Frieda River Limited

Sepik Development Project

Environmental Impact Statement

Appendix 6a – Site-wide Water Balance

SDP-6-G-00-01-T-003-015

A TETRA TECH COMPANY

Frieda River Project
Site-wide Water Balance

Report Prepared for

Frieda River Limited

Report Prepared by

SRK Consulting (Australasia) Pty Ltd

PNA009

August 2018

Frieda River Project

Site-wide Water Balance

Frieda River Limited

PO Box 2297, Fortitude Valley Business Centre
Brisbane QLD 4006

SRK Consulting (Australasia) Pty Ltd

Level 1, 10 Richardson Street
West Perth WA 6005

e-mail: perth@srk.com.au
website: asia-pacific.srk.com

Tel: +61 8 9288 2000
Fax: +61 8 9288 2001

SRK Project Number PNA009

August 2018

Compiled by

Brian Luinstra
Principal Hydrogeologist

Email: bluinstra@srk.com.au

Peer Reviewed by

John Chapman
Principal Consultant

Author:

Brian Luinstra.

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List of Abbreviations

Abbreviation	Meaning
ANZECC	Australian and New Zealand Water Quality Guidelines for Fresh and Marine Waters
AP	assessment point
AWBM -	Australian Water Balance Model
AWS	automatic weather station
ENSO	El Niño Southern Oscillation
ET	evapotranspiration
FRHEP	Frieda River Hydroelectric Project
FRL	Frieda River Limited
HIT	Horse-Ivaal-Trukai
HITEK	Horse-Ivaal-Trukai-Ekwai-Koki
IFC	International Finance Corporation
ISF	Integrated Storage Facility
FRCGP	Frieda River Copper-Gold Project
LOM	life of mine
L/s	litres per second
MAP	mean annual precipitation
masl	metres above sea level
mm/yr	millimetres per year
m RL	metres relative level
m/s	metres per second
m^3/s	cubic metres per second
m^3/day	cubic metres per day
ML/day	megalitres per day (million litres per day)
Mtpa	million tonnes per annum
PMP	probable maximum precipitation
SIP	Sepik Infrastructure Project
SPGP	Sepik Power Grid Project
SRK	SRK Consulting (Australasia) Pty Ltd
TSS	total suspended solids
WHO	World Health Organization
W/m^2	watts per square metre
WTP	water treatment plant

1 Introduction and Scope of Report

Frieda River Limited (FRL) is assessing the feasibility of the Sepik Development Project (the Project) in northwest Papua New Guinea (PNG). The Project is underpinned by the Frieda River Copper-Gold Project (FRCGP) and supported by three interdependent projects which provide key infrastructure including the Frieda River Hydroelectric Project (FRHEP), the Sepik Power Grid Project and the Sepik Infrastructure Project, all located in mountainous terrain in Sandaun and East Sepik provinces of PNG.

FRL proposes to also develop the Frieda River Hydroelectric Project and utilise the reservoir basin for tailings and mine waste disposal. This report provides a water balance for the FRGCP and Frieda River Integrated Storage Facility (ISF).

1.1 Study objectives

A site-wide water balance was constructed by SRK with the objective of providing information the sizing of and pumping from mine sumps within the open-pits, release of water from the ISF and effects on flows downstream of the facility. Additionally, the water balance model provides inputs for a load balance model, to investigate potential impacts of Project development on water quality. To inform the load balance, flows at a range of Assessment Point (AP) locations in creeks and rivers both upstream and downstream of the ISF, are required.

The water balance model provides predictions for average, wet and dry conditions for the pre-mining, life of mine (LOM), and post closure periods. The 90^{th} and 10^{th} percentile flow conditions from the stochastic water balance were selected to represent wet and dry conditions respectively.

Flow data from the model were also used to estimate pre-embankment flows at all APs which were then used to inform sediment load calculations and limnological studies for the FRHEP, and were undertaken by others.

Specific objectives of the study are to:

- Provide average, wet and dry flow estimates for designated APs for the pre-mining, LOM, and closure periods for use in the water quality load balance, sedimentological and limnological studies
- Provide an assessment of the proposed operations of the mine, including waste and water management, to determine potential impacts on watercourses downstream of the ISF impoundment, including the Frieda and Sepik Rivers
- Assess the viability of the ISF impoundment as a water supply for the mine and associated operations
- Provide an insight into the reliability of the ISF as a source of water for hydroelectric power generation.

1.2 Statement of SRK independence

Neither SRK nor any of the authors of this Report has any material present or contingent interest in the outcome of this Report, nor any pecuniary or other interest that could be reasonably regarded as being capable of affecting their independence or that of SRK.

SRK's fee for completing this Report is based on its normal professional daily rates plus reimbursement of incidental expenses. The payment of that professional fee is not contingent upon the outcome of the Report.

2 Site Summary

2.1 Location

The Project is located within the Sepik River catchment, approximately 200 km south of the northern coast of mainland PNG and 75 km east of the border with the Indonesian province of West Papua (Figure 2-1).

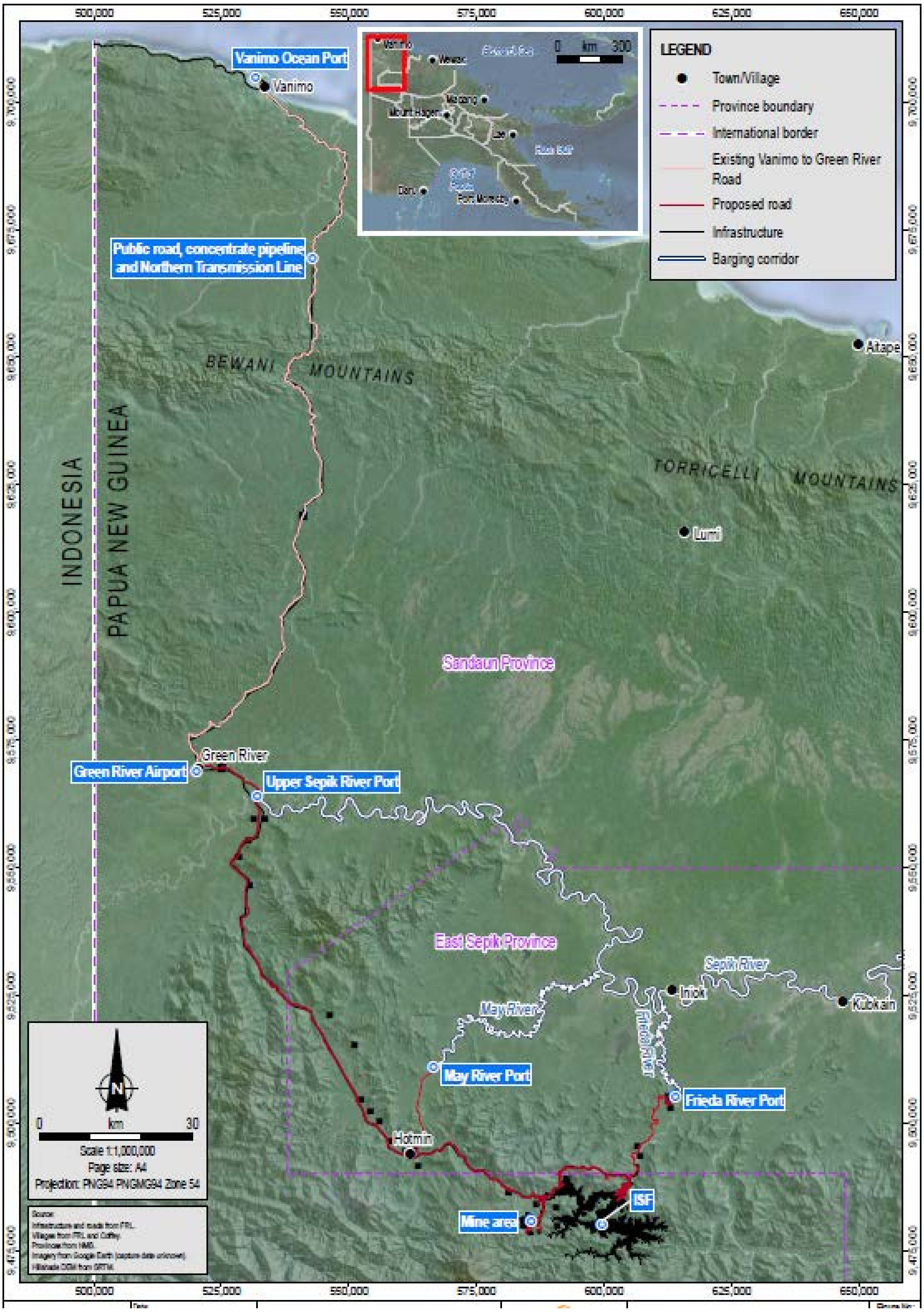

Figure 2-1: Frieda River Project location

The extent of the area covered by the current phase of water balance modelling is presented in Figure 2-2. The AP locations extend from the Ubai (AP1) and Uba (AP2) creeks, downstream of the proposed open-pits, and the Nena River, upstream of the ISF (AP3), to a location in the Sepik River (AP13) at Kubkain. Further details for the APs are provided in Table 2-1.

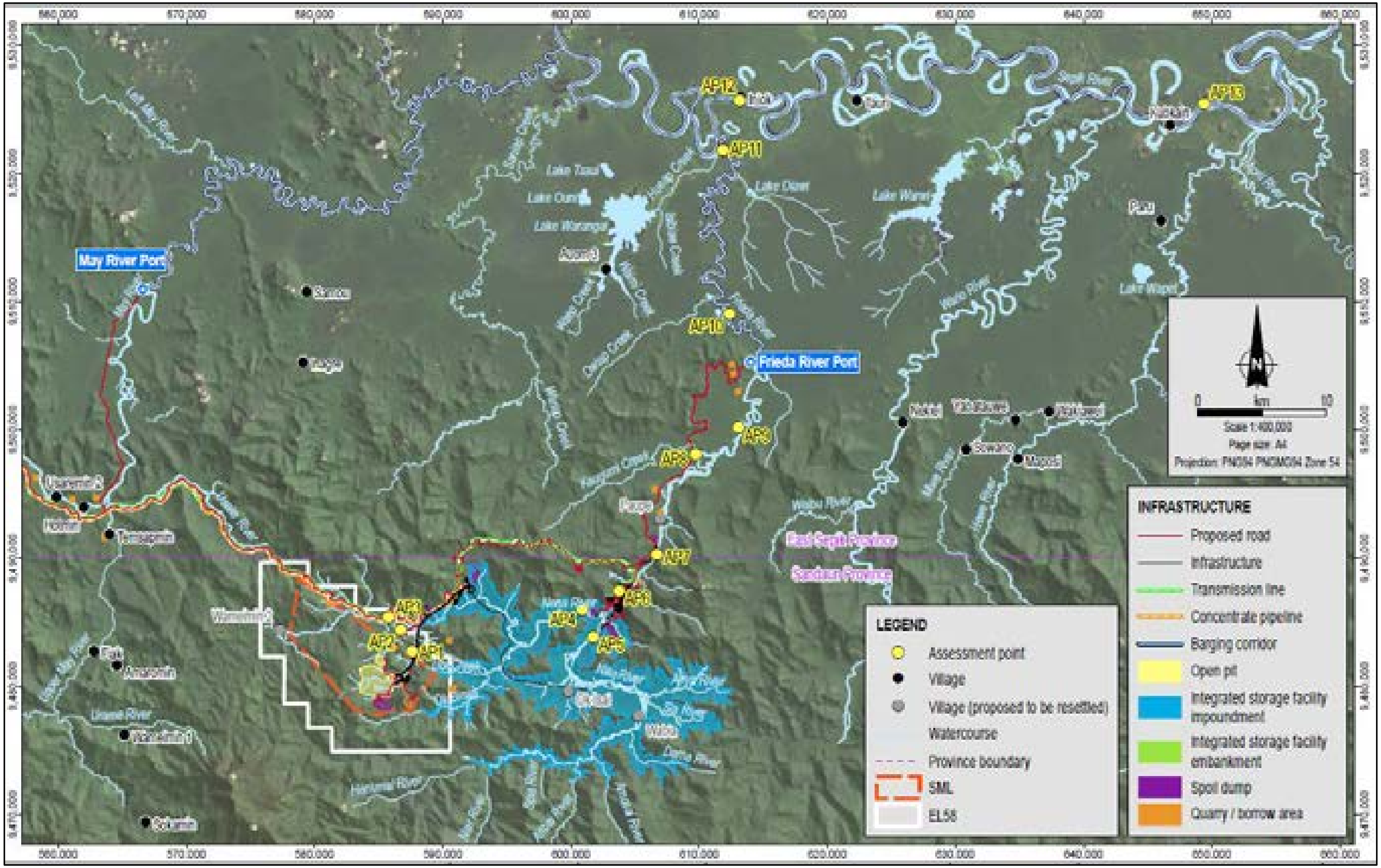

Figure 2-2: Locations of assessment points and mine facility areas

Table 2-1: Details of assessment point locations

Area	Site	Stream	Location	UTM coordinates		Catchment area (km^2)
				Easting (m)	Northing (m)	
Upstream of ISF	AP1	Ubai Creek	Ubai Creek upstream of Nena River	587,573	9,482,676	20
	AP2	Uba Creek	Uba Creek upstream of Nena River	586,686	9,484,334	6
	AP3	Nena River	Nena River upstream of Uba Creek confluence	585,756	9,485,363	151
Mid-catchment, within ISF	AP4	Nena River	Nena River upstream of Frieda River	600,888	9,485,937	351
	AP5	Niar River	Niar River upstream of Frieda River	601,723	9,483,812	652
Mid-catchment, downstream of ISF	AP6	Frieda River	ISF outflow	603,751	9,487,432	1,034
	AP7	Frieda River	Frieda River (airstrip)	606,698	9,490,214	1,047
	AP8	Frieda River	Frieda River (upstream of Kaugumi Creek)	609,703	9,498,016	1,092
	AP9	Frieda River	Frieda River (Frieda Mountain)	613,056	9,500,190	1,210
Lowland Plains	AP10	Frieda River	Frieda River (Lower Frieda River Gauging Station)	612,331	9,509,042	1,345
	AP11	Frieda River	Frieda River (upstream of Sepik River confluence)	611,840	9,521,775	1,466
	AP12	Sepik River	Sepik River (Iniok)	613,145	9,525,695	25,200
	AP13	Sepik River	Sepik River (Kubkain)	649,377	9,525,394	29,500

2.2 Meteorology

2.2.1 Climate setting

Detailed analysis of climatic data from the FRGCP region has been carried out (SRK, 2016c) and results are summarised herein. The FRGCP region has a tropical rainforest climate. Using the international Köppen-Geiger climate classification method, the region falls in the zone ‘Af’. By definition, ‘Af’ regions have the coldest month with temperatures over 18 °C and the precipitation of the driest month over 60 mm (Peel, Finlayson & McMahon, 2007). Monthly average relative humidity is consistently above 80%.

The FRGCP is located in the northern foothills of the New Guinea Highlands (Central Range) in Sandaun Province, with key infrastructure and transport corridors located in the East Sepik Province. It lies in a remote area approximately 200 km from the northern coast and 70 km from the closest navigable point on the Sepik River. The FRHEP is located in tributaries of the Sepik River, these being the Nena River, Frieda River and Ok Binai River.

The Ok Binai meets the Nena River, which then meets the Niar River to become the Frieda River. The Frieda River discharges into the Sepik River close to an elevation of 40 masl. The FRGCP area is limited to the north, west and south by a branch of the Central Range.

The climate and hydrology are seasonally influenced by the location with respect to the mountain ranges. Climatic conditions in the area may be separated into two zones, the uplands and the lowlands. The approximate boundary between the regions, for the purposes of the present study, is the Frieda River Airstrip (AP7).

2.2.2 Precipitation

Precipitation has historically been recorded at a number of tipping bucket rain gauges in the region surrounding the Project. As part of this study, precipitation data from 17 rain gauges have been analysed and the locations of these are presented in Figure 2-3 The rainfall data record available begins in 1994, though it is not continuous. Figure 2-4 graphically presents the data available for each rain gauge. Three locations in Figure 2-3 and Figure 2-4 are automatic weather stations, denoted by the suffix 'AWS', and additional climatic parameters are recorded at these stations. Of the AWS locations, Nena is at the highest elevation, close to the immediate Project area, Moraupi is in the mid-catchment area of the Frieda River, and Iniok is in the lowlands by the Sepik River. The locations in Figure 2-3 are coloured to indicate the approximate level of annual precipitation experienced at each site, and this highlights the difference between the upland and the lowland sites. The upland sites (coloured yellow or red) fall in the precipitation bands above 7,000 mm, whereas the lowland sites (grey, orange or green) represent precipitation bands less than 6,000 mm.

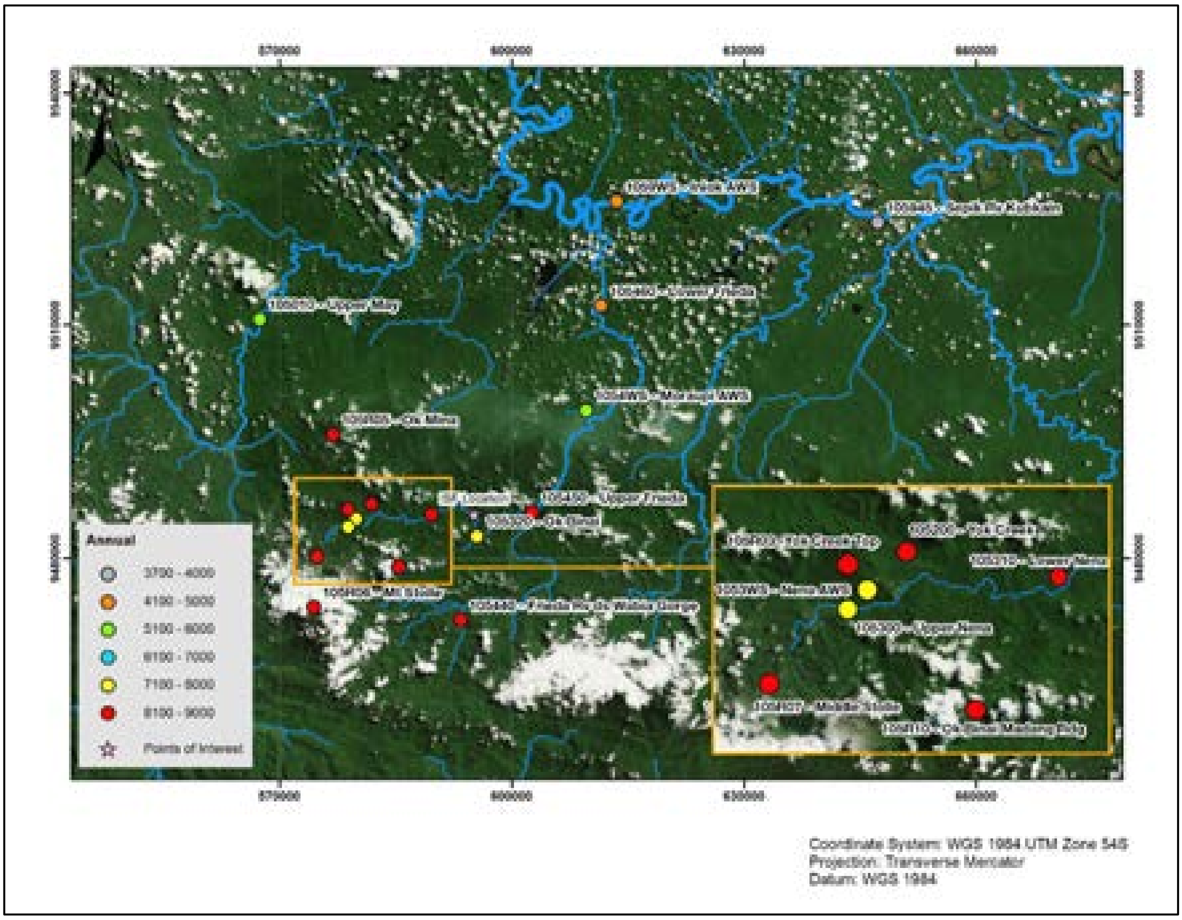

Figure 2-3: Station locations, elevations and mean annual precipitation

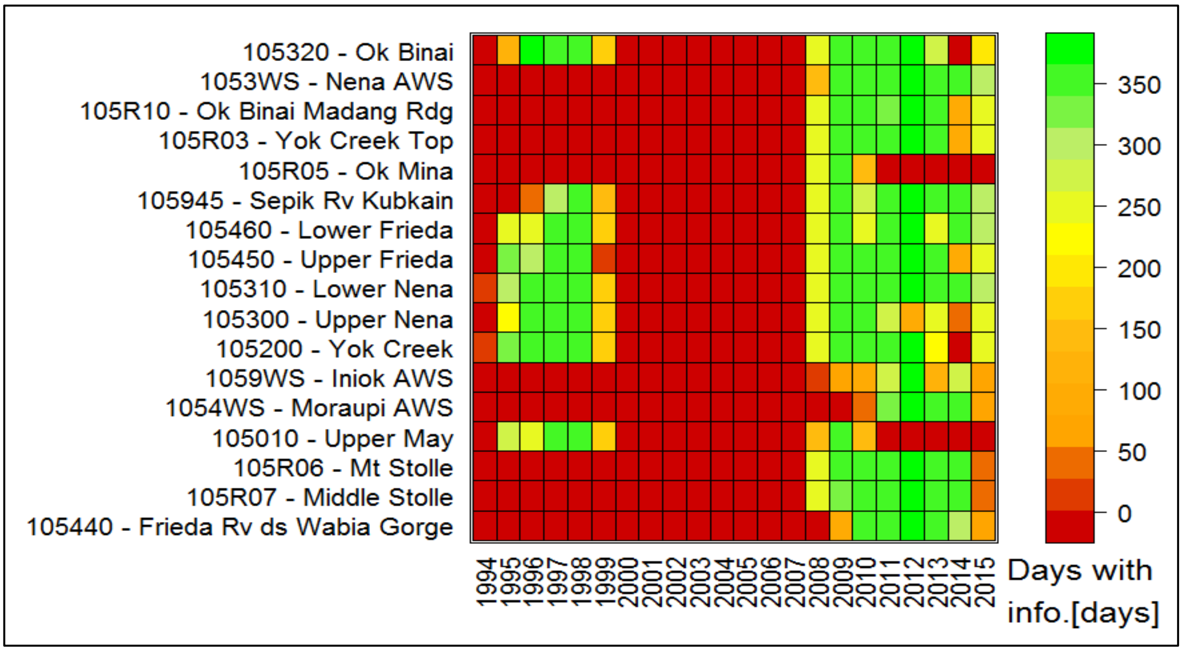

Figure 2-4: Precipitation data available at each station per year [days with data/ year]

Note: Green squares are years with 365 days of information grading down to red squares for years with no information.

The mean annual precipitation (MAP) values for the stations analysed are presented in Table 2-2, with sites indicated as being from the upland or lowland grouping.

Table 2-2: Mean annual precipitation values

Site number	Site name	Data years	MAP (mm)	Easting UTM Zone 54 PNGMG 1994	Northing UTM Zone 54 PNGMG 1994	Elevation (masl)	Grouping
105320	Ok Binai	11	7770	595494	9482874	110	Upland
1053WS	Nena AWS	7	8003	579857	9485084	840	Upland
105R10	Ok Binai Madang Ridge	7	8736	585396	9478946	627	Upland
105R03	Yok Creek Top	7	8281	578860	9486369	1062	Upland
105R05	Ok Mina	2	8572	576873	9495844	480	Upland
105450	Upper Frieda	11	7982	602597	9485957	100	Upland
105310	Lower Nena	10	8162	589619	9485727	190	Upland
105300	Upper Nena	11	7821	578858	9484081	635	Upland
105200	Yok Creek	10	8347	581856	9487016	425	Upland
105R06	Mt Stolle	6	8692	574411	9473706	2240	Upland
105R07	Middle Stolle	6	8338	574861	9480276	850	Upland
105440	Frieda River ds Wabia Gorge	4	8608	593374	9472027	361	Upland
105945	Sepik River Kubkain	8	3693	647260	9523328	20	Lowland
105460	Lower Frieda	10	4728	611508	9512509	25	Lowland
1059WS	Iniok AWS	2	4762	613524	9526037	20	Lowland
1054WS	Moraupi AWS	4	5446	609604	9499011	67	Lowland
105010	Upper May	5	5925	567456	9510800	46	Lowland

The upland region has MAP from 7,700 mm/yr to 8,600 mm/yr, and the average amount of available data for the 12 upland sites was eight years. The lowland region has MAP from 3,700 mm/yr to 6,000 mm/yr, and the average amount of available data for the five lowland sites was six years.

Mean monthly precipitation data are presented in Table 2-3 for the uplands and Table 2-4 for the lowlands. Seasonal variability in precipitation is typically higher in the lowlands than the uplands; however, in both regions the higher precipitation months are February to April, with a peak in March, and the lower precipitation months are May to August. The upland and lowland regions have monthly average precipitation levels of approximately 700 mm and 400 mm respectively. Average monthly precipitation data from the rain gauges are presented in Figure 2-5, and Table 2-3 and Table 2-4, separated into upland and lowland groupings, to illustrate seasonal variability.

Table 2-3: Mean monthly precipitation values for upland sites (values in mm)

Site number	Site location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
105320	Ok Binai	657	734	706	637	639	565	636	611	630	695	615	646
1053WS	Nena AWS	710	717	760	672	582	593	592	653	712	690	658	665
105R10	Ok Binai Madang Ridge	768	792	848	757	716	650	656	691	722	680	743	713
105R03	Yok Creek Top	748	713	766	676	639	641	639	675	709	654	711	711
105R05	Ok Mina	612	823	902	790	686	690	670	656	663	683	763	634
105450	Upper Frieda	698	695	746	673	638	570	639	622	643	731	632	695
105310	Lower Nena	640	732	697	682	620	613	630	685	683	818	721	641
105300	Upper Nena	716	769	776	712	564	567	591	605	607	653	558	703
105200	Yok Creek	692	749	770	766	644	630	609	632	707	722	672	753
105R06	Mt Stolle	655	751	864	795	648	663	755	809	802	671	606	672
105R07	Middle Stolle	706	696	859	746	650	592	665	710	738	683	627	665
105440	Frieda River downstream Wabia Gorge	697	629	849	709	764	702	691	782	766	683	672	663

Table 2-4: Mean monthly precipitation values for lowland sites (values in mm)

Site number	Site location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
105945	Sepik River Kubkain	326	364	447	306	272	219	236	221	278	296	339	390
105460	Lower Frieda	419	446	545	396	307	325	315	309	320	431	398	518
1059WS	Iniok AWS	691	295	652	436	195	225	175	317	338	375	634	430
1054WS	Moraupi AWS	662	291	420	432	375	403	386	443	519	521	494	500
105010	Upper May	442	592	651	609	470	355	395	401	384	493	518	612

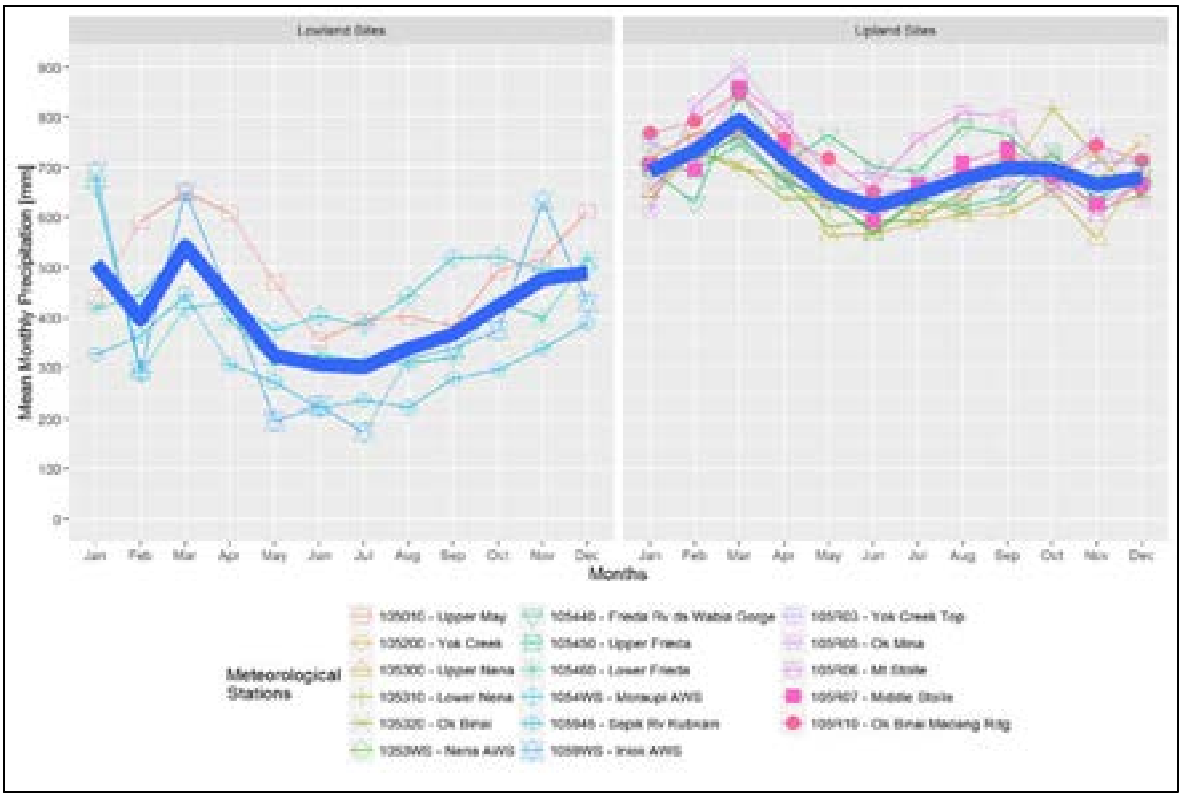

Figure 2-5: Monthly average precipitation data

Note: The thick blue line presents a smoothed line to illustrate the upland and lowland trends.

Storm frequency, intensity and duration interpretation was carried out for individual sites. Results are summarised in Table 2-5 as average upland and lowland precipitation levels for 24 hour storms, for return periods of 2 years, 10 years and 100 years.

Table 2-5: Average 24 hour storm precipitation levels

Return period (years)	Precipitation 24 hr (mm)	
	Upland	Lowland
2	175	156
10	205	196
100	244	240

The probable maximum precipitation (PMP) for locations in the uplands have been calculated. The uplands region was the focus of the PMP analysis as this is where the main mine facilities will be located, i.e. the ISF and the HITEK open-pits. The modelled PMP levels are presented in Table 2-6.

Table 2-6: Modelled PMP levels for ISF and HITEK open-pit areas

Site	PMP (mm)				
	1 hr	6 hr	24 hr	48 hr	72 hr
ISF (E 595120, N 9485670)	190	600	920	1370	1650
HITEK open-pit (E 584861, N 9480314)	210	700	1150	1720	2060

2.2.3 Automatic weather station data

In addition to the precipitation data presented above, monitoring was also carried out at the three automatic weather station (AWS) locations (Nena, Iniok and Moraupi) for relative humidity, solar radiation, temperature, and wind speed.

Monthly average relative humidity data are presented in Table 2-7. The data indicate that all sites experience similar levels of relative humidity, in the range between 81% and 86%, and that variability throughout the year is low.

Table 2-7: Monthly average humidity (values in %)

Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Nena AWS	84	84	85	85	85	86	86	84	85	84	84	84
Iniok AWS	82	82	83	83	84	84	84	82	82	82	83	82
Moraupi AWS	84	82	84	84	84	83	83	81	81	83	83	84

The average monthly solar radiation data are presented in Table 2-8 and indicate that at each site the average amount of solar radiation is relatively constant throughout the year.

Table 2-8: Monthly average solar radiation (values in W/m^2)

Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Nena AWS	194	191	195	178	173	173	171	173	182	194	194	194
Iniok AWS	157	156	172	166	150	147	145	171	159	160	153	155
Moraupi AWS	181	212	195	180	184	194	184	215	243	191	182	181

Monthly average temperatures are presented in Table 2-9. The data indicate that monthly average temperatures are relatively constant throughout the year. Temperatures at the Nena AWS, in the uplands, are typically around 4 °C lower than at the two stations in the lowlands.

Table 2-9: Monthly average temperature (values in degrees Celsius)

Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Nena AWS	23.1	22.9	23.0	22.8	22.9	22.7	22.5	22.6	22.8	22.9	23.1	23.2
Iniok AWS	27.1	27.0	26.9	27.0	26.7	27.0	26.6	26.6	26.7	26.9	27.0	27.2
Moraupi AWS	26.7	27.0	26.8	26.7	27.0	27.0	26.6	26.9	27.1	26.6	26.8	26.8

Monthly average wind speeds are presented in Table 2-10 and indicate that at each site there is minimal variance throughout the year. Wind speeds appear to increase with decreasing altitude, with the lowest values being recorded at the Nena AWS and the highest at the Moraupi AWS.

Table 2-10: Monthly average wind speed (values in m/s)

Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Nena AWS	0.47	0.49	0.46	0.50	0.52	0.46	0.46	0.56	0.50	0.55	0.51	0.51
Iniok AWS	0.83	0.84	0.91	0.78	0.71	0.87	0.88	0.91	0.77	0.85	0.85	0.81
Moraupi AWS	1.36	1.38	1.34	1.32	1.33	1.27	1.31	1.38	1.41	1.42	1.41	1.36

Evapotranspiration (ET) and evaporation were estimated using the complementary relationships for areal evapotranspiration (CRAE) and wet-surface evaporation (CRWE) methods (Morton, 1983). The model method uses different procedures to estimate reservoir evaporation and land ET. Morton's methodologies were applied to the Nena AWS station, which is in the same watershed as the Project. For model inputs, evaporation parameters, including air temperature relative humidity, and solar radiation, were obtained from the monthly average records at the Nena AWS station. Dewpoint temperature was estimated from the relative humidity values and monthly air temperature. Results for modelled reservoir evaporation and ET for the Nena AWS area are presented in Table 2-11 and are applicable to the uplands. ET for the lowlands is estimated to be approximately 1.25 times higher than in the uplands, based on information for differences in evaporation due to altitude, as presented in the handbook *Climate of Papua New Guinea* (McAlpine et al., 1983).

Table 2-11: Modelled evapotranspiration and pan evaporation (mine area)

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Evapotranspiration (mm)	152	134	156	134	131	120	122	124	133	150	145	151	1651
Reservoir evaporation (mm)	158	143	160	141	140	133	135	141	146	158	153	157	1765

2.3 Hydrology

Most of the Project infrastructure is within the catchments of the Nena, Niar and Frieda rivers, all draining into the Sepik River, which at 1,100 km long, is one of the longest rivers in Papua New Guinea.

The HIT and Ekwai open-pit areas are within the Ubai Creek catchment and the Koki open-pit is in the Uba Creek catchment. Both the Ubai and Uba creeks flow directly into the Nena River. The upper catchments flow through steeply incised valleys and are characterised by relatively narrow channels with steep banks and rocky beds containing large boulders. Flows are characteristically high energy and velocity due to the frequent rainfall events. As a result of conditions, there is typically little build-up of loose sediment or vegetation in the stream beds.

On moving downstream, the valley terrain remains relatively steep throughout the mid-catchment area where channels are typically wide, with straight to partly meandering channels containing cobble/ gravel beds and banks. Assessment points AP7 and AP8 are located in this mid-catchment area on the Frieda River.

From the area around location AP9 (Frieda Mountain), the Frieda River enters the lower gradient lowland plains and meanders through the Sepik River floodplain. In the floodplain area, river dynamics have created a series of oxbow lakes and main channels are commonly wide with highly braided sections.

Historical flow records from several locations have been analysed and mean monthly flow results from three of the gauge stations are presented in Table 2-12. Data availability for each of the gauging stations is presented in Figure 2-6. Gauge station 105310 is in the Nena River, at a location within the proposed ISF, downstream of the confluences with the Uba and Ubai creeks, station 109450 is on the Frieda River close to the airstrip (AP7), and station 105945 is in the Sepik River at Kubkain (close to AP14).

Table 2-12: Mean monthly flows in Nena, Frieda and Sepik Rivers (ML/day)

Location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
105310 (Nena River)	3,292	4,052	3,802	3,879	3,119	3,205	3,214	3,223	3,370	3,482	3,136	3,413	3,459
105450 (Upper Frieda)	19,440	22,205	21,600	21,082	18,576	17,971	18,230	18,922	18,144	19,786	17,971	18,317	19,267
105945 (Sepik River at Kubkain)	291,341	346,723	393,293	375,667	286,762	209,606	218,246	222,480	186,019	204,509	241,661	274,147	288,230

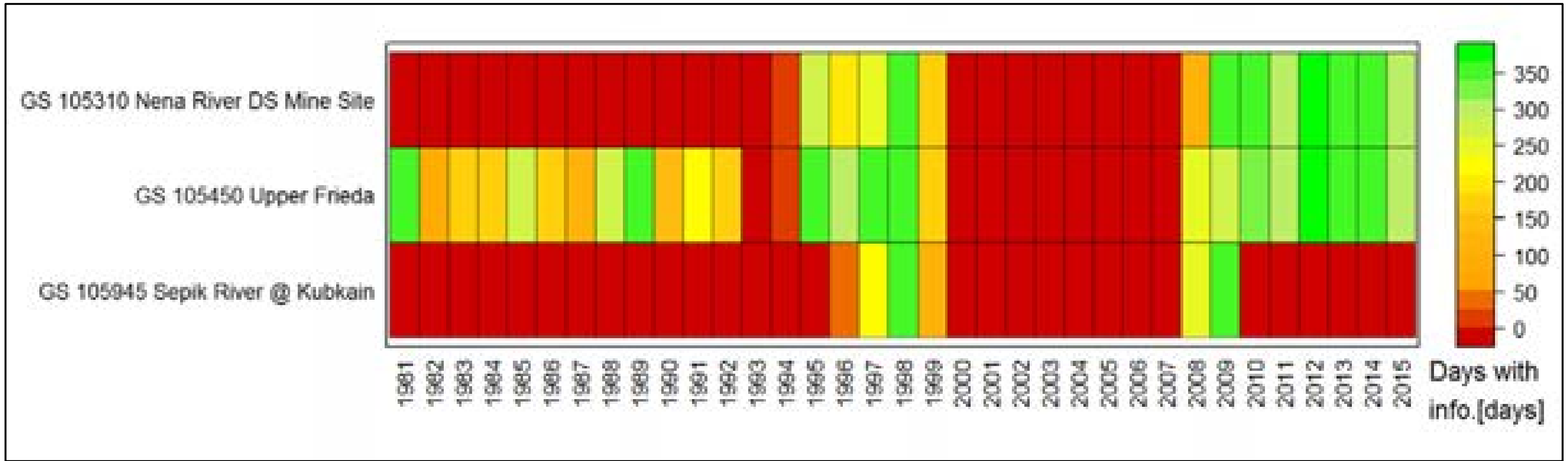

Figure 2-6: Flow gauging data available at each station per year [days with data/year]

Note: Green squares are years with 365 days of information grading down to red squares for years with no information.

The data in Table 2-12 are based on gauge data from the following years:

- 105310 Nena River: 1995–1999 and 2008–2015
- 105450 Upper Frieda: 1981–1992, 1995–1999 and 2008–2015
- 105945 Sepik River at Kubkain: 1996–1999 and 2008–2009.

Additional flow data for the three sites are presented in Table 2-13 to indicate the ranges experienced at each of the gauging stations.

Table 2-13: Flows in Nena, Frieda and Sepik rivers

Location	Flow (ML/day)				
	Minimum	10%	Median	90%	Maximum
105310 (Nena River)	423	1,202	2,699	6,518	23,239
105450 (Upper Frieda)	49	9,596	16,872	32,429	110,588
105945 (Sepik River at Kubkain)	989	127,940	260,656	378,167	520,320

The flows recorded at the gauge stations are presented as time-series graphs in Figure 2-7, Figure 2-8 and Figure 2-9 to further highlight variability and the periods of data availability. Flow duration curves for the sites are presented in Figure 2-10.

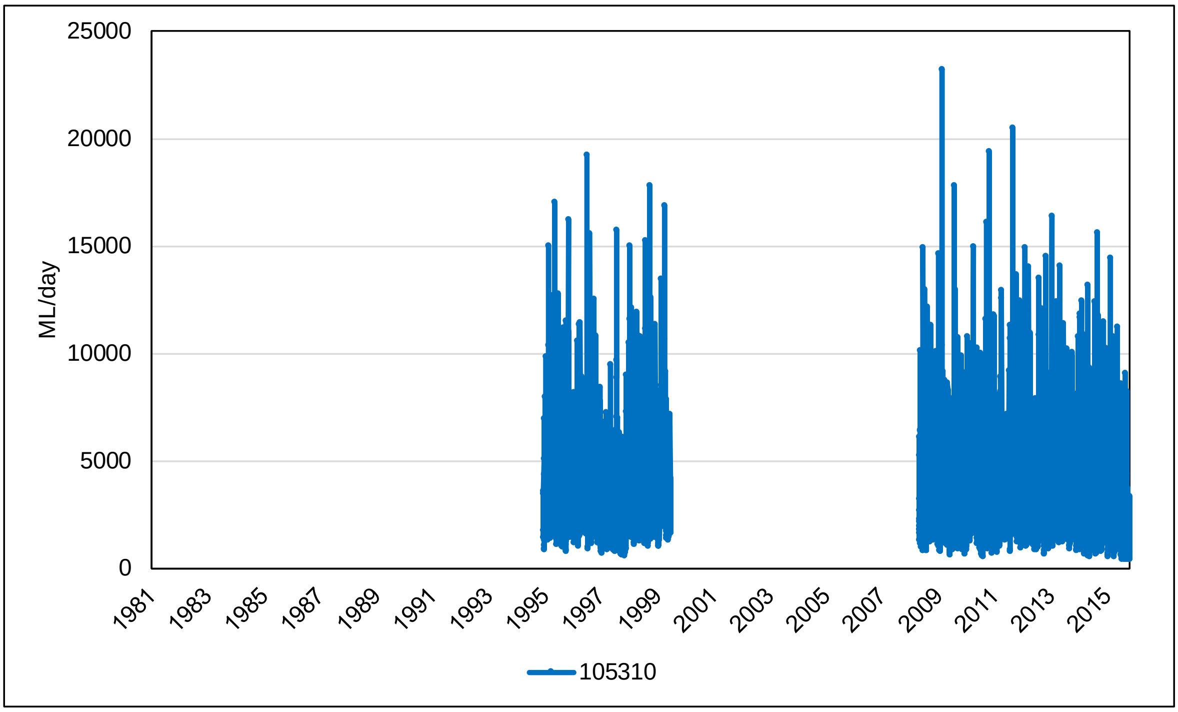

Figure 2-7: Time series flow data for station 105310, Nena River

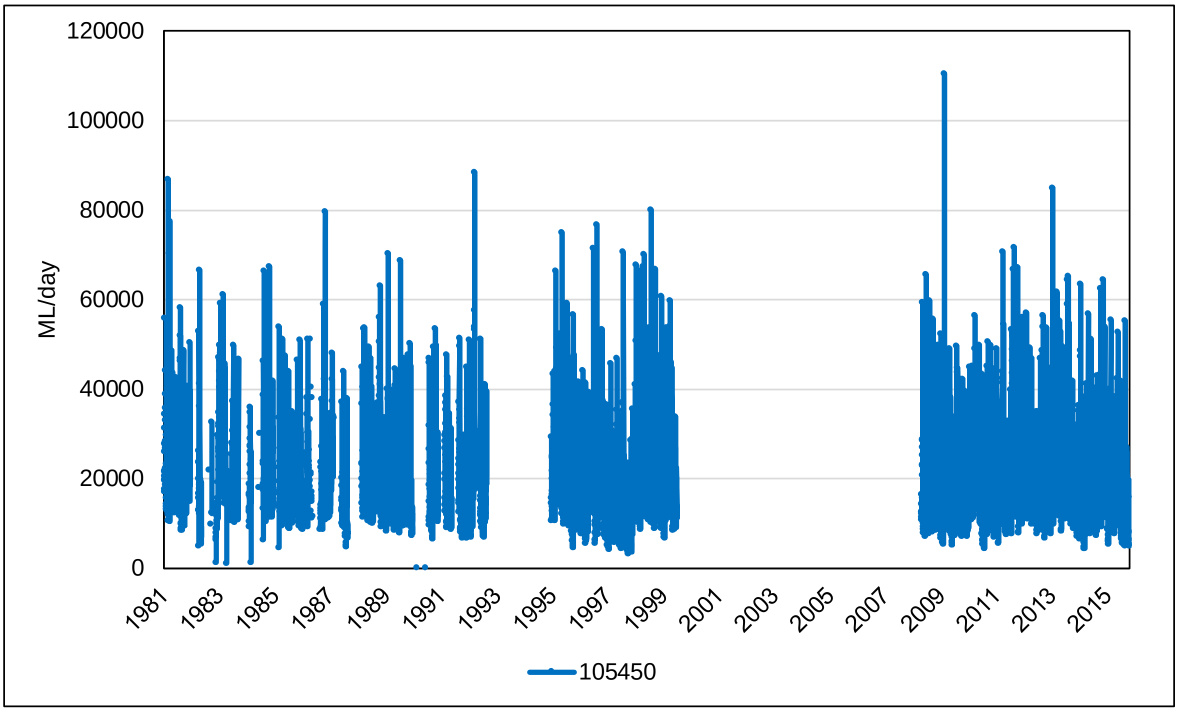

Figure 2-8: Time series flow data for station 105450, Frieda River

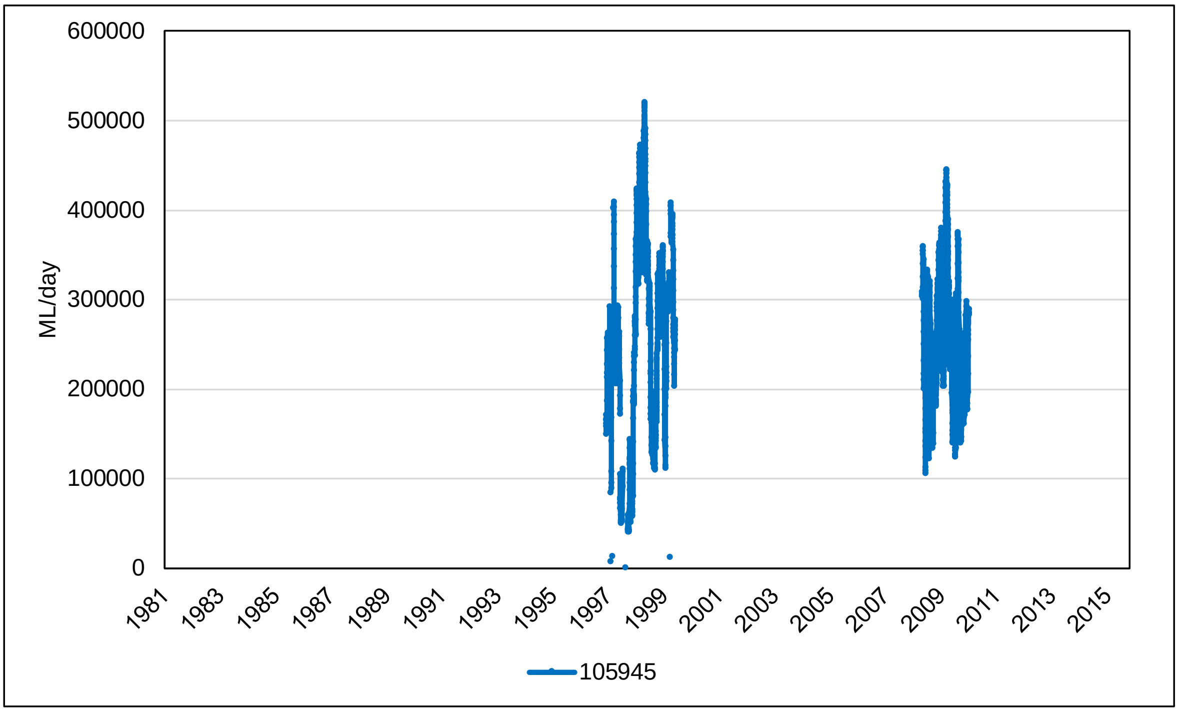

Figure 2-9: Time series flow data for station 105945, Sepik River

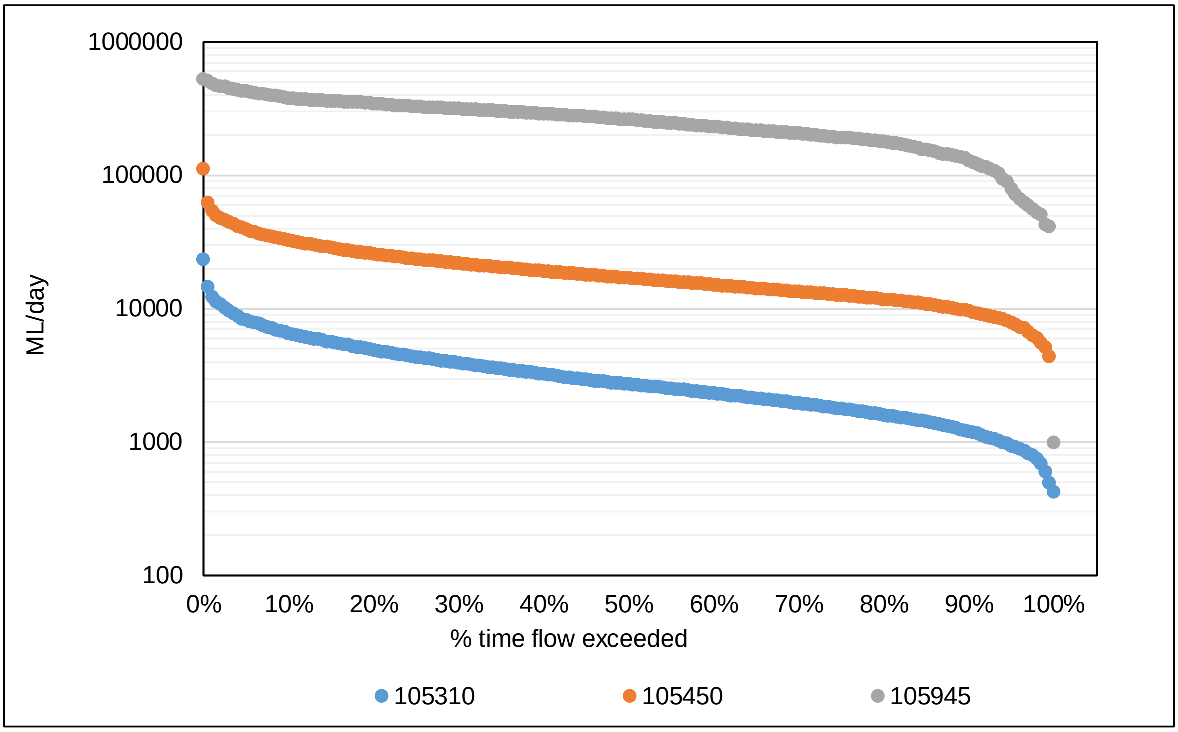

Figure 2-10: Flow duration curve for stations 105310, 105450 and 105945

2.4 Climate change

Climate change effects have been investigated (SRK, 2016c) and are expected to develop over long timescales relative to the Project life. Long-term trends are summarised in Table 2-14; however, these trends would not be expected to manifest within the timeframe of the Project. Because minimal increases and decreases over the length of the Project would be expected, the current water balance modelling has not included specific factors for climate change.

Table 2-14: Overall climate change trends

Climate factor	Trend	Justification
Frequency of drought	Unknown	Mean annual air temperature and precipitation have increasing trends, and the number of consecutive dry days is also increasing; as a result, there is insufficient information to predict the future frequency of drought.
Frequency of extreme temperatures	Unknown	Possible increase in extreme temperatures, but the strength of the trend is unknown.
Frequency of rainfall	Increasing	Trend is assumed to increase due to increasing trends in precipitation days and total precipitation.
Heavy rain	Increasing	Increase in heavy rainfall of 98% by 2100.
Total rainfall	Increasing	Increase in total precipitation of 281 mm (8.8%) by 2100.
Floods and storms	Unknown, likely Increasing	Increasing temperature, precipitation, and heavy rainfall trends to 2100 suggest that storms are increasing. Therefore, it is likely reasonable to assume that storms and floods will increase by 2100.
Average temperature	Increasing	Increase in mean annual air temperature of 4.1 °C by 2100.
Wind speed	Unknown	The wind speed trend tends to fluctuate, and the deviation from the baseline is minimal, making the overall trend difficult to understand.

3 Water Balance Model Components, Inputs and Operational Logic

A site-wide water balance model for the Project was constructed using the GoldSim modelling platform. Modelling was performed using a daily time step for a 56-year period, to estimate flows and volumes throughout the 33 year mine life as well as 20 years post-operations. Note that the model was set to commence (i.e. year "0" in all subsequent discussion) at the beginning of filling of the ISF (FRCGP Year -2).

3.1 Stochastic precipitation generation

Two stochastic rainfall modules were constructed in GoldSim to generate a wide range of climate sequences for the 56-year model run period. Separate stochastic rainfall modules were used for the upland and lowland regions, as data indicate the lowland region to be around 40% drier than the uplands. The stochastic modules generate precipitation sequences that are intended to have the same seasonality and statistical characteristics as the historical datasets for a range of parameters, including mean, variance, skew, and number of wet days or dry days. The stochastic inputs developed in this manner allow different temporal patterns to be simulated. For example, one model run might have wetter years at the start of the sequence, where another might have the wetter years towards the end of the sequence. Some of the stochastic rainfall sequences may be wetter or dryer than others, just as any given period of real climate data can be nominally different from, but statistically similar to, preceding years.

Full stochastic simulation of climate data for a region as extensive as that shown in Figure 2-1 is a highly involved process. However, the stochastic method adopted for this work was relatively simple, for two reasons. The first reason is that the ISF impoundment will smooth out much of the variability in upstream flows, making a more elaborate upstream model unnecessary. The second is that the data available from stations downstream of the ISF are very limited in comparison to the temporal and geographic extent of the model domain, making it difficult to support a more complex approach. SRK believes the limitations of the simple stochastic approach are reasonable for the intended purposes of this report, but caution is needed in interpreting the results.

Input parameters for stochastic precipitation generation for the uplands were selected from 11 years of data from the Lower Nena meteorological station (1995–1998, 2008–2014). This station was selected due to its proximity to proposed mine facility locations, and uplands precipitation was used to generate flows for stations AP1 to AP7. Numbers of dry and wet days were determined from daily statistics and depth of rain was developed by manually adjusting a Gumbel maximum distribution to fit monthly totals, +/- 1 and 2 standard deviations, and the annual frequency of 24 hour storms. A second order Markov chain was used to generate precipitation, where the probability of a wet day is based on one of two probabilities, chance of rain if the previous day was dry or chance of rain if the previous day was wet. If a wet day is predicted, a Gumbel distribution fit to all the wet days of the current month is used to generate the depth of the rainfall. The method used is based on the WGEN model (Richardson and Wright, 1984), which is commonly used in generation of stochastic precipitation data.

Stochastic precipitation for the lowlands (flow locations AP8 to AP13) was generated using data from 13 years (1995–1999, 2008–2015) for the Lower Frieda rain gauge. The methodology was similar to the one outlined for the uplands; however, a Johnson SB distribution was used to fit wet days, in place of the Gumbel distribution used for the uplands.

Two sets (upland and lowland) of 100 sequences of rainfall data were produced for a 56 year model run period, allowing a range of climatic conditions to be simulated. The maximum one-day rainfall generated for the upland sequence was 371 mm, equivalent to a 1,000 year return period, 24 hour storm, but significantly less than the calculated PMP for the ISF of 920 mm (Table 3-1). The maximum one-day rainfall generated for the lowland sequence was 225 mm.

Monthly rainfall data are compared in Figure 3-1 and Figure 3-2, which highlight an excellent comparison between measured and modelled mean values. Using the three-sigma rule, approximately 68% of data should fall within one standard deviation (1σ) below and above the mean (16%–84% in a normal distribution). The stochastically generated values at the 16th and 84th percentile are a good fit to the empirical data for mean -1σ and mean +1σ respectively, indicating that the model should reliably simulate normal periods of high and low flow. The match for wetter conditions (mean +2σ versus 98%) is good for the upland scenario and good for approximately half the months of the lowland scenario. Drier conditions are a good fit for approximately half of the months for both upland and lowland scenarios. Although the annual totals deviate at most 10% from the empirical values (Table 3-1), any discrepancy indicates a need for cautious interpretation of model results, particularly from the lowest flow periods.

Table 3-1: Annual precipitation at the Lower Nena and Lower Frieda stations and from stochastic data

Station	Statistic	(mm/yr)	Stochastic (mm/yr)		Stochastic as percentage of measured
			(mm/yr)	Percentile	
Lower Nena	Mean -2σ	6494	7158	2%	110%
	Mean -1σ	7207	7707	16%	107%
	Mean	7920	8214	Mean	104%
	Mean +1σ	8633	8726	84%	101%
	Mean +2σ	9346	9308	98%	100%
Lower Frieda	Mean -2σ	3668	3883	2%	106%
	Mean -1σ	4154	4284	16%	103%
	Mean	4640	4683	Mean	101%
	Mean +1σ	5126	5085	84%	99%
	Mean +2σ	5612	5594	98%	100%

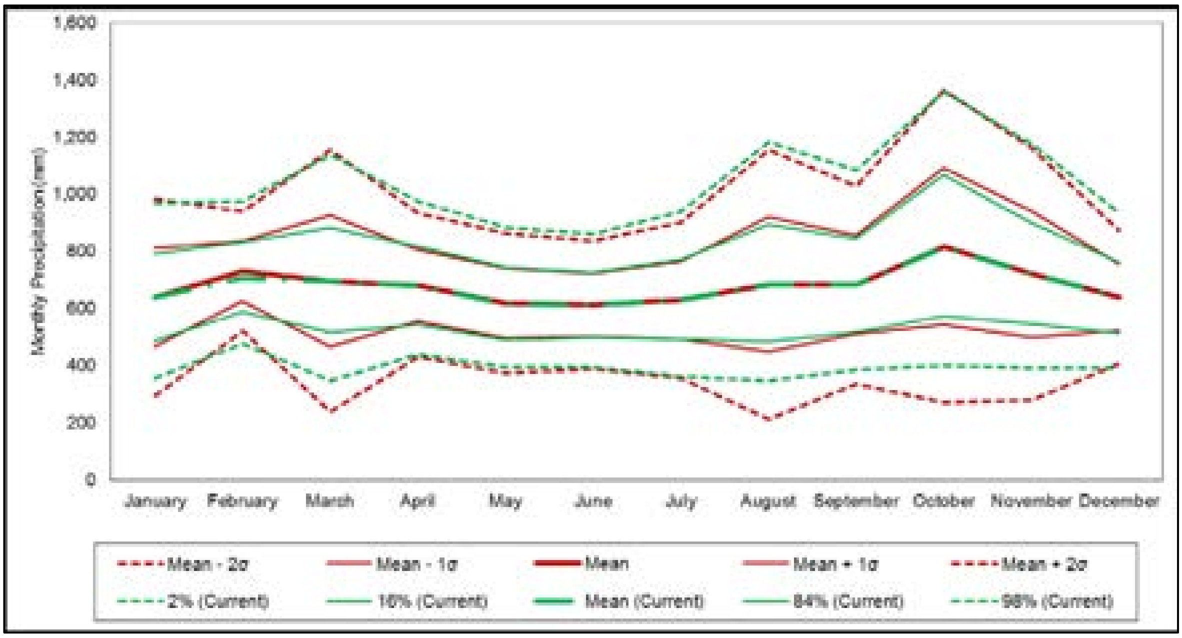

Figure 3-1: Comparison of measured and stochastic monthly rainfall – uplands

Note: Values from historic data are shown in red and values from stochastic rainfall generation are shown in green.

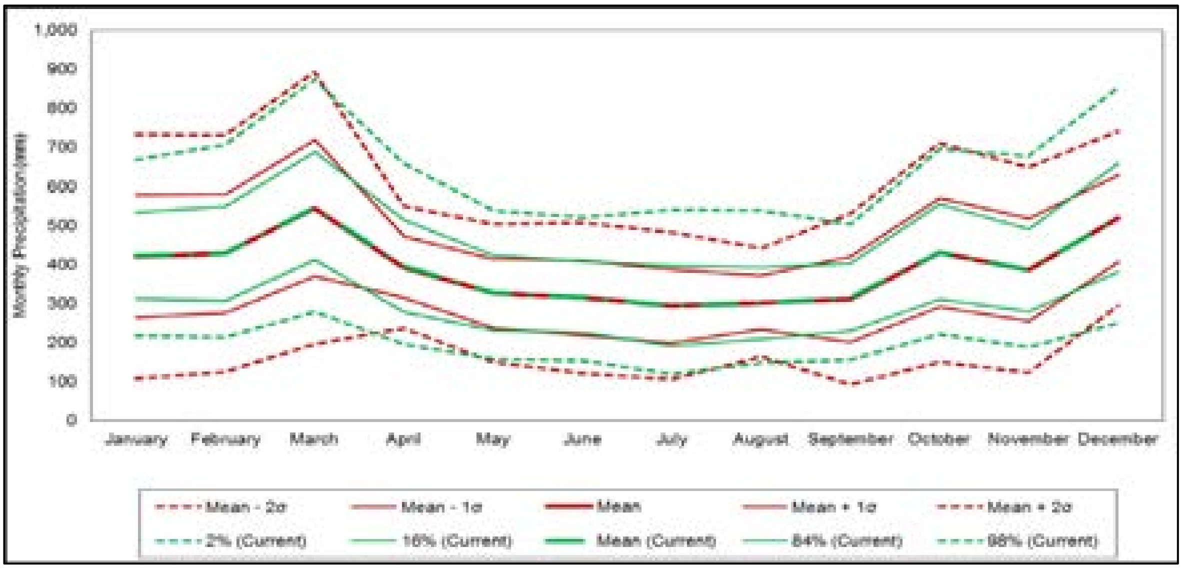

Figure 3-2: Comparison of measured and stochastic monthly rainfall – lowlands

Note: Values from historic data are shown in red and values from stochastic rainfall generation are shown in green.

The El Niño Southern Oscillation (ENSO) has an influence in driving year to year rainfall variability in Papua New Guinea. Approximately every 6 to 13 years, Papua New Guinea suffers from a lack of rainfall in many parts of the country. These events are almost always associated with El Niño phases of the ENSO. Conversely, La Niña episodes exhibit a weak trend in increased rainfall patterns. Although the historic dataset used to create the stochastic precipitation patterns covered a relatively short period, it did include the 1997–1998 El Niño event and a short La Niña event in 2008–2009. Variability related to the ENSO is therefore likely to have been at least partly captured in the stochastic variability. However, much drier and wetter conditions could be possible. Definition of these would require a larger empirical dataset.

3.2 Evaporation and evapotranspiration

Evaporation and ET are required as model inputs to account for losses from the ISF and from the land surface and vegetation. As outlined in Section 2.2.3, measured parameters including temperature, relative humidity and solar radiation from the Nena AWS station were used to model evaporation and ET. Modelled ET values in Table 2-11 were used directly for losses from the land surface and vegetation for the uplands and modelled ET values multiplied by 1.25 were used for the lowlands. The multiplication factor for the lowlands was based on information for differences in evaporation due to altitude in the region, as presented in the handbook *Climate of Papua New Guinea* (McAlpine et al., 1983). Modelled reservoir evaporation values were used for losses from the ISF.

3.3 Catchment modelling

An Australian Water Balance Model (AWBM) module (Boughton, 2004) was constructed within the Project GoldSim model to simulate runoff at each of the AP locations shown in Figure 2-2. The AWBM is a commonly-used catchment water balance model method which simulates storage within a catchment in a series of stores of different capacity, to simulate the variable nature of catchment morphologies. Overflow from the surface stores, when rainfall exceeds their capacities, is routed to a further two storages which allow generation of both baseflow and surface runoff components which feed stream flow. Values for surface storage capacities and factors for partitioning between surface flow and baseflow are ideally generated through calibration with gauging data from drainages within the study area. Figure 3-3 is a schematic representation of the AWBM.

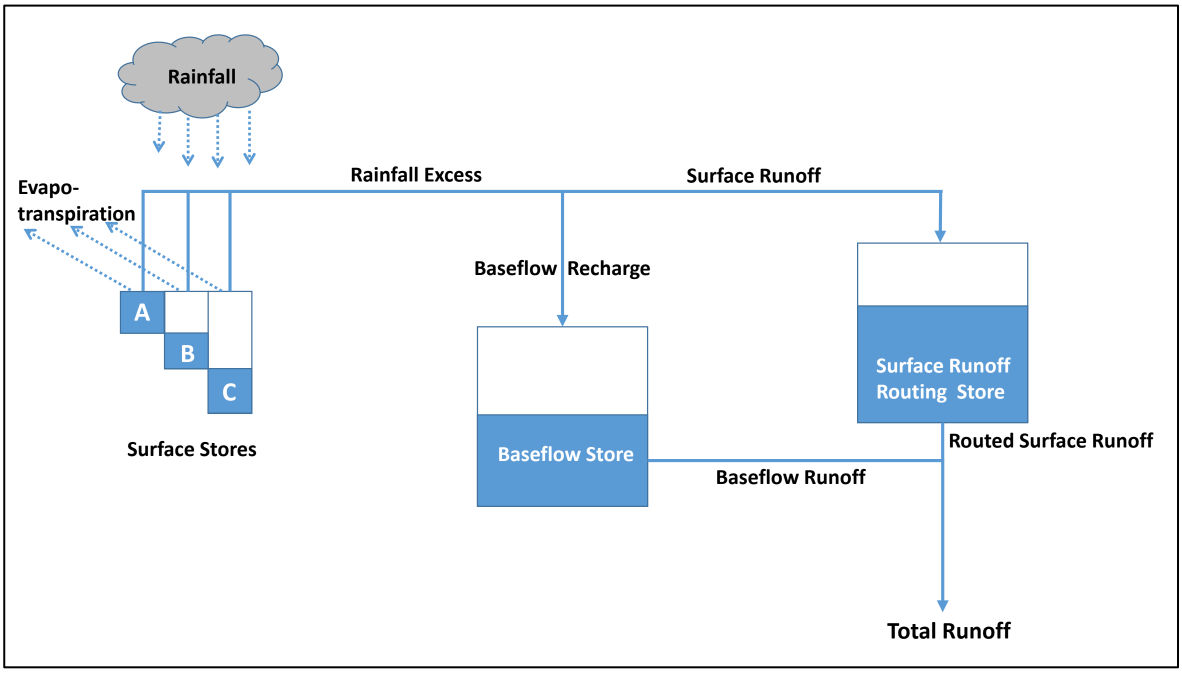

Figure 3-3: AWBM schematic

AWBM input parameters were derived from comparison of daily rainfall with stream flow at gauge stations in the Project area. Initial AWBM parameters were consistent with those previously defined for the Frieda River Project by SKM (2011) and SRK (2016a, 2016b). AWBM parameters for the uplands area (i.e. AP1–AP7 catchments) were then calibrated to the detailed flow data developed as part of a detailed hydrologic analysis of the Frieda Basin presented in SRK (2018).

Initial and calibrated parameter values used in the uplands AWBM calculations are presented in Table 3-2 and calibration results are presented in Table 3-3.

Table 3-2: Parameters used in AWBM

Parameter		Initial Value	Calibrated Value
Surface storage depths (mm)	A	28	7.9
	B	31	31.3
	C	247	247.2
Surface storage partial area proportions	A	0.253	0.253
	B	0.744	0.744
	C	0.003	0.003
Baseflow Index (proportion of surface store (A, B, C) excess which reports to baseflow store each day)		0.6	0.88
Proportion of water from baseflow store which reports to stream flow each day		0.55	0.55

Table 3-3: Calibrated flows for AP6

Parameter	Calibration Target (SRK, 2018)	Calibrated AWBM Value
Mean daily flow (m^3/s)	222	223.6
Minimum daily flow (m^3/s)	20	20.3
Maximum flow (m^3/s)	5000	2614.9

The calibration results show good correlation with minimum and average flow conditions, but underestimate high flows. This is due to the AWBM methodology of apportioning flow to storages before generating runoff, which generally leads to underestimation of peak flows. AWBM is designed as a long-term water balance model and is not considered suitable for peak flow estimation. With respect to the Project, and for the primary objectives of assessing ISF volumes and generating load estimates, the limitations of the AWBM should be considered conservative, and therefore SRK considers the AWBM runoff calculations to be appropriate for use in the water balance.

The AWBM method was used for all catchment areas within the Project GoldSim model, except open-pit walls. For open-pit wall surface sub-catchments, a runoff coefficient of 0.95 was used in conjunction with the assumption of zero storage capacity, i.e. in any day with rainfall, 95% of the precipitation was modelled as exiting the open-pit sub-catchments as runoff and the remaining 5% as evaporation.

3.4 Open-pit development

Open-pit surface areas at development in Years 7, 18 and 33 were used in the GoldSim model, together with the assumption that the final areas for each time period are also applicable to the preceding years, i.e. the areas on the first day of Year 8 are the same as those at the end of Year 18. This is not unreasonable as diversions will be established at the beginning of each major development stage and these would be retained for the duration of that stage (SRK, 2018). The Year 33 footprint areas were maintained for the closure period modelled.

Two zones of possible surface water diversions were defined within each open-pit: i) an upper zone from which runoff can be channelled to an ex-pit collection point, and, ii) a lower zone from which runoff is assumed to collect in a sump in the base of the open-pit. Water from the upper and lower zones was assumed to run off at a rate of 95% of incoming rainfall, as outlined in Section 3.3.

Footprint areas were also assigned to zones of natural catchment that report to bench diversion channels, and the water was assumed to mix with open-pit wall runoff. Runoff from these catchment areas was modelled using the AWBM method. Further diversion channels were also modelled, and water is diverted from clean catchment areas around the open-pit developments. Water from these diverted areas is assumed to remain clean, i.e. has no contact with open-pit walls.

The surface areas used in the GoldSim model for all three open-pit areas (HIT, Ekwai and Koki) are presented in Table 3-4. The HIT and Ekwai open-pits are in the catchment reporting to AP1, and the Koki pit is within the AP2 catchment (Figure 2-1).

Table 3-4: Surface areas used in open-pit contact water catchment runoff modelling

Years	Open-pit areas reporting to sumps (m^2)		Natural catchment reporting to sumps (m^2)		Diverted natural catchment reporting to streams (m^2)	
	HIT/Ekwai	Koki	HIT/Ekwai	Koki	HIT/Ekwai (AP2)	Koki (AP1)
1–7	1,021,900	303,383	514,700	195,942	5,091,997	97,246
7–18	1,779,900	303,383	1,179,700	195,942	2,404,136	97,246
18–33	645,610	303,383	3,134,397	195,942	2,005,593	97,246

3.5 Groundwater inflow to open-pits

All groundwater inflow to the open-pits is assumed to report to the basal open-pit sumps, where it is collected along with runoff from the lower open-pit walls. Inflow rates were provided from groundwater modelling work carried out for the Project (AGE, 2018). Average values from the AGE (2018) modelling are presented in Table 3-5.

Table 3-5: Average daily groundwater inflow rates

Year	Groundwater inflows to open-pits (ML/day)	
	HIT/Ekwai	Koki
2	4.86	0
5	12.22	0.5
10	17.36	2.83
15	19.80	2.59
33	25.88	2.16

3.6 Water treatment

All water collecting in the open-pit sumps and from the bench diversions is likely to be in contact with disturbed areas and will not be suitable for direct discharge. The water balance model therefore incorporated a treatment plant, through which all contact water (i.e. water that has been in contact with the pit surfaces) will be directed for treatment prior to discharge. The treated effluent was modelled as discharging to Ubai Creek upstream of location AP1.

3.7 ISF

The ISF is a major component of the Project water balance model. The reservoir will have solid inputs (tailings and waste rock from mining operations, and natural sediment present in runoff) and water inputs (tailings water, runoff and rain falling directly on the reservoir surface). Losses from the ISF will include evaporation from the reservoir surface, minor seepage through the embankment and seepage from the base of the impoundment, outflow through the integrated hydroelectric power system and outflow through the spillway. Water will also be attenuated within the pore space tailings and waste rock.

The sequence of the ISF embankment crest elevations was not incorporated into the water balance model, rather embankment construction was assumed to be completed in advance of the water levels. Reservoir capacity was estimated from the available topography and embankment design and a stage-storage volume to surface area relationship was established and incorporated in the water balance model. Reservoir capacity was reduced at each time-step to account for the introduction of solids in the form of tailings, waste rock and sediment deposition from natural drainages. A production schedule for tailings and waste rock was provided by FRL and included into the model (SRK, 2018). The sediment loads for operation and for the closure period were estimated from modelling data provided by Golder Associates (Golder, 2018). The ISF water balance inputs and outputs are discussed, along with other model results, in Section 4.

Water from the ISF flows to the downstream catchment either through the hydroelectric power system or via a spillway. Hydroelectric water demand was modelled for the facility and a series of operating rules incorporated into the model by Robinson Energy Ltd (Robinson, 2018). The generation of hydroelectric power in the first two years of operation will only be possible once water levels in the reservoir exceed 161.9 m RL. A minimum environmental flow of 50 m^3/s was established for the all periods. For FRGCP years 2–55, hydropower generation in the model is interrupted below 199.4 m RL. If water levels in the reservoir fall below the level required for hydroelectric power generation, minimum environmental flows are maintained through the hydroelectric power system. Where sufficient water is available (dependent on reservoir level), the flow rate for hydroelectric power generation has been provided by Robinson (2018). All outflow from the hydroelectric power system and the spillway will re-enter the Frieda River system upstream of location AP6 (Figure 2-2).

Prior to hydroelectric power operation, and during periods where volumes of inflows (minus storage) exceeds hydroelectric power requirements, excess flow will pass through the spillway system. Flow from the spillway was modelled to occur only when water levels in the ISF exceed 225 m RL (i.e. invert elevation of the spillway).

Operation of the ISF is designed to ensure that a water cover remains over the tailings and waste rock at all times to inhibit sulphide oxidation and generation of acidity. During barge-dumping operations, a minimum water depth of 10 m is required for barge passage. The maximum elevation of deposition will be about 160 m RL, whereas the minimum operating level for the hydroelectric facility is set at 199.4 m RL, which will ensure a water depth of water above the waste rock and tailings will always be in excess of 40 m. Evaporation (Section 2.2) and seepage through the bottom of the impoundment (see memorandum provided in Appendix A) have been captured in the water balance model.

4 Water Balance Modelling Results

4.1 Introduction

Following set up of the FRGCP GoldSim water balance model, 100 realisations were processed, each with two different stochastic precipitation patterns (uplands and lowlands). This allowed calculation of statistics to provide an indication of the likely range of flows that may be encountered. Additionally, the results facilitate verification of hydroelectric power and process water supply, planning for mine infrastructure such as open-pit sumps and water treatment capacity, and for water quality modelling. The results presented in this section are derived from the stochastic water balance model.

To address the objectives of the water balance exercise, two primary scenarios were developed for the FRGCP, a pre-embankment and post-embankment model. The pre-embankment scenario was developed to provide flow data for use in sediment load and limnological studies, and for assessment of baseline water quality loads for the Project area. The post-embankment scenario was developed primarily to inform the modelling of solute loads and to assess water levels in the ISF during the operational and closure phases.

4.2 Base Case: pre-embankment flows

4.2.1 Assessment point flows

The AP locations are shown in Figure 2-2. Stochastic modelling results indicate that flows at the AP sites will typically range over at least one order of magnitude; however, the range between the 10th (i.e. 'dry' rainfall) and 90th (i.e. 'wet' rainfall) percentiles is significantly smaller (Figure 4-1 and Table 4-1).

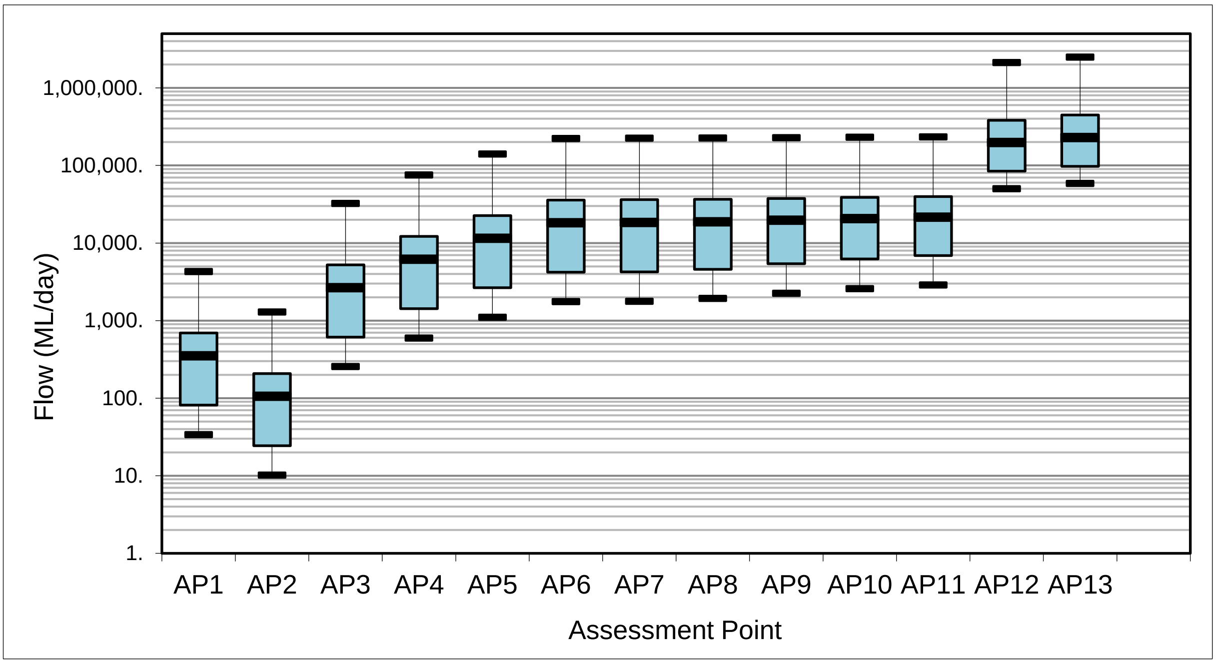

Figure 4-1: Modelled range of flows at the assessment points – pre-embankment scenario

Note: The plot shows mean and 10th/ 90th percentile values (box) along with minimum and maximum (whiskers).

Differences in catchment areas for the APs (Table 4-1) result in lower predicted average flows in the upper catchment sites (AP1, AP2 and AP3), with increasing flows downstream on the Nena-Niar-Frieda-Sepik system (AP6 - AP7 - AP8 - AP9 - AP10 - AP11 - AP12 - AP13: locations shown in Figure 2-2). The modelled flows are typical for systems of the area, with variability between

10th and 90th percentile flows similar with respect to minimum, maximum and average flows through the Frieda River system. The largest increases between sites are from AP4 and AP5 to AP6 due to the confluence of the Nena River and Niar River to form the Frieda River at AP6, and from AP11 to AP12 and AP13 which are in the larger Sepik River.

Table 4-1: Average modelled pre-embankment daily flow at each assessment point

Location	Catchment area (km^2)	Stochastic flow (ML/day)				
		Minimum	10th percentile	Average	90th percentile	Maximum
AP1	20	34	81	353	693	4,287
AP2	6	10	24	106	208	1,286
AP3	151	256	614	2,665	5,232	32,367
AP4	351	594	1,426	6,195	12,161	75,238
AP5	652	1,103	2,649	11,507	22,590	139,758
AP6	1,034	1,750	4,202	18,250	35,826	221,641
AP7	1,047	1,772	4,254	18,479	36,276	224,428
AP8	1,092	1,926	4,579	18,813	36,627	225,357
AP9	1,210	2,238	5,402	19,689	37,565	227,792
AP10	1,345	2,571	6,224	20,691	38,703	230,579
AP11	1,466	2,869	6,887	21,589	39,777	233,076
AP12	25,200	50,072	84,479	197,760	382,964	2,120,000
AP13	29,500	58,385	97,488	229,678	447,530	2,490,000

The results provide a pre-mining estimate of flows and, more importantly the ranges of flows for the modelled period (FRGCP years -2–54). Pre-mining flows were independently assessed at AP6 as part of the hydrological study for the Project presented in SRK (2018), and flows developed from the AWBM were calibrated against those flows (Table 3-3 in Section 3.3).

4.3 Base Case: post-embankment flows

The post-embankment scenario was developed primarily to inform the modelling of solute loads and to assess water levels in the ISF during the operational and closure phases, and to assess the capacity of storages, develop estimates of contact and non-contact water volumes, inform water treatment requirements and assess potential impacts to receiving water bodies. Modelled flow results for the APs for the post-embankment scenario were divided into two periods: an operational period (Model Years 0–35, corresponding to FRGCP operational years -2 to 33) and a closure period (Model Years 36–54; or FRGCP operational years 34 to 56).

During the operational period, flows into the ISF are influenced by mine operations and outflows from the ISF are dominated by hydroelectric power generation operations. For the closure period, the mine voids are modelled to persist and the hydroelectric facility is assumed to be decommissioned, with outflow from the ISF only via the spillway once the ISF capacity is exceeded.

4.3.1 Post-embankment flow – operational period (Model Years 0–35; FRGCP Operational Years -2 to 33)

Results of flows for the operational period are provided in Figure 4-2 and Table 4-2. Note the flows for AP4 and AP5 are not included in the summary table and plot as they are within the ISF impoundment (calculated flows were used only for impoundment water quality assessment).

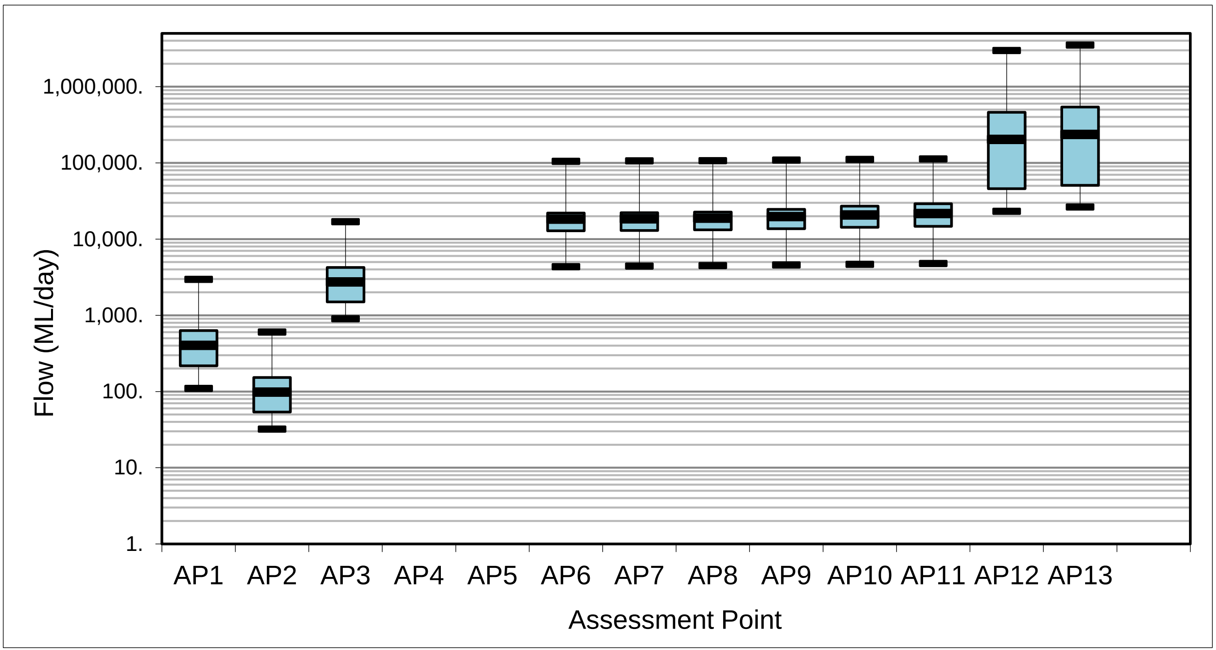

Figure 4-2: Daily average flow statistics at assessment points during FRGCP operations (FRGCP Years 1–33 inclusive)

Note: The plot shows mean and 10th/ 90th percentile values (box) along with minimum and maximum (whiskers).

Table 4-2: Average modelled post-embankment daily flow at each assessment point for the operational period (FRGCP Years 1–33 inclusive)

Location	Stochastic flow (ML/day)				
	Minimum	10th percentile	Average	90th percentile	Maximum
AP1	109	218	403	630	2,951
AP2	32	54	98	153	600
AP3	893	1,500	2,736	4,248	16,766
AP6	4,324	12,817	18,226	21,936	104,198
AP7	4,324	12,817	18,226	21,936	104,198
AP8	4,454	13,219	18,805	22,591	106,314
AP9	4,545	13,709	19,706	24,573	108,168
AP10	4,649	14,258	20,737	27,029	110,289
AP11	4,742	14,743	21,661	29,145	112,189
AP12	22,964	45,894	202,890	461,432	2,965,000
AP13	26,266	50,823	235,724	540,239	3,490,000

Modelled flow results are consistent with expected flow changes, with upstream catchments (AP1, and AP3) showing only minor changes in comparison with the pre-mining scenario, While AP2 shows more impacts due to effects of upstream diversions and increased capture of groundwater from the open-pits (note: treated water discharged upstream of AP1). Flows downstream of the ISF embankment show less variability between minimum and maximum values, and 10th and 90th percentile values in comparison with the pre-embankment scenario. There are also increases in average and 10th percentile flows, which are the result of the regulation of flows due to the ISF and the hydroelectric facility. The effects of the altered flow regime extend to the entire Frieda River system. Modelled flows in the Sepik River AP locations (AP12 and AP13) do not change significantly during operations. Maximum flows appear to have increased in the Sepik River, but this is likely due to a single realisation, and not reflective of the overall system.

Average (mean for all realisations) modelled hydroelectric water demand is provided in Figure 4-3, which includes all water losses from the ISF except for spillway flow. Mean hydroelectric water demand is consistent for periods based on a water demand schedule provided by Robinson (2018) and is the dominant outflow from the ISF during operations.

Water levels in the ISF may not be sufficient to allow for full hydroelectric power production during low precipitation realisations (typically below the 20th percentile), particularly during periods of high demand (~Years 11–33, inclusive).

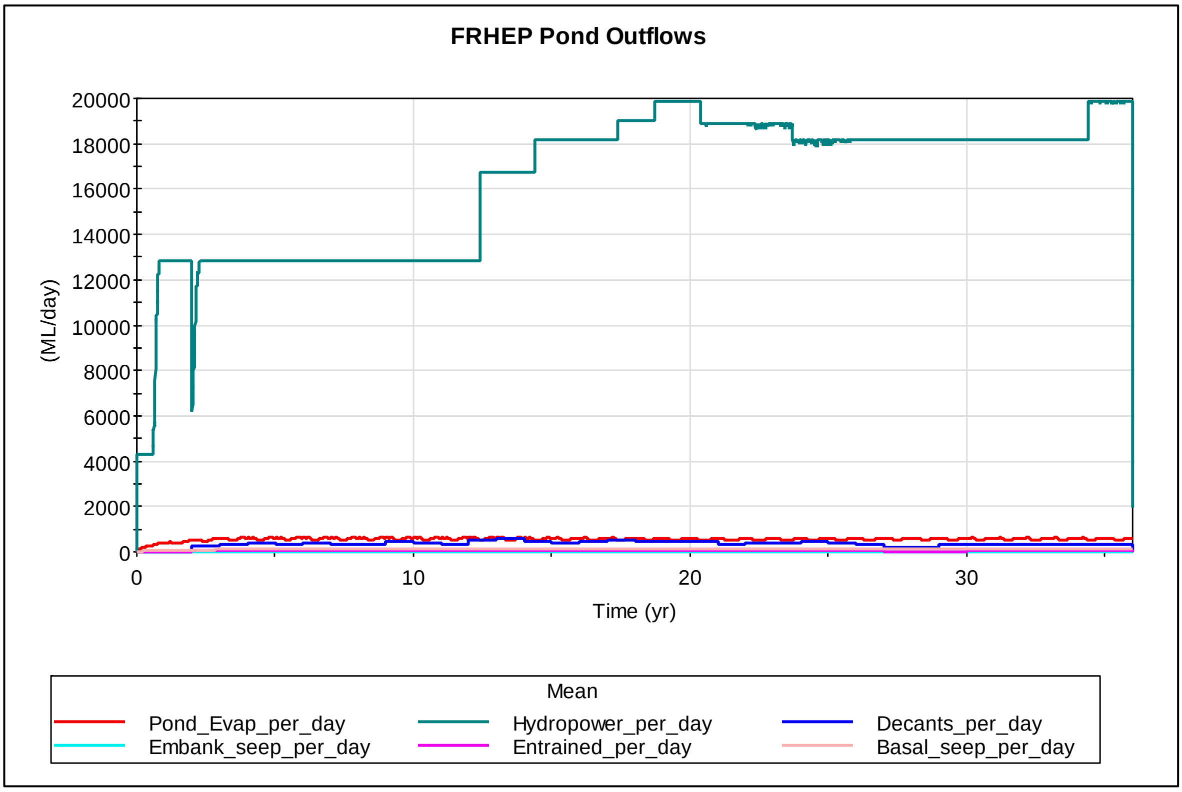

Figure 4-3: ISF outflows – operational period (Model year 0-36, FRGCP year -2 to 33 plus 1 year post FRGCP operations)

Note: Mean flow data for all model realisations.

ISF water levels reach the minimum operating level after approximately 9 months. When capacity in the ISF impoundment is exceeded in the modelled results, flows are conveyed via the spillway to the Frieda River immediately downstream of the embankment. Spillway flows, including the mean, median, 10th and 90th percentile values are provided in Figure 4-4. Flows via the spillway are predicted only during periods of lower hydroelectric water demand and occasionally during higher demand periods in response to wet periods. This is consistent with the simulated results of the ISF outflows (Figure 4-3) and the predicted water levels in the ISF impoundment provided in Figure 4-5.

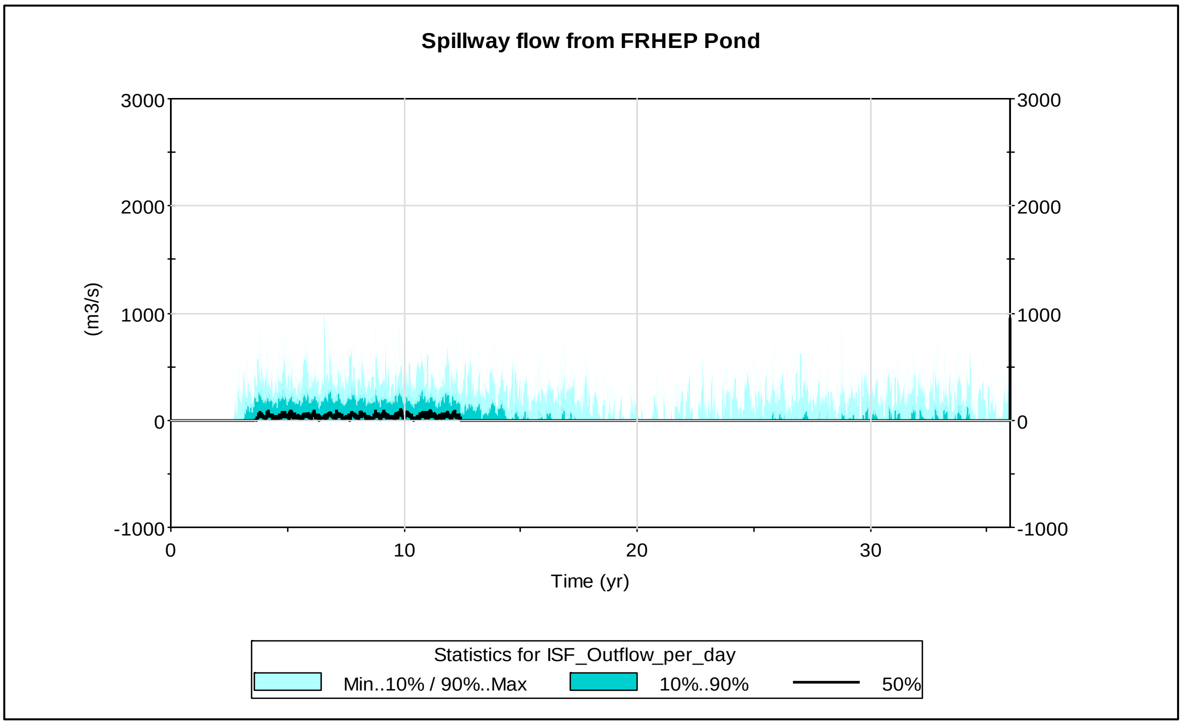

Figure 4-4: ISF spillway flow – operational period (Model year 0-36, FRGCP year -2 to 33 plus 1 year post FRGCP operations)

Note: The plot shows mean (red hatched line), median (black line) and 10th/ 90th percentile ranges of values (blue area).

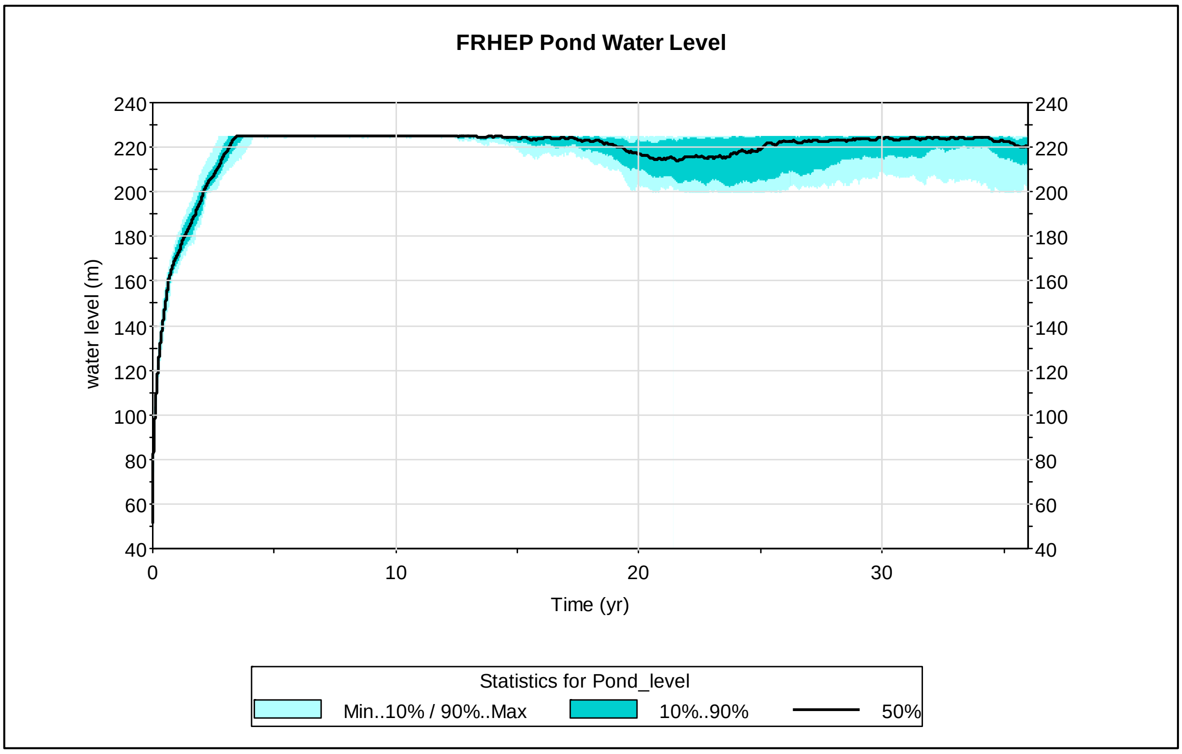

Figure 4-5: ISF water levels – operational period (Model year 0-36, FRGCP year -2 to 33 plus 1 year post FRGCP operations)

Note: The plot shows median (black line) and 10th/ 90th percentile ranges of values (blue area).

Water levels in the ISF (Figure 4-5) are regulated by the spillway invert (upper limit) and partially regulated by the minimum operating water level of the hydroelectric facility (199.4 m RL) although minimum flows of 50 m^3/s are maintained at all times. The model assumes that, after Year 2 (FRGCP year 1) of operations, when water levels fall below the minimum operating level hydroelectric power demand will cease and flows through the embankment reduce to maintain the minimum environmental flows in the Frieda River (50 m^3/s). Hydroelectric power production will recommence once water levels recover above the minimum operating level (199.4 m RL). The results of the stochastic modelling suggest that in the dry (i.e. 10th percentile) rainfall scenario, generation of hydroelectric power will be disrupted due to low water levels in the ISF.

It is understood that the operating assumptions regarding the hydroelectric power facility may be overly conservative because reduced water levels will likely lead to reduced hydroelectric power output and therefore water demand, rather than cessation of power generation altogether. However, the results of the stochastic model suggest that maintaining water levels in the ISF at a sufficient level to meet hydroelectric power demands may pose operational restrictions during extended dry periods.

Tailings and waste rock are proposed to be co-disposed within the ISF, and sediment load estimates were provided by Golder (2018). A combined schedule for waste rock, tailings and sediment loads ('combined solids') is provided in SRK (2018).

The schedule of waste rock, tailings and sediment were incorporated into the water balance model. The total depth of water over the combined tailings, waste and sediment solids was calculated and presented in Figure 4-6 and Figure 4-7. The model predicts significant depths of water above the surface of the accumulated solids for mean, median, 10th and 90th percentile rainfall results. The cumulative combined solids deposition relative to the mean ISF water levels are shown in Figure 4-7.

4.3.2 Mine water management

Mean daily pit sump inflows are presented in Figure 4-8. Pit sump inflows are dominated by the area of footprint of the pits and the diversions as outlined in SRK (2018) and in Table 3-4. For the operational period, it is assumed that all water from the pits will be pumped to the water treatment plant (WTP) for treatment for each time step (i.e. does not consider storage in the sumps). Average pumping requirements are approximately 65 ML/day (750 L/s) and 6.3 ML/day (70 L/s) for the HIT/Ekwai and Koki pits, respectively. If there is a requirement to maintain minimal volumes of water in the sumps, the maximum pumping requirements would be 123 ML/day (1,425 L/s) and 9.2 ML/day (106 L/s) for the HIT/Ekwai and Koki pits, respectively.

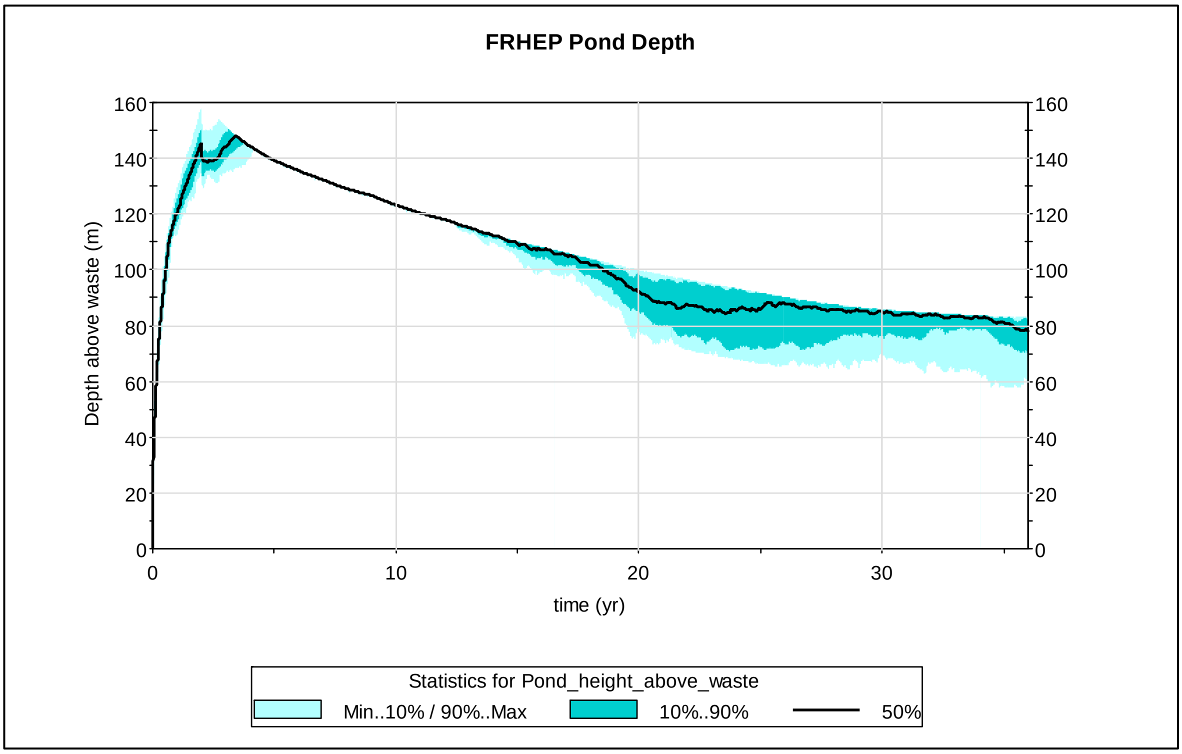

Figure 4-6: Modelled ISF water depth (above solids) – operational period (Model year 0-36, FRGCP year -2 to 33 plus 1 year post FRGCP operations)

Note: The plot shows mean (red hatched line), median (black line) and 10th/ 90th percentile ranges of values (blue area).

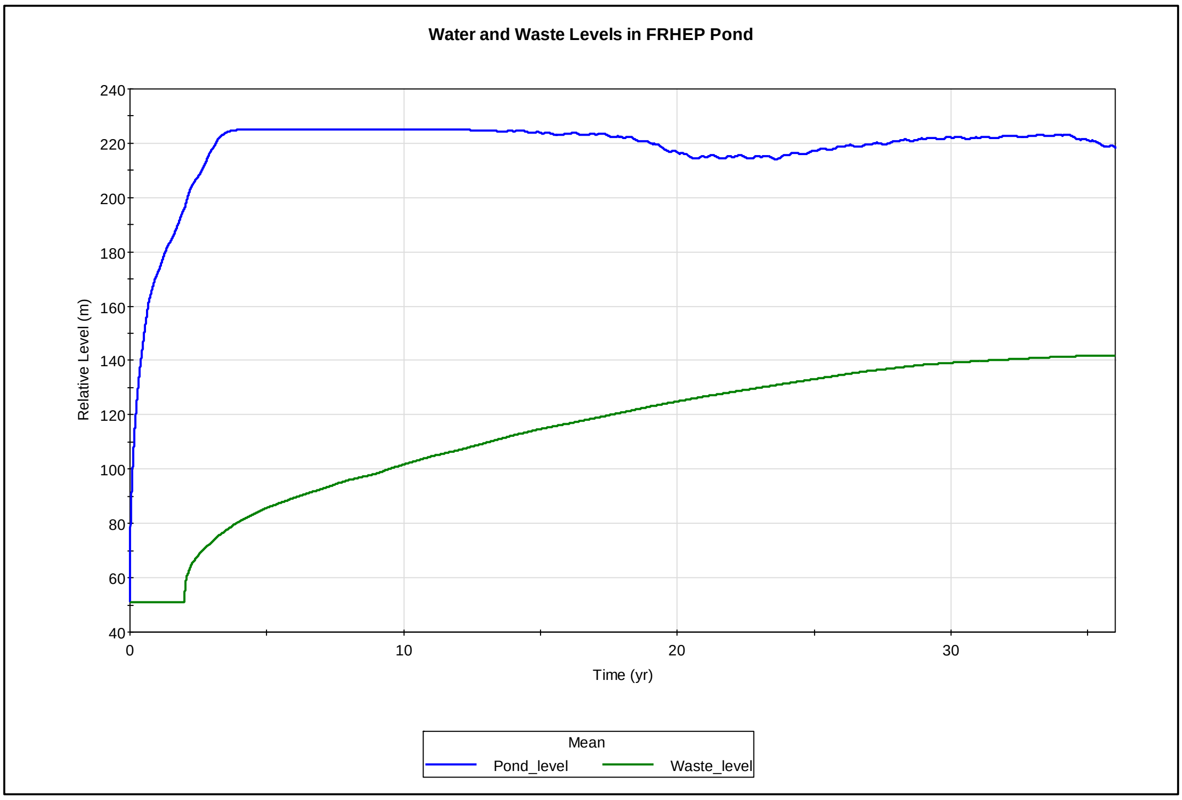

Figure 4-7: Modelled ISF level ranges – operational period (Model year 0-36, FRGCP year -2 to 33 plus 1 year post FRGCP operations)

Note: The plot shows mean water levels in the ISF (blue) and combined solids (green). Maximum level for the ISF impoundment is 225 m RL.

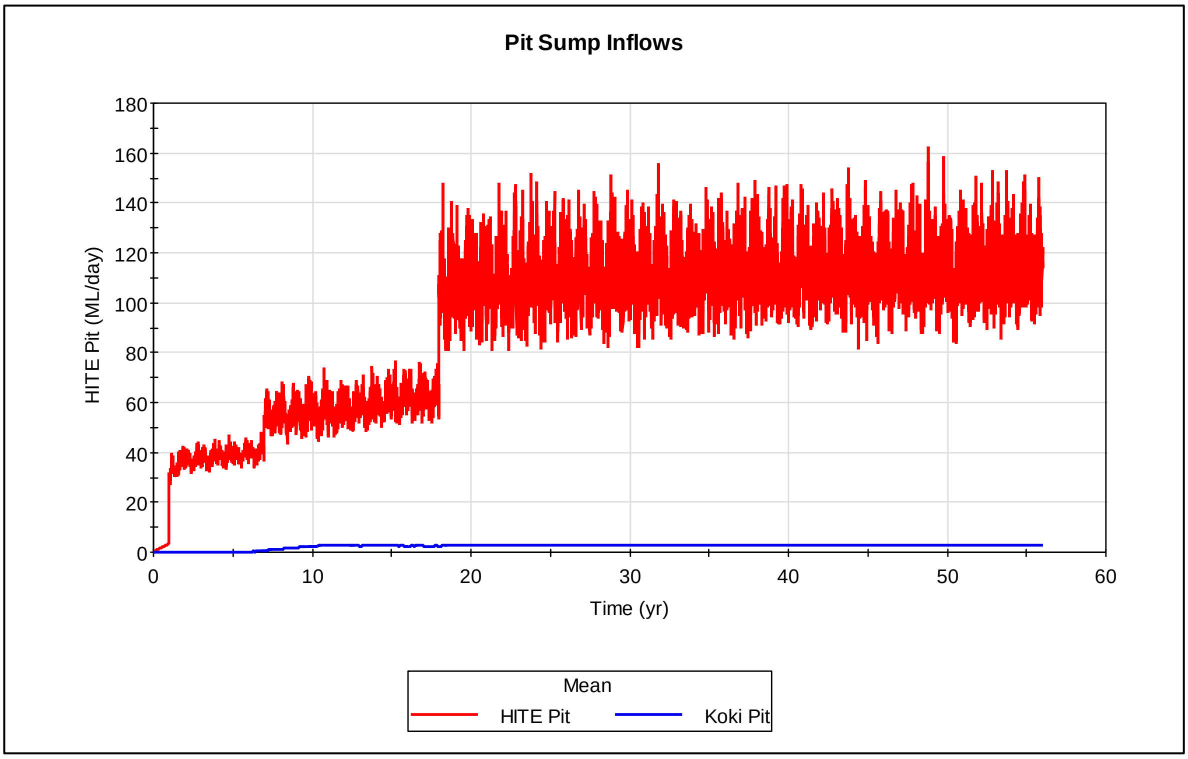

Figure 4-8: Pit sump inflows for the HIT/Ekwai and Koki pits

Note: The plot shows mean total inflows into the HIT/Ekawi (SP1_HIT_per_day) and Koki (SP3_Koki_per_day) pit sumps.

4.4 Closure period (Model Year 37-56, FRGCP Year 34–54)

Water balance results for the closure period were developed to assess the impacts of the project on flows after cessation of mining activities. A preliminary assessment was completed to determine the length of time for the HIT/Ekwai pit to be inundated on termination of dewatering activates. Three additional scenarios were developed to inform potential closure options for the Project, including:

- Scenario 1: Ongoing Hydropower – whereby hydropower is continued after mining beyond the modelled timeframe
- Scenario 2: No Hydropower – whereby all hydropower operations cease, hydroelectric spillways are decommissioned and all flow from the ISF impoundment is through the closure spillway
- Scenario 3: Maintained Environmental Flows – whereby all hydropower operations cease, and minimum environmental flows of 50 m^3/s are maintained from the ISF facility.

4.4.1 Pit inundation

Upon cessation of mining activities, the combined HIT and Ekwai pit will be inundated, and pumping will be stopped until a designated fill level is achieved and active treatment of discharged water will be recommenced (the Koki pit will already be inundated and excess water will be pumped to the HIT/ Ekwai pit rather than to the treatment plant.)

Pumping from the HIT/ Ekwai was terminated at the end of model year 36 (Project year 33) and the pit allowed to inundate whilst the diversions were maintained. The results are presented in Figure 4-9. The water balance model indicates that the pit will be flooded within 10 years (mean of all model realisations) after cessation of dewatering activities. An alternative scenario was developed whereby water in diversions around the pit were routed to the pit and indicate that the time to inundate the pit would be 3 years.

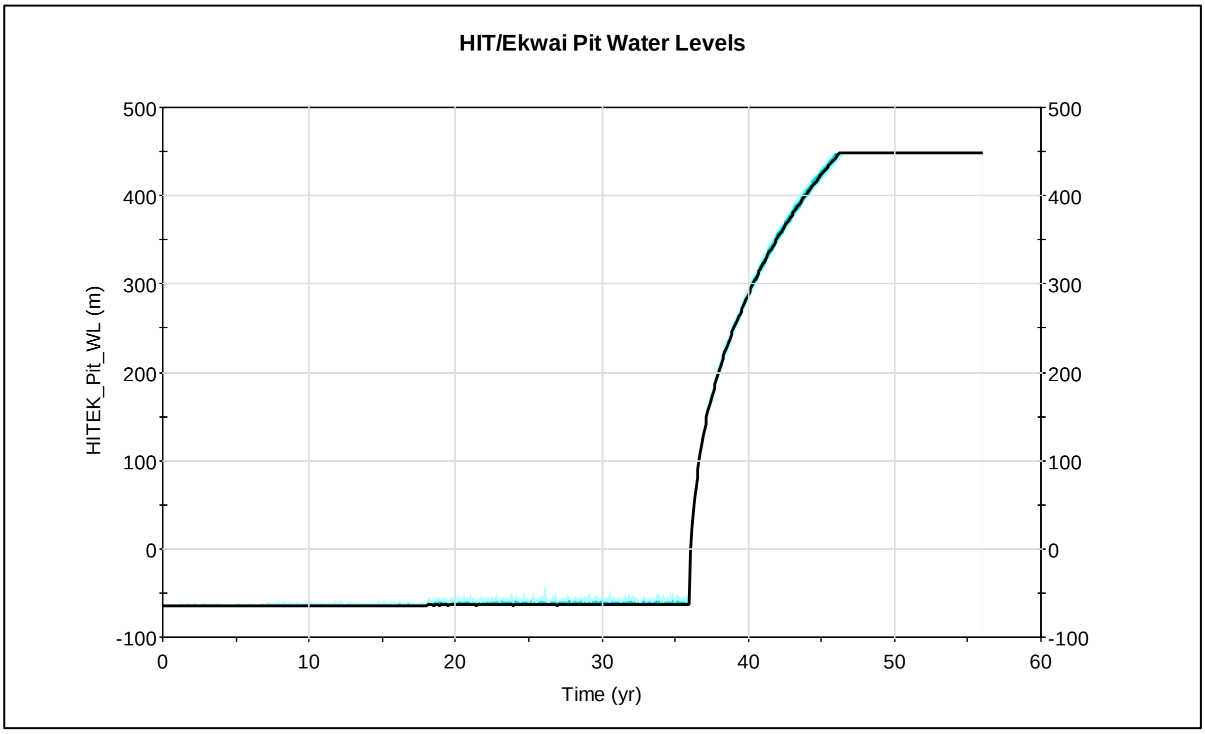

Figure 4-9: HIT/ Ekwai pit inundation post closure – diversions maintained

Note: The plot shows mean, 10th/ 90th percentile values (dark blue area) along with minimum and maximum (light blue).

4.4.2 Closure Scenario 1: ongoing hydropower

Results of flows for the closure period for Scenario 1 are provided are provided in Figure 4-10 and Table 4-5. As noted before flows for AP4 and AP5 are for reference and are not included in the results. For this scenario, hydroelectric water demand was assumed to be maintained at 210 m^3/s for the duration of the modelling period.

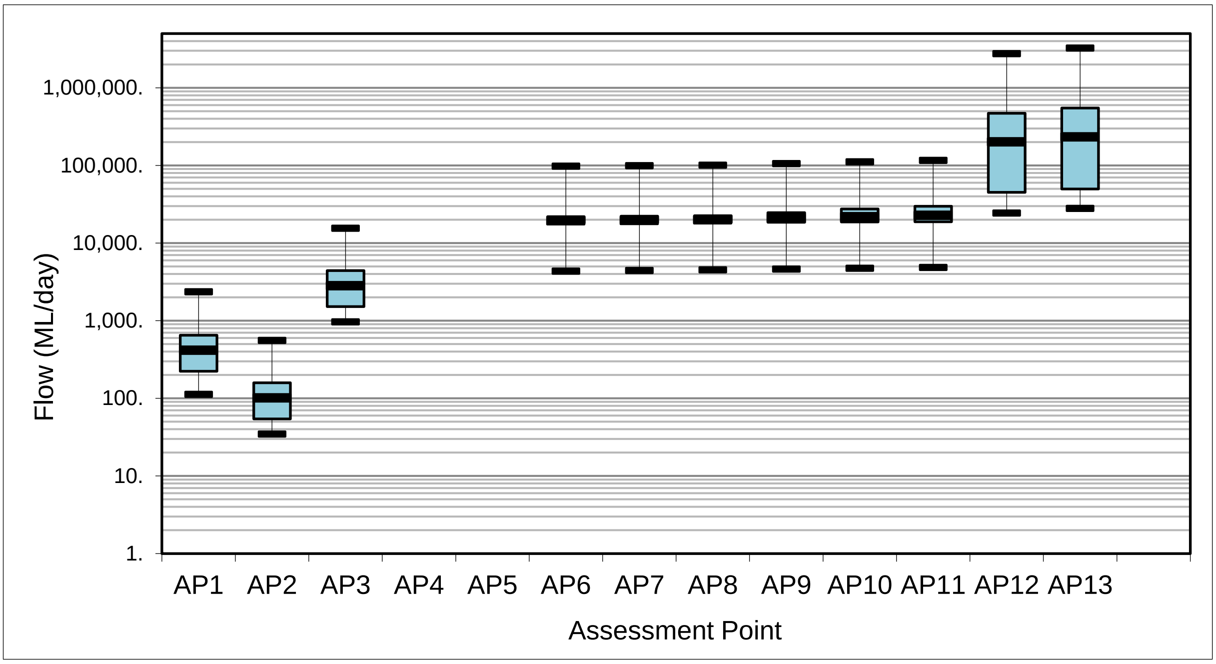

Figure 4-10: Daily average flow statistics at assessment points post closure with hydroelectric power operations maintained (FRGCP Year 34–54)

Note: The plot shows mean and 10th/ 90th percentile values (box) along with minimum and maximum (whiskers).

Table 4-3: Average modelled post-embankment daily flow at each assessment point for the closure period with Hydroelectric power operations maintained (FRGCP years 34–54)

Location	Stochastic flow (ML/day)				
	Minimum	10th percentile	Average	90th percentile	Maximum
AP1	112	223	417	650	2,343
AP2	34	54	101	158	554
AP3	963	1,519	2,827	4,427	15,483
AP6	4,324	18,148	19,386	21,866	97,582
AP7	4,422	18,278	19,629	22,159	98,683
AP8	4,497	18,384	19,968	22,517	100,473
AP9	4,608	18,543	20,856	24,707	105,166
AP10	4,719	18,705	21,873	27,464	110,535
AP11	4,818	18,845	22,784	29,754	115,348
AP12	24,288	45,040	201,493	469,141	2,745,000
AP13	27,815	49,731	233,870	549,086	3,230,000

Results for the post FRCGP closure period are consistent with expected flow changes, with upstream catchments (AP1, AP2 and AP3) showing little change, and flows in the Frieda River (AP6 to Ap11) similar to the operational period. The 10th and 90th percentile flows downstream of the ISF embankment show less variability than the operational period due to the lack of variability in hydroelectric water demand. Similar to the changes in flow during operations, the effects of the altered flow regime extend to the entire Frieda River system. Average and maximum flows within the Sepik River AP locations (AP12 and AP13) align well with operational values.

4.4.3 Closure Scenario 2: no hydropower

Scenario 2 was modelled assuming that hydroelectric operations were terminated upon closure, and that all flows are directed through the closure spillway (Invert at 210 m RL). Results of flows for the closure period for Scenario 2 are provided in Figure 4-11 and Table 4-4.

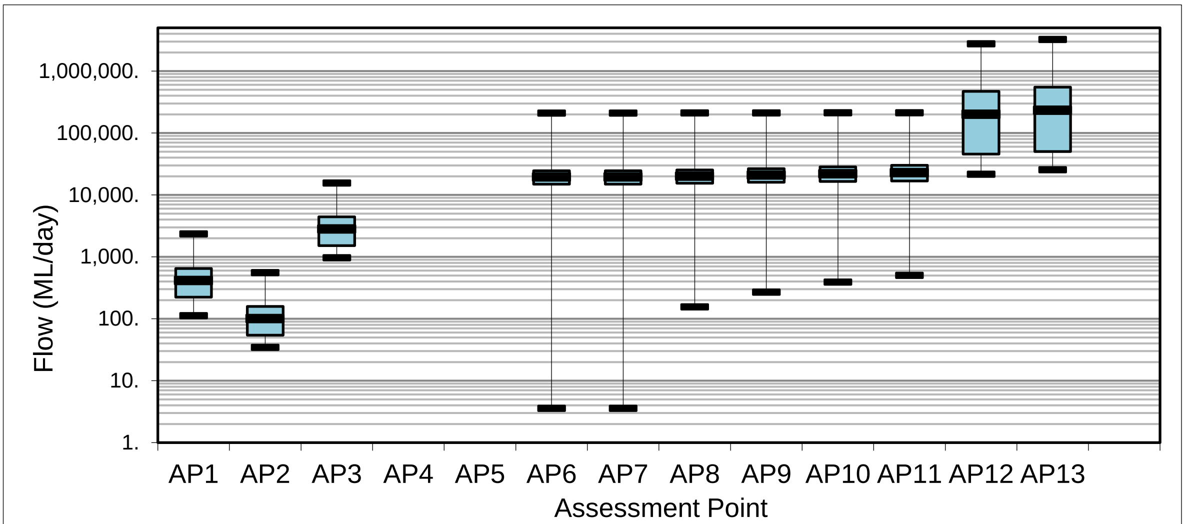

Figure 4-11: Daily average flow statistics at assessment points post closure with hydroelectric power terminated and all flow through the closure spillway (FRGCP years 34–54)

Note: The plot shows mean and 10th/ 90th percentile values (box) along with minimum and maximum (whiskers).

Table 4-4: Average modelled post-embankment daily flow at each assessment point for the closure period with hydroelectric power terminated and all flow through the closure spillway (FRGCP years 36–56 inclusive)

Location	Stochastic flow (ML/day)				
	Minimum	10th percentile	Average	90th percentile	Maximum
AP1	112	223	417	650	2,343
AP2	34	54	101	159	554
AP3	963	1,518	2,827	4,433	15,483
AP6	4	14,939	19,539	24,600	209,034
AP7	4	14,939	19,539	24,600	209,034
AP8	155	15,424	20,121	25,278	209,469
AP9	268	16,002	21,009	26,525	210,091
AP10	390	16,486	22,026	28,286	210,803
AP11	500	16,841	22,937	30,027	211,440
AP12	21,571	45,715	201,646	469,797	2,739,000
AP13	25,387	50,354	234,023	549,843	3,222,000

Results for the closure period are consistent with expected flow changes, with upstream catchments (AP1, AP2 and AP3) showing little change. Flows in the Frieda River (AP6 to AP11) show similar average conditions with those of the pre-embankment and operational base cases; however, there is more variability than those in the operational period due to the lack of any active regulation of flows. The range between the minimum and maximum flows, as well as the 10th and 90th percentile flows downstream of the ISF embankment are much larger. Most importantly, minimum flows are significantly reduced in comparison with the base cases and other scenarios. Analysis of the raw model data (i.e. all realisations) indicates that the probability of any daily flow being under the environmental minimum of 50 m^3/s is 0.025%, and that the maximum period where flows were below the threshold was 70 days (as part of the 1st percentile – i.e. extreme dry – realisation).

Similar to the changes in flow during operations, the effects of the altered flow regime extend to the entire Frieda River system.

4.4.4 Closure Scenario 3: maintained environmental flows

Scenario 3 was modelled similar to Scenario 2 but that environmental flows would be maintained post closure. Results of flows for the closure period for Scenario 3 are provided are provided in Figure 4-12 and Table 4-5.

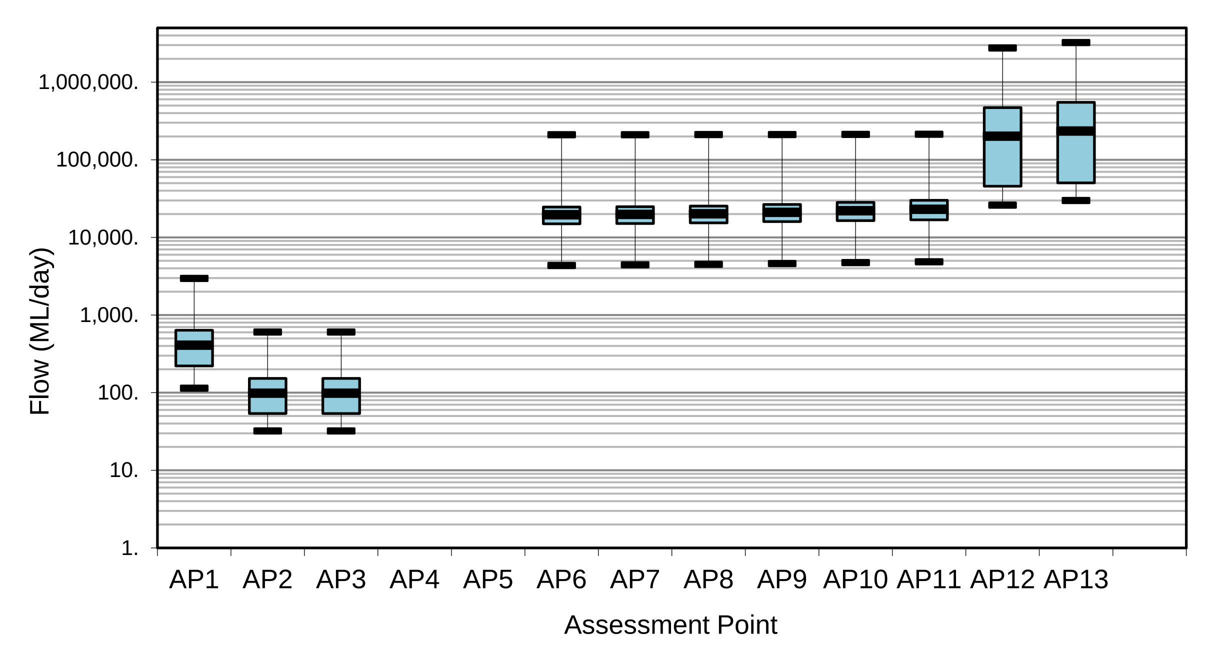

Figure 4-12: Daily average flow statistics at assessment points post closure with hydroelectric power terminated and environmental flows maintained (FRGCP years 34–54)

Note: The plot shows mean and 10th/ 90th percentile values (box) along with minimum and maximum (whiskers).

Table 4-5: Average modelled post-embankment daily flow at each assessment point for the closure period with hydroelectric power terminated and environmental flows maintained (FRGCP years 34–54)

Location	Stochastic flow (ML/day)				
	Minimum	10th percentile	Average	90th percentile	Maximum
AP1	78	170	425	764	3,405
AP2	34	54	101	159	554
AP3	34	54	101	159	554
AP6	4,324	14,937	19,553	24,621	209,267
AP7	4,416	15,126	19,796	24,926	209,464
AP8	4,476	15,421	20,135	25,300	209,705
AP9	4,588	15,999	21,024	26,550	210,338
AP10	4,711	16,484	22,040	28,309	211,063
AP11	4,820	16,839	22,951	30,050	211,712
AP12	25,891	45,725	201,660	469,819	2,739,000
AP13	29,707	50,362	234,038	549,849	3,223,000

Modelled flow results for the closure period are similar to those of Scenario 2, with the exception that minimum flows are maintained at 50 m^3/s (4,320 ML/day) downstream of the ISF at AP6.

5 Conclusions and Limitations

Results of the water balance modelling indicate the following:

- Predicted flow changes within upstream catchments (AP1, AP2 and AP3) are consistent with the diversions that will be implemented and groundwater that will be captured in the open-pits during the FRCGP mine operations and post-closure period.
- Average pit sump pumping requirements are approximately 65 ML/day (750 L/s) and 6.3 ML/day (70 L/s) for the combined HIT and Ekwai pit, and the Koki pit respectively. If there is a requirement to maintain minimal volumes of water in the sumps, the maximum pumping requirements would be 123 ML/day (1,425 L/s) and 9.2 ML/day (106 L/s) for the HIT/ Ekwai and Koki pits respectively.
- During operations, flows downstream of the ISF embankment show less variability between minimum and maximum values, and between 10th and 90th percentile values due to the regulation of flows from ISF. The altered flow regime extends to the entire Frieda River system, with no significant changes to the flow regime of the Sepik River.
- Water levels in the ISF may not be sufficient to support the planned hydroelectric power production during extended dry periods, with interruptions occurring in low rainfall realisations (typically below the 10th percentile) particularly during periods of high water demand for hydropower generation.
- The model predicts that complete inundation of the HIT/ Ekwai pit will be achieved after 10 years once dewatering has been terminated, and 3 years if all diverted flows are routed into the open-pit.
- During the post-closure period, three scenarios all indicate some changes to flows within the Frieda River system, with no significant changes in flow within the Sepik River system compared to baseline conditions. Under a scenario where hydropower is terminated post closure, minimum flows in the Frieda River may be lower than the target environmental flow rate of 50 $m^{3/}s$ (4,320 ML/day) specified for the Frieda River at AP6 downstream of the embankment during operations. Analysis of the results indicates that the probability of any daily flow being under the environmental minimum of 50 m^3/s is 0.025% for this scenario, and that the maximum period where flows could be below the threshold was 70 days (as part of the 1st percentile – i.e. extreme dry – realisation).

The water balance model has been developed using generally conservative methods and assumptions. Remaining model limitations or uncertainties that have been identified as potentially significant include:

- Conservative assumptions have been incorporated into the model, however, the limited long-term precipitation and flow data result in uncertainty in the understanding of low and high flow conditions.
- Flow conditions for the natural catchment areas have been estimated using the AWBM, which is designed for long-term water balance purposes and is suitable for assessing the effects on the hydrologic system, however, may not represent low and high flow conditions.
- Results for a number of outflows for the ISF are dependent on assumptions regarding operations for the site and, in some cases, information provided by third parties. These assumptions may not reflect the actual operation of the ISF.

Project Number: PNA009

Report Title: Frieda River Project – Water Balance

Compiled by

Brian Luinstra

Principal Hydrogeologist

Peer Reviewed by

John Chapman

Principal Consultant

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Appendices

Appendix A: FRHEP impoundment basal seepage assessment

Level 1, 10 Richardson Street
West Perth WA 6005, Australia

PO Box 943
West Perth WA 6872, Australia

T: +61 8 9288 2000
F: +61 8 9288 2001
E: perth@srk.com.au

www.asia-pacific.srk.com

Project Memo

Client:	PanAust Limited	**Date:**	17 May 2018
Attention:	Edward Chong	**From:**	Zip Boniecki
Project No:	PNA009	**Revision No:**	0
Project Name:	Frieda River		
Subject:	FRHEP impoundment basal seepage assessment		

1 Estimated seepage for the Frieda River Hydroelectric Project

PanAust, as part of a Selection Phase Study (SPS) for the Frieda River Hydroelectric Project (FRHEP), is proposing to construct an integrated tailings storage and hydroelectric facility at Frieda River. This memorandum has been prepared to provide a range of seepage estimates for the reservoir area for incorporation into the project water balance and energy model. Seepage estimates do not include any seepage at the embankment of the reservoir as this is included in a separate analysis by SRK Consulting (Australasia) Pty Ltd.

Once the diversion tunnels are blocked, the water level in the reservoir is expected to rise at the rate reflected in Figure 1-1.

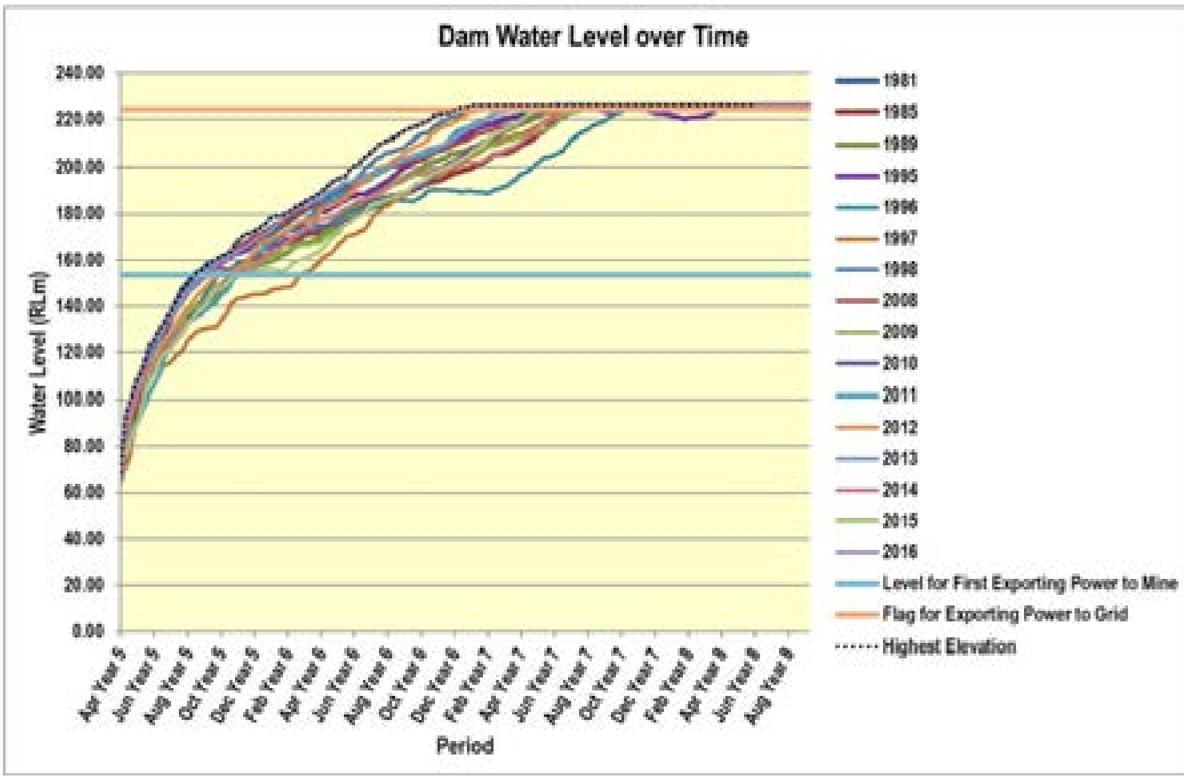

Figure 1-1: Reservoir water level

1.1 Regional Geology

The FRHEP is located on the northern part of the Indo-Australian plate, situated on the north flank of the Central Highlands.

The FRHEP site is situated in an area dominated by three major WNW–ESE to northwest–southeast northward dipping thrust faults (Figure 1-2). These faults are splays of the Leonard Schulze Fault, which lies about 100 km to the east. The major thrust splays, from south to north, are the Fiak, Frieda and Saniap faults. The Saniap Fault forms a boundary between rocks of the Wogamush Formation to the north and OK Binai Phyllites to the south. In 2015, a Mw 4.2 earthquake occurred at a depth of 43 km on the Saniap Fault, approximately 13 km to the southeast of the FRHEP site.

The oldest rocks in the area are the Jurassic- to middle Eocene-aged OK Binai Phyllite, which grade into the equivalent of the Wabia beds and Wahagi Group slate. The sequences comprise phyllitic mudstone, sandstone and volcanolithic rocks. The overlying Wogamush Formation consists of volcanogenic sequences and forms part of the late Oligocene to Miocene Maramuni Igneous Complex. The sequences consist of andesite to basaltic volcanics, volcanolithic sandstone, mudstone and limestone and have, in places, been intruded by numerous plutons.

Major slices of April Ophiolites have been thrust over the OK Binai and Wogamush sequences. The April Ophiolite is of Palaeogene age and consists of undifferentiated ultrabasic igneous rocks of basalt, gabbro and peridotite. These rocks represent the erosional remnants of a more extensive thrust sheet of oceanic crust. They are typically weathered, variously serpentinised and comprise layered to massive cumulate dunite (bedrock at FRHEP site), harzburgite and wehrlite (upper mantle).

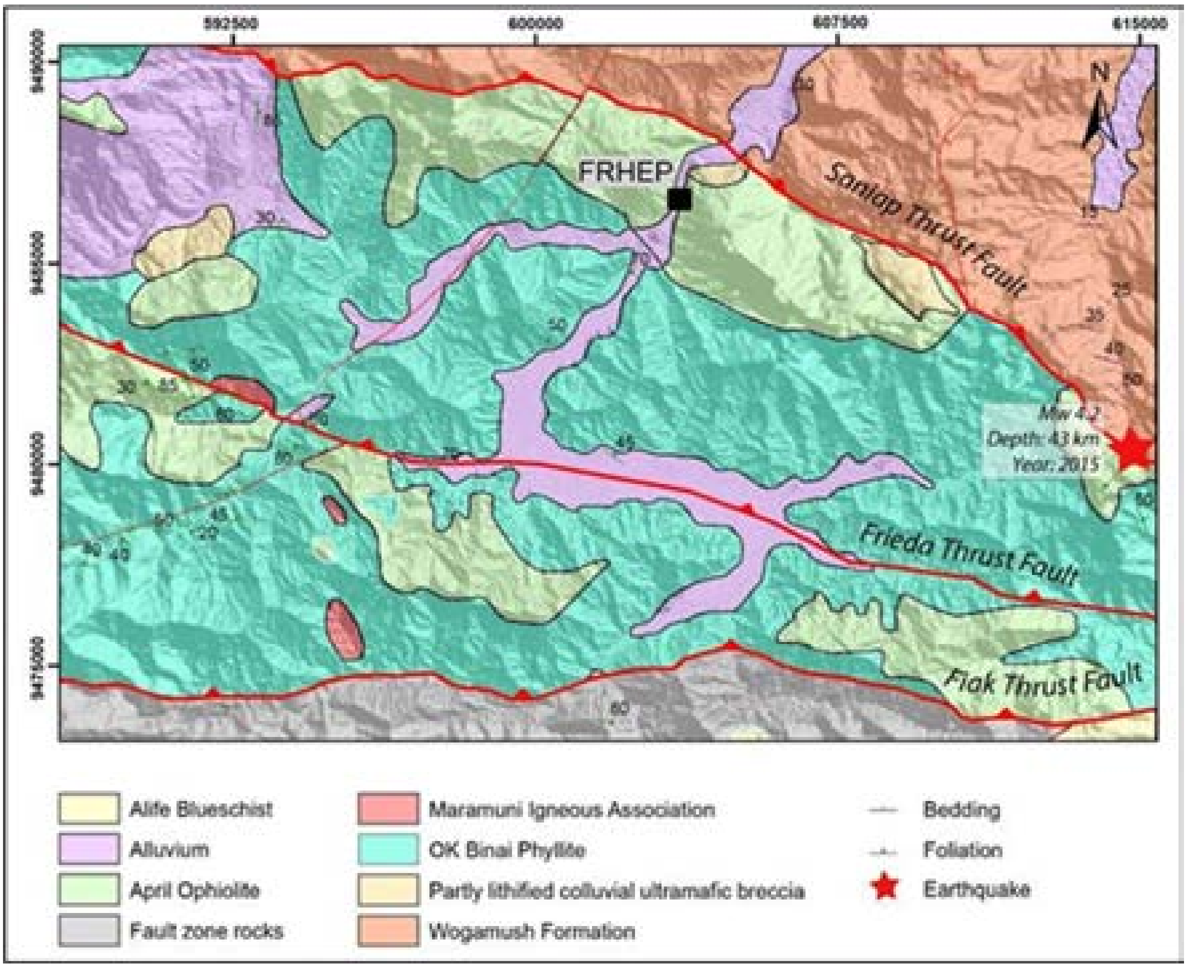

Figure 1-2: Geological setting of the FRHEP site and surrounds

1.2 Methodology

Groundwater seepage from the reservoir area during filling

Seepage was estimated using a relatively simple analytical method with a Darcy calculation, based on a range of hydraulic conductivity values and assumed groundwater levels to estimate groundwater gradients into the surrounding mountain ranges.

$$Q = K\frac{dh}{dx}A$$

Where:

Q = total seepage into the reservoir walls in m^3/s

K = the assumed hydraulic conductivity in m/s

$\frac{dh}{dx}$ = the assumed hydraulic gradient at a given reservoir water level; without numerical modelling this is difficult to predict as it will vary temporally and with different reservoir filling rates

A = area of the seepage face.

As the groundwater level at the groundwater divide is generally higher than the maximum level of the proposed reservoir, it is assumed that seepage into the rock walls will decrease over time as a new water equilibrium is reached; as such, the seepage estimates presented are considered to be estimates of peak seepage losses during filling.

A peak outflow was calculated for increase of reservoir water level at 50 m intervals. For each increase, only the area within the wall rock that would remain unsaturated from the previous calculation was considered (i.e. it was assumed that groundwater levels would stabilise and seepage from the previous interval would be negligible prior to filling to the next level). This may not be valid in areas close to the embankment or faulted zones as there could be a constant loss (covered in the next section); however, this is considered an acceptable estimate for the overall reservoir.

Groundwater seepage from fault zones and embankment valley

A constant seepage through the mapped fault zones through the eastern range separating the Frieda and Waria rivers was estimated using assumed values for hydraulic conductivities, and assumed depths and widths of the fault zone. As the range dividing the two valleys is less than 1,000 m across, with a height of only 330 m RL, it is likely that seepage in this area will commence after reservoir levels surpass 100–150 m RL; depending on the groundwater levels within the range.

Constant seepage through faults crosscutting the north range and along the western range separating the Frieda and May rivers has also been estimated, although this is highly speculative due to a lack of groundwater level data and the high topography; if groundwater levels are higher than reservoir levels in the fault zones, there will be no loss to groundwater through these zones.

Seepage through the weathered zone of the north valley walls and base (where the dam is to be placed) was also estimated. Seepage through the actual embankment has already been calculated (SRK, 2018) and was not estimated as part of this analysis.

Seepage through faults and the northern valley are likely to be the only constant and significant non-recoverable losses to groundwater system.

1.3 Hydrogeological conceptualisation

As hydrogeological data available for the Frieda River valley is highly localised, maximum and minimum seepage rates have been estimated. Maximum seepage rates are based on higher hydraulic conductivities, seepage through the weathered zone in the northern valley, no groundwater divide between Waria and Frieda river valleys at fault zones and seepage at the northern fault zones when reservoir levels surpass 200 m, and seepage at the western fault zone when reservoir levels surpass 150 m. Minimum seepage rates are based on lower hydraulic conductivities, a groundwater divide (groundwater levels at 150 m RL) between the Waria and Frieda river valleys in fault zones and no seepage from the northern range and western fault zone. Assumed values are shown in Table 1-1.

Table 1-1: Maximum seepage assumed values

Assumed values		Seepage into weathered zone	Seepage through embankment valley	Seepage into Waria River valley	Seepage along northern faults	Seepage along western faults
K (m/s)	Max.	5×10^{-7}	5×10^{-7}	1×10^{-5}	1×10^{-5}	1×10^{-5}
	Min.	1×10^{-7}	–		–	–
Gradient	Max.	0.1	(Head difference between Reservoir levels and groundwater divide or discharge area)/ (distance to groundwater divide or discharge area)			
	Min.		–	Same as above except with a groundwater level of 150 m at the groundwater divide	–	–
Area (m^2)		Calculated from topography	6000 + (increase in saturated thickness of 15 m of weathered rock either side of the valley)-	(Reservoir head + 50 m of below ground faulting) x (assumed fault zone width; 800 m to Waria River valley, 200 m otherwise)		

The conceptualisation is based on the following assumptions:

- It is assumed that water will enter the reservoir gradually and therefore a groundwater gradient will always be present (leakage will not be vertical as the ground beneath the reservoir should always be saturated).
- Where possible, hydraulic conductivities were estimated from packer testing data (provided in AGE, 2015) and from more recent work conducted by SRK). As packer tests are localised to three locations (Figure 1-3), it has been necessary to extrapolate hydraulic conductivities to the larger reservoir area, which may not be representative of the local hydrogeological conditions.
- Water levels are also based on a spatially limited monitoring network (Figure 1-3) and extrapolated to reflect topography. While this is likely to be a valid assumption, high permeability zones (such as faulted areas) may have a large influence on local water levels and the flow regimes. As such, drilling and subsequent packer tests or test pumping as well as groundwater level monitoring is recommended in faulted areas to determine their actual influence on the overall groundwater flow regime and to improve seepage estimates.

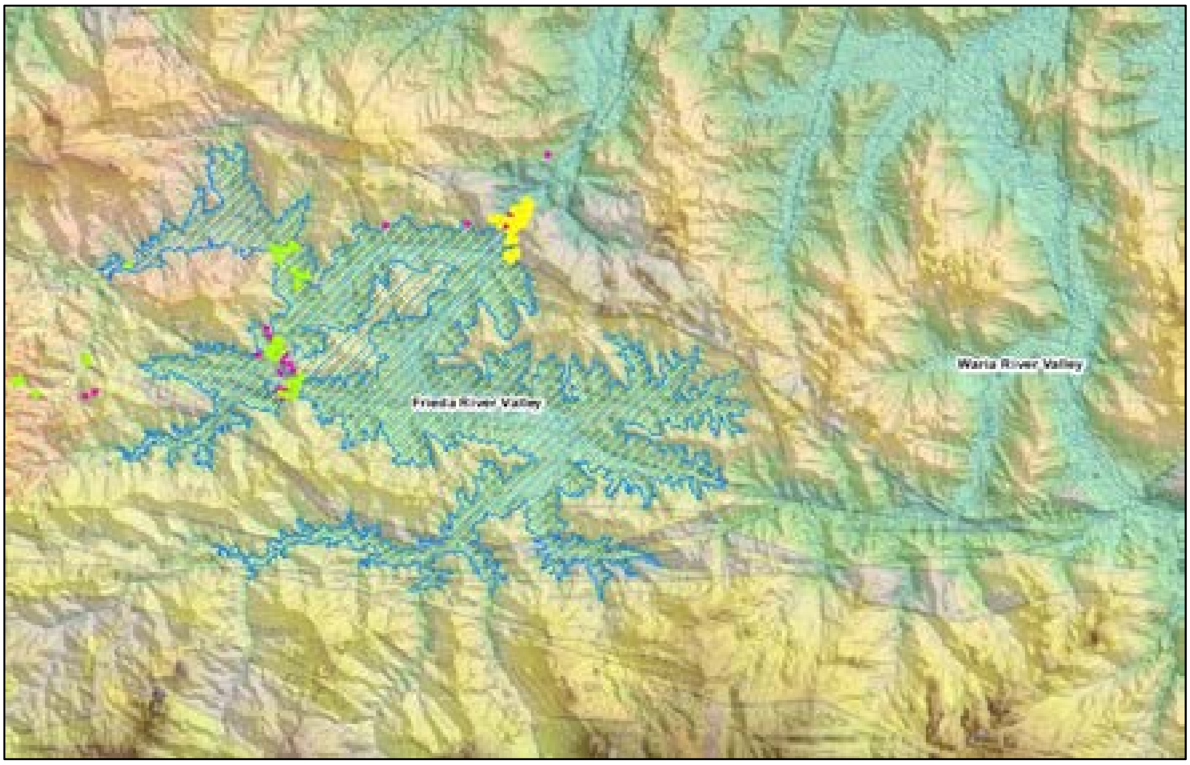

Figure 1-3: Spatial distribution of available hydraulic conductivity data from AGE (2015) (in green) and SRK (in yellow), and water level data from AGE (2015) (in pink)

Source: AGE, 2015.

- Seepage in the weathered rock around the embankment will occur as a result of increased head within the reservoir causing an increase in the gradient and saturated thickness of weathered rock, thereby increasing groundwater flow.
- Constant seepage losses to faults along the eastern range have been estimated assuming that groundwater levels follow topography in the Waria River valley and groundwater discharges to the Waria River. For the purposes of developing seepage estimates, the weathered zone area of the faulted zone is assumed to be 800 m wide and extend to 50 m RL (below the valley); an area proportional to head increase in the reservoir is added at each interval (Figure 1-4). Bulk hydraulic conductivities have been assumed as 1×10^{-5} m/s, extrapolated from packer testing at a distance of 19 km. The groundwater gradient is calculated based on reservoir level difference and distance to 100 m RL elevation in the Waria River valley.
- Constant seepage losses to faults along the northern and western ranges are not possible to predict accurately without further data, and so assumptions have been made to establish hydraulic conductivities and groundwater levels at faulted areas. For the purposes of developing seepage estimates, hydraulic conductivities have been assumed as 1×10^{-5} m/s and the fault zone is assumed to be 200 m wide and extends to 50 m RL. Furthermore, for the purposes of developing worst case scenario seepage estimates, it has been assumed that:
  - Faults continue past the northern and western ranges and act as a high flow conduit to discharge water from the system into downstream aquifers or to surface water via rivers and/ or springs
  - Faults have sufficiently high hydraulic conductivities and thicknesses to cause lowering of groundwater levels to 200 m RL in the northern range and 150 m RL in the western range.
- The thickness of the weathered zone is assumed to be 20–50 m thick based on drill hole data (AGE, 2015).

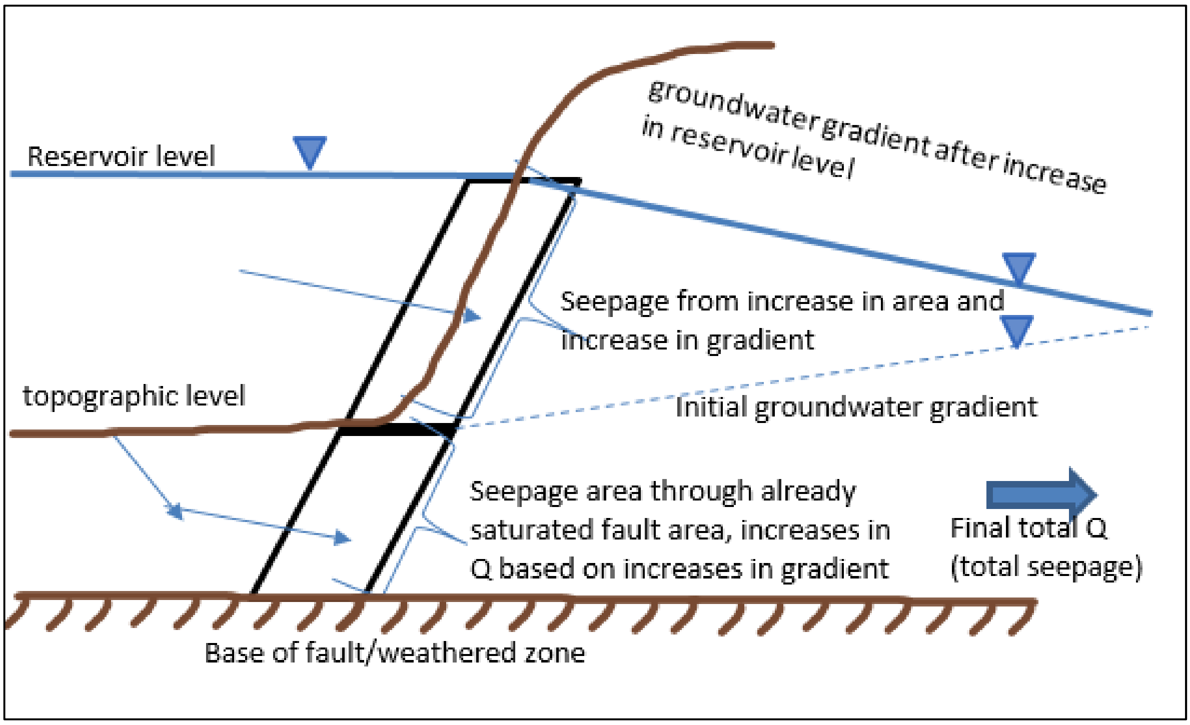

Figure 1-4: Conceptualisation of seepage through faults along eastern fault zone

1.4 Results

Based on the above methodology, total seepage estimates (excluding seepage through the embankment) range between 0.63 m^3/s and 2.18 m^3/s at full reservoir capacity of 250 m RL. Estimated seepage values for each interval are presented in Table 1-2 for maximum rates and Table 1-3 for minimum rates.

Table 1-2: Maximum seepage estimates

Reservoir level (m RL)	Seepage into weathered zone	Seepage through embankment valley	Seepage into Waria River valley	Seepage along northern faults	Seepage along western faults	Total seepage
	m^3/s					
100	0.12	0.0005	–	–	–	0.1254
150	1.83	0.00135	0.01	–	–	1.8355
200	1.98	0.00245	0.03	0.000	0.001	2.0130
250	2.12	0.0038	0.05	0.003	0.002	2.1818

Table 1-3: Minimum seepage estimates

Reservoir level (m RL)	Seepage into weathered zone	Seepage through embankment valley	Seepage into Waria River valley	Seepage along northern faults	Seepage along western faults	Total seepage
	m^3/s					
100	0.02	–	–	–	–	0.025
150	0.42	–	–	–	–	0.417
200	0.51	–	0.01	–	–	0.518
250	0.61	–	0.04	–	–	0.628

2 Limitations

The above seepage estimates have the following limitations:

- The lack of data around depths, thicknesses and hydraulic conductivities of fault/ weathered zones, particularly between the Frieda and Waria river valleys is a limitation for the accuracy of seepage estimates.
- The lack of data with respect to groundwater levels at fault/ weathered zones, particularly between the Frieda and Waria river valleys is a limitation for the accuracy of seepage estimates.
- The lack of information about storativity, a large unknown is the total volume of water that will be lost into the surrounding, unsaturated rock mass, in turn leads to uncertainty regarding duration of peak seepage losses and the rate of decrease of seepage until groundwater levels stabilise. Pumping tests in the surrounding weathered rock unit (defined in AGE, 2015) as a hydrostratigraphic unit) would be the most accurate method of determining storativity and estimating the total volume of water lost to groundwater seepage. Water lost to groundwater storage is likely to be recoverable, in part, when reservoir levels decrease and the hydraulic gradient is reversed. However, there will be a component that is permanently lost to capillary action; again, this volume is difficult to estimate without unsaturated moisture levels from the rock mass and without further data from pumping tests.

3 Recommendations

To establish a more reliable seepage estimate:

- Additional data on groundwater levels should be collected through installation of monitoring bores in the mountain ranges in faulted areas (Figure 3-1).
- Additional hydraulic conductivity data should be collected by means of packer testing or test pumping, especially in relevant valleys and faulted areas (Figure 3-1).

Furthermore, a numerical 3D groundwater model should be developed to better understand:

- Peak seepage into reservoir walls, rate of decline, total volume lost and total volume that is recoverable after reservoir level decline: Assuming a groundwater gradient is a weak point of this methodology (assumption of a static value in a dynamic system), a groundwater model would greatly improve accuracy of peak predictions.
- Total constant losses through faulted areas: Accuracy of any model would be highly dependent on reliable hydraulic conductivities and groundwater levels gathered from the faulted zones.

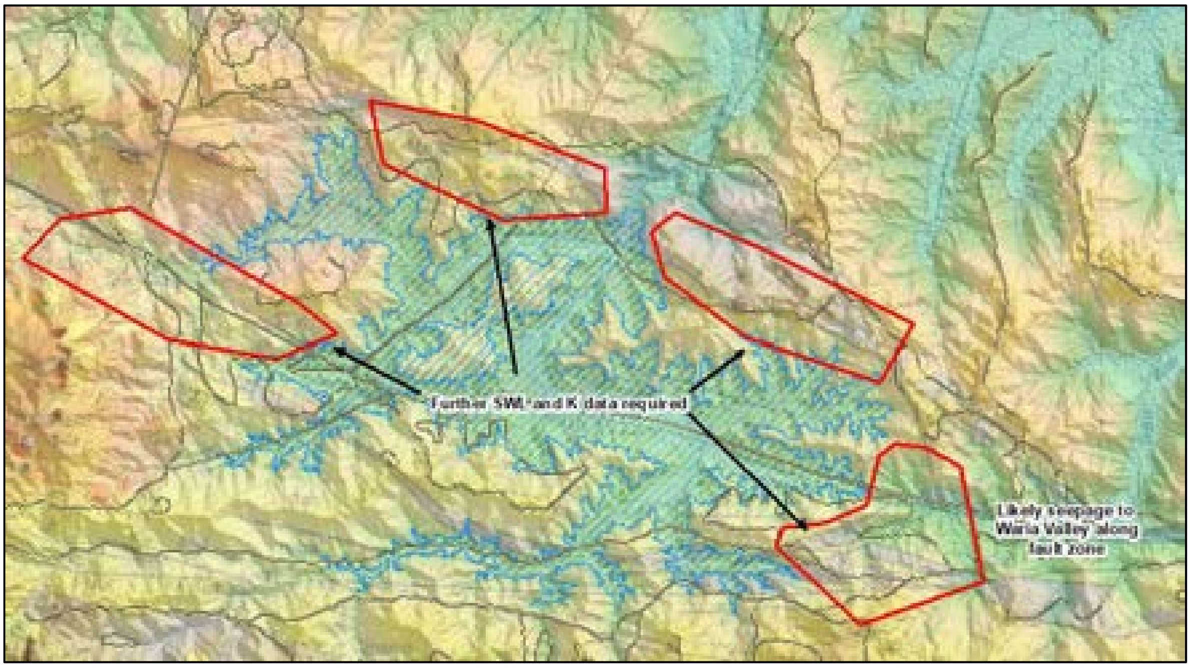

Figure 3-1: Areas requiring further information on groundwater levels and hydraulic conductivities

Notes: Blue outline is the 250 m RL reservoir boundary, yellow and black lines are faults.

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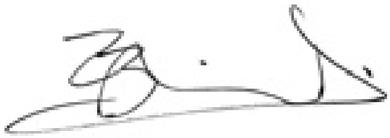
Yours faithfully

SRK Consulting (Australasia) Pty Ltd

Signed by:



Zbigniew Boniecki
Senior Hydrogeologist

Signed by:

Brian Luinstra
Principal Hydrogeologist

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0	19/04/2018	Brian Luinstra	Final Report
1	17/05/2018	Brian Luinstra	Revised Final Report
2	16/07/2018	Brian Luinstra	Revised Final Report
3	24/08/2018	Brian Luinstra	Revised Final Report