90	5.2	5.8
100	5.8	6.5

As seen in the table above, the current usual plant flow rate of 52 L/s can produce 3.0 to 3.4 ML/d in 20 hours operation. The deck pump flow rate of 62 L/s can produce 3.6 to 4.0 ML/d.

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

2.4 Current and Future Demand Scenarios

2.4.1 System Demand Studies

Based on plant data from the period 2004 – 2009 presented above, the historical demand based on the daily outflow volume has varied between 0.5 and 4.0 ML/day, with an average of around 2 ML/d.

Studies into the expected current and future demands from the Moura and Banana townships include:

- Ullman and Nolan P/L, February 1992, found an existing Moura population of 3008 EP (equivalent persons) with a maximum demand of 1800 L/EP/day giving a maximum daily demand of 5.4 ML/d;
- Cardno, January 2007, estimated current population water demand (Moura and Banana) to be around 3815 EP and the projected ultimate population water demand by the year 2026 to be up to 6000 EP.

The Cardno report identified several scenarios for development of Moura and Banana over the existing, intermediate and ultimate timescales. Cardno found that the ultimate extent of development may follow one of three scenarios:

- Nominal population growth;
- Nominal population growth plus 50 lot subdivision at Banana; or
- Nominal population growth plus 250 lot subdivision at Banana.

The middle development scenario of ‘nominal population growth plus 50 lot subdivision at Banana’ has been adopted for the purposes of this report to forecast likely demands for intermediate and ultimate development timeframes in Moura and Banana. Three development timeframes were investigated:

Timeframe 1: Intermediate development to 4362 EP;

Timeframe 2: Ultimate development to 5475 EP;

Timeframe 3: Maximum ultimate development to 6000 EP.

The instantaneous demand (in L/s) based on Cardno’s system modeling and the calculated daily demand (24 x instantaneous demand) associated with each development timeframe are outlined in the table below.

Water Demand Forecasts

Timeframe	Average Day, AD		Mean Day Max Month, MDMM		Peak Day, PD	
	L/s	ML/d	L/s	ML/d	L/s	ML/d
Timeframe 1: Intermediate	30.3	2.62	39.4	3.40	58	5.01
Timeframe 2: Ultimate	38	3.28	49.4	4.27	72.9	6.30
Timeframe 3: Maximum Ultimate	41.7	3.60	54.2	4.68	79.9	6.90

2.4.2 Plant Flow Rate Requirements for Future Demand Scenarios

For each of the future water demand scenarios identified earlier in this report, the table below was used to identify the nominal plant flow rates needed to meet the WTP sizing daily output. The flow rates are calculated allowing for process water losses of 10% and 20%.

Council use the Queensland DNRM rule of thumb 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). 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.

WTP Flow Requirements for Forecast Demands

Flow Parameter	Timeframe 1: Intermediate Development	Timeframe 2: Ultimate Development	Timeframe 3: Maximum Ultimate Development
WTP Output Sizing based on MDMM (ML/d)	3.40	4.27	4.68
Required WTP Raw Water Flow Rate (L/s):			
If 10% water losses:	52.0	65.2	71.5
If 20% water losses	56.7	71.1	78.0
***Alternative** WTP Sizing based on 2.0 x AD (ML/d)*	*5.24*	*6.57*	*7.21*
Required WTP Raw Water Flow Rate (L/s) if 10% losses	*80.0*	*100.3*	*110.1*

As seen in the table above, the intermediate timeframe MDMM demands could be met by the current proven WTP flow rates of up to 62 L/s. Ultimate development MDMM demands require flowrates higher than those currently proven but still within the original WTP design flow rate. The maximum ultimate timeframe demands would exceed the design flow rate of the plant. For the alternative sizing approach, the demand for all timeframes would exceed the design flow rate of the plant.

3. Water Quality Issues

3.1 River Source and Catchment Land Uses

Moura WTP draws water from the Dawson River, which also feeds the Theodore and Baralaba WTPs. The Moura WTP raw water draw-off is at an artificial impoundment between the Paranui weir and the weir at the Moura golf course.

The Dawson River catchment contains industries such as agriculture (cotton, sorghum, cattle) and mining.

The water-borne protozoan parasites *Giardia* and *Cryptosporidium* may be present in the faeces of cattle and other livestock. Because of the surrounding agricultural areas, the raw water is likely to contain some of these protozoan parasites in addition to pesticides/ herbicides.

The river source is usually high in turbidity due to suspended sediment, especially in times of fast flow. During low rainfall conditions the river stops flowing and can become stagnant, reportedly associated with high levels of iron and manganese in the raw water which may be associated with stratification in the raw water body. During low river flow periods, blue green algae and taste and odour problems may also be problematic.

3.2 Water Quality Monitoring Data

Raw and treated water quality is monitored daily at the WTP. Data for the period 2004 to 2010 graphed below has been mainly taken from the WTP log sheets where it is regularly recorded in electronic format.

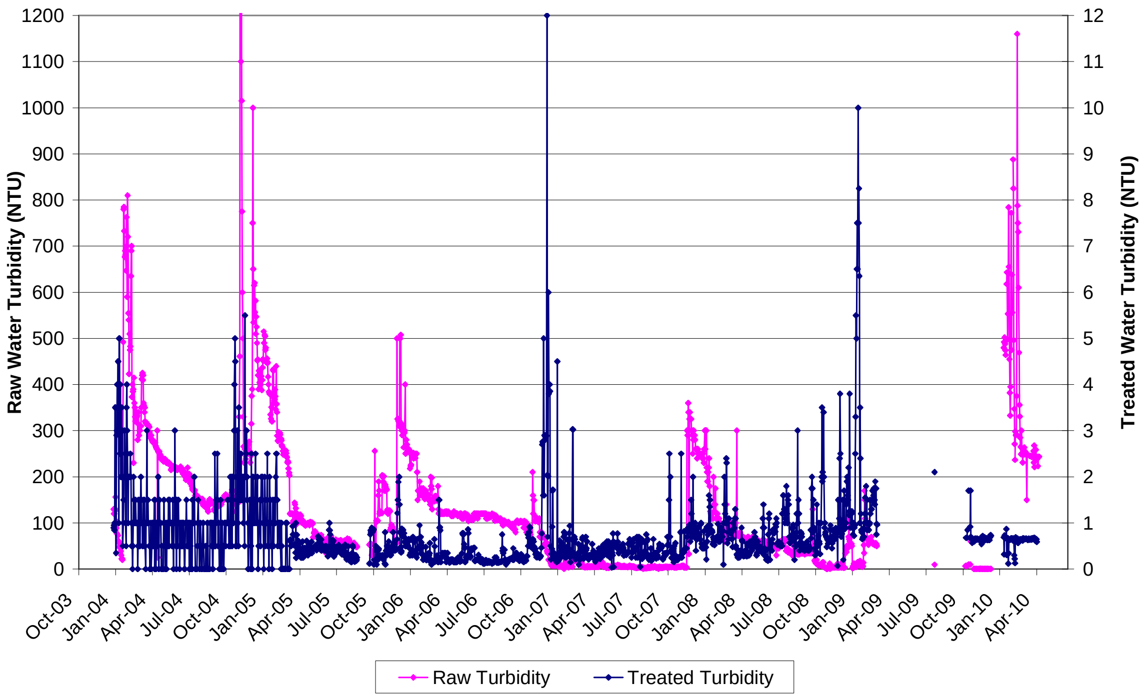
Raw and treated water samples have also been taken for external laboratory analyses, and the available data from this source has been summarised in Appendix A of this report.

Water quality results are discussed below for each relevant water quality parameter

3.3 WTP Raw and Treated Water Quality

3.3.1 Turbidity

The graph below shows the raw and filtered water turbidity for the period 2004 to 2010, based on WTP log sheets. It is understood that the raw and treated water turbidity was not measured between March and October 2009 due to instrument failure. Raw and particularly treated water turbidity is an important parameter and ideally **a spare turbidity analyser should be purchased and kept available so that regular turbidity measurements can be continued if the duty analyser fails.**



Graph of Raw and Treated Water Turbidity

A seen in the graph, the raw water turbidity normally varies seasonally, with turbidity peaks of 300 - 1000 NTU or more seen during most of the summer (rainy) seasons. Turbidity is generally < 300 NTU and more stable during the winter (dry) season. The turbidity peaks in summer are associated with high river flow conditions. This trend was less obvious during 2007 and 2009 when flow in the river was unusually low due to drought conditions. The raw water turbidity dropped to very low levels (below 10 NTU) during 2007, before a rainy period in January 2008 filled the river again.

The highest raw water turbidity reported in the daily log sheets was >2300 NTU in November 2004. Data from external laboratory analyses showed a peak of 485 NTU in January 2006.It is noted that the significant and sometimes rapid changes in raw water turbidity will require the operator to be vigilant in re-optimising coagulant doses at the WTP.

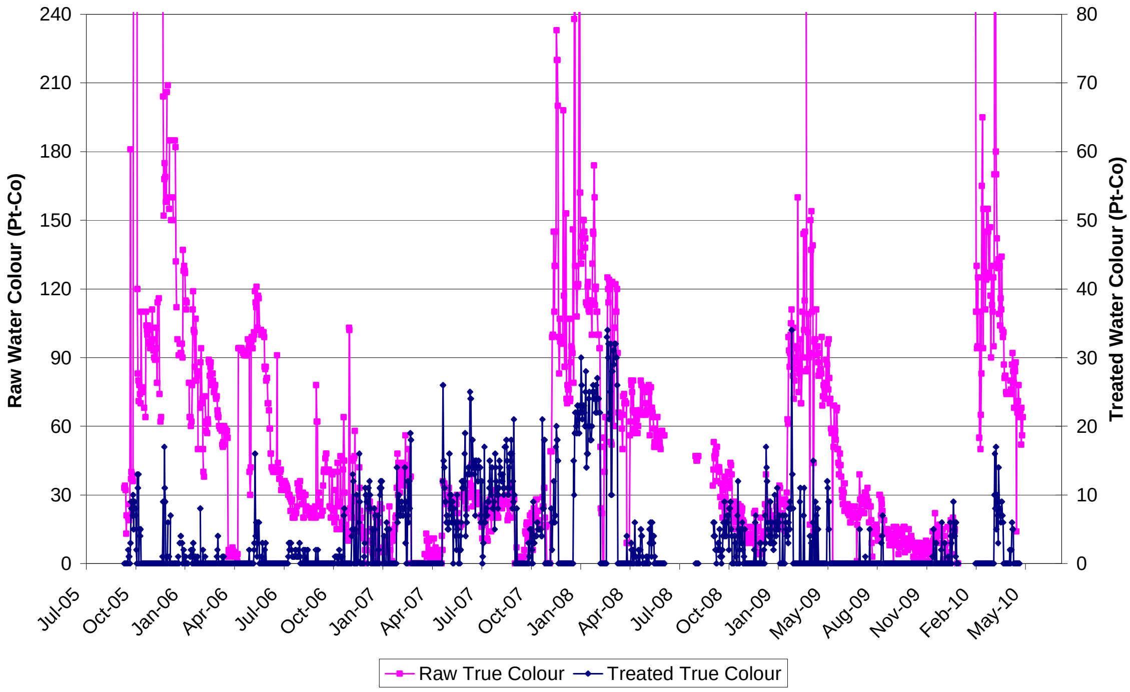
As also seen in the graph, the treated water turbidity is typically around 1 NTU however values are often >1 NTU, and some readings have been >5 NTU. The highest treated water turbidity level logged by the operators was 12 NTU in December 2006. The external laboratory analyses (see Appendix A) also mainly showed treated water turbidities of normally 1 NTU or less, although values up to 5 NTU were measured on some occasions and a single very high value of 33 NTU was logged for 19/8/02.

High treated water turbidities can result from high raw water turbidity, poor coagulation and/or poor filter performance. Elevated levels are a concern as they may indicate pathogens or contaminants are passing into the treated water, although another reason for elevated treated water turbidities is the effect of impurities in the lime dosed for post-filtration pH correction. As post-lime is regularly dosed at Moura WTP, this may have contributed to some of the high turbidities and it is noted that the treated water turbidity was generally lower after March 2005, when Nalco coagulant was introduced as the main coagulant leading to lower post-lime doses. The exact effect of the lime impurities is unknown. **Regular grab sample or online monitoring of the filtered water turbidity at a suitable point upstream of lime addition should be initiated to give more effective feedback on the performance of the filters.**

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 current ADWGs give a higher guideline level of <5 NTU as a limit for aesthetic purposes, as long as adequate disinfection can be achieved, however it is understood that the next revision of the ADWG is likely to recommend significantly lower turbidity levels. In the USA, the EPA approach to minimising the risk of *Cryptosporidium* contamination in drinking water is to specify that WTPs incorporating a conventional or direct filtration process should achieve < 0.3 NTU in 95% of measurements in the combined filter effluent stream and that the turbidity of this stream should never exceed 1 NTU. Most modern WTPs in Australia aim for a turbidity of <0.3 NTU or lower for best-case disinfection conditions.

3.3.2 Colour

Plant data from 2005 to 2010 was used to prepare the following graph of raw and treated true water colour. The measurement of 'true colour' requires the filtration of the sample to remove turbidity before colour analysis. This filtration step has only been performed since October 2005, when a suitable filtration flask was obtained for the WTP laboratory. It is understood that true colour is currently measured once each day by the plant operators.



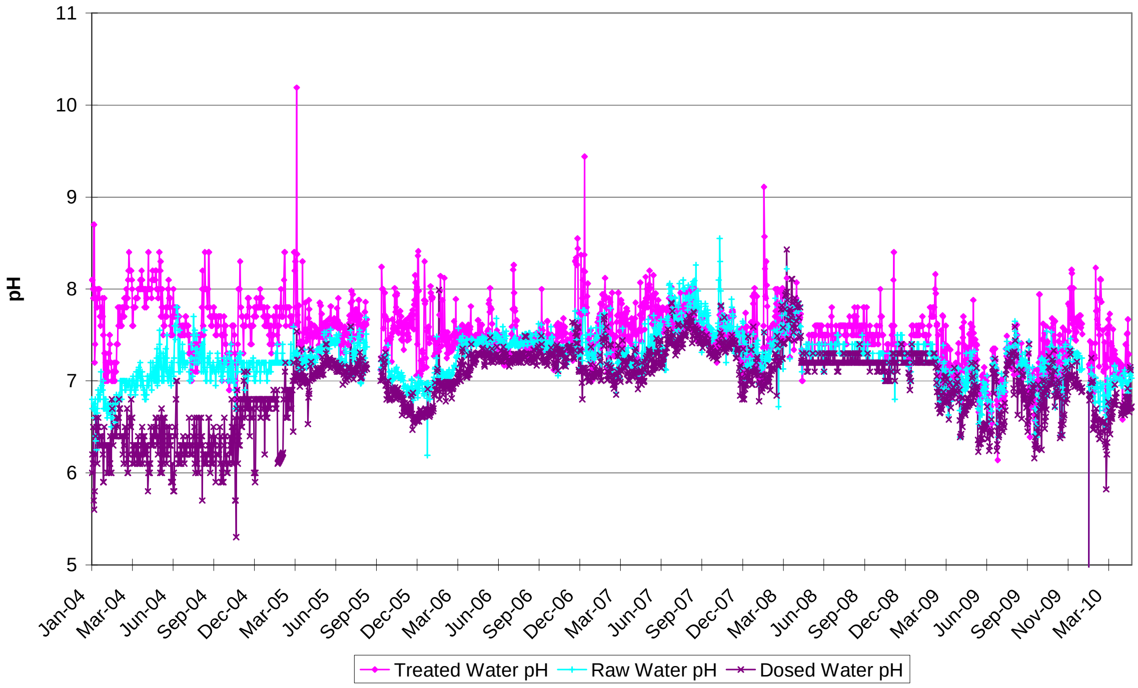
Graph of Raw and Treated Water True Colour

As seen in the graph, raw water colour has varied significantly over the period shown. Raw water true colour measurements between 0 and 480 Pt-Co have been reported. There is a rough correlation between true colour levels and the seasonal spikes of turbidity when the river is flowing fastest.

Treated water true colour has varied from 0 to 34 Pt-Co and has generally been less than 20 Pt-Co. The treated water colour has exceeded the ADWG recommended level of <15 Pt-Co at various times, particularly during 2006 - 2008. Higher treated water colour levels may indicate less than optimum coagulation. In particular, coagulation seems to have been poor during the unusually low raw turbidity and colour levels from December 2006 through 2007, conditions which can make water difficult to treat. Treated water true colour levels are not available for the period before March 2005 for comparison of the performance of alum with the Nalco coagulant.

3.3.3 pH and Alkalinity

Plant data from 2004 to 2010 is expanded in the following graph of pH levels.



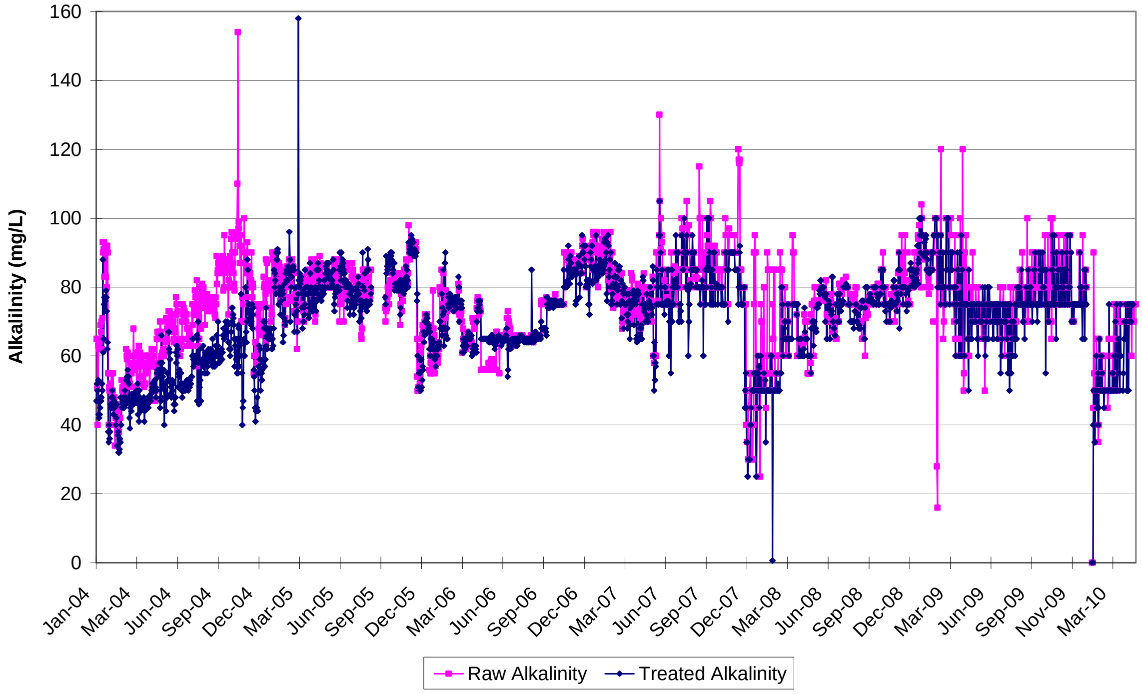
Graph of Raw, Dosed and Treated Water pH

As shown in the graph, the raw water pH is relatively constant, with measurements recorded between 6.1 and 8.6.

The dosed water pH has varied between 5.3 and 8.4 for the period shown. It is noted that alum was used as the main coagulant until around March 2005, after which the new Nalco proprietary coagulant was used. This is reflected in the trend, which shows a lower coagulation pH for alum compared to the Nalco coagulant.

The treated water pH has remained relatively stable at most times, commonly varying between 6.8 and 8.5. The typical pH achieved in the final water is around 7.6, which is within the current target range. Three treated water pH spikes were noted, where the treated water pH rose above 9. The treated water pH parameter is controlled by the operators manually adjusting the post-lime dose to achieve a suitable pH. The high pH spikes were reportedly caused by equipment failures, generally occurring overnight when the plant is unmanned but high levels could also be caused by operator error. Ideally, **an on-line pH analyser with associated callout alarms should be provided to give warning when the treated water pH goes outside the target range.**

Plant data from 2004 to 2010 was used to prepare the following graph of alkalinity levels.



Graph of Raw and Treated Water Alkalinity

The raw water alkalinity has varied from 16 mg/L to 154 mg/L. It is noted that recognisable drops in the alkalinity trend are associated with the seasonal river flood events, as at these times there is an influx of low alkalinity rain water into the river.

The treated water alkalinity level has varied, roughly following the trend of the raw water alkalinity levels. It is noted that during the period when alum was dosed, more of the raw water alkalinity was consumed by the coagulation reaction, compared to the period when Nalco coagulant dosing was used after March 2005.

3.3.4 Manganese

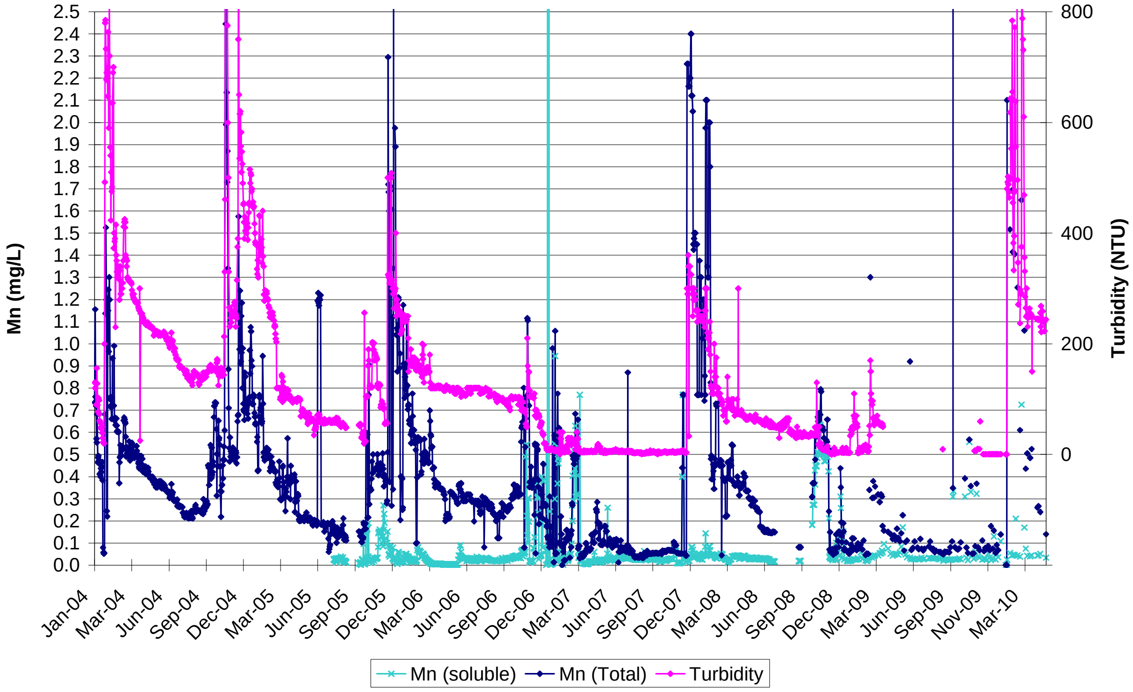
Manganese is an issue which affects many water treatment plants within Australia. In potable water it can lead to discoloured water (very dark brown to black) and staining of laundry and plumbing fixtures. Very high levels of manganese, often in conjunction with iron, will impart a taste to the water which is described as metallic, astringent, or medicinal. Manganese also interferes with the commonly used DPD method for determining chlorine residual which can lead to readings that indicate satisfactory chlorine residual when chlorine levels may be lower.

Manganese can build up on pipe walls in conjunction with various micro-organisms, with periodic sloughing off leading to dirty water problems. In some cases the onset of manganese problems in a system may be experienced only after several years of operation because of the manganese storage potential in the reticulation system.

Manganese is commonly found in soils and silt in the insoluble form as manganese oxide. Conditions such as low oxygen, acidic water or high carbon dioxide levels lead to the reduction of the manganese oxide into its soluble form.

Manganese treatment water chemistry is relatively complex and influenced by many factors. The nature of the water being treated can have a significant effect on the oxidant demand of the water and the type of manganese present. Manganese in water can be present both in particulate and soluble (dissolved) forms, depending on its chemical oxidation or reduction state. It can also be bound up with organics in the water as complex compounds. For effective treatment of manganese, it is critical to know the levels of both particulate (oxidised) and soluble (reduced) manganese, which together give the total manganese reading. Particulate manganese will generally be removed by coagulation and physical removal (settling/ filtration), while removal of soluble manganese requires an additional step of oxidation by chemical dosing or aeration to convert it to particulate form. The chemical doses or other treatment required to oxidise manganese in water depends on various factors, including the form of manganese present and the presence of organics and other species which will react with the oxidant.

Plant data from January 2004 to early 2010 was used to prepare the following graph of raw water manganese levels.



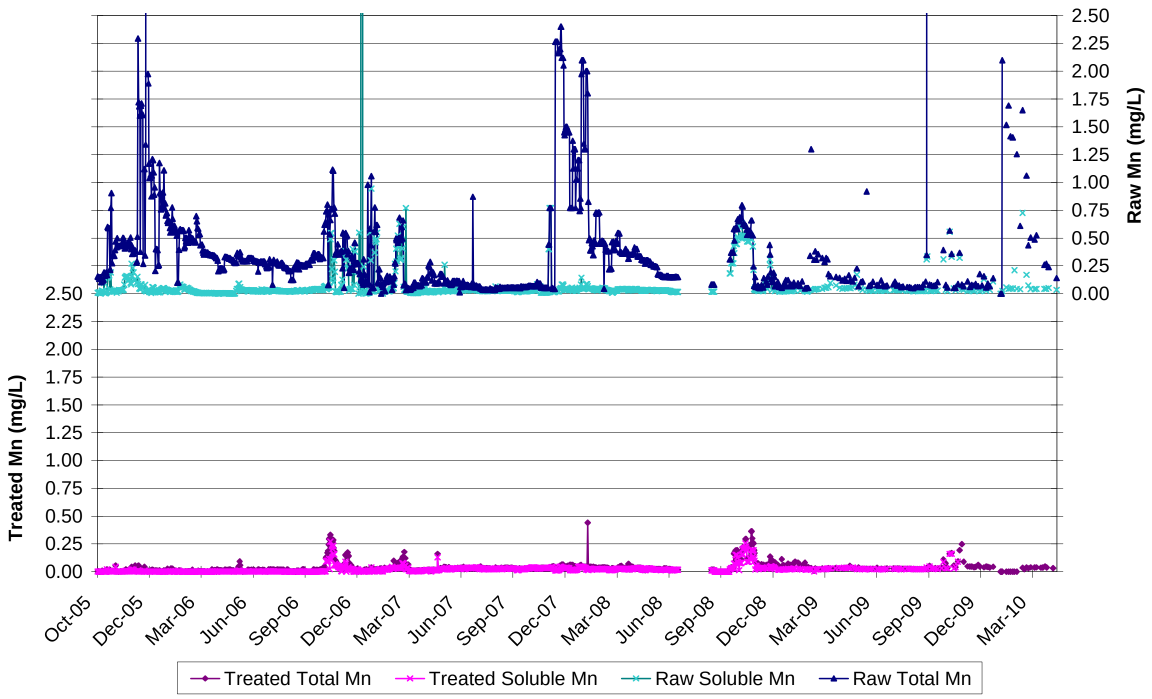
Graph of Raw Water Manganese

As seen in the graph above, total manganese is present in the raw water at Moura at quite high levels, periodically above 1 mg/L. Levels up to 9 mg/L total manganese have been reported. Total manganese spikes have tended to coincide with high turbidities, showing that much of the manganese is probably associated with sediments either washed into the river or resuspended by turbulent flow.

From the soluble manganese data available, it would appear that soluble manganese is normally a small proportion of the total manganese. However in particular during the low river conditions in December 2006 – February 2007 and December 2008, the soluble manganese did make up a significant proportion of the total manganese. Soluble manganese levels up to 0.95 mg/L were reported in January 2007. The high soluble manganese episodes coincided with very low river levels, when the water was being drawn from the lowest levels in the impoundment by the pontoon pumps and/ or the portable sled pump. It is noted from the trend that the soluble manganese level was highly variable during this period, which would potentially prove a challenge in terms of optimising chemical dosing for manganese oxidation.

Raw water total and soluble manganese levels should continue to be measured at least daily at the WTP. If high manganese levels are detected, more intensive monitoring (several times per day) could be undertaken.

Plant data from 2004 to 2010 was used to prepare the following graph of treated water manganese, with raw water manganese levels included for reference.



Graph of Raw and Treated Water Manganese Levels

It can be seen from the available data that the plant process generally has reduced both the soluble and the total manganese to significantly lower levels in the treated water. As shown in the graph, the treated water total manganese level is typically <0.02 mg/L, but has peaked a few times over 0.25 mg/L, particularly in December 2006 and 2008 when there were high levels of soluble manganese in the water. Most of the total manganese in treated water has been present as soluble manganese.

The higher treated water manganese levels have coincided with higher raw water soluble manganese levels, which is to be expected as soluble manganese is more difficult to remove than particulate manganese. Where soluble manganese has been removed by the WTP process, it is not clear whether this oxidation effect is caused by general aeration, pre-chlorine dosing or a combination of these processes.

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. Manganese can also accumulate in the reticulation system to emerge from taps later causing more complaints.

The data available to date suggests that the manganese passing through the plant process into the treated water is mainly unoxidised soluble manganese with a smaller proportion of manganese particles and. Achieving better manganese removal performance would therefore require optimisation of both the oxidation and the particle removal processes.

It is noted that a trial of pre-coagulation chlorine dosing to the raw water main (at the tip site) was carried out in 2007 but was inconclusive. **Further trials of manganese oxidation dosing, in conjunction with optimised particle removal in the filters, should be carried out in order to address the seasonal manganese treatment issues.**

3.3.5 Pesticides and Herbicides

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

Pesticide and Herbicide Analysis Results for Raw Water

Parameter	Units	23/10/02	[Date not reported]	River 11/07/06	WTP 28/11/06	Weir 16/5/07	Weir 12/11/07	Weir 28/1/09
OC Pesticides	µg/L	<0.1	<0.1	<0.3	<0.3	<0.3	<0.3	<0.3
OP Pesticides	µg/L	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Herbicide Atrazine	µg/L	**0.07**	**0.06**	**0.33**	**0.79**	**0.26**	**0.07**	**0.05**
Herbicide Desethyl Atrazine	-	-	-	**0.05**	**0.06**	**0.03**	<0.01	<0.01
Herbicide Desisopropyl Atrazine	-	-	-	**0.02**	**0.01**	**0.02**	<0.01	<0.01
Herbicide Diuron	µg/L	-	**0.01**	**0.02**	**0.03**	**0.29**	<0.01	<0.01
Herbicide Hexazinone	µg/L	-	<0.1	<0.01	**0.08**	**0.02**	<0.01	**0.01**
Herbicide Fluometuron	µg/L	-	**0.01**	**0.02**	**0.61**	**0.10**	<0.01	**0.02**
Herbicide Simazine	µg/L	-	-	<0.01	<0.01	<0.01	**0.02**	**0.01**
Herbicide Tebuthiuron	µg/L	-	-	**0.06**	**2.2**	**1.2**	**0.34**	**0.06**
Herbicide Metolachlor	µg/L	-	-	<0.01	**0.4**	**0.02**	<0.01	<0.01
Herbicide Prometryn	µg/L	-	-	**0.02**	**0.02**	**0.04**	<0.01	<0.01
Subs Urea Herbicides	µg/L	<0.01	-	-	-	-	-	-

As seen in the table above, pesticides were below the detectable levels in the samples taken, however a number of herbicides were found to be present.

Treated water herbicide and pesticide analyses available are shown in the table below.

Pesticide and Herbicide Analysis Results for Treated Water

Parameter	Units	ADWG Value	Health Value	Treated Water [Date not reported]
OC Pesticides	µg/L	-	-	<0.10
OP Pesticides	µg/L	-	-	<0.10
Herbicide Atrazine	µg/L	0.1	40	**0.06**

Parameter	Units	ADWG Value	Health Value	Treated Water [Date not reported]
Herbicide Diuron	µg/L	-	30	<0.01
Herbicide Hexazinone	µg/L	2	300	<0.10
Herbicide Fluometuron	µg/L	-	50	<0.01

As seen in the table, the herbicide Atrazine was detected in the treated water at levels less than the ADWG recommended level.

The operation and/ or design of the WTP treatment system should be improved with the aim of effectively removing pesticides and herbicides from the water. PAC dosing or ozone/ BAC are commonly employed treatment systems.

As herbicides have been detected, **pesticides and herbicides should continue to be monitored regularly in the raw water and these contaminants should be considered in any incident management procedures developed for raw water contamination events.**

3.3.6 Algae and Algal Toxins

Blue green algal blooms have been reported to occur in the river from time to time, especially when the river level is low and the water becomes stagnant. The WTP operators conduct regular visual checks of algal levels at the river, however no analysis data was available on algae types or counts for the Moura weir. It is understood that some algal monitoring is undertaken by Sunwater. **Council should ideally formalise a system for reviewing the algal analyses available from Sunwater.**

The operators report that tastes and odours can be a problem at the plant at times, however there is no data available to show which taste and odour compounds were present.

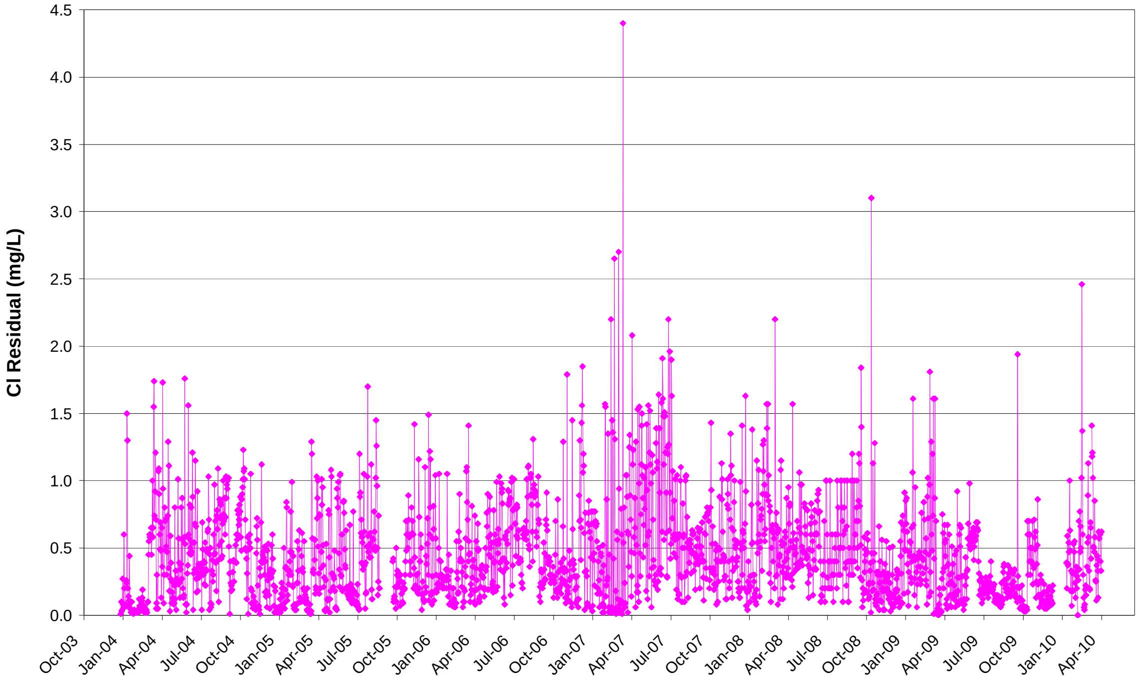
Algae and taste and odour compound analyses should be carried out when these contaminants are detected in the river water.

3.3.7 Aluminium

No plant data is available for treated water aluminium; however the treated water laboratory analyses tabulated in Appendix A show final water aluminium levels consistently below 0.05 mg/L.

3.3.8 Chlorine Residual

Plant data from 2004 to 2010 was used to prepare the following graph of treated water chlorine. The treated water chlorine level is controlled by the operators manually adjusting the post-chlorine dose. Measurement of the chlorine residual is carried out on the treated water, after it has passed through the clear water tanks.



Graph of Treated Chlorine Residual Levels

Chlorine residuals between 0.0 and 4.4 mg/L have been recorded in daily grab sample monitoring. A value of around 0.5 mg/L is achieved on average, however there have been significant daily variations.

An on-line chlorine analyser with associated callout alarms should be provided to give warning when the treated water chlorine residual goes outside the target range. Automatic chlorine dose adjustment for residual correction would also be possible if an on-line chlorine residual analyser were to be installed.

3.3.9 Microbiological Parameters

Treated water microbiological analyses available are shown in the table below.

Location	Date	Pass?	Parameters			
			Coliforms MPN per 100mL	E.Coli MPN per 100mL	HCC CFU per mL	Pseudomonas Aeruginosa CFU per 100mL
Moura Hospital	27/09/05	Y	>200	ND	-	-
	02/11/05	Y	9	ND	-	-
	29/11/05	Y	ND	ND	-	-
	10/01/06	Y	ND	ND	-	-
	03/03/08	Y	ND	ND	-	-
	31/03/08	Y	ND	ND	-	-
	28/04/08	Y	ND	ND	-	-
	26/05/08	Y	ND	ND	-	-

Location	Date	Pass?	Parameters			
			Coliforms MPN per 100mL	E.Coli MPN per 100mL	HCC CFU per mL	Pseudomonas Aeruginosa CFU per 100mL
	23/06/08	Y	ND	ND	-	-
	21/07/08	Y	ND	ND	-	-
	18/08/08	Y	ND	ND	-	-
	29/09/08	Y	ND	ND	-	-
	18/11/08	Y	ND	ND	-	-
	16/12/08	Y	20	ND	-	-
Moura Park	27/09/05	Y	ND	ND	-	-
Moura WTP	27/09/05	Y	59	ND	-	-
	02/11/05	Y	9	ND	-	-
	29/11/05	Y	ND	ND	-	-
	10/01/06	Y	ND	ND	-	-
	03/03/08	Y	ND	ND	-	-
	31/03/08	Y	ND	ND	-	-
	28/04/08	Y	ND	ND	-	-
	26/05/08	Y	ND	ND	-	-
	23/06/08	Y	ND	ND	-	-
	21/07/08	Y	ND	ND	-	-
	18/08/08	Y	ND	ND	-	-
	29/09/08	Y	ND	ND	-	-
	18/11/08	Y	ND	ND	-	-
	16/12/08	Y	ND	ND	-	-
Y=Yes, N=No, ND=not detected, HCC=Heterotrophic colony counts, MPN=Most Probable Number						

The above data indicates that total coliforms were present in samples taken at Moura Hospital and Moura WTP. In September 2005 high levels of total coliforms were detected in the distribution system samples and in November 2005 low levels were detected. Neither *E.Coli* nor Pseudomonas Aeruginosa were detected in any of the samples.

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.

3.3.10 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 - >2000	Summer >200; Winter 50 - 150	0 - 12	0.6 (for period with Nalco coagulant)
True Colour	Pt-Co	0 - 480	10 - 100	0 - 34	<10
pH	-	6.1 – 8.6	7.3	6.8 – 8.5	7.6
Manganese (total)	mg/L	0.012 – 9.2	0.400	0 – 0.440	<0.04
Manganese (soluble)	mg/L	0.001 – 0.95	0.04	0 – 0.272	<0.03
Aluminium	mg/L	<0.05 – 0.22	0.05	<0.05	<0.05
Alkalinity	mg/L $CaCO_3$	16 - 154	75	25 - 158	70
Chlorine	mg/L	-	-	0 – 4.4	0.5
Total coliforms		Not tested		Detected in some samples	
E.coli	MPN/100ml	Not tested		Not detected	
Pesticides	-	Not detected		Not detected	
Herbicides	-	Detected		Detected	
Blue Green Algae	-	Blooms known to occur		Blooms known to occur	
B-G Algal Toxins	-	Not tested		Not tested	
Taste and odour compounds	-	Reported to occur		Reported to occur	

3.4 Water Corrosivity Issues

3.4.1 Problems Caused By 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.

3.4.2 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);
- the Langelier Index; and
- the Larson Index.

The CCPP and Langelier Index are indicators of whether 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.

The Larson Index is an indicator of the potential aggressiveness of water in relation to the effect on oxide film formation on metals such as iron or mild steel. The Larson index is calculated as the ratio of chlorides and sulphides to alkalinity with all levels expressed in equivalents per million.

3.4.3 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
Larson Index	ratio	< 0.8	< 1.2

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.

3.4.4 Moura Corrosion Indicators

Corrosivity indices were modelled for Moura WTP raw and treated water and are set out in the table below with the data used as input to calculations included.

Corrosivity Indices for Moura Raw and Treated Water

Parameter	Units	Raw water Typical	Treated Water Typical	Treated Water Worst Case
Temperature	^{o} C	22	22	22
TDS	mg/L	152	164	217
Alkalinity	mg/L as $CaCO_3$	75	70	25
Calcium hardness	mg/L as $CaCO_3$	43	51	37.5
pH	-	7.3	7.6	6.8
Chloride	mg/L	30	35	57
Sulphate	mg/L	3.5	3.5	68
CCPP	**mg/L**	**-13.8**	**- 5.6**	**-18.4**
LI	**-**	**- 0.7**	**- 0.5**	**- 1.5**
Larson Index	**-**	**0.6**	**0.8**	**4.4**

The results of the modelling show that the raw water is likely to be corrosive in terms of CCPP and LI, but not in terms of the Larson Index.

The results for the typical treated water show that the water is just outside the recommended range for CCPP and LI but within the recommended range for the Larson Index. The water may therefore tend to be only mildly corrosive.

The worst case treated water quality is very likely to be corrosive, however it is noted that this type of quality would occur rarely and for short periods of time.

3.4.5 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 Moura WTP.

Treated Water Quality Targets & Guideline Values

Parameter	Units	ADWG		Common Industry Treated Water Targets	Current Moura 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	-
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.

4. WTP Process Description and Capacities

4.1 Process Overview

Moura WTP 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.

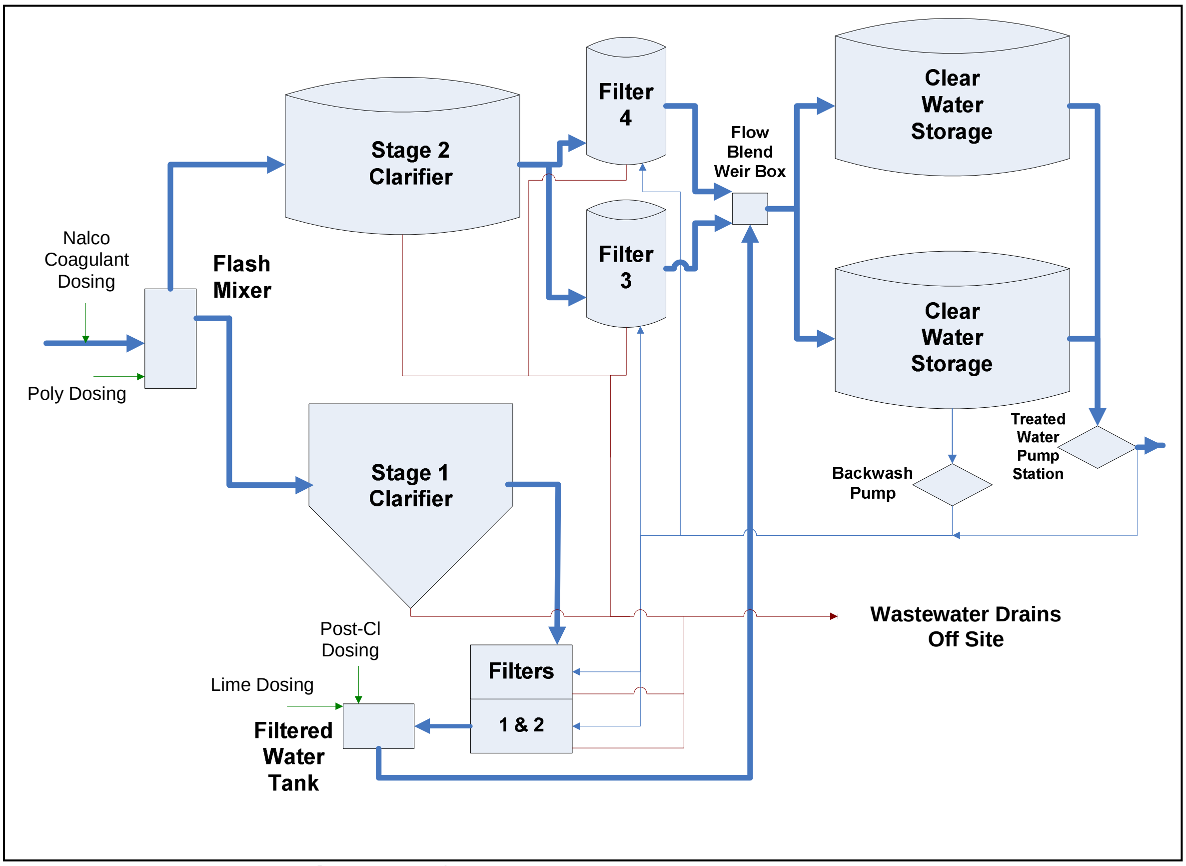


Diagram of Moura WTP Treatment Process

4.2 Raw Water Pumps and Plant Inlet

Raw water is drawn from the Moura Weir impoundment (Crest 104.6m AHD) at AMTD 155 km on the right bank by the raw water pumps. The artificial 7 – 10 km long impoundment lies between the Paranui weir and the weir at the Moura golf course and is around 200m wide and normally 10m deep.

There are two sets of raw water pumps for the WTP. The two new pumps are mounted on a floating pontoon and the two old pumps (which are retained as emergency backups) are mounted on a fixed deck.

The raw water is pumped from the river to the plant via a 250mm diameter asbestos cement (AC) rising main to the water treatment plant. The raw water pump operation is controlled by a float switch in the Clear Water Tank No. 1 at the WTP.

The main parameters of the raw water pumps and plant inlet are given in the table below.

Raw Water Pumps and Plant Inlet

Component	Parameter (Units)	Design Criteria	Comments
Dawson River Pumps	Intake type	Floating draw-off	
	No. and Capacity each (L/s)	New pumps 1a and 1b mounted on floating pontoon: 2 (duty/ standby) x 27 - 28 L/s each Old pumps 2 and 3 on decking: 1 x 60 L/s, 1 x 21 L/s	Combined capacity pumps 1a and 1b approx. 52 L/s Duty pumps are linked to level in WTP's Clear Water Tank No.1
	Raw water mains diameter, material, length	250 mm diameter, AC, 5.3 km	
Plant Inlet	Max hydraulic capacity	77 L/s	As tested during commissioning of QNP pumps

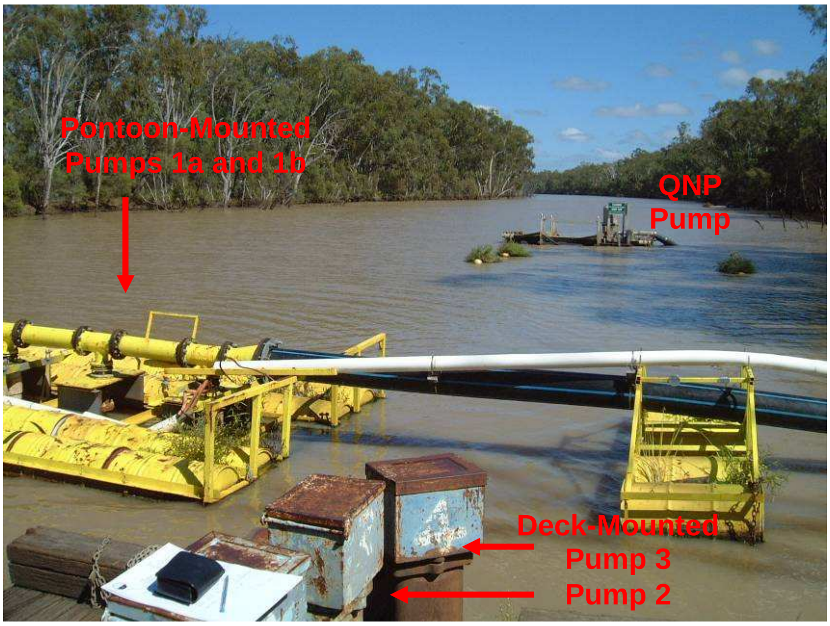


Photo of Floating Drawoff at River

The history of the raw water pumps includes:

- The decking pumps were originally used, however these were unable to be used when the water level dropped in the weir;
- The pontoon was installed in the 1990s, initially with a larger 62 L/s pump;
- After problems with the operation of the 62 L/s pump, it was replaced with the two existing raw water pumps 1a and 1b on the pontoon.

The existing duty raw water pumps are capable of a combined flow of approximately 52 L/s when the river is full. The provision of a third pump as an installed standby is planned by Council, however the installation has been delayed for several years due to re-design to address concerns about vortexing. The third pump could also be used as a third duty pump to give a greater flow rate to the plant.

The large deck mounted pump has a capacity 62 L/s, however it is expected by the operator that this pump will draw in poorer quality water than the pontoon mounted pumps as it is nearer the sediments near the bank. The pump intakes of the deck mounted pumps are around 3m below the decking and in very low river conditions operation will be problematic as they will not draw enough water. The operators report that their recent approach has been to give the deck pump a run each year if and when full river conditions allow.

The Queensland Nitrate (QNP) pump, rated at 77 L/s, is located nearby and able to be connected to the raw water main. This pump may be available for a short trial at higher plant flow rates. However this is possibly too high a flow rate for the plant process. This option would also not provide a long term solution for supplying higher flow rates to the plant.

The third pontoon mounted pump should be installed as soon as possible to provide standby capacity on the pontoon and to allow trials of higher flow rates.

The raw water pumping station is open to general public access, with associated security/vandalism issues. It is also noted that the site is not easily accessible in wet weather. It is understood that during short periods in 2009 and 2010 the failure of one of the pontoon pumps was unable to be addressed for some weeks due to access problems associated with flooding. Ideally, Council should determine methods of accessing the pumps safely during flood conditions when necessary.

It is noted that at times of extremely low water levels in the river, the pontoon mounted pumps will not draw enough water. Under such circumstances a sledge pump can be connected at a valve pit on the shore for emergency supply to the WTP.

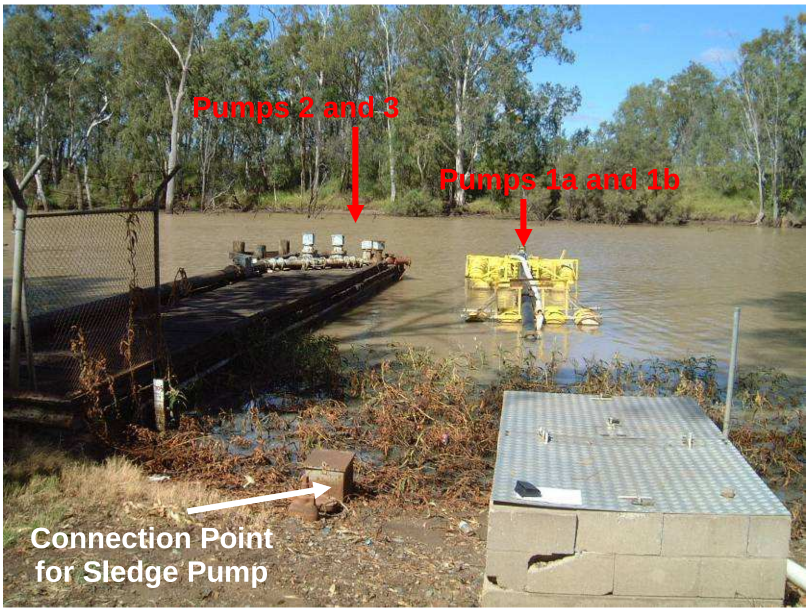


Photo of Pumping Facilities at River

4.3 Pre-Filtration Chemical Dosing

4.3.1 Chemical Dosing Locations

The chemicals currently dosed for pre-treatment and coagulation are:

- Nalco proprietary coagulant, product name Nalco DVS1 C001-D245, dosed into the main at the plant inlet several metres before the flash mixing tank;
- Aeration can be carried out in the first chamber of the flash mixing tank;
- Polyelectrolyte LT25 dosed into the second chamber of the flash mixer; and
- Pre-coagulation chlorine (oxidant) dosed into the bottom of the second chamber of the flash mixer.

The dosing points for these chemicals are shown in the figures below.

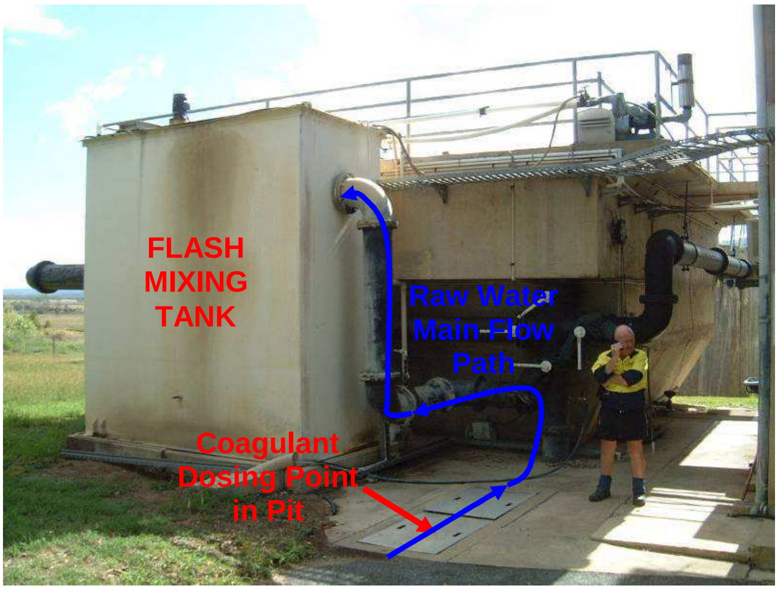


Photo of Nalco Coagulant Dosing Pit on Raw Water Main

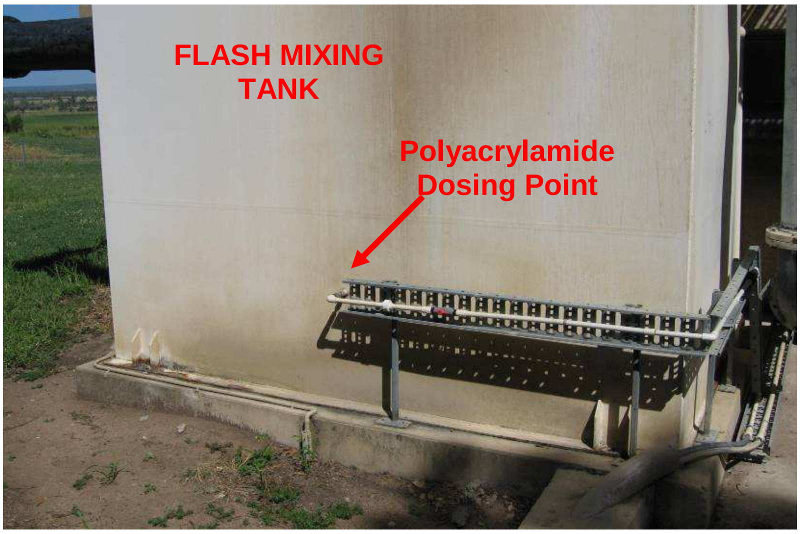


Photo of Polyacrylamide Dosing Point in Flash Mixer

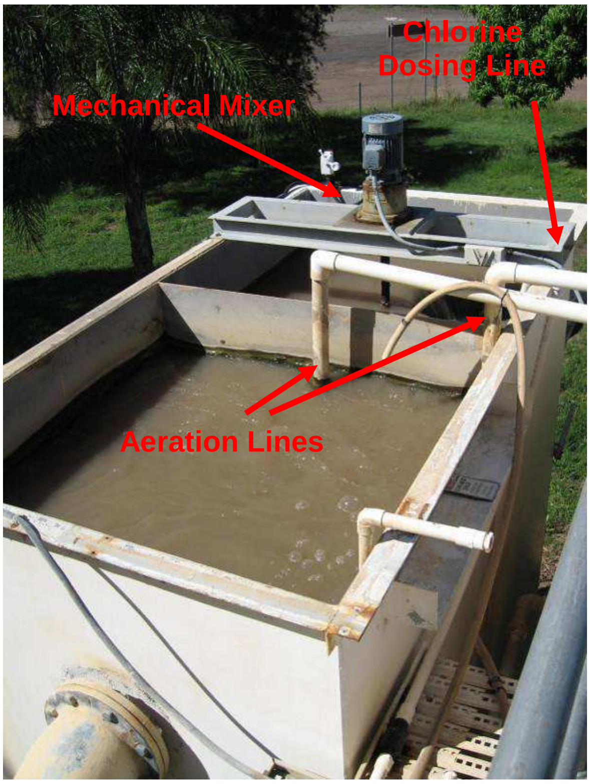


Photo of Aeration and Chlorine Dosing Points in Flash Mixer

4.3.2 Nalco Coagulant Dosing

The plant currently uses polymerised aluminium chemical Nalco DVS1 C001-D245 for coagulation. It was noted during discussions with Nalco that the last part of the product number usually used on site to name the product, D245, is in fact a reference to the container size and is common to many other Nalco chemicals. **The Nalco coagulant product should therefore more correctly be referred to using the first part of the number, DVS C001, in discussions with Nalco at least.** As no other Nalco coagulants are currently used at the Banana Shire plants, the chemical is referred to as 'Nalco coagulant' in this report.

Nalco coagulant dosing was implemented to replace alum and cationic polyDADMAC as the coagulant in March 2005. The operators report that the new coagulant is just as effective as alum plus cationoic polyDADMAC and has less effect on the water's pH (reducing the need for pre-lime dosing). Since its adoption for the Moura WTP, the Nalco proprietary coagulant chemical has also been implemented at the Baralaba and Theodore WTPs.

The Nalco coagulant is dosed into the raw water main at a valve pit, several metres upstream of the flash mixing tank.

The coagulant storage and dosing system capacities and actual chemical doses used are discussed in the next chapter of this report.

4.3.3 Flocculant Aid Polyacrylamide (LT25) Dosing

The polyacrylamide product Magnafloc LT25 is dosed as a flocculation aid into the flash mixing tank at the base of the second compartment. Anionic or nonionic polyacrylamides (also known as polyelectrolytes) are used as flocculation aids to assist in binding particles together during coagulation and flocculation. These long-chain polymers influence the bonding between floc particles and help to form larger and/or denser floc.

The dosing location in the flash mixer allows contact time and mixing of the coagulant chemicals before contact with the polyacrylamide. Flocculation aid polymers are best dosed downstream of the primary coagulant dosing point, allowing a suitable contact time for adequate mixing and initial floc development before the polymer is added.

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

4.3.4 Pre-Clarifier Aeration

Aeration can be carried out in the flash mixing tank if required. Aeration has been used in the past to try to reduce problem taste and odour compounds, reportedly with limited effectiveness.

The blower used for aeration in the flash mixer can be used as a backup blower for backwashing the filters if required.

Photo of Blower for Aeration of Flash Mixer

4.3.5 Pre-Clarifier Chlorine Dosing

Chlorine is dosed into the second chamber of the flash mixing tank, after the coagulant and polyacrylamide dosing points.

Pre-coagulation chlorination (‘pre-chlorine’) is generally used to oxidise soluble metals and organic compounds (including taste and odour compounds). Chlorine dosing can also assist by enhancing the coagulation process. It is understood that pre-chlorine is dosed mainly for oxidation of taste and odour compounds at Moura WTP. Pre-chlorination for dissolved metals oxidation would normally be dosed prior to coagulation so that the small particulates formed are effectively bound up in the floc.

The effectiveness of the chlorine dosing for manganese oxidation should be tested by measuring total and soluble manganese before and after the chlorine dosing when significant levels of soluble manganese are measured in the raw water.

One of the risks with pre-coagulation chlorination is that there is more chance that chlorine will react with the high levels of organic substances in the water to produce undesirable by-products such as trihalomethanes (THMs). **THMs and other chlorine byproducts should be monitored in the treated water. If high levels are found in the treated water, profiles through the process should be taken to investigate the source of these contaminants.** If blue-green algal cells are present, chlorination may also lyse the cells, potentially releasing any algal toxins into the water.

The chlorine dose is reportedly adjusted to achieve a residual of around 0.1 to 0.2 mg/L (measured by grab sample from the settling zone of the clarifier). The actual dose required to achieve a given chlorine residual depends on the raw water quality parameters, particularly turbidity and colour.

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

4.3.6 Pre-Coagulation Alkali Dosing

Pre-coagulation alkali dosing is not currently used at the plant, since Nalco coagulant dosing was commenced. It has been used in the past when high alum doses were required to treat poor quality raw water. When used, the lime solution was dosed into the flash mixer.

There is potential capacity to dose pre-coagulation lime (‘pre-lime’) again if required, as there are two lime dosing tanks (currently used as duty/ standby) and a spare lime dosing pump.

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

4.4 Flash Mixing Tank

The flash mixing tank provides mixing and chemical contact time for floc formation. The main parameters for the flash mixing tank are given in the table below.

Flash Mixing Tank

Component	Parameter (Units)	Design Criteria	Comments
Flash Mixing Tank	Dimensions (m), volume (m^3)	Each compartment 1.52 x 1.5 m x 3.07m deep, 7.0 kL	Total volume 14.0kL
	Detention time (mins)	3.1 min at 75 L/s 4.5 min at 52 L/s	Detention time each compartment =1.55 mins at 75 L/s
	Flash mixing	Mechanical (second compartment only), hydraulic	

As a flow of 77 L/s has briefly been trialled through the plant, it is understood that the hydraulic capacity if the flash mixing tank was adequate to carry at least this flow.

Hydraulic mixing of the coagulant chemicals occurs in the bends in the mains pipe and within the flash mixing tank. The second compartment of the flash mixer contains a mechanical mixer. The water being treated experiences additional flocculation mixing time and energy as it flows into the clarifiers, plus the Stage 2 clarifier also incorporates an additional flocculation zone.

As far as can be determined, the mixing time and energy provided in the flash mixing tank are adequate for suitable floc formation at the current flow rate of 52 L/s. The coagulant mixing issue may become more critical if higher plant flow rates were to be used. Poor mixing is typically indicated by high aluminium levels in the treated water. Coagulation mixing time and energy requirements could be studied further using jar testing.

Photo of Flash Mixer

4.5 Clarifiers

4.5.1 Flow Split Between Clarifiers

The flash mixing tank has two outlets, one to each of the clarifiers. The Stage 1 clarifier is linked to the flash mixing tank by a short pipe and then an inlet channel. The Stage 2 clarifier is linked to the flash mixing tank by a much longer inlet pipe.

Flow out of the flash mixing tank is hydraulically split between the clarifiers by adjusting a manual valve on the outlet to the Stage 2 clarifier. This valve is reportedly left fully open under current normal flow conditions (plant flow rate of 52 L/s).

The flow split between the clarifiers is difficult to check because there are no flow meters available to measure the flow rates through each clarifier. **Ideally, a flow meter would be installed on the inlet pipe to the Stage 2 clarifier to assist with flow balancing under different plant flow rates**.

Photo Showing Connection From Flash Mixing Tank to Stage 2 Clarifier

4.5.2 Stage 1 Clarifier

The stage 1 clarifier is a square (7m x 7m) settling basin. Dosed raw water flows into a channel along one side of the clarifier, and then through an inlet pipe to the base of the centre of the basin. The Stage 1 clarifier was reportedly designed to treat a flow of 25 L/s.

The stage 1 clarifier does not provide a separate flocculation zone. Solids and floc in the water sink due to gravity and settled water is collected in launders spaced approximately 1.75 m apart. The tank floor slopes to a central sump area. Based on available drawings, the settling basin is about 7.7 m deep and has vertical sides to around 2 m below the trough level then tapers (hopper-shaped) for the bottom 5.7 m.

Automatic sludge drawoff periodically removes sludge from sludge drawoff points in the corners of the tank, approximately 2m below the water surface. Sludge can also be drawn off manually from a sludge/ scour outlet pipe located approximately 1m from the bottom of the tank. This sludge drawoff design is not optimum, as sludge reportedly settles thickly in the base of the tank and is difficult to remove even with regular manual blowdowns. Accumulated sludge may tend to turn anaerobic and form reducing conditions, under which manganese will be converted to the soluble form and taste and odour compounds may be formed. Manual desludge is required once per day to prevent the buildup of old sludge at the bottom of the clarifier. **The desludge from the bottom scour outlet could be automated to assist with the regular removal of sludge from the base of the tank**.

The Stage 1 design surface rating of 1.8 m/h (at 25 L/s) is at the high end of the scale for settling basins. At the current plant flow rate of 52 L/s, assuming a 2:1 flow rate split between the Stage 2 and Stage 1 clarifiers respectively (i.e. a flow of 17.5 L/s through the Stage 1 clarifier), the clarifier is estimated to have a more acceptable loading rate of 1.3 m/h. The operators report, however, that even at a plant flow of 52 L/s they set the plant to send the minimum possible proportion of the flow rate to the Stage 1 clarifier due to unreliable operation at higher flow rate proportions.

Photo of Stage 1 Clarifier

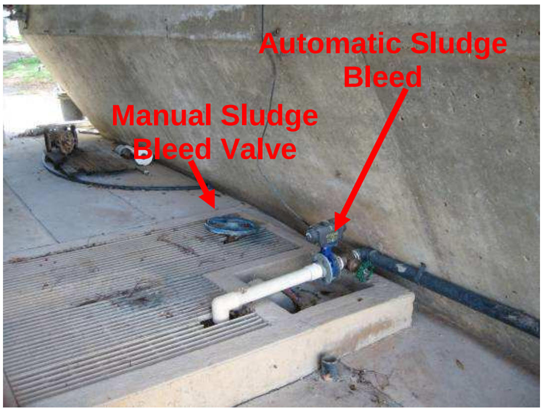


Photo of Stage 1 Clarifier Sludge Bleed Controls

The design of the clarifier has several limitations including:

- Significantly shorter flocculation time compared to Stage 2 clarifier;
- Problematic sludge withdrawal system, requiring manual sludge withdrawal and periodic cleanouts;
- Smaller unit and less robust settling system compared to Stage 2 clarifier.

Because of the information above, it is expected that the effective flow rate for stable operation of this clarifier is be likely to be less than the design rate of 25 L/s. The exact capacity should be determined by trials.

The clarifier loading rate could possibly be improved by fitting lamella tubes, to give a higher effective surface loading rate, although the suitability of the structure for this upgrade would need to be investigated further. **The option of fitting settling tubes to the Stage 1 clarifier or other upgrades to improve settling should be investigated further.**

4.5.3 Stage 2 Clarifier

The Stage 2 clarifier is a sludge blanket upflow design. It was reportedly designed to treat a flow of 50 L/s. Inflow to the clarifier is via the central flocculation zone, where hydraulic mixing assists in the formation and development of floc particles. The flocculation zone has two stirring paddles. The water then flows under the skirt to the outer annulus settling zone and upwards through the sludge blanket which aids floc removal.

The sludge rake is in the settling zone of the clarifier and operates when the plant is on to scrape the sludge to the centre of the conical floor. Settled sludge is periodically removed from the central sump at the bottom of the clarifier by operation of the automatic desludge valve.

The clarifier has an internal diameter of 13.72 m and a water depth of 5.145 m (with an additional 305 mm freeboard) at the deepest part of the sloped floor. The internal flocculation zone is approximately 5 m in diameter.

The design flocculation time of 10 minutes (at 50 L/s) is reasonable based on the expected warm water temperatures and additional floc formation time available in the flash mixer. However, flocculation time requirements in water treatment processes may sometimes be greater. Jar testing trials could be carried out to look at the minimum flocculation time required to form suitable floc.

The clarifier has a design surface rating of 1.4 m/h at 50 L/s which is typical for this type of clarifier. The operator reports that when the Stage 1 clarifier is taken off-line for cleaning, the Stage 2 clarifier is able to take the entire plant flow of 52 L/s, however only for around

3 - 4 hours before the water level tends to rise in the clarifier. This suggests that there is a hydraulic restriction through Stage 2 of less than 52 L/s. Based on the information above, it is expected that the maximum capacity of the clarifier is around 50 L/s. The exact capacity should be determined by trials.

Photos of Stage 2 Clarifier Showing Central Flocculation Well and Settling Zone

Photo of Stage 2 Clarifier Sludge Bleed Controls

4.5.4 Clarifier Summary Table

The main parameters for the clarifiers are given in the table below.

Clarifier (Flocculation and Settling Zones)

Component	Parameter (Units)	Design Criteria		Comments
		Stage 1	Stage 2	
Design Flowrate	Flowrate through clarifier (L/s)	25 design Actual flow limit probably lower	50 (design)	

Component	Parameter (Units)	Design Criteria		Comments
		Stage 1	Stage 2	
Flocculation Zone	Type	Separate flocculation is not provided	Flocculation occurs in central flocculation zone of clarifier	
	Dimensions: diameter, depth (m), volume (m^3)	-	Approx.5m, 5.15m deep, 101 m^3	
	Flocculation time	-	10 min	
	Flocculation mixing	-	Two mechanical mixer paddles (1 – 15 rpm)	Paddle currently run on 'slow' setting
Clarification Zone	Clarifier type	Square settling basin	Sludge blanket upflow design	
	Dimensions: diameter / base, depth (m)	7m x 7m (square), approx 7.7m	13.72 m (total internal diameter), 5.15m	Stage 1: Depth based on available drawings
	Settling zone surface area (m^2)	49 (including troughs)	128 (excluding flocculation zone)	
	Surface rating (m/h)	1.3 at 17.5 L/s 1.8 at 25 L/s	1.4 at 50 L/s	Stage 1: High settling rate at 25 L/s
	Sludge scraping	None – solids and floc sink due to gravity and settle in conical base	Sludge rake	
	Sludge drawoff system	Automatic desludge from 2m below surface, manual desludge from 1m above bottom of tank	Automatic drawoff from centre well via 150 mm diameter pipe	Stage 1: Not optimum – sludge settles thickly, difficult to remove.
	Typical sludge removal frequency, duration	Every hour, for 20 minutes	Every hour, for 20 minutes	Stage 1: Sludge bleed opened manually once per day

In the short term trial at 77 L/s for 8 hours, the clarifiers reportedly handled the flow rate reasonably, although the flow split between the two clarifiers during that trial is not known. **An extended high flow trial is required to prove the capacity limitations of both of the clarifiers, but in particular the Stage 1 clarifier.** The capacity of the Stage 1 clarifier could be tested at current plant flow rates by minimising flow to Stage 2 with the inlet valve so that at least 25 L/s goes to Stage 1, however a permanent or temporary method of measuring the flow rate to at least one Stage would be required to undertake effective trials.

It is noted in the table above that the sludge blowdown timers for both clarifiers are set to blow down for 20 minutes of every hour. This blowdown time is a little higher than typical. The clarifier sludge blowdown settings for frequency and duration of the blowdown should ideally be continually optimised in response to changes in raw water conditions and dosing levels to achieve the best operation of the clarifiers and minimise the amount of water lost to this waste stream.

4.6 Filtration and Backwashing

4.6.1 Flow Distribution Between Filters and Filtration Rates

The two Stage 1 plant filters were originally designed to treat a total flow of 25 L/s. However with the Stage 2 upgrade and addition of two more filters, it appears that the plant was then designed to split a combined flow of 75 L/s evenly between the four filters. An interconnection pipe is provided between the Stage 1 and Stage 2 filter inlets (after clarification) to allow the water flow rate to stabilise between the four filters. This means that the entire plant flow is split across all on-line filters, and that when one filter goes off-line (for a backwash or other reason), the flow is distributed between the remaining on-line filters. The actual flow distribution ratio is not certain, as the length of the connection pipe may hydraulically influence the evenness of flow distribution between the two stages.

Photo Showing Pipe Linking Stage 1 and 2 Filters

Assuming that the flow is divided equally between all on-line filters, a plant flow of 75 L/s gives the following filtration rates at each filter:

- With four filters on line (18.75 L/s through each filter): Stage 1 filters 9.0 m/h, Stage 2 filters 9.2 m/h;
- With three filters on line (25 L/s through each filter): Stage 1 filters 12.0 m/h, Stage 2 filters 12.3 m/h;
- With two filters on line (37.5 L/s through each filter): Stage 1 filters 18.0 m/h, Stage 2 filters 18.5 m.

The calculated filtration rates with four and three filters online at design flow rates are considered within the feasible range for these types of filters, although it is noted that the actual rates may be higher if the flow is not effectively distributed between the on-line filters. The calculated filtration rates of 18.0 – 18.5 m/h with only two filters on-line at design flow rates are considered outside the upper limit of the suitable range for sand filters and should be avoided by only taking one filter off-line at a time. At the current plant flow rate of 52 L/s it may be more feasible to have only two filters online as each filter would carry only 26 L/s each.

Based on available data, the total capacity of the filters is taken to be the design flow rate of 75 L/s, although **filter run times and water quality should be investigated and proved in long term trials at the maximum flow rate.**

4.6.2 Stage 1 Filter Design and Condition

The Stage 1 filters are housed in concrete filter structures attached to the original plant building. The filters contain fine sand, coarse sand and gravel media, as outlined in the table below. The effective size of the fine sand is 0.6 mm.

It should be noted that the reported media design (see summary table below) is missing a coarse sand layer of size range 1.7 to 3.2 mm. Ideally all media layers should overlap in size to minimise layers flowing through the layer below and onto the bottom of the filters. Depending on the nozzle design, sand can go through the nozzles into the filtered water well once it has passed through the gravel layers. This is especially prevalent with high rate backwashing.

The operators report that a significant amount of sand has been lost from the filters. It is not clear whether the media was lost through the base of the filters (e.g. because of the missing support layer) or during backwashing. It is estimated by operators that up to 400 mm depth of media may have been lost, equivalent to around 3 m^3 per filter. The media level was last topped up around five years ago, when around 4 t (roughly 2 m^3) was reportedly added to each filter. On surface inspection, the remaining media appears to be clean and in a satisfactory condition, however the amount of media lost over 5 years is very significant, and this issue should be investigated further.

Photo of Stage 1 Filter Showing Uneven Media and Concrete Etching

It is also noted that the surface of the media in the Stage 1 filters often appears uneven. This may be due to the way that water drops into the filters, or associated with uneven backwashing due to cracked nozzles, or may be a side effect of the media loss.

It is also noted that the concrete in the filter structures shows evidence of significant etching.

Backwashing issues are discussed further below.

Based on the significant filter media loss, **a detailed investigation of the Stage 1 filters should be undertaken to determine the reason for the media loss, including confirmation of the existing filter media strata, analysis of backwashing rates, inspection of the filter floor and underdrains, testing of the underdrains for the presence of sand etc. The filter media should then be refurbished with suitable sand layers, including the missing 1.7 to 3.2 mm coarse sand layer.**

4.6.3 Stage 2 Filter Design and Condition

The two Stage 2 filters are located in cylindrical tanks next to the Stage 2 clarifier.

The Stage 2 filters are presumed to have a similar media configuration to that of the Stage 1 filters, as no specific information on the Stage 2 filter media was available.

The filter structure and media in the Stage 2 filters appear to be in acceptable condition on surface inspection, and no significant media losses are reported.

Backwashing issues are discussed further below.

Photo of Stage 2 Filter Showing Level Switches

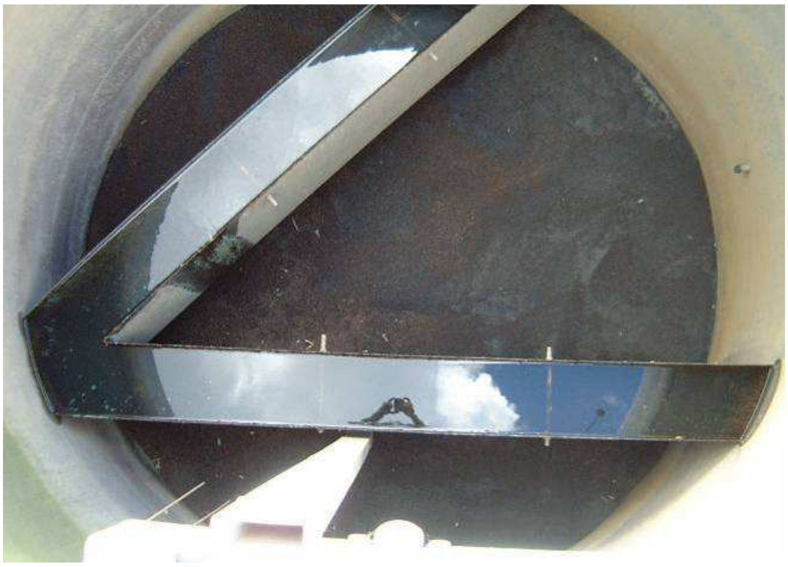


Photo of Stage 2 Filter Showing Media and Backwashing Trough

4.6.4 Filter Flow Control and Monitoring

The designs of both Stage 1 and Stage 2 filters use constant water level in the filters to control the rate of flow through each filter. The water level control devices are:

- Stage 1 filters: Bubbler-type level sensors which control modulating outlet valves to control filter level.
- Stage 2 filters: Float sensors which control modulating outlet valves to control filter level.

The systems are designed to open the outlet valve further if there is pressure for the water level to rise in the filter (from increasing headloss across the filter bed), in order to maintain a constant water level in the filter. There are also high level sensors in the filters, which are attached to local alarms (but not to SCADA/callout alarms). The filter water level control systems have been known to malfunction at times, leading to varying water levels and uneven flow distribution between the filters. **Further investigations should be**

undertaken to identify options for upgrading or replacing the filter flow control systems to achieve accurate, reliable control of water levels and flow distribution.

The flow rate through each filter is measured by a flow meter, with indication locally and on a chart recorder in the plant building. However the flow meters are aged and reportedly not reliable. **The filter flow meters should be checked and refurbished if possible, to give accurate, reliable flow rate readings. The flow meters and chart trends should then be used to further investigate flow distribution between the filters.** The flow meters could potentially be set up to record trends on the SCADA system and to trigger flow rate alarms.

It is noted that for optimum filter performance the provision of a constant flow rate through each filter is preferred. Some systems are designed so that when one filter is taken offline (e.g. for backwashing) the plant flow rate is reduced so that the flow rate through the remaining online filters is not increased significantly. This would however require modifications to allow control of the plant flow at the plant inlet (such as a control valve or the provision of VSDs on the raw water pumps), and flow pacing of the chemical dosing systems. **Measures to provide a steady flow rate through the filters should be considered in the design of any significant upgrades to the filters or their control system**.

Headloss is not monitored for the filters. This parameter would provide additional information on the build-up of solids in the filter bed to assist the operator in deciding when to backwash. **Headloss measurement devices, preferably differential pressure meters, should be installed in the filters to provide extra information to assist with filter monitoring and backwashing requirements.**

There is a partially installed online turbidimeter on the Stage 1 filtered water stream, which would be beneficial in terms of monitoring the turbidity trends. The monitoring of filtered water turbidity is a crucial factor in achieving satisfactory treated water quality and a major critical control point for the WTP. **Ideally the online turbidimeter or several similar meters would be set up to automatically log individual turbidity trends for all four filters.**

4.6.5 Filtration Parameters Summary

The main parameters for the filtration cycle of the filters are given in the table below.

Filters

Component	Parameter (Units)	Design Criteria		Comments
		Stage 1	Stage 2	
Filter Beds	Maximum flowrate for filtration stage (L/s)	75 L/s total		Flow divided between all on-line filters
	Type	Rectangular concrete vessels	Cylindrical steel vessels	
	Number of filters	2	2	
	Filter vessel: area dimensions (m)	2.438 x 3.072	3.05 diameter	
	Area per Filter (m^2)	7.49	7.31	
	Total Filter Area (m^2)	14.98	14.61	
	Filtration Rate (m/h):			

<table>
<tr><th rowspan="2">Component</th><th rowspan="2">Parameter (Units)</th><th colspan="3">Design Criteria</th><th rowspan="2">Comments</th></tr>
<tr><th colspan="2">Stage 1</th><th>Stage 2</th></tr>
<tr><td rowspan="2"></td><td>All filters operating</td><td colspan="2">9.01 at 18.75 per filter (75 L/s between 4 filters)
6.01 at 12.5 L/s (25L/s between 2 filters)</td><td>9.23 at 18.75 L/s per filter (75 L/s between 4 filters)
12.31 at 25 L/s (50L/s between 2 filters)</td><td rowspan="2">* Assuming plant flow of 75 L/s divided evenly between 3 on-line filters</td></tr>
<tr><td>One filter off-line</td><td colspan="2">12.02 at 25L/s* per filter</td><td>12.31 at 25 L/s* per filter
24.62 at 50L/s through 1 filter</td></tr>
<tr><td rowspan="4">Filter Media</td><td>Filter sand: size (mm), U.C., depth (mm)</td><td colspan="2">0.50 – 1.00, 750</td><td rowspan="2">Media details not available. Assumed to be similar to Stage 1 filters</td><td>Uniformity coefficient (UC) for filter sand not known</td></tr>
<tr><td>Coarse sand and gravel</td><td>Size (mm)
1.0–1.7
3.2–6.4
6.4–12.7
12.7–25.4</td><td>Depth (mm)
150
100
100
150</td><td>Profile should have sand layer in size range 1.7 to 3.2 mm</td></tr>
<tr><td>Underdrains</td><td colspan="2">Not known</td><td>Not known</td><td>No info available</td></tr>
<tr><td>Available Headloss (m)</td><td colspan="2">Not known</td><td>Not known</td><td>Headloss not currently measured</td></tr>
</table>

4.6.6 Filter Run Times and Backwash Initiation

The filters are normally run for several days between backwashes. There is no system for the automatic triggering of a backwash. Backwashes can only be manually initiated.

The backwashes are normally triggered based on a run time (2-3 days) of more often if the combined filtered water turbidity is higher (around 0.9 NTU).

Under the current control system, filter backwashes are initiated by the operator by manually pressing the backwash initiation button on the filter control panel. This system is problematic as the plant is unattended overnight and there are no meters to monitor headloss and no callout alarm to indicate a high water level in a filter, therefore filters are sometimes run too long. The arrangement has reportedly led to overflows from the filters when the headloss has built up to a level where no further flow can pass through the filter at a time when the plant was unattended. **The callout alarms list should be updated urgently to include filter high level alarms for all filters to prevent accidental overflows.**

In the medium term, the filter control system should be upgraded to allow the automatic initiation of a backwash on trigger levels for time, headloss (when headloss meters are installed) and filter high water level. Ideally, the filters would also eventually be upgraded to backwash automatically on filtered water turbidity.

4.6.7 Backwashing Sequence

The purpose of backwashing is to remove the floc and other solids from 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 Moura WTP, the backwashing sequence includes the following sequence steps:

- Drain down of water level to bottom of backwash troughs, or lower when manually controlled;
- Air scour, combined with low rate water flow – typically run for 10 minutes;
- High rate water wash – typically run for 10 minutes;
- Refill to operating water level by opening filter inlet valve.

The combination of air scour and low rate water as the initial phase of backwashing is unusual. In most filter designs the air scour is generally used alone for a period, before the low rate water is introduced. The danger of introducing the water flow too early is that water will overflow the backwash troughs while air is still causing turbulence, carrying filter media into the troughs. Media loss in the Stage 1 filters may be related to this issue. **The backwashing phases and rates should be reviewed in detail to determine their suitability in terms of potential media loss.**

4.6.8 Backwashing System Components

Air scour is provided by an air blower, located in the blower room in the chemical dosing shed.

The low rate water is supplied to the filters using the town water pressure via a small bore black poly pipe, separate to the backwash main. The low rate wash line has a solenoid valve fitted on it to automatically start/stop the flow as part of the backwashing sequence.

Water wash is provided by the backwash pumps, located in a separate shed near the clear water tanks. The backwash pumps are old and in very poor condition. One of the pumps cannot be used because a major component of the pump was removed years ago and never replaced. There is therefore no standby backwash pump.

It is understood from the operator that as an alternative to the backwash pumps, backwash water flow and pressure can be provided by the town supply mains pressure. An additional cross-connection from the treated water rising main to the backwash line was reportedly provided as part of the installation of the current treated water pressure pumps. **A system for backwashing the filters using the rising main water pressure should be tested and established as soon as possible to allow backwashing to continue even if the existing backwash pump fails. If backwashing using the rising main pressure is unsuccessful, a standby backwash pump or ideally two new backwash pumps should be installed urgently.**

Photo of Air Scour Blower

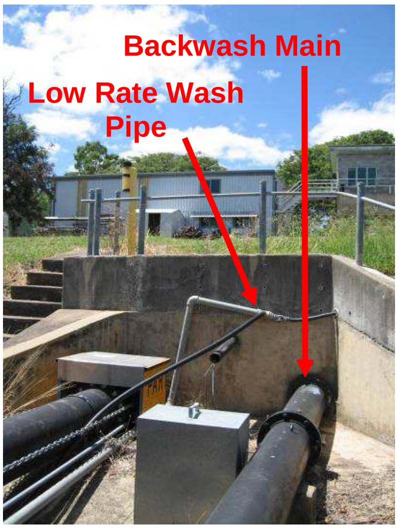


Photo of Backwash Main

Photo of Backwash Pumps

4.6.9 Backwashing Controls

The backwashing system for the Stage 1 and 2 filters was designed to undertake automatic backwashes (triggered manually at the backwash control panel), in which the filter control system automatically follows a pre-programmed backwash sequence with operator adjustable wash times. However, the filter controls have aged and the water pneumatic controls for the filter control valves are progressively being replaced with air pneumatic systems. The backwashing has therefore been carried out by manually controlling each valve and component for a number of years now, while Council staff upgrade the control system. **The repair of the valve controls should be completed as soon as possible to allow the reinstatement of automatic backwashing.**

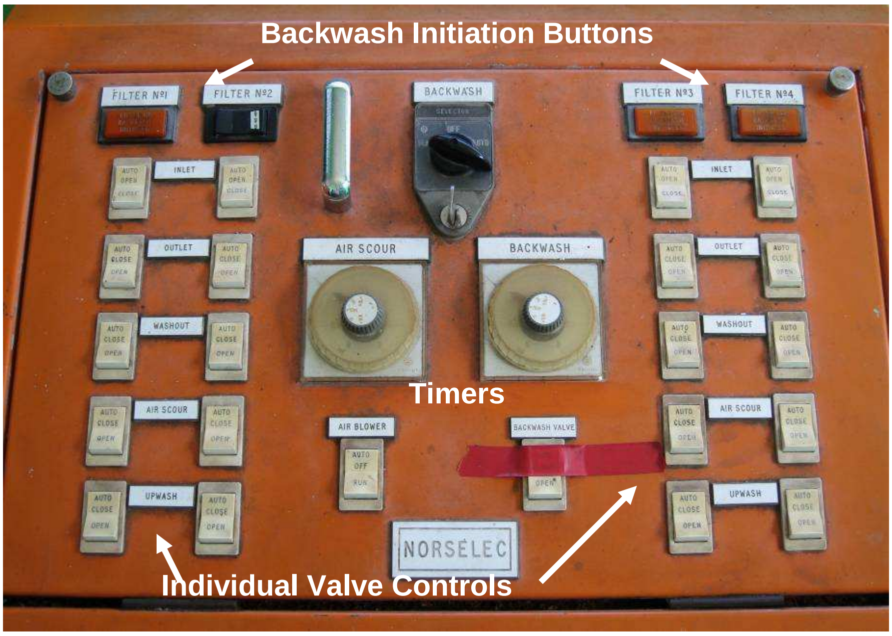


Photo of Backwash Control Panel

4.6.10 Backwashing Rates

The air scour flow rate is uncertain. There is an air scour flow rate dial gauge on the filter control panel, however the operator reports that this has not worked for many years. The dial gauge reads up to 4.5 m^3/min, which is equivalent to approximately 36 m/h, a fairly low rate for air scour. The operators report that the air scour appears adequate in terms of filter agitation. **Ideally the actual air scour flow rate should be determined and reviewed in relation to backwashing requirements.**

The backwash cycle includes air scour, combined air and low rate water and high rate water wash phases. Design backwash rates quoted from the Ullman and Nolan specification for Stage 1 filters and the equivalent volumes used over a 10 minute wash are as follows:

- Low water rate: 20.4 – 29.4 m/h: Volume used in 10 min (at 25 m/h): 31 kL;
- High water rate: 58 m/h: Volume used in 10 min: 72 kL.

The operators' rule of thumb, used over many years to calculate backwash water consumption, is that the typical backwash rates (assumed to be for the high rate wash) are:

- 5.8 kL/min for the Stage 1 filters: Volume used in 10 min: 58 kL;
- 6.0 kL/min for the Stage 2 filters: Volume used in 10 min: 60 kL.

There is thus a discrepancy between the information from the Ullman and Nolan specifications (for the Stage 1 filters) and the operators' rule of thumb. It is also noted that it is likely that the backwash pump would not produce the original capacity in its current rundown state. **The low and high rate backwashing flow rates should be confirmed if possible by filter rise rate tests. The rates should then be reviewed in terms of backwashing requirements, water consumption and the potential demands on a future WTP wastewater holding and treatment system.**

The actual backwashing rate or total water usage cannot easily be confirmed because there is no flow meter or totaliser on the backwash line. **Ideally, a flow meter should be fitted to the backwash water line for checking backwash flow rates and consumption**.

4.6.11 Backwashing Parameters Summary

Filter backwashing component capacities and settings for the Stage 1 and 2 filters are outlined in the table below.

Filter Backwashing – Stage 1 and 2 Filters

Component	Parameter (Units)	Design Criteria	Comments
Backwashing Parameters	Backwash control	Automatic, with manual option Backwash initiation can only be done manually	Automatic backwash function currently not working due to deteriorated valve controls
	Backwash phases	• Air scour (typically 10 min), combined with low rate water • High rate water wash (typically 10 min)	

Component	Parameter (Units)	Design Criteria	Comments
	Backwash triggers	Manually initiated, generally based on time	Few indications to warn operator when filter headloss high
	Backwash frequency	Filters are normally backwashed 2-3 times per week	
	Air scour rate (m/h)	Not known at this stage	To be tested/ calculated Est. 36 m/h max based in old air scour gauge range
	Air scour blower: capacity (m^3/min)	Not known at this stage	To be tested/ calculated
	Water scour rate (m/h)	From original specification for Stage 1 filters: Low rate: 20.4 – 29.4 m/h High rate: 58 m/h	Differs from operators' rule of thumb To be tested
	Backwash pumps capacity	Not known at this stage	To be tested

4.6.12 PAC Dosing to Filters

Powder Activated Carbon (PAC) is sometimes dosed at the WTP for treatment of tastes and odours. PAC, when required, is manually dosed into the filters. The PAC is mixed with water in a bucket and then poured down onto the surface of the media. Either a hose or turbulence from the filter inflow is used to distribute the PAC over the filter surface.

PAC particles contain many small pores which effectively adsorb molecules from the water. PAC will also effectively adsorb soluble manganese if it is present in the raw water. PAC particles remaining in the filter beds may currently provide ongoing removal of low levels of manganese. There is therefore some scope to use the adsorptive capacity of PAC, as an emergency response, to remove soluble manganese from the water.

PAC doses and system parameters are discussed further in the next chapter of this report.

4.7 Post-Filtration Chemical Dosing

4.7.1 Chemical Dosing Locations

The chemicals dosed after filtration are:

- Post-filtration chlorine (disinfectant);
- Post-filtration lime (alkali for pH correction).

Both chemicals are currently dosed into the Stage 1 post-filtration contact tank, which is located below the old plant building. Mixing between the Stage 1 and Stage 2 streams then occurs at the filtered water weir box.

4.7.2 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 to prevent the regrowth of biofilms in the reticulation pipes. The post-chlorine dose is adjusted to meet the final water chlorine residual target.

The operators have noted that the post-filtration chlorine residual, as measured in the treated water, tends to rise over time if the plant operates for long hours. The reason for this phenomenon is not clear.

Sufficient chlorine is dosed into the water flow from the Stage 1 filters at the current dosing point to give an appropriate residual in the combined water flow. However the current arrangement includes potential risk in that Stage 1 water is fully disinfected (primary disinfection) and then the chlorine residual (secondary disinfection) is employed to disinfect Stage 2 water, prior to contact time in the reservoirs. **It would be preferable to provide full disinfection to both streams of water in order to minimise risk of pathogen breakthrough through Plant 2 by providing disinfection in the pipeline after the two waters have combined**.

Chemical doses used and chlorine storage and dosing system capacities are discussed in the next chapter of this report. **Automatic dose trimming coupled with planned online chlorine residual measurement could also be considered, as discussed in the water quality section of this report.**

4.7.3 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 dosing of lime is currently controlled manually to meet the target pH, based on analysis of grab samples of the final water.

The operators have noted that the post-filtration pH, as measured in the treated water, tends to rise over time if the plant operates for long hours. The reason for this phenomenon is not clear.

Sufficient lime is dosed to the Stage 1 filtered water at the current dosing point to meet the target pH for the combined water from the two stages when they blend downstream. The dosing point in the Stage 1 contact tank, however, leads to problems with build-up of lime residues in the tank, which is difficult to access.

It would be preferable to dose lime downstream where the treated water from the two plants combines. The proposed new injection point is at the weir box where the Stage 1 and Stage 2 flows meet. It is expected that this location will be suitable as the weir provides mixing for the lime addition. Any insoluble material will then accumulate in the reservoirs which would be scoured/cleaned out as part of reservoir maintenance (but probably less frequently than is required in the Stage 1 filtered water well). **A new lime dosing point is therefore recommended at the weir box where the treated water from the two stages is blended.**

Chemical doses used and lime storage and dosing system capacities are discussed in the next chapter of this report. **Automatic dose trimming coupled with planned online pH measurement could also be considered, as discussed in the treated water section of this report.**

Photo of Filtered Water Weir Box Where Stage 1 and 2 Filtered Waters Blend

4.8 Clear Water Storage and Distribution

The final treated water is stored in the clear water tanks, from where it is pumped into the distribution system and to the elevated town reservoirs by the treated water pressure pumps.

The treated water pumps supply water to the town system and maintain the system pressure to the selected setpoint. The pumps start and stop based on system pressure requirements. The pump capacity is reportedly around 93 L/s, but will vary due to changing pressures within the town distribution system and varying levels in the reservoir.

The main parameters of the clear water system are shown in the table below.

Treated Water Systems

Component	Parameter (Units)	Design Criteria	Comments
Clear Water Storage Tanks	Type	2 x concrete reservoirs	
	Total Capacity (ML)	2.4 (old: 0.9, new: 1.5)	TWL 131m AHD
	Detention time	3 hours at 75 L/s to 8.1 hours at 75 L/s	For 0.81 ML assumed working volume of small tank For 2.2 ML assumed working volume of both tanks
Treated water pumps	No. and Capacity each (L/s)	5, 93 L/s total capacity Automatic duty changeover	Grundfos model CR90-4 (Rate and model based on Cardno report)
	Pressure Setpoint (kPa)	700	
Elevated Town Reservoirs	Capacity (ML)	Dawson Hwy 0.46 ML Burnham St 1.0 ML	TWL 159m AHD (standpipe for trucks) TWL 166m AHD (at pool)

Water is pumped out of the clear water tanks into the distribution system without further disinfection contact time. The contact time in the clear water tanks is expected to be well above required levels if the full clear water tank volumes are used and there is no short circuiting through the tanks, however this could be confirmed by **checking operational**

levels and the configuration of the tanks to estimate the potential risk of short-circuiting.

The current total system storage volume of 3.86 ML is equivalent to only approximately one day's storage at the current maximum output of the plant (at 52 L/s). Around 3 day's maximum demand is a used as a general rule for design of system storage. System storage requirements and reticulation design for Mora and Banana have been dealt with in a separate study and report undertaken by Cardno in 2006/07.

Components on the treated water rising main after the treated water pumps include the plant outlet ('consumption') magflow meter, a non-return valve and an isolation valve and takeoffs for WTP backwashing water and the treated water sample line. The consumption flow meter records to the SCADA system and a local display.

There have been complaints about town water pressure, so it appears that the pumping system does not accurately control the pressure in all parts of the system. Computer hydraulic modelling and calibration pressure chart recordings have confirmed that system pressure can regularly fall below 10m head. Council staff are progressively upgrading small diameter water services but investigations to overcome the general low pressure problems are continuing. **The findings of the water pressure investigations should be applied to optimize the performance of the treated water pumping station if necessary.**

Photo of Clear Water Tanks

Photo of Treated water pressure pumps

4.9 Wastewater System

Wastewater from the plant, including backwash water and sludge blowdown, is currently discharged directly onto the neighbouring landholder's property. Plant overflows are also discharged at the same release point. There are no WTP wastewater holding tanks or processing facilities on the Moura WTP site.

It is understood that the neighbouring landholder uses the water for irrigation and directs some of the water into a dam on his property. This landholder no doubt sees the water supply as a valuable resource. However there may be some environmental concerns about the continued application of WTP wastewater to land, the use for stock watering and/or the potential for runoff into water courses, in terms of the accumulated levels of metals (e.g. aluminium, iron and manganese), salts, nutrients, pH levels and possible pathogen transfer. **The water quality issues associated with the use of the wastewater for land irrigation/ stock watering and potential runoff into water courses should be reviewed. The effect on wastewater quality of the recent changeover to the Nalco proprietary coagulant should also be reviewed**. It is noted that the irrigation discharge method may no longer be allowed by the regulatory authorities in the future, due to environmental and/or water quality concerns.

The disposal of the wastewater to the neighbouring property represents the loss of a significant proportion of the influent water. In times of drought especially, some of this water could be recycled to the head of the treatment works rather than being lost from the water supply system, however this is currently not possible because of the lack of a wastewater holding tank.

It is likely that an effective wastewater treatment system will be needed in the future, to limit the flow released to the environment and/ or to allow some of the wastewater to be reclaimed for town water supply uses. The simplest wastewater system would be to install several lagoons and a supernatant return system, however this choice may be limited by land availability on the site. If a limited system footprint is required, various thickening and dewatering system design alternatives are available. **When required, the most appropriate type of wastewater holding, treatment and recycling facilities for the WTP should be identified and implemented.** Provision for such facilities should be included in the design of any major plant upgrades as appropriate.

4.10 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)
Raw Water Pumps	Pump condition/ capacity, river level	52 L/s	Add third pump (planned), and/ or enlarge rising main
Plant Inlet and Flash Mixer	Hydraulic design, required floc formation time	70 – 75 L/s	Augment/ replace with larger tank
Clarifier Stage 1	Surface loading rate for effective settling	< 25 L/s	Optimise settling/ Upgrade / Replace
Clarifier Stage 2	Flocculation time, Surface loading rate for effective settling	50 L/s	Optimise settling
Filters Stage 1	Filtration rate for effective performance	37.5 L/s (To be proved)	
Filters Stage 2	Filtration rate for effective performance	37.5 L/s (To be proved)	
Clear Water Tanks	Volume	2.4 ML	Provide another tank
Treated water pumps		93 L/s	
Sludge/ Wastewater Treatment Systems		No existing facilities	

From the summary above, it appears that **the most limiting factors for the overall plant capacity are:**

- **Raw water pump capacity;**
- **Clarifier capacity (Stage 1 clarifier especially).**

The filter capacity to treat 75 L/s split between the four filters also needs to be proved by trialling the higher plant flow.

5. Chemical System Descriptions, Doses and Capacities

5.1 Chemical System Descriptions

5.1.1 Nalco Coagulant (DVS1 C001-D245)

The Nalco coagulant has been used as the main coagulant since March 2005.

Polymerised aluminium chemical Nalco product DVS1 C001-D245 is supplied as a liquid product in bulky boxes. The dosing solution is delivered to the dosing point by a single metering pump.

The main details for the DVS1 C001 makeup and dosing system are given in the table below.

Nalco Coagulant System

Component	Parameter (Units)	Design Criteria	Comments
Nalco Coagulant System	Chemical product and Strength (%)	Nalco DVS1 C001-D245,	Chemical composition is proprietary information
	Batching concentration (g/L)	Dosed undiluted	Product density is 1260 g/L
	Storage tanks: No. of, capacity each (L)	1 – 2 bulky boxes (1000 L) stored in chemical shed	
	Dosing tank: No. of, capacity each (L)	1 x 1000 L	
	Dosing pumps: No. of, Capacity (L/h)	1 x 27 L/h	Capacity from name plate. Capacity should be tested Spare pump kept in storage
	Dilution water	Continuous carry water	

There is only one Nalco dosing pump, however it is understood that a suitable replacement pump is kept in the storeroom. **An installed standby dosing pump would improve process reliability by allowing plant operation to continue with minimal interruption in the event of pump failure.**

It is noted that the location of the dosing pump on the edge of the polymer tank bund is not ideal and could lead to spills onto the chemical shed floor. **A suitable bund should be provided.**

Photo of Nalco Dosing Pump

5.1.2 Alum

Alum was previously used as the main coagulant but was replaced by Nalco coagulant in March 2005. The existing alum system is reviewed below in case it is needed in the future.

Alum was supplied as granular alum in 25 kg bags. It was batched manually into a standard concentration dosing solution. The dosing solution was delivered to the dosing point by a single metering pump.

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	
	Storage capacity (25kg bags)	Minimal chemical stored on site as no longer used	
	Makeup and dosing system components	2 x solution tanks, 1 x dosing pump	Located in chemical dosing room
	Batching concentration (g/L)	200 (20% w/v) to 246 (24% w/v)	26 x 25 kg bags into 3250 L 32 x 25 kg bags into 3250 L
	Solution tank capacity (L)	2 x 3250 L tanks	
	Dosing pumps: No. of, Capacity (L/h)	1, 240	Pump capacity from name plate

The alum batching system is simple and reportedly ran well with minimal maintenance problems, but required manual handling by the operators in lifting the bags onto the unloading platform and emptying them into the solution tanks. **Manual handling and contact with the chemical would ideally be minimised in any planned alum system improvements.**

There is no dust extraction system for the bag unloading system. **A system for minimising dust escape should be provided.**

It is noted that there is only one alum dosing pump, however a suitable spare pump is kept in the storeroom. As alum is no longer dosed, this is not a critical pump, however an installed standby dosing pump should be considered if alum was to become the main coagulant again.

The alum pump has a calibration cylinder, and drop tests were performed weekly when dosing alum to check the pumping rate in relation to the required dose.

Photo of Alum Dosing Pump

Photo of Alum Level Testing

The alum pump and tanks are inadequately bunded. **An appropriately sized bund is required for the tanks and pump.**

The alum tank level is measured by using a 'dipper stick' marked with graduations. While rudimentary, this method allows the operators to measure the amount of water and chemical to add to top up the tank level. A less operator intensive method of measuring the tank level for chemical makeup calculations could be used, **such as providing a sight-tube or electronic level measurement device**. The dipping stick could be retained as a backup level measurement.

5.1.3 Cationic PolyDADMAC (LT425)

PolyDADMAC was previously used but was replaced by Nalco coagulant in March 2005. The existing polyDADMAC system is reviewed below in case it is needed in the future.

Cationic polyDADMAC was supplied as a liquid product in bulky boxes and diluted with water in the dosing tank. The dosing solution was delivered to the dosing point by a single metering pump. The equipment used to dose polyDADMAC is now used to dose the Nalco coagulant product.

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

Cationic PolyDADMAC System

Component	Parameter (Units)	Design Criteria	Comments
Cationic PolyDADMAC System	Chemical product and Strength (%)	Ciba Magnafloc LT425 liquid product	From data sheet, s.g.=1.09
	Batching system	Manual addition of product and dilution with treated water	
	Batching concentration (g/L)	10% v/v = 109 g/L	100 L (109 kg) product diluted into 1000 L per operator
	Storage space (bulky box)	Typically 1 x 1000 L bulky box stored in chemical shed	
	Dosing tank size	1000 L	
	Dosing pumps: No. of, Capacity (L/h)	1 x 27 L/h	Pump capacity from name plate

5.1.4 Polyacrylamide (LT25)

The polyacrylamide LT25 is supplied as a powder and batched manually in duty/ standby batching/ dosing tanks. A funnel fed eductor is used to wet up the polymer powder as it is added to the water during batching.

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

Polyacrylamide System

Component	Parameter (Units)	Design Criteria	Comments
Polyacrylamide System	Chemical product and Strength (%)	Ciba Magnafloc LT25, supplied as powder	
	Storage space (bags)	Several bags stored in chemical dosing shed	
	Batching system	Manual feed into mixing/ dosing tank via eductor	
	Batching concentration (g/L)	1.4 g/L	840 g into 589 L
	Dosing tank size	2 x 589 L	
	Dosing pumps: No. of, Capacity (L/h)	1 x 292 L/h	Pump capacity from name plate

Photo of Polyacrylamide Makeup Eductor and Mixing Tank

Photo of Polyacrylamide Dosing Pump

No batching or dosing concerns were noted for the polyacrylamide system, provided correct maintenance procedures were followed.

Pump and tanks are inadequately bunded. **A suitable bund should be provided for the system.**

5.1.5 PAC

PAC is dosed manually to the filters. The main details of the PAC storage facilities are shown 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	
	Storage space (PAC bags)	3 pallets of 20 kg bags stored in chemical shed	32 bags per pallet (640 kg)
	Dosing mechanism	Dosed by hand to filters	PAC dose added after backwash

Ideally, an automatic dosing facility for PAC should be provided if PAC dosing is required regularly.

Photo of PAC Storage

5.1.6 Chlorine

The chlorine room is a separate chemical dosing shed. The chlorine shed houses several cylinders and one 920 kg drum and the pre-chlorine and post-chlorine chlorinators.

The main details for the pre- and post-chlorine dosing systems are given in the table below.

Chlorine Systems

Component	Parameter (Units)	Design Criteria	Comments
Chlorine System	Chemical product	Chlorine gas	920 kg and 70 kg cylinders
	Chlorine room capacity (cylinders)	1 x 920 kg drum 4 x 70 kg cylinders (1 online and 3 standby)	Pre-chlorine draws from 920 kg drum. Post-chlorine draws from duty 70 kg cylinder
	Ejectors: No. of, Type, Capacity (g/h)	2 (1 x pre and 1 x post dosing), V100 Rotameters Capacity: • Pre: 1833 g/h • Post: 600 g/h	Noted that larger and smaller rotameters available for use for lower and higher dosing demands For 70 kg cylinder max discharge rates are approx 700 g/h at 10ºC and 400 g/h at 5ºC. For 920 kg drum max discharge rates are approx 3 kg/h at 10ºC and 1.6 kg/h at 5ºC.
	Booster water pumps: No. of, Capacity (L/s)	No booster pump. Runs from town pressure	
	Load cells capacity	Drum weight measured with scales by lifting drum on forklift	No auto changeover
	Service water	Supplied from treated water rising main	

The chlorine dosing systems do not have auto changeover when a cylinder is empty. The systems require the operator to be aware when a cylinder is empty and manually change over to a new cylinder. There are no alarms to remind the operator to change over cylinders, therefore continued chlorination depends on the competency and attendance of the operator.

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 installed as soon as possible and should be connected to the SCADA system, with an associated dial out alarm to reduce the potential response time if a chlorine cylinder should run out when the plant is unattended.**

Photo of Chlorine Room

Photo of Chlorinator

Photos of Chlorine Dosing System

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. Some of the requirements of the Standards which the Moura WTP system may not fully meet are shown in the table below. The review is undertaken on the basis that the chlorine qualities stored/ used are:

- 1200 kg total quantity onsite (4 x 70 kg cylinders plus 1 x 920 kg drum)
- 990 kg total quantity connected to withdrawal system (1 x cylinder plus 1 x drum)

Australian Standard Requirements for Chlorine Installations

Clause	Reqts Met?	Comments
All installations shall be appropriately secured against unauthorised persons (Clause 1.8)	Unsure	Security fence and lockable chlorine shed meet this requirement, however these are left open throughout the day, which may be a risk
The installation shall be located at least 15m (for <1000 kg) to 25m (for >1000 kg) away from a public place (Clause 3.3.1.1)	Unsure	The front fence-line is around 14m away from the chlorine shed. Shed walls will act as a vapour barrier, so actual separation distance is greater
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) Sign restricting entry ('Authorised personnel only') also required (Clause 6.4.1)	No	These specific signs have not been fitted.
Natural ventilation/ mechanical 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. Mechanical ventilation requires capacity 15 m^3/min, with suction intake near floor level (Clause 3.5.3)	No	Natural ventilation inadequate when door closed. Mechanical ventilation fan is located high on back wall of shed and unlikely to be effective. A pedestal fan is used continuously to enhance air flow.
Leak detectors shall be installed where chlorine is stored in tanks or where liquid chlorine is withdrawn. Leak detectors shall be tested each week (Clause 4.8.1)	No	Leak detector installed in suitable place but not tested regularly
Movement of cylinders on site shall be performed using a suitable trolley that has been designed to hold cylinders securely in place by a chain or clamp (Clause 5.7)	No	Current practice of 'rolling' cylinders does not meet requirements. Trolley should be provided
Eyewash and supply of clean water shall be available (Clause 5.12.2)	Unsure	Hose available outside chlorine shed. Eyewash located in chemical shed (around 15m from chlorine shed). Eyewash could be provided outside chlorine shed

As seen above, the installation does not meet some of the requirements of the Standards. **Ideally, the chlorine installation should be improved to meet the Australian Standards, as outlined in the table above.**

5.1.7 Lime

Lime is supplied in 20 kg bags. It is batched manually into a standard concentration dosing solution.

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. Varying purity	
	Makeup and dosing system components	Bag unloading chute Solution mixing tank Dosing pump	Manual loading into tank
	Number of makeup systems	2 solution tanks, used as duty/ standby	When one tank is empty, standby tank is brought on line and the empty tank refilled
	Storage capacity	8 – 10 pallets of lime in 20 kg bags	Each pallet has 1 tonne
	Bag unloading arrangement	Bags manually unloaded into solution tank	No dust extraction system
	Batching concentration (g/L)	100 g/L (10% w/v)	24 x 20 kg bags per full tank
	Solution tank capacity (L)	4830 L	
	Dosing pumps: No. of, Capacity (L/h)	2 (1 duty pre-lime, 1 duty post-lime), 283 L/h	Pump capacity from name plate
	Service water supply		

The lime batching system reportedly runs well most of the time, although blockages occur occasionally. The lime system is set up to automatically flush the pump and dosing line with service water both periodically during operation and on plant shutdown and start up.

Photo of Lime Tank and Mixer

Photo Showing Dilution Water and Flush Lines for Lime System

The lime pump and tanks are inadequately bunded. **An appropriately sized bund is required for the tanks and pump.**

The lime tank level is measured by using a 'dipper stick' marked with graduations. While rudimentary, this method allows the operators to measure the amount of water and chemical to add to top up the tank level. A less operator intensive method of measuring the tank level for chemical makeup calculations could be used, **such as providing a sight-tube or electronic level measurement device**. The dipping stick could be retained as a backup level measurement.

It is noted that there is one pre-lime and one post-lime dosing pump. Pre-lime is not generally used therefore this pump can be used as a standby.

Lime is similar to alum in that it requires manual handling by the operators in lifting the bags onto the unloading platform and emptying them into the solution tanks. **Manual handling and contact with the chemical would ideally be minimised in any planned lime system improvements.**

There is no dust extraction system for the bag unloading system. **A system for minimising dust escape should be provided.**

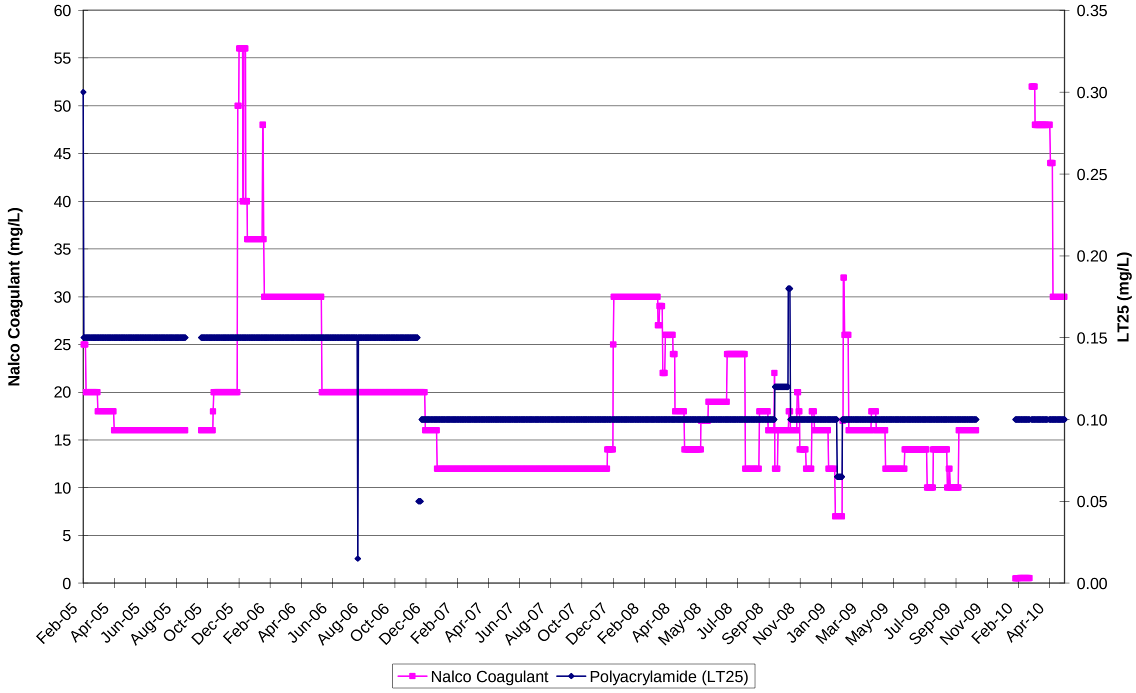
Photo of Lime Storage

Photo of Lime Unloading Chute

5.2 Chemical Doses Used

5.2.1 Nalco Coagulant (DVS1 C001-D245)

The doses of Nalco coagulant used since this coagulant was implemented in March 2005, as reported in operational data, are shown on the graph below. Polyacrylamide doses (as reported) are also shown in the graph. The doses shown in the graph are the numbers reported as 'Setting' on the WTP data sheets. It is understood Council staff that these settings represent the actual dose rate as determined in jar tests. **Coagulant dose rates and maximum pump capacity should ideally be confirmed by further testing**.



Graph of Coagulant Dose Levels

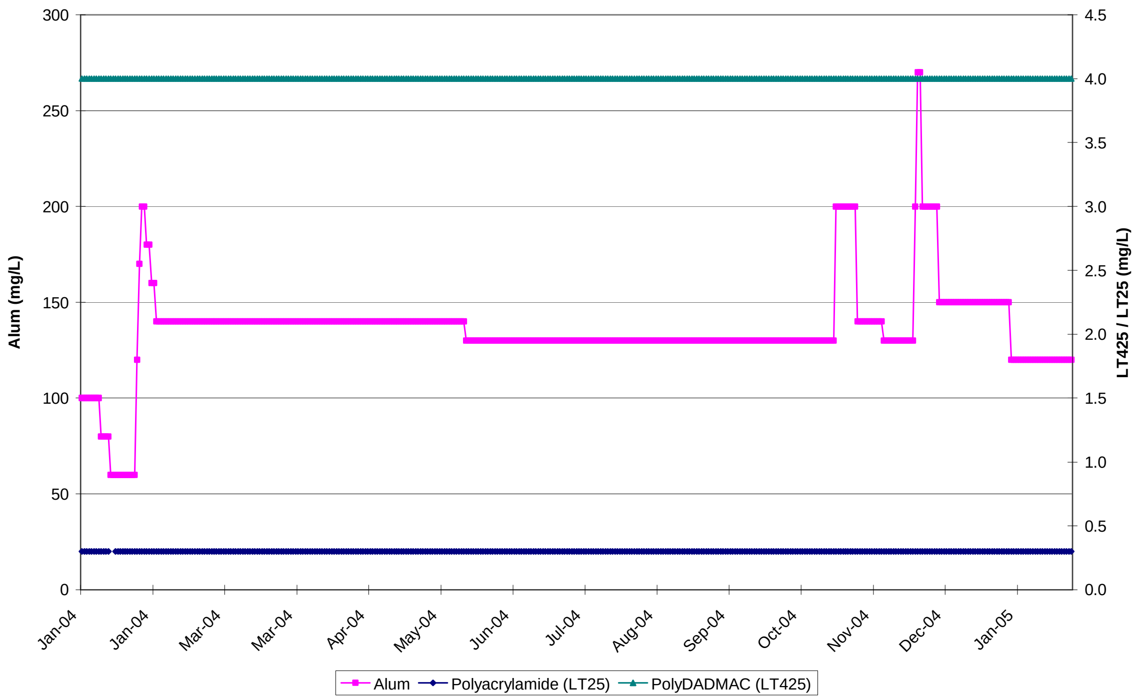
The graph shows that the Nalco coagulant dose remained at around 12 to 30 mg/L for most of the period shown, and peaked at up to 56 mg/L, coinciding with summer turbidity peaks.

It is understood that jar testing is performed regularly and the Nalco coagulant doses are adjusted accordingly.

The polyacrylamide dose ranged from 0.10 to 0.18 mg/L during the period shown in the graph.

5.2.2 Alum and Cationic PolyDADMAC (LT425)

The doses of alum and cationic polyDADMAC used up to March 2005, as reported in operational data, are shown on the graph below. Polyacrylamide doses (as reported) are also shown in the graph.



Graph of Alum, Polydadmac and Polyacrylamide Dose Levels

Alum doses ranged from 60 to 270 mg/L, with a typical dose of around 130 mg/L. Peak doses coincided with turbidity spikes in the river.

High alum doses are likely to reduce the pH of the water significantly, however it was noted from available data that when peak doses were used on 14/12/04 and 15/12/04, the reported dosed water pH was in the typical range at 7.8 and 7.6 respectively. It is likely that pre-lime was dosed at these times to compensate for the higher alum doses. The lowest pH events occurred at the time of alum doses around 200 mg/L.

Cationic polyDADMAC doses were reported to be around 4 mg/L for the entire period reported. This dose rate is at the high end of the range of cationic polyDADMAC doses normally applied.

5.2.3 Polyacrylamide (LT25)

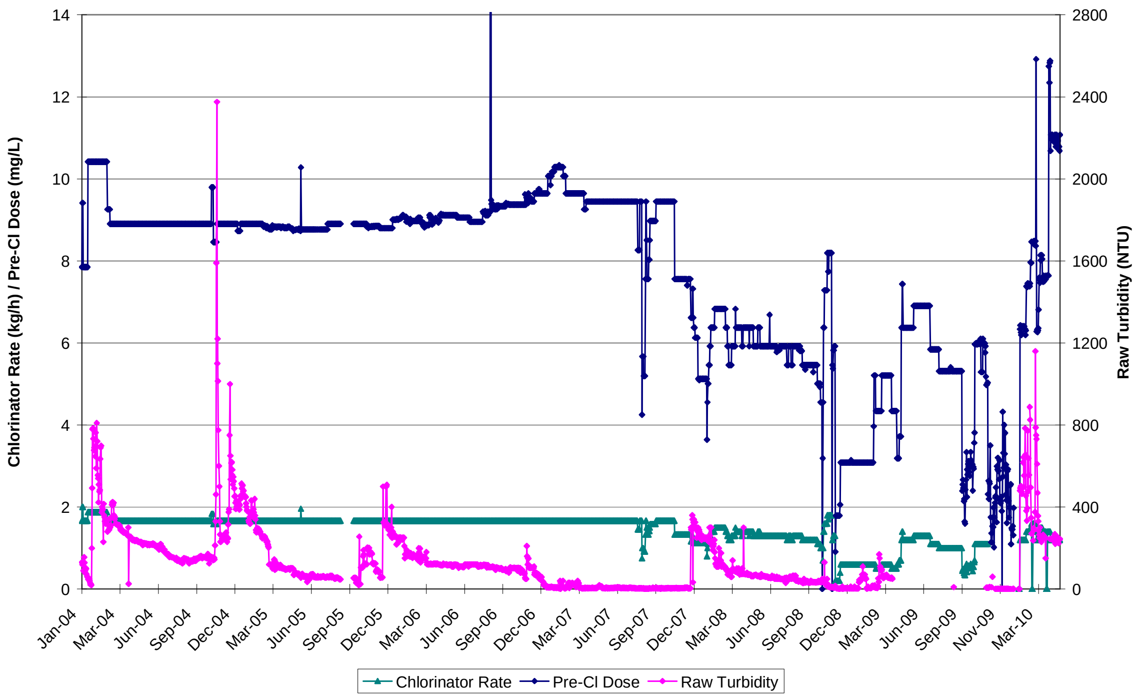
The polyacrylamide (LT25) flocculant aid doses reported in available data are shown on the coagulant dosing graphs in section 5.2.1 above. A dose of 0.3 mg/L was used up until March 2005. For the period shown in the graph, the dose was dropped to 0.15 mg/L and then 0.10 mg/L after the change from alum and polyDADMAC to Nalco coagulant was made, with a brief spike to 0.18 mg/L in October 2008.

5.2.4 PAC

The typical PAC dose is reportedly 0.5 to 1 kg per filter per run, added manually to the surface of the filter after backwashing. At times the dose has been increased up to 8 kg per filter per run, however this resulted in high headloss and short run times in the filter. As a guideline, the operator feels that a dose of 1 kg per filter per run is an appropriate maximum dosage giving reasonable run times.

5.2.5 Pre-Coagulation Chlorine

The graph below is based on available operational data and records of pre-coagulation chlorine dose rates. The chlorine dose (mg/L) was calculated based on the reported chlorine dosing rate (kg/day) and the treated water flow rate. Raw water turbidity levels are included in the graph.

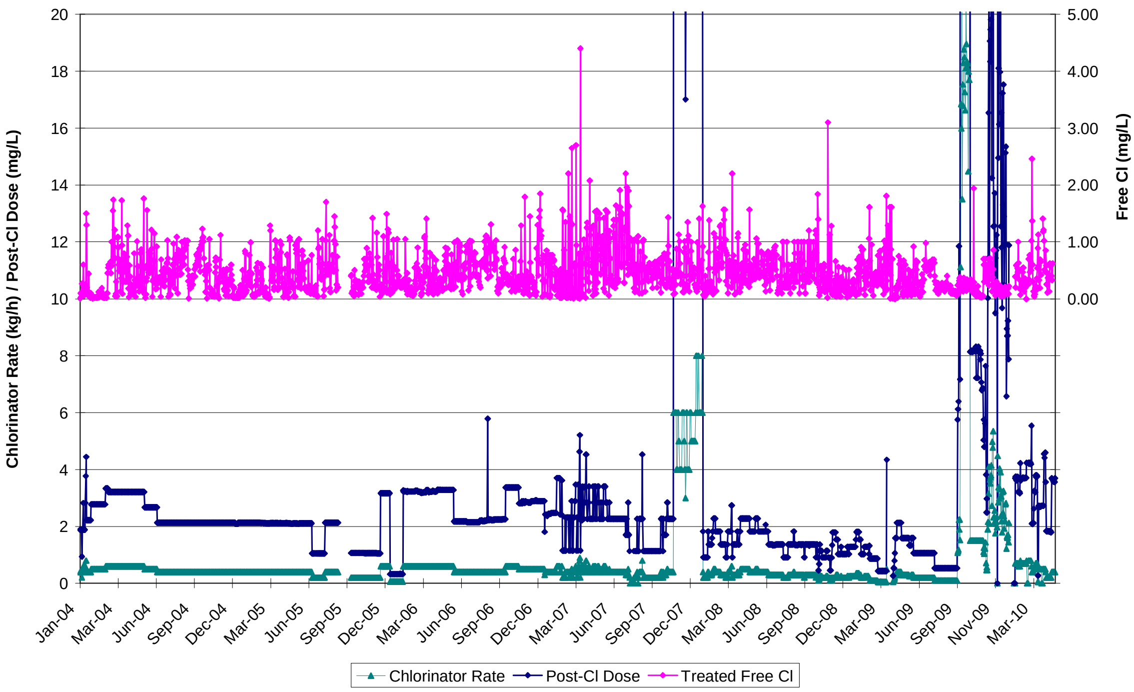


Graph of Pre-Coagulation Chlorine Dose Parameters

As seen in the graph, the pre-chlorine dose has mainly varied between approximately 1 and 11 mg/L. Up to the end of 2007 it was generally constant at around 9 mg/L, with little change in response to raw water turbidity peaks which would be expected to give a higher chlorine demand. From 2008 onwards, the dose was lower and more varied, ranging between 2 and 8 mg/L. The dose was then increased to around 11 mg/L again during early 2010.

5.2.6 Post-Filtration Chlorine

The graph below is based on available operational data and records of post-coagulation chlorine dosing rates. The chlorine dose (mg/L) was calculated based on the reported chlorine dosing rate (kg/day) and the plant flow rate. Treated water free chlorine residual measurements are also shown in the graph.



Graph of Post-Filtration Chlorine Dose Parameters

As seen in the graph, the post chlorine dose has ranged from 0.5 to 5 mg/L in most of the data.

During October – December 2007 and September – December 2009, much higher values for the chlorinator rate were recorded. It is suspected that this data was recorded or entered differently and is not a true representation of the dose as such large changes in the chlorine demand are unexpected.

Based on the free chlorine level measurements, it appears that the chlorine demand of the treated water is generally between 0.5 and 4.5 mg/L.

5.2.7 Post-Filtration Lime

No data was available for post-filtration lime dosing. This dose rate is adjusted in relation to the final water pH, however the actual dosing rate is not recorded. Based on Council's 2005 monthly reports outlining the total raw water intake and lime usage for each month, the average lime dose is estimated to be around 40 – 50 mg/L when alum was the main coagulant and 5 – 15 mg/L when Nalco coagulant was used.

5.3 Chemical Dose and System Capacity Summary

5.3.1 Calculated System Dosing Capacities

The current capacities of the chemical systems in mg/L for various plant flow rates were calculated, based on the review of systems outlined above and data shown in the following table. The capacity values are shown in the table, along with options for increasing the capacity of each system.

Chemical Dosing Capacities (Calculated from Available Data)

Chemical	System Parameters		Calculated Dose Capacity (mg/L)		Batch Tank/ Cylinder Usage Time		Options to Increase Capacity (if Required)
	Dosing capacty (L/h)	Dosing Soln (g/L)	52 L/s Plant Flow	75 L/s Plant Flow	Vessel volume (L)	Time to Empty* (h)	
Nalco Coag.	27	1260 (s.g. 1.26)	**182**	**126**	1000	37	Larger pump
Alum	240	240	**308**	**213**	6500	27	Larger pump
Cat Poly-DADMAC	27	109	**16**	**11**	1000	37	Stronger solution, larger pump
Polyacryl. (LT25)	292	1.4	**2.2**	**1.5**	1178	4	Stronger solution, larger pump
Pre-chlorine	1833 g/h	n/a	**9.8**	**6.8**	920	502	Fit larger rotameter
Post-chlorine	600 g/h	n/a	**3.2**	**2.2**	70	117	Fit larger rotameter and connect to 920kg drum
Post-lime	283	100	**151**	**105**	4830	17	Stronger solution, larger pump

* At maximum dosing rate

It is noted from the table above that the capacity of some of the chemical batch/ dosing tanks would not last long at maximum pumping rates, particularly for the polyacrylamide tank which would be emptied in 4 hours. However this time calculation is based on the maximum pumping rates, not on the actual maximum doses which are expected to be significantly lower than the pump capacities (see below).

5.3.2 Chemical Dose Summary and Comparison with Capacities

The actual chemical doses, as discussed above, are summarised in the table below, with the estimated system capacity for a 75 L/s plant flow repeated for comparison purposes.

Actual Doses Used (Estimated from Available Data)

Chemical	Actual Doses Used (mg/L)		Capacity at 75 L/s Plant Flow (mg/L)
	Range	Typical	
Nalco coagulant	16 - 56	18	**126**
Alum	60 - 270	130	**213**
Cationic PolyDADMAC	4	4	**11**

Chemical	**Actual Doses Used (mg/L)**		**Capacity at 75 L/s Plant Flow (mg/L)**
	Range	**Typical**	
Polyacrylamide (LT25)	0.15 – 0.3	0.15 with Nalco coagulant	**1.5**
PAC	-	-	**No dosing system**
Pre-chlorine	1 - 11	7	**6.8**
Post-chlorine	0.5 - 5	3	**2.2**
Post-lime	Not known	10 with Nalco coagulant	**105**

From the capacity and dose values estimated, it appears that most of the chemical dosing pumps have capacity to meet doses equal to the maximum recorded dose at a plant flow rate of 75 L/s.

The current capacity of the pre- and post-chlorine systems, however, would not meet the maximum recorded dose requirements at a plant flow of 75 L/s. The alum pump, if alum were required to be used again, would also not achieve the maximum recorded dose at 75L/s.

Based on this analysis, **capacity upgrade options for the pre- and post-chlorine systems should be investigated to meet likely maximum chemical demands at flows of 52 L/s or higher.** It is noted that some increased capacity may be achieved by fitting larger rotameters on the chlorinators, however for the post-chlorine the offtake from the 70 kg cylinder will be limited to around 700 g/h at 10ºC and less at lower temperatures. Thus further measures such as modifications to the drawoff arrangement and/ or heating of the chlorination room are also likely to be required.

The alum pump would also need to be enlarged if alum were to be dosed again at a plant flow rate of 75 L/s.

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 at present fully manual backwashing. The existing system is relatively simple, but requires greater operator input than more automated systems. **Improvements such as flow pacing could be considered in any upgrades.**

As discussed in the previous section on Filter Backwashing, the Stage 1 and 2 backwashing controls should be repaired and/or upgraded to allow the automatic backwashing sequence to be used again.

6.1.2 SCADA System

A RADTEL brand SCADA system common to all Council WTPs has been implemented over recent years. The Moura 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:

- Treated Water Pump Control: The town system pressure reading is used to generate a start/stop signal to the WTP treated water pressure pumps;
- 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 the SCADA (via radio telemetry). It is noted that the raw pumps have a 'soft start' setup, where one pump starts up before the other;
- Chemical Dosing Systems Control: The chemical dosing systems are programmed to start up at the same time as the raw water pumps (believed to be directly electrically triggered, rather than using a signal from a flow or level switch). It is noted that the post-filtration chemicals are delayed on start up: the lime dosing system performs a flush cycle on startup, and the chlorine dosing system startup is automatically delayed by 2 – 3 minutes.

It is noted that some of the valves on the flow path through the WTP remain open when the plant is off. The Stage 1 filter outlet valves (manually operated) remain open when the plant shuts down, whereas the Stage 2 filters are automatically closed when the plant shuts down.

The control systems for the raw water and treated water pumps are reported to work effectively, although the treated water pumps may not always achieve the setpoint water pressure in all parts of the system, as noted in the above section on the Clear Water System.

The detail of the electrical control logic for the chemical dosing systems start-up was not identified. Ideally the control should make sure that the chemical systems will not start up unless it has been confirmed that there is flow through the plant, to prevent the risk of overdosing. **The chemical control system should be investigated further to check that the system logic will prevent chemical dosing start-up if there is no flow through the plant.**

6.1.4 Process Impacts of Plant Start-up

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.

The operators feel that flow changes at the Moura WTP due to starting and stopping are not a great problem in terms of process stability and water quality achieved. It is noted, however, that **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:**

- **altering the start and stop trigger levels; and/ or**
- **fitting VSDs to the raw water pumps and making chemicals flow paced.**

6.1.5 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	**Partially installed – not yet operable**
	Sampling location	Stage 1 filtered water well	
pH	Type	**Planned, but not yet installed**	To be logged to SCADA
	Sampling location	Final water - **Not yet installed**	
Chlorine Residual	Type	**Planned, but not yet installed**	To be logged to SCADA
	Sampling location	Final water - **Not yet installed**	
Raw Water Flow	Type	Magflow meter	Connected to SCADA
	Sensor location	Raw water mains before flash mixing tank	
Treated Water Flow	Type	Magflow meter	‘Consumption’ flow meter connected to SCADA
	Sensor location	Downstream of pressure pumps	

Component	Parameter (Units)	Design Criteria	Comments
Clear Water Tank Level	Type	'Platypus' pressure sensor plus Hi/ Lo level float switches	Level sensor logged to SCADA Hi/ Lo float switches trigger alarms on SCADA
	Sensor location	Clear water tank	
Elevated (Town) Reservoir Levels	Type	'Platypus' pressure sensor plus Hi/ Lo level float switches	Level sensor logged to SCADA Hi/ Lo float switches trigger alarms on SCADA
	Sensor location	Reservoirs	
Filter Headloss	Type	-	No existing meters
	Sensor location	-	

As seen in the table, flow rates, filtered water turbidity, critical tank levels and filter headloss are monitored by on-line instruments. Most of these signals are connected to the SCADA system and thus are understood to be able to be trended.

The online turbidimeter has been partially installed at the Stage 1 filtered water well. **The installation of the turbidimeter should be completed and the instrument connected to the SCADA system.**

It is understood that the provision of on-line instruments for measuring treated water chlorine residual and pH is budgeted and that these instruments would also be connected to the SCADA system. As mentioned in the Water Quality and Chemical System sections above, automatic dose adjustment for lime and post-chlorine could be considered as a long term improvements, once the respective online meters have been installed. To allow for these further chemical dosing automation improvements, **the on-line chlorine and pH instruments selected should be suitable for use in chemical dosing trim control loops.**

It is understood that the operators generally receive information on river flows from upstream in the catchment to warn them when the river flow and raw water turbidity are likely to increase. An online instrument could also be used to more accurately measure any sudden deterioration in the water quality in the river. **The provision of an additional online turbidity meter to monitor the raw water turbidity level could be considered in future**.

6.1.6 SCADA Callout Alarms

The existing critical alarms are programmed to trigger a callout through the SCADA system during hours when the plant is unmanned:

- Power outage (phase failure);
- Raw water pump failure;
- Treated water pressure pump failure;
- Clear water tank low water level;
- Chlorine leak;

The following additional alarms should be included as callout alarms when possible:

- Individual filter high water level (overflow);

- Filtered water turbidity high;
- Treated water chlorine residual high/ low;
- Treated water pH high/ low;
- Clear water tank high level (overflow).

The callout alarms are reportedly currently set up to dial out to the Biloela WTP, which requires the Biloela WTP on-call operator to contact the Moura WTP on-call operator if attendance is required on site. **Council may wish to review this callout arrangement and to provide dial-out capacity to the local operator at Moura if considered more practical.**

It is noted that there is no laptop available for the on-call operator, which relies on the operators using a home computer if it is necessary to dial in after hours to review parameters on the SCADA screen. **An on-call laptop should be provided for operator use.**

6.1.7 Power Failure Protection

It is noted that there is no power backup system for the plant or the raw water pumps in the event of a power failure. It is noted that in the event of power failure, the treated water pressure pumps would not be available to pump water stored in the WTP clear water tank into the town system. Considering the limited town storage volume compared to the rising demands of the town, an extended power failure could present a significant risk of water supply failure. **Council should consider the issue of power failure in terms of providing additional system storage volume and/ or a backup power supply to power at least the treated water pressure pumps.**

It is noted that if the power fails, the plant stops but part of 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, **either all the filter inlet valves would be set to 'fail closed' so that water could not pass through the filters when the plant was off, or the controls of the raw water pumps should be interlocked so that the pumps will stop if the signal from the plant is lost.**

In the case of a power failure at the raw water pump station but not at the WTP, **the startup signal trigger for the chemical dosing systems should also be investigated, to confirm that there is no risk of the chemical dosing systems starting up when there is no flow through the WTP.**

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 many of the chemical solution tanks and dosing pumps in the chemical shed are not appropriately bunded. Chemical spills on the shed floor would be a health hazard. And it is likely that large spills would find their way outside the shed and potentially enter the environment.

It was noted that a spill kit containing equipment for cleaning up and isolating chemical spills is provided at the plant.

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

Manual handling issues identified on the plant include:

- Manual handling of all bagged chemicals on delivery;
- Manual handling of lime and alum bags for batching;
- Manual handling of PAC and dosing procedure requiring buckets to be carried up the stairs to the filters.

Any upgrades of the chemical systems should consider ways to further minimise manual handling requirements for the operators.

It is noted that mechanical equipment is provided for lifting the pallets onto the unloading platform. This helps to lift the bags more quickly and easily onto the platform, but does not remove the requirement for the manual lifting of bags to empty the contents into the unloading chutes.

In particular at Moura WTP, the unloading chutes are of a design where the operator needs to hold the bag above the chute while it is being unloaded, which is very unsatisfactory in terms of safety and manual handling. **At minimum, the design of the unloading chutes for lime and alum should be upgraded to allow easier and safer unloading of chemicals.**

6.2.3 Contact with Chemicals

Because chemicals are batched 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.

Dust masks and other PPE should be worn when handling chemicals.

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

6.2.4 Stairways and Unguarded Platforms

There a number of stairways and platforms which may not conform to modern safety standards. These include:

- The chemical unloading platform – Incomplete railing on chemical loading side and no railing on chemical tank side;
- Clarifier and filter platforms – Incomplete railings, no kickboards;

Railings in general should be checked against relevant standards and improvements to increase safety progressed as soon as possible.

Photo of Railings Around Clarifier

In particular, the chemical unloading platform where lime (and previously alum) is batched does not have a railing and presents the risk of falling from a height and/ or falling onto machinery or into the chemical tanks. This arrangement is considered unsatisfactory. **At the minimum, an effective railing system should be installed around the chemical dosing platform as soon as possible.**

Photo Showing Unguarded Chemical Dosing Platform

6.2.5 Laboratory and Office Facilities

The laboratory area provided at the WTP is small and rudimentary, but appears to serve the purpose as a place for daily water testing and jar testing. It is noted that the room is not air conditioned, which may be a concern for operator comfort and for the storage of reagents and instruments.

The office is located in a demountable shed. The level of security for computers and other equipment stored in the office shed may potentially be a concern.

It is noted that a security fence has been erected around the plant area. The plant gates should be locked when the site is unattended.

The office and laboratory facilities could be improved to provide better operator comfort and security. If a major upgrade of the plant is undertaken, it should include the addition of an air conditioned control room, office and laboratory.

7. WTP and System Upgrade Requirements

7.1 Upgrades to Achieve Higher Capacity

The need to upgrade the WTP or individual process components to achieve higher capacity is assessed below for the 'intermediate', 'ultimate' and 'maximum ultimate' demand timeframes developed in Section 2 of this report, considering the current flow rate capacities identified in Sections 3 and 4. The hydraulic capacity of the plant is understood to be at least 77 L/s, based on the 8 hour QNP pump commissioning test carried out in 1999/2000.

The table below shows the expected capacity upgrade requirements for each component of the WTP. Required inflow capacities for each demand timeframe are conservatively based on the raw water inflows required to allow for 20% water losses through the WTP process (as determined in section 2.4.2). The sizings of components downstream of the raw water delivery system would also be relevant in the event that a WTP wastewater treatment system was installed to recycle some of the water from the backwash and desludge waste streams, as water losses through the process would be reduced leading to lower raw water inflow requirements however the capacity of most WTP components would need to account for the addition of the recycled water stream (generally up to 10% of inflow) at the head of the works.

Upgrades not directly related to capacity issues are not included in the table below but are detailed in the following sub-sections of the report.

Expected WTP Capacity Upgrade Requirements

Parameter	Existing WTP Facilities	Expected Upgrade Requirements		
		Timeframe 1: Intermediate	Timeframe 2: Ultimate	Timeframe 3: Max Ultimate
Required WTP Inflow Capacity (L/s)	62 (proved) 70 – 75 (design)	**57** (*Alternative sizing: 80)	**71** (*Alternative sizing: 100)	**78** (*Alternative sizing: 110)
Raw Water Pump Station Upgrades	52 with 2 duty pontoon pumps	Add third pontoon pump 3 duty pumps to give 57 L/s flow rate**	Add third pontoon pump Confirm 3 duty pumps would give 71 L/s flow rate**	Add 2 more pontoon pumps or alternative arrangement to give 78 L/s flow rate**
Raw Water Main Upgrades	77 L/s based on QNP test	Upgrade not required	Upgrade not likely to be required	May need to upgrade/ duplicate main
WTP Inlet and Flash Mixing Tank	Designed for 70 - 75	Upgrade not required	Confirm chemical mixing time requirements met at 71 L/s	Upgrade to increase hydraulic capacity and chemical mixing time for 78 L/s
Clarifier Upgrades	Clarifier 1: < 25 L/s Clarifier 2: ≤ 50 L/s	Capacity upgrade not required	Further optimisation/ upgrade of clarifier 1 likely to	Upgrade or replace Clarifier 1 with new higher rate clarifier

Parameter	Existing WTP Facilities	Expected Upgrade Requirements		
		Timeframe 1: Intermediate	Timeframe 2: Ultimate	Timeframe 3: Max Ultimate
			be required	
Filter Upgrades	4 filters: Max. 75 L/s (18.75 L/s each)	Capacity upgrade not required	Capacity upgrade not likely to be required. Check flow distribution between filters	Comprehensive refurbishment and/ or additional filter may be required
Filtered Water Pipe and Weir Upgrades	77 L/s based on QNP test	Upgrade not required	Upgrade not likely to be required	Check hydraulic capacity of pipework
Clear Water Tank Upgrades	2.4 ML (8.1h storage at 75L/s)	Capacity upgrade not required	Capacity upgrade not required	Capacity upgrade not required
Treated Water Pump Station Upgrades	93 L/s	Capacity upgrade not required	Capacity upgrade not required	Capacity upgrade not required
Chemical Dosing System Upgrades	See Section 5 of this report	Upgrade capacity of pre- and post-chlorine systems	Upgrade capacity of pre- and post-chlorine (and alum if used again)	Upgrade capacity of pre- and post-chlorine (and alum if used again)

* WTP sizing based on MDMM. Alternative sizing based on 2 x AD. Refer to Section 2.4.2 of this report for further explanation.

** Reduced raw water delivery system sizes for intermediate, ultimate and maximum ultimate timeframes of 52, 65 and 72 L/s respectively could be applied if a WTP wastewater treatment and recycling system was added to reduce the water losses through the WTP process to 10%.

As shown in the table above:

- In the ‘intermediate’ timeframe, the third raw water pump will be required however the capacity of the other WTP process components is expected to be adequate. The capacity of the chlorine systems should be increased to achieve the maximum potential required doses. It is understood that the filters perform adequately at existing flow rates of up to 63 L/s, however their performance should be confirmed by further monitoring when more online instruments are available.
- In the ‘ultimate’ timeframe, in addition to the intermediate timeframe improvements above, further optimisation or upgrade of Clarifier 1 is likely to be required to achieve effective settling at the increased flow rates. Current filter performance is expected to be maintained, however the flow distribution between the filters should be checked to ensure each filter is taking similar flow rates.

If the ‘maximum ultimate’ conditions occur, the original design capacity of the WTP will be exceeded and most process components are likely to need some upgrade or enlargement. The capacity of Clarifier 1 could potentially be increased by upgrading it with settling tubes, otherwise this clarifier may need to be replaced with a new higher-rate clarifier. The capacity of the filters may be increased by comprehensively refurbishing them to include a high rate dual media design, with an alternative option of adding an extra filter. The hydraulic capacity of all components of the plant would also need to be checked.

Capacity upgrade options for the raw water, Clarifier 1 and chemical dosing and refurbishment of the filters are included in the list of potentials for improvement contained in the next sub-section of the report. The provision of a new clarifier and filter, potentially required to meet the 'maximum ultimate' demands, have not been investigated further at this stage.

It is noted that if the alternative 2xAD demand sizing was taken instead of the MDMM approach, the 'ultimate' timeframe design flow rate of 100 L/s would require warrant a whole new stage of the WTP, including new clarifier and filters. This upgrade option has not been considered further within this report.

7.2 Identified Potentials for WTP Improvement

The issues identified for potential improvement of the WTP are tabulated below, with time frame and priority level noted.

7.2.1 Treatment Process and Capacity Improvements

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Removal of organic compounds	Add PAC dosing system or alternative process to remove organics from the raw water	3.3.6	Medium-Long	Medium
Removal of manganese	Carry out further trials of manganese oxidation dosing, in conjunction with optimised particle removal in the filters, to address seasonal manganese issues	3.3.4	Short	**High**
WTP Flow Rate	Trial a higher plant flow rate of 65-75 L/s for at least a week	2.2.1, 4.5.4, 4.6.1	Short	**High**

7.2.2 Water Quality Monitoring Issues

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Manganese removal	Measure total and soluble manganese levels before and after pre-clarifier chlorine dosing.	4.3.5	Short	**High**
Raw water quality	Continue to regularly monitor pesticides and herbicides; Consider these contaminants in incident management procedures developed for raw water contamination events	3.3.5	Short-Medium	Medium
Raw water quality	Formalise a system for reviewing the algal analyses available from Sunwater.	3.3.6	Short-Medium	Medium
Raw water quality	Perform algae and taste and odour analyses when these compounds are detected or likely to be in river	3.3.6	Short-Medium	Medium
Raw water quality	Periodically analyse raw water for *Cryptosporidium* and *Giardia*	3.3.9	Short-Medium	Medium
Filtered water quality	Initiate regular sampling or online monitoring of filtered water turbidity from each filter stage	3.3.1	Short	**Very High**

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Raw and treated water quality	Purchase spare benchtop turbidity analyser to be used in case of failure of the duty analyser	3.3.1	Short-Medium	Medium
Treated water quality	Analyse treated water for THMs and other chlorine byproducts. If high levels are found in the treated water, profiles through the process should be taken to investigate the source of these contaminants	4.3.5	Short	Medium

7.2.3 Online Instruments

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Flow meters	Check the accuracy and calibration of the inlet and outlet flow meters	2.2.2	Medium	Medium
Flow meters	Provide flow meter on both backwash and desludge lines	2.3.1, 4.6.10	Medium	Medium
Flow meters	Install flow meter on the inlet pipe to the Stage 2 clarifier	4.5.1	Medium	Medium
Turbidimeter	Complete installation of filtered water turbidimeter. Link to SCADA system and provide callout alarm	6.1.5	Short	**Very High**
Post-chlorine	Install budgeted online chlorine residual meter. Link to SCADA system and provide callout alarm	3.38, 5.1.6, 6.1.5	Short	**Very High**
pH meters	Install online pH analyser. Link to SCADA system and provide callout alarm	6.1.5	Short	**High**
Headloss	Install differential pressure monitors in filters	4.6.4	Short-Medium	Medium
Turbidimeter	Set up the existing or additional turbidimeters to automatically log individual turbidity trends for all four filters	4.6.4	Medium	Medium
Turbidimeter	Install additional turbidimeter to monitor raw water	6.1.5	Long	Low

7.2.4 Chemical Dosing

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
WTP Flow Rate	Enlarge alum pump systems to cater for WTP capacity of 75 L/s (in case alum used again in future)	5.3.2	Medium	Low

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
WTP Flow Rate	Capacity upgrades for pre- and post-chlorine systems to achieve maximum required dose rates at flows of 52 L/s and higher	5.3.2	Short-Medium	**High**
Chemical control system	Check that system logic will prevent chemical dosing start up if there is no flow through the plant	6.1.3, 6.1.7	Medium	Medium
Chemical control system	Consider improvements such as flow pacing in any upgrades	6.1.1	Long	Low
Nalco coagulant	Install standby coagulant dosing pump	5.1.1	Short	Medium
Nalco coagulant and polyacryl-amide	Confirm dose rates and pump maximum capacity	5.2.1	Short	Medium
Lime (and alum if used again)	Minimise manual handling and contact with chemical. Minimise dust escape	5.1.7, 5.1.2	Medium	**High**
PAC	Install automatic PAC dosing facility if used regularly	5.1.5	Medium-Long	Medium
Post-chlorine	Provide disinfection in the pipeline after the two streams have combined	4.7.2	Short-Medium	Medium
Post-chlorine	Consider automatic dose trimming with planned online chlorine residual measurement	4.7.2, 6.1.5	Short-Medium	Medium
Chlorine	Improve chlorine installation to meet Australian standards	5.1.6	Short-Medium	Medium
Lime	Install a new lime dosing point at the weir box where the treated water from the two stages is blended	4.7.3	Short-Medium	Medium
Lime	Consider automatic dose trimming with planned online pH measurement	4.7.3, 6.1.5	Short-Medium	Medium
Alum	Provide sight tube or electronic level measuring device	5.1.2	Medium	Low
Lime	Provide sight tube or electronic level measuring device	5.1.7	Medium	Low

7.2.5 Raw Water Pump Station and WTP Inlet

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Raw Water Pumps	Install third pontoon mounted pump	4.2	Short	**High**

7.2.6 Clarifiers

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Stage 1 Clarifier	Consider automation of desludge from the bottom scour outlet to assist with the regular removal of sludge from the base of the tank	4.5.2	Medium	Medium
Stage 1 Clarifier	Investigate improved settling – Optimised flocculation and polymer dosing, with option of fitting settling tubes	4.5.2	Medium	Medium

7.2.7 Filters

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Filters	Upgrade filter operations and callout alarms to prevent filter overflows	2.2.2	Short	**Very High**
Filter backwash	Upgrade filter control system to allow automatic initiation of backwash on trigger levels for time, headloss, filter high water level, and ideally on filtered water turbidity.	4.6.6	Short-Medium	Medium
Stage 1 Filters	Perform detailed investigation to determine cause of media loss. Refurbish filter media	4.6.2	Medium	**High**
Filters	Check and refurbish filter flow meters and investigate flow distribution between filters by considering flow meter chart trends	4.6.4	Short	Medium
Filters	Investigate options to upgrade or replace filter flow control systems to accurately control levels and flow rates and ensure a steady flow rate through the filters	4.6.4	Short-Medium	Medium
Filter backwash	Review backwash phasing and rates in detail. Test water rates by rise rate test if possible.	4.6.7, 4.6.10	Short	**Very High**
Filter backwash	Test and establish system for backwashing using the treated water pumps or, if backwashing using the rising main pressure is unsuccessful, urgently install standby backwash pump or ideally two new backwash pumps	4.6.8		
Filter backwash	Repair valve controls	4.6.9	Short	Medium

7.2.8 Clear Water Tanks and Treated Water Pumps

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Clear Water Tank	Investigate operational levels and risk of short-circuiting through the tanks to confirm disinfection contact times	4.8	Medium	Low

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Treated water pumps	Optimize the performance of the treated water pumping station, if required by findings of water pressure investigations	4.8	Short-Medium	Low

7.2.9 General

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Wastewater system	Review issues associated with current disposal of WTP wastewater, including effect of Nalco proprietary coagulant on wastewater quality	4.9	Medium	Low
Wastewater system	Identify and implement the most appropriate type of wastewater holding, treatment and recycling facilities	4.9	Medium	Medium
Training	Provide ongoing training for operators and other staff in various SCADA capabilities	6.1.2	Medium	Medium
Alarms	Review call-out arrangement and consider providing dial-out capacity to the local operator at Moura	6.1.6	Medium	Medium
SCADA	Provide on call lap-top for operator use	6.1.6	Medium	Medium
Power failure	Consider providing additional system storage volume and/or backup power supply for treated water pumps	6.1.7	Medium	Low
Power failure	Set all the filter inlet valves to ‘fail closed’ so that water can not pass through the filters when the plant is off, or interlock the controls of the raw water pumps so that the pumps will stop if the signal from the plant is lost	6.1.7	Medium	Medium
Bunding	Upgrade all bunding of all chemical storage and dosing/batching tanks to conform with standards on design and materials.	5.1.1, 5.1.2, 5.1.4, 5.1.7, 6.1.7	Medium	Medium
Alarms	Attach callout alarms to clear water tank high level (overflow)	6.1.6	Medium	Medium
Manual Handling	Upgrade alum and lime unloading chutes to reduce manual handling.	5.1.2, 5.1.7, 6.2.1	Medium	**High**
Stairs/ platforms	Ensure railings are checked against relevant standards and improved where necessary	6.2.4.	Medium	Medium
Stairs/ platforms	Install effective railing system around chemical dosing platform	6.2.4	Medium	**High**

System	Issue/ Requirement	Refer Section No.	Time Frame	Priority
Laboratory	Improve laboratory and office facilities for operator comfort. Any major upgrades should include addition of air conditioned control room, office and laboratory.	6.2.5	Medium-Long	Medium
Nalco coagulant name	Refer to the Nalco coagulant product using the first part of the number, DVS C001, rather than D245	4.3.2	Short	Low
Start-ups	If the impact of start-ups is seen to be limiting further optimisation of the plant process, design control to limit number of daily startups.	6.1.4	Long	Low

7.3 High Priority Upgrade Requirements

7.3.1 High Priority, Short Timeframe Actions

The following improvement works are considered very high priority and should be carried out urgently:

- Install online turbidity and chlorine monitoring and link to SCADA trends and callout alarms– It is understood that Council are currently (June 2010) taking steps to purchase and install these online meters;
- Initiate regular sampling or install online monitoring of filtered water turbidity from each filter stage;
- Update filter control system to prevent filter overflows – This is likely to require connection of high level sensor to PLC/ SCADA and programming to create callout and/ or shut down alarms;
- Test and establish system for backwashing using the treated water pumps and, if backwashing using the rising main pressure is unsuccessful, install new/ refurbished standby backwash pump or ideally two new backwash pumps. In order to do this, backwash phasing and rates should be investigated in detail and the required rates confirmed by rise rate tests or other means.

The following improvement works are considered high priority and should be carried out as soon as possible:

- Install online pH monitoring and link to SCADA trends and alarms – It is understood that Council are currently (June 2010) taking steps to purchase and install this online meter;
- Install third pontoon mounted pump to provide standby capacity on the pontoon and to allow trials of higher flow rates;
- Trial higher plant flow rates of 65 – 75 L/s, when suitable raw water pump combinations available, for a duration of at least a week to confirm WTP performance at or near the design flow rate;
- Carry out further trials of manganese oxidation dosing in conjunction with optimised particle removal in the filters. When elevated raw water manganese levels are experienced, take regular measurements of total and soluble manganese levels before and after pre-clarifier chlorine dosing to confirm the effect of this process on the oxidation of manganese;

- Upgrade pre- and post-chlorine dosing systems to achieve higher capacity at flows of 52 L/s and higher - This may include purchase of larger rotameters and/ or modification of drawoff from cylinder/ drum and room heating to achieve higher discharge rates.

7.3.2 High Priority, Medium Timeframe Actions

The following improvement works are considered high priority actions which should be carried out in the medium term:

- For Stage 1 Filters, perform detailed investigation to determine cause of media loss, including confirmation of the existing filter media strata, analysis of backwashing rates, inspection of the filter floor and underdrains, testing of the underdrains for the presence of sand etc. The filter media should then be refurbished with suitable sand layers, including the missing 1.7 to 3.2 mm coarse sand layer. Underdrain refurbishment may also be required;
- Chemical system upgrades:
    - Install effective railing system around chemical dosing platform;
    - Upgrade lime system (and alum system if to be used again) to minimise manual handling, dust contact and dust escape.

7.4 Medium Priority Upgrade Requirements

7.4.1 Medium Priority, Short Timeframe Actions

The following improvement works are considered medium priority actions which should be carried out in the short to medium term:

- Install standby coagulant dosing pump – It is understood that a suitable spare coagulant dosing pump is always kept available on site. However, ideally a standby pump should be installed to provide rapid changeover if the duty pump fails;
- Filter checks and refurbishments:
    - Check and refurbish filter flow meters on each filter then investigate flow distribution between filters by considering flow meter chart trends;
    - Investigate options to upgrade or replace filter flow control systems to accurately control levels and flow rates and ensure a steady flow rate through the filters;
    - Install differential pressure (headloss) monitors in each filter;
    - Repair filter backwash valve controls;
    - Set up the existing or additional turbidimeters to automatically log individual turbidity trends for all four filters;
    - Upgrade filter control system to allow automatic initiation of backwash on trigger levels for time, headloss, filter high water level, and ideally on filtered water turbidity;
- Chemical dosing improvements:
    - Install a new lime dosing point at the weir box where the treated water from the two stages is blended;
    - Modify the post-chlorine dosing point to provide disinfection in the pipeline after the two filtered water streams have combined;

    - Consider automatic chlorine dose trimming with planned online chlorine residual measurement;
    - Consider automatic lime dose trimming with planned online pH measurement;
    - Improve chlorine installation to meet Australian standards;
- Confirm dose rates and maximum pump capacity for Nalco coagulant and polyacrylamide;
- Analyse treated water for THMs and other chlorine byproducts. If high levels are found in the treated water, profiles through the process should be taken to investigate the source of these contaminants;
- Analyse water for algal toxins and taste and odour compounds if these contaminants are detected or likely to be in the river water;
- Formalise a system for reviewing the algal analyses available from Sunwater;
- Continue to regularly monitor pesticides and herbicides. Consider these contaminants in any incident management procedures developed for raw water contamination events;
- Periodically analyse raw water for *Cryptosporidium* and *Giardia.*
- Purchase spare benchtop turbidimeter.

7.4.2 Medium Priority, Medium to Long Timeframe Actions

The following improvement works are considered medium priority actions which should be undertaken over the medium to long term:

- Automatic PAC dosing system or alternative process – If required, the preferred option for a PAC dosing system would be to size the system to dose at least 60 mg/L to address the potential need for algal toxin removal, equivalent to around 200 - 300 kg/day for WTP flows of 3.4 - 5 ML/d. The PAC dosing point would need to be upstream of the clarifiers to remove the high solids levels.
- Flow meter upgrades or improvements:
    - Check the accuracy and calibration of the inlet and outlet flow meters;
    - Install flow meter on the inlet pipe to the Stage 2 clarifier;
    - Provide flow meter on both backwash and desludge lines;
- Clarifier 1 improvements – Consider automatic sludge drawoff from scour valve to improve performance. Investigate improved settling by optimisation of polymer dosing and flocculation, with the option of fitting settling tubes to improve clarifier capacity. It is noted that further flocculation time could be achieved, if required, by modifications to the inlet tank;
- Wastewater potential upgrades:
    - Review issues associated with current disposal of WTP wastewater, including effect of Nalco proprietary coagulant on wastewater quality;
    - If required to reduce water losses through process or minimise releases of water from WTP site, identify and install most appropriate type of wastewater holding, treatment and recycling facilities. This may include sludge lagoons or a thickener on part or all of the waste streams.
- SCADA and control improvements:

    - Provide ongoing training for operators and other staff in various SCADA capabilities;
    - Review call-out arrangement and consider providing dial-out capacity to the local operator at Moura;
    - Provide on call lap-top for operator use.
    - Check that control system will prevent chemical dosing start up if there is a start signal but no flow through the plant (e.g. if a power failure or fault occurs at the raw water pump station but not at the WTP);
    - Set all the filter inlet valves to 'fail closed' so that water can not pass through the filters when the plant is off, or interlock the controls of the raw water pumps so that the pumps will stop if the signal from the plant is lost;
    - Attach callout alarms to clear water tank high level (overflow);
- Safety and environmental improvements:
    - Upgrade all bunding of all chemical storage and dosing/batching tanks to conform with standards on design and materials;
    - Ensure all railings are checked against relevant standards and improved where necessary;
    - Improve laboratory and office facilities for operator comfort.

7.5 Low Priority Upgrade Requirements

The following actions are recommended actions, considered lower priority than those actions listed above but nevertheless expected to bring benefits in terms of performance or operability of the WTP. Although listed as low priority under the current climate, some of these issues may become higher priority if conditions or concerns change in future:

- If the impact of start-ups is seen to be limiting further optimisation of the plant process, design control to limit number of daily startups;
- Install additional turbidimeter to monitor raw water;
- Investigate operating levels and risk of short-circuiting through the clear water tanks to confirm effective disinfection contact times are being achieved;
- Optimize the performance of the treated water pumping station, if required by findings of water pressure investigations;
- As protection against power interruptions, consider providing additional system storage volume and/or backup power supply for treated water pumps;
- Enlarge alum pump systems to cater for WTP capacity of 75 L/s (in case alum used again in future);
- Provide sight tube or electronic level measuring device for lime and alum chemical storage tanks;
- Undertake further examination of raw and treated water flows, with regard to water 'losses' to desludge and backwash;
- Consider improvements such as chemical system flow pacing in any upgrades;
- Refer to the Nalco coagulant product using the first part of the number, DVS C001, rather than D245.

7.6 Alternative of Providing a New WTP

An alternative to upgrading the existing WTP is to construct a whole new WTP. It is understood from Council that adequate land could be secured near the existing WTP site. This option has been costed below to allow Council to compare it with the cost of the required upgrades to the existing WTP.

For costing purposes it was assumed that the new WTP would be built to cater for maximum ultimate development, with an inlet flow of 78 L/s, equivalent to 5.6 ML/d inflow to produce 4.7 ML/d outflow (conservatively assuming 20% losses).

If a new WTP or a new stage of the existing WTP was to be designed, a conventional treatment settling process is considered the most appropriate process due to expected wet season turbidities of at least 300 NTU and occasionally up to 1000 NTU or more. Jar testing and further investigations would be recommended to refine the design and size individual components. The design of a new WTP should allow for the inclusion of pre-coagulation oxidation and automatic PAC dosing within the treatment process.

8. Budget Costs for Critical Upgrade Requirements

8.1 Short Timeframe Requirements

Estimated budget costs for the improvements designated as short and short to medium timeframe actions in Section 7 of this report are shown in the table below. Tasks which Council are already investigating are included in the table for reference but have not been costed within this report. The estimated costs do not include GST.

Estimated Short Timeframe Costs

Item	Estimated Cost	Cost Basis
High Priority		
Online meters for turbidity, chlorine and pH – Supply, installation, commissioning	Council investigating	-
Backwash rates and process investigation, including testing of backwashing from treated water main	Council to carry out internally, with external assistance if required	-
Raw water pump – installation of third pump	Council investigating	-
Long term (≥ 1 week) trials of 65-75 L/s WTP flow rates	Council to carry out internally, with external assistance if required	-
Further trials of chlorine for oxidation of manganese. Investigate effects of pre-chlorine dosing and optimisation of filtration		
Confirm dose rates and maximum pump capacity for Nalco coagulant and polyacrylamide		
Control system upgrades to prevent filter overflows – Wiring to connect high level sensor to PLC/ SCADA plus programming of call out/ shut down alarms	$ 5,000	Allowance
New backwash pump (estimated duty approx 450 m^3/h at 5m head), if required after investigation of alternative approaches – Pump supply, installation, commissioning	$ 50,000	CWT estimate based on pump supplier budget price
Pre- and post-chlorine dosing system upgrade to achieve maximum dosing rates at ≥ 52 L/s – Larger rotameters and/ or modification of drawoff from cylinder/ drum and/ or room heating	$ 5,000	Allowance – Further investigation required to confirm upgrade requirements
Medium Priority		
Turbidimeters for individual filters – 4 x new turbidimeters	Council investigating	-
Relocation of lime and chlorine dosing points to weir box or pipeline after the two filtered water streams have combined	Council to carry out internally, with assistance if required	-
Provision of automatic feedback control for	Council investigating	-

Item	Estimated Cost	Cost Basis
chlorine and pH		
Water quality analysis –THMs and other chlorine byproducts, algal toxins, Sunwater algal levels, pesticides and herbicides and *Cryptosporidium* and *Giardia.* Consider pesticides and herbicides in any incident management procedures	Council investigating	-
Spare benchtop turbidimeter	Council investigating	-
Standby coagulant dosing pump – Purchase and installation	$ 5,000	Allowance
Filter refurbishments – 4 x new flow meters, ultrasonic level sensors and DP cells. Supply, delivery and installation of instruments plus connection of instruments to SCADA system	$ 40,000	Estimate based on approx $2500 per level sensor and DP cell and $5,000 per flow meter
SCADA programming to allow automatic initiation of backwash on trigger levels for time, headloss, filter high water level, and filtered water turbidity	$ 10,000 Plus cost of general backwash valve control repairs/ upgrades	Allowance
Chlorine system upgrade to Aust Standards – Improve ventilation, add signs, add eyewash	$ 10,000	Allowance, including $3,000 for eyewash

8.2 Medium to Long Timeframe Requirements

Estimated budget costs for the improvements designated as medium to long timeframe actions in Section 7 of this report are shown in the table below. Tasks which Council are already investigating are included in the table for reference but have not been costed within this report. The estimated costs do not include GST.

Estimated Medium to Long Timeframe Costs

Item	Estimated Cost	Cost Basis
High Priority		
Stage 1 filter investigation, media and underdrains refurbishment - Detailed investigation into media loss issues, refurbishment of filter media. Refurbishment of underdrains if required	$ 30,000 Additional costs if underdrains also need replacing	Allowance for replacement of lost media in two filters plus further investigations
Upgrade of lime and alum systems – Tank platform railings. Supply and installation 2 x manual bag unloading cabinets. Supply and installation	 $ 2,000 $ 30,000	Allowance for railing. Bag unloader estimate based on information from supplier

Item	Estimated Cost	Cost Basis
Medium Priority		
Flow meters calibration check - Inlet and outlet (consumption) flow meters	Recalibration undertaken regularly by Council	-
Clarifier 1 investigation of options to improve settling. Trials of different polymer doses and dosing points, investigation of optimal flocculation time and energy	Council to carry out internally, with assistance if required	-
SCADA upgrades: • Operator training, review of call-out arrangements, on call lap-top; • Control system improvements to prevent chemical dosing start up if no flow through the plant and filter inlet valves to automatically 'fail closed' • Addition of callout alarm for clear water tank high level (overflow)		
Wastewater issues - Review issues associated with current disposal of WTP wastewater, effect of Nalco proprietary coagulant on wastewater quality		
PAC dosing system, if required – Bulk bag unloader, screw feeder system to dose approx 300 kg/day, shed to house dosing system. Design, engineering, installation, commissioning, contingency	$ 300,000	Estimate based on costs determined for Biloela WTP plus further info from supplier
Flow meters upgrade: 3 new flow meters - On inlet pipe to the Stage 2 clarifier, backwash and desludge lines. Supply, installation, commissioning	$ 40,000	Allowance based on supplier estimate of $10,000 per meter plus installation
Clarifier 1 improvements:		
Automation of sludge drawoff from scour valve, if required – Fitting of automatic valve and PLC/ SCADA connections	$ 5,000	Allowance
Retrofit of settling tubes to increase settling rate, if required – Supply and installation of settling tubes, installation of support frame	$ 180,000	Estimate based on similar projects
Wastewater system – Provision of appropriate type of wastewater treatment facilities, as required	$ 200,000	Allowance
Upgrade of bunding, check and upgrade of all railings (except lime and alum tank railings)	$ 10,000	Allowance

Item	Estimated Cost	Cost Basis
Improved laboratory and office facilities – New demountable building	$ 30,000	Allowance

Actions designated as low priority have not been included in the costing tables above, however these actions should still be progressed when possible.

8.3 Costs for a New WTP

In comparison to the costs given above, the cost for a new WTP to treat an inflow of 5.6 ML/d is estimated to be around $4 to 6 million. This cost estimate is for a standard conventional treatment process including basic chemical dosing systems, based on historical data and not including the purchase of land. Costs are expected to vary based on the level of automation, contract type, the requirements for specialised treatment systems such as pre-coagulation oxidation and PAC dosing and other inclusions.

9. Findings and Recommendations

9.1 Findings

9.1.1 WTP Flow Rate and Demand Issues

Moura WTP's original design flow rate was 70 – 75 L/s. The plant has been well proven at raw water flow rates of 52 to 62 L/s, but has only been tested at higher flow rates during an 8 hour test at 77 L/s.

Water demands for the three development timeframes and the corresponding WTP flow rate requirements are shown in the table below.

WTP Demands and Flow Requirements Summary

Flow Parameter	Timeframe 1: Intermediate Development	Timeframe 2: Ultimate Development	Timeframe 3: Maximum Ultimate Development
WTP Sizing based on MDMM (ML/d)	3.40	4.27	4.68
Required WTP Raw Water Flow Rate (L/s):			
If 10% water losses through WTP process:	52.0	65.2	71.5
If 20% water losses through WTP process:	56.7	71.1	78.0

Required treated water production capacity calculated using MDMM basis and 20 hours operation per day, as detailed in section 2.4.2. Alternative basis for capacity calculation discussed in section 2.4.2.

As shown in the table, the forecast 'intermediate' and 'ultimate' development demands are within the original WTP design capacity and are likely to be achieved without major upgrades to the existing WTP. The worst case 'maximum ultimate' development demand exceeds the current WTP design capacity and would require more significant upgrades to achieve the required output.

It is noted that permanent water restrictions may potentially reduce WTP demands for all timeframes, 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:

- Raw water issues include very high turbidity and colour from river flow events, periodic high manganese levels and the presence of herbicides;
- WTP treated water typically meets target values, with periodic excursions on treated water turbidity and true colour. The chlorine residual measured after the clear water tanks is highly variable and manganese targets are also sometimes exceeded;
- Total coliforms have been present in a number of treated water samples, but *E.Coli* has not been detected;
- From modelling of corrosivity potentials, the typical treated water is likely to be only mildly corrosive except under worst case conditions.

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 sized to achieve the design flow of 70-75 L/s although performance under these loadings has not been proven over a long term test. The most limiting process components are the raw water pontoon pumps, which can only supply 52 L/s to the WTP and Clarifier 1 which is generally operated well below its design rate due to poor settling at higher flows.
- Of the chemical dosing systems, the pre- and post-chlorine dosing systems would not achieve the maximum required doses at current flow rates without modifications. The alum pump (no longer used) would not achieve the maximum required doses at WTP design flow rates of 70 – 75 L/s.
- Plant control and automation, safety and maintenance issues were also reviewed, with various recommendations identified. Generally there is a need for more online monitoring and plant automation as well as some safety upgrades.

9.2 Recommendations

It is recommended that all upgrade requirements identified in this report be addressed as practical, with the priority and timeframe assigned in Sections 7 and 8 to be used as guidance for programming works. In particular, the following improvement works are considered very high priority and should be carried out urgently:

- Install online turbidity and chlorine monitoring;
- Initiate regular sampling/ online monitoring of filtered water turbidity;
- Update filter control system to prevent filter overflows;
- Test and establish system for backwashing using the treated water pumps. If unsuccessful, install a new/ refurbished standby backwash pump.

Budget costs for the high and medium priority upgrades are given in Section 8 to assist Council in budgeting for the upgrades. A budget cost has also been provided for the alternative approach of constructing a new WTP.

10. References

- Department of Natural Resources and Mines (DNRM), Planning Guidelines for Water Supply and Sewerage, March 2005;
- Cardno, Water Supply Planning Report – Moura and Banana, January 2007.

11. Appendices

APPENDIX A –WATER QUALITY - EXTERNAL LABORATORY ANALYSIS RESULTS

General Analysis Results for WTP Raw Water

Parameter	Units	2002	2005		2006		2007					2008			ADWG
		19/8	27/9	01/11	10/1	28/11	19/2	16/5	21/8	17/9	12/11	18/2	26/5	18/8	
Turbidity	NTU	160	48	209	485	146	22	3	1	2	5	154	87	37	<1
True Colour	HU	15	19	45	51	21	20	20	1	16	9	62	39	10	<15
pH @ 21oC	-	6.9	7.44	7.22	7.3	7.41	7.36	7.5	7.56	7.84	7.9	7.47	7.61	7.8	6.5 – 8.5
Conductivity	□s/cm	270	340	231	152	297	341	283	316	362	325	171	222	238	-
Total Dissolved Solids	mg/L	170	185	135	99	161	188	153	170	191	178	99	126	127	<500
Total Dissolved Ions	mg/L	190	225	167	123	205	231	191	210	247	229	123	155	161	-
Total Hardness	mg/L CaCO3	91	80	56	35	69	76	66	74	79	71	42	48	57	<60 Soft, may be corrosive
Temp Hardness	mg/L CaCO3	57	80	56	35	69	76	66	74	79	71	42	48	57	-
Alkalinity	mg/L CaCO3	57	86	74	61	88	88	82	80	103	95	62	77	78	-
Silica	mg/L	13	13	14	14	11	12	12	9	7	8	15	18	15	-
Sodium	mg/L	18	32	21	19	27	35	26	30	37	34	13	18	18	<180
Potassium	mg/L	7.4	6.8	6.6	5.3	7.1	8.4	7.6	7.9	7.8	8.2	6.9	7.1	7.2	-
Calcium	mg/L	28	21	15	9.6	18	19	17	19	20	18	12	12	15	-
Magnesium	mg/L	5.2	6.9	4.3	2.6	5.9	7	5.8	6.5	7.2	6.7	3.3	4.4	4.9	-
Hydrogen	mg/L	0	0	0	0	0	0	0	0	0	0	0	0	0	-
Bicarbonate	mg/L	70	104	90	74	108	108	100	97	124	115	75	93	95	-
Carbonate	mg/L	0	0.1	0.1	0.1	0.2	0.1	0.2	0.2	0.5	0.5	0.1	0.2	0.3	-

Parameter	Units	2002	2005		2006		2007					2008			ADWG
		19/8	27/9	01/11	10/1	28/11	19/2	16/5	21/8	17/9	12/11	18/2	26/5	18/8	
Hydroxide	mg/L	0	0	0	0	0	0	0	0	0	0	0	0	0	-
Chloride	mg/L	14.5	49	17	6.9	34	48	32	45	46	43	11	16	17	<250
Fluoride	mg/L	<0.1	0.2	0.2	0.1	0.17	0.19	0.18	0.16	0.2	0.17	0.08	0.11	0.12	<1.5
Nitrate	mg/L	1.2	<0.5	8.3	2	0.8	0.9	<0.5	<0.5	<0.5	<0.5	0.8	0.7	<0.5	<50
Sulphate	mg/L	51	5	4.3	3.3	5	4.7	2.4	3.5	4.7	3.5	1	3.4	3	<250
Iron	mg/L	0.02	0.02	0.04	0.11	<0.01	0.01	<0.01	<0.01	<0.01	<0.01	0.24	0.1	0.06	<0.3
Manganese	mg/L	<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	0.02	0.02	0.02	0.02	<0.01	0.01	0.05	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<3
Aluminium	mg/L	<0.05	<0.05	<0.05	0.11	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	0.22	0.06	0.05	<0.2
Boron	mg/L	0.06	0.04	0.05	0.17	0.05	0.05	0.05	0.07	0.05	0.05	0.03	0.05	0.04	<4
Copper	mg/L	0.11	<0.03	<0.03	0.04	<0.03	<0.03	0.12	<0.03	<0.03	0.04	<0.03	<0.03	<0.03	<1

General Analysis Results for WTP Treated Water

Parameter	Units	2002	2005		2006		2007					2008			2009	ADWG
		19/8	27/9	01/11	10/1	28/11	19/2	16/5	21/8	17/9	12/11	18/2	26/5	18/8	28/1	
Turbidity	NTU	33*	<1	<1	1	3	<1	1	<1	<1	<1	1	3	2	5	<1
True Colour	HU	<1	<1	9	4	9	<1	1	<1	4	<1	2	4	<1	1	<15
pH @ 21^{o}C	-	6.4*	7.41	7.86	7.52	7.53	7.85	7.9	7.66	7.57	7.55	7.62	7.75	7.99	7.71	6.5 – 8.5
Conductivity	μs/cm	265	339	275	192	304	407	322	322	380	342	204	253	261	359	-
Total Dissolved Solids	mg/L	170	184	156	108	164	217	169	179	198	185	111	138	140	182	<500
Total Dissolved Ions	mg/L	170	218	189	128	204	260	207	215	247	231	132	167	175	230	-
Total Hardness	mg/L $CaCO_3$	75	81	75	48	71	96	81	77	81	76	52	59	69	76	<60 Soft, may be corrosive
Temp Hardness	mg/L $CaCO_3$	26.5	77	75	48	71	89	81	77	81	76	52	59	69	76	-
Alkalinity	mg/L $CaCO_3$	26.5	77	75	54	82	89	81	83	92	87	56	76	81	93	-
Silica	mg/L	12	13	14	13	11	12	12	9	7	8	14	18	15	9	-
Sodium	mg/L	17.5	30	21	13	27	37	26	30	37	34	13	18	18	34	<180
Potassium	mg/L	7	7.1	6.5	5.8	7	8.7	7.7	7.8	7.9	8.2	7.2	7.1	7.2	7.8	-
Calcium	mg/L	23.5	22	23	15	19	27	23	20	21	19	16	16	20	20	-
Magnesium	mg/L	4	6.7	4.3	2.7	5.9	7.2	5.7	6.4	7.2	6.8	3.2	4.5	4.8	6.2	-
Hydrogen	mg/L	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-

Parameter	Units	2002	2005		2006		2007					2008			2009	ADWG
		19/8	27/9	01/11	10/1	28/11	19/2	16/5	21/8	17/9	12/11	18/2	26/5	18/8	28/1	
Bicarbonate	mg/L	32.5	94	91	66	100	107	98	101	112	105	68	92	97	113	-
Carbonate	mg/L	0	0.1	0.4	0.1	0.2	0.4	0.4	0.3	0.2	0.2	0.2	0.3	0.5	0.3	-
Hydroxide	mg/L	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-
Chloride	mg/L	17.5	54	29	21	38	67	43	45	57	53	22	25	24	46	<250
Fluoride	mg/L	<0.1	0.1	0.2	0.1	0.16	0.19	0.18	0.16	0.2	0.17	0.08	0.11	0.12	0.15	< 1.5
Nitrate	mg/L	1.1	<0.5	7.8	1.9	1.3	<0.5	<0.5	<0.5	<0.5	<0.5	0.9	0.7	<0.5	<0.5	<50
Sulphate	mg/L	68	4.8	5.7	2.5	5.2	5	2.6	3.6	5	3.6	1.1	3.3	3	2.2	<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	<0.01	<0.01	<0.3
Manganese	mg/L	<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	0.01	<0.1
Zinc	mg/L	0.02	0.02	<0.01	0.03	0.03	<0.01	<0.01	<0.01	0.01	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	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.05	<0.2
Boron	mg/L	0.05	0.04	0.05	0.06	0.08	0.05	0.05	0.06	0.05	0.05	0.03	0.05	0.04	0.05	<4
Copper	mg/L	0.09	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<1

* The turbidity and pH of the sample from 19/8/02 are out of target range for the treated water. Such a high turbidity result is considered unreasonable as no plant failures were reported for this period, and the unusual results may therefore be the result of incorrect labelling or sample contamination.