Appendix B

Concept design report

Googong Township water cycle project

Environmental Assesment

November 2010

Googong Integrated Water Cycle

Water and Wastewater Concept Design

Volume 1 – Main Report

Prepared for CIC Australia

Revision 9

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REVISION SCHEDULE

Rev No.	Date	Description	Prepared By	Reviewed By	Approved By
0	30/10/09	Final	SK, AM	PC	RP
1	30/11/09	SPS1 mods	RP	SK	RP
2	18/12/09	Peer Review	SK	RP	RP
3	22/12/09	Peer Review	RP	AJ	RP
4	05/02/01	Biosolids	AM	SK	RP
5	12/5/10	Site Update	AM	Susan Kitching	RP
6	12/5/10	VM Mods	AM	Susan Kitching	RP
7	20/7/10	VM Mods	PRP, AW	AJ, SK	RP
8	17/8/10	CIC Australia Review	AW	PRP	PRP
9	8/10/10	Minor Comments	AW	PRP	PRP

Glossary / Acronyms

Australian Capital Territory Environment and Water Corporation Ltd.	ACTEW
Australian Height Datum	AHD
Actuated inlet control valves	AICV
Alarm top water level	ATWL
A-Mixed liquor recycle	A-MLR
Anti-slammer air valves	ASAV
Average dry weather flow	ADWF
Average recurrence interval	ARI
Biological oxygen demand	BOD
Bulk water pumping station	BWPS
Chlorine Concentration x Contact Time	CT
Chlorine Contact Tank	CCT
Chemical dosing unit	CDU
Chemical oxygen demand	COD
Dept. Of Environment, Climate Change and Water	DECCW
Dimethyl Sulphides	DMS
Diameter nominale	DN
Ductile iron cement lined	DICL
Dry Solid Tonnes	DST
Equivalent population	EP
Extracellular Polymeric Substances	EPS
Fibreglass reinforced plastic	FRP
Galvanised Mild Steel	GMS
Glass reinforced plastic	GRP
Grams per head per day	g/hd/d
High voltage	HV
High-level zone	HLZ
High-level zone pumping station	HLZPS
Human machine interface	HMI
High-Voltage	HV
Instrumentation & Electrical	I & E
Integrated water cycle	IWC
Invert level	IL
kilo Volt-Amperes	kVA
Kilowatt	kW
Litres per equivalent population per day	L/EP/d
Mega-litres per day	ML/d
Membrane bioreactor	MBR
Mega-litre	ML
Mixed liquor suspended solids	MLSS
Motor control centre	MCC
Neighbourhood	NH
Non-return valve	NRV
Number	nr
Odour Control Facility	OCF
Peak dry weather flow	PDWF
High Level Potable Water Reservoir	PWHL
Peak wet weather flow	PWWF
Piping & instrumentation diagram	P & ID
Power factor	PF

Pressure sewerage system	PSS
Programmable logic controller	PLC
Potable Water	PW
Queanbeyan City Council	QCC
Reduced infiltration sewerage system	RISS
Rising main unit	RMU
Ring Main Units	RMU
Rotary drum thickener	RDT
Recycled Water	RW
High Level Recycled Water Reservoir	RWHL
Sewerage pumping station	SPS
Sewerage Treatment Plant	STP
Solids retention time	SRT
Supervisory control and data acquisition	SCADA
Surge anticipator valve	SAV
Suspended solids	SS
Switchgear control assembly	SCA
Total Kjeldahl Nitrogen	TKN
Ultraviolet	UV
Uninterruptable power supply	UPS
Volatile Organic Compounds	VOC
Variable speed drive	VSD
Waste activated sludge	WAS
Water recycling plant	WRP
Water Standards Authority	WSA
Water sensitive urban design	WSUD
Water treatment plant	WTP
μm	10^{-9} m

Executive Summary

Overview

18,000 people using the water that would traditionally sustain only some 6,000. That is the overall deliverable of the Integrated Water Cycle (IWC) planned for the new township of Googong – Australia's first purpose designed water efficient township.

A new township is being proposed by CIC Australia at Googong, located south of Queanbeyan in NSW (the 'Googong Development'). The new township of Googong will contain approximately 6,200 residences, supporting some 18,000 people, and will be developed over a 25 year period. As a township, the Googong development will include retail, commercial, education, recreation, and community services. Together these will provide containment of daily needs, and the general autonomy of the new town extends to the proposed water and sewer infrastructure

As a greenfields site of 784 ha remote from the existing Queanbeyan sewerage treatment plant – which is itself at capacity, Googong was identified as requiring new wastewater treatment facilities. Rather than seeing this as an obstacle, CIC Australia with MWH have designed an IWC which aims to cut potable water consumption by 62% via:

- The use of reticulated recycled water as source substitution
- Demand reduction through;
  - Water efficient designed landscaping
  - Mandated water efficient appliances
  - Rainwater tanks
  - WSUD employed within the overall subdivision design.

Additionally, the IWC aims to improve the quality of runoff into the adjacent Queanbeyan River through the retention of stormwater on site through rainwater use and a series of Water Sensitive Urban Design (WSUD) measures. Importantly, the IWC has been designed to protect the adjacent Googong Dam catchment, and it is vital to stress that the entire IWC, and developed area, will be <u>outside</u> the dam catchment, and thus drains to the Queanbeyan River or for a small portion in the west of the site, to the Jerrabomberra Creek.

Overall, the Water Recycling Plant (WRP) will recycle the vast majority of wastewater which will be reticulated for re-use, with excess water discharged from the site via the stormwater system. It is key to note that:

- Quality of discharge is as Class A recycled water fit for domestic use
- Volume of discharge is significantly reduced due to reticulation of recycled water.

This report presents the Concept Design for the Googong IWC, where the configuration, sizing, and staged construction sequence over the years of development have been addressed. It supports the planning application made for the IWC under part 3A of the NSW EP&A act (the Googong IWC being declared a Major Project). The Googong subdivision itself is being handled in a separate but parallel process under Part 4 of the EP&A act.

The IWC includes all elements associated with the supply of potable water, the collection of and treatment of sewage flows and the transfer of treated flows into the recycled water system for re-use in the community as in the schematic below.

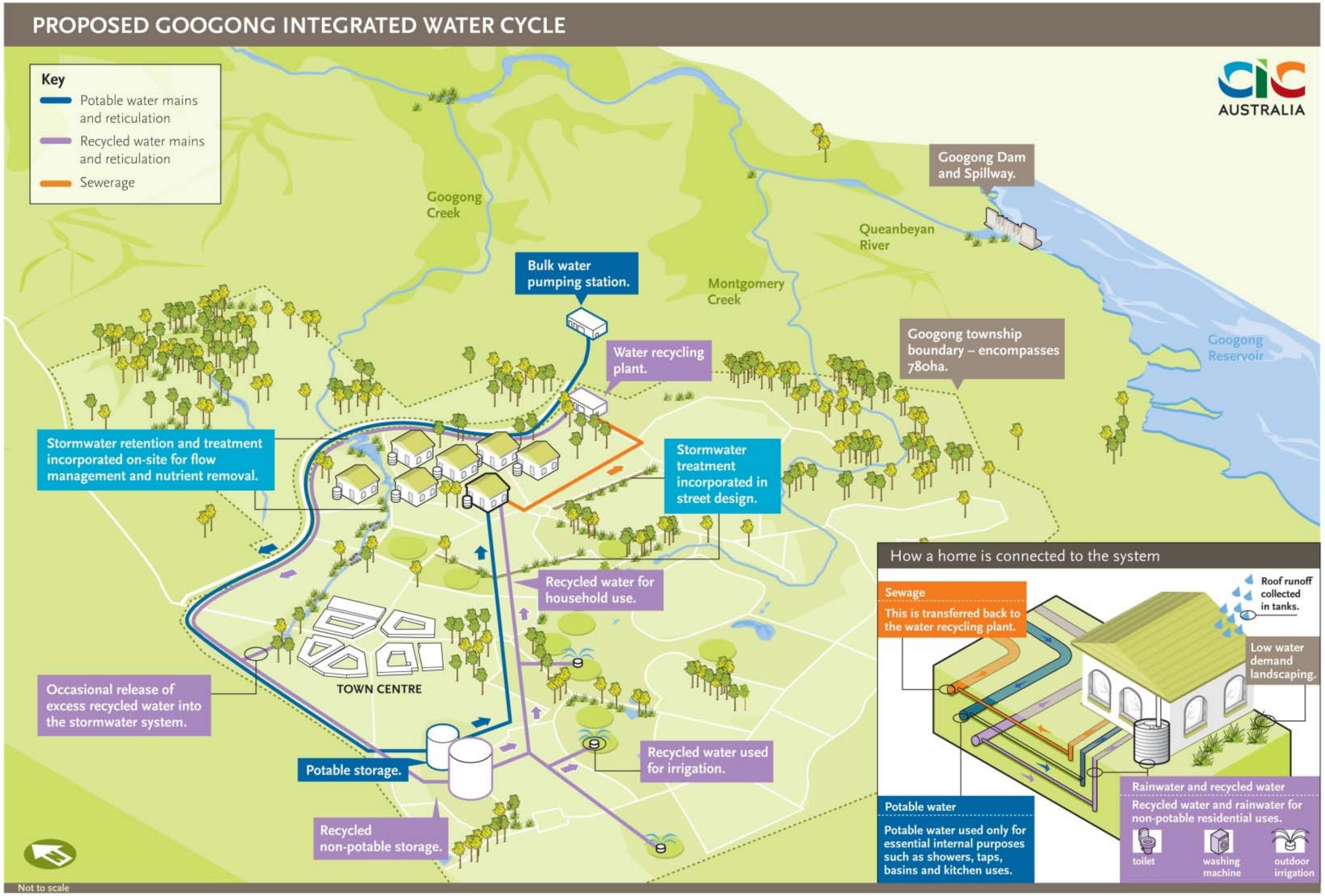


Figure 0-1 Googong Integrated Water Cycle schematic

The IWC includes all elements associated with the supply of potable water, the collection of and treatment of sewage flows and the transfer of treated flows into the recycled water system for re-use in the community. The system includes a potable water network, a recycled water network, sewerage collection networks and a Water Recycling Plant (WRP). Generally, for project planning purposes the IWC has been split into the Network (water, sewer, recycled water mains, pumps and storage) and the Water Recycling Plant (WRP).

Section 1 of this report presents an overview of the various potable water, recycled water and sewerage networks and the Water Recycling Plant. The Potable Water System will draw treated water from the ACTEW supply via a new Bulk Water Pump Station, located 170m down-slope from the existing Googong WTP. A Potable Water Supply Main will transfer flows to the Recycled Water Service Reservoir, for distribution to the potable supply reticulation system.

A conventional gravity sewerage system will collect sewage flows from the Googong community, draining by gravity to either Sewage Pump Station No.1 (SPS1) or SPS2. SPS1 and SPS2 will each deliver flows by sewer rising main directly into the inlet works of the WRP. SPS3 and SPS4 will pump into the gravity network upstream of SPS1 and SPS2 in the ultimate stage of design.

Recycled Water will be produced by the Water Recycling Plant (WRP) located along Googong Dam Road. The WRP will provide primary, secondary and tertiary treatment to produce recycled water for Googong. The WRP will be a Membrane Bio-Reactor (MBR) plant, complete with multiple disinfection barriers to meet the requirements of the Recycled Water Guidelines.

Recycled Water will be pumped through the Recycled Water Transfer Main to the Recycled Water Service Reservoir, for supply into the Googong community's recycled water reticulation system.

Staging Strategy

The Googong development will be constructed over a 20-25 year period. The construction of Googong water and wastewater infrastructure will be staged, such that capacity matches the population growth of the Googong development during its expansion. The staging strategy has been conceived in order to optimise the timing of capital expenditure throughout the life of the development, and comprises of seven discrete stages, each representing different population milestones in the development.

Section 1.7 of this report describes in detail the staging strategy for Googong. All of the major elements will be staged over a number of separate upgrade steps until the Ultimate configuration is reached. The Bulk Water Pump Station will be initially a small station providing flows to the potable service reservoir, but will be upgraded over time with larger pumping capacity to deliver potable water to also top up the recycled water system.

The first approximately 350 dwellings (being stages 1 and 2 of subdivision) will initially be supplied with Potable water only (in the separate Potable and Recycled Water systems), allowing time for the community to be developed and sufficient sewage load available to bring the WRP into reliable operation, producing recycled water for supply back to the community.

The potable and recycled water reservoirs will be constructed at an Interim Reservoir Site closer to the first stages of Googong at the high ground near Old Cooma Road and Googong Dam Road. At stage 3 of development, the interim site will be decommissioned and service reservoirs provided at the Hill 800 permanent reservoir site at the saddle feature further south along Old Cooma Road. Similarly, the WRP will be constructed in a number of stages beginning with a very small bioreactor sized for 1000EP, "turned down" to commence operations at say 150 Equivalent Population (EP), allowing tanker collection and disposal of sewage to cease. The various process-units of the WRP will be progressively upgraded over time to support the increasing sewage load as Googong continues to expand.

The following Table outlines the proposed stages and the corresponding Equivalent Population (EP) of the development at each milestone.

Table 0-1 Googong IWC staging strategy

Stage	EP Capacity (Network)	EP Capacity (WRP)	EP Minimum for This Stage	Sized for	Comment
Stage 1a	1,000	-	>0 EP	Neighbourhood 1A, Subdivisions 1 & 2	Initial subdivision, potable water network only, reservoir at Interim Site, tankering sewage from SPS1 wet well.
Stage 1b	-	1,000	>125 EP	Neighbourhood 1A, Subdivisions 1 & 2, Start Recycled Water (RW) production	Start SPS1, build 1/4 scale bioreactor at WRP, cease tankering of wastewater.
Stage 2a	3,600	2,350	>1,000 EP	Neighbourhood 1A	Construct recycled water network, 1/2 scale bioreactor at WRP
Stage 2b	-	4,700	>2,350 EP	First main WRP Bioreactor	WRP expanded to first full bioreactor
Stage 3a	9,400	-	>3,600 EP	50% Development	Network extended to final Reservoir Site, new potable reservoir and recycled reservoir
Stage 3b	-	9,400	>4,700 EP	50% Devt	WRP capacity expanded with second full bioreactor.
Stage 4	18,850	18,850	>9,400 EP	100% Devt	Reservoir capacity increased, WRP capacity expanded to Ultimate

Section 2 of this report details the various design criteria used for the concept design of Googong IWCMS infrastructure. The Bulk Water Pump Station, transfer pipelines and service reservoirs have been sized using Peak Day Demand values determined in accordance with water demand studies presented in **Appendix A**. For the Sewerage System, Peak Wet Weather Flows have been estimated based on the standard Water Standards Authority (WSA) methodology, for a storm Average Recurrence Interval of 3-months. The WRP design criteria are outlined further below in section 6.

Section 0 of this report describes the concept design of the Potable Supply system, including the Bulk Water Pump Station, and the two potable supply mains to the network (potable supply to the potable service reservoir, plus potable top-up to the recycled water system). The potable service reservoirs will supply the majority of the Googong development by gravity, meeting the desired residual head of 20m. Some small areas at higher elevation will need to be supplied from the High Level Zone booster pumps and elevated storages, sized for Peak Hour Demand, to meet the desired residual head. This system will include re-chlorination of the ACTEW potable water to meet the requirements of the drinking water guidelines and to limit biological re-growth in the supply system.

Section 4 of this report details the concept design of the Recycled Water (RW) supply system. This system comprises the RW Pump Station located at the Clear Water Tank of the WRP, the RW Transfer Main and the RW Service reservoir. The RW supply system will always be separate from the Potable supply system, but will in many ways will mirror that system. For example, the RW system will also utilise the re-chlorination system to maintain the desired chlorine residual to meet operational requirements. An air gap will be utilised to manage the risk of cross-contamination between the RW and potable water (PW) flows.

Section 5 of this report details the concept design of the sewerage system for Googong. Googong will be generally served by a conventional gravity sewerage system. The ultimate development will include at least four Sewage Pump Stations (SPS), with some fringe areas and hamlets potentially served with Low Pressure Sewerage Systems (LPSS) or similar. Googong sewerage systems will drain to either SPS1 or SPS2, which will both discharge via sewer rising mains directly to the WRP inlet works. The SPSs and gravity sewerage carriers will be naturally ventilated and it is expected that additional odour control measures will not be required.

Section 6 of this report details the concept design of the WRP, which is summarised below:

Water Recycling Plant

The Water Recycling Plant (WRP), as part of the overall Googong IWC has been included in this concept design to produce Recycled Water (RW) for domestic reuse. The WRP is critical to the overall IWC. RW will be used on site for applications including toilet flushing, cold water for washing and irrigation.

Strict RW quality requirements have been adhered to in the design of the WRP. The effluent quality required and receiving waters requirements were specified as the basis of design for the WRP to ensure safe domestic reuse within the Googong development. The proposed effluent consent conditions are outlined below. Environmental discharge limits are defined as the 50th and 90th percentile, representing the limit within which values will fall for 50% and 90% of all measurements, respectively.

A further condition of the basis of design specified a double barrier approach to disinfection, incorporating both UV treatment and chlorination, which ensures human health where domestic reuse is applied.

Table 0-2 Proposed WRP effluent conditions

Applicable Stage	Parameter	Proposed Consent Condition		
		Recycled Water [1]	Discharge to Environment	
			50 %ile	90%ile
All Stages (1-4)	BOD	-	5mg/L	10mg/L
	SS	-	10mg/L	20mg/L
	TDS		650mg/L	700mg/L
	Total Nitrogen	-	10mg/L	15mg/L

Applicable Stage	Parameter	Proposed Consent Condition		
		Recycled Water [1]	Discharge to Environment	
			50 %ile	90%ile
	Total Phosphorus	-	0.2mg/L	0.5mg/L
	Faecal Coliforms	< 1 cfu/100ml.		
	Total Coliforms	< 10 cfu/100ml (95%ile)		
	Virus	<1 cells/50L		
	Parasites	<1 cysts/50L		
	Turbidity	< 2 NTU		
	pH	6.5 – 8.0		
	Colour	< 15 TCU		
	Free Chlorine Residual	< 0.5 mg/l	-	

[1] NSW Guidelines for Urban & Residential Re-use of Reclaimed Water, May 1993

The selected treatment process allows for the removal of COD, TSS and nutrients (Nitrogen and Phosphorous) to the levels defined by the effluent quality requirements.

The removal of Viruses, Protozoa and Bacteria provided critical benchmarks for the design of the WRP. The indicative log reductions set a design requirement, as outlined below. Hence, the sequence of treatment outlined has been adopted for the Googong WRP design. The values given have been obtained from the Australian Guidelines for Water Recycling.

Table 0-3 Indicative log reductions for WRP processes

Treatment process unit	Indicative log reduction		
	Viruses	Protozoa	Bacteria
Secondary	0.5 - 2.0	0.5 - 1.5	1.0 - 3.0
Filtration	2.5 - 6.0	> 6.0	3.5 - 6.0
UV	> 1.0	2.0 - 6.0	1.0 - 4.0
Chlorination	1.0 - 3.0	0 - 1.5	2.0 - 6.0
TOTAL	**5.0 - 11.0**	**> 8.5**	**> 7.5**

The WRP combines primary, secondary and tertiary treatment processes to ensure effluent quality, as shown in Figure 0-2.

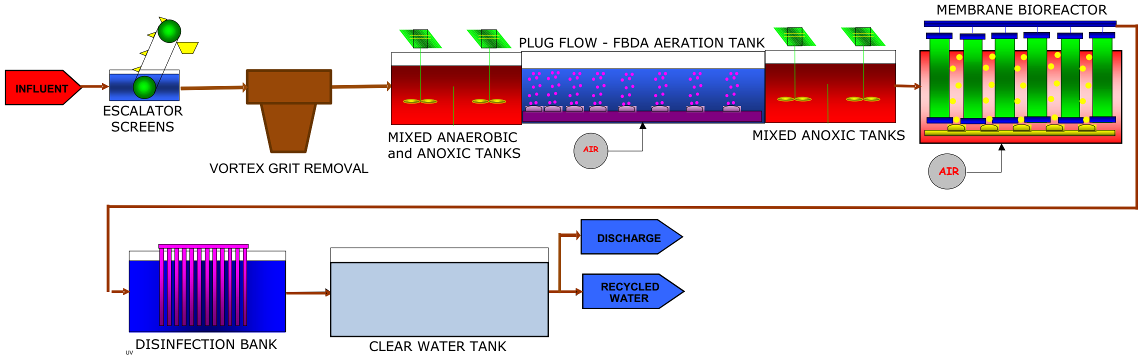


Figure 0-2 WRP process schematic

The major process units required within the WRP are outlined as follows:

<u>Preliminary Treatment</u>: 6mm then 1mm fine screens, followed by grit removal.

<u>Secondary Treatment</u>: Membrane Bio-Reactor (MBR) using a 5-stage “Bardenpho” treatment process incorporating biological phosphorus removal. The MBR will use immersed membranes to separate tertiary feed water from the biomass.

<u>Tertiary Treatment</u>: Ultra Violet (UV) disinfection followed by Chlorine Contact disinfection will be used to provide multiple disinfection barriers.

<u>Biosolids Treatment</u>: Waste biosolids from the MBR will be thickened then aerobically digested and dewatered, then stored in a sealed bin. Biosolids treatment will ensure a quality suitable for re-use through land application. This allows for re-use of useful organic matter and nutrient content, further limiting the waste product produced by the Googong development. The treatment of biosolids within the WRP will produce Grade B biosolids, for which many local land-applications exist. Generation of Grade A biosolids was deemed to be too resource intensive, given the limited additional re-use options which would become available.

<u>Odour Control</u>: The various treatment processes will be covered with sealed covers and foul air extracted. Foul air will be treated in a biological trickling filter (BTF) followed by activated carbon filter and discharged to atmosphere via a stack. The odour control strategy was modelled in CALPUFF and shown to achieve the DECCW 2 ou, 100th percentile criteria at the nearest sensitive receptor for Stage 2a and Stage 4, given the assumed residential zoning.

<u>Chemicals</u>: A number of chemicals will be used in the treatment process including Ferric Sulphate for odour control and (some) chemical phosphorus removal, Acetic Acid as a supplemental carbon source; Magnesium Hydroxide for alkalinity addition, Citric Acid for membrane cleaning and Sodium Hypochlorite for disinfection and membrane cleaning; and Polymer addition to waste biosolids for thickening and dewatering. However, the WRP has been designed with biological phosphorous removal, significantly limiting the requirement for Ferric Sulphate for phosphorous removal. The WRP will be configured to incorporate “biological phosphorus removal” to minimize chemical usage at the plant and reduce salt load on the treated recycled water.

Report Structure

This Concept Design Report has been prepared in three volumes as follows:

Volume 1	Main Report
Volume 2	Appendices
Volume 3	Drawings

1. Overview and Staging Strategy

1.1 Vision for Googong

A new residential community development is being proposed at Googong, located south of Queanbeyan in NSW (the 'Googong Development'). The community will contain approximately 5,500 residences, supported by retail, commercial and community services. This Integrated Water Cycle (IWC) design report outlines the design required to meet an optional capacity of 6,200 residences. Throughout the development, CIC Australia intends to employ a number of innovative water cycle management measures. These measures are intended to:

- Reduce the volume of potable water use through a series of water conservation, recycling and rainwater use initiatives;

- Improve the quality of runoff into the adjacent Queanbeyan River through the retention of stormwater on site through rainwater use and a series of Water Sensitive Urban Design (WSUD) measures; and

- Minimise the discharge of wastewater through the use of recycled water for toilet flushing and irrigation.

Googong will be a visually green and leafy town with landscaping and planting established in advance of the subdivision.

An IWC is fundamental to the successful establishment and future sustainability of the Googong Development.

In support of the vision of a sustainable township, the Googong Development will incorporate a Water Recycling Plant (WRP) which will receive sewage from the Googong Development and treat it to a standard suitable for re-use. The development of this infrastructure, along with a number of reservoirs, sewage pumping stations and other key management measures, forms part of the Googong IWC.

1.2 Integrated Water Cycle - Scheme Overview

An IWC for the Googong development has been developed to bring together a range of water efficiency and recycling measures into a combined strategy that includes potable water, recycled water, sewage collection and sewage treatment. This report presents the Concept Design for the Googong IWCMS, where the configuration, sizing, and staged construction sequence over the years of development have been addressed.

This report presents an overview of the IWC for Googong, and a synopsis of the adopted design criteria. The remainder of the report details the relevant engineering design aspects of each element of the IWC infrastructure.

The IWC includes all elements associated with the supply of potable water, the collection of and treatment of sewage flows and the transfer of treated flows into the recycled water system for re-use in the community. The general scheme layout is shown in **Drawing G-001**.

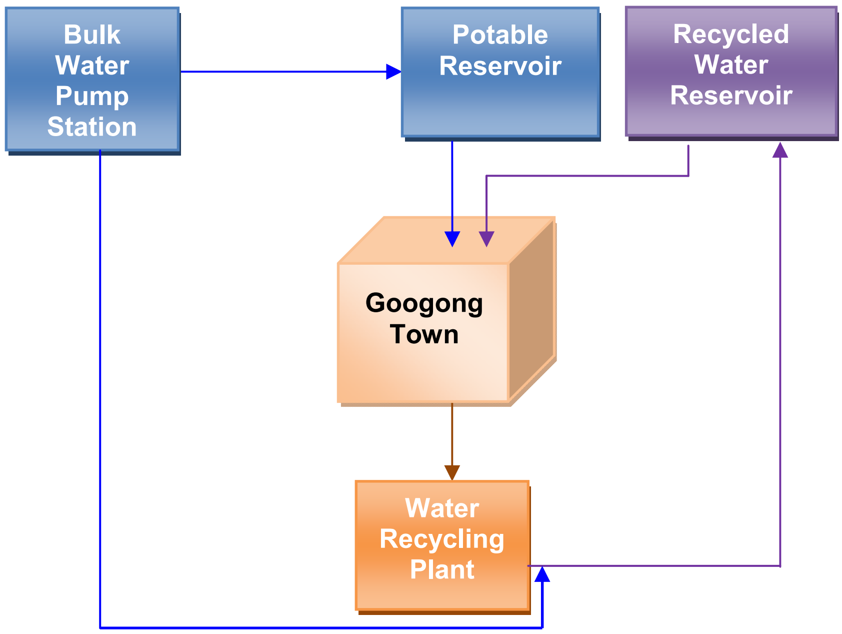


Figure 1-1 Integrated Water Cycle

The system includes a potable water network, a recycled water network, sewerage collection networks and a Water Recycling Plant (WRP). The following sections provide an overview of each major element of the IWC infrastructure.

1.3 Potable Water System

1.3.1 Existing Potable Water System

Consumers within the ACT are currently provided with potable water from existing ACTEW water treatment plants at Googong and Stromlo. Googong WTP is predominantly supplied from Googong Dam whilst the Stromlo WTP is supplied from the Cotter and Bendora dams.

Stromlo WTP has the capacity to provide up to 250ML/d of potable water to the residents and businesses of the ACT. The Googong WTP draws water from the Googong dam on the Queanbeyan River and has a capacity of 270 ML/day.

An existing pipeline is used to transfer flows in either direction between the Googong and Stromlo WTPs. This pipeline allows flow from Googong WTP to be transferred to Stromlo WTP to supplement supply, particularly during peak demand periods. Flow can also be passed from Stromlo WTP into the Googong dam to supplement the water supply to Googong WTP.

For operational reasons, ActewAGL's preferred and most common mode of operation is for flows to the ACT and Queanbeyan to be supplied from the Stromlo WTP. For this reason the supply arrangements for Googong will need to be capable of receiving flows from either WTP.

The existing arrangement is shown below in Figure 1-2.

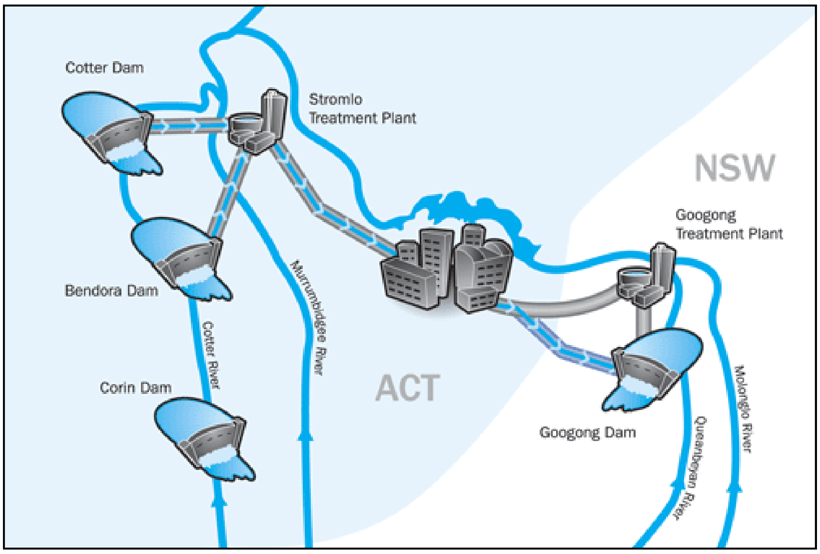


Figure 1-2 The location of dams and water treatment plants in the Googong region
(www.actew.com.au/watersecurity/majorprojects/cotter_googong_bulk_transfer.aspx)

1.3.2 Googong Potable Water System

A Potable water supply system will be provided for the Googong township, sourced from the ACTEW system. The Googong system will be capable of receiving flows from either Stromlo WTP or Googong WTP, in line with the normal operating practices of ActewAGL.

Because Googong will be gradually developed over time, it will not be necessary to construct the entire required infrastructure in the first instance. For this reason the IWC infrastructure for Googong will be constructed in a number of stages over time. For the early stages, an Interim Reservoir site will be utilised in high ground adjacent to Old Cooma Road, close to the initial subdivision areas. In later stages, the Permanent Reservoir Site will be established at a higher elevation further south along Old Cooma Road, to serve the entire Googong development, and the interim reservoir site will be decommissioned and demolished.

Similarly, the initial Bulk Water Pump Station drawing water from the DN1800 ACTEW main will be staged over time. The initial BWPS will need to be capable of supplying both the potable and recycled water networks of the development because tankering of sewage will be used in the first instance When the WRP is operational, the BWPS will be capable of meeting potable and recycled water demand as a potable “top up” of recycled water will be required from time to time to meet demand shortfalls. This potable top up of recycled water will initially be done at the Interim Reservoir Site, but in later stages the BWPS will be upgraded to supply this significant flow directly to the recycled water pump station for subsequent pumping to the permanent reservoir site recycled water reservoir. An air gap will be used in all potable top-up locations to ensure no cross contamination occurs.

As development of neighbourhood 1A (1,200 dwellings) progresses to some areas at higher elevation, minimum residual head of 20m will not be met. Therefore at the Interim Reservoir Site, it will be necessary to eventually construct an elevated storage tank and provide booster pumps to allow a High Level Zone to be supplied. The high level zone will make up approximately 20% of neighbourhood 1A, depending on the final detailed design planning.

As the Permanent Reservoir Site is constructed and the interim site decommissioned, the NH1A HLZ will be incorporated into the normal supply zone. However, the Permanent Reservoir Site will at some point also require elevated tanks and booster pumps to serve a separate High Level Zone. The ultimate number of high level properties will be assigned in detailed design. Therefore, the configuration and operation of the Interim and Permanent Reservoir Sites will be essentially the same, as is shown schematically in Figure 1-3 and Figure 1-4. Note that the separate main from the BWPS to the WRP supplying potable top up of recycled water system is shown in Figure 1.4.

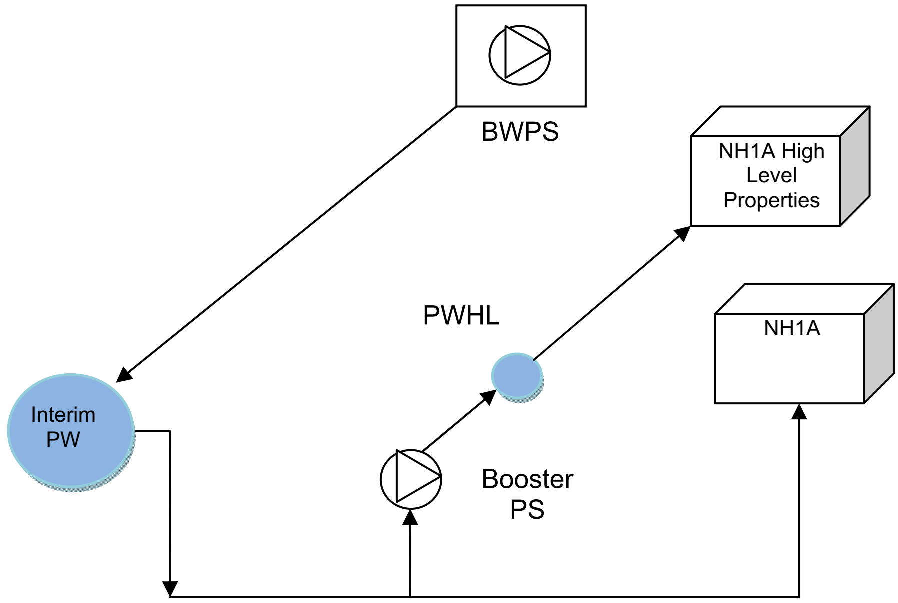


Figure 1-3 Schematic of Interim Potable Water Supply System (NH1A Only)

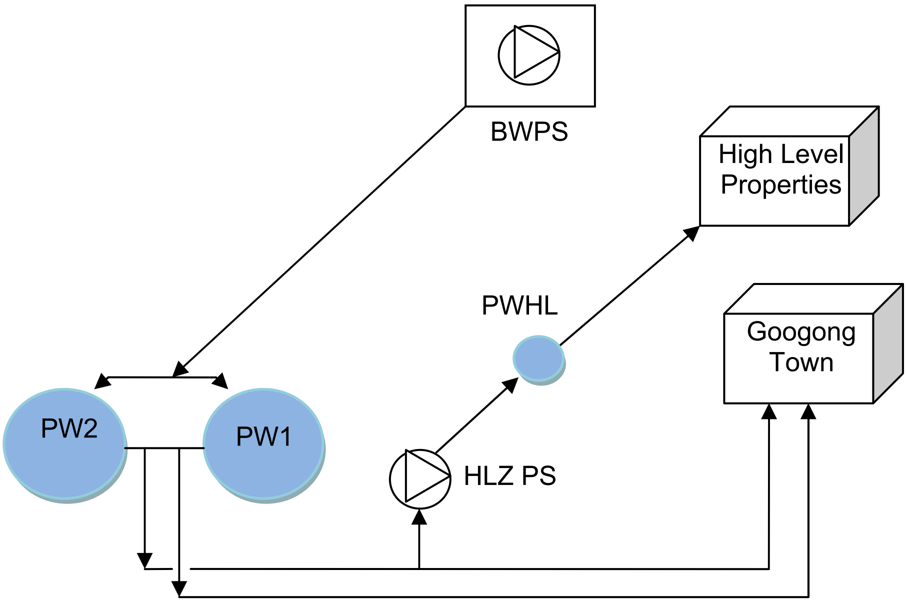


Figure 1-4 Schematic of Permanent Potable Water System (Whole Development)

Key elements of the Googong Potable water supply system are shown in Table 1-1.

Table 1-1 Key Elements of Googong Potable Water Supply System

Component	Major Elements	Description
Bulk Water Supply	Pipeline connecting ACT/Queanbeyan to ACTEW system (Existing, not part of this IWCMS)	Existing 1800mm diameter Googong WTP to Stromlo WTP pipeline
	Bulk Water Off-take (new)	Off-take and isolation valve arrangement from existing ACTEW 1800mm diameter pipeline
	Bulk water pumping station (BWPS)	Pump station configured to deliver flows to a) Potable water reservoir b) WRP Clear Water Tank for top up of recycled water system
Potable Water Transfer Mains	Potable Supply Main	Pipeline from BWPS to Potable Water service reservoirs.
	Potable Top Up of Recycled Water Main	Pipeline from BWPS to WRP Recycled Water Tank for top up. An air gap system will be utilised to ensure no cross contamination occurs into the potable water supply.
Interim Service Reservoirs (demolished approx year 5–6)	Interim service reservoirs	Potable Water and Recycled Water service reservoirs, located at 765m RL interim site along Old Cooma Road.
	Interim reservoir elevated tanks and booster pumps	Potable and Recycled water elevated tanks and booster pumps to supply minimum residual head to parts of Neighbourhood 1A.
Permanent service reservoirs	Permanent service reservoirs	Permanent Potable Water and Recycled Water reservoirs sized for Ultimate demand located at Hill 800 saddle feature further along Old Cooma Road
	High-level service reservoirs and pumping station.	Small high level Potable Water and Recycled Water service reservoirs and booster pumping stations to supply local high level zone.
Water Re-Chlorination	Re-chlorination facility	Located at each reservoir site, re-chlorination facility to top up chlorine residual in stored water to meet guideline requirements.
Other Works	Potable water outlet mains and distribution system	Potable water supply from service reservoirs to reticulation system supply zones.

The design criteria used in the concept design of the potable water system are shown in Section 2.2. The Potable water network Concept Design is described in Section 0.

1.4 Recycled Water System

A Recycled Water network will be provided for Googong to supply the recycled water reticulation system which in many ways mirrors the potable water system. Recycled Water will be used to supply mostly external demands such as garden watering, but will also be used as "top up" supply to washing machine cold water and toilets. Air gap systems will be used to ensure no cross-contamination occurs between RW and PW.

For the initial stage of the development, the small volumes of sewage produced by the growing community will be collected and tankered away for disposal at another regional treatment plant. The Water Recycling Plant (WRP) will be constructed after the first subdivision areas are developed. Because of the small number of houses in the initial development areas, sewage loads to the WRP will be small. The initial WRP will therefore be very lightly loaded and is expected to take some time to reach stable operation. For the initial period, potable water will be used to feed the reticulated recycled water system. Recycled water produced in Stage 1 of development will be transferred to the stormwater basin outlet, where it will be available for irrigation use. Further recycled water capacity from the WRP will be introduced at Stage 2, when a recycled water pump station, pipeline and reservoir are constructed. The recycled water system will be configured as shown in Figure 1-5.

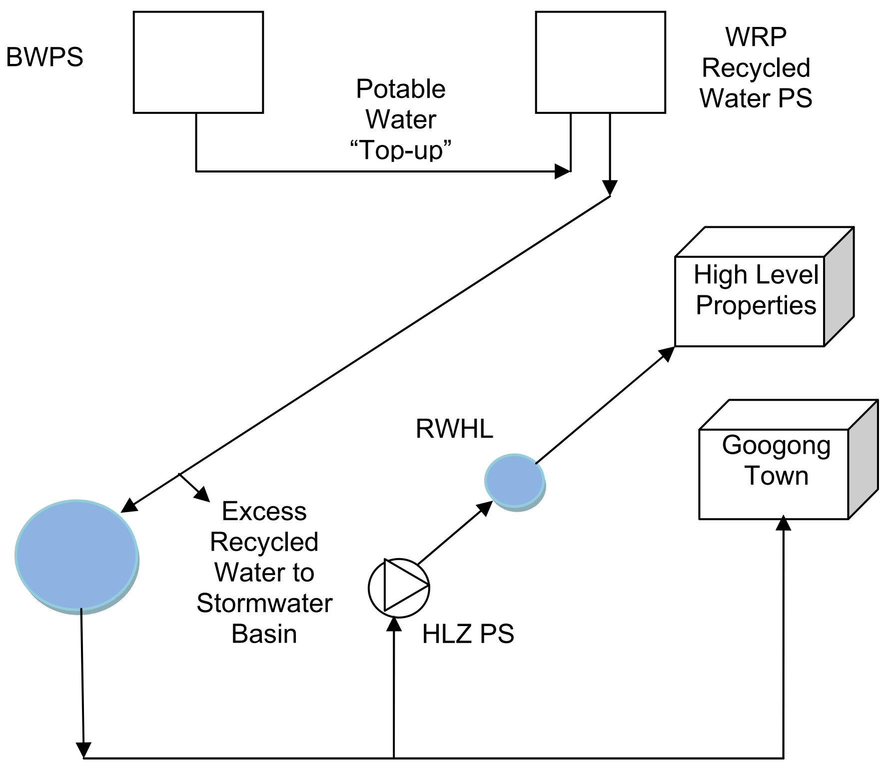


Figure 1-5 Schematic of Permanent Recycled Water System

In a similar way to the potable water system, the recycled water reservoir constructed at Stage 2 will be located at the Interim Reservoir Site adjacent to Old Cooma Road, close to the intersection with Googong Dam Road at elevation RL765m. At Stage 3, a reservoir will be constructed at the Permanent Reservoir Site at the twin hill and saddle feature further south on Old Cooma Road and the interim site will be decommissioned and demolished.

As for the potable supply system, both the Interim and Permanent Recycled Water systems will be predominantly supplied by gravity from an above ground reservoir, with a smaller High Level Zone supplied by booster pumps and elevated storage tank. Gravity feed will still be possible to the HLZ areas from the main reservoir in emergencies; however the booster pumps and elevated tank will allow minimum residual pressures to be met in normal conditions.

The recycled water system will function as a completely separate system in parallel to the potable water system. Key elements of the Recycled Water network are show in Table 1-2.

Table 1-2 Key Elements of Googong Recycled Water Supply System

Component	Major Elements	Description
Recycled Water Pump Station	Recycled Water PS	Dry mounted pumps located at the WRP, drawing water from the Recycled Water Tank.
	Potable Water Top Up of Recycled Water system	Supplied in a dedicated main from the Bulk Water PS, discharging to the WRP Recycled Water Tank to meet occasional shortfalls in recycled water production to meet demand. An air gap will be included to ensure no cross contamination between RW and PW.
Recycled Water Delivery Main	Recycled Water Supply Main	Pipeline from BWPS to Potable Water service reservoirs.
	Environmental Release	Pipeline Off-take from Recycled Water Supply Main releasing flows to stormwater pond system.
Interim Service Reservoirs	Interim service reservoirs	Recycled Water service reservoirs, located at interim site along Old Cooma Road.
	Interim reservoir elevated tanks and booster pumps	Recycled water elevated tanks and booster pumps to supply minimum residual head to parts of Neighbourhood 1A.
Permanent service reservoirs	Permanent service reservoirs	Permanent Recycled Water reservoirs sized for Ultimate demand located at saddle feature further along Old Cooma Road
	High-level service reservoirs and pumping station.	Small high level Potable Water and Recycled Water service reservoirs and booster pumping stations to supply local high level zone.
Recycled Water Re-Chlorination	Re-chlorination facility	Located at the reservoir site, re-chlorination facility to top up chlorine residual in stored water to meet guideline requirements.
Distribution System	Recycled water outlet mains and distribution system	Recycled water supply from service reservoirs to reticulation system supply zones.

The design criteria used in the concept design of the recycled water system are shown in Section 2.2. The recycled water network Concept Design is described in Section 4.

1.5 Sewerage System

Sewage generated within the development will be collected and transported to the water recycling plant for treatment and recycling. A combination of gravity sewers draining to a number of sewage pumping stations will be required. It is anticipated that some small areas within the development may require pressure sewage systems, however the extent of these areas cannot be determined until road layouts and land zonings are finalised.

In accordance with the overall development strategy, construction of the sewerage transfer system will be staged to minimise upfront costs and allow flexibility in the roll out of future infrastructure.

A minimum of four Sewage Pumping Stations (SPSs) and rising mains are required across the development, to transport and lift sewage to the WRP. For details on the sewerage catchments, refer to **Drawing G-002**.

SPS1 is proposed for Stage 1 and SPS 2 for Stage 2. SPS3 and SPS4 are proposed much later in the development for Stage 4. It should be noted that the final sewage catchment boundaries and hence pumping station locations is highly dependent on development road layouts and land zonings. It is recommended that these boundaries, pumping station locations and sizes be reviewed as town planning details become clearer.

Areas within NH1B and NH5 may require servicing by low pressure sewerage systems (LPSS) and/or an additional pumping stations.The extent of this area is highly dependent on the location of SPS2, which in turn has been influenced by preliminary environmental scanning. Should further environmental assessment indicated that SPS2 can be moved further downhill towards Montgomery Ck, a significant reduction in the area required to be serviced by a PSS network or additional pumping station can be achieved.

The concept design layout of SPS1 and SPS2 are shown on **Drawing G-001**. No design of the sewage collection system was undertaken, but consideration was given to the overall development sewerage philosophy to achieve a viable ultimate development solution. Table 1.3 shows key elements of the sewerage system.

Table 1-3 Key Elements of Googong Sewerage System

Component	Major Elements	Description
Sewage Pump Station 1	SPS 1	Underground, concrete wet well structure with submersible sewage pumps. Emergency storage provided for pump/power failure. Staging of pumps and emergency storage to be compatible with WRP capacity and catchment size. Initial capacity 10 L/s, increasing with staged upgrades over time to 67 L/s
	SPS 1 Rising Main	800m long DN225 buried pipeline from SPS1 to WRP inlet works.

Component	Major Elements	Description
Sewage Pump Station 2	SPS 2	Underground, concrete wet well structure with submersible sewage pumps. Emergency storage provided for pump/power failure. Staging of pumps and emergency storage to be compatible with WRP capacity and catchment size. Initial capacity 31 L/s, increasing with additional pump following staged upgrades over time to 55 L/s
	SPS2 Rising Main	800m long DN200 buried pipeline from SPS2 to WRP inlet works.
Sewage Pump Station 3	SPS3	Smaller, submersible pumping station will be required in Stage 4 to enable development of western side of Neighbourhood 3.
	SPS3 Rising Main	Short length of rising main discharging to adjacent gravity carrier in Neighbourhood 3.
Sewage Pump Station 4	SPS4	Submersible pumping station will be required in Stage 4 to enable development of the southern side of creek, for Neighbourhoods 4 and 5.
	SPS4 Rising Main	Approximately 500m rising main discharging to SPS2.

The design criteria used in the concept design of the sewerage system are shown in Section 2.4. The sewerage network Concept Design is described in Section 5.

1.6 Water Recycling Plant

1.6.1 Overview

The water recycling plant (WRP) described in this concept design will produce recycled water (RW) for non-potable use including; toilet flushing water, cold water for washing machines and irrigation. The production of recycled water is a fundamental element of the Googong IWC, allowing for significant savings in potable water use throughout the development.

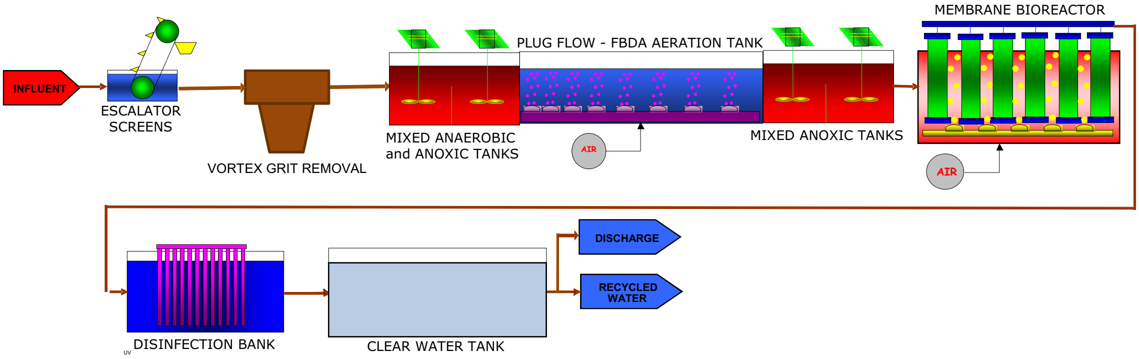


Figure 1-6 WRP Process Schematic

The WRP will treat sewage, generated from the Googong development to a quality suitable for recycling. The plant consists of; primary screens and grit removal, a Membrane Bioreactor (MBR) for nutrient and solids removal and a tertiary stage combining UV treatment and chlorination for disinfection. Key elements of the WRP are identified in Table 1-4.

Table 1-4 Key Elements of Googong WRP

Component	Major Elements	Description
Preliminary Treatment Inlet Works	Receiving Chamber	All sewage flows from the network will enter the inlet works via separate transfer pipelines and discharge into a common receiving chamber.
	6mm Screens	Coarse screening. Shaftless screw conveyor collecting screenings from coarse and fine screens.
	1mm Screens	Fine screens normally minimum requirement to protect membranes.
	Vortex Grit Removal	Standard vortex grit removal
	Screenings and Grit Handling	Screenings will be collected at ground level, washed and dewatered in a screw wash press, before being transferred to sealed bins. Grit will be washed and dewatered in a grit classifier and will be combined with the screenings in the bins
Secondary Treatment Biological Stage	Biological treatment in bioreactors: Anaerobic, Anoxic and Aeration Zones (5-stage Bardenpho process)	Secondary-biological treatment will be provided by a number of MBRs. The bio-reactors will remove organic material, nutrients (nitrogen and phosphorous) and suspended solids through membrane filtration. Biological Phosphorous Removal (BPR) has been implemented within the concept design to limit the overall requirement for chemical addition. BPR is used in conjunction with the addition of Ferric Sulphate in order to remove phosphorous to the licence condition.
	Membranes	The membrane tanks, fundamental to the operation of the MBRs, contain submerged ultra-filtration membranes. The effluent is drawn through the membranes whilst the mixed liquor from the bioreactor is contained within the tank. The pore size of the membranes is 0.45µm providing a physical barrier for solids removal.

Component	Major Elements	Description
Tertiary Treatment Disinfection	Disinfection with UV and Chlorine	Recycled water requires a double barrier approach to bacteria removal, to protect the end user. The exact requirement is determined under the Australian Recycled Water Guidelines by a risk assessment. The risk assessment will occur in parallel with detailed design activities. Experience from other plants with similar recycled water use, shows that two forms of disinfection provide an adequate barrier. The disinfection included for the WRP is ultraviolet (UV) disinfection followed by chlorination. Residual chlorine will be maintained, to prevent regrowth of bacteria within the recycled water reservoirs and pipe work.
Biosolids	Sludge Thickening, Digestion and Dewatering	Biosolids produced in the bioreactors will be thickened and then treated in an aerobic digester. Digested biosolids will then be dewatered using a centrifuge and stored on-site within a sealed bin for collection and off-site disposal.
Odour Control	Biological Trickling Filter	The WRP will be equipped with odour control facilities to minimise odour nuisance. Odour generation will be contained by provision of removable sealed covers on all process units. Foul air will be extracted and conveyed via sealed aboveground ducting, to a central odour control facility (OCF). The OCF will consist of biological trickling filters, followed by activated carbon filters. Treated air will be discharged to the atmosphere via a stack to aid dispersion. The odour control strategy was modelled in CALPUFF to achieve the DECCW 2 ou, 100th percentile criteria at the nearest sensitive receptor for Stage 2a and Stage 4, given the assumed residential zoning.
Chemicals	Ferric Sulphate	Odour control and chemical phosphorus removal[1]
	Acetic Acid	Supplementary carbon addition for de-nitrification
	Magnesium Hydroxide	Alkalinity addition
	Citric Acid	Membrane cleaning
	Sodium Hypochlorite	Disinfection and membrane cleaning
	Polymer	Sludge thickening and dewatering

Component	Major Elements	Description
Utilities and Services	Clear Water Tank	Storage of treated recycled water, used as feed wet well for Recycled Water Pump Station and location of potable water top up of Recycled Water system to meet demand shortfalls.
	Foul Water tank	Storage of backwash water from membrane cleaning cycle, flows returned to head of works.
	Service Water tank	Tank to store recycled effluent for various process uses around the plant
	Electricity Supply	Low Voltage supply required from Country Energy.
	Recycled Water Pump Station	The Recycled Water Pump Station will transfer recycled water from the WRP to the RW service reservoir.
Siteworks	Site Preparation and Roads	Benching and levelling, site roads and drainage as required.
	Recycled Water Pump Station and Discharge Pipelines	Recycled water (RW) will be pumped to the RW reservoir using the RW pumping station. RW produced in excess of demand will be discharged from the RW Main through the storm water system basins. A shorter pumped rising main to the stormwater basins fed by the membrane permeate pumps via the UV system will be an alternative point of discharge to the environment. This route will be used as the initial WRP discharge at Stage 1 until recycled water is available at Stage 2. The WRP is not able to discharge to the waterway draining from the WRP site.

When periods of high demand for recycled water coincide with an excess of stormwater and recycled water within the stormwater system, there may be an opportunity to transfer combined stormwater/recycled water to the WRP for treatment and subsequent transfer to the RW system. As a minimum this will involve provision of a pipe and actuated valve from Basin1 into SPS1. This will need to be considered further within the project to determine special arrangements.

The design criteria used in the concept design of the Water Recycling Plant are shown in Section 2.5. The WRP Concept Design is described in Section 6.

1.7 Staging Strategy

1.7.1 Staging Approach

Googong will be developed over a 20-25 year period. The works will be constructed in a staged approach to ensure that the infrastructure is correctly sized to meet the incremental level of demand, ensuring a balance is reached between cost-effectiveness and capacity. Each Stage will provide capacity ahead of the demand, with the next stage triggered as the particular stage reaches its capacity in response to population growth.

A Staging Strategy has been developed for construction of all of the Potable Water, Recycled Water, Sewerage and WRP works over the development period. The Staging Strategy consists if a series of integrated upgrade works that at all times meet the design criteria required for system performance and serviceability.

The upgrade stages and corresponding capacity listed in terms of Equivalent Population (EP) are shown in Table 1-5.

Table 1-5 Googong Works Staging Approach Versus Equivalent Population

Stage	EP Capacity (Network)	EP Capacity (WRP)	EP Minimum for This Stage	Sized for	Comment
Stage 1a	1,000	-	>0 EP	Neighbourhood 1A, Subdivisions 1 & 2	Initial subdivision, potable water network only, reservoir at Interim Site, sewage tankered away
Stage 1b	-	1,000	>150 EP	Neighbourhood 1A, Subdivisions 1 & 2	¼ scale bioreactor at WRP, cease tankering of wastewater.
Stage 2a	3,600	2,350	>1,000 EP	Neighbourhood 1A, Start RW	Construct recycled water network, 1/2 scale bioreactor at WRP
Stage 2b	-	4,700	>2,350 EP	First main WRP Bioreactor	WRP expanded to first full bioreactor
Stage 3a	9,400	-	>3,600 EP	50% Devt	Network extended to final Reservoir Site,
Stage 3b	-	9,400	>4,700 EP	50% Devt	WRP capacity expanded with second full bioreactor.
Stage 4	18,850	18,850	>9,400 EP	100% Devt	Reservoir capacity increased, WRP capacity expanded to Ultimate

1.7.2 Stage 1 – Initial construction to 1000 EP – Neighbourhood 1A St 1&2

1.7.2.1 Network for first 347 lots

Stage 1 of the Network is shown in **Drawing SK-001**. At Stage 1, the initial potable water system will also supply the "recycled water" distribution system, from a common potable water reservoir, as no recycled water will be reticulated from the WRP, however, RW will be available for irrigation. The potable reservoir will be located at the Interim Reservoir Site adjacent to Old Cooma Road, shown in plan and section respectively in **Drawings SK-303** and **SK-304**.

The Bulk Water Pumping Station will be configured with temporary skid-mounted pump and valve equipment, delivering flows to the potable reservoir via the Potable Supply Main. Stage 1 of the Bulk Water Pump Station is shown in **Drawing SK-601**. The DN225 potable supply main will be capable of supplying the potable and recycled demand of Stage 1 and 2. This pipeline has also been sized to meet the ultimate potable daily demand for Googong, without requiring augmentation. Later in the development, this pipeline will have the capacity to supply the potable reservoir only, as the potable top up of the recycled water system will be done at the WRP location.

1.7.2.2 Sewerage System and WRP

The conventional gravity sewerage system serving Googong will be constructed gradually, in line with the construction of the normal subdivision roads and services. The outlet of this first part of Neighbourhood 1A will be Sewage Pump Station No.1 (SPS1). The wet well and (part) of the emergency storage of SPS 1 will be constructed in Stage 1. However, instead of pumping to the WRP, SPS 1 will be used as a pump out location to tanker sewage away for disposal until enough houses are established, and sufficient pollution load is available to allow commencement of operations at the WRP – expected to occur at 45-50 dwellings..

Initially, sewage will be collected from the wet well of SPS 1 and tankered for disposal at another sewage treatment plant in the region. The WRP will be first constructed to have a reduced-scale bioreactor and membrane system with a capacity of 1000 EP. The WRP will be brought on line to receive sewage flows once 150 EP is available to provide sufficient load for the biological stages of the WRP to function correctly. At this point tanker disposal of sewage will cease.

Stage 1 of the WRP is shown in **Drawing SK-701**. As noted above, the WRP will not produce recycled water for domestic reuse in the initial stage. Treated effluent from the WRP will be discharged via a short pipeline to the constructed stormwater basin system. The recycled water flow in Stage 1 will be available for re-use through irrigation. Excess may be discharged to the environment.

While the ultimate plant will have four parallel concrete "bioreactor" tanks, each sized for about 4,700 EP giving a combined capacity of around 18,850EP, it is proposed in Stage 1 to construct only one of these tanks. However instead of configuring the tank for the full 4,700EP, it is proposed to utilise only a small fraction of the tank and configure it for 1000EP, so that sufficient turndown can be achieved to commence WRP operations as soon as possible, and tankering can cease. Temporary walls in the tank will be installed to seal off the smaller bioreactor. The unused remainder of the tank will be used for storing backwash water, sludge, inlet flow balancing or other purposes as defined in detailed design. A further option to consider at detailed design is to only construct 50% of the length of the first concrete tank, and construct the remaining 50% in Stage 2a, described below.

The proposed sequencing of bioreactors in the WRP is shown in **Drawing SK-608**, for each of the Stages 1 to 4.

The WRP at this early stage will include primary treatment of inlet screens (6mm and 1mm), secondary treatment (reduced scale bioreactor with membrane filtration), and tertiary treatment including UV disinfection and chlorination. The DN225 outfall pipeline will be utilised for chlorine contact during stage 1, where the recycled water produced may be used for irrigation of the growing development.

Due to the very small scale of the plant, biosolids digestion will not be provided, however waste biosolids will be thickened and dewatered and stored in sealed bins for disposal to landfill. Grit removal is not required at this early stage, as the first stage of the bioreactor can be occasionally cleaned to remove any grit entering the secondary treatment stage. Odour control will be provided from the first stage of WRP operation. Given the temporary nature of several elements of this initial stage, it is not proposed to construct the main administration or electrical building, but instead install an outdoor electrical kiosk to house electrical equipment.

1.7.3 Stage 2 – Completion of Neighbourhood 1A

1.7.3.1 Network – All of Neighbourhood 1A - 3600EP

As Neighbourhood 1A stages 1 and 2 are completed, approximately 1000EP demand is reached; additional components of the water network will be constructed to provide treated recycled water from the WRP to the Googong community. Stage 2 of the Network is shown in **Drawing SK-002**.

A Recycled Water Reservoir will be constructed at the Interim Reservoir Site, and a new feed line to the recycled water distribution system will be constructed, which replaces the common potable feed from Stage 1.

The Recycled Water Pipeline from the WRP to the service reservoir will be constructed. This pipe will be sized for the ultimate capacity, and will therefore have considerable excess capacity in the early years. The Recycled Water Pipeline will also serve a second purpose of acting as the Chlorine Contact Tank for the WRP, providing a second level of disinfection to meet the risk requirements of the recycled water guidelines.

Some areas of NH1A will be developed on land that is at a higher elevation, and residual gravity supply head from the main service reservoirs will fall below the minimum criteria of 20m. Therefore small elevated tanks and booster pumps will be provided at the Interim Reservoir Site to supply this separate High Level Zone.

1.7.3.2 WRP – Stage 2a to 2350 EP

At Stage 2a, SPS2 will be constructed to serve the eastern side of neighbourhood 1A. Stage 2a of the WRP is shown in **Drawing SK-702**. The sewer rising main for SPS2 will be brought into the WRP to discharge to the inlet works alongside the incoming main from SPS1.

Stage 2a represents the introduction of recycled water produced by the WRP for distribution to Googong community to be used for all recycled water applications. At the WRP, this stage requires expansion of biological capacity, as well as introduction of the Recycled Water Pump Station.

As the 1000EP capacity of the WRP from Stage 1 is reached, it is proposed to expand the first bioreactor to half its ultimate size by fitting out the other unused half of the first bioreactor tank, while the initial unit remains in service. It should be possible to configure this second, small scale stage by using all of the same equipment from the first stage, such as recirculation pumps, mixers and other equipment, with the installation of a number of new baffle walls to separate biological treatment zones.

An alternative approach is to simply construct the second full tank alongside the first tank, and fit this tank out for the 4,700 EP; however this alternative can be looked at in detail at the time of upgrade. A key factor in the decision is ensuring that the first bioreactor remains in uninterrupted service while the larger unit is constructed.

The Recycled Water Pump Station will be constructed, to deliver recycled water via the recycled water pipeline to the new recycled water service reservoir.

1.7.3.3 WRP – Stage 2b to 4700 EP

Stage 2b of the WRP is shown in **Drawing SK-703**. The WRP capacity will be increased to 4700 EP with the construction of the second full size bioreactor tank, while the first tank is left in service. After cutover to the new tank the old bioreactor will be decommissioned, and the empty space used as emergency storage until the next stage of development.

This major upgrade stage will see construction of a number of permanent elements of the WRP including the electrical/administration building, the dewatering and blower building and half of the ultimate aerobic digester volume, including thickening and dewatering.

1.7.4 Stage 3 – to 9,400 EP - 50% development of Googong

1.7.4.1 Network – to 9,400EP

Stage 3 of the Network is shown in **Drawing SK-003**. The Bulk Water Pump Station will undergo a major upgrade at this stage, with construction of the new Potable Supply pumps to deliver flows to the new, higher elevation Permanent Potable Reservoir, and the Potable Top Up pumps delivering flows to the WRP area. Upgrade of the Bulk Water Pump Station is shown in **Drawing SK-602**.

A new Potable Top Up pipeline from the BWPS will be constructed to the Recycled Water Pump Station at the WRP to provide potable top up of the recycled water system. An air gap will be utilised to ensure no cross-contamination occurs between the PW and RW streams.

The Potable Supply Main will be extended along Old Cooma Road to the new higher-elevation Permanent Reservoir Site located at the “Twin Hills” feature. The Permanent Reservoir Site plan and section are shown in **Drawings SK-301** and **SK302** respectively. The Interim Reservoir Site will be decommissioned and demolished. At the Permanent Reservoir Site located at the twin hill/saddle feature near Old Cooma Road, a new potable and a recycled water service reservoir will be provided.

1.7.4.2 WRP – to 9,400 EP

The first bioreactor Tank 1, which was initially used as the small-scale bioreactor (decommissioned in Stage 2), will be fitted out with mechanical equipment, baffle walls, etc, as a full size bioreactor running in parallel with the in-service Tank 1. This will give the plant half of the ultimate biological treatment capacity.

The proposed works at the WRP are shown in **Drawing SK-704**. The odour control system will be duplicated with an additional biological trickling filter and activated carbon filter. Various other upgrades will be required to expand the capacity of mechanical and electrical equipment including the membranes and permeate pumps, UV disinfection, chemical dosing and recycled water pumps.

1.7.5 Stage 4 – 100% Development

1.7.5.1 Network – to 18,850 EP

Stage 4 of the Network is shown in **Drawing SK-004**. The Bulk Water Pump Station will be upgraded to meet increased capacity requirements with an additional duty pump for both the Potable Supply main to the potable reservoir, and the Potable Top Up main to the WRP.

A larger recycled water reservoir will be constructed, and the Stage 3 RW reservoir will be converted to potable supply, giving two potable reservoirs and one recycled reservoir. Small potable and recycled reservoirs and booster stations will be constructed at higher level to supply the local High Level Zone in the higher elevation areas.

1.7.5.2 WRP – to 18,850 EP

The Stage 4 upgrade to the WRP would, in reality, occur as several steps over many years, however for simplicity have been represented as a single upgrade stage. Stage 4 of the WRP is shown in **Drawing SK-705**. Two new bio-reactor and membrane tanks would be constructed, giving a total of four tanks operating in parallel to meet the ultimate capacity of 18,850 EP. The aerobic digesters would be expanded to the ultimate capacity, and the blower building extended to accommodate the full complement of mechanical aeration equipment. The odour control system would be expanded, along with the various capacity upgrades to permeate pumps, chemical dosing, UV disinfection and site services.

1.8 Previous Studies

A number of previous studies associated with water infrastructure have been undertaken by MWH on behalf of CIC Australia. This Concept Design Report builds on the work prepared in the earlier work. A summary of the earlier studies and reports is shown in Table 1-6.

Table 1-6 Summary of previous reports

Report Title	Date	Description
The Googong New Community – Integrated Water Cycle Management Strategy – Initial Concepts	Mar 2004	Developed a potential water cycle for the new community and briefly assessed infrastructure options for the development.
Googong Service Reservoir and Sewage Treatment Plant – Site Options report	Dec 2004	Considered various options for the delivery of major infrastructure.
The Googong Integrated Water Cycle Management Strategy – Final concepts	May 2006	Developed a series of water cycle management options, identifying two stakeholder preferred water cycle management options (options 5b & 6a), and established three project objectives. It also built on the IWCMS work developed in the Initial Concepts IWCMS.
Googong Release Area Water and Wastewater Preliminary Design and Costing	Feb 2007	Follows on from previous work completed by MWH in support of CIC Australia's ongoing investigations of the potential release of land at Googong.
Googong Design Assumptions for Potable and Recycled Water Systems	Jan 2008 Revised July 2010	Outlined the revised water cycle assumptions and provided the proposed design demand unit rates for potable and non-potable water for each demand type in the Googong development.
Googong Water Recycling Plant Options Briefing Paper	Jan 2009	Outlined the different options available for the water recycling plant and determined the best option for this system was a 4-stage Bardenpho a membrane bioreactor.

2. Design Criteria

The design criteria applicable to the concept design of the IWC infrastructure are summarised in this Section. The design criteria are sourced from various design standards and codes, industry practice, modelling simulations and advice from Council. For a number of the design parameters, reference is made to earlier reports which describe the detailed modelling results.

2.1 Population Growth and Works Staging

The rate of population growth is the key driver of the required sizing of the various elements of water and wastewater infrastructure for Googong. The growth of population affects the planned staging of the works, which are constructed progressively, expanding in capacity to meet the needs of the growing population.

The population forecasts contained in this report are based on the growth rates provided by CIC Australia. The assumed growth rate is shown in Table 2-1, with the upgrade stage for the network and WRP also indicated. The forecasted growth rates will most likely vary over time in response to various factors such as the housing market, regional population growth, etc. For this reason, sizing of the various infrastructure elements and staging of the works has been linked to population rather than year, to better reflect the likely points in the future when the next upgrade stage will be required.

Table 2-1 Googong Forecast Rate of Development

Project Year	Calendar Year	Cumulative Settlements	Cumulative Equivalent Population	Network Upgrade Stage	WRP Upgrade Stage
1	2010	0	0	Stage 1a 0 to 1,000 EP	
2	2011	65	184		Stage 1b 125 to 1,000 EP
3	2012	249	706		
4	2013	505	1,428	Stage 2a 1,000 to 3,600 EP	Stage 2a 1,000 to 2,350 EP
5	2014	610	1,726		
6	2015	831	2,353		Stage 2b 2,350 to 4,700 EP
7	2016	1031	2,917		
8	2017	1223	3,460	Stage 3a 3,600 to 9,400 EP	
9	2018	1476	4,177		
10	2019	1702	4,816		Stage 3b 4,700 to 9,400 EP
11	2020	1928	5,456		
12	2021	2166	6,131		
13	2022	2405	6,806		
14	2023	2644	7,481		
15	2024	2897	8,200		
16	2025	3237	9,161		
17	2026	3422	9,684	Stage 4 9,400 to 18,850 EP	Stage 4 9,400 to 18,850 EP

Project Year	Calendar Year	Cumulative Settlements	Cumulative Equivalent Population	Network Upgrade Stage	WRP Upgrade Stage
18	2027	3713	10,507		
19	2028	4050	11,463		
20	2029	4247	12,019		
21	2030	4587	12,982		
22	2031	5010	14,177		
23	2032	5226	14,790		
24	2033	5475	15,495		
25	2034	5558	15,728		

It can be seen that each upgrade Stage will provide capacity ahead of demand, with the subsequent Stage triggered as the capacity of the current Stage is reached in response to population growth.

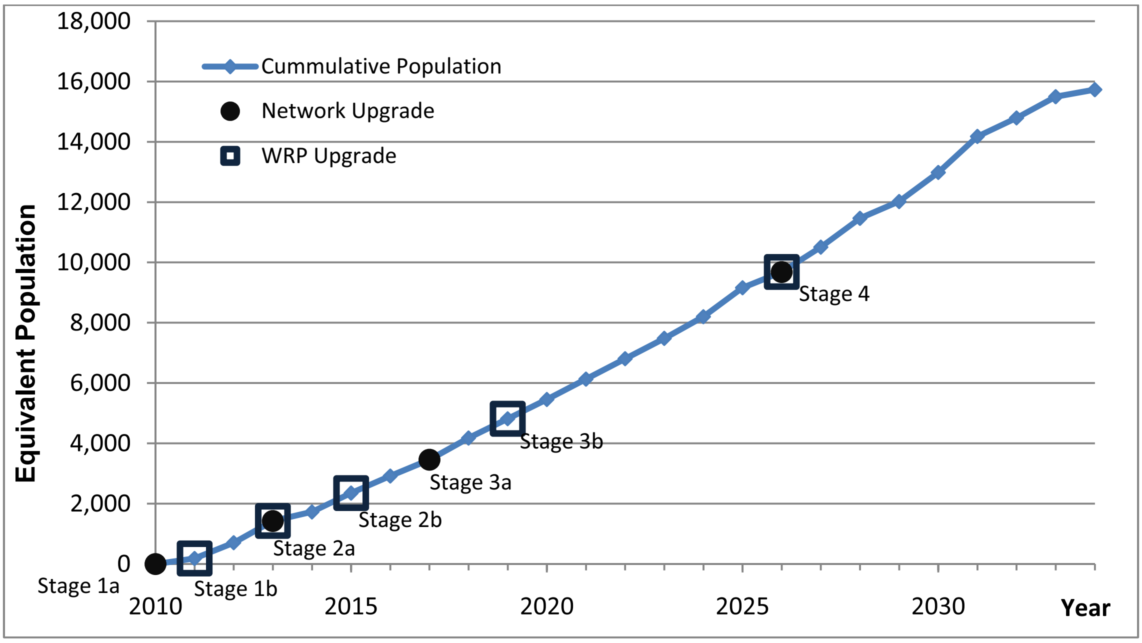


Figure 2-1 Graph of Googong Forecast Rate of Development

The Queanbeyan Residential and Economic Strategy 2031 (December 2008) places a cap of 5,550 dwellings within Googong. The strategy does however identify future investigation areas adjacent to the Googong re-zoning site, and so the IWC capacity has been designed for approximately 6,200 so as to enable any future expansion to be supported.

The population growth rates forecast by CIC Australia have been converted into Equivalent Population (EP) to account for all of the expected sewage flows and loads from both domestic and non-domestic sources. Table 2-2 shows the calculated total EP for Googong.

Table 2-2 Forecast Equivalent Population for Googong

Parameter	Number	Units
Final EP	18,850	EP
Final Dwellings	6,197	Dwelling

#*Excludes School and Commercial EP of 1678*

Googong will be progressively developed as a series of neighbourhoods which are constructed one after the other. The EP breakdown for the neighbourhoods is shown in Table 2-3. Further detail of the neighbourhood population forecasts is given in Appendix H.

Table 2-3 Predicted Equivalent Population by Neighbourhood

Development Precinct (NH)	Equivalent Population (EP)
NH1A	4,076
NH1B	1,395
NH2	5,323
NH3	2,529
NH4	2,611
NH5	2,196
Hamlet East	675
Hamlet West	44
TOTAL	**18,850**

2.2 Potable Water and Recycled Water Networks Design Flows

Googong will be supplied with both Potable Water and Recycled Water networks, designed as two completely separate systems. The split nature of the two water supply systems, and the innovative water saving features of the IWC for Googong, mean that traditional engineering design guidelines are not directly applicable. Therefore the concept designs for the potable and non-potable water systems are based on regular design guidelines, as well as an independently derived "end use" analysis. The two sources of information for sizing the water infrastructure are as follows:

1. Water Services Association of Australia Water Supply Code of Australia WSA03 – 2002 - 2.2, including Dual Water Supply Systems Version 1.2 Supplement to WSA 03.

2. Googong Design Assumptions for Potable and Recycled Water Systems, MWH, July 2010, Revision 4. This report has been included in **Appendix A.**

Although two separate systems, the potable network and recycled water network act together to meet the various internal and external water demands of Googong. The design assumptions document (MWH, 2010) outlines design demands established for the project, recognising the contribution of demand management measures, including the reuse of recycled water.

The water and recycled water design demand values proposed for sizing various aspects of the network are shown for Peak Day Demand and Peak Hour Demand in Table 2-4 and Table 2-5 respectively, subject to agreement with QCC. The Peak Day Demand values are typically used in the calculation of transfer pipelines from the source up to and including the service reservoirs. The Peak Hour Demand values are used for sizing the reticulation systems downstream of the service reservoirs.

Table 2-4 Peak Day Demand Summary

Peak Day Demand (ML/d)													
Potable				Recycled				Combined (Potable and Recycled)				Open Space Contribution	
Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2
0.2	0.8	1.7	3.4	0.9	2.3	4.6	9.3	1.1	3.1	6.3	12.6	0.5	0.9

Table 2-5 Peak Hour Demand Summary

Peak Hour Demand (ML/d)											
Potable				Recycled				Combined (Potable and Recycled)			
Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2	Stage 3	Stage 4
0.6	2.4	5.1	10.2	2.5	7.1	14.8	29.6	3.1	9.5	19.9	39.8

During the first five years of operation of the initial subdivision areas, system monitoring will be undertaken to confirm the design demand values. Monitoring of the performance of the system will provide real data to either validate or modify the design demands. To manage the risk associated with actual demands exceeding design assumptions, the sizing of the critical infrastructure with potential future constraints has been increased in the conceptual design phase:

- For Stage 1, excess capacity will be installed in the Potable Water Network. The bulk water pumping station and the delivery main to the reservoir site for the Stage 1 works are sized to meet the demand of not just Stage 1, but also Stage 2. After system monitoring data is available, any changes to demand assumptions can be incorporated into the next upgrade Stage.
- There is limited space available at the final reservoir site. However, sufficient space has been reserved to accommodate ultimate stage potable and recycled water reservoirs sized for maximum day demand volume plus at least 25%.

2.3 Reservoir Design and Performance Criteria

In accordance with section 2.7 of WSA 03 2002-2.2 Water Supply Code of Australia, and the Dual Water Supply Systems supplement to WSA 03, a number of criteria were considered during sizing of the service reservoirs. These are summarised in Table 2-6.

Table 2-6 Service Reservoir Design Criteria

Criteria	Adopted Approach
Operating storage versus pumping station or supply capacity	This is addressed by designing the BWPS with sufficient capacity to pump the combined potable and recycled water demands over a 22 hour period.
Reserve storage capacity	A minimum of 1/3rd maximum day demand has been provided as reserve storage for the low level reservoirs and 2hrs of maximum hour demand for the high level reservoir.
Water quality	The recycled water reservoirs receive water from two sources: the WRP and the ACTEW supply. The water quality within each recycled water reservoir will be maintained with the provision of a dedicated inlet and outlet main, to ensure turnover of water within the reservoirs, and with the provision of re-chlorination facilities and mixing.
Reservoir site aspect (space limitations)	There is limited space available for the provision of service storage capacity at the reservoir site. However, some additional space has been allocated to the largest reservoirs as a risk mitigation strategy against the possibility of actual demands exceeding anticipated demands. At the reservoir site, there is also limited elevation available to cater for the desired minimum residual water pressure of 20m (25m in commercial areas). This has resulted in the distribution of water storage across two separate elevations. The first, at a lower level, for the location of bulk storage, and the supply of low lying areas within the development. The second is on higher ground for the location of smaller elevated reservoirs used for the supply of elevated ground within the development.
Availability of emergency supply from adjacent systems.	The recycled water reservoir has an emergency backup supply with the potable system. The recycled reservoir sources of supply include: ▪ Recycled water via the recycled water pumping station and water recycling plant, ▪ Potable water via the bulk water pumping station and Stromlo WTP to Googong WTP transfer pipeline. Additional emergency measures have been taken to optimise pumping station reliability. These measures include: ▪ Provision of duty/standby pumping arrangements ▪ Provision of emergency generator connection points at each supply source.

Criteria	Adopted Approach
Other design criteria are as follows	▪ A minimum operating pressure head of 20m at residential properties has been designed for the recycled water system in accordance with WSA 03. The WSA03 supplement allows for differential pressure where specified by the water agency. MWH proposes the same minimum pressure. ▪ The main service reservoirs are able to service properties located below the 774m AHD contour with a minimum operating pressure of 20m. ▪ The high level reservoir requires a minimum reserve storage level of 815.5m AHD to provide an operating pressure of 20m.

2.4 Sewerage Network Design Flows

The sewerage network design flows are based on the Water Services Association of Australia Sewerage Code of Australia WSA02 -2002.

The sewerage catchment area is a significant factor in determining the design sewage flow. The catchment areas determined in calculations below exclude land uses within the development which do not contain a sewerage network. Public open space, drainage reserve, E2 Rural Zone and recreation areas, were excluded.

The sewage design flow or Peak Wet Weather Flow (PWWF) is the summation of the following three components:

- Dry weather sanitary flow;
- Groundwater infiltration (non-rainfall dependent); and
- Peak (rainfall dependent) inflow and infiltration.

Table 2-7 provides the values assumed for the key factors which influence the sewage flow calculation. For a more detailed analysis of the sewage design flow calculation methodology and assumptions, refer to the WSA Guidelines.

Table 2-7 Table of Adopted Key Sewage Design Flow factors

Factor	Value	Derivation / Basis
Sewage Generation Rate	180 L/person/day	WSA 02 Sewerage Code.
Groundwater Infiltration	Portion wet = 0	Proportion of pipe network with invert below water table is likely to be zero, based on geotechnical and groundwater level information obtained for the site.
Leakage Severity Coefficient	$C = S_{aspect} + N_{aspect}$ $S_{aspect} = 0.4$ $N_{aspect} = 0.4$	Soil condition and network conditions help determine rainfall dependent infiltration. Based on rock soil conditions, with trench stops, and a reasonable programme of maintenance in place.
Containment standard, (ARI)	3 months	One spill permitted every 3 months, in line with an allowable spill frequency of 40 per ten years elsewhere in NSW, subject to approval by DECCW.

The development assumes the installation of a conventional gravity sewerage system with a moderate leakage rate based on WSA 02-2002-2.2 guidelines. The use of a Reduced Infiltration Sewerage System (RISS) which would allow a lower leakage Severity coefficient to be used and result in lower design flows and reduced infrastructure costs, has been investigated and is promoted for the IWCMS. It is recommended that CIC Australia and Queanbeyan City Council (QCC) consider the use of the RISS approach at the detailed design stage to improve system performance and reduce long-term operating costs.

The sewerage design flows have been based on a 3 months Average Recurrence Interval (ARI) containment standard. This figure has been adopted in discussion with CIC Australia, but must be agreed and confirmed with DECCW in EA process and QCC during detailed design. Any changes to the containment standard will affect the Peak Wet Weather Flows (PWWF), and hence the sewerage reticulation and possibly the pump station design and peak WRP design flows. The adopted sewerage system design criteria are summarised in Table 2-8.

Table 2-8 Sewerage System Design Criteria

Wastewater Design Parameter	Value	Units	Basis
Average Dry Weather Flow (ADWF)	180	L/hd/d	WSA 02
Peak Dry Weather Flow (PDWF) / ADWF	2.2	:1	WSA 02
Peak Wet Weather Flow (PWWF) / ADWF	3.5	:1	WSA 02
Wet Weather Containment Standard (Average Recurrence Interval)	3	Month	WSA 02

2.5 Water Recycling Plant Design Criteria

2.5.1 Assumed Flows and Loads

The assumed influent flows, which have been calculated based on 180L/EP/d for ADWF, are outlined in detail in Table 2-9.

Table 2-9 Influent Flows to the WRP

IWC Stage	Units	Stage 1b	Stage 2a	Stage 2b	Stage 3b	Stage 4
Approximate development completion	%	5	12.5	25	50	100
Equivalent Population	nr	1,000	2,350	4,700	9,400	18,850
Flows						
ADWF	kL/d	180	423	846	1692	3393
	L/s	2.08	4.90	9.79	19.58	39.3
PDWF	kL/d	540	1269	2538	5076	7464
	L/s	6.25	14.69	29.38	58.75	86.4
PWWF	kL/d	900	2115	4230	8460	11875
	L/s	10.42	24.48	48.96	97.92	137.4

The flows given in Table 2-9 are greater than the corresponding flow rates in the water balance outputs (Site Water Balance Assessment Report, MWH Sept 2009) and hence have been adopted for design, as this corresponds to a conservative case. The discrepancy is generated by assuming a flow of 180 L/EP/day which has been adopted as the design standard, as given in Table 2-8. Using this conservative case allows variation from the forecast recycling rate or higher water usage of future residents. Using a conservative case allows for possible variations in the final development flows.

The biological loads used in the WRP design are listed in Table 2-9. The biological load treated at the plant is based on Equivalent Population and will not alter based on flow. Whilst water saving measures may reduce the ADWF, the mass of pollutant load produced per day from basic household functions remains the same, only the concentrations change.

Table 2-10 Influent Loads to WRP

Stage			Stage 1b	Stage 2a	Stage 2b	Stage 3b	Stage 4
Approximate development completion	%		5	12.5	25	50	100
Equivalent Population	nr		1,000	2,350	4,700	9,400	18,850
Loads		**per capita (g/hd/d)**					
COD	kg/d	120	120	282	564	1128	2262
BOD	kg/d	60	60	141	282	564	1131
SS	kg/d	65	65	153	306	611	1225
NH4	kg/d	10	10	24	47	94	188
P	kg/d	2.5	3	6	12	24	47

References:

COD = Data from other sites in NSW indicates that the COD:TKN ratio is between 8 and 10.5. The ration of 9.2 in this case gives a reasonable approach, whilst maintaining a conservative approach to the TKN.

BOD = Data from other NSW plants shows 54-60g/EP/d therefore a conservative approach has been taken

SS = This can vary widely in plants depending on the layout of the sewer system a nominal load used in other designs has been taken

TKN = Analysis of Sydney Water sites shows between 9 and 13.5mg/L therefore a conservative approach has been taken

TP = Data taken from other NSW sites

2.5.2 Effluent Conditions

The WRP is designed to produce Recycled Water for uncontrolled domestic reuse, from Stage 2 onwards, as the Recycled Water is reticulated throughout the Googong town. In Stage 1 the Recycled Water will not be available for distribution to the reticulation system. However, the same WRP effluent quality requirements will be applicable even at Stage 1, as the intent is to capture as much Recycled Water as possible for irrigation. The balance will be released to the environment.

The WRP concept design has been based on an assumed effluent quality required for discharge to a watercourse and an assumed quality for recycled water. However, these two assumptions mean:

- The effluent quality to discharge requires confirmation with DECCW.
- The Australian Guidelines for Water Recycling do not contain specific water quality targets. Whilst the design assumptions shown here are indicative, the actual requirements need to be gained from a formal risk assessment. The process of developing a recycled water management plan, including risk-based assessment of the scheme to ensure the recycled water product is fit for its intended use, will occur in parallel with ongoing design activities.

In normal conditions, effluent from the WRP will be directed to the Recycled Water Reservoir for distribution to the recycled water reticulation system for urban reuse. Effluent from the WRP which is in excess of the demand for urban reuse will be discharged to the nearest acceptable waterway from the recycled water main via the storm water system. In Stage 1, when reticulated Recycled Water is not produced, effluent not used for irrigation re-use will be discharged to the environment. Table 2-11 summarises the effluent quality criteria for each of the receiving watercourses for discharges from the WRP. Environmental discharge limits are defined as the 50th and 90th percentile, representing the limit within which values will fall for 50% and 90% of all measurements, respectively.

Table 2-11 Proposed Effluent Consent Conditions

Applicable Stage	Parameter	Proposed Consent Condition		
		Recycled Water [1]	Discharge to Environment	
			50 %ile	90%ile
All Stages (1-4)	BOD	-	5mg/L	10mg/L
	SS	-	10mg/L	20mg/L
	TDS		650mg/L	700mg/L
	Total Nitrogen	-	10mg/L	15mg/L
	Total Phosphorus	-	0.2mg/L	0.5mg/L
	Faecal Coliforms	< 1 cfu/100ml.		
	Total Coliforms	< 10 cfu/100ml (95%ile)		
	Virus	<1 cells/50L		
	Parasites	<1 cysts/50L		
	Turbidity	< 2 NTU		
	pH	6.5 – 8.0		
	Colour	< 15 TCU		
	Free Chlorine Residual	< 0.5 mg/l	-	

[1] NSW Guidelines for Urban & Residential Re-use of Reclaimed Water, May 1993

The Discharge to Environment conditions have been assumed as the conservative case based on earlier research of surrounding receiving waters and effluent quality. The final conditions need to be confirmed by NSW Department of Environment, Climate Change and Water (DECCW) during the discharge licence application process. Ideally a load based licence would be more appropriate to the Googong development due to varied volume of recycled water being produced.

The “Recycled Water” conditions have been determined from the NSW Guidelines for Urban & Residential Re-use of Reclaimed Water, May 1993, which have been superseded by the Australian Guidelines for Water Recycling, 2006. The latter is not a prescriptive guideline and as such does not provide water quality criteria. Instead, it requires a risk based approach to the development of the recycled water quality criteria. This exercise will need to be completed prior to commencement of detailed design.

The indicative pathogen log reductions achieved by different treatment processes can be seen in Table 2-12 below. The indicative values have been obtained from the Australian Guidelines for Water Recycling. The log reduction values associated with the specific process equipment used must be confirmed during detailed design to ensure the minimum log reductions are achieved.

Table 2-12 WRP treatment process with indicative log reductions

Treatment process unit	Indicative log reduction		
	Viruses	Protozoa	Bacteria
Secondary	0.5 - 2.0	0.5 - 1.5	1.0 - 3.0
Filtration	2.5 - 6.0	> 6.0	3.5 - 6.0
UV	> 1.0	2.0 - 6.0	1.0 - 4.0
Chlorination	1.0 - 3.0	0 - 1.5	2.0 - 6.0
TOTAL	**5.0 - 11.0**	**> 8.5**	**> 7.5**

3. Potable Water Supply

3.1 Operating Philosophy

The following section provides a description of the operational philosophy of the potable water supply system and should be read in conjunction with the piping and instrumentation drawings (for the ultimate system).

Under normal operation the new bulk water pumping station will draw water from the existing ACTEW transfer pipeline. This station will transfer water directly to the Potable service reservoir via the Potable Supply Main. For stages 1 and 2, potable water will be supplied to the Interim Potable Reservoir, at the interim site located adjacent to Old Cooma Road at RL765m AHD. For Stages 3 and 4, potable water will be supplied to the Permanent Potable Reservoir located on the Hill 800 saddle feature further south along Old Cooma Road at RL800m. By Stage 4, two potable reservoirs will be constructed at the permanent site.

Both the Interim and Permanent reservoir sites have large ground-mounted service reservoirs, as well as a smaller high-level zone system served from the main reservoir, with the addition of booster pumps and elevated tanks to provide additional gravity head. Both the Interim and Permanent sites will be operated in the same manner described in the following section.

To ensure the water in reservoirs is continuously cycled and the potential for water quality issues is reduced, and to reduce pressure fluctuations, no consumer off takes will be permitted off the BWPS delivery main. Furthermore, where the potable water system is used to top up recycled water demand throughout the development, air gap structures will be used to ensure the potable water is not contaminated.

The BWPS will be upgraded over time to provide more capacity to meet the potable water demand of the growing community. The BWPS will have an additional “standby” pump to provide spare capacity in the rare event of mechanical breakdown, and will be fully automated and controlled by a co-located PLC. The determination of alarms, remote monitoring requirements, remote manual operation and failure mode operation, etc, will be determined during detailed design in conjunction with the proposed operator.

The water level in the reservoir will be continuously monitored. When the level reaches the duty pump set point, the pumps will start and the reservoir inlet control valve will open, to allow the tank to fill. When the stop pump level is reached, the pump will shut down and the valve will close.

For Stage 4 onwards, when the BWPS is installed with a second duty pump, if the level in the reservoir continues to fall, the second duty pump unit will commence pumping until the pump set point has been reached. Once the reservoir level has increased to the set point, the pump unit will then shut down.

By Stage 4, a second potable reservoir will be required prior to ultimate development. The two reservoirs will be connected by a large diameter low-level gravity pipe, which will balance water levels between the reservoirs. Potable water stored in the service reservoirs will then be released into the potable water distribution system via gravity outlet mains.

A small high-level zone reservoir will be provided in Stage 4 to serve a small area at higher elevation that requires additional head to meet minimum residual pressure requirements. The potable water stored in the low-level service reservoirs will be used to fill the smaller high-level reservoirs. This will be achieved by connecting the suction pipework of the high-level reservoir pumping station to the potable water outlet main constructed for reservoir PW2. The pipework arrangement will allow water to be drawn from reservoir PW1, when reservoir PW2 is taken off-line for maintenance.

In the event of a power failure, PW1 and PW2 will have at least a third of a maximum day's demand below their reserve storage level. In addition, the bulk water pumping station will be provided with an emergency generator connection point, to allow the connection of a generator of sufficient capacity to permit the complete operation of this pumping station and all of its ancillary equipment.

The high-level reservoirs will be filled using the high-level reservoir pumping station. This pumping station will contain 2 pumping units at ultimate development in a duty/standby configuration and will be fully automated and controlled by a co-located PLC.

The water level in the reservoir PWHL will be continuously monitored. When the level reaches the set point for initiation of the duty unit, the pump will start and the inlet control valve will open. The duty unit will continue to operate until the water level reaches the set point for the duty unit shut down.

In the event of a power failure, reservoir PWHL will have at least 2 hours of reserve storage at maximum hour demand. In addition, the high-level reservoir pumping station will be provided with an emergency generator connection point to allow the connection of a generator of sufficient capacity to permit the complete operation of this pumping station and all of its ancillary equipment, as well as any other electrically actuated equipment and instrumentation located at the reservoir site. Once the reservoir supply is consumed, after 2 hours, the HLZ will still receive water from PW1 and PW2 reservoirs; however, this will occur at a reduced pressure.

Sodium hypochlorite is to be dosed into the drinking water stream to the Googong development to establish an appropriate level of residual chlorine at the homes of consumers, as specified in the Australian Drinking Water Guidelines. A chlorine dosing point will be provided into the common inlet main to PW1 and PW2 reservoirs. The dose rate will be based on one residual chlorine analyser on the feed water from BWPS, going into the tank and a flow signal. The dose rate will then be trimmed by a second chlorine analyser on the outlet to the reservoirs.

A second dosing point will be provided for the high-level reservoir. It will dose downstream of the HLZ booster pumps, using the analyser on the outlet of the low-level tank outlets and the flowmeter on the rising main up to the tank. Similarly, an analyser on the outlet of the HLZ tank would be used for trim control.

Dosing rates will be controlled by a proprietary PLC co-located within a proprietary sodium hypochlorite dosing system. This dosing system will be located as shown on drawing numbers A1081402- P301 and A1081402-P302. This dosing system will be provided with 100% standby dosing pumping capacity and a minimum sodium hypochlorite (conc.) storage capacity, equivalent to 30 days.

Mixers will be installed in PW1 and PW2 to assist with the mixing of sodium hypochlorite in each reservoir and to ensure uniformity of chlorine residual throughout each tank. These mixers will be submersible and continuously operational.

The potable water and recycled water systems will be kept entirely separate through each stage of construction. It will be necessary to manage change of operational heads to customers as the reservoir sites transition from the Interim Reservoir Site to the Permanent Reservoir Site.

It will be necessary to monitor potable water demands and recycled water demands at the feed points to the supply network to validate the design flow assumptions, for use in the next upgrade stages.

3.2 Bulk Water Off-take

3.2.1 Off-take Location

A water supply for Googong will require a connection to be made to the existing DN1800 main pipeline connection Googong WTP to the Queanbeyan and ACT bulk water network. A number of off-take location options were investigated, including a site in the grounds of the Googong WTP close to the clear water tank, as well as existing Queanbeyan off-takes located at Queanbeyan.

The existing Queanbeyan off-take locations were rejected because they are too remote from Googong, resulting in excessive pipeline and pump station costs. The site close to the Googong WTP clear water tank was rejected because its elevation will be too high to be supplied from the normal water source it the system, Stromlo WTP.

Following technical discussions with ACTEW representatives (ActewAGL), a site has been agreed for a new connection to the DN1800 main, located approximately 170m downslope from the Googong WTP clear water tank. The Bulk Water Pump Station must be designed to suit the minimum Hydraulic Grade Line (HGL) of 680m AHD and potential surge.

This site will require supply of power for the bulk water pump station, and an all weather access road will be run from the existing WTP access roads.

3.2.2 Off-Take Design Considerations

The off-take from the existing DN 1800mm transfer pipeline, will run in a cut and cover trench to the new Bulk Water Pump Station and enter the pump area above ground. The new off-take valve chamber will be constructed of reinforced concrete below ground. The top of this chamber will be provided with a lockable hinged grid-mesh cover and ladder. This chamber will be provided with a gravity drain with vermin cover draining to the creek system to the north-western side.

Works on the DN1800 main will be determined in consultation with ACTEW in the detailed design, but will require two DN450 off-take connections and two DN1800 valves in the main line for double isolation.

3.3 Bulk Water Pump Station

3.3.1 Purpose

The Bulk Water Pump Station (BWPS) will transfer flows from the DN1800 ACTEW pipeline to two separate points in the water supply network.

Potable Transfer: The first discharge will be to deliver flows via the Potable Water transfer main to the Potable Water Reservoirs. In the early stages, the discharge location will be to the Interim Reservoir Site located at Old Cooma Road, at RL765m AHD. In the later stages of the development, the discharge location will move to the Permanent Reservoir Site located on the saddle hill feature off Old Cooma Road at RL800m AHD. Because the flow and head requirements will increase over time with the increase in system demand and change in reservoir location, the pumping equipment and mechanical-electrical gear will be upgraded with each successive construction stage.

Potable Top Up of Recycled Water: The second discharge location will be to deliver flows via the Potable Water Top Up main to the Clear Water Tank at the WRP. This flow is used to top up shortfalls in the supply of Recycled Water to meet non-potable demand. An air gap will prevent cross-contamination between PW and RW systems.

3.3.2 Design Flows and Staging

To meet the staged requirements of flow and head as it increases over time, the BWPS construction will be staged to support the incremental increase in pump duty requirements. The Bulk Water Pump Station capacity is sized to meet the Peak Day Demand forecast, pumping for 22 hours in a day to allow some maintenance down time.

As the Recycled Water system serves mainly external demands, there will be periods when not enough recycled water can be produced to meet demand. In these occasions, potable water is used to top up the system to meet demand. This potable top up can be done either at the reservoir site, or at the WRP Recycled Water Pump Station. Table 3-1 shows that the adopted Potable Water top up location occurs at different places over the course of the Staging Strategy. It has been decided to take advantage of excess capacity in the potable system in the early years and carry out the top up at the reservoir site in Stage 2, whereas a dedicated component of the BPWS, and the corresponding transfer main, will be constructed at Stage 3 to deliver top up flows directly to the WRP's Clear Water Tank for supply to the Recycled Water Pumping Station. An air gap will be applied to manage the risk of cross-contamination where the potable water top up is used.

Table 3-2 summarises the capacity requirements for the Bulk Water Pump Station, for the two feed systems of Potable Supply and Potable top-up of Recycled Water. This table shows the Peak Day Demand (PDD) values taken from the demand forecasting work summarised earlier in Table 2-4. For each major upgrade stage, the required capacity of the Bulk Water Pump Station has been calculated from these PDD figures. The Potable Top Up of Recycled Water is sized for the full recycled water demand.

Table 3-1 Location of Potable Water Top Up of Recycled Water System

Stage	Potable Top Up Location	Comment
Stage 1 (0 to 1000 EP)	Not Required	No Recycled Water reticulation is provided in Stage 1, Potable and "Recycled Water" reticulation networks both fed from Potable Reservoir at interim Reservoir Site. Recycled water will be available at the environmental outfall to be used for irrigation.
Stage 2 (1000 to 3,600 EP)	Interim Reservoir Site	Sufficient capacity in BWPS and Potable Transfer main to meet Potable and Recycled Water Peak Day Demands.
Stage 3 (3,600 to 9,400EP)	WRP Recycled Water Pump Station (Clear Water Tank)	Dedicated Potable Top Up system feeding RWPS
Stage 4 (9,400 to 18,850EP)	WRP Recycled Water Pump Station (Clear Water Tank)	Dedicated Potable Top Up system feeding RWPS

The BWPS has been divided into two separate pumped systems to provide a more efficient staged construction to meet demand, to reduce operational complexity and improve pumping efficiency. The large difference in static head between the Potable Supply system and the Potable Top up of Recycled Water system, at around 140m and 55m respectively, mean that they will operate more simply as two separate pumped systems, rather than a combined system.

Table 3-2 summarises the required pumping capacity for the BWPS for each stage of the Potable Supply and the Potable Top up of Recycled Water systems. Table 3-3 shows how the design flows have been built up from the Peak Day Demand values and staging requirements.

Table 3-2 Bulk Water Pump Station Staged Capacity Summary

Stage	Potable Supply	Potable Top Up of Recycled Water
Stage 1 - 0 to 1000 EP	38 L/s @ 111m Duty/Duty/Standby Pumps	Not required
Stage 2 - 1000 to 3600 EP	No upgrade	Not required
Stage 3 - 3600 to 9400 EP	21 L/s @ 136m Duty/Standby Pumps	59 L/s @ 50m Duty/Standby Pumps
Stage 4 - 9400 to 18850 EP	43 L/s @ 142m Duty/Duty/Standby Pumps	118 L/s @ 55m Duty/Standby Pumps

Note: Pump duty heads to be verified in detailed design.

Table 3-3 Bulk Water Pump Station Staged Capacity Detailed Build-up

Stage	Configuration	Notes	Potable Peak Day Demand	Recycled Peak Day Demand	Total Demand	24/22 factor on PDD	Design Flow	Selected Capacity	RL PS upstream HGL	RL Discharge (Reservoir	Static Head	Dynamic Head	Total Head	Comments
			ML/d	ML/d	ML/d	ML/d	L/s	L/s	m AHD	m AHD	m	m	m	
Bulk Water Pump Station – Potable Transfer to Potable Water Reservoir														
Stage 1 - 0 to 1000 EP	Supply Interim Potable Reservoir	P+RW PDD for NH1A(1-2)	0.2	0.9	1.1	1.2	14	14	710	773	63	18	81	Upgrade for Stage 2 (37L/s)
Stage 2 - 1000 to 3600 EP	Supply Interim Potable Reservoir	P+RW PDD for NH1A(all)	0.8	2.3	3.1	3.2	38	38	680	773	93	18	111	
Stage 3 - 3600 to 9400 EP	Supply Final Potable Reservoir	P PDD 50% Development	1.7		1.7	1.9	21	21	680	805	125	11	136	Construct main BWPS upgrade, provide Duty/Standby Pumps
Stage 4 - 9400 to 18850 EP	Supply Final Potable Reservoir	P+RW PDD 100% Development	3.4		3.4	3.7	43	43	680	805	125	17	142	Additional Pump Unit to give Duty/ Duty/ Standby
Bulk Water Pump Station – Potable Top Up to WRP Recycled Water PS														
Stage 1 - 0 to 1000 EP	Not Required													Not required, recycled water not provided at Googong
Stage 2 - 1000 to 3600 EP	Not Required													Not required – Potable Water top up at reservoir, sufficient capacity
Stage 3 - 3600 to 9400 EP	Supply WRP Recycled Water Tank	RW PDD 50% Development		4.7	4.7	5.1	59	59	680	725	45	5	50	Construct main BWPS upgrade, provide Duty/Standby Pumps
Stage 4 - 9400 to 18850 EP	Supply WRP Recycled Water Tank	RW PDD 100% Development		9.3	9.3	10.1	118	118	680	725	45	10	55	Additional Pump Unit to give Duty/ Duty/ Standby

Note: Shaded cell denotes selected pumping duty for upgrade.

3.3.3 Site Layout

The BWPS site will be located approximately 170m north of the Googong WTP clear water storage tank immediately east of the existing DN 1800mm Googong WTP to Stromlo WTP transfer pipeline as it departs the Googong WTP site. The planned area of the pumping station site is approximately 40m long by 20m wide, including the valve chamber, pump slab, the MCC room, the access road and associated site works. The site is bounded by an existing four-wheel drive access track to the west along which the existing DN 1800mm transfer pipeline is located.

Layout details of this pumping station can be found on **Drawings SK-601** and **SK-602** for the Stage 1 and Stages 3 to 4 Upgrades respectively. The BWPS will consist of dry-mounted pumps located on a concrete slab, with associated piping and valves. The BPWS is located immediately adjacent to the DN1800 pipeline Off-take. The BWPS will include an unsealed access track, and outdoor electrical kiosk (Stage 1) and later a prefabricated electrical building (Stage 3). A pole mounted transformer and electrical supply will be required.

A new unsealed access track will be required for the BWPS. It is assumed that the existing road and turnaround area adjacent to the sludge drying beds at the WTP will be used for the first section of the road, which requires ongoing access through the ACTEW site, and a new road will be constructed from that point to the BWPS. The access road to the BWPS will be designed to provide heavy vehicle access to the site and platform to allow crane lifting of DN1800 valves on the ACTEW main, pumps and equipment at the BWPS. Heavy rigid vehicles will be provided with sufficient space and road modifications such that they will be able to complete a multi-point turn. On leaving the pumping station, these vehicles will be able to move directly on to the existing site road.

The BWPS site slopes steeply and falls approximately 4m over the footprint of the pumping station (approximate grade 1V:4H). No geotechnical investigations have been completed to date, however, the typical geology and topography suggests rock at shallow depth. The rear of the BWPS compound will be excavated into rock, with the front being built up in fill. The majority of the pumping station is likely to be constructed on a suspended reinforced concrete floor slab with pier supports founded on short piles to rock.

Noise studies are currently being undertaken by others to determine the potential noise impact of this facility on surrounding properties. If noise attenuation is required, acoustic covers for the pumps and valves could be provided.

3.3.4 Mechanical Equipment

Indicative mechanical pumping equipment sizing for the Potable Supply and Potable Top Up systems are summarised in Table 3-4 and Table 3-5 respectively.

Table 3-4 Bulk Water Pump Station – Potable Supply Pumps

Stage	Indicative Pump Configuration
Stage 1 - 0 to 1000 EP	Booster style pump system Skid mounted complete with valves and controls. 3 x 30 kW, 3000rpm fixed speed Soft Starters Nominal Inlet diameter: 150mm Nominal discharge diameter: 100mm Each 19l/sec x 111m head, Duty/Duty/Standby operation
Stage 2 - 1000 to 3600 EP	No Upgrade
Stage 3 - 3600 to 9400 EP	Horizontally mounted end suction centrifugal pumps 2 x 60kW, 3000 rpm,

	Variable speed drives Each 22 L/s x 142m head, Duty/Standby configuration
Stage 4 - 9400 to 18850 EP	Additional Pump, Duty/Duty/Standby configuration

Table 3-5 Bulk Water Pump Station – Potable Top Up of Recycled Water Pumps

Stage	Indicative Pump Configuration
Stage 1 - 0 to 1000 EP	Not Required
Stage 2 - 1000 to 3600 EP	Not Required
Stage 3 - 3600 to 9400 EP	Horizontally mounted end suction centrifugal pumps 2 x 75kW, 3000 rpm variable speed drives Nominal Inlet diameter: 250mm Nominal discharge diameter: 150mm Each 60 L/s x 55m head, Duty/Standby configuration
Stage 4 - 9400 to 18850 EP	Additional Pump, Duty/Duty/Standby configuration

Each pumping unit will be provided with a Non-Return Valve (NRV) for backflow prevention.

The elevation of the BWPS pumps has been determined using information provided by ActewAGL on the hydraulic grade line (HGL) of the existing DN1800 supply main at the connection location. The elevation selected allows for sufficient Net Positive Suction Head (NPSH) available to prevent cavitations occurring in the pumps and is based on a minimum HGL in the DN1800 supply main of **680mAHD** as advised.

Suction pipework will be fitted with a low suction pressure alarm that will initiate shutdown of all operating units when pressures less than the minimum suction pressure set point occur. Further, discharge pipework will be fitted with automatic air valves to allow purging of any entrained air. Pumping units will also be provided with a hard- wired no-flow alarm that will initiate pump shut-down in the event of no-flow conditions arising.

To cope with possible low suction pressures and negative surge in the DN 1800mm pipeline, NRVs are proposed on the discharge side of the pumps. In addition, a bypass pipeline between the BWPS suction pipe and discharge pipe, incorporating an NRV and a Surge Anticipator Valve (SAV), is proposed, for each of the two discharge pipeline systems. The bypass pipeline allows positive surge pressures generated within the DN 1800mm pipeline to bypass the pump station, thus protecting the pumps.

To prevent surge effects in the BWPS and delivery main system, approximately three Anti-Slam Air Valves (ASAV) and one NRV are proposed along each of the delivery mains. The bypass pipes allow negative surge pressures within the delivery pipework to draw water through the bypass, thus decreasing the surge effects at the BWPS.

A detailed water hammer investigation is to be completed during detailed design to further assess the effectiveness of this bypass arrangement in light of any additional available information. The investigation is to consider surge from the ACTEW 1800 pipeline and from the Googong system.

Each pumping unit will be provided with manual isolation valves upstream and downstream, and a NRV on the downstream side. DN 25mm pressure tapings will also be provided on pipework upstream and downstream of each unit to facilitate commissioning activities and ongoing operation and maintenance activities.

The discharges of each pumping unit will be connected to a common manifold that will be connected to the delivery main on one end and a scour outlet on the other. The scour will be drained to the creek system to the north-west of the site. A flow meter will also be provided on each of the two discharge pipes at the BWPS.

3.4 Potable Transfer Mains

3.4.1 Purpose

Potable water from the BWPS will be transferred to two separate locations, each with a dedicated pipeline.

The **Potable Supply Main** will deliver flows from the BWPS to the Potable Water Reservoir. In Stage 1 and 2, the potable Water Reservoir will be located at the Interim Reservoir Site adjacent to old Cooma Road. At Stage 3 the Potable Supply Main will be extended to supply the Potable Water Reservoir, located at the Permanent Reservoir Site at the RL800 saddle feature further south on Old Cooma Road.

The delivery main will enter through the concrete floor of the reservoir to avoid excessive loads on steel reservoir walls. In addition, the delivery main will also discharge into the recycled water reservoir via an up-and-over pipe in order to provide top-up water to the recycled water system, whilst maintaining an air gap to prevent any risk of cross- connection contamination of the potable supply.

The **Potable Top Up of Recycled Water Main** will deliver flows from the BWPS to the WRP Clear Water Tank. This pipeline will not be required until Stage 3, as sufficient capacity is available in the Potable Supply Main at the earlier stages to allow potable top up at the Interim Reservoir Site. An air gap will eliminate cross contamination between the PW and RW networks.

3.4.2 Staging and Pipeline Route

Several options are available in the staging and configuration of the Potable water supply pipeline and top up of the recycled water system. Two key options that have been considered and rejected include:

- Top up of recycled water with potable water at the Permanent Reservoir Site – This option requires construction of a large diameter pipeline all the way up to the reservoir site at the saddle feature at RL800. It was found that simply pumping to the WRP Recycled Water Tank, and using the available capacity of the Recycled Water Pump Station, was a lower cost option with a simpler operational requirement.
- Multiple smaller diameter pipelines constructed at shorter intervals – The option of installing smaller diameter pipelines, and coming back one or more times to constructed additional capacity over time, in line with population growth, was found to be highly disruptive to established road corridors and of little benefit from a Net Present Cost perspective.

The potable pipeline staging and sizing is summarised in Table 3-6. Because the BWPS and associated transfer mains will be staged in their delivery, different flow conditions will exist in the pipelines over time. For example, in Stage 1, the Potable Supply Main will have sufficient capacity to meet the potable and “recycled” water demands. By Stage 3, a dedicated pipeline will be required for the Potable Top Up of Recycled Water.

Table 3-6 Potable Transfer Main Details

Pipeline	Stage Constructed	Sizing Criteria	Diameter Nominal (mm)	Length (m)	Design Flow-rate (L/s)	Design Velocity (m/s)	Pipe Material and Class
Potable Supply Main	Stage 1	Ultimate (Stage 4)	DN225	4,100	38	1.0	DICL NP35
Potable Supply Main Extension	Stage 3	Ultimate (Stage 4)	DN225	1,460	43	1.0	DICL NP35
Potable Top Up of Recycled Water	Stage 3	Ultimate (Stage 4)	DN375	1,375	117L/s	1.0	DICL PN35

In the early years, these pipelines may experience flows lower than the design velocity, which are typically sized for the future development. This coupled with the very low chlorine residual from the ACTEW main may lead to bacterial re-growth within the delivery main and possible water quality issues. As a result, it may be necessary to periodically exercise this main with full flow rates to reduce the potential for bacterial re-growth. This can be done by introducing a flushing cycle in the operation of the bulk water pumping station.

The flushing cycle will require operation for around 2hrs and transfer approximately 1-2ML of water to reservoirs. The timing of this operation will need to be negotiated with the pumping station operator during detail design.

The delivery main will be laid at minimum depth in accordance with the approved materials and provisions of WSA 03 and fabricated of Ductile Iron Cement Lined (DICL) pipe, class PN35. DICL PN35 was selected for the pipe material because it is approved by Council and is in common industry use, but final pipe specification will need to be confirmed in detailed design in line with surge analysis and final design pressures.

From the BWPS, the buried pipelines will run along a constrained pipeline easement, still to be negotiated with ACTEW, containing the existing DN1800 ACTEW pipeline. The route passes through the Googong WTP site adjacent to the existing site roads, and turns westward into the verge of Googong Dam Road. The pipelines follow the road verge to their reservoir or WRP discharge locations for the respective pipelines. Small vehicle access will be possible along the pipeline route to inspect all accessible assets such as air valves, scour valves and any above ground infrastructure.

3.4.3 Mechanical

Mechanical fittings will be designed and installed in accordance with relevant guidelines. As a minimum, scour valves and pits will be provided at all low points with scour pipelines to the nearest stormwater system or drainage pathway. Manual air valves will be provided at all high points, reductions in upstream slope and increases in downstream slopes. Air valves shall also be provided at not greater than 800m intervals for all long horizontal, descending or ascending sections.

The Potable Supply Main will discharge through the concrete floor of the Potable Reservoir. Actuated Inlet Control Valves (AICVs) will be installed on the delivery mains at the inlet to both low level reservoirs. The provision of automatic air valves and sectioning valves will be determined during detailed design in consultation with the proposed pipeline operator.

The Potable Top Up of Recycled Water Main will discharge to the WRP Recycled Water Tank, with an air gap to prevent the possibility of cross contamination of the potable supply.

3.5 Service Reservoirs

3.5.1 Purpose

Service Reservoirs will be constructed to supply Potable Water to the Googong community. Recycled Water Reservoirs will also be constructed to support the parallel recycled water supply network.

Service reservoirs will be sited on high ground to meet Council's minimum residual pressure requirements of 20m head for domestic areas and 25m for commercial areas. Reservoir construction will be staged to better match the rate of development of Googong.

For Stages 1 and 2, the Service reservoirs will be initially constructed at the Interim Reservoir Site located adjacent to Old Cooma Road. At Stage 3, the Interim Reservoir Site will be demolished and service reservoirs will be located at the saddle feature known as Twin Hills at RL800 further south along Old Cooma Road.

3.5.2 Layout and Staging

3.5.2.1 Interim Reservoir Site

The **Interim Reservoir Site** has been selected to provide sufficient residual gravity head to the early subdivisions to meet Councils requirements. The site is to be located some 100m from Old Cooma Road at approximately RL765m AHD and is shown in plan and section on **Drawings SK-303** and **SK-304** respectively. At Stage 1, the Googong subdivisions will be supplied with Potable water only, from a single Potable Water reservoir. These early subdivisions will be constructed with both Potable and Recycled Water reticulation systems; however both systems will be supplied from the Potable Water reservoir.

At Stage 2, additional subdivisions to the East of the earliest areas will include some light commercial areas and are at slightly higher ground elevations. While these areas will still have a positive residual head, it will not be possible to supply some of these areas with the minimum 20m and 25m residual gravity head for domestic and commercial requirements respectively. Therefore it will be necessary to construct booster pumps and small elevated storage tanks, to supply the higher pressure network in these areas. An option at Stage 2 to reduce the visual impact of 25m high elevated tanks is to simply run these areas with in-line booster pumps with appropriate mechanical standby and connection points for an emergency generator. This requires risk assessment and confirmation from QCC, to be completed at later stages of design.

The site layout includes the 10m high covered steel Potable Water Reservoir and Recycled Water Reservoir, a Chlorine dosing kiosk, booster pumps, elevated storage tanks and site access tracks.

Table 3-7 Interim Reservoir Site Staging

Stage	Works
Stage 1 – 0 to 1,000 EP	Potable Water Reservoir Chlorine Dosing Kiosk Site Works
Stage 2 – 1,000 to 3,600 EP	Recycled Water Reservoir Potable Water Booster Pumps Recycled Water Booster Pumps Potable Water elevated Tank Recycled Water Elevated Tank
Stage 3 – 3,600 to 9,400 EP	Decommission and Demolish site

3.5.2.2 Permanent Reservoir Site

The Permanent Reservoir Site will be located at the Twin Hills site situated on the highest ground within the development. A level platform will be created at RL 795 mAHD to site the potable and recycled water reservoirs, and at RL 810 mAHD for two very small high level reservoirs (PWHL and RWHL), which are required to service a small local high level zone. The main reservoir platform will be constructed to full size at the beginning of Stage 1. It is possible that a low retaining wall will be required at the southern boundary of the reservoir site, depending on the final cut slope. This will need to be confirmed during detailed design.

The main service reservoirs at the Permanent Reservoir Site will have a minimum storage volume equivalent to the total Peak Day Demand for the development, for the respective potable and recycled water systems. From Stage 3 onwards, the majority of Googong will be supplied by gravity from these reservoirs.

The elevated reservoirs have been used to locate much smaller potable and recycled water reservoirs. These reservoirs will be much smaller and fed by a pumping station that draws water from the low-level reservoirs. These reservoirs will supply, by gravity, approximately 830 dwellings of the development that is located on higher ground within NH2, NH3 and NH4. The pumping station supplying the high-level reservoirs will be located adjacent to the low-level reservoirs. Access to these reservoirs will be provided from the Old Cooma Road.

Table 3-8 shows the proposed staging of the works at the Permanent Reservoir Site. At Stage 3, a Potable Reservoir and a Recycled Water Reservoir will be constructed at the site, complete with Chlorine dosing kiosk, access tracks and associated site-works.

At Stage 4, a larger Recycled Water Reservoir will be constructed, and the earlier Recycled Reservoir will be converted to a Potable Reservoir, leaving two smaller Potable Reservoirs and a large Recycled Reservoir at the site. At some point in Stage 4 the small Potable and Recycled Water High Level Zone service reservoirs will be constructed to service the local area near the hill feature.

Table 3-8 Permanent Reservoir Site Staging

Stage	Works
Stage 3 – 3,600 to 9,400 EP	Potable Water Reservoir Recycled Water Reservoir Site Works
Stage 4 – 9,400 to 18,850 EP	Convert Recycled Water Reservoir to second Potable Reservoir Recycled Water Reservoir Booster Pumps for High Level Zone Potable High Level Storage Recycled Water High Level Storage

The geotechnical investigation indicates rock is present at the reservoir site at depths ranging from 0.4m to 1.6m. Therefore, some rock excavation work will be required to create the level platform. The positioning of the reservoirs is such that each reservoir will be placed on in-situ ground and not on any fill material. The layout of the permanent reservoirs is based on maintaining a minimum distance between the tanks of 12m.

A 5m wide unsealed access road will be constructed from the nearest local road to the reservoir site. The road has been designed to accommodate a standard 12.5m rigid flat-top truck, as appropriate for general maintenance vehicles and a rigid 16,500L sodium hypochlorite delivery vehicle. A larger, 21,000L 19m long semi-articulated vehicle, has been checked and some kerb mounting would be anticipated in the lower level reservoir site. The larger vehicle would not be able to negotiate the high level reservoir access road. Orica have confirmed that both sized vehicles are available in the ACT/Queanbeyan area.

A minimum 5m wide access track will be provided around the perimeter of each reservoir, to allow access for maintenance vehicles.

The chemical unloading area adjacent to the chemical dosing unit should be provided with a sealed unloading area with a formed bund around the perimeter with capacity equivalent to 110% of the largest tanker truck required to attend site.

3.5.3 Reservoir Size

3.5.3.1 Reservoir Sizing Calculations

The service reservoirs will be sized giving consideration to the criteria outlined in section 2.7 of WSA 03, using the Peak Day Demands outlined earlier in Table 2-4. A larger footprint has been preserved for the ultimate stage reservoir at the Permanent Reservoir Site as a risk mitigation measure against unexpectedly high water demands.

The Staging Strategy for Googong has adopted the Interim Reservoir Site for Stage and Stage 2 water supply. Because Stage 1 will be supplied from a single Potable Reservoir, this reservoir is sized for the larger of two cases, being the Stage 1 Potable and Recycled Water PDD; and the Stage 2 Potable PDD Only.

Table 3-9 shows the selected Service Reservoir for Potable and Recycled Water, at each stage of the development.

Table 3-9 Reservoir Size and Stage

Stage	Service Reservoir	Sizing Criteria	Potable PDD	Recycled Water PDD	Total PDD	Selected Capacity	Comment
			ML/d	ML/d	ML/d	ML	
Stage 1 - 0 to 1000 EP	Interim Potable Reservoir at RL765 Cooma Rd	P+RW PDD (1000EP)	0.2	0.9	1.1	1.1	
Stage 2 - 1000 to 3600 EP	Interim Potable Reservoir at RL765 Cooma Rd	P PDD (3600EP)	0.8		0.8		No upgrade, Stage 1 size still ok
	Interim Recycled Reservoir at RL765 Cooma Rd	RW PDD (3600EP)		2.3	2.3	2.3	
	Interim High Level Potable Reservoir					0.05	
	Interim High Level Recycled Reservoir					0.15	

Stage	Service Reservoir	Sizing Criteria	Potable PDD	Recycled Water PDD	Total PDD	Selected Capacity	Comment
			ML/d	ML/d	ML/d	ML	
Stage 3 - 3600 to 9400 EP	Potable Reservoir at RL800	P PDD (9400EP)	1.7		1.7	1.7	
	Recycled Reservoir at RL800	RW PDD (9400EP)		4.7	4.7	4.7	
Stage 4 - 9400 to 18850 EP	Potable Reservoir at RL800	P PDD (18850EP)	3.4		3.4		Convert Stage 3 RW Reservoir to Potable
	Recycled Reservoir at RL800	RW PDD (18850EP)		9.3	9.3	9.3	
	High Level Potable Reservoir					0.08	
	High Level Recycled Reservoir					0.45	

3.5.3.2 Interim Reservoir Site

Based on the service reservoir sizes shown above in Table 3-9, specific dimensions and elevations of the reservoirs are summarised in Table 3-10. The elevated storage tanks shown here are sized for 30mins at Peak Hour Demand. In detailed design of the network distribution system for Stage 2, these tank sizes may be reduced to serve the High Level Zone only rather than all of Neighbourhood 1A (subdivision 3-6).

Table 3-10 Interim Reservoir Site – Reservoir Dimensions

Potable Water Reservoirs	Interim Potable Reservoir	Interim Potable High Level Tank
Storage Volume (ML)	1.1	0.050 (50m3)
Tank Diameter (m)	12	4.0
Tank Invert Level (mAHD)	765	785
Minimum Operating Water Level (mAHD)	765.5	
Reserve Storage Level (mAHD)	769	
Reserve Storage Volume (ML)	0.40	

Potable Water Reservoirs	Interim Potable Reservoir	Interim Potable High Level Tank
Top Water Level (mAHD)	775	789
Freeboard (m)	0.5	0.5
External Tank Height (m)	10.8	24.5*
Top of Roof Level (mAHD)	775.8	789.5
Inlet Pipe Diameter (mm)	225	200
Outlet Pipe Diameter (mm)	200	200
Overflow Pipe Diameter (mm)	300	300

* Note final tank height will be determined in detail design.

For Stage 1, the required residual gravity operating pressure of 20m head of water is available at the reservoir reserve storage level, in accordance with the WSA 03 Water Supply code.

If a reduced minimum operating pressure is acceptable to CIC Australia and QCC, it is possible to lower the overall height of the tanks, reducing the visual impact. This would require an increase in tank diameter in order to maintain the required reserve storage volume. These details should be confirmed during detailed design.

For Stage 2, booster pumps and elevated tanks have been adopted to meet the residual gravity head for the higher elevation areas in Neighbourhood 1, including the light commercial areas requiring an increase to 25m.

A telephone line will be connected to the MCC on site, as part of the SCADA control and telemetry requirements. A potable water connection from the reservoir outlet mains will be available on site for the safety shower washdown facility in the chemical dosing unit, and for general usage, such as drinking and hand washing.

3.5.3.3 Permanent Reservoir Site

The Permanent Reservoir Site located at the saddle feature adjacent to Old Cooma Road will require staged construction of five Potable and Recycled Water service reservoirs. The dimensions of the proposed service reservoirs are summarised in Table 3-11.

Table 3-11 Permanent Reservoir Site – Reservoir Dimensions

Potable Water Reservoirs	Permanent Potable Service Reservoir 1	Permanent Potable Service Reservoir 2	High Level Zone Reservoir
Storage Volume (ML)	1.7	Convert 4.7ML Recycled Water Reservoir to Potable*	0.08 (80m^3)
Tank Diameter (m)	14.7		5
Tank Invert Level (mAHD)	795		810
Minimum Operating Water Level (mAHD)	795.5		810.5
Reserve Storage Level (mAHD)	799		814.5

Potable Water Reservoirs	Permanent Potable Service Reservoir 1	Permanent Potable Service Reservoir 2	High Level Zone Reservoir
Reserve Storage Volume (ML)	0.60		0.08
Top Water Level (mAHD)	805		815.5
Freeboard (m)	0.5		0.5
External Tank Height (m)	10.8		6.3
Top of Roof Level (mAHD)	805.8		816.3
Inlet Pipe Diameter (mm)	225		200
Outlet Pipe Diameter (mm)	375		200
Overflow Pipe Diameter (mm)	450		300

‘* It should be noted that only 1.7ML additional potable storage is required at the final Stage 4 upgrade, hence use of the 4.7ML RW tank as the new potable tank provides well in excess of the Peak Day Demand for potable water.

3.5.4 Tank Construction

The tank construction material will be bolted steel panels constructed on a reinforced concrete base. A concrete ring beam will be constructed around the circumference of the reservoirs. The bottom panels will be affixed to a base plate attached to the ring beam. The tanks should be lined with a water grade epoxy. A gasket seal will be installed between the bottom panel and the concrete ring beam to achieve a watertight finish.

Inlet and outlet pipework for the reservoirs should be constructed through the concrete floor, rather than the side of the reservoir, as bolted panel tanks are less able to resist lateral thrust forces. Advice from the tank supplier should be sought during detailed design

The reservoirs will have aluminium or treated steel roof. Access hatches or ports will be required on the roof to enable the submersible mixers and level control instrumentation to be installed and maintained. Access and railing requirements to be determined in detailed design.

Ventilation will be required to control levels of chlorine in the airspace above the water. Ventilation will reduce moisture on the roof and corrosion. Typically a central spinner and side vents will be installed. A typical 10m diameter reservoir is shown in Figure 3-1.

Figure 3-1 Example of Typical Steel Service Reservoir

Roof runoff from each reservoir will be collected in guttering around the roof edge and discharged into the combined scour and overflow discharge chamber. A half-channel surface drain will be provided around the perimeter of each reservoir to collect surface runoff.

The proposed reservoir compound roads will be unsealed and consist of appropriately graded and compacted granular material. Construction with an appropriate camber will allow rainfall to runoff and infiltrate the ground or be collected into the surface channels proposed around the reservoirs. Therefore, formal kerb, gulley and road drainage are not proposed.

3.5.5 Mechanical

3.5.5.1 Reservoirs

In addition to the inlet and outlet pipes, the reservoirs will be provided with a high-level overflow pipe, to prevent water levels rising to uncontrolled levels in an emergency. A scour outlet pipe will be provided in the bottom of the reservoir to allow the tank to be drained completely for routine maintenance. Both the scour outlet and overflow pipework will discharge to a combined chamber and from there into the stormwater drainage system.

Manual inlet isolation valves will be installed on the incoming lines to the various reservoirs. A large diameter pipe is proposed to connect the two Permanent (Stage 3 and 4) Potable reservoirs, to enable the water levels to balance across the reservoirs. The outlet mains are also proposed to connect onto this pipe, with an isolation valve installed at each end, adjacent to the reservoir, to enable one tank to be taken offline for maintenance. This arrangement should be confirmed during detailed design.

Submersible mixer(s) should be installed in the reservoirs. The mixers should be sized to provide suitable mixing energy and sufficient number of mixers to suit the final tank geometry. Details shall be confirmed during detailed design. For maintenance purposes, the mixer should be installed on a guide rail which can be accessed from a cover located on the reservoir roof. Mixers are not considered necessary for the elevated tanks and high -level reservoirs because of the short residence time, and given that the water quality is nominally managed in the larger reservoir.

A potable water supply will be provided to site, for the safety shower wash down facility in the chemical dosing unit, and for general usage such as drinking and hand washing.

3.5.5.2 Permanent High Level Zone Pumping Station

A high-level reservoir is required to maintain sufficient water pressure supply for the high-level properties within the development. A pumping station will be located at the low-level reservoir site and will transfer water from reservoirs PW1 and PW2 to the high-level reservoir. The pumping station will be constructed in Stage 4 of development, as it is not required to be operational until the high-level properties, located above RL 774 mAHD in NH 3, 4 and 5, are built.

A cross connection should be provided from the low level reservoir outlet mains to the high level zone outlet pipework. A Pressure Sustaining Valve should be provided on the cross connection to allow water from the low level reservoirs to provide water to the high level zone potable water distribution network in an emergency situation. This water would be supplied at a lower positive pressure.

The pumping station will be located in the main reservoir compound, adjacent to reservoirs RW1 and PW1. It is proposed to install both the potable and recycled water pumps on the same raised concrete plinth. Two above-ground, dry-mounted potable water pumps will be installed in a duty/standby configuration. The pump suction main, will draw water from the outlet main constructed for the ultimate stage of development. The pumps will be sized for Peak Hour Demand. Pump details will be determined in Detailed Design.

3.5.6 Secondary Water Conditioning/Chemical Dosing

Googong will obtain water from the Googong WTP to Stromlo WTP 1800mm diameter transfer pipeline. Water quality data has been obtained from ActewAGL for the following sampling locations:

- Googong WTP supply;
- Stromlo WTP supply (after passing through Canberra);
- Queanbeyan water supply off-take #1; and
- Queanbeyan water supply off-take #2.

An assessment has been carried out of the drinking water quality from two treatment plants and is presented in **Appendix B**. In this assessment, it was seen that the water has a low alkalinity, however a high pH. Chlorine Residual data are summarised in Table 3-12.

Table 3-12 Flows and Loads Based on Potable Water Received from Googong and Stromlo WTPs

Parameter	Stage 2a	Stage 4 (Ultimate)	Source
Flows (ML/d)	0.6	2.6	See water balance assessment (Appendix A) for appropriate average day flows
Min Chlorine (mg/L)	0.3		Stromlo data – ACTEW AGL
Average Chlorine (mg/L)	1.5		Average of Stromlo and Googong data- ACTEW AGL
Max Chlorine (mg/L)	5.4		Googong maximum - ACTEW AGL

During periods when supply is from the Stromlo WTP, the new Googong development off take will be at the end of a large and long dead end pipeline. When supplied from the Googong WTP, water quality within the transfer pipeline is expected to improve at the Googong off take, such that re-chlorination demand will be reduced or not required. Other than Chlorine Residual, it has been assumed that ACTEW will provide water that meets the Australian Drinking Water Guidelines, regardless of supply source.

Therefore secondary water conditioning is required to ensure compliance with the Australian Drinking Water Guidelines for chlorine residual. This facility will dose sodium hypochlorite with the objective of meeting the requirements of Australian Drinking Water Guidelines for chlorine residual. Two dosing points will be provided. These will be on the inlet pipeline to the service reservoirs and a second top-up dosing point will be provided on the discharge main of the high level reservoir pumps. The use of two dosing points will also be applied to the interim configuration. The target performance requirements are nominated in Table 3-13.

Table 3-13 Performance Targets for Water Quality

Parameter	Minimum Value	Maximum Value	Units	Notes
Free Chlorine Concentration	0.1	2.0	mg/L	Free chlorine is to be adjusted as part of secondary water conditioning.
pH Range	6.9	8.2	-	Current discharge requirement for Googong and Stromlo WTPs. The pH will not be adjusted as part of secondary water conditioning.

Note: The pH is based on current water quality and no pH correction allowed for

The target free-chlorine concentration has been specified to a maximum of 2.0 mg/L, which is below the Australian Drinking Water Guidelines maximum of 5 mg/L, but high enough to ensure a residual level of chlorine to maintain disinfection. The target free-chlorine minimum has been set to 0.1 mg/L, as a guide for the design of dosing pump turndown.

For the chemical dosing facility, two chemical storage tanks are proposed for all the dosing points (both potable and recycled water) at the Interim Reservoir Site, and then transferred to the Permanent Reservoir Site. Each tank will be a volume of 15m^3 and the chemical dosing unit (CDU) will contain pumps for all dosing points.

This CDU may be supplied as a prefabricated unit or constructed on-site, but has been assumed to be a building or enclosure for security purposes as the reservoir site is deemed to be a remote site. The chemical tanks are assumed to be inside, however they may be placed outside if a suitable method of security can be found.

Table 3-14 summarises the chemical dosing equipment to be installed for the potable water system.

Table 3-14 Design Criteria for Chemical Dosing: Potable Water at Stage 4

Component	Value	Units
No. of chemical storage tanks	2	-
Volume of storage tanks	15	m^3
Number of dosing pumps	6 (2 for each dosing point)	-
Arrangement	Duty/Standby	-

Mechanical works associated with the sodium hypochlorite secondary conditioning system would include delivery kiosk, sodium hypochlorite storage tanks, dosing pump skids, pipework/valves/fittings, trace heating and insulation on above ground pipework, safety showers and eyebaths, ventilation fans and wash down hose. For 10% (w/w) sodium hypochlorite, the temperature of the tank needs to be maintained above 0°C to prevent crystallisation in the tank and lines. Therefore, appropriate trace heating and lagging should be used on above ground pipe work and tanks.

Sodium hypochlorite corrodes stainless steel, galvanised mild steel and most other metals. These materials should not be used in the construction of the dosing facility. The materials specification applies to areas where sodium hypochlorite vapour is prevalent also.

A typical contained chemical dosing unit is shown in Figure 3-2.

Figure 3-2 Example Self Contained Chemical Dosing Unit

The examples in Figure 3-2 illustrate two different size installed units. The units are delivered as containerised units and are self contained. They provide the bund required to contain spills, chemical dosing pumping skids and safety showers. They also ensure that the chemical tanks are kept secure which is important, particularly in off-site areas, such as the reservoir area.

4. Recycled Water System

4.1 Recycled Water Operating Philosophy

This text should be read in conjunction with the piping and instrumentation drawings. Figure 6-1 shows a schematic of the recycled water system.

Under normal operating circumstances, from Stage 2 onwards, fully treated recycled water from the WRP will be pumped from the recycled water pumping station at the WRP to the recycled water reservoir. The same WRP effluent quality will be produced in Stage 1, with recycled water to be collected for use at the Basin 1 discharge point and used for irrigation of the Googong development.

As for the potable water supply system, there will be an Interim Recycled Water Reservoir located at RL765 adjacent to Old Cooma Road, close to Googong Dam Road, and then at Stage 3 a Permanent Reservoir site located at the Twin Hills site off Old Cooma Road. Both sites will operate in the same way, as they have a main ground-mounted service reservoir, plus a high-level zone system with booster pumps and elevated tank to provide minimum residual pressures to the HLZ network.

If recycled water production exceeds usage then the recycled water service reservoir will be unable to receive recycled water flows. Excess recycled water will be discharged from the recycled water main through Basin 4 and the storm water system to the environment.

Level sensors within the recycled reservoir will control operation of the Recycled Water Pump Station (RWPS), located at the WRP.

For Stage 2, local High Level Zone (HLZ) recycled water elevated tank and booster pumps will be required to service properties at high elevations in Neighbourhood 1A. As Stage 3 is constructed, the interim Reservoir Site will be decommissioned and the NH1A networks supplied from the Permanent Reservoir Site.

For Stage 4, a local High Level Zone (HLZ) recycled water reservoir (RWHL) is required to service properties in NH2, NH3 and NH4 located above the 774mAHD contour and will be constructed prior to development above the 774m AHD contour. A HLZ pumping station and delivery main will transfer water to the RWHL reservoir. Suction pipework for the pumping station will draw water from the recycled water outlet main constructed for reservoir RW2. The pipework arrangement will enable water to be drawn from reservoir RW1 to allow reservoir RW2 to be taken off-line for maintenance.

For both the Interim and Permanent sites, a cross connection will be needed from the main recycled water reservoir outlet mains to the high level zone outlet pipework. A Pressure Sustaining Valve shall be provided on the cross connection to allow water from the low level reservoirs to provide water to the high level zone recycled water distribution network in an emergency situation. This water will be supplied at a lower pressure, so that while minimum residual head of 20m may not be met, residual heads of typically 10-20m will be possible to meet short term requirements.

From the end use modelling and system configuration detailed in **Appendix A**, it should be noted that recycled water demand in the town is considerably higher than the amount of recycled water which will be produced at the WRP each day. Therefore potable water from the ACTEW supply, pumped via the BWPS will be required to top up the recycled water supply. The network has been designed to receive the potable top-up water from the BWPS at the Recycled Water Pump Station located at the WRP. Level sensors in the Clear Water Tank at the WRP will control the BWPS operation. The determination of these control levels shall be developed during detailed design and refined during commissioning and operation.

Due to the staged nature of the construction of the recycled water system, and the provision of recycled water storage capacity ahead of its need, there may be water quality issues to consider. Provision of excess storage capacity ahead of its need may lead to water quality problems; in particular the maintenance of a satisfactory chlorine residual due to long recycled water residence times within these reservoirs. Accordingly, it is recommended that detail operating philosophy be developed during detail design that allows for only the partial utilisation of the storage volume available with the aim of minimising recycled water residence times within the constraints imposed by the predicted maximum water demands.

All inlet pipework into reservoirs will be controlled by Actuated Inlet Control Valves (AICV) via a Supervisory Control and Data Acquisition (SCADA) control system linked into the WRP control centre. This will ensure water is provided to the correct reservoir when the BWPS or recycled water pumping station operates.

Sodium hypochlorite dosing will be undertaken at the reservoirs to maintain the required chlorine residual. A self contained chemical dosing unit will be installed at the reservoir site. Individual dosing pumps and delivery lines will be installed for each reservoir.

4.2 Recycled Water Delivery Main

4.2.1 Purpose

In line with the Staging Strategy for Googong, Recycled Water will not be reticulated to the Googong township for unrestricted re-use for Stage 1, while the initial subdivisions are developed and the WRP treatment process stabilised. Instead, treated WRP effluent will be transferred via a short pipeline to Basin 1 of the Stormwater system, near to Googong Dam Road. This effluent will then be available for irrigation use. Any excess to the requirements for irrigation will be discharged to the environment. The outfall pipeline from the WRP to the Basin 1 location will be ID 225 to allow sufficient volume for chlorine contact, maintaining RW quality conditions.

The Recycled Water Delivery Main will not be constructed until Stage 2. At Stage 2, the Recycled Water Delivery Main will discharge directly to the Interim Recycled Water Reservoir at the Interim Reservoir Site. At Stage 3, the main will be extended to the Permanent Reservoir Site.

The normal operating regime will see the recycled water transferred to the recycled water reservoirs for storage and distribution to the town. Shortfalls in the supply of recycled water will be made up by the addition of potable water to top up the recycled water system.

However, if the recycled water reservoirs do not have sufficient capacity available to accept water from the WRP, recycled water flow will be directed via a side branch of the Recycled Water Delivery Main to Basin 4 of the stormwater system, for release to the environment.

4.2.2 Staging and Pipeline Route

As noted above, at Stage 1, a short outlet pipe will be constructed along the verge of Googong Dam Road to the Basin 1 outlet site, where it may be sued for irrigation or released to the environment. The transfer pipeline will operate as a chlorine contact tank, which must have a minimum internal diameter of 225mm. The Recycled Water Delivery Main will be constructed at Stage 2, normally discharging to the Interim Recycled Water reservoir, or alternatively excess flows discharging to the environment at Basin 4 of the stormwater system near Old Cooma Road. At Stage 3 the Recycled Water Delivery Main will be extended to the Permanent Reservoir Site as the interim site is decommissioned and demolished. The alternative environmental discharge point to Basin 4 will be retained for the permanent operation of the system. The Recycled Water Delivery Main staged construction and sizing is detailed in Table 4-1.

Table 4-1 Recycled Water Delivery Main Details

Pipeline	Stage Constructed	Sizing Criteria	Diameter Nominal (mm)	Length (m)	Design Flow rate (L/s)	Design Velocity (m/s)	Pipe Material and Class
Treated WRP Effluent to Basin 1	Stage 1	Stage 1	DN225	1,000	10	-	uPVC
Recycled Water Delivery Main from WRP to Interim Reservoir Site	Stage 2	Ultimate (Stage 4)	DN375	2,770	138	1.0	DICL PN35
Recycled Water delivery Main Extension to Permanent Reservoir Site	Stage 3	PDD Ultimate (Stage 4)	DN375	1,450	117	1.0	DICL PN35
		WRP PWWF (Stage 4)			138	1.1	

The WRP will not be permitted to discharge to the local waterway, but will discharge to the RW pipeline, either to the RW reservoir or the Basin4 discharge location close to Old Cooma Road. It will also be possible for the WRP to discharge to the Basin 1 of the stormwater system close to Googong Dam Road, using the permeate pumps via the UV disinfection system, bypassing the RW pump station and chlorine disinfection system, although this mode is only envisaged for Stage 1.

The pipeline route will be follow the road verge of Googong Dam Road and then Old Cooma Road from the WRP site to the Interim and Permanent Reservoir Sites, for the Stage 2 and Stage 3 construction stages respectively.

4.2.3 Mechanical

Mechanical pipeline fittings will be designed and installed in accordance with WSA 03 guidelines. As a minimum, scour pits with valves will be provided at all low points with scour pipelines to the nearest stormwater system or drainage pathway. Manual air release valves will be provided at all high points, reductions in upstream slope, increases in downstream slopes and at not greater than 800m intervals for all long horizontal, descending or ascending sections. The provision of automatic air valves and sectioning valves along the delivery main will be confirmed during detail design in consultation with the proposed pipeline operator.

Actuated Inlet Control Valves (AICVs) will be installed on the recycled water reservoir delivery main in below ground chambers at the inlet to the Recycled Water Reservoir, and at the Off-take to the Basin 4 release point. This will allow the recycled water pumping station to operate, and discharge water into either the reservoirs or Basin 4, as required.

4.3 Recycled Water Service Reservoir

4.3.1 Purpose

Recycled water service reservoir is required to store the recycled water produced at the WRP. The reservoirs are located in an area of elevated ground within the development so that water can be pumped to the reservoirs and then gravity fed to the development on demand, to meet the desired minimum requirements of 20m residual head.

Reservoir construction will be staged to meet the demands of the growth of Googong. For Stage 1, recycled water storage, for irrigation use, may be included at the Basin 1 outfall. This has not been included in this concept design. For Stage 2, the Interim Recycled Water Reservoir will be constructed at the interim site adjacent to Old Cooma Road. Stage 2 will also require the construction of booster pumps and elevated tanks to allow parts of Neighbourhood 1A to be meeting the minimum residual head requirements. At Stage 3, the interim site will be decommissioned and demolished, and a new reservoir constructed at the permanent site at the saddle feature further south along Old Cooma Road. At Stage 4, a larger Recycled Water reservoir will be constructed as the final storage, and the Stage 3 recycled reservoir converted to Potable supply.

4.3.2 Reservoir Sizing

The schematic in Figure 4-1 shows key reservoir operating sections.

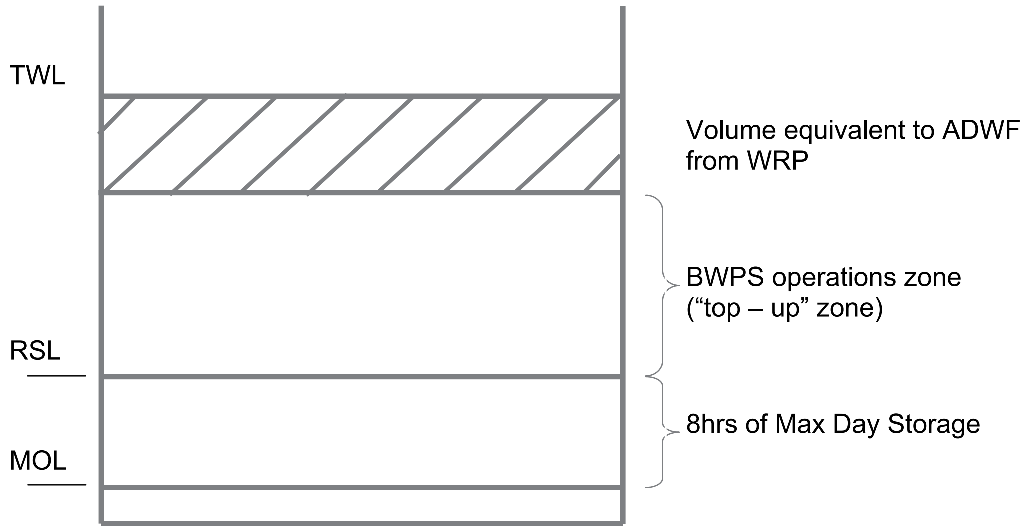


Figure 4-1 Reservoir Storage Zones

4.3.2.1 Interim Reservoir site

Based on the service reservoir sizes shown in Table 3-9, the specific dimensions and elevations of the recycled water reservoirs are summarised in Table 4-2. The elevated storage tanks shown here are sized for 30mins at Peak Hour Demand. In detailed design of the network distribution system for Stage 2, these tank sizes may be reduced to serve the High Level Zone only rather than all of Neighbourhood 1A (subdivision 3-6).

Table 4-2 Interim Recycled Water Reservoir Site – Reservoir Dimensions

Potable Water Reservoirs	Interim Recycled Water Reservoir	Interim Recycled Water High Level Tank
Storage Volume (ML)	2.1	0.150 (150 m3)
Tank Diameter (m)	14	5.8
Tank Invert Level (mAHD)	765	784
Minimum Operating Water Level (mAHD)	765.5	
Reserve Storage Level (mAHD)	769	
Reserve Storage Volume (ML)	0.54	
Top Water Level (mAHD)	775	789
Freeboard (m)	0.5	0.5
External Tank Height (m)	10.8	24.5
Top of Roof Level (mAHD)	775.8	789.5
Inlet Pipe Diameter (mm)	375	375
Outlet Pipe Diameter (mm)	200	200
Overflow Pipe Diameter (mm)	300	300

* Note final tank height will be determined in detail design.

Reservoir construction details are proposed to be the same as for the potable water reservoirs.

The potable water “top-up” inlet main shall be constructed up and over, rather than through the floor of the tank, and freely discharge above the top water level to maintain an air gap.

4.3.2.2 Permanent Reservoir site

The Permanent Reservoir Site located at the saddle feature adjacent to Old Cooma Road will require staged construction of five Potable and Recycled Water service reservoirs. The dimensions of the proposed service reservoirs are summarised in Table 4-3

Table 4-3 **Permanent Recycled Water Reservoir Site – Reservoir Dimensions**

Potable Water Reservoirs	Stage 3 Recycled Water Service Reservoir*	Permanent Recycled Water Service	High Level Zone Reservoir
Storage Volume (ML)	4.7	9.3	0.45 (450m3)
Tank Diameter (m)	24.3	34.4	10
Tank Invert Level (mAHD)	795	795	810
Minimum Operating Water Level (mAHD)	795.5	795.5	810.3
Reserve Storage Level (mAHD)	799	799	815.5
Reserve Storage Volume (ML)	1.63	3.26	0.43
Top Water Level (mAHD)	805	805	816.5
Freeboard (m)	0.5	0.5	0.5
External Tank Height (m)	10.8	10.8	7.3
Top of Roof Level (mAHD)	805.8	805.8	817.3
Inlet Pipe Diameter (mm)	225	375	225
Outlet Pipe Diameter (mm)	375	600	375
Overflow Pipe Diameter (mm)	450	750	300

** Stage 3 Recycled Water Service reservoir converted to Potable Service Reservoir at Stage 4.*

The High Level Zone Pump Station for Recycled Water will be required with similar details to the equivalent potable HLZ PS described in Section 3.5.5.2. The delivery main will be constructed of DICL, DN225 with pressure rating PN35. Two above ground dry mounted end suction horizontal pumps will be installed in duty/standby configuration. The pump will draw water using a single suction main connected to recycled water outlet main constructed to service the ultimate stage of development. Each pump will be equipped with a non-return valve for backflow prevention. The pumps are sized to deliver the maximum hour demand.

4.3.3 Secondary Water Conditioning/Chemical Dosing

Water quality will be maintained at the reservoir site by chemical dosing using sodium hypochlorite. A single chemical dosing unit (CDU) will be installed at the reservoir site to serve all of the reservoirs required for the ultimate development.

To limit bacterial regrowth within the recycled water network, chlorine residual will be maintained within the recycled water reservoirs to ensure a residual at the furthest end user. Recycled water will consume more chlorine than potable water and therefore the demand for this system will be higher than that of the potable water system.

Sodium hypochlorite will be dosed into the recycled water system at three locations.

- The first dosing point should be located in a suitable position for mixing prior to entry to the reservoirs, such as into the reservoir inlet pipework. This will be the primary dose and will be flow paced on the inlet flow to the reservoirs.
- The second dosing point will provide a much smaller "trim" dose to meet the set point chlorine residual required for discharge into the distribution network.

- A third sodium hypochlorite dosing point will be provided on the delivery main to the high level reservoir, downstream of the high level reservoir pumping station. This dosing point will provide a "trim" dose of Sodium Hypochlorite for supply to the high level reservoir. Dosing will be controlled by feedback from a chlorine residual analyser and flow meter on the outlet of the high level reservoir."

A single self-contained sodium hypochlorite dosing unit containing two 15m^3 chemical storage tank will be installed to serve all the reservoirs, including those located at the high level zone. The CDU will be in the order of 6.5m by 7.0m in area and 3.0m high, and will include chemical bund, emergency shower, MCC and the individual reservoir pump sets.

A separate pump and delivery line should be used for each dosing point. Two or three common standby pumps would be sufficient for the system. These details should be confirmed at detailed design stage.

5. Sewerage System

5.1 Sewerage System Operational Philosophy

5.1.1 Initial operating phase

In Stage 1, SPS1 will pump directly to the inlet works at the WRP. It is anticipated that SPS1 will not operate during the initial stages of development until sewage from an equivalent population of around 150 EP is being generated (50 houses). This is because the WRP will require sufficient biological loading to operate effectively. Stage 1 of the WRP has been configured as a 1000 EP bioreactor, with a turndown capability to 150EP to commence wastewater treatment.

During this initial period it is expected that sewage into SPS1 will be regularly tankered away for treatment and disposal to an operational Sewage Treatment Plant (STP), located in the area. This STP is yet to be determined. Nearby STPs exist at Queanbeyan and Fyshwick. It is anticipated that the operators will be responsible for developing a detailed methodology for the initial phase.

The frequency of tankering will depend on the connected population at the time and is expected to build up to a maximum of 3 trips per day immediately prior to pumping commencing for SPS1 at Stage 1. This is based on a population of 150 people, producing 180 L/person/day, and a standard tanker volume of $10m^3$.

The wet well volume at SPS1 is approximately 12.5 m^3 which, with regular tankering, should be adequate to contain the inflow during this initial period. Approximately $100m^3$ of storage will be provided at SPS1 initially, giving a safety buffer for tanker pump outs.

Once the WRP can begin receiving flows from SPS1, the pumps will be made operational.

SPS2 will be constructed in Stage 2, well after the WRP is operating, as development moves into this catchment.

5.1.2 Final Operating Phase

During the ultimate Stage 4 of development, SPS3 and SPS4 will constructed to receive flows from NH3, NH4 and NH5 and pump into the gravity networks upstream of SPS1 and SPS2. SPS1 and SPS2 will therefore receive significant additional flows, in the ultimate stage of development. The specific design of SPS3 and SPS4 are not considered in this report as they will not be required until the ultimate stages of development.

In order to operate effectively over the two stages of development, SPS1 and SPS2 are proposed to consist of three pumps in a duty / duty / standby arrangement, at ultimate development. Initially only duty / standby will be installed to cater for Stage 1 flows. The second duty pump will be installed when flows into the SPSs approach the capacity of the first pump. It is anticipated, that the second pump will be installed after approximately 6 years.

The pumps in each SPS will be controlled by wet well-level sensors, which switch the pumps on when the water level reaches the design cut-in level, and switches the pumps off, when the water level drops to the cut-out level.

Should inflow to the SPS exceed the pump discharge rate at any time, the emergency storage area will start to fill. If the emergency storage volume capacity is exceeded, flow will discharge in a controlled manner to a watercourse, via an emergency overflow pipe. At present, it is assumed that the overflow from SPS1 will discharge into the stormwater system at Basin 1 on Googong Creek, and SPS2 will overflow directly into Montgomery Creek. This is subject to obtaining the relevant environmental approvals.

A Supervisory Control and Data Acquisition (SCADA) system will be provided at each SPS and is proposed to be linked into the WRP control centre. This will allow operators at the WRP to control the pumping rate from the SPSs. This is subject to confirmation of operational and management arrangements at the WRP and SPSs.

5.2 Basis of Design

5.2.1 Equivalent Population

The Equivalent Population (EP) for each neighbourhood within the development was generated using the agreed development yield matrix and the method given in the WSA 02 Sewerage Code. Table 5-1provides the EP and catchment area used to calculate the design sewage flows for the ultimate development.

Table 5-1 Equivalent Population and Sewerage Catchment Area

Development Precinct	Equivalent Population#	Sewered Catchment Area (ha)
NH1A	4,076	101.8
NH1B	1,395	56.4
NH2	5,323	109
NH3	2,529	66.2
NH4	2,611	73.5
NH5	2,196	102.5
Hamlet East	675	24.4
Hamlet West	44	23.6
Total	**18,850**	**557.4**

#Including School Students (estimate 3000, 600 EP equivalent) and Commercial (1078 EP)

5.2.2 Design Sewage Flow

Table 5-2 shows the design flows entering the WRP. “Stage 2-3” includes all flows from NH1A and NH1B.

Table 5-2 Design Flows into Water Recycling Plant

Water Recycling Plant Design Flows	Equivalent Population (EP)	Average Dry Weather Flow (L/s)	Peak Dry Weather Flow (L/s)	Peak Wet Weather Flow (L/s)	Ratio of PWWF:ADWF
Stage 1	1000	2.1	7	10	5
Stage 2-3	5,473	11.4 (0.98 ML/d)	32.4	47.5	4.17
Ultimate Development	18,850	39.3 (3.40 ML/d)	84.2	127.4	3.24

5.2.3 Sewerage Catchments

Sewerage catchments do not follow the neighbourhood boundaries, but are demarked by topographical features such as valleys and ridges, and the fall of the land. This concept design is limited to the two main pumping stations SPS1 and SPS2. However, in order to determine the flows to SPS1 and SPS2, it is necessary to divide the entire development into sewerage catchments and distribute flows accordingly. Hence, consideration has been given to the location of the SPSs required to serve ultimate development.

A minimum of four SPSs are required to convey sewage from the development to the WRP. SPS1, SPS2 will be built in Stages 1 and 2, with SPS3 and SPS4 to follow during the ultimate Stage 4. The locations of the SPSs are shown in **Drawing G-002**.

SPS1 will receive flows from approximately 57ha of NH1A and will be pumped directly into the inlet works at the WRP. During the ultimate stage of development, NH2 (Town Centre), Hamlet East and Hamlet West, and a large portion of NH3 are assumed to drain via gravity to SPS1 and pump to the WRP. Flows from SPS3 are also assumed to discharge into the upper part of the SPS1 gravity sewer catchment and from there, drain to SPS1 and pump to the WRP.

As its catchment is developed, SPS2 will receive all flows from NH1B and approximately 52ha of NH1A and discharge directly to the inlet works at the WRP. During the ultimate stage, flows from SPS4 are assumed to discharge into the SPS2 gravity sewer catchment.

SPS3 is assumed to service sewage flows from the small 17.1 ha area adjoining the south-western boundary of NH3. SPS3 will discharge into the SPS1 sewerage catchment in the ultimate stage of development, and the precise discharge point will be determined during detailed design.

SPS4 is assumed to service all sewage flows from NH4 and NH5 and will discharge in the SPS2 sewerage catchment.

Some areas within the development cannot be served by the proposed SPSs. Options for these areas include:

- The use of pressure sewerage systems and/or;
- The use of additional SPSs; and

These options will need to be investigated during detailed design and as development stages are confirmed.

5.2.4 Sewage Pumping Station Design and Performance Criteria

Design and construction of the SPSs, will be in accordance with the WSA 04, 2005 Sewage Pumping Station code.

The pumps should be sized to discharge PWWF, in accordance with the WSA 04. However, during Stage 1, the WRP will only have one mini-bioreactor operating, sized for 1000 EP, with capacity to treat approximately 7-10 L/s. Sustained flows from SPS1 could be in excess of the WRP capacity. To prevent excessive flows entering the WRP during stage 1, the initial pump sizes will be de-rated to this smaller limiting flow. This could also be achieved by selecting smaller pumps for the early years. Any excess incoming flow to the SPS will therefore be spilled to the emergency storage tank. Alternatively, higher capacity pumps with VSD control could be used to limit flows then provide a full flow flushing cycle once a day.

The emergency storage volume is calculated as the higher of 4 hours at PDWF or 8 hours at ADWF. In this case, 4 hours at PDWF is the defining volume. Flows in excess of the emergency storage capacity will discharge to the environment at locations to be confirmed during detailed design. The emergency storage volume will be constructed in a staged approach over different upgrade stages. The WSA-04 code requires no dry weather overflows at PDWF over the maintenance/repair time.

A maximum of eight pump starts per hour, pumping at the design flows given below, has been used to determine the wet well operating volume.

Table 5-3 shows the flows used for the design of the SPSs.

Table 5-3 Sewage Pumping Station Design Flows for Stage 1 and Ultimate

	Equivalent Population	Average Dry Weather Flow (L/s)	Peak Dry Weather Flow (L/s)	Peak Wet Weather Flow (L/s)	Ratio of PWWF to ADWF	Pump Design Flow Rate (L/s)
Stage 1						
SPS1	1000	2.1	6.0	10	5	1 x 10
SPS2	0					
Stages 2-3						
SPS1	1987	4.1	11.9	17.0	4.15	1 x 33.5
SPS2	3486	7.3	20.5	30.5	4.18	1x 31.5
Ultimate						
SPS1	9,976	20.8	44.9	66.6	3.20	2 x 33.5
SPS2	8,874	18.5	39.3	60.8	3.29	2 x 31.5

5.2.5 Rising Main Design and Performance Criteria

The sewage rising mains will be designed and constructed in accordance with WSA 04, 2005 Sewage Pumping Station Code. A Colebrook White friction headloss coefficient of ks=0.6mm, has been used in the rising main analysis. Rising mains will be designed to ensure minimum sliming and self cleansing velocities of 0.9m/s are achieved, in accordance with WSA 04. A default maximum permitted flow velocity of 3.5m/s within the main is recommended as per WSA 04. However, it is recognised that in the early years rising main 1 will have a lower velocity than the WSA specified minimum, as a result of de-rating the first set of pumps to limit inflows to the WRP. Variable Speed Drives could be used to limit flows in normal operation yet provide a daily flushing flow at full capacity.

5.3 Sewage Pumping Stations

5.3.1 General

The topography of the development is such that SPSs are required to transport sewage to the WRP for treatment. Sewage will flow via a gravity sewerage network to the various SPSs serving the development and will be pumped to the WRP. The SPS1 and SPS2 wet well structures will be constructed to ultimate size in the first instance; however the emergency storage volume may be progressively staged over time, subject to the requirements of DECCW and other operational requirements.

The location and preliminary layout of SPS1 and SPS2 are shown on **Drawing G-001**. SPS1 compound is approximately 25m by 32m and is located in NH1A adjacent to Googong Dam Road and Stormwater Basin 1. SPS2 compound is 25m by 30m and located near Montgomery Creek, in the east of NH1B.

Each SPS compound will provide sufficient room to accommodate the SPS wet well and emergency storage, valve chamber, ventilation stack, 5m wide access track, electrical kiosk, inlet manhole, and potential future equipment such as a chemical dosing unit or a passive carbon odour unit. The site may also include a perimeter strip to allow for tree planting.

Access to SPS1 will be from Googong Dam Road, while SPS2 access will be from a local road, to be determined once final road layouts have been confirmed.

A security assessment should be undertaken in consultation with the SPS operators, during detailed design, to determine if fencing is required.

Ground conditions at the SPS site are expected to be consistent with those across the entire development, being underlain by low strength highly weathered dacite at willow depths of 0.7m and 0.9m respectively for SPS1 and SPS2. The dacite overlays rock at depths of approximately 1.3m and 1.8m. Refer to the Geotechnical Report produced by Douglas Partners in August 2009 for further details. Construction of the SPSs will therefore require a significant amount of excavation in rock material. Geotechnical conditions should be confirmed during detailed design.

5.3.2 Wet Well and Emergency Storage

The pumping station wet well will be a reinforced concrete structure and will be constructed to full size during stage 1. The emergency storage may be incorporated into the wet well structure itself or constructed as additional storage tanks alongside the wet well. The SPS wet well structures may be a cast in-situ or pre-cast concrete construction. The below-ground emergency storage tanks may also be concrete or alternatively FRP package units commonly available. Wet well requirements are summarised in Table 5-4.

The concrete roof of the pumping station will protrude 200mm above ground level across the length and breadth of the storage to reduce the potential for surface water inundating the structure. Access covers will be located above the wet well area to enable pump inspection, removal and maintenance to be carried out.

Below ground, normal operating flows will be contained in a 3.5m diameter wet well. Emergency storage will be provided within the combined structure, located above the Alarm Top Water Level. The emergency storage section of the combined structure will naturally fall towards the pumps to promote drainage and prevent the deposition of solids.

SPS1 and SPS2 will be provided with 655m^3 and 560m^3 of emergency storage respectively, corresponding to 4 hours at PDWF.

When the emergency storage area reaches capacity, the excess water will back up the SPS inlet pipe into the inlet maintenance manhole. An outlet pipe from the inlet maintenance manhole then discharges to an intermediate emergency relief structure before discharging via an outlet to the watercourse or stormwater basin.

5.3.3 Mechanical

Both SPSs are proposed to have submersible pumps installed in duty/duty/standby configuration at ultimate development. During Stage 1, one duty and one standby pump with a much smaller capacity will be installed. The second duty pump will be installed in Stage 4 as the flow into the SPS increases.

A valve chamber will adjoin each SPS. The chamber will include an emergency bypass pumping connection, reflux valves, stop valves and two ferric dosing points (if required).

Table 5-4 Key Sewage Pumping Station Details

Item	Quantity / Units		Comments
	SPS 1	SPS 2	
Average Dry Weather Flow (ADWF)	20.8 L/s	18.5 L/s	Ultimate Development
Peak Dry Weather Flow (PDWF)	44.9 L/s	39.3 L/s	Ultimate Development
Peak Wet Weather Flow (PWWF)	66.6 L/s	60.8 L/s	Ultimate Development. Design for 2 x 33.3 L/s and 2 x 30.4 L/s respectively
Duty Point	67L/s@25.4m	61L/s @48.2	
Operational Volume	7.5 m3	6.8 m3	Based on 8 pump starts/hr
Emergency Storage Volume	655 m3	560m3	4 hours PDWF
Wet Well Size	3.5m dia	3.5m dia	Refer Dwg A1081402 –SK101 & A1081402 –SK201
Ground level	RL 725.00	RL 713.0	Subject to confirmation by survey
1:100 year flood level	-	-	Not determined – confirm in detailed design
1:20 year flood level	-	-	Not determined – confirm in detailed design

Wet Well Levels			
Roof Level	RL 725.20	RL 713.2 mAHD	GL +200mm
Overflow Level	RL 724.50	RL 712.5	Nominally 0.5m below GL at station
Alarm level (ATWL)	RL 721.70	RL 709.00	
Emergency Cut-in	RL 721.55	RL 708.85	Emergency Cut-in
IL Incoming Sewer Inlet Maintenance Hole	RL 723.00	RL 711.00	Level into inlet manhole. Backdrop assumed.
Duty Cut-in 2	RL 721.45	RL 708.75	
Duty Cut-in 1	RL 721.35	RL 708.65	
Duty Cut-out 2	RL 721.05	RL 708.39	
Duty Cut-out 1	RL 720.95	RL 708.29	
Emergency Cut-out	RL 720.80	RL 708.14	
Suction Safety Cut-out	RL 720.40	RL 707.70	
Mean Water Level	RL 721.15	RL 708.47	
Floor Level	RL 720.00	RL 707.30	
Pump Characteristics			
Configuration	Submersibles	Submersibles	
No. Of Pumps	3	3	Duty /duty/ standby rotated

5.3.4 Odour

Sewage Pumping Stations receiving flows from local gravity catchments do not normally require mechanical odour control. Therefore SPS1 and SPS2 will be normally ventilated to the atmosphere with a standard 12m high DN150mm ventilation stack, connected to the wet well.

However, if odour complaints become a problem in future years, provision will be made to connect at a future odour control system. If required, this odour control system will be an activated carbon odour scrubber on the SPS outlet vent. The odour unit will be housed in a ventilated kiosk, mounted on a movable skid, approximately 3 m by 4 m in size. Extraction fans will be used to draw air from the wet well through the activated carbon scrubber. The kiosk will be approximately 1.5m (w) x 2.0m (d) x 1.5m (h), and will contain the carbon scrubber and fans.

Ventilation stacks will also be required throughout the sewerage network on branch and trunk sewers as required in WSA 02 Sewerage Code. They will typically be at a minimum spacing of every 400m, and be constructed of 150mm diameter, 12m in height.

As no septicity analysis has been performed on the network, space has been allowed for ferric sulphate dosing at the SPS1 and SPS2. Consideration should be given to using mixer pumps during the initial operating phase at SPS1 to reduce the potential for odours during this period.

5.3.5 Utility requirements

A telephone line will be connected to the MCC on site as part of the SCADA control and telemetry requirements.

Potable water supply will be provided to site for the safety shower wash down facility in the chemical dosing unit and for general usage such as drinking and hand washing. The potable water supply to the site shall be provided with a Reduced Pressure Zone Device in accordance with AS3500.

Recycled water will be supplied to site, to be used for washing down of the wet wells, as part of the maintenance programme.

5.4 Sewage Rising Mains

Sewage rising mains are required to transport sewage from the pumping stations to the WRP. The rising main routes from SPS1 and SPS2 to the WRP are outlined in **Drawing G-001**. The routes are based on preliminary road layouts and will need to be revised during detailed design once roads are finalised.

Construction of the rising mains will be timed to coincide with development of the relevant catchments, commencing with RM1 in Stage 1, and RM2 probably constructed in Stage 2, depending on the exact catchment boundaries adopted. However, the pumps will be staged which will mean different flow velocities in the rising mains for each stage. It is expected that minimum self cleansing flow velocities will not be achieved during Stage 1, because of the need to reduce peak flow into the first mini-bioreactor. The option of instead installing smaller mains, and duplicating in future years should be considered at detailed design stage.

Rising mains will generally be located within the road verge area. The depth of cover to the sewage rising mains and trench, as well as bedding details will be in accordance with WSA 04 Code or as agreed with Council. Manual air valves and scour valve arrangements will be provided in accordance with WSA 04 guidelines.

The ground conditions within the development generally consist of weathered dacite rock overlaying rock. Rock is commonly encountered at depths less than 1.0m, with excavation refusal often occurring around 1.0 to 1.5m. Douglas Partners have produced a geotechnical report which details specific ground conditions across the site.

Although DICL pipes are indicated in the table below, this does not preclude the use of alternative pipes by future designers or contractors.

Table 5-5 below, gives details of the rising mains required for SPS1 and SPS2 and indicates staging of the works.

Table 5-5 Rising Main Details for SPS1 and SPS2

	Stage	Diameter Nominal (mm)	Length (m)	Design Pumping Rate (L/s)	Velocity (m/s)	Pipe Material	Pipe Class
SPS1	Stage 1	DN225	800	10	0.25*	DICL	PN35
	Stage 2/3			33.5	0.75		
	Ultimate			67	1.49		
SPS2	Stage 1	DN200	860	30.5	0.86	DICL	PN35
	Ultimate	DN200	860	55	1.56		

' Use of Variable Speed Drive limiting normal pumping flows to 10 L/s with daily flushing cycle at full flow will be preferable for improved slime control.*

Table 5-6 Proposed Delivery Stages

Asset Description		Description of Stage
SPS1	Stage 1 (0-1000EP)	• Wet Well civil and structural works sized for ultimate duty • 100 KL Emergency Storage • 2 x 10L/s pumping units installed (duty/standby) • DN225 SPS1 Rising Main
	Stage 2/3 (1000-9400EP)	• 2 x 34 L/s pumping units installed (duty/standby) • Additional Emergency Storage
	Stage 4 (9400-18,850 EP)	• 1 additional pumping unit installed (duty/duty/standby) • Additional Emergency Storage
SPS2	Stage 1 (0-1000EP)	• Nil
	Stage 2 (1000-3600 EP)	• All civil and structural works sized for ultimate duty • 100 KL Emergency Storage • 2 x 30 L/s pumping units installed (duty/standby) • DN200 SPS2 Rising Main
	Ultimate	• 1 additional pumping unit installed (duty/duty/standby) • Additional Emergency Storage
SPS3	Ultimate	• Detailed Design
SPS4	Ultimate	• Detailed Design

6. Water Recycling Plant

6.1 Overview

The water recycling plant (WRP) described in this concept design will produce recycled water (RW) for non-potable use including; toilet flushing water, cold water for washing machines and irrigation. The production of recycled water is a fundamental element of the Googong IWC, allowing for significant savings in potable water use throughout the development.

The WRP will treat sewage, generated from the Googong development to a quality suitable for recycling. The plant consists of primary screens and grit removal, a Membrane Bioreactor (MBR) for nutrient and solids removal and a tertiary stage combining UV treatment and chlorination for disinfection. A process schematic is given in Figure 6-1.

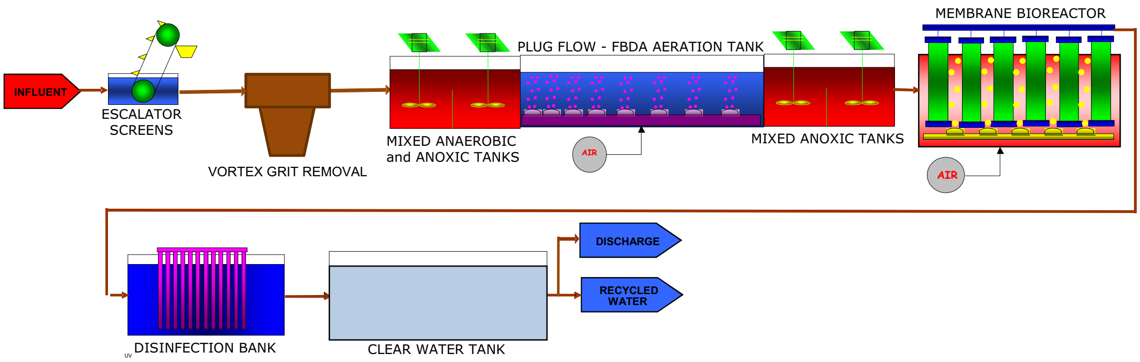


Figure 6-1 Overview of WRP Process

Flow from the network will enter the inlet works via separate transfer pipelines. The influent will pass through the inlet works, including screening and grit removal. Two stage screening will be provided, utilising 6mm screens followed by 1mm screens, which will provide protection for the membranes. Grit will be removed downstream of the screens via a vortex trap. The inlet works will be elevated above/adjacent to one of the bioreactors.

Screenings will be collected at ground level, washed and dewatered in a screw wash press, before being transferred to sealed bins. Grit will be washed and dewatered in a grit classifier and will be combined with the screenings in the bins.

Secondary-biological treatment will be provided by a number of MBRs. The bio-reactors will remove organic material, nutrients (nitrogen and phosphorous) and suspended solids through membrane filtration. Biological Phosphorous Removal (BPR) has been included within the concept design to limit the overall requirement for chemical addition. BPR is used in conjunction with the addition of Ferric Sulphate in order to remove phosphorous to the licence condition.

The membrane tanks, fundamental to the operation of the MBRs, contain submerged ultra-filtration membranes. The effluent is drawn through the membranes whilst the mixed liquor from the bioreactor is contained within the tank. The pore size of the membranes is 0.45μm providing a physical barrier for solids removal.

Recycled water requires a double barrier approach to pathogen removal, to protect the end user. The exact requirement is determined under the Australian Recycled Water Guidelines by a risk assessment which will be completed in parallel with ongoing design activities. Experience from other plants with similar recycled water use shows that two forms of disinfection provide an adequate barrier. The disinfection included for the WRP is ultraviolet (UV) disinfection followed by chlorination. Residual chlorine will be maintained, to prevent regrowth of bacteria within the recycled water reservoirs and pipe work.

Several chemicals will be stored on-site for different functions as part of the WRP. These include ferric sulphate, acetic acid, magnesium hydroxide, citric acid, sodium hypochlorite and polymer. The applications of these chemicals are outlined in Table 6-1.

Table 6-1 WRP Chemicals and their uses

Chemical Name	Purpose
Ferric Sulphate	Odour control and chemical phosphorus removal[1]
Acetic Acid	Supplementary carbon addition for de-nitrification
Magnesium Hydroxide	Alkalinity addition
Citric Acid	Membrane cleaning
Sodium Hypochlorite	Disinfection and membrane cleaning
Polymer	Sludge thickening and dewatering

[1]Phosphorous removal is achieved using a combination of biological process and chemical addition, discussed in Section 6.3.3.

It is noted that a number of chemicals may be used to achieve the above purposes; however chemicals have been selected for the basis of the concept design and should be revisited in detailed design based on operator requirements. The chemical use on site has been assessed in “Googong Water Cycle Project Risk Assessment SEPP33 Assessment Report” (Sherpa Consulting, 2009).

Recycled water (RW) will be pumped to the RW reservoir using the RW pumping station. Water in excess of the RW requirements will be discharged from the RW Main through the storm water system (basin 4) for discharge to the environment.

When periods of high demand for recycled water coincide with an excess of stormwater and recycled water within the stormwater system, there may be an opportunity to transfer combined stormwater/recycled water to the WRP for treatment and subsequent transfer to the RW system. As a minimum this will involve provision of a pipe and actuated valve from Basin1 into SPS1. This will need to be considered further within the project to determine:

- Any additional process requirements at the WRP
- Control arrangements
- Specific additional infrastructure requirements at Basin 1, SPS1 and the WRP

The WRP will be equipped with odour control facilities to minimise odour nuisance. Odour generation will be contained by provision of removable sealed covers on all process units. Foul air will be extracted and conveyed via sealed aboveground ducting, to a central odour control facility (OCF). The OCF will consist of biological trickling filters, followed by activated carbon filters. Treated air will be discharged to the atmosphere via a stack to aid dispersion. The requirements for the OCF have been determined by dispersion modelling.

Solids produced in the bioreactors will be treated in an aerobic digester and then dewatered using centrifuges. It will be stored on-site within a sealed hopper for collection and off-site disposal.

The adopted Design Criteria for the WRP, in terms of influent flows and loads, effluent criteria, are shown in Section 2.5.

6.2 Operational philosophy

A concept-level Process Flow Diagram (PFD) has been prepared in **Drawing P-101**, and indicative Piping & Instrumentation Diagrams (P&IDs) are shown in **Drawings P-201** to **P-238**. The PFD describes how all of the process units in the WRP interact, and reflects the staging of the WRP. The P&IDs show the Ultimate (Stage 4) Plant design although some design development has not been included. Further development of the P&IDs will be required in detail design to confirm final process configurations, standby units and instrumentation requirements.

The WRP will be controlled automatically via a centralised control system with a SCADA interface. The plant will operate automatically, treating flows as they arrive at the WRP.

Flow will gravitate through the inlet works to the bioreactors. The screens provided will be of sufficient capacity to operate as Duty/Standby although they will routinely operate Duty/Duty, whilst both units are available. The control will be based on the type of screen, however; for concept design purposes the control will be based on up-stream level within the channel (as measured by a level transmitter).

The flow will gravitate through the various sections of the bioreactors. Aeration will be controlled using dissolved oxygen probes in the aeration tank. REDOX probes will be provided in the anoxic zones for information and process optimisation only.

The bioreactor includes membrane ultra-filtration for solids removal. Pumps will feed the membrane tank, based on flow into the WRP, generating a raised water level in the membrane tank, subsequently driving the mixed liquor recycle stream. Recycle pumps will also be provided to transfer flows from the de-aeration zone to the primary anoxic zone (de-nitrification) and from the primary anoxic zone to the anaerobic zone (biological phosphate removal). Both sets of recycle pumps will also be controlled on WRP influent flow.

Permeate pumps will draw flow through the membranes and through the UV disinfection. Flow to be recycled, will then be further disinfected by the addition of chlorine, before being pumped to the RW reservoir. Chlorination will be operated to maintain a residual chlorine concentration in the RW system.

RW will be produced to match the diurnal profile of flow that comes into the plant, although in the first few years of RW production it will be more intermittent to allow for pump flows both from the SPS and the WRP. Excess flow will be discharged to the environment from the recycled water main through Basin 4.

Solids produced in the bioreactor, will be pumped using waste activated sludge (WAS) pumps to rotary drum thickeners for thickening. Sludge will be wasted, based on achieving sludge retention time in the bioreactors, and will be set to a time per day. The thickened sludge will then be treated in an aerobic digester, before being pumped to centrifuges within the building.

The plant will include mitigation of noise and odour by containment. Mechanical equipment that will cause noise will be housed in buildings and/or equipped with acoustic enclosures. Process units, which may emit odour, will be covered and foul air extracted, will be treated in an odour control unit. A noise assessment is being undertaken by Heggies with input from MWH on the expected noise levels from equipment on-site. Reference to applicable noise standards will be given in the Heggies report.

Truck movement will be required throughout the operation of the WRP in order to deliver chemicals and to remove solid waste. The anticipated frequency of truck movements are outlined in Table 6-2. The frequencies may be dependent on the load on the WRP through the life of the development; however, this is offset by increasing chemical storage as the load on the plant increases. Typical quantities of solid waste, which provides an estimate for the Googong WRP, are outlined in Table 6-3.

Table 6-2 Chemical Delivery Frequency

Product	Delivery Frequency (at Ultimate capacity)
Acetic Acid	28 days
Citric Acid	28 days
Polymer	28 days
Ferric Sulphate	28 days
Magnesium Hydroxide	28 days
Screenings / Grit bins	2 per week
Dewatered Sludge Cake	2 per week

The above estimates correspond to ultimate design and are approximate, based on predicted flows and loads into the plant.

Table 6-3 Typical Quantities of Waste produced by the WRP

Waste	Volume (at Ultimate capacity)
Screenings	2.6m^3/d
Grit	2m^3/d
Sludge (based on a five day week, approx 8m3/d)	41m^3/week

6.3 Primary Treatment

The purpose of the inlet works is to remove gross solids from the incoming sewage. The inlet works will comprise a reinforced concrete structure elevated above ground level and containing the following equipment:

- 6mm screens;
- 1mm screens;
- Grit removal (Stage 2b onwards); and
- Screenings and grit handling and washing (Stage 2b onwards).

For Stage 1, only screens will be provided due to the very small grit loads and influent flows. For Stage 3b onwards, an elevated concrete inlet works structure will be built next to one of the bioreactors to provide an influent receivals channel, vortex grit tank and 4-way flow splitter to the bioreactors.

Solids collection and removal activities from the inlet works will involve approximately two truck movements each week at ultimate WRP operation (based on use of 5m^3 collection bins).

6.3.1 Screens

Influent screening is required to protect downstream equipment from mechanical damage and aesthetic issues in biosolids, which will be recycled. As membrane technology is used in the WRP, effective screening is crucial, as large suspended solids may damage the membranes.

Material removed in the fine screening process will be washed to remove organics, in order to reduce odour nuisance, before being transported to landfill.

The flow into the WRP from SPS 1 and SPS 2 will be discharged into a covered, elevated inlet chamber and will gravitate through the inlet works. Ferric sulphate will be dosed into the inlet chamber for reduction of gaseous hydrogen sulphide to assist in the control of odour.

The 6mm screens will be provided as the first stage and 1mm aperture “punched hole” fine screens, will be provided downstream. For this concept design, the screens will be self contained “drum screen” units sitting on a deck above one of the bioreactor tanks. Pipes will deliver flow from the inlet works receivals channel to the screens, between screens and back from the screens to the grit channel. An alternative concept to consider at detailed design is to form concrete channels and place screens in the channel as either “step screen” or “band screen” configuration.

All inlet works channels and screen units will be covered for odour control and all equipment housed in shrouds to limit the volume of odorous air, which will be extracted and transferred to the odour control facility.

Screenings Handling and Washing

The screenings will be discharged from the screens into shaftless-screw conveyors that will transport the screenings to the screenings washing equipment. Screenings will be washed in a screw washpress. Screenings are stored in the vessel, and agitated and drained before the screw compactor dewaters the screening and transports the washed and dewatered screenings to a storage bin.

Table 6-4 summarises the key parameters of the screening system.

Table 6-4 Screening System Summary Parameter

Parameter	Units	Specification
Screen Flow Rate	L/s	140
No of screens		2 (Duty/Standby) 6mm
		2 (Duty/Standby) 1mm aperture (“punched hole” type)
Conveyors		1No shaftless screw
Screenings washing unit		1
Collection Bins		2
Corrosion protection	High levels of H_2S are likely to occur	
Material type to combat corrosive environment	Stainless Steel grade 316L	
Odour control	All plant covered to retain odorous air Forced abstraction of air from equipment and from covers to limit odour release	

6.3.2 Grit Removal

Vortex-type grit removal facilities will be installed downstream of the screens to remove grit and sand from the sewage and reduce wear on downstream equipment. Grit removal will be provided in the form of twin vortex grit removal chambers. A schematic of this type of grit removal can be seen in the Figure 6-2. As with the screens, the grit removal facility will be covered with close fitting covers and housed on the upper level of the Inlet Works building. Grit will be washed and transferred to the collection bins.

In Stage 1, in the early years of the WRP operation, low grit volumes at low incoming sewage allow for intermittent removal of grit from the bioreactor. Any grit entering the plant would settle out at the first anaerobic/anoxic tanks, and would be intermittently pumped from the system. From Stage 2 onwards, a vortex grit tank would be provided in the concrete elevated inlet works.

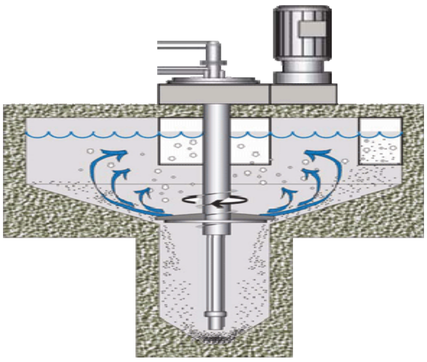


Figure 6-2 Typical Vortex Grit Removal System

Grit can be removed either by pumps or air blowers and is transferred to a classifier before being discharged into the screenings bins. The grit removal will be housed downstream of the screens and is connected into the formed concrete channel structure. The unit will be covered to allow air to be extracted and conveyed to the odour control facility. A summary of the grit removal system is presented as Table 6-5.

Table 6-5 Grit Removal System Summary Parameter

Parameter	Specification, for Ultimate Capacity
Grit flow-rate	65 L/s
Number of units	2
Type of unit	Vortex Type
Grit removal	95% removal of particles greater than 150 microns
Grit washing facility	Classifier

6.3.3 Civil

The Inlet Works will be located adjacent to the bioreactors, screens located on a platform above one of the bioreactors. Downstream of the screens, constructed at Stage 2b, there will be a single vortex grit chamber constructed from concrete. Screenings, grit handling and washing will be situated at ground level.

Access will be provided to all equipment. It is proposed that GRP, GMS or aluminium walkways be provided for this function. Covers will be provided on all channels for odour control.

6.3.4 Utility Requirements

An allowance of 2L/s of service water have been estimated for use by the screenings unit and 0.5L/s has been estimated for the grit classifier. This needs to be confirmed with the selected process equipment as part of detailed design.

6.4 Secondary Treatment

6.4.1 Treatment Process Configuration

Secondary treatment processes involve the use of biological methods to remove organic materials (BOD and COD) and nutrients such as nitrogen and phosphorus as well as total suspended solids. The Googong WRP will operate using a membrane bioreactor (MBR) design. A number of MBRs will be utilised for the Googong WRP which have been designed with additional capabilities to facilitate both biological nitrogen removal (de-nitrification) and biological phosphorous removal as well as the removal of dissolved solids and chemical and biological oxygen demand (COD and BOD).

The secondary treatment option was designed using Biowin process simulation software. The concept design consists of an anaerobic zone, a primary anoxic zone, an aeration zone, a de-aeration zone and a secondary anoxic zone followed by a membrane tank. This design utilises 4 identical trains which will be built as required by the overall development staging, discussed further in Section 6.14. Figure 6-3 shows the process schematic of one of the four bioreactor trains which will ultimately be required for biological treatment. Detailed analysis of the Biowin simulation outputs are given in **Appendix E**.

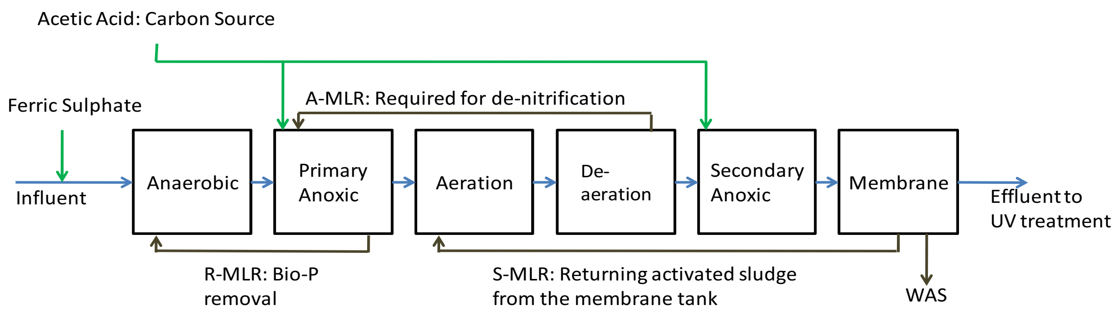


Figure 6-3 Biological treatment process schematic

Table 6-6 summarises the overall design parameters associated with the bioreactor. The process unit sizing included in this section relates to the Ultimate WRP size. With multiple bioreactors, the WRP will in practice be constructed in several stages, with a reduced size bioreactor in Stages 1a and 1b. Refer Section 6.14.2 for details.

Table 6-6 Design Parameters for Biological Treatment

Parameter	Units	Stage 1 (a and b)	Stage 2a	Stage 2b	Stage 3	Stage 4
Approximate Equivalent Population (EP)	No.	1,000	2,350	4,700	9,400	18,850
Bioreactor	No.	1	1	1	2	4
Membrane tanks	No.	2	2	2	2	4
ADWF	kL/d	180	423	850	1700	3400
PWWF factor	No.	5 x ADWF	5 x ADWF	3.5 x ADWF	3.5 x ADWF	3.5 x ADWF
Maximum Sewage Temperature	°C	24				
Minimum Sewage Temperature	°C	15				
Solids Retention Time (SRT) at capacity	Days	20				
MLSS in Bioreactor	mg/L	8,000				
Alpha factor	-	0.5				
Total Anoxic mass fraction	-	0.32				
Anaerobic mass fraction		0.15				

The effluent quality predicted by the process simulator used is given in Table 6-7. Biowin was used as a design tool to specify the plant effluent quality, internal recirculation flows and the overall plant equipment volumes. Biowin reported the effluent results based on the process conditions and standard input flows and loads. The detailed methodology used in the Biowin model to produce the results below is given in Appendix E.

Table 6-7 Bioreactor Effluent Quality

Element Name	Units	Biowin Model Prediction
Permeate Flow	m^3/d	890
Total COD	mg/L	47.32
Total BOD	mg/L	0.85
Total Kjeldahl Nitrogen (TKN)	mgN/L	3.16
Ammonia as N (NH_3-N)	mg/L	0.33
Nitrate (N)	mgN/L	0.99
Total N	mg/L	4.20
Total P	mgP/L	0.21
Total Suspended Solids (TSS)	mgTSS/L	0.01

The predicted total nitrogen does not include for unbiodegradable nitrogen recycling in the integrated water system as the Biowin model used considers only the WRP.

The bioreactor incorporates the following components:

- Distribution chamber at the inlet to the tank for Stage 3 onwards (not shown in Figure 6-3);
- Anaerobic zone to allow for phosphorus removal, which is aided by chemical removal through ferric sulphate dosing;
- Anoxic zone for the removal of nitrate to nitrogen gas; and
- Aeration zone for the removal of BOD, COD and oxidation of ammonia;
- Membrane tank for the microfiltration of bio-reactor effluent.

The final zone in the bioreactor contains submerged membranes, which act as a physical barrier for the removal of total suspended solids, with a pore size of 0.45µm. The membranes have sub-micron pore size which generates a high quality effluent from a reactor which contains a high level of suspended solids.

Aeration in the aerobic zone of the bioreactor will be provided in the form of submerged fine bubble diffusers positioned on the floor of the tank. Coarse Bubble Diffused Aerators (CBDA) will be required for aeration in the membrane tank. The blowers provided for this purpose will be located inside a building and will be equipped with individual acoustic enclosures. Blowers are required for the both the bioreactor and the membrane tank.

The volumes required for each section of the bio-reactor, shown in Figure 6-3, are given in Table 6-8, as were calculated in the Biowin simulation.

Table 6-8 Bio-reactor zone volumes at Ultimate

Bioreactor Zone	Volume (m3)	Area (m2)	Depth (m)
Anaerobic	154	31	5.0
Primary Anoxic	150	30	5.0
Aerobic	498	99.6	5.0
De-Aeration	48	9.6	5.0
Secondary Anoxic	176	35.2	5.0
Membrane Tank	80.4	33.5	2.4
Total	1106	239	27.4

The bioreactor design relies on the recycle flows to drive sludge growth and retention, de-nitrification and biological phosphorous removal. The flow of each recycle stream was optimized as part of the plant design within the modeling program. The final values achieved at concept level are given in Table 6-9

Table 6-9 MLSS recycle flow rates at Ultimate

Parameter	Units	Flow rate (xADWF)	Flow rate (kL/d at ADWF)	Purpose
A-Recycle (A-MLR)	kL/d	6	5340	De-nitrification
S-Recycle (S-MLR)	kL/d	6	5340	Returning activated sludge from the membrane tanks
R-Recycle (R-MLR)	kL/d	3	2670	Biological Phosphorous Removal

Detailed descriptions of the elements of the overall bio-reactor are given below.

The concept design of the IWC allows for a total of 6, 200 dwellings and an equivalent population (EP) of 18,850 in the ultimate design. This final population allows for potential increase above the Queanbeyan Residential and Economic Strategy 2031 cap of 5,550 residences, minimising any further construction which may be required. The WRP has been designed based on 180L/hd/d. With high levels of water reuse and water conservation, the per capita flow may be much lower. This value provides a conservative basis of design for the WRP. It should also be noted that the biological treatment at the plant is based on load and therefore the per capita flow will not result in smaller process units.

6.4.2 Flow Distribution Chamber

Flow from the inlet works will be conveyed to each of the four bioreactors via a flow distribution chamber, which will split the flow equally between the four bio-reactors. This infrastructure will not be required until more than one bioreactor is completed. It is proposed to construct this at the same stage as the vortex grit structure, so the two channel-based elements will probably be combined in one structure, refer to **Drawing SK-703**.

6.4.3 Anaerobic Tank

The anaerobic tank allows for biological phosphorous removal. Sludge with an elevated phosphate composition is taken from the primary anoxic zone and returned to the feed of the anaerobic tank. This process allows for a net increase in sludge phosphate concentration which is removed as WAS, allowing for a net reduction in the phosphate concentration of the MBR effluent.

Phosphorous removal is completed as a combination of biological phosphorous removal, described above, and chemical phosphorous removal, achieved by dosing ferric sulphate. Utilising biological phosphate removal minimises the required ferric sulphate addition to achieve the licence phosphate output, hence reducing the overall discharge of salt from the WRP into the environment. The combination of both ferric addition and biological phosphorous removal is shown in Figure 6-3. The biological phosphorous removal process used may increase acetic acid requirement due to additional COD demand. The balance of biological and chemical phosphorous removal must be further investigated in detailed design.

6.4.4 Primary Anoxic Tank

The primary anoxic tank converts nitrate to nitrogen gas. This process uses COD. There is likely to be insufficient carbon in the incoming sewage for this purpose and therefore supplementary carbon dosing will be provided for this tank, as described in Section 6.8. The volume of the primary anoxic tank will be approximately 150m^3 per bioreactor.

The primary anoxic tank will be divided into 3 equal zones using baffles. Mixers will be provided in each anoxic zone. Submersible mixers are proposed for this function.

6.4.5 Aerobic Tank

The aerobic tank introduces air into the bioreactor to oxidise organics and ammonia as well as uptake of other nutrients. The volume of each aerobic tank will be approximately 500m^3 per tank, which has been calculated as part of plant Biowin modelling. Mixed liquor will be returned to the aerobic tank from the membrane tank.

Three blowers will be provided for each set of the two tanks in the ultimate plant, totalling six blowers. Each set of three blowers will be configured in a duty/assist/standby configuration. Air from the blower building will be conveyed to the diffusers via elevated pipe work. Air pipe work will need to be designed so as to minimise noise, as it will be above ground.

6.4.6 De-Aeration Tank

The de-aeration tank has been included to reduce the dissolved oxygen before entering the secondary anoxic tank and before the recycle is returned to the primary anoxic tank. This will reduce air entrainment in these tanks. The volume of the de-aeration tank is 48m^3 per tank.

The recycle stream driving de-nitrification flows from the De-aeration tank, where 6xADWF is pumped to the first anoxic tank of each stream via the A -MLR pump.

6.4.7 Secondary Anoxic Tank

The volume of the secondary anoxic tank will be approximately 176m^3 per bioreactor. As with the primary anoxic tank, supplementary carbon dosing will be provided into this tank. This optimises the use of supplementary carbon dosing by providing operator flexibility and allowing the chemical to be dosed into the place where it will be needed most as the flow and load varies throughout the life of the plant.

The secondary anoxic tank will be divided into 2 equal zones using a baffle. Mixers will be provided in each anoxic zone. Submersible mixers are proposed for this function.

6.4.8 Scum Collection

Scum, or biological foam, is a product of a wastewater treatment plant incorporating long sludge age. It develops through the presence of filamentous bacteria which can produce stable, viscous, brown foam on the aeration tank surface. Whilst this is more of an operational issue than a process one, it can cause problems with the odour control process, if the foam enters the foul air extraction ductwork. It is therefore, usually removed frequently, to avoid build-up.

The proposed scum collection system consists of a mechanical scum harvester and scum pumping system. There is an individual system for each process stream.

6.4.9 Membrane Tank

Ultra-filtration submerged membranes are proposed to be used for the WRP. There are different types of membranes commercially available. The submerged membranes sit within the mixed liquor. Two types of submerged membrane systems have been considered; flat sheet and hollow fibre. Hollow fibre membranes have been selected for the concept design.

A typical submerged hollow fibre membrane system can be seen in Figure 6-4.

Figure 6-4 Typical Hollow Fibre Submerged Membranes

Each proprietary membrane has different requirements in terms of the following:

- Aeration requirements (in the membrane tank);
- Volume of membrane tank;
- Chemicals used for cleaning ; and
- Pumping arrangement.

These details will need to be assessed as part of the evaluation and selection of the preferred membrane supplier.

The concept design has been based on a volume of 70m^3 per membrane tank. The membrane tank is designed as a pump-in, gravity out recycle system, whereby the mixed liquor is pumped in to the tank via feed pumps and then recycled back to the aeration tank, along a weir and channel.

The membrane has agitation air supplied by blowers situated in the blower building. This air results in the contents of the tank being highly aerated, therefore, the recycle goes back to the aeration tank, where the excess dissolved oxygen can be utilised.

Effluent is separated from the mixed liquor in the membrane tank. Permeate pumps located in the Tertiary Treatment Building draw water through the membrane filters and discharge it to the chlorine contact tank following UV disinfection.

Waste sludge is pumped out to the solids handling stream for treatment.

The membranes are cleaned using sodium hypochlorite and citric acid (or similar). The sodium hypochlorite removes organic fouling and the citric acid removes inorganic fouling (e.g. from the chemical addition for phosphorus removal or from scaling). Automated maintenance cleans are performed approximately once to twice per week and recovery cleans (reliant on operator involvement), which involves extended chemical soaking, are performed approximately every six months.

6.4.10 Pumping and Blower Summary

Flows throughout the bioreactor will be driven by four sets of pumps, each with a specific purpose; de-nitrification (A-MLR), biological phosphorous removal (R-MLR), return mixed liquor (S-MLR) flow and MBR permeate flow, as follows:

- A-Mixed Liquor Recycle (A-MLR) pumps, one per bioreactor, positioned on a hardstand at ground level adjacent to the tank;
- R-Mixed Liquor Recycle (R-MLR) pumps, one per bioreactor, positioned on a hardstand at ground level adjacent to the tank;

- Membrane tank feed pumps, two per tank positioned on a hardstand at ground level adjacent to the tank. These generate an elevated liquid level in the membrane tanks which allows for the flow of mixed liquor to the aeration zone (S-MLR); and
- Permeate pumps to draw the water from the membrane tanks. These pumps will be housed in a tertiary treatment building to mitigate noise.

One A-MLR pump will be provided for each tank with a common spare unit retained in storage at the plant. The A-MLR stream will flow at 6 x ADWF from the de-aeration tank to the anoxic tank, returning nitrate to the anoxic tank for de-nitrification.

One R-MLR pump will be provided for each tank with a common spare unit in storage at the plant. The stream will flow at 3xADWF, returning phosphate to the anaerobic zone to drive biological phosphate removal.

Mixed Liquor will be pumped to the membrane tank from the anoxic zone of the bio-reactor and pump approximately 7 x the incoming flow. It is proposed that Duty/Standby pumps are used for each tank and cross connections are provided to allow for maintenance of mechanical equipment and cleaning of membranes.

Waste activated sludge (WAS) creates an additional pumping requirement which will be pumped from the MBR tank to the Rotary Drum Thickener located in the solids treatment stream. Two pumps will be provided (Duty/Standby) for each pair of bioreactors. The pumps will be installed within the tertiary treatment building.

Summaries of the pumping requirements for the A-MLR, R-MLR, MBR Feed, WAS and Permeate Pumps are outlined in Table 6-10 - Table 6-14.

Table 6-10 A-MLR Pump Summary

Parameter	Units	Stage 1 1,000 EP	Stage 2a 2,350 EP	Stage 2b 4,700 EP	Stage 4 Ultimate
Bioreactors		Mini reactor	Half size	1 full size	4 full size
Pumps	No.	1	1	1	4
Duty Arrangement		Duty	Duty	Duty/Standby	Duty/Standby per MBR
Pump Capacity	l/s	20	35	70	70
Discharge Head	m	1-5	1-5	1-5	1-5
Power Requirements	kW	0.6	1.1	2.2	2.2
Drive Type		VSD	VSD	VSD	VSD

Table 6-11 R-MLR Pump Summary

Parameter	Units	Stage 1 1,000 EP	Stage 2a 2,350 EP	Stage 2b 4,700 EP	Stage 4 Ultimate
Pumps	No.	2	2	2	8
Duty Arrangement		Duty	Duty	Duty/Standby	Duty/Standby per MBR
Pump Capacity	l/s	10	20	35	35
Discharge Head	m	1-5	1-5	1-5	1-5
Power Requirements	kW	0.4	0.6	1.1	1.1
Drive Type		VSD	VSD	VSD	VSD

Table 6-12 MBR Feed Pump Summary

Parameter	Units	Stage 1 1,000 EP	Stage 2a 2,350 EP	Stage 2b 4,700 EP	Stage 4 Ultimate
Pumps	No.	2	2	2	8
Duty Arrangement		Duty/Standby	Duty/Standby	Duty/Standby	Duty/Standby per MBR
Pump Capacity	l/s	30	70	140	140
Discharge Head	m	1-5	1-5	1-5	1-5
Power Requirements per pump	kW	1.1	2.2	4.0	4.0
Drive Type		VSD	VSD	VSD	VSD

Table 6-13 WAS Pump Summary

Parameter	Units	Stage 1 1,000 EP	Stage 2a 2,350 EP	Stage 2b 4,700 EP	Stage 4 Ultimate
Pumps	No.	2	2	2	2
Duty Arrangement		Duty/Standby	Duty/Standby	Duty/Standby	Duty/Standby
Pump Capacity	l/s	0.12	0.37	6	6
Discharge Head	m	8.0	8.0	8.0	8.0
Power Requirements per pump	kW	0.09	0.12	1.5	1.5
Drive Type		Fixed	Fixed	Fixed	Fixed

Table 6-14 Permeate Pump Summary

Parameter	Units	Stage 1 1,000 EP	Stage 2a 2,350 EP	Stage 2b 4,700 EP	Stage 4 Ultimate
Pumps	No.	4	4	4	12
Duty Arrangement		Duty/Standby	Duty/Standby	Duty/Standby	Duty/Duty/Standby for each pair of MBRs
Pump Capacity	l/s	15	35	35	35
Discharge Head	m	16	16	16	16
Power Requirements per pump	kW	1.1	2.2	5.5	5.5
Drive Type		VSD	VSD	VSD	VSD

Table 6-15 outlines the aeration requirements which are fulfilled by the process blowers. As discussed in Section 6.14.2, the interim mini-scale bioreactors may be instead fitted with a surface aerator to reduce complexity and to physically fit in the small space provided. For Stage 2b onwards, when the fill-size bioreactor tanks are constructed, aeration will be provided by in the form of submerged fine bubble aerators positioned on the floor of the aeration zone.

Table 6-15 Process Aeration Blower Summary

Parameter	Units	Stage 1 1,000 EP	Stage 2a 2,350 EP	Stage 2b 4,700 EP	Stage 4 Ultimate
Surface Aerators	No	1	1	-	-
Blowers	No.	-	-	3	6
Duty Arrangement		Duty	Duty	Duty/Assist/Standby	Duty/Assist/Standby per pair of bioreactors
Average Blower Capacity	m^3/hr at NTP			1,030 per bioreactor	1,030 per bioreactor
Maximum Blower Capacity	m^3/hr at NTP			2,268 per bioreactor	2,268 per bioreactor
Power Requirements per blower	kW			15	15
Drive Type				VSD	VSD
Turndown Required				6.6:1	6.6:1

Note: Aeration capacity for the modular MBR design (Stage 1 and Stage 2a) to be determined in detailed design

6.4.11 Civil

The bioreactor tank will be a partially above ground reinforced concrete structure. One part-bioreactor tank and one membrane tank, configured as two separate halves with internal dividing wall, will be provided in Stage 1b, constructed as a miniaturised bioreactor. At Stage 2a this reactor will be extended to occupy the other half of the full size Tank 1. At Stage 2b, the first full-size bioreactor will be constructed. Over the remaining stages, all four of the bioreactors will be installed.

The bioreactor tanks and associated flow distribution chambers will be covered and odour extracted to prevent odour emissions.

Each pair of bioreactor/membrane tanks will be 36m long, by 12m wide at ultimate development size, where the initial stage will utilise a bioreactor which is 6m wide. The structure of each MBR is shown in **Drawing SK-608**.

6.4.12 Utility Requirements

RW will be the main utility requirement which will be for the scum system sprays at an intermittent flow rate of 3.5L/s.

6.5 Tertiary Treatment

Due to the non-potable domestic re-use of the MBR effluent, both chlorination and UV disinfection are utilised in the WRP.

6.5.1 UV Disinfection

The effluent from the MBR will be disinfected to reduce the count of bacteria and viruses to ensure the product water is suitable for recycling and discharge.

Discharge from the permeate pumps will pass through an in-line ultraviolet (UV) disinfection system, located within the disinfection building. The UV lamps will treat 100% of the flow and be installed in duty/standby arrangement.

6.5.2 Chlorination

Chlorination provides a residual disinfectant which suppresses re-growth within the recycled water pipelines. A pipe line (DN375) will carry WRP effluent from the plant to the Interim (Stage 2b) and Permanent (Stage 3 and 4) Recycled Water Reservoir, which service the Googong township. Given the considerable length and volume, and ideal plug-flow characteristics, the Recycled Water Pipeline will be utilised as the WRP's chlorine contact tank (CCT). Sodium hypochlorite will be dosed into the effluent downstream of UV treatment to maintain the required residual chlorine concentration throughout the transfer pipeline, allowing for adequate disinfection. Utilising the pipeline for chlorine disinfection eliminate the need for a dedicated contact tank at the WRP.

During Stage 1, no RW will be supplied directly to the community. RW will be available for irrigation. WRP effluent will be discharged, via a DN225 outfall pipeline which will act as a chlorine contact tank. A collection tank for RW in stage 1 may be considered in detailed design.

The level of chlorine disinfection utilised in this concept design has been dictated by the CT value, taking both the chlorine concentration and resonance time into account. The contact time used in this design is 60 minutes and the free chlorine concentration is 1.3mg/L, which results in a CT value of 65.7 mg.Min/L based on HOCl concentration.

In times of shortfalls in the supply of Recycled Water to meet demand, potable water will be used to top up the supply. As discussed in Section 4, a dedicated pipeline from the Bulk Water Pump Station (BWPS) will be used to deliver potable water to the WRP's Clear Water Tank. The impact of using the Recycled Water Pipeline as the chlorine contact tank, in times when potable water is being also brought in at the Clear Water Tank is minimal. Where PW is required to make-up for limited RW, the maximum flow rate will be 388m3/hr to meet RW demand within the community. PW will require a lower dose of Chlorine to maintain a residual as it exerts a far lower chlorine demand than RW. Subsequently, the maximum required dose of Sodium Hypochlorite will be 8.8L/hr. This may be compared to the ADWF and PWWF sodium hypochlorite dose rates, under normal operating conditions, of 6.2 L/s and 21.8 L/s respectively.

6.5.3 Service Water

The service water tank supplies the WRP with recycled water for its own internal requirements. Service water can be used for membrane cleaning, screens and screenings and grit handling, scum sprays, UV, hose points, etc.

The service water tank will be fed with RW following UV disinfection. The volume of the service water tank will be 120m^3. As the RW will not be chlorinated at this point, chlorination will be required in the service water tank. Sodium Hypochlorite solution (13%) will be required at approximately 3.3 L/hr. This is based on a required service water flow of 21 L/s, as a conservative estimate, to cover all utility requirements and allow for hose points. The Service water demand requires further consideration in detailed design depending on the specific process unit service water requirements.

The service water tank will then overflow into the recycled water well to ensure that the tank is always full. A low level take-off will be provided back to the permeate pumps for used for back pulsing and cleaning of the membranes. A second higher level take-off will be provided for other uses around site.

The service water system will be a pumped system. Three pumps are proposed operating in a duty/assist/standby configuration. The tertiary treatment facilities are summarised in Table 6-16.

Table 6-16 Tertiary Treatment Summary

Parameter	Units	Stage 1b 1,000 EP	Stage 2a 2,350 EP	Stage 2b 4,700 EP	Stage 4 Ultimate
UV units	No.	2	3	4	6
Minimum Transmissivity	%	50	50	50	50
Dose rate	mJ/cm2	45	45	45	45
CT in RW transfer pipeline	mg.min/L	Nil	65	65	65
Volume of pipeline for Cl disinfection[1]	m^3	Nil	306	306	466
Volume of Service Water Tank	m^3	120	120	120	120

[1]The volume required to maintain a CT value of 60 mg/min/L are generally far smaller the volume of the pipeline.

6.5.4 Clear Water Tank

The Clear Water Tank (CWT) will be used to supply the Recycled Water Pumping Station with feed water, and the membrane tanks with backwash water. At 180m^3, the CWT will also have sufficient volume to provide two complete backwash cycles to the membranes.

The CWT will be a steel or FRP construction with roof, installed on concrete slab. The CWT will include an inlet from the Potable Top Up of Recycled Water Main from the BWPS, with an air-break to prevent any possibility of cross contamination of potable water with recycled water.

In Stage 1, the smaller Service Water Tank will be used as the CWT, however at Stage 2a when recycled water becomes available to the network, the CWT will be constructed.

6.5.5 Foul Water Tank

The Foul Water Tank (FWT) will have sufficient volume to contain two complete membrane tank backwashes, giving a requirement of $180m^3$. The FWT will be constructed alongside the CWT, also of FRP or steel construction with roof. For Stage 1, due to the smaller mini-tanks for the membranes, a smaller process volume of $40m^3$ is required. The Stage 1, the mini-bioreactor's adjacent emergency storage tank will be used as the FWT.

6.5.6 Civil

The UV system will be skid mounted close to the permeate pump area.

The service water tank could be a fibreglass tank located close to the skid mounted pump units. The tank will be uncovered apart from the service water tank which will be covered to prevent entry of foreign objects.

6.5.7 Utility Requirements

The UV system will require a constant flow of service water through it, of approximately 1.5L/s. The UV lamps will create heat in the process and therefore will require flow throughout them at all times, otherwise they will heat the water and cause fouling and overheating. The membrane back pulse and cleaning means that there are frequent times when there will be no flow through the UV from the membrane tanks, therefore, a service water flow should be provided.

6.6 Effluent Discharge

In Stage 1, the treated effluent will flow to Basin 1 where it may be captured for irrigation use or discharged to the local stormwater system.

From Stage 2 onwards, when Recycled Water is reticulated directly to the community, the recycled water reservoir will be located approximately 2800m and 4300m from the WRP for the Interim and Permanent Reservoir Site respectively. Pumps will transfer recycled water to the RW reservoir via the dedicated Recycled Water Pipeline, which as noted above, is also the WRP's chlorine contact tank.

The Recycled Water Pump Station (RWPS) will consist of suction pipework from the Clear Water Tank, dry mounted pumping units configured with standby capacity, and discharge manifold feeding the Recycled Water Pipeline. The RWPS will be located close to the Clear Water Tank.

When the RW reservoirs throughout the development are full, RW will be discharged to the environment via Basin 4.

6.6.1 Civil

The RWPS pumps and pipework will be situated within the tertiary treatment area, being close to the permeate pumps, UV equipment and Clear Water Tank. The pumps will be dry-mounted on a plinth on a concrete slab. The wet well for the RWPS is the Clear Water Tank itself, which is fed with treated Recycled Water from the WRP, or if demand shortfall is occurring, with potable water from the BWPS to augment supply to the recycled water system.

6.6.2 Mechanical

Centrifugal pumps have been used in this case and will be operated in a duty/standby configuration. A summary of the key parameters of the transfer pump is included in Table 6-17. The discharge head required by the pump station varies through the development as additional reservoirs are constructed

Table 6-17 Recycled Water Pump Station Summary

Parameter	Units	Stage 1b 1,000 EP	Stage 2a 3,600 EP	Stage 3a 9,400 EP	Stage 4 Ultimate
Pumps	No.	Nil	2	3	4
Duty Arrangement		Nil	Duty/Standby	D/D/Stdby	D/D/D/Stdby
Pumping Capacity	l/s	Nil	30	60	137
Discharge Head	m	Nil	63	97	100
Drive Type		Nil	VSD	VSD	VSD

The engineering details shown in Table 6-17 above will need to be confirmed in detailed design of the subsequent stages, when peak flows are confirmed and WRP operating modes are known.

6.7 Flow Management

Flows up to 3.5 x ADWF will be treated at the WRP.

Ensuring plant redundancy during power outages is an important aspect of the plant design. The WRP will have a single LV power feed from the local power system. Emergency backup power will be available with the provision of an emergency generator connection point. In addition, each SPS will be equipped with a covered emergency storage to temporarily store sewage during power outages, and emergency generator connection points to quickly restore power in an emergency.

The WRP will incorporate an emergency overflow facility located at the inlet chamber upstream of each of the bioreactors in the Stage 1 and Stage 2 configuration. Spare tank capacity has been included in the adjacent concrete tanks as part of the main bioreactor structure to accommodate emergency overflows.

As the plant expands and more process units are available, the relative impact of a mechanical failure is lower because multiple process trains are available. Therefore, due the high level of process and mechanical standby available in the later Stage 3 and Stage 4, and the ability to install temporary power with emergency generators, it is assumed that emergency storage will not be required in the longer term.

Beyond these contingency measures, WRP any overflows (comprising of screened and de-gritted sewage) will be conveyed via buried pipeline to the stormwater system, discharged to the adjacent watercourse.

6.8 Chemicals

Several chemicals will be utilised at the WRP comprising the following:

- Ferric sulphate for odour control at the inlet chamber and for chemical phosphorus removal in the bioreactor;
- Magnesium hydroxide for alkalinity addition to aid the biological processes that occur within the bioreactor;
- Sodium hypochlorite for disinfection of the secondary effluent and cleaning of the submerged membranes;
- Citric acid for cleaning the submerged membranes; and
- Acetic Acid as a supplementary carbon source to assist the biological processes within the bioreactor.

These chemicals will be stored on-site in sealed storage tanks, located together with dosing pumps, in a centralised bunded facility at the WRP (with a separate bund for each chemical). This facility will be roofed together with an adjacent bunded tanker delivery area. Chemical storage tanks have been sized for a minimum of 28 days storage. In stage 1, chemical dosing facilities will be implemented as IBC storage in portable bunds.

Chemical usage will have an impact on the salt levels in the effluent. It is proposed to configure Googong WRP for "Biological P removal". A Salt Balance has been prepared and is presented in **Appendix C**.

A separate nutrient storage and dosing facility will be located at the odour control facility for dosing of the biological trickling filter. Tanker delivery of the nutrient will be completed via the bunded tanker area located adjacent to the main chemical dosing facility.

Polymer will be utilised for thickening and dewatering and will be located on the ground floor within the dewatering building.

6.9 Solids Management

6.9.1 Disposal and Reuse

A number of sources of solids are present within the treatment process, including primary and secondary solids waste streams.

Primary solids, including screenings and grit, will be collected in self -contained bins in the inlet works building and will be removed from the site via these bins. Primary solids will be removed by truck typically twice weekly.

Secondary solids provide a re-use opportunity in the form of bio-solids. Solids produced as waste sludge from the bioreactor processes, will be mechanically thickened and then sent to an aerobic digester (from Stage 2b onwards), which will reduce the volatile solids and bacteria in order to ensure it is suitable for reuse. The digested sludge will then be mechanically dewatered prior to being transported off-site. In Stages 1 to 2a, no digestion has been included in order to limit capital expenditure where sludge volumes are low.

The solids processing system will include rotary drum thickeners (RDTs) which will be installed at ground level within an uncovered bunded area. Thickened sludge will be pumped to the aerobic digester, which will comprise an above-ground covered reinforced-concrete structure, equipped with odour extraction facilities.

Digested sludge will be pumped to a centrifuge, located in a dedicated room in the plant building. Polymer storage, makeup and dosing facilities, will be installed within a bunded area on the ground floor of this building.

Dewatered sludge cake will be taken by an off-loading conveyor from the centrifuge to a sealed storage bin. The bin will be collected by standard 10m3 skip truck for removal from the site.

The NSW DECCW released Environmental Guidelines on the Use and Disposal of Biosolids Products in 1997. This outlines the requirements for biosolids treatment and the potential uses of treated biosolids. The guidelines give classifications to the biosolids and determine what form of reuse the biosolids can have. Classification is given in terms of an allowed usage, within the categories outlined in Table 6-18. The definition of each Contaminant and Stabilisation grade is discussed in **Appendix G**.

Table 6-18 Classification of Biosolids (EPA, 1997)

Use	Contaminant Grade	Stabilisation Grade
Unrestricted Use	A	A
Restricted Use 1	B	A
Restricted Use 2	C	B
Restricted Use 3	D	B
Not Suitable for Use	E	C

Classification of biosolids, is based on the achieving a Contaminant Grade and a Stabilisation Grade. Contaminant grading is from Grade A to Grade D and is based on the concentration of certain contaminants in the biosolids.

Stabilisation grading depends on treatment methods used to reduce the pathogens present in the biosolids and the methods used for vector attraction reduction. The stabilisation grading can be between Grade A to Grade C, depending on the type of treatment method used.

Waste activated sludge from the bioreactors will be treated to achieve a stabilisation Grade B. It is assumed that there will be no major contaminants in the catchment and therefore, only pathogen removal has been included in the design.

Grade B biosolids can be used for the following purposes:

- Agriculture;
- Forestry;
- Soil and site rehabilitation;
- Landfill disposal; and
- Surface land disposal (to be applied within the boundaries of the sewage treatment plant site.

6.9.2 Process Description

Table 6-19 summarises the key parameters associated with the Waste Activated Sludge (WAS) System.

Table 6-19 WAS System Summary

Parameter	Units	Stage 1 1,000 EP	Stage 2b 4,700 EP	Stage 3a 9,400 EP	Stage 4 Ultimate
Aerobic Digester					
No of Units	-	Nil	1	1	1
Digester Sludge Age	Days	-	11	11	11
Feed Concentration	% dry solids	-	3.0 minimum	3.0 minimum	3.0 minimum
Blowers					
Duty Arrangement		-	Duty/Standby	Duty/Standby	Duty/Standby
Maximum Blower Capacity	m^3/hr at NTP	-	296	296	296
Power Requirements per blower	kW	-	3	3	3
Turndown Required		-	6.6:1	6.6:1	6.6:1
Rotary Drum Thickener					
No of Units	No.	-	1	1	1
Duty Arrangement		-	Duty	Duty	Duty
RDT Capacity	m^3/hr	-	20	20	20
Solids Capture Rate	%	-	95 minimum	95 minimum	95 minimum
Thickened Sludge Concentration	%DS	-	3.5	3.5	3.5
Average Polymer Dose Rate	kg/tds	-	3.0	3.0	3.0
Peak Polymer Dose Rate	kg/tds	-	4.0	4.0	4.0
Centrifuge feed pumps					
No of Pumps	No.	1	1	1	1
Duty Arrangement		Duty	Duty	Duty	Duty
Pump Capacity	l/s	5	5	5	5
Drive Type		VSD	VSD	VSD	VSD
Centrifuge					
Number		1	1	1	1
Capacity	m^3/h	13	13	13	13
Duty Arrangement		Duty	Duty	Duty	Duty
Average Polymer consumption	kg/Tds	8	8	8	8
Maximum Polymer consumption	kg/Tds	10	10	10	10
Storage Bins					
Number		1	1	1	2
Capacity	m^3	10	10	10	20

6.9.3 Civil

The RDT aerobic digester and pumps will be mounted on a drained reinforced concrete support slab. The aerobic digesters will consist of a concrete reinforced above ground tank, which will contain close fitting covers for odour control. The centrifuge will be housed in a dedicated room in the plant building. A conveyor will discharge from the centrifuge, through the wall of the plant building to a covered storage bin. The storage bin will require foul air extraction.

6.9.4 Utility Requirements

The RDT and centrifuge both require service water for their process. This will be dependent on the unit installed.

Sludge cake collection and removal activities will involve approximately two truck movements each week at ultimate WRP operation.

6.10 Works Drainage System

A small sewer system will be provided around the process areas of the plant to collect occasional small spills and equipment hose down drainage. The system will direct flows to the Works Pump Station, which will be a small prefabricated FRP unit located downslope from the main process units. The Works PS will return flows to the inlet works. Duty/Standby pumps will be provided.

6.11 Odour Management

The WRP will be equipped with centralised OCF comprising biological trickling filters, activated carbon filters, and two extraction fans (equipped with acoustic hoods) and associated ductwork and ancillary works. Treated air will be discharged via an exhaust stack. All units will be mounted on a reinforced concrete slab.

A detailed assessment of the odour control requirements of the WRP is presented in **Appendix D**. The key findings are shown in the following sub-section.

6.11.1 Odour Dispersion Modelling

Odour dispersion modelling has been undertaken to define the basis for the odour control system concept design. The modelling has been completed and documented in the Googong New Town WRP Odour Impact Assessment (MWH, 2009). This report concluded that, in order for the Googong WRP to ensure compliance with the DECCW requirements, a number of areas of the plant require coverage and odour control, as summarised in Table 6-20.

Some design development of the WRP has occurred since the odour modelling was completed. Furthermore, the stack location within the WRP site is similar to that of the previous case. Given that odour treatment covers will be provided on the same process units and that a biotrickling filter and carbon filter will be used in all cases, the risk of variation from the original modelling case is low. However, the odour modelling will need to be reviewed in detailed design where the final configuration of the WRP, along with ventilation volume, has been finalised.

MWH

Table 6-20 WRP Process Units to be Covered for Odour Control

WRP Area to be Covered	Typical Equipment
Preliminary Treatment Area	• Receiving chambers for discharging rising mains • Screens • Screenings handling and storage bin • Grit removal • Grit classifier and storage bin
Secondary Treatment Plant Anoxic and Anaerobic Zones	• Biological reactors
Secondary Treatment Plant Membrane Tank	• Membrane tanks
Sludge Digesters	• Aerobic digesters
Sludge Dewatering and Thickening	• Rotary Drum Thickener (RDT) • Dewatering centrifuge or belt filter press.
Sludge Storage	• Covered sludge storage bin

6.11.2 Foul Air Flows

Two cases have been studied in the odour control strategy of the WRP, the case at Stage 2b (4,700EP) and the full scale Stage 4 (18,849 EP). These two cases have given two indicative points of the foul air loads and odour treatment requirements across the staging of the WRP development. As plant equipment is specified for each stage of development within detailed design, the covered volumes and foul air flows may change and hence the final Odour control requirements will be specified at such time.

A summary of the foul air flows for the stages of WRP development are provided in Table 6-21

Table 6-21 OCF Process Equipment Sizes & Foul Air Flows – Stage 2b of development

Assessment horizon	Covered process equipment	Volume of equipment (m3)	Air changes per hour	Extraction rate (m3/h)
Stage 1, 2 and 2b	Inlet Work[1]	116	20 to 25	1,908
	Bioreactor Tank[2]	Based upon air demands		2,700
	Membrane Tank	Based upon air demands		2,954
	Sludge Treatment	90	6 to 25	596
	Total Foul Air Flow to be Treated in Central Odour Control Facility			8,167
	Design Airflow			**8,200**

[1]Inlet Works sizes have been taken from supplier details for screens, grit, and washing and classification facilities. The inlet works used in Stages 1and 2a will require further assessment as a smaller temporary unit will be utilised in these cases. A conservative case is taken for the inlet works equipment volume.
[2]Secondary treatment area footprints have been taken from conceptual layouts and aeration demands from process calculations.

Table 6-22 OCF Process Equipment Sizes & Foul Air Flows – Ultimate Development

Assessment horizon	Covered process equipment	Volume of process equipment (m3)	Air changes per hour	Extraction rate (m3/h)
Stage 3 onwards	Inlet Works[1]	116	20 to 25	1,908
	Bioreactor Tank[2]	Based upon air demands		10,833
	Membrane Tank	Based upon air demands		5,909
	Sludge Treatment	90	6 to 25	1,034
	Total Foul Air Flow to be Treated in Central Odour Control Facility			19,684
	Design Airflow			**19,700**

[1]Ultimate Inlet Works sizes have been taken from supplier details for screens, grit, and washing and classification facilities.
[2]Secondary treatment area footprints have been taken from conceptual layouts and aeration demands from process calculations.

6.11.3 Foul Air Loads

Summary load sheets for Googong WRP, at the various planning horizons are listed in Table 6-23. It is important to note that each WRP is individual in its odour generating potential. Different sewer networks, process units (and their mode of operation) all act to generate (and/or convey) odorous compounds in varying magnitudes. These calculations have been done based on ferric dosing upstream of the treatment process to reduce load to the odour control facility.

Table 6-23 Googong WRP Stages 1 and 2 Foul Air Flow and Load Sheet

Odorous compound	Average (ppm)	Maximum (ppm)
Hydrogen Sulphide (H_2S)	8.2	21.7
Mercaptans (R-SH)	0.8	1.6
Ammonia (NH_3)	0.4	1.1
Volatile Organic Compounds (VOC's)	5.5	15.6
Dimethyl Sulphides (DMS)	2.0	4.3
Design Airflow[1] = 8,200m^3/h		

[1]Only foul airflows requiring odour control treatment have been stated.

Table 6-24 Googong WRP Ultimate Foul Air Flow and Load Sheet

Odorous compound	Average (ppm)	Maximum (ppm)
Hydrogen Sulphide (H_2S)	4.0	20.0
Mercaptans (R-SH)	0.6	1.3
Ammonia (NH_3)	0.3	0.7
Volatile Organic Compounds (VOC's)	5.3	13.6
Dimethyl Sulphides (DMS)	1.4	2.9
Design Airflow[1] = 19,700 m^3/h		

[1]Only foul airflows requiring odour control treatment have been stated.

This odour will be treated and exhausted through a stack which has a height of 8m.

Based upon the odour dispersion modelling completed for the WRP the following capture efficiencies (and corresponding negative pressures) were derived:

- Inlet Works = Minimum capture efficiency of 99% (15.0 – 30.0 Pa negative pressure required); and
- Secondary Treatment Areas = 99.9% capture efficiency required (30.0 Pa negative pressure required).

6.11.4 Covers and Containment

At the Inlet Works areas, channels and the vortex grit chamber will be equipped with flat trafficable FRP covers similar to those used extensively throughout NSW for inlet works. They are lightweight, can be manufactured to seal complex structures and arrangements, and can be sealed under compression with suitable gaskets to ensure an excellent seal (often between 99 to 99.9% capture rate). All equipment within the Inlet Works (including screens, screenings dewatering, grit classifier, conveyors and collection bins) will be sealed and/or equipped with close-fitting odour containment hoods, and fitted with off-takes connected to the foul air extraction ductwork.

For the larger tank spans that require covers, such as the bioreactors or digesters, Arc span FRP covers have been applied to numerous existing treatment plants in QLD, WA & NSW to provide successful odour containment. Although the arc span nature of the cover generates a greater foul air volume than a flat cover, they can be sealed under compression with suitable gaskets to ensure an excellent seal. Previous experience has shown that ensuring a seal in odour cover designs may be the most problematic area of odour cover design.

Sludge treatment areas will be contained and foul air extracted and conveyed to the OCF. The aerobic digesters will have covers similar to those described for the bioreactor (FRP Arc Span). Other process units will be self contained. The RDT and centrifuge will have air directly extracted from the unit and the sludge storage bin will also be self contained.

6.11.5 Odour Treatment

Several methods can be utilised for odour treatment dependent on discharge requirements and air flows. Wet chemical scrubbers and biotrickling filters and carbon units both provide the appropriate level of treatment. A comparison of both treatment process options is included in **Appendix D**.

In the case of Googong WRP, a biotrickling filter followed by a carbon filter has been selected. The reduced level of maintenance and chemicals required for the biological treatment option makes it more suited to the size of installation expected for Googong WRP.

A biotrickling filter uses biomass attached to a media to remove some odorous constituents. The biomass needs to be kept wet and potentially fed additional nutrients to enhance growth. There are some constituents, such as some volatile organic carbons, which are not removed through biological means, and as such an activated carbon system is required to ensure odour removal to a level which is suitable for stack discharge.

A schematic of the biotrickling and activated carbon process is shown in Figure 6-5.

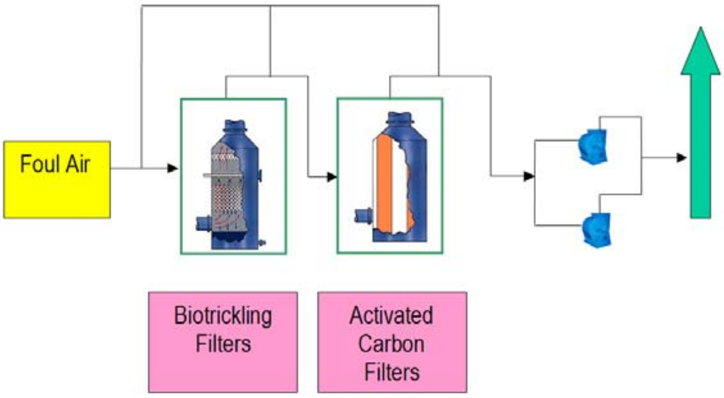


Figure 6-5 Biotrickling and Activated Carbon Process Schematic

6.12 Civil

6.12.1 WRP Structures/Foundations

The geotechnical investigation indicates the presence of dacite rock material at relatively willow depth across the development, including the WRP site. Douglas Partners 2009 geotechnical report indicates that the dacite allowable base bearing pressures for design purposes should be taken as 500 kPa. This material provides ideal founding conditions for all structures on site.

Due to the sloping site topography and required elevation of main structures, a number of structures will require cut and fill to provide a level foundation. Final cut and fill requirements will be determined in detailed design, however three general foundation options have been identified:

1. Excavate down to suitable dacite material, where possible across the length and breadth of the structure and construct foundation directly on the dacite.

2. Where fill is required, excavate down to suitable material, and build the ground level up by placement of mass concrete, or engineered fill, to the required level. Mass concrete is preferable under large or water retaining structures since this will not be subject to higher levels of differential settlement as would the engineered fill. Where engineered fill is used, the base slab of any structure should be designed with an allowance for differential settlement, such as slab thickening. The use of mass concrete or engineered fill will depend on the cost, and relative volumes of material required.

3. Support the elevated section of the structure on a suspended concrete floor slab, using concrete columns founded on suitable rock material. This option would be more suitable for smaller or lightweight structures, such as the administration building.

Groundwater levels are expected to be considerably deeper than the deepest structures proposed for site. However, groundwater lowering measures such as subsurface drains should be considered for around buried structures, during detailed design.

6.12.2 Roads

Site roads are proposed to be constructed of two coat seal and generally be 5m wide. Kerb and gutter will not be used; instead a surface scrape drain will be used.

6.12.3 Earthworks

A significant quantity of fill is required across the WRP site to raise ground levels for structures, and road levels. Excavated material on site will be much less than that required, so a significant amount of material will need to be imported. The imported material can be sourced in part from the excavations for the Sewage Pumping Stations, the potable and recycled water reservoir sites and trenching works for the pipe laying activities.

The fill requirements for each structure will need to be determined during detailed design once the use of engineered fill, mass concrete fill or elevated floor slab has been confirmed.

It is anticipated that significant amounts of spoil will be available from other parts of the Googong development. The WRP layout **Drawing SK-701** shows a bund surrounding the south and east boundaries of the WRP, which could be used to locate excess spoil from the development construction, to reduce noise and visual impacts of the WRP on the nearby communities.

6.12.4 Drainage

Proposed drainage will follow the topography of the site which falls from the north-western corner to the south-eastern corner of the site, using a combination of surface drainage and underground pipes, with details to be determined in detailed design. Stormwater discharge from the site would flow to the existing natural overland flow path located at the south eastern corner of the site. Some sediment control works will be required during construction.

6.12.5 Landscaping

As noted above, any excess excavated material from the general development could be utilised on site for landscaping, in the form of bunds to reduce noise and visual impacts. Tree planting will be undertaken around the perimeter of the WRP site to help reduce visual impact of the site. A 50m buffer zone has been provided around the outside of the WRP site, and this should be suitably planted and landscaped in accordance with the overall development plan.

6.12.6 Security

The site will be securely fenced to the standards of the local water agency and this is anticipated to consist of cyclone wire fencing.

Overhead site lighting will be required at buildings and key points around the site, such as entrances and the site frontage. Emergency lighting will be required in the event of power failure.

These details should be confirmed during the detailed design phase of works.

6.12.7 Buildings

The combined Administration/MCC building is could be constructed of either steel frame with infill block work, or tilt-up pre-fabricated concrete panels, or brickwork. A concrete base will be required. Coloursteel or similar roofing material is expected and will be appropriately painted to reduce visual impact. These details will need to be confirmed by an architectural designer during detailed design. A 2 hour fire rating will probably be required for the MCC room.

The mechanical plant room will accommodate process blowers and dewatering gear, and will consist of a simple unlined shed construction over a concrete slab.

6.13 Testing & Commissioning

The plant will be commissioned in a staged manner. The commissioning stages are discussed in more detail in Section 7.2 and are summarised as follows:

Testing – hydrostatic testing of tanks, point to point electrical testing, factory acceptance testing.

Dry commissioning – clean and inspect tanks and pipework,

Wet Commissioning - using potable water before sewage is introduced into the system. Wet commissioning will provide a method of testing mechanical kit and sequence testing the system through the automatic control system, checking each of the WRP sub systems.

Process Commissioning - once sewage is introduced to the tank, tanks be seeded with mixed liquor from another plant for this purpose in order to speed up the process. The plant provides biological treatment and therefore needs a minimum load to operate correctly. If this minimum load is not provided then the biomass may become unstable and issues such as foam may be apparent. There is also a chance if the loads are too low that the biomass may become stressed and create extracellular polymeric substances (EPS) which may foul the biomass although the risk of this is low in purely domestic sewage. With the 1000EP mini-bioreactor planned for Stage 1, preliminary calculations suggest that a minimum load will eventuate from 150 EP. Before this load is achieved sewage should be tankered to another treatment plant for treatment from SPS1.

Process Validation - Following commissioning there will be a period for validation of the process for Stage 1, then again at Stage 2a for recycled water. This will be developed in the risk assessment which has not yet been completed, however experience of this is that this period is a minimum of three months. The risk assessment is to be completed in parallel with ongoing design activities.

6.14 Staging

Construction of the WRP will be staged to match capacity with the growing needs of the Googong community. The proposed construction stages are summarised in Table 6-25.

Table 6-25 Equivalent population estimates to be accommodated by Staged WRP

Stage	EP Capacity (Network)	EP Capacity (WRP)	EP Minimum for This Stage	Sized for	Comment
Stage 1a	1,000	-	>0 EP	Neighbourhood 1A, Subdivisions 1 & 2	Initial subdivision, potable water network only, reservoir at Interim Site
Stage 1b	-	1,000	>125 EP	Neighbourhood 1A, Subdivisions 1 & 2	1/4 scale bioreactor at WRP, cease tankering of wastewater.
Stage 2a	3,600	2,350	>1,000 EP	Neighbourhood 1A, Start RW	Construct recycled water network, 1/2 scale bioreactor at WRP

Stage	EP Capacity (Network)	EP Capacity (WRP)	EP Minimum for This Stage	Sized for	Comment
Stage 2b	-	4,700	>2,350 EP	First main WRP Bioreactor	WRP expanded to first full bioreactor
Stage 3a	9,400	-	>3,600 EP	50% Devt	Network extended to final Reservoir Site,
Stage 3b	-	9,400	>4,700 EP	50% Devt	WRP capacity expanded with second full bioreactor.
Stage 4	18,850	18,850	>9,400 EP	100% Devt	Reservoir capacity increased, WRP capacity expanded to Ultimate

Table 6-26 outlines the major process equipment which will be required at each stage of WRP construction. The 4 stages outlined represent each modular case prior to the full scale development.

Table 6-26 Equipment requirements for modular WRP

Process unit	Stage 1	Stage 2a	Stage 2b	Stage 3	Stage 3 and Stage 4
EP	1,000	2,350	4,700	9,400	9,400 (st 3) 18,850 (St 4)
Plan Sketch	A1081402-SK701	A1081402-SK702	A1081402-SK703	A1081402-SK704	A1081402-SK705
6mm screens	Temporary construction above first tank	Temporary construction above first tank	Ultimate construction	Ultimate Construction	Ultimate Construction
1mm screens	Temporary construction above MBR	Temporary construction above MBR	Ultimate construction	Ultimate Construction	Ultimate Construction
Grit removal	none	none	1 tank	1 tank	2 tanks
Bioreactors	Mini-bioreactor	Half-scale bioreactor	1 full size bioreactor tank	2 full-size bioreactor tanks	4 full-size bioreactor tanks
Bioreactor aeration	Temporary surface aerators	Temporary surface aerators	3 blowers	3 blowers	6 blowers
Membrane Tanks	2 tanks with ballast	2 tanks with ballast	2 tanks	2 tanks	4 tanks

Process unit	Stage 1	Stage 2a	Stage 2b	Stage 3	Stage 3 and Stage 4
UV disinfection	2 units	3 units	4 units	5 units	7 units
Chemical Storage and dosing	Intermediate Bulk Container	Chemical dosing facility constructed	Staged as required	Staged as required	Ultimate construction
Odour control	1 biotrickling filter 1 carbon filter	1 biotrickling filter 1 carbon filters	1 biotrickling filter 1 carbon filters	2 biotrickling filters 2 carbon units	2 biotrickling filters 3 carbon units
Service Water System	All constructed in Stage 1	All constructed in Stage 1	All constructed in Stage 1	All constructed in Stage 1	All constructed in Stage 1
Clear Water Tank	Use Service water tank	Use service water tank	180m^3 tank	180m^3 tank	180m^3 tank
Foul Water Tank	Use emergency storage	Use emergency storage	180m^3 tank	180m^3 tank	180m^3 tank
Sludge thickener	1 unit	1 unit	1 unit	1 unit	1 unit
Aerobic Digesters	none	none	Half-scale digester	Half-scale digester	Full size digester staged
Centrifuge	1 unit	1 unit	1 unit	1 unit	1 unit

6.14.1 Staged Inlet Works

During construction Stages 1 and 2a, the full scale inlet works will not be necessary given the flows received by the WRP. In this instance a smaller temporary inlet works will be operational, mounted on a platform, above the scaled-down bioreactor tank. The full scale inlet works will be required as part of the Stage 2b upgrade, accommodating the first full scale bioreactor, sized for 4700EP. The full sized inlet works will include the elevated concrete structure accommodating the inlet channel and grit removal tank.

6.14.2 Staged MBR Design Approach

The limiting factor in the ability to design the WRP to treat loads represented by the growing development is the scalability of the bio-reactor. Two scaled-down design stages have been developed for the WRP which allow the bio-reactor to operate given the smaller equivalent population during early stages of development, Stage 1 and Stage 2a. The bioreactor has been scaled, in accordance with the Biowin modelling scenarios completed, to treat the flows which are expected in these two cases. The bioreactor volumes for each case are outlined in Table 6-27.

Table 6-27 MBR zone volumes for interim stage bio-reactor design

Zone	Unit	Volume (m3)		
		Stage 1a and 1b	Stage 2a	Stage 2b (1 x full scale bioreactor)
Maximum EP	number	1000	2350	4700
Maximum ADWF	kL/d	180	423	850
Anaerobic	m^3	32.6	76.6	154
Primary Anoxic	m^3	31.8	74.6	150
Aerobic	m^3	106	247.8	498
De-Aeration	m^3	10.2	23.9	48
Secondary Anoxic	m^3	37.3	87.6	176
Membrane Tank	m^3	17.0	40.0	80.4
Total	m^3	234	550	1106

The initial stage of MBR construction, Stage 1, will utilise a reactor to treat a maximum ADWF of 180kL/d, which is the product of 1000EP. The reactor sections, anaerobic, primary anoxic, aeration, de-aeration and secondary anoxic have been scaled to allow for a 20 day sludge age at this Flow rate.

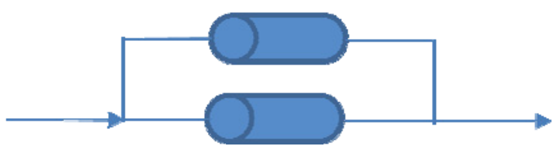
The stage 2a upgrade will be required as the stage 1 reactor reaches capacity. In order to construct this upgrade, the MBR tank will be extended to accommodate the full overall plant capacity. The newly cast section of tank will become the bio-reactor working volume, which will be appropriately partitioned with baffles to accommodate the required section volumes to treat 423kl/d, corresponding to 2350EP.

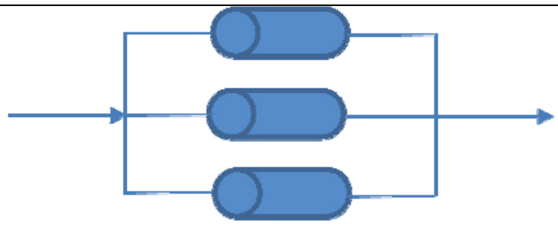
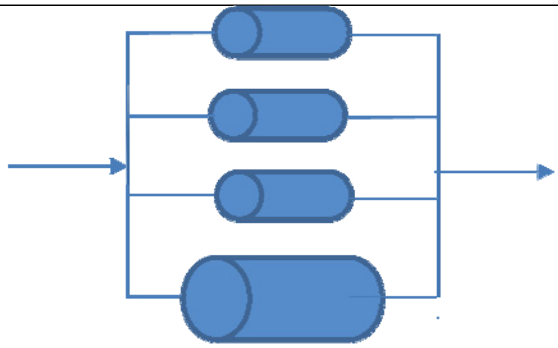
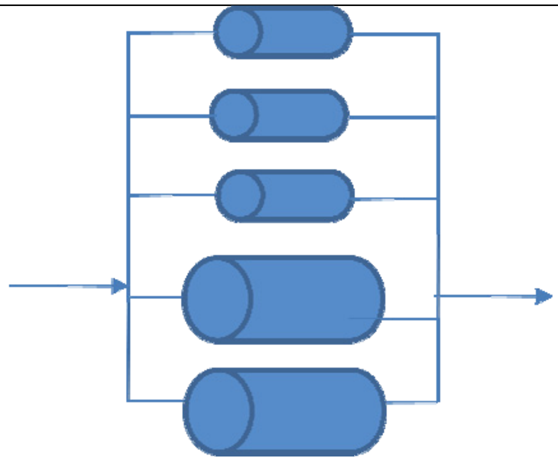
As the 2350EP plant reaches capacity, the stage 2b upgrade will be completed. This will involve casting a full scale mirror image bioreactor tank alongside the current structure. The new structure will be segmented with permanent partitions and baffles to in order to operate at a full maximum capacity of 840kl/d, corresponding to 4700 EP.

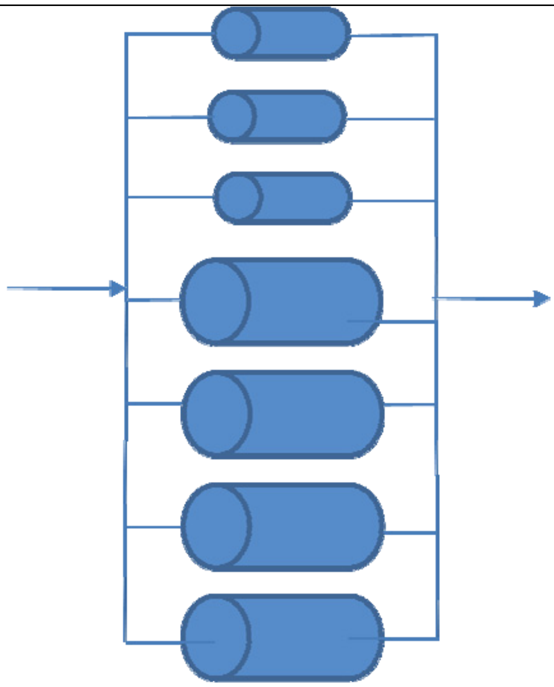
6.14.3 Staged Tertiary Treatment

The UV system has been designed in a modular arrangement to allow for the staging of the WRP. The stages used are outlined in Table 6-28. The stages are designed based on a 5x peak factor in stages 1a, 1b and 2a and a 3.5x peaking factor in stages 2b, 3 and 4.

Table 6-28 Growth Strategy for UV Disinfection System

Stage	Configuration	Description	Maximum ADWF at PF of 5 (kL/d)[1]	Maximum ADWF at PF of 3.5 (kL/d)[1]
Stage 1 (a and b)		2 Small units D/S Configuration	250	360

Stage	Configuration	Description	Maximum ADWF at PF of 5 (kL/d)[1]	Maximum ADWF at PF of 3.5 (kL/d)[1]
Stage 2a		3 Small units D/D/S Configuration	500	710
Stage 2b		3 Small units + 1 Medium unit D/D/D + S Configuration	640[2]	910[2]
Stage 3		3 Small units + 2 Medium units D/D/D + D/S Configuration	1,390	1,980

Stage	Configuration	Description	Maximum ADWF at PF of 5 (kL/d)[1]	Maximum ADWF at PF of 3.5 (kL/d)[1]
Stage 4		3 Small units + 4 Medium units D/D/D + D/D/D/S Configuration	2,670	3,800

[1] All Flow rates used based on Wedeco LBX90 for a 'small' unit and Wedeco LBX200 for a 'medium' unit
[2] Flow-rates capped at single duty medium unit to allow sufficient standby capacity

Chlorination will occur within the RW transfer pipeline. Staging for the network dictates that for Stage 2 the pipeline will be approximately 2770m long, from the WRP to an interim reservoir. In Stages 3 and 4, the pipeline will be approximately 4220m long. The PWWF which is expected, given an initial peaking factor of 5x ADWF, is 3.5 ML a day, which corresponds to a required disinfection pipeline length of 1320m in order to maintain the CT value and contact time. Further detail of the staged construction of the RW transfer pipeline is given in Section 4.2.2.

During stage 1, RW will be chlorinated within the DN225 transfer pipeline to basin 1. This water may then be used for irrigation. The pipeline will be approximately 950m in length to allow for a CT value and contact time to be maintained.

6.14.4 Equipment Delivery Times

A construction schedule is being prepared separately to this Concept Design Report by others. Approximate equipment delivery times have been obtained from suppliers and contractors to enable a critical procurement path to be determined. The delivery times are given in Table 6-29.

Table 6-29 Summary of Equipment Delivery Lead Times

Item	Minimum Delivery Time	Maximum Delivery Time	Source
Bulk Water Pumping Station			
Pumps	22 wks	24 wks	Supplier advice, KSB 17 Nov 09
Motor Control Centre	14	20 wks	SRS electric advice as used on various projects

Item	Minimum Delivery Time	Maximum Delivery Time	Source
Transformer	24	26 wks	ABB advice as used on Quakers Hill Sewage Treatment Plant
DN600 DICL Pipework	6	8 wks	Tyco advice
Small dia valves, fittings	8	10 wks	Tyco advice
DN1800 Valve		24wks	AVK Valve supplier advice
Reservoir Site			
Reservoir Tanks	10 wks	14 wks	Supplier advice, Tasman Tanks 17 Nov 09
High Level Zone Pumps		8 wks	Supplier quotation, KSB, 8 Sept 09
Motor Control Centre	12 wks	16wks	Based on Wollongong project delivery times
Chemical Dosing Unit		20 wks	Quotation for similar project
Water Recycling Plant			
Inlet Screens	22 wks	24 wks	Supplier quotation, VoR 27 Oct 09
Blower & Grit Classifier	18 wks	20 wks	Supplier quotation, VoR 28 Oct 09
Membranes	26 wks	52 wks	MWH Process Engineers advice
Bioreactor Aeration	24 wks	26 wks	Supplier quotation, ITT 29 Oct 09
Bioreactor Mixers		14 wks	Estimate
Sludge Thickener		13 wks	Supplier quotation, Alfa Laval 26 Oct 09
Sludge blowers and aeration	24 wks	26 wks	Supplier quotation, ITT 29 Oct 09
Scum Harvester	18 wks	20 wks	Supplier quotation, VoR 28 Oct 09
Sludge Centrifuge	22 wks	24 wks	Supplier quotation, Alfa Laval 26 Oct 09
WAS, Permeate, Sludge feed and Centrifuge Pumps	12 wks	14 wks	Supplier quotation, Liquitech, 30 Oct 09
Odour Plant - Biotrickling Filters		26 wks	MWH Process Engineers advice
Odour plant - Activated Carbon Units		20 wks	MWH Process Engineers advice
Sewage Pumping Stations			
Pumps – SPS1 & SPS2		14 wks	Flygt advice

6.14.5 Costs

Capital and operating cost estimates have been prepared separately to this Concept Design Report by others.

7. Other Design Considerations

7.1 Power Requirements

Power requirements for the project have been estimated and are presented in **Appendix F**. These power requirements are generally based on a 0.95 pf, and include a spare capacity allowance of 30%. Some of the power demands have changed in the Value Management work carried out as part of the development of this concept design report and may be slightly out of date. It is important to validate and confirm power demands at the Detailed Design Stage.

Within the concept design, the electrical design has been limited to the determination of electrical power demands only. The following assumptions have been made:

- The power supply for the WRP, Bulk Water Pump Station, Sewage Pump Stations, and other facilities are Low Voltage Customers. High voltage system and transformers to 415V will be provided and managed separately by the local power authority Country Energy.
- The backup strategy for power failure at any of these facilities will be a low voltage portable generator set connection point. This will enable emergency power to be supplied indefinitely until normal power is restored.
- The preliminary concept design of the WRP Motor Control Centre (MCC) for the WRP is based on the assumption that all Motor Starter Cubicles, Programmable Logic Controller (PLC) Cubicles, Power Factor/Harmonic Correction Cubicles, Uninterruptable Power Supply (UPS) Cubicles, Auxiliary Power Supply Distribution boards, Main Fire Alarm System Cubicle, and all other related cubicles will be located inside the MCC Room.
- The strategy for PLC Input/Output will be based upon centralised and distributed I/O relative to the location of equipment requiring connection to the PLC.

7.1.1 Bulk Water Pump Station

The design is based on a single power supply being provided to site, with a pole mounted transformer to supply the BWPS. In the event of power failure, the emergency response strategy for the BWPS is to provide a portable generator. An emergency generator connection point will be provided to the electrical kiosk.

From Stage 1, operation of the BWPS will be controlled by level in the Interim Potable Reservoir. From Stage 3 onwards, the operation of the BWPS will be controlled by a SCADA system which will monitor the water levels in the reservoirs, and operation of the Recycled Water Pump Station. The SCADA system will be located in the pump station MCC and will be linked into the control centre at the WRP to provide control over the entire potable, and recycled water system.

Table 7-1 Bulk Water Pump Station Power Demand

Stage	Stage 1 1000 EP	Stage 2 3,600 EP	Stage 3 9,400 EP	Stage 4 18,850 EP
Total kVA (@0.95 pf, including 30% spare capacity)	**87**	**87**	**226**	**417**

A telephone connection will need to be provided to the BWPS to provide a telemetry connection to the control centre at the WRP. Water and sewerage services will not be provided.

7.1.2 Interim Reservoir Site

Electrical requirements for the Interim Reservoir Site are shown in Table 7-2. This includes power required to operate the HLZ pumping station, AICVs, submersible mixer units, and the hypochlorite dosing.

Table 7-2 Interim Reservoir Site Power Demands

Power Demands	Interim Potable Reservoir (kVA)	Potable Water Booster PS and Elevated Tank (kVA)	Interim Recycled Water Reservoir (kVA)	Recycled Water Booster PS and Elevated Tank (kVA)
Interim Booster Pump Station Power Demand	NA	13	NA	40
Lighting	1.2	1.2	1.2	1.2
Submersible Mixers	2.1	NA	2.1	NA
Hypochlorite Dosing Pumps	1.1	1.1	1.1	1.1
Total kVA (including 30% spare capacity)	**5**	**16**	**5**	**43**

An existing 11KV power supply is available along Googong Dam Road, and it is proposed to extend this supply to the Interim Reservoir Site. A second power supply is expected to be provided along Old Cooma Road, in the 4th quarter of 2012.

In the case of electrical failure to the booster pumps at both the Interim and Final Reservoir Sites an emergency generator will be sourced to power the pumps. The pumping station MCC, will be provided with an external connection facility for a portable generator. Sufficient reserve storage, equating to an absolute minimum of 30 mins of Peak Hour Demand, will be maintained within the high-level reservoirs, to allow time for a temporary generator to be installed.

In the case of power failure at the BWPS, the service reservoirs will have a minimum of 8 hours Maximum Day Demand reserve storage in the reservoirs, to allow time for an emergency generator to be installed.

For Stage 1, a combined MCC unit will be constructed to serve both the potable and recycled water booster pumps. The MCC will be approximately a 3m (W) x 1m (D) x 2.2m (H) double-sided outdoor kiosk. It will be located adjacent to the pumps on the raised plinth. It will house the telemetry, power distribution boards and PLC control panels.

Telephone line will be connected to the MCC on site as part of the SCADA control and telemetry requirements.

The chlorine dosing plant is regarded as critical equipment; therefore supply of chlorinated water is required at all times, and needs to be considered in selection of the final emergency power supply strategy for the site. Control of the chlorine dosing system will be based on flow-meters and chlorine analysers. The design has assumed, that the site will be remotely monitored, through one control system.

7.1.3 Permanent Reservoir Site

Electrical requirements for the water infrastructure at the Permanent Reservoir Site are shown in Table 7-3. This includes power required to operate the High Level Zone pumping stations, Actuated Inlet Control Valves (AICV), chemical dosing pumps and submersible mixer units.

Table 7-3 Permanent Reservoir Site Power Demands

Power Demands	Ultimate Potable Reservoirs (kVA)	High Level Zone Potable Water Reservoir (kVA)	Ultimate Recycled Water Reservoir (kVA)	High Level Zone Recycled Water Reservoir (kVA)
Pump Station Power Demand	NA	7.3	NA	36.5
Lighting	1.2	1.2	1.2	1.2
AICV Power Demand	5.1	NA	5.1	NA
Submersible Mixers	2.1	NA	2.1	NA
Hypochlorite Dosing Pumps	1.1	1.1	1.1	1.1
Total kVA (including 30% spare capacity)	**9.5**	**10.8**	**10.8**	**48.9**

As for the Interim site, an emergency generator will be sourced to power the reservoir pumping station pumps. The pumping station MCC will be provided with an external connection facility for a portable generator. Sufficient reserve storage, equating to an absolute minimum of 2 hours of Maximum Hour Peak Day Demand will be maintained within the high level reservoirs to allow time for a temporary generator to be installed. It has been assumed that emergency generators are readily available for all installations with a power demand lower than 750kVA as is the case for the reservoir site. This approach should be confirmed during detailed design.

In the case of power failure at the BWPS, the main potable and recycled water service reservoirs will have a minimum of 8 hours Maximum Day Demand reserve storage in the reservoirs to allow time for an emergency generator to be installed.

A telephone line will be connected to the MCC on site as part of the SCADA control and telemetry requirements for the high level pump station.

A combined MCC unit will be constructed to serve both the potable and recycled water pumps. The MCC will be approximately 3m (W) x 2.2m (H) x 1m (D) and will be located adjacent to the pumps on the raised plinth. It will house the telemetry, power distribution boards, and the PLC.

7.1.4 Sewage Pumping Stations

No transformer has been allowed for on either SPS site. It is assumed that the power will be obtained from a pole-top step down transformer, located within close proximity to the SPS compound. The required power supply will be 415V AC. A singled sided outdoor MCC will be located within each SPS compound to contain the telemetry, pump starter, power distribution panel, and pump control panel. An external connection point for an emergency power generator will be provided on the MCC.

A SCADA system will also be provided at each MCC and will be linked into the WRP control centre via telemetry. This will allow the operators to control the pumps from a remote location. This will need to be confirmed during detailed design once the ownership and operational details are confirmed, as this will dictate the precise arrangement. Concept estimates of the power requirements for the SPS and MCC dimensions are provided in Table 7-4.

Table 7-4 Sewage Pumping Station Power Demands

Power Demands	Sewage Pumping Station 1	Sewage Pumping Station 2
Motor Control Centre Size	Approx. 3.0m(W) x 2.2m(H) x 0.5m(D)	Approx. 3.0m(W) x 2.2m(H) x 0.5m(D)
Pump Station Power Demand (kVA)	49.8	82.4
Chemical Dosing Unit Power Demand (kVA)	2.1	2.1
Lighting Demand (kVA)	1.2	1.2
Total (kVA) (including 30% spare capacity)	**69**	**111.5**

7.1.5 Water Recycling Plant

No transformer has been allowed for on the WRP site. It is assumed that the power will be obtained from a pole-top step down transformer, and/or a pad mounted transformer for Stage 1 and Stage 3-4 respectively, depending on Country Energy requirements. The transformer will be located outside the site fence to allow easy access by Country Energy, but within close proximity to the WRP compound. The required power supply will be 415V AC.

In case or power failure, it is recommended that a standby generator point is provided and a generator source is located.

For Stage 1, a singled sided outdoor MCC will be located near to the scaled-down bioreactor to contain the telemetry, starters, power distribution panel, and control panels. An external connection point for an emergency power generator will be provided on the MCC.

For Stage 2, a MCC room will be constructed as part of the combined Administration/MCC building to house the electrical equipment, and emergency generator connection point.

A suitably sized Power Factor correction /Harmonic Filter Unit will be installed for the power quality improvement. The power requirements of the WRP are summarised as follows:

Table 7-5 Water Recycling Plant Power Demands

Item	Stage 1/2	Stage 2b 4700 EP	Stage 4 18,850 EP
Motor Control Centre Size	Approx. 4.0m(W) x 2.2m(H) x 0.5m(D)	MCC Room approximately 15m x 8m	MCC Room approximately 15m x 8m
Total (kVA) (including 30% spare capacity)	To be determined in design	(1171kVA/ 1085kW @ 0.93 pf (lag)	(1611kVA/1494kW @ 0.93 pf(lag)

7.2 Testing & Commissioning

7.2.1 The Testing and Commissioning Process

The following discussion should be used as a guide to the possible commissioning procedure to be pursued at Googong. The actual procedure followed will depend on many issues such as:

- Operator preference
- Regulators requirements
- Contractual conditions of the party responsible for commissioning
- Schedule for delivery of various assets

In general, commissioning activities will commence with the preparation of an agreed commissioning plan followed by the development of the commissioning documentation nominated in the commissioning plan. Commissioning is likely to pass through the following phases: pre-commissioning followed by dry commissioning followed by wet commissioning. The latter stage may be followed with a period of system proving or testing.

7.2.1.1 Pre and Dry Commissioning

Pre-commissioning is likely to occur at an individual assets level, such as, a valve or a pump. This will include such activities as:

- Confirming that the equipment has been installed in accordance with the drawings and specifications (Has it been anchored correctly? Is it in its correct position? Is it the correct piece of equipment? etc),
- Pressure testing of all pipe work in accordance with Australian Standards (this may include x-ray and ultrasonic testing of welds),
- Point to point testing of all power supplies and instrumentation connections,
- Non-electric rotation testing of equipment to confirm that equipment can rotate freely as designed,
- Calibration of electrical instrumentation.
- All pipe work flushed and cleaned
- Lifting equipment to be pre-commissioned (primarily load and pressure tested and certified)
- Factory acceptance testing of the switch boards before delivery to site,
- Factory acceptance testing of the pumping units before delivered to site
- Communications testing of all telecommunications links

Once a piece of equipment has been successfully pre-commissioned it may then proceed into dry commissioning (if required by the commissioning plan) provided it does not need to be dry commissioned with an addition piece of equipment; in which case both pieces of equipment would need to be dry commissioned before each proceeding on to dry commissioning.

7.2.1.2 Dry commissioning

Dry commissioning would include activities such as:

- Rotation testing of equipment (does the equipment rotate in the correct direction when an electric current is applied)

- Confirming that instrumentation return the correct signal to the PLC when an induced signal is returned.

Once a piece of equipment has been successfully pre and dry commissioned it may then proceed into wet commissioning (if required) provided it does not need to be wet commissioned with an addition piece of equipment; in which case both pieces of equipment would need to be dry commissioned before each proceeding on to wet commissioning.

7.2.1.3 Wet Commissioning

Wet commissioning typically includes the run testing of an entire operating system or multiple components at the same time to prove that they all operate in-concert with one another as designed. The following activities are likely to be required:

- Site acceptance testing of the switch boards
- Site acceptance testing of mechanical equipment
- Testing of control logic in separate sub-systems
- Site acceptance testing of the PLC logic and SCADA system
- Water hammer testing

During this phase of commissioning the operating system will be operated and tested across the full range of functionality envisaged in the design.

7.2.1.4 Process Commissioning

The WRP will require process commissioning once sewage is available. This requires that all equipment is tested for performance and that effluent quality is proven to achieve the recycled water guidelines.

This proving period can be in excess of 3 months and will be determined in the risk assessment which should be completed in-line with the Australian Recycled Water Guidelines during ongoing design activities.

7.2.2 Potable Water System

It is assumed that commissioning of the potable water system at Googong will be broken down into the commissioning of the following sub-systems:

- Bulk water pumping station and delivery main
- Potable water reservoir;
- Chemical dosing system;
- Potable water outlet main; and
- Potable water distribution.

This section discusses specific requirements and timelines of the commissioning process of the potable water system and involves pre-commissioning, dry commissioning, wet commissioning and acceptance testing.

7.2.2.1 Pre-commissioning

Pre-commissioning and dry commissioning will require the:

- Pumping station pipework and valving to be pre-commissioned (primarily anchorage checking and pressure testing);

- All pipe work flushed and cleaned;
- Pumping station lifting equipment to be pre-commissioned (primarily load and pressure tested and certified);
- Delivery mains to be pre-commissioned (primarily pressure tested); and
- Instrumentation will need to be pre-commissioned (primarily instrument calibration and point to point testing).

Pre-commissioning and dry-commissioning activities are likely to take place in parallel and may require around 3-4 months to complete. However, this work is likely to be completed in parallel with the latter stages of construction, and may proceed as part of the mechanical and electrical installation work. Pre and dry commissioning is likely to extend beyond the completion of electrical installation by 2-3 months.

The pre-commissioning of the potable water network will be completed progressively throughout the network, from upstream locations to downstream, and will not commence until water can be released from the potable water reservoir. Water used in the cleaning and pressure testing of the network, may be discharged to the sewerage system for use in the commissioning of the sewage pumping stations. It may also be used to irrigate playing fields and landscaped areas of the development; however, it is likely that a large amount of water will need to be discharged to the stormwater system.

7.2.2.2 Wet Commissioning

During wet commissioning, the pumping station will be operated and tested across the full range of functionality, envisaged in the design. This may require the direction of substantial amounts of water to the potable water reservoir. Consequently, the potable reservoir will need to have been sufficiently commissioned, to accept this water, without compromising worker safety, structural integrity of the reservoir and its interconnecting pipework, and the scheduling of ongoing construction work at the reservoir site.

The coordination of the wet commissioning of the bulk water pumping station and delivery main, along with the construction and commissioning of the reservoirs, will need to be carefully planned, to avoid undue wastage of potable water. It is anticipated, the potable water reservoir will need to undergo sufficiently pre-commissioning to allow it to accept water from the bulk water pumping station and for this water to be used in the hydro-static testing of the reservoir and its outlet mains.

As part of the site acceptance testing work for the bulk water pumping station, the automatic inlet control valves at the reservoirs, will be tested to confirm that they can be operated remotely. When this is confirmed, the pumps will be tested to confirm that water can be directed to the reservoirs. The pumps will then be gradually tested over their design range and the PLC logic tested. This will lead to the reservoir being gradually filled with water, such that the reservoir can undergo hydrostatic testing.

Following hydrostatic testing of the reservoirs and full commissioning of the bulk water pumping station, the reservoirs will be emptied then cleaned and then filled with super-chlorinated water for 24 hours. They will then be emptied of super-chlorinated water and recharged (possibly being passed down to the potable water network for super-chlorination of the potable water network). It is anticipated that this activity will take approximately a month to execute.

Once emptied, the reservoirs will again be refilled and the chlorine residual in these tanks maintained at a level compliant with Australian Drinking Water Guidelines. The water in the reservoirs will then be tested for compliance with the Australian Drinking Water Guidelines. Once testing is successfully completed, water in the reservoirs may be released into the potable water network. It is anticipated that this activity may take approximately one month to successfully execute.

During this phase, the chlorine dosing system will be wet commissioned, using potable water, until the full range of PLC logic has been tested for this unit.

Once the chlorine dosing system logic is tested, sodium hypochlorite will be used in the commissioning of these units. Site acceptance testing, complete with a focus on tuning of the PID loops for the dosing units, to ensure that the water in the reservoirs and reservoir outlet mains can be reliability maintained at the required level of chlorine residuals, will be undertaken.

Wet commissioning of the potable water network will next require that all pipework is recharged with super-chlorinated potable water for a period of 24 hours. Next, the super-chlorinate water will be flushed out of the network and de-chlorinated before being discharged on to playing fields, and landscaped areas, then into the stormwater system. This phase of commissioning is likely to require around a month to organise and execute.

7.2.2.3 Acceptance Testing

Finally, the network will be recharged with water, originating from the potable water reservoirs, which is compliant with Australian Drinking Water Quality Guidelines. The water in the network will then undergo acceptance testing, to confirm that it complies with the Australian Drinking Water Guidelines. This phase of commissioning is likely to require approximately two months to complete and may require parts of the network to be re-flushed, super-chlorinated again and then re-tested, before the first customer may be permitted to be connected to the network.

7.2.3 Recycled Water System

It is assumed that commissioning of the recycled water system at Googong will be broken down into the commissioning of the following sub-systems:

- Recycled water pumping station and delivery main
- Recycled water reservoirs
- Chemical dosing system
- Recycled water outlet main
- Recycled water distribution

7.2.3.1 Pre-commissioning

Pre-commissioning and dry commissioning will require the:

- Pumping station pipework and valving to be pre-commissioned (primarily anchorage checking and pressure testing)
- All pipe work flushed and cleaned
- Pumping station lifting equipment to be pre-commissioned (primarily load and pressure tested and certified)
- Delivery mains to be pre-commissioned (primarily pressure tested)
- Instrumentation will need to be pre-commissioned (primarily instrument calibration and point to point testing),

Pre and dry commissioning activities are likely to take place in parallel and may require around 3-4mths to complete. However, this work is likely to be completed in parallel with the latter stages of construction, and may proceed as part of the mechanical and electrical installation work. Pre and dry commissioning is likely to extend beyond the completion of electrical installation by 2-3mths.

The recycled water network will be completed using potable water from the potable water system and will therefore required substantial completion of the commissioning of the bulk potable water system before the recycled network can be commissioned.

The Recycled Water pumping station will also need to have a power source at the WRP prior to commencement of the commissioning period.

The pre-commissioning of the recycled water network will be completed progressively throughout the network from upstream locations to downstream and will not commence until water can be released from the recycled water reservoir. Water used in the cleaning and pressure testing of the network may be discharge to the sewerage system for use in the commissioning of the sewage pumping stations. It may also be used to irrigate playing fields and landscaped areas of the development; however, it is likely that a large amount of water need to be discharged to the stormwater system.

7.2.3.2 Wet commissioning

During wet commissioning the recycled water pumping station will need to be charged and be operated and tested across the full range of functionality. This may require the direction of substantial amount of water to the recycled water reservoir. Consequently, the reservoir will need to have been sufficiently commissioned to accept this water without compromising worker safety, structural integrity of the reservoir and its interconnecting pipework, and the scheduling of ongoing construction work at the reservoir site.

The coordination of the wet commissioning of the recycled water pumping station and delivery main along with the construction and commissioning of the reservoirs will need to be carefully planned to avoid undue wastage of potable water. It is anticipated that the recycled water reservoir will need to undergo sufficiently pre-commissioning to allow it to accept water from the recycled water pumping station and for this water to be used in the hydro-static testing of the reservoir and its outlet mains.

Following hydrostatic testing of the reservoirs and full commissioning of the recycled water pumping station the reservoirs will be emptied then cleaned and then filled with super-chlorinated water for 24 hours. They will then be emptied of super-chlorinated water (possibly being passed down to the recycled water network for super-chlorination of the recycled water network) and recharged with potable water. It is anticipated that this activity will take approximately a month to execute.

The pumping station and pipeline will be commissioned in-line with the requirements above.

During this phase the chlorine dosing system will be wet commissioned using potable water until the full range of PLC logic has been tested for this unit.

Once the chlorine dosing system logic is tested, sodium hypochlorite will be used in the commissioning of these units and site acceptance testing complete with a focus on tuning of the PID loops for the dosing units to ensure that the water in the reservoirs and reservoir outlet mains can be reliability maintain at the required level of chlorine residuals.

Wet commissioning of the recycled water network will next require that all pipework is recharged with super-chlorinated potable water for a period of 24 hours. Next, the super-chlorinate water will be flushed out of the network and dechlorinated before being discharged on to playing fields, landscaped areas or into the stormwater system.

This phase of commissioning is likely to require approximately two months to complete and may require parts of the network to be re-flushed, super-chlorinated again and re-tested before the first customer may be permitted to connected to the network.

Upon successful wet commissioning of the recycled water system there will be a period of validation of the process, potable water will be used in the recycled water system until the Department of Health provide consent to allow the system to be charged with recycled water.

This will be done to protect public health until such time as the Department of Health is satisfied that the recycled water system is being operated as designed, that the public is sufficiently well educated in the use of recycled water, that operators have been able to demonstrate familiarity and compliance with operating procedures, and that the WRP has produced consistent recycled water quality. Until this time recycled water will be discharged into the storm water system for discharge. This time will be developed in a risk assessment in detailed design in accordance with the Australian Recycled Water Guidelines.

7.2.4 Sewage Transfer System

A detailed SPS and transfer main commissioning subplan and program will be required during detailed design as part of the overall commissioning plan. Commissioning of the SPSs and rising mains, will typically be undertaken in a two-stage process.

7.2.4.1 Pre and Dry Commissioning

Pre and Dry Commissioning checks and tests will be carried out as necessary, to prove that all structures, pipes, equipment and associated components, are completely installed and operational, to their respective manufacturer's recommendations and in accordance with the asset functional description. The tests and checks will include, but not limited to:

- Pressure testing of pipework and valves;
- Check completeness of installation;
- Alignment checks and final adjustments;
- Provision of sufficient equipment support;
- Ensuring all equipment is correctly lubricated and lubrication reservoirs are charged with adequate quantities of a suitable lubricant;
- Check conformance of equipment to requirements of the Contract;
- Check the direction of rotation and performance of electric motors;
- Testing of feedback, control and overload equipment including safety checks;
- Correct installation of guards, trip wires and other personnel safety equipment;
- Operation of all switches, interlocks and alarms;
- Electrical termination checks of all I/O and cabling;
- Checking all electrical and instrument wiring and connections to all termination points;
- Dry-run functional tests;
- Calibration and testing of all instruments; and
- All other checks as necessary and as specified in manufacturers' instructions.

Tests will be carried out on site, to ascertain that all plant and equipment perform satisfactorily under minimum, normal and maximum operational conditions. Normal operational conditions will include stoppages due to simulated power failure.

7.2.4.2 Wet Commissioning

Off-line wet commissioning will include running plant on water, prior to the introduction of sewage. It is anticipated that the entire plant will be operated in automatic mode by pumping (and recycling) water around the facility or up the transfer main. The purpose of wet commissioning is to ensure that all subsystems interact with each other in accordance with design requirements.

Site Acceptance Testing of the WRP's SCADA system will test all interactions between subsystems during this period.

Wet commissioning off-line, will be successfully completed before sewage can be introduced into any WRP, SPS or transfer main. As a minimum wet commissioning off-line will include the following:

- At SPS's the completion of all testing identified in the Pumping Station Acceptance Tests;
- Check conformance of equipment to requirements including verifying guaranteed performance;
- Testing of feedback, control and overload equipment including safety checks;
- 'Running in' of new equipment;
- Simulated fault condition tests;
- Final calibration tests;
- Hardwired logic checks; and
- Briefing of the handover personnel to allow commissioning to commence safely and effectively.

The SPS and rising mains are assumed to enter a performance proving period on hand over and introduction of raw sewage.

7.2.5 Water Recycling Plant

The following describes specific requirements of the WRP process only.

7.2.5.1 Dry commissioning

All mechanical equipment should be dry commissioned.

All tanks need to be cleaned out following hydrostatic testing and prior to installation of equipment. In particular this is important due to the membrane installation.

Pipework should also be cleaned and inspected. This could be done during sequence testing by placing strainers in the recycle pipe work to capture any left-over material.

The membrane manufacturer should provide a procedure for installation of the membranes and be on-site to witness the installation. It is a requirement that the membranes are submerged therefore potable water should be available when the membranes are installed.

7.2.5.2 Wet Commissioning

The plant should be wet commissioned using potable water before sewage is introduced into the system. Wet commissioning will provide a method of testing mechanical kit and sequence testing the system through the automatic control system.

Due to the requirements above it is suggested that this is separate to the water used within the sewage system to prevent contamination.

The plant should have each of the sub systems tested before full system testing is done.

- Inlet Works
- Bioreactors, excluding membranes
- Permeate pumps and tertiary treatment
- Solids processing

It should be noted that the timing of installation and wet commissioning is important due to warranty issues. As the plant will not be process commissioned until sometime after the plant is commissioned, a significant portion of the warranty period may have expired when sewage is first treated.

Waste water will need to either be removed by truck movement to another treatment plant or disposed of to the water course through special agreement with DECCW.

7.2.5.3 Process Commissioning and Start-up

Process commissioning describes the commissioning that takes place once sewage is introduced to the tank. This represents the period where the plant is brought online and its operating conditions are set. It is recommended that the tanks be seeded with mixed liquor from another plant for this purpose in order to speed up the process. Mixed liquor for seeding may be taken from another near-by activated sludge process following the establishment of a supply agreement with the operator.

The plant provides biological treatment and therefore needs a minimum load to operate correctly. If this minimum load is not provided then the biomass may become unstable and issues such as foam may be apparent. There is also a chance if the loads are too low that the biomass may become stressed and create extracellular polymeric substances (EPS) which may foul the biomass although the risk of this is low in purely domestic sewage.

With the scaled down bioreactor proposed for Stage 1, preliminary calculations suggest that a minimum load will eventuate from 150 EP. Before this load is achieved sewage should be tankered to another treatment plant for treatment from SPS1.Once the load is achieved within network then the plant can be process commissioned. This will involve growth of the biomass and optimisation of the process to achieve the desired discharge quality. The MLSS concentration in the bioreactor must be established during process commissioning prior to a stabilisation period. This requires operating the plant with no waste sludge stream, allowing the build-up of growing bacteria. Once sufficient sludge has been produced the plant may be set to run at the design waste rates and the stabilisation period may begin.

The stabilisation period will be required for approximately 3x SRT, or approximately 60 days. This allows for the process upsets caused by plant start up to stabilise and the operation to reach steady-state. Following stabilisation, the process validation may be completed prior to commencing normal operation.

It is recommended that a temporary licence for discharge is sought on start-up of the plant such that effluent can be discharged to the watercourse from plant start-up. This is a normal procedure to allow the plant to be optimised to achieve the licence conditions and should be included in the discussions with DECCW.

7.2.5.4 Process Validation

Following commissioning there will be a period for validation of the process for recycled water. This will be developed in the risk assessment which has not yet been completed, however experience of this is that this period is a minimum of three months. The discharge during this time should be discussed with DECCW. The risk assessment will be completed in parallel with ongoing design activities.

During this time water will be discharged to the environment via the stormwater system. The system will be separated from the recycled reservoir during this time to prevent cross contamination. Once the Process Validation period is complete then the system will be connected and commissioning will take place of the recycled water pipeline and pumping station.

The water from the commissioning will be disposed of from the system and the connection will not be made until the system has been super-chlorinated and the water is disposed of.

7.3 Operation and Maintenance

To facilitate routine maintenance on the reservoirs, separate inlets from the Potable Supply main should be constructed to the two (ultimate) Potable Reservoirs. Each reservoir can then be filled and emptied in isolation, whilst the other is taken offline for maintenance.

There will be times when the booster pump station or high level zone pump stations need to be taken off line for maintenance. For both the Interim and the Permanent sites, it is possible that the potable water reservoir can be used to meet the recycled water demand in the High Level Zone. This will require a cross connection to be made from the PWHL reservoir to the high level recycled water network. Appropriate safeguards, including double backflow prevention valves must be provided. Lower operating pressures and more frequent pump operating is likely to result during the maintenance period.

To take the elevated potable water reservoir PWHL off-line, potable water will need to be pumped from the low level reservoirs to the high level potable water network, bypassing the PWHL. Temporary pumps may need to be brought in to provide suitable flow rates and pressure to the high level zone properties.

General routine maintenance on the water network is expected to consist of the following items summarised in Table 7-6. Similarly, general routine maintenance of the Sewage Pump Stations and pipelines is summarised in Table 7-7.

Table 7-6 Reservoir Operations and Maintenance Requirements

Component	General O&M Requirements
Bulk Water Pump Station, Booster Pump Stations	• Pump inspections for vibration and coupling security would be undertaken on a monthly or bi-monthly basis, or as specified by the pump manufacturer; • Yearly inspection of the pumps for oil, wearing and general condition, would be required; • Annual internal condition inspection of the pumping station structure, including the roof; • External visual condition inspections of the reservoirs would be undertaken on a monthly basis; • Manual opening and closing check of all valves would be undertaken on a quarterly or four-monthly basis; • External paintwork to be inspected every 2 years, with repainting likely every 10 to 15 years, depending on the chosen finish; and • Periodical maintenance of mechanical and electrical items would be as per manufacturers' specification.
Bulk water delivery mains	• Operational staff will need to periodically remove silt that has built up in the low points of the pipeline. This will be done by sectioning off the pipeline and flushing the water out through the scour pits or back to the pump station for tankering away. • Small vehicle access, along the pipe route will be required to inspect all accessible assets such as air valves, scour valves and any above ground infrastructure, approximately every 6 to 12 months. The timing is subject to operational needs. • During the first year of operation, it is recommended that maintenance crews drive along the whole route on a monthly basis.
Potable water service reservoirs	To facilitate routine maintenance on the reservoirs, separate inlets from the BWPS delivery main should be constructed to reservoirs PW1 and PW2. Each reservoir can then be filled and emptied in isolation, whilst the other is taken offline for maintenance. To take the elevated potable water reservoir PWHL off-line, potable water will need to be pumped from the low-level reservoirs to the high-level potable water network, bypassing the PWHL. Temporary pumps may need to be brought, in to provide suitable flow rates and pressure, to the high level zone properties. However, lower operating pressures and more frequent pump operations, is likely to result during the maintenance period. General routine maintenance is expected to consist of the following: • An annual internal condition inspection of each reservoir including the roof structure by diver. A vacuum pipe would be used, to remove sediment from the reservoirs to reduce unnecessary discharge of water to the environment; • External visual condition inspections of the reservoirs would be undertaken on a monthly basis; • A manual opening and closing check of all valves, actuated or otherwise, greater than 100mm diameter would be undertaken on a quarterly or four-monthly basis; • External paintwork to be inspected every 2 years, with repainting likely every 10 to 15 years, depending on the chosen finish; and • An external roof inspection would be undertaken annually. • Periodical maintenance of mechanical items would be as per manufacturers' specification.

Component	General O&M Requirements
Secondary water conditioning/chemical dosing	There is sufficient redundancy in the chemical dosing system will allow mechanical items to be taken off-line for maintenance. The items assumed to need routine maintenance are: • Chemical dosing pumps • Chemical Dosing System Instruments This would be completed in-line with selected chemical dosing system manufacturer's requirements. There will be a requirement for security access or operational presence when the chemical tanks are being filled. Any drain valves in the tanker loading area would need to be checked that they are closed during this period to ensure there is no contamination with chemical to the storm water system

Table 7-7 Sewage Pump Station and Pipeline O&M Requirements

Component	General O&M Requirements
Sewage Pump Stations	• General pump inspections for vibration and coupling security would be undertaken on a monthly or bi-monthly basis, or as specified by the pump manufacturer. • Yearly inspection of the pumps for oil, wearing and general condition would be required. Pumps would be lifted out and hosed down at the SPS. Oil would be changed. A visual inspection of the wet well and emergency storage area would be undertaken at least annually. Washdown of the emergency storage area would be done at the same time. These works would need to be undertaken during periods of low flow, and the incoming sewage would be pumped out from the inlet manhole to a tanker and taken away for disposal. • An inspection of the wet well and emergency storage should generally be undertaken on a 6-monthly cycle, during which the emergency storage area will be hosed down and the wet well pumped out. A visual structural condition assessment of the SPS will be undertaken.
Sewage Rising Mains	• Operations staff would periodically remove silt that has built up in the low points of the pipeline. This would be done by draining water through the scour valves sited at each low point. It is anticipated that this operation would be undertaken during suitable weather periods, when the easement is dry. The scour valve pit would ideally be provided with a drainage pipe discharging into a nearby gravity sewer. Failing this, a suitably sized tanker with suction hose, would need to traverse the alignment from the nearest access point, to enable the scoured water to be collected from the scour pit. • Small vehicle access along the easement would be required to inspect all accessible assets such as air valves, scour valves and any above ground infrastructure, approximately every 6 to 12 months. The timing is negotiable subject to operational needs. • During the first years of operation, maintenance crew may drive along the whole route on a monthly basis. This would be in accordance with a commissioning / early operational plan that would be developed. • Larger vehicle access may be required along the route if any significant repair work is necessary. This would be a rare event

MWH

BUILDING A BETTER WORLD

Googong Integrated Water Cycle

Water and Wastewater Concept Design Volume 2 - Appendices

Prepared for CIC Australia

Revision 9

11 October 2010

CIC AUSTRALIA

Communities in the making

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APPENDIX A.
Googong Design Assumptions for Potable and Recycled Water Systems

GOOGONG DESIGN ASSUMPTIONS FOR POTABLE AND RECYCLED WATER SYSTEMS

PREPARED FOR CANBERRA INVESTMENT CORPORATION AND QUEANBEYAN CITY COUNCIL

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4.0	13 Jul 10	Peak day ratio revised in line with value mgmt.	Adam Joyner	Paul Phillips	Paul Phillips

GOOGONG DESIGN ASSUMPTIONS FOR POTABLE AND RECYCLED WATER SYSTEMS

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APPENDICES

1. INTRODUCTION

1.1 BACKGROUND

During the first phase of the Googong Integrated Water Cycle (IWC) investigation, preliminary assessment of water infrastructure requirements was made. As part of the Googong Phase 2 Water Cycle Work project MWH was engaged by CIC to review base water cycle assumptions with Queanbeyan City Council (QCC) and where appropriate, incorporate QCC data as opposed to Canberra data. Revisions 1.0 and 2.0 of this report reflected this review. Revision 2 (Jan 2008) received QCC's in-principle acceptance of the design criteria for the Googong development (QCC, 2008).

Subsequently, during the IWC concept design development, lot yields, lot layout and irrigation requirements have been refined. Additional water balance studies indicated higher total dissolved solid levels than originally anticipated resulting in the change for cooling water supply from non-potable supply to potable supply. Also, value management assessment of the concept design identified opportunities for cost reductions through system layout and design criteria modifications. These changes in the understanding of the IWC have resulted in refinement of the proposed potable and recycle water system design rates.

This report outlines the revised water cycle assumptions and provides the proposed design demand unit rates for potable and non-potable water for each demand type in the Googong development.

The layout of this revision of the report has not significantly changed from Version 2, however, to enable ready identification of report changes, changes in methodology and criteria are highlighted with a bar in the margin.

1.2 THE AVAILABILITY OF QUEANBEYAN DATA

As part of the process of developing design standards, water demand information was requested from QCC for use as the basis of the design standards. This information was:

- Daily bulk water production records; and
- Quarterly customer water consumption records.

The ACT daily bulk water production records proved useful in understanding daily demand variation in the region.

Unfortunately the customer consumption records were only available from more recent periods where water restrictions were in force and were deemed unsuitable for use as a basis for the development of design standards.

2. APPROACH

Traditional water supply design criteria exist for separated urban water, wastewater and stormwater systems, often without adequate consideration of demand management approaches. The IWC management approach proposed for the Googong development stretch the applicability of traditional design figures. The Water Services Association of Australia (WSAA) recognises that the development of appropriate standards maybe required by water agencies and provides initial guidance for dual water supply design rates (WSAA, 2002a and 2002b).

This section of the report outlines the approach used to develop the appropriate design standards for residential and non-residential uses at Googong.

2.1 RESIDENTIAL WATER DEMANDS

Residential demands will form the majority of water needs within the Googong development. Assessment of anticipated future residential demands, requires consideration of the conservation approaches and non-potable supply proposed for the Googong development. To assist with the assessment, a number of water industry studies are referenced.

A flow-chart summarising the methodology for the development of residential design standards is provided in Appendix A.

The Yarra Valley end-use studies (Yarra Valley Water 2004 and 2005) provide detailed information on internal (indoor) water usage and have been used in this study as the basis to determine the expected level of internal water use per capita.

The volumes of potable and recycled water use were estimated based on an end-use fixture and appliance allocation basis. Fixture allocations were based on a preferred IWC management strategy determined through stakeholder consultation in 2004 (MWH, 2004) and refined with development of the proposed concept (MWH, 2009 & 2010).

A study by ActewAGL titled *Household Consumption Trends – A Review of the Effect of Pressure, Elevation, Property Size and Income on Household Water Consumption* draft report by ActewAGL (2003) formed the basis for estimation of residential external demands in the January 2008 (Rev 2.0) version of this report. It now provides useful information on residential water demands in the ACT particularly related to lot size, prior to the introduction of the recent water restrictions.

Residential external water estimates have been based on:

- Irrigation modelling for each residential lot type with assumed irrigation type and areas in line with expectations for the development (Agsol, 2009);
- An allowance for cooling systems based on the Yarra Valley end use study (YVW, 2004); and,
- An assumed allowance for other external uses.

To the combined internal and external demands, an allowance for system leakage (assumed to be 10% of average day demand) and a factor of safety (assumed to be 20% for the design of long term assets and 40% for the assets "rolled-out" in the first 5yrs) were added to form the design rates. These allowances were determined in consultation with Queanbeyan City Council (QCC).

2.2 NON-RESIDENTIAL DEMANDS

In this study, demand based non-residential average and peak day usage rates have been developed through modification of the design standards published in WSA 03-2002, Table 2.1 (refer to Appendix B). In doing so, it is assumed that the design values nominated by WSAA make due allowance for leakage and embody an adequate factor of safety for design purposes.

A flow-chart summarising the methodology for the development of non-residential design standards is provided in Appendix A.

The WSA 03-2002 design values are peak hour figures. However, the end-use modelling (Demand Side Management Decision Support System - DSM DSS)[1] used to assess the system water balance demands, requires average consumption data with separate allowances made for peaking factors, system leakage and factor of safety. Accordingly, the WSA 03-2002 non-residential design demands for Canberra, were modified before putting them into the DSM DSS model. Adjustment to the WSA demands included:

1. Reducing the WSA 03 - 2002 design values back to average daily demands using the peak hour to peak day factor nominated in WSA 03-2002 and the peak day to average day ratio estimated by the DSM DSS model.

2. Reducing the average day demands by 20% for the assumed long term factor of safety.

3. Reducing the average day demands to allow for 10% assumed level of total system leakage.

There is limited information available for assessing end-uses in non-residential sectors.

For this study, the non-residential internal demands were estimated using an assumed percentage breakdown adopted from an American Waterworks Association Research Foundation non-residential end-use study (AWWARF 2000).

Non-residential average external water estimates have been based on:

- Irrigation modelling for each non-residential lot type with assumed irrigation type and areas in line with expectations for the development (Agsol, 2009)

- An assumed allowance for cooling systems.

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[1] The DSM DSS is an end use model, used for the assessment of water supply demands and demand management options. It performs the water balance calculation on best estimates of where water is used on an average basis. It also includes economic and energy considerations for demand management option comparison.

Combined internal and external average demands were returned to peak day rates using the reverse of the above described process for system leakage and factor of safety allowances. If this approach were not undertaken, there would in effect be a "double-dipping" of the above leakage allowance and factor of safety.

Significant assumptions have been necessary to develop the average non-residential demand rates used in the water balance assessment. In order to propose design rates for the Googong development, the combined demand based potable and recycled water peak day rates were compared to the WSA 03 design rates in each non-residential category:

1. Schools – The combined Googong potable and recycled peak design rate has been set at the WSA 03 design rate.
2. Open Space – the Googong demand based design rates, on a net area basis, are proposed.
3. Commercial - The combined Googong potable and recycled peak design rates have been set at the WSA 03 design rate.

Section 3 outlines the development of average residential and non-residential water demands. Section 4 outlines the development of potable and recycled water system design criteria using the water demands from Section 3.

3. WATER DEMAND ESTIMATES

This section of the report sets out the calculations used to develop average water demand estimates and end-usage breakdown.

3.1 RESIDENTIAL

3.1.1 INTERNAL

Estimation of end-use demands relies on interpretation of available monitoring and stock data. One of the more comprehensive end-use demand studies undertaken in recent years is the *Yarra Valley Water 2004 Residential End-Use Measurement Study* (YVW, 2005). This study combined with appliance market penetration and appliance usage frequency information gathered by MWH over the years was used to derive typical end use demands noted as "Yarra Valley" values in Table 3-1 below.

Table 3-1: Adaption of Yarra Valley Water End Use Data for Use in Googong

Type of Consumption	Assumed Number of Uses per User per Day	Yarra Valley Consumption per Capita (L/d)	Googong Consumption per Capita (L/d)	Reduction from Yarra Valley	Key Assumptions
Toilets	4.50	28.7	21.3	-25.9%	100% of 6/3 Dual Flush toilets
Baths	N/A	3.2	3.2	0.0%	
Showers	0.90	61.7	52.4	-15.1%	50% customers retain efficient showerheads
Taps/Sinks	N/A	27.0	24.3	-10.0%	10% reduction in use
Dishwashers	N/A	2.7	2.7	0.0%	
Washing Machines	0.33	43.1	38.3	-11.2%	50% of customers install efficient machines
Int. Leakage	N/A	11.9	11.9	0.0%	75% of total measured leakage is internal
Total Internal		**178.3**	**154.1**	**-13.6%**	

The resultant total internal per capita water usage is 178.3 L/c/d, which is in line with Yarra Valley Water's estimate for its entire residential customer base of 178 L/c/d (p3, YVW, 2005).

Table 3-1 also outlines the modifications made to the typical end-use demand assumptions for Googong. The key differences between the Yarra Valley customers and those expected in Googong were assumed to be:

- Toilets – in Googong 100% of customers are assumed to have 6/3 litre dual flush toilets, whereas in Yarra Valley only 78% of customers had dual flush toilets and a high proportion of those dual flush toilets (46%) were of the 9/4.5 litre dual flush variety.

- Showers – in Googong it has been assumed that 50% of customers will retain water efficient showerheads (9L/minute) whereas in Yarra Valley, only 32% of customers had low flow varieties;

- Washing Machines – in Googong 50% of customers are assumed to have efficient front loading washing machines whereas in Yarra Valley only 21% had efficient machines.

The resulting end-use model market penetrations are tabled below.

Table 3-2: Assumed Market Penetration - Toilets[2]

Type	Volume per Use (Litres)	% of Total - Yarra	% of Total - Googong
4.5/3 Dual Flush	4.1	0.0%	0.0%
6/3 Dual Flush	4.7	39.9%	100.0%
9/4.5 Dual Flush	6.7	38.2%	0.0%
High Flush	8.9	21.9%	0.0%

Table 3-3: Assumed Market Penetration - Showers

Type	Volume per Use (Litres)	% of Total - Yarra	% of Total - Googong
Water Miser	35.7	14.0%	0.0%
Low Flow	47.6	9.0%	50.0%
Medium Flow	58.2	9.0%	25.0%
High Flow	79.4	68.0%	25.0%

Table 3-4: Assumed Market Penetration – Washing Machines

Type	Volume per Use (Litres)	% of Total - Yarra	% of Total - Googong
Efficient Front Loader	80.0	5.0%	20.0%
Front Loader	100.0	16.0%	30.0%
Efficient Top Loader	130.0	39.5%	25.0%
Inefficient Top Loader	150.0	39.5%	25.0%

For the modified market penetration end-uses list above, the typical number of appliance uses per day have been estimated based on a combination of the Yarra Valley end-use study (YVW, 2005) in order to determine the impact on end-use demands.

3.1.2 EXTERNAL

The following chart (Figure 1) demonstrates the relationship between lot size and annual water consumption, pre-water restrictions, identified in the *Household Consumption Trends – A Review of the Effect of Pressure, Elevation, Property Size and Income on Household Water Consumption* draft report (ACTEW 2003).

[2] The DSM DSS combines the 9/4.5 and 11/6 dual flush stock.

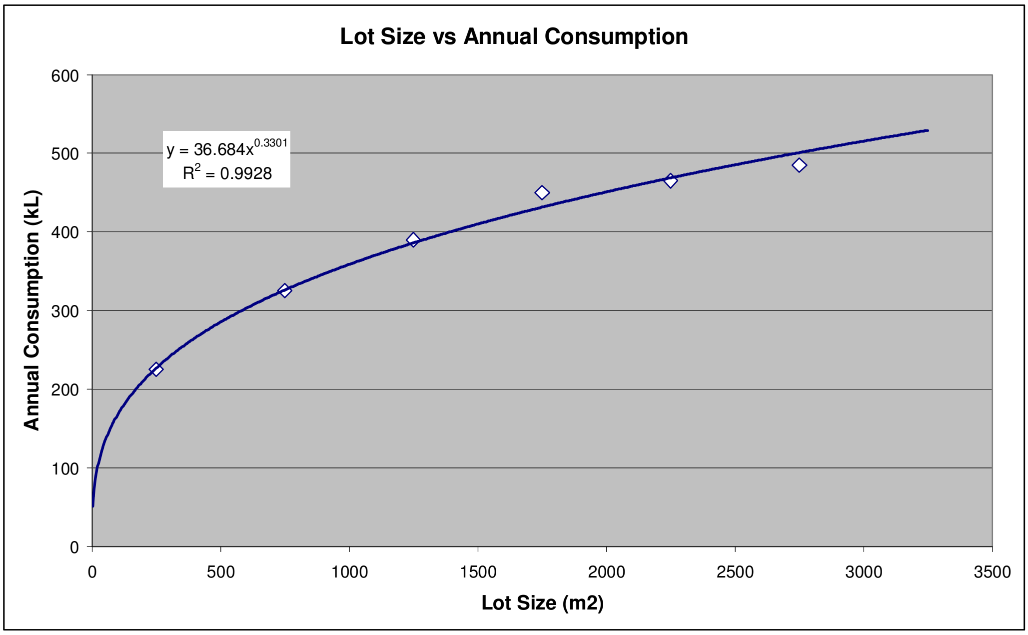

Figure 1: Total Annual Water Use vs Lot Size - A.C.T.

Water usage is observed to increase with lot size. External water usage, predominately irrigation, is likely to be a key factor influencing this relationship.

3.1.2.1 IRRIGATION

In recognition of the influence that irrigation has on external demands, specialist modelling was undertaken to determine annual average application rates using the development's proposed irrigation strategy and landscaping controls (Agsol, 2009).

Based on a *medium watering*[3] strategy for household gardens an average application rate of 467 mm/year is estimated. Urban design lot plans have been used to determine irrigation areas per customer type (Table 3-5).

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[3] For the *medium watering* strategy, sprinkler irrigation scheduling allows some stress to be developed in plants, but without permanent damage. Protected exposure and moderate to high advected energy is also assumed.

Table 3-5: Lot Irrigation Assumptions

Customer Type	Land Area per Dwelling (m2)	Irrigated Area per Dwelling (m2)
Apartments	125	11
Townhouse/Terrace	189	17
Small Courtyard	368	40
Large Courtyard	464	68
Single Lot	544	99
Large Lot	720	156
Estate Homes	1,500	360
Rural	15,000	720

3.1.2.2 COOLING

Although it is not intended that residential evaporative coolers be adopted at Googong, it is considered prudent to provide an allowance for residential cooling. The allowance has been based on an evaporative cooling study at Yarra Valley Water (YVW, 2005). This study identified a strong relationship between daily maximum temperature and air conditioner water usage on a daily basis. An algorithm was developed (YVW, 2005) which recognised daily maximum temperature and also the duration of use and day of the week, as below:

Daily volume per unit (L/d) = 3.58 x MaxTempExcess + 1.09 x Duration + 20.00 x Weekend

Where,

- *MaxTempExcess* = 0 if Maximum Daily Temperature <21^{0}C,
  otherwise *MaxTempExcess* = Max Daily Temperature – 21^{0}C.
- *Duration* = minutes of operation
- *Weekend* = 0 if weekday, 1 if Saturday or Sunday.

This relationship has been used to estimate the average cooling water usage at Googong by applying 40 years of Googong region daily maximum temperature data, with same average operation duration (106 minutes/day) and uptake of coolers (28%) as estimated at Yarra Valley, across all residential dwelling types (47L/d/dwelling). Evaporative air conditioner usage for each dwelling type was then estimated proportionally based on household size (average household size assumed as 2.5).

3.1.2.3 OTHER

It is recognised that there will be a relatively small component of *other* external uses, such as car washing. An allowance of five percent of external demands has been made for *other* uses.

An external end-use leakage allowance (leakage within a customer's property and not leakage in the supply system) is assumed as five percent of the external demand.

3.1.3 TOTAL

The residential internal and external demands are combined to estimate total average daily demands (ADD) (Table 3-6).

Table 3-6: Estimation of Average Residential Water Use

Customer Sector	Units	Total Internal ADD (L/d)	Total External ADD (L/d)	Total ADD (L/d)
Residential				
apartments (A)	per dwelling	291	27	318
townhouse / terrace (B1+B2)	per dwelling	393	39	432
small courtyard (C)	per dwelling	401	72	473
large courtyard (D)	per dwelling	414	112	527
single lot (E)	per dwelling	485	158	644
large lot (F)	per dwelling	502	241	743
estate homes	per dwelling	502	531	1,033
Rural	per dwelling	502	1,043	1,545

3.2 NON-RESIDENTIAL

Average non-residential internal demands have been estimated by manipulating the WSAA peak hour design standards (Appendix B) as described in Section 2.2.

Average external demands are assumed to be a combination of irrigation and cooling demands as discussed in Section 3.2.2.

3.2.1 INTERNAL

In the absence of better information from local water industry studies, the level of typical internal use is assumed to be 0% of the WSAA average demands for open space, and 70% for schools and commercial use (AWWARF, 2000).

Average internal demands are tabled below.

Table 3-7: Estimation of Non-residential Average Internal Water Use

Customer Type	WSAA Peak Hour Demand (L/s/Ha)	Peak Day Demand (no FOS) (L/d/Ha)	Average Day Consumption (no leakage) (L/d/Ha)	Average Day Internal Consumption (L/d/Ha)
Schools	1.1	39,600	13,362	9,353
Open Space	1.5	54,000	7,757	0
Commercial	0.7	25,200	8,503	5,952

3.2.1.1 ASSUMED END-USE BREAKDOWN

Estimates of end-use for the non-residential sector are rare due to the variety of water use in the sector and the fact that so few studies have been undertaken. One exception to this is the Non-Residential end-use study undertaken by the American Waterworks Association Research Foundation (AWWARF 2000). Figures have been used from this study for Googong end-use modelling, in the absence of local information.

Table 3-8: Assumed Breakdown of Internal Water Use (Adapted from AWWARF 2000b)

End Use	General Commercial	Schools
Toilets	38%	32%
Urinals	13%	20%
Showers	5%	6%
Taps/Sinks	10%	18%
Kitchens	5%	9%
Int. Other	25%	10%
Int. Leakage	5%	5%

3.2.2 EXTERNAL

3.2.2.1 IRRIGATION

As for development of the residential demands, specialist irrigation modelling (Agsol, 2009) has been made to estimate annual average irrigation application rates using the development's proposed irrigation strategy and landscaping controls (Table 3-9).

Table 3-9: Assumed Non-residential Irrigation Strategy and Application Rates

Customer Type	Non-residential Irrigation Type and Strategy Area Breakdown		
	Turf/Playing Fields Sprinkler Well Watered	Landscaping Sprinkler Restricted Watering	Landscaping (near buildings/roadways) Subsurface Restricted Watering
Application Rate	568mm/year	418mm/year	459mm/year
Schools	50%	0%	50%
Open Space	58%	40%	2%
Commercial	50%	0%	50%

Notes:

1. Area breakdown percentages are tabled for full development. Minor differences exist in the assumed breakdown for the initial stage of the development.
2. Well watered irrigation is scheduled to avoid any plant stress and address the anticipated high level of wear.
3. Restricted watering irrigation is to maintain aesthetics without using a liberal amount of water.

Planned net areas for irrigation have been adopted for open space development. Schools and commercial lots are assumed to have 50% and 7% of gross areas irrigated to develop average irrigation demands, respectively[4].

3.2.2.2 COOLING

Allowances for average cooling demands in schools and commercial properties have been made. It is assumed cooling demands represent 20% and 50% of the external component of WSAA average demands developed for schools and commercial areas, respectively[4].

[4] Ideally, schools and commercial lots would be assessed on an individual lot basis for design of supply systems.

3.2.2.3 OTHER

An external end-use leakage allowance (leakage within a customer's property and not leakage in the supply system) is assumed as five percent of the external demand.

3.2.3 TOTAL

The non-residential internal and external demands described above are combined to estimate the total average daily demands (ADD) (Table 3-10).

Table 3-10: Estimation of Average Non-residential Water Use

Customer Sector	Units	Total Internal ADD (L/d)	Total External ADD (L/d)	Total ADD (L/d)
Non-Residential				
Schools	per gross Ha	9,353	9,303	18,657
Open Spaces	per net Ha	0	14,587	14,587
Commercial Use	per gross Ha	5,952	2,266	8,218

4. DEVELOPMENT OF DESIGN STANDARDS

This section of the report outlines the development of design standards for Googong based on the water usage estimates in Section 3, peaking factors and adopted factors of safety. Appendix C includes a worked example of the derivation of the residential design standard.

4.1 CALCULATION OF DEMAND BASED PEAK DAY RATES

Peak day rates for the Googong supply area have been developed by the application of the assumed water savings and recycled water connections under the IWC described in the concept design[5] (MWH 2009). Rainwater tanks have been assumed to make no contribution to supply under peak demand conditions, on the basis that most tanks will be dry during the hot dry periods associated with a peak day demand.

The calculation of the demand based peak day rates involved:

1. Preparing the water balance with the expected average day demands outlined in Section 3.
2. Splitting the average day demands into potable and recycled water uses using the end-use demand breakdowns based on the anticipated servicing strategy (Table 4-1)
3. Applying peaking factors to outdoor uses (Section 4.2).
4. Adding in an overall 10% allowance for water supply system leakage (Section 4.2).
5. Adding the required factor of safety (residential: 20% long-term, 40% first five years; non-residential: 20%) (Section 4.4).

Table 4-1: Assumed Application of Recycled Water Under Peak Demand Conditions

End Use	% Recycled Water Applied			
	Residential	Schools	Open Space	Commercial
Internal				
Toilets	100.0%	100.0%	N/A	100.0%
Urinals	N/A	100.0%	N/A	100.0%
Baths	0.0%	N/A	N/A	N/A
Showers	0.0%	0.0%	N/A	0.0%
Taps/Sinks	0.0%	0.0%	N/A	0.0%
Kitchen	0.0%	0.0%	N/A	N/A
Dishwashers	0.0%	N/A	N/A	0.0%
Washing Machines	75.0%	N/A	N/A	N/A
Internal Other	N/A	0.0%	N/A	0.0%
Internal Leakage	50.0%	50.0%	N/A	50.0%
External				
Irrigation	100.0%	100.0%	100.0%	100.0%
Cooling	0.0%	0.0%	N/A	0.0%
Ext. Other	100.0%	100.0%	100.0%	100.0%
External Leakage	100.0%	100.0%	100.0%	100.0%

[5] The concept design IWC is similar to water cycle management Option 6A of the *Googong New Community Integrated Water Cycle Management Report* (MWH 2004). One key difference is cooling demand source supply.

In Table 4-1, it has been assuming that 50% of all on property, internal pipe leakage is attributable to leakage from the recycled water system plumbing; the other 50% coming off the potable water system plumbing. However, for external, on property leakage, it has been assumed that 100% of all leakage comes from the recycled water system plumbing.

4.2 AVERAGE DAY DEMANDS (WITH LEAKAGE)

Table 4-2 outlines the Year 2035 total average demands (potable and recycled per customer type) with a 10% allowance for system leakage included.

Table 4-2: Total Average Day Demand (Potable & Recycled Demand with Allowance for System Leakage)

Customer Sector	Units	Total Internal ADD (L/d)	Total External ADD (L/d)	Total ADD (L/d)
Residential				
apartments (A)	per dwelling	324	30	354
townhouse / terrace (B1+B2)	per dwelling	437	44	480
small courtyard (C)	per dwelling	445	80	525
large courtyard (D)	per dwelling	461	125	585
single lot (E)	per dwelling	539	176	715
large lot (F)	per dwelling	558	268	826
estate homes	per dwelling	558	590	1,148
Rural	per dwelling	558	1,158	1,717
Non-Residential				
Schools	per gross Ha	10,393	10,337	20,730
Open Spaces	per net Ha	0	16,208	16,208
Commercial Use	per gross Ha	6,614	2,518	9,131

4.3 PEAK TO AVERAGE DAY FACTORS

Previously, QCC recommended the system-wide peak to average demand ratio of 3.0 at ultimate development based on examination of historical daily production records in the region (MWH, Jan 08). However, value management assessment of the concept design identified the opportunity to reduce system costs through consideration of the WSA 03 peak to average daily demand factor[6]. The design rates presented in this report are based on a system wide peak to average demand ratio of 2.0 at ultimate development.

The majority of the increase in demand during peak conditions is associated with external water usage. For simplicity it, has been assumed that the peaking factor for all external usage is the same across all the customer categories. The peaking factor for internal water use and system leakage is assumed to equal 1.0. A peak to average day demand ratio of 4.0 for external water use is required to achieve an overall peak to average day factor of 2.0.

[6] WSA 03 (2002) recommends a peak to average day demand factor of 1.5 for populations greater than 10,000 and 2.0 for populations below 2,000, unless otherwise specified by the water agency.

4.4 GOOGONG DESIGN STANDARDS

4.4.1 DEMAND BASED PEAK DAY RATES

The final step in developing the demand based peak day rates was inclusion of an acceptable factor of safety. As previously noted, the factors of safety to be adopted were agreed with CIC and QCC as:

- residential: 20% long-term, 40% first five years;
- non-residential: 20%.

Table 4-3 outlines the resulting demand based peak day rates inclusive of these factors.

Table 4-3: Demand Based Potable and Recycled Water Peak Day Rates

Customer Sector	Potable ADD (L/d)	Expected Potable Peak Day Demand (L/d)	Potable Peak to Average Ratio	Potable PDD with FOS (Ultimate) (L/d)	Potable PDD with FOS (First Five Years) (L/d)	Recycled ADD (L/d)	Expected Recycled Peak Day Demand (L/d)	Recycled Peak to Average Ratio (L/d)	Recycled PDD with FOS (Ultimate) (L/d)	Recycled PDD with FOS (First Five Years) (L/d)	Overall Peak to Average Ratio
Residential (per dwelling)											
apartments (A)	217	247	1.14	296	346	136	184	1.35	220	257	1.22
townhouse / terrace (B1+B2)	293	333	1.14	400	466	187	259	1.38	310	362	1.23
small courtyard (C)	299	340	1.14	408	476	227	392	1.73	470	549	1.39
large courtyard (D)	309	351	1.14	422	492	276	553	2.00	664	774	1.55
single lot (E)	362	411	1.14	494	576	353	755	2.14	906	1,057	1.63
large lot (F)	375	426	1.14	511	596	451	1,086	2.41	1,303	1,520	1.83
estate homes	375	426	1.14	511	596	773	2,235	2.89	2,682	3,129	2.32
Rural	375	426	1.14	511	596	1,342	4,262	3.18	5,114	5,967	2.73
Non-Residential (per Hectare)											
Schools	6,677	12,088	1.81	14,506	Refer to ultimate rates.	14,053	35,156	2.50	42,187	Refer to ultimate rates.	2.279
Open Spaces	0	0	1.00	0		16,208	57,783	3.57	69,339		3.565
Commercial Use	4,417	7,861	1.78	9,433		4,715	7,729	1.64	9,275		1.707

Note: Additional significant figures have been shown in this table to assist with accuracy checking.

4.4.2 COMPARISON WITH TRADITIONAL DESIGN GUIDELINES

Table 4-4 compares NSW Public Works (1986), WSA 03 (2002) (for the ACT), and Sydney Water (2008) design standards with the Googong development demand based peak day rates. Note that the Googong and Sydney Water (SWC) figures are the sum of potable and recycled water.

Table 4-4: Googong Demands Comparison

Customer Sector	Units	Googong Peak Day Demand (Potable + Recycled)		Public Works NSW (1986)		WSAA (2002)		Sydney Water (2008) Dual Reticulation Systems (Combined potable + ReW)	
		Utlimate per Unit (L/d)	First 5 Years per Unit (L/d)	Nearest Applicable Sector	Peak Day Demand per Unit (L/d)	Nearest Applicable Sector	Peak Day Demand per Unit (L/d)	Nearest Applicable Sector	Peak Day Demand per Unit (L/d)
Residential									
apartments (A)	per dwelling	517	603	Flats/Units/townhouses	1,000	High Density	1,296	Multi-unit (61-100)	880
townhouse / terrace (B1+B2)	per dwelling	710	828	Flats/Units/townhouses	1,000	500m2 lots	1,296	Multi-unit (30-60)	1,350
small courtyard (C)	per dwelling	878	1,024	Houses	4,000	500m2 lots	3,240	Townhouse (<30)	1,570
large courtyard (D)	per dwelling	1,085	1,266	Houses	4,000	500m2 lots	3,240	Townhouse (<30)	1,570
single lot (E)	per dwelling	1,400	1,633	Houses	4,000	500m2 lots	3,240	Single Residential	2,200
large lot (F)	per dwelling	1,814	2,117	Houses	4,000	500m2 lots	3,240	Single Residential	2,200
estate homes	per dwelling	3,193	3,725	Houses	4,000	1,000m2 lots	6,480	Single Residential	2,200
Rural	per dwelling	5,625	6,563	Houses	4,000	1,000m2 lots	6,480	Single Residential	2,200
Non-Residential									
Schools	per gross Ha	56,693	56,693	Schools	N/A	Schools	47,520	Commercial	41,000
Open Spaces	per net Ha	69,339	69,339	Parks, Ovals Etc.	83,333	Parks	86,400	Irrigation	7,000
Commercial Use	per gross Ha	18,708	18,708	Offices/shops, etc	40,000	Commercial	30,240	Commercial	41,000

Notes:

1. WSA peak day estimates are based on peak hourly rates divided by the WSAA recommended peak hour to peak day ratio of 2.0
2. Public Works design guidelines for schools are based on the number of students, not the number of hectares, thus comparison is not possible.
3. Googong design standards including 20% factor of safety.
4. Public Works and WSA Open Space demands have been converted to net area rates assuming net area = 0.75 x gross area.

5. Googong peak day demand rates for schools and commercial use are modified for application as design rates (refer to Section 4.4.3).

Comparison of figures is difficult and requires multiple assumptions as described in the notes above.

The demand based *residential* peak daily rates are generally lower than the traditional and Sydney Water design figures.

The *open space* demand based rates are for net areas (actual areas to be irrigated) and traditional design figures are based on gross areas. Direct comparison requires case by case assessment. As specialist modelling forms the basis of open space demand estimates, it is proposed to maintain the demand based figures as design rates. However, it is recognised that the opportunity exists to decrease peak day rates, especially if irrigation application and scheduling during peak demand period can be controlled.

The other non-residential (*schools* and *commercial*) categories vary from the design rates and include assumptions in their development which may vary significantly on a case by case basis. Modification of demand based rates for the proposed *school*s and *commercial* design rates is described in the following section. A comparison summary of the estimated peak day demands for the development using different design standards is provided in Appendix E.

4.4.3 PROPOSED GOOGONG DESIGN RATES

The demand based rates reflect the assumptions made throughout this report including conservation measures and the system wide peak to average daily ratio. Application of these figures will require vigilance in insuring these assumptions are realised in the development. Incorporation of risk management approaches in the design and operation of the supply systems, including monitoring of demands, management of water usage and the ability to modify infrastructure particularly in the early development stages, is assumed.

It is recognised that key assumptions associated with development of the demand based peak day rates for the *schools* and *commercial* categories, may vary significantly on a case by case basis. For instance, actual areas to be irrigated in schools, and cooling demands in commercial properties, can significantly influence water supply needs. As such, at this stage it is difficult to justify design rates for these categories, which differ from standard design rates for the combined potable and recycled demands. The following modifications are proposed for Googong *schools* and *commercial* design rates:

1. Schools – The combined potable and recycled water design rates to equal the WSA 03 design rates. The *schools* recycled water peak demands have been reduced to achieve this.
2. Commercial - The combined potable and recycled water design rates to equal the WSA 03 design rates. The *commercial* potable water peak demands have been increased to achieve this.

Monitoring and evaluation of initial stage demands and flows will be used to confirm all design rates for subsequent stages of the development (Section 4.8).

Table 4-5: Proposed Potable and Recycled Water Design Standards (Peak Day)

Customer Sector	Potable ADD (L/d)	Expected Potable Peak Day Demand (L/d)	Potable Peak to Average Ratio	Potable PDD with FOS (Ultimate) (L/d)	Potable PDD with FOS (First Five Years) (L/d)	Recycled ADD (L/d)	Expected Recycled Peak Day Demand (L/d)	Recycled Peak to Average Ratio (L/d)	Recycled PDD with FOS (Ultimate) (L/d)	Recycled PDD with FOS (First Five Years) (L/d)
Residential (per dwelling)										
apartments (A)	217	247	1.14	296	346	136	184	1.35	220	257
townhouse / terrace (B1+B2)	293	333	1.14	400	466	187	259	1.38	310	362
small courtyard (C)	299	340	1.14	408	476	227	392	1.73	470	549
large courtyard (D)	309	351	1.14	422	492	276	553	2.00	664	774
single lot (E)	362	411	1.14	494	576	353	755	2.14	906	1,057
large lot (F)	375	426	1.14	511	596	451	1,086	2.41	1,303	1,520
estate homes	375	426	1.14	511	596	773	2,235	2.89	2,682	3,129
Rural	375	426	1.14	511	596	1,342	4,262	3.18	5,114	5,967
Non-Residential (per Hectare)										
Schools	6,677	12,088	1.81	14,506	Refer to ultimate rates.	14,053	35,156	1.96	33,014	Refer to ultimate rates.
Open Spaces	0	0	1.00	0		16,208	57,783	3.57	69,339	
Commercial Use	4,417	7,861	3.96	20,965		4,715	7,729	1.64	9,275	

4.5 GOOGONG PEAK DAY DEMAND FORECAST

The proposed potable and recycled water design standards have been adopted to prepare system peak day demand (PDD) estimates. Four development stages have been considered:

1. Stage 1 – Neighbourhood Area 1A, stages 1 and 2 only.
2. Stage 2 – Neighbourhood Area 1.
3. Stage 3 – representing half of full development.
4. Stage 4 – Full development.

Table 4-6 outlines the assumed development yield associated used to estimate peak day demands.

Table 4-6: Assumed Googong Development Yield

Customer Type	Units	Household Size	Stage 1	Stage 2	Stage 4
Apartments	Dwelling	1.89	0	26	566
Townhouse/Terrace	Dwelling	2.55	0	51	477
Small Courtyard	Dwelling	2.60	51	211	852
Large Courtyard	Dwelling	2.69	35	293	1,311
Single Lot	Dwelling	3.15	146	452	1,833
Large Lot	Dwelling	3.26	110	243	818
Estate Homes	Dwelling	3.26	0	0	281
Rural	Dwelling	3.26	0	0	58
Schools	Gross Ha	NA	0	5.1	21
Open Space	Net Ha	NA	7	13	45
Commercial	Gross Ha	NA	0.2	1.2	14

Table 4-7 summarises the estimated peak day demands for each development stage as well as the open space demands contribution during Stage 1 and Stage 2.

Table 4-7: Development Stage Peak Day Demands

Peak Day Demand (ML/d)													
Potable				Recycled				Combined (Potbale and Recycled)				Open Space Contribution	
Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2
0.2	0.8	1.7	3.4	0.9	2.3	4.6	9.3	1.1	3.1	6.3	12.6	0.5	0.9

PDD forecasts are tabled in Appendix E.

4.6 PEAK HOUR DESIGN FACTORS

WSA-03 (2002) recommends a total system peak hour factor (PHF) of 2.0 for populations greater than 10,000 and 5.0 for populations below 2,000. Limited studies have been completed on the PHFs observed in dual water supply systems, such as proposed at Googong.

In the absence of site specific assessment, PHFs and diurnal patterns developed by Sydney Water (SWC, 2009) for application in dual reticulation systems are proposed with minor modification for Googong, to facilitate design of distribution systems. Table 4-8 summarises the PHFs.

Table 4-8: Peak Hourly Factors

Customer Category	SWC Nearest Category	Peak Hour Factor	
		Potable	ReW
Apartments	*Multi-unit (61-100/ha)*	3.4	2.7
Townhouses	*Multi-unit (30-60/ha)*	3.4	3.2
Small Courtyard	*Townhouse (<30/ha)*	3.0	4.0
Large Courtyard	*Townhouse (<30/ha)*	3.0	4.0
Single Lot	*Single Dwelling Res.*	3.4	4.6
Large Lot	*Single Dwelling Res.*	3.4	4.6
Estate Homes	*Single Dwelling Res.*	3.4	4.6
Rural	*Single Dwelling Res.*	3.4	4.6
Schools (gross Ha)	*Commercial*	2.0	2.0
Open Spaces (net Ha)	*Irrigation*	NA	1.5
Commercial Use (gross Ha)	*Commercial*	2.0	2.0

Notes:
1. At system peak hour demand, the Open Spaces hourly factor is 1.04. Refer to assumed diurnal patterns.
2. Recycled water is assumed to be used for toilet, washing machine and external demands.

Table 4-7 outlines the estimated peak hour demands, assuming all peaks coincide.

Table 4-9: Development Stage Peak Hour Demands

Peak Hour Demand (ML/d)*											
Potable				Recycled				Combined			
Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2	Stage 3	Stage 4	Stage 1	Stage 2	Stage 3	Stage 4
0.6	2.4	5.1	10.2	2.5	7.1	14.8	29.6	3.1	9.5	19.9	39.8

Assumed diurnal patterns for system modelling are provided in Appendix F.

4.7 FIRE FLOWS

WSA 03 (2002) does not specifically stipulate design requirements for fire fighting capability. The dual water supplement (WSAA 2000b) suggests factors for determination of the appropriate system for fire fighting purposes including:

- security of supply
- available storage
- life cycle costs
- pressure of supplies
- other water agency nominated factors.

It is anticipated that the recycled water system will be used for fire fighting purposes. However, until this has been confirmed through relevant approvals, hydraulic modelling of both the potable and recycled distribution systems will include fire flow checks during minimum town main hydraulic characteristics[7] of:

- 10L/s at all nodes; and,
- 30L/s at commercial nodes.

4.8 MONITORING AND EVALUATION OF DESIGN DEMANDS

It is recognised that the demand numbers developed in this report are estimates, often made without the benefit of clearly established industry design criteria. Performance of the actual systems may vary compared to the modelled estimates for a number of reasons, including:

- Modelling limitations, in particular associated with the ability to replicate the extent and completeness of the future systems.
- Errors associated with model input assumptions. For instance, the inputs include assumptions in the way people will use water in the future.
- Variation in the assumed development lot yields, as well as arrangement, sizing and operation of the final integrated system components.

To manage any differences which may occur, design demand criteria have been set, in consultation with QCC, assuming the following:

- A staged approach to the provision of potable water and non-potable water supply major infrastructure. The initial stage is anticipated to represent the first five years of predicted growth.
- An increased factor of safety for residential demands during the initial stage of the development.

[7] Assumed to be represented by maintaining greater than 3m residual head throughout the supply system at peak hour flow demands with the supply reservoir at reserve storage level.

- The use of smart metering to monitor water usage and production during the initial stage of the development. The demand and production data will be used to evaluate the design criteria, including the adopted factor of safety and assumptions made in development of the standards, and where necessary modify design criteria for future development. Opportunities for improved system management, for instance targeting peak demand management, should also be investigated with the improved understanding of water usage.

- In the early development stages, there will be significant levels of control over non-domestic external irrigation, such as the ability to limit the scheduling of parks/gardens irrigation to out of peak hours.

- For the initial stage, and each stage of construction, system capacity significantly exceeds requirements, because each stage is sized for the population growth expected at the end of the stage. With review of monitoring data it will be possible to adjust design parameters based on measured rather than theoretical data.

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APPENDIX A

DEMAND DERIVATION FLOW CHARTS

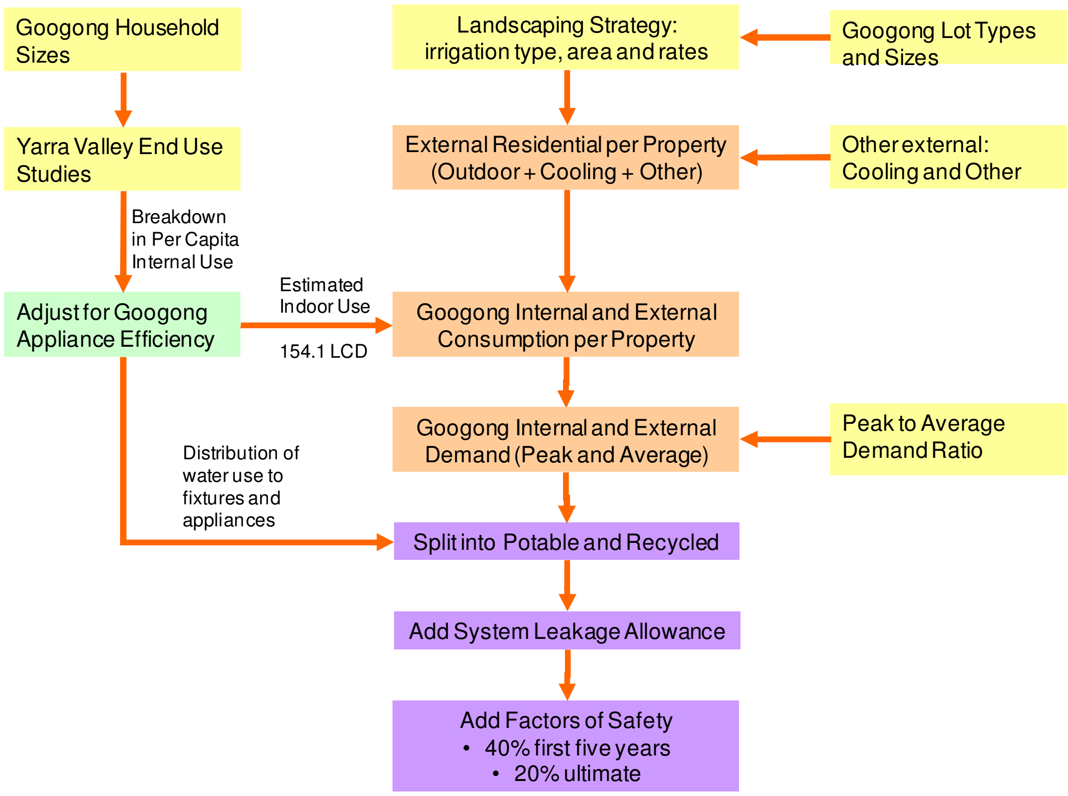

Figure 2: Demand Based Design Standards Development Process - Residential Uses

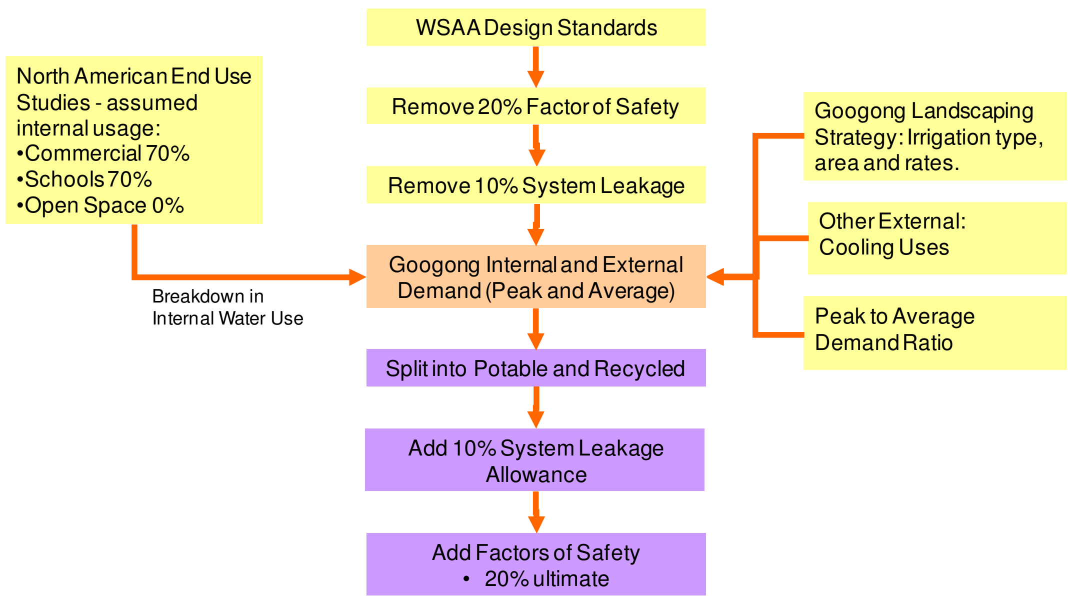

Figure 3: Demand Based Design Standards Development Process - Non-Residential Uses

APPENDIX B

COPY OF WSA 03-2002 TYPICAL PEAK HOUR DEMAND RATES

TABLE 2.1
TYPICAL PEAK HOUR DEMAND RATES

Demand type[A]	Demand rate units	Adelaide	Ballarat/Bendigo	Brisbane	Canberra[B]	Darwin	Gold Coast	Melbourne/Geelong	Newcastle	Perth	Sydney	Gippsland
Residential												
High density	L/s/100 units	4.25	3	6	3[C]	9.8[D]	7.75	3	1.6	9.7	3	[E]
1000 m^2 lots	L/s/100 lots	9	9	15	15	15.6[D]	7.75	10	2.9	18.7	12	[F]
500 m^2 lots	L/s/100 lots	6.35	6	10	7.5	15.6[D]	5.81	8	2.9	14	6.5	[F]
Commercial	L/s/ha	1.20	0.5	0.4	0.6 – 1.1	1.1	1.21	0.6	0.5	0.7	0.9	[E]
Industrial												
General heavy	≥200 ha L/s/ha			0.25			[G]	0.8	[E]	0.7	[E]	[E]
	≥40 ha L/s/ha			0.4			[G]	1.0	[E]	0.7	1.3	[E]
Light	L/s/ha	0.65	0.5	0.25	0.54	1.1	1.21	0.4	0.26	0.7	1.3	[E]
Designated high usage	L/s/ha		2	[E]			[G]	2.5	[E]	[E]	[E]	[E]
Public utilities												
Schools	L/s/ha L/s/school	0.25	0.25	2	1.1	1.7	[G]	0.23	0.3		4 High 2 Primary	[E]
General public purposes	L/s/ha	0.25	0.1	1.0	0.75		[G]	0.2			0.1	[E]
Hospitals	L/bed/day L/s/ha	0.25	4000	1000	1.7		1870	900	1675	938	1500	[E]
Reserves												
Parks	L/s/ha		0.1	0	1.5	3.5	[G]	0.2			0.1	[E]
Golf courses	L/s/ha		0.1	0	1.5		[G]	0.05	0.04		0.1	[E]
Market gardens	L/s/ha			0			[G]	0.2			0.25	[E]
Pastures	L/s/ha			0			[G]	0.01			[E]	[E]
Gardens	L/s/ha			0	1.5		[G]	0.32			0.25	[E]
Other												
Fire demand	L/s/hydrant			[H]	[J]							

APPENDIX C

WORKED EXAMPLE – DERIVATION OF RESIDENTIAL WATER DESIGN DEMANDS

To illustrate the process for the development of the design standards for residential use, the process has been set out for a Single Lot (e) dwelling size.

Step 1: Estimate the average internal consumption based on 154.1 L/p/d (Table 3-1) and 3.15 persons per dwelling (Table 4-6):

$$C_{INT} = 154.1 \times 3.15 = 485.4 L/day$$

Step 2: Estimate external irrigation consumption based on an irrigation rate of 467mm/year (Section 3.1.2.1) and an irrigated area of 99m^2 (Table 3-5):

$$C_{EXT,IRRIGATION} = \frac{467 \times 99}{365} = 126.7 L/day$$

Step 3: Estimate external cooling consumption by applying 28% dwellings (Section 3.1.2.2) with evaporative coolers at 47L/d (Section 3.1.2.2) for average household size of 2.5 (Section 3.1.2.2) and proportioning based on household size for single lot of 3.15 (Table 4-6).

$$C_{EXT,COOLING} = \frac{28\% \times 47 \times 3.15}{2.5} = 16.6 L/day$$

Step 4: Estimate external other consumption and external on-site leakage by applying 5% total external factors respectively.

$$C_{EXT,OTHER} = C_{EXT,LEAKAGE} = 5\% \times \frac{(126.0 + 16.5)}{(1 - (5\% + 5\%))} = 8.0 L/day$$

Step 5: Estimate total external consumption by summing external consumption values above:

$$C_{EXTERNAL} = 126.7 + 16.6 + 8.0 + 8.0 = 159.3 L/day$$

Step 6: Estimate total average consumption by summing internal and external consumption values above:

$$C_{TOTAL} = 485.4 + 159.3 = 644.7 L/day$$

Step 7: Estimate total average system demand by adding 10% allowance for system leakage (Section 4.1):

$$S_{TOTAL} = 644.7 \times (1 + 10\%) = 709.2 L/day$$

Step 8: The DSM DSS model is used to divide the total water system use into potable and non-potable on the basis of the assumed end-use breakdown (rounding errors <1% noted).

$$S_{POTABLE} = 362L/day$$

$$S_{RECYCLED} = 353L/day$$

Step 9: The DSM DSS model estimates the design peak demand on the basis of the assumed peaking factors for each end-use targeted by the recycled water (Table 4-3) where the external demands are assumed to have a peak to average ratio of approximately 4.0 to give a total system peak to average ratio of 2.0. It also applies the relevant factor of safety for the design case (Section 4.1) (rounding errors <1% noted) as below:

$$D_{POTABLE,PDD,ULTIMATE} = 362 \times 1.14 \times (1 + 20\%) = 495L/day$$

$$D_{RECYCLED,PDD,ULTIMATE} = 353 \times 2.14 \times (1 + 20\%) = 907\,L/day$$

APPENDIX D

ADDITIONAL ULTIMATE DEMAND INFORMATION

The following ultimate end-use breakdown information is provided for QCC, as requested.

Table 5-1: Example of the Application of Recycled Water to End-Uses (Single Lot – 544m^2)

Type of Consumption	% Recycled Water Applied	Potable Water Use (L/d)	Non-Potable Water Use (L/d)	Total (L/d)
Toilets	100.0%	0	67	67
Baths	0.0%	10	0	10
Showers	0.0%	165	0	165
Taps/Sinks	0.0%	77	0	77
Dishwashers	0.0%	9	0	9
Washing Machines	75.0%	30	90	121
Internal Leakage	50.0%	19	19	38
Total Internal		309	176	485
Irrigation	100.0%	0	126	126
Cooling	0.0%	17	0	17
Ext. Other	100.0%	0	8	8
External Leakage	100.0%	0	8	8
Total External		17	142	158
Total		326	318	644

With a total consumption of approximately 24% for non-residential demands, the breakdown of customer water use (potable + recycled) gives a reasonable split between residential and non-residential that is broadly consistent with the levels seen in most communities.

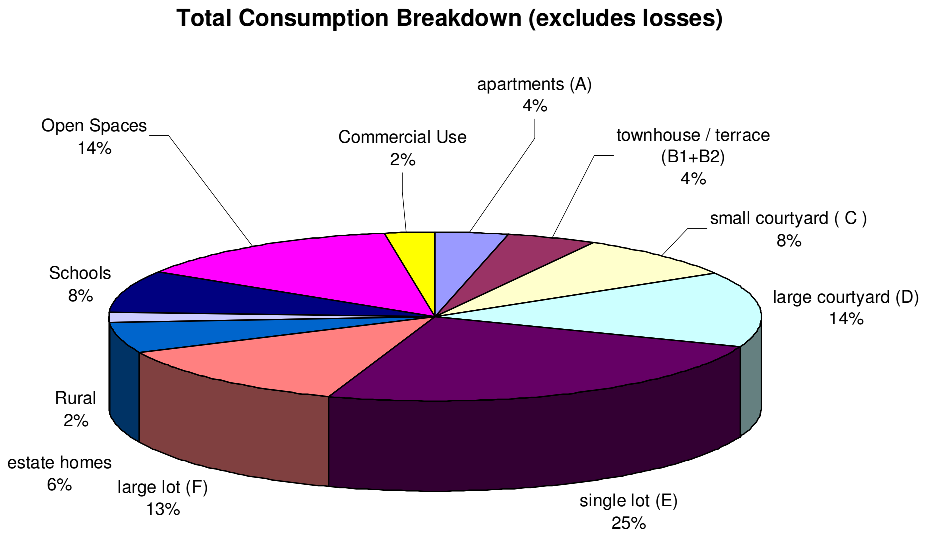

Figure 4: Breakdown in Total Annual Consumption (by customer sector - excludes system leakage)

APPENDIX E

PEAK DAY DEMAND FORECASTS

A comparison of the estimated peak day demands using the proposed Googong design rates with other design rates is tabled below.

Water Demand Basis	Peak Day Demand (ML/d)					
	Potable		Recycled		Combined	
Stage	NH1	Full	NH1	Full	NH1	Full
Googong Design Demands - Feb 10 (PDD:ADD=3.0)	1.2	3.8	4.8	15.1	6.0	19.0
WSA 03 -2002 Design Demands - Canberra Region	NA	NA	NA	NA	7.0	24.4
NSW Public Works (1986) Design Demands (assumes WSA 03 for Schools)	2.6	9.3	6.6	20.8	9.5	31.2
Sydney Water Design Demands - March 2008, Dual Reticulation Systems (ReW to toilets, washing machines & external)	1.1	4.2	2.5	8.6	3.7	12.9
Value Management Considerations	**NH1**	**Full**	**NH1**	**Full**	**NH1**	**Full**
Googong Design Demands - Jul 10 - PDD:ADD = 2.0	1.0	3.4	2.9	9.3	3.9	12.6

Peak day demand forecasts tabled below are based on constant growth between Year 1 (2011) and Year 7 for completion of Neighbourhood Area 1A, and Year 25 for full development.

Potable System - ML/d (Includes 20% FOS Unless Noted Otherwise)

Customer Category	2011	2012	2013	2014	2015	2016	2017	2021	2026	2036
apartments (A)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2
townhouse / terrace (B1+B2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.2
small courtyard (C)	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.3
large courtyard (D)	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.2	0.4	0.6
single lot (E)	0.0	0.1	0.1	0.1	0.2	0.2	0.2	0.4	0.6	0.9
large lot (F)	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.2	0.3	0.4
estate homes	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1
Rural	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Schools	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.3
Open Spaces	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Commercial Use	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2	0.3
Total	0.1	0.2	0.3	0.4	0.5	0.6	0.7	1.3	2.1	3.4
Total - 40% FOS on Res.	0.1	0.2	0.3	0.4	0.6					

Recycled System - ML/d (Includes 20% FOS Unless Noted Otherwise)

Customer Category	2011	2012	2013	2014	2015	2016	2017	2021	2026	2036
apartments (A)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1
townhouse / terrace (B1+B2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1
small courtyard (C)	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.3	0.4
large courtyard (D)	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.4	0.6	0.9
single lot (E)	0.1	0.1	0.2	0.2	0.3	0.4	0.4	0.7	1.1	1.7
large lot (F)	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.5	0.7	1.1
estate homes	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.4	0.8
Rural	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2	0.3
Schools	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.3	0.4	0.7
Open Spaces	0.1	0.3	0.4	0.5	0.7	0.8	0.9	1.4	2.1	3.1
Commercial Use	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1
Total	0.3	0.6	0.9	1.2	1.5	1.8	2.2	3.8	5.9	9.3
Total - 40% FOS on Res.	0.3	0.7	1.0	1.3	1.7					

Potable Plus Recycled Systems Combined - ML/d (Includes 20% FOS Unless Noted Otherwise)

Customer Category	2011	2012	2013	2014	2015	2016	2017	2021	2026	2036
apartments (A)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2	0.3
townhouse / terrace (B1+B2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2	0.3
small courtyard (C)	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.3	0.5	0.7
large courtyard (D)	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.6	0.9	1.4
single lot (E)	0.1	0.2	0.3	0.4	0.5	0.5	0.6	1.1	1.7	2.6
large lot (F)	0.1	0.1	0.2	0.3	0.3	0.4	0.4	0.7	1.0	1.5
estate homes	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.5	0.9
Rural	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2	0.3
Schools	0.0	0.1	0.1	0.1	0.2	0.2	0.2	0.4	0.6	1.0
Open Spaces	0.1	0.3	0.4	0.5	0.7	0.8	0.9	1.4	2.1	3.1
Commercial Use	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.2	0.4
Total	0.4	0.8	1.2	1.6	2.0	2.4	2.8	5.1	8.0	12.6
Total - 40% FOS on Res.	0.4	0.9	1.3	1.8	2.2					

APPENDIX F

PEAK DAY DIURNAL PATTERNS

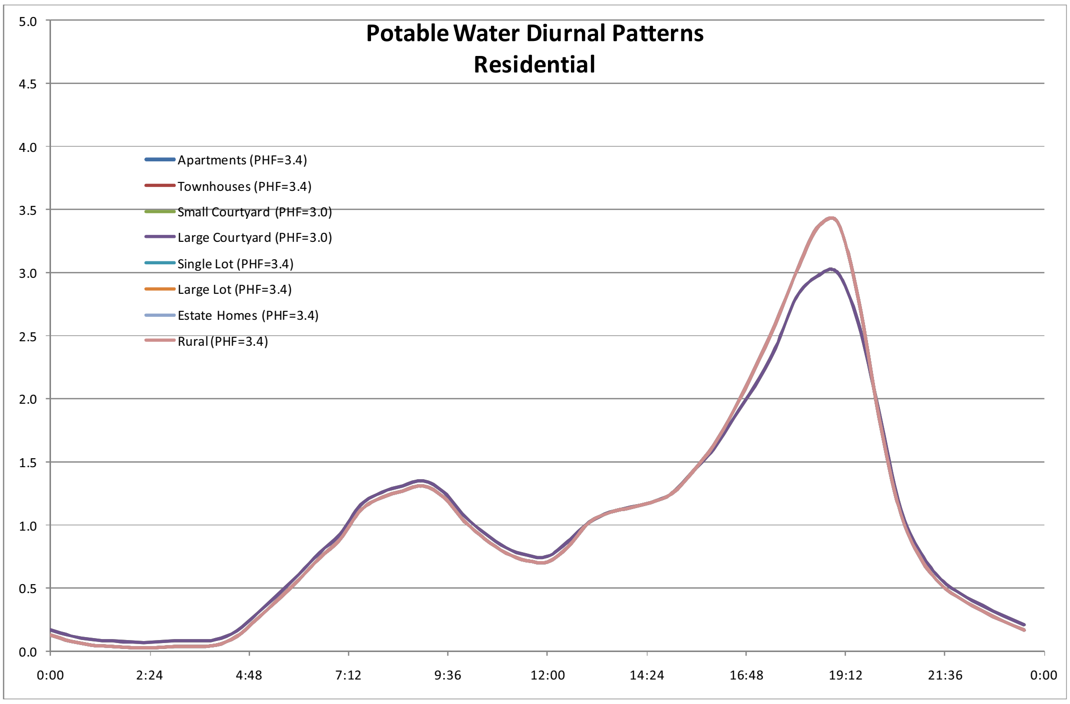

Figure 5: Potable Water Diurnal Patterns - Residential

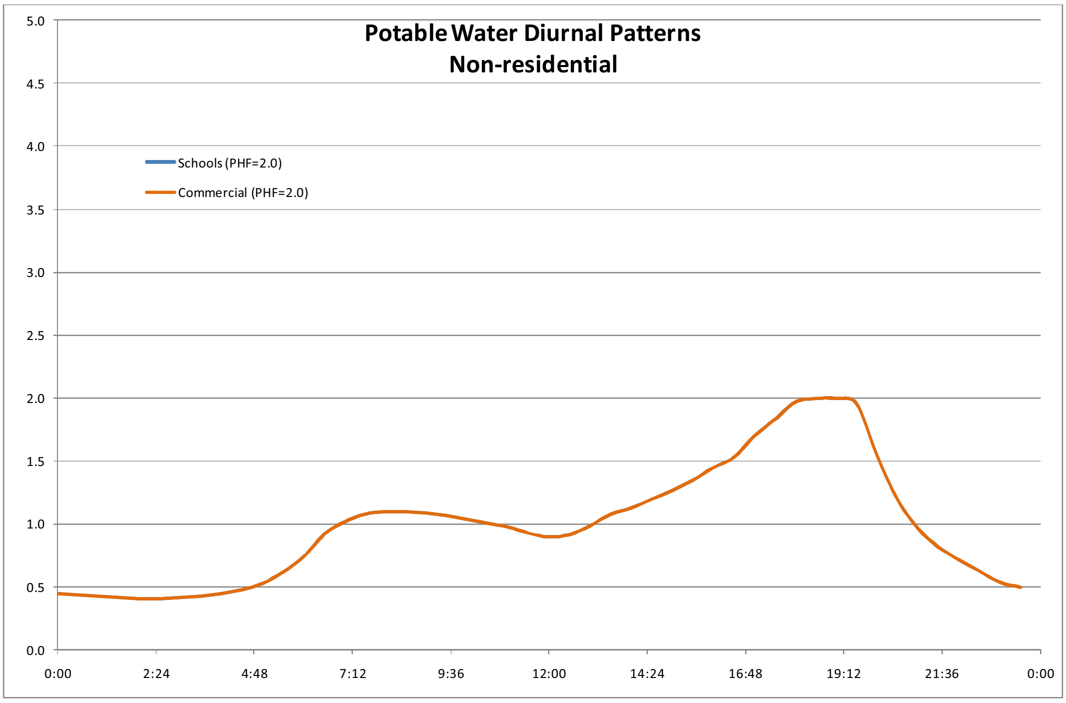

Figure 6: Potable Water Diurnal Patterns – Non-residential

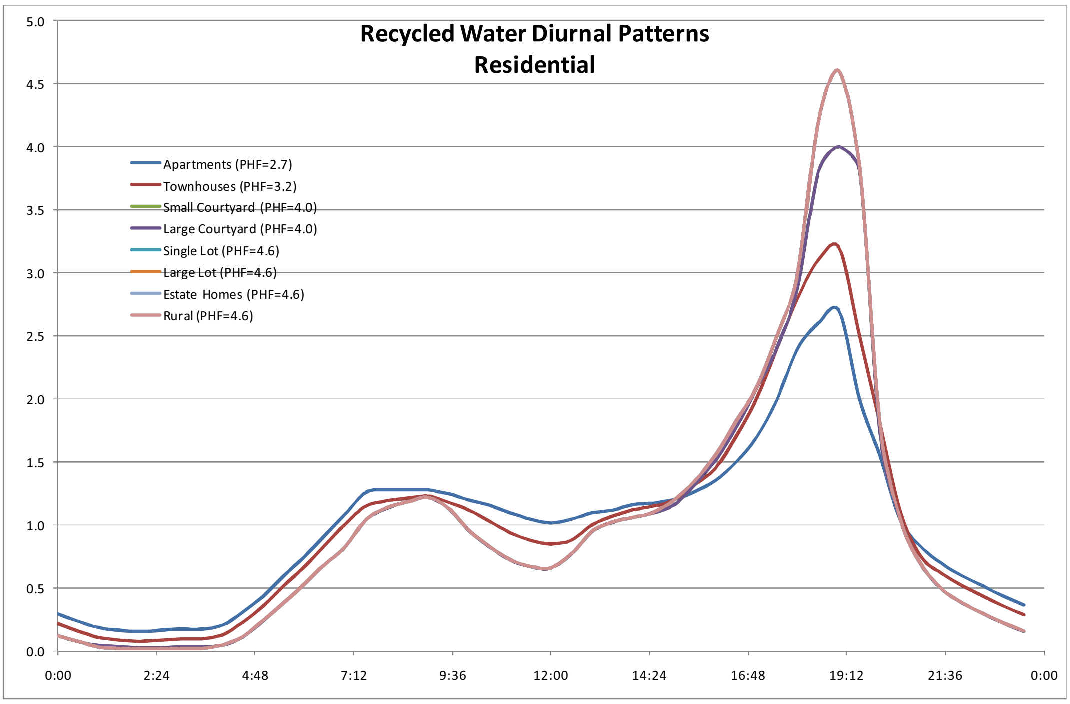

Figure 7: Recycled Water Diurnal Patterns – Residential

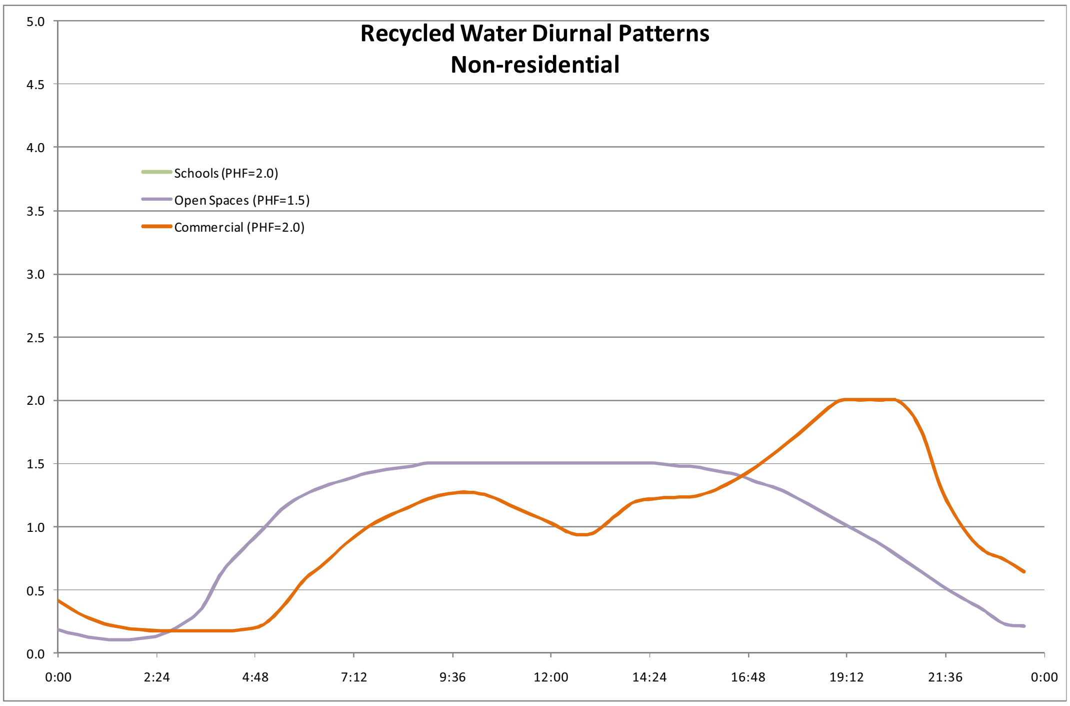

Figure 8: Recycled Water Diurnal Patterns – Non-residential

APPENDIX B.
Water Quality Analysis for ACTEW Supply

B.1 WATER SOURCES AND CHARACTERISATION

B.1.1 Introduction

Water Quality Data has been collected from ActewAGL to provide the basis for the design of the secondary water conditioning for Googong Town Potable Water System. The information below summarises that information, the requirements for the Australian Drinking Water Guidelines and the treatment processes required.

B.1.2 Water Characterisation

The water quality study has been based on data provided by ActewAGL from four sampling locations:

1. Googong WTP Effluent
2. Stromlo WTP Effluent (after passing through Canberra)
3. Queanbeyan water supply off take #1
4. Queanbeyan water supply off take #2

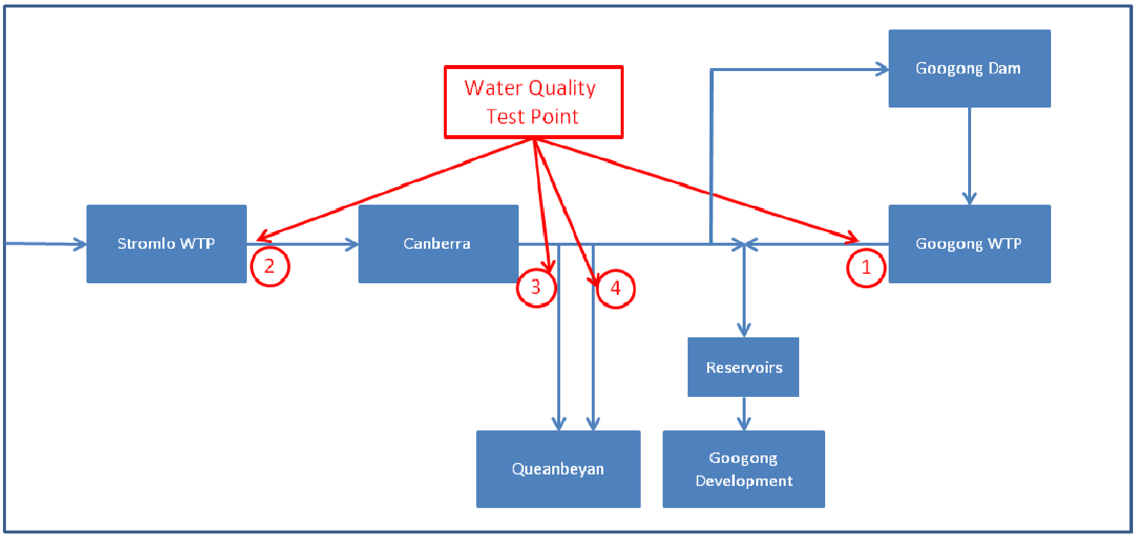

Figure B-1 Potable Water Sources for the Googong Development

Sampling from these locations was conducted by ActewAGL as part of their routine monitoring program. The sampling locations are identified in Figure B-1. Data provided for Queanbeyan is for two separate sampling points, which are assumed to be on separate off takes from the water supply pipeline connecting Stromlo and Googong WTPs.

The Googong development will draw its drinking water from two sources. During some parts of the year, the Googong development will draw water from Googong Water Treatment Plant (WTP). At other times, the Googong WTP will not be operating, and the development will draw water from Stromlo WTP plant.

Data on the quality of water provided by these water treatment plants has been sourced from ActewAGL and comes from the following locations:

Googong WTP: data available from sample point (1) in Figure B-1.

- This sampling point is located after chlorine is dosed into the network at the treatment plant.
- Stromlo WTP: data available from three sampling locations: Sample point (2) identified as “Stromlo”, is located on the outlet line from Stromlo WTP, after chlorine dosing;
- Sample points (3) and (4) cover the water quality to Queanbeyan. The pipeline to Queanbeyan draws water at a similar point to the proposed off take to the Googong development. As such, the data from this Queanbeyan off take is roughly representative of the quality of water expected at the Googong development when water is being drawn from Stromlo WTP.

The data provided by ActewAGL covers the following parameters:

- pH
- Free Chlorine concentration (mg/L)
- Hardness (mg CaCO3/L)
- Alkalinity (mg CaCO3/L)
- Temperature (°C)
- Turbidity (NTU)
- Total Iron (mg Fe/L)
- Total Manganese (mg Mn/L)
- Total dissolved solids (mg TDS/L)
- Conductivity (uS/cm)
- Langelier Saturation Index (LSI)

The data set for each of the locations was analysed, and a summary of the data, including values for the average, minimum, maximum and 90th percentile is provided in Table B-1. A count of the number of data points provided is also listed as “Count”. Some parameters have been recorded regularly, but others were recorded only sporadically.

The assessment of water quality for the Googong development will focus on 3 key parameters for each sampling location:

1. Free chlorine concentration
2. Alkalinity (measured by pH)
3. Langeliers Saturation Index (LSI)

B.2 WATER QUALITY SUMMARY

B.2.1 Googong WTP Source

B.2.1.1 Googong WTP Treated Water Quality Data

Data for Googong was collected by ActewAGL between February 2005 and May 2009. The dataset is somewhat sporadic, with no data available for some periods, most notably March – Jul 2005, March – October 2006 and August – October 2008. It has been assumed that in these periods (and others) the Googong WTP was offline.

Limited data is available for some months and not all factors were measured on all days, as indicated in the "Count" column of Table B-1.

Table B-1 Googong WTP Treated Water Quality

Contaminant	Units	Min	Ave	Max	90%ile	COUNT
pH	-	7.3	7.7	8.2	7.9	178
Chlorine	mg Cl_2/L	0.1	1.7	5.4	2.3	423
Hardness	mg $CaCO_3$/L	42.0	45.4	49.9	48.2	8
Alkalinity	mg $CaCO_3$/L	44.0	50.8	69.0	60.0	36
Temperature	^{o}C	7.0	18.4	28.0	22.7	436
Turbidity	NTU	0.1	0.5	1.5	0.8	167
Total Iron	mg Fe/L	0.02	0.02	0.07	0.03	79
Total Manganese	mg Mn/L	0.00	0.01	0.11	0.02	84
Total Dissolved Solids	mg TDS/L	82	124	140	140	25
Conductivity	uS/cm	120	184	210	200	28
Langelier Saturation Index	LSI	-0.43	-0.83	-1.23	-1.09	24

B.2.1.2 Free Chorine Concentration

The treated water quality data from the Googong WTP indicates relatively high chlorine concentrations (free chorine). Chlorine residual is required in the treated water discharge to ensure no bacteriological growth in the reticulation system occurs. Notwithstanding this, high chlorine concentrations have been known to cause taste and odour problems for consumers. A suitable balance must be found between the two that ensures compliance with the Australian Drinking Water Guidelines (ADWG).

Free chlorine must be kept below 5mg/L for health and 0.6 mg/L for aesthetics (0.2mg/L for some people) in accordance with Table 10.10 of the Australian Drinking Water Guidelines (Version 6) (2004). Values at Googong WTP frequently exceed the aesthetic value, owing to the length of the rising main and associated chlorine decay/demand. This practice is acknowledged by the Drinking Water Guidelines: “In some supplies it may be necessary to exceed the aesthetic guideline in order to maintain an effective disinfectant residual throughout the system.”

Free chlorine levels in the Googong WTP supply have exceeded the ADWG health guideline of 5mg/L (the maximum value reached is 5.4 mg/L). The target dose for Googong WTP is typically 1.6 – 2.0 mg/L but depends on chlorine residual targets, maintenance operations and the water temperature. It should be noted that the free chlorine residual has only exceeded the maximum health guideline of 5mg/L on one occasion between February 2005 and May 2009. Therefore, the 5.4mg/L has been considered an outlier and has been removed from the study.

B.2.1.3 LSI and Water Alkalinity

The Langelier Saturation Index (LSI) is a means of evaluating water quality data to determine if the water has a tendency to form a chemical scale or have a corrosive potential. The LSI is a calculated number used to predict the calcium carbonate stability of water. Essentially this is whether water will precipitate, dissolve, or be in equilibrium with calcium carbonate. Langelier developed a method for calculating the pH at which water is saturated in calcium carbonate. This pH is called the saturation pH, or pHS. The LSI is expressed as the difference between the actual system pH and the saturation pH:

B.2.1.4 LSI = pH – pHS.

If LSI < 0 — The actual pH of the water is below the calculated saturation pH and the water is under-saturated with respect to the calcium carbonate equilibrium. The water is more likely to be corrosive.

If LSI = 0 — The water is in equilibrium with calcium carbonate, and is unlikely to be particularly corrosive or result in the formation of scale

If LSI > 0 — The actual pH of the water exceeds the pHS and the water is super-saturated with calcium carbonate. The water has a tendency to form scale.

(Letterman, 1999)

The LSI is regarded as a straightforward means of evaluating the corrosion potential of water, however, literature suggests there is a potentially poor correlation between the LSI and the rate of corrosion (MWH Water Treatment Principles & Design, 2005). Other indices have been developed over the years including the Ryznar index for flowing systems or the CCPP (Calcium Carbonate Precipitation Potential), to more accurately predict the corrosion or precipitation potential of the water. However; because of the complex nature of the reactions and interactions all indices are suitable only as guides. Therefore, because the LSI is currently measured at Googong and Stromlo WTPs and is the most well known and established index for assessing the water quality it has been applied in this assessment.

The LSI of the water from Googong WTP is in the negative range, reaching levels of -1.23, indicating that it is under-saturated with respect to calcium carbonate and could be mildly corrosive. Generally this can be counteracted by dosing an alkaline chemical. However, the pH of the potable water from Googong WTP reaches a maximum of 8.2 (which meets discharge consent requirements of 6.9 to 8.2) and is frequently at or above pH 7.7 making the dosing of alkaline chemicals into the water supply pipeline unfavourable.

Since the LSI levels cannot be counteracted without raising the pH of the potable water supply, the levels of LSI will need to be managed at Googong WTP. Several chemicals can be considered to add alkalinity however by nature they are themselves all alkaline. Even weak bases such as sodium bicarbonate would raise the pH to some degree and therefore control of water that is already high in pH would be difficult in a network which is remote and has widely varying flows. It is easier to manage this issue at the WTP where processes, buffering and other chemicals are stored rather than in the network where raising the pH would result in another chemical being required. Without treatment, the water supplied to the Googong development may be mildly corrosive therefore discussions should occur with ActewAGL to address the issue in detailed design and ensure there is not significant water corrosion in the Googong new town.

B.2.2 Stromlo WTP Source

B.2.2.1 Stromlo WTP Treated Water Quality

Data for Stromlo was collected between 1 Jan 2005 and 30 June 2009. Data provided is for most days of the month and is consistent over the entire 4.5 year period. Not all of the factors were measured for as consistently as others, as indicated by the "Count" column of Table B-2.

Table B-2 Stromlo WTP Treated Water Quality

Contaminant	Units	Min	Ave	Max	90%ile	COUNT
pH	-	7	7.5	8.2	7.8	459
Chlorine	mg Cl_2/L	0.3	1.3	2.0	1.5	1343
Hardness	mg $CaCO_3$/L	27.9	38.8	76.4	52.8	18
Alkalinity	mg $CaCO_3$/L	18.0	37.2	70.0	45.5	96
Temperature	^oC	5.0	14.7	26.6	20.9	1367
Turbidity	NTU	0.1	0.3	1.8	0.5	454
Total Iron	mg Fe/L	0.02	0.02	0.07	0.02	209
Total Manganese	mg Mn/L	0.001	0.01	0.14	0.022	259
Total Dissolved Solids	mg TDS/L	52.0	70.1	140.0	81.0	47
Conductivity	uS/cm	76	105	200	120	61
Langelier Saturation Index	LSI	-0.67	-1.3	-2.11	-1.73	48

B.2.2.2 Free Chlorine Concentration

The free chlorine concentration of potable water from the Stromlo WTP peaks at 2.0 mg/L to ensure a downstream residual.

B.2.2.3 LSI and Water Alkalinity

Langelier saturation index (LSI) values for Stromlo WTP are more negative than for Googong WTP, reaching a value of -2.11 indicating that the water from Stromlo WTP is mild to moderately corrosive. The pH of the potable water varies between 7 and 8.2 and is frequently at or above pH 7.5. This meets regulatory requirements, but is unfavourable for the dosing of alkaline chemicals. This alkalinity will need to be addressed at Stromlo WTP, as it cannot be properly corrected in the network as described for Googong water supply.

B.2.3 Queanbeyan Off-take Sampling Locations

B.2.3.1 Queanbeyan Sampling Data

Queanbeyan data has been provided in two sets – one set for off take #1 and another set for off take #2. Discussions with staff at ActewAGL have revealed the following information:

- Both off takes are located in Jerrabomberra
- Off take 1 is located on Edwin Lan Parkway
- Off take 2 is located near the old Queanbeyan tip
- Both samples are taken before the secondary chlorine dose

Both sets have been included in the analysis below. The results for each off take show, in some cases, significant variation, possibly owing to the relative location of these off takes on the main water supply pipeline.

Data for Queanbeyan off take #1 is provided in Table B-3 for January 1998 to July 2009. The values from 2001 to 2009 have been collected by ActewAGL on a weekly basis. Prior to this, data was collected approximately twice a month. Not all of the factors listed were determined each time a sample was taken, as indicated by the "Count" column.

Table B-3 Queanbeyan and Off Take #1 Network Water Quality

Contaminant	Units	Min	Ave	Max	90%ile	COUNT
pH	-	7.0	7.6	8.0	7.9	98
Chlorine	mg Cl_2/L	0.01	1.07	2.68	1.55	513
Hardness	mg $CaCO_3$/L	27.5	33.3	37.0	36.7	4
Alkalinity	mg $CaCO_3$/L	6.8	30.6	57.0	50.1	97
Temperature	^oC	7.0	14.5	24.0	20.0	508
Turbidity	NTU	0.1	0.7	1.9	1.2	111
Total Iron	mg Fe/L	0.0	0.1	0.2	0.1	72
Total Manganese	mg Mn/L	0.001	0.007	0.038	0.012	115
Total Dissolved Solids	mg TDS/L	14	69	140	120	96
Conductivity	uS/cm	21	101	210	180	96
Langelier Saturation Index	LSI	-0.59	-1.64	-2.89	-2.40	97

Data for Queanbeyan off take #2 is provided in Table B-4 for January 2001 to July 2009. Data was collected weekly, but not all parameters were measured each week, as indicated in the "Count" column. No data was provided for off take # 2 and therefore discussions should be held with ActewAGL on this matter.

Table B-4 Queanbeyan and Off Take #2 Network Water Quality

Contaminant	Units	Min	Ave	Max	90%ile	COUNT
pH	-	6.9	7.9	8.7	8.4	62
Chlorine	mg Cl_2/L	0.01	0.88	3.16	1.62	454
Hardness	mg $CaCO_3$/L	38.0	41.5	44.9	44.2	2
Alkalinity	mg $CaCO_3$/L	13.3	38.3	62.0	53.8	61
Temperature	^oC	7.0	14.5	24.5	20.0	446
Turbidity	NTU	0.1	0.5	1.9	0.8	62
Total Iron	mg Fe/L	0.01	0.04	0.12	0.09	18
Total Manganese	mg Mn/L	0.001	0.007	0.042	0.016	65
Total Dissolved Solids	mg TDS/L	31	87	140	130	62
Conductivity	uS/cm	46	127	210	190	62
Langelier Saturation Index	LSI	-0.21	-1.13	-2.72	-1.74	59

B.2.3.2 Free Chlorine Concentration

The free chlorine concentration at each of the Queanbeyan off takes reaches a maximum of 2.7 mg/L and 3.2 mg/L for off takes 1 and 2 respectively. As with the data for Stromlo, these values exceed the Australian Drinking Water guideline for aesthetics, but are within the guideline's value of a 5 mg/L maximum for health.

B.2.3.3 LSI and Water Alkalinity

The Langelier saturation index (LSI) values for the Queanbeyan off takes are higher than for the Googong and Stromlo off takes, reaching a peak of -2.89 and -2.72 for off takes 1 and 2 respectively. Water with an LSI of this magnitude is moderately corrosive. As with the data from sampling locations at Googong and Stromlo, the pH of these streams is elevated – peaking at 8 for off take 1 and 8.7 for off take 2 and therefore discussions should be held with ActewAGL on this matter.

B.3 PERFORMANCE TARGETS

B.3.1 Residence Time

Discussions with ACTEW have indicated that up to 10 mega litres (ML) of water could reside in the pipeline (1800mm diameter) between the new connection to the Googong Dam and the Googong WTP itself (i.e. the idle leg). Stage 1 of the Googong development is expected to draw a maximum daily demand of 0.81 ML/d, potentially giving pipe residence times in excess of a week. In addition, water to be supplied to the Googong development will reside in one or more reservoirs prior to reaching Googong. These reservoirs will add to the residence time of the water.

Residence time affects the free chlorine concentration of the drinking water. Greater residence time results in lower levels of residual chlorine as the chemical tends to react with residual compounds. Additionally, chlorine concentration decays at a faster rate under higher temperatures.

A comparison has been made between the Stromlo and Queanbeyan free chlorine concentration data, presented in Figure B-2. The concentration of free chlorine in the water leaving Stromlo WTP is much higher than the concentration in the water received at Queanbeyan. This suggests that the chlorine levels are decreasing in the pipeline between the water treatment plant and the consumers of the water, identifying the need for a sodium hypochlorite dosing system.

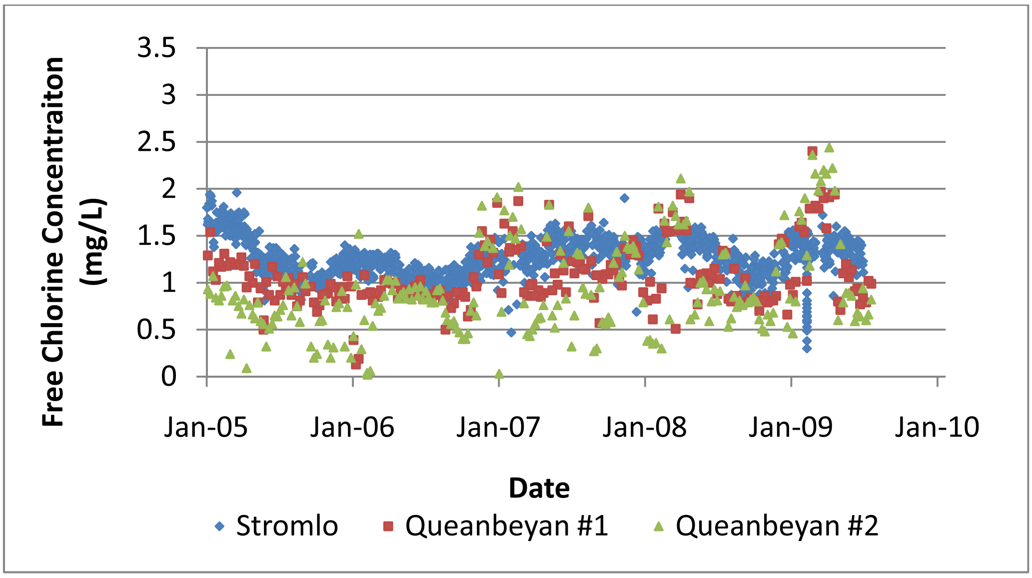

Figure B-2 Stromlo WTP and Queanbeyan Free Chlorine Concentrations

B.3.2 Free Chlorine Comparison

A comparison has been made between the free chlorine concentration in the potable water supplied by Googong and that received at Queanbeyan. The data provided by ActewAGL is presented in Figure B-3. In periods where the Googong WTP is operating, the Queanbeyan data has been removed as the Googong off take will only draw water from the Googong WTP.

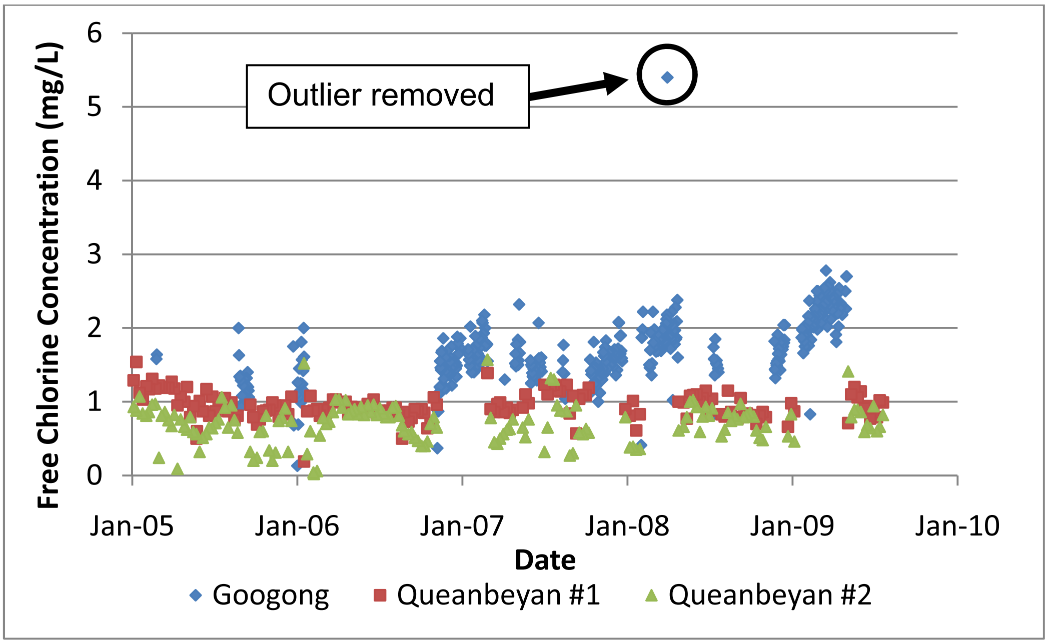

Figure B-3 Free Chlorine Concentration

Chlorine concentrations in the water from Googong WTP are generally higher than the water from Queanbeyan WTP. Incoming chlorine concentrations from Googong WTP have the following characteristics:

- Minimum = 0.1 mg/L
- Average = 1.7 mg/L
- Maximum = 5.4 mg/L

The chlorine concentration is expected to decrease as the water is conveyed from Googong WTP to the Googong development, as the free chlorine will react with residual compounds, a process which is more rapid under higher temperatures. Therefore it is recommended that secondary chlorine dosing is added to maintain acceptable levels of chlorine at times when residual chlorine is low.

The maximum chlorine concentration of 5.4 mg/L was reached only once in all of the data provided and is considered an outlier. All other measured chlorine concentrations were less than 3mg/L, which is within the standards set by the Australian Drinking Water Guidelines (ADWG). As such, a de-chlorination (i.e. sodium bisulphite) facility is not considered necessary.

B.3.3 pH Comparison

The pH of the raw water pH is depicted in Figure B-4 Little difference is observed in the pH of the water leaving Stromlo WTP and that of the water entering Queanbeyan. The water leaving Stromlo is consistently within the discharge guidelines of 6.9 to 8.2. The water received at Queanbeyan has a pH as high as 8.5, which is still within the ADWG values.

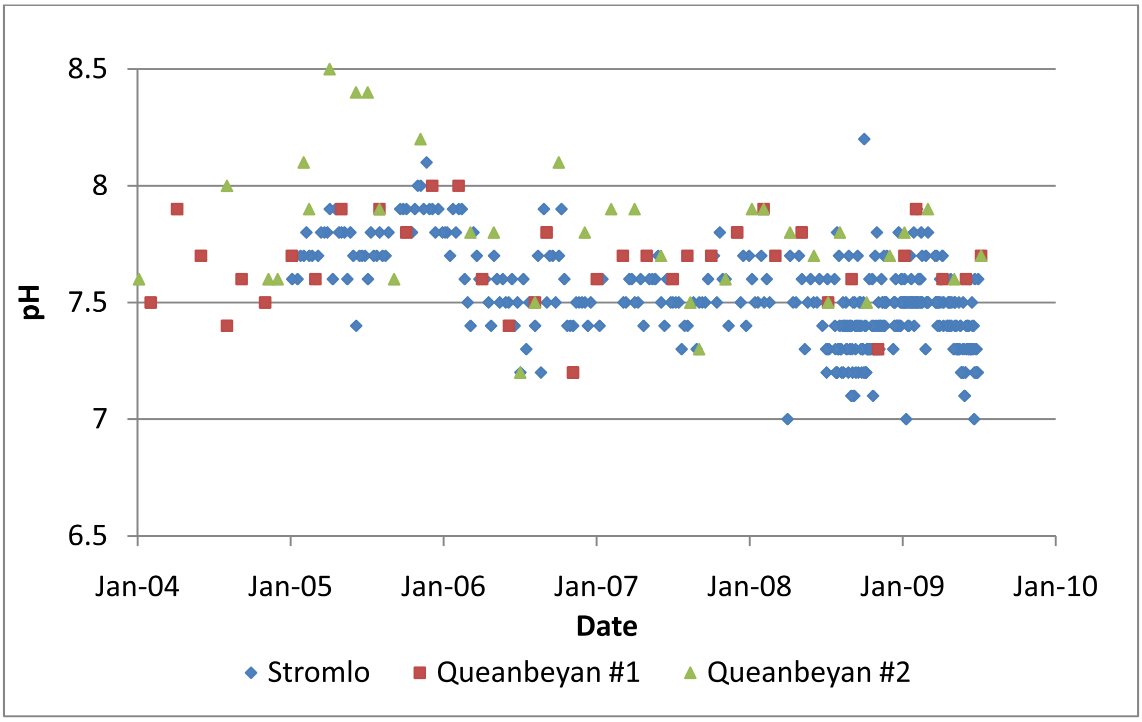

Figure B-4 Water from the Stromlo WTP and Queanbeyan Off-Takes

The pH of the water from Googong WTP varies between 7.3 and 8.2. A pH adjustment is not required for the water feeding the Googong development as the water from the existing water treatment plants is within acceptable pH ranges.

B.3.4 Performance Targets

It is proposed that a chlorine dosing facility be installed on all three service reservoirs, to maintain appropriate drinking water standards for the Googong development. In conjunction with dosing conducted onsite at Googong WTP, the targets presented in Table B-5 should be met with respect to water quality.

Table B-5 Potable Water Quality Performance Targets

Parameter	Minimum Value	Maximum Value	Units	Notes
Free Chlorine Concentration	0.1	2.0	mg/L	ADWGs specify a 5mg/L maximum for health and a 0.6mg/L maximum for aesthetics. Free chlorine is to be adjusted as part of secondary water conditioning.
pH Range	6.9	8.2	-	Current discharge requirement for Googong and Stromlo WTPs. The pH will not be adjusted as part of secondary water conditioning.

The target free-chlorine concentration has been specified to a maximum of 2.0 mg/L which is below the Australian Drinking Water Guidelines maximum of 5 mg/L, but high enough to ensure a residual level of chlorine to maintain disinfection. The target free-chlorine minimum has been set to 0.1 mg/L as a guide for the design of dosing pump turndown.

The pH range has been set to between 6.9 and 8.2 as this is the current discharge requirement for Googong and Stromlo WTPs.

No performance target has been set for the LSI value for the water as MWH recommends this parameter be adjusted at Stromlo and Googong water treatment plants.

B.4 References

Letterman, R.D. (1999) Water Quality and Treatment: A Handbook of Community Water Supplies, 5th Ed., New York: McGraw-Hill

APPENDIX C.
Salt Balance

C.1 Introduction

The proposed development of Googong will have its own designated water recycling plant. This will treat sewage from the new town to produce recycled water to a standard suitable for non potable urban reuse. Water above the requirements from the town will be discharged to a tributary to the Queanbeyan River.

The report "Water Treatment and Reuse Scheme – Preliminary Assessment", (December 2009, Ecowise) gave indicative parameters for discharge including stringent parameters on total nitrogen and phosphorus. A further study has been performed and is documented in "Googong Water Cycle Management, Water Salt and Nutrient Balance Analyses" (July 2009 Agricultural Water Management), which details the total dissolved solids (TDS) limits on soil application.

These two studies have led to a study of the TDS and nitrogen in the system which is detailed in the following report. This examines the build up of these two parameters and therefore the limits of capability of the treatment plant.

C.2 Parameters of Concern

C.2.1 Total Dissolved Solids (TDS)

Total dissolved solids have been examined using the water balance outcomes. The TDS in the system will increase over normal domestic sewage due to the following reasons:

- Common sewage treatment plants are not designed to treat / remove salts. With water recycling, salts could thus build up in the system
- The chemicals added at the WRP to treat the water contribute to dissolved solids.

C.2.2 Total Nitrogen

Total Nitrogen is made up of several main components, shown in .

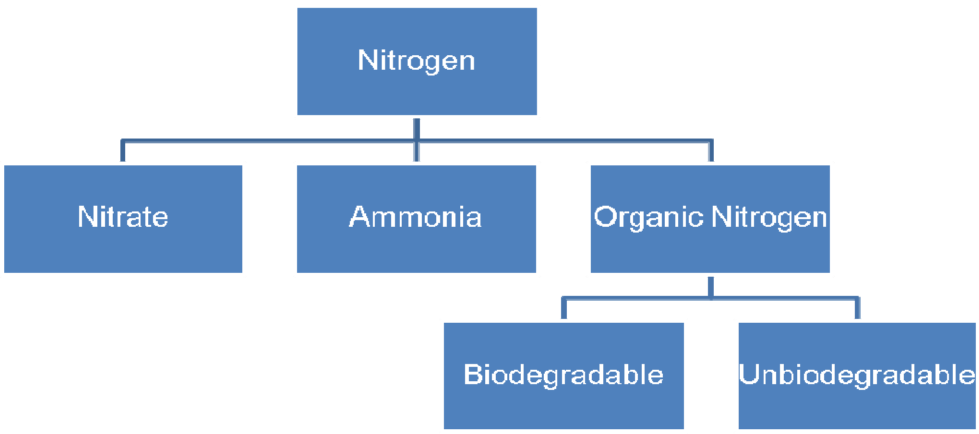

Figure C-1 Nitrogen Fractionation

A brief summary of the fractions is contained below:

- Ammonia in influent to WRP – converted to nitrate in bioreactor
- Nitrate created from oxidation of ammonia, released as nitrogen gas to atmosphere in bioreactor

- Biodegradable organic nitrogen converted to mostly ammonia in bioreactor
- Unbiodegradable organic nitrogen: Particulate fraction is retained by membranes and leaves the system as biosolids, while the soluble fraction will pass through the WRP into recycled water system.

The unbiodegradable soluble fraction will be recycled and therefore a total nitrogen balance needs to be performed to establish the concentrations of unbiodegradable soluble nitrogen as a result of water recycling.

C.3 Mass Balance Results

The mass balance has been done based on the outputs from the water balance for the system.

The time series has been used for the total dissolved solids and a steady state analysis has been completed for the total nitrogen balance. The nitrogen effluent is licensed on a 50 percentile annual basis and is affected by less parameters within the water balance therefore a simplified model can be used.

The assumptions made in the TDS balance are as follows:

- Recycled water produced on one day is used in the next day.
- Ultimate development with rainwater tanks functional.
- Rainwater has same salt content as Potable system
- Googong water quality has conductivity of 150 uS/cm (equivalent to 100 mg-TDS/L) from ACTEW website (April 1997 – July 2009, from Queanbeyan Off-take 1)
- Assume addition of salt to sewer is 135 mg/L in the form of Sodium, Chloride, Sulphate, Calcium, Magnesium and Silica. In addition, a 15 mg/L nitrate residual is generated as a result of the nitrogen transformation balance. A 100 mg/L TDS increase as a result of bicarbonate addition in the catchment minus consumption over the MBR plant.
- Salt addition, due to Ferric Sulphate dosing, is 315kg/d. This takes into account the reduced ferric required given the use of biological phosphorus removal in the bioreactor.
- Magnesium hydroxide addition is 613kg/d

It is important to note that the above assumptions are based on best available information given that no data is currently available for this future WRP. Major results for this salt balance will change if these assumptions are incorrect.

C.3.1 Total Dissolved Solids

Table C-1 TDS Results

	Potable Input (Assumed Constant)	Domestic Input (Assumed Constant)	Irrigation Water	Recycled Water to town	WRP Discharge
Average concentration (mg/L)	100	250	421	581	652
50 percentile concentration (mg/L)	100	250	403	621	648
90^{th} percentile concentration (mg/L)	100	250	538	695	703

The results of the mass balance show that land application is below 450mg/L on an average basis throughout the year. This is through dilution of the recycled water using rainwater tanks and the potable top-up into the system "Googong New Town Concept Design, Site water Balance Assessment" (July 2009, MWH). This has been taken as a daily time series from the volumes generated within this report over 41 years.

The 90^{th} percentile TDS concentration in the discharge from the treatment plant is 703mg/L.

C.3.2 Total Nitrogen

The total nitrogen balance has been done based on steady state calculations. In this case the input is from domestic catchment and removal takes place in the WRP.

Table C-2 Total Nitrogen Results

	Domestic Input	Recycled Water
Ammonia (mg-N/L)	84.3	0.3
Nitrate (mg-N/L)	0.0	3.0
Unbiodegradable soluble N (mg-N/L)	2.6	2.6
Unbiodegradable particulate N (mg-N/L)	12.8	0.0
Biodegradable organic fraction (mg-N/L)	28.1	0.1
Total Nitrogen (mg-N/L)	128	5.9

This shows that, based on the recycle and wastage rates from the model, the unbiodegradable nitrogen in the system will reach equilibrium at approximately 3 mg-N/L.

Normal domestic sewage to achieve effluent total nitrogen of 5 mg-N/L would be made up of

- Ammonia: 0.5 mg-N/L
- Nitrate: 3 mg-N/L
- Residual organic nitrogen: 0.5 – 1.5mg-N/L

Due to the water recycling process, an additional 2mg-N/L of soluble inert nitrogen will exist within the effluent at Googong.

It is important to note the nitrogen which is contained within the discharge is non-biodegradable, it will not react within the water and cause pollution as lower quality effluents would. Therefore it is recommended that a less stringent discharge limit for nitrogen is obtained from the DECCW considering overall benefits to the environment. It is noted that exceedingly stringent effluent limits could lead to excessive consumption of chemicals, and additional power requirements to operate the plant.

C.4 Conclusions

The TDS to land application is under 450mg/L as recommended by Agricultural Water Management due to dilution with rainwater tanks and potable top-up. The average discharge concentration from the plant is in under 700mg/L.

There will be higher levels of non-biodegradable TN in WRP plant flows due to the high recycled water content in the influent. The plant will be unable to remove this non-biodegradable fraction. Therefore we recommend a TN limit of 10:15 for Googong. A tighter licence of 5:7 would be very difficult to achieve because of this high non-biodegradable fraction. A more realistic TN concentration would be 7-10 mg/L. It should be noted that the higher effluent nitrogen concentration will not have a greater adverse impact on the environment as the additional material is inert.

It is suggested that the following parameters are suitable for discharge

Table C-3 Indicative Effluent Quality Results

Concentration (mg/L)	50 percentile	90 percentile
BOD	5	10
TSS	10	20
TDS	650	700
Total Nitrogen	10	15
Total phosphorus	0.2	0.5

There is some scope to reduce the TDS concentration by increasing the total phosphorus requirements which could be discussed with DECCW. Ferric Sulphate addition (for phosphorus removal) is a major contributor to the TDS and if this can be reduced then the TDS will be reduced. The use of biological phosphorous removal is currently limiting the requirement for Ferric Sulphate, yet not eliminating it.

It is recommended that a load based licence for TDS in particular is discussed with the DECCW as although concentrations are high, the discharge to the water course is intermittent.

C.5 References

“Googong New Town Concept Design, Site water Balance Assessment” (July 2009, MWH)

“Googong Water Cycle Management, Water Salt and Nutrient Balance Analyses” (July 2009 Agricultural Water Management),

“Water Treatment and Reuse Scheme – Preliminary Assessment”, (December 2009, Ecowise)

APPENDIX D.
WRP Odour Control Preliminary Assessment

D.1 Introduction

This report details a review of available odour control equipment and options for the Googong WRP, and makes recommendations for the preliminary odour management strategy for the plant. This report includes an estimate of foul air flow and load sheets for the new WRP at each stage of its construction and preliminary design and sizing a potential odour treatment facility for the new WRP.

D.2 Requirement for Odour Control

Odour assessment and dispersion modelling has been carried out to provide the foundation for this concept design. The dispersion modelling completed previously is summarised in Googong New Town WRP Draft Odour Impact Assessment (MWH 2009). This modelling determined which WRP areas require coverage and odour extraction. The purpose of this section is to provide a summary of the engineering requirements for the WRP to achieve NSW EPA compliance.

Because the WRP is progressively constructed in stages to match the rate of development, odour dispersion modelling has been completed at two development horizons, corresponding to a partly completed (4,700 EP) and fully completed WRP (18,849EP). At each horizon a baseline model run of the WRP without odour control has been compared to the WRP inclusive of odour control.

Table D-1 Dispersion Modelling Summary

Model no	Model description	Phase	NSW EPA Compliant ?	Areas covered and odour controlled
1	Googong WRP Baseline Model without Odour Control	1	✗	N/A
2	Googong WRP Phase 1 with Odour Control	1	✓	Preliminary treatment area, secondary treatment area (anoxic & membrane tanks), sludge treatment facilities.
3	Googong WRP Phase 2 Baseline Model	2	✗	Phase 1 preliminary treatment area, secondary treatment area (anoxic & membrane tanks), sludge treatment facilities.
4	Googong WRP Phase 2 with Odour Control	2	✓	Preliminary treatment area, secondary treatment area (anoxic & membrane tanks), sludge treatment facilities.

The odour dispersion modelling showed that in order for the Googong WRP to ensure compliance with the NSW EPA requirements the following areas of the plant require coverage and odour control:

Table D-2 WRP Process Units to be Covered for Odour Control

WRP Area to be Covered	Typical Equipment
Preliminary Treatment Area	• Receiving chambers for discharging rising mains • Screens • Screenings handling and storage bin • Grit removal • Grit classifier and storage bin
Secondary Treatment Plant Anoxic and Anaerobic Zones	• Biological reactors
Secondary Treatment Plant Membrane Tank	• Membrane tanks
Sludge Digesters	• Aerobic digesters
Sludge Dewatering and Thickening	• Rotary Drum Thickener (RDT) • Dewatering centrifuge or belt-filter press.
Sludge Storage	• Covered sludge storage bin

D.3 Foul Air Flow and Load Development

D.3.1 Introduction

The development of foul air flow and loads is typically the culmination of on -site odour characterisations (of similar existing WRP's), septicity modelling and a review of the MWH odour emission database and odour dispersion modelling.

The potential foul air concentrations have been based upon MWH knowledge and experience in odour control treatment plant designs to develop estimates of foul air flows and loads for the Googong WRP.

D.3.2 Level of Design

The negative pressure required to prevent fugitive emissions is a function of cover size, type and wind loading. The magnitude of the negative pressure which can be sustained under odour control covers is a function of:

- Negative pressure provided by the odour control system fan at the extraction point.
- Number of extraction points, and dispersal over the cover.
- Type of cover, number of cover joints, penetrations and cover flexibility which define the leakage rate.

- Air inlet configuration, e.g. provision of weighted air inlet dampers or grills.
- Degree of interconnection of process areas under the covers allowing inter-process distribution of air.

If a design negative pressure of -150Pa is assumed at the extraction point, then the negative pressure sustained is dependent upon the free area available for air ingress from the atmosphere (or other process area), and the gas flow rate through that space. As the free area is decreased, the gas velocity, and therefore pressure drop is increased, leading to a higher negative pressure sustained under the covers.

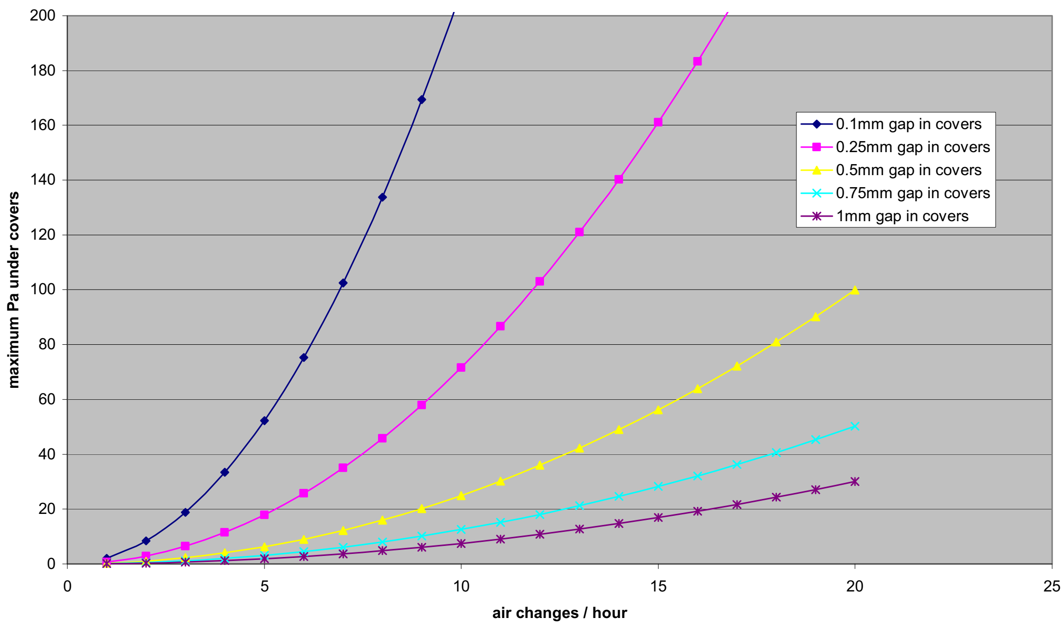

Figure D-1 Example of Achievable Negative Pressure for Varying Air Exchange Rates and Cover Seals for a Hypothetical Covered Tank

Figure D-1 shows the approximate relationship between maximum negative pressure that can be achieved under covers versus air exchange rate, derived from standard head loss equations. A hypothetical 10m by 5m rectangular covered structure with a 2m headspace is considered, with one number 100mm diameter air inlet. The covers are comprised of 10 No 1m by 5m FRP panels. Achievable negative pressure is shown with varying gaps between the cover sections.

The achievable negative pressure is dependant upon both the ventilation rate and the gaps between the cover sections, with a high sensitivity to the gaps between the cover sections and the number of penetrations within the cover. The higher the free area between the atmosphere and the tank air space volume, the higher the extraction required.

A static negative pressure of 25Pa under covers is generally accepted to achieve an odour capture rate of 99.9%. It should be noted that 0.1% of odour is still released from the cover, even at this sustained negative pressure, and should be included in any dispersion model.

Historically, in order to reduce the air infiltration to a level which enables a realistic extraction rate to be used (whilst still achieving -25Pa static pressure under the cover), the following is required;

1. Foam or rubber gasket seals are required between covers and concreate tank walls (and the seal held under compression by suitable bolt fixings).
2. Pipe penetrations must be carefully considered and be installed with effective collar seals.
3. Covers not requiring removal should be gas-sealed with a flexible sealant such a “Fosroc” product.
4. Weighted air inlet dampers to maintain a constant negative pressure under the covers during varying flow conditions should be employed.

D.3.3 Covered Enclosures Extraction Rates

Ventilation rates are selected for buildings and process units, taking into account the following factors:

- Air exchange rate to achieve suitable negative pressure to minimise odour release due to wind wake and solar effects.
- Suitable working atmospheres i.e. not creating an area which is a health or safety risk.
- Minimisation of corrosion rates.
- Preventing escape of fugitive emissions.
- Providing an acceptable operating environment for staff.

In deriving the foul air extraction rates for the areas requiring coverage at Googong WRP these criteria have been used together with MWH experience in odour control plant design.

D.4 Googong WRP

The odour control will be constructed in stages to correspond with the increasing size of the WRP plant as each construction stage is developed. For each of these stages there will be a corresponding foul air flow requiring treatment. This foul air flow may change, not only in quantity, but also in composition throughout the assessment period as different process units are brought on and off-line.

The key part of the flow and loads derivation exercise is to ensure the provision of adequate foul air treatment capacity. Therefore changes in foul air flows and loads have been considered to ensure sufficient required equipment turndown and potential peaks in contaminant concentrations are identified and incorporated into the designs completed at this stage.

D.4.1 Foul Air Flows

A summary of the development of the foul air flows for the Phase 1 and 2 Googong WRP is provided in Table D-3 and

Table **D-4** respectively. Only areas requiring coverage and odour extraction are included.

Table D-3 Summary of Stage 2b (4700 EP) WRP Process Equipment Sizes & Foul Air Flows

ASSESSMENT HORIZON	COVERED PROCESS EQUIPMENT	VOLUME OF PROCESS EQUIPMENT (m^3)	AIR CHANGES PER HOUR	EXTRACTION RATE (m^3/h)
Phase 1	PTA[1]	116	20 to 25	1,908
	Bioreactor Tank[2]	Based upon air demands		2,700
	Membrane Tank	Based upon air demands		2,954
	Sludge Treatment	90	6 to 25	596
	Total Foul Air Flow to be Treated in Central Odour Control Facility			8,167
	Design Airflow			**8,200**

Note 1. Phase 1 PTA sizes have been taken from supplier details for screens, grit, washing and classification facilities.
Note 2. Secondary treatment area footprints have been taken from the process calculations.

Table D-4 Summary of Stage 4 (18,849 EP) WRP Process Equipment Sizes & Foul Air Flows

ASSESSMENT HORIZON	COVERED PROCESS EQUIPMENT	VOLUME OF PROCESS EQUIPMENT (m^3)	AIR CHANGES PER HOUR	EXTRACTION RATE (m^3/h)
Phase 2	PTA[1]	116	20 to 25	1,908
	Bioreactor Tank[2]	Based upon air demands		10,833
	Membrane Tank	Based upon air demands		5,909
	Sludge Treatment	90	6 to 25	1,034
	Total Foul Air Flow to be Treated in Central Odour Control Facility			19,684
	Design Airflow			**19,700**

Note 1. Phase 1 PTA sizes have been taken from supplier details for screens, grit, washing and classification facilities.
Note 2. Secondary treatment area footprints have been taken from the process calculations.

D.4.2 Foul Air Flow and Loads

Foul air loads for potential odour control facilities at the Googong WRP have been developed by considering the following sources of information:

- A review of the MWH Global Odour Emission Database.

For each of the process areas requiring coverage and odour control, potential foul air contaminant concentrations have been selected for the following compounds which (based upon MWH experience) are expected to be present in the air to be extracted.

- Hydrogen Sulphide (H2S);
- Mercaptans (R-SH);
- Ammonia (NH3);

- Volatile Organic Compounds (VOC's);
- Dimethyl Sulphide (DMS).

The odorous compounds concentrations selected for each of the process units (i.e. Inlet Works, bioreactors etc) have been combined to form an overall load sheet for the Stage 1 and Ultimate Stage assessment horizons.

It should be noted that each WRP is individual in its odour generating potential. Different sewer networks, process units (and their mode of operation) all act to generate (and/or convey) odorous compounds in varying magnitudes.

Summary load sheets for Googong WRP, at the Phase 1 and 2 horizons, are presented in Table D-5 and

Table D-6. The objective of the sheets is to enable the selection and sizing of odour control equipment.

Table D-5 WRP Stage 2b (4700EP) Foul Air Flow and Load Sheet

ODOROUS COMPOUND	AVERAGE (ppm)	MAXIMUM (ppm)
Hydrogen Sulphide (H_2S)	8.2	21.7
Mercaptans (R-SH)	0.8	1.6
Ammonia (NH_3)	0.4	1.1
Volatile Organic Compounds (VOC's)	5.5	15.6
Dimethyl Sulphides (DMS)	2.0	4.3
Design Airflow[1] = 8,200m^3/h		

Note 1. Only foul airflows requiring odour control treatment have been stated.

Table D-6 WRP Stage 4 (18,849EP) Foul Air Flow and Load Sheet

ODOROUS COMPOUND	AVERAGE (ppm)	MAXIMUM (ppm
Hydrogen Sulphide (H_2S)	4.0	20.0
Mercaptans (R-SH)	0.6	1.3
Ammonia (NH_3)	0.3	0.7
Volatile Organic Compounds (VOC's)	5.3	13.6
Dimethyl Sulphides (DMS)	1.4	2.9
Design Airflow[1] = 19,700		

Note 1. Only foul airflows requiring odour control treatment have been stated.

D.5 Odour Capture

D.5.1 Introduction

An Odour Management Strategy (OMS) for the Googong WRP has been developed. The purpose of the OMS for Googong is to provide a clear way forward on how, with the expected population growth, it can achieve compliance with the NSW EPA odour nuisance guidelines. This is achieved through the completion of the following objectives:

- To provide details of potential odour capture and coverage techniques together with typical odour control technologies.
- To select, based upon MWH experience in odour control treatment technologies and their application, the most suitable odour treatment technology for Googong WRP,

A general discussion is provided, followed by the details required for each aspect of the Googong WRP. The OMS includes the following details:

1. Presentation of an odour management implementation strategy and identification of its expected phases.

2. Details of the potential odour treatment technologies for each assessment horizon.

Effective coverage and foul air extraction at Googong WRP is paramount to achieving compliance with the NSW EPA no nuisance odour guidelines. This section of the OMS emphasises the importance of effective cover capture and discusses the types of covers typically applied at sewage treatment works.

There is a common theme of coverage between select areas at the WRP involving the requirements to cover PTA's and select areas of secondary treatment facilities. Therefore the focus of this assessment is to provide details of the potential covers available for these areas.

D.5.2 Cover Capture Efficiencies

Prior to discussing available cover types a summary of effective cover capture efficiencies is provided within this section.

In determining the extent of coverage required at Googong WRP an assessment has been completed to establish the process units requiring covers and their required capture efficiency.

The types of covers available for the different areas of the WRP that require containment (and their close fitting ability) dictate the level of negative pressure that can be achieved underneath them. The achievable level of negative pressure can then be related to the percentage capture efficiency of the selected covers. Capture efficiencies of 99.9 to 100% equate to a required negative pressure of 15 to 30 Pa (Cadee et al, 2007). Therefore it can be assumed that for a cover arrangement capable of sustaining a 15-30 Pa negative pressure, a capture efficiency in excess of 99.9% can be achieved.

For different types (and installation) of covers, the achievable negative pressure varies, thus affecting the percentage capture efficiency. A review of the capture efficiencies of odour control covers installed at STW in Perth, Australia is presented in the literature by Cadee et al. A summary of the findings is presented in Table D-7.

Table D-7 Capture Efficiencies for Cover Subject to Wind Forces*

Static negative pressure (-pa)	Estimated capture efficiency (%)
< 5.0	95.0
5.0 – 10.0	95.0 – 99.0
10.0 – 15.0	99.0 – 99.9
15.0 – 30.0	99.9 -100.0

**After Cadee et al, 2007, for a project in Perth.*

Based upon the odour dispersion modelling completed for the WRP the following capture efficiencies (and corresponding negative pressures) were derived:

- Inlet Works = Minimum capture efficiency of 99.9% (15.0 – 30.0 Pa negative pressure required).
- Secondary Treatment Areas = 100% capture efficiency required (30.0 Pa negative pressure required).

D.5.3 Inlet Works Areas

Inlet works often give off some of the most offensive and strong odours at sewage treatment plants, and the following cover types are frequently applied successfully for Inlet Works:

- Aluminium covers;
- Concrete covers; and,
- Fibre Reinforced Plastic (FRP) covers.

D.5.3.1 Aluminium Covers

Aluminium covers are often used at Inlet Works because they are lightweight. They can be machined to seal complex structures (such as those with mechanical equipment) effectively if correctly fashioned and applied. Aluminium covers are largely corrosion-resistant, moderately expensive, and must be machined-to-measure. They would be suitable for use at Googong WRP.

An example of flat trafficable Aluminium covers is shown in Figure A8-3. These are frequently used in the covering of sewage channels, and other areas that do not require routine maintenance. Experience dictates that the overall capture rate of this type of Aluminium covers (if correctly installed and sealed) can be in the range of 99 to 100%. Therefore their application is recommended as suitable for Googong WRP.

	
Figure D-2 Example Aluminium Inlet Works Covers	**Figure D-3 Gas Assisted Aluminium Covers**

An example of gas-lift assisted aluminium covers in screenings areas in shown in Figure D-3. These types of covers are frequently applied prior to the screening facilities to allow for easier maintenance access. Experience dictates that the overall capture rate of this type of Aluminium covers (if correctly installed and sealed) can be in the range of 99 to 100%. Therefore their application is suitable for Googong WRP.

D.5.3.2 Concrete Covers

Figure D-4 shows that concrete covers have been used for the pre screen channels in the example Inlet Works. Concrete covers would provide the best seal of all of the potential cover options; however, due to their considerable weight, they make access extremely difficult. Experience dictates that the overall capture rate of concrete covers is generally 100%. Therefore their application is recommended as suitable for the Googong WRP in selected areas.

D.5.3.3 Fibre Reinforced Plastic Covers

FRP covers are used extensively at other STP's throughout NSW (i.e. Wollongong STP) and Queensland (i.e. Merrimac STP). They are lightweight, can be manufactured to seal complex structures and arrangements, and can be sealed under compression with suitable gaskets to ensure an excellent seal (often between 99 to 100% capture rate). An example of flat trafficable FRP covers at an existing inlet works is shown in Figure D-4.

These covers are highly corrosion-resistant; however they must be made to measure and are moderately expensive. Their application is recommended as suitable for Googong WRP.

Figure D-4 Flat Trafficable FRP Inlet works Covers

Figure D-5 Weighted Air Inlet Dampers

D.5.3.4 Air Inlet Systems

In order to prevent fugitive odour emissions, a sustained negative pressure under odour control covers is required.

The negative pressure obtained is dependant upon the pressure drop between the free atmosphere and the area beneath the covers, and the negative pressure at the extraction point. This pressure drop is a function of the free cross sectional area, the foul air extraction rate, and thus the gas velocity between the atmosphere and the enclosed area. The free cross sectional area can comprise of; (1) the air inlets (2) gaps between cover sections or around hatches and (3) gaps around cover penetrations (e.g. probes, cables etc).

In an ideal world, the covers would remain completely sealed and the air extraction rate will remain constant, however experience with numerous systems throughout the world has shown that this is not the case. Two options can be discussed further:

Option 1: Variable air inlet "holes"

For this option, the air holes are set on commissioning to achieve the design negative pressure at the extract rate at that time. This is done by closing the air holes to restrict the free cross sectional area manually until the design negative pressure under the covers is achieved with the design flowrate.

In the case where the free cross sectional area increases by gaps opening up in the covers, or the air extraction flow rate from the covered section varying, the air inlet holes will have to be re-set manually to re-achieve the design negative pressure.

Option 2: Automatic variable weighted air inlet dampers

For this option, weighted air inlet dampers are installed. These dampers are designed to be fully open at a differential pressure of (say) -50Pa. They open the required amount automatically to sustain this differential pressure.

In the case where the free cross sectional area increases by gaps opening up in the covers, or the air extraction flow rate from the covered section varying, the dampers close/open to restrict the cross sectional area automatically. This is done to the extent required to sustain the same differential pressure, thus the same negative pressure under the covers. This is performed automatically by the physical weight of the damper blades. Experience has shown that there is no re-calibration required.

An example of a weighted air inlet damper system is shown in Figure D-5, and their application is recommended as suitable for Googong WRP.

D.5.3.5 Screenings And Grit Handling Equipment Covers

Other inlet works areas frequently requiring coverage and odour extraction are screens washers, compactors and storage bins (including grit storage bins). This type of equipment is often vendor supplied, and care must be taken in ensuring adequate odour containment is provided. Close covering techniques have been successfully applied on previous sites. An example of this is shown in Figure D-6, where vendor supplied flange connections on screenings bins and conveyors are provided for foul air extraction.

D.5.3.6 Summary Of Potential Inlet Works Covers

A summary of the potential inlet works areas covers for foul air containments is provided in Table D-8.

Table D-8 Summary of Potential Inlet Works Covers

Cover types	Advantages	Disadvantages
Fixed Aluminium Covers	• Could provide cheaper alternative to GRP. • Will ensure air tightness and will meet air pressure and air change design values. • Expected odour capture of 99 to 100% (assuming correct installation ands seals).	• Can require coating to protect Operators from hot panels. • Could be heavy if pedestrian access is required. • Access to tanks only via pre-determined access hatches.
Mechanically Liftable Aluminium Covers (Gas Assisted)	• Frequently preferred by Operations; • Allows visual and physical access to all equipment and parts of reactor. • Expected odour capture of 99 to 100% (assuming correct installation and seals).	• May not give adequate seal due to gap between covers; • Pedestrian access/environmental impact could warp covers meaning seals do not work adequately.
Concrete Covers	• Will ensure air tightness and will meet air pressure and air change design values. • Expected odour capture of approx 100% (assuming correct installation).	• Increased areas of concrete requiring corrosion barrier coating. • Will mean access to tanks/channels only via pre-determined access hatches.
Fixed FRP Covers	• Widely used worldwide. • Strong and lightweight. • Will ensure air tightness and will meet air pressure and air change design values. • Expected odour capture of 99 to 100% (assuming correct installation).	• Can be prone to UV damage, will need adequate protective coating. • Will mean access to tanks/channels only via pre-determined access hatches.

Continued access at the Inlet Works is also considered to be essential, therefore correctly installed aluminium or FRP covers (with gas assisted access hatches) are considered to be the most suitable options for sealing the Googong WRP.

Positive experiences using FRP covers at the Merrimac STP (and other STP's), together with its slightly superior corrosion-resistance to aluminium drives the design towards FRP for the Inlet Works. Aluminium or a combination of aluminium and FRP covers would be ideal for sealing the mechanical equipment and channels.

D.5.4 Secondary Treatment Areas

Due to the sizes of the secondary treatment zones at the WRP, and covers span required, the types of applicable covers are reduced and the following cover types may be applicable to Googong WRP.

- Flat aluminium covers (Figure D-7)
- Arc span FRP covers. (Figure D-8)
- Cast concrete covers.

Figure D-6 Vendor Close Covered Screening Bins

Figure D-7 Fixed Aluminium Flat Cover Arrangements

Figure D-8 FRP Arc Span Anaerobic and Anoxic Zone Covers

A survey of US sites completed for previous odour management projects indicated that aluminium covers with fixed external trusses are used extensively. That many were located very close to residential developments proves that their seal is good. Their low headspace is also a considerable advantage (i.e. reduced foul air volume to be extracted).

Concrete covers, although effective in odour capture, have a detrimental impact to access and can be difficult to retrofit to existing structures (due to their weight impacting upon existing structural integrity). This has particular importance for Googong WRP, as a 4 or 5 stage Bardenpho process is the preferred process configuration. This type of process inherently comes with numerous mixed liquor recycles together with mixers to keep a homogenous wastewater profile. Access to these mixers is restricted with any covering arrangement; however, access difficulties could be further compounded by concrete covers.

Arc span FRP covers have been applied to numerous existing plants in QLD, WA & NSW to provide successful odour containments. Although the arc span nature of the cover generates a greater foul air volume that a flat cover, they can be sealed under compression with suitable gaskets to ensure an excellent seal. This type of cover has therefore been assumed to be applied for the secondary treatment areas.

D.5.5 General Cover Requirements

General design requirements for the covers shall include but not limited to:

- Cover and gaskets are to be fitted under compression to ensure adequate gas seal.
- Covers to be suitably fixed to the infrastructures and shall include lifting points to enable removal for maintenance.
- Cover concrete loading distance should be at least 250mm.
- Fixing of covers shall be by grade 316 stainless steel bolts chemically anchored into concrete.
- Hardware and fixing to hatches shall be grade 316 stainless steel.
- All removable covers to be designed for a maximum lifting weight of 15kg.
- The design temperature range is ambient conditions and consequential temperature affects.
- Any special tools for cover removal are minimised.

D.6 Potential Odour Treatment Technologies

Two options for treatment have been considered for Googong WRP:

- Wet Chemical Scrubbing Facility
- Biotrickling Filters & Activated Carbon Polishing

D.6.1 Wet Chemical Scrubbing

D.6.1.1 Packed Towers

Wet chemical scrubbers are designed in accordance to the requirements of mass transfer with respect to absorption theory. To remove and/or oxidise a compound present in the air stream (the gas phase), the compound must be first absorbed into the scrubbing liquid (the liquid phase). As a result, wet chemical scrubbing systems can treat only compounds that are soluble within the liquid phase. Solubility is dependent upon a number of factors including pH, temperature and Henrys constant (which in turn is dependent upon the ionic strength and temperature of the liquid amongst other factors).

The absorption of a given compound from the gas to the liquid phase is dependent upon mass transfer driving forces and its solubility. A high gas phase concentration (relative to the liquid phase), or a fast irreversible liquid phase reaction (which would decrease the liquid phase concentration) can both increase the driving force.

H2S is usually the most common compound that is targeted for removal within a wet scrubber treating air from a wastewater treatment plant. When dissolved in the liquid phase, the equilibrium ionisation reactions for H2S are as follows:

$$H_2S_{(aq)} \leftrightarrow H^+_{(aq)} + HS^-_{(aq)} \leftrightarrow H^+_{(aq)} + S^{2-}_{(aq)}$$

The relative proportions of H2S, HS-, and S2- are pH dependent. At pH 7, approximately 50% of the dissolved sulphide will be in the form of H2S. However at pH >9 more than 99% will be present as HS- or S2-. Therefore in scrubbing solutions operated at pH >9 the driving force for H2S absorption is much greater than when operating at pH 7, as there is a lower concentration of dissolved, unionized H2S within the liquid phase (this does not consider the implication of oxidation reactions on mass transfer rates).

Sodium hydroxide (caustic) is dosed into the recirculating stream of the scrubber to raise the pH to increase the transfer of H2S from the gas to the liquid phase.

The use of an oxidising agent within the recirculating liquid of a wet chemical scrubber enhances mass transfer by oxidising the dissolved species (in this case HS- and S2-) as they enter the liquid phase. If the HS- and S2- are removed from solution by oxidation, the equilibrium shifts, increasing the driving force, and therefore maintaining a high mass transfer driving rate. This is provided that there is a fast irreversible reaction in the liquid phase.

In addition to hydrogen sulphide, other compounds (particularly reduced organic sulphur compounds typically identified as mercaptans and dimethyl sulphide) maybe present in the gas stream. These compounds are highly odorous and need to be removed within the odour control system to meet stack discharge requirements. Although these compounds may not be present in the same concentration as hydrogen sulphide, they can have a much lower detection threshold. As a result, reduced organic sulphur compounds can make up a significant proportion of wastewater plant odour.

Organo-sulphides are typically associated with sewage sludge storage and treatment. These compounds are not routinely sampled for by plant operators and, unlike the extensive research carried out on hydrogen sulphide, there has been limited research undertaken on the formation and rate of production of these compounds. Compounds such as mercaptans and dimethyl sulphide are much less soluble than hydrogen sulphide at pH 9, and have more complex oxidation pathways. This results in such compounds requiring a greater stoichiometric quantity of oxidising agent to achieve complete oxidation. In some cases side reactions or partial oxidation can generate more odorous by products. As a result of solubility and reaction pathways, they represent a greater challenge for removal via convention wet chemical scrubbers.

Consequently conventional wet chemical scrubber using hypochlorite can encounter a number of difficulties:

1. The rate of oxidation reaction between hypochlorite and certain compounds (especially organo-sulphides) can be slow, requiring long sump residence times (larger unit sizing), leading to increased capital cost. Additionally conventional systems will need to be operated with high hypochlorite concentrations, which can lead to chlorine odour carry over (which can potentially damage downstream equipment and create additional odour) and higher operational costs.

2. Hypochlorite is non selective, and will readily react with a wide range of compounds including amines and phenols potential leading to the production of new odours.

3. The oxidation reactions between hypochlorite and hydrogen sulphide and organo-sulphide can take numerous pathways, leading to 'excessive' chemical consumption.

D.6.1.2 Odorgard Process

The 'Odorgard' process has been developed to resolve these issues. A description of a wet chemical scrubber with the Odorgard process is as follows:

Foul air enters a packed tower and is contacted counter-currently with the scrubbing liquor (a solution of sodium hypochlorite stabilised with sodium hydroxide) over the mass transfer media. Hydrogen sulphide, dimethyl sulphide, mercaptans and other odorous compounds are absorbed into the liquid phase.

Liquor saturated with odour compounds collects in the column sump where slow oxidation of the compounds takes place. The liquid leaving the sump is passed through the down-flow Odorgard vessel. The vessel contains a fixed bed nickel oxide based catalyst that substantially increases the oxidation rate of the absorbed species.

The Odorgard catalyst decomposes the hypochlorite ion, producing an intermediate metal super oxide, releasing the chloride ion. Dissolved compounds are adsorbed onto the catalyst where the metal super oxide (which is highly reactive) reacts with the compounds and is reduced back to its original state.

The liquor leaving the Odorgard vessel is then dosed with sodium hypochlorite and sodium hydroxide solution to replenish liquor concentrations after the required oxidation reaction has occurred. This occurs prior to the liquor re-entering the packed column.

A schematic of the wet chemical scrubbing process is displayed in Figure D-9.

The schematic indicates that foul air is to be drawn through the treatment stages by fans located pre discharge stack. This arrangement ensures that the treatment stages are held under a negative pressure during normal operation, minimising the risk of odour leakage, health and safety impacts, and improving maintenance access requirements.

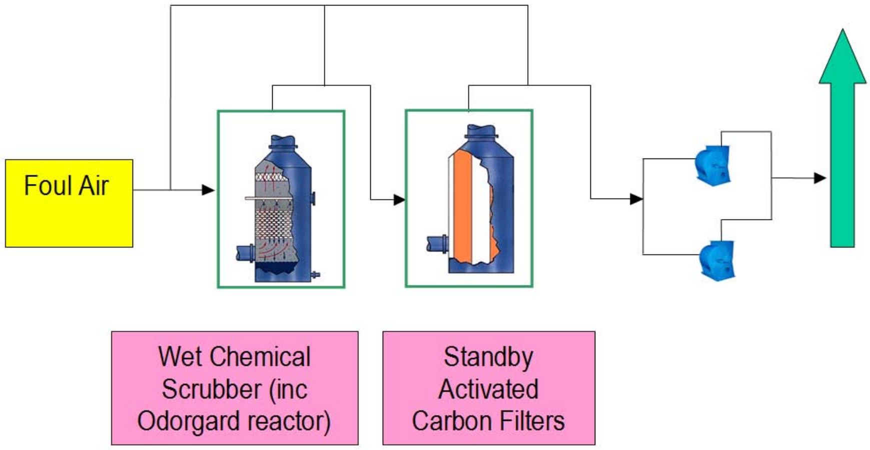

Figure D-9 Wet Chemical Scrubbing and Standby Activated Carbon Process Schematic

D.6.1.3 Advantages And Disadvantages

The advantages of chemical scrubbers are as follows:

- Able to provide odour removal of >99.9%.
- Chemical scrubber technology is well known and proven and many water utilities have extensive experience in their application.
- Chemical scrubbers can cope with a range of both acid and alkaline gases –depending on the scrubbing liquor and number of treatment stages - and can respond quickly to changes in odour load. Odour loads containing a mix of acid and alkaline gases will usually be scrubbed in separate sections/towers mounted in sequence.
- Operating parameters of scrubbers (i.e; pH, redox potential) can easily be altered depending on the changes that may be required over time.

The disadvantages of chemical scrubbers are as follows:

- The scrubber installation will require storage of dosing chemicals on site. These chemicals are to be held in Dangerous Goods stores as defined by Work Cover Regulations. Aside from the potential hazards of storing these chemicals on an unmanned site, and often in areas considered a public amenity, chemicals training of operational personnel would be required for those who may be in contact with the scrubber plant.
- Although wet chemical scrubbers are usually fitted with automated control systems, it is not a practice to operate this type of plant without regular visits by personnel (say, once or twice a shift).

D.6.2 Biotrickling Filters and Activated Carbon

These odour control devices utilise a biomass system whereby foul air is passed through a medium that supports a bacterial culture (the biomass). The airflow passes through the medium, where the odorous compounds are absorbed into the liquid film surrounding the biomass. The biomass consumes the contaminants, hence removing these odorous compounds from the air.

Biological filtration of contaminated airstreams have gained popularity in recent times as it is often seen as offering a benign solution to comparable systems such as activated carbon and chemical scrubbers. However, there are mixed responses from plant management across various wastewater authorities as to their success, and depending on selection and application. The type of biological treatment system must be carefully selected and deigned based upon the contaminants in the gas stream (e.g. Autotrophic system, Heterotrophic system, or a combination of both) in order to achieve the design intent.

Biological filtration efficiency is highly dependent on humidity, pH within the media and to the odour load to be treated. Biological filtration systems require ongoing maintenance that has been often neglected by many plant operators. While biological filtration can be an efficient and low cost means of odour control, the continued health of the biomass is important. Dangers to the biomass can come from a range of sources - compounds in the airstream that kill the biomass, changes in biomass pH over time, flooding or compaction of the biomass material over time (causing uneven air distribution), or excessive chlorine contamination of the water feed are some of the problems.

A trickling biofilter unit will appear at first glance to be similar to a chemical scrubber due to the arrangement of the vessel, the packing material and liquor circulation system. However, the trickling biofilter relies on the development and maintenance of a biomass on the scrubber media that is wetted and fed supplementary nutrients from the liquor circulation system. Trickling filters have gained in popularity in recent times due to their ability to perform with the similar advantages of a chemical scrubber system, without the need for storage for dosed chemicals, nor incurring the associated operating costs of these chemicals.

The use of specific bacterial types to provide odour control is not a new idea, and the concept of trickling biofilters has been used previously. However, new media materials for supporting the biomass that have developed in recent years, along with ongoing development of these types of systems has provided opportunities to successfully deploy these types of scrubbers within wastewater processes.

The scrubber construction is typically FRP, with varying types of internal packing media, depending upon supplier. Polypropylene randomly packed, foam squares, structured media and clay balls are all utilised depending upon system type and supplier.

Both recirculation and non recirculation systems are available, each with their own claims for success. The recirculation liquor may require supplement dosing of nutrients for bacterial growth, depending upon load and feed water quality. Typical nutrient chemicals may be ammonium phosphate and urea. Their addition may be only required intermittently.

Liquor operates at approximately pH 2 to 3 for autotrophic type systems and at pH 4 to 7 for heterotrophic systems; pH control is achieved through bleed and the addition of make-up water. Biomass growth at start-up may require up to 10 days before biofilter achieves optimum operating conditions.

Multi layers of media within the vessel allow for low and neutral pH levels so that the range of inorganic/organic sulphides & VOCs can be removed.

A schematic of the biotrickling and activated carbon process is shown in Figure D-10. The schematic indicates that foul air is to be drawn through the treatment stages by fans located pre discharge stack. This arrangement ensures that the treatment stages are held under a negative pressure during normal operation, minimising the risk of odour leakage, health and safety impacts, and improving maintenance access requirements.

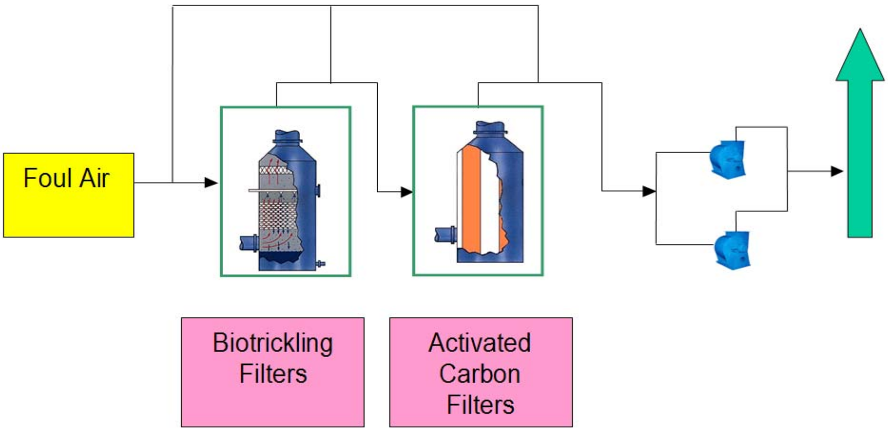

Figure D-10 Biotrickling and Activated Carbon Process Schematic

D.6.2.1 Advantages And Disadvantages

The advantages of biotrickling filter are as follows:

- Able to provide odour removal of 95%,
- Can be supplied and installed on a small footprint in comparison to other Biofiltration processes.
- Provides many of the advantages of a chemical scrubbing process without the need for chemical storage and handling plant.
- Can readily use recycled effluent liquor for make-up in lieu of potable/industrial water.
- The media bed usually provides less pressure drop in comparison to other odour control processes.
- Due to the low pH operating conditions these types of biofilters can handle low loads of ammonia – which in solution can provide nutrients for the biomass.

The disadvantages of a biotrickling filter are as follows:

- The system cannot respond as quickly as a chemical scrubber to sudden and significant changes in odour load.
- The high acidity of the bleed liquor can be a corrosion problem at the point of discharge at a plant. Multiple bed once through systems, although achieving superior VOC, DMS and R-SH removal, can have significant discharges of pH 1.5 liquor. In large systems with high hydrogen sulphide loads, this may require neutralisation prior to discharge in the inlet works to prevent adverse effects on works influent alkalinity.
- The trickling biofilter will require several days to reactivate its biomass if it has been shut down for any period of time.
- The treated air quality, for systems containing high quantities of reduced sulphides (or odorous VOCs) will generally need multiple treatment stages or activated carbon polishing of the discharge gas to achieve 500ou standard.

D.6.3 Performance Summary

Each of the potential technologies, or configurations of them, is required to be able to provide the same degree of treatment. The odour treatment requirements have been refined through the odour dispersion modelling completed, where a maximum discharges of 500 ou was considered for all odour treatment facilities.

Table D-9 Odour Treatment Requirements

Contaminant	Design % removal at peak inlet design load	Maximum discharge concentration at peak load (ppm)	Maximum discharge concentration at peak load
Hydrogen sulphide (H_2S)	>99.9%	0.05	Total including all contaminants not to exceed **500 ou** (based upon the results of the entire plant odour dispersion modelling).
Dimethyl sulphide (DMS)	>99%	0.01	
Mercaptans (R-SH)	>99%	0.01	
Chlorine	-	0.5	
VOC (average Mol. Wt 120)	There shall be sufficient removal of Ammonia and VOC's to achieve stack discharge max. concentration specified (at peak load)		

The summary of the various treatment technologies considered for this study provides initial information designed to allow familiarisation with the processes. In order to be able to complete a preferred option selection exercise, quantification of the treatment capabilities of all the options is required.

Table D-10 Summary of Process Technology Capabilities

Process technology	H_2s	r-sh	Dms	Voc	Nh^3	Ou[1]
Option 1: Wet Chemical Scrubbing Facility (Odorgard)	% Removal of >99.9[3]	% Removal of >99.9	% Removal of >99.9	% Removal of >40.0	% Removal of >99.0[2]	Discharge of < 500 ou
Option 2: Single Stage Biological Facility Followed by Activated Carbon Treatment	% Removal of >99.9[3]	% Removal of >99.9	% Removal of >99.9	% Removal of >99.9	% Removal of >99.9	Discharge of < 500 ou

Note 1. Odorous discharge concentrations are based on achieving performance on a 99.5%ile basis.
Note 2. With acid pre treatment stage required for concentrations above 5 ppm.
Note 3. H_2S removal capability has been stated assuming a low peak to mean ratio of H_2S in the foul air stream requiring treatment. This is expected to be achieved with the application of liquid phase odour control dosing measures (i.e. ferric chloride, magnesium hydroxide).

D.6.4 Process Unit Redundancy

All odour control technologies require down-time at some point, be it for maintenance, media replacement or unplanned work. Redundancy or standby capability is recommended to ensure continuous or alternative treatment is still possible while these works are being carried out.

Redundancy is handled in different ways depending on the owner, chosen technology, available footprint, odour sensitivity of the site, available budget etc. In other parts of Australia, duty-standby scrubbers are used. In some cases the trains are made smaller so that more than one individual unit can be used with an extra (n+1 approach) provided to cover redundancy. On occasions no standby capacity is afforded. Redundancy also comes at a cost.

Redundancy options considered sensible and viable for an odour control systems are as follows; in order of most to least risk-averse and highest to lowest cost.

1. "n+1" redundancy;

2. Carbon standby

3. Residual Carbon only.

Partial redundancy is often used where the technology consists of a number of units or trains. Here a philosophy of 'n+1' trains is often seen, n being the minimum number of vessels required to take the full flow, and +1 an extra which allows one unit to be off-line while the rest of the system carries the full flow.

A standby carbon system is often used where duplication of the primary system is not economical or where having a standby unit down most of the time is not recommended. Here, a carbon system would be available when the primary scrubbing train is offline. The standby carbon is eventually replaced when exhausted.

Partial carbon standby is where room in an activated carbon polisher is used to cover the downtime of a primary scrubbing system. This is a risky form of 'redundancy' as it is unavailable when the carbon is nearly exhausted. In the extreme case, foul air can still be passed through a bypass and up the stack just to maintain ventilation.

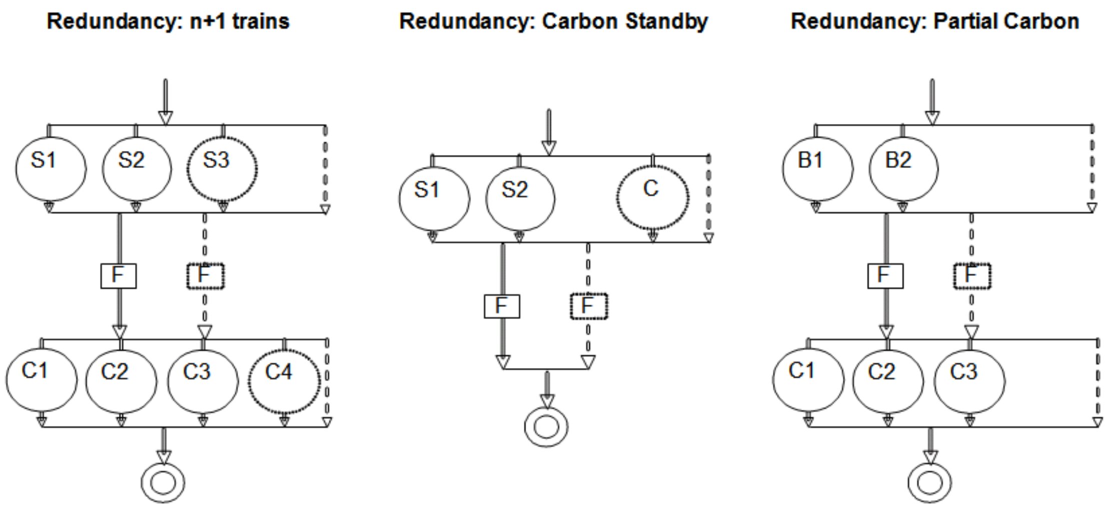

Figure A8-12: Redundancy Options for STP Biotrickling Filter-Carbon Odour Treatment Facilities

A carbon standby system was deemed most appropriate for the wet chemical scrubber options (including the bio/wet chemical option) whereas a “partial carbon system” was assumed for the biotrickling filter-carbon options.

As carbon only systems are inherently simple to operate and replacement of the carbon can be completed in a timely manner (on small systems), no standby streams have been included in the study. However, in systems where loads are erratic, or carbon in on long lead delivery times, it is prudent to include a standby vessel.

D.6.5 Conclusion

Option 2 has been selected as the best odour control treatment process option for Googong WRP, being a single-stage biological trickling filter followed by activated carbon.

APPENDIX E.
WRP Process Modelling using BioWin

PROCESS SIMULATION

GOOGONG WRP – MODULAR DESIGN SCENARIO

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This disclaimer shall apply notwithstanding that the report may be made available to other persons for an application for permission or approval or to fulfil a legal requirement.

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PREPARED BY:	**APPROVED FOR ISSUE BY:**
Stacey McCrystal	

REVISION SCHEDULE

REV. NO.	DATE	DESCRIPTION	PREPARED BY	REVIEWED BY	APPROVED BY
1	08/04/2010	Draft	S.McCrystal	S. Kitching	
2	13/04/2010	Draft Revision	S.McCrystal	S. Kitching	
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In line with our Quality System, this document has been prepared by Joseph Otter. It has been reviewed by Susan Kitching and signed off by Stephen Chapman.

STATUS: Draft | **PROJECT NUMBER:** A1068701 | February 2008
OUR REFERENCE: Googong WRP_Process Modelling Report.docx

CONTENTS

GOOGONG WATER RECYCLING PLANT PROCESS SIMULATION REPORT

1. INTRODUCTION

1.1 BACKGROUND

MWH has undertaken the concept design of a new Water Recycling Plant (WRP) within the Googong Urban Development. This design has been staged to allow some deferment of capital cost whilst the development is in construction and led to a First stage of the design allowed for a plant capacity of 850kL/d, allowing provision for further upgrades to the WRP to the projected ultimate plant capacity of 3.39ML/d (ADWF).

Costing models applied to the development have shown that further upfront capital cost savings are required for the project in the short term. As such a modular design has been investigated for the Water Recycling Plant. In addition the design of the plant with chemical phosphorus removal, is predicted to discharge a high total dissolved solids (TDS) concentration. This has arisen as a result of chemical dosing, high influent TDS concentrations in potable water and the effect of recycling flows in the system. Biological phosphorus removal would reduce this TDS concentration and allow CIC some flexibility in design.

This report has a two-fold objective:

1. To identify opportunities for the deferment of capital costs in the early years of the development, by assessing the feasibility of adopting a modular approach for the bioreactor and MBR, with additional modules being added as and when required. For this assessment a 50kL/d and 100kL/d option have been examined and compared to the base model (CDR Stage 1).

2. To provide preliminary assessment as to whether a biological phosphorus removal approach would be technically feasible (for both the existing 850kL/d plant design and the modular designs) and what added benefits this approach would offer.

Steady state modeling using the software package, BIOWIN, has been used to assess the feasibility of both the modular and biological phosphorus removal approaches.

The process simulation software (Biowin) is a computer simulation program developed by Envirosim for activated sludge processes that is based on the nitrification-denitrification practices described by the IAWPRC task group. The software has recently been extended for enhanced biological phosphorus removal using the enhanced culture models of Wentzel.

The simulation tool represents the current state-of-the-art in modelling activated sludge systems and is a significant improvement in activated sludge process calculations when used prudently.

MWH have in depth global experience in the application of the Biowin software to represent and calibrate existing treatment plants and for the design, or verification, of new facilities.

1.1.1 MODULAR DESIGN APPROACH

The proposed modular arrangement will involve modification of the existing CDR design for the bioreactor and MBR (850kL/d) into smaller 50kL/d or 100kL/d modules.

This technical assessment will be used to establish whether a smaller modular WRP approach can be used in lieu of the larger plant initially, by confirming the following:

1. Whether a modular arrangement will be able to consistently meet the proposed environmental constraints, including requirements for river discharge (ie. effluent quality).

2. Whether the effluent quality can be achieved with or without increased chemical dosage

3. What recirculation rates / reactor volumes are required for the modular reactors.

1.1.2 BIOLOGICAL PHOSPHORUS REMOVAL

Initially the Googong WRP was designed to achieve biological nitrogen removal only, with all phosphorus removed chemically, by the addition of ferric sulphate.

This technical assessment will be used to provide preliminary assessment as to whether biological phosphorus removal can be achieved (and to what degree) through modification of the existing plant design, and the modular designs. The following will be confirmed:

1. Whether a biological phosphorus (bio-P) removal process will be able to meet the proposed environmental constraints (ie. effluent quality)

2. What additional process units are required to achieve bio-P removal (and at what capital costs)

3. To what degree can Bio-P removal be achieved without the addition of ferric sulfate.

NB. Bio-P removal was assessed in all three design scenarios: 850kL/d, 100kL/d and 50kL/d.

1.2 REPORT SCOPE

The existing 850kL/d CDR Stage 1 design for the Bioreactor and MBR was initially performed using MWH process design calculations. As such, the same influent flow and load data have been applied in this Biowin modelling.

The scope of this report includes the following:

- Calibrated model of the existing 850kL/d Bioreactor and MBR design using existing flows and loads. The calibrated model will be used to establish the reliability of the Biowin modelling (by comparing outputs to the initial MWH process design calculations), and ensure accurate comparisons can be made between the design opportunities.

- Assessment of process performance using 50kL/d and 100kL/d modular designs (with flows and loads, and process units scaled accordingly).

- Optimisation of the 50kL/d and 100kL/d modular designs, by adjustment of chemical dose rates and recycle streams, to achieve the requirement effluent quality.

- Assessment of the feasibility in adopting a modular approach. The assessment will be largely based on ability of the modular design to consistently achieve the effluent quality requirements, and short and long term operational and capital expenditure. Ability to extend the modules in the available land will also be considered.

- Assessment of biological phosphorus removal performance in the 850kL/d, 100kL/d and 50kL/d design approaches.

- Adjustment of the anaerobic cell size and recycle rates to enhance the achievable bio-P (with final adjustment of chemical dose rates to achieve the required effluent TP quality).

- Assessment of the feasibility in adopting a bio-P approach in the design.

- Recommendations and Conclusions.

2. CURRENT PLANT SIMULATION

The initial plant process design has been conducted using MWH process design calculations. As such, an initial calibration of this design was performed in Biowin to ensure reliability of the Biowin software and enable accurate comparisons between the modular approach and the existing 850kL/d design.

The calibrated model will also form the basis of the biological phosphorus removal process assessment in the existing 850kL/d plant design, and the 50kL/d and 100kL/d

2.1 PROJECT INPUTS

2.1.1 FLOW CHART

Figure 2-1 below illustrates the Googong WRP process flow. The Googong WRP configuration is a four stage bioreactor (Primary Anoxic, Aerobic, De-Aeration and Secondary Anoxic zones) with a Membrane Bioreactor.

The process is primarily designed for biological nitrogen removal. All phosphorus removal is performed chemically, by the addition of ferric sulphate.

The same process configuration was adopted for both the 50kL/d and 100kL/d modular approach, with the process units scaled and sized appropriately

Figure 2-2 illustrates the adjusted process configuration required to achieve biological phosphorus removal. The adjusted process includes an upfront anaerobic cell, with nitrate and dissolved oxygen ‘poor’ recycle stream from the backend of the primary anoxic zones.

The same process configuration was adopted for both the 50kL/d and 100kL/d modular approach, with the process units scaled and sized appropriately.

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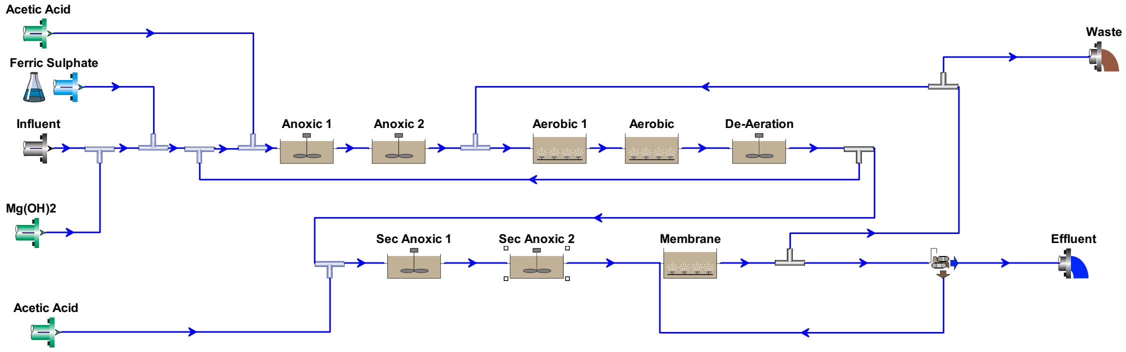

Figure 2-1 Googong WRP Flowchart

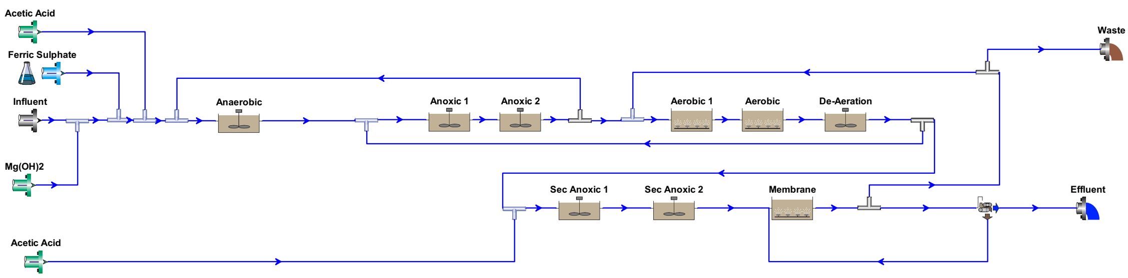

Figure 2-2 Googong WRP Flowchart (including Biological Phosphorus Removal)

2.1.2 PHYSICAL PARAMETERS

The size of the bioreactor units within the analysis have been taken from the MWH concept design. These volumes were then modified if required to obtain a design that would meet the effluent nutrient requirements and be efficient in tank size. No changes were required to the tank size.

The process units in the 50kL/d and 100kL/d modular design were sized and scaled appropriately to ensure the same aerobic and anoxic mass fractions were maintained across all design scenarios. Further detail of the 50kL/d and 100kL/d modular cell sizes is given in Section 2.2.2.1 below.

The size of each of the bioreactor cells and MBR used in the model calibration of the existing 850kL/d first stage design are displayed in Table 2-1.

Table 2-1. Existing Plant (850kL/d) Calibration Model Inputs

Element Name	Volume (m^3)	Area (m^2)	Depth (m)
Anoxic Cell 1	75	15	5
Anoxic Cell 2	75	15	5
Aerobic Cell 1	75	15	5
Aerobic Cell 2	423	84.6	5
De-Aeration	48	9.6	5
Secondary Anoxic 1	88	17.6	5
Secondary Anoxic 2	88	17.6	5
Membrane Tank	80.4	33.5	2.4

The size of each of the bioreactor cells and MBR used in the 50kL/d modular design are displayed in Table 2-2.

Table 2-2. 50kL/d Design Model Inputs

Element Name	Volume (m^3)	Area (m^2)	Depth (m)
Anoxic Cell 1	4.41	0.88	5
Anoxic Cell 2	4.41	0.88	5
Aerobic Cell 1	4.41	0.88	5
Aerobic Cell 2	24.9	4.98	5
De-Aeration	2.82	0.56	5
Secondary Anoxic 1	5.17	1.03	5
Secondary Anoxic 2	5.17	1.03	5
Membrane Tank	23.65	9.85	2.4

The size of each of the bioreactor cells and MBR used in the 100kL/d modular design are displayed in Table 2-3.

Table 2-3. 100kL/d Design Model Inputs

Element Name	Volume (m^3)	Area (m^2)	Depth (m)
Anoxic Cell 1	8.82	1.76	5
Anoxic Cell 2	8.82	1.76	5
Aerobic Cell 1	8.82	1.76	5
Aerobic Cell 2	49.76	9.95	5
De-Aeration	5.63	1.13	5
Secondary Anoxic 1	10.34	2.07	5
Secondary Anoxic 2	10.34	2.07	5
Membrane Tank	42.27	17.61	2.4

The size of the Anaerobic cell adopted for each of the design scenarios is given in Table 2-4 below, and represents approximately 15% of the overall process unit capacity. Typical anaerobic zone mass fractions would be between 10 – 15%. A number of scenarios were examined and this volume was selected to obtain the maximum biological phosphorus removal and therefore minimise chemical usage.

Table 2-4. Anaerobic Cell Size Model Inputs

Element Name	850kL/d Design	50kL/d Design	100kL/d Design
Anaerobic Cell Volume (m3)	153.8	9.05	18.10

2.1.3 OPERATING PARAMETERS

2.1.3.1 AERATION PARAMETERS

The dissolved oxygen setpoints applied in both the calibrated model (850kL/d) and the 50kL/d and 100kL/d modular designs are specified below. Biowin has the facility to model oxygen transfer, however as the system is not yet constructed it has been assumed that the plant is not oxygen limited and therefore a dissolved oxygen setpoint has been used.

Table 2-5. Dissolved Oxygen Concentrations

Element Name	Average Dissolved Oxygen (DO) Conc (mg/L)
Aerobic Cell 1 & 2	2.0
Membrane Tank	4.0

The membrane tank dissolved oxygen has been assumed. This may be higher depending on suppliers air flow requirements for scouring of the membranes. However, as oxygen modelling was not used in the model the effect of the dissolved oxygen on the recycles will be arbitrary and any nitrification in this zone will still occur with a positive DO concentration. Table 2-6 describes the design parameters used in both the calibrated model (850kL/d) and the 50kL/d and 100kL/d modular designs (NB. these design parameters were not altered across the design scenarios).

Table 2-6: Design Parameters

Parameter	Units	Calibration Model (850kL/d)	50kL/d and 100kL/d Modular Design
Minimum Temperature	^{o}C	15	15
MLSS in Bioreactor	mg/L	*	*
Alpha Factor	-	0.5	0.5
Beta Factor	-	0.95	0.95
Anoxic Growth Factor	-	1.0	1.0

Notes

*The MLSS concentration through the bioreactor varies. The target value to be achieved was 8000mg/L.

The anoxic growth factor has been changed from the Biowin default number. This is the only kinetic parameter that has been changed. MWH experience of calibration of plants within Australia and discussions with Envirosim have shown that for plants with Australian influents a value of between 0.8 – 1.0 is more typical than the default value.

2.1.3.2 RECYCLE RATES

Modular Design

The original Googong WRP configuration (see Figure 2-1 above) incorporates two recycle streams: A-Recycle, which recycles mixed liquor from the De-Aeration Cell back to the Anoxic Cell for denitrification, and S-Recycle pumps, which pump mixed liquor from the Membrane Bioreactor back to the Aerobic zone.

Sludge will be wasted, based on achieving a sludge retention time in the Bioreactor.

In order to achieve the desired effluent quality in the 50kL/d and 100kL/d modular design, the recycle rates were adjusted across a range of flows until the optimum recycle rates (to improve biological nitrogen removal) were achieved. Table 2-7 details these optimum recycle rates for the existing 850kL/d design and the 50kL/d and 100kL/d modular designs. Further analysis of these recycle rates is provided in Section 2.2.2.2 below.

Table 2-7 Googong WRP Recycle and Waste Streams

Element	Calibration Model (850kL/d) Flowrate	50kL/d Modular Design Flowrate	100kL/d Modular Design Flowrate
Influent Flow (kL/d)	890[a]	50	100
A-Recycle Flowrate (kL/d)	5340	400	800
A-Recycle Ratio	6 x ADWF	8 x ADWF	8 x ADWF
S-Recycle Flowrate	5340	400	800
S-Recycle Ratio	6 x ADWF	8 x ADWF	8 x ADWF
WAS Flow (kL/d)	38	2.15	4.3

[a] 40kL/d in return streams are included to the influent flow to give an effective influent flow of 890kL/d

The recycle rates have been adjusted along with the acetic acid dosing to the primary and secondary anoxic zones. One benefit of Biowin over steady state calculations is that this split can be seen and allows ease of optimisation during commissioning. It should be noted that the total acetic acid dosing requirements are the same.

Bio-P Removal

The Googong WRP configuration adopted for biological phosphorus removal (see Figure 2-2 above) incorporates an additional recycle stream, 'R-Recycle'. This recycle stream recycles nitrate and dissolved oxygen 'poor' mixed liquor post the primary anoxic cells to the front end fo the Anaerobic Cell for bio-P removal.

In order to enhance the Bio-P removal, the R-Recycle rates were adjusted across a range of flows (1 – 4 x ADWF) until the optimum recycle rate (for best bio-P removal) was achieved. Table 2-8 details these optimum R-Recycle rates for the existing 850kL/d design and the 50kL/d and 100kL/d modular designs. Further analysis of this R-Recycle rate is provided in Section 2.3.1.3 below.

Table 2-8 Googong WRP R-Recycle Stream (for bio-P removal)

Element	Calibration Model (850kL/d) Flowrate	50kL/d Modular Design Flowrate	100kL/d Modular Design Flowrate
Influent Flow (kL/d)	890	50	100
R-Recycle Flowrate (kL/d)	2670	150	300

R-Recycle Ratio	3 x ADWF	3 x ADWF	3 x ADWF

2.1.4 SETTLED SEWAGE INFLUENT DATA

The assumed influent flows and loads have been carried forward from the existing design and applied to the Biowin modelling for each design scenario.

As this is a new development, there is no existing data on which to perform influent characterisation. As such, data from existing NSW treatment plants has been used. As no diurnal flows patterns are available, only steady state modelling has been performed.

A review of the data is presented in Table 2-9.

Table 2-9 Googong WRP Raw Sewage Biowin Influent Parameters

Element Name	Influent (Calibration Model, 850kL/d)	Influent 50kL/d and 100kL/d Modular Design
Influent Flow m^3/d	890[a]	50 / 100
Total COD mg/L	667	667
Total Kjeldahl Nitrogen mgN/L	72	72
Total P mgP/L	14	14
Nitrate N mgN/L	0	0
pH	7.2	7.2
Alkalinity mmol/L	8.0	8.0
Inorganic S.S. mgTSS/L	41	41
Calcium mg/L	32	32
Magnesium mg/L	15.6	15.6
Dissolved oxygen mg/L	1.0	1.0

[a]The 890kL/d influent flow includes ~ 40kL/d return flows

As part of the process simulation inputs a fractionation of the raw sewage is required. The fractionation provides the simulation with the further breakdown of the incoming sewage constituents.

Wastewater component fractions are specific to each treatment plant however as no sewage exists default parameters have been used. As with the concept design there is some risk that these are incorrect.

It should be noted that the modular designs increase the risk with this lack of influent data. The concept design allowed for flexibility as it was designed for the first seven years of plant operation. Therefore real flows and loads could be determined in the first couple of years and used to more efficiently design the ultimate stage of development. Small modular systems do not allow as much flexibility in the first few years although the design can be modified in the longer term.

The fractionation is displayed in Table 2-10.

Table 2-10: Biowin Fractionation of Influent Data

Element Name	Influent
Fbs - Readily biodegradable (including Acetate) [gCOD/g of total COD]	0.190
Fac - Acetate [gCOD/g of readily biodegradable COD]	0.150
Fxsp - Non-colloidal slowly biodegradable [gCOD/g of slowly degradable COD]	0.750
Fus - Unbiodegradable soluble [gCOD/g of total COD]	0.050
Fup - Unbiodegradable particulate [gCOD/g of total COD]	0.130
Fna - Ammonia [gNH3-N/gTKN]	0.660
Fnox - Particulate organic nitrogen [gN/g Organic N]	0.500
Fnus - Soluble unbiodegradable TKN [gN/gTKN]	0.020
FupN - N:COD ratio for unbiodegradable part. COD [gN/gCOD]	0.035
Fpo4 - Phosphate [gPO4-P/gTP]	0.500
FupP - P:COD ratio for influent unbiodegradable part. COD [gP/gCOD]	0.011

2.1.5 CHEMICAL INPUT DATA

Several chemicals will be utilised at the WRP to assist the biological processes, as described below in Table 2-11.

The chemical addition in the existing plant calibration model were added in accordance with the original MWH Process Design Calculations. Chemical addition flows in Table 2-11 indicate that less ferric sulphate was dosed in the Biowin Calibration model than that used in the process design calculations.

It is difficult to model phosphorus removal using standard design methods and therefore a process simulation tool such as Biowin is normally used to predict performance. Therefore the results from the modelling will be taken as an accurate reflection of predicted performance.

In order to achieve the desired effluent quality in the 50kL/d and 100kL/d modular designs, the chemical dose rates were adjusted between the ranges advised, to achieve the required effluent results. These optimum chemical dose rates are detailed in Table 2-11 below.

Table 2-11: Optimum Chemical Input Parameters

Element	Calibration Model (850kL/d) Flowrate	50kL/d Modular Design Flowrate	100kL/d Modular Design Flowrate
Ferric Sulphate (for chemical phosphorus removal)	119 L/d	6.75 L/d	13.4 L/d
Acetic Acid (supplementary carbon source for denitrification)	0.1 m^3/d	0.009 m^3/d	0.0185 m3/d
Magnesium Hydroxide (for alkalinity addition)	0.165 m^3/d	0.0186 m^3/d	0.0744 m3/d

2.2 SIMULATION RESULTS (MODULAR DESIGN APPROACH)

2.2.1 INTRODUCTION

As described previously, MWH have completed a calibrated simulation using the Biowin software of the existing 850kL/d design plant. The model outputs compare well to the MWH process calculations. As such, the Biowin software can be reliably used to demonstrate the feasibility of a 50kL/d and 100kL/d modular design.

This section of the report details the results of the 50kL/d and 100kL/d modular design, and the operating conditions required to achieve the desired effluent results.

2.2.2 CALIBRATION OF EXISTING PLANT

The calibration of the existing 850kL/d plant, and modelling of a 50kL/d and 100kL/d modular design has been based upon the model inputs stated in section 2.1. The discussion of the simulation results is based upon the analysis of process operational requirements (i.e. recycle rates and chemical dosing) and effluent water quality comparisons.

2.2.2.1 BIOREACTOR VOLUMES AND MASSES

It must be noted that the initial bioreactor aerobic and anoxic mass fractions adopted in the initial 850kL/d process modelling have been adjusted slightly in the Biowin Calibrated model. The process calculation spreadsheet assumes that the same concentrations are maintained through each particular process cell, however, because Biowin models concentration changes through each cell, the output mass fractions (and required cell volumes) generated in Biowin are actual mass fractions rather than volumetric approximations.. As such, these adjusted mass fractions were applied across all the design scenarios. The total volume of the tank remained the same.

Table 2-14 below shows the adjusted Anoxic and Aerobic mass fractions between the initial 850kL/d model and the Biowin Calibration 850kL/d model. However, in summary, the Anoxic 3 cell in the existing model was converted to an Aerobic Cell to increase nitrification capacity, and avoid the anoxic cells turning anaerobic (due to complete denitrification occurring in Anoxic Cell 2).

Table 2-14 Bioreactor Mass Fractions

Element Name	MWH Process Calculations (850kL/d design)	Existing 850kL/d Calibrated Model
Anoxic Mass Fraction (%)	46	37.4
Aerobic Mass Fraction (%)	48.5	57.1

The process units in the 50kL/d and 100kL/d modular design were sized and scaled appropriately to ensure the same aerobic and anoxic mass fractions were maintained across the design scenarios. The cell sizes and mass fractions of the 50kL/d and 100kL/d modular design are detailed in the Table 2-15 and Table 2-16 below.

Careful consideration must be given to the modular configuration, in terms of cell length and width, as the ease of progressively adding additional modules (ie. through the removal of stainless steel walls etc) to the configuration adopted, so the eventual ultimate plant capacity of 3.4ML/d can be achieved on the land available (and in consideration of auxiliary infrastructure) is critical to the viability of the modular approach.

As illustrated in Tables 2-15 – 2-16 below, the cell sizes required for the 50kL/d and 100kL/d are very small ie. 4.41m3 (2.0m x 0.44m x 5.0m) for each Primary Anoxic Cell in the 50kL/d design. Based on the premilinary process configuration adopted for the purposes of this modelling, the overall Bioreactor dimensions for the 50kL/d and 100kL/d designs are:

- 50kL/d: 2.0m x 5.0m x 5.0m
- 100kL/d: 2.0m x 10.0m x 5.0m

It has been assumed that these tanks will be steel construction due to their small size. Careful consideration should be given by suppliers in terms of mixer and diffuser surface area requirements. The tanks may require resizing due to the requirements of mechanical equipment. This has not been assessed as part of this exercise.

The original designs allows for four process trains, each approx. 850kL/d capacity with bioreactor dimensions: 6.0m x 29 x 5.0m. These process trains can be easily incorporated in the land available. These four process trains also minimise the additional equipment / infrastructure required (ie. Recycle pumps, aeration blowers, tankage, process pipework and instrumentation etc).

A 50kL/d module will require the addition of up to 17 modules to reach an 850kL/d capacity. However, this would result in the following overall cell sizes:

- Anoxic Cell 1, 2: 34m x 0.44m x 5.0m (very long and very narrow)
- Aerobic Cell 1: 34m x 0.44m x 5.0m (very long and narrow)
- Aerobic Cell 2: 34m x 2.5m x 5.0m
- De-Aeration Cell: 34m x 0.28m x 5.0m
- Sec. Anoxic 1, 2: 34m x 0.52m x 5.0m

It is recommended that further analysis be performed on optimal cell sizing of the 50kL/d modules, or larger modules (ie. 200kL/d modules) are adopted.

Table 2-15 – 50kL Modular Design Bioreactor and MBR Cell Sizes and Configuration

Cell	Length	Width	Depth (@ ADWF)	Surface Area	Liquid Volume	Nominal Cell Volumes	Mass Fractions (excl. MBR)		Mass Fractions (incl. MBR)	
	m	m	m	m2	kL	kL				
Anoxic 1	0.44	2.0	5.0	0.88	4.4	4.4	Anoxic	17.2%	Anoxic	11.7%
Anoxic 2	0.44	2.0	5.0	0.88	4.4	4.4				
Aerobic 1	0.44	2.0	5.0	0.88	4.4	4.4	Aerobic	57.1%	Aerobic	39.1%
Aerobic 2	2.49	2.0	5.0	4.98	24.9	24.9				
De-Aeration	0.28	2.0	5.0	0.56	2.8	2.8	De-Aer	5.5%	De-Aer	3.7%
Sec. Anoxic 1	0.52	2.0	5.0	1.04	5.2	5.2	Sec. Anoxic	20.3%	Sec. Anoxic	13.9%
Sec. Anoxic 2	0.52	2.0	5.0	1.04	5.2	5.2				
		Total (excl. MBR)		**10.26**	**51.3**	**51.3**	check	100.0%		
MBR	4.92	2	2.4	9.84	23.6	23.6			MBR	31.5%
		Total (incl. MBR)		**20.1**	**74.9**	**74.9**			check	100.0%
						Total Aerobic		57.1%		70.6%
						Total Anoxic		37.4%		25.6%

Table 2-16 – 100kL/d Modular Design Bioreactor and MBR Cell Sizes and Configuration.

Cell	Length	Width	Depth (@ ADWF)	Surface Area	Liquid Volume	Nominal Cell Volumes	Mass Fractions (excl. MBR)		Mass Fractions (incl. MBR)	
	m	m	m	m2	kL	kL				
Anoxic 1	0.88	2.0	5.0	1.76	8.8	8.8	Anoxic	17.2%	Anoxic	11.8%
Anoxic 2	0.88	2.0	5.0	1.76	8.8	8.8				
Aerobic 1	0.88	2.0	5.0	1.76	8.8	8.8	Aerobic	57.2%	Aerobic	39.2%
Aerobic 2	4.98	2.0	5.0	9.96	49.8	49.8				
De-Aeration	0.56	2.0	5.0	1.12	5.6	5.6	De-Aer	5.5%	De-Aer	3.7%
Sec. Anoxic 1	1.03	2.0	5.0	2.06	10.3	10.3	Sec. Anoxic	20.1%	Sec. Anoxic	13.8%
Sec. Anoxic 2	1.03	2.0	5.0	2.06	10.3	10.3				
		Total (excl. MBR)		**20.48**	**102.4**	**102.4**	check	100.0%		
MBR	9.85	2	2.4	19.7	47.3	47.3			MBR	31.6%
		Total (incl. MBR)		**40.18**	**149.7**	**149.7**			check	100.0%
						Total Aerobic		57.2%		70.7%
						Total Anoxic		37.3%		25.5%

2.2.2.2 FINAL EFFLUENT QUALITY

Modular Design Approach

Table 2-12 compares the steady state MWH Process Calculation effluent results to the steady state model calibration results of the existing 850kL/d design, performed in Biowin. The results are also compared against the optimised 50kL/d and 100kL/d modular design models also performed in Biowin.

The 850kL/d calibrated model in Biowin compares well with the effluent results obtained from the MWH process calculations. The higher Total Nitrogen in the Calibrated Model may be accounted for by the Inorganic Nitrogen fraction determined in the Biowin models. In either case it shows that the total nitrogen effluent requirement can be achieved in all scenarios.

The effluent results obtained in the 50kL/d and 100kL/d modular design also compare well with those obtained in the MWH Process Calculations, and demonstrate that if a 50kL/d or 100kL/d modular design, and if operated correctly, may likely achieve the required effluent results. However, Section 2.2 discusses the modified operating parameters and cell configurations required to achieve this effluent quality in the 50kL/d and 100kL/d modular design.

Table 2-12. Final Effluent Quality

Element Name	MWH Process Calculations	Existing 850kL/d Calibrated Model	50kL/d Modular Design	100kL/d Modular Design
Permeate Flow m^3/d	890	890	50	100
Total COD mg/L	48	34.55	50.96	34.46
Total BOD mg/L	**5**	**0.81**	**0.76**	**0.74**
Total Kjeldahl Nitrogen (TKN) mgN/L	3.0	2.79	2.78	2.71
Ammonia as N (NH_3-N) mg/L	0.3	0.43	0.20	0.21
Nitrate N mgN/L	0.0	0.99	1.22	1.25
Total N mg/L	**3.3**	**3.89**	**3.98**	**4.01**
Total P mgP/L	**0.2**	**0.22**	**0.22**	**0.21**
Total Suspended Solids. mgTSS/L	2.0	0.01	0.01	0.00

2.2.2.3 RECYCLE RATES

The A-Recycle and S-Recycle rates for the 50kL/d and 100kL/d modular design plants are displayed in Table 2-17, and compared against the recycle rates for the existing 850kL/d plant.

It must be noted that in performing the optimisation of the 50kL/d and 100kL/d modular design to achieve the required effluent quality, the recycle rates were adjusted first, to maximise the process capacity in terms of achieving biological nutrient removal (before relying on chemical addition to improve results).

Table 2-17 Googong WRP Recycle Streams

Element	Calibration Model (850kL/d) Flowrate	50kL/d Modular Design Flowrate	100kL/d Modular Design Flowrate
Influent Flow (kL/d)	890[a]	50	100
A-Recycle Flowrate (kL/d)	5340	400	800
A-Recycle Ratio	6 x ADWF	8x ADWF	8 x ADWF
S-Recycle Flowrate	5340	400	800
S-Recycle Ratio	6 x ADWF	8 x ADWF	8 x ADWF

The above results indicate that greater recycle rates are required to achieve the same effluent results in the 50kL/d and 100kL/d modular design.

The power draw from the recycle pumps, and associated costs, were compared for all scenarios on a kWh / ML basis. Results indicate that the average power consumption for the 50kL/d and 100kL/d modular design is approximately 96kWh/ML and 144kWh/ML (respectively) as opposed to approximately 62kWh/ML for the 850kL/d design.

2.2.2.4 CHEMICAL REQUIREMENTS

The chemical requirements for the 50kL/d and 100kL/d modular design plant are displayed in Table 2-18 – Table 2-20, and compared against the chemical requirements for the existing 850kL/d plant.

Table 2-18 Ferric Sulphate Required Dose Rates to achieve effluent P requirements

Element	Calibration Model (850kL/d) Flowrate	50kL/d Modular Design Flowrate	100kL/d Modular Design Flowrate
Influent Flow (kL/d)	890	50	100
Ferric Dose Rate (L/d)	119	6.75	13.4
Ferric sulphate Specific Gravity	1.58		
Ferric Dose Rate (kg/d)	188	11	21
Ferric Dose Rate (mg/L)	221	213	211
Compared Dose Rate	0.96		

Table 2-19 Acetic Acid Required Dose Rates to achieve effluent N requirements

Element	Calibration Model (850kL/d) Flowrate	50kL/d Modular Design Flowrate	100kL/d Modular Design Flowrate

Influent Flow (kL/d)	890	50	100
Acetic Acid Dose Rate (m3/d)	0.1	0.009	0.0185
Acetic Acid COD equivalent	1024880 mgCOD / L Acetic Acid		
Acetic Acid Dose Rate (kg/d)	77	6.9	14.2
Ferric Dose Rate (mg/L)	90	138	142.2
Increased Dose Rate		**1.53**	**1.57**

Table 2-20 Magnesium Hydroxide Required Dose Rates for alkalinity requirements

Element	Calibration Model (850kL/d) Flowrate	50kL/d Modular Design Flowrate	100kL/d Modular Design Flowrate
Influent Flow (kL/d)	890	50	100
Mg(OH)2 Dose Rate (m3/d)	0.165	0.0186	0.0744
Mg(OH)2 Density (kg/m3)	1500		
Mg(OH)2 Dose Rate (kg/d)	247	28	112
Ferric Dose Rate (mg/L)	291	558	1116
Increased Dose Rate		**1.9**	**3.8**

The above results indicate that greater than 1.5 - 4 times more acetic acid and magnesium hydroxide is required to achieve the same effluent results in the 50kL/d and 100kL/d modular design.

Although the overall chemical usage appears to be smaller for the 50kL/d and 100kL/d modular designs, when these chemical consumptions are converted to a $/ML basis, the chemical costs for the 50kL/d and 100kL/d modular design are $235,700/ML and $373,500/ML (respectively). This compares to the $149,500/ML required for the 850kL/d design (ie. 1.6 – 2.5 times as much).

This would indicate that more money will be spent on chemicals in the 50kL/d and 100kL/d modular design to achieve the required effluent results.

2.2.2.5 SUMMARY

The following summarises the results from the existing 850kL/d design and the 50kL/d and 100kL/d modular design.

The model outputs indicate that the required effluent results can be achieved in a 50kL/d or 100kL/d modular design.

However, the feasibility of such an approach must also be considered in consideration of the ease of module addition (and overall process configuration and layout), changed operating conditions and subsequent increased operating cost efficiencies.

- Bioreactor Cell Dimensions

  The required cell dimensions in the 50kL/d and 100kL/d modular designs are very small. Further consideration must be given to ensure that the final cell dimensions can be constructed and that mechanical equipment can be located within the tank size.

- Recycle flows

  Slightly increased Recycle flows are required in the 50kL/d and 100kL/d modular designs in order to meet effluent quality requirements.

  The power draw from the recycle pumps, and associated costs, were compared for all scenarios on a kWh / ML basis. Results indicate that the average power consumption for the 50kL/d and 100kL/d modular design is approximately 96kWh/ML and 144kWh/ML (respectively) as opposed to approximately 62kWh/ML for the 850kL/d design.

  Therefore on a cost efficiency basis, the 50kL/d and 100kL/d modular designs represent increased operational costs.

- Chemical consumption

  The chemical consumption results indicate that greater than 1.5 - 4 times more acetic acid and magnesium hydroxide is required to achieve the same effluent results in the 50kL/d and 100kL/d modular design.

  Overall the chemical costs for the 50kL/d and 100kL/d modular design are $235,700/ML and $373,500/ML (respectively). This compares to the $149,500/ML required for the 850kL/d design (ie. 1.6 – 2.5 times as much). This has been based on the chemical costs used in the concept design.

  This would indicate that more money will be spent on chemicals in the 50kL/d and 100kL/d modular design to achieve the required effluent results.

2.3 SIMULATION RESULTS (BIO-P APPROACH)

2.3.1 INTRODUCTION

This section of the report details the results of the preliminary assessment as to whether a biological phosphorus removal approach would be technically feasible (for both the existing 850kL/d plant design and the modular designs) and what added benefits this approach would offer.

2.3.1.1 EFFLUENT RESULTS

Table 2-21 below demonstrates the effluent results achieved through augmentation of the existing process configuration, by including an upfront anaerobic zone, to enhance biological phosphorus removal. The results in Table 2-21 demonstrate the degree of biological phosphorus removed, without the addition of ferric sulphate.

NB. Two effluent TP results are displayed. The first results were achieved with additional acetic acid dose upfront, to enhance the soluble phosphorus release in the anaerobic cell. This additional acetic acid dose will be offset by the reduced ferric dose required to trim the effluent TP concentration. The second results were achieved without increasing the acetic acid dose. This will mean more ferric dose is required to trim the effluent TP concentration.

Table 2-21 Effluent TP achieved through bio-P removal only

Element Name	Existing 850kL/d Calibrated Model	50kL/d Modular Design	100kL/d Modular Design
Permeate Flow m^3/d	890	50	100
Total COD mg/L	47.29	50.96	50.37
Total BOD mg/L	0.84	0.76	0.76
Total Kjeldahl Nitrogen (TKN) mgN/L	3.16	3.31	3.28
Ammonia as N (NH_3-N) mg/L	0.33	0.15	0.15
Nitrate N mgN/L	1.07	0.98	1.09
Total N mg/L	4.31	4.32	4.41
Total P mgP/L	0.46 / 2.26	0.30 / 2.01	0.34 / 1.28
Total Suspended Solids. mgTSS/L	0.01	0.01	0.00

Table 2-22 below demonstrates the final effluent results with addition of ferric, in order for the process to achieve required effluent TP results.

Table 2-22 Effluent TP achieved through bio-P removal and Ferric addition.

Element Name	Existing 850kL/d Calibrated Model	50kL/d Modular Design	100kL/d Modular Design
Permeate Flow m^3/d	890	50	100
Total COD mg/L	47.32	50.96	50.37
Total BOD mg/L	0.85	0.76	0.76
Total Kjeldahl Nitrogen (TKN) mgN/L	3.16	3.31	3.28
Ammonia as N (NH_3-N) mg/L	0.33	0.15	0.15
Nitrate N mgN/L	0.99	0.98	1.09
Total N mg/L	4.20	4.32	4.41
Total P mgP/L	0.21	0.22	0.20
Total Suspended Solids. mgTSS/L	0.01	0.01	0.00

The anaerobic cell sizes and R-Recycle were optimised to achieve the best biological phosphorus removal possible. However, the results indicate that only particle phosphorus removal can be achieved biologically. As such, ferric sulphate dosing is still required in order for the plant to meet the required effluent TP parameters.

The Anaerobic Cell sizes, R-Recycle rates and chemical dosing required to achieve bio-P removal are discussed below.

2.3.1.2 BIOREACTOR VOLUMES AND MASSES

The optimum anaerobic mass fraction, to achieve best bio-P removal, was assessed over a range of mass fractions, between 10 – 15% of overall bioreactor volume (excl. the MBR volume).

Phosphorus results indicate that the optimum anaerobic mass fraction is approximately 15%, which is in inline with the typically adopted value.

As such, the adopted anaerobic cell volumes for bio-P modelling in the 850kL/d, 50kL/d and 100kL/d are as detailed in Table 2-23 below.

Table 2-23 Bioreactor Anaerobic Cell Volumes

Element Name	850kL/d Design	50kL/d Design	100kL/d Design
Anaerobic Cell Volume (m3)	153.8	9.05	18.10

Table 2-24 Overall Bioreactor Mass Fractions

Element Name (excl. MBR Volume)	Existing 850kL/d Calibrated Model	50kL/d Modular Design	100kL/d Modular Design
Anaerobic Mass Fraction (%)	15.0	15.0	15.0
Anoxic Mass Fraction (%)	31.8	31.8	31.8
Aerobic Mass Fraction (%)	48.6	48.6	48.6

2.3.1.3 RECYCLE RATES

The Googong WRP configuration adopted for biological phosphorus removal (see Figure 2-2 above) incorporates an additional recycle stream, ‘R-Recycle’.

In order to enhance the Bio-P removal, the R-Recycle rates were adjusted across a range of flows (1 – 4 x ADWF) until the optimum recycle rate (for best bio-P removal) was achieved. Table 2-25 details these optimum R-Recycle rates for the existing 850kL/d design and the 50kL/d and 100kL/d modular designs. NB. The A-Recycle and S-Recycle rates were not altered for bio-P removal.

Table 2-25 Googong WRP R-Recycle Stream (for bio-P removal)

Element	Calibration Model (850kL/d) Flowrate	50kL/d Modular Design Flowrate	100kL/d Modular Design Flowrate
Influent Flow (kL/d)	890	50	100
R-Recycle Flowrate (kL/d)	2670	150	300
R-Recycle Ratio	3 x ADWF	3 x ADWF	3 x ADWF

2.3.1.4 CHEMICAL REQUIREMENTS

By the incorporation of an anaerobic zone into the process configuration, to enhance biological phosphorus removal, an overall reduction in ferric dosing of between 40 – 50% could be achieved. NB. This reduction in ferric dosing was achieving without increasing the acetic acid dose rate.

If the acetic acid dosing was increased by up to 80% (to enhance soluble phosphorus release) an overall reduction in ferric dosing of between 75% - 85% could be achieved.

On a cost basis, if the acetic acid dose is not changed, the 40 – 50% reduction in ferric dosing provides a cost benefit, by reducing the overall chemical consumption costs by 4 – 9%, when compared to overall chemical costs when no anaerobic cell is included.

However, any cost benefit is eroded if a 75 – 85% reduction in ferric dosing is achieved through increasing the acetic acid dose by up to 80%. In fact, dosing chemical at these adjusted rates increases the overall chemical consumption costs by 3 – 6%, when compared to the overall chemical costs when no anaerobic cell is included (this is due to the more expensive nature of the acetic acid chemical).

When compared to the bioreactor designs, which do not incorporate biological phosphorus removal, the overall chemical costs compare as follows:

Table 2-26 Chemical Cost comparison

Overall Chemical Costs ($/ML)	Calibration Model (850kL/d) Overall chemical cost ($/ML)	50kL/d Modular Design Overall chemical cost ($/ML)	100kL/d Modular Design Overall chemical cost ($/ML)
No Bio-P Removal	149,500	235,700	373,500
Bio-P Removal (no change in acetic acid dose)	135,000	223,000	355,800
Variation in chemical costs	- 9.0%	- 5.4%	- 4.7%
Bio-P Removal (increase in acetic acid dose)	159,000	251,000	398,000
Variation in chemical costs	+ 6.3%	+ 6.5%	+ 6.5

Results indicate that an overall chemical cost saving of approximately 5 – 9% can be achieved by the incorporation of an upfront anaerobic cell to achieve partial biological phosphorus removal.

However this reduction in the ongoing chemical costs of the plant must be compared to the initial capital cost of installing an upfront anaerobic cell, with R-Recycle, and the cost of this additional pumping to establish any real long term financial benefits of incorporating biological phosphorus removal.

Assessment needs to be carried out to establish the potential reduction in TDS concentration throughout the system and the overall options that should be carried out within the design. This will be performed in a further exercise using the MWH water balance and is not included within this report.

2.3.1.5 SUMMARY

The following summarises the results from the bio-P assessment of the existing 850kL/d design and the 50kL/d and 100kL/d modular design.

The model outputs indicate that partial phosphorus removal can be achieved through the installation of an upfront anaerobic cell, which represents approximately 15% of the overall bioreactor volume, and R-Recycle.

The remaining phosphorus removal, to meet the effluent quality requirements, is achieved through reduced dosing of ferric sulphate.

The added benefit of reduced ferric dosing is the reduction in inorganic solids formed within the process, which also decreases the loading on the biosolids handling facility.

- Chemical consumptions

  Overall the chemical consumption of ferric sulphate can be reduced by 40 – 50% when an upfront anaerobic cell is incorporated in the process configuration, to achieve partial bio-P removal. This reduction in ferric dosing is realised in an overall reduction in the chemical costs of between 5 – 9%.

However, to understand any overall cost benefits of incorporating biological phosphorus removal into the design, the reduction in ongoing chemical costs must be compared to the initial capital costs of installing an upfront anaerobic cell, with R-Recycle pumps.

3. CONCLUSIONS AND RECOMMENDATIONS

Modular Design Approach

While the process simulations have demonstrated that a 50kL/d and 100kL/d modular designs will achieve the required effluent quality, as detailed in Table 2-9 above, the amount of chemical addition and increased recycle rates, when compared with the existing 850kL/d design demonstrate that while short term capital gain may be achieved in adopting the modular approach, ongoing operating costs (in the short and long term) are significantly greater (on a cost efficiency basis), and will effect the benefits of any short term capital gain.

In addition, careful consideration must be given to the bioreactor and MBR cell sizes and configuration to ensure the additional 50kL/d or 100kL/d modules required in staging can be easily installed, and in the land available (to the ultimate capacity of 3.4ML/d). A detailed analysis on bioreactor and MBR cell sizes and configurations, and ease of augmentation, has not been conducted in this report.

However, initial assessments would indicate that larger module designs (of say 200kL/d) may be more easily installed and augmented, while still achieving the required effluent results.

Bio-P Removal Approach

Preliminary assessment of bio-P removal, by incorporating an upfront anaerobic zone, with R-Recycle, into the process configuration, does provide partial phosphorus (in all design scenarios), with the remainder of the phosphorus being removed through the addition of ferric sulphate to meet the required effluent results.

However, to establish the overall financial benefits of a biological phosphorus design, the capital costs (of installing an anaerobic tank and R-Recycle pumps) must be compared against the long term chemical dosing and additional R-recycle pumping costs.

APPENDIX F.
Electrical Load

CIC - Googong SPS 1 - Maximum Demand Estimation (Rev. C)

Item No.	SPS 1 - Outdoor Kiosk	(P1) Rating per Unit kW	FLC, (A) per Drive or Equip	Motor Eff, (%)	pf	No of Units	No of Units running	Sub Total FLC, (A)	Diversity Factor	Total Current Drawn with Diversity Factor (A)	Total Power Absorbed, kW	Total Power Absorbed, kVA	Starting Method	Comment
1	Pump 1	22.00	34.64	93	0.95	1	1	34.64	1.0	34.64	23.66	24.90	VSD	
2	Pump 2	22.00	34.64	93	0.95	1	1	34.64	1.0	34.64	23.66	24.90	VSD	
3	Pump 3	22.00	34.64	93	0.95	1	0	0.00	0.0	0	0.00	0.00	VSD	
4	Chemical Dosing Facility	1.50	2.95	75	0.95	1	1	2.95	1.0	2.95	2.01	2.12	VSD	
5	Lighting and General Power Supply	2.00	3.44	90	0.9	1	1	3.44	0.5	1.72	1.11	1.23		
	Total:									**73.95**	**50.44**	**53.16**		
	Total (with 30% Spare Capacity):									**96.14**	**65.57**	**69.10**		

Estimated power capacity required is approximately (Includes 30% spare capacity)	69	kVA	or	66	kW

Formulas

Total Power Absorbed (kW) = 1.732 x Line Voltage x (FLC per drive) x pf x (No of Units running) x (Diversity Factor) / 1000
Total Power absorbed (kVA) = 1.732 x Line Voltage x (FLC per drive) x (No of Units running) x (Diversity Factor) / 1000
FLC per drive = (kW Rating per Unit) x 1000 / 1.732 / Line Voltage / [(Motor Eff) /100] / pf
Sub Total FLC = (kW Rating per Unit) x 1000 / 1.732 / Line Voltage / [Motor Eff/100] / pf x (No of Units running)

Assumption:

1. For DOL drive, the power factor is estimated to be same as the motor if there is insufficient information provided for the motor drive (the above estimate is based on TECO motor data)
2. For VSD drive, the power factor is estimated to be 0.95 if there is insufficient information provided for the motor drive
3. For Soft Starter drive, the power factor is estimated to be 0.90 if there is insufficient information provided for the motor drive
4. Assumption has been made on the diversity factor that duty/standby arrangement of identical motor drives will be configurated amongst MCC/SCA.
5. Miscellaneous single phase loads are assumed to be connected across 3 phases in a balanced loads configuration.