Zitholele Consulting
Reg. No. 2000/000392/07

PO Box 6002 Halfway House 1685, South Africa
Building 1, Maxwell Office Park, Magwa Crescent West
c/o Allandale Road & Maxwell Drive, Waterfall City, Midrand
T : 011 207 2060 **F** : 086 674 6121 **E** : mail@zitholele.co.za

REPORT

ESKOM-MEDUPI

FGD Effluent Waste Water Treatment Plant Concept Feasibility Report

Report No : 17041-45-Rep-001

Submitted to :

Eskom Holdings SOC Ltd
1 Maxwell Drive
Sunninghill
Sandton
2157

01 June 2018 17041

DOCUMENT CONTROL SHEET

Project Title: **Medupi FGD WWTP**

Project No: **17041**

Document Ref. No: 17041-43-Process Design Report

DOCUMENT APPROVAL

ACTION	DESIGNATION	NAME	DATE	SIGNATURE
Prepared	Civil Engineer	N Govender PrEng	1/6/2018	
Reviewed	Project Manager	N Rajasakran PrEng PrCPM	1/6/2018	
Approved	Project Manager	N Rajasakran PrEng PrCPM	1/6/2018	

RECORD OF REVISIONS

Date	Revision	Author	Comments
1/6/2018	0	N Govender PrEng/ B Muller PrEng	Issued for information

EXECUTIVE SUMMARY

Medupi Power Station, located in Limpopo, is in the process of designing and installing Flue Gas Desulphurisation (FGD) Technology to control sulphur dioxide emissions which is required to meet the South African Minimum Emission Standards. This report describes the process undertaken to evaluate and identify suitable process technologies to treat effluent from the FGD process at a new Wastewater Treatment Plant (WwTP).

Evaluation of the process technologies were conducted for two different water qualities (Case 1 and 2) during a trade off workshop. At the workshop robust discussions and interrogation of the evaluation criteria were undertaken to ensure that the scoring provided an accurate reflection of the technology being evaluated. Following the trade-off of the two options, a pre-treatment system making use of lime dosing; organo-sulphide dosing and ferric chloride dosing and a desalination system making use of thermal evaporation was ranked as the preferred option for Case 1 and 2. The designs will thus be further developed using this selected option.

TABLE OF CONTENTS

SECTION **PAGE**

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

LIST OF ACRONYMS

FGD	Flue Gas Desulphurisation
WHSF	Waste Handling and Storage Facility
WwTP	Wastewater Treatment Plant
ZLED	Zero Liquid Effluent Discharge

1 INTRODUCTION

Medupi Power Station, located in Limpopo, is in the process of designing and installing Flue Gas Desulphurisation (FGD) Technology to control sulphur dioxide emissions. This is required to meet the South African Minimum Emission Standards. The current design is based on the wet FGD process. This process utilises limestone (consisting primarily of $CaCO_3$) to react with gaseous sulphur dioxide (SO_2) to form gypsum ($CaSO_4 \bullet 2\ H_2O$) in a forced oxidation process. A stream concentrated with gypsum crystals is bled from the absorber to a gypsum dewatering system. A part of the bleed steam from the dewatering system (called FGD blowdown) needs to be treated in order to recover the water.

Eskom appointed Zitholele Consulting to design a waste water treatment plant (WwTP) to treat the FGD blowdown stream so that the water can be re-used. A requirement of the project is to have zero liquid waste discharge on the Medupi site.

The aim of this document is to describe the process design that was performed for the concept design.

2 BASIS OF DESIGN

Currently, two design cases for the FGD plant are being considered with respect to the feed limestone quality. The FGD blowdown quality is affected by the limestone used in the absorber thus resulting in two water quality cases that require assessment. The design feed water quality for each case that needs to be treated by the FGD waste water treatment plant (FGD WWTP) is shown in Table 1. The maximum design flows for each case is as follows:

- Case 1 = 44 m^3/h
- Case 2 = 45 m^3/h

The design should furthermore be able to cater for a minimum flow of 12 m^3/h.

Table 1: Design feed water quality

		85% limestone, crocodile water FGD BLOWDOWN (OPTION 1)		96% limestone, crocodile water FGD BLOWDOWN (OPTION 2)	
		mg/l AS SUCH	mg/l AS CaCO3	mg/l AS SUCH	mg/l AS CaCO3
Cations	Calcium, Ca	500		16400	
	Magnesium, Mg	17710		340	
	Sodium, Na	2601		1200	
	Potassium, K	200		200	
Anions	M-Alkalinity				
	Sulfate, SO4	30670		850	
	Sulfide, SO3	1		110	
	Chloride, Cl	31900		30000	
	Nitrate, NO3	600		600	
	Carbon Dioxide, CO2	10		10	
	Silica, SiO2	790			
Alkalinity	Bicarbonate, HCO3			800	
	Carbonate, CO3			0	
Other	pH (min/max)	4-7		6	
	Total Dissolved Solids, TDS	86000		75000	
	Total Suspended Solids, TSS	35600		16830	
	Temperature, °C (min/normal/max)	60		50	
	Biological Oxygen Demand, BOD	nv		60	
	Chemical Oxygen Demand, COD	min 670		250	
	Total Organic Carbon, TOC	min 360			
	Ammonium, NH4	nv		200	
Metals	Aluminum, Al	648		50	
	Antimiony, Sb	nv		1	
	Arsenic, As	-		1	
	Barium, Ba	2.4		30	
	Beryllium, Be	0.24			
	Boron, B	7.68		40	
	Cadmium, Cd	0.24		0.6	
	Chromium, Cr	0.72		3	
	Cobalt, Co	0.24		1	
	Copper, Cu	0.24		2	
	Fluoride, F	354		30	
	Iron, Fe	551		40	
	Lead, Pb	0.96		2	
	Manganese, Mn	165		30	
	Mercury, Hg	-		0.2	
	Molybdenum, Mo	-		2	
	Nickel, Ni	0.24		3	
	Selenium, Se	-		1	
	Silver, Ag	nv		2	
	Strontium, Sr	5.76		120	
	Thallium, Tl	nv			
	Titanium, Ti	292		0.6	
	Vanadium, V	0.72		0.8	
	Zinc, Zn	1.2		5	

The target water quality is derived from the raw water quality of the Mokolo Water supply system. The minimum, maximum and average values of the Mokolo water supply system is shown in Table 2, as well as the selected design basis values in the last column. The FGD WwTP must be designed such that the treated water quality meets the values listed in the Design Basis column.

Table 2: Treated water quality

Raw Water Analysis - Mokolo Water Supply				
Constituent/Water Quality	Raw Water - Maximum	Raw Water - Minimum	Raw Water - Average	Design Basis
Turbidity, NTU	3.6	0.7	1.5	1.8
Suspended Solids, mg/L	10.0	10.0	10.0	12.0
pH	9.5	6.0	8.1	8.8
Conductivity, K_{25}, µS/cm	112.3	66.7	88.6	106.3
Alkalinity to pH 8.3, P-alk as $CaCO_3$, mg/L	15.0	1.0	5.7	6.9
Alkalinity to pH 4.5, M-alk as $CaCO_3$, mg/L	36.9	22.1	31.3	37.6
Total Alkalinity, T-Alk, as $CaCO_3$, mg/L	50.0	22.1	32.6	39.1
Magnesium Hardness, MgH, as $CaCO_3$, mg/L	22.3	5.0	17.5	21.0
Calcium Hardness, CaH, as $CaCO_3$, mg/L	36.0	10.1	15.9	19.1
Total Hardness, TH, as $CaCO_3$, mg/L	56.0	18.0	32.0	38.5
Sodium, Na, mg/L	15.2	5.0	6.2	7.4
Potassium, K, mg/L	1.5	1.1	1.3	1.6
Ammonia NH_3, mg/L	1.5	0.0	0.6	0.7
Chloride, Cl, mg/L	24.8	5.3	10.0	12.0
Sulphate, SO_4, mg/L	3.7	0.5	1.8	2.2
Fluoride, F, mg/L	0.2	0.1	0.1	0.2
Nitrate, NO_3, mg/L	–	–	–	–
Oxygen Absorbed (OA), as $KMnO_4$, mg/L	3.3	1.2	2.3	2.7
Reactive Silica, as SiO_2, mg/L	99.2	4.9	15.8	19.0
Strontium, Sr, ug/L	90.0	90.0	90.0	108.0
Barium, Ba, ug/L	20.0	20.0	20.0	24.0
Iron, Fe, ug/L	5.0	5.0	5.0	6.0
Manganese, Mn, ug/L	5.0	5.0	5.0	6.0
Boron, B, ug/L	70.0	20.0	42.5	51.0

3 SCREENING OF OPTIONS

Based on literature surveys and previous FGD waste water treatment projects, various concept-level options were developed for the WwTP. After consultation with various experts in the field, some of these options could be eliminated as part of a screening stage, before developing them further. The options that were considered, as well as the reasons for eliminating or retaining them, are documented in this section. Concept designs were developed for the options that passed the screening stage.

It must be noted that this study was limited to the evaluation of treatment options to enable the re-use of the FGD effluent water. Waste produced will be transported to a waste disposal facility.

After analysing the feed water quality, it is proposed that the solution consists of some form of pre-treatment to remove suspended solids, metals and supersaturated constituents. To meet the required treated water quality with zero liquid discharge, further treatment using some form of desalination and waste management will be required. The pre-treatment and desalination options are described in more detail below.

4 PRE-TREATMENT OVERVIEW

While several pre-treatment options may be considered, a typical physical-chemical treatment process commonly used for FGD wastewater treatment was selected in this project for preliminary process development and cost estimation.

The aim of pre-treatment plant is flow equalization, calcium sulphate desaturation, suspended solids and trace metals removal, and pH adjustment. The main pre-treatment processes are described briefly below (for a detailed description, refer to section 4.1).

Flow Equalization: The purpose of flow equalization tank is to minimize variation in flows and water quality and optimize the downstream treatment plant size. Based on site conditions, it is assumed that the heat loss in the equalization tank will not be significant, and will not impact the calcium sulphate solubility, which increases as the temperature decreases.

Desaturation: This step is to reduce the concentration of sulphate in the wastewater stream by adding lime to raise the pH to approximately 8.5 to 9 to precipitate calcium sulphate. Raising the pH higher will result in calcium carbonate precipitation but would lead to higher lime costs and higher sludge processing and handling costs.

Primary Clarification: Removes the bulk of suspended solids and calcium sulphate produced in the desaturation reactor. A fraction of the sludge from the clarifier is recirculated to the desaturation reactor to provide additional sites for calcium sulphate precipitation and hence improves process efficiency.

Heavy Metals Removal: To meet low effluent limits for heavy metals including mercury, and as metal sulphide have lower solubility than metal hydroxides, organo-sulphides (for example TMT-15) is added to precipitate heavy metals. Further work during the subsequent design phases should critically evaluate the need for this aspect based on the intended re-use and the risk to equipment and human health.

Coagulation: Iron salt such as ferric chloride ($FeCl_3$) is typically added to neutralize particle charge and assist with the formation of dense flocs. It also enables the removal of Arsenic by co-precipitation with iron.

Flocculation: Polymer is typically added for floc agglomeration and to form dense flocs that can be removed in the downstream clarifier.

Secondary Clarification: To remove suspended solids, and metal precipitates. A fraction of the sludge is recycled to assist form dense stable flocs and improve process efficiency.

pH adjustment: pH is adjusted back to neutral by dosing acid (as required by the downstream processes).

Filtration: To reduce suspended solids load on the downstream treatment processes, the water is typically filtered using granular media filters having high solid holding capacity.

5 PRE-TREATMENT OPTIONS EVALUATED

Either Lime ($Ca(OH)_2$) or Caustic (NaOH) can be used for desaturation. Lime is typically used as it is substantially cheaper than Caustic. (It is important to note that soda ash dosing is being considered for Case 2 and this report will be updated to reflect this once the change is baselined).

Lime is normally dosed as a milk-of-lime solution, which can be prepared from either of the following two chemicals:

- Option 1: Quicklime (CaO)
- Option 2: Hydrated lime ($Ca(OH)_{2)}$

For Option 1, a slaker system is required to convert quicklime (CaO) to slaked or hydrated lime (Ca(OH)2). This is done by mixing water with the quicklime and allowing the following exothermic reaction to take place:

$$CaO + H2O \dashrightarrow Ca(OH)_2$$

The slaked lime can then be made up to a milk-of-lime solution by adding additional make-up water.

For Option 2, the lime is already hydrated, thus only water needs to be added to the lime to make up the milk-of-lime solution. The advantage of option 2 is that less infrastructure is required compared to option 1. The disadvantage of option 2 is that the bulk density of de-aerated hydrated lime is only 600 kg/m^3 (as per quote received from PPC Lime), compared to about 1000 to 1100 kg/m^3 for de-aerated quicklime. This means that the volume of dry feed material that needs to be transported to site for option 1 will be 40% less than option 2. Based on this, it was decided that, due to the savings in transport costs, Option 1 will be the preferred option.

Three permutations in terms of the dosing position and removal of the precipitated solids were considered (refer to Figure 1 to 3 for simplified flow diagrams of each option):

- Option 1:
  - Lime is dosed to Reactor 1.
  - Precipitated solids are removed in Primary Clarifier.
  - Organo-sulphide is dosed to Reactor 2.
  - Ferric is dosed to Reactor 3.
  - Remaining suspended solids and precipitated metal sulphides are removed in Secondary Clarifier.
  - This option is typically used when the suspended solids in the feed stream is high (above 1 to 2% solids).

- Option 2:
  - Similar to Option 1, except that the clarifier between Reactor 1 and 2 is removed.
  - Effluent from Reactor 1 flows directly into Reactor 2.
  - All solids are removed using one clarifier after Reactor 3.
  - This option can be used when the solids loading is not too high, e.g. if solids in the feed stream is below 1%.

- Option 3:
  - Similar to Option 2, except that the lime and organo-sulphides are dosed to the same reactor. Reactor 2 is therefore eliminated.
  - This option is also used when the solids loading is relatively low, although dosing lime and organo-sulphides in separate reactors seems to be the preferred option in most applications.

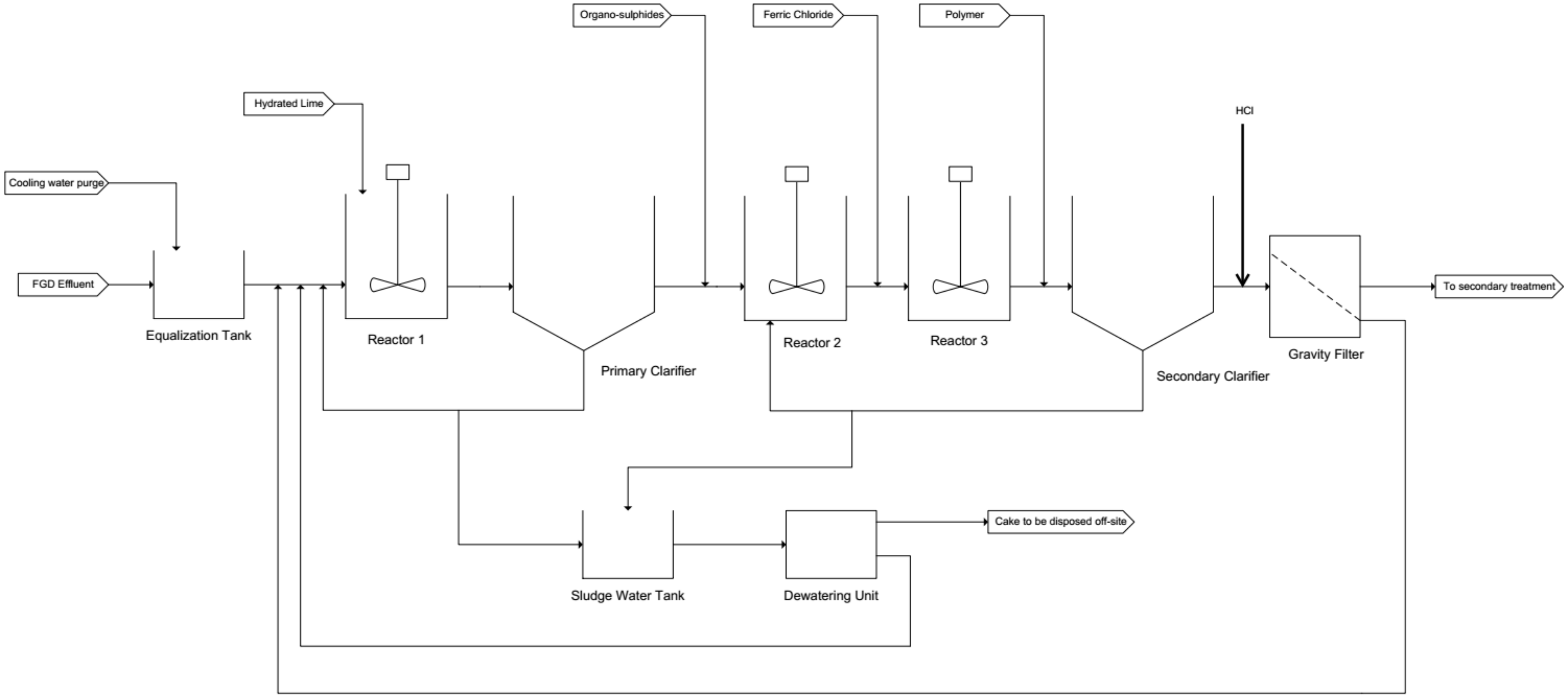


Figure 1: Pre-Treatment - Simplified Flow Diagram (Option 1)

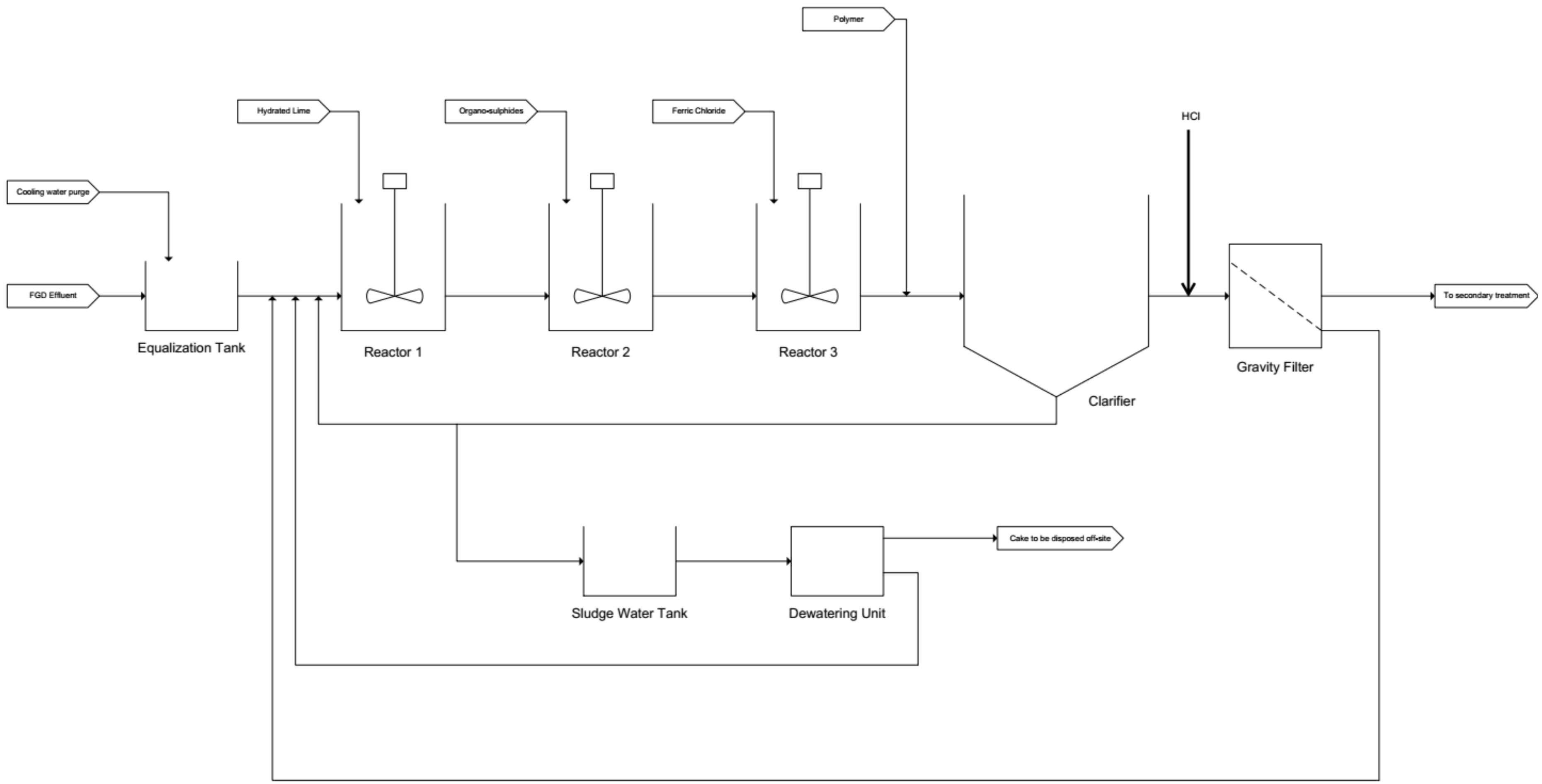


Figure 2: Pre-Treatment - Simplified Flow Diagram (Option 2)

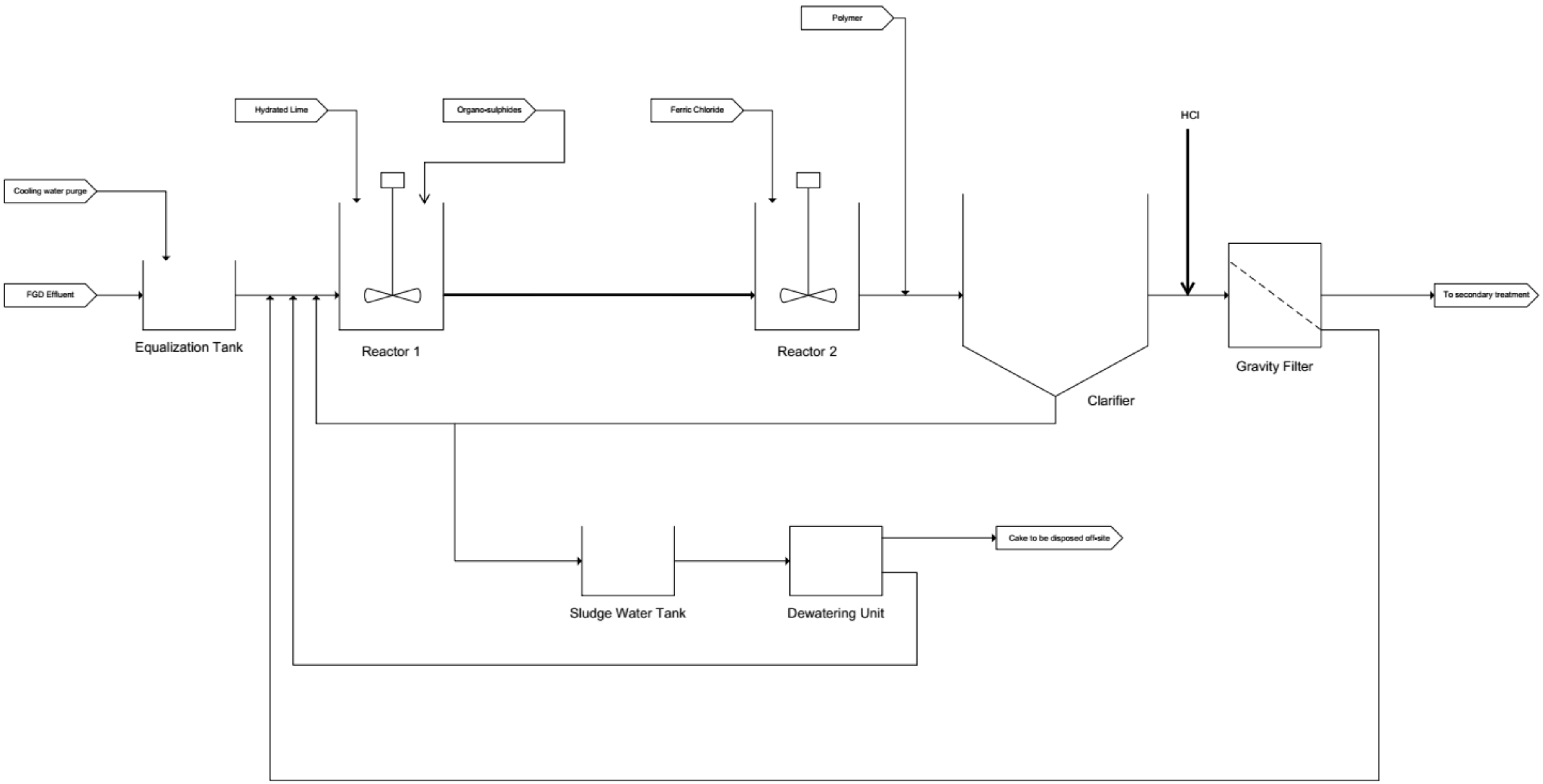


Figure 3: Pre-Treatment - Simplified Flow Diagram (Option 3)

Due to the high solids loading in the FGD purge stream (about 3.6% for Case 1 and 1.7% for case 2), as well as due to the fact that only a small saving will be achieved by eliminating Reactor 2, it was decided to use Option 1 described above for the pre-treatment. For a detailed description of this process, refer to section 7.1.

6 DESALINATION OPTIONS EVALUATION

The water from the pre-treatment section will still have a high TDS concentration that needs to be removed using desalination technology. For desalination, the following options were considered:

Option 1a: Reverse osmosis, followed by thermal evaporation and crystallisation of the brine to achieve a zero-liquid discharge.

Option 1b: Reverse osmosis, followed by freeze crystallisation of the brine to achieve a zero-liquid discharge.

Option 1c: Reverse osmosis, full brine stream is transported to a waste disposal facility.

Option 2a: Thermal evaporation and crystallisation of the full stream from the pre-treatment section.

Option 2b: Freeze crystallisation of the full stream from the pre-treatment section.

Option 3a: Forward osmosis, followed by thermal evaporation and crystallisation of the brine to achieve a zero-liquid discharge.

Option 3b: Forward osmosis, followed by freeze crystallisation of the brine to achieve a zero-liquid discharge.

After approaching some reverse osmosis suppliers with the given water qualities, the feedback received was that the TDS in the water is too high for reverse osmosis to be a feasible option. Although certain emerging advanced membrane technologies (e.g. VSEP and BKT FMX) claim to be able to treat waters with salinities in the range of the design basis for this project, these options were deemed to be unsuitable for this project. As a consequence, options 1a, 1b and 1c were eliminated.

Based on past experience and exposure to Forward Osmosis, it was concluded that forward osmosis (Option 3a and 3b) can also be ruled out for this project due to the following:

- Previous comparative studies have shown forward osmosis to be very expensive. Furthermore, preliminary evaluations conclude that this technology requires significant upstream softening, resulting in very high chemical costs.
- To our knowledge, there is only one full-scale installation of Forward Osmosis for FGD wastewater treatment in China (http://oasyswater.com/case-study-post/changxing/) and none in South Africa. It will therefore probably require extensive piloting, which is not an option for this project due to the tight time constraints.
- Difficulties might be experienced in obtaining local support for the technology, which will further increase the risk of using this technology.

Option 2a and 2b were selected for further evaluation as part of the concept design phase. These two options are described in more detail in the sections to follow. The pre-treatment process described earlier in the report was assumed to be applicable for both these options.

After developing concept designs for these two options (2a and 2b), they were evaluated in a trade-off study workshop to select the preferred option. The outcome of the trade-off study is also documented in this report.

It must be noted that there are some proprietary or patented technologies associated with specific vendors that could potentially be used. In order not to limit the solution to one specific vendor, these proprietary technologies were not included as options in the concept study. However, when the water treatment plant is put out on tender, it is recommended that tenderers be allowed to propose alternatives, which will open the door for these proprietary technologies to also be considered.

7 PROCESS DESCRIPTION OF CONCEPT DESIGNS

The WwTP process can be divided into two major sections:

- Pre-treatment and Desalination (two options were evaluated)
    - Option 1 Desalination: Thermal evaporation and crystallisation of the full stream from the pre-treatment section.
    - Option 2 Desalination: Freeze crystallisation of the full stream from the pre-treatment section. This option will require polishing treatment of the product water using UF and RO, as well as thermal evaporation and crystallisation of the brine stream to achieve a zero-liquid discharge solution.

Simplified block flow diagrams for the two options for the FGD waste water treatment plant are shown in Figure 4 and Figure 5 below. The detailed process flow diagrams of the common pre-treatment section, as well as the two desalination options, are given in Appendix B.

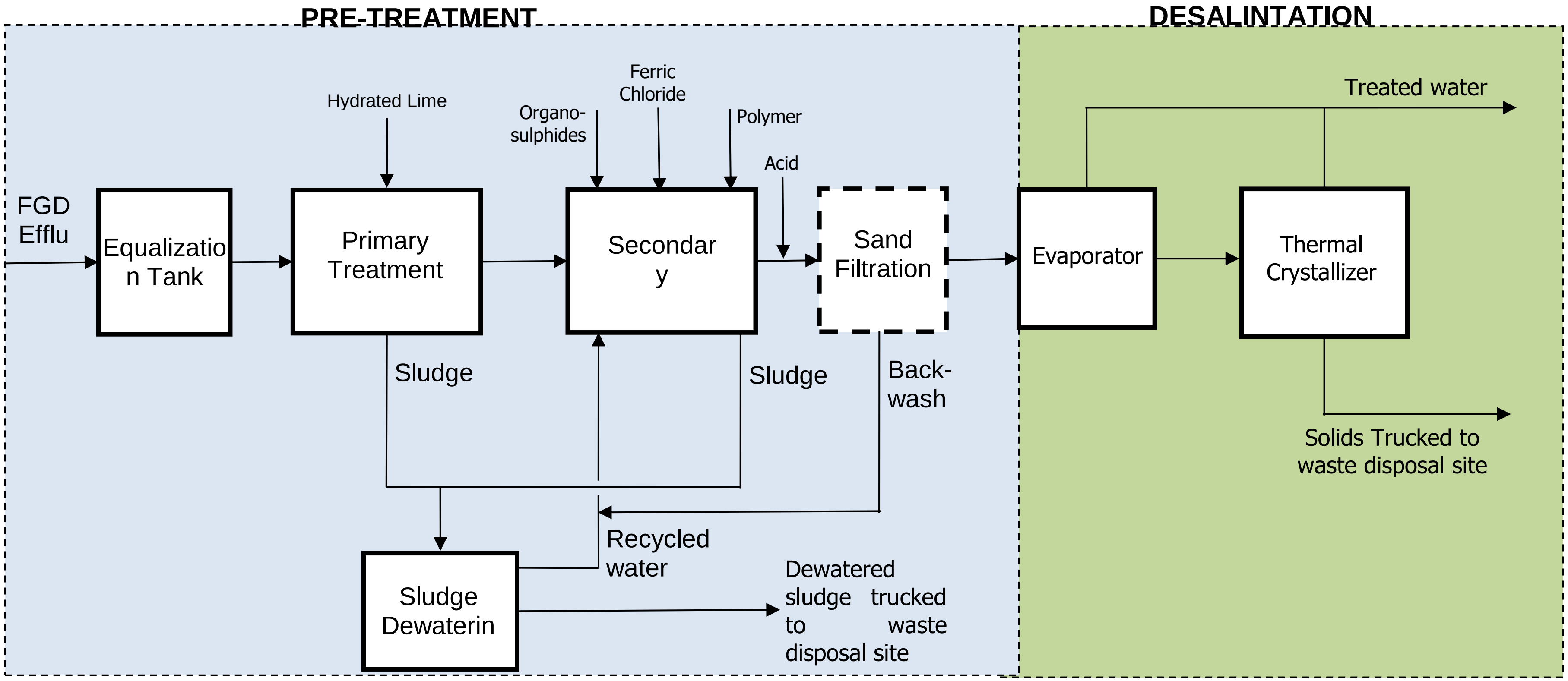


Figure 4 : Block flow diagram of Option 1

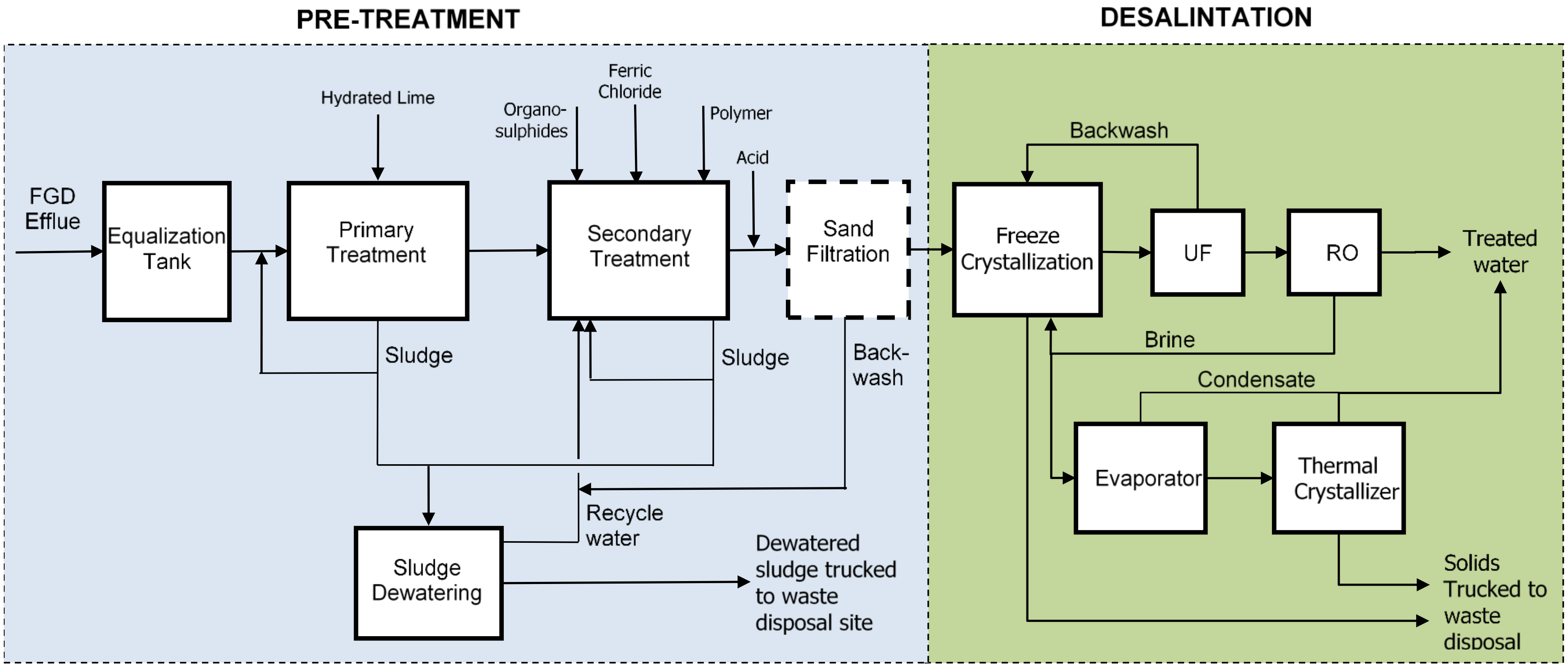


Figure 5 : Block flow diagram of Option 2

7.1 Pre-Treatment

7.1.1 Primary Treatment

The FGD effluent is fed to an equalisation tank which buffers flow and water quality variations. The equalisation tank is mixed using a motorised mechanical mixer to prevent the suspended solids from settling.

From the equalisation tank, the feed water flows under gravity to the first reaction tank (Reactor 1). A milk of lime solution is dosed to Reactor 1 using a lime dosing system (refer to Section 7.1.2 for details of the lime dosing system). The addition of an alkali (hydrated lime) is used to increase the pH of the equalization tank effluent to 9. The pH in Reactor 1 will be measured and the lime addition will be varied to control the pH. The Reactor is mixed using a motorised mechanical mixer.

Increasing the pH aids in the precipitation of metals and some heavy metals as metal hydroxides (metal solubility typically decreases with an increase in pH). Some water softening is also achieved through the precipitation of Ca and Mg as $CaCO_3$, $CaSO_4$ and $Mg(OH)_2$.

The effluent from Reactor 1 is directed to the Primary Clarifier, which removes the suspended solids from the stream. The overflow from the clarifier gravitates to the second reaction tank (assumed to contain less than 100 mg/L suspended solids). The sludge underflow is pumped back to Reactor 1 at a flow rate of equal to 100% of the feed flow to the plant. Recycling the underflow build up the solids concentration to about 10%. A purge stream is drawn off from the clarifier underflow and sent to the Thickener for further thickening.

The primary treatment is used to de-supersaturate and soften the water. This reduces scale formation in the downstream equipment, which increases the reliability and efficiency of the process.

7.1.2 Lime Dosing System

A brief process description for the quicklime dosing option is as follows:

Quicklime is delivered by bulk tankers and transferred into a quicklime silo, from where it is slaked with water in a detention-type slaker. The slaked lime is transferred using a slaker transfer pump to the lime slurry makeup tanks. Water is added to the slaked lime to dilute it to a 10% milk-of-lime solution, from where the milk-of-lime slurry is transferred to the dosing tank.

From the dosing tanks the lime slurry solution is dosed to Reactor 1 using a dosing pump. The dosage rate will be controlled based on the measured pH in Reactor 1. A pH of about 9 will be targeted.

7.1.3 Secondary Treatment

The overflow from the primary clarifier is directed to Reactor 2, where an organo-sulphide solution is dosed to further precipitate any heavy metals as metal sulphides. Reactor 2 is mixed using a motorised mechanical mixer. Reactor 2 overflows to Reactor 3.

To aid in flocculation of the precipitated metals, Ferric Chloride solution is dosed to Reactor 3. The iron salt helps to form denser flocs, which enhance the secondary clarifier performance. In addition, the iron salts also assist in co-precipitating remaining metals and some organic matter present in the feed.

Polymer is dosed to the effluent from Reactor 3 to aid with coagulation in the Secondary Clarifier. Since the suspended solids concentration in the feed to the Secondary Clarifier will be fairly low, a solid contact clarifier is used. The overflow from the Secondary Clarifier (assumed to contain less than 20 mg/L suspended solids) flows into the Sand Filter Feed Tank, from where it is pumped through a pressurised sand filter (refer to Section 4.1.5). The clarifier bottoms sludge (assumed to contain 1% solids) is recycled back to Reactor 2. A purge stream is send to the Thickener.

7.1.4 Sludge Thickening and Dewatering

The sludge purge streams from both the primary and secondary clarifiers are directed to the Sludge Thickener. The overflow from the thickener (assumed to contain less than 40 mg/L suspended solids) flows into the Recycle Water Tank. The thickened sludge from the bottom of the thickener is pumped to the Sludge Buffer Tank. The buffer tank is sized for 24 hours of storage to allow for maintenance time on the filter press. The sludge tank is equipped with a motorised mechanical mixer to keep the solids in suspension.

The effluent from the sludge buffer tank is directed to a dewatering unit. The dewatering unit consists of a plate-and-frame filter press. The dewatered sludge (assumed to contain 60% moisture) is sent to a sludge storage facility sized for storing 7 days of sludge. The dewatered sludge is trucked away for off-site disposed. The pressate water from the dewatering unit is directed to the Recycle Water Tank, from where it is pumped to Reactor 2.

7.1.5 Sand Filtration

The overflow from the secondary clarifier is cleaned further using pressurised sand filtration. There is a possibility that sand filtration will not be required depending on the requirements of the technology used for desalination.

The clarifier overflow is collected in the Sand Filter Feed Tank, from where it is pumped through multiple pressure filters. Acid is dosed upstream of the sand filters to neutralise the water. Backwashing of the filters is done one at a time using the filtrate from the other filters. The backwash water is sent to the Recycle Water tank for recycling back to Reactor 2. The filtrate is sent to the desalination process.

7.2 Desalination

7.2.1 Option 1: Thermal Evaporation and Crystallization

While there are various types and configuration of thermal evaporators, mechanical vapour compression (MVR) evaporators are typically used for FGD wastewater treatment with multiple existing full-scale installations. Hence MVR was selected for further evaluation in this project.

In thermal evaporation, heat is added to the high TDS concentrate to boil it. Steam is collected and condensed to form a purified distillate, whilst the concentrate that remains is further treated using crystallisation. Heat is added by mechanical compression of vapor. A combination of an evaporator, crystalliser and a filter press is typically used to achieve zero liquid discharge. Evaporators for the FGD wastewater application are often falling film type with or without a seeded slurry system. Crystallisers are typically forced circulation types.

In a falling film evaporator, the feed is pumped through a heat exchanger that raises the temperate of the feed water and cools the outflowing distillate/condensate. The heated feed is pumped to the evaporator sump, from where fluid is constantly pumped to the distribution box on top of tube bundle for heat transfer. As the concentrate flows down the tubes, it forms a thin film and a fraction of the flow evaporates. Calcium sulphate crystals form as the feed is concentrated. The seeded slurry provides precipitation nuclei and prevents scaling of the heat transfer tubes. The concreted fluids falls back into the sump and is recirculated. The vapour is passed through mist eliminators and directed to the vapour compressor, which compresses and heats the vapour. The heated vapour is transfer back to the evaporator where it exchanges heat with the recirculating hot concentrate and condenses on the outside heat exchanger tube. As the condensate flows down the exchanger tube, it transfers heat to the concentrate on the inside of the tube. This results in evaporation of the concentrate, and the evaporation cycle is sustained. The heat from the distillate is used to heat the incoming raw feed water as described earlier.

The following treatment components are typically included in a conventional thermal evaporation system:

Feed Tank: Adjust pH and neutralize bicarbonate alkalinity to enable preheating of the wastewater in plate heat exchangers.

Plate Heat Exchangers: To preheat the inlet wastewater with heat recovered from recovered distillate.

Deaerator: To remove dissolved carbon dioxide, dissolved oxygen, and non-condensable gases.

Brine Concentrator: Falling film evaporator for water evaporation.

Recirculation Pump: To recirculate brine and concentrate it to the desired concentration prior to discharge for further processing.

Mechanical Vapor Compressor: To compress the vapour formed and recycle the latent heat of vaporization.

Seed Crystal Addition and Recovery System: For addition of calcium-sulphate seed crystals. The dissolved calcium sulphate in FGD wastewater preferentially precipitates on the seed crystals rather than the brine concentrator tubes, thus reducing scaling.

In a forced-circulation crystalliser, concentrated brine from evaporator is pumped to an agitated crystalliser feed tank. From the tank, the brine is pumped through a shell and tube heat exchanger to a forced circulation crystalliser operating under vacuum. Brine is heated in the heat exchanger with heat recovered from vapor. The heated brine flashes as the pressure drops when entering the crystalliser body. Salt crystals form and crystallise in the concentrated brine (slurry) that collects in a sump at the bottom of the crystalliser body. The slurry is circulated and a portion is sent to solids handling system consisting of centrifuge or pressure filter, or is sent directly for solidification. The vapor collected from the crystalliser body is recompressed and introduced to the heat exchanger's shell side to provide thermal energy for continued evaporation.

Typical main components of crystalliser include:

- Feed Tank
- **Shell and tube heat Exchangers:** To preheat the inlet wastewater with heat recovered from recovered distillate.
- **Brine Concentrator:** Forced circulation evaporator for water evaporation.
- **Recirculation Pump:** To recirculate brine and concentrate it to the desired concentration prior to discharge for further processing.
- **Mechanical Vapor Compressor:** To compress the vapor formed and recycle the latent heat of vaporization.

7.2.2 Option 2: Freeze Crystallization

When water freezes, it generally forms ice crystals that are pure, leaving behind a more concentrated salt solution. The ice can be separated and allowed to melt to produce a product with low TDS. By removing the water in the form of ice, the remaining solution becomes supersaturated with the salt and crystals start to form. Since ice is less dense than water and brine, it floats to the surface, while the denser salt crystals settle to the bottom. The pure water (ice) and salt crystals can be separated according to density in a solids/solids separator.

Freeze crystallisation requires less energy compared to evaporative crystallisation, since the heat of fusion for ice is substantially less than the heat of evaporation. In addition, the temperature change required to freeze water is generally less compared to boiling it. However, various methods can be employed to improve the efficiency of both freeze crystallisation as well as thermal crystallisation, such as energy recovery through pre-heating the feed, etc.

A simplified flow schematic for the freeze crystallisation process is shown in the figure below.

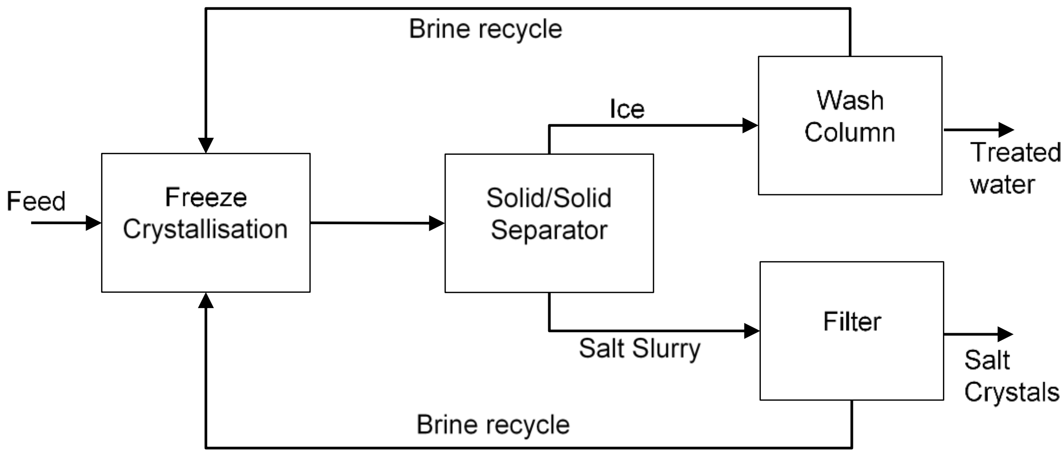


Figure 6 : Simplified block flow diagram of Freeze Crystallisation

8 PROCESS DESIGN

The process design for the two options is documented in this section.

8.1 Process Design Approach

A process design, as well as a detailed mass and component balance, was performed for both options evaluated, as well as for both feed water quality cases. A total of four design options were therefore developed as follows:

Option 1: Pre-treatment and thermal evaporation / crystallisation

- Case 1 feed water quality

- Case 2 feed water quality

Option 2: Pre-treatment and freeze crystallisation

- Case 1 feed water quality
- Case 2 feed water quality

The pre-treatment section is identical for Option 1 and 2. The sizing of the components does however differ for the Case 1 and Case 2 design feed water qualities

Dosing rates of chemicals, as well as sludge produced, were calculated for each of the four options as part of the mass balance.

Once the mass balance was fixed, the flows were used to size the equipment based on selected design criteria. The major infrastructure and equipment, as well as the design criteria used to size the various units, are given in the following sections.

8.2 Major Infrastructure and Electrical Equipment List

A summary of the major infrastructure is given in Table 3, including the design criteria used to size the infrastructure.

Note: Unless otherwise indicated, the volumes reported are the minimum required process volume and does not include dead zones or freeboard requirements.

The major electrical equipment is given in Table 4 below, including the design criteria used to size the equipment. The pumps are only preliminary sized based on an assumed required head, the exact sizes can only be determined once the required delivery head (including static head and losses in pipes) has been determined.

Different sizes for Case 1 and Case 2 feed water quality are given in the two tables. No details of the crystallisation processes are given; since these processes were treated as a black box (costing and footprint sizes were obtained directly from vendor). Additional equipment required for polishing treatment of the freeze crystallisation option is listed at the end of each table.

Table 3: Infrastructure of Case 1 and 2

Description	Type	Number (oper.)	Number (standby)	Number (total)	Case 1 Design size/unit	Case 2 Design size/unit	Units	Design criteria	Additional information - Case 1	Additional information - Case 2
Equalization Tank	Tank	1		1	360	360	m3	8 hours storage		
Reactor 1	Reactor	2		2	24	24	m3	30 minutes retention time		
Primary Clarifier	Central Drive with Rake Lift	1		1	8	9.5	m	0.5 m/h upflow rate	Side wall depth: 3 m Cone slope: 1/12 Sludge hopper volume: 1.2 m3	Side wall depth: 3 m Cone slope: 1/12 Sludge hopper volume: 1.0 m3
Reactor 2	Reactor	1		1	22	24	m3	15 minutes retention time		
Reactor 3	Reactor	1		1	22	24	m3	15 minutes retention time		
Secondary Clarifier	Solids Contact Clarifier	1		1	9	9.3	m	0.7 m/h upflow rate	Side wall depth: 3 m Cone slope: 1/12 Sludge hopper volume: 0.6 m3	Side wall depth: 3 m Cone slope: 1/12 Sludge hopper volume: 0.7 m3
Thickener	Central Drive Thickener	1		1	14	9.6	m	500 kg/(m2.d) solids loading rate	Side wall depth: 3 m Cone slope: 1/6 Sludge hopper volume: 0.6 m3	Side wall depth: 3 m Cone slope: 1/6 Sludge hopper volume: 0.4 m3
Sludge Buffer Tank	Tank	1		1	460	260	m3	24 hour storage		
Dewatering Press	Sludge Storage Facility	1		1	850	460	m3	7 days storage		
Sand filter feed tank	Tank	1		1	15	16	m3	20 minutes retention time		
Sand filter	Pressure Sand Filter	3	1	4	2.8	3	m3	10 m/hr filtration rate		
Recycle Water Tank	Tank	1		1	13	7	m3	20 minutes retention time		
Quick Lime Silo	Lime Silo	1		1	70	70	m3	7 days storage		
Lime Slaker Tank	Tank	2		2	28	28	m3	12 hour storage		
Lime Make-Up Tank	Tank	2		2	28	28	m3	12 hour storage		
Lime Dosing Tank	Tank	1		1	60	55	m3	12 hour storage		
Ferric Storage Tank	Drum	2		2	0.2	0.2	m3	7 days storage		
Polymer Make-Up and Curing Tank	Tank	1		1	0.4	0.5	m3	12 hour storage		
Polymer Dosing Tank	Tank	1		1	0.4	0.5	m3	12 hour storage		
Evaporator / Crystalliser	As per vendor information									
Polishing Treatment (only for Freeze Desalination)										
UF Feed Tank	Tank	1		1	17	18	m3		25 minutes retention time	
UF Filters		76		76						
UF Pressure Vessels		76		76						
UF Racks		2		2			m3			
UF CIP tank	Tank	1		1	0.4	0.4				
RO Filters		100		100						
RO Pressure Vessels		7		7						
RO Racks		1		1			m3			
RO Feed Tank	Tank	1		1	17	18	m3		25 minutes retention time	
Permeate flush tank	Tank	1		1	7.5	8	m3			
RO CIP tank	Tank	1		1	0.4	0.5	m3			
Brine Evaporator / Crystalliser	As per vendor information									

Table 4 : Electrical Equipment for Case Design 1 and 2

Description	Type	Number (operational)	Number (standby)	Number (total)	Case 1 Design size/unit	Case 2 Design size/unit	Units	Design criteria
Equalization Tank - Mixer	Rapid mixer	1		1	18.5	18.5	kW	45 W/m3
Equalization Tank - Pump	Centrifugal pump	1	1	2	2.2	2.2	kW	Assumed 10 m pump head and 70% efficiency. If gravity flow is possible, pump not required
Reactor 1 - Reactor Mixers	Rapid mixer	2		2	3	3	kW	120 W/m3
Primary Clarifier - Bridge motor	Bridge motor	1		1	4	4	kW	Assumed motor size
Primary Clarifier - Sludge recycle pump	Progressive cavity pump	1	1	2	2.2	2.2	kW	Assumed 10 m pump head and 70% efficiency.
Reactor 2 - Reactor Mixers	Rapid mixer	1		1	3	3	kW	120 W/m3
Reactor 3 - Reactor Mixers	Rapid mixer	1		1	3	3	kW	120 W/m3
Secondary Clarifier - Bridge motor	Bridge motor	1		1	4	4	kW	Assumed motor size
Secondary Clarifier - Sludge recycle pump	Progressive cavity pump	1	1	2	1.1	1.5	kW	Assumed 10 m pump head and 70% efficiency.
Stage 1 - Thickener - Bridge motor sizing	Bridge motor	1	1	2	7.5	5.5	kW	Assumed motor size
Stage 1 - Thickener - Waste sludge pump	Progressive cavity pump	1	1	2	5.5	3	kW	Assumed 10 m pump head and 70% efficiency.
Dewatering Press - Sludge Buffer Tank Mixer	Rapid mixer	1		1	45	30	kW	90 W/m3
Dewatering Press	Plate and Frame Filter	1	1	2	1.1	1.1	kW	Estimate
Dewatering Press - Sludge removal conveyor	Conveyor	1		1	0.75	0.37	kW	Estimate
Sand filter - Feed pumps	Centrifugal pump	1	1	2	TBD	TBD	kW	Still to be determined
Recycle Water Return Pump	Centrifugal pump	1		1	1.5	1.1	kW	Assumed 10 m pump head and 70% efficiency.
Dosing - H2SO4 - Dosing Pump	Peristaltic pump	1	1	2	0.18	0.18	kW	Assumed 5 m pump head and 70% efficiency.
Dosing - Lime Slaker Mixer	Rapid mixer	2		2	3	3	kW	Based on mixing intensity of 250 s^{-1}
Dosing - Slaked Lime Transfer Pump	Peristaltic pump	1	1	2	0.37	0.37	kW	Assumed 5 m pump head and 70% efficiency.
Dosing - Lime Slurry Mixer	Rapid mixer	2		2	3	3	kW	Based on mixing intensity of 250 s^{-1}
Dosing - Lime Slurry Transfer Pump	Peristaltic pump	1	1	2	0.37	0.37	kW	Assumed 5 m pump head and 70% efficiency.
Dosing - Lime Dosing Mixer	Rapid mixer	1		1	4	4	kW	Based on mixing intensity of 200 s^{-1}
Dosing - Lime Dosing Pump	Peristaltic pump	1	1	2	2.2	2.2	kW	Assumed 10 m pump head and 70% efficiency.
Dosing - Ferric - Mixer	Slow mixer	2		2	0.18	0.18	kW	Based on mixing intensity of 400 s^{-1}, minimum motor size
Dosing - Ferric - Dosing Pump	Peristaltic pump	1	1	2	0.18	0.18	kW	Assumed 10 m pump head and 70% efficiency, minimum motor size
Dosing - Polymer Make-Up Mixer	Mixer	1		1	0.18	0.18	kW	Based on mixing intensity of 25 s^{-1}, minimum motor size
Dosing - Polymer Transfer Pump	Peristaltic pump	1	1	2	0.18	0.18	kW	Assumed 5 m pump head and 70% efficiency.
Dosing - Polymer Dosing Mixer	Mixer	1		1	0.18	0.18	kW	Based on mixing intensity of 25 s^{-1}, minimum motor size
Dosing - Polymer Dosing Pump	Peristaltic pump	1	1	2	0.18	0.18	kW	Assumed 10 m pump head and 70% efficiency.
Dosing - Carrier Water Booster Pump	Centrifugal pump	1	1	2	0.18	0.18	kW	Assumed 10 m pump head and 70% efficiency.
Polishing Treatment (only for Freeze Desalination)								
UF Feed Pump	Centrifugal pump	1	1	2	5.5	5.5	kW	Assumed 30 m pump head and 70% efficiency.
UF Feed Backwash Pump	Centrifugal pump	1	1	2	55	55	kW	Assumed 32 m pump head and 70% efficiency.
UF Air Scour Blower	Blower	1		1	11	11	kW	
UF CIP pump	Peristaltic pump	1	1	2	0.75	0.75	kW	
UF CIP Tank Mixer	Rapid mixer	1		1	0.18	0.18	kW	50 W/m3
RO feed pump	Centrifugal pump	1	1	2	75	75	kW	Assumed 375 m pump head and 70% efficiency.
RO CIP pump	Centrifugal pump	1	1	2	2.2	2.2	kW	
RO CIP Tank Mixer	Rapid mixer	1		1	0.18	0.18	kW	50 W/m3

8.3 Chemical Consumption

The major chemicals that will be dosed, as well as the average chemical usage and the basis of calculation, are listed in Table 5 below. All the chemicals listed below are for the pre-treatment section, hence there is no distinction between Option 1 and Option 2.

Table 5: Dosing Chemicals

Chemical	Dosing calculation	Case 1	Case 2
Quick Lime (90% purity) dosed to Reactor 1	Target pH in the reactors = 9	9166 kg/d	8755 kg/d
Organo-sulphide	Based on vendor dosage rate	TBD (1)	TBD (1)
Ferric chloride	Assumed 10 mg/L dosing rate	19.3 kg/d	21.6 kg/d
Polymer	Assumed 2 mg/L dosing rate	3.9 kg/d	4.3 kg/d
Sulphuric Acid (98% w/w) dosed to pH correction tank	Target pH = 6.5	32.6 L/d	7.5 L/d

Note 1: The amount of organo-sulphide to be dosed needs to be informed by the vendor of the chosen organo-sulphide.

8.4 Waste Produced

An estimate of the waste quantities that will be produced for the two options and the two feed water cases are given in Table 6.

Table 6: Volume of waste produced

Waste Stream	Units	Option 1		Option 2	
		Case 1	Case 2	Case 1	Case 2
Dewatered Sludge Cake	kg/h	6708	3587	6708	3587
	m^3/h	5.0	2.7	5.0	2.7
Salt crystals	kg/h	3298	3168	8573	6046

9 WASTE HANDLING AND STORAGE FACILITY

The Waste Handling and Storage Facility (WHSF) has been designed to contain a Type 1 waste for a 7-day period. The facility will consist of an impermeable concrete surface bed with rear guard waterstops installed at the joints to render the surface watertight. The footprint of the impermeable concrete surface bed where the waste will be handled and stored is 2,370m^2. This is assuming that the waste is stored at a height of 0.6m. The perimeter of the facility will have 2m high reinforced concrete walls. A suspended slab is located within the building to support the plate-and-frame filter presses and the centrifuges. The slab contains openings under the various dewatering equipment to allow for the waste to be dropped onto the ground floor.

The initial option considered during the concept design entailed discharging waste from the dewatering units via openings in the slab onto the concrete surface bed on the level below. The surface bed will be designed with the appropriate concrete cover to accommodate severe conditions as per SANS 10100 – Part 2. The slab is located at the northern end of the building to enable maximum use of the ground floor footprint for the 7-day storage. This requires the use of a front-end loader to collect the waste from the northern end of the building and store it in rows at the entrance of the building. The front-end loader is also required for loading of the waste collection trucks. Additional considerations for this option include clearance heights for the front-end loader and for the crawl beams required for maintenance of the equipment. The current clearance to the beam supporting the suspended slab is 5.5m to enable full extension of the boom of the front-end loader (refer to Drawing 17041-73-16-101 S2). The height from the top of slab to the roof truss is approximately 4.3m. This height takes into consideration hand railings on the slab, height of the equipment and clearance from the roof truss and crawl beams.

Following Zitholele's trip to the USA to investigate FGD WwTPs another option that was previously assessed has been proposed again. The option entails the use of skips on rail systems, also located within the WHSF. For this option it will be desirable to locate the suspended slab with equipment away from the northern end of the building. This will have to be positioned strategically to allow storage of empty skips from the far end of the building before it reaches the dewatering equipment whilst also allowing sufficient 7-day storage space for loaded skips at the front of the building before collection.

Advantages of this option include the following:

- The prevention of double handling of the waste as skips can be loaded directly onto trucks for collection;
- Omission of the front-end loader will reduce clearance heights, consequently reducing the overall height of the building;
- The use of skips for storage will increase the amount of waste stored per m^2 in comparison to direct storage on the surface bed. This will reduce the required footprint to accommodate the 7-day storage which can reduce the overall building footprint;
- Containment of the waste by direct discharge into the skips also provide a cleaner operation within the WHSF and minimise the spillage of waste during operations at the entrance of the

WHSF. This will result in less washing of the area and a reduction in the generation of dirty water.

For both of the abovementioned options, a biodegradable lining system will be required in the truck or on the skip in which the waste will be discharged. This is due to the sticky consistency of the waste being handled and transported away. Additional options such as spray on lining systems are currently being investigated for use in a front-end loader, skip and/or truck. The two wastes may be mixed during operations as they will be transported together to the hazardous waste disposal site. A layout showing where the waste will be stored and handled is provided on Drawing 17041-73-16-101 S2).

Due to the various advantages of utilising a skip as mentioned above, this has been selected as the go forward option that will be developed further during the Basic Design Stage.

A structural steel roof cladded with IBR sheeting will be used to prevent rainfall from falling directly onto the surface bed. Transparent IBR panels will also be recommended on the roof and side cladding to enable natural lighting into the building. The choice of structural steel for the roof was due to the faster construction times as well as lighter weight of steel in comparison to other construction materials. This in turn has an effect on the transportation costs and can simplify the design of the building's foundation and other structural support systems. The building will be open on eastern and western sides to allow access for maintenance vehicles and a front-end loader. It will also ensure adequate ventilation inside the building.

The WHSF has been designed in terms of *GN 926 Norms and Standards for the Storage of Waste.* The following aspects have been incorporated into the design:

- An impermeable concrete surface bed where the waste will be stored and handled;
- All tanks used to store liquid waste will be contained in bunded areas that have impermeable floors and a capacity of at least 110% of the total contents of the liquid stored;
- Areas where spills may occur contain a sump that drains via a dirty water system into a dirty water holding sump;
- A stormwater interception channel has also been provided at the entrance of the WHSF that will divert contaminated run off via the dirty water system into a dirty water holding sump;
- The WHSF contains access gates to prevent unauthorised entry; and
- A perimeter fence will be provided around the facility with adequate signage. The signs will indicate the risks involved with entering the site, hours of operation, the name, address, telephone number and person responsible for the operation of the facility.

A layout of the WHSF is provided on Drawing 17041-73-15-101. Figure 7 and Figure 8 shows the 3D illustration of the WHSF and a typical detail of the impermeable concrete surface bed at the WHSF.

The design of reinforced concrete and structural steel elements for the WHSF will be done in accordance with the relevant SANS codes. These include but are not limited to the following:

- SANS 10160: The general procedures and loadings to be adopted in the design of buildings;
- SANS 10100-1: The structural use of concrete – Part 1: Design;
- SANS 10100-2: The structural use of concrete – Part 2: Materials and execution of work;
- SANS 10162-1: The structural use of steel – Part 1: Limit-state design of hot-rolled steelwork.

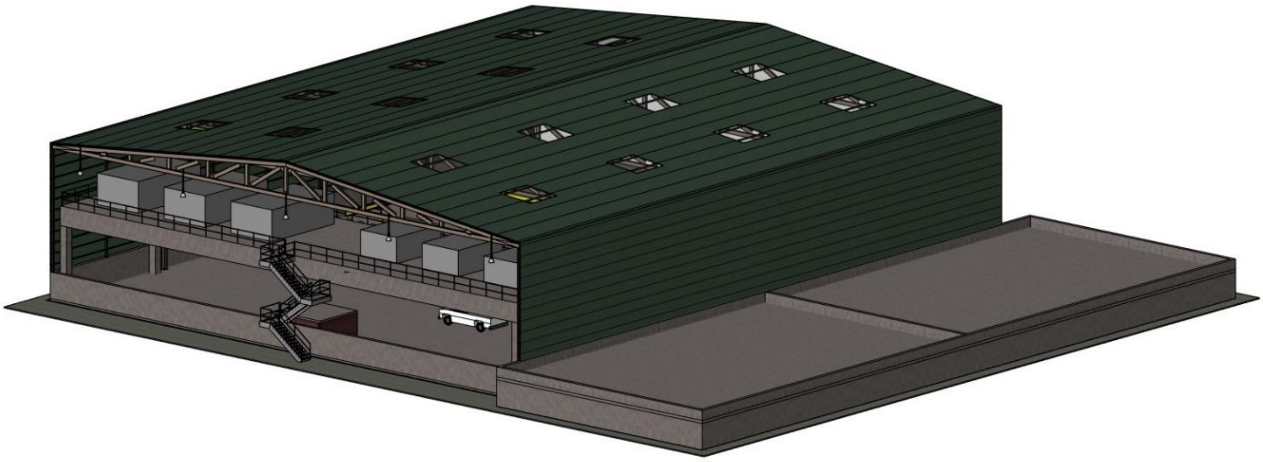


Figure 7: 3D illustration of WHSF

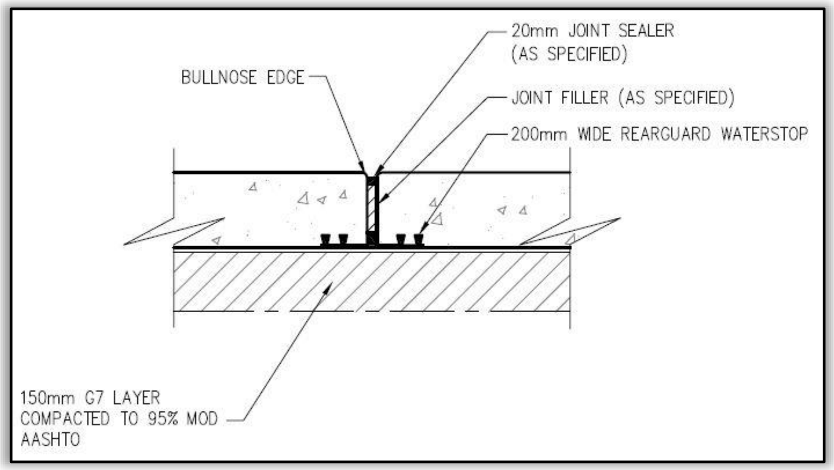


Figure 8: Typical detail of impermeable concrete surface bed at the WHSF

9.1.1 Waste handling and conveyance to hazardous waste disposal site

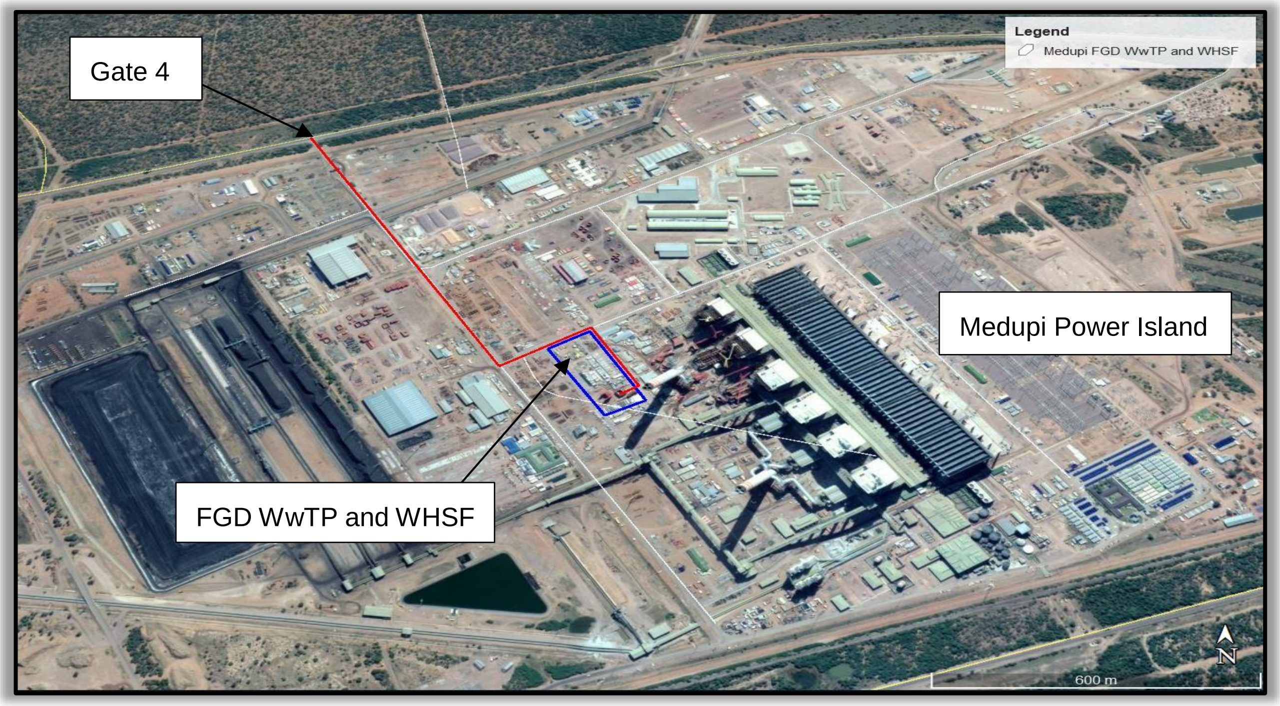
9.1.1.1 Overview

Waste from the WHSF will be transported to a hazardous waste disposal site on a daily basis. This will involve a continuous trucking operation. Information has been sourced from a hazardous waste disposal site which has informed operations philosophy described in the next Section.

9.1.1.2 Truck operation

The average number of trucks required to transport waste is based on the working hours at a typical hazardous waste disposal site which are between 6h00 and 22h00. A truck with a legal payload of 33t was used to calculate the number of trucks required to transport waste from the WHSF to the hazardous waste disposal site. Figure 9 shows the route that trucks will use when entering the WHSF.

Figure 9: Route for trucks to access the FGD WwTP and the WHSF



Sludge

Sludge will be discharged from the plate and frame presses at the dewatering section of the WHSF directly into a skip which will be situated a level below. Trucks will enter the WHSF and pick up the skips. Thereafter, they will exit the Power Station using Gate 4 and drive back to the hazardous waste disposal site. The daily amount of trucks required to transport sludge from the WHSF to hazardous waste disposal site is provided in Table 7.

Table 7: Daily amount of trucks required to transport sludge from the WHSF to hazardous waste disposal site

Scenario	Number of trucks required
Sludge - Case 1	5
Sludge - Case 2	3

Salts

The salts will be discharged from the dewatering section of the WHSF directly into a skip which will be situated a level below. Trucks will enter the WHSF and pick up the skips. Thereafter, they will exit the Power Station using Gate 4 and drive back to the hazardous waste disposal site. The daily amount of trucks required to transport salts from the WHSF to the hazardous waste disposal site is provided in Table 8.

Table 8: Daily amount of trucks required to transport salts from the WHSF to the hazardous waste disposal site

Scenario	Number of trucks required
Salts - Case 1	3
Salts - Case 2	3

10 STORMWATER MANAGEMENT

10.1 Stormwater management philosophy

Government Notice 704 (GN 704) Regulations on use of mining and related activities aimed at the protection of water resources relates to mining and not directly to coal fired power stations or industries in general. However, due to a lack of stormwater management legislation relating directly to coal fired power stations, GN 704 is used in terms of Best Practice. The clauses from Government Notice 704 (GN 704) and Government Notice 926 (GN 926) *National Norms and Standards for the Storage of Waste*, listed below, are applicable to the stormwater management philosophy for the Medupi FGD WwTP and its associated infrastructure.

GN 704 Clause 1 defines 'activity' as:

a) any mining related process on the mine including the operation of washing plants, mineral processing facilities, mineral refineries and extraction plants, and
b) the operation and the use of mineral loading and off-loading zones, transport facilities and mineral storage yards, whether situated at the mine or not,
(i) in which any substance is stockpiled, stored, accumulated or transported for use in such process; or
(ii) out of which process any residue is derived, stored, stockpiled, accumulated, dumped, disposed of or transported;

GN 704 Clause 6 stipulates the capacity requirements of clean and dirty systems as follows:

Every person in control of a mine or activity must-

(a) Confine any unpolluted water to a clean water system, away from any dirty area;

(b) design, construct, maintain and operate any clean water system at the mine or activity so that it is not likely to spill into any dirty water system more than once in 50 years;

(c) collect the water arising within any dirty area, including water seeping from mining operations, outcrops or any other activity, into a dirty water system;

(d) design, construct, maintain and operate any dirty water system at the mine or activity so that it is not likely to spill into any clean water system more than once in 50 years; and

(e) design, construct, maintain and operate any dam or tailings dam that forms part of a dirty water system to have a minimum freeboard of 0.8 metres above full supply level, unless otherwise specified in terms of Chapter 12 of the Act.

(f) design, construct and maintain all water systems in such a manner as to guarantee the serviceability of such conveyances for flows up to and including those arising as a result of the maximum flood with an average period of recurrence of once in 50 years.

GN 926 provides the Norms and Standards for the Storage of Waste, of which the following clauses were applied to the stormwater management philosophy:

Clause 7(5): A waste storage facility must be constructed to maintain on a continuous basis a drainage and containment system capable of collecting and storing all runoff water arising from the storage facility in the event of a flood. The system must under the said rainfall event, maintain a freeboard of half a meter.

In light of the above, the stormwater management plan outlined in Section 10.2 takes cognisance of all the relevant requirements in GN 704 and GN 926.

10.2 Stormwater management plan

The stormwater management design for the WHSF includes a clean and dirty water system as per GN 704. The two systems have been separated to prevent contamination of clean stormwater runoff and to contain dirty water.

The WHSF is a roofed facility that prevents clean stormwater runoff from being contaminated. The dirty footprint is limited to the area at the entrance of the WHSF and the bunded areas where the pumps are contained. Catchment 2 on Dwg. 17041-73-02-105 shows the dirty area at the entrance of the WHSF that will enable runoff to gravitate into the plant drain. The dirty stormwater from the bunded areas will be pumped using a mobile pump and drained via a lay flat pipe into the plant drain channel that connects to the plant drain. Details of the plant drain are provided in Section 10.3.6. The FGD WwTP dirty water system will not tie into the Medupi P.S. existing dirty water system. All other areas on site such as the terrace of the pre-treatment facility and Administration Building footprint have been classified as clean areas therefore, the runoff generated from those areas will flow into the Medupi P.S. existing clean stormwater system. The Medupi FGD WwTP terrace will be graded to ensure that all clean stormwater runoff flows into the Medupi P.S. existing clean stormwater system by means of a network of 450mm diameter pipes. The stormwater layout is provided on Drawing 17041-73-02-105.

10.3 Stormwater management infrastructure

The stormwater management system for the Medupi FGD WwTP contains the following infrastructure:

- Plant drain;
- Plant drain channel;
- Collection sumps in bunded areas ;
- Dirty water cut-off drain at the entrance of the WHSF;
- Oil and grease separator;
- Clean stormwater drainage inlets and piping network at the pre-treatment area; and
- Tie-in points to existing clean stormwater drainage system.

A description of the stormwater management philosophy on site is provided below:

10.3.1 Pre-treatment area

The pre-treatment area is classified as a clean area, with the exception of the bunded areas where the forwarding pumps are located. The bunded area is provided to contain any spills. The pre-treatment area is uncovered; therefore, rainwater which falls within the pump bunds is drained to a sump within the bund and emptied by means of a mobile pump and lay flat pipe into the plant drain channel. Any discharge as a result of pump maintenance or leakage is contained within the bund will also be removed with a mobile pump. The sumps will be monitored on a daily basis and cleaned out whenever they contain water.

10.3.2 Paved area surrounding WwTP and WHSF

Clean run off is generated from the pre-treatment area and the area surrounding the WHSF. These areas are shown as Catchment 1 & 3 on Drawing 17041-73-02-105. The clean run off will flow via the paving which is sloped towards stormwater inlets which connect to the clean stormwater tie-in points along roads 9 and 10. All dirty stormwater from the area in front of the WHSF as defined by Catchment 2 on Drawing 17041-73-02-105 reports to the plant drain. A description of the plant drain is provided in Section 10.3.6.

10.3.3 Wheel wash bay

A wheel wash facility has been provided for the trucks which will be transporting salt and sludge to the hazardous waste disposal facility. The dirty water from the wheel wash will flow into a small collection sump and via a pipe to the oil and grease separator (see Drawing 17041-73-18-101-S1) and then into the plant drain.

10.3.4 WHSF

The WHSF is covered by a roof. The stormwater runoff from the WHSF roof is drained via gutters and downpipes to the existing clean stormwater system.

Any surface runoff flowing towards the WHSF will be intercepted by a cut-off drain at the entrance of the WHSF and gravitate via a pipe into the plant drain (refer to Drawing 17041-73-02-105).

10.3.5 Administration building

Stormwater runoff in this area is classified as clean. The paving is sloped to direct clean runoff into clean stormwater inlets. The clean inlets will be tied into the existing clean stormwater system on Road 10.

Runoff from the administration building and carport roofs is drained by means of gutters and downpipes to the clean stormwater inlets.

10.3.6 Plant drain

The dirty stormwater system is designed as an isolated system which does not connect to the Medupi Power Station system. The plant drain is utilised to collect dirty runoff from the area in front of the WHSF (Catchment 2) as shown on Drawing 17041-73-02-105.

An estimated volume of 2940 litres of potable water per day will be used at the wheel wash bay. The wash water runoff will collect at a localised low point in a collection sump and gravitate via a pipe into an oil and grease separator (refer to Drawing 17041-73-17-101-S1) and then the plant drain.

In order to size the plant drain the following scenarios were assessed:

1. The volume to be contained for the 1:50 year storm event such that the plant drain does not overflow.
2. The volume required to empty the largest water retaining structure such that the plant drain does not overflow.

A water balance for the 1:50 year storm event was developed using historical rainfall data to determine the volume of the plant drain for Scenario 1. A volume of $405m^3$ is sufficient to ensure that the plant drain will not spill during a 50 year period.

A graphical representation of the results from the water balance model is provided below:

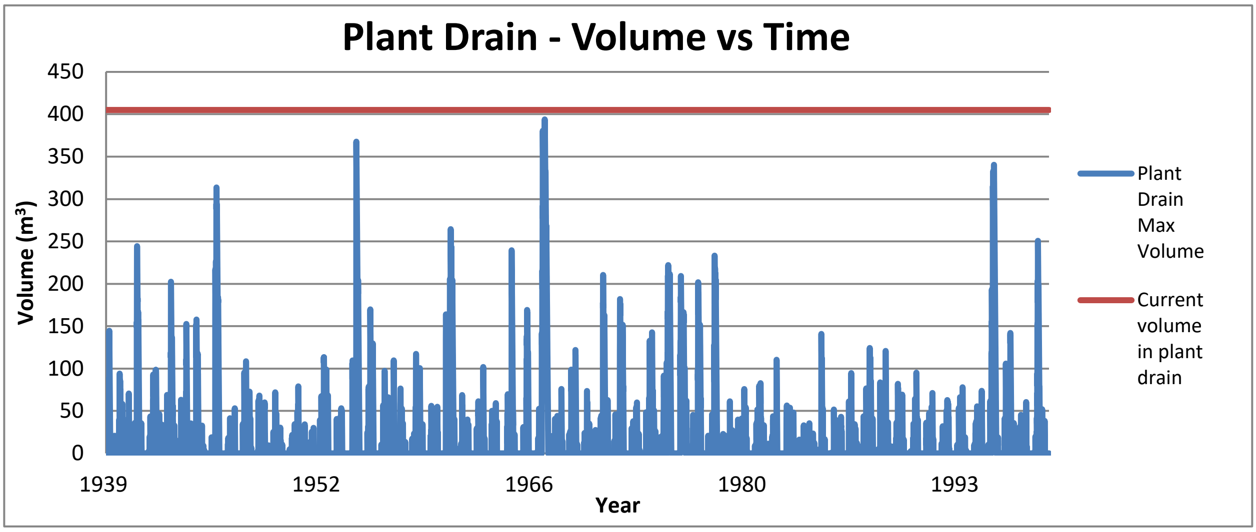


Figure 10: Graphical representation of water balance

Sizing of the plant drain for Scenario 2 considered the largest water retaining structure that has to be drained during maintenance. The largest water retaining structure is the Thickener with a volume of 525m^3.

The plant drain was sized using the larger volume from the two scenarios which was 525m^3 for Scenario 2.

<u>Plant drain operating philosophy</u>

Although the plant drain has been designed to allow draining of the water retaining structures, this should only be done one structure at a time and during the dry season. Each structure should be drained slowly and the level in the plant drain channel which flows to the plant drain should be monitored constantly as the structure is being drained. The plant drain channel is a 500mm x 500mm rectangular covered channel. A typical detail of the plant drain channel is shown on Dwg. 17041-73-17-101.

A level sensor will be fitted in the plant drain to determine the water level. As soon as the plant drain exceeds the minimum water level, the pumps will commence pumping from the plant drain to Reactor 1 (in both process trains) at a maximum flow rate of 8m^3/day. The plant drain is sized to ensure that no spillages will occur if water is pumped out as per the operating philosophy.

10.3.7 Stormwater Management Options

The following stormwater management options were considered to ensure that all the surface runoff generated from the development of the WwTP within Catchment 5 can be accommodated in the existing clean stormwater network:

1. <u>Tie in to Road 10 clean inlets</u>

This option was the ideal solution with respect to shortest drainage path; however the stormwater runoff from catchments 5d, 5e and 5f exceed the maximum capacity of inlets C27.3, C27.4 and C27.5 respectively. In order to achieve the stormwater runoff capacity requirements, an upgrade to the Road 10 network will be required. This is not favourable due to the additional construction costs that will be incurred when replacing the existing Medupi P.S network. This option is also likely to have downstream impacts on the existing system.

2. Attenuation of Flow

Attenuation can be considered in the event that the required Stormwater runoff capacities exceed the existing stormwater network capacity and the existing stormwater network capacity cannot be upgraded. Attenuation is not favourable due the reasons below:

- Water management of the attenuated flow may incur additional environmental requirements and permitting.
- Space constraints as the sizable footprint required and drainage slopes for an attenuation pond are note easily accommodated on the current footprint.
- The construction of an attenuation pond will result in additional costs.

3. Split the runoff between Road 10 clean inlets and Road 9 clean inlets

This option is the preferred solution with respect to stormwater runoff capacity requirements. Splitting the runoff between Road 9 & 10 reduces flow to both lines. The Road 9 stormwater network, C22.1 to C22.6 has available capacity due to minimal development in catchment 1, which was previously earmarked for the WHSF. To implement this option, the paving at the FGD WwTP will need to be sloped to direct flow to both the Road 9 and Road 10 stormwater network. This option has less construction cost impacts when compared to option 1; however the existing Medupi P.S. terrace levels will need to be revised to suit the new drainage requirements. Table 9 shows the stormwater flows based on splitting the runoff to flow into Road 9 and 10.

10.4 Stormwater Runoff Modelling

The stormwater design at Medupi Power Station conducted by Gibb was used as a reference for the stormwater infrastructure surrounding the WwTP terrace. Figure 11 depicts the catchments, annotated as per the existing design, and sub-catchments which were considered in establishing suitable stormwater routing and connection points. All clean runoff generated from the FGD WwTP terrace will be directed into the existing stormwater infrastructure on Road 9 and 10. The FGD WwTP terrace will be graded to allow clean runoff from 30% and 70% of the terrace to drain via the stormwater infrastructure on Road 9 and 10 respectively.

The capacities of the existing inlets were compared to the run-off flows generated from catchments 1 and 5. The comparison provided in Table 9 shows the stormwater system is adequate provided that the terrace is graded to allow the clean runoff to flow into the stormwater system on Road 9 and 10. The clean inlets identified along roads 9 and 10 are all kerb inlets; therefore minor modifications will be required to tie in the new stormwater infrastructure from the FGD WwTP to the existing stormwater infrastructure on Road 9 and 10.

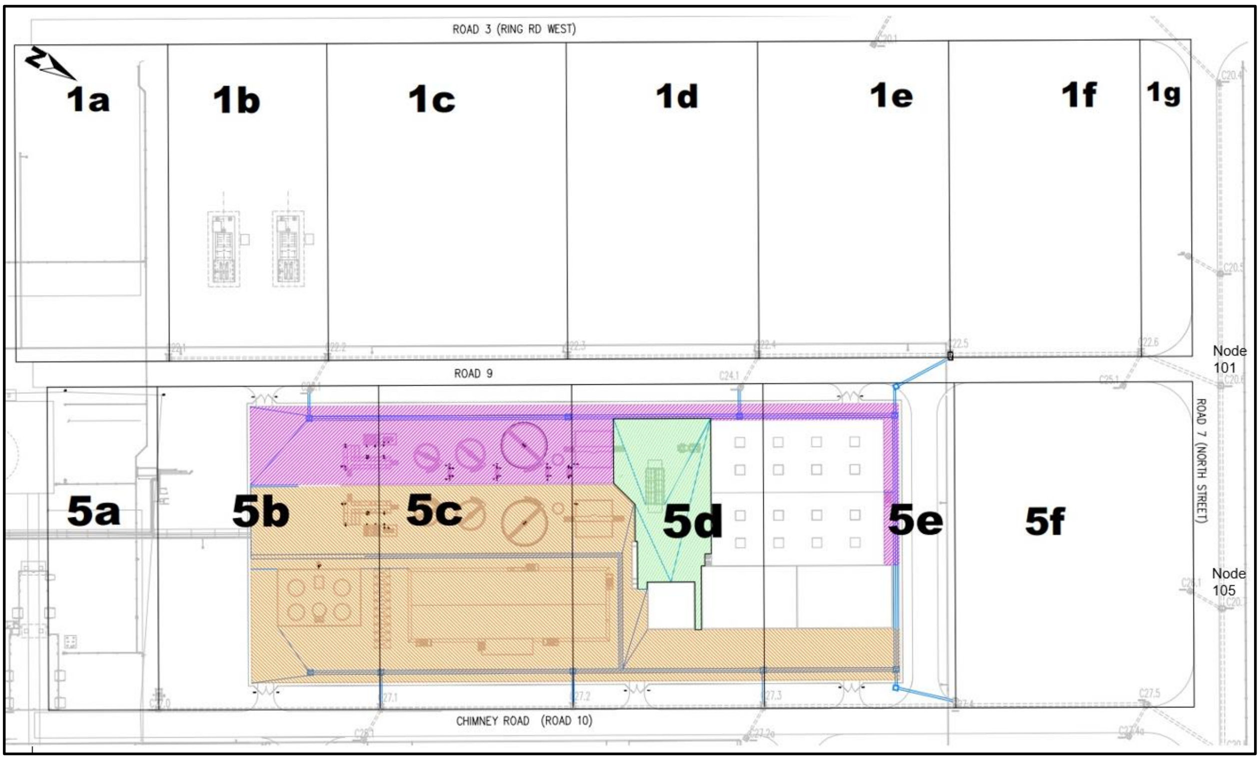


Figure 11: Layout Indicating Sub-Catchment Areas and Existing Stormwater Inlets

The runoff generated from the 1 in 50 year storm was calculated as shown in Table 9.

Table 9: Table of Stormwater Flows into Existing Inlets

Catchment	**Sub-Catchment**	**Runoff (m^3/s)**	**Cumulative Runoff (m^3/s)**	**Stormwater Inlet**	**Inlet Design Capacity (m^3/s)**
1	1a	0.048	0.048	C22.1	0.480
	1b	0.045	0.156	C22.2	0.480
	1c	0.066	0.222	C22.3	0.630
	1d	0.054	0.528	C22.4	0.828
	5c & 5d	0.189			
	1e	0.054	0.581	C22.5	1.061
	1f	0.054	0.887	C22.6	1.311
	5e & 5f	0.189			
	1g	0.014	0.014	C21.1	0.480
Road 3		0.066	0.067	C20.1	0.480

Catchment	Sub-Catchment	Runoff (m^3/s)	Cumulative Runoff (m^3/s)	Stormwater Inlet	Inlet Design Capacity (m^3/s)
		0.066	0.133	C20.2	0.480
		0.066	0.200	C20.3	0.480
Road 9		0.063	0.063	C23.1	0.480
		0.063	0.126	C24.1	0.480
		0.063	0.189	C25.1	0.480
Road 7		0.02	0.219	C20.4	0.480
		0.02	0.239	C20.5	0.627
		0.02	0.936	C20.6	1.311
		0.02	0.956	C20.7	1.930
		0.02	1.837	C20.8	2.756
5	5a	0.052	0.052	C27.0	0.48
	5b	0.057	0.172	C27.1	0.48
	5c	0.219	0.391	C27.2	0.63
	5d	0.158	0.612	C27.3	0.828
	5e	0.178	0.789	C27.4	1.061
	5f	0.038	0.038	C26.1	0.480
Road 10		0.063	0.063	C28.1	0.480
		0.063	0.126	C27.2a	0.480
		0.063	0.189	C27.4a	0.480

11 SITE SERVICES

The Admin Building at the FGD WwTP will contain a potable water and sewer reticulation that will be connected to the existing water supply and sewer system. Electricity for the building will be supplied by the existing electrical supply on site.

12 TRADE OFF WORKSHOP

A trade-off workshop was held on the 8th February 2018 and attended by Zitholele, Eskom Engineering and Eskom Environmental stakeholders. The workshop was utilised to evaluate the shortlisted process technologies for Case 1 and 2 water qualities. The criteria for the trade-off workshop were developed by Zitholele and Eskom's Process Engineers. Prior to the trade off workshop Zitholele populated the trade-off matrix as a basis for discussions. During the workshop, robust discussions were held and scoring of the various criteria was rigorously interrogated until the project team were satisfied that the scoring was representative of the technology being evaluated. The criteria that were evaluated during the workshop have been defined in Table 10.

Table 10: Description of trade off criteria

Theme	Criteria	Description
Environmental and Social	Site footprint	The area of the footprint for the WwTP and the waste handling facility – based on calculations
	Volume of waste	The total volume of waste produced by the process technology – based on calculations
	Type of waste	The Type of waste as per the waste assessment
Health and safety of people	Exposure of operating and maintenance staff	The potential harmful exposure of the technology on the operating and maintenance staff
	Inherent Safe Design	Safety risks associated with a particular technology
Financial	Life cycle cost analysis	Life cycle cost analysis of the technology and the WHSF – based on calculations
	Capital cost	Capital cost analysis of the technology and the WHSF – based on calculations
Constructability	Project execution schedule and time	The duration of construction for the process technology
	Ease of construction	The ease of construction particularly experience of other plants constructed globally
Operability	Flexibility of operation	The impact of variations in feedwater volumes and qualities
	Reliable achievement of the product flow and quality	The ability to reliably achieve the product flow and water quality on a continual basis
	Ease of operation	The ease of operating the process technology
Maintainability	Ease of cleaning/ maintenance and access	Easy access during cleaning and maintenance of the plant

Theme	Criteria	Description
	Plant availability	The availability of the plant locally
	Local availability of spares to support the plant	The availability of spares locally (i.e. Proximity to Lephalale)
	Maintainability	Maintainability during operations including local support for special maintenance activities
Utility Consumption	Energy	The amount of electricity and steam required to operate the process technology – calculated
	Chemicals	The amount of chemicals required to operate the process technology – calculated
	Cooling water	The amount of cooling water required to operate the process technology – calculated

Following evaluation of the two options, the thermal evaporation technology (Option 1) was ranked higher than the freeze crystallization technology (Option 2) for both Case 1 and 2.

13 PREFERRED TECHNOLOGY

Following the evaluation of the various options during the trade off workshop the technology with the best score for both Case 1 and 2 was Option 1 - Thermal Evaporation technology. A general arrangement of the site is illustrated on Drawing 17041-73-02-101-S1.

14 CONCLUSION

Since the Thermal Evaporation technology scored the highest during the trade-off workshop it will be developed further during the next design phase.

Appendix A : Process Flow Diagrams

Appendix B : Drawings