GWAING WWTW

MASTER PLAN

REV02

09 April 2025

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GEORGE MUNICIPALITY

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ABBREVIATIONS

AD	Anaerobic Digestion
ADWF	Average Dry Weather Flow
AOR	Actual Oxygen Requirement
BGCMA	Breede-Gouritz Water Management Area
BNR	Biological Nutrient Removal
CAPEX	Capital Expenditure
CAS	Conventional Activated Sludge
CCT	Chlorine Contact Tank
cfu	Colony Forming Unit
CHP	Combined Heat And Power
COD	Chemical Oxygen Demand
CPA	Contract Price Adjustment
CRR	Cumulative Risk Rating
DEADP	Department Of Environmental Affairs And Development Planning
DFFE	Department Of Forestry, Fisheries And The Environment’s
DO	Dissolved Oxygen
DPR	Direct Potable Reuse
DS	Dry Solids
DSVI	Diluted Sludge Volume Index
DWS	Department Of Water And Sanitation
EA	Environmental Authorisation
EBPR	Enhanced Biological Phosphate Removal
EC	Electrical Conductivity
FBDA	Fine Bubble Diffused Aeration
FBP	Filter Belt Press
FDBA	Fine Bubble Diffused Aeration
FSA	Free And Saline Ammonia
FSA-N	Free And Saline Ammonia As Nitrogen
GD	Green Drop
GM	George Municipality
GMSDF	George Municipality Spacial Development Framework

GT	Gravity Thickener
IGV	Inlet Guide Vane
IPR	Indirect Potable Reuse
ISS	Inert Suspended Solids
JHB	Johannesburg
LCE	Luckhozi Consulting Engineers
MBR	Membrane Bioreactor
MCC	Motor Control Centre
ML	Megalitre
MLD	Megalitre Per Day
MLE	Modified Ludzack-Ettinger
MLSS	Mixed Liquor Suspended Solids
MOV	Most Open Valve
MUCT	Modified UCT
NEMWA	National Environmental Management Waste Act
NTU	Nephelometric Turbidity Unit
OP	Ortho Phosphates
OPEX	Operational Expenditure
PAOs	Phosphate-Accumulating Organisms
PDWF	Peak Dry Weather Flow
PF	Peaking Factor
PS	Primary Sludge
PST	Primary Settling Tank
PV	Photovoltaic
PWWF	Peak Wet Weather Flow
RAS	Return Activated Sludge
RFP	Request for Proposal
SF	Safety Factor
SOR	Standard Oxygen Requirement
SOTE	Standard Oxygen Transfer Efficiency
SPC	Specific Power Consumption
SRT_m	Minimum Sludge Age For Nitrification
SST	Secondary Settling Tank
tCOD	ton COD
TKN	Total Kjeldahl Nitrogen
TOD	Total Oxygen Demand
TSA	Technical Site Assessment
TSS	Total Suspended Solids
UCT	University Of Cape Town
UF	Ultrafiltration
UPO	Unbiodegradable Particulate Organic
USO	Unbiodegradable Soluble Organic
UV	Ultraviolet
UVT	UV Transmittance
VAT	Value Added Tax
VFA	Volatile Fatty Acids

VFD	Variable Frequency Drive
VSS	Volatile Suspended Solids
VVD	Variable Vane Diffusers
WAS	Waste Activated Sludge
WRRF	Water Resource Recovery Facility
WSA	Water Services Act
WSI	Water Service Institutions
WUL	Water Use License
WULA	Water Use License Application
WW	Wastewater
WWTW	Wastewater Treatment Works
X_T	Reactor TSS Concentration

REPORT DETAILS

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1 EXECUTIVE SUMMARY

The Gwaing Wastewater Treatment Works in George, Western Cape, recently completed minor upgrades, resulting in a total average dry weather flow (ADWF) capacity of 10.4 million litres per day (MLD). Operating with a modified Ludzack-Ettinger (MLE) process, the plant achieves this capacity at a design chemical oxygen demand (COD) concentration of 782 mgCOD/l (or 10.9 MLD at a COD concentration of 700 mgCOD/l). Currently, the plant receives an ADWF of 10.4 MLD and is therefore operating at its capacity. Due to population growth in George, extending the wastewater treatment works is a priority.

George Municipality (GM) aims to upgrade the Gwaing WWTW to remain compliant with the effluent standards as dictated by the Water Use Licence (WUL) issued by the Department of Water and Sanitation (DWS). GM have appointed Lukhozi Consulting Engineers (Pty) Ltd (LCE) to create a Master Plan that will guide future upgrades. The Master Plan seeks an ultimate capacity of 50 MLD, allowing for phased intermediate upgrades aligned with the ultimate solution. Additionally, it optimizes spatial requirements on a site with various constraints.

The heart of the upgrades is additional biological reactor modules B and C and its associated SSTs to supplement the existing reactor module A. These biological reactors are conventional activated sludge (CAS) systems that can facilitate COD and nutrient (N and P) removal. The UCT process is an enhanced biological phosphate removal (EBPR) process and is the main process selected for the upgrades. Operational flexibility allows operators to choose between modified UCT, MLE or JHB processes to optimise the plant's capacity and/or effluent quality.

The upgrades will also include several other unit processes and peripheral facilities to be implemented in phases as summarized in Table 1-1. A key process change is envisaged during Phase D when PSTs and anaerobic digestion (AD) will be implemented. This will divert much of the COD loading from the reactors to the digestors and thereby create additional capacity in an energy efficient manner. The introduction of PSTs and AD is intentionally postponed until the WWTW capacity exceeds 25 MLD. Beyond 25 MLD the benefits of AD on energy consumption starts to occur for Gwaing WWTW. This delay ensures that the increased complexity associated with these processes is offset by their substantial size, which justifies higher-level operational and maintenance resources.

Table 1-1: Summary of infrastructure required for the upgrades and corresponding phasing

Infrastructure		Cumulative Total Process Units				
Unit Process	**Module**	**Existing***	**Phase A**	**Phase B**	**Phase C**	**Phase D**
Inlet Works		0	1	1	2	2
Regional Grit and Screenings Facility		0	0	1	1	1
Primary Settling Tanks		0	0	0	0	4
Primary Sludge Gravity Thickeners		0	0	0	0	2
Anaerobic Digestors		1	1	1	1	4
Biological Reactors	Module A	1	1	1	1	1
	Module B	0	0	1	1	1
	Module C	0	0	0	1	1
Secondary Settling Tanks	Module A	2	4	4	4	4
	Module B	0	4	4	4	4
	Module C	0	0	0	4	4
Blower House		0	0	1	1	1
RAS Pumpstation		0	1	1	1	1
UV / Chlorine Disinfection		0	0	1	2	2
WAS Dewatering Beltpresses		2	2	4	6	6
PS Dewatering facility		0	0	0	0	1
Sludge Solar Drying / Composting		0	0	1	1	2
Admin building		0	0	1	1	1

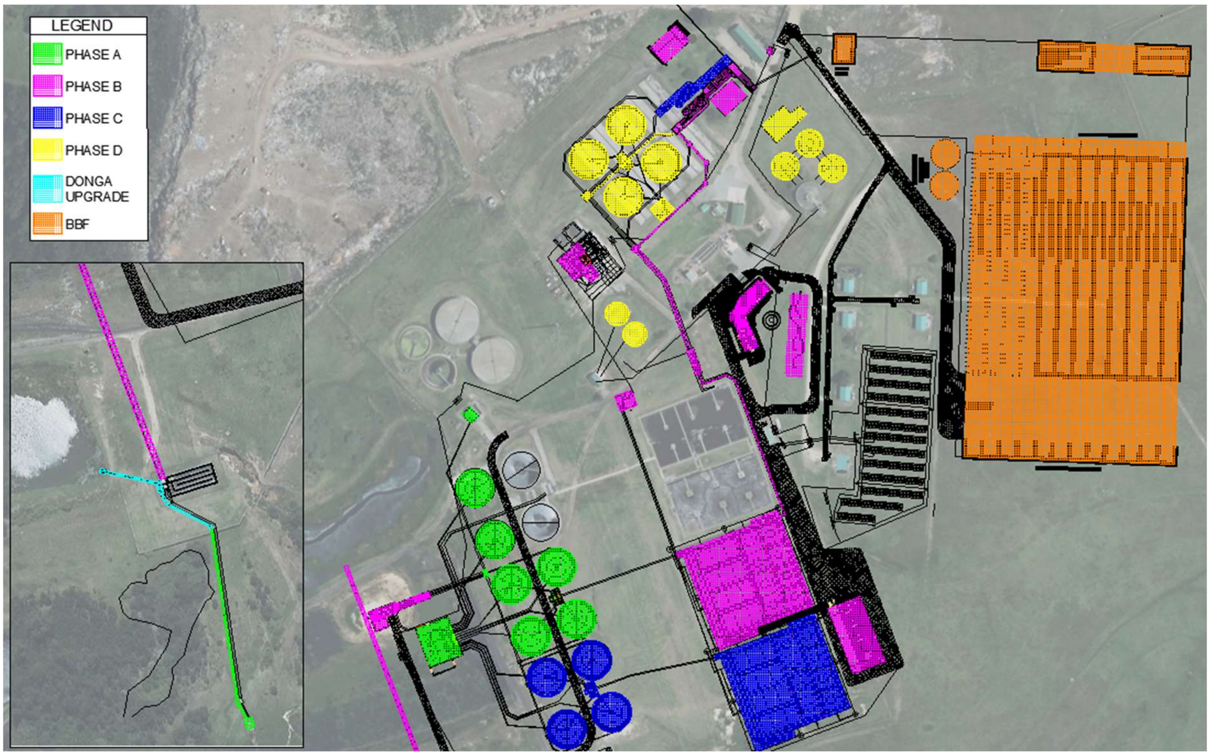
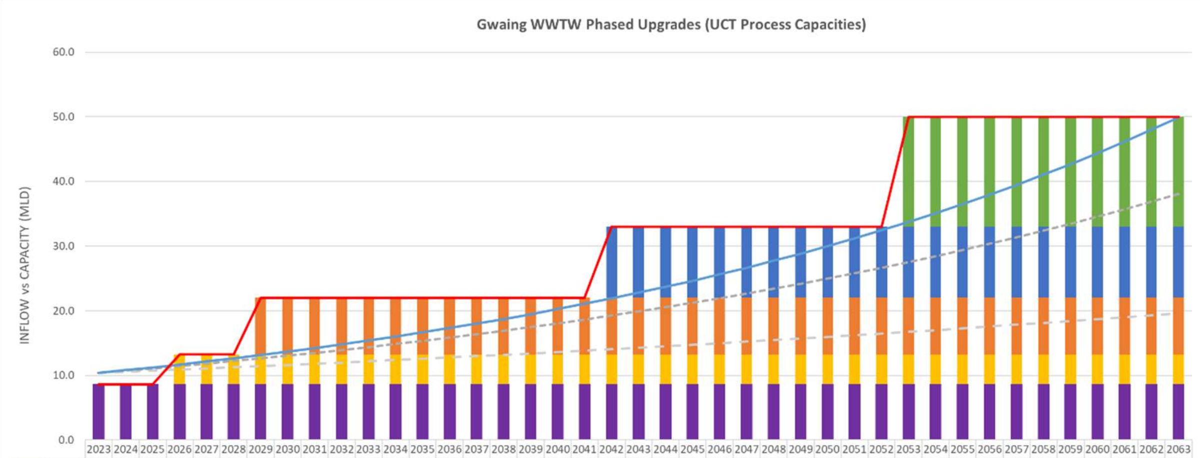


Figure 1-1: Phased Implementation Site Layout

The phased implementation must prioritize maintaining the capacity of the Gwaing WWTW ahead of the growing load. Figure 1-2 illustrates the essential timing for commissioning Phases A, B, C, and D. This timing ensures sufficient capacity at the works, even if the flow increases in line with the highest population growth projection of 4%. Additionally, the impact of lower growth rates on implementation timing is also evident. Note that the existing capacity in is shown as 8.6 MLD since this graph is produced for the UCT process (and not MLE) since this is the preferred process for the future of Gwaing WWTW.



	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042
Phase D																				
Phase C																				11.0
Phase B							8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8
Phase A				4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6
Current	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6
1.6%	10.4	10.6	10.7	10.9	11.1	11.3	11.4	11.6	11.8	12.0	12.2	12.4	12.6	12.8	13.0	13.2	13.4	13.6	13.8	14.1
3.3%	10.4	10.7	11.1	11.5	11.8	12.2	12.6	13.1	13.5	13.9	14.4	14.9	15.4	15.9	16.4	16.9	17.5	18.1	18.7	19.3
4.0%	10.4	10.8	11.2	11.7	12.2	12.7	13.2	13.7	14.2	14.8	15.4	16.0	16.7	17.3	18.0	18.7	19.5	20.3	21.1	21.9
TOTAL	8.6	8.6	8.6	13.2	13.2	13.2	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	33.0

	2043	2044	2045	2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063
Phase D											17.0	17.0	17.0	17.0	17.0	17.0	17.0	17.0	17.0	17.0	17.0
Phase C	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0
Phase B	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8
Phase A	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6
Current	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6
1.6%	14.3	14.5	14.7	15.0	15.2	15.5	15.7	16.0	16.2	16.5	16.7	17.0	17.3	17.6	17.8	18.1	18.4	18.7	19.0	19.3	19.6
3.3%	19.9	20.6	21.2	21.9	22.7	23.4	24.2	25.0	25.8	26.7	27.5	28.5	29.4	30.4	31.4	32.4	33.5	34.6	35.7	36.9	38.1
4.0%	22.8	23.7	24.6	25.6	26.7	27.7	28.8	30.0	31.2	32.4	33.7	35.1	36.5	37.9	39.5	41.0	42.7	44.4	46.2	48.0	49.9
TOTAL	33.0	33.0	33.0	33.0	33.0	33.0	33.0	33.0	33.0	33.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0

Figure 1-2: Phased capacity increases vs. increasing incoming flow rate projections for Gwaing WWTW.

The capital costs for the entire Gwaing WWTW upgrades according to the proposed phased implementation are shown in Table 1-2.

Table 1-2: Combined Civil and M&E capital cost estimate for all phases

Combined Cost Estimate - 2025 Rates		**TOTAL COST 50 MLD**		**BBF Construction**		**Phase A - 13.2 MLD UCT**		**Phase B - 22 MLD UCT**		**Phase C**		**Phase D**
Civil Cost Estimate	R	848 440 684.35	R	164 360 845	R	101 763 309	R	284 721 621	R	175 145 801	R	122 449 108
M&E Cost Estimate	R	796 885 657.02	R	75 240 000	R	67 706 766	R	338 034 717	R	160 818 718	R	155 085 456
TOTAL Excl VAT:	**R**	**1 645 326 341**	**R**	**239 600 845**	**R**	**169 470 075**	**R**	**622 756 338**	**R**	**335 964 520**	**R**	**277 534 563**

The Gwaing WWTW Master Plan serves as the blueprint for the plant's future upgrades over the next 40 to 50 years. It provides guidance for informed decision-making regarding the facility. By considering the entire scope, the full impact of each decision can be appreciated.

The vision for Gwaing WWTW extends beyond waste management. It aims to transform the facility into a Water Resource Recovery Facility (WRRF), emphasizing resource recovery. Key strategies include:

- Regional grit processing facilities to enable reuse of grit as part of composting or fill material.
- Regional screenings processing facility to minimise volume, odours, pathogens and vector attraction of screenings.
- Sludge beneficiation in the form of composting or fertilizer production is envisaged.
- The methane gas produced from anaerobic digestion will be used for generating heat and power (as part of Phase D).
- Effluent from the Gwaing WWTW can in future be pumped to neighbouring industries or golf courses for non-potable use. Alternatively, it can be further treated together with the effluent from Outeniqua WWTW before it is pumped to the Garden Route Dam as part of an indirect potable reuse scheme.

- Effluent will be recycled and pressurized on-site in a wash water ring main for various uses including irrigation, reducing the potable water demand of the WWTW.
- Energy efficient design principles will be used to reduce the power consumption of the plant, while a solar PV plant will both provide backup power during loadshedding events and shift the plant’s reliance from the national grid to renewable energy sources.

It remains crucial to ensure that the Gwaing WWTW’s primary task—producing compliant effluent—is executed effectively and consistently. This objective takes precedence over secondary goals like energy efficiency or automation.

It is recommended that the phasing as shown in Figure 1-2 be implemented. To facilitate this, completing the detailed design for Phases A and B promptly is essential. This step will solidify the design and pave the way for implementing the remaining phases. Additionally, having completed detail designs will enhance the Municipality’s readiness for implementation, making it easier to secure funding for the upgrades.

Given that the Gwaing WWTW is operating at the edge of its capacity, it is imperative to accelerate the implementation of at least Phase A. Doing so will ensure that the effluent from the works remains compliant. Similarly, the detail design and planning for Phase B should not be delayed to ensure that this phase can be commissioned before 2029 when the load on the plant is projected to exceed the capacity created by the implementation of Phase A. It would make sense to procure Phases A and B simultaneously, but to prioritize the scope of Phase A during implementation of this project.

The Gwaing BBF is poised to transform the way sludge is handled and perceived in the local market. New regulations are making the beneficiation of sludge a necessity. The Gwaing BBF will ensure that sludge handling complies to regulations and will facilitate a circular economy for sludge.

2 INTRODUCTION

2.1 Background

The Gwaing Wastewater Treatment works is one of the two major wastewater treatment works in George, Western Cape. Minor upgrades were recently completed, and the plant now has a total average dry weather flow (ADWF) capacity of 8.6 MLD when operating a UCT process at a design COD concentration of 782 mgCOD/l. The plant currently receives an ADWF of 10.7 MLD per day. The plant is operating over capacity. In addition, the population growth rate in George makes the extension of the wastewater treatment works a priority.

It is George Municipality's objective to upgrade the Gwaing WWTW and for the upgrade to comply with all current and relevant South African codes and standards. George Municipality appointed Lukhozi Consulting Engineers (Pty) Ltd (LCE) to provide professional engineering services necessary to implement the upgrade of the Gwaing WWTW to 21 MLD capacity. However, the first work package entailed the completion of a Master Plan (this Report) that will serve as a guiding document for all current and future upgrades to Gwaing WWTW.

2.2 Purpose of report

The purpose of the Master Plan is to provide a long-term solution to reach/achieve the ultimate capacity of 50 MLD. With the ultimate solution in mind, intermediate upgrades can be implemented over time to increase the capacity of the plant in small increments as budget allows and demand requires. The purpose of first designing a Master Plan with an ultimate solution, is to ensure that the intermediate upgrades align with the ultimate solution to prevent fruitless and wasteful expenditure. It also ensures the optimised use of spatial requirements on a site that has many spatial constraints. This report will present different phasing options for implementation as well as capital and operation/maintenance costs associated with the upgrades.

3 STUDY AREA AND WASTEWATER CHARACTERISTICS

3.1 Study Area

The Gwaing Wastewater Treatment Works falls under the jurisdiction of the George Municipality, situated in the Garden Route District within the Western Cape Province.

Figure 3-1 below indicates the position of the Gwaing WWTW in relation to George.

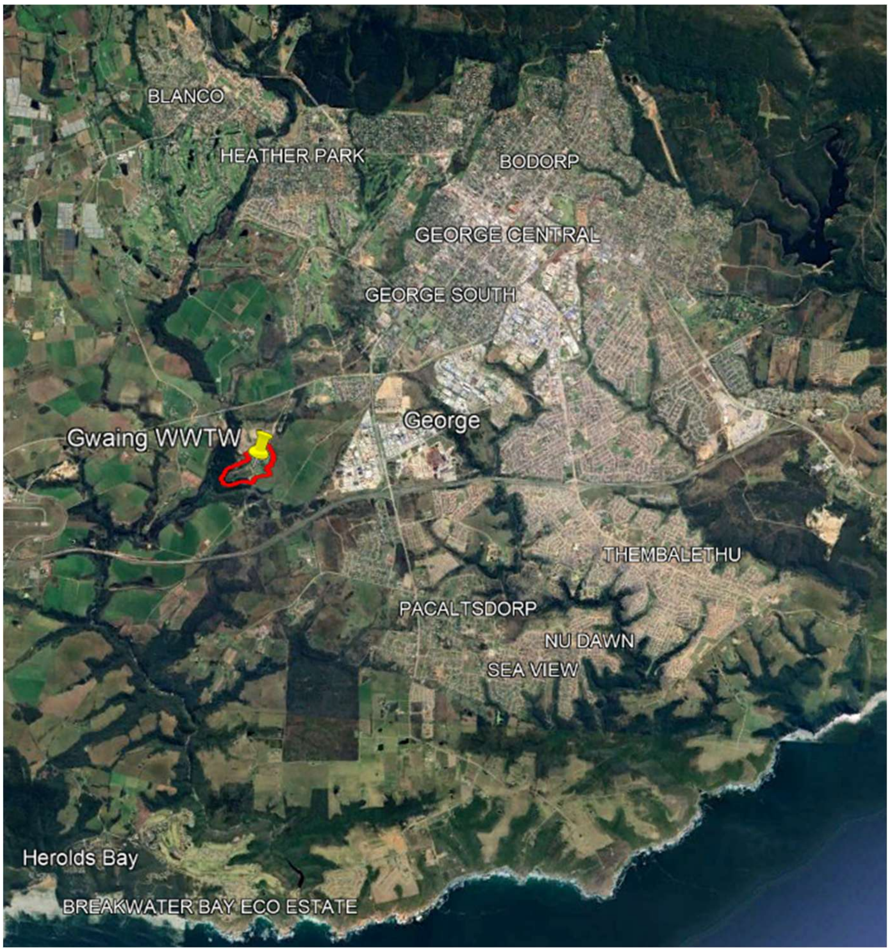


Figure 3-1: Gwaing WWTW Location (Google Earth)

The Gwaing WWTW is one of the two major wastewater treatment works serving the George area. The drainage area within George served by the Gwaing WWTW is highlighted in red in Figure 3-2.

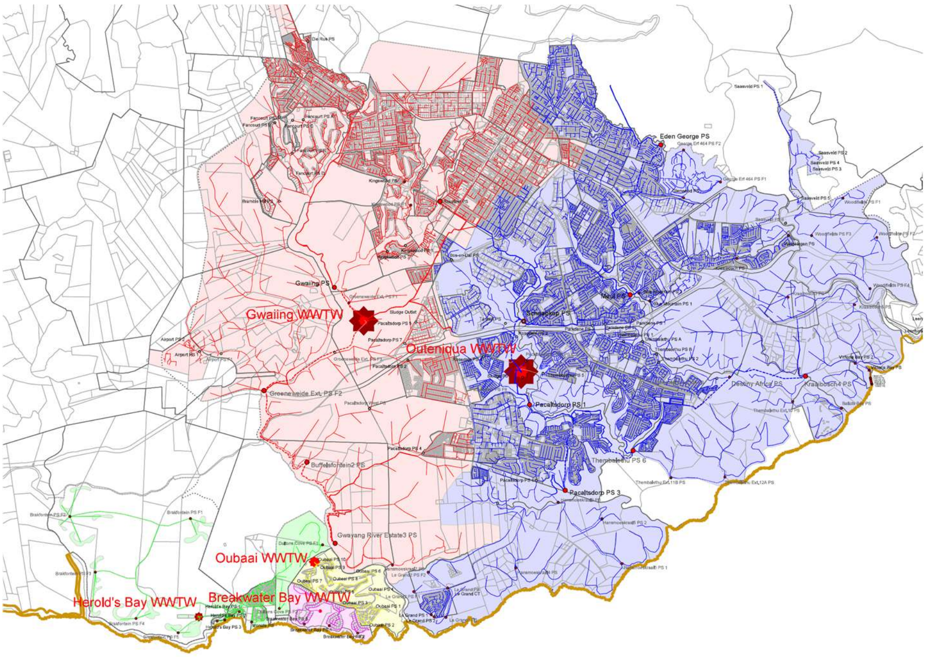


Figure 3-2: Gwaing Catchment Drainage Area (Master Planning of George Sewer Plan, 2022)

The proposed upgrades and refurbishments, as defined in the Sewer Master Plan are contained within the site boundary of the wastewater treatment works. The site boundary and aerial image of the existing infrastructure at Gwaing WWTW are shown in Figure 3-3 below.

Figure 3-3: Gwaing Site Boundary and Existing Infrastructure (Google Earth Image taken February 2023)

3.2 Population Projection

The George Municipal jurisdiction area is 5191 km^2 and spans the Southern Cape and Little Karoo regions of the Western Cape Province and is situated halfway between Cape Town and Gqberha (Port Elizabeth). The area administered by the George Municipality forms part of the larger Garden Route District Municipality's jurisdictional area.

The George city area is the primary urban centre of the Municipality with 84% of the municipal area's population located in the city. According to the Western Cape Government/Statistics SA, in 2011 the total population for George was estimated at 193 672. The 2016 Community Household Survey estimated George's total population to be 204,197 people or 61,441 households. The GMSDF of 2019 (George Municipal Spatial Development Framework, 2019) projected that the population will grow to 248,779 people by 2023, however, according to the 2022 Census, the population far surpassed the prediction with a total population of 294,929 (in 2022). Table 3-1 shows the growth percentages as per Census data provided by StatsSA.

Table 3-1: Population growth according to StatsSA Census Data

Year	Population	Growth %
2001	149 974	
2011	193 672	2.6%
2022	294 929	3.9%

According to the spatial development framework (GMSDF), a declining population growth rate per annum is evident, with the rate having dropped from a rate of 2.6% between 2001 and 2011 to 1.1% between 2011 and 2016. Conservative projections suggested that this growth rate would pick up slightly to 1.6% per annum between 2016 and 2023 (GMSDF, 2019). From the estimated population in 2018, the growth rate between 2016 and 2018 was 2%, which is an increase from the previous recorded period. Hence the observed growth population growth rates seem to be generally higher than predicted by the GMSDF of 2019.

Overall, the average annual growth rate observed from 2001 to 2022 according to StatsSA census data was 3.3%. The growth rates shown in Table 3-2 are comparatively used in the design when determining future flow rates.

Table 3-2: Population growth rates used for design

Growth %	Comment
1.6%	**Low growth scenario:** Growth rate prediction by GMSDF 2019
3.3%	**Medium growth scenario:** Growth rate observed between 2001 and 2022 according to census data.
4.0%	**High growth scenario:** Conservative growth rate. Since 2020 there has been an influx of people from other provinces into the Western Cape. There are many developments currently taking place in George that were not included in the GMSDF of 2019. For a conservative approach, this higher growth rate should be considered.

The growth rate does not impact the design of the ultimate capacity of the plant. The growth rate and how it plays out in the future will only impact the timing of the implementation of the phased upgrades towards the ultimate capacity. If the growth rate is near the upper limit (around 4%), the implementation of future phases will be sooner than if the actual growth rate is around 1.6%. The phasing options and timing are discussed in Section 7.

3.3 Wastewater Flows

3.3.1 Wastewater flow rates

Flow data for Gwaing WWTW's inflow totalizer and outflow totalizer was supplied by the Municipality. The inflow totalizer provides flow data from 28 November 2022 until 30 May 2023. The calibration of the inflow totalizer was however faulty, as the average flow during this period was 260 MLD, which does not make sense. The inflow totalizer data was therefore not used.

At the time of submitting this Concept Design report, the inflow totalizer had been replaced with a new level sensor upstream of the Parshall flume. In addition, the two Parshall flumes directly upstream of the main flume were also equipped with level sensors. Gauge plates were installed so that operators can easily verify digital readings with the observed flow rates. A reliable data set was obtained from 8 December 2023 to 28 February 2025 and is summarised in Table 3-3.

Table 3-3: Influent flow data from December 2023 to February 2025

	Dec '23 Jan '24	Feb '24	Mar '24	Apr '24 May '24	Jun '24	Jul '24	Aug '24	Sep '24 Oct '24	Nov ' 24	Dec '24	Jan '25 Feb '25
Channel 1	4.81	4.19	4.15	4.74	6.33	4.36	4.05	4.90	6.59	3.93	4.17
Channel 2	5.57	4.90	4.73	4.97	6.69	5.02	4.53	6.03	4.50	4.26	5.58
Channel 1+2	10.38	9.09	8.88	9.70	13.02	9.39	8.59	10.93	11.09	8.18	9.76
Common Channel	10.40	9.11	9.28	10.16	13.18	10.16	8.79	11.15	11.53		12.86

Figure 3-4 ows the data with instantaneous flow rates plotted approximately every 40 seconds for the full period. The diurnal pattern is clearly visible with the daily peak flow at around 11:00 and the daily low flow at around 5:00. The average flow rate for the period was **10.7 MLD**. There is enough confidence in this data for this to be adopted as the baseline ADWF currently received by Gwaing WWTW.

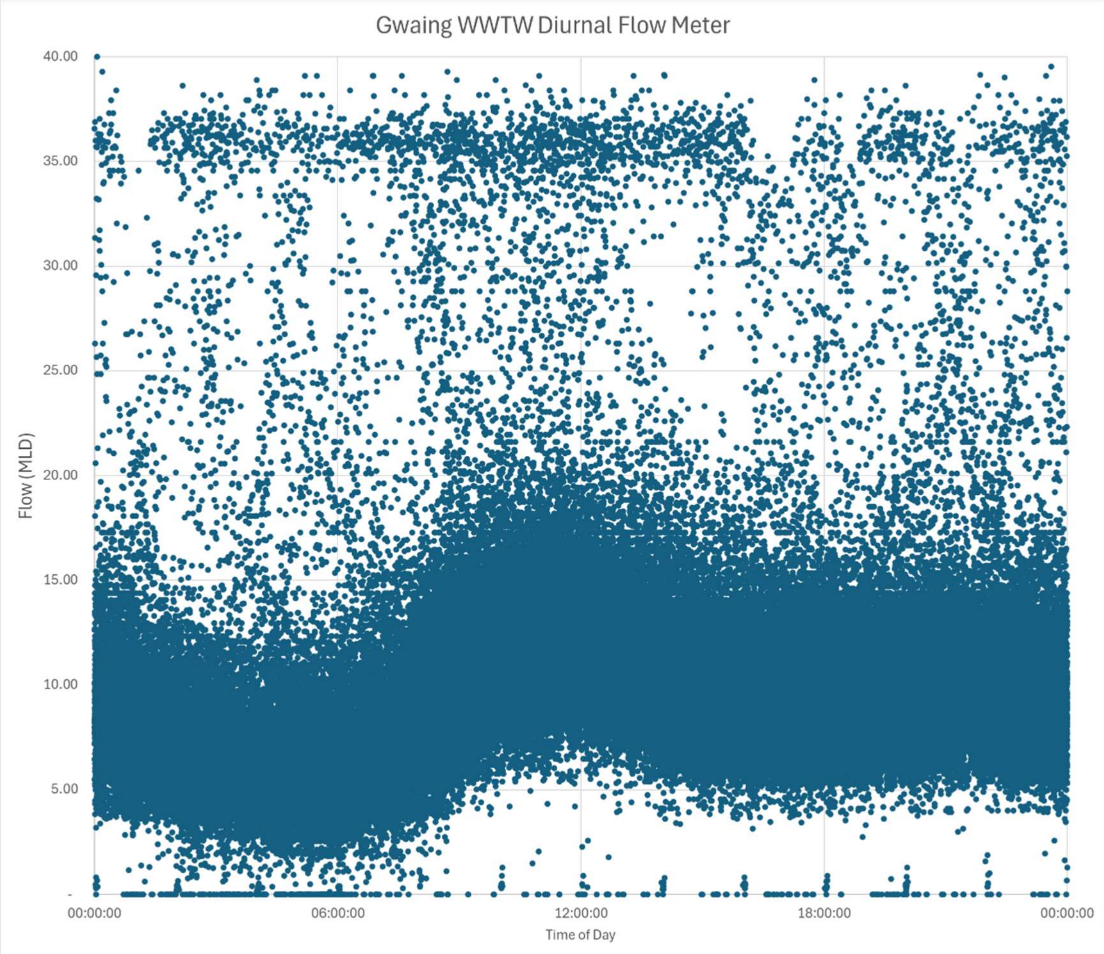


Figure 3-4: Diurnal flow pattern based on new level sensors at the inlet works Parshall flumes

3.3.2 Design flows

The design flow for the ultimate solution is 50 MLD for average dry weather flow. This is the design brief supplied by the Municipality in correspondence with the approximate capacity that can be treated within the existing site boundaries. The dry weather peaking factor was calculated theoretically according to the population size in the catchment using the Harmon formula:

$$PF = 1 + \frac{14}{4 + \sqrt{P}}$$

where: PF = peaking factor; P = population equivalent, in thousands.

The Harmon peaking factor correlates well with the dry weather peaking factor observed in the flow data. The stormwater infiltration selected for process design purposes is 30%. However, due to a history of isolated peak flows at Gwaing WWTW that exceed the PWWF designs, the hydraulic capacity of structures is designed for a stormwater infiltration of 50%. It was agreed with the

Municipality that a peaking factor (PWWF/ADWF) of 3 should be used for the inlet works to ensure that it can handle extreme wet weather peaks.

The peaking factors for PDWF and PWWF as well as the design flows used for the ultimate solution for 50 MLD are summarised in Table 3-4.

Table 3-4: Ultimate solution design flow summary

Characteristic	Unit	Process Design	Hydraulic Design	Inlet Works Hydraulic Design
Equivalent per capita flow rate	l/p/d	100	100	
Population equivalent	Pax	500 000	500 000	
Harmon Peaking Factor (PF) PDWF/ADWF Factor		1.53	1.53	
Stormwater infiltration		30%	50%	
PWWF/PDWF Factor		1.3	1.3	
PWWF/ADWF Factor		2.0	2.3	3.0*
Average Dry Weather Flow (ADWF)	MLD	50	50	50
Peak Dry Weather Flow (PDWF)	MLD	77	77	
Peak Wet Weather Flow (PWWF)	MLD	100	115	150

**It was agreed with George Municipality that the Inlet Works be hydraulically designed for a PWWF/ADWF factor of 3.*

3.3.3 Future flow projections

Flow projections based on Table 3-2 growth rate percentages for a 50-year projection are graphically shown in Figure 3-5. As discussed in Section 3.2, the growth rate does not impact the design of the plant, it will only impact the timing of the implementation of the phased upgrades towards the ultimate capacity.

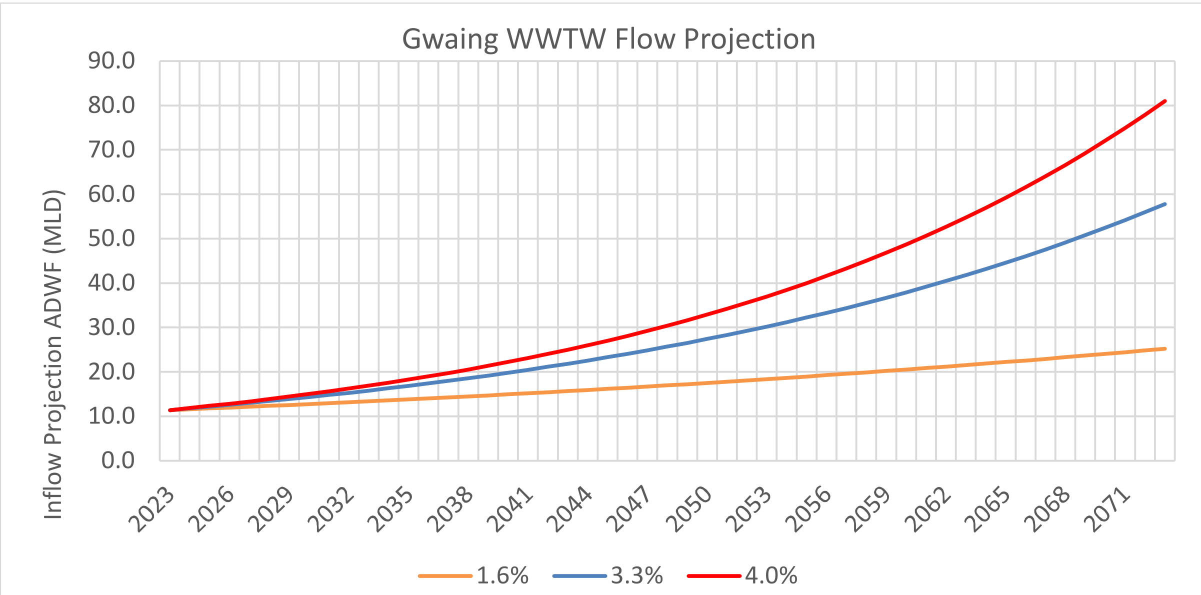


Figure 3-5: Gwaing WWTW flow projection graph

3.3.4 Emergency overflow strategy

A forward overflow strategy has been adopted for the design of processes and structures. Each structure is given an overflow and/or bypass option. In the case where the bypass or emergency overflow is blocked, the penstocks and internal walls of structures are designed at such a level, that the water will overflow in a forward direction and overflow to the next unit process. This is a worst-case scenario if the water or flow cannot be contained within a certain structure.

The alternative would be to provide emergency overflows of untreated or partially treated wastewater that discharge to a lined pond from where it can be pumped back to the inlet works when the overflow event ends. This option is not favoured since it creates odour, sedimentation and clogging risks at the emergency overflow pond.

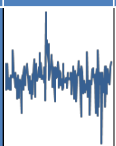
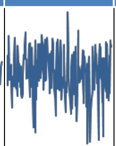
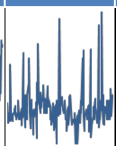
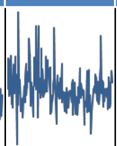
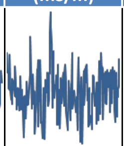
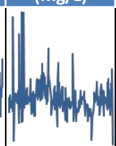
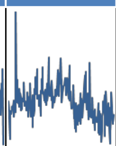
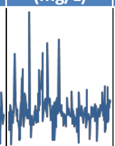
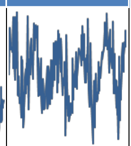
3.4 Wastewater Influent Characteristics

3.4.1 George Municipality samples

George Municipality samples the raw incoming wastewater every week (on a Tuesday) and tests for key wastewater parameters. A comprehensive data set has been developed and data from January 2018 to November 2022 was used for the design. The values highlighted in light blue in Table 3-5 have been used for design.

It may seem conservative to design for the 95th percentile. However, if a high degree of compliance is sought it is prudent to design for the worst case. Furthermore, it should be considered that the sampling data includes some dilution from stormwater infiltration and since George receives high annual rainfall, this is not insignificant. Using a higher percentile for design offsets this 'error' in the data.

Table 3-5: Summary of raw wastewater characteristics for Gwaing WWTW Jan 2018 - Nov 2022.

Parameter	Alkalinity as mg/l CaCO3	Ammonia as mg/l N	Chloride as mg/l Cl	COD (mg/L)	Electrical Conductivity (mS/m)	Ortho - Phosphate (mg/L)	pH	Suspended solids (mg/L)	Temperature (°C)
Variability over period									
minimum	25	4	80	0	55	0	6.89	3	**14.8**
10th percentile	200	36	110	323	71	3.4	7.1	153	18.2
25th percentile	**224**	42	120	398	83	4.2	7.2	205	19.6
Average	248	53	136	494	93	5.2	**7.3**	272	21.6
75th percentile	270	65	**144**	561	**103**	6.0	7.4	**312**	23.6
90th percentile	296	74	156	687	110	7.1	7.5	415	**25.1**
95th percentile	315	76	180	**782**	117	**8.0**	7.6	475	25.9
Maximum	480	96	298	1189	151	15.2	8.31	1077	27.3

The ammonia concentrations in Table 3-5 are shown in red since it was found to be higher than typically expected. The composite sampling discussed in Section 3.4.2 confirmed this and therefore the weekly sampling ammonia results were not used for design. Instead, the TKN/COD ratio of 0.10 found in the composite sampling results was adopted for the design. This means an incoming TKN concentration of 78.2 mgN/l was used for the design. Using a typical ammonia/TKN ratio of 0.75, the influent ammonia concentration is estimated at 58.7 mgN/l.

3.4.2 Composite sample test results

A supplementary composite sampling and testing regime was conducted from 3 August 2021 to 9 August 2021 by A.L Abbott and Associates as directed by iX Engineers. UCT assisted with lab-scale bioreactor tests to determine the fractionization of the COD. The main observation from this sampling data set was that the influent ammonia and TKN/COD ratio was lower than that derived from the weekly samples. The unbiodegradable particulate fraction of the COD is slightly higher than typical values used.

Table 3-6: Composite sample test results (August 2021)

	COD (mg/L)	COD (settled) (mg/L)	Ammonia as N (mg/L)	TKN (mg/L)	Total Phosphate as P (mg/L)	Ortho Phosphate as P (mg/L)
Average	522.3	176.4	36.7	46.9	6.2	4.1
95th percentile	759.0	233.7	41.3	52.9	6.7	5.6
Maximum	816	249	42.5	53.3	6.8	5.7

	Total Suspended Solids (mg/L)	Volatile Suspended Solids (mg/L)	pH (at 25 Deg)	Alkalinity as CaCO3 (mg/L)	Conductivity (mS/m)	Temp. (˚C)
Average	195.4	164.4	6.6	257.0	102.8	18.6
95th percentile	257.8	218.6	6.7	298.1	112.2	19.0
Maximum	277	224	6.75	302	114	19

3.4.3 Design parameters

The wastewater ‘strength’ as tested is diluted by rainwater infiltration and this needs to be taken into consideration when interpreting the data for high rainfall areas such as George. To account for this and to ensure that the plant remains compliant throughout the year the 95th percentile was used in design for ammonia, COD and ortho-phosphates.

3.4.3.1 Chemical Oxygen Demand (COD)

The municipal data and the composite sample data COD results follow a similar trend. A value of **782mgCOD/l** was selected for the raw influent COD. This is based on the 95th percentile of the George Municipality weekly samples. The value is slightly higher than the 95th percentile of the composite sample results, which recorded 759 mgCOD/l. Selecting the slightly higher COD for design purposes is a more conservative approach.

At least 30% of the COD concentration can be removed with primary settling. The concentration of settled COD used in the design is **547.4 mgCOD/l**. Higher COD removal rates can be achieved in the PST’s, although this will affect the TKN/COD ratio negatively, increasing effluent nitrate or phosphate concentrations. The current raw incoming COD load is therefore 11.4 MLD x 782 mgCOD/l = 8915 kgCOD/d.

3.4.3.2 Unbiodegradable COD Fractions

Wastewater contains particulate organic COD and soluble organic COD that are not biodegradable in the activated sludge model. Unbiodegradable particulate organic (UPO) COD settles out in the PST or overflows to the reactor and subsequently forms part of the reactor VSS mass. It only exits the system via the PST underflow (if settleable) or via the reactor waste stream and is not utilised in the system. Unbiodegradable soluble organic (USO) COD similarly is not utilised in the activated sludge system. Being soluble, it does not settle out in the PST or SST but escapes with the final effluent.

Unbiodegradable Soluble Fraction

The average USO COD fraction as measured of the composite sampling was 0.08 mgCOD/mgCOD. However, the typical range for USO COD is between 0.06 and 0.08 mgCOD/mgCOD with the recommended design USO COD fraction being **0.06 mgCOD/mgCOD**. The Client requested that the typical design value be used for design purposes.

Unbiodegradable Particulate Fraction

The average UPO COD fraction as measured of the composite sampling was 0.21 mgCOD/mgCOD. The typical range for UPO COD fractions ranges from 0.10 to 0.18 mgCOD/mgCOD. A value of **0.14 mgCOD/mgCOD**, which is the centre of the typical range was used for design purposes for raw wastewater. For settled wastewater **0.06 mgCOD/mgCOD** was used.

Soluble COD and VFA's

The average soluble COD fraction from the composite sampling was 0.37. However, a more conservative and typical value of **0.3** was adopted for the design. The influent soluble COD used for the design is therefore at 782 x 0.3 = **235 mgCOD/l**. Of this soluble COD it was estimated that **50 mgCOD/l** is present as volatile fatty acids (VFA's).

3.4.3.3 Total Kjeldahl Nitrogen (TKN) and TKN/COD ratio

The TKN values are not tested by the Municipality, the only nitrogen values that are tested is the ammonia. From the composite sample results, the average TKN/COD ratio was **0.10**. This ratio was adopted for design. The raw TKN based on a TKN/COD ratio of 0.10 is therefore **78.2 mgN/l**. Typically 15% of the TKN concentration is removed through primary settling. The settled TKN value is therefore **66.47 mgN/l**. The TKN/COD ratios for raw and settled wastewater are 0.10 and 0.12 respectively. This is on the high end of typical ranges found in domestic wastewater, but effluent nitrate standards can still be met, and biological phosphate removal is still effective at these ratios and external COD dosing is not required.

3.4.3.4 Ammonia (FSA)

The ammonia concentrations in the municipal data sampling are very high. The average ammonia concentration recorded by the municipality is 53 mgN/l whereas the composite sampling average ammonia result is 36.7 mgN/l. With the 95th percentile TKN concentration calculated as 78.2 mgN/l and using a typical ammonia/TKN ratio of 0.75, the influent ammonia concentration (95th percentile) is estimated at 58.7 mgN/l.

3.4.3.5 Maximum Specific Growth Rate of Nitrifiers (μ_{Am20})

The maximum specific growth rate of nitrifiers (at 20°C) ranges from 0.3 to 0.75 and has a significant effect on the sludge age required for nitrification. What exactly determines μ_{Am20} for a specific wastewater is not well understood although it has been shown that industrial effluent can reduce the growth rate of nitrifiers. For Gwaing WWTW a μ_{Am20} was assumed to be **0.45/d** for design purposes.

3.4.3.6 Suspended Solids

The raw influent wastewater suspended solids concentration used for design is the 75th percentile value of **312 mgTSS/l** from the weekly sampling data. The reason for not using a higher percentile value was that the maximum suspended solids concentration recorded during the week of composite sampling was 277 mgTSS/l and therefore the 75th percentile concentration was considered conservative enough. Primary settling removes about 55% of the TSS and 70% of ISS found in raw wastewater. These values were adopted for the design of settled wastewater. The suspended solids concentrations used for the design are summarised in Table 3-7.

Table 3-7: Suspended solids concentrations used in process design

Suspended Solids	Raw Concentration	Settled Concentration	Unit
TSS	312	140	mgTSS/l
VSS	250	122	mgVSS/l
ISS	62	19	mgISS/l

3.4.3.7 Ortho Phosphates

The 95th percentile influent ortho-phosphate concentration of the weekly samples was 8.0 mgP/l. This is higher than all the results from the composite sampling, but it was conservatively adopted for design. This concentration is still lower than the effluent standard of 10 mgP/l, which means that enhanced biological phosphate removal (EBPR) is not necessarily required in the process. However, it will be beneficial for the receiving water bodies to still include EBPR and thereby achieve lower effluent phosphate concentrations, limiting the risks of eutrophication in the river.

3.4.3.8 Temperature

The wastewater temperatures at Gwaing WWTW vary seasonally. The minimum temperature represents the worst-case scenario for nitrification in the biological process. The maximum temperature results in the maximum aeration requirements for the biological process. The minimum and maximum temperatures used in the design are 14˚C and 25˚C respectively. This is based approximately on the minimum and 90th percentile temperatures recorded in the municipal sampling data set shown in Table 3-5.

3.4.3.9 *Summary of wastewater design characteristics*

The wastewater characteristics used for the Master Plan design are summarised in Table 3-8.

Table 3-8: Wastewater Characteristics used in design

Wastewater Characteristic	Unit	Raw	Settled
COD	mg/L	782	547
TKN	mg/L	78.2	66.5
TKN/COD Ratio		0.10	0.12
Ammonia	mgN/l	42.5	42.5
TSS	mg/L	312	140
Ortho-Posphates	mgP/L	8.0	8.0
Unbiodegradable Particulate fraction of COD	mgCOD/mgCOD	0.14	0.06
Unbiodegradable Soluble fraction of COD	mgCOD/mgCOD	0.06	0.09

3.5 Effluent standards required according to the Water Use License

The Water Use License (WUL), dated 18 December 2015, stipulates the treated effluent compliance in terms of the General Limit Values as detailed in the Government Gazette of 6 September 2013, as shown in Table 3-9. The only deviation of the WUL is that E Coli is limited to 150 cfu/ 100 ml instead of the 1000 cfu/100 ml prescribed by the General Limit. Generally, the standard is achievable with a conventional BNR activated sludge plant including disinfection.

Table 3-9: Anticipated discharge Standards for the Gwaing WWTW based on the current 11 Mℓ/day WUL

Parameter	Units	General Limit	Current Water Use Licence Limit
Faecal coliforms	Count per 100 ml	1000	Not specified
E Coli	Count per 100 ml	Not specified	150
COD	mgCOD/l	75	75
pH		5.5-9.5	5.5-9.5
Ammonia (as N)	mgN/l	6.0	6.0
Nitrate (as N)	mgN/l	15	15
Chlorine as Free Chlorine	mg/l	0.25	0.25
Suspended Solids	mg/l	25	25
EC	m/mS	70*	70*
Ortho Phosphate (as P)	mgP/l	10	10
Fluoride	mg/l	1	1
Soap, oil and grease	mg/l	2.5	2.5

* 70 above intake to a maximum of 150 mS/m

Various reuse options are viable from Gwaing WWTW and to achieve them further tertiary treatment will be required. The reuse options and their associated treatment standards are discussed in Section 6.1.13.

3.6 Geotechnical Investigation Findings

Terra Geotechnical conducted a geotechnical investigation for the Gwaing WWTW upgrades. As part of the investigation 14 test pits were excavated (see Figure 3-6) and supported with 27 DPSH tests around the site.

Figure 3-6: Geotechnical Test Positions (Test Pits)

The geotechnical report is attached in Appendix A: Geotechnical Report. Based on the geotechnical investigation, several key findings have been identified:

1. **Transported Material Removal:** Due to the variable and organic nature of the upper transported material across the site, it is recommended to remove it to a depth of at least 300 mm beyond the perimeter of the proposed developments. Variations in this depth should be assessed during planned earthworks.
2. **Heave and Consolidation:** The soils covering the site may experience heave and/or consolidation (volume loss and gain) under loading or when saturated. Adequate strengthening of structures is necessary to prevent structural damage due to differential settlement beneath foundations.
3. **Moisture-Induced Differential Movements:** Differential movements will be exaggerated due to heave and shrinkage when moisture conditions change beneath structures.
4. **Foundation Recommendations:**
    a. For single- and double-storey structures, reinforced concrete strip/pad foundations are recommended.
    b. The foundation medium should achieve a minimum of 95% Mod AASHTO density or less than 20 mm penetration per blow of a Dynamic Cone Penetrometer (DCP).
    c. A recommended founding depth of 1 meter below the natural ground level (NGL) or below the transported soils ensures stability.
    d. Bearing pressures should not exceed 150 kPa to limit settlement.
    e. For heavier structures, consider deeper foundations (to weathered granite) or introduce imported structural fill.
    f. Light reinforced concrete rafts may also be suitable.
5. **Erodibility of Material:** The granitic soils encountered across the site are prone to erosion.
6. **Dispersive Soils:** Backfill should match the compaction of surrounding soil to avoid up-slope groundwater diversion and tunnel erosion.
7. **Slope Stability and Temporary Cuttings:**
    a. In general safe battering to 45° is proposed as a safe cut-back for deep excavations.
    b. Long-term stability decreases due to reduced cohesion and increased friction (safe cut slopes as low as 25°).
    c. Reworked residual granite remains stable if dry but can slump when subjected to standing water.
8. **Dewatering Measures:** Implement dewatering measures for open unsupported excavations prone to flooding. Safety precautions are crucial for excavations deeper than 1.5 meters.

Groundwater Occurrence: Perched groundwater seepage was observed across the site, generally with slow to moderate flow. Ferruginous material indicates seasonal fluctuating groundwater or excessive soil moisture movement.

4 EXISTING PLANT ASSESSMENT

4.1 Capacity analysis

A steady-state activated sludge model was used to determine the capacity of the existing Gwaing WWTW. The two main activated sludge processes that were modelled, were the modified Ludzack-Ettinger (MLE) process and the UCT process. Historically the plant was operated as a UCT process. However, the Municipality implemented mechanical upgrades to the plant to change from a UCT process to an MLE process to gain additional capacity as the plant is currently overloaded. Figure 4-1 shows the effect that the influent COD concentrations have on the plant's capacity with a constant volume and a constant sludge age of 20 days. Specifically shown on Figure 4-1 is the plant's capacity at an COD concentration of 700 mg/l (historical COD design concentration) and at this masterplan's COD design concentration of 782 mg/l.

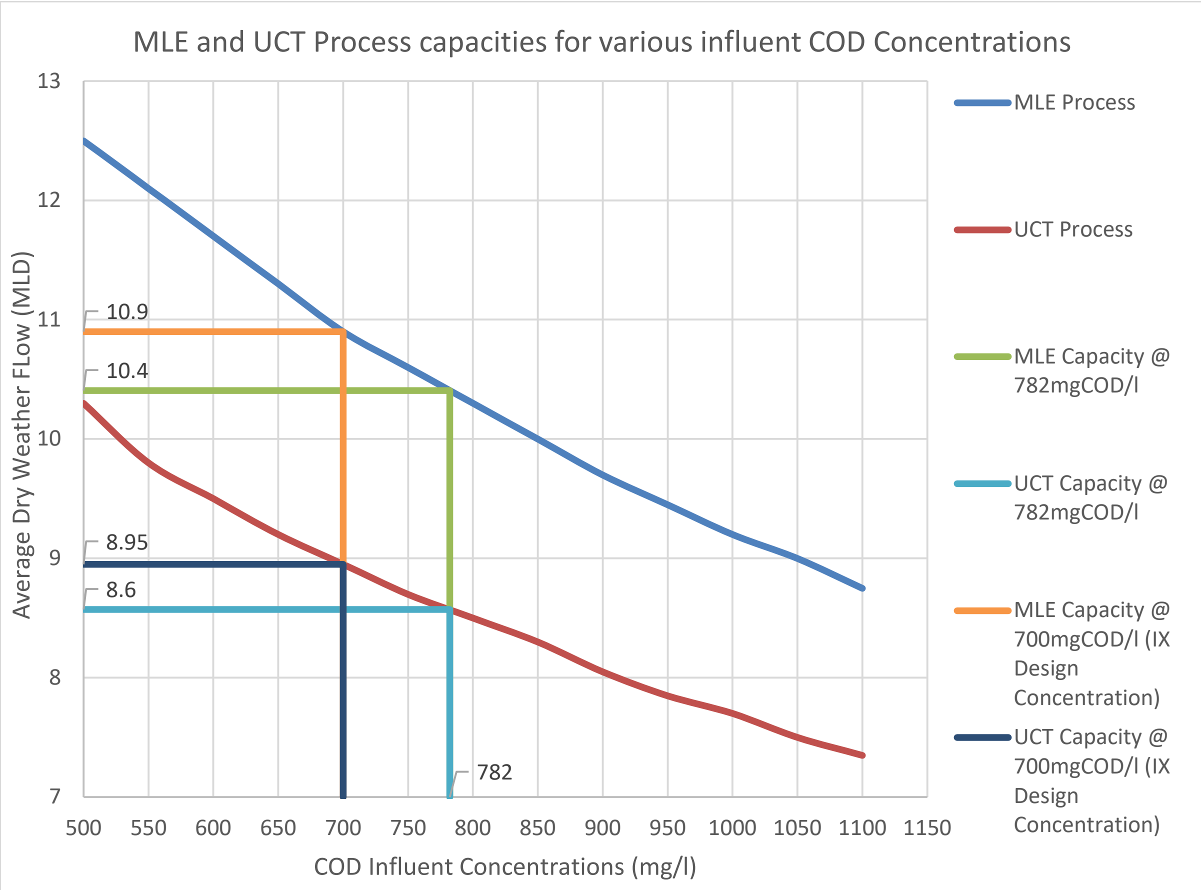


Figure 4-1: MLE and UCT process capacities (existing reactor) for various influent COD concentrations

The existing capacity is based on the operational existing infrastructure, which includes a biological reactor, with MLE and UCT operability and two secondary clarifiers of 25 m diameter each. There is no primary clarification currently in operation. Table 4-1 summarises the existing capacity of the plant for both the MLE and UCT processes based on a COD concentration of 782 mg/l.

Table 4-1: Existing plant capacity summary

Parameter	UCT Process (Raw WW)	MLE Process (Raw WW)
Raw influent flow (Ml/d)	8.6	10.4
Raw COD concentration (mgCOD/l)	782	782
COD flux (tCOD/d)	6.6	8.1
Minimum Temperature (°C)	14	14
Max Temperature (°C)	25	25
Sludge Age (days)	20	20
Reactor Volume (m^3)	14 864	14 864
Reactor concentration (mgTSS/l)	3850	3450
DSVI (ml/g)	135	135

4.2 Existing facilities and operations

The Gwaing WWTW contains various process units and equipment, of which some are currently in use, and others have been decommissioned. The existing plant is divided into two modules, namely the Bio-Trickling Filter module and the activated sludge module. Figure 4-2 shows the layout of the two modules. The head of works and the tertiary treatment are common to both modules. The current operation of the plant only makes use of the activated sludge module. The Bio-Trickling filter module has been decommissioned and is not in use.

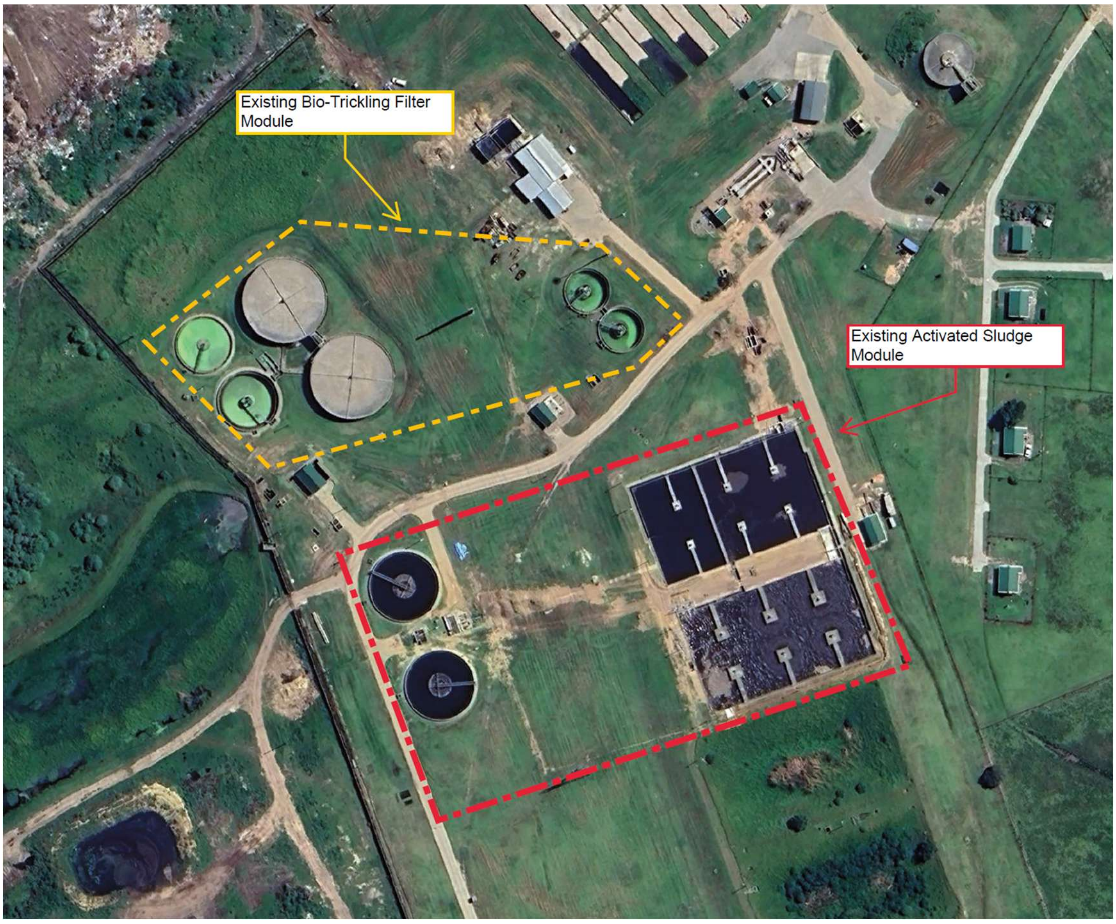


Figure 4-2: Gwaing WWTW existing plant layout (Google Earth Image taken February 2023)

4.2.1 Inlet Works

The Inlet Works consists of the following Infrastructure:

- 1 x Course Screen (12 mm appertures)
- 2 x Mechanical Drum Screens (6 mm appertures)
- 1 x 3 m dia. Vortex Degritter
- 3 x Degritter Channels (one of which has been changed to a bypass channel to bypass the collection chamber of the inlet works).

The inlet works is currently in working order and was designed to serve both the Bio-Trickling filter module and the Activated sludge module. The hydraulic capacity of the inlet works is sufficient for the current influent flows. However, the single vortex degritter is undersized. It will sufficiently remove grit up to 11 MLD, which matches the ADWF approximately, however for greater flow rates, that are experienced during peak dry weather flow (PDWF) or during peak wet weather flow (PWWF) the degritter will be undersized and grit will flow into the downstream processes such as the reactor.

The Parshall flume used for flow measurement is situated about 1m from where the 3 degritter channels join together into a common channel. The flow is extremely turbulent when it flows through the Parshall flume and thus inaccurate flow measurements are likely. Level sensors were installed at the Parshall flumes of the degritter channels to achieve more accurate flow measurements.

The position and layout of the existing inlet works make it challenging to upgrade the inlet works to a greater capacity. If in future phases, primary clarifiers are introduced, there is not sufficient hydraulic head between the inlet works and the existing biological reactor to include an additional process between the two.

4.2.2 Activated sludge module: Biological nutrient removal (BNR) reactor and secondary settling tanks (SSTs)

The existing activated sludge plant comprises of the following main components:

- Biological reactor – UCT and MLE process.
    - Anaerobic zone – 2 x vertical shaft mixers.
    - Anoxic zone – 4 x vertical shaft mixers.
    - Aerobic zone – 6 surface-mounted aerators.
        - 2 x 55 kW aerators.
        - 4 x 75 kW aerators.
    - a-recycle pumps (3)
    - r-recycle pumps (2)
- 2 x return activated sludge (RAS) pumps.
- 2 x waste activated sludge (WAS) pumps.
- 2 x 25 m diameter secondary settling tanks [flat bottom].

4.2.2.1 Biological Reactor

The BNR reactor is in a good functioning condition. It can operate in either the UCT or MLE process configuration. The reactor is currently operating as an MLE process and is operating at capacity. If high COD loads, or peak flows are experienced, the secondary clarifiers may experience sludge carry over. Although in working order, the current mechanical mixers and surface aerators in the reactor have a high energy demand and are not energy efficient.

4.2.2.2 Secondary Settling Tanks

The secondary settling tanks and associated mechanical equipment are in good working order. Occasionally the waste activated sludge dewatering system is out of operation, this means the waste flow from secondary settling tanks is stopped which leads to the MLSS concentration in the reactors increasing, causing sludge carry-over from the SSTs. The side wall depth of the SSTs is 3.5 m which is marginally less than the desired depth of 4 m. The SSTs have flat bottom floors with a syphon lift sludge withdrawal system mounted on a half-bridge. The concrete launder is positioned inside the SST circumference, creating the Stamford baffle effect which prevents sludge density currents from increasing the effluent TSS concentration.

4.2.2.3 RAS and WAS Pumpstations

The return activated sludge (RAS) and waste activated sludge (WAS) pumps withdraw from the same sump which is situated between the existing SSTs. For proper sludge age control, it is better to waste from the reactor since the reactor MLSS concentration is relatively stable and the mass of sludge wasted is therefore better controlled. Wasting from the SSTs underflow (RAS) gives a higher concentration of sludge (thicker) which is easier to dewater, however it poses challenges for sludge age control since the variation in the concentration of the RAS stream varies considerably. A mechanical upgrade was completed in October 2023 where pumps were installed at the reactor to waste directly from the reactor instead of wasting from the RAS sump.

4.2.3 Bio-trickling filter module (Decommissioned)

The bio-trickling filters comprise of the following main components:

- 2 x 15.5 m diameter PSTs
- 20 x sludge drying beds
- 2 x 30 m diameter Bio-trickling filters (4m deep) and associated mechanical equipment.
- 2 x 20 m diameter humus tanks
- 18 m diameter, 3000 m^3 capacity Anaerobic digester for stabilisation of primary and humus sludge.

4.2.3.1 Bio-trickling filters

The existing Bio-trickling filters are the oldest part of the Gwaing WWTW, the module has been decommissioned and is currently inoperable. Bio-trickling filters perform relatively well in COD removal and nitrification of ammonia. It performs poorly however in denitrification and will not be able to meet the effluent standard.

An investigation was done by an independent laboratory in May 2023 to determine the structural integrity of the Bio-trickling filters. It was determined that the existing Bio-trickling filters cannot be re-purposed as a water retaining structure since the walls are not integrated with the floor and the condition of the concrete and reinforcement drastically deteriorated over the years. To reinstate the trickling filters will require extensive repair work involving the removal of the stones, potential replacement with plastic carrier media and repair of the concrete walls. There are also some of the underground pipes that are blocked or broken that will need to be assessed and repaired or replaced. It is not advisable to spend more funds on an old technology that cannot meet the effluent standard consistently.

Figure 4-3: Cracks visible in bio-trickling filter walls

4.2.3.2 Primary settling tanks (PSTs)

The decommissioned PSTs are currently inoperable. Vandalism to the installed equipment and potentially dilapidated underground sludge withdrawal pipework impact the feasibility of reinstating the PST structures and ancillary equipment to its original functionality. At an overflow rate of 1.2 m/h (at ADWF) the two PSTs are undersized for the existing biological reactors for both a settled MLE and a settled UCT process. The PSTs are also constructed at a lower level than the current biological reactors and thus due to hydraulic constraints cannot be utilised with the activated sludge module.

The feed and outflow pipelines of the PSTs as well as the underflow sludge line would need thorough investigation to determine the feasibility of reinstating the PSTs. The existing PSTs can possibly be used as sludge thickeners before dewatering or digestion if new PSTs with sufficient capacity are constructed at a higher level, this is discussed in more detail in Section 5.5. Figure 4-4 shows an image of the inside of one of the existing PSTs.

Figure 4-4: Inside of Existing PST

4.2.3.3 Anaerobic Digestor

The Bio-trickling filter/Humus tank configuration as well as the PSTs cannot function without a fully operational Anaerobic digester to digest and stabilise the primary- and humus sludge. The Anaerobic digester has not operated in the last 17 years. Significant refurbishment of the pipework and the mixing system is required on the anaerobic digestor. The condition of the concrete also needs to be assessed. A detailed investigation of the existing concrete work and internal mechanical work would need to be conducted to determine the feasibility of reinstating or refurbishing the existing anaerobic digestor.

4.2.4 Maturation ponds

Gwaing WWTW has four maturation ponds of approximately equal size. The total area of the ponds is 44 000 m^2. At an approximate depth of 1.5 m, this equates to a volume of 66 000 m^3. At the current ADWF a retention time of 6 days is achieved as shown in Table 4-2.

Table 4-2: Maturation Pond Retention Time Summary

Parameter	Units	Flow Rate (MLD)		
		ADWF	PDWF	PWWF
		10.4	21	27
Area	m^2	44000		
Depth	m	1.5		
Volume	m^3	66000		
Retention Time	hrs	152	75	59
	Days	6.3	3.1	2.4

The maturation ponds are partially filled with sludge carry over from the secondary settling tanks that has accumulated in the ponds over the years. The first two ponds will most likely have more sludge than the latter ponds. This sludge will digest and reduce in volume over time, however there is a buildup of unbiodegradable residue over time. It would be good practice to clean the ponds approximately once in every 5 years.

The maturation pond configuration resembles a horseshoe, with effluent flowing in an anti-clockwise direction. The area between the ponds is being used for sludge stockpiling, which cannot be deemed either a temporary or long-term solution. There are regulatory issues associated with this practice that should be addressed. The removal of sludge should be a priority as part of the first planned upgrade. Since neither the sludge stockpiling area between the ponds, nor the ponds themselves are lined, the nutrients from the sludge seeps into the maturation ponds and the effluent quality is negatively affected.

Figure 4-5: Aerial image of Maturation Ponds

4.2.5 Chlorine Disinfection

The existing Chlorine Contact Disinfection Infrastructure consists of the following:

- Chlorine dosing building and equipment
- Chlorine contact tank

The chlorine contact dosing system and tank is in working condition. The contact tank is however on the upper limit of its hydraulic capacity. At the current PWWF the retention time is approximately 20 mins.

Remedial work to the pipework feeding the chlorine contact tank as well as the discharge pipeline is required. The outflow discharge structure from where water flows into the Gwaing River, is hydraulically undersized for current peak flow events. It is also apparent that the last maturation pond has been overtopped at times when reeds have blocked the outlet structure. These issues are being addressed in the short term donga rehabilitation project.

4.2.6 Waste Activated Sludge (WAS) Dewatering Facility

The dewatering facility is operational, despite some minor mechanical issues. The Dewatering infrastructure installation at Gwaing consists of the following infrastructure:

- Sludge Holding Tanks
  - 2 off 190 m^3 sludge holding tanks in Duty/Duty operation or parallel configuration
    - In Duty/Duty mode, the facility can dewater Outeniqua- and Gwaing sludge separately
    - In Parallel mode, Outeniqua- and Gwaing sludge can be blended before dewatering
  - Mixing/aeration, each tank is fitted with:
    - 1 off venturi aerator (can only operate when the tank is full due to air entrainment), and;
    - 1 off submersible mixer.
- Sludge Feed Pumps
  - Filter belt press (FBP) 1 is serviced by 2 x progressive cavity, variable frequency drive (VFD) controlled, positive displacement pumps
  - FBP 2 is serviced by 2 x progressive cavity, VFD controlled, positive displacement pumps
- Instrumentation
  - FBP 1
    - 1 x Turbidity Probe
    - 1 x Flow Meter
  - FBP 2
    - 1 x Turbidity Probe
    - 1 x Flow Meter
- Polyelectrolyte
  - Shared, package-type polyelectrolyte make-up unit
  - Dosing to FBP 1
    - VFD controlled, positive displacement pump
    - Dosing inline before orifice plate mixer
  - Dosing to FBP 2
    - VFD controlled, positive displacement pump
    - Dosing inline before orifice plate mixer
- Wash water
  - Wash water tanks
  - 2 x Vertical, multi-stage, centrifugal pumps in Duty/Standby configuration for FBP 1
  - 2 x Vertical, multi-stage, centrifugal pumps in Duty/Standby configuration for FBP 2
- Filter belt presses (FBP) equipment:
  - FBP 1 comprises of:
    - 1.8 meter wide linear, Gravity Belt Thickener table
    - 1.8 meter wide Filter Belt Press
  - FBP 2 comprises of:
    - 1.8 meter wide linear, Gravity Belt Thickener table

        - 1.8 meter wide Filter Belt Press
    - FBP 3 comprises of:
        - 1.2 meter wide Filter Belt Press (not in use)
- Dewatered sludge handling
    - Sludge Conveyor, 600mm wide – shared,
    - 2 x 3t Sludge skips for dewatered cake storage

The sludge dewatering capacity of the filter belt presses at Gwaing WWTW is given in Table 4-3.

Table 4-3: Dewatering Capacity of Filter Beltpresses installed at Gwaing WWTW

Filter Belt Presses	Hydraulic Capacity		Solids Loading Capacity	
	(m^3/hr)	(m^3/24 hr d)	(kg/hr)	(kg/24 hr d)
FBP No 1 1.8 m nominal belt complete with integral 1.5 m linear table	58	1392	525	12600
FBP No 2 1.8 m nominal belt complete with integral 1.5 m linear table	58	1392	525	12600
FBP No 3 1.2 m nominal belt without linear table	10	240	350	8400

4.2.7 Sludge Handling/Disposal

George Municipality's current sludge disposal method is not compliant with sludge management guidelines with the sludge being stored between the maturation ponds in an unlined area. This causes seepage of nutrients to the maturation ponds and underlying aquifer. The sludge produced currently is classified as class B1a. This places restrictions on how the sludge can be utilised due to the presence of some microbiological contaminants.

4.3 Chemical and Energy Usage

The Existing Gwaing WWTW does not require many chemicals to function since the process is mainly biological. The main chemical consumption is chlorine gas for disinfection and polyelectrolyte (Zetag 7557) powder for sludge dewatering.

The BNR Reactor is equipped with surface aerators that account for the largest energy usage on site. The reactor was designed with sloped sides which makes it difficult to retrofit it with more efficient fine bubble diffused aeration (FBDA) systems. As discussed in Section 5.3.2.1 the decision was made to remain with surface aeration in the existing reactor while future reactors are designed for FBDA systems. Surface aeration is generally simpler and more reliable, so thereby a balance between reliability and efficiency can be achieved.

The opportunity to optimise mixing energy usage in the anaerobic and anoxic zones will be investigated during detail design. The existing mixers are most likely of the backswept radial mixer type with non-clogging plate. New mixer technologies could be employed that reduce the mixing requirements from the typical range of 5 W/m^3 to 2 W/m^3.

At present, the anaerobic digestors are not operational. Generally for a plant of this size anaerobic digestion including a CHP generator is not a net producer of power since mixing of the digestor sludge also requires a significant amount of energy. The energy benefit however is realized through the introduction of PSTs that divert the COD load and thereby reduce the aeration requirements per ML treated in the reactor by 30% to 50%. The existing PSTs cannot be used for the BNR reactor because it is constructed at a lower level than the reactors and is undersized for future requirements.

Also see Section 4.9.1.

4.4 Strengths and weaknesses

Some of the key strengths and weaknesses of the existing Gwaing WWTW are summarized in Table 4-4 and Table 4-5 respectively.

Table 4-4: Existing Gwaing WWTW Strengths

Strengths
The BNR reactor and associated SSTs are a simple reliable process that achieves the effluent standards.
WAS dewatering infrastructure is in working condition.
Chlorine gas disinfection infrastructure is in working condition.
A solar PV plant will be implemented soon that will assist with backup power supply during loadshedding. Only portions of the WWTW will be powered by the solar PV plant to reduce the use of generators.

Table 4-5: Existing Gwaing WWTW Weaknesses

Weaknesses
The disposal of waste activated sludge (WAS) has not been resolved. Currently, sludge is stockpiled between the maturation ponds.
The inlet works is too small for future requirements and its level is too low to accommodate PSTs upstream of BNR Reactors.
The maturation ponds suffer from vegetation overgrowth and eutrophication, potentially due to nutrient seepage from the sludge stockpile.
The effluent outfall from the final maturation pond and/or chlorine contact tank has created an erosion ‘donga’ during peak flow events.
The bio-trickling filter infrastructure has deteriorated structurally. The process is not able to meet the effluent nitrate standard.

4.5 Regulatory Requirements

4.5.1 Water Use License

The Water Use License (WUL), dated 18 December 2015, stipulates the treated effluent compliance requirements in terms of the General Limit concentrations as detailed in the Government Gazette of 6 September 2013. See Section 3.5.

4.5.2 Sludge Handling

4.5.2.1 Water Services Act (Act 108 of 1997)

The Water Services Act (WSA) 108 of 1997 mandates the Minister responsible for water and sanitation to prescribe compulsory national norms and standards in accordance with Sections 9 and 10 of the Act. The National norms and standards for domestic water and sanitation services (GN R. 982 of 2017; DWS, 2017) set out the national norms and standards for levels of water services, including sanitation, which will be applicable from 2017 until the Minister requests another revision.

According to section 6.2.4 of the norms and standards, wastewater sludge management must adhere to the Guidelines for the utilisation and disposal of wastewater sludge, Volumes 1 – 5 (Sludge Guidelines):

• Guidelines for the utilisation and disposal of wastewater sludge Volume 1: Selection of management options (Snyman & Herselman, 2006a);

• Guidelines for the utilisation and disposal of wastewater sludge Volume 2: Requirements for agricultural use of wastewater sludge (Snyman & Herselman, 2006b);

• Guidelines for the utilisation and disposal of wastewater sludge Volume 3: Requirements for the on-site and off-site disposal of sludge (Herselman & Snyman, 2009a);

• Guidelines for the utilisation and disposal of wastewater sludge Volume 4: Requirements for the beneficial use of sludge at high loading rates (Herselman & Snyman, 2009b); and

• Guidelines for the utilisation and disposal of wastewater sludge Volume 5: Requirements for thermal sludge management practices and for commercial products containing sludge (Herselman & Snyman, 2009c).

4.5.2.2 National Environmental Management: Waste Act (Act no. 59 of 2008)

Wastewater sludge falls in the definition of waste under National Environmental Management:

Waste Act (NEMWA): '…any substance, material or object, that is unwanted, rejected, abandoned, discarded or disposed of, by the holder of the substance, material or object, whether or not such substance, material or object can be re-used, recycled or recovered…'

Therefore, the waste regulations and norms and standards have to be considered in sludge management, especially when disposal is the preferred management option. The NEMWA norms and standards applicable to sludge storage and disposal are:

• National norms and standards for the storage of waste (GN R. 926 of 2013); and

• National norms and standards for the assessment of waste for landfill disposal (GN R.635 of 2013).

4.6 Proposed Improvements (not linked to capacity upgrades)

The following improvements are proposed for the existing plant:

- Construct a more centralized admin facility and process control centre for process controllers.
- Rehabilitate the erosion 'donga' at the chlorine contact tank and install pipework to prevent future erosion in this area. (Separate Contract ongoing: Refer to Section 7.3.1)
- Install better aeration equipment in the WAS sludge holding tanks that can aerate at different liquid depths.
- Discontinue sludge stockpiling at the maturation ponds – consider viable alternative options such as solar drying and fertilizer production – See Section 6.1.8.
- Install more energy efficient mixers in the existing Reactor A.
- Improve flow measurement at the inlet works (completed).

4.7 Site Constraints

4.7.1 Eskom Overhead Power Lines

There are two Eskom overhead powerlines running through the Gwaing WWTW site between the existing biological reactor and the existing SSTs. The powerlines run in a north-south direction as indicated in Figure 4-6.

A wayleave was issued by Eskom on 26/10/2023 with reference number 12245-23 for the approval of work surrounding the overhead power lines at Gwaing WWTW. The restrictions of construction around the overhead power lines are indicated in the extract from the wayleave in Figure 4-7. The Wayleave is not an approval to commence with construction, but an approval that states that Eskom has no objection to the planned construction. A separate wayleave needs to be applied for in order to commence with construction. The Wayleave issued by Eskom is valid for 12 months whereafter a reapplication is to be submitted.

The powerline to the right is a 132 kV line and the powerline on the left is a 66 kV line. In Clause 3(a) of the wayleave supplied by Eskom, as shown in the extract in Figure 4-7, it states that a building restriction on either side of a 66 kV line is 11 m and for a 132 kV line the restriction is 15.5 m. The total width of the servitude is 50 m. The servitude impacts the layout of infrastructure and restricts reactor and SST sizing and positions.

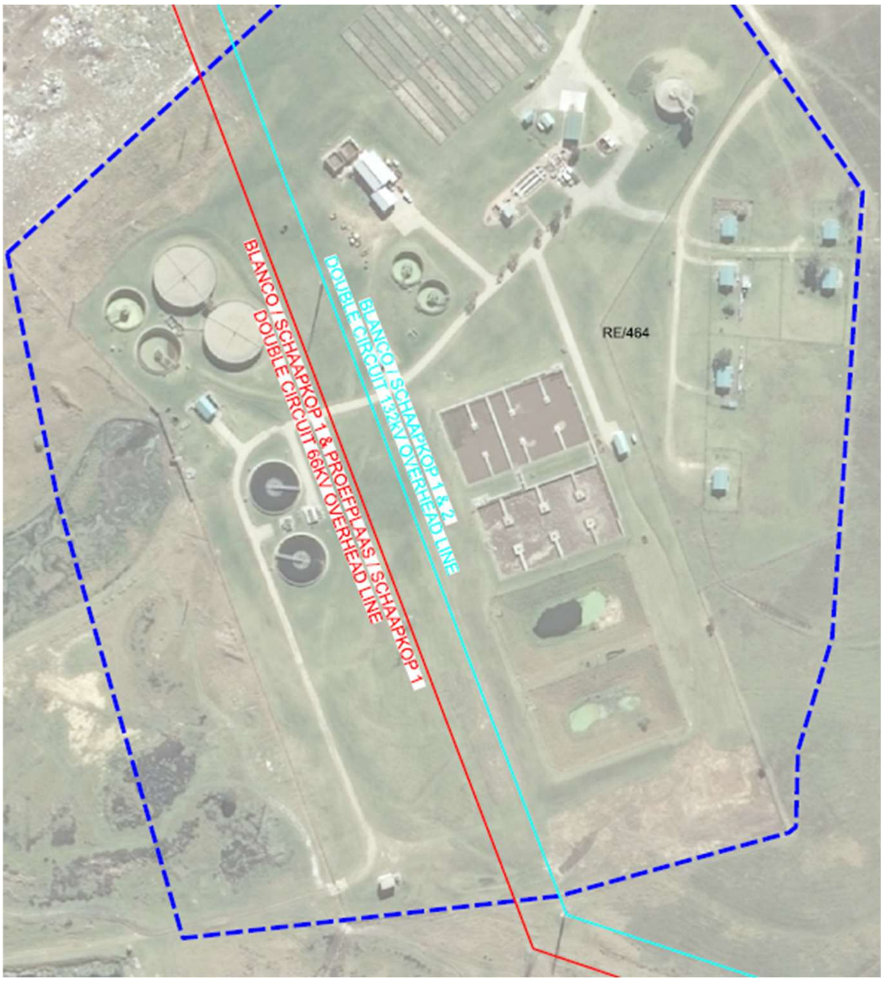


Figure 4-6: Eskom overhead powerline positions

3. **O.H. Line Services:**

a) The following building and tree restriction on **either side of centre line** of overhead power line must be observed:

Voltage	Building restriction either side of centre line
11 / 22kV	9.0 m
66kV	11.0 m
132kV	15.5 m

b) No construction work may be executed closer than **6 (SIX) metres** from any Eskom structure or structure-supporting mechanism.

c) No work or no machinery nearer than the following **distances from the conductors**:

Voltage	Not closer than:
11 / 22kV	3.0 m
66kV	3.2 m
132kV	3.8 m

d) Natural ground level must be maintained within Eskom reserve areas and servitudes.

e) That a **minimum ground clearance** of the overhead power line must be maintained to the following clearances:

Voltage	Safety clearance above road:
11 / 22kV	6.3 m
66kV	6.9 m
132kV	7.5 m

Figure 4-7: Extract from Eskom Wayleave for overhead powerline restrictions

4.7.2 George Municipality Overhead Powerline

George Municipality has an overhead powerline running on the western side of the existing secondary settling tanks. This is a single overhead power line and runs parallel to the Eskom power lines. The registered servitude for the powerline has a total width of 22 m (11 m on either side of the cable). The George Municipal power line together with the Eskom powerline influences the layout of the Master Plan solution. Although the Master Plan's ultimate capacity of 50 MLD is achievable while adhering to the restrictions of these two powerlines, it does to an extent dictate the layout of the ultimate solution. Figure 4-8 shows the extent of the two servitudes overlayed on the existing site layout.

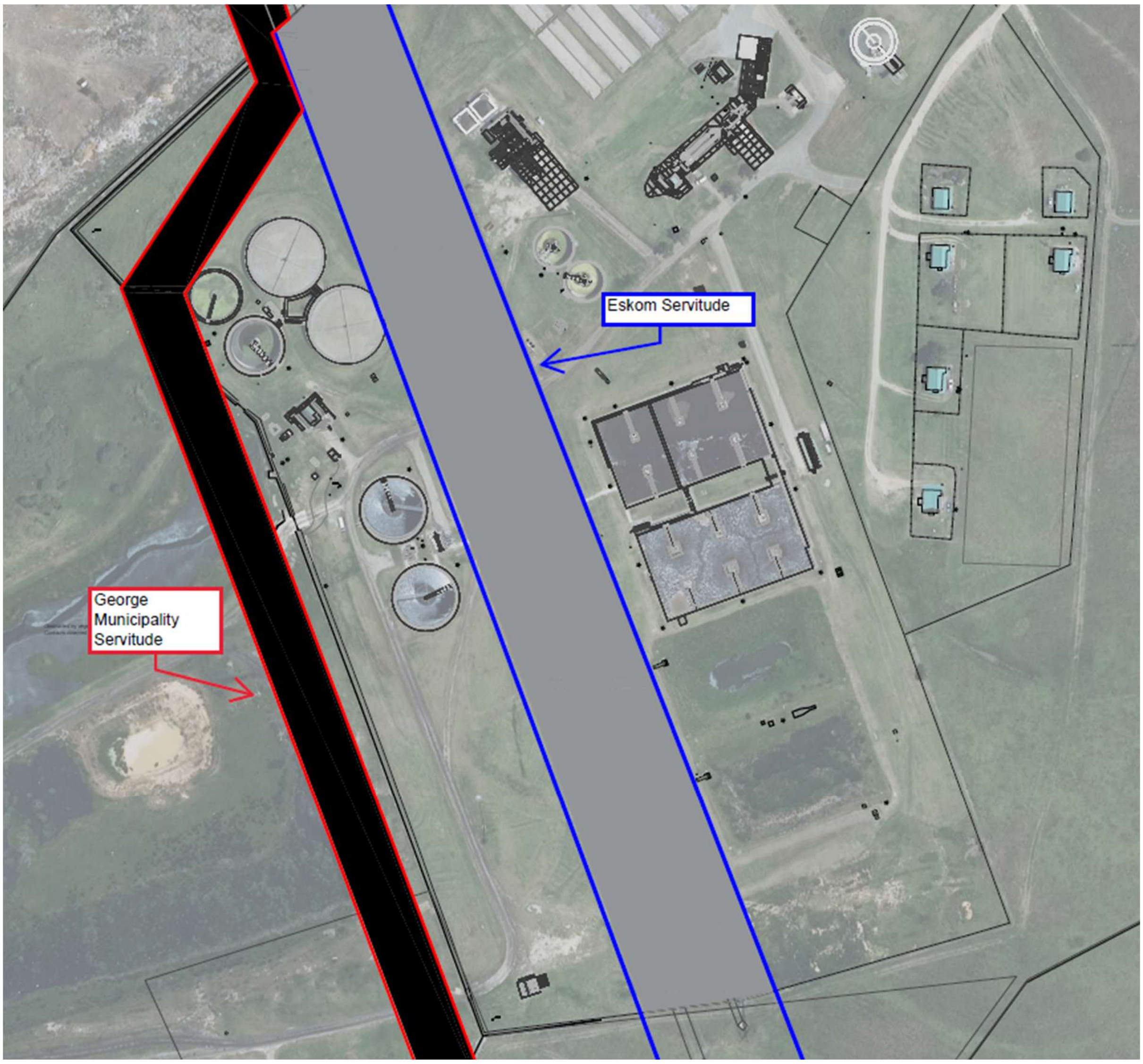


Figure 4-8: Eskom and George Municipality powerline servitudes

4.7.3 Site Topography

Gwaing WWTW is situated on a site that has a relatively steep gradient. The site falls from the North-East down to the South-West. The gradient of the site has both advantages and disadvantages. The advantage of the gradient is that structures can be constructed at ground level and there is sufficient fall between unit processes that the water can flow from the inlet works through to the outfall without intermittent pumping. The hydraulic gradient through the plant has a similar profile to the ground level. The disadvantage, however, is that it restricts the layout of the plant to fit in with the fall, it leaves little flexibility to optimise the layout for maximised usage of the site boundary. If unit processes are to be constructed in areas that do not follow the gradient of the natural ground level, structures will need to be either very deep in the ground, requiring large excavation work, or they will be elevated in the air and require large volumes of concrete for support structures the construction of earthworks. The gradient of Gwaing WWTW is of such a nature, that it can be utilized advantageously without uncommon amounts of earthworks and platform construction. The contours with schematic fall direction arrows are shown in Figure 4-9.

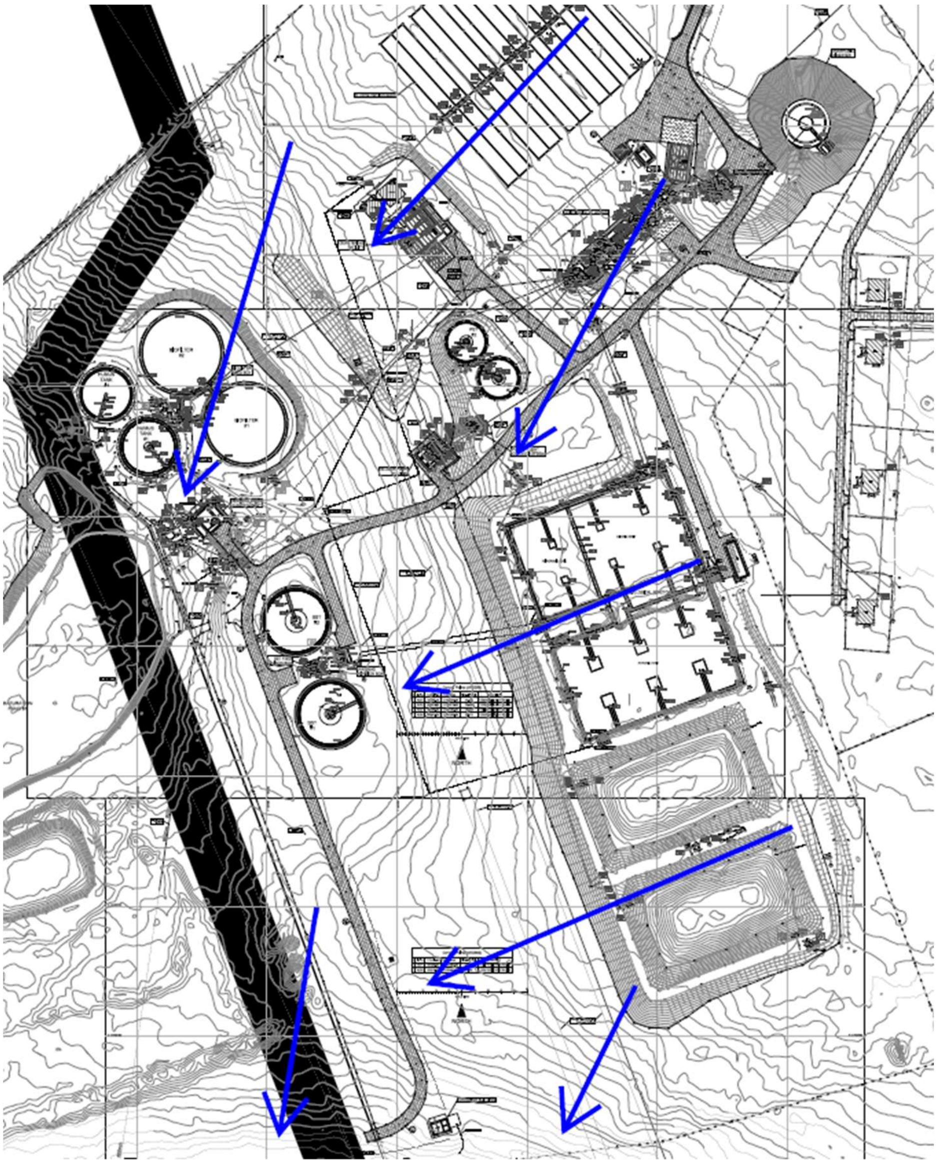


Figure 4-9: Site contours and fall directions

4.8 Personnel

The current works is classified as a Class A Works. According to George Municipality, the personnel on site are summarised in Table 4-6.

Table 4-6: Gwaing WWTW personnel on site

Position	Active Personnel	Vacant Positions	Total
Senior Manager	1	0	1
Superintendent: Operations	1	0	1
Chief Process Controller	1	0	1
Clerk	1	0	1
Process Controller	4	5	9
Water Purification Assistant	2	6	8
Supervisor Driver	0	1	1
Handyman	2	1	3
Wastewater Treatment Assistant	4	0	4
Total:	**16**	**13**	**29**

According to the Water Services Act of 1997's Regulations Relating To Compulsory National Standards For Process Controllers And Water Services Works, a Class A works requires 6 process controllers and a supervisor on site, with qualifications and skills as described in Table 6-20 of Section 6.5. The Gwaing WWTW currently has 16 active personnel and 13 vacant positions. According to the requirements as defined in the Water Services Act of 1997, a class A-works must always have a Class IV process controller on site and a Class V supervisor on standby. Depending on how Gwaing WWTW rotates its staff and their qualifications, it seems to have insufficient process controllers.

4.9 Green Drop Report 2022 and 2023 Assessments

The Green Drop programme seeks to induce changes in behaviour of individual institutions to facilitate continuous improvement and adoption of best practice management of wastewater collection and treatment systems. Consequently, progressive improvement and excellent performance are recognised and rewarded.

The Green Drop score reflects the status of the complete wastewater business over a period of 12 months based on a full Green Drop audit, whereas the Green Drop Risk Rating Assessment focuses on specific risk indictors at a specific moment in time, or a more prolonged period (i.e. 12-month period). This is undertaken by calculation of the Cumulative Risk Rating (CRR) of each Wastewater Treatment Works (WWTWs). The CRR calculation is a concise and focussed benchmarking exercise that extracts four key risk areas that would individually and collectively, give a snapshot view of the status of wastewater treatment.

Each Green Drop audit cycle is marked by incremental change in the audit criteria, guided by the status and priorities of wastewater sector. It is therefore important for Water Service Institutions (WSI) to note that merely maintaining the previous cycle's Green Drop evidence and performance will not warrant the same Green Drop score. The Green Drop Report of 2022 and 2023 for Gwaing WWTW was reviewed.

4.9.1 Energy efficiency (Green Drop Report 2022)

Part of the Green Drop report is a review of the energy consumption and efficiency of wastewater treatment plants. This diagnostic investigates the status of energy efficiency management at a provincial and municipal level to motivate improved operational wastewater treatment efficiency. The energy consumption was compared to industry benchmarks. Gwaing WWTW achieved a specific power consumption (SPC) score of 0.93 kWh/m^3. The SPC's in the Western Cape ranged from 0.5-3.6 kWh/m^3 with the average SPC for advanced treatment systems being 0.8 kWh/m^3. According to the Green Drop Report (2022), the industry benchmark for similar treatment plants ranges from 0.27 to 0.41 kWh/m^3. This shows that Gwaing WWTW's energy consumption is well above the benchmark and slightly above the average SPC in the Western Cape, which indicates that there is considerable opportunity for energy efficiency improvements.

4.9.2 Green Drop Score and Technical Site Assessment (Green Drop Report 2022)

The Green Drop (GD) process makes provision for the desktop audit to be followed by a Technical Site Assessment (TSA) to verify the desktop evidence. The assessment includes a physical inspection of the sewer network, pump stations, and treatment facility, coupled with asset condition checks to determine an approximate cost to restore existing infrastructure to functional status. A deviation of >10% between the GD and TSA score indicate a misalignment between the administrative aspects and the work on the ground. The Regulator regards a wastewater system with a TSA score of >80% as one that has an acceptable level of process control and functional equipment. 90% would represent an excellent plant that complies with most of the Green Drop TSA standards. An extract of the Gwaing WWTW results is shown in Figure 4-10.

WSA Name	TSA WWTW Name	WWTW GD Score (%)	%TSA	Key Hardware Problems	Difference between TSA and GD score
Breede Valley	De Doorns	75%	54%	1. Need to get the 2 x 20% A/S modules reconfigured and commissioned; 2. Sludge recycle pumps need to be working; 3. Sludge wasting; 4. Chlorine gas disinfection	21%
Theewaterskloof	Grabouw	87%	61%	1. Urgently desludge maturation dams and repair; 2. Repair weirs of clarifiers; 3. Repair composting plant; 4. Replace sludge thickening; 5. Implement more regular desludging	26%
Swellendam	Klipperivier	31%	54%	1. Unlined sludge ponds; 2. None of the mixers are operational, with phased repair; 3. Lined solar drying pad required	23%
Cape Agulhas	Bredasdorp	50%	67%	1. Unlined sludge ponds; 2. Network pump station needs fencing; 3. Staff Facilities needs improvements	17%
Hessequa	Heidelberg	36%	68%	There are no serious hardware issues	32%
Mossel Bay	Mossel Bay	92%	80%	There were no major hardware risks	12%
George	Gwaing	71%	70%	1. Erosion at CCT; 2. Sludge Stockpile; 3. Cow in inlet, major safety risk in reticulation network	1%
Knysna	Sedgefield	73%	75%	1. Clarity in CCT is poor, sludge present in CCT consider secondary clarification; 2. Problems with disinfection evident from poor micro-bio results; 3. Establish FE measurement point after final polishing (maturation Ponds); 4. Securing of the network pump station	2%

Figure 4-10: Technical Site Assessment extract from the Green Drop Report 2022

The Green Drop score and TSA score were similar at 70% and 71% respectively. This is lower than 80%, which shows that there is room for improvement in process and control and functional equipment. Given that the WWTW underwent an upgrade since the audit was conducted, there may already be an improvement in the scoring. One of the concerns mentioned is the erosion at the chlorine contact channel. This is currently being rectified with the Donga Rehabilitation project being in the detail design phase at the time writing this Report.

Figure 4-11 shows a summary of Gwaing's Green Drop score summary for 2009, 2011, 2013 and 2021. From the scoring, it is evident that the Gwaing's score has slightly decreased over the last decade. The Green Drop report recorded that only 68% of the plant's capacity of 11 MLD was utilized in 2021. The flow data received from the Municipality, as discussed in Section 3.3.1, shows an average dry weather flow of 11.4 MLD in 2022, which means that the capacity utilization as reported by the Green Drop may be slightly lower than reality.

Key Performance Area	Unit	Gwaing
Green Drop Score (2021)		71%
2013 Green Drop Score		91%
2011 Green Drop Score		95%
2009 Green Drop Score		83%
System Design Capacity	Ml/d	11
Design Capacity Utilisation (%)		68%
Resource Discharged into		Gwaing River
Wastewater Risk Rating (CRR% of CRR_{max})		Gwaing
CRR (2011)	%	40.9%
CRR (2013)	%	40.9%
CRR (2021)	%	40.9%

Figure 4-11: Gwaing WWTW Green Drop Score Summary

Gwaing WWTW's cumulative risk rating (CRR) as reported in the 2023 Green Drop Report showed an increased CRR from 40.9% to 52.4% from 2022 to 2023. Figure 4-12 shows a summary of the risk assessment as reported in the Green Drop Progress Report published on 05 December 2023. The CRR is explained below:

Cumulative Risk Rating (CRR) = A x B + C + D where:

- A = Design Capacity of a plant which also represents the hydraulic loading onto the receiving water body
- B = Operational flow exceeding-, on- and below capacity
- C = Number of non-compliance trends in terms of effluent quality as discharged to the receiving water body
- D = Compliance or non-compliance in terms of technical skills

Example 1: a 95% CRR% deviation value means the plant has only 5% space remaining before the system reaches its maximum critical state (100%).

Example 2: a 25% CRR% deviation value means the plant holds a low and manageable risk position and is not close to the limits that define a critical state (90-100%).

Risk Assessment Areas	Weight	Gwaing
Class of Works		A : Approved
Treatment Technology		Activated Sludge
A: Total Design Capacity	**kl/d**	**11000**
B: Operational Capacity (% inflow/design)	**%**	**73.9%**
C: Effluent Quality Non-compliance	**#**	**2**
% Microbiological Compliance	%	91.8%
% Physical Compliance	%	87.1%
% Chemical Compliance	%	95.3%
D: Technical Skills Compliance	**%**	**83.3%**
Process Controller Compliance	%	50%
Supervisor Compliance	%	100.0%
Maintenance Team Compliance	%	100.0%
CRR 2023 (%CRR/CRRmax)	**%**	**52.4%**
CRR 2022	**%**	**40.9%**
CRR 2013	**%**	**40.9%**
CRR 2011	**%**	**40.9%**
W2RAP Status: 2022 Green Drop Report		In planning stage
W2RAP Status: 2023 Green Drop PAT		Final document (approved by Council)
Capital & Refurbishment Projects (Rand in millions)		5
Description of Capital & Refurbishment Projects		Reinstatement of 3.5Ml/day biofilter module, refurbishment of belt presses and sludge stockpile area, paving of roads, repair of flood damaged maturation pond and donga.
2022 GD Score	%	71.0%
GD Improvement Plan (GDIP)	Y/N	Yes
Corrective Action Plan (CAP)	Y/N	No

Figure 4-12: Green Drop Report 2023 Risk Assessment summary

5 PROCESS DESIGN

5.1 Process Selection

The effluent standards required by the WULA, as shown in Table 3-9, are of such a nature that an activated sludge treatment process is required. The nitrate effluent standard is 15 mg/l, which requires a configuration that includes nitrification and denitrification. The phosphorus effluent standard is 10 mg/l, which is not a strict limit and can be achieved without an enhanced biological phosphorus removal (EBPR) configuration (depending on the influent). The two processes considered in this design are the Modified Ludzack-Ettinger (MLE) process and the UCT process. Implementing a conventional activated sludge plant for the ultimate capacity has various advantages, some of which are as follows:

a. The existing plant only has a single reactor in operation, making it difficult to do maintenance on the tank or mechanical equipment when needed. More reactors will provide a valuable degree of redundancy.
b. Reactors and SSTs are tried and tested technologies that provide a high degree of reliability.
c. CAS reactors and SSTs are well understood by designers and operators and the operation thereof is comparatively simple.
d. Additional reactors will provide the opportunity to install more efficient aeration equipment such as fine bubble diffusers and blowers in the new reactors. This has the potential to reduce the aeration power consumption of the new reactor by 50% when compared to surface aeration, which is extremely valuable currently with the national deficit in power supply. It will also make it more feasible to operate the plant with solar power as is currently envisaged.
e. The new reactor and SST infrastructure are expected to last for more than 50 years compared to costly repairs to old infrastructure without the expectation that it will last as long.

5.1.1 Modified Ludzack-Ettinger (MLE) Process

The MLE process is a nitrification-denitrification system utilizing the biodegradable organics in the influent as organics for denitrification. The MLE process has two basins in series partially separated from each other. The first basin is the anoxic zone, in which denitrification takes place, and the second basin is the aerobic zone, in which nitrification takes place. With the same volume reactor, and all other constants being the same, the MLE process has a larger treatment capacity than the UCT process. The MLE process configuration is shown in Figure 5-1.

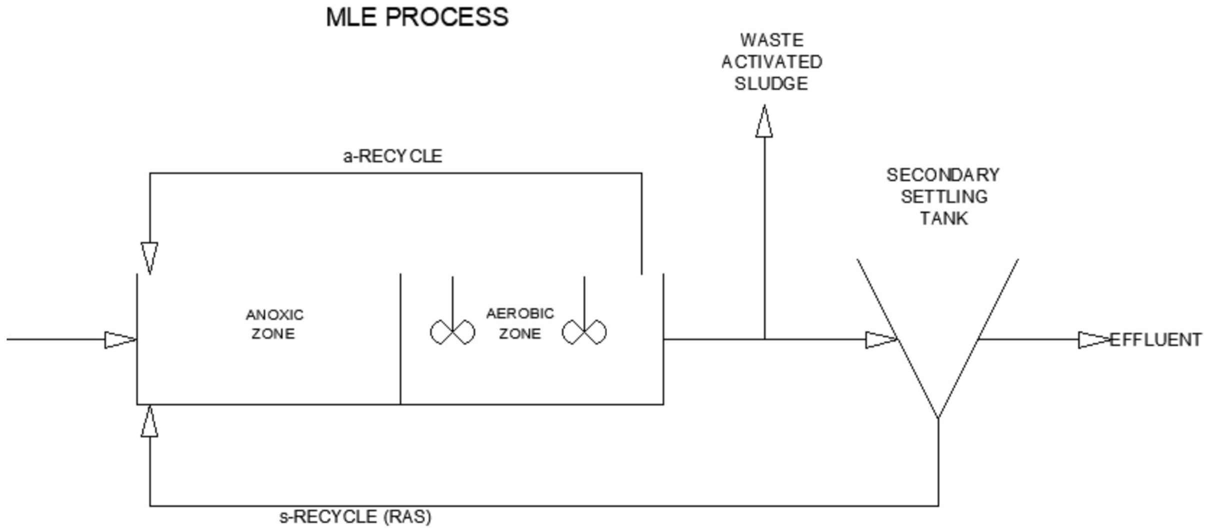


Figure 5-1: MLE process configuration

5.1.2 UCT Process

The UCT process is a nitrification-denitrification system as well as an enhanced biological phosphorus removal (EBPR) process. The SST underflow sludge recycle (RAS or s-recycle) is discharged into the anoxic zone. The r-recycle draws mixed liquor from the end of the anoxic zone and discharges it into the anaerobic zone. Mixed liquor is also recycled from the aerobic zone to the anoxic zone (a-recycle). The UCT process is effective for phosphate and nitrogen removal. However, due to the anaerobic zone operating at a reduced MLSS concentration and the increased MLSS concentration due to the presence of poly-phosphate accumulating organisms (PAO's), the UCT process, with the same reactor size as the MLE system, has a smaller treatment capacity. However, it achieves lower effluent phosphate concentrations than the MLE process. Figure 5-2 shows a process flow diagram of the UCT process.

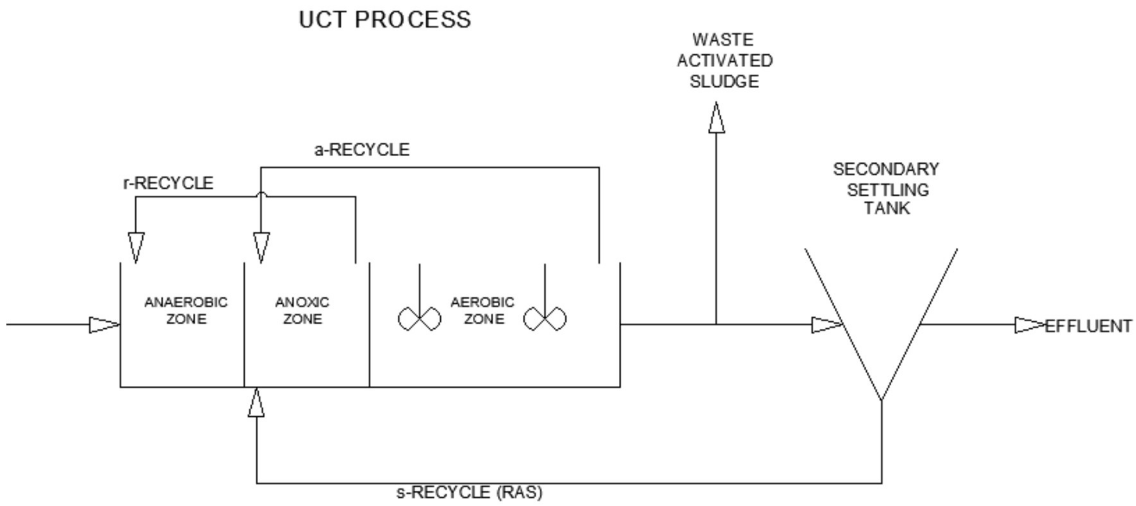


Figure 5-2: UCT process configuration

5.1.3 Process selected for design

The current water use licence does not have a strict effluent phosphorus requirement, however it needs to be considered that the effluent requirements for phosphorus may become stricter in future years. Even if it does not, it will be good for the receiving water body to limit effluent phosphate as far as possible since it is the limiting nutrient for eutrophication. As a result of this eventuality, all upgrades and phases leading up to the Master Plan design are designed with the option of operating it as a UCT or MLE process. Additional process configurations such as the modified UCT process and the Johannesburg process will also be included without the need for more equipment or infrastructure.

5.2 Ultimate Solution

The ultimate Master Plan for Gwaing WWTW is a plant that has the capacity to treat 50 MLD (ADWF) with an effluent quality that complies with the requirements as set out in the WUL. The current capacity of the plant is 8.6 MLD when operating as a UCT process and 10.4 MLD when operating as a MLE process, as summarised in Table 4-1. The ultimate solution was designed as a settled UCT system.

Given the boundary of the Gwaing WWTW and the site constraints as discussed in Section 4.7, the only way to achieve a capacity of 50 MLD (ADWF) with the UCT or MLE process is by including PSTs. An MBR Process would also be a space efficient option, but is not considered due to higher lifecycle costs and operational complexity. PSTs provide the immediate benefit that it removes between 30% and 40% of the influent COD loads and 50% to 60% of the TSS from the influent wastewater. The capacity of a given reactor and SST configuration can be increased by roughly 50% by adding PSTs to the process. The addition of PSTs effectively reduces the COD and TSS load on the reactor and thereby increases hydraulic treatment capacity.

However, when PSTs are introduced, primary sludge is generated. The only feasible means of dealing with primary sludge is anaerobic digestion, which is required to stabilise the sludge. The benefit of anaerobic digestion is that heat and power can be generated from the methane that is released in the process. Anaerobic digestion is discussed in more detail in Section 5.6.

5.3 Activated Sludge Design

5.3.1 Modular Design

The ultimate solution of 50 MLD was designed in such a way that it can be implemented in phases. The modular design was based on the existing reactor and SSTs. The existing reactor has a total volume 14,864 m^3. The two existing SSTs both have a diameter of 25 m. Various alternate solutions were investigated, such as including a single additional reactor with a larger volume of around 30,000 m^3. This would however require a large capital input and increase the capacity far beyond what would be required for a phased approach. Adding additional SSTs of a greater diameter of 30m or 35m was also investigated, however the constraint of the Eskom and George Municipal power lines restricted the number of SSTs of such a large diameter to a point where there would not be enough space to achieve the required capacity. Implementing equal-sized modules also simplifies flow splitting as an equal flow can be sent to each module for treatment. Due to the fall/gradient of the site coupled with the restrictions of the power lines, the available spaces for the reactors and SSTs are limited to the areas shown in Figure 5-3.

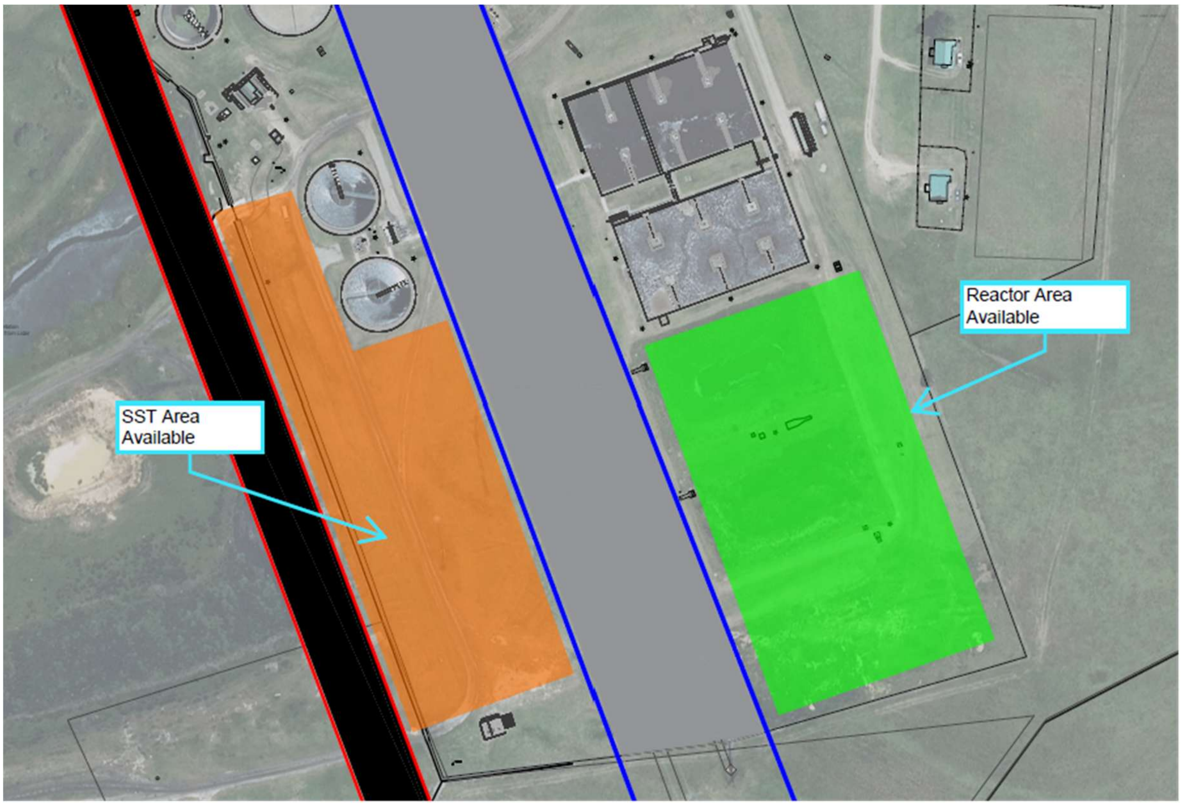


Figure 5-3: Reactor and SST construction area available

When utilizing the existing reactor volume, together with four SSTs of 25 m diameter each, with a settled UCT process, the hydraulic capacity of the module is 16,67 MLD (ADWF). This is exactly a third of the total capacity of 50MLD. The space available for the reactors and SSTs allows for three equal modules of a reactor with a volume of 14,864 m^3 and four SSTs with a diameter of 25m each. The layout of the three modules is shown in Figure 5-4.

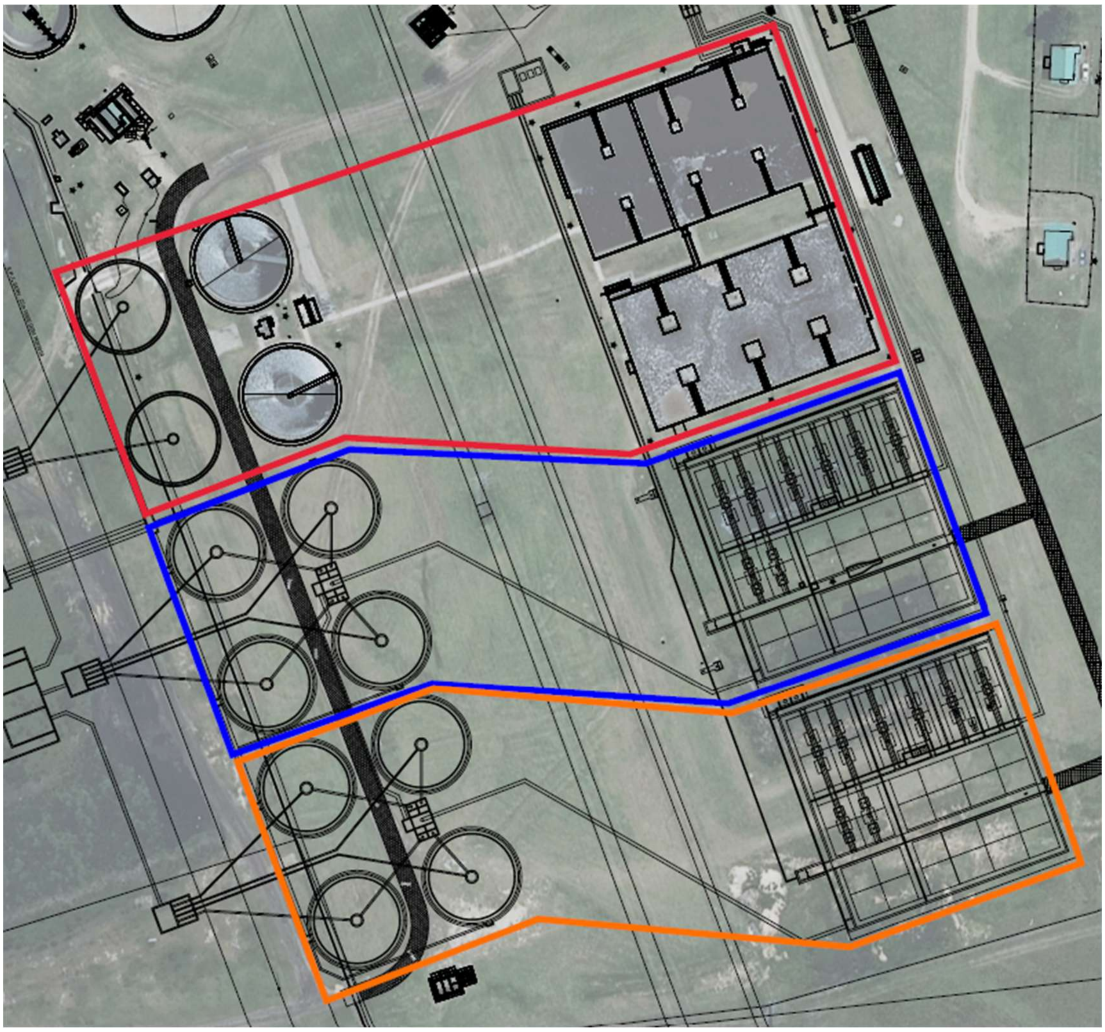


Figure 5-4: Reactor and SST module layouts for ultimate capacity

5.3.2 Biological Reactor Design

Two new reactors are required for the ultimate Gwaing WWTW solution. Each new reactor is designed in a single tank configuration with division walls to separate the different zones. The reactor is approximately 60 m × 60 m in plan and 5.5 m deep (4.2 m deep water level with 1.3 m freeboard). Figure 5-5 presents a layout of the reactor design.

The reactor is designed to have at least 37.5% (zones 1 to 5) of its volume as unaerated zones with mixer platforms. 12.5% of the volume (zone 6) is a swing zone which can be equipped with mixing and aeration equipment. Depending on the reactor process being utilized the swing zone can either function as an unaerated zone or aerated zone. The rest of the volume (50%, which includes zones 7,8,9 and 10) is aerated, fitted with fine bubble diffusers.

Screened and degritted sewage enters the reactor in the channel as shown. The r-recycle pump station is also placed in this channel resulting in a denitrified r-recycle stream joining the influent where it enters the reactor. The channel at the top and right-hand side of the image is dedicated to the Return activated sludge (RAS) stream from the SSTs and the a-recycle stream respectively. These channels join together for some of the processes that can be selected. The a-recycle pump station is also fitted in this channel. The reactor effluent flows over a weir from the last aerated zone to the

SSTs. Waste activated sludge (WAS) is taken from the overflow channel of the aerobic zones utilizing a flow meter and modulating valve to control the sludge age. Service ducts between the four aerated zones will be dedicated to the air headers which will be fitted with drop legs to the submerged FBDA network. Zones 7, 8, 9 and 10 can be individually isolated for maintenance of the FBDA networks.

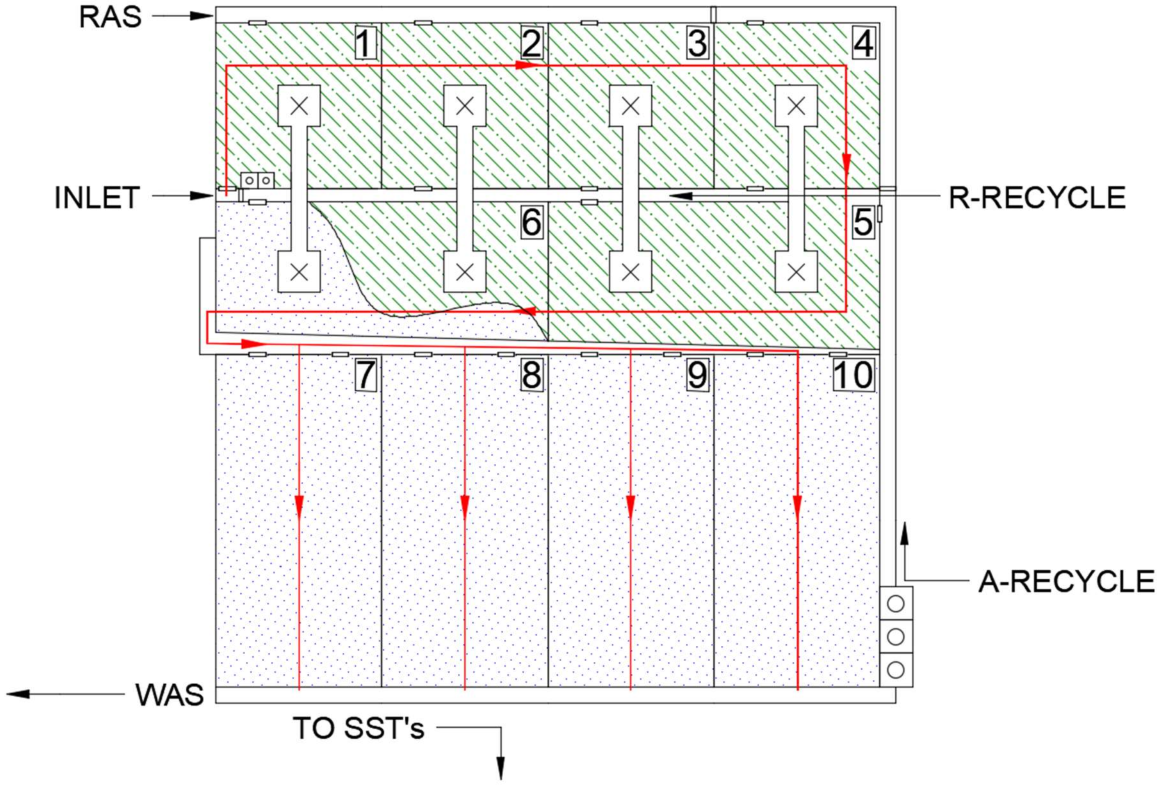


Figure 5-5: Reactor Layout Design

Actuated Penstocks from the channels to the respective zones will allow the reactor to operate in various configurations. With this reactor design, the modified Ludzack-Ettinger (MLE) process, the UCT process, the Modified UCT (MUCT) process and the Johannesburg (JHB) process can be utilized in various configurations. These configurations are visually shown in Figure 5-6.

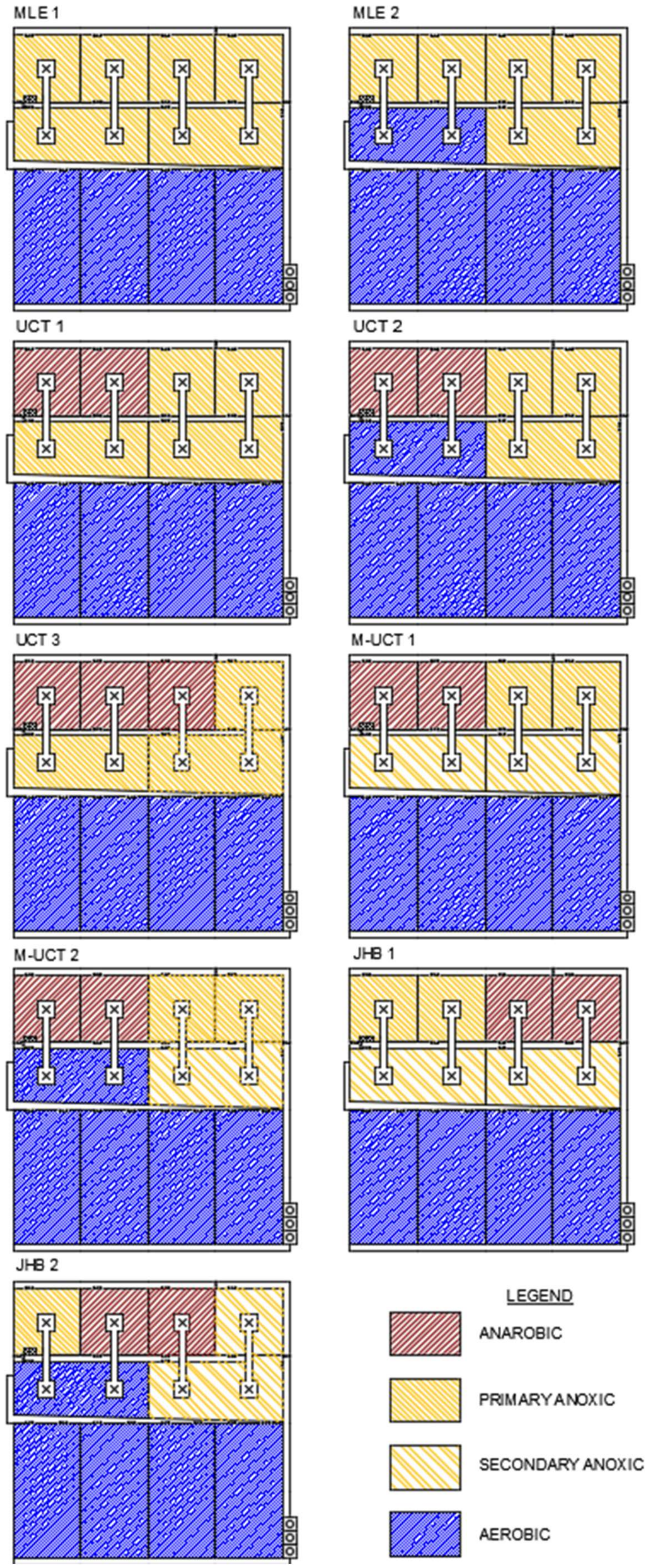


Figure 5-6: Reactor configurations

The reactor configurations best suited to treat settled wastewater according to the current sampling data are the MLE1 or UCT1 configurations. Orthophosphate concentrations in the MLE effluent will be just within the general limit whereas the UCT process produces much lower orthophosphate levels which is better for the environment. For settled wastewater, given how close the MLE settled process' orthophosphate concentrations are to the general limit, it is better suited to operate the UCT settled process. The mass fractions and expected effluent quality of the MLE and UCT processes associated with the reactor configurations in Figure 5-6 are presented in Table 5-1. For each reactor configuration the minimum required sludge age required for stable nitrification at 14 °C is also shown.

Table 5-1: Reactor Mass Fractions and corresponding effluent quality for Gwaing WWTW reactor configurations

	MLE 1	MLE 2	UCT 1	UCT 2	UCT 3
By Mass Fraction:					
Anaerobic	0.0%	0.0%	6.7%	6.7%	10.3%
Anoxic	50.0%	37.5%	40.0%	26.7%	34.5%
Aerobic	50.0%	62.5%	53.3%	66.7%	55.2%
	100%	100%	100%	100%	100%
Minimum Sludge Age for nitrification (SRT_m) at 14 °C [days]	*19*	*13*	*17*	*12*	*16*
RAW: Steady State Effluent Concentrations at 20-day sludge age and 14 °C					
Unbiod Sol COD (mgCOD/l)	46.9	46.9	46.9	46.9	46.9
FSA-N (mgFSA-N/l)	1.47	0.74	1.04	0.59	0.89
Nitrate ($mgNO_3$-N/l)	6.47	11.83	6.33	9.81	6.32
Ortho P as P (mgP/l)	8.96	8.96	0.00	0.00	0.00
SETTLED: Steady State Effluent Concentrations at 20-day sludge age and 14 °C					
Unbiod Sol COD (mgCOD/l)	46.9	46.9	46.9	46.9	46.9
FSA-N (mgFSA-N/l)	1.47	0.74	1.04	0.59	0.89
Nitrate ($mgNO_3$-N/l)	8.28	13.84	5.90	14.74	9.45
Ortho P as P (mgP/l)	9.56	9.56	0.00	0.00	0.00

5.3.2.1 Surface Aeration vs FBDA

Surface aeration and fine bubble diffused aeration (FBDA) were considered as aeration options for the aeration zones of the biological reactors. To compare these two options, their response to Gwaing WWTW's expected diurnal total oxygen demand (TOD) was modelled as shown in Figure 5-7. The following inputs were used in the model:

a. Raw COD concentration – 782 mgCOD/l
b. Ammonia – 58.7 mgN/l
c. Alpha factor surface aeration = 0.85
d. Alpha factor FBDA = 0.6

e. Theta (Temperature sensitivity) for FBDA = 1.024
f. Theta (Temperature sensitivity) for surface aeration = 1.012
g. Standard Oxygen Transfer Efficiency (SOTE) = 7%/m depth of reactor for FBDA
h. Surface aeration on/off control steps = 55 kW

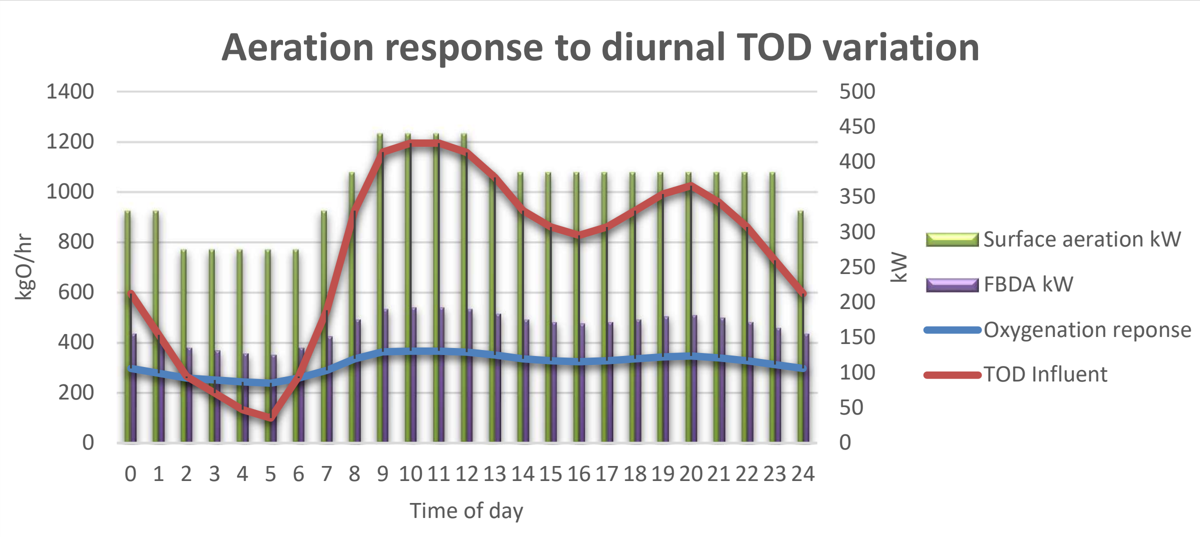


Figure 5-7: FBDA vs Surface Aeration Response to Diurnal TOD Variation

The model predicts an actual oxygenation aeration efficiency of 0.94 kgO/kWh for surface aeration and 1.9 kgO/kWh for FBDA which makes FBDA 2 times more efficient in transferring the same amount of oxygen. Thus, to maintain the same dissolved oxygen (DO) in the reactor, surface aeration will have double the energy costs compared to FBDA. Surface aeration cannot be effectively turned up or down to maintain a DO level. Instead, surface aerators are switched on or off at 55 kW increments in this case to keep DO concentrations within an acceptable range. An FBDA system on the other hand allows for continuous adjustment of airflow to individual aeration zones by modulating control valves and ramping of the blower speed. This optimisation will increase the efficiency of an FBDA system to 2.2 times better than surface aeration. Consequently, we recommend that FDBA be installed for the new reactors.

Table 5-2: kW requirements for surface aeration vs FBDA (based on UCT process with PST's for Reactor 2)

	FBDA	Surface Aeration
kW requirements/reactor at 50 MLD for Settled UCT process	193 kW blower / reactor	387 kW aerators / reactor

Table 5-3: Aeration Installed kW requirements for surface aeration vs FBDA for respective reactors and processes. (Note that the aeration requirements correspond with the flow rates (capacities) as indicated.

	Existing Reactor 1				Reactor 2				Reactor 3				**Total [kW]**			
	Raw		Settled		Raw		Settled		Raw		Settled		**Raw**		**Settled**	
	MLE	UCT	MLE	UCT	MLE	UCT	MLE	UCT	MLE	UCT	MLE	UCT	**MLE**	**UCT**	**MLE**	**UCT**
Vertical Shaft Aeration [kW installed]	448	327	548	387									**897**	**653**	**1120**	**784**
Fine Bubble Diffused Aeration (FBDA) [kW installed]					224	163	286	198	224	163	286	198				
Capacity in MLD	14	11	22.5	16.7	14	11	22.5	16.7	14	11	22.5	16.7	**42**	**33**	**68**	**50**

The existing reactor utilises surface aeration. It had four 55 kW and two 45 kW surface aerators giving a total installed aeration capacity of 310 kW. To provide more surface aeration capacity for the master plan upgrade, these aerators have recently been upgraded to four 75 kW and two 55 kW aerators which give a total aeration capacity of 410 kW. Surface aeration will be maintained in the existing reactor for two reasons:

- The reactor was constructed with sloped sides which are not ideal for FBDA.
- Surface aeration is generally less complex and more reliable than FBDA blower and diffuser systems. Hence the resilience of the overall plant is somewhat increased by maintaining surface aeration in the existing reactor, even if the energy costs are higher.

For FBDA centrifugal blowers with inlet guide vane (IGV) and Variable Vane Diffusers (VVD) are recommended. For the most optimum energy usage, they should be used with DO control in combination with the most open valve (MOV) algorithm to control air flow and maintain the lowest pressure towards the reactors.

The aeration equipment for the blower house needs to be sized to maintain a Dissolved Oxygen concentration of 2 mg/l in the bulk liquid of the aerobic zone, with the stepped aeration aiming at 1 mg/l for the last portion of the aerobic zone to minimise DO recycle to the anoxic zone via the a-recycle.

Fine bubble diffusers with a specific oxygen transfer efficiency (SOTE) of at least 7% per metre submergence will be specified. The aerobic/anoxic swing zone (zone 6) in the biological reactor will be equipped with combined mixing and aeration equipment. Ideally, this will not include FBDA but a maintenance free aeration method since this zone is not designed to be isolated for maintenance.

5.3.3 Secondary Settling Tank (SST) Design

The SSTs are designed with a side wall depth of 4.0 m and sloped floors to the centre sludge collection well. The overflow collection launders are internal launders and create a Stamford baffle effect, deflecting sludge density currents away from the overflow weir. The deep (4 m) side walls and Stamford baffle result in a flux rating of 0.8. A diluted sludge volume index (DSVI) of 135 ml/g was used for the design. The return activated sludge (RAS) recycle (or s-Recycle) ratio selected is 2 times the ADWF. Figure 5-8 shows the design and operation chart of a single SST for various reactor concentrations ranging from 4.1 kgTSS/m^3 (4100 mgTSS/l) to 4.85 kgTSS/m^3 (4850 mgTSS/l). With the above-mentioned inputs, the models show SST failure at reactor concentrations greater than 4.5 kgTSS/m^3.

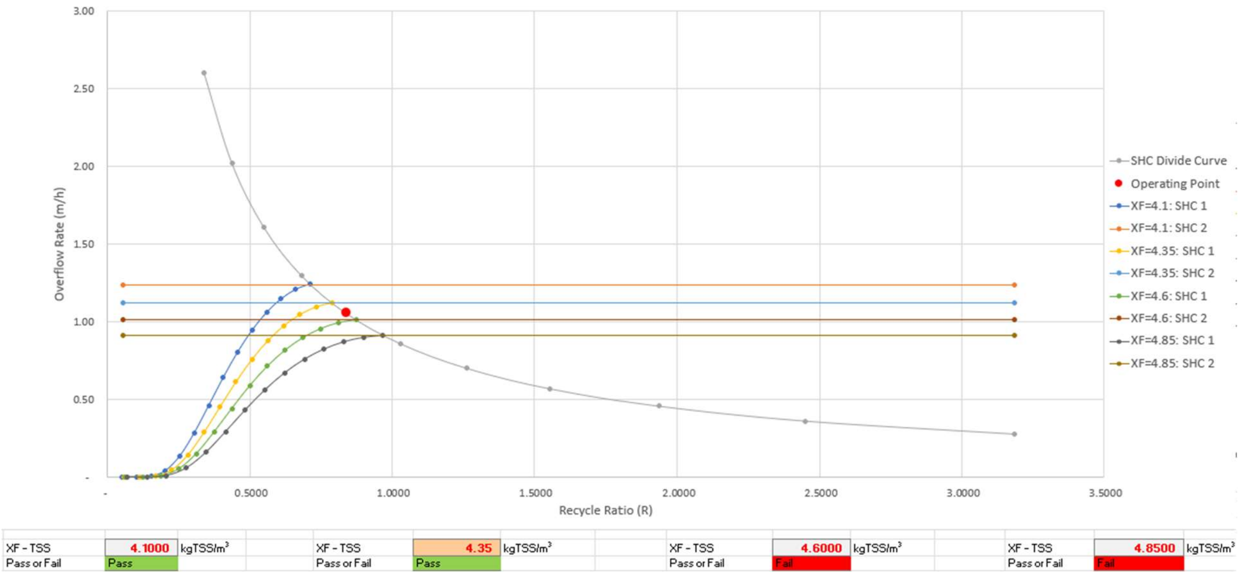


Figure 5-8: Design and Operation chart of SST based on a single reactor with 4x25m Dia. SSTs (UCT settled process)

The D&O chart is based on the UCT process with settled wastewater (PSTs included) for an ADWF of 16.7 MLD per reactor train (per four SSTs). A PWWF of 39.6 MLD per reactor train was used, based on a Harmon peaking factor of 1.83 and a stormwater infiltration of 30%, giving a total peak factor of 2.4. Design values for the SSTs are summarised in Table 5-4.

Table 5-4: SST design values per module

Parameter	Module A	Module B	Module C
DSVI (ml/g)	135	135	135
SST Diameter (m)	25	25	25
No. of SST's	4	4	4
Recycle ratio w.r.t ADWF	2	2	2
SST Area (m^2)	1963	1963	1963
Flux Rating	0.8	0.8	0.8
SST effective Area (m^2)	1571	1571	1571

5.3.4 Summary of Activated Sludge Design

5.3.4.1 Module Capacities for Settled and Raw WW

The capacities for each module for the given reactor volumes and number of SSTs are summarised in Table 5-4. The capacity for each module is shown for the MLE and UCT processes for both settled and raw wastewater. It is evident when comparing the ultimate capacity for a given reactor volume and number of SSTs that the UCT process has a lower treatment capacity than the MLE process in a given volume. The reasons for selecting the UCT process for the ultimate capacity design are discussed in Section 5.

Table 5-5: Module capacities for various processes

	Existing Reactor A				Reactor B				Reactor C				**Total [kW]**			
	Raw		Settled		Raw		Settled		Raw		Settled		**Raw**		**Settled**	
	MLE	UCT	MLE	UCT	MLE	UCT	MLE	UCT	MLE	UCT	MLE	UCT	**MLE**	**UCT**	**MLE**	**UCT**
ADWF (MLD)	14.0	11.0	22.5	16.7	14.0	11.0	22.5	16.7	14.0	11.0	22.5	16.7	**42.0**	**33.0**	**67.5**	**50.0**
Flow to PST (MLD)	-	-	0.2	0.2	-	-	0.2	0.2	-	-	0.2	0.2	-	-	**0.7**	**0.5**
ADWF to Reactor (MLD)	14.0	11.0	22.3	16.5	14.0	11.0	22.3	16.5	14.0	11.0	22.3	16.5	**42.0**	**33.0**	**66.8**	**49.5**
Sludge Age (days)	20	20	20	20	20	20	20	20	20	20	20	20	-	-	-	-
SRT_{min} with SF	14.3	17.0	19.0	17.0	13.5	16.3	13.5	16.3	13.5	16.3	13.5	16.3	-	-	-	-
Reactor TSS Concentration (mgTSS/l)	4700	4900	3700	4350	4700	4900	3700	4350	4700	4900	3700	4350	-	-	-	-
Reactor Volume (m^3)	14864	14864	14864	14864	14864	14864	14864	14864	14864	14864	14864	14864	**44592**	**44592**	**44592**	**44592**
Harmon Peaking Factor	1.9	2.0	1.7	1.8	1.9	2.0	1.7	1.8	1.9	2.0	1.7	1.8	**1.6**	**1.6**	**1.5**	**1.5**
PDWF (MLD)	26	22	39	30	26	22	39	30	26	22	39	30	**66**	**54**	**98**	**77**
Stormwater infiltration	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	30%	**30%**	**30%**	**30%**	**50%**
PWWF (MLD)	34	28	50	39	34	28	50	39	34	28	50	39	**86**	**70**	**128**	**115**
PWWF/ADWF peak factor	2.4	2.6	2.3	2.4	2.4	2.6	2.3	2.4	2.4	2.6	2.3	2.4	**2.0**	**2.1**	**1.9**	**2.3**

5.3.4.2 Recycle Ratios

The recycle ratios and mass fractions for the MLE and UCT processes for both settled and raw wastewater are summarised in Table 5-6. The recycle ratios are per reactor module for the ultimate solution.

Table 5-6: Recycle Ratios in relation to ADWF for various processes

Parameter	Values Per Reactor Module			
	MLE Raw	MLE Settled	UCT Raw	UCT Settled
r-recycle ratio	N/A	N/A	1.00	1.00
s-recycle ratio (RAS)	2.00	2.00	2.00	2.00
a-recycle DO concentration	2.00	2.00	2.00	2.00
Maximum a-recycle ratio (operationally variable)	6.00	6.00	6.00	6.00

5.4 Primary Settling Tank (PST) Design

The primary settling tanks are designed to have a maximum overflow rate of 1.2 m/h during average dry weather conditions and a maximum overflow rate of 2.4 m/h during peak wet weather flow conditions. The PST underflow used for the design is 1% of the ADWF. For the ultimate capacity of 50 MLD, the underflow rate of the PSTs will be 0.5 MLD. The ultimate capacity is designed with four PSTs of 25 m diameter each to have sufficient capacity during PWWF conditions. A summary of the design values is indicated in Table 5-7.

Table 5-7: PST Design parameters

Total Capacity - 50MLD

PST Underflow Ratio		1.00%			
		Raw	Settled	Underflow	
Average Dry Weather Flow	Q_{ADWF}	50.0	49.5	0.50	ML/d
Peak Dry Weather Flow	Q_{PDWF}	76.6	76.1	0.50	ML/d
Peak Wet Weather Flow	Q_{PWWF}	99.5	99.0	0.50	ML/d

Selected PST Sizing

New PSTs

PST Diameter	Ø	25	m
Number of PST's	No.	4	
	Area	1 963	m^2

	Variable	Unit	ADWF	PWWF
Overflow Rate	Vo	m/h	1.2	2.4
Incoming Flow	Q_i	m^3/h	2 083	4 147
Overflow	Q_e	m^3/h	2 063	4 126
Underflow	Q_{ps}	m^3/h	21	21
Min Required Area	A_{Req}	m^2	**1 719**	**1 719**
			Pass	Pass

5.5 Primary Sludge Thickening

Primary sludge will be thickened using gravity thickeners. Thickening of the primary sludge is done to reduce its volume and thereby reducing the volumes of anaerobic digestors needed for a given hydraulic retention time.

If sludge is withdrawn from the PSTs at a rate of 1% of ADWF, then the underflow concentration will be approximately 1.7%, although this can vary considerably. A thickened concentration of minimum 4% will be targeted. The design loading rate for the thickeners is 80 kgTSS/m^2/d. For the ultimate capacity of 50 MLD ADWF, the primary sludge flux will be 8650 kgTSS/d. A thickener with an area of 8650/80 = 108 m^2 is required. It is good practice to provide two digestors so that batch operation can be done whereby one digestor is filled at a time, while the sludge in the other digestor is left to settle and thicken.

The existing two PSTs upstream of the bio-trickling filters could potentially be repurposed as gravity thickeners in the future. Each tank has a diameter of 15m which equates to an area of 177 m^2. Its capacity is therefore sufficient for thickening of the primary sludge of a 50 MLD plant.

The suitability of the PST scraper bridges will have to be reviewed since the sludge in a gravity thickener is more viscous than primary sludge and therefore the bridge structure needs to be stronger. The rotation of the bridge assists the thickening process by disrupting inter-particle bridging.

The existing primary sludge pump station can also be repurposed to pump the thickened sludge to the anaerobic digestors. The integrity of the pipeline from the PSTs to the pump station needs to be reviewed.

The overflow (supernatant) from the thickener can be returned to then inlet works, PSTs or biological reactor.

5.6 Anaerobic Digestion (AD)

The benefits of anaerobic digestion include the following:

- Energy conservation: A reduction in the organic load entering the biological reactor that consequently decreases the amount of air needed for nitrification and oxidation.
- Energy production: The potential to generate electricity with a combined heat and power generator can increase energy independence.
- Waste management: Anaerobic digestion reduce the strength and volume of the solid waste stream while still preserving its fertilizer value and removing odorous compounds and pathogens in sludge.

Unfortunately, anaerobic digestion has a poor track record in South Africa. There are very few properly operating anaerobic digestors in the country. While the theory of anaerobic digestion is simple enough it is often small operational challenges that lead to the digestors eventually being mothballed. Digestor overloading causes a pH drop which causes the digestor to turn sour and not achieve the required sludge quality. Digestor heating systems often fail. When mixing systems fail and are not repaired timeously the sludge in the digestors solidifies and when the digestor sludge needs to be removed there are safety concerns due to trapped methane. All these aspects can be overcome with good design, quality equipment and operational discipline. However, this process introduces more risks to the works compared to a simple CAS system consisting of a reactor and SSTs alone. Nonetheless, anaerobic digestion forms part of the future phases of Gwaing WWTW's Master Plan to upgrade the works to 50MLD. The increased scale in future phases will make the energy benefits unlocked by AD more attainable.

Table 5-8 shows the sludge treatment capacity of the four proposed anaerobic digestors.

Table 5-8: Anaerobic Digestors Sludge Treatment Capacity.

	VSS	TSS	ISS
Sludge Mass In (kg/d)	6457	8650	2193
Sludge Mass Out (kg/d)	3163	5357	2193
Sludge Mass reduction (kg/d)	3293	3293	0

A summary of the energy generation potential and heat generation potential in the proposed ADs is presented in Figure 5-9. It is estimated that 394 kW of energy and 562 kW of heat (including 169 kW of losses) can be generated at Gwaing WWTW's ultimate solution (50 MLD).

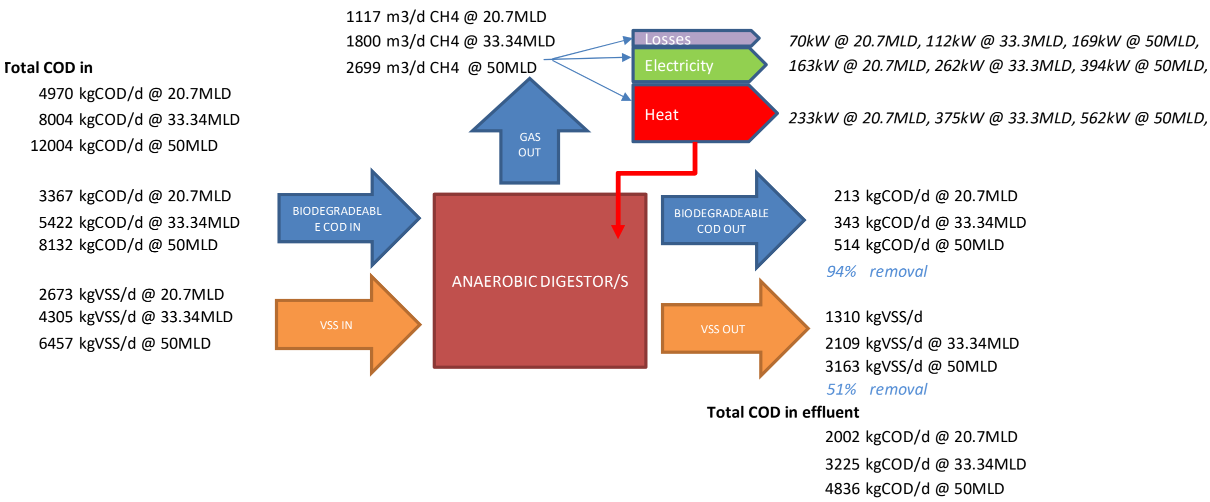


Figure 5-9: Gwaing WWTW Anaerobic Digestion Treatment Summary

Whilst anaerobic digestion is part of Gwaing WWTW's ultimate solution, the timing of its implementation was considered. Adding PSTs will reduce the COD load on the biological reactors and consequently the aeration requirements in it. But at the same time, many additional loads are added with the addition of PSTs, primary sludge pumping, AD mixing, gas treatment, and sludge dewatering. A simulation was done to determine when the benefit of reduced aeration requirements (reduced COD load to the reactors) and the energy generated (in kWh) by the CHP generators would materialize given the additional loads added by the equipment needed for anaerobic digestion. This simulation considered the operational hours of equipment and is graphically shown in Figure 5-10 (note with surface aeration the model will look different). As can be observed from the figure, as the size of the WWTW grows beyond 25 MLD it would make sense to include PSTs and anaerobic digestion to increase the plant's capacity and to reduce its energy consumption. Therefore, for the first upgrade, it would make more sense for the municipality to install a simpler and more robust process with the addition of another reactor and SSTs. Below 25MLD the benefits of energy conservation in the reactors and energy generation are less than the additional electrical loads added by all the anaerobic equipment. Consequently the implementation of anaerobic digestion (and by implication PSTs) is deferred to future upgrades only after the capacity of the reactors and SSTs have been increased.

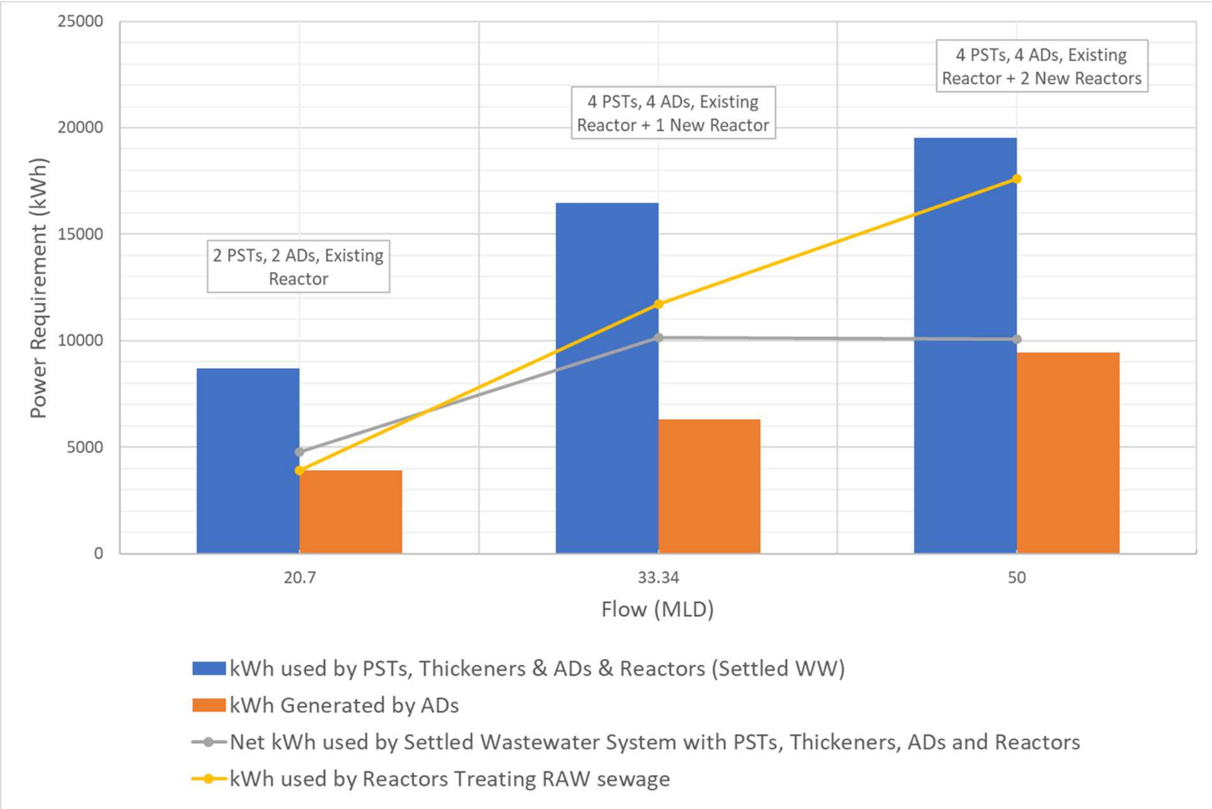


Figure 5-10: Energy Requirements - Treating Raw vs Settled Wastewater

It is not recommended that waste activated sludge (WAS) be anaerobically digested. WAS has very little active biomass left after treatment in the biological reactors and hence adds very little extra energy generation capacity to the ADs. Phosphate is captured within the biomass in the phosphate-accumulating organisms (PAOs). When the biomass is introduced to anaerobic conditions the PAOs release the phosphate they have accumulated. This release can cause issues such as struvite precipitation in the ADs. Additionally, nutrients (including nitrates and nitrites) pass through the ADs and will add an extra nutrient load for the WWTW to treat.

Figure 5-11 and Figure 5-12 show the results from a study UCT did on the topic and clearly show how little energy anaerobic digestion of WAS adds in comparison with primary sludge. The marginal benefit is lost by the fact that effluent nitrate and phosphate concentrations are increased.

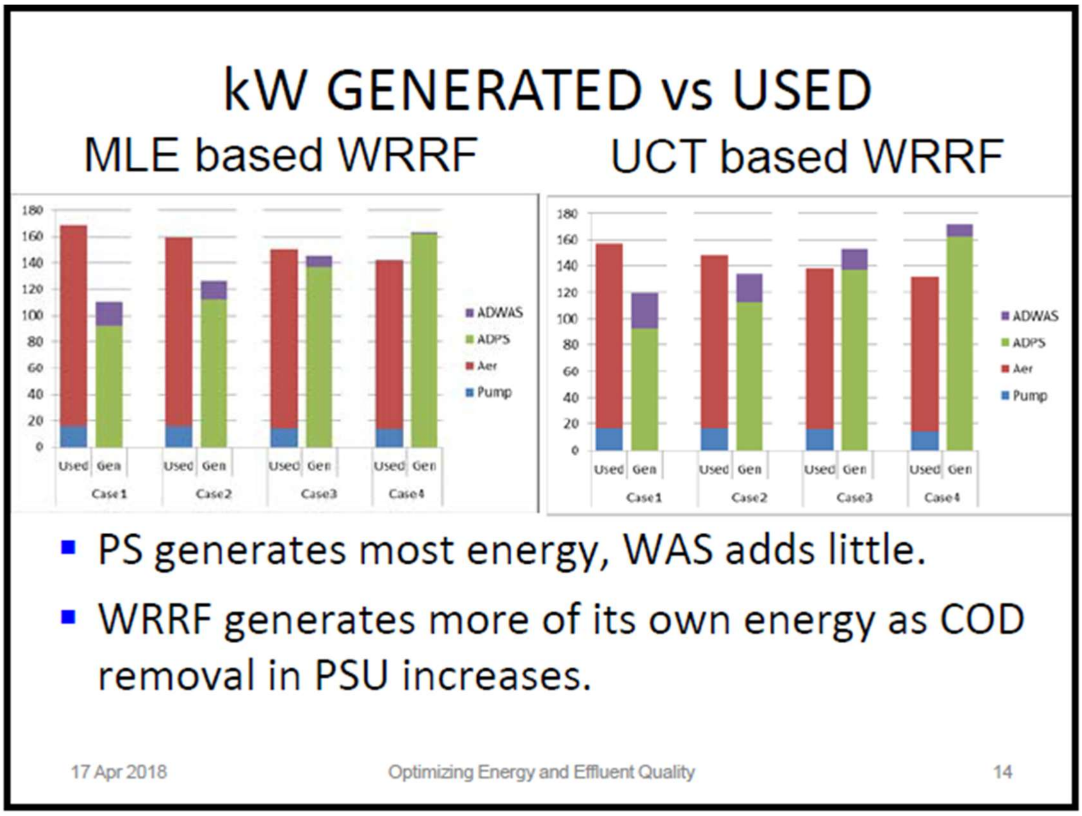


Figure 5-11: Comparison between the energy used in aeration and pumping with that generated by anaerobic digestion of WAS and Primary Sludge in UCT and MLE systems. Cases 1, 2, 3 and 4 represent increasing COD removal efficiency at the PST's. (Ekama et al. 2018)

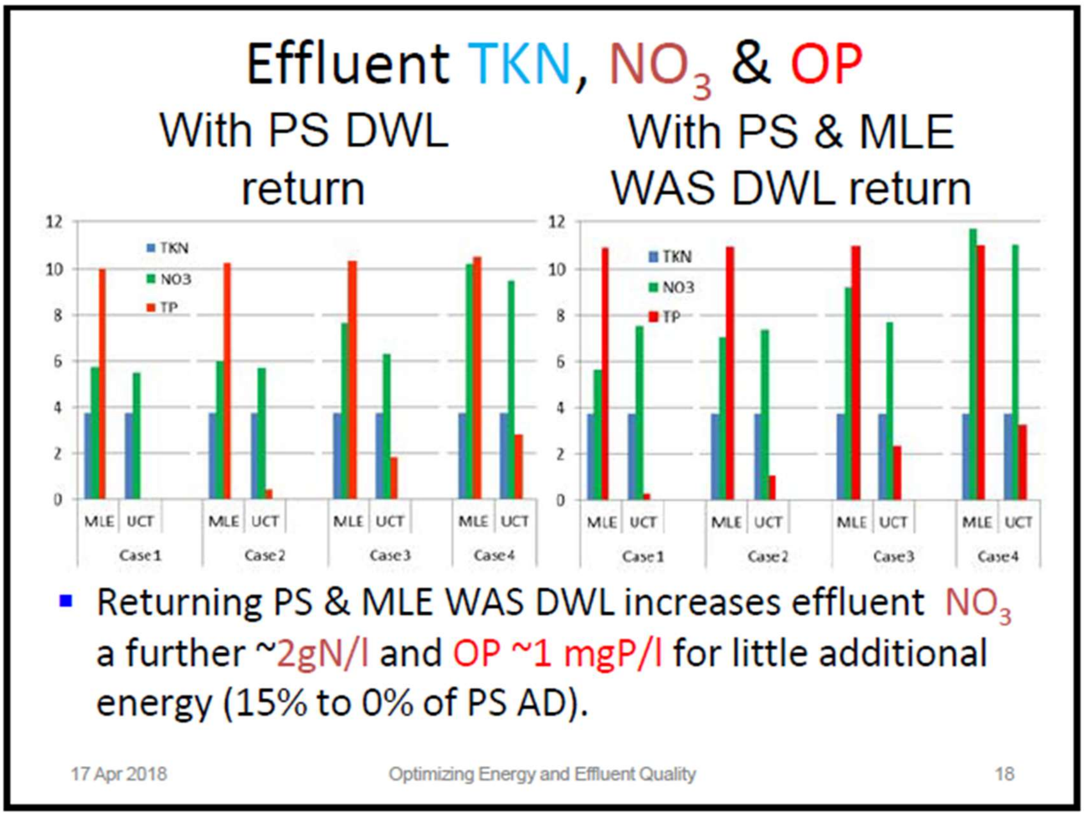


Figure 5-12: Comparison of Effluent TKN, NO3 and OP generated by anaerobic digestion of WAS and Primary Sludge in UCT and MLE systems. Cases 1, 2, 3 and 4 represent increasing COD removal efficiency at the PST's. (Ekama et al. 2018)

5.7 Disinfection

All clarified effluent from the secondary clarifiers will gravitate to a disinfection facility. Disinfection will be done upstream of the maturation ponds with chlorine in new chlorine contact tanks or with UV reactors in new UV channels. The existing chlorine contact tank will be retained as an optional final disinfection step after the maturation ponds. It is recommended that the maturation ponds be retained as it provides additional disinfection and naturalisation of the effluent, also reducing the TSS concentration and residual chlorine levels before it is discharged to the river. Means to bypass the ponds will be provided. If UV disinfection is done, this should be done upstream of the maturation ponds to prevent algae from fouling the UV light sleeves.

5.7.1 Chlorine Option

The design of the chlorine contact channel shall allow for a 2-phased upgrade of the new disinfection facility to suit the ultimate capacity of the Gwaing WWTW.

This upgraded chlorine contact facility shall receive overflow from the secondary clarifiers. The chlorine contact tank will be designed as two parallel systems so that each tank can be isolated for maintenance. The volume of the chlorine contact tank shall be such that the hydraulic retention time is at least 20 mins during PWWF. The dosing equipment will be sized to dose chlorine at any rate from 5 mg/l to 20 mg/l. Residual chlorine in the effluent should be below 0.25 mg/l as free chlorine.

The existing chlorine contact channel infrastructure will remain unchanged, except for the existing bypass causing the problematic erosion at the donga, which flow will be diverted to the existing chlorine contact channel.

A turbulent zone will be incorporated either at the outlet of the channels or further downstream to cater for re-aeration of the treated effluent and help dissipate residual chlorine in the treated effluent. It is not anticipated that formal chemical dosing for chlorine residual destruction will be required and therefore no dosing facility for de-chlorination has been allowed for.

Sludge accumulating at the bottom of the channels can be withdrawn from a small sump area into which a small, removable submersible pump can be placed as needed.

5.7.2 UV Disinfection Option

UV disinfection is a viable alternative to chlorine disinfection. UV disinfection can be grouped into two main categories for wastewater disinfection:

- Low-pressure high output (LPHO) in open channels or closed vessels
- Medium-pressure lamps in closed vessels

In general, LPHO lamps have a longer guaranteed lamp life, but more LPHO lamps are needed compared to similar MP lamp systems. MP lamps are shorter which is better for maintenance, but has a higher fouling risk. MP lamps operate at extremely high temperatures (600-900°C) resulting in the risk of algae formation and the precipitation compounds with lower solubility at higher temperatures like $CaCO_3$, $CaSO_4$, $MgSO_4$, and $FePO_4$. The majority of UV wastewater installations are done in open channels, which allows operators to maintain equipment without having to isolate channels or bypass flows. For Gwaing WWTW, an LPHO open-channel UV system is recommended.

For the design of a UV system, key decisions must be made such as the minimum design UV transmittance (UVT), level of confidence of effluent concentration (7day average, 30day average, or 1-day max), the log reduction to be achieved, maximum flow to be disinfected and whether the system should be bioassay validated by an accredited third party. A summary of the recommended UV design parameters is shown in Table 5-9.

Table 5-9: UV Design Summary for Ultimate Solution

Design Summary	
Maximum disinfection flow	115 MLD
Average disinfection flow	50 MLD
Minimum disinfection flow	11 MLD
Minimum Ultraviolet Transmittance (UVT) @ 253.7 nm	40%
Treatment objective at all conditions specified in this table	4 log reduction of Escherichia coli (E.coli)
UV System Influent E.coli count	1000000 cfu/100ml
Confidence level in E. coli count in UV outflow	1 day max (95%ile)
Effluent standards to be achieved	150cfu E.coli per 100 ml at 95%ile confidence level at all conditions specified in this table.
Third-party Bioassay Validated	Yes

The UV system will be designed such that the UV banks (with 25% redundancy built-in) are located on two open channels covered with a roof. Electrical equipment will be housed in a closed building next to the channels. The channels will be sufficiently sized such that the UV system for future phases can be easily slotted in. An example of an open channel UV installation with an automatic lamp lifting mechanism for ease of maintenance is shown in Figure 5-13.

Figure 5-13: Example Image of open channel UV installation with automatic lamp lifting system.

5.7.3 UV vs Chlorine Disinfection

Both UV disinfection and chlorine disinfection are proven methods for effectively disinfecting treated wastewater effluent. However, they differ in terms of effectiveness, cost, environmental impact, and operational considerations. A high-level comparison of these differences is shown in Table 5-10.

Table 5-10: UV vs Chlorine Comparison

	UV	Chlorine
Particle Shielding	Susceptible to particle shielding	Susceptible to particle shielding
Disinfectant Byproducts formation	No	Do form disinfection byproducts such as trihalomethanes (THMs) and haloacetic acids (HAAs) when chlorine reacts with organic matter in water.
Electricity Usage	High	Low
Chemical usage	Low	High
Capital Cost	High	Low
Operational Costs	High	High
Health and Safety Considerations	Risks associated with occupational exposure to UV-C radiation and high-voltage electricity	Storage and handling of hazardous chemicals: Qualify as a major hazardous installation under the Occupational Health and Safety Act (Act 85 of 1993)
Environmental Effects	No chemical residual	Chlorine residual if not dissipated can have a negative on aquatic life

To decide between UV and Chlorine disinfection for Gwaing WWTW a life cycle cost analysis (LCCA) was done. The following inputs were used in the LCCA:

- Chlorine dosing of 10mgCl_2/l @ ADWF to reduce E.Coli from 650 000 cfu to 150 cfu
- UV design to reduce E.Coli from 650 000 cfu to 150 cfu with 95% confidence at 40 % UVT
- Electricity cost – R2.04/kWh @ ADWF
- Electricity inflation - 10%
- Chlorine Costs – R32.6/kgCl_2 (based on quotes received from ChlorCape 2024)
- Chlorine inflation - 10%
- Total maintenance cost (Civil and M&E) – 1.5% of capital cost per annum
- Discount rate – 6%
- Civil inflation – 6%
- M&E equipment inflation – 8%

UV designs and costs were obtained from two UV suppliers and a chlorine supplier. A summary of the capital and associated operation and maintenance costs from these suppliers is shown in Table 5-11.

Table 5-11: Chlorine and UV Capital and Annual O&M Costs (2024 costs)

	Phase A&B	Phase C	Phase D	Total
Capital Cost (2024)				
M&E Cost UV	R39 732 000.00	R18 356 184.00	R41 122 620.00	**R99 210 804.00**
UV Civil Cost	R6 018 891.06	R0.00	R0.00	**R6 018 891.06**
Chlorine M&E Cost	R17 731 871.25	R8 865 935.62	R3 657 558.02	**R30 255 364.89**
Chlorine Civil Cost	R12 223 566.40	R4 807 566.40	R0.00	**R17 031 132.80**
Annual O&M Costs (2024 Value)				
UV Annual Operation Cost	R2 001 484.80	R2 299 920.48	R3 731 339.52	
UV Annual Maintenance Cost	R2 663 522.70	R520 666.76	R1 041 333.51	
Chlorine Annual Operation Cost	R2 617 780.00	R3 926 670.00	R5 949 500.00	
Chlorine Annual Maintenance Cost	R476 873.09	R702 267.47	R775 418.63	

The net present value of the LCCA discounted at 6% over 45 years between 2025 and 2070 is shown in Figure 5-14. Over this period, as derived from Figure 5-14, there is a net present value difference of **R23 405 134.67** in the favour of UV disinfection.

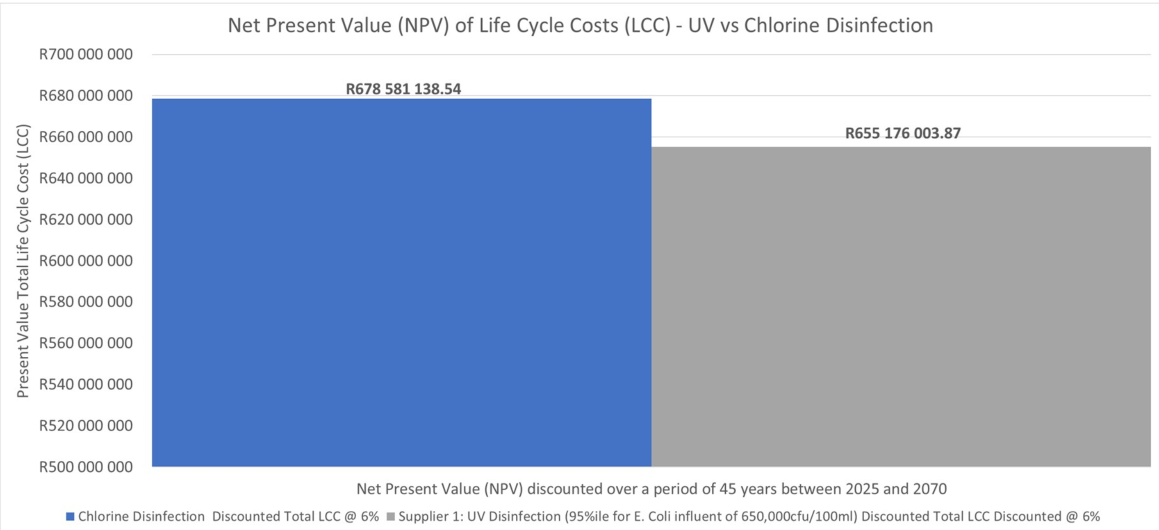


Figure 5-14: Discounted Life Cycle Cost Comparison between UV and Chlorine

The life cycle cost analysis is depicted in Figure 5-15. The projection shows that UV becomes more cost effective than Chlorine disinfection in 2068. When interpreting the comparison, it must be kept in mind that the electricity increase and the chlorine gas cost increase were both estimated at 10% for an equal comparison. If either one of these deviates from the estimated increase percentages, it would have a significant effect on the life cycle analysis result.

For the overall life cycle costs for the entire Gwaing WWTW upgrade, the difference between UV and chlorine treatment is minimal. Thus, from a long-term economic perspective, UV and chlorine are relatively similar. However, from an environmental and regulatory standpoint, UV offers more advantages (see Table 5-10). For instance, UV treatment leaves no chemical residuals, which is better for the environment, and a UV installation doesn't have to comply with the legislative demands of a major hazardous installation.

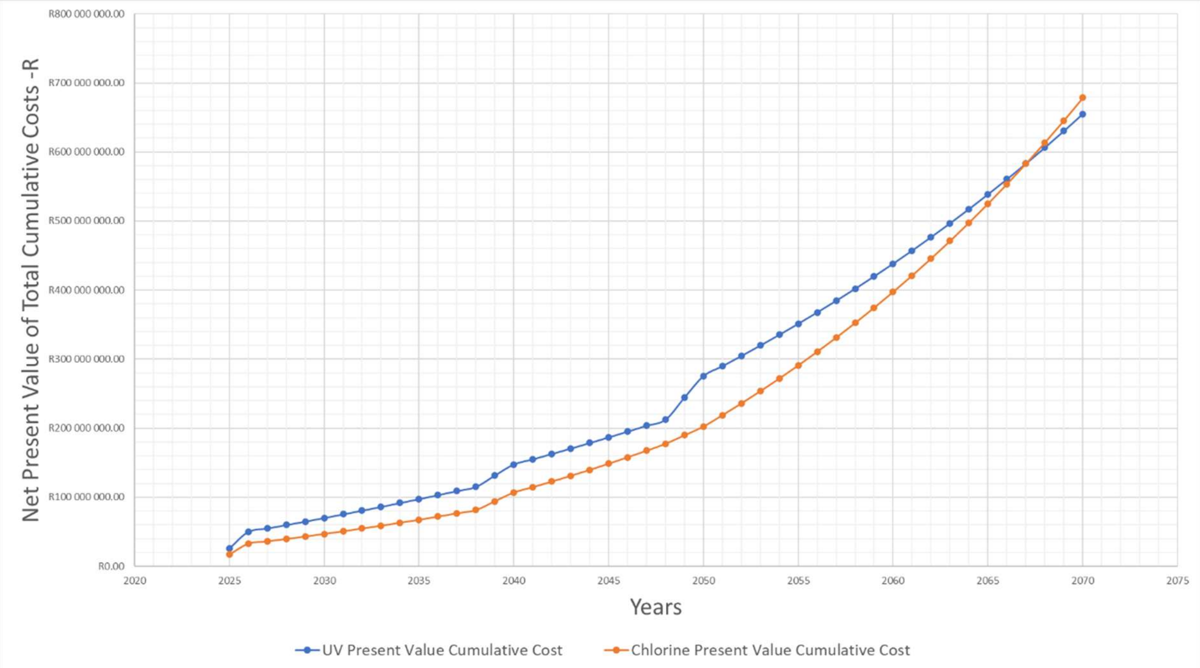


Figure 5-15: Net Present Cumulative Cost Comparison UV vs Chlorine

Given UV's long-term economic benefits, UV disinfection's environmental advantages and lower health and safety risk, UV disinfection was adopted as the disinfection method for the Master Plan. It is recommended that George Municipality opt for UV disinfection as the disinfection alternative.

A final decision needs to be made regarding the confidence level of E. coli count in UV outflow. The LCCA was based on a 1-day maximum concentration (95% confidence of compliance). If the level of compliance is based on a 7-day average geometric mean, UV becomes significantly more cost-effective compared to chlorine. The values used for the comparison is based on the Master Plan ultimate solution. The entire life cycle and future upgrades need to be taken into consideration when deciding on the disinfection alternative.

5.8 Alternative Process Options Considered

5.8.1 External Nitrification Process

The external nitrification process is effectively a combination of conventional activated sludge (CAS) and trickling filters. COD removal, denitrification, and P-removal happen in the CAS reactor while nitrification is outsourced to the bio-trickling filter. This allows the CAS reactor to be operated at a much lower sludge age of around 5 to 8 days.

Figure 5-16 shows a typical external nitrification process flow diagram. It may have been a feasible upgrade option for the Gwaing WWTW if the trickling filters had been in a better structural condition, but due to its current state, it is not the recommended option for the upgrade.

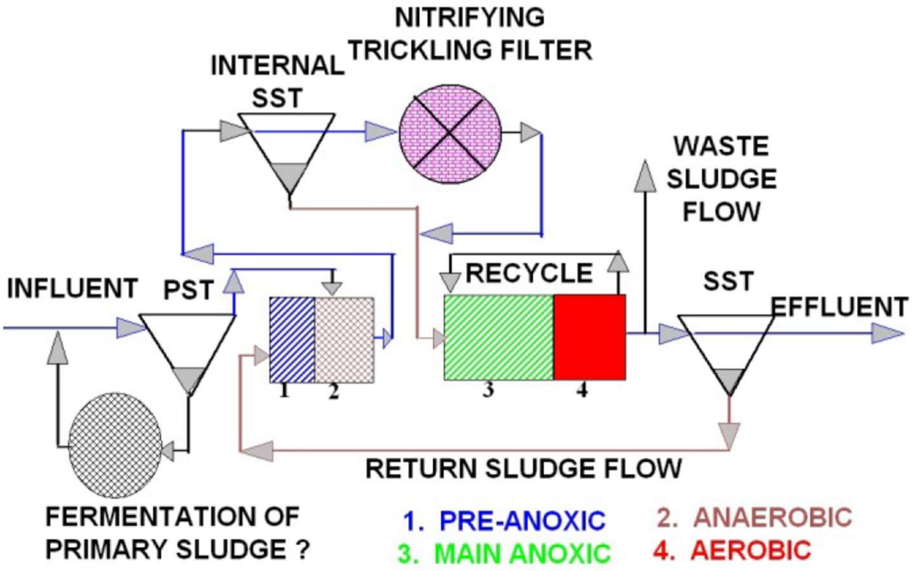


Figure 5-16: Typical Process Flow Diagram of an external nitrification process. (Ekama, 2014)

5.8.2 Conversion of Bio-trickling filters to CAS plant

The option of converting the existing bio-trickling filters to a CAS reactor was considered by converting one trickling filter into anaerobic and anoxic zones and the other trickling filter into an aerobic zone. The humus tanks could then be converted to SSTs. This would create a relatively small capacity upgrade. The main problem with this approach is the structural condition of the bio-trickling filters and the fact that they are not designed as water-retaining structures. Figure 5-17 shows how the trickling filter walls rest freely on the floor as opposed to being integrated with the floor. Significant and costly changes to these tanks would be required to render them water-retaining and suitable to host a CAS process. For this reason, this option is not favoured.

Figure 5-17: Bio-trickling filter walls can be seen as resting on the floor slab and not integrated into the floor slab.

6 FUTURE REQUIREMENTS

6.1 Future required facilities

The future facilities required for the ultimate solution of 50 MLD are discussed in this section. The phasing options for intermediate upgrades will be discussed in Section 7. The activated sludge portion of the works, namely the biological reactors and secondary settling tanks (SSTs), is divided into three equal modules (modular design discussed in Section 5.3). The head of works and the primary settling tanks are common structures for the three activated sludge modules. The flow split for the three reactor modules takes place upstream of the reactor. The flow from the PSTs to the reactor splitting unit is in a common conduit (pipe or channel). The SST supernatant flows to a common chlorine contact channel, and from there into the maturation ponds. The waste activated sludge is dewatered and sent to a new proposed biosolids beneficiation facility (BBF) for solar drying. The primary sludge stream will be anaerobically digested.

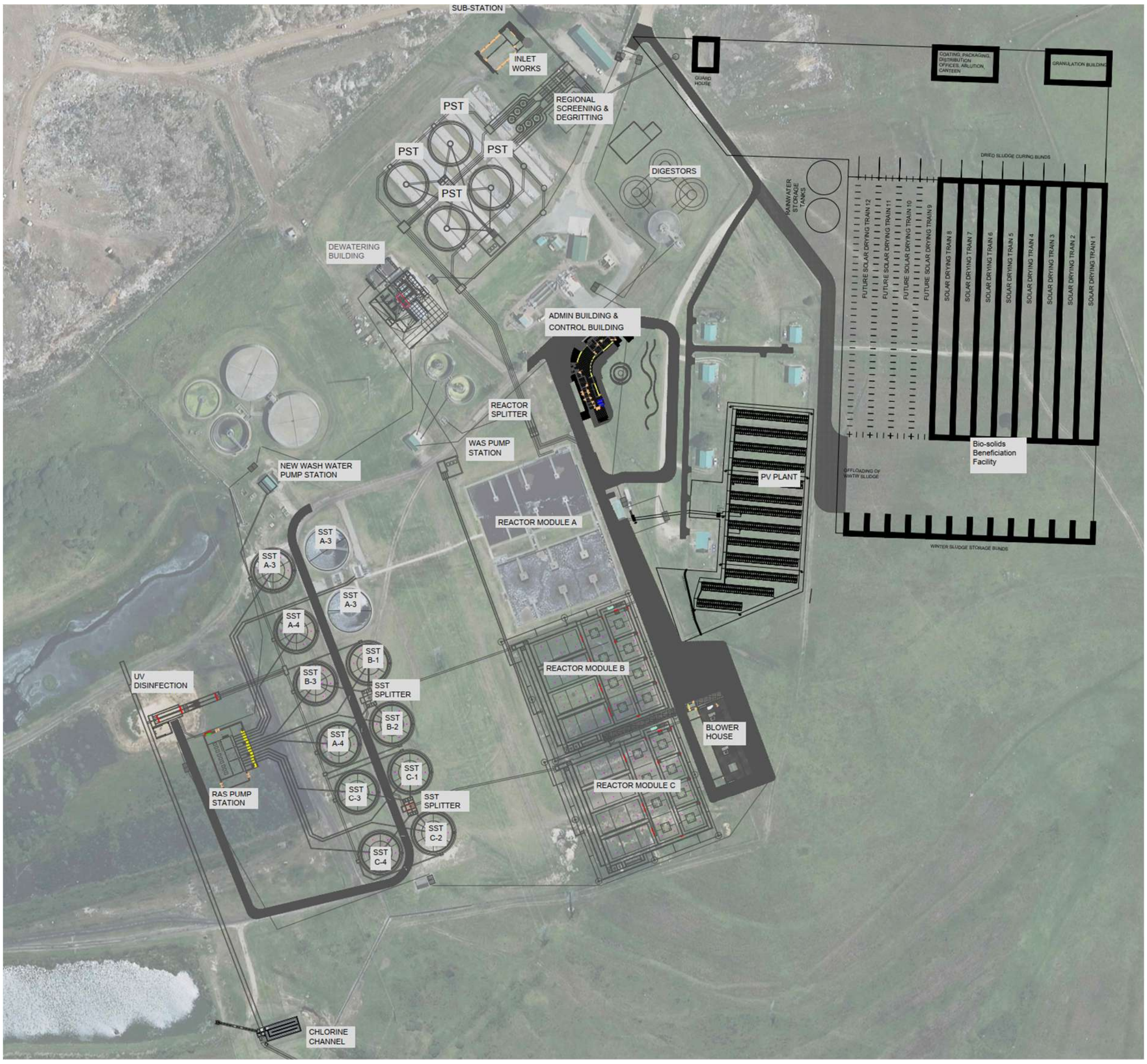


Figure 6-1: Ultimate solution (50 MLD) site layout

6.1.1 Head of Works

The head of works collects the total incoming flow into the Gwaing WWTW, including the flow discharged by vacuum tankers. The head of works is a single structure that serves the entire WWTW. The head of works is designed in such a way that it can be constructed in two separate phases. This is to allow for capital offset and construction of each phase when required. The phasing options will be discussed in more detail in Section 7.

Two pipelines discharge raw sewage into the existing inlet works. One being a 900 mm diameter gravity sewer main and the other a 350 mm diameter rising main from Gwaing pumpstation. The indicative positions of these two pipelines are shown in Figure 6-2. Future flows are expected to join the inlet works from two pump stations Groeneweide pumpstation 1 and 2.

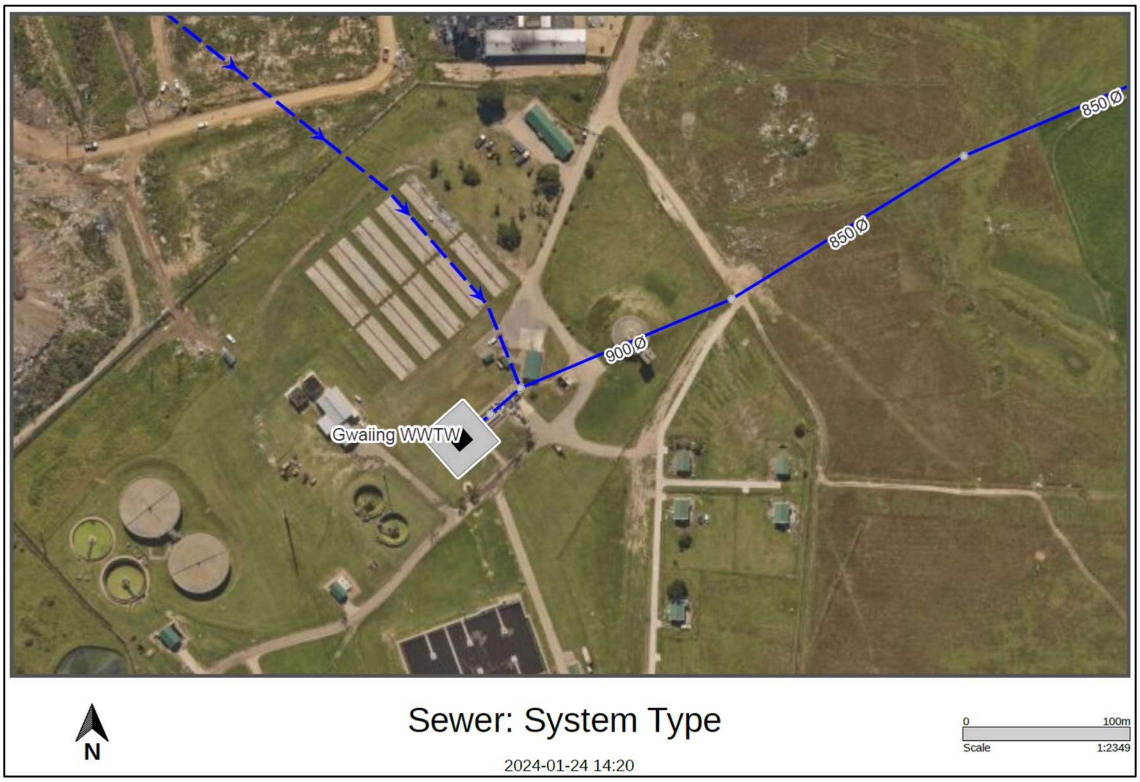


Figure 6-2: Main Incoming Raw Sewage Pipelines (GLS)

Table 6-1 shows the following flows for the existing and future pipelines:

- Estimated existing operational flows (based on currently developed plots),
- The existing theoretical flows (based on both undeveloped and developed plots),
- Future flows (based on planned new developments and changes in the catchment) and
- The maximum pipeline capacity of the current and planned future pipelines.

The total future theoretical maximum pipeline capacity feeding Gwaing WWTW is estimated to be 134.87 MLD. The bulk, 74 MLD, of this theoretical capacity lies in the 900 mm gravity main which comes from more developed areas. To cater for the ultimate PWWF design capacity (115 MLD) of Gwaing WWTW's masterplan, bigger or additional pipelines from new developments might be required if the projected population growth, as shown in Figure 3-5, is realised.

Table 6-1: Estimated Flows Feeding Gwaing WWTW (Source: GLS Consulting Hydraulic Models)

Estimated Flows (MLD)	Existing Operational - MLD (PWWF)	Existing Theoretical - MLD (PWWF)	Future - MLD (PWWF)	Theoretical Maximum Pipeline capacity
900mm Gravity main	25.6	28.4	19.01	74.0
Gwaing Pump station (Proefplaas)	15.1	15.1	8.64	15.1
Groeneweide PS 1 (Future)			2.07	2.1
Groeneweide PS 2 (Future)			43.63	43.6
Total	**40.69**	**43.55**	**73.35**	**134.87**

The proposed position of the new head of works (inlet works) is located on top of the redundant sludge drying beds. The existing two raw sewer pipelines will have to be re-routed to join the new inlet works at this position. All pumped pipelines shall be routed to the new inlet works such that they enter the structure from the top with a gooseneck to free discharge into the stream.

Due to the hydraulic head required to gravitate from the new inlet works through the new PST's and into the existing and new reactors, the new inlet works will need to be a few meters higher than the existing inlet works. A detailed hydraulic analysis in association with GLS Consulting indicated that it is unfeasible to gravitate to the higher level of the new inlet works. Various pipe gradients, pipe sizes and pipe routes were considered and even without the losses through the PSTs and PST splitter structures the manholes of the gravity main are expected to overflow with increased future flows. For the gravity sewer line, a low lift pump station will be required before the inlet works to provide the hydraulic head to cater for the additional losses through the future PSTs, flow measurement structures and chambers. It is proposed that this low lift pumpstation form part of the inlet works.

The Head of Works consists of the following:

- Low lift pump station for the 900 mm raw sewer gravity main
- Honeysucker discharge point at the head of the works.
- Front raked mechanical coarse (12 mm) bar screens, screenings conveyor and screenings washer/compactor.
- Emergency bypass channels to bypass the coarse screening. The bypass channels will each be fitted with hand-raked bar screens (30 mm).
- Fine perforated plate screens (3.5 mm apertures), screenings conveyor and screenings washer/compactor.
- Hydraulically assisted vortex degritters, each fitted with a grit loosening system, air lift pump grit removal system, and an inclined screw classifier.

The 3D model of the Head of Works is shown in Figure 6-3 and the plan layout in Figure 6-4. The figures exclude any mechanical infrastructure and the channel cover slabs.

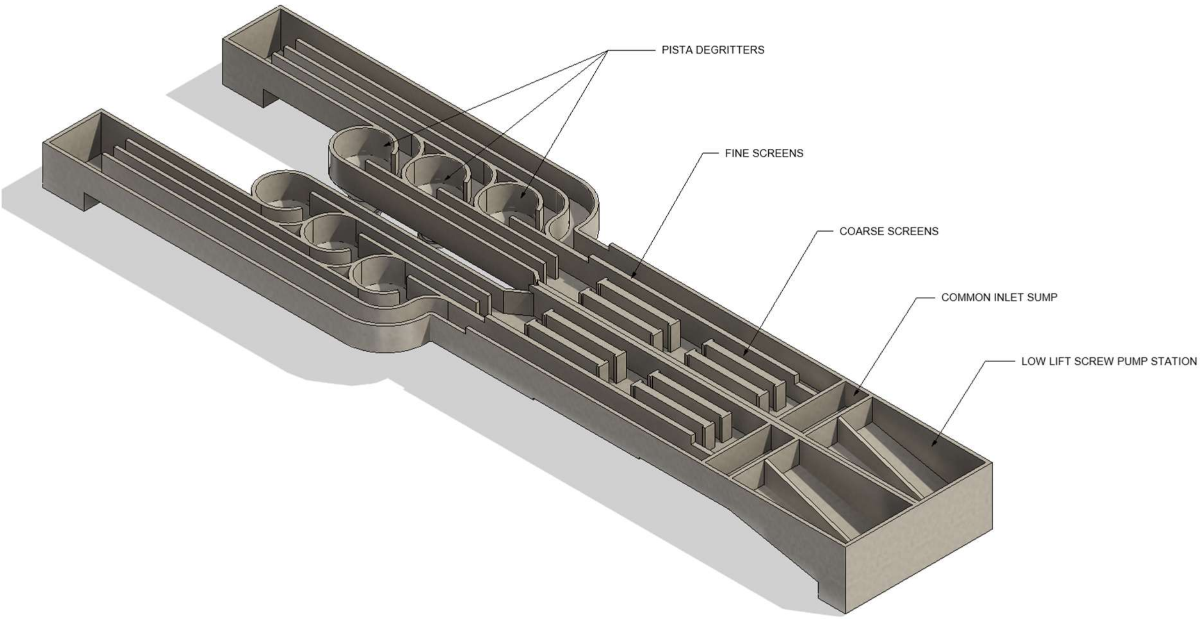


Figure 6-3: Head of Works 3D model

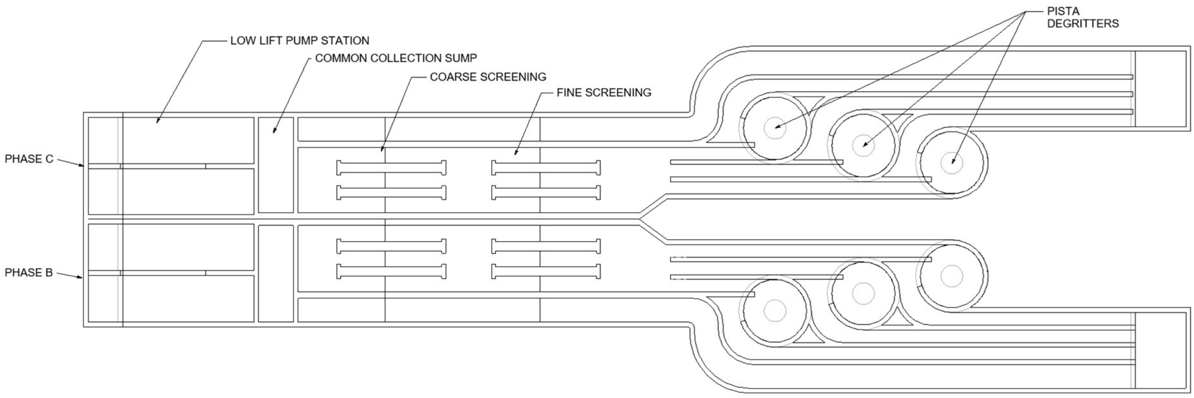


Figure 6-4: Head of works channel plan layout

6.1.1.1 Screening channels

The wastewater enters the Head of Works in a common sump, the flow is hydraulically split into the coarse- and fine-screening channels. Each channel can be isolated upstream and downstream of the respective coarse screens by means of penstocks.

Two bypass channels are provided in the event that the screens are all blinded or if the screens need to be bypassed for maintenance purposes. The bypass channels are designed to cater for all flows up to PWWF. Both bypass channels are fitted with manually raked coarse bar screens. The bypass channel joins the common chamber downstream of the fine screens. From the common chamber downstream of the fine screens, the wastewater flow is hydraulically split between the hydraulically assisted vortex degritters.

Table 6-2: Screenings channels configuration – 50 MLD

Screening Channels	Duty	Standby
Channels – Hydraulic capacity	4	2
Bypass channels – Hydraulic capacity	0	2
Course Screens – 12 mm front raked screens	4	2
Fine Screens – 3.5 mm perforated plate screens	4	2
Water Launders	4	0
Washer Compactors	2	2

6.1.1.2 Vortex degritters

The wastewater is hydraulically split between six hydraulically assisted vortex degritters. Each of the six degritter trains can be isolated by means of sluice gates installed upstream and downstream of each respective degritter. The wastewater is again combined downstream of the vortex degritter installation before entering the main distribution box.

Return water from the coarse screening, degritting and fine screening installations is reintroduced into the inlet channel of each respective installation.

Two bypass channels are provided in the event where the flow into the degritters is blocked, either by closed sluice gates, or if maintenance is being conducted. The bypass channels are sized to cater for the full PWWF. The bypass channels join the main distribution box downstream of the degritter channels before flowing to the PST splitter box. The design parameters for the vortex degritters are indicated in Table 6-3. The vortex degritters are designed to have a duty standby configuration as described in Table 6-4.

Table 6-3: Degritter design parameters

Degritter Design Parameters	
Degritter Type	Vortex Degritters
Rotation	270 degrees
Particle Settling Velocity	25 mm/s
Max. Horizontal Velocity	0.3 m/s
Min. Horizontal Velocity	0.1 m/s

Table 6-4: Vortex degritter configuration

Vortex Degritters	No. Installed	Duty Hydraulic Design Peak (150 MLD)	Duty Process Design Peak (115 MLD)
Vortex Degritters – 4 m diameter	6	6	5
Degritter paddles – 0.3 m/s rotation speed	6	6	5
Grit classifiers	6	6	5

6.1.2 Regional Grit and Screenings Facility

George Municipality has identified Gwaing WWTW as a possible location for a regional degritting and screening facility for washing of grit and screenings from surrounding wastewater treatment works and pumpstations. These works include Gwaing WWTW, Outeniqua WWTW, Kleinkrantz WWTW, Uniondale WWTW and several pumpstations that have screenings and degritting infrastructure. Grit and screenings are collected in separate waste skips at the above-mentioned facilities. From there they will be transported to Gwaing WWTW for further cleaning. The purpose of a centralised facility, is to further clean or wash the grit and screenings to remove any nutrients or biological matter still entrapped in the material. The cleaned screenings can then be disposed of at the George solid waste facility, which borders Gwaing WWTW. The cleaned grit material can be disposed of or used in several ways mentioned in Section 6.1.2.1 below.

6.1.2.1 Degritting facility

The proposed regional grit facility will be receiving grit that is that is both raw and processed. The raw grit will be received from Uniondale WWTW, which has only grit channels and no grit classifers. The grit is removed manually and placed into containers for removal from site. The processed grit will be received from Outeniqua WWTW, Kleinkrantz WWTW and on site from Gwaing WWTW. These three facilities have grit classifiers which are used to wash and dewater grit. The grit received from all the plants will be dry volumes. The average daily grit volumes generated by each of the four wastewater treatment works is summarised in Table 6-5.

Table 6-5: Average daily grit volumes generated at each WWTW

WWTW Site	Average Daily Grit Volume (m^3)
Gwaing WWTW	0.107
Outiniqua WWTW	0.120
Kleinkrantz WWTW	0.005
Uniondale WWTW	0.043
Total:	**0.276**

The purpose of the centralized grit treatment facility is to further wash and clean the grit in order that it is fit for commercial use. The main targets for the grit facility are as follows:

- Remove organic matter and nutrients from the grit. This implies that the VSS/TSS ratio is reduced to less than 5%.
- Reduce the odour of the grit.
- Reduce microbial content within the grit. If the grit is classified as a sludge, a sludge classification of A1a is targeted. This means that faecal coliforms and Helminth ova counts should comply with the following limits:
    - Faecal coliforms < 1000 CFU/g_{dry}
    - Helminth ova < 1 Viable ova/g_{dry}
- The washed and dried grit can also be used as a composting additive or backfilling material (selected fill).

The total volume produced by the four wastewater treatment works is approximately 0.276 m^3 per day on average. The size of a standard waste skip is 6 m^3, this means that a single waste skip will be received at the regional facility once every 21 days (on average). The daily average volumes will increase as the flow to each of the plants increase over time. Even when considering the increased volumes, the regional facility will not be a continuous operating system with a constant flow, but rather a batch operating system that only operates when a waste skip of grit is delivered to the facility. Figure 6-5 shows the process flow diagram of the mechanical grit washing system proposed.

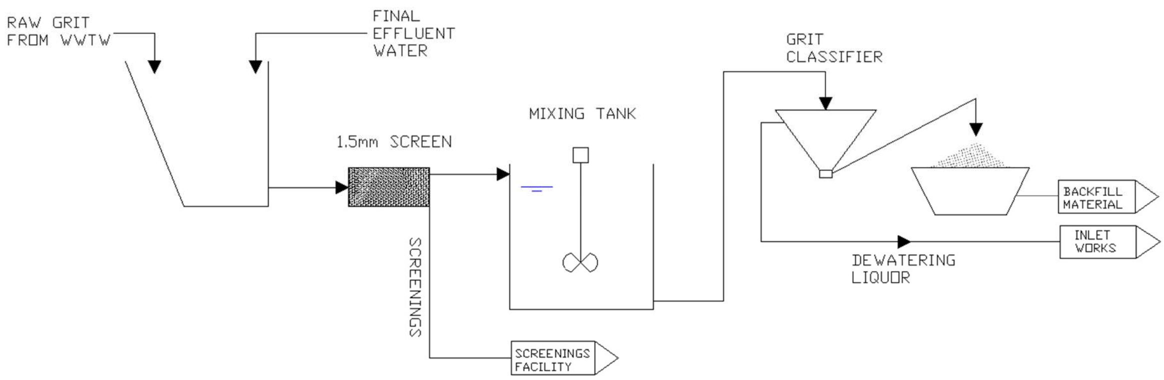


Figure 6-5: Mechanical grit washing system process flow diagram

The grit received from the various WWTWs will be tipped into a collection sump and will be diluted with final effluent water to create a flowable slurry. From the collection sump, the slurry will flow through a fine screen with a 1.5 mm aperture size. This is to remove any larger items that may be present in the grit. The screenings collected will be taken to the adjacent screening facility. The screened slurry will flow into a mixing tank where the slurry will be mechanically agitated. The agitation process will detach biological matter from the grit and bring it into suspension. From the mixing tank the slurry will be pumped to a grit classifier where through a vortex rotation system, the water, together with the volatile suspended solids is separated from the grit. The dewatering liquor, with high VSS content is returned to the inlet works of Gwaing WWTW. The extracted grit is transported via a screw press, which further dewaters the grit, into skip bins. At this point, the grit can be used as a commercial filling material.

It is recommended that a grit sample is taken from each of the waste water treatment works that are envisaged to send grit to Gwaing WWTW, and the organic (VSS/TSS) and microbial content be tested. This will give a better understanding of the quality of the grit to be received at the regional facility.

6.1.2.2 Screening facility

The screenings received from Kleinkrantz WWTW, Uniondale WWTW and that generated at Gwaing WWTW are raw uncompacted screenings. Outeniqua WWTW is the only plant that currently has washer compacters for the screenings. Table 6-6 shows the average screenings volumes generated by each wastewater treatment works per day on average.

Table 6-6: Average daily screenings volumes generated at each WWTW

WWTW Site	Average Daily Screenings Volume (m^3)
Gwaing WWTW	0.175
Outiniqua WWTW	0.107
Kleinkrantz WWTW	0.026
Uniondale WWTW	0.029
Total:	**0.336**

As with the degritting facility, the volumes are too small to have a continuous operating system. The system proposed is a batch operating system that only operates when a waste skip of screenings is delivered to the facility.

The main targets of the centralised screenings facility is as follows:

- Washed screenings quality < 40 mg COD/g_{dry}
- The quality of screenings increased to non-hazardous to allow for safe disposal at solid waste facility.
- Compaction of screenings reduces waste volume

The proposed screenings process flow diagram is shown Figure 6-6. The screenings received from the various WWTWs will be dumped into a collection chamber. The dry raw screenings will be transported to a washer compactor by means of a screw conveyor. The washer compactor will wash the screenings with final effluent water. The washed screenings are dewatered with a screw press and discharged into a waste skip. The dewatering liquor from the washer compactor is returned to the inlet works of Gwaing WWTW and the compacted dry screenings are in a state to be safely disposed of.

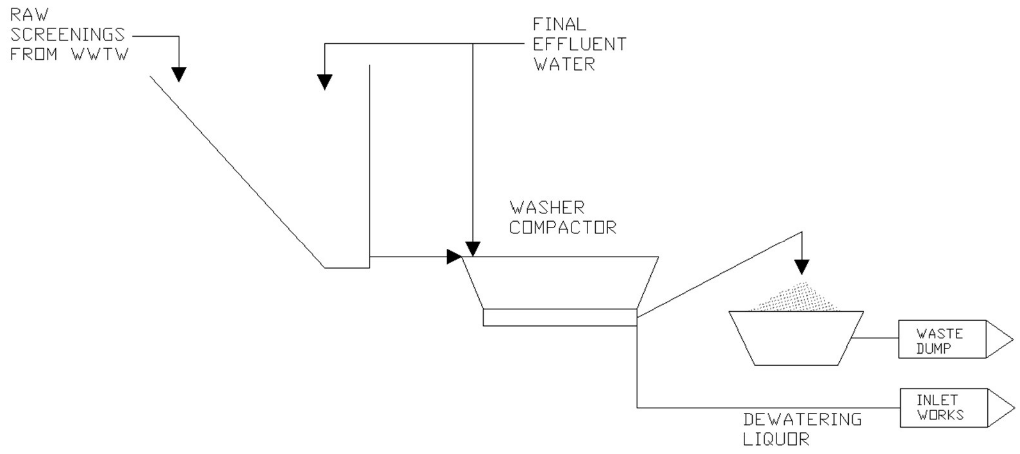


Figure 6-6: Regional screenings facility process flow diagram

The proposed location of the regional screenings and degritting facility is adjacent to the proposed new inlet works as shown in Figure 6-7.

Figure 6-7: Regional screening and degritting schematic position

6.1.3 Primary settling tanks

The primary settling tanks are designed with the underflow and overflow parameters as described in Section 5.4. The raw wastewater from the inlet works flows to a common splitter chamber in the centre of the four PSTs. The splitter chamber splits the flow equally between the four PSTs. The flow to each PST can be stopped by closing a penstock in the splitter chamber. Some or all of the PSTs can be bypassed with the bypass pipelines, which are sized for the hydraulic PWWF. The flow can be optionally bypassed by opening the penstocks to the bypass lines. If the PSTs are blocked or the penstocks closed, the water will overflow into the bypass pipelines. The PST and splitter configuration proposed is shown in Figure 6-8.

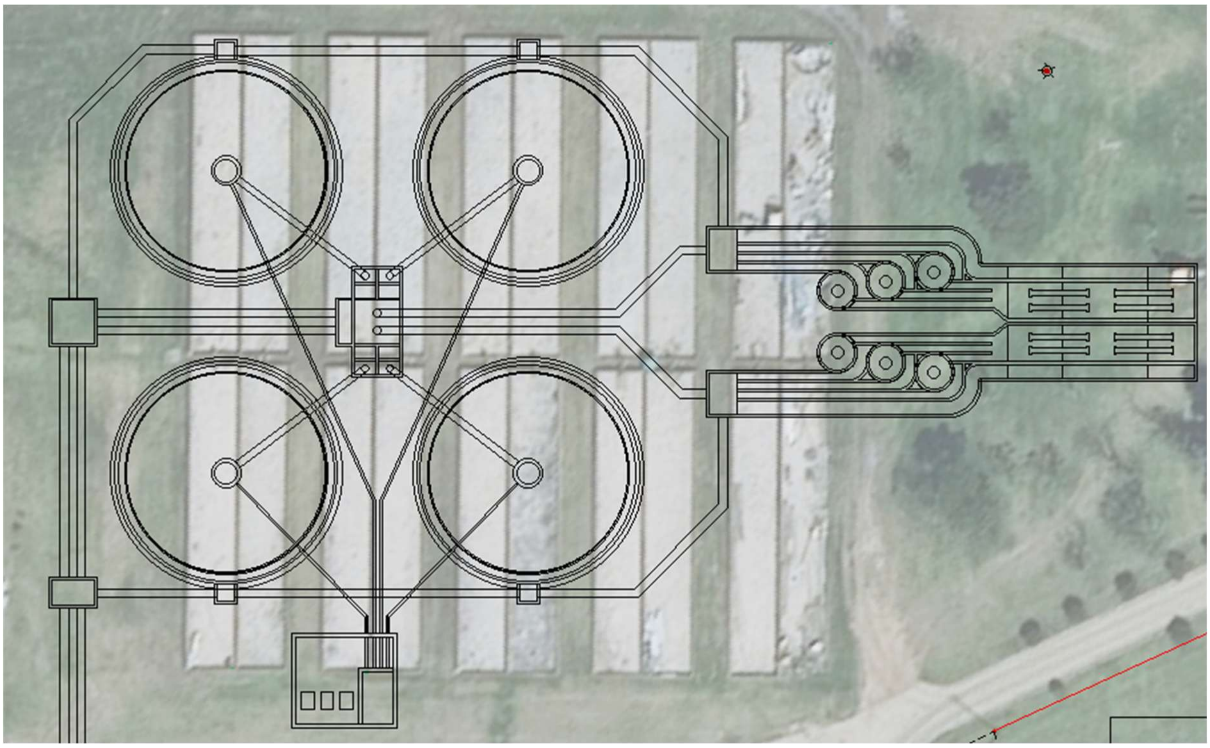


Figure 6-8: PST configuration

The underflow (primary sludge) is abstracted from the sludge collection sump of the PSTs and flows to the primary sludge pump station, which is situated at the bottom of Figure 6-8. The primary sludge handling is discussed in more detail in Section 6.1.9. A typical section of a 25 m PST is shown in Figure 6-9 below. The figure shows only the civil infrastructure and does not indicate the pipework or scraper bridge.

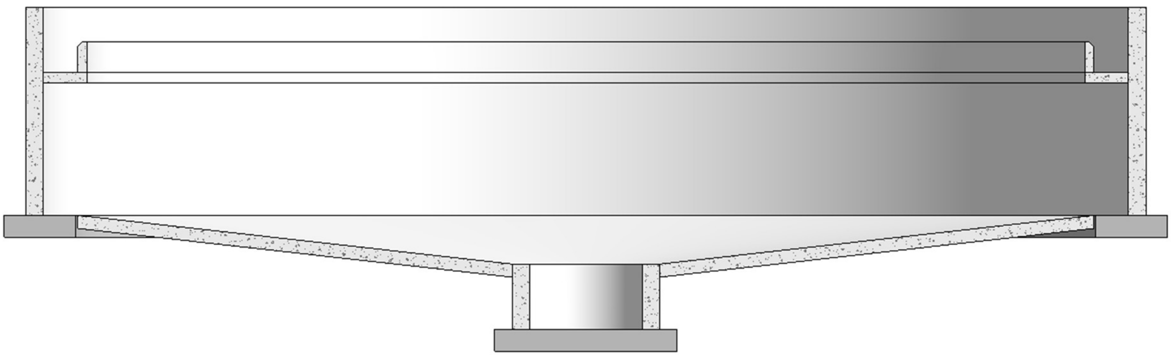


Figure 6-9: Typical 25m PST cross-section

The PSTs have sidewall depths of 4.0 m, with sloped floors for efficient sludge removal. The PSTs will also be installed with a scum baffle to prevent scum from overflowing to the reactors. The scum is removed by a scraper and scum removal mechanism which will flow into the primary sludge pump station and be treated with the primary sludge.

6.1.4 Biological Reactors

The existing reactor is configured to operate as either an MLE or a UCT process. The MLE process includes anoxic and aerobic zones only while the UCT process includes an anaerobic zone upstream that together with the r-recycle stream enables enhanced biological phosphate removal. Two additional reactors are required with the same volume as the existing reactor for the ultimate capacity of 50 MLD ADWF. The layout of the existing reactor and two additional reactors is shown in Figure 6-10.

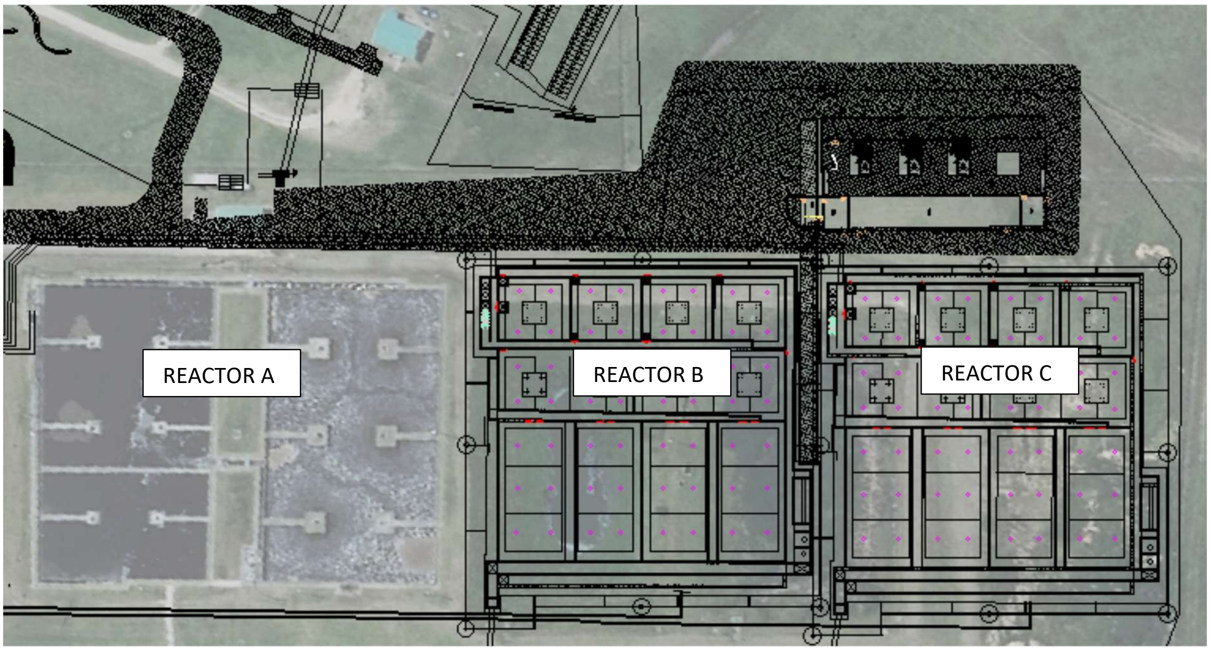


Figure 6-10: Reactor site layout configuration

A common channel or pipe will flow from the PSTs to the flow split unit (left side of Figure 6-10) upstream of the reactors. The flow split is achieved by three equally sized open-channel rectangular flumes. The flumes have a dual purpose in that it is used to measure the flow to each reactor and give an equal flow split to each. The waste activated sludge (WAS) is abstracted from the end of the aerobic zone of each reactor. The WAS sludge pump station is situated on the bottom left side of Figure 6-10. From there the WAS is pumped to the dewatering building. The WAS handling is discussed in more detail in Section 6.1.7.

The following recycle pump stations are included in the reactor design.

- The a-recycle transfers mixed liquor from the end of the aerobic zone to the beginning of the anoxic zone.
- The r-recycle transfers mixed liquor from the end of the anoxic zone to the beginning of the anaerobic zone.

The internal dividing walls between the zones will be submerged walls (submerged weirs), to ensure scum forming on the surface of the reactor passes through the reactor in the direction of flow. The 3D model of the biological reactor is shown in Figure 6-11

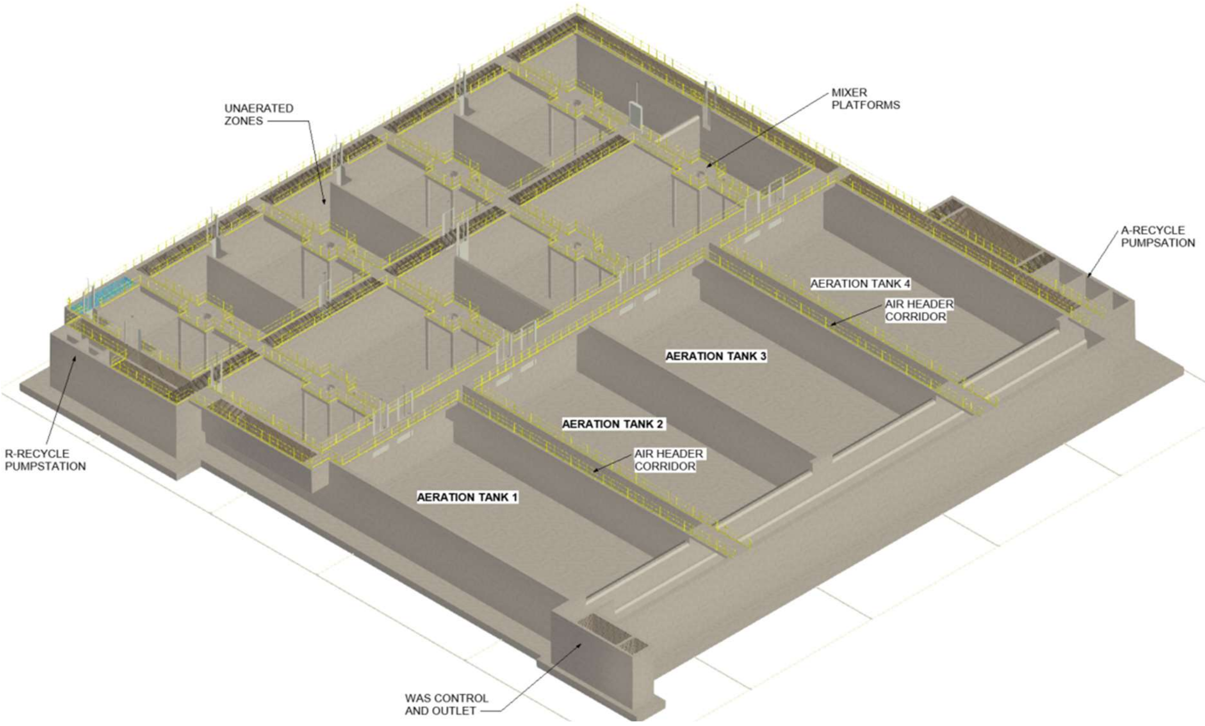


Figure 6-11: Biological reactor 3D model

The anoxic zones will be mechanically mixed with vertical shaft mixers, while air will be introduced into the mixed liquor contained in the aerobic zones by means of a fine bubble diffused aeration system for Modules B and C. The aeration system and blower house is discussed in more detail in Section 6.1.10.

6.1.5 Secondary Settling Tanks (SSTs)

Mixed liquor from each bioreactor module gravitates to a flow division box, where the activated sludge is hydraulically divided between the SSTs. Each reactor module has 4 dedicated SSTs. The existing reactor has two existing SSTs. Two additional SSTs will be constructed for the existing reactor. The ultimate solution of 50 MLD will have 12 SSTs in total. The existing two SSTs are 25 m in diameter and ten new SSTs having a diameter of 25 m are required for the ultimate solution. The SSTs are designed with a side wall depth of 4.0 m and sloped floors to the centre sludge collection well. The overflow collection launders are internal launders and form a Stamford-type baffle.

Similar to the PST design, the typical cross section of a 25 m SST is shown in Figure 6-12.

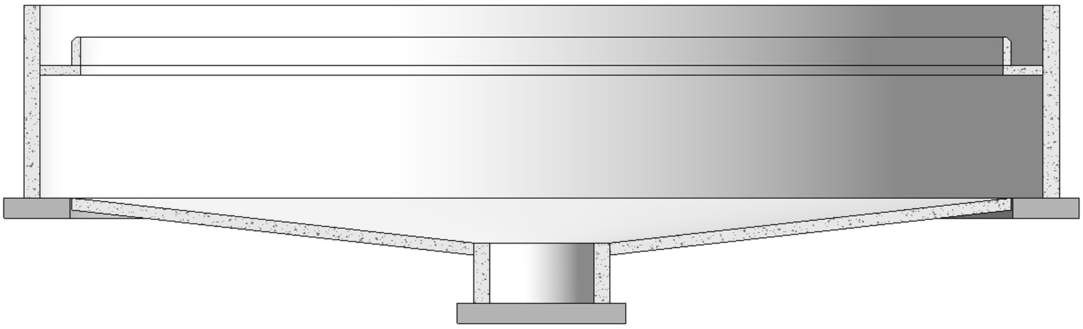


Figure 6-12: Typical 25m SST cross section

Interconnection between all the SSTs will be provided such that there is optional interconnectivity between the three modules' SSTs. If one of the reactors is out of operation for maintenance, the SSTs for that module can still be used. Or if one or more SSTs are out of operation, the reactor's flow can be split between the remaining operational SSTs to maximise the treatment of all modules. The SST splitter chambers are designed in a manner that the reactors and SSTs can either be operated as isolated modules, or as an interconnected system. The SST configuration for the ultimate upgrade is shown in Figure 6-13.

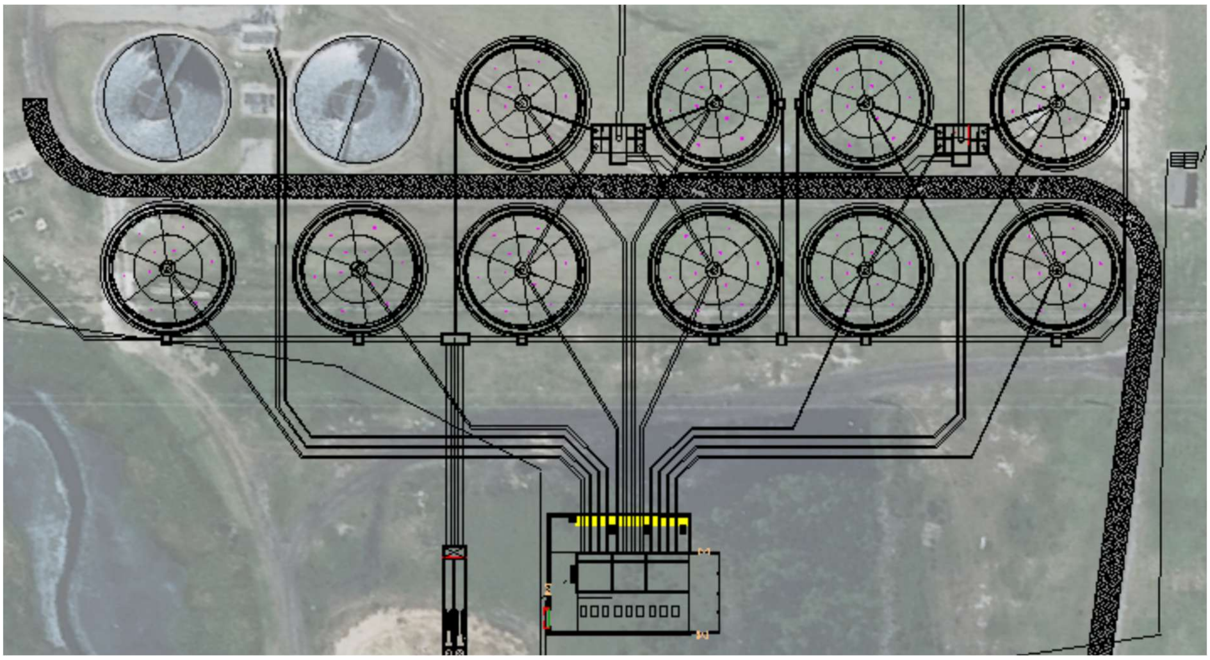


Figure 6-13: Secondary settling tank configuration

The overflow from each Module's set of secondary settling tanks will flow under gravity into a common channel, and from there into the chlorine disinfection contact channel. The underflow return activated sludge (RAS) from each Module gravitates through a flow-meter chamber and into the RAS pump station. The RAS flow (or s-recycle) from each SST is set to a constant flow of 2 x ADWF and is controlled by a telescopic bell-mouth. The RAS extracted from each SST is measured in the flow meter chamber to ensure the recycle ratio is correct. From the RAS pumpstation the sludge is returned to the anoxic zone compartment of each bioreactor.

6.1.6 Disinfection

All clarified effluent from the secondary settling tanks will gravitate to the UV channels. Two UV channels are proposed to provide standby for critical components that control the UV dose. For Phase C and D of the upgrades, additional banks will be fitted into the channels. Each channel will be fitted with a flume to split the flow equally between the two channels and to measure the flow. The flow measurement will be used to adjust the dose of the UV banks. In combination to flow measurement, a UVT analyser in each channel will adjust the dose based on the flow and the UVT reading. A modulating weir gate in each channel will ensure that the UV lights are always submerged under water for all flows. The location of the UV channels is indicated in Figure 6-14.

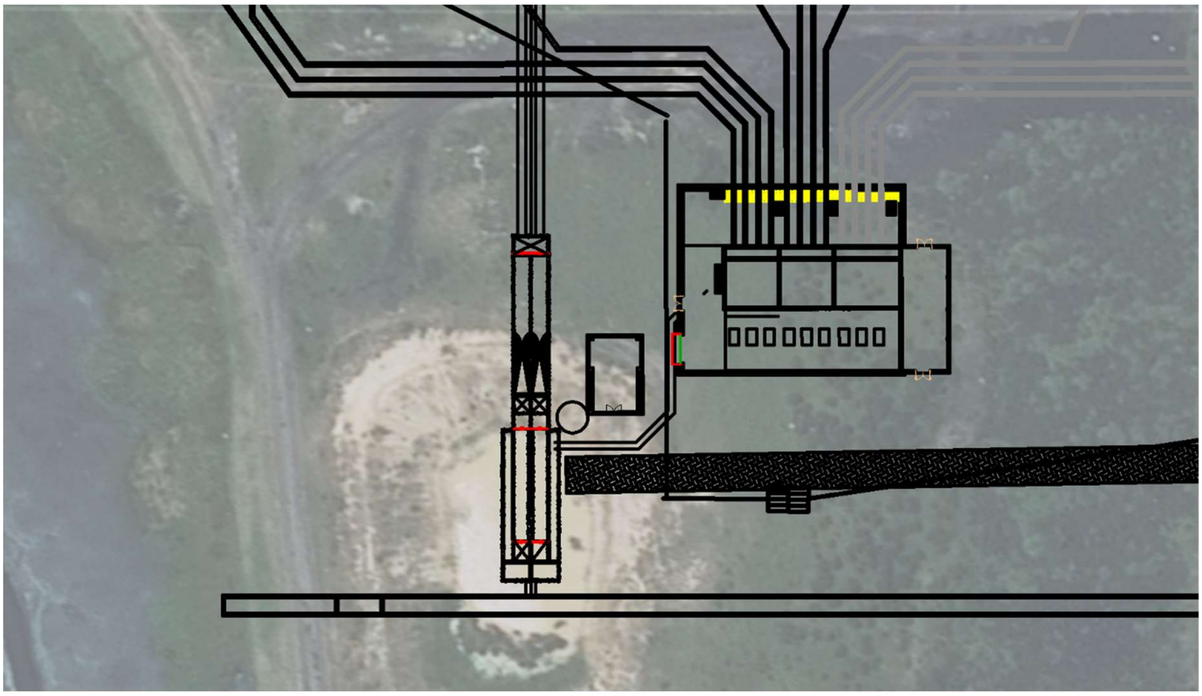


Figure 6-14: UV Channels site location

Once through the UV channels, the effluent can either flow into the maturation ponds, or be diverted directly to the existing chlorine contact tank where it will be discharged to the Gwaing River. The existing Chlorine dosing facility will remain in operation until the new UV disinfection facility is commissioned. Once the UV disinfection is commissioned, the existing dosing building can be de-commissioned. The existing chlorine contact tank will not serve any function once the UV disinfection is commissioned. If the George Municipality opt to implement Chlorine disinfection under Phase B instead of UV disinfection, the existing chlorine contact channel may serve as additional contact time before the final disinfected effluent is discharged into the environment.

As discussed in Section 5.7, for implementation, a final decision needs to be made whether UV disinfection will be selected as a preferred alternative to chlorine disinfection.

6.1.7 Waste Activated Sludge (WAS) Dewatering

The waste activated sludge is pumped from the reactors to the dewatering building. The volume of sludge produced by each reactor at Gwaing WWTW is summarised in Table 6-8 for the UCT raw and UCT settled processes respectively. Table 6-8 does not include the sludge from Outeniqua WWTW.

At present the design philosophy is to optionally dewater sludge from Outeniqua WWTW separately or combined with the sludge from Gwaing WWTW. There will be benefits in combining the dewatering of the two WWTW's, since some equipment will be common to both dewatering plants and the ability to dewater on all the beltpresses combined increases redundancy.

Table 6-7: Existing Belt Press Capacities

Existing Beltpress Capacities	Hydraulic Capacity	Solids Loading Capacity
	(m^3/hr)	(kg/hr)
FBO No 1 1.8m nominal belt complete with integral 1.5m linear table	58	525
FBO No 2 **1.8m nominal belt complete with integral 1.5m linear table**	58	525

Table 6-8: Capacity Check of 1, 2 or 3 beltpresses allocated to Gwaing WWTW (and the other beltpress dedicated to Outeniqua WWTW) when operating as UCT raw or UCT settled process for 8,12,16 or 24 hours per day. Cells highlighted red represents overloading of the beltpresses. Green text represents the options chosen for design.

			1 Beltpress for Gwaing WWTW				2 Beltpresses for Gwaing WWTW				3 Beltpresses for Gwaing WWTW			
Raw UCT process -			Beltpress Operating Hours per day				Beltpress Operating Hours per day				Beltpress Operating Hours per day			
			24	16	12	8	24	16	12	8	24	16	12	8
			Hydraulic Loading Rate (m^3/hr)				Hydraulic Loading Rate (m^3/hr)				Hydraulic Loading Rate (m^3/hr)			
Waste Flow Rate for 1 Reactor	672	m^3/d	28	42	56	84	14	21	28	42	9	14	19	28
Waste Flow Rate for 2 Reactors	1344	m^3/d	56	84	112	168	28	42	56	84	19	28	37	56
Waste Flow Rate for 3 Reactors	2016	m^3/d	84	126	168	252	42	63	84	126	28	42	56	84
			Solids Loading Rate (kg/hr)				Solids Loading Rate (kg/hr)				Solids Loading Rate (kg/hr)			
Mass TSS wasted for 1 Reactor	3293	kgTSS/d	137	206	274	412	69	103	137	206	46	69	91	137
Mass TSS wasted for 2 reactors	6586	kgTSS/d	274	412	549	823	137	206	274	412	91	137	183	274
Mass TSS wasted for 3 reactors	9879	kgTSS/d	412	617	823	1235	206	309	412	617	137	206	274	412
Settled UCT process -			Beltpress Operating Hours per day				Beltpress Operating Hours per day				Beltpress Operating Hours per day			
			24	16	12	8	24	16	12	8	24	16	12	8
			Hydraulic Loading Rate (m^3/hr)				Hydraulic Loading Rate (m^3/hr)				Hydraulic Loading Rate (m^3/hr)			
Waste Flow Rate for 1 Reactor	669	m^3/d	28	42	56	84	14	21	28	42	9	14	19	28
Waste Flow Rate for 2 Reactors	1338	m^3/d	56	84	112	167	28	42	56	84	19	28	37	56
Waste Flow Rate for 3 Reactors	2007	m^3/d	84	125	167	251	42	63	84	125	28	42	56	84
			Solids Loading Rate (kg/hr)				Solids Loading Rate (kg/hr)				Solids Loading Rate (kg/hr)			
Mass TSS wasted for 1 Reactor	2910	kgTSS/d	121	182	243	364	61	91	121	182	40	61	81	121
Mass TSS wasted for 2 reactors	5821	kgTSS/d	243	364	485	728	121	182	243	364	81	121	162	243
Mass TSS wasted for 3 reactors	8731	kgTSS/d	364	546	728	1091	182	273	364	546	121	182	243	364

The WAS generated at Gwaing WWTW when for the ultimate capacity of the UCT settled process, is 8731 kg/d. For the same number of modules, operating a UCT Raw process, a total mass of 9879 kg/d is wasted. The limiting factor however, is the hydraulic loading rate of the belt presses. The existing belt-presses have a hydraulic capacity of 58 m^3/hr, which means the belt-presses will be overloaded as shown by the red highlights in Table 6-8 according to the operating hours and the number of Reactors in operation.

The green text in Table 6-8 represents the chosen operating hours and number of beltpresses required for each reactor at Gwaing WWTW. A single beltpress per reactor is required, operated for 12 hours per day. By the time that three reactors are operational at Gwaing WWTW, there will be three 1.8 m wide beltpresses required to dewater the WAS sludge of Gwaing WWTW only. No standby beltpress is envisioned since there is room to increase the operating hours per day, should one of the beltpresses fail.

Ultimately Gwaing WWTW and Outeniqua WWTW will each require three dedicated 1.8 m wide beltpresses with linear screens (gravity tables), if we assume that Outeniqua WWTW's capacity will grow at a similar rate to Gwaing WWTW. This results in a total of six beltpresses required for the ultimate solution.

It is proposed that the dewatering facility be upgraded in two phases, with each phase adding an additional two 1.8 m beltpresses and ancillary equipment. The first set of two beltpresses should be installed as part of Phases A and/or B while the second set of two beltpresses should be installed as part of Phase C (see Section 7.3 for phasing). The operational flexibility to dewater WAS from Outeniqua WWTW separately or to mix it with that of Gwaing WWTW will be maintained.

Waste Activated Sludge (WAS) that is released from the reactors needs to be stored in WAS holding tanks before it is pumped to the beltpresses. This storage serves to balance the difference in flow of what is coming from the reactors to what is going to the beltpresses. By wasting from the reactors at a consistent frequency the need for large WAS storage volumes is reduced. These WAS holding tanks also provide some contingency storage to allow wasting from the reactors to continue if there is an operational issue at the dewatering facility. As the capacity of the Gwaing WWTW increases, the storage of the WAS holding tanks should increase accordingly. It is proposed that an additional WAS storage volume of 380 m^3 be provided for the WAS from Gwaing WWTW, resulting in the retention times as shown in Table 6-9.

Table 6-9: WAS holding tank additional volume and corresponding retention times for UCT Raw process.

Raw UCT Process WAS Holding Tank Volumes	Volume	
Existing Holding Tank Volume	**190**	**m^3**
Existing Holding Tank Retention 1 Reactor (Existing)	*6.8*	*hrs*
Existing Holding Tank Retention 2 Reactors (Phase A&B)	*3.4*	*hrs*
Existing Holding Tank Retention 3 Reactors (Phase C&D)	*2.3*	*hrs*
Additional Holding Tank Volume	**380**	**m^3**
Additional Holding Tank Retention 1 Reactor (Existing)	*13.6*	*hrs*
Additional Holding Tank Retention 2 Reactors (Phase A&B)	*6.8*	*hrs*
Additional Holding Tank Retention 3 Reactors (Phase C&D)	*4.5*	*hrs*
Total Holding Tank Volume	**570**	**m^3**
Total Holding Tank Retention 1 Reactor (Existing)	*20.4*	*hrs*
Total Holding Tank Retention 2 Reactors (Phase A&B)	*10.2*	*hrs*
Total Holding Tank Retention 3 Reactors (Phase C&D)	*6.8*	*hrs*

It is important that the WAS holding tanks be simultaneously mixed and aerated. Mixing ensures that the feed to the beltpresses is homogenous and a consistent TSS concentration is fed to the beltpresses. Aeration is required to prevent anaerobic conditions so that phosphate is not released by the PAO's to the liquid stream. If this happens, phosphates won't exit the system as part of the dewatered WAS sludge, but will escape with the beltpress filtrate back to the reactors. This will in turn increase the effluent phosphate concentration, nullifying the benefits of the UCT process as an enhanced biological phosphate removal (EBPR) process.

Since the level in the WAS holding tank will rise and fall, the mixing and aeration system should be able to cope with these changes in the level. The current venturi type submerged aerators cannot deal with these variations effectively. Alternatives include floating surface aerators and low maintenance medium bubble diffused aeration. Both options are viable, however the medium bubble diffused aeration system is preferred for its efficiency and greater degree of control. Roots-type (rotary lobe) positive displacement blowers can be used since their operation is less sensitive to variable discharge pressures. These blowers can be equipped with VSD's so that airflow rates can be varied as the levels rise and fall for optimal energy usage. Surface aeration will not have the same turn-down capability and will not achieve an equally homogenous sludge, but will be a simpler solution in some regards. Surface aeration also causes more shearing of flocs than diffused aeration which could reduce the dry solids achieved on the beltpress or increase poly consumption. The final decision will be made during detail design.

WAS is typically dewatered to between 15% and 20% dry solids. Table 6-10 shows the expected mass and volume of WAS generated for Gwaing WWTW. Note that these volumes would approximately double if Outeniqua WWTW's WAS is added.

Table 6-10: Dewatered WAS produced and stored on site for Gwaing WWTW operated as Raw UCT Process (excluding Outeniqua WWTW WAS).

Average dewatering achieved (%DS)	**17%**			
Estimated dulk density of dewatered WAS sludge*	750 kg/m^3			
	Mass		Volume	
Dewatered sludge (incl moisture) for 1 Reactor	*19371*	*kg/d*	*26*	*m^3/d*
Dewatered sludge (incl moisture) for 2 Reactors	*38742*	*kg/d*	*52*	*m^3/d*
Dewatered sludge (incl moisture) for 3 Reactors	*58113*	*kg/d*	*77*	*m^3/d*
Days of sludge storage required on site	30 days			
Volume of dry sludge to be stored on site - 1 Reactor	775 m^3			
Volume of dry sludge to be stored on site - 2 Reactors	1550 m^3			
Volume of dry sludge to be stored on site - 3 Reactors	2325 m^3			

*Could vary between 750 kg/m^3 and 1200 kg/m^3

6.1.8 Biosolids Beneficiation Facility

6.1.8.1 Status Quo

George Municipality's current sludge disposal method is not compliant with sludge management guidelines. Sludge is currently being stored between the maturation ponds in an unlined area. This causes seepage of nutrients to the maturation ponds and the underlying aquifer.

The sludge produced currently is classified as class B1a according to a report by Herselman Consulting Services compiled in October 2021. The 'B' designation refers to microbiological class with the presence of faecal coliforms above 1000 CFU/g_{dry} and Helminth ova above 0.25 viable ova/g_{dry} preventing the sludge from achieving an 'A' designation for microbiological class. This places restrictions on how the sludge can be utilised. The designation '1' refers to the stability class while the ‘a’ refers to the pollutant class (metals). The sludge at Gwaing WWTW achieved the highest designation for these two classes.

The dewatered sludge from the beltpresses has 14-17% dry solids (DS). While this is dry enough to be carted away, it is still too ‘wet’ for most commercial uses. Composting or fertilizer facilities require drier sludge and new legislation requires that sludge have at least 40% DS before it can be applied to landfills in South Africa. The Western Cape Government’s DEADP and Waste Management Directorate has set targets to reduce organic waste to landfills by 50% by 2022 and to ban all organic waste from landfills by 2027. Hence application of sludge to landfills will not be a viable option in the near future.

To make the sludge a more attractive commodity for either the municipal composting facility or private compost and fertilizer manufacturers the sludge needs to be processed further at Gwaing WWTW to achieve a higher dryness (solids content) and/or a classification of A1a.

George Municipality is investigating several options for sludge beneficiation and as well as a diversified approach to be less dependent on a single solution.

6.1.8.2 Sludge Volumes

The combined waste activated sludge (WAS) and Primary Sludge (PS) wet, dewatered and dry masses and volumes for Gwaing WWTW are summarized in Table 6-11. The corresponding phases as defined in Section 7.3 are indicated in red text.

Table 6-11: WAS, PS and Total Sludge Quantities for Gwaing WWTW (excluding Outeniqua WWTW sludge) for the UCT process with and without PSTs. Phasing text in red.

Without PSTs		From Reactor to Dewatering				After Dewatering				After Solar Drying			
WAS from RAW UCT	Mass dry solids kgTSS/d	DS %	ρ kg/m³	Wet Volume	Wet Mass	DS %	Bulk ρ kg/m³	Moist Volume	Moist Mass	DS %	Bulk ρ kg/m³	Dry Volume	Dry Mass
WAS for 1 Reactor	3293	0.490%	1000	672 m³/d	672 t/d	17%	750	25.8 m³/d	19 t/d	70%	750	6.3 m³/d	4.7 t/d
WAS for 2 Reactors **[Phase B]**	6586			1344 m³/d	1344 t/d			51.7 m³/d	39 t/d			12.5 m³/d	9.4 t/d
WAS for 3 Reactors **[Phase C]**	9879			2016 m³/d	2016 t/d			77.5 m³/d	58 t/d			18.8 m³/d	14.1 t/d

With PSTs		From Reactor to Dewatering				After Dewatering				After Solar Drying			
WAS from Settled UCT	Mass dry solids kgTSS/d	DS %	ρ kg/m³	Wet Volume	Wet Mass	DS %	Bulk ρ kg/m³	Moist Volume	Moist Mass	DS %	Bulk ρ kg/m³	Dry Volume	Dry Mass
WAS for 1 Reactor	2910	0.435%	1000	669 m³/d	669 t/d	17%	750	22.8 m³/d	17 t/d	70%	750	5.5 m³/d	4.2 t/d
WAS for 2 Reactors	5821			1338 m³/d	1338 t/d			45.7 m³/d	34 t/d			11.1 m³/d	8.3 t/d
WAS for 3 Reactors **[Phase D]**	8731			2007 m³/d	2007 t/d			68.5 m³/d	51 t/d			16.6 m³/d	12.5 t/d
		From Anaerobic Digestor to Dewatering				After Dewatering				After Solar Drying			
Primary Sludge (PS)	Mass dry solids kgTSS/d	DS %	ρ kg/m³	Wet Volume	Wet Mass	DS %	Bulk ρ kg/m³	Moist Volume	Moist Mass	DS %	Bulk ρ kg/m³	Dry Volume	Dry Mass
PS for 1 Reactor	1786	1.86%	1000	96 m³/d	96 t/d	17%	750	14.0 m³/d	11 t/d	70%	750	3.4 m³/d	2.6 t/d
PS for 2 Reactors	3571			192 m³/d	192 t/d			28.0 m³/d	21 t/d			6.8 m³/d	5.1 t/d
PS for 3 Reactors **[Phase D]**	5357			288 m³/d	288 t/d			42.0 m³/d	32 t/d			10.2 m³/d	7.7 t/d
		Total to Dewatering				After Dewatering				After Solar Drying			
Total (WAS+PS)	Mass dry solids kgTSS/d	DS %	ρ kg/m³	Wet Volume	Wet Mass	DS %	Bulk ρ kg/m³	Moist Volume	Moist Mass	DS %	Bulk ρ kg/m³	Dry Volume	Dry Mass
WAS+PS for 1 Reactor	4696	0.61%	1000	765 m³/d	765 t/d	17%	750	37 m³/d	28 t/d	70%	750	9 m³/d	6.7 t/d
WAS+PS for 2 Reactors	9392			1530 m³/d	1530 t/d			74 m³/d	55 t/d			18 m³/d	13.4 t/d
WAS+PS for 3 Reactors **[Phase D]**	14088			2295 m³/d	2295 t/d			110 m³/d	83 t/d			27 m³/d	20.1 t/d

Since it is planned that Outeniqua WWTW’s sludge will also be dewatered and potentially dried at Gwaing WWTW, it is important that its sludge quantities are added to that of Gwaing WWTW. In the absence of detailed flow projections for Outeniqua WWTW at present it is estimated that the sludge from Outeniqua WWTW will approximately match that of Gwaing WWTW in the future. [This will be refined during detail design]. Therefore the actual dewatering and sludge handling or beneficiation capacities will be approximately double the quantities listed in Table 6-11.

6.1.8.3 Sources of Sludge

The George Municipal Area is serviced by six (6) wastewater treatment works (WWTW), excluding private wastewater treatment works. The wastewater treatment works vary in capacity and the volume of sludge they produce.

Wastewater streams are generally domestic in nature. Several registered industries and businesses discharge their waste through the bulk sewer system for treatment at the wastewater treatment works, these include cheese factories, restaurants etc. Outeniqua and the Gwaing WWTW both receive some industrial effluent.

A summarised description of each WWTW is provided below.

Gwaing WWTW

As discussed in this Report.

Outeniqua WWTW

The WWTW was upgraded in 2023 to 25 Ml/day. The WWTW is intended to be upgraded to 30 Ml/day in future (currently not a priority upgrade). The current ADWF is measured at 13 Ml/day.

The WWTW consists of the following main components:

- Inlet works (fine mechanical perforated screens, grit traps and classifier)
- Activated sludge module no. 1 (15 Ml/day) - carousel
  - Biological nutrient removal reactor (MLE process functionality), surface aeration
  - 2 x 35 m diameter secondary settling tanks
  - RAS archimedean screw pumps x3 (common pump station)
  - WAS pumps
- Activated sludge module no. 2 (10 Ml/day)
  - Fine bubble diffused aeration
  - 2 x 35 m diameter clarifiers (flat bottom)
  - RAS archimedean screw pumps x2 (common pump station)
  - WAS pump station
- Chlorine disinfection
- Waste activated sludge dewatering facility
  - None provided.
  - Sludge is pumped to the Gwaing WWTW dewatering facility

Kleinkrantz WWTW

The WWTW was upgraded in 2012 to 2.5 Ml/day. The WWTW is intended to be upgraded to 3 Ml/day in future (currently not a priority upgrade). The ADWF is measured at 1.1 Ml/day.

The WWTW consists of the following main components:

- Inlet works (fine screen, vortex degritter and classifier)
- Biological reactor no. 1 (2.5 Ml/day)
  - Surface aeration
  - 14 m and 16 m diameter Secondary Settling Tanks
  - RAS pumps
  - WAS pumps
- Chlorine disinfection
- Sludge drying beds.

A tender to construct a dewatering facility to replace the drying beds is underway and should be commissioned by 2026.

The dewatering facility will consist of a gravity belt thickener.

Uniondale WWTW

The WWTW has a capacity of 1ML/day and is intended to be upgraded to 1.3 Ml/day in future (currently not a priority upgrade). The treatment works currently has sufficient capacity to treat the wastewater.

The WWTW consists of the following main components:

- Inlet works (grit channels& mechanical screening removal)
- Biological reactor (1.0 Ml/day)
  - Surface aeration (MLE)
  - 2 x 9.5 m diameter secondary settling tanks
  - RAS pumps
  - WAS pumps
- Chlorine disinfection
- Sludge drying beds

Dried sludge is currently stockpiled on site.

Haarlem WWTW

The WWTW has a capacity of 0.1 ML/day and is intended to be upgraded to 0.8 ML/day (majority of the rural village have septic tanks per household). The treatment works currently has sufficient capacity to treat the wastewater.

The WWTW consists of the following main components:

- 8 x septic tanks, and
- 3 x maturation ponds

Septic tanks are emptied twice a year (every 4 – 5 months) and transported to the Uniondale WWTW. The sewage discharged at the Uniondale WWTW is then treated through the Uniondale WWTW process train.

Herold's Bay WWTW

The WWTW treats the incoming waste stream by means of waste stabilisation ponds. The treatment facility commissioning date is unknown, but the ponds were upgrade in 1991. The treatment works has a capacity of 0.3 Ml/day (estimated) and is intended to be upgraded to 1.0 Ml/day in future (funding dependent). The treatment works currently has sufficient capacity to treat the wastewater.

The WWTW consists of the following main components:

- Inlet works
- 2 x Anaerobic ponds
- 2 x Primary facultative pond
- 1 x Secondary facultative pond
- 3 x Maturation ponds
- 1 x Infiltration pond
- Chlorine contact tank

Ponds are de-sludged twice a year. The sludge is sun dried and transported to Gwaing WWTW.

6.1.8.4 Beneficiation Option 1: Producing Fertilizer

The preferred option for disposal of sludge is to produce fertilizer from it. Solar dried sludge (>80% DS) granules are optionally mixed with chemical fertilizers and sold to farmers for application to agricultural land. This option creates a high-value product that warrants the additional capital and operational expenditure required for a solar drying plant. George Municipality is currently busy with a Request for Proposal (RFP) process to ascertain whether private industry would be interested in using the sludge for fertilizer, composting or other beneficiation projects.

If there are offtake agreements established with private companies an important consideration would be to establish suitable battery limits. George Municipality can decide for example between these options, although more variations will be possible:

- The Service Provider operates the beltpress dewatering facility, solar drying plant including all sludge handling and transport.
- George Municipality operates the beltpress dewatering facility and solar drying plant and sells dried sludge to the Service Provider.
- George Municipality operates the beltpress dewatering facility and a service provider operates the solar drying plant. This is the preferred option for George Municipality.

The current intention is for George Municipality to construct a solar drying and granulation plant. This will be referred to as the George Biosolids Beneficiation Facility, or Gwaing BBF. George Municipality plans to construct the capital infrastructure and only outsource operation of the facility, including the selling of the granulated sludge as fertilizer.

6.1.8.5 Beneficiation Option 2: Composting

Composting could be employed to sterilize the sludge to a class A1a sludge. If this is achieved the sludge can be sold as compost for agriculture or horticulture use, reducing the need for sludge storage or landfill application. *Delta Built Environmental Consultants* were appointed to investigate the feasibility of composting as a sludge beneficiation strategy for George Municipality at the newly implemented Municipal Composting Facility. Their Report titled: *Sludge Utilisation Within George Municipality Compost Facilities Recommendations Report* is currently in draft format.

The following is an extract from this report:

Municipal sewage sludge is generated in large quantities and contains high organic loads. To reduce its environmental risk, composting is a common treatment technique. Key parameters include moisture content and C/N ratio. Sludge variability makes it challenging to establish treatment protocols, but minimum requirements are necessary for proper composting.

6.1.8.5.1 Sludge Composting Process

The following outlines the main activities associated with composting of sewage sludge:

- Aerobic digestion is the key principle behind sludge composting.
- Sewage sludge can be combined with other waste materials such as wood chips, straw, or green wastes before composting.
- The mixture undergoes natural mesophilic (moderate temperature) and thermophilic (high temperature) aerobic degradation within a largely static system.
- The composting process is low in energy demand as it relies on natural diffusion for aeration.
- Approximately 20-30% of the volatile solids in the sludge can be converted to carbon dioxide during composting.
- The process can take around six months, but will be influenced by climatic conditions like rainfall, humidity, and temperature.
- The resulting composted product can be used for soil conditioning or other land applications, provided the metals content is sufficiently low.

6.1.8.5.2 Windrow Composting

The most common method for sludge composting is by using windrows. The following aspects related to windrow composting should be taken into consideration:

- Windrows are simple piles of material, with cross-section dimensions of up to approximately 2 meters deep, and 4 meters wide.
- Windrows are periodically mechanically turned to ensure even distribution of organic materials and adequate contact with air.
- To facilitate degradation and destroy pathogenic microorganisms, the process temperature must be maintained between 55°C and 65°C, with a moisture content of 35-65%.
- The composting process proceeds through stages including (i) preprocessing, (ii) high-rate decomposition, (iii) recovery of bulking agents, (Iv) curing, and (v) post-processing (screening or grinding).

6.1.8.5.3 Materials For Composting:

Materials combined with sludge prior to composting fall into two categories:

- Bulking agents: These (such as wood chips, or shredded leaves) support the structure of the sludge by increasing its porosity for effective aeration. It also serves to reduce the moisture content of sewage sludge.
- Amendments: Examples include sawdust, straw, rice hulls, or recycled compost, which primarily increase the organic content and enhance biodegradability.

6.1.8.5.4 Factors That Affect The Composting Process:

The physical, chemical, and thermodynamic characteristics of the starting material determine the composting evolution, the process efficiency, and the compost quality. Some parameters, as described below, must be considered before sewage sludge composting is undertaken:

- Particle size and free air space (FAS)
- Moisture
- C/N ratio
- Temperature
- Oxygen
- Bulking agent proportions

6.1.8.5.5 Gwaing Compost Facility Capacity

The capacity of the existing Gwaing Compost Facility was reviewed. Based on the windrow area available and using a 2:1 (bulking agent volume : sludge volume) and using a composting period of 60 days, it was found that Platform A has capacity for about 30% of the combined dewatered WAS sludge from Gwaing WWTW and Outeniqua WWTW. When platform B is completed in 2027, the two platforms combined will have sufficient capacity for 84% of the sludge from Gwaing WWTW and Outeniqua WWTW (Refer to Table 6-12). This is a significant portion of the sludge, but from the onset, there should be planning to expand the composting facility, find additional composting facilities or look at alternative sludge utilisation options in addition to composting.

6.1.8.5.6 Potential Compost End Users

Overall, the use of sludge by private composting facilities was not well received. This is due to their target market being domestic end users and the possible health risks that are perceived with sludge. They were slightly more willing to consider receiving final compost that includes sludge, but still sceptical to resell this to customers.

If packaged properly and at a very low cost, or free of charge, the agricultural sector in and around GM would be interested in utilising dewatered sludge from the WWTWs. The use of sludge within the agricultural sector is also promising due to farmers being able to make use of the sludge on their fields. There are cases of farmers collecting dewatered sludge from WWTWs when full truckloads are available and then making use of the sludge, thus allowing for almost no sludge needing to be disposed of.

The recommendation is that sludge of a quality A1a be produced in the Gwaing composting facility. The facility will have adequate volumes to make use of significant amounts of sludge to be incorporated into the compost. The facility currently has the capacity to take 30% of the sludge produced with platform A, once platform B is complete they should be able to take roughly 84% of the sludge produced. The use of sludge within the agricultural sector is also promising due to farmers being able to make use of the sludge on their fields. With in-depth interaction from GM and the agricultural sector around the WWTWs, there should be significant interest from farmers to make use of the sludge if the quality is A1a.

Table 6-12: Sludge volumes from Gwaing WWTW and Outeniqua WWTW combined and the potential % utilisation at the Gwaing Composting Facility at Platform A (Existing) and Platform B (online 2027) combined.

YEAR	SLUDGE VOLUME			GWAING COMPOST FACILITY	% UTILISED
	LOW	MEDIUM	HIGH	PLATFORM A + PLATFORM B (2026)	
2023	58	58	58	17.5	30%
2024	59	60	60	17.5	29%
2025	60	62	62	17.5	28%
2026	61	64	65	17.5	27%
2027	62	66	67	56.5	84%
2028	63	68	70	56.5	81%
2029	64	70	73	56.5	78%
2030	65	72	75	56.5	75%
2031	66	75	78	56.5	72%
2032	67	77	81	56.5	70%
2033	68	79	84	56.5	67%
2034	70	82	87	56.5	65%
2035	71	84	91	56.5	62%
2036	72	97	94	56.5	60%
2037	73	90	98	56.5	58%
2038	74	92	101	56.5	56%
2039	75	95	105	56.5	54%
2040	77	98	109	56.5	52%
2041	78	101	113	56.5	50%
2042	79	104	118	56.5	48%
2043	80	107	122	56.5	46%
2044	82	111	127	56.5	45%
2045	83	114	132	56.5	43%

6.1.8.5.7 Composting Option Conclusion

Presently the decision is not to pursue composting as a direct option for the beneficiation of the Gwaing WWTW sludge. However, with the implementation of a solar drying facility that achieves a class A1a sludge, the dried sludge will be more palatable for composting plants and end users and it is foreseen that the sludge could be sold or given to these facilities as an alternative option to fertilizer production.

6.1.8.6 Sludge Storage

Regardless of the sludge beneficiation option chosen by GM, there may well be a need for the temporary storage/stockpiling of sludge. Such a storage facility would be valuable if the composting facility is not able to receive sludge for a period. If solar drying is employed, the drying rate is much lower in winter and therefore it may be sensible to store a portion of the sludge during winter so that it can be dried in summer when higher drying rates are achievable.

Due to the high rainfall in George, it is advisable to cover the sludge storage area to prevent rainwater ingress. By making the covers translucent, some consequential solar drying will also take place in the stockpiles. The bunded areas must include impermeable floors and contained stormwater retention so that nutrient-rich runoff does not enter the maturation ponds or stormwater networks. Sludge must be easily transportable by means of a TLB or similar. Figure 6-15 shows an example of typical storage bunds that may be viable for Gwaing WWTW.

Figure 6-15: An Example of sludge storage bunds with concrete floors and translucent roof covers.

6.1.8.7 Solar Drying

Solar drying of sewage sludge is typically done after initial dewatering to 14% - 17% dry solids (DS). Solar drying can be done to achieve between 65% and 90% DS. Above 65% DS the sludge forms granules or powder and is not lumpy or sticky any longer. The drying process reduces pathogens and faecal coliforms. A microbiological class of A could potentially be achieved to reach an overall sludge classification of A1a. However, it should be noted that temperature has been found to be the main parameter in the removal of helminth eggs and therefore the achievement of A1a may be dependent on the temperatures reached during the solar drying process. Stockpiling and curing of the sludge after drying has also been effective for pathogen reduction.

Another benefit of solar drying is the reduction of moisture content, leading to the reduction of mass and volume of the sludge. This reduces transport costs and simplifies sludge handling.

During detail design, a solar assessment is required at Gwaing WWTW to determine what irradiance and temperature and resulting drying rates can be expected during every season. It may be worthwhile to consider a pilot-scale solar drying plant to obtain as much as possible data prior to finalising the design of the complete solution.

George experiences a horizontal solar irradiation of 1500-1850 kWh/m^2 per annum. This is low compared to other parts of South Africa (see Figure 6-16), but relatively high when compared to central and northern Europe for example. George experiences a subtropical oceanic climate, characterized by mild winters and warm summers with monthly mean temperatures ranging from 12°C to 22°C. The annual precipitation is about 715 mm, spread quite evenly over the year so that monthly averages range from 45 to 70 mm per month.

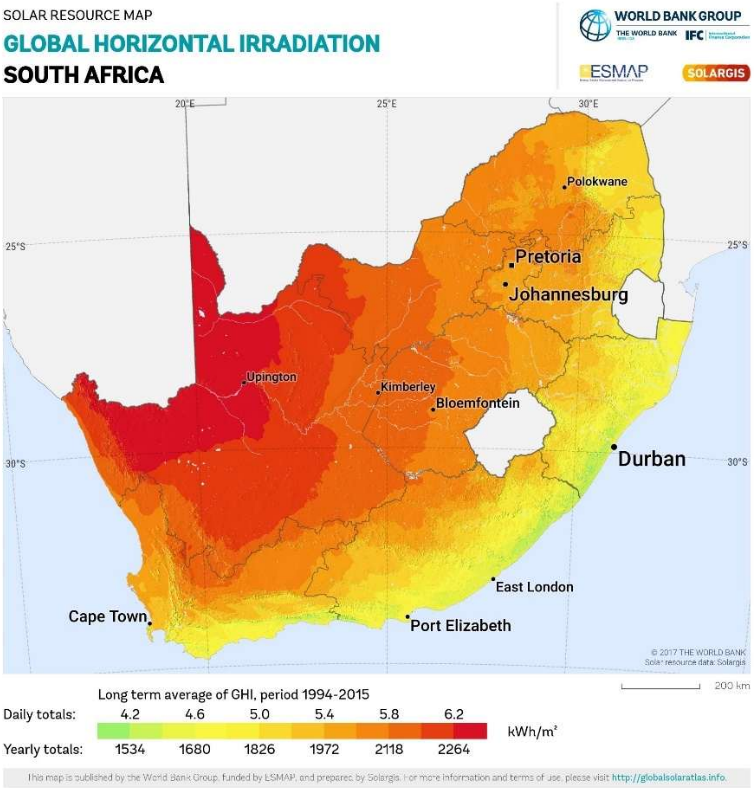


Figure 6-16: Horizontal solar irradiation map of South Africa.

The weather station nearest to the Gwaing WWTW was used to determine the weather conditions for 2024 as shown in Figure 6-17.

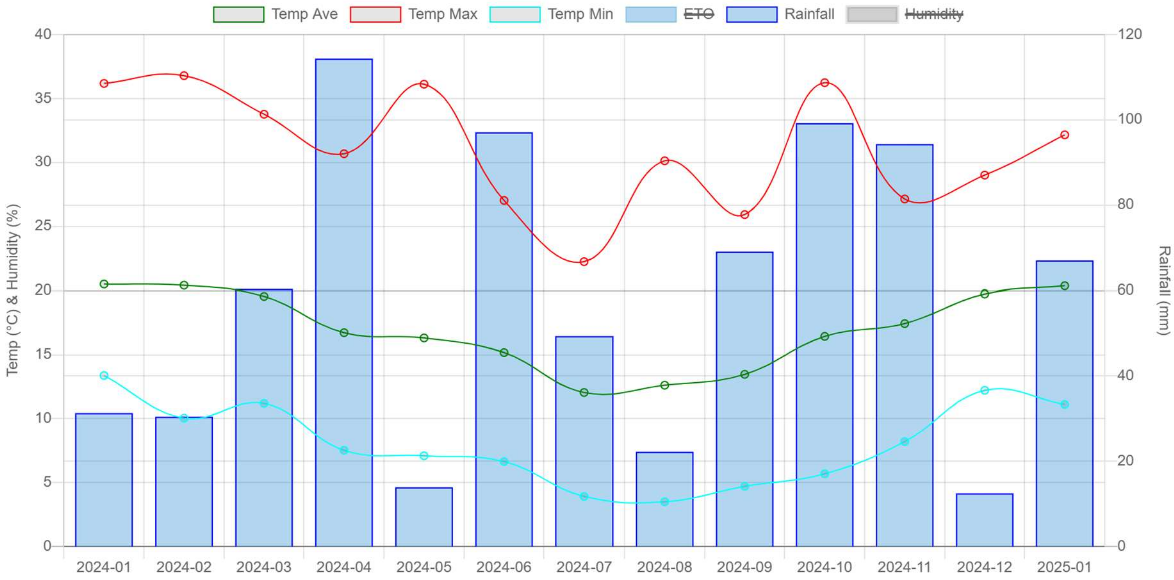


Figure 6-17: Gwaing WWTW Weather conditions for 2024 as obtained from gis.elsenburg.com/apps/wsp/

6.1.8.7.1 Requirement for Translucent Roof Sheeting

Solar drying can be done with or without roof coverings. Figure 6-18 shows an aerial view of a solar drying plant at Rooiwal WWTW north of Pretoria. The precipitation here is lower (600 mm per annum, summer rainfall) and the solar irradiance is higher than in George. The evaporation rates are also higher than in George. This makes it feasible to operate the drying facility without any roof structure. Simple concrete slabs with allowance for drainage are sufficient, with mechanical plant used to spread and turn the sludge periodically.

Figure 6-19 shows a solution often employed in colder climates. This includes translucent roof sheeting, forced ventilation and automated sludge spreading and turning. It seems apparent at this stage that translucent roof sheeting may be required for a solar drying plant at Gwaing WWTW to limit the footprint required to within reasonable limits. Different options for sludge spreading and turning can be considered. This approach results in a drastic reduction of processing time or footprint and produces a better quality sludge.

Figure 6-18: Aerial view of the solar drying facility at Rooiwal WWTW north of Pretoria.

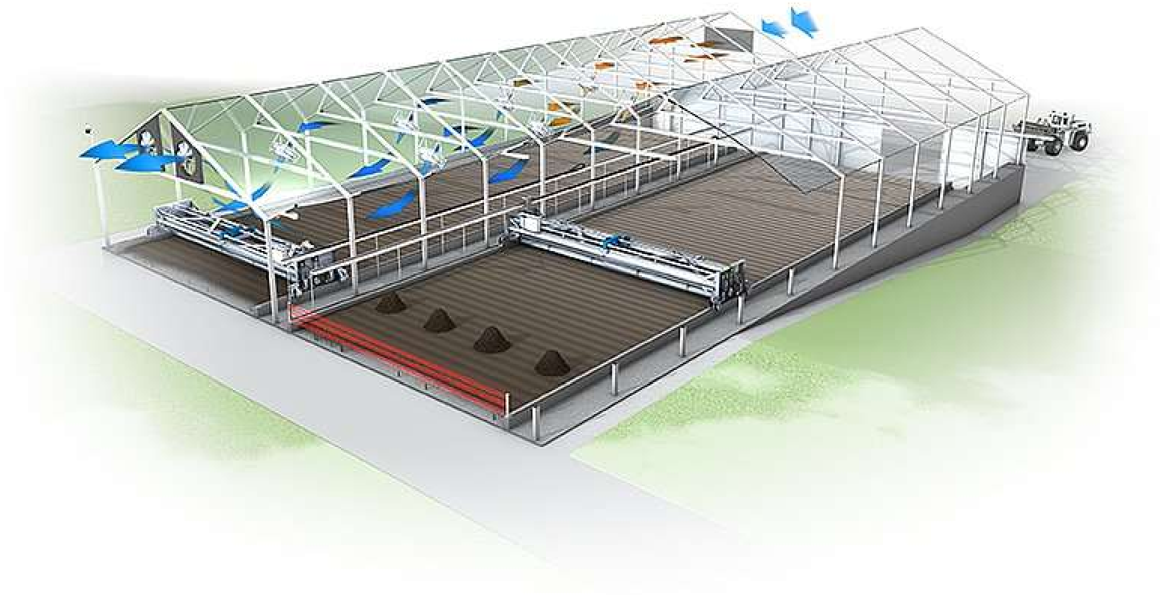


Figure 6-19: Example of advanced solar drying facility including translucent roof sheeting, forced ventilation and a sludge turner and spreader (Huber).

6.1.8.7.2 Continuous drying vs. batch drying

Continuous solar sludge drying and batch solar sludge drying are two distinct methods used for reducing the moisture content of sludge using solar energy. Continuous solar sludge drying involves a steady, ongoing process where sludge is continuously fed into the drying system, typically spread in thin layers within a greenhouse structure. This method ensures uniform drying through regular agitation and optimal air circulation, leading to efficient moisture evaporation and consistent output quality. An example of a continuous drying process is seen in Figure 6-19 where a Huber sludge turning traveling bridge continuously turns the sludge and gradually transports it from the inlet side of the greenhouse structure to the outlet side. Wendewolf and Thermo-Systems are other prominent suppliers who offer similar sludge drying travelling brides as part of a continuous drying system.

In contrast, batch solar sludge drying processes sludge in discrete batches, where each batch is dried separately before the next one begins. This method can be less efficient due to the downtime between batches and potential inconsistencies in drying conditions. However, batch drying allows for greater control over individual batches, which can be beneficial for handling varying sludge characteristics. Overall, continuous solar sludge drying is generally more suitable for large-scale operations where automated sludge feeding with conveyor belts are utilised, while batch solar sludge drying offers flexibility and control for smaller or more variable sludge volumes. An example of a batch system is the SolarBatch electric mole system from Thermo Systems. For Gwaing WWTW this would typically entail that a greenhouse train is loaded to capacity with sludge with front-end loaders, and then the sludge is left to dry in that train for about 28 days while continuously being rotated by the electric mole.

Figure 6-20: Electric mole as part of the SolarBatch system by Thermo-Systems.

Figure 6-21 shows a comparison table of continuous vs batch solar drying plants.

Figure 6-21: Comparison of continuous vs batch solar drying plants.

Aspect	Continuous Solar Sludge Drying	Batch Solar Sludge Drying (Electric Mole)
Initial DS Content (min-max)	15% - 30%	2% - 30%
Filling Height (min-max)	10 - 40 cm	5 - 30 cm
Storage of winter sludge	Possible in greenhouse structure by increasing sludge depth	Separate storage area required
Processing of Pasty Sludges	Remixing required; problems expected	No restrictions; smooth processing
Access	Single side access possible, but limited transport capacity	One or both sides
Tolerance of Floor Unevenness	Low (bridge-type turning device cannot adapt)	High (Electric Mole can adapt)
Tolerances for Walls	Low (turning device needs to drive on mural crown)	High (Electric Mole does not drive on walls)
Width of Drying Chambers	Up to 12 m	Up to 20 m
Automatic Feeding	Possible (but expensive due to additional devices)	Possible with smaller plant concepts (StorageDryer)
Automatic Extraction	Possible (but expensive due to additional devices)	Not possible (only by wheel loader)
Pathogen Reduction	Medium (Recontamination of dry sludge by travelling bridge to be carefully managed.)	High (Class A can be reached)
Removal of Turning Device for Maintenance	Possible, but complicated	Easily possible
Machine Failure Impact	Leads to process stop	Five days with minimal impact since mole is not used for transporting sludge.
Maintenance Costs	Medium	Low

Aspect	**Continuous Solar Sludge Drying**	**Batch Solar Sludge Drying (Electric Mole)**
Operation Costs	Low	Low
Total Cost per t H_2O Evap.	Medium	Low
Overall Complexity	Medium	Low
Flexibility in Dimensioning	Low	High
"Dead Zones" on Drying Surface	Yes; on both ends and sides of drying chamber	No; whole drying surface can be reached
Total Surface vs. Drying Surface	Higher surface requirement for concrete works & steel structure	Maximum of total surface used as drying surface
Turning Performance	High	High
Risk of Strong Odor Formation	Low due to back-mixing	Medium but manageable via turning and ventilation rate
Efficiency	High, due to consistent operation and minimal downtime	Lower, due to downtime between batches
Control	Less control over individual batches	Greater control over each batch
Scalability	Suitable for large-scale operations	More suitable for smaller or variable sludge volumes. Also suitable for large scale.
Drying Uniformity	High	High
Operational Complexity	Requires continuous monitoring and maintenance	Simpler operation with clear start and end points
Flexibility	Less flexible, designed for continuous input	More flexible, can handle varying sludge characteristics
Output Quality	Consistent output quality	Variable output quality depending on batch conditions

Both options are feasible with several international reference plants available. The batch plant seems better suited for the Gwaing BBF for the following reasons:

- Lower capital cost
- Operational simplicity

- Automatic feeding is not an option since the Gwaing WWTW dewatering plant is too far away from the solar drying plant. Batch feeding with front end loaders more suitable.
- The process has higher resistance to cross-contamination of dried sludge by raw sludge
- The process is more forgiving if the sludge from the dewatering plant is too wet, whereas the continuous process would prefer >17%DS, which would be difficult to achieve with only WAS at the beltpresses.

It is an option to do sludge turning in the greenhouse structures with more rudimentary mechanical equipment such as a tractor pulling a rotovator or mechanical broom. The benefit of this option is the reduced capital costs, but the disadvantageous are as follows:

1. **Labor-Intensive**: Manual operation requires significant human effort, which can be physically demanding and time-consuming. This can lead to increased labour costs and potential worker fatigue

2. **Inconsistent Results**: Manual rotation may not achieve the uniformity and consistency needed for effective sludge drying. Variations in the rotation process can result in uneven drying, leading to areas of sludge that are either too wet or over-dried

3. **Operational Inefficiency**: Manual processes are generally slower and less efficient compared to automated systems. This can reduce the overall throughput of the drying operation, making it less suitable for large-scale applications

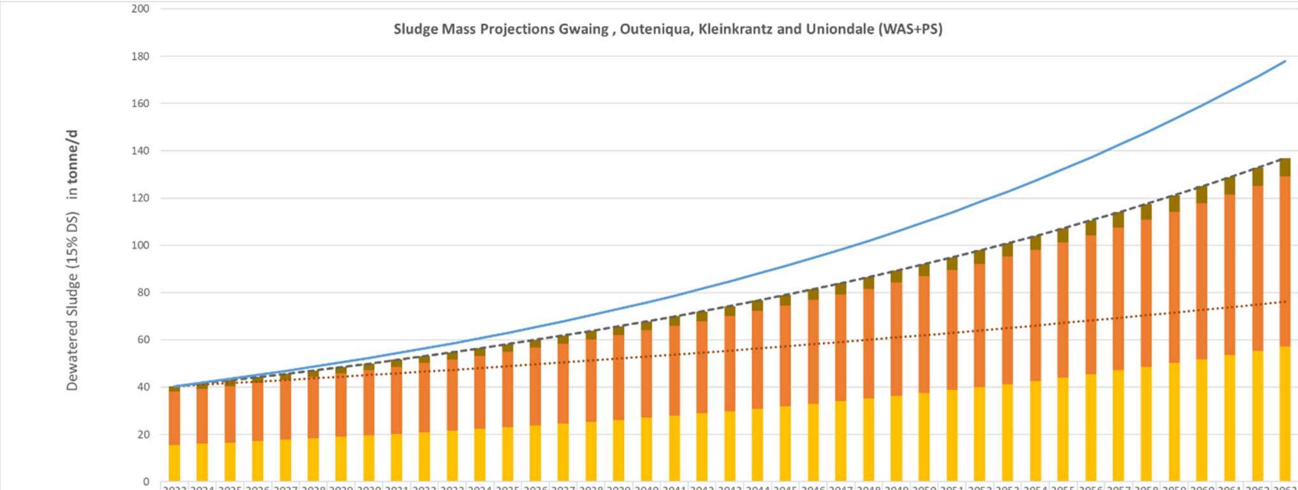
4. **Health and Safety Risks**: Working with rotovators and handling sludge manually can pose health and safety risks to workers, including exposure to pathogens and physical injuries from operating machinery. The temperature in the greenhouse structures can reach 55°C, meaning that plant operators would rely on tractor air-conditioning for safe operating conditions.

5. **Environmental Control**: Greenhouse structures require precise environmental control to optimize drying conditions. Manual rotation can disrupt the controlled environment, leading to fluctuations in temperature and humidity that can negatively impact the drying process

Overall, while manual sludge rotation with rotovators might be feasible for small-scale operations, it is generally less efficient, more labour-intensive, and potentially more hazardous compared to automated systems. A more detailed comparison will be done during the detail design phase.

6.1.8.7.3 Solar Drying Sludge Volumes

The sludge volumes to be received at the BBF solar drying facility are shown in Figure 6-22**. Note that the sludge volume is less than that shown in Section 6.1.7**. Some conservative factors in the process design were removed so that the sludge quantities expected at the BBF are not over-estimated.

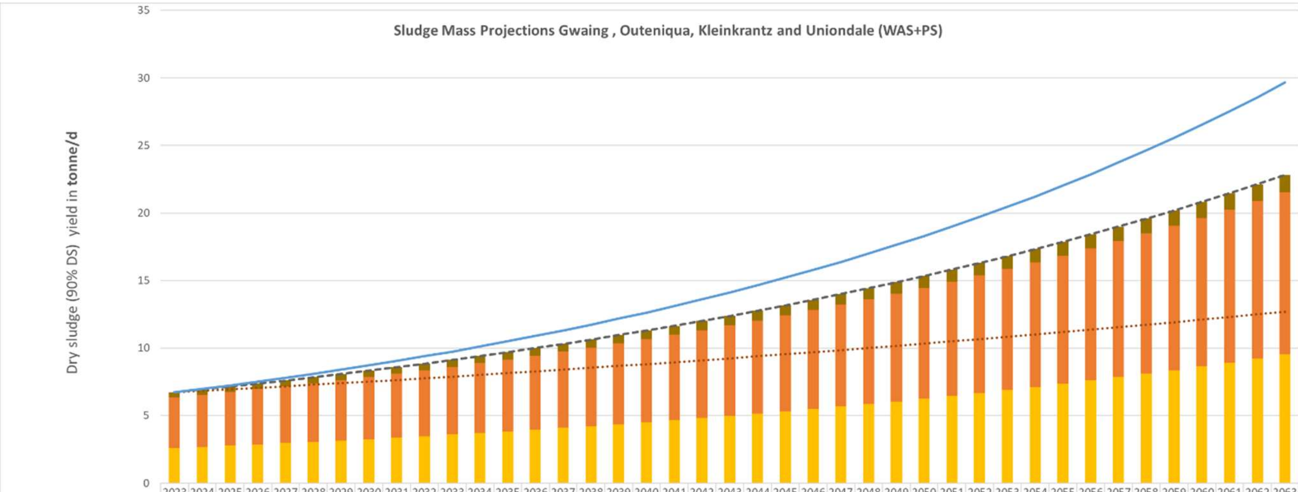


	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043
Uniondale & kleinkranz	2	2	2	3	3	3	3	3	3	3	3	3	3	3	4	4	4	4	4	4	4
Outeniqua WWTW	23	23	24	25	25	26	27	28	28	29	30	31	32	33	34	35	36	37	38	39	40
Gwaing WWTW	16	16	17	17	18	18	19	20	20	21	22	22	23	24	25	25	26	27	28	29	30
Low [1.6%]	40	41	42	42	43	44	44	45	46	47	47	48	49	50	50	51	52	53	54	55	55
Med [3%-3.3%]	40	42	43	44	46	47	48	50	52	53	55	56	58	60	62	64	66	68	70	72	74
High [3.6%-4%]	40	42	43	45	47	49	50	52	54	56	58	61	63	65	68	70	73	76	79	82	85

	2044	2045	2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063
Uniondale & kleinkranz	4	4	5	5	5	5	5	5	6	6	6	6	6	6	7	7	7	7	8	8
Outeniqua WWTW	41	43	44	45	47	48	49	51	52	54	55	57	59	60	62	64	66	68	70	72
Gwaing WWTW	31	32	33	34	35	36	37	39	40	41	43	44	46	47	49	50	52	54	55	57
Low [1.6%]	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	72	73	74	75	76
Med [3%-3.3%]	77	79	81	84	87	89	92	95	98	101	104	107	111	114	117	121	125	129	133	137
High [3.6%-4%]	88	91	95	98	102	106	110	114	118	123	127	132	137	142	148	153	159	165	171	178

Figure 6-22 Dewatered sludge (at 15% DS) mass projections for George Municipal WWTW's combined current and future projections

The future projected combined sludge mass (as dried sludge from the Gwaing BBF) is shown in Figure 6-23. Note that future projections are estimations only ranging from low (1.6%) to high (3.6 – 4%) population growth scenarios.



	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043
Uniondale & kleinkranz	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	1	1	1	1	1	1
Outeniqua WWTW	4	4	4	4	4	4	4	5	5	5	5	5	5	5	6	6	6	6	6	7	7
Gwaing WWTW	3	3	3	3	3	3	3	3	3	3	4	4	4	4	4	4	4	5	5	5	5
Low [1.6%]	7	7	7	7	7	7	7	8	8	8	8	8	8	8	8	9	9	9	9	9	9
Med [3%-3.3%]	7	7	7	7	8	8	8	8	9	9	9	9	10	10	10	11	11	11	12	12	12
High [3.6%-4%]	7	7	7	8	8	8	8	9	9	9	10	10	10	11	11	12	12	13	13	14	14

	2044	2045	2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063
Uniondale & kleinkranz	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Outeniqua WWTW	7	7	7	8	8	8	8	8	9	9	9	10	10	10	10	11	11	11	12	12
Gwaing WWTW	5	5	5	6	6	6	6	6	7	7	7	7	8	8	8	8	9	9	9	10
Low [1.6%]	9	10	10	10	10	10	10	10	11	11	11	11	11	12	12	12	12	12	13	13
Med [3%-3.3%]	13	13	14	14	14	15	15	16	16	17	17	18	18	19	20	20	21	21	22	23
High [3.6%-4%]	15	15	16	16	17	18	18	19	20	20	21	22	23	24	25	26	27	28	29	30

Figure 6-23: Dry sludge (90% DS) mass projections for George WWTW's combined current and future projections

It is proposed that the facility be sized initially to receive approximately 50 tonne/d at 15% DS which will result in a dried mass of about 8.3 tonne/d at 90% DS. Additional drying trains can be added in future in line with the realized population rates. The capacity of the BBF should be sufficient until at least 2030, depending on the population growth rate.

6.1.8.8 BBF Infrastructure Layout

The BBF process comprises primarily of the following steps:

i. Receiving dewatered sludge from the WWTW with front end loaders, skips or similar.
ii. During winter when the temperatures and solar radiation is lower and the drying capacity of the plant is reduced, excess sludge will be stockpiled in bunds. (Note this will be done if a batch system is used as opposed to a continuous drying system). During summer the bunds will gradually be emptied as the drying capacity increases.
iii. Loading of the solar drying trains with front end loaders, approximately one train every 3rd day.
iv. Solar drying of the sludge while sludge is continuously being turned and spread by an electric mole or similar equipment. This process will take approximately 30 days.
v. Removing of the dried sludge with front end loaders, approximately one train every 3rd day.
vi. Stockpiling the dried sludge in curing bunds for 6-8 weeks to get additional pathogen removal in order to obtain class A1a sludge.
vii. The dried sludge is taken to a granulation plant where it is granulated to a size suitable for agricultural applications.
viii. After granulation the product is coated and packaged before being transported to an off-site fertilizer production facility.

The infrastructure required for the Gwaing BBF facility can be summarized as follows:

i. Guard House
ii. Perimeter fencing and access gate
iii. Approximately 30 000 m^2 of concrete slabs for the various stages of sludge stockpiling, solar drying, composing and sludge handling. This includes the areas under translucent roof sheeting for solar drying.
iv. Approximately 13 000 m^2 in plan view of translucent roof sheeting ('greenhouse') structures.
v. One 18m x 36m shed with a clear height of 4.5m and without any columns inside the building for the sludge granulation plant.
vi. A second building of similar footprint for the packaging plant and distribution depot. This building is to include offices, ablution and a canteen for the operating staff of approximately 6 people.
vii. Movable precast concrete walls placed on slabs to demarcate separated process areas and to prevent contamination of treated sludge by raw sludge.
viii. Access Roads
ix. Rainwater collection and storage from all roof structures
x. Stormwater collection and drainage from concrete slabs with pipeline to Gwaing WWTW inlet works.

The schematic layout in Figure 6-24 shows some of the key infrastructure components and the basic process flow.

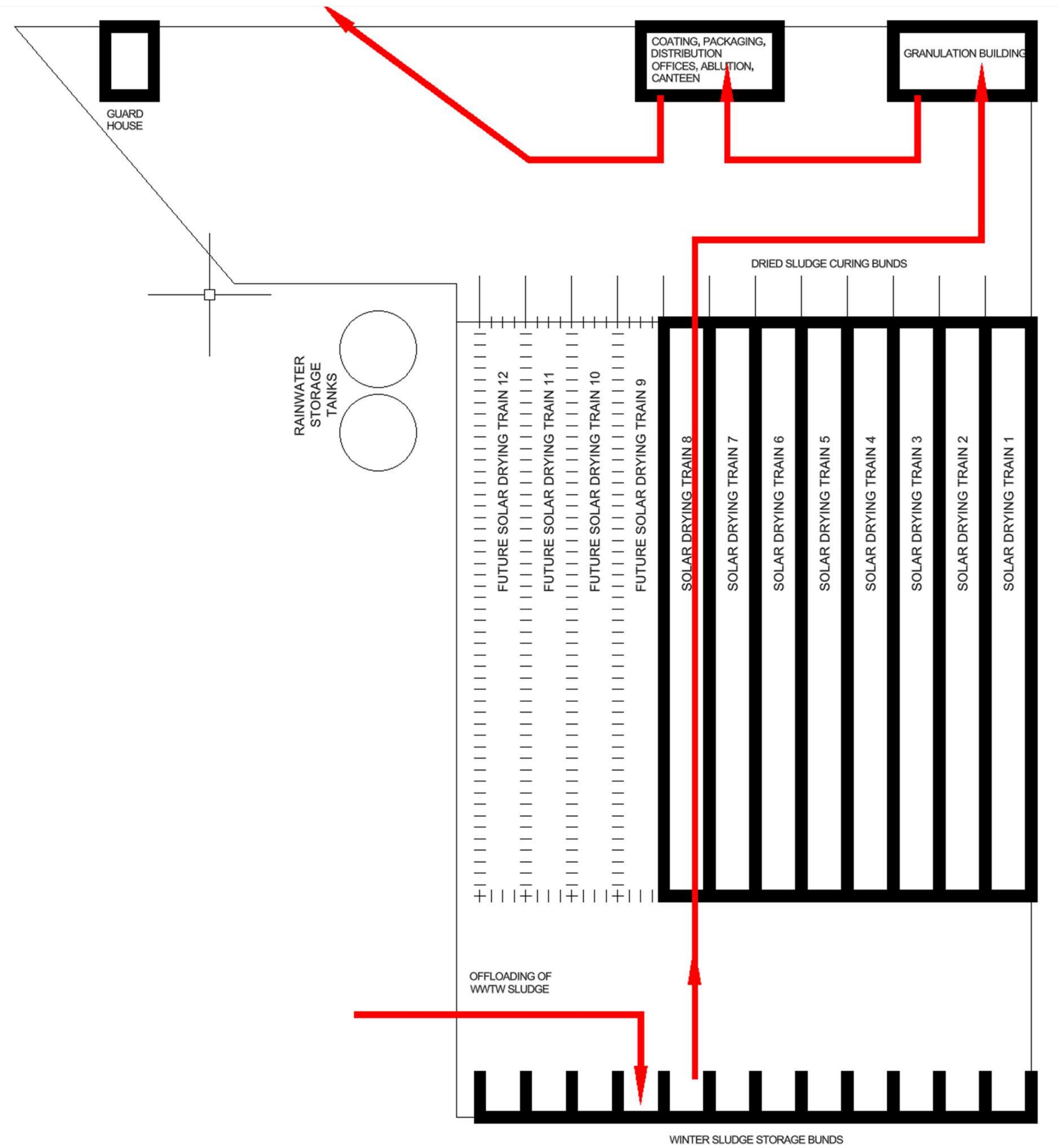


Figure 6-24: Gwaing BBF schematic layout with basic process flow.

The trains can have a width ranging between 11m and 20m. Factors that influence the chosen width are:

i. The weight of the translucent sheeting. Glass is heavier than polycarbonate sheeting and therefore may require a shorter span.
ii. The sludge turning equipment – travelling bridges from different suppliers come in specific sizes. An electric mole can operate over a wider range of widths.
iii. The design of the steel structure.

The trains can be up to 150m long. The main limitation in the length is the electrical equipment required for the travelling bridges or moles when moving up and down the train.

The height of the structures is governed by the size of the front-end loader that loads and unloads the trains.

The layout of the BBF is shown with reference to the WWTW and how it fits onto erven 57, 59, 61 and 63 of the proposed Gwayang development.

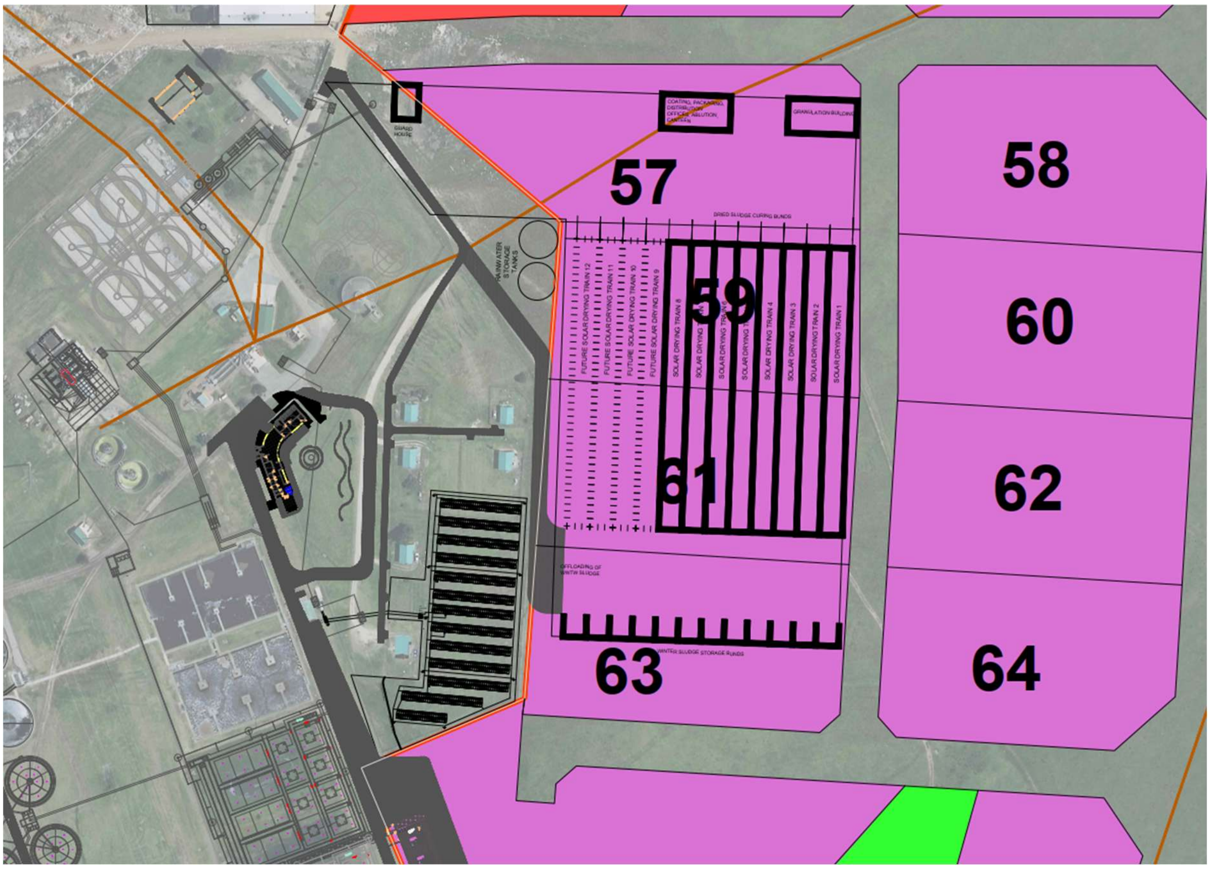


Figure 6-25: Proposed position for the BBF site.

6.1.8.9 Stormwater Management

It is foreseen that the BBF will have approximately 13 000 m^2 of roof area available. Rainwater harvesting will be done from the translucent roofs. It is foreseen that about 1000 kl of rainwater storage will be provided. This water will be used in the BBF and may be used at the WWTW as washwater at the inlet works or beltpress facility.

The remaining area of approximately 13 000 m^2 will primarily comprise of concrete slabs that will accommodate sludge stockpiles. These slabs will be sloped at approximately 1:200 to open v-drains. The slopes should not be too steep to prevent fluidisation and transport of sludge stockpiles during heavy rainfall. At the perimeter of the slabs kerbs will be provided to ensure that sludge and contaminated stormwater does not flow to the adjacent environment.

A combination of open v-drains and stormwater pipes will collect all the stormwater from the concrete slabs at the south-western corner of the site. This is the lowest point on the site, and the nearest point to the WWTW. The stormwater from the slabs will drain to the inlet works of the Gwaing WWTW. Since the stormwater from the concrete slabs will contain some sludge and organic matter it should not be discharged to a retention pond since it will become eutrophic and may produce a foul smell. The nature of the organic matter discharged to the WWTW will be beneficial to the WWTW process.

Figure 6-26 shows a schematic layout of the stormwater management plan for the Gwaing BBF.

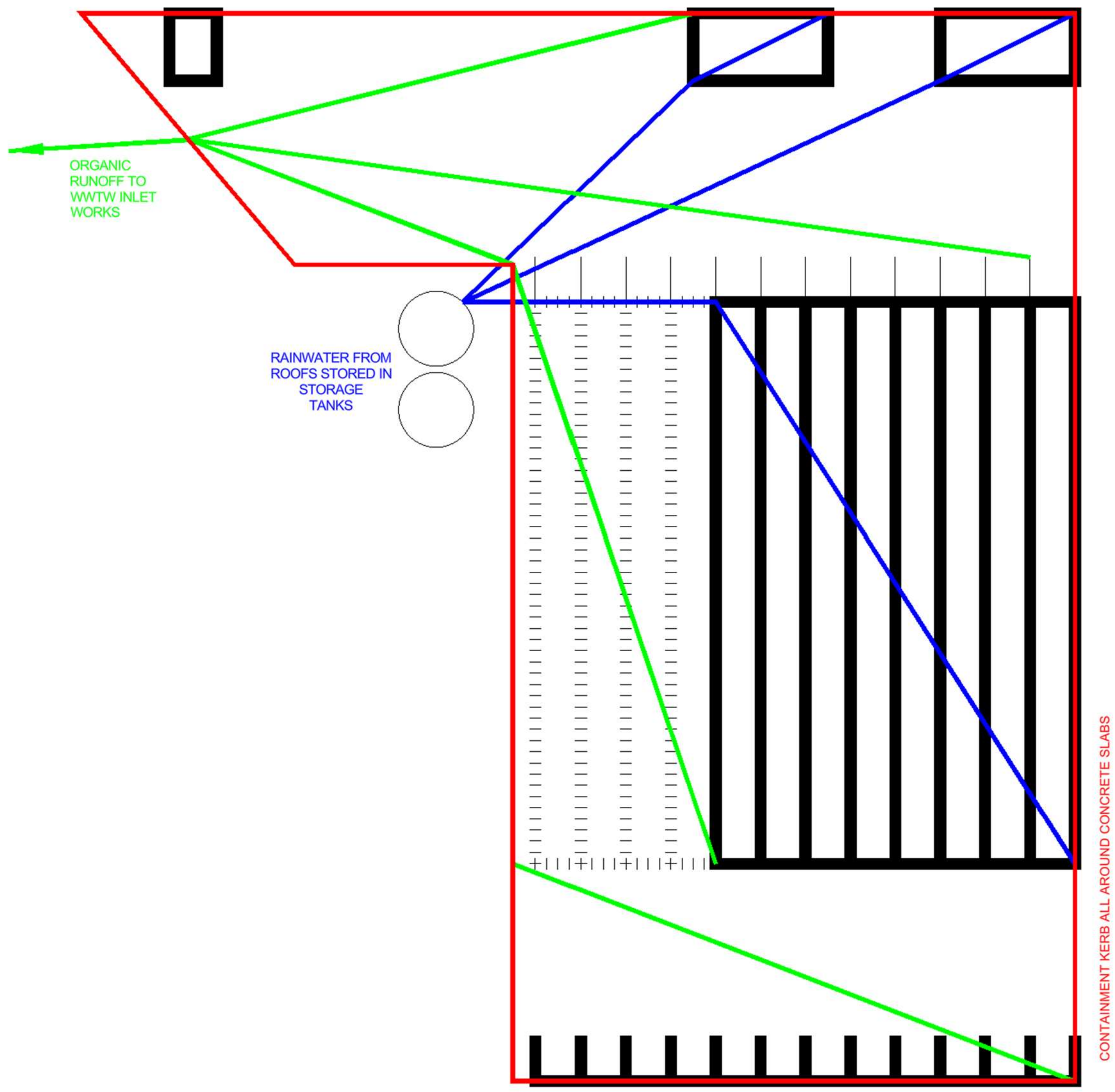


Figure 6-26: Schematic Stormwater Management Plan for Gwaing BBF

6.1.9 Anaerobic Digestion of Primary Sludge

Gwaing WWTW has one existing anaerobic digestor (AD) which is not presently in operation but is scheduled to be recommissioned eventually as part of Gwaing WWTW's ultimate phases. For the ultimate solution, three new ADs are required that will be used in combination with the existing AD. The three new ADs are sized to be similar in size to the existing AD to facilitate the use of similar equipment in all ADs. To reduce the size of the ADs, primary sludge from the primary settling tanks (PSTs) must first be thickened before being pumped into the ADs. It is recommended that the existing PSTs of the decommissioned bio-trickling filter module be used as sludge thickening tanks and equipped with new pumps for pumping thickened sludge to the ADs. Table 6-13 presents a summary of the AD design parameters.

Table 6-13: Summary of the Anaerobic Digestors Design.

Parameter	Value
Retention time (days)	25
Minimum Thickening Achieved	3.0%
Digester Diameter (m)	18.0
Digester Depth (m)	8.50
No. of Digestors	4 (For 50 MLD)

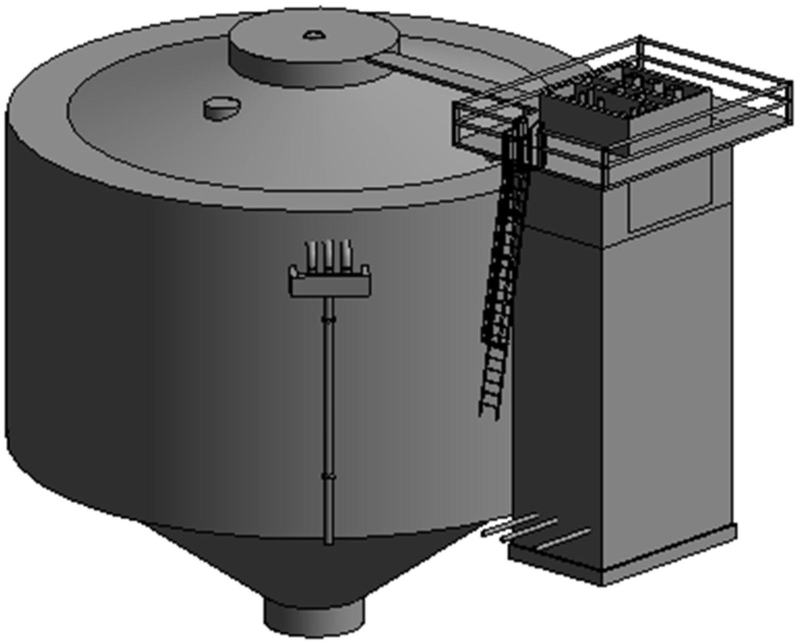


Figure 6-27: 3D Model of existing anaerobic digestor.

To capture the benefits of treating primary sludge anaerobically, a train of equipment is needed to treat and harvest the gas stream from the digestors. The proposed gas treatment train is presented in Figure 6-28. The first step in the train is a dehumidification unit to remove the moisture in the abstracted gas. The second step is the removal of H_2S (gas). Relatively low concentrations of H_2S form in anaerobic digesters and must be removed before the gas can be utilized. In a combined heat and power (CHP) unit, H_2S would form sulphuric acid and SO_2. These gasses are highly corrosive and can contaminate the oil. Operational problems can be prevented with the use of external biological desulphurisation, which is the most cost-effective means of removing H_2S in this case. For the sake of final H_2S removal prior to gas utilization an activated carbon filter needs to be installed to meet the requirements of the CHP unit. As the activated carbon will become saturated with contaminants, it needs to be replaced periodically.

In order to produce electrical energy, a piece of equipment is needed to convert the embodied chemical energy of the biogas into electrical energy. This can be done by combusting the biogas in a combustion engine and using the resulting mechanical energy to drive a generator. As an internal combustion engine produces a lot of heat and heating is required in the digesters, a combined heat and power (CHP) unit is a suitable solution. In case the biogas cannot be utilized (e.g. when the downstream equipment is down), is out of specification (e.g. during start-up) or excess gas is produced, it must be possible to safely dispose of it. Simply venting biogas into the atmosphere would be environmentally unsound, as it is a greenhouse gas, thus it must be combusted in a flare.

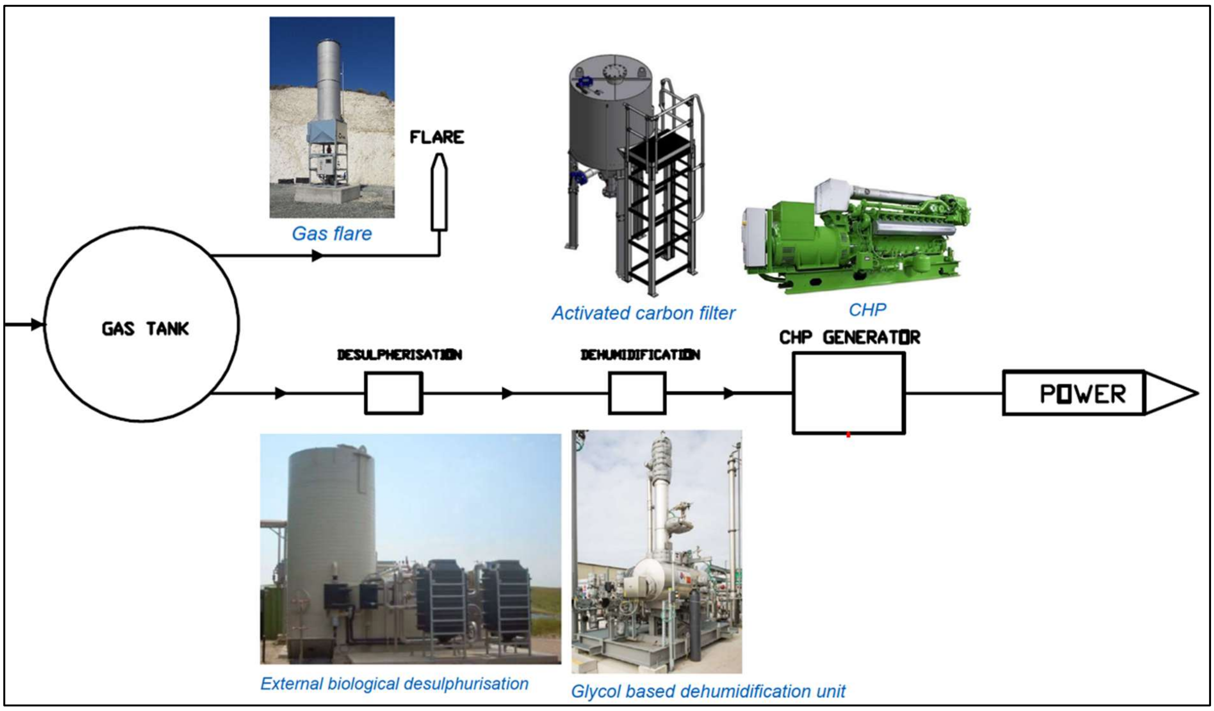


Figure 6-28: Anaerobic Digestion Equipment for the Gas Treatment Stream

As energy generation is not the primary service/function of the George Municipality wastewater treatment division, such a service/function should ideally be outsourced to a specialist service provider.

After anaerobic digestion, digested sludge will need to be abstracted from the digestors and further thickened to target dryness of around 20%. For this dewatering process, a sludge screw press is recommended. A picture of a typical screw press is shown in Figure 6-29. Beltpresses would be viable alternatives. A detailed comparison should be done during detail design phase.

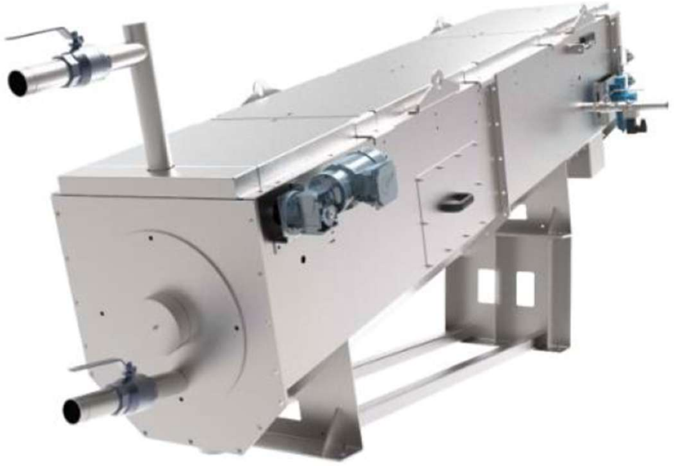


Figure 6-29: Sludge Screw Press for Digested Sludge Treatment

6.1.10 Blower House and Service Corridor

To optimize operational costs, fine bubble diffused aeration (FBDA) is preferred over surface aeration. Consequently, a blower house is necessary to accommodate the blowers. These blowers will supply air through a large air header pipe to the biological reactors. In an effort to minimize air header length and service corridors, the blower house is strategically positioned as close as possible to the reactors. The proposed blower house floor plan and associated 3D rendering are shown in Figure 6-30.

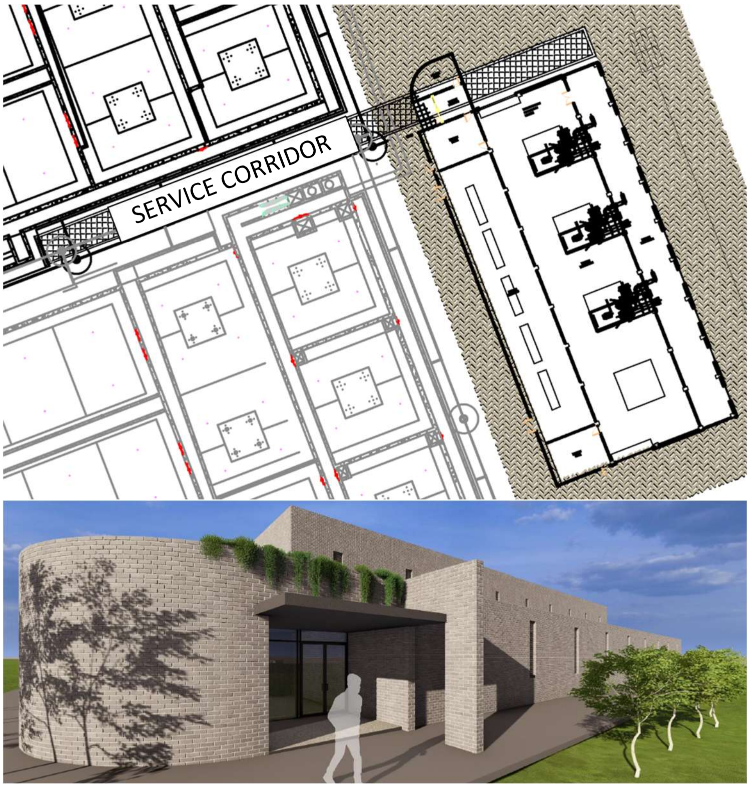


Figure 6-30: Blower house floor plan and associated 3D rendering

The blower house will consist of storerooms, a small entrance/office space, electrical equipment rooms (for the blowers and reactor equipment), the main blower hall and an air intake plenum housing the air intake filters.

6.1.10.1 Mechanical Equipment for Aeration

A summary of the mechanical equipment for the reactor aeration system is provided in Table 6-14. Centrifugal blowers with aeration control are proposed. Aeration control offers an energy-efficient approach to enhance the performance of an aeration process. These controls utilize automated direct flow control, Most-Open-Valve (MOV) technology, and control algorithms, providing advantages over pressure control systems. By utilizing direct flow control with MOV logic, the required system pressure is minimized, resulting in reduced pressure and power demands on the aeration compressor.

Table 6-14: Mechanical Equipment for the Blower Aeration System

Equipment	Description	Duty+Standby (Redundancy)	Operation
Blowers	Centrifugal blower driven by an electric motor through a speed-increasing gearbox. Supplied with inlet guide vane and variable vane diffuser control	2+1	Using aeration control and four control valves
Diffusers	Membrane disc-type diffusers	4 networks, one in each aeration basin	Fine bubbles are created from slots on the EPDM membrane of the disc-type diffuser
Control Valves	Jet control valve	4	Controlled to a set DO level.
Isolation valves	Knife gate valve	10	Manual

A typical blower that will be kept in the blower house is shown in Figure 6-31.

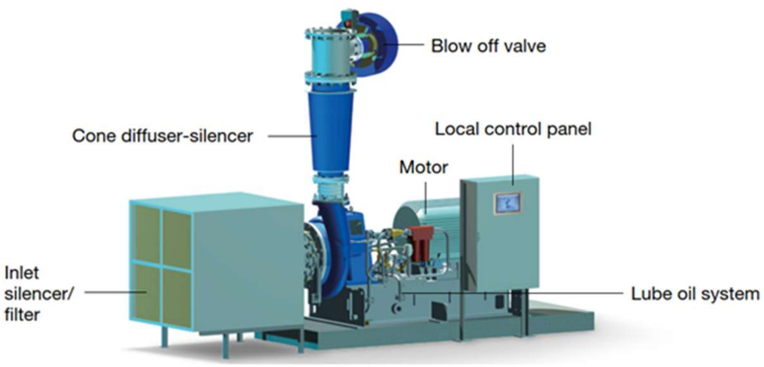


Figure 6-31: Isometric view of a typical centrifugal blower to be housed in the blower house (acoustic hood not shown)

The proposed aeration control system is shown in Figure 6-32. The system will use four control loops to regulate air flow rate and pressure. The first control loop establishes the dissolved oxygen level set-point based on ammonia readings in the reactor. The second loop adjusts the air flow set-point of the air control valves using the deviation from the dissolved oxygen level (DOL) set-point. The third loop sets the main header pressure (or airflow set-point) based on the most open control valve position, while the fourth loop governs the airflow produced by the online blowers.

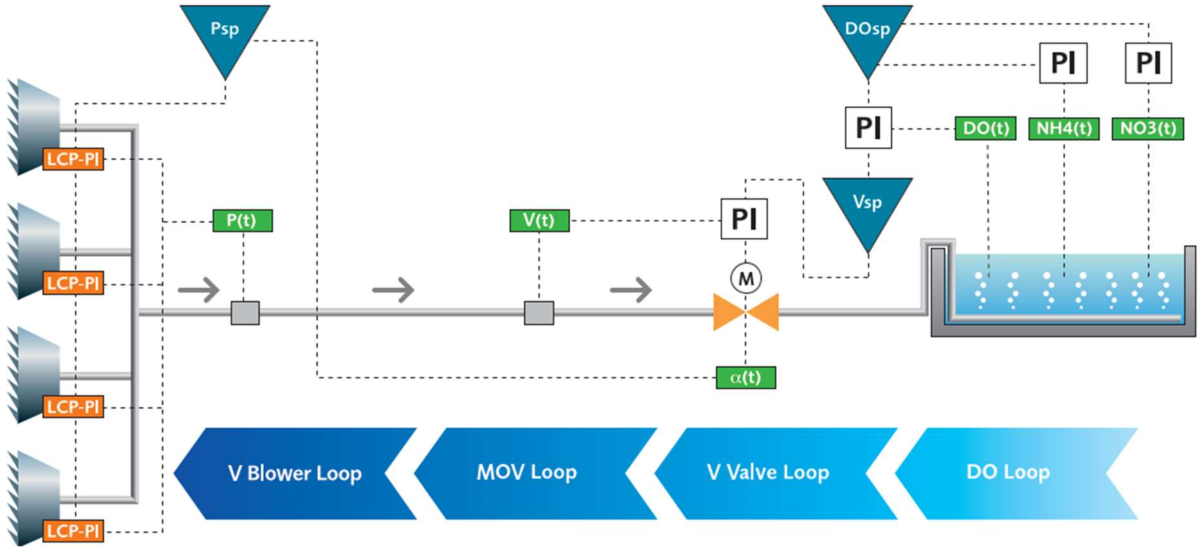


Figure 6-32: Aeration control response system

6.1.11 Wash Water (Effluent) Return

As part of the Gwaing WWTW upgrade, the existing pipeline to the ponds and chlorine contact tank will be rerouted through the new UV disinfection channels. Consequently, the existing pipe leading to the wash water pump station sump will become redundant. The upgrade will introduce additional wash water demands, rendering the existing wash water pump station insufficient in size. As a result of the rerouting of the outlet pipe and the additional demands, a new wash water pump station is proposed together with a wash water pipeline ring main around the site.

During the master plan stage, the existing humus tank underflow pumpstation was identified as an ideal building to be reused as a wash water pump station. Currently, one side of the humus tank underflow pump station is used as a low point to which on-site sewage drains. This sewage gets pumped back to the inlet works.

Final effluent will be directed (through a new pipe) from the new SST outflow system to the old humus underflow pump station suction sump. From there, a new wash water pump set inside the humus underflow pump station will feed a service water ring main to supply the main process units with service water. The proposed location of the new wash water pump station is shown in Figure 6-33.

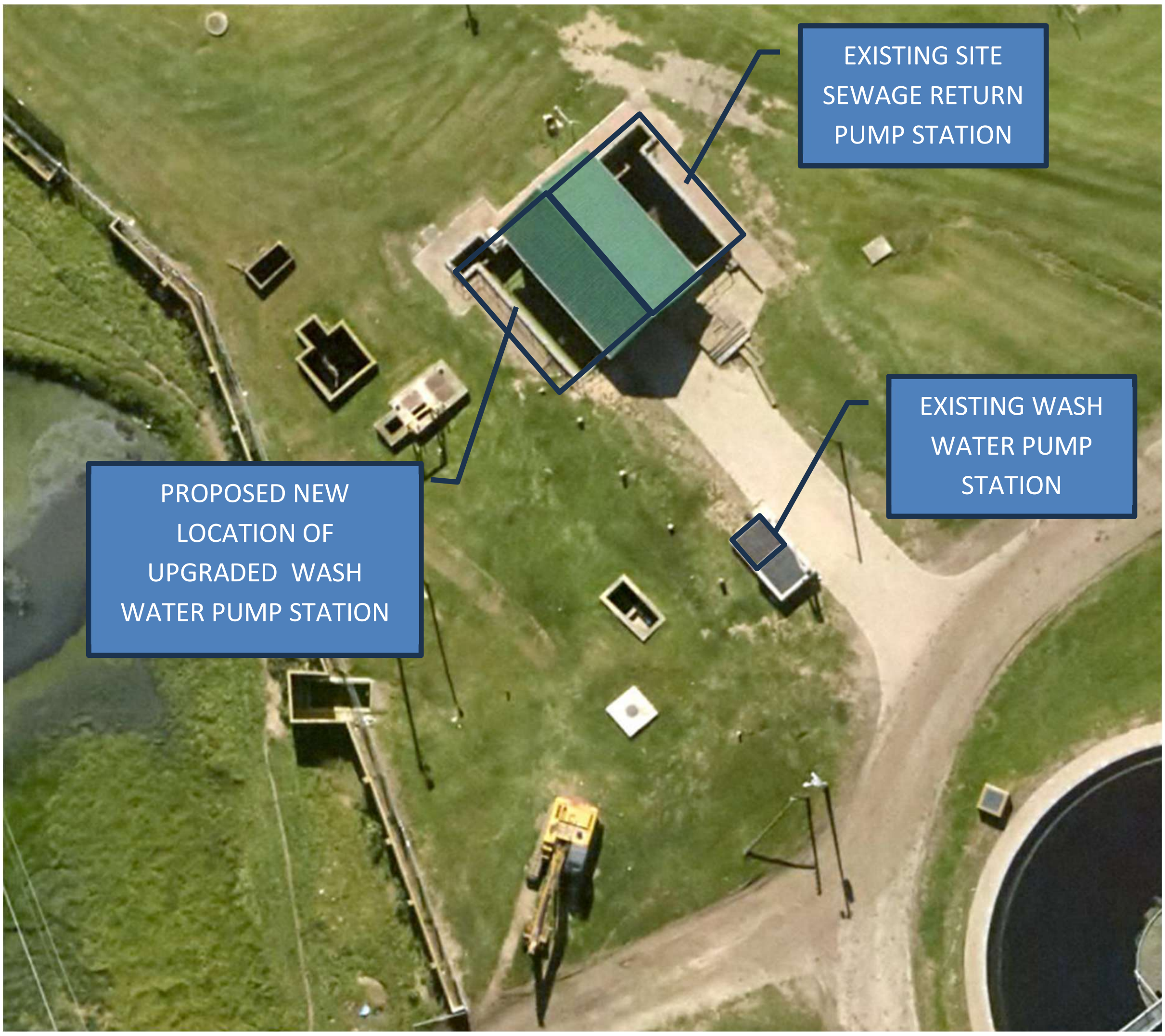


Figure 6-33: Location of New Wash Water Pump Station

6.1.12 Admin Building

A new administration building is proposed as part of the upgrade. The following design principles was used for the admin building:

- Processing and general worker areas on the ground floor
- Offices, labs, control room etc on the first floor
- Services to back
- Aesthetic screen to front (timber)
- Electrical building to set up courtyard space – green

The admin and electrical building complex are situated in the centre of the site in such a manor for the operators to have a view of the main process units. A conceptual layout of the admin building is shown in Figure 6-34 and a 3D render of the building in Figure 6-35.

GROUND FLOOR PLAN

1. Green Courtyard
2. Covered Parking
3. Entrance Foyer
4. Locker Rooms and Kitchenette
5. Stores (external access)
6. Staircase (fire escape)
7. Proposed ring road
8. Service Yard

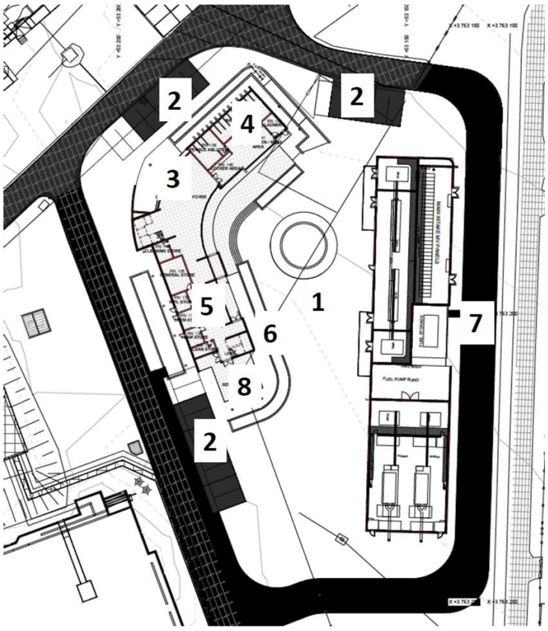


Figure 6-34: Conceptual Layout of the Admin Building

Figure 6-35: 3D rendering of the Admin Building

The accommodation schedule for the admin building is shown in Table 6-15.

Table 6-15: Accommodation Schedule for the Admin Building

ACCOMMODATION REQUIREMENTS	Rooms	Requirements	Est Area (m^2)
GROUND FLOOR: GENERAL WORKERS	Kitchen area (separate from office staff)	Lockers with seating for 10 (lockers separate)	10
	Staff mess room for 10 workers		
	Ablution facilities with change areas	Male showers and toilets -4-5 Female -2 10 lockers	30-50
	General storeroom	Tools, equipment, herbicides,	
	Separate store area for		

ACCOMMODATION REQUIREMENTS	Rooms	Requirements	Est Area (m^2)
	flammables		
STORAGE AREAS and OTHER	Chemical storage	No longer required	Old Admin Building
	3 Small storerooms for cleaning materials	External access, not through the main entrance	5
	Implement Store	Outside Access	5
	Boardroom Store	Off boardroom	6
	Lab Store	Off Lab	5
	Laundry		12
	Server Room	off SCADA	8
FIRST FLOOR: OFFICE STAFF	Rooms	Requirements	Est Area (m2)
	Reception and printing area	All stationery and printer	10-15
	Process controller (PC) office with overview of the plant	2 PC and 2 PC assistants on shift (future 4 + 4)	25-30
	3x Offices (in addition to above)	Superintendent, Snr PC, foreman	12
	Boardroom (large) with kitchenette -12-15 ppl	Close to reception and restrooms	20
	Ablution facilities for PCs and management staff	2 showers and 2 toilets per gender	20
	Kitchen for PCs and management staff	With seating tables	12
	Lab	Near offices	15
	SCADA Station	Overlooking inlet and external treatment works	20
	Control Room	Overlooking inlet and external treatment works	20
OTHER	Parking	10-12 for staff and visitors	
	Entrance Foyer and staircase		20-30
	Fire escape staircases	3	
	Disabled Lift/Hoist		4
	Circulation / Non-Assignable Area	Allowance	18%

6.1.13 Reuse Opportunities

Given the risk of future droughts, population growth and limited additional surface water sources for GM, direct, indirect, and industrial reuse was also considered as part of the Gwaing WWTW Master Plan. Final effluent is a substantial water source considering that about two-thirds of George's potable water consumption ends up at its WWTWs.

Since 2010 GM has been operating a 10 MLD indirect reuse plant from Outeniqua WWTW. The reuse treatment train consists of phosphorous removal with ferric chloride, screening, Ultrafiltration (UF) and chlorination before being diffused into the Garden Route Dam. The pipeline from the Outeniqua WWTW to the garden route dam has been sized for an ultimate capacity of 35 MLD. The additional capacity in the pipeline was provided to unlock future reuse opportunities at Outeniqua WWTW and Gwaing WWTW via the Garden Route Dam. Outeniqua WWTW has recently been upgraded to a capacity of 25 MLD. If all the final effluent from Outeniqua WWTW is reused via the Garden Route Dam, about 10MLD of the 35MLD pipeline capacity is left to be used by Gwaing WWTW.

Having considered the points mentioned above, three reuse options at Gwaing WWTW are identified:

1. Tie into the Outeniqua WWTW reuse system through the Garden Route Dam indirect potable reuse (IPR) system,
    a. Option 1 will require the following:
        i. A pump station and pipeline from Gwaing WWTW to Outeniqua WWTW.
        ii. The humus tanks of the trickling filters could potentially be used as tanks from which to pump to Outeniqua WWTW.
        iii. Upgrade of the Outeniqua UF, chlorination and UF facilities.
2. Implement an independent industrial reuse system from Gwaing WWTW
    a. Option 2 will require the following:
        i. Advanced tertiary treatment at Gwaing WWTW.
        ii. Pump station and distribution network from Gwaing WWTW to industrial users.
3. Implement an independent direct potable reuse (DPR) system from Gwaing WWTW,
    a. Option 3 will require the following:
        i. Advanced tertiary treatment at Gwaing WWTW.
        ii. Pump station and distribution network from Gwaing WWTW to the potable water network.

If 10 MLD is used via option 1 through the Outeniqua WWTW IPR scheme then 40MLD of Gwaing WWTW's ultimate capacity of 50MLD is left to be used through options 2 and 3. The international best practice guidelines adopted for the City of Cape Town reuse projects are summarised in Table 6-16.

Table 6-16: Treatment Requirements for Industrial Reuse, IPR and DPR

Treatment Performance requirements	Industrial Reuse	Indirect Potable Reuse (IPR) *	Direct Potable Reuse (DPR)
Virus	5-$\log_{10}$ Removal	8-$\log_{10}$ Removal	12-$\log_{10}$ Removal
Giardia		7-$\log_{10}$ Removal	10-$\log_{10}$ Removal
Cryptosporidium		8-$\log_{10}$ Removal	10-$\log_{10}$ Removal
Turbidity	2NTU		
Total Coliform Bacteria	<2.2/100ml		
Minimum no. treatment processes		≥2	≥2

Treatment Performance requirements	Industrial Reuse	Indirect Potable Reuse (IPR) *	Direct Potable Reuse (DPR)
* treatment requirement before injection into a surface water reservoir			

The potential verifiable log removals achieved with the current treatment train of the IPR treatment plant at Outeniqua WWTW IPR treatment plant are summarised in Table 6-17. If GM aims to go by the international best practice guidelines an additional advanced treatment step will be required at Outeniqua WWTW that will get the log-removals up to those depicted in Table 6-16.

Table 6-17: Evaluation of Outeniqua WWTW IPR scheme log removals

Treatment Performance requirements	Estimated log removals achieved with the IPR treatment plant at Outeniqua WWTW	Required log removals needed from the IPR treatment plant at Outeniqua WWTW
Virus	4.7	8
Giardia	5	7
Cryptosporidium	4.5	8

The proposed area for future reuse infrastructure is shown in Figure 6-36. This will depend on which site is selected for sludge beneficiation.

Figure 6-36: Proposed Area for Future Reuse Infrastructure

6.2 Electrical

Various internal meetings were held between Lukhozi Consulting Engineers and Reflekt Water to understand the electrical requirements and the scope of service to be provided by Lukhozi Consulting Engineers electrical department.

Subsequent a site inspection was conducted to better understand the existing electrical infrastructure and how the alterations will affect and/or modify the electricity network.

6.2.1 Design Standards

The following design standards were used for the electrical and electronic design:

- SANS 10139 – Code of practice for design, installation, commissioning and maintenance of fire detection and alarm systems in non-domestic premises.
- SANS 10114-1 – Interior lighting Part 1: Artificial lighting of interiors.
- SANS 10114-2 – Interior lighting Part 2: Emergency Lighting.
- SANS 10142-1 – The Wiring of Premises Part 1: Low-voltage installations.
- SANS 10400 – National Building Regulations, Part T.

6.2.2 Existing Services

There is an existing 11 kV electricity network for the facility which (from electricity accounts provided) do not utilise more than 600 kVA maximum demand for a single month. The electricity network is currently a straight-line network and not a ring network.

There are street lighting and security lighting, however it could not be confirmed if it is operational, however the lighting in general is old and of older technology.

The existing building’s electricity installations are in various stages of age due to some buildings being upgraded in the past few years.

6.2.3 Recommendations

The electrical and electronical recommendations for the Gwaing WWTW upgrades are subdivided into each phase of the masterplan in the following sections. The proposed electrical MV ring main is shown in pink on Figure 6-37.

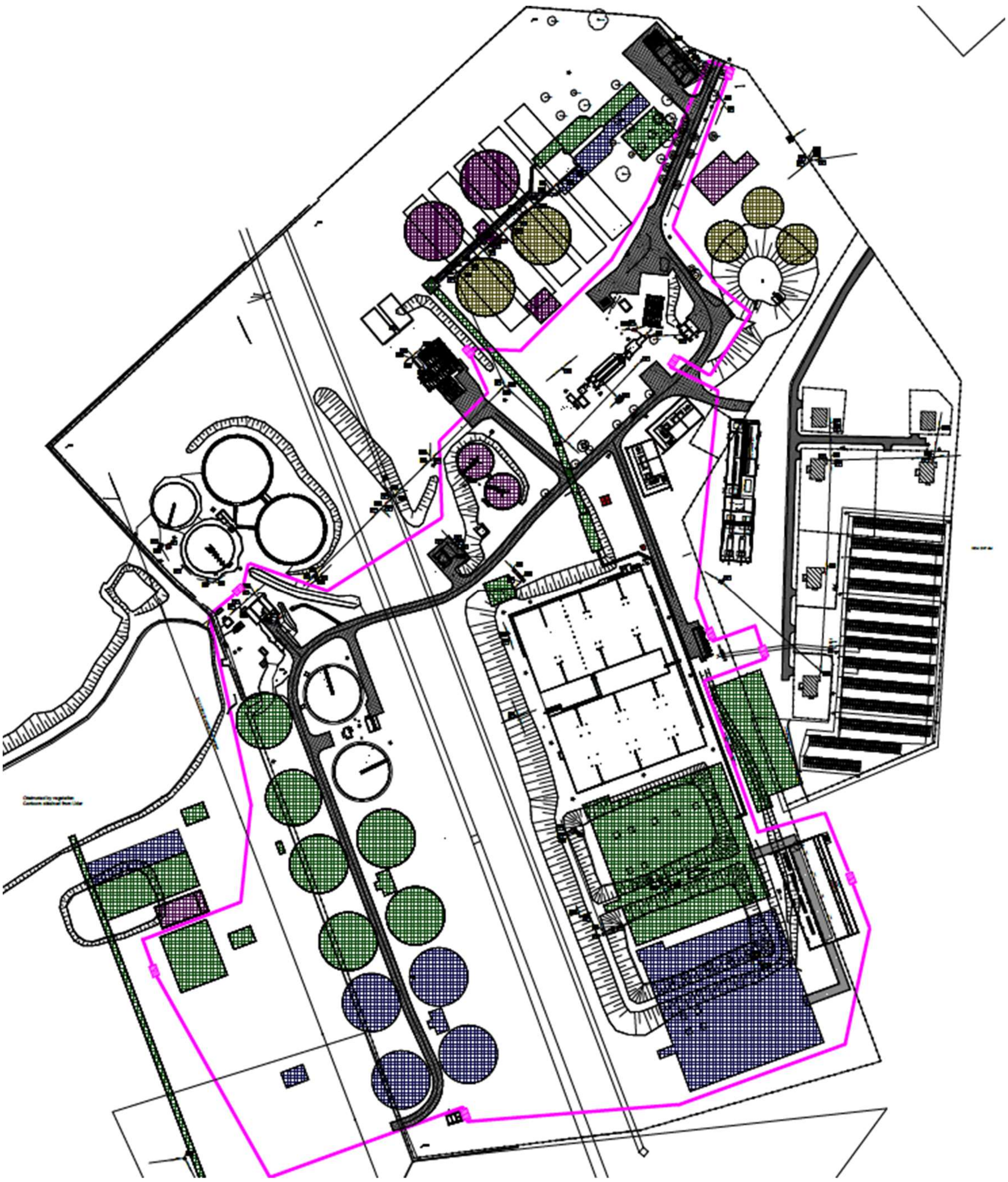


Figure 6-37: Electrical MV Ring Main Layout

6.2.3.1 Phase A

There is an allowance made for a new substation building which will house the switchgear for the required 2.5 MVA demand to accommodate the existing and the additional power requirements for Phase A. This phase of the project will allow for standby generator capacity to accommodate the 2.5 MVA electricity demand.

As part of Phase A, the medium voltage cables will be installed throughout the site to allow for a ring electricity network. There are also allowances made for street lighting and security lighting.

Various of the existing buildings will be modernised and new power and lighting allowed. There are motor control centres that will be upgraded and modernised with power factor correction.

The following electrical work and equipment are foreseen for phase A:

- Make safe and remove redundant equipment
- Medium Voltage Network
- Distribution Boards
- Cables
- Earthing
- Luminaires
- Power
- Street lighting
- Standby Generator

6.2.3.2 Phase B

Phase B will allow for an additional 1 MVA electricity demand and the additional switchgear installed in the substation building. An additional standby generator will be installed to accommodate the additional electricity load.

There is also an allowance for maintenance to be done on some of the buildings, not included in Phase A.

The following electrical work and equipment are foreseen for phase B:

- Medium Voltage Network
- Distribution Boards
- Cables
- Power
- Standby Generator

6.2.3.3 Phase C

There is an allowance made for the additional miniature substation for the equipment required in Phase C as well as additional generator capacity. An allowance for maintenance is included for some of the buildings, not included in Phase A and B.

The following electrical work and equipment are foreseen for phase C:

- Medium Voltage Network
- Distribution Boards
- Cables
- Power
- Standby Generator

6.2.3.4 Phase D

There is an allowance made for the additional miniature substation for the equipment required in Phase D as well as additional generator capacity. An allowance for maintenance is included for some of the buildings, not included in Phase A, B and C.

The following electrical work and equipment are foreseen for phase D:

- Medium Voltage Network
- Distribution Boards
- Cables
- Power

6.2.3.5 Electronic Services

Various electronic services have been allowed for the project. During concept and detail design, the Client needs to confirm which equipment must be included. If certain of the electronic services are excluded from the scope of work, the Client also has to confirm which will be installed in the future to allow for wireways.

The following electronic work and equipment are foreseen:

- Access Control
- Alarm System
- Closed-Circuit Television
- Fibre
- Fire Detection
- IT
- SCADA

6.2.4 Summary of Electrical Requirements

The phases for the building works and the electrical works are out of sync due to the infrastructure and the complexity of the medium voltage network. The electrical master plan provided also provides maximum flexibility for alteration and/or additions in the future.

There is an additional high increase in electricity capacity requirement in Phase A and Phase B has marginally less equipment than Phase A. However, Phases C and D does not have a large electricity increase associated to the additional equipment.

6.3 Chemical and Energy Usage

For the ultimate solution, Gwaing WWTW may be using chlorine for final disinfection (if UV is not selected) and polymer for waste activated sludge and primary sludge dewatering. A summary of the estimated chemical usage at 50 MLD and the associated estimated costs are provided in Table 6-18.

Table 6-18: Chemical usage and cost summary for Gwaing WWTW ultimate solution

Chemical	Dosing rate	Dose/annum (kg/a)	Rate (R/kg)	Annual Cost (2023 value)
Chlorine	12.00mg/l	219000	R32.60	R7 139 400.00
Polymer for primary sludge dewatering	5kg/ton TSS	9777	R44.77	R437 738.04
Polymer for waste activated sludge dewatering	5kg/ton TSS	15768	R44.77	R705 985.43
Total				**R8 283 123.47**

Based on a detailed equipment list, and each equipment's associated electric motor and operational hours, the electricity cost for operating Gwaing WWTW's ultimate solution was estimated. The costs were determined using an average unit price of R2.04/kWh for electricity and are summarised in Table 6-19. Total electricity costs including energy generation by anaerobic digestion are estimated to be R 26 231 766.39 in 2023.

Table 6-19: Electricity usage cost estimate for Gwaing WWTW ultimate solution

Description	Total Duty Power (kW)	kWh/d	Total -R/d (2024)
Inlet Works	341	1065	R2 172.48
Low Lift Screw Pump Station	31.5	660.4	R1 347.22
PSTs	99.8	1009	R2 058.46
Thickeners	81	925	R1 887.31
Anaerobic Digestion	29	1161	R2 368.23
Existing Reactor - Module A (Including Surface Aeration)	529	11644	R23 754.58
New Reactor - Module B	166	2920	R5 957.62
New Reactor - Module C	166	2920	R5 957.62
Blowers and Aeration	514	12240	R24 970.42
New WAS Pump Station	16	276	R563.86
Module A SSTs (Existing Reactor SSTs)	18	250	R510.82
Module B SSTs	37	501	R1 021.63
Module C SSTs	37	501	R1 021.63
RAS Pump Station 1	160	3732	R7 614.10
RAS Pump Station 2	160	3732	R7 614.10
Existing and Additional WAS Dewatering Equipment	42	544	R1 108.84
Wash Water Pump Station	16	276	R563.86
Disinfection: Chlorine	11	264	R538.56
Subtotal	**2451**	**44623**	**R91 031.30**
Contingency Allowance	368	6693	R13 654.69
Annual Electricity Cost (2024)			**R38 210 387.94**

The total annual chemical and electricity costs are estimated to be R46 493 511.41 (in 2024 Rands).

6.4 Waste Reduction and Resource Recovery

The vision for Gwaing WWTW is to change the focus from simply dealing with waste to recovering multiple resources and thereby transitioning it from being a WWTW to a WRRF (Water Resource

Recovery Facility). Several waste reduction and resource recovery strategies are employed in the design of the upgrades, including:

- Regional grit processing facilities to enable reuse of grit as part of composting or fill material.
- Regional screenings processing facility to minimise volume, odours, pathogens and vector attraction of screenings.
- Sludge beneficiation in the form of composting or fertilizer production is envisaged.
- The methane gas produced from anaerobic digestion will be used for generating heat and power (as part of Phase D)
- Effluent from the Gwaing WWTW can in future be pumped to neighbouring industries or golf courses for non-potable use. Alternatively, it can be further treated together with the effluent from Outeniqua WWTW before it is pumped to the dam as part of an indirect potable reuse scheme.
- Effluent will be recycled and pressurized on site in a wash water ring main for various uses and irrigation, reducing the potable water demand of the WWTW.

6.5 Personnel Requirements

A preliminary rating based on the Water Services Act of 1997's Regulations Relating To Compulsory National Standards For Process Controllers And Water Services Works was done. Gwaing WWTW scored more than 70 points even without anaerobic digestion. This means that with the upgrade, Gwaing WWTW is likely to be rated as a class A works. The staffing requirements of a class A works according to the act are presented in Table 6-20.

Table 6-20: Gwaing WWTW Staffing Requirements

Work class	Class and number of persons as process controllers	Class of person as supervisor	Class of Person for weekly inspection
A	1 x Trainee 2 x I 1 x II 1 x III 1 x IV	V	-

6.6 Environmental Authorisation Requirements

The proposed upgrades will require an Environmental Authorisation (EA) and a Water Use License (WUL). A screening was done on the Department of Forestry, Fisheries and the Environment's (DFFE) National Web-based Environmental Screening Tool. The Screening Tool identifies related exclusions and/ or specific requirements including specialist studies applicable to the proposed site and/or development, based on the national sector classification and the environmental sensitivity of the site. The Screening tool report indicated that the following studies should be undertaken for the EA process:

Table 6-21: EA Screening test for specialist studies.

1	Agricultural Impact Assessment	7	Geotechnical Assessment
2	Archaeological and Cultural Heritage Impact Assessment	8	Health Impact Assessment
3	Palaeontology Impact Assessment	9	Socio-Economic Assessment
4	Terrestrial Biodiversity Impact Assessment	10	Ambient Air Quality Impact Assessment
5	Aquatic Biodiversity Impact Assessment	11	Plant Species Assessment
6	Hydrology Assessment	12	Animal Species Assessment

A motivation was included in the Notice of Intent (NOI) and Site Sensitivity Verification Report (SSVR) against Studies 1, 2, 3, 4, 8, 9, 10, 11 and 12 as the proposal is for the upgrading of an existing facility within the current footprint and those areas have been previously disturbed and shaped. The following studies can be expected as part of the next upgrade:

- 5 – Aquatic Assessment will have to be undertaken due to the proximity to the river west of the facility and is also required for the WULA (uncertain at this stage if it will be a new WULA or Amendment of the existing WUL).
- 6 – Hydrological assessment will be required and will determine whether a groundwater monitoring programme is required.
- 7 – Geotechnical will be undertaken for more detail to inform the engineering design.

A notice of intent (NOI) to submit an application for environmental authorization was sent to the Western Cape Department of Environmental Affairs and Development Planning (DEADP) on 26 April 2024. On 25 June 2024, DEADP responded with a letter containing comments on the NOI. A summary of their comments is provided below:

a. Based on the information submitted to this Directorate a Basic Assessment process must be followed in order to apply for Environmental Authorisation.
b. EAP requires clarification to include only neighbouring property owners within a 1km radius of the existing Gwaiing WWTW in a public participation process.
c. A site sensitivity verification report(s) (SSVR) that confirms or disputes the site sensitivities for each of the themes identified in the Screening Tool Report is required.
    i. the report(s) must include a motivation for the exclusion of any of the specialist assessments identified in the Screening Tool Report, which in the opinion of the EAP are not considered relevant or required.
d. Confirm with Western Cape Government: Department of Agriculture (DoA) that the sensitivity of the site on the agricultural theme is not high.

e. Consult CapeNature in the public participation process and specifically obtain written confirmation Gwaing WWTW site doesn't have a HIGH sensitivity rating for the Animal Species Theme.
f. A very high sensitivity rating for the Aquatic Biodiversity Theme was identified and the Aquatic Impact Assessment must adhere to the Aquatic Biodiversity Specialist Assessment protocols.
g. DEADP strongly advises that a Notice of Intent to Develop (NID) be submitted to Heritage Western Cape (HWC).
h. DEADP does not anticipate that a civil aviation-related assessment would be required.

The aquatic specialist will use this concept design report to assist with the Section 27 Motivational Report for the WULA. The Environmental Assessment Practitioner (EAP) will also use this concept design report to continue compiling the Basic Assessment Report for the proposed upgrades.

More studies, such as Health Impact Assessment and Ambient Air Quality Assessment, may be required depending on DFFE, DEADP and BGCMA's feedback.

6.7 Architectural

Based on a precedent study, analysing the architectural type (wastewater treatment works buildings), the architectural team has drawn a number of conclusions that will act as a guideline reference for best practice in the design of the blower house and admin building. These principles are discussed below in sections 6.7.1 to 6.7.6, along with some sketches and images that will assist in the readers' understanding of the principles presented.

6.7.1 Spatial Design – the arrangement of the various user requirements within the building in relation to one another

In almost all cases analysed, the design of the WWTW building can be divided into 2 major parts – "Plant" areas, comprising program such as the electrical plant, blower hall, and MCC, and "User" areas, made up of offices, labs, meeting rooms, ablutions, and stores. Typically, the Plant items are all located on the ground floor, along with general workers' ablutions and change rooms, and the user spaces are all located on the first floor, slightly removed from the industrial processes and with views overlooking the facility and the operations. In the case of this project, the Blower Hall and all "Plant" areas will be housed separately from the Admin Building. The Admin Building will house all the "User" areas. These (User areas) are also often located in a linear arrangement, along the street or entrance edge of the site, creating the opportunity for an aesthetically pleasing façade treatment and the creation of a strong street edge, with the building being scaled down towards the back. Figure 6-38 shows an annotated section through the Athlone WWTW blower house by SALT Architects, a good indication of this principle. Although the buildings will be separated, this principle will still apply to the design of the Admin Building.

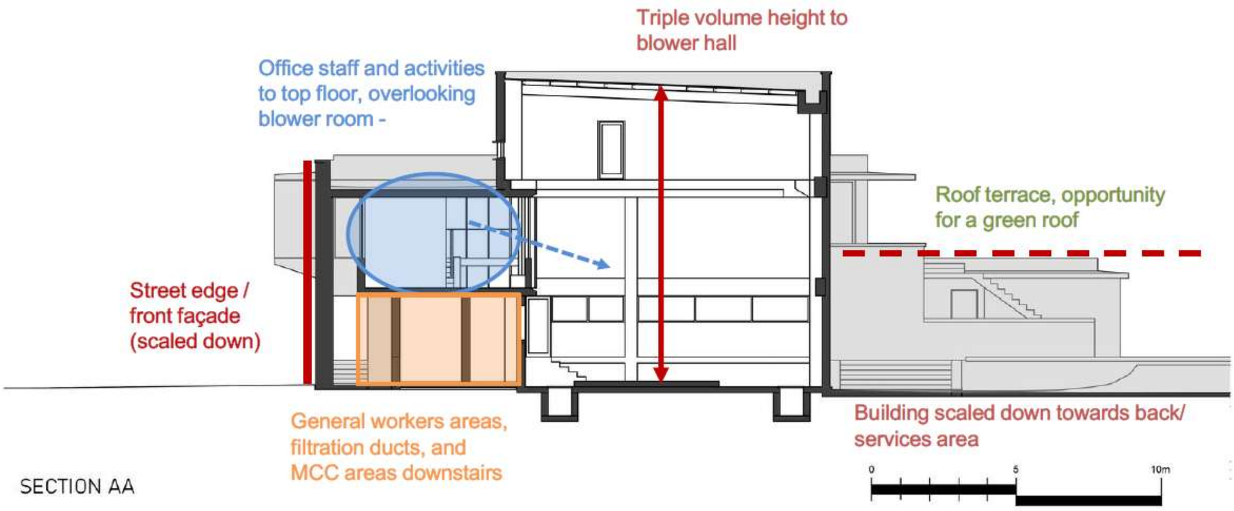


Figure 6-38: Section of Athlone WWTW's blower house

6.7.2 Textured/Repetitive Facades

The nature and function of the blower house complex, being largely industrial/process-based, lends itself to monolithic, box-type forms, being essentially a covering for an industrial process to take place. With most of the "User" activity being grouped together, an opportunity is presented to design a building that merges the "humane" aspect of human activity and the industrial aspect of the site. This is often done through a specific, interesting façade treatment, or screened element, that is aesthetically pleasing and provides a sense of warmth to an otherwise industrial environment. This is normally introduced near the entrance, or along the street-facing side of the building, but can also be site-dependent. A good example of this, as shown in Figure 6-39, is found in the design of a Water Treatment Plant in France by AWP.

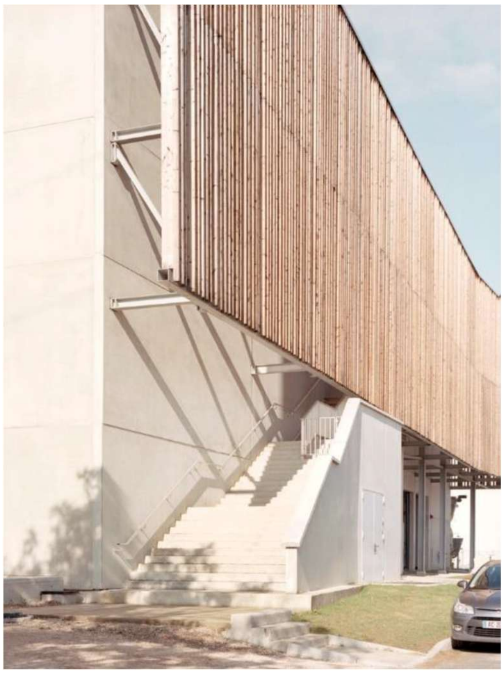


Figure 6-39: Picture of a Water Treatment Plant in France

6.7.3 Use of Materials

It is generally accepted as good practice, when designing for industrial applications, to make use of low-maintenance materials. We also feel that it is important to make use of materials that reflect the nature of the site and its surroundings, and are as natural and sustainable as possible. Materials such as raw concrete, clay-brick, and natural finished timber immediately come to mind. The Athlone WWTW by SALT Architects is a precedent that manages this well from which inspiration for the proposed material pallet can be drawn. The proposed materials are indicated in Figure 6-40.

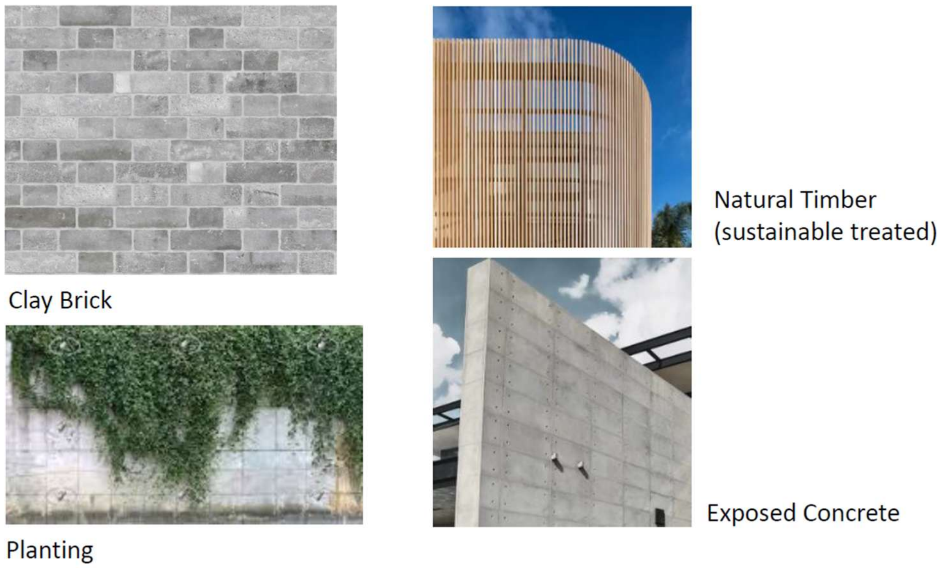


Figure 6-40: Proposed Building Materials

6.7.4 Sustainability Principles

This is a wide-ranging topic with many sub-topics. Elements such as building management, air quality, energy and water usage, materials, and emissions, among others, begin to play a significant role in building design. Some items that we intend to incorporate into the design include the use of natural ventilation to improve air quality (see Figure 6-41), daylight glare control, creating external views for the users, energy and water management, rainwater harvesting, and community involvement.

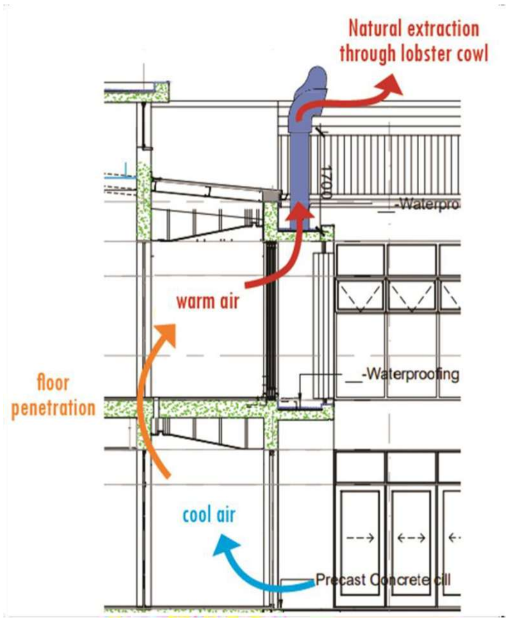


Figure 6-41: Natural ventilation flow

One of the more prominent design elements will be the incorporation of a natural, planted courtyard. This has multiple benefits for the building, its users, and the environment. Figure 6-42 shows this idea with an image of a courtyard incorporated into a previous 6-star green-rated building designed by Imbono Architects. This can be achieved in principle, even in a case where the buildings don't necessarily enclose a formalised courtyard.

Figure 6-42: Picture of a natural Planted Courtyard

6.7.5 Response to Site and Concept Design Proposal

Elements relating to the site are the main indicators when it comes to the initial design process, with site shape, orientation, prevailing weather conditions, and existing infrastructure on site all playing a role in how the building finds itself positioned on the site.

With the building site demarcated in the masterplan, we have proposed a high-level concept design that considers the previously mentioned points, and positions itself logically and responsively. Figure 6-43 provides a conceptual site approach. The buildings are positioned in such a way as to create a street-facing entrance and façade, with the buildings extending linearly along the axis of the existing plant. This allows for the operators to have easy viewing access over both internal and external processes. Future buildings on the site can be positioned to set up a green courtyard behind the admin building, which is experienced on approach to the building.

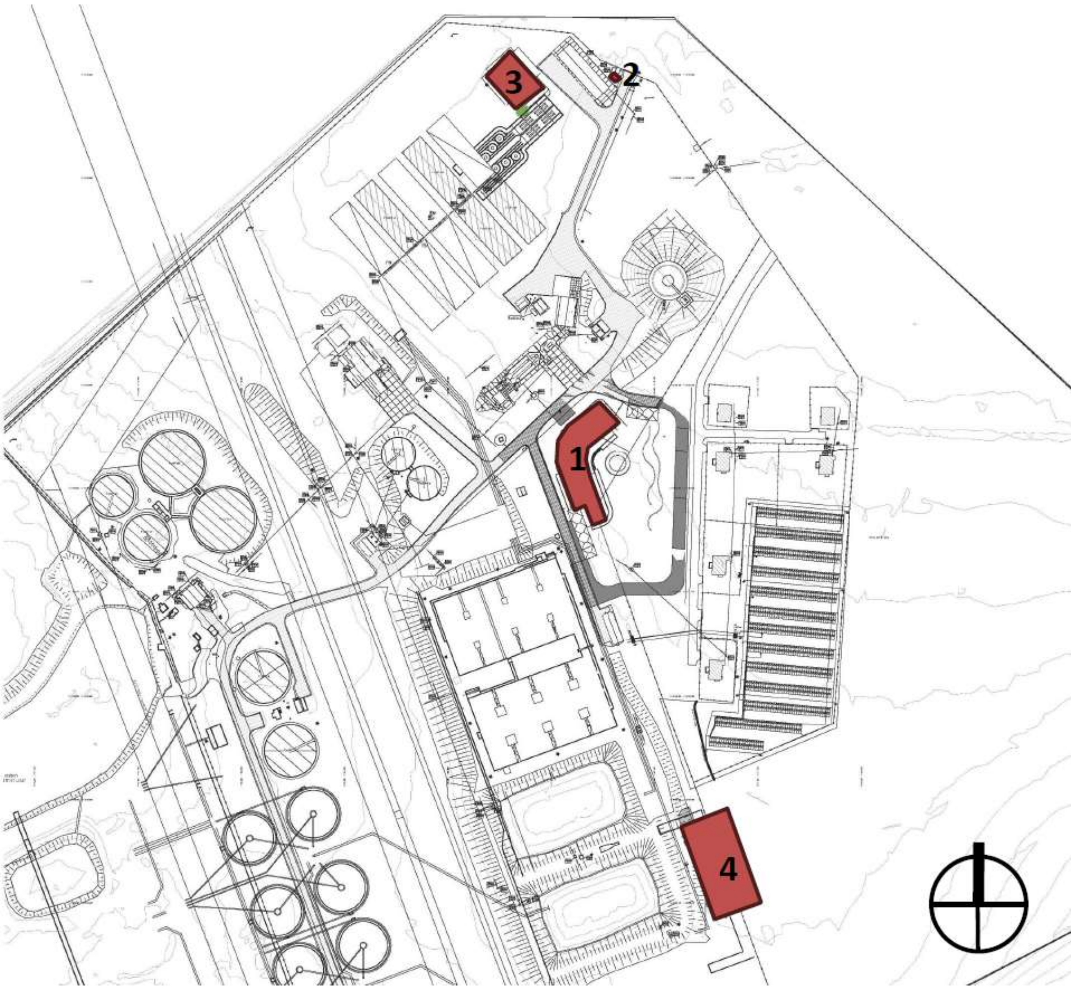


Figure 6-43: Positioning of Architectural Buildings on Site

As shown and numbered in Figure 6-43, the larger site incorporates several new buildings, namely:

1. New Admin Building

The new Admin Building is located in an open portion of land on the site, in a central position providing access and visibility to the majority of the infrastructure and sewer treatment processes that take place on-site. The building has been designed in alignment with the above-mentioned principles and incorporates a green/planted courtyard on the east side of the building.

2. New Guard House

The new Guard House is necessarily located at the entrance to the site. The layout of the Guard House will follow the standard Guard House design implemented by the George municipality on other George Municipality sites. The Architectural Character of the Guard House will echo the design features and same material palette established in the design of the Admin Building, with the idea of creating a consistency and uniformity throughout the site. Figure 6-44 shows extracts from the George Municipality's standard Guard House design. Figure 6-45 shows the 3D rendering for the proposed new Guard House which forms part of the upgrades to the Gwaing WWTW complex.

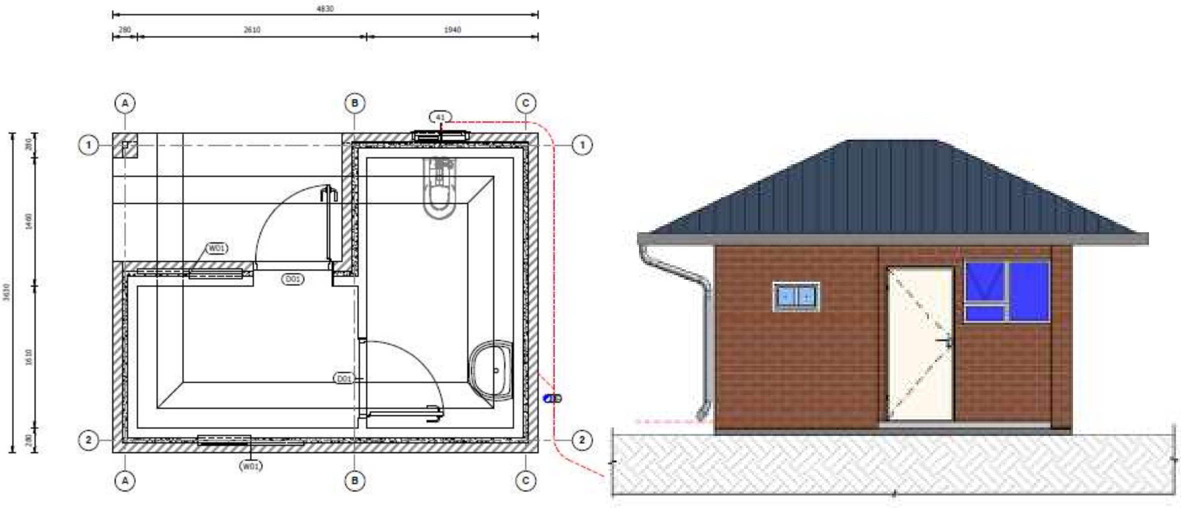


Figure 6-44: Extracts from George Municipality Standard Guard House Drawings

Figure 6-45: 3D Rendering of proposed Guard House Building

3. New Electrical Sub-Station

The new Electrical Substation is located near the entrance to the site to pick up on the incoming electrical supply and has also been aesthetically designed in line with the Admin Building.

4. New Blower House

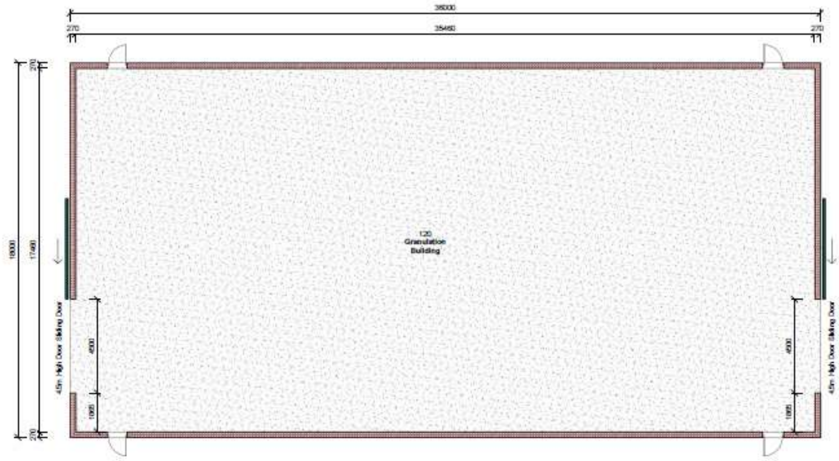
The Blower House is located south of the new Admin Building, as dictated by the industrial process on site. The spaces required are dictated by the function of the building and the materials used are also in line with the general site aesthetic that has been established.

5. Upgrade/Extension to existing De-Watering Facility (not shown in Figure 6-43)

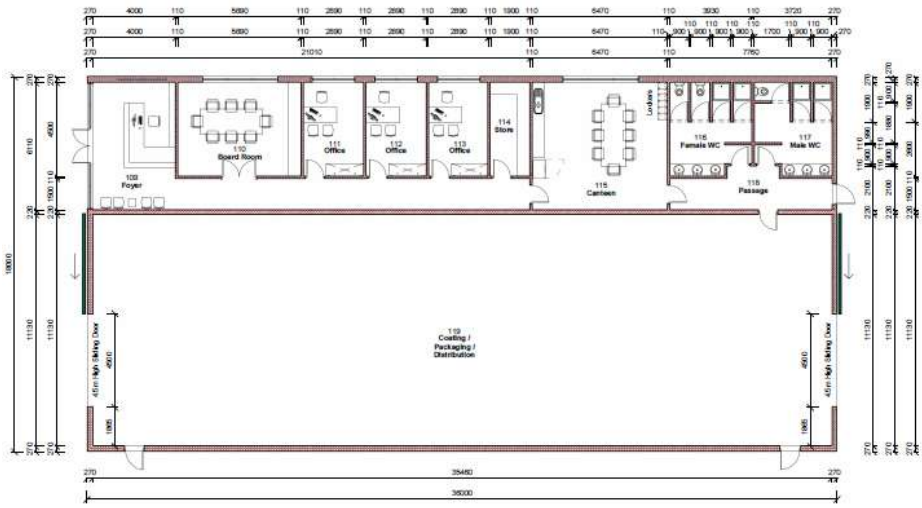
A small extension is planned for the existing De-Watering Facility which has not yet been developed in full detail.

6. Biosolids Beneficiation Facility (not shown in Figure 6-43)

A Biosolids Beneficiation Facility (BBF) is proposed as an option for disposal of sludge at Gwaing WWTW. The proposed facility will include a Solar Drying facility (including area allocated for future expansion), a granulation building, and an Admin Block. The Admin Block will house the staff facilities as well as the Coating, Packaging and Distribution functions of the facility. Figure 6-46 shows extracts from the concept drawings for the BBF which need to be adjusted to the specific site allocated for the facility.



Granulation Building
SCALE 1 : 100



Admin & Packaging Block
SCALE 1 : 100

Figure 6-46: Proposed Floor plans For Granulation Building and Admin Block

6.7.6 Landscaping

The proposed WWTW aims to establish an environmentally conscious and visually appealing space that seamlessly integrates with the natural surroundings of the Eden/Garden Route area. The focus is not only on the functional requirements of waste treatment but also on contributing positively to local ecosystems and community well-being through thoughtful landscape design.

George is situated in a region rich in biodiversity, featuring diverse biomes, including the renowned Fynbos and Cape Floral Kingdom, as well as indigenous forests, and unique vegetation. The natural beauty of the Cape Floristic Region (see Figure 6-47) underscores the importance of an approach that is seamlessly integrated into these existing contexts.

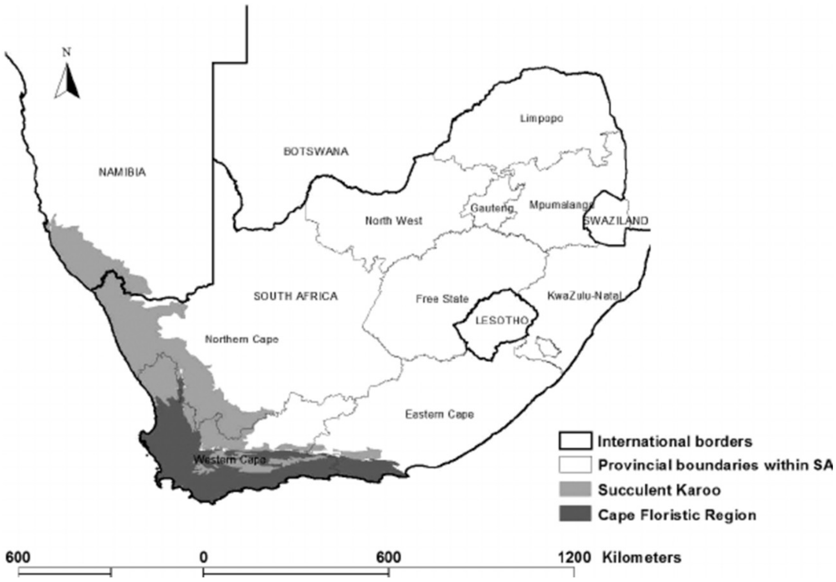


Figure 6-47: Map of the Cape Floristic Region

The design approach for the landscaping of the site followed the following principles:

- **Ecological Sensitivity:**
    - The design aims to prioritize ecological sensitivity by respecting the fynbos biome and preserving indigenous vegetation, including iconic species such as the Protea Repens (Sugarbush) and Leucospermum (Pincushion). Upon further site investigation, the layout will be planned, as far as possible, to minimize disturbance to the natural terrain, ensuring that the facility blends harmoniously with its surroundings.
- **Green Courtyard with Biodiversity Enhancement:**
    - A central feature of the landscape design is the incorporation of a green courtyard within the treatment area. This space will serve as an aesthetically pleasing focal point, providing an opportunity to introduce a variety of native trees and plants, including species like Protea cynaroides (King Protea) and Leucadendron argenteum (Silver Tree). The design will specifically focus on biodiversity enhancement within the green courtyard, supporting local flora, attracting native bird species, and promoting overall biodiversity and contributing towards the health of the Cape Floristic Region.

Figure 6-48 presents a picture of the landscape plants envisioned to be used for the Gwaing WWTW upgrades.

Protea Repens

Fynbos Type

King Protea

Silver tree

Figure 6-48: Landscaping Concepts

6.8 Site Security

A new guardhouse is recommended at the main entrance next to the existing admin building to provide access control for the site. The guard house will include a toilet and a small kitchenette for the guards. Most of the new infrastructure, except for the parts of the SSTs, the RAS pump station and the chlorine contact tank will be built within the existing fence of the Gwaing WWTW. The existing fence will be extended to include these structures inside the fence for security reasons.

The proposed layout and rendering of the guardhouse are shown in Figure 6-49.

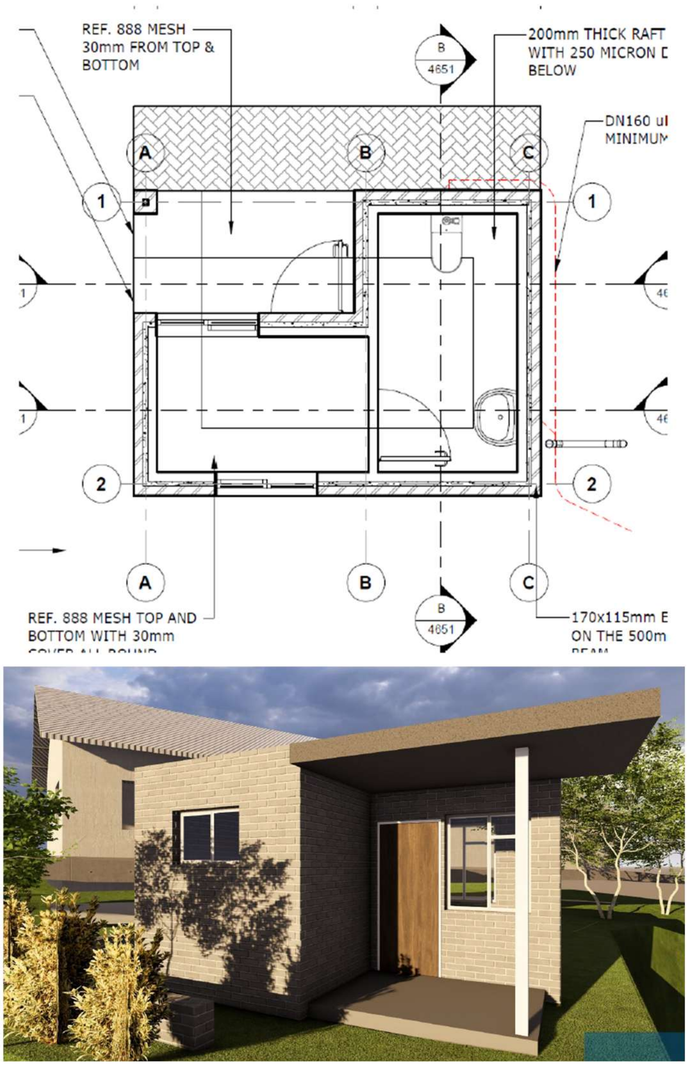


Figure 6-49: Proposed layout and rendering of the Guardhouse

6.9 Roads and Stormwater Network

The details of the roads and stormwater infrastructure will be developed during the detail design phase. GM requested that new roads be built to the operator's houses next to Gwaing WWTW. The cost for these roads is included in the Gwaing WWTW phase A&B upgrades. All new roads including the roads to the operator's houses will be constructed with interlocking pavers. All existing loads will be refurbished. A layout of the proposed new roads is shown in Figure 6-50. Due to the nature of the plant and future upgrades, future services (pipes and cables) will inevitably need to cross new and existing roads. Pavers are easy to remove and re-use in the case where excavation through roads is required.

A new stormwater system will convey stormwater through concrete pipes. It is envisaged that stormwater will drain to the existing maturation ponds on site since it is located at the lowest point of the site and has sufficient capacity to attenuate the flow.

Figure 6-50: Layout of proposed new roads

6.10 Demolition Work

Figure 6-51 shows the structures to be demolished as part of the Gwaing WWTW phase A&B upgrade. The structures that need to be demolished is the old sludge drying beds and the bio trickling filter process train. The old sludge drying beds at Gwaing WWTW are not operational anymore and need to be demolished to make space for the new inlet works, the PSTs, and the primary sludge pump station. The bio-trickling filter process train is no longer operational and has been decommissioned for some years.

Once Phase B is commissioned, the existing inlet works will no longer receive any flow. Thus, if required, demolition of the existing inlet works filters can be done to make space for future infrastructure.

Figure 6-51: Structures to be Demolished as part of Gwaing WWTW upgrades

7 IMPLEMENTATION AND PHASING

All infrastructure was designed with a phased approach in mind. The plant's current capacity, when operating an MLE process, is 10.4 MLD and when operating a UCT process is 8.6 MLD. The ultimate capacity of the Master Plan was designed for 50 MLD based on a UCT process. The site layout of the total capacity of the Master Plan is shown in Figure 7-1 and the 3D model is shown in Figure 7-2.

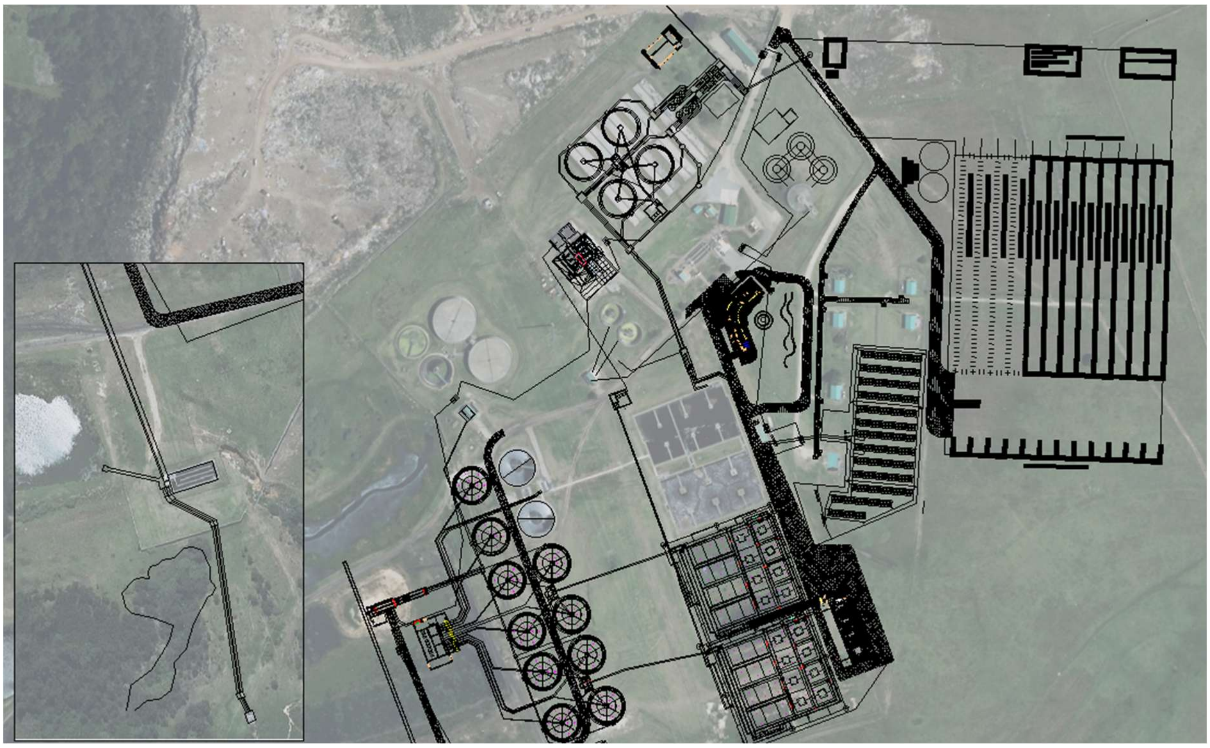


Figure 7-1: Master plan site layout

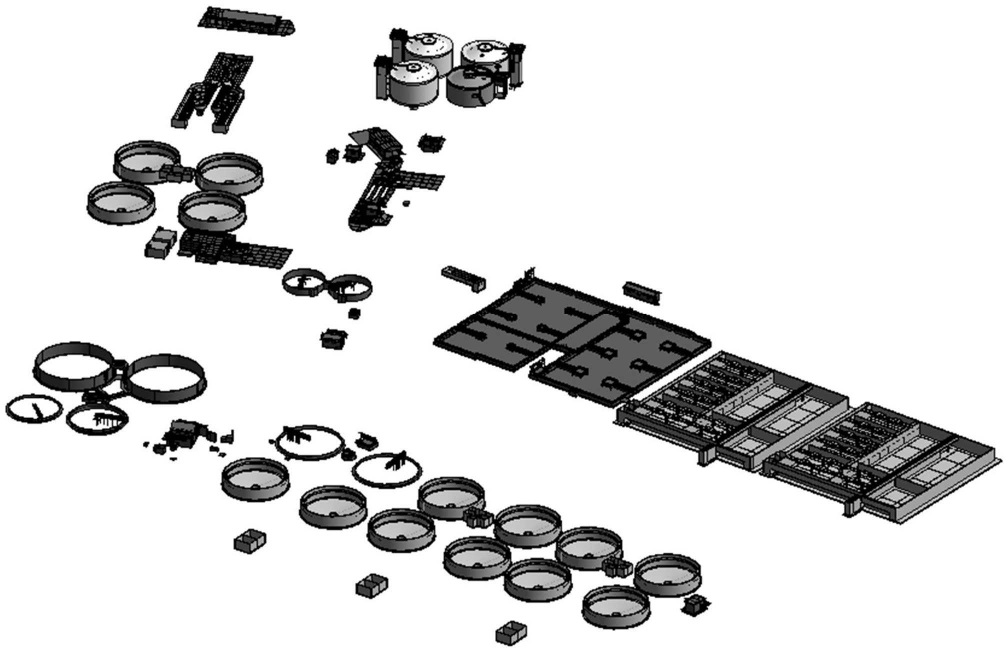


Figure 7-2: Site layout 3-D Model

7.1 Process unit and module naming conventions

For phasing purposes, specific naming conventions have been selected for ease of reference to specific structures or parts of the treatment works. Table 7-1 summarizes each unit-process for the ultimate capacity and indicates each unit's process capacity. The table summarizes the main unit processes and does not include ancillary pumpstations, flow split units, or flow measurement infrastructure.

Table 7-1: Unit process naming conventions and process capacities

<table>
<tr><th>Unit Process</th><th>No. Units</th><th>Unit Name</th><th>Process Capacity in terms of ADWF (MLD)</th></tr>
<tr><td rowspan="2">Inlet Works</td><td rowspan="2">2</td><td>Inlet Works 1</td><td>25</td></tr>
<tr><td>Inlet Works 2</td><td>25</td></tr>
<tr><td rowspan="4">Primary Settling Tanks</td><td rowspan="4">4</td><td>PST 01</td><td>14</td></tr>
<tr><td>PST 02</td><td>14</td></tr>
<tr><td>PST 03</td><td>14</td></tr>
<tr><td>PST 04</td><td>14</td></tr>
<tr><td rowspan="2">Primary Sludge Gravity Thickeners</td><td rowspan="2">2</td><td>GT 01</td><td></td></tr>
<tr><td>GT 02</td><td></td></tr>
<tr><td rowspan="4">Anaerobic Digestors</td><td rowspan="4">4</td><td>AD 01</td><td></td></tr>
<tr><td>AD 02</td><td></td></tr>
<tr><td>AD 03</td><td></td></tr>
<tr><td>AD 04</td><td></td></tr>
<tr><td rowspan="3">Biological Reactors</td><td rowspan="3">3</td><td>Reactor A</td><td rowspan="15">Reactor and SST process capacity depends on the process type and configuration selected.</td></tr>
<tr><td>Reactor B</td></tr>
<tr><td>Reactor C</td></tr>
<tr><td rowspan="12">Secondary Settling Tanks</td><td rowspan="12">12</td><td>SST A-1</td></tr>
<tr><td>SST A-2</td></tr>
<tr><td>SST A-3</td></tr>
<tr><td>SST A-4</td></tr>
<tr><td>SST B-1</td></tr>
<tr><td>SST B-2</td></tr>
<tr><td>SST B-3</td></tr>
<tr><td>SST B-4</td></tr>
<tr><td>SST C-1</td></tr>
<tr><td>SST C-2</td></tr>
<tr><td>SST C-3</td></tr>
<tr><td>SST C-4</td></tr>
<tr><td rowspan="2">Chlorine Contact Tank</td><td rowspan="2">2</td><td>CCT 01</td><td>25</td></tr>
<tr><td>CCT 02</td><td>25</td></tr>
</table>

The biological reactors and SSTs are divided into three modules, as described in Section 5.3.1. The three modules and their naming conventions are shown in Figure 7-3. Although the reactors and SSTs are divided into three specific modules, the sequencing of implementation may result in the Module A and Module B SSTs being commissioned before Reactor B is constructed, thus meaning Module A and B SSTs will operate together with Reactor A. The module naming conventions are as per the flow split of the ultimate solution, but do not limit the interconnectivity of the modules during prior phases.

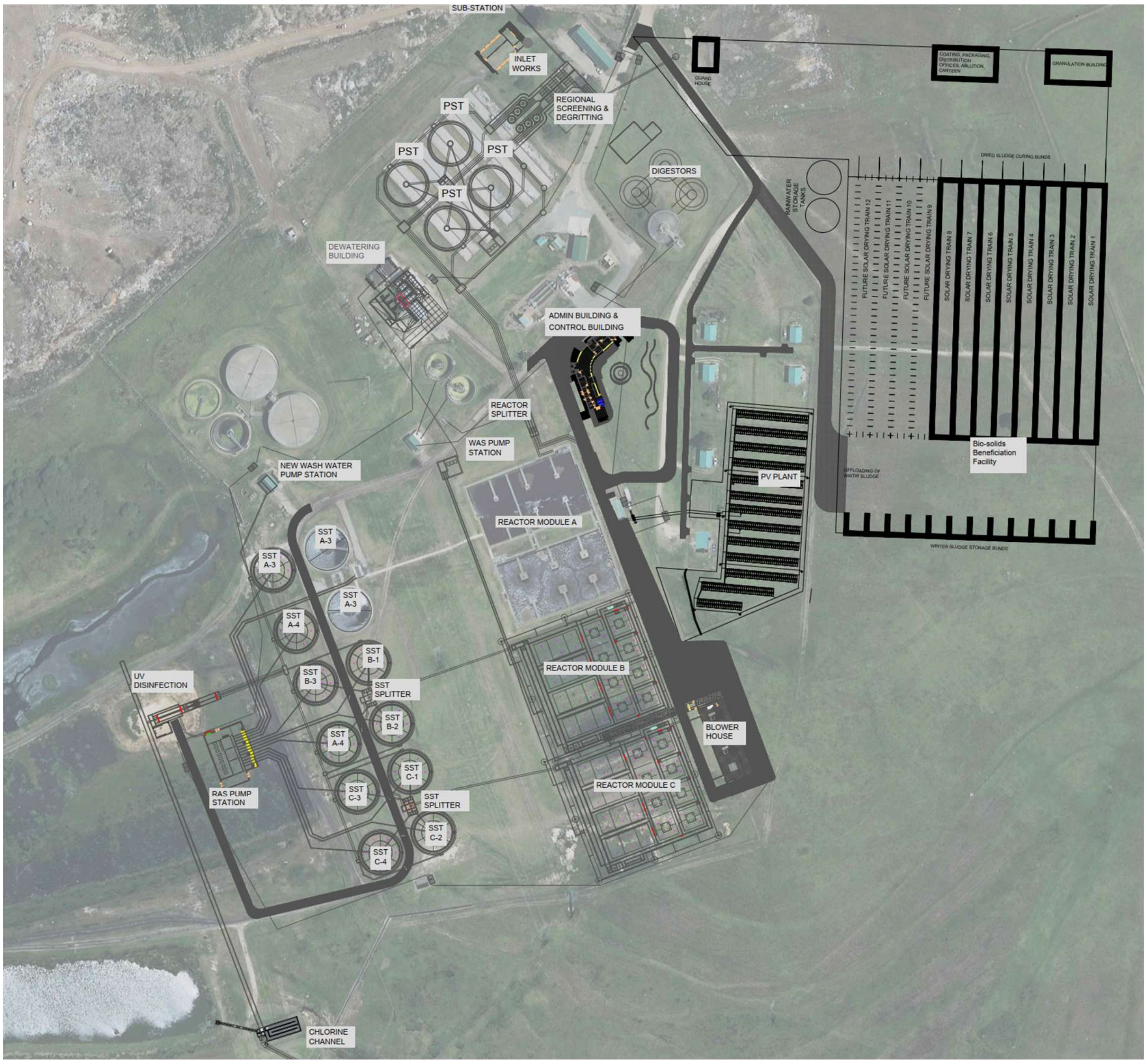
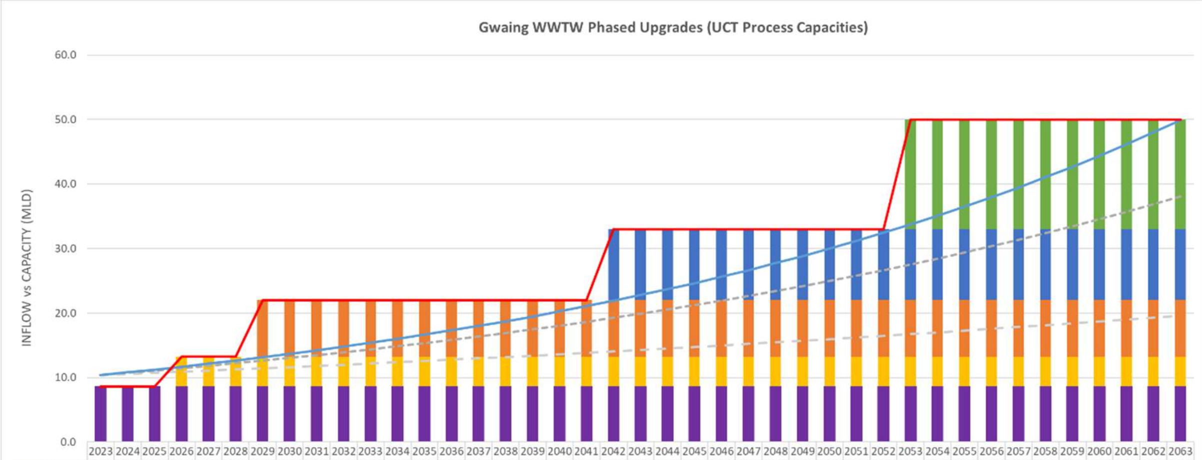


Figure 7-3: Activated sludge module naming convention

7.2 Summary of Ultimate Capacity's Phases

The four phases and their capacities are graphically shown in Figure 7-4 to align with the population growth based on 4% population growth. Note that the existing capacity in Figure 7-4is shown as 8.6 MLD since this graph is produced for the UCT process (and not MLE) since this is the preferred process for the future of Gwaing WWTW.



	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042
Phase D																				
Phase C																				11.0
Phase B							8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8
Phase A				4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6
Current	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6
1.6%	10.4	10.6	10.7	10.9	11.1	11.3	11.4	11.6	11.8	12.0	12.2	12.4	12.6	12.8	13.0	13.2	13.4	13.6	13.8	14.1
3.3%	10.4	10.7	11.1	11.5	11.8	12.2	12.6	13.1	13.5	13.9	14.4	14.9	15.4	15.9	16.4	16.9	17.5	18.1	18.7	19.3
4.0%	10.4	10.8	11.2	11.7	12.2	12.7	13.2	13.7	14.2	14.8	15.4	16.0	16.7	17.3	18.0	18.7	19.5	20.3	21.1	21.9
TOTAL	8.6	8.6	8.6	13.2	13.2	13.2	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	22.0	33.0

	2043	2044	2045	2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063
Phase D											17.0	17.0	17.0	17.0	17.0	17.0	17.0	17.0	17.0	17.0	17.0
Phase C	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0
Phase B	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8	8.8
Phase A	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6
Current	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6	8.6
1.6%	14.3	14.5	14.7	15.0	15.2	15.5	15.7	16.0	16.2	16.5	16.7	17.0	17.3	17.6	17.8	18.1	18.4	18.7	19.0	19.3	19.6
3.3%	19.9	20.6	21.2	21.9	22.7	23.4	24.2	25.0	25.8	26.7	27.5	28.5	29.4	30.4	31.4	32.4	33.5	34.6	35.7	36.9	38.1
4.0%	22.8	23.7	24.6	25.6	26.7	27.7	28.8	30.0	31.2	32.4	33.7	35.1	36.5	37.9	39.5	41.0	42.7	44.4	46.2	48.0	49.9
TOTAL	33.0	33.0	33.0	33.0	33.0	33.0	33.0	33.0	33.0	33.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0

Figure 7-4: Population growth and phases of capacity upgrades

The four phases proposed, with the relevant processes and capacities are summarised in Table 7-2. The commissioning dates for each phase were selected based on a population growth of 4%. The exact dates of implementation will be determined as time progresses and as the demand increase becomes more apparent with actual figures. The 4% growth selected is the worst-case scenario and is used for illustration purposes.

Table 7-2: Summary of phasing capacities

Phase	Process	Date of commissioning (Based on 4% population growth)	Additional Capacity (MLD)	Total Capacity (MLD)
Existing Plant	UCT Raw			8.6
Phase A	UCT Raw	2026	4.6	13.2
Phase B	UCT Raw	2029	8.8	22
Phase C	UCT Raw	2041	11	33
Phase D	UCT Settled	2051	17	50

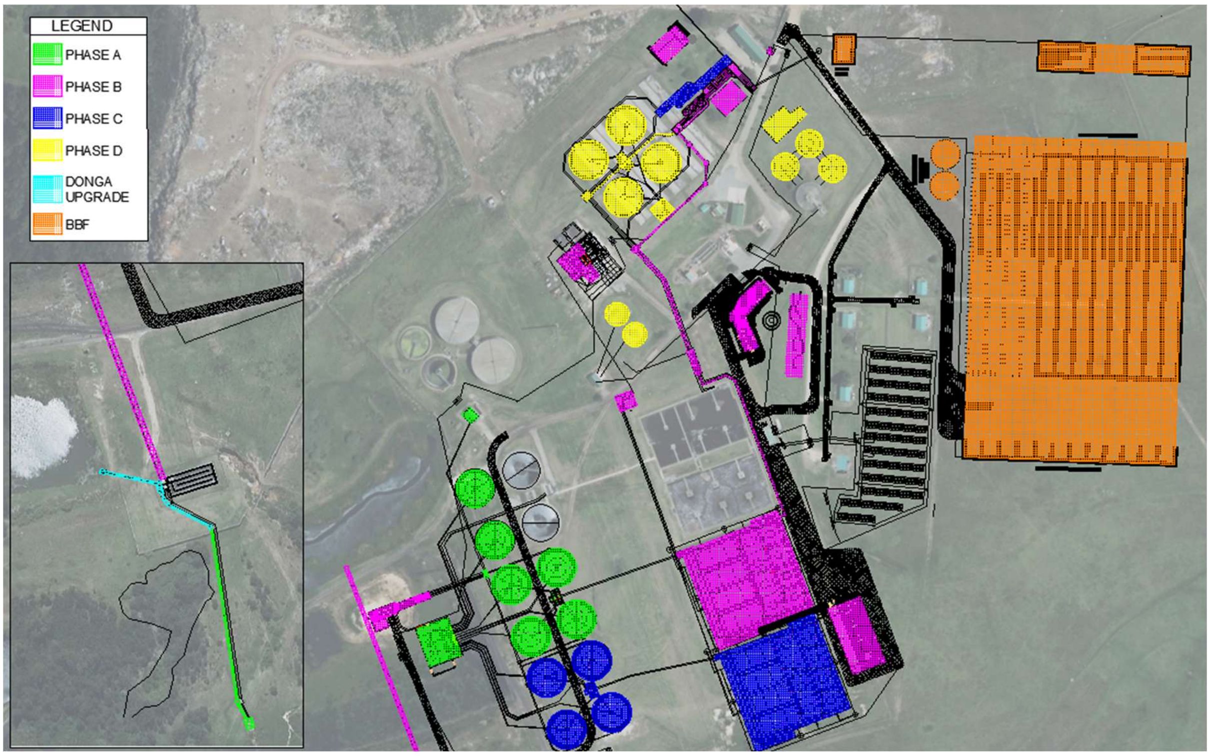


Figure 7-5: Phases of Master Plan Upgrade

7.3 Details of Phases

The phasing and implementation of the total capacity can be approached in various ways. Through various discussions and workshops with George Municipality, the phasing options have been refined. The phasing proposed in this Report is not the only possible but has been carefully considered and selected based on process selection possibilities and suitable upgrade intervals to align with population growth. The Master Plan is divided into 4 separate phases, namely Phases A, B, C and D.

7.3.1 Donga Rehabilitation (Separate Contract)

The Donga Rehabilitation project which includes the upgrading of Maturation Pond No. 4 outlet structure as well as upgrades to the existing Chlorine Contact channel is in the Construction Phase of the project at the time of this report. During the design of the Donga Rehabilitation Project, which includes the new Maturation Pond No. 4 outlet structure, the ultimate capacity upgrade of the Gwaing WWTW according to the Master Plan.

The new outlet pipe from maturation pond No. 4 will be linked directly to the existing chlorine contract tank, discharging into a new concrete stilling chamber with two new penstocks that can control and re-direct flow as necessary. Figure 7-6 below shows the upgrade to the existing chlorine contact channel.

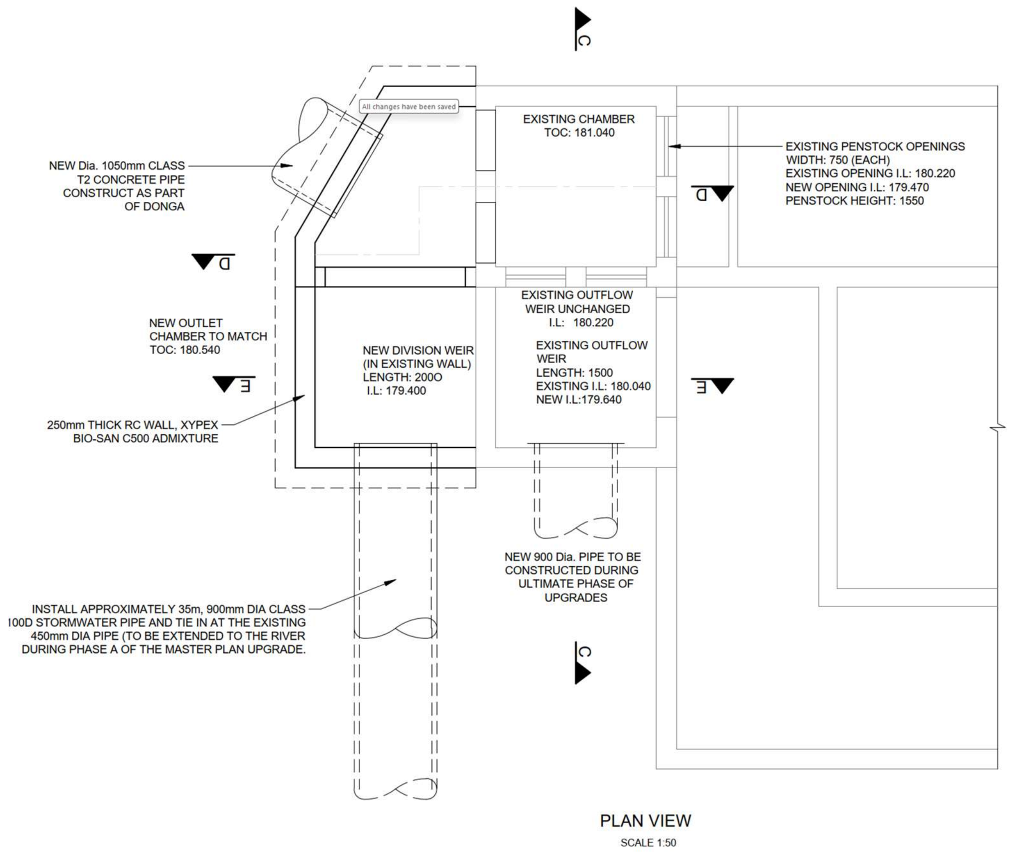


Figure 7-6: Chlorine Contact channel upgrade as part of Donga Rehabilitation Contact

The design of the new stilling chamber at the chlorine contact tank, allows flow to be diverted directly to the outflow pipes, thus bypassing the chlorine contact channel if so required. With the adjustment of existing weir levels at the chlorine contact tank and a new division weir to the new outlet chamber, capacity is improved to handle peak storm weather conditions up to the end of Phase B of the Master Plan upgrade. The design of the new concrete overflow chamber also provides flexibility to isolate areas of the existing chlorine contact channel to ensure that construction during Phase B of the Master Plan upgrade can take place without constructing temporary infrastructure to divert the flow.

7.3.2 Phase A

This phase includes:

- 2 additional SSTs for Module A
- 4 SSTs for Module B (can operate with Reactor A)
- New RAS Pumpstation
- New Substation building
- Replacement of the DN450 with a DN950 pipe from the existing chlorine contact channel to the river outlet.
- Electrical Equipment
- Associated road and stormwater infrastructure

Capacity achieved:

- 13.2 MLD ADWF as a Raw UCT process
- 17 MLD ADWF as Raw MLE process

Gwaing WWTW receives on average around 10.7 MLD is currently operating an MLE system which gives a capacity of 10.4 MLD with no spare or redundant capacity. When operating as a UCT process with the existing infrastructure the capacity is 8.6 MLD. The primary purpose of Phase A is to increase the capacity of the plant in the shortest possible time to ensure the works have enough capacity to sufficiently treat wastewater to comply with effluent requirements. The proposed solution is to construct 6 additional SSTs to operate together with the existing Reactor A.

The 8 SSTs in total, together with Reactor A will give an additional capacity of 4.6MLD (from the existing 8.6 MLD when operating the UCT process) resulting in a total capacity of 13.2 MLD (ADWF). When operated as an MLE process a capacity of 17 MLD can be achieved. The additional infrastructure of Phase A is highlighted in Figure 7-7.

Figure 7-7: Phase A site layout

The outlet structure which discharges the final effluent is in a poor condition and is undersized for the current capacity. Included in Phase A of the upgrade will be the construction of a new outlet chamber sufficient for the ultimate solution. The donga and maturation pond outlet channel to the existing chlorine contact channel will be upgraded on a separate contract due to the urgency of restoring the donga. The pipe and channel sizing and positions as part of the donga upgrade contract will be aligned with the Master Plan upgrade. Figure 7-8 shows the layout of the outlet structure upgrade and the channel from the maturation pond to the chlorine contact channel as part of the donga upgrade.

Figure 7-8: Layout of the outlet structure upgrade

7.3.3 Phase B

There were two options investigated for Phase B of the upgrade. The first option is implementing an additional reactor and operating a UCT system with unsettled wastewater. The second option is to implement primary settling (including all primary sludge handling) and operate a UCT settled process with the existing Reactor A. The two options were compared to each other and workshopped together with George Municipality. The total structures required for the two options are summarised in Table 7-3.

Table 7-3: Phase A + B - Option summary

Unit Process	**Phase A & B: Option 1**	**Phase A & B: Option 2**
Phase A + B Capacity	22 MLD	20.7 MLD
Inlet Works	1	1
Primary Settling Tanks	-	2
Gravity Thickeners	-	2
Anaerobic Digestors	-	2
Biological Reactors	2	1
Secondary Settling Tanks	8	8
Chlorine Contact Tank	1	1

7.3.3.1 Phase B – Option 1 and Option 2 layouts

The layouts of Option 1 and 2 are shown in Figure 7-9 and Figure 7-10 respectively. The figures show the combined infrastructure upgraded in both Phase A and B of each of the two options.

Figure 7-9: Phase A & B - Option 1 Layout

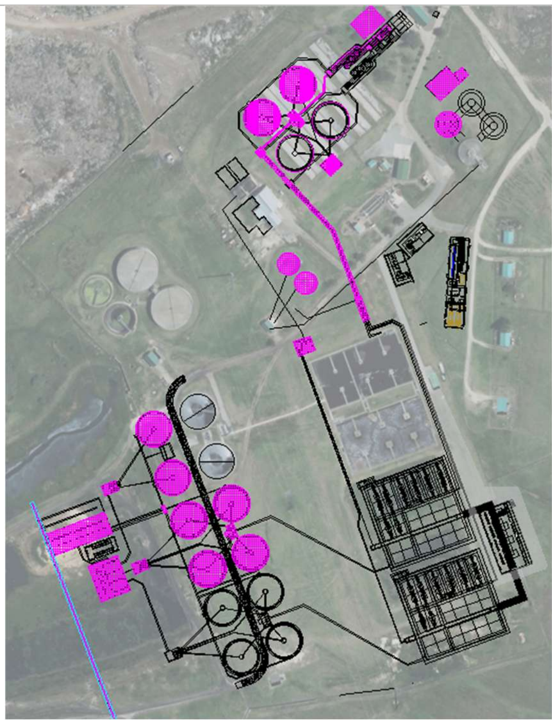


Figure 7-10: Phase A & B - Option 2 Layout

7.3.3.2 Phase B – Option 1 and Option 2 cost comparison

The civil and mechanical and electrical (M&E) costs of the two options are shown in Table 7-4. The total cost of Phase A and B are combined for the cost comparison. Phase A for both options is identical.

Table 7-4: Phase A & B - Option 1 and 2 cost comparison (2025 rates)

Combined Cost Estimate		Phase A & B: Option 1 22 MLD	Phase A & B: Option 2 20.7 MLD
1	Civil Cost Estimate	R386 484 929.93	R311 777 365.66
2	M&E Cost Estimate	R405 741 482.96	R390 797 266.51
	Total	**R792 226 412.89**	**R702 574 632.17**

7.3.3.3 Phase B – Optioneering

An optioneering exercise was conducted at a Workshop with the Consultants, Municipal Project Team and Process Controllers present. The optioneering exercise was conducted to compare key attributes between Option 1 and Option 2 for Phase B of the upgrades. The attributes and their weighting are graphically indicated in Figure 7-11.

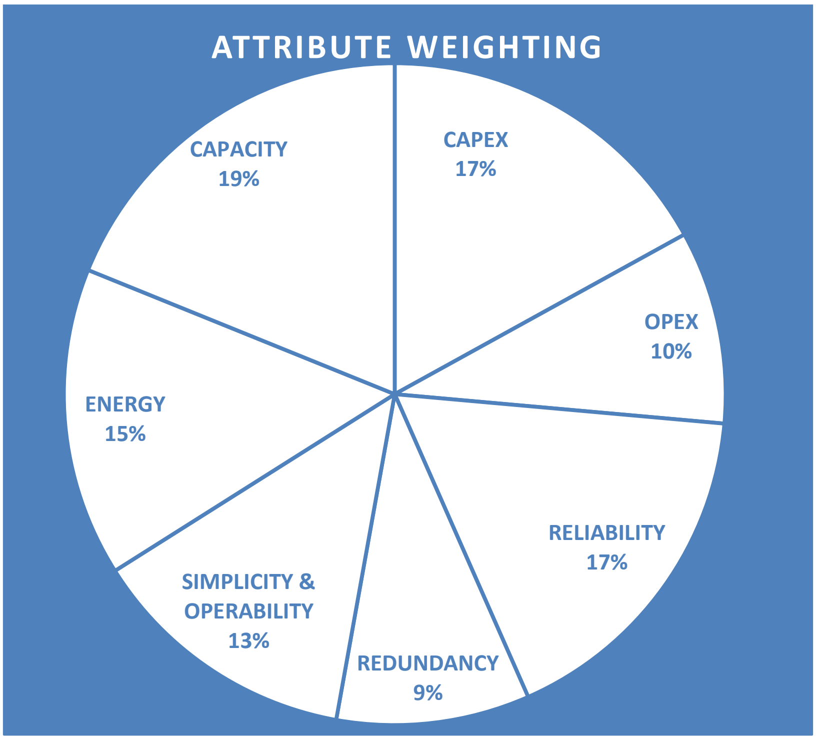


Figure 7-11: Phase B optioneering attribute weighting

Each attribute was discussed and compared between the two options. The result of the optioneering is shown in Table 7-5.

Table 7-5: Phase B Optioneering summary

Attribute	Attribute Weighting 1 to 10	OPTION 1 - New Reactor			OPTION 2 - New PSTs		
		Comments	Score /5	Weighted score	Comments	Score /5	Weighted score
CAPEX	**9**	Total Capex: R792 226 412.89	4	36	Total Capex: R702 574 632.17	5	45
OPEX	**5**	High power requirements increase operational costs	4	20	Energy and heat generation reduce operation costs of digestors.	5	25
RELIABILITY	**9**	Biological reactors and SSTs are a proven and robust technology and very familiar in South African WWTWs.	5	45	Anaerobic digestion has a poor track record in South Africa, with few digestors standing the test of time.	3	27

Attribute	Attribute Weighting 1 to 10	OPTION 1 - New Reactor			OPTION 2 - New PSTs		
		Comments	**Score /5**	**Weighted score**	**Comments**	**Score /5**	**Weighted score**
REDUNDANCY	**5**	Constructing an additional reactor increases redundancy should maintenance be required on either of the reactors.	5	25	If something goes wrong with a digestor or PST, or gravity thickeners, the entire sludge handling unit process will be unable to operate. Thus there is very little redundancy.	2	10
SIMPLICITY & OPERABILITY	**7**	The activated sludge process is familiar and relatively simple to operate.	5	35	Anaerobic digestion is unfamiliar and has various complex components in its operation.	3	21
ENERGY	**8**	The activated sludge process has a high energy consumption as a result of the blowers which introduce oxygen into the process.	2	16	Primary sludge is digested and produces energy and heat which can be used in the process.	5	40
CAPACITY	**10**	Phase A + B: Option 1 = 22 MLD	5	50	Phase A + B: Option 2 = 20.7 MLD	4	40
TOTALS	**53**		**29**	**222**		**27**	**208**

7.3.3.4 Selected Option for Phase B

The optioneering exercise was a qualitative test based on parameters and attributes weighted by the relevant stakeholders. The optioneering exercise resulted in **Option 1** being the preferred option for Phase B. This is in line with the recommendation by the Consultants for reasons as summarised in the optioneering exercise. The Municipality indicated that they are comfortable to move forward utilizing Option 1 for Phase B of the upgrade. (Refer to Figure 7-9)

Therefore, Phase B includes:

- New Inlet Works Train 1
- Regional Grit and Screenings Facility (Construction may be in a later phase or on a separate contract depending on funding availability)
- New biological reactor (Module B)
- New Blower House and aeration system
- Service corridor for air header

- New WAS pumpstation
- New UV disinfection system (Subject to approval by George Municipality)
- Extension to WAS Dewatering Facility
- New Process Control including Admin Building (Construction of Admin Building may be in a later phase or on a separate contract depending on funding availability)
- Electrical Equipment
- Potentially sludge storage bunds and/or sludge drying facility
- Demolition of sludge drying beds
- Associated roads and stormwater infrastructure

Capacity achieved:

- 28 MLD ADWF as MLE
- 22 MLD ADWF as UCT

Phase B will see the construction of a new inlet works (half the ultimate upgrade proposed inlet works), including regional screening and degritting facility, for the washing of screenings and grit from other pumpstations and wastewater treatments works within the Municipal area. An additional reactor (Reactor Module B) will be constructed together with its associated pipework to connect to the SSTs constructed in Phase A. The additional reactor will be aerated with fine bubble diffusers and therefore a blower house will be constructed. UV disinfection and WAS dewatering are also included in the construction of Phase B. Phase B will give an additional capacity of 8.8 MLD from the 13.2MLD achieved in Phase A, resulting in a total capacity of 22 MLD (ADWF). The additional infrastructure of Phase B is highlighted in Figure 7-12.

Figure 7-12: Phase B Site Layout

7.3.4 Phase C

This phase includes:

- 1 New biological reactor (Module C)
- Extension of Blower House and aeration system
- 4 new SSTs (Module C)
- Additional UV banks (M&E) (If approved by George Municipality)
- New Inlet Works Train 2
- Additional DN950 outlet pipe from existing chlorine contact channel to the river outlet
- Electrical Equipment
- Associated roads and stormwater infrastructure

Capacity achieved:

- 42 MLD ADWF as MLE
- 33 MLD ADWF as UCT

Phase C of the upgrade will be to construct Module C's reactor and SSTs. It is proposed to construct the final reactor and SSTs prior to constructing the PSTs and associated primary sludge handling unit processes as all the ancillary infrastructure for the reactors and SSTs would have been constructed as part of Phase B. This includes the Blower House, RAS pump station and WAS pumpstation. It would also give more redundancy with the additional reactor and SSTs should maintenance on any of the existing infrastructure be required. The site layout for the proposed Phase C of the upgrades is shown in Figure 7-13. The total capacity of the plant after the Phase C upgrade will be 33 MLD operating a UCT process.

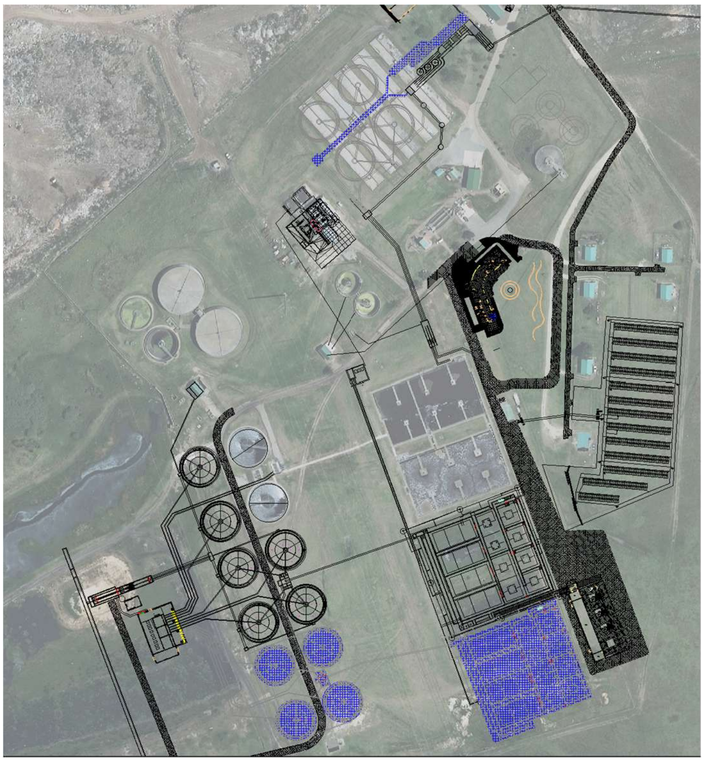


Figure 7-13: Phase C site layout

7.3.5 Phase D

This phase includes:

- 4 New PSTs
- Primary Sludge Pump Station
- 2 Gravity Thickeners (repurpose old PSTs)
- 4 Anaerobic Digesters
- Primary Sludge Dewatering Facility
- Electrical Equipment
- Associated roads and stormwater infrastructure

Capacity achieved:

- 68 MLD ADWF as MLE
- 50 MLD ADWF as UCT

Phase D of the upgrades will be the final phase of the Master Plan. The phase will see the construction of the four PSTs, primary sludge pumpstation and three additional anaerobic digestors. The existing PSTs will be refurbished and used as gravity thickeners for the primary sludge. Phase D will increase the plant's capacity from 33 MLD to 50 MLD, operating a UCT settled process. The sequencing of Phase C and D can be switched around if the Municipality chooses to do so. Switching the two phases will have the same impact on the capacity. Figure 7-14 shows the site layout of the proposed Phase D upgrade.

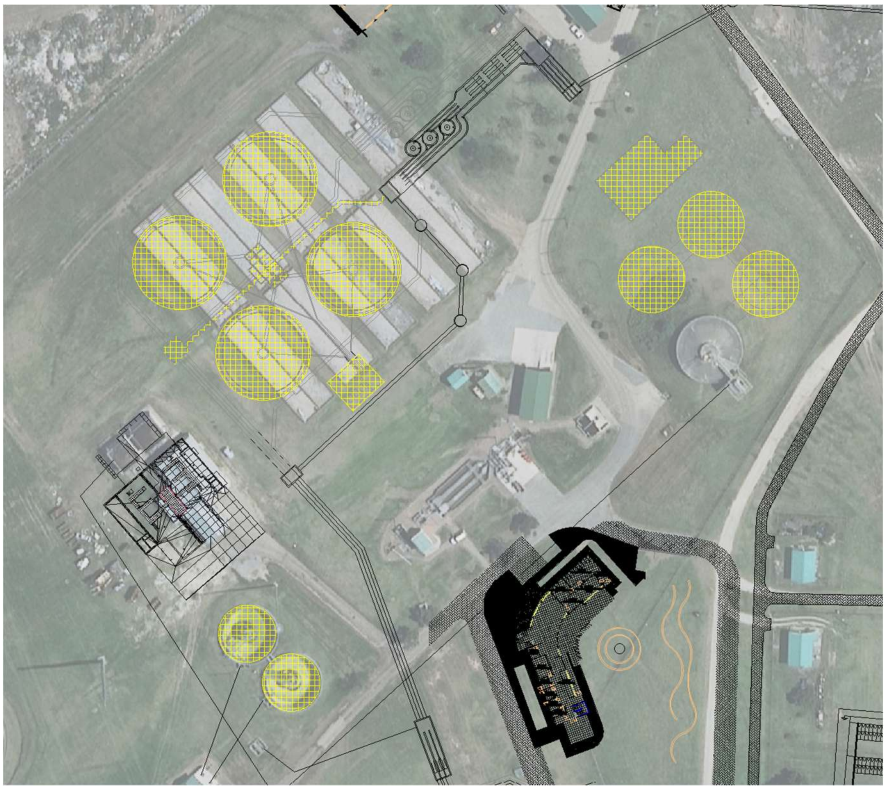


Figure 7-14: Phase D site layout

7.3.6 Biosolids Beneficiation Facility (BBF)

This phase includes the new biosolids beneficiation plant which comprises of the following infrastructure:

i. Guard House
ii. Perimeter fencing and access gate
iii. Approximately 30 000 m^2 of concrete slabs for the various stages of sludge stockpiling, solar drying, composing and sludge handling. This includes the areas under translucent roof sheeting for solar drying.
iv. Approximately 13 000 m^2 in plan view of translucent roof sheeting ('greenhouse') structures.
v. One 18m x 36m shed with a clear height of 4.5m and without any columns inside the building for the sludge granulation plant.
vi. A second building of similar footprint for the packaging plant and distribution depot. This building is to include offices, ablution and a canteen for the operating staff of approximately 6 people.
vii. Movable precast concrete walls placed on slabs to demarcate separated process areas and to prevent contamination of treated sludge by raw sludge.
viii. Access Roads
ix. Rainwater collection and storage from all roof structures
x. Stormwater collection and drainage from concrete slabs with pipeline to Gwaing WWTW inlet works.

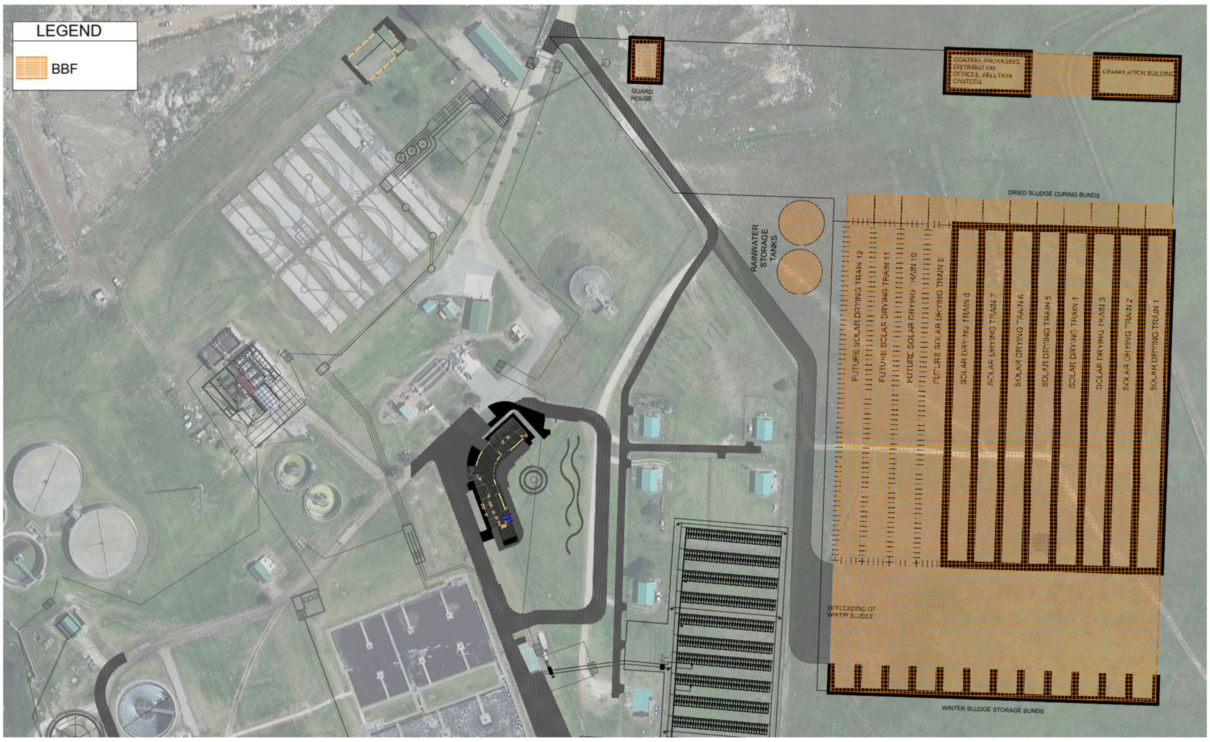


Figure 7-15: Layout of the Bio-Solids Beneficiation Facility phase layout

8 FINANCIAL

8.1 Capital Cost Estimate

A conceptual-level estimate of the four phases of the Master Plan was compiled. The estimate was based on rates from similar projects completed in recent years with relevant escalation values. All values are current values, although the project will extend over several years, the values exclude any contract price adjustment (CPA). All costs are shown excluding VAT. A breakdown of the Civil capital costs is shown in Table 8-1 and the M&E capital costs of each unit process are shown in Table 8-2 below.

Table 8-1: Civil capital cost estimate

				13.2 MLD UCT	22 MLD UCT	33 MLD UCT	50 MLD UCT
	Civil Phasing Cost Estimates - 2025 Rates	**TOTAL COST 50 MLD**	**BBF Construction**	**Phase A**	**Phase B**	**Phase C**	**Phase D**
1	Site Works	R 20 548 241.11		R 6 164 472	R 14 383 769	R -	R -
2	Raw Sewer Pipelines (Re-Routing)	R 7 412 052.12		R -	R 7 412 052	R -	R -
3	Inlet Works	R 25 221 507.38		R -	R 12 610 754	R 12 610 754	R -
4	Primary Settling Tanks	R 32 806 872.80		R -	R -	R -	R 32 806 873
5	Anaerobic Digestors (3off.)	R 25 750 686.00					R 25 750 686
6	Refurbish Existing Digestor	R 3 239 080.00					R 3 239 080
7	Primary Sludge Dewatering Building	R 7 079 499.08					R 7 079 499
8	Refurbish Existing PST's into Thickeners	R 5 846 876.11					R 5 846 876
9	Primary Sludge Pumpstation	R 4 204 914.12					R 4 204 914
10	Thickener Pumpstation (Refurbish Existing)	R 1 388 415.04					R 1 388 415
11	Reactor - Module B	R 79 660 681.19		R -	R 79 660 681	R -	R -
12	Reactor - Module C	R 79 660 681.19		R -	R -	R 79 660 681	R -
13	SST's - Modula A (2off.)	R 14 475 338.80		R 14 475 339	R -	R -	R -
14	SST's - Modula B (4off.)	R 28 950 677.60		R 28 950 678	R -	R -	R -
15	SST's - Modula C (4off.)	R 28 950 677.60		R -	R -	R 28 950 678	R -
16	UV Channels	R 5 173 307.79		R -	R 5 173 308	R -	R -
17	UV Electrical Building	R 1 206 716.74		R -	R 1 206 717	R -	R -
18	RAS Pumpstation	R 10 861 574.41		R 10 861 574	R -	R -	R -
19	WAS Pumpstation	R 653 654.30		R -	R 653 654	R -	R -
20	Wash Water Pumpstation (Refurbish existing)	R 839 508.98		R 839 509	R -	R -	R -
21	Extension of WAS Dewatering Building	R 6 745 437.35		R -	R 6 745 437	R -	R -
22	Guard House	R 345 476.94		R -	R 345 477	R -	R -
23	Blower House & Electrical Building	R 34 658 247.09		R -	R 34 658 247	R -	R -
24	Admin Building	R 31 235 181.12		R -	R 31 235 181	R -	R -
25	Interconnecting Pipework	R 29 505 174.20		R 8 261 449	R 6 491 138	R 5 901 035	R 8 851 552
26	Channels & Chambers - PST's	R 3 596 580.00					R 3 596 580
27	Channels & Chambers - SST's & Reactors	R 9 681 690.28		R 2 904 507	R 6 777 183	R -	R -
28	Demolition Work	R 18 543 552.83		R 4 635 888	R 8 344 599	R 5 563 066	R -
29	Biosolids Beneficiation Facility	R 124 515 792.02	R 124 515 792	R -	R -	R -	R -
	Sub Total	**R 642 758 094.20**	**R 124 515 792**	**R 77 093 416**	**R 215 698 197**	**R 132 686 213**	**R 92 764 475**
	PRELIMINARY AND GENERAL	R 128 551 619	R 24 903 158	R 15 418 683	R 43 139 639	R 26 537 243	R 18 552 895
	CONTINGENCIES	R 77 130 971	R 14 941 895	R 9 251 210	R 25 883 784	R 15 922 346	R 11 131 737
	TOTAL Excl VAT:	**R 848 440 684.35**	**R 164 360 845**	**R 101 763 309**	**R 284 721 621**	**R 175 145 801**	**R 122 449 108**

Table 8-2: M&E capital cost estimate

	M&E Equipment Phasing Cost Estimates - 2025 Rates			BBF Construction	13.2 MLD UCT	22 MLD UCT	33 MLD UCT	50 MLD UCT
	Description		50 MLD TOTAL		Phase A	Phase B	Phase C	Phase D
1	Inlet Works		R 65 741 616			R 34 107 928	R 31 633 688	R -
2	PSTs		R 14 912 050				R -	R 14 912 050
3	Thickeners		R 9 179 514				R -	R 9 179 514
4	Anaerobic Digestion		R 41 987 508				R -	R 41 987 508
5	New Reactor - Module B		R 11 190 344			R 11 190 344	R -	R -
6	New Reactor - Module C		R 11 190 344				R 11 190 344	R -
7	Blowers and Aeration		R 78 744 780			R 56 701 620	R 22 043 160	R -
8	New WAS Pump Station		R 2 501 380			R 2 052 460	R 448 920	R -
9	SSTs - Module A		R 5 501 816		R 5 501 816		R -	R -
10	SSTs - Module B		R 11 003 632			R 11 003 632		R -
11	SSTs - Module C		R 11 003 632				R 11 003 632	R -
12	Washwater Pumpstation		R 2 363 132		R 2 363 132		R -	R -
13	RAS Pump Station		R 8 833 850		R 6 354 700		R 2 479 150	R -
14	WAS Dewatering Equipment		R 44 733 844			R 29 269 844	R 15 464 000	R -
15	Disinfection		R 107 147 668			R 42 910 560	R 19 824 679	R 44 412 430
16	Biosolids Beneficiation Facility		R 47 000 000	R 47 000 000				
	Subtotal:		**R 473 035 111**	**R 47 000 000**	**R 14 219 648**	**R 187 236 388**	**R 114 087 573**	**R 110 491 503**
	Electrical Equipment		R 130 666 144	R 10 000 000	R 37 073 356	R 68 850 519	R 7 744 790	R 6 997 479
	Preliminary and General	20%	R 120 740 251	R 11 400 000	R 10 258 601	R 51 217 381	R 24 366 472	R 23 497 796
	Contingencies	10%	R 72 444 151	R 6 840 000	R 6 155 161	R 30 730 429	R 14 619 883	R 14 098 678
	TOTAL Excl VAT:		**R 796 885 657**	**R 75 240 000**	**R 67 706 766**	**R 338 034 717**	**R 160 818 718**	**R 155 085 456**

The combined civil and M&E cost estimate for each phase is summarised in Table 8-3.

Table 8-3: Combined Civil and M&E cost estimate

Combined Cost Estimate - 2025 Rates	TOTAL COST 50 MLD	BBF Construction	Phase A - 13.2 MLD UCT	Phase B - 22 MLD UCT	Phase C	Phase D
Civil Cost Estimate	R 848 440 684.35	R 164 360 845	R 101 763 309	R 284 721 621	R 175 145 801	R 122 449 108
M&E Cost Estimate	R 796 885 657.02	R 75 240 000	R 67 706 766	R 338 034 717	R 160 818 718	R 155 085 456
TOTAL Excl VAT:	**R 1 645 326 341**	**R 239 600 845**	**R 169 470 075**	**R 622 756 338**	**R 335 964 520**	**R 277 534 563**

8.2 Operation and Maintenance Costs

The following inputs were considered for operation and maintenance costs:

- Maintenance costs – 1.5% of capital value for M&E and 1% of capital value for civil
- Polymer usage for sludge dewatering – 5 kg/ton TSS
- Electrical cost based on kilowatts of equipment, operational hours and R2.16/kWh (value of R2.04/kWh in 2024 escalated by 6% to 2025 value)
- Inflation at 6% to determine future value costs (at date of implementation)

Table 8-4 represents the annual chemical costs per phase, Table 8-5 represents the annual electrical costs per phase and Table 8-6 represents the annual maintenance costs for the four phases of Gwaing WWTW upgrades.

Table 8-4: Annual Chemical Costs (Polymer)

Parameter	BBF (2028)	Phase A&B (2028)	Phase C (2041)	Phase D (2051)
Capacity (MLD)		22	33	50
Polymer				
Cost/day (R/d)		R1 656.66	R2 470.01	R2 173.27
R/a - 2025		R604 680.69	R901 555.17	R793 245.23
R/annum in future value *		R720 184.38	R2 427 683.24	R3 825 302.93
*** in the year shown in the column heading**				

Table 8-5: Annual Electrical Costs

Parameter	BBF (2028)	Phase A&B (2028)	Phase C (2041)	Phase D (2051)
Capacity (MLD)		22	33	50
Cost/day (R/d)	R20 865.60	R74 538.65	R104 237.08	R113 726.52
R/a – 2025	R7 615 944.00	R27 206 606.27	R38 046 532.62	R41 510 180.54
R/annum in future value *	R9 070 711.16	R33 862 106.13	R63 051 564.78	R79 754 473.26
*** in the year shown in the column heading**				

Table 8-6: Annual Maintenance costs

Parameter	BBF (2028)	Phase A&B (2028)	Phase C (2041)	Phase D (2051)
Capacity (MLD)		22	33	50
R/a - 2025	R2 772 208.45	R9 950 971.54	R14 114 710.33	R17 665 483.24
R/annum in future value *	R3 301 744.62	R11 851 766.32	R23 736 488.97	R41 886 927.63
*** in the year shown in the column heading**				

9 URBAN ECONOMIST

Urban economists, *Conningarth Economists*, were appointed to conduct an economic feasibility assessment and socio-economic impact of the Gwaing WWTW upgrade for George Municipality. Their findings are captured in the report titled *George Municipality upgrade of the Gwaing WWTW – Economic Feasibility Assessment and Socio-Economic Impact of the WWTW Project for the Municipality of George,* dated 5 February 2025. This section summarizes the aim and conclusion of the abovementioned report.

9.1 Aim of Investigation

The project assignment calls for performing a Financial Cost Benefit Analysis, underpinned by an Economic Costs Benefit Analysis (ECBA) to determine the financial and economic feasibility of the upgrade to the Gwaing Wastewater Treatment Works (WWTW). In order to develop the Financial Cost Benefit Analysis, it was regarded as of crucial importance that a Pure Financial Analysis, also referred to as a Cash Flow Analysis, be undertaken. The Financial Analysis factor in key aspects such as an optimal tariff structure, affordability and ability to pay by various households and water users as well as the sustainability of the upgrade of the project.

Furthermore, a socio–economic impact assessment was performed to determine the socio–economic impacts of the project. In the final instance a Bankability and Funding analysis was performed to determine the possible funding options.

A sensitivity and risk analysis was also performed to determine the financial and economic impacts under different population growth and service demand scenarios.

9.2 Conclusions

The George LM is planning the expansion of the existing Gwaing Wastewater Treatment Works (WWTW) to accommodate the projected increase in wastewater. The planning of this intended expansion of the WWTW is the result of a high population growth rate of about 4% per annum. Wastewater treatment is essential to remove contaminants from wastewater to a level where the effluent is suitable to be discharged to the surrounding environment or possible reuse, thereby preventing water pollution from raw sewage discharges.

Lukhozi Engineers has recommended a four phased approach for expanding the WWTW to accommodate the fact that the future population growth cannot be predicted accurately. Three different population growth rates namely 4%, 3,26% and 1,6% were used for planning purposes. Conningarth added a fourth one based on a downward stepped population growth rate.

A financial analysis was performed consisting of a Financial Cost Benefit Analysis (FCBA) and Cash Flow Analysis for all four options. The FCBA and Cash Flow Analysis produced positive sets of answers for all the options.

An Economic Cost Benefit Analysis (ECBA) and Socio-Economic Impact Analysis was undertaken for the proposed phases and the four options to determine the economic viability of the proposed expansion of the Gwaing WWTW. The results of the ECBA and Socio-Economic impact analysis also provide positive economic results.

A detailed Sensitivity and Risk analysis was also performed and within the risks of a downward stepped population growth scenario, a set of results were obtained.

The outcome of both the financial analysis and the economic analysis support the proposed extension of the Gwaing WWTW, subject to the projected population growth models as used in the analyses.

9.2.1 Way Forward

It is recommended that the population growth tendencies be closely monitored, and the construction programme be adjusted accordingly.

It is recommended that the implementation of the proposed upgrade of the Gwaing WWTW commence and be implemented, taking full cognisance of the population growth patterns. The implementation of at least phases A and B should, in view of the current demand, as well as the short to medium term population growth rate projections, be started simultaneously.

It is recommended that, in view of the fact that the project proves to be bankable and can be considered for loan funding, the municipality should explore the possibility of a funding mix, making use of available existing intergovernmental grant transfers to partly fund the development associated to providing services to communities and loan funding where loan conditions will be appropriate for a loan product suitable to the Municipality's borrowing capacity.

10 CONCLUSION AND RECOMMENDATIONS

The Gwaing WWTW Master Plan serves as the blueprint for the plant's future upgrades over the next 40 to 50 years. It provides guidance for informed decision-making regarding the facility. By considering the entire scope, the full impact of each decision can be appreciated.

The vision for Gwaing WWTW extends beyond waste management. It aims to transform the facility into a Water Resource Recovery Facility (WRRF), emphasizing resource recovery. Key strategies include:

- Regional grit processing facilities to enable reuse of grit as part of composting or fill material.
- Regional screenings processing facility to minimise volume, odours, pathogens and vector attraction of screenings.
- Sludge beneficiation in the form of composting or fertilizer production is envisaged.
- The methane gas produced from anaerobic digestion will be used for generating heat and power (as part of Phase D).
- Effluent from the Gwaing WWTW can in future be pumped to neighbouring industries or golf courses for non-potable use. Alternatively, it can be further treated together with the effluent from Outeniqua WWTW before it is pumped to the Garden Route Dam as part of an indirect potable reuse scheme.
- Effluent will be recycled and pressurized on site in a wash water ring main for various uses including irrigation, reducing the potable water demand of the WWTW.
- Energy efficient design principles will be used to reduce the power consumption of the plant, while a solar PV plant will both provide backup power during loadshedding events and shift the plant's reliance from the national grid to renewable energy sources.

All the while it will remain important to ensure that the primary task of Gwaing WWTW, which is to produce compliant final effluent, is executed effectively and consistently. The design approach is therefore not to (simply) sacrifice reliability at the expense of secondary goals such as energy efficiency or automation. Two examples of design decisions that were made on this basis include:

- Surface aeration will be maintained initially in Reactor A even though there would be a 50% energy saving by replacing it with FBDA. Surface aeration is a much more simple - and therefore reliable - technology and for this reason (as well as the sloped floors) it was decided to keep surface aeration for Reactor A while including FBDA for Reactors B and C to obtain the energy efficiency benefits.
- Including PSTs and anaerobic digestion (AD) has a significant theoretical energy savings advantage over reactors without PSTs for plants above 25 MLD capacity. However, AD has a bad track record in South Africa due to several operational aspects discussed briefly in this report. While PSTs and AD do form part of the Master Plan for Gwaing WWTW, these unit processes are intentionally delayed until Phase D to ensure that the scale of the plant at the time of implementation warrants sufficient operational resourcing and attention for it to succeed.

The fact that Gwaing WWTW and Outeniqua WWTW are only 4 km apart has several advantages. It is proposed that the benefits of centralisation and economies of scale be harnessed in the following ways:

- Continue to use Gwaing WWTW as a centralized sludge dewatering and beneficiation location for both WWTWs in the region as well as other WWTWs in the district.
- Re-establish a centralized effluent reuse plant at Outeniqua WWTW and include pumping of effluent from Gwaing WWTW to Outeniqua WWTW if required. This can include industrial reuse, irrigation and indirect potable reuse schemes.
- Establish cross connection for raw sewage to be transferred (pumped) between the two WWTWs to shift load from the one plant to the other during planned maintenance periods or unforeseen operational issues. Alternatively, this flexibility can be provided further upstream in the sewerage reticulation network.

It is recommended that the phasing be implemented as discussed in Section 7.3 of this Master Plan. It would be beneficial to complete the detail design of Phases A and B as soon as possible as this will fix the design and pave the way for the implementation of the rest of the Master Plan. Completed detail designs will also bring the Municipality closer to implementation readiness that will make it easier to secure funding for the upgrades.

The Gwaing WWTW is operating at the edge of its capacity and therefore at least Phase A must be accelerated to implementation as soon as possible for the effluent from the works to remain compliant. The detail design and planning for Phase B should also not be delayed to ensure that this phase can be commissioned before 2029 when the load on the plant is projected to exceed the capacity created by the implementation of Phase A. It would make sense to procure Phases A and B simultaneously, but to prioritize the scope of Phase A during implementation of this project.

The Gwaing BBF is poised to transform the way sludge is handled and perceived in the local market. New regulations are making the beneficiation of sludge a necessity. The Gwaing BBF will ensure that sludge handling complies to regulations and will facilitate a circular economy for sludge.

11 APPENDICES

11.1 Appendix A: Geotechnical Report

11.2 Appendix B: GLS Electrical Capacity Investigation Study

APPENDIX A: GEOTECHNICAL REPORT

APPENDIX B: GLS ELECTRICAL CAPACITY INVESTIGATION STUDY