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TWIN HILLS SURFACE WATER STUDY AND WATER BALANCE

DEFINITIVE FEASIBILITY STUDY SUMMARY REPORT

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EXECUTIVE SUMMARY

Osino Gold Exploration and Mining (Pty) Ltd is completing Definitive Feasibility Studies (DFS) for the Twin Hills gold mining project (the Project), located near Karibib, in the central region of Namibia.

The project comprises four open pits, a processing plant, a flood diversion structure, as well as waste rock and tailings storage facilities to accommodate a planned annual 65 million tonnes of ore throughput for the gold mining operation (measured, indicated and inferred). Knight Piésold Consulting (Pty) Ltd was appointed to complete surface and ground water management studies for the project. The groundwater studies are presented in a separate report while this report summarizes surface water studies. This includes the site hydrology assessment, site flood mitigation measures for the Okawayo River and a site wide water balance inclusive of groundwater and pit dewatering potential supply for the processing operation.

The main findings of the surface water studies are summarised in what follows hereunder.

CLIMATIC AND HYDROLOGIC CONDITIONS

The Project site is characterized by arid conditions with a rainfall season generally spanning between December to March/April. Precipitation is generally short and of higher intensity, occasionally developing into flash flood rainfall events. The mean annual rainfall estimate has been refined from past hydrological studies to 184 mm, although drought periods in Namibia are known to last over several years [NMET, 2021]. With gross annual potential evaporation exceeding the mean annual rainfall significantly (3200 to 3400 mm A-pan evaporation per annum), the site is in a water deficit for most of the year.

Climate change studies and models from the World Bank Group Climatic Portal show that periods of drought are expected to increase and flood events are expected to be of higher magnitude and intensity [WBG, 2021].

The site is characterised by generally flat catchment areas bordered by a marble ridge on its southern side. The soil is generally rocky with sand and alluvium material and scarce to medium dense bushes. The proposed mine area is part of the Khan River hydrologic basin, a relatively large ephemeral river leading to the Swakop River. The site naturally slopes in a north western directly following a gentle slope of about 1% to steep slopes closer to the marble ridges.

OKAWAYO FLOOD ATTENUATION

The Main Pit will be developed across the natural flow path of the Okawayo River, an ephemeral river which occasionally flows during flash flood events and often experiences little to no flow during low return period storms. The Okawayo River catchment area at the Main Pit southern rim is estimated to be approximately 92 km² according to available digital elevation models [NASA/SRTM, 2013].

The catchment slope is relatively flat, comprising mainly sand cover with marble outcrops where it crosses the marble ridge south of the Main Pit.

Without any diversion of the Okawayo River, large flood events could have a significant impact on the Main Pit operation. Figure 1, below, depicts the anticipated floodline level that could arise as a consequence of flooding the Main pit for various flood Annual Exceedance Probabilities (AEP).

The figure illustrates that the Main Pit could be flooded to a depth of up to 50 m owing to runoff originating from only a 2-year AEP flood event (estimated at 0.6 Mm³ from 42 mm rainfall-runoff

generated over the entire catchment) and up to 140 m depth in the case of a 200-year AEP flood event occurring (5.8 Mm³ from 118 mm rainfall-runoff generated over the entire catchment).

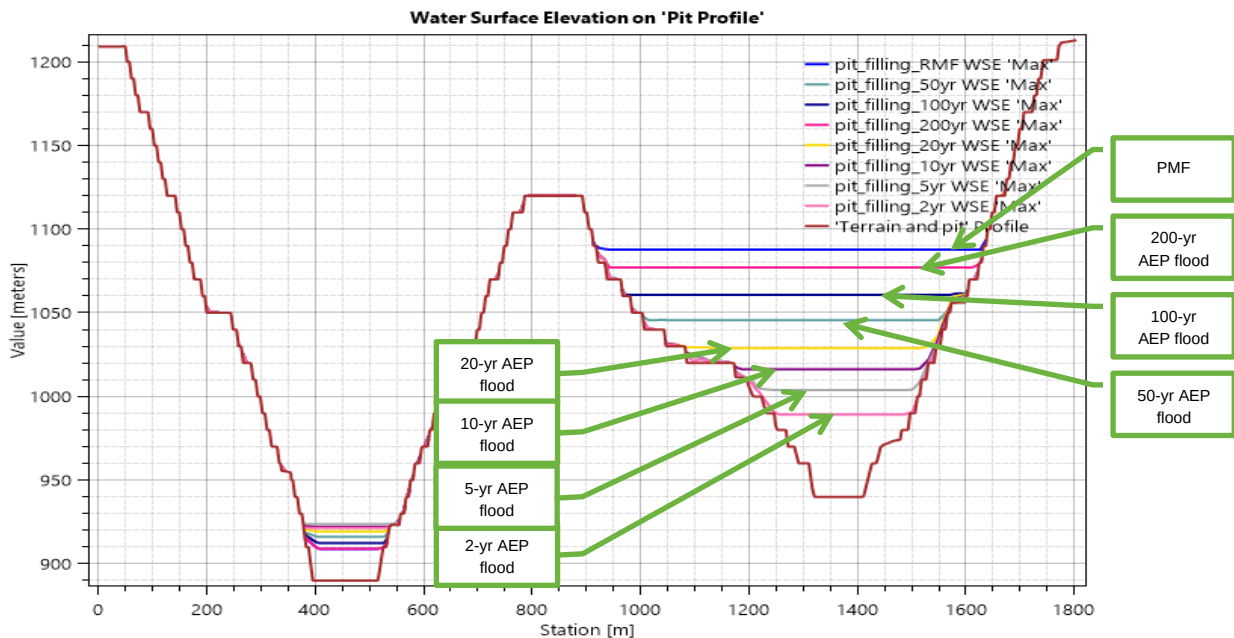


Figure 1 – Anticipated depth of flooding the Main Central Pit for various AEP floods without diversion measures

To mitigate against the pit flooding risk for small duration storms, construction of a stream diversion structure upstream of the pit diverting flood flows through a channel around the pit was proposed in the Prefeasibility Study, PFS.

At the time of the PFS, the river diversion philosophy proposed a 20 m high dam wall with a large capacity spillway discharge channel to divert all AEP flood events around the pit. Apart from the environmental and safety risk associated with the construction of a medium sized dam wall immediately upstream of the pit to arrest ephemeral flood flows, this solution would require deep excavations to accommodate the spillway channel and would ultimately have a significant impact on Capex expenditure and overall bankability of the project.

Alternative river diversion solutions were developed to reduce implementation cost, with an understanding that the Mine would then accept a higher risk of pit flooding in the event of a larger flood event occurring.

In this regard, the diversion structure has been sized to convey up to the 50-year AEP peak flow around the Main Pit to discharge to the Cloud West Pit. Floods with greater return periods will potentially overtop the dam wall's overflow weir, and be routed towards the Main Pit, where provision for pit lake and dewatering following extreme events will need to be managed by the mining operation.

The dam structure must therefore be designed to allow for safe overtopping during extreme floods without compromising the integrity of the structure and the spillway channel must be fully operational during all flood flows. Inspection of the spillway channel is possible when the flood subsides.

It is expected that the small reservoir formed upstream of the dam structure will fill with sand sediment transported during flood events over time and that little to no flood attenuation can be assumed once

this happens. The spillway and discharge channel capacity are therefore sized to accommodate the unrouted design flood peaks (without allowing for attenuation), which is appropriately conservative. A secondary benefit of upstream sedimentation is that the diversion structure will then essentially act as a sub-surface recharge dam that will provide a higher, beneficial, head for groundwater recharge.

The proposed 8.8 m high dam for the main diversion considered two wall types, namely a concrete faced rockfill dam (CFRD) wall and a rubble masonry concrete (RMC) weir. Considering the presence of competent marble foundation rock mass at the proposed dam site, an abundance of stone plums that can be sourced for the wall construction and the inherent ability of the structure to pass flood flows directly over the wall both during construction and after completion, the RMC weir is the preferred option dam type. It is also the most cost-effective option.

Up to the 1/50 AEP design flood, discharge will be controlled through a spillway channel located on the right flank that will convey the water to the Cloud West Pit (see figure below). Stored water in this pit could be used as an additional storage facility to balance water supply requirements over the life of the mine. The pit capacity is adequate to store up to approximately the 1/5 AEP and will overtop for larger duration storm events. Water flowing from the pit must be routed back into the downstream watercourse; the details of this to be confirmed.

The construction of the spillway and channel is expected to require excavation in medium to hard rock and would require little to no additional lining to protect against erosion during operation. This assumption will need to be re-evaluated once a more thorough geotechnical investigation along the proposed alignment is carried out in the next stage.

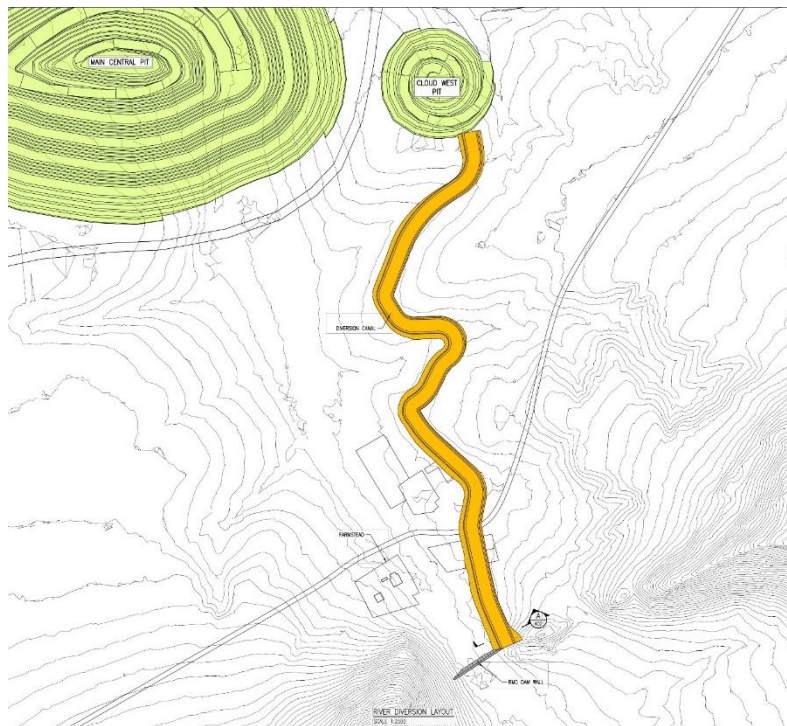


Figure 2: Layout of diversion wall and discharge channel

CLASSIFICATION

The RMC weir will be designed as a gravity monolith with the majority of its length assumed to be an emergency spillway (operating for flood return periods between 1/50 AEP and PMF).

As a recharge/ raw water dam, classification is made in terms of the RSA dam safety regulations and guidelines described by South African National Committee on Large Dams (SANCOLD), 1991 for selecting spillway design floods and freeboard provisions.

Together with the reduced (small) wall size, a low-risk hazard category can be assigned to the dam, provided that no persons downstream will be harmed in the event of a dam breach or major flood event arriving.

Considering the flat catchment and ephemeral flow regime, effective early flood warning systems can be developed and applied as part of the emergency response and preparedness plans, to ensure that personnel are prevented from access to the main pit in the event of a major flood event.

The selected spillway design flood (1/50 AEP) for a dam with a low hazard risk is in accordance with SANCOLD guidelines. A 1/50 AEP is also a typical design return period for considering environmental flows in terms of clean/dirty water separation.

Should a flood with a higher AEP be experienced, the RMC weir will begin to safely overtop with its crest shaped as a broad crested weir (see cross section below) and the flow rate in the spillway channel will progressively increase. In the extreme case of a PMF arriving, the RMC structure will overtop by approximately 0.75 m and the spillway channel will be discharging the equivalent of the 1/100 AEP, with zero secondary freeboard.

This low classification considers the small wall height and the adoption of an effective early warning system by the mine (reducing probable loss of life to zero in the event of a dam breach or spillway flow) in the area downstream.

This does not consider the economic impact of lost production or pit dewatering requirements, should the Main Pit flood. The mine must accordingly accept this risk.

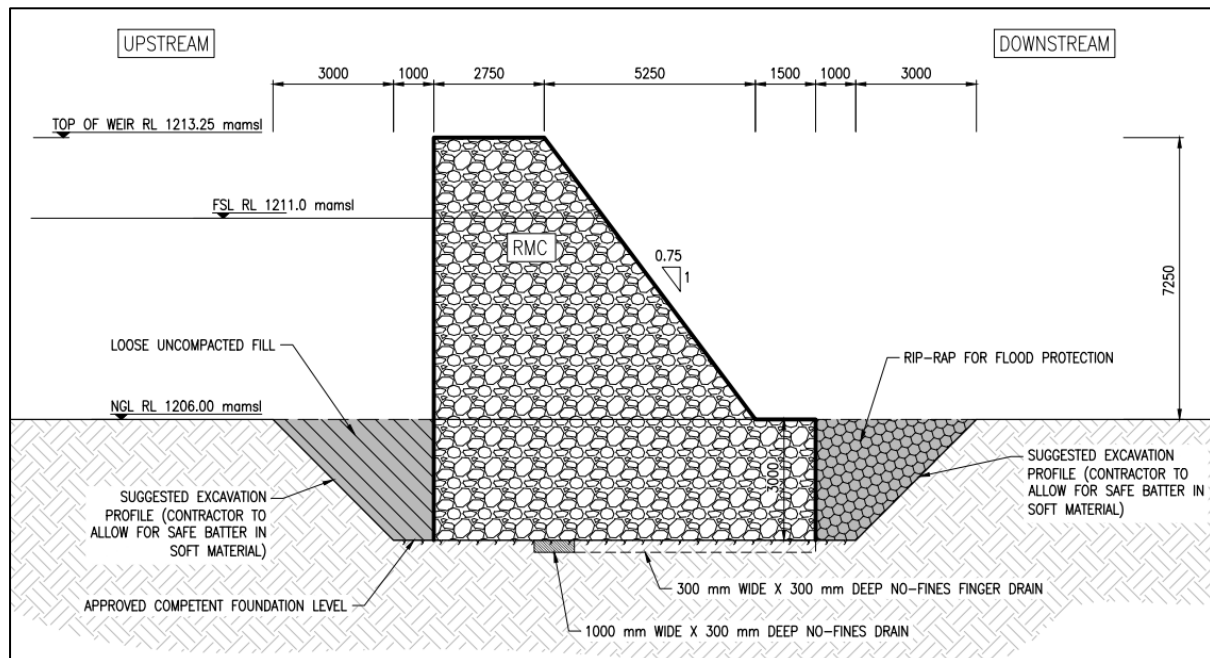


Figure 3: Cross section of RMC wall

A high level bill of quantities was extracted, and construction rates obtained from various contractors were applied to the major materials for the RMC and CFRD wall options, as well as the spillway discharge channel.

The tables below summarize the expected construction costs for the RMC wall and the channel in Year 1 and Year 2, respectively, as well as the total combined cost of construction.

Table 1-1: Cost of RMC Wall in Year 1

Okawayo Diversion Stream P&G's	873 883.35
	10.43%
Okawayo Diversion Stream Construction Costs	7 979 576.81
5% on design development	398 978.84
Year 1 RMC Dam	9 252 439.00

Table 1-2: Cost of Channel Excavation in Year 2

Okawayo Diversion Stream P&G's	954 026.34
	10.43%
Okawayo Diversion Stream Construction Costs	8 711 376.02
5% on design development	435 568.80
Year 2 Channel Excavation	10 100 971.16

Table 1-3: Combined cost of diversion structure

Okawayo Diversion Stream P&G's	1 827 909.70
	10.43%
Okawayo Diversion Stream Construction Costs	16 690 952.83
5% on design development	834 547.64
TOTAL OKAWAYO STREAM DIVERSION	19 353 410.17

WATER BALANCE

The climate for the site has a net water deficit considering annual precipitation and evaporation estimates. As such, the project requires the investigation of supplementary water sources to meet the water demand of the process plant. Groundwater supply by means of borehole pumping and pit dewatering are key water sources investigated for on site supply.

Alternative supply from NamWater, Karibib Town Treatment Works, and an augmentation scheme on the Khan River, were suggested in the PFS, and are being investigated by other consultants in this phase of the study.

The water balance for this phase of the study focused on the supply from surface water runoff and groundwater supply, with the alternative supplies only being considered as supplementary to supply and deficits identified in the water balance.

The site water balance is based on a peak process throughput of 5.2 Mtpa and has a water demand of approximately 1.112 million cubic meters per annum (Mm³/a). The process design aims to maximise the reuse of water by recycling process solutions wherever possible through filtration systems in the

plant. A monthly increment was selected for the BFS and includes inflows and outflows to the process operation including the open pits.

The plant demand assumes that approximately 77 % of the water contained in tailings is recycled at the plant before the cake is sent to the tailings storage facility. The remaining water is lost in the filtered tailings cake sent to the tailings storage facility. This also includes losses to plant operations and gland water.

Evaporation estimates from the open pit lake, groundwater inflows, and direct rainfall are considered in the water balance model.

The DFS water balance shows that the plant demand can be supplied by groundwater from boreholes and pit dewatering operations through the life of the project (see Figure 4). The supply from groundwater is a main source of water and essential for operations. Occasional surface water runoff will also be diverted from the Okawayo river to the Cloud West pit, after the pit is mined out. Surface water runoff frequency during the life of mine is, however, uncertain and would not be sustainable over the life of mine and does not eliminate the requirement for groundwater supply. The impact of the Cloud West Pit as a storage reservoir on the overall groundwater recharge will need to be evaluated in greater detail in the next phase of the study.

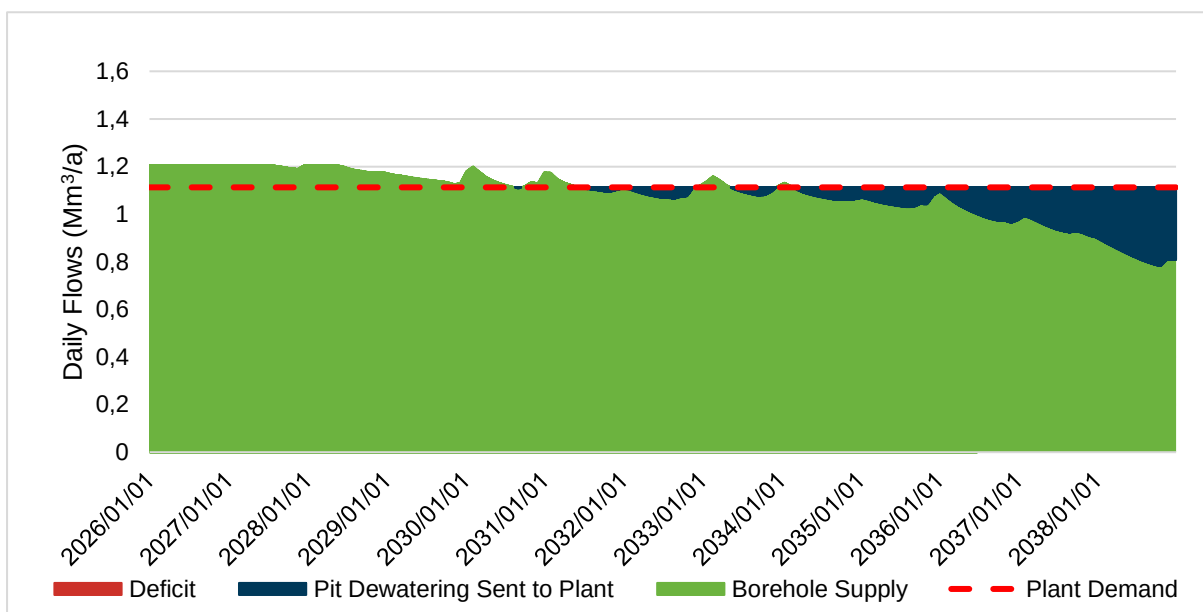


Figure 4: Water supply/deficit for Scenario 1 with full borehole supply

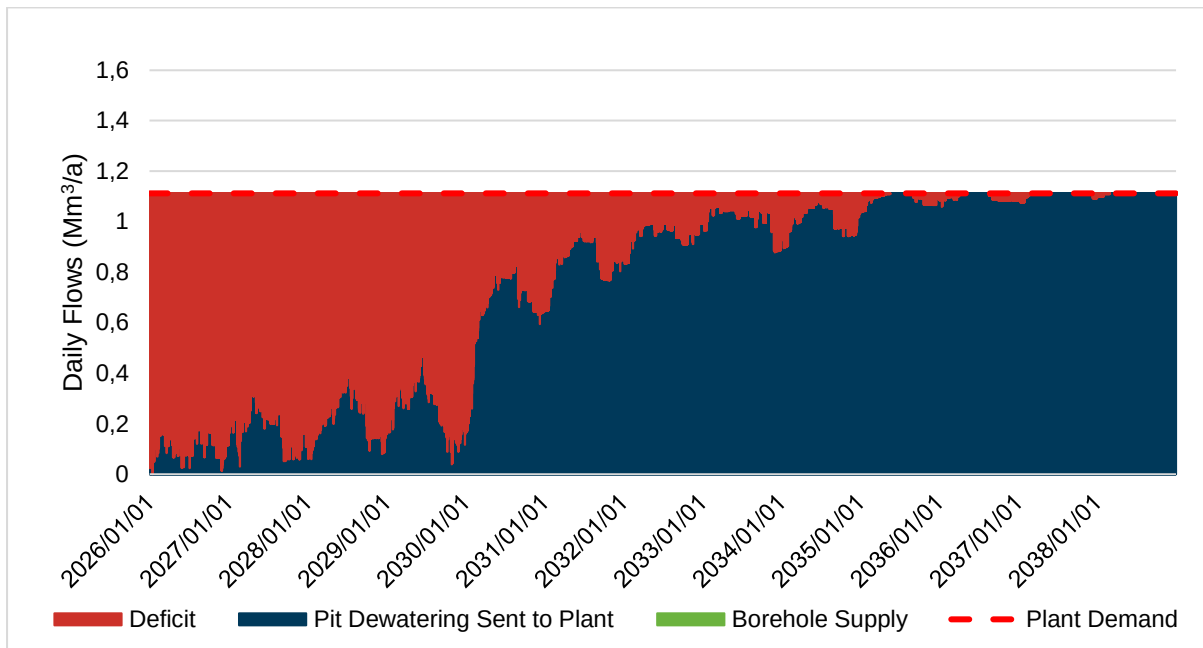


Figure 5: Water supply/deficit for scenario 2 with no borehole supply

The water balance considered the sensitivity of supply to increased plant demands, the exclusion of borehole supply, an extended drought period, and the exclusion of the Cloud West pit as a storage reservoir. It was found that the security of water supply is extremely sensitive to borehole supply (see Figure 4), with deficits expected in the initial years of the project and in the dry seasons of the latter years if borehole supply is ignored. These deficits would require supply from the alternative source mentioned above.

If borehole supply is considered, the impact of droughts will be negligible. The use of the cloud west pit for storage is not a necessity if the modelled borehole supply is achieved, however, the results show a clear benefit if borehole supply is compromised or if the plant demand were to increase.

Alternate sources of water supply options are also being considered by other consultants (which includes supply from the Karibib Town Water Treatment Works and augmentation from a scheme on the Khan River).

CONCLUSIONS, RISKS AND OPPORTUNITIES

Surface Water Management

- The current climate change models indicate that the climate will become more variable, tending toward longer more severe drought periods and shorter more severe flood events, which could result in changes to the flood peaks. The selection of the rubble masonry concrete flood mitigation structure would mitigate increased flood peaks to some degree as it can be designed to overtop with less risk than the concrete faced rockfill dam option.
- Accurate estimates of direct catchment runoff from precipitation events typically require measurements from multiple weather stations and flow gauges along the catchment confluences. As these measuring stations are not available for this particular catchment, runoff is estimated by using rainfall runoff estimation methods. A runoff coefficient estimation study was carried out to validate the coefficients used in the flood peak estimates, however, with limited contours information. The quality of the survey needs to be refined to further validate the estimates.

- Part of the main pit is situated directly in the flow path of the Okawayo River, at the confluence of a 92 km² catchment area. The pit would be subject to flooding under extreme event conditions if no flood mitigation measures are implemented. The pit development schedule would dictate the timeline for the implementation of flood mitigation measures, however, constructing the diversion structure early in the project cycle would aid the groundwater recharge by effectively creating a sub-surface dam once the diversion reservoir fills with sediment.
- The AEP to design flood mitigation infrastructure is not clearly defined by local and regional regulations. The diversion structure is not intended for use as a permanent water supply but rather to route flood flows to the Cloud West pit. Whereas diverting all AEP flood flows is possible, this would require significantly larger and more expensive diversion infrastructure.
- The Client requested that alternative river diversion solutions be developed that offered a trade off between diverting smaller return period floods at a reduced implementation cost, but the mine would then accept the higher risk of pit flooding in the case of larger flood events.
- A small dam constructed using RMC is recommended for the diversion wall, with small size class (8.8 m) and a low hazard rating classification (SANCOLD). The spillway is sized for a 50-year AEP flood capacity but larger floods can be passed directly over the wall. From a business perspective, there is risk to the operations if lower AEP flood events were to occur. Early flood warning systems would need to be implemented by the mine for evacuating plant and personnel from the pit before the dam overtops and operations in the main pit would be suspended until the pit is dewatered. The dam wall must therefore be designed to allow for safe overtopping during extreme floods without compromising the integrity of the structure and the spillway channel must be fully operational during all flood flows. Inspection of the spillway channel is only possible when the flood subsides.
- As the main pit is situated in the direct flow path of the Okawayo River, the environmental impact of diverting flow to the cloud west pit will need to be assessed further. The incremental impact of the small reservoir pool upstream of the dam is no more severe than the case of intercepting the natural river it in the Central pit without diversion measures being instated.
- The expectation that the diversion dam reservoir would silt up has the benefit of creating a sub-surface recharge dam, but the consequence is that there is little to no attenuation achieve during flood events (i.e. the reduction in storage capacity by silt build-up reduces how much water the dam can store before flowing over the spillway). This results in the spillway being sized for an unrouted flood peak, although these peaks are temporary and not expected to be sustained during flooding.
- The excavation of the spillway discharge channel is expected to be predominantly in rock. The requirement to line the channel will need to be assessed during the construction and/or the operation of the channel. This will be based on the extent erosion/scouring that takes place. The side slopes used in the estimation of the excavation quantities are also based on the assumption that the channel will be excavated in competent material – should the material require flatter side slopes, the excavation quantities will change.
- The surface water management referred to herein is specific to the management of the upstream catchment. Management of runoff emanating from the waste rock dumps and the TSF is looked at under the scope of those designs.

Water Balance

- The key component contributing to water supply has been identified as borehole supply. The variability/reliability of this supply is addressed in the groundwater study and has not been addressed in the surface water study, specifically the water balance. The impact of not having borehole supply has been shown to be a significant risk to the overall water security for the site.

- The seasonal variability in groundwater was not considered in the PFS study. This was addressed in this phase of the study by including drought conditions for both the groundwater and the surface water components of the water balance.
- The full evaporation potential on the pit inflows seeping through the pit walls was not fully considered in the previous phase of the study. This has been addressed in this phase of the study. The significance of the borehole supply compared to the supply from pit dewatering reduces the risk of under/overestimating this component of evaporation, but a reduction in borehole supply may require this estimate to be redefined.
- The borehole supply has been shown to decrease over time, however, as the pits are excavated to greater depths, the pit inflows increase which offsets the risk of the reduced borehole supply.
- Reliability of supply for the external supply options has not been considered in this study. The supply potential and reliability from these separate studies is intended to assess the potential to meet plant demand in the case of deficits from site supply.
- A detailed sensitivity analysis was not considered in the previous phase. The sensitivity of site supply has been addressed in this study based on the scenarios selected.
- The high-level water balance from the PFS was based on a monthly time step. This has been refined in this phase of the study by running the model on a daily time step to allow for better refinement of the daily response to water flow/transfers.
- The current operating philosophy of the water balance relies on all pit inflow being dewatered and water being stored in the Cloud West pit. The practicality of achieving full dewatering has not yet been considered. The pit drainage management will need to be designed in the next phase and incorporated into the water balance operating philosophy.

RECOMMENDATIONS

Surface Water

Surface water recommendations can be summarized as follows:

- Complete a detailed topographic survey (1 m contours accuracy) of the Okawayo catchment area, and update hydrological and hydraulic models to assess the potential runoff and peak runoff for various flood events. This should also include the B1 highway crossing and location of roadway water crossings along the Okawayo catchment.
- Integrate and detail the overall site water management to include mining infrastructures/dewatering, downstream infrastructures such as roads, utilities, and final landscape of waste rock dumps. Detailed design should include:
 - Accurate sizing of channels, flood protection berms and storage containment facilities;
 - Detailed stormwater design plan around the mine pit infrastructures development.
 - Overflow structures for the Cloud West Pit
- Complete detailed design of the Okawayo flood mitigation measures with detailed peak flood assessment upstream and downstream of the structures, transient analysis and operational and environmental impact of various flood conditions on the downstream environment. This should consider a stepped development of the pit and details of flood protection berms, pit water management landscapes (pit lake), other site water management structures and dewatering capacity.
- Complete a sediment transport model and impact assessment for various conditions as part of the detailed design.

- Consider the mine development activities (phased development) and detailed infrastructures in a flood risk assessment, operational and emergency procedure requirement to mitigate risk.
- Complete, as part of the detailed design, additional geotechnical investigation to detail the founding conditions of the diversion structure and spillway discharge channel, as well as required suitable erosion protection measures.
- It is recommended that a pit water management system be put in place during the detailed design of the stormwater management plan.
- Peak flows and all detailed design criteria must be included in the stormwater management report.
- Impacts of the diversion structure must be considered in a detailed environmental impact assessment stemming from detailed designs.
- The impact of sedimentation should be investigated in more detail in the detailed design
- Consider the mine development activities (phased development) and detailed infrastructures in a flood risk assessment, operational and emergency procedure requirement to mitigate risk

Water Balance

- The design/management of the pit dewatering, which incorporates the pit development, will need to be incorporated into the water balance design philosophy.
- The timing of when the diversion structure and the cloud west pit is used for storage could be refined in the detailed design.
- Details regarding the TSF should be refined and the surface water collection from this facility should be included in the overall water balance.
- The results of the study for the Khan River augmentation options should be incorporated into an overall site water supply report.
- The variability of the groundwater supply could be refined in the detailed design, with the impact on reliable yield assessed based on using the boreholes for secondary supply rather than primary supply, as is the current case. As the pit dewatering requirement increases with time, this could be used as primary supply and borehole abstractions could be used to supplement the plant supply. This may have a positive impact on the available yield from the boreholes in the latter years of the project.

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ABBREVIATIONS

2D	Two Dimensional
AEP	Annual Exceedance Probability
CFR	Concrete Faced Rockfill
CFRD	Concrete Faced Rockfill Dam
DFS	Definitive Feasibility Study
EFL	Estimated Floor Level
EIA	Environmental Impact Assessment
FoS	Factor of Safety
FSL	Full Supply Level
ICOLD	International Committee on Large Dams
km ²	squared kilometres
kN/m ³	kilo Newton per cubic meter
KP	Knight Piésold Consulting (Pty) Ltd
LOM	Life of Mine
M	million
m	Meters
m/day	meters per day
m ³ /d	cubic meters per day
mamsl	meters above mean sea-level
MAP	Mean Annual Precipitation
MAR	Mean Annual Runoff
mbs	meters below surface
MCE	Maximum Credible Earthquake
MDE	Maximum Design Earthquake
mm/year	millimetres per year
Mm ³	million cubic meters
Mm ³ /a	million cubic meters per annum
MPa	Mega Pascal
NOC	Non-overspill Crest
OBE	Operating Basic Earthquake
PFS	Pre-Feasibility Study
RDD	Recommended Design Discharge
SANCOLD	South African National Committee on Large Dams
SEF	Safety Evaluation Flood
SF	Safety Factor
SOW	Scope of Work
SRTM	Satellite Radar Topography Mission
TSF	Tailings Storage Facility
WRD	Waste Rock Dump

1 INTRODUCTION

Osino Resources anticipate that the Twin Hills Gold Project's minimum overall water demand will be in the order of 1.1 million cubic meters (Mm³) per annum, depending on the processing throughput. Osino Resources appointed Knight Piésold Consulting (Pty) Ltd (KP) to conduct surface water and groundwater assessments for the Twin Hills Gold Project in Namibia to support the Definitive Feasibility Study (BFS). This report documents the surface water component of the appointment. The surface water study covers hydrology, surface water management, and a water balance that focusses on confirming the available water supply and/or deficit for the plant.

Based on the pre-feasibility study, it is envisaged that the mine's water requirements would need to be supplemented with groundwater, and/or sourced through NamWater. This report includes the results and interpretation of the additional data collated during this phase of the study.

1.1 OBJECTIVES

The following objectives are outlined:

- Confirm site surface hydrology.
- Determine base flood lines and the impact of flooding on the pit, as well as progress the flood mitigation measures identified in the PFS to a definitive level of accuracy.
- Update and refine the mine supply water balance with updated inputs from the groundwater study.
- Size and cost required infrastructure for the Okawayo flood mitigation structures.

2 PROJECT DESCRIPTION

2.1 GENERAL LAYOUT

The Twin Hills Project area is in the Erongo Region of Namibia, approximately 17 km northeast of Karibib Town. The Study Area is north of the D1941 District Road with the C33 Road and airport to the west. Figure 2-1 shows the location of the study area relative to important landmarks.

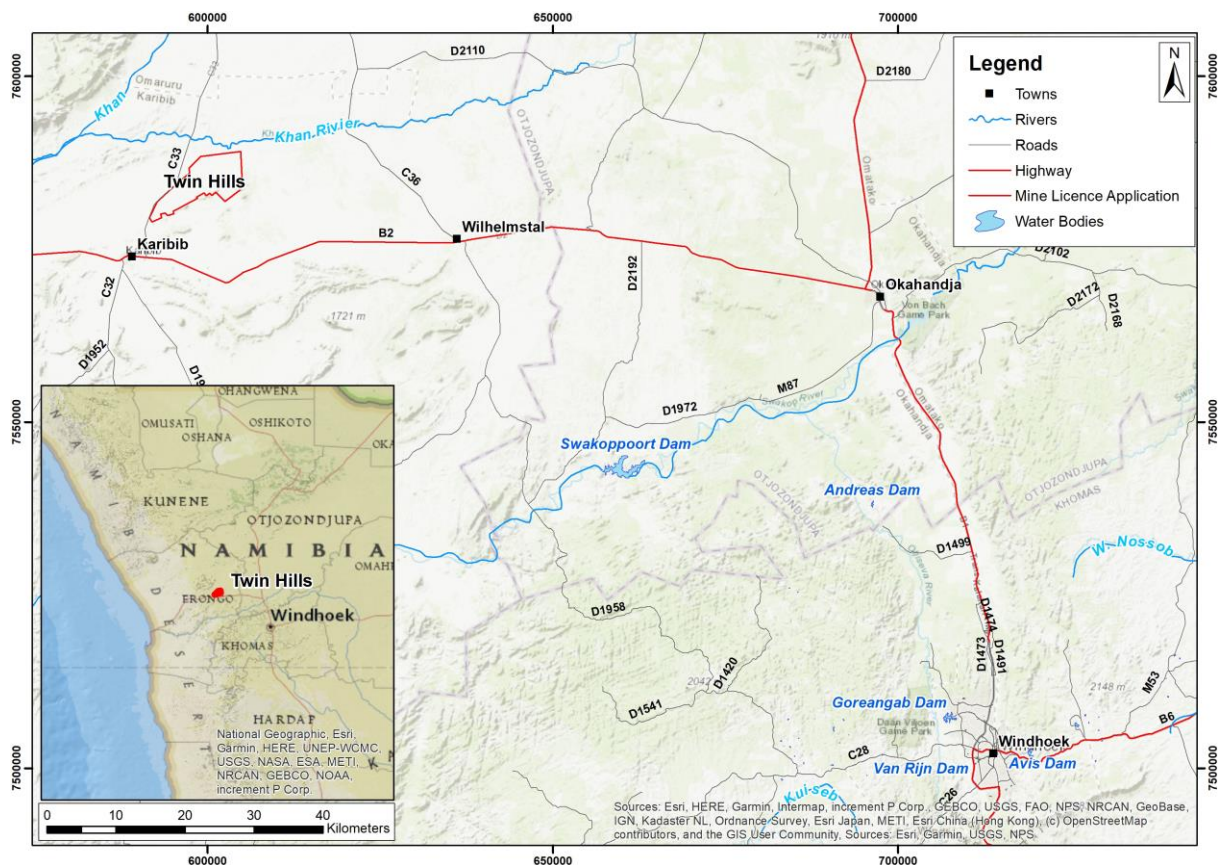


Figure 2-1: The Location of the Twin Hills Study Area.

2.2 MINE INFRASTRUCTURE

The preliminary proposed infrastructure for the mine is shown in Figure 2-2 and includes:

- Development of four open pit areas to a depth of 335 meters below the surface (Main Pit (Bulge and Twin Hills Central), Cloud, Cloud West, and Twin Hills West).
- A groundwater supply and distribution network
- A Tailings storage facility
- Waste rock dumps.
- An alternative route for the D1941, with a mine access road and Haul network.
- Plant, mine offices and a PV installation site.

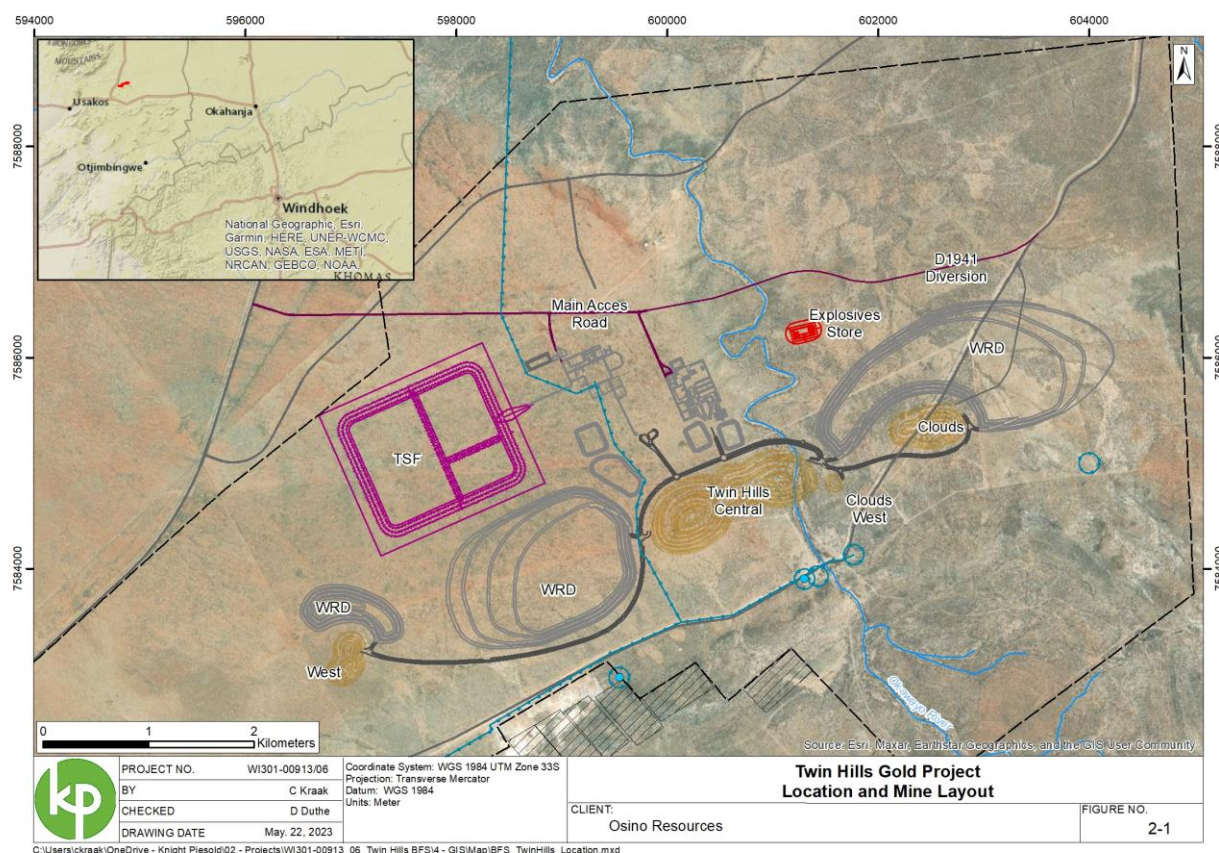


Figure 2-2: Mine Infrastructure and Layout

2.3 METEOROLOGICAL INFORMATION

2.3.1 CLIMATE

The daytime temperatures near the town of Karibib are warm to hot in summer, with average temperatures of 35°C, and cold winter temperatures falling below the freezing point. The climate is characterized as arid with low humidity and the predominant wind direction is from the North-East.

The current climate change models, indicate that the climate will become more variable, tending towards longer more severe drought periods and shorter more severe flood events [WBG, 2021].

2.3.2 RAINFALL

The PFS evaluated several sources of precipitation data and selected the data provided by the MET services for the Karibib station. This data was presented in monthly totals for the years 1980 to 2015, with a mean annual precipitation (MAP) of 268 mm. The cumulative annual precipitation is plotted on Figure 2-3. Typically, a good data set will have a linear cumulative annual precipitation. If there is a distinct change in the slope of the cumulative progression, it is an indication of a change in climatic conditions or a change/error with the way the data is recorded. It is clear from Figure 2-3 that there is an increase in the annual precipitation from 2003 onwards. The increase is considered to have skewed the data which resulted in an overestimated MAP estimate. To correct this for the BFS, daily data from

the same station was sourced for the period 1980 to 2002. This data was used to determine a new MAP of 184 mm, with Figure 2-4 showing the monthly precipitation.

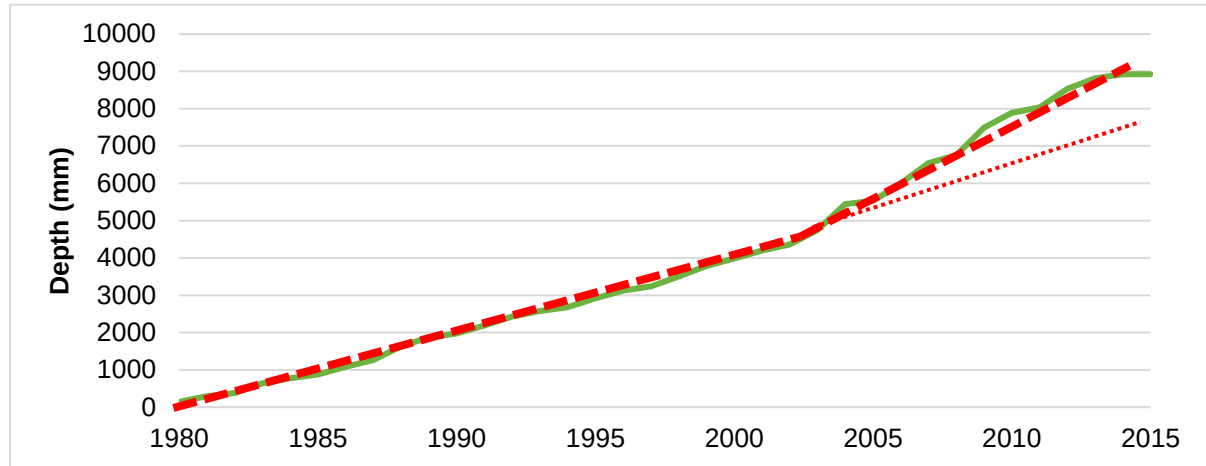


Figure 2-3: Cumulative Annual Precipitation for Karibib Station [1980-2015] showing a change in rainfall trend from 2002.

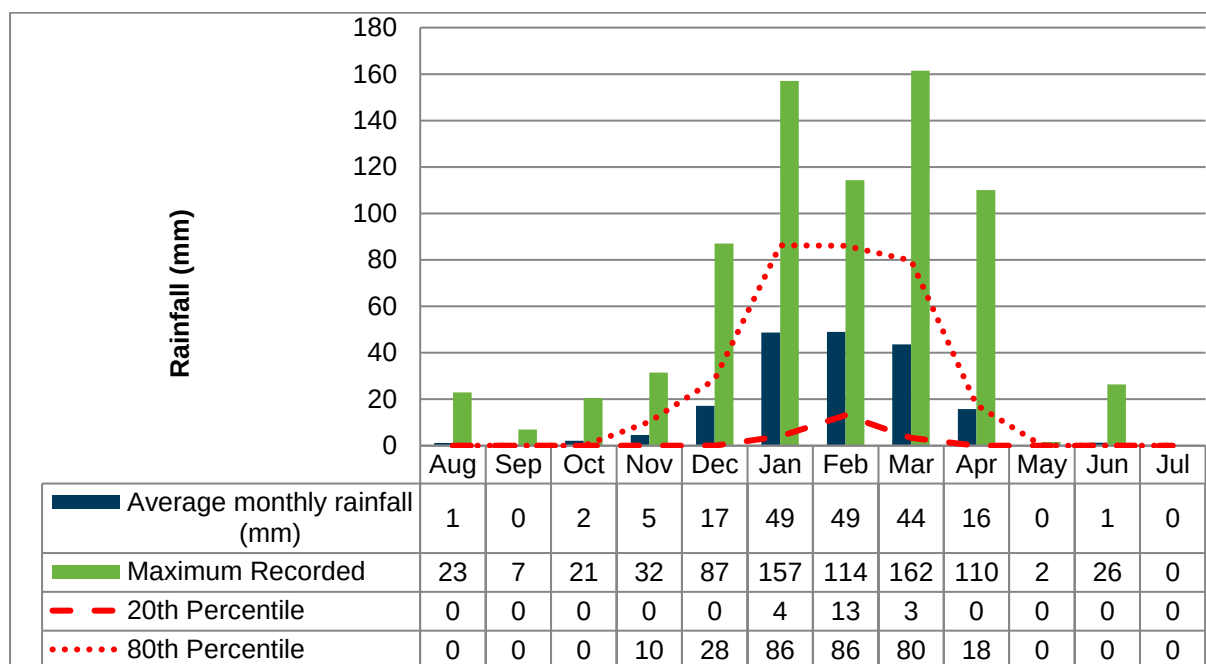


Figure 2-4: Average Monthly Precipitation

The same data set was used to update the 24-hour storm depths, with the estimates for each return period changing from the previous estimate due to the change in data set used. The annual maximum rainfall from the daily rainfall record was fitted to a Log Pearson III distribution to attain the estimates presented.

Table 2-1: 24-hour Duration Storm Depths

Return Period (years)	2	5	10	20	25	50	100	200	500	1000
Depth (mm)	42	64	77	88	91	101	110	118	128	135

2.4 EVAPORATION

The gross potential evaporation near the site is reported to be between 3 200 to 3 400 mm per annum (MET, 2002). The A-Pan and open water evaporation from the SLR Groundwater and Supply Study are shown in Figure 2-5 and Table 2-2. According to the SLR Groundwater and Supply Study, a conversion factor of 0.7 should be used to convert the A-Pan evaporation to Open Water evaporation (Kambinda and Bittner, 2020b), which correlates to the conversion recommended in TR102 (Adamson, 1980).

Table 2-2: Monthly A-Pan and Open Water Evaporation (Kambinda and Bittner, 2020b)

Month	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total
Monthly A-Pan Evaporation (mm)	346	343	356	327	256	237	215	202	179	199	243	301	3203
Open Water Monthly Evaporation (mm)	242	240	249	229	179	166	150	141	126	139	170	211	2242

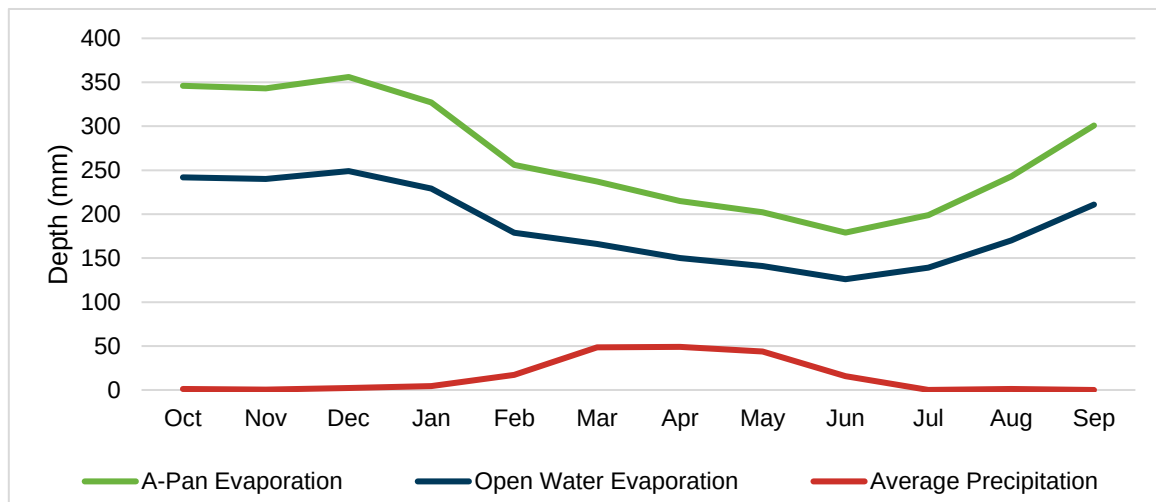


Figure 2-5: Average Monthly Rainfall vs. Evaporation

2.5 TOPOGRAPHY

The Twin Hills project area is at an altitude of approximately 1215 meters above mean sea level (mamsl) sloping gently towards the north and the Khan River valley at an altitude of 1070 mamsl. Surface geomorphology indicates high surface water runoff potential.

The sharp ridges occurring to the south and north of the proposed project area are the outcropping Karibib Marbles rising to a maximum elevation of 1420 mamsl. The hills have been and are still mined extensively for high-quality marble.

2.6 SITE GEOLOGY

The Twin Hills site is in the inland arm of the Pan-African Neoproterozoic Damara Belt (Miller, 2008). The Damara Belt is defined by a northeast striking Neoproterozoic fold, thrust and metamorphic belt and has been metamorphosed to greenschist facies from several deformation events. The Twin Hills ore reserves are hosted in the Kuiseb Formation of the Damara Supergroup lithologies, including the finely laminated mica schists (Biotite), Massive and bedded meta-greywacke, Cordierite Meta Greywacke (both massive and bedded) and finely laminated Cordierite Schist.

The ore zone occurs in the southern limb of the tightly folded syncline, slightly overturned towards the south. The meta-greywacke core of the syncline is approximately 100 m wide but thickens to over 200 m, caused by thrusting and repetition in the western portion of Twin Hills Central. Gold (Au) mineralisation is associated with three sulphide phases, namely pyrrhotite, arsenopyrite and pyrite. The host rocks include biotite and muscovite schist (Au in quartz-sulphide veins parallel or sub-parallel to foliation) and quartzite and turbidite (Au in breccia, stockwork and cross-cutting quartz-sulphide veins).

The Karibib Formation consists entirely of a thick marble succession with a few interlayered calcsilicate facies. It is a very characteristic and regionally developed unit throughout the area. The Karibib Formation occurs as massive- to banded, grey and white calcitic and dolomitic marbles. These are often interspersed with marble breccia horizons, particularly in the lower parts of the formation. The topographical ridges that dominate the landscape to the south and north of the ore bodies are comprised of the Karibib marbles and the Arandis–Spes Bona formations.

The Damara lithologies are overlain by thick quaternary deposits in the western portion consisting of sandy soils and calcrete units, ranging from nodular to hardpan in places. The estimated depth of this unit is around 30 mbs, thinning out towards the east.

3 SURFACE HYDROLOGY

3.1 CATCHMENT

The Twin Hills project area is situated within the Okawayo Catchment, which is a tributary to the Khan River, which feeds into the Swakop River and eventually discharges into the Atlantic Ocean. The Khan River catchment is approximately 8400 km² [SLR, 2020], and the Okawayo River catchment in which the Twin Hills Gold Project falls is 92 km² which is 1% of the total Khan catchment area.

The catchment, refer Figure 3-1, was delineated using Quantum Geographical Information Systems (QGIS) software and Shuttle Radar Topography Mission (SRTM, 2021) survey data. The proposed open pit mining operations occur towards the lower end of the catchment as shown.

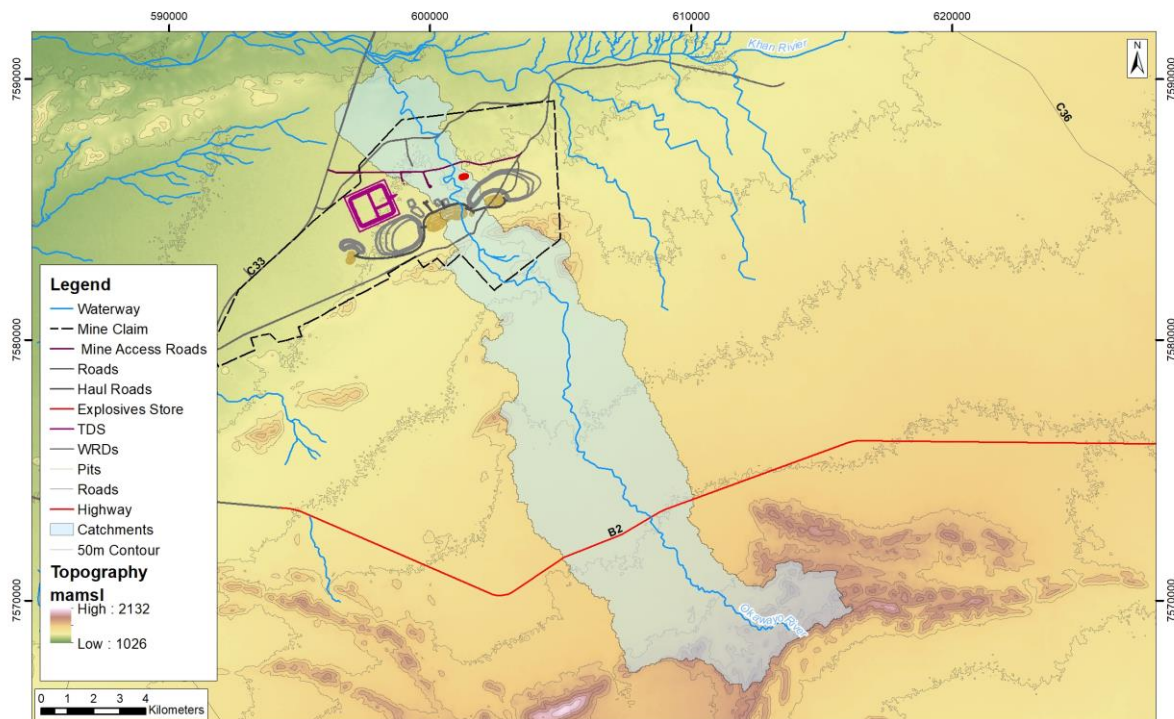


Figure 3-1: Okawayo Catchment Relative to the Kahn River

A summary of the Okawayo Catchment characteristics as well as that for the catchment specifically upstream of the proposed pits is shown in Table 3-1.

Table 3-1: Catchment Characteristics

Characteristics	Unit	Value
Catchment area upstream of pit	km ²	92
The total catchment area of the tributary (Okawayo River)		120
Watercourse length upstream of pit	km	28.3
Watercourse length for entire tributary		39.5
Slope upstream of pit	m/m	0.0082
Slope for the entire tributary		0.0078
Time of concentration ⁽¹⁾ upstream of pit	hours	5.85
Time of concentration for entire tributary		7.07

3.2 FLOOD PEAKS

Flood peaks can be estimated using a variety of methods based on the information available. Ideally, reliable flow data from a gauging station is used to determine the flood peaks of various annual exceedance probabilities (AEP). This data can also be used to determine the runoff response to rainfall events (such as representative runoff coefficients). If there is a lack of reliable flow data, deterministic methods are required where flood peaks and runoff response are calculated using estimates of the runoff coefficients.

The Rational Method has been adopted for this study. Runoff coefficients for the different AEP storm events are estimated based on the topography, vegetation, soils, catchment development, and other factors (SANRAL, 2013) and range from 0.143 to 0.355 for the floods presented.

The results of the flood peak assessment are presented in Table 3-2 and assume that the full Okawayo catchment contributes to the flood. This is assumed relatively conservative for the smaller probability events.

Table 3-2: Flood Peak Flows for the Twin Hills Catchment Area (m³/s)

Return Period	Flood Peak Rational Method (m ³ /s)
2	20
5	31
10	42
20	58
50	89
100	127
200	185
Q _{PMF}	233

3.2.1 RUNOFF COEFFICIENT CALIBRATION

In an effort to validate/calibrate the runoff coefficients used to calculate the flood peaks, a site inspection was conducted by Dr. Gert Cloete, hydrotechnical specialist, to assess historical flood line estimates and potential monitoring stations to be used in future monitoring. Information relating to historic floods along the Okawayo was investigated and used to calibrate a HEC-RAS 2D flood model.

Flood debris were observed at 6 stations upstream of the Twin Hills Site, with cross sections at these stations surveyed to capture the data for the HEC-RAS model. It should be noted that the observed data cannot be linked to a specific historic rainfall/storm event, the model will only be able to determine the flood peak that correlate to the observed debris level. Additional limitations/constraints to the model include:

- The accuracy of the terrain data – a detailed survey should be conducted in the next phase of the study to further refine the flood hydrology.
- There is no reliable weather station situated directly in the catchment – runoff coefficients will be based on inferences from other rainfall stations applied to this catchment.

It is proposed to install climatic stations in secured places within the catchment and to keep monitoring the floodline stations following rainfall events in the 2023-2024 rainy season. This should be combined with more accurate survey of the Okawayo hydrologic basin area.

3.2.1.1 SITE INSPECTION DATA

Table 3-3 below reflects a summary of the historic debris that was observed along the Okawayo river.

Table 3-3: Okawayo River Site Inspection Data

Cross Section Name	Station Observed 03/02/2023	Lowest River Invert (mamsl)	Debris observed (mamsl)	Flow Depth (m)	Debris Description
01_Fence	XS-114	1202.87	1203.53	0.66	Debris in middle of river against solid tree (not fence)
02_DS_Weir	Xsc-331	1204.58	N/A	N/A	Cross section 5 m D/S of weir structure
03_Weir	XSW-336	1205.25	1206.29	1.04	Debris on RB slope 3 m upstream of weir (more accurate)
04_Bend	XS-544	1206.44	1207.43	0.99	LB large, transported tree stump, rotted
05_Bend2	XSc-678	1207.61	N/A	N/A	Cross section 342 m U/S of weir, in low energy pool upstream of control point.
06_Tributary	XS-848	1208.06	1208.88	0.82	RB debris washed up against sloping rock
07_45degree Rock	XS-1098	1209.79	1210.765	0.98	RB debris trapped in bushes 4 m D/S of section (more representative)
08_US_Drop	XS-11163	1210.97	1211.868	0.90	Top gravel bar; gravel bar usually stationary, relates to low probability floods.

NOTES:

D/S SLOPE OF 0.0553 m/m

3.2.1.2 HEC-RAS MODEL

Terrain

The study area is in the catchment of a tributary of the Khan River, which joins the Swakop River and discharges into the Atlantic Ocean at the coastal town of Swakopmund. The topography and catchment details were derived from the Shuttle Radar Topography Mission (SRTM, 2021) dataset which was improved in 2020 and is called NASADEM, with a 1 arc second resolution. The catchments and main streams relevant to the study area were delineated using automated procedures in Quantum

Geographical Information Systems (QGIS) and manually verified using satellite images (Gericke & du Plessis, 2012). The topography can be described as steep in the upper reaches, with a maximum elevation of 1 550 meters above mean sea level (mamsl), up to the lowest elevation of 1 150 mamsl, Figure 3-2 below reflects the HEC-RAS model.

The catchment characteristics presented in Table 3-1 were used to define a 2D flow are in HEC-RAS. Figure 3-2 depicts the 2D flow grid/mesh for the catchment specifically upstream of the diversion structure, along with the centreline of the Okawayo River.

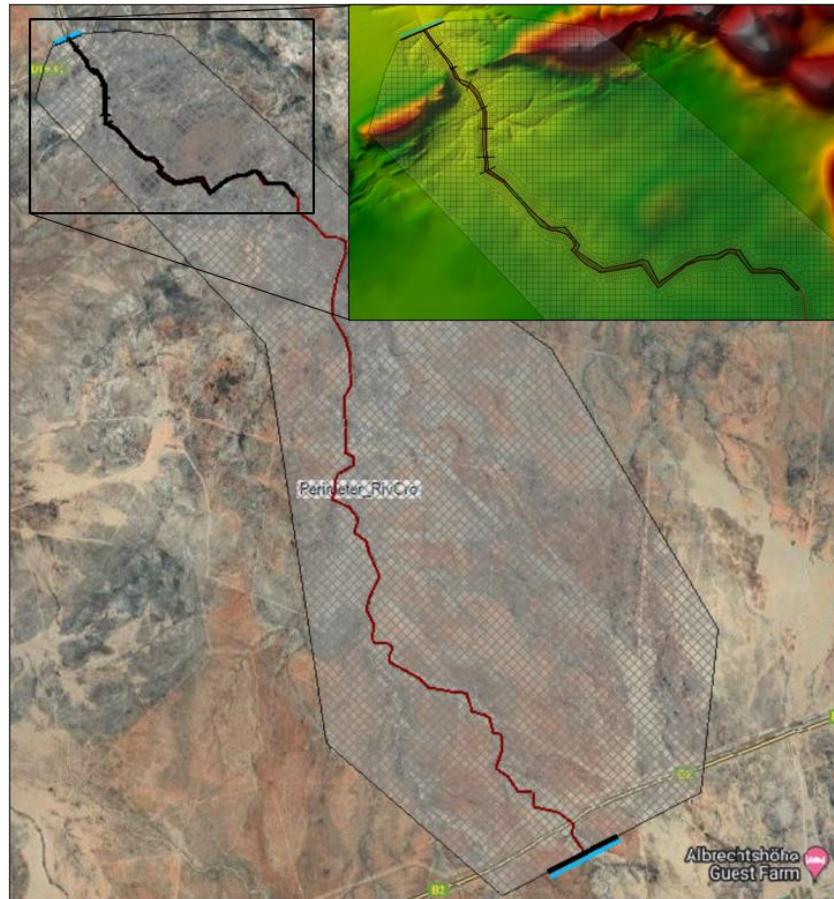


Figure 3-2: HEC-RAS Model Layout

It should be noted that the resolution of terrain data did not accurately capture all the river reaches in the Okawayo catchment, the satellite imagery visually reflects more detail of river reaches. Figure 3-3 shows that several of the tributaries shown in the imagery (left) are not adequately reflected in the terrain model (right). This justifies the recommendation that an updated detailed survey be undertaken.

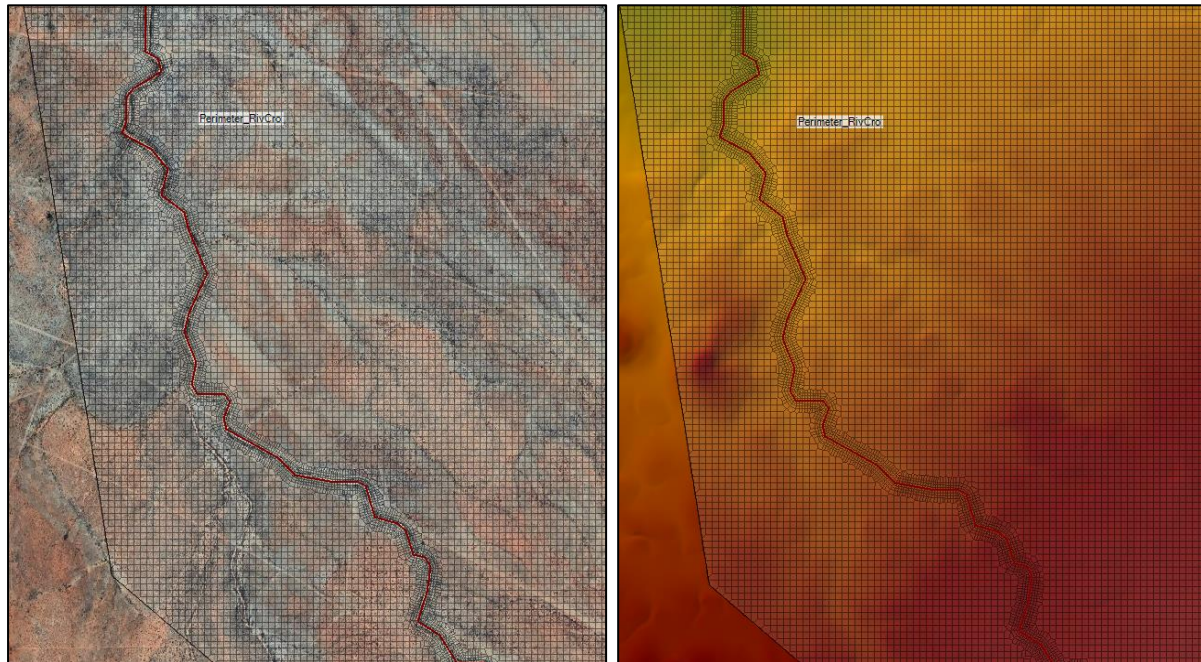


Figure 3-3: Terrain Data Accuracy

Geometry

A two-dimensional (2D) flow area was created on the Terrain, with a perimeter larger than the catchment area reporting to the confluence of the Khan and Okawayo river. A 50 x 50 m grid size was adopted for the overall geometry, and a 20 x 20 m grid was selected along the reach of the Okawayo river (refer Figure 3-2).

A land cover area was incorporated in the model to distinguish between the roughness coefficients of the river stream and the riverbeds. Table 3-4 reflects the roughness coefficients that were allocated in the model; these values were selected in consultation with the flood hydrology expert. It is envisaged that a more detailed analysis will be conducted once a detailed survey has been obtained for the next phase of the project.

Table 3-4: Manning's Roughness Coefficient

Roughness Coefficient	River Stream	Riverbeds
Manning's (n)	0.033	0.4

Unsteady Flow Data

Two boundary conditions were created for the unsteady flow analysis: The downstream boundary conditions considers normal flow depth and a downstream slope of 0.0055 m/m. The upstream boundary condition considers a flow hydrograph and an upstream slope of 0.008 m/m. An extensive range of flows was used to compare flood depths at the cross sections where flood debris was identified. The flow was increased by 10 m³/s over 6-hour intervals to allow for the range of flows to be evaluated in a single simulation. The 6-hour interval is greater than the calculated time of concentration for the catchment, which allows for the flow to gradually change with the changing flow rate. Figure 3-4 depicts the hydrograph used.

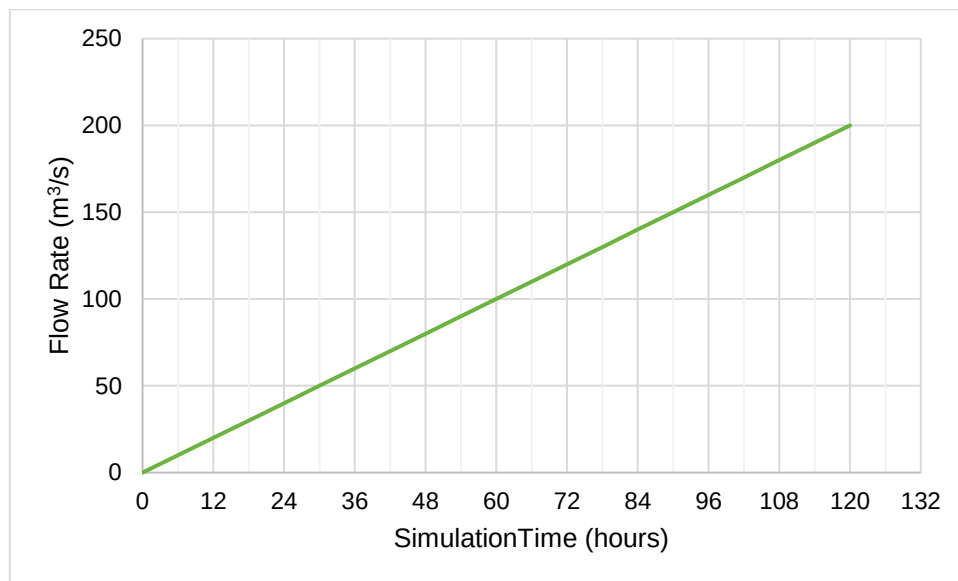


Figure 3-4: Hydrograph Used to Model Range of Flows for Calibrating the Runoff Coefficient

3.2.1.3 RESULTS

Six debris levels at various stations were observed. The simulation was run and the flow depth at each cross section was observed. When the flow depth at a cross section matched the observed debris level, the flow rate at that cross section was recorded. The flow depths and flow rates are presented in Table 3-5, below. The table reflects the flood peak required for the water surface to reach the observed debris level at that station. The flow rates were then compared to the flood peaks presented in Table 3-2 to estimate the return period, as calculated using the estimated runoff coefficients.

Table 3-5: Results Summary Table

HEC-RAS Section Name	Station Observed	Flood Depth Observed (m)	Water depth measured in the HEC-RAS Model H (m) to correlate to the Debris level.					
01_Fence	XS-114	0.66	0.66	1.63	1.39	1	1.71	*
02_DS_Weir	Xsc-331	n/a	0.41	1.14	0.94	0.64	1.32	*
03_Weir	XSW-336	1.04	0.35	1.04	0.85	0.57	1.22	*
04_Bend	XS-544	0.99	0.5	1.16	0.99	0.72	1.31	*
05_Bend2	XSc-678	n/a	0.68	1.48	1.28	0.93	1.61	*
06_Tributary	XS-848	0.82	0.55	1.31	1.11	0.82	1.44	0.14
07_45degree_Rock	XS-1098	0.98	0.5	0.85	0.65	0.57	0.98	0.2
08_US_Drop	XS-11163	0.90	1.09	1.57	1.46	1.26	1.68	0.9*
Flow Rate (m ³ /s)			16	105	76	37	137	3
Return Period (1:x years)			2	75	35	5	100	<1
Return Period (% AEP)			50%	1%	3%	20%	1%	100%

NOTES

1. IT IS NOT POSSIBLE TO KNOW THE DATE WHEN THE DEBRIS WAS TRANSPORTED AND/OR IF THERE WAS ANY HUMAN INTERFERENCE SUBSEQUENT TO THE FLOOD.
2. THE RIVER MAKES A NATURAL DAM IN THE VICINITY OF STATION XS-11163, ONLY ONCE THE RIVER WATER LEVEL REACHES ELEVATION 1221,56 MAMSL (ABOUT 1 METRE FLOW DEPTH) WILL STATION XS-114 RECEIVE ANY FLOW FROM THE U/S FLOOD.

3.2.1.4 LOW RETURN PERIOD RUNOFF COEFFICIENTS

Runoff coefficients are generally calculated for storm events that are less frequent in nature. For more regular rainfall, i.e. annual seasonal rainfall, runoff can be much lower than for high return period storms and in some cases the runoff down stream of a storm can be zero. To estimate the runoff coefficients to use for these types of events, the Okawayo catchment was compared to similar catchments in the Northern Cape of South Africa. Data relating to mean annual runoff (MAR, not to be confused with the MAR used in groundwater context) and mean annual precipitation (MAP) is well documented for these comparable catchments and an effective runoff coefficient can be estimated from the catchment area, the runoff and the MAP. These were then used to justify the low runoff coefficients presented in Table 3-6 for low return period precipitation depths.

3.2.1.5 CONCLUSION

The nature of the debris identified at each cross section gives an indication of the likely magnitude of storm needed to transport and deposit said debris. Larger debris and evidence of large scour would indicate that a lower AEP flood occurred (higher return period e.g. 50 to 100-year return period), whereas smaller scour and smaller washed debris likely indicate greater AEP floods (lower return period e.g. 1 to 5-year return period).

The results suggest that the higher AEP peak floods estimates are reasonable. The lower AEP flood estimates are difficult to relate directly to the results as it is impossible to know exactly when the debris

would have been deposited; it is only reasonable to assess the difference in the nature of the debris compared to the higher AEP floods. Without having flow records and rainfall gauge stations specific to the catchment in question, the results suggest that the runoff coefficients used to estimate the peak floods are reasonable.

The coefficients are summarized in Table 3-6, and are used in both the flood estimate and the runoff estimates used in the water balance.

Table 3-6: Runoff Coefficients

Return Period	Precipitation Depth (mm)	Runoff Coefficient (C)
<1	0	0
	2	0.003
	5	0.01
1	20	0.05
2	42	0.143
5	64	0.156
10	77	0.17
20	88	0.191
25	91	0.2
50	101	0.235
100	110	0.285
200	118	0.355

4 UPSTREAM STORMWATER MANAGEMENT

As there are no specific legislation/guidelines in Namibia at the date of this study relating specifically to infrastructure for surface water management, the decisions taken forward are based on industry standards and guidelines and the risk that the owner is prepared to adopt. The risks relate to the magnitude of flood that infrastructure is sized for, and the cost associated with mitigating the selected risk profile.

4.1 PRE-DEVELOPMENT FLOOD LINE

To identify the flood mitigation requirements, an understanding of the flooding pre-development is required. Figure 4-1 shows that the 50-year annual exceedance probability (AEP) flood will pass directly through the planned footprint of the Main pit. Mitigation for the flooding is identified in Section 4.2.

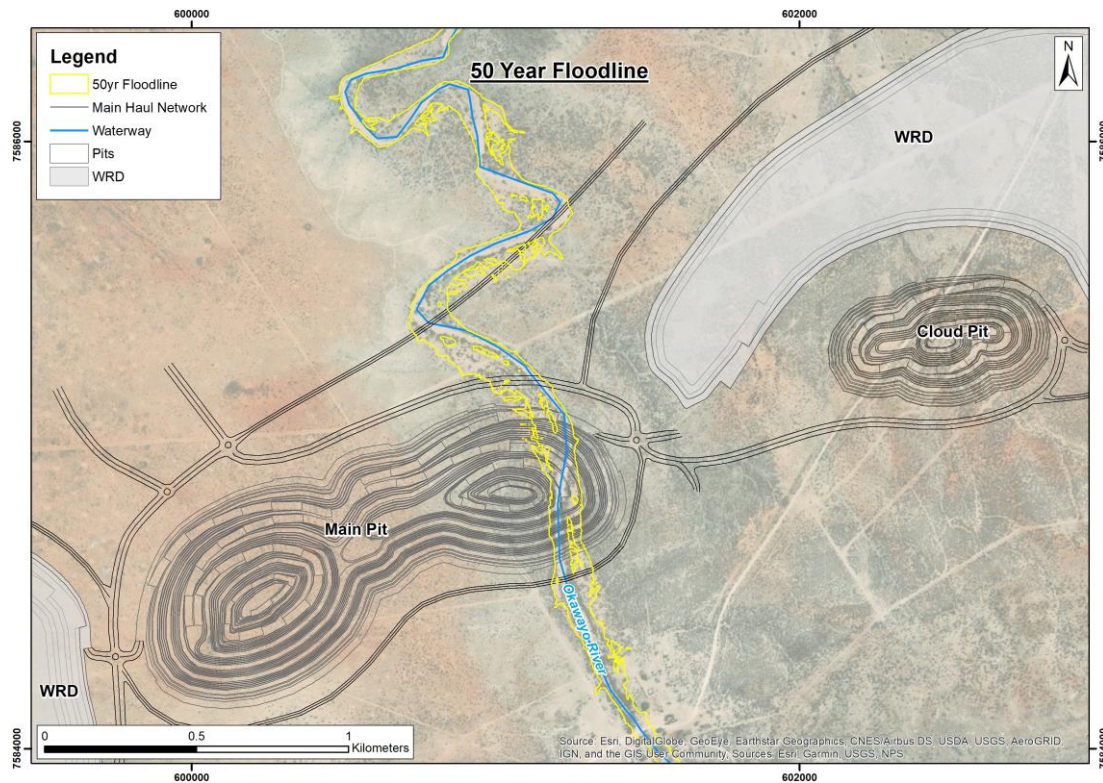


Figure 4-1: 50-year AEP Floodline of Okawayo river

4.2 FLOOD MITIGATION

Flood mitigation requires an understanding of the impact of flood risk. The preferred mitigation measure identified, is an upstream structure with an emergency spillway that conveys the discharge to the cloud west pit.

4.2.1 NO MITIGATION

If the pit were to be developed with no mitigation infrastructure in place, the impact of a 50-year AEP flood is shown in Figure 4-2 (flood impact modelled on V07 of pit design), while Figure 4-3 shows the impact of flooding on the pit for several AEP flood events. The mine schedule could offset the immediate requirement for flood mitigation if the western part of the pit were excavated first, but it is strongly recommended that flood mitigation infrastructure be constructed before the eastern sector of the Main pit is excavated.

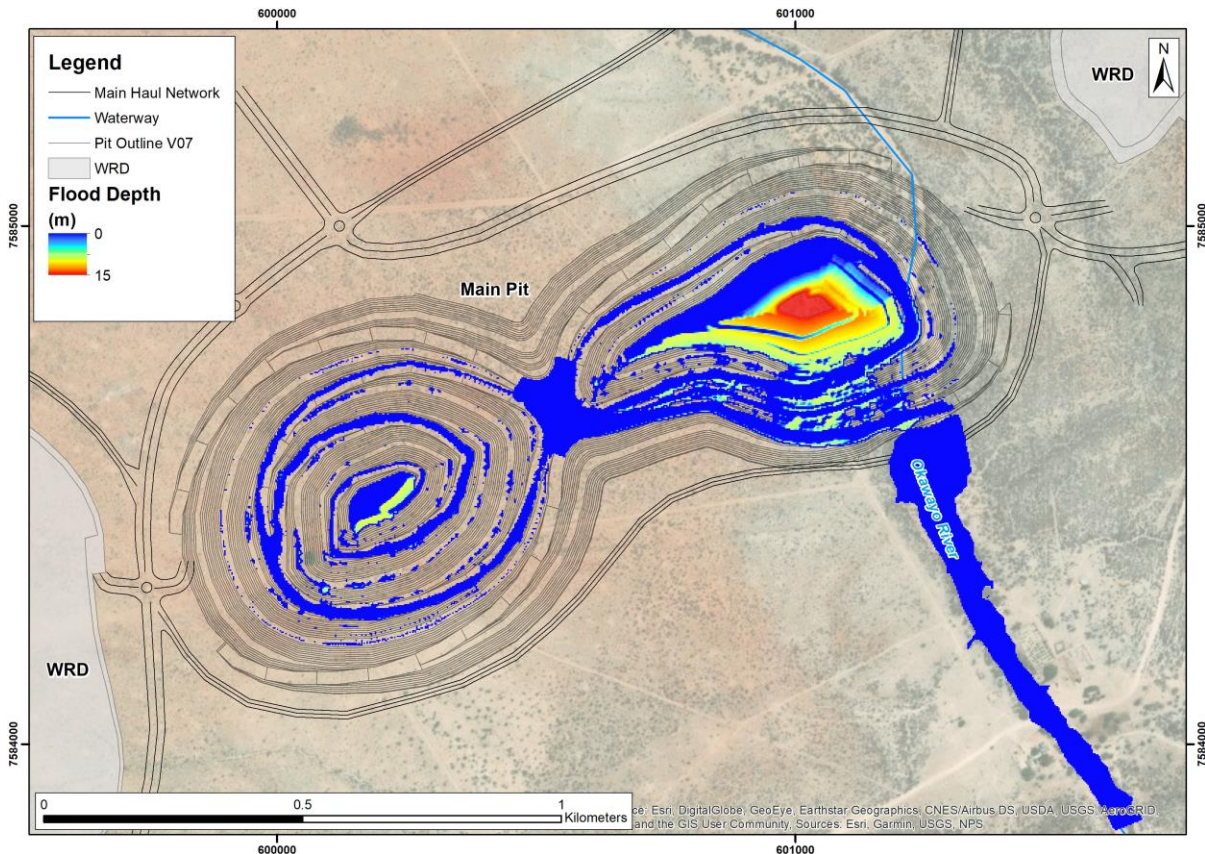


Figure 4-2: Plan View of the Pit Following a 50-year AEP Flood With No Mitigation Infrastructure (Flood Impact Modelling on V07 of Pit Design)

Figure 4-3 shows that the consequence of flooding is significant with a 2-year AEP flood resulting in the eastern section of the main pit potentially flooding up to 40 m in depth. The difference in consequence for the 20 year and 50 year AEP floods is 14% when comparing the flood depths of 90 m and 105 m, respectively.

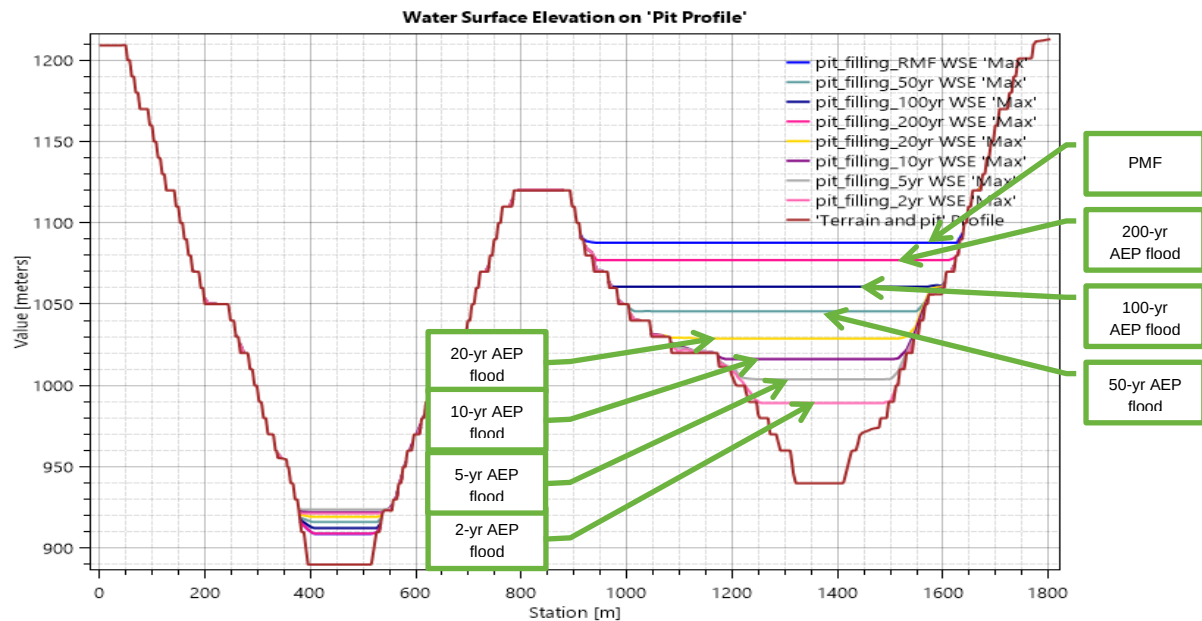


Figure 4-3: Cross Section of the Pit for Various AEP floods with No Mitigation Infrastructure

4.2.2 DIVERSION STRUCTURE

The proposed flood mitigation structure consists of a wall being built across the Okawayo river to divert flow away from the main pit. The flow will be routed to the cloud west pit via a spillway and discharge channel (the pit will require a downstream overflow channel for flood events larger than 1:5 AEP). Figure 4-4 shows the layout infrastructure.

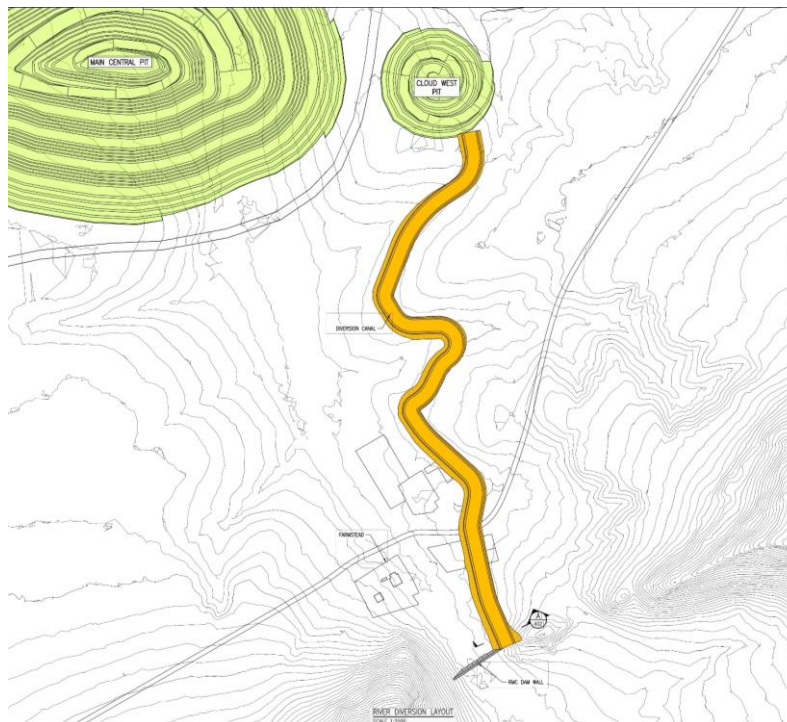


Figure 4-4: Flood Mitigation Infrastructure Layout

4.2.2.1 DAM AND SPILLWAY SIZING

The proposed flood mitigation structure consists of a diversion dam wall built across the Okawayo river to divert flow away from the main pit and a spillway channel to route flow to the cloud west pit.

The flood handling capacity of the infrastructure was determined based on the assumed low risk classification, which is a function of the wall height and downstream hazard classification (Risk to persons and the environment in the event of a dam failure occurring).

The hydraulic design considers that the spillway channel will come into operation for all flood events and can discharge to the cloud west pit exclusively up to the 1/50 AEP. For greater return period flood events, the main diversion dam wall will begin to overtop in tandem with the ongoing spillway channel releases to cloud west. The incremental releases over the main dam will be routed to the Main Pit, but the system (dam and spillway channel) is sized to accommodate the PMF.

As such, the mine must accept the risk of the main pit flooding for return periods in excess of a 1/50 AEP.

The intention to route flow from the diversion wall to the Cloud West pit requires a channel between the wall and the pit to have an appropriate grade to allow for flow to be driven by gravity. Flatter bed slopes require a bigger cross section to handle the required flow rates, but steeper slopes require a higher elevation and greater erosion protection requirements.

The selected channel slope of 0.15% allows for the spillway invert to be set at RL 1211 mamsl (5 m above base of wall), which requires the wall with a height of 7.25 m from base to overflow weir at RL 1213.25 mamsl, and a total height of 8.8 m from base to non-overspill crest (NOC) at RL 1214.80 mamsl.

For this height, the wall can be classified as a low-risk small dam, based on SANCOLD guidelines. However, an effective early warning system must be implemented by the mine (reducing probable loss of life to zero in the event of a dam breach or spillway flow) in the area downstream occurring.

4.2.3 DAM TYPE OPTIONS

Two options were assessed for the construction of the diversion wall, namely a rubble masonry concrete wall (RMC) and a concrete faced rockfill dam (CFRD). The selection of these types of wall was based on the likely availability of construction materials.

The profile of each dam option has been selected based on a stability assessment of each, with a cost comparison then being completed to consider in the selection of the preferred option.

4.2.4 AVAILABLE INFORMATION FOR STABILITY ASSESSMENTS

It should be noted that only a high-level geotechnical investigation has been conducted for the BFS, it should be expected that the estimated foundation level (EFL) will be different during the excavations which will impact the quantities as reflected in this report. It is recommended that a full geotechnical investigation be conducted for the detailed design stage to reduce the level of uncertainty on the foundation conditions.

The following available information was provided:

- Test pit logs (March 2023),
- Borehole test data (March 2023),
- Soil laboratory test results (March 2023),
- Rock laboratory test results (February 2023), and
- Open pit flood attenuation measures (January 2023).

The abovementioned data are the results from the high-level geotechnical investigation that was carried out at Twin Hills. It is estimated that foundation level is approximately 3 m below the natural ground level (NGL). This was adopted to determine the quantities and the stability of the dam types. This level may change once a detailed geotechnical investigation have been conducted.

4.2.5 RUBBLE MASONARY CONCRETE (RMC) DAM

4.2.5.1 GENERAL

RMC dams have been constructed successfully in South Africa, refer to Table 4-1 for recent dams.

Table 4-1: Recent RMC Dam Precedents in SA.

Dam	Max. Height (m)	Arch Radius Intrados (m)	Arch Thickness (m)	Arch Angle (Degrees)	Year of Completion
Mndwaka Dam	29	30	2.0 - 7.0	160	2015
Molepo Dam	18	14	1.8 - 3.6	140	2010
Zir Thangi	12	13	1.5	120	2006
Ghorqand Ghuriah	13	14	1.8	140	2005
Lucilla Pout	42	Log-spiral arch	4 - 7		2005
Bellair	20	14	2	140	2005
Kef-a	9	N/A	N/A	N A	2001
Aloe Cove	21	25	2	140	2005
Star	11	21	1.0 - 1.6	120	2000
Welgevonden	15	28	1.8	100	1999
Hogsback	11.5	20	1.0 - 1.6	143	1999
Genadendal	9	N/A	N/A	N A	1996
Bakubung	14.5	12	0.9 - 1.6	120	1996
Maritsane	18	60	2.4	105	1996
Castle Coombe	12	12	1.0 - 1.8	120	1990

Masonry structures have proven to be superior to the simple embankment dams in the past, with the oldest dating back to 4500 B.C. (Jansen, 1980).

Some advantages of RMC include:

- Conventional formwork is omitted,
- Little to no complications arising from heat of hydration effects,
- No vertical construction joints,
- Reduce river diversion strategy infrastructure and cost,
- RMC dams seep, but tend to self-seal after a few months in operation,
- Simplicity of construction activities and use of unskilled labour,
- Highly labour-intensive construction,
- Durability:

- Ability to pass floods directly over the wall during construction,
 - Allows for continuous construction, or construction over an indefinite period of time,
 - Reduced river diversion facility requirements,
- Skills transfer:
 - Masonry skills developed are useful in many other construction disciplines,
 - Site laboratories and offices are often maintained and staffed by local persons.

4.2.5.2 RMC DAM GRAVITY SECTION

The stability of an RMC wall is a function of the total mass of the structure and the ability for this mass to resist the moment forces created by the loads behind the wall. The RMC gravity profile was selected after conducting numerous stability analyses on a computer aided stability analysis program called CADAM. The program is based on the gravity method using rigid body equilibrium and beam theory to perform stress analyses, compute crack lengths, and safety factors. Seismic analyses could be done using either the pseudo-static or a simplified response spectrum method. CADAM provides a large number of modelling options, including:

- Crack initiation and propagation,
- Effects of drainage and cracking under static, seismic, post-seismic uplift pressure conditions,
- Safety evaluation procedures using deterministic, allowable stresses and limit states probabilistic analyses (Monte-Carlo simulations).

The profile in Figure 4-5 conformed to all the safety factors and allowable stresses, based on the assumed material properties. The section can still be optimised in the detailed design, once all the material and foundation properties have been refined.

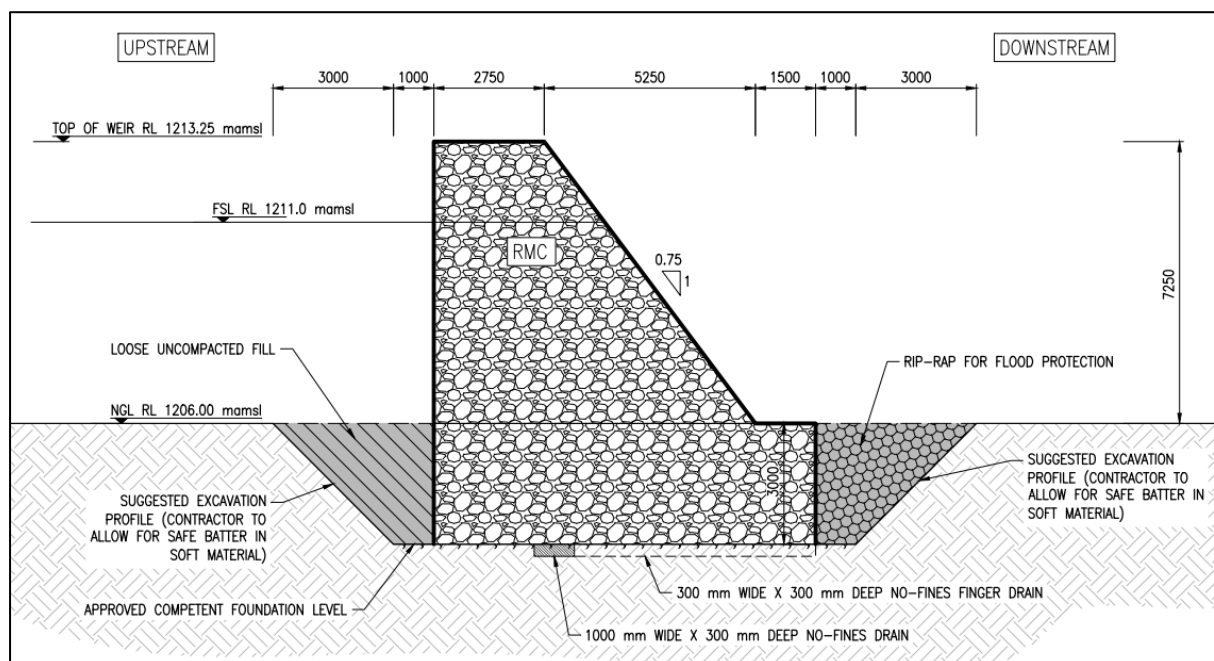


Figure 4-5: RMC Gravity Option Profile

4.2.5.3 LOAD CONDITIONS

The RMC stability analysis consider six load cases, which are sub categorised as follows:

- a) **Service Loading:** This load category includes loads to which the structure is subjected either permanently or for an extended period.
- b) **Abnormal Loading:** This load category includes abnormal temporary loads that can realistically be anticipated during the lifespan of the structure. These are usually the design level loads.
- c) **Extreme Loading:** This load category includes the most severe load conditions on the structure.

These loads are a combination of static and dynamic forces as discussed below.

Static Loads

The loads applied for the analyses addressed herein include hydrostatic, uplift, silt, and gravity.

- I. **Dead Loads and Gravity:** The RMC parameters are reflected in Table 4-2.

Table 4-2: Rubble Masonry Concrete (RMC) Parameters.

Description	Magnitude	Unit
RMC Density	2400	kg/m ³
Mortar Strength	14	MPa
RMC Comp Strength (use mortar strength)	14	MPa
RMC Tensile Strength	0.95	MPa
Cohesion between RMC and foundation (Peak)	250	kPa
Cohesion between RMC and foundation (Residual)	150	kPa
Peak Friction angle	45	degrees
Residual Friction angle	40	degrees
Elastic Modulus	18	GPa
Poisson's Ratio	0.2	-

- II. **Hydrostatic Forces:** This force is a horizontal load due to the upstream and downstream reservoir water levels. The FSL, RDF, and the SEF, together with the associated tailwater levels are normally considered for the stability analysis.

Table 4-3, below, reflects the catchment characteristics and the flood routing results, which were used in the analysis.

Table 4-3: Catchment Characteristics (KP, 2021).

Characteristics	Magnitude	Unit
Catchment area upstream of pit	92	km ²
The total Catchment area of the tributary (Okawayo River)	120	
Watercourse Length upstream of pit	28.3	km
Watercourse length for entire tributary	39.5	
Slope upstream of pit	0.0082	m/m
The slope for the entire tributary	0.0078	
Time of concentration ⁽¹⁾ upstream of pit	5.85	hours
Time of concentration for entire tributary	7.07	
Non-overspill Crest (NOC)	1214.8.	mamsl
Full Supply Level (FSL)	1211	mamsl
Recommended Design Discharge (RDD)	1213.25	mamsl
Tailwater Level RDD	-	mamsl
Safety Evaluation Discharge (SED)	1214	mamsl
Tailwater Level SED	1208	mamsl

- III. **Uplift Forces:** These forces act vertically under the base of the RMC structure, they occur because of the head water and tailwater pressures that permeate through cracks, joints, and pores into the foundation. The uplift pressure has a major destabilising effect on the structure. However, drainage is provided in the form of a no-fines subsurface drainage system, which helps reduce the uplift pressure. The stability analysis, however, only considered undrained conditions, for the sizing of the initial RMC gravity section, this section will be refined in the detailed design. The head water and tail water levels are reflected in Table 4-3.
- IV. **Silt Loads:** It is assumed that the flood mitigation structure will be fully silted up in the future, i.e. silt level will be at the FSL at RL 1211 mamsl. This assumption will be beneficial for the closing out of the mine since no additional work will be required to safeguard the structure. A friction angle and submerged density of 26 degrees and 9 kN/m³ was assumed respectively. It is assumed that the lateral pressure of the silt against the spillway will be in the “at-rest” state ($K_0 = 1 - \sin\phi = 0,56$).

Dynamic Loads

The analysis considers seismic loads for both the Operating Basis Earthquake (OBE) and the Maximum Credible Earthquake (MCE). The pseudo-static method is used to represent the seismic loads of an earthquake as a constant horizontal and vertical force by multiplying the inertia forces of the spillway with a coefficient. A conservative approach included the application of the peak horizontal acceleration in the most unfavourable direction, perpendicular to the dam axis, and the simultaneous application of horizontal and vertical accelerations.

In accordance with current seismic regulations provided in Bulletin 72 of ICOLD (1989), Eurocode 8 (2004) and ASCE (2005), three seismic designated levels were considered. namely the operating basis earthquake (OBE), maximum design earthquake (MDE) and maximum credible earthquake (MCE).

The results of the PSHA are given in terms of mean return periods and probabilities of being exceeded for horizontal component of PGA:

- Operating basis earthquake (return period 144 years) = 0,1g
- Maximum design earthquake (return period 475 years) = 0,021g
- Maximum credible earthquake - MCE (return period 10 000 years) = 0,114g

The Westergaard theory was considered to determine the dynamic water effect on the spillway due to the earthquake.

4.2.5.4 LOAD CASES AND COMBINATIONS

Table 4-4 describes the various load cases and combinations that will be used for the analysis.

Table 4-4: Concrete Gravity Loading Case and Combinations for Analyses.

Load Case	Loading Condition	U/S Water Level	Self-weight	Silt Load	Tailwater level	Uplift/ Drainage	Seismic Loading
1	Normal	No	Yes	Yes	No	No	
2	Normal	FSL	Yes	Yes	No	Undrained	
3	Unusual	RDF	Yes	Yes	RDF	Undrained	
4	Unusual	No	Yes	Yes	No	No	OBE
5	Unusual	FSL	Yes	Yes	No	Undrained	OBE
6	Extreme	SEF	Yes	Yes	SEF	Undrained	

NOTES

OBE: OPERATING BASIS EARTHQUAKE (0.1 g)

The factors of safety and allowable stresses for the gravity spillway structures were evaluated against the criteria listed in Table 4-5. For the base crack analysis, it is assumed that a crack will initiate and propagate only once the allowable tensile stress is exceeded.

Table 4-5: Minimum Stability Factors of Safety for Gravity Dam Structures.

Load Category	Load Case	Tensile Stress (MPa)	Compressive Stress (MPa)	Shear Friction SF (Peak)	Shear Friction SF (Residual)	Overturning SF
Normal	1	0.0	4.2	2.5	1.5	1.5
Normal	2	0.0	7	2.5	1.5	1.5
Unusual	3	0.29	7	2.0	1.1	1.2
Unusual	4	0.29	12.6	2.0	1.1	1.2
Unusual	5	0.29	12.6	2.0	1.1	1.2
Extreme	6	0.87	4.2	1.5	1.0	1.1

NOTES

SF: factor of Safety

Figure 4-6 reflects the load combinations as defined in Table 4-4.

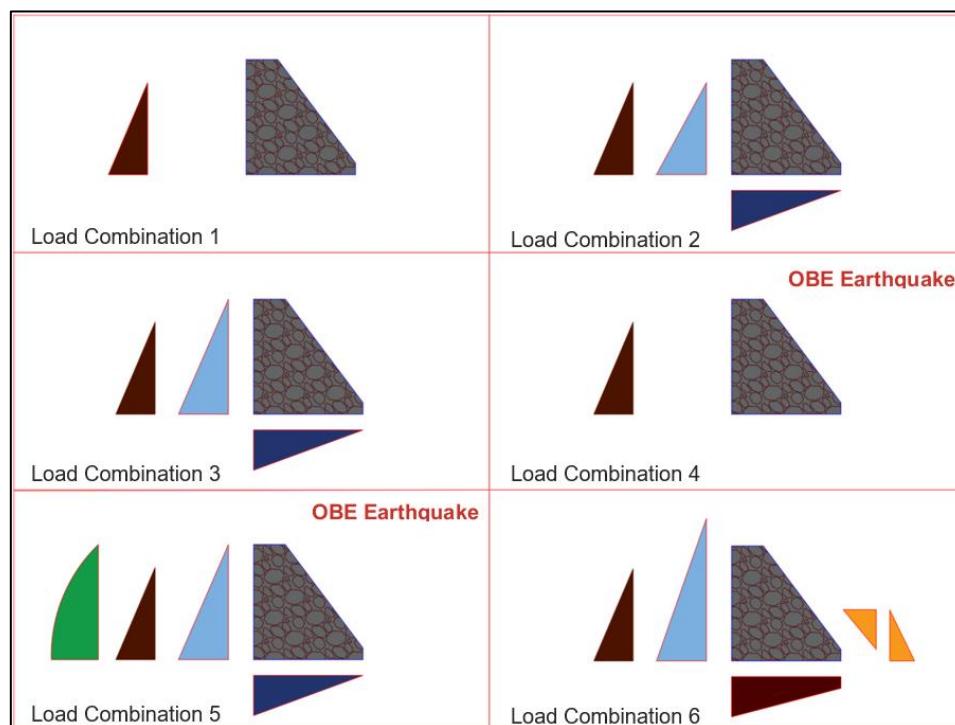


Figure 4-6: RMC Structure Load Diagrams

4.2.5.5 STABILITY RESULTS

The results of the stability analysis for the deepest RMC section (NOC to EFL) are reflected in Table 4-6. It is evident that all the stresses and factors of safety meet the minimum/maximum criteria. Due to the stresses calculated not exceeding any of the allowable stresses, it meant that no crack analyses or lengths were undertaken or determined respectively. Appendix B reflects the detailed stability drawings of the CADAM stability analyses.

Table 4-6: RMC Stability Analysis Results.

Load		Calculated Stress (MPa) ⁽¹⁾		Allowable Stress (MPa) ⁽¹⁾		Calculated SF for Sliding		Allowable SF for Sliding		Overturning SF	
Category	Case	Heel	Toe	Heel	Toe	Peak	Residual	Peak	Residual	Actual	Allowable
Normal	1	-0.25	-0.07	0	4.2	25.1	17.6	2.5	1.5	20.9	1.5
Normal	2	-0.12	-0.13	0	7	7.7	5.2	2.5	1.5	2.4	1.5
Unusual	3	-0.04	-0.18	0.29	7	5.4	3.7	2	1.1	1.8	1.2
Unusual	4	-0.19	-0.11	0.29	12.6	11.8	8.2	2	1.1	14.9	1.2
Unusual	5	-0.04	-0.18	0.29	12.6	5.1	3.4	2	1.1	1.7	1.2
Extreme	6	0.03	-0.21	0.87	12.6	3.8	2.5	1.5	1	1.3	1.1

NOTES

(1) +ve = Tension, -ve = Compression

4.2.6 CONCRETE FACED ROCKFILL (CFR) DAM

The design and construction of Concrete Faced Rockfill Dams (CFRD) has improved substantially during the last 40 years. This type of dam has proven to be a safe structure long term and is well suited to withstand static and dynamic earthquake loading. In some circumstances CFRDs can offer a cost-effective alternative in both wide and narrow valleys compared to other types of rockfill and concrete dams.

Compared to other types of dams, CFRDs offer advantages that include:

- Foundation grouting does not affect the critical path activities, since the grout curtain is located on the upstream side of the CFRD embankment,
- CFRDs have historically performed well during earthquakes,
- CFRDs with heights exceeding 150 m have been constructed at numerous sites,
- CFRDs can perform well even if large leakages occur through the upstream concrete face,
- CFRD construction offers a cost-effective solution for many sites,
- Rockfill of various quality can successfully be included in construction of the CFRD embankment.

4.2.6.1 GEOMETRY

Based on the proposed design, the CFR embankment has a crest width of 6 m, height of 10 m, and a 330 mm thick concrete face on the upstream slope of the embankment as seen in Figure 4-7. Side slopes of 1:2 (V:H) were considered. The full supply level of the dam is set at 5 m and non-overspill crest (NOC) level was set at 7 m.

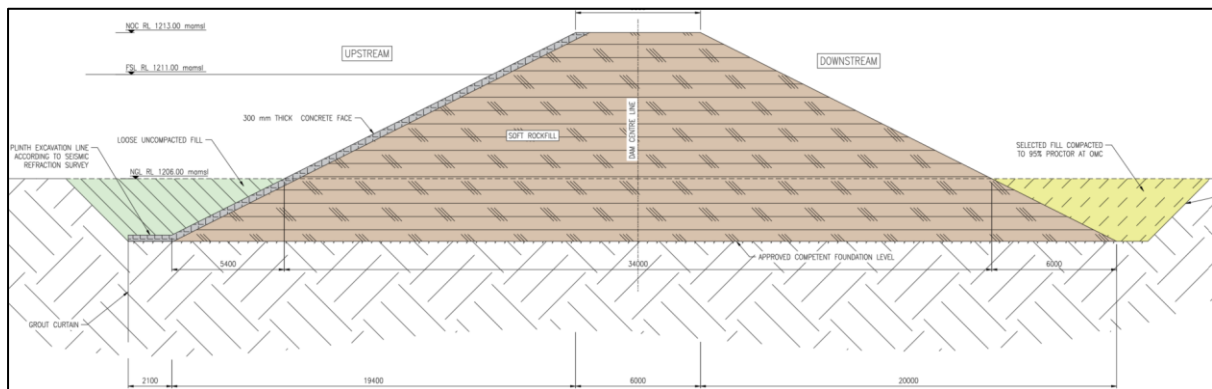


Figure 4-7: Embankment Cross Section.

4.2.6.2 GEOTECHNICAL PARAMETERS

Geotechnical parameters for the stability analysis were inferred from the results of the geotechnical investigation completed in March 2023. BH01 and BH03, and TP13, TP01 and TP14 are located near the footprint area of the embankment, as seen in Figure 4-8.



Figure 4-8: Layout of Test Pit and Borehole Positions.

The available borehole data was used to determine the soil profile below the embankment. The borehole data indicated that the bedrock level varies along the proposed footprint with a maximum depth of 5.5 m below NGL. Bedrock is overlain by alluvium and residual soil of varying thickness.

It is expected that soft or loose material will be removed from the embankment footprint before construction of the embankment will commence. The model used for stability analysis consists of a rockfill embankment underlain by a 3 m thick layer of residual material followed by bedrock.

The strength parameters of the residual material were inferred from the available geotechnical information. Typical shear strength parameters from literature were used for the rockfill and concrete. Permeability of the materials was based on literature. The material parameters used in the model can be seen in Table 4-7.

Table 4-7: Material Parameters Embankment.

Material	Unit Weight (kN/m ³)	Phi (°)	Cohesion (kPa)	Permeability (m/s)
Concrete	23.5	40	500	1x10 ⁻¹⁰
Rockfill	19	38	0	1x10 ⁻⁴
Sandy-Gravel	21.6	32	0	3x10 ⁻⁶
Bedrock	21.5	39	88	1x10 ⁻⁹
Silt	16	26	0	1x10 ⁻⁵

4.2.6.3 PHREATIC SURFACE

The location of the phreatic surface was based on a two-dimensional finite element seepage analysis conducted in Slide2 software from Rocscience. The upstream water level was varied according to the load case being analysed. Due to the low permeability of concrete, the phreatic surface is reduced as it passed through the concrete and into the rockfill embankment as seen in Figure 4-9. The level of the phreatic surface at the downstream toe is highly influenced by the groundwater level.

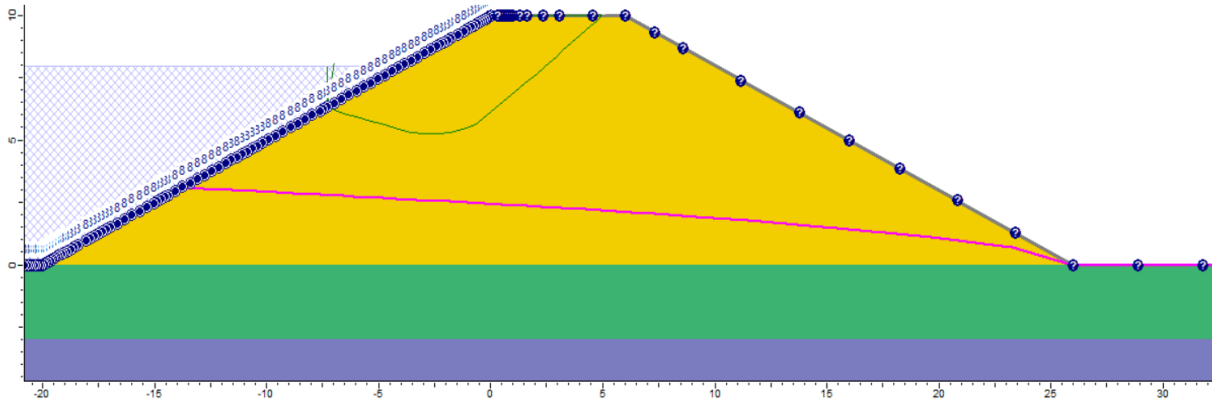


Figure 4-9: Phreatic Surface Location (Pink Line).

Groundwater was not encountered in the boreholes completed in the embankment footprint area. It is therefore not clear what the water table level is. It was assumed that the water table level would be at least 3 m below ground level for the stability model.

4.2.6.4 STABILITY ANALYSIS AND RESULTS

A two-dimensional limit equilibrium analysis of a cross section of the embankment (Figure 4-10) was conducted using Slide2 software from Rocscience. The factor of safety was analysed using both the Morgenstern and Price, and Spencer methods along with non-circular auto refine search method. The minimum depth of the computed slip planes was limited to 4 m to ensure that surficial and near surficial surfaces were not included in the calculation of the factor of safety. The following load cases were analysed to determine whether the embankment complies with the international best practice and ICOLD standards:

1. End of construction (no water),
2. Long-term: steady seepage from full supply level,
3. Maximum surcharge pool (water level at NOC),
4. Seismic MCE (maximum credible earthquake) at full supply level, and
5. Long-term: silt at full supply level.

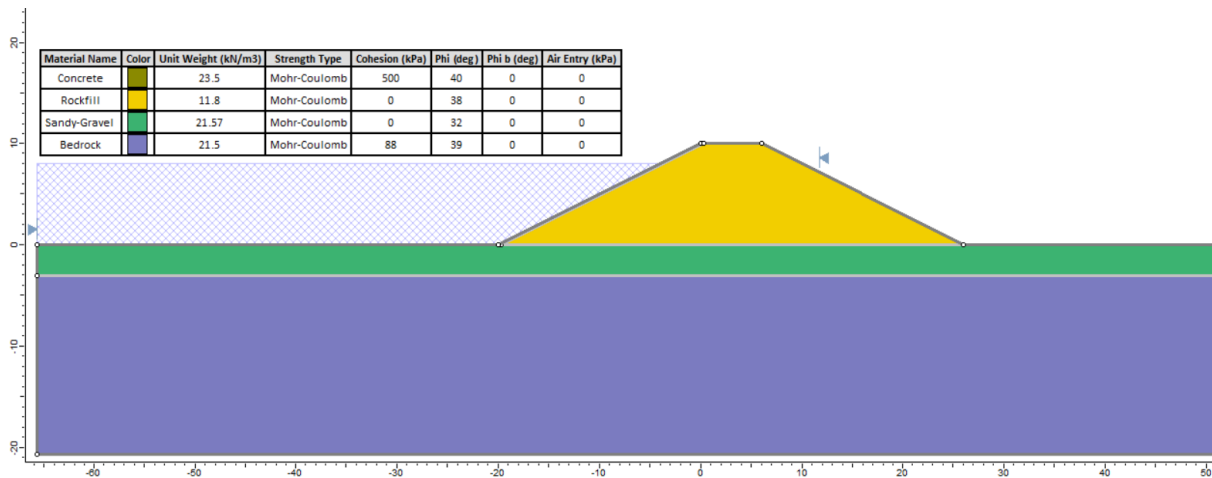


Figure 4-10: Cross Section of the Embankment.

The required factor of safety (FoS) for the five load cases that were considered are indicated in Table 4-8. The analyses were undertaken in accordance with the recommended approach set out in ICOLD Bulletin 157-Small Dams.

Table 4-8: Load Case and Required Factor of Safety.

Load Case	Minimum Factor of Safety (FoS)
End of construction	1.3
Long term-steady seepage	1.5
Maximum surcharge pool	1.4
Seismic	1.1
Long term-silt build-up	1.3

The seismic analysis conducted for the maximum considered earthquake represented a 144-year recurrence interval seismic event. A peak ground acceleration of 0.1 g was used for the analysis (WI301-00913/06-A, March 2023). The results obtained after computing the model is seen in Table 4-9.

Table 4-9: Results of the Stability Analysis.

Load Case	Factor of Safety (FOS)		
	Required	1:2 (V:H) Slope	
		Upstream	Downstream
End of construction	1.3	1.7	1.7
Long term-steady seepage	1.5	6	1.5
Maximum surcharge pool	1.4	15.2	1.4
Seismic	1.1	3.3	1.2
Long term-silt build-up	1.3	5.5	1.5

The results of the slope stability analysis indicate that the load cases comply with the required factors of safety for the 1:2 (V:H) side slopes. It should be noted that the seepage analysis is dependent on the groundwater level. A conservative assumption was made regarding the groundwater level for the analysis. The groundwater level and the permeability of the residual material in the foundation should be confirmed for further analysis.

4.2.1 SUMMARY

The results from the stability analysis indicated that all five load cases complied with the required factors of safety both upstream and downstream of the embankment for both slope scenarios. It can be deduced that the overall stability of the embankment is expected to be good. This indicates that a Concrete Faced Rockfill (CFR) Dam Wall is a suitable design for this embankment with side slopes of 1:2 (V:H).

The addition of a concrete face lowers the phreatic surface and increases the stability of the embankment, as confirmed by the factors of safety achieved. It should be noted that the stability of the embankment is significantly influenced by the phreatic surface which is affected by both the groundwater elevation and the upstream head water level. The geotechnical investigation indicated that the ground water level lies deep in the foundation, but the elevation is unknown at this point. The upstream headwater level will fluctuate seasonally, which will result in the stability of the dam fluctuating accordingly.

The stability analysis was based on the analysis and interpretation of the geotechnical investigation and literature. It should be noted that there is a possibility that the actual material parameters differ from those used as the input parameters in the analysis.

4.2.2 SPILLWAY DISCHARGE CHANNEL

The spillway discharge channel will convey water from the diversion structure to the cloud west pit, approximately 1.1 km downstream of the diversion structure. The channel is sized to safely convey the 50-year AEP flow while providing nominal freeboard (20 m wide, 3 m deep). The RMC weir would provide additional capacity for larger flood events with a flood up to PMF being able to pass over the RMC weir and spillway without compromising the structure. In this event, the proportion of flow discharging over the spillway is equivalent to the 1:100-year AEP flow with no freeboard into the discharge channel, while the RMC weir would discharge the remaining proportion.

The premise of the channel design is to minimize the need to line the channel which will reduce the CAPEX of the channel. The performance will then be monitored during the operational life and should the channel require maintenance it will be done in the dry seasons.

The geotechnical investigation indicates that bedrock is expected to be encountered in depths ranging from as shallow as 0.3 m to 2.3 m. Given the depth of channel required (refer Figure 4-11), this will require excavation in hard rock, but does have the benefit of being more durable than an unlined excavated in softs.

The mild slope of the channel (see long section in Figure 4-12) results in a subcritical flow regime for the design flow, with a flow velocity of approximately 1.2 m/s. These flow conditions are favourable for the design philosophy described above, however, conditions will need to be evaluated during excavation to assess the final suitability.

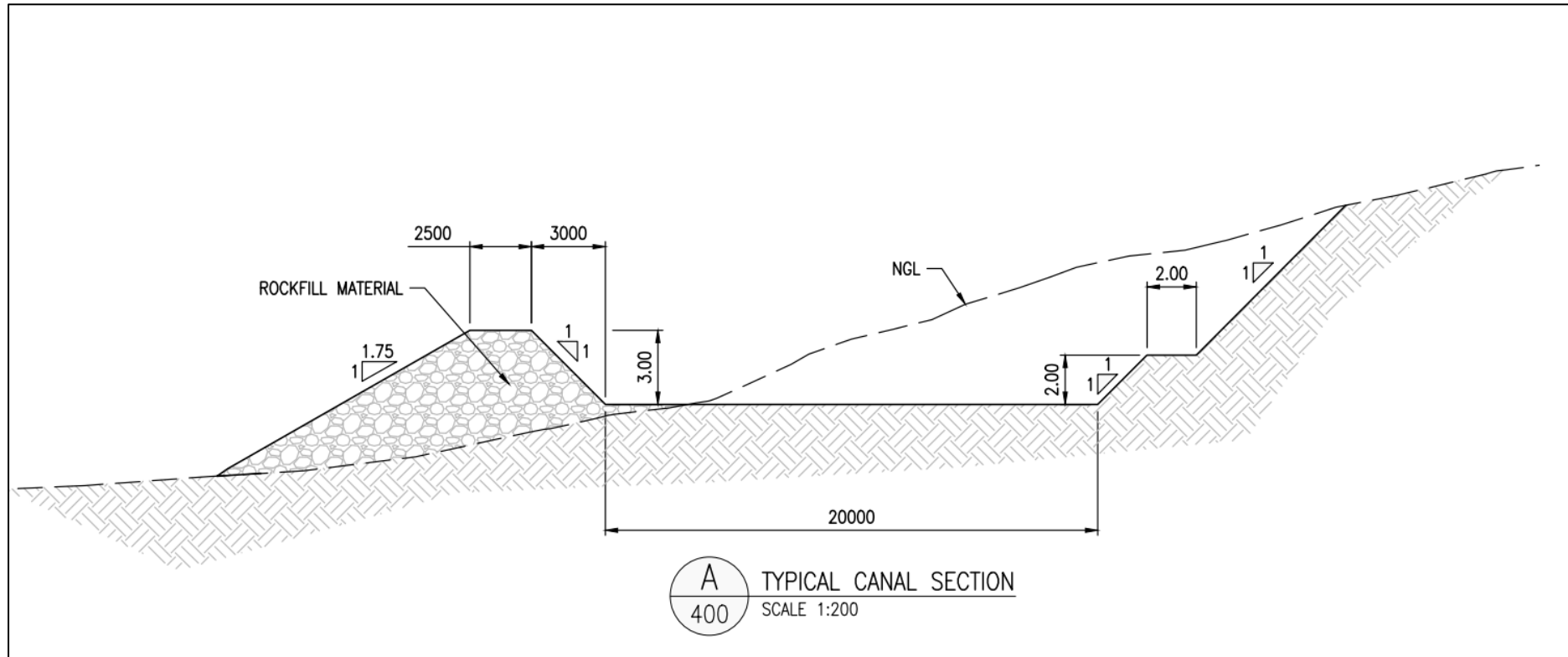


Figure 4-11: Typical Cross Section of Spillway Discharge Channel

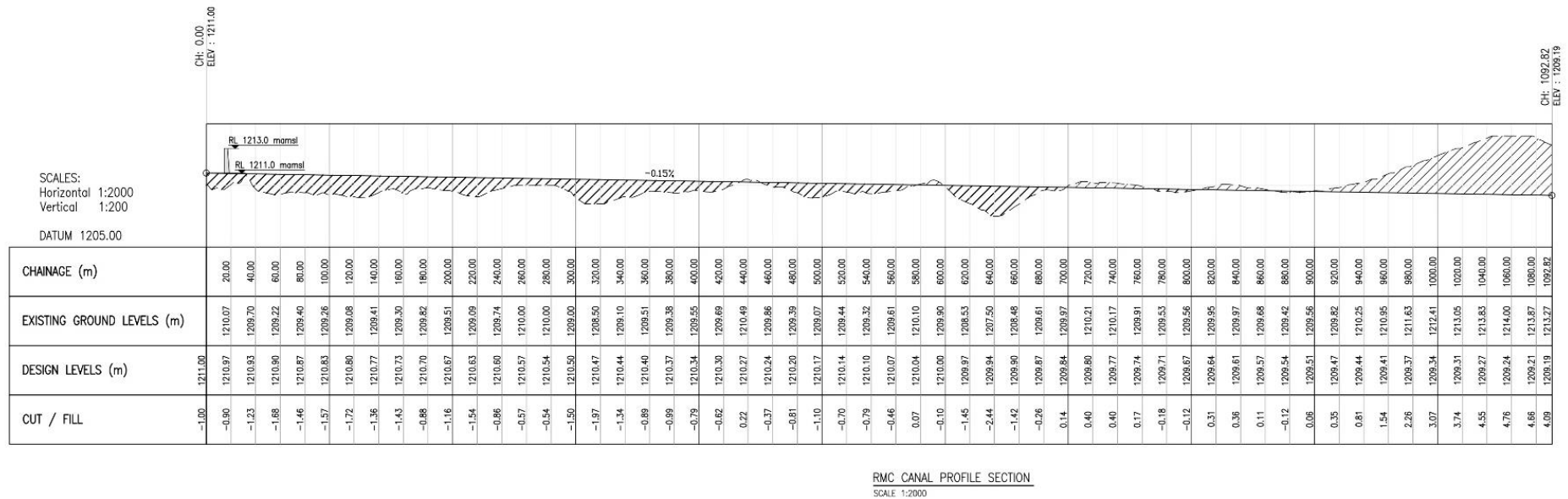


Figure 4-12: Long Profile of spillway Discharge Channel

4.3 COST ESTIMATE

A cost comparison has been conducted to determine the most cost-effective option to take to the detailed design. The details of the channel are the same for both the RMC and the CFRD wall options and the comparison is made between the wall options only.

The cost of the RMC wall amounts to 7.98 Million Namibian Dollars at the time of reporting, while the CFRD cost amounts to 12.79 Million Namibian Dollars. The more cost-effective option is therefore the RMC wall.

Table 4-10 to Table 4-12 summarizes the total cost of construction, including estimated P&G costs and engineering fees for developing the design in the next phase.

Table 4-10: Cost of RMC Wall in Year 1 (Rates in Namibian Dollars)

Okawayo Diversion Stream P&G's	873 883.35
	10.43%
Okawayo Diversion Stream Construction Costs	7 979 576.81
5% on design development	398 978.84
Year 1 RMC Dam	9 252 439.00

Table 4-11: Cost of Channel Excavation in Year 2 (Rates in Namibian Dollars)

Okawayo Diversion Stream P&G's	954 026.34
	10.43%
Okawayo Diversion Stream Construction Costs	8 711 376.02
5% on design development	435 568.80
Year 2 Channel Excavation	10 100 971.16

Table 4-12: Combined Cost of Diversion Structure (Rates in Namibian Dollars)

Okawayo Diversion Stream P&G's	1 827 909.70
	10.43%
Okawayo Diversion Stream Construction Costs	16 690 952.83
5% on design development	834 547.64
TOTAL OKAWAYO STREAM DIVERSION	19 353 410.17

4.4 PREFERRED OPTION

Based on the cost, the RMC dam wall is the preferred option to take forward to the detailed design. The option provides additional advantages in that it provides lower consequences in overtopping and does not require significant diversion requirements during construction (refer Section 4.2.5.1). The total cost for this option is estimated at **Nam \$19 353 410.17** and excludes contingencies.

5 WATER SUPPLY AND WATER BALANCE

5.1 GENERAL

The PFS evaluated the potential for several water supply options and the potential annual quantities that could be required to make up the deficit of water supply that could be obtained from pit dewatering and borehole supply.

These options consisted of water recycled from a Karibib Town Treatment Plant, water sourced by an offtake to NamWater supply pipelines, and possibly creating a source by building a dam on the Khan River. These options do not form part of the scope of this study as the feasibility of these sources is being investigated by other consultants. The basis of this study is to quantify the overall supply available from the surface and groundwater options of the site itself, and quantify the deficit, if any exists.

This is achieved by undertaking a site specific water balance, encompassing the groundwater recharge for the open pits, the borehole supply, and the surface runoff along the Okawayo river, which will be managed by means of the flood mitigation structure discussed in Section 4.

5.2 WATER BALANCE

The water balance represents a site-wide review of the surface water components, with inclusions from the groundwater model. The key components that contribute to water supply are (refer Figure 5-1):

1. Subsurface water supply abstracted through boreholes
2. Seasonal runoff generated from the upstream catchment and stored in the reservoir created by the diversion structure, as well as the mined out pits;
3. Ground water recharge seeping into the pits;
4. Supply from the Khan River sand dam storage system;
5. Fresh water supply from connection to the NamWater supply pipeline;
6. Recycled water from Karibib Town Treatment works;
7. The TSF seepage collection pond return

NOTE: ITEMS 4 THROUGH 6 ARE CONSIDERED BACK UPS TO COVER DEFICITS IN SUPPLY AND THEREFORE NOT FOCUSED ON IN THIS WATER BALANCE, ITEM 7 IS YET TO BE CONFIRMED AS THE SEEPAGE POND IS INTENDED TO BE AN EVAPORATION POND AND THE POTENTIAL TO PUMP NEEDS TO BE CONFIRMED.

The process flow diagram indicates the diversion structure upstream of the pits - this is only relevant after year 4 of mine operation. For years 1 to 4 the runoff from the upstream Okawayo catchment will flow directly into the main pit which is situated in the Okawayo flow path.

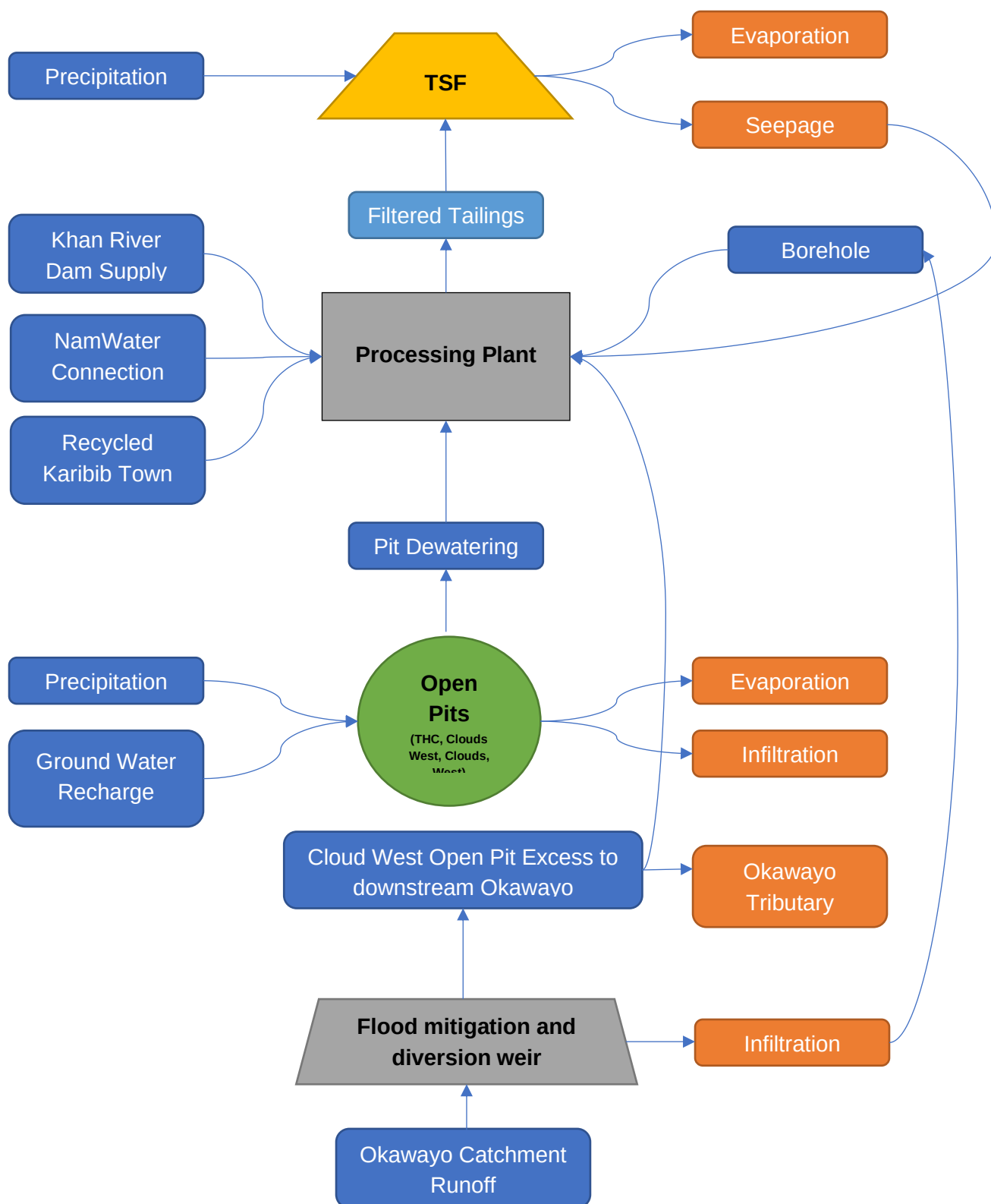


Figure 5-1: Water Balance Flow Chart

5.2.1 METHODOLOGY

The water transfers and storages were modelled using a GoldSim 14.0 model. The model is based on deterministic inputs with deterministic outputs. The climatic inputs are based on historical records, while the pit pushbacks were provided to KP. The inputs for the diversion structure are taken from the stage capacity curve and spillway characteristics developed by KP. The model is calculated on a daily time step, with results of the inflows presented on an annual time step and results of the plant supply/deficit presented on a daily time step.

This model forms the third iteration of the water balance. This iteration takes into consideration the four separate pits that are to be developed with the pit inflows and available storage capacity changing with time according to the pit progression provided to KP. The model also considers the use of the cloud wet pit as a general water supply reservoir.

The intention behind this is that when the volume of water dewatered from the main, west, and cloud pits is greater than the plant demand, the excess water will be pumped to the cloud west pit to be stored. This will increase the efficiency of water utilization for the site, as excess water from pit dewatering will not be discharged directly to the environment. The exact operating philosophy of the cloud west pit will require refinement in the next phase with regard to the water pumped to it from the other pits to ensure that the pits remain dewatered for active mining while also only sending water to cloud west pit when there is capacity to receive additional water.

The outcomes from this model are a time series of results that include parameters such as:

- Inflow volumes
- Water supply to the plant by different sources
- Reliability of supply to the plant

Several scenarios have been modelled to test the sensitivity of the supply. The results of these have been included for comparison.

5.2.1 DESIGN CRITERIA, ASSUMPTIONS, AND LIMITATIONS

The key inputs into the water balance are summarized below:

- **Processing Plant Water Requirement:** The process water requirement is 1.112 Mm³/a [Burkes, S., Correspondence May 04, 2023]. This based on 7008 hours of plant operation per year at 740 tonnes per hour and includes an additional component for general site dust suppression. This estimate forms the base demand for the scenarios, with demand increased to 1.5 Mm³/a for to assess sensitivity.
- **Mean Annual Evaporation (MAE):** Annual pan evaporation is in the order of 3 200-3 400 mm [MET, 2002]. A conversion factor of 0.7 was applied for estimating the open water evaporation (Kambinda and Bittner, 2020b). Evaporation was also considered to occur as water seeps from the pit faces. This was done by calculating the pit wall surface area and determining the equivalent depth of seepage seeping from the walls. The evaporation potential on this seepage depth was estimated by considering that direct sunlight can only radiate onto a fraction of the wall throughout the day and the moving water reduces the potential to absorb the radiation required to induce evaporation. 25% of the wall is assumed to be exposed to sunlight at any point in the day, and the evaporation potential for the seeping water is assumed to be 15% of the open water evaporation potential.

- **Water Management in the Pit:** The size of the pumps inside the pit is yet to be confirmed. The water balance assumes that an appropriate pump capacity will be installed to fully dewater the inflows. Should there be a limit on the capacity to be installed, this will need to be incorporated into the water balance in the next phase of the design. The design of the pit sumps and the relocation of the sumps as the pits progresses will also need to be considered in the next phase of the design. The exact operating philosophy regarding the cloud west pit will also affect the abstraction of water from the pits as water can only be should only be sent to the cloud west pit if there is sufficient capacity to receive the water.
- **Monthly rainfall data:** A 22-year daily rainfall record [Namibian MET Office] was used in the analysis. A stochastic analysis was performed to produce a 14-year project cycle rainfall record for the average condition. The surface water conditions were linked to the groundwater supply inflow files, i.e. if average conditions were used for groundwater supply then average conditions were used for surface water flows. Where a drought was considered for the groundwater supply, the same drought conditions were applied to the surface water supply. This was done primarily because the pit inflow and borehole abstraction flows are fixed input files in the model.
- **Borehole Water and Pit Seepage Flow:** The groundwater investigation was performed concurrently with the water balance and results from the investigation were fed into the water balance for water supply. The scenarios run in this water balance were chosen to allow compatibility with the scenarios and output from the groundwater study.
- **External water supply:** Investigations into sourcing water from the Karibib Town Treatment plant and the Khan River Abstraction Study are not specifically assessed in this water balance. The results of those studies will feed into servicing potential deficits from the surface water and groundwater supply.
- **Pit Push Backs:** The excavation of each pit through each year of the project life cycle provides inputs to both the groundwater study and the surface water balance. The development of each pit will influence the volume of water that seeps into the pits, as well as change the available storage that needs to be considered in the water balance. Although the current assumption is that the pits will be fully dewatered (except for cloud west when it is used as a storage reservoir), a change in the operating philosophy may require temporary storage in each pit which is dependant on the storage available at each point in time. Refinement of the pit pushbacks will need to be considered in the next phase. Additionally, the pit push backs influence the direct rainfall volumes that fall into the pit. The pump capacities in the pits have yet to be defined as the design of the pit sumps has not been completed – it is assumed that the pump capacity will be designed to meet the requirements as set out in the water balance.
- **Diversion Structure:** The diversion structure, as defined in Section 4, is assumed to only become operational at the same time as the cloud west pit is to be considered for additional storage. This is currently set for year 4 of the project cycle. The development of the main pit in years 1 to 4 will need to consider the potential for runoff from the upstream catchment.

5.2.2 SCENARIOS

Table 5-1 provides a summary of the scenarios, as described below:

- **Scenario 1:** The base scenario consists of the full groundwater component being available for plant supply i.e. borehole abstraction provides the primary water supply with pit seepage being dewatered and used to make up deficits. The daily rainfall record is used to create the rainfall/runoff estimates of surface water supply. The climatic data is applied stochastically with average climatic conditions being reported. The diversion structure is applied in year 4 of

operation. The cloud west pit is used as an additional storage reservoir when the diversion structure is applied. The plant demand is 1.112 Mm³/a.

- **Scenario 2:** This scenario assesses the ability to supply water if borehole abstractions are ignored. The daily rainfall record is used to create the rainfall/runoff estimates of surface water supply. The climatic data is applied stochastically with average climatic conditions being reported. The diversion structure is applied in year 4 of operation. The cloud west pit is used as an additional storage reservoir when the diversion structure is applied. The plant demand is 1.112 Mm³/a.
- **Scenario 3:** This scenario assesses the impact of not using cloud west pit when supply is constrained. Borehole abstractions are ignored in this scenario. The daily rainfall record is used to create the rainfall/runoff estimates of surface water supply. The climatic data is applied stochastically with average climatic conditions being reported. The diversion structure is applied in year 4 of operation. The cloud west pit is not used as an additional storage reservoir when the diversion structure is applied. The plant demand is 1.112 Mm³/a.
- **Scenario 4:** This scenario assesses the impact of not using cloud west pit when supply is not constrained i.e. the full groundwater component is available for supply. The daily rainfall record is used to create the rainfall/runoff estimates of surface water supply. The climatic data is applied stochastically with average climatic conditions being reported. The diversion structure is applied in year 4 of operation. The cloud west pit is not used as an additional storage reservoir when the diversion structure is applied. The plant demand is 1.112 Mm³/a..
- **Scenario 5:** The base scenario consists of the full groundwater component being available for plant supply, however, a drought condition is applied. The drought condition is a forced 7 year drought over the first 7 years of mine operation, after which average climatic conditions apply. The groundwater inputs are adjusted for the drought based on results from the groundwater study. The daily rainfall record is used to create the rainfall/runoff estimates of surface water supply during the non-drought years. The climatic data is applied stochastically with average climatic conditions being reported. The diversion structure is applied in year 4 of operation. The cloud west pit is used as an additional storage reservoir when the diversion structure is applied. The plant demand is 1.112 Mm³/a.
- **Scenario 6:** This scenario shows the impact of increasing the plant demand while having the full groundwater component being available for plant supply i.e. borehole abstraction provides the primary water supply with pit seepage being dewatered and used to make up deficits. The daily rainfall record is used to create the rainfall/runoff estimates of surface water supply. The climatic data is applied stochastically with average climatic conditions being reported. The diversion structure is applied in year 4 of operation. The cloud west pit is used as an additional storage reservoir when the diversion structure is applied. The plant demand is 1.5 Mm³/a.
- **Scenario 7:** This scenario shows the impact of not using the cloud west pit for additional storage when there is full groundwater supply but also an increased plant demand. The daily rainfall record is used to create the rainfall/runoff estimates of surface water supply. The climatic data is applied stochastically with average climatic conditions being reported. The diversion structure is applied in year 4 of operation. The cloud west pit is not used as an additional storage reservoir when the diversion structure is applied. The plant demand is 1.5 Mm³/a.

Table 5-1: Water Balance Scenarios

Scenario	Pit Inflows	Borehole Supply	Cloud West Storage	Drought Conditions	Demand (Mm ³ /a)
1	x	x	x		1.112
2	x		x		1.112
3	x				1.112
4	x	x			1.112
5	x	x	x	x	1.112
6	x	x	x		1.500
7	x	x			1.500

5.2.3 RESULTS

5.2.3.1 SCENARIO 1

Figure 5-2 shows the stacked annual inflow volume for Scenario 1. It is evident that the majority of the water supply available to the plant comes from the groundwater component. The groundwater supply from boreholes reduces with time, while the volume available from dewatering the pit seepage increases (this is expected as the pits get deeper with time). Although the boreholes supply reduces with time, the overall groundwater supply increases due to the pit wall surface areas getting larger which allows greater seepage potential. The volume available from direct precipitation increases as the pits grow (larger catchment area as the pit footprint increases) while the catchment runoff component shows small variations.

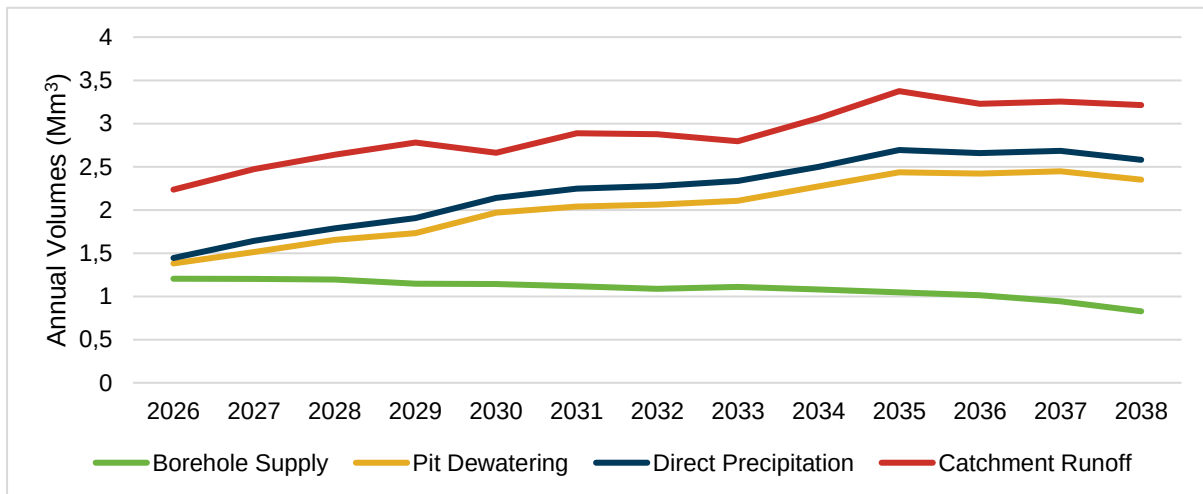


Figure 5-2: Annual Inflows for Scenario 1

Figure 5-3 shows that the base case Scenario 1 can meet the full plant demand through the full project cycle. Supply during the start up phase will be entirely supplied through borehole abstractions, while pit dewatering (seepage and direct precipitation from the pits and any catchment runoff) will make up the additional demand as the borehole supply reduces with time.

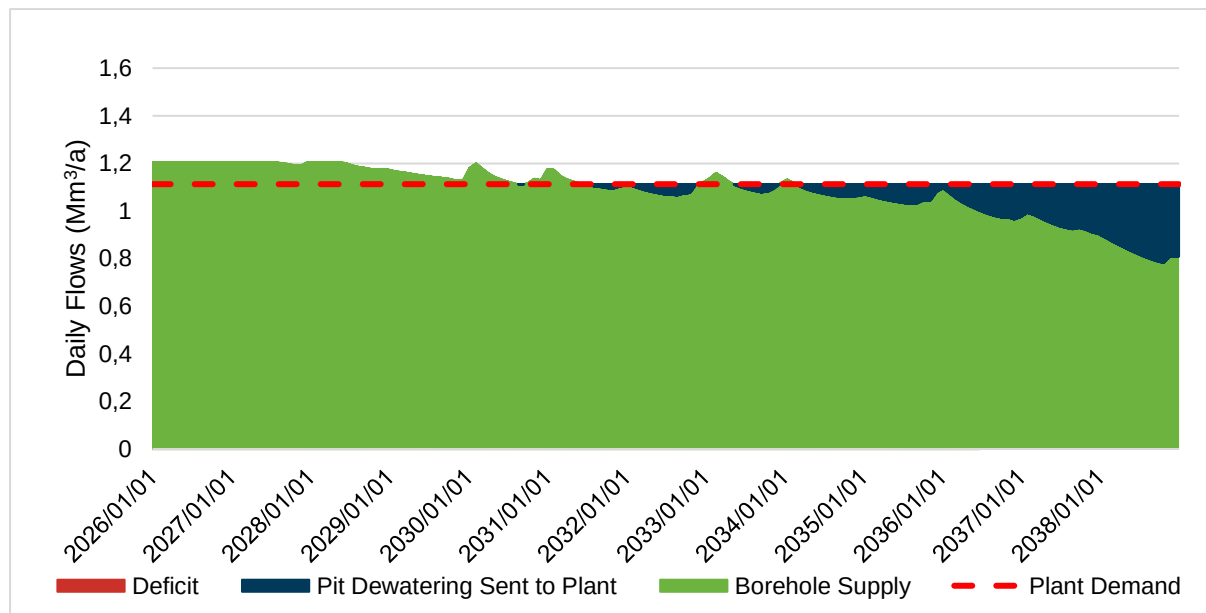


Figure 5-3: Plant Supply and Deficit for Scenario 1

Figure 5-4 indicates the reliability of supply is certain for the selected design criteria.

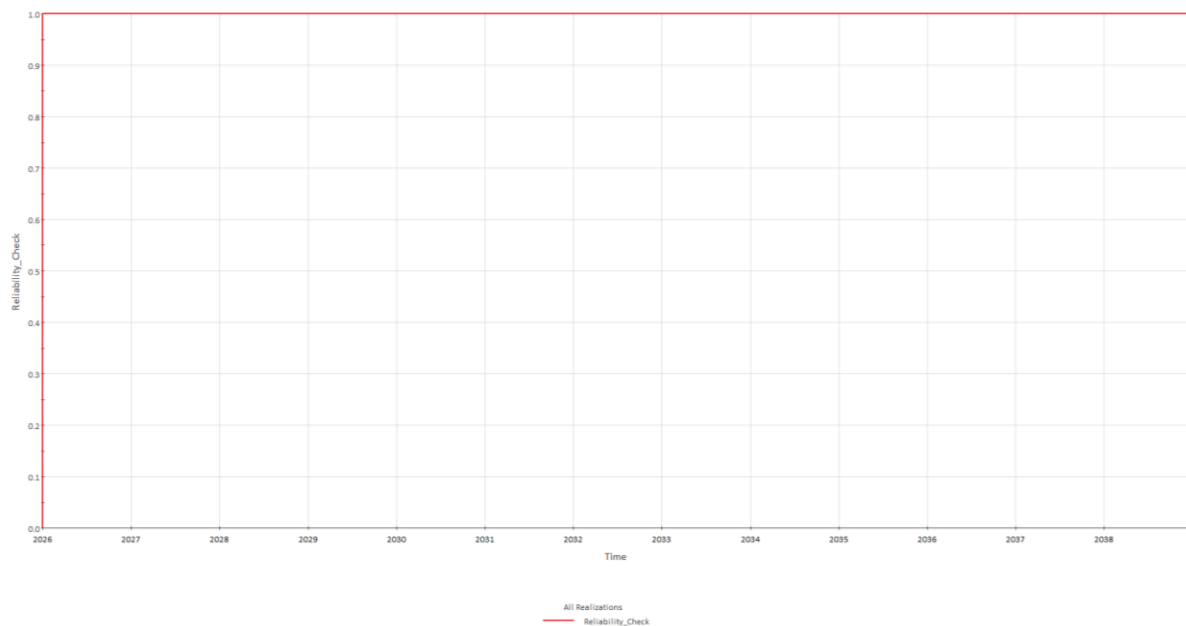


Figure 5-4: Progressive Reliability of Supply for Scenario 1

5.2.3.2 SCENARIO 2

Figure 5-5 shows the stacked annual inflow volume for Scenario 2. Without the borehole supply, the total water availability during the project start up is low. The volume available from direct precipitation increases as the pits grow (larger catchment area as the pit footprint increases) while the catchment runoff component shows small variations.

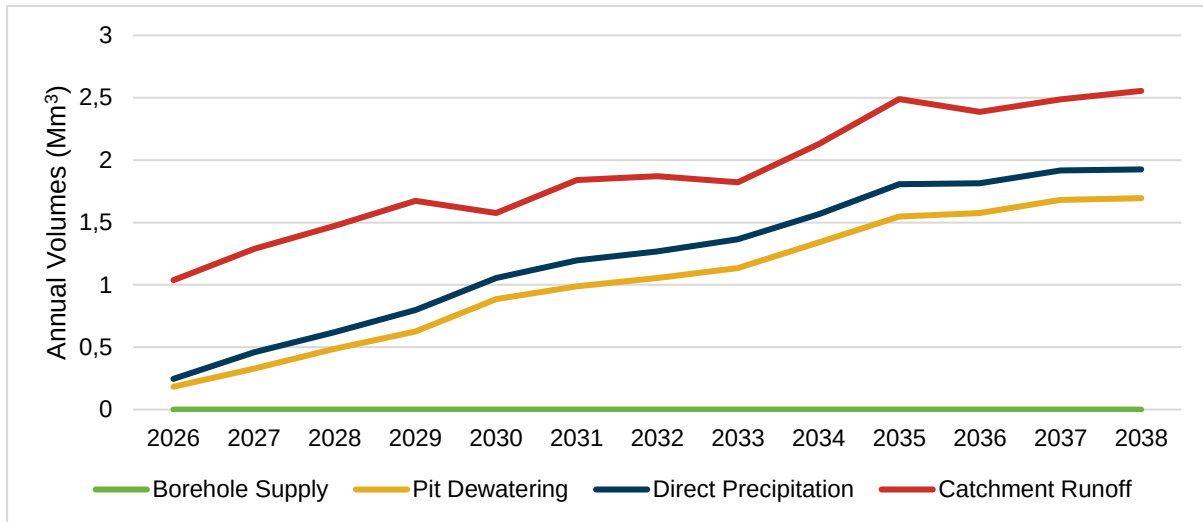


Figure 5-5: Annual Inflows for Scenario 2

Figure 5-6 shows that without borehole supply, the start up phase will have a significant deficit. Although Figure 5-5 shows the annual catchment runoff contributes to a total annual inflow being approximately equal to the annual plant demand, the rainfall/runoff events are not consistent and the efficiency in using this source is low while there is no diversion structure. Once the diversion structure is built after 4 and the cloud west pit is utilized for storage, the deficits reduce significantly.

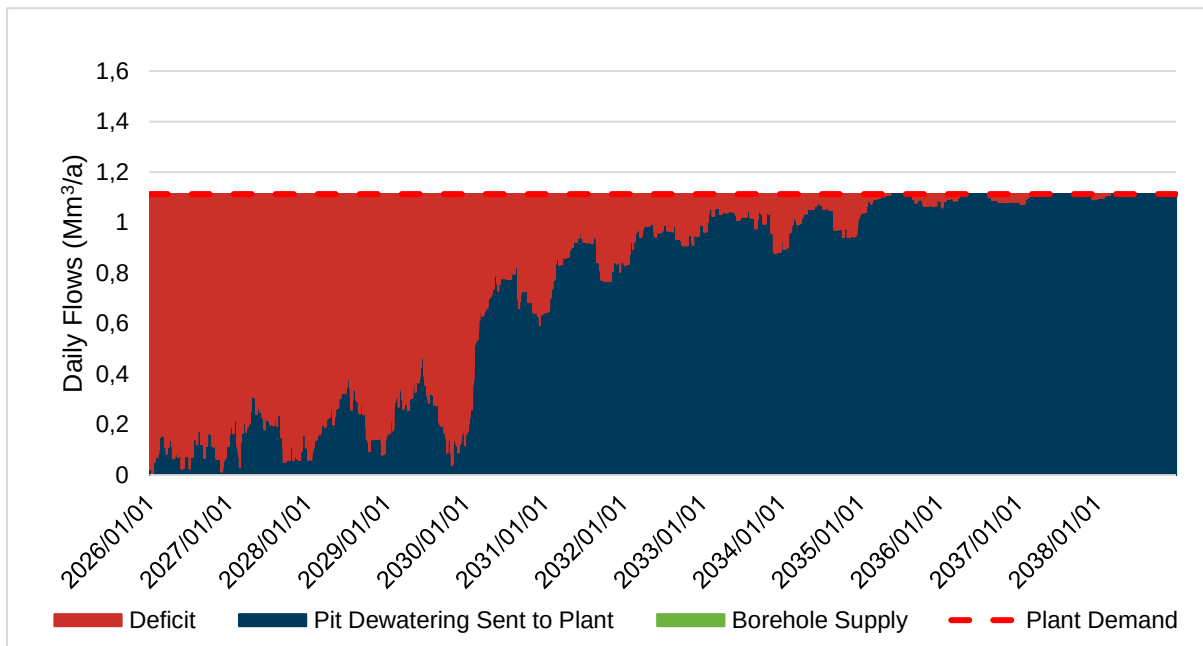


Figure 5-6: Plant Supply and Deficit for Scenario 2

Figure 5-7 shows that without borehole supply, the reliability of supply is significantly reduced (approximately 10%) during project start up, but increases after cloud west pit is used for storage.

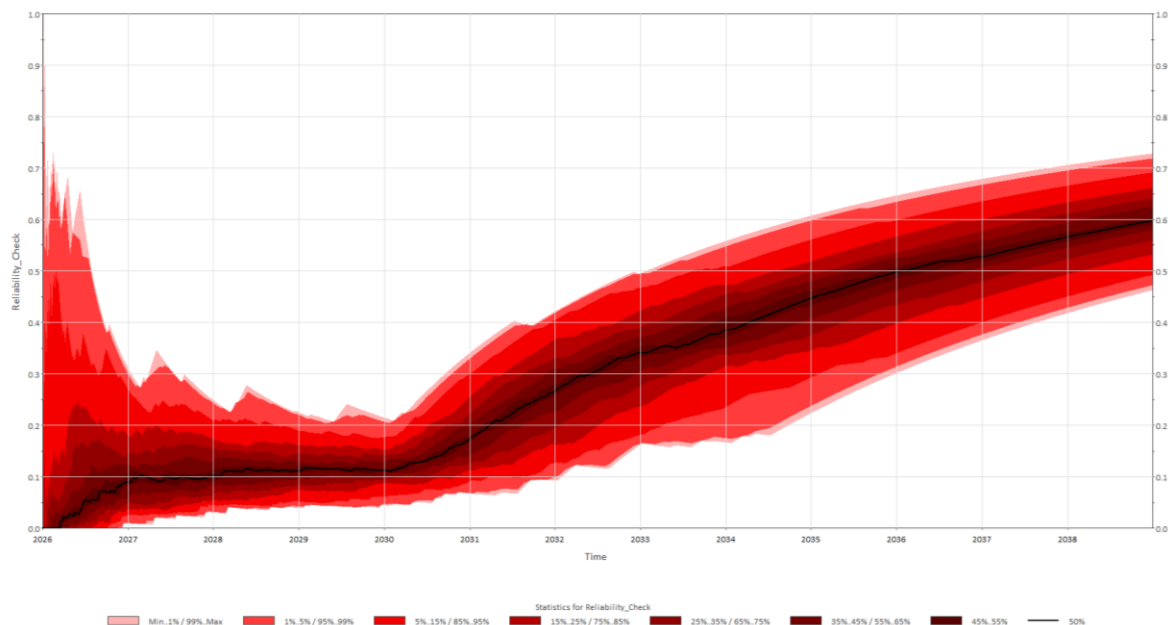


Figure 5-7: Progressive Reliability of Supply for Scenario 2

5.2.3.3 SCENARIO 3

Figure 5-8 shows the water supply for Scenario 3 to be the same as that for Scenario 2.

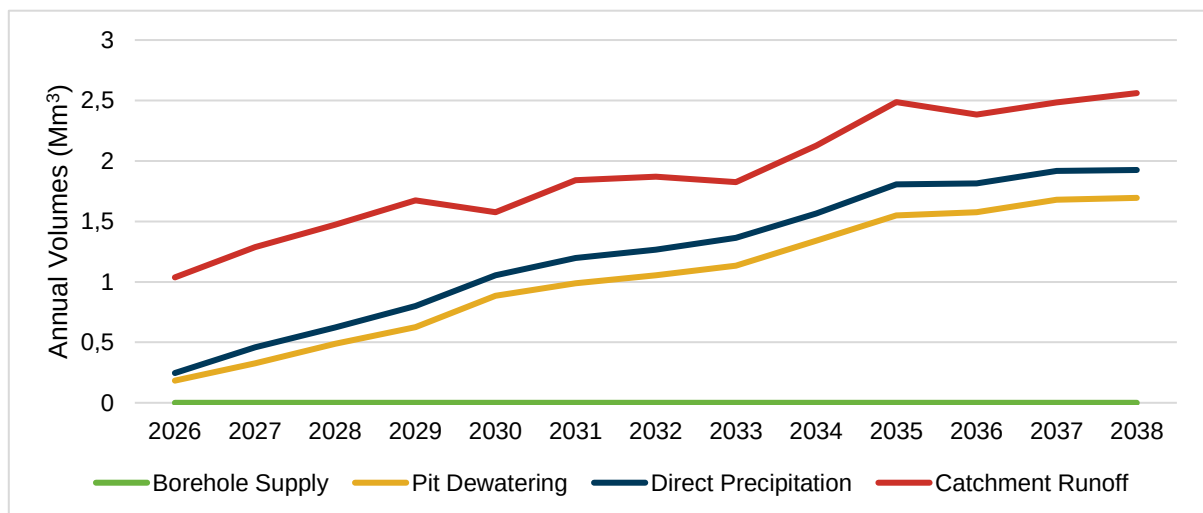


Figure 5-8: Annual Inflows for Scenario 3

Figure 5-9 shows the contrast in supply compared to Scenario 2 that if there is no borehole supply and cloud west pit is not used for additional storage. Supply is only assured during the wet seasons, compared to Scenario 2 where the cloud west pit created reserves for use in the dry season.

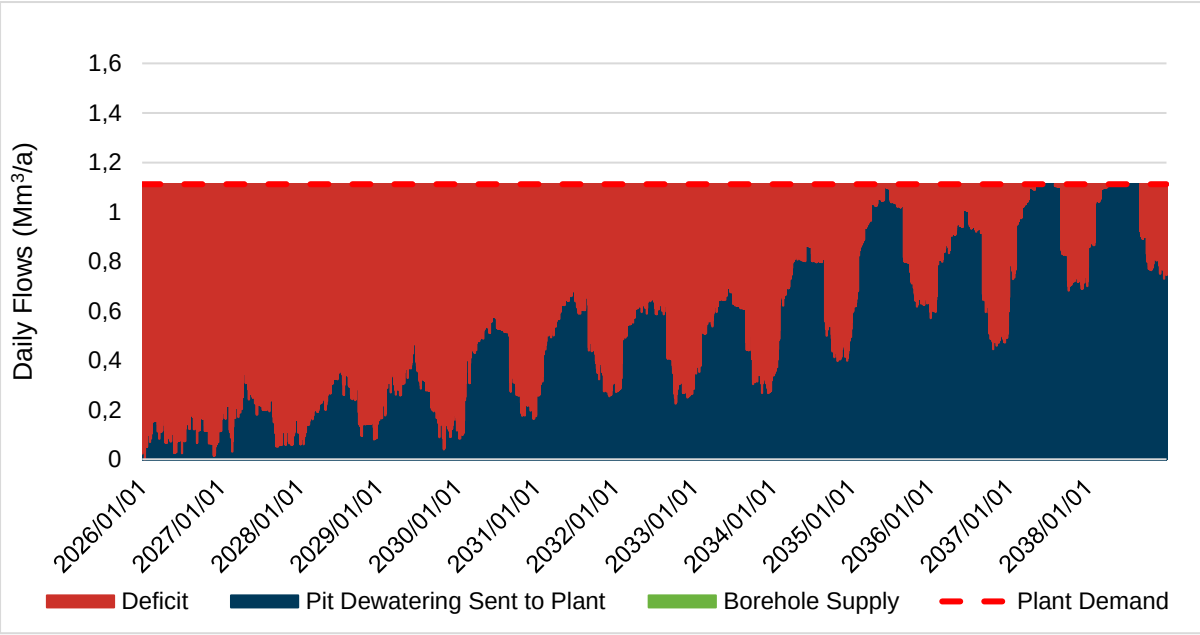


Figure 5-9: Plant Supply and Deficit for Scenario 3

Figure 5-10 shows the reliability of supply does not increase beyond 15% if cloud west pit is not utilized.

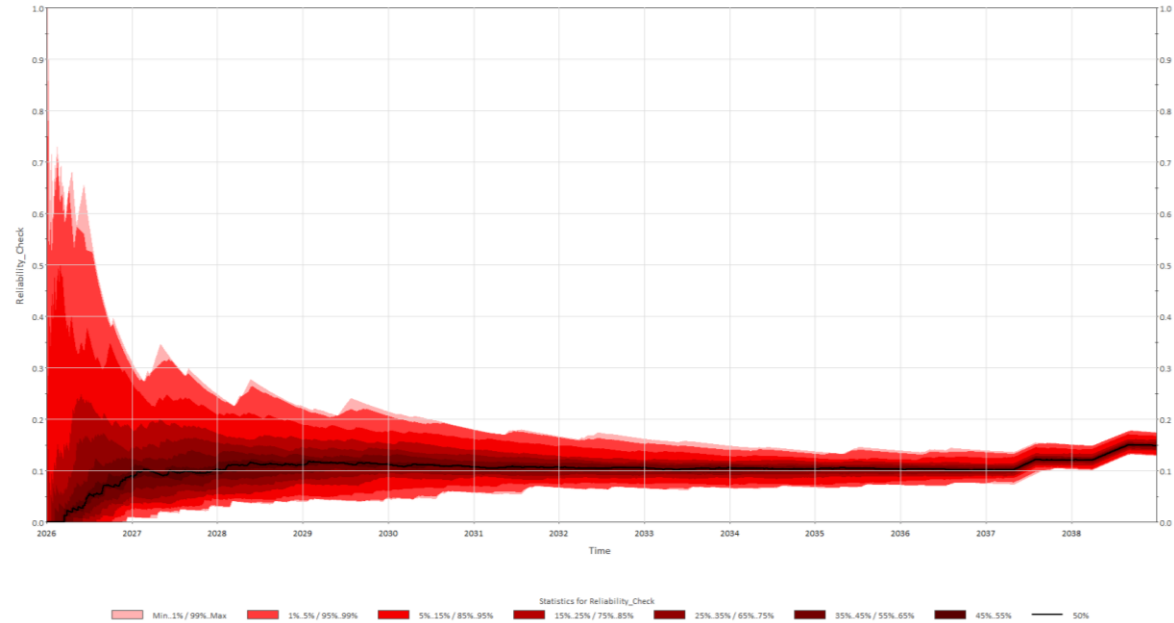


Figure 5-10: Progressive Reliability of Supply for Scenario 3

5.2.3.4 SCENARIO 4

Figure 5-11 shows the water supply for Scenario 4 to be the same as that for Scenario 1.

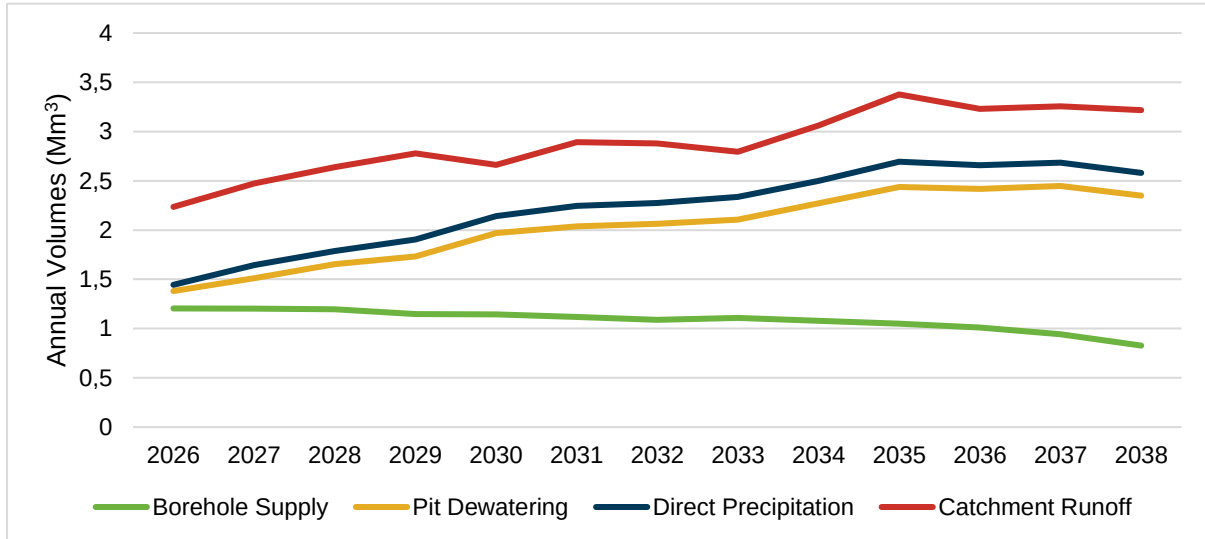


Figure 5-11: Annual Inflows for Scenario 4

Figure 5-12 shows that even if cloud west pit is not utilized, the borehole supply is sufficient to meet the plant demand with pit dewatering making up the shortfall in the latter years of the project.

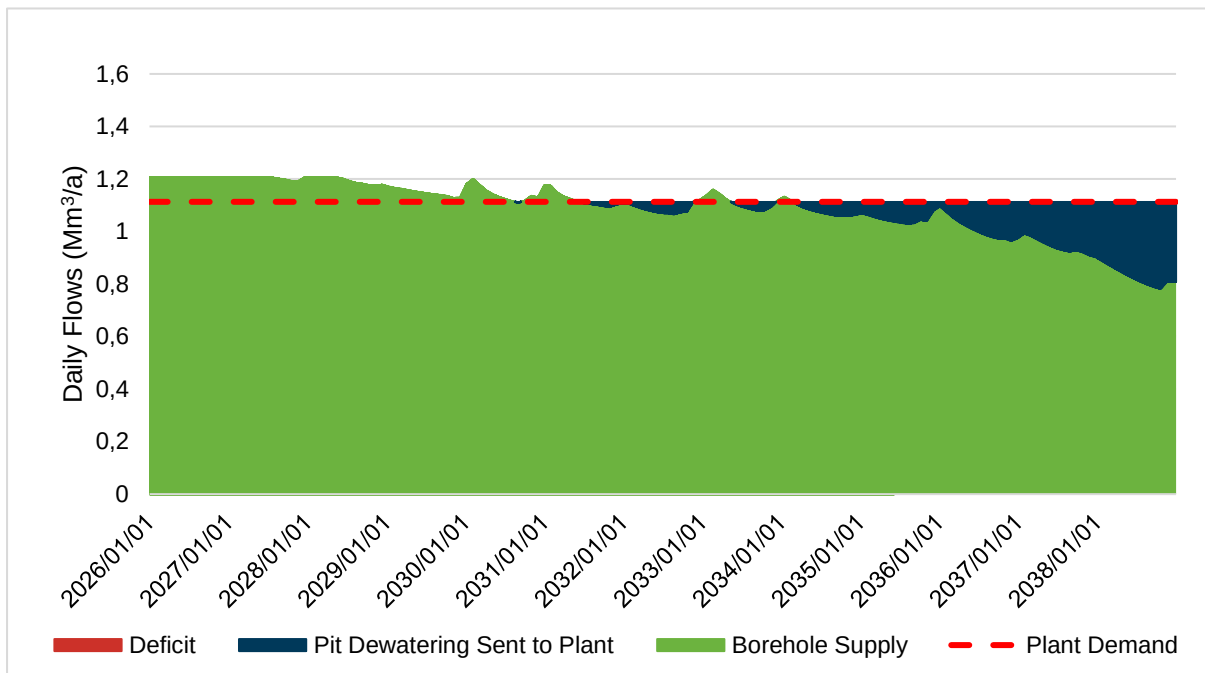


Figure 5-12: Plant Supply and Deficit for Scenario 4

Figure 5-13 shows the reliability of supply is assured where there is borehole supply for the selected plant demand.

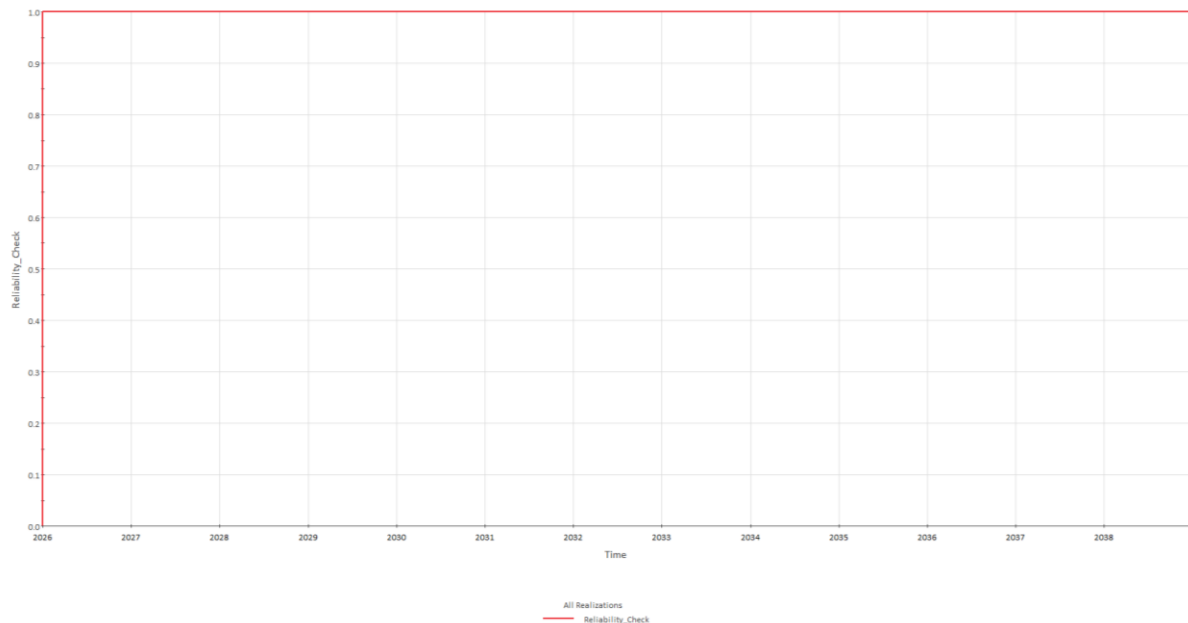


Figure 5-13: Progressive Reliability of Supply for Scenario 4

5.2.3.5 SCENARIO 5

Figure 5-14 shows the stacked annual inflow volume for Scenario 5. The majority of the water supply available to the plant comes from the groundwater component. The groundwater trend follows the same trend as described in Scenario 1. Due to the forced drought, there is no direct precipitation or catchment runoff component in the first 7 years of the simulation, after which the trends are similar to that of the previous scenarios.

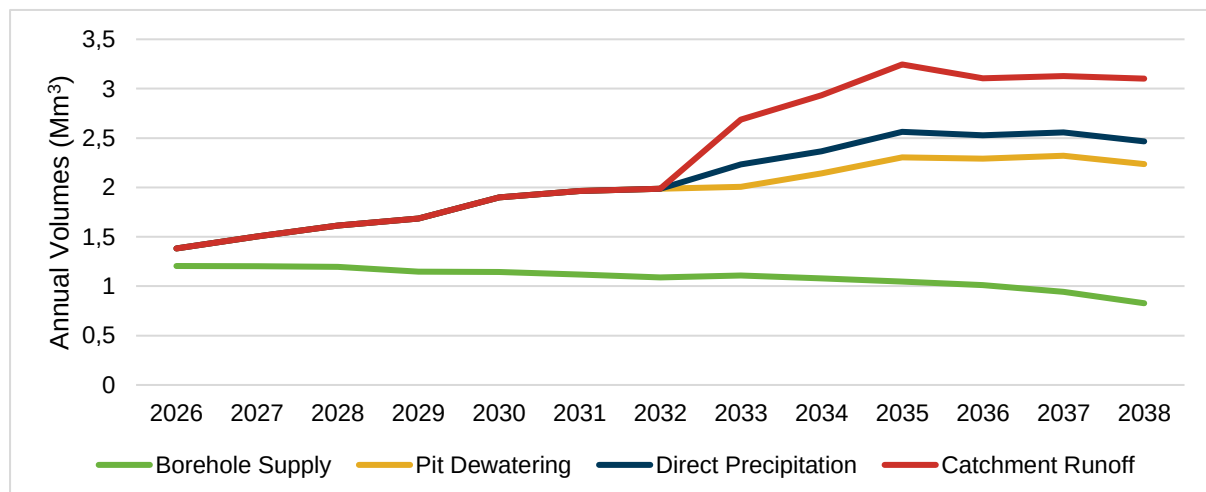


Figure 5-14: Annual Inflows for Scenario 5

Figure 5-15 shows that although there is a drought condition, the borehole supply and pit dewatering is still able to meet the plant demand.

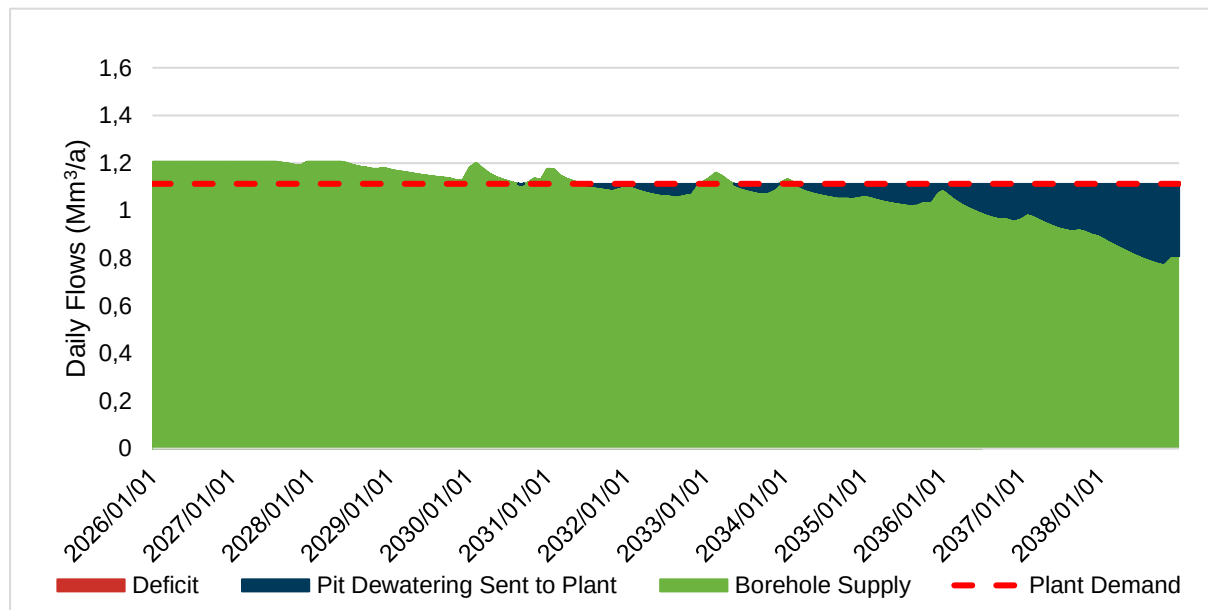


Figure 5-15: Plant Supply and Deficit for Scenario 5

Figure 5-16 shows that for the selected plant demand, drought conditions should have no bearing on reliability of supply from boreholes and pit dewatering.

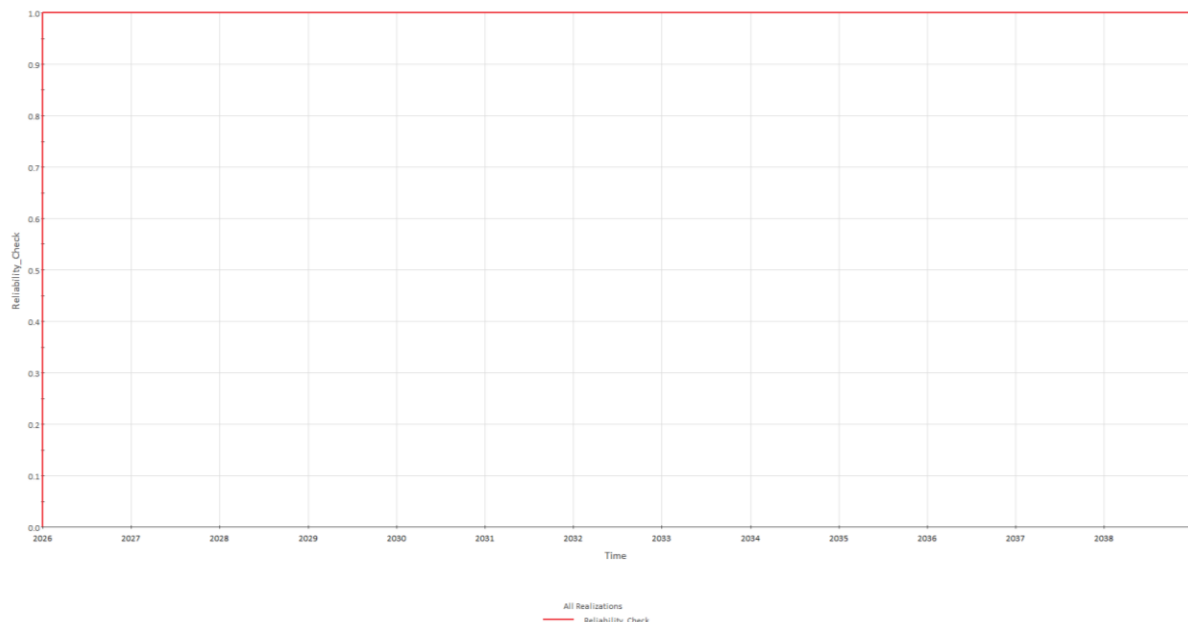


Figure 5-16: Progressive Reliability of Supply for Scenario 5

5.2.3.6 SCENARIO 6

Figure 5-17 shows the water supply for Scenario 6 to be the same as that for Scenario 1.

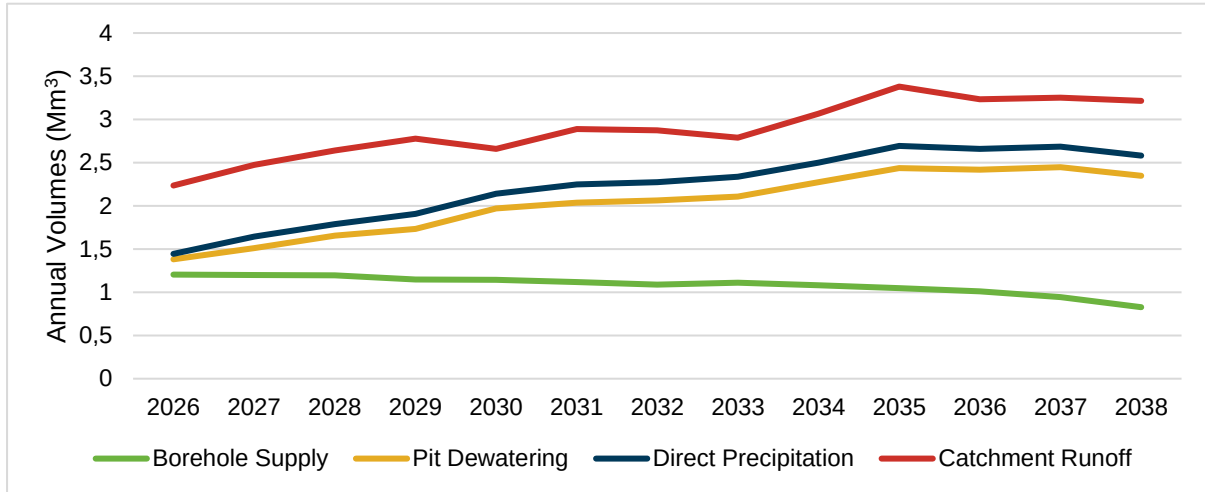


Figure 5-17: Annual Inflows for Scenario 6

Figure 5-18 shows that if the plant demand were to increase, the plant would experience deficits in the start-up phase. The use of cloud west pit for storage after year 4 provides contingency in the dry seasons which reduces the deficits.

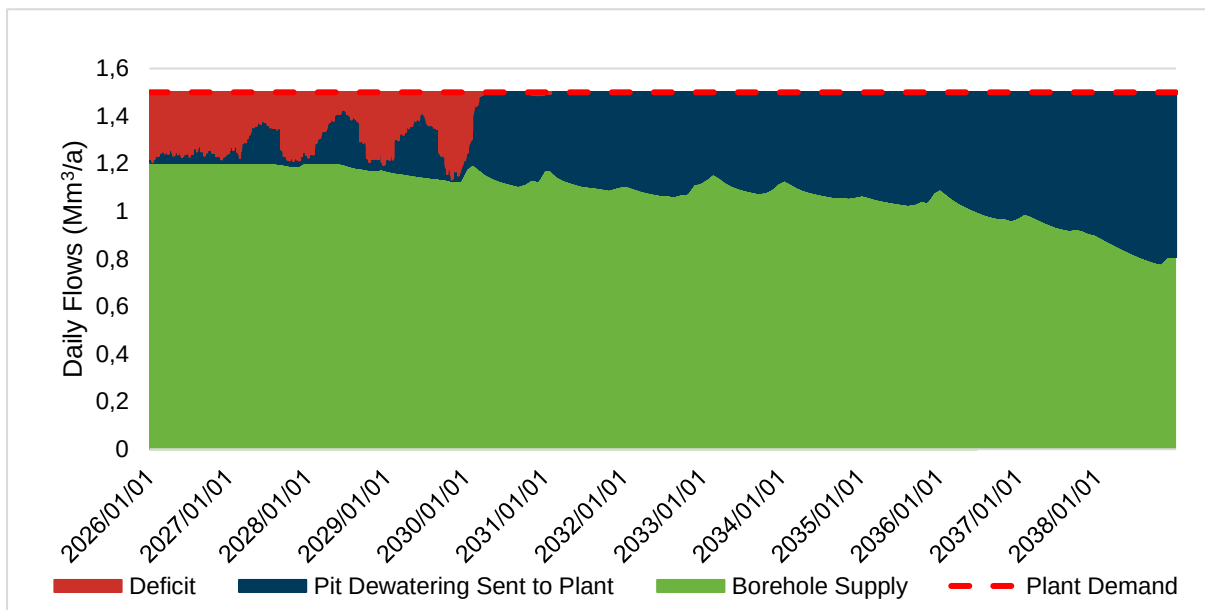


Figure 5-18: Plant Supply and Deficit for Scenario 6

Figure 5-19 shows that for an increased plant demand project life cycle reliability of supply would be below 15% until the cloud west pit starts being used for storage, after which the reliability increases due to the elimination of deficits.

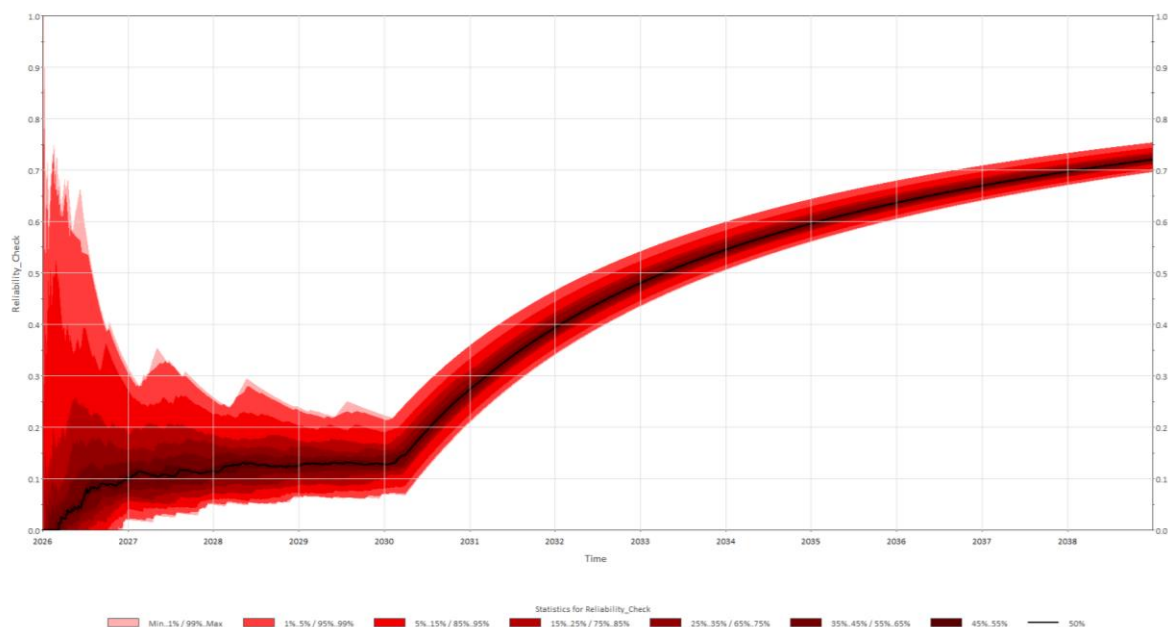


Figure 5-19: Progressive Reliability of Supply for Scenario 6

5.2.3.7 SCENARIO 7

Figure 5-20 shows the water supply for Scenario 7 to be the same as that for Scenario 1.

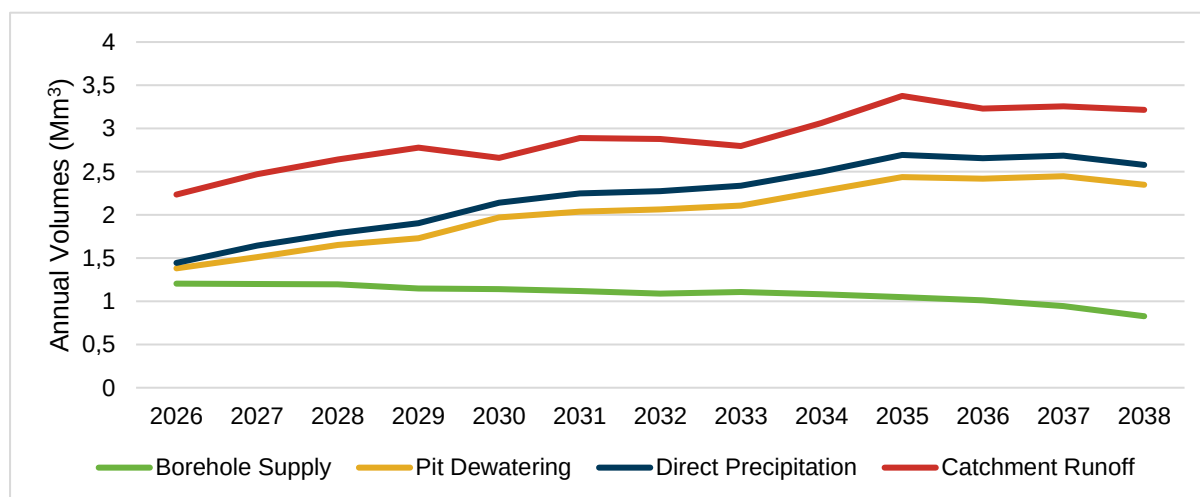


Figure 5-20: Annual Inflows for Scenario 7

Figure 5-21 shows that, compared to Scenario 6, not using the cloud west pit for storage would result in seasonal deficits in the latter years of the project.

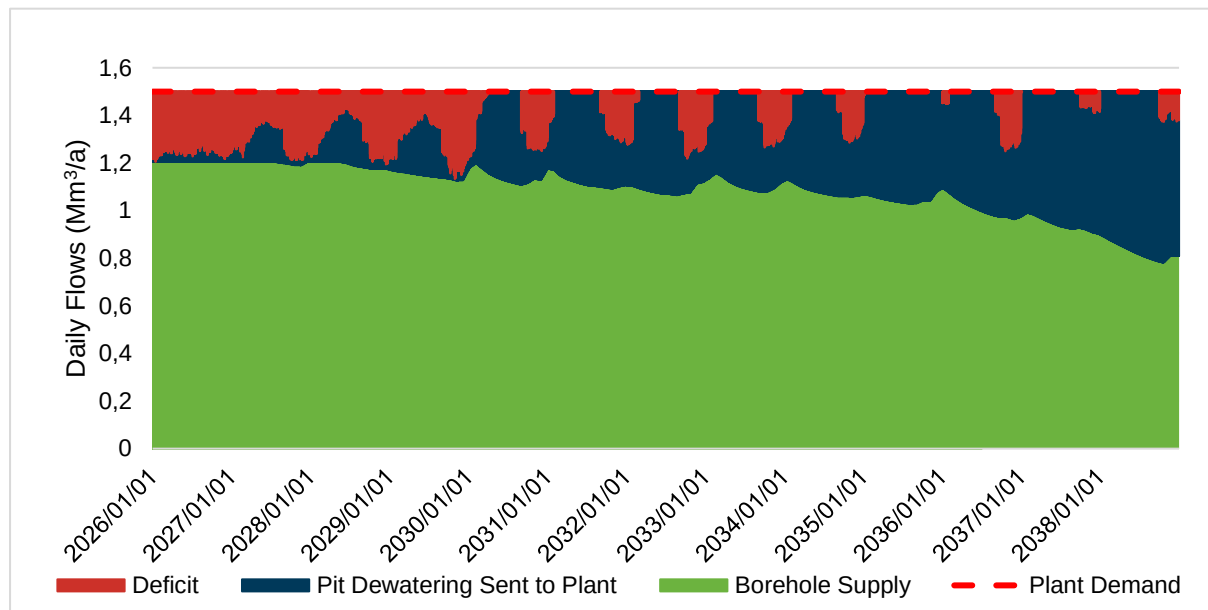


Figure 5-21: Plant Supply and Deficit for Scenario 7

Figure 5-22 shows that although the greater volume of water from pit dewatering increases the reliability of supply, the reliability is not increased as much as in Scenario 6.

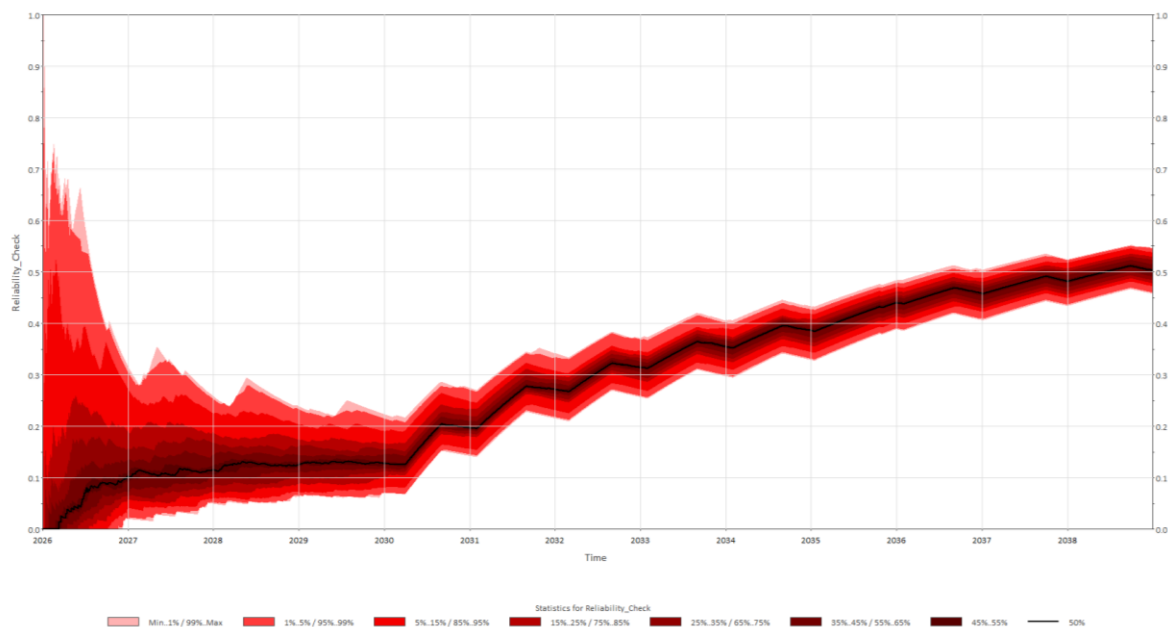


Figure 5-22: Progressive Reliability of Supply for Scenario 7

5.2.4 WATER BALANCE FINDINGS

Figure 5-23 shows a comparison of the annual deficits for each scenario. The following conclusions can be drawn from the results and the comparison shown:

- Where the plant demand is 1.112 Mm³/a, deficits are not expected if the borehole supply can be assured (refer Scenarios 1, 4, and 5). Where borehole supply cannot be assured (refer Scenarios 2 and 3), the annual deficit during the project startup is as much as 0.975 Mm³/a and reduces as the pit inflows increase with time;
- If the demand is increased, the potential for deficits is increased (refer Scenarios 6 and 7). This is particularly sensitive during the project startup until such time as the pits are developed to a depth that allows for increased pit seepage and subsequent dewatering;
- For a plant demand of 1.112 Mm³/a, drought conditions have little impact on plant supply, as long as borehole supply is consistent with the estimates used in this model (refer Scenario 4);
- The use of cloud west pit as an additional storage reservoir has a significant impact on reliability of supply where there is potential for deficits (through reduced borehole supply or increased demand refer Scenarios 2 and 6 compared to 3 and 7 for cloud west storage vs no storage respectively).
- The maximum pit dewatering rate of abstraction is estimated to be 292 m³/hr, from the main pit. The maximum dewatering rates for the cloud, cloud west, and west pits are 65 m³/hr, 127 m³/hr, and 35 m³/hr, respectively.

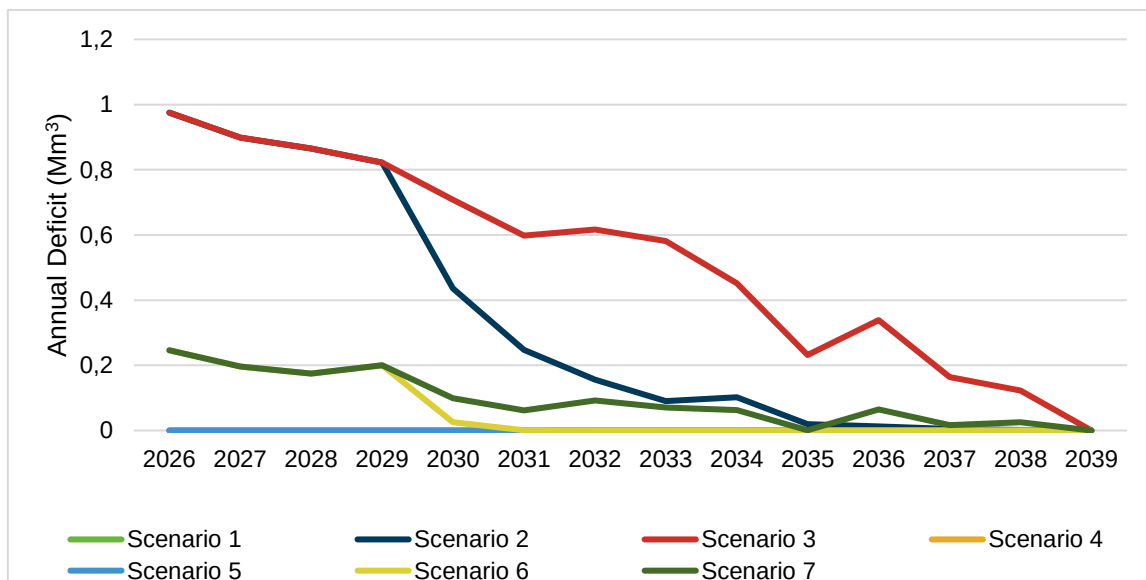


Figure 5-23: Comparison of Plant Supply Deficits for Each Scenario

6 RISKS

6.1 SURFACE WATER

- The current climate change models indicate that the climate will become more variable, tending toward longer more severe drought periods and shorter more severe flood events, which could result in changes to the flood peaks. The selection of the rubble masonry concrete flood mitigation structure would mitigate increased flood peaks to some degree as it can be designed to overtop with less risk than the concrete faced rockfill dam option.
- Accurate estimates of direct catchment runoff from precipitation events typically require measurements from multiple weather stations and flow gauges along the catchment confluences. As these measuring stations are not available for this particular catchment, runoff is estimated by using rainfall runoff estimation methods. A runoff coefficient estimation study was carried out to validate the coefficients used in the flood peak estimates, however, the quality of the survey needs to be refined to further validate the estimates.
- Part of the main pit is situated directly in the flow path of the Okawayo River, at the confluence of a 92 km² catchment area. The pit would be subject to flooding under extreme event conditions if no flood mitigation measures are implemented. The pit development schedule would dictate the timeline for the implementation of flood mitigation measures, however, constructing the diversion structure early in the project cycle would aid the groundwater recharge by effectively creating a sub-surface dam once the diversion reservoir fills with sediment.
- The annual exceedance probability to design flood mitigation infrastructure is not clearly defined by local regulations. The diversion structure is not intended for use as a permanent water supply but rather to route flood flows to the Cloud West pit. Whereas diverting all AEP flood flows is possible, this would require extensive and costly river diversion infrastructure.
- The Client requested that alternative river diversion solutions be developed that offered a trade off between diverting smaller return period floods at a reduced implementation cost, but the mine would then accept the higher risk of pit flooding in the case of larger flood events.
- A small dam constructed using RMC is recommended for the diversion wall, with small size class (8.8 m) and a low hazard rating classification (SANCOLD). The spillway is sized for a 50-year AEP flood capacity but larger floods can be passed directly over the wall. From a business perspective, there is risk to the operations if lower AEP flood events were to occur. Early flood warning systems would need to be implemented by the mine for evacuating plant and personnel from the pit before the dam overtops and operations in the main pit would be suspended until the pit is dewatered. The dam wall must therefore be designed to allow for safe overtopping during extreme floods without compromising the integrity of the structure and the spillway channel must be fully operational during all flood flows. Inspection of the spillway channel is possible when the flood subsides.
- As the main pit is situated in the direct flow path of the Okawayo River, the environmental impact of diverting flow to the cloud west pit will need to be assessed further. The incremental impact of the small reservoir pool upstream of the dam is no more severe than the case of intercepting the natural river it in the Central pit without diversion measures being instated.
- The expectation that the diversion dam reservoir would silt up has the benefit of creating a sub-surface recharge dam, but the consequence is that there is little to no attenuation achieved during flood events (i.e. the reduction in storage capacity by silt build-up reduces how much water the dam

can store before flowing over the spillway). This results in the spillway being sized for an unrouted flood peak, although these peaks are temporary and not expected to be sustained during flooding.

- The excavation of the spillway discharge channel is expected to be predominantly in rock. The requirement to line the channel will need to be assessed during the construction and/or the operation of the channel. This will be based on the extent erosion/scouring that takes place. The side slopes used in the estimation of the excavation quantities are also based on the assumption that the channel will be excavated in competent material – should the material require flatter side slopes, the excavation quantities will change.
- The surface water management referred to herein is specific to the management of the upstream catchment. Management of runoff emanating from the waste rock dumps and the TSF is looked at under the scope of those designs.

6.2 WATER BALANCE AND WATER SUPPLY

- The key component contributing to water supply has been identified as borehole supply. The variability/reliability of this supply is addressed in the groundwater study and has not been addressed in the surface water study, specifically the water balance. The impact of not having borehole supply has been shown to be a significant risk to the overall water security for the site.
- The seasonal variability in groundwater was not considered in the PFS study. This was addressed in this phase of the study by including drought conditions for both the groundwater and the surface water components of the water balance.
- The full evaporation potential on the pit inflows seeping through the pit walls was not fully considered in the previous phase of the study. This has been addressed in this phase of the study. The significance of the borehole supply compared to the supply from pit dewatering reduces the risk of under/overestimating this component of evaporation, but a reduction in borehole supply may require this estimate to be redefined.
- The borehole supply has been shown to decrease over time, however, as the pits are excavated to greater depths, the pit inflows increase which offsets the risk of the reduced borehole supply.
- Reliability of supply for the external supply options has not been considered in this study. The supply potential and reliability from these separate studies is intended to assess the potential to meet plant demand in the case of deficits from site supply.
- A detailed sensitivity analysis was not considered in the previous phase. The sensitivity of site supply has been addressed in this study based on the scenarios selected.
- The high-level water balance from the PFS was based on a monthly time step. This has been refined in this phase of the study by running the model on a daily time step to allow for better refinement of the daily response to water flow/transfers.
- The current operating philosophy of the water balance relies on all pit inflow being dewatered and water being stored in the Cloud West pit. The practicality of achieving full dewatering has not yet been considered. The pit drainage management will need to be designed in the next phase and incorporated into the water balance operating philosophy.

7 OPPORTUNITIES

7.1 SURFACE WATER

- Refinement of the site hydrology/runoff coefficients can be done with updated terrain surveys.
- General flood protection berms around each pit should be determined and included in the detailed stormwater management plan.
- The detailed designs of the channels could consider suitable erosion protection measures including channel stepping, internal check weirs/velocity reduction bunds, and block stone or riprap protection.
- Detailed calculation of design storm (Peak flows) for sediment ponds, channels, catchment, and silt traps must be included in the detailed stormwater management plan design.
- A stream diversion study must be undertaken, with a detailed environmental impact assessment stemming from detailed designs.

7.2 WATER BALANCE

- Refining the operating philosophy of the water balance, specifically relating to pit dewatering capacity, should be done once the final pit push backs are determined. The timing of when the cloud west pit starts being used for additional storage could also be considered.
- The water balance currently prioritizes borehole supply for meeting the plant demand. This could be reviewed and used as the make-up requirement for shortfalls from pit dewatering which would improve the potential reliable yield from the boreholes during the latter years of operation. This can only be confirmed once the pit dewatering sump designs are done to then feed into the overall operating philosophy.
- Details regarding the TSF should be refined and the surface water collection from this facility should be included in the overall water balance.
- A yield study for the Khan River augmentation options should be conducted to determine the optimal size to make the preferred dam option. Once this dam size has been determined, the dam can be included in the overall site water balance.
- Water quality tests should be undertaken in the detailed design to confirm the suitability of using the cloud west pit as a storage reservoir.

8 CONCLUSION

The following conclusions can be noted:

Surface Water

- Part of the main pit is situated directly in the flow path of the Okawayo River. The pit would be subject to extensive flooding if no flood mitigation measures are implemented. The pit development schedule would dictate the timeline for the implementation of flood mitigation measures which could allow for a phased implementation and positively impact the capital outlay requirements.
- A diversion wall with a spillway discharge channel was considered for flood mitigation. An RMC wall being the preferred wall type to take forward to the detailed design. The material costs for the wall and the channel are estimated..

Water Balance

- The site is a surface water deficit site and requires additional sources to supplement the plant's water demand.
- The GoldSim water balance has been modelled on a daily timestep and considers groundwater supply from pit inflows and boreholes, the use of the cloud west pit for storage, and the diversion structure routing runoff to the cloud west pit.
- The results of the water balance indicate that the groundwater supply will likely meet the plants annual water demand of 1.1 Mm³.
- The reliability of supply is sensitive to the borehole supply and the demand remaining at the current estimate. The sensitivity to drought conditions is not high if borehole supply can be assured.
- The use of the cloud west pit as an additional storage basin will be beneficial if groundwater supply is compromised or if the plant demand increases.

9 RECOMMENDATIONS

It is recommended to advance the following scope of work to refine the water supply and model assessment to a detailed design level. A detailed Scope of Work (SOW) and associated costs will be provided to the Client for the next phase.

Surface Water

- Complete a detailed topographic survey (1 m contours accuracy) of the Okawayo catchment area, and update hydrological and hydraulic models to assess the potential runoff and peak runoff for various flood events. This should also include the B1 highway crossing and location of roadway water crossings along the Okawayo catchment.
- Integrate and detail the overall site water management to include mining infrastructures/dewatering, downstream infrastructures such as roads, utilities, and final landscape of waste rock dumps. Detailed design should include:
 - Accurate sizing of channels, flood protection berms and storage containment facilities;
 - Detailed stormwater design plan around the mine pit infrastructures development.
 - Overflow structures for the Cloud West Pit
- Complete detailed design of the Okawayo flood mitigation measures with detailed peak flood assessment upstream and downstream of the structures, transient analysis and operational and environmental impact of various flood conditions on the downstream environment. This should consider a stepped development of the pit and details of flood protection berms, pit water management landscapes (pit lake), other site water management structures and dewatering capacity.
- Install additional site rainfall stations and monitor the site flood station identified in this study.
- Complete a sediment transport model and impact assessment for various conditions as part of the detailed design.
- Consider the mine development activities (phased development) and detailed infrastructures in a flood risk assessment, operational and emergency procedure requirement to mitigate risk.
- Complete, as part of the detailed design, additional geotechnical investigation to detail the founding conditions of the diversion structure and spillway discharge channel, as well as required suitable erosion protection measures.
- It is recommended that a pit water management system be put in place during the detailed design of the stormwater management plan.
- Impacts of the diversion structure must be considered in a detailed environmental impact assessment stemming from detailed designs.
- Consider the mine development activities (phased development) and detailed infrastructures in a flood risk assessment, operational and emergency procedure requirement to mitigate risk

Water Balance

- The design/management of the pit dewatering, which incorporates the pit development, will need to be incorporated into the water balance design philosophy.
- The timing of when the diversion structure and the cloud west pit is used for storage could be refined in the detailed design.

- Details regarding the TSF should be refined and the surface water collection from this facility should be included in the overall water balance.
- The results of the study for the Khan River augmentation options should be incorporated into an overall site water supply report.
- The variability of the groundwater supply could be refined in the detailed design, with the impact on reliable yield assessed based on using the boreholes for secondary supply rather than primary supply, as is the current case. As the pit dewatering requirement increases with time, this could be used as primary supply and borehole abstractions could be used to supplement the plant supply. This may have a positive impact on the available yield from the boreholes in the latter years of the project.

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11 CERTIFICATION

This report was prepared and reviewed by the undersigned.

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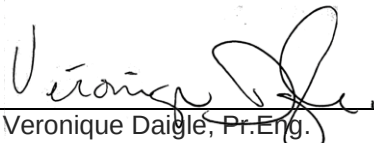
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