# BANANA SHIRE COUNCIL





REPORT

# THEODORE SEWAGE TREATMENT AND EFFLUENT DISPOSAL PLANNING REPORT

September 2008
Job No.7612/01

# TABLE OF CONTENTS

## Document Control

| Version | Date | Author | | Reviewer | |
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# 1.0 INTRODUCTION

## 1.1 Wastewater Treatment in Theodore

Banana Shire Council is responsible for the collection, treatment and disposal of wastewater for the town of Theodore. The wastewater is delivered to the wastewater treatment plant located North-East of the town.

The treated effluent is directed to a storage pond, and there after is used to irrigate the balance area of the treatment plant site that is managed by the local branch of the Apex Club to grow commercial crops (usually sorghum or cotton) to raise funds for the Club. There is an emergency overflow pipeline from the treatment plant to Lonesome Creek which ultimately discharges to the Dawson River.

## 1.2 History of the Treatment Plant

The plant was originally built in the late 1960's, the treatment process comprises an Imhoff tank, trickling filter, humus tank and disinfection system which includes a chlorine contact tank. The treatment plant is typical of those built in Queensland in this period.

The effluent management scheme currently incorporates an agreement with the Apex Club for lease of the Council owned land. The agreement does not address environmental and legal issues.

The quality of effluent exiting the storage pond is of low quality with elevated concentrations of suspended solids (SS), and Biochemical oxygen demand (BOD), and algae caused by bird activities and rain flushing.

## 1.3 Objective of this Report

The objective of this Planning Report for Banana Shire Council is to review the current Theodore sewage treatment plant (STP) and provide recommendations for upgrading the system to produce effluent of an appropriate quality for a sustainable effluent disposal/water reuse scheme. The augmentations are required to sustain current and future loadings for the next 10 and 20 years.

This report addresses the following key issues;

- A prediction of the Theodore population growth and future sewage loadings.
- A description of each process unit of the sewage treatment plant and the system as a whole.
- An assessment of the operational efficiency of the STP and its ability to treat the raw sewage to the required effluent quality for current and future loadings.
- Recommendations of process augmentations to achieve to the required effluent quality.
- Recommendations of appropriate and environmentally sustainable recycled water reuse schemes.
- An estimate of the operating and capital cost for the process augmentation recommendations and the water recycling schemes.
- Investigation and review of existing treatment plant operations and processes;
- Undertake physical audit of each plant for actual operation and safety aspects and condition of existing infrastructure;

- Undertake review of daily STP operations and provide recommendations accordingly (operational practices, maintenance practices, process monitoring, sampling/analysis, data capture, etc);
- Consider disposal options for septic tank/grease trap waste, etc, at each plant;
- Review existing sludge disposal practices;
- Determine limiting factors affecting ability to treat and dispose of existing and future plant loadings;
- Assess inflows to STP's and estimate design loads (short, medium, and long term) having due regard to potential development and likely population growth and ultimate development in accordance with Council's draft Town Planning Scheme;
- Undertake a review of existing effluent disposal/re-use practices both on-site, and off-site by Third Parties with respect to:-
- Council's Environmental Authority issued by the EPA and likely updates;
- Queensland Guidelines for the Safe use of Recycled Water;
- Long term sustainability of effluent management practices;
- Requirements of current and potential future third party users;
- Consider effluent quality and suitability for re-use (both now and future having regard to STP upgrades, etc);
- Review irrigation demands, etc, and review land suitability for effluent and sludge disposal (consider site characteristics, crop types, water quality, soil types and profiles, suitability of soils for irrigation);
- Undertake water balance modelling and consider soil nutrient budget for each current/future disposal site;
- Consider effluent storage requirements (available versus required/recommended) including wet weather storage requirements;
- Consider potential impacts on groundwater and surface water;
- Consider relevant Workplace Health and Safety issues;
- Review suitability and sustainability of existing effluent disposal/re-use practices having due regard to long term impacts and EPA requirements/guidelines;
- Investigate additional/alternative use of effluent generated at each STP and consider viability of potential options (eg additional sites, parkland/open space, sporting/recreational venues, etc);
- Meet EPA representatives (Gladstone) together with Council Officers to discuss relevant issues;
- Review compliance with existing environmental authority and likely future compliance requirements;
- Assess existing/future options for effluent reuse and investigate capital and operational costs and make recommendations regarding available effluent reuse options;
- Submit draft documentation for Council comment/review;
- Finalise documentation for incorporation into planning reports.

# 2.0 COMMENTS ON THE SINCLAIR KNIGHT'S REPORT

Sinclair Knight produced the Banana Shire Council tender documents for '*Preparation of Sewage Treatment and Effluent Management Studies for Biloela STP, Moura STP and Theodore STP*', Dec 2004. The document included a scoping study for the three mentioned Sewage Treatment Plants (STPs).

Some of the key issues from the report particular to the Theodore sewage works and effluent reuse scheme are:

- The existing sewage treatment plant is dated technology, and the infrastructure is in poor condition and shows clear signs of structural deterioration.
- Current effluent reuse incorporates irrigation cropping, and in the past effluent has been released into Lonesome Creek which connects to the local Dawson River.
- The soil at the STP site and adjacent land is considered suitable for irrigation.
- The current operational and maintenance expenditure on the Theodore STP is considered low.
- Key issues for the upgrading the treatment process are not necessarily related to increased loading to cater for population growth, but rather to ensure the process incorporates appropriate disinfection for protection of people who may come into contact with the treated effluent (Council workers and Apex Club members) and nutrient/water balance for the effluent disposal to land, which includes irrigation to the Apex Club land and potential other areas such as the recreational reserve.
- A Recycled Water Use Agreement and Recycled Water Safety Plan for Theodore should be considered concurrently with Biloela and Moura.
- A range of environmental issues are required to be addressed to ensure long term sustainability of effluent reuse, including but not limited to training, monitoring and reporting of effluent reuse and effects on land, waterways and public health.

# 3.0 EXPECTED POPULATION GROWTH

The town of Theodore is a small inland commercial country town which is 140m above sea level, and serves the agricultural and pastoral needs for a portion of Banana Shire.

There is minimal information on the population of Theodore obtained from the Census from 1996 and 2001, which was 508 and 420 respectively. This indicates a 4% fall in population in the five years, however there is not enough data that this indicates an overall trend in the population for Theodore.

For the purpose of planning it is relevant to know the population of a community to predict loadings to the sewage treatment plant. There is limited information on actual population, however there is some recorded data of the flows received by the Theodore sewage treatment plant (STP). These flows are taken from the Sinclair Knight Merz Scoping Study report, and are used to predict the equivalent persons load to the sewage treatment plant.

For domestic sewage loading, the Water Resources '*Guidelines for Planning and Design of Sewerage Schemes'*, Vol 1, Sep 1992, pg 5, apply a general loading of 250L/EP/day. Using this loading and the monthly average received flow to the STP, an equivalent persons population figure is calculated. It is not expected that Theodore has a high growth rate, as there are no major commercial industry, housing or tourism development plans. Considering the new town planning scheme, the demand for housing being placed on Theodore by mining workers, and a 37 lot subdivision proposed by Council, it is reasonable to assume a conservative population growth rate of 0.5% per annum.

The expected trend is shown in Table 3.1.

**Table 3.1: Average equivalent persons population for the town of Theodore.**

| Year | Flow (kL/day) | Population from Census | Equivalent Population (EP) |
|---|---|---|---|
| Real Data | | | |
| **1996** | | 508 | |
| **2001** | | 420 | |
| Flow Data | | | |
| **2001** | 163.5 | | 654 |
| **2002** | 191.7 | | 767 |
| **2003** | 181.6 | | 726 |
| Expected Future | | | |
| **2010** | 188 | | 752 |
| **2015** | 193 | | 771 |
| **2020** | 198 | | 791 |
| **2025** | 203 | | 811 |

# 4.0 RAW WASTEWATER

## 4.1 Sewage Reticulation System

The wastewater generated from the town of Theodore is pumped to the Theodore sewage treatment plant for treatment.

## 4.2 Effects of Flow Fluctuations

Fluctuations in the raw sewage arriving at the treatment plant are relevant to the efficiency of the process plant. Fluctuating influent flows affect the hydraulic loadings of all the process units, which inturn affect the treatment efficiency. Ideally a wastewater treatment plant should receive a constant, consistent flow 24hrs a day, 7 days a week for optimal biological and physical treatment.

Treatment units usually have hydraulic load considerations as part of the design. Flows in excess of these design parameters will have a detrimental effect on the treatment performance.

The raw sewage inflow to the plant is controlled by the operation of the inlet station, in which the running and stopping of the pumps is actuated by the water level in the wet well. It is likely that Theodore STP would receive the bulk of the daily volume to the plant during peak times and there would be significant periods throughout the day and night where the plant would receive no flow.

## 4.3 Load Fluctuations and Raw Sewage Quality

### 4.3.1 Hydraulic Load

The volume of sewage received can fluctuate on a daily basis. Wastewater received by the Theodore sewage treatment plant was recorded monthly between May 2001 and March 2004. Figure 4.1 graphically displays the calculated average daily flows for each recorded month as well as the calculated maximum and minimum daily flows.

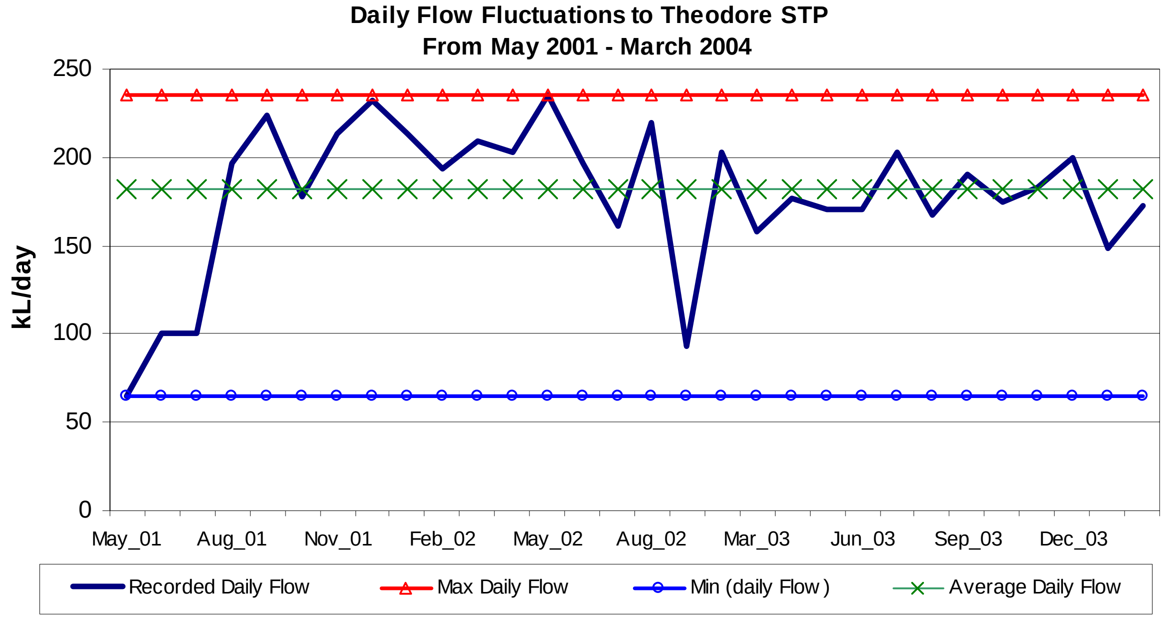



**Figure 4-1: Calculated average daily flow to the Theodore sewage treatment plant.**

Figure 4.1 shows that during mid to late 2001 until mid to late 2002 the water usage was on average higher than 2003 and beyond. This could be indicative of a drop in population, or a more conservative approach to water usage being adopted by the community. As the cause for the decrease is unknown, a water usage of 250 L/p/d has been assumed during the capacity assessment.

Storm water can infiltrate into the sewerage system and during heavy rain fall significantly increase the hydraulic loading to a sewage treatment plant. Figure 4.2 shows the recorded rain fall data for the same period as shown in Figure 4.1, and compares the loadings to the STP and the rainfall to determine if the sewage system suffers from significant stormwater infiltration.

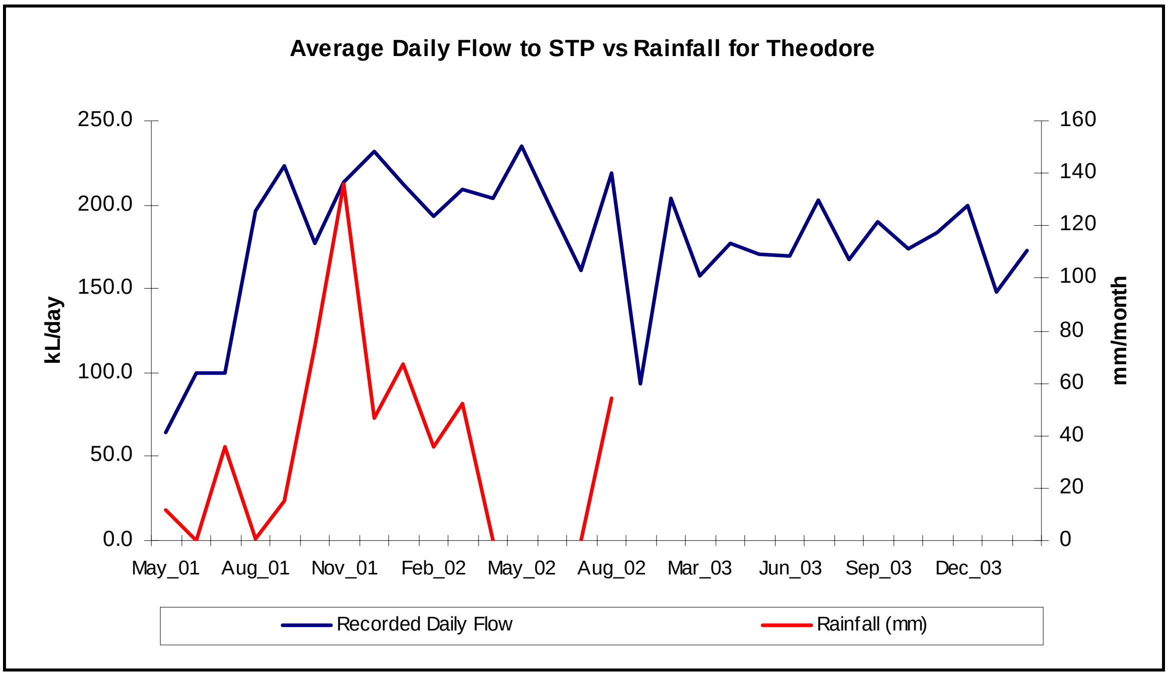



**Figure 4-2: Rainfall Data for the Town of Theodore.**

Figures 4.1 and 4.2 do not show a strong correlation between recorded flow into the sewage treatment plant and the periods of high rainfall, suggesting a low degree of stormwater infiltration to the sewer system.

### 4.3.2 Raw Sewage Quality

There is no current data available that represents the variation of concentrations of the raw sewage during particular times of the day. There are however analytical results for raw sewage quality taken for different months.

Table 4.1 gives a summary of the raw sewage quality received at Theodore STP.

The samples taken were composite grab samples, which are not always indicative of the true average concentration of raw sewage received. The measured concentration of constituents in the raw sewage is considered unusual for a demographic such as Theodore and so at this stage the assessment of the current sewage treatment plant is based on receiving typical medium strength sewage. Further sampling and analysis should be performed on composite 24 hour samples of the raw sewage before detail design of augmentation commences.

**Table 4.1: Theodore Raw Sewage Quality**

| Parameter | Units | Recorded Range | Average Concentration | Typical Medium Strength Value |
|---|---|---|---|---|
| Total Suspended Solids | mg/L | 160 - 270 | 210 | 240 |
| BOD | mg/L | 136 -513 | 236 | 280 |
| pH | - | 7.3 - 7.7 | 7.5 | 6.5 – 8.0 |
| Ammonia as N | mg/L | 49 - 53 | 45 | 40 |
| Nitrate as N | mg/L | < 0.1 | < 0.1 | < 0.1 |
| Total Nitrogen as N | mg/L | 48 - 72 | 59 | 55 |
| Phosphorous | mg/L | 8.8 - 12 | 10.5 | 10 |
| Total Dissolved Solids | mg/L | 480 - 540 | 503 | 650 |

## 4.4 Expected Future Loadings

Using the predicted population in Table 3.1, and the assumed loading of 250L/EP/day (Sewerage Code of Australia, Vol 2, 1992) the predicted loadings for the sewage treatment plant at Theodore is shown in Table 4.3.

**Table4.2: Predicted future loadings for the Theodore Sewage Treatment Plant**

| YEAR | FLOW (KL/DAY) |
|---|---|
| 2005 | 183 |
| 2010 | 188 |
| 2015 | 193 |
| 2020 | 198 |
| 2025 | 203 |

## 5.0 TREATED WASTEWATER

### 5.1 Water recycling – Classification for Irrigation

Banana Shire Council has an Environmental Licence set by the Environmental Protection Agency (EPA) which does not set specify effluent contaminant limits. Rather than set a standard limit the EPA requires that the Theodore sewage treatment plant implement an effluent disposal strategy which includes a Recycled Water Management Plan for irrigation detailing the following;

- Soil capability and assimilative capacity
- Depth of groundwater and effect effluent is having on groundwater
- Nutrient loading and nutrient harvesting
- Sustainability of irrigation practices
- Alternatives to current practices.

The appropriate quality of water used for recycling will depend on the intended end use, site characteristics, and risk factors.

The '*Queensland Water Recycling Guidelines',* herein referred to as the *Guidelines* detail the quality of recycled water required for various uses.

Currently the effluent is predominantly being used by the Apex Club for irrigating the balance of the STP land adjacent to the treatment plant site. The land is approximately 11.0 hectares and is used to grow sorghum, cotton or similar crops under a lease agreement between the Council and the Apex Club. There was, at the time the lease agreement was entered into, no consideration given to the long term sustainability of the nutrient loading onto the land and the effect on the surrounding environment.

A sustainability assessment is considered necessary and an evaluation of other potential uses for the effluent

Treated effluent is considered a valuable resource and when effectively managed can be applied to land and be beneficial to the surrounding environment. Suggested applications for the Theodore STP treated effluent include;

- Apex Club's irrigation area (Sorghum) 11.0Ha – currently supplied with effluent for irrigation. (Shown in Appendix B)
- Alternative irrigation sites on neighbouring properties, many of which are irrigation lots. A neighbouring property (Edwards farm approximately 30Ha available area negotiable is one option).
- Irrigation of the neighbouring lot of 11 ha that Council is in the process of acquiring.
- Supplying effluent to irrigate areas of public land spaces, such as parks, road landscaping or sports and school play grounds.
- Proposed new truck washdown bay.
- Supply to a treated effluent standpipe for other uses (dust suppression, drilling operations, other industrial uses, etc)

The *Guidelines* provide different classifications for the various applications of recycled water. These are described in Table 5.2.

**Table 5.1: Classification of recycled water for use in Queensland**

| Class | E. coli (cfu/100m) median | $BOD_5$ (mg/L) median | Turbidity (NTU) 95%ile (max.) | SS (mg/L) median | TDS, mg/L or EC, µS/cm Median TDS / EC | pH |
|---|---|---|---|---|---|---|
| A+ | <1 | | <2 (5) | - | 1000/1600 | 6-8.5 |
| A | <10 | 20 | <2 (5) | 5 | 1000/1600 | 6-8.5 |
| B | <100 | 20 | - | 30 | 1000/1600 | 6-8.5 |
| C | <1000 | 20 | - | 30 | 1000/1600 | 6-8.5 |
| D | <10,000 | - | - | - | 1000/1600 | 6-8.5 |

Note: For more detail on the criteria of each class of water and recommended end uses, please refer to the 'Queensland Water Recycling Guidelines' , December 2005.'

For the applications suited to Theodore's proposed effluent reuse schemes the following guidelines are applicable,

**Pasture Irrigation, stock watering, and agricultural wash down**

- Where there is no assurance of effective control over the timing of public access to any area irrigated with recycled water, and above-ground irrigation delivery systems are used, only Class A recycled water should be used. Where sub surface irrigation is used, Class C recycled water may be used with uncontrolled access. Drip irrigation may not lead to ponding of the water.

- Class C recycled water could also be used for spray irrigation in areas where public access can be prevented during irrigation and for long enough after irrigation wetted surface has dried, or be used for subsurface irrigation.

- Class B recycled water can be used for irrigation of pasture and fodder for dairy animals where there is no withholding period between irrigation and feeding;

- Class C recycled water can be used for irrigation of pasture and fodder for dairy animals where there is a 5 day with holding period;

- Recycled water for stock drinking water should meet the requirements for Class B, with the exception that stock should not be exposed to recycled water that contains Helminth (tapeworm) eggs;

- Recycled water for non food crops such as silviculture, cotton, turf production and nurseries should be of at least Class D quality; and

- Vehicle washdown and standpipe supply should be considered as unrestricted access and should be Class A+

These guidelines are primarily aimed at ensuring public safety. The EPA also requires long term environmental sustainability to be assessed and shown that the application of the effluent for the various uses is not detrimental to the environment in the long term.

Currently the EPA's accepted method to assess the irrigation application rate, and sustainability of recycled water applications is by using an effluent irrigation modelling program developed by the Department of Natural Resources and Water (DNRW) called MEDLI (Model for Effluent Disposal Using Land Irrigation).

## 5.2 Current Treated Wastewater Quality

The final point of discharge is from the effluent storage pond, Samples were taken from these ponds and analytical results are shown in Table 5.1.

**Table5.2: Quality of treated wastewater exiting the effluent storage pond**

| Parameter | Units | 08/03/05 | 19/04/05 | 04/05/05 | 14/06/05 | Average Concentration |
|---|---|---|---|---|---|---|
| Total Suspended Solids | mg/L | 6 | 32 | 60 | 34 | 33 |
| BOD | mg/L | 13 | 7 | 14 | 14 | 12 |
| pH | - | 7.1 | 7.8 | 7.2 | 7.7 | 7.5 |
| Ammonia as N | mg/L | | 2.4 | 3.1 | 9.7 | 5.1 |
| Nitrate as N | mg/L | | 0.18 | 0.62 | 0.36 | 0.4* |
| Total Nitrogen as N | mg/L | 18 | 91 | 9.2 | 15 | 14 |
| Elements (Phosphorous) | mg/L | 6.9 | 5.3 | 6.2 | 7.4 | 6.5 |
| Total Dissolved Solids | mg/L | 384 | 420 | 480 | 540 | 456 |

Note: The highlighted figure is believed to be an analytical or information transfer error, therefore not considered to be true and accurate.

* Average excluding unusual highlighted result.

It is evident that the treatment process is generally effective in treating the effluent to meet acceptable BOD concentration with some overall nitrogen removal and limited phosphorous removal. The suspended solids concentrations are higher than would be expected from an efficiently operating system.

With suspended solids at these higher levels, irrigation targeted disinfection will be difficult to achieve. Accordingly the treated effluent from Theodore STP does not meet the water quality for the irrigation and reuse schemes and upgrading of the existed plant is required to achieve the irrigation water quality.

# 6.0 EXISTING SEWAGE TREATMENT PLANT

The sewage treatment plant at Theodore is typical of plants built in the late 60's early 70's, based on primary sedimentation, biological treatment by trickling filters and discharge to local waterway. These types of plants were designed to typically produce a secondary treated effluent quality of 20mg/L BOD/ 30mg/L Suspended Solids, with no nutrient removal, although some nitrification may occur in the trickling filter under low-load conditions and ammonia volatilisation may occur in the storage ponds.

The process system at Theodore consists of the following units;

- Raked screen
- Imhoff tank (combined Primary Sedimentation and Sludge Digester)
- Trickling filter;
- Humus Tank (Secondary Clarifier);
- Disinfection contact tank.
- Effluent Lagoon
- Effluent irrigation

## 6.1 Imhoff Tank

### 6.1.1 General Consideration

The Imhoff tank performs two functions in a single tank. It acts as:

- A primary sedimentation tank, and
- An anaerobic sludge digester.

Settleable solids in the raw sewage settle as a sludge into the conical hopper bottom of the tank where it anaerobically digests and is regularly withdrawn and transferred to the sludge drying beds. The clarified wastewater flows over a weir to the trickling filters for further treatment.

There is only a manually raked coarse screen at Theodore; the *QLD Guidelines for Planning & Design of Sewerage Schemes,* recommend that screening and grit removal should always precede an Imhoff Tank, therefore, the solid loading to the Imhoff Tank is higher than desirable with considerable coarse inorganic materials such as paper, grit etc passing into the tank.

The physical design of an Imhoff Tank is based on the surface loading rate, retention time, digestion chamber volume and floor slope of hopper bottom. The surface loading rate is given in terms of cubic metres of flow per square meter of surface area per unit of time, usually per day.

Typical design parameters for an Imhoff tank are shown in Table 6.1, taken from the *QLD Guidelines for Planning & Design of Sewerage Schemes,* design criteria. It should be noted that these criteria relate to the individual compartments of the tank ie sedimentation compartment or digestion compartment not to the tank as a whole

**Table 6.1: Imhoff Tank Design Values**

| Item | Units | Criteria |
|---|---|---|
| Surface loading rate at 3ADWF | $m^3/m^2/d$ | <25 |
| Retention time | hrs | >2.0 |
| Anaerobic Digester volume for primary and secondary sludge | $m^3/EP$ | 0.15 |
| Suspended solids removal rate | % | 40 – 70 |
| BOD removal | % | 20 – 50 |

The surface loading rate for optimal operation is given in literature as $25m^3/m^2/day$, (Water Resources Commissions Department of Primary Industries, *QLD Guidelines for Planning & Design of Sewerage Schemes, September 1992).* This rate is designed to achieve a removal efficiency of 30% for BOD and 60% for suspended solids.

For normal primary sedimentation tanks if the retention time is too long the content of the tank, especially the settled sludge, can become anaerobic and generate unpleasant odours and if the retention time is too short the tank will be inefficient and the suspended solids and BOD removal efficiency will be reduced. The same can be said about the Imhoff tank, however anaerobic digestion is encouraged. The internal baffles prevent the rising sludge from entering the clarified central effluent zone, encouraging the biogas bubbles through to the side zones. In theory this should not cause solids to pass through to the clarified effluent, but be collected through a scum collection system.

### 6.1.2 Assessment of Imhoff Tank

Theodore's raw sewage is pumped via a rising main from the town directly into the Imhoff tank. The Imhoff tank is an elevated structure with the top of the tank some 3.3 m above ground level.

Due to fluctuations in raw sewage flow, the surface loading rate varies throughout the day. These flow fluctuations have the potential to significantly reduce the efficiency of the process units.

The Theodore STP plant is not designed to receive significant recycle streams, the recycle flows entering the Imhoff tank include;

- Subnatant draw off from the drying beds
- Humus sludge return from the humus tank
- Effluent sprays from the chlorine contact tank (used to control the scum layer)

Table 6.2 provides design detail of the current Imhoff tank and Table 6.3 provides detail on current and expected future performance.

**Table 6.2: Theodore STP Imhoff Tank**

| Item | Units | Value |
|---|---|---|
| Length | m | 7.01 |
| Width | m | 5.79 |
| Surface Area (Total) | $m^2$ | 40.6 |
| Surface Area (Sedimentation) | $m^2$ | 21.4 |
| Side wall depth (Sedimentation) | m | 1.50 |
| Approximate volume (Total) | $m^3$ | 230 |
| Approximate volume (Sedimentation) | $m^3$ | 164 |
| Approximate Volume (Sludge Digestion) | $m^3$ | 66 |

**Table 6.3: Theodore Imhoff Tank current and future operation for Year 2005 and 2025.**

| Item | Units | Year 2005 | | Year 2025 | |
|---|---|---|---|---|---|
| | | ADWF | 3ADWF | ADWF | 3ADWF |
| Flow rate | $m^3/d$ | 183 | 553 | 203 | 609 |
| Surface loading rate (Hydraulic) | $m^3/m^2/d$ | 8.6 | 25.8 | 9.5 | 28.5 |
| Hydraulic Retention time | hrs | 21 | 7 | 19 | 6 |
| Digestion Volume | $m^3/EP$ | 0.09 | - | 0.08 | - |

The surface loading rate is relatively low with a high retention time. This retention time allows for significant sludge removal to the anaerobic digestion chamber, and therefore as is revealed by the low BOD concentration of the tank effluent, reduces the BOD loading to downstream processes. The suspended solids concentration is also lower than otherwise would be expected. The volume of the digestion compartment is lower than the recommended *Guidelines* value of 0.15 $m^3/EP$ which will increase the required rate of sludge removal with consequent reduction in the stability of the sludge

**Figure 6-1: Surface of Imhoff tank at Theodore STP**

Effluent sprays have been set up over the Imhoff Tank to reduce scum formation.

Visual assessment of the Imhoff tank indicates that the general structure of the tank may be in a poor condition. The concrete surface in general is in good condition however there are a number of cracks and flaws most noticeably in the supporting legs with the potential to lead to a structural failure of the tank.

**Figure 6-2: Cracks in supporting legs of Imhoff Tank**

Overall, while the use of an Imhoff Tank is dated technology, it is at Theodore currently reliable as a treatment unit in the process train. Structurally the tank, due to the appearance of cracks, appears to be in a poor condition and the structural integrity should be investigated further.

## 6.2 Trickling Filter

### 6.2.1 General Considerations

The trickling filter is the major treatment unit in the purification process. The trickling filter treats the soluble organic matter in the clarified wastewater from the Imhoff tank. The clarified wastewater is distributed evenly over the filter surface through the rotating distributor arms and flows down through the rock media of the filter. A biological slime containing bacteria and protozoa grows on the media and as the wastewater passes over the slime the bacteria purifies the wastewater by converting the organic material into harmless compounds – mainly carbon dioxide and water.

The trickling filter in the application at Theodore STP is not designed to perform significant nitrification - that is convert ammonia in the sewage to nitrate. However some nitrification could occur if the filter has a light organic loading resulting in subsequent reduction in ammonia concentration in the wastewater.

All tricking filters produce a fine sludge (or humus sludge) that is washed from the filter in the effluent. The sludge comprises the algal slime, bodies of the protozoa, worms and insects that inhabit the eco-system within the filter. Effluent from the base of the filter is collected and directed to the secondary clarifier (humus tanks), where the humus solids are allowed to settle out. The clarified effluent is directed to effluent disposal or some tertiary treatment.

The design parameters for a trickling filter of this type at Theodore is provided in Table 6.4

**Table 6.4: Optimal Design Parameters for a Low Rate Trickling Filter**

| Parameter | Unit | Value |
|---|---|---|
| Hydraulic Loading Rate | $m^3/m^3$ media /day | 0.3 – 0.8 |
| Organic Loading Rate | kg BOD/$m^3$/day | 0.07-0.22 |

The hydraulic loading rate is the rate at which the effluent passes through the media. This should not exceed 0.8 $m^3/m^3$ media per day. Above this rate the effluent will not have adequate contact time with the biological slime on the media to ensure full treatment. Conversely, the filter media must remain moist at all times. If the media is allowed to dry out for a prolonged period, the bacteria starts to die and the treatment efficiency is reduced. To prevent the media drying out during periods of low flow it is normal practice to recirculate treated effluent to the filter.

### 6.2.2 Assessment of the Trickling Filter.

The overflow from the Imhoff tank flows into the chamber which feeds the trickling filter. The feed flow is delivered to the central well in the trickling filter and disperses out through the outlet holes in the distribution arms. The effluent flowing out of the arms generates enough force to drive the arms across the surface of the media of the trickling filter.

The effluent is then collected in the underdrainage system and directed to the humus tank.

**Figure 6-3: Trickling filter at Theodore**

The appearance of the media surface at Theodore suggests that the effluent is not evenly distributed over the surface of the filter, some patches appear dry. This reduces the effectiveness of treatment and the overall performance of the filter.

At the time of the site visit, it is obvious that the flow was not continuously entering the filter, the arms of the trickling filter were rotating intermittently and a garden sprinkler connected to a hose was spraying treated effluent water on the top of the filter to keep the filter media moist, this uneven distribution will cause the filter media to dry out.

The result of our assessment of the trickling filter is shown in Table 6.5 below. The BOD concentration of the influent was taken from theoretical calculated values.

The preliminary assessment of the trickling filter at Theodore STP, on a 24 hour basis, indicates that the filter would receive an acceptable hydraulic and organic load at ADWF under normal operation conditions.

**Table 6.5: Theodore STP Trickling Filter.**

| Items | Units | Year 2005 | | Year 2025 | |
|---|---|---|---|---|---|
| | | ADWF | 3ADWF | ADWF | 3ADWF |
| Flow (2005) | $m^3/d$ | 183 | 553 | 203 | 406 |
| Diameter | m | 12.5 | | | |
| Depth of Media | m | 2.1 | | | |
| Volume of Media | $m^3$ | 258 | | | |
| Hydraulic loading rate | $m^3/m^3/d$ | 0.71 | 2.14 | 0.79 | 2.36 |
| Organic load | kg/d | 40 | | 45 | |
| Organic loading rate | kg BOD/$m^3$/d | 0.14 | 0.42 | 0.15 | 0.396 |
| BOD removal | % | 85 | 76.8 | 84.5 | 76 |
| BOD – Trickling filter discharge | mg/L | 29 | 45.4 | 30 | 40.4 |

From Table 6.5 above, the average hydraulic loading to the filter is 0.71 $m^3/m^3/d$ which is within the accepted range of 0.3 - 0.8 $m^3/m^3/d$. This indicates that, if the filter was receiving a constant flow rate, the filter would be operating at the high end of the acceptable optimal hydraulic loading. Based upon influent BOD estimates the organic loading rate is within the desirable range.

From the above we have concluded that the trickling filter is near to its maximum capacity under the condition of daily average flow, and is likely exceed the maximum during periods of peak daily flow periods when the hydraulic loading would be in the order of 2.14 $m^3/m^3/d$ and the organic load around 0.42 kg/$m^3$/d at x3 ADWF. During these periods the plant is receiving peak flows, deterioration of the effluent quality may be expected.

## 6.3 Humus Tank/Secondary Clarifier

### 6.3.1 General Consideration

The purpose of the humus tank is to separate the humus solids from the trickling filter effluent. The clarified effluent passes to the disinfection tank while the sludge settles to the tank floor and is returned upstream to the inlet works.

### 6.3.2 Assessment of the Humus Tank

The design of humus tank in this application is similar to the design of primary sedimentation tanks. The accepted design parameters for a humus tank are shown in Table 6.6 below.

**Table 6.6: Humus tank design values**

| Item | Units | Range |
|---|---|---|
| Surface Loading Rate at Peak flow* | $m^3/m^2/d$ | 25 |
| Retention time at 3ADWF | hrs | 2.0 |
| Weir overflow rate | $m^3/m/d$ | 250 |

* For design purposes peak flow = 5ADWF (*QLD Guidelines for Planning & Design of Sewerage Schemes, Vol 2, Section 12)*

The design parameters and performance for the Theodore STP humus tank is shown in Table 6.7 below.

**Table 6.7: Theodore STP humus tank**

| Item | Units | Year 2005 | | Year 2025 | | |
|---|---|---|---|---|---|---|
| | | ADWF | 5ADWF | ADWF | 3ADWF | 5ADWF |
| Flow | kL/d | 183 | 915 | 203 | 609 | 1015 |
| Diameter | m | 5.8 | | | | |
| Surface Area (Total) | $m^2$ | 26.3 | | | | |
| Side wall depth | m | 1.42 | | | | |
| Estimated volume | $m^3$ | 29.4 | | | | |
| Surface loading rate | $m^3/m^2/d$ | 7.0 | 34.7 | 7.7 | 23.1 | 38.6 |
| Retention time | hrs | 3.8 | 0.8 | 3.5 | 0.9 | 0.7 |
| Weir overflow rate | $m^3/m/d$ | 10 | 50 | 11 | 33 | 55 |

The Surface Loading Rate is high for peak design flow of 5ADWF but could be considered reasonable for 3ADWF

The humus tank for Theodore is in the design of a Dortmund tank. A Dortmund Tank is a cylindrical unit with a shallow vertical wall height, with a large hopper bottom, and a steep side wall slope to aid sludge settlement.

The tank is required to settle the solids, and allow clarified effluent to pass through to the chlorination contact tank. Longer retention time will allow for greater solid settlement, may cause anoxic conditions to occur resulting in denitrification of any nitrate that has formed in the trickling filters releasing bubbles of nitrogen gas. These rising bubbles disturb the sludge blanket reducing settling efficiency. This can be avoided with regular sludge withdrawal.

The current practice of manual sludge draw off is undertaken in the morning and the humus sludge returned to the Imhoff tank. Analytical results show that from the raw influent to the final chlorine contact tank there is some loss of ammonia (57%). Nitrogen is lost through the system through settled organic particulate material, ammonia volatilisation, and ammonia nitrification followed by nitrate denitrification. It is not known exactly which process dominates, however as some nitrates are evident in the final effluent; it is assumed that a level of nitrification is occurring followed by some denitrification.

Whether the denitrification is occurring in the humus tank can only be determined by further analysis or visual observation of rising bubbles in the tank. Denitrification should be avoided to optimise settling efficiency.

It is therefore preferred to initiate more regular humus sludge draw-off procedures, preferably automatically during low flow periods to assist with operation of the trickling filter by maintaining moist condition of the media.

During the site visit, the clarified effluent leaving the humus tank looked to be turbid and little changed from the appearance of the clarified sewage leaving the Imhoff tank. The analytical results reveal the effluent suspended solids ranges from 6 -60mg/L, indicating performance inconsistent with the design values and possibly due to sludge retained in the tank.

**Figure 6-4: Humus tank at Theodore STP.**

## 6.4 Chlorine Contact Tank

### 6.4.1 General

The chlorine contact tank ensures that the chlorine (gaseous chlorine at Theodore) has adequate retention time to achieve the maximum bacteriological kill rate. A 20 to 30 minute retention time is generally considered adequate.

### 6.4.2 Assessment of Final Disinfection Contact Chamber

The current volume of the contact tank is approximately $18m^3$. At the ADWF the retention time in the contact tank is nearly 2.1 hrs, and at 3 times ADWF the detention time is approximately 42 minutes. Therefore the current chlorine contact tank is considered more than adequately sized.

The quality of the effluent will determine the effectiveness of disinfection. Organic matter in the effluent can react with the chlorine creating harmful by products as well as generating a higher demand for chlorine to achieve the desirable residual chlorine concentration, therefore reducing disinfection effectiveness.

As seen in Figure 6.5, the effluent quality is turbid. It is predicted that there would be a considerable build up of sludge on the tank floor. With such quality effluent, disinfection is unlikely to be effective.

From the chlorination tank the effluent is pumped to the final effluent lagoon.

**Figure 6-5: Chlorine Contact Tank**

# 6.5 Effluent Storage Pond

## 6.5.1 General

The effluent storage pond is used to store the treated effluent until it is required for irrigation. While in the pond some reduction in nutrient concentrations can be expected especially if algae are present. Similarly some reduction in BOD and suspended solids may occur but this reduction may be masked or negated by the presence of organic material from wildlife and algae in the samples.

The contribution made by the pond to the treatment process is dependent on the climate particularly temperature and hours of sunlight, and as such is variable.

## 6.5.2 Assessment of Lagoon

The total volume of the lagoon at Theodore and retention time are not known accurately as there are no available drawings. The capacity is estimated at 12.6 ML.

**Figure 6-6: Pond**

The pond at Theodore is green in colour and appears to have significant algal population. The pond receives a relatively high solid loading, and considerable organic and nutrient loading in the treated effluent. The range and average concentration of contaminants for influent and effluent from the pond and the removal efficiency of the various contaminants is shown in Table 6.8 (taken from average figures)

**Table 6.8: Pond Performance**

| Item | Units | Pond Inlet | | Pond outlet | | Reduction in pond |
|---|---|---|---|---|---|---|
| | | Range | Average | Range | Average | |
| Ammonia as N | mg/L | 16-22 | 19 | 2.4 - 9.7 | 5.1 | 73% |
| Nitrate as N | mg/L | 33-49 | 40 | 0.18 - 0.12 | 0.4 | 99% |
| Total Nitrogen | mg/L | 12-35 | 27.3 | 9.2 -18 | 14.1 | 48% |
| BOD | mg/L | 12-40 | 21 | 7 -14 | 12 | 20% |
| Suspended Solids | mg/L | 12-140 | 57.5 | 6 - 60 | 33 | 43% |
| pH | - | 7-7.6 | 7.3 | 7.7 - 7.8 | 7.5 | |

Table 6.8 shows that significant nitrogen removal is achieved in the pond. The final component of nitrogen is primarily made up of inorganic nitrogen which is difficult to remove. Little BOD is removed and the final suspended solids concentrations remains high.

The pond at Theodore is considered useful to reduce nutrient concentration in the effluent. However due to the presence of wildlife and algae the effluent will be of poor bacteriological quality. This contamination may however be accepted by the EPA for irrigation purposes as it is not from human sources. None the less, the quality should be regularly monitored to ensure bacterial quality remains at a reasonable level.

## 6.6 Sludge Treatment and Dewatering

### 6.6.1 Sludge Production

Sludge produced in a sewage treatment plant may be defined as a concentrated dispersion of solids from the treatment processes suspended in water. The solids in the sludge are mainly of a biological nature (biosolids) produced as a waste product of the biological purification. In general, the biosolids are in the order of 70 - 80% organic matter with the balance being inert, inorganic material.

The nature and physical characteristics of the sludge depends on its origins. At the Theodore STP, sludge is removed from the bottom of the Imhoff Tank at the operator’s discretion. This sludge originates from the raw or primary sludge settled out from the influent and humus sludge returned from the Humus Tank. In a well operated Imhoff Tank this sludge should be a relatively stable anaerobically digested sludge

Raw or primary sludge is usually a grey colour with a viscous, lumpy consistency due to its organic nature and has an objectionable odour when exposed to air. It is produced by the settlement of the organic matter contained in the raw sewage. The raw sludge anaerobically digests in the bottom hopper of the Imhoff Tank which changes its characteristics. The digested sludge is a black colour with a characteristic “tarry” odour and a creamy appearance.

Sludge production can be approximated using flow data, the average suspended solid concentration in the influent and assuming a 50% solid reduction by digestion.

The sludge removed from the Imhoff tank is likely to be around 2 - 4% dry solids and at Theodore is transferred directly to sludge beds for drying.

The estimated daily sludge production for the Theodore STP is shown in Table 6.9 below.

**Table 6.9: Theodore STP Sludge Production**

| Items | Units | Value (Year 2005) | Value (Year 2025) |
|---|---|---|---|
| Current EP | | 732 | 811 |
| Suspended solids loading | g/EP | 70 | 70 |
| Suspended solids load to treatment plant | kg/d | 51.2 | 56.8 |
| Suspended solids captured in Imhoff tank (assume 60%) | kg/d | 30.7 | 34.1 |
| Estimated sludge dry solids content | % | 2 | 2 |
| Solid loss through anaerobic digestion | % | 40 | 40 |
| Sludge volume transferred to drying beds | $m^3/d$ | 0.92 | 1.02 |

### 6.6.2 Sludge Treatment

The sludge treatment process in place at Theodore STP is based on anaerobic digestion. The sludge is held for up to 30 days at ambient temperatures under anaerobic conditions. In the process, specific groups of anaerobic bacteria decompose the various groups of organic matter in the sludge, breaking them down to simple compounds, mainly water, carbon dioxide, methane and biomass.

The mixture of carbon dioxide and methane produced is commonly known as "biogas" and can be used to generate heat or power if sufficient gas is produced. A well operated anaerobic digester can reduce the organic matter in the sludge by 40 – 50% and produce a stabilised sludge.

There is a wide range of sludge digestion processes available. The anaerobic process employed at Theodore is classified as a low-rate process based on its' extended retention time and operation at ambient temperatures.

The bottom section of the Imhoff Tank operates as an unheated, low rate anaerobic digestion chamber. From drawings provided the Imhoff tank has a calculated maximum working volume of approximately $230m^3$, with the sludge hopper having an approximate volume of 66 $m^3$. It is likely that the calculated volume will be significantly reduced by a build up of grit and other settled inorganic solids that have settled in the tank.

Digested sludge is removed from the hopper bottom every 3 to 4 weeks and transferred to the drying beds.

Anaerobic digestion produces biogas, a mixture of carbon dioxide and methane. The "biogas" produced rises through the gas chamber to the surface and escapes into the atmosphere. As the gas travels upwards through the digesting sludge some mixing of the sludge is induced. A scum layer is formed from the sludge rising with the gas to the surface; this layer helps contain the odour.

There are no exact design requirements for an Imhoff tank, it can however be likened to that required for a typical low rate anaerobic digester. Table 6.10 provides a brief summary of the design requirements for a low rate digester, and compared to actual design of the sludge collection chamber of the Imhoff Tank.

| Item | Units | Guidelines | Value |
|---|---|---|---|
| Calculated capacity | $m^3$ | | 66 |
| Raw sludge volume | $m^3/d$ | | 3.75 |
| Retention | d | 30 – 60 | 17 |
| Volatile solids loading | kg VSS/$m^3 \cdot$d | 0.64 – 1.00 | 0.60 |
| Digester volume per EP | $m^3$capacity/ep | 0.1 – 0.14 | 0.09 |
| Expected EP Year 2005 | | | 732 |
| Digester volume required | $m^3$ | | 88 |

**Table 6.10: Design criteria for a low rate anaerobic digester**

The above calculations indicate that the volume of the Imhoff Tank is inadequate for the current loading.

### 6.6.3 Sludge Dewatering - Sludge Drying beds

Sludge removed from the Imhoff tank is expected to have a dry solids content in the order of 2 – 4%. At this concentration, the sludge is still 96 – 98% water and must be dewatered (dried) to a solid consistency so that it can be handled and transported from site without spillage.

There are various methods of sludge dewatering and currently the Theodore treatment plant dewaters the sludge on drying beds allowing the sludge to dry to a “spadeable” consistency before lifting and eventually removing from site to landfill.

The Queensland guidelines for sludge drying beds are based on the EP loading to the plant and the sewage treatment process, specifically the sludge quality. For a plant such as Theodore, the sludge quality would be consistent with that of an Imhoff tank. Therefore the sludge beds are recommended to be 0.05$m^2$ per EP loading to the plant.

There are six sludge drying beds at Theodore STP, these are in extremely poor condition and require replacement.

**Table 6.11: Existing Drying Beds**

| Item | Units | Value |
|---|---|---|
| Number of drying beds | | 6 |
| Dimensions (L x W) | m | 3.5 x 2.5 |
| Area (per bed) | $m^2$ | 8.75 |
| Area (total) | $m^2$ | 52.5 |
| Underdrain system | | Yes |
| Bed topping | | Sand |
| General condition | | Poor |
| Area required (Year 2005) | $m^2$ | 36.75 |
| Area required (Year 2025) | $m^2$ | 40.55 |

**Figure 6-7: Sludge drying beds at Theodore**

### 6.6.4 Summary of Existing Plant Processes

| Element | Current | Recommendations |
|---|---|---|
| Inlet Screens | Manually raked coarse screen | Automatic fine screening to be installed to reduce solids load on downstream process units |
| Grit Removal | None | Grit removal to be implemented to reduce solids load on downstream process units |
| Imhoff Tank | Current capacity adequate as sedimentation tank, inadequate for sludge digestion and stabilisation. | Process capacity inadequate for future loading<br>Structural integrity also needs evaluating |
| Trickling Filter | Current capacity adequate | Flow distribution need improving<br>Capacity should be adequate for future loading under average flows provided humus recycle flows are controlled to apportion loading throughout day |
| Humus Tank | Current capacity adequate for average flows and anticipated peak flows | Capacity should be adequate for future loading under expected peak hydraulic loads<br>Humus return flows need to be automated and controlled |
| Disinfection | Current capacity adequate | Capacity should be adequate for future loading under expected peak hydraulic loads<br>Tank to be desludged to improve capacity and improve effluent turbidity. |
| Effluent Disposal | Current pond capacity adequate | Capacity should be adequate for future loading under expected peak hydraulic load<br>Effluent quality is affected by algal growth within lagoon and presence of wildlife |
| Sludge Disposal | Sludge Drying Beds - unserviceable | Sludge Drying Beds to be replaced |

# 7.0 FUTURE SEWAGE TREATMENT AT THEODORE

## 7.1 General Considerations

The assessment of the treatment plant and process has demonstrated that the existing Theodore treatment plant works is in poor structural condition and the process overall is producing a poor to reasonable quality effluent and cannot consistently achieve the current irrigation water quality requirement of Class B.

There are a number of options are available for continued sewage treatment at Theodore:

- Upgrade the existing treatment plant to produce a good quality treated effluent;
- Build a new treatment plant using modern treatment technology that will produce a treated effluent more suited to disposal by irrigation;
- Provide an additional treatment module in parallel with the existing plant to treat a portion of the effluent to higher quality standard suitable for unrestricted reuse.

## 7.2 Design Flows for Augmentation

Table 3.1 shows the predicted population growth for Theodore to the Year 2025 when the predicted daily flow will be 203 kL.

In the design of a treatment plant it is usual practice to allow all flows up to 3ADWF to pass to the secondary treatment process and hence receive full treatment. The maximum flow to the Theodore STP is expected to be no more than 8 L/s, which is equivalent to 3ADWF; the existed raw sewage pump station must be sized to match the increased flow rate to the plant.

While the design of the augmentation is based on 203 kL/d, the process hydraulics has been sized to accommodate a maximum flow of 8 L/s to the secondary treatment process. This will ensure all the treated effluent produced by the upgraded plant will be of the required, consistent quality.

## 7.3 Treatment By-Pass

As a precaution in the event of flows greater than 8 L/s reaching the treatment plant, a by-pass will be included in the design. During wet weather events, the sewage flow received at the treatment plant is higher but generally weaker due to the dilution by infiltration water and apart from prolonged events (>3 days) the treated effluent quality would not be affected.

As discussed in Section 4.3.1 there is a minimal correlation between rainfall and received flow into sewage plant, indicating low infiltration into the sewers. None the less, extreme rainfall can cause hydraulic over flow to the plant, and this situation is recommended to be considered when designing a process system.

## 7.4 Treated Effluent Quality

Currently at Theodore the treated effluent is predominantly used for irrigation with only emergency overflow from the storage pond discharging to the Dawson River via the local creek.

When a treated effluent is to be used for crop irrigation, the main points that must be considered include:

- Method of irrigation
- Long-term environmental sustainability of the irrigation scheme,
- Nutrient levels in the treated effluent;
- Crop nutrient requirements,
- Soil types on the irrigation area and its capacity to handle nutrients, and
- The fate of any excess treated effluent, either from over-irrigation or effluent overflows from the storage pond.

Using soil analysis from the cropped area at Theodore and with knowledge of the crops grown, MEDLI was used to predict nitrogen and phosphorous uptake rates for the crop and from the rates the allowable concentration of nitrogen and phosphorous in the treated effluent. The irrigation of pasture crops requires Class B effluent quality.

When treated water is to be used for truck wash and/or supply to a standpipe for applications such as dust suppression, drilling operations etc, Class A+ quality of the treated effluent must be achieved.

The treated water from Theodore STP will predominantly be recycled for irrigation of crops with only minor quantities being used for the truck wash bay and standpipe supply which have a high likelihood for unrestricted human contact. Therefore Class B recycled water quality is targeted for any upgrading of the entire plant with the potential to include a small side stream Class A+ treatment system to meet the demands of the truck wash bay and standpipes.

The Class A+ quality (median) prior to disinfection is shown as Table 8.1:

**Table 7.1: Recommended Treated Effluent Quality for irrigation**

| Parameter | Unit | Class A+ (median) | Targeted (median) |
|---|---|---|---|
| pH | | 6.5 – 8.5 | 6.5 – 8.5 |
| BOD | mg/L | 20 mg/L | 10 |
| Suspended solids | mg/L | <5 mg/L | 5 |
| Total nitrogen | mg/L | - | 10 |
| Total phosphorous | mg/L | - | 5 |
| E. coli | Cfu/100mL | 1 | 1 |

The need for low BOD and suspended solids in the treated effluent is related to the efficiency and effectiveness of the disinfection process. Organic matter will adsorb and react with chlorine to form harmful by-products while suspended solids will prove protection for bacteria and pathogens.

Upgrading of the existing treatment plant and processes will need to consistently achieve the treated effluent quality shown in Table 8.1 above.

# 8.0 OPTIONS FOR UPGRADING

## 8.1 Current Situation

The main aim in upgrading the treatment plant is to install a treatment plant that will consistently produce the required quality treated effluent. The process should not rely on the storage pond for additional treatment to meet the effluent quality criteria.

The existing secondary treatment; the trickling filter is, under average daily flow conditions, adequate for the predicted Year 2025 loading. Operation of the trickling filters can be improved by optimising flow distribution and recycle flows.

If the existing treatment plant is to meet the performance requirements for *uncontrolled* treated effluent reuse, the existing treatment plant processes must be augmented or replaced.

## 8.2 Options for Augmentation

There is a range of options available for augmentation of the secondary treatment process to ensure full compliance with the treated water quality requirement. As the primary re-use of the recycled water will be irrigation of the land currently leased by the Apex Club improved operational practices will allow effluent of Class B quality to be targeted.

Effluent destined for uses, the truck wash bay and the recycled water stand pipe should be of Class A+ quality.

The capital cost estimates for all augmentation options evaluated include components for contingency and for engineering costs but excludes any GST payable.

There are two options for the augmentation of the plant to meet the desired effluent quality. The first is to utilise the existing facilities as much as possible in the upgrade, the second is to decommission the existing facilities and to build a new treatment plant.

It is assumed for all augmentation options that:

- A new inlet works will need to be provided at an estimated cost of $235,000.
- The current Imhoff Tank is considered unsuitable to be used as part of any upgrade because of current structural defects and will be replaced if required.
- The existing trickling filter can be utilised when required as the structural integrity is considered sound**.**
- A Primary Sedimentation Tank (PST) to replace the sedimentation compartment of the Imhoff Tank would be 5.6 m diameter with a side wall depth (SWD) of 3.0m. The estimated cost of this tank complete with pipework and equipment is $294,000.
- A low rate Anaerobic Digester to replace the Imhoff Tank digestion compartment would be 5.6 m diameter with SWD of 3.6 m and conical base section. The estimated cost of this work complete with pipework and equipment is $252,000.

### 8.2.1 Option 1 - Additional trickling filters

To meet the predicted flow and loading requirements for the Year 2025 additional trickling filter capacity would be required to prevent effluent quality deterioration at peak flows. Duplication of the existing filter is advisable to consistently achieve a 20mg/L BOD and 30mg/L suspended solids quality treated effluent, and achieve some nitrification.

In addition to the trickling filters, improvement of the current basic screenings system and installation of grit removal should be included.

The existing humus tank should be able to be utilised after modifications to sludge draw-off as it is technically underloaded for the predicted flows.

For optimal performance of the trickling filters, a recycle flow will be included in the design. This flow will be taken downstream from the humus tank, from the clarified effluent prior to chlorination and pumped to the outlet of the Imhoff Tank to ensure the filter media remains wet.

Treated effluent from the humus tank will pass to the disinfection process and a Residual Chlorine monitor included in the system to control chlorination.

**Treated Effluent Quality**

At best, a trickling filter system can only be relied upon to produce effluent of 20 mg/L BOD, 30 mg/L suspended solids quality (ie Class B) with some nitrification, full nitrification is unlikely. Any phosphorous removal required for sustainable irrigation or discharge to water courses will only be achieved by chemical dosing.

**Process Design**

The main process design parameters for the trickling filter option for Year 2025 are shown in Table 8.1. below:

**Table 8.1: Option 1 Process Design Parameters**

| Parameter | Unit | Value |
|---|---|---|
| Organic load from PST/Imhoff Tank (40% reduction) | kg BOD/d | 34 |
| Hydraulic load | kL/d | 203 |
| Design organic loading | kg BOD/m$^3$/d | 0.07 |
| Design hydraulic loading | m$^3$/m$^3$/d | 0.41 |
| Controlling parameter | | Hydraulic loading |
| Total volume of media required | m$^3$ | 495 |
| Depth of filter | m | 2 |
| Diameter of new filter | m | 12.5 |

***Advantages:***

- Low operating cost
- Simple to operate
- Will achieve some nitrification
- Existing humus tank can be utilised.

***Disadvantages:***

- Requires large land space
- New PST and Digester will be required for this option.
- Unlikely to achieve significant overall nutrient removal

**Cost Estimates**

Estimated capital cost: $ 1,106,000

Estimated operating cost $ 64,000 per annum

### 8.2.2 Option 2 – Aerated Pond

Primary settled sewage or effluent from the trickling filters would be aerated in a large open pond. To achieve treatment and minimise sludge production, a minimum retention time of 5 days is recommended. At Theodore STP this could be built as a new pond with an impervious liner rather than upgrading the existing pond.

This system would again require a new inlet works for screenings and grit removal. The existing Imhoff Tank, trickling filter and humus tank will be retained.

The pond would be a minimum 3.5 m deep and would be aerated by surface aerators.
An important design feature of aerated ponds is the need to ensure the solids in the pond remain full suspended at all times, hence mixing must be applied at all times. The mixing is usually provided by the surface aerators and in most instances the mixing power required far outweighs the power need for biological treatment.

Nutrient removal is limited in the biological process; phosphorus removal has to completely rely on the chemical dosing and near full nitrification will be expected in the aerated pond while some denitrification will take place in the trickling filter through the recycling of the aerated pond effluent.

**Process Design**

The main process design parameters for the aerated pond option for Year 2025 are shown in Table 9.2 below. In addition to that a new inlet works and PST would be required

**Table 8.2: Option 2 Process Design Parameters**

| Parameter | Unit | Value |
|---|---|---|
| Design retention time | d | 5 |
| ADWF for Year 2025 | kL/d | 203 |
| Pond volume | $m^3$ | 1015 |
| Approximate dimensions L x W x D | m | 18 x 18 x 3.5 |
| Mixing intensity | $W/m^3$ | 30 |
| Installed power for mixing | kW | 30 |
| Installed power for biological treatment | kW | 7.6 |

Separation of the solid and liquid components of the pond will need to be undertaken prior to reuse/discharge. One option is to utilise the existing humus tank which is hydraulically adequate for the proposed future loading. Alternatively an additional filtration unit is required after the aerated pond to separate sludge and water.

**Treated Effluent Quality**

This option will produce a treated effluent BOD in the order of 10 to 20 mg/L, the suspended solids could be in the range 20 – 30 mg/L. The process will reduce ammonia in the sewage by aeration and some denitrification may occur. The process will not reduce phosphorous except by limited adsorption on the biomass with a high probability of release in the solids settlement/filtration stage. Filtration or sedimentation will be required after the aerated pond process.

***Advantages:***

- Low capital cost
- Simple to operate
- Robust
- Will achieve some nutrient removal via volatilisation

***Disadvantages:***

- High operating cost
- Requires PST, Digester and trickling filter

**Cost Estimates**

| | |
|---|---|
| Estimated capital cost: | $ 792,000 |
| Estimated operating cost | $ 73,200 per annum |

This option has a high operating cost primarily due to the power required to ensure the pond is fully mixed for 24 hours per day.

### 8.2.3 Option 3 - Combined Processes, Trickling Filter/Activated Sludge

Several treatment processes are available that combine trickling filters with activated sludge based processes. These processes are known generally as combined processes and were originally developed for upgrading trickling filter plants, however; the advent of BNR processes and the environmental requirement to remove nutrients from the wastewater has seen these processes little used in recent years.

The trickling filter/activated sludge process takes effluent from the trickling filter and aerates the effluent in an aeration tank with a retention of 3 – 4 hours, before recycling the aerated effluent back to the filter from the humus tank. The aerated water with a high dissolved oxygen concentration provides additional oxygen for the micro-organisms in the trickling filter and hence increases the treatment capacity of the filter.

**Process Design**

A new inlet works would be required.

Effluent from the trickling filter is directed to a small aeration tank of 35 $m^3$ capacity. An aeration system in the contact tank will control the dissolved oxygen concentration. The process will use the existing humus tank to separate solids from the system. The effluent is then recycled back to the trickling filter at a rate of up to ADWF. Table 9.3 provides the design parameters for the combined process, trickling filter/Activated Sludge system.

**Table 8.3: Option 3, Process Design Parameters**

| Parameter | Unit | Value |
|---|---|---|
| Contact tank volume | $m^3$ | 35 |
| Retention time | hrs | 4.0 |
| Volume of air | $m^3/h$ | 1.1 |
| Blower power installed | kW | 2.5 |

#### Treated Effluent Quality

The treated effluent quality will be in the order of 20mg/L BOD and 30 mg/L suspended solids after secondary treatment. There will be no appreciable reduction of nutrients. Following the secondary treatment filtration/sedimentation is required to reach targeted effluent quality.

#### Advantages

- The existing trickling filter will be utilised;
- Moderate capital cost;
- Low operating cost;
- Simple operation;
- Potential to improve BOD and suspended solids removal.

#### Disadvantages:

- Dependent on life expectancy of the existing structures;
- New PST and Digester required
- Trickling filter may become prone to blockages as the BOD load increased
- Limited nutrient removal

#### Cost Estimates

Estimated capital cost: $ 985,000

Estimated operating cost $ 76,800 per annum

### 8.2.4 Option 4 - Continuous Activated Sludge

#### General Comments

This option would decommission the trickling filter and construct a new activated sludge plant comprising a bioreactor, clarifier and Return Activated Sludge (RAS) pumping station, as with other options a new inlet works will be included.

The new bioreactor would be a single tank divided into two compartments to accommodate the aerobic and anoxic zones. The process is not designed to specifically remove phosphorous although some reduction will occur due to adsorption on the biomass. A new clarifier will be required to separate the biomass from the treated effluent.

A return pump line will be required to return the activated sludge from the clarifier to bioreactor. If the process is operated at a Sludge Age of greater than 15 days waste activated sludge from the bioreactor would be pumped directly to sludge dewatering without the need for a new digester.

### Process Design

The flow to the activated sludge plant would be limited to 3ADWF. Screened and degritted sewage from the Inlet Works would be transported to the bioreactor. The process will incorporate an anoxic zone and aerobic zone for nitrification/denitrification. Any requirement for phosphorous removal will be achieved by chemical dosing external to the bioreactor. Along with the usual return activated sludge recycle the process will incorporate a recycle from the aerobic zone back to the inlet of the anoxic zone. The bioreactor will have a total volume of 220 $m^3$ and be divided into 2 separate cells. The existing humus tank can be utilised for the secondary clarifier.

A new return activated sludge pump station would be built and used to remove waste activated sludge from the process through a separate pipe.

**Figure 8-1: Process Flow Diagram of Continuous Aeration System**

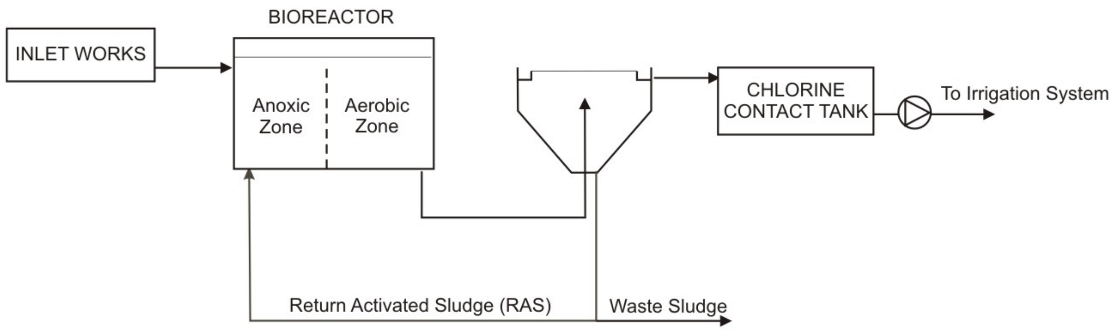



**Table 8.4: Process Design Parameters**

| Parameter | | Unit | Value |
|---|---|---|---|
| Design flow | | kL/d | 203 |
| Bioreactor volume | | $m^3$ | 220 |
| Bioreactor volume fractions – | Aerobic | | 0.59 |
| | Anoxic 1 | | 0.41 |
| Bioreactor retention times – | Aerobic | h | 11.5 |
| | Anoxic | h | 7.8 |
| MLSS | | mg/L | 4000 |
| Sludge Age | | d | 15 |
| Oxygen requirement (SOTR) | | kg $O_2$/d | 272 |
| Peak factor = 1.4 | | kg $O_2$/h | 15.9 |
| Aeration power installed (blower) | | kW | 5.0 |

### Treated Effluent Quality

Modelling of the process using BioWin predicts the following treated effluent quality:

| | |
|---|---|
| BOD | 6.4 mg/L |
| Suspended solids | 16 mg/L |
| Total nitrogen | 4.9 mg/L |
| Total phosphorous | 6.2 mg/L |

**Advantages**

- Moderate to high capital cost;
- Small land space requirements
- Some complexity of operation but process will be fully automated
- Excellent BOD and suspended solids in treated effluent;
- Good nitrogen removal;
- Chemical dosing for phosphorous removal can be retrofitted;
- Produces a good settling sludge, and
- Would allow for discharge to waterways without severe environmental concern.

**Disadvantages:**

- Limited phosphorous removal – may require increased irrigation area;
- Would need chemical dosing for appreciable phosphorous reduction;
- High operating cost when compared to the existing trickling filter plant;
- Would require new pumping for sludge return.

**Cost Estimates**

Tertiary filtration is not included in the cost estimation.

Estimated capital cost: $ 610,000

Estimated operating cost $ 84,500 per annum

### 8.2.5 Option 5 - Intermittent Aeration

The trickling filter would be decommissioned and replaced with an intermittent aeration bioreactor with a volume of 400 $m^3$. A new inlet works would be included to remove screenings and grit from the raw sewage. Treated effluent would discharge into a balance tank to await disinfection. The process would be designed to provide biological nitrification and denitrification. Any phosphorous removal required will be by chemical precipitation and filtration. The process does not require a separate secondary clarifier.

**Process Design**

Raw sewage from the inlet works will be transferred to the bioreactor. Decanted supernatant discharges from the reactors at a high rate and will be discharged into a balance tank or pond from where it will be pumped at a controlled rate to the tertiary treatment process.

The process will operate in a sequence of Aeration/Settlement/Decant. The time for each full sequence will be in the order of 4 – 8 hours. Figure 8.2 shows a schematic process flow diagram of the Intermittent Aeration treatment process.

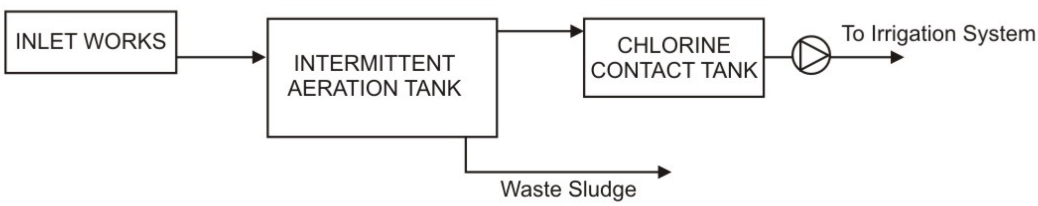



**Figure 8-2: Process Flow Diagram of Intermittent Aeration System**

Table 8.6 provides the Design parameters for the Intermittent Aeration process system.

**Table 8.5: Option 5 Process Design Parameters**

| Parameter | Unit | Value |
|---|---|---|
| Bioreactor volume (total) | $m^3$ | 400 |
| Full liquid depth | m | 3.5 |
| Settled fraction | | 0.25 |
| MLSS | mg/L | 3500 |
| Sludge age | d | 30 |
| Oxygen requirement (AOTR) | kg $O_2$/d | 54 |
| Oxygen requirement (SOTR)<br>Peak factor = 1.4 | kg $O_2$/d<br>kg $O_2$/h | 119<br>14 |
| Blower power absorbed | kW | 10 |

### Treated Effluent Quality

The process will produce a treated effluent quality with 10 mg/L BOD, 10 mg/L suspended solids and <10 mg/L total nitrogen can be achieved. Treated effluent phosphorous concentrations as low as 1.0 mg/L can be achieved by chemical dosing.

Intermittent aeration, secondary treatment can achieve the targeted effluent quality of Class B; additional tertiary treatment facilities must be employed in the process train to achieve Class A.

### Advantages

- Moderate to high operating cost;
- Medium complexity of operation in combining SBR technology with biological nitrogen removal;
- Does not require a separate clarifier;
- Stable process with good buffering capacity due to large reactor volume;
- Good nitrogen removal can be achieved, and
- Stabilised waste sludge produced.

### Disadvantages:

- Medium to high capital cost
- Requires a decant balance tank;
- New Digester required
- Large tankage required and consequently a large land area, and
- Requires chemical dosing to achieve phosphorous removal.

### Cost Estimates

The tertiary filtration system is not included in the cost estimation.

Estimated capital cost: $ 880,000

Estimated operating cost $ 71,000 per annum

### 8.2.6 Option 6 – MBR process

Membrane bioreactors are a recent innovation in sewage treatment technology. The membranes, specially developed for use in sewage treatment, are used to separate the activated sludge biomass from the treated effluent. The membranes used are ultra-filtration membranes with a nominal pore size around 0.02 microns. At this size the membranes will remove virtually all suspended solids from the treated effluent and produce a treated water meeting Class A+ requirements with the exception of disinfection, in order to meet the desired Log-removal of pathogens UV disinfection or Ozonation is required together with chlorination to provide a residual. In effect the membrane bioreactor can replace the secondary clarifier and media filter of the tertiary filtration process. A further major benefit of the process is the ability to operate at much higher mixed liquor suspended solids concentrations than conventional activated sludge systems – operating in the range 9,000 – 15,000 mg/L MLSS compared with the 2,000 – 5,000 mg/L for conventional systems. The effect of this is a significant reduction in bioreactor volumes.

**Process design**

The existing Imhoff tank, trickling filter and clarifier will be decommissioned and replaced with a completely new MBR plant. The existing facilities will be kept operating until the new facilities are commissioned.

The MBR process consists of the following units and the facilities:

- Inlet works, which includes raw sewage balancing tank, coarse screen, grit removal and fine screen;
- Bioreactor, which includes Anoxic and aerobic compartments, ‘A’ recycle and fine bubble aeration system;
- Membrane chamber, which includes membrane cassette, air scouring system, sludge turn, backwashing and membrane cleaning systems;
- Disinfection, which includes sodium hypochlorite dosing and chlorine contact chamber;
- UV Disinfection;
- Treated water storage tank;
- Sludge treatment, which includes sludge dewatering and supernatant recycling.

The sewage flow to the plant is affected by the residence activities, especially to those small catchment of medium and small sizes, the flowrate fluctuates greatly with relative high flow during peak hour and a small flow around mid-night time. To reduce the capital cost of the treatment facilities of the option, raw sewage balancing tank is recommended for the MBR process to balance the raw sewage flowrate and raw quality.

Figure 8.3 shows a schematic process flow diagram of the Membrane Bioreactor treatment process.

The MBR option is developed based on MLE process which will achieve Nitrogen removal by biological nitrification and denitrification, the phosphorous removal of the process relies on the alum dosing.

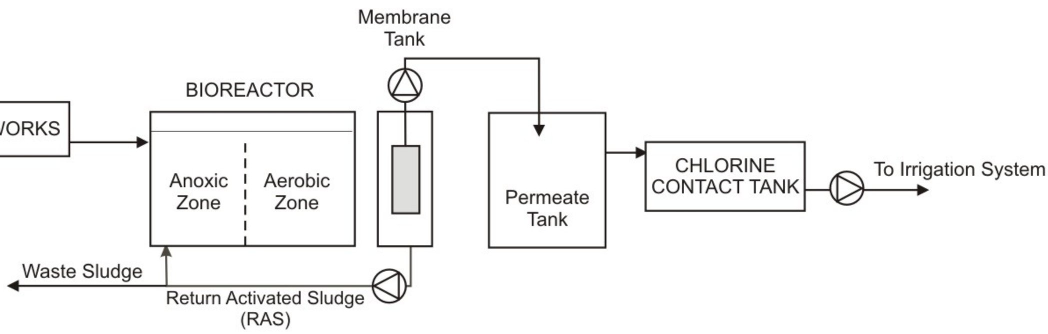



Figure 8.3: Process flow diagram of MBR process

Table 8.6 shows the MBR process design parameters and the main facilities in the treatment train.

**Table 8.6: Main Process Design Parameters**

| Parameter | Unit | Value |
|---|---|---|
| Design flow (ADWF) | kL/d | 203 |
| Coarse screen aperture | mm | 10 |
| Fine screen aperture | mm | 1 |
| Diameter of grit chamber | m | 1.0 |
| Raw sewage balancing tank volume | kL | 85 |
| Wet weather storage pond | kL | 1000 |
| Bioreactor volume | $m^3$ | 80 |
| Bioreactor volume fractions – Aerobic<br>Anoxic 1 | fraction<br>fraction | 0.59<br>0.41 |
| Bioreactor retention times – Aerobic<br>Anoxic | h<br>h | 5.5<br>4.0 |
| MLSS | mg/L | 8000 |
| Sludge Age | d | 12.5 |
| Oxygen requirement (SOTR)<br>Peak factor = 1.4 | kg $O_2$/d<br>kg $O_2$/h | 272<br>15.9 |
| Aeration power installed (blower) | kW | 5.0 |
| RAS recycle ratio | % | 400 |
| 'A' recycle ratio | % | 300 |
| Membrane flux (ADWF) | L/$m^2$.h | 10.6 |
| Membrane area (installed) | $m^2$ | 800 |
| Operating flux at 1.5xADWF | L/$m^2$.h | 15.9 |
| Membrane chamber volume | kL | 25 |
| Treated water storage tank volume | kL | 3,500 |
| Sludge production | kg/d | 43 |
| Sludge wasted from membrane chamber | kL/d | 3.6 |

#### Treated Effluent Quality

It is predicted the following treated effluent quality:

| | |
|---|---|
| BOD | 5.0 mg/L |
| Turbidity | 2.0 NTU |
| Suspended solids | 5.0 mg/L |
| Total nitrogen | 10.0 mg/L |
| pH | 6.0 – 8.5 |
| Conductivity | 1,600 µS/cm |

The option will achieve Class A or Class A+ recycled water quality. The TP removal rate can be controlled by alum dosing, and TN removal rate can be controlled by adjusting the 'A' recycle ratio based on the requirement in the future.

#### Advantages

- Good BOD and suspended solid removal;
- Good nitrogen removal;
- Would allow for discharge to surrounding water bodies without causing environmental harm.
- Small footprint with high MLSS concentration.

#### Disadvantages:

- High operating cost when compared to the existing trickling filter plant;
- High level of operator skill required; and
- High capital cost.

#### Cost Estimates

Estimated capital cost: $ 1,230,000

Estimated operating cost $ 194,000 per annum (includes provision for membrane replacement)

### 8.2.7 Option 7 - Package Treatment Plant

A number of commercially produced "Package Sewage Treatment Plants" are available that will economically treat flow from a community the size of Theodore.

These units are generally based on the aerobic activated sludge process although versions are available based on fixed film technology similar to trickling filters.

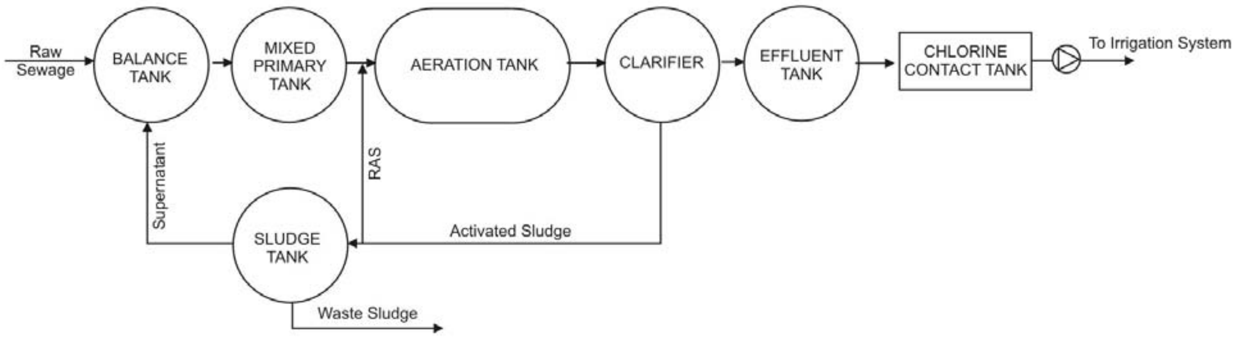
*Enviroflow* produce a suitable unit that has been in use for a number of years. *Enviroflow* have provided a budget price for a packaged system to treat an average dry weather flow of 250kL/day to produce Class C effluent. These units could be used in conjunction with the existing clarifier and chlorine contact tank to upgrade the effluent to Class B suitable for irrigation. A number of other companies produce similar systems and Enviroflow is only used as an example, if Council decides to go with this option it would be recommended that acquisition be based on a performance based supply, install and commission basis.

The *Enviroflow* Offer is attached as an Appendix to this report for information only.

In summary the *Enviroflow* system (and similar commercially available systems) consists of the following:

- Balance Tank/Inlet pump station complete with grinder pumps (this negates the need for new inlet screens etc);
- Mixed Primary Tank which acts as a Anoxic Zone
- Aerobic Tank(s) complete with diffused air system for aerobic biological treatment
- Clarifier for separation of sludge from effluent
- Sludge Storage Tank – to allow for additional consolidation of waste sludge
- Disinfection system consisting of chlorine contact tank.

The layout of the proposed system is shown below:




### Treated Effluent Quality

It is predicted the following treated effluent quality:

| | |
|---|---|
| BOD | 20 mg/L |
| Suspended solids | 30 mg/L |
| Total nitrogen | 30.0 mg/L |
| pH | 6.0 – 8.5 |
| Coliforms | 1,000 cfu/100mL |

The option will achieve Class C recycled water quality. The total Phosphorus concentration can be controlled by including Alum dosing prior to the existing Clarifier. The use of the existing Chlorine contact Tank will allow for better disinfection and reduce the Coliform count to Class B levels.

### Advantages

- Good BOD and suspended solid removal;
- Reasonable Nitrogen removal;
- Would allow for discharge to irrigation without causing environmental harm;
- Can, if desired, be acquired on a lease basis with options for operation and maintenance;
- Can be readily de-commissioned and related to other sites.

### Disadvantages:

- Higher operating cost when compared to the existing trickling filter plant; and
- Higher level of operator skill required.

**Cost Estimates**

| | |
|---|---|
| Estimated capital cost: | $ 850,000 |
| Estimated operating cost | $ 71,000 per annum |

## 8.3 Summary of Options

The options considered for upgrading the secondary treatment process at Theodore are summarised below in Table 8.7.

**Table 8.7: Summary of Options**

| Option | Description | Capex $('000s) | Opex $/a | 20 year NPV @6% $,000 |
|---|---|---|---|---|
| 1 | Trickling filters | 1,106 | 58,700 | 989 |
| 2 | Aerated pond | 792 | 68,000 | 1,117 |
| 3 | Trickling filter/activated sludge | 985 | 82,600 | 1,342 |
| 4 | Continuous activated sludge | 610 | 84,500 | 1,357 |
| 5 | SBR | 883 | 71,000 | 1,165 |
| 6 | MBR | 1,230 | 194,000 | 2,338 |
| 7 | Package STP | 860 | 68,000 | 1,087 |

All the options except Option 6; the MBR process, will require additional treatment to achieve Class A  and A+ recycled water quality.

- Options 1, 2, 3 and 7 do not include any significant nutrient removal and should consistently produce effluent of Class B standard suitable for irrigation;
- Options 4 and 5 will produce some nutrient removal; mainly nitrogen and should consistently produce Class B effluent;
- Option 6 MBR configured on MLE process is a technology able to achieve Class A+ treated water quality;
- If phosphorous reduction required because of a change in the soil types or condition, on the irrigation areas or for discharge to surrounding water bodies, then chemical dosing will be required for all Options.

## 8.4 Selection of the Preferred Option

All processes will produce an effluent that can be used for irrigation of crops.  Option 6 has phosphorous removal as part of the system but this adds a significant level of complexity to the process.  At this time the soil analysis indicates that phosphorous removal is not required.

Should phosphorous removal become necessary at some time in the future it can be retrofitted to all options as a chemical precipitation process to remove the phosphorous.

Option 6 will produce sustained Class A+ effluent quality, and there is no further process upgrading requirement in the future, however as the majority of the effluent produced only needs to meet Class B quality the additional operational costs are not warranted at this time.

Option 1; the provision of an additional Trickling Filter with PST and Digester is the best financial option based on the results of a NPV analysis. The process is simple to operate and will require minimal operator attention. The construction works could be staged with provision of new inlet works and trickling filter as Stage 1 with new Primary Sedimentation Tank and Digester added at a later date.

Option 7; the provision of a "Package" STP is second preference based on the NPV assessment. The advantage of the Package STP option is that it could be acquired on a purchase or lease arrangement with the potential for Council into entering into an operation and maintenance agreement with the supplier. In addition the system can be readily de-commissioned and relocated if required.

The estimated cost of this option is $850,000 with an additional $20,000 required for provision of connecting pipework etc, the requirement for a new inlet works has also been removed.. This option will produce Class B effluent which is suitable for the majority of uses. Additional polishing of effluent to Class A+ will be required for use in the truck wash bay etc. at an estimated cost of $380,000.

In view of the uncertainties regarding growth in Theodore and ultimate effluent discharge and quality requirement the most cost-effective, flexible option is Option 7; a Package STP.

# 9.0 CLASS A+

According to Risk Assessment techniques Class A+ recycled water is required for the truck washdown bays and standpipes. The quality must satisfy the requirements as per the *Queensland Water Recycling Guidelines*. The process should also provide $Cl_2$ residual in the supplied recycled water to maintain the disinfection effect within tankers. A new treatment unit is required in addition to the conventional treatment plant to produce effluent of this quality.

The *Queensland Recycle Water Guidelines* specify as part of the *"Treatment objective from raw sewage"* log reduction requirements to meet Class A+ water quality. Indicative log reduction capabilities of various treatment elements included in the *Guidelines* and reproduced below:

**Table 9-1 Indicative Log Reductions** (Table 6.1 of Guidelines)

| | **Indicative Log Reductions** | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Treatment Process** | E.coli | Bacterial pathogens | Viruses | Phage | Giardia | Crypto | Clostridium perfringens | Helminths |
| Primary Treatment | 0-0.5 | 0-0.5 | 0-0.1 | N/A | 0.5-1.0 | 0-0.5 | 0-0.5 | 0-2.0 |
| Secondary Treatment | 1.0-3.0 | 1.0-3.0 | 0-2.0 | 0.5-2.5 | 0.5-1.5 | 0.5-1.0 | 0.5-1.0 | 0-2.0 |
| Dual Media Filtration | 0-1.0 | 0-1.0 | 0.5-3.0 | 1.0-4.0 | 1.0-3.0 | 1.5-2.5 | 0-1.0 | 2.0-3.0 |
| Membrane Filtration | 0 – 1.0 | 3.5 - >6.0 | 2.5 - >6.0 | 3.0 – 6.0 | >6.0 | >6.0 | .6.0 | >3.0 |
| Lagoon Storage | 1.0-5.0 | 1.0-5.0 | 1.0-4.0 | 1.0-4.0 | 3.0-4.0 | 1.0-3.5 | N/A | 1.5->3.0 |
| Chlorination | 2.0-6.0 | 2.0-6.0 | 1.0-3.0 | 0-2.5 | 0.5-1.5 | 0-0.5 | 1.0-2.0 | 0-1.0 |
| Ozonation | 2.0-6.0 | 2.0-6.0 | 3.0-6.0 | 2.0-6.0 | N/A | N/A | 0-0.5 | N/A |
| UV Light | 2.0->4.0 | 2.0->4.0 | >1.0 adenovirus >3.0 enterovirus, hepatitis A | 3.0-6.0 | >3.0 | >3.0 | N/A | N/A |
| Wetlands – surface flow | 1.5-2.5 | 1.0 | N/A | 1.5-2.0 | 0.5-1.5 | 0.5-1.0 | 1.5 | 0-2.0 |
| Wetlands –subsurface flow | 0.5-3.0 | 1.0-3.0 | N/A | 1.5-2.0 | 1.5-2.0 | 0.5-1.0 | 1.0-3.0 | N/A |

The total log reduction of the existing treatment processes at Theodore and Biloela can be calculated by adding the log reduction of all the existing treatment processes as per Table 9.2 below:

**Table 9-2 Log Reductions of Theodore STP**

| Treatment | Indicative Log Reductions | | |
|---|---|---|---|
| | E.coli | Bacteriophage | Clostridium Perfringens |
| 1) Primary Treatment | 0 - 0.5 | N/A | 0 – 0.5 |
| 2) Secondary Treatment | 1 – 3 | 0.5 – 2.5 | 0.5 – 1 |
| 3) Chlorination | 2 – 6 | 0 – 2.5 | 1 – 2 |
| Total Existing Log Reduction 1) +2) +3) = | 3 – 9.5 | 0.5 – 5 | 1.5 – 3.5 |

The total existing log reduction is then compared to Class A+ recycled water requirements to determine the required log reduction of the new augmentation. Each process element of the new augmentation is then chosen so the total log reduction meets the objective.

A treatment process that satisfies the above objective may comprise multiple processes:

- Filtration of the existing effluent to remove any residual particulate matter and to ensure the treated effluent has a turbidity <2 NTU (as per the guideline).
- Additional disinfection such as Ultraviolet application or Ozonation to kill any residual pathogens.
- Chlorine Dosing to provide the recycled water with free chlorine residual to prevent regrowth of any micro-organisms

## 9.1 Media Filtration

Media filtration can remove any residual fine particles in the clarified effluent. The turbidity of the filtered water can meet the Class A+ requirements as well as removing some of the micro-organisms present.

As the clarified water passes downwards through the media bed residual particles will be trapped in the media and being removed from the flow. Backwashing is required to release the solids from within the media bed and to flush the solids from the filter. The wastewater produced by backwashing can be returned to the works inlet for treatment.

**Advantages**

- Moderate capital cost
- Moderate operating cost

**Disadvantages**

- May not be reliable to achieve <1 NTU if the inlet water quality is inconsistent

## 9.2 Membrane Processes

Pressure-driven membrane processes are common in water treatment. These processes are classified in accordance with pore size and include reverse osmosis (RO), nanofiltration (NF), ultrafiltration (UF) and microfiltration (MF).

Ultrafiltration membrane filtration is also capable of providing microbiological removal to meet the *Guidelines*. Due to the very small physical pore size, membrane filtration is also capable of reducing turbidity down to less than 1 NTU.

The process would involve passing the treated effluent through the membrane filtration modules under pressure using the cross-flow process where the dirty water is on the outside of the membrane and the clean water passes through to the centre of the membrane.

**Advantages**

- Provide high level of reduction in all pathogens and organisms
- Can reliable achieve <1NTU; therefore, there is no need for a media filter.
- Low suspended solids in the final effluent
- Small footprint.

**Disadvantages**

- Membranes may foul under high suspended solids loading
- Moderately high capital cost (replacement of membranes)

## 9.3 Ultra Violet Irradiation (UV)

Ultra Violet (UV) irradiation has been included to provide a higher microbiological kill rate. UV disinfection is relatively effective in removing all pathogens and organism with its strongest phages reduction.

Standard UV systems for the disinfection of sewage treatment usually operate in the order of 80 – 120 mW.s/cm$^2$. High intensity UV systems are design to provide 240 – 280 mW.s/cm$^2$ and provide a higher level of disinfection than standard systems.

For Theodore, a low-pressure system would be used and mounted in a spool piece inserted into the filtered water pipe line.

The UV tubes will require regular cleaning to ensure there is no build up of film on the tubes and maximum transmission efficiency is maintained. It is usual to change the UV lamps every 8,000 -10,000 hours.

**Advantages**

- Strong phages reduction and relatively effective in reducing a broad range of organisms
- Low to moderate capital cost
- Low complexity
- Low operational cost

**Disadvantages**

- Less effective if water contains high suspended solid, colour and turbidity
- Does not provide residual disinfection effect

## 9.4 Chlorine Dosing

The *Guidelines* and the risk assessment, require a chlorine residual for the recycled water to meet Class A+ qualities. In this situation chlorine dosing will also be an additional disinfection after membrane filtering and UV.

Chlorine solution is dosed to ensure a 0.5 – 1.0 mg/L residual concentration of free chlorine after filtration. Free chlorine would be measured directly on-line by a chlorine analyser linked to the chlorine dosing pump. This will ensure the correct residual is maintained at all times.

**Advantages**

- Effective at creating residual disinfection
- Moderate capital cost
- Low complexity
- Moderate operational cost

**Disadvantages**

- Chemical handling requires extra care due to corrosive properties and potential harm to health if inhaled, handled or ingested.

## 9.5 Ozonation

Ozone must be produced on site as it is a highly reactive, unstable gas. It is produced from air or oxygen subjected to an electrical discharge. It dissociates rapidly back to oxygen. Having an unstable oxygen radical, ozone is the most powerful disinfect available and requires only 5 – 10 minutes contact time.

**Advantages**

- Most powerful disinfectant commonly available;
- Generated on site from air.

**Disadvantages**

- Higher capital cost than chlorination and UV technology
- Does not produce a residual disinfection effect in the treated water and chlorination would still be required
- High operating cost (power consumption)

## 9.6 Treatment Process Selection

To minimise human health risk, a two-stage system followed by additional chlorination is recommended to ensure effective disinfection and a lasting residual in the final effluent.

The use of membrane filtration will act as an effective disinfectant due to the reduction of a wide range of organisms. It also can eliminate the need of using media filter as it is capable of reducing turbidity similar to media filtration.

Additional disinfection is recommended as double protection in case of membrane breakthrough. UV is suggested for its strong virus reduction and relatively effective in reducing a broad range of organisms. It is also easy and cheap to operate and maintain.

Ozonation can be a practical option for large treatment plants but generally for smaller plants it is not considered economically practical unless special circumstances dictate its use.

If membrane filtration and UV disinfection is provided the log reduction requirement of the *Guidelines* is achieved as is shown in Table 9.3

**Table 9-3 Indicative Log Reductions of the Recommended Augmentation**

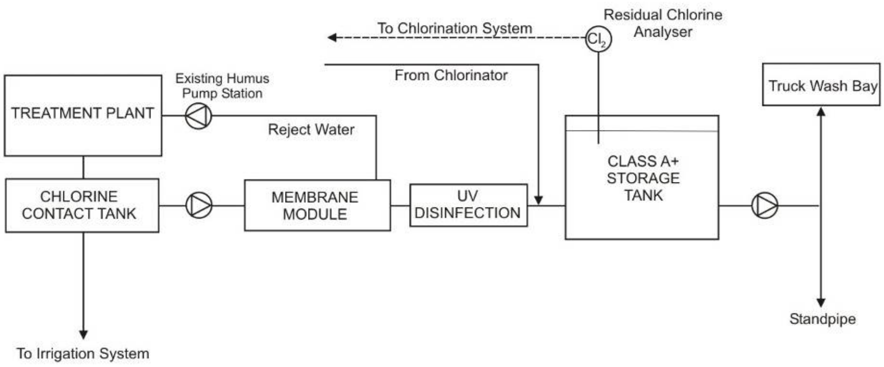
| Treatment | Indicative Log Reductions | | |
|---|---|---|---|
| | E.coli | Bacteriophage | Clostridium Perfringens |
| Total Existing Log Reduction = | 3 – 9.5 | 0.5 – 5 | 1.5 – 3.5 |
| Class A+ Requirement | 5 | 6 | 5 |
| Additional Log Reduction Required (minimum) | 2 | 5.5 | 3.5 |
| Recommended Augmentation: | | | |
| a) Membrane Module | 3.5 - 6 | 3 - 6 | > 6 |
| b) UV Disinfection | 2 - 4 | 3 - 6 | N/A |
| Augmentation log reduction: a) + b) = | 5.5 – 10 | 6 – 12 | > 6 |
| Total log reduction after the augmentation = | 8.5 – 19.5 | 6.5 – 17 | > 6 |

* Log reduction credit for new chlorination is not counted toward the total log reduction as it is already included in the existing total log reduction.

## 9.7 Proposed Treatment Process Components

The recommended augmentation comprises of a membrane module, an UV disinfection unit, a chlorine dosing unit. A 10m$^3$ Class A+ storage tank will be required to provide minimum chlorine contact time and with transfer pumps to provide pressure at the standpipes and truck washdown bay.

The process flow diagram is shown as follow:




The chlorinated effluent from the treatment plant will be transferred to the new treatment module from the outlet of the Chlorine Contact Tank. The treatment flow rate of 1 L/sec is controlled by the level of the effluent in the Class A+ storage tank. The required flow rate to service the standpipes and washdown bay is 4 L/s. The feed water flows through the membrane module and UV system. The reject water is discharged back to the STP during backwashing steps through the existing Humus Pump Station.

The clear water exiting the membrane module flows through the UV disinfection system further disinfection and is dosed with chlorine solution before entering the storage tank. The storage tank will provide chlorine contact time to ensure the complete disinfection and also act as a balance storage. The $Cl_2$ residual level is monitored and the controlled automatically by adjusting the stroke of the dosing pumps.

The estimated cost of these works is:

| | Estimated Cost |
|---|---|
| UF Membrane Module | $180,000 |
| UV Disinfection Module | $26,000 |
| Chlorine Dosing system and $Cl_2$ monitoring and control | $8,000 |
| Water Storage Tank | $15,000 |
| Pumps | $12,000 |
| Electrical and Controls | $75,000 |
| Contingency cost (20%) | $64,000 |
| **TOTAL:** | **$380,000** |

Please note that it in order to limit the risk to Council validation and verification of achievement of Class A+ quality is required to be undertaken. The above costing does not include the expenses for those procedures.

# 10.0 SLUDGE TREATMENT AND DEWATERING

## 10.1 Considerations

Considering the findings of the treatment plant review and the above recommendations for a new treatment module to be added, the existing anaerobic sludge digestion compartment of the Imhoff Tank may be available in the short-term to digest the raw and humus sludges produced by the process.

## 10.2 Treatment Process Options

All biological treatment processes will produce a sludge that is mainly organic in nature. If the sludge is not properly handled it can be a major source of odours from the treatment plant. Treatment of the sludge is aimed at reducing the organic matter in the sludge to a point where the sludge can be considered “stable” and will not generate a nuisance.

There are a range of processes, aerobic and anaerobic, available for the treatment of sludges. In selecting a suitable process, consideration must be given to the main treatment process. In the case of the proposed augmentation the existing anaerobic digestion system is appropriate and can easily be augmented by replacement of the Imhoff Tank’s sludge digestion compartment with a dedicated slow rate anaerobic digester of $90m^3$ capacity.

## 10.3 Sludge Dewatering

Currently the Theodore treatment plant dewaters the sludge on drying beds allowing the sludge to dry to a “spadeable” consistency before lifting and eventually removing from site to landfill, and experimenting with composting mixtures.

The current beds are unserviceable and should be replaced.

There are several other methods can be used for replacing the existed dewatering methods such as use of Geotube® and mechanical dewatering etc.

### 10.3.1 Sludge Drying Beds

The Queensland Guidelines for sludge drying beds are based on the EP loading to the plant and the sewage treatment plant, specifically the sludge quality. For a plant such as that recommended for Theodore, the sludge would be activated sludge for which the Guidelines recommend the sludge drying bed area be $0.1m^2$ per EP loading to the plant. On this basis the drying bed areas required for the upgraded treatment plant in 2025 is 81 $m^2$.

It is considered that the new beds would be hard surfaced to allow removal of dried sludge by Bobcat or similar equipment to reduce manual labour.

The estimated cost of sludge drying beds is $156,000 with operational costs of $8,000 per annum.

### 10.3.2 Geotube®

A Geotube® – a porous geotextile bag. Sludge is discharged into the bag and filtrate drains out through the pores in the geotextile. The bags are designed to accept multiple fillings and once the capacity of the bag is reached it is left to drain before being opened and the sludge allowed to dry.

The use of Geotube® will require provision of a bunded area similar to a sludge drying bed, the cost of this work is estimated at $60,000.

In addition the Geotube® will cost approximately $8,000 per annum.

### 10.3.3 Mechanical Dewatering

The use of mechanical dewatering equipment such as a belt filter press or centrifuge is alternative option. A belt filter press is preferable to a centrifuge considering the relatively small sludge volume of around 3.6 kL/d. The filtrate will be returned to inlet works for treatment. Dewatered sludge from the press will generally achieve a dry solids content of 15% – 20%, dewatered sludge is stockpiled on site within a bunded area and then can be removed to landfill site at a convenient time.

The estimated cost of a Belt Filter Press for Theodore similar to the system being installed at Moura STP is $100,000 with annual costs of $5,000

**Table 10.1: Summary of Options**

| **Option** | **Description** | **Capex** $(‘000s) | **Opex** $/a | **20 year NPV** @6% $,000 |
|---|---|---|---|---|
| 1 | Sludge Drying Beds | 156 | $8,000 | 187.8 |
| 2 | Geotubes | 60 | $8,000 | 184.2 |
| 3 | Belt Filter Press | 100 | $5,000 | 132.2 |

The sludge dewatering by belt press is recommended for new Theodore sewage treatment plant.

# 11.0 WATER RECYCLING

## 11.1 General Consideration

Treated wastewater is being increasingly recognised as a valuable resource and it is becoming common practice to re-use treated wastewater for applications that do not necessarily require potable water quality.

Such re-use applications include;

- Treated effluent returned to residential and commercial properties for watering gardens, toilet flushing, wash-down water, or
- Irrigation of commercial crops, public parklands and open space, sports ovals, golf courses, truck washing bay and standpipe etc.

To ensure treated wastewater re-use schemes are environmentally sustainable in the long term, the Queensland Environmental Protection Agency (EPA) has developed draft guidelines to protect public health and to protect the environment.

The guidelines provide classifications of recycled water for various applications, and these are based on the bacteriological and aesthetic quality rather than chemical quality.

The guidelines provide some discussion on the effects of nutrient loading onto surrounding environments such as water bodies, and soil conditions, however do not provide a standard application rate that will ensure long term environmental sustainability. This balance between the water and nutrient loading from specific effluent irrigation schemes to certain soil profiles, crop types and rate of harvesting, condition and location of surrounding water bodies (ground and surface) are specific for each site. There is an accepted effluent irrigation modelling program developed that analyses the effect of irrigation and can determine an appropriate rate that will reduce potential environmental harm. This model is referred to as MEDLI – Model for Effluent Disposal for Land Irrigation.

MEDLI simulates the effects of effluent loading to a land application by modeling the effects to the surrounding environment. The EPA accepted computerized hydraulic model to evaluate the sustainability of effluent re-use on land applications is MEDLI (Modeling Effluent Disposal to Land Irrigation). The following factors are considered in the hydraulic modeling program;

- Soil capability and assimilative capacity
- Depth of groundwater and effect effluent is having on groundwater
- Nutrient loading and nutrient harvesting
- Sustainability of irrigation practices
- Wet weather storage capacity required to ensure minimal site run off.

Details of the MEDLI program and it results are provided in Section 11.4.

## 11.2 Current Effluent Reuse Scheme

Effluent reuse schemes and sewage treatment plant operations are closely interrelated components of the treatment process with each component having a direct impact on the successful management of pollutants. Without effective implementation of operational and management procedures, pollutants can adversely impact on the environment, or cause risk to human health. The reuse scheme that Theodore has comprises of;

- Irrigation by the local Apex Club to grow crop, usually sorghum. (11.00ha)
- Irrigation of the public spaces, truck wash bay and standpipe for industrial uses,
- In the past, excess effluent has been released to Lonesome Creek which flows into the Dawson River.

The volume pumped to the Apex Club varies approximately between 2 – 7 ML/month. There is no flow meter on the discharge to Lonesome Creek, however it is assumed that any additional effluent will flow into the creek.

There are additional areas available for irrigation close to the treatment plant but are not currently being utilized, in addition to that, truck wash bay and standpipe for supplying treated water to be use for industrial purposes. The most likely of these to be used would be the Edwards property with approximately 35ha available for irrigation.

## 11.3 Quality of irrigation water

The effluent produced at the Theodore STP is currently considered to be inconsistent and below Class B standard. With the recommended process augmentations implemented, (particularly for tertiary treatment), Class A or Class A+ effluent is expected to be consistently produced. The effluent that is currently being irrigated has relatively low nutrient concentrations, with an average concentration of 14 mg/L total nitrogen, and 6.5 mg/L total phosphorous. There are however relatively high solids loading and significant organic loading, at on average 33mg/L suspended solids and 12mg/L BOD.

The predicted quality of the treated effluent from the new treatment plant is shown in Table 10.1 below.

**Table 11.1**: **Predicted average concentrations of effluent.**

| Contaminants | Units | Effluent Quality |
|---|---|---|
| Biochemical Oxygen Demand | mg/L | 10 |
| Suspended Solids | mg/L | < 10 |
| Total Nitrogen | mg/L | <10 |
| Total phosphorous | mg/L | < |
| Conductivity | µS/cm | 700 |
| pH | - | 6.0 – 8.5 |
| E.coli | CFU/100mL | < 10 |

According to Queensland Guidelines for Safe Use of Recycled Water Class A+ effluent may be used for irrigation in public areas, truck wash and standpipe for industrial dust control etc without controlled conditions. This quality is to be achieved at the end of the treatment train, prior to entering the storage pond Process verification is required followed by regular sampling to demonstrate compliance.

Intermediate storage downstream from the treatment process is often not covered and therefore prone to wild life inhabitation, and plant growth. This often causes contamination from wild life faeces and algae growth, however this is considered to be natural and not sewage related contamination, and accepted for irrigation practices.

Therefore effluent lagoons and uncovered storage dams are acceptable as storage facilities. If these lagoons and dams have adequate void volume for storage during wet weather periods (no irrigation) these can also be utilised as wet weather storage facilities (details in section 12.8).

## 11.4 Model for Effluent Disposal using Land Irrigation (MEDLI)

A computer based hydraulic modeling program has been jointly developed by the CRC for Waste Management and Pollution Control, the Queensland Department of Primary Industries and Natural Resources and Mines (NRM) for the purpose of designing and analysing effluent disposal systems for rural industries and waste water treatment plants using land irrigation. The program is called MEDLI (Model for Effluent Disposal using Land Irrigation). Using actual historical climatic data (rainfall, temperature, evaporation and solar radiation), soil profiles, irrigation area characteristics, and effluent flows and quality for the situation at Theodore MEDLI has been used to provide;

- Nutrient balance over the soil, plants and water tables to ensure long term sustainability;
- Suitable and sustainable irrigation application rate for specified irrigation area;
- Wet weather storage capacity required for a predicted 95% effluent usage (5% loss due to overflowing during periods of excessive wet weather).

A MEDLI model was run for the Theodore effluent irrigation scheme, using the effluent quality predicted to be produced by the augmented treatment system. The model is based on the assumption that rainfall and climatic patterns are evolved over time, and using historical data, a reasonable prediction of future rainfall can be formed. Climatic data for the area of Theodore was obtained from the Department of Natural resources and Mines for the period between the Years 1957 to 2004. Using the predicted effluent quality and soil sample analysis MEDLI provides some water and nutrient balances over the available land areas.

## 11.5 Soil Analysis

Soil samples were taken at various sites across the irrigation area and were used to determine a sustainable irrigation application rate for the sights. There are two aspects of soil sampling, the soil profile analysis and the current concentration of various nutrients, and salts in the soil.

***Soil profiles*** were analyses as to define the type of soil that will be irrigated. This is useful in determining the drainage rates and porosity of the soil.

The ***initial soil condition*** is relevant to determine the current loading and its capacity to accept additional nutrients. Excessive nutrient concentrations in the soil can mean that effluent with elevated concentrations of nutrients must be applied at a reduced rate to ensure the nutrients are not leached into the groundwater or allow high concentrations to run off the land.

The soil analysis results are available in Appendix E (Tony De Vere & Associates) and Table 11.2 gives a qualitative brief summary of the soil samples analysed.

**Table 11.2: Qualitative summary of soil analysis**

| Sample Site | Phosphorous | Nitrogen | Sulphur | Sodicity | Conductivity |
|---|---|---|---|---|---|
| Apex Club Crop (Sorghum) | Medium (Med capacity to Store P) | Low | Medium | Med in top soil High sub soil | Low |

| Sample Site | Phosphorous | Nitrogen | Sulphur | Sodicity | Conductivity |
|---|---|---|---|---|---|
| Edwards Farm Land | Medium (Med capacity to Store P) | Low | Medium | Med in top soil High sub soil | Low |

Note: Top soil is 0-100mm and sub soil is 100-600mm

The soils of the irrigation area have been receiving effluent from the treatment plant for around 20 years. The nutrient analysis for the soils is shown below in Table 11.3.

**Table 11.3: Soil analysis***

| Parameter | Units | Value in Soil | | Guideline Values | | | Comments |
|---|---|---|---|---|---|---|---|
| | | 0 – 10 cm | 50 – 60 cm | Min | Opt | Max | |
| **Nitrate nitrogen** | ppm | 4.1 | 0.7 | 10 | 20 | 50 | Ideal <10 in subsoil |
| **Ammonia nitrogen** | ppm | 4.0 | 3.7 | | | 40 | |
| **Total nitrogen** | ppm | 0.1 | 0.1 | | | | |
| **Phosphorous – Mehlich III** | ppm | 75 | 29 | 25 | | 100 | |
| | | | | | | | |
| **Calcium** | % | 68.01 | 54.82 | 68 | 68 | | |
| **Magnesium** | % | 19.30 | 28.27 | 12 | 12 | | Excess causes poor soil structure |
| **Potassium** | % | 3.17 | 1.59 | 2 | | 8 | |
| **Sodium** | % | 1.93 | 11.43 | 0.5 | | 3 | |

* Tony de Vere & Associates

### Nutrients

Table 11.3 shows that after over 20 years irrigation with effluent, the nitrogen and phosphorous concentrations in the soil are well below the maximum sustainable concentrations.

All soils on the irrigation area can sustain an increased nutrient loading as long as the plant and vegetation on the land is regularly removed to ensure the nutrients continue to be taken up by the grass/crops and removed from site. This is ensured by regularly harvesting of crops.

### Sodicity

Table 11.2 generally suggests that the soils at Theodore are sodic in the sub soil layer. High sodicity which is related to the elevated concentrations of sodium in the soils can cause the soils to agglomerate, reduce percolation, and encourage water logging, erosion and increase run off. This is a concern if the irrigation rate is too high.

The soil profile and description in the soil report indicate that the soil layers are generally sandy and loamy, and these soil generally have good drainage properties, therefore although the soils are slightly sodic, it is not expected to generate excessive run off.

Sodicity coupled with elevated conductivity can be beneficial to the soil, the dissolved sodium chloride in the soil solution provide an ion exchange between the sodium and chlorides which reduces the dispersion of clay particles, and therefore the elevated conductivity increases the permeability and reduces run off.

Therefore irrigation of the effluent which has a conductivity of approximately 700 µS/cm can be considered beneficial for soil conditioning.

**Overall Soil Condition**

Overall the soils at Theodore are considered suitable for irrigation with the Theodore sewage treatment plant effluent as long as a suitable and sustainable irrigation application rate is applied. The soil profile suggests a relatively high drainage capacity, therefore nitrate leaching into the groundwater source is of concern and therefore the constraining factor

The suitable application rate of effluent is generated from the MEDLI runs.

## MEDLI Model Outputs

Outputs of the MEDLI model investigation are summarised in Table 10.4, and the nutrient balance over the soils are also determined and shown in Table 10.5. The treated effluent is assumed to be of the Class A+ quality with the concentration of contaminants stipulated in Section 10.1.

**Table 11.4: Irrigation/water balance results of MEDLI model analysis**

| Medli Parameters | Units | Apex | | Edwards | |
|---|---|---|---|---|---|
| **WATER BALANCE** | | | | | |
| Trigger Irrigation at Soil Water Deficit | mm | 20 | | 20 | |
| Land Area | ha | 11.25 | | 10 | |
| Rainfall | mm/yr | 673.6 | | 673.6 | |
| Soil Evaporation | mm/yr | 633.4 | | 779.6 | |
| Transpiration | mm/yr | 668.3 | | 478.7 | |
| Run off with irrigation | mm/yr | 20.5 | | 113.2 | |
| Run off with no irrigation | mm/yr | 40 | | 112.5 | |
| Drainage with irrigation | mm/yr | 73.4 | | 25.8 | |
| Drainage with no irrigation | mm/yr | 33 | | 5 | |
| Change in soil moisture | mm/yr | -1.4 | | -0.2 | |
| ***Monthly application*** | | **mm** | **ML** | **mm** | **ML** |
| January | | 65 | 7.3 | 71 | 7.1 |
| February | | 55 | 6.2 | 58 | 5.8 |
| March | | 65 | 7.3 | 63 | 6.3 |
| April | | 61 | 6.9 | 60 | 6.0 |
| May | | 55 | 6.2 | 54 | 5.4 |
| June | | 52 | 5.9 | 41 | 4.1 |
| July | | 45 | 5.1 | 29 | 2.9 |
| August | | 54 | 6.1 | 36 | 3.6 |
| September | | 70 | 7.9 | 59 | 5.9 |
| October | | 69 | 7.8 | 89 | 8.9 |
| November | | 60 | 6.8 | 82 | 8.2 |
| December | | 68 | 7.7 | 80 | 8.0 |
| Yearly application | | **719** | **80.9** | **722** | **81.2** |
| % days prevent irrigation | % | 18.8 | | 18.8 | |

**Table 11.5: Irrigation Nutrient Balance results of MEDLI analysis**

| Medli Parameters | Units | Apex | Edwards |
|---|---|---|---|
| **NUTRIENT BALANCE** | | | |
| Nitrogen applied by irrigant | kg/ha/yr | 54.2 | 53.3 |
| Nitrogen plant uptake | kg/ha/yr | 62.9 | 60.4 |
| Leached Nitrate | kg/ha/yr | 0.6 | 0.3 |
| Adsorbed Ammonia | kg/ha/yr | 0 | 0 |
| | | | |
| Phosphorous applied by irrigant | kg/ha/yr | 72.1 | 72.4 |
| Phosphorous plant uptake | kg/ha/yr | 32.7 | 33.6 |
| Phosphorous leached | kg/ha/yr | 0 | 0 |
| Phosphorous adsorbed | kg/ha/yr | 39.3 | 38.3 |

The soil profile as well as the initial soil chemistry is important to determine the drainage rate and potential run off. It is important to ensure that the run off and drainage rates are at an acceptable level.

There is limited groundwater data available for the town of Theodore, and the MEDLI program makes the assumption that the groundwater starts at the bottom of the inputted soil profile. This is not always the case, therefore elevated drainage rates are not necessarily a sign of groundwater infiltration.

The extent of nitrogen leaching is contributed to by both the organic component naturally occurring in the soil, as well as in the effluent applied. The optimal procedure for assessing the extent of nitrogen leaching into ground waters or any other surrounding water bodies is to implement a monitoring program which trends the behaviour of nitrogen concentrations.

## 11.7 Summary of MEDLI Output and discussion

The conclusion from the MEDLI runs shows that the effluent can be applied at a sustainable rate to the Apex club land, if the crop of Sorghum is regularly harvested.

The crop will suffer some nitrogen stress, and will require extra nitrogen through fertiliser. This will aid crop growth which in turn will also increase crop phosphorous uptake. Although phosphorous loading is greater than the uptake from the plant, there is no leaching evident from the soil during the 47 year irrigation period. This indicates that this irrigation rate is sustainable for the soil and crop type.

Run off from the Apex land is actually reduced with effective irrigation, and drainage is considered within an acceptable range.

The crop is likely to suffer some stress due to water deficit, and climatic temperature.

A MEDLI run was also performed on the Edwards land to indicate that if it should occur that the Apex crop was no longer harvested, then the effluent is more that capable of being used to irrigate 10 hectares on Edwards farm land.

Table 10.6 shows the sustainable application rate and the available effluent volume for each month. The difference is the deficit or surplus volume of effluent (shown by negative or positive figure).

**Table 11.6: Summary of Sustainable Irrigation Application rate and Available Effluent**

| | **Irrigation Sites (Apex Club)** | | |
|---|---|---|---|
| ***Month*** | ***Sustainable Application rate (ML)*** | ***Available supply (ML)*** | ***Surplus / Deficit (ML)*** |
| January | 7.3 | 6.3 | -1.0 |
| February | 6.2 | 5.7 | -0.5 |
| March | 7.3 | 6.3 | -1.0 |
| April | 6.9 | 6.1 | -0.8 |
| May | 6.2 | 6.3 | 0.1 |
| June | 5.9 | 6.1 | 0.2 |
| July | 5.1 | 6.3 | 1.2 |
| August | 6.1 | 6.3 | 0.2 |
| September | 7.9 | 6.1 | -1.8 |
| October | 7.8 | 6.3 | -1.5 |
| November | 6.8 | 6.1 | -0.7 |
| December | 7.7 | 6.3 | -1.4 |
| **Yearly** | **80.9** | **74.1** | **-6.8** |

**Sustainable allocation of effluent**

The results show that the expected effluent production by the Theodore sewage treatment plant for Year 2025 can be used to irrigate the Apex Club land. This is assumed to be sustainable if the land is regularly harvested, and the irrigation is only triggered at a soil water deficit of 20mm.

**Irrigation rate**

There is not a significant variation in demand for effluent for each month, therefore the application of irrigation to the entire 11.25 hectares of the irrigation site is recommended to be on average 6.8ML/month.

The existing irrigation system of flood irrigation is considered suitable, as long as the area is bunded to ensure run off is contained and is not permitted to run off to any surrounding water bodies.

## 11.8 Wet Weather Storage

During wet weather there is no irrigation of the effluent to land. Therefore the effluent is required to be stored.

MEDLI was modelled to provide the storage capacity required to ensure a 95% reuse of effluent. The wet weather storage volume required is 3.5ML.

The model was based on an uncovered storage tank, with a depth of 4m, therefore there is some loss due to evaporation and transpiration.

## Groundwater

No information is readily available on groundwater in the area. To monitor the impact of the effluent storage pond and treated effluent irrigation on the any groundwater, a series of monitoring piezometers will be installed to enable samples of groundwater to be taken for analysis.

## 11.10 Recommendations for Effluent Re-use

It is suggested that for the Theodore effluent reuse scheme to be long term environmentally sustainable, as well as ensure that the effluent is utilised to full fill its maximum beneficial potential, the following is recommended;

- The expected effluent production for the Year 2025, a 3.5ML wet weather storage volume is required for 95% effluent re-use;
- The model was based on the existing system of irrigation at the Apex land, flow irrigation and will be triggered at a soil water deficit of 20mm, this will be monitored using a soil moisture probe.
- The monthly application of effluent to the irrigation site should not exceed 6.8ML/month;
- The existing irrigation scheme is to be assessed to ensure that the effluent is not permitted to run off into surrounding water bodies;
- It is recommended that a third party agreement be formed between Council and the Apex Club to ensure the allocation of effluent is agreed upon and conditions for receiving the Recycled Water are clearly stipulated. These conditions include regular harvesting of the crop and the irrigation system and land is maintained to an acceptable standard. This will include the implementation of a Recycled Water Management Plan for both supplier and user.

# 12.0 ENVIRONMENTAL MANAGEMENT

## 12.1 Recycled Water Management Plan

Recycled Water Management Plan should be prepared for the Apex Club and any other user, including Council.

The Management Plan will include as a minimum:

- A formal Agreement between Council and the User specifying the terms and conditions of use;
- Advice on the safe use of recycled water;
- Site specific treated effluent application rates
- Monitoring requirements.

## 12.2 Environmental Management

A comprehensive Environmental Management Plan should be prepared for the sewage treatment and effluent disposal at Theodore. The Plan should cover:

**The Sewage Treatment Process**

- The Theodore sewage treatment process including;
  - Recording of raw sewage flow;
  - Analysis of the raw sewage;
  - Analysis of treated effluent ;
  - Analysis of the effluent from the treated water storage tank to irrigation.
- Odour from the treatment plant

**Treated Effluent Disposal by Irrigation**

- Analysis of the soils on all irrigation areas
- Monitoring of the volumes applied

**Groundwater**

- Establish piezometers;
- Measurement of groundwater levels;
- Sampling of groundwater.

# 13.0 CONCLUSIONS

From the above it has been concluded that:

- The existing treatment plant at Theodore is reaching the limit of its process capacity. In addition, the structure of the Imhoff tank is in poor condition with major cracks in the concrete works;
- A new treatment plant is required that will produce effluent of Class B recycled water quality suitable for disposal by irrigation. Additional treatment will be required for the proposed truck wash bay and standpipe;
- The existing practice of irrigating to a crop is environmentally sustainable;
- An Environmental Management Plan should be developed and put into place to monitor the treatment plant and irrigation disposal.

# 14.0 RECOMMENDATIONS

It is recommended that:

- A new treatment plant is built at Theodore;
- The process for the new treatment plant should be a "Package STP" system based on the activated sludge process (Option 7), and should include inlet screenings;
- The current practice of using the treated effluent to irrigate a crop is continued; and treated effluent will be used for public open space irrigation without controlled access;
- New treated effluent be used for wash truck bay and to industrial uses through standpipes
- An Environmental Management Plan is prepared for Theodore.

# References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Environmental Guidelines for the Use and Disposal of Biosolids Products, NSW EPA, 1997

# Appendix A

# Theodore Sewage Treatment Plant Flow and Analytical Data

| | |
|---|---|
| **Project No** | 7612/01 |
| **Project name:** | Banana Shire Council - STP Review and Augmentation |
| **Particular:** | Theodore Analytical Results |
| **By:** | Dominique Keirens |

| **Theodore Raw Sewage** | | 8/03/2005 | 19/04/2005 | 4/05/2005 | 14/06/2005 | |
|---|---|---|---|---|---|---|
| **Contaminants** | **Units** | **Raw** | **Raw** | **Raw** | **Raw** | **AVERAGE (Raw)** |
| **Ammonia as N** | mg/L | | 53 | 49 | 32 | *44.7* |
| **Nitrate as NO3** | mg/L | | 0.5 | 0.5 | 0.5 | *0.5* |
| **Nitrate as N** | mg/L | | 0.11 | 0.11 | 0.11 | *0.1* |
| **BOD** | mg/L | 137 | 513 | 136 | 158 | *236.0* |
| **COD** | mg/L | 240 | 560 | 330 | 390 | *380.0* |
| **TN as N** | mg/L | 48 | 72 | 65 | 50 | *58.8* |
| **Calculated Organic N** | mg/L | | 18.89 | 15.89 | 17.89 | *17.6* |
| **Conductivity** | uS/cm | 900 | 1100 | 920 | 900.00 | *955.0* |
| **Elements (Phosphorous)** | mg/L | 11 | 12 | 8.8 | 10 | *10.5* |
| **Mercury** | mg/L | | 0.001 | 0.001 | | |
| **Arsenic** | mg/L | | 0.005 | 0.005 | | |
| **Cadmium** | mg/L | | 0.005 | 0.005 | | |
| **Chromium** | mg/L | | 0.005 | 0.005 | | |
| **Copper** | mg/L | | 0.012 | 0.006 | | |
| **Lead** | mg/L | | 0.005 | 0.005 | | |
| **Nickel** | mg/L | | 0.005 | 0.005 | | |
| **Zinc** | mg/L | | 0.05 | 0.019 | | |
| **pH** | - | 7.3 | 7.7 | 7.4 | | *7.5* |
| **Total Dissolved Solids (TDS)** | mg/L | 540 | 480 | 490 | | *503.3* |
| **Total Suspended Solids (TSS)** | mg/L | 160 | 270 | 200 | | *210.0* |

| **Theodore PST Effluent** | | 8/03/2005 | 19/04/2005 | 4/05/2005 | 14/06/2005 | |
|---|---|---|---|---|---|---|
| **Contaminants** | **Units** | **PST** | **PST** | **PST** | **PST** | ***AVERAGE (PST)*** |
| **Ammonia as N** | mg/L | | 16 | 19 | 22 | *19.0* |
| **Nitrate as NO3** | mg/L | | 22 | 15 | 17 | *18.0* |
| **Nitrate as N** | mg/L | | 4.9 | 3.3 | 3.8 | *4.0* |
| **BOD** | mg/L | 40 | 12 | 13 | 20 | *21.3* |
| **COD** | mg/L | 180 | 130 | 52 | 78 | *110.0* |
| **TN as N** | mg/L | 12 | 30 | 32 | 35 | *27.3* |
| **Calculated Organic N** | mg/L | | 9.1 | 9.7 | 9.2 | *9.3* |
| **Conductivity** | uS/cm | 620 | | | | *620.0* |
| **Elements (Phosphorous)** | mg/L | 4 | 10 | 11 | 11 | *9.0* |
| **Mercury** | mg/L | | | | | |
| **Arsenic** | mg/L | | | | | |
| **Cadmium** | mg/L | | | | | |
| **Chromium** | mg/L | | | | | |
| **Copper** | mg/L | | | | | |
| **Lead** | mg/L | | | | | |
| **Nickel** | mg/L | | | | | |
| **Zinc** | mg/L | | | | | |
| **pH** | - | 7.1 | 7 | 7.3 | 7.6 | *7.3* |
| **Total Dissolved Solids (TDS)** | mg/L | 372 | | | | *372.0* |
| **Total Suspended Solids (TSS)** | mg/L | 140 | 42 | 12 | 36 | *57.5* |

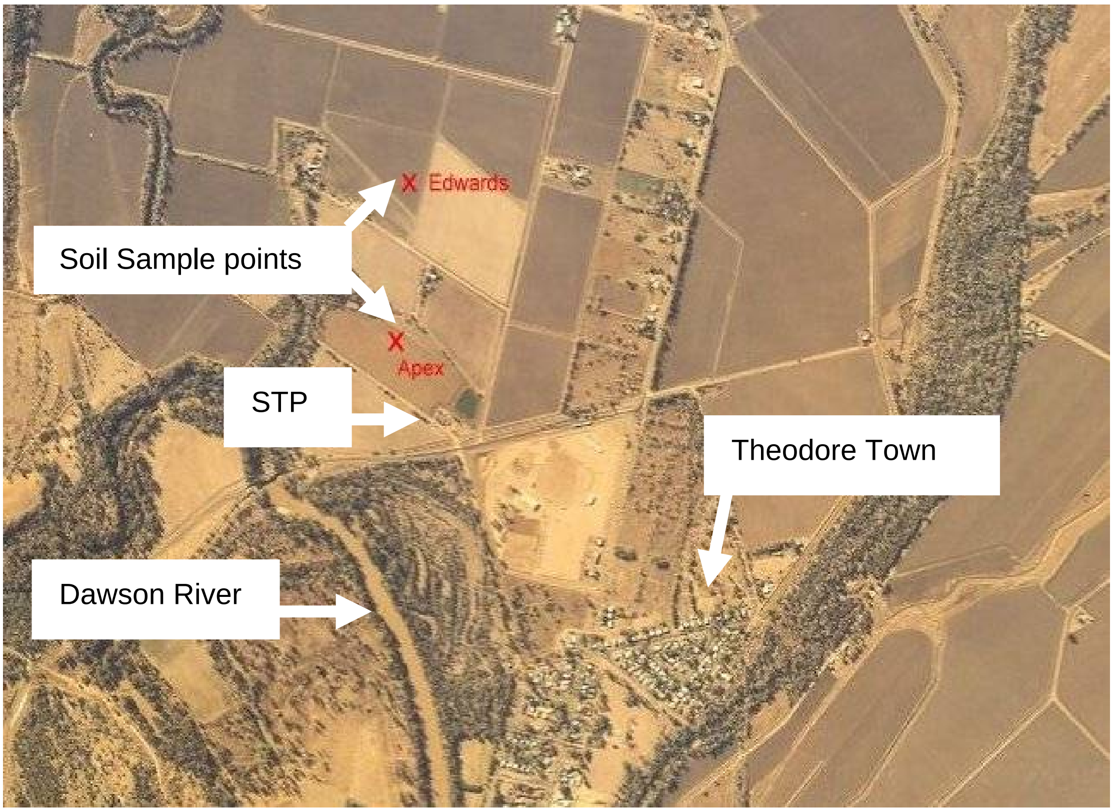
| | |
|---|---|
| **Project No** | 7612/01 |
| **Project name:** | Banana Shire Council - STP Review and Augmentation |
| **Particular:** | Theodore Analytical Results |
| **By:** | Dominique Keirens |

| **Theodore Final Ponds Effluent** | | 8/03/2005 | 19/04/2005 | 4/05/2005 | 14/06/2005 | |
|---|---|---|---|---|---|---|
| **Contaminants** | **Units** | **Final Ponds** | **Final Ponds** | **Final Ponds** | **Final Ponds** | ***AVERAGE (Final Ponds)*** |
| **Ammonia as N** | mg/L | | 2.4 | 3.1 | 9.7 | *5.1* |
| **Nitrate** | mg/L | | 0.8 | 2.8 | 1.6 | *1.7* |
| **Nitrate as N** | mg/L | | 0.18 | 0.62 | 0.36 | *0.4* |
| **BOD** | mg/L | 13 | 7 | 14 | 14 | *12.0* |
| **COD** | mg/L | 49 | 87 | 52 | 38 | *56.5* |
| **TN as N** | mg/L | 18 | 91 | 9.2 | 15 | *33.3* |
| **Calculates organic N** | mg/L | | 88.42 | 5.48 | 4.94 | *32.9* |
| **Conductivity** | uS/cm | 640 | 710 | 680 | 770 | *700.0* |
| **Elements (Phosphorous)** | mg/L | 6.9 | 5.3 | 6.2 | 7.4 | *6.5* |
| **Mercury** | mg/L | | | | | |
| **Arsenic** | mg/L | | | | | |
| **Cadmium** | mg/L | | | | | |
| **Chromium** | mg/L | | | | | |
| **Copper** | mg/L | | | | | |
| **Lead** | mg/L | | | | | |
| **Nickel** | mg/L | | | | | |
| **Zinc** | mg/L | | | | | |
| **pH** | - | 7.1 | 7.8 | 7.2 | 7.7 | *7.5* |
| **Total Dissolved Solids (TDS)** | mg/L | 384 | 420 | 480 | 540 | *456.0* |
| **Total Suspended Solids (TSS)** | mg/L | 6 | 32 | 60 | 34 | *33.0* |

# Appendix B

# Locations of irrigation Land Sites

Irrigation Areas are located around the point where soil samples where taken for analysis by Tony De Vere.



# Appendix C

# Soil Report – By Tony De Vere

# Appendix D

# MEDLI Output Summary