Appendix

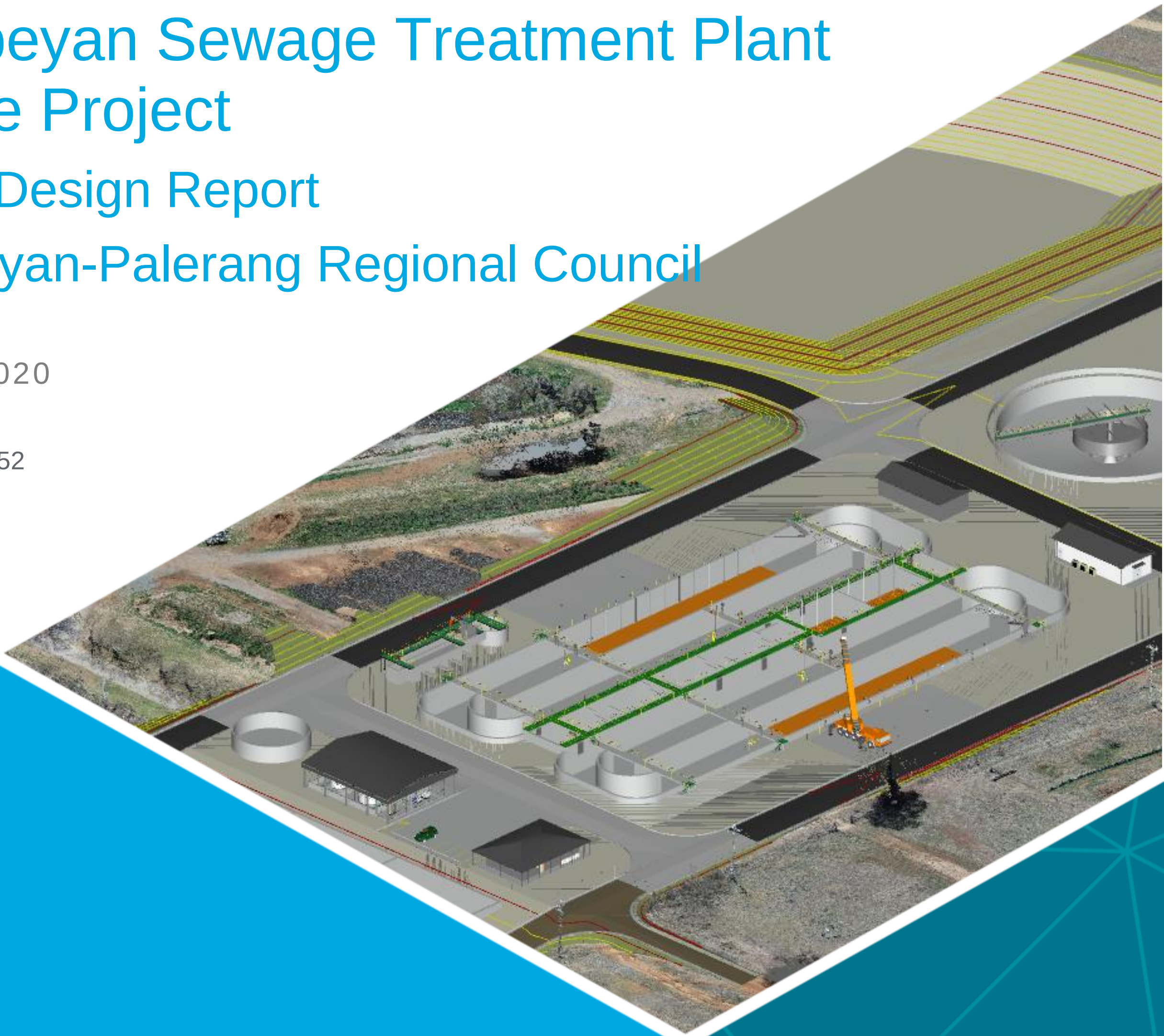
G1 Concept Design Report

hunterh2o

Queanbeyan Sewage Treatment Plant Upgrade Project

Concept Design Report

Queanbeyan-Palerang Regional Council



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A	Draft	Craig White Jeremy Smith Mark Lynn David Perry	David Perry	Jeremy Smith	15 April 2020
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C	Update for Draft EIS	Craig White Jeremy Smith Mark Lynn David Perry	David Perry	Jeremy Smith	27 October 2020

Executive Summary

Queanbeyan-Palerang Regional Council (QPRC) is undertaking an upgrade of the Queanbeyan Sewage Treatment Plant (STP) which treats sewage from Queanbeyan prior to discharge to the Molonglo River. The Queanbeyan STP upgrade will replace the existing sewage treatment plant which is approaching the end of its asset life with a modern treatment facility that will improve treatment reliability. The upgrade provides additional capacity for growth and development in Queanbeyan and will enable QPRC to continue to meet its regulatory requirements. Compared with the existing STP, the upgrade will provide a higher level of treatment at every treatment step. Hunter H2O has been engaged as the design consultant to prepare the design and tender documentation for the upgrade.

This report describes the concept design of the Queanbeyan STP upgrade. This concept will be used to inform the preparation of a Business Case and Environmental Impact Statement for the project. The concept design will also form the basis for further design development prior to inviting tenders for the construction of the works.

The project aims to have the Queanbeyan STP upgrade operational by 2024.

Upgrade Definition

The concept design for the Queanbeyan STP upgrade has been developed based on the design criteria and assumptions report for the upgrade (December 2019) and the option study that selected the preferred treatment process for the upgrade (November 2019).

The Queanbeyan STP Upgrade will provide treatment capacity for an equivalent population (EP) of 75,000. The upgrade is designed to meet the effluent quality objectives of the existing Environmental Authorisation licence. The licence requirements however present some technical challenges in terms of ammonia and TDS concentrations. The sampling requirements will also need to be updated to provide appropriate monitoring of the new STP. A modification to the licence is therefore required.

Recycled water produced by the plant will be used onsite for treatment processes and site hose reels for washdown. A standpipe will also be provided to supply recycled water to tankers for offsite uses managed by QPRC such as dust suppression.

Biosolids produced by the treatment process will be stabilised using aerobic digestion to achieve Stabilisation Grade B. This treatment enables the plant biosolids to be disposed of to landfill or reused for agriculture.

Concept Overview

Process Configuration

The Queanbeyan STP Upgrade proposes the construction of a new treatment plant complete with screening and grit removal, a continuous oxidation ditch activated sludge process with gravity clarifiers, tertiary filtration and UV disinfection. Waste sludge produced by the treatment process is stabilised in an aerobic digester and dewatered; producing a biosolids product that is suitable for reuse. Treated effluent is discharged via an on-bank discharge structure adjacent to the Molonglo River. Infrequent overflows from the Storm Pond discharge at the same location.

An overview of the process showing major treatment processes is shown below.

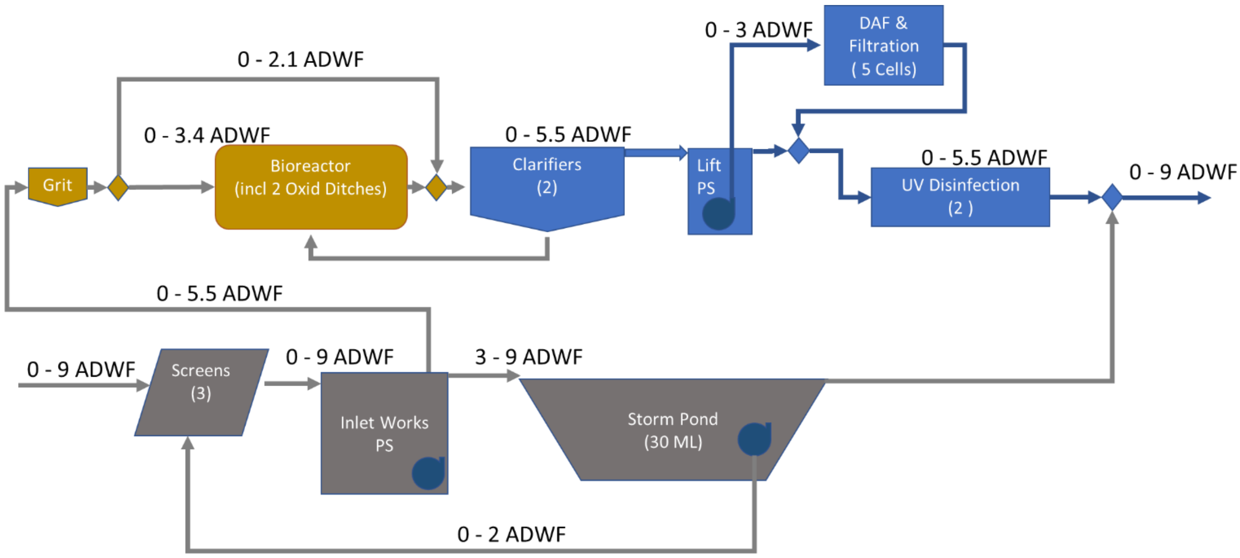


Figure A – Process Configuration Overview

The design has been developed to provide a simple and robust process that provides reliable removal of nitrogen and phosphorus and treatment of storm flows.

A two-stage phosphorus removal process has been used to provide a low phosphorus concentrations for discharge. The bioreactor and clarifier system have been configured with both biological and chemical phosphorus removal to remove the bulk of the phosphorus. Dissolved Air Flotation (DAF) followed by dual media filters (i.e. DAFF) after the clarifiers provide tertiary filtration to polish the phosphorus to the low levels required. This tertiary treatment produces a high quality of treated effluent that will support future reuse opportunities. Initially recycled water from the upgraded plant will used for onsite as well as being provided to water tankers for offsite uses such as dust suppression.

Queanbeyan STP receives wet weather flows of up to 9 times Average Dry Weather Flow (ADWF) from the catchment. The proposed upgrade provides screening of all flows received using 2-dimensional 5 mm aperture band screens. The bioreactor and clarifiers are configured to provide enhanced storm flow treatment using a solids contact process for flows of up to 5.5 ADWF. The upgrade design includes a 30 ML Storm Pond that is used to store wet weather flows greater than 5.5 ADWF during significant wet weather events and return them to rescreening and treatment as inlet flows subside. Controlled overflows from the Storm Pond are rare and are estimated at approximately a 1 in 10 year event. In these instances, the overflow is likely to be discharged into a high river flow environment.

A summary of the upgrade process units is provided in the table below.

Table A – Overview of the Unit Operations of the Design

Category	Design equipment
Inlet works screening	Band screen – 5 mm 2 dimensional
Inlet Pump station	Lifts flow to treatment post screening
Storm Pond	30 ML storm pond downstream of the inlet screens
Inlet works grit removal	Vortex grit system
Bioreactor	Biological phosphorus removal configured reactor. Anaerobic zones followed by two oxidation ditches and two final aerobic zones
Clarification	Two 40 m clarifiers to treat bioreactor and storm flows treated using the solids contact process
Filter lift pump station	Lifts flow to filtration
Tertiary DAF and filtration	DAF and granular media filtration dual coal / sand media
Disinfection for river discharge	Ultraviolet (UV)
Disinfection for Recycled Water	Filtration +UV + chlorination
Biosolids stabilisation	Aerobic digestion
Biosolids dewatering and handling	Two centrifuges that are to be out-loaded directly into truck for transport off-site

Site Arrangement and Civil Works

The new STP will be constructed on the existing lease area to the south east of the existing treatment process. The site location provides a predominantly level area where the new treatment process may be constructed while maintaining operation of the existing STP.

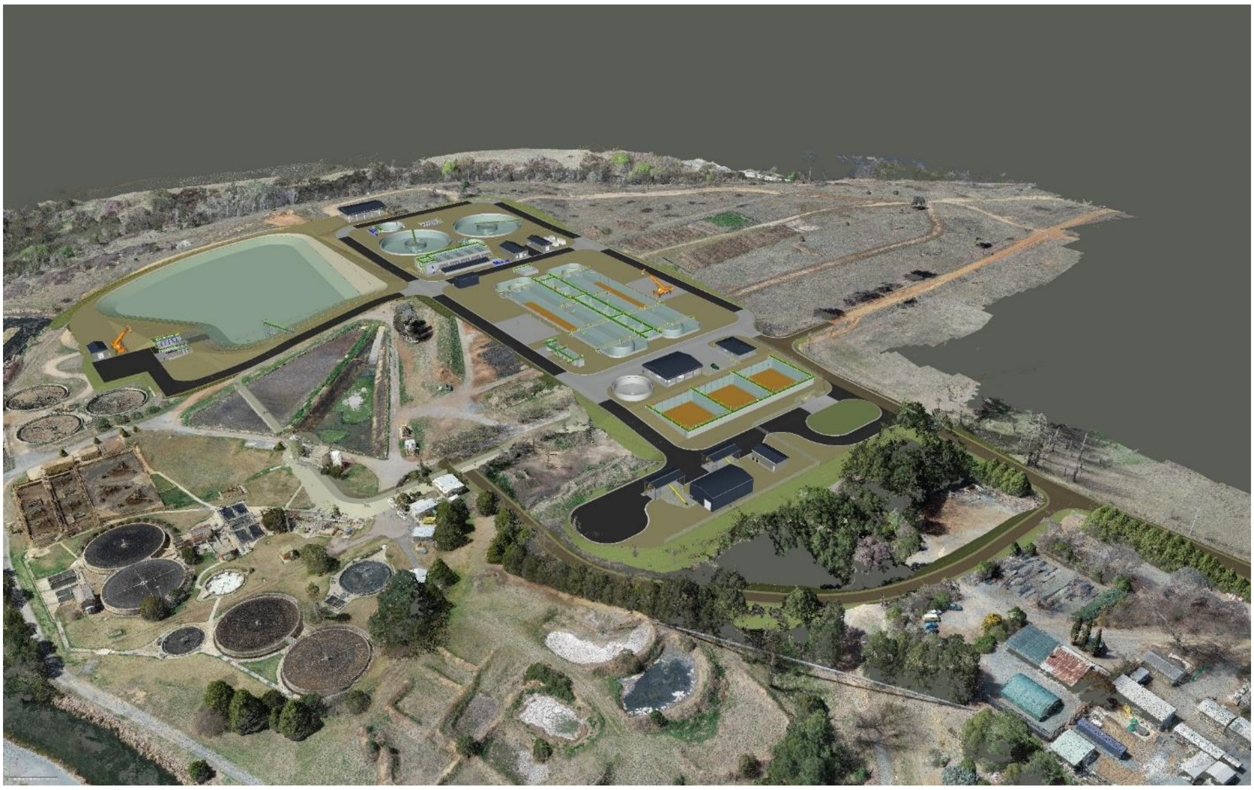


Figure B – Overview of the Queanbeyan STP Upgrade (existing plant in foreground)

The Queanbeyan STP Upgrade is primarily situated above the nominated design flood level reducing the risk of damage to major structures, mechanical and electrical equipment during flood events (refer to Figure C).

The site layout has been developed in consultation with QPRC and informed by site investigations including survey, services location, geotechnical, ecology, contamination and heritage investigations completed to date. Key considerations in development of the layout include:

- Locating the hydraulic grade line and height of structures to ensure bioreactor, clarifiers and UV are positioned at ground level (i.e. top of structure is at handrail height generally) to simplify operation and reduce costs associated with access to elevated structures and lift pumping
- Minimising hydraulic losses of major pipe runs through the treatment process
- Providing adequate space between structures for operation and maintenance access as well as for installation of below ground pipework and electrical conduits
- Site operation, monitoring and security requirements
- Construction sequencing requirements.

Development of the upgrade layout has considered the potential future requirements of the site. The site arrangement leaves space for a future stage 2 upgrade to expand the treatment capacity by 50% to 112,500 EP. Consideration to the needs of the future upgrade has been given as part of the hydraulic design, site layout and the process sizing of some aspects of the treatment facility which would be difficult to upgrade at a later date, such as the inlet works and grit removal facilities.

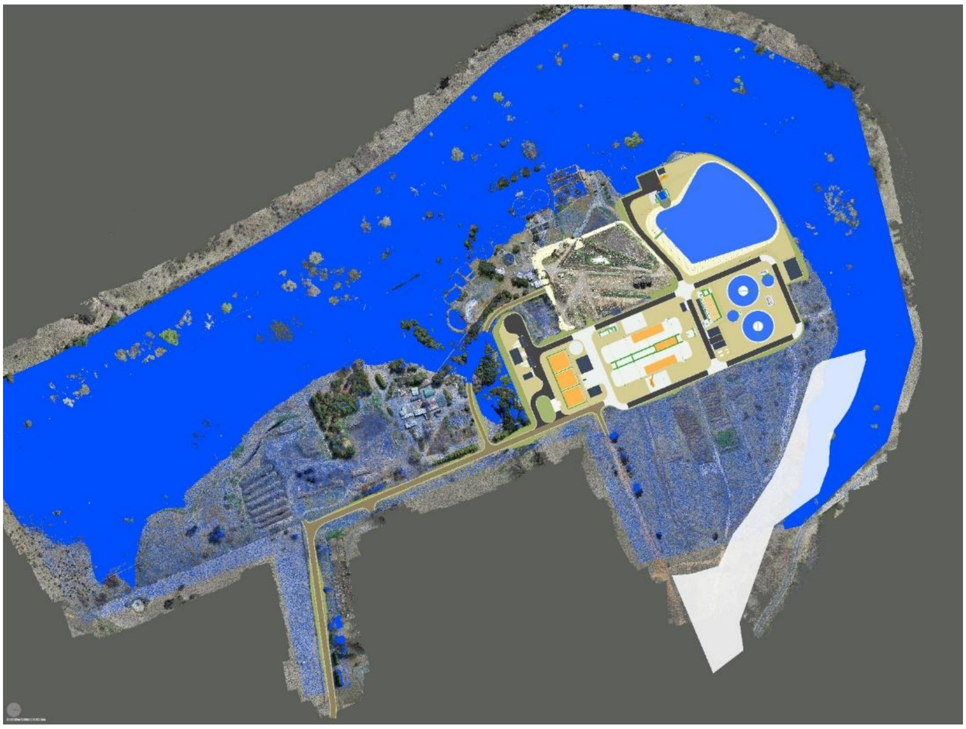


Figure C – Queanbeyan STP Upgrade Site Layout Relative to the Nominated Design Flood Level of 1% AEP (design has additionally considered an allowance for climate change)

Additional civil works included in the Queanbeyan STP Upgrade includes:

- Diversion of the incoming Morisset and Jerrabomberra Trunk Sewers from cut-in locations within the STP lease boundary to the new Inlet Works
- Construction of a new effluent discharge point to a new on-bank discharge structure downstream of the UV facility
- Upgrade of the existing Mountain Road access
- Decommissioning and modification of the three existing Maturation Ponds that are located in the flood zone and are no longer required for treatment.

Electrical and Control

The power system architecture for the upgrade uses two pad mounted transformers with separate distribution across the site. Backup onsite diesel generators are provided to automatically start and transfer power on loss of transformer supply.

The power supply authority for the site (Evoenergy) has advised that there is currently insufficient power supply capacity for the new plant. Evoenergy have a planned upgrade to increase supply capacity in the area that they expect can be made available in time for commissioning of the upgraded Queanbeyan STP.

Cost Estimate

A risk-based engineering cost estimate for the project has been prepared in conjunction with this Concept Design.

At the time of this concept design, the construction value of the works is estimated to be in the order of $100m. This is in line with expectations for a plant of this capacity and QRPC's previous planning for this project. This cost estimate is subject to change over time as the project scope is further defined through the design process. A revised cost estimate will be provided during the reference design phase.

Procurement Program

Key milestones for the delivery of the Queanbeyan STP upgrade are shown in Table B. These dates are for planning purposes and will be refined as the project progresses.

Table B – Key Delivery Milestones for Queanbeyan STP Upgrade Project

Milestone	Current estimate
Concept design approval	May 2020
Draft EIS	November 2020
Final EIS	July 2021
Detailed design and tender documentation	November 2021
Tendering and contract award	November 2021 – June 2022
Construction and commissioning. Decommissioning of maturation ponds	July 2022 – December 2024
Completion of HV power supply upgrade by Evoenergy	June 2023

Contents

Figures

Tables

Appendices

A full list of design report appendices is provided below. Appendix A, L and M are provided for the concept report accompanying the EIS.

Appendix A	Site Arrangement
Appendix B	Flow Balance and Hydraulic Profile
Appendix C	Process Flow and Process and Instrumentation Diagrams
Appendix D	Navisworks Model Screenshots
Appendix E	Electrical Drawings
Appendix F	Safety in Design Summary
Appendix G	Maximum Power Demand Calculation
Appendix H	Cost Estimate
Appendix I	Geotechnical Investigation – Factual Report
Appendix J	Geotechnical Investigation – Interpretive Report
Appendix K	UVT Jar Testing Report
Appendix L	ACT EPA Feedback on Concept Design
Appendix M	NSW DPIE Feedback and Reponse

Glossary of Terms

AC	Activated Carbon
ACT	Australian Capital Territory
ADWF	Average Dry Weather Flow
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ASCE	American Society of Civil Engineers
ATF	Air Treatment Facility
BFR	Best for Region
BOD	Biochemical Oxygen demand (5 day) not inhibited, sometimes expressed as BOD_5
BOM	Bureau of Meteorology
BTF	Biological Trickling Filter
BWL	Bottom Water Level
COD	Chemical Oxygen Demand
EPA	Environment Protection Authority (ACT / NSW)
DAF	Dissolved Air Flotation
DAFF	Dissolved Air Flotation Filter (or Filtration)
DICL	Ductile Iron Cement Lined
DO	Dissolved Oxygen
DOL	Direct Online
DOP	Dissolved Organic Phosphorus
DWAS	Digested Waste Activate Sludge
EBPR	Enhanced Biological Phosphorus Removal
EC	Electrical Conductivity (an indirect measure of water salt level)
EP	Equivalent Person
ET	Equivalent Tenement
F_{bs}	Fraction of COD which is biodegradable and soluble
F_{na}	Fraction of TKN which is ammonia
F_{nus}	Fraction of TKN which is unbiodegradable and soluble
F_{up}	Fraction of COD which is unbiodegradable and particulate
F_{us}	Fraction of COD which is unbiodegradable and soluble
FAD	Free Air Delivery
FC	Faecal Coliforms
FRP	Free Reactive Phosphorus
FS	Functional Specification
FE	Flow Element
FIT	Flow Indicator Transmitter

HDPE	High Density Polyethylene
HGL	Hydraulic Grade Line
HV	High Voltage
IBC	Intermediate Bulk Container
ICON	Icon Water Limited – manages water and sewage in the ACT
LCS	Local Control Station
IDE	Influent Design Envelope
LV	Low Voltage
MCC	Motor Control Centre
MLS	Mixed Liquor Splitter
MSB	Main Switchboard
MLSS	Mixed Liquor Suspended Solids (or TSS, TSS in the bioreactor)
Nm^3	Normal m^3 expressed at 20°C and 1 atm. European standard is 0°C.
MSCL	Mild Steel Cement Lined
NSW	New South Wales
NTU	Nephelometric Turbidity Units
OCU	Odour Control Unit
OTR	Oxygen Transfer Rate, or actual oxygen transfer rate
P&ID	Piping & Instrumentation Diagram
PDWF	Peak Dry Weather Flow (hourly average peak flow in dry weather)
PE	Pressure Element
PFD	Process Flow Diagram
PFTF	Peak Full Treatment Flow
PIF	Peak Instantaneous Flow
PIT	Pressure Indicator Transmitter
PLC	Programmable Logic Controller
PS	Pumping Station
PWWF	Peak Wet Weather Flow (daily total 24 hour integrated)
QPRC	Queanbeyan-Palerang Regional Council
RW	Recycled Water
RIO	Remote Inputs & Outputs
rDOP	refractory Dissolved Organic Phosphorus.
sAHP	Soluble Acid Hydrolysable phosphorus
SCADA	Supervisory Control and Data Acquisition
SOTE	Standard Oxygen Transfer Efficiency. Clean Water zero DO oxygen transfer efficiency typically expressed as %, %/m, gO_2/Nm^3 or $gO_2/Nm^3/m$
SOTR	Standard Oxygen Transfer Rate, oxygen transfer into clean potable water at zero DO and 20°C as per ASCE standard
SRT	Solids Retention Time in days

STP	Sewage Treatment Plant
TDS	Total Dissolved Solids
TKN	Total Kjeldahl Nitrogen (organic + ammonia nitrogen)
TN	Total Nitrogen (all nitrogen forms expressed as N, organic, ammonia and oxidised)
TOG	Total Oil & Grease
TOR	Terms of Reference. Document RFT Volume 3 Design Consulting Services Terms of Reference
TP	Total Phosphorus (all phosphorus forms expressed as P, organic and phosphate)
TSS	Total Suspended Solids
TWAS	Thickened WAS
TWL	Top Water Level
VFD	Variable Frequency Drive
VSD	Variable Speed Drive
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge

1 Introduction

1.1 Background

Queanbeyan-Palerang Regional Council (QPRC) is undertaking an upgrade of the Queanbeyan Sewage Treatment Plant (STP) which treats sewage from Queanbeyan prior to discharge into the Molonglo River. The upgrade will improve treatment reliability by replacing the existing treatment plant, which is approaching the end of its asset life, with a modern treatment facility. The new Queanbeyan STP will also provide additional capacity to support population growth and development within the Queanbeyan area. Hunter H2O has been engaged as the design consultant to prepare the design and tender documentation for the upgrade.

The new Queanbeyan STP will be constructed at Council's existing STP site, which is located off Mountain Road, Jerrabomberra ACT.

1.2 Project Objective

The objective of the project is to deliver an STP that protects public health and the environment for future generations. In developing the project, the upgrade seeks to deliver:

- A robust and "Best for Region" solution that provides for both immediate service needs and plans for identified needs of the future and
- An upgrade solution that:
  - Represents value for money
  - Achieves targeted sustainability and public health outcomes and
  - Meets regulatory requirements.

The project is referred to as the Queanbeyan STP Upgrade and will provide treatment capacity for an equivalent population (EP) of 75,000. The upgrade aims to be operational by 2024.

1.3 Purpose of this Report

This concept design report and accompanying drawings describe the proposed scope of work for the Queanbeyan STP Upgrade project. The report has been prepared to assist QPRC and project stakeholders in reviewing the proposed upgrade prior to further design stages and the preparation of an Environmental Impact Statement for the project.

Appendices L and M contain feedback from the ACT EPA and NSW DPIE on a draft concept design that was prepared in April 2020. The concept design has been refined since this time with the following:

- The proposed tertiary filters have been upgraded to dissolved air flotation filters (DAFF) to provide a higher quality of treated effluent with lower phosphorus concentrations
- The plant hydraulics, flow splitter structures and capacity of the inlet works screening and grit removal has been revised to better provide for a staged increase to the plant capacity to 112,500 EP in the future
- The proposed capacity of the aerobic digestor has been increased to a 30-day SRT to provide improved stability of the biosolids produced
- Auxiliary plant such as the septage receival and number of pump stations have been simplified as part of value management.

The design continues to be refined and will be further developed prior to construction with final structure sizing and layout being developed to take into account equipment selection, safety, operational and maintenance requirements.

2 Upgrade Definition

2.1 Overview

Development of the design of the Queanbeyan STP upgrade has followed a staged approach. The design presented here builds on investigations and consultation with project stakeholders carried out to date.

Assessments and studies used to develop the concept design have included:

- A review of flow and Equivalent Population (EP) projections
- A sewage and effluent quality assessment including a review of available data and a further raw sewage and effluent quality monitoring program
- A review of biosolids quality
- A review of durability, criticality and redundancy requirements
- A site selection study that reviewed potential areas for construction of the new STP
- An option study that selected the preferred treatment process for the upgrade using a multi-criteria assessment of various treatment options based on cost and non-cost factors
- A site arrangement workshop that reviewed the hydraulic grade and proposed location of the plant to minimise pumping
- Site investigations including survey, location of services, geotechnical investigation, preliminary contamination assessment, heritage and ecological assessment
- Site flood levels determined by the *Queanbeyan Flood Plain Risk Management Study and Plan* (Queanbeyan-Palerang Regional Council, March 2019)
- Consideration of adaptation opportunities to mitigate against risks to construction and operation as a result of climate change
- A Safety in Design review to eliminate or minimise health and safety risks associated with construction and operation of the STP.
- Various workshops covering both risk and sustainability.

The *Queanbeyan Sewage Treatment Plant Upgrade Project: Option Selection Report* (Hunter H2O, November 2019) documents the process that was used to select the treatment process configuration for the upgrade. The treatment process selected for the upgrade is a conventional continuous oxidation ditch activated sludge process with gravity clarifiers, tertiary treatment with dissolved air flotation filtration, ultraviolet (UV) disinfection, aerobic digestion and centrifuge dewatering.

The clarifiers have been configured in a solids contact mode to enable enhanced storm water treatment. The design also includes a storm pond to contain storm flows which exceed the flow capacity of the treatment process in significant wet weather events and return them for screening and treatment.

2.2 References

The above items are documented in the following reports as well as those listed in the References section at the end of this report:

- *Queanbeyan Sewage Treatment Plant Upgrade Project: Design Criteria and Assumptions Report, Rev C* (Hunter H2O, December 2019)
- *Queanbeyan Sewage Treatment Plant Upgrade Project: Option Selection Report, Rev B* (Hunter H2O, November 2019)

- Minutes of the *QSTP Site Arrangement Workshop* (Hunter H2O, December 2019).
- Minutes of various workshops to date

These reports should be consulted for a detailed understanding of the design basis. Key design criteria for the upgrade are summarised in the next section.

3 Design Envelope

The design envelope for the upgrade is presented in the *Queanbeyan Sewage Treatment Plant Upgrade Project: Design Criteria and Assumptions Report* (Hunter H2O, December 2019). A summary of the influent design envelope and treatment objectives for treated effluent, recycled water and treated biosolids is presented below.

3.1 Influent Design Envelope

3.1.1 Design Population

This Queanbeyan STP Upgrade provides treatment capacity for 75,000 EP.

A detailed review of existing sewage loads and growth projections identified that a capacity of 75,000 EP was required for the upgrade to ensure that a further expansion of treatment capacity was not required for at least 15 years after commissioning.

Development of the upgrade has also considered the potential future requirements of the site. The site arrangement leaves space for a future stage 2 upgrade to expand the treatment capacity by 50% to 112,500 EP. Consideration for this potential future upgrade has been given as part of the hydraulic design, site layout and the process sizing of some aspects of the treatment facility which would be difficult to upgrade at a later date, such as the inlet works and grit removal facilities. The space requirement considerations are based on the future expansion using the same process technology as the 75,000 EP upgrade.

Initial planning for the site included consideration of space requirements for an ultimate capacity of 150,000 EP. This figure has been revised following the decision from ICON Water not to transfer sewage to QSTP.

3.1.2 Design Flow

The flow design basis is summarised in Table 3-1.

Table 3-1 Flow Design Basis at 75,000 EP

Parameter	Value	Units	Notes
EP	75,000	EP	
ADWF Flow loading	230	L/EP/day	200 L/EP day measured. Additional 15% allowance if flow loading allows for increase in inflow and infiltration as sewer degrades over time.
Design ADWF	17.25	ML/d	
PDWF	1.35	ML/hr	Based on measured 95 %ile of 1.88 x average
PFTF	599	L/s	Minimum flow equivalent to 3 ADWF on an instantaneous basis.
PWWF	155.25	ML/d	Note one event of 9 ADWF was recorded
PIF	1,797	L/s	Set equal to PWWF

3.1.3 Raw Sewage Quality

Table 3-2 displays the sewage quality design summary.

The design is mostly governed by the load of pollutants accepted by the plant, rather than pollutant concentrations. For this reason, the influent has been specified in terms of load. For convenience the concentration of pollutants at the current ADWF loading (200 L/EP/day) has also been presented. Concentrations referenced except for unbiodegradable soluble nitrogen and phosphorus are for information only and will vary as flow varies over time.

Table 3-2 Sewage Quality Design Summary at 75,000 EP

Parameter				Notes
	Median Load kg/day (mg/L)*	90%ile Load kg/day	Maximum Load kg/day	*Equivalent concentration at 200 L/EP/d which is current observed value. At the design flow loading of 230 L/EP concentrations will be approximately 9% lower.
COD	9,000 (600)*	13,680	20,700	Based on 120 g/EP/d. Higher load reflects adjustments to account for settling of sewage.
BOD5	4,090 (273)*	N/A	N/A	Ratio of COD/BOD from extensive grab sampling database used.
ISS	570 (36)*	N/A	N/A	Higher value used in HWA data set. Not measured in more extensive grab sampling database.
TKN	938 (63)*	1,173	1,332	Based on 12.5 g/EP/d.
NH_3-N	750 (50)*	891	939	Measured ratios of ammonia to TKN adopted from grab sampling database
TP	165 (11)*	228	297	2.2 g/EP/d adopted which was observed in HWA and MW data. A much lower value of ~ 1.6 g/EP/d was observed in grab sampling data.
OP	116 (7.7)*	161	209	OP/TP ratio is ~ 0.7 MW/HWA data versus 0.6 for grab data. The lower ratio appears too low for domestic sewage. Higher adopted.
Alkalinity	3,855 (257)*			Limited data available. Lower value of HWA data set adopted to ensure dosing is not undersized.
TDS	6,900 (460)*	7,865	8,625	Excluding sewage organics, NH_3-N and PO_4
10%ile COD/TKN	7.8 COD/TKN			
	Median			
pH	7.2			Typical domestic sewage value.
	Key Sewage Fractions/ Concentrations			
F_{bs}			0.18 - 50%ile	Design to allow for variance + or – 40%
F_{up}			0.2 - 50%ile	Design to all for variance + or – 20%
Unbiodegradable Soluble N			1.3 mg/L 50 %ile 2.7 mg/L 90 %ile	From extensive assessment of clarifier effluent quality.
Unbiodegradable Soluble P			0.01 mg/L 50 %ile 0.02 mg/L 90 %ile	Based on clarifier jar testing program.

3.1.4 Septage Waste

There are a significant number of domestic septic tanks and aerated wastewater treatment systems within the QPRC local government area. Currently most of this septic and sewage tank waste is transported for treatment at facilities operated by ICON Water in the ACT. As part of the STP upgrade, QPRC wish to include a Septage Receival facility to accept and treat septic tank and aerated wastewater treatment system waste collected from within the QPRC local government area.

The design allowance for additional flow and load from the Septage Receival facility is outlined in Table 3-3. For operational management these loads have be expressed as a maximum number of 10 kL tankers. The number of tankers will be assessed and reviewed with QPRC as part of ongoing operations based on the strength of the tankers.

As the septage waste material consists of partially digested sludge, the septage will be separately screened and directed to the STP digester.

Table 3-3 Current and Design Flow and Load Design Basis for the Septage Receival Facility

Parameter	Current kg/d (mg/L)	Design kg/d (mg/L)
Volume (kL/week)	Up to 110	Up to 150
Number of 10 kL Tankers (Tankers per week)	Up to 11 Tankers	Up to 15 Tankers
Number of Onsite Sewage Management Systems	4,554	6,618
EP	12,026	17,870
TSS	282 (20,000 to 40,000)	420 (20,000 to 40,000)
VSS	203 (14,400 to 28,900)	302 (14,400 to 28,900)
ISS	79	117
BOD_5	4 (3,100 to 6,200)	65 (3,100 to 6,200)
COD	320 (22,700 to 45,300)	475
COD_{up} + End	177	355
TKN	24 (1700 to 3,400)	36 (1700 to 3,400)
NH_3-N	11 (750 to 1,500)	16 (750 to 1,500)
TP	9 (630 to 1270)	13 (630 to 1270)

3.1.5 Raw Sewage and Bioreactor Temperature

The operating range for raw sewage and the contents of the activated sludge bioreactors is 12 to 24°C.

3.2 Effluent Quality Objectives

3.2.1 Licence Conditions

QPRC met with ACT EPA and NSW Department of Planning, Industry and Environment on 9 September 2019 to provide an initial briefing on the upgrade project. It was indicated at this time that no change to the existing Environmental Authorisation (referred to in this report as EA or "licence") would occur with the proposed upgrade.

The position of the ACT EPA was communicated via email to QPRC on 9 September 2019 which included the following statement:

> *"The upgrade of the STP itself does not trigger the need for a variation as such and as discussed we do not intend to change the discharge limits in your current EA."*

The current STP operates under the ACT EPA Environmental Authorisation No. 0417. The existing licence conditions are summarised in Table 3-4 and Table 3-5. The routine sampling frequency under the existing licence are summarised in Table 3-6.

Table 3-4 Allowable Concentrations and Loads in Wastewater under ACT EPA Environmental Authorisation No. 0417

Parameter	Concentration 50%ile Limit (mg/L)	Concentration 90%ile Limit (mg/L)	Average Daily Load Limit (kg/d)	Average Performance Period (months)	Sample Method
Biological oxygen demand (BOD_5)	5	10	50	3	24 hr composite
Total phosphorous (TP)	0.2	0.3	6	3	24 hr composite*
Total nitrogen (TN)	30	35	300	12	24 hr composite*
Suspended solids (SS)	8	20	90	3	24 hr composite
Total dissolved solids (TDS)	600	650	6000	12	24 hr composite
Parameter	**Concentration 50%ile Limit (cfu/100 mL)**	**Concentration 80%ile Limit (cfu/100 mL)**	**Performance Period (days)**		
Thermotolerant coliforms	200	1,000	35 days – grab sampling		
Parameter	**Lower Limit (pH value)**	**Upper Limit (pH value)**	**Sample Method**		**Sample Method**
pH	6.5	8.5	Average of continuous daily (via online analyser)		Grab

*Samples stored and analysed weekly.

Note the average performance for the parameters in Table 3-4 are calculated for a rolling period of the length specified in each "Average Performance Period" column for each parameter.

Table 3-5 Maximum Allowable Concentration of Ammonia in Wastewater as a Function of Receiving Water pH and Temperature under ACT EPA Environmental Authorisation No. 0417

Temperature of receiving waters °C	pH of receiving waters					
	6.5	7.0	7.5	8.0	8.5	9.0
	Ammonia as N mg/L (Monitored Daily)					
0	2.53	2.53	2.53	1.53	0.49	0.16
5	2.36	2.40	2.40	1.44	**1.47**	0.16
10	2.24	2.20	2.20	1.37	0.45	0.16
15	2.15	2.16	2.17	1.33	0.44	0.16
20	1.46	1.49	1.50	0.93	0.32	0.12
25	1.03	1.04	1.05	0.66	0.23	**1.10**
30	0.73	0.74	0.75	0.47	0.17	0.08

Note the values in bold in Table 3-5 appear to be miss-typed in the Environmental Authorisation. It appears the first number should be zero not 1 in both instances. In early versions of the Environmental Authorisation the figures appear correct.

Table 3-6 Required Sampling Frequency for Routine Wastewater Quality Monitoring under ACT EPA Environmental Authorisation No. 0417

Parameter	Unit	Wastewater Quality Monitoring
Acidity	pH value	Daily average from continuous monitoring
Ammonia	mg/L	Daily (24 hr composite)
Biochemical oxygen demand (BOD_5)	mg/L	Weekly (24 hr composite)
Total nitrogen (TN)	mg/L	Daily (24 hr composite)*
Total phosphorous (TP)	mg/L	Daily (24 hr composite)*
Suspended solids (SS)	mg/L	Daily (24 hr composite)
Temperature	°C	Daily Grab
Thermotolerant coliforms	cfu/100 mL	Weekly
Total dissolved solids (TDS)	mg/L	Weekly
Total daily flow	ML/day	Daily
Peak daily flow	L/s	Daily
Monthly irrigation volume	ML	Monthly

*Nitrogen and phosphorous samples are stored and analysed weekly.

3.3 Risks Associated with Existing Licence Conditions

The upgraded STP will provide an improvement in the level of treatment offered, compared against the existing STP, at every process step. However, the existing licence limits pose challenges for the upgrade project. Key risks of retaining the existing licence for the upgraded STP are:

- Load limits for total dissolved solids (TDS), total suspended solids (TSS) and biological oxygen demand (BOD) are very low and are not practical for the upgraded STP
- Total dissolved solids (TDS) concentration limits require liquid trade waste management in the catchment and cannot be achieved through treatment; and
- Ammonia concentration limits are hard to achieve, particularly in summer.

3.3.1 Load Limits

Table 3-7 indicates the calculated equivalent 50 percentile concentration that is required to meet the licence load limits at current and design EP. The load limits in the existing licence would be difficult for a design to guarantee. The existing TDS load limits are very low and not achievable. The BOD load limit is also very low and is close to the limit of detection for the analytical method. The TSS limits is low and well below the licence concentration limit.

Load limits for the existing STP are likely to have been based on lower flows. It would therefore be more appropriate for the design to be based on the current licence 50%ile concentration limits refer Table 3-4. As agreed at the Options Selection Workshop on 13 September 2019 the STP design is to be based on the concentration limits as per Table 3-4 up to 75,000 EP.

Table 3-7 Equivalent 50%ile Concentrations to Ensure Compliance with the Load Limits

Parameter	50%ile Target Concentration for Load Compliance 45,000 EP (mg/L)	50%ile Target Concentration for Load Compliance 75,000 EP (mg/L)	50%ile Licence Concentration Limit (mg/L)
Biological oxygen demand (BOD_5)	3.6	2.1	5
Total phosphorous (TP)	0.43	0.26	0.2
Total nitrogen (TN)	25	15	30
Total Suspended solids (TSS)	6.4	3.9	8
Total dissolved solids (TDS)	500	300	600

3.3.2 TDS Concentration Limits

Meeting the TDS concentration limits requires liquid trade waste management in the catchment. It is not practical or economic for the STP to remove salt to meet a TDS limit.

This upgrade has been designed to minimise the increase in TDS concentration by minimising salt addition that occurs through chemical dosing. In this context, the design will facilitate optimum biological phosphorous removal and hence, limit the need for alum addition for chemical phosphorous removal.

3.3.3 Ammonia Concentration Limits

More detailed commentary of what has been allowed for in terms of ammonia performance is provided in section 4.4 of this report.

3.4 Recycled Water

Recycled water produced by the treatment plant will be reticulated for use in onsite treatment process (e.g. screen washing) and site hose reels used for washdown. A standpipe will also be provided for filling tankers with recycled water for offsite uses managed by QPRC such as dust suppression.

Recycled water treatment standards will meet the requirements in the *NSW Guidelines for Recycled Water Management Systems* (NSW Department of Primary Industries, Office of Water, May 2015) and in the *Australian Guidelines for Water Recycling* (AGWR) (NRMMC, 2006). Further details are provided in section 6.11.

3.5 Biosolids

The *NSW Environmental Guidelines for Use and Disposal of Biosolids Products* (Environment Protection Authority, 2000) categorise biosolids using two grading systems: contaminant and stabilisation grade. The sludge handling and digestion facilities of the upgrade are designed to maintain stabilisation grade B based on the current guidelines.

Achieving stabilisation grade B places the biosolids in the Restricted Use 2 category; allowing biosolids to be disposed of to landfill or reused for agriculture, forestry, solid and site rehabilitation and land disposal.

Reuse for these purposes is dependent on suitable reuse sites being available and is only possible if the required contaminant grade is also met through appropriate implementation of QPRC's Liquid Trade Waste Policy (Queanbeyan Palerang Regional Council, March 2018).

The current NSW Biosolids Guidelines were originally published in 1997 and have been under review for some years. It is possible the aerobic digestion may be impacted by changes to the guidelines in future and the volume of the digester might need to be increased. Prudent steps to manage the uncertainty regarding possible changes to the guidelines are discussed in the *Queanbeyan Sewage Treatment Plant Upgrade Project: Design Criteria and Assumptions Report* (Hunter H2O, December 2019). The limitations with the current activated sludge models used in digestion are also discussed in that report.

4 Concept Overview

This section provides a high-level overview and discussion of the key elements of the design. Subsequent sections of the report provide detailed information on the overall hydraulic design and each unit operation.

The STP upgrade has been designed to provide an improved level of treatment compared with the existing STP.

Two key effluent quality aspects, disinfection and ammonia removal are discussed in this section in the context of the overall design as they have a large bearing on the treatment capacity.

4.1 Process Configuration

The flow diagram in Figure 4-1 shows an overview the process configuration and major unit operations of the upgrade. Flow ranges are shown for the major unit operations to indicate the range of flows of incoming sewage that receives treatment and each stage. The flow ranges are expressed as multiples of ADWF of incoming sewage at 75,000 EP. (Refer to the flow balance section for a more detailed discussion of hydraulic design.)

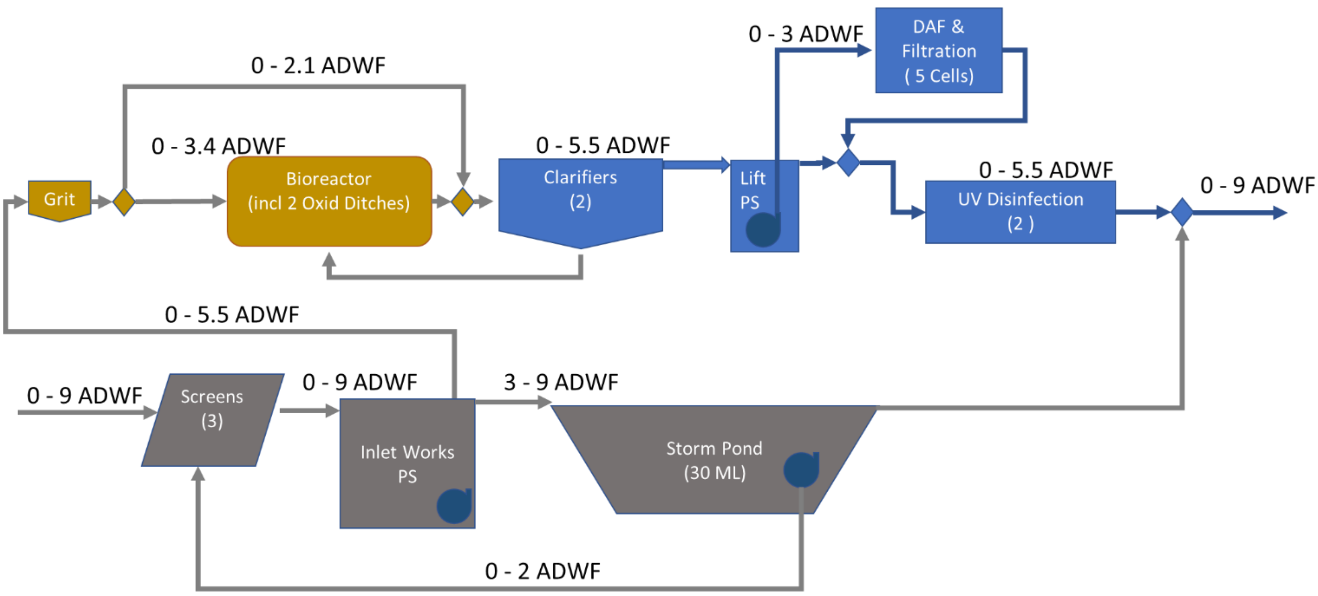


Figure 4-1 Concept Overview of Unit Operations and Liquid Stream Bulk Flows

Wet weather flows of up to 9 ADWF are received from the catchment. All flow is screened by 2-dimensional 5 mm aperture band screens. There are three automatic band screens configured in a duty, assist and standby arrangement. If all automatic screens fail, a 20 mm manual bar screen is also included to ensure all flows are screened.

Screened flow of up to 5.5 ADWF is lifted to the treatment and disinfection process by the Inlet Pump Station. During significant wet weather events, screened flows greater than 5.5 ADWF overflow by gravity to the Storm Pond where they are stored and returned.

The treatment process includes grit removal, two oxidation ditch bioreactors (including common anaerobic and final aerobic zones), two gravity clarifiers, dual media filters and UV disinfection.

The bioreactor /clarifier treatment process has two flow operating modes which include:

- **Full treatment.** The bioreactors are configured for nitrogen and phosphorus removal and can pass to peak flow of 3.4 ADWF through the bioreactors.

- **Solids contact treatment.** The clarifiers are configured in a solids contact mode to enable enhanced treatment of storm flows. During high wet weather flow events, a passive weir splitting structure downstream of the grit system starts directing flow above 3 ADWF (599 L/s) to the clarifiers. As wet weather flow received from the Inlet Pump Station increases, the flow to the bioreactors increases until a peak flow of 3.4 ADWF is reached. At this peak flow 3.4 ADWF (1,098 L/s) enters the bioreactor and 2.1 ADWF (419 L/s) flows via the bioreactor bypass and shandies with the bioreactor outflow flow in a splitter prior to the clarifiers. The shandy process provides solids contact treatment and enables particles and colloids in the bypass to attach to activated sludge flocs from the bioreactor and settle in the clarifier. As sludge is returned to the bioreactor at a high rate (up to 3 ADWF) up to half the ammonia and nutrients in the bypass are returned to the bioreactor for treatment using this enhanced process.

A two-stage phosphorus removal process has been used to provide a low level of phosphorus for discharge. The bioreactor / clarifier system has been configured with both biological and chemical phosphorus removal to remove the bulk of the phosphorus. Dissolved air flotation with dual media filters (DAFF) are provided after the clarifiers to polish the phosphorus to the low levels required for median and 90%ile compliance with the licence limits. Based on available flow data, instantaneous flows above 3 ADWF occur at least 6 times per year. On this basis a peak filtration capacity of 3 ADWF has been selected to ensure the upper 90%ile phosphorus limit is met.

The key benefit of the solids contact treatment operating mode is that it enables treatment of storm flows by making the clarifiers work much harder in wet weather. All clarifiers are mass flux limited. This means their main function is to thicken and settle the larger concentration (i.e. ~4,000 mg/L) of biomass existing the bioreactor. This need to thicken biomass in the clarifier restricts the solids load which can be applied per unit surface area. Clarifiers however, can accept much higher volumes of low concentration solids as this does not increase the solids load on them significantly. If the 5.5 ADWF treatment flow was sent directly through the bioreactor, the clarifiers would need to be 80% larger to treat the same flow handled by the solids contract treatment operating mode.

For wet weather flows there would not be any significant benefit in much larger clarifiers to treat all flows through the bioreactor. The flocculation of bypassed sewage with bioreactor solids provides good solids capture. Also, the return activated sludge process returns 50% of the bypass to the bioreactor for nutrient removal. This is demonstrated further below where disinfection and ammonia performance of the solids contact process are explored.

The dissolved air flotation filter has been designed to sit off the hydraulic grade line of the treatment process. That is flows are lifted to it from the clarifier. Using this approach if the filter fails or is unavailable, flow will overflow by gravity to the UV system.

A UV system will be utilised to treat up to 5.5 ADWF (1,098 L/s) or all flows that are presented to the clarifiers. It has been specified conservatively with a low UVT assuming the filters are not operating, and it is seeing only effluent from the clarifier in solids contact operation. A two-channel system will be provided with each channel having a 2.75 ADWF (549 L/s) capacity.

An extensive review of sewage flow and gauging stations identified the instantaneous peak dry weather flow is as a 95 percentile less than 2 ADWF. Therefore, in dry weather the instantaneous flow is not expected to rise above 2 ADWF which at 75,000 EP is 399 L/s. The total daily flows are less than 3 ADWF for most rainfall events less than 50 mm/day. Typically, total daily flows greater than 3 ADWF occurs 2 to 3 times per year. On this basis in all dry weather and most wet weather scenarios all flow will pass through every unit operation with the exception of the Storm Pond.

A 30 ML storm pond will provide a buffer storage for wet weather flows. It can operate in two configurations which include:

- **Maximise full treatment.** Flows above 3 ADWF overflow to the Storm Pond up to the 10 ML storage level. Provided the pond is not above 10 ML, flows are returned automatically to treatment when there is spare capacity under the 3 ADWF peak full treatment flow envelope. When the pond exceeds 10 ML the peak capacity of the treatment capacity increases to 5.5 ADWF and only flows above 5.5 ADWF enter the Storm Pond. Flows under the 30 ML total storage volume are returned automatically to the screens and treatment plant as there is spare capacity up to the 5.5 ADWF capacity envelope. The Storm Pond peak return rate is 2 ADWF.

- **Peak treatment operation only.** Flows above 5.5 ADWF only overflow to the Storm Pond. Flows are returned automatically to the screens and treatment plant at a rate up to 2 ADWF when there is capacity under the peak 5.5 ADWF treatment capacity envelope.

The maximise full treatment approach uses the storm pond to temporarily store moderate to high wet weather events ensuring the treatment plant only operates in solids contact mode for more extreme events.

The peak treatment operation approach is more aggressive and seeks to reduce the frequency of potential pond overflow. This mode may be selected if significant wet weather is predicted to minimise the likelihood of pond overflow. It is envisaged the plant would normally be configured in maximise peak treatment as it will provide enhanced treatment for most wet weather events.

In significant wet weather events, the storm pond has been designed to overflow and shandy with treated and disinfected effluent prior to discharge to an on-bank discharge structure. The effluent which overflows from the pond will be less than 3.5 ADWF at 75,000 EP (699 L/s) and receive a minimum of full screening to 5 mm by the band screens and sedimentation in the Storm Pond. A median of 60% solids, 30% BOD and 12 % TP and TN removal is expected in the Storm Pond. The settled solids and nutrients will be eventually returned to the treatment process via the pump at the base of the Storm Pond.

If the pond overflows, the quality will be reasonable given heavy dilution by rainfall and the solids removal which occurs in the Storm Pond. A median of 20 mg/L TSS, 30 mg/L BOD, 2 mg/L TP and 8 mg/L TN is expected for the effluent overflow from the pond prior to shandy with treated effluent.

A diesel generator will back up the power supply. In the event of power failure, the generator will provide power to ensure flow can be lifted into the treatment process and run the Inlet Works, Storm Pond return pumps and liquid treatment stream (i.e. bioreactor, clarifier, filters and UV). As an additional protection measure, the overflow from the Storm Pond to the on-bank discharge structure has been sized to pass the full flow which can be received (9 ADWF). This is to ensure a controlled release of flow under an extreme situation of no power or generator and a full pond which is very rare.

The sludge produced by the biological and filtration processes will be thickened in a gravity thickener and aerobically digested. Digested sludge will be drawn from the digester and dewatered in two centrifuges. Biosolids produced from the centrifuges will be conveyed to one of two pivoting conveyers that will spread the material into a truck body. Up to a semi-trailer sized truck body has been allowed for.

4.2 Storm Pond Sizing

The only extensive sewage flow database available was 24-hour integrated flow received at the treatment process. The database consisted of approximately 10 years of data (i.e. from 2009 to current). Instantaneous sewage flow data was available for two sewer gauges over a shorter much drier period from 2015 and missed 2017 which was a wet year. This data is unlikely to have picked up peak flow events associated with significant wet weather needed for sizing of the Storm Pond.

A daily step model was developed using the existing 24-hour integrated flow database. Normalised (i.e. flow/ADWF) flow was developed from the database and used to project future flows at a higher 75,000 EP design loading. Based on the peak treatment capacity of 5.5 ADWF the volume needed to reduce the frequency of overflow from the storm pond to 1 event in 10 years was calculated. The volume required was 20 ML.

The figures below show the annual rainfall statistics for the Queanbeyan Bowling Club as well as measured rainfall during the sewage flow database period. Most of the rainfall data is at the long-term median with two of the years at or above the long term 90%ile. The year 2010 was above the 95%ile and closest to highest on record. Based on a review of the rainfall records, the shorter 10-year flow database is believed to be a reasonable representation of the long-term rainfall records for the region.

To maximise peak treatment an additional volume of 10 ML (total 30 ML) was added to enable operation in “maximise full treatment mode” noted above. Based on an initial peak treatment capacity threshold of 3 ADWF a volume of 10 ML will on average mean the pond temporarily receives flow up to 5 times per year up to the 10 ML threshold, where it switches to high capacity 5.5 ADWF treatment. The additional buffer volume above 10 ML (i.e. 20 ML) and operation at a higher peak flow of 5.5 ADWF based on the above modelling should limit overflows from the Storm Pond to a 1 in 10-year frequency.

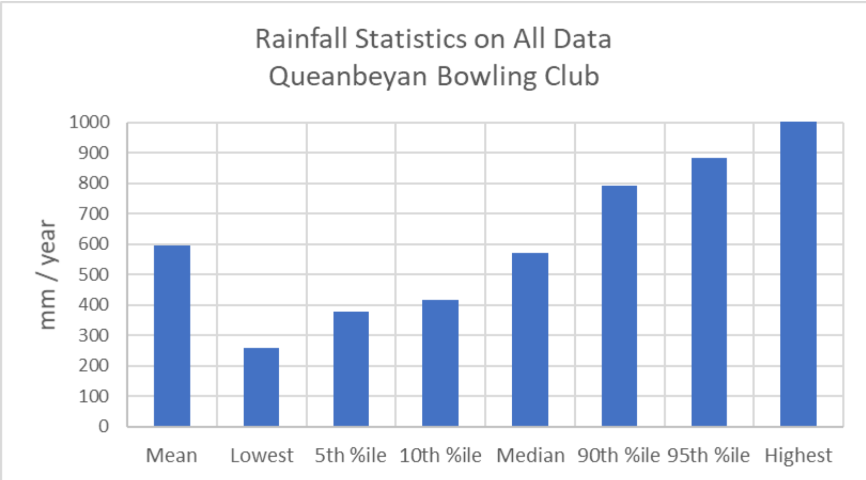


Figure 4-2 Rainfall Statistics for Queanbeyan Bowling Club Based on 149 years of Data

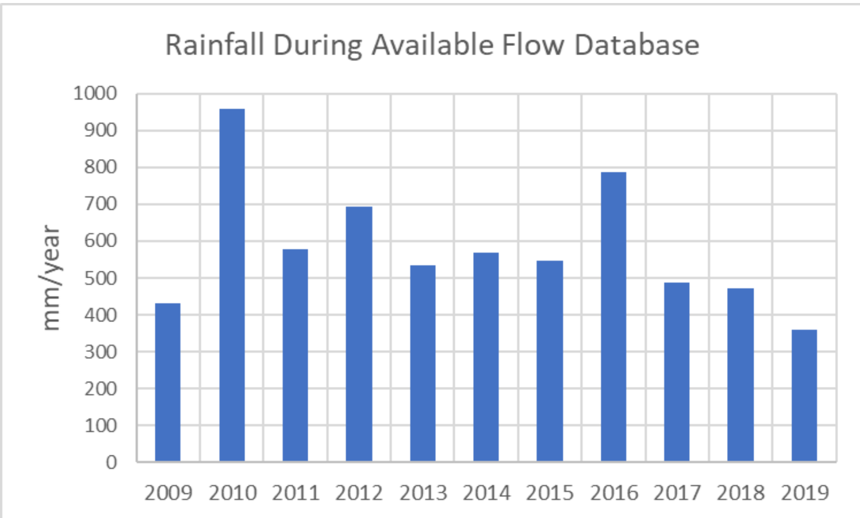


Figure 4-3 Annual Rainfall during Sewage Database Period

4.3 Disinfection System Sizing

The UV system has been designed to treat up to 5.5 ADWF at 75,000 EP. Bypass of the disinfection can only occur if the Storm Pond overflows, as most wet weather events will either be treated or captured by the pond for later treatment. Based on the Storm Pond modelling above, the combination of the pond and a high treatment capacity means a bypass of the disinfection system will occur at an expected frequency near the 1 in 10-year recurrence interval. This will coincide with significant wet weather events. In these instances, flows will be high in the receiving environment.

A key design consideration for UV systems is the UV Transmittance (UVT). UVT is the percentage of the light source at 254 nm that passes through 1 cm of a sample. Particles or dissolved organic substances can adsorb light. Setting the minimum likely UVT is important to ensure the UV system is sized with enough UV light power to handle possible reduction in UVT at high flows.

In the case of the Queanbeyan STP, the worst case likely UVT assuming no filtration and the plant operating in solids contact mode was explored. This was achieved by running a series of jar tests with plant sewage and activated sludge from the current Queanbeyan STP reactor. The plant sewage was diluted with potable water at various ratios associated with different flow/ADWF ratios in wet weather. The diluted sewage was then combined with activated sludge in the ratios expected for various plant operating scenarios (e.g. 4 ADWF 3 parts activated sludge settled effluent and 1-part diluted sewage). To test the impact of the solids contact process, the diluted sewage and activated sludge where combined in different ways as follows:

- **No Solids Contact.** Diluted sewage was combined with the existing plant clarifier effluent. This effectively assumes the solids contact process provides no flocculation/sedimentation benefit and it is a straight blending process.
- **Solids Contact.** Diluted sewage was blended with suspended activated sludge in a jar, gently stirred to simulate the mixing which will occur in the mixed liquor splitter, feed pipe and the flocculation well in the clarifier. The combined mixture was then allowed to settle, and a sample decanted.

The no solids contact tests acts as a control by which to assess the performance improvement the solids contact flocculation process provides. The results from these site-based jar tests are presented in Figure 4-4. The report including the experimental procedure are outlined in Appendix K.

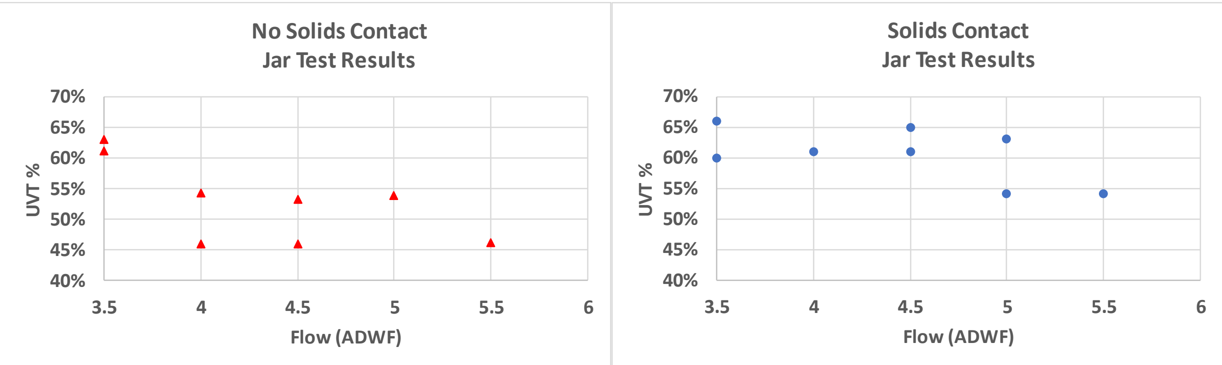


Figure 4-4 UVT Results with and without Solids Contact in the Clarifiers versus Flow Treatment Multiple

It is clear from the results in Figure 4-4, a solid contact process will actively flocculate material in the bypass around the reactor. Compared to the straight blend results the UVT stays much higher indicating particle capture and some organics adsorption to flocs is occurring.

In the concept design a UV reactor was selected based on the median UVT dropping to no lower than 50% at the extreme of 5.5 ADWF treatment. Based on this jar test run this initial allowance appears reasonable. Further equivalent jar testing is planned in the next reference design stage to assess this aspect further and to finalise the UV design to ensure it can treat and meet the pathogen limits with confidence up to 5.5 ADWF.

4.4 Ammonia Performance

The temperature and pH data for Molonglo River as measured at Oaks Estate (receiving environment) are plotted in Figure 4-5. The corresponding ammonia limits (in accordance with the licence) for the pH and temperature extremes of the receiving environment are presented in Table 4-1. Note the current in river baseline water quality monitoring will provide further data for analysis of river condition during the reference design stage.

The summer extremes show the maximum allowable ammonia limit to meet licence conditions can drop as low as 0.23 mgN/L in summer. This low limit will be difficult for any plant configuration to guarantee as a maximum for all flows. In storm conditions it is likely this limit would be breached for any plant even if a much higher plant capacity was provided.

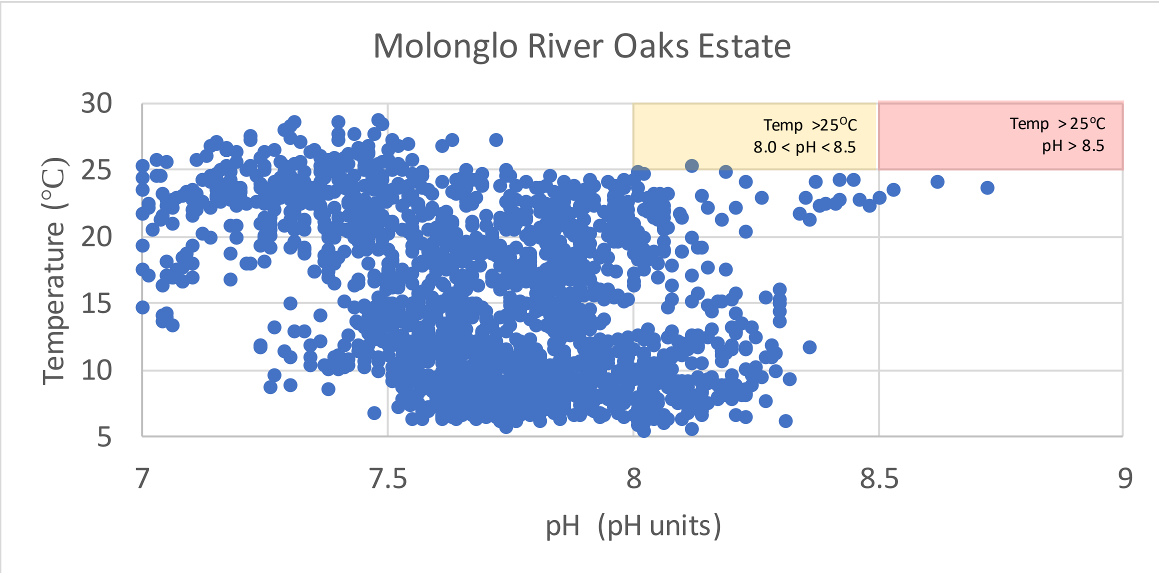


Figure 4-5 River Temperature as a Function of River pH for STP Receiving Environment (Molonglo River Oaks Estate) (03/04/11 - 04/06/19) (Gauge at Molonglo River at Oaks Estate Site 410729)

Table 4-1 River Summer and Winter Extremes of Temperature and pH, and the subsequent Maximum Allowable Ammonia Concentration in Wastewater under current ACT EPA Environmental Authorisation No. 0417

River Temperature (°C)	River pH	Maximum allowable NH3-N concentration (mgN/L)
Summer Extreme		
28	7.5	0.87
25	8	0.66
25	8.5	0.23
Winter Extreme		
5	7.5	2.4
5	8.3	1.46

For ammonia removal to occur, ammonia needs to be oxidised by autotrophic bacteria in the bioreactors. To meet very low maximum limits under all flow conditions would mean some portion of storm flows would need to pass through the bioreactor. In this design there are two ways sewage can enter the bioreactors which include:

- Bioreactor inlet
- Return Activated Sludge (RAS) system. Bypasses which enter the clarifier in solids contact operation are partially recycled back to the bioreactor by the RAS pumps.

During wet weather the ammonia concentration will reduce due to dilution with rainwater. Taking rainfall dilution into account, modelling was undertaken to assess the likely ammonia effluent quality for a range of different clarifier operating conditions and the results are expressed in Figure 4-6. Three curves are presented which includes the current concept where the clarifiers have a Peak Full Treatment Flow (PFTF) (i.e. flow passed directly to bioreactor) of a minimum of 3 ADWF and a higher Solids Contact (SC) combined bioreactor / bypass capacity of 5.5 ADWF. The two other curves are for more clarifier surface area which can accommodate up to 5 ADWF PFTF and full treatment of storm flows through the bioreactor and clarifiers.

The key assumption with this modelling includes:

- No trade waste inhibitors are impacting biological growth of autotroph bacteria
- Rainwater dilution does not contribute an additional ammonia load. The ammonia load is from the sewage only
- All bioreactors and clarifiers are operating.

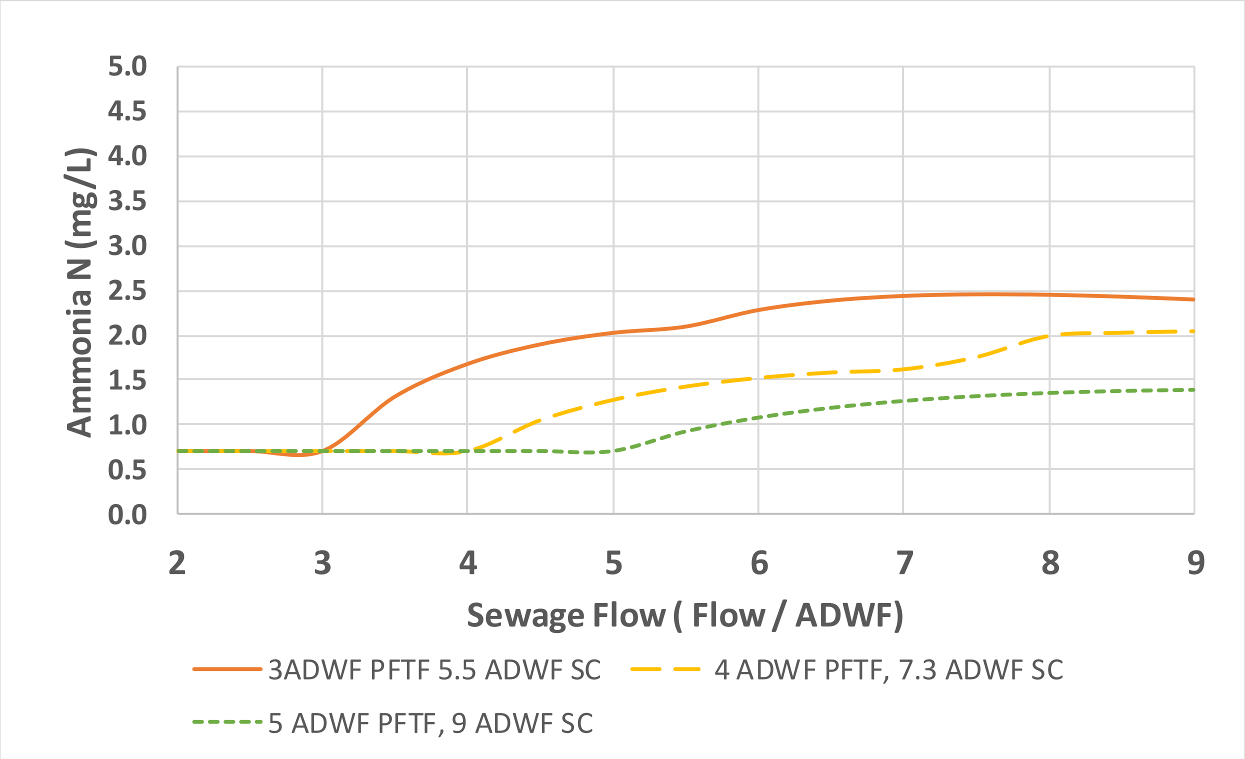


Figure 4-6 Expected Ammonia Quality versus Flow Treatment Multiple for Various Peak Full Treatment (PFTF) and Solids Contact (SC) Capacities

It can be seen with the proposed 3 ADWF minimum PFTF and 5.5 ADWF solids contact process there is a slight decline in ammonia removal with increasing flow. However, a significant increase in treatment capacity from 3 to 5 ADWF PFTF provides minimal improvement in ammonia, despite almost twice as much clarifier area being provided. Even with such a large clarifier system the very low maximum ammonia limits which are all less than 1 mg/L (refer Table 4-1) cannot be met. In summary at the wet weather extremes, exceedances of the maximum ammonia limit would be likely even if significant clarifier capacity was installed.

When the plant is operating at high flow events it is likely the Molonglo River will see a deterioration in upstream ammonia concentration. Figure 4-7 shows results from online ammonia water quality from the Oaks Estate Road Quality Station and flow from the Molonglo road station. Both are upstream of the existing QSTP discharge location. It can be seen during a wet weather event on the 5 March 2020 (38 mm at Canberra Airport) ammonia considerably rises from ~ 1 mg/L to 2.5 mg/L over a 24-hour period. For this event the proposed 3 ADWF PFTF 5.5 ADWF solids contact clarifier configuration is still likely to be operating in the peak full treatment envelop using the Storm Pond to buffer storm flows. Under this scenario the ammonia discharged will be less than 1 mg/L and the plant effluent would improve the ammonia quality in the receiving environment. Note, that, as above, current in river baseline water quality monitoring will provide further data for analysis of river condition during the reference design stage.

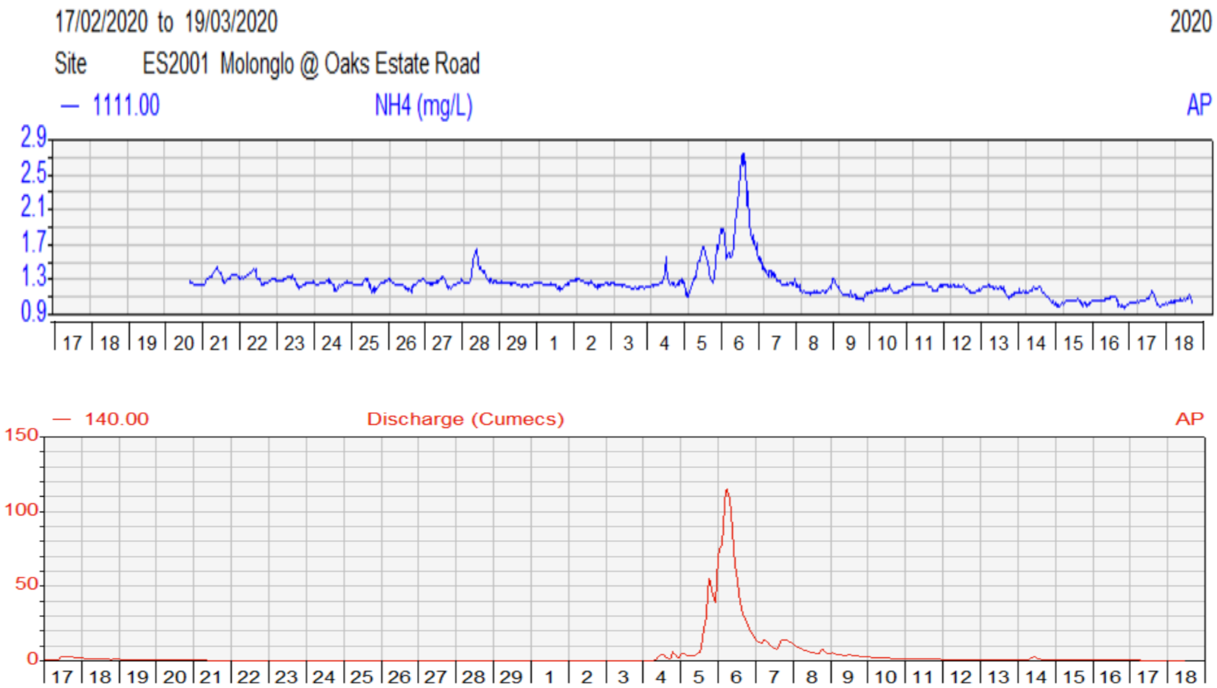


Figure 4-7 Recent Upstream Ammonia Concentration and River Flow (20/02/20 - 18/02/20) (QSTP Stream Station ES2001 & QSTP Flow Station 410729)

The cost of a 5 ADWF PFTF 9 ADWF solids contact clarifier process is significant with a likely project capital cost increase of $11 M over the proposed concept. However, this will only improve ammonia removal performance in extreme wet weather events by a very minor amount of 1 mg/L at a time when in river conditions are substantially deteriorated.

Providing large clarifiers for occasional use in wet weather can also cause problems with effluent quality performance in dry weather which include:

- Too much clarifier surface area increases the amount of biomass stored in the clarifier system. This can reduce the aerobic mass fraction of the biomass in the activated sludge system which slightly elevates dry weather ammonia.
- In lowly loaded clarifiers it can be difficult to effectively convey the small amount of biomass per unit area to the clarifier hopper. This can lead to sludge accumulation and flotation of sludge if storage is too long, leading to poor suspended solids effluent quality.
- Excessive sludge storage can result in anaerobic conditions forming in the clarifier sludge blanket and phosphorus release by the Polyphosphate Accumulating Bacteria (PAO) responsible for biological phosphorus removal.

It is likely that if larger clarifier surface area was provided than the proposed concept, clarifier(s) would need to be taken offline in dry weather. This will increase the operational complexity of the plant in returning them to service when needed. The stored effluent in the offline clarifier will likely degrade and grow algae which will interfere with the effluent quality when the clarifier is returned to service.

The modelling indicated long term daily expected composite ammonia performance. It cannot pick up load transient issues associated with initial first flush loads on a bioreactor. Most plants typically observe ammonia concentration spikes up to a few milligrams per litre for short periods at the start of wet weather events. This occurs in all activated sludge processes as the ammonia load rapidly increase within a reactor (assuming no bypass) due to a sudden wet weather flow ramp up. There is a stable constant population of autotroph bacteria in the reactor which have grown in reaction to the normal dry weather load. Under rapid load changes their population is not high enough to respond and some ammonia loss occurs. This phenomenon normally occurs at the start-up of wet weather events when dry weather sewage is flushed at high concentration to the bioreactor. Once dilution occurs in the network and the ammonia loads return to normal values performance stabilises. This can happen for some hours with sudden storm events occur.

Based on the above considerations, a 3 PFTF and 5.5 solids contact clarifier design will provide significant ammonia removal and has been recommended. Further improvements in treatment capacity will provide very minor ammonia removal benefit and very limited river improvement at considerable extra cost, further adding to operational complexity, degrade dry weather effluent quality and likely result in algae issues in the effluent.

5 Flow Design

5.1 Flow Balance

A detailed flow balance for the entire plant has been developed and used in the hydraulic design of the plant. A sketch of the flow balance including details on all flows is provided in Appendix B. The flow balance allows for the bulk flows noted in Section 3.5 as well as:

- The need for recycled water and / or potable water to operate key unit operations such as band screens, screen washing, grit classification, scum removal and chemical dilution
- Internal process recycles
- Process flows associated with management of the sludge digestion, dewatering and filtration system
- Rainfall on key plant surface which add to the treatment flows.

A rainfall allowance has been calculated by considering Bureau of Meteorology (BOM) data. A 30 min 2% Annual Exceedance Probability (AEP) has been allowed for which is 72 mm/hr. This value has been used to calculate instantaneous rainfall flow for plant catchment areas such as the bioreactor, clarifier and Storm Pond (when full) that will pass rainfall flow on promptly to other units as it is collected.

Rainfall flows have been considered for the digester. The freeboard vessel allowance is considered adequate to manage rainfall. Slight adjustment to thickening operation can be made after rainfall to re-thicken the biomass to the original value.

5.2 Hydraulic Grade Line

The hydraulic grade line is provided in Appendix B.

Two Hydraulic Grade Line (HGL) drawings have been developed which include the two main gravity flow paths of:

- Sewage inlet receival, screening, Inlet Pump Station, Storm Pond to the on-bank discharge structure
- Grit removal, bioreactors, clarifiers, UV to the on-bank discharge structure. Sewage is pumped initially from the Inlet Pump Station to grit removal.

The 1 in 100 flood level is 572 m AHD in the vicinity of the proposed on-bank discharge structure.

The flow path from sewage receival to on-bank discharge structure has been designed assuming full failure of the plant and backup generator with PIF flows of 9 ADWF (1797 L/s + rainfall flow) discharging by gravity. Gravity flow can just be achieved and meet the current 1 in 100 flood level with 50 mm of contingency. It must be noted this is an extreme flow scenario and not intended to occur. The plant if operating with power supply will only discharge 3.5 ADWF (699 L/s + rainfall flow) as 5.5 ADWF will be pumped to treatment. As noted in Section 4.2, overflow from the pond is a rare event (1 in 10 year expected).

Both the Morisset and Jerrabomberra trunk sewer mains will be received in a new inlet works. The Top Water Level (TWL) at the Inlet Receival Pit of the Inlet Works is 574.3 m AHD. This level is set to ensure there is no impact on the gravity sections of the Morisset sewer main.

The treatment plant hydraulic grade line has been positioned to ensure bioreactor, clarifiers and UV are positioned at ground level (i.e. top of structure is at handrail height generally). This simplifies operation and reduces costs associated with access to elevated structures. It also reduces the sewage lift which lowers plant operations cost.

To ensure the treatment plant sits on the ground level, relatively low design velocities at peak flow (< 1 m/s) have been used for sewage and effluent pipework. For pipelines between the bioreactor and clarifiers the velocities have needed to be kept high (up to 2 m/s). This results in reasonable head loss (~ 2.4 m) between the bioreactor and clarifier water levels. This is necessary to ensure the velocities are not too low for dry weather operation where activated sludge settlement may occur.

For the treatment plant HGL a tailwater was achieved with a 500 mm contingency above the 1 in 100 level of 572 m AHD. That is the treatment plant (not Storm Pond overflow) could operate at full capacity even if the river level at the on-bank discharge structure was at 572.5 m AHD. The invert and top water level for the UV are 572.8 m and 573.8 m AHD. This puts the UV structure well clear (> 1.8 m) of the 1 in 100 flood contour.

In a range of equipment/structure offline scenarios the hydraulic limitations at 75,000 EP are:

- Common anaerobic zones operating, one oxidation ditch only (one offline) and both final aerobic zones operating, flows up to 2 ADWF (400 L/s) can only pass to the bioreactor inlet
- Common anaerobic zones are offline, flow is received at the first anaerobic and is diverted to one oxidation ditch only (one offline) and both final aerobic zones operating, flows up to 3.4 ADWF (679 L/s) can pass to the bioreactor inlet
- A clarifier is offline. After allowing for the RAS flow the clarifier hydraulic capacity is limited to 525 L/s based on sewage flow. However, as noted in Section 6.7 the process capacity is limited to 399 L/s to prevent sludge blanket accumulation.

These hydraulic restrictions will be addressed using standard operating procedures for taking the plant offline. Practically, the operator will need to set a peak flow limit on the Inlet Works Pumping Station to match the hydraulic restriction while it occurs.

Refer to Appendix B for a summary of the HGL calculations. Also refer to the HGL drawings for a representation of peak water levels and structure positions.

The system network hydraulics are proposed to be reviewed further once updated survey data is compared to current system performance and static model. This will require updating the current network model during the reference design phase to confirm no further impacts to the plant hydraulics in terms of incoming diversions etc.

5.3 Allowance for Plant Staging

The hydraulic design has been revised following consultation with NSW DPIE to better facilitate a potential future stage 2 upgrade of the facility to increase the treatment capacity by 50% to a total of 112,500 EP. The revised hydraulic design includes the following allowances to facilitate the incorporation of a future upgrade:

- Inlet works treatment and hydraulic capacity including screening and bypass hydraulics are sized for Stage 2
- Grit removal facility treatment and hydraulic capacity are sized for Stage 2
- Flow splitter structures including the Bioreactor and Clarifier Weir Splitter have been designed for Stage 2 hydraulic requirements and include future connection points for a Stage 2 bioreactor and clarifier
- The UV channel has been designed with hydraulic capacity and space for the additional UV lamps required for stage 2.

Further details are provided in the relevant sections of section 5.3.

6 Process and Mechanical

Presented in this section of the report is a summary of the key design requirements for process and mechanical equipment. Further details of the process and mechanical design is provided in the P&ID in Appendix C.

6.1 Septage Receival

A septage receival facility has been allowed for. The septage receival facility will be located in a separate compound. Consideration will be given to ensuring the facility is secure including locks and fencing.

Septage will be screened using a 20 mm aperture manually raked bar screen before being pumped to the inlet works to be screened with the automatic screens. Tankers up to 10 kL will connect to a 100 mm diameter rigid pipeline including flow meter which will direct the flow into a manual screen chamber. Downstream of the screen chamber will be a sump where a macerating pump will be used to pump the material to the inlet works.

A 20 L/s pump and a 3.6 m^3 pumping station sump will be used to balance the discharge flow from a 10 kL tanker assuming the tankers height above the facility is 3.1 m and has a depth of the tank is no more than 1.6 m. In this situation a 100 mm hose of up to 5 m length connected to the rigid pipe will ensure the sump does not overtop. If a larger diameter hose of up to 150 mm was connected to rigid pipe the screen structure is likely to fill up to 50% of the volume in the screen chamber. That is the pumping station sump will overtop, however it will likely be contained in the screen sump until the tanker stops discharging.

It is recommended hoses no larger than 100mm are connected to the pipe which feeds the receival facility. Also, it is recommended tankers no larger than 10 kL can free drain into the facility. Larger tankers may be able to discharge if the tanker driver carefully watches and slows / stops flow if the level rises too high in the screen well. Tankers must also not discharge until the pump has stopped and cleared the well. The facility can accept and clear a 10 kL delivery within 9 minutes.

The levels in the vicinity of the receival area make a gravity discharge option to the inlet works not feasible.

6.2 Inlet Works

6.2.1 Screening

The Inlet Works treatment consists of the receival pit, automatic band screens, screen washing and Inlet Works Pumping Station.

The automatic band screens are designed to accept PIF (9 ADWF – 1797 L/s + recycles) using two of the three screens. In the event of full failure of the automatic screens the level will rise in the Inlet Works and overflow a weir to a manually raked screen. This will provide coarse screening of PIF for a period until the issue with the automatic screen can be resolved.

The band screens use recycled water to rinse screenings off the screen. This water flows by launder sluice to one of two screen washing systems. Some recycled water can be used to rinse the launder and assist with conveying screenings to the screen washing systems. The rinse water is used in the screen washing system to wash the screenings.

A common issue at many treatment facilities the accumulation of screening material in the bioreactors over time. This can lead to ragging of aeration diffusers and other equipment. This upgrade includes the ability for operators to return WAS to the Inlet Works for rescreening by the band screens. This would be done from time to time during low flow periods.

The screen washing systems compact and remove water from the screenings. This water which is drained is returned via gravity to the Receival Pit ahead of the screens.

The height of the manually raked screen is set to ensure PIF flows can overflow to the Inlet Works Pumping Station without surcharging the Inlet Works Structure if the screen becomes blocked due to an extensive automatic screen outage.

6.2.2 Inlet Works Pumping Station

The Inlet Works Pumping Station is designed to lift to 5.5 ADWF (1,098 L/s) at 75,000 EP to the treatment process. A mixer is included in the pump station to minimise sedimentation of sewage.

The Inlet Works Pumping Station includes at least 100 m^3 of control volume to provide the required flow balancing of peak dry weather flows from the incoming trunk sewer mains. Further flow balancing is provided by the variable speed pumps which pump flows to treatment.

6.2.3 Odour Management

The odour assessment undertaken as part of the EIS indicates that the new treatment plant is likely to reduce odour impacts in the area compared with the existing treatment plant.

As part odour management for the new STP, sodium hydroxide can be dosed into the Inlet Works to raise the pH to supress odour generation. At higher pH values the chemical equilibrium for H_2S shifts away from gaseous H_2S to the ionised form (HS^-) which is less odorous. It is likely at times some sodium hydroxide will need to be dosed to support chemical phosphorus removal in the treatment plant. When needed, the sodium hydroxide will be dosed at the Inlet Work to boost alkalinity as it will provide the added benefit of lowering odour in the Inlet Works and grit removal structures.

Hydrogen sulphide concentrations at the existing inlet works were measured over a two-week period in April 2019 and compared with workplace exposure standards (Safe Work Australia, 2013). The observed concentrations were well below exposure standards.

- The short term exposure limit (STEL) concentration over a 15 minute period for hydrogen sulphide is 15 ppm. The eight hour time weighted average (TWA) exposure limit is 10 ppm.
- The monitoring at QSTP recorded a maximum 15 minute time weighted average of 1.5 ppm and a maximum eight hour time weighted average of 6.3 ppm.

6.2.4 Consideration for Future Plant Staging

The capacity of the inlet works would be difficult to upgrade at a later date. For this reason, the proposed inlet works has been sized for the future 112,500 EP using the following approach:

- The manual screen can take the future PIF for 112,500 EP of 2,695 L/s (9 ADWF). The screen can take this flow over the screen if it blocks without surcharging the structure.
- The screen washing system will be sized for stage 2. The minimum unit size for stage 1 will handle the higher stage 2 loads.
- The automatic screen philosophy will be changed from duty, assist and standby to three duty for 112,500 EP PIF operation. This will provide up to 2,850 L/s which provides some contingency for future recycles.
- The pumps installed at 75,000 EP will be replaced with equivalent pumps which can provide the 112,500 EP duty of 412.5 L/s/pump.
- The overflow weir to the storm pond will be sized for the future PIF of 2,850 L/s assuming full pump station failure

6.2.5 Summary

A summary of the design for the Inlet Works is provided in Table 6-1.

Table 6-1 Inlet Works Design Summary

Item	Value	Units	Notes
Automatic Screens			
Automatic band screen No.	3		Operates as a duty / assist / standby
Screen aperture	5	mm	Punched hole two-dimensional screening
Automatic band screen capacity	945	L/s/screen	
Band screen rinse flow	5	L/s/screen	
Common screens launder rinse flow	4	L/s	
Manual Screen			
Number of manual screens	1		
Manual screen aperture	20	mm	1 dimensional bar screen
Screen Washing			
Number of units	2		Duty / standby
Screenings load	15 150	kg/ML kg/ML	dry basis wet basis (10% assumed solids contents)
Capacity of unit	1.5 19	m^3/h/unit L/s/unit	Wet screenings. 1.5 required for Stage 2. At stage 1 1 m^3/h is expected. Total launder inlet flow will not change for stage 2 as allowance for all flow at one time incl. Volume of screening is the wet load the exists the band screen. The allowed solids content is 10%
Inlet Works Pump Station			
Number of pumps	5		Duty / assist / assist / assist/ standby
Pump capacity	275	L/s/pump	With 4 operating (variable speed)
Minimum pump flow (at 30 Hz speed – full turndown)	50	L/s/pump	0.25 x ADWF with one pump 0.38 x ADWF at commissioning

6.3 Storm Pond and Return Pump Station

Screened effluent which cannot be pumped to the treatment plant will overflow to a storm pond.

Minor storms may result is some influent being stored for a short period in the Storm Pond. The operation philosophy of this pond will be to return the stored flows as soon as possible when there is spare treatment capacity. There two operating philosophies are outlined in Section 4.2 for the pond system.

A pump station in the pond has been included to return the stored diluted sewage to the Receival Pit for further screening and pumping to the treatment plant. This return will occur automatically, and the pump will be operated to provide the maximum flow the treatment plant can tolerate. This may mean more flow returns at lower flow periods overnight.

In extreme wet weather events, the Storm Pond may reach top water level. In these instances, it is designed to overflow via a weir system and combine with treated effluent and be discharged to the on-bank discharge structure. The storm pond overflow can accept up to PIF to allow for full failure of the power supply and plant generator.

The drawings have allowed for the later addition of a second equivalent sized pond and pumping system for the potential future expansion of the plant.

A summary of the design for the Storm Pond system is provided in Table 6-2.

Table 6-2 Storm Pond and Return Pump Station Design Summary

Item	Value	Units	Notes
Storm Pond			
Number	1		Operates as a duty / assist / standby
Volume	30	ML	10 ML provide for operation of the treatment plant with full treatment (at least 3 ADWF) 20 ML provided to allow for operation of the treatment plant up to 5.5 ADWF when operating in solids contact mode.
Storm Pond Return Pumps			
Number of return pumps	2		Duty / standby (variable speed drives)
Return pump capacity	350	L/s/pump	Sized to return the entire volume in 24 hours
Minimum pump capacity	150	L/s/pump	30 Hz speed full turndown.

6.4 Grit Removal

All flow directed to treatment initially passes through a JETA style vortex grit removal system. The grit system can be manually bypassed if required for maintenance. The grit removal system is positioned ahead of the bioreactors and does not form part of the Inlet Works structure. This structure has been sized for with consideration of stage 2 treatment capacity and hydraulic capacity of 1,647 L/s (i.e. 5.5 ADWF at 112,500 EP).

The settled grit is periodically pumped to a grit classifier. The classifier both washes and concentrates the grit. Separated foul water from the grit system is returned to the Receival Pit of the Inlet Works.

A summary of the design for the grit system is provided Table 6-3.

Table 6-3 Grit Removal Design Summary

Item	Value	Units	Notes
Vortex Grit Removal			JETA style of vortex grit trap.
Number	1		Operates as a duty. Can be bypassed.
Capacity	1750	L/s	1,650 L/s required based on the flow balance for future plant upgrade. The next standard size up (1750 L/s) was selected.
Diameter	5.48	m	
Recycled water supply	3	L/s	For flushing of hopper prior to pumping
Grit Pumps			
Number	1		One duty and one cold standby
Pump capacity	23	L/s	To suit requirements of classifier. Direct online pump.
Grit Classifier			BACHE style constant velocity trough and auger clarifier separator.
Number	1		Duty
Recycled water supply	5	L/s	For washing cycle.

6.5 Bioreactor and Clarifier Weir Splitter

After the grit chamber sewage, enters a passive orifice/weir splitter. This splitter is designed to enable bypass to the clarifiers at flows above 3 ADWF at 75,000 EP (599 L/s). Flow passes normally along a channel for flows less than 3 ADWF and flows through an orifice to the bioreactor. As flow exceeds 3 ADWF the level in the channel starts to overtop a long bypass weir (10 m, 2 x 5 m weirs on each side of the channel) and gradually introduces flow to the clarifier. The combination of the long bypass weir (10 m) and orifice (0.25 m^2 for stage 1 and 0.39 m^2 for stage 2) ensure only minimal extra flow is sent the bioreactor. This regulation of the bioreactor flow is to prevent overloading the solids flux capacity of the clarifiers. However, at full solids contact flow of 5.5 AWDF, 3.4 ADWF is flowing to the bioreactor and 2.1 ADWF via the bypass to the clarifier.

The flows during operation of the splitting weir are shown in Figure 6-1.

The passive weir/orifice system is designed to be robust and operationally simple. There is no control automation required to make it function. The flow split is sensitive to the positioning of the bypass weir and orifice opening. For all weirs in this splitter, metal weir plates with elongated bolt holes, to adjust the weir position if movement occurs, are recommended. An adjustable orifice by penstock will also be provided. It will normally stay and be locked in one position.

The Bioreactor and Clarifier Weir Splitter design includes consideration of the hydraulic requirements of a future stage 2 upgrade. A slight increase in the orifice area would enable stage 2 to continue to achieve the correct split with no further modification to the bypass weir. Flows for the future splitter are very similar to that presented in Figure 6-1, however the orifice size needs to be increased to 0.39 m^2

The bioreactor flow discharging from the orifice enters a second splitter chamber to be split between the 75,000EP bioreactor and the future third bioreactor. The connection for the future bioreactor will be constructed and sealed off until required.

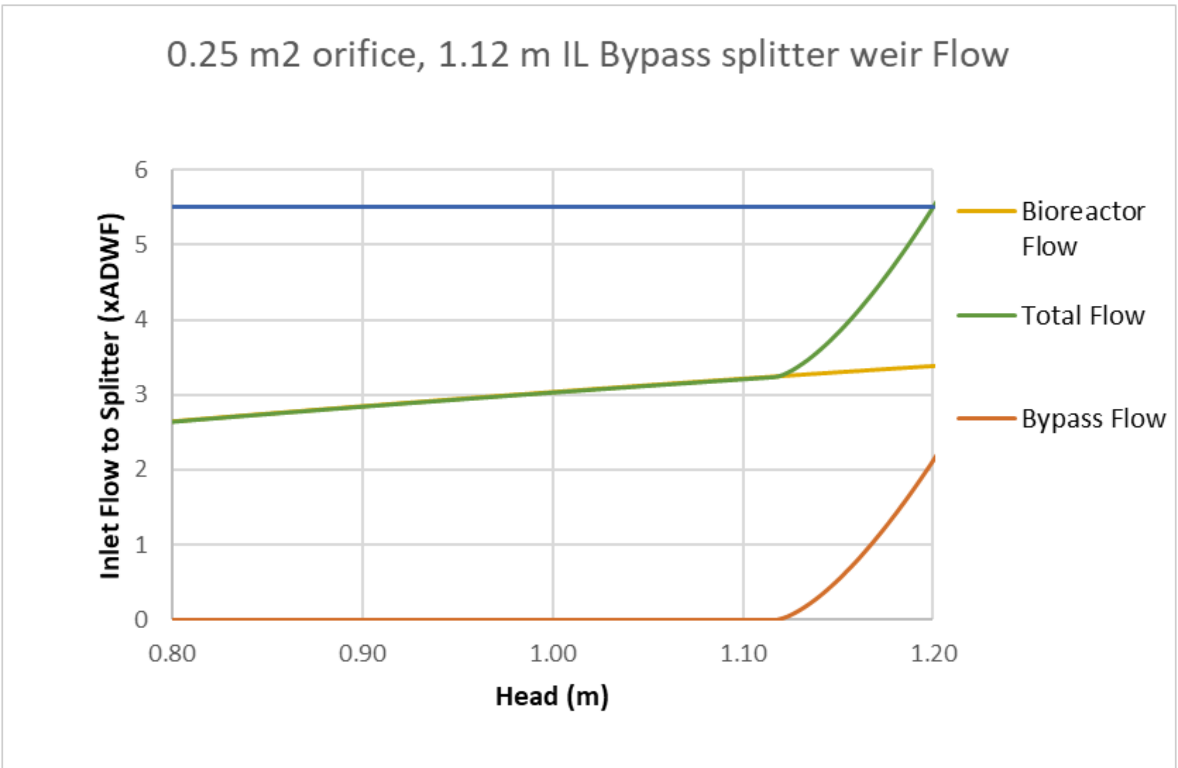


Figure 6-1 Flow Splitting Weir Flow Variation between the Bioreactor and Clarifiers for Stage 1

6.6 Bioreactor

6.6.1 Overview

An overview of the bioreactor is provided in Figure 6-2. Up to 3.4 ADWF (677 L/s) at 75,000 EP is fed to the bioreactor. Up to 2.1 ADWF (418 L/s) at 75,000 EP can bypass the bioreactor and recombine in the Mixed Liquor Splitter (MLS). As discussed in Section 4.1 combining flows at this point enables flocculation of bypassed material and sedimentation in the clarifiers. The clarifier RAS system returns part of the bypassed organics and nutrients for processing in the bioreactor.

The bioreactor consists of the following:

- Hydraulically mixed anaerobic zones to support Enhanced Biological Phosphorus Removal (EBPR)
- A splitter in the last anaerobic zone to divide flow to each oxidation ditch
- Three feed points for anaerobic effluent to enter the oxidation ditch. Movement of the feed point can increase or decrease the amount of oxidised nitrogen in the effluent.
- Two oxidation ditches with a series of 4 low speed high efficiency mixers per ditch which create a constant velocity of typically 0.3 m/s to keep activated sludge in suspension. Each ditch has a dedicated bank of diffuser grids to provide fine bubbles for aeration at the location shown in Figure 6-2.

- Two final aerobic zones to provide ammonia polishing and assist with phosphorus removal via the EBPR mechanism.

The bioreactor has been designed for all diffusers and mechanical equipment to be retrievable while it is online. Using this approach, it will be very unusual to take the bioreactor offline. However, the bioreactor has been configured to enable the anaerobic zone, each oxidation ditch and each final aerobic tank to be taken offline while other parts of the tankage are online. Options for structures to be taken offline include the following by positioning stop boards shown on Figure 6-2:

- Anaerobic 2 to 9 by shutting stop board 2 and opening stop boards 1 and 3
- An oxidation ditch (one at a time only) by shutting stop board either 4 or 5
- Both final aerobic zones by shutting stop boards 6 and 7 and opening 8 and 9.

Taking the anaerobic zones offline will likely lead to an increase in effluent phosphorus which will need to be managed by increasing the alum dose.

With the anaerobic zones offline the feed to each oxidation ditch will be sub-optimal. The carbon source for denitrification will be added much further down the anoxic zone and some increase in oxidised nitrogen may occur. However, no issue with compliance with the licence total nitrogen limit is expected. Nitrification (ammonia removal) will be not impacted.

Removal of the final aerobic zones may increase both the phosphorus and ammonia levels slightly. Both can be offset by moving the feed point to each ditch further downstream and changing the DO set point. This will increase the aerobic mass fraction which assists EBPR and nitrification.

Taking an oxidation ditch offline and emptying it is a major undertaking. Prior to being taken offline the MLSS will need to be lowered to under 2,000 mg/L by increasing wasting (this may take up to a month). Once the liquid contents are being removed from a ditch the aeration grids from it will need to be progressively moved to the online ditch. Positions and locators will be included to allow to move all grids from any ditch to the other. The extra aeration capacity will be needed in the online single oxidation ditch to manage the higher loadings.

An oxidation ditch should only be taken offline in the warmer period of the year. The autotroph bacteria responsible for nitrification are slow growers and are impacted by temperature and bioreactor SRT. The required SRT will need to drop to half the design value at 75,000 EP (~ 12.5 days) for one oxidation ditch operation. This can only be sustained in the hotter months when the bioreactor temperatures are above 19°C. The median ammonia from the clarifiers is likely to slightly rise to a value at or above 1 mgN/L median during these periods. At lower connection loads the oxidation ditch may be able to be taken offline at lower temperatures.

For the future upgrade an allowance has been made to hydraulically connect the Common Mixed Liquor Chamber at each oxidation ditch. This will ensure future clarifiers and bioreactors can share unit operations.

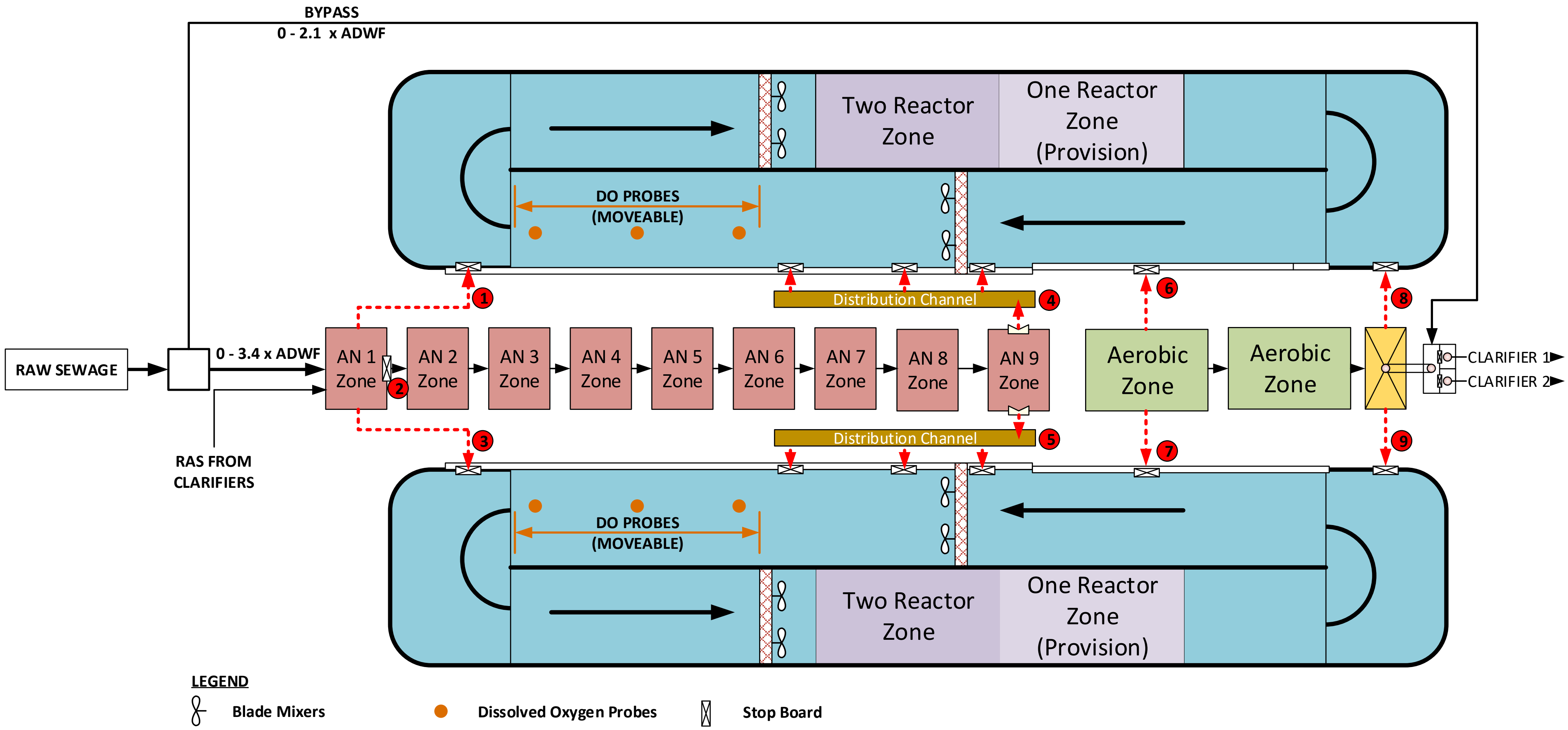


Figure 6-2 Bioreactor Configuration

6.6.2 Enhanced Biological Phosphorus Removal

In a review of options during the project defining stage it was agreed to configure the process for EBPR. To enable EBPR a series of anaerobic (no oxidised nitrogen or oxygen) zones are needed ahead of the process. This assists in the selection and fermentation of wastewater to enable Polyphosphate Accumulating Organisms (PAO) to grow. PAOs can store excess quantities of phosphorus internally with the cell structure as polyphosphate. The storage process occurs under both anoxic and aerobic conditions in the oxidation ditch and final aerobic zones. Wasting sludge from the final most aerobic zone effectively removes phosphorus from the plant. A detail description of the EBPR mechanism is provided in the QSTP Upgrade Project Options Selection Report, November 2019.

With EBPR, phosphorus is removed biologically and stored within the PAO cells as polyphosphate, however digestion processes can to some degree reverse this process. Digestion actively results in cell decay which will return some stored polyphosphate back to solution as phosphate in the digester. This will be returned via the centrifuge centrate to the head of the plant.

To counter the release of phosphorus in digestion, a range of phosphorus removal processes for the centrifuge centrate were considered in the options stage. The adopted approach for EBPR agreed at the Options Workshop on 24 July 2019 was to include anaerobic reactors ahead of the process. However, nutrient recovery in the form of calcium hydroxyapatite would not be included. Nutrient recovery is needed to ensure full biological phosphorus removal. The decision was made at the workshop that nutrient recovery would not be added in the upgrade, however it could be added at a later stage. Without nutrient recovery, limited biological phosphorus is likely (30 to 50% as opposed to up to 90%). EBPR is also heavily influenced by sewage quality characteristics, temperature, wet weather and equipment failure which can limit phosphorus removal even with the provision of centrate treatment.

With EBPR it is also good practice to return all recycle streams containing oxidised nitrogen downstream of the anaerobic zones. Oxidised nitrogen competes for the Short Chain Volatile Fatty Acids (SCVFA) required by PAOs to release phosphorus in the anaerobic zones. This can limit the growth of PAOs and reduce EBPR. Given the decision to allow not design for EBPR most recycles are directed to the inlet works. At a later stage, these flows could be intercepted and pumped separately to the oxidation ditch or final aerobic zones.

EBPR has a range of cost (i.e. lowers chemical dosing) environment benefits which include:

- Less TDS in the effluent given lower reliance on alum and caustic dosing
- Phosphorus removed remains in the phosphorus cycle and is not chemically locked up
- The biosolids has a higher phosphorus nutrient value
- A separate phosphorus resource (calcium hydroxyapatite) could be produced at a later stage by addition of a lime clarifier to the centrate stream.

Alum dosing has been allowed to ensure full chemical phosphorus removal (i.e. no EBPR at all) in the bioreactor in case there are issues with EBPR. For the alum dose rates allowed, a median of less than 0.7 mg/L is expected post the clarifiers. The filters are designed to polish the remaining phosphorus to meet the licence limits.

Under normal operation, phosphorus will be removed using biological phosphorus removal with chemical phosphorus removal with alum being used as a polishing step to meet concentration requirements.

6.6.3 Nitrogen Removal

The oxidation ditch is approximately 50% aerobic (i.e. some Dissolved Oxygen (DO) is present > ~ 0.2 mg/L) and 50% anoxic (zero to minimal DO). The anoxic portion of the ditch can be increased by moving the position the feed is introduced further downstream.

The DO is controlled at a point far downstream of the aeration grids and just prior to the feed. Using a controller, airflow is adjusted to the ditches grids in equal amounts based on a downstream DO measurement point. The DO setting is typically set low at 0.2 to 0.6 mg/L to ensure anoxic conditions form just after the DO probes and the feed is anoxic (see DO probe location above). Using this approach, the zone of aeration or portion of the ditch which is aerobic is controlled, however DO inside the aerobic zone is not necessarily controlled. This concept is illustrated in Figure 6-3. At high organic and TKN loads the peak DO is much higher for the same downstream DO then lower load periods. The reason for this is there is a higher oxygen uptake rate (i.e. DO slope after the last aeration grid) at higher loadings.

Nitrification (oxidisation of ammonia to nitrate) relies on aeration to occur and the rate is partially influenced by the DO within the aeration zone, with higher values increasing the rate. Therefore, the rate of nitrification is partly a function of the area under the DO curve in the aerobic portion, which is influenced by loading rate, not necessarily the DO at the control point. This control approach works well in oxidation ditches as the nitrification rate is slightly increased at higher loading rates (i.e. DO increases to cope with the higher oxygen uptake rate with higher loads refer Figure 6-3).

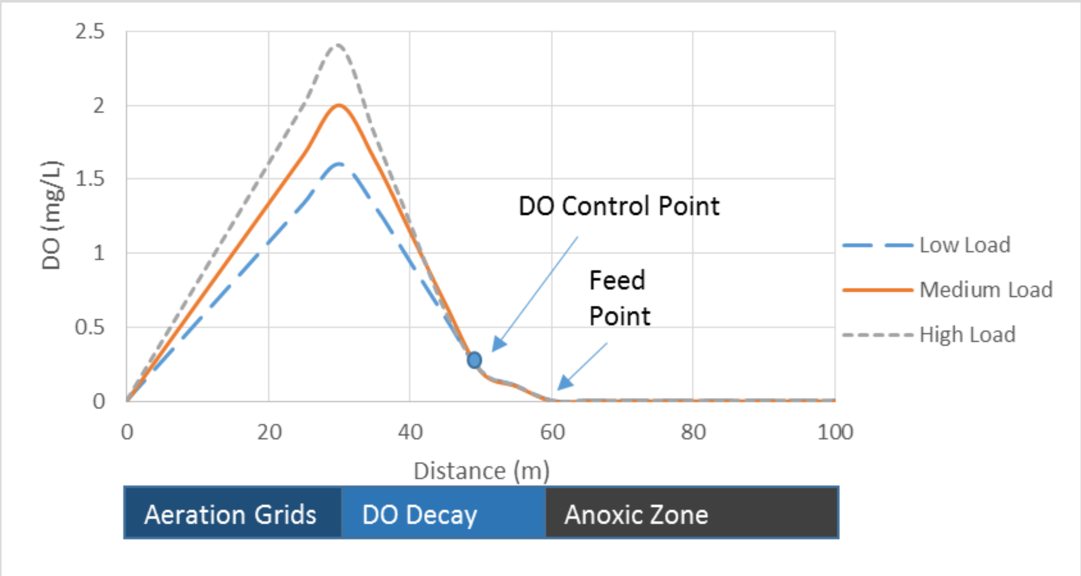


Figure 6-3 Illustration of Typical DO Profile in Each Oxidation Ditch

The nitrification rate can also be adjusted by the following operational measures:

- Modifying the channel velocity by adjusting mixer speed settings. For example, a lower speed will reduce the liquid volume rate the air is diffused in which increases the peak DO (area under the curve).
- Modifying diffuser banks which are online. At commission not all diffuser banks are needed and shutting down a few downstream increases the peak DO and nitrification rate.
- Moving the feed point. For example, moving it downstream increases the aerobic mass fraction in the whole process and enhances the nitrification rate. This can also reduce denitrification.
- Modify DO at the control point. For example, a rise in the DO setpoint will increase the entire amount of DO in the ditch which improves the nitrification rate.
- Operate the final aerobic zones at different DO set points. The last aerobic zones can be used to lower the ammonia further. Increasing the DO set point will lower ammonia levels.

- Adjusting SRT. Increasing SRT increases the nitrification and to a lesser extent the denitrification rate.

It is important to note that denitrification (i.e. oxidised nitrogen conversion to nitrogen gas) only can occur in the anoxic portion (i.e. downstream of the feed and just prior to the aeration grids) of the oxidation ditch. Generally, the controls above that adjust ammonia will work in the opposite direction for oxidised nitrogen.

If required the oxidised nitrogen levels can be increased in the plant by a combination of factors which include, moving the feed downstream, high DO settings and high channel velocities. It is understood this may have some environmental benefits in the receiving environment. However, attempting to actively target high oxidised nitrogen can have the following consequences:

- High plant power use. More power is needed to transfer oxygen to the process at a higher DO and more oxygen is used to oxidise more BOD aerobically. An oxidised nitrogen increase from 2 (typical for oxidation ditch processes) to 10 mg/L can increase power costs by up to 15 to 20%.
- Elevated oxidised nitrogen can cause denitrification in the sludge blanket of the clarifier, nitrogen gas formation and flotation of sludge. This can lead to elevated effluent TSS and impact on filter operation (low run times).

As part of optimisation of this process further consideration will be given to the use of Oxidation Reduction Potential (ORP) probes as part of commissioning and ongoing monitoring. It is not recommended that the median total oxidised nitrogen post the clarifier be any higher than 8 mgN/L as a median to avoid clarifier issues.

6.6.4 Scum Harvester

For biological foams or scum to form a combination of filamentous bacteria and fats and oils are needed. As well as enabling EBPR the anaerobic zones act as a high Food to Microorganism (F/M) selector. High F/M conditions encourage the growth of floc forming organisms. This can limit the growth of filamentous bacteria which can lead to bulking (poor sludge settling), clarifier capacity issues and scum formation.

Selectors cannot always prevent formation of scum, therefore, a harvester has been allowed for in each oxidation ditch. They will be installed just downstream of the aeration grids where most of the biological foam will form if generated. It will include a plate across the ditch that directs scum to a flight scraper system. Collected scum will be pumped to either the WAS Thickener or digester.

Scum if present tends to move through the oxidation ditch quickly and enter the scum harvester. However, reverse flow currents occur on the surface near the mixers which provide the channel velocity. These typically act to trap scum. To ensure scum to pass these zones the mixers will have a programmed pause in mixer operation for a few minutes each hour. This is all that is needed to move the scum forward to the harvester. This can be timed to occur with the scum harvester operation.

6.6.5 Bioreactor Design Summary

A summary of the bioreactor design is provided in Table 6-4.

Table 6-4 Bioreactor Design Summary

Item	Value	Units	Notes
Overall Bioreactor			
Temperature range	12 to 24	°C	Based on monitoring range of existing reactor

Item	Value	Units	Notes
Design average SRT	25 22	days days	Septage directed to digester Septage directed to plant
Maximum SRT	12.5	days	One oxidation ditch offline at 75,000 EP
Minimum SRT (limited by WAS thickener)	11 6.3	days days	All bioreactor components online One oxidation ditch online
MLSS (Design load)	4,000	mg/L	When operating at design SRT. Note different SRT depending on where septic waste is fed.
Alum Dose	3	mol AL/mol P	Molar dose allowance per mole of P removed assuming no biological P removal occurring.
Anaerobic Zones			
Number	9	cells	
Volume	278	m^3/cell	Cells are hydraulically mixed 10 % biomass mass fraction
Oxidation Ditches			
Number	2		
Volume	10,625	m^3/ditch	85% biomass mass fraction
Mixers	4		Duty / duty / duty / standby
Maximum channel velocity	0.4	m/s	Rate can be varied from 0.2 to 0.4 m/s
Scum harvesters	1	per ditch	Scraper and chain style
Final Aerobic			
Number	2	cells	For ammonia polishing
Volume	625	m^3/cell	5% biomass mass fraction
Aeration			
Type			Fine bubble diffused aeration with common blower group suppling air to entire bioreactor Centrifugal for enhanced efficiency
Number of blowers	4		Duty / assist / assist / standby
Capacity per blower	5,000	Nm^3/h	Subject to actual diffuser selection
Required blower minimum	2,500	Nm^3/h	Subject to actual diffuser selection
Design fouled alpha factor	0.55		With type 1 and II fouling at diffuser life with peak MLSS.
Expected clean alpha factor	0.7		
Adopted beta factor	0.95		

Item	Value	Units	Notes
Oxidation ditch DO (aerobic zone)	1-2	mg/L	Varies depending on plant load and circulation velocity.
Oxidation ditch maximum SOTR design (fouled)	550	kg/h/ditch	At peak diurnal and design load
Oxidation ditch SOTR minimum Commissioning (clean)	90	kg/h/ditch	At minimum diurnal and commissioning load
Final Aerobic DO	3	mg/L	Maximum vale can be adjusted.
Final Aerobic SOTR maximum (fouled)	60	kg/h/zone	At peak diurnal and design load
Final Aerobic SOTR minimum (fouled)	13	kg/h/zone	At minimum diurnal and commissioning load

6.7 Clarifiers

The clarifier system has been designed with two clarifiers which can accept flows up to 5.5 ADWF at 75,000 EP while operating in solids contact configuration (3.4 ADWF through bioreactor and 2.1 ADWF bypass).

Prior to the clarifiers a splitting structure blends and mixes bioreactor and solids contact bypass flow and splits them using equal length weirs. This splitting structure has been designed with hydraulic capacity for stage 2 flows and a third weir is included in the structure for a future clarifier.

As a clarifier has revolving submerged equipment there is a minor risk that a clarifier will fail and require shut down. At a minimum it is important that the plant can treat the Peak Dry Weather Flow (PDWF) in the event of one clarifier failing. This requirement to treat PDWF with one clarifier out of service was adopted for this design.

Effective removal of settled sludge from the clarifier base is important for the following reasons:

- Sludge accumulation can reduce the aerobic mass fraction leading to issues with nitrification (ammonia removal).
- A failure to remove sludge can result in sludge accumulation which can negatively impact on EBPR if anaerobic conditions form in the blanket. This can lead to an effect known as secondary release where stored phosphorus is released by PAOs under anaerobic conditions.
- If sludge accumulates in the clarifier anoxic conditions can be readily established in the sludge blanket. If issues with denitrification are being experienced (i.e. elevated effluent nitrate) it is possible if a sludge blanket is present in the clarifiers that denitrification could occur resulting in the release of nitrogen gas. If the gas build-up is significant rafting (or flotation) of clumps of sludge can occur resulting in poor suspended solids effluent quality performance.

To prevent the above from happening, as well as a good mass flux design with adequate clarifier area and RAS capacity, it is important that an effective sludge scraper is used. To ensure this a full radius plus a 1/3rd radius log spiral scraper has been specified with adequate blade height to ensure removal of settled sludge in one revolution of the scraper arm.

The EPA limits are challenging for phosphorus. In these instances, good solids removal and solids capture is necessary. Therefore, a clarifier design with the following features which assist in solids removal have been provided:

- An energy dissipating inlet and flocculation well. This is a design feature which allows time for flocs which may have been sheared in transport to the clarifier to reform and flocculate prior to entry into the clarifier settlement zone. This particularly is helpful in the flocculation of the bypass stream around the bioreactor when it is occurring.
- A Stamford Baffle. This is a peripheral baffle which redirects the flow away from the weir. This avoids the “chimney effect” or region of high velocity near the clarifier wall and creates a lower velocity region conducive to low solids levels near the weir.
- An adequate sidewall depth. There is a relationship between effluent quality and clarifier sidewall depth with greater depths leading to improved quality. The selected clarifier depth is in line with the recommendations in IWA Secondary Settling Tank Scientific & Technical Report No. 6 (1997) and Hunter H2O experience to achieve optimal effluent suspended solids.

The settling characteristics of the biomass generated in the bioreactor have a significant influence on the clarifier capacity. Filamentous bacteria, which are bacteria which upon cell division do not separate and form long strings, if they proliferate, they can bulk up the sludge which effectively reduces the settling rates. The Diluted Sludge Volume Index (DSVI) has units of mL/g and is a measure of bulking. It is indicative of the volume occupied by one gram of settled sludge after 30 minutes.

For a flow and bioreactor MLSS concentration, the DSVI will limit the solids flux ($kg/m^2/h$) and determine the clarifier area. At DSVI values above the peak value at design load (75,000 EP) the clarifier will fail to thicken and sludge storage is likely to occur in the clarifier. For the reasons noted above this is not desirable and can impact effluent quality.

Figure 6-4 shows expected DSVI trends based on Hunter H2O’s operational database of DSVI results for similar design plants to QSTP. The figure plots DSVI versus percentile of occurrence based on long run plant data. DSVI trends are for EBPR (i.e. biological phosphorus removal only), EBPR with a chemical trim of alum (~30% of the full CPR figure) and full Chemical Phosphorus Removal (CPR) . QSTP will mostly operate in the latter two categories. The clarifiers have been sized using a DSVI of 145 mL/g which is within the upper percentile of what is expected to be experienced for this style of plant. At this upper DSVI one clarifier can tolerate 2 ADWF at 75,000 EP (399 L/s) which is just above PDWF. Effectively the process can operate for all dry weather flow regimes on one clarifier at 75,000 with no storage of sludge in the clarifier.

The anaerobic zones will assist in improving the DSVI. Also, alum dosing ballasts the sludge which can improve the DSVI.

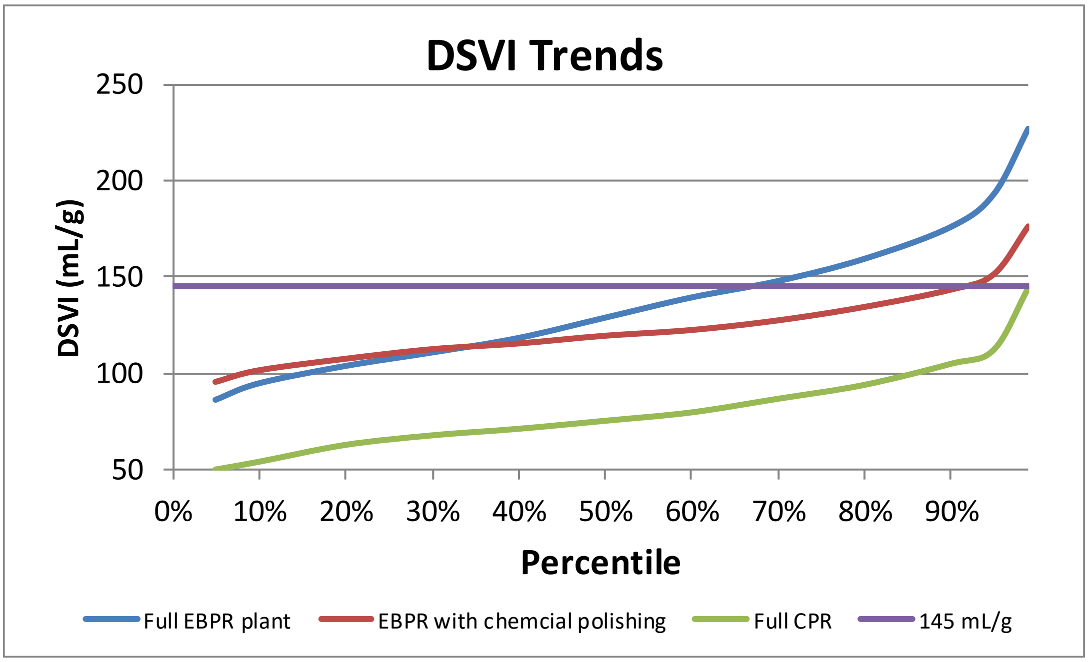


Figure 6-4 DSVI Trends for Different Plant Types

A summary of the clarifier design is provided in Table 6-5.

Table 6-5 Clarifier Design Summary

Item	Value	Units	Notes
Clarifiers			
Number	2		
Diameter	40	m	
Sidewall depth	4	m	Depth to weir
Energy dissipating inlet (EDI) diameter	3.8	m	Device to create a swirl path in the flocculation chamber. 1.0 m depth.
Flocculation Chamber Diameter Flocculation Chamber Depth	12.4 2.0	m m	
Stamford Baffle Width	1.24	m	Projecting at 53.4 degree angle . Vertically 1.37 m below the weir.
Scraper system			1 radius and 1/3 30 degree log spiral
Peak Mass Flux	7.2 7.9	$kg/m^2/h$ $kg/m^2/h$	At 3.4 ADWF and peak RAS (2 clarifiers) At 2 ADWF and peak RAS (1 clarifier)
Overflow Rate	1.57 0.97 1.14	m/h m/h m/h	At Peak flow 5.5 ADWF (549 L/s/clarifier) At Peak flow 3.4 ADWF (339 L/s/clarifier) At 2 ADWF with one clarifier (399 L/s)
Maximum DSVI at peak solids flux (one clarifier operation)	145	mL/g	Equates to Veslind values of: • V_o 7.6 m/h • n 0.47 m3/kg
One Clarifier peak flow at maximum DSVI	2	ADWF	399 L/s, limited by mass flux
RAS PS			
Number pumps per clarifier	TBA		Duty / assist / standby Pump selection will confirm the number of pumps required for the required turndown.
Pump capacity	152.5	L/s/pump	3.05 ADWF in total at 75,000 EP with two pumps operating on both clarifiers
Scum Removal			
Number of launder troughs	2	/ clarifier	Flooded beach style mechanically actuated
Trough volume	150	L	

6.8 Dissolved Air Flotation Filtration

Dissolved Air Flotation (DAF) followed by dual coal / sand granular media filters known as a Dissolved Air Flotation Filter (DAFF) are installed after the clarifiers as a tertiary treatment stage to provide fine phosphorus polishing and solids removal. The DAFF is not on the hydraulic grade line. Flows are pumped to the DAFF by the Filter Lift Pumping Station. If the DAFF is unavailable flow will overflow a weir to UV disinfection and discharge.

The DAFF have been designed for a net capacity (i.e. after allowance for backwash) of 3 ADWF at 75,000 EP (599 L/s).

6.8.1 Filter Lift Pumping Station

The Filter Lift Pumping Station has been sized to ensure the flow increases by no more than 25% per 30-minute period during typically dry weather operation. This has been managed by providing enough control volume in the Filter Lift Pumping Station to achieve this. Sudden flow increases as a result of ramp up in the diurnal profile can lead to some solids loss.

Flow balancing is more of a concern for 50%ile TP limit compliance, therefore the focus on dry weather operation. In wet weather, removal will still occur, and the upper percentile limits met over the licence annual period.

The pumps have been sized with a total capacity of 725 L/s to allow for operation of the backwash system which returns 125 L/s at peak capacity and ensures a net filter water capacity of 599 L/s is produced (3 ADWF at 75,000 EP).

For stage 2 additional pumps can be installed in this station or the existing pumps replaced, however the flow increase per 30 minute period will increase. To address this if it is an issue based on historic DAFF operation, additional balance volume could also added by the addition of another hydraulically connected tank.

Table 6-6 Filter Lift Pumping Station Design Summary

Item	Value	Units	Notes
Control volume	1,000	m^3	Volume enable flow increase to be limited to 25 %/30 min in peak diurnal conditions
Number of pumps	3		Duty / assist / standby
Pump capacity	363	L/s/pump	
Minimum pump flow	50	L/s/pump	

6.8.2 Dissolved Air Flotation Filtration (DAFF)

Both alum and caustic are dosed prior to a wafer style non clog static mixer which feeds flow into two flocculation tanks. Caustic dosing has been added at this point to enable the adjustment of the pH if required to the ideal value (i.e. a pH of 6.8) for the lowest dissolved phosphate residual.

Slow speed adjustable mixers are provided in each flocculation tank to allow growth and optimum size flocs to form for flotation and filtration. If needed directly after flocculation a filter aid non-ionic polymer can be dosed. This acts to make the filter media more amenable to capture flocs.

Filtered water will be pressured to up to 600 kPa and compressed air injected via a saturation vessel to dissolved air in the water. This flow (on average 10% of the inflow) will be directed to the inlet of each cell where a series of globe valves will release all the pressure. The sudden pressure release will cause micro bubble formation and the floc will be floated to the surface of the cell via a chimney. A series of sprays will direct the float periodically to a wash out trough. The separated water will flow down through the filtration stage.

A dual coarse coal (1.4 mm) and fine sand (0.6 mm) filter has been selected. The coal media is coarser than typical (1.1 mm) to improve the filters ability to capture and store solids associated with alum precipitation and solids existing the clarifier.

During filtration the level in the filter will be controlled by modulating the filter outlet valve to achieve a target level. The level in each filter is measured by a filter level element.

The head loss across the filter media will be measured and used as well as time to trigger a filter backwash. A filter head loss of up to 2.75 m has been allowed for. Typically, up to 2.5 m is observed at the floc loading rates and peak flow allowed for in this design.

The Clear Water Tank has been sized to ensure there is adequate water supply at the peak expected backwash rate (55 m/h) and time (10 minutes).

Expansion or fluidisation of the bed by up to 25% during backwash has been allowed for. The backwash rate has been designed to be variable to account for different expansions which occur as water viscosity changes with temperature. The highest backwash rate is needed for the highest summer temperature where the water viscosity is the lowest.

The backwash includes initial air scour prior to clear water backwash. Dedicated air scour blowers have been provided for the filters.

DAF on filter (DAFF) is a method used in conjunction with alum and polymer addition to achieve a better phosphorous removal efficiency. A jar testing regime was used to asses straight filtration and DAFF operation during the period of the 14th to the 21st of July 2020. The DAF component will involve the following equipment and methods to achieve the lower phosphorous levels:

- Recirculation flow and pump, a saturator (operating up to 600 kPa), compressed air injection into the saturator, pressure reduction and dispersion in the flocculated feed
- A downflow and up flow chimney section in the filter structure ahead of the filter to accept the saturator flow and disperse it into the flocculated feed
- Installation of a DAF float removal system and sprays which would direct DAF float to the dirty backwash system.

For the DAF float periodically the filter outlet control valve will increase the level to a point where surface water will flow over a weir at the opposite end of the inlet. This will direct float assisted with sprays over the weir to the DBT. Once the surface is clear the level will lower below the float outlet weir. At all times DAF and filtration operation will continue. In a filtration cycle float removal may occur many times before a backwash of the filter is needed.

The pneumatic compressed air compressor which runs the whole DAFF system including control valves will supply air to the DAF saturator. The small amount of air required for this system will be included in this compressor package.

Table 6-7 Filter Design Summary

Item	Value	Units	Notes
Flocculation			
Flash mixer	1		In pipe wafer non clog static type
Number flocculators	2		
Flocculator volume	220	m^3/flocculator	Sized for > 15 mins at PDWF. 11 minutes actual at peak flow

Item	Value	Units	Notes
DAF Part of DAFF			Common system for the 5 filter cells
Number of cells			DAF inlet chimney, filter and float overflow
Recycle ratio	10	%	of inflow.
Saturator pressure	600	kPa	Maximum. Can operate 400 to 600 kPa
Saturator diameter	1,500	mm	
Saturator height	3,750	mm	
Hydraulic Residence Time (HRT)	60	s	
Drop downflow zone cross sectional area	8.7	m^2/cell	After inlet weir split and prior to chimney
Chimney reaction zone area	12.0	m^2/cell	
Chimney reaction zone rise rate	60	m/h	
Float zone area (Filter)	59.85	m^2	Same as area of filter
Number of pressure reduction valves	4	per cell	Connected to two up turned bullhorns outlets.
Number of Bullhorns	8	per filter cell	
Diameter of Bullhorns	0.06	m	
Filtration (Dual Media) – Lower Section of DAFF			
Filter area	57.85	m^2	
Filter media depth	0.7 0.5	m coal m sand	1.4 mm effective size, 500 L/d 0.6 mm effective size, 830 L/d
Peak flow	181	L/s/filter	With one filter offline for backwash
Peak filtration rate	12.4	m/h	With one filter offline for backwash
Peak filtration rate	9.9	m/h	All filters online.
Peak alum dose (DAF offline)	41	mg/L	As supplied (4.24 % Al) Equivalent to 5.5 mol Al/ mol P with 0.4 mgP/L inlet
Peak alum dose (DAF online)	104	mg/L	As supplied (4.24 % Al) Equivalent to 14 mol Al/ mol P with 0.4 mgP/L inlet
Peak clarifier filter solids	15	mg/L	< 5 mg/L typical for dry weather
Floc storage ability	1	kg/m^3	Before head loss required backwash
Peak Solids Load (DAF Offline)	54.6	kg/h	Combined clarifier and alum sludge
Peak Solids Load (DAF Online)	14.9	kg/h	Combined clarifier and alum sludge
Minimum filter run time (DAF offline)	6.4	hours	Worst solids and alum concentration at peak flow and DAF Offline.

Item	Value	Units	Notes
Minimum filter run time (DAF online)	23.2	hours	Worst solids and alum concentration at peak flow and DAF Offline.
Typical filter run time with (DAF offline) (at 5 mg/L clarifier TSS, 41 mg/L alum)	16 >24	hours hours	At PDWF (440 L/s) sustained At ADWF
Alum dose pump	208	L/h	Duty / standby pumps
Caustic Dosing	150	L/h	Duty / standby pumps
Filter aid (non-ionic polymer)	40	L/h	0.05 mg/L maximum dose rate
Clear Water Tank & Backwash			
Tank size	510	m^3	Sized for required net drawn down during longest peak backwash time and no inlet flow.
Backwash rate fast	55	m/h	890 L/s per filter
Backwash rate slow	16	m/h	256 L/s per filter
Backwash cycle(maximum)			17 min drain down, 5 min air scour, 2 min air scour slow backwash, 1 min slow backwash, 10 min fast backwash and 1 min slow backwash.
Backwash troughs per filter	4		
Number back wash pumps	2		
Pump flow range	250 890	L/s L/s	Minimum (slow backwash) Maximum
Air scour rate	55	m/h	3,180 Nm3/h
Number of Air Scour Blowers	2		Duty / standby
Air Scour Capacity	3,300	$Nm^3/h/$ blower	3,180 at peak. Some below contingency included.

6.8.3 Dirty Backwash Tank and Return System

The dirty backwash system has been sized to accept the high instantaneous flow during a backwash, balance it and return it at a lower controlled rate to limit the impact on the secondary clarifiers and ensure a net flow of 3 ADWF at 75,000 EP (599 L/s) leaves the filters with one filter offline for backwash. A summary of the design is provided in Table 6-8.

In peak flow (599 L/s) and solids load conditions, the dirty backwash return pump will be limited to 125 L/s and 510 m^3 of volume has been included to enable one backwash to return over a 1.31 hour period with all filters progressively washed over 6.6 hours. This total time equals the worst-case excepted run time of a filter with the DAF offline at peak flow. Provided the filters are operated in a staggered backwash configuration prior to a peak flow event (i.e. backwashes evenly spaced) there will not be a capacity issue associated with the backwash system with the DAF offline. However, it is important operationally to ensure the filters are evenly spaced in terms of backwash operation during normal operation to prevent a filter stacking event which can limit capacity.

A filter stacking event occurs if filter backwashes are not spaced out and are all operational backwashed in low flow conditions at very similar time (e.g. over 4 hours). This would mean at peak flow conditions they all call for backwash (based on head loss trigger) in a short period based on them having similar run times.

In DAF operation under peak flow conditions the filter run time is much longer at 23 hours and the backwash can be returned at a slow rate and less frequently.

A higher capacity dirty backwash return pump rate of 200 L/s has been included to resolve an issue quickly in low flow periods where they may be stacked up due to say maintenance activity. At 200 L/s all filters can be promptly backwashed in 0.8 hours each. This may be needed to rapidly provide capacity for dry weather operation if the DAF is offline. Then the filter run time controller will be used to ensure all filters are staggered.

It is possible to design a system to rapidly clear backwashes in peak flow conditions, however this approach will lead to the following cost impacts on the design:

- Larger dirty backwash return pump capacity, for example clearing all filters backwashes in 3 hours increases the dirty backwash capacity to 260 L/s
- Greater clarifier area to handle the high return flow. This can increase the area required by 26% for a 3 hour clearing cycle.
- Hydraulic grade line impact. Larger flows through the clarifiers increases the head difference between the bioreactor and clarifier as pipe velocity cannot be lowered given the need to maintain a minimum dry weather velocity for process reasons (refer Section 5.2). This can make it difficult to site the reactors and clarifiers and increase the RAS and lift pumping costs.

The approach above where the backwash system is minimised for peak flows is considered acceptable provided adequate controls are included in the plant control system to ensure staggering of filter backwashes. This will be addressed in the development of the control system functional specification.

The backwash solids consist of a combination of treated biological flocs not captured in the filters and alum sludge. The combined material settles readily, and it is proposed to be directed to the clarifiers directly via the Mixed Liquor Splitter. Adding this flow to the Mixed Liquor Splitter reduces the mass flux on the clarifiers and is a better location in terms of maximising clarifier capacity. However, an option to add this flow to the bioreactor has been included. This location provides a minor benefit as more time for flocculation prior to clarification is provided.

Table 6-8 Dirty Backwash System Design Summary

Item	Value	Units	Notes
Control volume	510	m^3	Volume form BWL to TWL
Number of pumps	2		Duty / standby
Pump capacity	200	L/s/pump	Used for dry weather operation.
Pump minimum flow	125	L/s/pump	Sized to limit return flow to ensure 3 ADWF can be met in peak conditions
Time backwash all filters	8.6 5.4	Hours hours	at 125 L/s dirty backwash return flow at 200 L/s dirty backwash return flow

6.9 Ultraviolet Disinfection

A channel Ultraviolet (UV) low pressure lamp system is proposed to disinfect the effluent. Sizing is proposed to achieve Faecal Coliform (FC) reduction only to meet the licence discharge effluent quality limits. A summary of the design is provided in Table 6-9.

A proprietary system is shown in the design based on discussions with a vendor, however other equivalent and similar systems are available from other suppliers which are acceptable. To optimise power use and control UV dose an ability to shut down banks and dim light output is recommended for any system installed. The proprietary design includes a high degree of power turndown with two channel 8 UV bank configuration. Minimum turndown is 3 banks at 50% output and 8 at 100%. Above the 3 minimum each bank comes on progressively one at a time. Other systems may provide equivalent or less turndown. However, it is recommended at least the same overall UV dose turndown is provided to ensure power is not wasted in normal dry weather flows at commissioning.

The sizing of the UV system is influenced by the worst-case solids, feed FC and UVT. Refer to Section 4.3 for a discussion on the basis for the UVT value adopted. The values adopted have been selected assuming a loss of filtration and disinfection of clarified only effluent in peak flow conditions.

Disinfection of only the flows that are lifted to the treatment plant (i.e. 5.5 ADWF) occur. Flow which may overflow from the Storm Pond in rare events (1 in 10 year). There is insufficient hydraulic grade to pass these storm pond overflows through the UV system. This storm pond merges downstream of the UV and discharges at the on-bank discharge structure. Overflows of this nature are likely to be accompanied by high river flow events providing substantial dilution.

The system is fully automated with power automatically adjusted based on measured intensity in each channel. The level is controlled in the channel by modulating an outlet penstock in reference to a level element in each channel. The vendor uses the level over the penstock to measure flow for use in UV dose calculations. This flow is available for display on the plants SCADA system for operational use. However, it is typically not recommended to be used for licence reporting. Only as a backup flow meter if needed for flow discharge reporting purposes.

To provide enhanced turndown and minimise power use only one channel will operate at lower flows (i.e. under 2.75 ADWF). The other channel will be maintained as standby by rising the outlet weir to the closed position. Regular rotation of channels and banks will occur to ensure even use of lamps.

The UV channel has been designed with hydraulic capacity and space for the additional UV lamps required for stage 2. Fillets will be installed in the UV channel and replaced during Stage 2 with the UV lamps required to increase the UV lamp bank size.

Table 6-9 UV Disinfection System Design Summary

Item	Value	Units	Notes
Number of Channels	2		
Banks per Channel	8		28 lamps per bank equivalent to Wedeco Duron system
UV Dose Turndown	5.3 10.6	to 1 to 1	Per channel Overall system both channels
Capacity per Channel	706	L/s/channel	Allows for 5.5 ADWF over 2 channels plus rainfall flow on upstream catchments 3.5 ADWF capacity per channel
Feed quality basis:			
TSS	15	mg/L	Median (30 mg/L max)
Faecal coliforms	10^6	mg/L	Maximum
UVT	50	%	Minimum
Cleaning			Automatic wiper
Level Control			Outlet adjustable weir

6.10 Sludge Handling

Sludge is wasted continually from the bioreactor to a gravity sludge thickener. A picket fence style thickener produces a concentrated sludge which is continually pumped to the Aerobic Digester. A three cell aerobic digester stabilises the sludge and includes balance storage for periodic operation of dewatering. Sludge is drawn from the last digester cell and is dewatered using two centrifuges. The dewatered biosolids is discharged to two pivoting conveyors which can load a semi-trailer truck body.

6.10.1 WAS Thickening

A summary of the WAS thickening design is provided Table 6-10.

The WAS pumps are sized so the bioreactor cannot operate any lower than an 11-day SRT when all components are online. The limitation is set by the WAS and thickener capacity chosen. The 11-day SRT was chosen so the whole system can be wasted down to a sludge inventory level to enable one oxidation ditch operation and not exceed the clarifier settling capacity (refer Section 6.6.1). With one oxidation ditch offline the lowest SRT possible is 6.3 days (i.e. lower than 11 days as the total bioreactor volume has reduced but the WAS pump minimum flow is capped).

In operation some slight adjustment of the SRT is not unusual in the range 18 to 25 days. Adjustments can be used to manage poor settling, scum formation, ammonia and biosolids stability.

The WAS pumps have been sized to enable operation continually up to a 25-day SRT. SRTs above this can be achieved below 75,000 EP or if the DSVI is better than the design value (refer Section 6.7). If a SRT above 25 days is required, and the pump minimum flow, pulse operation of the WAS pump will occur to enable higher SRTs to be achieved if possible. There will be guidance provided in the plant operating manual of the possible peak SRT versus connected EP. This is to avoid building up too much sludge inventory in the bioreactor and overloading the clarifiers.

A single gravity thickener will be provided. Given the simplicity of the thickener there is low risk of failure. Where there is a risk duty/standby pumps have been provided. It is recommended a spare scraper centre drive and gear box parts be held on site for this single drive.

A bypass of the thickener has been included for periods where the thickener may be out of service. In an extreme outage (many weeks) the digester will thin down and the centrifuges will need to operate more often. This worst-case scenario is discussed in the dewatering section. In this scenario Grade B stabilisation via the Specific Oxygen Uptake Rate (SOUR) method is unlikely to be met. However, sludge will be able to be removed from the plant to ensure effluent quality is maintained. In this situation the biosolids would need to be classified as Grade C stabilisation and not be suitable for reuse.

The thickener is likely to achieve the target thickness for the digester without the need for polymer dosing. However, in instances where the is DSVI is poor (i.e. filamentous sludge) the target thickness may not be met without the aid of a polymer. A polymer dosing system has been provided which will use the dewatering system polymer to ensure the digester target thickness is met.

The sludge thickener pumps will operate to achieve a target solids concertation based on operator set bioreactor solids levels. In most cases the provided pump capacity range will be enough for continuous slow speed operation of the sludge pumps. However, if a lower rate is required the pumps will pulse on a cycle to achieve the flow.

Table 6-10 WAS Pumping and Thickening Design Summary

Item	Value	Units	Notes
Design minimum SRT	11	days	All bioreactors components online.
Maximum SRT for Continuous operation	25	days	For longer SRTs if needed periodic WAS operation will occur
WAS Pumps			
Number WAS Pumps	2		Duty / standby
WAS Pump Range	7.4-28.2	L/s/pump	
Thickener			
Number of	1		
Diameter	15.5	m	3 m sidewall depth, 1/6 floor slope
Peak Solids Load	1.2	$kg/m^2/h$	At minimum SRT
	1.0	$kg/m^2/h$	At maximum SRT
Overflow rate	0.53	m/h	At minimum SRT
	0.22	m/h	At maximum SRT
Sludge Thickener Pump			
Number Pumps	2		Duty / standby
Design Solids Range	0.8-1.25	%	
Pump Range	4-8	L/s/pump	Capacity to handle lowest %solids at highest expected sludge production
Polymer Dosing Pump			
Number Pumps	2		Duty / standby

Item	Value	Units	Notes
Design Solids Range	0.8-1.25	%	
Pump Range	4-8	L/s/pump	Capacity to handle lowest % solids at highest expected sludge production

6.10.2 Aerobic Digestion

An overview of the aerobic digester design is provided in Table 6-11.

Sludge from the thickener is continually fed to the first cell of the digester. It flows through each cell in series with sludge extracted from the final cell for dewatering. Essentially aerobic and anoxic processes are used in the digester to break down the sludge. Decaying microorganisms release organics which are oxidised biologically. The net effect is a loss of sludge mass and its conversion to carbon dioxide. Some nitrogen removal also occurs with nitrogen gas produced.

Removable aeration grids are installed in each cell. The number of diffusers in each cell reduces as it passes to the final cell. As the sludge stabilises less aeration is needed.

Aeration is provided using an aeration sequence. Typically, Cell 1 will aerate for 120 minutes and cease for 120 minutes. During the aeration off phase of Cell 1 both Cell 2 and 3 will receive air and have an equivalent aeration off phase. Essentially one set of blowers will sequence air back and forth aerating Cell 1 followed by Cell 2 and 3.

With a three-cell digester system the aeration demand for Cell 1 roughly equals the combined demand for Cells 2 and 3. Therefore, a three-cell system cycling the aeration allows the blower size to be minimised.

Aeration on and off cycling is important from a digester stability perspective. During the aeration phase bacteria cells decay and release ammonia which is nitrified to nitrate (i.e. nitrification). As nitrification is an acidic process the pH will drop as aeration proceeds. If it drops too low this can inhibit the biological processes. By providing 50% of the total aeration on / off cycle as anoxic. Slow denitrification (i.e. nitrate removal and nitrogen gas formation) occurs in the aeration off phase which can completely recover the alkalinity lost by nitrification and recover the pH. The reason for the long off period is due to the slow release of COD by decaying cells which is required for denitrification. This process achieves some nitrogen removal which reduces the nitrogen load returned to the bioreactor via the dewatering centrate.

For EBPR too long an aeration off phase can be an issue and lead to phosphorus release in the digester. This can increase the mass of phosphorus returned in the dewatering centrate and lower EBPR in the treatment process. To manage this the aeration on and off time can be adjusted to ensure anaerobic conditions do not form. The aim is to only have enough off time to use up the nitrate and return to aeration to ensure only anoxic conditions occur in the digester.

Adjustments to the aeration cycle are used to limit phosphorus release, however release cannot be prevented. With digestion, the PAO bacteria response for EBPR will die off and release phosphorus. However, the PAO decay rate is much slower than ordinary bacteria which make up the bulk of the sludge mass. Therefore, full release of phosphorus is unlikely to occur and some overall EBPR in the plant occurs. Refer to Section 6.6.2 for further discussion on this aspect.

An automated valve has been included between Cells 1 and 2. The valve opens during the aeration phase of Cell 1 to ensure sludge migrates to Cell 2. Prior to the end of the aeration phase the valve will close and stay closed during aeration of Cells 2 and 3. If all cells are hydraulically interconnected flow moves back and forth amongst the cells as the aeration starts and stops due to level changes when aeration occurs. This movement of sludge prevents series flow through the digester cells and reduces the biosolids stabilisation achieved.

The NSW Biosolids Guidelines calls for compliance with either the Specific Oxygen Uptake Rate (SOUR) or Further Volatile Solids Reduction (FVSR) tests to meet the Grade B stabilisation requirement. A minimum digester SRT of 30 days has been allowed for at BWL to provide the necessary confidence the SOUR requirement will be met. For the digester design we have not allowed to meet the FVSR requirement for Grade B. Research undertaken by Hunter H2O and others (refer reference 1) identified refractory organics in sewage are digestible in aerobic digesters. This leads to much higher solids destruction than predicted by commercially available models. This research also identified it is difficult to meet the FVSR requirement unless very large capacity aerobic digesters are provided (i.e. > 35 days SRT).

The NSW Biosolids Guidelines are under review and adjustment may be made to the stabilisation requirements. A SOUR slightly below the current guidelines has been targeted to provide a small contingency if this value lowers in future. However, at this stage we cannot be certain if the digester size will be adequate for possible future changes to the guidelines. At commissioning loads the SRT could be 50% higher than the design value and will provide a lower SOUR (improved stability) than if at design load. This will provide some buffer to allow QPRC to react to future guideline changes if they occur.

The aeration system has been sized for a typically minimum bioreactor SRT of 20 days and a digester SRT of 30 days. If one oxidation ditch is taken offline the bioreactor SRT will need to reduce to around 11 days at 75,000 EP (SRT can be higher for lower EP loads). The lower SRT will present less stable sludge to the digester which will increase aeration demand. Rather than allow for higher capacity blowers and aeration grid capacity for such a rare event, the operation of the digester will need to be modified for the oxidation ditch offline scenario. The modification will include operating no higher than a 22.5-day digester SRT by adjusting the WAS Thickener to target a lower solids concentration. At the lower SRT the aeration demand will match the installed aeration for normal operating scenario with two oxidations ditches. The SOUR during this offline scenario will be higher but still likely meet Grade B.

The digester has been designed with volumetric freeboard above the minimum design SRT to enable the WAS thickener to operate continually, however the dewatering can be offline for up to 3 days. This is to avoid the need to operate and remove biosolids during a long weekend period. However, for the 15 days SRT oxidation ditch offline scenario the freeboard will only provide 2 days of buffer storage.

Flexibility has been provided to take any digestion cell offline. This has been achieved by providing pipework to feed any cell and draw from any cell to dewatering. If a Cell 1 is taken offline there is less aeration capacity to cope with the thickened WAS. For this scenario feeding of both Cells 1 and 2 is recommended.

As removable diffusers have been provided the need to take a cell offline is rare. It is likely only to be required if there is an issue with the structure.

Table 6-11 Aerobic Digestion Design Summary at 75,000 EP

Item	Value	Units	Notes
Digester			
Minimum SRT at BWL (normal operation)	30	days	At design load and a feed concentration of 12,500 mg/L. Minimum bioreactor SRT 30 days.
Minimum SRT at BWL (oxidation ditch offline)	22.5	days	At design load and a feed concentration of 10,000 mg/L. Minimum bioreactor SRT 11 day for one oxidation ditch operation.
Balance time above BWL	3	days	Enables continuous thickened WAS feed over long weekend with no dewatering. 3 days volume freeboard for normal allowed. 2 days freeboard for oxidation ditch offline.

Item	Value	Units	Notes
			Volume equals1,570 m^3 and accounts for a lower feed of 10,000 mg/L.
Number digester cells	3		Series operation
TWL	5.6	m	0.3 m freeboard above TWL
BWL	4.8	m	Minimum SRT meet at this level
Total volume at TWL	12523	m^3	To meet minimum SRT plus weekend buffer.
Total volume at BWL	10,950	m^3	To meet minimum SRT.
Expected SOUR	0.9-1.1	mgO_2/h/gTSS	20 day SRT bioreactor + Digester minimum SRT
Expected SOUR	1.1-1.3	mgO_2/h/gTSS	11 day SRT bioreactor + Digester minimum SRT. Oxidation ditch offline scenario
Aeration			
Aeration cycle typical			120 min aeration on / 120 min aeration off Cell 1 aerated then Cells 2 and 3
Expected fouled Alpha factor			Cell 1 0.33, Cell 2 0.36 & Cell 3 0.38 (15 % fouling factor allowed)
Adopted beta factor	0.95		
Aeration DO	2	mg/L	Max, 1 mg/L more typical
Peak SOTR	530 250 150	kgO_2/h kgO_2/h kgO_2/h	6,800 Nm^3/h expected blower flow required 3,300 Nm^3/h expected blower flow required 2,180 Nm^3/h expected blower flow required
Number Blowers	3		Duty/ assist / standby
Peak blower capacity	3,400	Nm^3/h/blower	
Minimum blower capacity	1,700	Nm^3/h/blower	

6.10.3 Sludge Dewatering

Two centrifuges will be provided with a combined ability to dewater produced sludge at 75,000 EP over 35 hours per week (5 days for 7 hours per day). If a centrifuge is not available for an extended period, dewatering at 75,000 EP would need to occur over 7 days at 10 hours per day or 5 days over 14 hours per day. From experience centrifuge operation is typically stable and can generally operate well unattended.

The system is designed to use a pivoting conveyor to fill up to two semi-trailers. A common belt conveyor will feed each pivoting conveyor. Therefore, only one truck can be filled at a time. At peak production rate it will take up to 3.5 hours to fill one truck body (7 hours if one centrifuge is operating) to its 20 m^3 holding capacity. Auto change over to the other semi-trailer loader can occur once the fill time has elapsed.

Initially up to 6 trucks per week are expected for a 50,000 EP commission load. At 75,000 this will increase to 9 trucks per week. Initially if QPRC wished to only provide one pivoting conveyor they would likely need to run at least 6 days per week. For the design load two conveyors will be required as more than 1 truck per day will need to leave the site. It is recommended that two pivoting conveyors be provided with the upgrade to better manage removal logistics and enable the full 75,000 EP to be realised without the need for a later minor plant upgrade. Initially this will mean a double load will need to be produced on one day per week, with the other days being single loads if a 5-day dewatering basis is used. At 75,000 EP, 4 out of 5 days per week a double load will need to be produced for a 5-day dewatering basis.

Designs were sought from two centrifuge suppliers to progress the dewatering design. The option to provide pre-thickening ahead of the centrifuge was explored with one supplier. This can lower the power use as less water needs to be handled by the centrifuge. However, it increases the complexity with a machine ahead of the centrifuge. A cost benefit analysis identified there was no financial benefit in providing pre-thickening.

As noted in Section 6.10.1 it is unlikely there will be long term failure of the WAS thickener. If it did occur dilute WAS would need to be directly wasted to the digester as such the MLSS concentration will be much lower than normal and the flow rate will more than double. For this failure scenario two centrifuges would need to operate at 10 hours per day 7 days per week (20 hours/day for one centrifuge). This scenario was reviewed with the suppliers and the centrifuge can handle the diluted sludge.

The above is a rare event. If it occurred for more than 10 days, the digester SRT would drop considerably to approximately 9 days. It is likely under this scenario that the SOUR will not meet Grade B stabilisation requirements and the biosolids would need to be classified as Grade C stabilisation. Even with the lower SRT the overall SRT is reasonable (i.e. 20 days in the bioreactor + 9 days in the digester) and the dewatered cake is unlikely to present any odour risk for short term storage (i.e. up to 3 days).

Table 6-12 Sludge Dewatering Design Summary

Item	Value	Units	Notes
Dewatering			
Design operating time	35	Hours/week	two centrifuges operating
Number of centrifuges	2		
Centrifuge capacity	452 57	kg/h/centrifuge m^3/h/centrifuge	
Capture efficiency	90	%	
DWAS Fed Pumps	3		Duty / duty / standby
DWAS Pump Capacity	16	L/s/pump	
DWAS Pump Minimum	8	L/s/pump	
Polymer System			Liquid emulsion 0.5% makeup solution
Number pumps	3		Duty / duty / standby
Peak polymer dose	12	kg/ dry tonne	Likely value is 7-9 kg/dry tonne
Polymer Pump Capacity	1,100	L/h/pump	
Polymer Pump Minimum	440	L/h/pump	To enable 5 kg/ dry tonne
Dilution water post pump	2.5	L/s	To lower polymer to 0.15 prior to dosing.

Item	Value	Units	Notes
Polymer Emulsion Storage	1.5	kL	Transferred to tank from IBCs
Pivoting Conveyor			
Number	2		One needed initially. Two for 75,000 EP
Capacity	7 7.4	tonne/h m^3/h	Based on worst case 13 % cake thickness, > 16 % expected.

6.11 Recycled Water System

The site Recycled Water system provides water for use in the treatment plant ring main and the recycled water standpipe for offsite use.

The plant ring main provides recycled water to most of the plants water use demands. Refer to the P&IDs. Most of these uses are sprays or flows to enclosed controlled areas such as plant inlet works screens. Recycled water is also provided to supply hose reels on site. It is not intended for this water to leave the plant site.

The recycled water standpipe provides recycled water to tankers for uses managed by QPRC such as road construction and dust suppression.

The *NSW Guidelines for Recycled Water Management Systems* sets out minimum pathogen Log Reduction Values (LRVs) for various reuse classifications (NSW Department of Primary Industries, Office of Water, May 2015). The LRVs required for Municipal Use category which includes unrestricted access application and nominates dust suppression are outlined in Table 6-13. LRVs are also set out in the Australian Guidelines for Water Recycling (AGWR) (NRMMC, 2006), however, in this case, and many others, the LRVs set out in the NSW guidelines are more stringent than those in the AGWR, therefore achieving compliance with the NSW guidelines will ensure the system meets the equivalent AGWR requirements.

The proposed recycled water system can source effluent from two locations being post the UV or from the Clear Water Tank after filtration. Typically, flow would be taken from post UV to provide this additional disinfection barrier. However, in wet weather events bypassing of filtration will occur and the UV will see a higher particulate load and disinfection performance will decline, however quality will be within the licence limits. In these instances, recycled water can be sourced directly from filtration if operational issues are experienced with the plant ring main.

Outlined in Table 6-13 are the LRVs likely achieved for the concept design for both the plant ring main and effluent standpipe with UV disinfection online or not. Justification is provided in the notes section in the table below.

Table 6-13 Log Reduction Values for Recycled Effluent

Item	Protozoa	Virus	Bacteria	Notes
Municipal Use Dust Suppression Target	3.7	5.2	4	NSW DPI Recycled Water Guidelines Table 4
Process Concept – LRVs Likely Achieved				
Activated Sludge	0.5	0.5	1	Minimum value DPI guidelines Table 8
Dual Media Filter with coagulation	1.4	1.2	1	Minimum value DPI guidelines Table 8

Item	Protozoa	Virus	Bacteria	Notes
UV	3	-	3	No virus removal unless specific validated unit selected.
Chlorination				
• Plant Main	-	~1	~1	Estimated based on ring main contact on residual basis
• Standpipe	-	4	4	Dosed to achieve 4 log Keegan (2012) on primer kill basis
Total Achieved - Treatment Plant Ring Main Only				
Total Achieved				
Filter + UV + chlorine	4.9	2.7	9	
Filter + chlorine	1.9	2.7	6	
Total Achieved - Stand Pipe Only				
Filter + UV + chlorine + CT	4.9	5.7	9	**Stand Pipe Only**
Filter + chlorine +CT	1.9	5.7	6	CT = pipe contact time at 1.3 mg/L free chlorine residual x baffling factor x contact time = 11 mg/L.min required for 4 log Keegan (2012)

LRV for offsite controls can be claimed, if required, and if the controls were fully managed. However, the risk is reduced if recycled water supplied offsite meets the NSW guidelines LRVs for unrestricted use. These can be met if the UV system is online and adequate chlorine CT (i.e. contact time x free chlorine residual) are provided for 4 log virus removal (i.e. 11 mg/L.min). With the available pipeline length for the RW facility to the standpipe location, a 200 mm main will provide the necessary CT based on a 1.3 mg/L free chlorine residual. The CT has been calculated assuming progressive tankers load one after another and only the dwell time in the main can be relied upon when filling occurs (i.e. standing time in the main is not acceptable between fills).

Online monitoring of free chlorine has been included to demonstrate the required CT for virus inactivation will be met for the standpipe.

If the UV is not utilised during peak wet weather events or the conditions of the proprietary UV system are outside the requirements for 3 log protozoa inactivation, then recycled water supply to the standpipe must stop. It is likely this will be at a time when no reuse will be required due to moderate to heavy rainfall.

The LRV likely achieved for the site ring main have also been shown in Table 6-13. We are not aware of specific standards for internal STP use. However, moderate LRVs are met and this water source is more controlled in an STP. The virus inactivation is lower than the standpipe value. This is due to there being no dedicated chlorine contact time for virus LRVs. However, chlorination is provided and up to 1 log virus removal is expected based on the contact in the recycled water storage and to the first draw off from the ring main. This chlorination has been primarily provided for slime control in the ring main, however it also provides a disinfection barrier.

An outline of the design of the recycled water system is provided in Table 6-14.

In the event of major failure of the recycled water supply pumps a potable water supply has been included as a backup water supply to the Recycled Water Tank. It has been limited to just supply the two duty Band Screen only so screening removal can continue in the event of recycled water supply loss. This is the most critical system for the recycled water supply system.

Other systems can be offline for a period while supply is restored. It is not practical to supply the whole system with potable water due to potable main supply limitations.

In the later reference design stage, the ability to provide 10 L/s of potable water will need to be reviewed. There are other requirements for the potable supply to meet the firefighting code and BCA considerations. The existing potable supply main will need to be assessed.

Liquid chlorine (sodium hypochlorite) is required in small quantities to provided chlorination for the recycled water system. Refer Section 6.12.3 for design details on this system.

Table 6-14 Recycled Water Design Summary

Item	Value	Units	Notes
Peak RW ring main use	L/s	34	Most major uses occurring at one.
Tanker Fill Rate	L/s	25	For under 7 min fill time
Total RW Pump Demand	L/s	59	
Number RW Pumps	2		
Pump Range	L/s	7 - 60	Proprietary variable speed multistage pressure pumping system with accumulator pressure vessel for very low demands 0 to 7 L/s
Pump Design Pressure	TBA		To be set in reference design once peak pressure for worst case demand is known. Pressure will be adjustable.
RW Tank Size	TBA		To be set in reference design once sequencing of demands is known subject to more design.
RW Supply Pumps No	2		Duty / standby
RW Supply Pumps Range	30 -60	L/s	
Potable water backup	10	L/s	To supply band screens (2 off) only in the event of supply pump failure.

6.12 Bulk Chemicals

This section deals with bulk chemical used in the common bulk storage area. Smaller quantities of liquid emulsion polymer and powder polymer are used in dewatering and filtration respectively. Refer to Sections 6.8.2 and 6.10.3 for details on these polymer chemicals.

6.12.1 Alum

The design of the alum system is summarised in Table 6-15. Alum is used primarily in chemical phosphorus removal.

The alum usage is dependent on the raw sewage concentration and how much EBPR can be achieved (refer Section 6.6.2). In Table 6-15 the usage and storage time for both worst case (no EBPR) and some moderated (30%) EBPR is quoted. EBPR of 30% is expected, however actual performance will depend on how much digestion occurs (P is released in digestion).

Note EBPR is the usage beyond what is needed for BOD removal (typically 30 % of raw sewage phosphorus is removed to enable bacteria growth). After allowing for phosphorus usage for BOD removal, the remaining phosphorus removal through EBPR is 30% what remains. This equates to an overall phosphorus removal of 49% (i.e. 100 % x (1-30) % x (1-30 %) = 49%).

To achieve a high degree of phosphorus removal alum is dosed in two locations (bioreactor and filters) by two separate dosing pumps.

Table 6-15 Alum Design Summary

Item	Value	Units	Notes
Strength	4.24	%Al w/w	Alum solution of $AL_2SO_4.14H_2O$
Number Tanks	2		
Capacity of Tank	50	kL	100 kL overall
Storage Time 75,000 EP Full Chemical P 30% Biological P	 25 35	 days days	 30 % EBPR (noting 30% occurs without EBPR)
Bulk Transfer Possible In Storage Volume	4		Based on 20 kL/tanker and assuming some residual to manage order (20 kL)
Peak Usage 75,000 EP Full Chemical P 30% Biological P	 4.0 2.5	 kL/d kL/d	 30 % EBPR (noting 30% occurs without EBPR)
Bioreactor dose pumps No	2		Duty standby
Bioreactor Pump Range	40 -770	L/h	Digital dosing pump with larger turndown.
Peak bioreactor dose rate	3	mol P/mol Al	
Filter dose pumps No	2		Duty standby
Filter Pump Range	25 -186	L/h	Digital dosing pump with larger turndown.

6.12.2 Caustic

The design of caustic (i.e. sodium hydroxide solution) dosing system is summarised in Table 6-16.

Caustic is only needed to keep the pH within licence and at the optimal level for the high levels of phosphorus removal required.

There are two acidic processes (pH lowering) in wastewater treatment which include biological nitrification or ammonia removal (nitric acid forms) and chemical phosphorus removal (sulphuric acid addition). There are basic processes (pH increasing) which offset these acidic reactions somewhat and include biological denitrification, organic nitrogen mineralisation and liquid chlorine dosing. However, more acid is added than alkali, especially when low TP limits are targeted.

Modelling shows caustic dosing will be required. However, as there are many processes occurring which impact the pH, the amount of dosing required can vary significantly operationally. For example, elevated nitrate can increase caustic dose as less alkalinity is recovered by denitrification. Also, the degree of EBPR occurring has a significant impact on the caustic dosed required due to lower alum usage.

The worst case and expected 30 % EBPR doses are shown in Table 6-16. However, as noted above dose may vary outside of these values if other variables change (i.e. denitrification, raw sewage alkalinity and filtration target TP).

Two dosing locations have been provided to better optimise the use of caustic. A base dose will be needed in the bioreactor system. However, it is important the pH can be adjusted at filtration to target the optimal value for phosphorus removal.

Table 6-16 Sodium Hydroxide Design Summary

Item	Value	Units	Notes
Strength	25	%NaOH w/w	Low strength to minimise freezing risk.
Number Tanks	2		
Capacity of Tank	25	kL	50 kL overall
Sewage alkalinity	257	mg/L	$CaCO_3$
Target Effluent Alkalinity	90	mg/L	$CaCO_3$
Storage Time 75,000 EP Full Chemical P 30% Biological P	 17 73	 days days	As well as alum dose for P removal dose depends on raw sewage alkalinity and effluent nitrate. Low raw alkalinity and high nitrate will increase dose and vice versa. 30% EBPR (noting 30% occurs without EBPR)
Bulk Transfer Possible In Storage Volume	2.5		Based on 20 kL/tanker and assuming some residual to manage order (10 kL)
Peak Usage 75,000 EP Full Chemical P 30% Biological P	 3 0.8	 kL/d kL/d	30% EBPR (noting 30% occurs without EBPR)
Bioreactor dose pumps No	2		Duty standby
Bioreactor Pump Range	100 - 900	L/h	Digital dosing pump with larger turndown.
Filter dose pumps No	2		Duty standby
Filter Pump Range	20 - 150	L/h	Digital dosing pump with larger turndown.

6.12.3 Sodium Hypochlorite

The design of sodium hypochlorite system (liquid chlorine) is summarised in Table 6-17. Chlorine dosing is not used for effluent disinfection for discharge. It is used to provide slime control in the RW ring main and disinfection for site use and the tanker standpipe. Refer to Section 3.4 for a description of the design of the chlorine disinfection system for recycled water.

As the usage rate is low only a small 1.5 kL dose storage tank is required. This is to allow acceptance of 1 kL IBC containers. Up to 3 IBCs can be stored in the bunded area along with a storage tank.

Direct dosing out of an IBC was explored to remove the need for a dedicated storage tank. Sodium hypochlorite breaks down over time and forms gas. A bleed back to the storage tank is needed to handle this gas from the suction side of the pumps. The bleed to the tank is preferred to avoid spillage if accidental pressurisation of the line occurred. With an IBC as no rigid line could be provided for the bleed line it was considered a potential safety risk. Therefore, a dedicated tank with rigid return bleed line is recommended.

Table 6-17 Sodium Hypochlorite Design Summary

Item	Value	Units	Notes
Strength	13	% w/w	As supplied NaClO. 10 % w/w assumed to account for degassing in usage and tank size calculations
Assumed chlorine demand	6	mg/L	Dose before free chlorine forms
Target free chlorine residual	2	mg/L	
Number Tanks	1		One bulk. Fed by stored IBCs
Capacity of Tank	1.5	kL	
Storage Time 75,000 EP Tank 2 Stored IBCs Total (2 IBC's and Tank)	 18 24 42	 days days days	Average usage. Usage will increase for wet periods slightly. Temperature in summer may degrade chlorine and may decrease storage time available. 3 can be stored.
Bulk Transfer Possible In Storage Volume	1		Based on 1 kL/IBC and assuming some residual to IBC transfer (0.5 kL)
Peak chemical Usage 75,000 EP	67(16)	L/d	Plant + (20 standpipe tankers per day)
Filter dose pumps No	2		Duty standby
Filter Pump Range	0.2 - 17	L/h	Digital dosing pump with larger turndown.

7 Civil / Structural Design

7.1 General

A detailed 3D Navisworks model of the proposed upgrade has been prepared as part of the concept design. The civil / structural design of the upgrade is presented in the Site Arrangement in Appendix A and 3D Navisworks Model Views in Appendix D.

This section provides additional details on the civil and structural design elements of the proposed upgrade.

7.1.1 Site Layout

The new Queanbeyan STP will be located to the south and east of the existing STP. The new STP is primarily situated above the 100-year ARI nominated design flood level with the location of structures, mechanical and electrical equipment also giving further consideration to the potential for increases in the flood level with climate change (Figure 7-2).

The site footprint of the new STP is set out across a predominantly level area, with the exception of the new storm pond which is located in a part of the site that grades downwards to the north-east. The centre of the site is highest, with a minor drop in elevation to the east and slightly more prominent drop-off to the west. The site layout has been developed to use the topography to enable gravity flow through the processes where possible. Following diversion of the incoming Morisset and Jerrabomberra trunk sewers to the new inlet works located adjacent to the storm pond, influent is pumped up to the main plant pad area for grit removal before passing to the bioreactors. The liquid stream then flows by gravity to the east to the clarifiers. Flows are pumped to tertiary treatment by the DAFF and then flow by gravity to UV treatment and discharge via the site discharge manifold to the Molonglo River. Sludge from the central processes is pumped to the west to the WAS thickener, aerobic digesters and dewatering area.

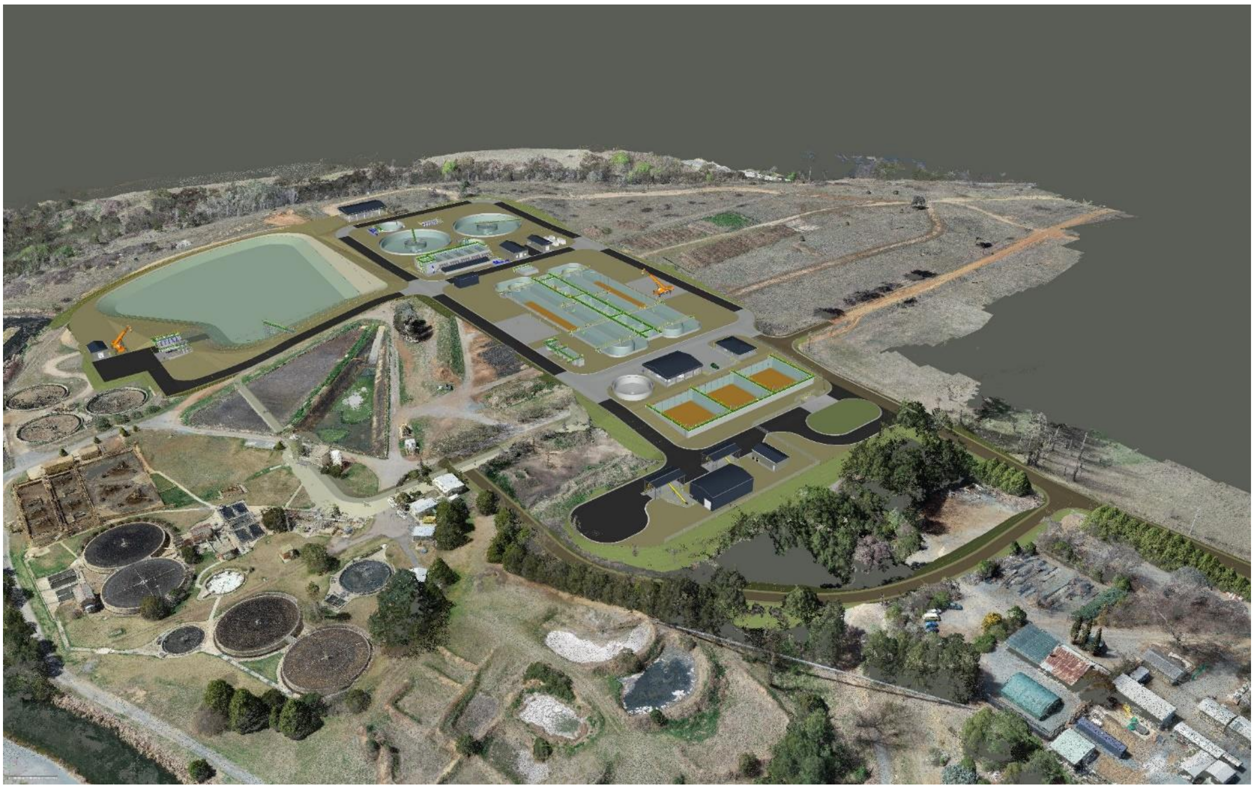


Figure 7-1 Overview of the Queanbeyan STP Upgrade (existing plant in foreground)

The location of structures on the site layout has been developed in consultation with QPRC and has given consideration to a range of factors including:

- Locating the hydraulic grade line and height of structures to ensure bioreactor, clarifiers and UV are positioned at ground level (i.e. top of structure is at handrail height generally) to simplify operation and reduce costs associated with access to elevated structures and sewage lift pumping.
- Protecting major structures and critical mechanical and electrical equipment from flooding (including an allowance for increased flood levels with climate change)
- Site geotechnical considerations
- Minimising hydraulic losses of major pipe runs through the treatment process
- Providing adequate space between structures for operation and maintenance access as well as for installation of below ground pipework and electrical conduits.
- Site operational monitoring and security requirements
- Consideration of construction sequencing requirements.

Access ring roads are provided for the new works, with new internal site pavements servicing all areas of the site within a grid arrangement. The layout accommodates the movement of heavy rigid vehicles with predominantly looping roadways and only two dead ends which can be avoided by larger vehicles. Provision has been made for permanent crane set down pads to access designated areas to allow for maintenance and repairs in the vicinity of mechanical / process infrastructure.

The site layout will continue to be refined and further developed during the design process to take into account final equipment selection and structural sizing as well as safety, construction, operation and maintenance requirements.

An upgrade to the site access road, Mountain Road is also included in the project, consisting of a two coat seal pavement upgrade to meet ACT Roads requirements. Further consultation with ACT Roads will occur in the reference design phase.

Depending on future growth there is the potential for STP expansion to be undertaken in the future resulting in a stage 2 plant capacity of 112,500 EP. This project has given consideration to the site space requirements of this future STP expansion. The general arrangement provides sufficient space in the area to the immediate south of the upgrade site for construction of the future expansion. The additional storm pond for the expansion can be constructed in the area currently occupied by the existing STP which will be decommissioned.

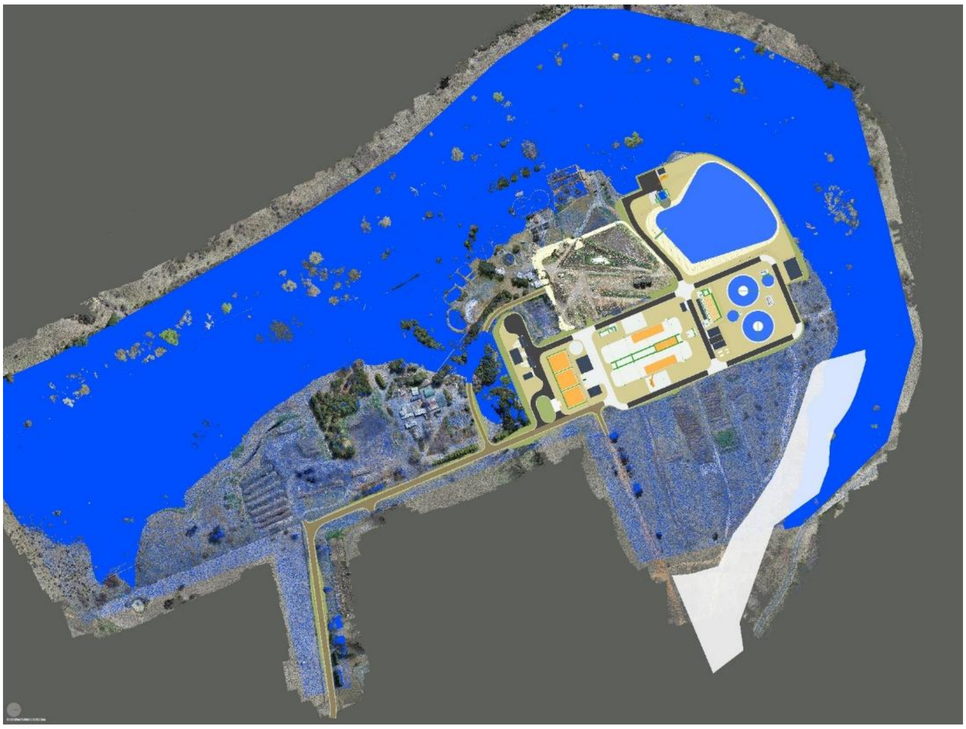


Figure 7-2 Queanbeyan STP Upgrade Site Layout Relative to the Nominated Design Flood Level

7.1.2 Design Service Life Requirements

Table 7-1 details the design service life requirements for all components of the contract works as nominated in the *Queanbeyan STP Upgrade Project: Durability Plan, Rev A* (Hunter H2O, June 2019).

Table 7-1 Design Service Life for New Infrastructure

Component	Service Life
Buildings – steel framing (structural)	30
Buildings – blockwork (structural)	50
Buildings (non-structural elements i.e. roofing/cladding/girts)	20
Buried pipelines	80
Road pavements	20
Liquid retaining structures	60
Structural steelwork	50
Stairs, platforms and similar metal work	30
Mechanical assets – incl. valves/actuators, motors & pumps, and other equipment	15
Electrical assets – switchboards	25
Electrical cabinets (field)	20
Instrumentation and control (incl. PLC, telemetry & SCADA)	15
Protective coatings	10
FRP bulk chemical storage tanks	25

7.1.3 Design Criteria

The design is being undertaken in accordance with the following Australian Standards:

- *AS1170.1 Permanent, Imposed and other Actions*
- *AS1170.2 Wind Loads*
- *AS1170.3 Earthquake Actions in Australia*
- *AS2159 Piling – Design and Installation*
- *AS2870 Residential Slabs and Footings*
- *AS3600 Concrete Structures*
- *AS3735 Concrete Structures for Retaining Liquids*
- *AS3700 Masonry Structures*
- *AS4100 Steel Structures.*

All structures for this project are being designed for Importance Level 3 (Wastewater Treatment Facility), in accordance with Table 3.1 of *AS/NZS 1170.0:2002: Structural Design Actions – General Principles.*

7.1.4 Building Code of Australia Considerations

All buildings including prefabricated structures are required to be constructed in accordance with the Building Code of Australia (BCA) and relevant Australian Standards.

A deemed to satisfy review of each building design will be undertaken in the reference design phase to establish relevant building classifications and clauses applicable under the BCA.

Final BCA compliance and certification responsibility would be included in the Construction Contractors scope by a private certifier for all buildings.

7.1.5 Site Fire Compliance

As a wastewater treatment plant, QSTP is considered critical infrastructure by the ACT Strategic Bushfire Management Plan (ACT Government, 2019).

ACT Fire & Rescue has primary responsibility for fire response in urban areas of the ACT. A deed of agreement between Icon Water and ACT Fire & Rescue outlines Icon Water's obligations to provide fire hydrant access and water supply to ACT Fire & Rescue within Icon Water's network area. Schedule 2 of the deed outlines the Fire Risk Type and minimum firefighting flow for a range of different developments. Wastewater treatment plants are not readily classified within the listed Fire Risk Types. The firefighting requirements for the facility will depend on factors including the scale, number of workers and building class.

The existing DN100 watermain along Nimrod Road has been assessed by ICON Water for its ability to provide firefighting flow requirements. The results of that assessment shown in Table 7-2.

Table 7-2 Water supply assessment of Nimrod Rd DN100 watermain at customer connection

Supply condition	Water supply pressure (m)
Max Static Pressure (m)	70
Min Pr @ Peak Demand	70
Min Pr @ Peak Demand + 10 L/s (m)	54
Min Pr @ Peak Demand + 20 L/s (m)	7
Min Pr @ Peak Demand + 21 L/s (m)	0

Further assessment of options to meet firefighting requirements of the facility will be undertaken during detailed design including an assessment against the Building Code of Australia requirements once building details are confirmed and further consultation with ACT Fire & Rescue and Icon Water. Final requirements may require an upgrade to the existing DN100 water supply or provision of onsite storage to meet firefighting requirements.

7.2 Geotechnical

7.2.1 Preliminary Investigation

A preliminary geotechnical investigation was undertaken by ARUP in June 2019. The preliminary investigation included a desktop review followed by excavation of a select number of test pits to between 0.6 m and 2.5 m depth and a limited set of in-situ and laboratory testing.

The findings of the investigation are detailed in the following reports:

- *Queanbeyan STP Upgrade: Site Investigation – Geotechnical Desktop Study* (Arup, August 2019)
- *Queanbeyan STP Upgrade: Factual Geotechnical Investigation* (Arup, August 2019).

The investigation area was limited to the proposed STP footprint build area and indicated that the rock layer was typically located at approximately 2 m below ground surface levels in this area, overlaid by alluvial and residual soil units with occasional shallow filling.

7.2.2 Detailed Investigation

A second, more detailed round of geotechnical investigation was undertaken by Douglas Partners in the period 21 January to 7 February 2020. The investigation comprised drilling of 40 boreholes to between 0.9m and 15.75m depth, supplemented by in-situ and laboratory testing.

The findings of the investigation are detailed in the following reports:

- *Queanbeyan Sewage Treatment Plant Upgrade: Factual Report on Geotechnical Investigation* (Douglas Partners, March 2020).
- *Queanbeyan Sewage Treatment Plant Upgrade: Interpretive Report on Geotechnical Investigation* (Douglas Partners, March 2020).

The investigation area comprised the proposed STP footprint, the length of Mountain Road and the vicinity of the proposed Morisset sewer trunk main realignment. The subsurface conditions encountered in the proposed STP footprint indicated that the top of rock is significantly deeper than indicated by the preliminary ARUP investigation, nominally between 2.3m and 10.0m depth.

The discrepancies in logged subsurface conditions between the ARUP investigation and that of Douglas Partners are likely due to the limited nature of the preliminary investigation. The information provided by Douglas Partners has therefore been adopted for the design.

7.2.3 Subsurface Conditions

The site primarily consists of interbedded medium-dense to dense sand and gravel units and very stiff to hard clays, underlain by bedrock. Some surficial filling was present in parts of the site.

The rock shelf within the new STP footprint is at between 2.3 m and 10.0 m depth below existing ground surface levels, generally shallower in the south-western portion of the site, increasing in depth in a north-easterly direction.

The typical subsurface profile encountered within the STP footprint is as outlined in Table 7-3.

Table 7-3 Summary of Subsurface Conditions within STP Footprint

Subsurface Conditions	Top Depth (m)	Bottom Depth (m)
Uncontrolled Filling	0.0	0.0 to 0.8
Alluvium / Residual Soils (Gravel/Sand/Silt/Clay)	0.0 to 0.8	2.3 to 10.0
Bedrock (Sandstone/Siltstone)	2.3 to 10.0	N/A

7.2.4 Groundwater

Groundwater was encountered at two boreholes only during the investigation, between approximately 561.2m AHD and 563.8m AHD (BHs 27 and 40). Groundwater wells were installed at these locations to facilitate groundwater monitoring.

Minor seepage was also observed in bores adjacent the existing sludge lagoons, possibly attributed to minor subsurface leakage. It is noted that groundwater observations were obscured during rock coring at all locations where undertaken and therefore the groundwater table may have been intersected but not observed. Groundwater conditions are also dependent on climatic conditions and will vary with time.

Buoyancy considerations will be required to be reviewed for all structures subject to flood inundation as part of the reference design phase. The current proposed approach would be either additional mass concrete in base elements of structures or where uneconomic use of floor relief valves in liquid retaining tanks. The storm pond will require careful consideration as the invert is located below the flood level. Currently the storm pond liner is proposed to be a combination of clay with a ‘Tiltex’ or equivalent cementitious fabric overlay which incorporates the floor relief valves. It is considered uneconomic to provide a full reinforced concrete liner.

7.2.5 Site Classification

The site is generally classified as Class S to AS2870:2011. This is the minimum foundation requirement to the code; however, the classification for parts of the site where more than 0.4m of filling is present, or where greater than 0.4m of engineered filling is to be placed as part of the works, would be Class P.

7.2.6 Soil Aggressiveness

All soils encountered on the site are classified as non-aggressive for both concrete and steel structures. Classifications are based on AS2159:2009.

7.2.7 Foundation Summary

The variability of the rock profile within the STP footprint has implications for structural foundations, with differential settlements a key issue to address in reference design for selection of foundation improvement. This is a pertinent issue for the major liquid retaining structures e.g. the bioreactor, clarifiers, filters etc due to the variability of founding strata properties across their large footprints.

The footing systems currently proposed are as per Table 7-4.

Table 7-4 Preliminary Foundation Loading for Each Structure

Structure	Approximate Load (kPa)
Clarifiers	75
Filters	50
Blower Room	25
Electrical Switchrooms	25
Bioreactor/ Oxidation Ditch	75
Chemical Dosing Facility	25
Aerobic Digesters	75

7.2.8 Contamination Assessment

A preliminary contamination assessment of the site was undertaken in June 2019 by Senversa in conjunction with the preliminary geotechnical investigation. The preliminary assessment identified a number of contaminants (hydrocarbons, zinc, copper and phosphorus) present which are likely to be a result of historical stockpiling of biosolids on the site (Senversa, October 2019). Asbestos containing material was also detected in a number of locations.

Further contamination assessment was undertaken in conjunction with the detailed geotechnical investigation. Further details will be provided as part of the Environmental Impact Statement.

7.3 Survey and Services

Initial site survey has been completed using an aerial drone survey, supplemented by a site control survey and survey of discrete existing manhole invert levels.

Existing services have been identified through existing record drawings and Dial Before You Dig Enquiries. Location and survey of site services within the geotechnical investigation area was undertaken in the period 21 to 22 January 2020 by Hunter H2O's surveyor and Commence Communications' service locator.

Further location of survey and services location will be undertaken as needed to support development of the reference design.

7.4 Diversion of Incoming Sewer Trunk Mains

Sewage is currently conveyed to the existing STP by two incoming sewer trunk mains; the Morisset trunk main to the south; and the Jerrabomberra trunk main to the west. Both mains currently terminate at the existing inlet works which is situated approximately 180m south-west of the new primary inlet works. As part of the upgrade, both mains will be diverted to the new inlet works receival pit.

The new sections of main within the site should be constructed from the new inlet works back to the proposed cut-in points. Then, to facilitate construction of the diversion, a temporary bypass along each sewer main either side of the proposed cut-in point will be required, involving the isolation of a section of main between consecutive manholes using temporary lift pump or vacuum trucks as deemed appropriate. Excavation around the cut-in point and installation of a prefabricated tee/elbow would then connect the incoming main to the new section of main leading into the site.

Figure 7-3 Diversion of Jerrabomberra and Morisset Trunk Sewers to New Inlet Works

7.5 On-Bank River Discharge

The upgrade to the Queanbeyan STP requires the location of the effluent discharge to the Molonglo River to be relocated to be adjacent to the new site. The location has been selected to minimise impact to the riverbank and avoid in river construction works.

Treated effluent from the UV outlet pit and emergency overflow from the Storm Pond will be discharged to an overland discharge structure adjacent the river. The discharge structure will consist of a concrete anchored, HDPE diffuser heads. The header pipe distributes effluent flow through a series of holes at different levels in the header, this is designed so that flow is distributed across rip rap at a range of flow rates.

An area of rip rap would be provided at the discharge structure to prevent scouring of the riverbank.

7.6 Roadworks

7.6.1 Mountain Road Upgrade

The Queanbeyan STP is accessed via Mountain Road. The existing road is unsealed and is in poor condition.

As part of this upgrade project, Mountain Road is to be upgraded from the existing intersection with Railway St to accommodate access for larger vehicles and provide suitable road conditions for operational traffic. Preliminary discussions with ACT Roads has been undertaken and consultation will be finalised as part of the reference design phase.

The new road pavement will comprise the following, based on a design traffic loading of 6×10^4 ESA (design safety factor of 2.0 applied):

- Surfacing: two-coat spray seal
- 150 mm thick DGB20 or equivalent basecourse
- 150 mm thick DGS20 or equivalent subbase.

The vertical alignment of the existing road is to be largely adopted with minor alterations to blend with existing / new pavement areas. It is noted that areas requiring significant filling are limited laterally by the ACT Roads easement and resultant pavement width (including embankments / batters) which cannot infringe on adjacent land to the south (not owned by QPRC).

The road is proposed to be constructed with sufficient crossfall to facilitate stormwater drainage. No kerb and guttering system are proposed, with revegetation along the road reserve instead recommended to mitigate erosion due to stormwater runoff. A suitably sized culvert will be required beneath the road at its single low point.

7.6.2 Plant Roads

The access road within the STP will enable up to a 19m articulated vehicle to access the proposed structures and equipment for construction, maintenance and operational purposes.

The new road pavement design will be based on a design traffic loading of $6x10^4$ ESA (design safety factor of 2.0 applied).

The roads are proposed to be constructed with sufficient crossfall to facilitate stormwater drainage. Kerb and guttering will be present along the edges of all sealed internal site pavements.

7.7 Amenities Building

The upgrade will include the construction of a new amenities building. The building will be located close to the entry to the plant for security and operational monitoring. It is proposed the building will be constructed of masonry to provide good thermal insulation.

The amenities building will include:

- Control room with SCADA terminals
- Kitchen and eating area
- Bathroom with toilet, shower and washing machine
- Sample storage / testing area.

Further consideration will be given to incorporating a community information and training area.

7.8 Stormwater Drainage

Site buildings will be constructed with suitably sized roof guttering and downpipe systems to direct stormwater to road level. The internal site pavements will be constructed with crossfall to direct stormwater towards kerb and guttering, which will contain suitably spaced inlet pits and adjoining pipework to collect and distribute discharge to a stormwater detention basin. The subsurface stormwater network configuration and final basin location will be determined in reference design. Based on site topography, it is currently proposed that site stormwater will drain to an onsite stormwater detention basin via stormwater drainage along the northern boundary and eastern edge of the site. The basin will discharge via a broad crested weir with low flow outlet structure incorporated into the design.

Along Mountain Road, no formed kerb and guttering is currently proposed; however, a culvert or similar will be required at the low point situated approximately 150 m south-west of the existing site entrance to prevent localised flooding. The downstream (northern) end of the culvert will require construction of a suitable diversion channel to promote the stormwater flow to the natural gulley orientated north-west towards the Molonglo River.

7.9 Maturation Pond and Riverbank Stabilisation

The existing treatment plant process includes three maturation ponds that are located on the southern side of the Molonglo River. The three maturation ponds operate in series with an area in the order of 7.6 ha and a volume approaching 200 ML. Effluent from Pond 3 discharges to the Molonglo River at the existing licenced discharge point.

The maturation ponds are located within the extents of the 1% AEP flood zone and are at risk of failure during flood events. The property boundary of the Queanbeyan STP extends only to the downstream crest of the pond embankments. The land downstream of embankments is under authority of the Act Government.

The maturation ponds will not form part of the treatment process once the new treatment plant has been constructed and commissioned. As part of the upgrade project, it is proposed that the ponds are decommissioned and the area remediated to extend the riparian zone along the Molonglo River bank.

The condition of the embankments has been the subject of two studies as presented in the following documents:

- *Queanbeyan WWTP: Maturation Pond Embankment Investigation* (Hunter Water Australia, January 2011)
- *Queanbeyan Sewage Treatment Plant Maturation Ponds: Condition and Consequence Assessment* (Public Works Advisory, March 2019).

The major finding of these investigations is that the ongoing stability of the embankments is of concern, and there is an ongoing risk that the riverbank or pond embankments may fail during the next significant flood event.

As the new STP treatment does not require use of maturation ponds, it is proposed to decommission them in such a way to eliminate the potential downstream risk of embankment failure without retaining water.

The proposed method of decommissioning the maturation ponds is as follows, to be undertaken following commissioning of the new STP:

- Dewater existing ponds, including removal of fish
- Descale internal faces of lagoons/embankments (i.e. remove sludge)
- Partially backfill empty ponds with excess spoil generated from new STP construction, including compaction in layers not exceeding 300mm thickness, placement of geotextiles and subsequent revegetation to mitigate future erosion
- Breach embankments of each pond to allow floodwater ingress and recession without retention. Breaching should include recontouring, placement of geotextile/riprap/rock armouring/concrete reinforcement and revegetation around the breached embankment face to mitigate future erosion.

Further details on the proposed construction sequence is provided in Section 9.

8 Electrical, Instrumentation and Control

8.1 Electrical Concept Design

This section provides additional details on the electrical and control elements of the proposed upgrade. The electrical concept design drawings are included in Appendix E. The following drawings are included:

- Single Line Diagrams
  - Main Switchboards, accepting transformer and generator feeds and distributing power to site
  - Motor Control Centres, located at three major plant areas supplying power to local equipment in each area (Inlet Works, Main Plant, Sludge Handling)
- PLC and Control Network diagrams, showing the site wide optical fibre networks and local sub-networks
- General Arrangement Diagrams
  - Main Switchboard arrangements
  - Motor Control Centre arrangements
  - Switchroom layouts.

8.2 Supply Authority

The existing site is supplied by Evoenergy via the Abattoir feeder coming out of Fyshwick Zone Substation. The existing Evoenergy distribution is shown in Figure 8-1.

Evoenergy has advised that the Abattoir and neighbouring feeders currently do not have any spare capacity to support the upgraded plant. Evoenergy is initiating a project to augment the network to increase the supply capacity in the Fyshwick area and are expecting that by June 2024 2.4 MVA capacity can be made available for the new plant either through the network augmentation or load transfer to adjacent / neighbouring feeders, which aligns with the present project schedule.

The upgrade approach discussed with Evoenergy is to retain the power supply to the existing plant while the new plant is constructed and commissioned, at which time the existing plant supply will be released. This arrangement will also provide temporary power supply for use during construction. This avoids the need to operate the existing or new treatment plant with diesel generators or other temporary supplies during commissioning.

The existing Abattoir feeder runs through the proposed site and segregates he current upgrade site (75,000 EP) and space identified for use in a future expansion. Preliminary discussions with Evoenergy have identified issues with sharing of an underground cable easement, however if the easement remains overhead, pipes and electrical crossings associated with the future expansion should be acceptable to Evoenergy. The crossing of the Evoenergy HV easement will need to be detailed during the detailed design phase to ensure any upgrades to Evoenergy assets are aligned with the future requirements.

Figure 8-1 Evoenergy Distribution

8.3 Power System Architecture

The power system architecture for the site is shown in the single line diagrams in Appendix E. As a climate change adaption measure, the power system architecture uses two pad mounted transformers with separate distribution across the site. Each transformer supplies a dedicated Main Switchboard (MSB) and in turn each MSB has a separate distribution system. The calculated STP Upgrade demand loading is below Evoenergy's requirements to be HV metered, and therefore each transformer will be connected via a dedicated LV metering system.

The MSBs are used for power distribution to the Motor Control Centres (MCCs) and do not contain any motor controls themselves. This design strategy reduces the maintenance requirements for accessing the MSBs which improves switchroom safety and reliability. The MCCs contain all of the motor controls and final distribution for the motors and process loads. Allocation of these process loads across each distribution system is generally by Duty/Standby roles in order to minimise the disruption to operations due to the loss of one distribution system.

8.4 Backup Generators

Backup power to each MSB is provided by dedicated onsite diesel generators connected with an Automatic Transfer Switch (ATS) to automatically start and transfer power on loss of the associated transformer supply. The provision of a backup generator is a climate change adaption measure. The switchgear and transfer switch arrangement will prevent paralleling of the generator and transformer.

An investigation into the fuel requirements for the generators was undertaken to determine the fuel storage requirements for different periods of operation, based on the expected Maximum Demand of the plant discussed in Section 8.5 below.

The generator fuel curve and generator fuel usage are provided in Table 8-1 and Table 8-2, respectively.

Table 8-1 Generator Fuel Curve

Fuel Consumption (L/hr)		
25%	82	82
50%	155	155
75%	222	222
100%	289	289

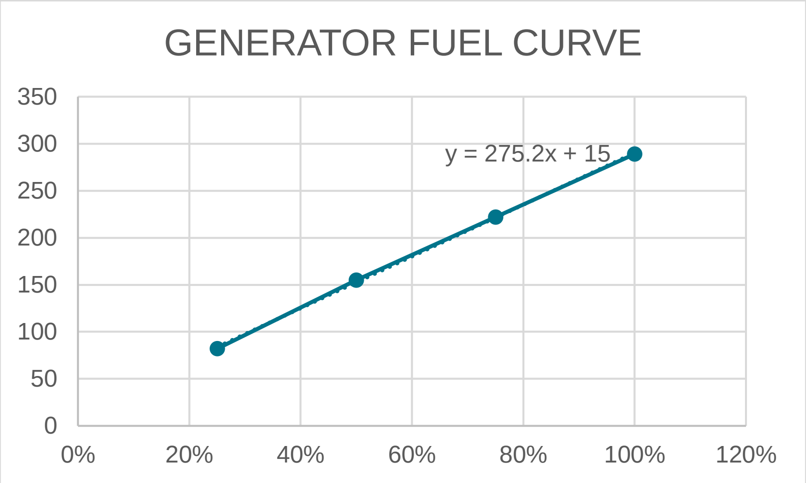


Table 8-2 Generator Fuel Usage

	GEN-100	GEN-200
Model	C1675 D5	C1675 D5
Engine	KTA50-G8	KTA50-G8
Prime rating (kVA)	1400	1400
Loading kVA (Max Demand)	938.1	938.1
Loading percentage	67%	67%
Fuel usage (L/hr)	200	200
Fuel usage (L/Day)	4,800	4,800

Summary	
400	Litres/hr
9,600	Litres/day
19,200	Litres/2 days

Based on consideration of various failure modes for the power supply at site, a 20 kL above-ground diesel storage facility is to be provided. This storage volume is sufficient to maintain backup power supply for 48 hours with the treatment plant running continuously at peak treatment capacity.

As the generators will be used only in emergency circumstances the diesel system will include an inbuilt conditioning system to keep the diesel fresh. Further consideration of the safety classifications and storage design requirements will be developed during the detailed design.

8.5 Maximum Demand

The maximum demand for the upgraded site is calculated to be 2,708 Amps (1,876 kVA) based on the Concept Design equipment loads and using a 0.85 diversity factor. The detailed calculations are provided in Appendix G, and a summary provided below in Table 8-3.

Table 8-3 Maximum Demand Summary

DIVERSIFIED SUMMARY CALCULATIONS										
SWITCHBOARD	MAXIMUM DEMAND				POWER FACTOR			HARMONICS		
	AMPS	kVAR	kW	kVA	POWER FACTOR	MAX kVAR	PFC Required KVAR	MAXIMUM AMPS	THDi AMPS	THDi %
MCC110	1156.7	217.6	771.3	801.4	0.96	253.5	-	1304.0	418.7	32.1%
MCC120	464.0	95.5	306.9	321.5	0.95	100.9	-	529.7	171.6	32.4%
MCC130	336.0	48.9	227.6	232.8	0.98	74.8	-	382.6	150.0	39.2%
MSB100	1955.9	362.0	1305.9	1355.1						
MCC210	1197.3	220.2	799.8	829.5	0.96	262.9	-	1356.4	443.3	32.7%
MCC220	432.6	79.6	289.0	299.7	0.96	95.0	-	496.3	173.1	34.9%
MCC230	481.0	68.9	326.0	333.2	0.98	107.2	-	545.9	215.3	39.4%
MSB200	2110.3	368.7	1414.8	1462.0						
SITE TOTAL	2708.0	554.4	1792.4	1876.2	0.96	589.1	0.0	1833.7	590.3	32.2%

Loads have been assigned to each transformer distribution based on the process Duty/Standby assignment. The loading on each transformer is dynamic and dependant on which loads are selected as duty loads.

To ensure the electrical infrastructure has capacity to supply all connected duty loads, it is necessary to consider the Standby loads as Duty loads on the individual distribution systems. Table 8-3 calculates the full load rating of the loads connected to each switchboard for this purpose, and the site total shows the true demand load for the site.

Based on this, two 1500kVA transformers have been proposed for the site which will have capacity to run all connected load to each distribution system and be loaded to approximately 62% during normal operation. During extreme circumstances, it may be possible to run the majority of the plant from one transformer while rectifying a fault on the other supply, due to the Duty/Standby loads being split across the boards.

The power factor and harmonic distortion is calculated on the same basis and are included in the summary table. The power factor is calculated at each switchboard, and overall has a calculated power factor of 0.96 lagging and therefore there is no dedicated power factor correction equipment proposed for this project. The good power factor is predominately due to the large number of VSDs used in the project which return good front end power factor regardless of the connected motor.

The harmonic current expected to be generated by these VSDs and injected into each distribution system is in the order of 740 Amps on MSB100, and 831 Amps on MSB200 with all loads running. Under normal operation, the expected site harmonic load is calculated to be 590 Amps. For this Concept design, a 300A active harmonic filter has been included on each MSB, which will have capacity for full correction under normal operation, and partial correction under the worst-case loading.

8.6 Switchrooms and Site Arrangements

Due to the size of the site, there are three switchrooms in the design which are located at beneficial locations across the site to service the power and control needs of the local equipment. The switchrooms are identified in the design as the Main Switchroom, Dewatering Switchroom and Inlet Switchroom.

The switchroom designs are similar, with the MCCs installed in the centre of each switchroom with the duty/standby MCCs installed back-to-back. The VSDs, distribution boards and ancillary equipment are installed along the switchroom walls. The MSBs in the main switchroom are installed along the switchroom wall to provide easy access for the large incoming cables from the supply transformers and reduce overloading the underfloor cabling system.

Cabling between the switchrooms is via underground conduits, and the main routes are indicated on the Civil drawings.

8.7 Control and Communications

The switchrooms and control room are connected via 24 core single mode optical fibre cable designed in a redundant ring architecture. The SCADA Network and the Control Network are physically segregated networks on the fibre ring, with dedicated fibre pairs allocated to each.

Each MCC contains a dedicated PLC panel which is used to marshal I/O within that MCC to a Remote IO (RIO) PLC rack. A control network switch connects the smart starters, power meter and any other communications equipment related to the individual MCC.

Each switchroom has a stand-alone PLC panel containing RIO racks and communications equipment, which is used for all of the field-mounted process instrumentation and valves, and switchroom-mounted equipment such as VSDs.

Remote access to the plant will be provided via Council's secure network and a dedicated site VPN connection. This will allow operators to access the plant SCADA remotely for detailed monitoring, control and alarm management.

A provision has been included for an offsite radio link to connect the plant to Council's main Head Office SCADA system. This will allow critical plant status information to be displayed and monitored remotely. Further development will be undertaken during the Reference Design, including desktop radio path survey.

8.8 Site Lighting

All light fittings will be LED for energy efficiency and longevity.

Lighting within buildings will be by motion sensor.

Outdoor general access lighting will be provided for safe access to the plant. Lighting will typically be provided for general transit through plant areas, on stairways and along raised walkways/structures. Operators will operate outdoor lighting via the SCADA. Lighting will be designed to mitigate light noise to the surrounding areas.

8.9 Lightning Protection

During the Reference Design period, an AS1768 Lightning Protection Assessment will be undertaken. As the plant is classified as critical infrastructure under this Standard, the protection measures will likely include surge protection and lightning conductors.

8.10 Climate Change Considerations

Development of the electrical design for the site has considered ways to reduce power use and associated emissions and means to improve resilience to climate change.

Measures adopted in the design include:

- Improving energy efficiency through power quality monitoring, harmonic correction and power factor correction
- Selection of key equipment based on best operating point

- Active power monitoring of drives using Next Generation Operating Systems to monitor efficiency, identify maintenance requirements and optimise energy use
- Providing resilient power supply architecture including two transformers and back-up power generators
- Locating critical electrical equipment above flood level
- Including a connection for future PV generation to connect to the site at the Main Switchboard
- Inclusion of a weather station connection to SCADA to provide real time and historical weather information
- The plant is capable of being operated using field push buttons whenever there is a failure of automatic operation

Further consideration will be given to:

- Use of local onsite PV solar generation for powering the amenities building
- Use of solar hot water heating for amenities buildings.

8.11 Further Design Development

During the development of the electrical Concept Design, several items of design were identified that require further investigation or consideration. These items are captured here, and will be considered during the Reference Design phase:

- Hazardous Area assessment to identify which areas of plant may require classification and development of Hazardous Area Dossier
- CCTV camera system for remote plant monitoring and operation
- Site security system and offsite alarming, including automated access gate

9 Construction Issues and Constraints

It's anticipated that a staged construction approach will be adopted by the construction contractor to enable controlled sequencing of construction, cut-over and commissioning within the construction contract period.

Development of the concept design included consideration of potential construction issues and constraints that would inform the construction program for the upgrade. These issues will be further developed by the reference design, detailed design and technical specifications.

9.1 Constraints

The selected build zone of the STP is to the south and east of the existing treatment plant. This location will largely enable construction to be undertaken within a separate area while existing operation is maintained at the current STP.

Sequencing of the construction of the works will need to give consideration to:

- Maintaining operation of the existing works including treatment of sewage and compliance with the existing environmental requirements throughout construction and commissioning
- The location and space constraints of the build area
- The designated Golden Sun Moth exclusion zone to the south of the proposed build area
- Safe and efficient construction sequence and allocation of works
- Location of existing power supply services and timing of high voltage upgrade work by EvoEnergy
- Procurement times for long-lead time equipment.

9.2 Early Works

Early works to be undertaken prior to construction include:

- Testing of trial concrete mixes pre-contract so results are available to the tenderers
- Reuse of existing biosolids stockpiles for future landscaping on site
- Demolishing of existing redundant buildings in Old Nursery area.
- Relocation of services including a number of power poles along Mountain Road and the power supply to the existing site to provide clear site access for construction phase
- Construction of a temporary operational access road to the existing treatment plant to maintain access during the construction period and separate operational and construction traffic as much as possible
- Excavation and installation of the Morisset trunk sewer diversion which runs through the proposed build zone from the existing sewer to the proposed inlet works
- Upgrade Mountain Road to support construction traffic
- Establishment of the construction compound.

A layout of the early works has been provided in Figure 9-1.

Figure 9-1 Layout of Early Works (showing temporary access road, construction zone and connection to Morisset Trunk Sewer)

9.3 Site Mobilisation and Construction

The key activities during construction of the main plant will include:

- Site establishment and mobilisation
- Installation of environmental controls
- Site shed establishment and lay down areas
- Early procurement of items including:
    - Cast in pipe stubs for major structures
    - Imported equipment (e.g. inlet screens, centrifuges, blowers, etc)
    - Switchboards
- Installation of the Jerrabomberra trunk sewer diversion which runs through the existing works to the proposed inlet works
- Bulk earthworks including excavation for the bioreactor and clarifier structure areas
- Construction of major reinforced concrete liquid retaining structures – inlet works, bioreactor, clarifiers, filters, digesters.
- Construction of electrical switchrooms
- Plant roads, pumping stations, chemical storage area, hardstands and buildings
- Yard pipework, electrical and instrumentation
- Connect to new power supply
- Hydrostatic testing
- Mechanical testing
- Cut over of sewage flows and commissioning of whole plant
- Final finish to the upgrade to Mountain Road
- Demobilisation including restoration of site roads, drainage and final landscaping.

A schematic of the earth works has been provided in Figure 9-2. Structures have been located so switchrooms are outside the excavation zone and can be commenced early to avoid construction program constraints.

Figure 9-2 Schematic of Bulk Earthworks

9.4 Decommissioning and Modification of the Maturation Ponds

Decommissioning and modification of the maturation ponds will take place following cut-over of sewage to the new STP. It is important to note that the existing maturation ponds are registered dams under the ACT Utilities (Technical Regulation) Act 2014 and will need to be officially decommissioned.

The proposed construction sequence for decommissioning and remediation of the maturation pond areas is outlined below.

- Inflow to the maturation ponds from the existing STP will discontinue once the new STP is commissioned.
- Treated effluent from Maturation Ponds 1 and 2 will be drawn down and discharged to the Molonglo River from the existing discharge point at Pond 3. This may be done by isolating Ponds 1 and 2 from Pond 3 and then pumping effluent from Pond 2 to Pond 3 to maintain the gravity discharge to the river. The pumping rate will be selected to maintain a hydraulic residence time suitable for continued settling and treatment of the effluent. Testing of the effluent discharge will be undertaken to ensure compliance with existing effluent quality requirements. Fish stock within Ponds 1 & 2 would need to be collected and removed from site to a suitable facility off site. Pumping from Pond 2 will cease as the sludge blanket interface is reached.
- Pond 3 will then be drawn down using pumping to discharge treated effluent to the Molonglo River from the existing discharge point. Testing of the effluent discharge will be undertaken for compliance with existing effluent quality licence requirements. Pumping

from ponds will cease as the sludge blanket interface is reached. Exotic fish stock from Pond 3 will also need to be collected and removed.

- The maturation ponds are expected to contain a layer of stabilised organic sludge that has accumulated through years of operation. The sludge may also contain ferric and other chemicals used in the treatment process. Liquid sludge will be removed using contract dewatering equipment. Remaining sludge that cannot be removed from the lagoons using dewatering equipment will likely need to be blended and moisture conditioned with clean fill from the site, then removed using an excavator and stockpiled for further drying. This blended material as well as dewatered sludge will then be used on-site as part of the remedial and landscaping works. For on-site placement it is expected that the site material will be classified as General Solid Waste.
- Vegetation will be removed from the existing embankment where required and stockpiled for remediation of the site.
- The existing embankments are proposed to be modified in several locations to form a broad crest along each maturation pond wall. This bank re-shaping approach will allow the flood water to rise and attenuate and remove the water retaining properties of the pond areas.
- The material from the embankment modifications and excess fill from the STP upgrade will be used as filling material for the ponds to ensure the pond areas are no longer water retaining and allow free drain to the adjacent river.
- Revegetation of the banks will be undertaken to extend the riparian zone of the riverbank. The extent of weed management along the banks is to be confirmed.
- Acid sulphate soil management of the earthworks is to confirmed noting that ASS mapping indicates that ASS and potentially ASS are classified as a low probability of occurrence.
- Asbestos clearance requirements to be confirmed
- Contamination testing extent to be confirmed.
- All earthworks will adopt AS3798 commercial criteria (Level 1 inspection & testing).
- Sediment and erosion control measures would be installed prior to excavation and construction works and would be maintained in an effective condition until earthworks have been completed and the sites are rehabilitated. Sediment and erosion control devices would be inspected regularly and after rainfall events, maintained to ensure effectiveness over the entire duration of the Project, and cleaned out before 30% capacity is reached. Sediment control devices (e.g. silt fences, straw bales wrapped in geotextile etc.) would be installed parallel with the contours of the site and immediately downslope of any areas where the natural ground surface has been disturbed.
- Rainfall and any encountered groundwater seepage would be permitted to evaporate or if required, pumped and passed through erosion and sediment controls (sediment filters and traps, barley bales) and across grassed areas prior to discharge.
- Any spoil storage areas or stockpiles would have appropriate erosion control devices installed to control runoff and prevent sedimentation. Silt barriers/sediment filters/traps around any stockpiles of soil would be provided to prevent the loss of material.
- Dust suppression techniques would be employed where required.
- Official decommissioning of maturation ponds as registered dams under the ACT Utilities (Technical Regulation) Act 2014.

9.5 Decommissioning of Existing STP

The project does not currently include decommissioning of the existing STP. A desktop decommissioning plan will be prepared for the existing plant in the next phase of this project.

10 Sustainability

QPRC has adopted the Infrastructure Sustainability Council of Australia (ISCA) Rating Tool version 1.2 and aims to achieve an “Excellent" Design and As-Built Rating for the STP with a score in the range of 65 to 75.

As part of this process, the team brainstormed a long-list of sustainability initiatives to embed in design at a “Sustainability in Design” workshop held on 15 May 2019 and short-listed initiatives in another workshop on 25 March 2020. Table 10-1 outlines how the ISCA criteria have been incorporated into the concept design for the relevant ISCA credits.

Table 10-1 ISCA Tracking Against Current Design

Credit	Credit Name	Reference with this report	Comments
Cli-2	Adaptation measures	Section 8.10	The concept design has included adaptation measures to address climate risks that were identified at the climate change risk assessment workshop.
Ene-1	Energy and carbon monitoring and reduction	Section 7.1.1 and Section 8.10	The concept design has included initiatives to improve energy efficiency, some of which are outlined in Section 7.1.1 and Section 8.10.
Ene-2	Use of Renewable Energy	Section 8.10	A connection has been included in the main switchboard to connect potential future PV generation.
Wat-2	Replace Potable Water	Section 3.4 and Section 6.12	Recycled water produced by the plant will be used onsite for treatment processes (used as sprays or flows to enclosed controlled areas such as plant inlet works screens) and to site hose reels for washdown.
Mat-1	Materials lifecycle impact measurement and reduction		Reducing materials use, in particular through detailed design of structures and structural steel work. Material selection to avoid composite materials where possible and maximise use of recyclable materials.
Dis-1	Receiving Water Quality	Section 3.2.1	The concept design has been developed so that the Queanbeyan STP meets the ACT EPA discharge licence limits for wastewater effluent (note that it has also highlighted areas of risk in maintaining this licence)
Dis-4	Air Quality	Section 6.1	Odour dispersion modelling is being undertaken for the concept design.
Lan-2	Conservation of on-site resources	Section 9.4	Excess fill (including topsoil) from the STP upgrade will be used as fill for the maturation ponds that will be decommissioned as part of the existing STP.
Was-1	Waste management	Section 3.5	The sludge handling and digestion facilities of the upgrade are designed to maintain stabilisation grade B which categorises them for Restricted Use 2 based on the current guidelines.

Credit	Credit Name	Reference with this report	Comments
Was-3	Deconstruction / Disassembly / Adaptability	Section 3.1.1	The concept design has incorporated considerations for future expansion. It has also considered how the process may be modified to meet potential tightening of phosphorous limits in the future.
Eco-1	Ecological value	Executive Summary	Site layout has been informed by site investigations including ecology especially regarding the existing habitat of the Golden Sun Moth
Her-1	Heritage assessment and management	Executive Summary	Site layout has been informed by site investigations including heritage
Inn-1	Innovation strategies and technologies	Section 8.10 Section 6.6.1 Section 6.2 Section 8.10	There has been significant innovation in the design so far including: ▪ Smart energy management system ▪ Eliminated the need for mechanical mixing of anaerobic zone ▪ WAS returned to the inlet works for rescreening ▪ Next Gen Operating System

11 Safety in Design

As part of the development of this concept design, Hunter H2O has adopted an approach of progressive consultation with QPRC about how the risks associated with construction, operation and decommissioning of this upgrade can be eliminated or minimised as far as reasonably practicable.

Safety in design activities during the design development to date has included:

- Identifying lessons learnt recommendations from QPRC's existing operational sites and previous projects delivered by Hunter H2O
- Design development reviews considering constructability, operation and maintenance and access requirements
- Review of criticality and redundancy including a preliminary HAZOP of the concept P&ID
- Including QPRC operational staff in design reviews
- A concept phase safety in design workshop on 19-20 February 2020.

Safety in design aspects of the upgrade will be developed further during the Reference and Detailed Design phases of the project through:

- A formal risk review CHAIR workshop
- A HAZOP on the completed design P&IDs
- Consultation with the construction contractor on project specific safety issues and constructability as part of the planned early tenderer involvement (ETI) process.

A Safety in Design Report for the Concept Design is provided in Appendix F.

12 Cost Estimate

A risk-based engineering cost estimate for the project has been prepared in conjunction with this Concept Design.

At the time of this concept design, the construction value of the works is estimated to be in the order of $100m. This is in line with expectations for a plant of this capacity and QRPC's previous planning for this project. This cost estimate is subject to change over time as the project scope is further defined through the design process. A revised cost estimate will be provided during the reference design phase.

The cost estimate for construction has been prepared based on the proposed plant in this report. Detailed quantities have been extracted from the detailed 3D Navisworks model of the proposed upgrade. Costs have been developed using the following primarily first principle methods and the following sources:

- Rawlinson's Construction Handbook 2020 and other first principle estimating tools
- Supplier quotes sourced specifically for the proposed upgrade and this estimate
- Known contract rates and quotes from previous relevant wastewater plant construction projects
- Rates from independent estimator and contractor databases.

Where appropriate, Building Price Indices have been applied to bring rates inline with financially current values. An estimate for electrical, control and instrumentation (ECI) costs have been made using 20% of the total civil and mechanical works which is consistent with a design of this level.

The cost estimate is based on the procurement model indicated by QPRC and assumes that the project will be delivered in Construct Only model by a prime contractor.

Estimates of the operating cost for the plant have been provided as part of the Option Selection Report.

13 Procurement

13.1 Background

The procurement strategy for the project has been developed through procurement strategy workshops held in Queanbeyan on 14 March 2019 and in Newcastle on 22 August 2019.

These workshops were attended by key personnel from Council and considered:

- Project objectives, characteristics and risks
- Various contract and tendering systems including their advantages and disadvantages
- The capacity and capability of both the market and Council to deliver the project.

13.2 Procurement Strategy

The key features of the procurement strategy for delivery of the Queanbeyan STP Upgrade project are:

- Hunter H2O will prepare a concept design, reference design, detailed design and tender documentation for the works
- The upgrade will be undertaken as a Construct Only contract using the NSW Government GC21 General Conditions of Contract
- A separate early works contract may be used to establish a temporary access road and the construction compound prior to the main STP Upgrade contract
- Council will invite open Expressions of Interest for the purpose of establishing a list of prequalified tenderers which will tender for the construction contract
- The prequalified tenderers will participate in an Early Tender Involvement process to ensure appropriate allocation of risk (technical and commercial) and provide input to constructability issues
- Tenders for Construct contract will be evaluated based on price and non-price evaluation criteria.

13.3 Preliminary Procurement Program

Key milestones for the delivery of the Queanbeyan STP Upgrade project are shown in Table 13-1. These dates are for initial planning purposes and will be refined as the project progresses.

Table 13-1 Key Delivery Milestones for Queanbeyan STP Upgrade Project

Milestone	Current estimate
Concept design approval	May 2020
Draft EIS	Nov 2020
Final EIS	Jul 2021
Detailed design complete	Nov 2021
Contract award	Jun 2022
Construction	Jul 2022 – Jun 2024
Decommissioning of the maturation ponds	Dec 2024

14 Further Design Development

The design presented in this concept will be further developed by Hunter H2O through the preparation of a Reference Design and subsequent detailed design for construction. Further studies and design development to be undertaken during the Reference Design includes:

- Final structural sizing and site arrangement layout adjustments to take into account final equipment selection as well as safety, operational and maintenance requirements
- Assessment of Building Code of Australia requirements and confirmation of firefighting requirements
- Further geotechnical investigation
- Additional location of services
- Equipment selection for design
- Confirmation of materials for construction
- Optimisation of pipeline hydraulics
- Further development of trunk main connections and on-bank effluent outfall
- Value management and sustainability reviews
- Safety in design reviews of construction, operation and maintenance requirements
- Plant yard pipework
- Road and stormwater design
- Potable & recycled water ring mains
- Electrical transformer conduits, diesel fuel storage and generator details
- Further development of riverbank and ponds decommissioning.

15 References

ACT Government. (2019). *Strategic Bushfire Management Plan 2019-2024.*

Arup. (August 2019). *Queanbeyan STP Upgrade: Factual Geotechnical Investigation.*

Arup. (August 2019). *Queanbeyan STP Upgrade: Site Investigation – Geotechnical Desktop Study.*

Douglas Partners. (March 2020). *Queanbeyan Sewage Treatment Plant Upgrade: Factual Report on Geotechnical Investigation.*

Douglas Partners. (March 2020). *Queanbeyan Sewage Treatment Plant Upgrade: Interpretive Report on Geotechnical Investigation.*

Environment Protection Authority. (2000). *Environmental Guidelines: Use and Disposal of Biosolids Products.* Sydney: Environment Protection Authority.

Hunter H2O. (December 2019). Minutes of the QSTP Site Arrangment Workshop.

Hunter H2O. (December 2019). *Queanbeyan Sewage Treatment Plant Upgrade Project: Design Criteria and Assumptions Report, Revision C.*

Hunter H2O. (June 2019). *Queanbeyan STP Upgrade Project: Durability Plan Rev A.*

Hunter H2O. (November 2019). *Queanbeyan Sewage Treatment Plant Upgrade Project: Option Selection Report, Rev B .*

Hunter Water Australia. (January 2011). *Queanbeyan WWTP: Maturation Pond Embankment Investigation.*

NRMMC. (2006). *Australian Guidelines for Water Recycling (AGWR).* Canberra, Australia: Biotext Pty Ltd.

NSW Department of Primary Industries, Office of Water. (May 2015). *NSW Guidelines for Recycled Water Management Systems.*

Public Works Advisory. (March 2019). *Queanbeyan Sewage Treatment Plant Maturation Ponds: Condition and Consequence Assessment.*

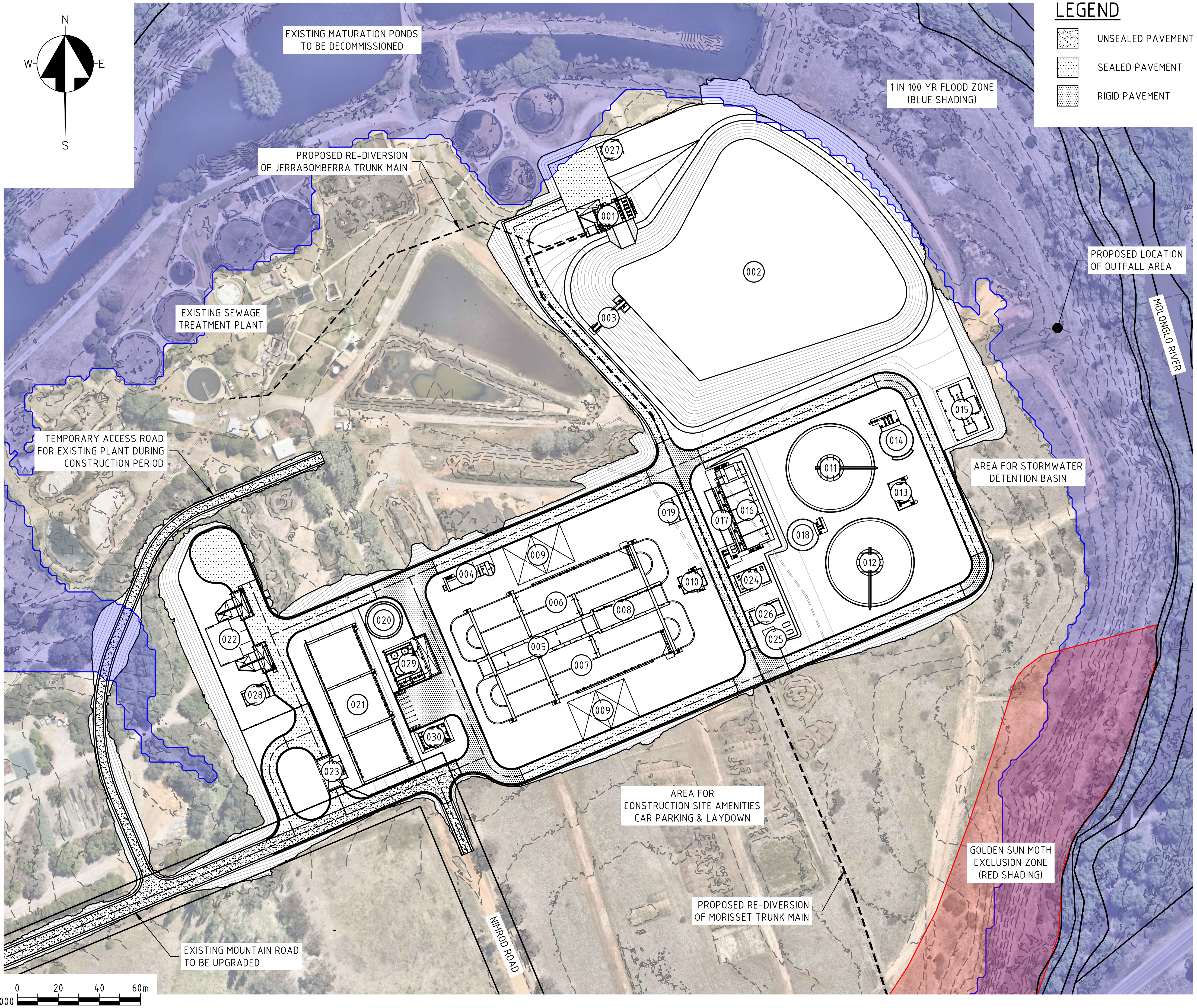
Queanbeyan Palerang Regional Council. (March 2018). *QPRC Liquid Trade Waste Policy.*

Queanbeyan-Palerang Regional Council. (March 2019). *Queanbeyan Flood Plain Risk Management Study and Plan.*

Safe Work Australia. (2013). *Workplace Exposure Standards for Airborne Contaminants.* Canberra.

Senversa. (October 2019). *Preliminary Contamination Assessment: Block 27 Nimrod Road Jerrabomberra ACT.*

Appendix A Site Arrangement



STRUCTURE LIST

ITEM NUMBER	DESCRIPTION
001	INLET WORKS
002	STORM POND (30 ML)
003	STORM RETURN PUMPING STATION
004	GRIT REMOVAL FACILITY
005	ANAEROBIC ZONES
006	OXIDATION DITCH 1
007	OXIDATION DITCH 2
008	FINAL AEROBIC ZONES
009	AERATION GRID WASH DOWN AREA
010	MIXED LIQUOR SPLITTER
011	CLARIFIER 1
012	CLARIFIER 2
013	RAS PUMPING STATION
014	FILTER LIFT PUMPING STATION
015	UV DISINFECTION FACILITY
016	DAFF (DISSOLVED AIR FLOTATION FILTERS)
017	CLEAR WATER TANK
018	DIRTY BACKWASH TANK
019	POLYMER DOSING FACILITY
020	WAS THICKENER
021	AEROBIC DIGESTER
022	DEWATERING FACILITY
023	SEPTAGE RECEIVAL
024	BLOWER FACILITY
025	HV ELECTRICAL AREA
026	MAIN SWITCHROOM
027	INLET SWITCHROOM
028	SLUDGE HANDLING SWITCHROOM
029	CHEMICAL DOSING FACILITY
030	AMENITIES

AMENDMENTS

Ver	Date	Description	Drawn
B	23/10/20	CONCEPT	GG
A	14/04/20	CONCEPT	GG

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Drawn GG	Checked RKB
Approved JLS	
Date 14/04/20	
HH2O Project Number 5431	

QPRC

QUEANBEYAN-PALERANG REGIONAL COUNCIL
QUEANBEYAN STP UPGRADE

SITE ARRANGEMENT

Scale 1:2000 at A3

Document No **QSTP-HH2O-DWG-CV-XX-XX-006**

Status **DRAFT**

Version **B**

Plot Date: 23/10/20 - 10:39 Cad File: P:\QPRC\5431 Queanbeyan STP Upgrade\2. Tasks\0. Drafting\1 - CURRENT WORKING\QSTP-HH2O-DWG-CV-XX-XX-006 (Site Arrangement - Stage 1).dwg

Ver.1.00

Appendix B Flow Balance and Hydraulic Profile

Included in the full design report

Appendix C Process Flow and Process and Instrumentation Diagrams

Included in the full design report

Appendix D Navisworks Model Screenshots

Included in the full design report

Appendix E Electrical Drawings

Included in the full design report

Appendix F Safety in Design Summary

Included in the full design report

Appendix G Maximum Power Demand Calculation

Included in the full design report

Appendix H Cost Estimate

A copy of the engineering cost estimate has been provided to QPRC.

Appendix I Geotechnical Investigation – Factual Report

Included in the full design report

Appendix J Geotechnical Investigation – Interpretive Report

Included in the full design report

Appendix K UVT Jar Testing Report

Included in the full design report

Appendix L ACT EPA Feedback on Concept Design

ACT Government
Chief Minister, Treasury and Economic Development

Our ref: 20/21968

Mr Phil Hansen
General Manager Community Connection
Queanbeyan-Palerang Regional Council
PO Box 90
QUEANBEYAN NSW 2620

Dear Mr Hansen

Queanbeyan Sewage Treatment Plant Upgrade – Concept Design Report

Thank you for the opportunity to review and provide comment on the Queanbeyan Sewage Treatment Plant (QSTP) Upgrade Concept Design Report (CDR) Draft A and be involved in the workshop on the CDR held on 28 April 2020.

The EPA notes the design report shows that the proposed upgraded plant may match the existing plants performance and will be able to cater for the growth of Queanbeyan. There is still some uncertainty regarding the critical parameter of phosphorus. Additionally, there were some issues flagged regarding other limits in the current Authorisation and with the location of the measurement of effluent characteristics. However, it was stated in the workshop that the design consultants for QPRC are currently collecting an extensive dataset of water quality parameters (upstream/downstream and within plant). Analysis of the data can be used to help resolve the phosphorus issue and to support proposals to vary the Authorisation so that plant processes can be optimised whilst maintaining protection of the aquatic environment.

The workshop discussions on the concept design indicate scope for some variance on elements within the plant. The consideration of such changes in the detailed design phase must not compromise the concept design outcomes. An example of such a consideration is changes to the screening arrangements and potential for discharge of large sewage components to the river during times of high plant flow. The design should ensure that any discharge of partially treated or untreated sewage cannot be recognised visually as having originated from the QSTP.

Partial treatment events are considered undesirable and the duration and frequency of partial treatment events that are likely to happen from the upgraded plant will require further articulation to better understand if the events may cause environmental impact. The design consultant, Hunter H2O, has demonstrated that a sewage treatment plant having a capacity of up to 150,000 EP can be accommodated on QPRC's existing site.

The upgrade of the QSTP is a major project and it is pleasing to see QPRC progressing it in a diligent manner for the benefit of the community. I appreciate the opportunities that QPRC has afforded EPA representatives to be involved in the planning processes.

Further comment will be provided through the planning process, including on the Development Application as more details become available.

Please contact Mr Heath Chester, Assistant Director Water Regulation, on (02) 6207 5728 for further assistance.

Yours sincerely

Ben Green
Environment Protection Authority

12 May 2020

cc: Mr Ben Ponton
Director-General
ACT Environment, Planning and Sustainable Development Directorate

cc: Mr Robert Mitchell
Principal Technical Assessor
NSW Department of Planning, Industry and Environment

cc: Mr David Perry
Principal Engineer
HunterH20

Appendix M NSW DPIE Feedback and Response

5 June 2020

Robert Mitchell

Principal Technical Assessor, Water & Sewage

NSW Department of Planning, Industry and Environment

robert.mitchell@dpie.nsw.gov.au

RE: Queanbeyan STP Upgrade Project – DPIE Notes on Concept Design

Dear Robert,

Thank you for providing your notes on the Queanbeyan STP draft concept design (by email 4 May 2020) and for meeting with us to discuss this feedback (20 May 2020). This letter summarises how the strategic comments provided in your notes will be addressed as part of further development of the proposed upgrade during the Reference Design.

A. Plant Staging

The site arrangement presented in the draft concept design report gave consideration to the space requirements of a future expansion to an ultimate plant capacity of 150,000 EP (referred to in the draft concept as Stage 2) based on duplicating the 75,000 EP treatment plant structures. The site arrangement for this ultimate capacity was presented to demonstrate that there was sufficient space on site.

We agree that both the timing and staging of any upgrade to QSTP beyond the current project is uncertain. The ultimate capacity of 150,000 EP is also intentionally a conservative estimate of the maximum population that could be served by QSTP.

We have changed the language in the final concept design report to more clearly communicate the uncertainty of future upgrade staging to readers and to refer to the 150,000 EP treatment plant as an 'ultimate capacity' rather than 'Stage 2'.

We acknowledge your comment that the next upgrade to the QSTP may be a smaller upgrade than presented in the concept design (e.g. a 50% upgrade in capacity that adds a third oxidation ditch, third clarifier etc).

We will give further consideration to potential flow splitting arrangements required to facilitate a 50% increase in capacity and your suggestion to relocate the grit facility to a more central location. Any decision to include capital works now to facilitate a future upgrade would be reviewed by QPRC as part of value management.

B. Inlet Works

Your initial feedback comments on the inlet works and screening arrangement are noted.

As discussed with you on 20 May 2020, QPRC's key considerations for providing automatic screening for all flows arriving at the works is:

- to reduce the risk of screening material being discharge to the Molonglo River during a storm pond overflow event
- to reduce the risk of screening material from a storm pond overflow blocking the on-bank discharge structure and
- to reduce the potential accumulation of screening material in the Storm Pond, which introduces an avoidable manual handling task for operators to remove screening material from the pond after wet weather events.

The upgrade will retain automatic screening of all flows to the works prior to the Inlet Works Pumping Station and Storm Pond as presented in the Concept Design.

As a value management measure, we will reduce the size of the automatic inlet screens based on providing screening capacity for the PIF using 3 operating screens (rather than 2 + a standby as presented in the draft concept).

Further consideration will be given to your suggestion to add a manually raked bar screen to the overflow from the Storm Pond to reduce the risk of the discharge structure being blocked by leaf matter etc.

C. Waste Pump Stations

The concept for the upgrade includes numerous pump stations (Septage Pumping Station, Main Foul Water Pumping Station, Inlet Foul Water Pumping Station, Scum Pumping Station, Dirty Backwash Pumps etc).

We will review the number of pump stations and consider options to simplify operation by consolidating pump stations or removing pumps. (As an example, we agree with your suggestion to review whether the Backwash Pumps can be removed from the Dirty Backwash Tank and the dirty backwash returned to the Inlet Works Pumping Station)

D. Aerobic Digester

Your comments and follow up discussion on the solids retention time in the aerobic digesters is noted. Increasing the SRT will provide improved biosolids stability for QPRC.

The SRT of the aerobic digestors will be increased from 20 to 30 days.

E. Plant Layout

We support the suggestion to simply plant pipework wherever possible. As discussed, the proposed location of the clarifiers was based on plant hydraulics and consideration of earthworks required to achieve a final surface level that provided clarifier walls at handrail height for ease of operator inspection.

We will review the option of relocating the filter structure to the area adjacent of the UV treatment system to see if this provides an improved solution.

F. Hydraulic Profile Drawings

More detailed hydraulic profile drawings will be prepared during the Reference Design.

Other Comments

DPIE's additional comments are noted and will be further considered during design development.

In particular we note your suggestions to:

- Simplify the septage receival facility by providing a flow meter, pit and manual screen arrangement similar to the Hunter H2O design at Gunnedah STP
- Review the bioreactor / clarifier splitter weirs and consider an orifice type arrangement
- Review instrumentation on the P&ID based on a consistent approach to control philosophy.

We welcome the opportunity to contact the department from time to time during the Reference Design Phase to update you on how your comments have been considered and addressed.

Thank you again for your support of this important project for Queanbeyan.

Kind regards,

Dr David Perry

Principal Engineer

BE (Chem), PhD, DipPM, MIEAust, CPEng, NER, APEC Engineer, IntPE (Aus)

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QUEANBEYAN STP
NOTES ON CONCEPT DESIGN

As advised at the meeting the overall approach for the upgrade of the Queanbeyan STP is good with the main issues around inlet arrangements and staging. It is important the upgrade provides the required capacity/performance, is relatively easy to operate is cost effective, and also looks to the future in terms of upgrade and expansion. Consequently, the Department would request to be included on discussions around the strategic items below.

Notes have been divided into two sections, strategic and more detailed comments on the concept information. Action items are highlighted.

1. STRATEGIC COMMENTS PROVIDED DURING THE VIDEO WORKSHOP 28/04/20

At the video conference session, the following summarises DPIE's main comments that are relevant the broad / big picture strategy the be adopted in the QSTP concept design:

A. Plant Staging

Duplication of Stage 1 plant in Stage 2 serves to demonstrate that there are no site area constraints on future plant layout. But, there is scope that Stage 1 may down the track prove to have somewhat more than 75,000 EP capacity. In that case, Stage 2 could be reduced in size and addition of another half of Stage 1 (i.e. a third oxidation ditch, third clarifier, etc) would be a more likely scenario. This places a different perspective on the need to identify features that should be incorporated in Stage 1 works to facilitate the incorporation of a reduced Stage 2 plant capacity increment in the future, eg:

- How better the provide shared use of 3 clarifier among available ditches?
- Integration of staged inlet works, ie one big & one small or a consolidated ST1 & ST2 inlet? (we believe this would provide the greatest flexibility for future planning).
- Stage 1 grit removal would not be suitable for Stage 2 – Would centrally located grit removal be a better option? (as per above point)

An assessment of what/how a reduced Stage 2 works (3 ditches) would look like is required. This must consider flow splitting arrangements between units for operational flexibility.

B. Inlet Works

The inlet works (screening, lift pumping and storm attenuation) has a few unusual features.

- Automated fine screening upstream of lift pumping resulting in a very large screening units.
- Issues with screening capture of raw sewage (gross solids from gravity catchment areas).
- Return pumping from storm pond to lift pump well (similar heights).
- Stage 2 layout is a mirror image of Stage 1 (not well explained in terms of how it would work). Ie running two lift PS how is the raw sewage flow is split between stages?

The inlet works could be rationalised/simplified as follows:

- Single lift PS for Stages 1 & 2, ie the lift SPS expandable for a reduced stage 2 and ultimately a full duplication.
- Using the inlet PS to undertake storm return function by lowering the floor level of the station so volumes detained in the storm pond can drain back to the Inlet PS well. Ie an inlet sump in the storm pond.
- The inlet PS should make allowance for future connection of additional sewage main(s) as new mains are likely needed to transport increased sewage flows to the plant in Stage 2 if not Stage 1 as well.

- Consolidation of inlet works treatment functions (screening and grit removal) at the head of the plant once flows are lifted up there by the inlet PS. This means that the inlet works would be sized for 5.5xADWF rather than 9xADWF with a substantial saving on screening and size of equipment. The inlet works may possibly then look at 2 x band screen units (duty /assist) in Stage 1 and include space for a third band screen in Stage 2. Flow division downstream of preliminary treatment could be achieved by a simple flow divider arrangement as opposed to separate pumping stations.
- Inclined coarse bar screens would then be incorporate in the overflow from the Inlet PS to the Storm Pond and also overflow discharge from the Storm Pond.

A revision of the current lift pumping and preliminary treatment arrangement is required. Again attention is required for Stage 2 and the real possibility of an additional process train (not full duplication).

C. Waste Pump Stations

There are 3 pumping station to cater for waste flows such as backwash waste, dewatering liquor and general wastewater. These stations are not insignificant. With the inlet PS located at a low elevation of the site the management of these wastes can be simplified by directing them to the inlet PS via gravity pipelines. This will also minimise power supply sizing as well as standby generator capacity.

Revise waste PS for cost savings and plant simplification.

D. Aerobic Digester

A design solids retention time of 20 days is considered low for a plant which will experience very cold winters. Another plant that is currently being designed by Hunter H2O has an aerobic digester with a design SRT of 40 days, which in addition to a 25 day design sludge age in the upstream activated sludge treatment process, provides a total sludge age of 65 days. Published design criteria have recommended SRTs of 60 days for cold climates.

Design aerobic digester SRT for a minimum of 30 days.

E. Plant Layout

Reason for locating the filters between the oxidation ditches and the clarifiers is unusual and potentially increase complexity of pipework arrangement and pumping filter feed pumping requirements. More practical to locate the filters to the right of the clarifiers, closer to the UV disinfection system.

Also the pipe work needs to flexible to accommodate likely Stage 2 works. Ie various ditch clarifier combinations.

Rationalise location of filters and clarifiers.

F. Hydraulic profile drawing

Septage receival, WAS thickener, aerobic digester, dewatering facility and associated foul water pump station are not shown in the hydraulic profile drawings.

Include in the drawings.

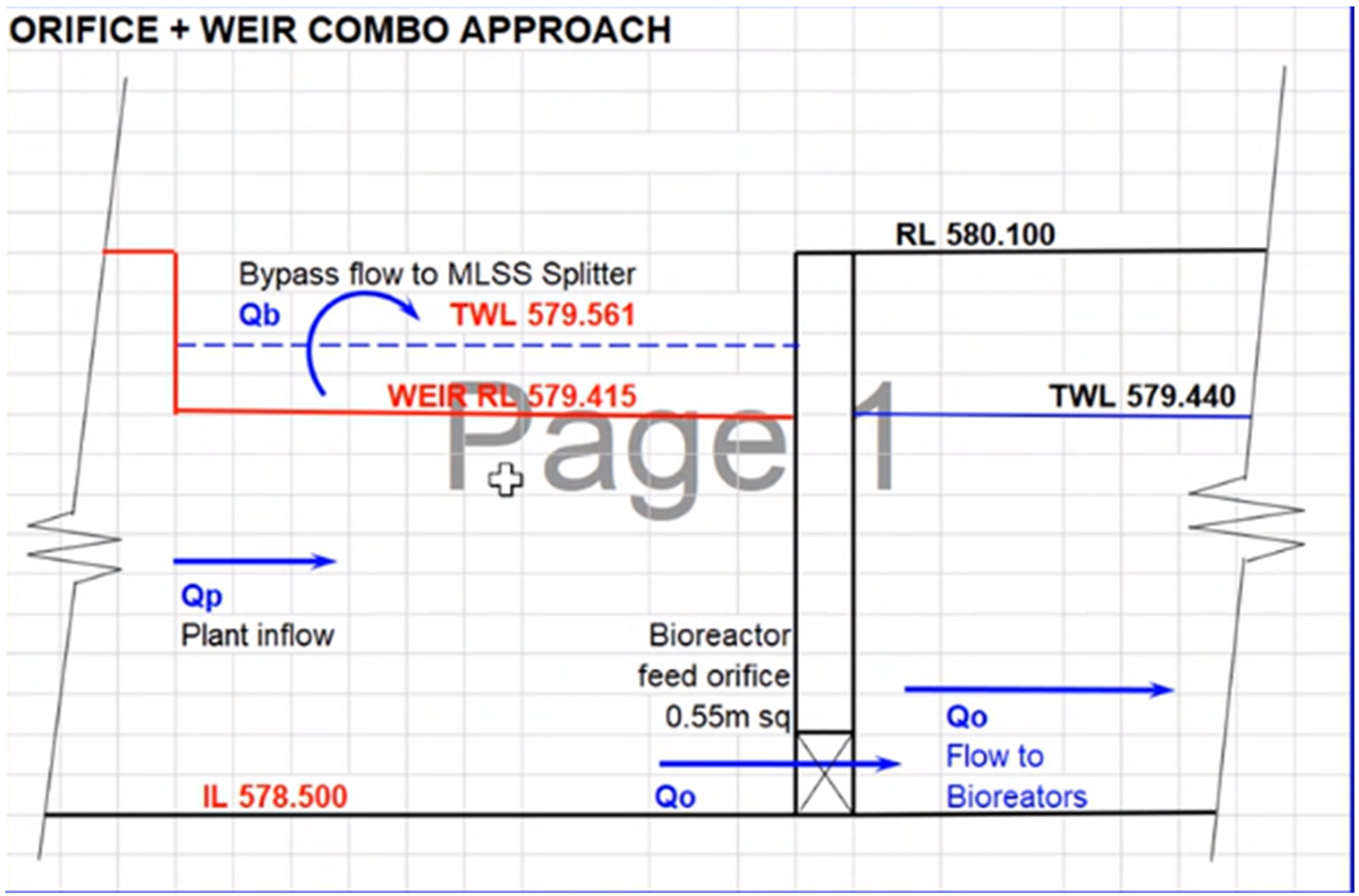
2. OTHER COMMENTS ON CONCEPT DOCUMENTS

DPIE's additional concept review comments presented in the vain of achieving refinements in aspects of plant simplification, economy or enhanced operation are tabulated below:

Ref	Details	COMMENT
Wastewater quality/ plant loads		
S3.1.3	Alkalinity based on limited data.	Site sewage sampling required.
S3.1.3	pH is typical value not QSTP data.	
S3.1.3	Septage quality – septage can be somewhat stronger than the values given. Further, have taken lower end of the concentration ranges.	Based on comments by QPRC, the 150 kL weekly septage volume allowed should be reduced proportionate to increased septage strength allowances. (refer to the DLG silver book '*On-site Sewage Management for Single Households'*).
Hydraulics		
S5.2	Velocity < 1m/s at peak flow so must be pretty low at average flows.	This suggests that velocities in average or low flow periods are likely to be quite low – should indicate what they are to see if they are suitable.
Plant operational configurations		
S5.2	Operator to set peak flow limits	Once units taken off line it would be expected that SCADA would automatically adopt appropriate limits rather than have Operator set them
Septage Reception		
S6.2	Manual and mechanical fine screens for septage reception	Practicality of auto screen for septage use and if so if they should be course size rather than fine.
Inlet Works PS		
S6.3	Mixer in Inlet Works PS	Any need for mixer – presumably no mixers in network PSs
S6.3	5xIWks pumps with VSDs	IWPS needs to accommodate twin duties of 5.5xADWF and 3xADWF being secondary bypass and no-bypass situations. Multiple pumps with VSDs are proposed to meet this. But should provide max. and minimum output flows with1, 2, 3 and 4 pump operating scenarios to demonstrate this.
Storm Return PS		
S6.3	Return PS discharge flows are 150 L/s minimum up to 350 L/s.	Corresponds to 0.75 to 1.75xADWF – so how do you get design capacity of 2xADWF?
Bioreactors		
S6.6	Bioreactor/clarifier split with 2x11.5m weirs	DPIE calculations show that 2 x11.5m weir is insufficient to bypass 2.1 x ADWF. Longer weirs appear a poor solution. Alternatively a orifice/ bypass weir combination would provide a in the order 2 x 3.2m weirs at the cost of deepening of the floor level. (refer to attached sketch)
S6.7	9xanaerobic zones all taken offline together.	Would be handy if can only a few can be offline rather than all cells.
S6.7	Removal of air grids form one ditch to other	Likely high cranage reach requirement

Filtration		
S6.9	50 L/s minimum feed pump flow is given.	Need 600 L/s for 3xADWF design flow capacity – so how do we get down to 50 L/s (turn down to 14%)?
S6.9	Filter run times	What are filter run times at peak load of 3xADWF?
Effluent reuse		
S6.12	LRVs for reuse	FYI draft AGWR revisions are heading towards LRV targets for dust suppression municipal reuse of V=5.0, P=4.5 and B= 5.0 but these don't significantly impact on proposals (v target achieved of 4.9 log is nominally short)
S6.12	Quality/treatment requirements for on-site reuse	DPIE consider that, given that operators can variously be exposed to sewage, septage, mixed liquor, sludge, etc. the RE water for on-site use should be sourced form the best quality effluent otherwise achieved from the plant. This would be ex the UV outlet if there is no of-site reuse or ex the chlorine storage tank supplying the standpipe for tankers for off-site dust suppression use.
Hydraulic Prifile		
B-QBal/ HProf	MLSS Splitter – TWL depth 3.08m Weir height 2.75m	Does splitter need to be this deep
B-QBal HProf	Morriset PS 900 L/s Jerrabombera 900 L/s	Same flow both inlet mains – coincidence or is one actually bigger
Process flow diagram		
PFD	Scum from ditches goes to WAS thickener and clarifier scum goes direct to digesters	Scum should all discharge to one place
P&IDs - Various		
PI-007	Mixer in septage PS well	Is the mixer really required – see comment for IWPS below.
PI-007	Screens (manual + fine) with no overflow bypass	Include high level overflow if screens blocked
PI-009	2xlevel sensors	Would 1 suffice
PI-009	Mixer in inlet PS well	Is the mixer really required. This PS is essentially the same as a catchment PS which we assume have no mixers and operate OK.
PI-011	Storm Pond has LSL as sole level setting	Need LSH level setting for 10 ML control of 3x vs 5.5x IWPS operation and LSHH (high) level setting for overflow monitoring/alarm
PI-012	Bache Trough receiving grit before classifier	Needed or not??
PI-012	No switches/alarms on grit classifier	Needed or not??
PI-014	8xanaerobic cells – but report has 9	Final anaerobic cell should be labelled.
PI-014	No instruments in anaerobic cell	Some (say 2) Redox probe(s) would be handy.
PI-014	No pH probes in ditches	A pH probe in each ditch or in downstream aerobic cell would be handy.
PI-015	Provision for WAS pumping to receival pit	WAS only to thicken not required to receival pit

PI-016	Large clarifiers may benefit from spray sprays	Include surface sprays
PI-019	UV 1 inlet chamber and separate UV 2 inlet chamber	Could have a combined/common inlet chamber
PI-019	pH probe is shown for receiving water	What or receiving water monitoring requirements
PI-022	Turbidity meter is on liners to DRTBW	Add turbidity meter on lines to CWT- would normally expect filtered water turbidity monitored for each filter cell
PI-027	Have CWT to Recycle Water line	RE offtake should be from UV outlet
PI-028	Mixer included in DWBWPS	Would not expect mixer to be required.
PI-029	No pressure switch(es) on pressure vessel	Would expect PSL and PSH
PI-029	Source RE water should be from UV outlet	RE offtake should be from UV outlet
PI-029	No scour on RE storage tank	Include scour outlet
PI-030	No SS sensor in WAS thickener	Include SS probe
PI-031	No SS sensor in digesters	Include SS probe
PI-031	No pH sensor in digesters	Include pH probe
PI-032	Transfer poly from IBC to storage tank	Appears manual transfer
PI-032	3xmixers in poly blending tank	Why not single mixer – wouldn't expect it to be an overly large tank
PI-033	Poly dose to WAS thickener	Is poly needed for gravity thickening to 1.25%
PI-034	No drive/sensors/instruments on horizontal conveyor	Include drive/sensors/instruments
PI-041	Filter aid uses powdered poly	Is liquid poly possible instead so that all poly across site is liquid.
PI-043	RPZs shown on potable service to individual process units	Provide single RPZ then feed to various process units – cost/maintenance savings
P&IDs - Flow meters		
PI-008	Flow meter on amenities ablutions	Need consistent/rational flow metering details, eg Major/critical PS = flow meter with valved bypass. Minor PS (eg foul, scum) = flow meter. Inconsequential PS (eg ablutions) = None. Chemical dosing = flow meter. Plant discharge lines = flow monitoring on pond overflow, plant bypass, UV bypass, UV outlet discharge streams.
PI-011	No flow monitoring of pond overflow discharges	
PI-013	No flow meter on PS discharge	
PI-017	No valved bypass around flow meter	
PI-018	No flow meter on Scum PS discharge	
PI-019	No valved bypass around feed PS flow meter	
PI-019	No flow meter on river discharge	
PI-029	No flow meter on RE use	
PI-036	No flow meters on air scour delivery to filters	
PI-040	No flow meter on hypo dosing line	
PI-041	No flow meter on filter poly dosing line	


ORIFICE + WEIR COMBO APPROACH
Bypass flow to MLSS Splitter
Qb
TWL 579.561
WEIR RL 579.415
RL 580.100
TWL 579.440
Page 1
Qp
Plant inflow
Bioreactor
feed orifice
0.55m sq
Qo
Flow to
Bioreators
IL 578.500
Qo