



Appendix Z

Tailings Management Engineering Design

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The EIS has been prepared by, for and on behalf of Wafi Mining Limited and Newcrest PNG 2 Limited (together the "**WGJV Participants**"), being the participants in the Wafi-Golpu Joint Venture ("**WGJV**") and the registered holders of exploration licences EL 440 and EL1105, for the sole purpose of an application (the "**Permit Application**") by them for environmental approval under the Environment Act 2000 (the "**Act**") for the proposed construction, operation and (ultimately) closure of an underground copper-gold mine and associated ore processing, concentrate transport and handling, power generation, water and tailings management, and related support facilities and services (the "**Project**") in Morobe Province, Independent State of Papua New Guinea. The EIS was prepared with input from consultants engaged by the WGJV Participants and/or their related bodies corporate ("**Consultants**").

The Permit Application is to be lodged with the Conservation and Environment Protection Authority ("**CEPA**"), Independent State of Papua New Guinea.

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Engineering design and other studies are continuing and aspects of the proposed Project design and timetable may change.

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Newcrest Mining Limited ("**Newcrest**") is the ultimate holding company of Newcrest PNG 2 Limited and any reference below to "Newcrest" or the "Company" includes both Newcrest Mining Limited and Newcrest PNG 2 Limited.

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Competent Person's Statement

The information in the EIS that relates to Golpu Ore Reserves is based on information compiled by the Competent Person, Mr Pasqualino Manca, who is a member of The Australasian Institute of Mining and Metallurgy. Mr Pasqualino Manca, is a full-time employee of Newcrest Mining Limited or its relevant subsidiaries, holds options and/or shares in Newcrest Mining Limited and is entitled to participate in Newcrest's executive equity long term incentive plan, details of which are included in Newcrest's 2017 Remuneration Report. Ore Reserve growth is one of the performance measures under recent long term incentive plans. Mr Pasqualino Manca has sufficient experience which is relevant to the styles of mineralisation and type of deposit under consideration and to the activity which he is undertaking to qualify as a Competent Person as defined in the JORC Code 2012. Mr Pasqualino Manca consents to the inclusion of material of the matters based on his information in the form and context in which it appears.

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Competent Person's Statement

The Wafi-Golpu Joint Venture is an unincorporated joint venture between a wholly-owned subsidiary of Harmony Gold Mining Company Limited and a wholly-owned subsidiary of Newcrest Mining Limited.

The information in the EIS that relates to Golpu Ore Reserves is based on information compiled by the Competent Person, Mr Pasqualino Manca, who is a member of The Australasian Institute of Mining and Metallurgy. Mr Pasqualino Manca, is a full-time employee of Newcrest Mining Limited or its relevant subsidiaries, holds options and/ or shares in Newcrest Mining Limited and is entitled to participate in Newcrest's executive equity long term incentive plan, details of which are included in Newcrest's 2017 Remuneration Report. Ore Reserve growth is one of the performance measures under recent long term incentive plans. Mr Pasqualino Manca has sufficient experience which is relevant to the styles of mineralisation and type of deposit under consideration and to the activity which he is undertaking to qualify as a Competent Person as defined in the JORC Code 2012. Mr Pasqualino Manca consents to the inclusion of material of the matters based on his information in the form and context in which it appears.

WAFI-GOLPU PROJECT

DSTP Engineering Design Report

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EXECUTIVE SUMMARY:

This document has been prepared to provide a technical summary of the Engineering Design of the marine component of the proposed Deep Sea Tailings Placement (DSTP) system for the Wafi-Golpu Project. The technical study has shown that hydraulically, the tailings can be transported from the Watut process plant in the Mine Area to a mix/de-aeration tank situated 6.3km east of the Port Facilities Area in Lae, and approximately 120m inshore of the coast. From the near shore mix/de-aeration tank, the tailings would be passively transported (i.e., gravity-driven) via the DSTP outfall seabed-mounted pipelines and discharged approximately 200m below sea level.

The engineering design is for a system that can meet the requirements of a maximum design throughput of 2050 tonnes per hour (tph) (capable of handling 16.57 Million tonnes per annum) during the mine life. The proposed DSTP system (terrestrial and marine components) has an installation schedule of 19 months from the commencement of construction contractor mobilisation and hence deliver a DSTP system that will be available to meet the first production milestone.

The proposed DSTP Engineering Design by Tetra Tech implements a viable engineering solution for adopting DSTP as the preferred method of tailings management for the Wafi-Golpu Project. This Engineering Design forms part of the multi-disciplinary data collection, studies and Environmental Impact Statement (EIS) for a DSTP system.The marine components of the DSTP engineering design are as described in this report, with the marine outfall system consisting of the following:

- A 14m diameter x 15m high reinforced concrete mix/de-aeration tank inside a dry moat (31.9m [north-south] x 21m [east-west]) that is 12.2m deep, constructed using secant piles, located 120m inland from shore. Energetic flows mix the tailings slurry from the plant with seawater, while still allowing any entrained air to escape to the atmosphere;
- Two 505m long seawater intake pipelines (1000mm Outside Diameter (OD), High Density Polyethylene (HDPE), Standard Diameter Ratio SDR11) which convey seawater passively into the mix/de-aeration tank from 60m depth (i.e. flow is driven by the head difference between the operating mix tank level and sea level, accounting for the density difference between the tailings slurry and seawater); and
- Two 985m long slurry outfall pipelines (900mm OD, HDPE SDR11) which convey diluted tailings slurry (after mixing with seawater) from the mix/de-aeration tank to the terminal outfall depth (i.e., approximately 200m below sea level). Immediately beyond the discharge location the seabed slope exceeds 11°, leading to the Markham Canyon. The slope between 300m depth and the Markham Canyon floor ranges between 7° and 15°.
- Dry moat fitted with a 2.8kW submersible sump pump to pump rainfall from the dry moat and choke station concrete bund into the mix/de-aeration tank
- Ballast blocks to provide stability for seawater intake and DSTP outfall pipeline support and stability
- Instrumentation for the monitoring and control of the mix/de-aeration tank
- Electrical infrastructure.

The DSTP design presented in this report is based on information that was collected to support the preparation of an EIS and the specific details of the design may be refined and updated as the project is progressed.

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1. INTRODUCTION

1.1 Project Definition

Wafi Mining Limited and Newcrest PNG 2 Limited (WGJV Participants) are equal participants in the Wafi-Golpu Joint Venture (the WGJV). The WGJV is proposing to construct, operate and (ultimately) close an underground copper-gold mine and associated ore processing, concentrate transport and handling, power generation, water and tailings management and related support facilities and services (hereafter referred to as the "Wafi-Golpu Project" or the "Project"). The Project is located approximately 300 kilometres (km) north-northwest of Port Moresby and 65 km south-west of Lae in the Morobe Province of the Independent State of Papua New Guinea (PNG) (Figure 1.1).

As part of this development, WGJV proposes a Deep Sea Tailings Placement (DSTP) system incorporating pumping facilities, approximately 103km of terrestrial tailings pipeline to bring tailings from the Watut Process Plant to the coast (near Lae), a mix/de-aeration tank, and a marine tailings outfall system to transfer diluted tailings to the deep waters of the Huon Gulf. An overview of the locations of the DSTP Project components is shown in Figure 1.1 including the overall terrestrial and marine pipeline route alignments from the Watut Process Plant to the outfall terminus east of Lae.

Figure 1.1: Terrestrial Portion of DSTP Project - Pipeline Route

1.2 Engineering Context and Scope of Facilities

Wafi-Golpu Joint Venture engaged Tetra Tech to complete a DSTP engineering design for the marine components of the proposed DSTP system that would be capable of safely managing the final approved mine tailings production rate.

The engineering design scope includes a single tailings pump station at the Watut Process Plant with a single buried terrestrial tailings pipeline, followed by a choke station and a passive tailings mix/de-aeration tank and tailings outfall. The system is designed to handle tailings during the production ramp up, ramp down, and full production phases at the Watut Process Plant production with rates of up to 2050 tonnes per hour (tph) (capable of handling 16.57 Million tonnes per annum).

This report focuses on the engineering aspects of the marine components of the DSTP system, consisting of the mix/de-aeration tank, tailings outfall pipelines, and seawater intake pipelines.

1.3 Background Information

During August 2016, Tetra Tech was engaged by WGJV as the Engineering consultant to undertake technical studies for the proposed DSTP system. The engineering design presented herein, for the marine components of the system, is the outcome of several phased engineering investigations.

The design tailings through-put for the DSTP system is a maximum hourly throughput of up to 1934tph (accounting for a utilization rate of 91.3%); for engineering design, a maximum rate of 2050 tph (i.e. 1934tph + 6%) is considered. These rates will allow for handling 16.57Mtpa of tailings.

1.4 Concept of a DSTP System

A DSTP system usually consists of the following main components:

- A terrestrial tailings transport pipeline carrying tailings from the plant, discharging into the mix/de-aeration tank;
- A mix/de-aeration tank, in which slurry from the plant is mixed with seawater, for discharge to the sea;
- Seawater intake pipe(s) leading into the mix/de-aeration tank, either pumped or gravity-driven (by the head difference between the operating level in the tank and sea level, considering the relative densities of tailings slurry and sea water); and
- Tailing outfall pipe(s) from the mix/de-aeration tank, transporting the tailings/seawater mixture to the outfall at depth.

Seawater dilution is beneficial with respect to reducing the concentration of dissolved metals and metalloids in the tailings slurry, and with respect to the hydraulic balance of the system. Furthermore, the mix/de-aeration tank allows entrained air in the tailings slurry to be released to the atmosphere, prior to discharging into the ocean at depth. A gravity driven system (i.e., not a pumped system) provides flexibility where there may be a range of tailings input process parameters, because a gravity driven system automatically responds to fluctuations in input by adjusting the seawater intake rate.

The efficiency of a DSTP system depends on its ability to create a density current after the tailings and seawater mixture exits the outfall pipeline terminus. In general, slurry (tailings solids and liquid, including seawater) discharged through the outfall pipe at depths usually greater than 100m, will form a density current that travels along the seabed to the deep ocean, i.e. to depths greater than 1,000m. As the density current descends, it casts off dissolved material and some fine sediment to the overlying water column, thereby forming sub-sea plumes, usually at depths greater than 300m. Sub-sea plumes generally stay at the depth at which they are formed and spread horizontally under the action of ocean currents where they become progressively diluted. Sediment settles out from the density current and the sub-sea plumes. The interaction with the upper part of the water column is non-existent since these processes occur at great depths, where the tailings will remain. It is generally an important design consideration that the tailings liquids and solids do not enter the surface mixed layer or the euphotic zone (i.e. the region where primary production occurs).

1.5 Geographical Location

The proposed mine and process plant are located on the north side of the Owen Stanley Ranges of PNG approximately 300km north-northwest of Port Moresby and some 65km south-west of Lae in the Morobe Province of PNG.

The selection of the preferred marine pipe route alignment depended on a number of factors, including:

- Seabed morphology (e.g., avoidance of ridge-like features in the 0m to 350m depth range) for laying the pipe, minimizing the potential for lateral rolling of the pipe, and for the initial pathway of the descending density current.
- Transition from onshore to offshore conditions, i.e. the adequacy of the route along the submarine pipe but also downstream of it to convey a coherent density current down into the Markham Canyon. A relatively steeper slope at and immediately below the outfall terminus minimizes the likelihood that the outfall pipeline will 'plug' due to build-up of deposited tails. Similarly, such a slope is recommended for the formation and stability of the density current along its flow path; and
- Distance from the active part of the Busu River delta.

The marine component of the Project is located approximately 6.3km east of the Port Facilities Area in Lae at a site named Outfall "A". Outfall "A" is located adjacent to the Busu River alluvial plain. Figure 1.2 shows the proposed mix/de-aeration tank location for Outfall "A" as well as the proposed outfall pipe route. The seawater intake pipeline inlets are sited at 60m below sea level at approximately 505m from the mix/de-aeration tank. The DSTP outfall pipelines are sited at a terminus depth of approximately 200m below sea level at approximately 985m from mix/de-aeration tank.

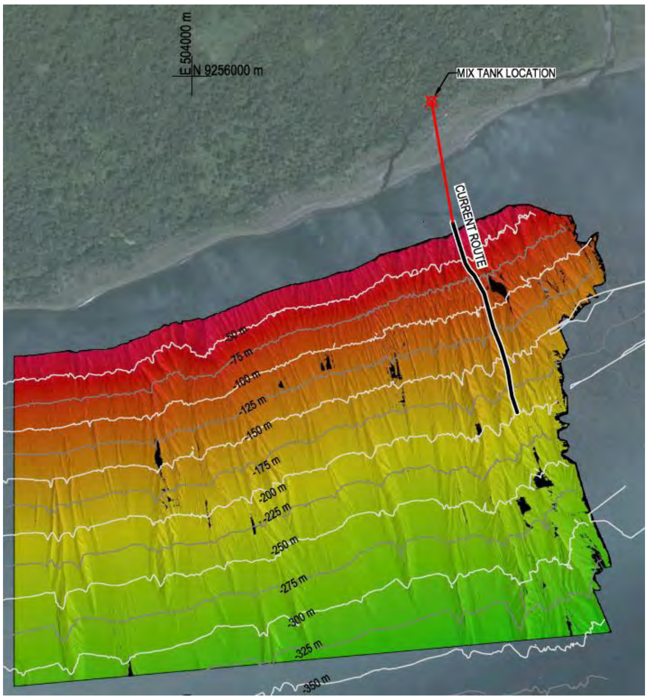

Figure 1.2: Marine Portion of DSTP Project overview map ("Outfall A")

1.6 Seismology

The project area is a seismically active area. Historical evidence suggests the region can produce crustal events with a magnitude of up to 7.7 and subduction events of Earthquake Magnitude (Ms) = 8.4.

A regional seismic assessment (SRK, 2007) was undertaken using deterministic and probabilistic analyses for the mine ore body and the Watut and Bavaga TSF sites and determined:

- Peak Ground Acceleration (PGA) for Operating Basis Earthquake (OBE) is 0.34g
- PGA for Maximum Credible Earthquake (MCE) is 0.43g
- Seismic coefficient for OBE and MCE are respectively 0.23 and 0.29

Additional geohazard, seismic and geotechnical work was undertaken by Advisian and Environmental Earth Sciences. Advisian (2017) included a detailed probabilistic seismic hazard assessment as well as a study of potential major fault displacements impacting the pipeline alignment. The adopted seismic design parameters along the pipeline are as follows:

- Wave Magnitude Earthquake (Mw) = 6.9 for 475 years return period
- PGA at Mix/de-aeration Tank location (Wagang Village) = 0.22g

1.7 Climate Data

1.7.1 Precipitation

The average annual rainfall at the Wafi mine camp is 2871mm, equivalent to an average daily rainfall of 7.8mm. It rains throughout the year, with the months of December to April being the wettest (Figure 1.3). During the year 1995, the area received 3440mm making it the wettest year on record, including the maximum daily rainfall event recorded to date (134.5mm) in February 1995. Rainfall at the Outfall Area is expected to be similar.

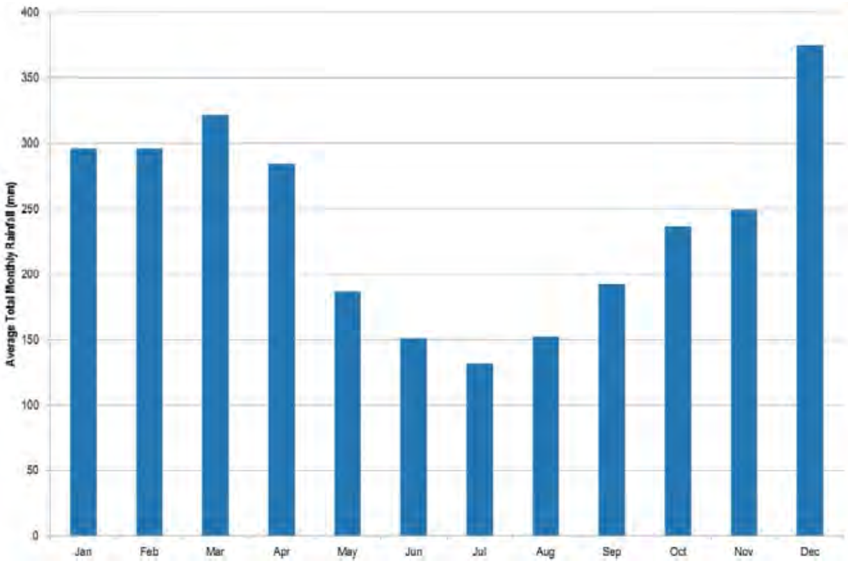

Figure 1.3: Average monthly rainfall recorded at Wafi Camp rain gauge (1990-2016)

1.7.2 Air Temperature and Relative Humidity

Air temperature and relative humidity observations are recorded at Wagang at 10-minute intervals. The average air temperature measured from 15 May 2017 to 1 December 2017 at the Wagang Station was 26°C, with a maximum value of 33.3°C occurring in October and a minimum value of 21.8°C occurring in July. The average measured relative humidity was 87.5%, with a maximum value of 99.8% occurring in November and a minimum value of 48.4% occurring in October.

2 DSTP ENGINEERING DESIGN OVERVIEW

The DSTP engineering design has been based on the following principles:

- The Watut Process Plant tailings thickener underflow will be discharged to the tailings feed tank. The tailings feed tank will provide a controlled tailings density feed to the charge and tailings pumps for discharge through the DSTP terrestrial pipeline, choke station, mix/de-aeration tank and outfall pipeline.
- The near shore mix/de-aeration tank will operate passively to dilute the tailings at a nominal dilution rate of four parts seawater to one part tailings slurry.
- The DSTP outfall pipelines will convey diluted tailings from the mix/de-aeration tank to the outfall terminus at a depth of approximately 200m below mean sea level (MSL).
- Pipeline engineering based on rheology test work carried out by Slurry Systems Engineering Pty Ltd (SSE) for a composite tailings sample (Section 5.2)

2.1 Design Process Parameters

The engineering design is based on plant production throughput rates corresponding to the mine production schedule tonnage over a 27-year life of mine (LOM), as envisaged at the time of engineering design[1]. A maximum design operating rate that considers a 6% increase in the maximum operating flow rate has also been incorporated in order to account for expected periodic fluctuations and minor changes in pipeline length. Furthermore, a design utilisation rate of 91.3% has been incorporated for all years of operation. The tailings process parameters utilised for this design are outlined in Table 2.1 and Table 2.2 below. For all years, the solids specific gravity (SG) is 2.82, d_{50} is 50.6µm and d_{90} is 123µm; the slurry pH is between 7 and 8.

The lower design solids concentration (35% C_w) specified for years 2 – 3, compared to years 4 – 27 (55% C_w), enables a single terrestrial pipeline and mix/de-aeration tank to manage the entire range of tailings throughput properties over the LOM.

Table 2.1: Process Parameters (Years 2 and 3)

Operating Case	Operate Minimum	Operate Maximum	Design Maximum (Max flow x 1.06)
Design Solids Throughput (tph)	912	967	1025
Solids Concentration by weight (%)	35		
Slurry Design Flow Rate (m^3/h)	2022	2144	2272

[1] While a 27 year mine life was envisaged at the time of this engineering design, the operations phase for the Project as described within the Environmental Impact Statement is proposed to continue for some 28 years.

Table 2.2: Process Parameters (Years 4 to 27)

Operating Case	Operate Minimum	Operate Maximum	Design Maximum (Max flow x 1.06)
Design Solids Throughput (tph)	1824	1934	2050
Solids Concentration by weight (%)	55		
Slurry Design Flow Rate (m^3/h)	2144	2273	2409

2.2 Marine Component of DSTP System

2.2.1 Choke Station

A horizontal choke station is provided at the terminus of the terrestrial pipeline, approximately 50m upstream of the mix/de-aeration tank. The choke station contains two duty and one standby choke, all of which are automatically brought into and out of circuit as required via actuated bypass valves. The purpose of the choke station is to prevent “slack flow” (pipeline pressure falls below vapor pressure of transporting fluid, leading to formation of vapor bubbles) and the resulting high liner wear in the terrestrial tailings pipeline that would otherwise occur downstream of the local high point at CH 99.1km.

Two DN700 hydraulically operated terminal valves are provided upstream of the chokes to prevent the tailings from draining out of the pipeline on shutdown and provide isolation for the choke station and mix/de-aeration tank. Two bursting discs are provided for over-pressure protection of the pipeline. One is upstream of the terminal valves and the other upstream of the chokes. On rupture, each discharges to the mix/de-aeration tank via a DN500 carbon steel pipe.

A storage container, hose water tank and pump, duty and standby diesel generating sets and a diesel storage tank are provided at the choke station.

Figure 2.1 shows the choke station layout and position relative to the mix/de-aeration tank.

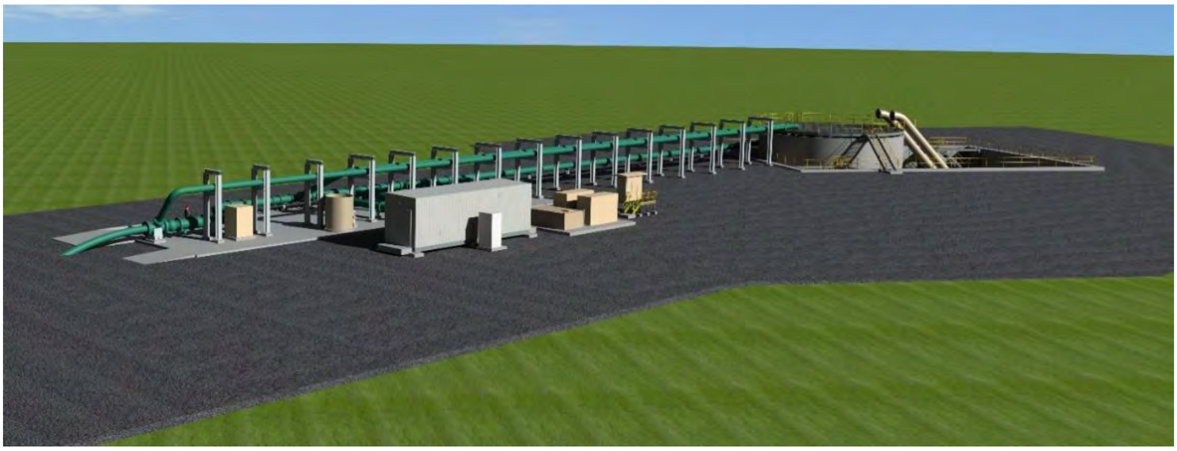

Figure 2.1: Choke station and mix/de-aeration tank view from north

2.2.2 Mix/de-aeration Tank and Dry Moat

Downstream of the choke station and 120m inland from the shoreline, a passively operating mix/de-aeration tank (14m inside diameter, 15m tall) will be provided to allow any entrained air in the inflowing tailings to escape to the atmosphere and to dilute the tailings with seawater in a one to four ratio of tailings slurry to seawater. An internal baffle wall within the mix/de-aeration tank will help to generate turbulent mixing of the tailings with seawater, prevents short-circuiting within the tank (i.e. prevent a direct flow path from developing between the inflow and outflow pipes), and encourage the detrainment of air from the inflowing tailings. The mix/de-aeration tank will be located in a dry moat with the bottom of the tank (and moat) at 8.25m below MSL. Although the marine intake and outfall pipelines will be constructed from HDPE, within the walls of the dry moat the seawater intake piping is DN750 (Sch5S) duplex stainless steel, while ocean outfall piping is DN750 (SchS10) rubber lined carbon steel. The tank receives inflow from one DN600 tailings feed pipeline and two DN750 stainless steel seawater intake pipelines, and discharges diluted tailings out of two DN750 rubber lined steel outfall pipelines. The seawater intake and outfall pipelines pass through the walls of the mix/de-aeration tank via 28% chrome alloy steel nozzles such that the bottoms of the pipes are all approximately 7.75m below MSL. The inflowing tailings feed line will pass through the wall of the mix/de-aeration tank 1.75m above the tank bottom (i.e. 0.5m above inflowing seawater pipelines) such that its outlet will be situated within the area semi-isolated by the internal baffle, adjacent to and above the location of the inflowing seawater intake pipelines. The mix/de-aeration tank will have provision to receive flow from one/two of the choke station’s bursting disc pipelines that enter the tank at the top. Figure 2.2 provides a three-dimensional drawing of the mix/de-aeration tank and dry moat.

The system design enables passive operation: i.e., gravity provides the energy required to move the tailings to the discharge point, rather than requiring the use of pumps. This system requires only nominal power consumption for valves and instrumentation in order to function during steady state and transient operating conditions.

An ultrasonic level transmitter, in conjunction with the Programmable Logic Controller (PLC), will monitor the fluid level in the tank. An alarm will be triggered if tank levels rise or fall outside of normal operating limits. Two pressure transducers will be connected to the side of the mix/de-aeration tank in a high/low arrangement to determine the density in the tank on a continuous basis. The pressure transducers also serve as a redundant level measurement. This data will be monitored by the PLC and will trigger an alarm if the fluid density rises or falls beyond specified thresholds or if the tank level rises or falls outside of normal operating limits.

Two stair towers at opposing corners (south-east and north-west) of the dry moat will be provided for access to the dry moat bottom, as well as egress purposes in the event of an emergency such as the unlikely event of the moat becoming flooded. The sump pump within the dry moat sump has been selected to accommodate a 200-year rainfall event.

All actuated valves in this area will be hydraulically actuated with the power packs located out of the dry moat. Electromagnetic flow meters will be located on each pipeline within the dry moat, monitored by the local HMI (Human-Machine Interface) and PLC, in order to provide continuous flow measurement and trigger alarms if flow rates fall below or exceed specific threshold values.

Remote operating vehicle (ROV) access pipes are provided at the top of the mix/de-aeration tank, connected to each outfall pipeline, in order to allow an ROV to be fed into each of the ocean outfall pipelines during scheduled maintenance periods. The ROV will be used to inspect the outfall pipelines for internal wear and to check for the unlikely presence of a deposited bed forming within the pipelines.

One tee-nozzle is located on each seawater intake pipeline within the dry moat such that a hired pump may be connected to each of these pipelines during scheduled maintenance shut-down periods to allow for pumping seawater into the mix/de-aeration tank and/or into the ROV launcher pipes to flush the system of any deposited solids and to maintain flow in the seawater intake pipelines.

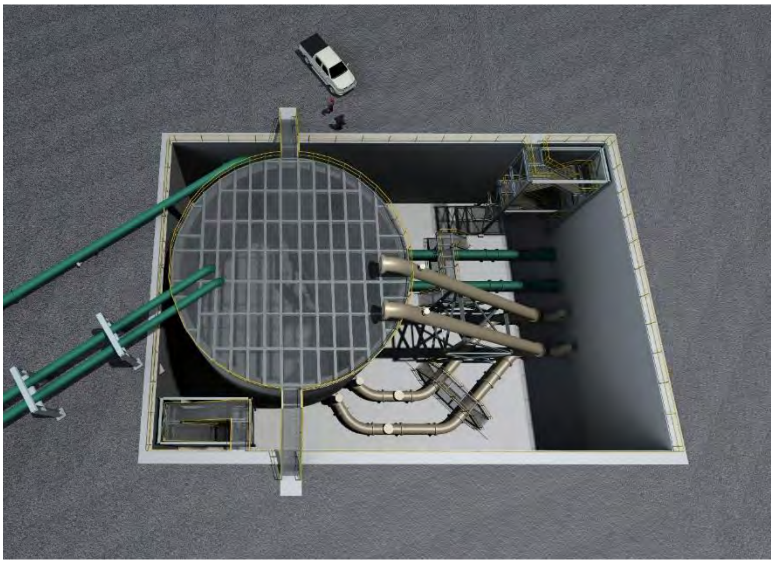

Figure 2.2: Mix/De-aeration Tank Top View

2.2.3 Seawater Intake Piping

The marine pipeline design has been based on a trenched and seabed-mounted alignment for the two seawater intake pipelines. Outside of the dry moat, the seawater intake system consists of two 1000mm OD HDPE (814.9mm ID) SDR 11 pipelines. The seawater intake pipelines pass through the wall of the dry moat via stainless steel nozzles at approximately 7.75m below MSL (bottom of pipelines). The seawater intake pipelines terminate at 60m depth, which is below the euphotic zone in the Huon Gulf (i.e. below the depth at which photosynthesis is expected to occur). The intakes of each of these pipelines are covered by steel screens to prevent the entrainment of deleterious debris and aquatic wildlife. These pipelines are each approximately 505m long from the wall of the dry moat.

From the mix/de-aeration tank to the point at which the pipelines daylight at 10m below MSL, the pipes are installed in a 12m wide trench. Beyond this point (10m depth at Station 305m, i.e. horizontal distance of 305m from mix/de-aeration tank), the seawater intake pipelines are installed on top of the existing seabed down to 60m depth. The ballast blocks for the seawater intake pipelines are designed for a 25% air offset (1583kg each at about 5m spacing).

Spare seawater intake pipelines will be stored adjacent to the mix/de-aeration tank throughout the life of the mine, in accordance with the Draft General Guidelines for DSTP in PNG (SAMS, 2010). However, pipe selection has been designed for the life of the mine.

2.2.4 DSTP Outfall Piping

Outside of the dry moat, the DSTP outfall pipelines consist of two 900mm OD HDPE (732.4mm ID) SDR 11 pipelines. These pipelines pass through the wall of the dry moat via

28% chrome-alloy nozzles at approximately 7.75m below MSL (bottom of pipes). The DSTP outfall pipelines each terminate at approximately 200m depth and are approximately 985m long from the wall of the dry moat.

The DSTP outfall pipelines will be installed within the same 12m-wide trench as the seawater intake pipelines.

The average slope along the DSTP outfall pipeline route, beyond the trenched section, is about 18°. At the outlet of these pipes, the slope is maintained greater than 11° down to approximately 300m depth, after which the downward slope primarily ranges between 7° and 15°, favoring the formation of a coherent descending density current to carry the tailings down to the floor of the Markham Canyon.

The ballast blocks for the DSTP outfall pipelines are also designed for a 25% air offset (1320kg each). These blocks will be spaced at 6m spacing from the dry moat to 25m depth (end of armour layer protection), at 5m spacing from 25m depth to 75m depth, and 6m spacing from 75m depth to the pipeline termini at 200m depth.

Similarly to the seawater intake pipelines, spare DSTP outfall pipelines will be stored adjacent to the mix/de-aeration tank throughout the life of the mine, in accordance with guidelines. The outfall DSTP pipelines have also been designed for the life of the mine.

All steel connections on the outfall and seawater intake pipelines (i.e. flanged connections and ballast block assemblies) are protected from corrosion by aluminium sacrificial anodes.

2.2.5 Flood Mitigation Design

The Outfall Area ground level will be constructed on a pad, elevated to 5003.5m (3.5m above MSL, RL5000m is at MSL) to reduce the risk of flood inundation from the Busu River. The expected maximum Busu River water surface level for a 1:200yr flood event is 5002.8m, based on a comprehensive hydrological and hydrodynamic study of extreme rainfall events in the Busu catchment (Tetra Tech, 2017a; 2017b). The berms surrounding the elevated area are protected from scour/erosion with riprap (class of 10kg and an approximate average rock size of 300mm), having an overall armour thickness of 450mm.

2.2.6 Security Fencing

Provision is made for security fencing around the mix/de-aeration tank area comprised of 2.4m high topped with security wire. Lighting and CCTVs will be provided.

2.2.7 Utilities and Communications

The entire DSTP operations will be managed from the main Watut Process Plant as all the operation centres along the pipeline route will be normally unmanned. A fibre optic cable connecting the Watut Process Plant and the Outfall Area ensures real time monitoring of the DSTP system. A combination of communications technology (i.e. VoIP, Ethernet, PLC, fibre optics, and Cat6 copper UTP cable) will facilitate this communication.

2.2.8 Electrical Interface and Energy Consumption

Power at the Outfall Area will be supplied by duty/standby 100kVA diesel driven generator sets sized on largest motor starting. Nominal loads at the Outfall Area (i.e. for operation of valves and instrumentation) do not depend on plant throughput and are relatively constant for the LOM. Energy required for the mix/de-aeration tank support systems, i.e. valve actuators, will be 21.6kW, area lighting will be 4kW, and IT/communications will be 4kW.

3 ENGINEERING STUDIES AND DESIGN DECISIONS

During the development of the engineering design, several investigations, often in the form of trade-off studies, were conducted, the outcomes of which directly influenced the design. These trade-off studies, described in detail below, include:

1. Pipe material selection: HDPE versus steel
2. Mix-tank material selection.
3. Pig vs ROV for pipe inspection during maintenance
4. Pump-driven system versus passive gravity driven system
5. Depth of outfall terminus
6. Appropriate dilution in the mix-tank
7. Pipe Route Selection anfd Mix/de-aeration Tank Location
8. Number and Size of Pipelines
9. Marine pipeline installation: HDD vs. Seabed-Mounted Option

3.1 Pipe Material Selection

There are several characteristics required of the pipe used for the outfall and seawater intakes of a DSTP system. Three candidate piping materials were considered: HDPE, steel and fibreglass. Although fibreglass is suitable in many situations in a minerals processing plant, it does not have the ductility that would make it suitable for a marine pipeline that would be laid on an un-prepared seabed, and be subject to earthquake damage. Therefore, fiberglass was not considered further. The choice between steel and HDPE merited some consideration, as summarized below.

Adequate Pressure Rating

The maximum operating pressure in the marine pipelines is about 20psi relative to atmospheric pressure (not accounting for overburden pressure or ambient seawater). This is a very low pressure requirement, easily met by both steel and plastic.

Chemical Resistance

Generally, HDPE is resistant to chemical damage, except for very high concentration acids, and except for some organics. Experience has shown that steel pipe is often not appropriate for piping in mineral processing plants due to corrosion. It should be noted that the terrestrial pipeline will consist of HDPE-lined carbon steel. The selection of an HDPE liner for the terrestrial pipeline was based on laboratory testing conducted by Slurry Systems.

Wear Resistance

PPI (2006) states: “The advantage of polyethylene in these applications is its wear resistance, which has been shown in laboratory tests to be three to five times longer than normal or fine-grained steel pipe at a typical velocity of under 15 ft/s.” However, HDPE can suffer wear when transporting slurries, such as at the Batu Hijau DSTP system, where the wear is thought to be attributable to the large angular particles from the slurry (Tedd Dowd, pers. comm.). Control measures for such wear are to control the grain size of the slurry solids being transported, and to design the system so that flow velocities are above the deposit velocity, i.e., so that a bed of moving solids does not form along the pipe bottom.

Biological Resistance

Polyethylene is resistant to some biological growth (i.e. bacteria) adhering to its surface, which is one reason for its use in sewage piping. However, HDPE pipes are subject to fouling by bio-organisms, including both seaweed and crustaceans (i.e. barnacles). While not a problem for the pipe’s exterior, the interior will need to be inspected and maintained.

Pipe Strength and Ductility

One of the biggest issues for the DSTP system for the Project is the ability to survive earthquakes. HDPE pipe was shown to resist damage during the 2010 Chilean magnitude 8.9 earthquake. It was found that all but two marine outfalls in Chile survived the earthquake and associated tsunami (Labbe, 2011). The two pipes that did not survive suffered damage and failure because of inadequate shore protection through the surf zone and because of inadequate ballasting, respectively.

Liquifaction of the seabed is deemed to be the most potentially damaging of the earthquake effects on the pipeline. The proposed pipelines possess adequate strength to withstand the tensile stress applied by liquefied soil moving down slope (Section 6.8).

Ease of Installation and Repair

The ductility of HDPE makes it a material of choice for installation in the marine environment where the S-bend method is usually being used.

Summary

Based on the above consiederations, HDPE was determined to be the superior material for the marine pipelines.

3.2 Mix-Tank material selection

Several materials were considered: concrete, stainless steel, and lined carbon steel. All three were similar in cost. Concrete was chosen because it could be fabricated locally, it did not need a liner, the outer surface did not need corrosion protection and the engineering principles for seismic design are well known and readily implemented for concrete.

3.3 Pipeline Inspection Gauge (Pig) versus Remotely Operated Vehicle (ROV)

As part of maintenance, pipe inspection will be conducted on a regular basis. Two options are available: a pig and a ROV. The pig can use the pipe pressure to move through the pipe. The ROV will have an umbilical cord linking it to an operator at the mix/de-aeration tank facility. It was determined that the pig had flaws for use in inspection. Foremost, the pig would exit the outfall pipe at a depth of approximately 200m, making recovery difficult. Furthermore, the pig is generally carried by ambient flow, making close examination of potential damage to the pipe challenging. An ROV can pause at any time in order to examine in more detail a particular section of pipe. It can be equipped with lights, cameras, and radius-measuring equipment to quantify wear.

Based on these considerations, it was determined that an ROV offered the best features for pipe inspection, and a special section of pipe was added to the design, angled up from the outfall pipe to the mix-tank, to allow launching and control of the ROV.

3.4 Mix/de-aeration Tank Gravity versus Pumped System

A thorough investigation was conducted in order to assess the trade-off between a pumped seawater system and a passive (gravity-fed) seawater system. This evaluation included an assessment of the net present cost (capital costs and operating costs) and inherent risks associated with both options.

For the purpose of this evaluation the tank arrangement was consistent for both options; however, the passive system required that the mix/de-aeration tank be situated with the base of the tank below sea level within a dry moat, whereas the mix/de-aeration tank is situated at finished grade elevation for the pumped system and contained by a dry bund.

The diluted slurry (4:1 dilution) from the mix-de-aeration tank flows out through the outfall pipeline(s) by gravity for both systems.

The evaluation of capital cost for each system included all of the critical items that have a significant impact on the cost associated with each system. Replacement and maintenance costs were also factored into the evaluation where appropriate. The passive system has negligible operating costs over the assumed 27 year life of the mine compared to the pumped system, but does however have a higher capital cost, largely due to the cost of the dry moat. Electricity was assumed to be provided by a diesel generator.

The difference in net present cost between these two systems over the life of the mine is relatively insignificant (i.e. ~5%). Therefore, the trade-off between these two systems is dictated largely by the inherent risks associated with either option. Risks considered as part of this study included those related to: bio-fouling, vandalism, power failure, mechanical failure, and extended shut-down of mine. A risk matrix was utilized to outline the possible risks associated with each system and to identify the mitigating measures that would be recommended. In summary, there are risks associated with both systems; however, providing the proper mitigating measures are employed, both systems have low overall risk with respect to impeding regular operation of the system.

Nonetheless, there is a higher risk associated with the pumped system, in that it relies entirely on the operating condition of the electrical and mechanical components (pumps, motors, variable speed drives, and generator), as well as availability of diesel fuel. Although back-up systems were included in the design (i.e. two generators and four pumps), if more than two pumps, both generators or the electrical components were to become significantly damaged due to any unforeseen circumstances, then significant delays would result due to the relative remoteness of the Outfall Area location and limited access to supplies, replacement components, and trained professionals to conduct replacement/maintenance.

The largest concern associated with the passive system is that extended periods of shut-down in mine operations could result in stagnant seawater being present in the pipelines, which would then present opportunity for biological growth to occur in the seawater intake pipelines. If no preventative measures were employed, then this could result in increasing the frictional resistance within the pipe (i.e. up to nearly 80% observed at another installation in tropical waters). The currently proposed design would continue to function in this scenario; however, the fluid level in the tank would decrease significantly, posing a high risk of air entrainment into the outfall pipeline during transients or due to the formation of a vortex. In this case, the intake pipelines would need to be replaced. In accordance with Draft General Guidelines for DSTP in PNG (SAMS, 2010), provision for spare pipelines is recommended for either the pumped or the passive system. Given that adequate preventative measures are taken, this risk can be significantly reduced by using temporary pumps during extended shut-downs which will be connected to the tees on the seawater intake pipelines within the dry moat and used to discharge seawater into the ROV pipelines, pumping seawater through the piping system to maintain flow and prevent biological growth.

3.5 Tailings Ocean Outfall Terminus Depth

Several alternative outfall terminus depths were considered: 150m, 200m, 250m, and 300m. It was found that, from a hydraulics perspective for the passively operating system, any of these outfall depths would allow for satisfactory operating conditions to exist (i.e., in terms of tank operating level, dilution achieved, outfall pipe velocities, etc.). Therefore, the critical factors for determining the selected outfall depth are:

- Compliance with the Draft General Guidelines for DSTP in PNG (SAMS, 2010); and
- Submarine topology along the pipe route and density current path.

SAMS (2010) specifies recommended outfall depths based on the depth of the euphotic zone. Where the euphotic zone is less than or equal to 80m depth, the outfall depth must be a minimum depth of 120m. If the maximum depth of the euphotic zone is greater than 80m, then the outfall depth must be at least 50% greater than the deeper of:

(a) the maximum observed mixed layer depth; and

(b) the maximum observed euphotic zone depth.

Based on a 12-month period of CTD (Conductivity-Temperature-Depth) profile data from the Huon Gulf, collected by IHAConsult (Oceanography consultant), the depth of the euphotic zone appears to be consistently less than 60m.

Given the strong southeasterly winds that can occur in the Huon Gulf and in order to ensure the tailings outfall is below the layer of weakly stratified surface waters to minimize the formation of tailings plumes in the water column, Tetra Tech and WGJV selected the more stringent criteria from SAMS (2010) as the design basis. Based on this 12-month period of CTD profile data from the Huon Gulf, the strongest pycnocline, that would be a barrier to upward mixing of tailings, was consistently located above 96m. Based on SAMS (2010), the outfall ldepth should be 50% greater than 96m, i.e., greater than 144m. To ensure compliance with SAMS (2010), a 200m outfall depth was selected.

Following the selection of the final pipe route alignment, several near-field density current simulations were initialized to compare and evaluate eleven different potential outfall locations along this route between 193m and 220m depth in terms of the overall behaviour of the density current (i.e. thickness, dilution, deposited solids, entrainment, and depletion, etc.). It was concluded that, from a density current perspective, there was no benefit of having the discharge location below 200m depth, and in fact the density current behaved more coherently when discharged at approximately 200m depth. From an installation and cost perspective, having the outfall termini locations positioned at approximately 200m depth is also preferable.

3.6 Mix-Tank Dilutions

Several different dilution rates were analysed from a hydraulics and density current perspective: 2:1, 4:1, and 8:1. A target dilution of 4:1 was selected as appropriate. Simulations of the various dilution options were conducted using the near-field density current model. It was found that by about 100m from the outfall pipe, there is minimal difference in the behaviour of the near-field density currents between the 8:1, 4:1, and 2:1 diluted tailings slurries in terms of dilution. It was found that the 4:1 dilution ratio provides an optimal balance between achieving an outflowing tailings with properties that enable the generation of a coherent density current, providing sufficiently high entrainment velocities, providing desired hydraulic characteristics in the mix/de-aeration tank and outfall system, and preventing the formation of slug or slack flow, while also yielding a desirable dilution ratio from an environmental perspective.

It should be noted that higher dilution ratios, i.e. 8:1, are primarily used in gold mines where the concentration of residual cyanide is high. The dissolved metal with the highest concentration in the Project tailings slurry discharge in comparison to PNG ambient marine water quality standards is cobalt. From elutriate testwork (i.e., mixing tailings in seawater to measure the concentration of metals released from the tailings into the dissolved phase) it was found that cobalt will require 1,800 dilutions for the tailings to meet PNG ambient marine water quality standards. Dispersion modelling has shown that required water quality levels are predicted to be met at 2,174m from the outfall (Appendix J - Density Current, Plume Dispersion and Hydrodynamic Modelling).

3.7 Mix/De-aeration Tank Location and Marine Pipeline Alignment

The proposed DSTP outfall pipeline route (Figure 3.1 and Figure 3.2) lies between a designated cultural heritage site (east of Wagang Village) and the Busu River. The location of the cultural heritage site was taken into consideration when siting the mix/deaeration tank to avoid the cultural heritage site (including a buffer zone). The selected mix de-aeration tank location (504,703 E / 9,255,925 S) will be situated approximately 6.3km east of the Port Facilities Area in Lae and 120m inland from the shoreline on a built-up pad; the discharge location will be (504,957 E / 9,255,011 S) at a depth of approximately 200m. Review of satellite imagery along Huon Gulf shoreline at this location suggests the shoreline may be receding; hence a 120m setback of the mix/de-aeration tank relative to the shoreline will be provided. This route was favoured for the following reasons:

- High-precision bathmetry by the Neptune Bathymetric Survey (2017), Kongsberg AUV Survey (2016), and Singaua Survey (2012) provides confidence to the accuracy of the submarine topology and the engineering design to address this bathmetry. Analysis of the selected pipe route using data from these surveys shows strong agreement in submarine topology along this route with no significant discrepancies or indication of significant changes in bathymetry (i.e. accumulation or erosion of sediment or land movement).
- A nearly constant slope is observed below 80m depth and an adequate slope is present for density current formation (above 11º).
- The density current will be conveyed through a reasonably wide submarine valley that modelling has shown will allow the density current to proceed unhindered to the Markham Canyon (i.e. non-constricting flow).
- An adequate slope along the density current's main axis (slope below 300m depth primarily ranging between 7° - 15°).
- An inundation study related to potential risk of floods from the Busu River confirmed that a mix/de-aeration tank and choke station in this area could be protected from inundation during the 200-year flood by raising the ground elevation to a finished grade of 3.5m above MSL.

Figure 3.1: Location of mix/de-aeration tank and DSTP outfall

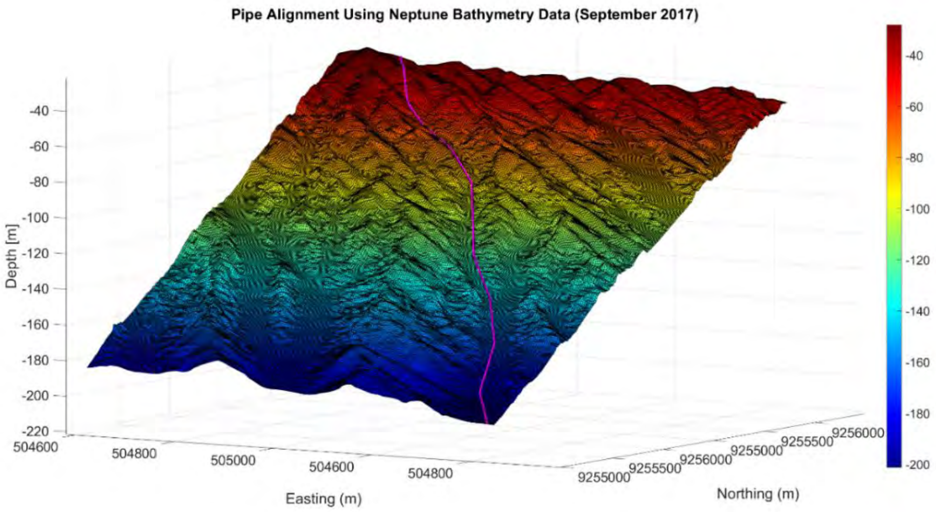

Figure 3.2: Final Selected Pipeline Alignment (Note vertical exaggeration)

3.8 Number and Size of Pipelines and Mix/De-Aeration Tanks

The option of using multiple tanks and multiple pipes was considered for the tailings throughput for the Project. The number and size of marine pipelines, as well as the number and size of mix/de-aeration tanks is largely driven by the hydraulics of the passive system. Although a wide range of potential operating points are possible with a gravity-fed mix/de-aeration tank system, the passively operating system relies on a specific balance between the potential energy resulting from the tailings density (relative to sea water) and mass flux and the frictional losses in the seawater intake and outfall pipelines in order to achieve a specified dilution ratio of seawater to tailings. The inflowing tailings density is specified within the design criteria and the pre-selected location for the mix/de-aeration tank and pipe terminus depths governs the length of pipelines required. Therefore, the hydraulic engineering design is focused on altering the geometry of the system in terms of the tank bottom depth, tank cross-sectional area, tank baffle design, pipe invert depths and orientations, pipe numbers, and pipe sizes. Frictional losses in the pipelines are directly proportional to the square of the pipeline fluid velocity, inversely proportional to pipe diameter, and directly proportional to pipe length. Additional considerations include:

- Ensuring that the operating level of the tank and lowest level during any transients is maintained at least one pipe diameter above the top of the outfall pipe(s) to prevent a vortex from forming and/or air from being entrained into the outfall pipeline, while also ensuring the operating level of the tank remains at least 30cm above the top of the baffle wall in order to limit potential scouring due to high local tangential velocities along the top of the baffle wall and in order to prevent a billowing-type flow from developing over the top of the baffle wall (depends on tank bottom elevation, tank size/numbers, pipe size/numbers, and baffle wall design).
- Ensuring that the outfall pipe velocities remain above the depositional velocity of the diluted tailings solids and below the velocity at which an increased risk of pipe wear is expected to occur (depends on pipe size/numbers).
- Ensuring adequate residence time of the tailings inside the tank to maximize the potential for the release of any entrained air in the inflowing tailings (depends on tank size/numbers, pipe size/numbers, and baffle wall design).

A single large (14m inside diameter) mix/de-aeration tank has been selected, which can manage the proposed tailings throughputs for the entire life of the mine. The tank diameter is governed by the requisite cross-sectional area of the tank that provides a sufficiently high residence time for given design throughput flow rates to ensure adequate liberation of any entrained air in the slurry. Having one large tank instead of two smaller tanks reduces the dimensions of the dry moat and simplifies the arrangement of pipelines within the dry moat.

The various pipeline combinations that were evaluated include:

1. Two Sea Water Intake Pipelines (1.0m OD, 0.8149m ID) and Two Outfall Pipelines (0.9m OD, 0.7324m ID), all of which would be in operation during the life of the mine.
2. Three Sea Water Intake Pipelines (0.90m OD, 0.7324m ID) and Three Outfall Pipelines (0.71m OD, 0.5776m ID).
3. Four Sea Water Intake Pipelines (0.80m OD, 0.651m ID) and Four Outfall Pipelines (0.63m OD, 0.5126m ID).

The Project design criteria specified SDR11 pipeline in order to ensure the pipeline can withstand the stresses imparted to it during installation, a liquefaction event, and in the event that up to three ballast blocks become unsupported and held up between two ballast blocks by the pipeline (following installation), as well as to ensure its integrity (i.e. wear resistance and mechanical fatigue due to cyclical loading of currents and wave action) over the LOM, and its ability to resist overburden loads within the trenched section.

Option (1) was selected based on:

- Constructability and installation: Having fewer pipelines reduces the complexity of piping in proximity to the mix/de-aeration tank and simplifies installation;
- Operability: The dual tailings ocean outfall and dual seawater intake pipeline system yields optimal operating characteristics of the passive mix/de-aeration tank system over the range of throughput process parameters for the LOM (i.e. target dilution, outfall velocities, mix/de-aeration tank fluid level, etc.).

The selected mix/de-aeration tank and pipeline design infrastructure is as follows:

- One large mix/de-aeration tank: 15m tall, 14m inside diameter, with 500mm thick reinforced concrete walls
- Two 900mm OD SDR 11 (732.4mm ID) HDPE Outfall Pipelines
- Two 1000mm OD SDR11 (814.9mm ID) HDPE Seawater Intake Pipelines
- Sea water intake pipelines within dry moat are stainless steel (DN750 Sch 5S)
- Outfall pipelines within dry moat are rubber-lined steel (DN750 Sch10 Rubber Lined)

3.9 Marine Pipeline HDD versus Trenched/Seabed-Mounted Option

Three potential options are available for installing the marine pipelines from the wall of the dry moat (7.75m below MSL) to their termini at 60m (seawater intake) and approximately 200m (outfall) depths respectively, which have been evaluated for the engineering design:

1. Trench excavation from the wall of the dry moat to approximately 10m depth below MSL and seabed-mounted pipe installation from 10m depth to pipe terminus;
2. Horizontal Directional Drilling (HDD) of the pipelines from the wall of the dry moat to the pipe terminus; and
3. HDD of the pipelines from the wall of the dry moat to approximately 10m depth below MSL (i.e. in lieu of trench operation for approximately the first 300m of each pipeline) and seabed-mounted installation from 10m depth to the pipe terminus.

It has been decided to proceed with the seabed-mounted option, based on three major considerations:

1. The risks associated with the seabed-mounted option are well understood, and the installation method has been used successfully many times in similar situations whereas the risks associated with the application of HDD for such a long route and with relatively large pipes are significant.
2. The seabed-mounted option is considerably less expensive and will require a shorter schedule.
3. The only practical option for HDD involved four outfall pipes (630mm OD HDPE with 800mm casing), and four intake pipes (800mm OD HDPE and 1000mm casing), based on lower construction risks associated with smaller-diameter pipelines and on hydraulic considerations. The selection of four intake and four outfall pipelines to facilitate using HDD would complicate the configuration of pipelines within the dry moat, increase the number of nozzles through the dry moat and through the mix/de-aeration tank wall, as well as the number of valves, EM flow meters, and ROV launcher pipes. These factors would also influence the total cost associated with the DSTP system, or would result in custom nozzles being manufactured at this interface to connect the eight HDD pipelines with the currently proposed, four larger diameter pipelines within the dry moat, such that the currently proposed dry moat configuration would remain unaltered.
4. The additional complications that arise with the third alternative are that following the termini of the HDD section, there would be either eight pipelines to install, with ballast blocks on each pipeline, which would increase the schedule time required for installation, and increase the required spatial footprint of the pipelines on the seabed; or custom nozzles would again converge the HDD pipes into half the number of correspondingly larger pipes for the seabed-mounted configuration.

4 BASIS OF DESIGN

4.1 LEAN Design

The success of the project business case is dependent upon the ongoing development and realisation of the LEAN Business Engineering approach (optimised scope, specification, cost, schedule, and execution strategy).

The purpose of the Wafi-Golpu Project LEAN Business design guideline was to outline the key principles and processes to be employed and managed in the definition phases to ensure that the LEAN business engineering opportunities are properly and continually explored, developed and achieved in each project life cycle phase.

4.2 Design Reviews

Regular design reviews, including by independent competent reviewers, were performed at pre-determined stages of the project development to identify issues that could be expected to adversely affect the suitability, adequacy and effectiveness of the design, prior to proceeding to the next stage of design. Design reviews focused on different aspects of the design, i.e., environmental, functional, standardisation, maintainability, etc. and considered components, sub-systems and systems. Actions arising from the design review were recorded and integrated back into the designs.

In addition to the scheduled stage design reviews, regular internal design reviews and reviews with sub-consultants (Slurry Systems Engineering and Staheli Trenchless Consultants) were held during the on-going design phase and when engineering deliverables were completed at various stages during the project phase. These reviews were both discipline specific as well as multi-disciplinary. Peer Reviews

The intention of the DSTP Competent Independent Review (CIR) peer review process was to engage a CIR early in the design process to ensure timely input that could be incorporated as the project progressed. The scope of the CIR was to complete a detailed review of the DSTP reports and recommended Forward Work Plans. These CIR reviews were performed at pre-determined stages of the project development to:

- Review the Project in terms of compliance to internal and WGJV standards, project set-up and engineering designs,
- Identify all issues that could adversely affect the suitability, adequacy and effectiveness of the design,
- Identify any weaknesses that have material impact on methodology adopted during data collection and studies,
- Ensure project team has identified key risks associated with DSTP and has mitigation measures in place,
- Identify any opportunities for project enhancement, and
- Verify that the forward work plans are adequate

4.3 Discipline Design Criteria

The purpose of the DSTP Design Criteria for the Project is to clearly define the criteria to be used as part of the engineering design. This DSTP Design Crietria is to be considered as an approved source of input data for the engineering team and has been utilised for the design. The criteria outline clear definitions of relevant standards and statutory requirements. A brief summary of the critical design parameters for the marine components are summarized below:

- All permanent infrastructure for the Outfall System is designed for a minimum 27 year life[2].
- The DSTP seawater intake and outfall pipelines are HDPE, grade PE4710, SDR 11, PN16. The material is in accordance with ASTM D 3350 (Cell classification 445574).
- The seawater intake and DSTP outfall piping is HDPE and thus will not be subject to corrosion, apart from the flange connections and ballast block clamp bolts, which will be fitted with cathodic protection. This material is sufficiently robust and will provide the greatest resistance to damage from seismic events on the sea floor and during the installation phase.
- No allowance for future expansion has been applied to the design of the Outfall System.
- Geohazard and seismic loads: design requirements for effects of liquefaction-induced ground displacement.
- The control system will provide the necessary control and instrumentation to ensure safety of personnel and protection of equipment, as well as assurance of recovery from upset conditions.

[2] 27 years being the mine life as envisaged at the time of this engineering design. The operations phase for the Project as described within the Environmental Impact Statement is proposed tol continue for some 28 years.

- The DSTP system control philosophy ensures automated plant control to prevent need for continuous manned operations, with provision for manual override and control if required.
- The number of types of mechanical equipment, e.g., pumps, valves, motors, has been kept to a minimum through equipment standardisation.
- Hydraulic power packs have been included for all actuated valves at the mix/de-aeration tank and at the terminal valves.
- A spare tailings outfall pipeline and seawater intake pipeline, complete with ballast blocks, is included in the design and assumed to be located at the Outfall Area.
- The power generation plant at the choke station and mix/de-aeration tank area comprises self-contained diesel generation units with the appropriate duty standby arrangements to allow for the required maintenance and designed availability.

Table 4.1: Key marine component design data

Parameter	Design Value
Minimum mix/de-aeration tank fluid depth	2.8m
Target Dilution (Seawater:Tailings)	4:1
Range of Operating solids concentrations by weight (C_w)	9-15%
Estimated Slurry deposition velocity @ C_w = 9-10%	1.4 – 1.7 m/s
Estimated Slurry deposition velocity @ C_w = 13-15%	1.5 – 1.8 m/s
Slurry deposition velocity (@ C_w = 11-15.3%) (range considers p80 = 93um and p80 = 150um)(1)	1.07 – 1.8 m/s
Minimum Slurry Operating Velocity for p80 = 93um(1)	1.4 m/s
Minimum Slurry Operating Velocity for p80 = 150um(1)	2.1 m/s
Minimum slope at outfall terminus location	12°
Minimum outfall terminus depth	200m
Minimum seawater intake depth	60m
Minimum pipeline ballasting (air offset, %v/v)	25%
Earthquake Design Return Period	475 years
Busu River Flood Protection Design Return Period	200-year event

[1] Determined by SSE

4.4 Engineering Design Drivers

The key and material aspects considered during the design phase include:

- LEAN approach to provide the least up-front cost and efficient construction schedule
- Application of appropriate industry standards with respect to safety to persons, operability and maintainability
- Minimum variation in equipment specifications and types to limit spares holdings
- Commonly available equipment and materials
- Technology that is tried and tested
- Designing infrastructure and equipment that, through the life of the mine, may be used for an alternative function or may be relocated for reuse elsewhere when required.

4.5 Constructability Issues Identified and Provided for in the Design Basis

In addition to the HAZOP meetings, constructability reviews of each area of the DSTP system were conducted by multi-disciplinary teams from Tetra Tech and WGJV. The purpose of the constructability reviews was to ensure that construction aspects of the Project have been analysed and optimised solutions provided. For example in Health, Safety, Environment and Community (HSEC) items such as schedule, construction sequence, ease of construction, materials of construction, weight, size, construction technique, skills and resources were identified. Key actions required to develop a coherent constructability framework during the detailed design phase were identified via a rigorous analysis of inputs that could affect construction (i.e. designs, assumptions, other data, etc.).

The constructability reviews will be revisited during the detail design phase to confirm that the strategy and recommendations are still applicable and to identify additional improvements to be carried forward into project execution and construction.

5 TEST WORK

5.1 Oceanographic Test Work

The Draft General Guidelines for DSTP in PNG (SAMS, 2010) recommend that in order to meet the standards for international best practice, a preliminary site evaluation and environmental baseline study must be performed.

IHAconsult has been engaged by WGJV since August 2016 and has conducted over one year of oceanographic data collection and analysis in order to inform the DSTP design. Data from these investigations has been shared between IHAconsult and Tetra Tech throughout the monitoring period, such that it could be utilized to:

- Determine seasonality in mixing, flushing, depth of the weakly stratified surface layer, and euphotic zone;
- Inform the DSTP hydrodynamic modelling work (H3D modelling, density current modelling and hydrodynamic modelling of the fate of the tailings within the Huon Gulf);
- Inform the Environmental Impact Statement.

The work conducted by IHAconsult during this time relevant to the DSTP engineering design included the following:

- Monthly CTD profiling along two transects (5 stations per transect) on the west and east side of the Busu River with the deepest casts being >1000m depth, to measure: conductivity, temperature, depth, dissolved oxygen, chlorophyll-a, and turbidity;
- Inner Huon Gulf Instrument Moorings at 5 locations: “outfall mooring”, “canyon mooring”, “basin mooring”, “far-field mooring”, and “trench” that include: Acoustic Doppler Current Profiler (ADCP), sediment traps;
- Marine sedimentology, including gravel samples from Markham Canyon, and box core and multi-core sampling in the Huon Gulf;
- Echo sounder transects to assess bed sediment transport on the canyon bed;
- Bed altimeters (basin, canyon, and trench) to monitor change in bed elevation;
- Trial of icListen hydrophone technology on the Canyon mooring to measure mass movement events;

The data listed above (amongst other data from Wave Watch III (wind/wave data), HYCOM, and NOAA) has provided integral inputs to the determination of environmental forcing on the pipelines, as well as inputs to the DSTP hydrodynamic modelling and the density current model to determine the behaviour and footprint of the tailings in the Huon Gulf over the duration of operations. This density current modelling is described in Tetra Tech (2018).

5.2 Tailings Slurry Test Work

In order to complete the hydraulic design and select the piping materials to be used for the terrestrial tailings pipeline, a number of laboratory tests were undertaken to establish the tailings slurry properties. Two samples (provided by WGJV) were tested:

- "A17234 MN1604 Final Product" (rheology at as received pH, rheology at pH10, particle sizing, solids SG, settling test, slope settling test, and slurry and water corrosion tests).
- "Domain 29 Upper South Flotation Tails", or "Domain 29". This sample was produced using 100% metasediments product, considered to be the most challenging tailings with respect to hydrotransport, and expected to be produced by the plant towards the end of the mine life. Testing for this sample included rheology at as received pH, rheology at pH10, particle sizing, solids SG, settling test and slope settling test.

The key results from the testing are described below.

5.2.1 Particle Sizing

A particle size comparison by Slurry Systems Engineering is shown in Figure 5.1, below. The Domain 29 sample is slightly coarser, which will marginally reduce DSTP capacity.

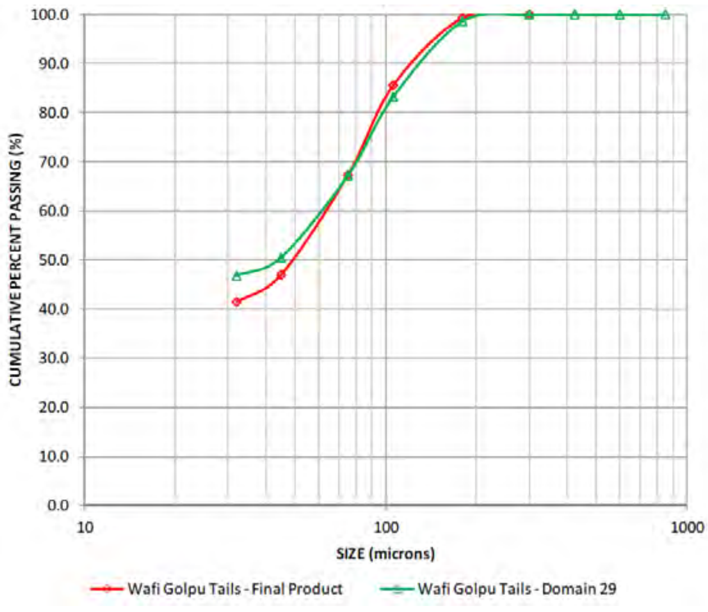

Figure 5.1: Particle Sizing Comparison (SSE, 2017a)

5.2.2 Tailings with Seawater Rheology

To establish the expected slurry rheology data for the outfall pipeline, additional testing was carried out by SSE (2017b, 2017c) on Wafi-Golpu tails mixed with seawater. A seawater to slurry mix ratio of 4:1 was used for the test (i.e. design concentration of 15% C_w) using the A17234 MN1604 Final Product sample. The seawater was sourced from the ocean in the Perth, Western Australia, area. The findings of this study can be summarized as follows:

The yield stress (Pa) can approximately be described by the equation:

$$Yield\ Stress\ (Pa) = 9.766x10^{-3}\exp(6.892x10^{-3} * {C_w}^2)$$

Where:

- C_w is the concentration by weight of the slurry, valid from about 3% < C_w < 19%.

Plastic viscosity can approximately be described by the equation:

$$Plastic\ Viscosity\ at\ 20^oC\ (mPas) = 1.094\exp(6.5566x10^{-3} * C_w{}^{1.2})$$

Where:

- C_w is the concentration by weight of the slurry, valid from about 0% < C_w < 19%.

The predicted deposit velocity for the Final Product Sample is 1.07m/s at C_w of 11-15.3%. In Slurry System Report (SSE, 2017c), the effect of possible coarser grinds on the system was investigated, with an analytic scenario using a p80 of 150μm. In such case, the predicted deposit velocity is 1.8m/s. As a result, minimum recommended outfall pipe velocity is 2.1m/s.

5.3 Geotechnical Test Work

5.3.1 Preliminary Slope Stability Analysis (August – November, 2016)

Tetra Tech was engaged by WGJV in August 2016 to conduct an offshore preliminary slope stability analysis to evaluate the suitability of three areas (“Area A” [red], “Area D” [yellow], and the area east of the Busu River [green]) in the Huon Gulf for the seabed-mounted installation of a DSTP system (Figure 5.2).

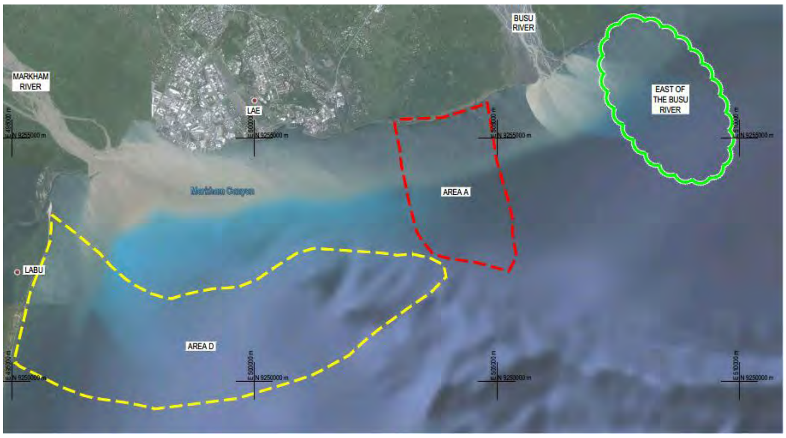

Figure 5.2: Preliminary Slope Stability Analysis Site Location Plan

This study included a review of existing information that was available prior to 2016, including technical/scientific published literature, and involved conducting the following surveys:

- August 2016 Survey: Multibeam Echosounder (MBES), Subbottom Profiler (SBP) and ADCP measurements for the Huon Gulf; and
- September 2016 Survey: Multibeam Echosounder (MBES), Subbottom Profiler (SBP) and ADCP measurements focused on areas near Lae, and limited soil sampling at depths greater than 300m. This survey was carried out at a considerably higher spatial resolution than the August 2016 survey.

Based on this study, it was determined that submarine landslides in this area may be triggered by earthquakes, over-steepening of sediments at the delta front of rivers (i.e. the Markham and Busu Rivers), and by erosion of the Markham Canyon. “Area A” was determined to be the most suitable location investigated, and five potential pipeline routes were identified within this area.

5.3.2 Updated Slope Stability Analysis

The Preliminary Slope Stability Assessment concluded that further geotechnical investigations should be conducted along the proposed pipe alignment, including sampling, drilling, and in situ testing (i.e., Cone Penetration Testing (CPT)), in order to obtain the required soil strength data to reassess the seismic and seabed slumping analyses discussed above.

As a result, during August to October 2017, Advisian was engaged by WGJV to conduct a series of geotechnical and geophysical investigations along the marine component of the proposed DSTP system alignment from the mix/de-aeration tank to the pipeline terminus.

The following geophysical work was completed during these investigations:

- Multibeam Echosounder (MBES) and Singlebeam Echosounder (SBES) to measure water depth to determine topography, seabed gradient, and seabed sediment composition (via backscatter).
- Sidescan Sonar (SSS) to provide seafloor reflectivity based on sediment type as well as identify geologic or man-made features at the seabed (i.e. identify potential hazards to pipelines and measurement of seabed composition to support MBES backscatter datasets).
- Velocity profiling consisting of direct SVP (Sound Velocity Profiling), XBT (Expendable Bathythermograph) and CTD for proper velocity control.
- Subbottom Profiler (SBP) to image subsurface soil, stratigraphic, and structural conditions.
- 2D Ultra High Resolution (2DUHR) seismic profiling to image the subsurface soil, stratigraphic, and structural conditions.
- Sparker survey (to provide deep imaging of the subsurface for the characterisation of geologic structure and stratigraphy in order to assess and properly characterise geohazards and conditions relevant to the DSTP pipelines).

The following geotechnical investigations were completed:

- Seabed sampling using a 3m Kullenberg Piston Core, Aimers McLean 3m Vibrocore, and offshore geotechnical laboratory to investigate soil composition, water content, density, and undrained shear strength of soil along pipe alignment.
- 70m deep (100mm diameter) Sonic borehole log and Standard Penetration Testing (STP) to determine soil composition and relative density of granular deposits at proposed mix/de-aeration tank location
- Cone Penetration Testing (CPT) using a dual-drive 50kN CPT Unit with 6m mast to determine cone resistance (qt, MPa), friction (fs, MPa), pore pressure (U2, kPa), ambient pore pressure (kPa), pore pressure ratio (Bq), friction ratio (Rf, %), undrained shear strength (Su, kPa), and relative density (Dr, %) along pipe alignment

Based on the review of the 2017 investigations, information from previous investigations, and understanding of the geology of the area, a geological model was developed for the slope stability analyses. The model makes a distinction between non-cemented granular deposits present below the seafloor with varying thickness and cemented granular deposits (considered to be likely the Leron Formation) present below the non-cemented deposits.

The results of the 2017 geophysical survey concluded there was an absence of deep seated failures, active faults and fluid escape features in “Area A”. While conducting the updated slope stability analyses and reviewing the recent geophysical surveys, Tetra Tech also arrived at the same conclusion. The new multibeam survey of the area did not reveal any significant difference from the 2016 AUV survey, but was useful in precisely positioning the 2DHRS lines.

The main source of slope instability is from sediment erosion/accumulation and earthquakes. In “Area A”, in the absence of any deep seated sliding surface or signatures, it is expected that only shallow instabilities could develop along channels or channel slopes. Instabilities along the Markham canyon, i.e. depth of 600m and deeper, are also possible, but their impacts on the pipeline located down to approximately 200m depth are considered minimal since retroactive failures are not expected to extend from the Markham Canyon up to the Outfall Area.

Morphological analysis of the DSTP pipeline alignment seaward of the trenched section indicated the potential for small localized failures along the channel walls. These localized failed masses may have a small volume but they could be widespread along the channel walls. By observing the floor morphology of the channel, for as long as the channel remains active, the slide material will be rapidly eroded and carried to greater depths into the Markham Canyon.

Liquefaction analyses performed using the shallow CPT soundings indicated the presence of liquefiable soil at all CPT locations within the vicinity of the pipeline route. Further, liquefaction analyses performed using the shear wave velocity data onshore near the mix tank area showed that, while most of the shear wave points indicated “no liquefaction”, few points indicated “liquefaction” extending to depths of up to about 35m. Liquefiable soil based on V_s analysis are located in discontinuous layers at depths up to 35m. Within these 35m it is possible that some depth intervals within the deposit will not experience liquefaction.

The updated slope stability models and data analyses concluded that the factor of safety for static and seismic conditions met geotechnical engineering standards. However, the factor of safety (FoS) for post-liquefaction analyses were well below 1.1, indicating the potential for instability.

Given that the calculated post-seismic FoS is less than 1.1, flow slide is anticipated in the event of an earthquake with estimated lateral deformation in the order of several meters. However, based on the geophysical data, traces of deep-seated failure surfaces were not observed and it is likely that such a flow slide will be shallow. Therefore, it has been demonstrated that the DSTP pipeline is designed to withstand the post-seismic forces on the pipe/ballast blocks due to potential soil movement. An assessment of the forces on the pipeline due to liquefaction of the soil beneath the ballast blocks (Section 6.8) has confirmed that the pipelines’ tensile strength (wall thickness and material selection) is adequate to resist yielding failure.

6 ENGINEERING ANALYSIS AND FRONT END LOADING

6.1 Front End Loading

The objective of Front End Loading (FEL) is to gain a detailed understanding of the Project as early as possible in order to minimise the number of changes during Project Execution. FEL proceeds across the project life cycle until a full design-basis package is completed.

6.2 Flood Mitigation Design and Analysis

A rainfall-runoff hydrological analysis was completed to estimate the Busu River flood hydrographs during various extreme flood events. The flood hydrographs were then routed through approximately 8km of the Busu River to determine the extent of flooding. Corresponding results were used to evaluate the potential impact of flooding episodes on the Outfall Area and to inform the required flood protection measures to prevent impacts to the proposed facilities.

The terrain surface for the Busu River was developed from LiDAR data (0.5mx0.5m) provided by WGJV. Bathymetric data from the Busu River was incorporated into the terrain surface for the last 8km of the Busu River upstream of the Huon Gulf.

6.2.1 Hydrology and Hydraulic Analyses

A hydrological analysis was conducted in order to estimate the design return period flood magnitudes from the Busu watershed, based on available rainfall data. A climate data analysis informed an estimate of the extreme rainfall events. Available climate data in the vicinity of the Busu watershed includes:

- World Bank (WB) group climate change knowledge portal and precipitation flux data (daily interval) from 1961 to 1999;
- National Centers for Environmental Information (NOAA) [monthly averages];
- PNG Remote Sensing Centre Ltd, (10 minute interval) from 2014 to 2016; and
- Wafi camp rainfall data (daily interval) from 1990 and 2017.

Flood hydrographs from the rainfall runoff analysis were input into a hydraulic model to be routed through the last 8km of the Busu River in order to assess the flood impact on the mix/de-aeration tank location. It was found that while the 20 year flood will not inundate the choke station and mix/de-aeration tank location, the 50 year flood will overtop the Busu River overbank and flow towards the mix/de-aeration tank location.

6.2.2 Flood Mitigation Design

The existing grade elevation at the Outfall Area prior to implementing flood inundation protection procedures is around 5001 – 5002m (i.e. 1-2m above MSL). In order to avoid flooding of the Outfall Area due to the Busu River 200 year flood event, the existing ground surrounding the choke station and dry moat will be elevated to 5003.5m (i.e., about 3.5m above MSL), which leaves a freeboard of about 0.6m. The elevation of the finished grade immediately adjacent to the dry moat has been elevated to 5003.76m (i.e. an additional 0.26m), and this gradually slopes away from the dry moat to the aforementioned elevation of 5003.5m. The top of the secant pile wall around the dry moat will be at an elevation of 5003.91m, which provides an additional freeboard of 0.15m above the top of the finished grade immediately adjacent to the dry moat. The raised elevation around the dry moat and choke station will also serve to protect the dry moat and choke station during storm surge or tsunamis by providing approximately 3m of freeboard above highest astronomical tide. Velocities adjacent to the mix/de-aeration tank location from a 1 in 200 year flood of the Busu River were calculated to be relatively small (maximum of 1.3m/s).

6.3 Structure Design: Mix/De-Aeration Tank Dimensions

The mix/de-aeration tank has an internal diameter of 14m with an inner area of 154m^2 (500mm thick reinforced concrete walls). The tank bottom is situated 8.25m below MSL, and the tank walls are 15m in height (extending to 6.75m above MSL), with the top of the tank being covered with a steel grating to allow personnel to walk across the top. The elevation of the tank bottom, as well as the lateral and vertical dimensions, are dictated by the hydraulics of the passively operating system. There will be two access points in the steel grating to allow entry into the inside of the mix/de-aeration tank, and for allowing servicing/maintenance. A safety handrail will be located along the top perimeter of the mix/de-aeration tank as a safety precaution.

One ROV launcher pipe (Y-Joint with 20m segment of DN750 Sch10 rubber lined steel pipe) extends from each outfall pipeline to the top of the mix/de-aeration tank in order to enable personnel to safely insert an ROV into the outfall lines from an elevation above sea level. The ROV inspection is to be conducted during maintenance periods when there is no inflowing tailings into the tank.

A clearance of 0.5m is provided between the tank bottom and the bottom of the intake and outlet pipes. This ensures that if there are any solids that settle to the bottom of the tank, then these will not present a risk of blocking the seawater intake check valves or creep towards the outfall pipe (although such deposition is not expected, i.e. Section 6.3.2).

The DSTP outfall pipes will enter the tank on the south-southwest side of the tank, separated by a circumferential distance of 2.3m (centre-to-centre). The tailings feed pipe will enter the tank 1.75m from the tank bottom on the east side of the tank. The seawater intake pipes both enter on the south-southeast side of the tank 0.5m from the tank bottom, separated by a circumferential distance of 1.8m (DWG-30202 and -30203). This arrangement ensures the pathlines of the inflowing slurry will be intersected by those of the inflowing seawater, which converge in a turbulent manner within a semi-isolated internal baffle region. This internal baffle region comprises approximately 23% of the cross-sectional area of the tank for the bottom 2.5m of the tank. A 500mm-thick 3-panel wall (2.5m high) separates this region from the remainder of the tank.

6.3.1 Detrainment of Air

The tank diameter, orientation of inflowing and outflowing pipelines, and baffle wall size and position were selected to promote mixing, increase the time inflowing tailings spends within the tank, and promote the release of any entrained air in the slurry to the atmosphere.

Bubble rise velocity is proportional to bubble size (Calderbank, 1967; Ohta et al., 2011; Van Rijn, 1993; Celata et al., 2007) and therefore the characteristic rise velocity of smaller bubbles provides the threshold for the downward velocity in the tank. Based on a combination of experience and literature review, minimum median bubbles could potentially be as small as 0.5mm in diameter. Using equations provided by Calderbank (1967) and Van Rijn (1993), assuming a conservative minimum air bubble diameter of 0.5mm, and not accounting for the upward momentum of the slurry/gas mixture provided by the internal baffle, the minimum expected bubble rising velocity is around 5.9cm/s, which is twice the maximum expected tank fluid level average downward velocity (calculated as the cross-sectional area outside of the inner baffle zone divided by the maximum outflow from the tank). Therefore if any air enters the mix tank via the inflowing tailings pipeline, which should not be a regular occurrence under normal operation, this air should readily escape into the atmosphere.

This functionality has been further evaluated with a CFD model (Section 6.3.2).

6.3.2 CFD Modelling of the Mix/De-Aeration Tank

A multiphase Eulerian-granular computational fluid dynamics (CFD) model was constructed using ANSYS Fluent in order to simulate the turbulent mixing of seawater and tailings within the mix/de-aeration tank. The scenario evaluated as an example in this section corresponds to the Operating Maximum tailings throughput rate for years 4 – 27.

6.3.2.1 Flow paths of tailings and residence time within mix/de-aeration tank

Figure 6.1 shows the pathlines of slurry coloured by tailings solids velocity. The inflowing tailings is mixed around within the inner baffle zone before becoming entrained by inflowing seawater towards the free surface, dispersed around the tank and eventually towards the outfall pipelines. This flow path maximizes the distance traveled by the tailings within the tank before the diluted tailings/seawater mixture is carried to the outfall pipelines.

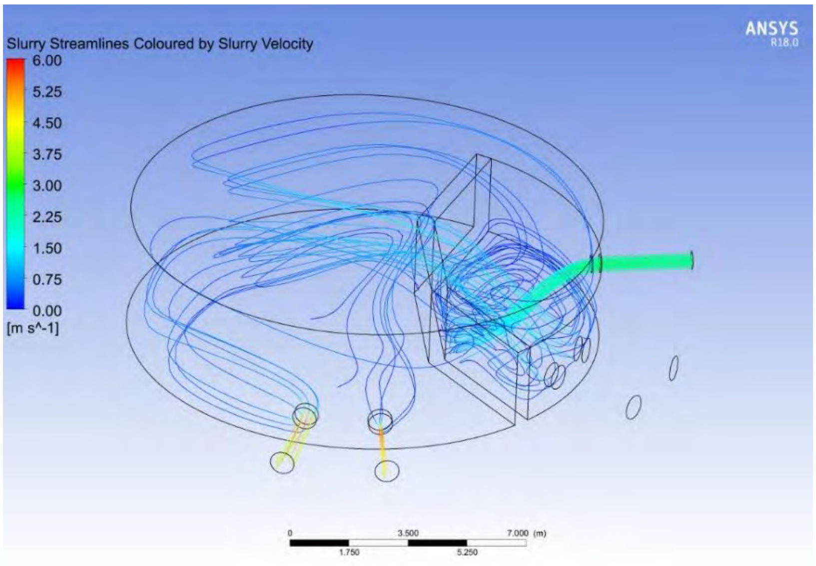

Figure 6.1: Seawater streamlines coloured by seawater velocity

The air detrainment potential can be assessed by the residence time (estimated average time that a "parcel of tailings fluid" spends in the tank) and by the vertical velocities within the tank. By evaluating elapsed time along streamlines, the average residence time of tailings within the tank is between 100 – 200 seconds, but typically around 200 seconds (i.e. no short circuiting of tailings within tank). It requires about 67 seconds for a 0.5mm air bubble to rise from the bottom of the tank to the top of the tank, if the fluid were stagnant. In comparison, the shortest observed duration for tailings to reach the outfall pipes is about 75 – 100 seconds. Furthermore, a comparison of the average vertical velocities inside and outside the baffle zone show that average upward velocity of tailings within the baffle region is more than twice the average falling velocity of tailings outside the baffle region, thereby indicating that the baffle wall is providing excellent potential for air detrainment from the inflowing tailings (i.e. redirecting inflowing seawater/tailings towards the free surface).

6.3.2.2 Concentration and deposition of tailings solids in mix/de-aeration tank

The CFD model simulates the processes of deposition, scouring, and re-suspension. There is predicted to be a very minor deposition pattern observed in the bottom of the mix/de-aeration tank, with a small amount of deposition (i.e. $<1.5\times10^{-4}$m) predicted to occur below the slurry feed pipe, below one of the seawater intake pipelines, and below the outfall pipelines. As the simulation continued, a pattern of deposition and scour was observed wherein the overall pattern remained indifferent.

The solids concentration of the majority of the fluid within the large mixing zone above 0.1 – 0.2m is equal to the fully diluted outfall concentration of 5.7% v/v. Within the inner baffle region the solids concentration decreases from 20% v/v at 0.2m height to 5.7% v/v at the top of the baffle wall. The highest concentrations occur within the inner mixing zone along the tank walls adjacent to the inflowing pipes and within the larger tank area along the outside wall adjacent to the outfall pipes.

6.3.2.3 Scour and Wear Potential of Inside of Mix/De-Aeration Tank Walls

One of the reasons that concrete was selected as the material with which to construct the mix/de-aeration tank walls was its high resistance to corrosion and wear. The shear stress along the walls of the mix/de-aeration tank is directly proportional to the forces responsible for causing surface deformation. Reported permissible shear stress values for various forms of concrete range between 598.5Pa and 980Pa, while the reported maximum permissible velocities for similar materials range between 2.5m/s – 4.6m/s (California Department of Transportation, 2014; American Excelsior Company, n.d.; IOWADOT, 2017; NPTEL IIT Kharagpur, n.d). In comparison, the permissible velocity to avoid scour of vitrified tiles is 4.5m/s – 5.5m/s, indicative of its higher wear resistance (NPTEL IIT Kharagpur, n.d).

The maximum tangential velocities along the baffle wall are around 1m/s, while absolute maximum tangential velocities around 3m/s occur along the tank wall adjacent to the outfall pipes. The largest corresponding observed shear stress values are between 12Pa and 31Pa. These values are well below the design limits discussed above.

A criterion incorporated into the design is that the fluid level in the tank must remain at least 30cm above the top of the baffle wall during steady-state operations to ensure that flow behavior over the top of the baffle wall does not introduce opportunity to entrain additional air towards the outflow, as may occur if the flow becomes non-steady and sporadically billows over the top of the baffle wall at low fluid levels. Furthermore, if the tank level drops to within 30cm above the top of the baffle wall, the maximum tangential velocity and shear stress values along the baffle wall increase by approximately 3-fold (maximum of 30Pa, which remains safely below design limits).

The inclusion of ceramic tiles within the inner mixing zone and the stainless steel and 28% chrome alloy re-entrant nozzles for the intake and outfall pipes respectively will help ensure there is limited scouring over time of the mix/de-aeration tank walls or nozzles. It is recommended to consider including ceramic tiles around the wall of the mix/de-aeration tank adjacent to all of the nozzles, where the localized high shear stresses, tangential velocities, and normal velocities occur.

6.4 Structure Design: Dry Moat

The mix/de-aeration tank must be situated within a dry moat in order to accommodate its base being 8.25m below sea level, the configuration of pipelines around the mix/de-aeration tank, and provide access for maintenance to key instruments/connections. Rather than passing beneath the ROV launchers, the outfall pipelines deviate from a straight path towards the mix/de-aeration tank and enter (orthogonally) the south-west side of the tank. The seawater intake pipelines also enter the tank orthogonally on the south-east side of the tank. Amongst this arrangement of pipelines, there are several instruments and valves, as well as the tee-connections on the seawater intake pipelines, that may need to be accessed over the life of the mine. Hence the mix/de-aeration is positioned within a rectangular (31.9m x 21m) dry moat that is 12.16m deep.

In terms of civil works required for construction of the dry moat (excluding the mix/de-aeration tank and additional facilities within the moat), the primary components are:

- **Ground Preparation:** Free-draining well-graded granular fill material will be delivered to site and dynamically compacted in a maximum of 300mm thick lifts to 95% of Standard Proctor Maximum Dry Density to raise the existing ground elevation to 5003.5m.
- **Raft Slab:** A 700mm thick reinforced concrete raft slab, designed to support the self-weight of the tank and other facilities within the dry moat, as well as the operational

loads (i.e. weight of fluid in mix/de-aeration tank, equipment, and other live loads). This slab contains a 600mm deep sump in the northeast corner, and has concrete pads located on the surface for the pipe (ROV launcher), stairs, and stile supports.

- **Secant Pile Walls and Deadman Anchors:** Secant pile retaining walls are proposed to support the sides of the dry moat excavation. Because the exposed height of the wall (12.2m) is greater than the maximum practical height for a cantilevered secant pile wall, tieback construction wire ropes supported by deadman anchors are required. A 1x1m continuous concrete pile cap extends along the top of the deadmen.
- **Shotcrete Facing:** 250mm thick reinforced shotcrete facing will provide a smooth, water-tight facing to walls within dry moat and also provides long-term resistance to external soil/water pressures on outside of dry moat wall.
- **Perimeter waler beam and temporary restraining steel frame for south side of secant pile wall:** The construction wire rope extending from each deadman pile surrounding the dry moat is attached to a waler beam embedded within the shotcrete facing on the inside of the dry moat wall at 2.5m distance from the top of the secant pile wall.
- **Jet Grout Plug:** Due to the depth that the dry moat will extend to below the groundwater table, and the presence of highly permeable granular soils (gravelly sand and sandy gravel as shown on Advisian soil log from the mix/de-aeration tank location), a 7m thick jet grout plug will be installed to reduce seepage of groundwater into the excavation and provide resistance against uplifting buoyancy forces.

The geotechnical analyses were performed using the following tools:

- **Sheet pile wall:** Conventional analyses based on the Coulomb's earth pressure theory, which was used to calculate the driving and resisting soil lateral pressures on the wall, were performed using the computer program, SPW911 version 2.40.
- **Deadman anchor piles:** Pile lateral deformation analyses were performed for an elastic section using the commercially available software LPile version 2013 by Ensoft, Inc. LPile calculates pile deformations, shear forces and bending moments with depth.

The soil friction angle (36°) and secant pile wall-soil interface friction angle (18°) were informed by Advisian soil logs, i.e. SBH-5 at the Outfall Area location.

This structural design ensures that the dry moat that houses the mix/de-aeration tank remains dry (i.e. leak-proof). Therefore, given that this moat is completely sealed, it also serves as a containment area with a total volume of about 6000m^3 (excluding the volume occupied by the mix/de-aeration tank). This volume equates to containment of 2.5 hours of inflowing tailings from the terrestrial pipeline at the maximum design operating flow rate. A sump will be located within the dry moat, in the northeast corner, in order to pump any accumulated rainwater and/or overflow/spill from the mix/de-aeration tank back into the tank. This pump has been selected with capacity to manage the 200-year rainfall event.

The dry moat will have two access stairways in order to provide access between the top and bottom of the dry moat. Two walkways also extend from the surrounding terrain around the dry moat to the top of the mix-de-aeration tank in order to provide access to the top of the tank. A safety handrail will wrap around the perimeter of dry moat. The pipes within the dry moat all have walkover ramps positioned over them in order to provide safe ease of access around the large pipes within the dry moat.

6.5 Seawater Intake and DSTP Outfall Piping Design

The important design considerations with regard to the sea water intake and tailings outfall pipelines are the selection of pipe routes, dimensions for the pipes, and the pipe material.

The seawater intake pipes selected are the largest available HDPE SDR 11 pipeline that is manufactured. The selection of this pipeline size is predominantly governed by minimizing head losses (i.e. larger diameter pipelines have lower throughput velocities, and hence lower frictional losses), which impact on operating fluid level and dilution achieved in the mix/de-aeration tank. The diameters of the seawater intake pipelines and outfall pipelines have been selected such that the operating level of the tank is satisfactory and the resulting energy balance in the passive system can achieve the nominal dilution ratio of 4:1 (four parts seawater to one part tailings) for all years of operation, without requiring operator intervention. Additional consideration in the outfall pipe selection is made to ensure the velocity of diluted tailings in outfall pipe is above expected depositional velocity.

The pipe material and thickness (HDPE SDR11) was selected for its resistance to expected wear over the life of the mine, its ductility as well as its strength. The latter two parameters are critical characteristics for a DSTP pipeline during installation (S-bend and J-bend sinking), as well as during its lifetime in order to withstand various environmental stresses (i.e., seabed movement, seismic forcing, currents, etc.). Additional criteria is outlined in Section 6.7.3. SSE (2017d) predicted the following expected wear rates in the terrestrial pipeline (HDPE lined pipeline):

1. 0.067mm/year at normal operating velocity of 1.65m/s
2. 0.1mm/year at maximum operating velocity of 1.81m/s
3. Where velocities are 5% higher, the wear rates are approximately 15% higher
4. 6mm/year for a slack flow velocity of 7.5m/s

Based on this evaluation, although the outfall pipeline velocities are just over twice the velocities in the terrestrial pipeline, the tailings are diluted at a 4:1 ratio in the outfall pipelines, and thus it is expected that the wear rate would be on the same order or less than those noted above. The design flow velocities in the outfall pipelines are 3.0 – 4.0m/s and in the intake pipelines are 1.9 – 2.6m/s.

6.5.1 Solids Depositional Velocity

The deposition velocity of suspended solids in the outfall pipeline is the minimum velocity that must be maintained in the pipe in order to prevent deposition of solids in the pipe and consequent increases in head requirements, hydraulic instabilities and pipeline wear. The deposition velocity depends on: particle size, concentration of particles in slurry, density of the particles and carrying fluid, pipeline diameter, and pipeline slope. The rheology parameters utilized in calculations is based on analysis from Slurry Systems (Section 5.2) for the Final Product Sample: solids specific gravity is 2.82, the median particle diameter for the tailings is 50μm, p80 is 93μm.

Many studies have been conducted and formulae developed for determining settling and depositional velocities of suspended solids in slurry transport (i.e. Turian (1987), Roitto (2014), Shook & Roco (1991), Matourek (2011), Bbosa (2016), Wilson (2006;1992), Shook, Gillies, and Sanders (2002), Gillies et al. (2000), Thomas (1979) as discussed by Wilson (2006), Etchell (1986) as discussed by Shook&Roco (1991), Durand&Condolios (1952) as discussed by Roitto (2014), Schiller&Herbich (1991) as discussed by Roitto (2014), Jufin&Lopatin (1966) as discussed by Matourek (2011), Wasp& Aude (1970) as dicussed by Roitto (2014)). Based on the most relevant formulae, for particles with similar values of d_{50}, the average calculated expected deposition velocities in a horizontal pipeline lie between 1.4 and 1.8m/s. When considering particles less than 74μm to be part of the liquid fraction (Shook & Roco,1991), the depositional velocity for the coarse fraction of particles (+74μm) was also found to be within this aforementioned range based on three different formulae. Beyond the shoreline, in the sloping portion (-15%) of the outfall pipeline, the expected deposition velocities may be reduced by at least 20% (Wilson, 1992).

SSE also determined the predicted deposit velocity to be 1.07m/s for the Final Product sample, with p80 of 93μm at C_w of 11-15.3%. The minimum recommended velocity in the outfall pipelines is thus 1.4m/s. Note that design operating velocities in the outfall pipelines (3-4m/s) are well above these recommended minimum operating velocities, and hence it is expected that the tailings particles will remain in suspension.

During any short-term or extended term shut-down periods, it is prudent to connect a hired pump to each of the tees on the seawater intake pipelines within the dry moat and to pump seawater through the outfall pipelines in order to ensure that these pipelines are cleared of any deposited solids. Note that during freshwater flushing of the terrestrial line the corresponding velocity in the outfall pipelines is only around 0.74m/s (i.e. any remaining solids in the outfall pipelines could settle if not manually flushed). Following flushing, an ROV should be sent down each outfall pipeline in order to inspect these pipelines and ensure that they have been cleared of all tailings solids.

6.5.2 Hydraulic Components and Instrumentation

The proposed system will have several hydraulic components designed to optimize operation of the mix/de-aeration tank and slurry outfall characteristics under the projected range of slurry feed throughputs, and also to help enable safe and efficient control of the system. In this regard, the design includes:

- Bar screen on intake end of each seawater (SW) intake pipeline to prevent foreign material from entering pipelines;
- Electronic flow meter on each SW intake pipeline and outfall pipeline;
- Check Valve on each SW pipeline located on inside of tank wall to prevent backflow of tailings or tailings-SW mixture into the intake pipelines;
- Two butterfly valves on each SW intake pipeline, located (1) outside tank wall and (2) adjacent to dry moat wall, to enable shutdown and isolation of SW intake pipelines for maintenance/emergencies (maintained in 100% open position for normal operations);
- One pinch valve on each outfall pipeline. These valves are maintained in 100% open position for normal operations based on expected tailings throughput. These valves lend additional control to passive system if required.
- Two knife gate valves on each tailings outfall pipeline, located (1) outside tank wall and (2) adjacent to dry moat wall, to enable isolation of the pinch valves, ROV launchers, and outfall pipelines within dry moat, for maintenance;
- One Y-Joint with 20m segment of DN750 Sch10 rubber lined steel pipe extending from each outfall pipeline up to top of handrail above tank platform to allow ROV access to each outfall pipeline for inspection and maintenance. The ROV launchers also serve as standpipes (i.e. air release valves) on the outfall pipelines to release any trapped air from the outfall pipelines;
- Ultrasonic level monitor (in conjunction with PLC) to monitor fluid level in tank and trigger an alarm if the tank level falls/rises to levels below/above specified thresholds;
- Two pressure transducers connected to side of mix/de-aeration tank in a high/low arrangement to determine fluid density in the tank, and also to serve as a redundant level measurement (monitored by PLC).

6.6 Hydraulic Analysis

The mix/de-aeration tank and pipelines have been designed to operate passively (i.e. via gravity alone) and to optimize target dilution and detrainment of air for the final approved mine production schedule tonnage, considering tailings rheology test work by SSE (Section 5.2).

6.6.1 Steady State Operation

Operating characteristics of the system have been assessed under steady-state operating conditions. A summary of the critical operating parameters is provided in Table 6.3 below.

Table 6.1: Mix/de-aeration tank, intake/outfall pipe design parameters steady-state operation

Parameter	Years 2 – 3 912 – 1025tph @ 35% w/w	Years 4 – 27 1824 – 2050tph @ 55% w/w
Inflowing Slurry %v/v	16%	30%
Inflowing Slurry Flow Rate (m^3/hr)	2022 – 2272	2144 – 2409
Inflowing Slurry Density (kg/m^3)	1289	1547
Inflowing Seawater Flow Rate (m^3/hr)	7207 - 7355	9554 - 9774
Outfall Slurry Flow Rate (m^3/hr)	9230 – 9626	11700 – 12182
Outflow Slurry Concentration %v/v	3.5 – 3.8%	5.5 – 5.9%
Outfall Slurry Density (kg/m^3)	1080 – 1085	1118 – 1126
Outfall Slurry Velocity (m/s)	3.04 – 3.17	3.86 – 4.02
Mix Tank Surface Level (m below sea	2.54 – 2.64	4.23 – 4.41

During years 2 – 3, the dilution achieved in the mix/de-aeration tank varies between 3.2 – 3.6 parts seawater to 1 part tailings (i.e. slightly less than 4:1), whereas in years 4 – 27 the dilution is 4.1 – 4.5 parts seawater to 1 part tailings (i.e. slightly higher than 4:1). However, the outfall characteristics highlighted in Table 6.3 show that although the dilution achieved in the mix/-de-aeration tank during years 2 – 3 is slightly less than in years 4 – 27, the outflow tailings solids concentration (i.e. 3.5 – 3.8% v/v) is still less than in years 4 – 27 (i.e. 5.5 – 5.9%) when the tailings is diluted slightly more. Analysis based on various dilution ratios between 2:1 and 8:1 have shown that all scenarios achieved similar dilution after about 100m from the outfall and, therefore, variations in initial dilution should not influence the ability of the Project to achieve the required total dilutions within the conservative 2,200m mixing zone proposed. The outfall densities for all years of operation are well above the ambient sea water density at 200m depth.

The passively operating system will have a varying fluid surface level during operation, the timing of which will coincide with the local tidal cycle as described below. During regular steady state operation (not flushing), the fluid surface level in the tank will be:

1. Less than or equal to 8.7m below top of tank when tide is at heighest astronomical tide (HAT) (no overtopping risk)
2. Greater than or equal to 1.9m above top of outlet pipes when tide is at LLW (i.e. 2.5 times the diameter of each outfall pipe, thus ensuring minimal risk of a vortex forming at the outlet pipes and/or air being ingested into the outfall pipelines).
3. Greater than or equal to 0.64m above the top of the baffle wall

The amount of freeboard between the surface level of fluid in the tank and the top of the tank walls for the range of operations can be summarized as follows:

1. During regular operation, the tank fluid surface level will fluctuate depending on the inflowing flow rate, density, and tide level: 8.65m to 11.85m of freeboard will be provided. (i.e. significant freeboard to ensure the tank does not overflow).
2. During a planned short-term shut down (no freshwater enters the tank), when sea level is at HAT (5000.6m), there will be 6.15m of freeboard in the tank.
3. During freshwater flushing, the fluid surface level will be 5.5m above sea level; there will be a minimum 0.65m of freeboard provided at HAT (5000.6m). During a 17 hour flushing period, it is expected to observe water levels in the tank at or above 14.15m depth (when sea level is at or above mean higher high water (MHHW)) for a period of up to about 5 hours.
4. If a freshwater flush is occurring and the operator shuts down the procedure and closes the terrestrial pipeline's terminal valve, leaving the tank full of freshwater, then the fluid surface level in the tank will fall from 5.5m above sea level to 4.9m above sea level. At HAT (5000.6m), this corresponds to 1.24m of freeboard.

The operating fluid level in the mix/de-aeration tank will vary between 2.5m and 4.4m below sea level (i.e. 3.1 – 7m depth), depending on tailings throughput. Under normal conditions, the sea level ranges tidally between Lower Low Water (LLW) and HAT; a difference of 1.3m based on hydrographic charts for Lae. This is supported by nine months of observed data, measured at Lae Yacht Club where the total range in tidal level is 1.19m; the maximum level is 1.25m above lowest astronomical tide (LAT), and the lowest level observed was 0.06m above LAT. Based on these observations, values above MHHW (0.4m above MSL) occured only 10.5% of the time and values below mean lower low water (MLLW) (0.5m below MSL) occured only 0.6% of the time.

An allowance for tidal fluctuations (1.3m), plus an allowance for storm surge (~0.5m), and an allowance for tsunamis have been accounted for in determining the tank wall height. The maximum recorded tsunami wave height near Lae is 2.4m, generated by a submarine landslide in 1972 near Voco Point (Buleka et al., 1999). During regular operation, the freeboard provided by the mix tank walls above regular steady-state fluid operating levels should ensure that, even during storm surge or a potential tsunami, the tank walls will not be overtopped by the fluid within the tank.

Following a planned extended shutdown period, the mix/de-aeration tank will receive freshwater flushing from the terrestrial pipeline for approximately 17 hours, followed by tailings. During this time, there is no inflow from the seawater intake pipelines. It is important that the operator does not initiate this start-up sequence if there is immediate risk of a tsunami or large storm surge occurring within the following day while freshwater is flowing into the mix/de-aeration tank, as there would be a potential risk in this case alone of overtopping the tank walls. Also, as described above (4), if the flow of freshwater into the mix/de-aeration tank has already been initiated, then upon receiving a tsunami warning, the operator can shut down the flow of freshwater into the tank in order to increase the available freeboard between the fluid level in the tank and the top of the tank walls to between 1.24m and 2.54m (depending on tide level). Note that if the tank were to overflow at this time, the overtopping fluid would only be freshwater, and would be contained within the dry moat. Once the event has passed, the sump would be used to pump this overflow (freshwater) back into the tank. This procedure can be carefully planned for by the operator, thereby significantly reducing the risk of this event occurring in practice.

6.6.2 Transient Analysis

Transients, such as those that occur during start-up and shut-down scenarios or when transitioning from tailings to freshwater (or vice versa) in the tailings feed pipeline, will result in momentary fluctuations in the fluid surface level in the mix/de-aeration tank that are proportional to the flow rates involved, as well as the magnitude of the change in inflowing fluid properties (Table 6.4). The terrestrial pipeline terminal valves can be opened/closed in 15 seconds, which is used as the design transition period for evaluating transients.

Table 6.2: Mix/de-aeration tank and slurry outfall pipe design parameters during transients

Parameter	Years 2 – 3 912 – 1025tph @ 35% w/w	Years 4 – 27 1824 – 2050tph @ 55% w/w
Max Transient Fluid Level (m above sea level)	0.45 – 0.51	0.43 – 0.49
Min Transient Fluid Level (m below sea level)	2.98 – 3.14	4.67 – 4.84
Max Outfall Velocity During Transients (m/s)	3.49 – 3.64	4.69 – 4.89

The maximum transient fluid levels in the tank (Table 6.4) will be approximately 5.6 – 5.7 meters below the top of the tank during HAT, and thus present no risk of overtopping the tank. At this time, there is also a short-duration surge in the velocities in the outfall pipelines. However the values are well below 5m/s, which is considered to be a conservative operating threshold to prevent scouring given the low concentrations of solids in the outfall pipelines and small median particles size (d50 = 50μm), which help to reduce expected pipe wear rates (Goddard, 1994; Badr et al., 2002).

The tank level should not be allowed to fall within one pipe diameter of the top of the outfall pipelines or to the height of the top of the baffle wall (2.5m above tank bottom) (Section 6.3.2). During years 4 – 27, the absolute minimum transient fluid height is predicted to be 2.7m if tide is at LAT (most challenging tidal condition), remaining 1.5m above the elevation of the top of the outlet pipes and 0.2m above the top of the internal baffle wall. Although this is lower than the 0.3m Project design criteria (Section 6.3.2), the fluid level will be below the normal operating level for a short duration. This is considered acceptable and to present negligible risk of scouring the baffle wall or causing air to become entrained into the outfall pipelines. This transient is associated with a controlled/planned change from a short-term shut-down state to normal operation, and hence its coincident occurrence with LAT can be avoided. When sea level is at or above MLLW, the lowest transient levels in the tank for all throughput scenarios do not fall within 30cm of the top of the baffle wall.

In conclusion, the tank fluid level is expected to always remain within safe operating limits (i.e. below the top of the tank walls and above the lower threshold limit at which risk of vortex formation and/or air entrainment into the outfall pipelines, or scouring of the internal baffle wall becomes of concern), and the outfall pipeline velocities are expected to remain below the limit at which higher rates of scouring may occur and above the thresholds at which deposition of suspended sediment in the pipelines is expected to occur.

6.6.3 Scenario Analysis and Operational Considerations

Several additional scenarios have also been investigated in order to assess the resiliency of the system to any potential detrimental events or occurrences that could develop over the life of the mine, although these are considered to be unlikely:

- Blockage of one or both of the seawater intake pipelines;
- Development of biological growth within the intake pipelines (i.e. biofouling);

- Blockage of one or both of the outfall pipelines;

The safe operating level for maintaining a sufficient buffer over the top of the baffle wall was taken as 4.95m below sea level (i.e. the minimum permissible operating point in the mix/de-aeration tank during regular operation based on MLLW).

6.6.3.1 Blockage of One or Both Seawater Intake Pipelines

Two scenarios that were investigated were the events that foreign material causes (1) a partial blockage of both seawater intake pipelines; or (2) a complete blockage of one or both of the seawater intake pipelines. In order to help prevent the seawater intake pipelines from being blocked by deleterious objects, the seawater intake pipelines both have bar screens on their inlets.

In order to evaluate the first scenario (1), a stepwise blockage was assessed, simulated as a sharp-edged thin orifice, in accordance with Miller (1994). As the seawater intake pipelines become progressively blocked by foreign material, the seawater inflow rate and dilution would decrease and the mix/de-aeration tank surface fluid elevation would fall.

The design of the system allows for a partial blockage of both seawater intake pipelines to occur, while still operating safely (i.e. with an operating fluid level that is more than 30cm above the top of the inner baffle wall and hence also more than two pipe diameters (+1.55m) above the elevation of the top of the outfall pipes). The blockage limit for the seawater intake pipelines before the tank level falls below the minimum design level for safe operation and should be shut down to remediate the blockage, is as follows:

- Years 2 – 3: up to 60% blocked
- Years 4 – 27 at 1824tph – 1934tph: up to 40% blocked
- Years 4 – 27 at 2050tph: up to 30% blocked

The DSTP system instrumentation (flow meters, dual pressure transducers on the side of the tank, and ultrasonic level transmitter) will serve as redundant indicators that this incident has developed. If fluctuations in these monitored variables occur for reasons not explained by factors upstream (i.e. rheology, flow rates, etc.), it is recommended that the system be shut-down for inspection and/or maintenance as required. Note that the corresponding fluid levels for these blockages are well above one pipe diameter above the top of the outfall pipelines.

If both of the seawater intake pipelines were to become blocked somewhat instantaneously from a debris flow in the marine environment, then the response of the system would be similar to above; however, the time scale would be shorter. If this event were to occur, the monitoring system (instruments and alarms) would record a sudden flow decrease in both intake pipelines, a monotonic increase in the tank fluid density, and a monotonic decrease in the tank fluid level. If these changes are noted, it is critical that the system be shut down in order to limit the entrainment of air into the outfall pipelines. Assuming about 120 seconds for the control room operator at the main plant to shut-down the terrestrial system, the fluid level in the tank would be at or just above the elevation of the top of the outlet pipes. The fluid level in the outfall pipelines could fall to a maximum expected depth of about 15m below the bottom of the mix/de-aeration tank, which is within the range of which the outfall pipelines are trenched and/or covered by riprap, in addition to ballasting (for 25% air offset). Hence even if the fluid level in the outfall pipelines were to fall to this depth, the outfall pipelines would not rise from the seafloor. Eventually, an exchange-flow would occur in the outfall pipelines and seawater would pass upwards through the outfall pipelines and eventually into the mix/de-aeration tank to an equilibrium level with seawater.

If only one of the seawater intake pipelines were to become blocked suddenly as a result of a debris flow in the submarine environment, the monitoring system (instruments) would note the same changes as above, however, at a slower rate. The rate at which the fluid level in the tank would fall is 2.3 times less than if both seawater pipes were simultaneously blocked. This would provide additional time for the operator at the control plant to take action and prevent the level in the tank from falling below the elevation of the top of the outfall pipelines. It would be recommended in this case that the operator shuts down both the terrestrial feed pipeline as well as one of the outfall pipelines. Assuming the same response and action times as above, under this approach, the fluid level in the tank would remain more than one pipe diameter above the top of the outfall pipeline and would only fall below the top of the baffle wall for a short duration at LAT. If tide were at or above MSL then the fluid level would remain safely above 30cm above the top of the baffle wall throughout this scenario. Maintenance can then be performed to remove any pipe blockage(s).

6.6.3.2 Biofouling

An investigation has been conducted in order to determine what the response of the DSTP system would be if the seawater intake pipelines were to experience a build-up of biological growth. From past experience, it has been found that biofouling can significantly increase the frictional resistance along the inside of the HDPE pipelines. A new, clean pipeline has a Darcy-Weisbach friction coefficient of 1.02e-2; however, a DSTP intake pipeline from an undisclosed project that was subject to significant biofouling was found to have a friction coefficient of 1.86e-2 (i.e. an increase of over 80%). Tetra Tech evaluted the potential effect of a progressive accumulation of biological growth on the inside of the pipeline by considering frictional coefficients from 1.02e-2 to 1.86e-2 for all design throughputs.

During years 2 – 3 (912 – 1025tph at 35% Cw), the seawater intake pipelines can tolerate over an 82% increase in the Darcy-Weisbach friction coefficient, and yet the tank level will remain within the specified operating range. For years 4 – 27, the system can tolerate up to a 24% increase in the frictional resistance within the intake pipelines due to biological growth, before the tank levels would fall below recommended operating levels for both continuous operation and transients. This still offers a considerable margin of safety with regards to the design for several reasons:

- During years 2 – 3, regular planned inspections of the intake pipelines with ROV units will have identified if biological growth within the pipelines is occuring;
- The response of the system to biological growth (i.e. decrease in tank level during regular operation, decrease in intake flow rates, and decrease in outfall flow rates) would indicate this issue to the operator before progressing to this limiting extent;
- The inlets of the intake pipelines are located below the euphotic zone, and hence significant biological growth within the pipelines is not expected to occur;

If the plant is shut down for an extended period of time, then it is recommended that an ROV be guided through both seawater intake pipelines, in order to inspect the inlet locations as well as the walls for accumulation of biological growth (and to remove any if present).

6.6.3.3 Blockage of One or Both of the Outfall Pipelines

The final situation that was assessed, was a submarine landslide or other debris flow causing one or both of the outfall pipelines to become blocked. For this assessment, the operating maximum flow rate was used for example purposes.

If only one of the outfall pipelines were to become blocked, then the tank level would increase by one half of the distance from sea level to the normal operating level, the seawater intake flow rate (and hence dilution) would decrease to 60% of its original value, the density in the mix/de-aeration tank would increase by over 4%, and the outfall velocity though the operating outfall pipeline would increase to over 5m/s. Under these operating conditions, alarms would indicate these changes have occured. It is recommended that the system would be shut down; however, note that under these circumstances the tank would not overflow and could be safely shut down.

If both outfall pipelines were to become blocked, then the tailings feed pipeline must be shut down in order to prevent the mix/de-aeration tank from overflowing. If this event were to occur, then the rate of increase in the tank level is bimodal. It would require about 5 minutes for the tank level to rise from the normal operating level to sea level. Prior to reaching this level, alarms would indicate to the operator that there is zero flow through both outfall pipelines, decreasing flow in the seawater intake pipelines, and a rising fluid level in the mix/de-aeration tank and hence a shut-down procedure would be initiated. Therefore it is unlikely that the fluid level in the tank would exceed this level if this event were to occur and planned emergency procedures were followed. However, if this does not occur within this aforementioned 5 minutes, then once the tank level exceeds sea level, then seawater will stop flowing into the tank through the intake pipelines and the rate of increase in the tank fluid level surface will decrease, requiring an additional 25 – 30 minutes for the fluid within the tank to reach the top of the tank walls. Therefore it is very unlikely that the tank would overflow before the system is shut down. Even if the tank were to overflow, there is a sump pump located within the dry moat around the tank to evacuate the dry moat of the overflow material once the tank is operational again (Section 6.4).

6.7 Pipe Stability and Scour Protection

For a seabed-mounted installation, a combination of trenching, scour protection (riprap), and ballasting is used to ensure the maintenance of pipeline position and stability (i.e. in order to prevent sideway or vertical motion of the pipelines). Ballast blocks are required along the entire pipe length; however, in the near-shore environment (from 5m depth down to 25m depth), scour protection (riprap) is also required in order to prevent the erosion of sediment around the pipelines due to wave action and currents. From the wall of the dry moat down to 10m depth the pipelines are also situated in a trench with approximately 5 – 10m of native fill overtop of them down to 5m depth (where the riprap begins) and then progressively less backfill overburden on top of the riprap from this point until the location where the pipelines daylight at 10m depth.

Accurate near-shore bathymetry (<10m depth) was not available at the time of this report, and thus the nearshore bathymetry along the pipe alignment between the HHW mark and approximately 10m depth has been interpolated between available LiDAR data on the land portion and the Singuau (>10m depth), AUV (>30m depth), and Neptune (>30m depth) bathymetric survey data. Given that the pipeline is contained within a trench for this portion of the alignment, then the overall impact of this is not significant.

6.7.1 Design Storm Conditions

In relatively shallow near shore waters, wave loading on the pipelines can be significant. To estimate the forces resulting from waves, a series of wave models has been developed to produce estimates of wave height, period and orbital velocity along the pipeline route.

To estimate a 200 year return period design storm event at the pipeline location, 10 years of hindcast wind data has been obtained from the 0.5 degree resolution Climate Forecast System Re-Analysis and Reforecast (CRSRR) model, as reported in the National Oceanic and Atmospheric Administration's (NOAA) Wave Watch III global wave model. This model is operated by the National Centers for Environmental Information (NCEP). Wave data for the same period has been obtained directly from NOAA's 0.5 degree resolution global Wave

Watch III model. This data has been combined with 8 years of hindcast data provided by IHAconsult, for a total wind and wave record of 18 years in the Huon Gulf. Based on this data, an extreme value analysis has been undertaken on the wind and wave record to determine the design wind speed and wave height. The 200 year return period sustained wind speed from the south-east was determined to be 22.44m/s, yielding a 3.75m significant wave height from the south east in the central Huon Gulf.

The wave model SWAN was used to estimate the nearshore wave conditions at the Outfall Area. A coarse resolution model of the Huon Gulf driven by Wave Watch III winds and waves at the offshore boundary is used to provide boundary conditions for a fine resolution wave model in the vicinity of the of the Outfall Area. The coarse grid and fine grid models have nominal resolutions of 40m and 3.5m respectively at the Outfall Area. The progression of wind and wave heights during the design storm event was scaled from past storms such that its wind speed and wave height match the design storm levels. Wave period was scaled along with wave height to preserve the spectral steepness (i.e. the relative distribution of wave energy across various wave periods) of the wave field. The pipeline depth, significant wave height [H_s] and near bed orbital velocity [U_{bot}] along the nearshore sections of the pipeline route is presented in Figure 6.2 with the daylight location shown as a red dot and the end of the scour protection shown as a black dot. H_s at 300m depth is 4.8m and H_s at the daylight point in 10m of water is approximately 3.6m. A peak near bed orbital velocity of 1.3m/s occurs in approximately 5m of water, with a peak orbital velocity of 0.9m/s at the daylight location (10m depth). From 10m to approximately 25m depth, U_{bot} decreases exponentially to 0.4m/s (slower decay below this depth, eventually reaching zero at greater depths).

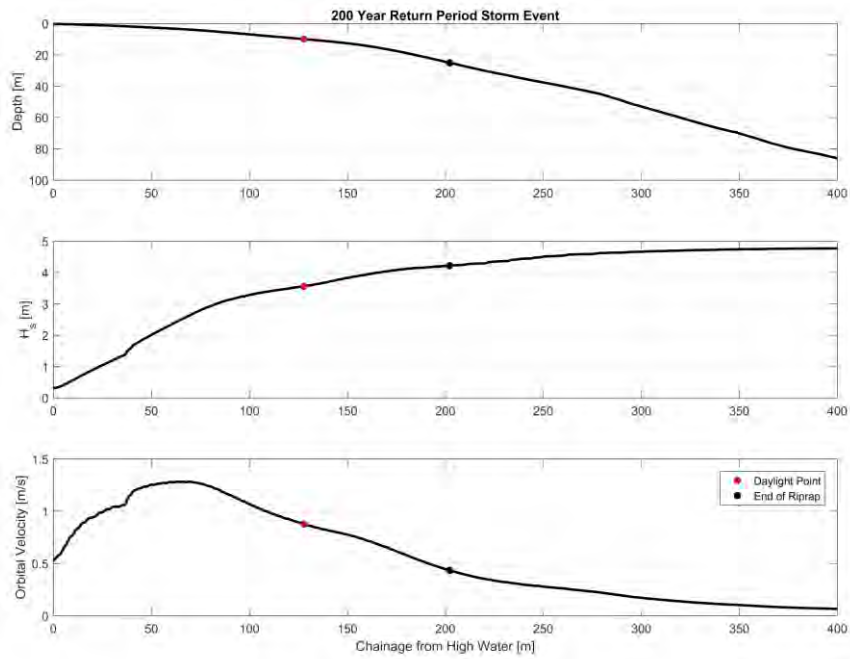

Figure 6.2: Pipeline depth (top panel), significant wave height (middle panel) and near bed orbital velocity (bottom panel) along the nearshore sections of the pipeline route

6.7.2 Scour Protection

The main purpose of the scour protection is to prevent the movement and undermining of the pipelines during the design storm event in the vicinity of their daylight point at about 10m below MSL. Loose rock scour protection, comprised of an armour and filter layer is to be provided on either side, beneath and overtop of both subsea pipelines in waters shallower than 25m. The pipes will be covered by scour protection from the point at which the embedment depth is less than 5m to a depth of 25m. The purpose of providing scour protection for a short section of buried pipeline is to mitigate against seasonal and inter-annual variations in the nearshore morphology, for example the onshore-offshore movement of sand bars or long term erosion. Google Earth imagery (2010), satellite imagery (2016/2017) and local reports suggest a history of ongoing periodic recession and accretion of the shoreline over time at the project site. The purpose of providing scour protection (including a filter layer and an armour layer) to a depth of 25m is to ensure that the scour protection extends well below the depth of significant wave action to mitigate against undermining of the offshore toe of the scour protection layer. It is important to ensure that the gradation of the filter layer rock (75mm minus) is geometrically closed within the armour layer stone (d50 of 0.3m) such that the filter layer is not scoured away from the pipelines within the armour layer and "sucked" through the gaps between the armour stone. The gradation of these materials have been selected based on well-reviewed and used standard ratios such that a geotextile layer should not be required to contain the filter material within the armour layer.

The ballasting on the pipeline, overburden from the gravel filter layer and the native backfill within the trench down to 10m depth (Station 0+0m to 0+305m), as well as the gravel filter layer and the scour protection down to 25m depth will also serve to protect the pipelines during the drawdown and potential air ingestion into the pipelines that may accompany a tsunami by providing additional resistance against buoyancy. Furthermore, the ROV launcher on each outfall pipeline act as large stand pipes, thereby allowing air to escape, rather than be dragged down into either of the outfall pipelines.

6.7.3 Ballast Design

The tailings ocean outfall pipelines and seawater intake pipelines will be saddled by ballast blocks at specified intervals. Concrete ballast is typically designed to keep a pipe submerged despite a certain percentage of the pipe being filled with air. This "air offset" is calculated as the volume of air in the pipe that would cause the system to begin to rise off the seabed, as a percentage of the interior volume of the pipe. A larger offset requires more ballast and implies a more stable system. Submerged systems can be designed with air offsets between 10-100%. The appropriate offset depends on the risk of air entrainment into the pipe. Based on risk analyses (including two HAZOP multidisciplinary workshops) and past project experience, it was determined that a 25% air offset was appropriate for this DSTP system. From the perspective of tensile yield during liquefaction, the designer is motivated to increase the ballast block spacing and decrease the air offset. However, there are practical upper limits to ballast block spacing.

The specified mass, shape and spacing of the ballasting along the pipelines is controlled by several important factors, including:

1. The bottom thickness (height) of the ballast block below the pipeline must be equal to at least half of one pipe diameter in order to reduce the risk of localized high scouring velocities underneath the pipeline and to ensure the pipe remains supported off of the seabed, even in loose/soft granular soils;
2. The spacing (maximum) is designed for a minimum 25% air offset;
3. The spacing (maximum) is selected to accommodate pipe handling on land with the ballast blocks installed such that the bending stress experienced by the pipe with one unsupported block suspended between two adjacent supported ballast blocks is within tolerable design limits for HDPE;

4. The spacing (maximum) is selected such that if the seabed were to slough away beneath three of the ballast blocks, then the bending stress experienced by the pipe suspended between two other ballast blocks while carrying the weight of the unsupported ballast blocks is within tolerable design limits for HDPE;
5. The spacing (minimum) is designed to enable the pipe to float when it is 100% full of air, eliminating the need for additional buoyancy during the sinking process;
6. Consideration of tensile yield during a liquefaction event;

For design of the ballasting system, the following physical parameters were assumed:

- Outfall pipes (900mm OD, SDR11 HDPE) have a linear mass of 213kg/m
- Seawater intake pipes (1000mm OD, SDR11 HDPE) have a linear mass of 261kg/m
- Marine concrete density for constructing the ballast blocks has a density of 2400kg/m^3
- Seawater density surrounding pipelines is 1024.5kg/m^3 (2016-2017 CTD data);
- The design scenario for ballast design occurs when the pipelines contain only seawater with an assumed density of 1022kg/m^3 (i.e. heavier effluent contributes marginally to pipeline stability). Air ingestion during freshwater flushing is not likely given the tank fluid level presents no opportunity for air ingestion into outfall pipelines.
- Lateral drag exerted by currents was represented by a drag coefficient of 1.0 in the standard drag equation for flow around a cylinder. A lift coefficient of 0.9 was applied.
- Average downward slope is 17.6° (local maximum of 24° at station 0+770m);
- Alignment of seawater intake and outfall pipelines are relatively indifferent from each other in terms of critical factors related to ballast design (Aside from length and size).

6.7.3.1 Installation and Handling Constraints

Beam-bending equations were employed to assess the pipe’s bending strength in order to ensure the bending stress experienced during (3) and (4) above is within design limits for the selected SDR11 HDPE pipelines (maximum design stress of 8MPa) (AS/NZS 4131). The allowable bending moment in the pipe was calculated based on bending stresses and on pipe wall buckling. For the proposed outfall and intake pipelines, bending stresses govern. The maximum spacing is 12.3m for a 1320kg block on the outfall pipelines and the the maximum spacing is 13.3m for a 1583kg block (seawater intake pipelines).

6.7.3.2 Environmental Conditions

The consideration of environmental forcing is not independent from the design buoyancy (air offset). If a pipeline becomes more buoyant due to air ingestion, then the impact due to given current and wave conditions on the potential for ballast blocks to slip along the seabed or to tip sideways increases. The bathymetry (i.e. transverse and longitudinal slopes) is also very important for this analysis (i.e. gentler slopes are more stable).

Currents for the design of the ballasting were estimated from the ADCP record at Outfall A, Deployment 1 (23 October to 11 December 2016). The Outfall A location is approximately 2.3km west of the pipe route, 1.1km offshore, in a depth of 300m. The maximum hourly-averaged currents observed during Deployment 1 was 0.22m/s perpendicular to the route in a westward direction. For the purposes of ballast design, a uniform current of 0.22m/s was applied laterally along the full length of the pipe. The effects of waves were accounted for by considering the wave conditions associated with the 200-year design storm wind and wave conditions described above.

6.7.3.3 Summary

In the near-shore field, where the pipelines are within the trench and covered by an armour layer, the ballast block spacing are designed for 25% air offset in terms of buoyancy only. This is due to the fact that these pipelines within this portion of the alignment will be protected from environmental forces (tipping/sliding due to waves/currents) by the trench and armour layer (station 0+000 to 0+350m). In general, this ballasting within this section will also help hold the pipelines in place (as installed) over the lifetime of the mine. The ballast blocks on the outfall pipelines in this section will be spaced at 6.0m intervals with a total mass of 1320kg each. The ballast blocks on the seawater intake pipelines in this section are spaced at 5.0m intervals (center-to center) with a total mass of 1583kg each.

The ballast blocks on the outfall pipelines beyond the trench and riprap section are indifferent in terms of size and shape, and are also designed for a minimum air offset in terms of buoyancy of 25%; however, the spacing will be reduced to 5.0m down to 75m depth in order to account for the maximum possible orbital velocities associated with the design 200-year storm event described above. From 75m depth down to approximately 200m depth (outfall pipe termini) the requisite spacing is once again increased to 6m, given the decrease in environmental forces below this depth. The spacing for the ballasting on the seawater intake pipelines will be constant (5m) along the entire pipeline.

The ballast design is summarized in Table 6.5.

Table 6.3: Ballast Block Spacing

Station	Outfall Pipe	Intake Pipe
0+000m to 0+350m	6m spacing (center to center) between each ballast block	5m spacing (center to center) between each ballast block
0+350m to 0+520m	5m spacing (center to center) between each ballast block	5m spacing (center to center) between each ballast block (terminus at 0+471m)
0+520m to 0+930m	6m spacing (center to center) between each ballast block	-

6.8 Liquefaction Analysis

If the seabed were to liquefy beneath and around the pipelines, then the seabed soil may slide downslope, essentially losing some or all of its ability to support the weight of the DSTP system. In this situation, the soil drag force (exerted on the pipeline and ballast blocks) and the axial component of the self-weight of the DSTP system components are carried by tension in the HDPE pipe itself. The pipe strength must be selected to manage this loading.

The following assumptions were incorporated into this investigation:

- The soil along a portion or all of the length of the pipes could liquefy from the pipe daylighting point at 0+305m to the pipe termini. On the outfall pipe, this full length along the route is approximately 650m (average downward slope is 17°).
- Axial component of system's self-weight is the sum of the buoyant weights of the individual components (pipe, ballast, effluent) multiplied by sine of the slope angle;
- Outfall pipes are full of diluted tailings during liquefaction event, with a density of 1,126kg/m^3 (worst-case scenario, i.e. pipes will be heavier);
- The drag along the pipeline due to skin friction between the liquefied soil and the outer surface of the pipe was calculated as the product of the outer surface area of the

pipeline in contact with the seabed and the residual shear strength of the soil. The soil will move downslope and pile up against the ballast blocks forming a stepwise distribution between each set of ballast blocks relative to the pipeline and therefore less than the full length of the pipe may be in contact with the soil.

- Pipe tension prior to liquefaction event is negligible (relaxation of HDPE over time).
- Liquefied soil has a bulk unit weight of 18kN/m^3 and a residual strength of between 1kPa and 4kPa based on a 1 in 475 year Earthquake Hazard analysis.
- Pipe's tensile yield strength is estimated as the yield stress of PE4710 multiplied by the pipe's cross-sectional area at minimum wall thickness. The tensile yield stress for HDPE is 25MPa (3600psi; per JMM spec sheet for PE4710).
- During a liquefaction event, the concrete ballast blocks (more dense than the liquefied soil) in the liquefied zone will sink into the soil with no resistance, reaching an equilibrium depth determined by buoyancy (i.e. mud-line on the pipe estimated based on geometry and relative buoyancy).
- Liquefied soil provides no vertical support to pipeline, except through buoyancy.

As the liquefied soil moves downslope, the ballast blocks and pipelines will remain effectively stationary since adequately supported at the upslope end; hence the moving soil will pile up behind and eventually move around each of the ballast blocks. This action will produce a pressure force on the upslope face of each block and a drag force on the bottom and sides of each block (as well as on the pipeline itself). The soil pressure on the upslope face of each block was estimated from P-Y curves derived using the commercially available software, LPile. These curves are applicable for cohesive soils with a given residual strength and provide an ultimate pressure for each depth at large deflections, representative of conditions where liquefied soil flows around a ballast block. These ultimate pressures were integrated over the submerged face of the ballast blocks to estimate a total force per block. The soil pressure on the upslope block faces was also estimated in a 2-D simulation of soil deformation in the software Plaxis; pressures predicted by Plaxis were less than those estimated from P-Y curves, indicating that the latter is conservative. The drag force of the soil against the bottom and sides of each block and the outside of the pipeline was estimated by multiplying the residual shear strength of the liquefied soil by the exposed surface area.

The tension in the pipe at the upslope end of the liquefied zone is the sum of the axial forces: drag and pressure on each block, drag along the pipeline, plus the system's self-weight. The factor of safety against tensile yield is defined as the tensile yield strength (5,210kN for the outfall pipe) divided by the predicted tension. Therefore a FoS greater than one implies that the tensile strength exceeds the force exerted, i.e. the strength of the pipe would be sufficient to resist to the simulated liquefaction event.

Although the intake pipes' proposed ballast blocks are larger than those for the outfall pipes, the intake pipes' walls have a larger cross-sectional area and the route is much shorter. Therefore, the outfall pipelines are more susceptible to a higher risk due to liquefaction.

A sensitivity analysis has been conducted in order to determine how the FoS varies in response to varying the input parameters over their expected ranges:

- Residual shear strength of soil: 1 – 4 kPa
- Length of pipeline over which liquefaction occurs: 0 – 650 m (i.e. a range from none to the entire length)
- Fraction of pipeline subject to drag due to shear between the moving liquefied soil and the relatively stationary pipeline: 0 – 100%

The variation in FoS against tensile yield was examined in response to simultaneously considering the range of potential values for the variables discussed above. The minimum FoS against tensile yield for all possible ranges of these three variables is 1.2, indicating that the tensile strength of the pipeline exceeds the tensile force imparted to it for all possible combinations of parameters investigated.

Key elements of uncertainty remaining in this analysis are:

- Soil liquefaction and the resulting flow of material that applies pressure/drag on the pipes/blocks is a complex series of events. The calculation of soil pressures against the ballast blocks during a liquefaction event was carried out using several 'simplified' methods with widely varying results. The calculation method used to produce the results above was demonstrated to be the most conservative;
- The upper layer of soil may not liquefy, or may have sufficient residual strength to support the blocks, in which case soil drag against the blocks could be substantially less than predicted in this analysis and result in a much larger factor of safety; and
- The profile of the soil passing around and between ballast blocks, imparting frictional drag along the sides/bottom walls of the pipeline is uncertain. It has been considered that up to 100% of the pipeline length may be exposed to this frictional drag, although it is expected that it would be less than this, as the soil piles up against the ballast blocks at the downslope end of each interval between the ballast blocks.

6.9 Installation (Civil/Structural) of Marine Components of DSTP System

This section describes the specific materials, design, and construction sequence associated with the marine component of the DSTP system, including but not limited to the mix/de-aeration tank, dry moat (and structural ancillary facilities), trench, and marine pipelines.

The structural design related to the dry moat and mix/de-aeration tank has accounted for design loads in conformance with NBCC 2015 (Canada). All design materials are local and in conformance with Australian standards.

6.9.1 Dry Moat Civil Works Construction Sequence

The secant pile walls will consist of overlapping primary reinforced and secondary non-reinforced piles. First, the secondary non-reinforced piles will be installed by drilling minimum 880mm diameter cased vertical shafts around the perimeter of the moat at 1.5m centre-to-centre spacing. The shafts will be filled with 5MPa concrete and the casing is then pulled. After allowing sufficient time for the concrete to cure and to gain the required design strength, the primary reinforced piles will be installed in cased shafts (minimum 880mm) between (and cut into) the non-reinforced piles (i.e. primary piles overlap secondary piles). The steel reinforcement for the primary piles will consist of a steel wide flange (DWG-30202). The deadman anchor piles will be installed using the same equipment as for secant wall piles.

A cursory review of several wall options was considered, which included sheet pile walls, secant pile walls, diaphragm walls, and reinforced soil mix walls. The secant pile wall option was considered to be the most appropriate for the dry moat for ease of construction, water tightness, and integration into permanent design (i.e. structural beams will be exposed and structurally connected to the finished interior wall surface).

After the secant piles are installed from the existing site grade, a jet-grout rig will be mobilized to construct a jet-grout groundwater cut-off below the proposed underside of the raft slab within the secant pile wall area. The jet-grout cut-off will be constructed by advancing a jet-grout nozzle from existing ground surface down to the proposed elevation at the bottom of the moat. The jet grout nozzle will then be rotated and high pressure grout will be sprayed to create a vertical column of grout that extends to a depth of approximately 7.0m below the bottom of the future structural slab. These jet-grout columns are horizontally overlapped such that a continuous groundwater cut-off is created. The jet-grout is assumed to have a unit weight of 24kN/m^3 and strength of 25Mpa to 40MPa.

Once the concrete for the secant pile wall, deadman anchors, and jet grout plug has set, a pump will be installed to dewater the inside of the dry moat and excavators will be used to remove the material from the inside of the moat. The excavation will proceed down in stages as follows:

1. Excavate within the dry moat to approximately 3.5m below top of secant pile wall.
2. Shave off the inside face of the secant piles to expose the steel beams down to just below 2.5m below the top of the secant piles and install waler beam.
3. Excavate behind the secant pile wall to a maximum depth of approximately 3.5m at the wall location and to about 1.0m depth at deadman anchor locations using trenches as required to facilitate the installation of the walers and tieback wire cables. The tieback cable will be connected to the deadmen pile cap 500mm below the top of the pile cap. To connect the tieback cables to the secant pile wall, drill through the secant pile wall and feed the tieback cable through the hole, seal the hole to prevent water from entering the excavation, install the waler along the interior perimeter of the excavation, then connect the tieback cable to the waler and tension the cable.
4. After installing and connecting the tieback cables, the dry moat area will be excavated to the underside elevation of the proposed raft slab (i.e. top of jet grout).
5. Following these procedures, the secant pile wall and the jet grout cutoff will be checked for leakage; if any leaks are observed, they will be sealed by drilling through and grouting the leak.
6. Install reinforcement and construct the moat bottom, complete with sump, which will act as the finished working surface.
7. Shave off remaining inside face of secant pile walls to expose steel flanges of beams.

The cast-in nozzles must be installed through the secant pile wall no later than the installation of the moat wall reinforcement, and must be installed prior to placing the shotcrete on the shaven inside surface of secant pile wall. The centrelines of these nozzles must align with the (2.75m) gaps between the deadmen anchors on the south side of the dry moat wall. Subsequently, the wall reinforcement and 250mm thick shotcrete wall will be installed. The permanent structural facing will be connected to the flanges with Nelson studs to strengthen the wall, provide sufficient resistance against the external pressures from the surrounding soil, and help seal the inside of the moat.

6.9.2 Mix/De-Aeration Tank Construction Sequence

Upon completion of construction of the structural facing and concrete slab within the dry moat, and once all concrete has fully cured to satisfactory strength, the mix/de-aeration tank may be installed into its desired position, as follows:

1. Install reinforcement and form the mix/de-aeration tank wall and the baffle wall
2. Install step brackets to access the mix/de-aeration tank bottom

3. Install embed plates (must wait for moat bottom slab concrete to cure [~7days] before workers can go on top of concrete slab and start placing rebar and forms)
4. Install five cast-in nozzles no later than installation of mix/de-aeration tank wall reinforcement. These nozzles must be installed before pouring concrete for the mix/de-aeration tank wall (i.e. installation must be coordinated with piping).
5. Pour concrete into the mix/de-aeration tank wall and the baffle wall

The following assumptions have been made regarding concrete structures:

- Special Marine concrete will be cast in situ.
- Site located batch plant will use specific dry mix and technology by the local supplier.
- Reinforcement is provided by the local supplier.
- Forms and shoring will be designed by the contractor

6.9.3 Ancillary Structural Components Related to Dry Moat and Mix/De-Aeration Tank

Once all of the basic dry moat components have been constructed, the following remaining steel structures and piping within the dry moat can be installed:

- Two access stairs from the finished grade to the bottom of the dry moat
- Two access stairs from the top of the finished grade surrounding the dry moat to the top of the mix/de-aeration tank
- Maintenance stile and walk bridge over the pipes at the bottom of dry moat
- Pipe support frame (for two ROV launchers)
- Mix/de-aeration tank cover steel beam framing complete with perimeter beam, grating, removable grating (access), and safety handrails
- Safety handrails around perimeter of dry moat

The following construction sequence will be followed:

1. Pour concrete for pads for pipe, stairs and stile supports.
2. Install steel structures (i.e. pipe support frame, access stairs) at bottom of dry moat
3. Install steel structures (cover steel beam framing complete with perimeter beam, grating, and removable grating) on top of the mix/de-aeration tank
4. Install safety guard rails on top of mix/de-aeration tank circumference and around top of dry moat perimeter
5. Install piping
6. Install steel structures for maintenance stile and walk over bridge over pipes in bottom of dry moat

Note that to complete this construction, working personnel must be certified to work at heights, in confined spaces and with H_2S presence. Also, for this construction, a long boom (>12m) crane will be required. It has also been assumed that corrosion protection, if necessary, will be provided by vendor and/or contractor. Stainless steel, or hot-dip galvanized bolts can be used; however, it is recommended that cathodic protection is used for overall steel protection, given that the moat will also serve as a potential containment area.

6.9.4 Trench and Rip-Rap

The two outfall pipelines and two seawater intake pipelines will extend from the nozzles (top of each nozzle 7m below MSL) at the wall of the dry moat towards the Huon Gulf in a 12m wide trench, where the pipelines daylight at about 10m below MSL in the Huon Gulf at Station 305m.

The following soil parameters were assumed:

- Friction angle of sand: 30 degrees (accounts for sand loosening by breaking waves);
- Cohesion of sand: 0kPa;
- Dry unit weight of sand: 20kN/m^3;
- Shoring design method: fixed earth support;
- Maximum mobilized passive earth pressure coefficient for deadmen: 3.0;
- The software (SPW911) was used to determine the sheetpile section required as well as the lateral load demand on the deadmen.

Prior to excavating the trench in proximity to the dry moat, temporary steel bracing must be installed into the dry moat in order to help resist the external water and soil pressures on the outside face of the south wall during excavation of this portion of the trench (i.e. in-situ soil around at least three of the deadmen supporting the south wall of the moat will be removed).

From station 0 to station 200m, the trench will be excavated using temporary sheet piling, supported by deadmen via construction wire rope (DWG-30204) and a long reach excavator or dredge. This sheet piling will extend past the surf zone in order to help provide protection for the pipes and excavation in the vicinity of the shore and breaking waves. From the end of the sheet piles until the location that the pipes daylight (Station 305m), an open-trench excavation method will be employed using a barge. No ground preparation has been proposed for the seabed at depths greater than 10m depth, where the pipelines will be positioned on top of the existing seabed.

Following completion of the installation of the pipelines (Section 6.9.5), the trench will be backfilled. The pipelines will be covered with 75mm and smaller rock filter layer of uniform gradation (below, beside, and between the pipelines and ballasting). Within the trench section, the contractor must ensure adequate compaction of the gravel around the pipelines is achieved in order to provide satisfactory embedment and protect the pipelines from being compressed, deflecting, or buckling under the weight of the remaining backfill overburden.

From station 0 to station 200m, the rest of the native excavated material will then be backfilled (and compacted in layers) into the trench, after which the sheet piles, deadmen, and temporary bracing within the dry moat can be removed. From station 200m to station 305m, the filter layer will also be positioned over/around the pipelines and ballasting. The beginning of the riprap coincides with the end of the sheet pile construction and the beginning of the open-trench excavation. The armour layer (two layers of 0.30m D50 angular stone with a maximum side slope of 2H:1V) will then be placed over top of the gravel filter layer. Subsequently, the native backfill material will be placed back on top of the scour protection in such a manner as to return the seabed bathymetry to its pre-disturbed state. From station 305m to station 350m (i.e., 10m depth to 25m depth), the pipelines and ballasting will be installed on top of the existing seabed. This portion of the pipeline will also be covered with a gravel filter layer and armour layer that matches that described above; however, no native backfill material is placed on top of the armour layer.

6.9.5 Marine Pipelines: Pipe Stresses during Sinking

Prior to installation, the pipelines will be assembled in a lay-down area (nominally within the Lae Tidal Basin) and subsequently floated with capped ends into the harbour. Ballast blocks will be attached to the pipelines (Section 6.7.3), and then these pipelines will be pulled over to the installation site, and subsequently floated into the open trench. The connection between the nozzles at the wall of the moat and the pipelines will be made via 10-15m long spool pieces, fabricated on-site.

The pipelines will be installed using S-bend sinking. Each pipe will be fitted with ballast blocks and blind flanges at both ends, and towed into its approximate alignment. The terminus of each pipe near the dry moat will be anchored about 10m from the wall of the dry moat, and not attached to the nozzles extending from the dry moat until after the sinking process is completed. During sinking, water will be pumped into the landward end of each floating pipe, causing that part of the pipe to sink. A dynamic positioning vessel will maintain tension on the weighted pipe, and gradually more water will be introduced, accompanied by release of air at the floating seaward end. A tug will support the dynamic positioning vessel by helping to control sideward movement of the pipeline perpendicular to its alignment, and thereby control the positioning of the pipelines. As this procedure progresses and the pipeline sinks, the contact point of the pipe on the bottom will move seaward. If the pipe must be raised during sinking, then air will be re-introduced, preferably from the seaward end, and water drained from the landward end.

During S-bend sinking, the part of the pipe that is suspended in the water column is supported to a large extent by the buoyancy of the air in the seaward part of the pipe. At some point, the amount of air-filled pipe on the sea surface is insufficient to support the pipe, and the pipe must then be lowered to the sea-bed under control of the dynamic positioning vessel, but with some assistance from the air in the pipe (i.e. “J-bend sinking”).

The sinking process will be monitored by an ROV equipped with an inertial navigation system designed to record and report ROVs/AUV position such that the position of each ballast block on the seabed can be monitored and recorded.

A detailed analysis of the sinking process has been conducted, using the software package ZENRISER 6.3. Figure 6.3 shows the outfall pipe near the end of the S-bend sinking process, under a tug tension of 14t. The pipe is coloured by bending moment. The allowable bending moment for 8MPa bending stress is 320kN*m, indicated by magenta tones in the colour bar. The maximum curvature and bending moment in the pipe occur just below the surface, and are within acceptable limits, with the exception of possible sharp points in the bathymetry. Sharp points should be avoided during installation by careful planning of the route and monitoring via the ROV. The tug tension is therefore shown to be adequate. During J-bend sinking, pipe stresses are generally lower than during S-bend sinking provided the rate of descent is adequately controlled. A rate of descent of 1.5m/s was shown to be acceptable through analysis in ZENRISER.

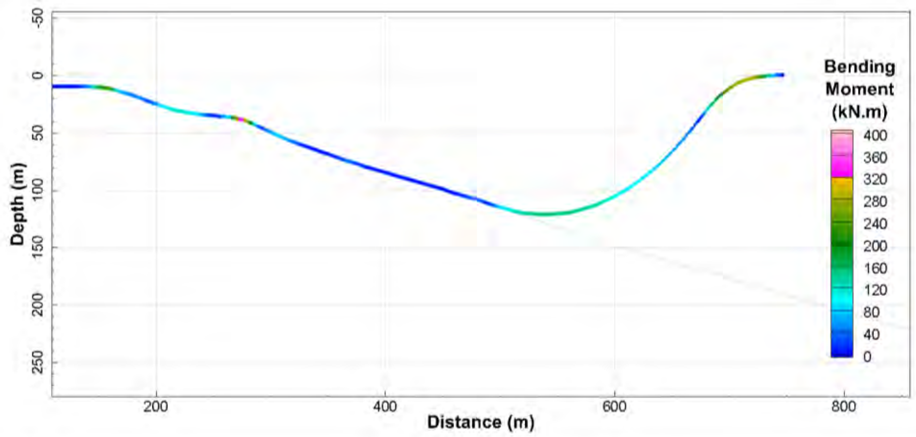

Figure 6.3: Outfall Pipe Stress during S-Bend Sinking with 14 ton Tug Pull

6.10 Near-Field Density Current

Once exiting the outfall, discharged slurry will form a density current. This section focuses on the behaviour of this density current in the near-field to confirm that the slurry exiting the outfall pipe flows in an unhindered manner away from the pipe and forms a discrete density current. Small, transient levees are built a few metres downstream from the pipe terminus to the sides of the pipe's centre axis. The growth and collapse of these levees needs to be understood to ensure they do not pose a threat to the system by plugging the pipe.

6.10.1 Background Information on the Density Current Model

The density current model is a two-dimensional reduced-gravity model of the density current created by the tailings discharge. The model considers the two-layer flow consisting of the density current flowing under the receiving water as if the density current were a one layer flow, but with a reduced acceleration of gravity in the equations of motion modified according based on the water depth, thickness of the density current, and relative density of the density current (including effects of entrained ambient seawater and solids) to ambient seawater. The model is a time-stepping model, and uses a spatial grid of uniformly spaced cells. The grid size is 20cm for assessment of the behaviour of the discharged slurry in the near-field (150m x 200m domain).

The solids form a coherent density current within the model domain selected, extending about 150m seaward from the pipe terminus. Beyond that depth, the solids are not a concern from an engineering sense (potential for pipe blocking).

The simulation starts with a bare seabed, and with all model cells devoid of the denser tailing-bearing fluid. As tailings are discharged, more and more computational cells are turned on, i.e. flooded, as the density current distribution builds in a down-slope direction with time. Ultimately, a quasi-steady state is reached, with the solids either being carried out of the model domain, settling to the seabed within the model domain, or being carried off through the generation of subsurface plumes. The model simulates the frontal advance of the density current and proceeds to evaluate the fluxes of mass into each cell, velocity components, and concentrations of dissolved scalars and sediment. The net result is that the density current readily flows in the downstream direction, but also spreads laterally, the expected behaviour for a density current discharging down a slope, and similar to the configuration of a turbulent wall jet (Rajaratnam and Pani, 1974).

The model hydrodynamics include:

- Advective terms;
- Coriolis term;
- Bottom friction and interfacial friction, expressed as a fixed fraction of bottom friction;
- Entrainment of ambient fluid based on Ellison and Turner (1959) observations
- Depletion (removes material from top of density current when it is unstable, based on a Richardson Number criteria);
- Horizontal eddy viscosity, and
- Drag by ambient currents.

Close to the outfall terminus, the high velocities lead to a high entrainment of the overlying water into the density current. This process leads to an increase in thickness of the density current in the downstream direction as well as dilution of the density current by seawater. As the density current moves away from the outfall terminus, velocities become lower and differences in density between the density current and the overlying water column become smaller, leading to lower entrainment and higher depletion velocities. Sub-surface plume generation usually does not occur in the near-field density model, because the density of the density current usually remains well above seawater density, stabilizing the density current/seawater interface over this small domain. The competing processes of entrainment and depletion were validated against data reported by Hurzeler et al. (1996).

The model has been validated against laboratory data (Stronach et al., 1999, Stronach et al., 2000). As well, aspects of the model such as the depositional footprint and the production of subsurface plumes have been shown to compare well with field observations at another DSTP site in Papua New Guinea.

6.10.2 Inputs to the Density Current Model

Solid tailings were divided in two categories for the density current modelling based on test work by Slurry Systems. A cut-off at 38 microns was selected to separate the fine from the coarse material.

- Fine fraction with grain size less than 38 microns representing 45% of the tailings solids: d50 = 13.5μm/d90 = 30.4μm/sinking velocity: 0.17mm/s
- Coarse fraction with grain size greater than 38 microns representing 55% of the tailings solids: d50 = 83.0μm/d90 = 148.0μm/sinking velocity: 6.47mm/s

Sinking velocities were obtained based on Van Rijn's terminal fall velocity for non-spherical particles (Van Rign, 1993). Slurry System's glass graduated cylinder settling test confirmed the minimum fall velocity for fine particles.

The bathymetry data was provided by the geotechnical AUV survey conducted in September 2016. The ambient seawater density was based on IHA's CTD data.

Complete results of the density current model are reported in Tetra Tech (2018). Only the deposition pattern is discussed below, because of its relevance to the engineering design.

Figure 6.4 below shows the depositional footprint after the simulation has reached steady-state. The steady-state condition was reached within one hour of simulated time from the start of tailings released. As the density current exits the pipe, the central core maintains a high velocity. The flow along the two outer boundaries, to either side of the main core, is slower. This results in the formation of small levees along the lateral boundaries. The height of the levees is limited because once they are tall enough to exceed a critical slope, the model predicts that they collapse into the main core of the flow where the high turbulence and velocity carry the remobilised sediment away from the pipes.

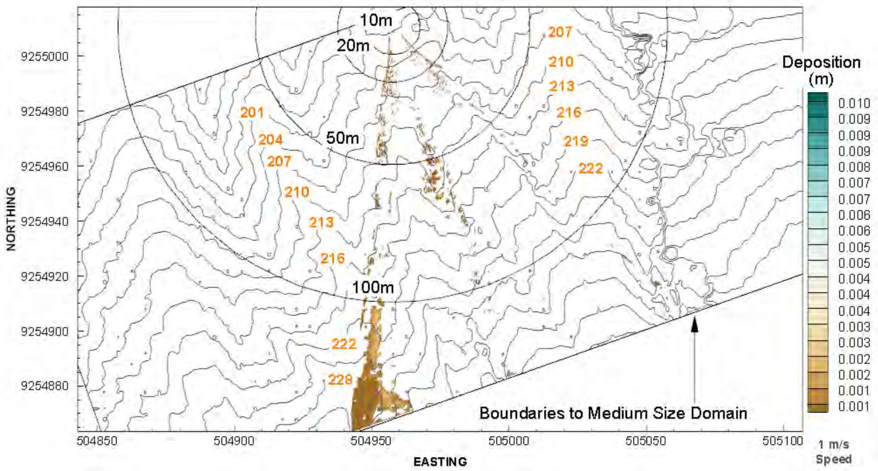

Figure 6.4: Predicted Deposition from the Density Current in the Near-Field after Reaching Steady State

6.11 Value Improving Practices

Value Improving Practices are a series of processes that are designed to ensure that any facility is conceptualised, designed, constructed and operated in such a way as to achieve the organisational objectives. A number of Value Improvement Practices were chosen to be focused on during the DSTP Design. The main focus of the chosen VIPs was of capital and operational cost optimisation of the eventual operation. The main VIPs chosen were:

- Process simplification
- Design to capacity
- Value engineering
- Energy optimisation

The Constructability VIP was enhanced by the engagement of a PNG experienced pipeline construction contractor who provided experience of the hazards and risks related to pipeline construction during the constructability review and construction cost estimation.

The VIP of 3D design was utilised for the DSTP Design. The designs and layouts were undertaken into intelligent design environments for the different engineering disciplines and ultimately integrated into the Bentley AECOsim intelligent design environment such that errors and related rework were minimised.

6.11.1 Process Simplification

Process simplification is a high-level review of the overall process or concept to ensure it does not include unnecessary steps, components, or features, thereby reducing investment and operating costs. The following process simplification has been incorporated:

- Single Mix/de-aeration tank designed to manage range of tailings throughputs
- Choke station area utilizes gravity line to the dry moat sump
- Sufficient tank height provided such that no flushing water pump required

6.11.2 Design to Capacity

This VIP eliminates excess capacity from the design of each piece of equipment. Due consideration is given to the amount of knowledge available on calculation methods, tailings rheology, pipeline route and product consistency. Capacity is controlled and safety factors are used to avoid duplication, provide auditability and consistency in calculations, and to ensure that consistent design margins are applied for equivalent equipment and systems across the project. The following design to capacity work has been incorporated:

- Design margin of 6% included, over the plant maximum operating mass flow rate
- Conducted test work to establish the following tailings rheology, water corrosive properties, solids settling velocities, tailings Rheology bookend tests, system sensitivity to changing grind size, and predicted wear rates based on upstream process plant roping cyclones.

It is recomended that final hydraulic and transient analysis calculations be conducted based on firm survey data, final pipe route, and mine production profiles.

6.11.3 Value Engineering

Value Engineering is a disciplined method used during front-end design aimed at eliminating or modifying items that do not add value to meeting business needs, and also includes the assessment of construction execution strategies to ensure that the most efficient cost design basis is adopted without compromising safety or environmental performance. The Value Engineering VIP involves detailed consideration of facilities and equipment, and determining better ways to provide functions of major equipment. The following Value Engineering VIP items have been incorporated:

- ANSYS Fluent 18.0 CFD modelling (Eulerian Granular model) employed to generate a 3-D CFD model of the mix/de-aeration tank to assess wear potential, evaluate baffle design and orientation of inflowing/outflowing pipes to optimize mixing of tailings and seawater and air release.
- Mix Tank Hydraulic Analysis Model to evaluate various scenarios (i.e. sea water pipe blockage, tailings ocean outfall pipe blockage, bio-fouling of sea water intake pipelines, transients, etc.).
- Density current modelling, Plume Dispersion modelling, and three-dimensional hydrodynamic modelling of near and far-field tailings deposition, scour and behaviour within Huon Gulf.
- Modelling of pipe installation (S-bend and J-bend) using ZENRISER software to ensure pipeline can withstand stresses during installation and to recommend best practices.
- Mix/De-aeration tank material and pipeline material selected to withstand anticipated wear/pressure over LOM.
- Flood mitigation design based on hydrological and hydrodynamic study and model
- Early specialised construction contractor involvement for development of construction methodology and schedule

6.11.4 Energy Optimisation

- No pump required at mix/de-aeration tank during flushing of terrestrial pipeline, reducing the overall electrical load at the mix/de-aeration tank area
- Passive gravity DSTP system (no high flow sea water pumping station required)

7 SAFETY IN DESIGN

7.1 Safety Parameters Identified and Provided for in the Design Basis

Tetra Tech has a specific focus on Safety-in-Design (SID) which is integrated into the engineering processes for each discipline. The objectives of identifying and applying SID disciplines is to apply risk control measures to the plan definition, and to flow these through to subsequent execution of engineering design work. This is aimed at having greater focus on addressing basic safety requirements as an integral part of the design effort; with the objective of circumventing costly re-work later in the execution of detailed design.

Hazard and Operability Analysis (HAZOP) level 2 studies were conducted by the project team on the DSTP system during the development of the design, wherein different risks and safety-in-design elements were identified. Safety parameters in design basis were also reviewed within the project HAZOP and constructability reviews. Due regard was given to the requirements for the safety of appropriate maintenance, management and operational personnel in specific areas. Furthermore, emergency evacuation requirements related to an appropriate variety of emergency situations as identified during the HAZOP reviews were considered and incorporated into the area layout designs.

7.2 Safety in Design

The following safety considerations (Table 7.1 on following page) have been incorporated into the design of the marine component of the DSTP system.

Table 7.1: Safety in Design Considerations

Area	Safety Consideration
Tailings Mix Tank and Valves	• The dry moat design incorporates two stairways as well as two egress ladders in opposite corners to provide opportunity for safe evacuation of personnel.
	• Multiple access points to each area within dry moat
	• Allowance for second removable grate at top of mix/de-aeration tank to provide emergency response access for stretcher to be hoisted out in the event of an emergency.
	• Safety handrails around top perimeter of dry moat
	• Safety handrails around top perimeter of mix/de-aeration tank
	• Safety steel mesh (i.e. "Handimesh" or similar) built into stairway to catch any fallen objects/tools.
	• Safety handrails on all stairs, stiles, and access ramps.
	• Working personnel to be certified for working at heights, in confined spaces, and with H_2S present
	• Factor of Safety incorporated into all geotechnical and structural design work to ensure design can withstand expected dead and live loads experienced overLOM.
	• Passive system does not require personnel to be regularly working around the mix/de-aeration tank or within the dry moat.
	• Sump pump within dry moat will prevent accumulation of rainwater or tailings/sea water and thereby reduces risk of slips/trips/falls or drowning within dry moat.
	• Ease of access to all mechanical and instrumentation components (i.e. valves and flow meters) that may need to be serviced during LOM (i.e. the ROV launcher pipes do not pass directly overhead of the control/isolation valves within the dry moat).
	• Piping materials within dry moat (Rubber-lined steel and stainless steel) selected to withstand anticipated wear and pressures over LOM.
	• Mix/de-aeration tank material selected to withstand expected wear and fluid pressures over LOM.
	• Dry moat designed to be "water-tight" (isolated from groundwater)
Tailings Ocean Outfall	• Inspection of pipeline via ROV launcher situated with access point above sea level and adjacent to top of tank, so crews are not subject to high water pressures.
	• Inspection of pipeline via ROV and not by personnel.

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