GWAING WWTW

CONCEPT DESIGN REPORT: PHASE A & B

REV02

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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 Spatial Development Framework
GT	Gravity Thickener
IGV	Inlet Guide Vane
IPR	Indirect Potable Reuse
ISS	Inert Suspended Solids
JHB	Johannesburg
LCE	Lukhozi 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, has a total average dry weather flow (ADWF) capacity of 8.6 million litres per day (MLD). Operating with a UCT process, the plant is overloaded at a design chemical oxygen demand (COD) concentration (95th percentile) of 782 mgCOD/l. Currently, the plant receives an ADWF of 10 MLD. Due to population growth in George, expanding 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 firstly create a Master Plan that will guide future upgrades in appropriate phases. The Master Plan has been completed and this Concept Design report shifts the focus towards becoming implementation-ready for the short- and medium-term requirements identified in the Master Plan. GM appointed Lukhozi Consulting Engineers (Pty) Ltd (LCE) to do the detail design for upgrades to achieve a capacity of 21 MLD. These requirements are encapsulated in the first two phases of the ultimate capacity design, namely Phase A and Phase B that is shown to achieve an ADWF capacity of 22 MLD.

Phase A represents the solution that can be implemented the soonest and most cost effectively to increase the capacity of the works by a meaningful margin. This entails the construction of the four SSTs associated with Reactor B as well as an additional two SSTs for Reactor A, with all these SSTs operating with Reactor A until Phase B is implemented. The main infrastructure included in Phase A is:

- 2 additional SSTs for Module A
- 4 SSTs for Module B (can operate with Reactor A)
- New RAS Pumpstation
- New MV Substation building

The overall capacity achieved by implementing Phase A is as follows:

- 13.2 MLD ADWF as a Raw UCT process

The heart of the Phase B upgrades is an additional biological reactor (B) that will be equipped with energy efficient fine bubble diffused aeration (FBDA) equipment and will boast with the flexibility to operate several different process configurations. The biological reactor is a conventional activated sludge (CAS) system that can facilitate COD and nutrient (N and P) removal. The UCT, MUCT and JHB processes are enhanced biological phosphate removal (EBPR) processes that can be facilitated in the reactor along with the MLE process that excludes EBPR.

Since Gwaing WWTW is earmarked to remain the central point for sludge dewatering and beneficiation, Phase B also includes necessary upgrades to the WAS dewatering plant. Sludge beneficiation options including primarily composting and/or solar drying for fertilizer production is discussed.

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, PSTs 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. Hence a new inlet works is included in Phase B that includes improved screening and degritting as well as a regional grit and screenings receiving facility.

The main infrastructure included in Phase B is:

- New Inlet Works Train 1
- Regional Grit and Screenings Facility
- New biological reactor (Module B)
- New Blower House and aeration system
- New WAS pumpstation
- New UV disinfection system
- Extension to WAS Dewatering Facility
- New Process Control (Admin) Building
- Electrical Equipment
- Potentially sludge storage bunds and/or sludge drying facility

The overall capacity achieved by implementing Phase A and B is as follows:

- 22 MLD ADWF as Raw UCT Process

The capital costs for Phase A and B Gwaing WWTW upgrades are shown in Table 1-1.

Table 1-1: Combined Civil and M&E capital cost estimate for Phases A and B the BBF Construction

Combined Cost Estimate - 2025 Rates		TOTAL COST Phase A & B & BBF (22MLD)		BBF Construction		Phase A 13.2 MLD UCT		Phase B 22 MLD UCT
Civil Cost Estimate	R	550 845 775.40	R	164 360 845	R	101 763 309	R	284 721 621
M&E Cost Estimate	R	480 981 482.96	R	75 240 000	R	67 706 766	R	338 034 717
TOTAL Excl VAT:	**R**	**1 031 827 258**	**R**	**239 600 845**	**R**	**169 470 075**	**R**	**622 756 338**

Figure 1-1 below shows the layout for Phase A and B of the Gwaing WWTW upgrade.

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 solar drying and 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 are 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.

Figure 1-1: Site layout showing new infrastructure for Phase A, and Phase B and the BBF Facility.

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.

The completion of the detailed design for Phases A and B will solidify the design and pave the way for implementing the remaining phases identified in the Master Plan. 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 ensuring 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 implementation and phasing is subject to availability of funding by the Municipality.

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. The first work package entailed the completion of a Master Plan that will serve as a guiding document for all current and future upgrades to Gwaing WWTW. This Concept Design Report describes the proposed design to achieve the required capacity of 21 MLD. The Report is the main deliverable for Stage 2 - Concept and Viability of Civil, Mechanical, Electrical and Electronic Engineering Services for Lukhozi Consulting Engineers' appointment under T/ING/010/2020 - Project no: 22, Work package no.3.

2.2 Purpose of Report

The Concept Design Report elaborates on the comprehensive strategy outlined in the Master Plan for the phased upgrading of the Gwaing Wastewater Treatment Works (WWTW). This report zeroes in on the specifics of the infrastructure development necessary to achieve the required short to medium term capacity of 21 MLD. It delineates two main stages of development, referred to as Phase A and Phase B. The execution of these phases, whether they proceed as distinct stages or are combined or further segmented, will be contingent upon budgetary allocations.

This report provides an analysis of the study area, including wastewater flow and characteristics. It offers an evaluation of the current plant infrastructure, examining its capacity for treatment and its compatibility with the envisioned future expansion of Gwaing's treatment capabilities. The report details the process design and reiterates the phased approach as stipulated in the Master Plan. Furthermore, it outlines the proposed new infrastructure, itemizes the associated costs, and discusses the projected timeline for implementation.

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.

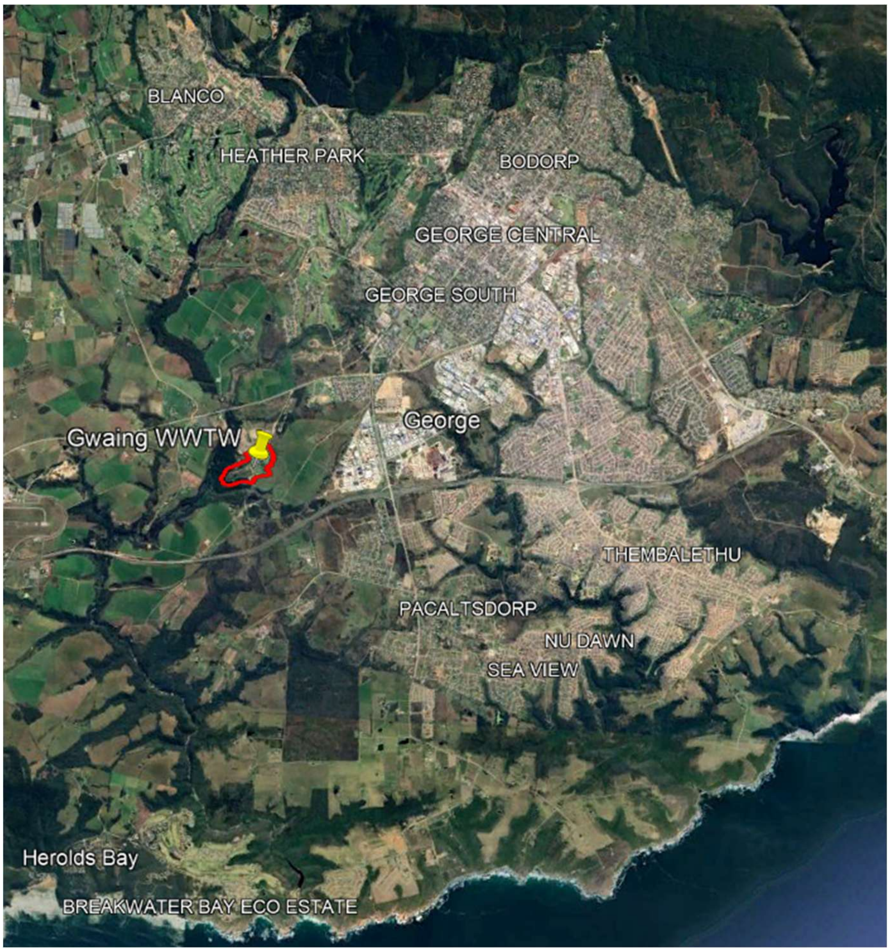


Figure 3-1: Gwaing WWTW Location (Google Earth)

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.

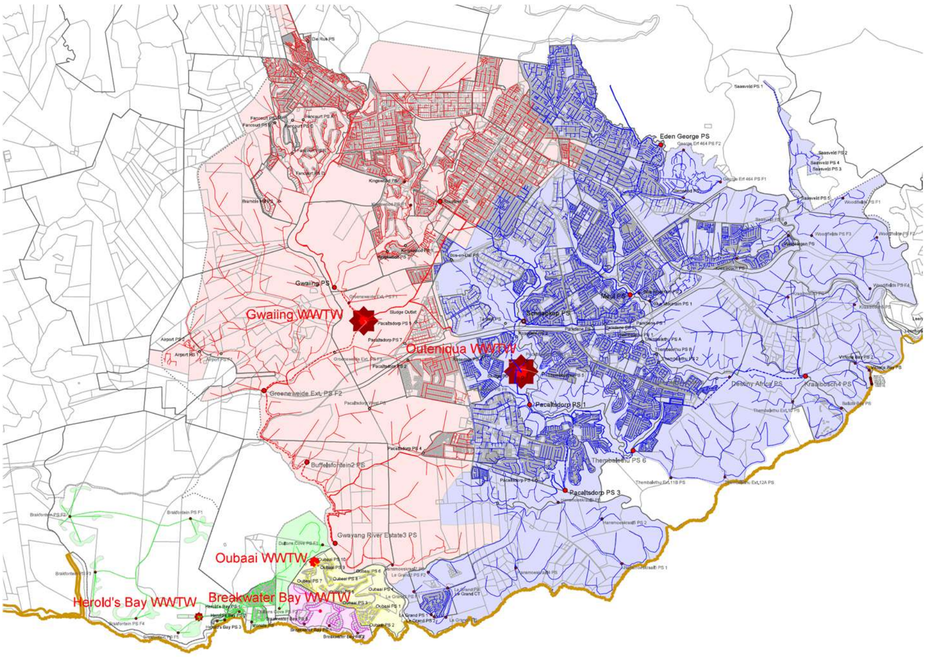


Figure 3-2: Gwaing Catchment Drainage Area (Master Planning of George Sewer Plan, 2022)

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 Gqeberha (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 the 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 6.

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 shows 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.

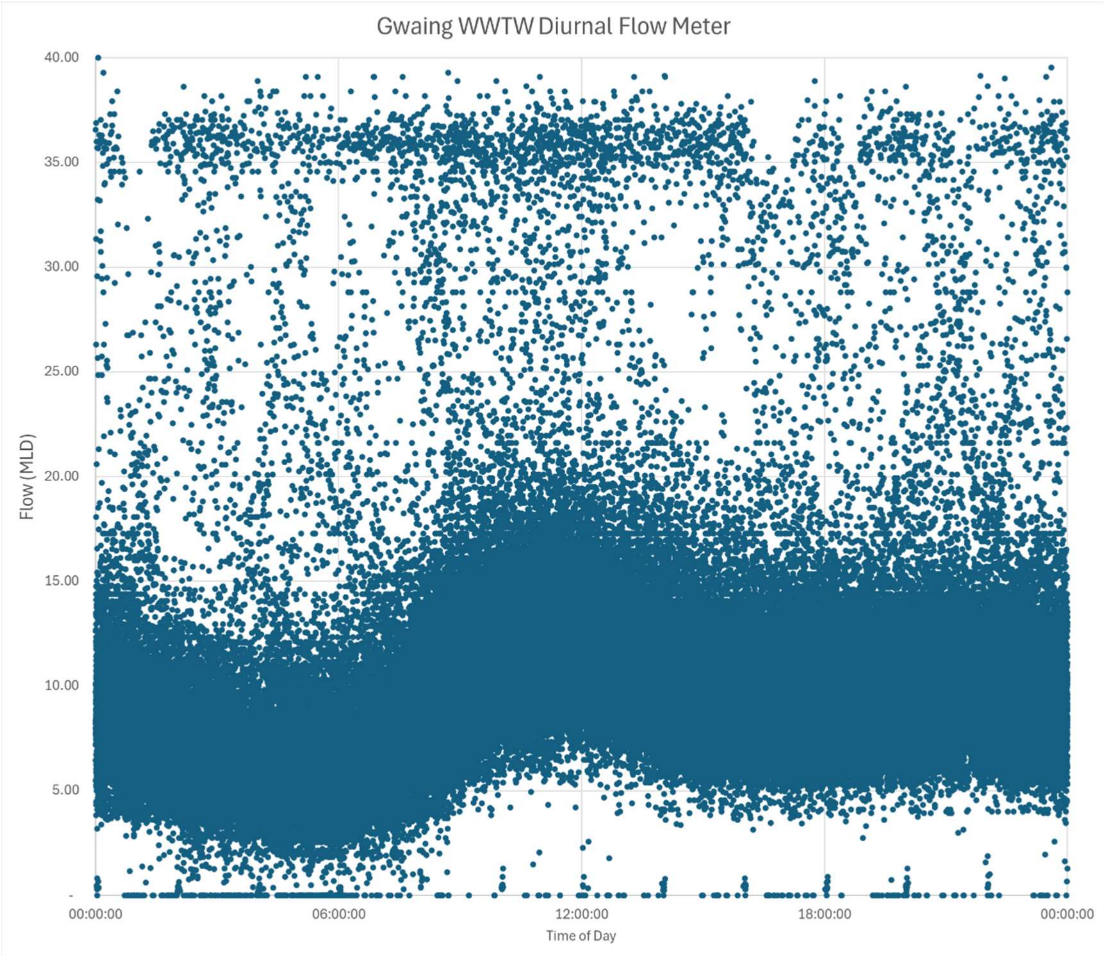


Figure 3-4: Diurnal flow pattern based on new level sensors at the inlet works Parshall flumes

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 design flow for this report as required by George Municipality is 21 MLD as this is the required capacity of Gwaing WWTW to be achieved in the short to medium term. Due to the practicalities of sizing the modules in will be shown that Phases A and B as defined in Section 6 amount to 22 MLD.

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 design purposes of Phase A and B is 30%. 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 22 MLD are summarised in Table 3-4.

Table 3-4: Phase A and B flow summary

Characteristic	Unit	Process & Hydraulic Design	Hydraulic Design	Inlet Works Hydraulic Design
Equivalent per capita flow rate	l/p/d	100	100	
Population equivalent	Pax	220 000	220 000	
Harmon Peaking Factor (PF) PDWF/ADWF Factor		2	2	
Stormwater infiltration		30%	30%	
PWWF/PDWF Factor		1.3	1.3	
PWWF/ADWF Factor		2.5	2.5	3.0
Average Dry Weather Flow (ADWF)	MLD	22	22	22
Peak Dry Weather Flow (PDWF)	MLD	42	42	
Peak Wet Weather Flow (PWWF)	MLD	55	55	66

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.

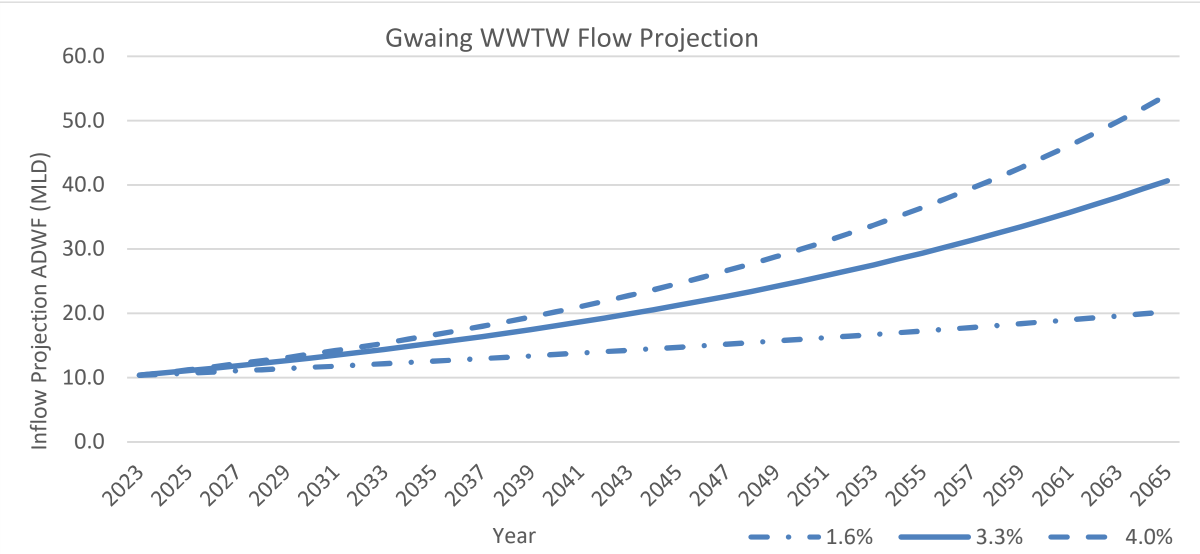


Figure 3-5: Gwaing WWTW flow projection graph

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.

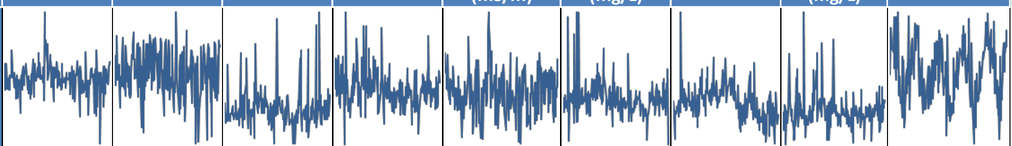
3.4 Wastewater Influent Characteristics

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 PSTs, although this will affect the TKN/COD ratio negatively, increasing effluent nitrate or phosphate concentrations. The current raw incoming COD load is therefore 10 MLD x 782 mgCOD/l = 7820 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 by 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 by 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 95^{th} percentile TKN concentration calculated as 78.2 mgN/l and using a typical ammonia/TKN ratio of 0.75, the influent ammonia concentration (95^{th} 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 design are summarised in Table 3-8. The Raw data applies to Phases A and B, although the reactors are designed with the addition of PSTs in the future in mind.

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-Phosphates	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 7.11.

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.
9. **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, functionality will be included to operate the plant as an MLE process (and other processes) when additional capacity is required. 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 a COD concentration of 700 mg/l (historical COD design concentration) and at this report's COD design concentration of 782 mg/l.

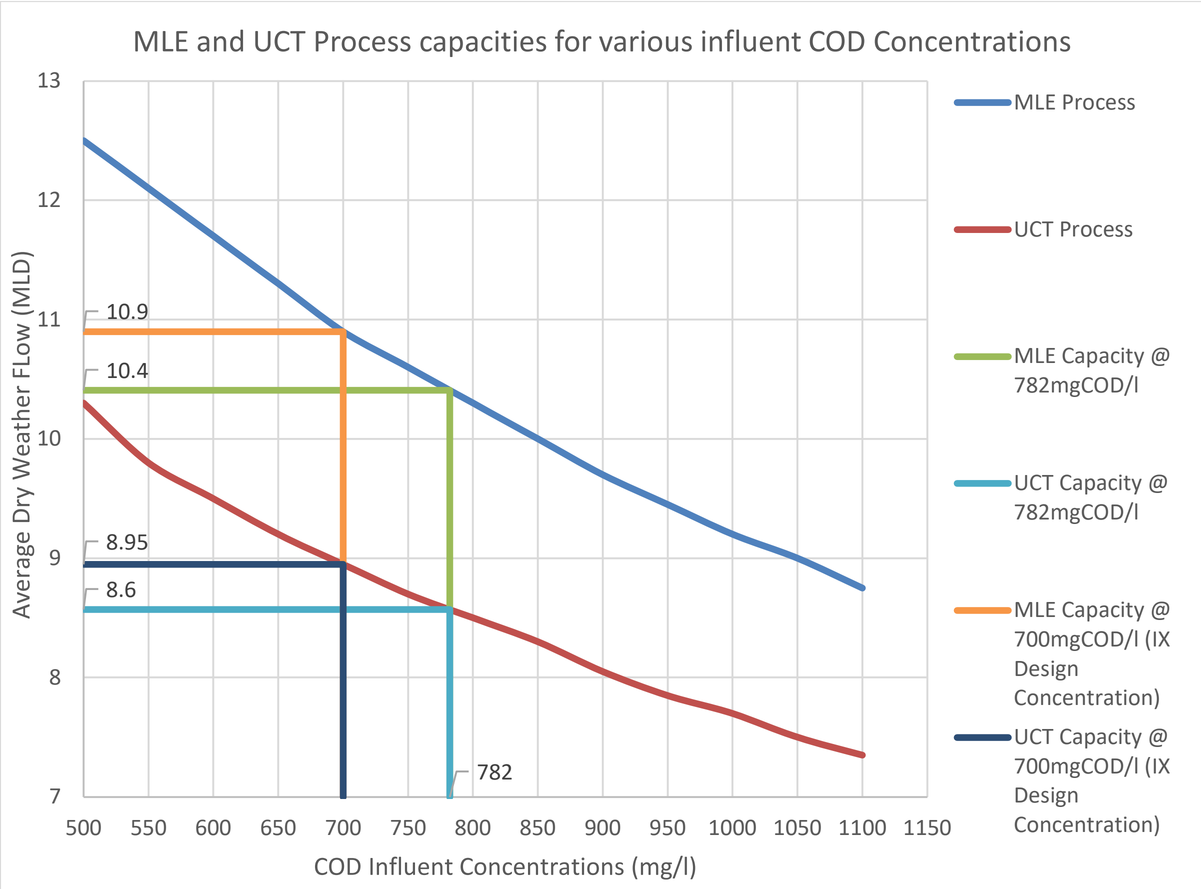


Figure 4-1: MLE and UCT process capacities (existing reactor) for various influent COD concentrations

The existing capacity is based on the operational existing infrastructure, which includes a biological reactor and two secondary clarifiers of 25 m diameter each. There is no primary settling 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
Optimal 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.

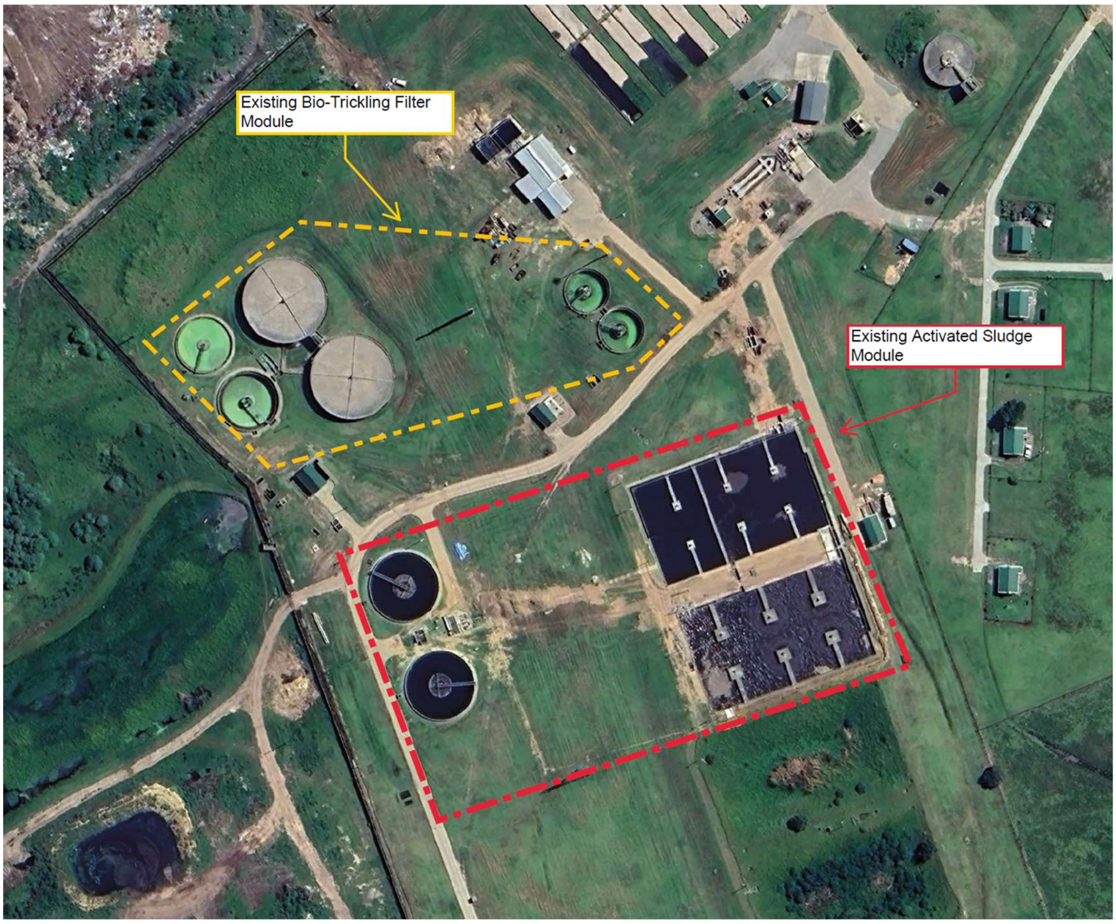


Figure 4-2: Gwaing WWTW existing plant layout (Google Earth Image taken February 2023)

4.2.1 Inlet Works

The Inlet Works consists of the following Infrastructure:

- 1 x Course Screen (12 mm apertures)
- 2 x Mechanical Drum Screens (6 mm apertures)
- 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 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, PSTs 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 process.
  - Anaerobic zone – 2 x vertical shaft mixers (11 kW each).
  - Anoxic zone – 4 x vertical shaft mixers (11 kW each).
  - 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 operates in the UCT process configuration. 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 the Master Plan. 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	22	28
Area	m^2	44000		
Depth	m	1.5		
Volume	m^3	66000		
Retention Time	hrs	158	72	57
	Days	6.6	3.0	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 (TSS) Probe
    - 1 x Flow Meter
  - FBP 2
    - 1 x Turbidity (TSS) 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

- o 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. Refer to Section 7.6 for the future requirements for sludge handling.

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. The 6 mixers in the anaerobic and anoxic zones have 11 kW motors each which means the installed mixing intensity capacity is 8.9 W/m^3. New mixer technologies could be employed that reduce the mixing requirements to about 2.4 W/m^3 (6 mixers of 3 kW each) which would reduce the installed mixer power by 48 kW. Even though the more energy efficient mixers will be more expensive, it is expected that the payback period will be no more than 3 years.

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.

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 has been implemented 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, norms and standards must 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 6.3.1)

- 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 7.6.
- 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.

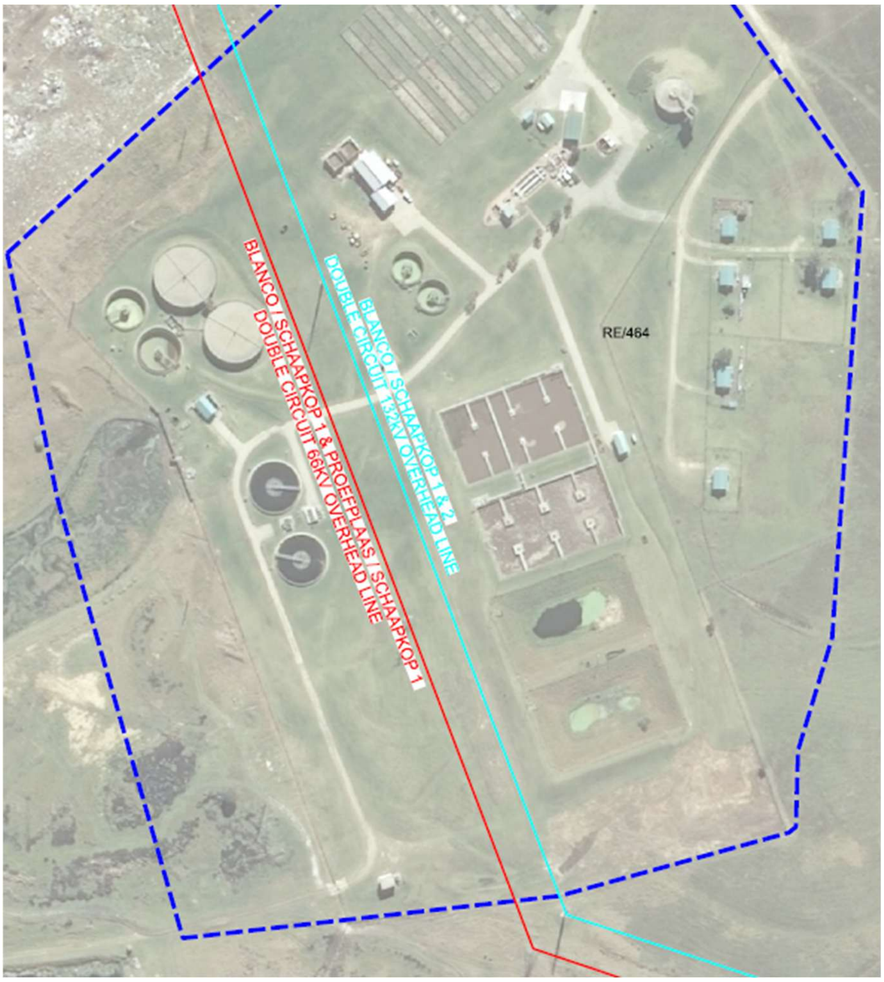


Figure 4-6: Eskom overhead powerline positions

3. **<u>O.H. Line Services:</u>**

a) The following building and tree restriction on **<u>either side of centre line</u>** 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 **<u>distances from the conductors</u>**:

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 **<u>minimum ground clearance</u>** 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.

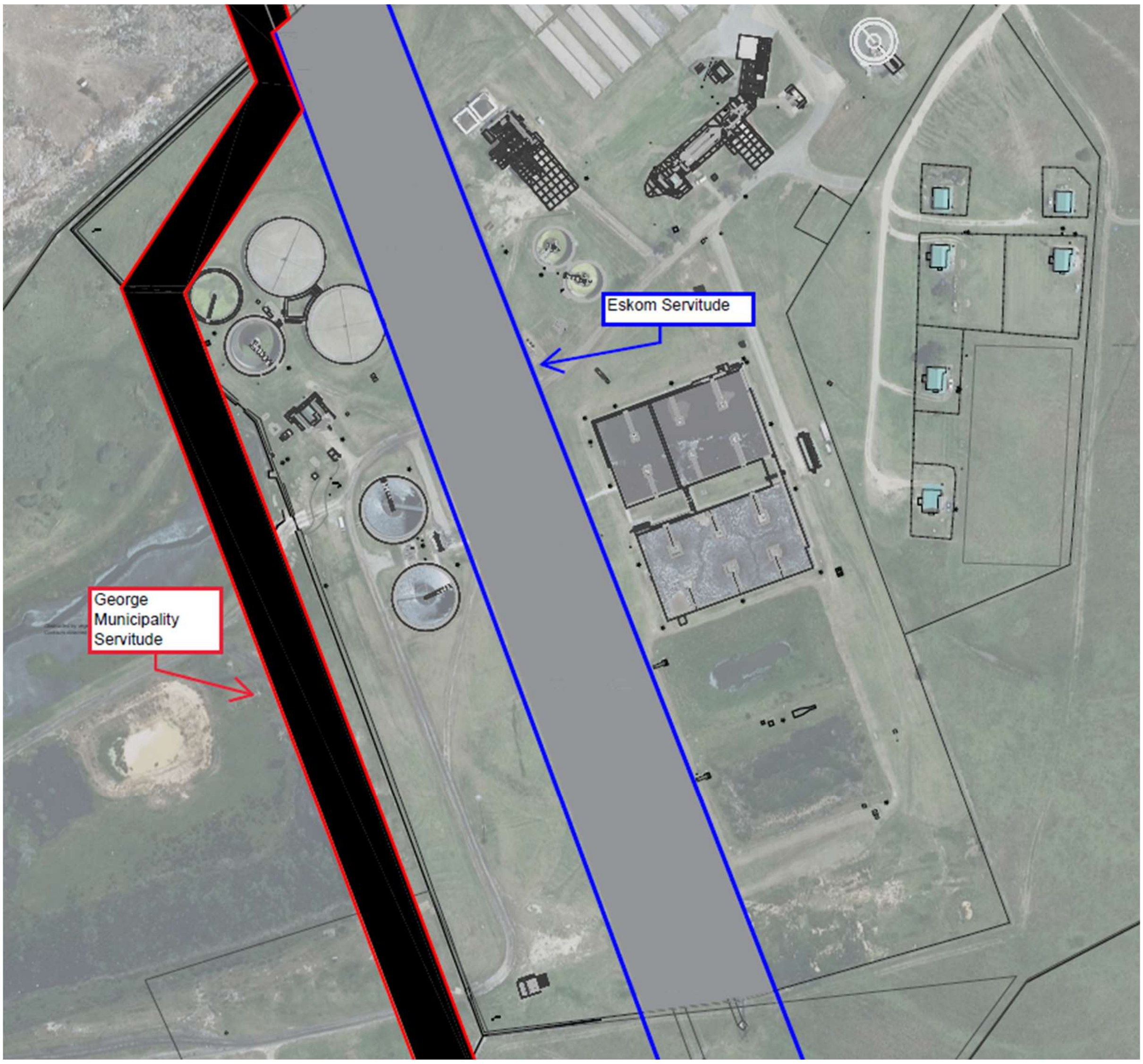


Figure 4-8: Eskom and George Municipality powerline servitudes

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 imported earthworks and extensive concrete support structures. 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.

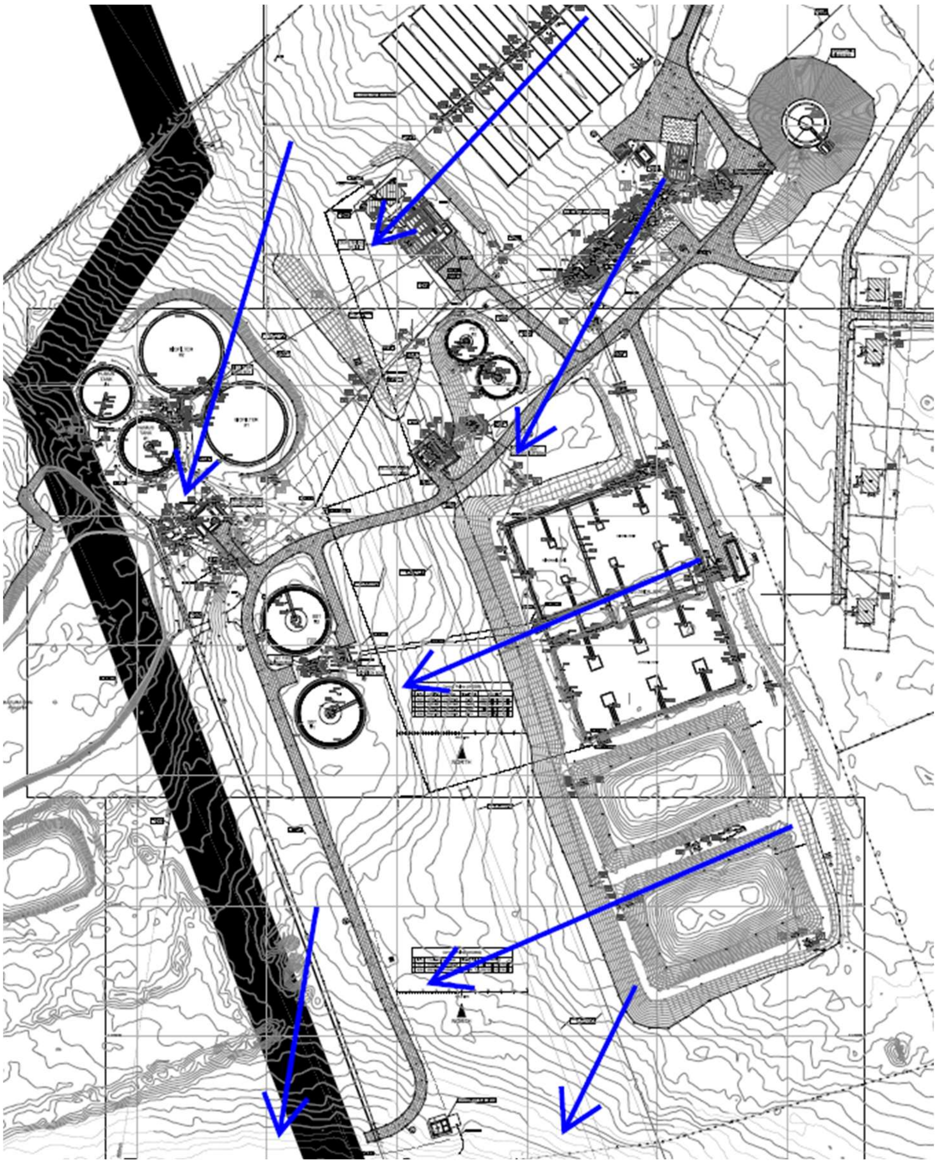


Figure 4-9: Site contours and fall directions

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.

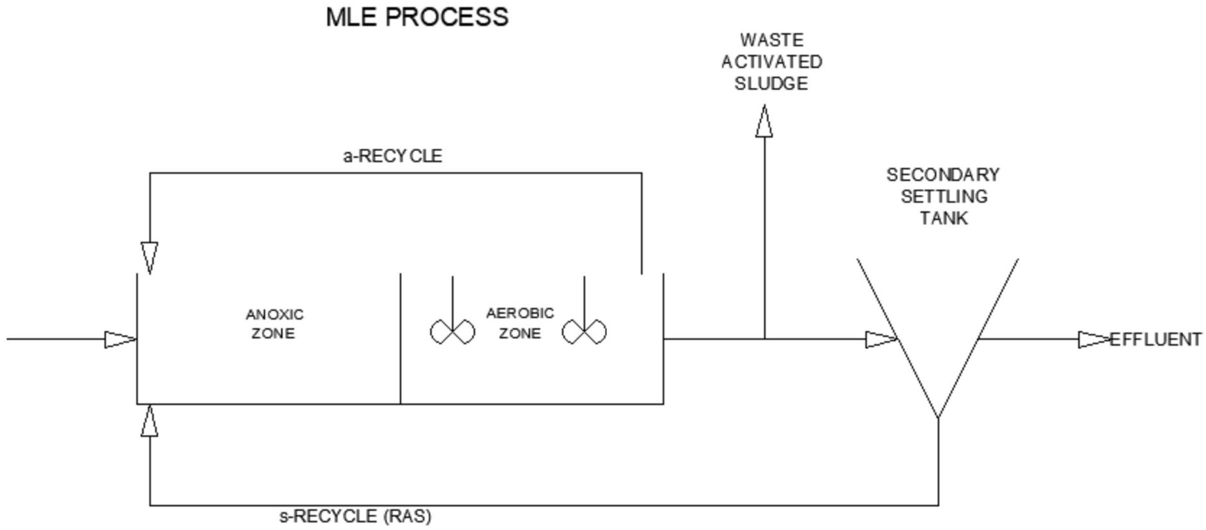


Figure 5-1: MLE process configuration

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.

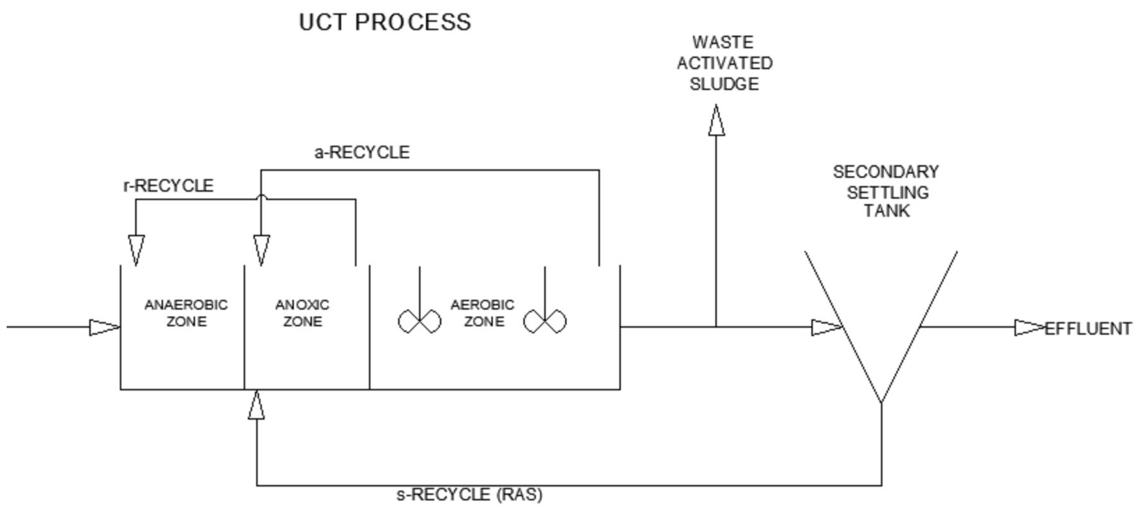


Figure 5-2: UCT process configuration

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.

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.

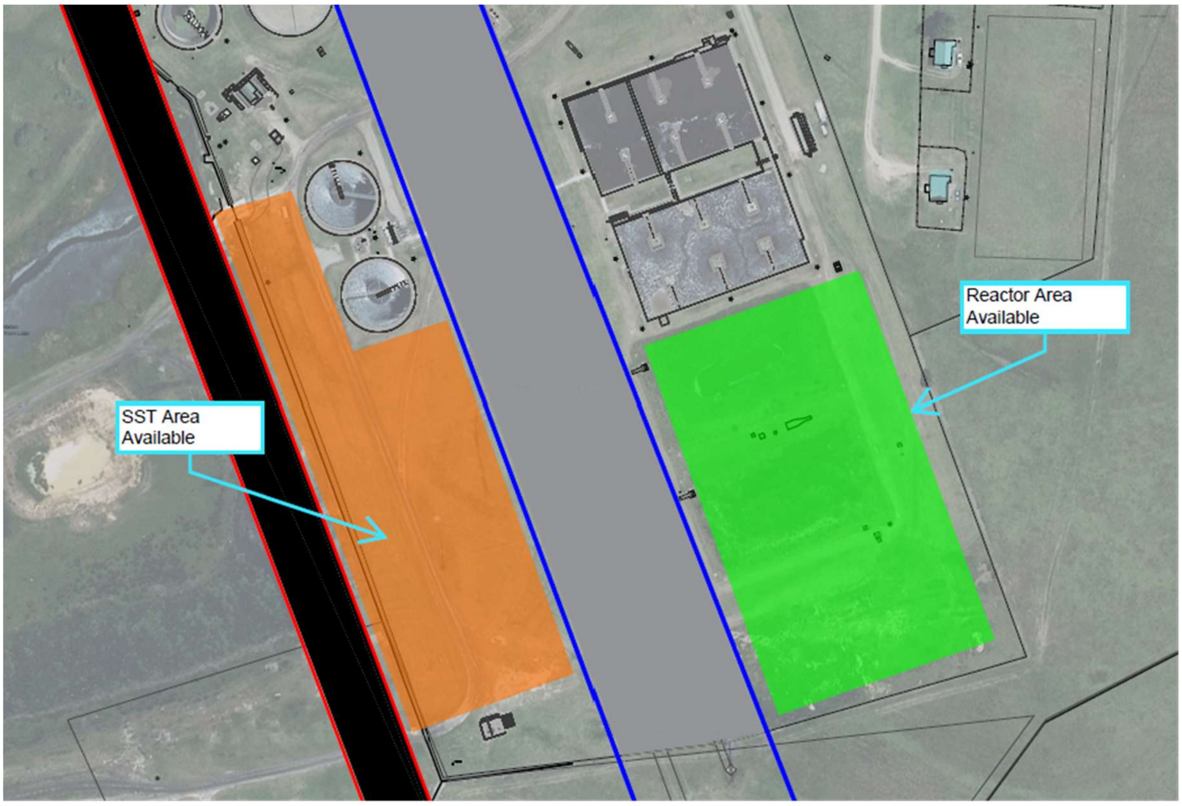


Figure 5-3: Reactor and SST construction area available

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 16,7 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.

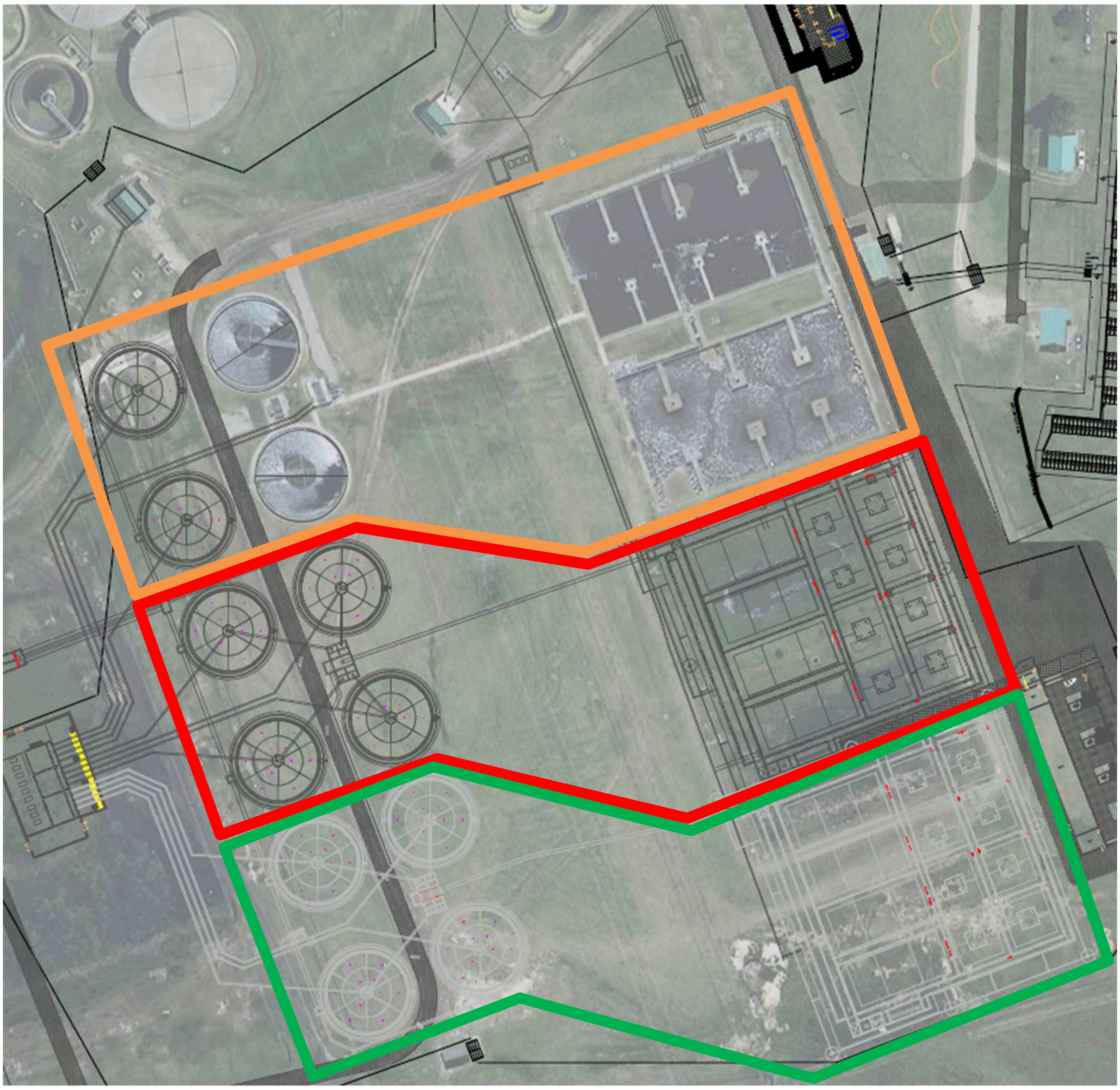


Figure 5-4: Reactor and SST module layouts for ultimate capacity

5.3.2 Biological Reactor Design

Two new reactors are required for the ultimate Gwaing WWTW solution. Phase B includes the first of the new reactors, called Reactor B. 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 5.7 m deep (4.5 m deep water level with 1.2 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 that 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 remaining volume (50%, which includes zones 7,8,9 and 10) is aerated, and fitted with fine bubble diffusers.

Screened and degritted sewage enters the reactor in the channel as shown in Figure 5-5. 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 zones (Zone 7 to 10) to the SSTs. Waste activated sludge (WAS) is taken from the overflow channel of the aerobic zones and is pumped to the WAS Dewatering Sump at specific rates to control the sludge age, while considering the level in the WAS dewatering sump. Service ducts between the four aerated zones (zone 7 to 10) 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.

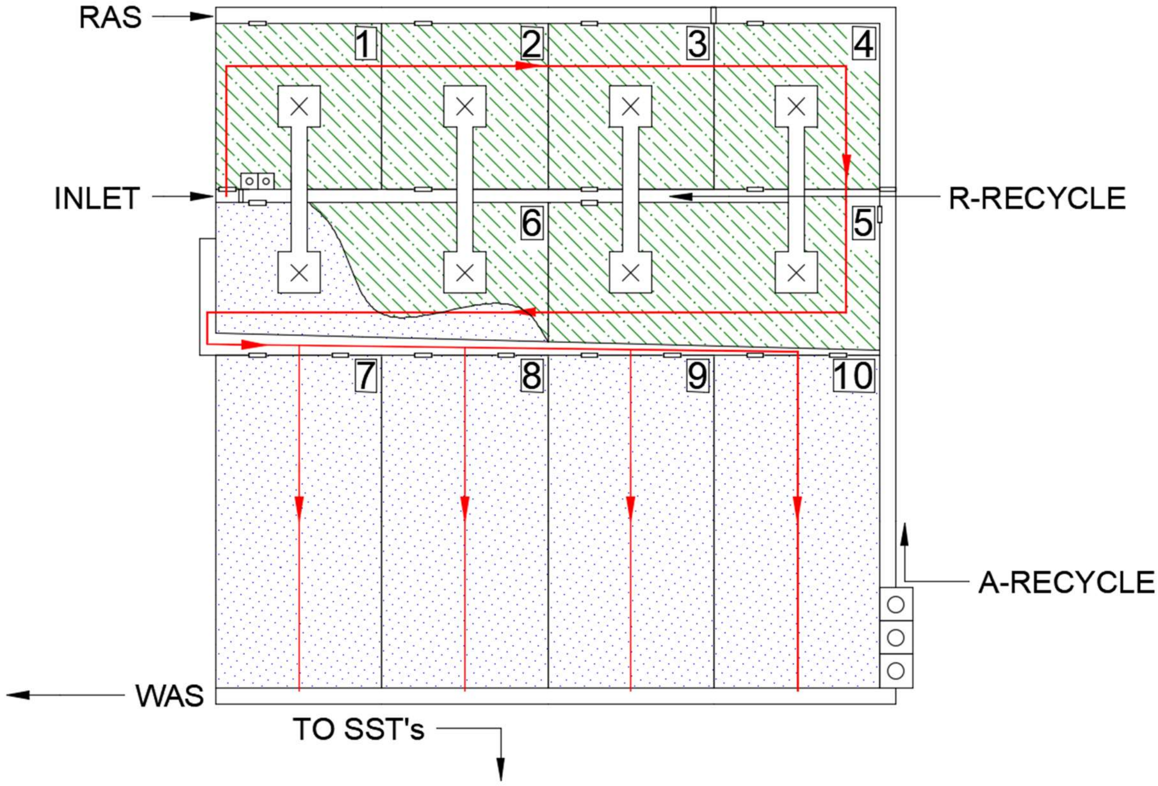


Figure 5-5: Reactor Layout Design

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 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. Given how close the MLE process' orthophosphate concentrations are to the general limit, it is better suited to operate the UCT process when the plant is not overloaded. 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 (lower temperatures reduces nitrification ability). For the Phase B reactor that will receive **raw** sewage, the UCT 2 process will be well suited during winter to support nitrification while the MUCT 1 or JHB 1 might be best suited for summer.

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. ADWF – 11 MLD (Reactor B Raw UCT capacity)
b. Raw COD concentration – 782 mgCOD/l

c. Ammonia – 58.7 mgN/l
d. Alpha factor surface aeration = 0.85
e. Alpha factor FBDA = 0.6
f. Theta (Temperature sensitivity) for FBDA = 1.024
g. Theta (Temperature sensitivity) for surface aeration = 1.012
h. Standard Oxygen Transfer Efficiency (SOTE) = 7%/m depth of reactor for FBDA
i. Surface aeration on/off control steps = 55 kW

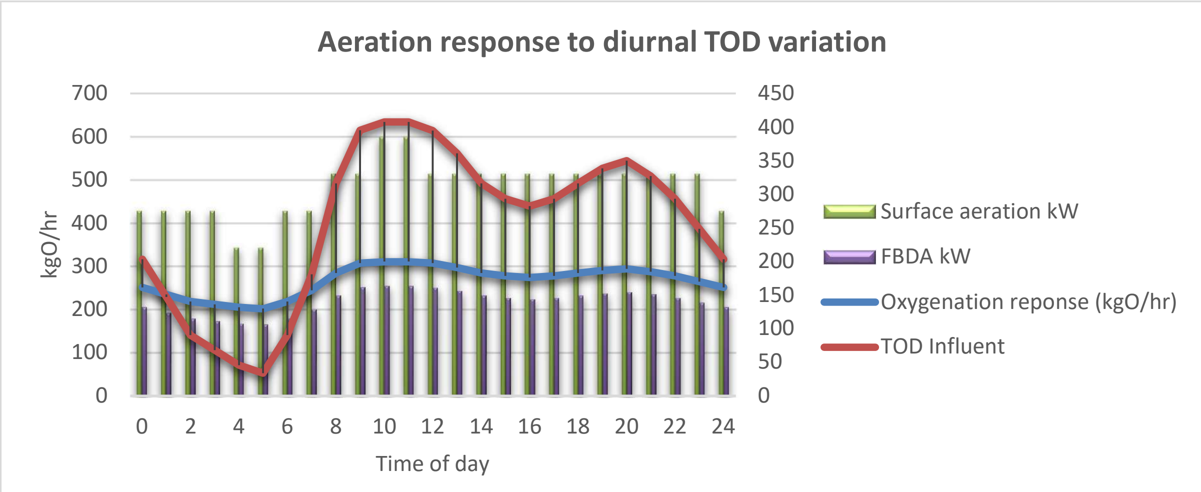


Figure 5-7: FBDA vs Surface Aeration Response to Diurnal TOD Variation

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 factor of surface aeration to FBDA power consumption to 2.2. Consequently, we recommend that FDBA be installed for the new reactors.

Table 5-2: 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				**Total [kW]**			
	Raw		Settled		Raw		Settled		**Raw**		**Settled**	
	MLE	UCT	MLE	UCT	MLE	UCT	MLE	UCT	**MLE**	**UCT**	**MLE**	**UCT**
Vertical Shaft Aeration [kW installed]	440	327	548	397					**664**	**490**	**834**	**596**
Fine Bubble Diffused Aeration (FBDA) [kW installed]					224	163	286	198				
Capacity in MLD	14.0	11.0	22.5	16.7	14.0	11.0	22.5	16.7	**28.0**	**22.0**	**45.0**	**33.3**

The centrifugal blowers needed for phase B were sized using the oxygen required for running an MLE process treating 14 MLD of raw sewage (maximum oxygen demand case). A diurnal oxygenation response curve indicated that the following oxygen requirements:

- Minimum Oxygenation rate – 278 kgO_2/hr
- Average Oxygenation rate – 367 kgO_2/hr
- Minimum Oxygenation rate – 428 kgO_2/hr

Based on the diurnal oxygenation response curve the blowers will need to be able to achieve a turndown ratio of 40% necessitating a 2 duty and 1 standby blower configuration.

The existing reactor utilises surface aeration. 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. The SST design parameters are summarised in Table 5-3.

Table 5-3: SST design values per phase

Parameter	Phase A	Phase B
ADWF (MLD)	13.2	22
PWWF (MLD)	33	55
DSVI (ml/g)	135	135
SST Diameter (m)	25	25
No. of SST's	8	8
No. of Reactors	1	2
Recycle ratio w.r.t ADWF	2	2
SST Area (m^2)	3927	3927
Flux Rating	0.8	0.8
SST effective Area (m^2)	3142	3142

5.3.3.1 Phase A

Figure 5-8 shows the design and operation (D&O) chart for Phase A of a single SST for various reactor concentrations ranging from 5.65 kgTSS/m^3 (5650 mgTSS/l) to 6.40 kgTSS/m^3 (6400 mgTSS/l). With the above-mentioned inputs, the model shows SST failure at reactor concentrations greater than 5.90 kgTSS/m^3. The D&O chart is based on the raw UCT process with a total ADWF of 13.2 MLD and PWWF of 33 MLD into the plant. The peak flow is based on a Harmon peaking factor of 1.9 and a stormwater infiltration of 30%, giving a total peak factor of 2.5.

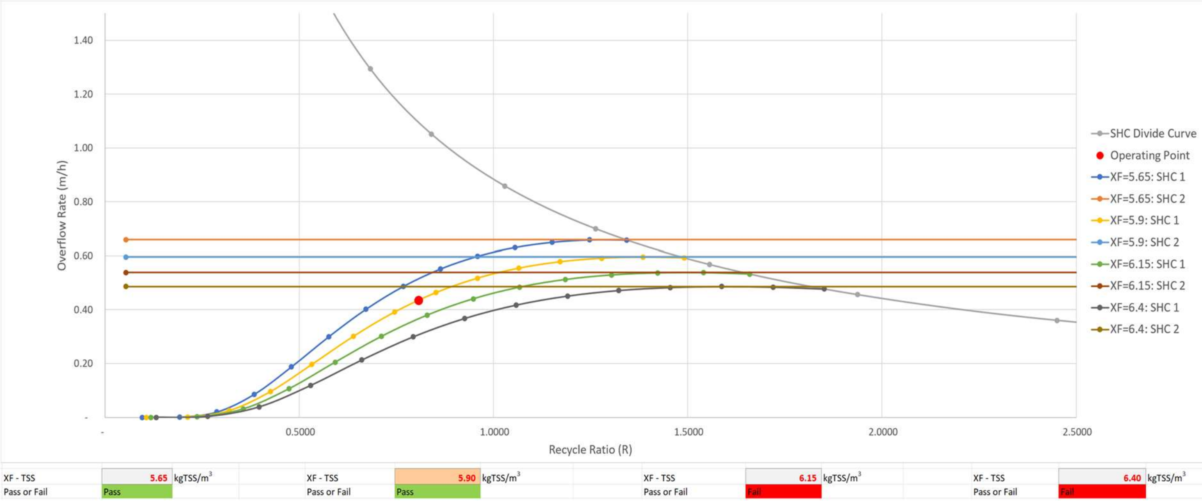


Figure 5-8: Design and Operation chart of SST for Phase A

5.3.3.2 Phase B

Figure 5-9 shows the design and operation chart for Phase B of a single SST for various reactor concentrations ranging from 4.65 kgTSS/m^3 (4650 mgTSS/l) to 5.40 kgTSS/m^3 (5400 mgTSS/l). The Phase B configuration includes two biological reactors and eight SSTs, with four SSTs per reactor. The steady-state model shows that the SSTs fail at reactor concentrations greater than 4.90 kgTSS/m^3. The D&O chart in Figure 5-9 is based on the raw UCT process with an ADWF of 22 MLD and a PWWF of 55 MLD. The PWWF is based on a Harmon peaking factor of 1.9 and a stormwater infiltration of 30%, giving a total peak factor of 2.5.

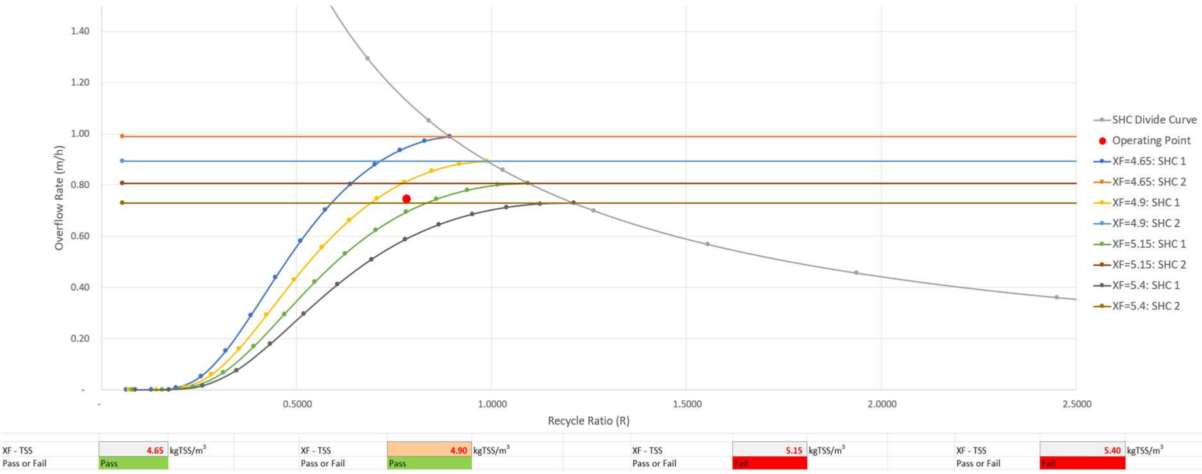


Figure 5-9: Design and Operation chart of SST for Phase B

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 5.1.3. The infrastructure included in Phases A and B is discussed in Section 6.3.

Table 5-4: Module capacities for various processes for ultimate capacity design

	Existing Reactor A				Reactor B				Total			
	Raw		Settled		Raw		Settled		Raw		Settled	
	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	**28**	**22**	**45**	**33.34**
Flow to PST (MLD)	0.0	0.0	0.2	0.2	0.0	0.0	0.2	0.2	**0**	**0**	**0.45**	**0.333**
ADWF to Reactor (MLD)	14.0	11.0	22.3	16.5	14.0	11.0	22.3	16.5	**28**	**22**	**44.55**	**33**
Sludge Age (days)	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0	**40**	**40**	**40**	**40**
SRT_{min} with SF	19.0	16.3	19.0	16.3	13.5	16.3	13.5	16.3	**32.5**	**32.59**	**32.5**	**32.59**
Reactor TSS Concentration (mgTSS/l)	4700.0	4900.0	3700.0	4350.0	4700.0	4900.0	3700.0	4350.0	**9400**	**9800**	**7400**	**8700**
Reactor Volume (m^3)	14634.8	14777.5	14764.4	14709.4	14634.8	14831.6	14764.4	14765.3	**29270**	**29609**	**29529**	**29475**
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**
PDWF (MLD)	26.4	21.6	38.8	30.2	26.4	21.6	38.8	30.2	**52.8**	**43.3**	**77.5**	**60.4**
Stormwater infiltration	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	**0.3**	**0.3**	**0.3**	**0.3**
PWWF (MLD)	34.3	28.1	50.4	39.3	34.3	28.1	50.4	39.3	**68.6**	**56.2**	**100.8**	**78.6**
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**

5.3.4.2 Recycle Ratios

The biological reactor's recycle ratios and operating strategies are summarised in Table 5-5.

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

Parameter	**Recycle Ratio w.r.t ADWF**	**Operating Strategy**
r-recycle ratio	1.00	Constant flow rate
s-recycle ratio (RAS)	2.00	Could be varied according to the actual flow entering the plant to save energy. (1.0 x incoming flow rate)
Maximum a-recycle ratio (operationally variable)	6.00	The actual a-recycle ratio will be set according to the chosen process and the available denitrification potential in the anoxic zone.

5.4 Disinfection

All clarified effluent from the secondary settling tanks 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.

5.4.1 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 5-6.

Table 5-6: 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 effect on aquatic life

To assist with deciding 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 capital cost inflation – 6%
- M&E capital equipment inflation – 8%

UV costs were obtained from UV and chlorine equipment suppliers. A summary of the capital and associated operation and maintenance costs from these suppliers is shown in Table 5-7.

Table 5-7: Chlorine and UV Estimated Capital and Annual O&M Costs (2024 Rates)

	Phase A&B	
	2024 Rates	2025 Rates*
Capital Cost		
M&E UV	R39 732 000.00	R42 910 560.00
UV Civil Cost	R6 018 891.06	R6 380 024.52
M&E Chlorine	R17 731 871.25	R19 150 420.94
Chlorine Civil Cost	R12 223 566.40	R12 956 980.38
Annual O&M Costs (2024 Value)		
UV Annual Operation Cost	R2 001 484.80	R2 201 633.28
UV Annual Maintenance Cost	R2 663 522.70	R2 929 874.97
Chlorine Annual Operation Cost	R2 617 780.00	R2 879 558.00
Chlorine Annual Maintenance Cost	R476 873.09	R524 560.40

** Quotes and pricing were acquired in 2024, all rates were adjusted as defined in the paragraph preceding the Table.*

The net present value of the LCCA discounted at 6% over 45 years between 2025 and 2070 is shown in Figure 5-10. Over this period, as derived from Figure 5-10, there is a net present value difference of **R23 405 134.67** in the favour of UV disinfection.

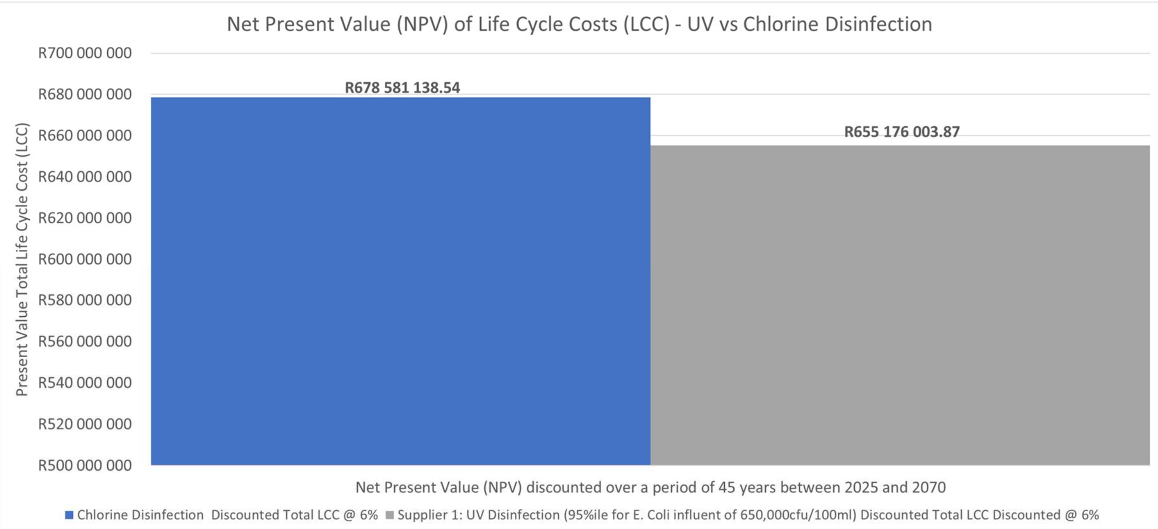


Figure 5-10: Discounted Life Cycle Cost Comparison between UV and Chlorine

The life cycle cost analysis is depicted in Figure 5-11. 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 deviate 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 5-6). 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.

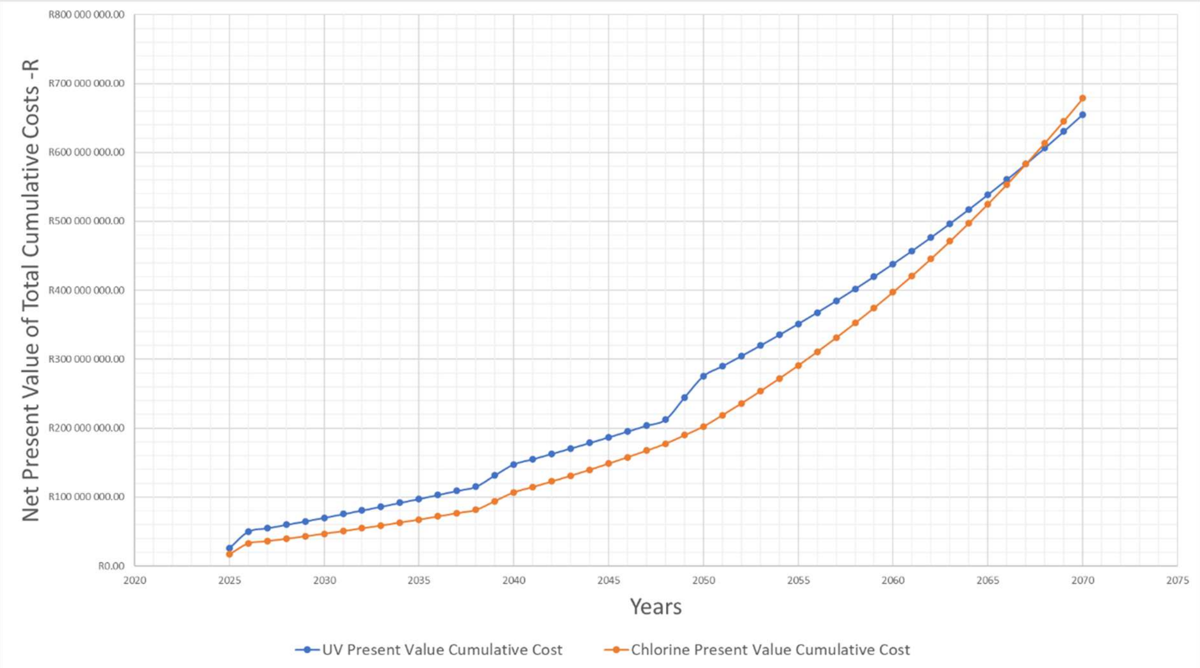


Figure 5-11: Net Present Cumulative Cost Comparison UV vs Chlorine

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.

5.4.2 UV Disinfection

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 5-8.

Table 5-8: UV Design Summary

Design Summary	
Maximum disinfection flow	55 MLD
Average disinfection flow	22 MLD
Minimum disinfection flow	4 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 5-12.

Figure 5-12: Example Image of open channel UV installation with automatic lamp lifting system.

6 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 6-1.

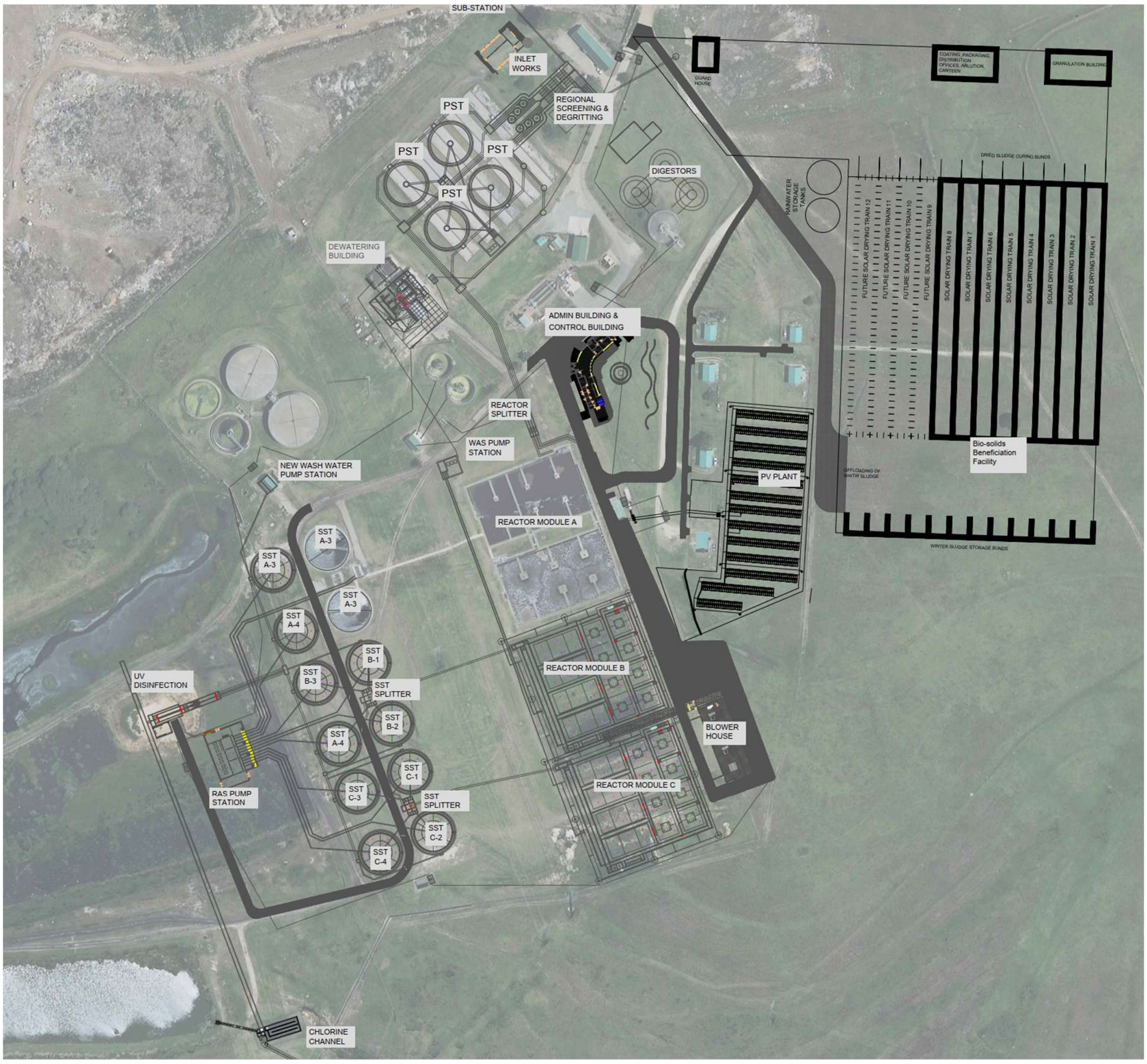


Figure 6-1: Master plan site layout

6.1 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 6-1 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 6-1: Unit process naming conventions and process capacities

Unit Process	No. Units	Unit Name	Process Capacity in terms of ADWF (MLD)
Inlet Works	2	Inlet Works 1	25
		Inlet Works 2	25
Primary Settling Tanks	4	PST 01	14
		PST 02	14
		PST 03	14
		PST 04	14
Primary Sludge Gravity Thickeners	2	GT 01	
		GT 02	
Anaerobic Digestors	4	AD 01	
		AD 02	
		AD 03	
		AD 04	
Biological Reactors	3	Reactor A	Reactor and SST process capacity depends on the process type and configuration selected.
		Reactor B	
		Reactor C	
Secondary Settling Tanks	12	SST A-1	
		SST A-2	
		SST A-3	
		SST A-4	
		SST B-1	
		SST B-2	
		SST B-3	
		SST B-4	
		SST C-1	
		SST C-2	
		SST C-3	
		SST C-4	
UV Disinfection	2	UV 01	25
		UV 02	25

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 6-2. 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.

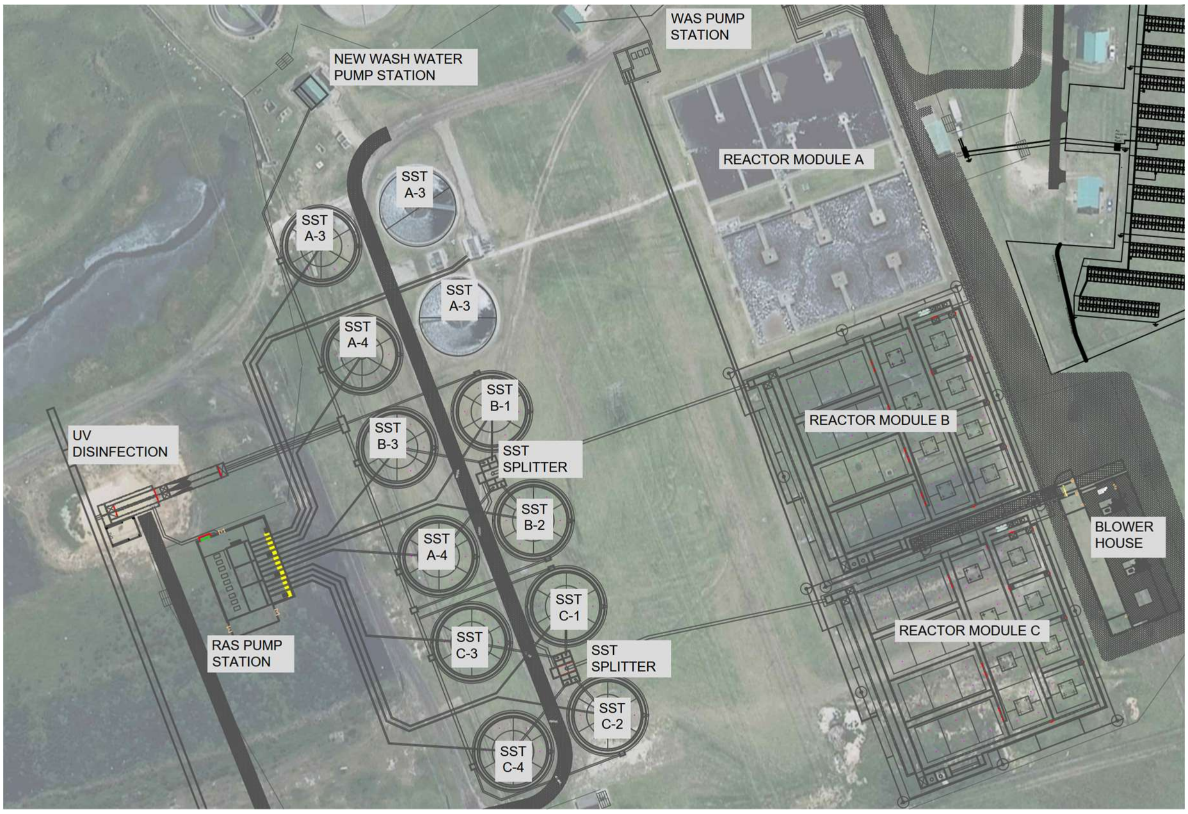
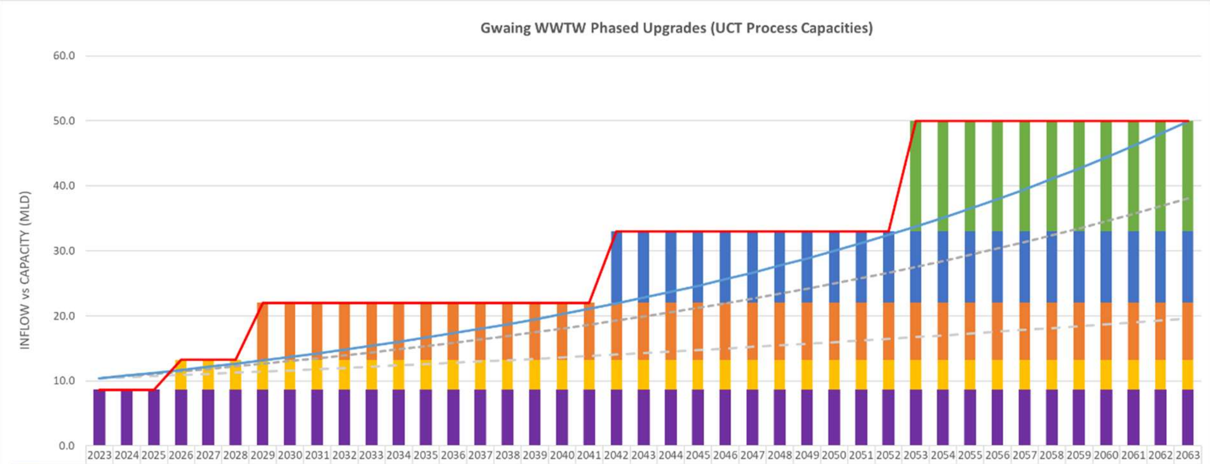


Figure 6-2: Activated sludge module naming convention

6.2 Summary of Ultimate Capacity's Phases

The four phases and their capacities are graphically shown in Figure 6-3 to align with the population growth based on 4% population growth. Note that the existing capacity in Figure 6-3 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	2043
Phase D																					
Phase C																				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
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
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%	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	14.3
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	19.9
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	22.8
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	33.0

	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
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
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
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%	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%	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%	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	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0

Figure 6-3: Population growth and phases of capacity upgrades

The four phases proposed, with the relevant processes and capacities are summarised in Table 6-2. 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 6-2: 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 6-4: Phases of Master Plan Upgrade

6.3 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 phasing configuration 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.

6.3.1 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 and the Gwaing WWTW Upgrade Concept Design Report: Phase A and B was considered.

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 6-5 below shows the upgrade to the existing chlorine contact channel.

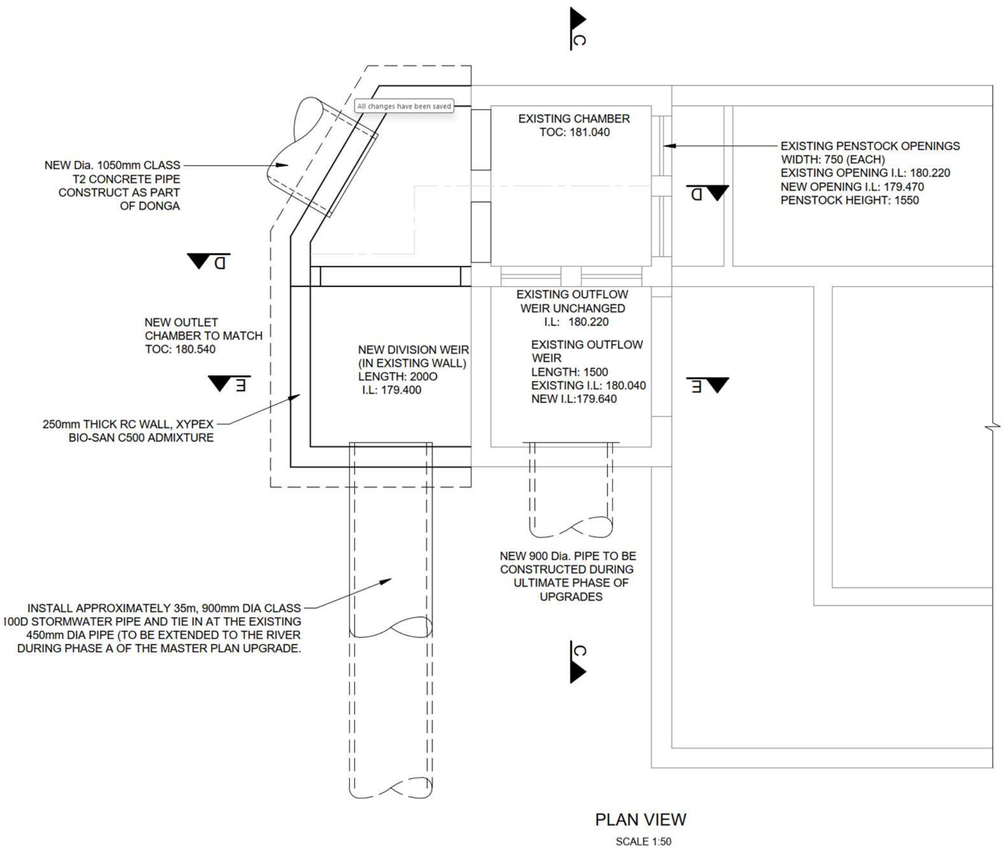


Figure 6-5: Chlorine Contact channel upgrade as part of Donga Rehabilitation Contact

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.

6.3.2 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

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 6-6.

Figure 6-6: 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 6-7 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 6-7: Layout of the outlet structure upgrade

6.3.3 Phase B

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:

- 22 MLD ADWF as Raw UCT process

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 6-8.

Figure 6-8: Phase B Site Layout

6.3.4 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:

- 33 MLD ADWF as Raw UCT process

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 6-9. The total capacity of the plant after the Phase C upgrade will be 33 MLD operating a UCT process, this is an 11 MLD upgrade from Phase B.

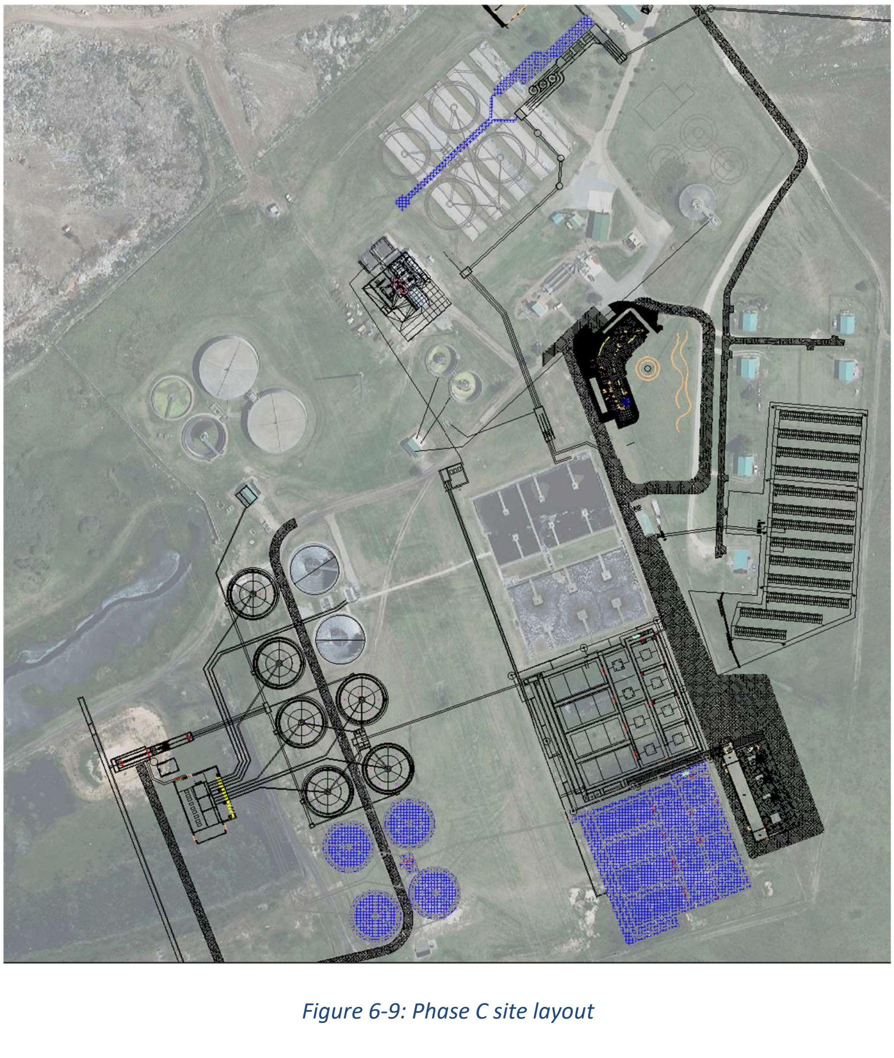


Figure 6-9: Phase C site layout

6.3.5 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:

- 50 MLD ADWF as settled UCT process

Phase D of the upgrades will be the final phase of the Master Plan Upgrade. The phase will see the construction of the four PSTs, a 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 6-10 shows the site layout of the proposed Phase D upgrade.

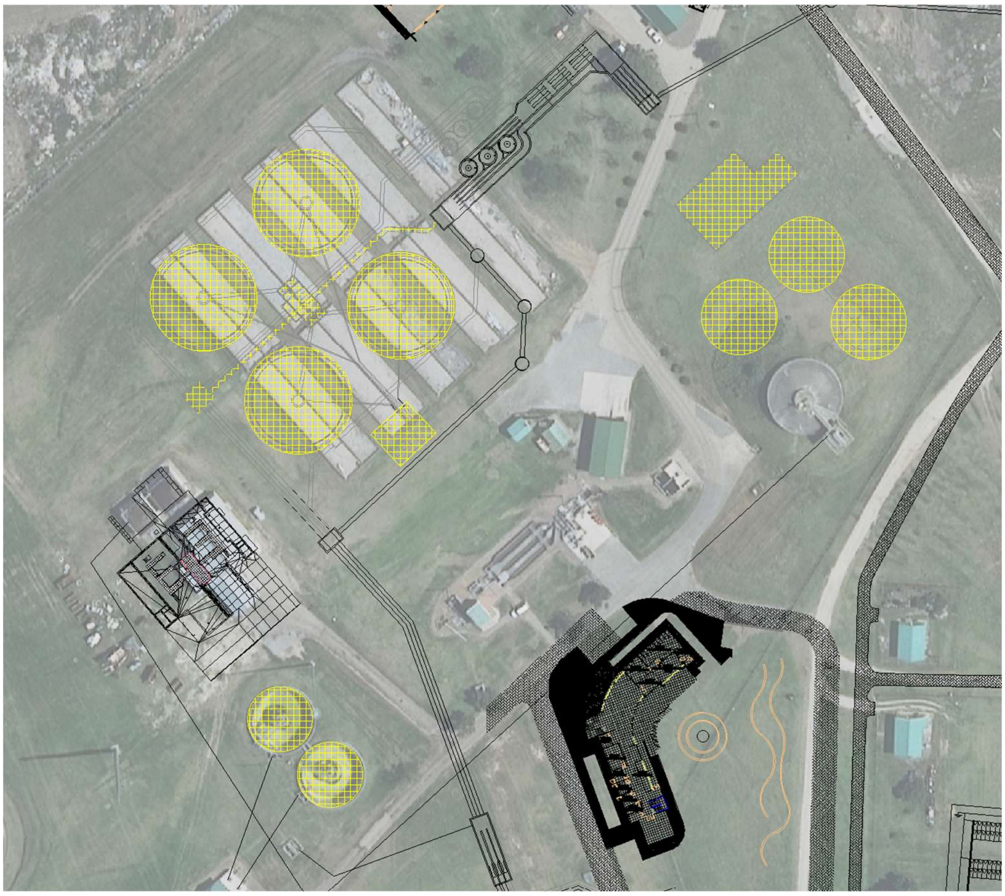


Figure 6-10: Phase D site layout

6.3.6 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.

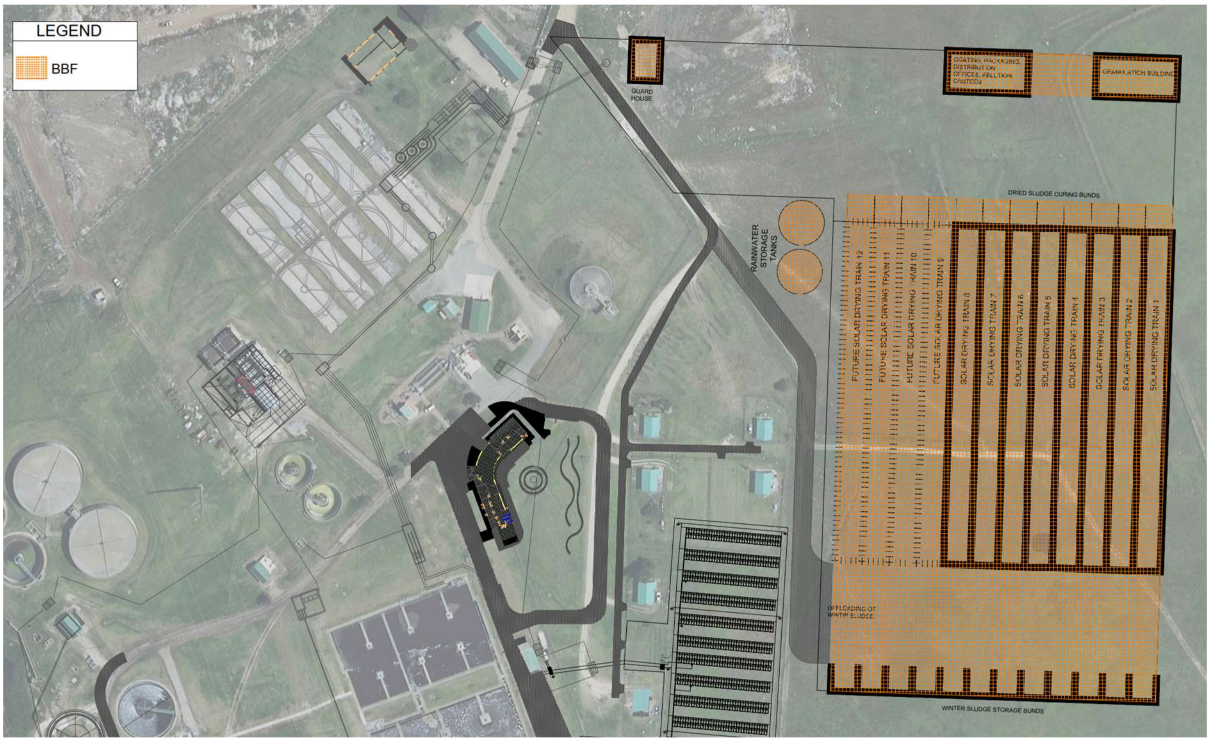


Figure 6-11: Layout of the Bio-Solids Beneficiation Facility phase layout

7 NEW INFRASTRUCTURE CONCEPT DESIGN (PHASES A & B)

This section includes the infrastructure that will be implemented as part of Phase A, Phase B and the Bio-Solids Beneficiation Facility (BBF) construction.

The new facilities required for Phases A and B to reach a combined capacity of 22 MLD are discussed in this section. The activated sludge portion of Phase A and B, namely the biological reactors and secondary settling tanks (SSTs), will ultimately be two equal modules (modular design discussed in Section 5.3). The head of works is a common structure for both activated sludge modules. The flow split for the two reactor modules takes place upstream of the reactors. The SST supernatant flows to the disinfection unit process, and from there into the maturation ponds.

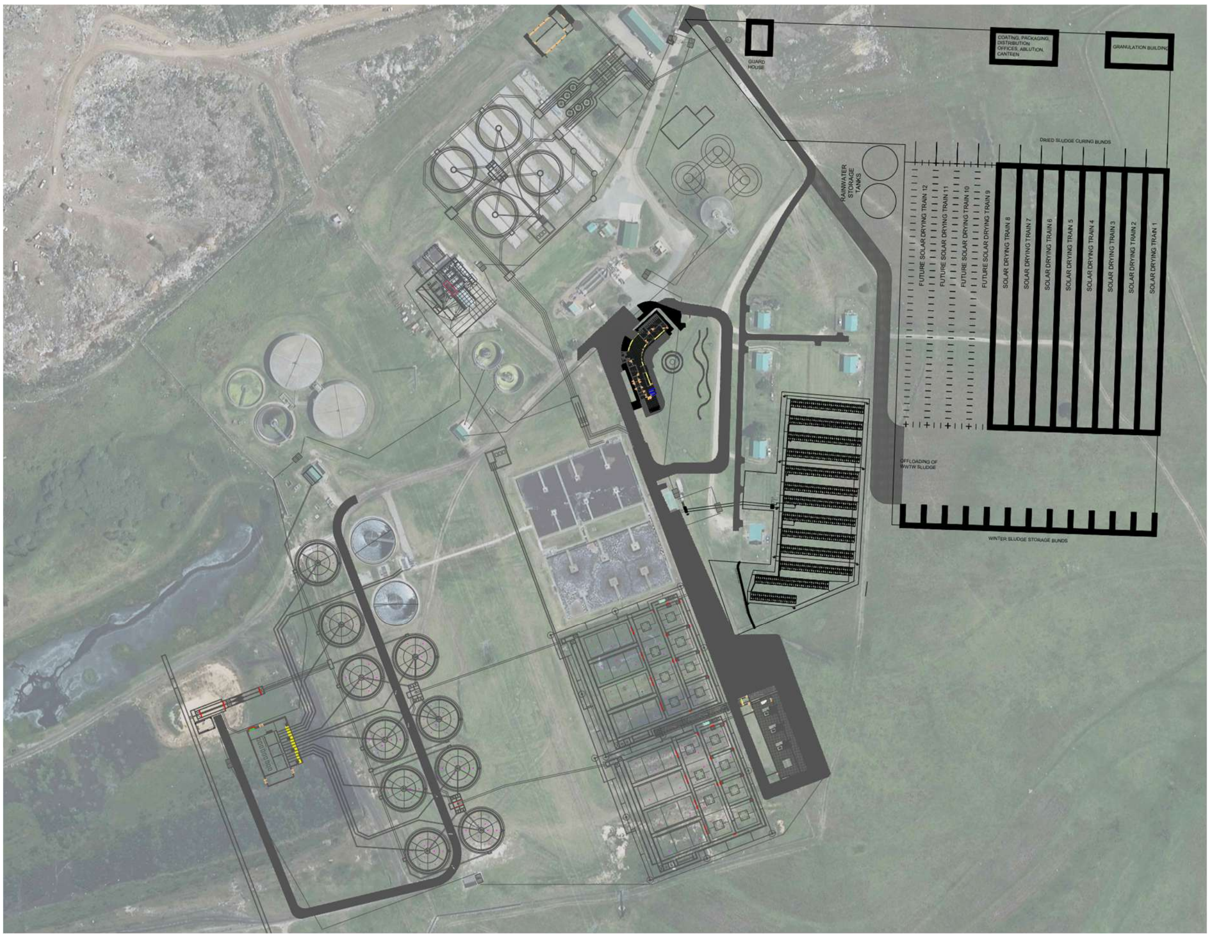


Figure 7-1: Site layout of Phase A and B

7.1 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. During Phase B of the upgrades, the first half of the Inlet Works will be built, which will adequately serve the capacity requirements for Phases A and B. The second half of the inlet works is only required in phase C.

Two pipelines discharge raw sewage into the existing inlet works. One is a 900 mm diameter gravity sewer main and the other is a 350 mm diameter rising main from Gwaing pumpstation. The indicative positions of these two pipelines are shown in Figure 7-2. Future flows are expected to join the inlet works from two pump stations, namely the Groeneweide pumpstation 1 and 2.

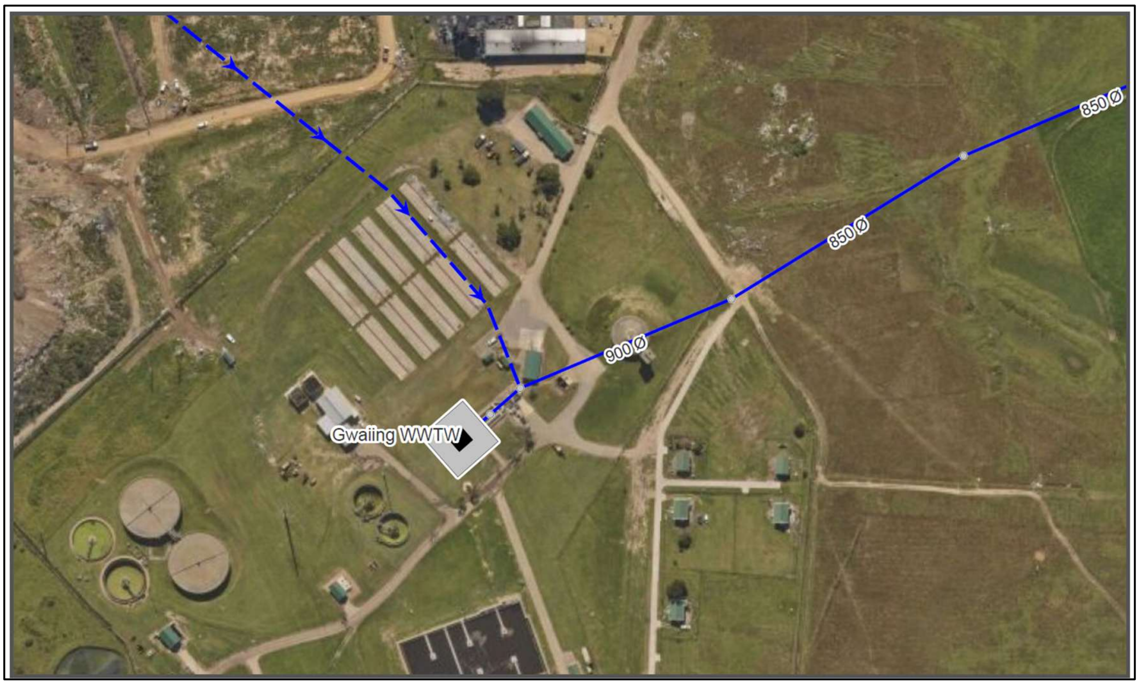


Figure 7-2: Main Incoming Raw Sewage Pipelines (GLS)

Table 7-1 shows the following flows for the existing pipelines:

- Estimated existing operational flows (based on currently developed plots).
- The theoretical flow capacities (based on the approved zoning and fully occupied properties in the catchment).

Table 7-1: Estimated Flows Feeding Gwaing WWTW (Source: GLS Consulting Hydraulic Models)

Estimated Flows (MLD)	Existing Operational - MLD (PDWF)	Theoretical Capacity (Approved zoning) - MLD (PDWF)
900mm Gravity main	25.6	28.4
Gwaing Pump station (Proefplaas)	15.1	15.1
Total	**40.69**	**43.55**

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 with a free discharge into the stream.

Due to the hydraulic head required to gravitate from the new inlet works through the new PSTs (Phase D) 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
- Vacuum tanker discharge point at the head of the works.
- 12 mm front raked mechanical coarse screens, screenings conveyor and screenings washer/ compactor.
- 3.5 mm perforated plate fine screens, screenings conveyor and screenings washer/ compactor.
- Emergency bypass channels to bypass the screenings in case of a blockage. The bypass channels will each be fitted with hand-raked bar screens with 30 mm apertures.
- 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 7-3 and the plan layout in Figure 7-4. The figures exclude any mechanical infrastructure and the channel cover slabs.

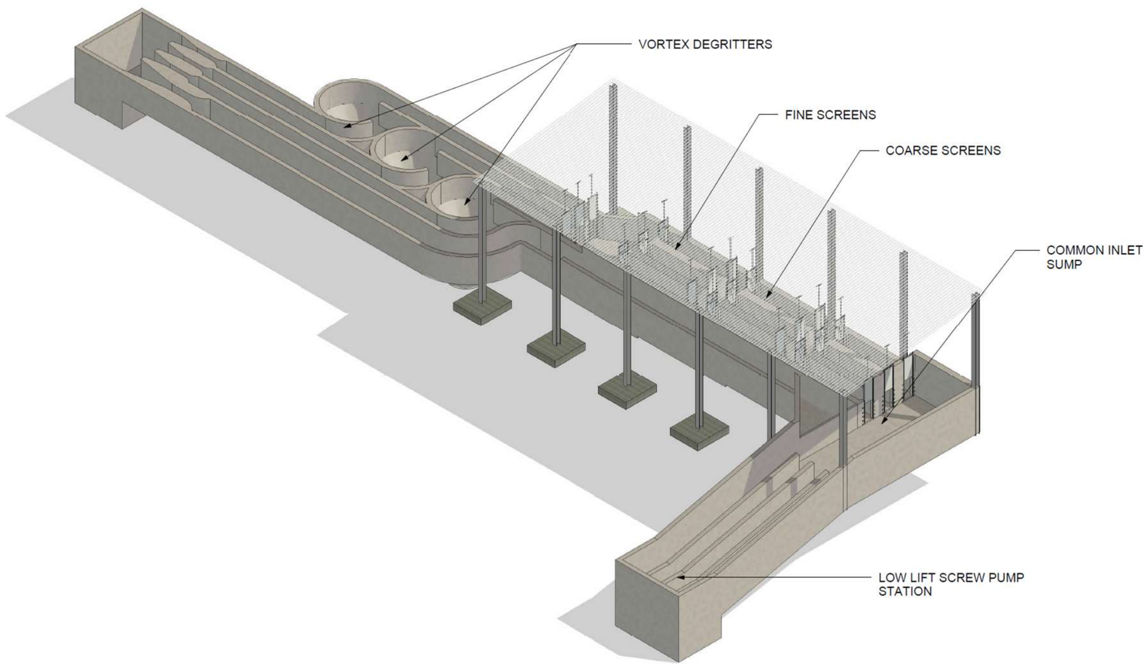


Figure 7-3: Head of Works 3D model (Phase A&B)

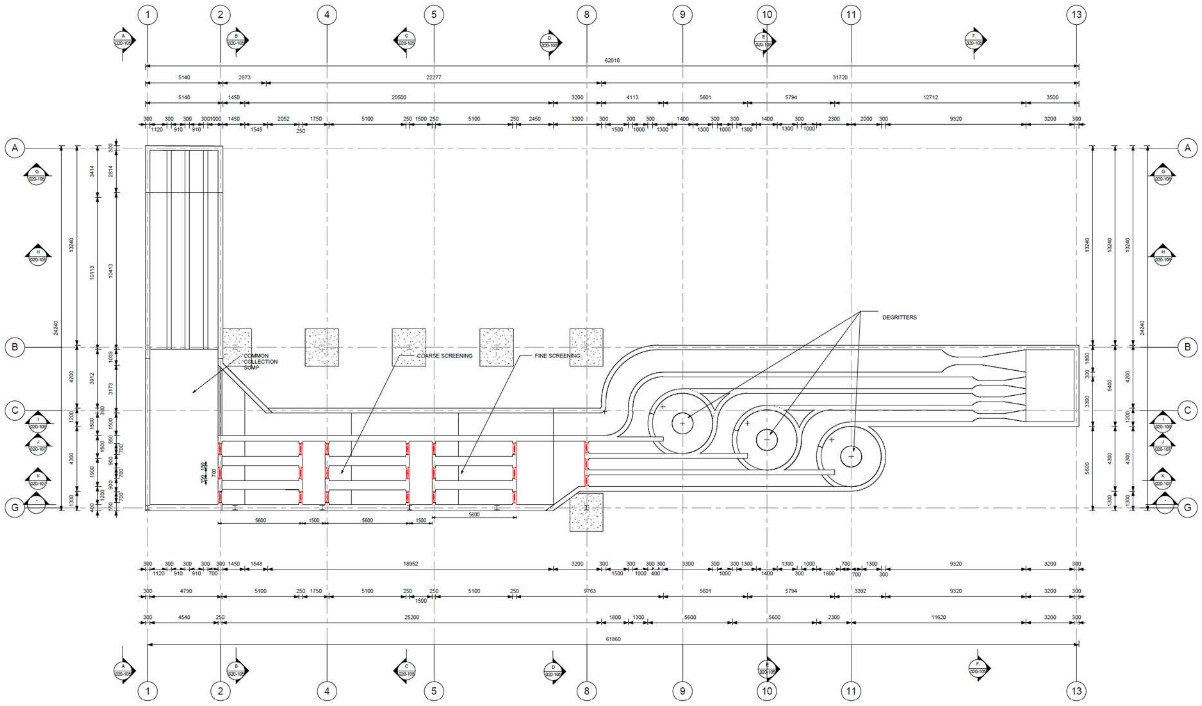


Figure 7-4: Head of works channel plan layout

7.1.1 Low lift pumpstation

The highest level to which the 900 mm gravity main can freely discharge is 196 masl. The top water level for the new head of works inlet chamber is 200.68 masl. The total pumping height required is 4.68 m. The maximum theoretical pipeline capacity, based on approved developments in the catchment area of the 900 mm gravity sewer, according to GLS, is 28.4 MLD (PDWF). The peak flow to be received in the low lift pumpstation, based on an ADWF/PWWF factor of 3, is 56.9 MLD. The screw pumpstation is designed to have a 3 duty and 1 standby screw configuration. This allows pumps to be turned off during lower flows to ensure energy efficiency. Table 7-2 shows a summary of the design flows and duty standby configurations for various flows.

Table 7-2: Screw pumpstation design flows and duty-standby configuration

	PDWF (Based on pipeline capacity)	ADWF	PWWF
Design Flow (MLD)	28.4	19.0	56.9
Design Flow (m^3/h)	1184	790	2369
Total Pumps	4	4	4
No of Duty Pumps Required	2	1	3
Flow per pump (m^3/h)	592	789.6	789.6
Flow per pump (l/s)	164.5	219.3	219.3

7.1.2 Screening channels

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

A bypass channel is provided in the event that the screens are all blocked or if the screens need to be bypassed for maintenance purposes. The bypass channels are designed to cater for all flows up to PWWF. The bypass channels are fitted with manually raked coarse screens with 30 mm apertures. 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 split between the hydraulically assisted vortex degritters. Table 7-3 shows the screen configuration.

Table 7-3: Screenings channels configuration – 22 MLD

Screening Channels	Duty	Standby
Channels – Hydraulic capacity	2	1
Bypass channels – Hydraulic capacity	0	1
Course Screens – 12 mm front raked screens	2	1
Fine Screens – 3.5 mm perforated plate screens	2	1
Water Launders	2	0
Washer Compactors	1	1

7.1.2.1 12mm Coarse Screens – Front Raked screens

The first set of screens through which the water will flow, are 12 mm front raked screens (coarse screens). These mechanical screens are continuously cleaned, and have teardrop-shaped bars to ensure minimised head losses through the screens. Figure 7-5 shows a schematic of the 12 mm front-raked screens in operation as well as the bar configuration. The screenings are lifted by the mechanical rakes and discharged into a water launder which conveys the screenings to a washer compacter.

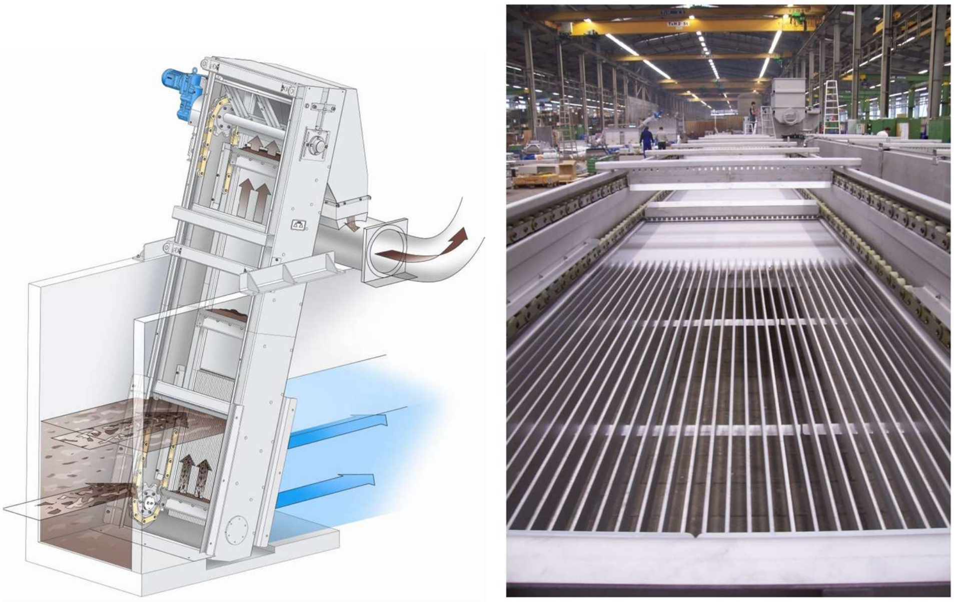


Figure 7-5: 12 mm front raked screen schematic

7.1.2.2 3.5 mm perforated plate mechanical screens

Mechanically operated perforated plate screens with 3.5 mm apertures will be installed as the final screening step before the water flows to the degritters. Figure 7-6 shows a schematic of the perforated plate screen in operation. The perforated plates continuously rotate, lifting screenings out of the water. A counter-rotating brush roller scrapes the screenings off the perforated plates and into a water launder which conveys the screenings to a washer compacted. Figure 7-7 shows the perforated plate aperture configuration.

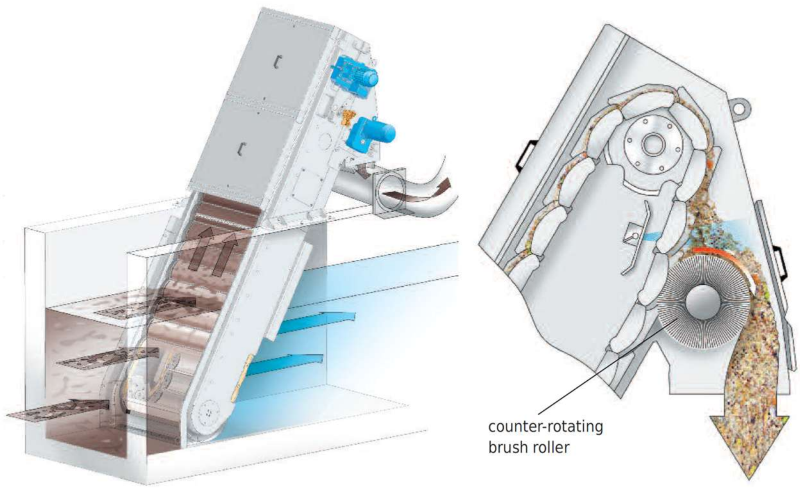


Figure 7-6: Perforated plate screen schematic

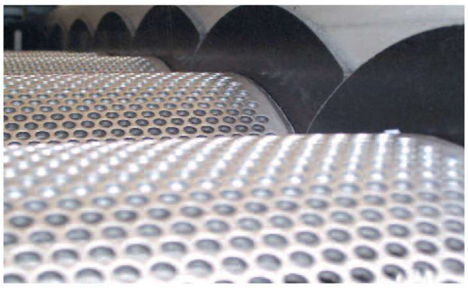


Figure 7-7: Perforated plate aperture configuration

7.1.3 Vortex degritters

The wastewater is hydraulically split between three hydraulically assisted vortex degritters. Each of the three degritter trains can be isolated by means of sluice gates installed upstream and downstream of each respective degritter.

A bypass channel is 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 channel is sized to cater for the full PWWF of train 1 of the inlet works. The bypass channel joins the main distribution box downstream of the degritter channels. The design parameters for the vortex degritters are indicated in Table 7-4. The vortex degritters are designed to have a duty standby configuration as described in Table 7-5.

Table 7-4: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 7-5: Vortex degritter configuration

Vortex Degritters	**No. Installed**	**Duty**	**Duty**
		Hydraulic Design Peak (75 MLD)	Process Design Peak (57.5 MLD)
Vortex Degritters – 4 m diameter	3	3	2
Degritter paddles – 0.3 m/s rotation speed	3	3	2
Grit classifiers	3	3	2

The layout of the vortex degritters is shown in Figure 7-8. The grit collected in the degritter collection sumps is removed with an airlift pump which creates a vacuum and sucks the grit through a pipe that discharges the grit and water mixture into a grit classifier. The grit cleaning and disposal operation is discussed in Section 7.1.4.2.

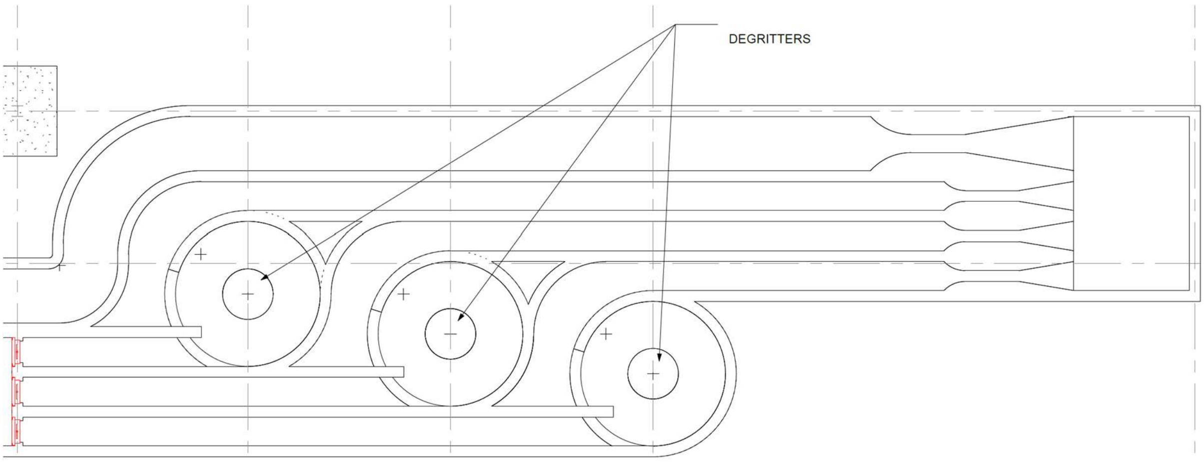


Figure 7-8: Vortex degritter layout

7.1.4 Regional Grit and Screenings Facility

George Municipality has identified Gwaing WWTW as a possible location for a regional degritting and screening facility for the 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 7.1.4.2 below.

The proposed location of the regional screenings and degritting facility is adjacent to the proposed new inlet works as shown in Figure 7-9.

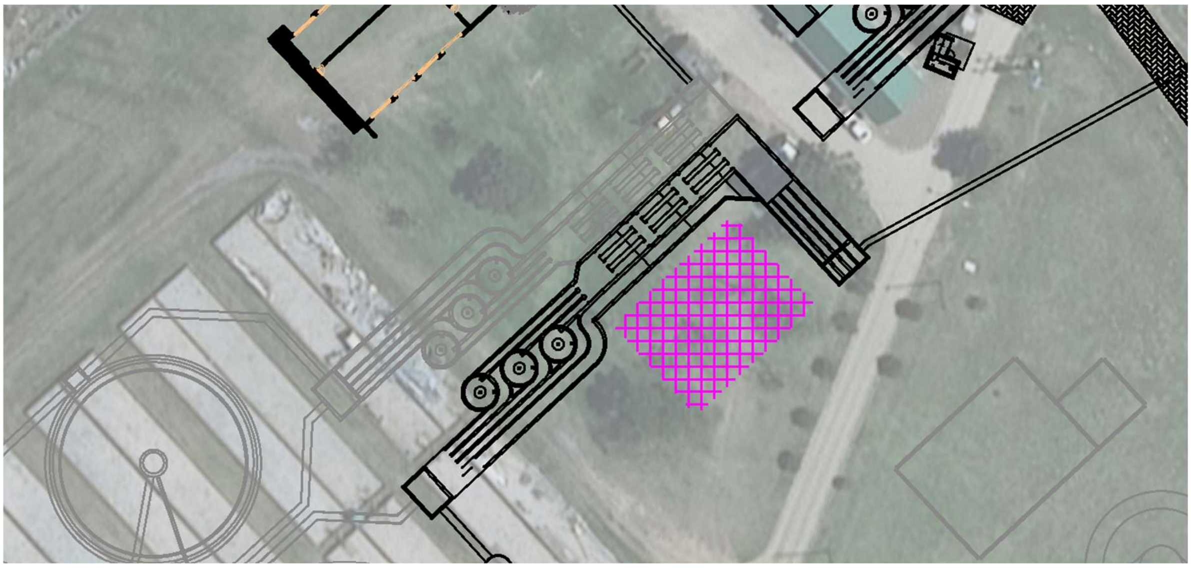


Figure 7-9: Regional screening and degritting position

7.1.4.1 Screenings washing 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 7-6 shows the average screening volumes generated by each wastewater treatment works per day on average between July 2022 and December 2023.

Table 7-6: Average daily screening volumes generated at each WWTW between July 2022 and December 2023

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

The volume of screenings generated is 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 are as follows:

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

The proposed screenings process flow diagram is shown in Figure 7-10.

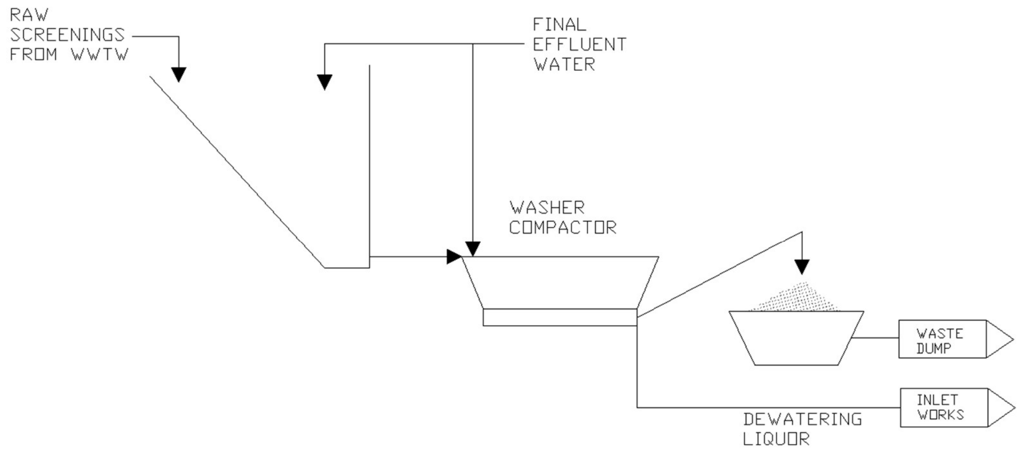


Figure 7-10: Regional screenings facility process flow diagram

The screenings received from the various WWTWs will be dumped into a collection chamber. The screenings generated at Gwaing WWTW will be discharged into the sump via a water launder from the screens at the inlet works. The raw screenings will be transported to a washer compactor by means of a screw conveyor. Figure 7-11 shows a typical installation and diagram of a screenings washer compactor.

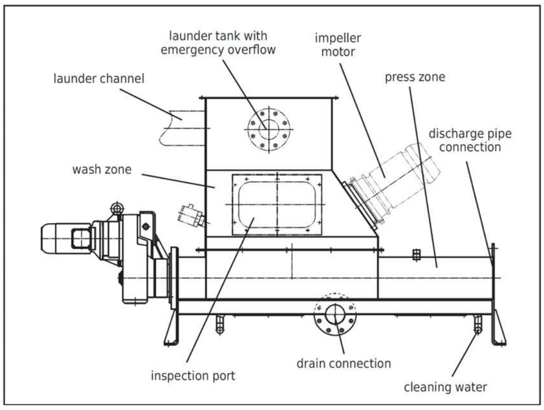


Figure 7-11: Typical installation and diagram of a screenings washer compactor

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 7-10 shows the process flow diagram for the screenings facility.

7.1.4.2 Degritting facility

The proposed regional grit facility will be receiving grit from Uniondale WWTW, Outeniqua WWTW, Kleinkrantz WWTW, on-site from Gwaing WWTW and from various pumpstations. The average daily grit volumes generated by each of the four-wastewater treatment works between July 2022 and December 2023 are summarised in Table 7-7.

Table 7-7: Average daily grit volumes generated at each WWTW between July 2022 and December 2023

	Average Daily Grit Volume (m^3)
Gwaing WWTW	0.107
Outeniqua 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 to standard that it is fit for commercial use or disposal. 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 (between July 2022 and December 2023). 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 7-12 shows the process flow diagram of the mechanical grit washing system.

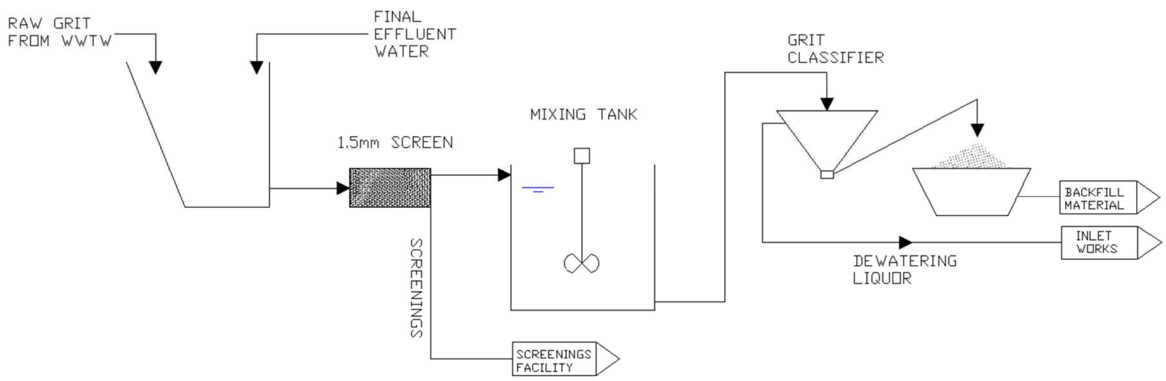


Figure 7-12: Mechanical grit washing system process flow diagram

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 transported to the adjacent screening facility via a water launder or mechanical auger. 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.

During the Detail Design stage, it is recommended that a grit sample be taken from each of the wastewater treatment works and various pumpstations 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.

7.2 Biological Reactors

The existing reactor in Module A is operated as a UCT (University of Cape Town) process. The reactor contains anaerobic, anoxic and aerobic zones. Together with the appropriate recycle streams, this enables the UCT process to achieve enhanced biological phosphate removal. To extend Gwaing WWTW's capacity to 22 MLD (Phase B), an additional reactor with the same volume as the existing one is required. The position of both reactors is depicted in Figure 7-13.

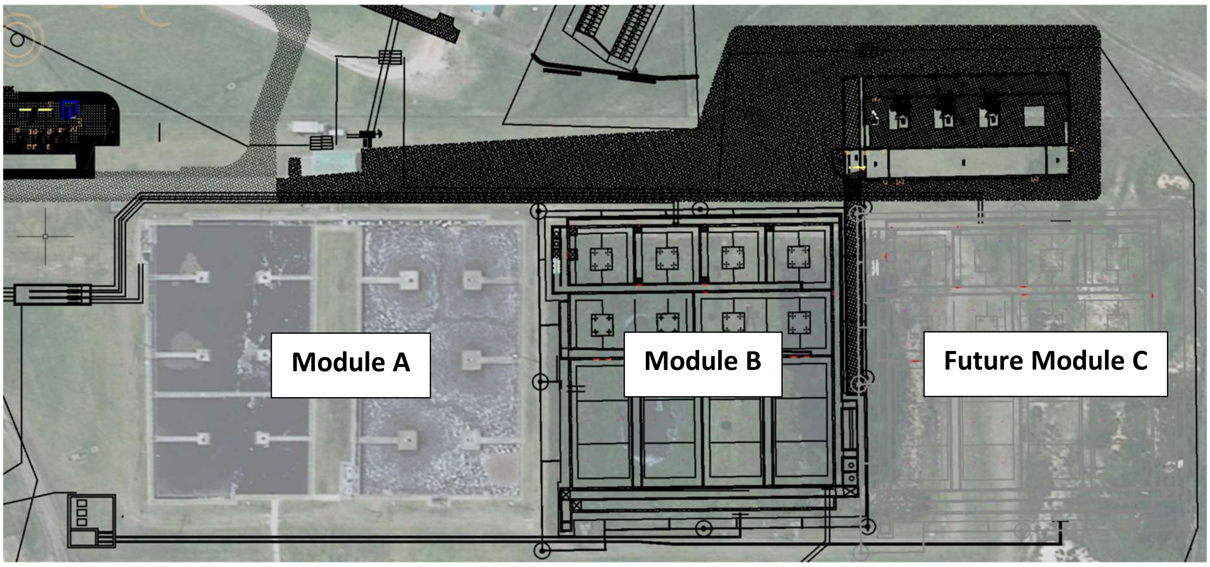


Figure 7-13: Reactor site layout configuration

A pipe from the new inlet works will bring screened and degritted raw sewage to the biological reactor flow splitter box (situated on the left side of Figure 7-13). The flow split is achieved by equally sized open-channel rectangular flumes, one for each reactor. 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. 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 7-13. From there the WAS is pumped to the sludge holding tanks at the dewatering building. The WAS handling is discussed in more detail in Section 7.5.

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 7-14.

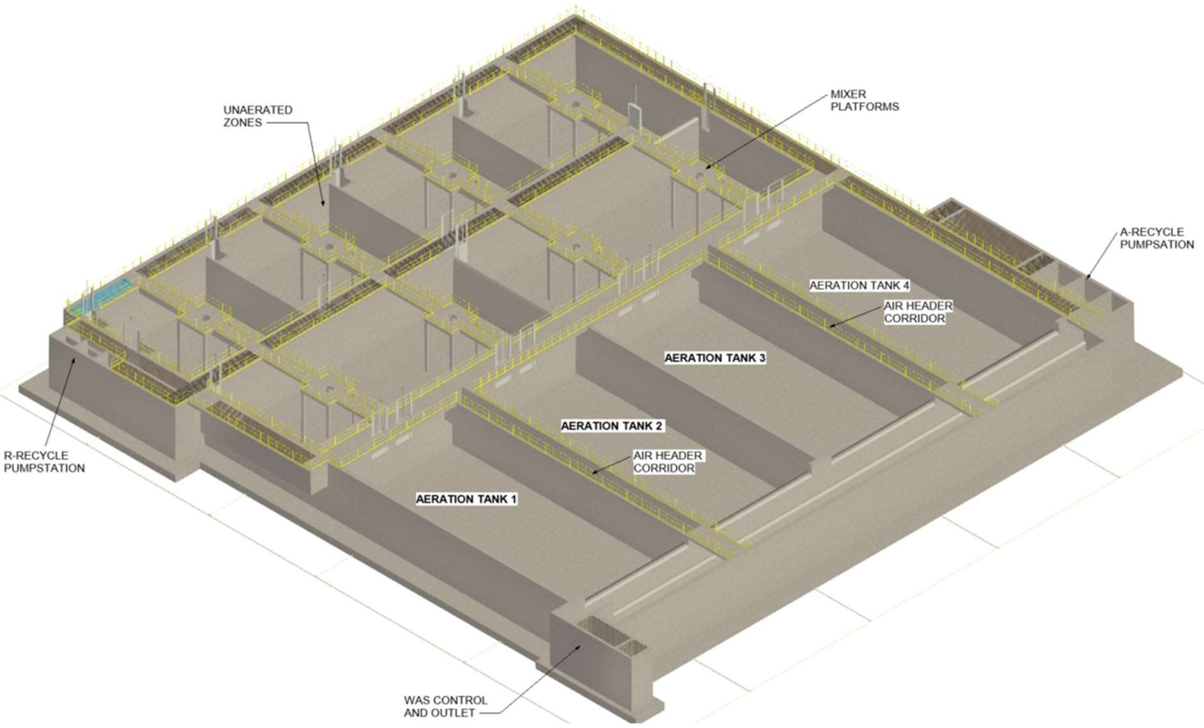


Figure 7-14: Biological reactor 3D model

Figure 7-15 shows the layout of the new reactor for module B and the blue arrows the direction of flow through the reactor. The reactor consists of 5 unaerated zones which can utilized as either anaerobic or anoxic zones. Zone 6 is designed as a swing zone that can be operated as either aerobic or anoxic depending on the process configuration selected.

The unaerated zones will be mechanically mixed with vertical shaft mixers, while the swing zone will be mixed with a vertical shaft mixer installed with a specially designed sparger system under the mixer body (see Figure 7-16). The equipment for the reactor is discussed in more detail in the next Section 7.2.1. For the aeration zones, air will be introduced into the mixed liquor contained in the aerobic zones by means of a fine bubble diffused aeration system for Module B. The aeration system and blower house are discussed in more detail in Section 7.7.

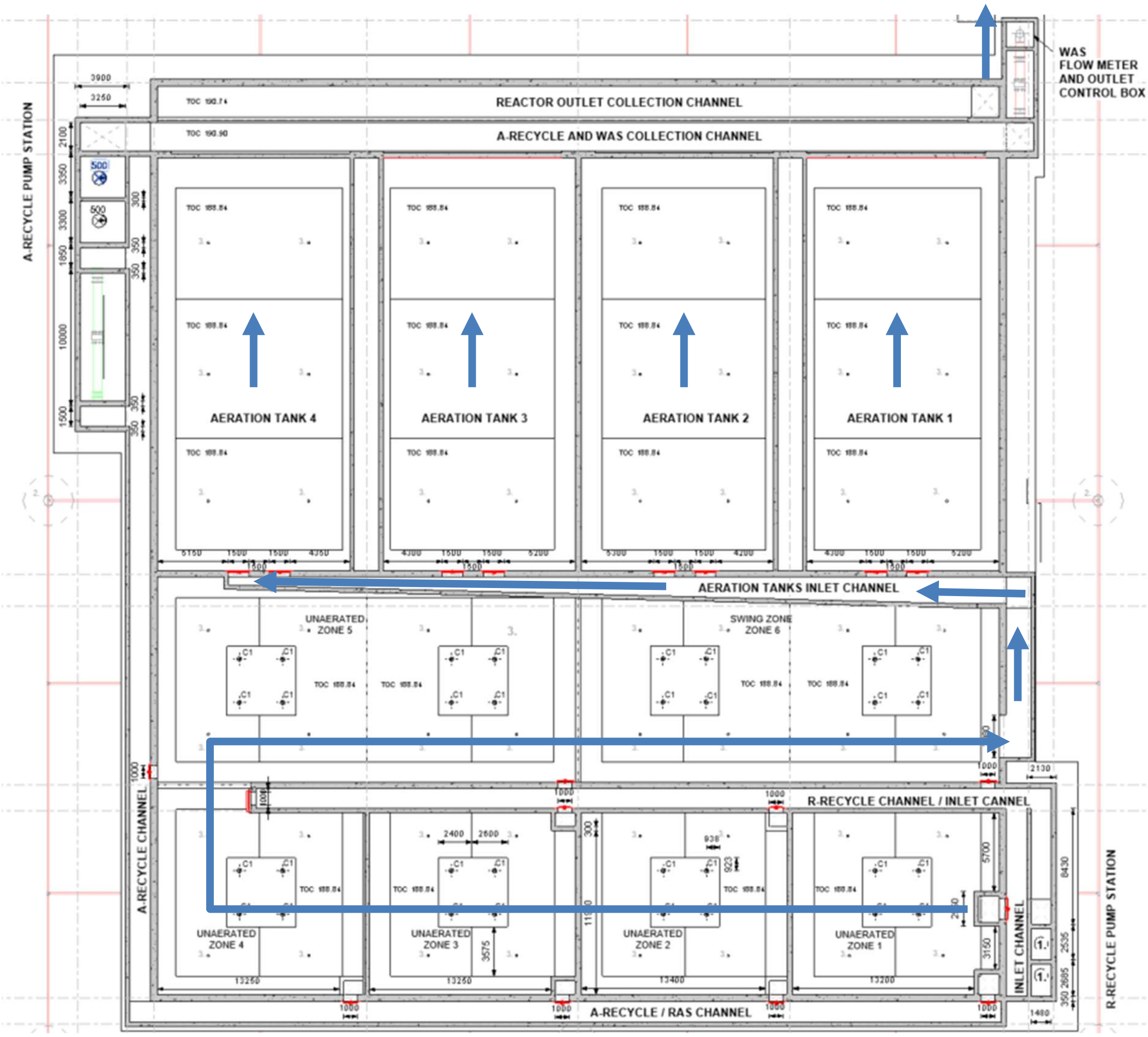


Figure 7-15: Layout of the New Reactor for Module B

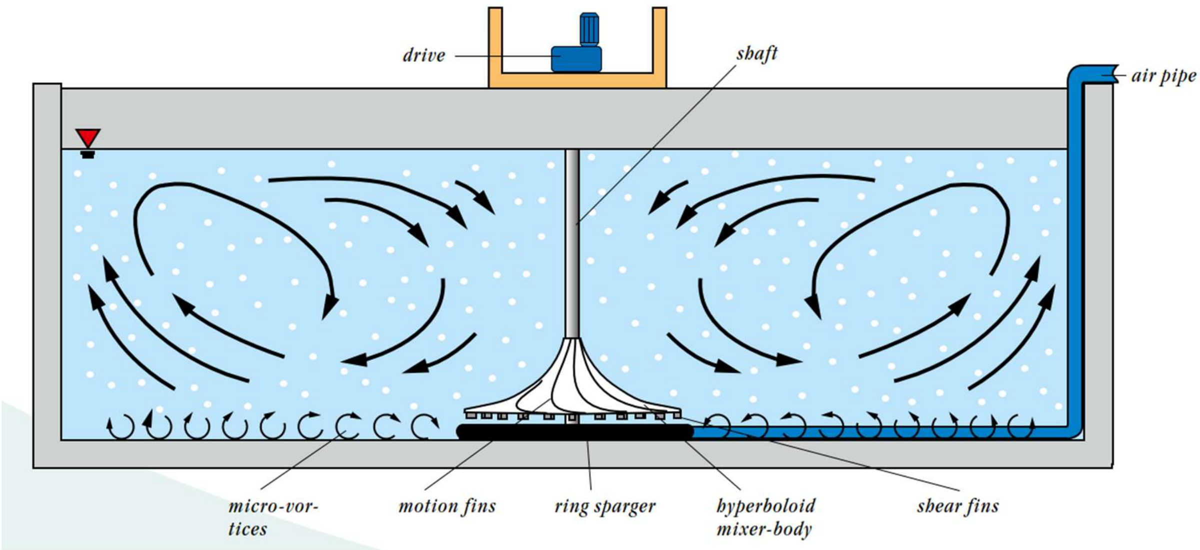


Figure 7-16: Mixer with specially designed sparger system under the mixer-body

7.2.1 Mechanical Equipment for Reactor

The successful operation of the new biological reactor necessitates mixers, recycle pumps, WAS pumps, penstocks, and a blower and FBDA system. A summary of the mechanical equipment (excluding the blower and FBDA system discussed in Section 7.7) is provided in Table 7-8.

Table 7-8: Mechanical Equipment for the New Reactor

Equipment	Description	Duty+Standby (Redundancy)	Operation
Mixers	Vertical shaft mixer with a hyperboloid-shaped mixer body installed close to the bottom of the reactor	8	Fixed speed with gearbox
Mixer with integrated aeration system for swing zone	Vertical shaft hyperboloid-shaped mixer with a specially designed sparger system under the hyperboloid mixer-body	2	Fixed speed with gearbox
Aeration Control Valve	Jet Control valve	2	Controlled to a set DO level.
A-Recycle Pumps	Vertically installed axial flow pumps	1+1	VSD controlled based on a set flow rate and level in the suction channel
R-Recycle Pumps	Vertically installed axial flow pumps	1+1	VSD controlled based on a set flow rate and level in the suction channel
Penstocks	Channel Gates (sealed on 3 sides)	20	Actuated penstocks with an option to automatically open and close based on the process selected.
WAS Pumps	Positive displacement sludge pumps	1+1	VSD controlled to achieve a set flow rate (based on the selected sludge age) to WAS holding tanks

The proposed hyperboloid-shaped mixers exhibit remarkable efficiency gains. These mixers can achieve energy savings of 40% to 60% when compared to radially backswept mixers. Their unique design, depicted in Figure 7-17, contributes to this improved performance.

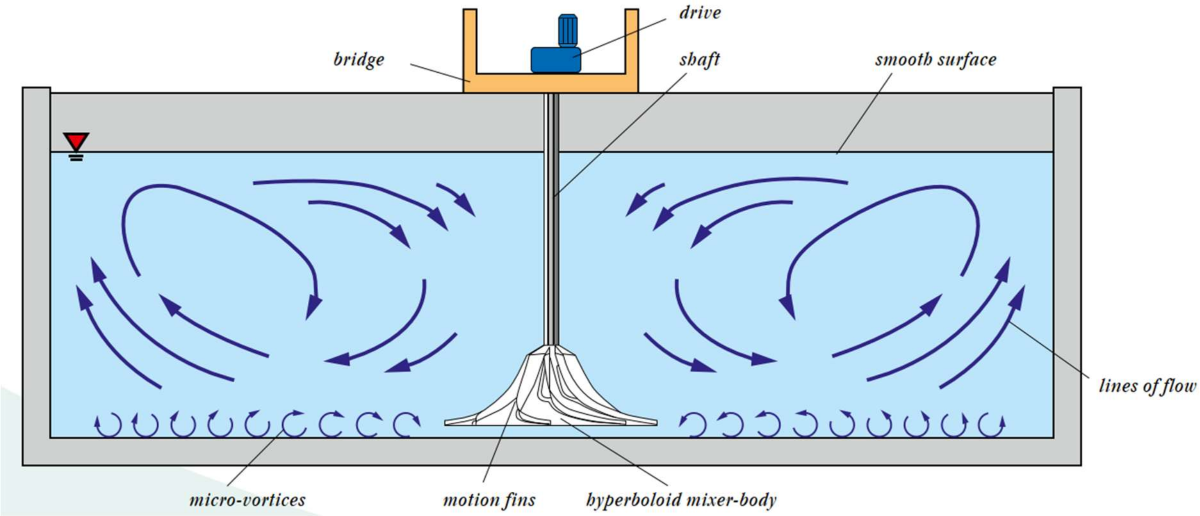


Figure 7-17: Schematic Indicating the functioning of the hyperboloid mixer

7.3 Settling and Return activated sludge (RAS)

The existing plant currently has two operational flat-bottom SSTs. Phase A includes the construction of six new 25 m SSTs with conical bottom, resulting in a total of 8 SSTs. All 8 SSTs will be dedicated to Reactor Module A. At the commissioning of Phase B, which includes an additional reactor module (Module B), four SSTs will be dedicated to each of the reactor modules. The layout of Phase A is shown in Figure 7-18 below.

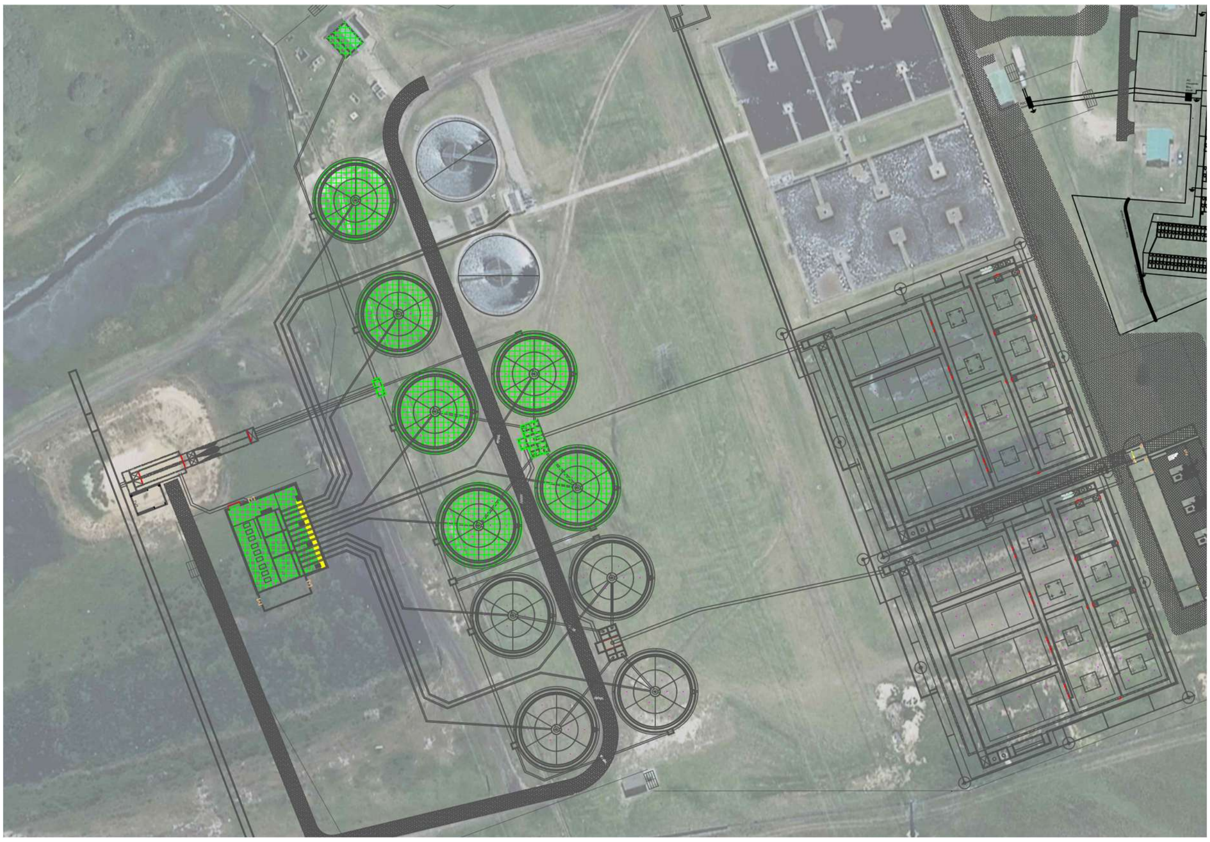


Figure 7-18: Phase A SST layout

The mixed liquor from each bioreactor gravitates to a division chamber which hydraulically splits the flow equally to the SSTs dedicated to that reactor. Due to the topography of the site, the existing SSTs were constructed a few meters lower than the biological reactors. For ease of operation in terms of flow split between the SSTs (new and old) and return activated sludge control, the new SSTs are designed to have the same top water level as the existing SSTs.

7.3.1 Energy recovery (hydropower turbine)

The level difference between the reactors and SSTs results in excess head available. This presents an opportunity for potential energy recovery in the form of a hydropower turbine. The maximum available head when taking into account all possible head losses between the bioreactors and hydropower turbines upstream of the SST division chambers, is 1.86 m (at PWWF). Table 7-9 shows the average potential energy recovery with a hydropower turbine installed upstream of the SST division chamber during Phase A and Phase B.

Table 7-9: Potential power generation from hydropower turbines

	Units	Phase A	Phase B
Q_{inflow} (ADWF)	MLD	13.20	22.00
RAS recycle ratio w.r.t. ADWF		2	2
Q_{RAS}	MLD	26.40	44.00
$Q_{Inflow + RAS\ recycle}$	MLD	39.60	66.00
No. Turbines	No.	2	2
$Q_{flow\ per\ Turbine}$	MLD	19.80	33.00
$Q_{flow\ per\ Turbine}$	m^3/s	0.229	0.382
H (Head available for energy recovery)	m	1.86	1.86
Turbine Efficiency		0.8	0.8
Power generated per turbine	**kW**	**5.23**	**8.71**

Installation of hydropower turbines would have a large capital cost and would increase the operational complexity and cost of the plant. The payback period per turbine, based on R2.04 per kWh, based on ADWF is 24 years for Phase A flow rates and 14.5 years for Phase B flow rates. This payback period is merely for the capital cost and does not include any maintenance or operational costs. Due to the low cost-benefit of hydropower turbines, it is not included in the upgrade of the plant. Table 7-10 shows a summary of the payback period of the turbines for each phase.

Table 7-10: Capital cost payback period of turbine

	Phase A	Phase B
Operational Hours per day	24	24
Total kWh per day	125.4	209.0
R/kWh =	R 2.04	R 2.04
Potential Cost saving per day	R 255.82	R 426.37
Potential Cost saving per anum	R 93 374.02	R 155 623.36
Capital cost of Turbine	R 2 250 000.00	R 2 250 000.00
Payback period (Years)	**24.1**	**14.5**

If the George Municipality would like a detailed life-cycle cost analysis, including operational and maintenance costs, a detailed investigation can be done during the Detail Design phase of the project.

7.3.2 Secondary Settling Tanks (SSTs)

The new SSTs are designed to be 25 m in diameter (the same as the existing SSTs) and are designed with a side wall depth of 4.0 m and sloped floors. The overflow collection launders are internal launders and form a Stamford-type baffle. The clear water overflows into the launder and flows to the disinfection unit process. A rotating mechanical bridge with scrapers guides the sludge that collects on the floor to the collection sump in the centre of the SST. From the sump, the activated sludge gravitates to the return activated sludge (RAS) pumpstation where it is pumped back to the reactor. A typical cross-section of the 25 m SSTs is shown in Figure 7-19.

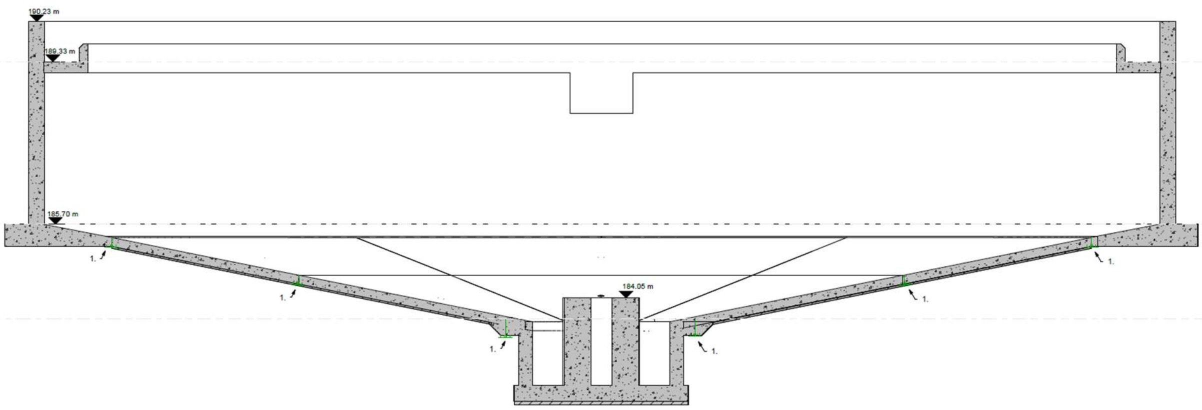


Figure 7-19: Typical 25m SST cross-section

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.

7.3.3 Return Activated Sludge Pumpstation

The underflow return activated sludge (RAS) from each SST gravitates to the RAS pumpstation. The RAS or s-recycle ratio is 2 times the average dry weather flow (ADWF). Each SST has a dedicated underflow pipeline that leads into the RAS Pumpstation. In the pumpstation each line has a dedicated magnetic flow meter. The RAS pumpstation sump contains a fixed weir level where the flow from the SST enters, thus ensuring a constant flow rate (fixed head losses). The rate of flow from the SST is adjusted by slightly opening or closing a valve upstream of the sump. Closing the valve slightly will induce an increased local head loss and thus reduce the RAS flow, opening the valve will have the inverse effect. The RAS should be adjusted from time to time based on the number of operational SSTs and the ADWF entering the plant.

The RAS pumpstation is designed so that each reactor module can be operated separately or be integrated. Each module has its own sump with dedicated recycle pumps. A two-duty and one standby pump configuration per module is implemented.

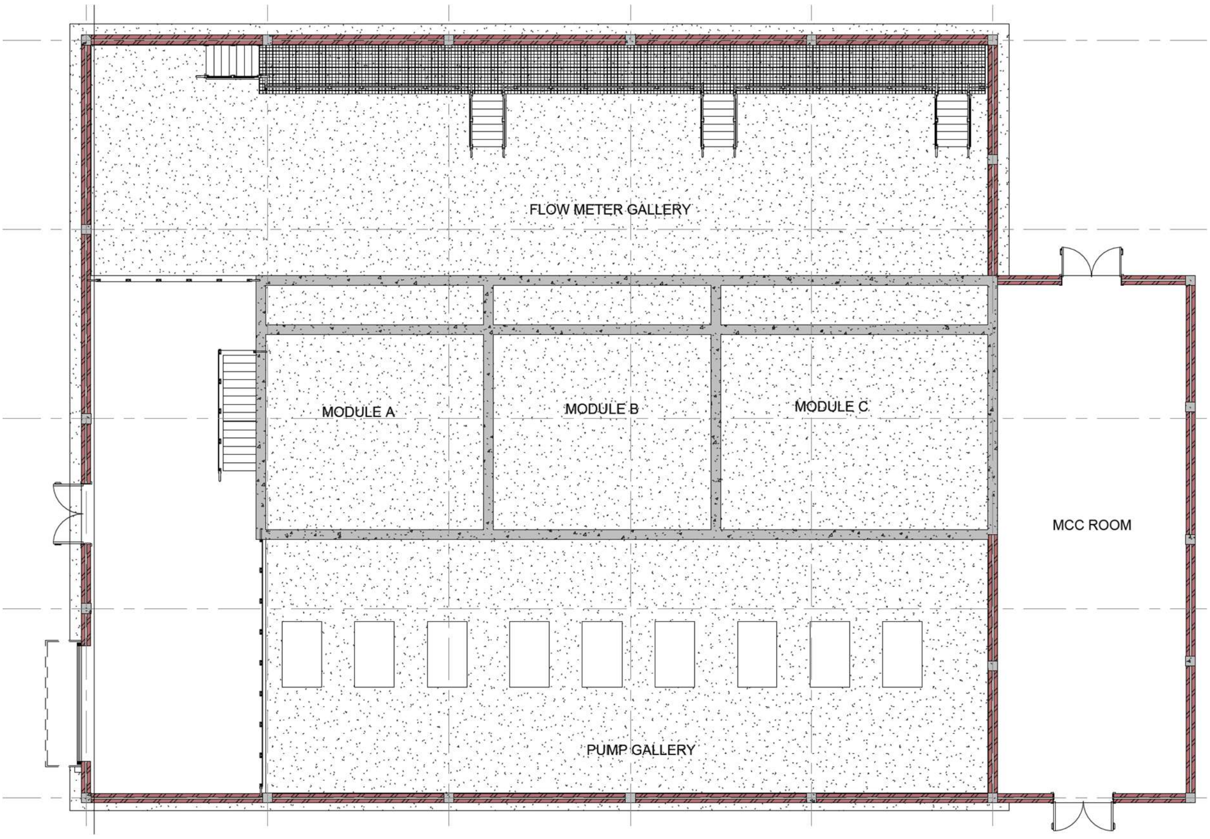


Figure 7-20: RAS pumpstation floor plan layout

7.4 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 future upgrades, additional banks can 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 7-21.

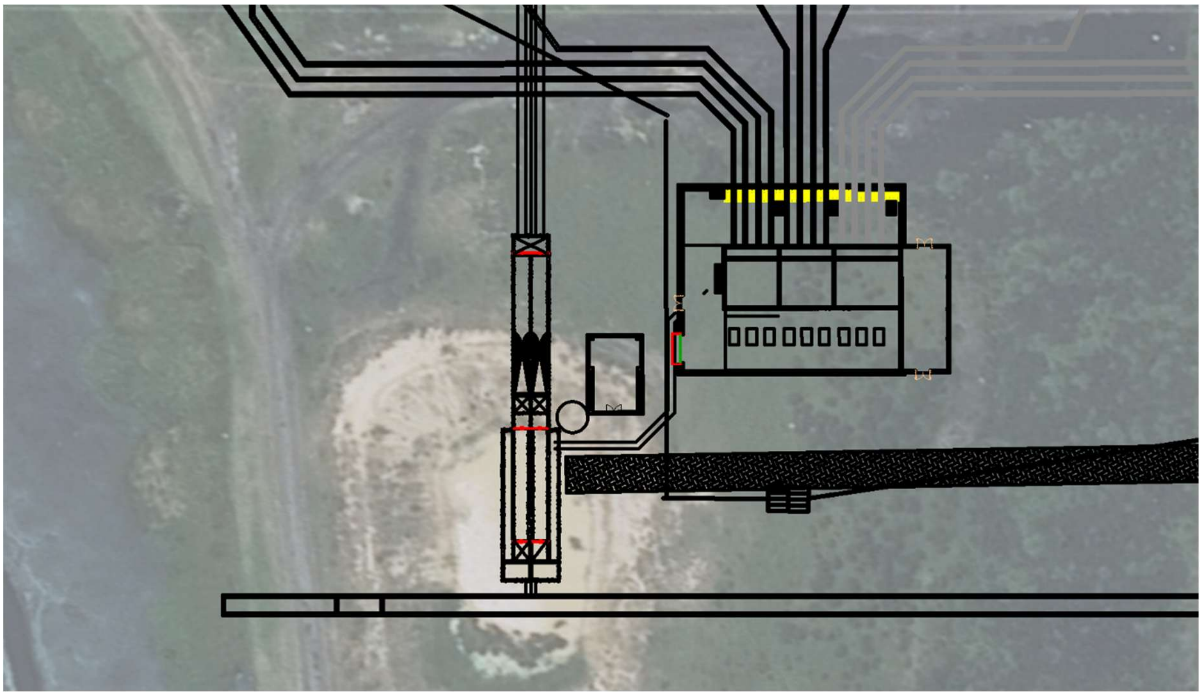


Figure 7-21: UV Channels site location

An isometric view, plan view and section of the UV channels structure are shown in Figure 7-22. 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.

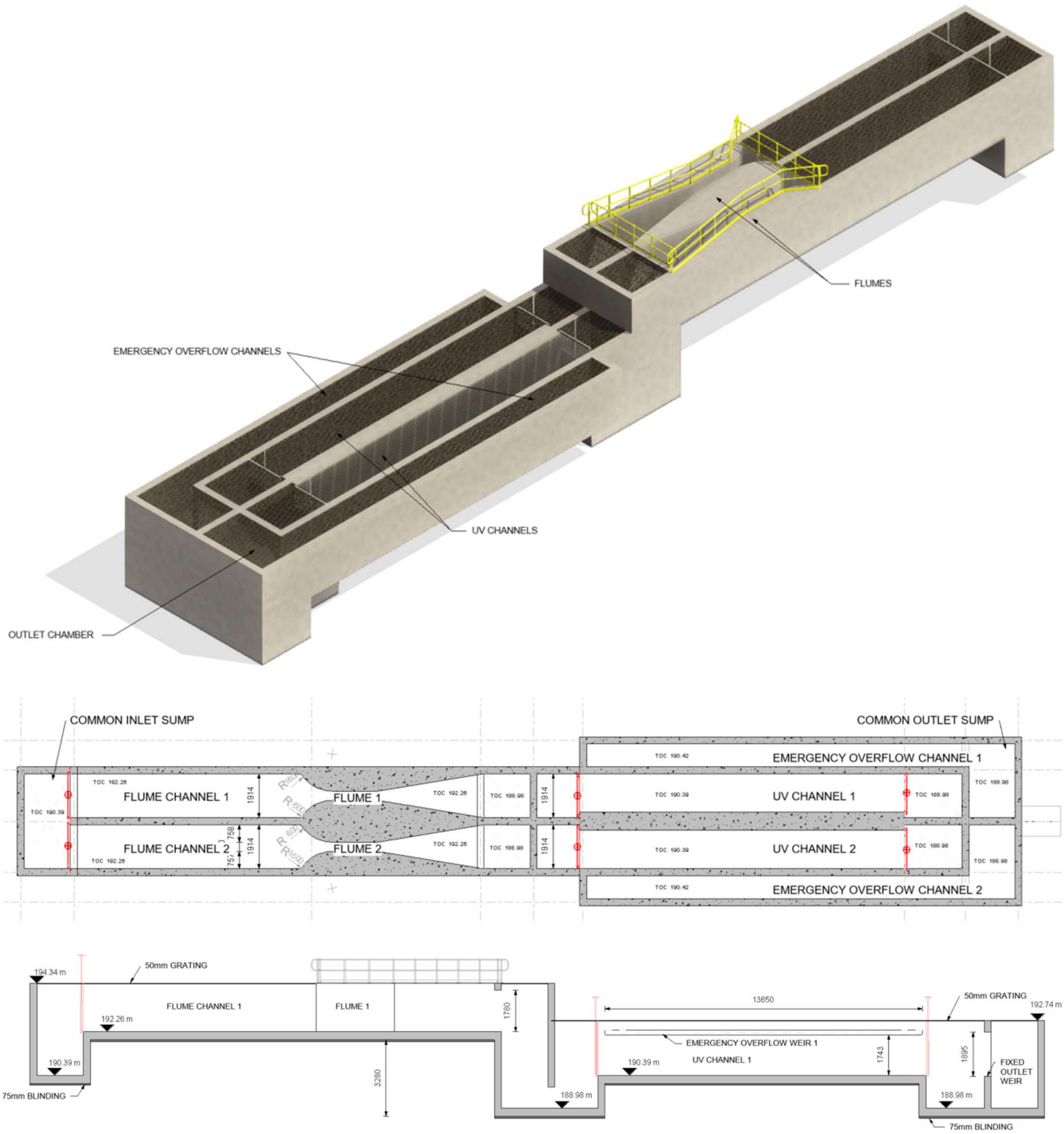


Figure 7-22: Plan and Section of the UV Channels Structure

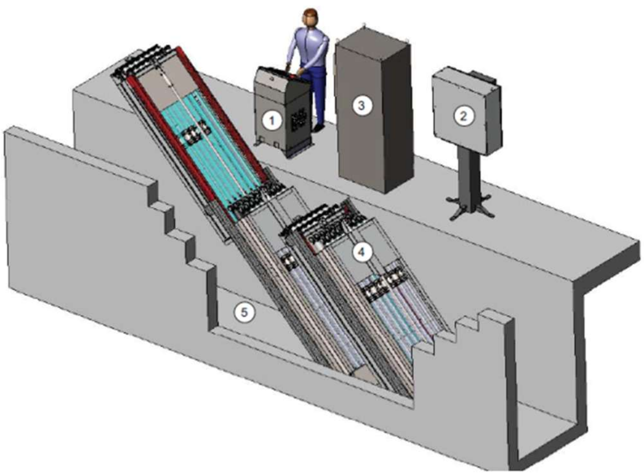
7.4.1 Mechanical Equipment for UV Disinfection

A summary of the mechanical equipment for the UV disinfection system is provided in Table 7-11. The proposed type of UV disinfection is low-pressure high-output (LPHO) lamps in banks installed in an open channel. To assist with maintenance it is proposed that the UV banks be supplied with an automatic raising mechanism. Final effluent poses a high degree of lamp sleeve fouling risk and hence mechanical and chemical cleaning of the sleeves is advised.

Table 7-11: Mechanical Equipment for the UV Disinfection System

Equipment	Description	Duty+Standby (Redundancy)	Operation
UV System	Low-pressure high output (LPHO) open channel UV disinfection system with automatic raising mechanism and mechanical and chemical cleaning	8 UV Banks total each with 25% redundancy (4 banks per channel)	Validated dose control (dose-pacing) as a function of flow rate, UVT and UV intensity in a validated dose equation
Modulating Weir Gates	Downward opening weir gates	2	Actuated weir gates that modulate to control a set level in the UV Channels
Penstocks	Channel Gates (sealed on 3 sides)	4	Penstocks for isolation and maintenance purposes

A system overview of a typical open channel UV installation (including electrical components and instrumentation) is shown in Figure 7-23.



1. Hydraulic System Center	2. System Control Center
3. Power Distribution Center	4. UV Bank
5. UV Channel	6. Level Controller (not shown)
7. Low Water Level Sensor (not shown)	8. High Water Level Sensor (not shown)

Figure 7-23: Components of an open channel UV System

7.5 Waste Activated Sludge (WAS)

7.5.1 WAS Pumping from Reactors

The waste activated sludge is pumped from the reactors to the dewatering building. Each of Reactors A and B will be equipped with a dedicated WAS pump set. The WAS pumps will draw MLSS from the Aerobic zones of Reactor A and B. The WAS pump rate will be carefully controlled to ensure that the desired sludge age is maintained. It is envisaged that a target wasting rate will be displayed on the HMI and SCADA to assist process controllers. The target wasting rate will take into account the average wasting rate over the previous 10 days, 20 days and 30 days (See Figure 7-24). The waste pump rate and hours can then automatically adjust to target the desired sludge age (current design is for a sludge age of 20 days).

It is important to note that the waste pumps will only pump when the WAS dewatering sump level (at the dewatering building) is below its maximum. Therefore, no wasting will happen if the beltpresses are not operating. If an insufficient quantity of WAS is dewatered the sludge age in the reactors will gradually increase, eventually resulting in sludge carry-over at the SSTs. Any downtime at the beltpresses will result in an increased sludge age in the reactors (refer to Section 7.5.3 for duty unit configuration). After any dewatering downtime, the wasting rate and dewatering rate will need to increase to get rid of the excess MLSS in the system and to lower the sludge age towards the desired 20 days.

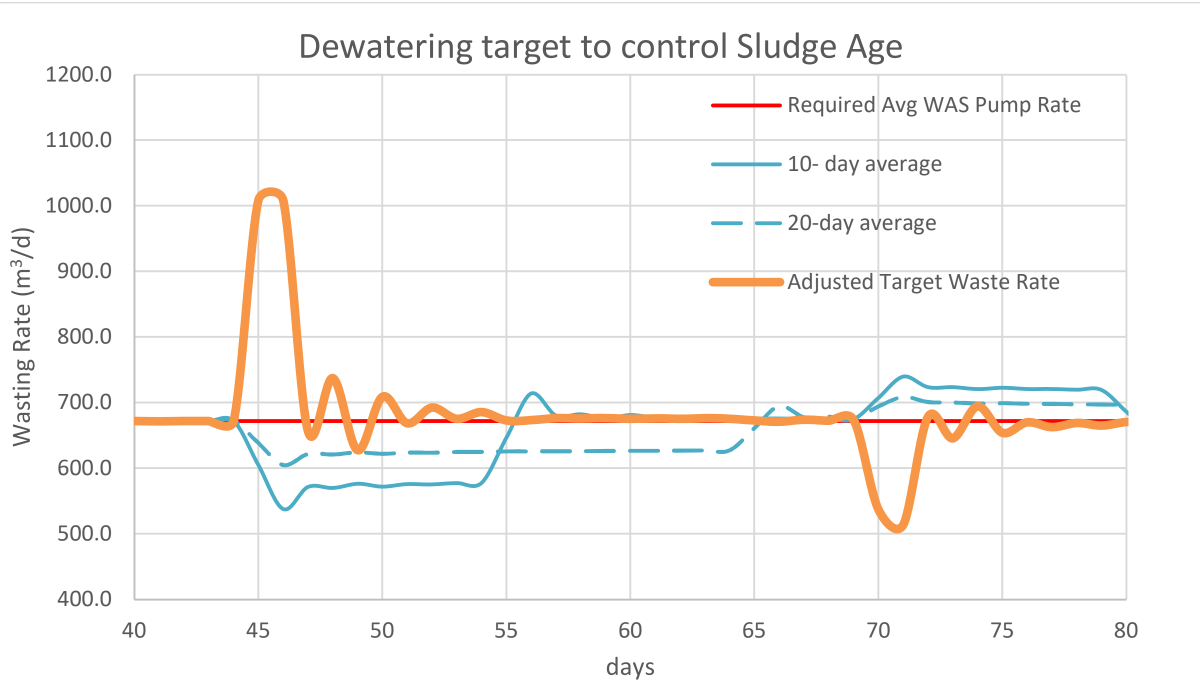


Figure 7-24: Target Wasting (& Dewatering) Rate - proposed HMI/SCADA information (example data shown with 2 days with zero dewatering at 45 days and 2 days of excessive dewatering at 70 days)

7.5.2 WAS Holding Sump

The WAS holding sumps at the dewatering plant need to be aerated as a means of mixing and to ensure that aerobic conditions prevail causing phosphate to remain bound in the poly-P accumulating organisms (PAOs). If the sump turns anaerobic the phosphates will be released to the bulk liquid and won't exit the system as dried sludge from the beltpresses but will escape as beltpress filtrate to be returned to the reactor. This will lead to increased effluent phosphate concentrations, nullifying the benefits of the UCT process as an enhanced biological phosphate removal (EBPR) process. Mixing also ensures that the feed to the beltpresses is homogenous and a consistent TSS concentration is fed to the beltpresses.

The existing WAS holding sumps do not need to be increased in size since this has little benefit. Increasing the sump size will necessitate larger aeration equipment and increased energy costs for aeration. Having a very large WAS holding sump could serve as a buffer between the reactor wasting and the dewatering plant, however it is more pragmatic to simply keep the MLSS in the reactor until it can be dewatered. The main purpose of the WAS holding sump is that of combining the flow from the respective reactors (and Outeniqua WWTW) and then feeding the individual beltpresses at suitable rates.

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 VSDs so that airflow rates can be varied as the levels rise and fall for optimal energy usage.

Medium bubble diffusers are less efficient at oxygen transfer than fine bubble diffusers, but are virtually maintenance-free. It is foreseen that a blower will be dedicated to each sump with a common standby for the two sumps.

The two sumps can be used to keep the WAS from Outeniqua WWTW and Gwaing WWTW separate when required or it can be combined by simply opening a valve between the two sumps. There needs to be communication by telemetry between the WAS holding sump and the WAS pumpstation at Outeniqua WWTW to ensure that the WAS is not pumped when the WAS sump is full.

Table 7-12: 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*

7.5.3 WAS Dewatering

The existing belt press capacities are shown in Table 7-13. The volume of sludge produced by each reactor at Gwaing WWTW is summarised in Table 7-14 for 1, 2 and 3 reactors operating at full capacity. It does not include the sludge from Outeniqua WWTW. At present the design philosophy is to optionally dewater sludge from Outeniqua WWTW on separate beltpresses or mixed 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 and results in maximum utilisation of the equipment.

Table 7-13: Existing Belt Press Capacities

Existing Beltpress Capacities	Hydraulic Capacity (m^3/hr)	Solids Loading Capacity (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 7-14: 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

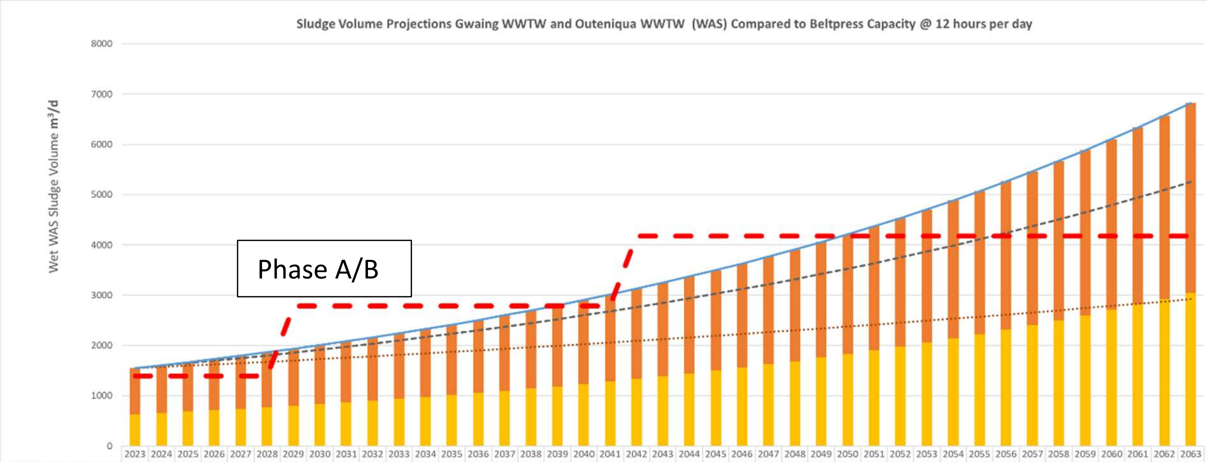
			1 Beltpress for Gwaing WWTW				2 Beltpresses for Gwaing WWTW				3 Beltpresses for Gwaing WWTW			
Raw UCT process -			Beltpress Operating Hours per day				Beltpress Operating Hours per day				Beltpress Operating Hours per day			
			24	16	12	8	24	16	12	8	24	16	12	8
			Hydraulic Loading Rate (m^3/hr)				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

The WAS generated at Gwaing WWTW for the ultimate capacity of the UCT Raw process is a total mass of 9879 kg/d. 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 7-13 according to the operating hours and the number of Reactors in operation.

The green text in Table 7-14 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. For Phase B there will be two dedicated beltpresses required for Gwaing WWTW.

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). This results in a total of six beltpresses required for the ultimate solution. For Phase B a total of four beltpresses are required, two dedicated to Gwaing WWTW and two dedicated to Outeniqua WWTW. Phase B will see the installation of two additional beltpresses to the existing two beltpresses.



	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042
Outeniqua WWTW	915	948	982	1018	1054	1093	1132	1173	1215	1259	1304	1352	1400	1451	1503	1558	1614	1672	1732	1795
Gwaing WWTW	634	660	686	714	742	772	803	835	868	903	939	977	1016	1056	1099	1143	1188	1236	1285	1337
Low [1.6%]	1549	1574	1599	1625	1651	1677	1704	1731	1759	1787	1816	1845	1875	1905	1935	1966	1997	2029	2062	2095
Med [3%-3.3%]	1549	1597	1647	1698	1750	1804	1860	1918	1977	2038	2101	2167	2234	2303	2374	2448	2523	2602	2682	2765
High [3.6%-4%]	1549	1608	1668	1731	1797	1864	1935	2008	2083	2162	2244	2328	2416	2507	2602	2700	2802	2908	3018	3132
beltpress capacity	1392	1392	1392	1392	1392	1392	2784	2784	2784	2784	2784	2784	2784	2784	2784	2784	2784	2784	2784	4176

	2043	2044	2045	2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063
Outeniqua WWTW	1860	1927	1996	2069	2143	2221	2301	2384	2470	2559	2651	2747	2846	2949	3055	3166	3280	3398	3521	3648	3780
Gwaing WWTW	1390	1446	1503	1564	1626	1691	1759	1829	1902	1978	2058	2140	2226	2315	2407	2503	2604	2708	2816	2929	3046
Low [1.6%]	2128	2162	2197	2232	2268	2304	2341	2378	2417	2455	2494	2534	2575	2616	2658	2700	2744	2788	2832	2878	2924
Med [3%-3.3%]	2851	2939	3031	3124	3221	3321	3424	3530	3640	3753	3869	3989	4113	4240	4372	4508	4648	4792	4941	5094	5252
High [3.6%-4%]	3250	3373	3500	3632	3769	3912	4060	4213	4372	4537	4709	4887	5072	5264	5463	5669	5884	6106	6337	6577	6826
beltpress capacity	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176	4176

Figure 7-25: Sludge Volume Projection Gwaing WWTW and Outeniqua WWTW WAS compared to beltpress Capacity @ 12 hours per day.

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 or B. The operational flexibility to dewater WAS from Outeniqua WWTW separately or to mix it with that of Gwaing WWTW will be maintained. A Concept Layout of the upgraded dewatering plant is shown in Figure 7-26.

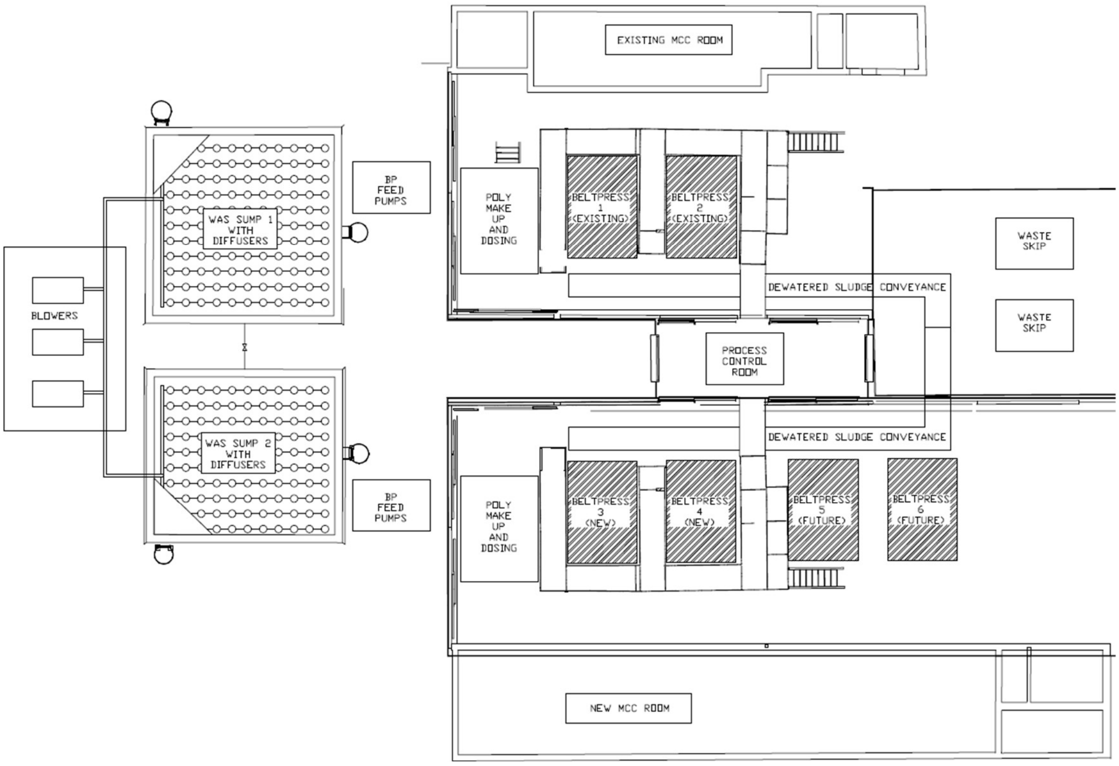


Figure 7-26: Concept Layout of upgraded WAS Dewatering Plant

7.5.4 Mechanical Equipment for WAS Dewatering

The following main mechanical equipment, as summarised in Table 7-15, is proposed for the WAS dewatering plant as part of Phases A and B:

Table 7-15: Was Dewatering Mechanical Equipment

Equipment	Description	Duty+Standby (Redundancy)	Operation
Reactor A WAS Pumps	Positive displacement sludge pumps	1+1	VSD Controlled based on sludge age and WAS sump level
Reactor B WAS Pumps	Positive displacement sludge pumps	1+1	VSD Controlled based on sludge age and WAS sump level
WAS Sump Blowers	Rotary Lobe Blowers (roots type)	2+1	VSD controlled according to the WAS sump level
WAS Sump Diffusers	Fine bubble or medium bubble diffuser network and pipework.	Each WAS sump can be isolated for maintenance.	Blower controls aeration rate. Safety interlocks prevent over-aeration and damage to diffusers.
Beltpress Feed Pumps for Beltpresses 3 and 4	Positive displacement sludge pumps	1 for BP 3 1 for BP 4 Common standby	VSD controlled according to beltpress capacity, TSS concentration and WAS sump level.
Beltpresses 3 and 4	1.8m wide Gravity table (linear screens) mounted on a 1.8m wide filter belt press	BP 3 = duty BP 4 = duty (spare capacity in longer operating hours)	OEM control logic according to the solids and hydraulic loading rates
Poly Make-Up and Dosing Equipment	Packaged type or discrete equipment poly dosing system dedicated to BP3 and BP4.	Minimum 50% redundancy on all key components	Dosing rate controlled according to beltpress feed flow rate and TSS concentration
Poly storage facility	Storage facility for polymer for BP1, 2, 3 and 4.	Single storage area for all 4 beltpress trains	
Dried Sludge Conveyance	Screw conveyors for dried sludge to waste skips	Common duty for BP 3 and BP 4	Always on when beltpress operates.
Washwater Booster pumps	The wash water ringmain will supply wash water to the dewatering plant. Additional pressure-boosting pumps may be required for beltpress spray water.	1+1	VSD controlled pump set to maintain constant pressure

Figure 7-27 shows a typical belt press installation.

Figure 7-27: Example beltpress installation showing 3 filter belt presses

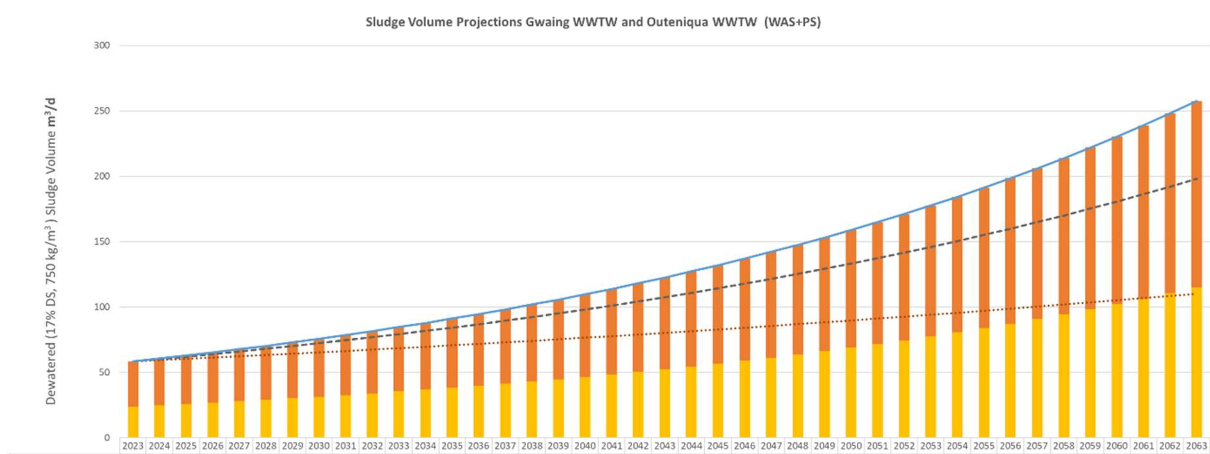
WAS is dewatered to between 15% and 20% dry solids. Table 7-16 shows the expected mass and volume of WAS generated for Gwaing WWTW (only) when all three reactors operate at capacity with the UCT Raw Process.

Table 7-16: 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 bulk 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

The dewatered sludge volumes of Gwaing WWTW and Outeniqua WWTW combined are shown in Figure 7-28 with low, medium and high catchment population growth scenarios plotted over time.



	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043
Outeniqua WWTW	35	36	37	38	40	41	43	44	46	47	49	51	53	55	57	59	61	63	65	68	70
Gwaing WWTW	24	25	26	27	28	29	30	31	33	34	35	37	38	40	41	43	45	47	48	50	52
Low [1.6%]	58	59	60	61	62	63	64	65	66	67	68	70	71	72	73	74	75	77	78	79	80
Med [3%-3.3%]	58	60	62	64	66	68	70	72	75	77	79	82	84	87	90	92	95	98	101	104	107
High [3.6%-4%]	58	61	63	65	68	70	73	76	79	82	85	88	91	95	98	102	106	110	114	118	123

	2044	2045	2046	2047	2048	2049	2050	2051	2052	2053	2054	2055	2056	2057	2058	2059	2060	2061	2062	2063
Outeniqua WWTW	73	75	78	81	84	87	90	93	96	100	104	107	111	115	119	124	128	133	138	143
Gwaing WWTW	55	57	59	61	64	66	69	72	75	78	81	84	87	91	94	98	102	106	110	115
Low [1.6%]	82	83	84	86	87	88	90	91	93	94	96	97	99	100	102	103	105	107	108	110
Med [3%-3.3%]	111	114	118	121	125	129	133	137	141	146	150	155	160	165	170	175	181	186	192	198
High [3.6%-4%]	127	132	137	142	147	153	159	165	171	178	184	191	198	206	214	222	230	239	248	257

Figure 7-28:Dewatered Sludge Volume Projection Gwaing WWTW and Outeniqua WWTW WAS

7.6 Biosolids Beneficiation Facility

7.6.1 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.

7.6.2 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 7-17. The corresponding phases as defined in Section 6.3 are indicated in red text.

Table 7-17: WAS 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/m^3	Wet Volume	Wet Mass	DS %	Bulk ρ kg/m^3	Moist Volume	Moist Mass	DS %	Bulk ρ kg/m^3	Dry Volume	Dry Mass
WAS for 1 Reactor	3293	0.490%	1000	672 m^3/d	672 t/d	17%	750	25.8 m^3/d	19 t/d	70%	750	6.3 m^3/d	4.7 t/d
WAS for 2 Reactors **[Phase B]**	6586			1344 m^3/d	1344 t/d			51.7 m^3/d	39 t/d			12.5 m^3/d	9.4 t/d
WAS for 3 Reactors **[Phase C]**	9879			2016 m^3/d	2016 t/d			77.5 m^3/d	58 t/d			18.8 m^3/d	14.1 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. Refer to Figure 7-25 and Figure 7-28 for the sludge volumes including Outeniqua WWTW Sludge.

7.6.3 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.

7.6.4 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.

7.6.5 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.

7.6.5.1 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.

7.6.5.2 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).

7.6.5.3 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.

7.6.5.4 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

7.6.5.5 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 7-18). 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.

7.6.5.6 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 7-18: 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%

7.6.5.7 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.

7.6.6 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 7-29 shows an example of typical storage bunds that may be viable for Gwaing WWTW.

Figure 7-29: An Example of sludge storage bunds with concrete floors and translucent roof covers.

7.6.7 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 7-30), 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.

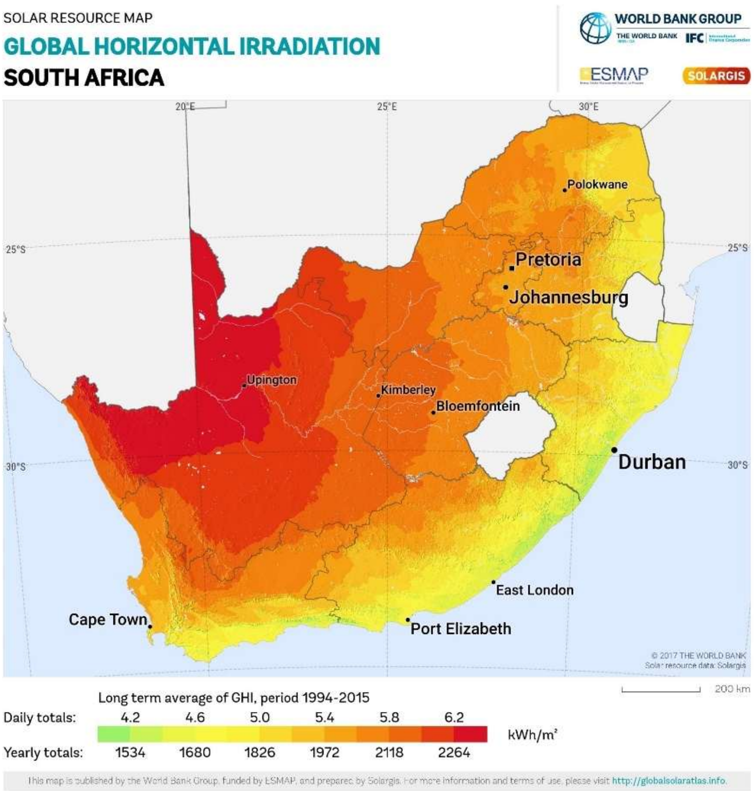


Figure 7-30: Horizontal solar irradiation map of South Africa.

The weather station nearest to the Gwaing WWTW was used to determine the weather conditions for 2024 as shown in Figure 7-31.

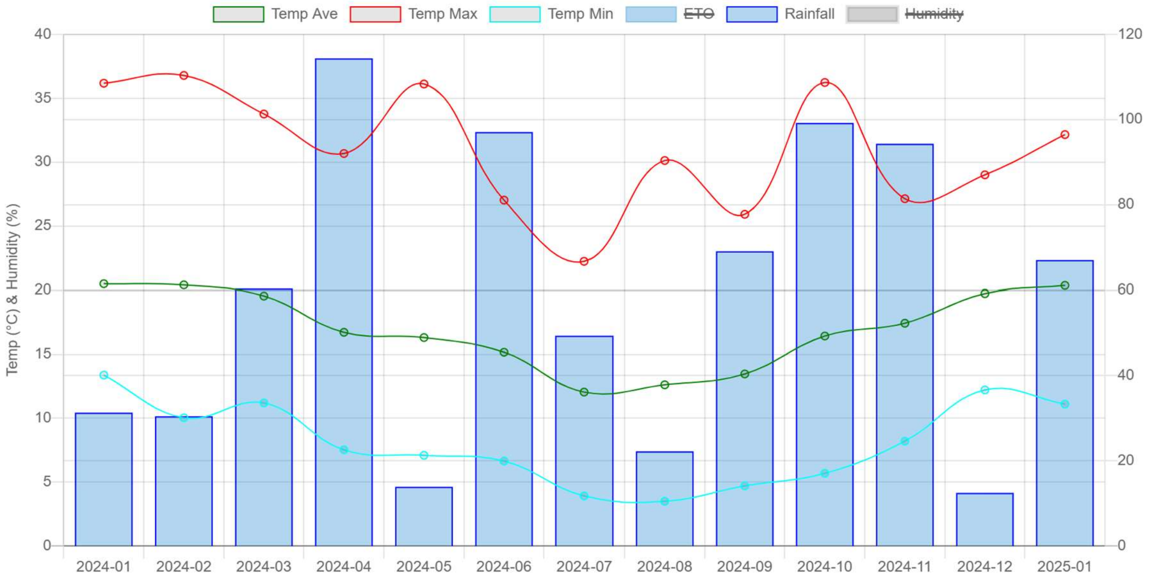


Figure 7-31: Gwaing WWTW Weather conditions for 2024 as obtained from gis.elsenburg.com/apps/wsp/

7.6.7.1 Requirement for Translucent Roof Sheeting

Solar drying can be done with or without roof coverings. Figure 7-32 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 7-33 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 7-32: Aerial view of the solar drying facility at Rooiwal WWTW north of Pretoria.

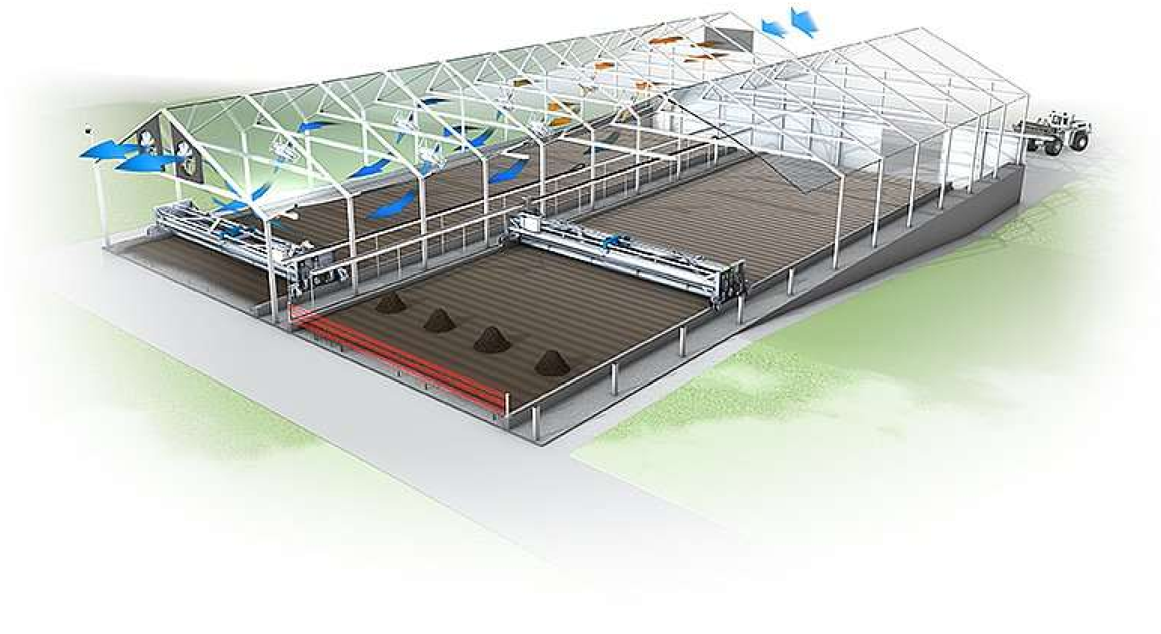


Figure 7-33: Example of advanced solar drying facility including translucent roof sheeting, forced ventilation and a sludge turner and spreader (Huber).

7.6.7.2 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 7-33 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 7-34: Electric mole as part of the SolarBatch system by Thermo-Systems.

Figure 7-35 shows a comparison table of continuous vs batch solar drying plants.

Figure 7-35: 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
Operation Costs	Low	Low

Aspect	Continuous Solar Sludge Drying	Batch Solar Sludge Drying (Electric Mole)
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

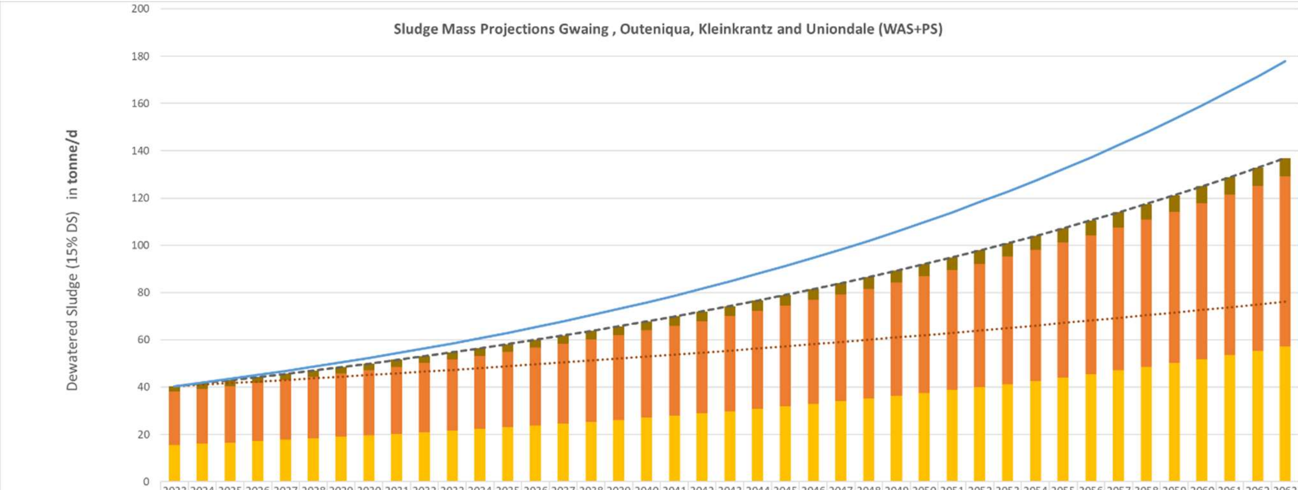
4. **Health and Safety Risks**: Working with rotovators and handling sludge manually can pose health and safety risks to workers, including exposure to pathogens and physical injuries from operating machinery. The temperature in the greenhouse structures can reach 55°C, meaning that plant operators would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

7.6.7.3 Solar Drying Sludge Volumes

The sludge volumes to be received at the BBF solar drying facility are shown in Figure 7-36**. Note that the sludge volume is less than that shown in Section 7.5**. Some conservative factors in the process design were removed so that the sludge quantities expected at the BBF are not over-estimated.

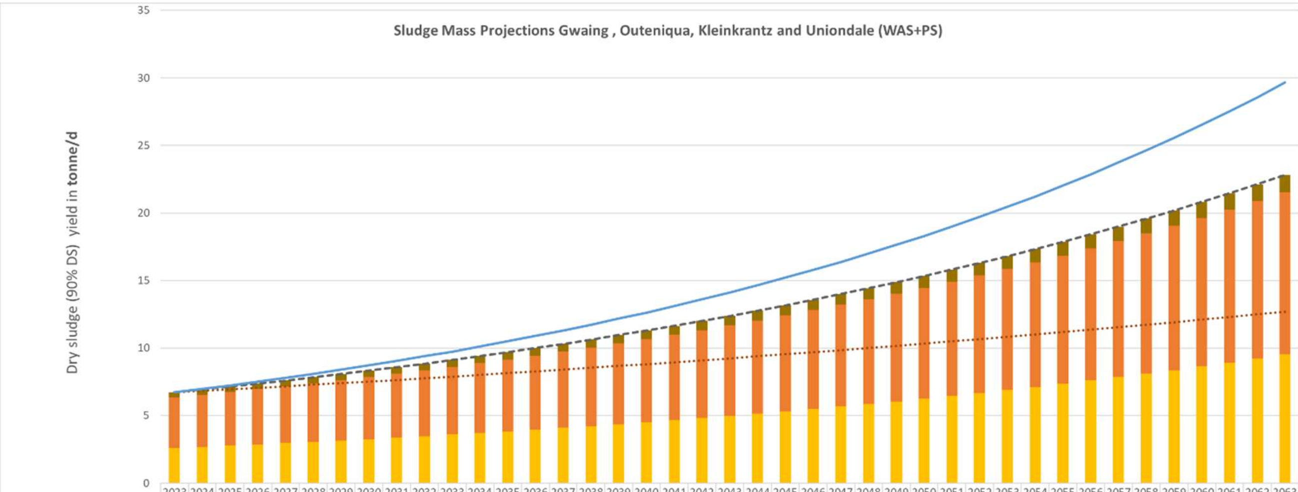


	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039	2040	2041	2042	2043
Uniondale & kleinkranz	2	2	2	3	3	3	3	3	3	3	3	3	3	3	4	4	4	4	4	4	4
Outeniqua WWTW	23	23	24	25	25	26	27	28	28	29	30	31	32	33	34	35	36	37	38	39	40
Gwaing WWTW	16	16	17	17	18	18	19	20	20	21	22	22	23	24	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 7-36 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 7-37. 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 7-37: 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.

7.6.8 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:

xi. Guard House
xii. Perimeter fencing and access gate
xiii. 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.
xiv. Approximately 13 000 m^2 in plan view of translucent roof sheeting ('greenhouse') structures.
xv. One 18m x 36m shed with a clear height of 4.5m and without any columns inside the building for the sludge granulation plant.
xvi. 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.
xvii. Movable precast concrete walls placed on slabs to demarcate separated process areas and to prevent contamination of treated sludge by raw sludge.
xviii. Access Roads
xix. Rainwater collection and storage from all roof structures
xx. Stormwater collection and drainage from concrete slabs with pipeline to Gwaing WWTW inlet works.

The schematic layout in Figure 7-38 shows some of the key infrastructure components and the basic process flow.

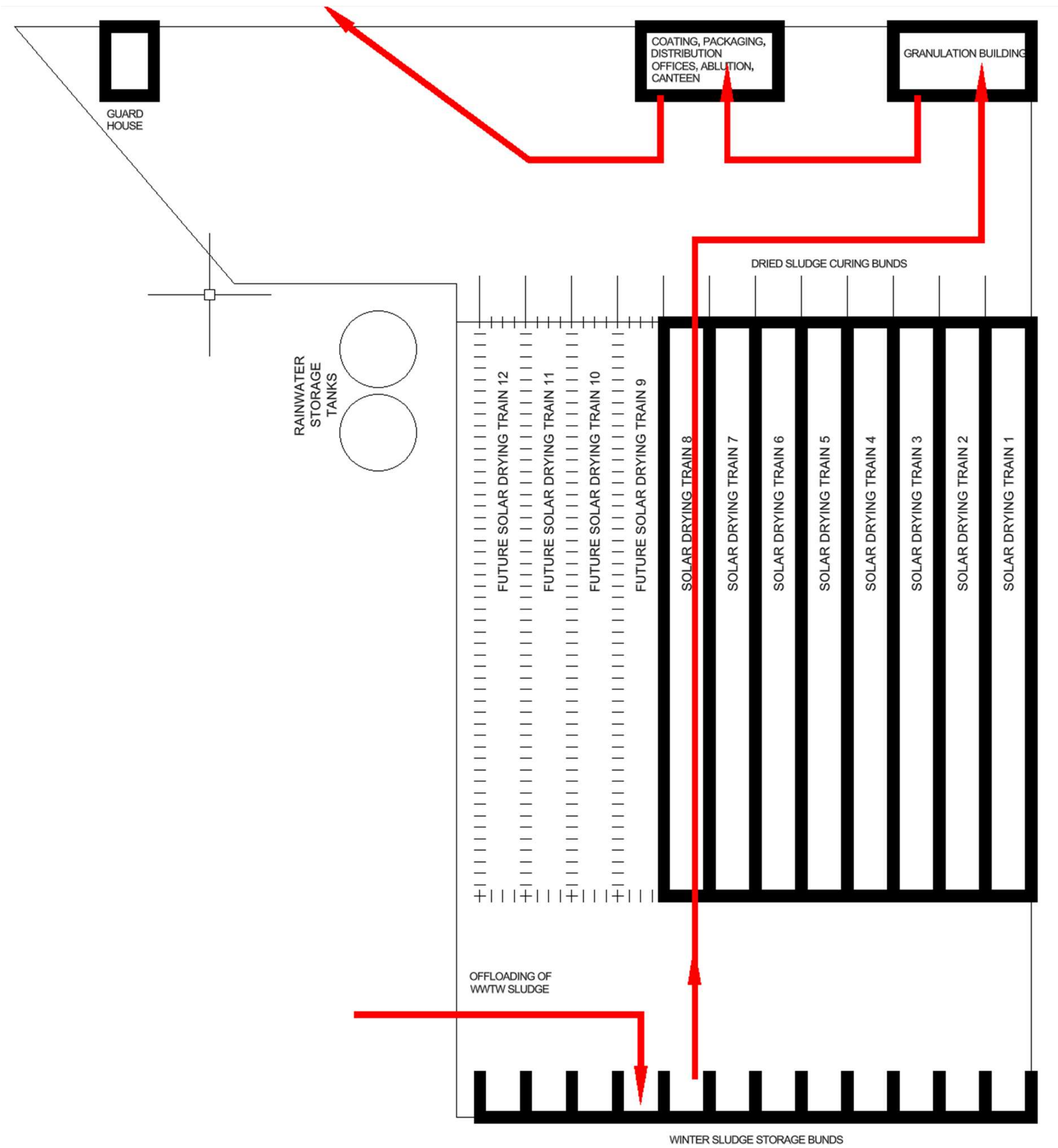


Figure 7-38: Gwaing BBF schematic layout with basic process flow.

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.

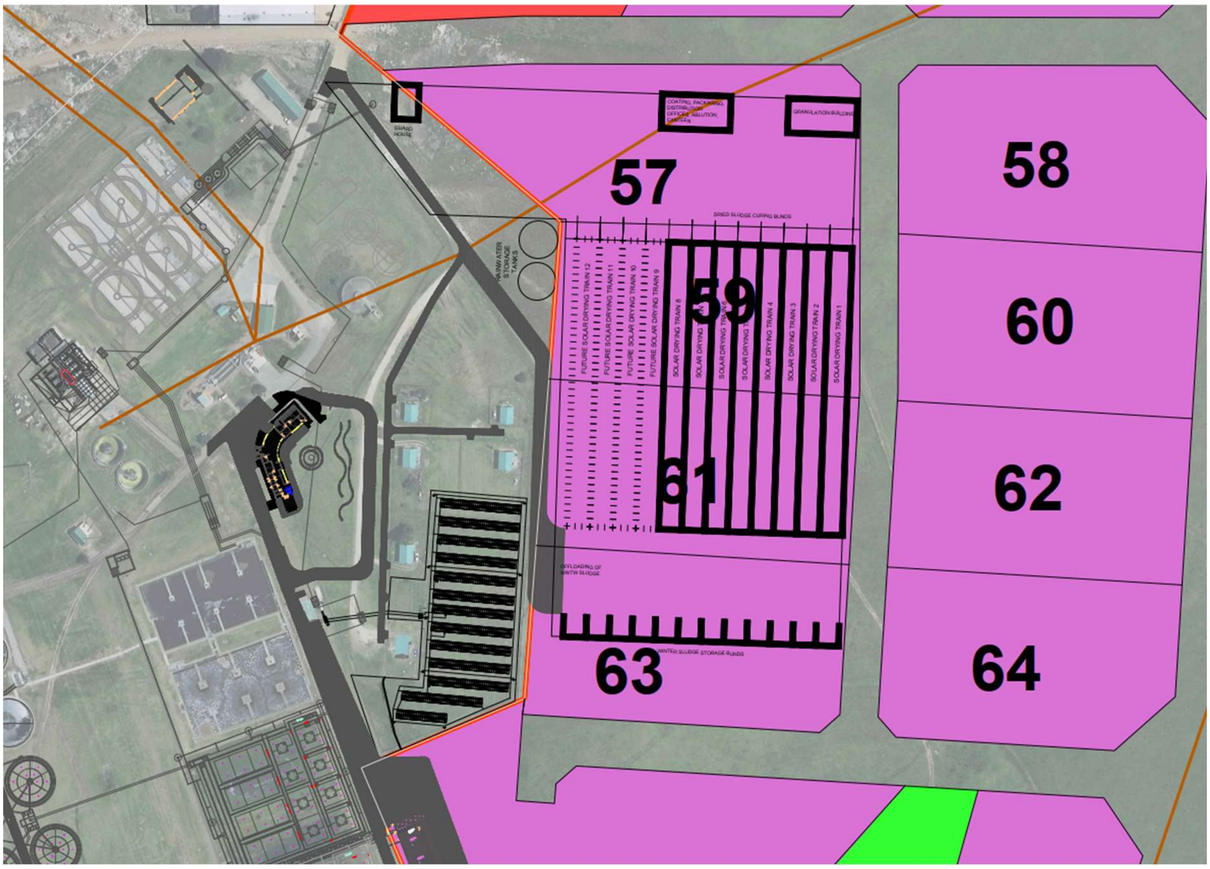


Figure 7-39: Proposed position for the BBF site.

7.6.9 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 7-40 shows a schematic layout of the stormwater management plan for the Gwaing BBF.

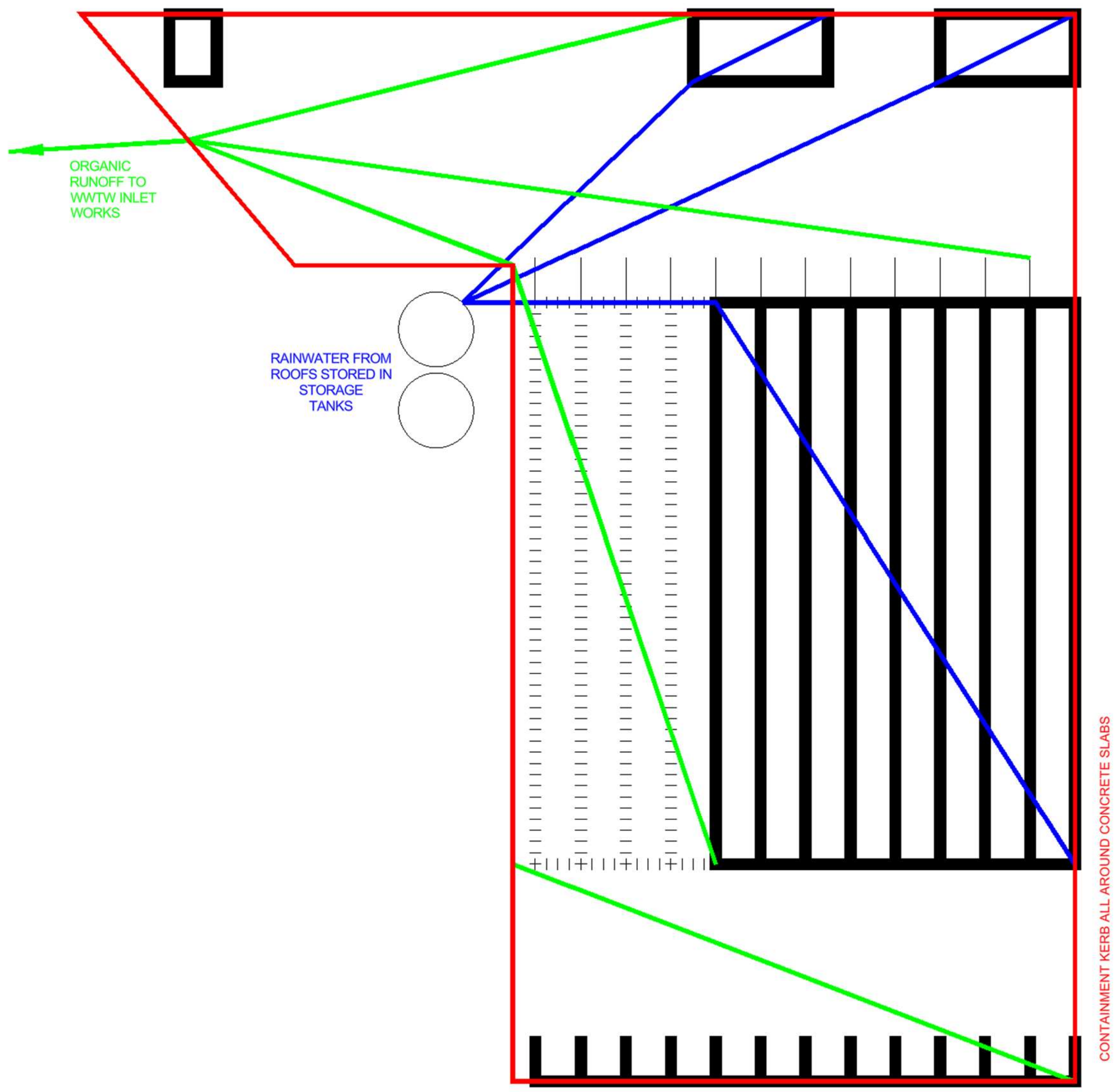


Figure 7-40: Schematic Stormwater Management Plan for Gwaing BBF

7.7 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 7-41.

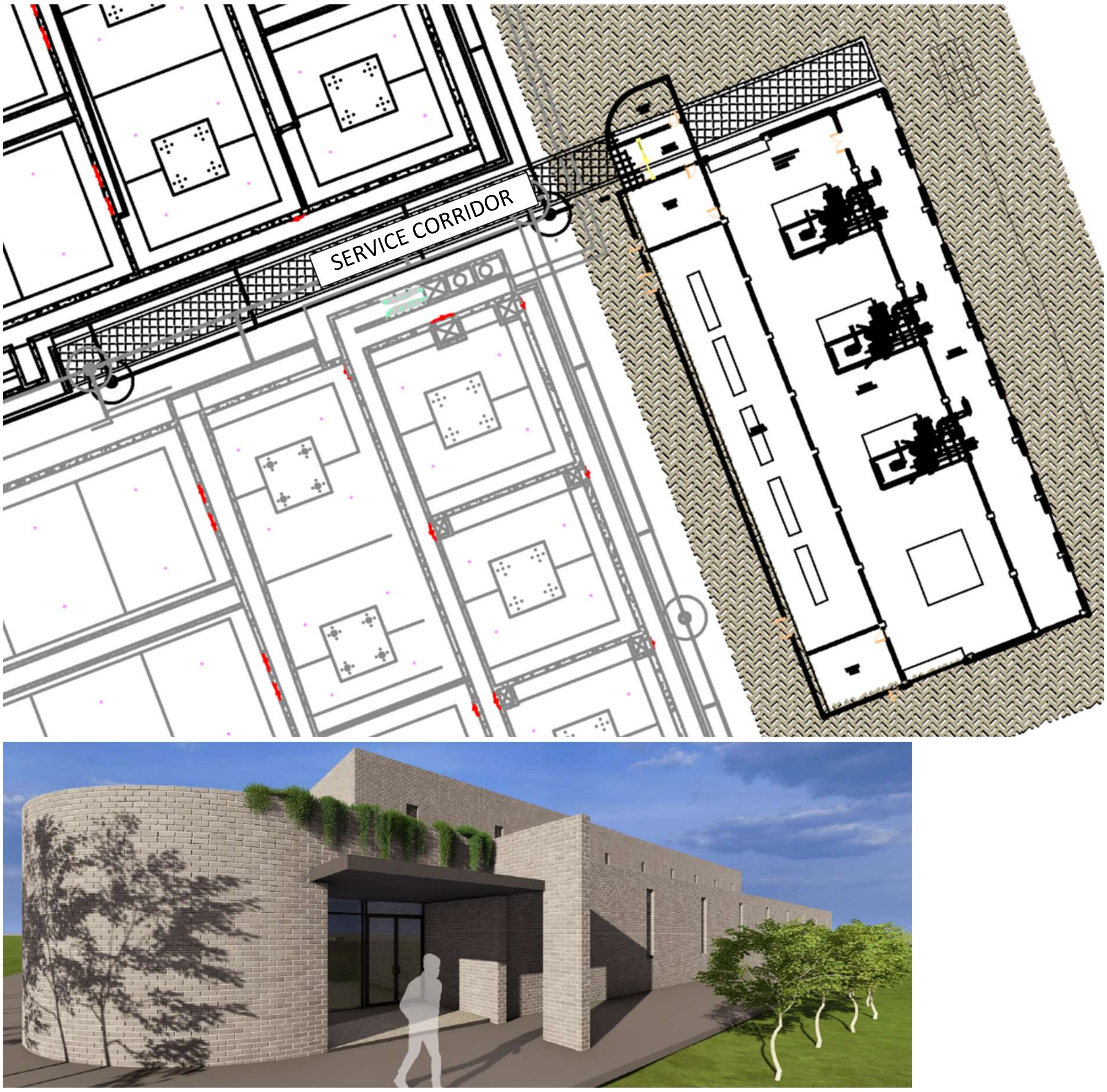


Figure 7-41: Blower house floor plan and associated 3D rendering

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.

7.7.1 Mechanical Equipment for Aeration

A summary of the mechanical equipment for the reactor aeration system is provided in Table 7-19. 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 7-19: 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 7-42.

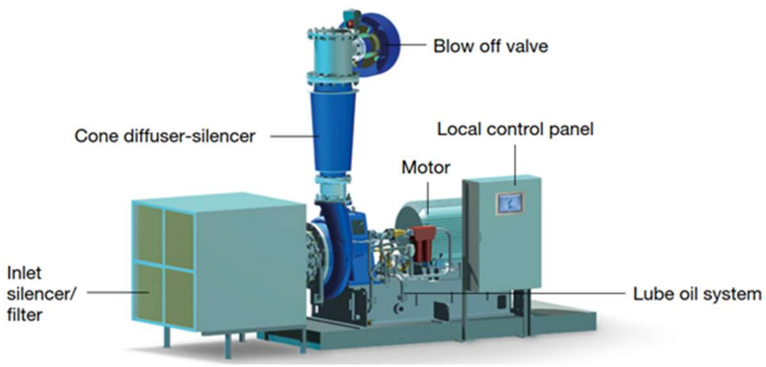


Figure 7-42: Isometric view of a typical centrifugal blower to be housed in the blower house (acoustic hood not shown)

The proposed aeration control system is shown in Figure 7-43. 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.

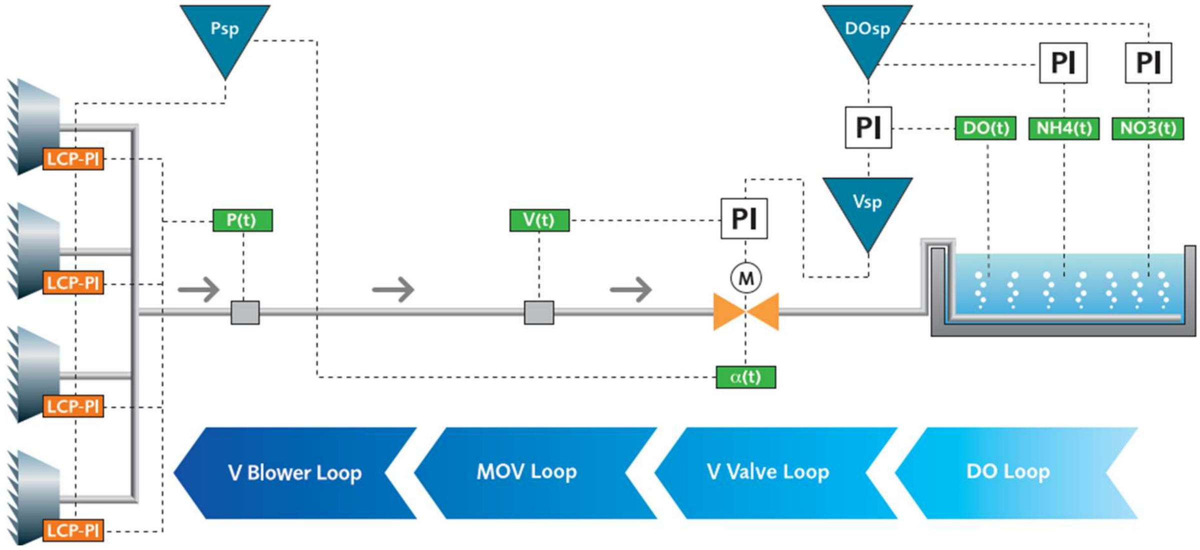


Figure 7-43: Aeration control response system

7.8 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 7-44.

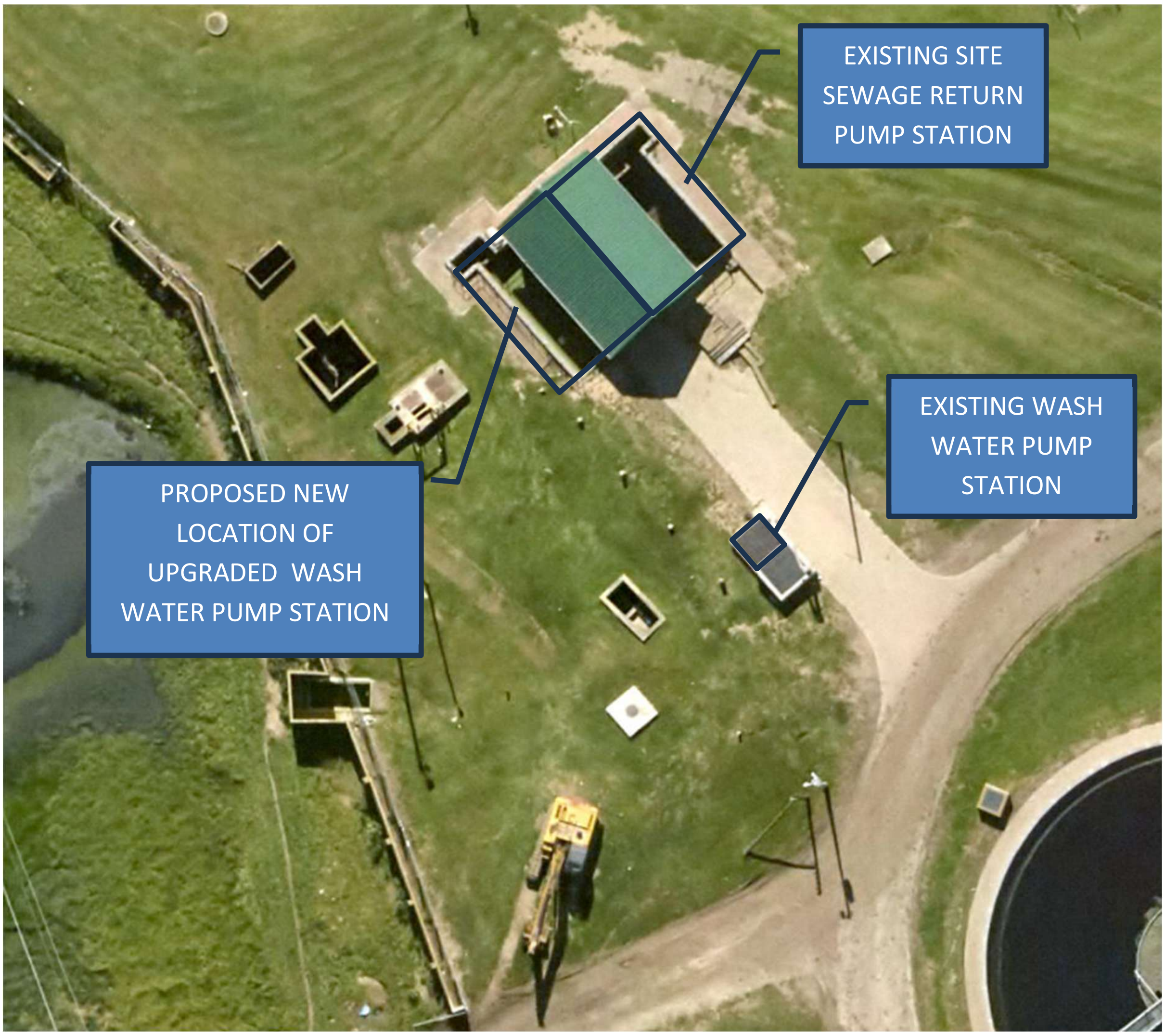


Figure 7-44: Location of New Wash Water Pump Station

7.9 Admin Building

A new administration building is proposed as part of the upgrade. The following design principles were 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 complex is situated in the centre of the site in such a manner that the operators have a view of the main process units. A conceptual layout of the admin building is shown in Figure 7-45 and a 3D render of the building in Figure 7-46.

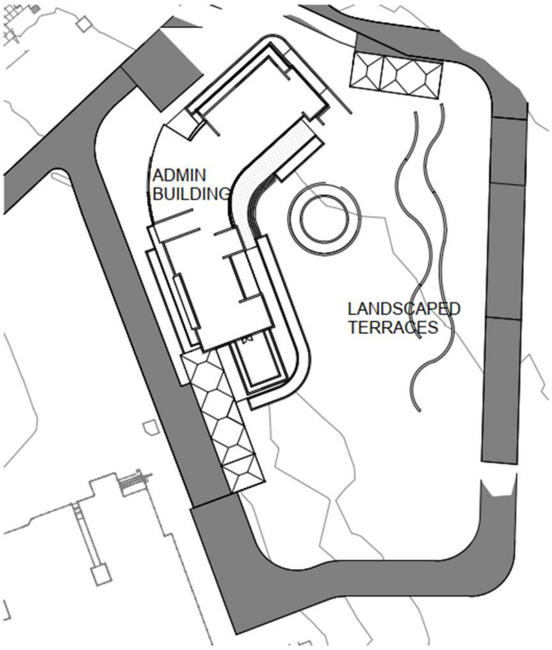


Figure 7-45: Conceptual Layout of the Admin Building

Figure 7-46: 3D rendering of the Admin Building

The accommodation schedule for the admin building is shown in Table 7-20.

Table 7-20: 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,	

ACCOMMODATION REQUIREMENTS	Rooms	Requirements	Est Area (m^2)
	Separate store area for 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	Reception and printing area	All stationery and printer	10-15
	Process controller (PC) office with view 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%

7.10 Interconnecting Pipework

All pipework is sized based on specific hydraulic requirements. External or internal pipes connecting to puddle pipes will be joined with flanges, flange adapters or ranger couplings suitable for the two pipe materials being joined. Sufficient corrosion protection will be specified for steel couplings or flange adapters that will be installed below ground or in corrosive environments. The pipe material and pressure ratings of the interconnecting pipework for the Gwaing WWTW upgrade are summarised in Table 7-21.

Table 7-21: Interconnecting pipework specifications

Description	DN (mm)	Material
Gravity Pipework - External		
Puddle pipes cast into concrete	< 300	SS316
	> 300	GRP
Process related gravity lines (underground)	< 300	uPVC Class 9
	> 300	GRP SN5000 PN10
Pressure Pipework - External		
Puddle Pipes cast into concrete	All	SS316
Wash water ring main	All	HDPE or uPVC
Process-related pressure lines (underground)	< 300	uPVC Class 9
	> 300	GRP SN5000 PN10
Internal pipework (inside structures)		
Internal pipework	All	SS316 or uPVC
Air pipework		
Air Header	All	SS316

7.11 Reuse Opportunities for Gwaing WWTW

As per the CSIR Green Book, George faces a significant risk of increased drought exposure. Currently, 1.9 years per decade are susceptible to drought, and this is projected to rise to 3.1 out of every 10 years by 2050. In response to the 2009/10 draught, George Municipality implemented the 10MLD indirect re-use plant as part of the emergency relief draught measures. The plant only ran for around 128 hours and was then decommissioned according the George Municipality. The Municipality is currently within the planning stages of upgrading and recommissioning of the plant. Given population growth and limited surface water sources, planning for direct, indirect, and industrial reuse is crucial. Notably, final effluent from George's WWTWs constitutes a substantial water source, with about two-thirds of the city's potable water consumption ending up there. Consequently, reuse is also part of the Gwaing WWTW site planning.

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 reuse 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.

The international best practice guidelines adopted for the City of Cape Town reuse projects are summarised in Table 7-22.

Table 7-22: 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 requirement before injection into a surface water reservoir			

The proposed area for future reuse infrastructure is shown in Figure 7-47. This will depend on which site is selected for sludge beneficiation.

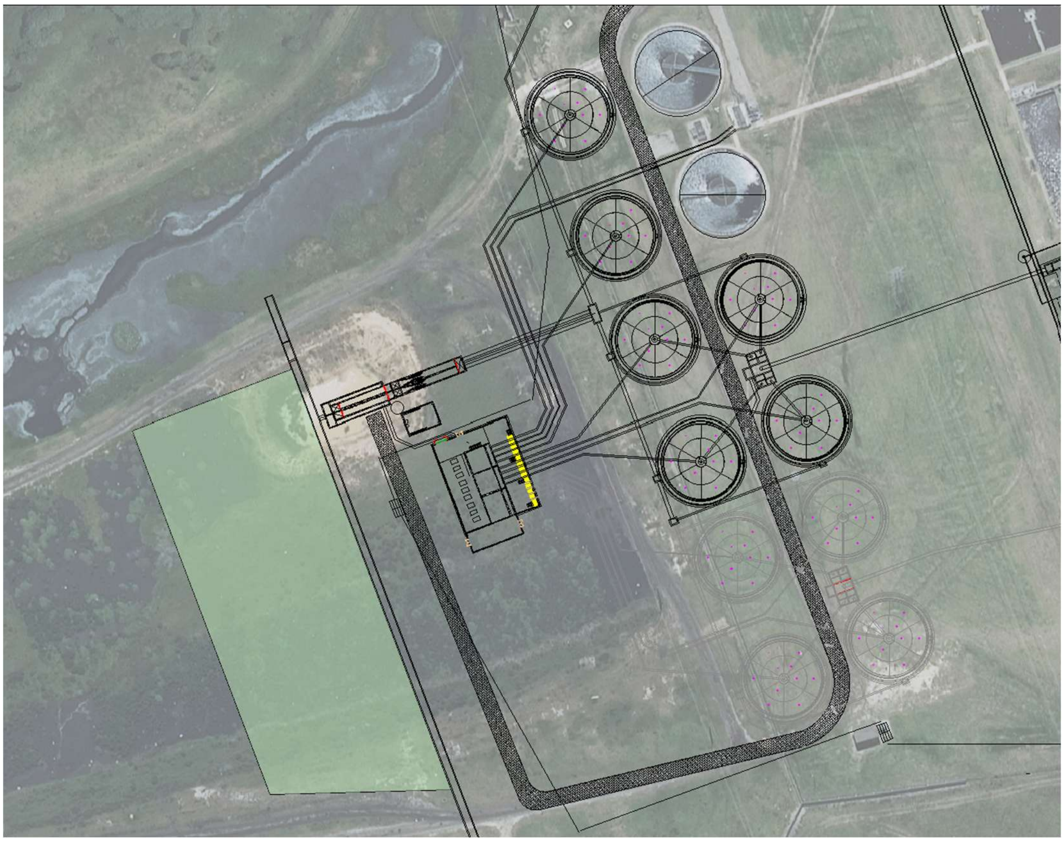


Figure 7-47: Proposed Area for Future Reuse Infrastructure

8 ELECTRICAL

Following the completion of the Gwaing WWTW master plan, stage two (concept) commenced and the designs were developed for phases A and B combined.

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.

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

The Architects provided concept drawings for buildings which were used to establish the small power and lighting requirements. There was also an equipment schedule/list provided to determine the sizing of the motor control centres for each required building.

8.1 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.

8.2 Existing Services

There is an existing 11 kV electricity network for the facility which (from GM electricity accounts provided) does 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.

8.3 Electrical Concept Design

The next subsections discuss the electrical scope of phases A and B.

8.3.1 Medium Voltage Network

The proposal is for a new four (4) MVA with full backup (standby generators) substation to be constructed near the entrance of the facility behind the existing administration building (to be converted to stores).

The existing 11kV overhead electricity supply/line will be re-aligned to be taken underground and to the new proposed substation with new medium voltage switchgear and metering equipment.

Currently, the requirement is for three (3) MVA electricity supply with three (3) one (1) MVA 400-volt standby generators to be installed to cope with the existing and additional electricity load to the facility.

The proposal is for a new medium voltage line for the full load capacity of four (4) MVA to be installed to form a ring medium voltage network and existing miniature substation to be upgraded and an additional miniature substation to be installed.

The proposed electrical MV ring main is shown in red in Figure 8-1.

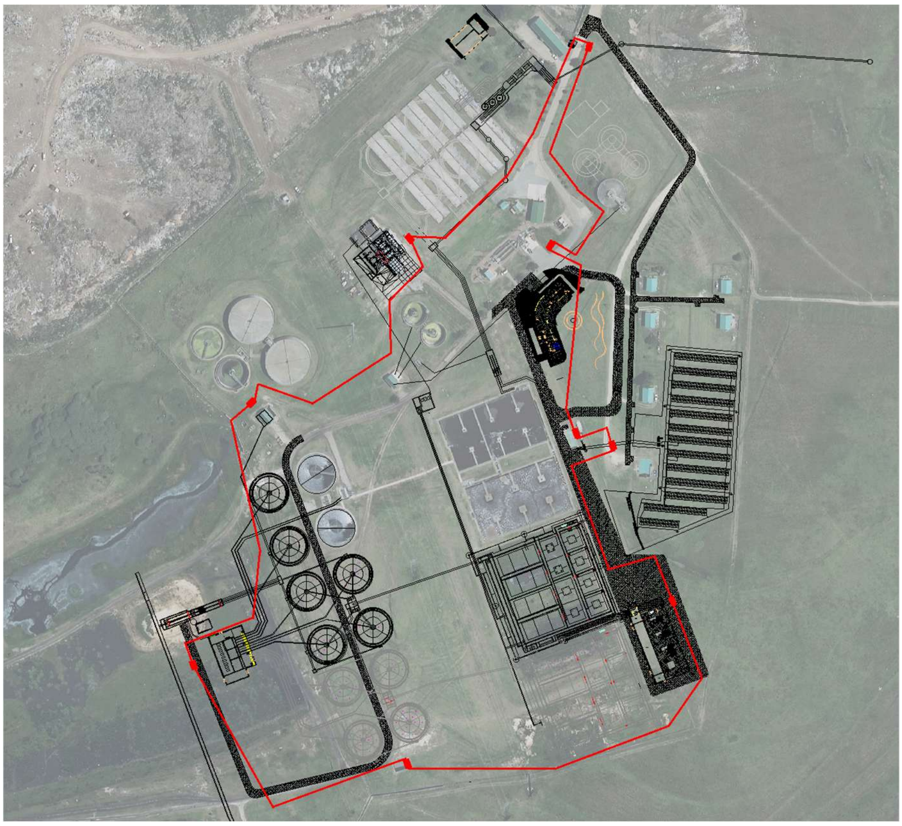


Figure 8-1: Electrical MV Ring Main Layout

8.3.2 Motor Control Centres

Each new building will be equipped with a motor control centre with various controlling equipment to indoor and field mechanical equipment. From the motor control centres, there are electricity supply cables allowed to all field mechanical equipment. The SCADA system will be integrated with all instruments and mechanical equipment for the upgrade.

8.3.3 Small Power and Lighting

Each new building will be equipped with local power outlets (in accordance with SANS 10142) and area purpose/specific lighting (in accordance with SANS 10114).

There is also a general allowance made for street lighting, which will aid security as well as additional security lighting on buildings.

8.3.4 Fire Detection

The fire detection design for each building and collectively for the entire premises will be designed in accordance with SANS 10139. Each building will have its own fire panel and a master panel in the guard house at the entrance. The guardhouse is the main point of entry and therefore the master fire panel is installed in the guardhouse for the fire department to identify the building that has the fire detection activated.

8.3.5 Closed-Circuit Television

The site was assessed, and additional closed-circuit cameras will be installed allowing security to provide better surveillance for the premises.

8.3.6 Access Control

Each outside door to the buildings on site and the new proposed buildings on site will have doors that are accessed controlled.

8.3.7 Fibre and Data

A new sleeve and manhole network is proposed allowing fibre cables to be run within the site and connect each building to the network aiming to reduce the quantity of wireless transmitted poles to data transfer across the site.

8.4 Summary of Electrical Requirements

There is an allowance made for a new substation building which will house the switchgear for the required three (3) MVA demand to accommodate the existing and the additional power requirements for phases A and B. Phase A and B would require standby generator capacity to accommodate the three (3) MVA electricity demand.

As part of phases A and B, the medium voltage cables will be installed throughout the site to allow for a ring 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.

A network capacity study was received from GLS Consulting (Pty) Ltd which indicated that there is sufficient medium voltage capacity for the upgrade of the facility. There was a request for a second electricity supply line to the facility. Refer to Appendix B: GLS Electrical Capacity Investigation Study.

The phases for the processing work and the electrical works are out of sync due to the infrastructure and the complexity of the medium voltage network. The current concept design provides maximum flexibility for alteration and/or additions in the future.

9 SCADA AND TELEMETRY

Gwaing WWTW existing Supervisory Control And Data Acquisition (SCADA) and telemetry systems were installed and are currently being maintained by Spectrum Communications.

9.1 Existing Installation

The site was visited, and the following information was obtained. The existing SCADA and telemetry installation is based in the control room. An Adroit SCADA is implemented. The Adroit license information obtained from the system is as follows:

- Adroit version - 1000
- HASP type – HASP-HL
- Licensed Clients – 1
- Licensed Scanpoints – 750
- Used Scanpoints – 376

The graphic mimics that are implemented are in the Classic format.

A server rack is installed in the control room housing two servers and a battery backup system. A computer accessing the SCADA is housed at one of the control room desks along with a wall-mounted display showing the SCADA information.

A Spectrum tele-Flex radio telemetry system is installed. The surrounding pump stations feeding into the works are displayed on the SCADA. Some of the pump stations' information was not updating on the SCADA. This could be due to faults at the pump stations or on the communication links.

9.2 Required Upgrades

It is recommended to upgrade the SCADA to the latest version of Adroit with SmartUI format and to upgrade to 1500 Scanpoints. This will ensure that sufficient scanpoints are available for Phase A and B and that all the latest functionalities of the Adroit SCADA are available for implementation. It is also recommended to upgrade the SCADA server to a rack-mounted server.

The SCADA and telemetry scope of work will comprise the following items:

a. SCADA liscence 1500 tag
b. SCADA gold support
c. SCADA client liscence (5user)
d. SCADA server
e. SCADA client PC
f. UPS
g. Server rack
h. Server switch
i. SCADA development existing plant and Phase A&B
j. commissioning

10 CHEMICAL AND ENERGY USAGE

For phases A and B, Gwaing WWTW will be using polymer for waste activated sludge dewatering. A summary of the estimated chemical usage at 22 MLD and the associated estimated costs are provided in Table 10-1.

Table 10-1: Chemical usage and cost summary for Gwaing WWTW phase A&B solution

Chemical	Dosing rate	Dose/annum (kg/a)	Rate (R/kg)	Annual Cost (2023 value)	Annual Cost (2025 Value)*
Polymer for waste activated sludge dewatering	5kg/ton TSS	12019	R44.77	R538 163.66	R604 680.69
Total					**R604 680.69**

*Actual values recorded in 2023 were increased at an inflation rate of 6% to calculate 2025 values.

Based on a detailed equipment list, and each equipment's associated electric motor and operational hours, the electricity cost for operating Gwaing WWTW's phase A and B solution was estimated. The costs were determined using an average unit price of R2.16/kWh (value of R2.04/kWh in 2024 escalated by 6% to 2025 value) for electricity and are summarised in Table 10-2.

Table 10-2: Electricity usage cost estimate for Gwaing WWTW Phase A and B

Description	Total Duty Power (kW)	kWh/d	Total -R/d (2025)
Inlet Works	145	374	R807.52
Low Lift Screw Pump Station	37	621	R1 341.96
Existing Reactor - Module A (Including Surface Aeration)	523	11641	R25 144.06
New Reactor - Module B	164	2854	R6 163.88
Blowers and Aeration	472	5945	R12 841.63
New WAS Pump Station	29	345	R745.80
SSTs associated with Existing Reactor - Module A	12	247	R533.23
SSTs associated with New Reactor - Module B	12	247	R533.23
RAS Pump Station - Module A&B	227	3606	R7 789.75
Existing and Additional WAS Dewatering Equipment	74	1073	R2 318.66
Effluent Return/ Wash Water Pump Station	24	345	R745.80
Disinfection: UV	113	2709	R5 850.66
Subtotal:	**1831**	**30008**	**R64 816.22**
Contingency Allowance (15%)	275	4501	R9 722.43
Total:			**R74 538.65**
Annual Electricity Cost - R/a (2025)			**R27 206 606.27**

The total annual chemical and electricity costs are estimated to be R27 811 286.97 (in 2025).

11 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 the reuse of grit as part of composting or fill material. [Phase B]
- Regional screenings processing facility to minimise volume, odours, pathogens and vector attraction of screenings. [Phase B]
- Sludge beneficiation in the form of composting or fertilizer production. [Phase B]
- The methane gas produced from anaerobic digestion will be used for generating heat and power. [Phase D]
- Effluent from the Gwaing WWTW can in the 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. [Future]
- 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. [Phase B]

12 LAND REQUIREMENTS

It is not foreseen that additional land will be required for the Gwaing WWTW until 50 MLD capacity is exceeded. As shown in Figure 7-39, additional land may be required for solar drying of sludge if the area at the maturation ponds is not suitable.

13 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 for both Phase A & B.

Phase A scored 67 points, which classifies the works as a Class B works. Phase B scored more than 73 points which classifies the works as a Class A works. The staffing requirements for both phases according to the act are presented in Table 13-1. Gwaing WWTW operates 24 hours a day and operates on a shift basis. Table 13-1 shows the minimum staffing requirements per shift required.

Table 13-1: Gwaing WWTW Staffing Requirements

Phase	Capacity	Works Class	Class and number of persons as process controllers	Class of person as supervisor	Class of Person for weekly inspection
Phase A	13.2 MLD	B	1 x Trainee 2 x I 1 x II 1 x III	IV	V
Phase B	22 MLD	A	1 x Trainee 2 x I 1 x II 1 x III 1 x IV	V	-

According to George Municipality, the current personnel on site is as summarised in Table 13-2.

Table 13-2: 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 B works requires 5 process controllers and a supervisor on site, with qualifications and skills as described in Table 13-1 of Section 13 of this report. 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 B-works must always have a Class IV supervisor on site and a Class V supervisor conducting weekly inspections. Depending on how Gwaing WWTW rotates its staff and their qualifications, it seems to have insufficient process controllers.

14 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 14-1: 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.

15 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 15.1 to 15.6, along with some sketches and images that will assist in the readers' understanding of the principles presented.

15.1 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 15-1 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.

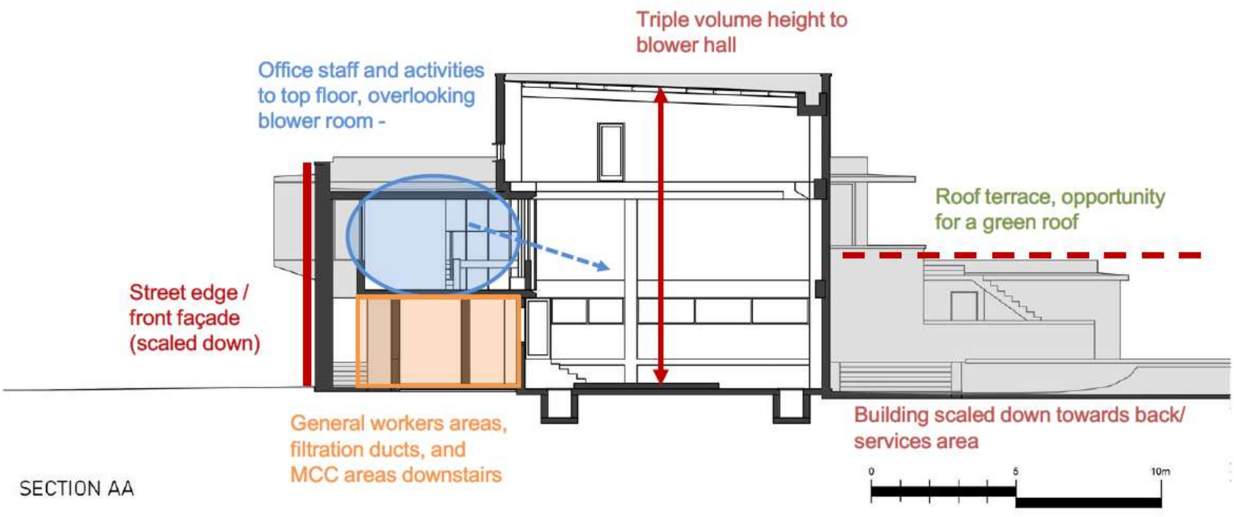


Figure 15-1: Section of Athlone WWTW's blower house

15.2 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 15-2, is found in the design of a Water Treatment Plant in France by AWP.

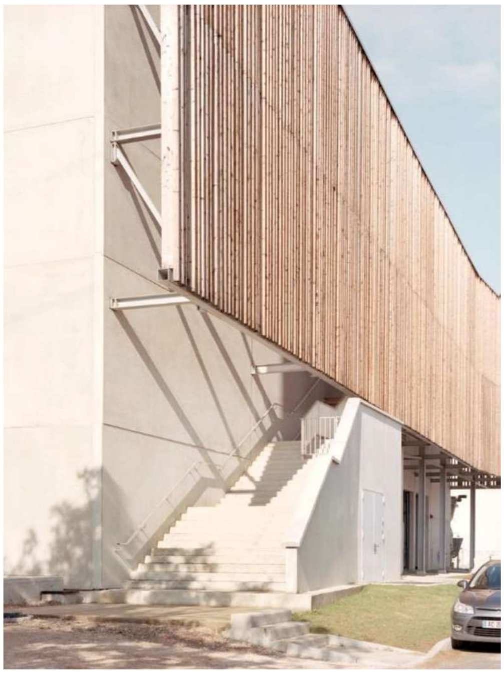
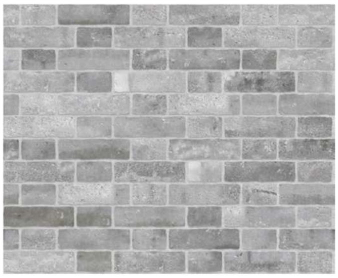


Figure 15-2: Picture of a Water Treatment Plant in France

15.3 Use of Materials

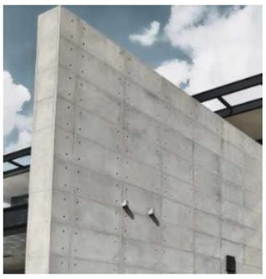
It is generally accepted as good practice, when designing for industrial applications, to make use of low-maintenance materials. We also feel that it is important to make use of materials that reflect the nature of the site and its surroundings, and are as natural and sustainable as possible. Materials 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 15-3.



Clay Brick

Natural Timber
(sustainable treated)

Planting



Exposed Concrete

Figure 15-3: Proposed Building Materials

15.4 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 15-4), daylight glare control, creating external views for the users, energy and water management, rainwater harvesting, and community involvement.

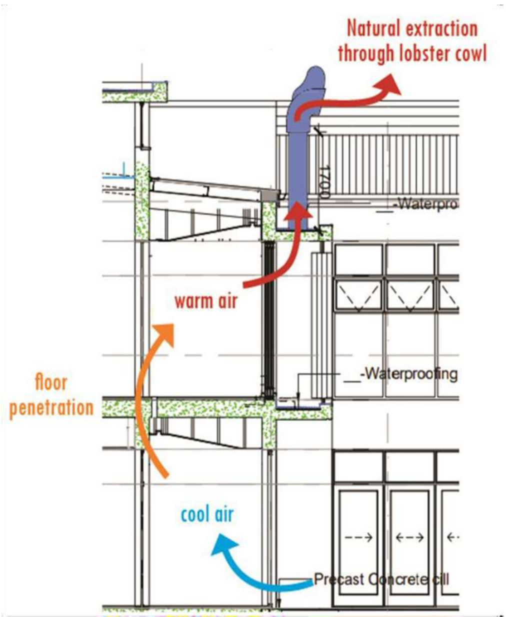


Figure 15-4: Natural ventilation flow

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 15-5 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 15-5: Picture of a natural Planted Courtyard

15.5 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 15-6 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.

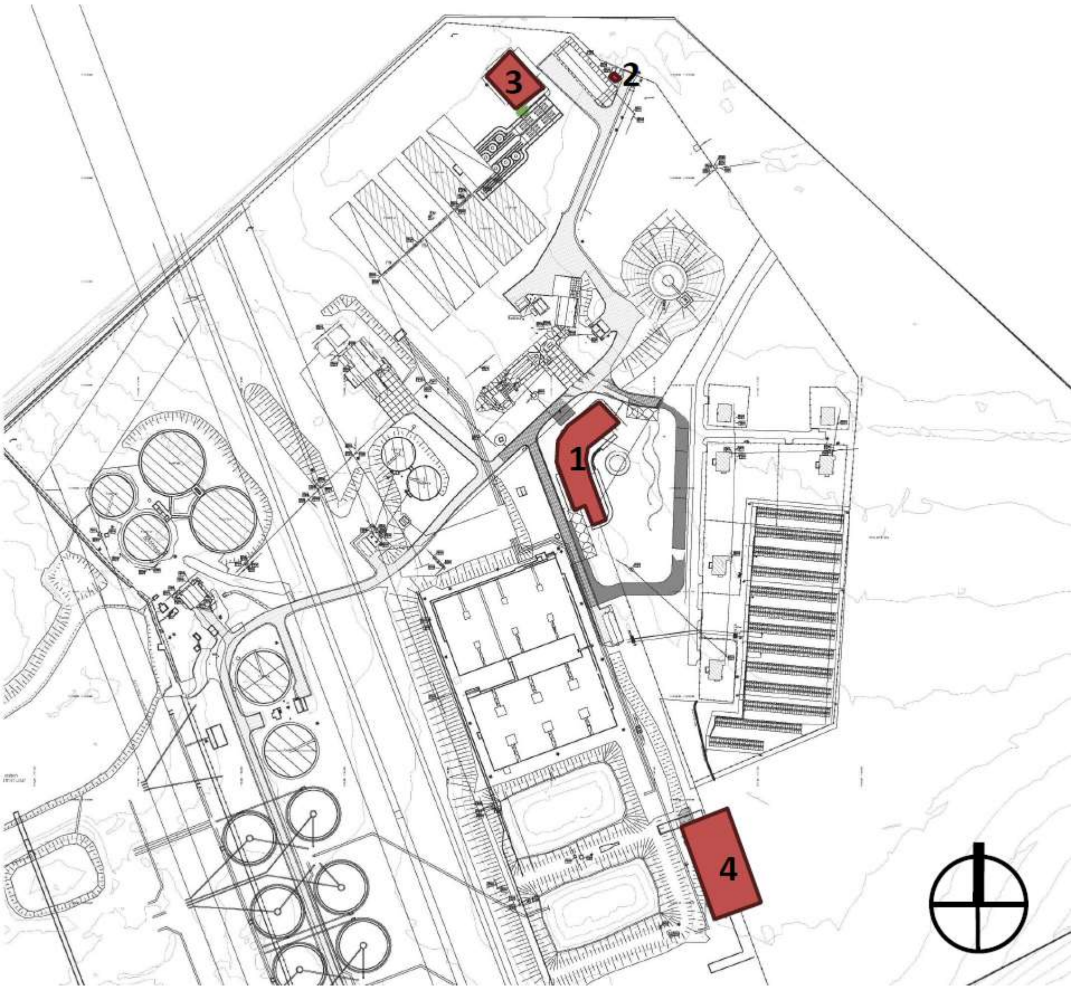


Figure 15-6: Positioning of Architectural Buildings on Site

As shown and numbered in Figure 15-6, 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 15-7 shows extracts from the George Municipality's standard Guard House design. Figure 15-8 shows the 3D rendering for the proposed new Guard House which forms part of the upgrades to the Gwaing WWTW complex.

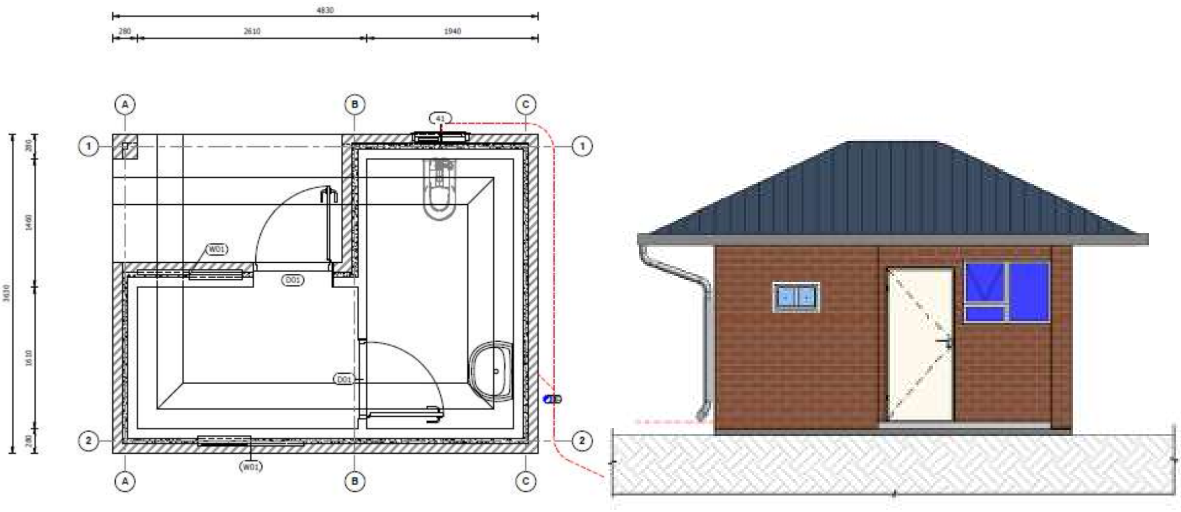


Figure 15-7: Extracts from George Municipality Standard Guard House Drawings

Figure 15-8: 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 15-6)

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 15-6)

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 15-9 shows extracts from the concept drawings for the BBF which need to be adjusted to the specific site allocated for the facility.

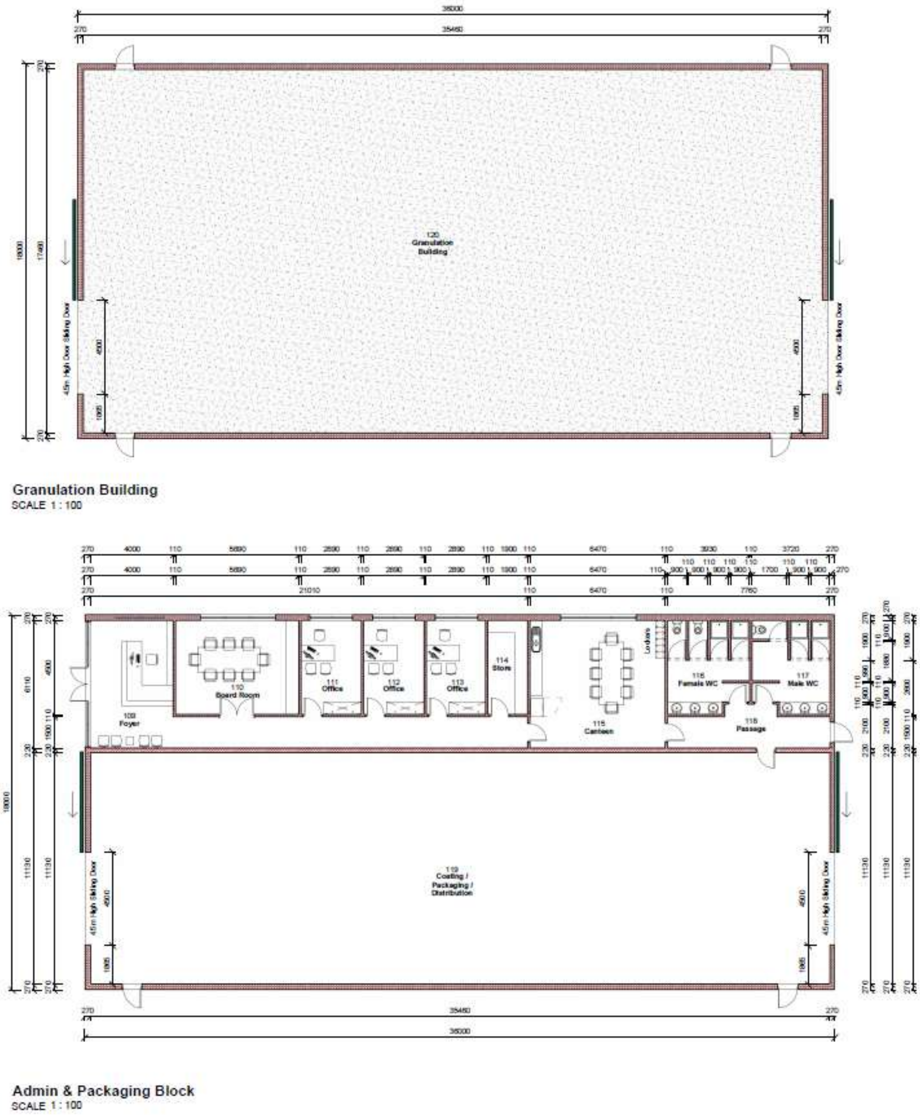


Figure 15-9: Proposed Floor plans For Granulation Building and Admin Block

15.6 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 15-10) underscores the importance of an approach that is seamlessly integrated into these existing contexts.

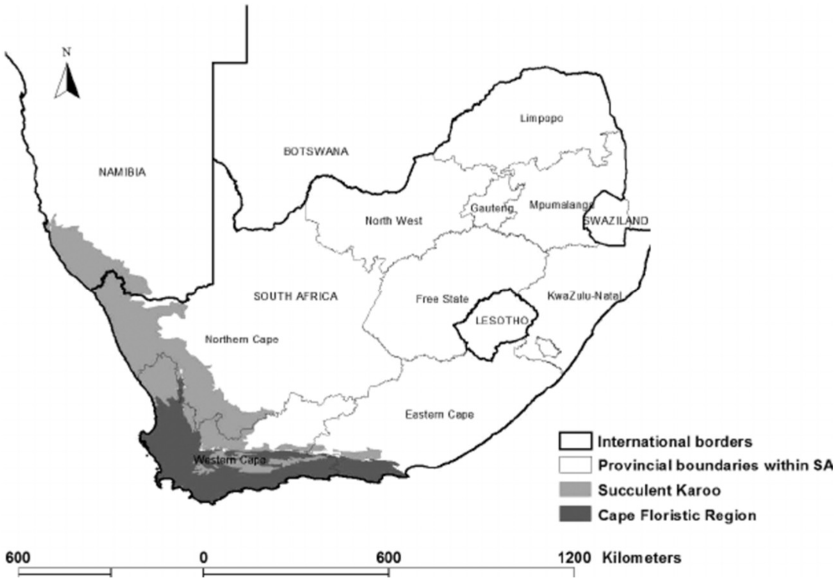


Figure 15-10: Map of the Cape Floristic Region

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 15-11 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 15-11: Landscaping Concepts

16 SITE SECURITY

A new guardhouse is recommended adjacent to the existing administrative building at the main entrance. This guardhouse will serve as an access control point for the site and will include facilities such as a toilet and a small kitchenette for the guards. The majority of the new infrastructure will be constructed within the existing fence of the Gwaing WWTW, with the exception of specific components like the SSTs, the RAS pump station, and the chlorine contact tank. To enhance security, the existing fence will be extended to encompass these structures.

The proposed layout and rendering of the guardhouse are shown in Figure 16-1.

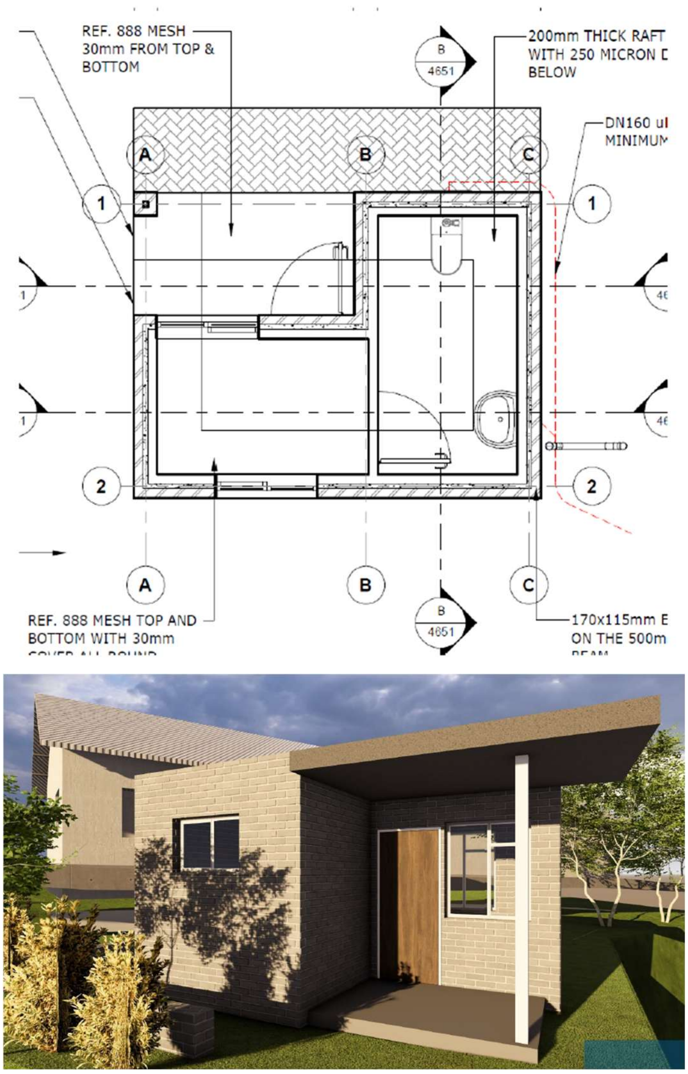


Figure 16-1: Proposed layout and rendering of the Guardhouse

17 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 17-1. 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 17-1: Layout of proposed new roads

18 DEMOLITION WORK

Figure 18-1 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 18-1: Structures to be Demolished as part of Gwaing WWTW upgrades

19 FINANCIAL

19.1 Capital Cost Estimate

A concept-level cost estimate of Phases A and B 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 19-1 and the M&E capital costs of each unit process are shown in Table 19-2 below.

Table 19-1: Civil capital cost estimate

	Civil Phasing Cost Estimates - 2025 Rates			TOTAL COST Phase A & B & BBF (22 MLD)		BBF Construction		13.2 MLD UCT Phase A		22 MLD UCT Phase B
1	Site Works		R	20 548 241.11	R	-	R	6 164 472	R	14 383 769
2	Raw Sewer Pipelines (Re-Routing)		R	7 412 052.12	R	-	R	-	R	7 412 052
3	Inlet Works		R	12 610 753.69	R	-	R	-	R	12 610 754
4	Reactor - Module B		R	79 660 681.19	R	-	R	-	R	79 660 681
5	SST's - Modula A (2off.)		R	14 475 338.80	R	-	R	14 475 339	R	-
6	SST's - Modula B (4off.)		R	28 950 677.60	R	-	R	28 950 678	R	-
7	UV Channels		R	5 173 307.79	R	-	R	-	R	5 173 308
8	UV Electrical Building		R	1 206 716.74	R	-	R	-	R	1 206 717
9	RAS Pumpstation		R	10 861 574.41	R	-	R	10 861 574	R	-
10	WAS Pumpstation		R	653 654.30	R	-	R	-	R	653 654
11	Wash Water Pumpstation (Refurbish existing)		R	839 508.98	R	-	R	839 509	R	-
12	Extension of WAS Dewatering Building		R	6 745 437.35	R	-	R	-	R	6 745 437
13	Guard House		R	345 476.94	R	-	R	-	R	345 477
14	Blower House & Electrical Building		R	34 658 247.09	R	-	R	-	R	34 658 247
15	Admin Building		R	31 235 181.12	R	-	R	-	R	31 235 181
16	Interconnecting Pipework		R	14 752 587.10	R	-	R	8 261 449	R	6 491 138
17	Channels & Chambers - SST's & Reactors		R	9 681 690.28	R	-	R	2 904 507	R	6 777 183
18	Demolition Work		R	12 980 486.98	R	-	R	4 635 888	R	8 344 599
19	Biosolids Beneficiation Facility		R	124 515 792.02	R	124 515 792	R	-	R	-
	Sub Total		**R**	**417 307 405.61**	**R**	**124 515 792**	**R**	**77 093 416**	**R**	**215 698 197**
	PRELIMINARY AND GENERAL	20%	R	83 461 481.12	R	24 903 158	R	15 418 683	R	43 139 639
	CONTINGENCIES	10%	R	50 076 888.67	R	14 941 895	R	9 251 210	R	25 883 784
	TOTAL Excl VAT:		**R**	**550 845 775.40**	**R**	**164 360 845**	**R**	**101 763 309**	**R**	**284 721 621**

Table 19-2: M&E capital cost estimate

	M&E Equipment Phasing Cost Estimates - 2025 Rates			TOTAL COST Phase A & B & BBF (22 MLD)		BBF Construction		13.2 MLD UCT Phase A		22 MLD UCT Phase B
	Description									
1	Inlet Works		R	34 107 928	R	-	R	-	R	34 107 928
2	New Reactor - Module B		R	11 190 344	R	-	R	-	R	11 190 344
3	Blowers and Aeration		R	56 701 620	R	-	R	-	R	56 701 620
4	New WAS Pump Station		R	2 052 460	R	-	R	-	R	2 052 460
5	SSTs - Module A		R	5 501 816	R	-	R	5 501 816	R	-
6	SSTs - Module B		R	11 003 632	R	-	R	-	R	11 003 632
7	Washwater Pumpstation		R	2 363 132	R	-	R	2 363 132	R	-
8	RAS Pump Station		R	6 354 700	R	-	R	6 354 700	R	-
9	WAS Dewatering Equipment		R	29 269 844	R	-	R	-	R	29 269 844
10	Disinfection		R	42 910 560	R	-	R	-	R	42 910 560
11	Biosolids Beneficiation Facility		R	47 000 000	R	47 000 000	R	-	R	-
	Subtotal:		**R**	**248 456 036**	**R**	**47 000 000**	**R**	**14 219 648**	**R**	**187 236 388**
	Electrical Equipment		R	115 923 875	R	10 000 000	R	37 073 356	R	68 850 519
	Preliminary and Gen	20%	R	72 875 982	R	11 400 000	R	10 258 601	R	51 217 381
	Contingencies	10%	R	43 725 589	R	6 840 000	R	6 155 161	R	30 730 429
	TOTAL Excl VAT:		**R**	**480 981 483**	**R**	**75 240 000**	**R**	**67 706 766**	**R**	**338 034 717**

The combined civil and M&E cost estimate for each phase is summarised in Table 19-3.

Table 19-3: Combined Civil and M&E cost estimate

Combined Cost Estimate - 2025 Rates	TOTAL COST Phase A & B & BBF (22MLD)		BBF Construction		Phase A 13.2 MLD UCT		Phase B 22 MLD UCT	
Civil Cost Estimate	R	550 845 775.40	R	164 360 845	R	101 763 309	R	284 721 621
M&E Cost Estimate	R	480 981 482.96	R	75 240 000	R	67 706 766	R	338 034 717
TOTAL Excl VAT:	**R**	**1 031 827 258**	**R**	**239 600 845**	**R**	**169 470 075**	**R**	**622 756 338**

While the main components that separate Phases A and B are clear, there are several peripheral items that could be included in either Phase A or B. Therefore, the cost split between Phases A and B should not be considered fixed. The availability and timing of funding may determine in which phase these items can be incorporated.

Table 19-4 shows the indirect costs associated with the project for Phase A & B and Table 19-5 shows the indirect costs associated with the BBF construction.

Table 19-4: Indirect Costs associated with Phases A and B

Item	Description	Cost Estimate	
A2	**INDIRECT COSTS**		
A2.1	Percentage Fee (Stages 2-6)	R	54 060 000.00
A2.2	Construction Monitoring (36 months)	R	2 880 000.00
A2.3	Subconsultants		
A2.3.1	Survey	R	362 000.00
A2.3.2	Geotechnical	R	160 000.00
A2.3.3	EAP	R	624 000.00
A2.3.4	OH&S (GM appoints)	R	200 000.00
A2.3.5	Architect	R	3 800 000.00
A2.4	Recoverable Expenses	R	150 000.00
	Total Indirect Costs (Excl. VAT)	**R**	**62 236 000.00**

Table 19-5: Indirect Costs associated with the BBF Construction

Item	Description	Cost Estimate	
B2	**INDIRECT COSTS**		
B2.1	Percentage Fee (Stages 2-6)	R	19 070 000.00
B2.2	Construction Monitoring (36 months)	R	960 000.00
B2.3	Subconsultants	R	-
B2.3.1	Survey	R	70 000.00
B2.3.2	Geotechnical	R	170 000.00
B2.3.3	EAP	R	-
B2.3.4	OH&S (GM appoints)	R	120 000.00
B2.3.5	Architect	R	1 050 000.00
B2.4	Recoverable Expenses	R	100 000.00
	Total Indirect Costs (Excl. VAT)	**R**	**21 540 000.00**

19.2 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 19-6 represents the annual chemical costs per phase, Table 19-7 represents the annual electrical costs per phase and Table 19-8 represents the annual maintenance costs for the four phases of Gwaing WWTW upgrades.

Table 19-6: Estimated Annual Chemical Costs (Polymer)

Parameter	Phase A (2026)	Phase B (2028)
Capacity (MLD)	13.2	22
Cost/day (R/d)	R865.10	R1 656.66
R/a - 2025	R315 761.06	R604 680.69
R/annum in future value *	R334 706.73	R720 184.38
* in the year shown in the column heading		

Table 19-7: Estimated Annual Electrical Costs

Parameter	BBF (2028)	Phase A (2026)	Phase B (2028)
Capacity (MLD)		13.2	22
Cost/day (R/d)	R20 865.60	R38 966.95	R74 538.65
R/a – 2025	R7 615 944.00	R14 222 937.73	R27 206 606.27
R/annum in future value *	R9 070 711.16	R15 076 314.00	R33 862 106.13
	* in the year shown in the column heading		

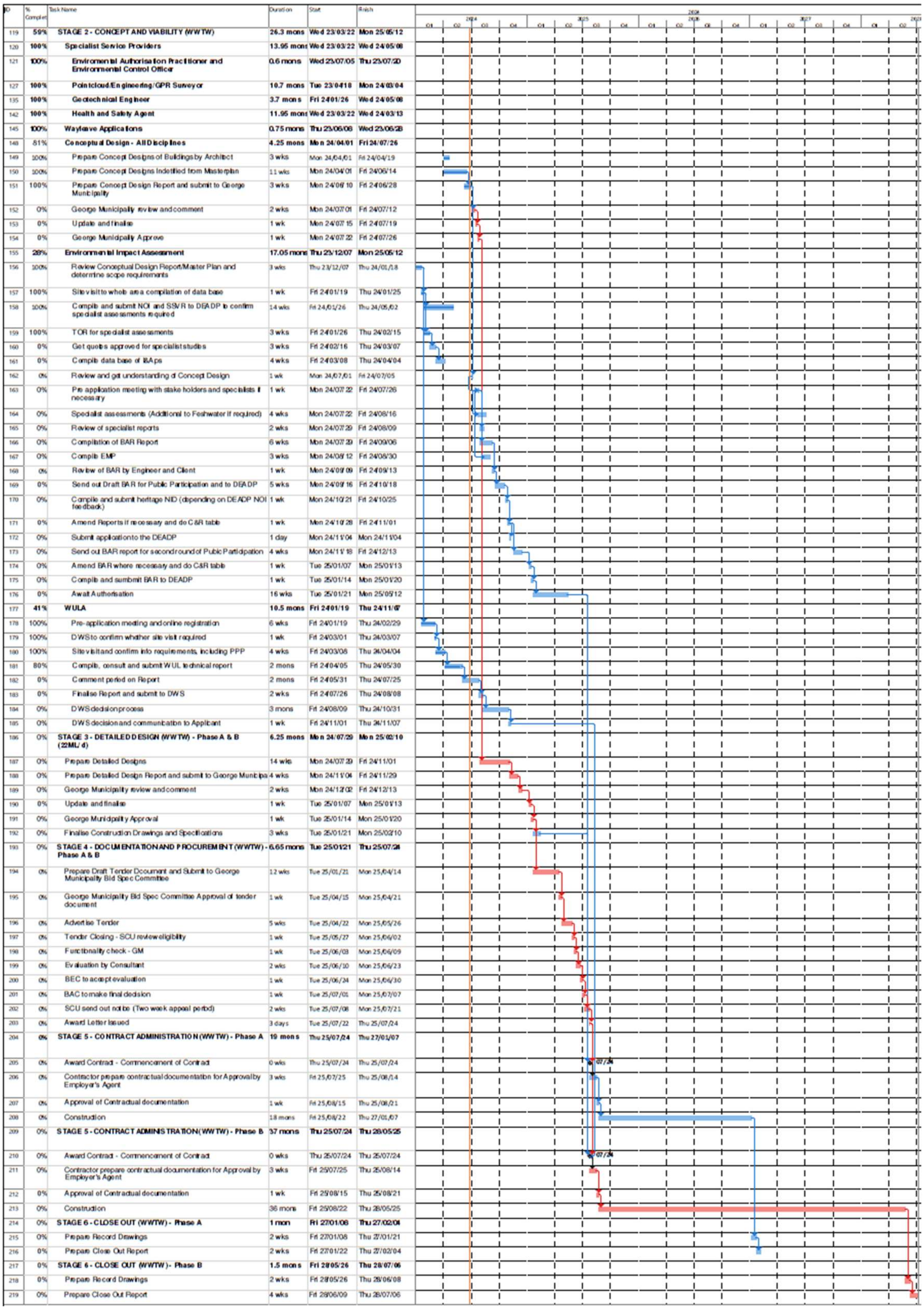
Table 19-8: Estimated Annual Maintenance costs

Parameter	BBF (2028)	Phase A (2026)	Phase B (2028)
Capacity (MLD)		13.2	22
R/a - 2025	R2 772 208.45	R2 184 791.50	R9 950 971.54
R/annum in future value *	R3 301 744.62	R2 315 878.99	R11 851 766.32
	* in the year shown in the column heading		

20 IMPLEMENTATION TIMELINE

The proposed project timeline for the implementation of CESA Stages 3 to 6 is shown in Table 20-1.

Table 20-1: Implementation Timeline proposed for Phase A and B of Gwaing WWTW Upgrade.



ID	% Complete	Task Name	Duration	Start	Finish
119	59%	**STAGE 2 - CONCEPT AND VIABILITY (WWTW)**	**26.3 mons**	**Wed 23/03/22**	**Mon 25/05/12**
120	100%	**Specialist Service Providers**	**13.95 mons**	**Wed 23/03/22**	**Wed 24/05/08**
121	100%	**Environmental Authorisation Practitioner and Environmental Control Officer**	**0.6 mons**	**Wed 23/07/05**	**Thu 23/07/20**
127	100%	**Pointcloud/Engineering/GPR Surveyor**	**10.7 mons**	**Tue 23/04/18**	**Mon 24/03/04**
135	100%	**Geotechnical Engineer**	**3.7 mons**	**Fri 24/01/26**	**Wed 24/05/08**
142	100%	**Health and Safety Agent**	**11.95 mons**	**Wed 23/03/22**	**Wed 24/03/13**
145	100%	**Wayleave Applications**	**0.75 mons**	**Thu 23/06/08**	**Wed 23/06/28**
148	81%	**Conceptual Design - All Disciplines**	**4.25 mons**	**Mon 24/04/01**	**Fri 24/07/26**
149	100%	Prepare Concept Designs of Buildings by Architect	3 wks	Mon 24/04/01	Fri 24/04/19
150	100%	Prepare Concept Designs Indetified from Masterplan	11 wks	Mon 24/04/01	Fri 24/06/14
151	100%	Prepare Concept Design Report and submit to George Municipality	3 wks	Mon 24/06/10	Fri 24/06/28
152	0%	George Municipality review and comment	2 wks	Mon 24/07/01	Fri 24/07/12
153	0%	Update and finalise	1 wk	Mon 24/07/15	Fri 24/07/19
154	0%	George Municipality Approve	1 wk	Mon 24/07/22	Fri 24/07/26
155	28%	**Environmental Impact Assessment**	**17.05 mons**	**Thu 23/12/07**	**Mon 25/05/12**
156	100%	Review Conceptual Design Report/Master Plan and determine scope requirements	3 wks	Thu 23/12/07	Thu 24/01/18
157	100%	Site visit to whole area compilation of data base	1 wk	Fri 24/01/19	Thu 24/01/25
158	100%	Compile and submit NOI and SSVR to DEADP to confirm specialist assessments required	14 wks	Fri 24/01/26	Thu 24/05/02
159	100%	TOR for specialist assessments	3 wks	Fri 24/01/26	Thu 24/02/15
160	0%	Get quotes approved for specialist studies	3 wks	Fri 24/02/16	Thu 24/03/07
161	0%	Compile data base of I&APs	4 wks	Fri 24/03/08	Thu 24/04/04
162	0%	Review and get understanding of Concept Design	1 wk	Mon 24/07/01	Fri 24/07/05
163	0%	Pre application meeting with stake holders and specialists if necessary	1 wk	Mon 24/07/22	Fri 24/07/26
164	0%	Specialist assessments (Additional to Feshwater if required)	4 wks	Mon 24/07/22	Fri 24/08/16
165	0%	Review of specialist reports	2 wks	Mon 24/07/29	Fri 24/08/09
166	0%	Compilation of BAR Report	6 wks	Mon 24/07/29	Fri 24/09/06
167	0%	Compile EMP	3 wks	Mon 24/08/12	Fri 24/08/30
168	0%	Review of BAR by Engineer and Client	1 wk	Mon 24/09/09	Fri 24/09/13
169	0%	Send out Draft BAR for Public Participation and to DEADP	5 wks	Mon 24/09/16	Fri 24/10/18
170	0%	Compile and submit heritage NID (depending on DEADP NOI feedback)	1 wk	Mon 24/10/21	Fri 24/10/25
171	0%	Amend Reports if necessary and do C&R table	1 wk	Mon 24/10/28	Fri 24/11/01
172	0%	Submit application to the DEADP	1 day	Mon 24/11/04	Mon 24/11/04
173	0%	Send out BAR report for second round of Public Participation	4 wks	Mon 24/11/18	Fri 24/12/13
174	0%	Amend BAR where necessary and do C&R table	1 wk	Tue 25/01/07	Mon 25/01/13
175	0%	Compile and sumbmit BAR to DEADP	1 wk	Tue 25/01/14	Mon 25/01/20
176	0%	Await Authorisation	16 wks	Tue 25/01/21	Mon 25/05/12
177	41%	**WULA**	**10.5 mons**	**Fri 24/01/19**	**Thu 24/11/07**
178	100%	Pre-application meeting and online registration	6 wks	Fri 24/01/19	Thu 24/02/29
179	100%	DWS to confirm whether site visit required	1 wk	Fri 24/03/01	Thu 24/03/07
180	100%	Site visit and confirm info requirements, including PPP	4 wks	Fri 24/03/08	Thu 24/04/04
181	80%	Compile, consult and submit WUL technical report	2 mons	Fri 24/04/05	Thu 24/05/30
182	0%	Comment period on Report	2 mons	Fri 24/05/31	Thu 24/07/25
183	0%	Finalise Report and submit to DWS	2 wks	Fri 24/07/26	Thu 24/08/08
184	0%	DWS decision process	3 mons	Fri 24/08/09	Thu 24/10/31
185	0%	DWS decision and communication to Applicant	1 wk	Fri 24/11/01	Thu 24/11/07
186	0%	**STAGE 3 - DETAILED DESIGN (WWTW) - Phase A & B (22ML/d)**	**6.25 mons**	**Mon 24/07/29**	**Mon 25/02/10**
187	0%	Prepare Detailed Designs	14 wks	Mon 24/07/29	Fri 24/11/01
188	0%	Prepare Detailed Design Report and submit to George Municipa	4 wks	Mon 24/11/04	Fri 24/11/29
189	0%	George Municipality review and comment	2 wks	Mon 24/12/02	Fri 24/12/13
190	0%	Update and finalise	1 wk	Tue 25/01/07	Mon 25/01/13
191	0%	George Municipality Approval	1 wk	Tue 25/01/14	Mon 25/01/20
192	0%	Finalise Construction Drawings and Specifications	3 wks	Tue 25/01/21	Mon 25/02/10
193	0%	**STAGE 4 - DOCUMENTATION AND PROCUREMENT (WWTW) - Phase A & B**	**6.65 mons**	**Tue 25/01/21**	**Thu 25/07/24**
194	0%	Prepare Draft Tender Dcoument and Submit to George Municipality Bid Spec Committee	12 wks	Tue 25/01/21	Mon 25/04/14
195	0%	George Municipality Bid Spec Committee Approval of tender document	1 wk	Tue 25/04/15	Mon 25/04/21
196	0%	Advertise Tender	5 wks	Tue 25/04/22	Mon 25/05/26
197	0%	Tender Closing - SCU review eligibility	1 wk	Tue 25/05/27	Mon 25/06/02
198	0%	Functionality check - GM	1 wk	Tue 25/06/03	Mon 25/06/09
199	0%	Evaluation by Consultant	2 wks	Tue 25/06/10	Mon 25/06/23
200	0%	BEC to accept evaluation	1 wk	Tue 25/06/24	Mon 25/06/30
201	0%	BAC to make final decision	1 wk	Tue 25/07/01	Mon 25/07/07
202	0%	SCU send out notice (Two week appeal period)	2 wks	Tue 25/07/08	Mon 25/07/21
203	0%	Award Letter Issued	3 days	Tue 25/07/22	Thu 25/07/24
204	0%	**STAGE 5 - CONTRACT ADMINISTRATION (WWTW) - Phase A**	**19 mons**	**Thu 25/07/24**	**Thu 27/01/07**
205	0%	Award Contract - Commencement of Contract	0 wks	Thu 25/07/24	Thu 25/07/24
206	0%	Contractor prepare contractual documentation for Approval by Employer's Agent	3 wks	Fri 25/07/25	Thu 25/08/14
207	0%	Approval of Contractual documentation	1 wk	Fri 25/08/15	Thu 25/08/21
208	0%	Construction	18 mons	Fri 25/08/22	Thu 27/01/07
209	0%	**STAGE 5 - CONTRACT ADMINISTRATION (WWTW) - Phase B**	**37 mons**	**Thu 25/07/24**	**Thu 28/05/25**
210	0%	Award Contract - Commencement of Contract	0 wks	Thu 25/07/24	Thu 25/07/24
211	0%	Contractor prepare contractual documentation for Approval by Employer's Agent	3 wks	Fri 25/07/25	Thu 25/08/14
212	0%	Approval of Contractual documentation	1 wk	Fri 25/08/15	Thu 25/08/21
213	0%	Construction	36 mons	Fri 25/08/22	Thu 28/05/25
214	0%	**STAGE 6 - CLOSE OUT (WWTW) - Phase A**	**1 mon**	**Fri 27/01/08**	**Thu 27/02/04**
215	0%	Prepare Record Drawings	2 wks	Fri 27/01/08	Thu 27/01/21
216	0%	Prepare Close Out Report	2 wks	Fri 27/01/22	Thu 27/02/04
217	0%	**STAGE 6 - CLOSE OUT (WWTW) - Phase B**	**1.5 mons**	**Fri 28/05/26**	**Thu 28/07/06**
218	0%	Prepare Record Drawings	2 wks	Fri 28/05/26	Thu 28/06/08
219	0%	Prepare Close Out Report	4 wks	Fri 28/06/09	Thu 28/07/06

21 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.

21.1 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.

21.2 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.

21.2.1 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.

22 CONCLUSION AND RECOMMENDATIONS

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 soar drying and 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.

The completion of the detailed design for Phases A and B will solidify the design and pave the way for implementing the remaining phases identified in the Master Plan. Additionally, having completed detail designs will enhance the Municipality's readiness for implementation, making 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 is recommended to procure Phases A and B simultaneously, but to prioritize the construction 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.

23 DRAWING REGISTER

The concept design drawings register is shown in Table 23-1. The drawings are attached in Appendix C: Concept Design Drawings.

Table 23-1: Drawing Register

Drawing Register GWAING WWTW - Concept Design				
Drawing Number	**Title**	**Size**	**Status**	**Revision**
002-009 PFD and P&ID				
100005-005-001-PRO-REV A	P&ID - HEAD OF WORKS	A1	I	A
100005-005-002-PRO-REV A	P&ID - SCREENING	A1	I	A
100005-005-003-PRO-REV A	P&ID - DEGRITTING	A1	I	A
100005-005-004-PRO-REV A	P&ID - REACTOR FLUMES	A1	I	A
100005-005-005-PRO-REV A	P&ID - REACTOR	A1	I	A
100005-005-006-PRO-REV A	P&ID - SSTs B	A1	I	A
100005-005-007-PRO-REV A	P&ID - SSTs A	A1	I	A
100005-005-008-PRO-REV A	P&ID - RAS PUMP STATION	A1	I	A
100005-005-009-PRO-REV A	P&ID - BLOWERS	A1	I	A
100005-005-010-PRO-REV A	P&ID - DEWATERING SHEET 1	A1	I	A
100005-005-011-PRO-REV A	P&ID - DEWATERING SHEET 2	A1	I	A
010-019 SITE WORKS				
100005-010-201-CIV-REV B	SITE LAYOUT WITH LABELS	A1	I	B
100005-010-202-CIV-REV B	SITE LAYOUT	A1	I	B
100005-010-203-CIV-REV B	PHASE A LAYOUT	A1	I	B
100005-010-204-CIV-REV B	PHASE B LAYOUT	A1	I	B
100005-010-205-CIV-REV B	BBF LAYOUT	A1	I	B
100005-010-206-CIV-REV B	SITE LAYOUT COMBINED PHASES	A1	I	B
100005-010-207-CIV-REV B	ROADS LAYOUT	A1	I	B
100005-010-208-CIV-REV A	DEMOLITION LAYOUT	A1	I	A
100005-010-209-CIV-REV A	MUNICIPAL ERF LAYOUT	A1	I	A
020-029 PRE-TREATMENT / INLET WORKS / SCREENING/ DEGRITTING /				
100005-020-201-CIV-REV A	INLET WORKS - ISOMETRIC VIEW	A1	I	A
100005-020-202-CIV-REV A	INLET WORKS - ELEVATIONS SHEET 1	A1	I	A
100005-020-203-CIV-REV A	INLET WORKS - FLOOR PLAN 1	A1	I	A
100005-020-204-CIV-REV A	INLET WORKS - FLOOR PLAN 2	A1	I	A
100005-020-205-CIV-REV A	INLET WORKS - SECTION A,B,C,D,E,F	A1	I	A
100005-020-206-CIV-REV A	INLET WORKS - SECTION G,H,I	A1	I	A
100005-020-207-CIV-REV A	INLET WORKS - SECTION J,K	A1	I	A
100005-020-208-CIV-REV A	INLET WORKS - SECTION L	A1	I	A
100005-020-209-CIV-REV A	INLET WORKS - DETAILS	A1	I	A
040-049 REACTOR SPLITTER				
100005-040-201-CIV-REV A	REACTOR SPLITTER - ISOMATRIC VIEW	A1	I	A
100005-040-202-CIV-REV A	REACTOR SPLITTER - ELEVATION 1	A1	I	A
100005-040-203-CIV-REV A	REACTOR SPLITTER - FLOOR PLAN 1	A1	I	A
100005-040-204-CIV-REV A	REACTOR SPLITTER - FLOOR PLAN 2	A1	I	A
100005-040-205-CIV-REV A	REACTOR SPLITTER - SECTION 1	A1	I	A
100005-040-206-CIV-REV A	REACTOR SPLITTER - SECTION 2	A1	I	A
041-049 BIOLOGICAL REACTOR				
100005-041-201-CIV-REV A	BIOLOGICAL REACTOR - ISOMATRIC VIEW SHEET 1	A1	I	A
100005-041-202-CIV-REV A	BIOLOGICAL REACTOR - ELEVATION SHEET 1	A1	I	A
100005-041-203-CIV-REV A	BIOLOGICAL REACTOR - ELEVATION SHEET 2	A1	I	A
100005-041-204-CIV-REV A	BIOLOGICAL REACTOR - FLOOR PLAN SHEET 1	A1	I	A
100005-041-205-CIV-REV A	BIOLOGICAL REACTOR - FLOOR PLAN SHEET 2	A1	I	A
100005-041-206-CIV-REV A	BIOLOGICAL REACTOR - FLOOR PLAN SHEET 3	A1	I	A
100005-041-207-CIV-REV A	BIOLOGICAL REACTOR - SECTION A,B	A1	I	A
100005-041-208-CIV-REV A	BIOLOGICAL REACTOR - SECTION C,D	A1	I	A
100005-041-209-CIV-REV A	BIOLOGICAL REACTOR - SECTION E,F	A1	I	A
100005-041-210-CIV-REV A	BIOLOGICAL REACTOR - SECTIONS G,H	A1	I	A
100005-041-211-CIV-REV A	BIOLOGICAL REACTOR - SECTION I,J	A1	I	A
051-059 SECONDARY SETTLING SPLITTER				
100005-050-201-CIV-REV A	SECONDARY SETTLING TANK SPLITTER - ISOMETRIC VIEW	A1	I	A
100005-050-202-CIV-REV A	SECONDARY SETTLING TANK SPLITTER - ELEVATION SHEET 1	A1	I	A
100005-050-203-CIV-REV A	SECONDARY SETTLING TANK SPLITTER - FLOOR PLAN 1	A1	I	A
100005-050-204-CIV-REV A	SECONDARY SETTLING TANK SPLITTER - FLOOR PLAN 2	A1	I	A
100005-050-205-CIV-REV A	SECONDARY SETTLING TANK SPLITTER - FLOOR PLAN 3	A1	I	A
100005-050-206-CIV-REV A	SECONDARY SETTLING TANK SPLITTER - SECTION A,B,C	A1	I	A
100005-050-207-CIV-REV A	SECONDARY SETTLING TANK SPLITTER - SECTION SHEET 2	A1	I	A
100005-050-208-CIV-REV A	SECONDARY SETTLING TANK SPLITTER - SECTION SHEET 3	A1	I	A

050-059 SST's				
100005-051-201-CIV-REV A	SECONDARY SETTLING TANK - ISOMETRIC	A1	I	A
100005-051-202-CIV-REV A	SECONDARY SETTLING TANK - ELEVATION SHEET 1	A1	I	A
100005-051-203-CIV-REV A	SECONDARY SETTLING TANK - FLOOR PLAN 1	A1	I	A
100005-051-204-CIV-REV A	SECONDARY SETTLING TANK - FLOOR PLAN 2	A1	I	A
100005-051-205-CIV-REV A	SECONDARY SETTLING TANK - FLOOR PLAN 3	A1	I	A
100005-051-206-CIV-REV A	SECONDARY SETTLING TANK - FLOOR PLAN 4	A1	I	A
100005-051-207-CIV-REV A	SECONDARY SETTLING TANK - SECTION 1	A1	I	A
100005-051-208-CIV-REV A	SECONDARY SETTLING TANK - SECTION 2	A1	I	A
100005-051-209-CIV-REV A	SECONDARY SETTLING TANK - SECTION 3	A1	I	A
070-089 RAS PUMP STATION				
100005-070-201-CIV-REV A	RAS PUMP STATION - ISOMETRIC	A1	I	A
100005-070-202-CIV-REV A	RAS PUMP STATION - ELEVATION SHEET 1	A1	I	A
100005-070-203-CIV-REV A	RAS PUMP STATION - FLOOR PLAN 1	A1	I	A
100005-070-204-CIV-REV A	RAS PUMP STATION - FLOOR PLAN 2	A1	I	A
100005-070-205-CIV-REV A	RAS PUMP STATION - FLOOR PLAN 3	A1	I	A
100005-070-206-CIV-REV A	RAS PUMP STATION - SECTION A-A	A1	I	A
100005-070-207-CIV-REV A	RAS PUMP STATION - SECTION B-B	A1	I	A
100005-070-208-CIV-REV A	RAS PUMP STATION - SECTION C-C	A1	I	A
100005-070-209-CIV-REV A	RAS PUMPSTATION - SECTION D-D	A1	I	A
100005-070-210-CIV-REV A	RAS PUMPSTATION - SECTION E-E	A1	I	A
100005-070-211-CIV-REV A	RAS PUMPSTATION - SECTION F-F	A1	I	A
100005-070-212-CIV-REV A	RAS PUMPSTATION - SECTION G-G	A1	I	A
100005-070-213-CIV-REV A	RAS PUMPSTATION - SECTION H-H	A1	I	A
090-099 DIFINFECTION / CHLORINE CONTACT CHANNEL / CHLORINE DOSING / BUILDING / UV CHANNELS				
100005-090-201-CIV-REV A	UV CHANNEL - ISOMETRIC VIEW	A1	I	A
100005-090-202-CIV-REV A	UV CHANNEL - ELEVATION	A1	I	A
100005-090-203-CIV-REV A	UV CHANNEL - FLOOR PLAN 1	A1	I	A
100005-090-204-CIV-REV A	UV CHANNEL - FLOOR PLAN 2	A1	I	A
100005-090-205-CIV-REV A	UV CHANNEL - FLOOR PLAN 3	A1	I	A
100005-090-206-CIV-REV A	UV CHANNEL - FLOOR PLAN 4	A1	I	A
100005-090-207-CIV-REV A	UV CHANNEL - SECTION A-B	A1	I	A
100005-090-208-CIV-REV A	UV CHANNEL - SECTION C,D	A1	I	A
100005-090-209-CIV-REV A	UV CHANNEL - SECTION E,F,G	A1	I	A
100005-090-210-CIV-REV A	UV CHANNEL - SECTION H,I,J	A1	I	A
140-159 DEWATERING BUILDING				
100005-141-201-CIV-REV A	DEWATERING BUILDING - ISOMETRIC VIEW	A1	I	A
100005-141-202-CIV-REV A	DEWATERING BUILDING - ELEVATION 1	A1	I	A
100005-141-203-CIV-REV A	DEWATERING BUILDING - ELEVATION 2	A1	I	A
100005-141-204-CIV-REV A	DEWATERING BUILDING - FLOOR PLAN 1	A1	I	A
100005-141-205-CIV-REV A	DEWATERING BUILDING - FLOOR PLAN 2	A1	I	A
100005-141-206-CIV-REV A	DEWATERING BUILDING - FLOOR PLAN 3	A1	I	A
100005-141-207-CIV-REV A	DEWATERING BUILDING - SECTION A-A	A1	I	A
100005-141-208-CIV-REV A	DEWATERING BUILDING - SECTION B-B	A1	I	A
100005-141-209-CIV-REV A	DEWATERING BUILDING - SECTION C-C	A1	I	A
100005-141-210-CIV-REV A	DEWATERING BUILDING - SECTION D-D	A1	I	A
100005-141-211-CIV-REV A	DEWATERING BUILDING - SECTION E-E	A1	I	A
100005-141-212-CIV-REV A	DEWATERING BUILDING - SECTION F-F	A1	I	A
ARCHITECTURAL DRAWINGS				
PE23-006-SP2.02	Guard House GUARD HOUSE / FLOOR PLAN -ELEVATIONS-3D VIEW	A1	I	A
PE23-006-SP2.03	SUB-STATION / FLOOR PLAN -ELEVATIONS-3D VIEW	A1	I	A
PE23-006-SP2.01	BLOWER HOUSE LAYOUT	A1	I	A
PE23-006-SP3.00	BLOWER HOUSE ELEVATIONS	A1	I	A
PE23-006-SP2.00	ADMIN BUILDING PLANS	A1	I	A
PE23-006-SP3.00	ADMIN BUILDING ELEVATIONS	A1	I	A
PE23-006-SP4.00	ADMIN BUILDING ELEVATIONS	A1	I	A
ELECTRICAL DRAWINGS				
1752-ELE-R001	ELECTRICAL INSTALLATION SITE RETICULATION	A1	I	A

Drawing Status	
I	For Information
T	For Tender
C	For Construction
A	As-Built

24 APPENDICES

24.1 Appendix A: Geotechnical Report

24.2 Appendix B: GLS Electrical Capacity Investigation Study

24.3 Appendix C: Concept Design Drawings

APPENDIX A: GEOTECHNICAL REPORT

APPENDIX B: GLS ELECTRICITY CAPACITY INVESTIGATION STUDY

APPENDIX C: CONCEPT DESIGN DRAWINGS