ARL GROUP

LABORATORY REPORT

Job Number: 18-00471
Revision: 00
Date: 5 February 2018

ADDRESS: **Liquid Labs WA**
4/96 Briggs Street
Welshpool WA 6106

ATTENTION: Matt Van Herk

DATE RECEIVED: 11/01/2018

YOUR REFERENCE: North Star Magnetite Project

PURCHASE ORDER:

APPROVALS:

DouglasTodd
Laboratory Manager

Sam Becker
Inorganics Manager

REPORT COMMENTS:

This report is issued by Analytical Reference Laboratory (WA) Pty Ltd
Samples are analysed on an as received basis unless otherwise noted.
Phosphorus Colwell, Potassium Colwell, & Sulphur analysis subcontracted to CSBP, Report Number CTS18162, 64-78

METHOD REFERENCES:

ARL No. 314	NOx in Soil and Sediment by Discrete Analyser
ARL No. 118	Total Phosphorus and TKN in Soil and Biosolids
ARL No. 401/403	Metals in Soil and Sediment by ICPOES/MS
ARL No. 138	pH in Soil and Biosolid
ARL No. 140	Conductivity in Soil and Biosolid
ARL No. 064	Total Organic Carbon in Sediment
ARL No. 136	Lime Equivalence in Biosolids
ARL No. 213	Exchangeable Bases
ARL No. 212	Exchangeable Acidity
Subcontracting	See Report Comments section for more information.

LABORATORY REPORT

Liquid Labs WA
ARL Job No: 18-00471 *Revision:* 00 *Date:* 5 February 2018

RESULTS:

Nutrients in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
NOx-N	1	mg/kg	3	1	1	<1	1
Total Nitrogen	10	mg/kg	420	250	360	230	450

Nutrients in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-6 LLS18/32 4/01/2018	18-00471-7 LLS18/33 4/01/2018	18-00471-8 LLS18/34 4/01/2018	18-00471-9 LLS18/35 4/01/2018	18-00471-10 LLS18/36 4/01/2018
NOx-N	1	mg/kg	<1	<1	<1	23	1
Total Nitrogen	10	mg/kg	360	270	240	280	530

Nutrients in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-11 LLS18/37 4/01/2018	18-00471-12 LLS18/38 4/01/2018	18-00471-13 LLS18/39 4/01/2018	18-00471-14 LLS18/40 4/01/2018	18-00471-15 LLS18/41 4/01/2018
NOx-N	1	mg/kg	2	<1	<1	<1	<1
Total Nitrogen	10	mg/kg	350	200	160	500	150

Nutrients in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-16 LLS18/42 4/01/2018
NOx-N	1	mg/kg	18
Total Nitrogen	10	mg/kg	330

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
Sulphur	0.01	% w/w	<0.01	<0.01	<0.01	<0.01	<0.01
pH	0.1	pH units	7.1	6.9	7.3	7.1	7.9
Conductivity	0.01	mS/cm	0.04	0.03	0.03	0.03	0.07
TOC	0.1	%	0.5	0.5	0.5	0.4	0.5
Maximum Potential Acidity	2	kgH2SO4/t	<2	<2	<2	<2	<2
Acid Neutralising Capacity BT (19A2)	2	kgH2SO4/t	<2	<2	<2	<2	5

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
Net Acid Producing Potential	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 4.5	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 7.0	2	kgH2SO4/t	<2	<2	<2	<2	<2

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-6 LLS18/32 4/01/2018	18-00471-7 LLS18/33 4/01/2018	18-00471-8 LLS18/34 4/01/2018	18-00471-9 LLS18/35 4/01/2018	18-00471-10 LLS18/36 4/01/2018
Sulphur	0.01	% w/w	<0.01	<0.01	<0.01	0.01	<0.01
pH	0.1	pH units	7.6	6.8	8.5	7.1	6.8
Conductivity	0.01	mS/cm	0.05	0.01	0.08	1.3	0.04
TOC	0.1	%	0.5	0.3	0.5	0.3	0.9
Maximum Potential Acidity	2	kgH2SO4/t	<2	<2	<2	<2	<2
Acid Neutralising Capacity BT (19A2)	2	kgH2SO4/t	<2	<2	11	<2	<2
Net Acid Producing Potential	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 4.5	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 7.0	2	kgH2SO4/t	<2	<2	<2	<2	<2

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-11 LLS18/37 4/01/2018	18-00471-12 LLS18/38 4/01/2018	18-00471-13 LLS18/39 4/01/2018	18-00471-14 LLS18/40 4/01/2018	18-00471-15 LLS18/41 4/01/2018
Sulphur	0.01	% w/w	<0.01	0.01	<0.01	<0.01	<0.01
pH	0.1	pH units	7.1	7.2	7.0	7.4	6.7
Conductivity	0.01	mS/cm	0.03	0.02	0.01	0.04	0.01
TOC	0.1	%	0.5	0.2	0.3	0.7	0.2
Maximum Potential Acidity	2	kgH2SO4/t	<2	<2	<2	<2	<2
Acid Neutralising Capacity BT (19A2)	2	kgH2SO4/t	<2	<2	<2	3	<2
Net Acid Producing Potential	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 4.5	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 7.0	2	kgH2SO4/t	<2	<2	<2	<2	<2

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-16 LLS18/42 4/01/2018
Sulphur	0.01	% w/w	<0.01
pH	0.1	pH units	7.2
Conductivity	0.01	mS/cm	0.08
TOC	0.1	%	0.4
Maximum Potential Acidity	2	kgH2SO4/t	<2
Acid Neutralising Capacity BT (19A2)	2	kgH2SO4/t	<2
Net Acid Producing Potential	2	kgH2SO4/t	<2
Net Acid Generation to pH 4.5	2	kgH2SO4/t	<2
Net Acid Generation to pH 7.0	2	kgH2SO4/t	<2

Cation Exchange Capacity Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
Exchangeable Calcium	0.2	cmolc/kg	3.0	3.7	3.8	3.2	8.9
Exchangeable Acidity	0.1	cmolc/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Exchangeable Potassium	0.05	cmolc/kg	1.5	0.98	2.4	1.5	2.0
Exchangeable Magnesium	0.2	cmolc/kg	2.2	1.7	2.0	2.8	3.2
Exchangeable Sodium	0.2	cmolc/kg	0.6	1.3	0.6	0.6	1.2
Cation Exchange Capacity	1	cmolc/kg	7	8	9	8	15.3

Cation Exchange Capacity Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-6 LLS18/32 4/01/2018	18-00471-7 LLS18/33 4/01/2018	18-00471-8 LLS18/34 4/01/2018	18-00471-9 LLS18/35 4/01/2018	18-00471-10 LLS18/36 4/01/2018
Exchangeable Calcium	0.2	cmolc/kg	4.8	1.9	22	2.2	3.6
Exchangeable Acidity	0.1	cmolc/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Exchangeable Potassium	0.05	cmolc/kg	1.6	1.5	1.7	1.8	1.4
Exchangeable Magnesium	0.2	cmolc/kg	2.6	2.5	3.8	4.4	2.4
Exchangeable Sodium	0.2	cmolc/kg	0.8	0.8	0.4	3.9	0.5
Cation Exchange Capacity	1	cmolc/kg	10	7	27.9	12.3	8

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ARL Job No: *18-00471* *Revision:* *00* *Date:* 5 February 2018

Cation Exchange Capacity Sample No:	**LOR**	**UNITS**	**18-00471-11**	**18-00471-12**	**18-00471-13**	**18-00471-14**	**18-00471-15**
Sample Description:			**LLS18/37**	**LLS18/38**	**LLS18/39**	**LLS18/40**	**LLS18/41**
Sample Date:			**4/01/2018**	**4/01/2018**	**4/01/2018**	**4/01/2018**	**4/01/2018**
Exchangeable Calcium	0.2	cmolc/kg	4.3	2.6	2.6	8.0	1.6
Exchangeable Acidity	0.1	cmolc/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Exchangeable Potassium	0.05	cmolc/kg	0.74	0.75	0.65	1.4	0.50
Exchangeable Magnesium	0.2	cmolc/kg	2.5	2.5	5.3	2.6	2.7
Exchangeable Sodium	0.2	cmolc/kg	0.9	0.7	0.8	0.5	0.9
Cation Exchange Capacity	1	cmolc/kg	8	7	9	12.5	6

Cation Exchange Capacity Sample No:	**LOR**	**UNITS**	**18-00471-16**
Sample Description:			**LLS18/42**
Sample Date:			**4/01/2018**
Exchangeable Calcium	0.2	cmolc/kg	2.3
Exchangeable Acidity	0.1	cmolc/kg	<0.1
Exchangeable Potassium	0.05	cmolc/kg	0.80
Exchangeable Magnesium	0.2	cmolc/kg	2.3
Exchangeable Sodium	0.2	cmolc/kg	0.6
Cation Exchange Capacity	1	cmolc/kg	6

Subcontracting Sample No:	**LOR**	**UNITS**	**18-00471-1**	**18-00471-2**	**18-00471-3**	**18-00471-4**	**18-00471-5**
Sample Description:			**LLS18/27**	**LLS18/28**	**LLS18/29**	**LLS18/30**	**LLS18/31**
Sample Date:			**4/01/2018**	**4/01/2018**	**4/01/2018**	**4/01/2018**	**4/01/2018**
Phosphorus Colwell	2	mg/kg	4	4	5	3	4
Potassium Colwell	2	mg/kg	200	180	210	160	170
Available Sulphur	0.5	mg/kg	1.7	1.4	1.3	1.5	2.1

Subcontracting Sample No:	**LOR**	**UNITS**	**18-00471-6**	**18-00471-7**	**18-00471-8**	**18-00471-9**	**18-00471-10**
Sample Description:			**LLS18/32**	**LLS18/33**	**LLS18/34**	**LLS18/35**	**LLS18/36**
Sample Date:			**4/01/2018**	**4/01/2018**	**4/01/2018**	**4/01/2018**	**4/01/2018**
Phosphorus Colwell	2	mg/kg	3	<2	4	6	6
Potassium Colwell	2	mg/kg	160	96	130	310	330
Available Sulphur	0.5	mg/kg	1.0	1.5	0.5	110	1.9

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Subcontracting Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-11 LLS18/37 4/01/2018	18-00471-12 LLS18/38 4/01/2018	18-00471-13 LLS18/39 4/01/2018	18-00471-14 LLS18/40 4/01/2018	18-00471-15 LLS18/41 4/01/2018
Phosphorus Colwell	2	mg/kg	4	<2	3	<2	3
Potassium Colwell	2	mg/kg	220	250	170	190	83
Available Sulphur	0.5	mg/kg	1.0	0.9	<0.5	<0.5	1.0

Subcontracting Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-16 LLS18/42 4/01/2018
Phosphorus Colwell	2	mg/kg	3
Potassium Colwell	2	mg/kg	160
Available Sulphur	0.5	mg/kg	3.3

Result Definitions

LOR Limit of Reporting [NT] Not Tested [ND] Not Detected at indicated Limit of Reporting

* Denotes test not covered by NATA Accreditation

FOR MICROBIOLOGICAL TESTING - The data in this report may not be representative of a lot, batch or other samples and may not necessarily justify the acceptance or rejection of a lot or batch, a product recall or support legal proceedings. Tests are not routinely performed as duplicates unless specifically requested. Changes occur in the bacterial content of biological samples. Samples should be examined as soon as possible after collection, preferably within 6 hrs and must be stored at 4 degrees Celsius or below. Samples tested after 24 hrs cannot be regarded as satisfactory because of temperature abuse and variations.

CERTIFICATE OF ANALYSIS

Work Order	: **EP1802462**	Page	: 1 of 4
Client	: **ATC WILLIAMS P/L**	Laboratory	: Environmental Division Perth
Contact	: ALPESH PATEL	Contact	: Customer Services EP
Address	: 1141 HAY STREET WEST PERTH W.A. 6005	Address	: 26 Rigali Way Wangara WA Australia 6065
Telephone	: ----	Telephone	: +61-8-9406 1301
Project	: North Star Magnetite	Date Samples Received	: 15-Feb-2018 12:35
Order number	: 114185.1	Date Analysis Commenced	: 15-Feb-2018
C-O-C number	: ----	Issue Date	: 22-Feb-2018 17:08
Sampler	: ALPESH PATEL		
Site	: ----		
Quote number	: EN/222/17		
No. of samples received	: 4		
No. of samples analysed	: 4		

Accreditation No. 825
Accredited for compliance with
ISO/IEC 17025 - Testing

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted. This document shall not be reproduced, except in full.

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

Additional information pertinent to this report will be found in the following separate attachments: Quality Control Report, QA/QC Compliance Assessment to assist with Quality Review and Sample Receipt Notification.

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	*Position*	*Accreditation Category*
Canhuang Ke	Metals Instrument Chemist	Perth Inorganics, Wangara, WA
Dilani Fernando	Senior Inorganic Chemist	Melbourne Inorganics, Springvale, VIC
Indra Astuty	Instrument Chemist	Perth Inorganics, Wangara, WA
Jeremy Truong	Laboratory Manager	Perth Inorganics, Wangara, WA
Tyrone Cole	Inorganics Preparation Supervisor	Perth Inorganics, Wangara, WA

General Comments

The analytical procedures used by the Environmental Division have been developed from established internationally recognized procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are employed in the absence of documented standards or by client request.

Where moisture determination has been performed, results are reported on a dry weight basis.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.

Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

When sampling time information is not provided by the client, sampling dates are shown without a time component. In these instances, the time component has been assumed by the laboratory for processing purposes.

Where a result is required to meet compliance limits the associated uncertainty must be considered. Refer to the ALS Contact for details.

Key : CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
^ = This result is computed from individual analyte detections at or above the level of reporting
ø = ALS is not NATA accredited for these tests.
~ = Indicates an estimated value.

- Fluoride analysis conducted by ALS Melbourne, NATA accreditation no. 825, site no 13778

Analytical Results

Sub-Matrix: GROUNDWATER (Matrix: WATER)			Client sample ID	BH2A-01	BH2A-04	BH2A-05	DUPLICATE	----
			Client sampling date / time	11-Feb-2018 00:00	11-Feb-2018 00:00	11-Feb-2018 00:00	11-Feb-2018 00:00	----
Compound	CAS Number	LOR	Unit	EP1802462-001	EP1802462-002	EP1802462-003	EP1802462-004	--------
				Result	Result	Result	Result	----
EA005P: pH by PC Titrator								
pH Value	----	0.01	pH Unit	8.29	8.08	8.62	8.31	----
EA015: Total Dissolved Solids dried at 180 ± 5 °C								
Total Dissolved Solids @180°C	----	10	mg/L	660	694	565	618	----
EA025: Total Suspended Solids dried at 104 ± 2°C								
Suspended Solids (SS)	----	5	mg/L	234	792	210	219	----
EA065: Total Hardness as CaCO3								
Total Hardness as CaCO3	----	1	mg/L	491	429	457	476	----
ED037P: Alkalinity by PC Titrator								
Hydroxide Alkalinity as CaCO3	DMO-210-001	1	mg/L	<1	<1	<1	<1	----
Carbonate Alkalinity as CaCO3	3812-32-6	1	mg/L	<1	<1	41	3	----
Bicarbonate Alkalinity as CaCO3	71-52-3	1	mg/L	401	419	354	397	----
Total Alkalinity as CaCO3	----	1	mg/L	401	419	395	400	----
ED040F: Dissolved Major Anions								
Sulfur as S	63705-05-5	1	mg/L	28	20	20	29	----
Silicon as SiO2	14464-46-1	0.1	mg/L	22.0	23.6	13.6	22.4	----
ED041G: Sulfate (Turbidimetric) as SO4 2- by DA								
Sulfate as SO4 - Turbidimetric	14808-79-8	1	mg/L	94	68	65	92	----
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	1	mg/L	138	118	102	134	----
ED093F: Dissolved Major Cations								
Calcium	7440-70-2	1	mg/L	17	20	15	16	----
Magnesium	7439-95-4	1	mg/L	109	92	102	106	----
Sodium	7440-23-5	1	mg/L	97	102	81	94	----
Potassium	7440-09-7	1	mg/L	3	9	2	3	----
EG020F: Dissolved Metals by ICP-MS								
Aluminium	7429-90-5	0.01	mg/L	0.17	0.02	0.02	0.02	----
Antimony	7440-36-0	0.001	mg/L	<0.001	<0.001	<0.001	<0.001	----
Arsenic	7440-38-2	0.001	mg/L	0.019	0.009	0.034	0.020	----
Boron	7440-42-8	0.05	mg/L	0.38	0.43	0.36	0.40	----
Barium	7440-39-3	0.001	mg/L	0.014	0.022	0.010	0.012	----
Beryllium	7440-41-7	0.001	mg/L	<0.001	<0.001	<0.001	<0.001	----
Cadmium	7440-43-9	0.0001	mg/L	<0.0001	<0.0001	<0.0001	<0.0001	----
Cobalt	7440-48-4	0.001	mg/L	0.001	0.003	<0.001	<0.001	----
Chromium	7440-47-3	0.001	mg/L	<0.001	<0.001	<0.001	<0.001	----

Analytical Results

Sub-Matrix: GROUNDWATER (Matrix: WATER)			Client sample ID	BH2A-01	BH2A-04	BH2A-05	DUPLICATE	----
			Client sampling date / time	11-Feb-2018 00:00	11-Feb-2018 00:00	11-Feb-2018 00:00	11-Feb-2018 00:00	----
Compound	CAS Number	LOR	Unit	EP1802462-001	EP1802462-002	EP1802462-003	EP1802462-004	--------
				Result	Result	Result	Result	----
EG020F: Dissolved Metals by ICP-MS - Continued								
Copper	7440-50-8	0.001	mg/L	**0.001**	**0.006**	**0.001**	**0.001**	----
Manganese	7439-96-5	0.001	mg/L	**0.026**	**0.039**	<0.001	**0.021**	----
Nickel	7440-02-0	0.001	mg/L	**0.003**	**0.012**	**0.003**	**0.003**	----
Lead	7439-92-1	0.001	mg/L	<0.001	<0.001	<0.001	<0.001	----
Selenium	7782-49-2	0.01	mg/L	<0.01	<0.01	<0.01	<0.01	----
Vanadium	7440-62-2	0.01	mg/L	<0.01	<0.01	<0.01	<0.01	----
Zinc	7440-66-6	0.005	mg/L	<0.005	**0.009**	<0.005	<0.005	----
Molybdenum	7439-98-7	0.001	mg/L	**0.005**	**0.005**	**0.007**	**0.005**	----
Silver	7440-22-4	0.001	mg/L	<0.001	<0.001	**0.001**	<0.001	----
Strontium	7440-24-6	0.001	mg/L	**0.145**	**0.208**	**0.082**	**0.142**	----
Thorium	7440-29-1	0.001	mg/L	<0.001	<0.001	<0.001	<0.001	----
Tin	7440-31-5	0.001	mg/L	<0.001	<0.001	<0.001	<0.001	----
Titanium	7440-32-6	0.01	mg/L	<0.01	<0.01	<0.01	<0.01	----
Uranium	7440-61-1	0.001	mg/L	**0.002**	**0.002**	<0.001	**0.002**	----
Iron	7439-89-6	0.05	mg/L	<0.05	<0.05	<0.05	<0.05	----
EG020T: Total Metals by ICP-MS								
Aluminium	7429-90-5	0.01	mg/L	**6.67**	**16.4**	**4.55**	**6.53**	----
Iron	7439-89-6	0.05	mg/L	**6.14**	**19.1**	**7.82**	**5.84**	----
EG035F: Dissolved Mercury by FIMS								
Mercury	7439-97-6	0.0001	mg/L	<0.0001	<0.0001	<0.0001	<0.0001	----
EK040P: Fluoride by PC Titrator								
Fluoride	16984-48-8	0.1	mg/L	**0.5**	**0.5**	**0.3**	**0.5**	----
EK057G: Nitrite as N by Discrete Analyser								
Nitrite as N	14797-65-0	0.01	mg/L	<0.01	<0.01	<0.01	<0.01	----
EK058G: Nitrate as N by Discrete Analyser								
Nitrate as N	14797-55-8	0.01	mg/L	**0.03**	**0.36**	**0.02**	**0.03**	----
EK059G: Nitrite plus Nitrate as N (NOx) by Discrete Analyser								
Nitrite + Nitrate as N	----	0.01	mg/L	**0.03**	**0.36**	**0.02**	**0.03**	----
EK067G: Total Phosphorus as P by Discrete Analyser								
Total Phosphorus as P	----	0.01	mg/L	**0.02**	**0.09**	**0.05**	**0.04**	----
EN055: Ionic Balance								
Total Anions	----	0.01	meq/L	**13.9**	**13.1**	**12.1**	**13.7**	----
Total Cations	----	0.01	meq/L	**14.1**	**13.2**	**12.7**	**13.7**	----
Ionic Balance	----	0.01	%	**0.91**	**0.46**	**2.40**	<0.01	----

APPENDIX C1

REPORT

IRON BRIDGE OPERATIONS PTY LTD

North Star Project - Stage 2
Pilbara, Western Australia

Tailings Storage Facility & Return Water Pond

Mining Proposal Design Report

Appendix C1 - Tailings Laboratory Testing

October 2018

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ANDREW FYANS
Author

CRAIG NOSKE
Reviewer

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TABLE OF CONTENTS

FIGURES

1 INTRODUCTION

Since ATC Williams (ATCW) commenced work for the North Star Magnetite - Stage 2 Project by Iron Bridge Operations (IBO) Pty. Ltd, a range of geotechnical testing has been carried out on tailings samples at the ATCW laboratory in Melbourne.

Two rounds of tailings testing have been completed to date, the first of which was completed in March 2015 **[1]**, and the second completed in late 2016, and reported in January 2017 **[2]**.

This Report is provided as an Appendix to the October 2018 Mining Submission Design Report for the Site 2A Tailings Storage Facility (TSF) and Return Water Pond, and provides a consolidated summary of the relevant "wet" tailings test results conducted in 2015 **[1]** and 2016 **[2]**.

2 SAMPLE DETAILS

2.1 Background

As the North Star Magnetite Project has progressed, the composition of the tailings has been modified, and as such a number of test results are no longer relevant for the Stage 2 Project. This Report discusses only those samples which are still relevant to the North Star TSF.

As advised by IBO, the current Project Stage 2 design tailings particle size distribution is finer than what was used for previous studies, with a P_{80} reducing from 75 µm to approximately 40 µm.

Hence, until updated testwork can be completed, the ATCW laboratory characterisation of "wet" tailings samples conducted in 2015 **[1]** represent the most appropriate data for design purposes.

Testing in 2015 included basic characterisation tests (particle density, plasticity) and specialised tailings testing (initial settled density and rheology).

Unfortunately other important specialised tests (shrinkage limit density and segregation threshold) were not conducted on the 2015 samples. In order to account for this, a comparison and interpolation exercise has been conducted using the slightly coarser grind, 2016 wet tailings testwork **[2]**.

2.2 Tailings Sample Receipt

2.2.1 2015 Samples

On 5 November 2014, the ATCW laboratory in Melbourne received the followings samples (as 3 x 20L buckets):

- "Fig 5. Combination of primary wet LIMS from batch plant (fresh) (coarse – 63%) and regrind wet LIMS (P80 30 micron -37%)"; and
- "Fig 6. Combination of primary wet LIMS (fresh)(coarse – 63%) and secondary wet LIMS (P80 20 micron -37%) from batch plant".

The samples are referred to as "Figure 5" and "Figure 6", respectively, throughout this Report.

2.2.2 2016 Samples

On 19 December 2016, the ATCW laboratory received the followings samples:

- PP25 North Star, RMS Non-Mags 11/10/16 (5 x10 L buckets); and
- PP32 North Star, LMR Non-Mags 11/10/16 (3 x 10 L buckets).

These samples were combined using a mixing ratio of 1.88 : 1 (RMS to LMR) provided by IBO. The 2016 combined sample is herein referred to as "Combined Tailings".

2.3 Sample Preparation and Flocculation

The procedure for initial sample preparation and flocculation are detailed in **[1]** and **[2]**.

3 TESTING PROGRAM

General material characterisation tests were conducted for both the 2015 and 2016 samples. These tests included the following:

- Particle Size Distribution (PSD) and hydrometer;
- Atterberg Limits; and
- Soil Particle Density / Specific Gravity (SG).

The abovementioned tests were performed on un-flocculated samples for Figure 5 and Figure 6. As outlined in **[2]**, the Combined Tailings arrived at the ATCW laboratory already flocculated and therefore the tests were conducted on flocculated samples.

Table 1 summarises the testing program that was performed on the samples after flocculation.

Table 1
Testing Program on Flocculated Samples

Test	Samples		
	Figure 5	Figure 6	Combined Tailings
Segregation Threshold			X
Initial Settled Density	X	X	X
Shrinkage Limit Density			X
Rheology (Rotational)	X	X	X
Rheology (Pipe Loop)	X		X

4 TEST RESULTS

4.1 Combined Sieve and Hydrometer Analysis

PSD and Hydrometer tests were carried out on representative sub-samples of Figure 5, Figure 6 and the Combined Tailings. These tests were in accordance with the Australian standard test methods AS 1289.3.6.1 **[3]** and AS 1289.3.6.3 **[4]**.

The NATA accredited test reports for these tests have been presented in **Figure C1**, **Figure C2**, and **Figure C3**.

4.2 Atterberg limits

Representative sub-samples were taken from the un-flocculated Figure 5 and Figure 6, and the flocculated Combined Tailings, tested in accordance with the following Australian Standard test methods:

- AS 1289.3.1.2 – Liquid limit (one point Casagrande method) **[5]**
- AS 1289.3.2.1 – Plastic Limit **[6]**
- AS 1289.3.3.1 – Calculation of the Plasticity Index **[7]**

The results of the testing are shown in **Table 2**.

Table 2
Atterberg Limits

Sample	Liquid Limit	Plastic Limit	Plasticity Index
Figure 5	21	17	4
Figure 6	19	17	2
Combined Tailings	18	16	2

NATA accredited test reports for these tests have been included in **Figure C4** and **Figure C5**.

4.3 Particle Density

Representative sub-samples were taken from Figure 5, Figure 6 and the Combined Tailings and tested for particle density in accordance with AS 1289.3.5.1 **[8]**. The result of the testing are summarised in **Table 3**.

Table 3
Particle Density

Sample	Particle Density (t/m^3)
Figure 5	2.97
Figure 6	2.97
Combined Tailings	3.04

NATA endorsed test reports for these tests have been included in **Figure C6** and **Figure C7**.

4.4 Segregation Threshold

The purpose of this test is to estimate the solids content at which separation of the coarser grading fraction first occurs during static settling. This is known as the segregation threshold.

The test procedure requires a sample of slurry to be mixed in a glass cylinder and allowed to settle. The solids concentration is varied by adding decant water as required and the test is repeated until some sign of segregation is observed. Two sub-samples of equal volume are then taken using a small pipette, one from the bottom of the cylinder and the other from just below the surface.

These samples are put into separate test tubes and both are diluted, agitated and decanted to separate the coarser particles. The volumes of the coarser particles remaining are compared and the whole test repeated after solids concentration adjustment, until the volumes of coarse particles are judged to be different. The slurry in the cylinder is then tested for solids concentration and this is the threshold value.

Segregation threshold testing has only been performed for the Combined Tailings sample. The segregation threshold for the Combined Tailings Sample is 58%.

A NATA endorsed report for the this segregation threshold test is presented in **Figure C8**.

4.5 Initial Settled Density

The test is performed by taking a sub-sample of the flocculated material at the required initial solids content and pouring into a large diameter pan to a target depth. A large diameter pan is used to limit the effect of friction between the settling sample and the side wall. The settlement of the surface of the tailings over time is recorded until settlement is judged to be complete.

Sub-samples of the flocculated Figure 5 and Figure 6 samples were tested at a nominal initial solids concentration of 65% and a nominal initial depth of 100 mm. The Combined Tailings sample was also tested at a nominal initial solids concentration of 65% and at nominal initial depths of 100 mm, 150 mm and 200 mm.

The settlement results for the tests are plotted in **Figure C9** to **Figure C14**.

A summary of the data obtained from the testing is also presented in **Table 4**.

Table 4
Initial Settled Density Results

Sample Description	Approx. Time to Reach t_{100} (min)	Solids Concentration (%)		Sample Depth (mm)			$\frac{H_F}{H_O} \times 100\%$	Settled Dry Density (t/m³)	Bleed Water	
		Initial	Final	Initial H_o	Final H_F	ΔH			% of Initial Water	m³/t of Dry Solids
Figure 5	470	64.9	73.9	98.6	77.5	21.1	78.6	1.45	34.8	0.19
Figure 6	520	65.1	73.4	100.0	80.1	19.9	80.1	1.43	32.4	0.17
Combined Tailings	420	65.1	75.2	150.0	123.5	26.5	82.3	1.52	36.8	0.21

t_{100} - Time taken in minutes for prepared samples to reach a depth of 100 mm
H_o - Initial sample depth in mm
H_F - Final sample depth in mm at the end of the test
ΔH - Change in sample depth from start to end of test
H_F/H_0 x 100% - Percentage of final depth vs starting sample depth from start to end of tests

4.6 Shrinkage Limit Density

The shrinkage limit test is carried out to determine the maximum dry density that could result from the evaporative drying of the tailings on a beach.

Shrinkage limit density was only determined for the Combined Tailings sample, which returned a value of 1.87 t/m^3.

The shrinkage limit density result for the Combined Tailings sample is plotted in **Figure C15**.

4.7 Rotational Rheology

4.7.1 Methodology

The rheology testing was carried out using a Thermo Haake VT550 Viscotester. A MV2P Couette flow (bob in cup) sensor system was used for the measurement of flow curve rheology. The sample was mixed and the cup was filled to the correct level prior to testing. When the flow curve testing was complete, the sample in the cup was discarded, the bob and cup were cleaned and dried, the remaining sample was remixed and the flow curve test was repeated.

Herschel – Bulkley modelling was performed on the flow curve data in accordance with the following pseudo-plastic model relationship:

$\tau = \tau_y + K\dot{\gamma}^n$ Where: τ = Shear Stress

τ_y = Yield Stress

K = Consistency Coefficient

$\dot{\gamma}$ = Shear Rate

n = Flow Behaviour Index

Modelling was conducted at shear rates above the observed slippage in order to provide a theoretical line of best fit of the experimental data and for inferred yield strength parameters only. The parameters derived have not been compared against each other for overall material characterization modelling. Modelling was performed using the Thermo Haake Rheowin Data Manager software.

A FL100 shear vane sensor system was used for the direct measurement of the yield stress using a vane rotation of 0.1 RPM.

Testing was performed on the flocculated Figure 5, Figure 6 and Combined Tailings samples.

4.7.2 Figure 5 Rheology

Testing on the Figure 5 sample for both flow curve rheology and shear vane testing was performed at solids concentrations of 61.7%, 63.9%, 65.9% and 67.9%. Flow curve rheology testing of the Figure 5 sample was also performed on sub-samples taken from the pipe loop at solids concentrations of 60.1% and 64.4%.

In order to mimic the effects of long distance pipeline transport of the tailings, rheology testing was also performed on a sample sheared using a laboratory mixer. The sample was prepared and mixed in a wide mouth 1 litre sample container using a standard laboratory mixer at 1400 RPM for 2 hours. This test was performed at measured solids concentration of 65.0%

The data derived from the flow curves is plotted on **Figures C16** and **Figure C17**. The plastic viscosity has been defined as the gradient of the tangent of the flow curve at a shear rate of 100 sec^{-1}, while the intercept of this tangent is shown as the tangent intercept stress. The yield stress values have derived from the Herschel – Bulkley models. Yield stress measurements made on the samples using the FL100 vane are also plotted on **Figure C16**.

The shear vane test results are shown as shear strength versus time curves and are presented on **Figure C18.** A copy of the flow curves are presented on **Figure C19** and **Figure C20**.

4.7.3 Figure 6 Rheology

Testing on the Figure 6 sample for both flow curve rheology and shear vane testing was performed at solids concentrations of 61.6%, 63.1%, 65.2% and 66.6%.

The data derived from the flow curves is plotted on **Figures C21** and **Figure C22**. Yield stress measurements made on the samples using the FL100 vane are also plotted on **Figure C21**.

The plastic viscosity definitions and yield stress derivations are as described for the Figure 5 tailings in **Section 4.7.2** above.

The shear vane test results are shown as shear strength versus time curves and are presented in **Figure C23.** A copy of the flow curves is included in **Figure C24** and **Figure C25.**

4.7.4 Combined Tailings Rheology

Testing on the Combined Tailings sample for both flow curve rheology and shear vane testing was performed at solids concentrations of 65.0%, 68.98% and 73.41%.

The data derived from the flow curves is plotted on **Figures C26** and **Figure C27**. Yield stress measurements made on the samples using the FL100 vane are also plotted on **Figure C26**.

The plastic viscosity definitions and yield stress derivations and are as described for the Figure 5 tailings in **Section 4.7.2** above.

The shear vane test results are shown as shear strength versus time curves and are presented in **Figure C28.** A copy of the flow curves is included in **Figure C29** and **Figure C30.**

4.8 Pipe Loop Rheology

4.8.1 Figure 5

The Fig 5 sample was tested at three solids concentrations in the pipe loop and the results are presented in **Figure C16** and **Figure C17**. Details on pipe loop testing were provided in the technical memo submitted by ATCW in March 2015 **[9]**.

5 CLOSURE

Your attention is drawn to the “Conditions of Investigation” which appear after the document history and status page of this report.

ATC WILLIAMS PTY LTD

ATC Williams

REFERENCES

[1] ATCW Williams, March 2015, “Interim Work – Laboratory Testing Report”, Ref. 114185.03R01 Rev 2.

[2] ATCW Williams, February 2017, “Laboratory Testing Results for 2016 Wet Tailings Sample”, 114185.08 M01_Rev0.

[3] AS 1289.3.6.1 – Soil classification tests – Determination of the particle distribution of a soil – Standard method of analysis by sieving. ISBN 0 7337 9138 7

[4] AS 1289.3.6.3 - Soil classification tests – Determination of the particle distribution of a soil – Standard method of fine analysis using a hydrometer. ISBN 0 7337 5325 6

[5] AS 1289.3.1.2 – Soil classification tests – Determination of the liquid limit of a soil – One point Casagrande method (subsidiary method). ISBN 0 7262 9451 9

[6] AS 1289.3.2.1 – Soil classification tests – Determination of the plastic limit of a soil – Standard method. ISBN 0 7337 9005 4

[7] AS 1289.3.3.1 – Soil classification tests – Calculation of the plasticity index of a soil. ISBN 0 7337 9006 2

[8] AS 1289.3.5.1 – Soil classification tests – Determination of the soil particle density of a soil – Standard method. ISBN 0 7337 7475 X

[9] ATC Williams, 2015, Technical Memo - Pipe Loop Testing, Ref. 114185 03 M04

FIGURES

Particle Size Distribution Results

TEST IN ACCORDANCE WITH AS 1289

☑ Method 3.6.1 ☑ Method 3.6.3

☑ Oven Drying Method 2.1.1

ATC Williams GROUNDED IN DESIGN

Client: Iron Bridge Operations Pty Ltd

Address: Level 2, 87 Adelaide Terrace East Perth, WA

Project: North Star Magnetite Project-Stage 2

NATA Report No.: R02015

Job No.: 114185.03

Register No.: 25414

Location: WA

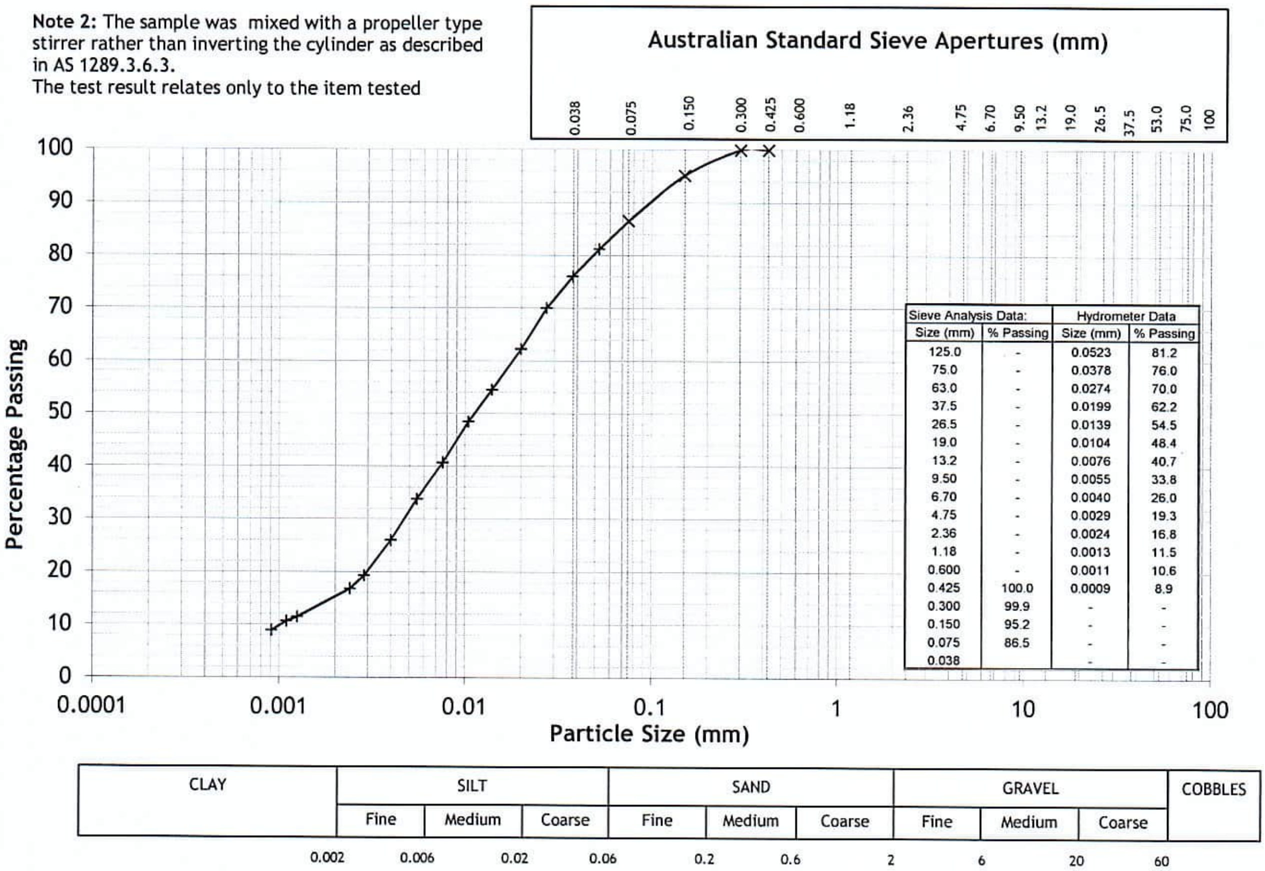
Sample Description: Sample Figure 5	Borehole ☐ No:	Test Pit ☐ Depth:

Dispersion Method used: Mechanical Stirrer for 1 minute **Hydrometer Type Used:** ASTM 152H

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1 Clause 6.5.4 ☑ Sample provided by the Client

Note 1: The sample was oven-dried during sample preparation and not air-dried as stated in AS 1289.3.6.3

Note 2: The sample was mixed with a propeller type stirrer rather than inverting the cylinder as described in AS 1289.3.6.3.
The test result relates only to the item tested



Sieve Analysis Data:		Hydrometer Data	
Size (mm)	% Passing	Size (mm)	% Passing
125.0	-	0.0523	81.2
75.0	-	0.0378	76.0
63.0	-	0.0274	70.0
37.5	-	0.0199	62.2
26.5	-	0.0139	54.5
19.0	-	0.0104	48.4
13.2	-	0.0076	40.7
9.50	-	0.0055	33.8
6.70	-	0.0040	26.0
4.75	-	0.0029	19.3
2.36	-	0.0024	16.8
1.18	-	0.0013	11.5
0.600	-	0.0011	10.6
0.425	100.0	0.0009	8.9
0.300	99.9	-	-
0.150	95.2	-	-
0.075	86.5	-	-
0.038		-	-

NATA ACCREDITED LABORATORY NUMBER: 3372

NATA ACCREDITED FOR TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025

Approved Signatory: [signature] Date: 2/02/2015

Name of Signatory: Peter Lam

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FIGURE C1

Particle Size Distribution Results

TEST IN ACCORDANCE WITH AS 1289

☑ Method 3.6.1 ☑ Method 3.6.3

☑ Oven Drying Method 2.1.1

ATC Williams
GROUNDED IN DESIGN

Client: Iron Bridge Operations Pty Ltd

Address: Level 2, 87 Adelaide Terrace East Perth, WA

Project: North Star Magnetite Project-Stage 2

NATA Report No.: R02115

Job No.: 114185.03

Register No.: 25514

Location: WA

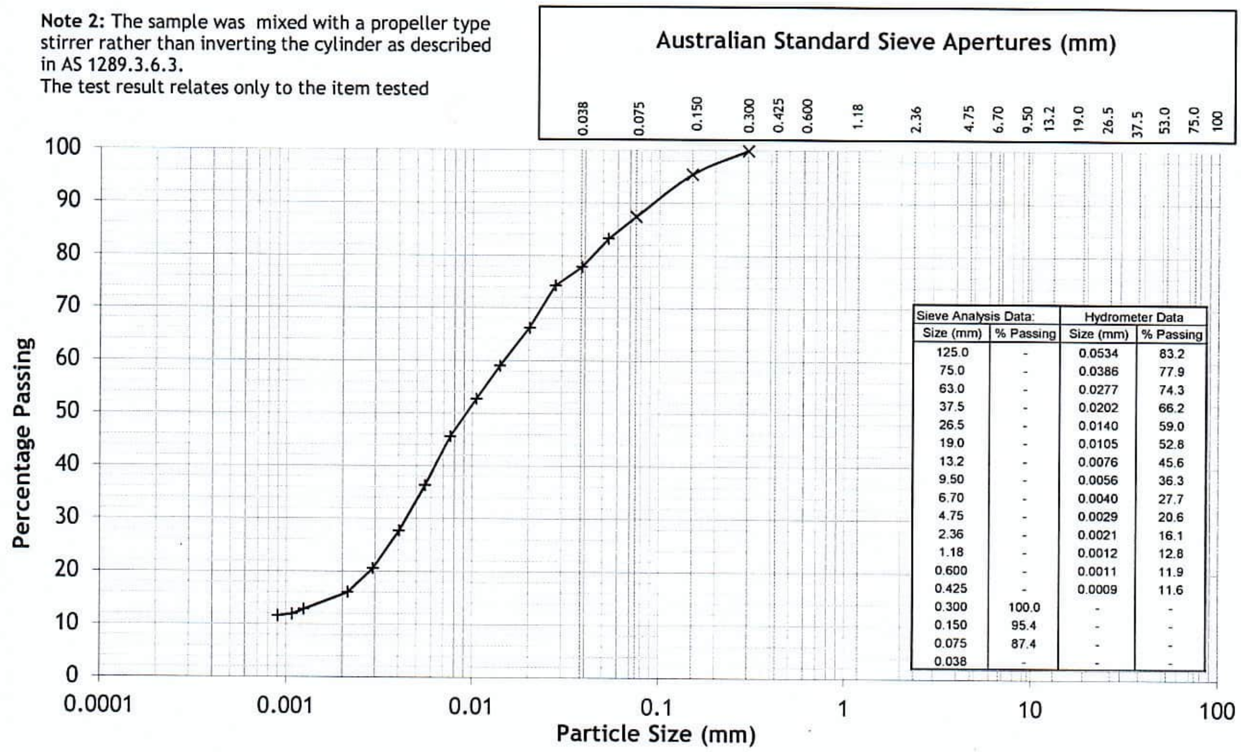
Sample Description: Sample Figure 6	Borehole ☐ No:	Test Pit ☐ Depth:

Dispersion Method used: Mechanical Stirrer for 1 minute **Hydrometer Type Used:** ASTM 152H

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1 Clause 6.5.4 ☑ Sample provided by the Client

Note 1: The sample was oven-dried during sample preparation and not air-dried as stated in AS 1289.3.6.3

Note 2: The sample was mixed with a propeller type stirrer rather than inverting the cylinder as described in AS 1289.3.6.3.
The test result relates only to the item tested



Sieve Analysis Data:		Hydrometer Data	
Size (mm)	% Passing	Size (mm)	% Passing
125.0	-	0.0534	83.2
75.0	-	0.0386	77.9
63.0	-	0.0277	74.3
37.5	-	0.0202	66.2
26.5	-	0.0140	59.0
19.0	-	0.0105	52.8
13.2	-	0.0076	45.6
9.50	-	0.0056	36.3
6.70	-	0.0040	27.7
4.75	-	0.0029	20.6
2.36	-	0.0021	16.1
1.18	-	0.0012	12.8
0.600	-	0.0011	11.9
0.425	-	0.0009	11.6
0.300	100.0	-	-
0.150	95.4	-	-
0.075	87.4	-	-
0.038	-	-	-

CLAY	SILT			SAND			GRAVEL			COBBLES
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	

0.002 0.006 0.02 0.06 0.2 0.6 2 6 20 60

NATA ACCREDITED LABORATORY NUMBER: 3372

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025

Approved Signatory: [signature] Date: 2/02/2015

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FIGURE C2

Particle Size Distribution Results

TEST IN ACCORDANCE WITH AS 1289

☑ Method 3.6.1 ☑ Method 3.6.3

☑ Oven Drying Method 2.1.1

ATC Williams

Client: Iron Bridge Operations Pty Ltd

Address: Level 2, 87 Adelaide Terrace, East Perth, WA 6004

Project: North Star Project - 2016 Tailings Testing

NATA Report No.: R00217

Job No.: 114185.08

Register No.: 00417

Location: WA

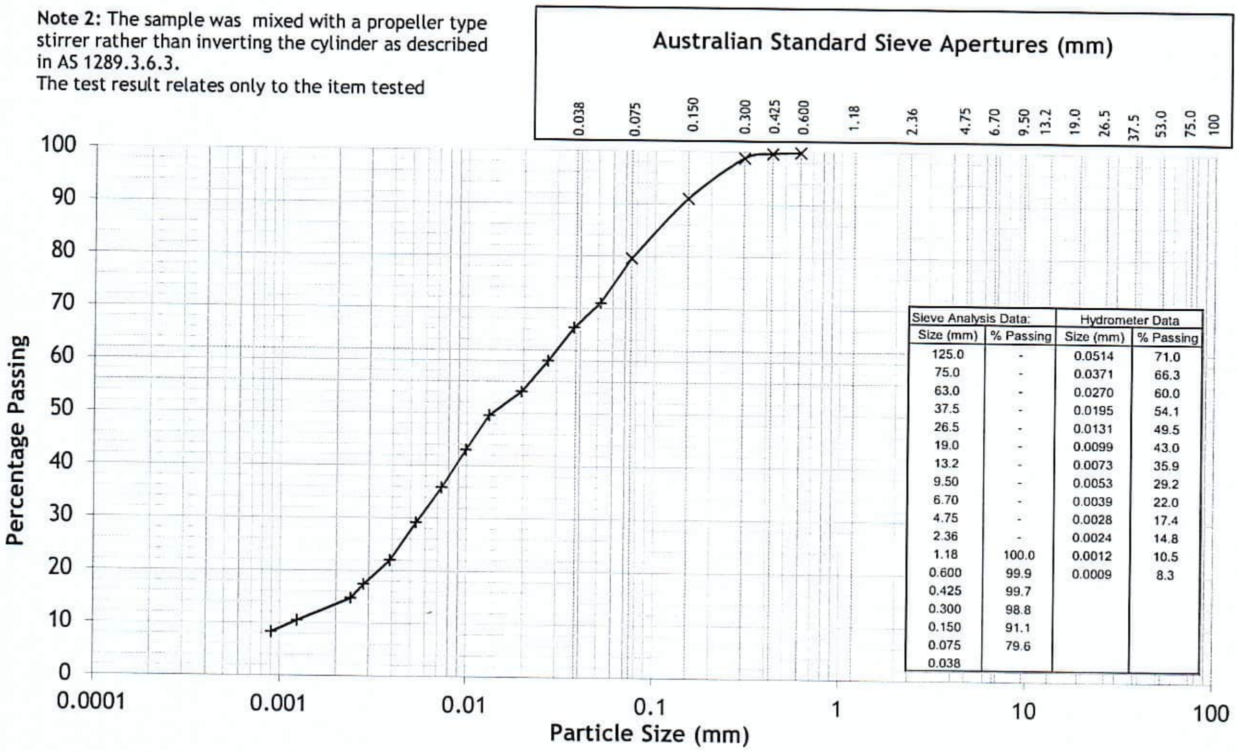
Sample Description: Combined Tailings	Borehole ☐ No:	Test Pit ☐ Depth:

Dispersion Method used: Mechanical Stirrer for 1 minute **Hydrometer Type Used:** ASTM 152H

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1 Clause 6.5.4 ☑ Sample provided by the Client

Note 1: The sample was oven-dried during sample preparation and not air-dried as stated in AS 1289.3.6.3

Note 2: The sample was mixed with a propeller type stirrer rather than inverting the cylinder as described in AS 1289.3.6.3.
The test result relates only to the item tested



Sieve Analysis Data:		Hydrometer Data	
Size (mm)	% Passing	Size (mm)	% Passing
125.0	-	0.0514	71.0
75.0	-	0.0371	66.3
63.0	-	0.0270	60.0
37.5	-	0.0195	54.1
26.5	-	0.0131	49.5
19.0	-	0.0099	43.0
13.2	-	0.0073	35.9
9.50	-	0.0053	29.2
6.70	-	0.0039	22.0
4.75	-	0.0028	17.4
2.36	-	0.0024	14.8
1.18	100.0	0.0012	10.5
0.600	99.9	0.0009	8.3
0.425	99.7		
0.300	98.8		
0.150	91.1		
0.075	79.6		
0.038			

CLAY	SILT			SAND			GRAVEL			COBBLES
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	

0.002 0.006 0.02 0.06 0.2 0.6 2 6 20 60

NATA ACCREDITED LABORATORY NUMBER: 3372

NATA ACCREDITED FOR TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025

Approved Signatory: [signature]

Date: 19/01/2017

Name of Signatory: John Walker

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FIGURE C3

Classification (Atterberg Limits)

TEST IN ACCORDANCE WITH AS 1289

ATC Williams
GROUNDED IN DESIGN

Client: Iron Bridge Operations Pty Ltd

Address: Level 2, 87 Adelaide Terrace East Perth, WA

Project: North Star Magnetite Project - Stage 2

NATA Report No.: R46314

Job No.: 114185.03

Location: WA

Register Number	Sample Description	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Sample Curled (CU) / Crumbled (CR)
25414	Fig 5. Combination of Primary Wet LIMS from Batch Plant and Regrind Wet LIMS.	21	17	4	-	-
25514	Fig 6. Combination of Primary Wet LIMS and Secondary Wet LIMS from Batch Plant.	19	17	2	-	-

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1

☒ Sample provided by the client

The test results relate only to the items tested.

Test Methods:

- ☐ Liquid Limit AS 1289.3.1.1 (Standard method)
- ☒ Liquid Limit AS 1289.3.1.2 (Subsidiary method)
- ☒ Plastic Limit AS 1289.3.2.1
- ☒ Plasticity Index AS 1289.3.3.1
- ☐ Linear Shrinkage AS 1289.3.4.1
- ☒ Moisture Content AS 1289.2.1.1

Sample Preparation:

- ☐ Natural Moisture ☒ Air Dried ☐ Oven Dried ☐ Unknown
- ☐ Wet Sieved ☒ Dry Sieved ☐ Unsieved

NATA ACCREDITED FOR TECHNICAL COMPETENCE

NATA ACCREDITED LABORATORY NUMBER: 3372

Accredited for compliance with ISO/IEC 17025

Approved Signatory .... Date 18th December 2014

Name of Signatory John Walker

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ATC Williams unites the companies of Australian Tailings Consultants and MPA Williams & Associates

FIGURE C4

Classification (Atterberg Limits)

TEST IN ACCORDANCE WITH AS 1289

ATC Williams

Client: Iron Bridge Operations Pty Limited

NATA Report No.: R00417

Address: Level 2, 87 Adelaide Terrace, East Perth WA 6004

Job No.: 114185.08

Project: North Star Project - 2016 Tailings Testing

Location: WA

Register Number	Sample Description	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Sample Curled (CU) Crumbled (CR) Cracked (CK)
00417	Combined Tailings	18	16	2	-	-

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1

☒ Sample provided by the client

The test results relate only to the items tested.

Test Methods:

- ☐ Liquid Limit AS 1289.3.1.1 (Standard method)
- ☒ Liquid Limit AS 1289.3.1.2 (Subsidiary method)
- ☒ Plastic Limit AS 1289.3.2.1
- ☒ Plasticity Index AS 1289.3.3.1
- ☐ Linear Shrinkage AS 1289.3.4.1
- ☒ Moisture Content AS 1289.2.1.1

Sample Preparation:

☐ Natural Moisture ☒ Air Dried ☐ Oven Dried ☐ Unknown

☐ Wet Sieved ☒ Dry Sieved ☐ Unsieved

NATA ACCREDITED FOR TECHNICAL COMPETENCE

NATA ACCREDITED LABORATORY NUMBER: 3372

Accredited for compliance with ISO/IEC 17025

Approved Signatory .... Date 19/01/2017

Name of Signatory: John Walker

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FIGURE C5

Determination of the Soil Particle Density of a Soil

TEST IN ACCORDANCE WITH AS 1289.3.5.1

ATC Williams
GROUNDED IN DESIGN

Client: Iron Bridge Operations Pty Ltd ....

NATA Report No.: R46014 ....

Address: Level 2, 87 Adelaide Terrace East Perth WA ....

Job No.: 114185.03 ....

....

Project: North Star Project-Stage 2....................................

Location: WA ....

Register Number	Sample Description	Test Temperature (°C)	% of Sample >2.36 mm	Particle Density (g/cm^3)
21214	Bulk fresh non-magnetic coarse cobbing (-3.35mm)	24.2	0	3.03
25314	Fig 2 -Combination of minus 1mm coarse cobbing rejects from batch plant	23.9	0	3.01
25414	Fig 5 Combination of primary wet LIMS from batch plant and regrind wet LIMS	23.8	0	2.97
25514	Fig 6 Combination of Primary Wet LIMS and Secondary ewt LIMS from batch plant	23.4	0	2.97
25614	Combination of minus 3mm coarse rejects and primary Wet LIMS & Reground wet LIMS from batch plant	23.2	0	2.97
25714	Combination of minus 0.5mm coarse rejects & primary wet LIMS & reground wet LIMS from batch plant	23.3	0	2.98

Notes:

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1

☒ Sample provided by the client

* = apparent average soil particle density - particle size less than 2.36 mm

X = apparent average soil particle density - particle size greater than 2.36 mm

= soil particle density of the total sample

The test results relate only to the items tested.

NATA ACCREDITED FOR TECHNICAL COMPETENCE

NATA ACCREDITED LABORATORY NUMBER: 3372

Accredited for compliance with ISO/IEC 17025

Approved Signatory .... Date December 17th, 2014 ....

Name of Signatory Peter Lam ....

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ATC Williams unites the companies of Australian Tailings Consultants and MPA Williams & Associates

FIGURE C6

Determination of the Soil Particle Density of a Soil

ATC Williams

TEST IN ACCORDANCE WITH AS 1289.3.5.1

Client: Iron Bridge Operations Pty Limited........................ **NATA Report No.: R00517** ....

Address: Level 2, 87 Adelaide Terrace, East Perth............... **Job No.:** 114185.08 ....

WA 6004..

Project: North Star Project - 2016 Tailings Testing.............. **Location:** WA......................

Register Number	Sample Description	Test Temperature (°C)	% of Sample >2.36 mm	Particle Density (g/cm^3)
00417	Combined Tailings Sample	24.5	None	3.04 #

Notes:

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1

☒ Sample provided by the client

* = apparent average soil particle density - particle size less than 2.36 mm

X = apparent average soil particle density - particle size greater than 2.36 mm

= soil particle density of the total sample

The test results relate only to the items tested.

NATA ACCREDITED FOR TECHNICAL COMPETENCE

NATA ACCREDITED LABORATORY NUMBER: 3372

Accredited for compliance with ISO/IEC 17025

Approved Signatory .... Date 20/01/2017 ....

Name of Signatory John Walker..................................

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FIGURE C7

Segregation Threshold of Tailings

ATC Williams

TEST IN ACCORDANCE WITH - IN HOUSE PROCEDURE IHM 5.0

Client: Iron Bridge Operations Pty Limited

Laboratory Report No.: R00317

Address: Level 2, 87 Adelaide Terrace, East Perth WA 6004

Job No.: 114185.08

Project: North Star Project - 2016 Tailings Testing

Location: WA

Sample Register Number	Sample Description	Condition	Segregation Threshold Solids Concentration (%)
00417	Combined Tailings	Flocculated with Magnafloc 338 at a dose rate of 15g/t	58 (static)

Notes:

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1.

☒ Sample provided by the client

The test results relate only to the items tested.

Note:
Segregation Threshold is carried out in accordance with ATC Williams In House Test Procedure IHM 5.0 and relates to the minimum solids concentration at which coarse particles do not settle out of the slurry in a static condition. Shearing however can induce segregation in slurries at solids concentrations above the static threshold point.

This document is issued in accordance with the laboratory quality system requirements of ATC Williams Pty Ltd.

Signatory .... Date 19/01/2017

Name of Signatory John Walker

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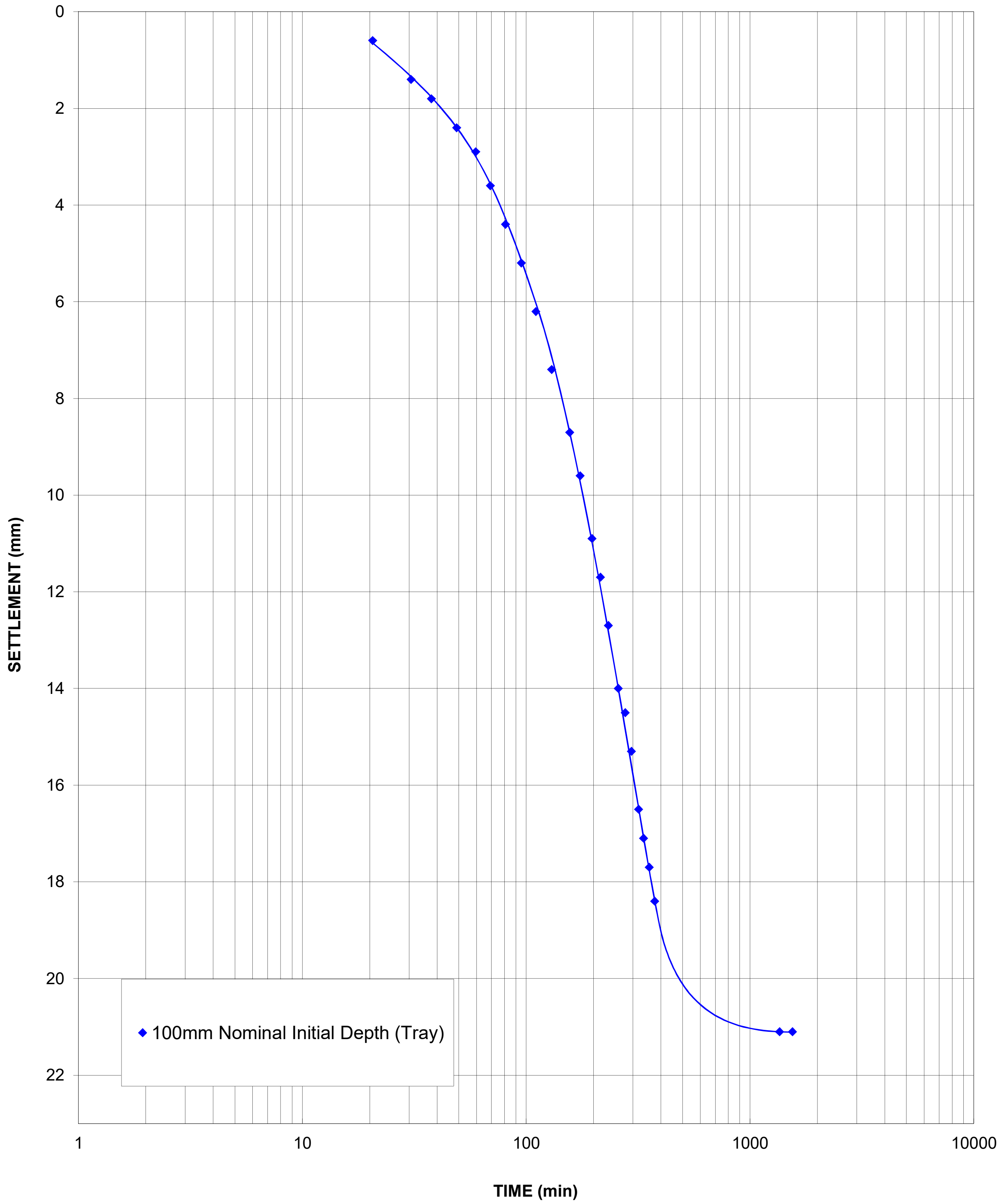
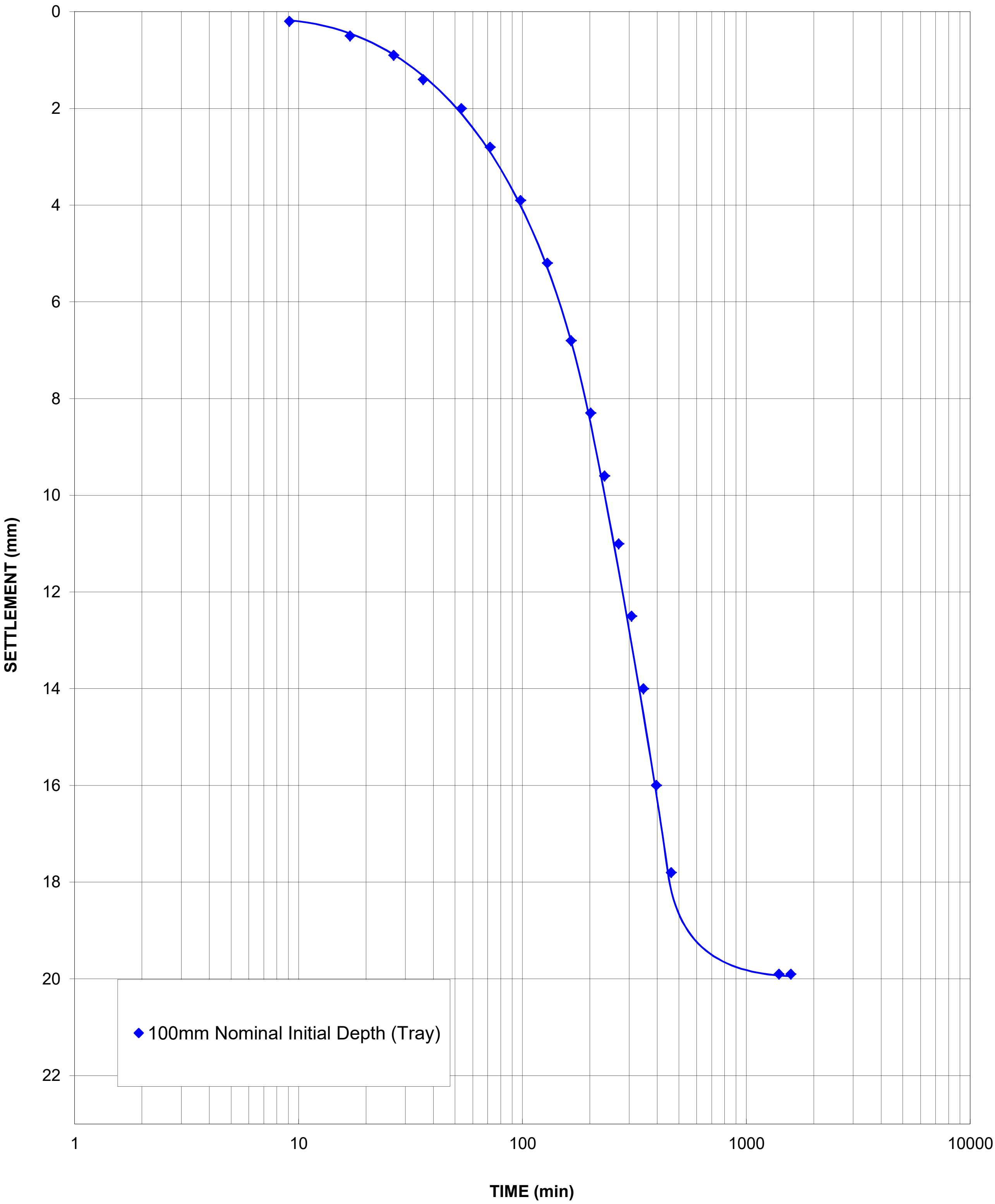
FIGURE C8

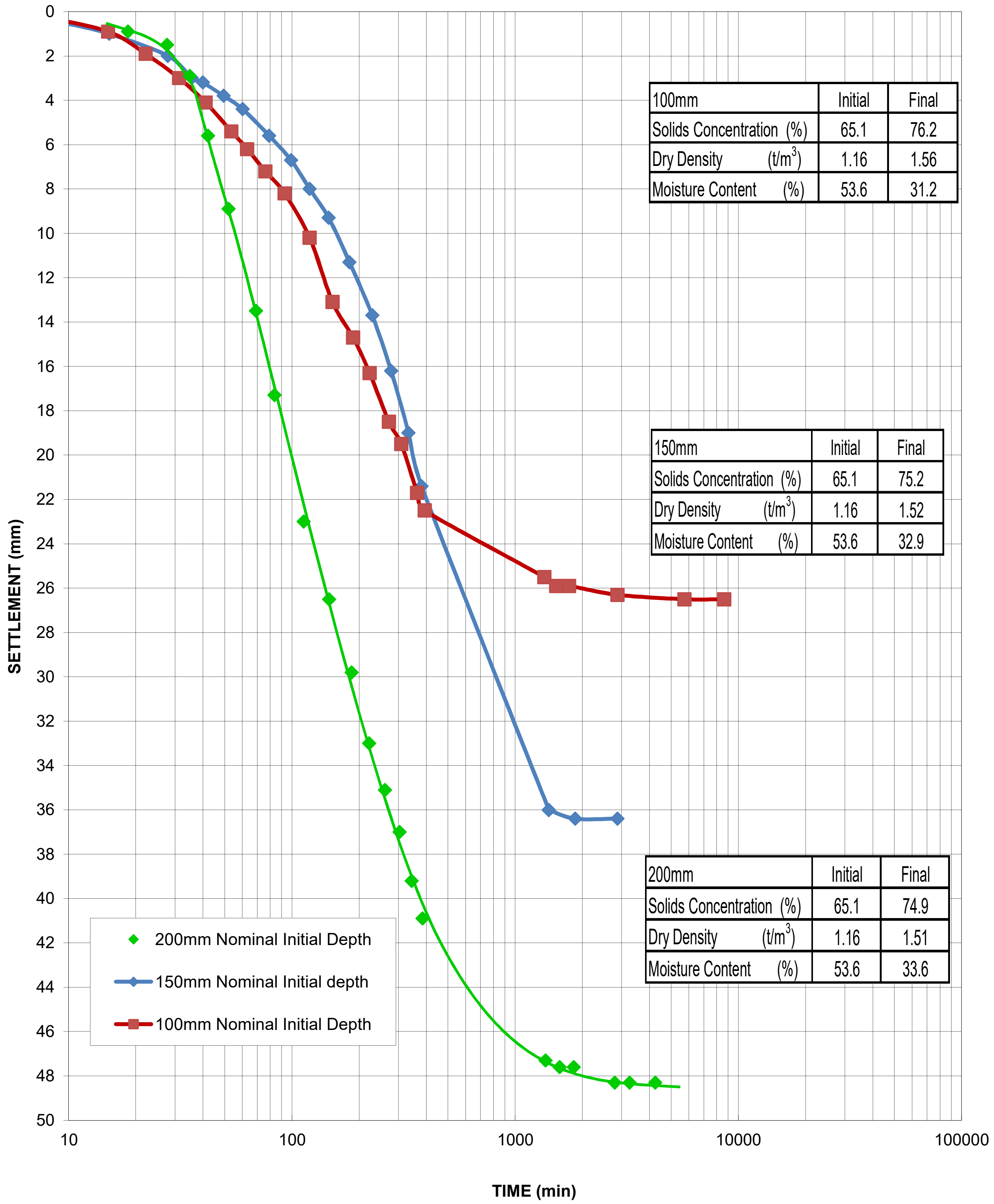


IRON BRIDGE OPERATIONS PTY LTD North Star Project- Stage 2 Figure 5 -Flocculated Sample	INITIAL SETTLED DENSITY 64.9 INITIAL SOLIDS
ATC Williams GROUNDED IN DESIGN ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	Job No: 114185.03 Date: 30/01/2015 FIGURE C9



IRON BRIDGE OPERATIONS PTY LTD North Star Project- Stage 2 Figure 6 -Flocculated Sample	INITIAL SETTLED DENSITY 65.1% INITIAL SOLIDS
ATC Williams GROUNDED IN DESIGN ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	**Job No:** 114185.03 **Date:** 30/01/2015 **FIGURE C10**

:\2014\114185.08 North Star Project\114185.03 - North Star Magnetite\Draft Lab Report\Figures



100mm	Initial	Final
Solids Concentration (%)	65.1	76.2
Dry Density (t/m^3)	1.16	1.56
Moisture Content (%)	53.6	31.2

150mm	Initial	Final
Solids Concentration (%)	65.1	75.2
Dry Density (t/m^3)	1.16	1.52
Moisture Content (%)	53.6	32.9

200mm	Initial	Final
Solids Concentration (%)	65.1	74.9
Dry Density (t/m^3)	1.16	1.51
Moisture Content (%)	53.6	33.6

Iron Bridge Operations Pty Ltd

North Star Project

Combined Tailings

INITIAL SETTLED DENSITY

INITIAL SETTLED DENSITY

65 % INITIAL SOLIDS

Job No: 114185.08

Date: 25/09/2018

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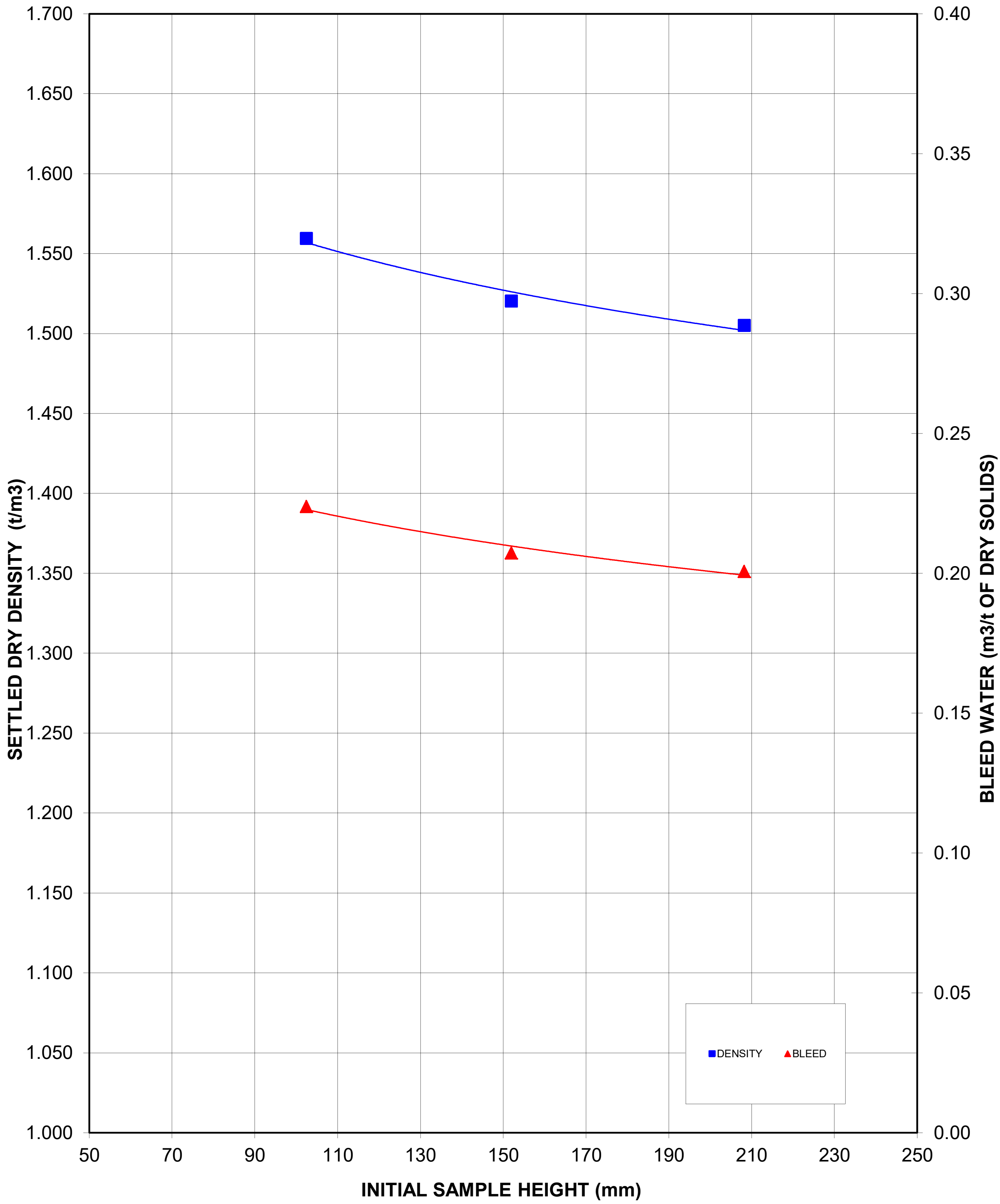
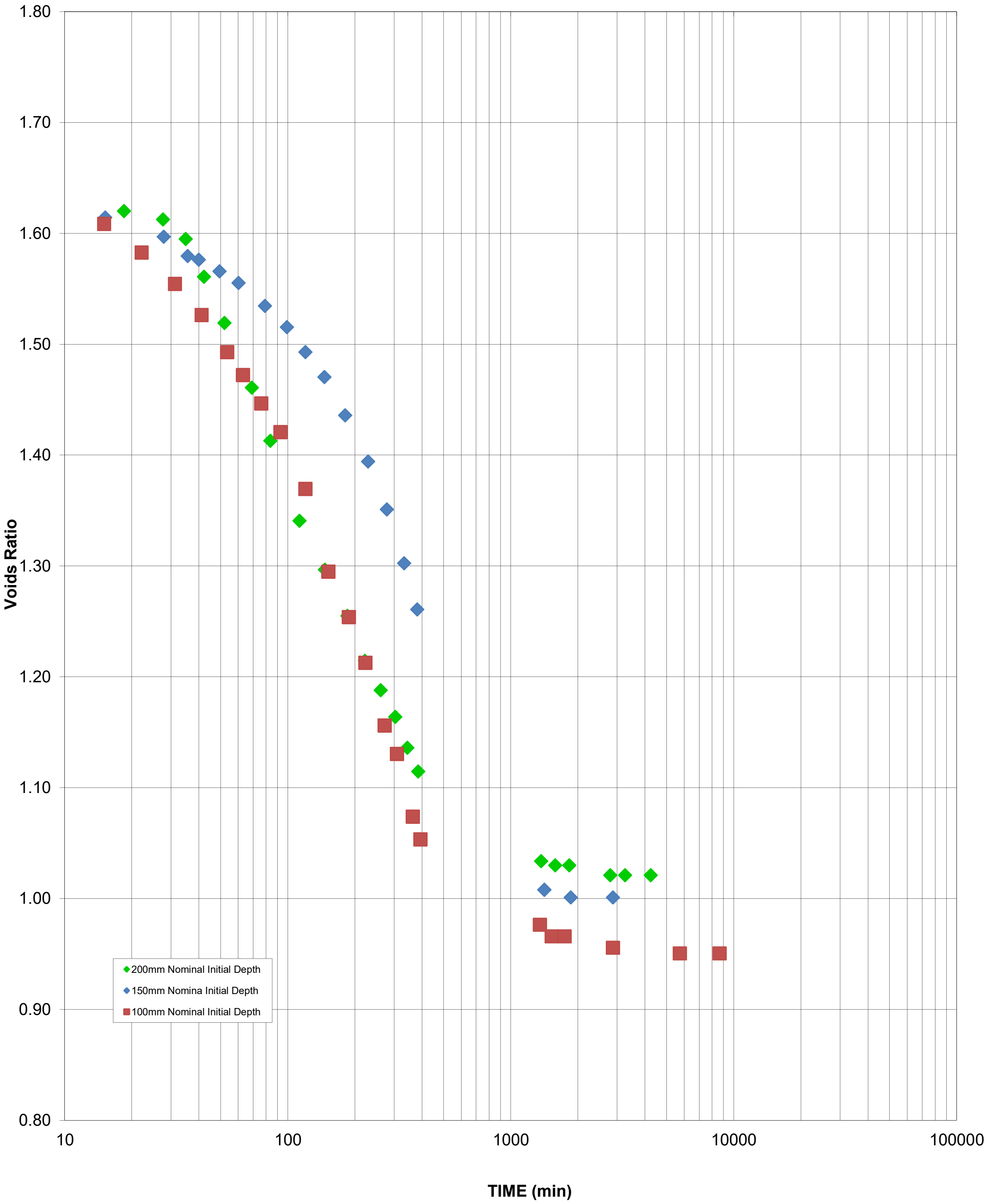
FIGURE C11

:\2014\114185.08 North Star Project\00417-Tray Settlement 200,150,100 mm



Iron Bridge Operations Pty Ltd North Star Project Combined Tailings	INITIAL SETTLED DENSITY DENSITY AND BLEED 65 % INITIAL SOLIDS
ATC Williams GROUNDED IN DESIGN ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	**Job No:** 114185.08 **Date:** 25/09/2018 **FIGURE C12**

..\2014\114185.08 North Star Project\00417-Tray Settlement 200,150,100 mm



Iron Bridge Operations Pty Ltd

North Star Project

Combined Tailings

INITIAL SETTLED DENSITY

INITIAL SETTLED DENSITY

65 % INITIAL SOLIDS

Job No: 114185.08

Date: 25/09/2018

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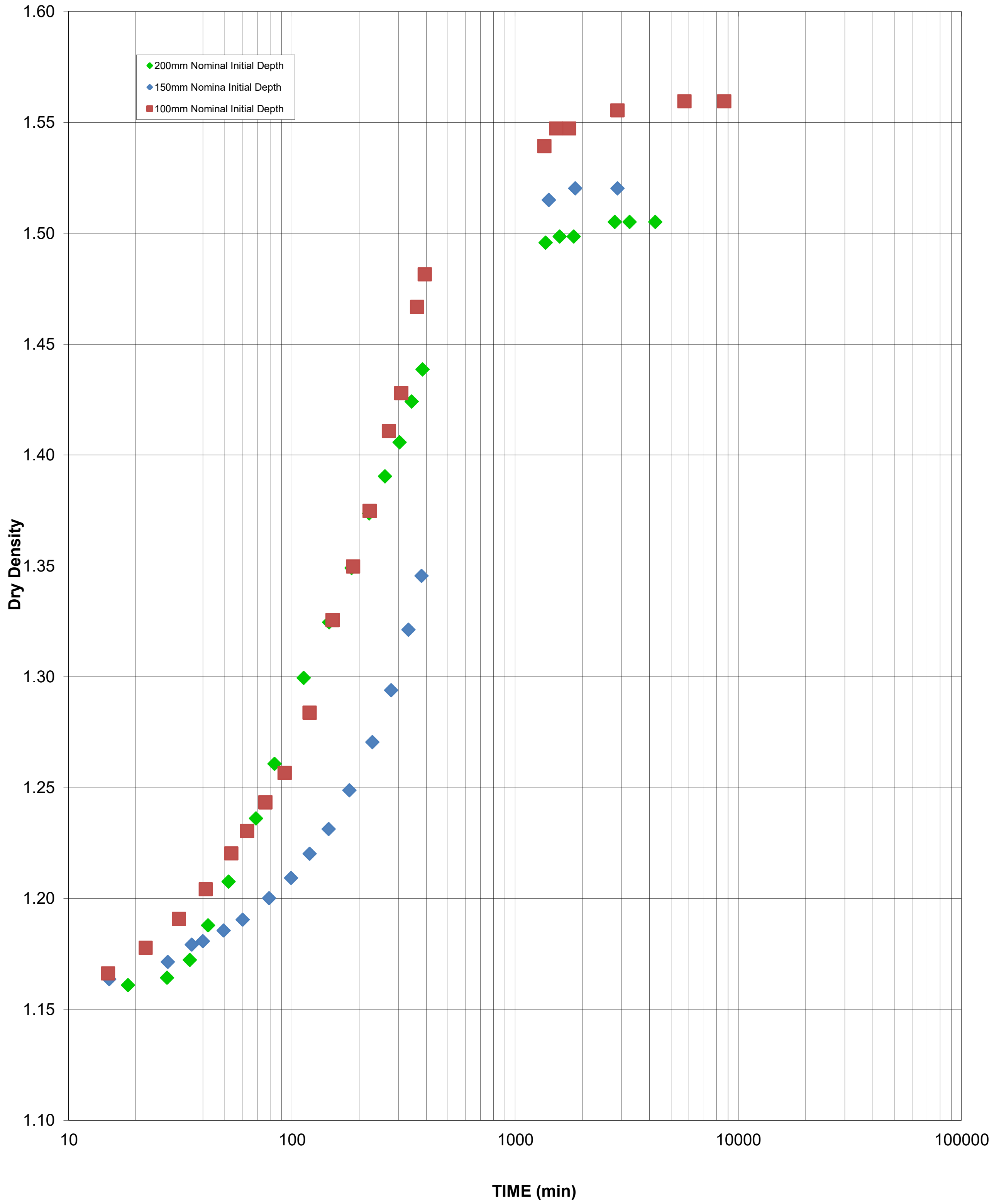
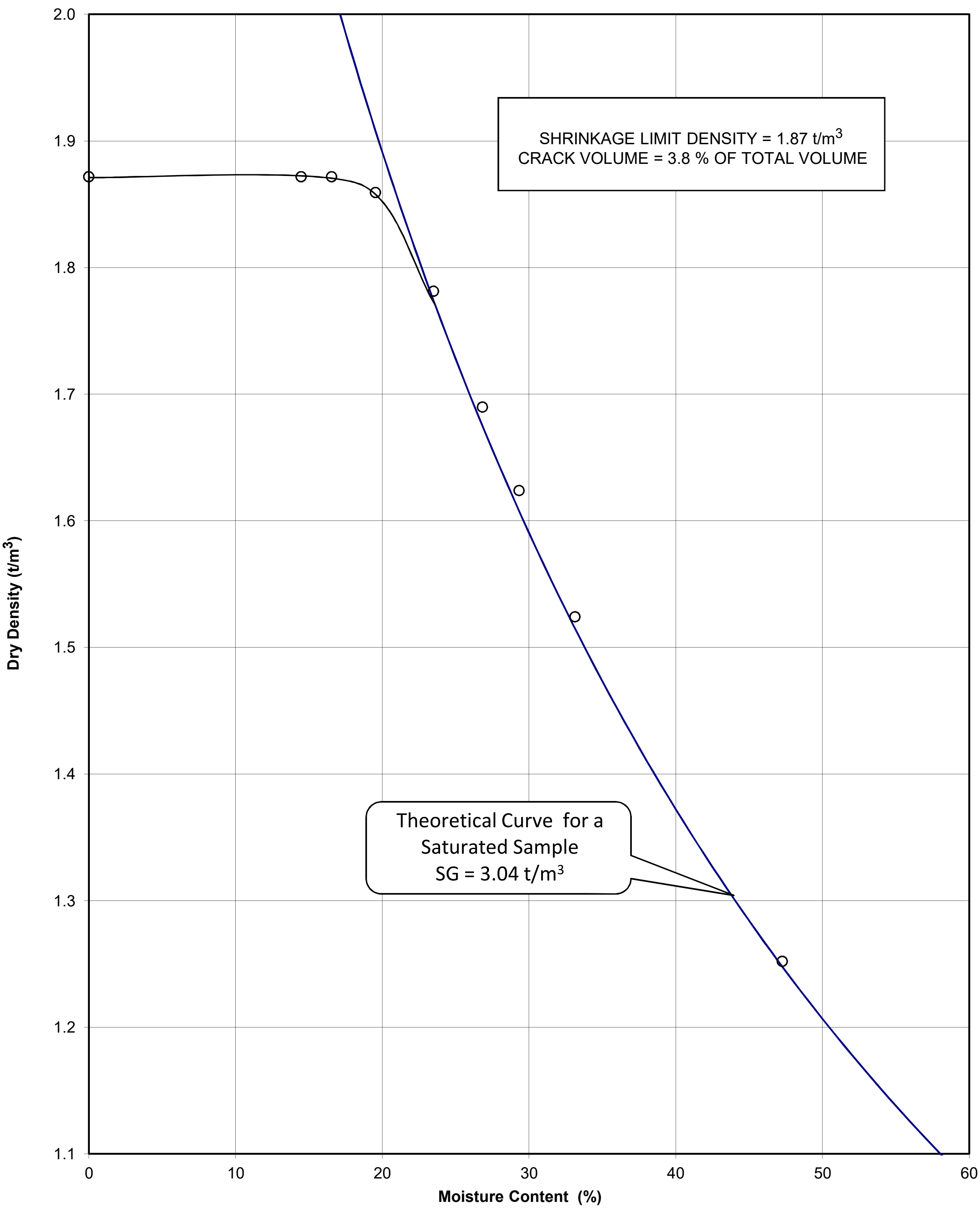
FIGURE C13

..\2014\114185.08 North Star Project\00417-Tray Settlement 200,150,100 mm



Iron Bridge Operations Pty Ltd North Star Project Combined Tailings	INITIAL SETTLED DENSITY INITIAL SETTLED DENSITY 65 % INITIAL SOLIDS
ATC Williams GROUNDED IN DESIGN ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	Job No: 114185.08 Date: 25/09/2018 **FIGURE C14**

..\2014\114185.08 North Star Project\00417-Tray Settlement 200,150,100 mm



IRON BRIDGE OPERATIONS

NORTH STAR PROJECT-2016 TAILINGS TESTING

Flocculated Tailings

Shrinkage Limit Density

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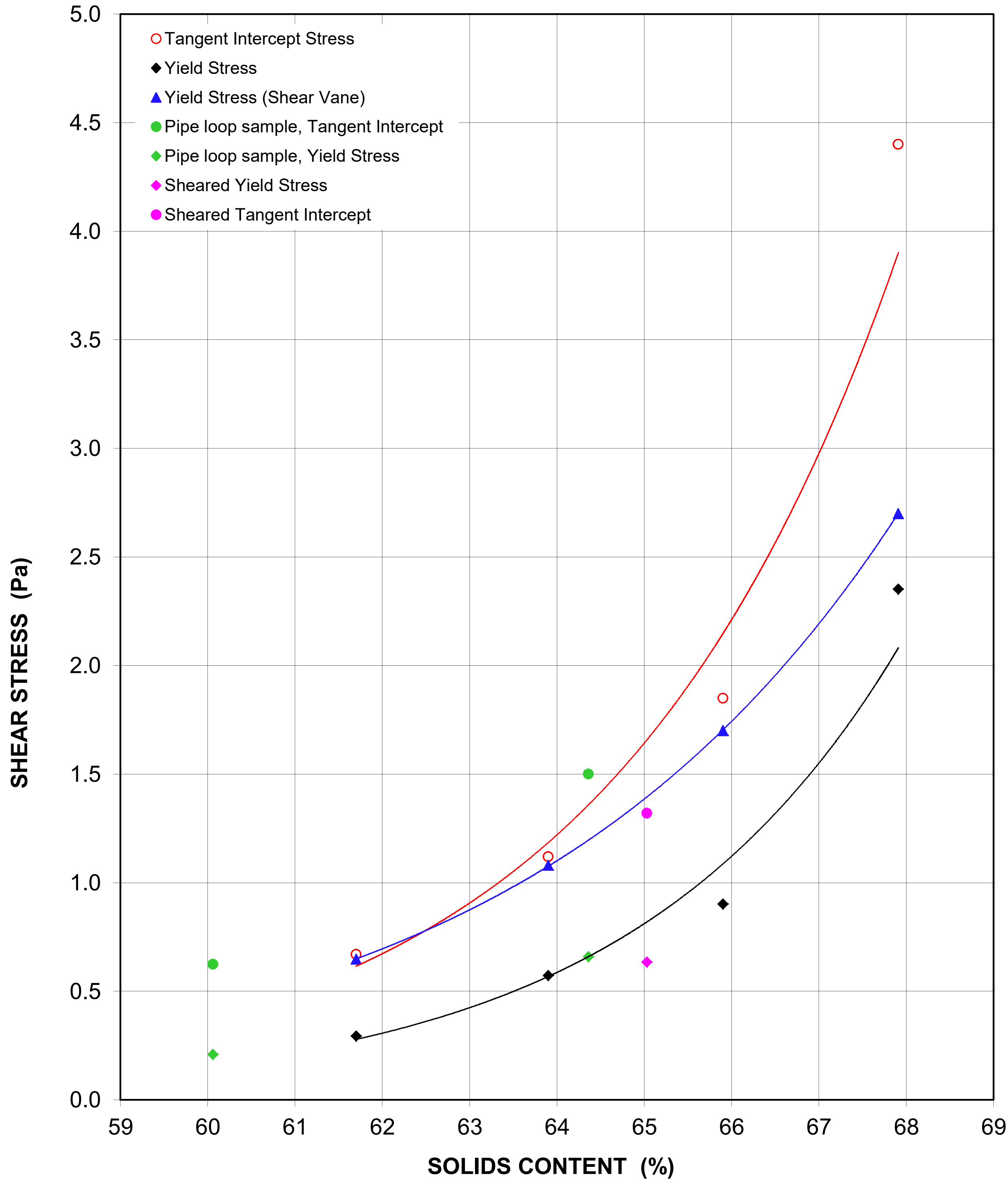
Job No: 114185.08

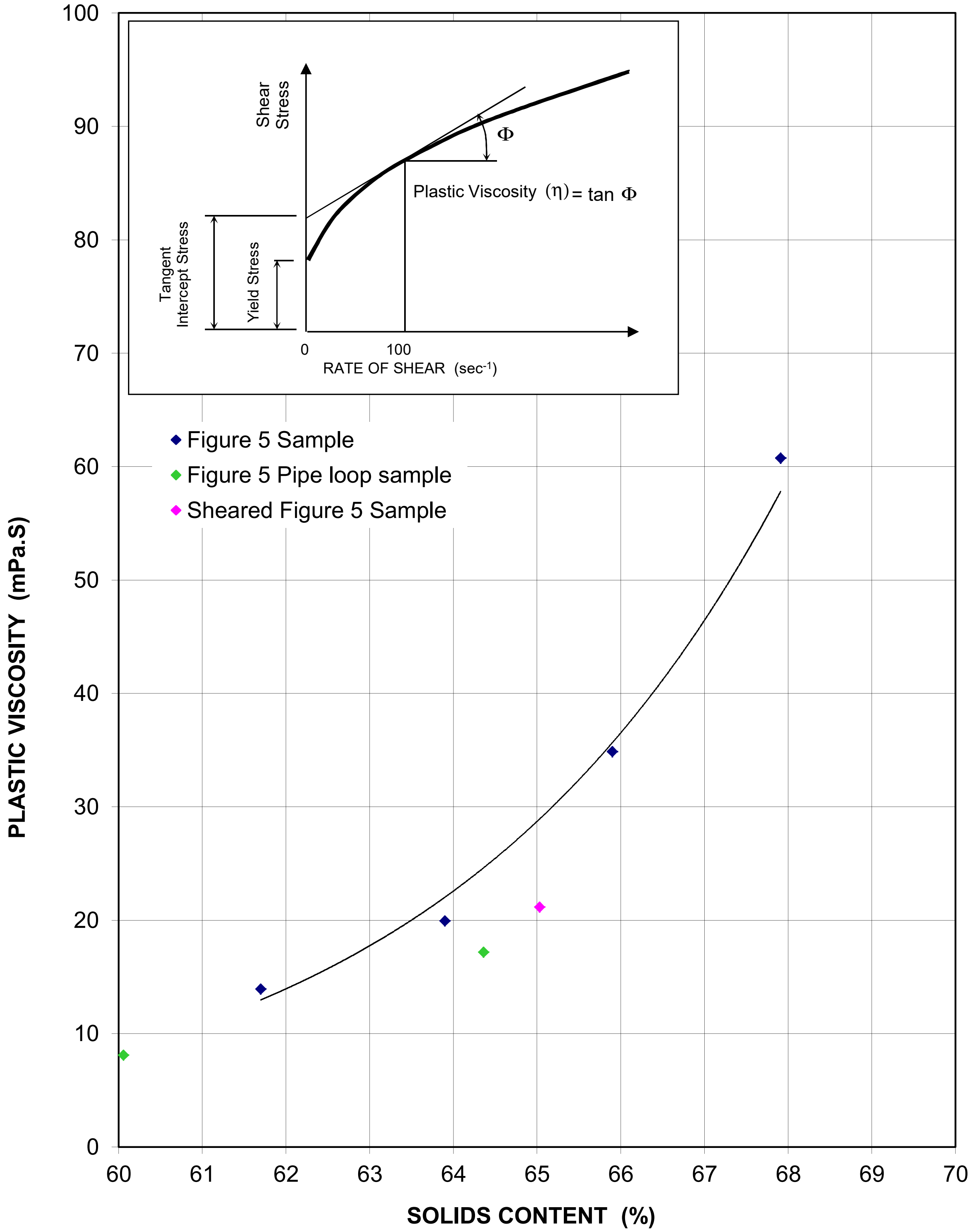
Date: 17/02/2017

FIGURE C15



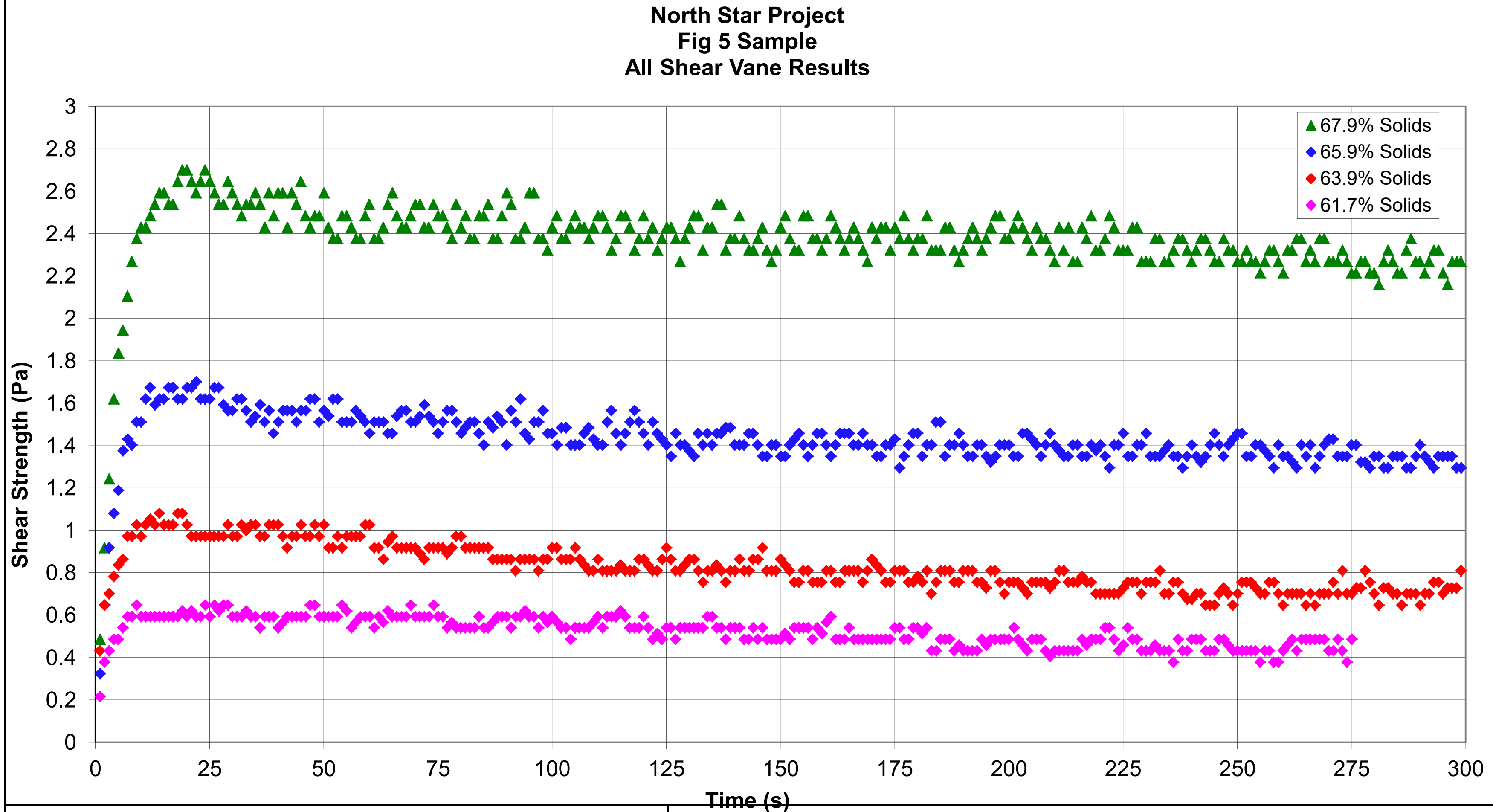
Iron Bridge Operations Pty Ltd North Star Project - Stage 2 Fig 5 Sample	Rheology Testing Yield Stress Versus Solids Concentration
ATC Williams ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	**Job No:** 114185.03 **Date:** 4/02/2015 **FIGURE C16**

:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Documents\R03 (Design Report)\Appendices\C - Tailings .ab Testing\Lab Sheets\Individual\Rheology\Figure 5\26214 - Rheology



Iron Bridge Operations Pty Ltd North Star Project - Stage 2 Fig 5 Sample	Rheology Testing Plastic Viscosity Versus Solids Concentration
ATC Williams ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	**Job No:** 114185.03 **Date:** 4/02/2015 **FIGURE C17**

:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Documents\R03 (Design Report)\Appendices\C - Tailings .ab Testing\Lab Sheets\Individual\Rheology\Figure 5\26214 - Rheology



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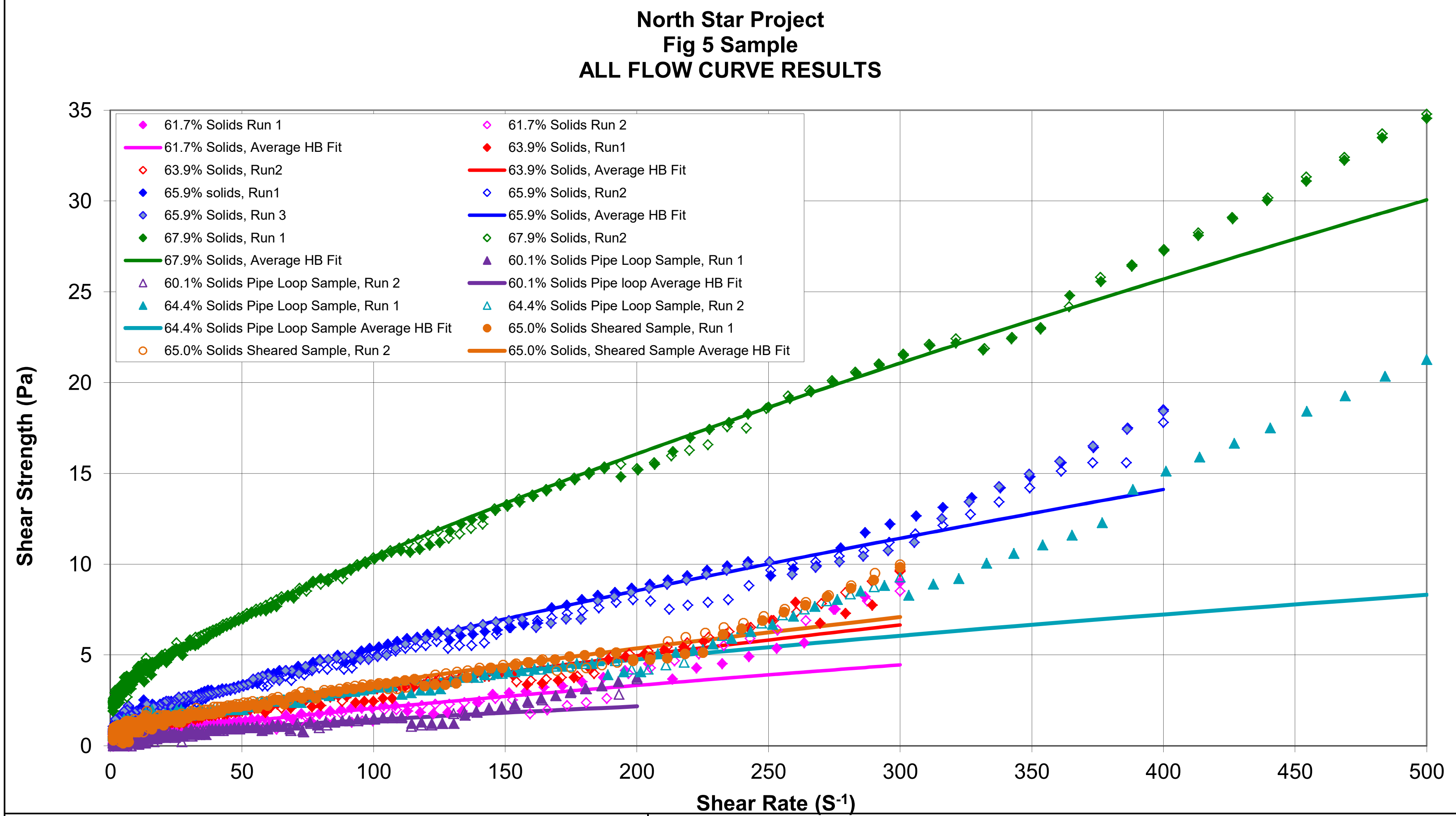
Iron Bridge Operations Pty Ltd

North Star Project - Stage 2

Fig 5 Sample

Tailings Rheology

Date:	15/02/2017	Job No:	114185.08	FIGURE C18



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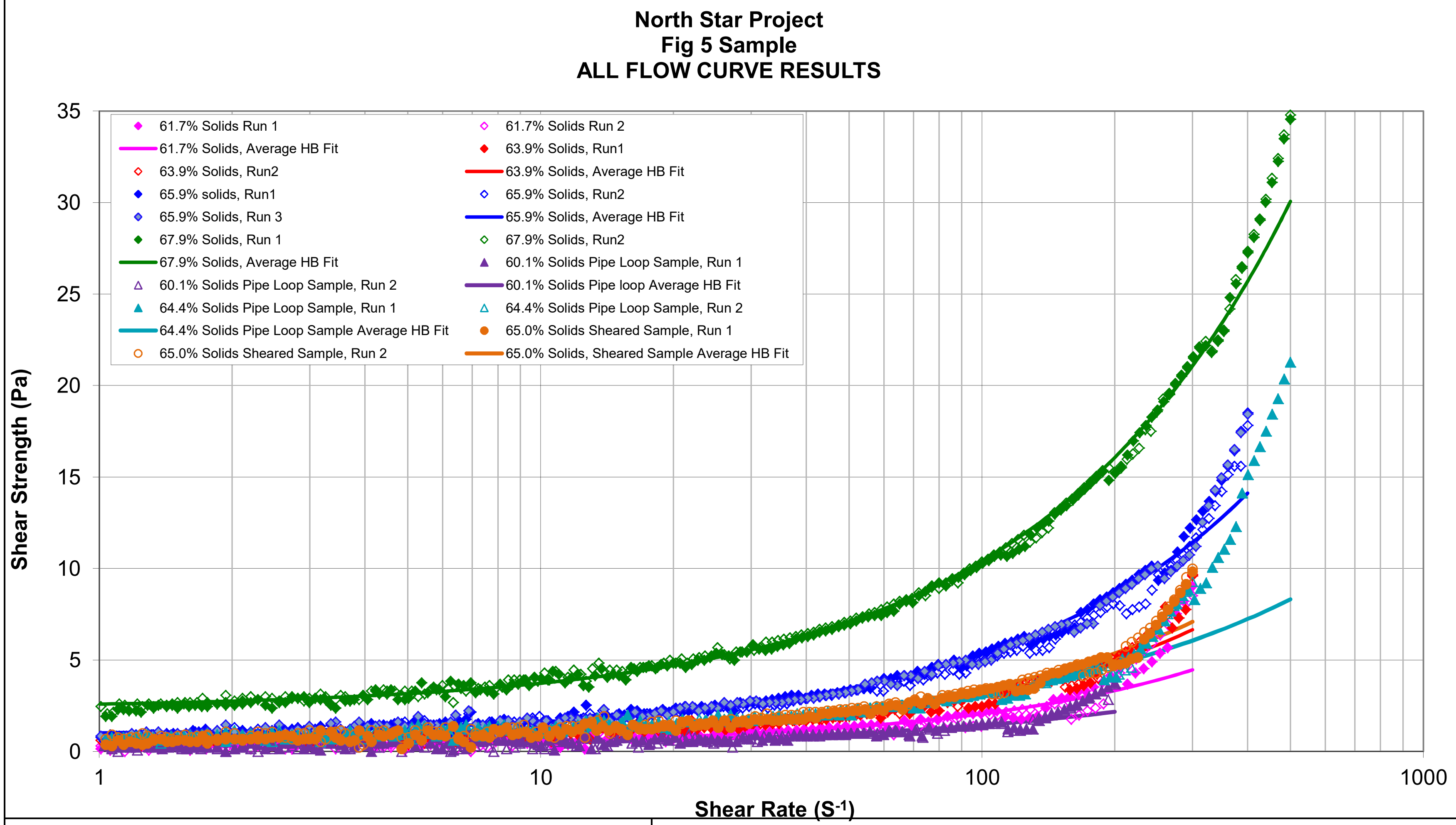
North Star Project - Stage 2

Fig 5 Sample

Tailings Rheology

Date:	15/02/2017	Job No:	114185.08	FIGURE C19



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Iron Bridge Operations Pty Ltd

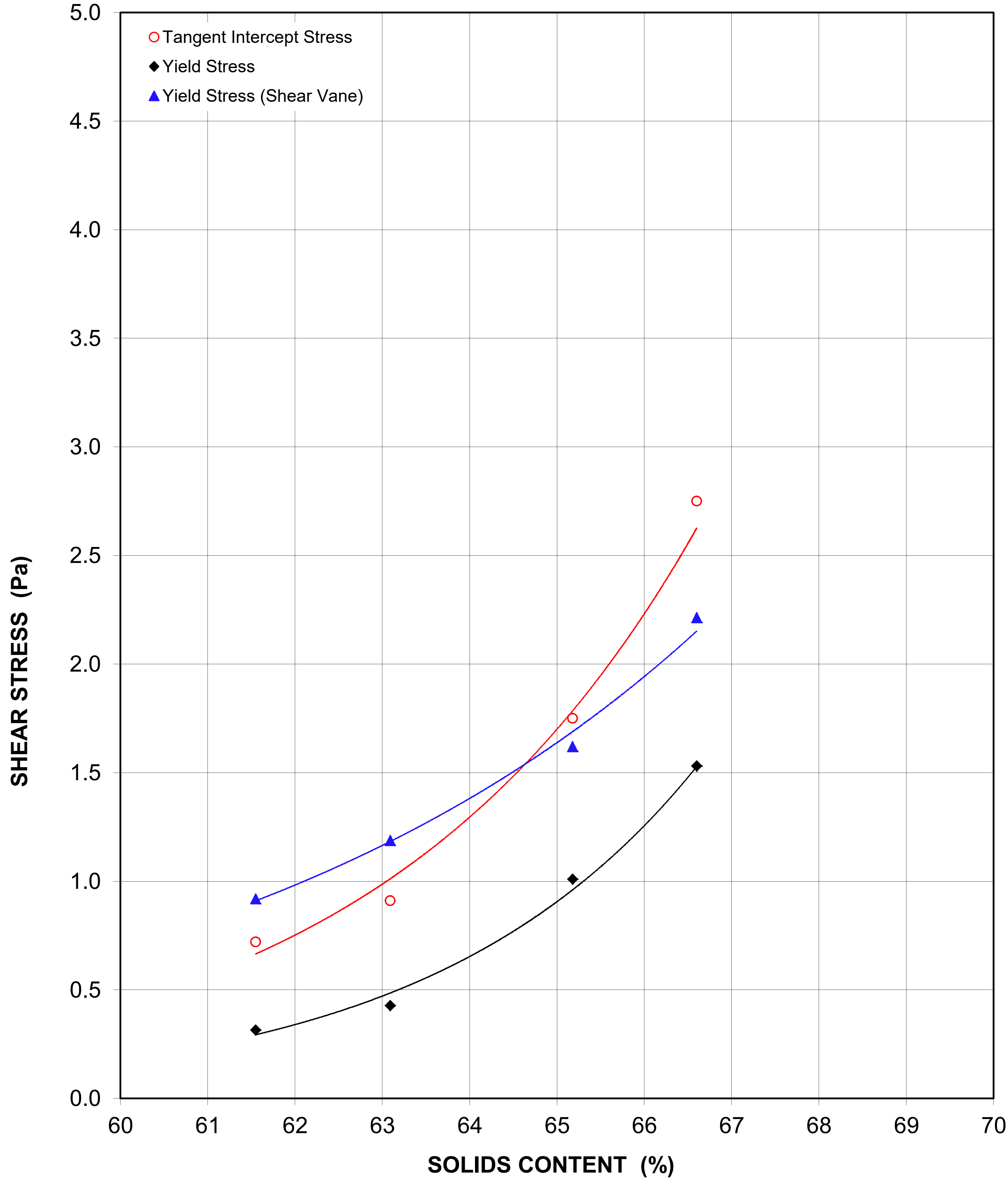
North Star Project - Stage 2

Fig 5 Sample

Tailings Rheology

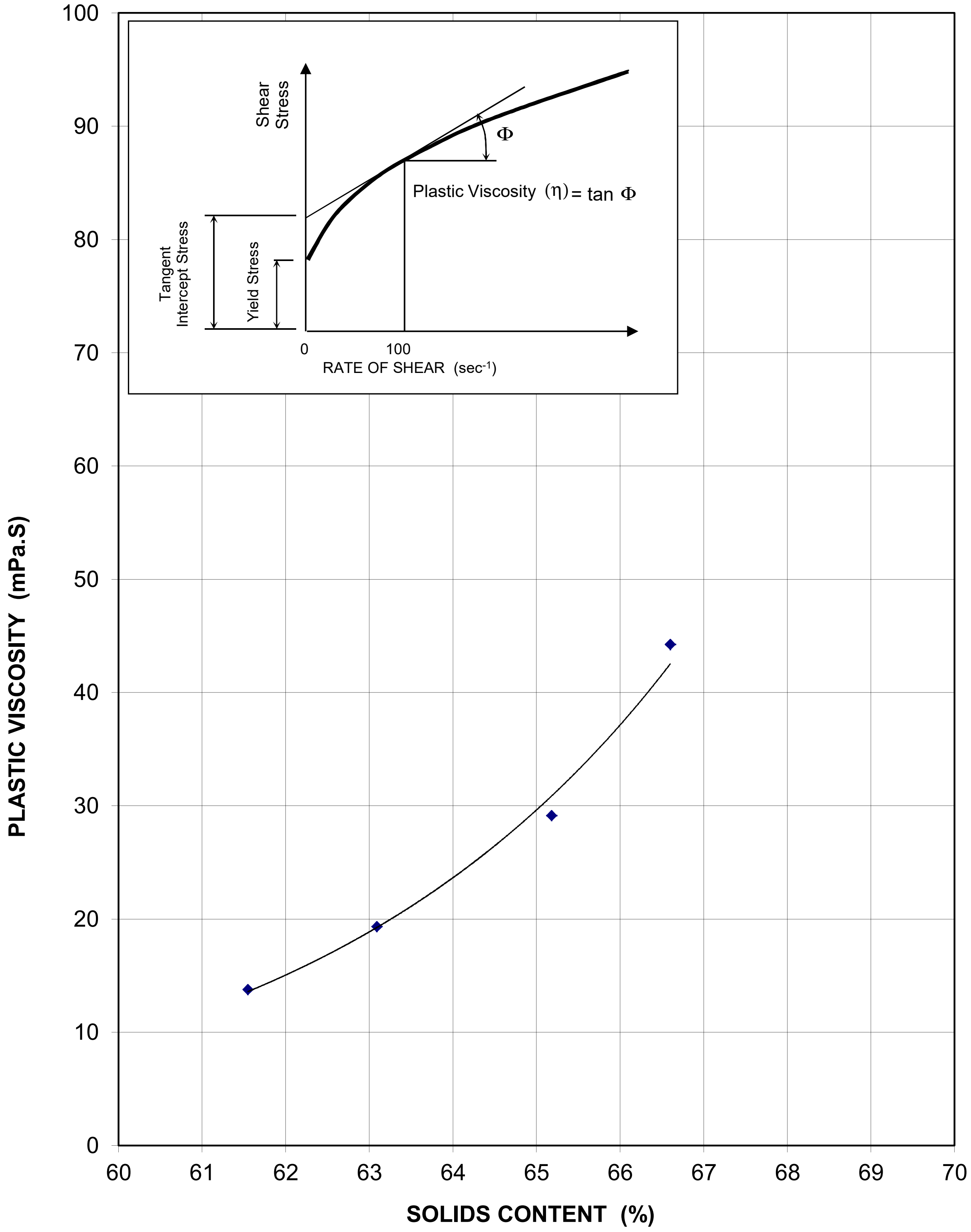
Date: 15/02/2017 **Job No:** 114185.08

FIGURE C20



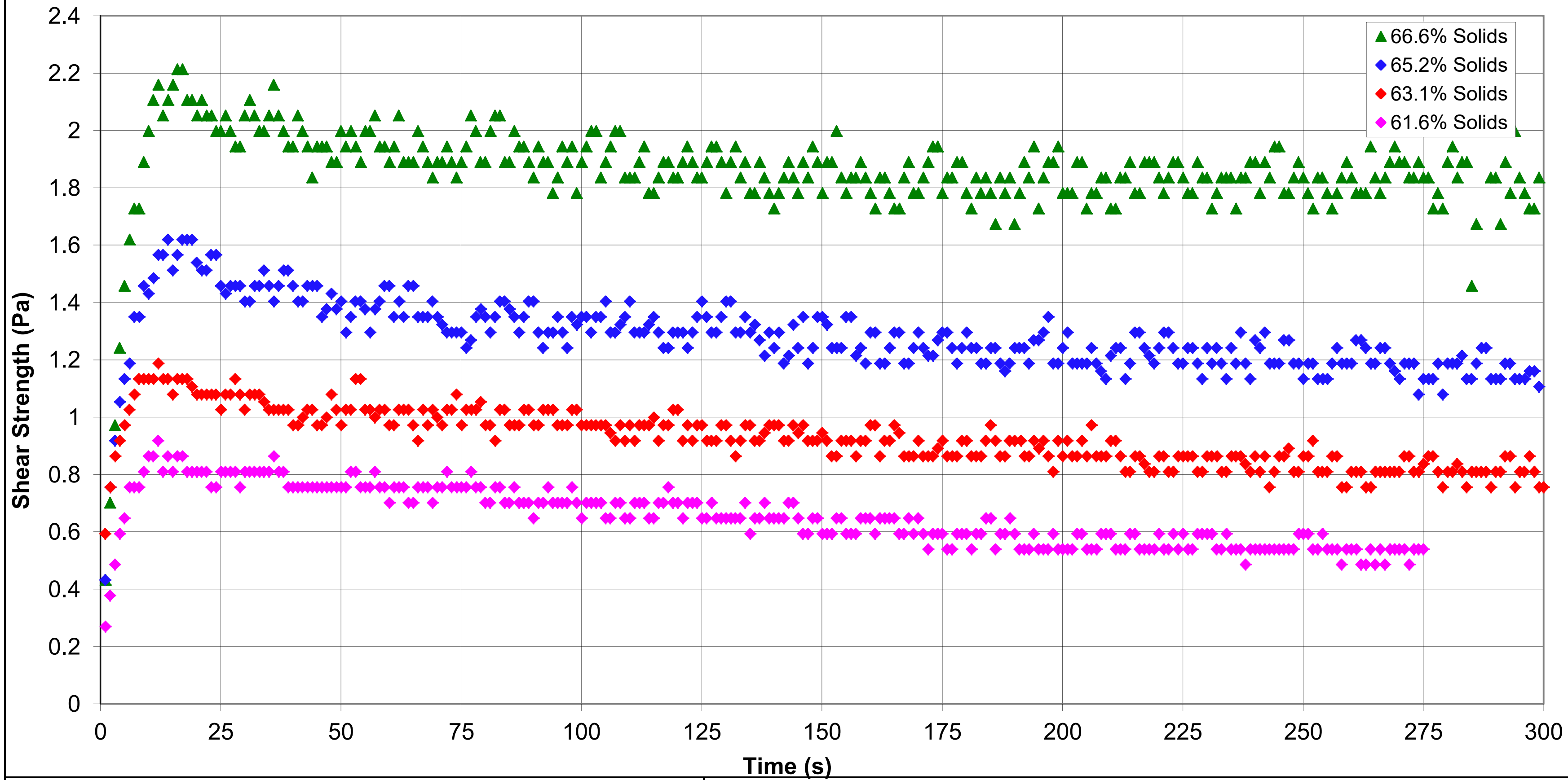
Iron Bridge Operations Pty Ltd North Star Project - Stage 2 Fig 6 Sample	Rheology Testing Yield Stress Versus Solids Concentration
ATC Williams ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	**Job No:** 114185.03 **Date:** 9/01/2015 **FIGURE C21**

:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Documents\R03 (Design Report)\Appendices\C - Tailings .ab Testing\Lab Sheets\Individual\Rheology\Figure 6\26314 - Rheology



Iron Bridge Operations Pty Ltd **North Star Project - Stage 2** **Fig 6 Sample**	**Rheology Testing** **Plastic Viscosity Versus** **Solids Concentration**
ATC Williams ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	**Job No:** 114185.03 **Date:** 9/01/2015 **FIGURE C22**

North Star Project
Fig 6 Sample
All Shear Vane Results



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Iron Bridge Operations Pty Ltd

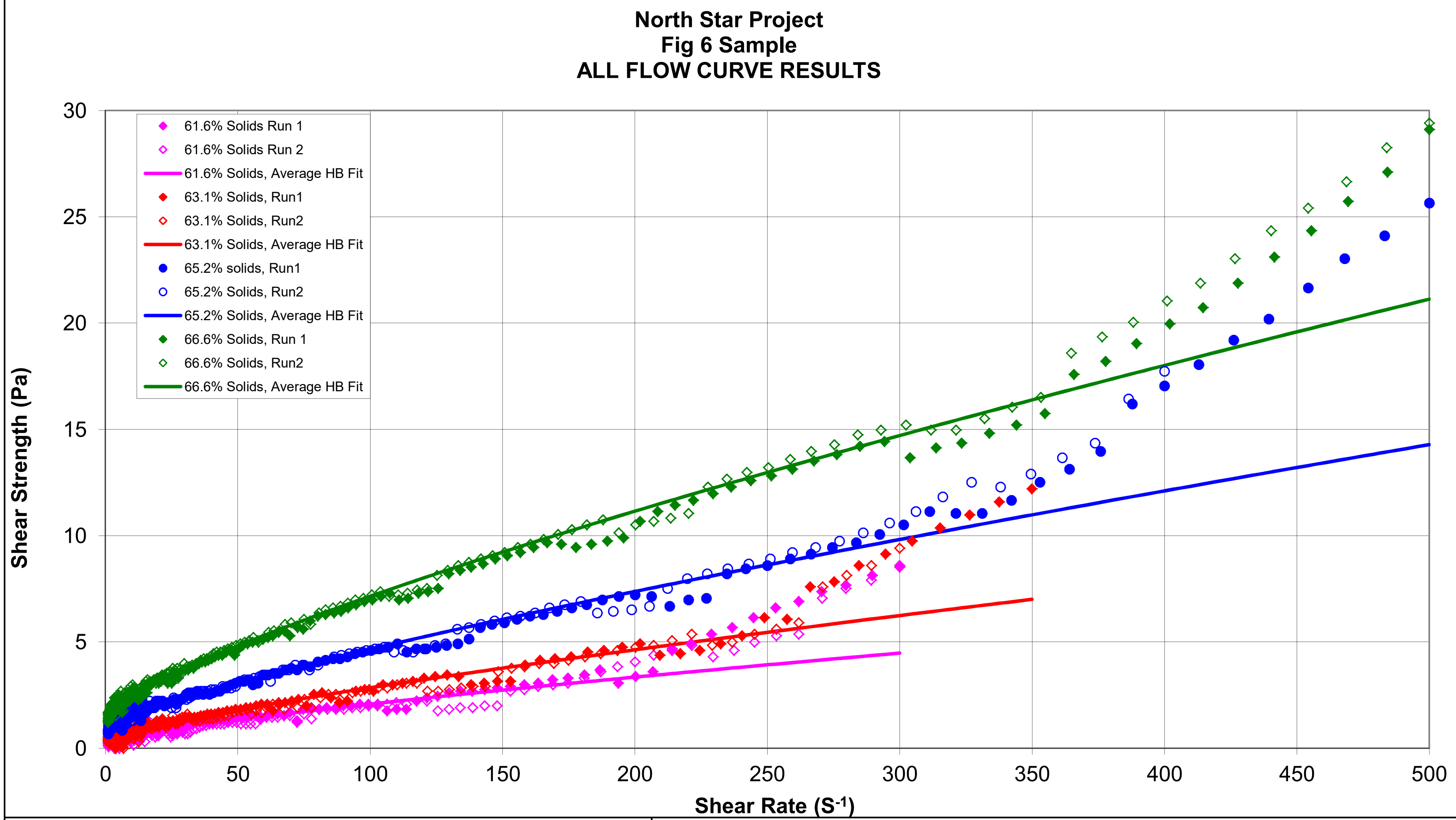
North Star Project - Stage 2

Fig 6 Sample

Tailings Rheology

Date: 15/02/2017 **Job No:** 114185.08

FIGURE C23



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Iron Bridge Operations Pty Ltd

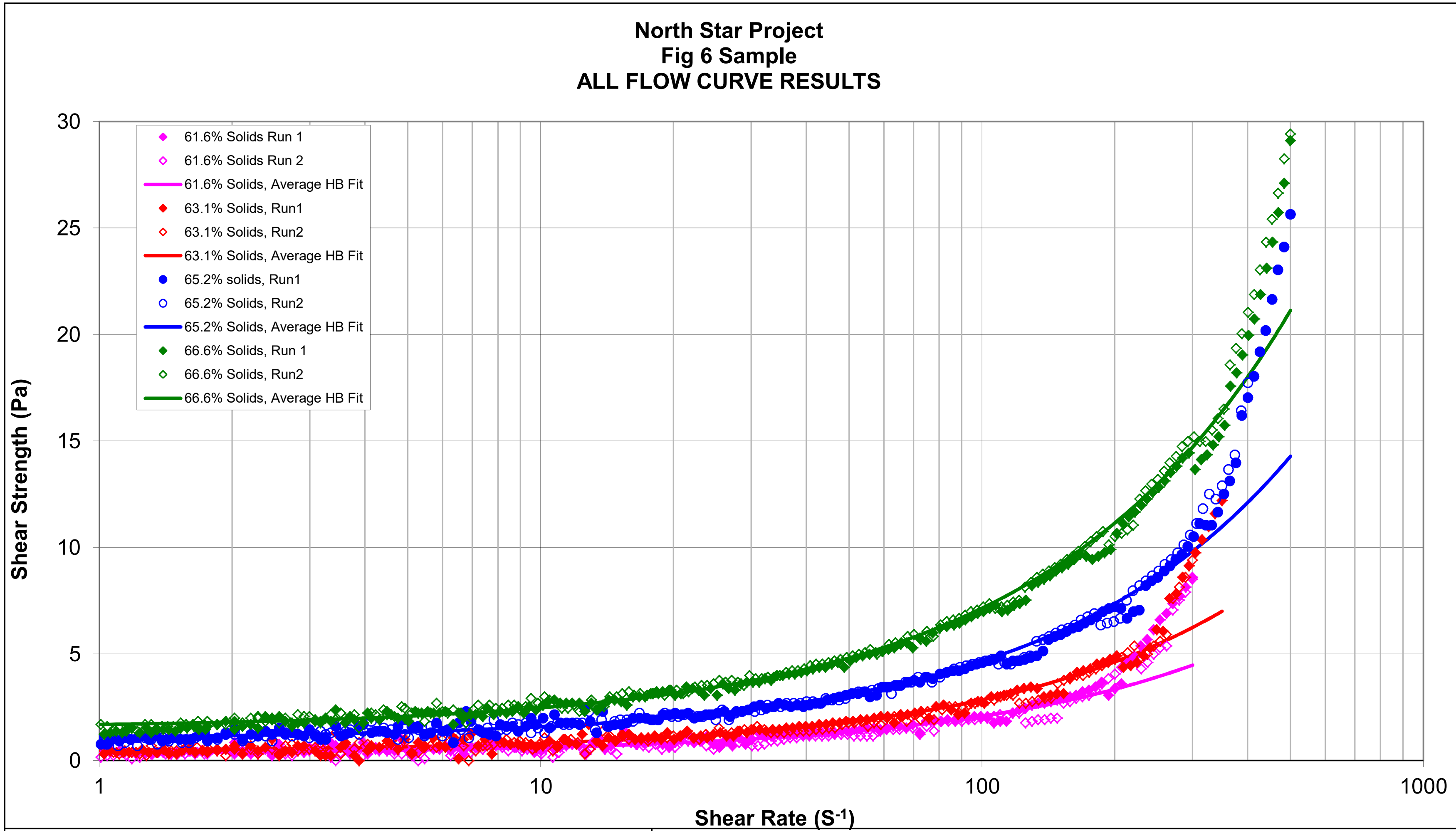
North Star Project - Stage 2

Fig 6 Sample

Tailings Rheology

Date: 15/02/2017 **Job No:** 114185.08

FIGURE C24



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Iron Bridge Operations Pty Ltd

North Star Project - Stage 2

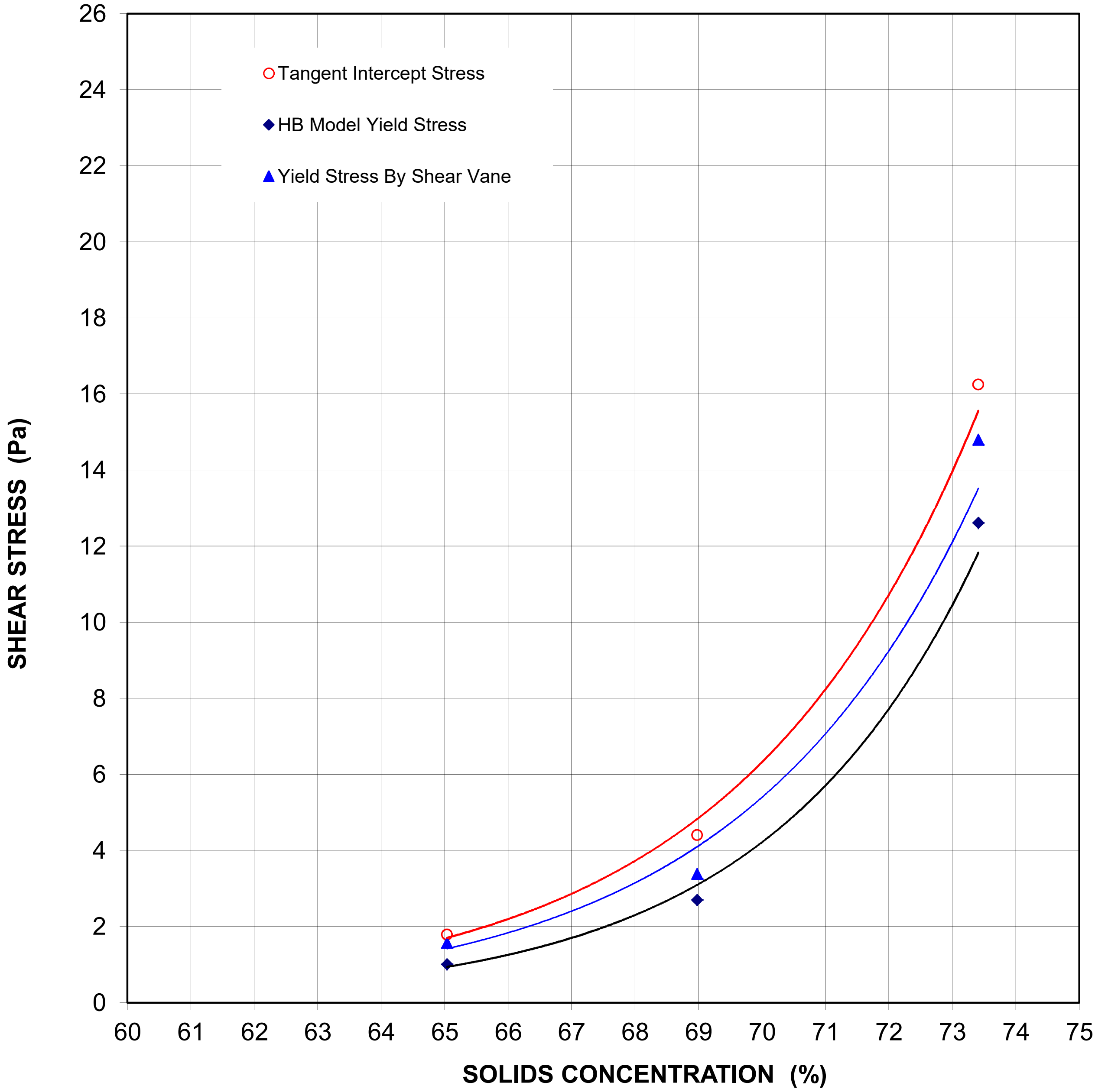
Fig 6 Sample

Tailings Rheology

Date: 15/02/2017 **Job No:** 114185.08

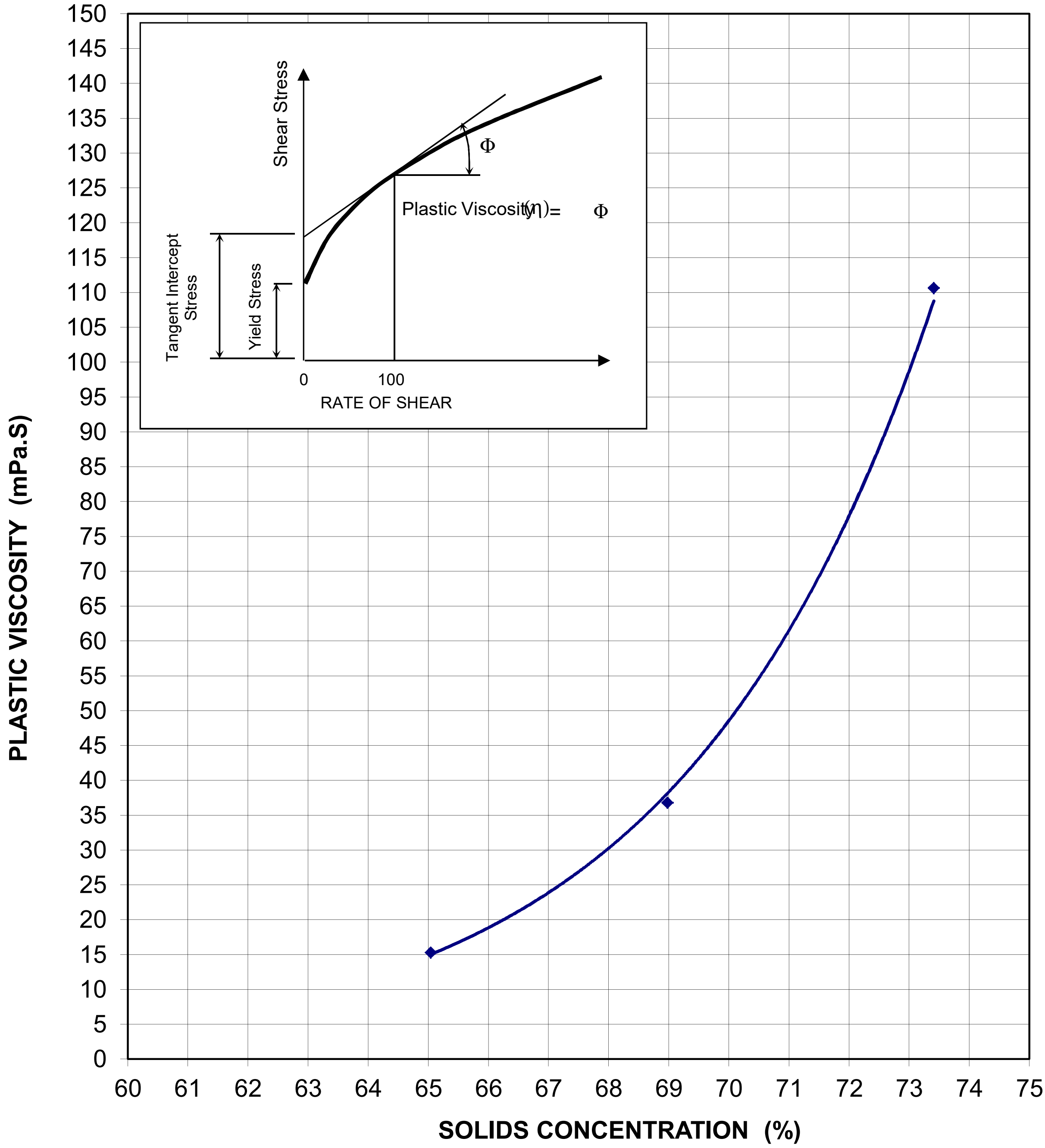
FIGURE C25

114185.08 - North Star Project - Tailings
Flocculated Combined Tailings Sample
SHEAR STRESS

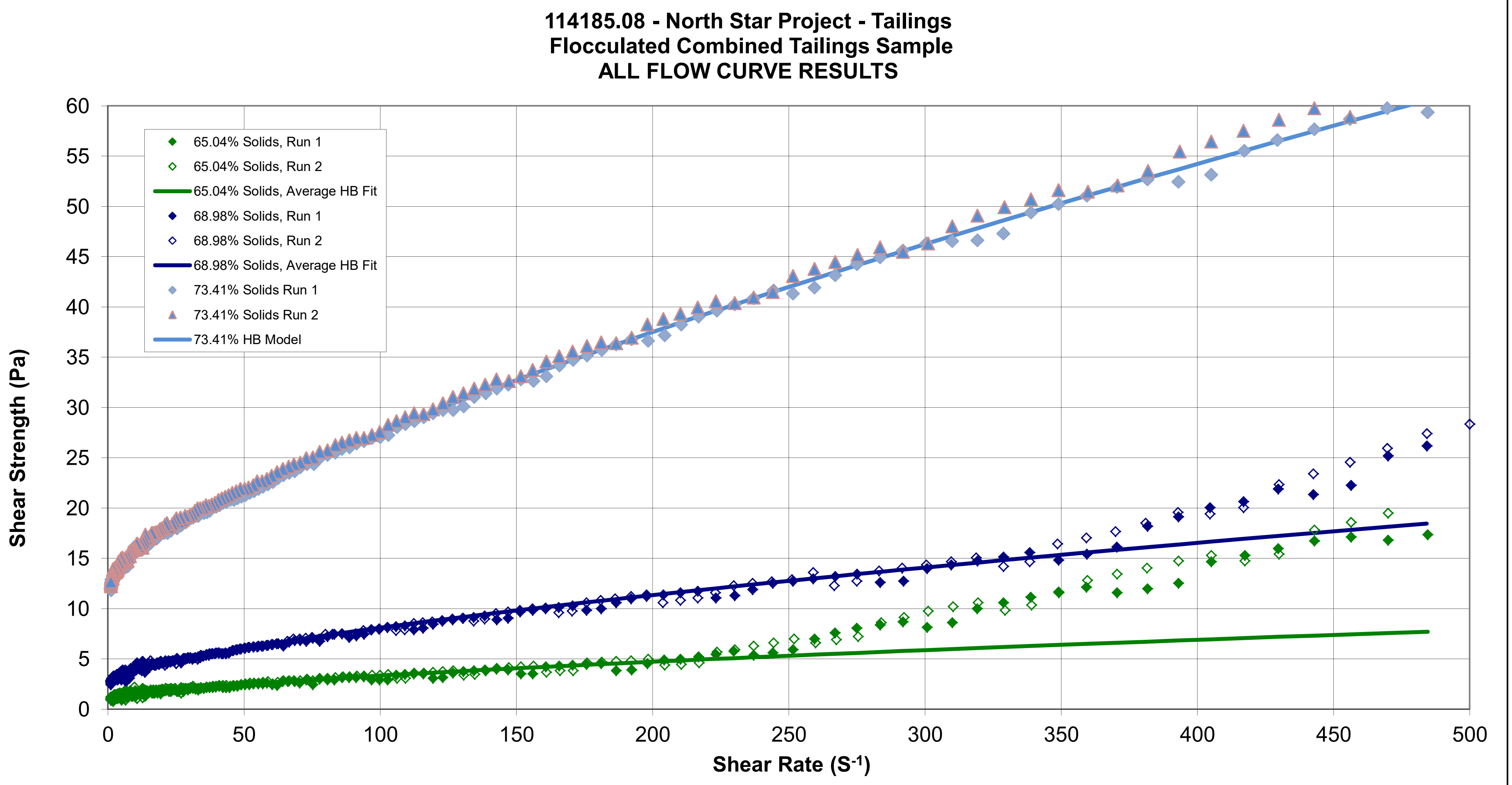


Iron Bridge Operations Pty Ltd North Star Project - 2016 Tailings Testing Flocculated Combined Tailings	Rheology Testing Yield Stress Versus Solids Concentration
ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	**Job No:** 114185.08 **Date:** 15/02/2017 **FIGURE C26**

114185.08 - North Star Project - Tailings
Flocculated Combined Tailings Sample
PLASTIC VISCOSITY



Iron Bridge Operations Pty Ltd North Star Project - 2016 Tailings Testing Flocculated Combined Tailings	Rheology Testing Plastic Viscosity Versus Solids Concentration
ATC Williams ATC Williams Pty Ltd 19 Beach Avenue Mordialloc Vic 3195 AUSTRALIA T +61 3 9590 9222 F +61 3 9590 9228 melb@atcwilliams.com.au www.atcwilliams.com.au	Job No: 114185.08 Date: 15/02/2017 **FIGURE C27**



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Iron Bridge Operations Pty ltd

North Star Project - 2016 Tailings Testing

Flocculated Combined Tailings

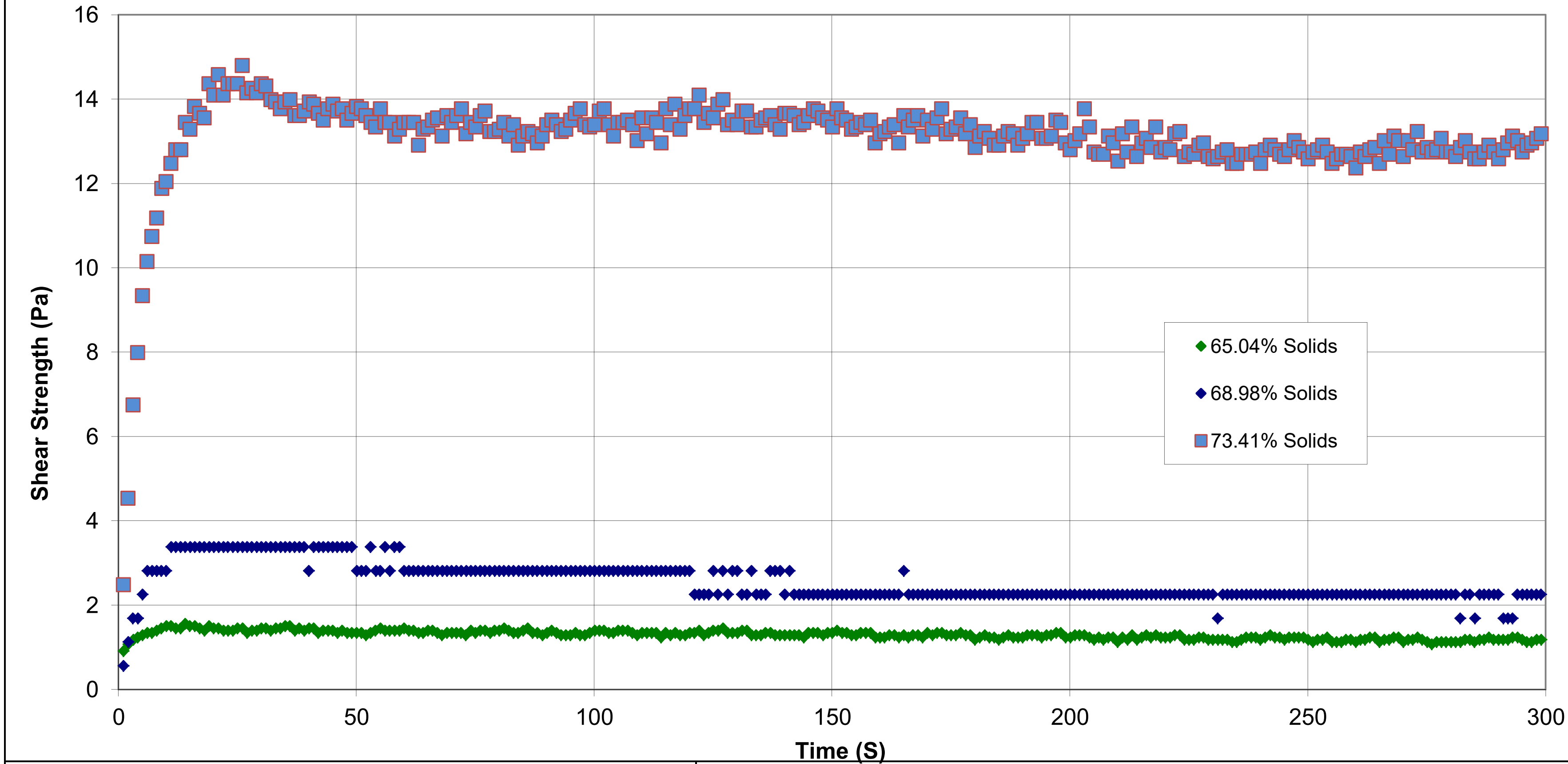
Tailings Rheology

Date: 15/02/2017 **Job No:** 114185.08

FIGURE C28

26/09/2018

114185.08
Flocculated Combined Tailings
ALL SHEAR VANE RESULTS



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Iron Bridge Operations Pty Ltd

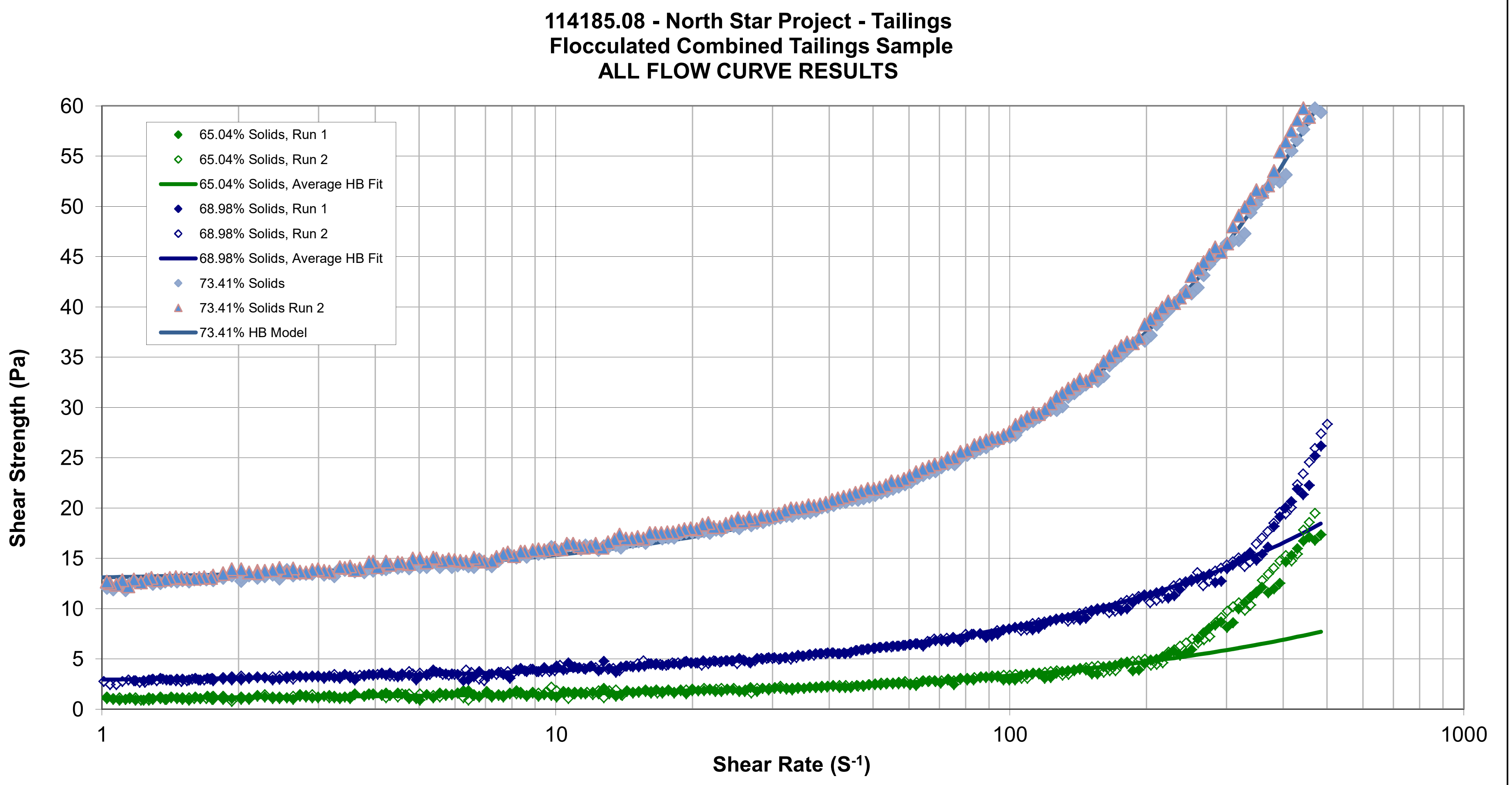
North Star Project - 2016 Tailings Testing

Flocculated Combined Tailings

Tailings Rheology

Date: 15/02/2017 **Job No:** 114185.08

FIGURE C29



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Iron Bridge Operations Pty Ltd

North Star Project - 2016 Tailings Testing

Flocculated Combined Tailings

Tailings Rheology

Date:	15/02/2017	**Job No:**	114185.08	**FIGURE C30**

APPENDIX C2

ATC Williams

REPORT

IRON BRIDGE OPERATIONS PTY LTD

North Star Magnetite - Stage 2
Pilbara, Western Australia

Tailings Storage Facility & Return Water Pond

Appendix C2-
Tailings Test Work - Factual Report

October 2019

114185.20 R01

Document History and Status

Title:	Appendix C2 - IBO North Star Magnetite TSF & RWP, Tailings Test Work – Factual Report
Job Number/Extension	114185.20
Document Number:	114185.20 R01
Project Office:	Melbourne
File Path:	K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Documents\R01\Text\114185.20 R01.docx
Author:	Andrew Fyans
Reviewer:	Craig Noske
Job Manager:	Craig Noske

Rev.	Status	Issued to	Issue Date	Signatures	
				Author	Reviewer
A	Issued for Approval	IBO	11/11/2019	AF	CN

This Revision distributed To:	Hard Copy	Electronic Copy
IBO	-	☑
ATCW Melbourne Office	-	On network

ANDREW FYANS
Author

CRAIG NOSKE
Reviewer

IMPORTANT NOTICE
Please refer to our Conditions of Investigation and Report

CONDITIONS OF INVESTIGATION AND REPORT

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ATC Williams

TABLE OF CONTENTS

APPENDICES

1 INTRODUCTION

1.1 Overview

This report presents the results of the laboratory testing of tailings for the Iron Bridge Operations (IBO) North Star Magnetite Stage 2 Tailings Storage Facility (TSF).

The work has been carried out in accordance with the ATC Williams Pty Ltd (ATCW) proposal 114185.20-P01 'Proposal for Tailings Test Work and Consolidation Modelling' submitted to IBO on 29 May 2019, and MMG Purchase Order Number 4300170573.

A key purpose for the tailings test work was to define tailings material characteristics for use in subsequent tailings consolidation modelling work. Results of the consolidation modelling will be presented in a separate report, 114185.20-R02.

1.2 Background

ATCW submitted the Mining Proposal Design Report **[Ref. 1]** for the North Star Magnetite Stage 2 TSF & RWP in March 2019.

The primary purpose of the test work was to confirm the geotechnical, density and consolidation characteristics of the North Star Magnetite tailings with the existing data and assumptions made in the Mining Proposal Design Report **[Ref. 1].**

2 LABORATORY TESTING

2.1 Overview

The tailings sample geotechnical and depositional (density) characteristics were tested to confirm the existing data and assumptions used in the TSF design, which were documented in the Mining Proposal Design Report **[Ref. 1]**.

The laboratory test work presented herein can be divided into three sub-categories:

i. Material characterisation testing;
ii. Depositional (density) characterisation testing; and
iii. Consolidation testing.

These sub-categories are described in detail in the subsequent sections.

2.2 Sample

A bucket containing approximately 10 kg of dry tailings was received at the ATCW Melbourne laboratory on 28 May 2019. The sample was generated by the pilot plant and the North Star site.

Tailings samples were not flocculated prior to material characterisation and depositional characterisation testing, based on the assumption and previous experience with flocculated tailings that the flocculant breaks down during transportation and has minimal impact on the depositional characteristics.

Samples for depositional characterisation testing were mixed with water to achieve a solids concentration of approximately 65%, in accordance with the proposed discharge solids content advised by IBO and documented in the Mining Proposal Design Report **[Ref. 1]**.

ATCW has since been advised by IBO that the proposed discharge solids content has reduced from 65% to 62%.

2.3 Material Characterisation

2.3.1 Overview

The following material characterisation test work was completed:

i. Soil Particle Density (SG);
ii. Particle Size Distribution (PSD) & Hydrometer; and
iii. Atterberg Limits (PI).

NATA accredited laboratory report sheets for the material characterisation tests conducted by ATCW are presented in **Appendix A**.

2.3.2 Results

Material characterisation test results are presented in **Table 1**.

Table 1
Laboratory Test Results –Material Characterisation

SG (t/m^3)	PSD			Hydrometer		Atterberg Limits		Classification
	P_{80} (mm)	Sand (%)	Fines (%)	Silt (%)	Clay (%)	LL (%)	PI (%)	
2.92	0.035	13.3	86.7	72.7	14.0	29	9	CL

Email correspondence received from IBO indicated that tailings sample was expected to have a P_{80} of approximately 0.039 mm and that the Stage 2 tailings are predicted to have a P_{80} in the range of 0.035 to 0.045 mm. Results of the material characterisation testing indicate that the tailings sample is within the predicted range, albeit at the lower bound.

2.4 Depositional Characteristic Testing

2.4.1 Overview

The following depositional (density) characterisation test work was completed:

i. Initial Settled Density (ISD); and
ii. Shrinkage Limit Density (SLD).

As described in **Section 2.2**, the depositional characteristic testing was undertaken on tailings prepared to the design discharge solids content of 65%.

NATA accredited laboratory report sheets for the depositional (density)characterisation tests conducted by ATCW are presented in **Appendix B**.

2.4.2 Initial Settled Density

The test is performed by taking a sub-sample of the material at the required initial solids concentration and pouring into a large diameter test cell to an initial test depth. A large diameter cell is used to limit the effect of friction between the settling sample and the side wall. The settlement of the tailings surface over time is recorded until settlement is judged to be complete.

Samples were prepared at an initial solids concentration of approximately 65% and poured at a nominal depth of 100 mm.

The results of the ISD test work are presented in **Table 2**.

Table 2
Laboratory Test Results – Initial Settled Density

Solids Concentration (%)		Sample Depth (mm)			$\frac{H_F}{H_0} \times 100$	Settled Dry Density (t/m³)	Bleed Water	
Initial	Final	Initial H_0	Final H_F	Change ΔH			% of Initial Water	m³/t of Dry Solids
64.88	73.55	100.1	79.5	20.6	79.42	1.425	33.58	0.182

2.4.3 Shrinkage Limit Density

The SLD is the limiting dry density that could result from the evaporative drying of the tailings on a beach.

The SLD is determined by allowing a saturated sample to gradually dry over time. A point will be reached in the drying process at which the reduction in moisture content will no longer result in a reduction in the volume of the soil mass. This point is defined as the SLD.

A summary of the SLD test results is presented in **Table 3**.

Table 3
Laboratory Test Results – Shrinkage Limit Density

Shrinkage Limit Density (t/m³)	Crack Volume (%)
1.66	2.0

2.5 Rowe Cell (Consolidation Testing)

Rowe Cell testing involves loading a sample hydraulically in a stainless steel cell under drained conditions. Pore-water pressures, drainage, volume changes are all measured during incrementally increased loading stages. Results provide geotechnical data defining the compressibility and permeability characteristics of the tailings.

Rowe Cell test results are summarised in **Table 4**.

Table 4
Laboratory Test Results – Rowe Cell Consolidation

Material Compressibility		Material Permeability	
Pressure (kPa)	Void Ratio	Permeability (m/s)	Cv (m^2/year)
1.25	0.90	1.6×10^{-7}	8.8
640	0.68	7.3×10^{-8}	3687.8

Laboratory report sheets for the consolidation test work are presented in **Appendix C**.

3 COMPARISON WITH PREVIOUS ASSESSMENTS

ATCW has previously made an assessment (from prior test work and assumptions) of design tailings characteristics as part of the Mining Proposal Design Report **[Ref. 1]**.

Presented in **Table 5** is a comparison of ATCW's previous assessment with the lab results presented herein.

Table 5
Laboratory Test Results – Comparison with Previous Assumptions

Parameter	Previous Assumptions	Current Test Results
SG	2.97	2.92
ISD (t/m^3)	1.45	1.43
SLD (t/m^3)	1.75 – 1.80*	1.66
Overall In-situ Density (t/m^3)	1.66	^1.53 t/m^3 @ 1.25 kPa ^1.74 t/m^3 @ 640 kPa

*No testing conducted, estimate based on previous experience and internal database of similar materials.
^Rowe Cell results do not provide a direct comparison and are shown only as an indication (without further analyses)

The tested tailings sample has an SG of 2.92 t/m^3, which is slightly lower than the previously tested 2.97 t/m^3, presented in the Mining Proposal Design Report **[Ref. 1]**.

ATCW previously adopted an initial settled density of 1.45 t/m^3 in the Mining Proposal Design Report **[Ref. 1]**, which is comparable with the recent test result of 1.43 t/m^3.

The SLD result from the test work indicate a significantly lower density than previously estimated. This could have the impact of achieving a lower Overall In-situ Density (OID) than previously adopted in the Mining Proposal Design Report **[Ref. 1]**, which was estimated at 1.66 t/m^3.

The OID at the end of filling will be accurately assessed as part of the consolidation modelling (to be presented in a subsequent report). However, the Rowe Cell test results show that the tailings will reach a maximum dry density in excess of 1.74 t/m^3.

This indicates that the original estimate of cumulative density at end of filling of 1.66 t/m^3 falls within the range of expected results, which will be confirmed on submission of the consolidation modelling report.

4 RECOMMENDATIONS

The test work completed for tailings has been conducted on a generated sample from the pilot plant, and was provided as a dry sample to the ATCW laboratory.

It is recommended that once deposition has commenced into the TSF, that confirmatory laboratory testing be undertaken on the actual production tailings to compare with the material characterisation testing.

5 CLOSURE

Your attention is drawn to the “Conditions of Investigation and Report” which appear after the title page of this report.

REFERENCES

[Ref. 1] ATC Williams (March 2019), “*Iron Bridge operations Pty Ltd, North Star Magnetite – Stage 2, Pilabar, Western Australia, Tailings Storage Facility & Return Water Pond, Mining Proposal Design Report*”, Ref. 114185.14 R03 Rev 1.

APPENDICES

APPENDIX A

Determination of the Soil Particle Density of a Soil

TEST IN ACCORDANCE WITH AS 1289.3.5.1

Client: Fortescue Metals Group Ltd ....

NATA Report No.: R32619........

Address: Level 2, 87 Adelaide Terrace, East Perth, WA .... 6004..

Job No.: 114185.20..................

Project: North Star Magnetite Stage 2 - Tailings Test Work ..

Location: WA ....

Register Number	Sample Description	Test Temperature (°C)	% of Sample >2.36 mm	Particle Density (g/cm^3)
17919	Wet Tailings Blend	18.6	None	2.92 #

Notes:

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1

☒ Sample provided by the client

* = apparent average soil particle density – particle size less than 2.36 mm

X = apparent average soil particle density – particle size greater than 2.36 mm

= soil particle density of the total sample

The test results relate only to the items tested.

NATA ACCREDITED FOR TECHNICAL COMPETENCE

NATA ACCREDITED LABORATORY NUMBER: 3372

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory ....

Date Tested: 28/06/2019

Name of Signatory *Luke Renkin*

Date Reported: 22/07/2019

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Form RSN 003.7
Date of Issue: August 2018

Particle Size Distribution Results

TEST IN ACCORDANCE WITH AS 1289

☑ Method 3.6.1 ☑ Method 3.6.3
☑ Oven Drying Method 2.1.1

ATC Williams

Client: Fortescue Metals Group Ltd
Address: Level 2, Adelaide Terrace, West Perth WA, 6004
Project: North Star Magnetite Stage 2 - Tailings Test Work

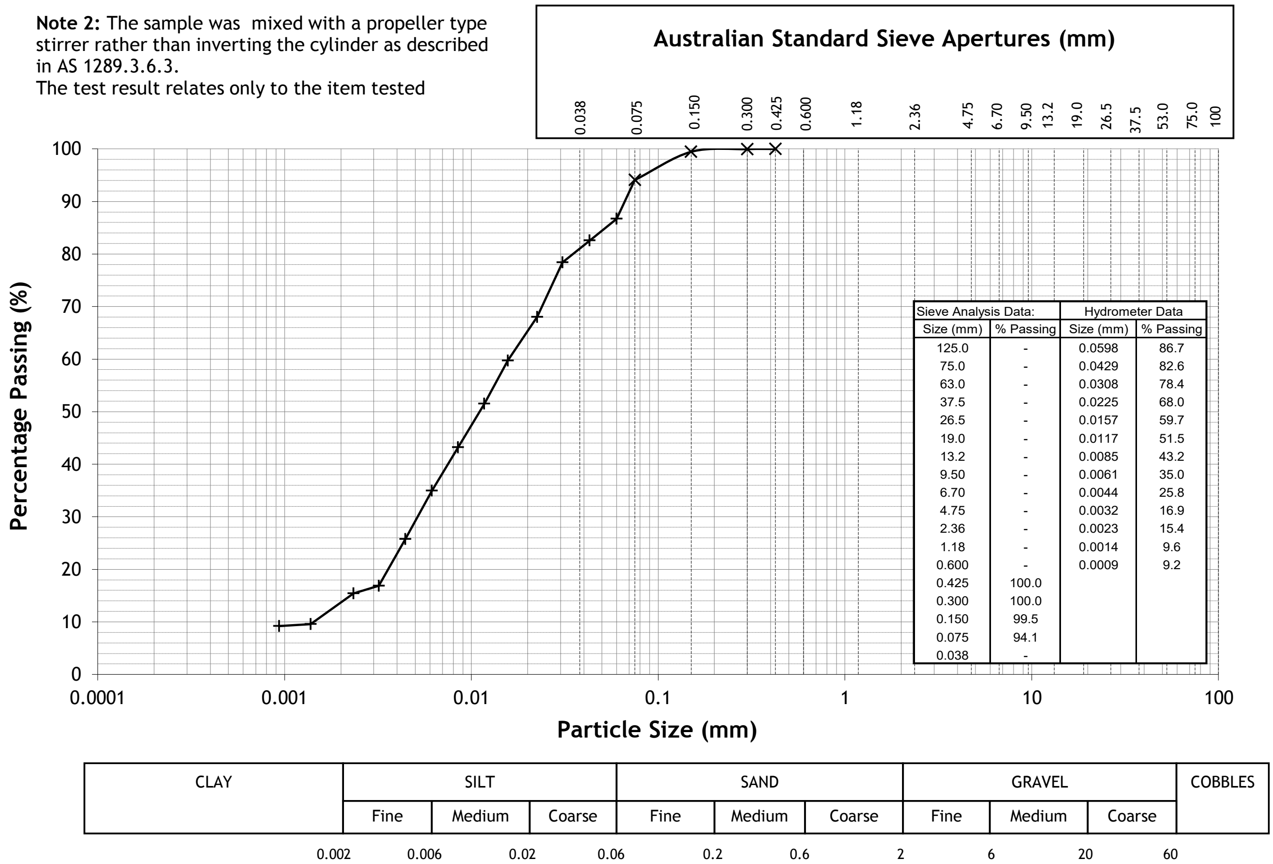
NATA Report No.: R32819
Job No.: 114185.20
Register No.: 17919
Location: WA

Sample Description: Wet Tailings Blend	Borehole ☐ No:	Test Pit ☐ Depth:

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1 Clause 6.5.4 ☑ Sample provided by the Client

Note 1: The sample was oven-dried during sample preparation and not air-dried as stated in AS 1289.3.6.3

Note 2: The sample was mixed with a propeller type stirrer rather than inverting the cylinder as described in AS 1289.3.6.3.
The test result relates only to the item tested



Sieve Analysis Data:		Hydrometer Data	
Size (mm)	% Passing	Size (mm)	% Passing
125.0	-	0.0598	86.7
75.0	-	0.0429	82.6
63.0	-	0.0308	78.4
37.5	-	0.0225	68.0
26.5	-	0.0157	59.7
19.0	-	0.0117	51.5
13.2	-	0.0085	43.2
9.50	-	0.0061	35.0
6.70	-	0.0044	25.8
4.75	-	0.0032	16.9
2.36	-	0.0023	15.4
1.18	-	0.0014	9.6
0.600	-	0.0009	9.2
0.425	100.0		
0.300	100.0		
0.150	99.5		
0.075	94.1		
0.038	-		

NATA ACCREDITED LABORATORY NUMBER: 3372

NATA ACCREDITED FOR TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: [signature] Date Tested: 8-10/07/2019

Name of Signatory: Luke Renkin Date Reported: 22/07/2019

Classification (Atterberg Limits)

TEST IN ACCORDANCE WITH AS 1289 – CONE PENETROMETER

Client: Fortescue Metals Group Ltd

Address: Level 2, 87 Adelaide Terrace, East Perth, WA 6004

Project: North Star Magnetite Stage 2 - Tailings Test Work

NATA Report No.: R32719

Job No.: 114185.20

Location: WA

Register Number	Sample Description	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Sample Curled (CU) Crumbled (CR) Cracked (CK)
17919	Wet Tailings Blend	29	20	9	-	-

☐ Sampled by ATC Williams Pty Ltd in accordance with AS 1289.1.2.1
☒ Sample provided by the client

The test results relate only to the items tested.

Test Methods:

☐	Liquid Limit	AS 1289.3.9.1 (Standard - Penetrometer Method)
☒	Liquid Limit	AS 1289.3.9.2 (One Point – Penetrometer Method)
☒	Plastic Limit	AS 1289.3.2.1
☒	Plasticity Index	AS 1289.3.3.2 (Cone Plasticity Index)
☐	Linear Shrinkage	AS 1289.3.4.1
☒	Moisture Content	AS 1289.2.1.1

Sample Preparation:

☐ Natural Moisture ☒ Air Dried ☐ Oven Dried ☐ Unknown
☐ Wet Sieved ☒ Dry Sieved ☐ Unsieved

NATA ACCREDITED LABORATORY NUMBER: 3372

Accredited for compliance with ISO/IEC 17025 - Testing

ACCREDITED FOR TECHNICAL COMPETENCE

Approved Signatory ....

Date Tested: 28/06/2019

Name of Signatory: *Luke Renkin*

Date Reported: 22/07/2019

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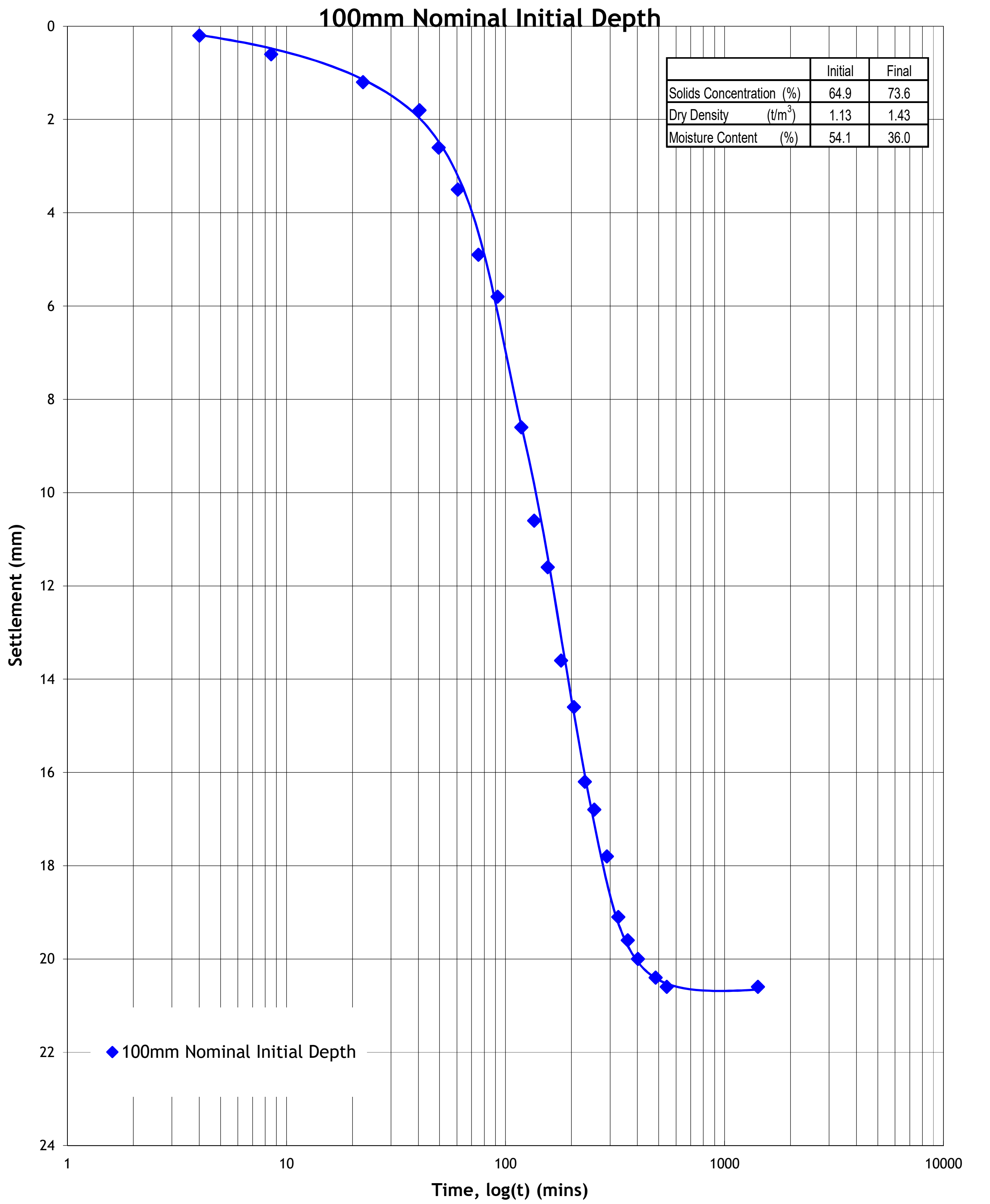
APPENDIX B



	Initial	Final
Solids Concentration (%)	64.9	73.6
Dry Density (t/m^3)	1.13	1.43
Moisture Content (%)	54.1	36.0

Fortescue Metals Group LTD

North Star Magnatite Stage 2- Tailings Testing

Wet Tailings Blend

Undrained Settled Density

Settlement vs Time

(Sample No. 17919)

Job No: 114185.20

Date: 10/09/2019

FIGURE B1

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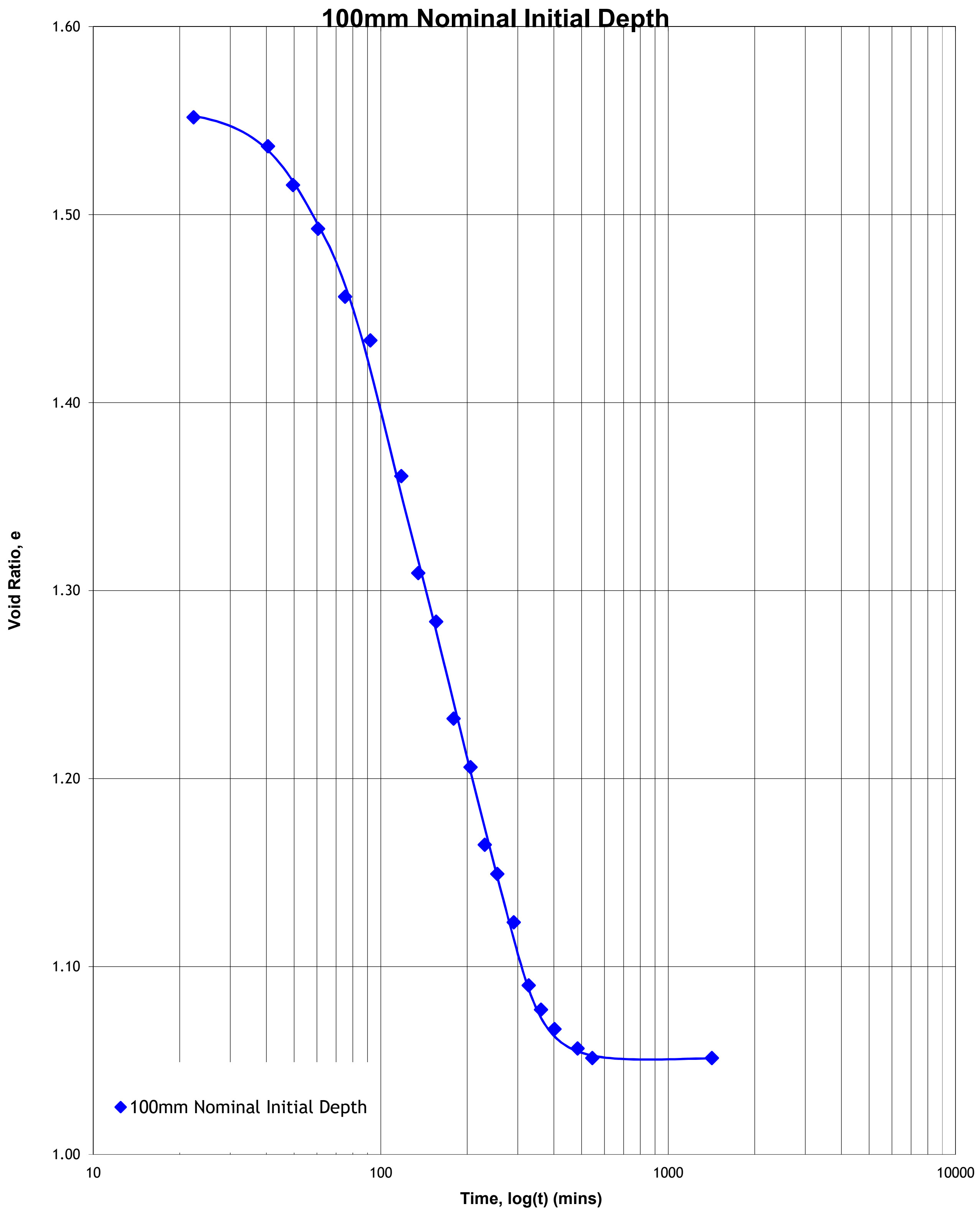
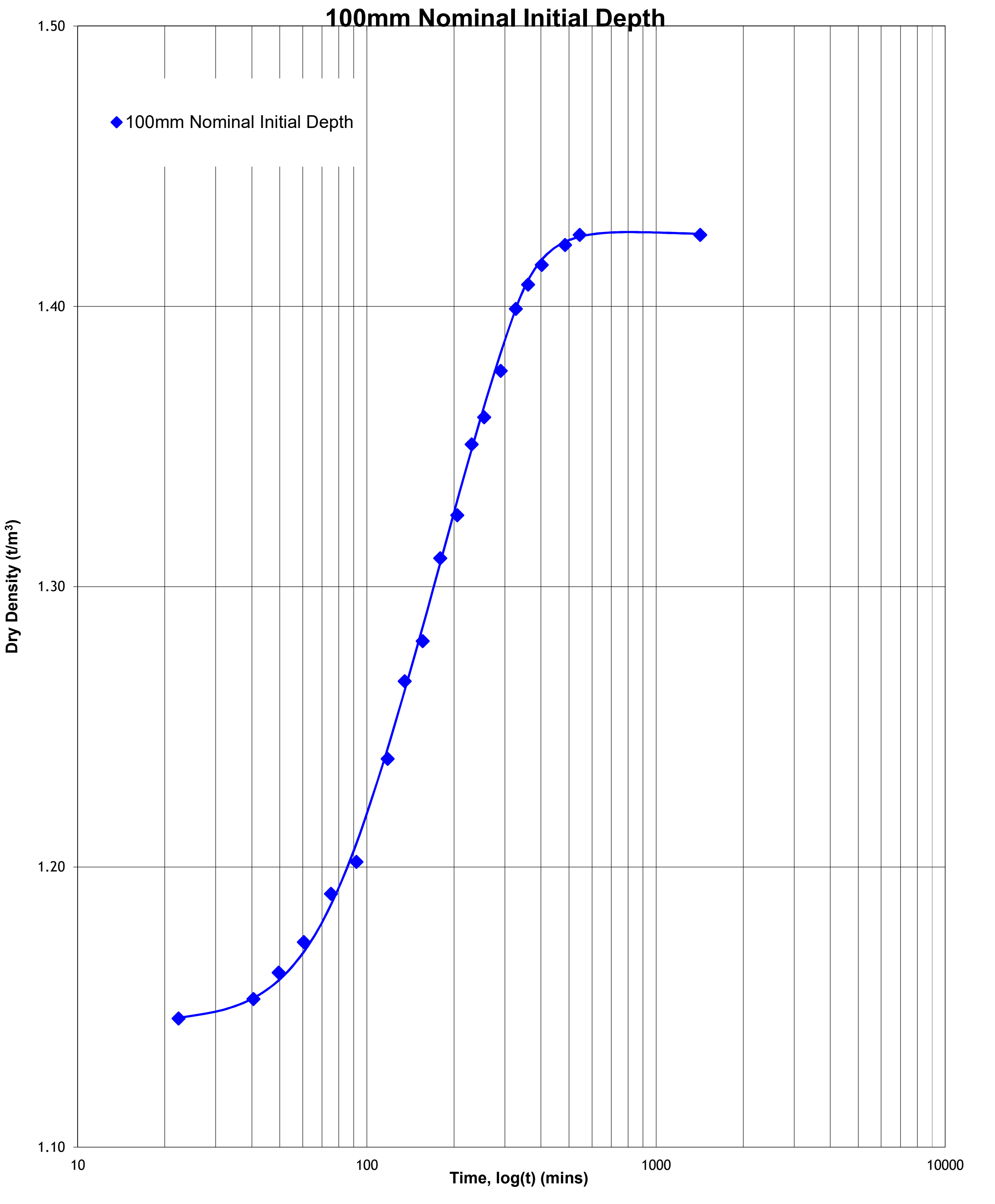
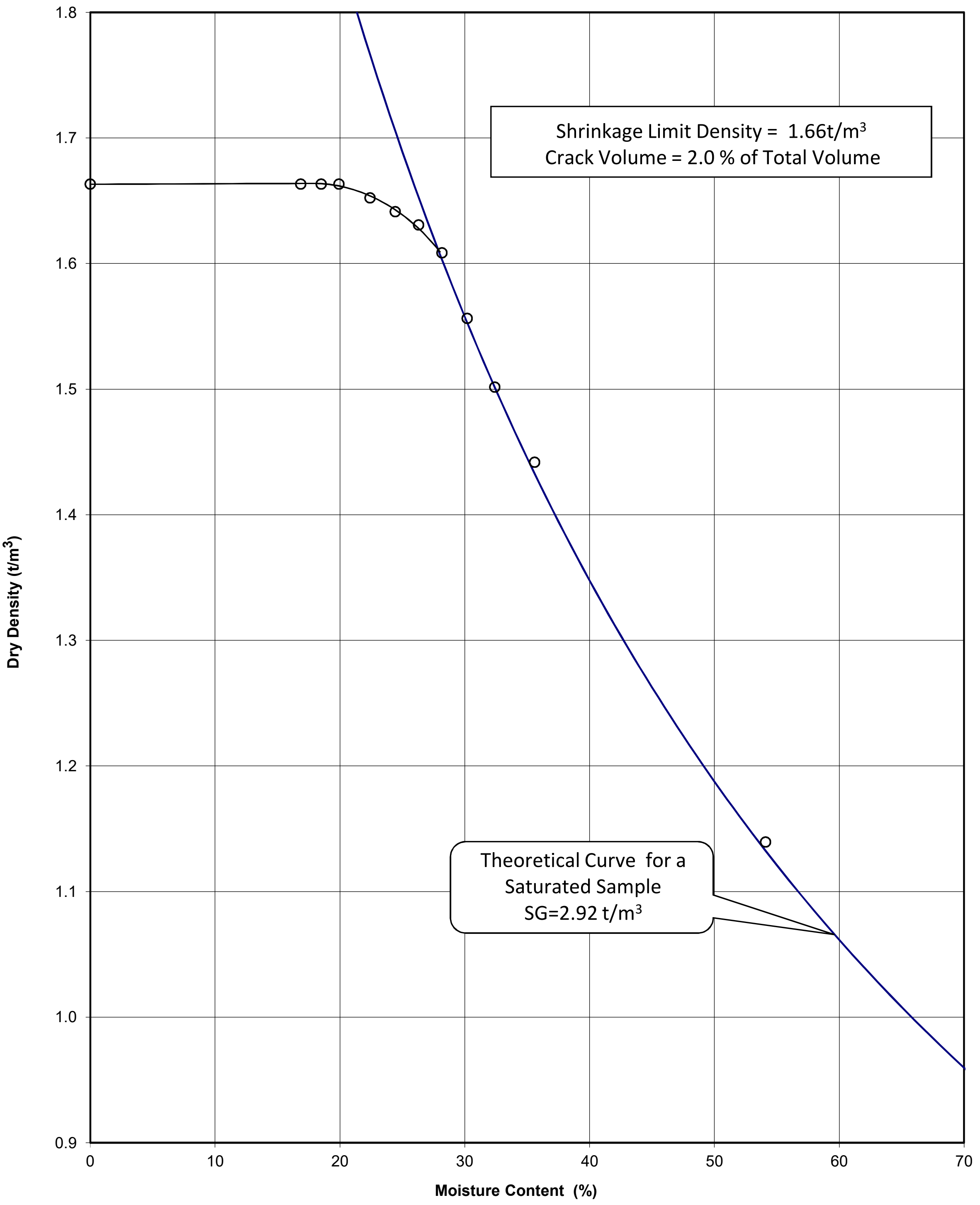


North Star	Undrained Settled Density
North Star Magnatite Stage 2	Void Ratio vs Time
Unflocculated Tailings	Flocc'd Tails (Sample No. 17919)
ATC Williams	Job No: 114185.20
	Date: 10/09/2019
www.atcwilliams.com.au	FIGURE B2



North Star North Star Magnatite Stage 2 Unflocculated Tailings	Undrained Settled Density Dry Density vs Time Flocc'd Tails (Sample No. 17919)
ATC Williams www.atcwilliams.com.au	Job No: 114185.20 Date: 10/09/2019
	FIGURE B3

..:\2014\114185.20 North Star\17919- ISD Undrained with density and solids concentration ATC Report (Dry Density)



Fortescue Metals Group Ltd

North Star Magnatite Stage2 - Tailings Test Work

Wet Tailings Blend

Shrinkage Limit Density

Sample No: 17919

Job No:	114185.20
Date:	22/07/2019

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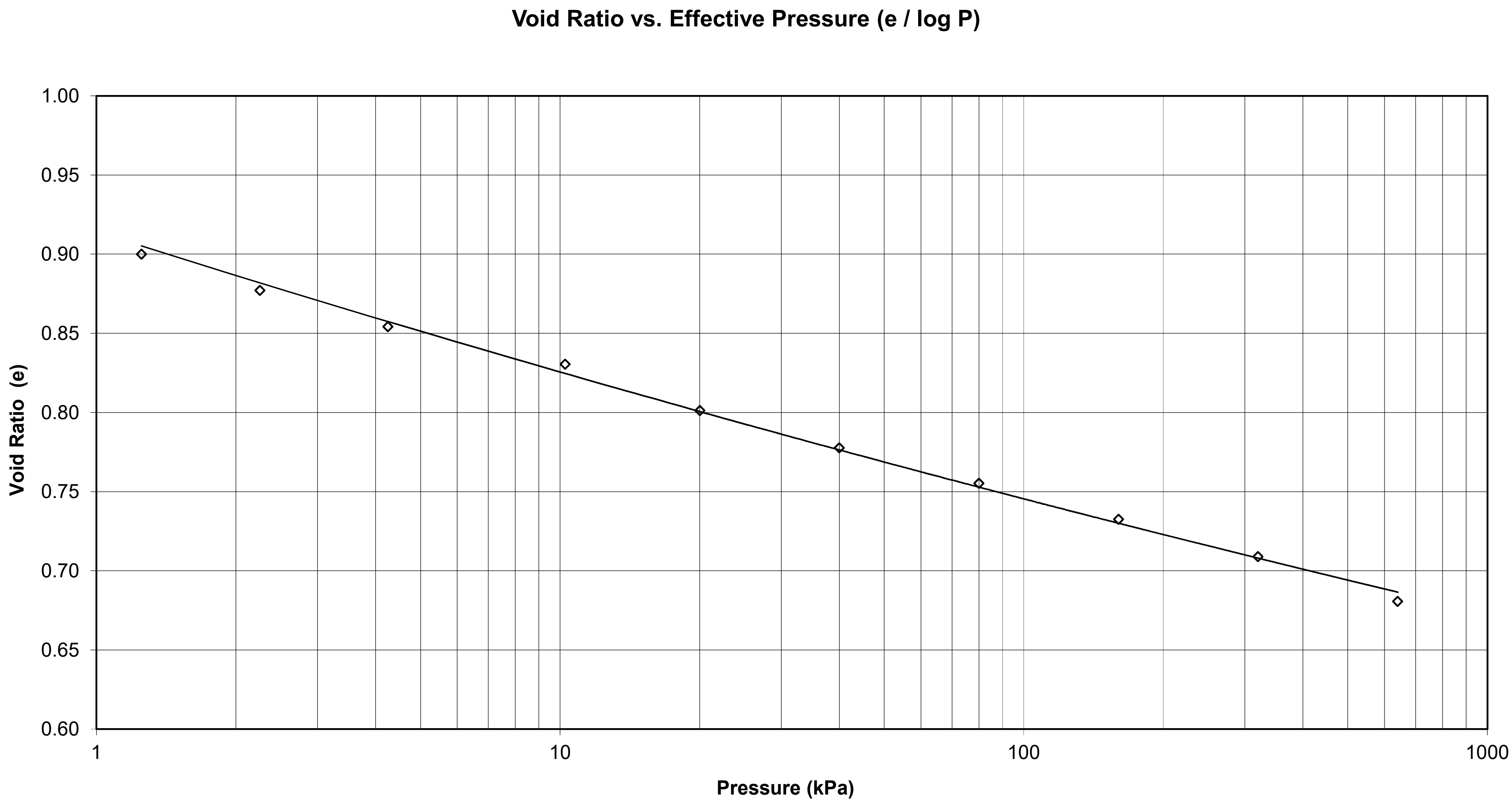
FIGURE B4

APPENDIX C



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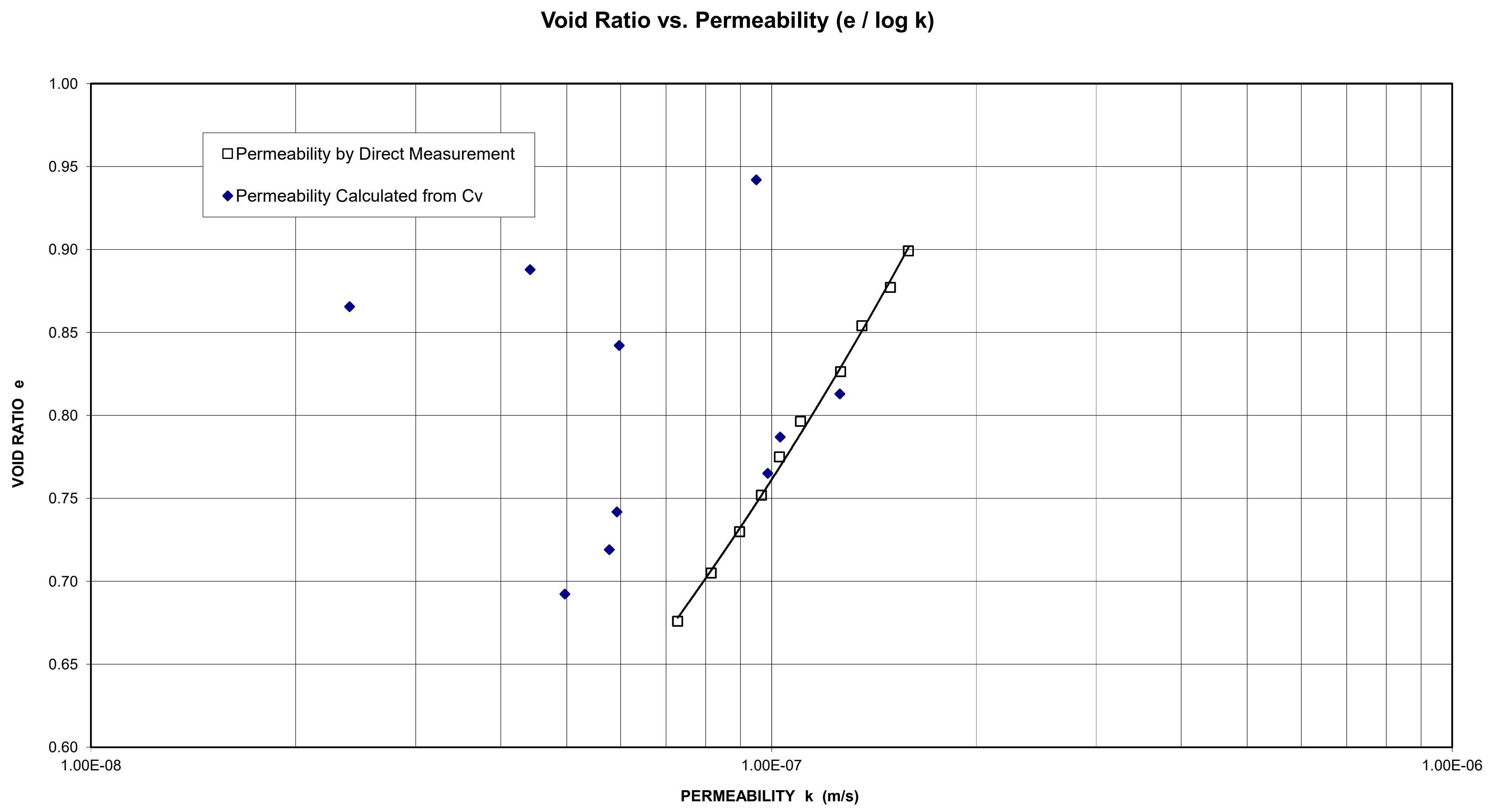
FORTESCUE METALS GROUP LTD

NORTH STAR MAGNETITE STAGE 2 - TAILINGS TEST WORK AND CONSOLIDATION MODEL

ROWE CELL CONSOLIDATION

WET TAILINGS BLEND

Date:	16/08/2019	Job No:	114185.20	FIGURE C1



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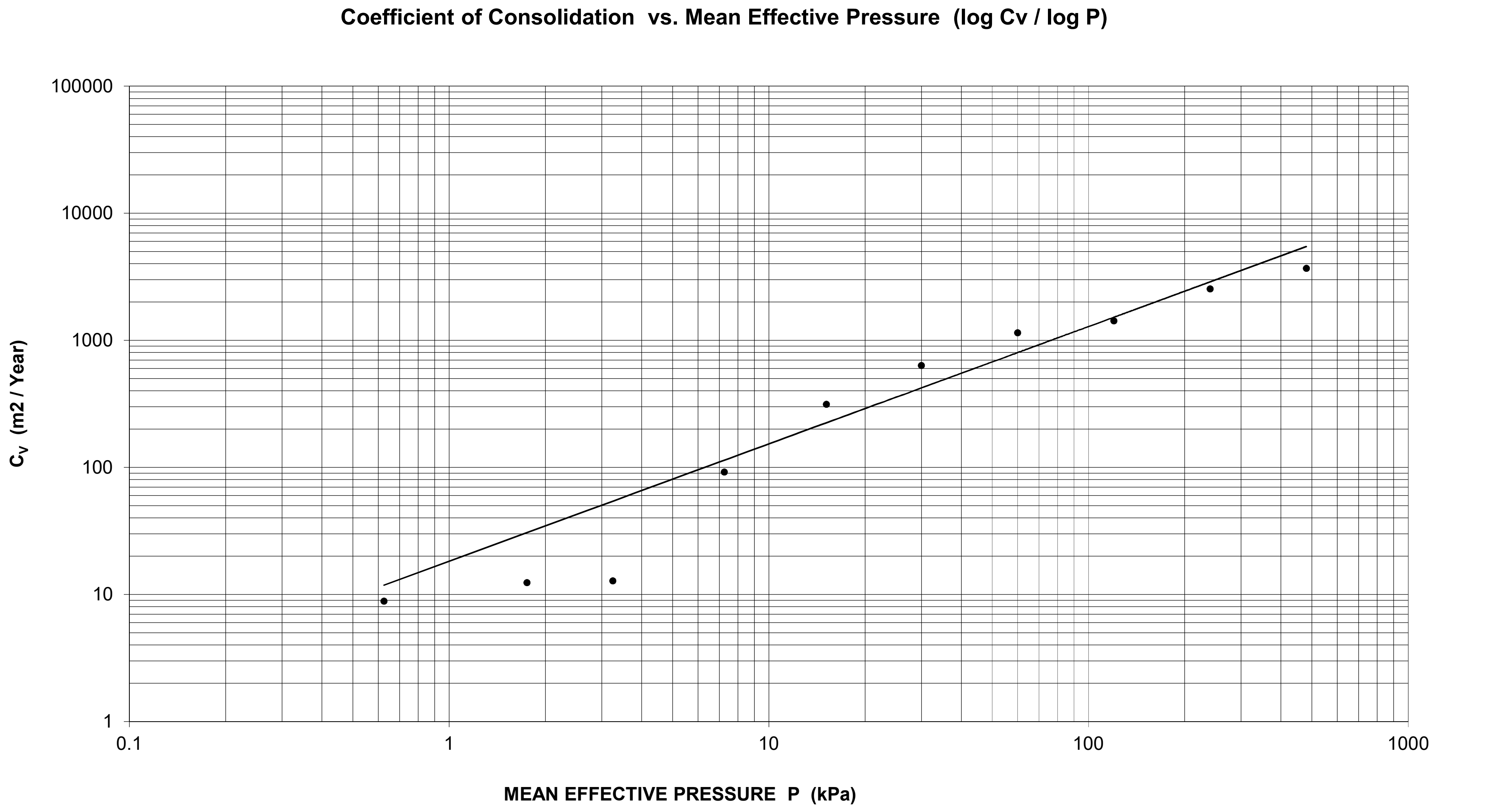
NORTH STAR MAGNETITE STAGE 2 - TAILINGS TEST WORK AND CONSOLIDATION MODEL

ROWE CELL CONSOLIDATION

WET TAILINGS BLEND

Date: 16/08/2019 **Job No:** 114185.20

FIGURE C2



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NORTH STAR MAGNETITE STAGE 2 - TAILINGS TEST WORK AND CONSOLIDATION MODEL

ROWE CELL CONSOLIDATION

WET TAILINGS BLEND

Date: 16/08/2019 **Job No:** 114185.20

FIGURE C3

L:\2014\114185.20 North Star\Rowe Cell\17919 - Rowe Cell Report Log Cv logP Report

APPENDIX C3

REPORT

IRON BRIDGE OPERATIONS PTY LTD

North Star Magnetite – Stage 2
Pilbara, Western Australia

Tailings Storage Facility

Appendix C3 -
Tailings Consolidation Modelling

November 2019

114185.20-R02

Document History and Status

Title:	Appendix C3 - Tailings Consolidation Modelling
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Authors:	Andrew Fyans
Reviewer:	Behrooz Ghahramen-Nejad
Job Manager:	Craig Noske

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				Author	Reviewer
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This Revision distributed to:	Hard Copy	Electronic Copy
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ATC Melbourne Office		On network

ANDREW FYANS
Author

CRAIG NOSKE
Reviewer

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TABLE OF CONTENTS

CHARTS

TABLES

APPENDIX A – Consolidation Modelling Output

1 INTRODUCTION

This report documents the results of the one-dimensional tailings consolidation modelling undertaken for the North Star Magnetite – Stage 2, Tailings Storage Facility (TSF) for the Iron Bridge Operations Pty Ltd (IBO).

The work has been carried out in accordance with the ATC Williams Pty Ltd (ATCW) proposal 114185.20-P01 'Proposal for Tailings Test Work & Consolidation Modelling' submitted to Fortescue Metals Group Ltd (FMG) on behalf of IBO on 29 May 2019, and FMG Purchase Order Number 4300170573.

The primary purpose of the work is to define the consolidation characteristics of the tailings, verify predictions of insitu density and estimate that the amount of consolidation that will occur adjacent to the TSF embankment upstream face decant structure, in order to adequately estimate the forces acting on the concrete box culvert.

The consolidation modelling presented herein makes use of the results collected from the laboratory testing program, covered by the same proposal and purchase order number, the results of which were presented in Tailings Laboratory Test Work Factual Report **[Ref. 1]**, provided to IBO in September 2019.

2 CONTEXT & SCOPE

2.1 Overview

The deposition method in Stage 2A TSF comprises down valley discharge of tailings, with construction of cross-valley embankments to form an impoundment within a wide valley. The largest embankment will be in excess of 68m high at the final construction stage and approximately 2.9 km in length. Embankments will be constructed in seven stages using downstream methods. Supernatant water will be transferred to the Return Water Pond (RWP) located immediately downstream of the TSF main embankment. The design data for the TSF is summarised as follows:

- Particle size, P_{80} (particle size for which 80% of material is finer) = 40 μm
- Life of mine (LOM) = 24 years;
- TSF operational life = 20 years (with contingency for extension to LOM capacity);
- Required TSF starter (Stage 1) storage capacity = 2 years;
- Required minimum TSF Stage life (Stage 2 to 7) = 3 years;
- Annual process plant operating hours = 7,400 hrs;
- Tailings LOM production = 691.4 Mt (24 years), 621.9 Mt (20 years)
- Tailings pumping rate to TSF = 3,800 to 4,000 dry tonnes per hour (dtph);
- Solids concentration at thickener underflow by weight = 62% and
- Maximum return water pumping rate (from RWP) = 1,980 m^3/hr.

The tailings will be thickened to a target solids content of 62% before discharge into TSF. Settled in-situ density of 1.5 t/m^3 during the first 6 months of operation followed by 1.66 t/m^3 have been adopted for the design of TSF with its capacity being 338 million m^3. Detail description of design of TSF and RWP can be found in the Mining Proposal Design Report **[Ref. 2]**.

2.2 Scope of Work

The scope of work carried out to produce this report includes the development of a representative finite-strain tailings consolidation model to evaluate the time rate of consolidation within the tailings profile, primarily based on laboratory Rowe cell testing of production tailings **[Ref. 1]**.

The consolidation modelling has been used to estimate the amount of tailings consolidation settlement that will take place over time. The results can be used to estimate the overall dry density of the tailings deposit, the pore water pressure conditions (including excess pore water pressure) and the hydraulic conductivity of the tailings profile.

The results will also be used to assess the impact of consolidation settlement on the decant structure box culverts (due to 'positive' projection) located on the upstream face of the embankment.

2.3 Definitions

<u>Consolidation</u>
Is the time dependent process whereby water is "squeezed" from the pore spaces in the tailings due to their self-weight. In rapidly filled, deep deposits of fine-grained tailings with a low permeability base, the drainage path for this consolidation water is predominantly upwards to the surface. This drainage can take many years (decades), meaning that the tailings will be "under-consolidated" (i.e. exhibit excess pore water pressure) for the entire active period of the TSF, and many years after cessation of deposition.

Typically, consolidation modelling is undertaken to enable insitu tailings dry densities and rates of release of entrained water during the deposition phase to be estimated. Rowe Cell test data provides information that can define the tailings characteristics in terms of material *compressibility* and *permeability*, which forms the primary input data that is required to undertake consolidation modelling.

<u>Positive Projection</u>
The decant structure box culverts incorporated in the design of the TSF are an example of 'positive' projecting installations. In 'positive' projection, the culvert projects above the natural ground level and differential fill settlement over the top of the culvert can occur, compared to that on each side. In such a case, the fill load acting on the top of the culvert is taken as the mass of the fill material in a trapezoidal prism over the width of the culvert, plus the frictional forces developed between the soil prism and the adjacent fill. Often, 'positive' projection installations are also referred to as 'embankment' installations.

3 MODELLING METHODOLOGY

3.1 Overview

3.1.1 Tailings Consolidation Model

The tailings were modelled as a 1-dimensional column using SV Consolidation software **[Ref. 3]**. Material properties, filling rates, and boundary conditions were derived, and applied as discussed in the following sections of this Report.

A model has been created using laboratory test results **[Ref. 1]** on the tailings sample. It should be noted that IBO has prepared the tailings sample and shipped to the ATCW Melbourne laboratory for subsequent laboratory testings.

The model included incompressible layers of foundation materials with permeability values of 1×10^{-7} m/sec for upper foundation layer below the tailings and 1×10^{-6} m/sec for lower foundation layer, based on data presented in the Mining Proposal Design Report **[Ref. 2]**, with base flux measured at the interface between tailings and the foundation material.

Excess Pore Water Pressure (EPWP) was set to zero as the base boundary condition to allow dissipation of EPWP.

4 CONSOLIDATION MODELLING INPUT PARAMETERS

4.1 Initial Conditions

4.1.1 Material Properties

As part of the laboratory testing program **[Ref. 1]**, Initial Settled Density (ISD) and particle density (SG) testing was conducted.

ISD results were used to describe the initial conditions in the model, and SG results to estimate the material properties. **Table 1** presents the values used in the model.

Table 1

Initial Conditions - Material Properties

Material Property	Unit	Laboratory Test Result
SG	t/m3	2.92
ISD Void Ratio (e_0)	-	1.051
ISD Dry Density (DD_0)	t/m3	1.425

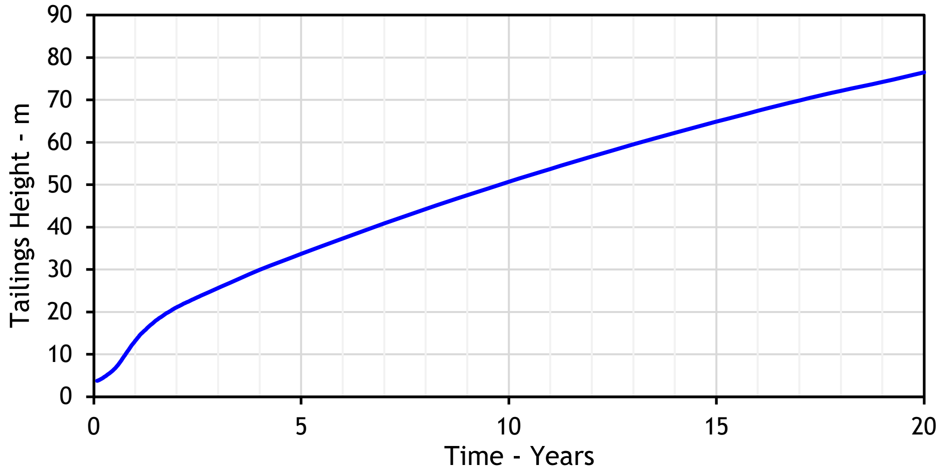
4.1.2 Rate of Rise

Since producing the Mining Proposal Design Report **[Ref. 2]**, IBO has advised ATCW that the production rates have been increased. The filling curve used in the TSF Design Report **[Ref. 2]** was therefore revised to reflect the revised production rates. The updated filling curve was used as an initial starting point for the model. Several iterations of the consolidation model were then completed to arrive at a final consolidated height of 64 m. The final iteration was then calibrated by calculating the predicted total tonnages across the entire facility using the results of the one-dimensional model, as discussed in **Section 5**.

The resultant input filling curve culminated to a final tailings depth of 76.2 m (unconsolidated), which is presented graphically in **Chart 1**.

Chart 1

Initial Conditions -TSF Filling Curve (Maximum Tailings Height)



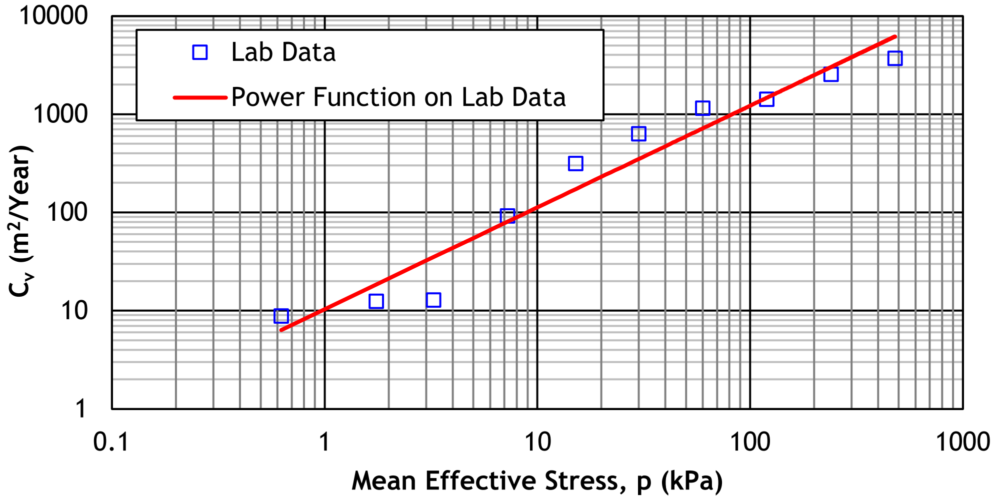
4.2 Material Compressibility and Permeability Functions (Rowe Cell Data)

The Coefficient of Consolidation (C_v) is the geotechnical parameter governing the rate by which compression of a soil occurs. The rate and amount of compression in tailings varies with the rate at which pore water is dissipated and is therefore dependent on permeability.

The Rowe cell consolidation test was performed to accurately estimate the compressibility and permeability of the tailings. Results as shown in **Chart 2** indicate that the Cv values for the tailings ranged from approximately 10 to 4000 m^2/year. Based on ATCW's experience and internal database, the Cv value is at the upper bound of what can be expected for iron ore tailings.

Chart 2

Rowe Cell Results - Coefficient of Consolidation



SV Consolidation software **[Ref. 3]** couples SV Solid **[Ref. 4]** and SV Flux **[Ref. 5]** programs. The individual software packages require separate input properties to define a material in terms of compressibility (SV Solid **[Ref. 4]**) and permeability (SV Flux **[Ref. 5]**). Both properties are defined by Rowe Cell testing.

Material compressibility is determined by relationships between the void ratio and mean effective stress. Similarly, material permeability is defined by relationships between the void ratio and hydraulic conductivity. Both properties are defined by power functions, as follows:

Material compressibility power function (Somogyi, 1980) $\boldsymbol{e = A \cdot \sigma'^{B}}$

Material permeability power function (Somogyi, 1980) $\boldsymbol{k_{sat} = C \cdot e^{D}}$

Where: $e = void\ ratio,$
$\sigma' = effective\ stress\ (kPa),$
$k_{sat} = saturated\ hydraulic\ conductivity\ (m/s), and$
$A, B, C, D = experimental\ parameters\ (refer\ to\ \boldsymbol{Table\ 3}).$

These parameters are derived from the Rowe cell test by applying power functions to the data in an Excel spread sheet, and are presented in **Chart 3** and **Chart 4**.

Chart 3

Rowe Cell Result - Material Compressibility

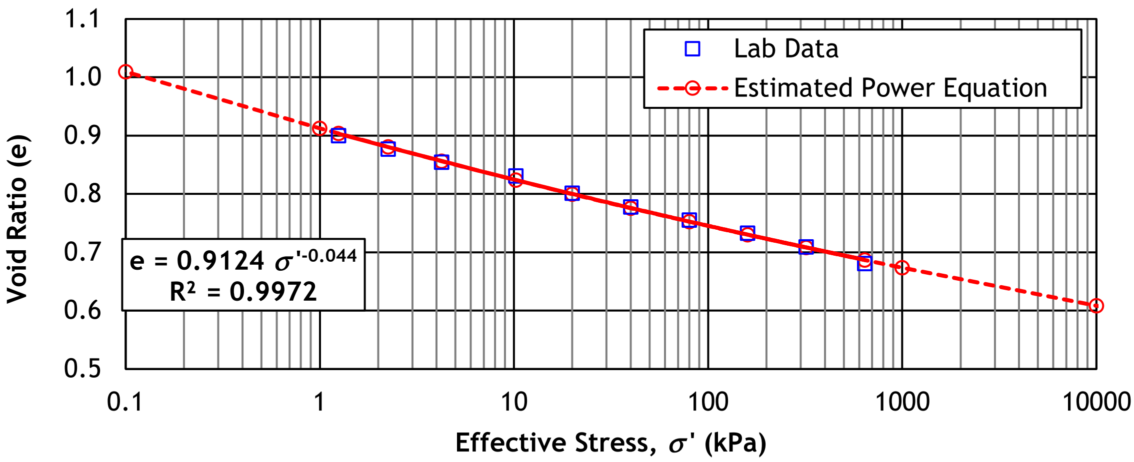
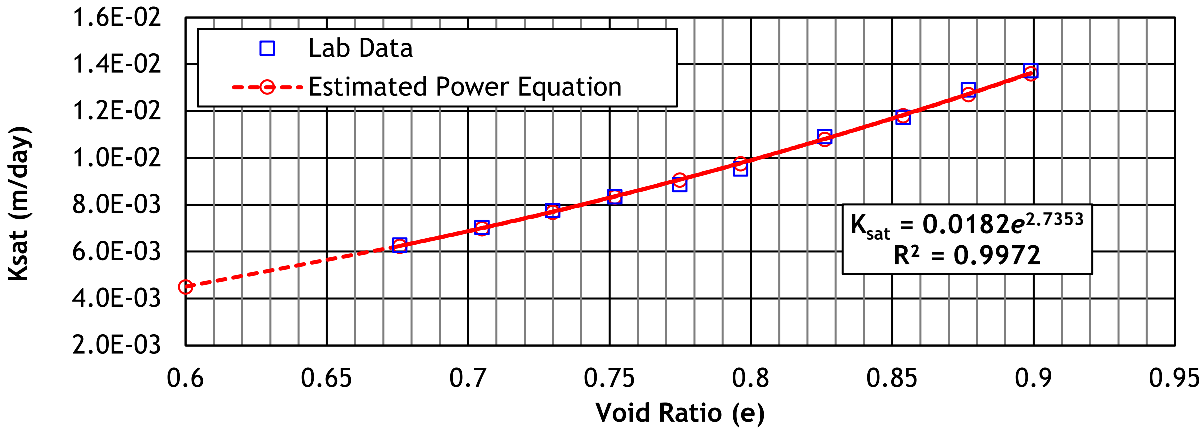


Chart 4

Rowe Cell Result - Material Permeability



As presented in **Chart 3** and **Chart 4**, the R-squared value for the plotted functions is 99.7%. Considering that the power functions also visually appear to closely mimic the data points, the generated power functions are well fitted to the Rowe Cell data.

The required parameters for defining the compressibility and permeability of the tailings are summarised in **Table 2**.

Table 2

Tailings Compressibility and Permeability Properties (Rowe Cell Data)

Material Property	Experimental Parameter			
	A	B	C	D
Compressibility	0.9124	-0.044	-	-
Permeability	-	-	0.0182	2.7353

4.3 Boundary Conditions

SV Consolidation **[Ref. 3]** allows the user to define different boundary conditions for SV Solid **[Ref. 4]** and SV Flux **[Ref. 5]**. SV Solid **[Ref. 4]** requires boundary conditions to be entered in terms of the degree of movement allowed in the material at certain locations. SV Flux **[Ref. 5]** boundary conditions control the drainage characteristics of the boundary, and can apply pressures as required.

Table 3 presents the boundary conditions used in the SV Consolidation **[Ref. 3]** model.

Table 3

Boundary Conditions

<table>
<tr><th rowspan="2">Boundary</th><th colspan="4">Boundary Conditions</th></tr>
<tr><th>SV Solid [Ref. 4]</th><th>Description</th><th>SV Flux [Ref. 5]</th><th>Description</th></tr>
<tr><td>Top of Model</td><td>Free</td><td rowspan="2">Allows free movement in the vertical direction</td><td>Pressure Head = 0.4m</td><td>Maintains a constant pressure head of 400 mm to reflect the normal operating level of the decant pond</td></tr>
<tr><td>Intermediate Boundaries</td><td>Free</td><td>No BC</td><td>Unrestricted</td></tr>
<tr><td>Base of Tailings/ Surface of Foundations</td><td>Fixed</td><td rowspan="2">Does not allow movement in any direction</td><td>No BC</td><td>Unrestricted</td></tr>
<tr><td>Base of Foundations</td><td>Fixed</td><td>EPWP Constant = 0</td><td>Prevents excess pore water pressure building, allowing a base drainage.</td></tr>
</table>

Two-way drainage has been considered in the model with the assumption that some pore water also escapes from the foundation. This was based on premise that, at least during the early stages of deposition, the least path of resistance for water flow will be to the base. Two way drainage has therefore been allowed in the model by taking two layers of foundation material at the base of the tailings with a permeability of 1 x 10^{-7} m/sec and 1 x 10^{-6} m/sec, based on in-situ permeability testing of the foundation, discussed in the Mining Proposal Design Report **[Ref. 2]**.

A zero Excess Pore Water Pressure (EPWP) boundary condition was applied at the base of the foundation material to allow drainage from the tailings to the foundations and allow some degree of EPWP generation at the base of the tailings, if the model parameters calculated it.

A constant pressure head boundary condition of 0.4 m was applied to the surface of the tailings to allow water to pond at the surface due to drainage through the top of the consolidation model and to account decant water at the surface of the tailings.

Foundation materials were modelled to be incompressible (i.e. no vertical movement allowed) by setting fixed boundary conditions at the base and surface of the foundations. All other boundary conditions were set to 'free' which allows the tailings to consolidate with time.

5 TONNAGE CALIBRATION

5.1 Overview

As previously discussed, this Report documents the results of one-dimensional consolidation modelling. In reality, tailings profiles are geometrically complex, and one-dimensional modelling is not representative of the entire tailings profile, which varies in depth and area with time. It is, therefore, necessary to calibrate the tonnages with the model, to ensure that the model is representative of the entire beach area and reflects that actual tonnages that will be deposited into the facility.

The tonnage calibration was conducted using an iterative process to arrive at the final filling curve, detailed in **Section 4.1.2**.

5.2 Methodology

Prior to commencing the tonnage calibration, the initial results of the modelling were first analyses to identify the consolidation behaviour of the tailings in the TSF. The results indicated that at no time during deposition does any meaningful Excess Pore Water Pressure (EPWP) generate (i.e. Maximum EPWP = 3 kPa). This suggests that the tailings rapidly consolidate and are essentially normally consolidated immediately after being deposited.

Since the tailings rapidly consolidate, it was assessed that the rate of rise will have negligible impact on the consolidation behaviour of these tailings. Therefore, it was decided that a single profile would be suitable for the modelling.

The tailings beach was then divided into 9 areas based on tailings depth. The tonnage was then estimated by multiplying each beach area by the Overall Insitu Density (OID) for the corresponding depth of tailings, determined from the consolidation model. The tailings volume was also assessed based on the calculated tonnage and OID.

Individual tailings tonnages and volumes for each beach area were then summed and compared to the target values. If the results varied by 5% or less, it was considered that the model was representative of the entire tailings beach area.

5.3 Results

Results of the tonnage calibration are presented in **Table 4**.

Table 4

Consolidation Model – Tonnage Calibration

Tailings Depth m	Tailings Beach Area m^2	OID t/m^3	Tonnage t	Volume m^3
2.5	50,000	1.541	192,635	125,000
5.0	477,554	1.512	4,374,441	2,893,976
10.0	612,119	1.670	11,135,060	6,667,701
15.0	469,608	1.670	14,555,584	8,715,919
25.0	1,125,553.32	1.680	49,749,978	29,613,082
35.0	2,045,888	1.700	124,144,576	74,790,927
45.0	2,700,593	1.700	210,616,046	123,891,792
55.0	1,953,980	1.710	185,489,659	108,473,485
60.0	314,705	1.710	34,204,579	20,002,678
		Σ	637,462,558	375,174,559
		Target	621,890,000	368,957,317
		% Difference	2.50	1.69
		< 5% Difference	Yes	Yes

As presented in **Table 4**, the tailings deposition from combining the consolidation modelling results with the tailings beach geometry culminates to a total tonnage of 637,462,558 tonnes and a volume of 375,174,559 m^3. The modelled tonnage and volume differ from the target values by only 2.50% and 1.69%, respectively. Therefore, ATCW has assessed the results of the modelling to be representative of the entire tailings beach area.

Detailed results are presented in the following Sections.

6 CONSOLIDATION MODEL

6.1 Graphical Plots

Graphical results of the initial modelling are provided in **Appendix A**. These results are presented in terms of:

i. Pore water pressure profiles and excess pore water pressure profiles (**Figures A1 - A2**).

ii. Dry density, Void ratio profiles, solid content profiles and saturated hydraulic conductivity profiles (**Figures A3 – A6**).

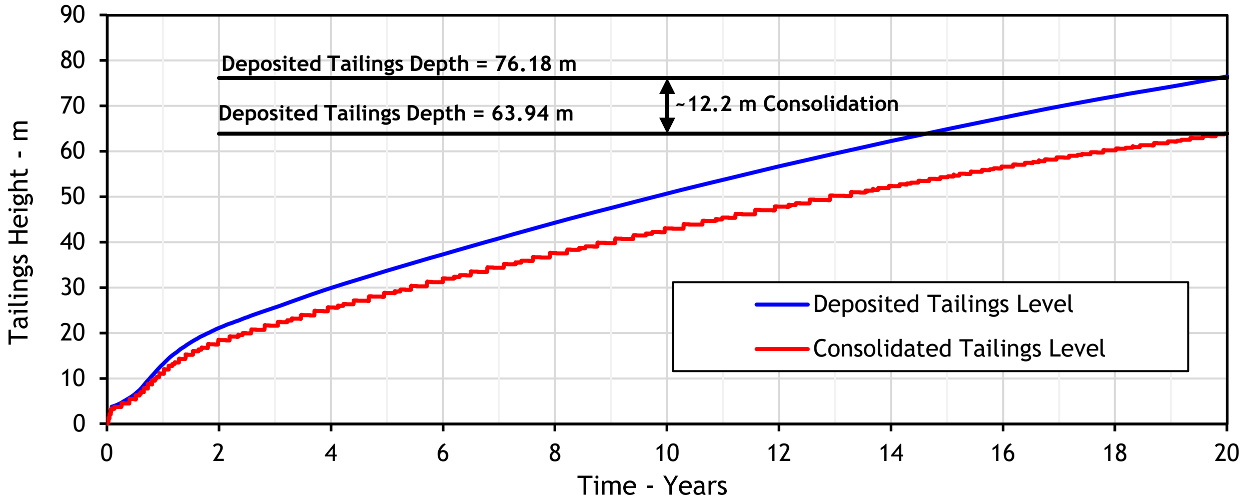
6.2 Consolidation (Tailings Deposition Period)

To replicate the predicted tonnages (refer **Section 5**), 76.2 m of tailings were deposited in the modelled column. Approximately 12.2 m of consolidation occurred during 20 years of deposition, resulting in a final consolidated height of nearly 64.0 m. Note that the column modelled represents the maximum tailings height in the beach.

This is presented graphically in **Chart 5** (over page).

Chart 5

Consolidation Model Results – Consolidation During Deposition



6.3 Pore Water and Excess Pore Water Pressure

Results of the consolidation model show that only negligible EPWP generates throughout the depositional life of the TSF (maximum EPWP = 3 kPa). The PWP profile therefore essentially represents the hydrostatic conditions for the entire operational life of the TSF, indicating that the tailings are in a normally consolidated state almost immediately after they are deposited into the facility.

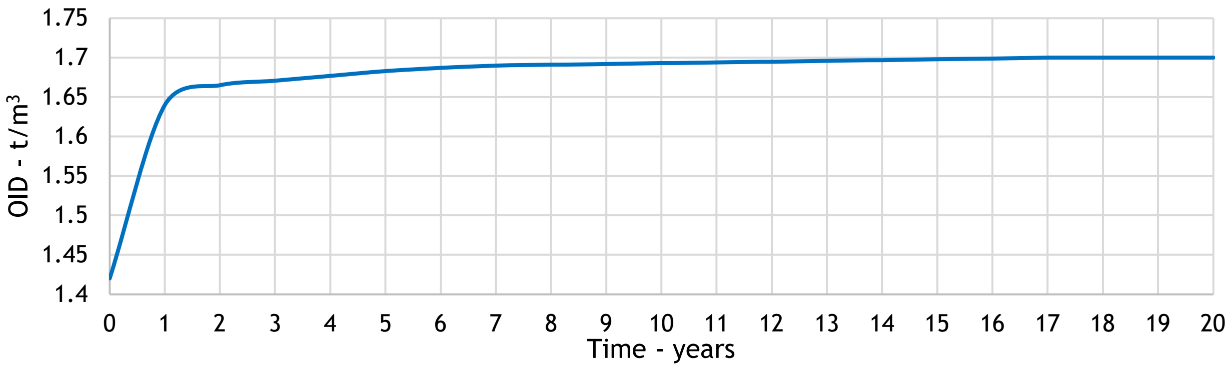
PWP and EPWP profiles are presented in **Figures A1** and **A2** of **Appendix A.**

6.4 Overall Insitu (Dry) Density

Interpretation of the dry density profiles from the consolidation modelling indicates that the tailings will quickly achieve an OID of around 1.64 t/m^3 within the first year of deposition. The OID will gradually increase to around 1.70 t/m^3 by the end of year 8 and maintains fairly constant for the remaining depositional life between 1.70 – 1.71 t/m^3, as shown in **Chart 6**, below.

Chart 6

Consolidation Model Results – Overall Insitu Density



Complete consolidation modelling outputs relating to material, showing dry density, void ratio and solids content profiles are presented in **Figures A3** to **A5** of **Appendix A**.

6.5 Hydraulic Conductivity

Hydraulic conductivity outputs indicate that the permeability of the tailings will reduce at the base of the tailings profile from approximately 9.2 x 10^{-8} m/s to 7.4 x 10^{-8} m/s by 20 years after commencement of deposition.

Hydraulic conductivity results from the consolidation modelling are presented graphically in **Figure A6** of **Appendix A**.

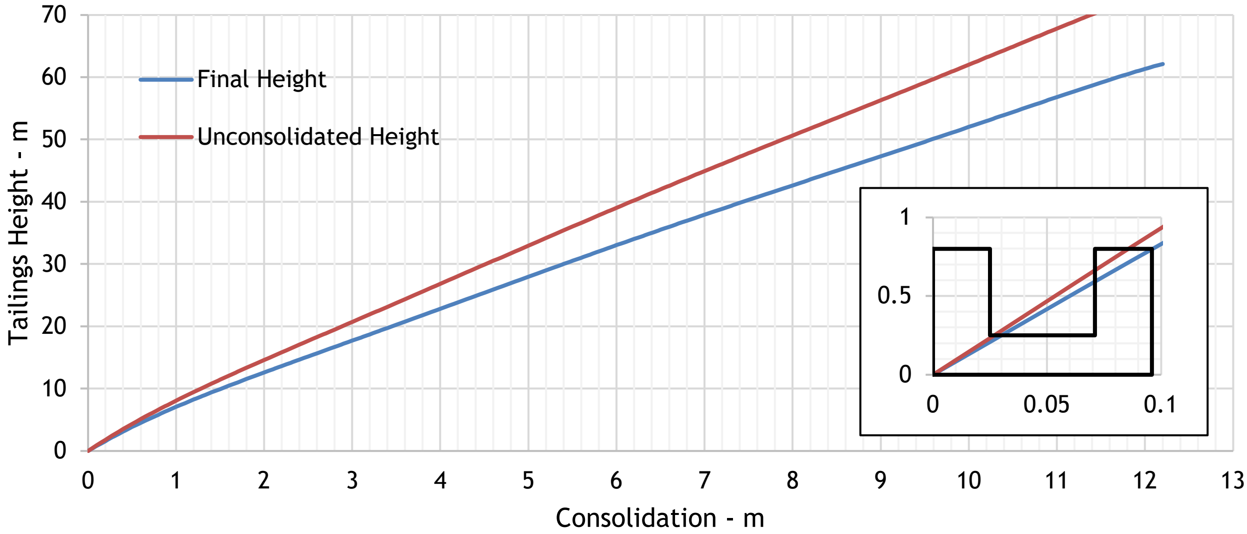
6.6 (Dry) Density at Decant Structure

The dry density at the level of the deepest box culvert is predicted to increase from 1.675 t/m^3 after the first year of deposition to 1.735 t/m^3 by the end of filling (at 20 years).

The box culvert protrudes from the face of the embankment by a maximum of 0.8 m (vertically). Throughout the depositional life of the TSF, a total of 9.6 cm of consolidation occurs at the level of the deepest box culvert. Total consolidation versus the tailings height (unconsolidated and final height) is presented in **Chart 7**.

Chart 7

Consolidation Model Results – (Dry) Density at Culvert



As discussed in **Section 6.3**, the North Star tailings consolidate rapidly resulting in an approximately normally consolidated profile throughout the depositional period. Hence, after only approximately 6 months of deposition into the TSF, only the tailings adjacent to the box culvert will experience only 3 cm of additional consolidation over the remaining 20 years. At this stage a vertical stress of approximately 150 kPa is acting on the culvert.

In the final 10 years of deposition, only a further 0.7 cm of consolidation will occur at the level of the deepest box culvert.

This indicates that by the time the full load of the tailings is acting on the box culverts, there will be negligible differential consolidation between the tailings column acting on the box culvert and the adjacent tailings columns.

7 DISCUSSION

7.1 Overall Insitu (Dry) Density

The results presented in **Section 6.2** and **Section 6.4** show that the OID during the first year of deposition increases from approximately 1.43 t/m^3 to 1.64 t/m^3, thereafter increasing to approximately 1.70 t/m^3.

The design report adopted an OID of 1.5 t/m^3 for the Stage 1 (starter) embankment, which is in operation for approximately 6 months, and 1.66 t/m^3 thereafter. The consolidation modelling results correlate with the adopted OID for the Stage 1 (starter) embankment, indicating an OID of between 1.5 t/m^3 to 1.55 t/m^3 after 6 months of deposition. The consolidation modelling results indicate that the assumption of 1.66 t/m^3, adopted in the design report, is likely to be slightly conservative, with the average modelled OID being 1.70 t/m^3 after the first eight years of deposition.

ATCW therefore recommends that the assumed design OID for the Stage 1 (starter) embankment be retained, however that the adopted OID be increased from 1.66 t/m^3 to 1.7 t/m^3 for the remaining depositional life.

7.2 Hydraulic Conductivity

As indicated in **Section 6.5**, the tailings will reach a hydraulic conductivity of 7.4×10^{-8} m/s at the upstream toe of the embankment by the end of deposition. This value is higher than the adopted value of 1×10^{-8} m/s, which was used in previous seepage modelling.

It is not expected that this will significantly impact the results, however ATCW recommends that for future modelling, a saturated hydraulic conductivity of 7.4×10^{-8} m/s be adopted for the tailings.

7.3 Box Culverts (Positive Projection)

Section 6.3 and **Section 6.6** showed that the tailings rapidly reach a normally consolidated profile, and negligible consolidation will occur at the level of the deepest box culvert after the first year of deposition, well before the maximum depth of tailings is reached.

Thereby, negligible relative consolidation settlement is expected to occur in the tailings column directly above the culvert compared to the adjacent tailings columns. This, in effect, means that although the box culverts are initially installed in a state of ‘positive’ projection, by the time the tailings column has developed adjacent and above the box culverts, this will no longer be the case.

Nonetheless, the design of the box culverts has been completed by Humes Holcim adopting soil-structure interaction factors for the ‘positive’ projection installation case, in accordance with AS 1597.2 for design of large box culverts, which based on the results presented herein, is appropriately conservative.

8 CONCLUSIONS AND RECOMMENDATIONS

The results of tailings consolidation modelling presented herein can be summarised as follows:

An average OID of 1.70 t/m^3 has been estimated at completion of filling of the TSF, with the OID during Stage 1 (i.e. first 6 months of operation) being retained from the design report at 1.55 t/m^3.

The permeability of the tailings column at the upstream toe of the embankment is approximately 7.4×10^{-8} m/s, which should be adopted for any subsequent seepage analyses.

The design of the decant system box culverts has adopted conservative factors for 'positive' projection installation and further assessment is not warranted.

9 CLOSURE

Your attention is drawn to the "Conditions of Report" which appear after the document history and status page of this report.

REFERENCES

[Ref. 1] ATC Williams (September 2019), "Iron Bridge Operations Pty Ltd, North Star Magnetite – Stage 2 Project, Tailings Laboratory Test Work Factual Report", Ref. 114185.20-R01.

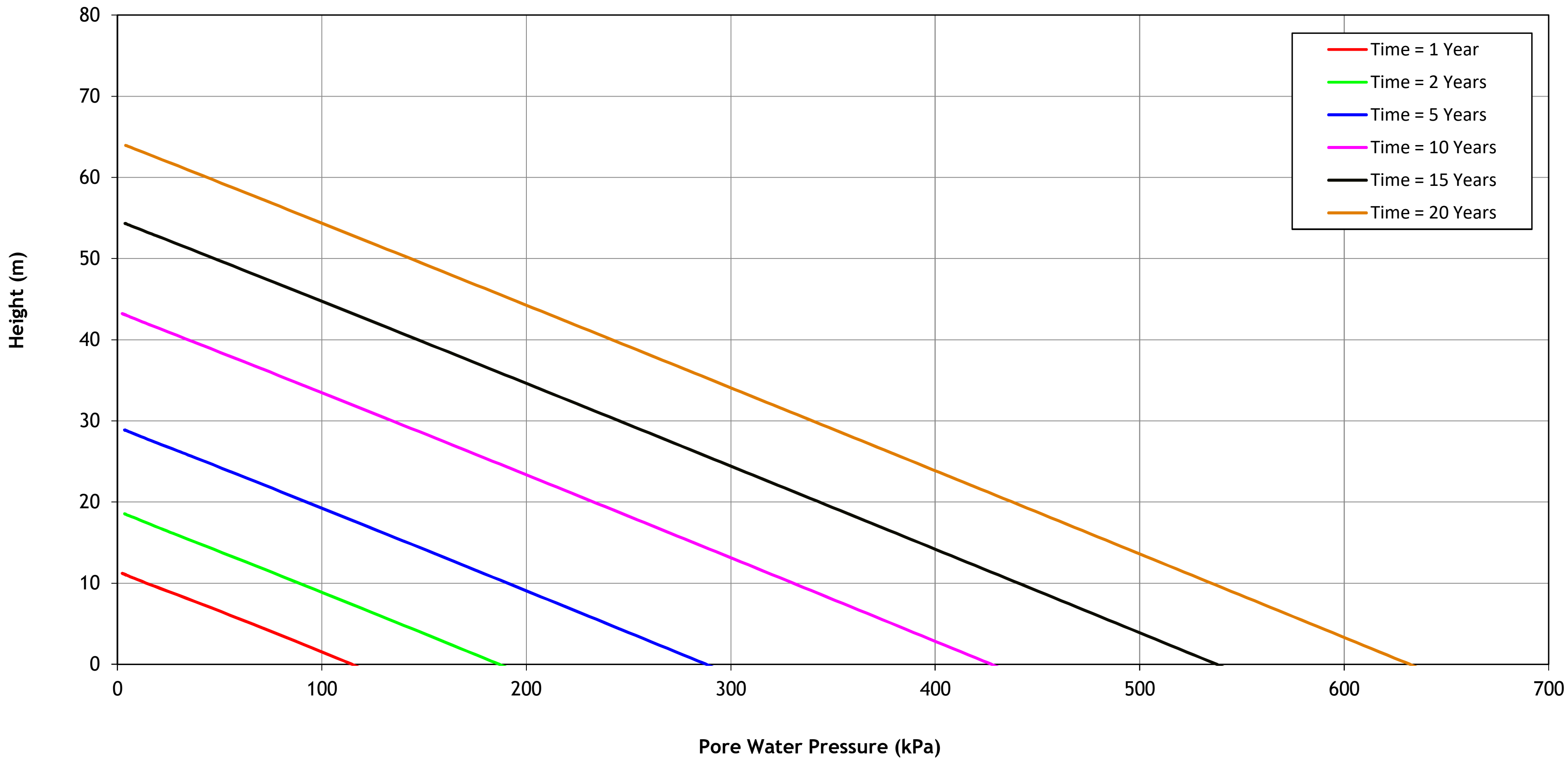
[Ref. 2] ATC Williams (March 2019), "Iron Bridge Operations Pty Ltd, North Star Magnetite – Stage 2 Project, Tailings Storage Facility & Return Water Pond, Mining Proposal Design Report", Ref. 114185.14-R03.

[Ref. 3] SVOffice (2009), "*SVSolid & SVFlux 1D/2D/3D Consolidation Modelling Software (Version 2.4.29)*", Soil Vision Systems Ltd.

[Ref. 4] SVOffice (2009), "*SV Solid 1D/2D/3D Finite Element Stress Deformation Modelling*", Soil Vision Systems Ltd.

[Ref. 5] SVOffice (2009), "*SV Flux 1D/2D/3D Saturated/Unsaturated Fine Element Groundwater Seepage Modelling*", Soil Vision Systems Ltd.

APPENDICES



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NORTH STAR MAGNETITE - STAGE 2

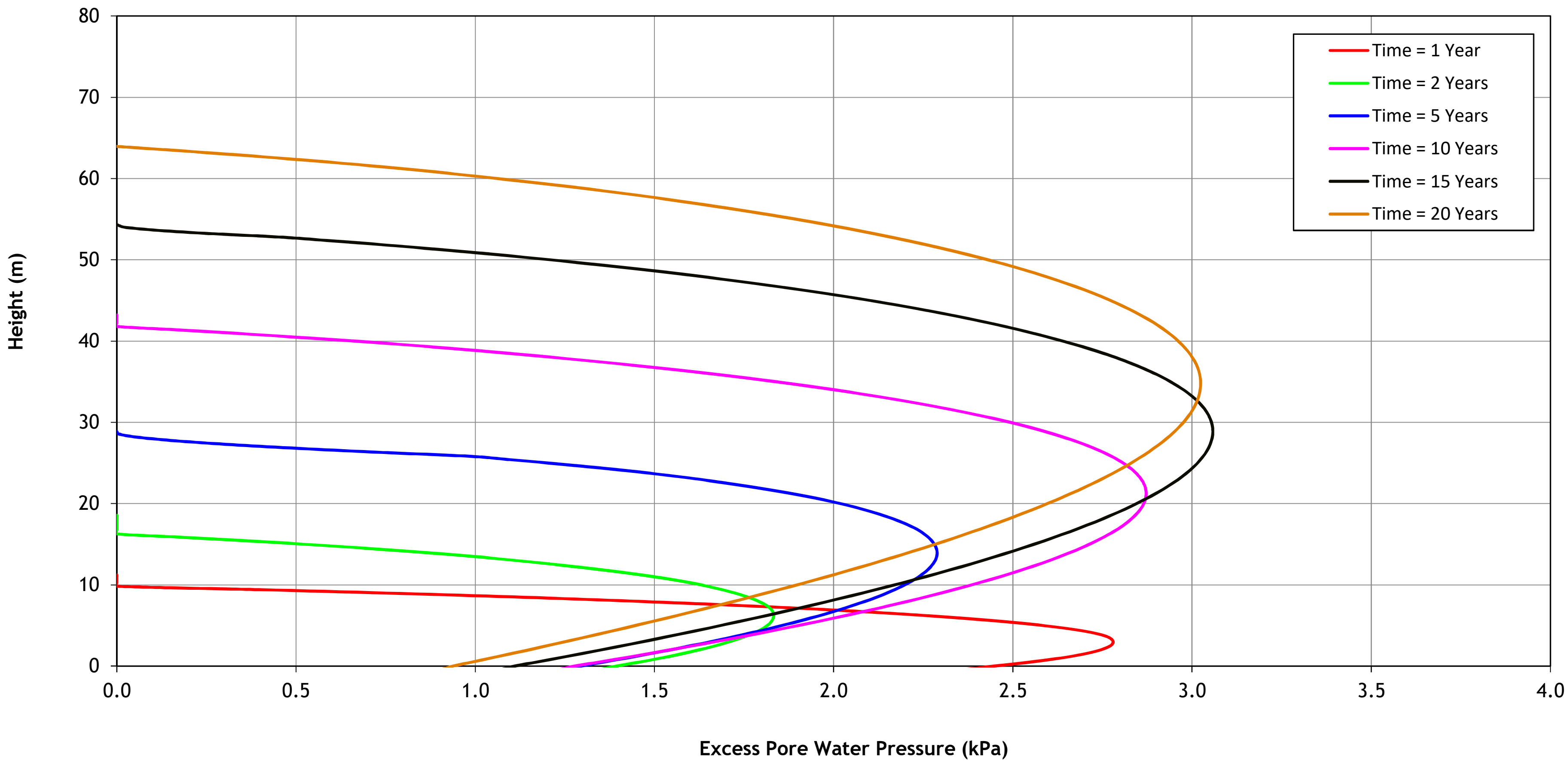
Consolidation Modelling

Consolidated Height of Tailings Column by the End of LOM = 64m

Pore Water Pressure Profiles

Date:	28/10/2019	**Job No:**	114185.20-R02	**FIGURE A1**

K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Data and Calcs\AF\Base Model_76m FIG A1



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NORTH STAR MAGNETITE - STAGE 2

Consolidation Modelling

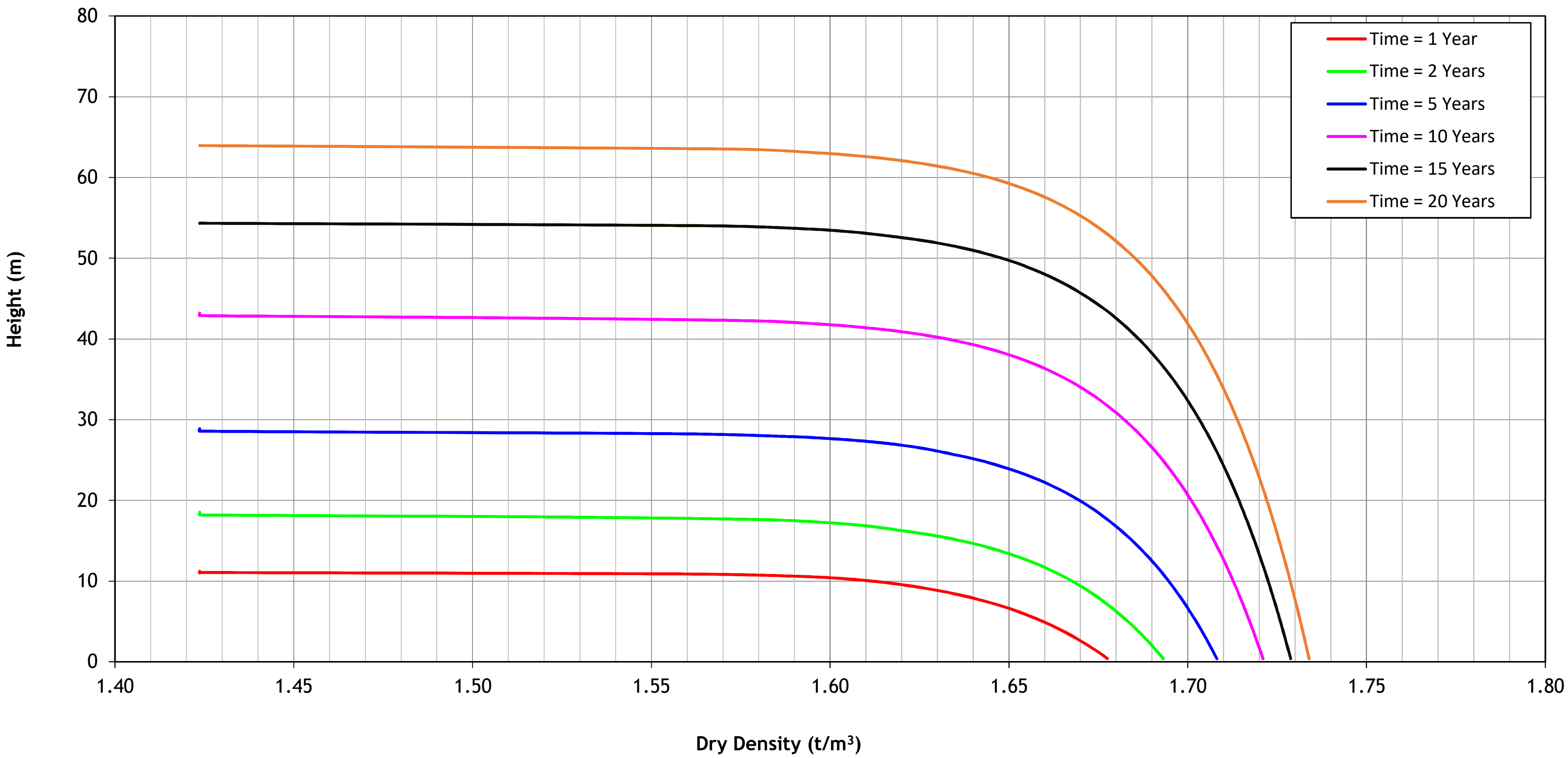
Consolidated Height of Tailings Column by the End of LOM = 64m

Excess Pore Water Pressure Profiles

Date: 28/10/2019 **Job No:** 114185.20-R02

FIGURE A2

K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Data and Calcs\AF\Base Model_76m FIG A2



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NORTH STAR MAGNETITE - STAGE 2

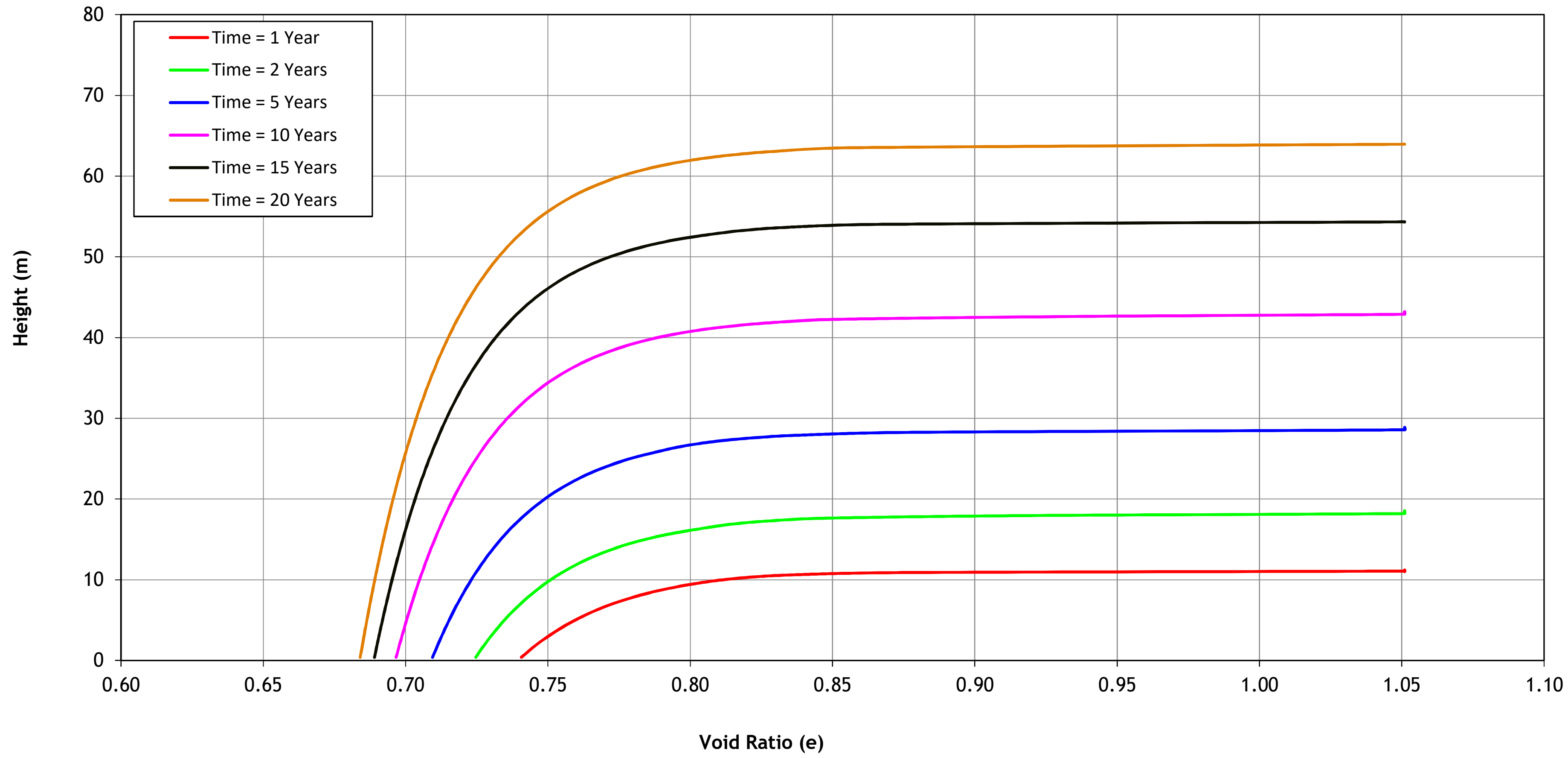
Consolidation Modelling

Consolidated Height of Tailings Column by the End of LOM = 64m

Dry Density Profiles

Date:	28/10/2019	**Job No:**	114185.20-R02	**FIGURE A3**

K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Data and Calcs\AF\Base Model_76m FIG A3



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NORTH STAR MAGNETITE - STAGE 2

Consolidation Modelling

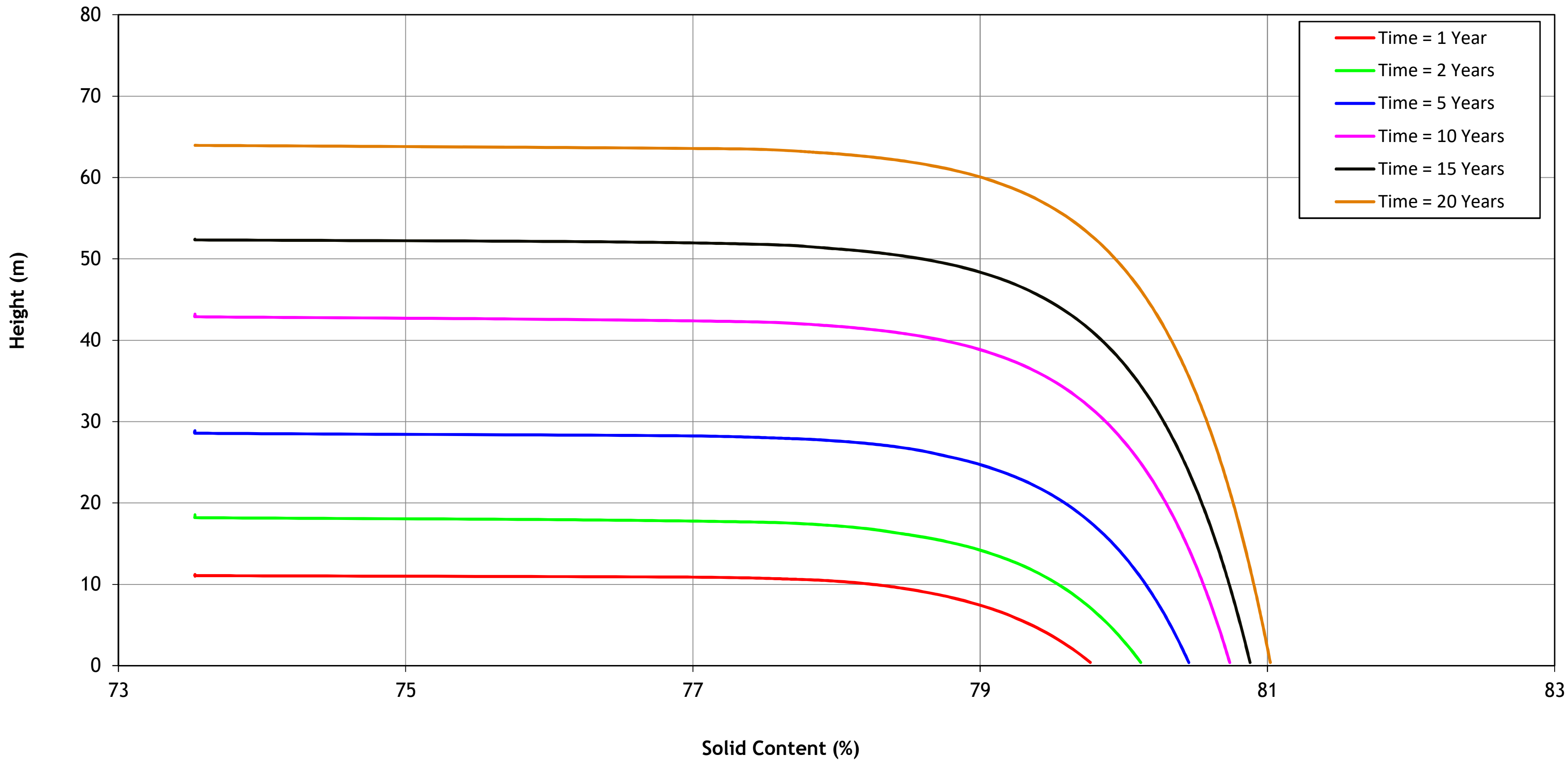
Consolidated Height of Tailings Column by the End of LOM = 64m

Void Ratio Profiles

Date: 28/10/2019 **Job No:** 114185.20-R02

FIGURE A4

K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Data and Calcs\AF\Base Model_76m FIG A4



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NORTH STAR MAGNETITE - STAGE 2

Consolidation Modelling

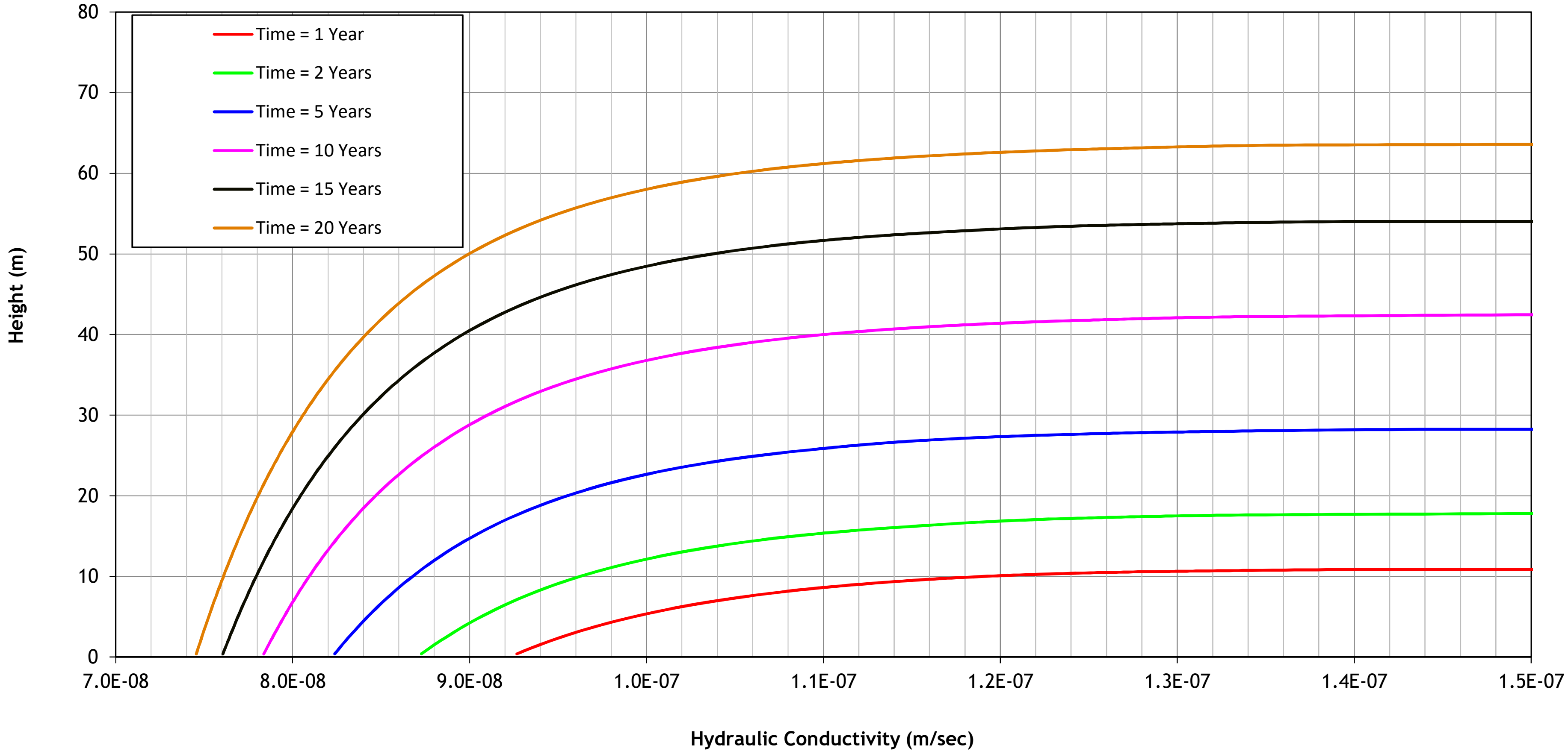
Consolidated Height of Tailings Column by the End of LOM = 64m

Solid Content Profiles

Date: 28/10/2019 **Job No:** 114185.20-R02

FIGURE A5

K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Data and Calcs\AF\Base Model_76m FIG A5



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NORTH STAR MAGNETITE - STAGE 2

Consolidation Modelling

Consolidated Height of Tailings Column by the End of LOM = 64m

Saturated Hydraulic Conductivity Profiles

Date: 28/10/2019 **Job No:** 114185.20-R02

FIGURE A6

K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Data and Calcs\AF\Base Model_76m FIG A6

APPENDIX D

ATC Williams

REPORT

IRON BRIDGE OPERATIONS PTY LTD

North Star Magnetite - Stage 2
Pilbara, Western Australia

Tailings Storage Facility &
Return Water Dam

Mining Proposal Design Report

Appendix D -
Dam Break Study

November 2018

Doc. No. 114185.14-R02
Revision Status: A

Document History and Status

Title:	North Star Magnetite TSF - Dam Break Study
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Authors:	Thomas Lu & Craig Noske
Reviewer:	John Leavy
Job Manager:	Craig Noske

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				Author	Reviewer
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ATCW Melbourne Office	-	On network

Craig Noske & Thomas Lu
Authors

John Leavy
Reviewer

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TABLE OF CONTENTS

FIGURES

1 INTRODUCTION

This Report documents the results of a Sunny Day Failure (SDF) dam break event of the proposed Tailings Storage Facility 2A (TSF 2A) and the Return Water Pond (RWP) at the North Star Magnetite – Stage 2 Project. The project is operated by Iron Bridge Operations (IBO) in the Pilbara Region of Western Australia.

The study was undertaken to assess the potential effects of a dam break situation on the downstream receiving environment and identified public infrastructure. The outcome of the study forms the basis for validation of an appropriate risk category for the facility.

A conservative approach was adopted for this dam break study, comprising the worst-case scenario where the TSF and RWP embankments fail consecutively in the same event (i.e. a coupled or cascading failure mechanism).

2 PROJECT BACKGROUND

2.1 Overview

The North Star TSF 2A and RWP site occupies a broad valley area in between a series of plateaus (mesas), hills and ridges. Tailings from the nearby process plant will be deposited down-valley from the east, and form an impoundment against the proposed cross-valley TSF embankments.

Tailings deposition and TSF embankment construction will take place in 7 stages over a nominal 20-year Life of Mine (LOM). For this dam break study, the final stage (end of filling condition) was adopted as the basis of the model.

Supernatant water from the tailings will be transferred via decant structures to the RWP located immediately downstream (west) of the TSF, where two embankments, one each side of a central hill abutment will be constructed to form a water storage pond for process plant reuse.

2.2 Site Description

The TSF 2A and RWP impoundment valley is approximately 4,400 m long, and is generally 2,000 m wide within the tailings storage area. The sides of the valley are generally steep, with a perimeter level mostly in excess of RL 350 m. The valley floor, by comparison, is relatively flat with small hills, and is generally sloping 1% from south-east to north-west.

The single drainage outlet is a narrow valley at the north-western end of the RWP. This valley runs for approximately 2,000 m before reaching the western extent of the ranges and discharging into Chinnamon Creek. The creek eventually leads to the tributaries of Turner River, which is some 45 km north-west of the North Star TSF.

Vegetation within the valley is relatively sparse, with ground generally covered with minor brush. Dense trees from small to medium sizes exist along the creeks or in the surrounding areas.

2.3 TSF Design

2.3.1 Embankment Geometry

The TSF main embankment crest levels and hydrological design criteria have been based upon the ANCOLD Tailings Dam Guidelines **[Ref. 1]**, with a dam failure consequence category of "High C" adopted for design purposes.

The TSF main embankment will have a crest width of 15 m, with upstream and downstream slopes of 2 : 1 (H : V). The embankment will be constructed in seven stages of downstream construction, with a final crest elevation of 323.3 m AHD. This will result in a maximum embankment height of approximately 68 m.

The length of the embankment from south abutment to north abutment is approximately 2,400 m. Each embankment stage will be equipped with an emergency spillway, designed to pass the peak flow from the critical duration storm event. This results in a final spillway invert of 322.1 m AHD.

2.3.2 Embankment Type

The TSF main embankment will be constructed with rockfill and granular filter materials, and lined with a bituminous geomembrane anchored on the upstream face of the embankment.

2.3.3 Capacity

The TSF has been designed to store 560 Mt of tailings over a 20 year LOM. Based on the design overall in-situ dry density, this equates to a storage capacity of 338 million m^3.

The estimated final tailings beach level at the TSF main embankment is RL 319.7 m. The remaining 2.4 m of freeboard to spillway invert level is to provide for the normal operating pond and the critical duration flood storage in the event that the decant system from the TSF to the RWP is not operational. This configuration corresponds to a maximum decant pond storage volume of 3,730 ML to the spillway invert level on top of the final tailings beach at the end of filling.

2.4 RWP Design

2.4.1 Embankment Geometry

Confinement for accumulated water in the RWP is achieved by the construction of two embankments (16 m and 10 m high respectively, with a crest elevation of 259 m AHD) approximately 1.5 km downstream of the TSF embankment. These embankments will be constructed to full height during the initial construction works, with a crest width of 10 m, and upstream and downstream slopes of 2 : 1 (H : V).

An emergency spillway is located at the north abutment, directly adjacent to the RWP northern embankment. The spillway invert level is designed at an elevation of 256.8 m AHD.

2.4.2 Embankment Type

The RWP Embankments will be water retaining, bituminous geomembrane-lined rockfill embankments.

2.4.3 Capacity

The maximum water storage capacity for the RWP is 5,140 ML at the spillway invert level.

3 DOWNSTREAM RECEIVING AREA

3.1 Overview

TSF2A and the RWP are located within a valley impoundment area at the head of Chinnamon Creek. Chinnamon Creek drains to the north-west for approximately 45 km until it intersects with a component tributary of the Turner River. The tributary crosses the Pilgangoora mining area access road, Pippingarra Road, the Port Hedland – Wittenoom Road and flows beneath three railway bridges (BHP, FMG and Roy Hill) before it flows into the Turner River.

Figure 114185.14_D1 presents a general layout plan of the TSF and RWP sites and downstream receiving area comprising Chinnamon Creek, Turner River, roads and rail crossings.

3.2 Chinnamon Creek and Turner River

Based on information provided by IBO, the area along Chinnamon Creek and the tributary of Turner River provides habitat for a number of significant fauna species, including:

- Pilbara Leaf-nosed Bat;
- Pilbara Olive Python;
- Northern Quoll; and
- Rainbow Bee-eater.

There are also individual specimens of the Declared Rare Flora (DRF) *Pityrodia sp Marble Bar* in this area.

Generally, a dam break situation would cause direct environmental impacts which include, but are not limited to, high velocity flows with the potential for erosion and scouring, and a release of pollutants along with tailings and water. In this case, a dam break event would likely result in the loss of habitat for the significant species and potential loss of the DRF specimens.

3.3 Public Infrastructure

From the initial investigation on the potential flow path, it was identified that a dam break flood would mainly flow along the creeks, which are crossed by several access roads and three railway bridges. The following is a summary:

- North Star Borefield Road (immediately downstream of the RWP Embankments);
- The Pilgangoora Mining Area access road (approximately 28 km north-west);
- Pippingarra Road and the adjacent Port Hedland – Wittenoom Road, approximately 37 km north-west; and
- Three railway crossings (BHP, FMG, Roy Hill, 37 km to 39 km north-west).

The first road that intersects the flow path is the North Star borefield road immediately downstream of the RWP Embankments. This is an IBO road, with only intermittent use and no public access.

The Pilgangoora access road crosses Chinnamon Creek further downstream, which connects the Lithium mines operated by Pilbara Minerals and Altura at Pilgangoora to the Great Northern Highway.

Pippingarra Road and the nearby Port Hedland – Wittenoom Road run northwards alongside the BHP rail line and cross Chinnamon creek approximately 10 km downstream of the Pilgangoora access road crossing. Each road has a broad floodway at the creek.

About 2 km further downstream, the FMG and Roy Hill railways cross Chinnamon Creek.

The railway crossings comprise bridges which have been designed to high standards to withstand extreme flood conditions. The railway bridges were inspected and identified to have minimum clearances from the creek bed of 4.3 m (BHP) and 6 m (FMG and Roy Hill). The crossings are illustrated in **Photos 1,2 & 3**.

With respect to other infrastructure in the downstream receiving area, satellite imagery and aerial photos indicate two open pits adjacent to Chinnamon Creek some 18 km downstream of the RWP embankments, which could be inundated in a dam break situation. However, these two pits are abandoned and rehabilitated gold mines that ceased operation in 1998. For the purpose of this dam break study, these pits were not examined for potential impacts from dam break inundation.

PHOTO 1 – BHP RAIL CROSSING

PHOTO 2 – FMG RAIL CROSSING

PHOTO 3 – ROY HILL RAIL CROSSING

4 SCOPE & MODELLING METHODOLOGY

4.1 Scope of Work

The scope of work for the TSF and RWP coupled dam break analysis has considered a two-stage process: firstly, the initial loss of impounded decant pond water and secondly the run-out of the flowable portion of the deposited tailings through the breach. The steps can be summarised as follows:

i. Assemble a 3-dimensional topographic model of the TSF2A and RWP site, together with the downstream receiving environment, extending north-west from the embankments;
ii. Estimate TSF and RWP embankment breach parameters and set up an initial condition for the critical case and volume of stored water, which is taken to be the maximum stored decant pond volume at the end of TSF filling and full RWP volume at spillway invert level;
iii. Based on the topographic model and the embankment alignments, create a 2-dimensional unsteady flow hydraulic model to simulate the dam break scenario;
iv. Run the hydraulic dam break model for the simulated breach of the TSF main embankment followed by the overtopping of the RWP Embankments, incorporating a total loss of 8,870 ML (refer to **Section 4.2.2**) of retained water;
v. Create flood inundation maps and estimate flood depths and velocities at the critical downstream locations identified in **Section 3**;
vi. Assess and estimate representative tailings density, shear strength and rheological properties for the end of filling scenario;
vii. Estimate the proportion of deposited tailings likely to be mobilised and released through the embankment breach
viii. Using a two-dimensional, non-Newtonian method, calculate, model and map the extent of tailings run-out through the embankment breach for the dam break scenario; and
ix. Provide assessments and discussions regarding the dam break outcomes.

4.2 Methodology

4.2.1 Failure Condition

Dam breach evaluations typically consider two initial hydrologic conditions:

- Sunny Day Failure (SDF); and
- Incremental Flood Failure (IFF).

A Sunny Day Failure represents a sudden dam failure that occurs for reasons other than flood induced overtopping, which may be caused by internal erosion, piping or earthquake. This failure condition generally involves little to no direct flooding of the facility or the downstream receiving environment. However, the existing conditions in the storage may conservatively adopt flood storage conditions, provided that they are within the design capacities of the facility.

Under an IFF situation, the failure is caused by uncontrolled overtopping of embankments due to extreme rainfall. In this case, the antecedent conditions in Chinnamon Creek and the Turner River catchment area would generate significantly larger flows than what could be expected due to the failure of the TSF and RWP embankments. This is due to the TSF2A site being at the head of the catchment, with the catchment area at the location of critical public infrastructure identified in **Section 3.3** being several time larger.

Therefore, for the IFF case, it is considered that a release of the maximum theoretical 8,870 ML of water stored in the TSF and the RWP at the end of filling would produce a negligible increase in the existing downstream flood flow depths and velocities.

Also, under an extreme flood condition due to storm surge, it would be expected that downstream public infrastructure would be closed until flood levels subside, eliminating risks for loss of life.

Conversely, a breach under the SDF condition is expected to induce considerable incremental flows into the downstream receiving area, especially during the dry season due to the ephemeral nature of Chinnamon Creek.

For this dam break study, the SDF condition (but allowing for maximum water accumulation in the RWP and TSF following extreme rainfall) was adopted since it was considered to be the critical case based on a comparison of expected incremental effects.

The adopted embankment breach failure modes were as follows:

- TSF Main Embankment – gradual piping failure starting from spillway invert level; and
- RWP Embankments – instantaneous overtopping failure once retained water reaches crest level.

4.2.2 Adopted Initial Conditions

The adopted TSF, RWP and downstream receiving conditions upon initiation of the coupled TSF and RWP dam break can be summarised as follows:

- TSF configuration:
  - Tailings beach at final (Year 20) level of RL 319.7 m ; and
  - Decant pond at maximum capacity of 3,730 ML, i.e. filled to spillway invert level of RL 322.1 m.
- RWP configuration:
  - Pond at maximum capacity of 5,140 ML, i.e. filled to spillway invert of RL 256.8 m.
- Downstream receiving area configuration:
  - Normal dry season flow condition, i.e. no antecedent flow in Chinnamon Creek.

It is important to note that whilst the above configuration has been adopted for the purposes of this dam break study, it is an extremely conservative case valid only at the very end of TSF operations and requiring the TSF decants to operate during an extreme rainfall event large enough to fill the RWP (5,140 ML) then become instantaneously blocked, accumulating the maximum amount of water (3,730 ML) behind the TSF embankment. In reality, this would be very unlikely to occur.

For the TSF decant pond and the RWP to both fill to spillway level without major storm events, a significant long-term change to the specified TSF and RWP water management practices would have to have occurred. In essence, this would need to involve the cessation of process water return pumping throughout the latter filling years, potentially coupled with closing of the TSF decants and the storage of additional mine water streams within the TSF during the majority of the operating life.

Nevertheless, the adopted configuration is considered valid for the purposes of understanding the maximum theoretical incremental effects of a coupled TSF and RWP dam break on the downstream receiving environment.

4.2.3 Adopted Modelling Techniques

Generally, a breach of a tailings retaining structure would result in the discharge of the entire supernatant pond volume, but not necessarily have to result in the full discharge of the impounded tailings volume. Therefore, the dam break modelling has adopted a two-stage process involving the following discharge mechanisms:

i. Initial flood wave – TSF decant pond and RWP water discharge; and

ii. Tailings run-out – the slumping or flow of the mobilised tailings from the TSF impoundment.

The initial flood wave is modelled as water, or Newtonian fluid, which is a simplifying assumption that conservatively results in high flow velocities and discharge rates, and longer run-out distances. This is discussed in detail in **Section 5**.

The piping breach formation at the TSF main embankment is modelled using Froehlich's empirical equations **[Ref. 2]**, where supernatant water in the TSF forms the flood wave surging into the RWP. The water level in the RWP then reaches the crest level, triggering the overtopping failure and the sudden destruction of the RWP embankments. The combined breach flow will form the flood wave that drains into the downstream receiving environment.

The tailings run-out modelling is then achieved separately using Jeyapalan's two-dimensional, non-Newtonian fluid modelling method **[Ref. 3]**, based on the rheological properties of the tailings (viscosity and yield strength). This is discussed in detail in **Section 6**.

5 DAM BREAK ANALYSIS – INITIAL FLOOD WAVE

5.1 Modelling Overview

5.1.1 Breach Parameter Estimation

For the SDF scenario, the characteristics of the TSF embankment breach, through which release into the RWP would occur, have been determined using Froehlich's empirical equations **[Ref. 2]**. Key inputs required for the Froehlich parameter estimation are the following:

- Volume of water lost through breach (V_w); and
- Ultimate breach depth.

The adopted breach geometry is assumed to be trapezoidal-shaped, with key breach outcomes comprising:

- Ultimate breach width (B_w);
- Breach formation side slopes (0.7 H: 1 V, as for piping failures); and
- Breach development time (t_f).

The breach parameters of the RWP embankments have been manually defined to simulate a full embankment breach.

5.1.2 Hydraulic (Flood) Model

2-Dimensional (2D) dam breach modelling was performed using the software package HEC-RAS 5.0.5 **[Ref. 4]**. HEC-RAS was developed by the U.S. Army Corps of Engineers Hydraulic Engineering Centre originally as a 1D flow modelling tool, with the 2D modelling capacities developed and released in 2016. For this dam break analysis, the TSF decant pond and the RWP were all modelled as 2D storage areas, and the downstream receiving area is modelled as a 2D flow grid.

The downstream extent is modelled to approximately 60 km north-west of the RWP, following the direction of Chinnamon Creek and the Turner River. Based on high-level assessments of the topography, the downstream inundation would initially be contained within the creeks and the overbank areas. Towards the model downstream boundary, the flows will essentially be confined to the Turner River banks.

HEC-RAS 2D hydraulic modelling does not require a pre-determined failure path, which helps to identify more realistic potential failure impact zones. The model estimates the flow paths, velocities and depths based on the hydraulic conditions and topography, and allows assessment of inundation of critical infrastructure and locations caused from the flood flows.

5.2 Model Inputs

5.2.1 Digital Elevation Model (DEM)

Accurate topographical information is required for the TSF, RWP and downstream receiving area to perform the dam break analysis. IBO had previously provided ATCW with 10 m grid aerial survey for the region covering the TSF and RWP area.

However, significant regions required for the dam break study to the north and west of the TSF and RWP were not provided. Thus, the underlying terrain DEM was obtained from ELVIS Elevation Foundation Spatial Data, Geoscience Australia **[Ref. 5]**. The selected ELVIS dataset is a 1 second SRTM derived Hydrological DEM (DEM-H), with a grid spacing of approximately 30 m. This DEM has been hydrologically conditioned and drainage enforced, hence is suitable for use in hydrological analyses including catchment definition and flood mapping.

To further improve the accuracy of the model terrain, the two data sources were combined to generate a final 3D terrain model for use in HEC-RAS.

5.2.2 Dam Breach Parameters

Flow into the system is generated by the breach of the embankments. One of the key considerations in the dam break modelling is the vertical extent of the breach propagation. This is essentially an engineering judgement, based on the embankment construction materials and compaction methodology, and the erosive actions of mobilised materials flowing through the breach.

The TSF main embankment will be geomembrane-lined and constructed predominantly of rockfill, with a granular upstream supporting zone for the liner. Although the granular supporting zone may erode, the high strength, compacted rockfill will be non-erosive and will hence provide resistance to the propagation of the embankment breach.

For the adopted initial conditions, a vertical breach depth of approximately 3.6 m will reach the final tailings beach surface, and will result in the outflow of all stored decant pond water. Beyond that level, it is estimated that the breach will continue to propagate as the flowable proportion of the deposited tailings mobilise, but will reach a point of equilibrium with increased depth as higher strength, more consolidated tailings are exposed.

Another factor which will limit the vertical extent of the breach is the increasing width of the embankment rockfill with depth.

For the dam break model, the TSF embankment has therefore been assumed to breach to approximately half of its total height (an approximate depth of 36.6 m, extending 33 m below the tailings surface level). Breach parameters are calculated using Froehlich's empirical equations **[Ref. 2]** and are summarised in **Table 1.**

TABLE 1
TSF MAIN EMBANKMENT BREACH PARAMETERS

Parameter	Value
Breach Top RL	323.3 m
Breach Base RL	286.7 m
Breach Bottom Width (B_w)	14 m
Breach Side Slope	0.7 H : 1 V
Breach Development Time (t_f)	0.3 hr

At the base of the breach the embankment width will be in excess of 150 m.

For the purpose of hydraulic modelling of the decant pond outflow, the portion of the breach below the base of the pond has been ignored (tailings outflow and run-out is analysed separately in **Section 6**). The breach bottom width was therefore increased linearly to 60 m to reflect the shallower but wider breach bottom related to the pond discharge model.

Figure 114185.14_D7 presents in "Section A", the adopted breach geometry for the TSF main embankment used in the HEC-RAS hydraulic modelling. Note that in the cross-section a larger breach is also shown to demonstrate the potential ultimate breach geometry after the release of the tailings. This is further discussed in **Section 6.5**.

With respect to the breaching of the RWP embankments, since they are smaller in scale and likely to be eroded completely once overtopped by the cascading flood inflow from the TSF, the breach was conservatively assumed to comprise the complete removal of the embankments in 0.05 hours (3 mins).

5.2.3 Computational Mesh

The computational mesh was defined to enable accurate calculations without sacrificing efficiency. A flow grid with 20 m spacing was adopted for the region of the TSF and RWP sites, and a 90 m mesh spacing was used for the downstream receiving area.

5.2.4 Roughness Coefficients

A Manning's roughness coefficient (n) value was assigned to the downstream catchment. Roughness coefficients represent the resistance to flood flows in channels and floodplains.

Appropriate Manning's 'n' values were assigned as per Chow V.T. **[Ref. 6]**. Photographs taken by ATCW on past site visits were used as a guide when assigning roughness coefficients.

The adopted Manning's 'n' value for modelling purpose was 0.05. This value is considered slightly conservative, but was also chosen to compensate for the lack of topographical roughness of the downstream hydro-enforced DEM.

5.2.5 Boundary Conditions

A downstream boundary condition was applied at the boundary of the model extent, approximately 60 km north-west. This boundary condition assumes normal flow conditions in the Turner River, and removes dam break flood water from the modelling domain.

5.3 Model Analysis

5.3.1 Model Run-Time

The HEC-RAS model was run for a total duration of 36 hours which allowed the following simulation sequences to be completed:

i. Pre-breach TSF decant pond and RWP conditions comprising water storage to spillway levels before TSF main embankment breach is triggered;
ii. Flood wave from TSF dam breach propagates into the RWP and brings the pond surface to RWP Embankment crest level (RL 259 m), which triggers the breach of the RWP embankments; and
iii. Flood wave propagates downstream into Chinnamon Creek, through the Turner River catchment and into the Turner River.

5.3.2 Time Step

The adopted time step is another critical aspect of the model computational setting. A fine time step setting is able to capture flow conditions during the rapid discharge period, but will increase the computational time and reduce efficiency. In the recent update of HEC-RAS **[Ref. 4]**, the capability of using adaptive time steps was introduced to increase the computation efficiency and was implemented in this dam break study.

During the model analysis, HEC-RAS can maintain suitable modelling conditions by adapting the time steps under control by Courant conditions. A basic time step of 2 seconds was used for the simulation, and it was adaptively controlled between 1 second to 32 seconds. During the time period around the peak breach flow, the time step was on average set at 1 second.

5.4 Model Verification

No historical flood information or stream flow gauge data was available for Chinnamon Creek to calibrate the hydraulic model. However, several manual verification calculations were performed at selected locations, and an acceptable overall correlation was found.

5.5 Summary of Results

5.5.1 Overview

As the dam break flood wave travels downstream, the depth and velocity generally dissipate. The maximum inundation depth and velocity recorded across the downstream receiving area is shown in **Figures 114185.14_D2** and **D3** respectively.

Before assessing **Figures 114185.14_D2** and **D3**, it is important to note that whilst they present the maximum inundation characteristics, the water surfaces as well as the values shown in the figures never actually occur simultaneously. These are rather a representation of the maximum inundation area and the maximum values over the entire model duration.

Staged inundation maps, at 0, 2, 2.5, 3, 8, 11, 15 and 19 hours after the initial breach are shown in **Figures 114185.14_D4** and **D5** which represent instantaneous flood depths as the flood wave is routed through the downstream receiving area.

The **T = 0** inundation plot in **Figure 114185.14_D4** shows the stabilised initial condition for the model before the breach, where both the TSF decant pond and the RWP are filled to spillway invert level. Then after the initial TSF main embankment breach, as the breach formation progresses, the RWP is gradually filled and the spillway becomes operational. From the **T = 2 hours** inundation plot, the RWP is shown to be filled to embankment crest level at RL 259 m; the embankment breach is then triggered whilst breach flow from the TSF is still filling the pond.

Upon breaching the RWP embankments, the combined flood wave travels rapidly along Chinnamon Creek and its overbank area, and inundates the North Star borefield road immediately (after 20 - 30 minutes, shown in the **T = 2.5 hours** inundation plot).

Figure 114185.14_D5 demonstrates further flood propagation within Chinnamon Creek, arriving at the downstream public infrastructure defined in **Section 3.3**, and eventually joining the Turner River.

A summary of the results at the North Star borefield road, the Pilgangoora access road, the Port Hedland – Wittenoom Road and the railway bridges are presented in **Table 2**.

TABLE 2
FLOOD INUNDATION RESULTS

Location	Maximum Inundation Width (m)	Maximum Depth (m)	Maximum Velocity (m/s)	Time from Breach to Inundation (hrs)
North Star Borefield Road	450	3.3	3.3	0.5
Pilgangoora Access Road	650	2.1	1.6	8.0
Port Hedland - Wittenoom Road	850	1.3	1.0	11.0
Railway bridges	600	2.0	0.8	11.5

5.5.2 Discussion

Extent of Inundation

From **Figures 114185.14_D2** and **D3**, it is estimated that the maximum area of inundation within the modelled extents is approximately 4,450 hectares.

Generally, the flood within the valley area downstream of the RWP is confined to a narrow width with significant flood depths (>7 m), whilst the inundated area in Chinnamon Creek spans a maximum width of approximately 1.3 km. The inundated width in the Turner River is between 500 m to 1 km (confined to the river banks), and at a shallower depth of 0.6 m to 1.5 m.

The estimated inundated widths over the downstream infrastructure shown in **Table 2** demonstrate that the dam break flooding can extend over a significant area over the roads, with the potential to cause loss of life to itinerant road users.

Inundation Velocity

The maximum inundation velocities summarised in **Table 2** also indicate the potential risk to the downstream infrastructure users, especially the North Star Borefield Road immediately downstream of the RWP. However, as identified in **Section 3.3**, this is an IBO road, with only intermittent use and no public access.

From **Figure 114185.14_D3**, it is identified that the maximum velocity in Chinnamon Creek immediately downstream of the RWP can reach up to 5 m/s. Such velocities are capable of causing significant scouring and erosion, as well as bulk transport of solid particles such as gravels alongside the creek bank.

The inundation velocity in Chinnamon Creek further downstream is reduced to less than 1.3 – 2.0 m/s, while the velocity for the flood in the Turner River is generally below 1.0 m/s. Such velocities would normally cause minor scouring and erosion, with limited potential for vegetation stripping or bulk transport of solid particles.

Based on the height of the railway bridges (4.3 m and 6 m), the flood will not inundate the railway tracks on top of the bridges, nor would the velocities be expected to cause damage to these structures.

It is also noted that the flood wave will take 8 hours to reach the nearest public road, by which time evacuation notifications would have been initiated.

6 TAILINGS RUN-OUT MODELLING

6.1 Overview

The dam break assessment for the North Star TSF 2A has been conducted in two stages, with the release of stored water considered to occur first (as described in **Section 5**), followed by the release and run-out of mobilised tailings as the embankment breach propagates.

The modelling of conventional water outflows from a dam break is generally well-understood and utilises well-established methodologies. However, the modelling and estimation of tailings run-out volumes and distances is more complex due to the variables involved, and requires greater qualitative engineering assessment in order to adopt representative modelling techniques and input parameters.

The procedure involves three general steps, as follows:

i. Identify breach location and determine breach parameters;

ii. Estimate the volume of tailings likely to be released from the dam break; and

iii. Estimate the run-out distance and inundation area resulting from the dam break.

Step **i.** has been previously described in **Section 5**, whilst the remaining steps are outlined in the following sections.

6.2 Released Tailings Volume

6.2.1 Background

Estimating the proportion of deposited tailings likely to be mobilised and released through the embankment breach is the most difficult and subjective analysis step.

Some of the more commonly used methods include "rule of thumb" approaches, assumed post-failure slopes, and empirically-derived storage-height relationships.

"Rule of thumb" estimates (arbitrarily picking a proportion of the total stored volume to be released) are not considered to be current best practice. Empirically-derived approaches are also highly variable with no broad consensus. For example, ICOLD **[Ref. 7]** present statistics from 40 historical failures where the proportion of released tailings ranged from 1% to 100% of the stored volume, with an average of 37%. However, this data takes no account of the storage configuration, tailings consolidation density and shear strength characteristics, or historical water management practices, all of which are significant contributors to the potential quantity of released tailings in a dam break.

6.2.2 Adopted Approach

The adopted approach for the North Star TSF 2A has hence been to estimate a post-failure slope, with the gradient being a function of the consolidation density and shear strength profiles within the tailings. The slope has been estimated using liquefaction flow slide theory, as described by Seddon **[Ref. 8]**. The post-failure, or equilibrium tailings slope resulting from an embankment breach is essentially a function of the post-liquefaction strength of the deposited tailings at the base of the breach.

The post-liquefaction steady-state (residual) shear strength is not constant, but is dependent on the vertical effective stress assuming a normally consolidated deposit. On the basis of previous ATCW test results for North Star tailings and tailings samples of similar characteristics as the North Star Magnetite materials, the post-liquefaction shear strength of tailings can be expressed as:

S_{us} = $k \times \sigma_{vo}'$, where

S_{us} = (steady-state) undrained, post liquefaction shear strength,

k = constant, strength reduction factor,

σ_{vo}' = vertical effective stress

The results for laboratory shear vane testing on similar tailings samples indicate that a value of k = 0.07 is appropriate. Results from triaxial testing suggest higher values of k, but for this analysis, we have conservatively taken the lower value.

The equations derived from the stability of long, shallow slopes ("infinite slope theory") may be used to analyse this situation. Based on the North Star tailings properties (refer to **Section 6.4**), the estimated tailings failure slope within the TSF resulting from a breach-induced mobilisation of the deposited tailings is approximately 4%.

For the embankment breach dimensions outlined in **Section 5.2.2** (i.e. a breach depth of approximately 33 m below the final tailings beach surface), this equates to a released volume in the order of 16 Mm^3.

6.3 Tailings Run-out Analysis Methodology

The tailings run-out analysis was performed using a two-dimensional, non-Newtonian method described by Jeyapalan **[Ref. 3]**. The liquefied flow of the tailings material is assumed to follow the Bingham plastic model (i.e. flow commences once the liquefied shear strength is exceeded).

The method is applicable to laminar flow, as observed with most liquefied tailings materials. The method does not take into account the frictional resistance from the flow channel sides and may, therefore, overestimate the inundation distance for situations where the flow slide runs through narrow valley channels. However, the downstream area from the TSF main embankment is located in the RWP impoundment, which is a wide valley with generally flat ground surface.

The model takes into account the following parameters when completing the calculations:

- Average slope over which the tailings material will flow;
- Height of the overall failure (vertical face);
- Total saturated density of the tailings material;
- Yield strength (post-liquefaction shear strength); and
- Tailings plastic viscosity.

The results of the analysis identify the equilibrium state of the mobilised tailings and the overall distance the liquefied tailings will flow. These results facilitate the creation of the tailings run-out inundation map.

6.4 Tailings Run-out Analysis Parameters

6.4.1 Average Bed Slope

The average bed slope is calculated perpendicular to the expected breach location and is presented as a percentage value. A positive slope represents flow downhill while a negative slope identifies flow that is contained within an area where the natural surface is sloping uphill. The average slope within the downstream area of the TSF main embankment is estimated to be 0.5%.

6.4.2 Height of Overall Failure

The estimated elevation of the final tailings beach profile immediately adjacent to the TSF main embankment is 319.7 m AHD. From **Section 5.2.2**, the base of the embankment breach is estimated to be 286.7 m AHD, which equates to approximately 33 m of tailings.

6.4.3 Total Density

Liquefied tailings are likely to be fully saturated under flow conditions. The total density parameter utilised is therefore the saturated unit weight of 20.7 kN/m^3, determined from the tailings specific gravity of 3.04 t/m^3 and adopted settled dry density of the deposited tailings of 1.65 t/m^3.

6.4.4 Yield Strength

For a wedge-shaped tailings failure block, the final yield strength can be determined from its representative effective strength and a conservative tailings liquefied shear strength ratio of 0.07. The adopted tailings yield strength is approximately 8.4 kPa.

6.4.5 Plastic Viscosity

The plastic viscosity of the tailings material has been assigned a very low value of 0.1 Pa·s for calculation purposes. This is a typical viscosity value observed in ATCW's past laboratory testing, and will provide additional conservatism in the tailings run-out estimation.

6.4.6 Run-out Gradient

The tailings run-out modelling using Jeyapalan's estimation method concludes that the tailings run-out will most likely conform to an average longitudinal gradient of 2.4%.

6.5 Adopted Tailings Run-out

The tailings run-out distance and inundation area has been estimated using a simple mass balance of "volume mobilised" equals "volume of outflow", and conforming to an average longitudinal gradient of 4% for the surface of the stored tailings post-failure and 2.4% for the tailings run-out.

The estimated tailings run-out volume is around 16 Mm^3, and the tailings are expected to flow approximately 1,600 m downstream of the TSF main embankment in the RWP area. This equates to a deposition area of approximately 128 ha.

As the tailings are being released, the initial embankment breach is expected to be further scoured and enlarged by the tailings flow. For this study, the ultimate breach bottom width is adopted to be the same with the maximum embankment width of approximately 300 m at the breach location.

Figure 114185.14_D6 presents an inundation map showing the approximate extents of tailings run-out, whilst **Figure 114185.14_D7** demonstrates the cross-section of the TSF embankment breach and a cross-sectional profile along the run-out extent.

6.6 Discussion

From **Figure 114185.14_D6**, it is identified that the tailings run-out is generally contained within the RWP impoundment, and only extends slightly beyond the location of the RWP embankments.

In this study, it was assumed that the tailings run-out in a dam break situation would only happen after the complete release of the stored water, and would reach the maximum distance once the flowing potential is counter-balanced by the resistance.

However, a review of case histories of tailings dam failures show that whilst all of the water flows out of the dam, there is generally a degree of mixing with the tailings.

As the breach develops and the water outflow reaches its peak discharge rate, the water entrains some tailings, which are carried as suspended solids. The released water flows for many kilometres downstream, to wherever the topography leads it, and the suspended tailings eventually settle out.

This mechanism is not covered in the analyses described in this report, however should be taken into consideration when the risks associated with the dam break and tailings run-out are being assessed.

7 CONCLUSIONS

Under a Sunny Day Failure condition, a dam break event at the North Star TSF and RWP can, in the worst-case scenario, produce a coupled failure of both TSF and RWP embankments. This is predicted to result in a sudden release of a maximum 8,870 ML of stored water, followed by the run-out of 16 Mm^3 of mobilised tailings.

On the basis of the summary provided in the above **Sections 5 and 6**, it is concluded that the overall dam break situation poses material risks associated with the potential loss of life, general environmental harm and damage to public assets. It is therefore recommended that a semi-quantitative Population-at-Risk (PAR) analysis be performed to establish an appropriate Consequence Category for the North Star TSF 2A.

8 CLOSURE

Your attention is drawn to the "Conditions of Report" which appear after the document and history page of this report.

ATC Williams

REFERENCES

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[2] Froehlich, David C., (December 2008) "Embankment Dam Breach Parameters and Their Uncertainties," Journal of Hydraulic Engineering, Vol 134, No. 12, pp. 1708-1721.

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[4] U.S. Army Corps of Engineers (2018), "Hydraulic Engineering Centre - River Analysis System, HEC-RAS v5.0.5" Available from: http://www.hec.usace.army.mil/

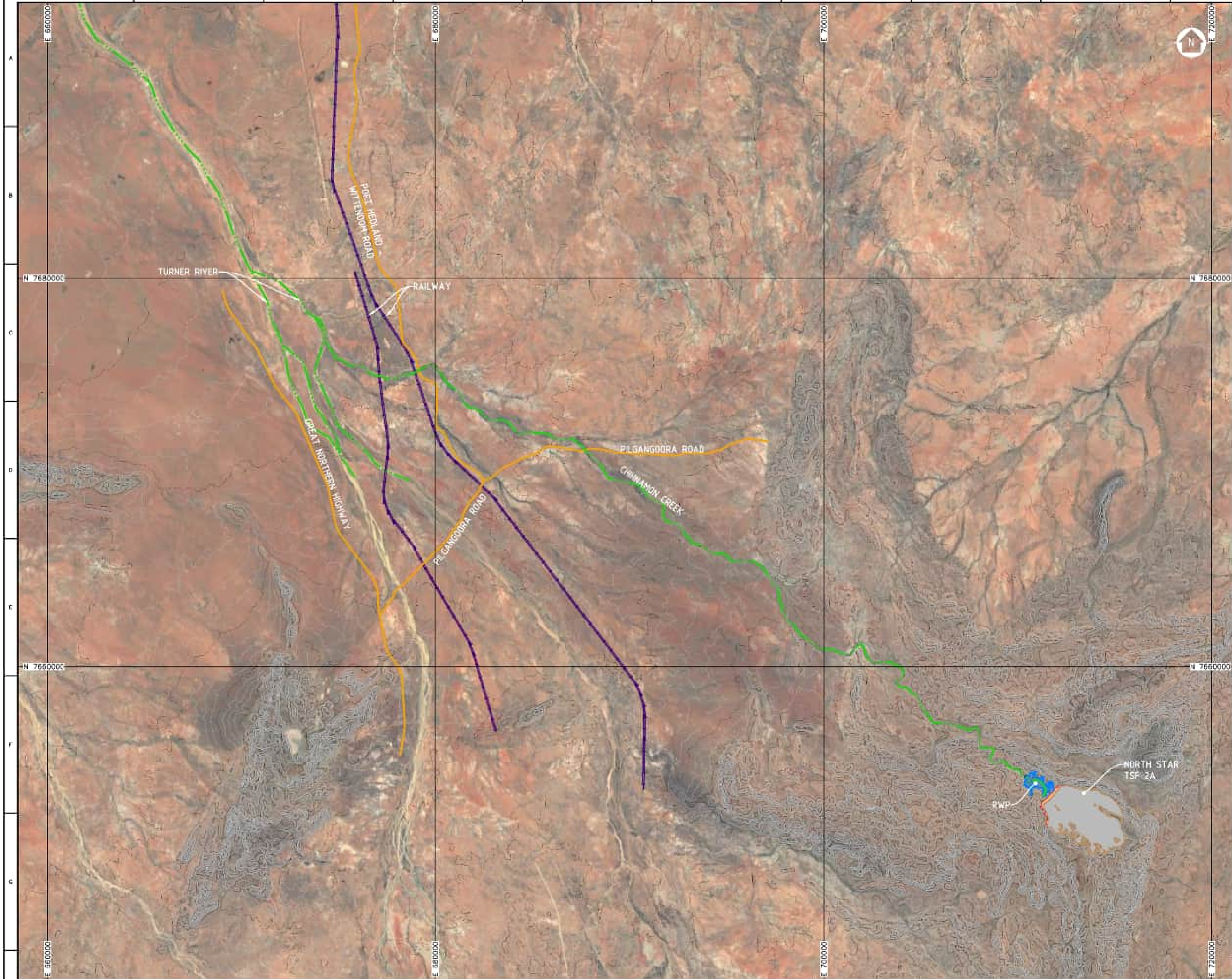
[5] Geoscience Australia. (2018), Elevation - Foundation Spatial Data. Elevation.fsdf.org.au [online]. 2018. [Accessed 9 July 2018]. Available from: http://elevation.fsdf.org.au/

[6] Chow, V.T., (1959), "Open-Channel Hydraulics", New York, McGraw-Hill, pp.680

[7] ICOLD (2001), "Tailings Dams – Incident Case Records, Tailings Dams Risk of Dangerous Occurrences", CIB ICOLD & UNEP PNUE, Paris, pp. 85-97.

[8] Seddon, K.D. (2007), "Post-Liquefaction of Thickened Tailings Beaches", Proceedings of the 10th International Seminar on Paste and Thickened Tailings (Paste 2007), Perth, Australia, pp. 395-411.

FIGURES



NOTES

1. COORDINATE SYSTEM WGS 84 / UTM ZONE 50S.
2. TSF DECANT POND EXTENT AND DEPTH INFORMATION SHOWN ARE REFERENCIAL.

LEGEND

- TSF AND RWP MAIN EMBANKMENTS
- ROAD
- RAILWAY
- WATERCOURSE

0 1000 2000 4000 8000
(IN METERS)
1:100000

ISSUED FOR REVIEW
NOT FOR CONSTRUCTION

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
114185.14_D1	GENERAL SITE LAYOUT	A	17.09.18	DRAFT ISSUE	TL	CN	-	-	-	-	

NORTH

PROJECT NUMBER: 114185.14

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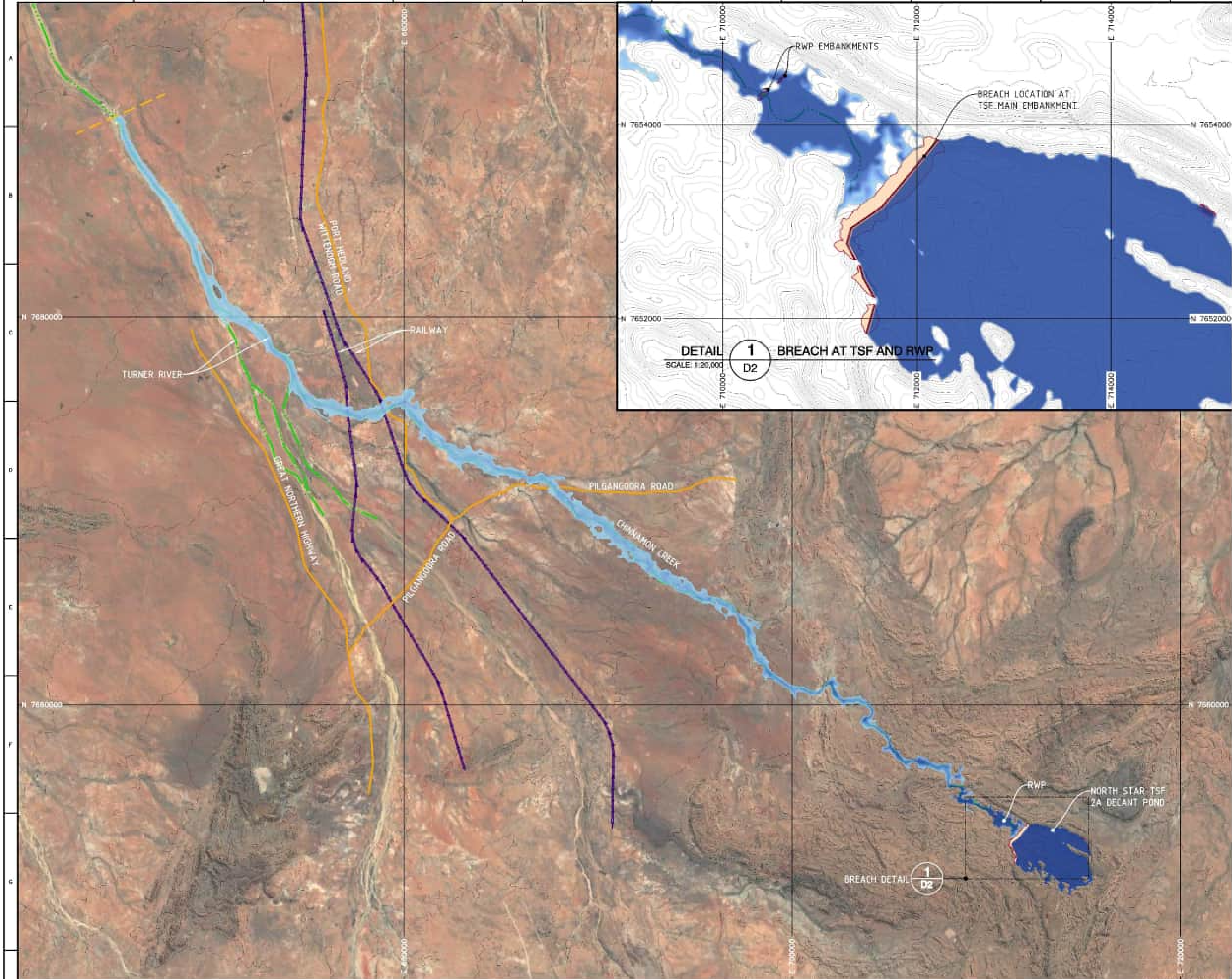
NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY 2A
DAM BREAK STUDY
GENERAL SITE LAYOUT

A1 — SCALE 1:100,000 — DRG No: — REV:



NOTES

1. COORDINATE SYSTEM WGS 84 / UTM ZONE 50S.
2. TSF DECANT POND EXTENT AND DEPTH INFORMATION SHOWN ARE REFERENCIAL.

LEGEND

- DEPTH 0-1m
- DEPTH 1-2m
- DEPTH 2-3m
- DEPTH 3-4m
- DEPTH 4-5m
- DEPTH 5-6m
- DEPTH >6m
- TSF AND RWP EMBANKMENTS
- ROAD
- RAILWAY
- WATERCOURSE
- MODEL BOUNDARY

0 1000 2000 4000 8000
(IN METERS)
1:100000

ISSUED FOR REVIEW
NOT FOR CONSTRUCTION

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
114185.14_D2	MAXIMUM INUNDATION DEPTH	A	17.09.18	DRAFT ISSUE	TL	CN	-	-	-	-	

VENDOR/DESIGNER DRG No. 114185.14_D2 REV: A

PROJECT NUMBER 114185.14

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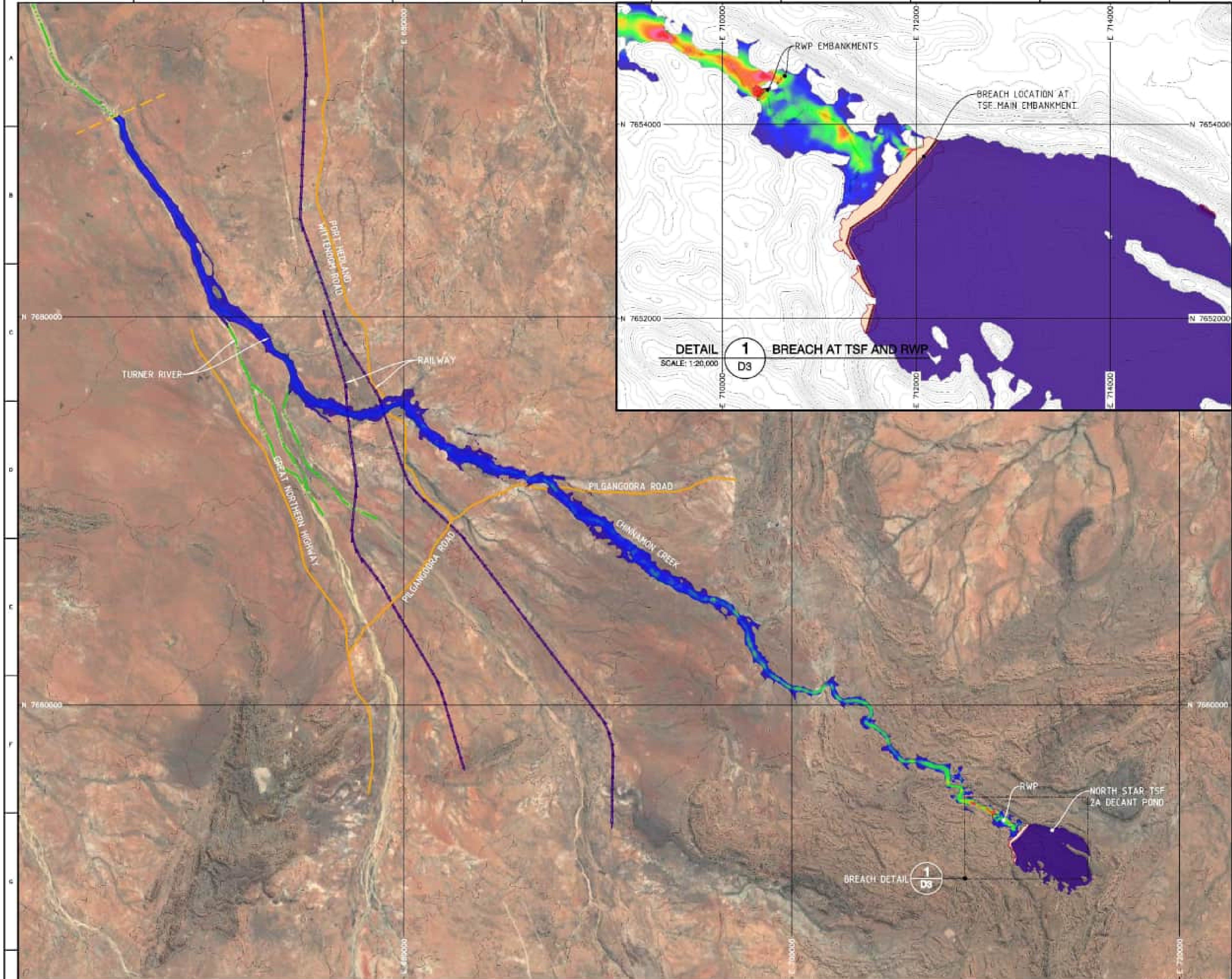
NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY 2A
DAM BREAK STUDY
MAXIMUM INUNDATION DEPTH

ATC Williams

NORTH

A1 SCALE 1:100,000 DRG No: REV:



NOTES

1. COORDINATE SYSTEM WGS 84 / UTM ZONE 50S.
2. TSF DECANT POND EXTENT AND VELOCITY INFORMATION SHOWN ARE REFERENCIAL.

LEGEND

- VELOCITY 0.0-1.0m/s
- VELOCITY 1.0-2.0m/s
- VELOCITY 2.0-3.0m/s
- VELOCITY 3.0-4.0m/s
- VELOCITY 4.0-5.0m/s
- VELOCITY 5.0-6.0m/s
- VELOCITY 6.0-7.0m/s
- VELOCITY >7.0m/s
- TSF AND RWP EMBANKMENTS
- ROAD
- RAILWAY
- WATERCOURSE
- MODEL BOUNDARY

0 1000 2000 4000 8000
(IN METERS)
1:100000

ISSUED FOR REVIEW
NOT FOR CONSTRUCTION

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
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VENDOR/DESIGNER DRG No. 114185.14_D3 REV: A

PROJECT NUMBER 114185.14

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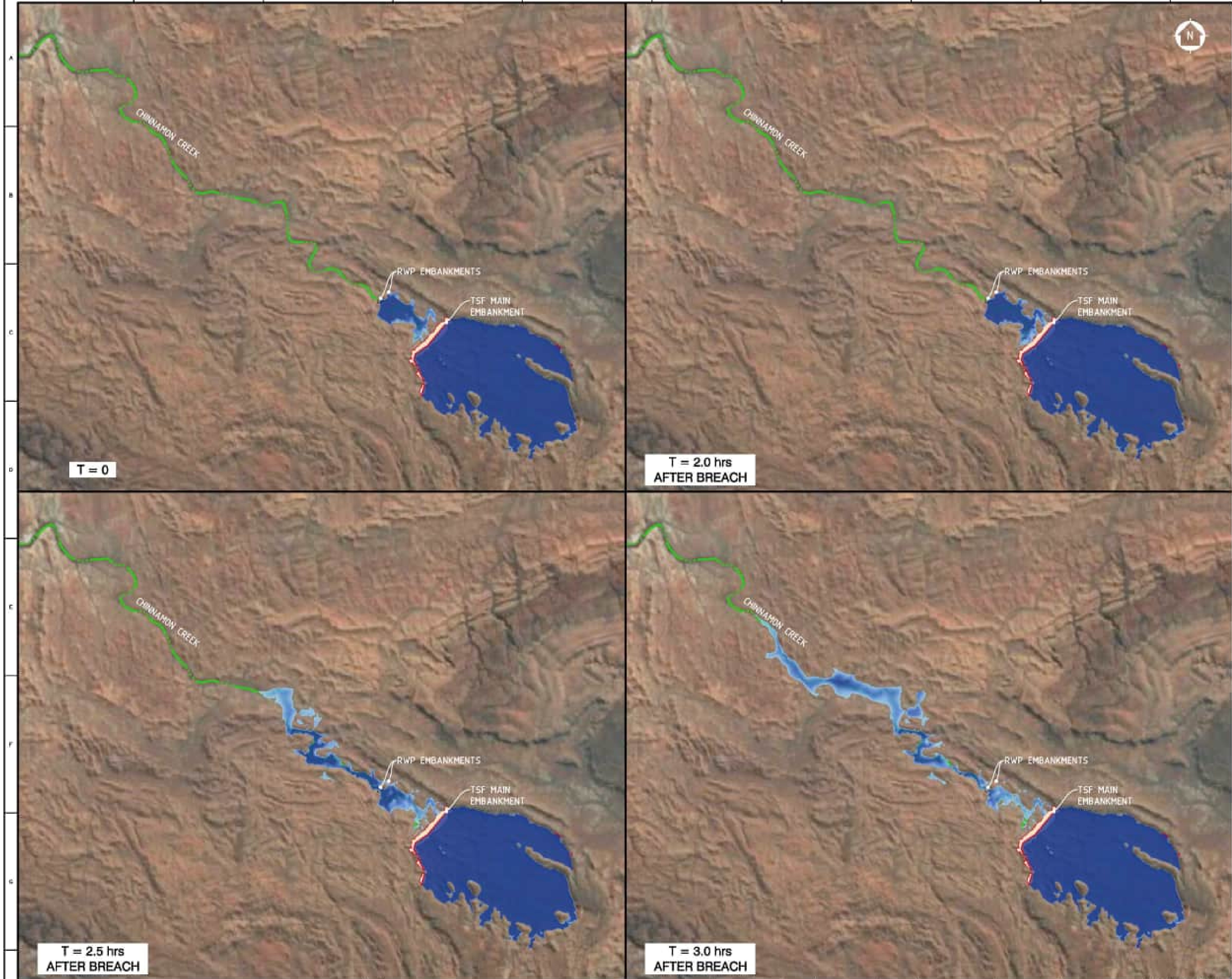
NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY 2A
DAM BREAK STUDY
MAXIMUM INUNDATION VELOCITY

A1 SCALE 1:100,000 DRG No: REV:



NOTES

1. COORDINATE SYSTEM WGS 84 / UTM ZONE 50S
2. TSF DECANT POND EXTENT AND DEPTH INFORMATION SHOWN ARE REFERENCIAL.
3. ESTIMATED TIME FOR EACH INUNDATION STAGE IS CALCULATED FROM THE INITIATION OF THE TSF MAIN EMBANKMENT BREACH.

LEGEND

- DEPTH 0-1m
- DEPTH 1-2m
- DEPTH 2-3m
- DEPTH 3-4m
- DEPTH 4-5m
- DEPTH 5-6m
- DEPTH >6m
- TSF AND RWP EMBANKMENTS
- WATERCOURSE

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REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
114185.14_D4	STAGES OF INUNDATION (1)	A	17.09.18	DRAFT ISSUE	TL	CN	-	-	-	-	

FMG

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NORTH

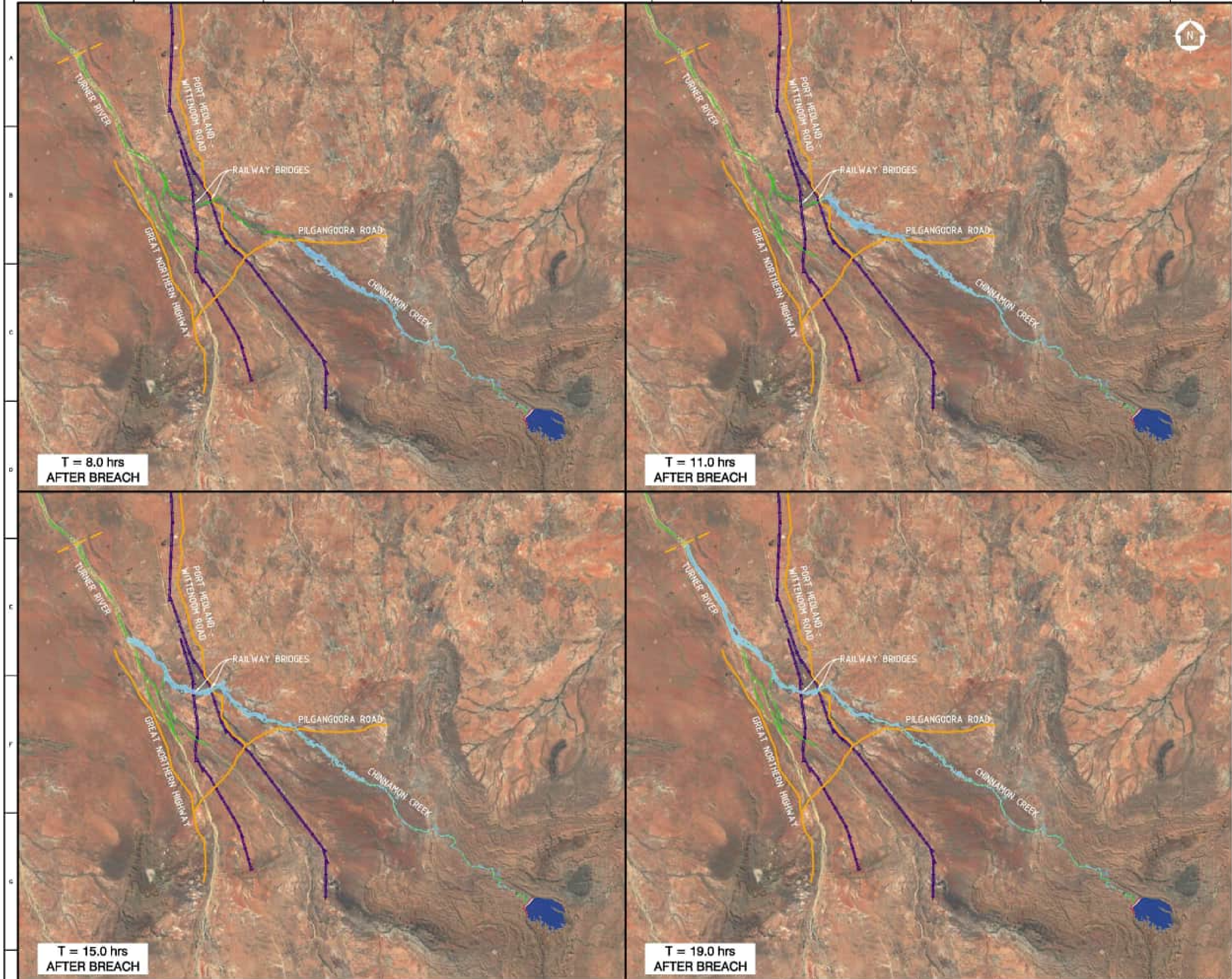
PROJECT NUMBER 114185.14

VENDOR/DESIGNER DRG No. 114185.14_D4 REV: A

Fortescue Metals Group Ltd

NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY 2A
DAM BREAK STUDY
STAGES OF INUNDATION (1)

A1 SCALE 1:50,000 DRG No: REV:



NOTES

1. COORDINATE SYSTEM WGS 84 / UTM ZONE 50S
2. TSF DECANT POND EXTENT AND DEPTH INFORMATION SHOWN ARE REFERENCIAL.
3. ESTIMATED TIME FOR EACH INUNDATION STAGE IS CALCULATED FROM THE INITIATION OF THE TSF MAIN EMBANKMENT BREACH.

LEGEND

- DEPTH 0-1m
- DEPTH 1-2m
- DEPTH 2-3m
- DEPTH 3-4m
- DEPTH 4-5m
- DEPTH 5-6m
- DEPTH >6m
- TSF AND RWP EMBANKMENTS
- ROAD
- RAILWAY
- WATERCOURSE
- MODEL BOUNDARY

(IN METERS) 1:200000

ISSUED FOR REVIEW
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NORTH

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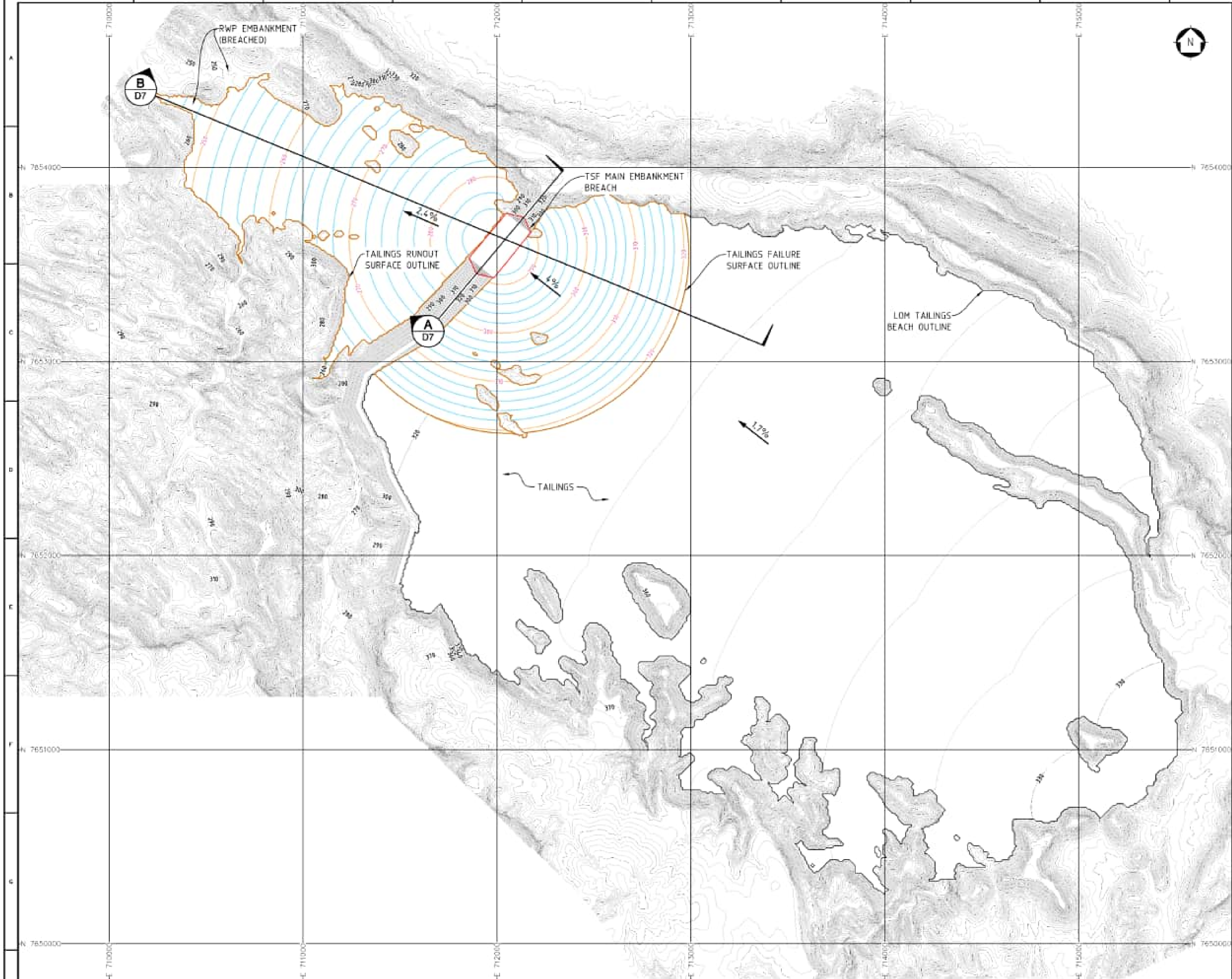
NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY 2A
DAM BREAK STUDY
STAGES OF INUNDATION (2)

A1 | SCALE 1:200,000 | DRG No: | REV:



NOTES

1. ESTIMATED VOLUME OF TAILINGS RUNOUT IS APPROX. 16Mm3
2. ESTIMATED TAILINGS RUNOUT DISTANCE IS APPROX. 1,600m

LEGEND

FINAL BREACH FOOTPRINT FOR TSF MAIN EMBANKMENT

OUTLINE FOR TAILINGS FAILURE SURFACE AND RUNOUT SURFACE

0 100 200 400 800 (IN METERS) 1:10000

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NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14_D6 REV: A

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
114185.14_D6	TAILINGS RUNOUT PLAN	A	11.09.18	DRAFT ISSUE	TL	CN	-	-	-	-	

FMG

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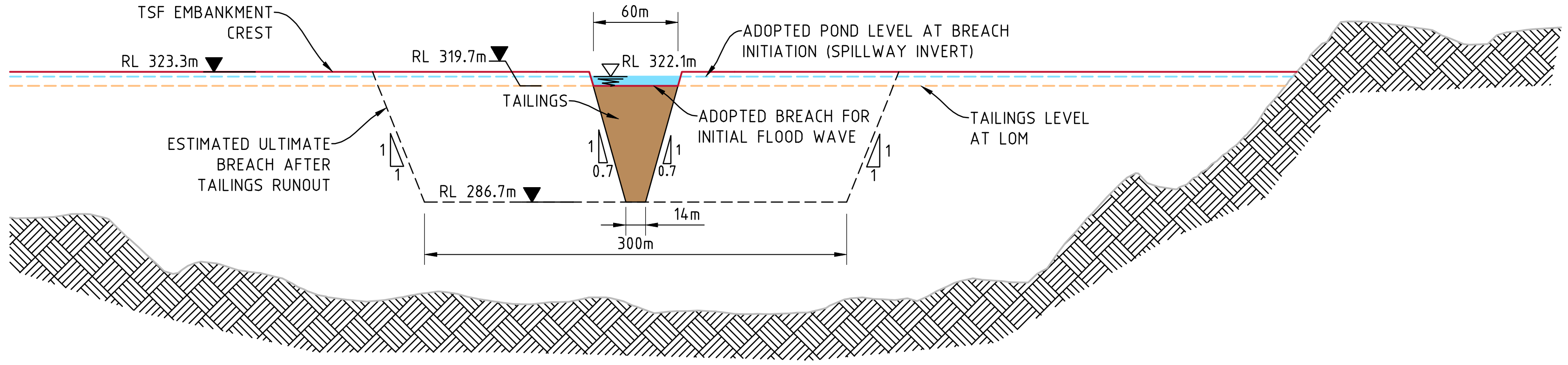
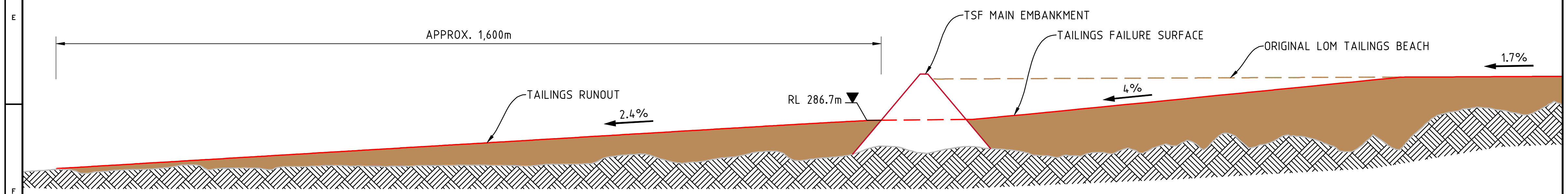
NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY 2A
DAM BREAK STUDY
TAILINGS RUNOUT PLAN

A1 SCALE 1:10,000 DRG No: REV:



SECTION A D6 TSF MAIN EMBANKMENT BREACH SECTION
SCALE: 1:2,500 (H)
1:1,000 (V)



SECTION B D6 TAILINGS RUNOUT CROSS SECTION
SCALE: 1:5,000 (H)
1:2,000 (V)

ISSUED FOR REVIEW
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REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
114185.14_D7	BREACH GEOMETRY & RUNOUT CROSS-SECTION	A	17.09.18	DRAFT ISSUE	TL	CN	-	-	-	-	

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VENDOR/DESIGNER DRG No. 114185.14_D7 REV: A

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NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY 2A
DAM BREAK STUDY
BREACH GEOMETRY & RUNOUT CROSS-SECTION

A1 SCALE AS DRAWN DRG No: REV:

APPENDIX E

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REPORT

IRON BRIDGE OPERATIONS PTY LTD

North Star Magnetite - Stage 2
Pilbara, Western Australia

Tailings Storage Facility & Return Water Dam

Mining Proposal Design Report

Appendix E - Rainfall Analysis & Water Balance

November 2018

Doc. No: 114185.14-R01
Revision Status: A

Document History and Status

Title:	Rainfall Analysis and Water Balance Report
Job Number/Extension	114185.14
Document Number:	114185.14R01
Project Office:	Melbourne
File Path:	K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Documents\R01 (Water Balance)\Text\114185.14R01 (RevA).docx
Author:	Thomas Lu/Craig Noske
Reviewer:	John Leavy
Job Manager:	Craig Noske

Rev.	Status	Issued to	Issue Date	Signatures	
				Author	Reviewer
A	Issue for Client Review	IBO	5.11.2018	TL/CN	JL

This Revision distributed To:	Hard Copy	Electronic Copy
IBO	-	☑
ATCW Melbourne Office	-	On network

Craig Noske & Thomas Lu
Authors

John Leavy
Reviewer

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EXECUTIVE SUMMARY

1. INTRODUCTION

This Report documents the collation and analysis of a representative climatic database and the development and operation of a 20 year Life Of Mine (LOM) water balance model for the North Star Magnetite Tailings Storage Facility 2A (TSF 2A).

In order to construct the climatic database, rainfall and evaporation records from the SILO (Science Information for Land Owners) database were used. The SILO patched records span for 100 years from 1918 to 2018. This climate dataset was then analysed and used to generate 1,000 years of statistically congruent synthetic rainfall data to be used in the subsequent water balance.

The water balance model used multiple realisations, each simulating a 20-year LOM situation, to generate statistical results.

2. CLIMATIC DATA

100 years of daily rainfall and evaporation data were obtained from SILO as the basis for the water balance modelling input. SILO patched data is a database of complete rainfall, evaporation and climatic records for locations across Australia with interpolated infills for missing data. The datasets are constructed from regional observation data obtained from the Australian Bureau of Meteorology.

However, the SILO historical dataset is not necessarily representative for future predictions. In order to achieve a statistical analysis on the water balance results using the limited historical climate dataset, it was decided to synthesise further data. This was achieved by developing 1,000 years of climate data using the same statistical characteristics as the existing 100-year historical record.

The Stochastic Climatic Library (SCL) computer program was used to generate synthetic daily climate data for use in the water balance. As a result, 1,000 years of synthetic climate data were generated and used for the water balance modelling. The synthetic climate datasets were also interrogated against the original records and confirmed on the same statistical characteristics.

These synthetic records were used in the water balance to test many alternative realisations that have equal probability of occurring. This allows for a much broader analysis in the water balance which includes more iterations than is possible with the original climate dataset.

3. WATER BALANCE OVERVIEW

The following are the key objectives of the water balance study:

- Assess the water level fluctuation and hence the statistical range of expected water levels in the TSF and RWP due to the seasonal impacts of the Pilbara regional climate;
- Determine the probable likelihood of TSF and RWP spillway discharge; and
- Determine the probable availability of water in the RWP for return to the process plant.

The geometry of the facility was divided into different sub-areas. The TSF is divided into the "Catchment Surrounds", "Dry Beach Area", "Wet Beach Area" and "Decant Pond Area". The RWP is divided into the "Catchment Surrounds" and "Pond Area".

For the purpose of this study, seepage from the TSF impoundment was assumed to be zero. The reasons for this are primarily due to the low permeability of the tailings, the impermeable liners on the embankment faces and the grout curtain under the RWP embankments.

The TSF decant pond was modelled at different stages as it moves in location and level as the TSF fills. These stages were interpolated to yield interim values used in the water balance.

A combination of runoff coefficients and the Boughton SFB yield model were used to calculate the runoff from the different geometric areas of the TSF and RWP. A runoff coefficient of 1 was utilized in areas where most or all of the rainfall is converted to runoff. In the other areas, the Boughton SFB model was utilised to introduce more parameters in the correlation process to achieve a more representative approach.

Gravity decant structures were modelled to withdraw water from the TSF decant pond into the RWP, where the stored water will be pumped back for process plant reuse. Two return water pumps (nominally Weir ME-420EXHV- 2) mounted on a floating pontoon will return the water, and the maximum combined pumping rate adopted was 1,980 m^3/hr over the LOM.

4. WATER BALANCE MODELLING

The computer program GoldSim was utilized to run the 1,000 runs/realisations of the water balance for the 1,000 years of rainfall and evaporation data. The realisations were stepped through the rainfall and evaporation dataset to utilize every ordered combination.

Besides the stochastic time history output from all the realisations in the GoldSim modelling, two representative realisations were also selected based on the probability study on the LOM total rainfall, namely:

- Realisation 786, representing the 50% LOM total rainfall; and
- Realisation 750, representing the maximum LOM total rainfall.

5. WATER BALANCE OUTCOMES

Stochastic output for the TSF decant pond and RWP behaviours are presented and discussed.

The water balance model demonstrated that the TSF can generally provide enough capacity for the decant pond storage by removing water with the gravity decant structures. TSF spillway overflow can still potentially happen when the TSF capacity is low and a major (for example, a 1:100 AEP, 72 hr) rainfall event happens. This situation could arise when tailings are at end of stage deposition height and the operating pond elevation is higher than the design nominal level adopted for contingency storage. However, the probability for this type of spillage is low, and since the overflow will be transferred and contained in the RWP, any TSF spillway discharge is considered internal and controllable.

From results for the representative realisations, it is identified that for an average LOM total rainfall case (Realisation 786), the RWP should be able to provide enough storage capacity and hence no spillway discharge to the exterior environment is expected. However, the maximum LOM total rainfall case (Realisation 750) and certain stochastic results still demonstrate potential for spillway discharges, with a maximum cumulative RWP spill volume of 11 GL and 40 days of spilling in total.

Spillage from the RWP may be regarded as an emergency discharge. The probability for RWP spillage to happen in any day among all the simulations in the model is assessed to be 1 in 2,560. However, it is decided that the assessment of the probability for these spillway discharges to happen in reality is impractical due to the combined effects from coupled events. Thus, the stochastic results should be fully understood and referred to only as possible scenarios for all events that could happen during the operation of the RWP.

The availability of return water was also investigated. From the modelling results, it can be identified that the return water pumps will be able to return water back to the process plant for most of the mine life, as there is a constant daily inflow from the tailings bleed water. The total number of days over the LOM in which the RWP pumps are operating at a very "low" rate (defined as a pumping rate of 1,980 m^3/day, or approximately 4% of the adopted pump capacity of 1,980 m^3/hr) is estimated to be in the range of 75 to 165 days, and the total availability of return water is estimated to be between 120 GL and 210 GL.

ATC Williams

TABLE OF CONTENTS

FIGURES

1 INTRODUCTION

1.1 Overview

This Report documents the collation and analysis of a representative climatic database and a Life of Mine (LOM) water balance model for the North Star Magnetite Tailings Storage Facility 2A (TSF 2A) and its associated Return Water Pond (RWP). The project is being developed by Iron Bridge Operations (IBO) in the Pilbara Region of Western Australia.

The studies described herein constitute a key component of the required submissions for regulatory approval, and this report forms Appendix E of the overall TSF 2A and RWP Design Report **[Ref. 1]**.

1.2 Design Basis

The current EPA approval conditions for the project require that no environmental spillage into the downstream environment occurs under normal operating conditions. Consequently, the spillway for the RWP should only function as an emergency spillway to protect the embankments from overtopping during flood events.

Contingency storm water storage design requirements for the facility are based on superposition of a particular storm event on a nominal operating pond volume with additional freeboard included.

These requirements are catered for in the TSF and RWP embankment design. The purpose of the water balance is to examine the potential fluctuations in stored water volume that could occur over the life of mine. One of the outcomes of this is that an informed assessment of the size of a design nominal operating pond can be determined for assessment of required design freeboard.

The key Design Basis parameters for the RWP water balance were determined using an iterative water balance approach, which included the spillway invert level (i.e. the maximum storage capacity of the RWP) and the return water pumping rate as input variables. A description of the adopted derivation methodologies and outcomes to derive these parameters are provided in Section 7 of the Design Report **[Ref. 1]**. The parameters are summarised as follows:

- RWP spillway invert level: RL 256.8 m AHD (storage capacity of approximately 5.1 GL); and
- Maximum return water pumping rate: 2 pumps combined (= 1,980 m^3/hr).

The water balance described herein incorporates current mine operating information and design parameters, and hence revises, updates and supersedes previous analysis outcomes for North Star water balance studies.

1.3 Tailings Storage and Water Management Concept

The water management concept for TSF 2A and the adjacent RWP is relatively simple, combining all tailings decant water and rainfall runoff from the TSF and RWP impoundments into one storage system, with a spillway incorporated for emergency discharge provision.

Tailings will be discharged from the eastern and south-eastern sides of the TSF 2A impoundment through a nominal 20-year LOM. Tailings will be retained by the TSF main embankment (which will be constructed in seven stages over the LOM), where the decant pond will form. The TSF 2A impoundment will collect bleed water from the tailings as well as catchment run-off and incident rainfall into the decant pond. The water ponding on the tailings surface will then be transferred through high capacity gravity decant structures into the RWP, where incidental rainfall and run-off from the RWP catchment will also accumulate. The RWP is located directly downstream of the TSF, and its containment is provided by two embankments along with the surrounding basin topography. From the RWP, water will be pumped back to the process plant for reuse via pumps mounted on a floating pontoon. A decant water return pipeline and access road will be required to link the RWP to the mine process plant.

TSF 2A has a large natural catchment of 13.5 km^2 and the total RWP catchment is 7.1 km^2.

During the first 16 months of the operation (Stage 1A), a small sacrificial bund will be constructed inside the TSF impoundment to separate it into northern and southern valley deposition areas. This will limit the size of the initial TSF catchment area to 12.1 km^2, and the tailings will only be deposited into the northern side of the TSF. For Stage 1B, the construction of the southern section of the main embankment will be finished and tailings will be deposited into both valley areas.

The final TSF main embankment height will be approximately 68 m, with emergency spillways for each stage constructed in cut in the surrounding topography. The TSF main embankment is designed as a water retaining structure and hence will be constructed of rockfill material with an impermeable bituminous geomembrane liner fitted on the upstream face. As a single water management system, the TSF spillways will potentially be used in extreme cases to transfer internal overflow discharge into the RWP.

The two RWP embankments are adjacent to each other and will be approximately 15 m and 9 m in height, with an emergency spillway constructed next to the Northern RWP embankment. The two embankments will be constructed in a similar fashion to the TSF main embankment.

1.4 Scope

In order to construct a representative climatic database, rainfall and evaporation records from the SILO patched database were used. The obtained SILO climate records span approximately 100 years. The climate records were then used as the basis for the generation of 1,000 years of statistically congruent synthetic climate data to be used in the subsequent water balance model.

A multi-realisation water balance model was developed to simulate the LOM water management for TSF 2A and RWP, and statistical results were generated and assessed.

2 CLIMATE DATA

2.1 SILO Climate Data

SILO (Scientific Information for Land Owners) is a database of Australian climate data hosted by the Science Division of the Queensland Government's Department of Environment and Science (DES) **[Ref. 2]**. It provides daily datasets for a range of climate variables across Australia with interpolated infills for missing data. The datasets are constructed from observational data obtained from the Australian Bureau of Meteorology.

For this modelling study, the climate data were obtained as SILO patched dataset for Hillside weather station (station number 4015), which is approximately 60 km southeast of the North Star TSF 2A. The data contains 100 years of daily rainfall and evaporation records from 1918 to 2018.

A summary of the average monthly rainfall and evaporation is provided in **Table 1**, and a plot for the annual rainfall sum and evaporation sum is shown in **Figure E1**. It is evident that the evaporation data set is patched from monthly averages prior to 1969.

TABLE 1
SUMMARY OF AVERAGE MONTHLY CLIMATE DATA

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Rainfall (mm)	77	81	52	21	20	20	12	6	1	3	9	40	342
Evaporation (mm)	355	289	295	247	194	155	168	205	264	345	376	388	3,281

2.2 Synthetic Climate Data

2.2.1 Purpose

The obtained climate dataset from SILO provides the historical records for the past 100 years, however it is not necessarily representative for future predictions. In reality, the "wet" and "dry" year climatic cycles will be unlikely to follow the same historical trend, hence predictive modelling (i.e. water balance models) should follow a stochastic approach with multiple realisations using various climate datasets (*stochastic is defined as having a random probability distribution or pattern that may be analysed statistically but may not be predicted precisely*).

In order to achieve a statistical analysis of the water balance results using the limited historical climate dataset, it was decided to synthesise further data. This was achieved by developing 1,000 years of climate data using the same statistical characteristics as the existing 100-year historical record. The enlarged climate dataset then enables multiple realisations for the water balance modelling, where each realisation has an equal probability of occurring and statistical results can be constituted.

Stochastic climate data allows for the water balance to quantify the inherent uncertainty in environmental systems associated with climate variability, and provide a more complete modelling picture that fully utilises the real data available.

2.2.2 Methodology

The Stochastic Climatic Library (SCL) computer program **[Ref. 3]** was used to generate synthetic daily climate data for use in the water balance. The SILO patched climate data was entered into SCL and the statistical characteristics of the set including mean, variance, skew, and long-term persistency were calculated by the program. These statistical characteristics were then used as a basis to generate a synthetic dataset with the same statistical characteristics as the original climate dataset.

The SCL program essentially produces replicates, which follow the same length as the original input dataset. Depending on the length of the synthetic dataset required, any number of replicate sets can be produced and stacked together to make up the synthetic dataset. For this study, 10 replicates of the original 100 years of climate data was produced and combined to generate 1,000 years of synthetic data. The annual total rainfall for all 1,000 years is plotted in **Figure E2**.

Each individual replicate set will have different statistical characteristics than the original input set, but the average of all the replicate sets combined will have the same statistical characteristics of the original set. For example, with 100 years of real rainfall records, 10 replicates of 100 year lengths can be generated to create 1,000 years of synthetic data.

An individual 100 year replicate may not have the same statistical characteristics as the original data, but if all of the statistical characteristics are averaged for all of the 10 replicate sets together, they will match the original data within the specified tolerances.

The actual and synthetic annual rainfall sums are plotted together along with the fitted normal distribution lines in **Figure E3**. It can be identified that both actual and synthetic datasets share similar statistical characteristics. A summary of the synthetic data is also given in **Table 2.**

TABLE 2
SYNTHETIC ANNUAL CLIMATE DATA STATISTICAL SUMMARY

Annual Statistics		
	Rainfall (mm)	Evaporation (mm)
Mean	366	3,281
Max	1,071	3,974
Min	0	2,568

2.3 Regional Climate Assessment

It is well-documented (e.g. **[Ref. 4]**) that rainfall in the Pilbara region where the North Star project is located is most influenced by tropical and monsoonal drivers, which are predominantly active in summer and autumn. The region has the second highest inter-annual variability in rainfall (second only to central Australia), and is the most cyclone-prone area along the Australian coastline. This is demonstrated by the extreme range in annual rainfall statistics tabulated in **Section 2.2** above.

Tropical cyclones cause the most extreme rainfall events and are reported to generate 25% to 34% of the total annual rainfall near the Pilbara coast and as much as 21% up to 450 km inland.

It is therefore evident that the water balance outcomes will be driven by extreme, low probability rainfall events in the climate database, due to the above factors.

3 WATER BALANCE OVERVIEW

3.1 Objectives

The water balance was used to investigate specific design parameters for TSF 2A and the RWP. The following are the key objectives of the water balance study:

(i) Assess the water level fluctuation and hence the statistical range of expected water levels/volumes in the TSF decant pond and RWP due to the seasonal impacts of the regional climate;

(ii) Determine the probable likelihood of TSF and RWP spillway discharge; and

(iii) Determine the probable availability of water in the RWP for return to the process plant.

3.2 Modelling Assumptions, Inputs and Logic

The water balance model has been prepared using a multi-stage mass balance approach:

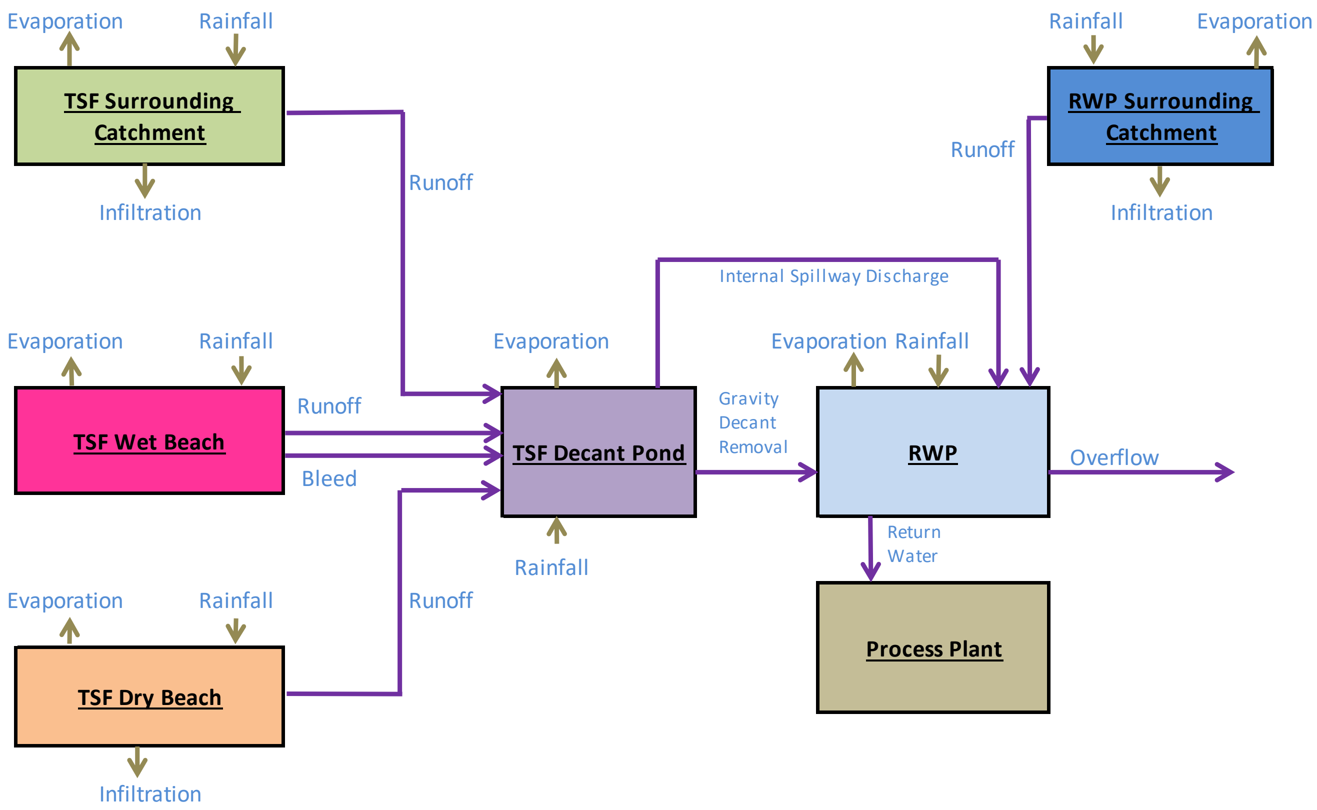
$$\Delta \text{ TSF Storage Volume} = \text{TSF Inputs} - \text{TSF Outputs}$$

$$\Delta \text{ RWP Storage Volume} = \text{RWP Inputs} - \text{RWP Outputs}$$

Inputs into the TSF include tailings bleed, surface runoff from the TSF catchment and tailings beach, and direct rainfall into the decant pond, with outputs from the TSF system being decant water transfer and evaporation.

The transferred decant water then becomes a part of the RWP inputs, along with the catchment runoff and direct rainfall into the RWP. The RWP outputs include returned process water, evaporation and overflow. Seepage through the TSF or RWP embankments was not quantified due to the reasons discussed in **Section 3.3**. A schematic flow logic diagram is shown in **Chart 1**.

CHART 1
WATER BALANCE MODEL SCHEMATIC LOGIC FLOW DIAGRAM



3.3 Impoundment Seepage

For the purpose of this study, seepage from the TSF impoundment (other than near surface infiltration contributing to baseflow) was assumed to be zero. Both TSF and RWP embankments will be lined with bituminous geomembrane on the upstream faces to form an impermeable layer, which will prevent seepage through the embankments. Also, the low permeability of the tailings material and the proposed construction of a grout curtain beneath the RWP South embankment will also confine the zone of saturation, hence flow out of the impoundment is not expected to be material in the context of this study.

3.4 Rainfall and Evaporation

As discussed in **Section 2** of this Report, rainfall and evaporation records from the SILO database spanning 1918 to 2018 were used to generate 1,000 years of synthetic climate data. These were then used to evaluate the water balance.

For evaporation calculations, a pan factor of 0.7 was adopted to account for ground and atmospheric conditions on the tailings beach and decant pond. This pan factor was adopted using the regional Western Australia Class A pan evaporation summary plot by Luke et al. **[Ref. 5]**.

3.5 Tailings Bleed

Bleed water is the difference between the water in the tailings slurry as it leaves the thickener, and the retained water in the freshly deposited tailings at the Initial Settled Density (ISD). Bleed water is considered to arrive instantaneously at the surface of the tailings, where it then flows down the beach to the decant pond. Further reductions in tailings water content due to beach evaporation have been assumed to be losses. Any water displaced to the surface as a result of long term consolidation is considered to evaporate and is taken to have been accounted for in the retained water loss on deposition.

Bleed is a function of the tailings production rate, the tailings discharge solids concentration, and the tailings sedimented density. These parameters are summarised in **Table 3.**

TABLE 3
TAILINGS PARAMETERS FOR BLEED CALCULATION

Tailings Pumping Rate	Solids Concentration	Initial Settled Density
3,800 to 4,000 dry tonnes per hour	65%	1.45 t/m^3

The resultant theoretical bleed water available is approximately 0.2 m^3/tonne of tailings.

On average, this equates to approximately 15,400 m^3/day of bleed water from tailings deposition after the initial production ramp up. However, the actual volume arriving at the decant pond is adjusted to account for the effect of evaporation along the bleed water flow path. For this water balance study, the bleed water was assumed to be flowing in a 0.1 m deep channel on the wet tailings beach surface before arriving at the decant pond.

3.6 Water Balance Model Geometry

3.6.1 Overview

The geometry of the water balance model was divided into different areas as follows:

- TSF
    - "Catchment Surrounds"
    - "Dry Beach Area"
    - "Wet Beach Area"
    - "Decant Pond Area"
- RWP
    - "Catchment Surrounds"
    - "Pond Area"

3.6.2 TSF Model Geometry

The "Catchment Surrounds" is the natural catchment area surrounding the deposited tailings. Runoff from the catchment surrounds is collected in the TSF decant pond.

The tailings beach area is divided into the dry beach, wet beach and the decant pond. The "Wet Beach Area" is a function of the total beach area and the "Decant Pond Area".

The "Wet Beach Area" is considered to be the area that has had tailings deposited recently, but has only begun to dry out slightly. The "Dry Beach Area" is considered to be the portion of the beach that has had tailings deposited earlier and is dried significantly by evaporation. For simplification and based on previous ATCW experience with similar sized TSFs in comparable climates, the "Wet Beach Area" for this study was defined to be 35% of the total beach area, with the "Dry Beach Area" accounting for the remainder of the exposed beach.

The "Decant Pond Area" is the surface accumulation of water at the low points of the tailings beach near the TSF main embankment.

With the sacrificial bund during Stage 1A operation, the TSF total catchment is limited to an area of 12.07 km^2. From Stage 1B onwards, the total catchment area for the TSF is 13.52 km^2.

The TSF total beach area (i.e. the "dry" beach + "wet" beach + "decant pond area") will range from 2.4 km^2 at the end of Stage 1A, to a final (Stage 7) area of 9.4 km^2.

3.6.3 TSF Decant Pond

The tailings beach was modelled at various stages to facilitate interpolation of the decant pond volume and area. As the TSF fills with tailings, the decant pond moves in position and elevation. In the water balance model, the decant pond volume versus elevation versus area relationships were interpolated from the modelled tailings beach stages.

The decant pond capacity will fluctuate with each TSF embankment stage raise, with the minimum capacity being at the end of each stage when the available freeboard reaches the design minimum value.

The minimum TSF embankment freeboard (to spillway invert level) is defined and detailed in Sections 8 and 9 of the Design Report **[Ref. 1]**, respectively. It is equal to a minimum operating pond depth (adopted to be 0.4 m), plus a "wet" freeboard required to store a 1:100 Annual Exceedance Probability (AEP), 72 hr run-off in the event that the decant system is not operational. The minimum TSF embankment freeboard for each stage is as follows:

- Stage 1 = 2.6 m
- Stages 2 to 7 = 2.4 m

It was assumed in the water balance model that the initial TSF decant pond volume is 0 m^3.

3.6.4 RWP Model Geometry

Similarly to the TSF, the RWP collects runoff from its surrounding natural catchment area, and the "Pond Area" is the accumulation of water surface against the RWP embankments.

The RWP at Stage 1A has a total catchment area of 8.58 km^2, due to the implementation of the sacrificial bund and a smaller TSF impoundment area. From Stage 1B onwards, the total catchment area for the RWP is 7.13 km^2.

3.7 Catchment Yield

3.7.1 Overview

To relate the rainfall to runoff in the different geometric areas of the water balance, both runoff coefficients and the Boughton SFB yield model were used. The runoff coefficients were utilised in areas where most or all of the rainfall is converted to runoff and a simpler model is appropriate.

In the other areas, the Boughton SFB model was utilised to introduce more parameters in the correlation process to achieve a more representative yield assessment.

3.7.2 Boughton Model

In an attempt to provide a rational basis for determination of runoff from the catchment surrounds the Boughton SFB method **[Ref. 6]** has been used. The Boughton SFB model was developed by correlating real rainfall records and catchment yields for catchments in Australia. It takes into account the residual moisture stored in the soil, downward migration of water through the soil strata to base flow, and upward migration of water up the soil strata to evaporation to calculate the total water available for runoff.

The Boughton SFB method is applied to rainfall records to estimate the runoff, and is appropriate for small, ungauged catchments based upon a three parameter model (S, F and B) where for the model section:

S is the surface storage capacity in mm;

F is the daily infiltration capacity in mm/day controlling percolation from the surface store to the groundwater store; and

B is a baseflow factor in the range 0 to 1, which determines the portion of the daily depletion of groundwater that appears as baseflow runoff.

For the North Star TSF water balance, the Boughton model was utilised for both the surrounding catchment areas and the "Dry Beach Area" of the tailings.

Due to the lack of gauged data, the values for the rest of the parameters (i.e. S and F) could not be calibrated against any recorded data. Hence, these parameters were calibrated against Australian Rainfall & Runoff 2016 **[Ref. 7]** published loss parameters (IL and CL) for the Pilbara and arid Australian regions, as well as from previous water balance studies in similar terrain/climate were catchment yield data is available.

The parameter B was conservatively set to zero, as there is not considered to be any significant baseflow due to the ephemeral characteristic of the local hydrology. A summary of the adopted Boughton model parameters is shown in **Table 4**.

TABLE 4
BOUGHTON MODEL PARAMETERS SUMMARY

Area	Boughton Parameters		
	S (mm)	F (mm/day)	B
Surrounding Catchment Area	23	2	0
Dry Beach Area	20	1	0

3.7.3 Catchment Yields

The catchment yield is the percentage of the total precipitation that is considered as actual surface runoff after adjustment by the Boughton model, which is similar to the general runoff coefficients. **Figure E4** and **Figure E5** show statistical outcomes from the water balance model for the surrounding catchments and tailings dry beach runoff yields. In this water balance study, it was identified that the average long term yields are approximately 0.37 and 0.41 respectively.

It is noted that the catchment yields calculated empirically by applying initial loss and continuing loss parameters for the Pilbara **[Ref. 7]** to IFD rainfall depths are significantly lower than those derived above. However, when the yields were further interrogated against ATCW's previous experience with similar catchment geology and geomorphology. The derived values were considered as appropriate and conservative (in the context of water management) for this water balance study.

3.7.4 Runoff Coefficients

For the remaining catchment areas, losses due to infiltration are not expected. Hence a runoff coefficient of 1.0 (i.e. 100% catchment yield) was utilised for the "Wet Beach Area", "Decant Pond Area" and RWP "Pond Area".

3.8 Water Management

3.8.1 Decant Structures

As discussed in **Section 1.3**, decant and runoff water will be stored in the TSF and RWP as one single system linked by two decant structures. The TSF main embankment decant structures drain water via gravity into outfall pipes, which direct water underneath the TSF embankment to the RWP. The decant structures collect decant water from the TSF pond using inverted box culverts with segmented stoplogs on top. The water is then transferred through the pipeline into the RWP.

An operational TSF decant pond depth is designed to be 400 mm to allow for tailings solids to settle out onto the submerged beach before being decanted. The flow rate is controlled by the water head above the segmented stoplogs as well as the flow capacity of the decant structure and outlet pipes.

The inlet box culverts have a design dimension of 900 mm by 900 mm, and the diameter of the outfall pipelines is 750 mm.

The two decant structures will be constructed and raised sequentially with each TSF stage, and this is also reflected in the water balance model. During Stage 1A operation, the TSF catchment will be limited to a smaller area and only the northern section of the TSF embankment will be constructed. Hence, only one decant structure will be constructed for decant water transfer during Stage 1A. The second decant structure will be constructed along with the southern TSF embankment and will commence operation from Stage 1B.

The water balance modelling logic is based on the TSF Decant Pond levels and volumes relative to those of the adjacent RWP storage. It is hence apparent that the decant transfer capacity will increase with each TSF embankment stage raise, due to the increase in head.

Under normal operating conditions (i.e. 400 mm deep decant pond), the combined discharge capacity of the TSF decant return system will be mainly controlled by the weir flow at the segmented stoplogs on top of the box culverts. The discharge rate under this condition is approximately 10,400 m^3/day.

This capacity is considered sufficient to manage normal operating inflows (i.e. bleed water, refer to **Section 3.5**), when taking into account evaporation losses from the wet beach and pond.

However, during rainfall events it is necessary to significantly increase the discharge capacity to remove ponded water from the tailings. The decant system has been designed such that what when the TSF decant pond depth extends beyond 800 mm, the decant outlet pipelines will be completely filled with decant water, hence the discharge rate will be dominated by the head difference between TSF decant pond and the RWP. In order to illustrate the capacity under flood clearance conditions, the nominal decant structure discharge rates (for a 800 mm decant pond, and the RWP at minimum operating level) are summarised for the beginning of each stage in **Table 5**.

TABLE 5
NOMINAL DECANT STRUCTURE DISCHARGE RATES

Stages (Beginning)	Decant Structure Discharge Rates (m^3/day)
1A	250,000 (1 Decant Structure)
1B	680,000
2	720,000
3	820,000
4	880,000
5	940,000
6	980,000
7	1,000,000

3.8.2 Return Water Pumping

The decant water and the runoff collected at the RWP will be returned via a pontoon-mounted pumping station installed within the RWP, and will be re-utilised within the process plant and for various other mine activities.

As described in **Section 1.2**, two return water pumps with a total pumping rate of 1,980 m^3/hr, corresponding to 47.52 ML/day, have been adopted in this study.

In the water balance model, a nominal minimum pond volume corresponding to a water depth of 3.0 m against the RWP embankment was set for the return water pump, in order to avoid the pump running dry and being damaged.

3.8.3 Internal Spillway Discharge

Given that the TSF 2A and the RWP are a linked containment system, it is a design condition that during large flood events at the end of each stage, internal discharge of the decant pond water from the TSF spillway into the RWP is acceptable, since the water is still contained within the system.

From the water balance model outcomes, it was identified that the internal spillway discharge could possibly happen at the end of Stage 1A (16 months) when the TSF decant pond capacity is at a very low value (approximately 0.7 GL) and only one decant structure is operational. Detailed results are discussed in **Section 5.2**.

4 WATER BALANCE MODELLING

4.1 Model Realisations

The computer program GoldSim **[Ref. 6]** was used to develop and run the daily water balance model. 1,000 realisations were run to stochastically analyse all possible climatic scenarios. Each realisation represents a 20-year LOM with a unique LOM climate dataset.

The realisations were stepped through the 1,000 years of synthetic rainfall and evaporation data. For example, the climate data for Realisation 1 starts at Year 1 and ends at Year 20; the next realisation in the model then progresses into the following 20 years of climate data starting from Year 2.

When the data stepping reaches the end of the 1,000 year dataset, it then loops back to the beginning of the set. As such, the final realisation (Realisation 1,000) utilises the climate data from Year 1,000 to Year 19, which prevents any repetition of climate data for a 20-year LOM duration.

4.2 Stochastic Output and Representative Realisations

GoldSim outputs values for various elements within the water balance model as both individual realisation results and as statistical results. The individual realisation results comprise daily output following the unique climate input for that realisation. The statistical results depict the percentile values based on a certain day across all realisations, and hence represent a collection of values from all realisations for the model duration rather than representing any specific realisation.

For example, the "maximum" result is merely demonstrating all the maximum values achieved on each simulated day from all the realisations, and does not reflect results from any specific realisation. Therefore, certain realisations would have results partially aligning with the "maximum" result, but they would never exceed the "maximum".

Two representative individual realisations were selected among the stochastic outputs for the purpose of investigating possible scenarios during the operation of the North Star TSF. The selection was done by statistically analysing the LOM total rainfall (20-year sum) for all 1,000 realisations. A normal distribution plot of the LOM total rainfall for all realisations is shown in **Figure E6**, and the representative realisations were selected accordingly.

These individual realisations include:

- Realisation 786 - representing the average (50 percentile) LOM total rainfall case (7,259 mm in 20 years); and
- Realisation 750 - representing the maximum LOM total rainfall case (11,779 mm in 20 years).

The average realisation (Realisation 786) was also inspected to ensure that no isolated extreme rainfall events were present in the dataset.

A third realisation (Realisation 949) was also selected for the purpose of assessing the potential for TSF spillway flow during Stage 1A filling. This is described in **Section 5.2**.

5 WATER BALANCE OUTCOMES

5.1 TSF Decant Pond Volume Results

Presented in **Table 6** is a statistical analysis of the average TSF decant pond volume outcomes based on the water balance stochastic results.

TABLE 6
STATISTICS OF TSF DECANT POND VOLUMES

	Decant Pond Volume (Average) (ML)
Mean	146
25 Percentile	135
50 Percentile (Median)	136
75 Percentile	138
95 Percentile	165
99 Percentile	393
Maximum	1,315

A selection of stochastic output together with the results for the representative individual realisations (as defined in **Section 4.2**) for the modelled TSF decant pond behaviour are presented in **Figure E7**.

When comparing the data in **Table 6** with the pond volume versus mine year in **Figure E7**, it is important to note that the **Table 6** statistics are derived from the daily volumes output from the model for all 1,000 LOM realisations. For example, the mean TSF decant pond volume of 146 ML is the average of (1,000 x 20 x 365) No. daily volume outcomes.

In order to provide context to the **Figure E7** volume data with respect to available capacity and spill risk, the realisation 750 (Maximum LOM total rainfall scenario) and the "maximum" stochastic data are plotted with the actual TSF decant pond capacity in **Figure E8**. This indicates that the spill risk is limited to short periods at the end of each stage, particularly Stage 1A.

Stochastic outcomes are also presented in terms of decant pond elevation versus mine year in **Figure E9**. For the purposes of demonstrating how these outcomes relate to the filling of the TSF over the LOM, the modelled tailings surface level at the TSF embankment decants has been included, together with the ultimately adopted TSF spillway invert levels (refer to **Ref. 1**).

5.2 TSF Capacity and Internal Spillway Discharge

Generally, it can be concluded from the water balance model that the decant water in the TSF can be transferred effectively via the gravity decant structure, hence the TSF can provide enough decant pond capacity.

However, as discussed in **Section 1.3**, during Stage 1A of the TSF operation, the tailings will only be deposited in the northern section of the TSF, where decant pond capacity will be limited by the available width of beach . **Figure E10** plots the cumulative spillway discharge over the LOM from the TSF and indicates that the TSF spillway will only discharge towards the end of Stage 1A in the "99 percentile" and "Maximum" condition, with a maximum spill volume of approximately 0.9 GL (this is also illustrated in **Figure E8).**

Upon investigating the spillway discharge results, the maximum TSF spillway discharge happens in Realisation 949 at around 15 months. **Figure E11** shows the decant pond volume, spill discharge volume and daily rainfall for Realisation 949. The existing decant pond volume prior to the spillage (based on an interrogation of the data) was approximately 1.3 GL, and a subsequent rainfall of 180 mm in one day expands the decant pond beyond its capacity at this stage of TSF development (1.8 GL), resulting in the spill.

This demonstrates that TSF spillway discharges are effectively triggered by coupled events, such as a major storm with a large antecedent pond volume. The probability of such events cannot be properly assessed, but is generally considered to be extremely low.

As outlined in **Section 3.8.3**, it is an acceptable design condition that a TSF spillway discharge may occur in extreme flood cases, particularly given that the spillage will be transferred and contained in the RWP, hence this type of spillage is considered as internal and controllable.

5.3 TSF Decant System Results

Presented in **Table 7** is a statistical analysis of the average decant system discharge outcomes based on the water balance stochastic results.

A selection of stochastic outputs for the modelled TSF decant system discharge behaviour are presented in **Figure E12**.

TABLE 7
STATISTICS OF TSF DECANT SYSTEM DISCHARGES

	Decant System Discharge (Average) (m^3/day)
Mean	18,900
25 Percentile	11,100
50 Percentile (Median)	12,000
75 Percentile	13,100
95 Percentile	30,100
99 Percentile	230,100
Maximum	664,100

When comparing the data in **Table 7** with the decant system discharge versus mine year in **Figure E12**, it is important to note that the **Table 7** statistics are derived from the daily discharges output from the model for all 1,000 LOM realisations. For example, the mean TSF discharge of 18,900 m^3/day is the average of (1,000 x 20 x 365) No. daily discharge outcomes.

5.4 RWP Volume Results

Presented in **Table 8** is a statistical analysis of the average RWP volume outcomes based on the water balance stochastic results.

A selection of stochastic output together with the results for the representative individual realisations for the modelled RWP behaviour are presented in **Figure E13**.

TABLE 8
STATISTICS OF RWP VOLUMES

	RWP Volume (Average) (ML)
Mean	305
25 Percentile	39
50 Percentile (Median)	40
75 Percentile	285
95 Percentile	1,530
99 Percentile	2,628
Maximum	4,225

Stochastic outcomes are also presented in terms of RWP pond elevation versus mine year in **Figure E14**. The RWP spillway invert level is also included to demonstrate the relationship between the pond levels and the spill potential. It can be seen from the stochastic data in **Figures E13** and **E14** that the "maximum" percentile graph comprises the peak outcomes of Realisation 750 (the maximum LOM rainfall case) on a number of occasions.

5.5 RWP Spillage and Spill Risk

5.5.1 Spillway Discharge Results

From **Figure E13** and **Figure E14**, it is identified that on an average LOM total rainfall case (Realisation 786), there should be no spillway discharge from the RWP as the pond never reaches the maximum capacity (spillway invert level). With the maximum LOM total rainfall case (Realisation 750), it is identified that the RWP water level exceeds the spillway invert level in a number of situations, resulting in emergency spillway discharges.

To demonstrate the spill frequency, the cumulative number of days for the RWP to have spillway discharge throughout LOM is presented in **Figure E15**. It is identified that the maximum number of spilling days is small compared to the 20 year (7,300 days) mine life, with a maximum number of 40 days. For the maximum LOM total rainfall case (Realisation 750), the cumulative number of spilling days is 22, which equates to a 1 in 330 chance of spilling over the 20 year LOM.

The stochastic results for cumulative spill volume from the RWP are plotted in **Figure E16**, which implies a maximum cumulative RWP spill volume of 11 GL. However, as discussed in **Section 4.2**, the stochastic results only represent percentile values among results from all realisations, and cannot be associated with the likelihood of a particular spillway discharge. Also, similar to the TSF spillway discharge, the RWP spillage can be triggered by a combined effect from coupled events.

It is important to discuss the apparent increased risk of spillage during Stage 1A, based on the "maximum" stochastic plots in **Figures E15** and **E16**. The TSF impoundment area is reduced during Stage 1A, meaning that there is less flood inflow attenuation within the TSF. This results in higher peak RWP inflows due to an extreme, monsoonal rainfall event. Such events are statistical outliers in the climate database, however when one occurs early in a particular water balance realisation, the "maximum" stochastic outcomes indicate that RWP emergency spillway discharges are possible. In many respects a better representation of the apparent spill susceptibility of the Stage 1A configuration is achieved by looking at the 95%ile, and Realisations 750 and 786 in **Figures E15** and **E16**.

In order to demonstrate, **Figure E17** plots the RWP volume, cumulative spill volume and daily rainfall for the maximum LOM total rainfall case (Realisation 750). It is identified that the majority of the RWP spillway discharges happen when:

- The antecedent RWP volume is large (over 2 GL); and
- A cluster of major rainfall events (over 100 mm/day) or an extreme rainfall event (350 mm/day) occurs.

In **Figure E17** at around Year 8 when there is a cluster of high rainfall events, the RWP volume accumulates quickly and results in a spill. Also, at around Year 18 when the antecedent RWP volume is at approximately 2 GL, an extreme rainfall event of 350 mm (the highest in the 1,000 year synthetic daily rainfall database and likely to be rarer than 1 in 800 year ARI) causes a large spillway discharge from the RWP.

These spillway discharges result from coupled events comprising high antecedent pond volume and extreme rainfall, for which the joint-probability is not able to be directly assessed.

Therefore, these stochastic results should be put in context, and referred to only as possible scenarios for all events that could happen during the operation of the RWP.

5.5.2 Estimated Spill Probability

From the water balance modelling results, a probability for RWP spillage can be assessed based on the total number of days simulated throughout the whole model, i.e. 1,000 LOM realisations × 20 years × 365.25 days = 7,305,000 days.

From the modelling results for all 1,000 realisations, a total number of 2,854 days is observed to have a RWP spillway discharge, with a maximum of 22 for any individual realisation. Based on this, the chance for any simulated day in this water balance model to have a RWP spillway discharge is assessed to be 1 in 2,560.

5.6 Assessment of RWP Capacity

The daily RWP volume data for the two representative individual realisations (as defined in **Section 4.2**) were ordered, ranked, and assigned a probability. **Figure E18** shows the volume probability distribution plot for the Realisation 786 (Average LOM Total Rainfall) and Realisation 750 (Maximum LOM Total Rainfall) scenarios, together with resultant return periods.

Presented in **Table 9** are RWP volumes corresponding to various return periods for Realisation 786 (Average) and 750 (Maximum) LOM total rainfall scenarios.

TABLE 9
RWP VOLUME - PROBABILTY OF OCCURANCE (ONCE IN THE LOM)

Probability	Maximum Pond Volume (ML)	
	Realisation 786 (Average LOM Total Rainfall Scenario)	Realisation 750 (Maximum LOM Total Rainfall Scenario)
Mean	260	886
1 in 10	875	2,967
1 in 20	1,675	3,927
1 in 100	2,740	5,100 (Max. Capacity)

It is important to note the probabilities in **Table 9** relate to the chance of the given volume being reached or exceeded once in the 20 year LOM for that rainfall scenario. The variation in the data for the two rainfall scenarios provides a clear indication of the impact of tropical and monsoonal events within the rainfall database, as discussed in **Section 2.3**.

5.7 Availability of Return Water

As evident from **Section 2**, the evaporation at the North Star site is significantly higher than the rainfall, which will impact the availability of pond water to be returned to the process plant. From the water balance modelling results, both the TSF decant pond and the RWP water level can be expected to exhibit large fluctuations over the LOM, and the volume of water in the RWP may periodically be at the minimum pump operating level. In general, both the decant pond and the RWP will operate under "normal" operating conditions due to the daily inflow of tailings bleed water.

Figure E19 shows the statistical analysis of the number of days in the LOM in which the RWP pumps are operating at a very "low" rate (defined in **Figure E19** as a pumping rate of 1,980 m^3/day, or approximately 4% of the adopted pump capacity of 1,980 m^3/hr). Results for

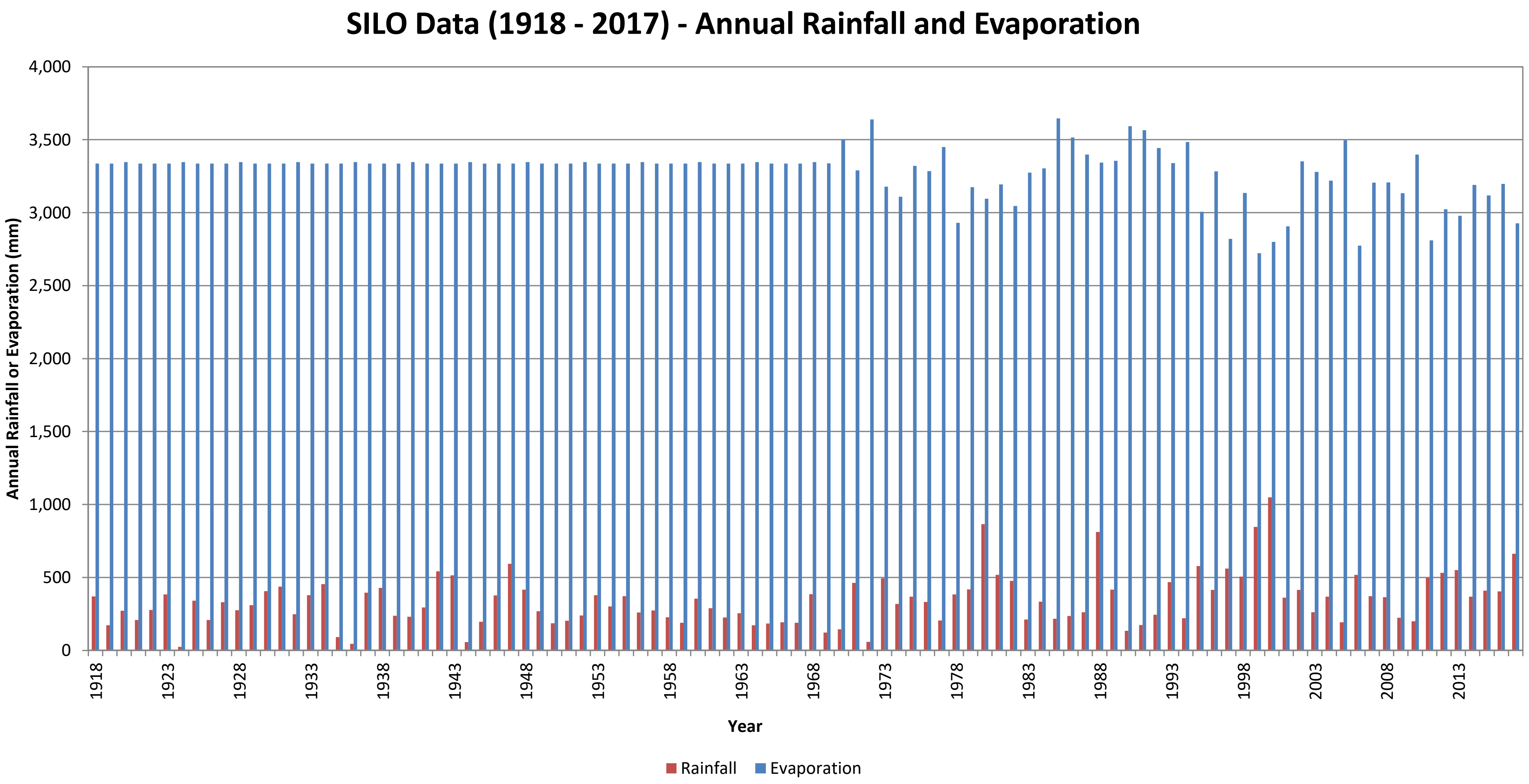
Realisation 786 (Average LOM Total Rainfall) and Realisation 750 (Maximum LOM Total Rainfall) are also included to demonstrate representative scenarios. It can be identified that on average, the number of days with such significantly reduced water return over the LOM can be expected to be in the range of 75 to 165 days. This equates to 1% to 2% of the total mine life.

Figure E20 shows the statistical analysis of the cumulative volume of water returned back to the process plant, along with the representative realisations. These results demonstrate that the range of the total availability of return water through LOM for the process plant is expected to be 120 GL to 210 GL, which equates to an average of 6 GL/yr to 10.5 GL/yr of return water.

6 CLOSURE

Your attention is drawn to the "Conditions of Report" which appear after the document and history page of this report.

FIGURES



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NORTH STAR MAGNETITE TSF STAGE 2

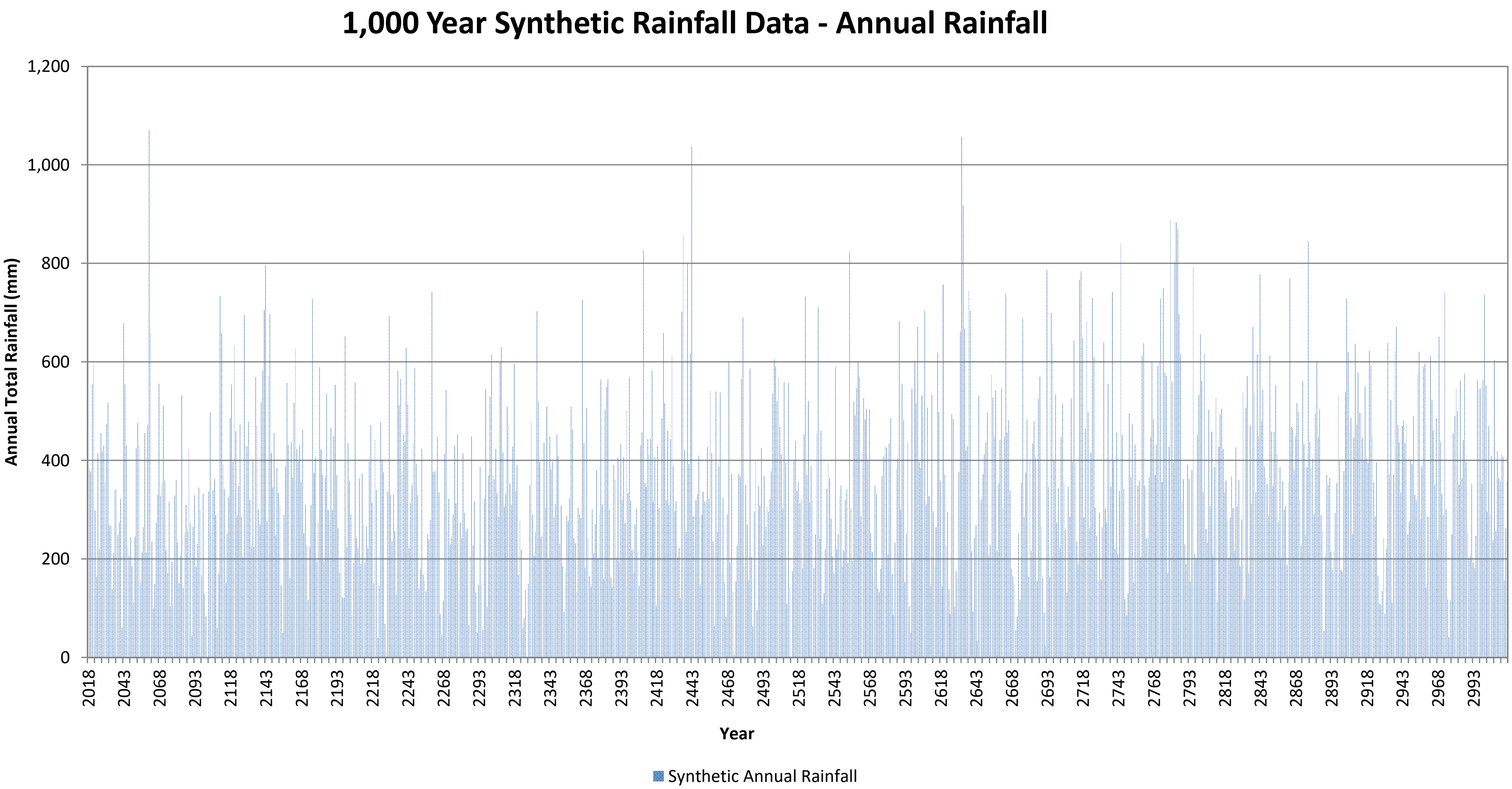
RAINFALL ANALYSIS AND WATER BALANCE REPORT

SILO Actual Climate Data - Annual Rainfall and Evaporation Totals

Date: 29/10/2018 **Job No:** 114185.14

FIGURE E1

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\SILO - Hillside Weather Station - 100 Years.xlsx Plot



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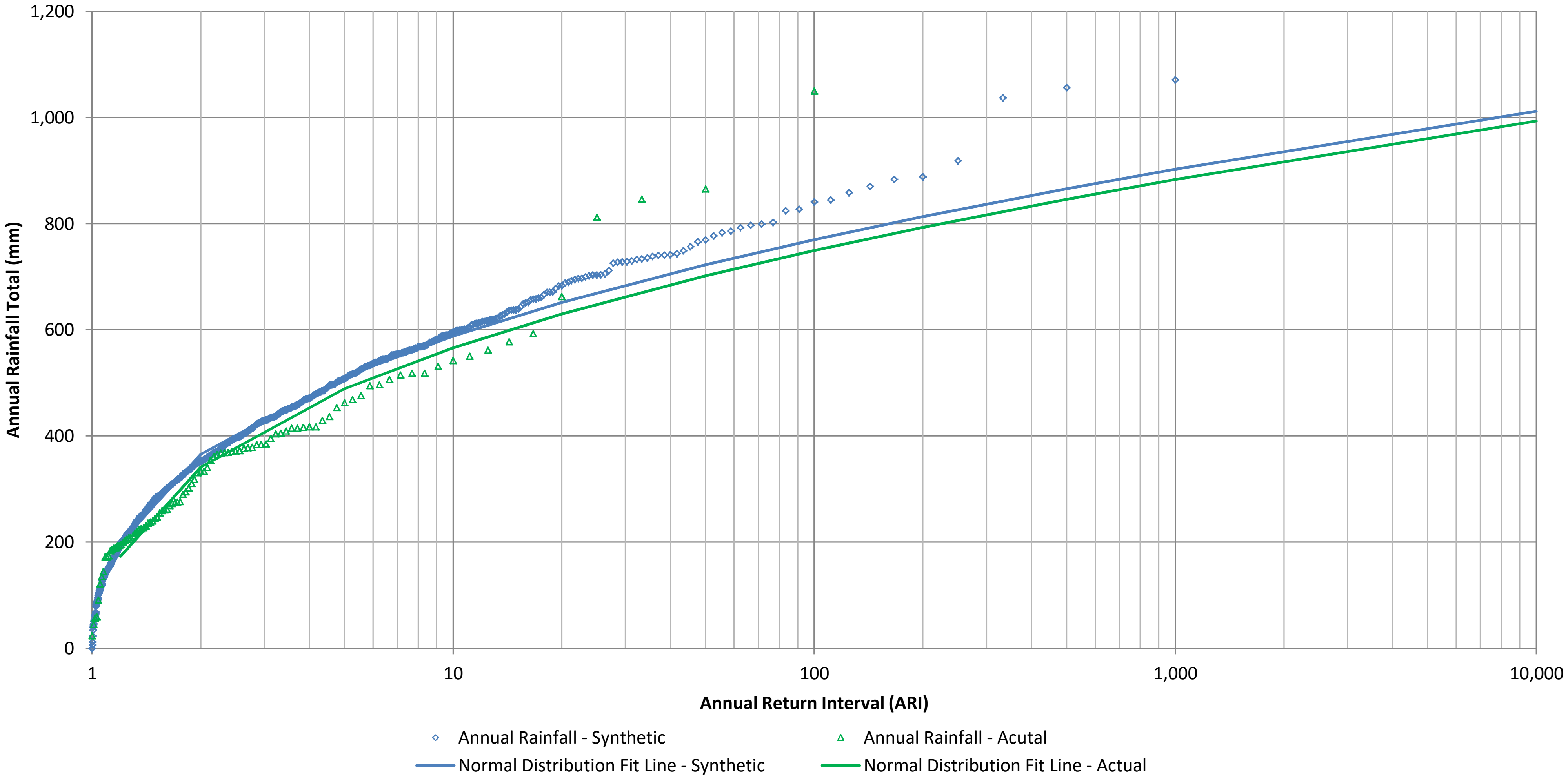
IRON BRIDGE OPERATIONS **NORTH STAR MAGNETITE TSF STAGE 2** **RAINFALL ANALYSIS AND WATER BALANCE REPORT**	
Synthetic Data - Annual Total Rainfall	
Date: 29/10/2018 **Job No:** 114185.14	**FIGURE E2**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Synthetic Climate Data for GoldSim.xlsx Annual Rain Plot

Normal Distribution for Actual and Synthetic Rainfall Data



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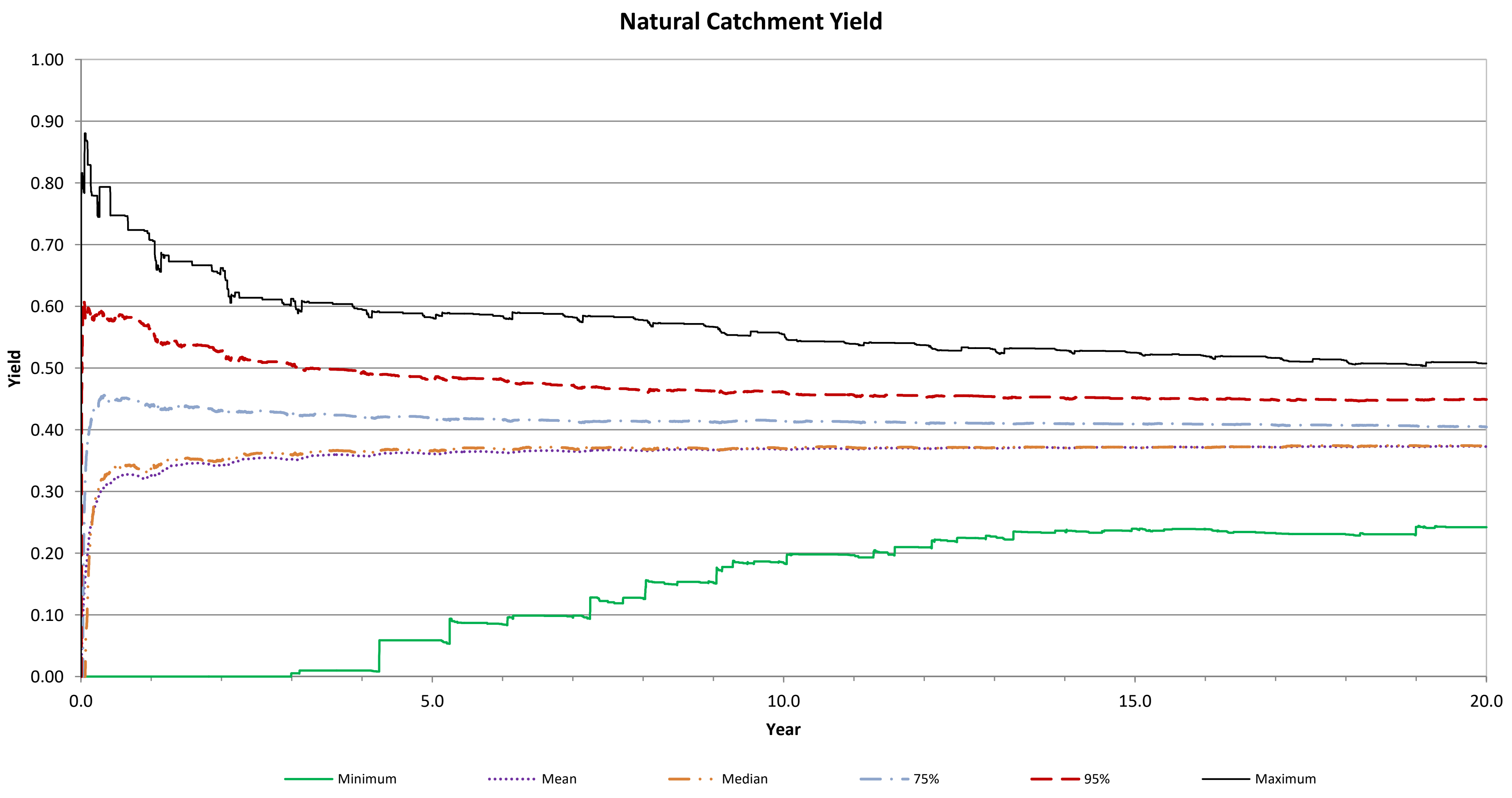
NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

Actual and Synthetic Rainfall Annual Totals - Normal Distribution Graph

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E3**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Synthetic Climate Data for GoldSim.xlsx Normal Distribution



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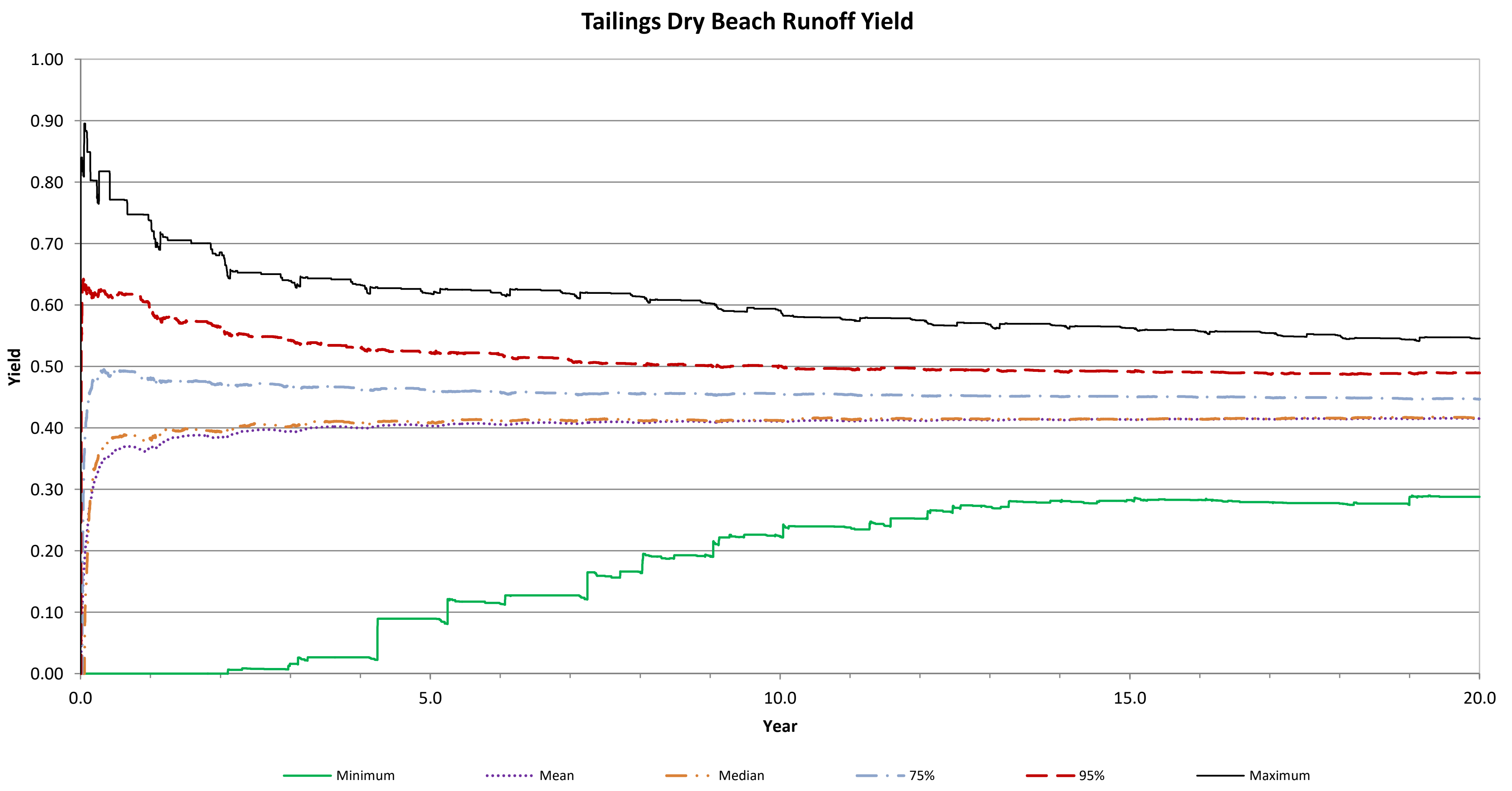
NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

Stochastic Results - Natural Catchment Yield

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E4**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Catchment Yield Plot.xlsx Nat_Catch Yield



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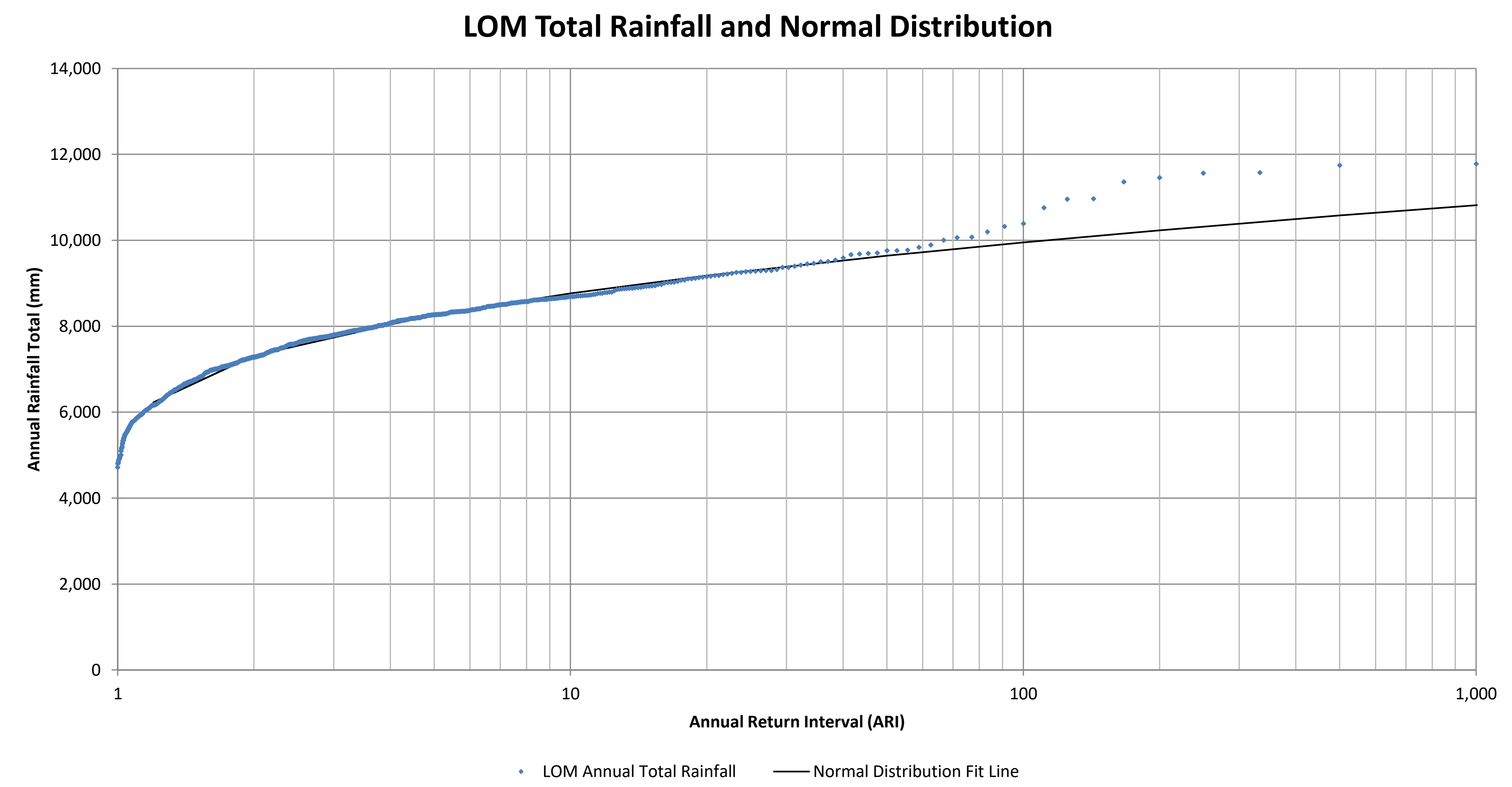
NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

Tailings Dry Beach Runoff Yield

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E5**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Catchment Yield Plot.xlsx Dry Beach Yield



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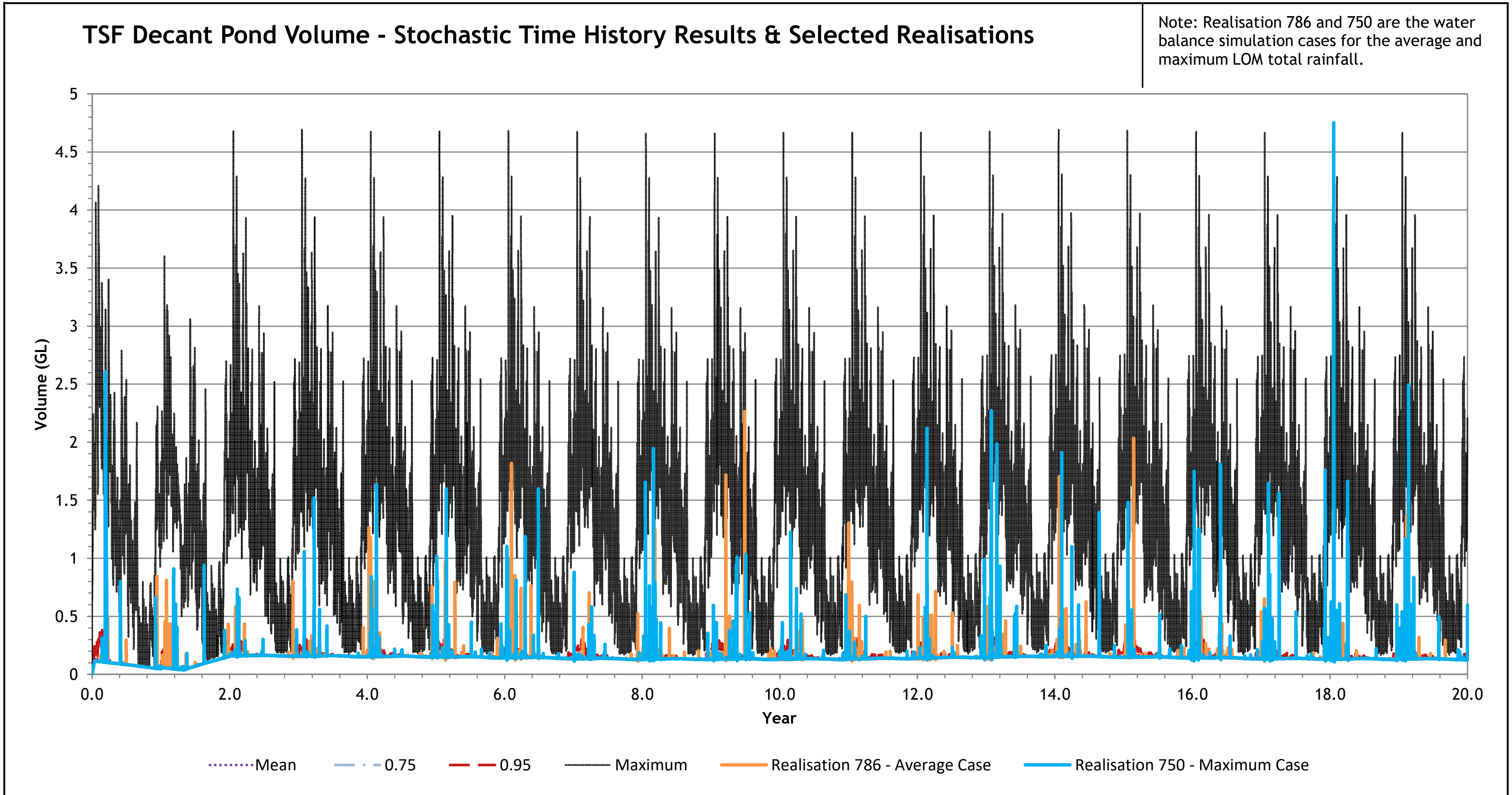
IRON BRIDGE OPERATIONS NORTH STAR MAGNETITE TSF STAGE 2 RAINFALL ANALYSIS AND WATER BALANCE REPORT				
LOM Annual Total Rainfall - Normal Distribution Graph				
Date: 29/10/2018	**Job No:** 114185.14			**FIGURE E6**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\20-year Total Rainfall Distribution.xlsx Normal Distribution



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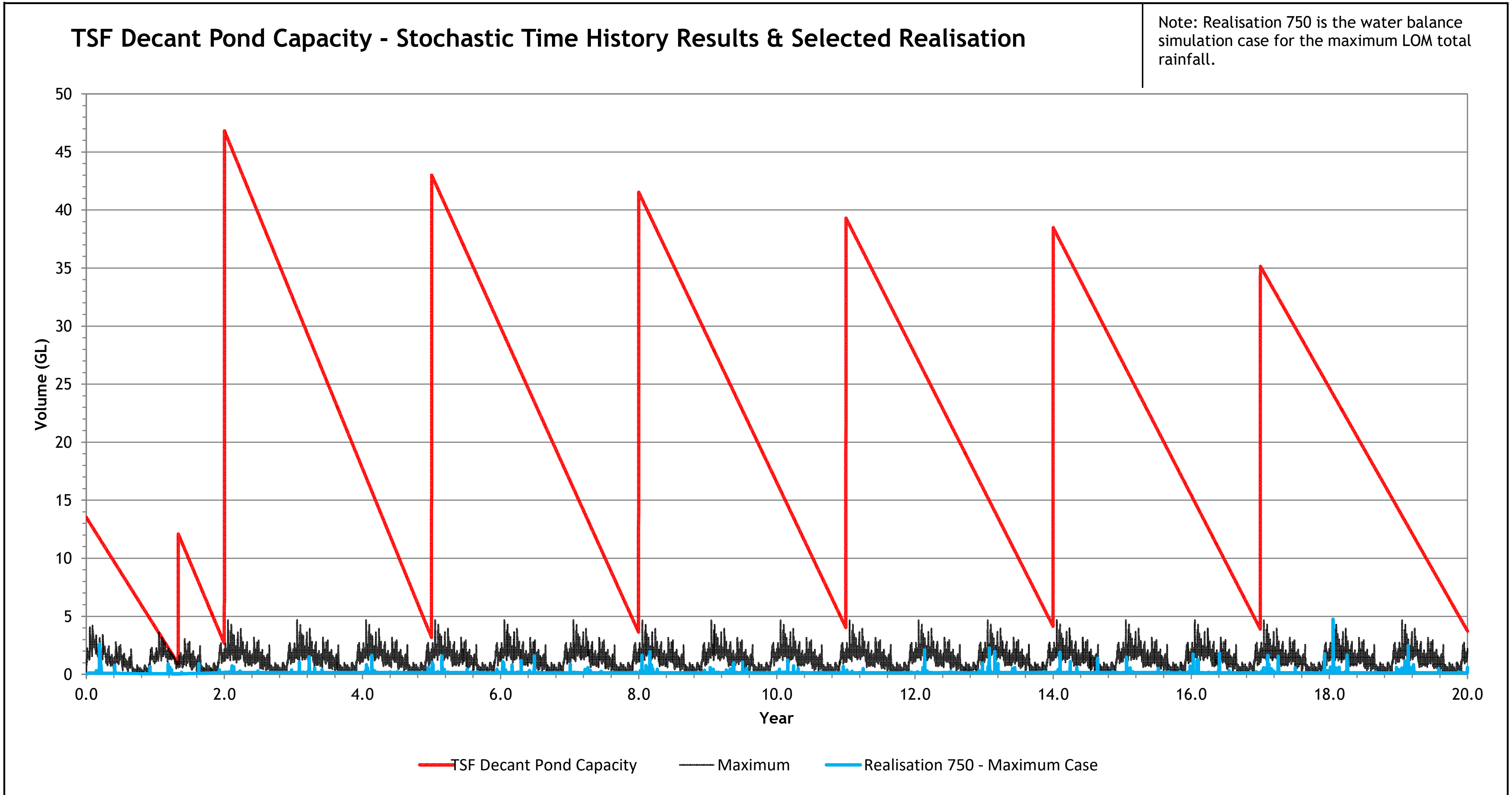
NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

TSF Decant Pond Volumes - Stochastic Results and Selected Realisations

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E7**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx Decant Pond Volume



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NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

TSF Decant Pond Capacity - Stochastic Results and Selected Realisation

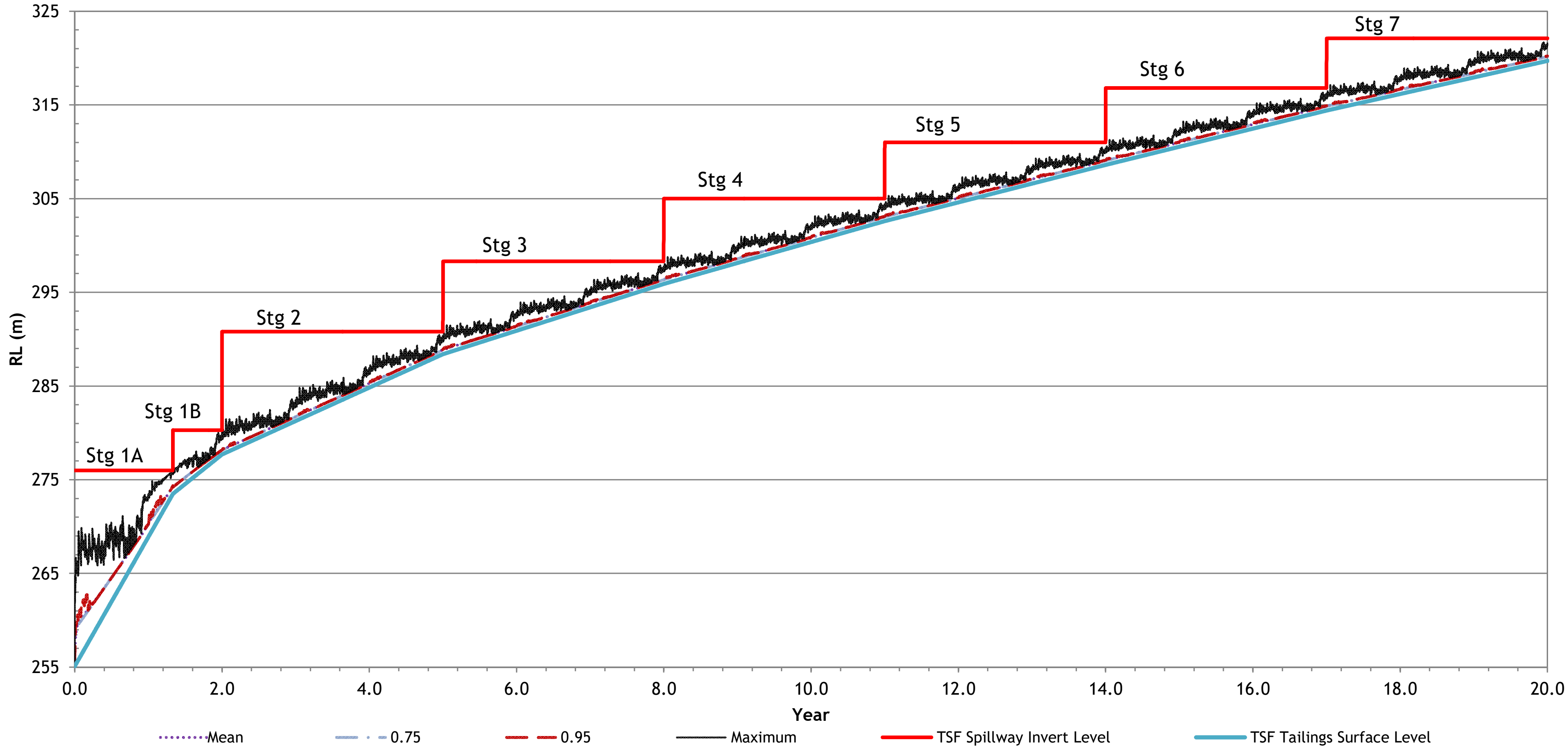
Date: 29/10/2018 **Job No:** 114185.14

FIGURE E8

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx Decant Pond Capacity

TSF Decant Pond RL - Stochastic Time History Results & Selected Realisations

Note: Realisation 786 and 750 are the water balance simulation cases for the average and maximum LOM total rainfall.



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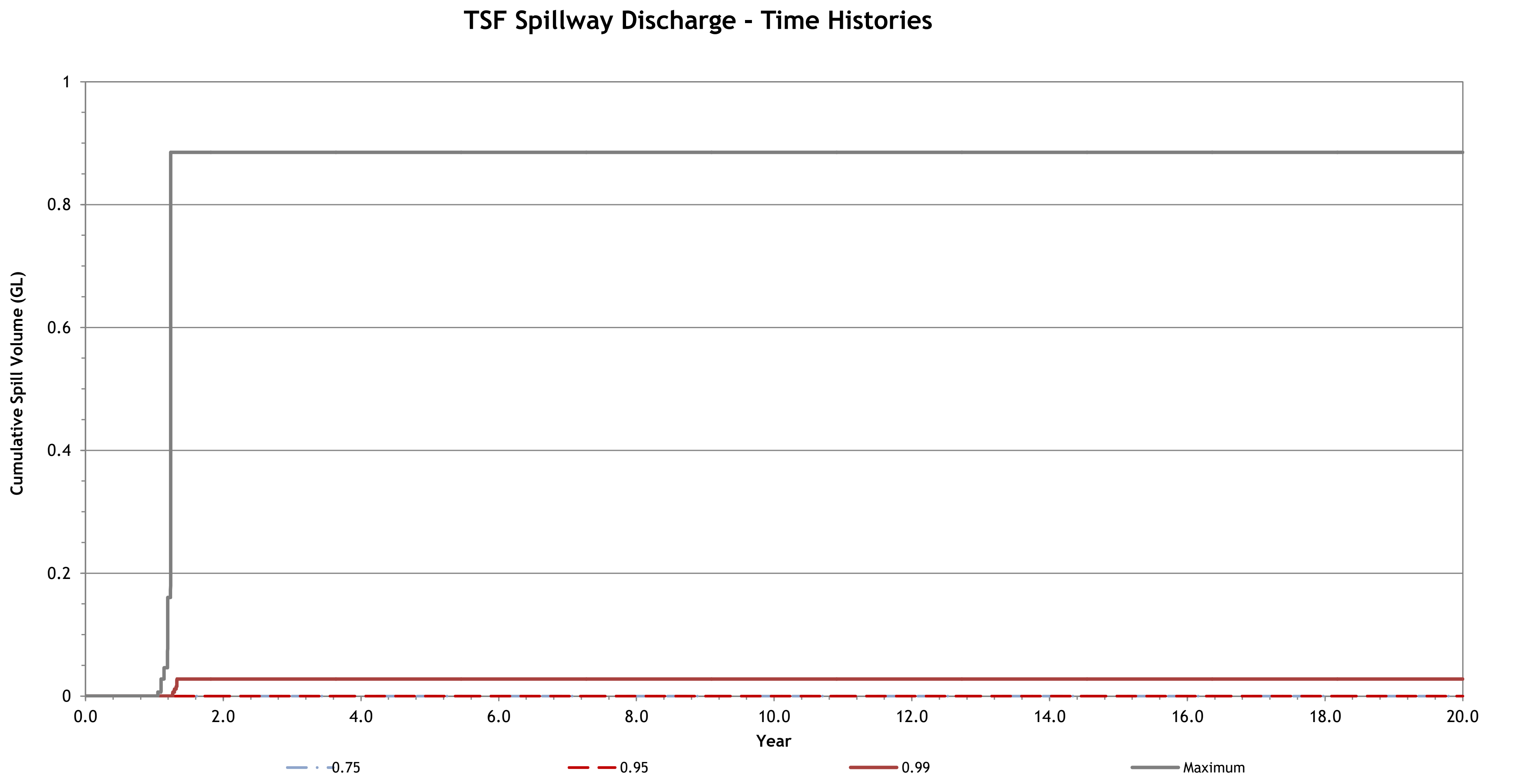
NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

TSF Decant Pond RL - Stochastic Results and Selected Realisations

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E9**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx Decant Pond RL



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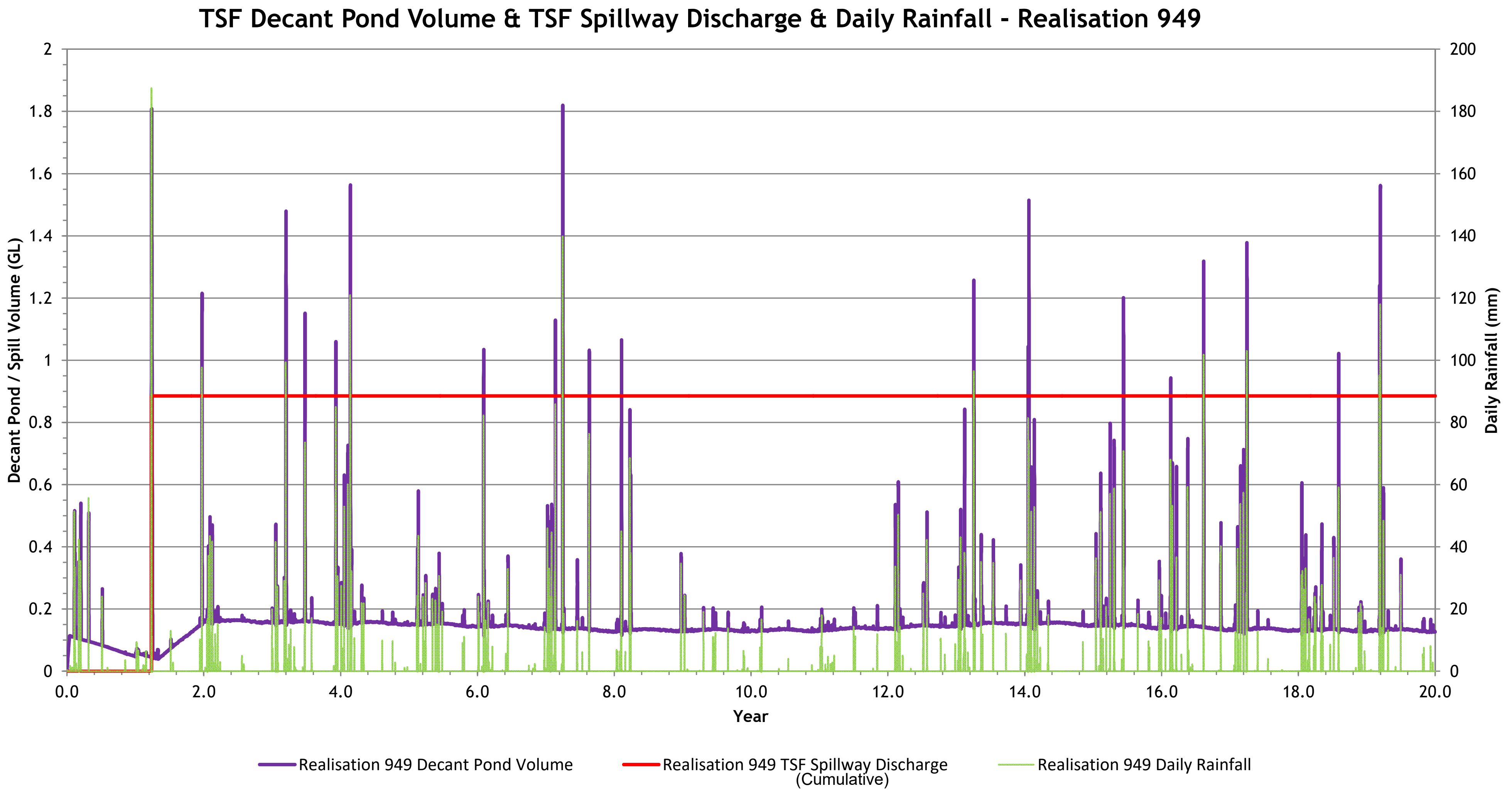
NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

TSF Cumulative Spillway Discharge - Volume Stochastic Results

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E10**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx Internal Spill



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NORTH STAR MAGNETITE TSF STAGE 2

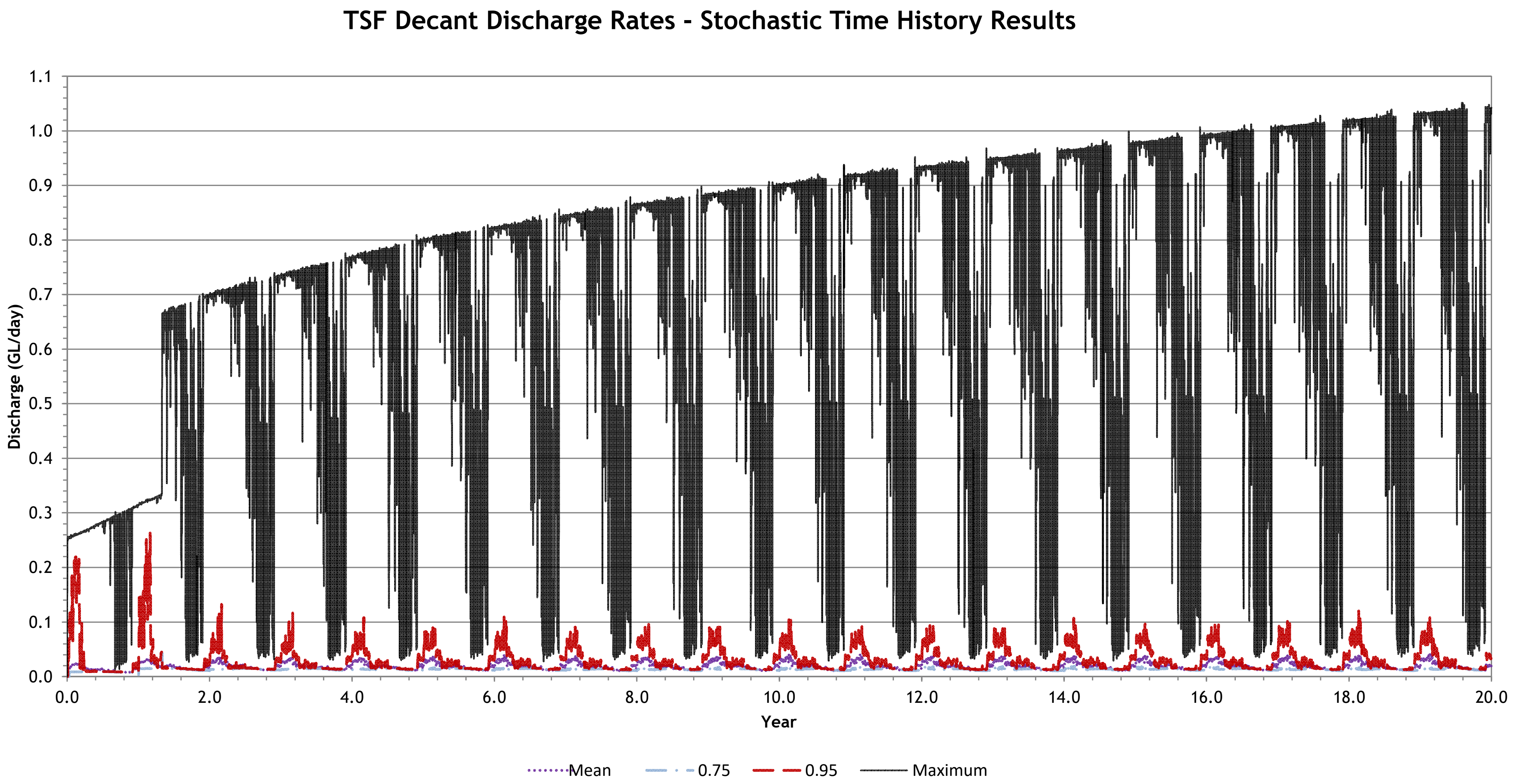
RAINFALL ANALYSIS AND WATER BALANCE REPORT

TSF Decant Pond Volumes, Cumulative Spill and Rainfall - Realisation 949

Date: 29/10/2018 **Job No:** 114185.14

FIGURE E11

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx Realisation 949



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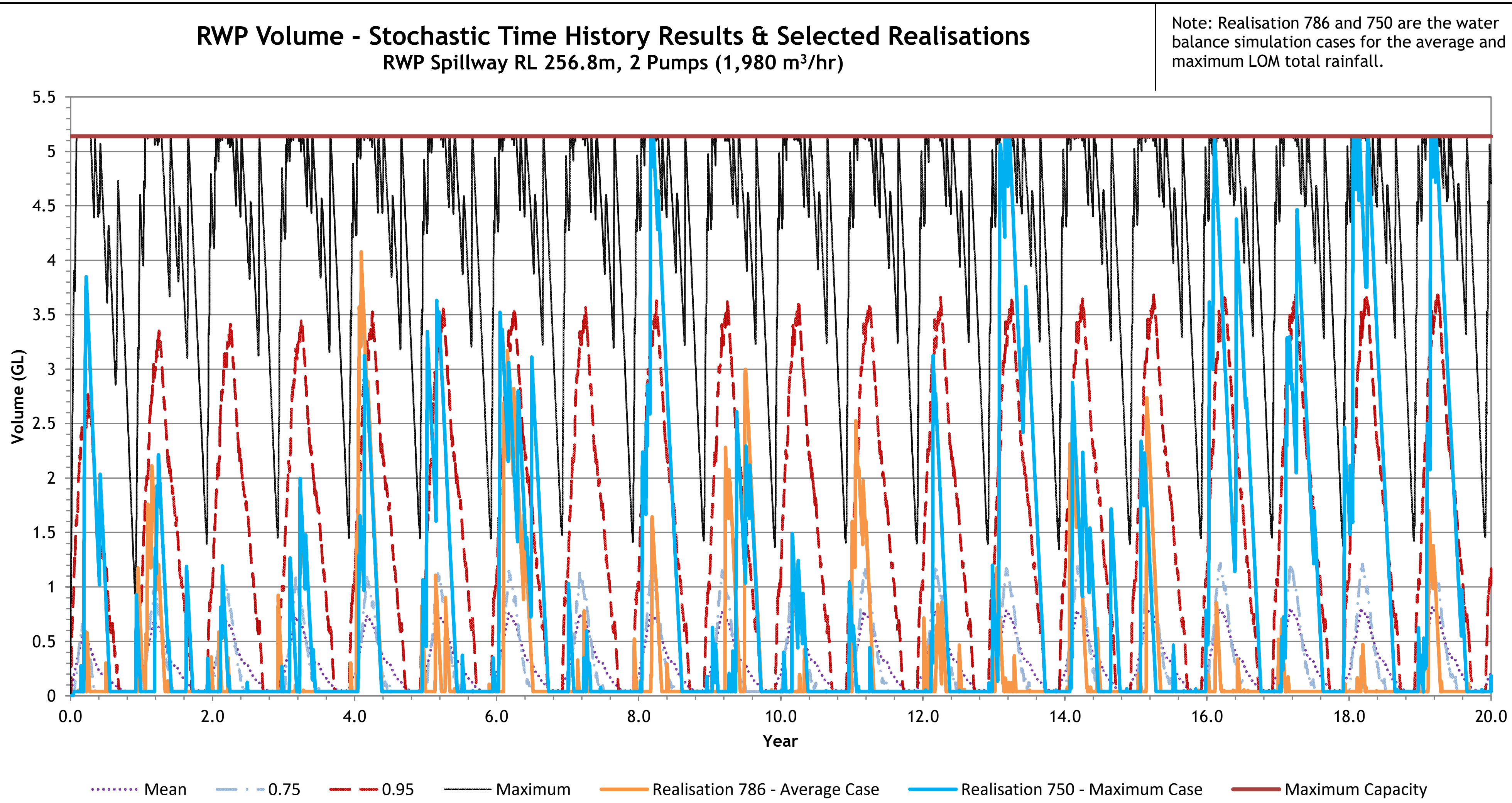
NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

TSF Decant Discharge Rates - Stochastic Results

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E12**

Note: Realisation 786 and 750 are the water balance simulation cases for the average and maximum LOM total rainfall.



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NORTH STAR MAGNETITE TSF STAGE 2

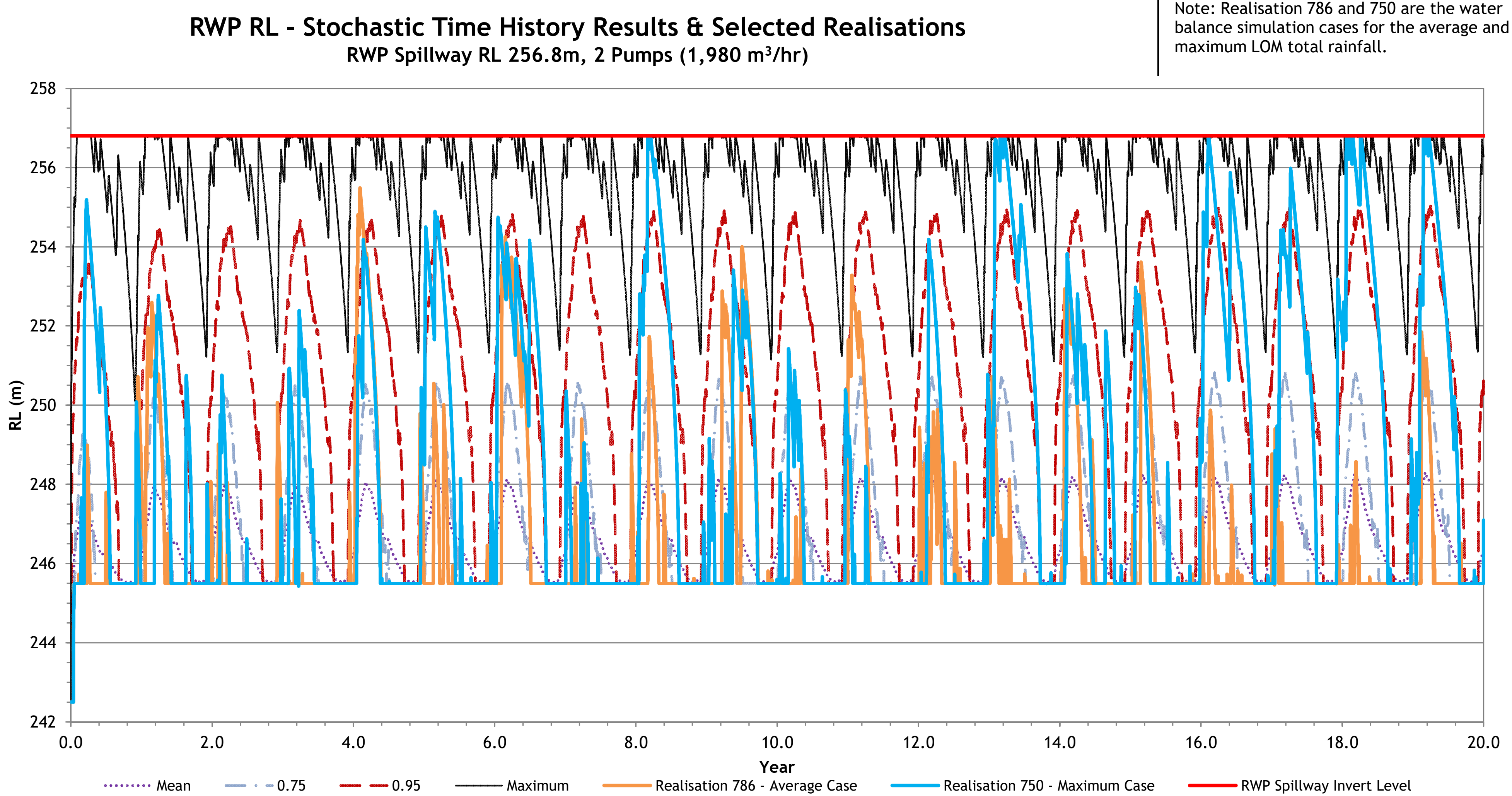
RAINFALL ANALYSIS AND WATER BALANCE REPORT

RWP Volumes - Stochastic Results and Selected Realisations

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E13**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx RWP Vol

Note: Realisation 786 and 750 are the water balance simulation cases for the average and maximum LOM total rainfall.



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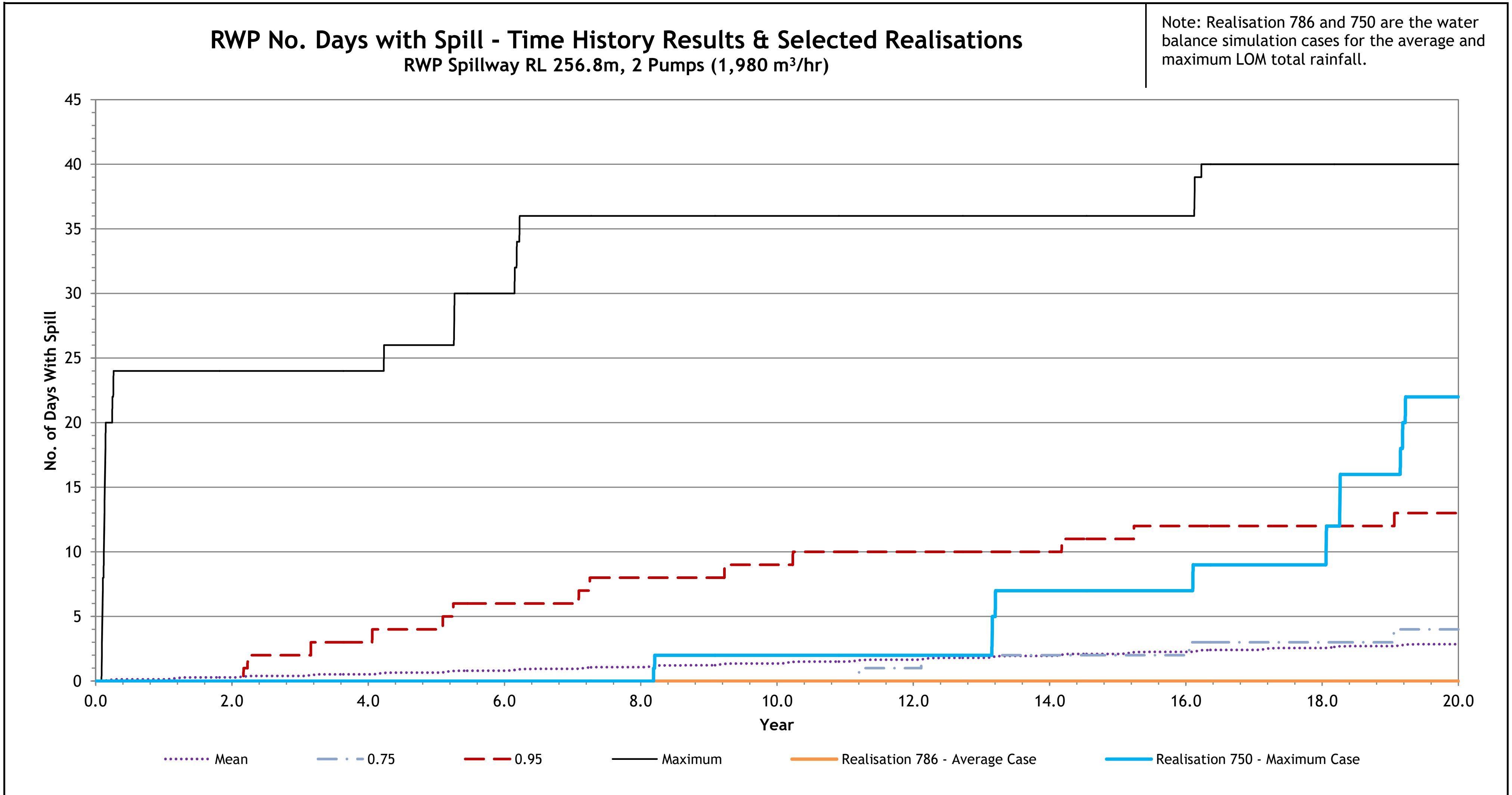
RAINFALL ANALYSIS AND WATER BALANCE REPORT

RWP RL - Stochastic Results and Selected Realisations

Date: 29/10/2018 **Job No:** 114185.14

FIGURE E14

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx RWP RL



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NORTH STAR MAGNETITE TSF STAGE 2			
RAINFALL ANALYSIS AND WATER BALANCE REPORT			
RWP Number of Days With Spillway Discharge			
Date: 29/10/2018	**Job No:** 114185.14		**FIGURE E15**

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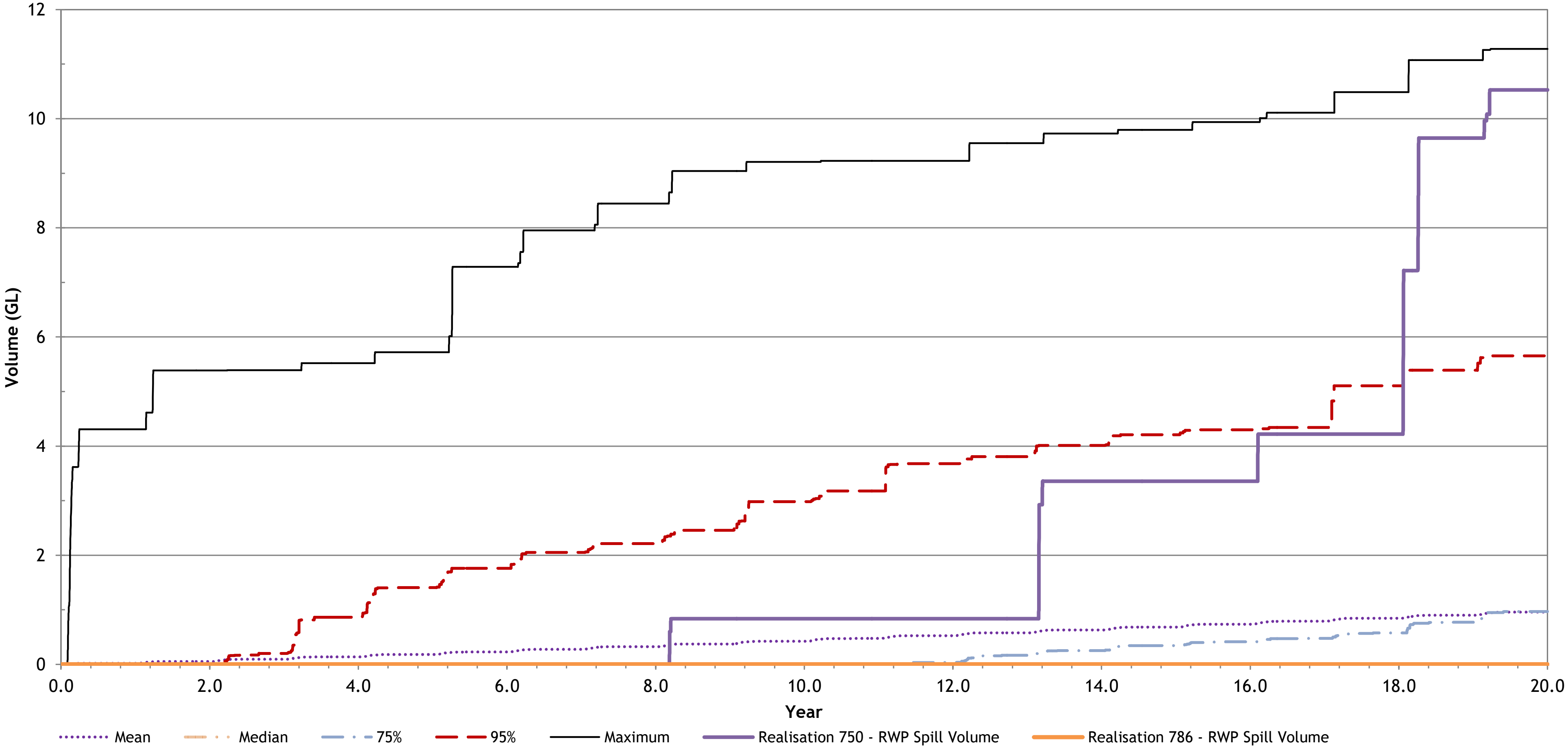
K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx No Spill Days

RWP Cum. Spill Volume - Stochastic Time History Results & Selected Realisations

RWP Spillway RL 256.8m, 2 Pumps (1,980 m³/hr)

Note: Realisation 786 and 750 are the water balance simulation cases for the average and maximum LOM total rainfall.



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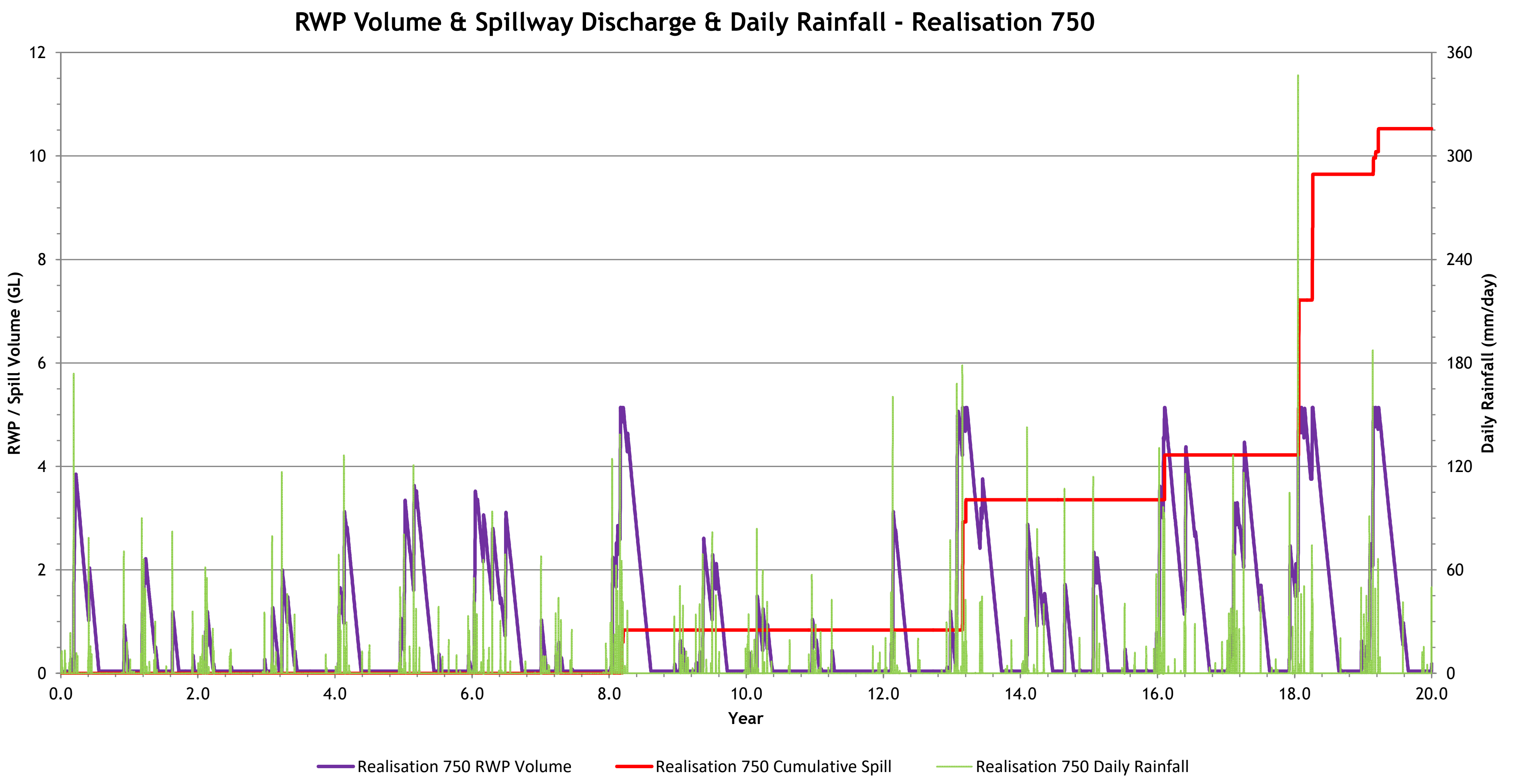
NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

RWP Cumulative Spill Volumes - Stochastic Results and Selected Realisations

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E16**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx RWP Cum Spill Vol



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NORTH STAR MAGNETITE TSF STAGE 2

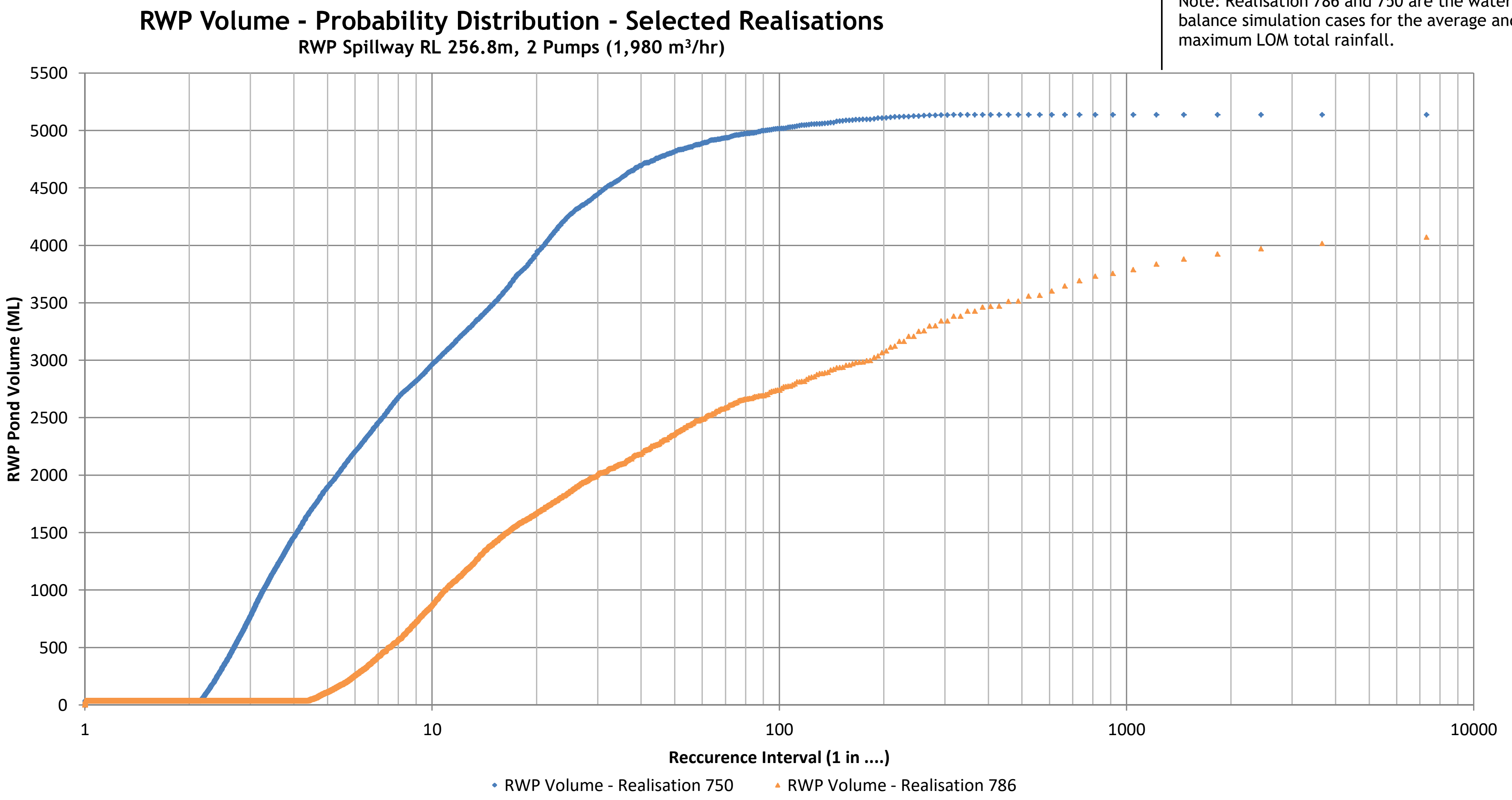
RAINFALL ANALYSIS AND WATER BALANCE REPORT

RWP Volumes, Cumulative Spill and Rainfall - Realisation 750

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E17**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx Realisation 750

Note: Realisation 786 and 750 are the water balance simulation cases for the average and maximum LOM total rainfall.



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NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

RWP Daily Volume Probability Distribution - Realisation 786 and 750

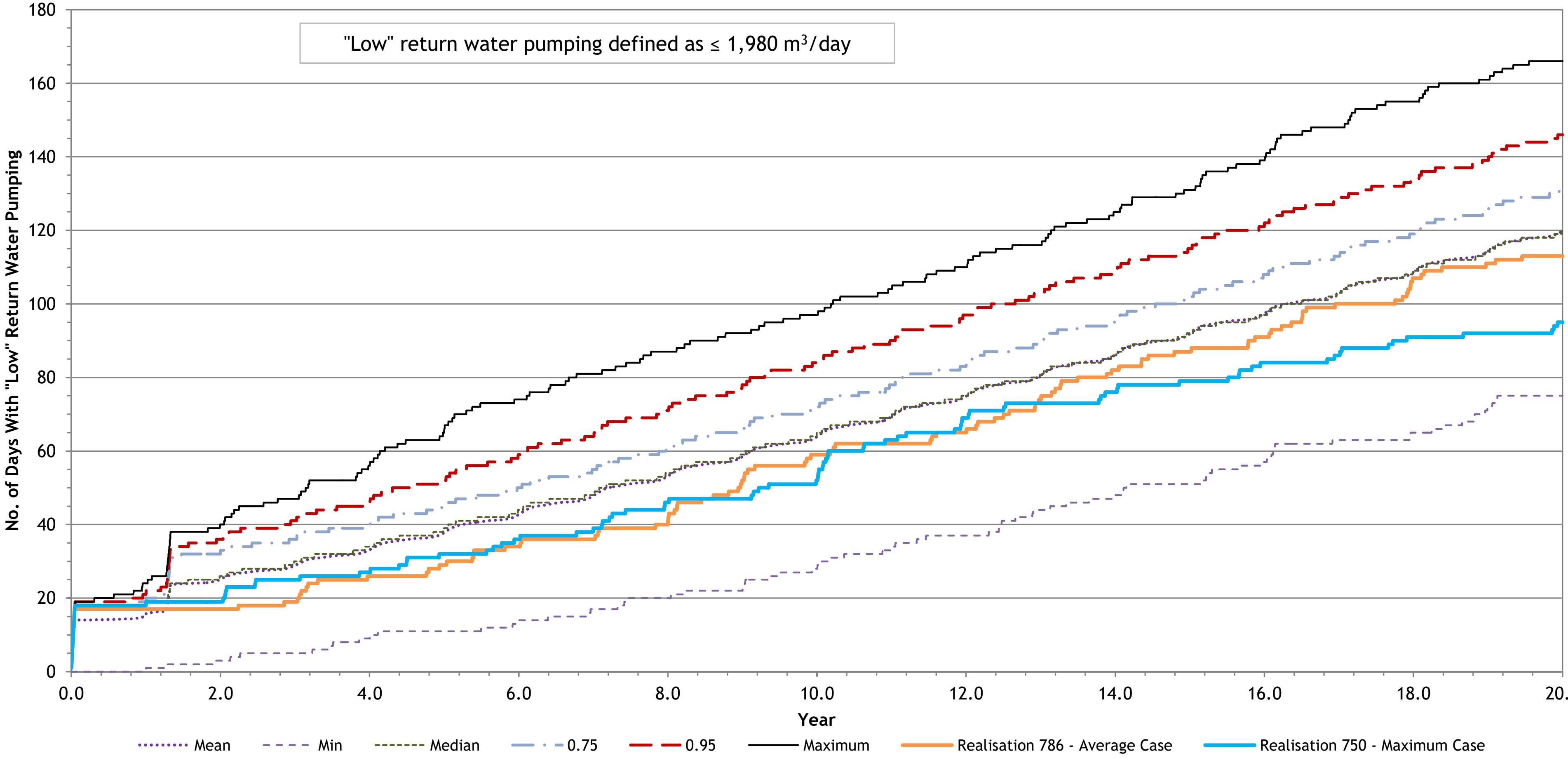
Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E18**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx RWP Vol Dist

RWP No. Days with "Low" Return Water - Time Histories & Selected Realisations

RWP Spillway RL 256.8m, 2 Pumps (1,980 m³/hr)

Note: Realisation 786 and 750 are the water balance simulation cases for the average and maximum LOM total rainfall.



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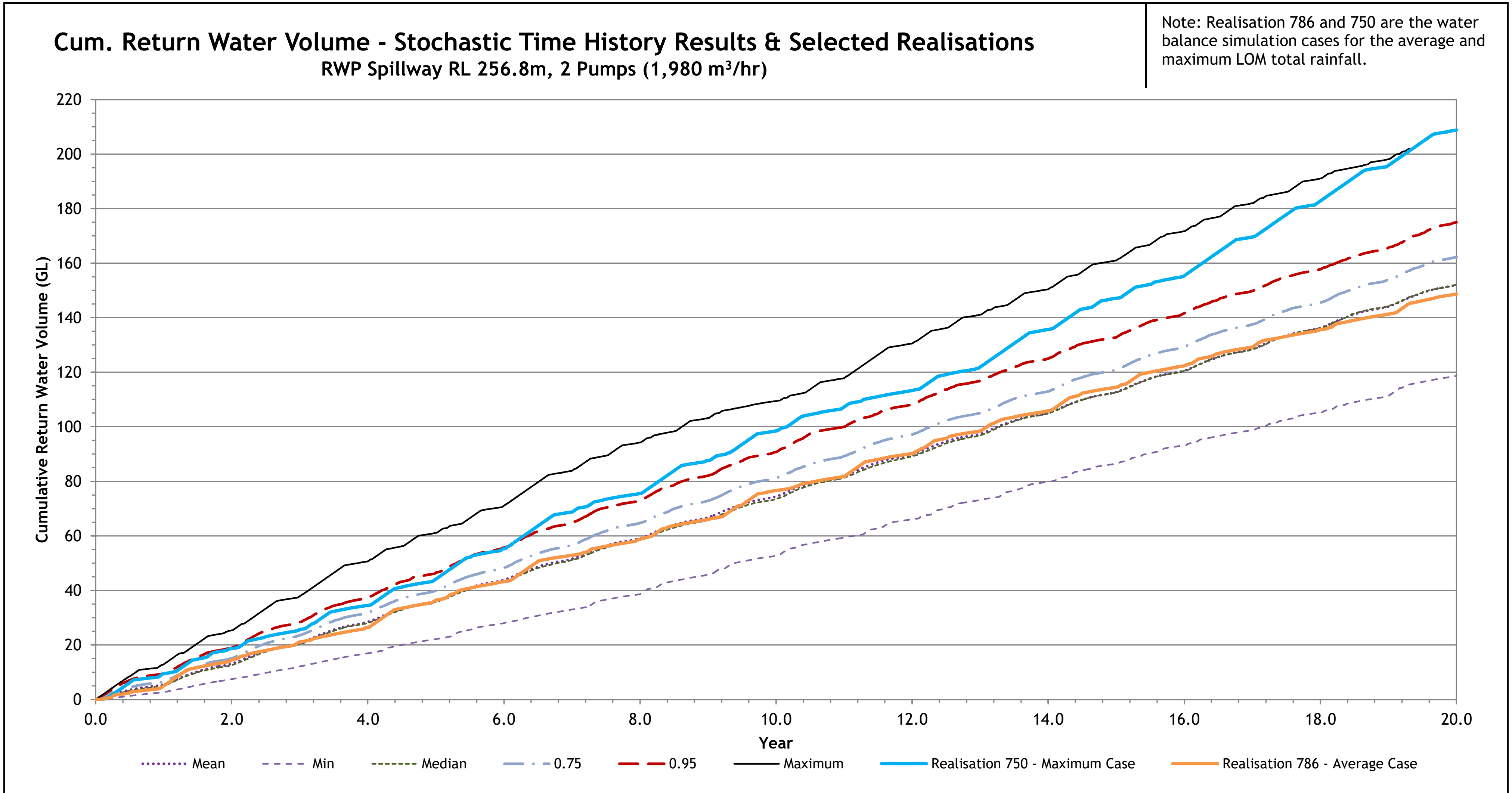
RAINFALL ANALYSIS AND WATER BALANCE REPORT

RWP - Number of Days with "Low" Return Water Pumping

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E19**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx Low Pump Days



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NORTH STAR MAGNETITE TSF STAGE 2

RAINFALL ANALYSIS AND WATER BALANCE REPORT

Cumulative Return Water Volumes - Stochastic Results and Selected Realisations

Date:	29/10/2018	**Job No:**	114185.14	**FIGURE E20**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\Water Balance\Water Balance Model Output.xlsx Cum Return Vol

REFERENCES

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[2] SILO: Scientific information for land owners (2018), “SILO - Queensland Government” [online at https://silo.longpaddock.qld.gov.au/].

[3] Srikanthan, S., Chiew, F and Frost, A, (February 2007), “User Guide, Stochatic Climate Library (SCL)”, Revision 2.1b, Cooperative Research Centre for Catchment Hydrology – Climate Variability Program.

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[5] Luke, G. J., Burke, K. L., & O'Brien, T. M. (1988), “Evaporation data for Western Australia”. Western Australia Department of Agriculture, Division of Resource Management.

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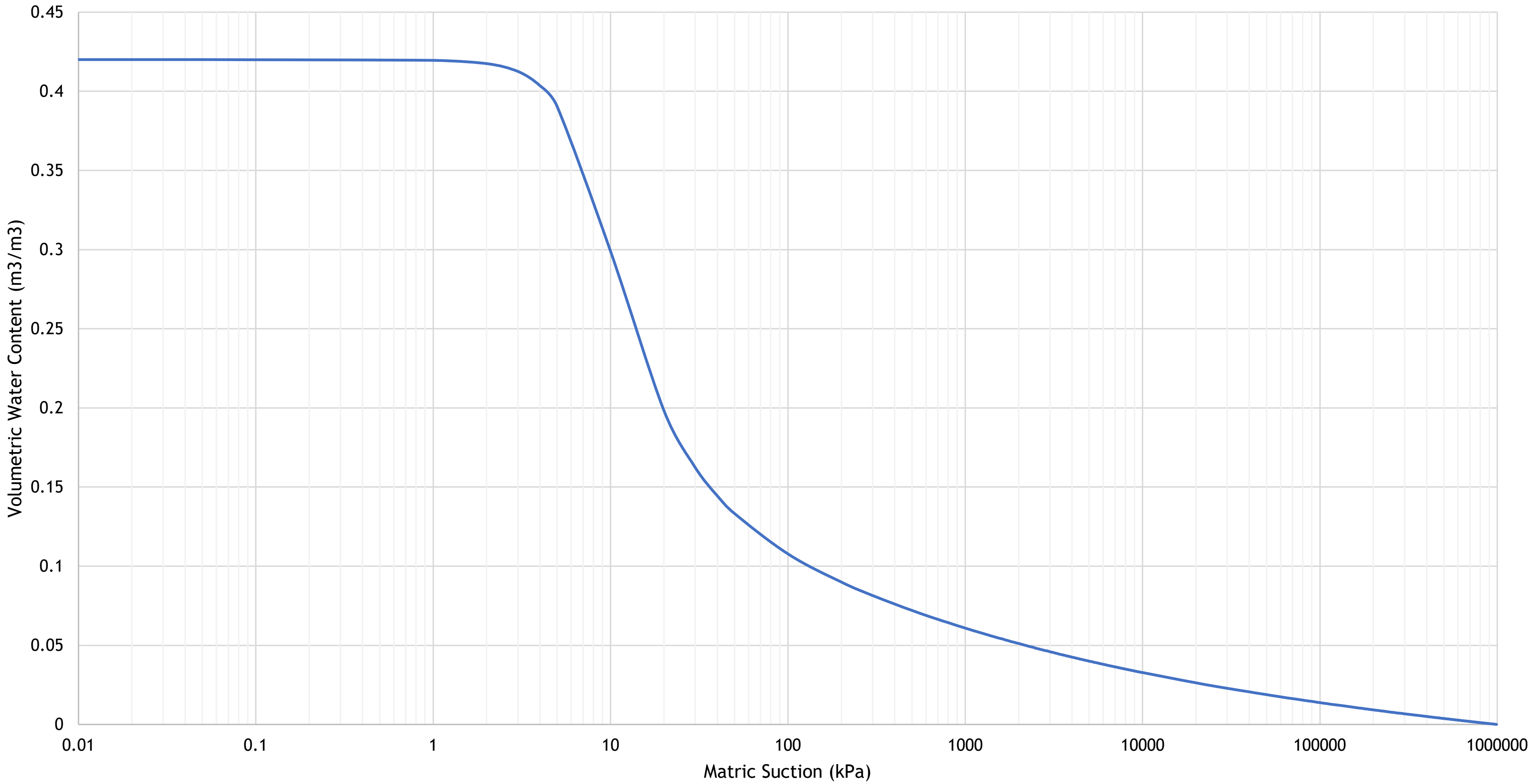
[8] GoldSim Technology Group (Feb 2017), “GoldSim Probabilistic Simulation Environment”, Version 12.0, Washington, USA.

APPENDIX F



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NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

North Star Tailings Soil Moisture Characteristic Curve (Estimated)

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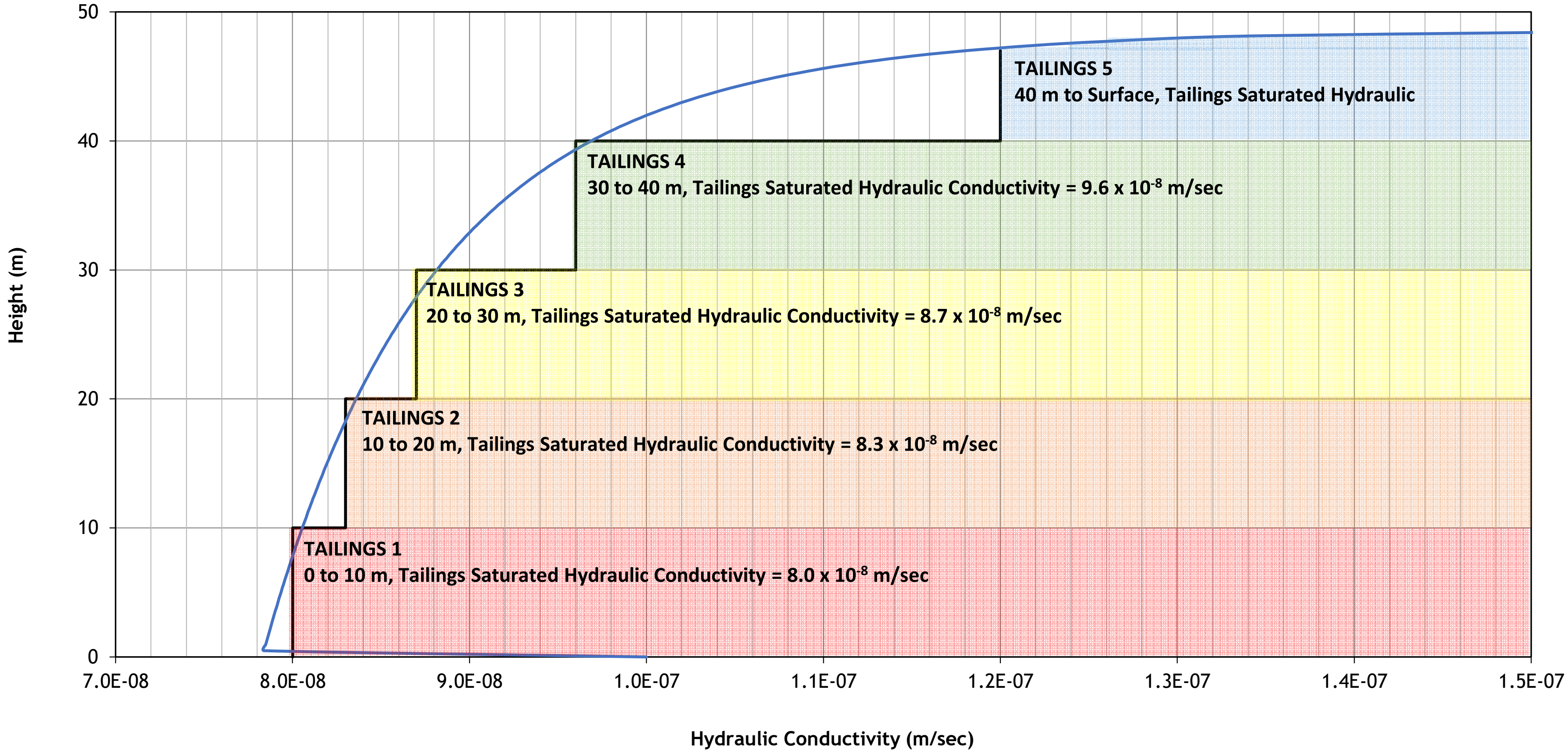
Date:	22/04/2020	**Job No:**	114185.26	**FIGURE F1**

K:\Projects\114\114185 North Star Magnetite Stage 2\21 Design Engineer Support\Data and Calcs\SeepW\JW\VWC Data.xlsx Fig 6



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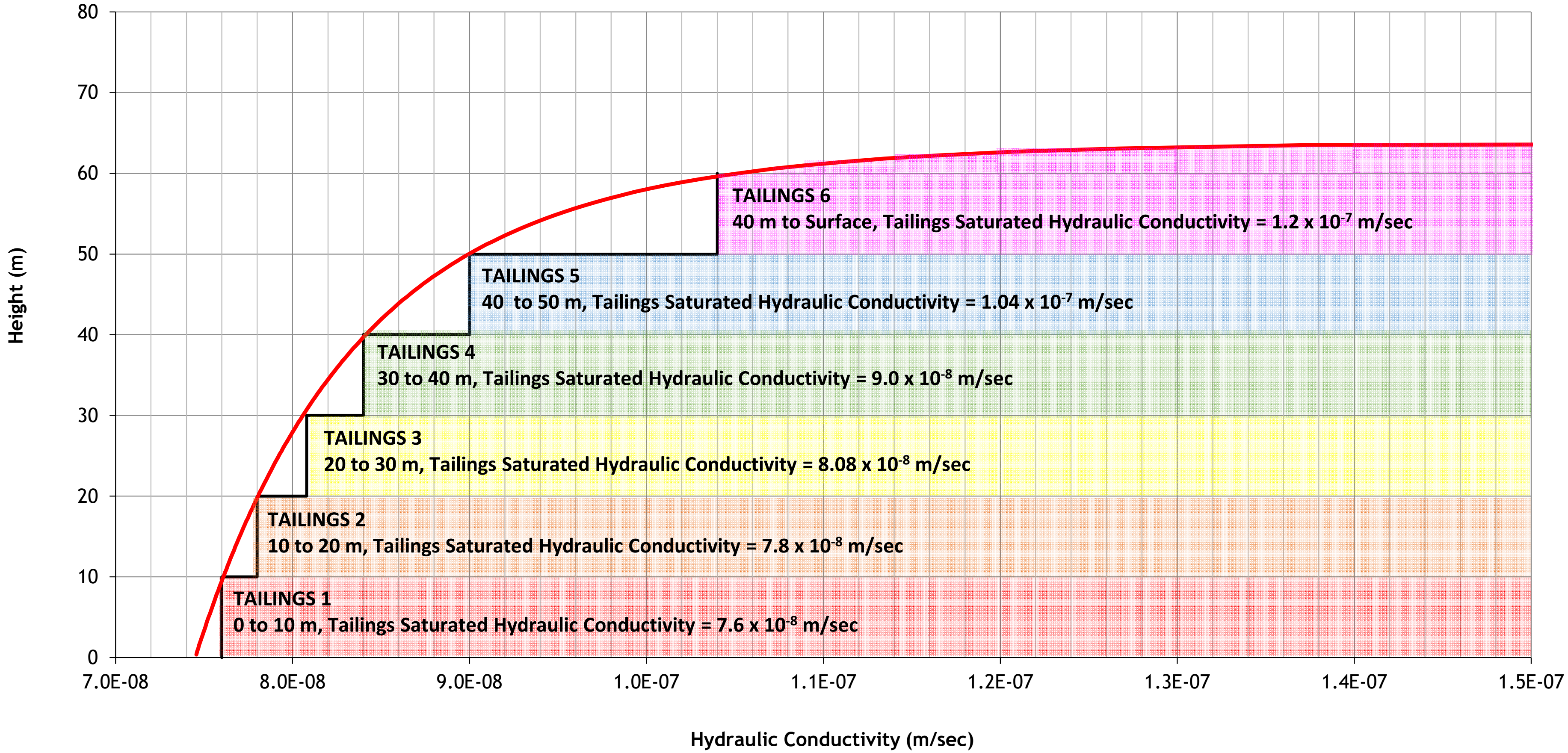
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

Main Embankment B - Stage 7 Saturated Hydraulic Conductivty Profiles

Date:	20/03/2020	**Job No:**	114185.14	**FIGURE F2**

K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Data and Calcs\AF\Base Model_76m.xlsx FIG 3



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NORTH STAR MAGNETITE - STAGE 2

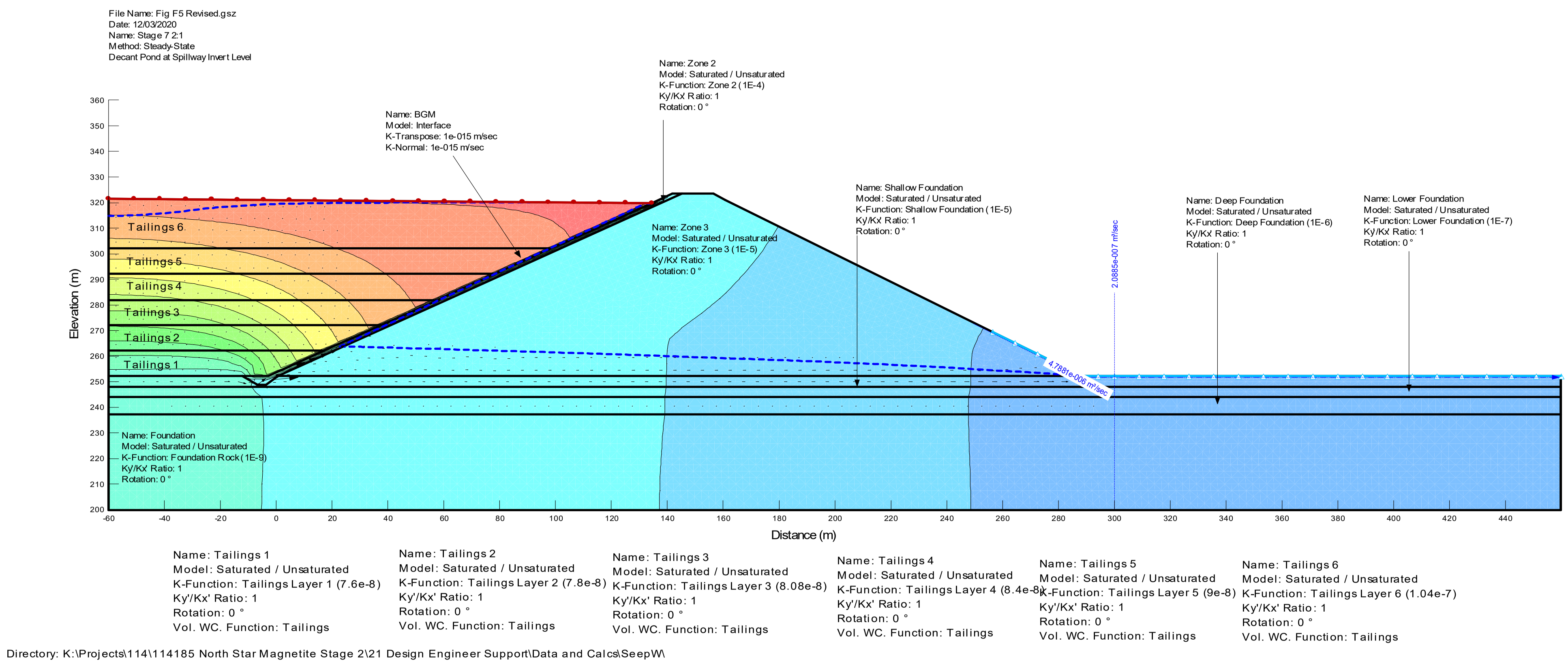
DESIGN REPORT FOR TAILINGS STORAGE FACILITY

Main Embankment A, C & D - Stage 7 Saturated Hydraulic Conductivty Profiles

Date: 20/03/2020 **Job No:** 114185.14

FIGURE F3

K:\Projects\114\114185 North Star Magnetite Stage 2\20 Tailings Test Work & Consolidation Modelling\Data and Calcs\AF\Base Model_76m.xlsx FIG 5



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NORTH STAR MAGNETITE - STAGE 2

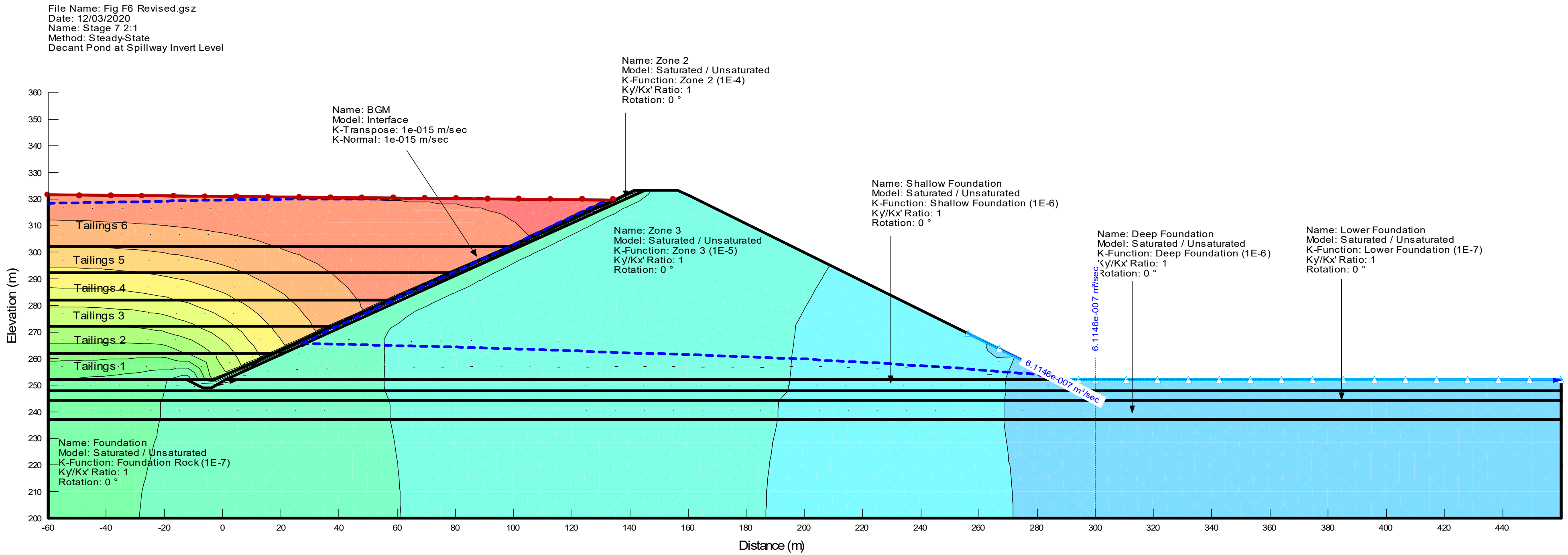
DESIGN REPORT FOR TAILINGS STORAGE FACILITY

Seepage Analysis for Final Stage (Stage 7) - Main Embankment A

Date: 22/04/2020 **Job No:** 114185.14

FIGURE F4

K:\Projects\114\114185 North Star Magnetite Stage 2\21 Design Engineer Support\Data and Calcs\SeepW\Fig 4, 4A, 5 and 6.xlsx Fig 5



Name: Tailings 1
Model: Saturated / Unsaturated
K-Function: Tailings Layer 1 (7.6e-8)
Ky/Kx' Ratio: 1
Rotation: 0 °
Vol. WC. Function: Tailings

Name: Tailings 2
Model: Saturated / Unsaturated
K-Function: Tailings Layer 2 (7.8e-8)
Ky/Kx' Ratio: 1
Rotation: 0 °
Vol. WC. Function: Tailings

Name: Tailings 3
Model: Saturated / Unsaturated
K-Function: Tailings Layer 3 (8.08e-8)
Ky/Kx' Ratio: 1
Rotation: 0 °
Vol. WC. Function: Tailings

Name: Tailings 4
Model: Saturated / Unsaturated
K-Function: Tailings Layer 4 (8.4e-8)
Ky/Kx' Ratio: 1
Rotation: 0 °
Vol. WC. Function: Tailings

Name: Tailings 5
Model: Saturated / Unsaturated
K-Function: Tailings Layer 5 (9e-8)
Ky/Kx' Ratio: 1
Rotation: 0 °
Vol. WC. Function: Tailings

Name: Tailings 6
Model: Saturated / Unsaturated
K-Function: Tailings Layer 6(1.04e-7)
Ky/Kx' Ratio: 1
Rotation: 0 °
Vol. WC. Function: Tailings

Directory: K:\Projects\114\114185 North Star Magnetite Stage 2\21 Design Engineer Support\Data and Calcs\SeepW\

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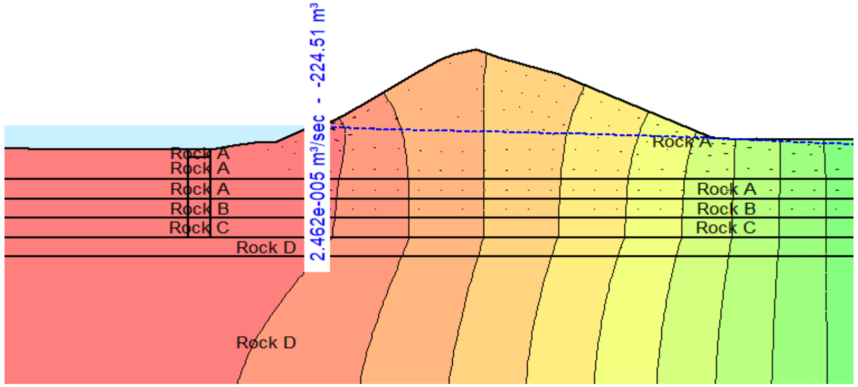
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

Seppage Analysis for Final Stage (Stage 7) - Main Embankment C

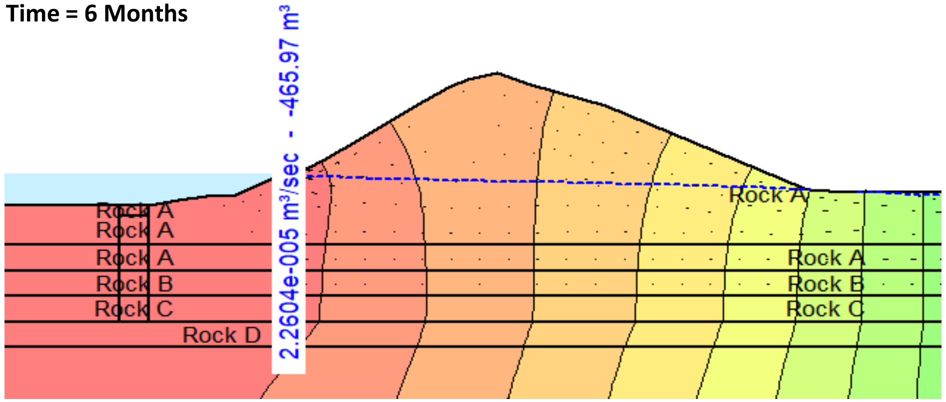
Date: 22/04/2020 **Job No:** 114185.14 **FIGURE F5**

Time = 2 Months

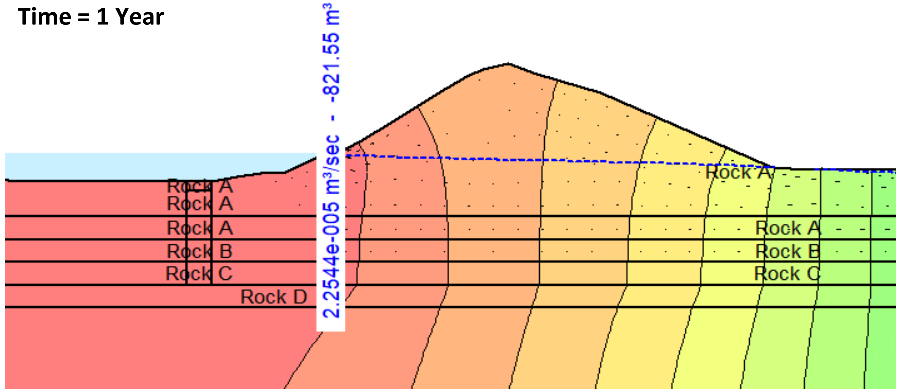


Name: Rock A K-Function: Rock A 4e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock B K-Function: Rock B 2.5e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock C K-Function: Rock C 3e-6 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock D K-Function: Rock D 2e-7 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: BGM K-Transpose: 6e-014 m/sec K-Normal: 6e-014 m/sec
Name: Zone 2 K-Function: Zone 2 5e-4 Ky'/Kx' Ratio: 1 Vol. WC. Function: Zone 2
Name: Zone 3 K-Function: Zone 3 1e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Zone 3
Name: Creek Sand K-Function: Creek Sand 2e-4 Ky'/Kx' Ratio: 1 Vol. WC. Function: Sand

Time = 6 Months



Time = 1 Year



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NORTH STAR MAGNETITE - STAGE 2

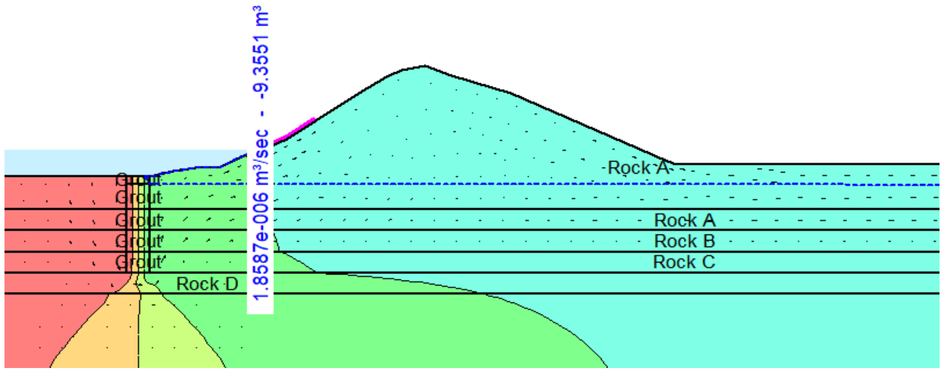
DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP Hill - No Grout ot Liner - Transient Seepage Analysis

Date: 10/10/2018 **Job No:** 114185.14

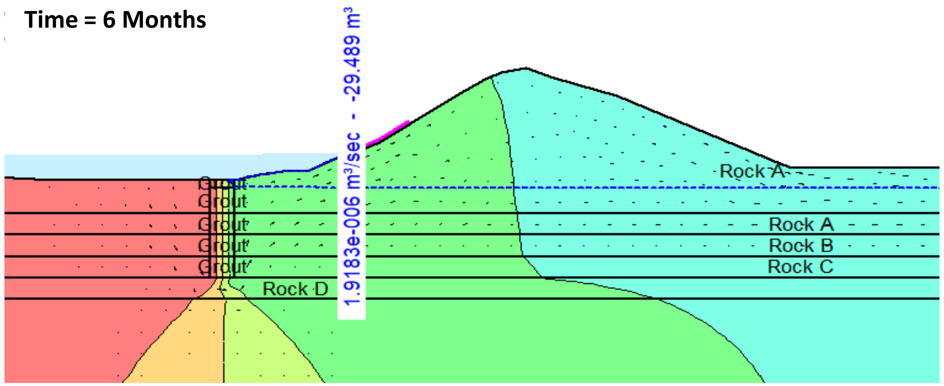
FIGURE F6

Time = 2 Months

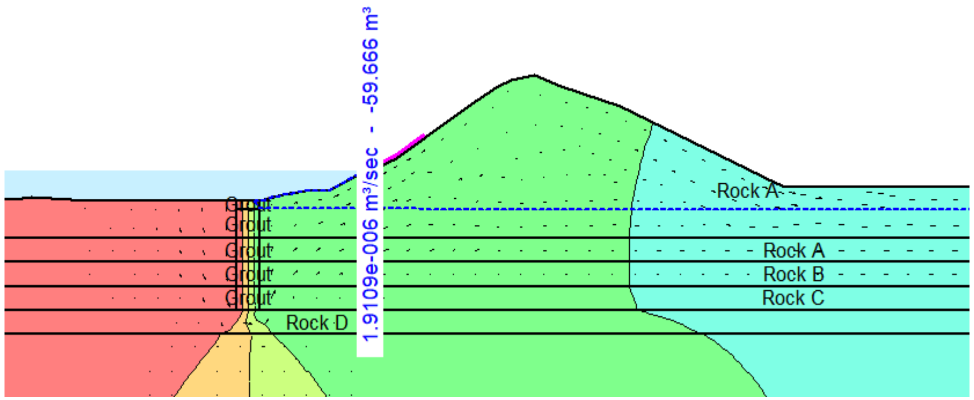


Name: Rock A K-Function: Rock A 4e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock B K-Function: Rock B 2.5e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock C K-Function: Rock C 3e-6 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock D K-Function: Rock D 2e-7 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: BGM K-Transpose: 6e-014 m/sec K-Normal: 6e-014 m/sec
Name: Zone 2 K-Function: Zone 2 5e-4 Ky'/Kx' Ratio: 1 Vol. WC. Function: Zone 2
Name: Zone 3 K-Function: Zone 3 1e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Zone 3
Name: Creek Sand K-Function: Creek Sand 2e-4 Ky'/Kx' Ratio: 1 Vol. WC. Function: Sand

Time = 6 Months



Time = 1 Year



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NORTH STAR MAGNETITE - STAGE 2

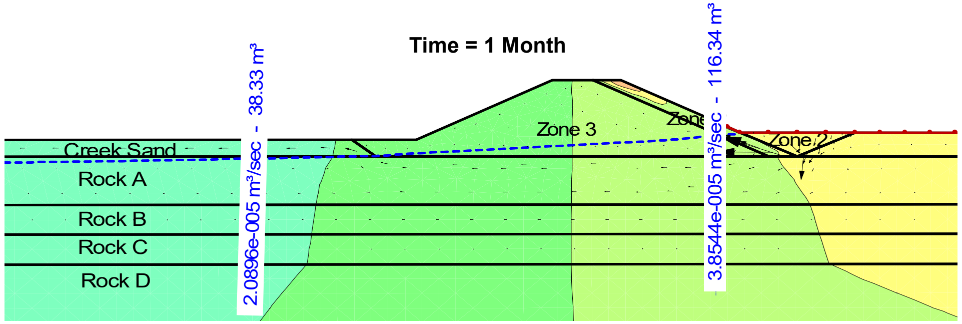
DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP Hill - Grout Curtain and BGM Liner - Transient Seepage Analysis

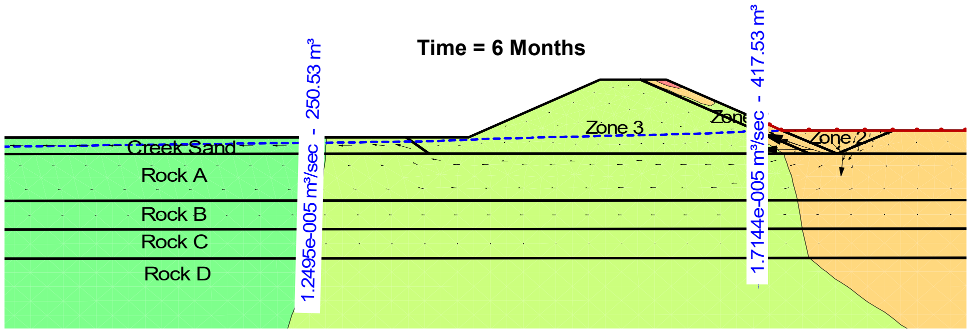
Date: 10/10/2018 **Job No:** 114185.14

FIGURE F7

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Documents\R03 (Design Report)\Appendices\F - Seepage\Appendix F.xlsx F6

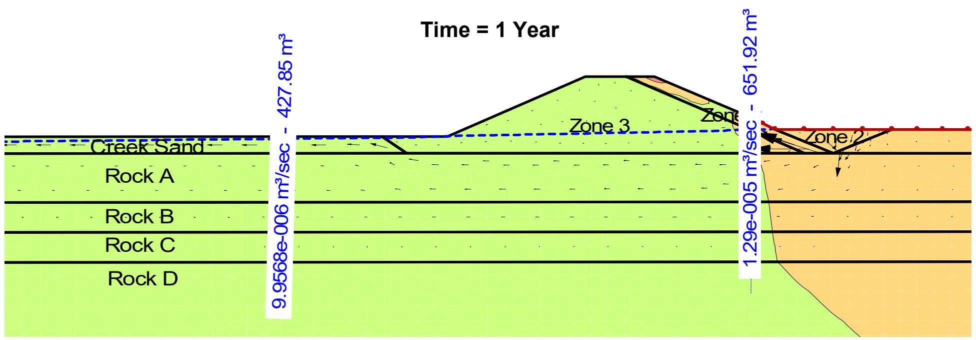


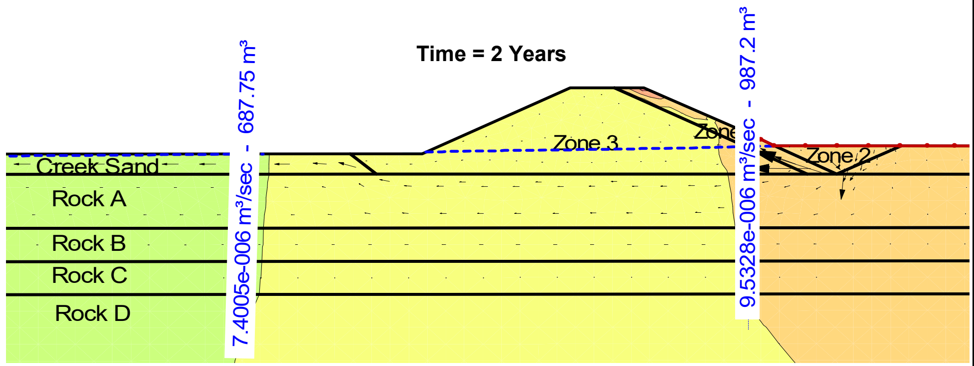
Name: Rock A K-Function: Rock A 4e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock B K-Function: Rock B 2.5e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock C K-Function: Rock C 3e-6 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: Rock D K-Function: Rock D 2e-7 Ky'/Kx' Ratio: 1 Vol. WC. Function: Rock
Name: BGM K-Transpose: 6e-014 m/sec K-Normal: 6e-014 m/sec
Name: Zone 2 K-Function: Zone 2 5e-4 Ky'/Kx' Ratio: 1 Vol. WC. Function: Zone 2
Name: Zone 3 K-Function: Zone 3 1e-5 Ky'/Kx' Ratio: 1 Vol. WC. Function: Zone 3
Name: Creek Sand K-Function: Creek Sand 2e-4 Ky'/Kx' Ratio: 1 Vol. WC. Function: Sand



RWP Pond Level = RL 250 m

Initial Ground Water Level = 6.5 m below natural





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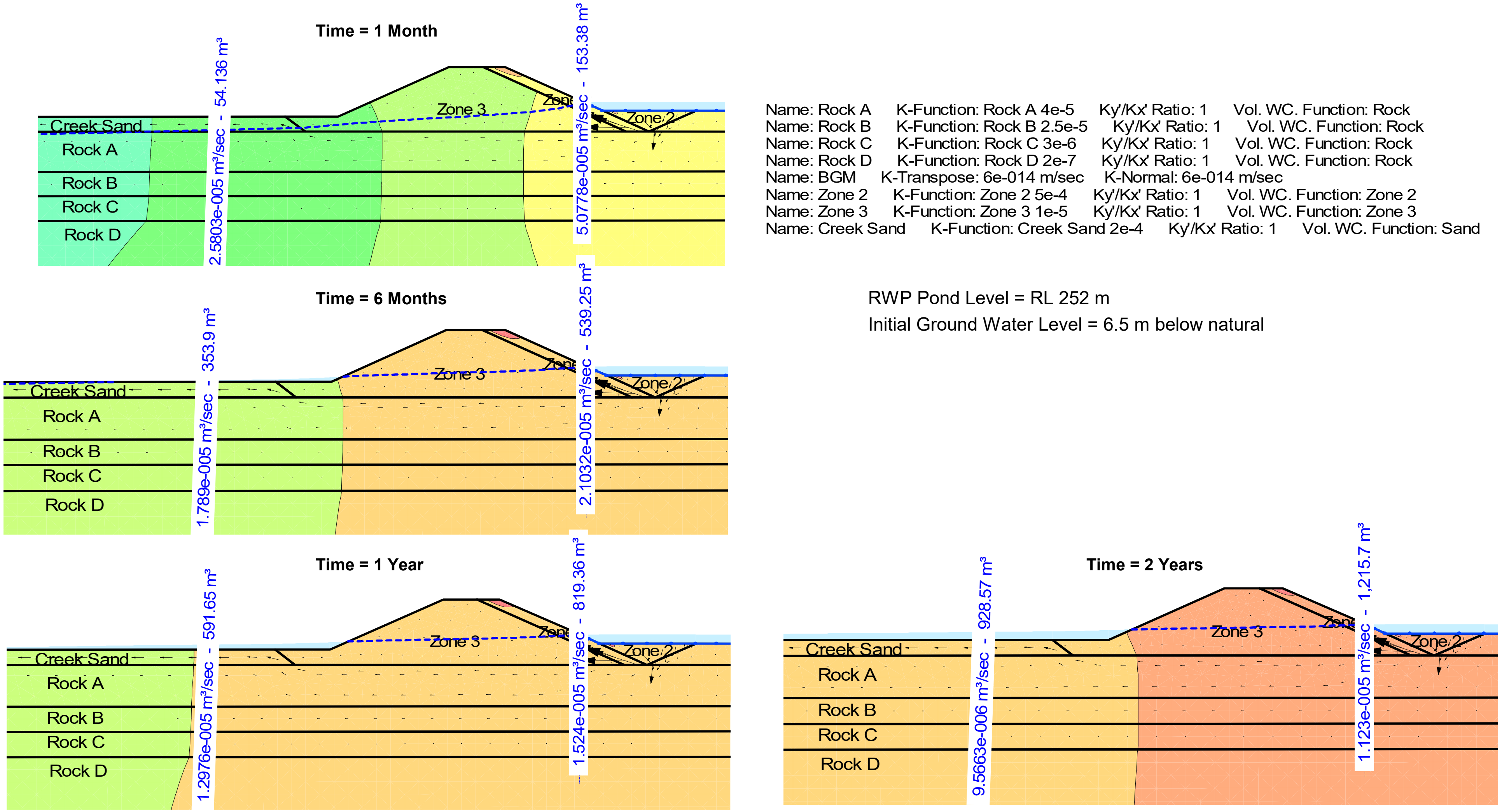
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP North Embankment - No Grout Curtain - Transient Seepage Analysis

Date: 10/10/2018 **Job No:** 114185.14 **FIGURE F8**



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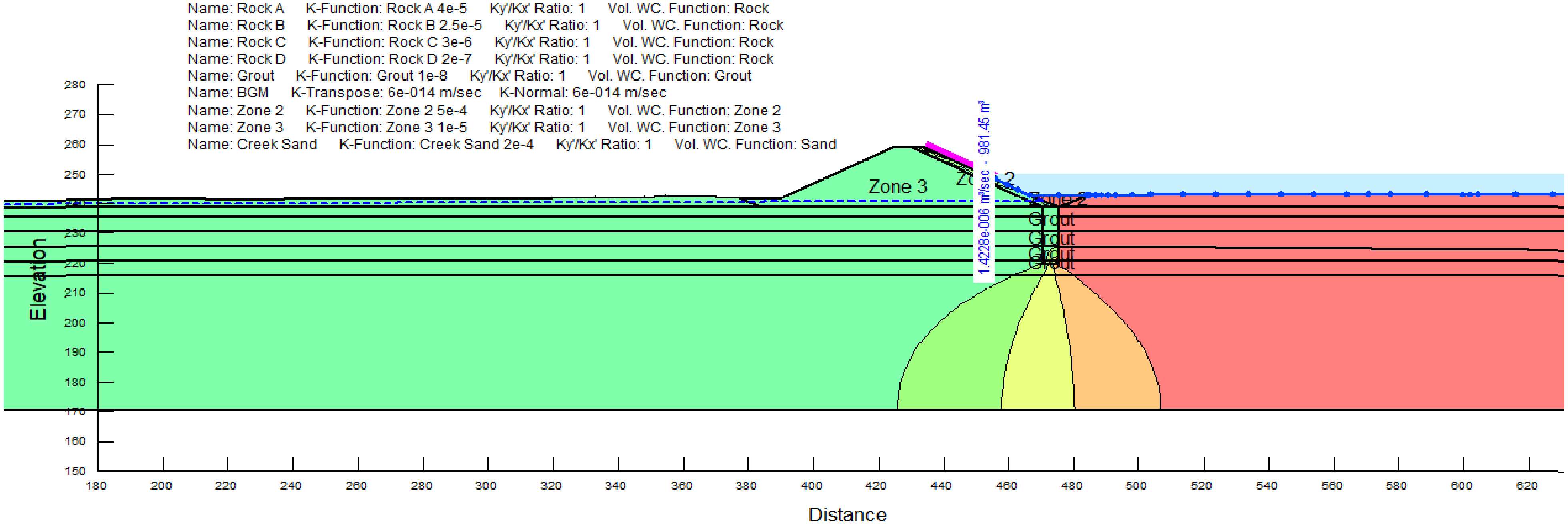
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP North Embankment - No Grout Curtain - Transient Seepage Analysis

Date:	10/10/2018	Job No:	114185.14	FIGURE F9

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Documents\R03 (Design Report)\Appendices\F - Seepage\Appendix F.xlsx F8



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NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP South Embankment - with Grout Curtain and BGM Liner - Transient Seepage Analysis

Date:	10/10/2018	**Job No:**	114185.14	**FIGURE F10**

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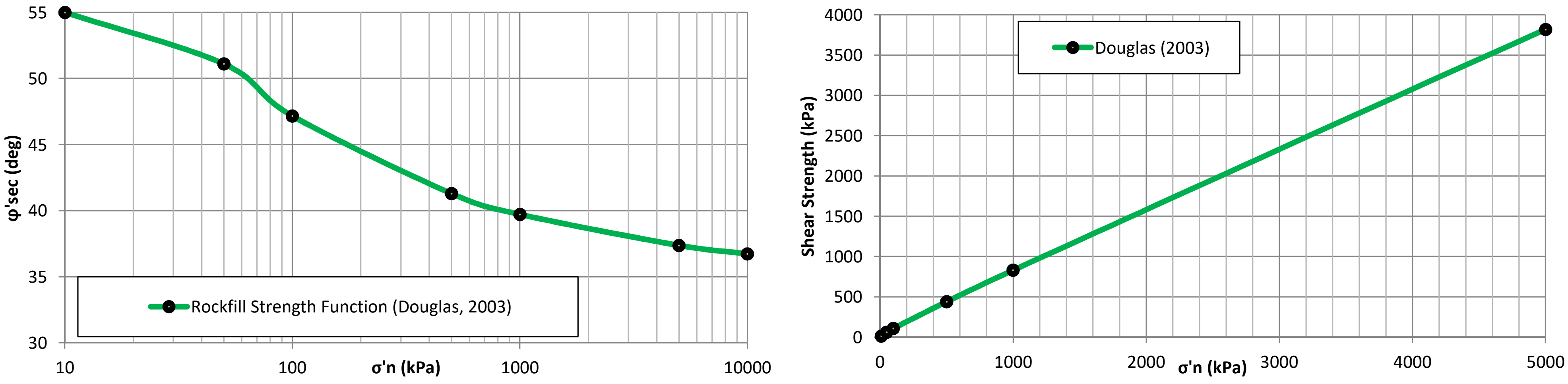
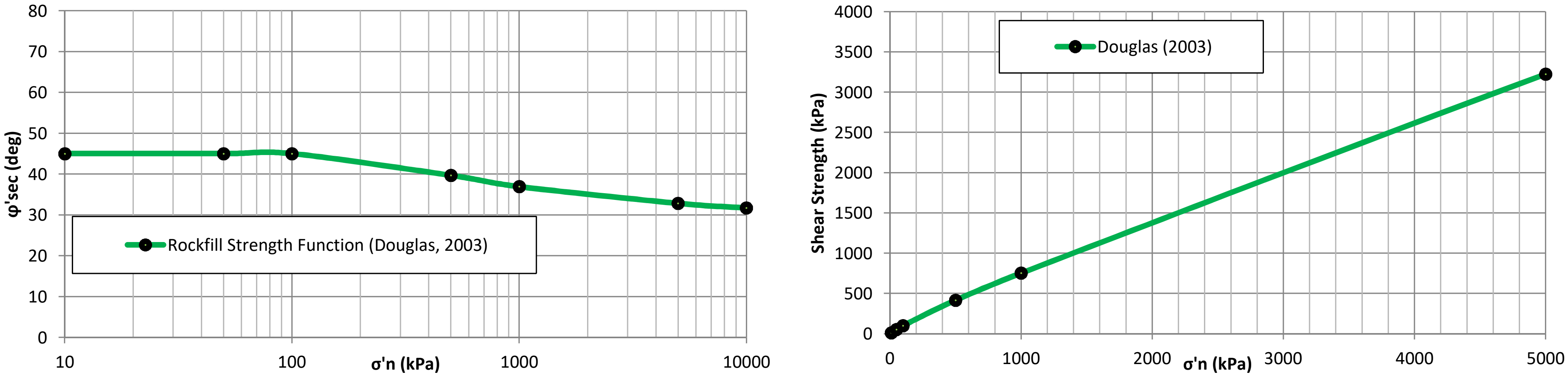
APPENDIX G

ZONE 2A SHEAR STRESS FUINCTIONS



ZONE 3B SHEAR STRESS FUINCTIONS



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NORTH STAR MAGNETITE - STAGE 2

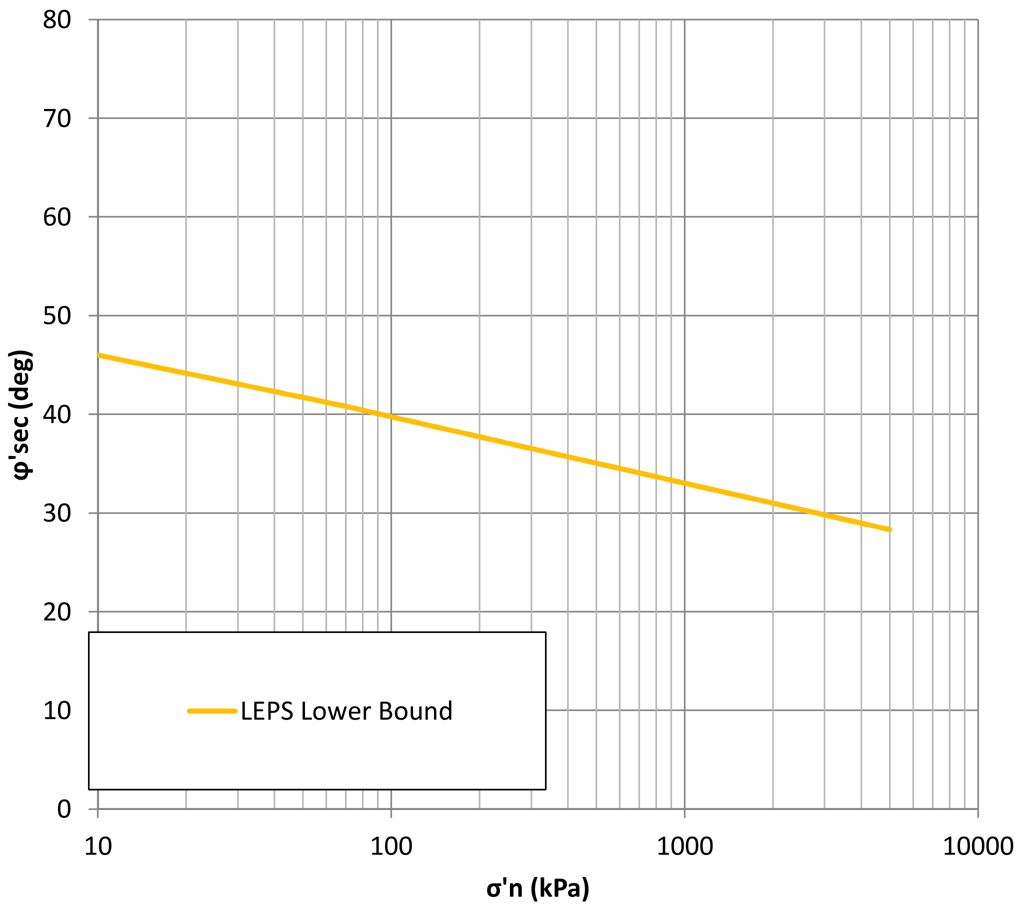
DESIGN REPORT FOR TAILINGS STORAGE FACILITY

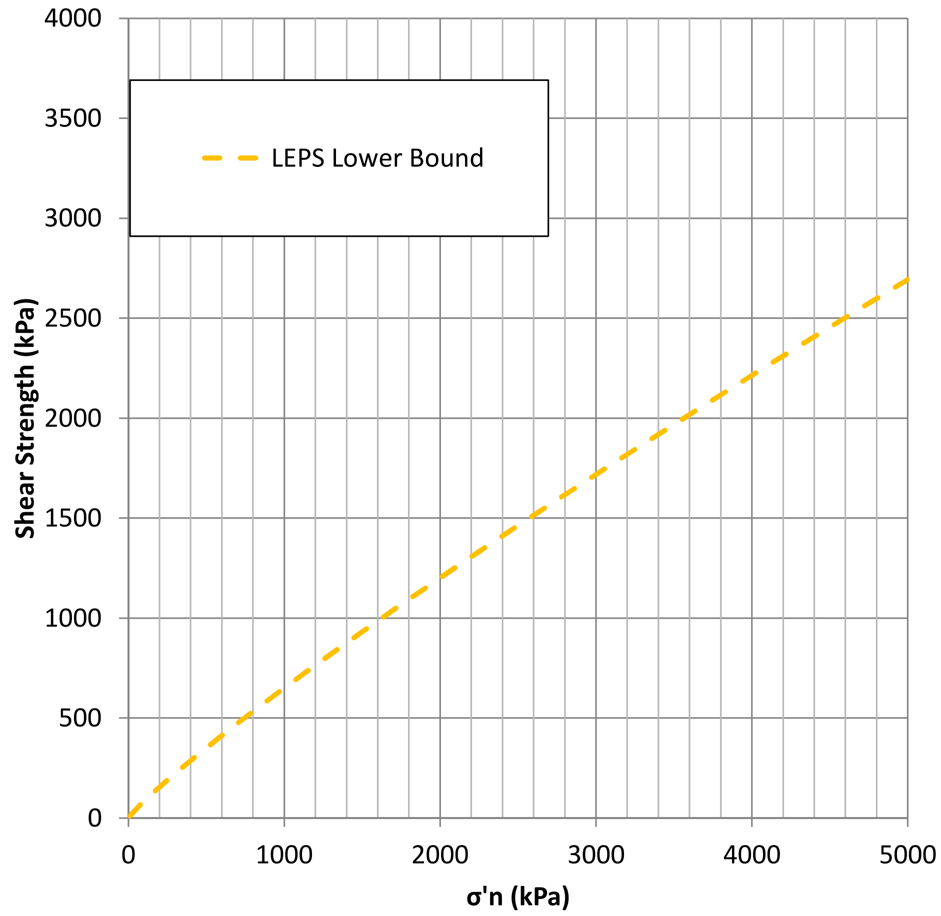
Douglas (2003) Shear Stress-Normal Stress Functions - Zone 2A and Zone 3B

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE G1**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G1

ZONE 3A SHEAR STRESS FUINCTIONS





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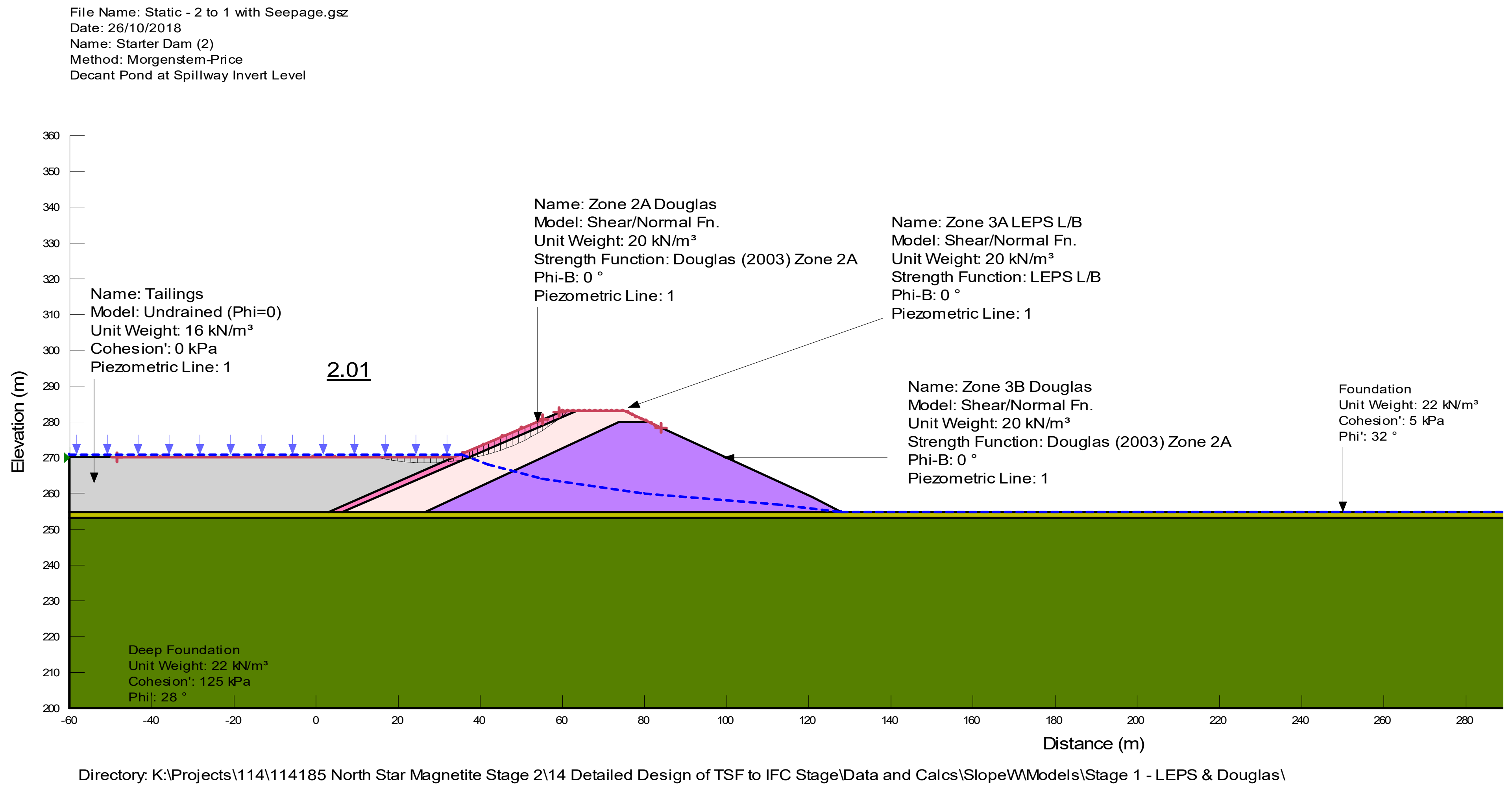
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

LEPS Lower Bound Shear Stress-Normal Stress Functions - Zone 3A

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE G2**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G2



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NORTH STAR MAGNETITE - STAGE 2

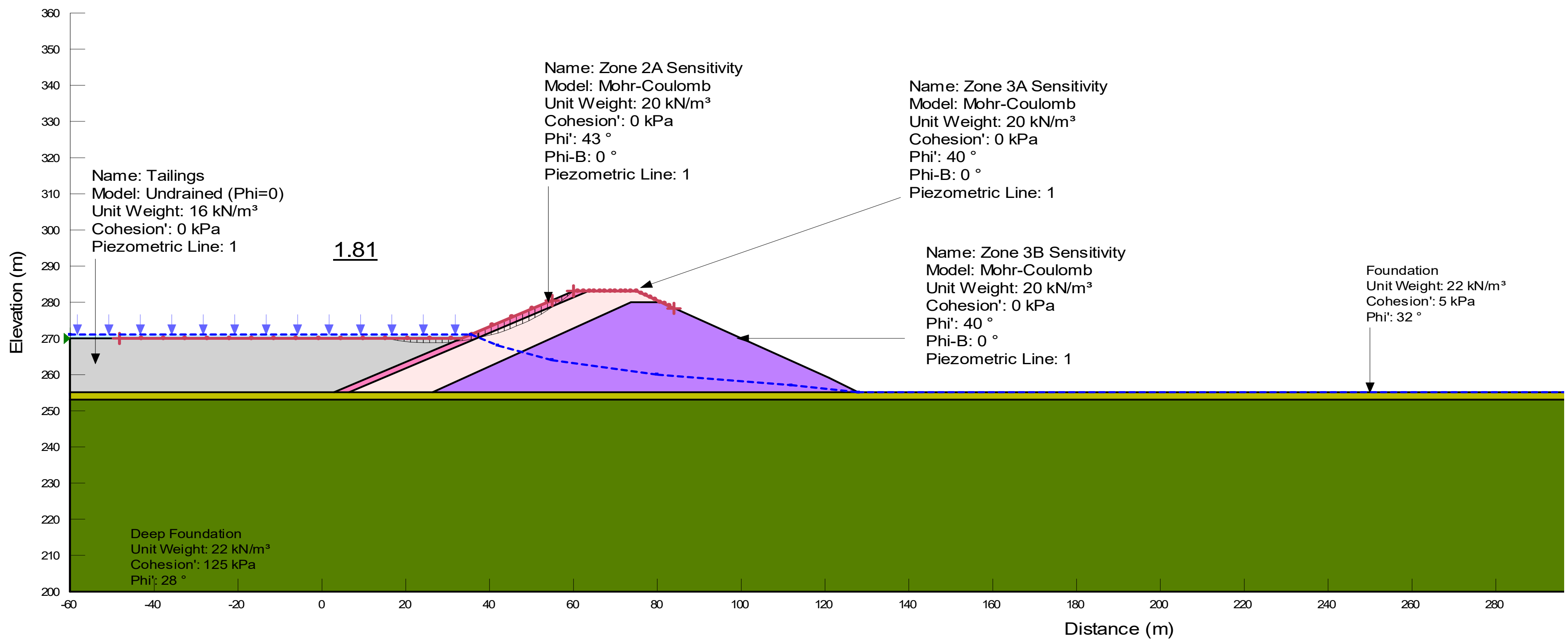
DESIGN REPORT FOR TAILINGS STORAGE FACILITY

TSF Starter Dam - (2:1 D/S Slope) - Static (LEPS and Douglas 2003 Strength Functions)

Date: 14/02/2019 **Job No:** 114185.14

FIGURE G3

File Name: Stage 1 Sensitivity (Static).gsz
Date: 31/01/2019
Name: Starter Dam
Method: Morgenstern-Price
Decant Pond at Spillway Invert Level



Directory: K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Models\Sensitivity Analyses\

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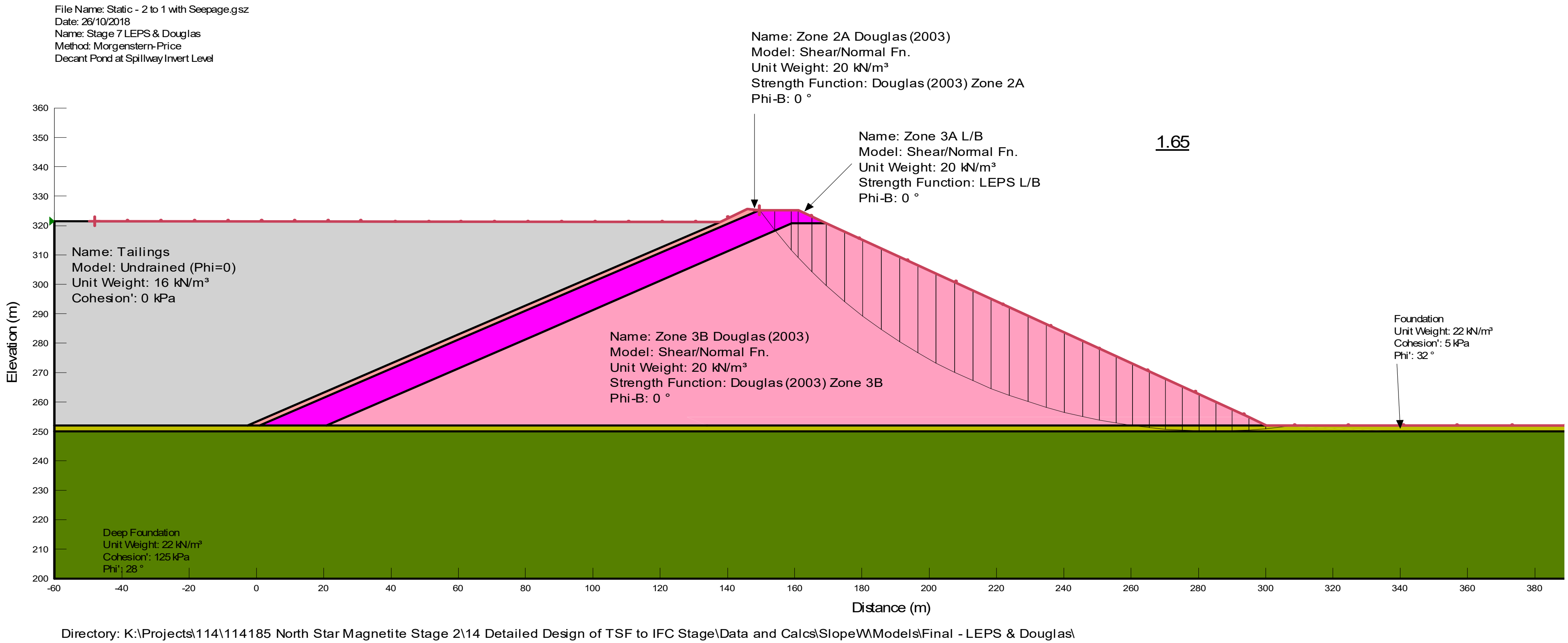
NORTH STAR MAGNETITE - STAGE 2

STABILITY ANALYSES SENSITIVITY ANALYSES

TSF Starter Dam - (2:1 D/S Slope) - Static

Date: 14/02/2019 **Job No:** 114185.14 **FIGURE G4**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G4



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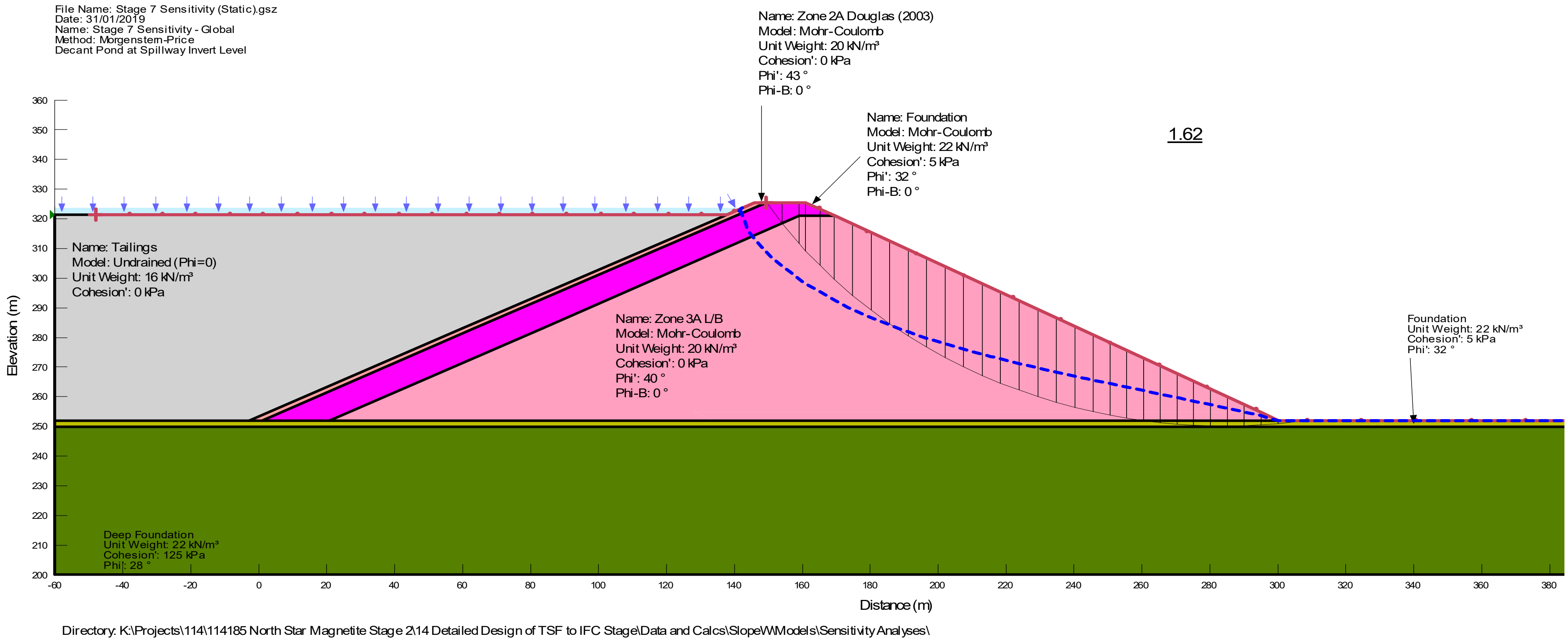
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

TSF Stage 7 - (2:1 D/S Slope) - Static

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE G5**



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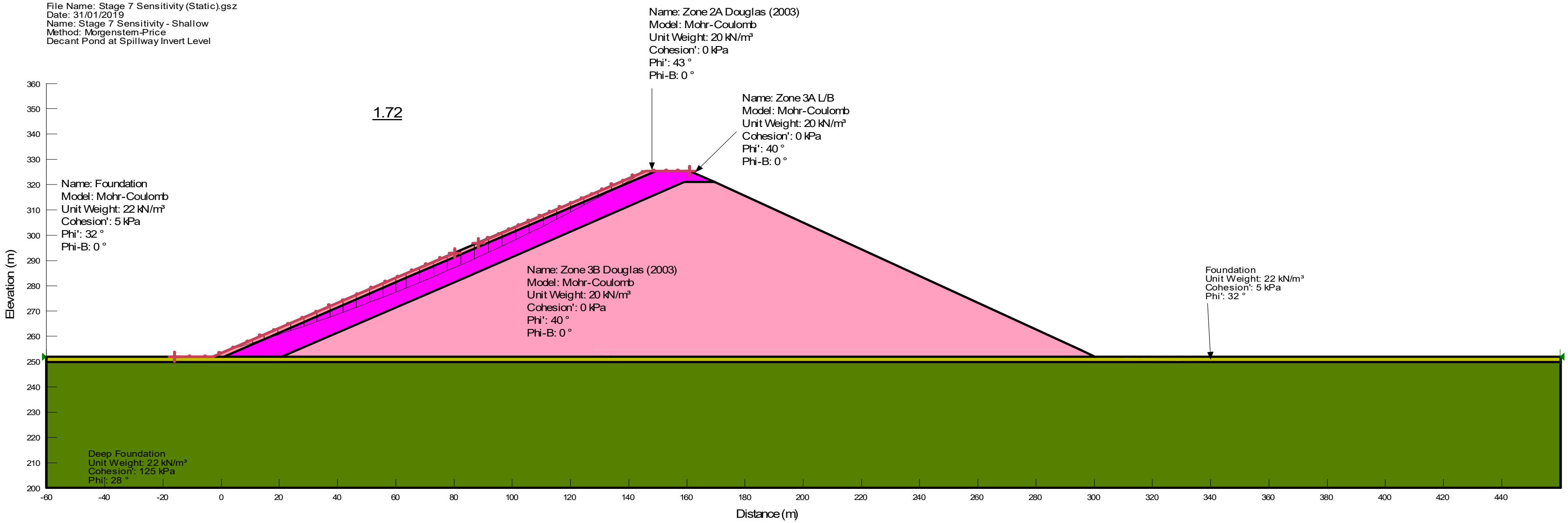
NORTH STAR MAGNETITE - STAGE 2

STABILITY ANALYSES SENSITIVITY ANALYSES

TSF Stage 7 - (2:1 D/S Slope) - Static

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE G6**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G6



Directory: K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Models\Sensitivity Analyses\

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NORTH STAR MAGNETITE - STAGE 2

STABILITY ANALYSES SENSITIVITY ANALYSES

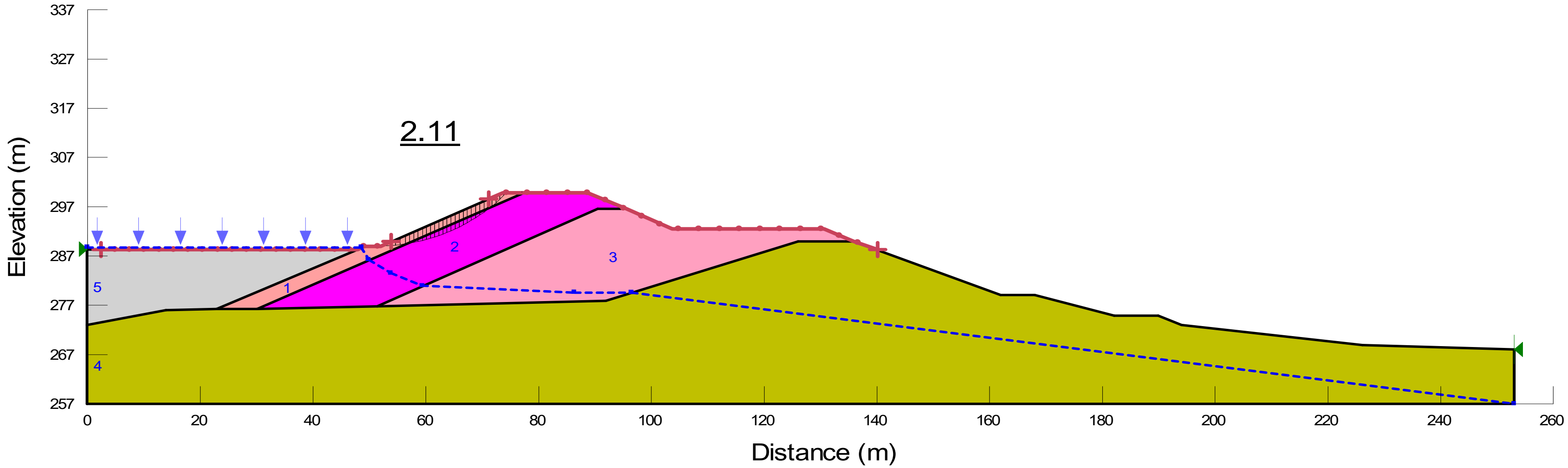
TSF Stage 7 - (2:1 D/S Slope) - Static - Shallow Failure

Date: 14/02/2019 **Job No:** 114185.14

FIGURE G7

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G7

File Name: Static - Unlined to RL 289.gsz
Date: 30/08/2018
Name: Stage 7 LEPS & Douglas
Method: Morgenstern-Price
Tailings Level at end of Stage 2



1
Name: Zone 2A Douglas (2003)
Model: Shear/Normal Fn.
Unit Weight: 20 kN/m³
Strength Function: Douglas (2003) Zone 2A
Phi-B: 0 °
Piezometric Line: 1

2
Name: Zone 3A L/B
Model: Shear/Normal Fn.
Unit Weight: 20 kN/m³
Strength Function: LEPS L/B
Phi-B: 0 °
Piezometric Line: 1

3
Name: Zone 3B Douglas (2003)
Model: Shear/Normal Fn.
Unit Weight: 20 kN/m³
Strength Function: Douglas (2003) Zone 3B
Phi-B: 0 °
Piezometric Line: 1

4
Name: Foundation
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion': 125 kPa
Phi': 28 °
Phi-B: 0 °
Piezometric Line: 1

5
Name: Tailings
Model: Undrained (Phi=0)
Unit Weight: 16 kN/m³
Cohesion': 0 kPa
Piezometric Line: 1

Directory: K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Models\Unlined Section\

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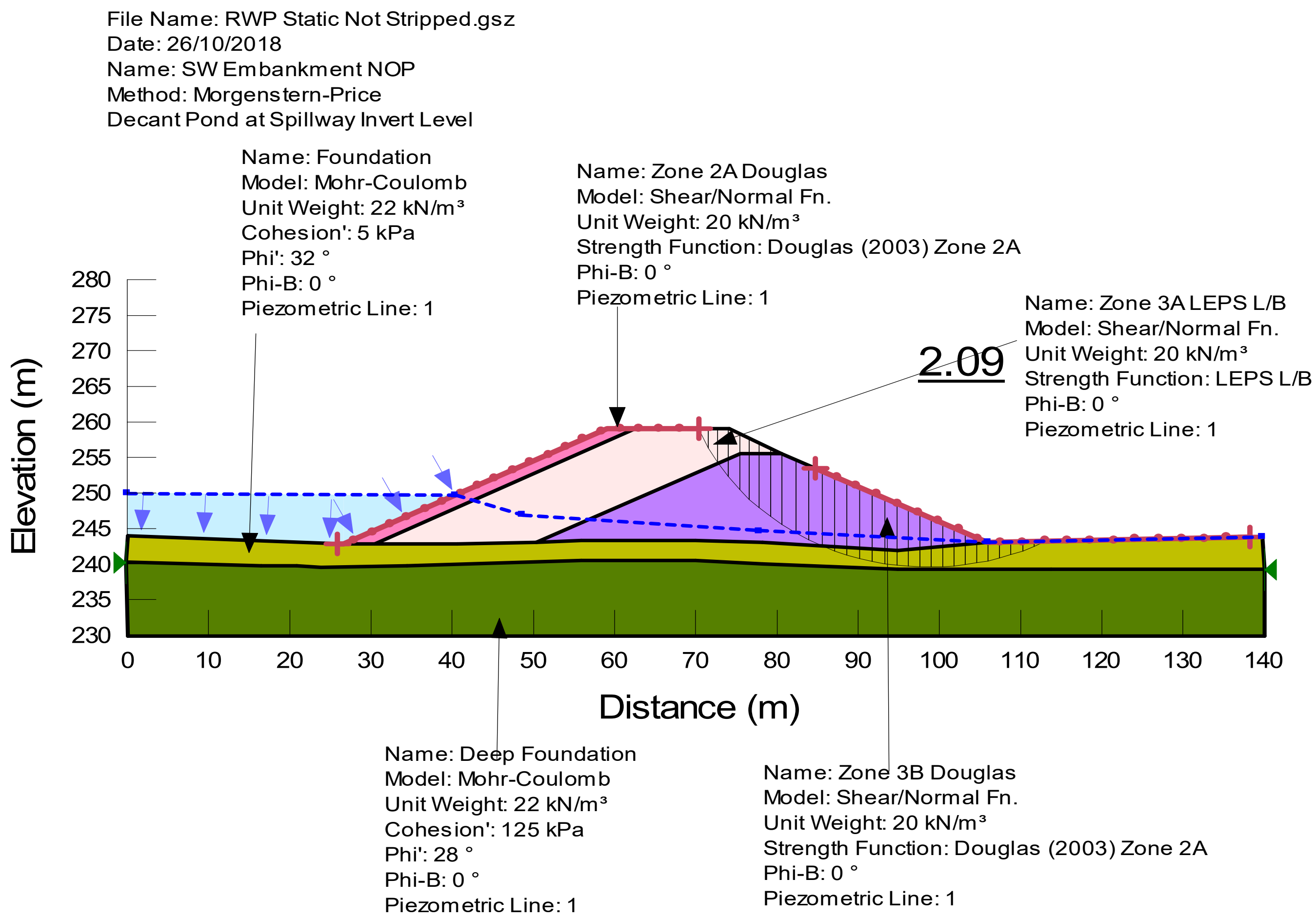
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

TSF Stage 3 - (2:1 D/S Slope) - Static - Unlined to RL 289 m

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE G8**



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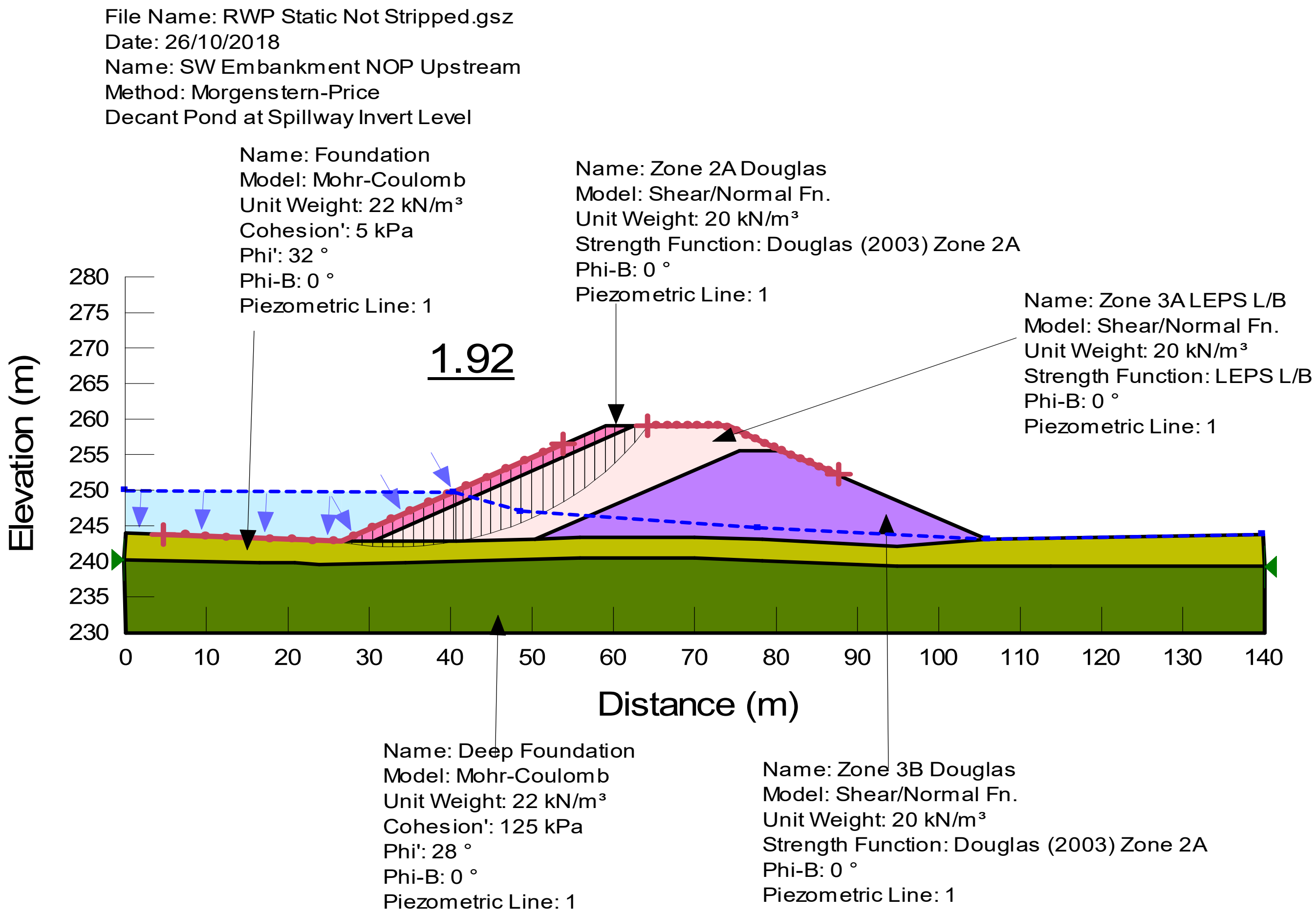
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP - (2:1 Side Slope) - Static - Normal Operating Pond Downstream

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE G9**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G9



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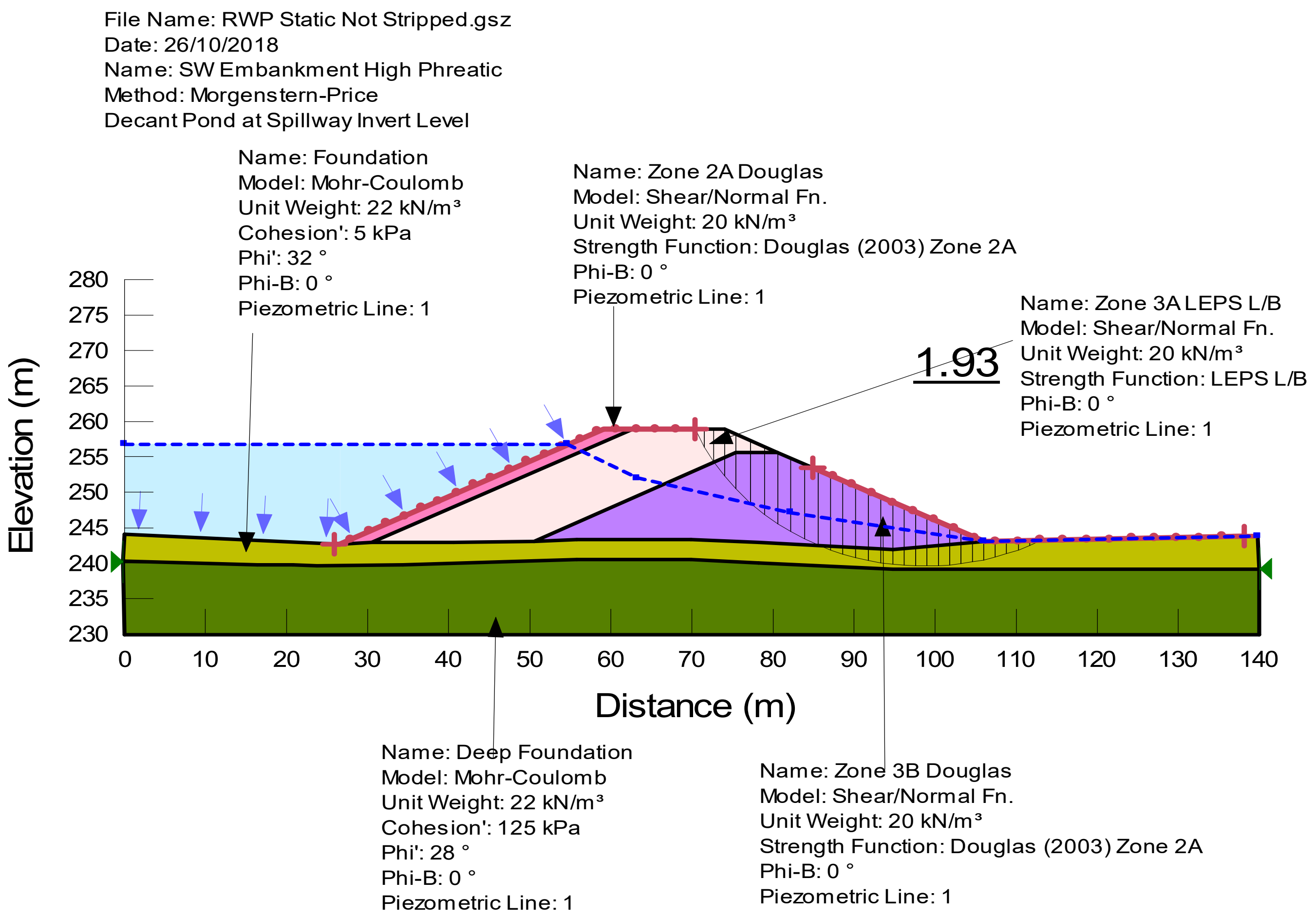
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP - (2:1 Side Slope) - Static - Normal Operating Pond Upstream

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE G10**

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G10



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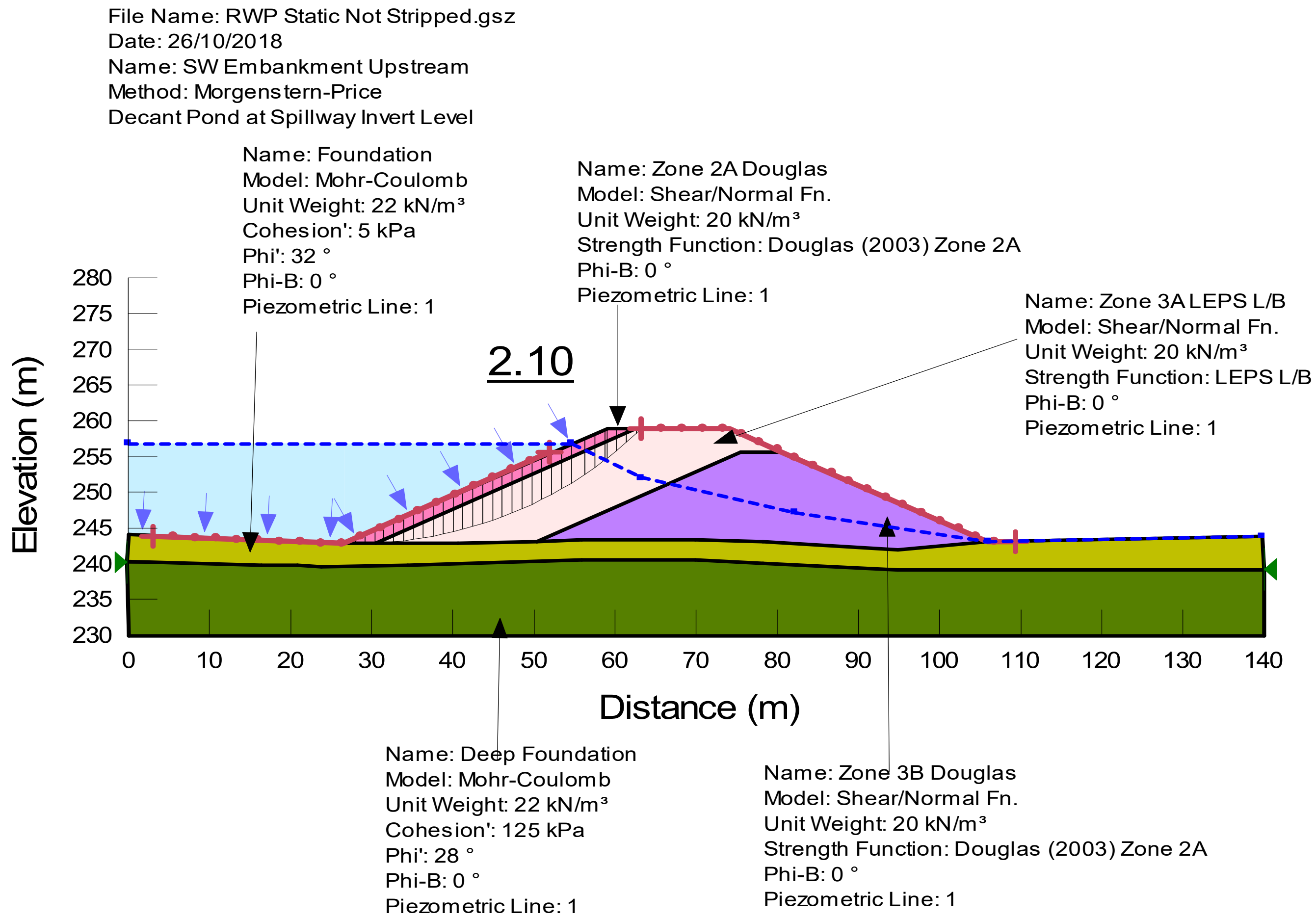
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP - (2:1 Side Slope) - Static - Pond at Spillway Invert Level Downstream

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE G11**



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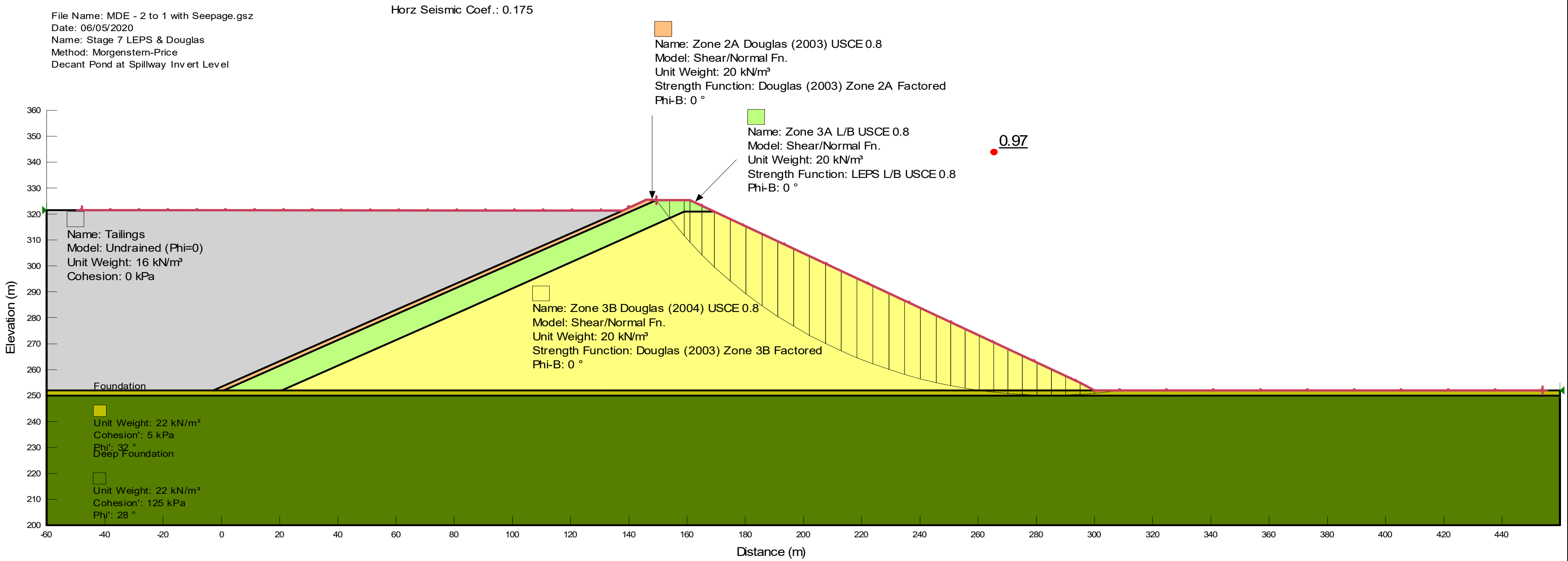
NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

RWP - (2:1 Side Slope) - Static - Pond at Spillway Invert Level Upstream

Date:	14/02/2019	Job No:	114185.14	FIGURE G12

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G12



Directory: K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Models\Final - LEPS & Douglas\

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NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

TSF Stage 7 - (2:1 D/S Slope) - MDE

Date:	6/05/2020	Job No:	114185.14	FIGURE G13

APPENDIX H

REPORT

IRON BRIDGE OPERATIONS PTY LTD

North Star Magnetite - Stage 2

Pilbara, Western Australia

Tailings Storage Facility Closure Design

Date: 12/12/2019

Project Report Number: 114185.15 R01

Document History and Status

Title:	Tailings Storage Facility Closure Plan
Job Number/Extension	114185.15
Document Number:	114185.15 R01
Project Office:	Perth
File Path:	G:\Synergy\Projects\114\114185 North Star Magnetite Stage 2\15 TSF Closure Design\Documents\R01\Text\114185.15 R01 Rev 1.docx
Author:	John Leavy
Reviewer:	Craig Noske
Job Manager:	John Leavy

Rev.	Status	Issued to	Issue Date	Signatures	
				Author	Reviewer
A	Draft Issue	IBO	25/02/19	JL	GB
0	Approvals issue	IBO	17/04/19	JL	CN
1	Revised Client Issue	IBO	12/12/19	JL	CN

This Revision distributed To:	Hard Copy	Electronic Copy
IBO		☑
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DN: cn=John Leavy, o=ATC Williams, ou=Perth, email=johnl@atcwilliams.com.au, c=AU
Date: 2019.12.11 10:04:07 +08'00'

JOHN LEAVY
Author

Digitally signed by Craig Noske
Reason: I have reviewed this document
Date: 2019.12.11 17:49:35+11'00'

CRAIG NOSKE
Reviewer

IMPORTANT NOTICE
Please refer to our Conditions of Investigation and Report

CONDITIONS OF INVESTIGATION AND REPORT

Conditions of Report

1. This report has been prepared by us for the purposes stated herein. We do not accept responsibility for the consequences of extrapolation, extension or transference of the findings and recommendations of this report to different sites, cases or conditions.

2. This report is based in part on information which was provided to us by the client and/or others and which is not under our control. We do not warrant or guarantee the accuracy of this information.

3. We believe the conclusions and recommendations contained herein were reasonable and appropriate at the time of issue of the report. However, the user is cautioned that fundamental input assumptions upon which this report is based may change with time. It is the user's responsibility to ensure that input assumptions remain valid.

4. This report must be read in its entirety. This notice constitutes an integral part of the report, and must be reproduced with every copy.

5. This report is prepared solely for the use of the person or company to whom it is addressed. No responsibility or liability to any third party is accepted for any damages howsoever arising out of the use of this report by any third party.

6. Unless specifically agreed otherwise in the contract of engagement, ATC Williams retains Intellectual Property Rights over the contents of this report. The client is granted a licence to use the report for the purposes for which it was commissioned.

Geotechnical Investigation

7. Geotechnical site investigation necessarily involves the investigation of the subsurface conditions at a site at a few isolated locations, and the interpretation and extrapolation of those conditions to elsewhere on the site not so investigated. This procedure has been adopted at the site that is the subject of this report and due care and skill has been applied in carrying out and reporting on the work. Thus the findings, conclusions and comments contained in this report represent professional estimates and opinions and are not to be read as facts unless the context makes it clear to the contrary. In general, statements of fact are confined to statements as to what was done and/or what was observed. Other statements have been based on professional judgement.

8. The scope of the work has been planned in the absence of any fore-knowledge of the site other than that stated in the report. Unless otherwise stated we consider that the number of locations investigated and the depths to which they have been investigated are reasonable bearing in mind the scale and nature of the project, and the defined purpose for which the investigation was undertaken.

9. We do not accept any responsibility for any variance between the interpreted and extrapolated conditions and those that are revealed by any means subsequently. Specific warning is also given that many factors, either natural or artificial, may render ground conditions different from those which pertained at the time of the investigation. Should there be revealed during the construction or at any other time any apparent difference from subsurface conditions described or assessed in this report, it is strongly recommended that such differences be brought to our attention so that its significance may be assessed and appropriate advice given.

EXECUTIVE SUMMARY

TSF Details

The North Star Stage 2 tailings storage facility (TSF) has been designed to store 664 Mt of fine tailings over a projected mine life of 20 years (with potential for an additional 4 years of storage allowed for in the design). The maximum throughput of tailings is 29 million tonnes per annum discharged at between 62% and 65% solids concentration.

The TSF is a valley type facility, whereby tailings are discharged from the head of a broad east west trending valley. During operations, a separate return water pond (RWP) area immediately downstream of the main embankment will collect tailings bleed water and excess rainfall run off for return to the processing plant. The RWP will be decommissioned and rehabilitated as part of the overall mine closure plan.

The TSF/RWP Site 2A valley has a large combined catchment of over 20 km^2, with no diversion drains proposed.

The proposed North Star pit is located immediately to the south and proposed waste rock dump immediately to the south east of the TSF. An existing pilot plant tailings storage facility (TSF 1) and Stage 1 plant site also lie immediately to the south of the proposed TSF. These will be decommissioned and removed as part of the operational mine development.

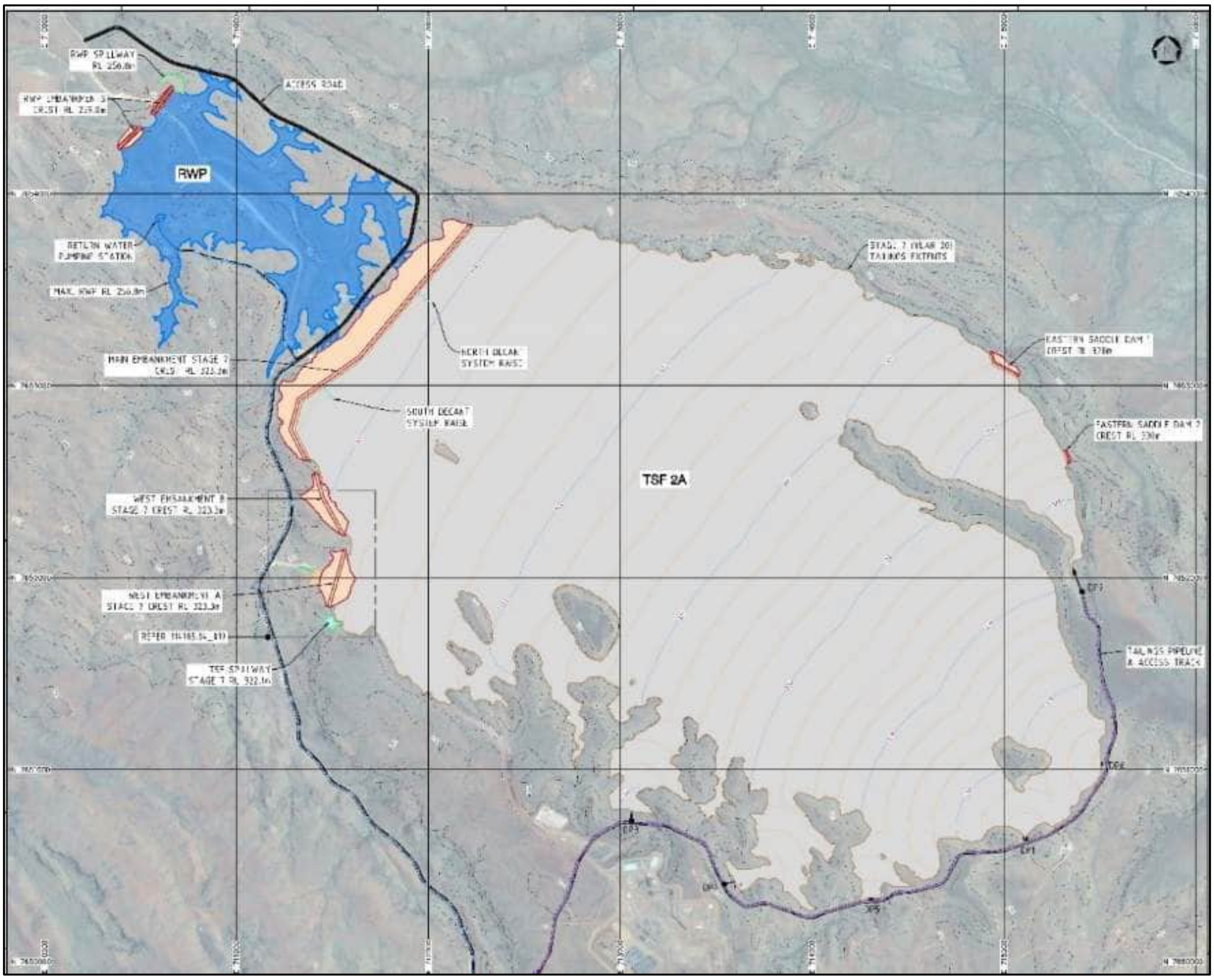
The surrounding land usage is pastoral.

The TSF failure consequence category adopted for design and operation, in accordance with ANCOLD guidelines **[21]** is **High C.**

At completion of tailings deposition, the tailings beach will have an area of approximately 1300 ha. Tailings will primarily be contained by the natural valley sides, which are up to 90 m high, a 68 m high main embankment and four saddle embankments constructed across topographic lows around the impoundment perimeter and the tailings pipeline access road formation which will fill the small gullies around the south-eastern side of the TSF.

The TSF arrangement prior to completion of closure works is illustrated on **Figure A**.

Figure A LOM TSF Arrangement



Note: 1:100 ARI, 72 hr RWP Pond Extent Shown

Closure Details

The primary objectives of TSF closure are to to safely decommission the infrastructure and rehabilitate the TSF such that it is safe, stable, non-contaminating, erosion-resistant and supports a self-sustaining ecosystem.

Several options have been considered to address these objectives and design analyses undertaken to validate these options, specifically with respect to the methodology associated with the formation of a closure cover on the tailings beach and re-shaping of the operational outer slopes of the engineered containment embankments.

The outcome of the options assessment and analyses is described below.

Closure for the RWP area will comprise draining of the residual pond, removal of the RWP embankments (for use in buttressing the TSF main embankment), grading of the RWP impoundment to remove any transported tailings residue on the ground surface and re-establishment of the drainage lines. The extent to which vegetation within the remainder of the RWP impoundment has been impacted during facility operation (during temporary flooding events) should be assessed prior to closure to develop a site-specific rehabilitation plan for this domain.

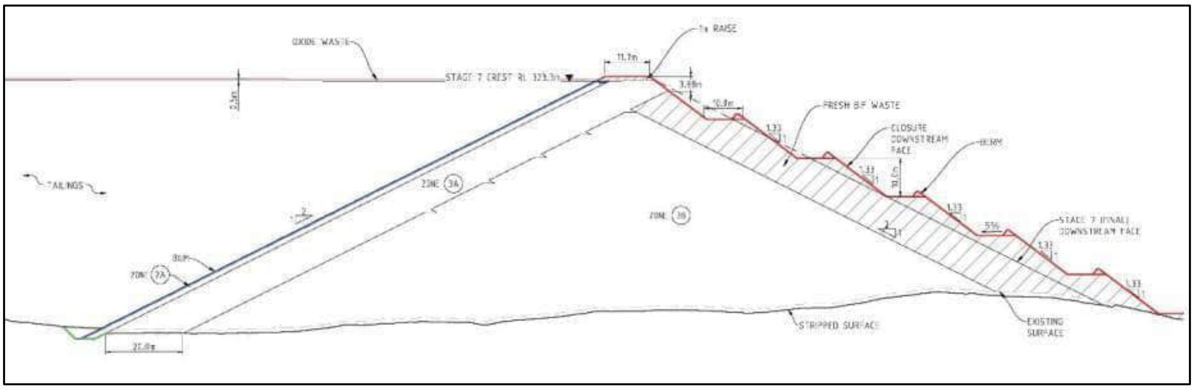
Re-establishment of vegetation in the drained operating pond area will be improved through the targeted application of topsoil.

The operational downstream slopes of the LOM TSF embankments are 1H:2V. In preparation for closure, the final operational stage raise will be constructed with highly durable, high permeability fresh BIF mine waste. The final surface will be regraded and buttressed with fresh BIF mine waste to form benched slopes at a nominal angle of repose of 37° (1H:1.33V) with bench height of 10 m and berm width of 10 m. This arrangement satisfies the criteria for acceptable rates of erosion associated with a high risk ranking, based on site-specific erosion modelling studies performed by Landloch **[11].**

The adoption of this embankment profile also allows the natural drainage line below the closure spillway to remain viable for transfer of external catchment run-off and spillway flows to the downstream environment.

A typical section through the re-profiled embankments is given on **Figure B**.

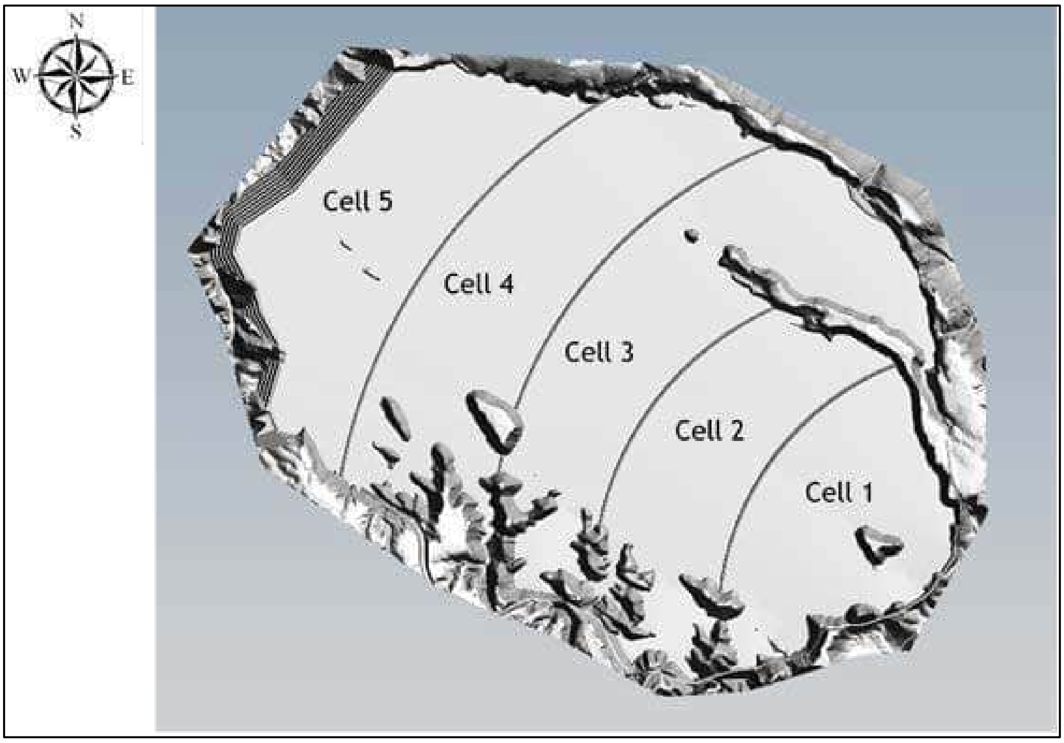
Figure B
TSF Embankments Closure Profile



The surface cover material to the tailings beach will comprise a 500 mm thick layer of oxide and fresh waste material from the Stage 2 mining operations. A series of four contour bunds will be constructed across the closure surface to attenuate excess rainfall run-off and create localised areas designated for topsoil spreading, fertilisation and establishment of vegetation.

The arrangement of the facility following closure is illustrated on **Figure C**.

Figure C
TSF Closure Arrangement



Risk Based Design

A risk-based approach has been implemented in the design development of the North Star TSF, whereby design features have been adopted that address recognised, credible failure modes for the key elements of the TSF embankments. These design features will be maintained post closure of the TSF landform.

Potential credible failure modes are overtopping, embankment stability and internal erosion and piping.

In order to address the potential for water overtopping of the TSF embankments, a high capacity, 100 m wide emergency closure spillway, together with conservatively adopted freeboard allowances for both flood height and wave action have been adopted, that will enable routing of Probable Maximum Flood (PMF) conditions across the landform surface.

Whilst surface contour bunding is incorporated into the design for the purpose of flood attenuation, sediment capture and to aid establishment of vegetation, the spillway has been designed to accommodate PMF flows without consideration of the reduced flow volumes and velocities provided by these features (i.e if the contour bunds were to be eroded over time, the risk of overtopping would still be mitigated).

To address the potential for tailings displacement over the operational TSF embankments associated with post seismic destabilisation of the adjacent saturated beach, the design provides for minimal sustained water ponding on the tailings surface during operation. For the post closure scenario, ponding on the closure cover above the lower beach would only occur for short periods following rare or extreme rainfall events and the underlying tailings will desaturate, such that the potential for liquefaction will diminish significantly.

Potential initiators for embankment instability generally comprise unidentified foundation weaknesses, inappropriate design criteria, construction flaws (material selection, compaction etc.), or sustained operation of the dam in contravention to the design intent.

To address these issues, the design has adopted the stripping of all weak alluvial materials from the embankment foundation to expose competent weathered sandstone and siltstone which were identified at shallow depth across the embankment alignments by geotechnical investigation.

The embankment materials will comprise rockfill and granular materials which will essentially be too coarse to liquefy, will be free-draining and have low risk of developing excess pore water pressures. Compaction specifications will be to a high standard to maximise material shear strength and minimise post construction settlements.

The risk of internal erosion or piping, commonly associated with earth embankments is significantly reduced since the TSF embankment does not have a low permeability core which could be subject to dispersion or erosion as a result of seepage forces. The embankment design incorporates compacted rockfill, an upstream compacted granular filter and an exposed geomembrane liner. The liner provides the primary water and tailings retaining element, with the filter forming the liner subgrade and providing protection against tailings migration into the rockfill due to a concentrated leak in the liner.

Possible failure mechanisms associated with internal erosion that were considered for the operational stage design include the development of a liner defect, earthquake-induced cracking of the embankment and associated liner rupture, or unidentified erosive foundation materials.

The compaction and quality control of the construction materials, coupled with the filter and lining system, were considered to reduce the risk of erosion progressing to an embankment breach to an acceptable level. For the post closure case where a sustained source of seepage is not present and the tailings are desaturating, the risk of instability associated with embankment seepage is considerably reduced compared to the operational case.

Design Analyses

Analyses conducted for the purpose of closure design validation included PMF flood routing, seepage analysis, slope stability and deformation analysis and revision of the operational stage water balance. The operational stage dam break analysis was also reviewed for the post closure scenario.

The outcomes of the flood routing modelling were to increase the width of the LOM spillway from 50 m to 100 m and to deepen the spillway by 1 m.

Seepage analyses indicated that full desaturation of the tailings beneath the former pond area would occur over approximately 25 years. Whilst a desiccated crust is expected to form rapidly on removal of the 400 mm deep decant pond, such that machine access can be gained, full desaturation of the upper part of this area is unlikely to be complete until approximately three years.

Seepage analyses were also conducted to assess the water holding and infiltration capacity of the embankment berms under PMF flood conditions. The outcomes indicate that due to the high infiltration capacity of the fresh BIF waste materials, overtopping of the berms would not occur, although provision is made for incorporation of low edge bunds to assist in the retention of short duration, high intensity PMF events.

The results of slope stability analyses indicated acceptable factors of safety for all scenarios considered. Associated analysis to assess the potential degree of seismically induced deformation indicated that embankment freeboard would not be comprised under this scenario.

The outcomes of water balance modelling for the closure scenario indicated that embankment overtopping will not occur and the closure spillway will rarely be required to operate (at reduced flow compared to PMF) with a maximum inferred spill frequency of less than 90 days over a 25 year period. Whilst the cross-landform contour bunds remain intact, spillway flow frequency would be less than 11 days over a 25 year period.

Review of the dam break analysis indicates that the potential impacts of dam break would be significantly reduced after closure, primarily since the RWP will not be present and the shear strength of the tailings will have increased as a result of consolidation and desaturation.

Material Balance

The available waste materials for use in closure earthworks will include stockpiled topsoil, oxide waste rock, fresh waste rock and coarse rejects material. For design, it is envisaged that the construction materials for embankment slope formation and tailings cover will comprise fresh BIF waste rock and oxide waste rock.

Based on information derived from geotechnical investigation in the TSF impoundment area and from mine schedule information provided by IBO, the anticipated quantities of materials available for use in closure cover placement and embankment re-profiling at LOM are given in **Table A**. These estimates are subject to ongoing review during development of the mine plan; however, it is considered that construction material requirements are insignificant in comparison to the expected volume of waste to be produced.

Table A
Material Balance

Material	Required	Available
Topsoil/Subsoil	407,000 m^3	407,000 m^3
Coarse Rejects	Blending option	111 Mm^3
Oxide Waste Rock	Surface Cover 4.97 Mm^3	@160 Mm^3
Fresh Waste Rock	Embankments 0.97 Mm^3	@160 Mm^3

A combined recoverable topsoil and subsoil depth of 150 mm is assumed within the valley floor areas which comprise approximately 30% of the total TSF impoundment area.

A bulking factor of 1.3 has been adopted to convert mined waste bcm to volumetric equivalents for embankment construction.

The available volume of re-useable embankment fill within the RWP embankments is 95,000 m^3.

The available volume of material resulting from formation of the closure spillway is 221,000 m^3.

Material Characterisation

Based on materials characterisation tests completed to date, the topsoil which will be recovered from the TSF area is acidic to neutral and marginally to strongly sodic with low salinity levels. Measured electrical conductivity (EC) is below the threshold level for plant growth (0.4 dS/m for topsoil and 1.2 dS/m for subsoil).

Total Nitrogen, available Potassium and available Phosphorous concentrations are generally anticipated to be high enough to support plant growth without significant fertilizer addition, although available Sulphur is generally very low. Addition of elemental Sulphur fertilisers would be beneficial to stimulate plant growth, although the potential for stimulating weed generation may outweigh the potential benefits.

The soils are generally deficient in Copper and Zinc, but have moderate to high concentrations of Iron, Manganese, Arsenic, Cadmium, Chromium and Cobalt.

The oxide waste is likely to be reasonably well graded, neutral, non-dispersive, non-saline and are considered as a potential foundation for growth medium. The material has low Total N values (although higher than some topsoil materials sampled to the east of the pit).

The fresh waste is expected to predominantly comprise cobble and boulder size material of extremely strong and extremely durable material (as supported by laboratory testing of available core). This material is suitable for long term erosion protection to embankments and design drainage pathways on the landform surface.

Closure Implementation

Due to the design intent to remove water from the tailings surface and the associated increase in shear strength as a result of beach development and evaporative drying, access to the tailings beach will be available prior to completion of operations. This will enable progressive closure of the TSF to commence from the beach head in advance of completion of ore processing. To facilitate this, the tailings discharge pipes will be advanced across the beach head or the deposition location can be transferred westwards around the impoundment perimeter.

Creation of the contour bunds across the tailings surface will provide stable access for construction equipment to place the oxide cover across the beach surface.

Within a period of less than three years following cessation of tailings deposition it is anticipated that access to the lower beach area (former operational pond) will be readily available.

Regrading of the embankment batters may be initiated during the final stage of operation or earlier than this if mine waste movements are scheduled to accelerate progressive closure.

Deepening and widening of the final stage spillway may take place once tailings deposition has ceased and the operational pond is no longer present. Similarly, removal of the RWP embankments, remediation of the RWP impoundment and decommissioning and grouting of the decant structures and transfer pipes will take place following cessation of deposition.

Following completion of implementation and subject to ongoing monitoring, it is considered that the closure works will satisfy the primary design objectives of creating a stable, non-polluting landform on which natural vegetation can be re-established.

TABLE OF CONTENTS

DRAWINGS

APPENDICES

1 INTRODUCTION

ATC Williams (ATCW) are pleased to present our report on the Closure Design of the Tailings Storage Facility 2A (TSF 2A) at Iron Bridge Operation's (IBO) North Star Magnetite Project. The scope of work for this report was outlined in our proposal reference 114185.15P01 Rev 0 dated June 2018.

Design of TSF 2A was completed by ATCW in November 2018 **[1]** in support of a Mining Proposal submission. The final design report completed following a peer review process was issued in April 2019 **[2]**.

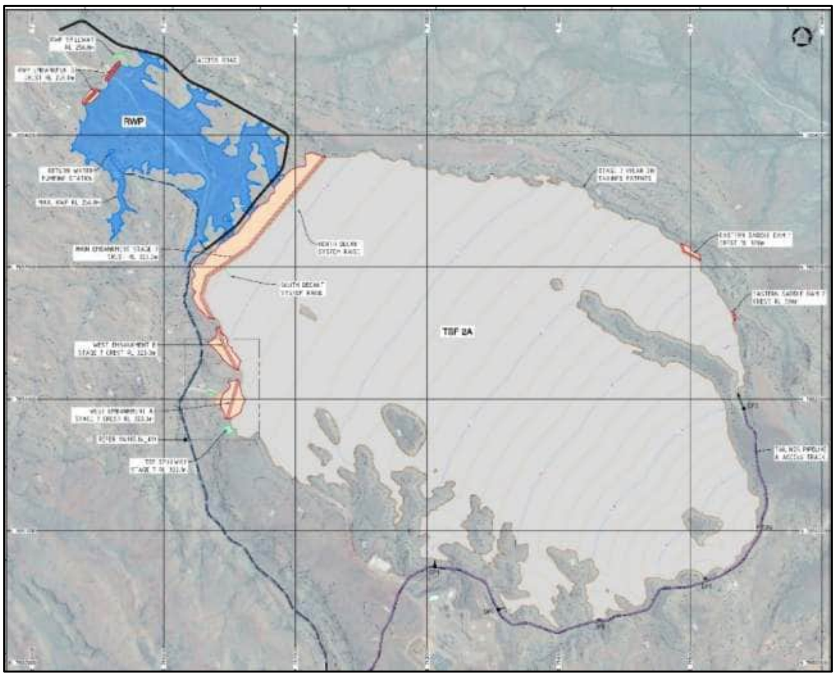
It is intended that this closure design report is read in conjunction with the final design report; consequently, duplication of information contained in the design report has been minimised where possible.

The facility will store wet tailings from processing of Magnetite ore and has a design life of 24 years. The TSF is a down valley type facility, whereby thickened tailings will be discharged from the head of a valley and will be retained by a downstream embankment. The main embankment and secondary saddle embankments will be progressively constructed during the operational life of the facility.

The design provides for gravity decanting of water accumulating against the main embankment, such that a large sustained water pond does not develop on the tailings surface. The decanted water will be stored in a separate downstream Return Water Pond (RWP), contained by low embankments.

The overall arrangement of the facility at the end of life of mine (LOM) and prior to closure is illustrated on **Figure 1** .

Figure 1
North Star TSF 2A LOM Arrangement



2 PROJECT BACKGROUND

The Project is being developed by Iron Bridge Operations Pty Ltd (IBO), and will use conventional open-cut mining and ore processing techniques to produce high-grade iron ore concentrate. A pilot plant has previously been constructed (Stage 1) and minor amounts of tailings produced, which were stored in a small tailings storage facility (TSF) designated TSF 1.

Mining will be undertaken within two open pits to provide sufficient ore for mill feed which will be treated at a process plant located approximately 5.0 km to the south of the proposed location of the Stage 2 tailings storage facility (TSF 2A).

Wet tailings production associated with Stage 2 development will be approximately 664 million tonnes at a maximum rate of 29 million tonnes per annum over a 24-year Life of Mine (LOM). The wet tailings will be thickened to between 62% and 65% solids concentration and pumped to TSF2A at 3,800 to 4,000 dry tonnes per hour (dtph);

ATCW completed an option study for the disposal of tailings from the proposed Stage 2 processing plant in 2014 **[3]**. Following a series of laboratory tests on tailings materials produced from metallurgical test work campaigns, a Feasibility Design report was completed in 2015 **[4].**

The feasibility study considered storage of "all-in" tailings with a P_{80} of 300 µm which comprised of both 'wet tailings' and 'coarse rejects' in a single stream of slurry pumped to TSF 2A, the storage requirement at this time was approximately 457 Mt of "all-in" tailings over a period of 20 years.

IBO issued a revised mass balance scenario upon completion of the feasibility study that determined the two tailings streams would remain separated at the process plant i.e. coarse rejects and wet tailings. The coarse rejects will be stockpiled and only wet tailings will be stored in the TSF.

In 2017, additional tailings laboratory test work was completed **[5]** on new sample material considered to be representative of the proposed wet tailings stream which had a P_{80} of 75 µm.

The final project mass balance provides for an overall wet tailings production of 664 Mt with the remainder of waste process material comprising <12 mm coarse HMS rejects which will be stockpiled in the vicinity of the Stage 2 process plant. The tailings will comprise low plasticity silt with a P_{80} of approximately 40 µm, similar in physical properties to one of the wet tailings samples tested during the feasibility study stage. The TSF has been designed for a 20-year life, with a design tailings tonnage of 560.2 Mt.

The tailings will be thickened to a target solids content of 65% at discharge. Settled in-situ dry density of 1.5 t/m^3 during the first 6 months of operation and 1.66 t/m^3 thereafter have been determined for TSF design purposes. The required TSF capacity is thus 338 Mm^3.

Geochemical characterisation test work on "oxide" tailings samples has indicated that the tailings will be non-acid forming (NAF).

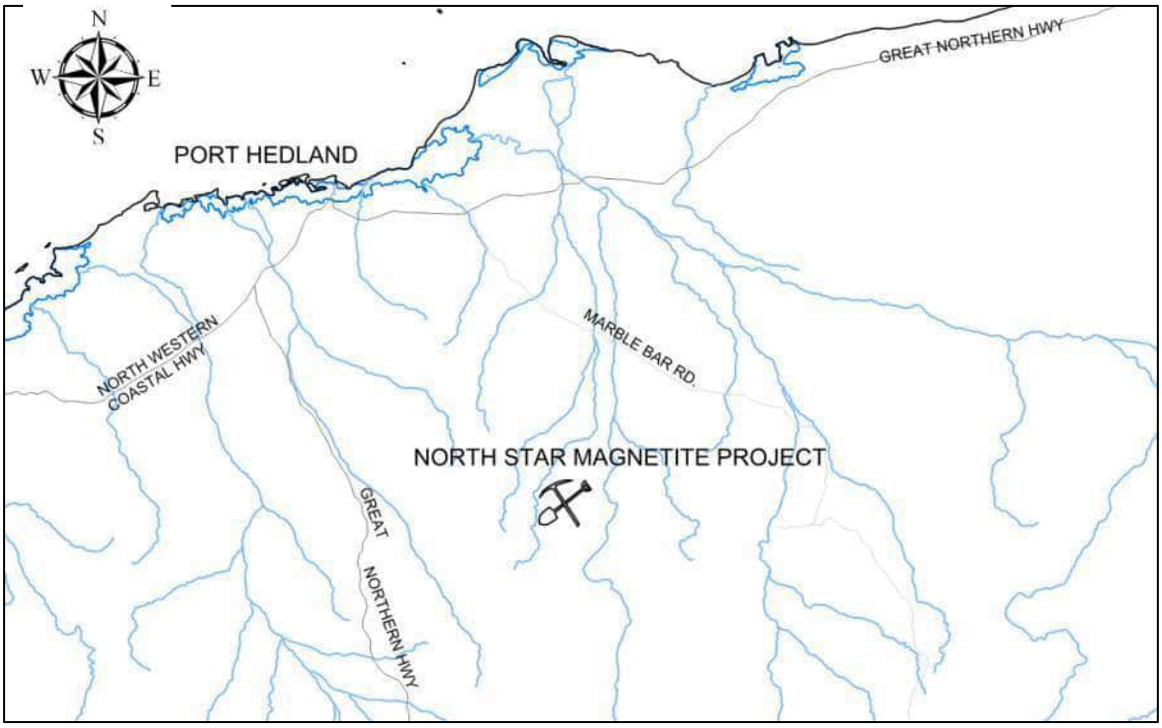
Geotechnical investigation for TSF design was completed between December 2017 and February 2018 **[6].** This supplemented previous geotechnical and hydrogeological investigations completed in 2011 by Golder Associates.

3 SITE CHARACTERISTICS

3.1 Site Location

The Project is located approximately 1,230 km north east of Perth and 110 km south east of Port Hedland in the Pilbara region of Western Australia as shown in **Map 3.1**.

Map 3.1
Project Location



The existing pilot plant tailings storage facility (TSF 1) and Stage 1 plant site lie immediately to the south of the proposed TSF 2A. The TSF 1 and Stage 1 plant have been decommissioned and will be removed as part of Stage 2 mine development; consequently, closure of TSF 1 is not considered in this design study.

The overall site layout is illustrated on **Drawing 114185.15_002**. The mining tenement on which these facilities are located is M45/1226.

3.2 Site Description

The TSF and RWP impoundments are situated in an east west trending, broad valley area confined by a narrow, banded iron formation (BIF) and silicified chert mesa landform approximately 90 m high on the northern side and a broad dissected ridge of sandstone of similar height on the southern side. Several sandstone hills generally between 10 m and 70 m high are also present within the overall valley area in the western part of the TSF impoundment.

The site is also delineated by the tenement boundary to the east, proposed North Star pits to the south and proposed waste rock dump to the south east.

The TSF site is the largest valley in the area of the mine tenement. The eastern part of the valley area is split into two sub-valleys by an elongated mesa ridge extending approximately 2 km in a north westerly direction from the eastern side of the TSF impoundment.

The TSF and RWP extents have been limited to within the current IBO North Star lease tenure. Containment will almost entirely be provided by the valley topography, with the only additional retaining features being the Main TSF embankment and the RWP embankments on the west side of the TSF, two small saddle dams on the north-eastern side of the TSF, and the tailings pipeline access road formation which will fill the small gullies around the south-eastern discharge side of the TSF.

The TSF/RWP Site 2A valley has a large combined catchment of over 20 km^2, with no diversion drains proposed. This catchment is however a benefit of the site, in that it can be used to collect and temporarily store runoff during the wet season for use in the process plant.

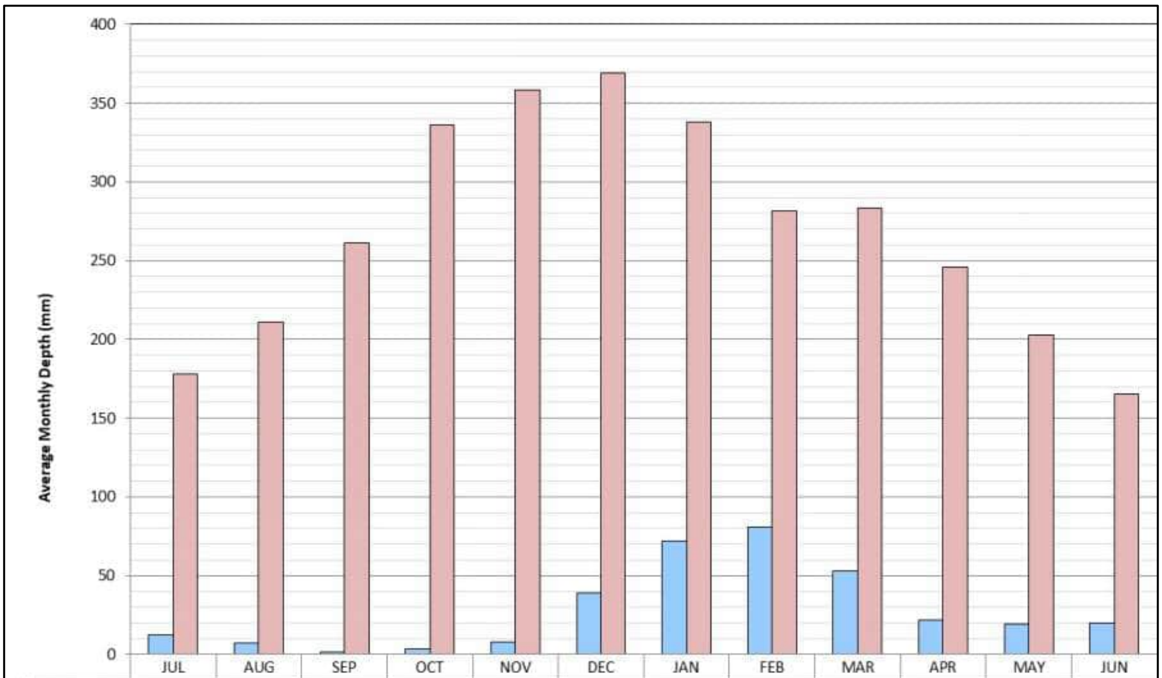
The surrounding land usage is pastoral.

3.3 Climate

The Pilbara region experiences an arid climate, characterised by hot, humid summers (wet season) and relatively cooler, drier winters (dry season). The site is within a region prone to remnant cyclonic activity, particularly between November and April.

Average annual rainfall for the project area is approximately 347 mm (Hillside station) and average annual evaporation is 3,300 mm (Marble Bar). Average monthly rainfall and evaporation totals are depicted in **Chart 3.1**.

Chart 3.1
Average Monthly Rainfall And Evaporation

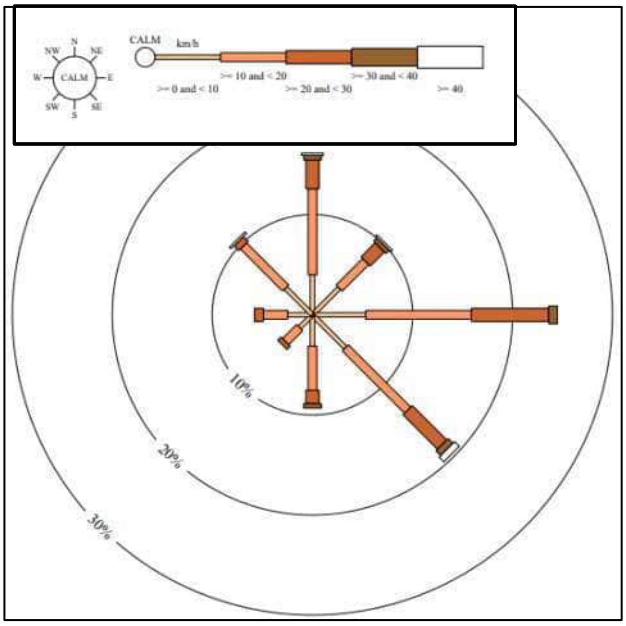


	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN
Rainfall	12	7	1	3	8	39	72	81	53	22	19	20
Evaporation	178	211	261	336	359	369	338	281	284	246	203	165

The highest wind speeds occur between September and December. Review of all months of wind rose data indicated that sustained (i.e. 10 minute) winds generally do not exceed 30 km/hr and very rarely exceed 40 km/hr. **Chart 3.2** presents the wind rose diagram for February, which is representative of the strongest winds.

Chart 3.2

Wind Rose Diagram For February (Marble Bar)



3.4 Topography

The project area is located within the George Range on an eroded peneplain with remnants forming plateaus (Mesas), hills, ridges and valleys within the landscape.

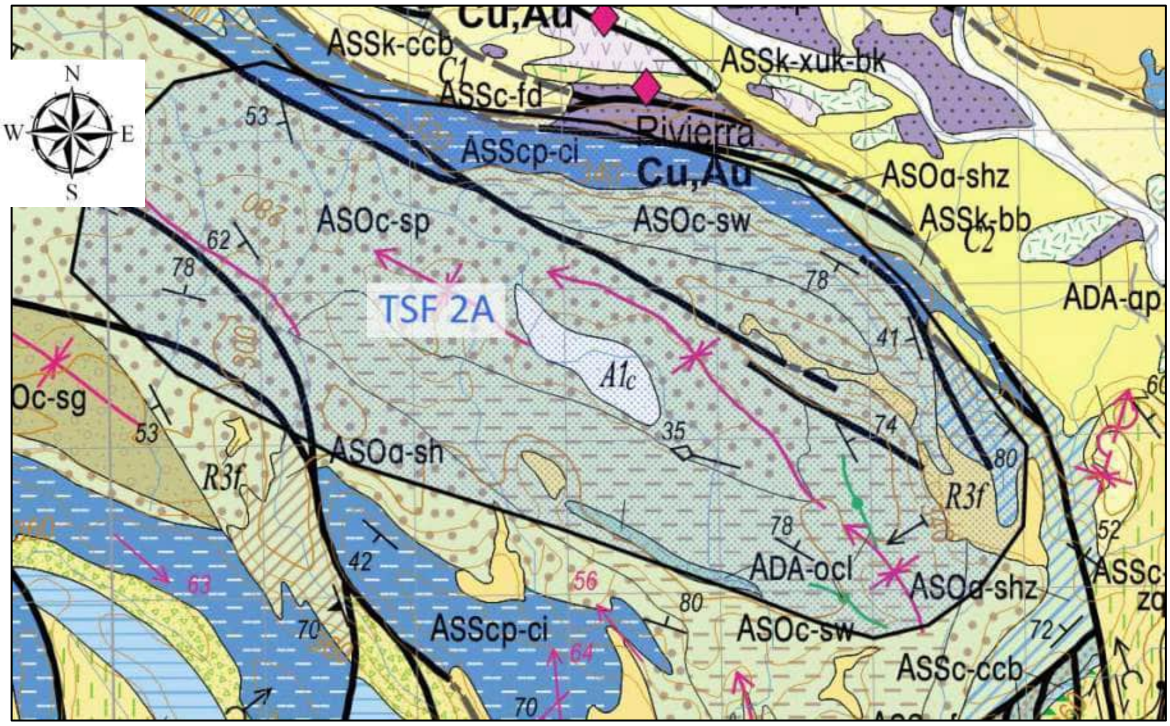
The valley within which TSF 2A is located generally grade to the west and north with approximately 1% gradient. The elevation in the area ranges between RL 270 m in the western part of the valley floor to RL 400 m on the surrounding plateaus. The existing regional and site topography is illustrated on **Drawing 114185.15_003**.

3.5 Geology

The project geology is illustrated on **Map 3.2**. [7].

The Proterozoic Soanesville meta - sedimentary group is dominant in the TSF area and is represented by rocks of the Corboy and Cardinal formations. The Corboy formation rocks (ASOc) comprise sandstones, wackes (clayey sandstone) and lithic arenites (sandstone with > 50% included rock fragments, typically of fine and medium gravel size). Local interbeds of conglomerate, siltstone and shale are also present. The Cardinal formation (ASOa) predominantly comprises ferruginised black shale with local siltstone, banded iron formations (BIF), sandstone and conglomerate. Silicified shales are also present at the eastern edge of the TSF.

Map **3.2**
North Shaw Regional Geology [7]



Rocks of the older Sulphur Springs group are localised, poorly represented and generally confined to the perimeter of the TSF area. They include components of the Kangaroo caves formation (ASSc) including the Pincuna BIF member, layered cherts, dacite and rhyolite volcanics and also Basalts of the Kunagunarrina formation (ASSk).

The rock units have an overall east -west strike and generally dip at about 70° to the North. They have been subject to low grade metamorphism by compression in an east – west direction and contacts between sandstone and interbedded shale units are frequently sinuous. Small scale, tight isoclinal folding is frequently observed in the relatively deformable shale units.

The sandstone outcrops are blocky in nature with typical block dimensions of a few hundred millimetres. The rock mass is discoloured red brown on the surface; however, this is superficial in nature, being a few millimetres thick. Breaking of blocks with sturdy blows of a geological hammer reveals grey, relatively fresh rock material of moderate to high strength beneath.

The shales are thinly bedded, typically with alternating purple grey and pale-yellow grey bands. Contacts between individual beds are tight. The rock is typically of low and medium strength.

Superficial deposits are restricted in depth and lateral extent, generally being confined to the valley floor area and the drainage channels that meander westwards between the sandstone hills.

3.6 Soils

The classification of the soils at the North Star project was assessed by GHD in 2011 and is described in the Soil and Landform report prepared by FMG **[8]**. The soils are inferred to be associated with the Capricorn land system as mapped by Van Vreeswyk *et al* **[9]**.

The Capricorn System is typically represented by:

Ridges, hills and upper slopes – Stony soils
Lower footslopes – Red shallow loams
Stony plains – Red shallow sands and red shallow loams
Narrow drainage floors and channels – River bed soils

Within the system, topsoils are commonly shallow (~5cm), are covered in a stony mantle and contain a high percentage of coarse fragments. Subsoils are usually absent; although, where they are present, they contain a high percentage of hard, weathered and unweathered rock.

The soils have been classified predominantly as tenosols or rudosols. Tenosols have weak pedological development and tend to capture a wide range of soils that do not fit the requirements of other soil orders within the Australian Soil Classification Scheme **[10]**. Similarly, rudosols have little pedological development though they tend to be younger in terms of their development that tenosols.

Based on the outcomes of the GHD assessment and subsequent soil tests by Landloch **[11]** and ATCW **[6]**, the key characteristics of these soils are:

- Soil pH is acidic to neutral in some areas and alkaline in others and tend to increase in pH with depth. Measured pH values generally range from 5.7 to 8.7 for topsoils.

- Salinity levels in the topsoils and subsoils are generally low, with measured electrical conductivity (EC) below the threshold level for plant growth (0.4 dS/m for topsoil and 1.2 dS/m for subsoil).

- Total N values measured in topsoil samples recovered by ATCW ranged from 150 mg/kg to 530 mg/kg. Landloch consider total N values >300mg/kg may be used to define materials with suitable Total N values to support plant growth without significant fertilizer addition. Nitrite and Nitrate (NOx) concentrations were low (<8 mg/kg) with occasional exceptions (18 mg/kg and 23 mg/kg).

- Available Potassium and Phosphorous concentrations are generally within ranges that could support plant growth, with limited fertilizer addition. Available Sulphur is generally very low and addition of elemental Sulphur fertilisers would be beneficial to stimulate plant growth.

- Measured exchangeable sodium percentage (ESP) values variably classify the soils as marginally to strongly sodic.

- Soils are generally deficient in Copper and Zinc, but have moderate to high Iron and Manganese levels.

- Heavy metal concentrations are moderate to high for Arsenic, Cadmium, Chromium and Cobalt.

3.7 Hydrology

Drainage lines in the region are ephemeral and generally only flow for short durations following rainfall events. Intermittent flows normally occur during the wet season with long periods of no flow during the dry season.

The mine area lies on the catchment boundary of the Turner River and Strelley River. The Turner River has a catchment of 4,802 km^2 and is a major river of the Port Hedland coast catchment while the Strelley River has a catchment area of 2,805 km^2 and is a sub-catchment of the Shaw River **[12]**.

Numerous ephemeral drainages are present across the site. The main drainage lines in the TSF 2A area are:

- Lost Boys Creek to the north of the proposed mine pit, within the area required for the TSF, which ultimately flows into the Turner River via Chinnamon Creek; and
- An unnamed creek which roughly parallels the mine access road and flows into the Turner River just south of Pincunah Waterhole.

Several small drainages on the eastern edge of the mine area flow into Six Mile Creek which ultimately flows into East Strelley River.

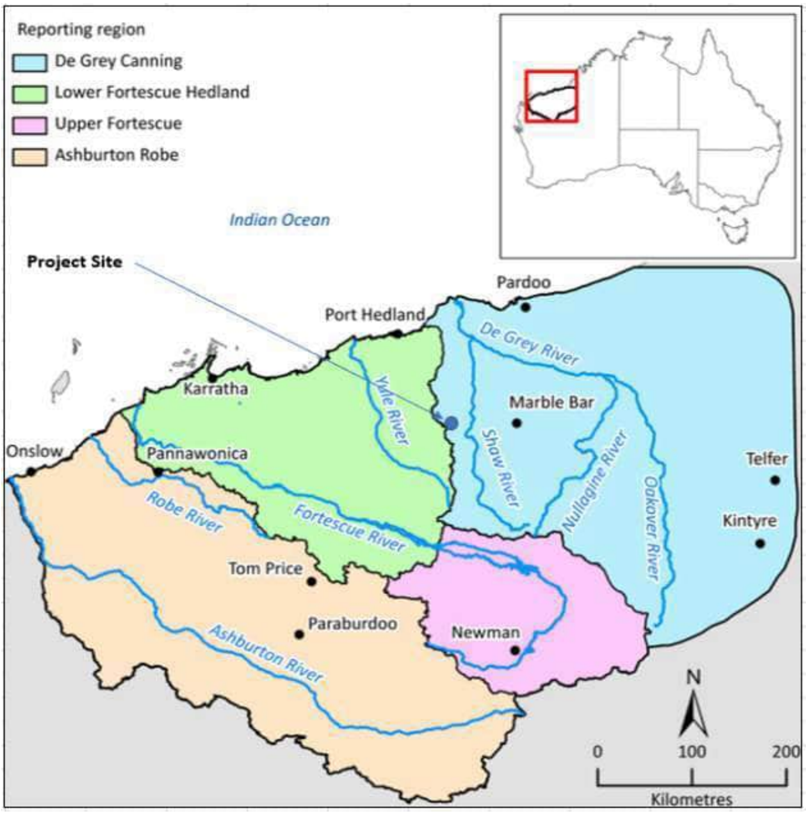
3.8 Hydrogeology

CSIRO has conducted an overview of the current and future climate and water resources of the Pilbara for the Government of Western Australia and industry partners **[13]**. They have assessed surface water and groundwater resources and their environmental significance in four regions referred to as: Ashburton Robe, Upper Fortescue, Lower Fortescue Hedland and De Grey Canning.

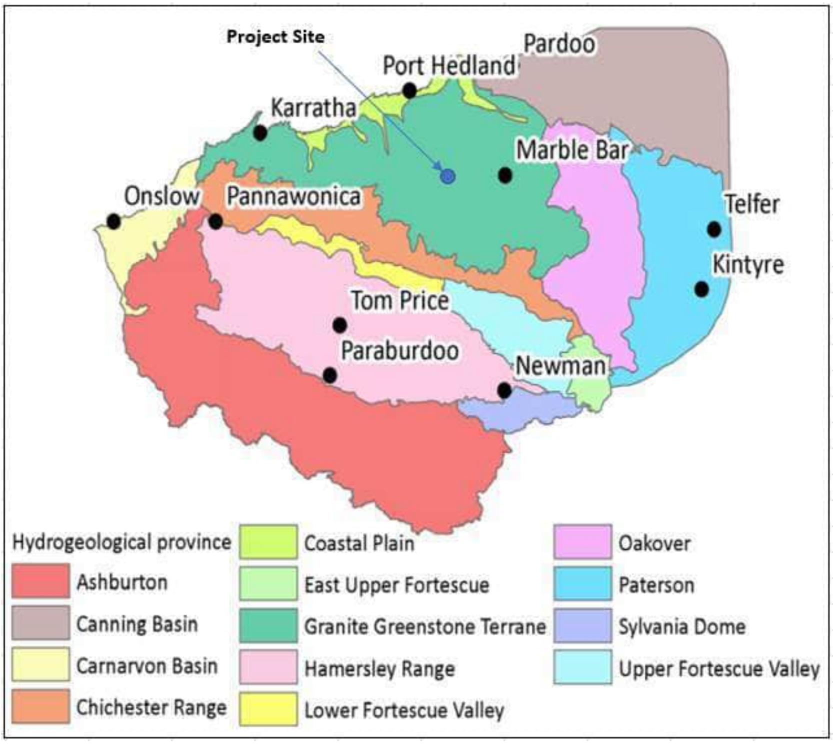
Map 3.3 shows the regions assessed by CSIRO, with the Project site being located within the De Grey Canning region.

The site is also located within the granite greenstone terrane hydrological region, as shown in **Map 3.4**. Groundwater resources of this province are limited to alluvial aquifers and fracture zones in greenstones.

Map 3.3
Pilbara Water Resource Assessment Regions - CSIRO

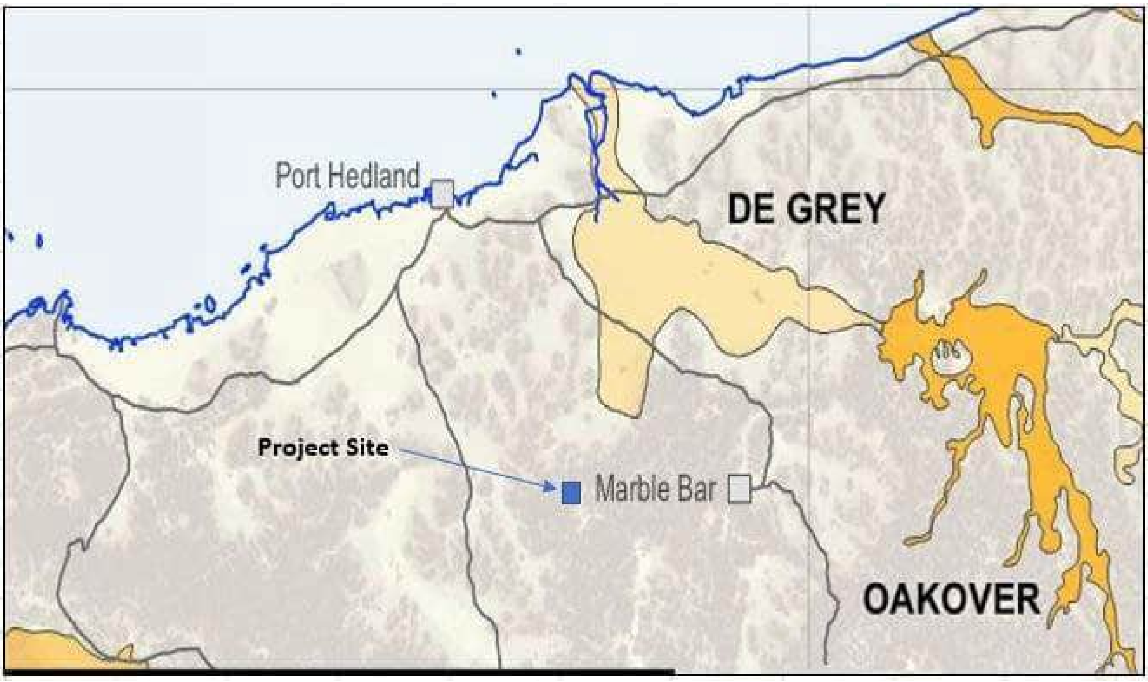


Map 3.4
Pilbara Hydrogeological Provinces - CSIRO



Map 3.5 shows the interpreted distribution of palaeovalleys in arid and semi-arid parts of Western Australia, South Australia and the Northern Territory (or “WASANT”), published by the National Waters Commission in 2012 **[14]**. The coloured areas are the palaeovalleys, and it is apparent that the Project site is not located within any of the identified palaeovalleys.

Map 3.5
Location Of Site Within The WASANT Palaeovalley Map



Within the project area, unconsolidated alluvial deposits represent a shallow alluvial aquifer to the north-east of the ore body (including the TSF area). A weathered zone of basement rock and a south west - north east trending fault present possible sources of groundwater at depth **[15]**.

Based on previous **[16]** and recent geotechnical investigations **[6]**, the inferred groundwater elevation ranges from RL 241.7 m to RL 270.4 m in the TSF 2A site area, with an inferred hydraulic gradient of approximately 1:100 towards the north west.

Groundwater was encountered in all the geotechnical investigation boreholes in the valley floor area at depths ranging of 2.3 m to 6.6 m. At the location of the RWP embankments, groundwater is close to the creek bed level and seasonally daylights in small ephemeral pools on the outer bends of the creek channel (**Photo 1**).

Comparison of groundwater monitoring bore water levels in the closest monitoring bore (5A) between July 2018 (when a pool was visible) and October 2018 indicated a fall in elevation of 600 mm during this period.

Photo 1
Ephemeral Pool At RWP Embankment Location

3.9 Seismicity

A probabilistic seismic hazard assessment (PSHA) of the region was conducted in 2012 by TetraTech for the nearby Fortescue Metals Group (FMG) Solomon Operation **[17]** with all identified faults added to the seismotectonic model. The Solomon Review presents an ARI versus ground motion relationship determined for the seismotectonic province in which the North Star project is also located.

Subsequently, site specific PSHAs was undertaken by ATCW for the various FMG Pilbara operations including the North Star (Iron Bridge) operation **[18]**.

The horizontal peak ground acceleration (PGA) inferred for the operational basis earthquake (OBE) and Safety Evaluation Earthquake (SEE) are presented in **Table 3.1**, where 'g' refers to the acceleration due to gravity. The AEPs selected for assessment of OBE and MDE are based on the ANCOLD dam failure consequence category of "High C" assigned to the facility (**Section 4.1.2**). For closure purposes, it is also necessary to consider the maximum credible earthquake (MCE) which has been taken as the probabilistic ground motion from a 1:10,000 AEP event.

Table 3.1
Summary of Seismic Design Criteria

Design Event	OBE	SEE	MCE
Annual Exceedance Probability (AEP)	1 in 475	1 in 2,000	N/A
Peak Ground Acceleration (PGA)	0.05 g	0.13 g	0.35 g

3.10 Vegetation

Mapping of the vegetation of the Pilbara region **[19]** indicates the principal vegetation associations in the area to comprise:

- Vegetation Association 82: Hummock grasslands, low tree steppe, snappy gum over *Triodia wiseana*; and
- Vegetation Association 93: Hummock grasslands, shrub steppe, kanji over soft spinifex.

Project specific vegetation and flora surveys conducted in 2015 By Ecologia Environment **[20]** concluded that the records of Pityrodia sp. Marble Bar are found overwhelmingly within the Capricorn land system, with only six records (from all surveys) recorded in any other land system (these six records are all within the Rocklea land system).

Several potentially important ecological observations were also made during the survey, including observations of potential pollinators (jewel beetles and feral honey bees) visiting Pityrodia sp. Marble Bar flowers, Marble Bar individuals reshooting after fire, with prolific flowering/fruiting following fire and evidence of recruitment observed only in areas recently burnt. Some individuals which appeared to be less than one year old were also observed flowering. Therefore, an increase in fire frequency is not expected to be detrimental to the Pityrodia sp. Marble Bar population.

4 TSF DESIGN DETAILS

4.1.1 Summary

The project involves construction of cross valley embankments to form an impoundment within a wide valley. The largest embankment will be 68 m high at the final construction stage (20 years LOM) and approximately 2.5 km in length. Embankments will be constructed in 7 stages using downstream methods.

Each stage will include emergency spillways constructed in cut through natural rock exposure along the alignment of the final embankment. The interim stage spillways will be infilled as part of the construction works for the subsequent stage embankment raise.

Ultimately the TSF retaining structures will comprise a main embankment at the north-western end of the TSF (68 m high); two western embankments; and two eastern saddle embankments.

The TSF concept will involve down-valley discharge on a rotational basis from three primary discharge points at the head of tributary creek valleys at the eastern end of the TSF area. Supplementary discharge points will be provided from the delivery pipe corridor around the south-eastern side of the valley to infill local creek valleys. Tailings will flow down existing drainage lines and accumulate against the main cross-valley TSF embankment.

All decant and runoff water collecting on the tailings beach will be conveyed via two gravity decant structures constructed on the upstream face of the main TSF embankment to the RWP directly downstream. The TSF decant system comprises two decant inlet chutes, constructed of upturned precast concrete box culvert sections, with a reinforced concrete base slab. The decant chutes discharge to reinforced concrete-encased steel, 700 mm diameter outlet pipes through the base of the embankment, which transfer the decant water downstream to the RWP. Emergency shut-off valves will be installed at the outlet of each pipe.

The purpose of the TSF embankment is to retain tailings, with practically all bleed and excess rainfall run-off being transferred to the downstream RWP area. A nominal sustained TSF operating pond depth of 400 mm represents the height of water required to effectively spill into the decant structures without carrying disturbed tailings material.

Confinement for accumulated water in the RWP will be achieved by the construction of two small embankments (16 m and 10 m high respectively) either side of a small sandstone hill approximately 1.5 km m downstream of the TSF embankment. These embankments will be constructed to full height during the initial construction works.

All the embankments will be founded on competent rock, with minimal requirement for removal of superficial deposits in alluvial channel areas. The TSF embankment and the smaller northern RWP embankment will adopt a conventional upstream key trench to manage seepage. The larger southern RWP embankment will require a grout curtain to a nominal depth of 20 m to manage seepage.

As there is insufficient clayey material available in the project area, the embankments will be constructed of rockfill with an upstream filter and cushion layer of select sand and gravel size material (coarse rejects) upon which a thin, impermeable Bituminous Geomembrane (BGM) will be placed. The subgrade for the BGM Liner has been designed to act as a filter for the tailings, such that any ingress of tailings fines through liner defects which could potentially migrate further through the embankment will be minimised.

The initial embankments will be primarily constructed of competent sandstone rockfill sourced from hills within the TSF impoundment area. Subsequent construction stages will utilise an increasing proportion of mine waste rock as access to the TSF impoundment for quarrying becomes restricted due to accumulation of tailings.

The embankment zoning is:

BGM Bituminous Geomembrane Liner
2A Upstream BGM Support Layer (coarse rejects, < 25mm particle size)
3A Transition Rockfill (coarse rejects, waste rock, quarries, < 400 mm particle size)
3B General Rockfill (waste rock, designated excavations, quarries < 800 mm particle size)

A summary of major earthworks and BGM liner quantities estimated over the seven embankment construction and tailings deposition stages is presented in **Table 4.1**.

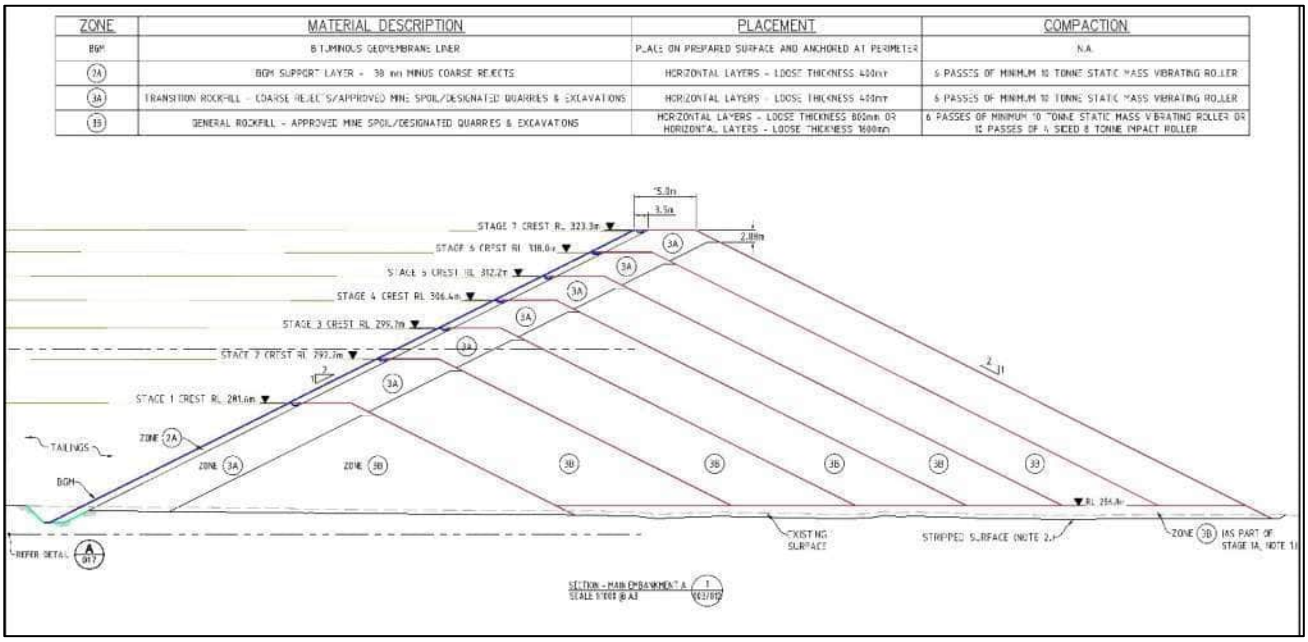
Table 4.1
Summary of Embankment Fill and BGM Liner Quantities

Storage	Stage	Zone 2A m^3	Zone 3A m^3	Zone 3B m^3	Stage Volume m^3	BGM Liner m^2
RWP	**1A**	13,500	22,000	60,000	95,000	10,000
TSF	**1A**	50,000	240,000	440,000	730,000	30,500
	1B	16,000	90,000	300,000	406,000	12,000
	2	50,000	270,000	685,000	1,005,000	38,000
	3	45,000	230,000	970,000	1,245,000	33,000
	4	45,000	235,000	1,215,000	1,495,000	33,100

Storage	Stage	Zone 2A m^3	Zone 3A m^3	Zone 3B m^3	Stage Volume m^3	BGM Liner m^2
	5	45,000	240,000	1,420,000	1,705,000	34,500
	6	45,000	245,000	1,710,000	2,000,000	35,500
	7	45,000	235,000	1,950,000	2,230,000	34,000
Total		354,500	1,807,000	8,750,000	10,911,000	260,600

A design section through the operational main TSF embankment is illustrated in **Figure 2.**

Figure 2
North Star TSF 2A LOM Section



The return water pumping system will consist of a pontoon-mounted pumping station located within a channel excavated to access the central part of the RWP area from its southern perimeter.

The facility has a large catchment of over 20 km^2 comprised of 13.5 km^2 in the tailings storage area and 7.1 km2 in the water storage area. Bleed water from the tailings and excess rainfall run-off from both sub-catchments will be captured behind the RWP for use as return water to the process plant.

For design purposes, a tailings beach slope of 0.4% has been determined for the upper (eastern) part of the tailings beach, decreasing to 0.3% and 0.23% in the middle to lower parts of the beach and to 0.17% at the toe of the beach close to the main embankment.

4.1.2 Facility Classification

The North Star TSF2A and RWP embankment failure consequence category adopted for design and operation, in accordance with ANCOLD guidelines **[21] (**given a PAR > 1 to 10 and a "Major" severity of damage or loss) is **High C.**

The environmental spill consequence category as a result of water release is generally lower than the embankment failure consequence category, particularly if no loss of life is expected and water spilling from the facility is controlled by spillways.

For this project, the decant water is not toxic or likely to become acidic and the impact on the natural environment as a result of unseasonal flow along the Turner River channel is expected to be "Minor ". Hence an environmental spill consequence category of **Low** is considered appropriate for the North Star TSF2A and RWP facility.

In accordance with DMIRS guidelines **[22]**, TSF 2A is considered a **Category 1** facility by virtue of having a final embankment height > 15 m (irrespective of adopted hazard rating).

4.1.3 Geotechnical Investigations

Geotechnical investigations at the TSF 2A site have been completed by Golder Associates **[23]** and ATCW **[6]**.

Beneath discontinuous superficial cover, highly to moderately weathered sandstone or interbedded siltstones and shales were identified at the main TSF embankment. Slightly weathered to fresh sandstone was typically intersected in boreholes at depths of around 17 m.

At the RWP embankment location, moderately weathered fine to coarse grained sandstones of high strength are the dominant rock type in the south and central abutment areas. This rock type forms the low hills throughout the RWP and TSF impoundment areas. Moderately weathered shales and siltstones of medium strength are prevalent at the main TSF embankment and in most of the valley floor areas in the impoundments.

At the main and intermediate hill abutments, a thin (0.1 m) horizon of topsoil overlies up to 2 m of colluvium, which in turn overlies the weathered rock profile. Colluvial deposits typically consist of dense clayey gravels and sandy gravels, with frequent cobbles and boulders.

It is considered that foundation stripping to a typical depth of 0.5 m will generally reveal a competent foundation profile, providing a satisfactory foundation for the TSF and RWP embankments.

In elevated areas outside of creeks and valley floors, where superficial deposits are thin or absent, it is expected that a stripping depth of 0.2 m will reveal competent foundation materials.

In creek and valley floor areas, sandy topsoil of up to 0.4 m thickness overlies alluvial deposits, which extended to a typical depth of less than 2 m in the creek bed through the RWP South embankment. Alluvial deposits in this area consisted of dense silty sandy gravels with cobbles.

Colluvial deposits are present on the lower slopes of the embankment abutments, with a maximum thickness of approximately 2 m at the base of the valley sides. The colluvium overlies the weathered rock profile, and is typically composed of clayey gravels and sandy gravels.

The thinly banded shale units identified in the vicinity of the embankments are generally of moderate strength and are not susceptible to slaking when wet.

The sandstone rock mass found in the numerous hills within the western part of the TSF impoundment is generally of medium to high strength and durable.

Based on insitu testing, the near surface and exposed sandstone rock mass is considered highly permeable whereas the shale and siltstone units are of low to moderate permeability.

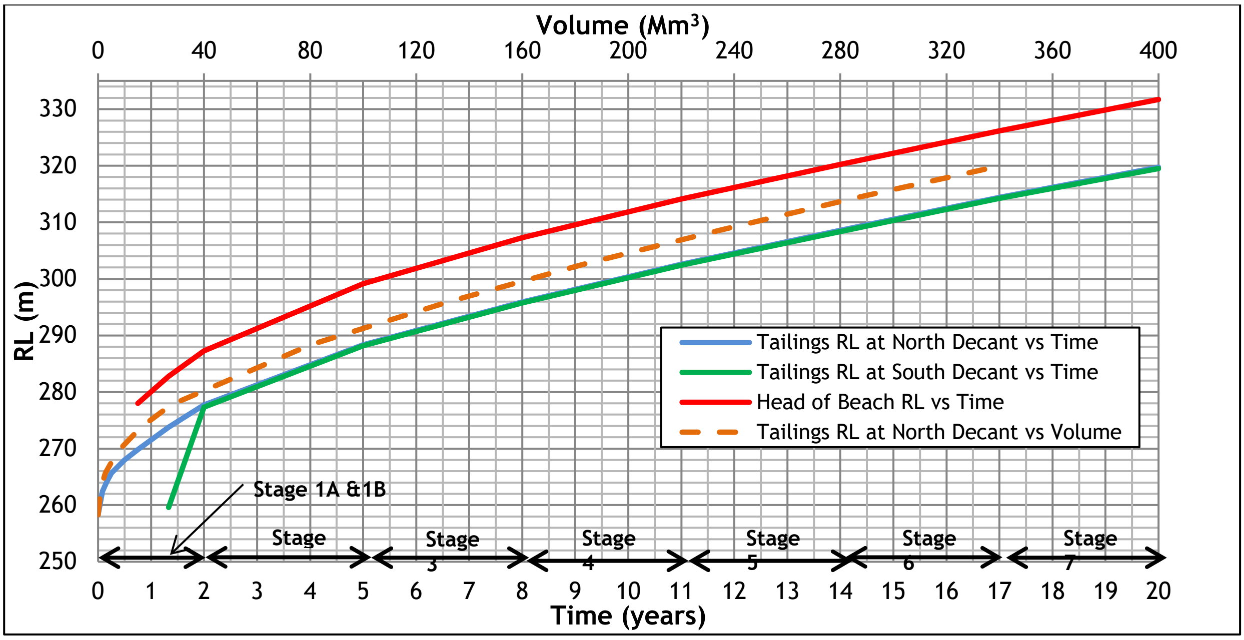
4.1.4 TSF Filling

It is expected that the base of the TSF valleys will fill quickly on initial deposition and the beach toe (at the TSF main embankment) will rise moderately quickly during Stage 1 (average 9.7 m per year). However, due to the topography of the valley, the rate will reduce markedly, to an average of 2.8 m per year from Stage 2 to Stage 4.

The tailings level is predicted to rise, on average, at a rate of 1.9 m per year from Stage 5 to Stage 7 (end of filling). Rates of rise of this order are considered beneficial with respect to achieving optimum density in the deposited tailings prior to closure.

The projected filling curve for TSF 2A is illustrated on **Chart 4.1**.

Chart 4.1
TSF Filling Vs Time Relationship



4.1.5 Downstream Staging Benches

The TSF embankment will be raised in the downstream direction and will thus require downstream benches to be constructed in channel areas at strategic stages of embankment raises to protect subsequent embankment stage foundations from inundation in the event of temporary expansion of the RWP pond due to extreme rainfall events.

The elevation of the staging benches has been set at the RWP maximum water level (RL 256.8 m). The timing of bench construction will be based on when the downstream toe of a particular stage encroaches upon the RWP maximum water level. Staging benches will be included in the construction of Stages 1A, 1B and 4 of the TSF embankment raises.

5 CLOSURE DESIGN CONSIDERATIONS

5.1 Post-Mining Land Use

The project has been operational since 2015, operating a pilot plant, during which a robust stakeholder consultation process has been established. There are currently no known items or sites of significant state, national or aboriginal heritage impacted by the proposed TSF.

The surrounding land use is pastoral, and the final post closure land use objective will be to return the site back to pastoral land.

5.2 Objectives

The general objectives for closure and rehabilitation of the TSF are as follows:

- Protect public health, safety and property;
- Ensure long term physical stability of the facility;
- Ensure long term chemical stability of the facility;
- Design for a sustainable ecosystem and land use;
- Employ rehabilitation methods that are technically effective and cost efficient; and
- Implement standard and proven engineering practices to minimise on-going maintenance.

DMIRS guidelines **[24]** recommend the following process for TSF closure design:

- Decommissioning: to be commenced near or at the cessation of deposition;
- Rehabilitation: to progressively return the disturbed land to a self-sustaining condition; and
- Performance monitoring: to demonstrate the proposed closure design outcomes are achieved.

5.3 Materials Characterisation

5.3.1 Topsoil

Topsoil within the TSF impoundment is severely limited in lateral extent due to the predominance of rock outcrops in the sandstone hills and throughout the lower valley area.

Data from test pits within the impoundment conducted during the geotechnical investigations suggests that maximum topsoil thickness in the valley area is approximately 0.2 m. The material typically comprised dense, pale brown, red brown and dark grey silty sand and was generally directly underlain by shale or sandstone except adjacent to drainage lines where alluvial deposits of sand or clayey sand were locally encountered. Colluvial clayey gravels or gravel /sand mixtures were identified in places near the base of the hills. These superficial deposits were rarely greater than 1 m thickness.

Sixteen samples of topsoil were collected by ATCW and analysed for a range of nutrients, inorganic analytes, cation exchange capacity, and available P, S in general accordance with the DMIRS draft guidelines on materials characterisation **[25]**. The results are given in **Appendix A**.

The topsoil materials have similar particle size distribution to the underlying subsoils. Based on analyses of these materials, the proportion of aggregates > 0.85 mm is generally > 30% where gravelly material is present in the subsoil, giving a low wind erodibility rating. Where less gravelly subsoil is present, the proportion of aggregates > 0.85 mm is between 20% and 30%, giving a moderate wind erodibility rating.

The WA soil group is classified according to Schoknecht and Pathan **[26]** and has been assigned as Soil Group 203 Stony Soil with a qualifier of LMM (Loamy Matrix). This is based on the relatively neutral pH values and description of the topsoil as either silty gravel or silty sand. Soils within this category are considered to have a moderate inherent fertility level.

The salinity of the topsoil samples is interpreted to be mostly low based on electrical conductivity results and thus the material is suitable as a growth medium. An exception is test pit TSF-TP2A-39 a higher salinity level classes the material as being suitable for some salt tolerant species.

Based on the measured Exchangeable Sodium Percentage (ESP), the majority of test pits possess topsoil that would fall within the marginally sodic to sodic category (ESP 6 -14), while three test pits (TSF-TP2A-06, TSF-TP2A-39, TSF-TP2A-65) possess topsoil that would be considered strongly sodic (ESP >14).

As the topsoil material at all locations may be considered as sodic, consists of silty gravel or silty sand, and has low total organic carbon content, it should be considered as highly susceptible to water erosion. This is further reinforced by the fact that the majority of locations have been assigned a Class 1 dispersion risk. Due to this, a conservative erosion hazard for water of "High" is suggested for locations with Class 1 dispersion risk and "Moderate" for locations with Class 2A.

For materials with low clay content (<10 % clay) sodicity is of a low risk and subsequently the dispersion risk is lower.

Twelve grab samples of stockpiled topsoil from other areas of the North Star site (Gatehouse, east of pit and Japal area) were recovered by IBO in 2018 and supplied to Landloch for analysis **[11]**.

Landloch concluded that:

The topsoil samples have suitable soil pH1:5 and are non-saline. They contain appreciable gravel contents. Based on their ESP, they are generally not prone to clay dispersion, though the low EC1:5 values may mean that they are prone to hard setting. Two materials did have elevated ESP values and sufficient clay and ECEC for them to present a risk of dispersion. However, the rocky nature of the soils will act to mitigate some of the potential structural decline.

The nutrient status of the stockpiled soils is lower for the East of Pit soils than measured for undisturbed soils and when compared to typical values for soils in the Pilbara region. The values measured for the Gatehouse and Japal areas seem reasonable. Addition of N, P, and K may improve plant growth for the East of Pit soils.

Overall the site soil characteristics indicate that if properly managed, the topsoil would provide a suitable medium for growth during rehabilitation of the TSF. Addition of significant amounts of fertiliser is generally not encouraged in the Pilbara since this tends to increase the likelihood of weed generation.

5.3.2 Tailings

5.3.2.1 Physical properties

The tailings produced during the North Star operation are expected to comprise low plasticity silt with liquid limit in the range 28% - 25% and Plasticity index < 4%. Specific gravity of the tailings solids is expected to be approximately 3 t/m^3.

Based on consideration of laboratory test results, potential for evaporative drying and likely rate of consolidation of the deposited tailings, an overall in-situ density of 1.5 t/m^3 has been adopted for TSF design during the the first six months of operation and 1.66 t/m^3 thereafter.

At closure, most of the beach surface is expected to have reached a shrinkage limit density (SLD) of approximately 1.8 t/m^3 and in the absence of slurry deposition and decant pond operation, the beach surface in the pond area is expected to reach SLD within a period of several months.

Based on the tailings characterisation test work and proposed deposition methodology, predicted **[27]** beach slopes are approximately 0.4% at the eastern (upper) side of the TSF, decreasing to less than 0.2% at the western side.

5.3.2.2 Geochemistry

Based on the results of geochemical analyses conducted on tailings samples in 2014 **[28],** and 2015 **[29]**, the tailings are considered to be non-acid forming (NAF).

Chart 5.1 presents the tailings sample classification following the AMIRA classification method **[30]**. The range of NAPP (-30 to -90 kg H_2SO_4/t) and ANC/MPA ratio values (2.4 to 193) placed all the samples in the acid consuming category (**Chart 5.2)**.

X-ray diffraction (XRD) analyses identified that the oxide tailings were mainly quartz and iron oxide in the form of hematite and goethite. No sulphide or carbonate minerals were identified in the oxide samples. A wide variety of minerals were encountered in the fresh tailings samples; however, Pyrite or any other sulphide phase was not present in detectible quantities. Significant sulphate mineral content (fibroferrite) was observed; however, the potential for acid formation from fibroferrite is much lower than pyrite.

Further substantiation that the tailings are non-acid forming was provided by RGS in 2017 **[31]** and it was also determined that most of the waste rock materials are likely to be NAF with excess ANC. Some waste rock materials were identified as being potentially acid forming (PAF) and further work has been recommended to fully assess the geochemical characterisation of waste rock materials. It is recommended that routine tests are conducted on the waste rock during all stages of construction with the aim of eliminating the use of any PAF material in embankment construction or as closure cover for the TSF facility.

No geochemical characterisation testing has been conducted on decant water to date.

Chart 5.1
NAG pH & NAPP Values For Tailings Samples

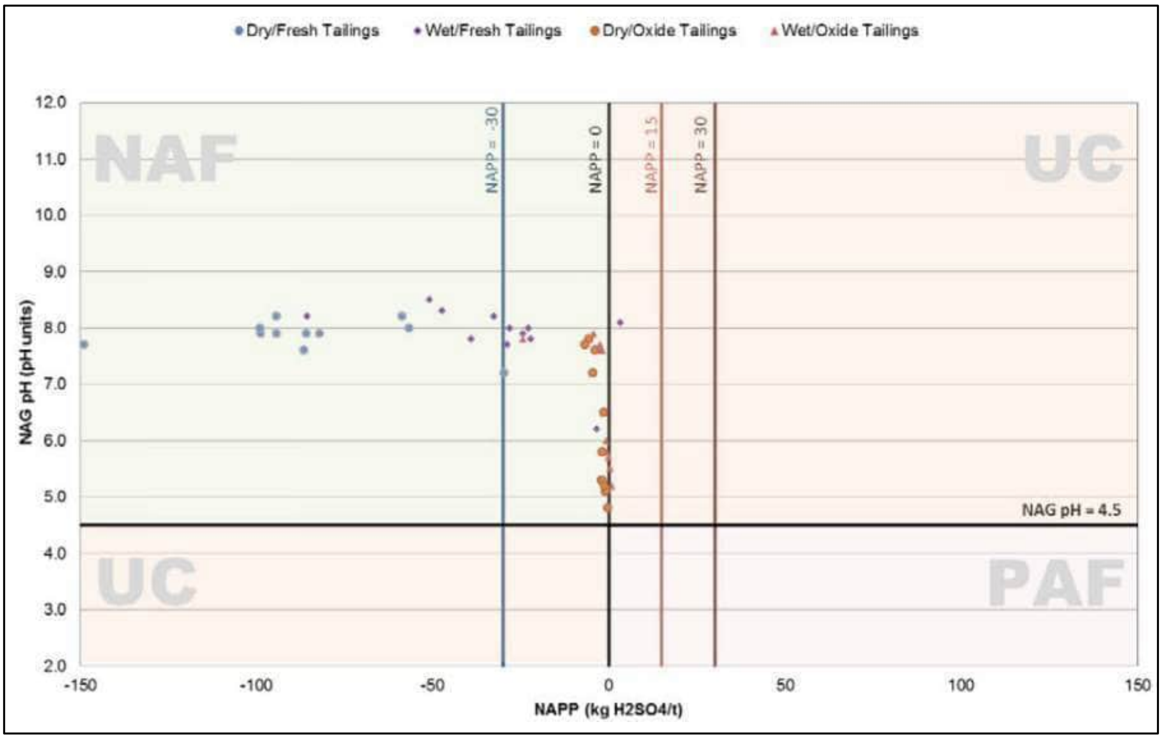
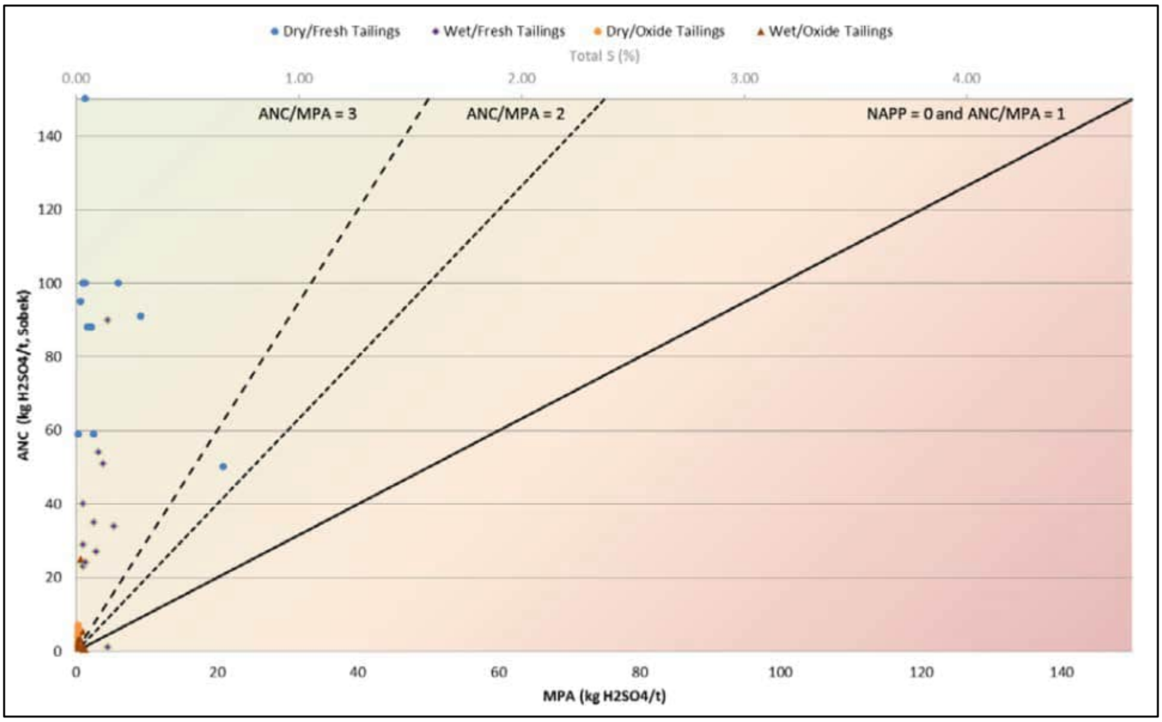


Chart 5.2
ANC & MPA Values For Tailings Samples



Landloch **[11]** made a preliminary assessment of the suitability of the existing Stage 1 tailings for use as growth medium and concluded that:

The tailings tend to be more saline than the natural soils (and coarse rejects) and that their salinity may affect they germination of growth of salt sensitive species, but is likely to not eliminate all plant growth.

The tailings are alkaline compared to the soils, with pH1:5 values ranging from 8.60 -9.24 and averaging 8.9. The high pH of these materials may impact on the growth of some plants.

Two of the five tailings samples tested had elevated ESP values. They also had appreciable clay, though they are still dominated by fine sands. These materials should be considered prone to structural decline through clay dispersion and possibly tunnel erosion due to wetting and surface liquefaction.

The tailings have low Total N and Available P and high Available S values compared to the soils. although high S values may not require further management. The low N and P values could be amended through application of fertiliser.

Fixation of P in the tailings could be considerable and these rates would require verification via pot trials before they could be considered optimal application rates.

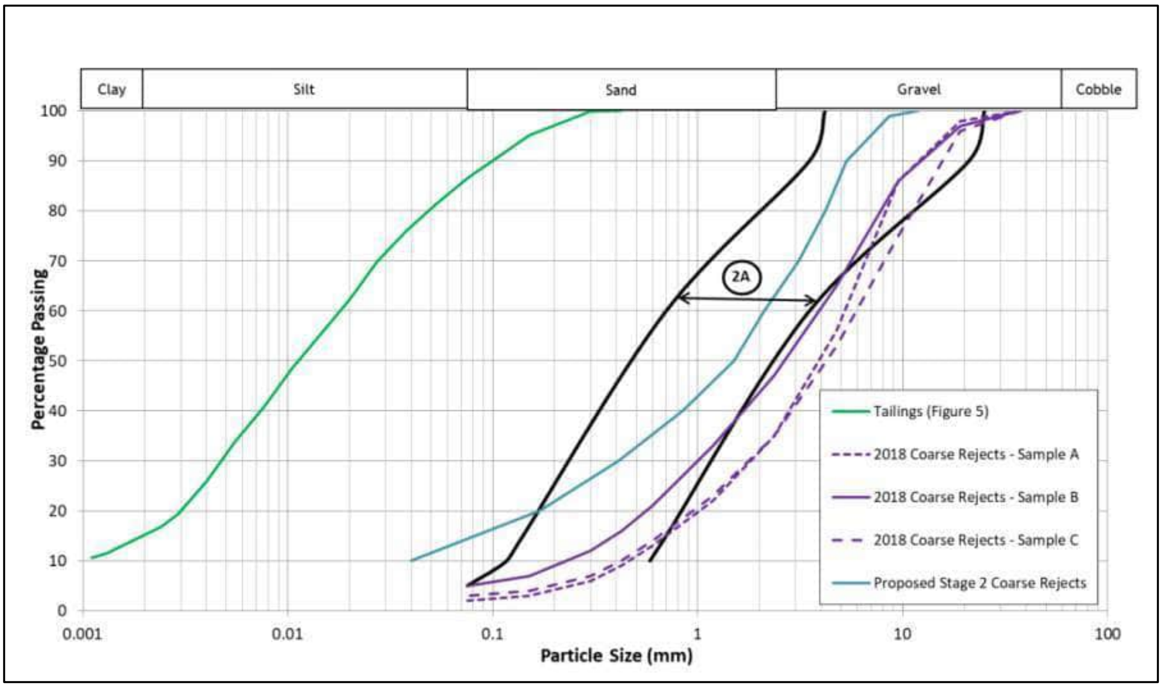
5.3.3 Coarse Rejects

Zones 2A and 2B embankment materials are expected to be sourced from the existing coarse reject stockpile located at the Stage 1 North Star process plant site and subsequently from Stage 2 production of similar material. Coarse rejects may also be used to reshape strategic areas of the landform surface prior to erosion resistant cover placement or could potentially be blended into the final cover layer.

Particle size distributions for the existing Stage 1 and proposed North Star Stage 2 coarse rejects (as advised by IBO) are plotted in **Chart 5.3**.

It is evident that the rejects are generally blended silt-sand-gravel materials with fines contents (percent passing 75 μm, i.e. silt and/or clay) ranging from 5% to 10%. Based on laboratory testing, the fines are non-plastic.

Chart 5.3
Particle Size Distribution – Coarse Rejects & Zone 2A Filter Criteria



Landloch **[11]** conducted an assessment of the suitability of coarse (dry) reject samples from the Stage 1 stockpile and concluded:

Dry rejects are non-saline alkaline compared to the soils, with pH1:5 values ranging from 8.67-8.90 and averaging 8.8. The is similar to the pH of the most alkaline soils reported by GHD (2011). However, on average the pH of the soils was lower (6.8). The high pH of these materials may impact on the growth of some plants although detail on pH effects on specific native species in the Pilbara is scarce.

The dry rejects have low Total N and Available P and high Available S values compared to the soils. Issues with Sulphur are more related to sulphur deficiencies rather than toxicities. Therefore, the high S value may not require further management. The low N and P values could be amended through application of fertiliser.

The fixation of P in the dry rejects could be considerable and these rates would require verification via pot trials before they could be considered optimal application rates.

5.3.4 Oxide Waste

The oxide waste is likely to be reasonably well graded and a potential foundation for growth medium.

Landloch **[11]** conducted analysis of 15 samples of oxide waste from the existing stage 1 waste dump and concluded:

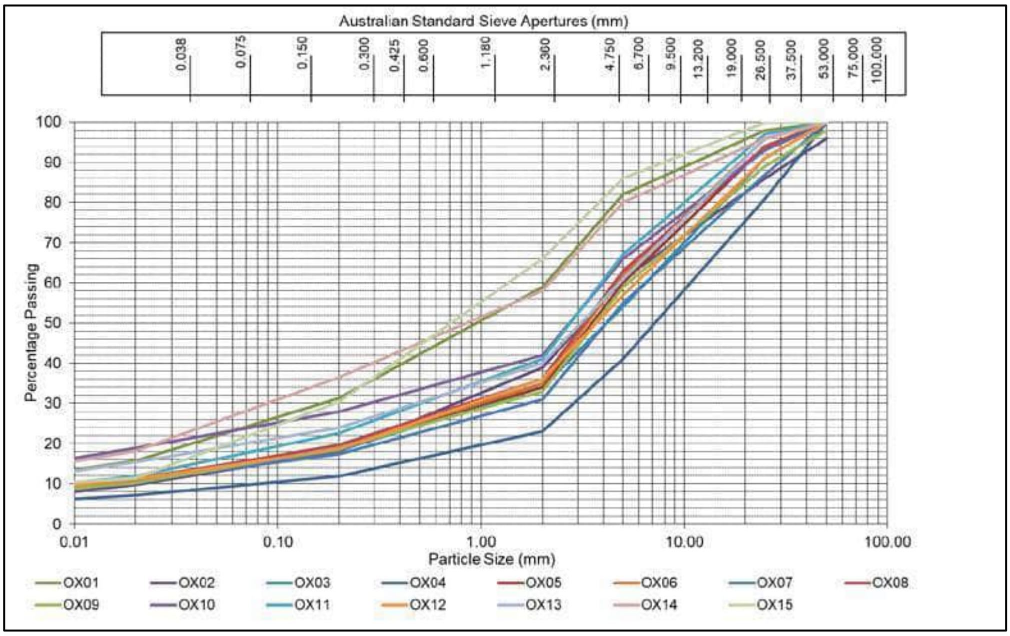
Oxide wastes have suitable soil pH1:5 and are non-saline. They are not prone to clay dispersion and hence are at low risk of tunnel erosion.

The oxides have generally suitable fertility except for low Total N values, which could be adjusted by fertilizer addition.

An image of the oxide waste material tested by Landloch is given on **Photo 2**. Grading curves interpolated from the Landloch data are given on **Chart 5.4**.

Photo 2
Oxide Waste Material – Landloch Sample

Chart 5.4
Particle Size Distribution – Oxide Waste



Geotechnical characterization of Oxide BIF waste, completed by SRK **[32]** generally inferred a coarser grading for the oxide waste, with D_{50} ranging from 25 mm to 170 mm. Mean Unconfined Compressive Strength was 36 MPa (Medium strength (ISRM)), inferred lower bound permeability was 1 x 10^{-2} m/s and based on results of slake durability tests, the material was classed as having very high to extremely high durability.

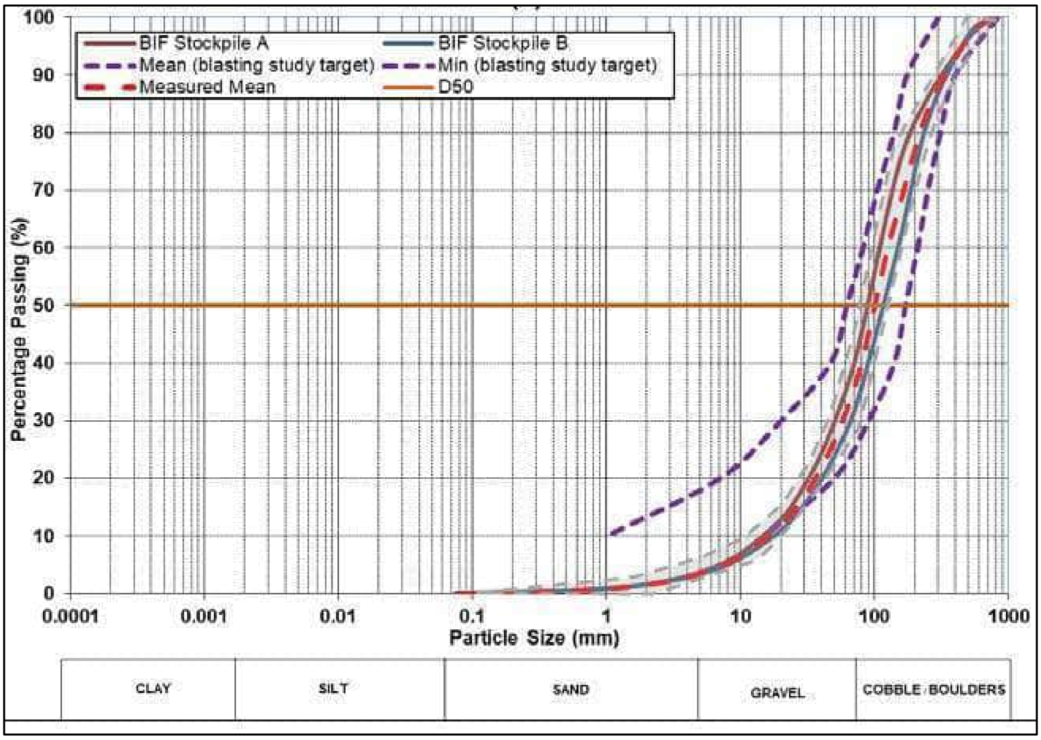
5.3.5 Fresh Waste

The fresh waste is expected to predominantly comprise cobble and boulder size material of extremely hard and durable material, suitable for long term erosion protection to design drainage pathways on the landform surface.

Geotechnical characterization of fresh BIF waste, completed by SRK **[32]** generally inferred a D_{50} ranging from 60 mm to 170 mm. The inferred particle size distribution is illustrated on **Chart 5.5**.

Mean Unconfined Compressive Strength ranged from 167 MPa to 306 MPa (Very high to extremely high strength (ISRM)), inferred lower bound permeability was 2 x 10^{-2} m/s and based on results of slake durability tests (**Appendix A**), the material was classed as having very high to extremely high durability.

Chart 5.5
Particle Size Distribution – Fresh BIF Waste [32]



5.4 Completion Criteria

To inform the basis of design for closure of TSF2A, relevant completion criteria have been considered. These criteria and the status of available closure data associated with the criteria are described below. The basis of design (**Section 7**) summarises the adopted design objectives and parameters to address the completion criteria.

5.4.1 Embankment Stability

The post closure landform should be geotechnically stable under both static and dynamic conditions.

Geotechnical parameters for use in limit equilibrium slope stability analysis have been defined for the purpose of TSF 2A design. Additional non-linear shear strength envelopes have also been defined for the various oxide and fresh waste materials where used as rockfill **[32]**. Development of these parameters is based on the outcomes of soil laboratory testing, adopted material selection and compaction control specification for embankment construction.

The embankments associated with the post closure landform have been designed to meet specific factors of safety with respect to geotechnical stability.

Validation of geotechnical design parameters will be achieved by periodic geotechnical investigation during the course of TSF development.

5.4.2 Erosional Stability of Landforms

The final landforms should have;

No significant rill or gully erosion
No sediment fans at the landform toe
No sedimentation of local drainage lines; and
The presence of vegetation.

The materials characterisation completed for the purpose of project approvals indicates that the materials expected to be available for closure purposes (topsoil, coarse rejects, oxide waste and fresh waste) all have potential for beneficial use in rehabilitating and stabilising the final landform.

Placing durable waste rock material over the tailings surface of the TSF landform will result in a wide, very gently sloping uniform surface of less than 1% grade, with little potential for attenuation of high velocity water flows.

Concentration of surface water run-off at the perimeter embankments has been considered in the assessment of potential batter and berm configurations.

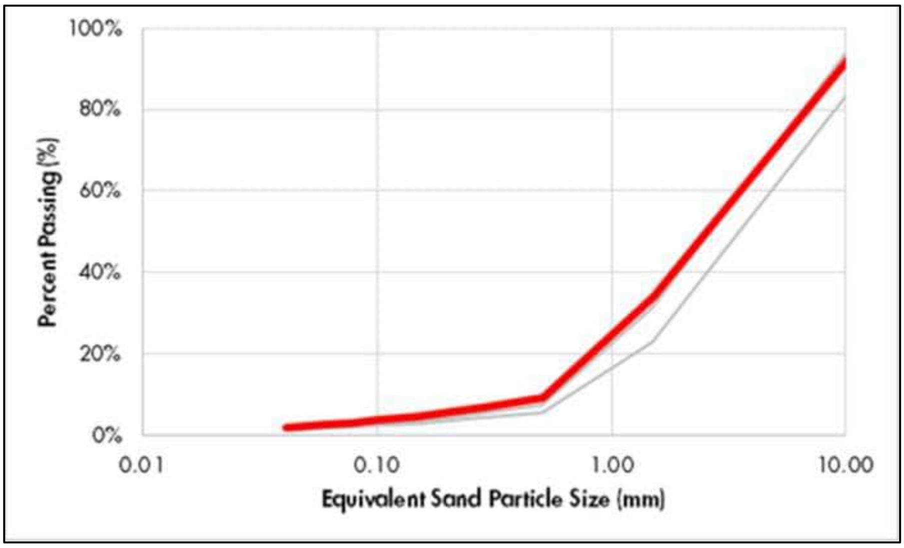
Based on data obtained for existing oxide waste material at the Stage 1 waste dump area, a preliminary WEPP model was developed by Landloch **[11]** to assess the potential erosion rate of various landform batter slopes if covered with oxide waste.

The model input parameters derived by Landloch from laboratory testing are summarised in **Table 5.1** and **Chart 5.6**.

Table 5.1
WEPP Parameters Derived From Laboratory Testing – Oxide Waste

Material	Effective hydraulic conductivity, K_e	Interrill erodibility, K_i	Rill erodibility, K_R	Critical shear, τ_c
	mm/h	kg.s/m^4	s/m	Pa
Oxide	12	302,986	0.00597	37

Chart **5.6**
Equivalent Sand Particle Size Distribution Curves For The Oxide Material



A risk matrix derived by Landloch to assess acceptable landform erosion risk is illustrated on **Table 5.2**.

Critical rock diameter values under typical Pilbara conditions nominated by Landloch (for a typical batter slope of 18°) range from 55 mm to 125 mm for lift heights ranging between 10 m and 60 m.

The inferred D_{50} of the oxide waste sampled by Landloch at North Star Stage 1 is <10 mm, which is less than 50% of the nominated CRD values. This implies that erosion is likely and there is a moderate or high risk of localised impact due to sediment movement from the landform if oxide waste is used as batter cover material.

Table 5.2
Landform Design Risk Matrix [11]

Likelihood of high rates of erosion			Consequence: Potential environmental impact of high rates of sediment movement from a waste landform*		
			Localised impact but not affecting wider ecosystem function	Ongoing & expanding ecosystem impairment (degradation not loss)	Loss of high value species, habitat or ecosystem value
Potential for active gullying	D_{50} =<50% of CRD^A	Likely	Moderate	High	High
	50% of CRD <D_{50}< CRD	Possible	Low	Moderate	High
	D_{50} =>CRD	Unlikely	Low	Low	Moderate

** Matrix assumes hostile (e.g. PAF) wastes are suitably encapsulated.*
^A: Critical Rock Diameter. To avoid having a small number of very large particles bias the D_{50} estimates, it is also stipulated that D_{80} should be no larger than 3 times D_{50}.

D_{50} estimates for the fresh BIF waste material have been estimated by SRK **[32]** to range between 60 mm and 170 mm. On this basis, a D_{50} value similar or greater than the critical rock diameter is inferred and the overall risk of erosion will be low or moderate if fresh BIF waste is used for batter cover material.

As the potential environmental impact due to landform batter erosion at the North Star site could potentially include localised loss of high value species habitat or ecosystem value, it is concluded that if oxide waste is used as cover material to the embankment closure batters, a “High” risk rating is appropriate and if Fresh BIF waste is used, a “Moderate” risk rating may be appropriate. The potential for high rates of erosion may be considered as “possible” pending field assessment of the quality of the fresh BIF waste, consequently, at this stage a “High” risk rating is considered applicable for either case.

Acceptable erosion rates for various risk ratings are indicated in .

Table 5.3
Acceptable Erosion Rates for Various Risk Rating [11]

Risk Rating	Mean Average Annual Erosion (t/ha/y)	Mean Peak Annual Erosion (t/ha/y)
Low	9	18
Moderate	6	12
High	3	6

The outcome of the WEPP modelling for various batter heights and slope angles using oxide waste is summarised in **Table 5.4**.

Table 5.4
WEPP Model Simulation Result Outcomes [11]

Simulation No.	Height (m)	Gradient (°)	Suitable batter configurations based on an acceptable erosion threshold consistent with sites that are:	
			Moderate Risk	High Risk
1	40	37	×	×
2	30	37	×	×
3	20	37	×	×
4	10	37	✓	✓
5	10	18	✓	✓
6	20	18	×	×
7	10	16	✓	✓
8	20	16	×	×
9	10	10	✓	✓
10	20	10	×	×
11	100	5	✓	×
12	50	5	✓	×
13	30	37-20-10	✓	×
14	20	37-25	×	×
15	10	37-25	✓	✓
16	20	37-25	×	×

The results indicate that for sites with a high risk rating, berm heights should be restricted to 10 m for oxide waste material. The predicted erosion rates for the various batter angles considered are indicated in **Table 5.5**. The simulations shaded in green satisfy the mean average and mean peak acceptable erosion rates criteria for a high risk rating (**Table 5.3)**.

Table 5.5
WEPP Model Predicted Erosion Rates [11]

Simulation No.	Height (m)	Gradient (°)	WEPP-Predicted Long-Term Mean Annual Erosion (t/ha/y)	
			Mean	Peak
1	40	37	46	125
2	30	37	29	100
3	20	37	12	56
4	10	37	0.6	5.0
5	10	18	0.4	3.6
6	20	18	9.0	45
7	10	16	0.3	2.7
8	20	16	7.5	40
9	10	10	0.1	0.6
10	20	10	2.5	19
11	100	5	3.4	9.9
12	50	5	3.1	12
13	30	37-20-10	1.4	11
14	20	37-25	4.8	30
15	10	37-25	0.2	1.0
16	20	37-25	3.6	21

The outcome of the modelling indicated that several potential batter geometries would be suitable to maintain acceptable rates of erosion to minimise rilling and gullying of oxide waste cover material.

For the scenario where fresh BIF waste material is adopted as cover material, the likelihood of significant erosion rates occurring will be significantly reduced, giving an improved risk outcome. Batter heights greater than 10 m are likely to give acceptable outcomes for a range of slope angles using fresh BIF waste; however, at this stage, in the absence of modelling and site trials to demonstrate this, a maximum height of 10 m is considered appropriate, irrespective of material type adopted for cover profiling.

5.4.3 Dust Generation

Dust generated post closure of the facility should not exceed that which may reasonably be expected to occur from the surrounding natural landscape.

Notwithstanding other completion criteria, the feasibility of leaving the tailings beach uncovered at closure has been assessed.

Factors that would assist in minimisation of dust generation post closure include:

The presence of strong inter-particle forces due to the fine grain size and cohesive nature of the tailings;

The anticipated generation of shrinkage and desiccation cracks and cementation of tailings on the surface of dessication polygons which will resist further breakdown into silt and clay particles (unless disturbed or very wide dessication cracks form) that may lead to dusting; and

The presence of continuous containment embankments elevated above the tailings surface which limits the potential for dust transportation across the tailings surface and off the surface.

At closure, it is anticipated that a thin cemented surface crust will be retained on the tailings surface which will inhibit dust generation whilst intact. In the long term; however, it must be assumed that the crust will be broken and placement of a suitable cover material over the tailings beach will be required (wind erosion features have been observed on the stage 1 TSF 1 surface since it was decommissioned).

For design purposes, a minimum cover requirement of 500 mm has been adopted. A lesser cover thickness would minimise potential for dusting, but for the purpose of practical material placement, 500 mm is considered appropriate for creation of stable access for construction machinery during closure implementation.

5.4.4 Groundwater Quality

Long term groundwater quality should be comparable to baseline quality prior to operations. Leachate generation through long term flushing should not be of lesser quality than groundwater. Long term seepage associated with consolidation of the deposited tailings should not have a significant detrimental impact to groundwater or surface water.

Baseline groundwater quality data currently available comprises measurement of field parameters (EC, pH) between March 2012 and January 2019 at three monitoring bores in the vicinity of TSF 2A (BH05A, Obs-13 and PB02). The data indicates that the groundwater is neutral (pH 7.4) and brackish to saline (average EC 2300 – 2600 µs/cm).

Testing of process water introduced in the tailings slurry will be completed on a quarterly basis during operations.

Desaturation of the tailings will occur over time due to consolidation. Pore water released during consolidation is expected to be of similar quality to process water. Post de-saturation, flushing with rain water infiltration will occupy available pore space but will be limited to the upper part of the tailings deposit due to the short-term transient nature of recharge and excess evaporative potential.

Transient state analyses to assess the likely post closure phreatic levels within the final embankments and closure landform have been completed for the purposes of design. The analyses indicate that the rate of release of pore water during consolidation will be very low due to the low permeability of the tailings material.

5.4.5 Contaminant Migration

The risk of migration of contaminants within the tailings deposit via leaching and seepage should be minimised or eliminated for the post closure condition.

The tailings seepage water is expected to be clean of contaminants, however the potential for localised deposition of PAF tailings resulting from processing of PAF ore could result in elevated acidity levels in leachate.

Identification and differentiation of any PAF ore will be undertaken during mine development and the location within the overall tailings deposit of any tailings produced from processing PAF ore should be established so that the potential for long term oxidation of the material can be assessed.

Provided that PAF ore processing does not take place during the final stages of operation, the potential for long term oxidation of these materials will be minimised by burial with NAF tailings.

5.4.6 Performance of Surface Water Management Measures

The post closure landform should be capable of storing or passing the storm water run-off associated with PMP rainfall events without significant erosion of the landform being initiated.

The TSF has a large catchment and as such, severe storm events will cause peak flows across the landform which can be modelled and calculated. Predicted flow rates and depths can be used to assess the likely performance of the closure cover and design and construct stormwater management structures such as spillways and external diversion drainage features. The final landform is shaped to accommodate floods with limited:

- final landform erosion,
- points of concentrated flow,
- gradient changes (rapid or sharp gradient changes invite erosion and degradation over time)
- channelling or restriction of flow.

5.4.7 Vegetation Establishment and Maintenance

The final landform should be capable of supporting native vegetation to a similar extent as the surrounding land.

Opportunities for vegetation establishment on the final landform are limited effectively to areas with low surface gradient and the least risk of erosion. Such areas comprise the upper surface of the final landform and to a lesser extent the embankment batter berms.

The design of the surface cover profile includes provision for cross bunding to attenuate rainfall and concentrate moisture behind the bunds, provide a focus for placement of available topsoil and encourage the initiation of vegetation establishment on the closure surface.

To facilitate rapid infiltration of storm water run off into the high permeability fresh BIF waste on the embankment batter closure profile, it is not intended to place growth medium on the batter berms as this would impede infiltration. Utilisation of the limited growth medium available will be focussed on strategic establishment of vegetation on the upper landform surface.

6 OPTIONS ANALYSIS

To establish the basis of design criteria for closure of the TSF 2A facility, an options analysis was conducted to determine suitable options which would address the outcome of the closure design considerations described above.

Details of the options analysis are given in **Appendix B**.

In summary, the analysis considered:

- Final embankment profile
- Final embankment drainage
- Final tailings surface cover
- Final tailings surface drainage
- Growth media and substrate

A nominal cover (minimum 500 mm) to the beach is considered necessary to reduce the long-term likelihood of dust generation and scour due to concentration of rainfall run off.

6.1 TSF Embankment Closure

Options considered for the final embankment profile comprised;

1. Leave embankments at LOM geometry (1v:2H);

2. Reprofile via cut to fill and additional import at the toe to create a single concave slope of approximately 20° for the upper 50% and 15° for the lower 50%. Three material placement sub-options were considered;

3. Reprofile via a two-lift, concave batter berm configuration with maximum upper and lower lifts each of approximately 35m vertical height, 30m wide berms and individual batter faces of approximately 20° upper and 15° lower sections. The as presented material would be utilised for ripping and seeding. Four material placement sub-options were considered;

4. Reprofile via a three-lift, concave batter berm configuration with maximum upper and lower lifts each of approximately 20 m – 25 m vertical height, 20 m wide berms and individual batter faces of approximately 20° upper and 15° lower sections. The as presented material would be utilised for ripping and seeding. Four material placement sub-options were considered;

5. Import waste material and tip with no reprofiling to form 10 m high benches with berms up to 20 m wide between toe of batter above and crest of batter below. Two material placement sub-options were considered;

6. Import material to form a 5° slope. Two material placement sub-options were considered.

Options considered for drainage of the final embankment profile comprised;

a. No specific drainage provision on concave slopes;

b. Berms would be constructed to be level, back sloping with cross bunds to ensure that the only water temporarily ponding on berms is from the batter immediately above;

c. Berms graded, nominally at 1-2% to facilitate low velocity run off from berms to natural ground via hardened drains on the berms;

d. The majority of the length of berms would be constructed as per option b. Where sections of berms are adjoining natural ground, and where discharge can be effectively achieved, short sections would be constructed consistent with option c.

The selected configuration, based on consideration of earthworks volumes required, erosion resistance, potential for establishment of vegetation and impacts to the downstream drainage flow regime (including spillway operation) was Option 5b, using fresh BIF waste with level, back sloping berms (drainage options b/d).

Analyses to validate the optimum berm width are given in **Section 9**. The selected width is 10 m with 5% backslope and 1.4 m edge bunds.

6.2 TSF Surface Closure

Options considered for the final surface profile comprised;

1. Leave the top surface as deposited during operations. Make safe;
2. Place an even layer of oxide waste over the beach surface, of sufficient thickness to be placed in a practical and uniform manner and to minimise potential for exposure of tailings which could be subjected to long term wind erosion;
3. Place an even layer of fresh waste over the beach surface, of sufficient thickness to be placed in a practical and uniform manner and to minimise potential for exposure of tailings which could be subjected to long term wind erosion;
4. Place a mosaic of blocky durable waste and oxide in patches across the tailings surface strategically placed in lower and higher surface water flow locations.
5. Place a layer of blocky durable waste overlain by a layer of oxide waste material.

Options considered for drainage of the closure surface profile comprised;

a. No specific drainage provision on surface; Spillway only.

b. Form multiple contour banks with in situ material in order to establish haulage access to import cover material. The haulage routes would be sheeted with conditioned/compacted material as a running surface. They would be armoured on the upstream face with a minimum 1m durable blocky fresh waste;

c. Form waste rock bunds at strategic locations to create flow interruption and limit velocity locally. Attenuation behind rock bunds reduces likelihood and frequency of discharge over spillway.

d. Construct formal drains over cover surface which concentrate flow from throughout the facility to larger drains which discharge at the spillway.

The selected configuration, based on consideration of earthworks volumes required, erosion resistance, flow velocity and potential for establishment of vegetation is option 2, using 500 mm thickness of oxide waste and provision of waste rock bunds across the surface to attenuate run off and create areas for initial vegetation establishment.

Analysis to validate the capability of the selected option to pass PMF events without overtopping or excessive erosion is given in **Section 9**. It is assumed that the cross bunding could be eroded and re-distributed over time, such that the system must maintain the ability to manage PMF events without such bunds being present.

On the basis of hydraulic flood routing modelling, sizing of the closure spillway has been undertaken such that it can safely pass PMF run off and flow resulting from the critical PMP storm event.

6.3 Growth Media

Options considered for the placement of growth media comprised;

1. No growth media anywhere on closed TSF;

2. All topsoil resources placed as islands on the TSF cover surface. Most effectively over oxides and in lower/no flow velocity parts of the facility (in the lees of island or promontories or behind contour banks);

3. All topsoil resources placed on the embankment batters. Most effectively over options where oxides and durable blocky waste are mixed/integrated. This may involve placement of soils only on the lower sections of batters where velocity is likely to be higher.

4. All topsoil resources placed on the embankment berms. Most effectively over options where oxides and durable block waste are mixed/integrated.

5. Mixed deployment which may involve a combination of very small islands of topsoil on the top surface within large islands of oxide to create biodiversity distribution zones, deployment of material on berms and /or batters or deployment of soils on the lower half of concave slopes to establish vegetation where flow velocities may be higher.

The selected option, based on limited topsoil availability and the inferred properties of the available waste materials is option 2, whereby,

- Deployment primarily supports surface stability
- Some volumes are allocated for sustainability/aesthetic objectives
- Flat surfaces of oxide material are likely to be suitable to revegetate with seed only

7 BASIS OF DESIGN

Based on the defined closure objectives, completion criteria and the outcomes of options analysis, together with review of relevant regulatory criteria, the basis of design for the final landform is summarised in Table **7.1**. A breakdown of the design criteria considerations is given in **Appendix C.**

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Table 7.1
Basis of Design Final Landform

Design Aspect	Adopted Basis			
	Source	Criteria 1	Criteria 2	Criteria 3
Geotechnical Stability	ANCOLD	Limit equilibrium FOS > 1.5	MCE deformation < PMP freeboard	Minimum 2m thickness of durable waste rock cover on batters
Minimum Design Life	DMIRS	300 yrs	-	-
Closure surface storm water management	DMIRS	Pass or Store PMF	Spillway to pass critical PMF	-
Embankment storm water management	FMG/DMIRS	Batter/Berms to infiltrate/contain PMP excess rainfall	Transient saturation to have no significant impact on geotechnical stability (FOS > 1.3)	-
Surface cover	GARD, Design Outcomes	Limit net infiltration to “hostile” waste	Support native plant growth except in areas designed as angle of repose slopes or durable fresh waste armour	-
Erosional stability - Water	Landloch WEPP model	Average mean and peak annual sediment loss less than acceptability criteria for high risk ranking.	Self-stabilising erosion features	Sediment emissions do not have significant off-site impact
Erosional stability - Wind	DMIRS	Erosionally stable	No dust leaving Lease boundary	All tailings exposures will be covered with rock

8 CLOSURE STRATEGY

8.1 Summary

Closure for the RWP area will comprise draining of the residual pond, removal of the RWP embankments (for use in buttressing the TSF main embankment) and grading of the RWP impoundment to remove any transported tailings residue on the ground surface. The extent to which vegetation within the remainder of the RWP impoundment has been impacted during facility operation (during temporary flooding events) should be assessed prior to closure to develop a site specific rehabilitation plan for this domain.

Re-establishment of vegetation in the drained operating pond area will be improved through the targeted application of topsoil.

The downstream slopes of the LOM TSF embankments will be buttressed and regraded to form a stable, erosion resistant long-term landform. Based on current best practice for landform rehabilitation in the Pilbara region and the outcomes of erosional stability modelling by Landloch **[11]**, the regrading will comprise creation of benched slopes at a nominal angle of repose of 37° with bench height of 10 m and berms of sufficient width to infiltrate and contain run off from PMP storm events. Provision of strategic edge bunds to cater for short duration, high intensity PMP events is also proposed.

Typical sections through the re-profiled embankments are given on **Drawings 114185.15_004** to **114185.15_006**.

It is not anticipated that the TSF embankments will be constructed to the final closure profile during the operational life of the TSF; nonetheless, if minimising final closure costs becomes a priority in the future it will be possible to adopt the closure profile or at least develop the lower benches at an earlier stage of TSF embankment construction.

As there will be insufficient topsoil recovered from the TSF impoundment to fully cover the LOM landform, the surface cover material is likely to predominantly comprise oxide and fresh waste material from the Stage 2 mining operations, with localised areas designated for topsoil spreading, fertilisation and establishment of vegetation.

The tailings beach is likely to have growth medium characteristics similar to the stockpiled 20-year old topsoil; consequently, subject to ongoing material characterisation and completion of rehabilitation trials, consideration may be given to blending tailings material with residual topsoil or screened oxide waste to increase growth medium coverage.

The available waste materials will include oxide waste rock, fresh waste rock and coarse rejects material.

8.2 Decommissioning

8.2.1 General

Decommissioning of the TSF and RWP begins when the final raise construction is completed. The final raise should be constructed to the dimensions set out in the closure design.

8.2.2 Return Water Pond

Any stored water remaining in the return water pond impoundment at closure will be drawn down to enable access to the full impoundment floor area for grading and general rehabilitation of the ground surface and original drainage line.

Any residual tailings slimes that may have been transferred through the decant system or settled out of the return water pond during operations will be recovered from the impoundment surface and deposited in the TSF.

The RWP embankments will be removed and the waste rock materials recovered will be used to provide erosion protection locally or will be used in the TSF embankment re-profiling earthworks.

The BGM liner will be removed from the upstream face and discarded to landfill.

8.2.3 Spillways

The Stage 7 TSF spillway will be widened and deepened to form the closure spillway arrangement. Excavated rock will be used in the embankment re-profiling works.

The RWP spillway floor will be covered with a minimum of 0.5 m thickness of oxide waste and topsoil to promote vegetation growth.

Sections and details of the closure spillway arrangement are given on **Drawing 114185.15_007**.

8.2.4 Decant towers

Two decant systems (North and South decants) will be constructed on the upstream slope of the main TSF embankment. The objective of the decant system is to remove the maximum practicable amount of decant and storm water runoff from the surface of the tailings, and pass this water to the downstream RWP.

The system consists of the following components:

(i) Decant inlet chute system - comprising the decant chute, aluminium stoplogs, safety grates and trash racks, access stairway, handrails, and connection with the decant outlet.

(ii) Decant outlet – comprising reinforced concrete-encased steel outlet pipe (NB700, API 5l X52, STD wall thickness 9.5 mm) pipe trench backfill, emergency isolation gate valve, precast concrete headwall and downstream stilling basin.

(iii) Decant operations system – comprising decant access pontoon and stoplog installation arrangements.

The decant inlet chute will be constructed of precast concrete box culvert sections (made to suit), within a reinforced concrete base slab.

The culvert sections will be constructed to the maximum height of each stage. Each pre-cast unit will be placed on a concrete slab, and will have a water-tight seal between each unit as well as having a commercial joint sealing tape on the outside of the joint. Cast in-situ steps and safety hand-rails will allow access full length of the decant chute.

The decant outlet pipes have been designed to function as a gravity system which will remove water from the decant chutes for discharge in the downstream RWP. The selected pipe is steel pipe of NB700, API 5l X52, STD wall thickness 9.5mm with internal diameter of 692mm.

The outlet pipes will be completely encased in reinforced concrete (to withstand embankment loading).

A conventional precast outlet headwall will be installed at the outlet, which will discharge into a Zone 3C rockfill – lined stilling basin to dissipate the excess energy which would otherwise scour the RWP base between the RWP operating pond and the outlet points.

Decommissioning of the decant system will comprise:

- Sealing of the 692 mm decant outlet pipes at the downstream toe with plugs of cement grout;
- Grouting of the decant chutes from the tailings beach surface using a 1.5 MPa tailings/cement/bentonite mix;
- Capping of the grouted chutes with a concrete slab;

- Breaking out and removal of any decant chutes exposed above the final tailings surface;
- Burial of the upper decant section under the beach cover material;
- Burial of the lower decant sections and headwalls under the lower embankment buttress benches.

Details of the decant system decommissioning are given on **Drawings 114185.15_008** and **114185.15_009**.

8.2.5 Diversion drains

The TSF 2A impoundment and its containment embankments effectively fills the valley catchment in which it is located, such that there are no significant diversion drains required during operation or closure of the facility.

8.2.6 Pumps, Pipes and Cables

All pipework, pumps, valves, cables and associated tailings delivery and water return infrastructure will be dismantled and removed from site at cessation of tailings deposition.

8.2.7 Access Roads

The access road and tailings delivery pipe corridor along the southern side of TSF 2A will be rehabilitated on closure. Construction of the road will predominantly be in cut through a competent rock mass; however, there are four short sections of embankment which are up to 25 m high at $1_V:2_H$ batters.

Specific closure measures are not considered necessary for the road cuttings, but the embankment sections will be re-profiled with 10 m high x 10 m wide benches as proposed for the TSF embankments.

8.3 Proposed Closure Landform

8.3.1 Embankment Profile

At closure, the downstream faces of the TSF embankments will be battered to create benched angle of repose slopes (37 degrees) using fresh BIF waste materials and incorporating up to seven 10 m high and 10 m wide benches as shown in the drawings.

8.3.2 Tailings Cover Profile

The design cover profile comprises 500 mm of oxide waste material with incorporation of four cross beach berms of 2 m nominal height to promote water retention in areas targeted for preferential topsoil placement and establishment of initial vegetation growth areas.

As the tailings surface is expected to comprise a wide, gently sloping beach, significant reprofiling of the surface is not considered necessary.

8.3.3 Closure Spillway

To provide sufficient spill capacity to cater for PMF conditions on the landform surface, and limit the extent of PMF ponding against the main embankment, the Stage 7 operational spillway will be widened by 50 m and deepened by 1 m.

Flood routing analyses to validate the closure spillway capacity are described in **Section 9**.

8.4 Materials Availability

Based on information derived from geotechnical investigation in the TSF impoundment area and from mine schedule information provided by IBO, the anticipated quantities of materials available for use in closure cover placement and embankment re-profiling at LOM are:

Topsoil/Subsoil	407,000 m^3 (Max area = 272 ha x 0.15m)
Coarse Rejects	111 Mm^3
Total Waste Rock	320 Mm^3

A combined recoverable topsoil and subsoil depth of 150 mm is assumed within the valley floor areas which comprise approximately 30% of the total TSF impoundment area.

A bulking factor of 1.3 has been adopted to convert mined waste bcm to volumetric equivalents for embankment construction.

The proportions of oxide and fresh waste rock materials have yet to be determined as part of the mine plan development but it is envisaged that at least 160 Mm^3 of each material type will be available.

The available volume of re-useable embankment fill within the RWP embankments is 95,000 m^3.

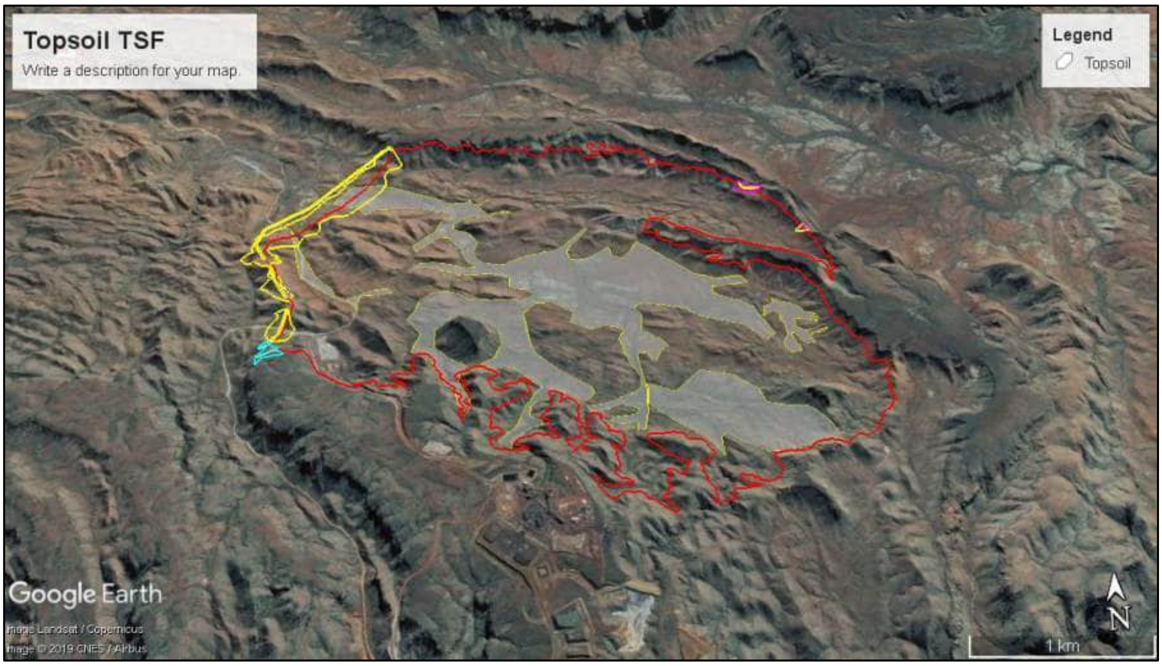
The volume requirements to be subtracted from the inventory for TSF embankment construction during the operations stage are:

Coarse Rejects	0.35 Mm^3
Total Waste Rock	10.6 Mm^3

These construction requirements are insignificant in comparison to the expected volume of waste to be produced.

The approximate extent of the areas within which topsoil/subsoil could reasonably be recovered are indicated on **Figure 3.**

Figure 3
Approximate Topsoil Recovery Areas – TSF Impoundment



9 ANALYSES

9.1 Flood Routing

9.1.1 Summary

Hydrological flood routing across the final design landform has been completed, up to and including probable maximum flood (PMF) conditions.

Initial flood routing calculations to size the LOM operating spillway are described in the TSF design report **[2]**.

Flood routing for the closure landform has been completed using the hydraulic software package TUFLOW.

9.1.2 LOM Assessment

For the purposes of sizing the operational stage spillways, in accordance with ANCOLD guidelines **[21]** a design rainfall ARI of 1:100,000 was adopted on the basis of the facility dam failure classification of "High C".

The procedures outlined in Australian Rainfall and Runoff (ARR) **[33]** were utilised in the calculation of the design rainfall depths for durations of 1 hour to 72 hours. The procedure involves first calculating 1 in 50 year and 1 in 100 year rainfall, then the Probable Maximum Precipitation (PMP) using the GSDM method **[34]** and the ARR preliminary prediction equations.

The design rainfall events are then interpolated from the calculated data sets, and temporal distributions applied using the parameters recommended by ARR. In accordance with ARR, the following methods have been adopted in assigning temporal patterns to specific rainfall durations:

(i) GSDM **[34]** for short durations (i.e. up to 6 hour rainfall durations), and;

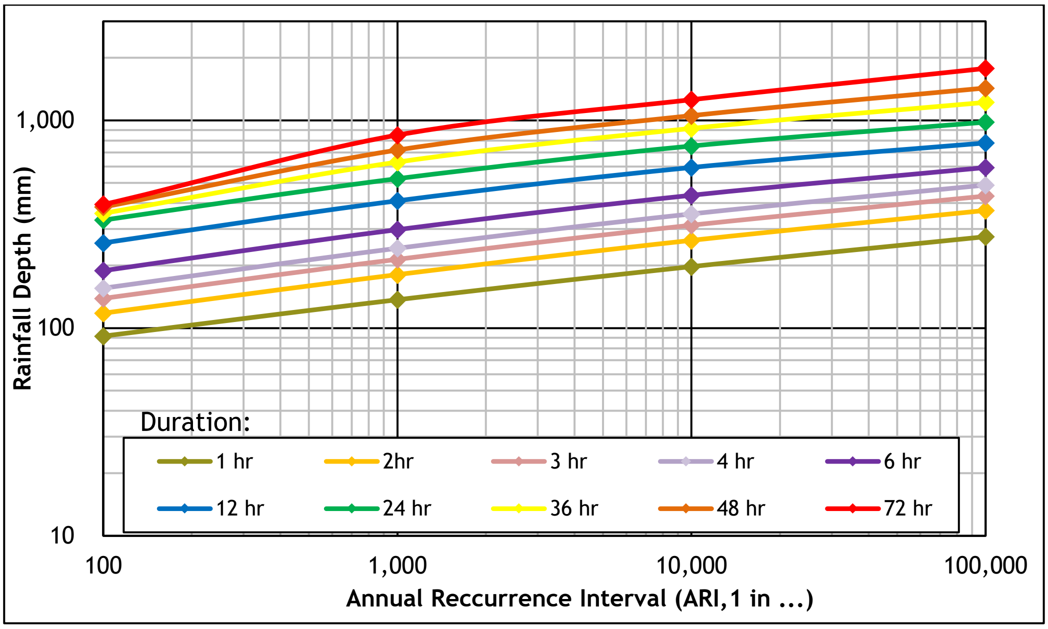
(ii) GSTMR **[35]** for intermediate durations (i.e. up to 24 hour rainfall durations).

The estimated rainfall depths for various ARIs and durations up to 1:100,000 are shown in **Chart 9.1**.

The inputs in the derivation of the design flood for a given duration are the design rainfall event and the design inflow hydrograph. The inflow hydrograph for a particular duration storm is the summation of individual component hydrographs from the pond and the surrounding catchment.

The total catchment area of the TSF impoundment and contributing natural catchment is 1,352 ha. As the catchment is ungauged, the flood hydrograph was modelled using the synthetic unit hydrograph technique. A synthetic unit hydrograph was constructed for each individual contributing catchment using the key parameters of the catchment (time of concentration, area, average slope), in accordance with the method described in ARR. The tailings impoundment flood hydrograph was derived using the time-area histogram method.

Chart **9.1**
North Star Design Rainfall Depths



Once the design flood for particular storm durations was derived, spillway sizing was undertaken by routing the flood through the storage. This was accomplished using the storage indication method, a direct numerical procedure which is described in ARR.

A flood wave passing through the storage is both delayed and attenuated as it enters and spreads over the pond surface. The surcharge storage is then gradually released over the spillway. In order to perform satisfactorily, the spillway configuration must be able to pass the critical duration design AEP flood without overtopping of the embankment crest.

Floods from storms of increasing duration were progressively routed through the storage, until a peak outflow was obtained. The spillway configuration (i.e. depth and width) was considered satisfactory only if the capacity was greater than the critical peak outflow.

The results of the TSF embankment spillway routing calculations for various stages of TSF construction are summarised in **Table 9.1**.

Table 9.1
Operational Spillway Flood Routing Results For TSF

Stage	Design Storm (AEP)	Critical Duration Event	Results		
			Parameter	Unit	Outcome
1A	1:100,000	6 hour	Sacrificial Bund Crest Width: Peak Flood Depth: Peak Outflow:	(m) (m) (m^3/s)	45 1.9 236
1B	1:1,000	18 hour	Spillway Width: Peak Flood Depth: Peak Outflow:	(m) (m) (m^3/s)	20 1.3 54
2	1:100,000	12 hour	Spillway Width: Peak Flood Depth: Peak Outflow:	(m) (m) (m^3/s)	50 1.4 147
7 (Final)	1:100,000	18 hour	Spillway Width: Peak Flood Depth: Peak Outflow:	(m) (m) (m^3/s)	50 1.2 114

The spillway channel outlets were positioned to divert any flows into a stream located directly downstream of the main and west embankments, and away from the embankment toe areas to minimise the potential for erosion.

9.1.3 Closure Assessment

9.1.3.1 Methodology

For the purposes of performing the closure flood routing assessment in a manner consistent with previous site wide hydrological modelling undertaken by BGE **[36]**, ATCW were provided with an existing TUFLOW hydraulic model which had been compiled to assess flooding characteristics of the TSF 2A area. The model received was named *'M06'* and occupies the part of the North Star Mine Site which includes TSF 2A.

The hydraulic modelling domain incorporated in the 'M06' model is illustrated in **Figure 4**.

Modifications were made to the model domain and resultant digital elevation model (DEM) grid to include the TSF 2A closure landform, part of the proposed waste rock dump and the LOM open pit configurations. A DEM grid size of 5 m was adopted for the model.

The Boundary Conditions for the TUFLOW hydraulic model consist of the following features:

- 'Rain on Grid' rainfall modelling method
- Downstream model outflow location (position where flows can exit the hydraulic modelling domain)

- Surface 'roughness' conditions
- Surface infiltration loss model

Rainfall data files were provided to ATCW within the TUFLOW model files for use within this assessment.

Manning's roughness coefficient '*n*' values were developed for the TSF closure surfaces. Natural surface roughness coefficients were supplied as part of the existing TUFLOW model. The values used within the model are summarized in Table **9.2**.

Figure 4

TUFLOW Hydraulic Model M06 Domain Extent

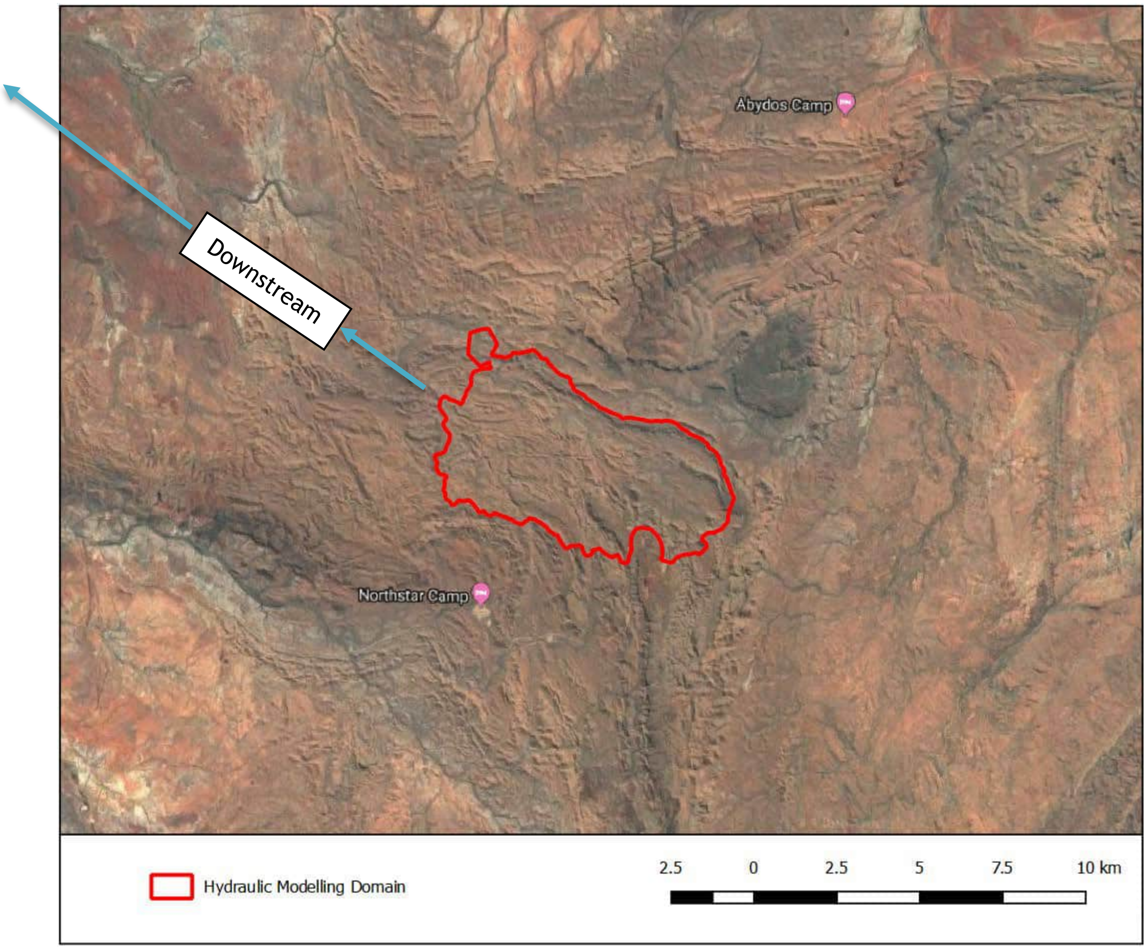


Table 9.2

Manning's Roughness Values Adopted

Surface Type	Manning's n	Description
Natural Surface	0.055	Undulating ground and scattered shrubs and rocks
Tailings Closure Surface	0.030	Machine graded surface and some grasses or small shrubs
Embankments	0.025	Machine trimmed uniform rock fill. Generally no vegetation.

Manning's 'n' roughness values were generally selected toward the upper range of what would produce higher impact modelled conditions. As the purpose of this model was to assess the potential impact of flood levels, conservative values were adopted.

The surface infiltration loss parameters used within the TUFLOW model were provided to ATCW with the exception of the tailings closure surface. The loss parameters adopted for the closure surface were derived based on the Particle Size Distribution (PSD) of the Oxide waste presented in the Landloch materials characterization report **[11]** which were used to infer saturated hydraulic conductivity rates. The underlaying tailings were inferred as possessing the lowest infiltration rate, and therefore the governing infiltration rate for the TSF closure surface.

Rainfall loss models are generally described as an Initial Loss (IL) and Continuing Loss (CL) with the IL value being subtracted from the rainfall model at the start of a design storm and the CL being applied as a subtraction to the rainfall amount distributed evenly over time (per hour) for the remainder of the storm. The loss values used within the model for the PMP rainfall events are shown in Table **9.3**. Additionally, the model contains a buffer at the domain boundary with a high continuing loss to avoid water falsely 'ponding' against the boundary of the model domain.

Table 9.3
Infiltration/Loss Model Values Adopted For PMP

Material	Initial Loss (IL - mm)	Continuing Loss (CL - mm)
Roadways	1.0	2.0
Embankments	1.0	10.0
TSF Closure Surface	1.0	5.0
Domain Boundary Buffer	1.0	75.0
Other existing natural surfaces	1.0	2.0

9.1.3.2 PMP Spillway Assessment

To assess the closure spillway peak operational requirement, the closure surface without any allowance for contouring or bunding was adopted.

The closure surface is represented by 500 mm oxide waste cover over the LOM tailings beach and 10 m high benches with 10 m wide berms on the containing embankments and has the following key features:

- Spillway width 100m
- Spillway invert RL 321.1m
- Main embankment crest level of 324.3m (LOM crest plus 1 m of oxide waste cover)

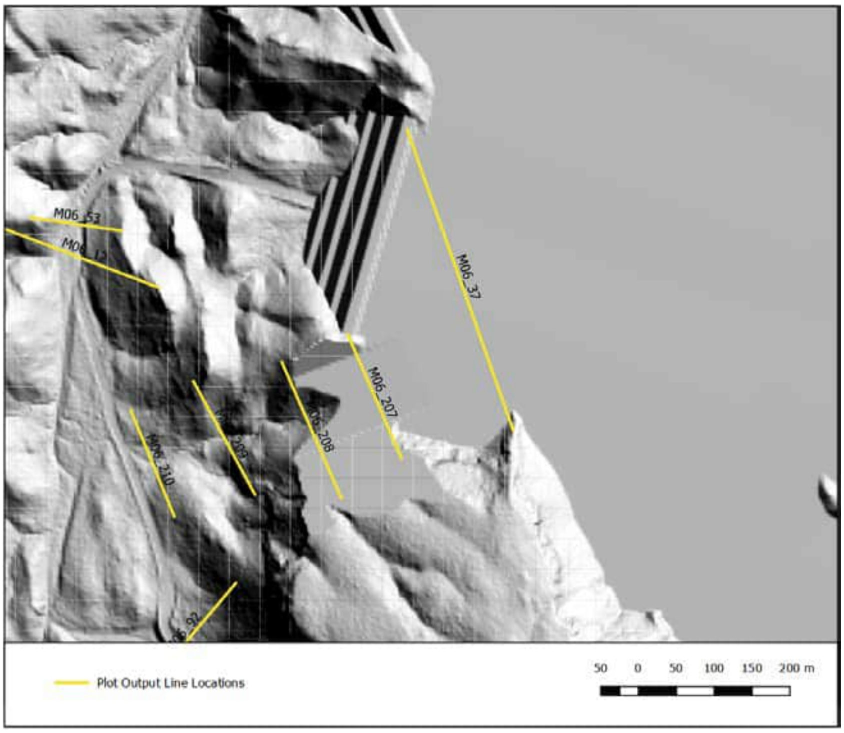
The following PMP storm durations were modelled and assessed:

- 60 minutes (1 hour)
- 120 minutes (2 hours)
- 180 minutes (3 hours)
- 360 minutes (6 hours)
- 720 minutes (12 hours)

‘Plot Output’ (PO) lines are marked within the hydraulic model at selected locations and the flow rates across these lines are recorded throughout the duration of the model runs. The PO line locations are shown on Figure 5.

PO line M06_207 was used to assess the performance of the spillway.

Figure 5
Spillway Plot Output Line Location Within Hydraulic Model



The spillway discharge flow rates modelled for the PMP storm event durations are plotted and shown in **Chart 9.2** and a summary of the modelled flow rates is provided in **Table 9.4**.

Chart 9.2
Spillway Plot Output Line (M06_207) **Maximum Modelled Flow Rate** (CUMECS)

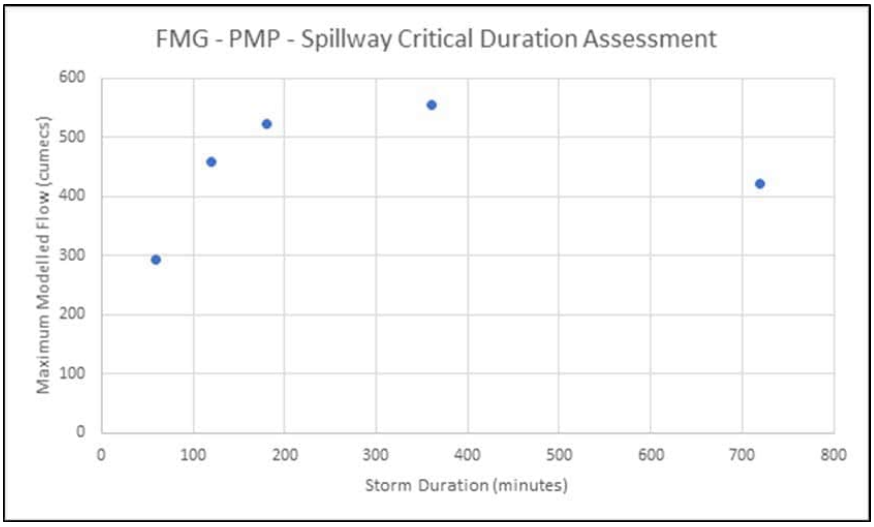


Table 9.4
Peak Modelled Spillway Discharge Rate (Po Line M06_207)

Event Duration	Peak Modelled Flow Rate (m^3/sec)
60 mins	293.06
120 mins	459.27
180 mins	523.58
360 mins	556.15
720 mins	421.51

The maximum modelled water surface elevation during the PMP storm durations was assessed to determine the freeboard available at the main embankment crest. The results of the assessment are shown in **Table 9.5**.

Table 9.5
Peak (PMP) Modelled Water Surface Elevation And Freeboard – Main Embankment

Event Duration	Peak Water Surface Elevation - PMP (RL m)	PMP Spillway flow depth (m)	Calculated Embankment Freeboard (m) - PMP
60 mins	322.92	1.82	1.38
120 mins	323.29	2.19	1.01
180 mins	323.44	2.34	0.86
360 mins	323.52	2.42	0.78
720 mins	323.21	2.11	1.09

The results of the PMP modelling indicate that the 6 hour PMP storm event is the critical case for spillway operation, giving a spillway flood depth of 2.42 m and peak flow rate of 556.15 m^3/s. The maximum flood water depth at the main embankment is 1.9 m and the storm freeboard at the embankment is approximately 0.8 m.

The sensitivity of the maximum modelled water surface elevation to the infiltration loss parameter of the closure surface was assessed by reducing the Continuing Loss parameter of the closure surface from 5mm/hr to 1mm/hr. The maximum modelled water surface elevation for the critical duration PMP event was modelled to increase by 1mm.

Peak modelled water velocity was assessed for the critical duration PMP storm event to consider whether the TSF closure surface would be subject to erosion. A summary of the modelled velocities is shown in Table **9.6**.

Velocity and depth maps are given in **Appendix D**.

Table 9.6
Maximum Modelled Velocity At Key Locations

Location	Velocity PMP (m/s)	Velocity 0.2% AEP (m/s)	Expected Surface Condition	Required Protection
TSF closure surface	0.36	0.05	Partially revegetated capping material	No specific protection required
Spillway (approach)	1.5	0.3	Partially revegetated capping material	No specific protection required
Spillway (central)	3.2	0.6	Cut into rock	None – as surface cut into basement rock
Spillway (chute)	> 10.0	4.0	Existing ground formations	Strip to basement rock material or allow to erode to basement

The predicted flow velocities across the broad, low relief closure surface are very low and below that which would be considered critical for erosion of either the oxide waste cover or bare tailings exposures. For loess earth (silt), typical critical velocities for gully initiation are in the range 0.65 m/s – 0.75 m/s and for gravel loam, velocities of 1.5 m/s to 2 m/s are generally reasonable assumptions **[37]**. For oxide waste with a D_{50} of 10 mm, a critical velocity of 1.2 m/s is estimated on the basis of grain size correlation.

The PMP 360 minute modelling output maps are included as it was determined to be the critical storm.

9.1.3.3 Surface Assessment during Frequent to Rare Rainfall events

The proposed TSF closure surface with bunding has cross beach bunds of 2 m nominal height at approximately 750 m spacing but otherwise retains the key features of the non-bunded surface adopted for the PMP modelling.

Modelling of the 0.2% AEP (1:500 ARI) 360 minute and 5% AEP (1:20 ARI) 360 minute storms was undertaken to demonstrate the effect of the proposed closure surface bunds and their modelled area of containment.

The 5% AEP storm maximum modelled depth shows almost no modelled water depth which is indicative of the low level of rainfall and high rainfall losses resulting in low modelled runoff.

The graphical outcomes of the hydraulic modelling are presented in **Appendix D.**

Table 9.7 provides reference for the result figures produced and their relevant modelled scenario.

Table 9.7
Figures and TUFLOW Modelling Results Reference Table

Figure	Title
D001	PMP 360m – Maximum Modelled Depth
D002	PMP 360m – Maximum Modelled Velocity
D003	0.2% AEP 360m – With Surface Bunds – Maximum Modelled Depth
D004	0.2% AEP 360m – With Surface Bunds – Maximum Modelled Velocity
D005	5% AEP 360m – With Surface Bunds – Maximum Modelled Depth
D006	5% AEP 360m – With Surface Bunds – Maximum Modelled Velocity

9.1.3.4 Model Limitations

It is noted that there are limitations to the modelling assessment conducted for this report. The limitations are generally based around, but not limited to, the following items:

- Rainfall estimation and impact of climate change (none made for this assessment)
- Quality of survey data
    - Survey data for large areas will often lack stream bed and bank profiles which can impact water levels, flow velocities and storage,
- Grid size used within the model can impact water levels as terrain features can be lost when smaller than the grid,
- Changes to terrain that occur during flooding can significantly alter flow paths and impact the hydraulic calculation outcomes,
- Flood waves can often create blockages of debris in narrow channels which can impact the hydraulics,
- Variability in surface conditions and materials can impact soil infiltration capacities.

9.2 Seepage Analysis

9.2.1 Tailings

To assess the likely seepage behaviour associated with consolidation and desaturation of the tailings deposit post closure, transient seepage analyses were conducted over a period of 25 years from the end of LOM, where sustained ponding on the landform surface is not applicable.

The initial phreatic surface was determined by conducting a staged transient analysis, whereby each filling stage and its operating pond (represented by a 400 mm pressure head boundary condition) were modelled incrementally up to the end of LOM. For post closure assessment, the pond boundary condition was removed.

The design report cross section adopted was taken through Main Embankment C, reflecting foundation conditions within the southern stream bed. This section had given the highest operational stage seepage estimate of 90 m^3/day (which is very low compared to the daily throughput of the TSF decants (bleed water only is approximately 16,000 m^3/day)).

Table 9.8 shows the saturated permeability estimates adopted for the embankment materials (and tailings), based on the geotechnical investigations, tailings laboratory testing, ATCW previous experience (regarding embankment Zone 2 and Zone 3 materials) and the inferred Oxide waste permeability adopted by Landloch.

Table 9.8
Saturated Permeability Estimates for TSF & RWP Embankment Materials

Material / Embankment Zone	Saturated Permeability (k_{sat}, m/s)
Oxide Waste Cover	3.3×10^{-6}
Tailings:	1×10^{-8}
Zone 2 – BGM Liner Subgrade:	1×10^{-4}
Zone 3A and Zone 3B – Embankment Rockfill:	1×10^{-5}
BGM Liner:	1×10^{-15}

The tailings material was modelled as "saturated/unsaturated" to allow for consideration of unsaturated flow and the effects of negative pore pressure development during desaturation. Based on three-point consolidation test data, a coefficient of volume change (mv) of 7×10^{-5} m^2/kPa was adopted to allow for the effects of pore space reduction during consolidation.

The boundary functions adopted for hydraulic conductivity and volumetric water content in the tailings are illustrated in **Chart 9.3** and **Chart 9.4**. Saturated and residual water contents of 0.4 and 0.05 were assumed for the tailings material. The volumetric water content function was derived using the Van Genuchten correlation with particle size and plasticity data derived from laboratory testing.

The results of the seepage analysis are given in **Appendix E** and indicate that full desaturation of the tailings beneath the former pond area would occur over approximately 25 years. Whilst a desiccated crust is expected to form rapidly on removal of the 400 mm deep decant pond, such that machine access can be gained, full desaturation of the upper part of this area is unlikely to be complete until approximately three years.

Chart 9.3

Hydraulic Conductivity boundary function – Tailings

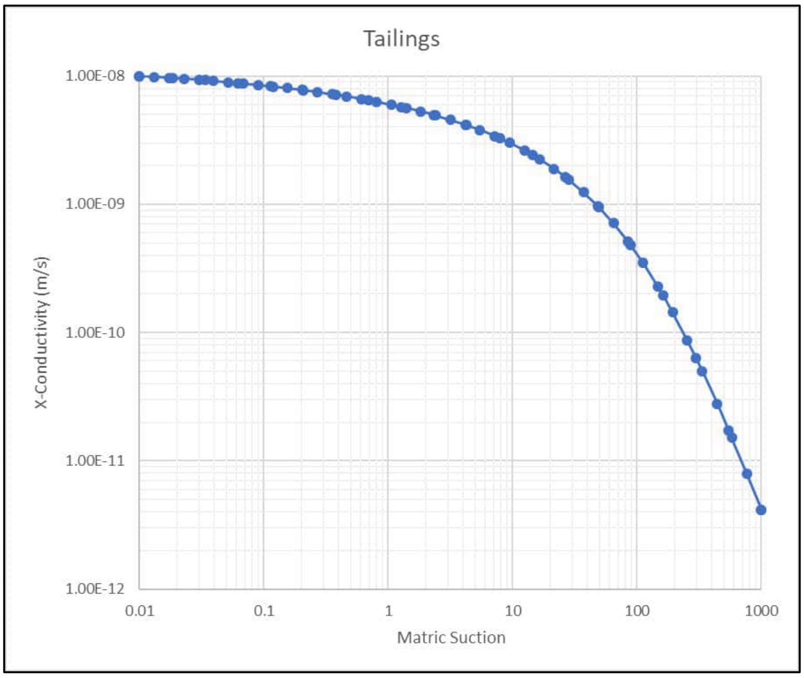
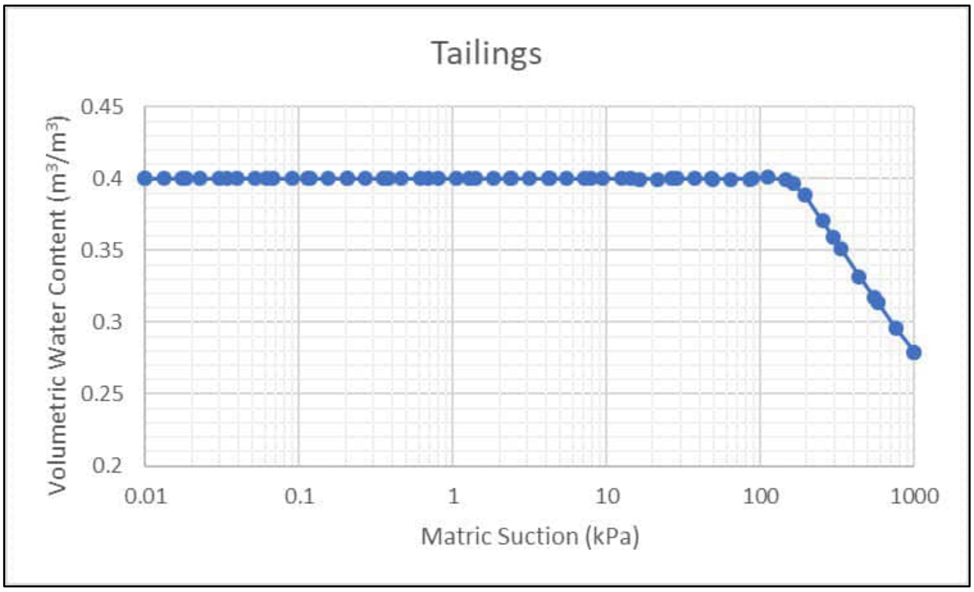


Chart 9.4

Volumetric water content boundary function – tailings



9.2.2 Closure Berms

To assess the capability of the closure slope berms to retain and infiltrate stormwater during PMP rainfall events, transient seepage analyses were conducted for successive storm durations to identify whether berm overtopping would occur.

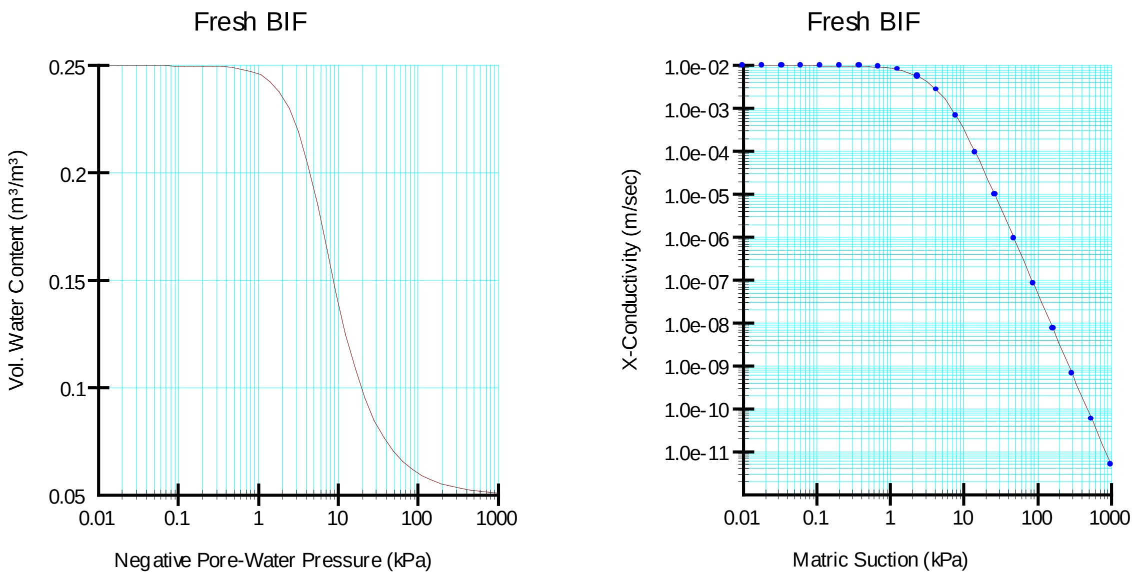
The seepage model geometry assumes that fresh waste is used to form the final embankment raise and closure benches. This material is expected to have a high degree of void interconnectivity such that infiltration and saturation of the upper layers of the material can occur rapidly. A saturated hydraulic conductivity (k_{sat}) of 1 x 10^{-2} m/s was adopted for the material (lower bound value as assessed by SRK in the geotechnical characterisation assessment [32]). For the existing embankment materials below the Stage 7 raise, the permeability values used in the seepage analyses for TSF 2A design were adopted.

Boundary conditions included:

- Development of a thin surface mesh with fine elements and k_{sat} > the applied surface rainfall flux. This allows the model to account for surface run off from the sloping bench faces onto the berms.
- Application of surface unit flux equivalent to the rainfall intensity calculated for the design storm duration.

The volumetric water content and hydraulic conductivity functions adopted for the fresh BIF waste are shown on **Chart 9.5**.

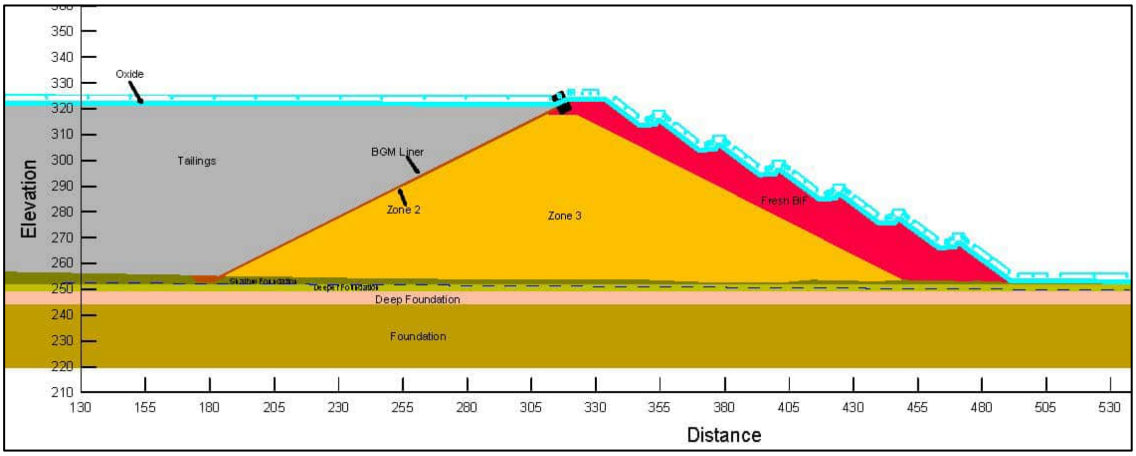
Chart 9.5
Hydraulic functions – Fresh BIF Waste



Preliminary screening was conducted by running an empirical spreadsheet model to assess the range of storm durations which could potentially produce berm overflow with the conservative assumption of no infiltration. The screening indicated that incorporation of an edge bund would be appropriate to contain run off from short duration, high intensity events where run off onto the benches could have the potential to exceed infiltration from the bench surface, while saturation of the upper surface occurs. For an adopted bund height of 1.4 m, satisfactory containment would be provided for PMP events up to 1.5 hr duration, with no infiltration.

The seepage model section is illustrated on **Figure 6.**

Figure 6
Berm Seepage Model Section



The results of the seepage analyses indicated that for longer duration events, storage and infiltration begin to dominate in the upper part of the slope profile with concurrent drainage occurring at the toe of the slope; consequently overtopping of the berms is not anticipated.

A comparison of the development of total head profiles for storm durations up to 72 hours is shown on **Chart 9.6**, illustrating that in each case, total head remains below the embankment and berm surface.

Chart 9.6
Berm Total Head Profiles – PMP

Seepage analyses outputs illustrating the extent of saturation at the end of the modelled storm events are given in **Appendix E.** Effective drainage of the water stored in the fresh BIF waste material is estimated to take between five days (6 hr event) and three weeks (72 hr event) after the storm event.

9.3 Slope Stability Analysis

9.3.1 Summary

Geotechnical slope stability analyses have been completed for the post closure embankment profile using limit equilibrium analysis in SLOPE/W software, adopting the Spencer analysis type which satisfies all conditions of static equilibrium.

The final tailings level with a “remnant” normal operating pond against the highest section of the Stage 7 TSF embankment was assumed in the analysis.

The phreatic surface conditions derived from the transient seepage analysis at the time of cessation of tailings deposition has been assumed.

Static analyses were performed to establish the long-term factor of safety (FS) against limit equilibrium failure of the downstream embankment bench profile.

As liquefaction of the embankment foundations (rock) is not anticipated, the potential for seismically induced dynamic failure under MCE conditions was assessed using the USCE method **[38]**, whereby a horizontal acceleration of half the peak ground acceleration is applied and the material shear strengths are reduced by 20%.

Embankment stability has been assessed using the following minimum FS as recommended in the ANCOLD **[21]** guidelines:

Long term (drained)	-	FS ≥ 1.5
Short term (no potential loss of containment)	-	FS ≥ 1.3
Downstream slope (seismic coefficient screening)	-	FS ≥ 1.1

9.3.2 Material Properties

The material properties used in the stability analyses are summarised in **Table 9.9**.

Table 9.9
Materials Parameters for Embankment Stability and Screening Analyses

<table>
<tr><th rowspan="2">Zone</th><th colspan="2">Static</th><th colspan="2">Seismic Coefficient Screening</th><th rowspan="2">Unit Weight, γ (kN/m³)</th></tr>
<tr><th>c’ (kPa)</th><th>φ' (Degrees)</th><th>c’ (kPa)</th><th>φ' (Degrees)</th></tr>
<tr><td>Oxide Waste</td><td colspan="4">Leps (1970) [40] Lower Bound Shear Stress-Normal Stress Function.
Note: 20% reduction factor applied for MCE Analyses[1]</td><td>20</td></tr>
<tr><td>Fresh BIF Waste</td><td colspan="4">SRK (2019) [32] Lower Bound Shear Stress-Normal Stress Function (East Limb South).
Note: 20% reduction factor applied for MCE Analyses[1]</td><td>23</td></tr>
<tr><td>Oxide or Fresh Waste (Sensitivity)</td><td>0</td><td>33</td><td>-</td><td>-</td><td>20</td></tr>
<tr><td>2A</td><td colspan="4">Douglas (2003) Shear Stress-Normal Stress Function
Note: 20% reduction factor applied for Seismic Coefficient Screening</td><td>20</td></tr>
<tr><td>2A (Sensitivity)</td><td>0</td><td>43</td><td>-</td><td>-</td><td>20</td></tr>
<tr><td>3A</td><td colspan="4">Leps (1970) Lower Bound Shear Stress-Normal Stress Function. Note: 20% reduction factor applied for MDE Analyses[1]</td><td>20</td></tr>
<tr><td>3B</td><td colspan="4">Douglas (2003) Shear Stress-Normal Stress Function.Note: 20% reduction factor applied for Seismic Coefficient Screening</td><td>20</td></tr>
<tr><td>3A / 3B (Sensitivity)</td><td>0</td><td>40</td><td>-</td><td>-</td><td>20</td></tr>
<tr><td>Tailings</td><td>0</td><td>0</td><td>0</td><td>0</td><td>16</td></tr>
<tr><td>Foundation Gravels</td><td>5</td><td>32</td><td>3</td><td>27</td><td></td></tr>
<tr><td>Foundation Rock</td><td>125</td><td>28</td><td>125</td><td>28</td><td>22</td></tr>
</table>

It is well established that rockfill shear strength is a function of the normal effective stress, dry density, particle roughness, particle crushing strength, grain size angularity and uniformity of grading. To reflect this, a number of rockfill shear strength functions are available which results in a curved strength envelope, which best represents the higher frictional strengths at low confining pressures, and the lower strengths at high overburden pressures due to suppressed dilation and particle crushing. These methods are preferred by ATCW as they allow the development of a more realistic model. To justify the use of rockfill shear strength functions, sensitivity analyses have been conducted on the TSF embankment stability, adopting lower bound constant friction angles with overburden stress.

The Douglas (2003) **[39]** and Leps (1970) **[40]** shear stress-normal stress functions were both considered in assigning material parameters to the operational embankment granular and rockfill materials. The lower bound shear stress-normal stress function derived by SRK for the fresh BIF waste was adopted for the stage 7 and closure profile outer embankment geometry.

Leps (1970) is a database method which adopts upper, average and lower bound strength functions based on varying materials of different grading, density and strength. Alternatively, Douglas (2003) adopts a more empirical approach, quantifying strength parameters based on the angularity, grading and measured Unconfined Compressive Strength (UCS) of rock material.

The Douglas (2003) method was selected for Zone 2A and Zone 3B due to the availability of inferred values to correlate to shear strength. Zone 3A was incorporated into the embankment design to accommodate blasting product from more weathered and/or finer grained rock bands encountered in quarry areas. Therefore, it was deemed suitable to adopt the more conservative Leps (1970) lower bound function, which is based on a database of comparatively lower density, more poorly graded, less durable particles.

Unit weight values for the compacted rockfill have been adopted based on past experience with compacted, well graded sedimentary rockfill.

The Douglas (2003) **[39]** method uses the following relationships:

$$\varphi'_{sec} = a + b \cdot {\sigma'_n}^{c}$$

$$a = 36.43 - 0.267ANG - 0.172FINES + 0.756(\text{Cc-2}) + 0.0459(\text{UCS-150})$$

$$b = 69.51 + 10.27ANG + 0.549FINES - 5.105(Cc - 2) - 0.408(UCS - 150) - 0.408$$

ANG	=	*Angularity rating determined by particle shape*
FINES	=	*Percent passing 0.075 mm*
Cc	=	*Coefficient of Curvature* $(d_{30}^2/(d_{10}*d_{60}))$

Particle size distribution (PSD) results from geotechnical investigations have been used for selecting parameters for Zone 2A, together with information provided by IBO. For Zone 3B, estimations were made for material PSD based on target material specifications and observations made by ATCW during geotechnical investigations and geological mapping in the area.

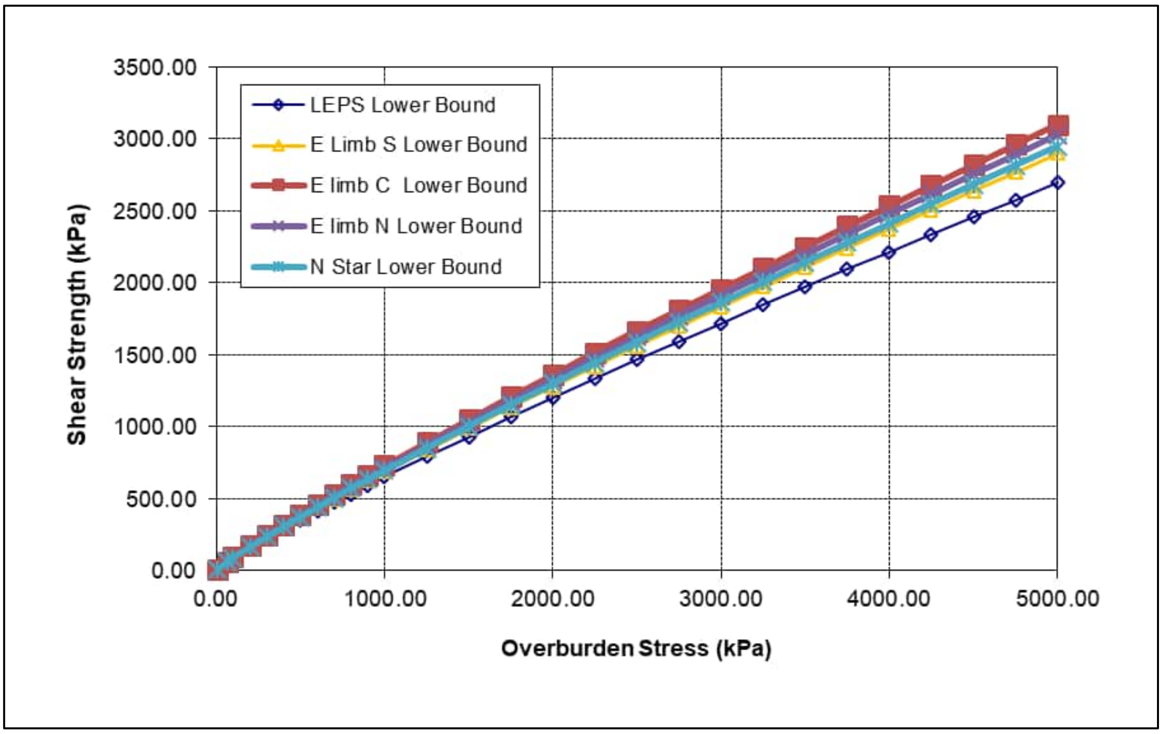
Leps (1970) has shown that shear strength, as expressed by its friction angle, varies markedly as a function of the effective normal stress.

Zone 3A rockfill and oxide waste has been assessed as being comparable to the Leps (1970) lower bound function. The rockfill which will be used in construction of the operational embankment (to stage 6) is potentially of higher quality than these estimates; however, the adopted parameters have been derived to make allowance for areas of lower strength, or more weathered materials being present in the proposed rockfill quarries.

For the stage 7 and closure berm profile, the lower bound fresh BIF waste shear stress-normal stress function derived by SRK for the East Limb South Pit waste material was adopted.

A comparison of the lower bound BIF waste strength functions with the LEPS lower bound function is given on **Chart 9.7**.

Chart 9.7
Fresh BIF Waste Strength Functions [32]



Since the TSF embankments will be downstream raised, the strength parameters for the tailings will not impact on the stability results, hence zero strength has been assigned, although a unit weight of 16 kN/m^3 has been adopted.

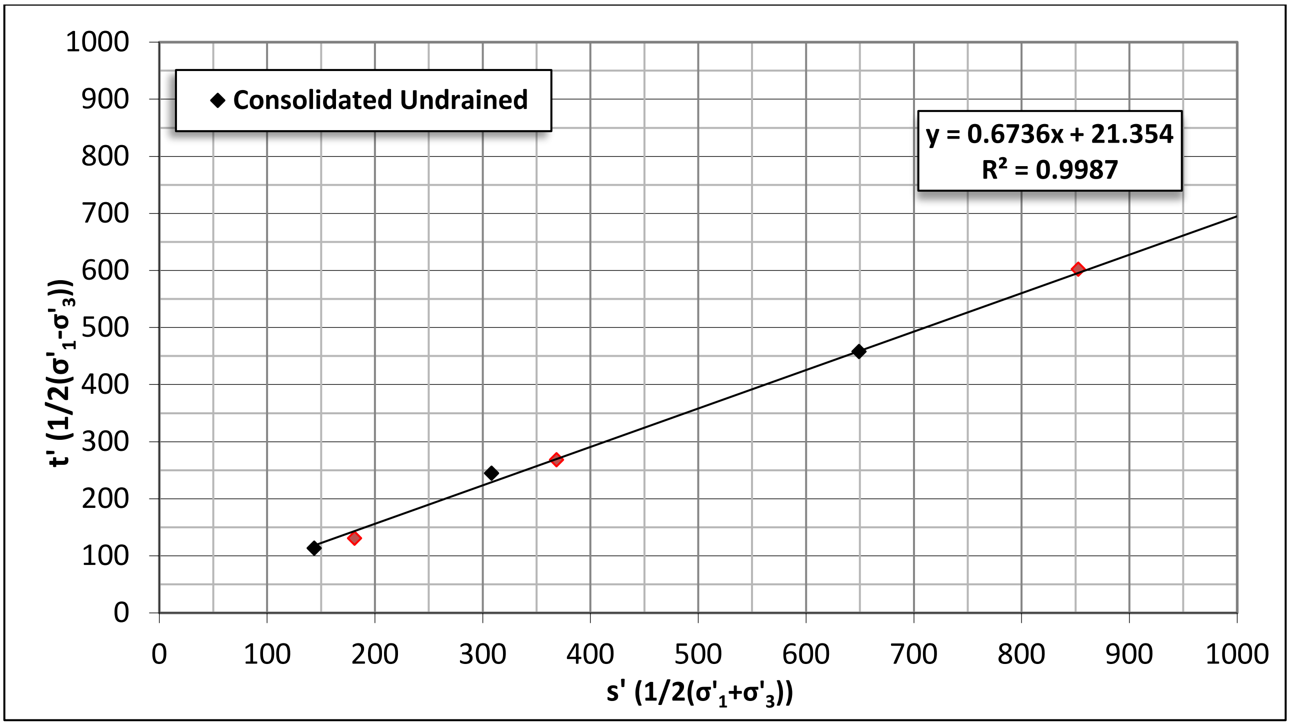
9.3.2.1 Sensitivity Analyses - Zone 2A

Single stage Consolidated Drained (CD) and Consolidated Undrained (CU) triaxial tests were conducted on proposed Stage 2 coarse rejects material by E-Precision Laboratory in December 2018. Results of these tests were provided to ATCW by IBO in January 2019. The individual test results are summarised on **Table 9.10** and the combined results are presented in the s'-t' plot on **Chart 9.8**.

Table 9.10
Summary of Coarse Rejects Triaxial Data (Zone 2A)

Test Type	CU	CD
Cohesion c' (kPa)	61	23
Friction Angle φ' (deg)	40	44

Chart 9.8
Coarse Rejects - Triaxial Test Data



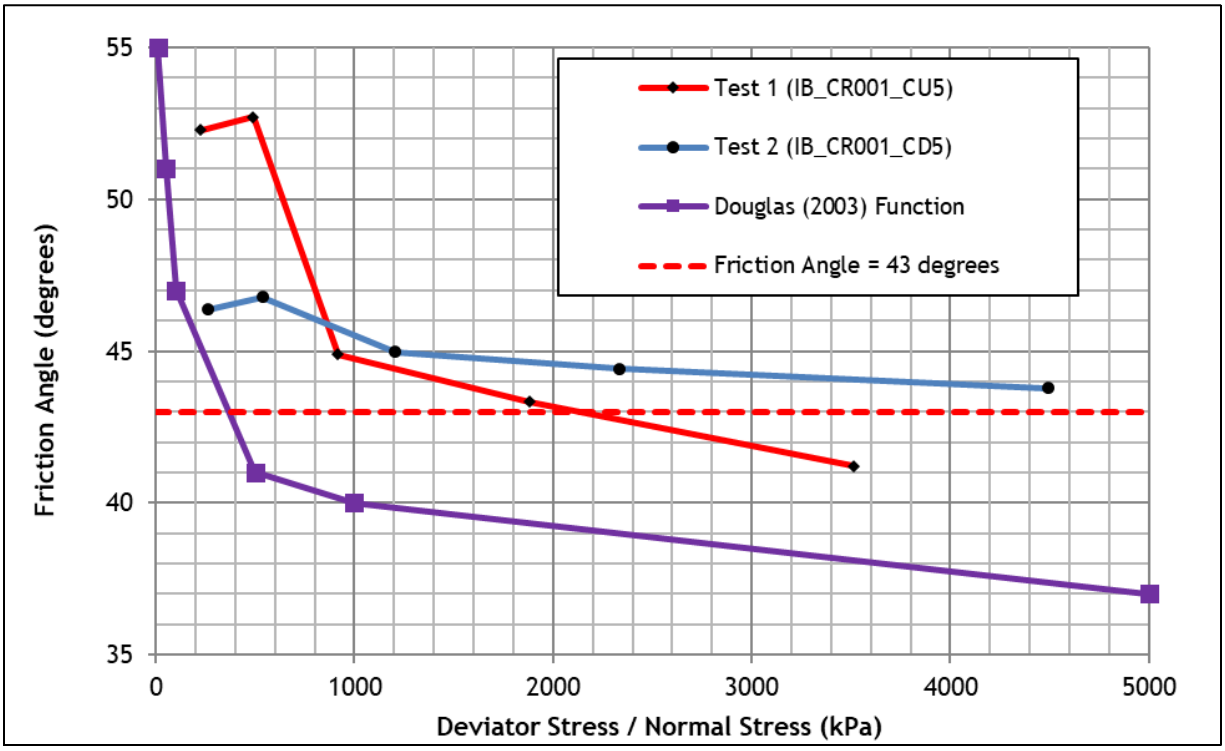
The values of cohesion and friction angle derived from the plot are c' = 26 kPa and φ' = 42°.

It is commonly accepted practice for granular materials to adjust the reported shear strength for zero apparent cohesion, which results in a higher friction angle, with a cohesion of 0 kPa. Setting the s'-t' plot trendline intercept to zero gives an inferred friction angle (φ') of 43°.

To demonstrate the correlation between the non-linear rockfill shear strength function and the above test results, **Chart 9.9** plots the friction angle for each triaxial stage with the deviator stress, together with the derived Zone 2A Douglas (2003) functions.

As evident in **Chart 9.9**, the Douglas (2003) function is appropriately conservative when compared to the triaxial data.

Chart 9.9
Friction Angle Comparison - Coarse Rejects



9.3.2.2 Sensitivity Analyses - Zone 3A, Zone 3B and Waste Materials

A conservative constant frictional angle of 40 degrees has been applied to both Zone 3A and Zone 3B rockfill zones to provide comparison with the rockfill strength functions adopted. An elevated phreatic surface was also adopted.

As for the Zone 2A analysis, this is less conservative than the rockfill strength function at depth, however more conservative when applied to shallow slip circles, which for the downstream slope are considered less significant as they will not result in loss of containment.

For the oxide waste, sensitivity was also assessed by adopting a constant friction angle; however, the approach was to vary friction angle until a FS marginally above the acceptable value was obtained. The resultant friction angle was then compared to the equivalent inferred from the Leps function (shown on Chart 9.10).

Chart 9.10
Leps (1970) Shear strength function – Friction Angle

For the fresh BIF waste, sensitivity was assessed by conservatively adopting the Leps lower bound function.

9.3.3 Embankment Stability Results

9.3.3.1 Static

A summary of the stability analyses results for the TSF embankment under static loading prior to and following closure reprofiling are presented in **Table 9.11**. The figure numbers in the table refer to graphical output of the critical failure surfaces, which are presented in **Appendix F**.

FS values for each scenario modelled satisfy the stability criteria recommended by ANCOLD. For the closure case, the effect of flattening the overall (inter ramp) slope angle from $1_V:2_H$ to $1_V:2.4_H$ results in an increased FS.

As shown in **Table 9.11**, the FS values for the sensitivity analyses are reduced slightly, however all still exceed the minimum required FS.

Table 9.11
Stability Analysis Results – TSF Main Embankment LOM And Closure Profiles

Stage	Scenario	Description	Failure	FS		Appendix Figure No.
				Required	Modelled	
7	Static	Final tailings level, Normal operating pond	Downstream	1.5	1.65	Figure F1
7	Static	Sensitivity analyses as above	Downstream	1.5	1.62	Figure F2
Closure	Static	Final tailings level, Normal operating pond with modelled transient phreatic surface.	Downstream	1.5	1.99	Figure F3
Closure	Static	Sensitivity analyses as above	Downstream	1.5	1.90	Figure F4
Closure	Static	Transient Saturation PMP	Downstream	1.3	1.43	Figure F5

9.3.3.2 Pseudo-Static (Screening of Downstream Slope)

Pseudo-static analyses have been used as an initial screening tool to assess the stability of the downstream slope of the TSF embankment, under the SEE operational case and post closure MCE loading case. Results of the seismic coefficient screening assessment satisfy the target FS, as presented in **Table 9.12**. Modification of the overall slope profile and adopting fresh BIF waste for closure results in a similar FOS for both scenarios.

Table 9.12
Seismic Coefficient Screening Results – TSF Embankment Downstream Slope

Stage	Scenario	Description	Failure	FoS		Appendix Figure No.
				Required	Modelled	
7	SEE	Final tailings level, Normal operating pond with elevated phreatic surface.	Downstream	1.1	1.12	Figure F6
Closure	MCE	Final tailings level, Normal operating pond at end of deposition.	Downstream	1.1	1.12	Figure F7

9.3.4 Liquefaction Potential

Potential liquefaction of the tailings near the embankment does not present a direct risk of embankment failure since the embankments will be founded on rock and will be constructed in the downstream direction. Nevertheless, it is considered appropriate to assess the potential for liquefaction mobilising tailings from higher up the beach profile, which if this were to occur could potentially result in embankment overtopping.

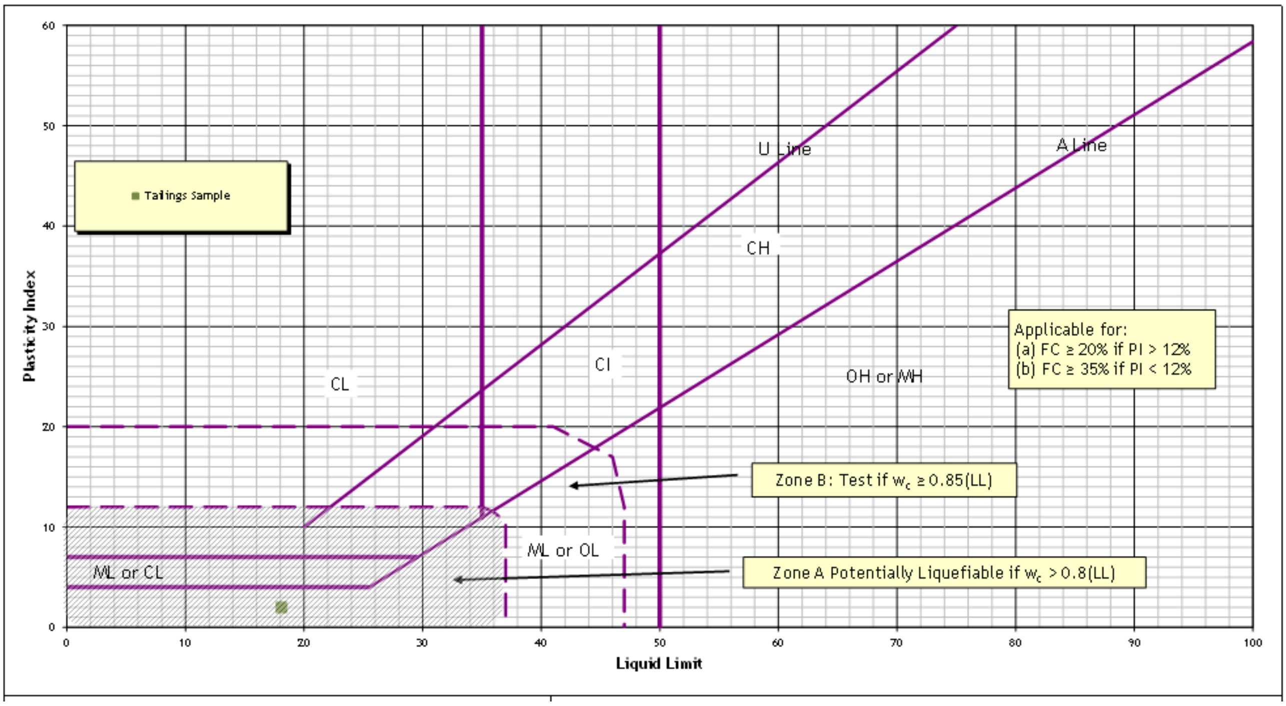
Most tailings can be susceptible to cyclic liquefaction during an earthquake event of sufficient strength if they meet some or all of the following criteria:

- Are predominantly silty or sandy, with fines of low or no plasticity;
- Are saturated; and
- Have low density and strength.

The potential for a material to liquefy under adverse induced stress conditions can be assessed using the Seed et al. **[41]** criteria is based on a database of field liquefaction performance and data from laboratory plasticity characterisation tests. The method is applicable for materials with Fines Content (FC) greater than 20%, and splits materials into three Zones based on plasticity, as shown **Chart 9.11.**

Chart 9.11

Method for Assessment of Liquefiable Soil Types - Seed et al



The three Zones are defined as:

Zone A - Materials susceptible to "classic" cyclic liquefaction;

Zone B - Materials may be liquefiable; and

Zone C - Materials not generally susceptible to "classic" cyclic liquefaction.

Screening indicates that the North Star tailings fall in Zone A, and would potentially be susceptible to cyclic liquefaction under the following conditions:

- The tailings are saturated (or at least have a moisture content greater than 85% of the liquid limit); and

- The Factor of Safety (FS) against liquefaction must be less than unity for the given earthquake. This can be calculated by comparing -

  a. The Cyclic Stress Ratio (CSR), which is a measure of the cyclic load applied to the tailings material by the earthquake (i.e. the magnitude of the design earthquake and the ground acceleration), with

  b. The Cycle Resistance Ratio (CRR), which is the capacity of the tailings material to resist liquefaction. (i.e. the density, in-situ stress conditions and the strength of the potentially liquefiable tailings).

  If the CSR is greater than the CRR, then the FS < 1.0, and liquefaction is likely to occur.

Due to the excess of evaporation over rainfall at the site, it is expected that the tailings beach will not be in a fully saturated condition, and hence not susceptible to liquefaction processes.

However, since the liquid limit of the tailings is low (20%), much of the tailings will have a moisture content in excess of 0.85 x LL and given the large catchment and potential for periods of flood water inundation, it is assumed that during operations the tailings may be sufficiently saturated to satisfy condition (i) above.

Experience indicates that for tailings deposits with a saturated tailings beach, a degree of deformation may occur during seismic events (cyclic mobility), depending on the angle of the beach, the strength of the tailings, and the intensity and duration of shaking. Continuing deformations (liquefaction flow sliding) after the end of shaking are not anticipated.

However, for verification purposes, a post-liquefaction stability analysis **[42]** has been carried out for the upper and lower slopes of the final Stage 7 tailings beach using infinite-slope theory.

For the post-liquefaction stability analysis, it has been assumed that the tailings have been deposited and dried to at least the OID of 1.66 t/m^3. A saturated bulk density, γ = 2.10 t/m^3 is assumed, given the density of water, γ_w = 1.0 t/m^3 and zero air voids moisture content = 29%.

The post-liquefaction steady-state (residual) shear strength is not constant, but is dependent on the vertical effective stress distribution within the tailings deposit. On the basis of previous ATCW test results obtained for testing of tailings samples with similar characteristics to the North Star materials, the post-liquefaction shear strength of normally consolidated tailings can be expressed as:

S_{us} = $k \times \sigma_{vo}'$, where

S_{us} = (steady-state) undrained, post liquefaction shear strength,

k = constant

σ_{vo}' = vertical effective stress

The results of laboratory shear vane testing indicate that a value of k = 0.07 is appropriate. Results from triaxial testing suggest higher values of k, but for this analysis, we have conservatively taken the lower value.

The equations derived from the stability of long, shallow slopes ("infinite slope theory") may be used to analyse this situation. The concave beach slope profile inferred during design has been adopted. Two cases may be considered:

1. Upper Slope

For the upper, steepest portion of the beach:

Slope angle β = 0.4% (sin β = 0.004)

It is conservatively assumed that the slope is saturated, but does not have a phreatic surface. For this condition, the stability equation for the Factor of Safety (FS) reduces to:

$$FS = \frac{k}{\sin\beta} = \frac{0.07}{0.004} = 17.5$$ i.e. the slope will remain stable.

2. Lower Slope

For the lower, flatter portion of the tailings beach:

Slope angle β = 0.23% (sin β = 0.0023)

In this case it is assumed that the slope is fully saturated, and has a phreatic surface at the top of the tailings (i.e. has applied hydrostatic pore pressures). This situation would occur below the decant pond. For this condition, the equation for stability is:

$$FS = \frac{k}{\sin\beta}\left(1 - \frac{\gamma_w}{\gamma}\right) = \frac{0.07}{0.0023}\left(1 - \frac{1}{2.1}\right) = 15.9$$ i.e. the slope will remain stable.

Whilst the tailings physical characteristics suggest they are potentially liquefiable, the post liquefaction stability assessment shows that the adopted tailings beach slopes will remain stable, even when hydrostatic pore pressure conditions are assumed after seismic-induced liquefaction.

Following capping and closure, the tailings beach will become desaturated below the capping layer, further reducing the likelihood of cyclic mobility occurring.

9.4 Deformation Analysis

It is implied from the outcome of the limit equilibrium stability analyses that deformations resulting from the MCE seismic event would be insignificant for the TSF embankment.

Empirical database methods have been used to assess the seismic deformation of the TSF embankment profile. In accordance with ANCOLD **[21]**, empirical database methods are considered applicable in cases where there is no potential for liquefaction or significant strain softening, as is the case for the North Star TSF.

The embankment foundations or materials are not susceptible to liquefaction or significant strain weakening. Hence the following simplified empirical database methods have been utilised for this assessment:

* Swaisgood (1998, 2003) **[43][44]**
* Pells and Fell (2002, 2003) **[45][46]**

The results of Swaisgood (1998, 2003) estimates MCE crest settlements of less than 56 mm, which indicates a relative degree of damage of “None” to “Minor” based on **Chart 9.12**.

The empirical database method developed by Pells and Fell (2002, 2003) uses a larger database than Swaisgood (1998, 2003). The method uses two correlations, one for earthfill dams and one for earth and rockfill dams. The latter correlation has been adopted for this analysis. **Chart 9.13** shows that the TSF Embankment Damage Class Number plots as Class 1. **Table 9.13** provides an interpretation of the Damage Class Number.

Chart 9.12
Swaisgood (2003) Relative Crest Settlement Vs. MCE Peak Ground Acceleration

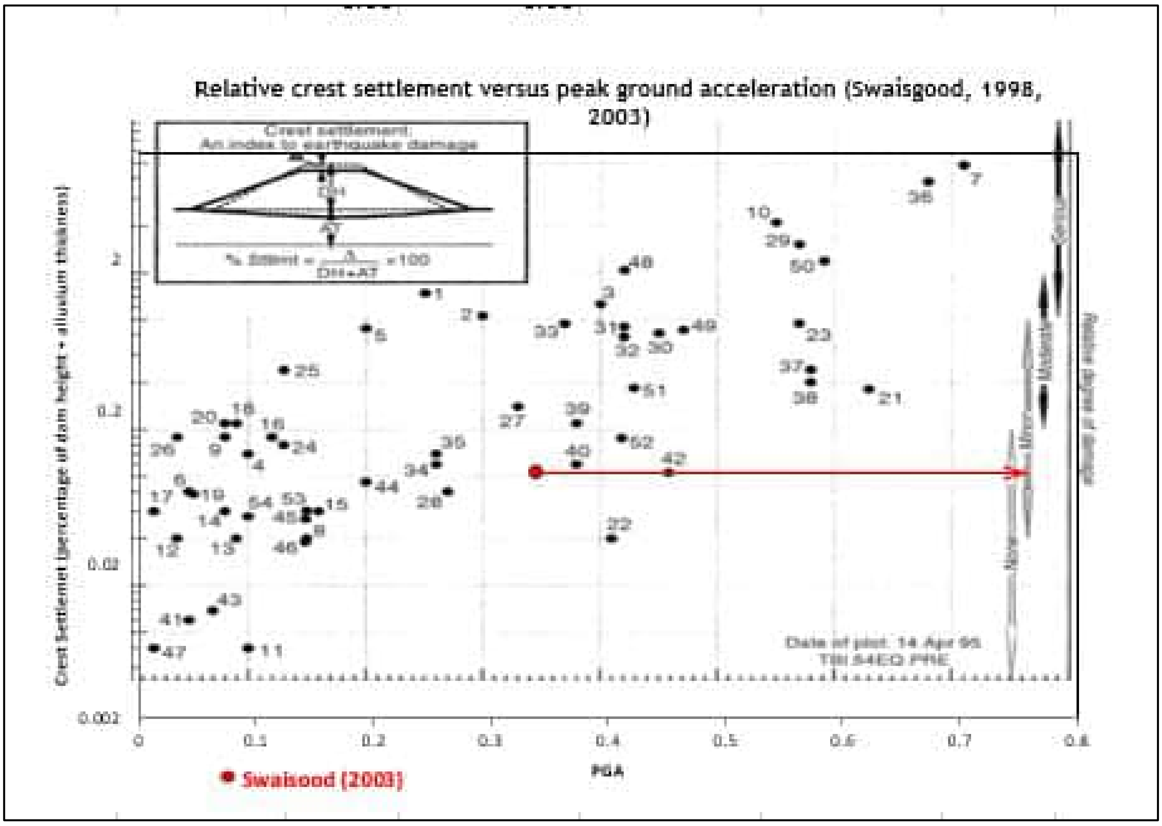


Chart 9.13

Contours of Damage Class Vs. Earthquake Magnitude and PGA (Pells and Fell, 2003)

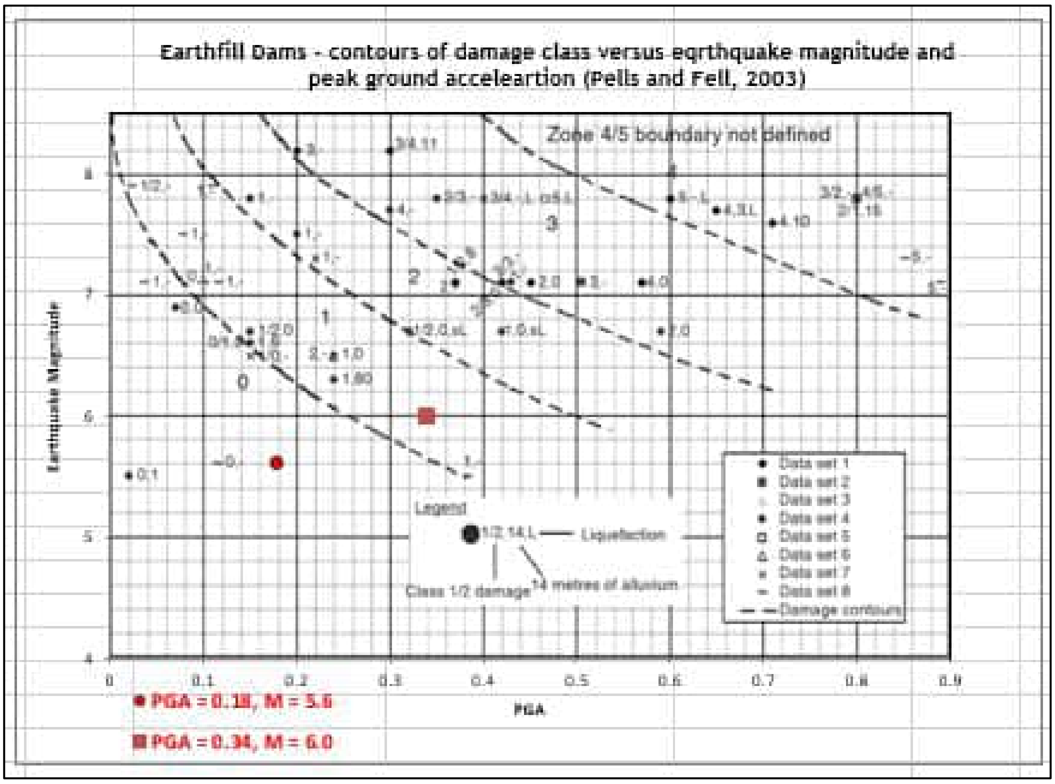


Table 9.13

Damage Classification System (Pells and Fell, 2003)

Damage Class Number	Description	Max. Longitudinal Crack Width (mm)	Max. Relative Crest Settlement (%)	Estimated Settlement (mm)
0	No or Slight	< 10	< 0.03	0 to 19.5
1	Minor	10 to 30	0.03 to 0.2	19.5 to 130
2	Moderate	30 to 80	0.2 to 0.5	130 to 325
3	Major	80 to 150	0.5 to 1.5	325 to 975

By comparison, the Pells and Fell (2002, 2003) method appears to produce a slightly more conservative result than Swaisgood (1998, 2003), however the maximum estimated settlement for class 1 is only 130 mm, which would not impact on the post closure functionality of the TSF.

The reprofiled LOM TSF embankment with has a minimum freeboard to spillway invert level of 3.4 m. Evidently, there is a significant margin between the amount of seismic deformation predicted by the empirical database methods (maximum 130 mm) and the embankment freeboard.

Hence, in accordance with ANCOLD **[21]**, the use of empirical database methods is considered a sufficient level of study and it is not considered necessary to apply more complex numerical methods for estimating deformations during earthquakes.

9.5 Dam Break Analysis Review

A dam break analysis was performed for the purpose of TSF 2A design. The outcomes of the assessment may also be considered relevant to the post closure case, although the risk of dam failure will be significantly reduced in the absence of stored water (particularly as the RWP will have been removed) and as the tailings desaturates.

The area of impact downstream of the TSF is largely undeveloped with only road and rail transport infrastructure and potential small mining developments possibly in the path of a tailings and water release.

In the event of a dam break, the direction of flow would be to the north-west along existing drainage channels (Chinnamon Creek) for approximately 45 km until it intersects with the tributary of the Turner River. The tributary crosses the Pilgangoora mining area access road, Pippingarra Road, the Port Hedland – Wittenoom Road and flows beneath three railway bridges (BHP, FMG and Roy Hill) before it flows into the Turner River.

The area along Chinnamon Creek and the tributary of Turner River provides habitat for a number of significant fauna species, and there are also individual specimens of Declared Rare Flora (DRF) in this area.

Regarding major infrastructure in the downstream receiving area, the following is a summary:

- North Star borefield road (immediately downstream of the RWP Embankments). This would not be operational at closure;
- The Pilgangoora mining area access road (Wodgina Road), approximately 28 km north-west;
- Pippingarra Road and the adjacent Port Hedland – Wittenoom Road (approximately 37 km north-west); and
- Three railway crossings (BHP, FMG, Roy Hill, 37 km to 39 km north-west).

The Sunny Day Failure (SDF) condition was considered to be the critical case for the facility under operating conditions and this remains the case for the post closure scenario.

The analysis conducted for TSF 2A design was a two-stage process:

(iii) Firstly, the initial loss of impounded decant pond water (5,100 ML of RWP water, plus 3,700 ML of decant pond water in the TSF); followed by

(iv) The run-out of the flowable portion of the deposited tailings through the breach.

For the closure scenario, the maximum water volume that could have been stored by the RWP embankments (5,100 ML) will not be available to contribute to failure, consequently the maximum flow depths and velocities predicted for the operational case situation would be greatly reduced.

As a result, maximum flood flow depths of less than 900 mm are inferred with maximum flow velocities of less than 1 m/s. This level of flood velocity would result in potential for only minor scouring and erosion, with limited potential for vegetation stripping or bulk transport of solid particulates.

The analyses for tailings run out performed for the operational case remains valid for the closure scenario (although the likelihood of breach occurring is reduced since the tailings will gain strength on consolidation). The analysis indicated that the top surface of the tailings runout will most likely conform to an average equilibrium longitudinal gradient of 2.4%. For the adopted breach scenario, the tailings are expected to flow of the order of 1,600 m downstream of the TSF Main Embankment, passing through the former RWP Embankment area. This equates to an inundation area of 128 ha.

As determined for the operational case analysis, It is considered that the level of risk that people will be in the path of the breach flood flow at the time of the breach is very low, nevertheless, based on the potential for vehicles crossing Chinnamon Creek on the Pilgangoora access road, it is considered that a PAR of 1 to 10 should be conservatively maintained for risk assessment purposes.

9.6 Water Balance Review

A comprehensive water balance model was developed by ATCW for the operational TSF 2A case, to investigate specific design parameters for the TSF and RWP.

The stochastic climate data realisations (1000 years) developed for the model have been re-run for the closure case geometry to investigate the patterns of pond development that could eventuate behind the cross-contour berms.

The key objective of this assessment were to;

(i) Determine the likelihood of the closure embankment spillway being engaged;

(ii) Identify the optimum locations for establishment of vegetation on the closure surface.

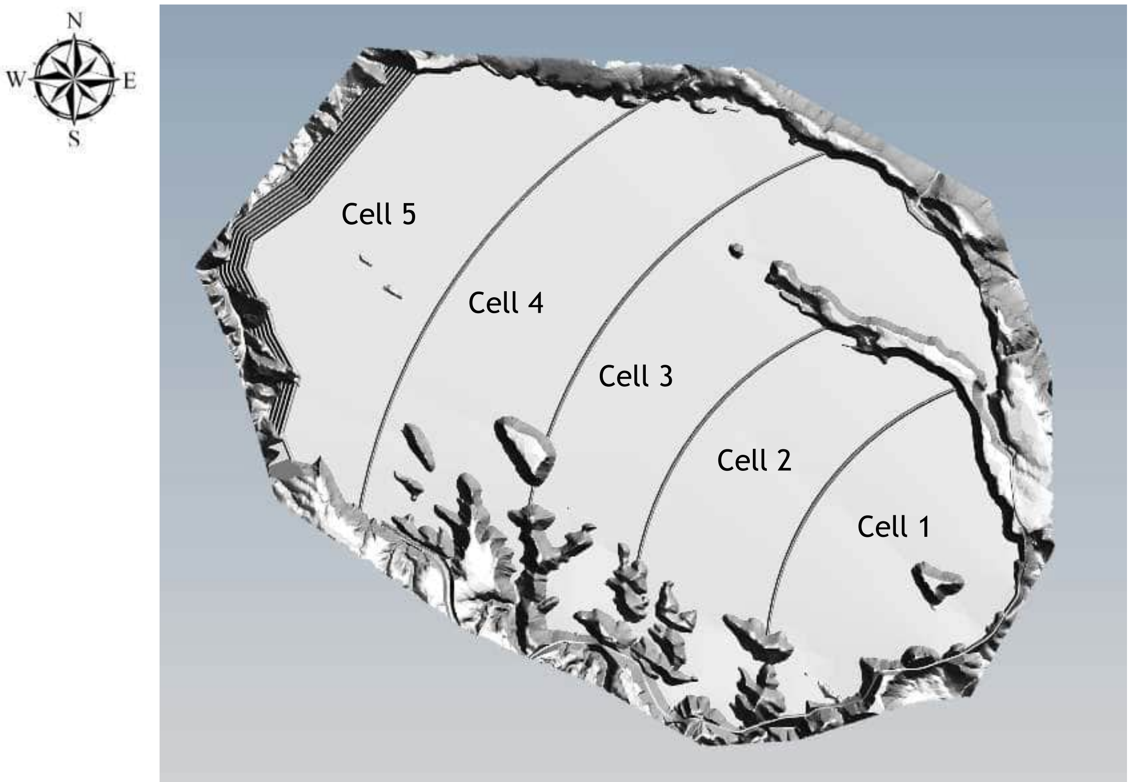
A detailed presentation of the water balance methodologies, inputs and outcomes is provided in Appendix **E** of the TSF 2A design report **[2]**.

At closure of the facility, the RWP embankments will be removed and tailings slurry will no longer be discharged to the TSF. To investigate the likely frequency of water ponding on the closure surface, the TSF design water balance model was modified and re-run through the full sequence of stochastic rainfall simulations adopted for design.

Each realisation represents the 25 year period after mine closure. The realisations were stepped through the 1,000 years of synthetic rainfall and evaporation data. For example, the climate data for Realisation 1 starts at Year 1 and ends at Year 25; the next realisation in the model then progresses into the following 25 years of climate data starting from Year 2.

Pond storage curves have been developed for each of the component storage areas on the closure surface cells defined by the cross tailings bunds as shown in **Figure 7.**

Figure 7
Cell Configurations



Each cell is divided to three areas with respect to infiltration characteristics, comprising the natural catchment area, dry surface area and pond area. For the natural catchment at the edge of the cells, the Boughton SFB yield model parameters adopted for the operational case water balance were adopted. For the ponded areas, a run off coefficient of 1 was adopted and for the closure surface, infiltration was estimated on the basis of flux estimates from the seepage analyses described in **Section 9.2**.

An inferred seepage flux of 7×10^{-9} m^3/s per unit area (equivalent to 0.6 mm/day) beneath ponded areas and through the cross bunds was adopted for initial analysis. This may be considered conservative since the tailings surface is assumed to be unsaturated. The seepage rate was multiplied by the pond area at any particular time to estimate the amount of seepage exiting the ponds.

Two representative individual realisations were selected among the stochastic outputs for the purpose of investigating possible scenarios after the closure of the North Star TSF. The selection was made by statistically analysing the total rainfall (25-year sum) for all 1,000 realisations.

These individual realisations include:

- Realisation 377 - representing the median (50 percentile) total rainfall case (9,136 mm in 25 years); and
- Realisation 748 - representing the maximum total rainfall case (13,821 mm in 25 years).

The stochastic water volumes, elevations and spillway flows for the initial analysis are presented in **Appendix G.** The bunded cell areas tend to follow seasonal patterns of filling and emptying.

Bund overtopping did not take place in any of the realisation runs and partial flows across the spillway as a result of Cell 5 rainfall and base flow transfer through the oxide layer occurred less than 0.2 % of the time.

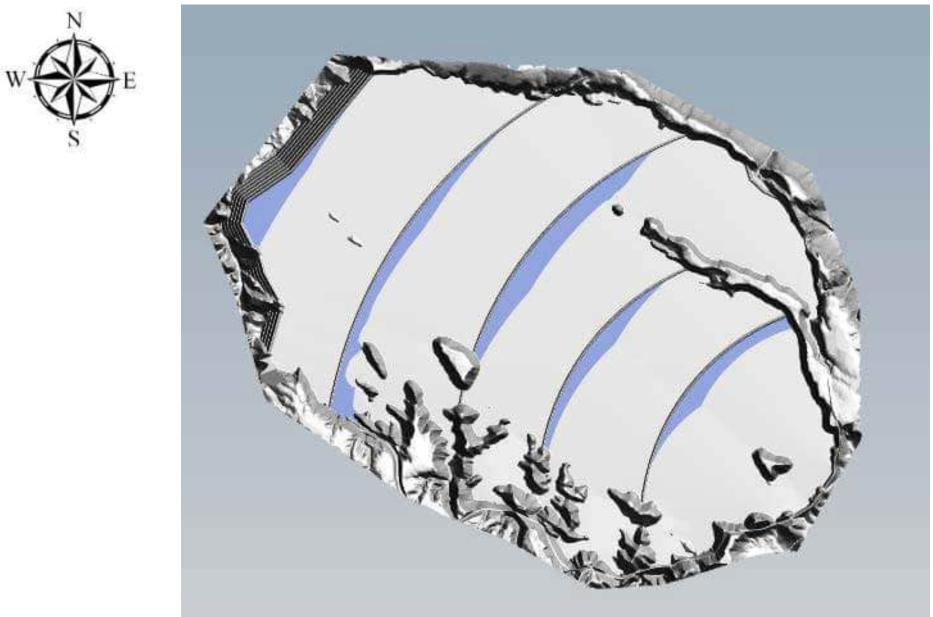
A summary of predicted spillway flow frequency is given in **Table 9.14**.

Table 9.14
Spillway Flow Frequency

	Realisation #377	Realisation #748
Spill Frequency (Days)	2	11
Spill Percentage Over 25 Years	0.02%	0.12%
Total Spill Volume (m^3)	5.38×10^5	4.66×10^6
Average Spill Volume (m^3/day)	59	511

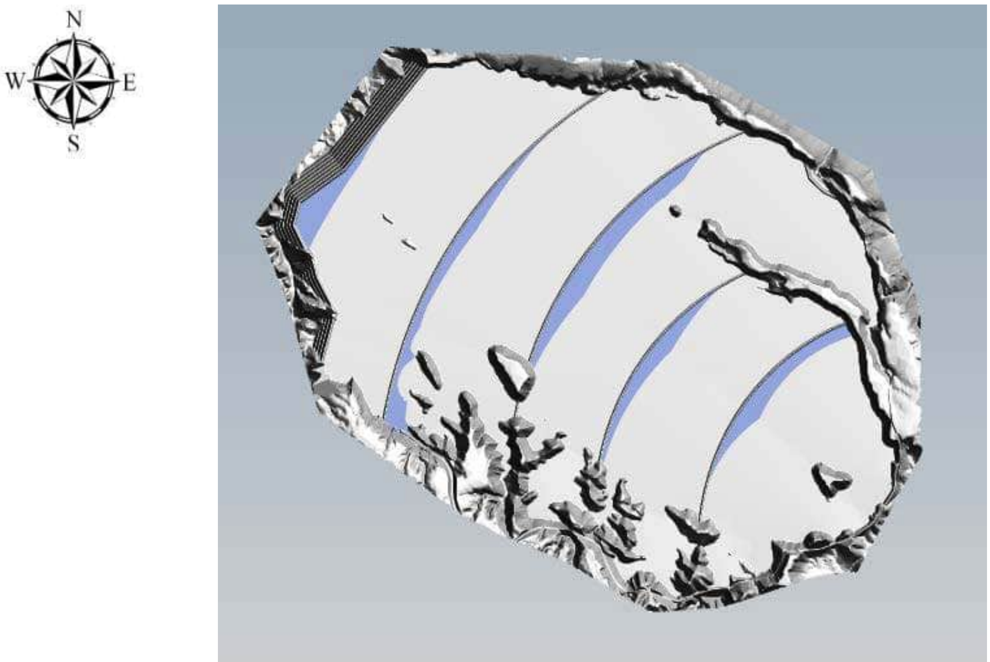
To visually illustrate the median distribution of ponding on the closure surface where the underlying tailings are assumed to remain unsaturated, the inferred median wet season pond extents for the Realisation #377 case are shown in **Figure 8**. The pond width is approximately 120 m. Pond depth is in the range 0.2 m to 0.4 m.

Figure 8
Median Pond Extents



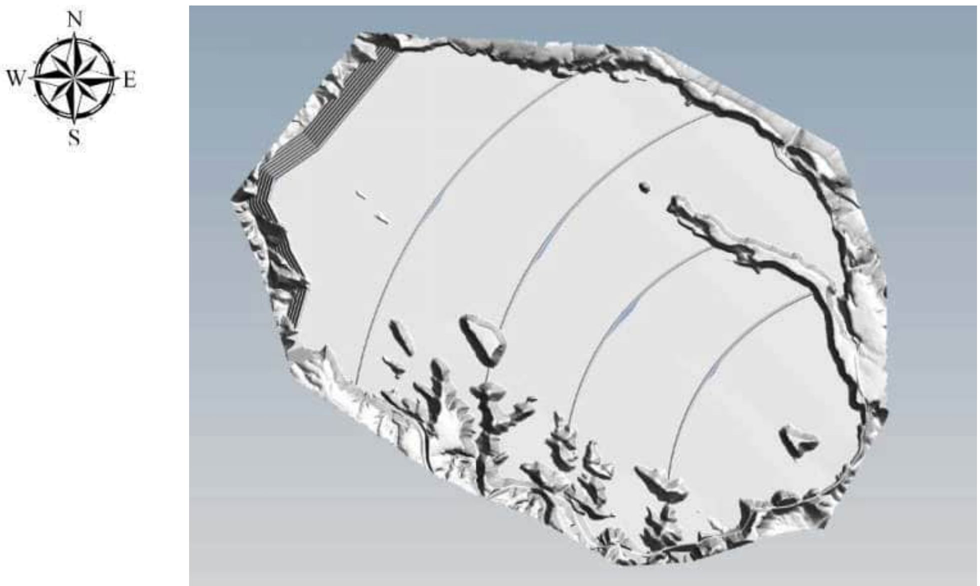
To investigate the effect of assumed seepage rate on pond volumes, the seepage model in Section **9.2** was run with an assumed permeability of 1.15×10^{-5} m/s for the cover material. The resultant estimated vertical flux of 1.14×10^{-8} m/s was adopted to rerun the water balance model. The outcome was to decrease the inferred median wet season pond width by approximately 20 m as illustrated on **Figure 9**.

Figure 9
Median Pond Extents – Cover Permeability 1.15 x 10^{-5} m/s



The case where the underlying tailings become saturated (towards the latter part of the wet season) was also assessed. The resultant seepage flux at the base of the ponds increases to 1.5 x 10^{-7} m^3/s per unit area and median pond development is significantly restricted as illustrated in **Figure 10**, with ponding being restricted to the central bund areas. The median pond depth fluctuation is illustrated on **Figure G4** of **Appendix G**.

Figure 10
Median Pond Extents – Saturated base soil (tailings)



To assess the likely range of ponding behind the bunds in more detail for the purpose of vegetation assessment, small scale trial tailings, cover and bund areas should be developed at the head of the tailings deposition areas during early operations. In addition to monitoring the rate of dissipation of ponding following site rainfall events, the areas could also be artificially irrigated or flooded periodically to establish the range of seepage rates expected for various design storm cases.

For conservative preliminary assessment, a flood buffer width of 100 m from the rear of the bunds is suggested.

Attenuation of flood water behind the bunds will provide a reservoir of water to release moisture via seepage and baseflow to the upper reaches of the cell below the bund, which are likely to present enhanced opportunities for sustained vegetation development.

9.7 Schedule of Quantities

A preliminary schedule of quantities detailing the materials required for earthworks to complete the closure and rehabilitation of the facilities is included in **Appendix H.**

A summary of quantities required, in comparison to the inferred material availability is given in **Table 9.15**. More refined estimates of material availabilities will be developed during completion of the mine plan; however, it is apparent that the availability of waste materials will far exceed the requirements for TSF closure.

Table 9.15
Material Balance

Material	Required	Available
Topsoil/Subsoil	407,000 m^3	407,000 m^3
Coarse Rejects	Blending option	111 Mm^3
Fresh BIF Waste Rock	Embankments 0.97 Mm^3	160 Mm^3
	Surface Cover 4.97 Mm^3	
Oxide Waste Rock	Blending option	160 Mm^3

10 CLOSURE IMPLEMENTATION

10.1 Summary

Implementation of the closure process will take place incrementally throughout the operational life of mine, culminating in decommissioning of tailings delivery infrastructure and appurtenant structures.

The envisaged stages of implementation are:

1. Development of a research, investigation and trial plan;
2. Monitoring, maintenance and observation of landform behaviour during operations stage – water, dust, erosion stability;
3. Implementation of progressive closure works where and when possible;
4. Pre-closure works – final stages of TSF construction;
5. Commencement of closure earthworks – re-profiling of final embankment batters;
6. Deposition of tailings ceases

7. Decanting of excess water in operational TSF pond;
8. Commencement of cover placement from head of beach;
9. Completion of closure earthworks;
10. Closure auditing
11. Final closure report
12. Site relinquishment.

10.2 Research, Investigation and Trials

It is envisaged that data collection, material characterisation and rehabilitation pot trials will be ongoing throughout operations.

As the TSF embankments will be constructed in the downstream direction, any trials on the as-constructed stage batters will be restricted in duration; however, a long-term trial area could be constructed in the final stage 7 downstream toe area, to ultimately be incorporated into the final closure profile.

10.3 Progressive Rehabilitation

Towards the end of deposition, progressive rehabilitation of the tailings beach head and strategic areas of the LOM beach may be implemented by extending the spigot lines across the beach head.

Deposition could potentially be concentrated in the smaller norther arm of the valley, such that closure of this area could take place in advance of completion of filling in the main valley area.

Targeted tailings discharge to strategic parts of the lower beach may also be implemented to reshape the tailings surface prior to cover placement. Progressive rehabilitation in a mine scale context may include creation of a new waste dump at the beach head area, thus removing the need to clear virgin ground for waste dump siting.

10.4 Premature Closure

In the event that TSF 2A is closed prior to completion of stage 7 filling, the detailed closure design will require modification; however, the closure strategy described for the LOM case remains valid, such that closure implementation for premature closure will comprise:

- Removal of the RWP embankments and rehabilitation of the RWP pond footprint;
- Removal of all tailings delivery infrastructure;
- Placement of 500 mm oxide waste cover and cross beach bunds on the tailings surface;
- Re-profiling of downstream batters to form 10 m high x 10 m wide benches using oxide waste material;
- Widening and deepening of the operational stage spillway at the time of closure;
- Rehabilitation of access roads.

Where closure occurs in the early part of stage filling, it is likely that cut and fill embankment re-profiling will include removal of the upper part of the embankments for use as fill lower down the slope. The final embankment crest elevation will be such that the closure landform and spillway can cater for the critical PMF event.

11 CLOSURE MONITORING AND MAINTENANCE

11.1 Monitoring Regime

11.1.1 Groundwater

Monitoring of groundwater levels and quality (such as contaminant levels, pH, and total dissolved solids) will continue on a monthly to quarterly basis at the existing and proposed monitoring bores which will be associated with TSF 2A for a period necessary to demonstrate that the closure completion criteria are being met.

An embankment piezometer network is also proposed for the facility. At the TSF embankment, three sets of Vibrating Wire Piezometers (VWP) will be installed, two sets in the foundations of TSF Main Embankment A, and one set in the foundations of TSF Main Embankment C. The VWP sets will each be comprised of 3 x VWP's located within the embankment footprint. The two sets located within TSF Main Embankment A will each include an additional VWP located directly upstream of the Key Trench. The purpose of the upstream VWP's will be to monitor excess pore pressures in the tailings beach, and hence enable evaluation of drainage and consolidation performance.

The proposed arrangement, and sections and details of the VWP's and the RWP downstream monitoring bores are shown on **Figures 114185.14_025** and **114185.14_026** of the TSF 2A Design report, respectively.

11.1.2 Deformation, Erosion and Sediment

A series of survey movement monitoring prisms will be installed on the crests of the RWP North and RWP South embankments to monitor any embankment deformation.

On completion of closure profiling, it will be necessary to install new prisms on the embankment crest and additional monitoring stations should be installed at strategic locations on the closure surface and embankment batter berms. Although these instruments are generally used to monitor settlement, they can also be used as datum references for quantitative measurement of sediment build up and/or erosion. In areas targeted for erosion measurement, the prisms should be installed on stainless steel rods, embedded to at least 1 m depth.

Geotechnical investigation and stability review should be carried out if the settlement or sediment build up exceeds 10 mm quarterly.

The proposed locations of survey movement monitoring prisms at closure are presented **Figure 114185.15_010.**

11.2 Maintenance and Assessment

11.2.1 General

Post closure maintenance activities will be minimal provided the closure implementation is successful completed.

In addition to post closure instrumentation monitoring and groundwater sampling, such activities will necessarily involve regular inspections and audits of the final landform to validate its performance under applied climatic conditions.

Some degree of seepage and surface water management may be required based on the outcomes of monitoring or observations made after heavy rainfall events.

11.2.2 Assessment and Auditing

Assessment and auditing involve the monitoring of set criteria, with associated maintenance and re-adjustment of closure strategy if required. This means that if an implemented closure strategy is demonstrated to not be achieving the desired outcome, that strategy should be changed – perhaps with some additional works, to meet the required outcomes.

For TSF's in Western Australia, DMIRS **[24]** require the preparation of a final TSF specific closure plan to demonstrate how long-term objectives will be achieved based on the final status of the TSF. Prior to the submission of the TSF closure report, a decommissioning and rehabilitation review should be conducted by a competent person to provide an engineering status report of the TSF to assist with planning for closure of the TSF. This would normally be expected to be a "live" document developed in association with the annual geotechnical audit of the facility.

11.2.3 Seepage Management

It is proposed to install a seepage monitoring trench and contingency recovery sump directly downstream of each RWP embankment (where the greatest potential for seepage exists).

Ground water monitoring bores proposed immediately downstream of the RWP will be drilled to sufficient diameter and screened to allow conversion to seepage recovery bores as a contingency in the event of seepage occurring around the South embankment abutments or through the hill between the South and North embankments. Contingency seepage monitoring and collection trenches and sumps will also be provided.

During the course of operations, should seepage beneath the TSF embankments be identified through monitoring, similar contingency measures may be implemented closer to the TSF embankments.

At closure, it is envisaged that any seepage mounding associated with the operational stage will cease and water levels will begin to return to baseline levels. Hydraulic connectivity between seasonally ponded water on the landform surface and the receding groundwater will be broken as the upper tailings desaturates.

11.3 Relinquishment

Relinquishment of the TSF 2A landform requires formal regulatory acceptance that all obligations under the mine closure plan, including achievement of completion criteria have been satisfied **[47]**.

Where any ongoing requirements for future management and maintenance are defined, these must be agreed to by the subsequent land owners.

Such agreement, including legal aspects must be formally documented and approved by the regulators. To be recognised under the contaminated sites act 2003, the written approval of the CEO of the Department of Water and Environmental Regulation is required.

It is understood that DMIRS are currently reviewing its formal relinquishment process, to be incorporated in future issues of the guidelines for preparing mine closure plans.

With respect to Part V of the environmental protection act, a closure notice may be issued requiring ongoing monitoring and management of a decommissioned facility after a licence has ceased to have effect.

It is envisaged that monitoring data, materials characterisation data, rehabilitation trial data and audit information collected during TSF operation will be used to fully define the TSF 2A completion criteria for closure, including definition of the measurement tools used to demonstrate compliance following substantial completion of closure implementation.

12 RISK ASSESSMENT

A detailed, quantitative risk assessment is proposed for the TSF 2A design, which by extension will be required to address risks associated with closure of the facility.

It is intended that the closure design documentation will be updated on completion of the assessment which should include the following potential risks for consideration:

Embankment failure
Erosion of embankment batters and berms
Erosion damage to spillway
Erosion and sediment movement off closure surface
Dust control
Management of PAF material during operations (waste and tailings)
Availability of closure construction materials
Topsoil fate and availability for TSF closure
Cover material suitability
Groundwater management and seepage mitigation
Surface water management
Vegetation establishment and support

13 FUTURE DESIGN AND INVESTIGATION WORK

It is a critical requirement that verification of the fundamental TSF design parameters be undertaken at defined intervals during facility operation to reconcile the performance of the TSF with design expectations. Where possible, similar validation of adopted closure performance design assumptions should be undertaken.

It is considered that annual intervals coinciding with the annual surveillance audits would be appropriate for routine checks to be made on tailings properties. Parameters to be assessed should include:

- Sampling and laboratory characterisation of tailings particle size distribution, specific gravity, Atterberg limits, initial settled density and shrinkage limit density;
- Tailings beach slope and insitu dry density;
- AMD potential (SPOCAS or SCr suite of analyses)
- Tailings production rates (to enable calibration of TSF filling rates and ultimate filling level); and
- Water inputs (rainfall runoff, paste thickener overflow) and outputs (evaporation, return water pumping) to enable calibration of water balance parameters.

Regular geotechnical investigation to assess the in-situ tailings strength and compressibility profile, composition and degree of saturation should also be undertaken by CPTu probing, perhaps on a two-yearly cycle.

During Stage 1 deposition, a tailings sample should be taken for Rowe Cell consolidation testing to confirm the tailings consolidation parameters and enable calibration of TSF filling rates, ultimate filling level and magnitude and rate of settlement that may be expected during operations and post closure. It is likely that this testing would be one off, however if tailings characterisation changes throughout the LoM, additional consolidation testing may be necessary during subsequent verifications.

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DRAWINGS

IRON BRIDGE OPERATIONS PTY LTD
NORTH STAR MAGNETITE - STAGE 2

TAILINGS STORAGE FACILITY CLOSURE DESIGN

DRAWING NUMBER	DRAWING TITLE
114185.15_001	COVER PAGE
114185.15_002	OVERALL MINE PLAN
114185.15_003	SITE LAYOUT
114185.15_004	SECTIONS & DETAILS (SHEET 1 OF 3)
114185.15_005	SECTIONS & DETAILS (SHEET 2 OF 3)
114185.15_006	SECTIONS & DETAILS (SHEET 3 OF 3)
114185.15_007	TSF SPILLWAY LAYOUT & SECTION
114185.15_008	TSF DECANT SYSTEM - LAYOUT AND SECTION
114185.15_009	TSF DECANT SYSTEM DETAILS
114185.15_010	SURVEY MONITORING PRISM LOCATIONS LAYOUT

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		B	27.02.2019	DRAFT ISSUE	AH	JS					
		A	26.02.2019	DRAFT ISSUE	PC	JL					

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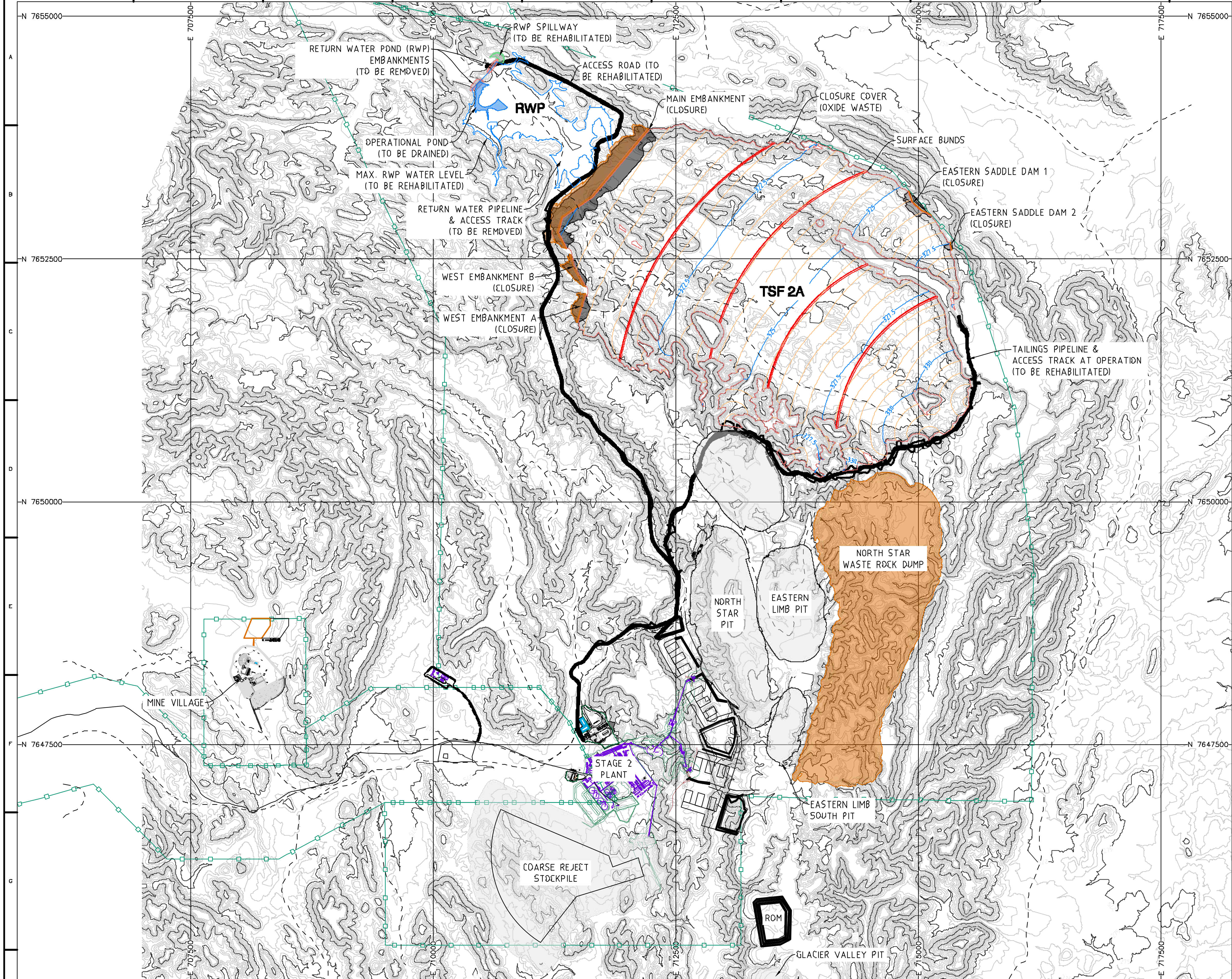
NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
COVER PAGE

A3 | SCALE: NTS | DRG No: | REV:



LEGEND
- EXISTING ACCESS ROADS
- PART IV BOUNDARY
- CONTOUR 25m
- CONTOUR 5m

NOTES:

1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50

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REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
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		B	27.02.2019	DRAFT ISSUE	AH	JS					
		A	26.02.2019	DRAFT ISSUE	AH	JL					

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PROJECT NUMBER: 114185.15

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VENDOR/DESIGNER DRG No. 114185.15_002 REV: C

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NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
OVERALL MINE PLAN

A3 | SCALE 1:40000 | DRG No: | REV:

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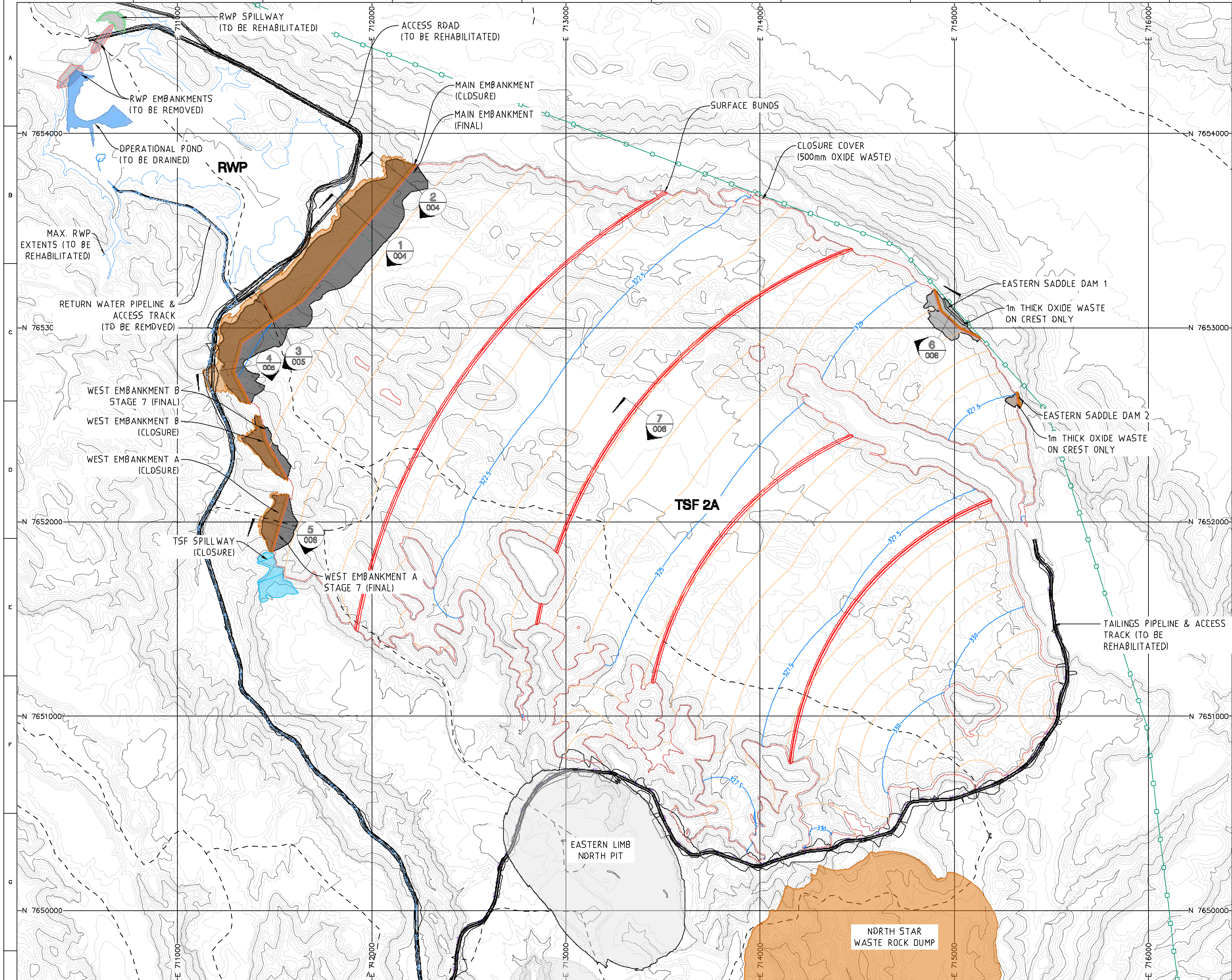

LEGEND

- EXISTING ACCESS ROADS
- PART IV BOUNDARY
- TAILINGS SLURRY PIPELINE ON ACCESS TRACK
- RETURN WATER PIPELINE ON ACCESS TRACK
- CONTOUR 25m
- CONTOUR 5m
- CONTOUR 0.5m (TSF)
- CONTOUR 2.5m (TSF)

NOTES:

1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50

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REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
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B	27.02.2019	DRAFT ISSUE	AH	JS					
A	26.02.2019	DRAFT ISSUE	AH	JL					

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NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
SITE LAYOUT

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A3 | SCALE 1:20000 | DRG No: | REV:

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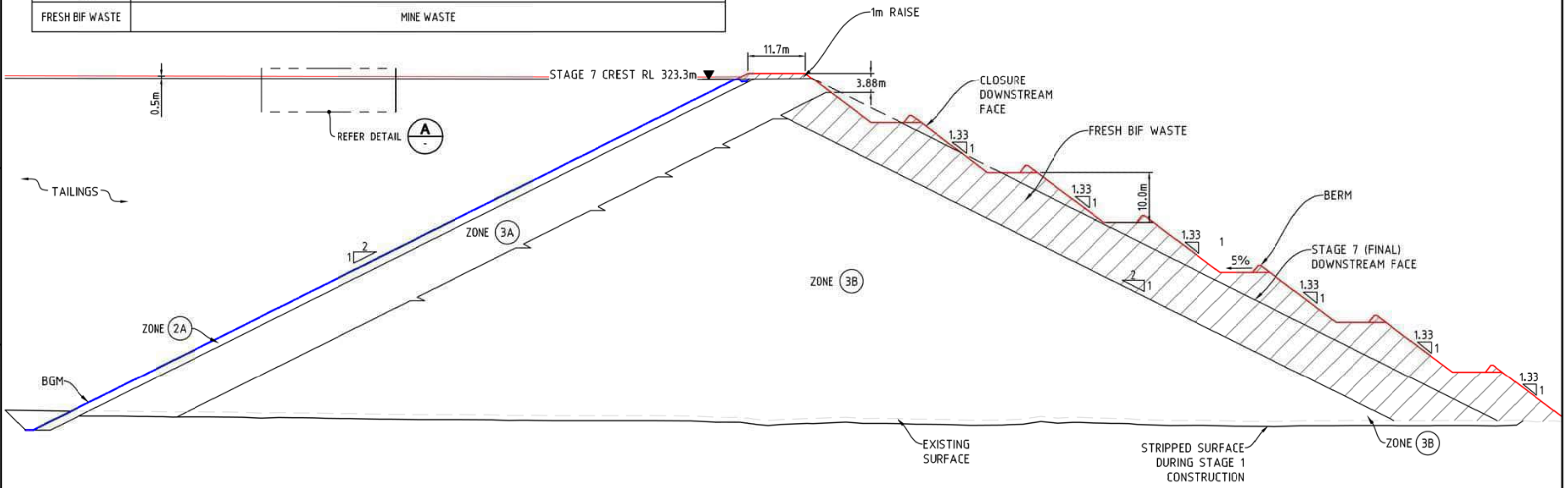
ZONE	MATERIAL DESCRIPTION
BGM	BITUMINOUS GEOMEMBRANE LINER
2A	BGM SUPPORT LAYER - 30 mm MINUS COARSE REJECTS
3A	TRANSITION ROCKFILL - COARSE REJECTS/APPROVED MINE SPOIL/DESIGNATED QUARRIES & EXCAVATIONS
3B	GENERAL ROCKFILL - APPROVED MINE SPOIL/DESIGNATED QUARRIES & EXCAVATIONS
FRESH BIF WASTE	MINE WASTE

LEGEND

\# MATERIAL ZONE NUMBER (REFER TO 114185.15_004 FOR DESCRIPTIONS).

FRESH BIF WASTE



SECTION - MAIN EMBANKMENT A 1/003
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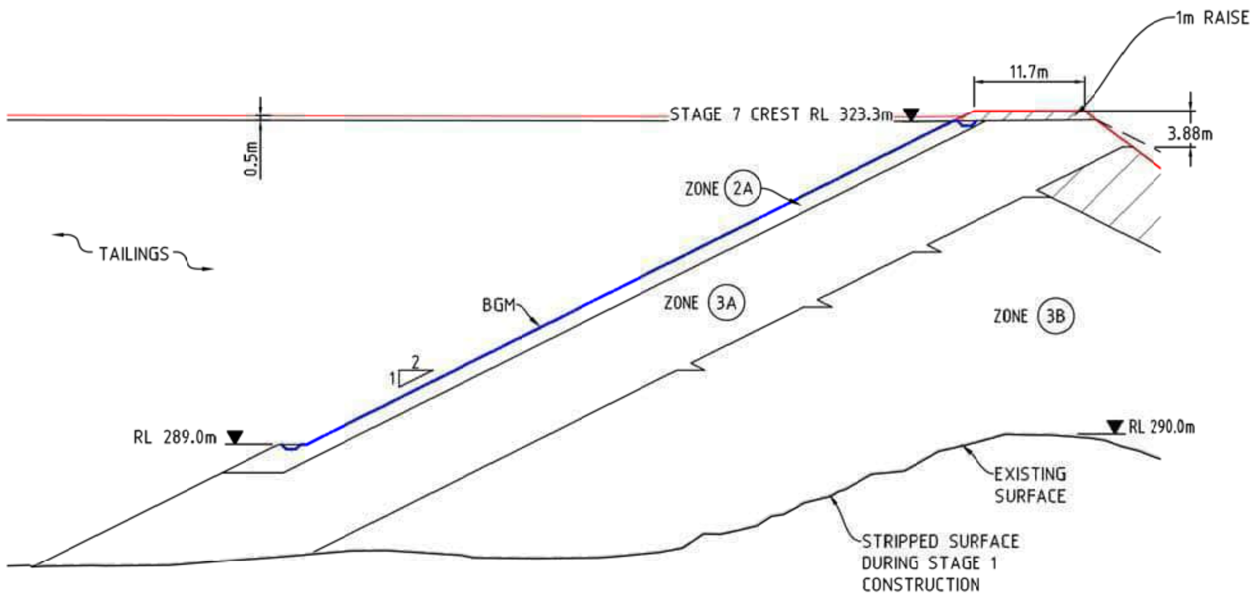

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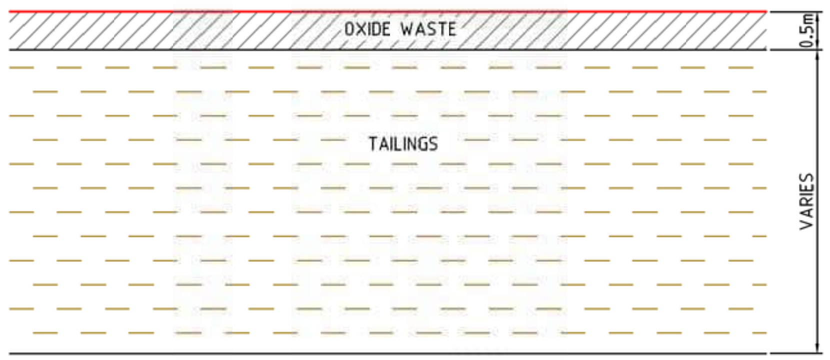

PROFILE DETAIL A/-
SCALE 1:100 @ A3

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PROJECT NUMBER 114185.15

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NORTH STAR MAGNETITE STAGE 1
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
SECTIONS & DETAILS (SHEET 1 OF 3)

A3 | SCALE AS SHOWN | DRG No: | REV:

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LEGEND

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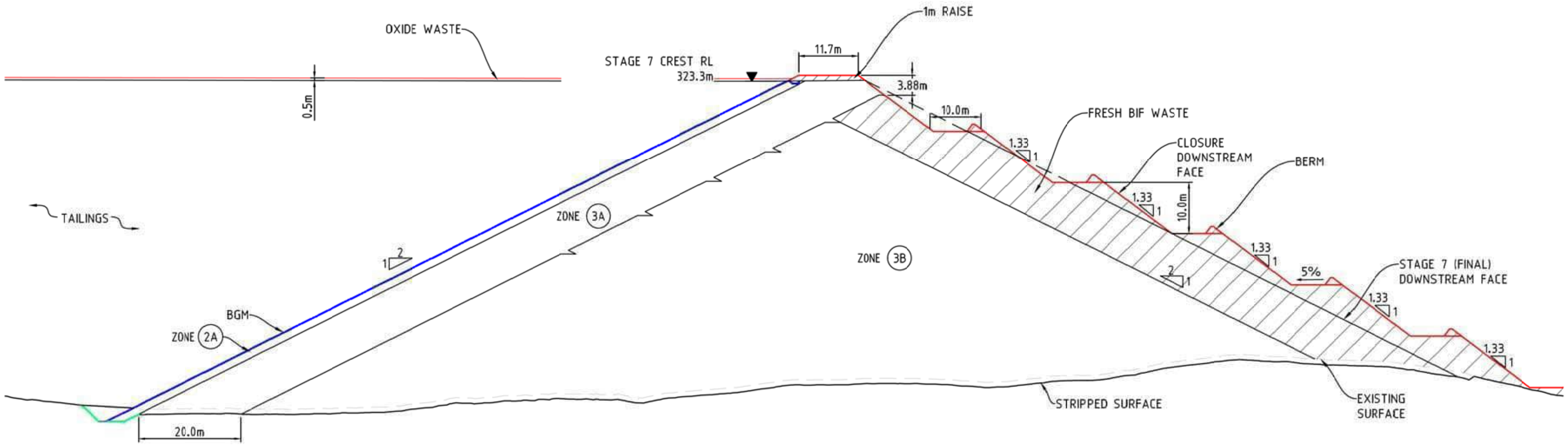
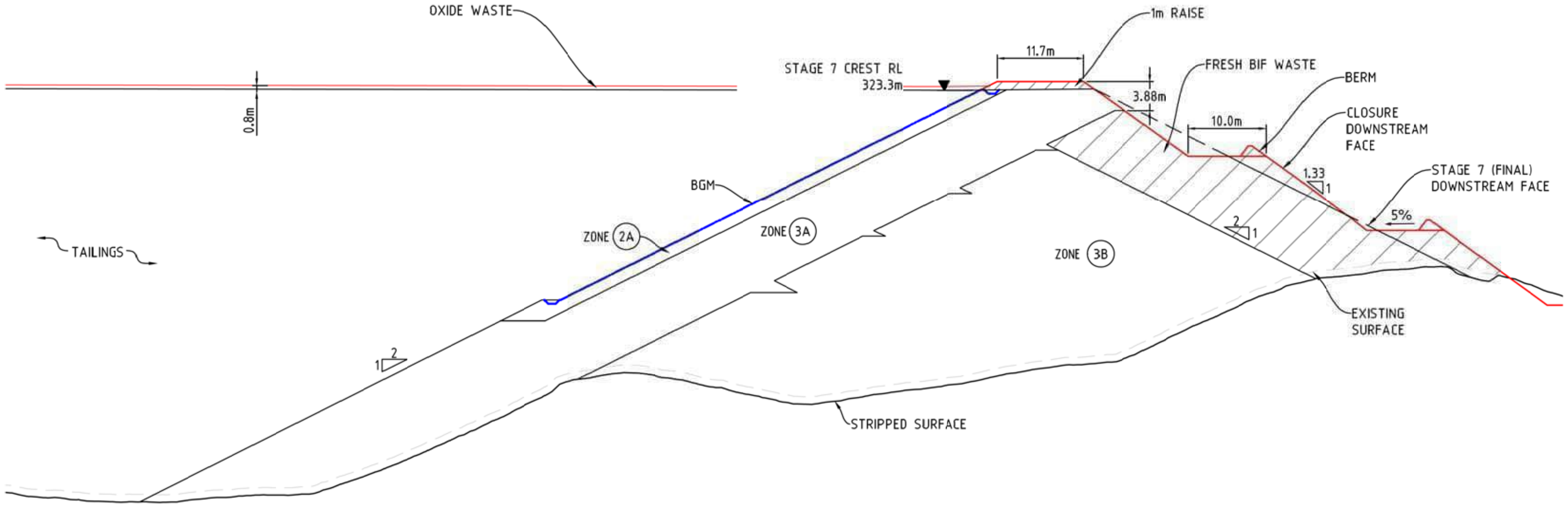
FRESH BIF WASTE



SECTION - MAIN EMBANKMENT C (3/003)
SCALE 1:1000 @ A3



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SCALE 1:750 @ A3

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NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITIY
CLOSURE DESIGN
SECTIONS & DETAILS (SHEET 2 OF 3)

A3 SCALE AS SHOWN DRG No: REV:

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LEGEND

(#) MATERIAL ZONE NUMBER (REFER TO 114185.15_004 FOR DESCRIPTIONS).

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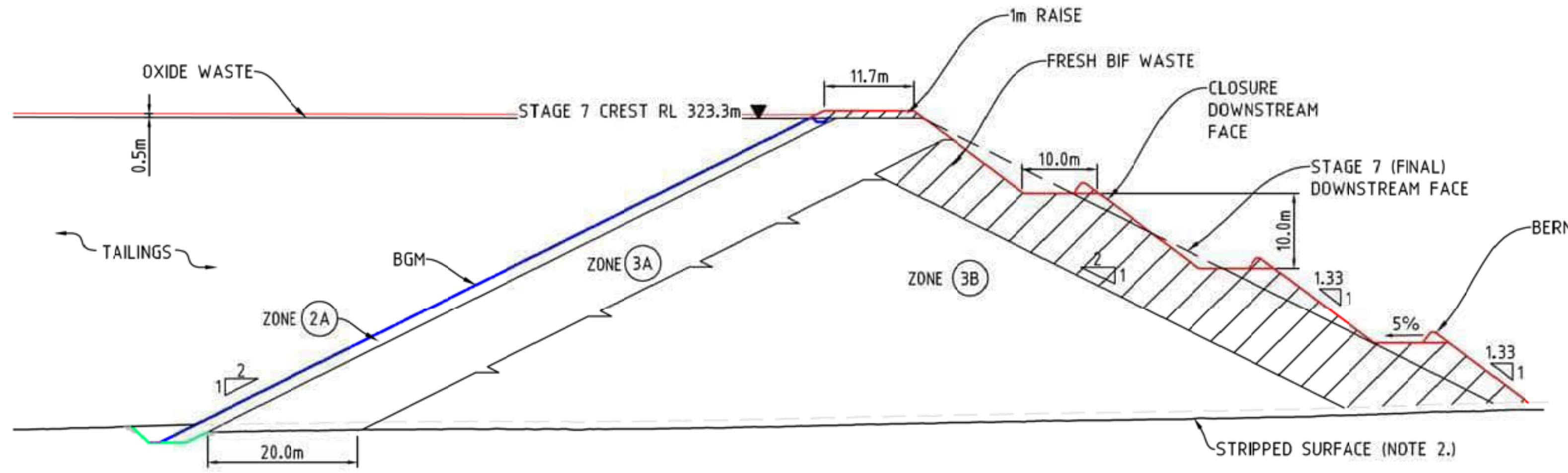

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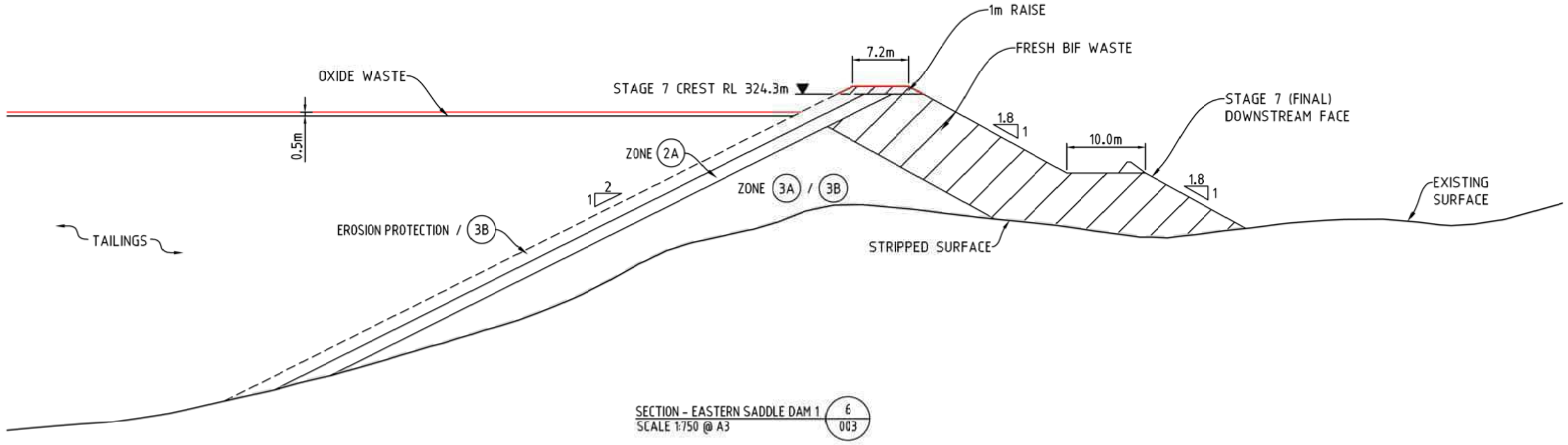

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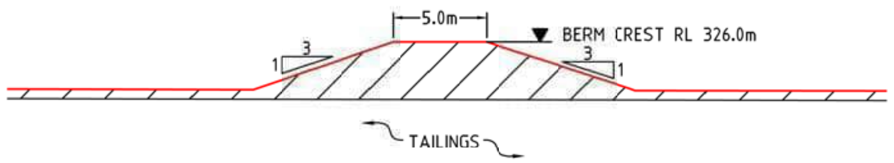

SURFACE CROSS BEACH BERM (7 / 003)
SCALE 1:500 @ A3

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NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
SECTIONS & DETAILS (SHEET 3 OF 3)

A3 | SCALE: AS SHOWN | DRG No: | REV:

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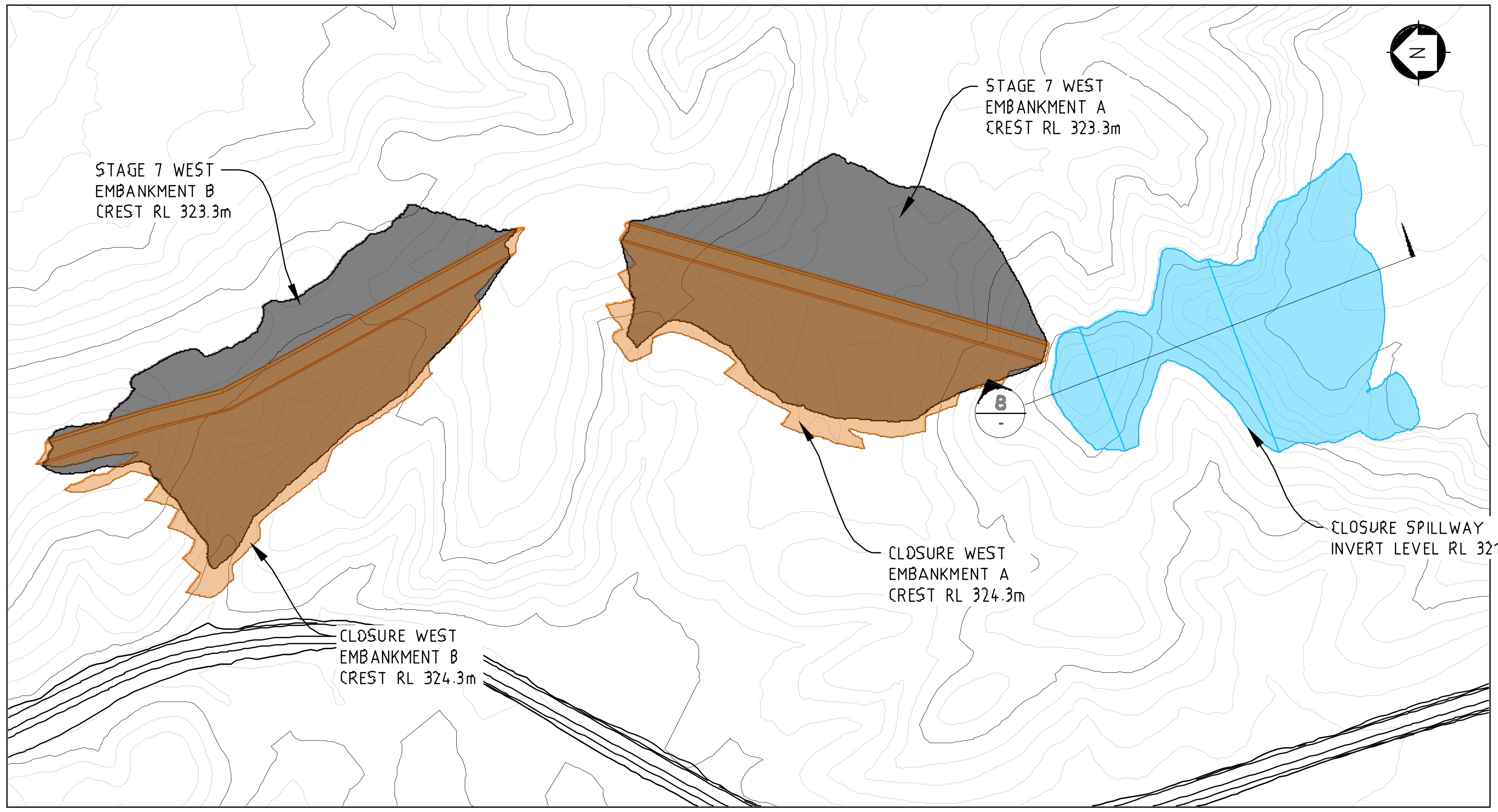
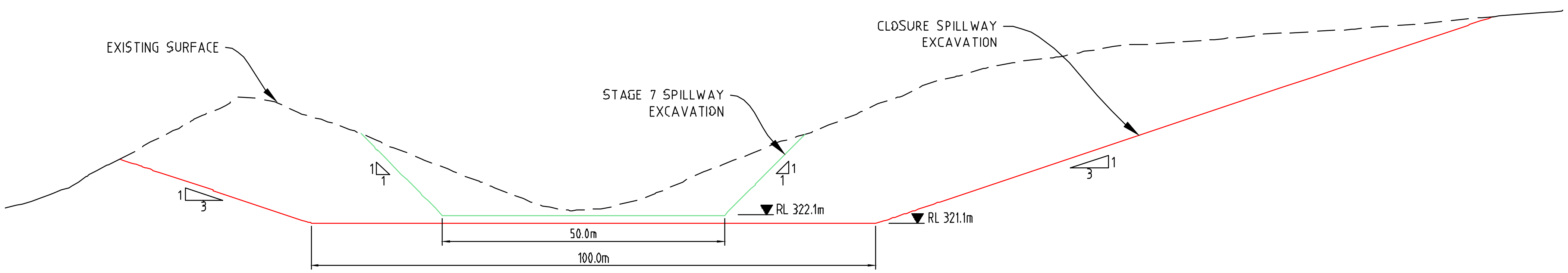


CLOSURE SPILLWAY LAYOUT
SCALE 1:5000 @ A3



SECTION - TSF CLOSURE SPILLWAY (B / -)
SCALE 1:1000 @ A3

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REV: C

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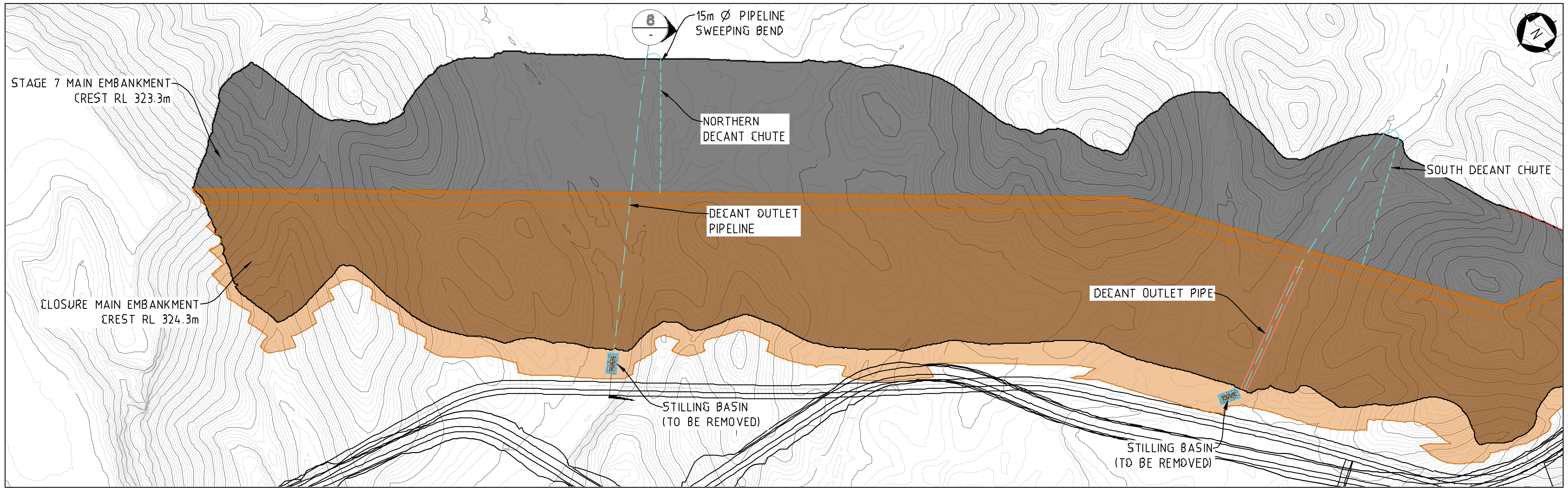
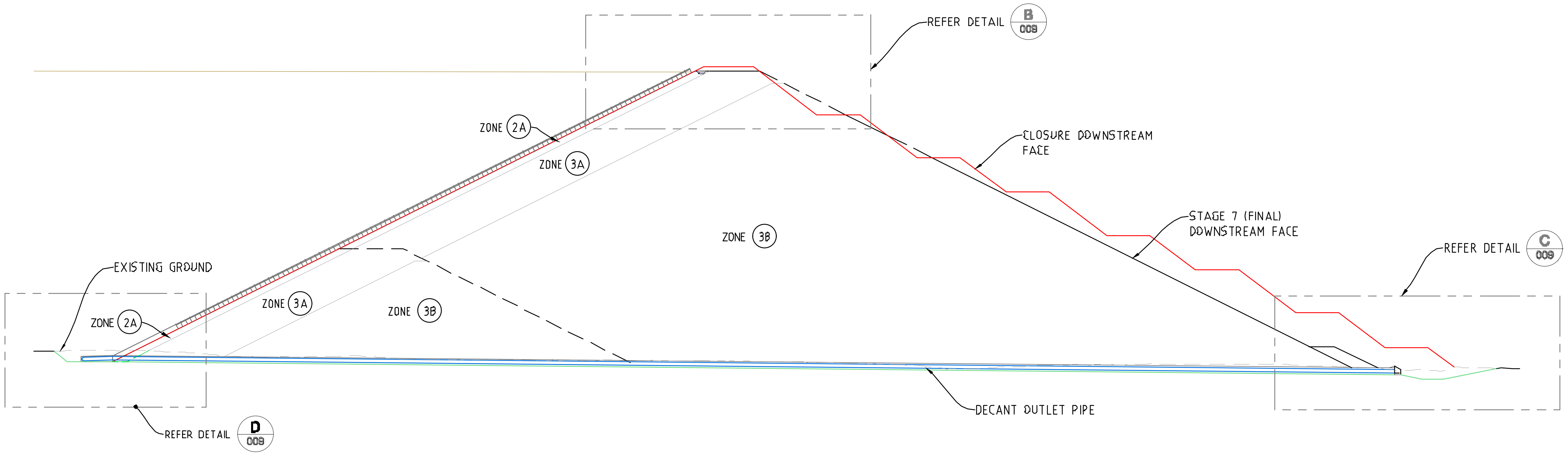
NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
TSF SPILLWAY LAYOUT & SECTIONS

A3 | SCALE: AS SHOWN | DRG No: | REV:



DECANT LAYOUT
SCALE 1:5000 @ A3



SECTION - NORTH DECANT SYSTEM B -
SCALE 1:120 @ A3

NOTES

1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50.

LEGEND

DECANT OUTLET PIPELINE
CONTOUR 5m
CONTOUR 1m
MATERIAL ZONE NUMBER (REFER TO 114185.15_004 FOR DESCRIPTIONS).

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REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
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PROJECT NUMBER 114185.15

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VENDOR/DESIGNER DRG No. 114185.15_008 REV: C

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NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
TSF DECANT SYSTEM - LAYOUT AND SECTION

A3 SCALE AS SHOWN DRG No: REV:

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LEGEND

(#) MATERIAL ZONE NUMBER

NOTES:

1. NORTH DECANT SYSTEM DETAILS ARE TYPICAL FOR THE SOUTH DECANT SYSTEM.

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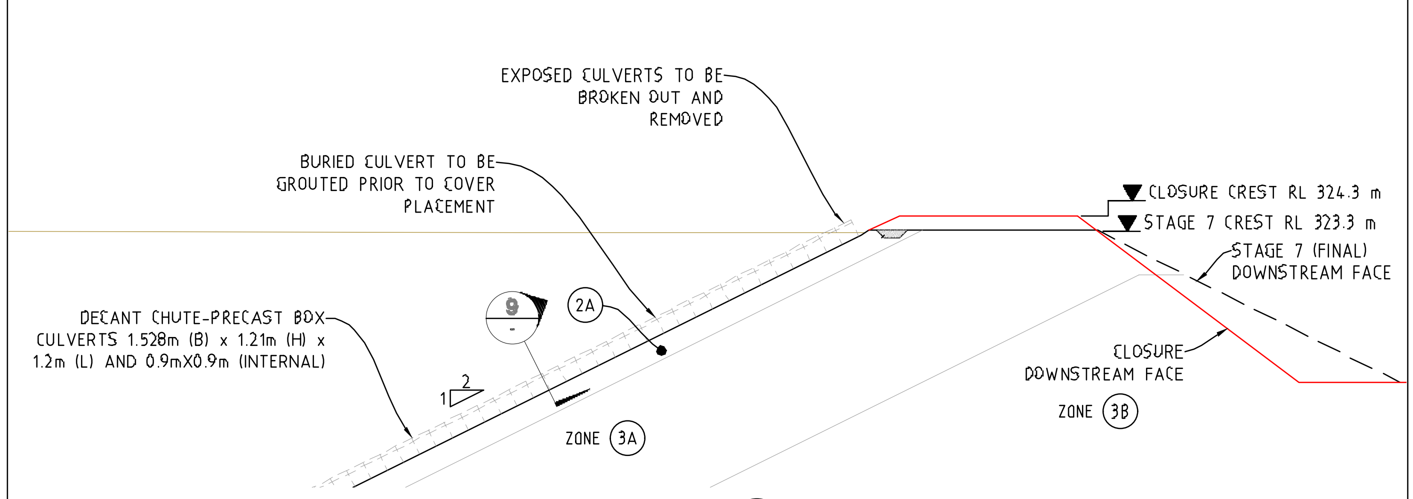
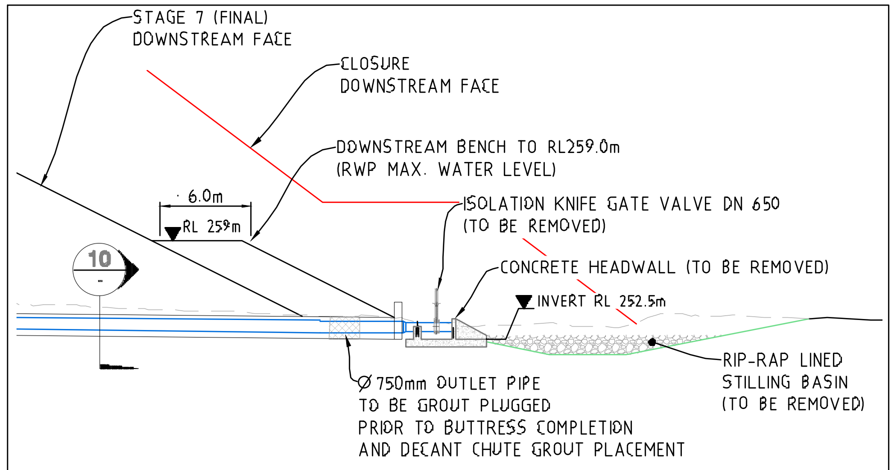
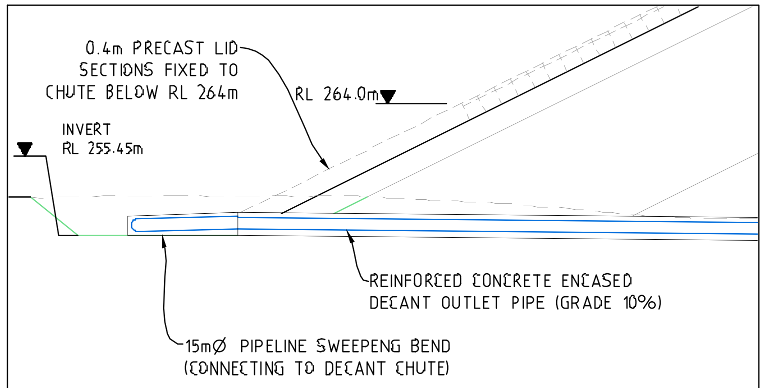


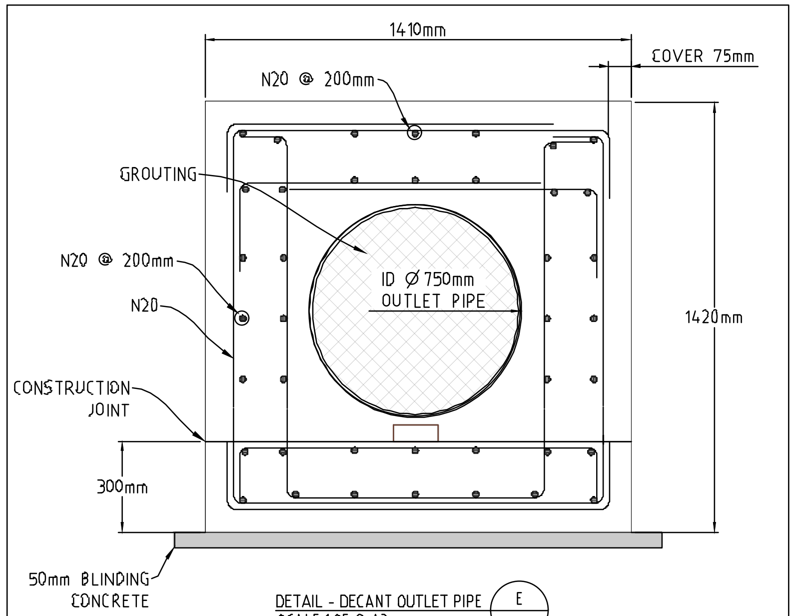
DETAIL - CLOSURE CREST B / 008
SCALE 1:500 @ A3



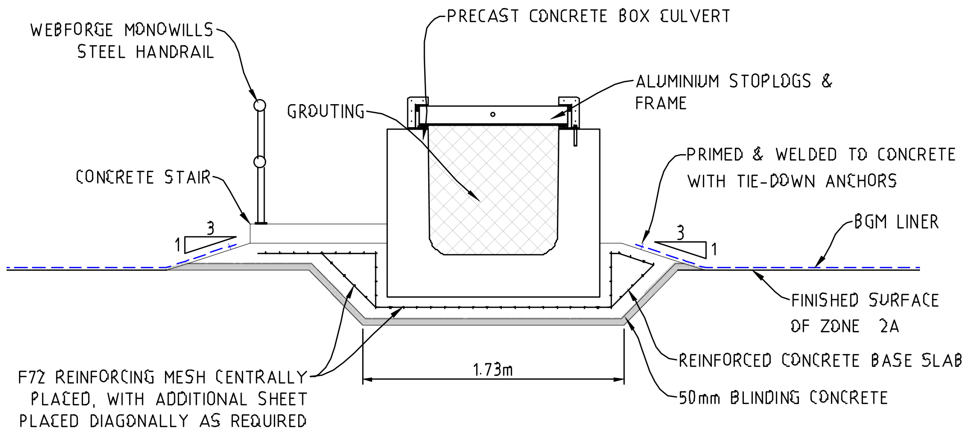
DETAIL - CLOSURE DOWNSTREAM TOE C / 008
SCALE 1:500 @ A3



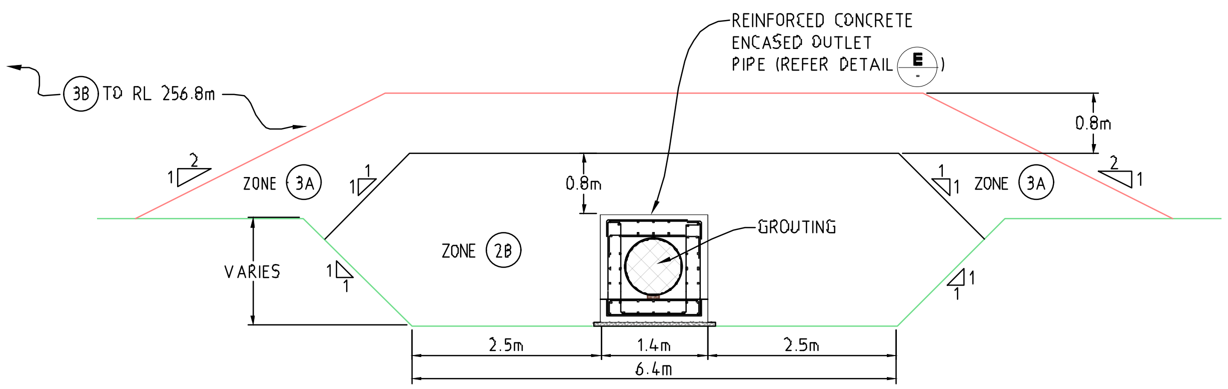
DETAIL - CLOSURE UPSTREAM TOE D / 008
SCALE 1:500 @ A3



DETAIL - DECANT OUTLET PIPE E / -
SCALE 1:25 @ A3



BURIED SECTION - CLOSURE NORTHERN DECANT 9 / -
SCALE 1:50 @ A3



SECTION - CLOSURE NORTH DECANT OUTLET PIPE TRENCH 10 / -
SCALE 1:100 @ A3

REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
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FMG | ATC Williams | NORTH

PROJECT NUMBER: 114185.14

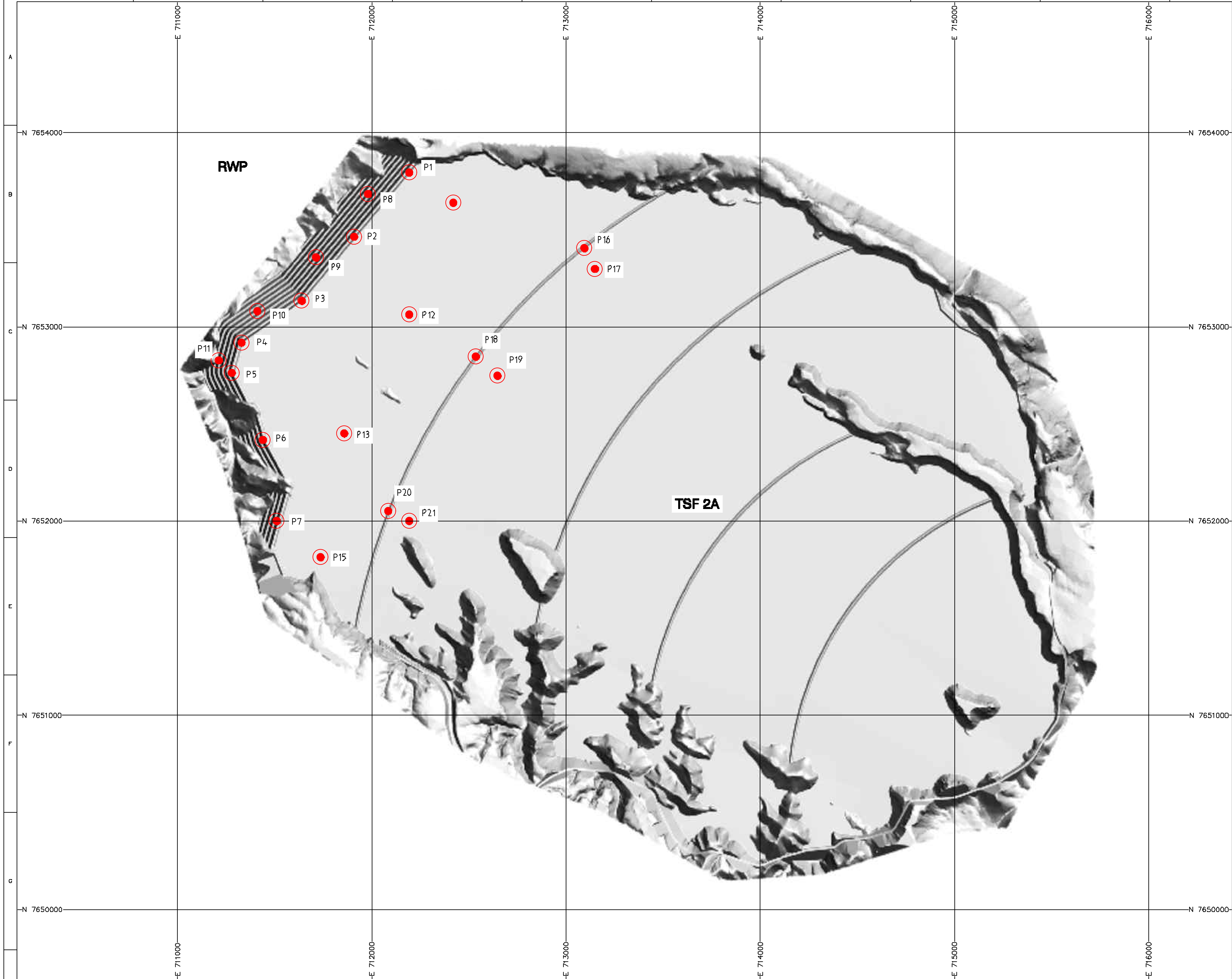
VENDOR/DESIGNER DRG No. 114185.15_009 REV: C

Fortescue Metals Group Ltd

NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
TSF DECANT SYSTEM DETAILS

A3 SCALE: AS SHOWN DRG No: REV:

G:\Synergy\Projects\114\114185 North Star Magnetite Stage 2\15 TSF Closure Design\CADD\CAD\2019-01-31 Issued for Review\114185.15_009.dwg



LEGEND

SURVEY MONITORING PRISMS

NOTES:

1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50

ISSUED FOR REVIEW
NOT FOR CONSTRUCTION

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		0	18.04.2019	APPROVALS ISSUE	AH	JS					
		B	27.02.2019	DRAFT ISSUE	AH	JS					
		A	26.02.2019	DRAFT ISSUE	AH	JL					

ATC Williams

NORTH

VENDOR/DESIGNER DRG No. 114185.15_010 REV: C

PROJECT NUMBER 114185.15

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Fortescue Metals Group Ltd

NORTH STAR MAGNETITE STAGE 2
TAILINGS STORAGE FACILITY
CLOSURE DESIGN
SURVEY MONITORING PRISM LOCATIONS LAYOUT

A3 SCALE 1:20000 DRG No: REV:

APPENDIX A

ARL
Analytical Reference Laboratory

ARL GROUP

ProMicro
Science with Ethics

LABORATORY REPORT

Job Number:	18-00471
Revision:	00
Date:	5 February 2018

ADDRESS: **Liquid Labs WA**
4/96 Briggs Street
Welshpool WA 6106

ATTENTION: Matt Van Herk

DATE RECEIVED: 11/01/2018

YOUR REFERENCE: North Star Magnetite Project

PURCHASE ORDER:

APPROVALS:

Douglas Todd
Laboratory Manager

Sam Becker
Inorganics Manager

REPORT COMMENTS:

This report is issued by Analytical Reference Laboratory (WA) Pty Ltd
Samples are analysed on an as received basis unless otherwise noted.
Phosphorus Colwell, Potassium Colwell, & Sulphur analysis subcontracted to CSBP, Report Number CTS18162, 64-78

METHOD REFERENCES:

ARL No. 314	NOx in Soil and Sediment by Discrete Analyser
ARL No. 118	Total Phosphorus and TKN in Soil and Biosolids
ARL No. 401/403	Metals in Soil and Sediment by ICPOES/MS
ARL No. 138	pH in Soil and Biosolid
ARL No. 140	Conductivity in Soil and Biosolid
ARL No. 064	Total Organic Carbon in Sediment
ARL No. 136	Lime Equivalence in Biosolids
ARL No. 213	Exchangeable Bases
ARL No. 212	Exchangeable Acidity
Subcontracting	See Report Comments section for more information.

LABORATORY REPORT

Liquid Labs WA
ARL Job No: *18-00471* *Revision:* *00* *Date:* 5 February 2018

RESULTS:

Nutrients in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
NOx-N	1	mg/kg	3	1	1	<1	1
Total Nitrogen	10	mg/kg	420	250	360	230	450

Nutrients in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-6 LLS18/32 4/01/2018	18-00471-7 LLS18/33 4/01/2018	18-00471-8 LLS18/34 4/01/2018	18-00471-9 LLS18/35 4/01/2018	18-00471-10 LLS18/36 4/01/2018
NOx-N	1	mg/kg	<1	<1	<1	23	1
Total Nitrogen	10	mg/kg	360	270	240	280	530

Nutrients in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-11 LLS18/37 4/01/2018	18-00471-12 LLS18/38 4/01/2018	18-00471-13 LLS18/39 4/01/2018	18-00471-14 LLS18/40 4/01/2018	18-00471-15 LLS18/41 4/01/2018
NOx-N	1	mg/kg	2	<1	<1	<1	<1
Total Nitrogen	10	mg/kg	350	200	160	500	150

Nutrients in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-16 LLS18/42 4/01/2018
NOx-N	1	mg/kg	18
Total Nitrogen	10	mg/kg	330

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
Sulphur	0.01	% w/w	<0.01	<0.01	<0.01	<0.01	<0.01
pH	0.1	pH units	7.1	6.9	7.3	7.1	7.9
Conductivity	0.01	mS/cm	0.04	0.03	0.03	0.03	0.07
TOC	0.1	%	0.5	0.5	0.5	0.4	0.5
Maximum Potential Acidity	2	kgH2SO4/t	<2	<2	<2	<2	<2
Acid Neutralising Capacity BT (19A2)	2	kgH2SO4/t	<2	<2	<2	<2	5

LABORATORY REPORT

Liquid Labs WA

ARL Job No: *18-00471* *Revision:* *00* *Date:* 5 February 2018

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
Net Acid Producing Potential	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 4.5	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 7.0	2	kgH2SO4/t	<2	<2	<2	<2	<2

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-6 LLS18/32 4/01/2018	18-00471-7 LLS18/33 4/01/2018	18-00471-8 LLS18/34 4/01/2018	18-00471-9 LLS18/35 4/01/2018	18-00471-10 LLS18/36 4/01/2018
Sulphur	0.01	% w/w	<0.01	<0.01	<0.01	0.01	<0.01
pH	0.1	pH units	7.6	6.8	8.5	7.1	6.8
Conductivity	0.01	mS/cm	0.05	0.01	0.08	1.3	0.04
TOC	0.1	%	0.5	0.3	0.5	0.3	0.9
Maximum Potential Acidity	2	kgH2SO4/t	<2	<2	<2	<2	<2
Acid Neutralising Capacity BT (19A2)	2	kgH2SO4/t	<2	<2	11	<2	<2
Net Acid Producing Potential	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 4.5	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 7.0	2	kgH2SO4/t	<2	<2	<2	<2	<2

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-11 LLS18/37 4/01/2018	18-00471-12 LLS18/38 4/01/2018	18-00471-13 LLS18/39 4/01/2018	18-00471-14 LLS18/40 4/01/2018	18-00471-15 LLS18/41 4/01/2018
Sulphur	0.01	% w/w	<0.01	0.01	<0.01	<0.01	<0.01
pH	0.1	pH units	7.1	7.2	7.0	7.4	6.7
Conductivity	0.01	mS/cm	0.03	0.02	0.01	0.04	0.01
TOC	0.1	%	0.5	0.2	0.3	0.7	0.2
Maximum Potential Acidity	2	kgH2SO4/t	<2	<2	<2	<2	<2
Acid Neutralising Capacity BT (19A2)	2	kgH2SO4/t	<2	<2	<2	3	<2
Net Acid Producing Potential	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 4.5	2	kgH2SO4/t	<2	<2	<2	<2	<2
Net Acid Generation to pH 7.0	2	kgH2SO4/t	<2	<2	<2	<2	<2

LABORATORY REPORT

Liquid Labs WA

ARL Job No: *18-00471* *Revision:* *00* *Date:* 5 February 2018

Misc. Inorganics in Soil Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-16 LLS18/42 4/01/2018
Sulphur	0.01	% w/w	<0.01
pH	0.1	pH units	7.2
Conductivity	0.01	mS/cm	0.08
TOC	0.1	%	0.4
Maximum Potential Acidity	2	kgH2SO4/t	<2
Acid Neutralising Capacity BT (19A2)	2	kgH2SO4/t	<2
Net Acid Producing Potential	2	kgH2SO4/t	<2
Net Acid Generation to pH 4.5	2	kgH2SO4/t	<2
Net Acid Generation to pH 7.0	2	kgH2SO4/t	<2

Cation Exchange Capacity Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
Exchangeable Calcium	0.2	cmolc/kg	3.0	3.7	3.8	3.2	8.9
Exchangeable Acidity	0.1	cmolc/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Exchangeable Potassium	0.05	cmolc/kg	1.5	0.98	2.4	1.5	2.0
Exchangeable Magnesium	0.2	cmolc/kg	2.2	1.7	2.0	2.8	3.2
Exchangeable Sodium	0.2	cmolc/kg	0.6	1.3	0.6	0.6	1.2
Cation Exchange Capacity	1	cmolc/kg	7	8	9	8	15.3

Cation Exchange Capacity Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-6 LLS18/32 4/01/2018	18-00471-7 LLS18/33 4/01/2018	18-00471-8 LLS18/34 4/01/2018	18-00471-9 LLS18/35 4/01/2018	18-00471-10 LLS18/36 4/01/2018
Exchangeable Calcium	0.2	cmolc/kg	4.8	1.9	22	2.2	3.6
Exchangeable Acidity	0.1	cmolc/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Exchangeable Potassium	0.05	cmolc/kg	1.6	1.5	1.7	1.8	1.4
Exchangeable Magnesium	0.2	cmolc/kg	2.6	2.5	3.8	4.4	2.4
Exchangeable Sodium	0.2	cmolc/kg	0.8	0.8	0.4	3.9	0.5
Cation Exchange Capacity	1	cmolc/kg	10	7	27.9	12.3	8

Cation Exchange Capacity Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-11 LLS18/37 4/01/2018	18-00471-12 LLS18/38 4/01/2018	18-00471-13 LLS18/39 4/01/2018	18-00471-14 LLS18/40 4/01/2018	18-00471-15 LLS18/41 4/01/2018
Exchangeable Calcium	0.2	cmolc/kg	4.3	2.6	2.6	8.0	1.6
Exchangeable Acidity	0.1	cmolc/kg	<0.1	<0.1	<0.1	<0.1	<0.1
Exchangeable Potassium	0.05	cmolc/kg	0.74	0.75	0.65	1.4	0.50
Exchangeable Magnesium	0.2	cmolc/kg	2.5	2.5	5.3	2.6	2.7
Exchangeable Sodium	0.2	cmolc/kg	0.9	0.7	0.8	0.5	0.9
Cation Exchange Capacity	1	cmolc/kg	8	7	9	12.5	6

Cation Exchange Capacity Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-16 LLS18/42 4/01/2018
Exchangeable Calcium	0.2	cmolc/kg	2.3
Exchangeable Acidity	0.1	cmolc/kg	<0.1
Exchangeable Potassium	0.05	cmolc/kg	0.80
Exchangeable Magnesium	0.2	cmolc/kg	2.3
Exchangeable Sodium	0.2	cmolc/kg	0.6
Cation Exchange Capacity	1	cmolc/kg	6

Subcontracting Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-1 LLS18/27 4/01/2018	18-00471-2 LLS18/28 4/01/2018	18-00471-3 LLS18/29 4/01/2018	18-00471-4 LLS18/30 4/01/2018	18-00471-5 LLS18/31 4/01/2018
Phosphorus Colwell	2	mg/kg	4	4	5	3	4
Potassium Colwell	2	mg/kg	200	180	210	160	170
Available Sulphur	0.5	mg/kg	1.7	1.4	1.3	1.5	2.1

Subcontracting Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-6 LLS18/32 4/01/2018	18-00471-7 LLS18/33 4/01/2018	18-00471-8 LLS18/34 4/01/2018	18-00471-9 LLS18/35 4/01/2018	18-00471-10 LLS18/36 4/01/2018
Phosphorus Colwell	2	mg/kg	3	<2	4	6	6
Potassium Colwell	2	mg/kg	160	96	130	310	330
Available Sulphur	0.5	mg/kg	1.0	1.5	0.5	110	1.9

LABORATORY REPORT

Liquid Labs WA

ARL Job No: *18-00471* *Revision:* *00* *Date:* 5 February 2018

Subcontracting Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-11 LLS18/37 4/01/2018	18-00471-12 LLS18/38 4/01/2018	18-00471-13 LLS18/39 4/01/2018	18-00471-14 LLS18/40 4/01/2018	18-00471-15 LLS18/41 4/01/2018
Phosphorus Colwell	2	mg/kg	4	<2	3	<2	3
Potassium Colwell	2	mg/kg	220	250	170	190	83
Available Sulphur	0.5	mg/kg	1.0	0.9	<0.5	<0.5	1.0

Subcontracting Sample No: Sample Description: Sample Date:	LOR	UNITS	18-00471-16 LLS18/42 4/01/2018
Phosphorus Colwell	2	mg/kg	3
Potassium Colwell	2	mg/kg	160
Available Sulphur	0.5	mg/kg	3.3

Result Definitions

LOR Limit of Reporting [NT] Not Tested [ND] Not Detected at indicated Limit of Reporting

* Denotes test not covered by NATA Accreditation

FOR MICROBIOLOGICAL TESTING - The data in this report may not be representative of a lot, batch or other samples and may not necessarily justify the acceptance or rejection of a lot or batch, a product recall or support legal proceedings. Tests are not routinely performed as duplicates unless specifically requested. Changes occur in the bacterial content of biological samples. Samples should be examined as soon as possible after collection, preferably within 6 hrs and must be stored at 4 degrees Celsius or below. Samples tested after 24 hrs cannot be regarded as satisfactory because of temperature abuse and variations.

Perth
Unit 3, 34 Sphinx Way
Bibra Lake
WA 6163
Ph: (08) 9418 8742
Mob: 0422 814 231
E-mail: Phillip.li@eprecisionlab.com

E-PRECISION LABORATORY

SLAKE DURABILITY TEST REPORT

Test Method: AS4133.3.4

Client:	Fortescue Metals Group	Date Tested:	03/12/2018
Project:	Iron Bridge 2018	Date Reported:	10/12/2018
		EP Lab Job Number:	FMG

Lab:	EPLAB
Tested by:	Phil
Checked by:	Phil

Lab ID:	NSD0055-03_SD	NSD0066-01_SD	NSD0064-05_SD	NSD0066-03_SD	NSD0067-02_SD
Client ID:	NSD0055-03	NSD0066-01	NSD0064-05	NSD0066-03	NSD0067-02
Depth (m):	317.70 - 318.03	99.80 - 100.10	214.65 - 214.86	124.15 - 124.41	120.48 - 120.74
Lithology/Description:	Pinc. BIF	Kang. SHL	Kang. SHL	Kang. SHL	Pinc. BIF
Moisture Content (%):	0	0	0	0	0
Slaking Fluid:	Potable Water	Potable Water	Potable Water	Potable Water	Potable Water
Temperature of Fluid (°):	20	20	20	20	20
1st Cycle Slake Durability Index (%)	**98.5**	**96.5**	**95.6**	**96**	**99.1**
Appear Particle Retained	*Broken*	*Broken*	*Broken*	*Broken*	*Broken*
Appear Particle Passing	*Fine Silt*	*Fine Silt*	*Fine Silt*	*Fine Silt*	*Fine Silt*
2nd Cycle Slake Durability Index (%)	**98.1**	**92.7**	**91.1**	**92.3**	**97.6**
Appear Particle Retained	*Broken*	*Broken*	*Broken*	*Broken*	*Broken*
Appear Particle Passing	*Fine Silt*	*Fine Silt*	*Fine Silt*	*Fine Silt*	*Fine Silt*

Notes: *Samples crushed down and sieved prior to testing*

Stored and Tested the Sample as received

Samples supplied by the Client

NATA: 19078 **Authorised Signatory (Geotechnical Engineer):**

Perth
Unit 3, 34 Sphinx Way
Bibra Lake
WA 6163
Ph: (08) 9418 8742
Mob: 0422 814 231
E-mail: Phillip.li@eprecisionlab.com

E-PRECISION LABORATORY

SLAKE DURABILITY TEST REPORT

Test Method: AS4133.3.4

Client:	Fortescue Metals Group	Date Tested:	03/12/2018
Project:	Iron Bridge 2018	Date Reported:	10/12/2018
		EP Lab Job Number:	FMG

Lab: EPLAB
Tested by: Phil
Checked by: Phil

Lab ID:	NSD0067-101_SD	NSD0068-01_SD	NSD0068-06_SD		
Client ID:	NSD0067-101	NSD0068-01	NSD0068-06		
Depth (m):	100.28 - 100.44	210.15	246.5		
Lithology/Description:	Pinc. BIF	Corboy SHL	Corboy SHL		
Moisture Content (%):	0	0	0		
Slaking Fluid:	Potable Water	Potable Water	Potable Water		
Temperature of Fluid (°):	20	20	20		
1st Cycle Slake Durability Index (%)	**96.7**	**94.5**	**95.2**		
Appear Particle Retained	*Broken*	*Broken*	*Broken*		
Appear Particle Passing	*Fine Silt / Sand*	*Fine Silt*	*Fine Silt / Sand*		
2nd Cycle Slake Durability Index (%)	**93.6**	**90.7**	**91**		
Appear Particle Retained	*Broken*	*Broken*	*Broken*		
Appear Particle Passing	*Fine Silt*	*Fine Silt*	*Fine Silt*		

Notes: *Samples crushed down and sieved prior to testing*
Stored and Tested the Sample as received

Samples supplied by the Client

NATA: 19078 **Authorised Signatory (Geotechnical Engineer):**

APPENDIX B

TSF Closure Options Analysis – North Star Magnetite Stage 2

1. TSF Embankment Closure - Reprofiling

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
TSF Closure - Embankment Reprofiling						
Assumptions: 1. The TSF Landform classifies as NAF 2. The discharge arising from the waste rock used for construction either to groundwater or as surface seepage is not impacted by acid or neutral mine drainage and is dischargeable 3. There is insufficient growth media available to cover the entire embankment and it is likely that only a small proportion of the embankment could be covered with growth media 4. There is sufficient Oxide material available for import for all Options 5. There is sufficient durable blocky waste available for import for all Options 6. Volumes are a surrogate for costs and should be considered as indicative only						
1	**Do Nothing**	Leave the embankment as constructed during operations. Make Safe.	Minor Cost Negligible Earthworks	Unlikely to conform to Approvals Unlikely to satisfy Stakeholders Likely Unacceptable/ High erosion Limited revegetation	**Nil**	12
2a	**Single concave slope with oxide**	The operational era embankment would be reprofiled via a cut to fill and additional import at the toe to create a concave slope of approximately 20° for the upper 50% and 15° for the lower 50%. 2m of Oxide would be spread with dozers (where oxide not already present as import)	Less earthworks than Option 3 and above Some potential for revegetation No Berms and hence no water concentration or drainage structures required	Probable unacceptable/high rates of erosion	3,742,000	14

# Option Title		Description	Advantages	Disadvantages	Indicative Volumes (m3)	Ranking
2b	**Single concave slope with rock mulch**	As for 2a but with 0.5m of durable blocky waste placed over the oxide batters and ripped in as a rock mulch.	As for 2a and Improved erosional stability	As for 2a	3,742,000	6
2c	**Single concave slope with durable fresh waste**	As for 2a but with 2m of Durable Blocky waste rock would be spread with dozers instead of oxide	Less earthworks than Option 3 and above No Berms and hence no water concentration or drainage structures required. Erosionally Stable	Little or no revegetation Specific case not yet assessed in erosion modelling	3,742,000	4
2d	**Single concave slope with durable fresh waste and oxide**	As for 2c but with 1.5m of Durable Blocky and 0.3 -0.5m of oxide and ripped in/integrated into more durable material	As for Option 2c but with higher likelihood of some revegetation	As for 2c but less erosionally stable, primarily in establishment phase	3,742,000	3
3a	**Batter berm (concave batters) with two lifts – Oxide or mixed waste rock**	The operational era embankment would be reprofiled via oxide/mixed waste rock import to create a batter berm configuration with upper and lower lifts each of approximately 35m vertical height, 30m wide berms and individual batter faces of approximately 20° upper and 15° lower sections. The as presented material would be utilised for ripping and seeding.	Shorter Slopes Maintain Concave Slopes on individual batters Less Import than cases 4 and above	More import than Option 2 Moderate Erosional Stability Does not align with findings of Erosion modelling.	5,555,000	8

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
3b	**Batter berm (concave batters) with two lifts – rock mulch**	As for 3a but with 0.5m of durable blocky waste placed over the batters and ripped in as a rock mulch.	As for 3a Improved slope erosion resistance	As for 3a with With two spread passes	5,555,000	9
3c	**Batter berm (concave batters) with two lifts – Blocky durable waste rock**	As for 3a but with 2m of durable blocky waste placed over the batters	Erosionally Stable	As for 3a with Potentially greater additional import if reprofiling material oxide/mixed waste already in place Little or no vegetation	5,555,000	5
3d	**Batter berm (concave batters) with two lifts – Blocky durable waste rock and oxide**	As for 3a but with 1.5m of durable blocky waste and 1.5m of oxide placed over the batters	As for 3a and More Erosionally Stable	As for 3a with Two spread passes Limited revegetation	5,555,000	6
4a	**Batter berm (concave batters) with three lifts - Oxide**	The operational era embankment would be reprofiled via a cut to fill and oxide/mixed waste rock import to create a batter berm configuration with three lifts of 20-25m, 20m wide berms and individual batter faces of approximately 20º upper and 15º lower. The as presented material would be utilised for ripping and seeding.	Shorter Slopes Maintain Concave Slopes Less Import than cases 5 and above Some revegetation	May have unacceptable rates of erosion Lift heights greater than current erosion modelling recommendations for oxide.	6,348,000	9

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
4b	**Batter berm (concave batters) with three lifts – rock mulch**	As for 4a but with 0.5m of durable blocky waste placed over the batters and ripped in as a rock mulch.	As for 3a Improved slope erosion resistance than oxide only	As for 3a with With two spread passes	6,348,000	7
4c	**Batter berm (concave batters) with three lifts – Blocky durable waste rock**	As for 4a but with 2m of durable blocky waste placed over the batters	Shorter Slopes Maintain Concave Slopes Erosionally Stable Less Import than cases 5	As for 3a with Potentially greater additional import if reprofiling material oxide/mixed waste already in place Little or no vegetation	6,348,000	6
4d	**Batter berm (concave batters) with three lifts – Blocky durable waste rock and oxide**	As for 4a but with 1.5m of durable blocky waste and 5m of oxide placed over the batters	As for 3a and More Erosionally Stable	As for 3a with Two spread passes Limited revegetation	6,348,000	7
5a	**Oxide Angle of repose slopes. Maximum seven 10m lifts. Berms To Be Sized Accordingly.**	Import oxide material and tip with no reprofiling. Seven ten metre lifts with 10m – 20 m berm between toe of batter above and crest of batter below	Consistent with erosion modelling where berm accepted as an effective break to water/sediment	Potential large toe extension and import volume if wider berms adopted. No vegetation on batters	998,000 - 4,031,000	2
5b	**Blocky Durable fresh waste Angle of repose slopes. Maximum seven lifts. Berms To Be Sized Accordingly.**	As for 5a with the batter faces constructed with, or having a minimum depth of, 2m durable blocky fresh waste	Consistent with erosion modelling where berm accepted as an effective break to water/sediment Higher erosional stability than 5a. Higher infiltration capacity than 5a.	Potential large toe extension and import volume if wider berms adopted. No vegetation on batters or berms.	998,000 - 4,031,000	1

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
6a	**Single Slope at 5 degrees**	Import material to form a 5 degree slope	Consistent with erosion modelling	Very large toe extension Very large import volume Long slope length and hence large catchment	68,756,000	11
6b-d	**As for 3b-d except for embankment profile**	Import material to form a 5-degree slope and sheet as per 3. b-d	Consistent with erosion modelling Higher erosional stability to varying degrees	Very large toe extension Very large import volume Long slope length and hence large catchment	68,756,000	10

Justification for Preliminary Preferred Option – 5 (with optimised berm widths)

Lower bound import volumes

- Upper Bound Erosional Stability
- Short slope lengths
- Low Cost

2. TSF Embankment Closure - Drainage

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
TSF Closure – Embankment Drainage Design						
Assumptions: 1. The discharge arising from the waste rock used for construction either to groundwater or as surface seepage is not impacted by acid or neutral mine drainage 4. There is sufficient durable waste for select material supply 6. Volumes should be considered as indicative only						
1	**No Drainage Measures**	Leave the embankment as constructed during operations. Make Safe.	Minor Cost Negligible Earthworks	Does not conform to approvals Will not satisfy Stakeholders Unacceptable erosion Limited if any revegetation	NA	6
2a	**Single Slope average 17.5 degrees 2m Oxide Sheeting (No concentrating drainage structures)**	The operational era embankment would be reprofiled via a cut to fill and additional import at the toe to create a slope of approximately 20° upper and 15° lower. 2m of Oxide would be spread with dozers (where oxide not already present as import)	No concentrating surface water structures No water structure construction costs The same action achieves slope stability	May have unacceptable rates of erosion Does not align with findings of Erosion modelling. Limited revegetation		5
2b	**Single Slope average 17.5 degrees and sheet with 2m durable blocky waste rock (No concentrating drainage structures)**	As for 2a but with min 2m of durable blocky waste placed over the batters. Ideally, based on mine life schedule, all of the buttress would be created from durable blocky waste rock. (As in 2d for Embankment Profile – 500mm of oxide could be spread over the surface and ripped into the waste rock)	No concentrating surface water structures No water structure construction costs The same action achieves slope stability	Vegetation will not contribute to stabilisation		3

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
3a	**Berms – fully contained on berms with cross bunds**	Berms would be constructed to be level, back sloping with cross bunds to ensure that the only water retained on berms is from the batter immediately above. Final berms widths and back slopes based on field derived infiltration and run off study	Smaller individual batter catchments Passive/no velocity approach Berms can be revegetation opportunity Berms can be sized based on catchment, infiltration capacity and local run off characteristics	Berms must contain / infiltrate inflow in addition to providing freeboard for sediment Long term potential for bund failure		2
3b	**Fully discharging berms to natural ground**	Berms graded, nominally at 1-2% to facilitate low velocity run off from berms to natural ground via hardened drains on the berms	Berms can be narrower	Concentrating flow more likely to scour/fail over the long term May require select material May be impacted by differential settlement		4
4a	**Contained on berms on the majority of length with discharging drains near the interface with natural ground**	The majority of the length of berms would be constructed as per 3a with those sections of berms adjoining natural ground, and where discharge can be effectively achieved, short sections constructed consistent with 3b to a very high standard of engineering rigor	Balanced approach to generally minimise concentrating flow but take logical opportunities to discharge/integrate with natural ground	As for 3a and 3b		1

Justification for Preliminary Preferred Option – 3a/4a

- Drainage balance containment/ infiltration and discharge
- Aligns with profile option

3. TSF Surface Closure – Top Surface Cover

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
TSF Closure Tailings Surface Cover						
Assumptions: 1. There is sufficient Oxide material available for import for Options 2. There is sufficient durable blocky waste available for import for Options 3. Volumes should be considered as indicative only						
1	**No Cover**	Leave the top surface as deposited during operations. Make Safe.	Minor Cost Negligible Earthworks	Does not conform to approvals/guidelines Will not satisfy Stakeholders Unacceptable erosion Limited if any revegetation Potential for Dusting Tailings discharge via spillway likely	Nil	7
2a	**Oxide Cover 1m**	An even layer of 1m of oxide is top over the beach surface	Deeper rooting depth than 2b if tailings less benign than oxide Some potential revegetation in oxide material	Greater import volumes than 2b Material prone to erosion but longer lifespan than 2b	9,405,000	5
2b	**Oxide Cover .5m**	An even layer of 0.5m of oxide is spread over the top surface	Less import volume than 2a Closer to PAW in tailings	Less root medium if tailings hostile to plant establishment Material prone to erosion Higher probability of earlier exposed tailings over long term	4,702,500	3

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
3a	**Durable Fresh Waste Rock Cover – 1m**	An even layer of 1m of durable fresh waste rock is spread over the top surface	Material less prone to erosion	Greater import volumes than 3b Little or no vegetation	9,405,000	4
3b	**Durable Fresh Waste Rock Cover – .5m**	An even layer of .5m of durable fresh waste rock is spread over the beach surface	As for 3a and Less import volumes that 3b	Little or no vegetation	4,702,500	2
4a	**Mixed Waste Type Cover with Oxide patches in low flow/velocity areas to promote some revegetation -1m**	Mosaic of Blocky durable waste and oxide in patches across the tailings surface strategically placed in lower and higher flow locations strategically at an average depth specified of 1m.	Creates opportunities to balance revegetation establishment with erosion resistance and dust control	Requires more earthworks controls/management. More import than 4b	9,405,000	3
4b	**Mixed Waste Type Cover with Oxide patches in low flow/velocity areas to promote some revegetation -0.5m average**	Mosaic of Blocky durable waste and oxide in patches across the tailings surface strategically placed in lower and higher flow locations strategically at a an average depth specified of 0.5m though depth may vary locally	Requires less import than 4a	Higher probability of earlier exposed tailings over long term	4,702,500	1
5	**Capillary Break Cover**	0.5m of blocky durable waste overlain by 0.5m oxide material	May reduce vertical rise of salts	Vegetation in perennial drought Two layers of spreading Capillary break benefits lost by second level and spread of next layer	9,405,000	6
Justification for Preliminary Preferred Option – 4b • Balances erosional stability and revegetation with local placement in response to flow characteristics • Lower bound material import • Some soil forming/revegetation potential						

4. TSF Surface Closure - Top Surface Drainage

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
TSF Closure – Surface Cover Drainage						
Assumptions: 1. ARI and PMP calculations are developed based on acceptable inputs 2. There is sufficient durable fresh waste to supply options including for the generation of select material 3. Volumes should be considered as indicative only						
1	**No Drainage or Attenuation Measures – Spillway only**	Leave the embankment and spillway as constructed during operations. Make Safe.	Minor Cost Negligible Earthworks	May not conform to approvals Unlikely to satisfy Stakeholders	**Nil**	5
2a	**Multiple Contour Banks to 1:500 ARI plus spillway**	Contour banks would be formed via the utilisation of in situ material in order to establish haulage access to import of cover (an unavoidable cost). The haulage routes must be hardened with conditioned/compacted material as a running surface. The RL of the causeways can be sized to contain a 1:500ARI. They would be armoured on the upstream face with a minimum 1m durable blocky waste	Passive drainage management (non concentration) No interaction between one sub catchment and the next for < 500-year ARI Banks receiving natural catchment flow can be sized accordingly	Contour banks may degrade	TBA	1
2b	**Multiple Contour Banks to PMP**	As for 2a but to a PMP scale. No spillway	No Spills. No LOM spillway modification required. Passive drainage management (non-concentration) No interaction between one sub catchment and the next	Incremental additional in situ push up of in situ material. Contour banks may degrade.	TBA	3

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes (m^3)	Ranking
			Banks receiving natural catchment flow can be sized accordingly			
3	**Velocity Reduction Rock Walls**	Rock walls placed at strategic locations to create flow interruption and limit velocity locally. Attenuation behind rock walls reduces frequency of discharge over spillway	Reduces flow velocity on higher potential flow velocity area on cover. Reduces frequency and scale of discharges Localised preferential vegetation behind rock walls Can anticipate differential settlement	Does not eliminate flow along entire length of catchment Full catchment includes natural ground run on	**TBA**	2
4	**Spine Drain**	Formal drains constructed over cover which concentrate flow from throughout the facility to larger drains which discharge at the spillway.		Select material likely to be required Concentrating structures over tails likely to scour over the long term Prone to differential settlement Must manage natural catchment flow in addition to tailings footprint	**TBA**	4

Justification for Preliminary Preferred Option 2a – Multiple contour banks to 1:550 plus spillway

- Limits cover erosion
- Spillway can accommodate degradation of bunds
- Low flow velocities anticipated

5. Growth Media and Substrate

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes	Ranking
TSF Closure – Embankment and Surface Cover Growth Media						
Assumptions: 1. Topsoil and subsoils from the TSF footprint only will be available 2. Subsoil and topsoil will not be separate and will be utilised as growth media 3. Timber mulch is negligible 4. Only topsoil which is non dispersive will be utilised on batters. Highly dispersive, sodic or saline soils will not be utilised as growth media 5. Other material such as coarse rejects will not be included in the soil inventory						
1	**No growth media anywhere on Closed TSF**	No topsoil or subsoil spread	Minor Cost Negligible Earthworks There is very little topsoil and it may be better deployed on the WRD	Regulators expect soil resources to be utilised Incremental revegetation opportunities will not be realised locally The stabilising effect of revegetation will not be realised locally	**NA**	5
2a	**Preferentially placed as islands on top surface**	All topsoil resources placed as islands on the TSF cover surface. Most effectively over oxides and in lower/no flow velocity parts of the facility (in the lees of island or promontories or behind contour banks)	Potential for enhanced revegetation on TSF surface. Likely to benefit from surrounding areas seed natural seed transfer	No enhanced revegetation on embankments Top surface less visible (to be confirmed)	Volume as per inventory	4
2b	**Preferentially placed on embankment batters**	All topsoil resources placed on the TSF embankment batters. Most effectively over options where oxides and durable	Embankments most difficult sub domain to stabilise and	May be insufficient inventory to cover embankments	Volume as per inventory	2

#	Option Title	Description	Advantages	Disadvantages	Indicative Volumes	Ranking
		block waste are mixed/integrated. This may involve placement of soils only on the lower sections of batters where velocity is likely to be higher.	revegetation contribution to land stabilisation focused here	Any material which is dispersive or sodic to any extent is not suitable for slopes		
3a	**Preferentially placed on embankment berms**	All topsoil resources placed on the TSF embankment berms. Most effectively over options where oxides and durable block waste are mixed/integrated.	Placed on flatter Slopes Less run off Placed in more erosionally stable location Less import than case 2	Potential flooding Impedes berm drainage	Volume as per inventory	3
3a	**Mixed deployment on selected surfaces based on balance of land stability, ecosystem reestablishment and aesthetic considerations.**	May involve small islands of topsoil on top surface within large islands of oxide to create biodiversity distribution zones. Some deployment of material on berms only, batters or deployment of soils on the lower half of concave slopes where flow velocities may be higher	Shorter Slopes Maintain Concave Slopes		Volume as per inventory	1

Justification for Preliminary Preferred Option 3a – Balance of Deployment

- Deployment primarily supports land stability
- Some volumes allocated for sustainability/aesthetic objectives
- Flat surfaces (oxide) easier to revegetate with seed only

APPENDIX C

BASIS OF DESIGN - MASTER

Site	Group	Domain	Design Aspect	High Level Design Objective	Element	Statutory obligation	External	WAIO standard	FMG standard	Regulatory Guidance	Range (Low)	Range (High)	Design Criteria	Basis for selection (ref)
North Star Magnetite	Landforms	TSFs	Geotechnical stability	Safety, stability	Entire facility	Maximum Credible Earthquake (MCE). FOS minimum >1.0 to be defined.					MCE	MCE	Geotechnical - Factor of Safety >1.5 Static, >1 Post Seismic (ANCOLD) Seismicity - MCE	ICOLD/ANCOLD Guidelines (ICOLD - International Committee on Large Dams; ANCOLD - Australian National Committee on Large Dams)
			Design life	Safety, stability, non-polluting	For all TSF design aspects (cover, drainage, embankments etc.)	None	200 year design life			300 yr. design life	150 yr.	1000 yr.	300 year	DMIRS Guideline as minimum standards
			Drainage - Top surface	Stability, non-polluting	Return event	PMP Critical design storm event	PMP				4 hr. PMP	72 hr. PMP	PMF - part contain on facility & pass remainder through spillway	DMIRS/ANCOLD Guideline to ensure long term structural integrity of the facility
			Drainage - Embankments / benches	Stability, non-polluting	Return event	PMP Critical design storm event		1:200 year ARI rainfall events	1:500 year ARI rainfall events		1:200 yr. ARI	PMP	PMP	High range - conservative
			Drainage - Perimeter	Stability, non-polluting	Return event	PMP Critical design storm event	PMP / 1:300-1:500	1:10,000 for flood events for diverted creeks			1:300 yr. ARI	PMP	PMP - manage	
				Land Use, non-polluting	Limit net infiltration to HOSTILE waste	DMIRS requirement							Dependant on nature of hostile material - North Star Tailings Classify as benign	The International Network for Acid Prevention - GARD (Global Acid Rock Drainage) Guide. Minimise net infiltration and oxygen diffusion to hostile waste and subsequent solute release to environment.
			Surface cover	Land Use, non-polluting	Support plant growth	Vegetated outcome "TSF cover to withstand terrestrial fauna"					500mm (over tailings plus growth medium)	2,000mm	Support native plant growth except in areas designed as angle of repose slopes or durable fresh waste armour	Minimum thickness required to achieve nominated end land use (e.g. vegetated outcome etc.). Practicality of placing suitable rock on poorly consolidated and low strength tailings surfaces and some angle of repose batters will significantly limit vegetation establishment locally
			Erosion stability - water	Stability, land Use, non-polluting	Erosion loss	Erosionally Stable	5 t/ha/yr. mean erosion loss				30 t/ha/yr.	5 t/ha/yr.	Does not affect cover integrity, erosion features are self stabilising and sediment emissions do not have significant off site impacts	Mid -High range based on risk assessment. Surface materials to be durable and enduring to meet specified design life.
			Erosion - wind	Non-polluting	Dust release	Erosionally Stable	$4g/m^2$/month (Environmental Protection Authority Victoria, Publication 1191. Protocol for Environmental Management State Environment Protection Policy Published in December 2007)				No dust leaving the lease boundary	No dust leaving the TSF boundary	All tailings exposures will be covered with rock	High range - conservative

Option	Embankment Profile		Embankment Cover			Top Surface Cover	
	Total Volume (m3)	Surface area (m2)	2m Cover	1m Cover	.5m Cover	.5m	1m
20/15 single slope	3,742,000	465,000	930,000	465,000	232,500	4,702,500	9,405,000
2X 35m high lifts, 18 degree slopes with 30m wide berm	5,555,000	547,000	1,094,000	547,000	273,500	4,702,500	9,405,000
3X 23m high lifts 18 degree slopes with 20m wide berms	6,348,000	579,000	1,158,000	579,000	289,700	4,702,500	9,405,000
five degree slope from crest	68,756,000	2,960,000	5,920,000	2,960,000	1,480,000	4,702,500	9,405,000
7 X 10m lifts at 37 degrees with 20m berms	4,031,000	516,000	1,032,000	516,000	258,000	4,702,500	9,405,000

20/15 single slope	Reprofile	Cover Embankment (inc)	Cover Top	Total
2m Embankment Cover and 1m Top Cover	3,742,000	930,000	9,405,000	13,147,000
1m Embankment Cover and 1m Top Cover	3,742,000	465,000	9,405,000	13,147,000
.5m Embankment Cover and 1m Top Cover	3,742,000	232,500	9,405,000	13,147,000
0m Embankment Cover and 1m Top Cover	3,742,000	0	9,405,000	13,147,000
2m Embankment Cover and .5m Top Cover	3,742,000	930,000	4,702,500	8,444,500
1m Embankment Cover and .5m Top Cover	3,742,000	465,000	4,702,500	8,444,500
.5m Embankment Cover and .5m Top Cover	3,742,000	232,500	4,702,500	8,444,500
0m Embankment Cover and .5m Top Cover	3,742,000	0	4,702,500	8,444,500

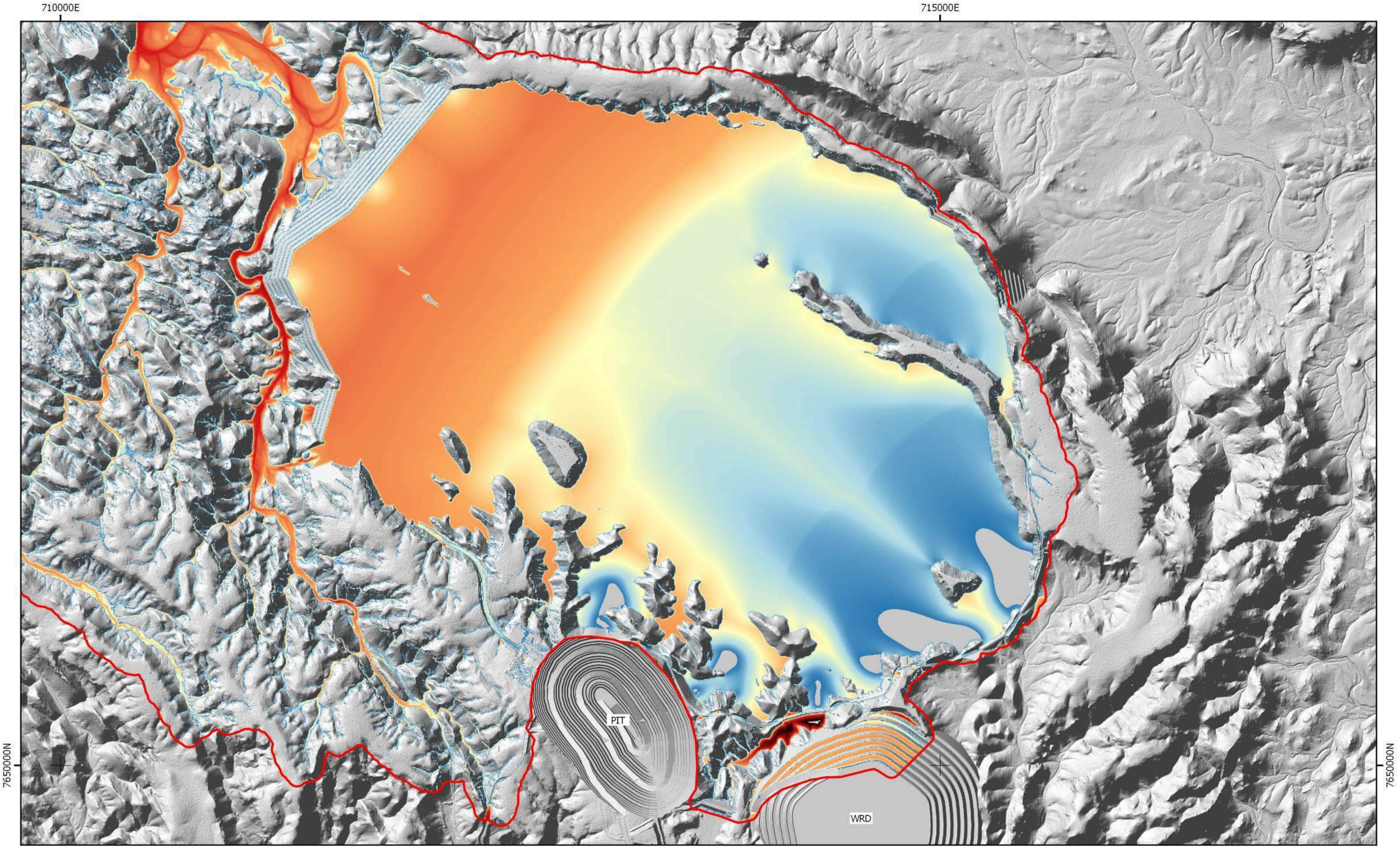
2X 35m high lifts, 18 degree slopes with 30m B	Reprofile	Cover Embankment (inc)	Cover Top	Total
2m Embankment Cover and 1m Top Cover	5,555,000	1,094,000	9,405,000	14,960,000
1m Embankment Cover and 1m Top Cover	5,555,000	547,000	9,405,000	14,960,000
.5m Embankment Cover and 1m Top Cover	5,555,000	273,500	9,405,000	14,960,000
0m Embankment Cover and 1m Top Cover	5,555,000	0	9,405,000	14,960,000
2m Embankment Cover and .5m Top Cover	5,555,000	1,094,000	4,702,500	10,257,500
1m Embankment Cover and .5m Top Cover	5,555,000	547,000	4,702,500	10,257,500
.5m Embankment Cover and .5m Top Cover	5,555,000	273,500	4,702,500	10,257,500
0m Embankment Cover and .5m Top Cover	5,555,000	0	4,702,500	10,257,500

3X 23m high lifts 18 degree slopes with 20m wide berms	Reprofile	Cover Embankment (inc)	Cover Top	Total
2m Embankment Cover and 1m Top Cover	6,348,000	1,158,000	9,405,000	15,753,000
1m Embankment Cover and 1m Top Cover	6,348,000	579,000	9,405,000	15,753,000
.5m Embankment Cover and 1m Top Cover	6,348,000	289,700	9,405,000	15,753,000
0m Embankment Cover and 1m Top Cover	6,348,000	0	9,405,000	15,753,000
2m Embankment Cover and .5m Top Cover	6,348,000	1,158,000	4,702,500	11,050,500
1m Embankment Cover and .5m Top Cover	6,348,000	579,000	4,702,500	11,050,500
.5m Embankment Cover and .5m Top Cover	6,348,000	289,700	4,702,500	11,050,500
0m Embankment Cover and .5m Top Cover	6,348,000	0	4,702,500	11,050,500

Five degree slope from crest	Reprofile	Cover Embankment (inc)	Cover Top	Total
2m Embankment Cover and 1m Top Cover	68,756,000	5,920,000	9,405,000	78,161,000
1m Embankment Cover and 1m Top Cover	68,756,000	2,960,000	9,405,000	78,161,000
.5m Embankment Cover and 1m Top Cover	68,756,000	1,480,000	9,405,000	78,161,000
0m Embankment Cover and 1m Top Cover	68,756,000	0	9,405,000	78,161,000
2m Embankment Cover and .5m Top Cover	68,756,000	5,920,000	4,702,500	73,458,500
1m Embankment Cover and .5m Top Cover	68,756,000	2,960,000	4,702,500	73,458,500
.5m Embankment Cover and .5m Top Cover	68,756,000	1,480,000	4,702,500	73,458,500
0m Embankment Cover and .5m Top Cover	68,756,000	0	4,702,500	73,458,500

7 X 10m lifts at 37 degrees with 20m berms	Reprofile	Cover Embankment (inc)	Cover Top	Total
2m Embankment Cover and 1m Top Cover	4,031,000	1,032,000	9,405,000	13,436,000
1m Embankment Cover and 1m Top Cover	4,031,000	516,000	9,405,000	13,436,000
.5m Embankment Cover and 1m Top Cover	4,031,000	258,000	9,405,000	13,436,000
0m Embankment Cover and 1m Top Cover	4,031,000	0	9,405,000	13,436,000
2m Embankment Cover and .5m Top Cover	4,031,000	1,032,000	4,702,500	8,733,500
1m Embankment Cover and .5m Top Cover	4,031,000	516,000	4,702,500	8,733,500
.5m Embankment Cover and .5m Top Cover	4,031,000	258,000	4,702,500	8,733,500
0m Embankment Cover and .5m Top Cover	4,031,000	0	4,702,500	8,733,500

APPENDIX D



LEGEND

Hydraulic Model Domain

MODELLED DEPTH (M)

0.1	1.0
0.25	5.0
0.5	20.0

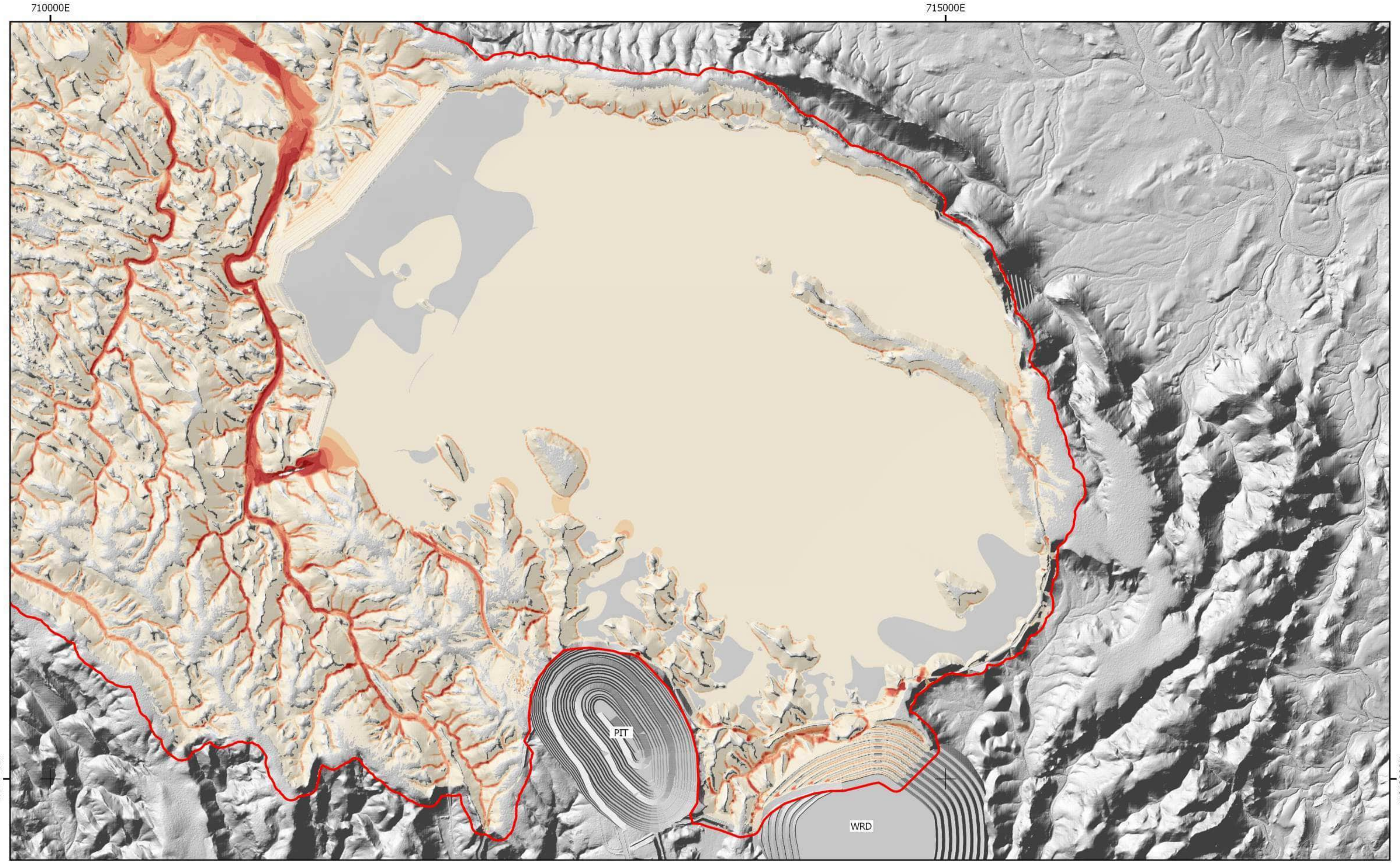
0 0.5 1 1.5 2 km

FIGURE 001 - PMP 360 MINUTE - MAXIMUM MODELLED DEPTH

FORTESCUE METALS GROUP - NORTH STAR
TAILINGS STORAGE FACILITY
CLOSURE HYDRAULIC PERFORMANCE ASSESSMENT

ATC Williams

DATE: 2019-02-22
CRS: GDA94 / MGA Zone 50



LEGEND

Hydraulic Model Domain

MODELLED VELOCITY (m/s)

- < 0.8 m/s
- 0.8 - 1.0 m/s
- 1.0 - 1.5 m/s
- 1.5 - 2.0 m/s
- 2.0 - 3.0 m/s
- 3.0 - 10.0 m/s

FIGURE 002 - PMP 360 MINUTE - MAXIMUM MODELLED VELOCITY

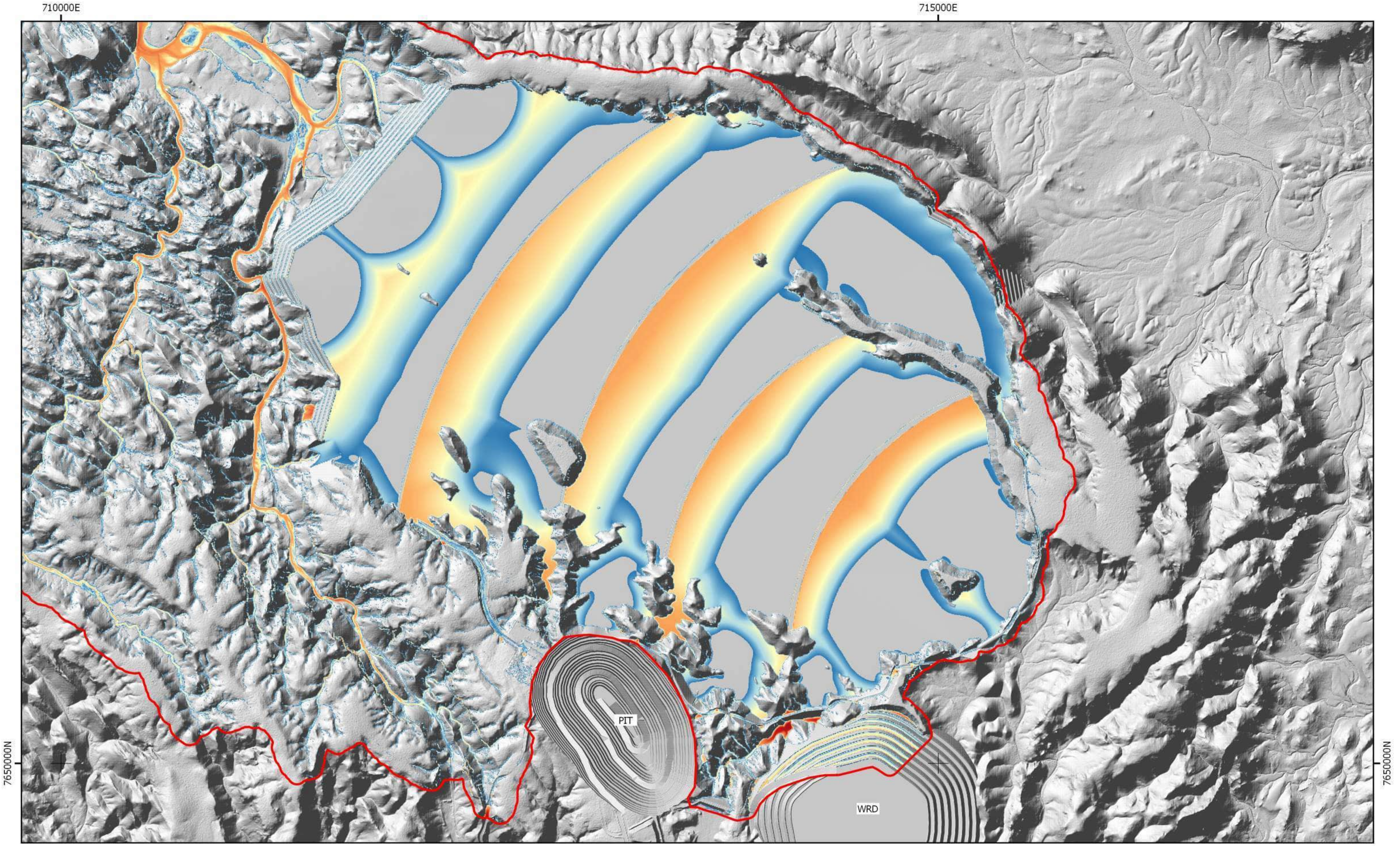
FORTESCUE METALS GROUP - NORTH STAR
TAILINGS STORAGE FACILITY
CLOSURE HYDRAULIC PERFORMANCE ASSESSMENT

0 0.5 1 1.5 2 km

N

ATC Williams

DATE: 2019-02-22
CRS: GDA94 / MGA Zone 50



LEGEND

Hydraulic Model Domain

MODELLED DEPTH (M)

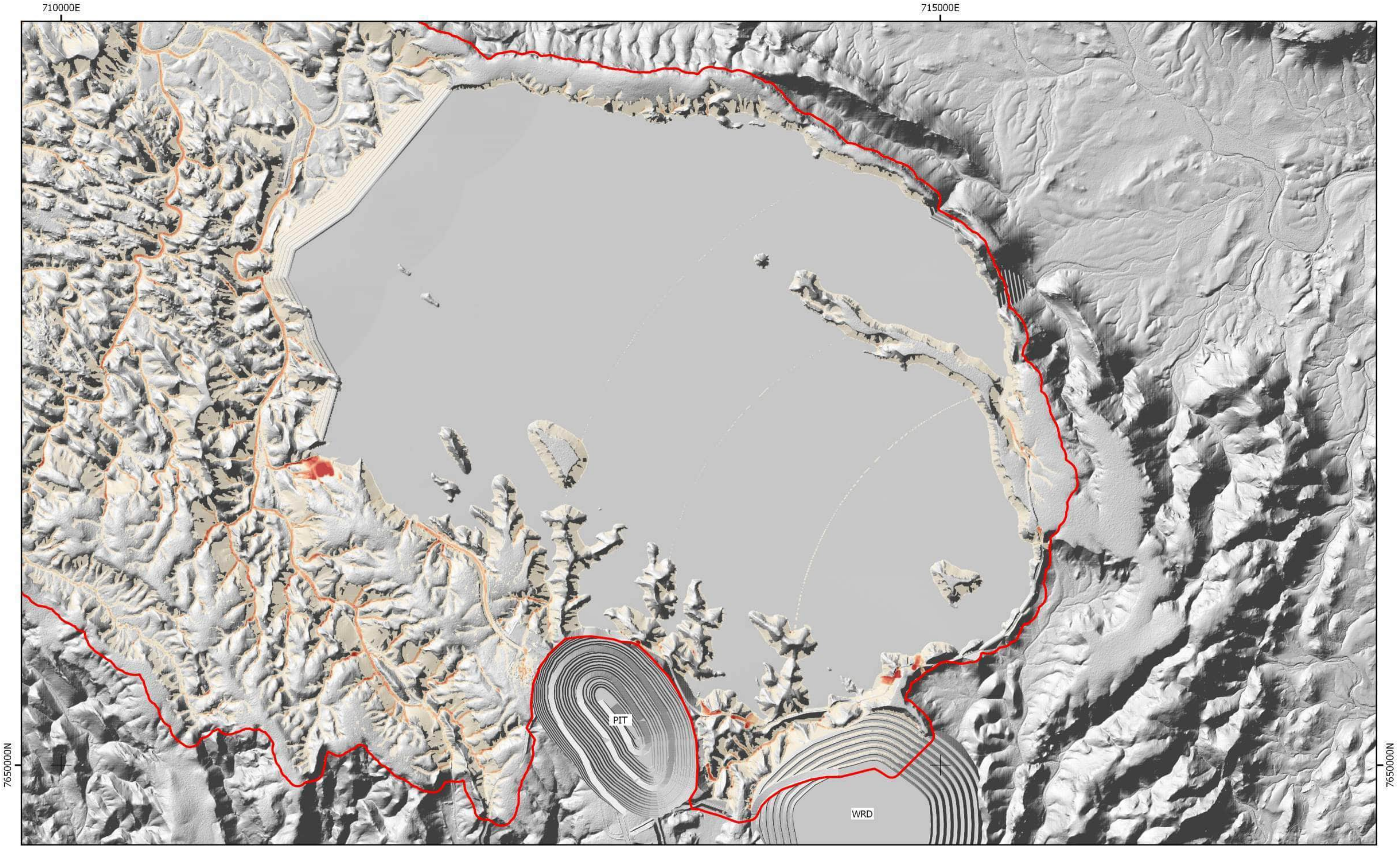
0.1
0.25
0.5
1.0
5.0
20.0

FIGURE 003 - 0.2% AEP 360 MINUTE MAXIMUM MODELLED DEPTH
(CLOSURE SURFACE BUNDING MODELLED)

FORTESCUE METALS GROUP - NORTH STAR
TAILINGS STORAGE FACILITY
CLOSURE HYDRAULIC PERFORMANCE ASSESSMENT

ATC Williams

DATE: 2019-02-22
CRS: GDA94 / MGA Zone 50



LEGEND

Hydraulic Model Domain

MODELLED VELOCITY (m/s)

< 0.8 m/s
0.8 - 1.0 m/s
1.0 - 1.5 m/s
1.5 - 2.0 m/s
2.0 - 3.0 m/s
3.0 - 10.0 m/s

FIGURE 004 - 0.2% AEP 360 MINUTE MAXIMUM MODELLED DEPTH
(CLOSURE SURFACE BUNDING MODELLED)

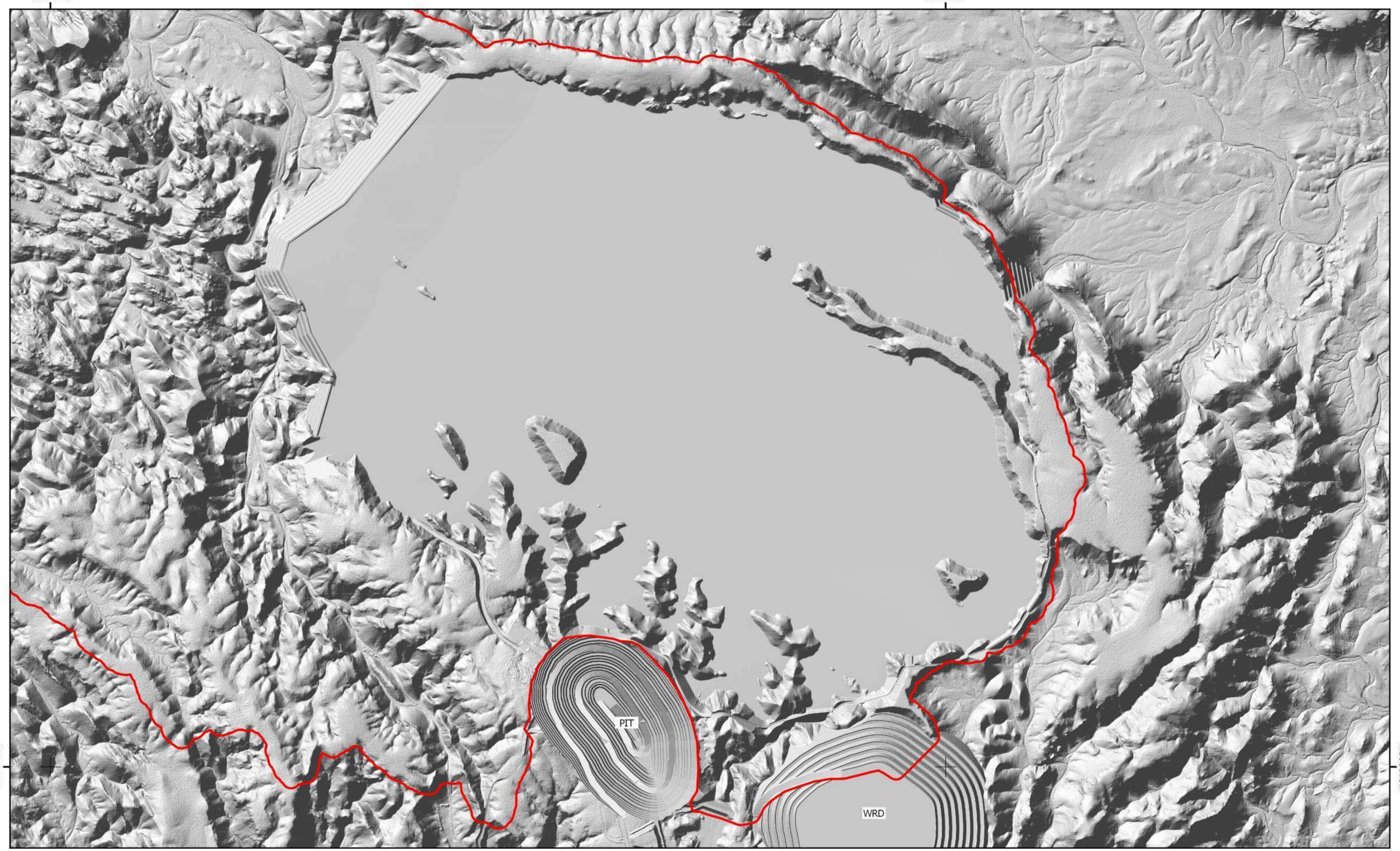
FORTESCUE METALS GROUP - NORTH STAR
TAILINGS STORAGE FACILITY
CLOSURE HYDRAULIC PERFORMANCE ASSESSMENT

0 0.5 1 1.5 2 km

N

ATC Williams

DATE: 2019-02-22
CRS: GDA94 / MGA Zone 50



LEGEND

Hydraulic Model Domain

MODELLED DEPTH (M)

0.1
0.25
0.5
1.0
5.0
20.0

0 0.5 1 1.5 2 km

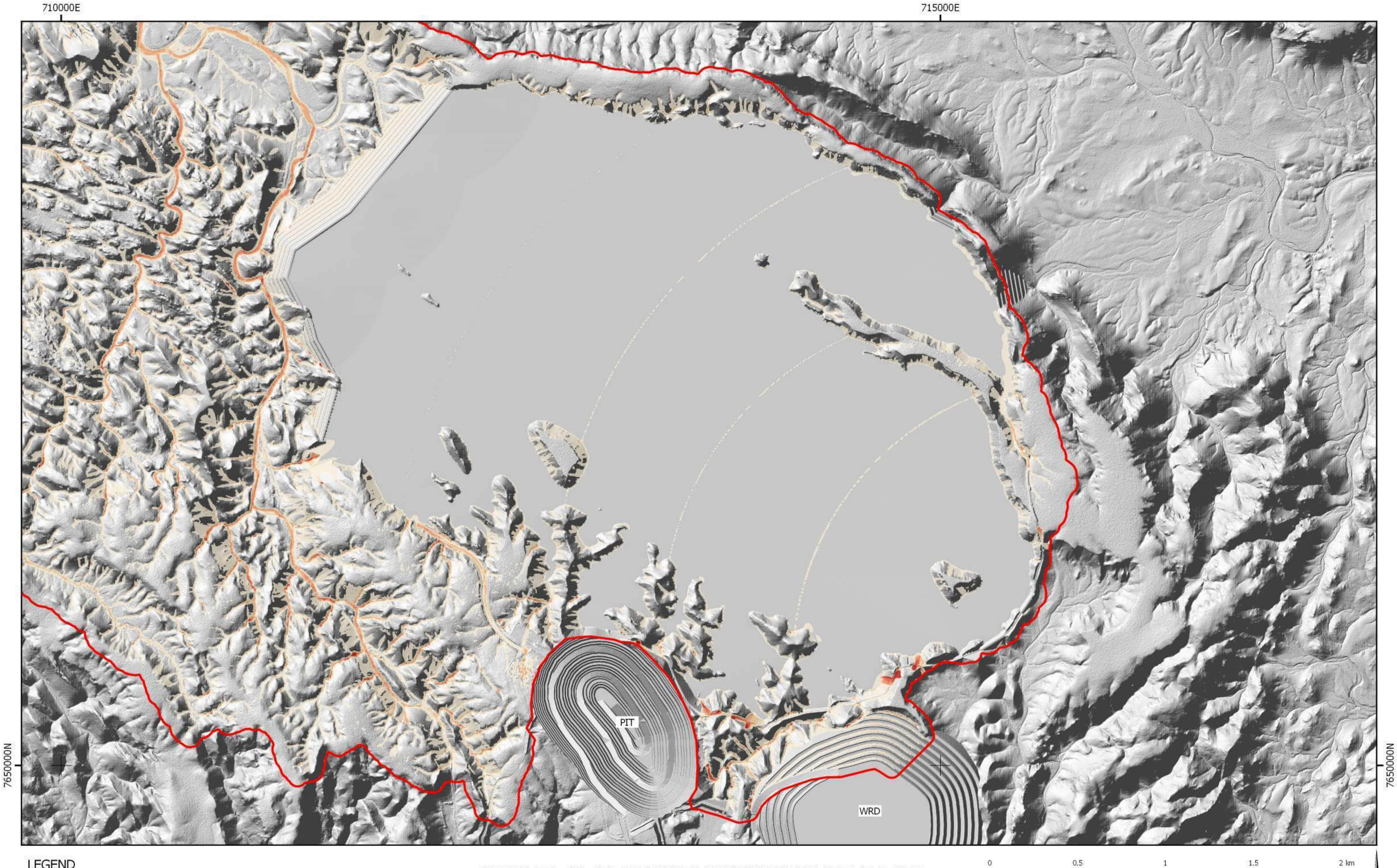
N

FIGURE 005 - 5% AEP 360 MINUTE MAXIMUM MODELLED DEPTH
(CLOSURE SURFACE BUNDING MODELLED)

FORTESCUE METALS GROUP - NORTH STAR
TAILINGS STORAGE FACILITY
CLOSURE HYDRAULIC PERFORMANCE ASSESSMENT

ATC Williams

DATE: 2019-02-22
CRS: GDA94 / MGA Zone 50



LEGEND

Hydraulic Model Domain

MODELLED VELOCITY (m/s)

< 0.8 m/s
0.8 - 1.0 m/s
1.0 - 1.5 m/s
1.5 - 2.0 m/s
2.0 - 3.0 m/s
3.0 - 10.0 m/s

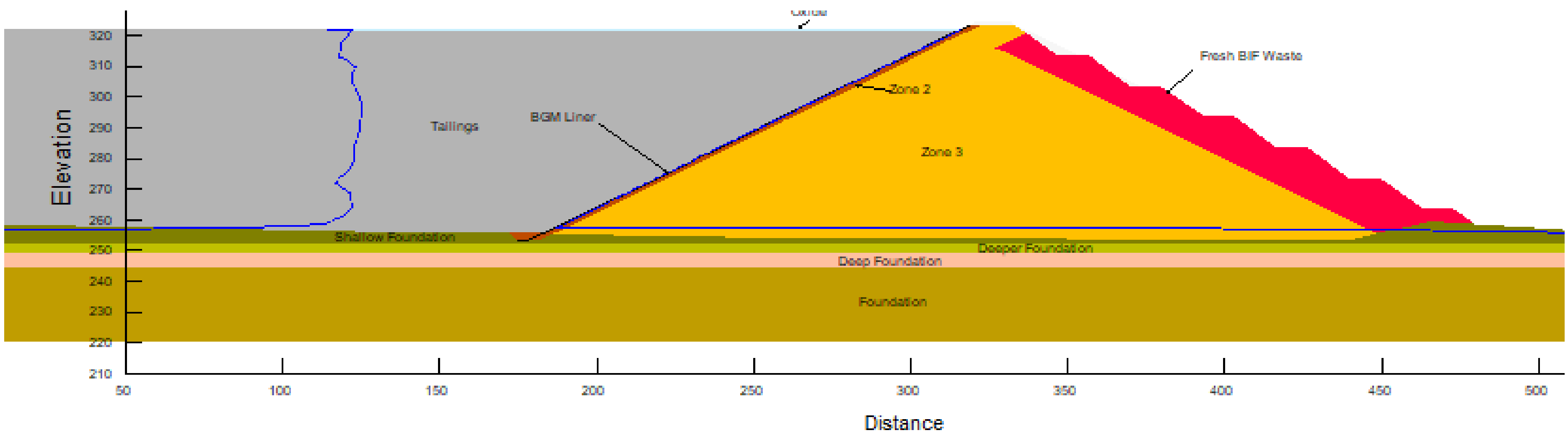
FIGURE 006 - 5% AEP 360 MINUTE MAXIMUM MODELLED VELOCITY
(CLOSURE SURFACE BUNDING MODELLED)

FORTESCUE METALS GROUP - NORTH STAR
TAILINGS STORAGE FACILITY
CLOSURE HYDRAULIC PERFORMANCE ASSESSMENT

ATC Williams

DATE: 2019-02-22
CRS: GDA94 / MGA Zone 50

APPENDIX E



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IRON BRIDGE OPERATIONS PTY LTD

NORTH STAR MAGNETITE - STAGE 2

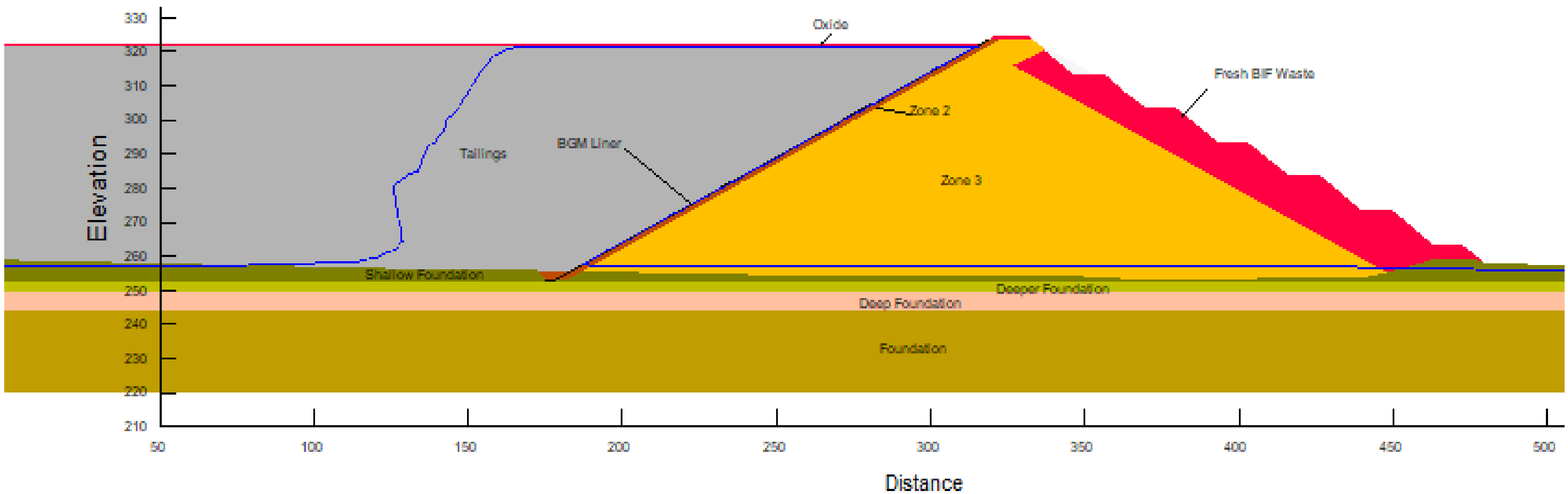
CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis - End of TSF Stage 7

Date: 12-12-19 **Job No:** 114185.15

FIGURE E1

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 1



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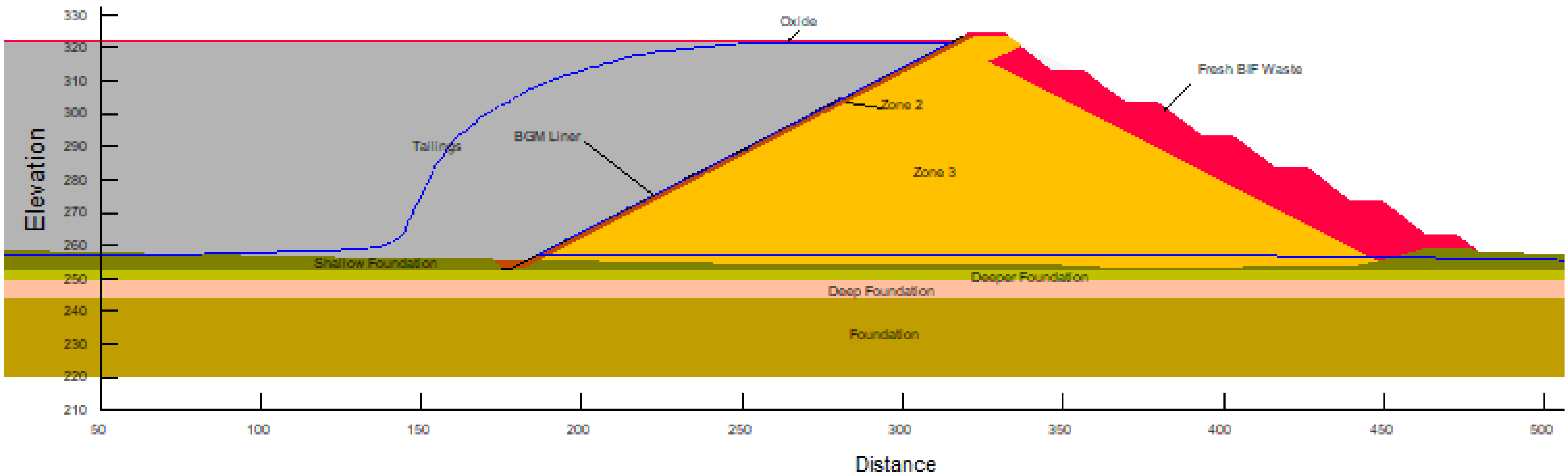
NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis - 6 Months After Mining

Date:	12-12-19	**Job No:**	114185.15	**FIGURE E2**

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 2



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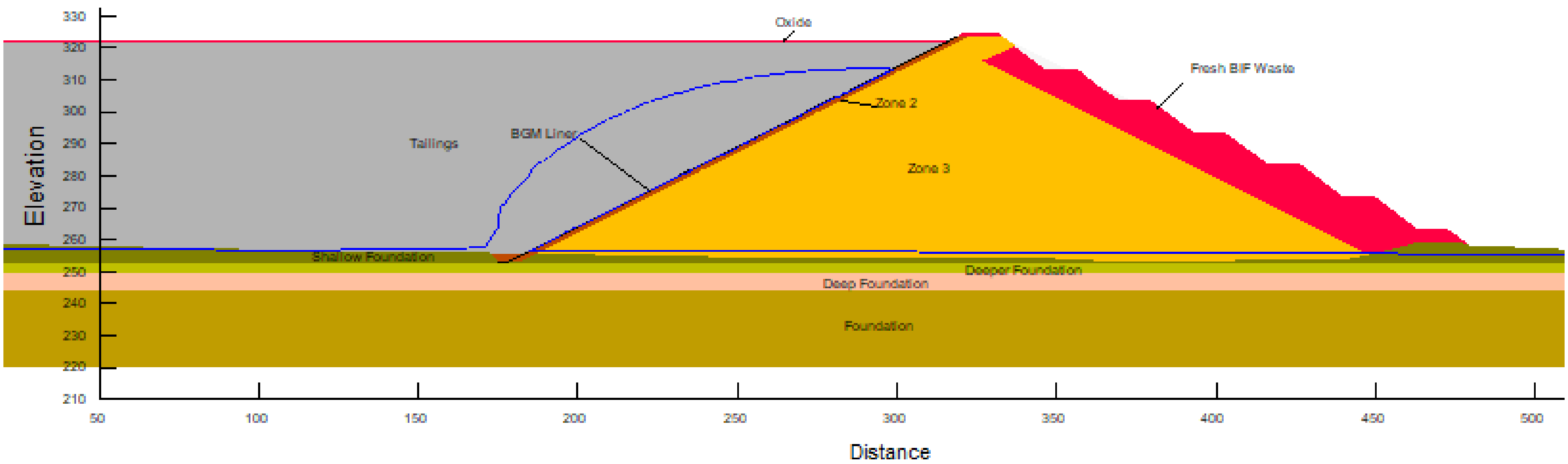
NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis - 1 Year After Mining

Date:	12-12-19	**Job No:**	114185.15	**FIGURE E3**

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 3



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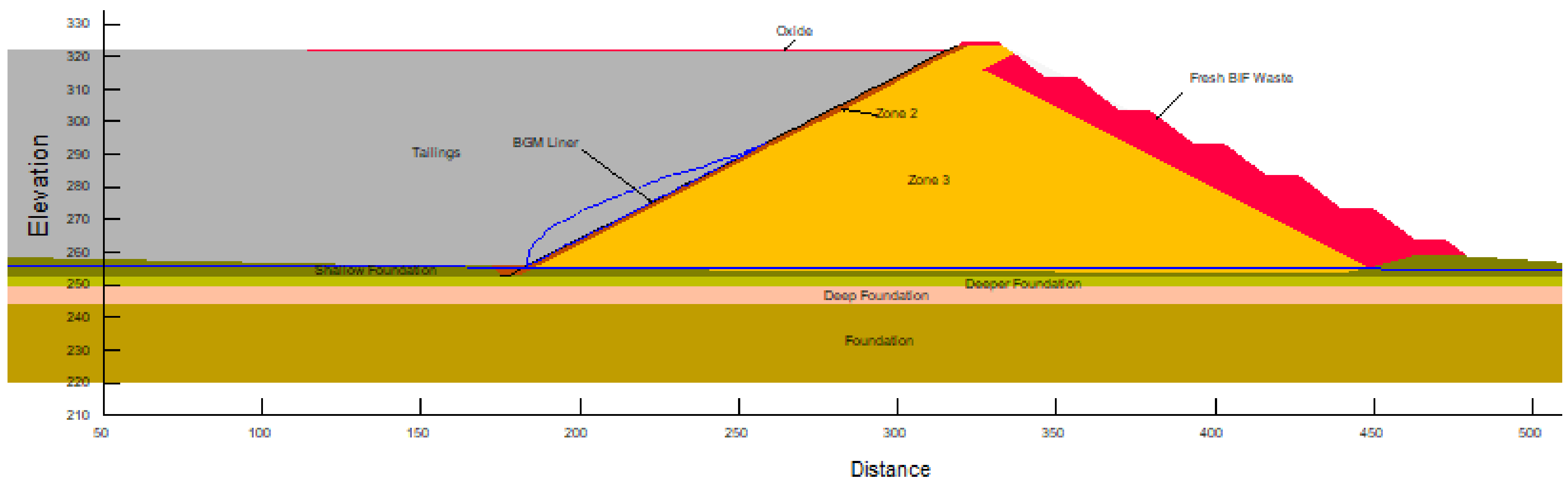
NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis - 5 Years After Mining

Date:	12-12-19	**Job No:**	114185.15	**FIGURE E4**

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 4



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NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis - 10 Years After Mining

Date:	12-12-19	**Job No:**	114185.15	**FIGURE E5**

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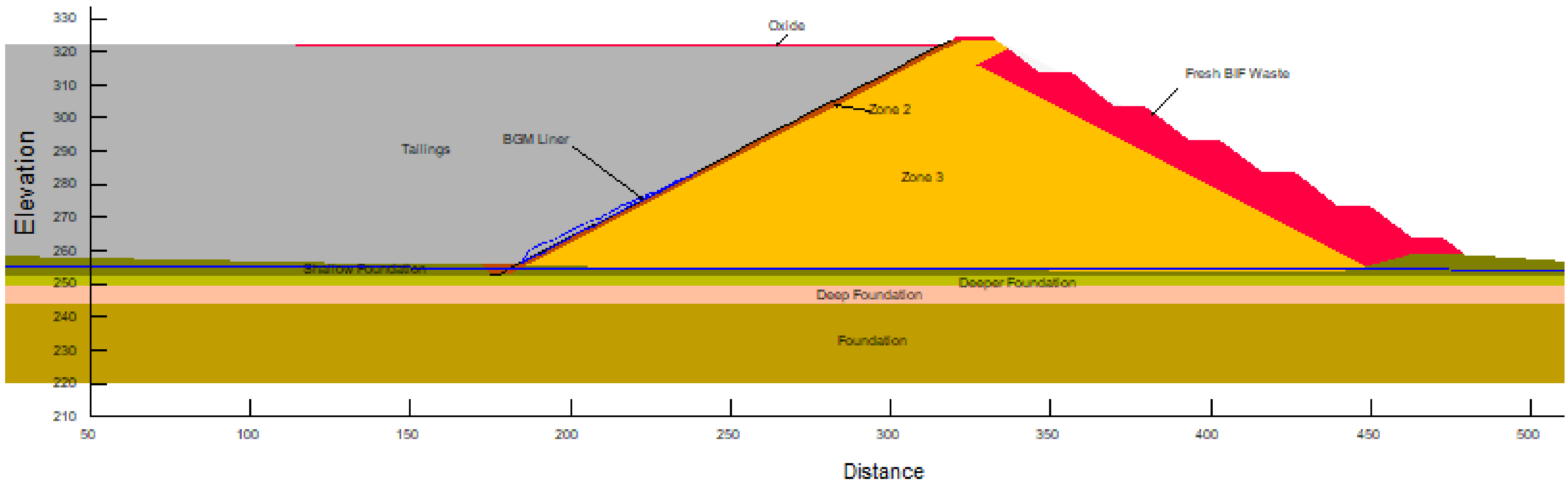

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NORTH STAR MAGNETITE - STAGE 2

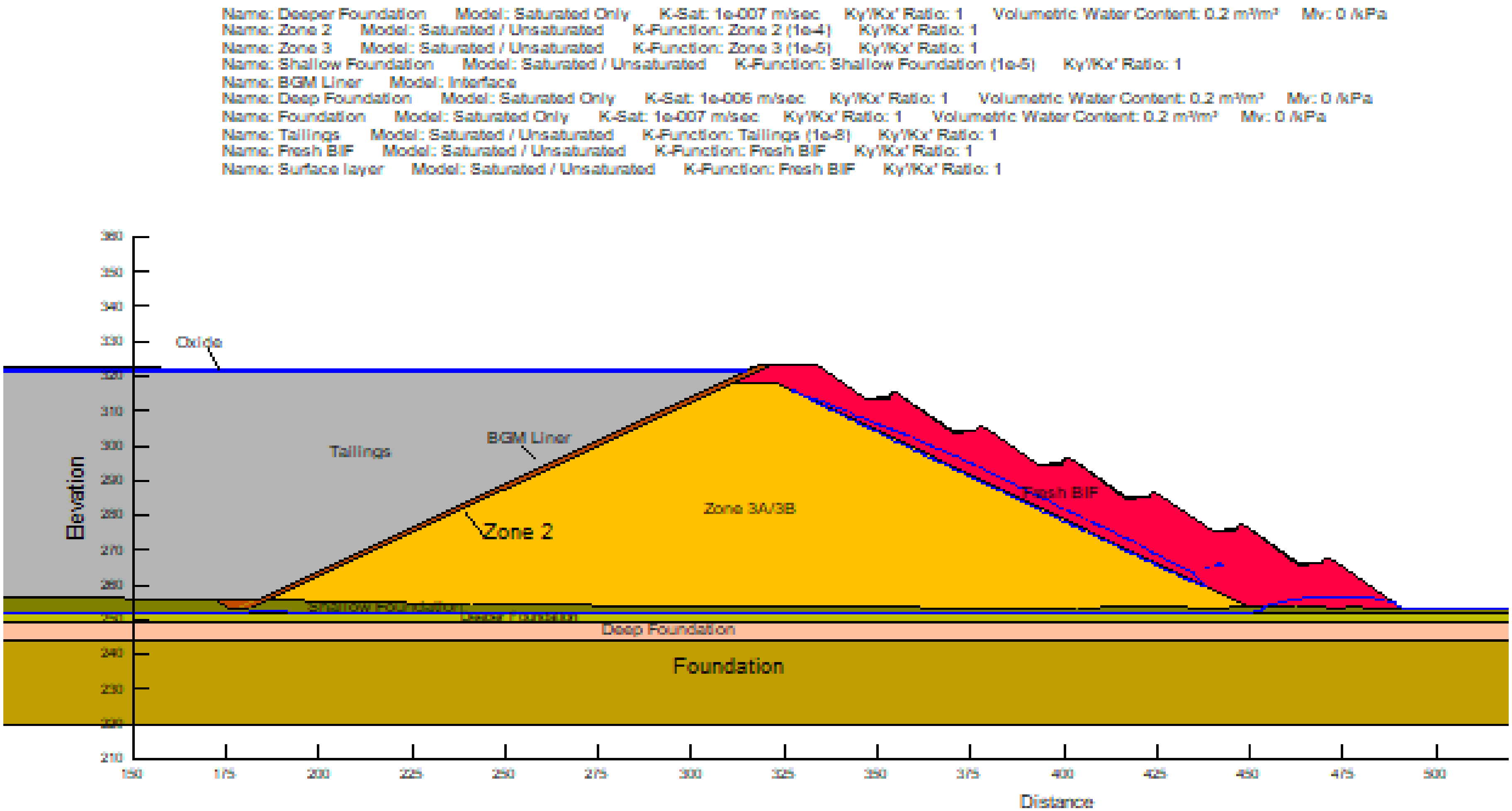
CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis - 25 Years After Mining

Date: 12-12-19 **Job No:** 114185.15

FIGURE E6

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 6



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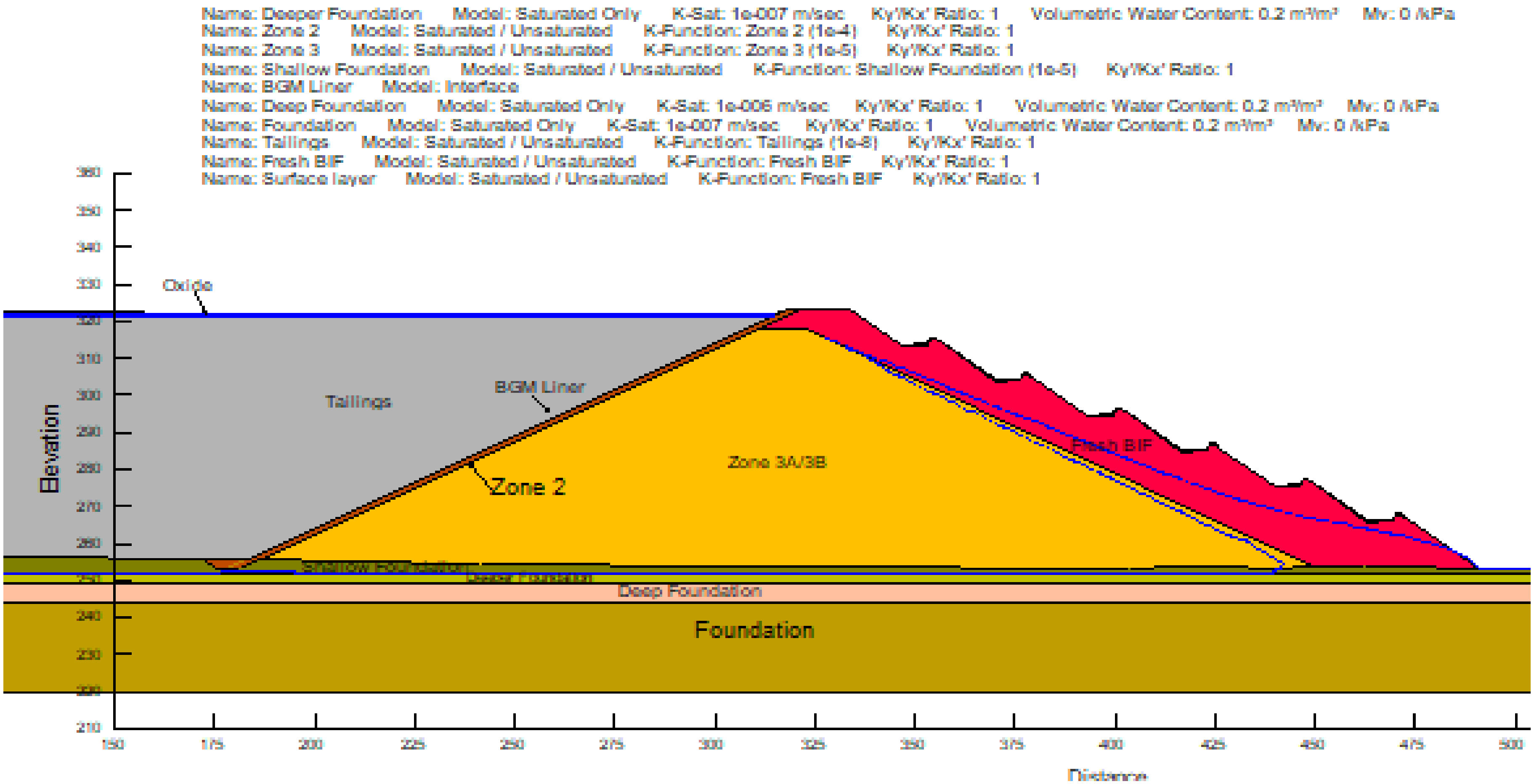
NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis - 6 HOUR PMP

Date:	12-12-19	Job No:	114185.15	FIGURE E7

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 7



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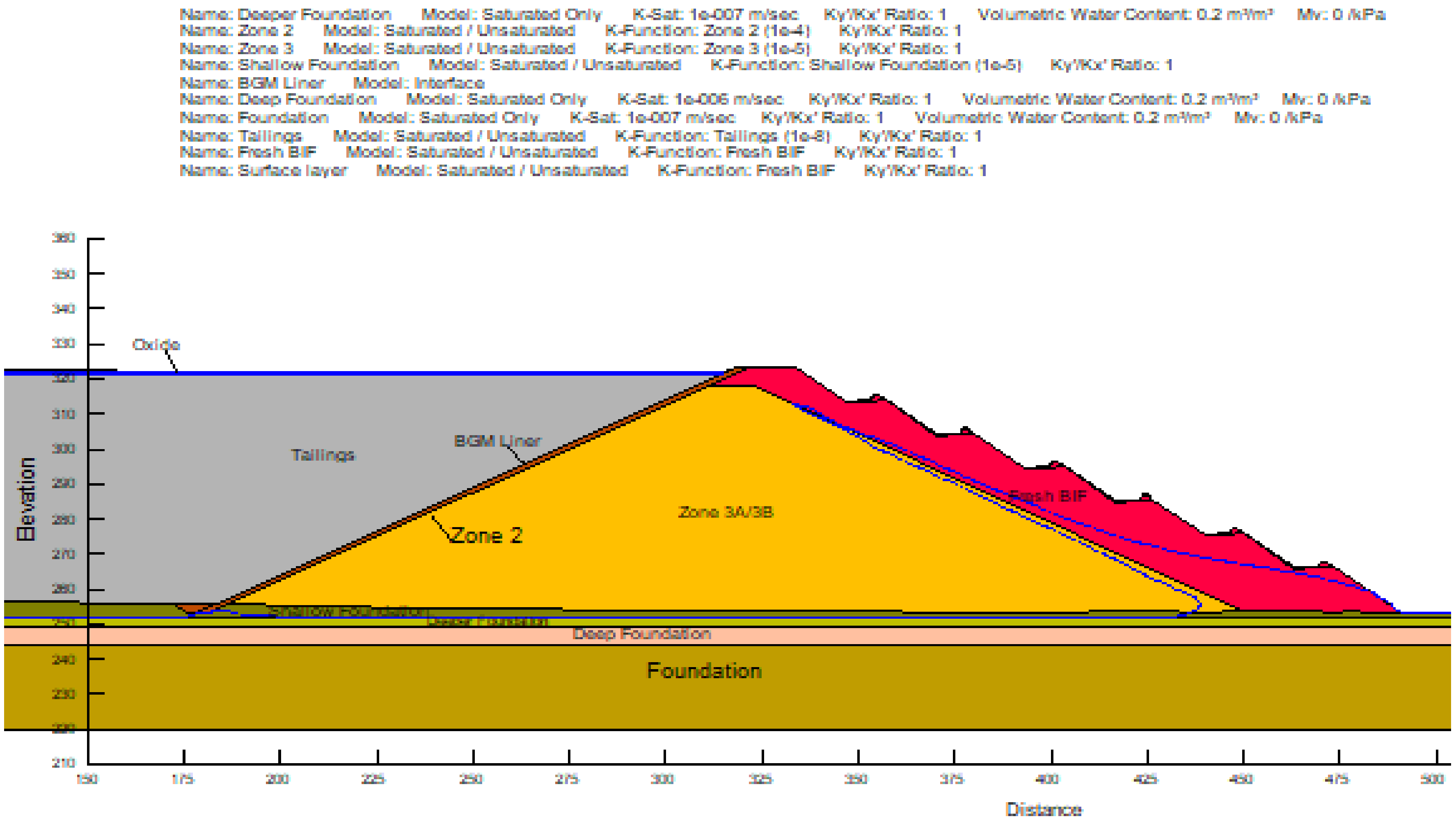
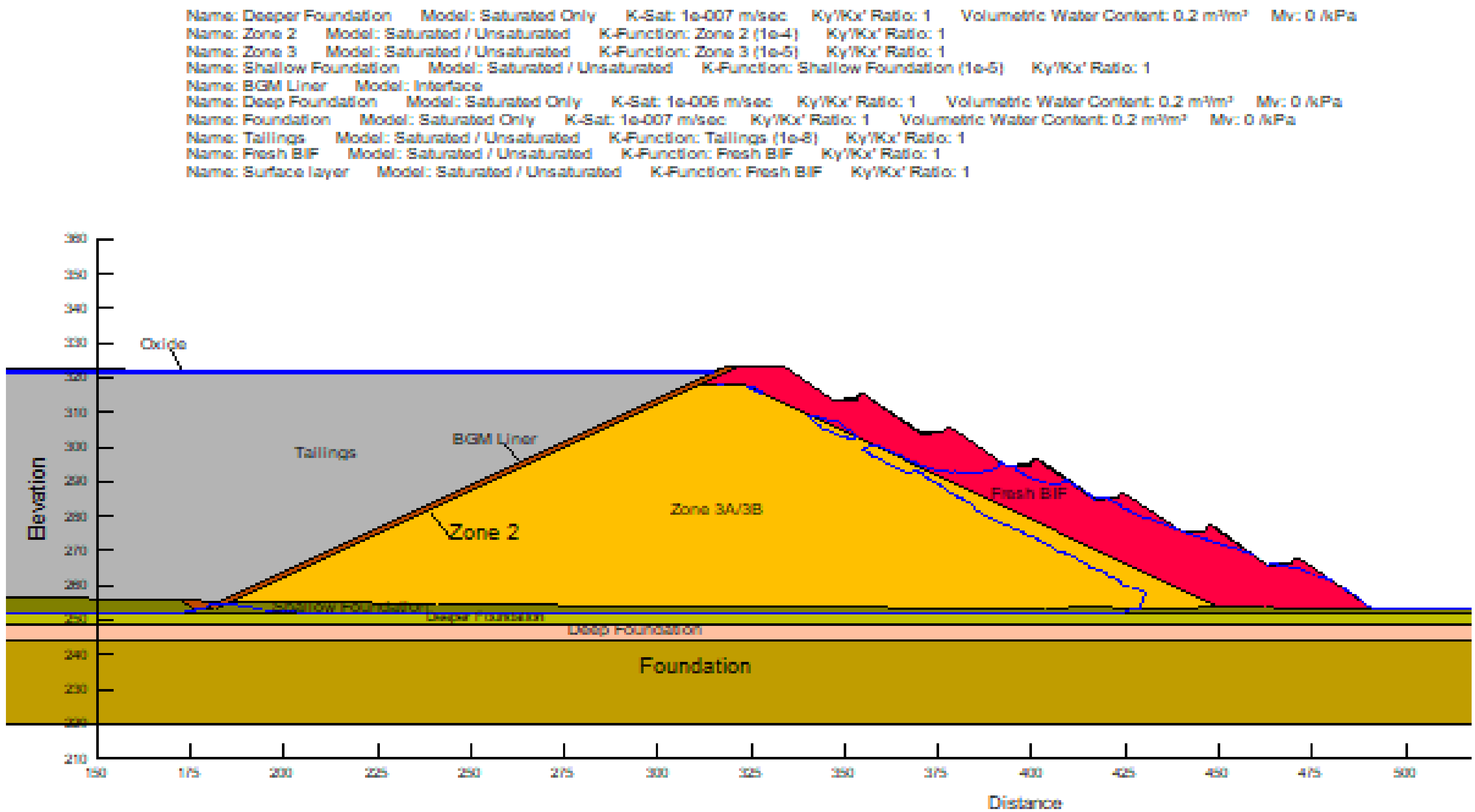
	Seepage Analysis -12 HOUR PMP		
	Date: 12-12-19	**Job No:** 114185.15	**FIGURE E8**



ATC Williams www.atcwilliams.com.au	**IRON BRIDGE OPERATIONS PTY LTD** **NORTH STAR MAGNETITE - STAGE 2** **CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY**
	Seepage Analysis - 24 HOUR PMP
	Date: 12-12-19 **Job No:** 114185.15 — **FIGURE E9**

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 9



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NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis -48 HOUR PMP

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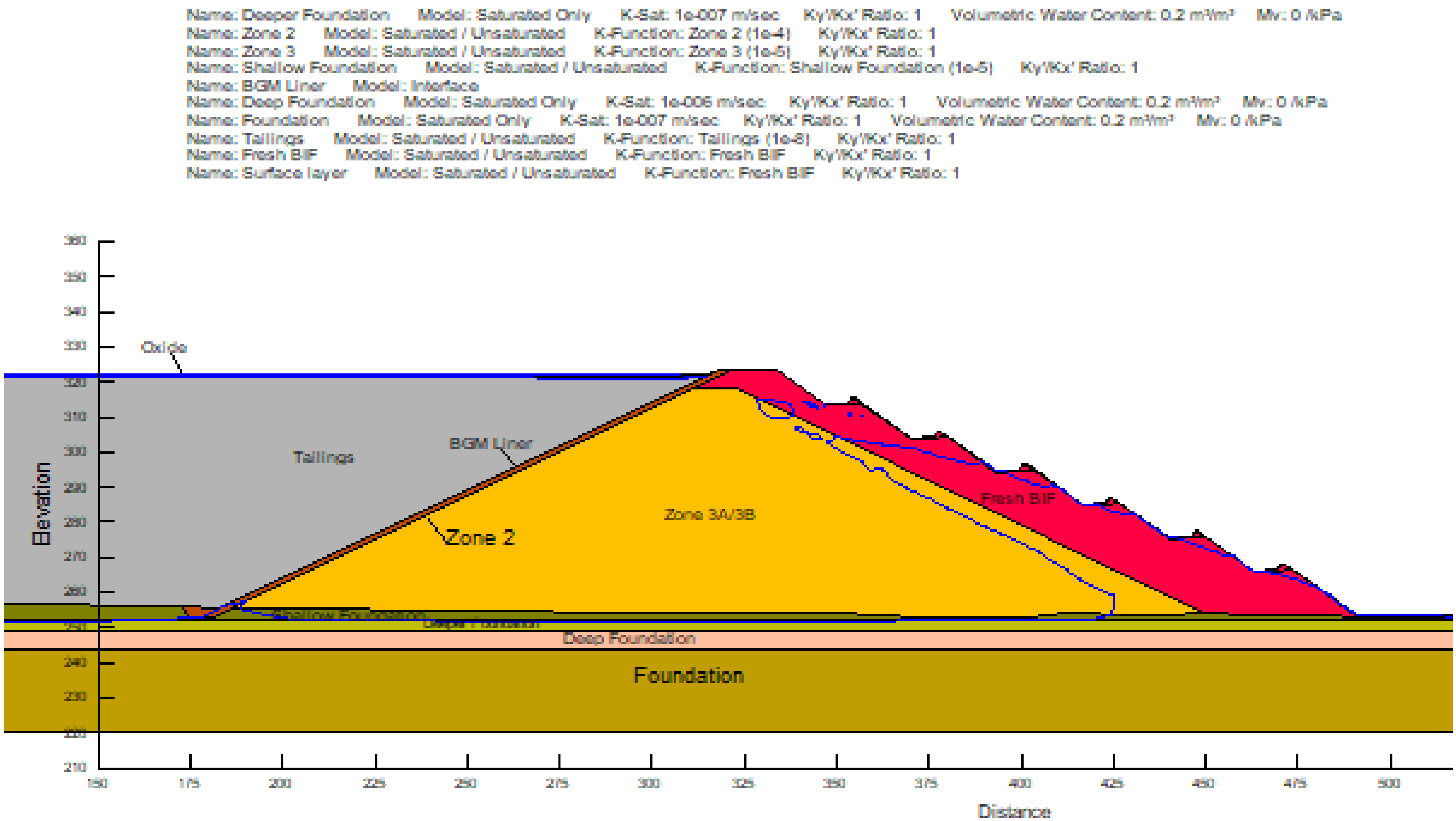
Date:	12-12-19	**Job No:**	114185.15	**FIGURE E10**

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 10



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NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Seepage Analysis -72 HOUR PMP

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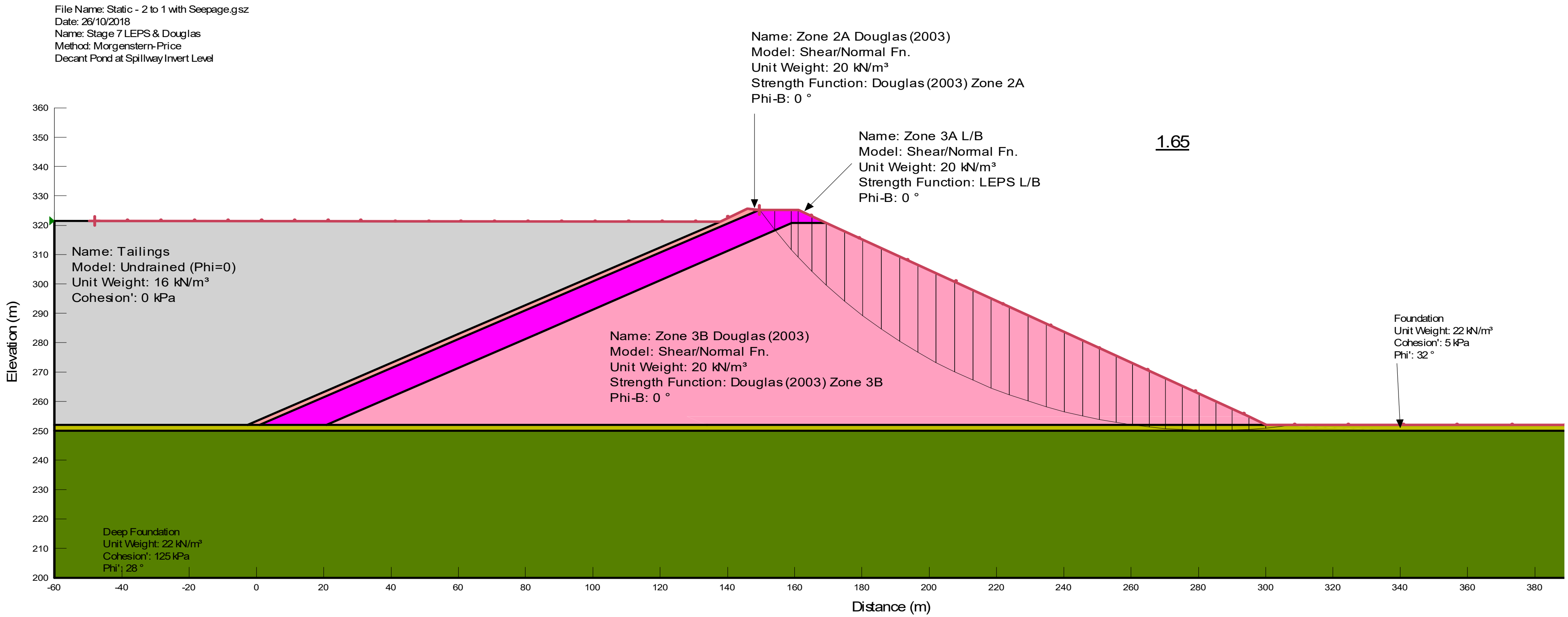
Date:	12-12-19	Job No:	114185.15	FIGURE E11

C:\Users\johnl\Documents\Job Files\New work\North Star\VG 2e-2\Unsat\Z3 corrected\Seepage Results Rev2.xlsx 11

APPENDIX F



Directory: K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Models\Final - LEPS & Douglas\

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NORTH STAR MAGNETITE - STAGE 2

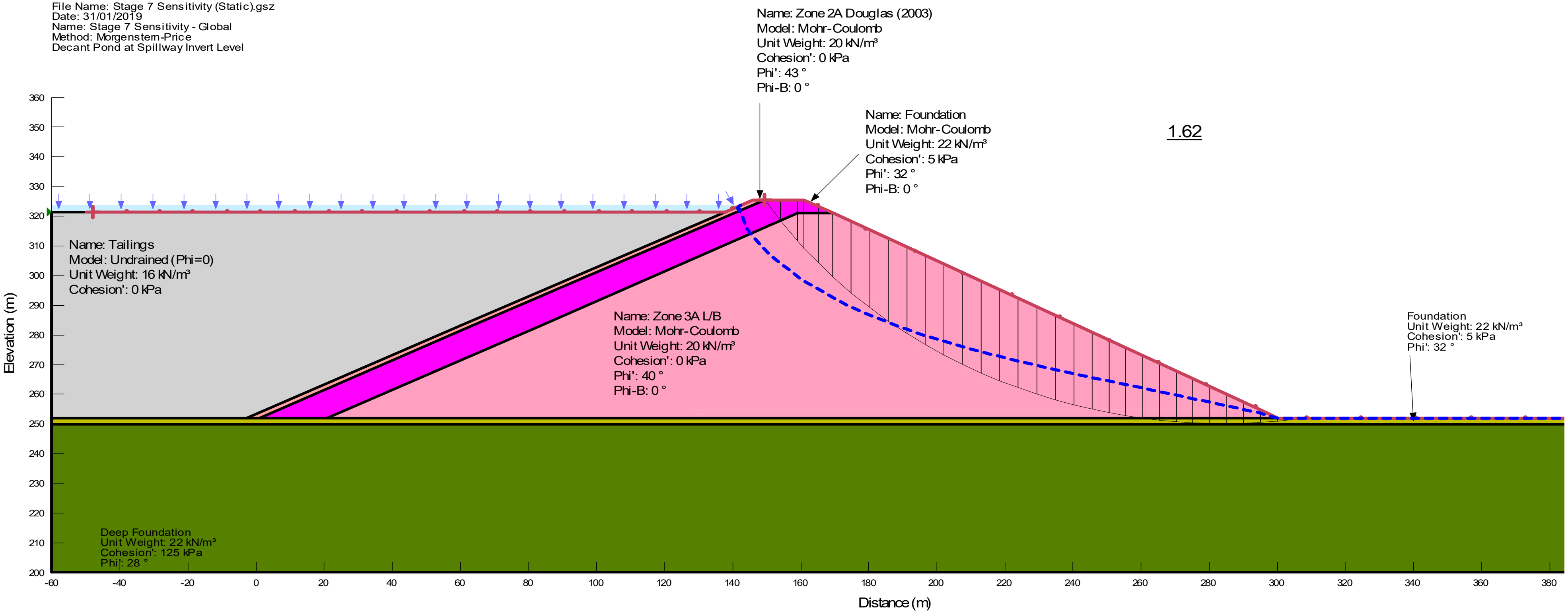
DESIGN REPORT FOR TAILINGS STORAGE FACILITY

TSF Stage 7 - (2:1 D/S Slope) - Static

Date: 14/02/2019 **Job No:** 114185.14

FIGURE F1

K:\Projects\114\114185 North Star Magnetite Stage 2\14 Detailed Design of TSF to IFC Stage\Data and Calcs\SlopeW\Excel\Stability Summary Rev2.xlsx G5



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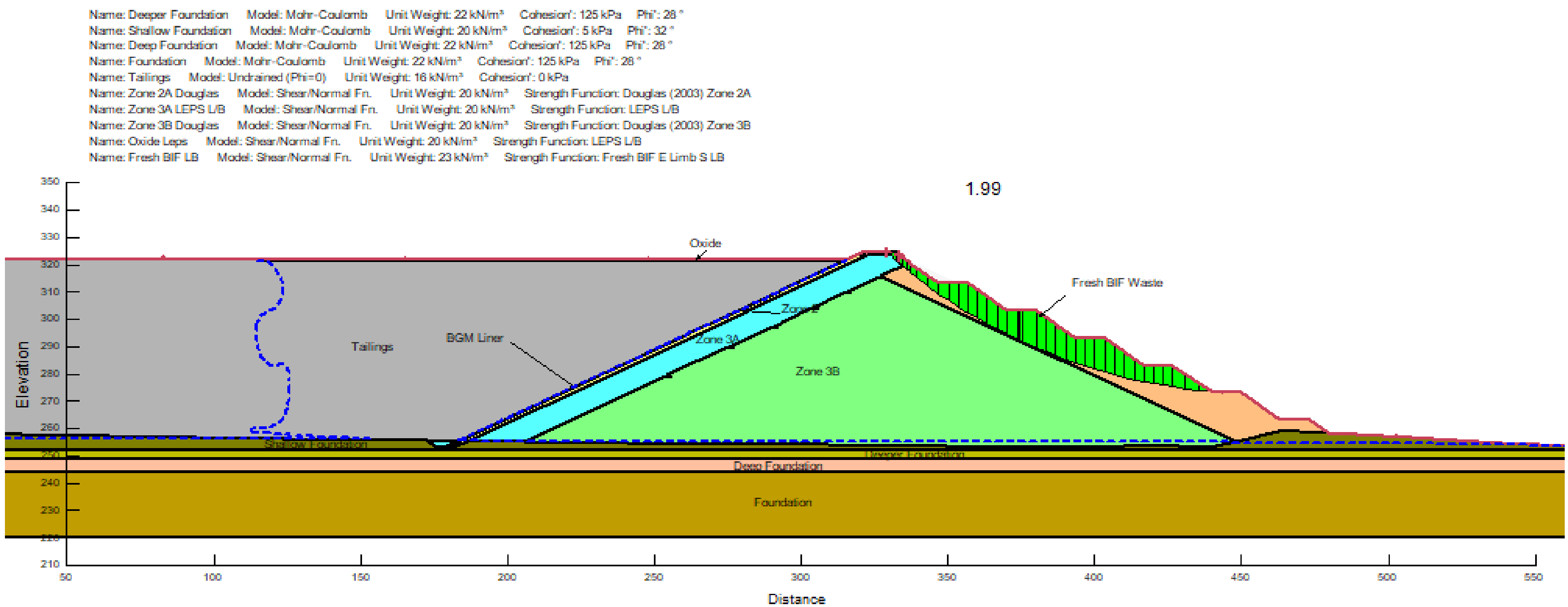
NORTH STAR MAGNETITE - STAGE 2

STABILITY ANALYSES SENSITIVITY ANALYSES

TSF Stage 7 - (2:1 D/S Slope) - Static

Date:	14/02/2019	Job No:	114185.14	FIGURE F2



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NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Stability Analysis - Closure Face Fresh BIF (E Limb S)

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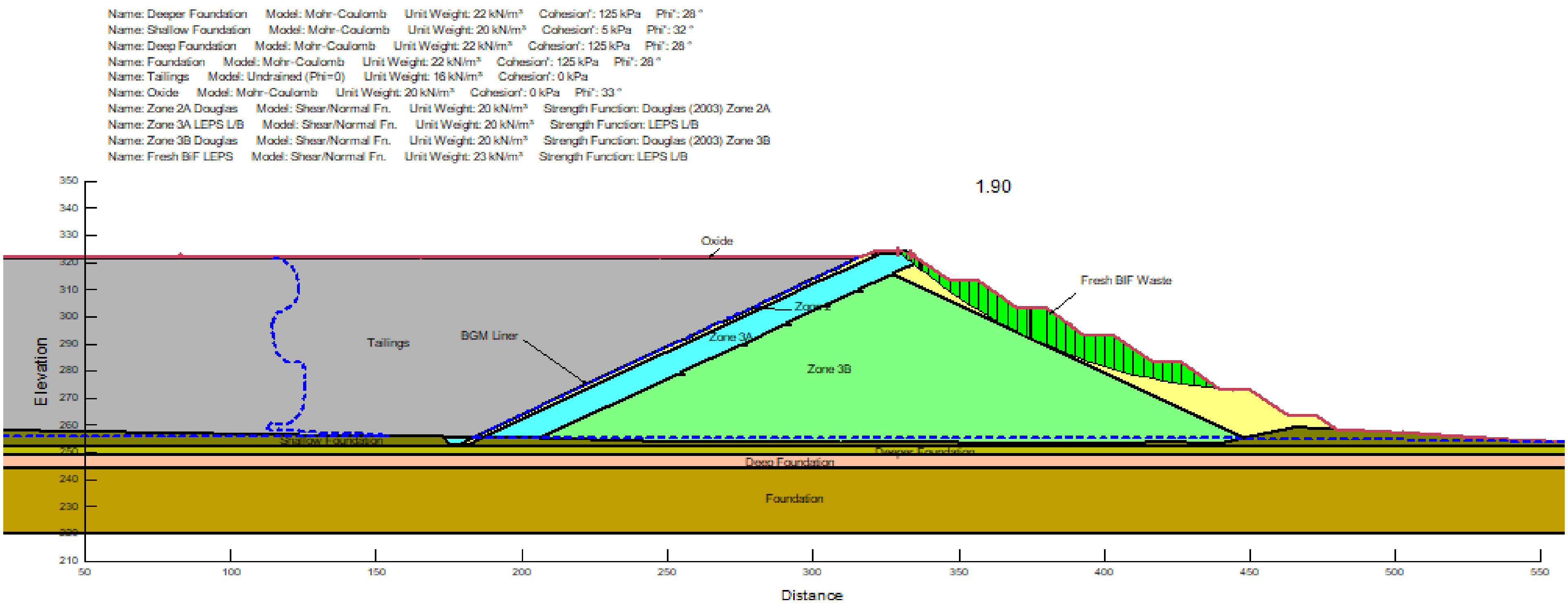
Date: 10-12-19 **Job No:** 114185.15 **FIGURE F3**

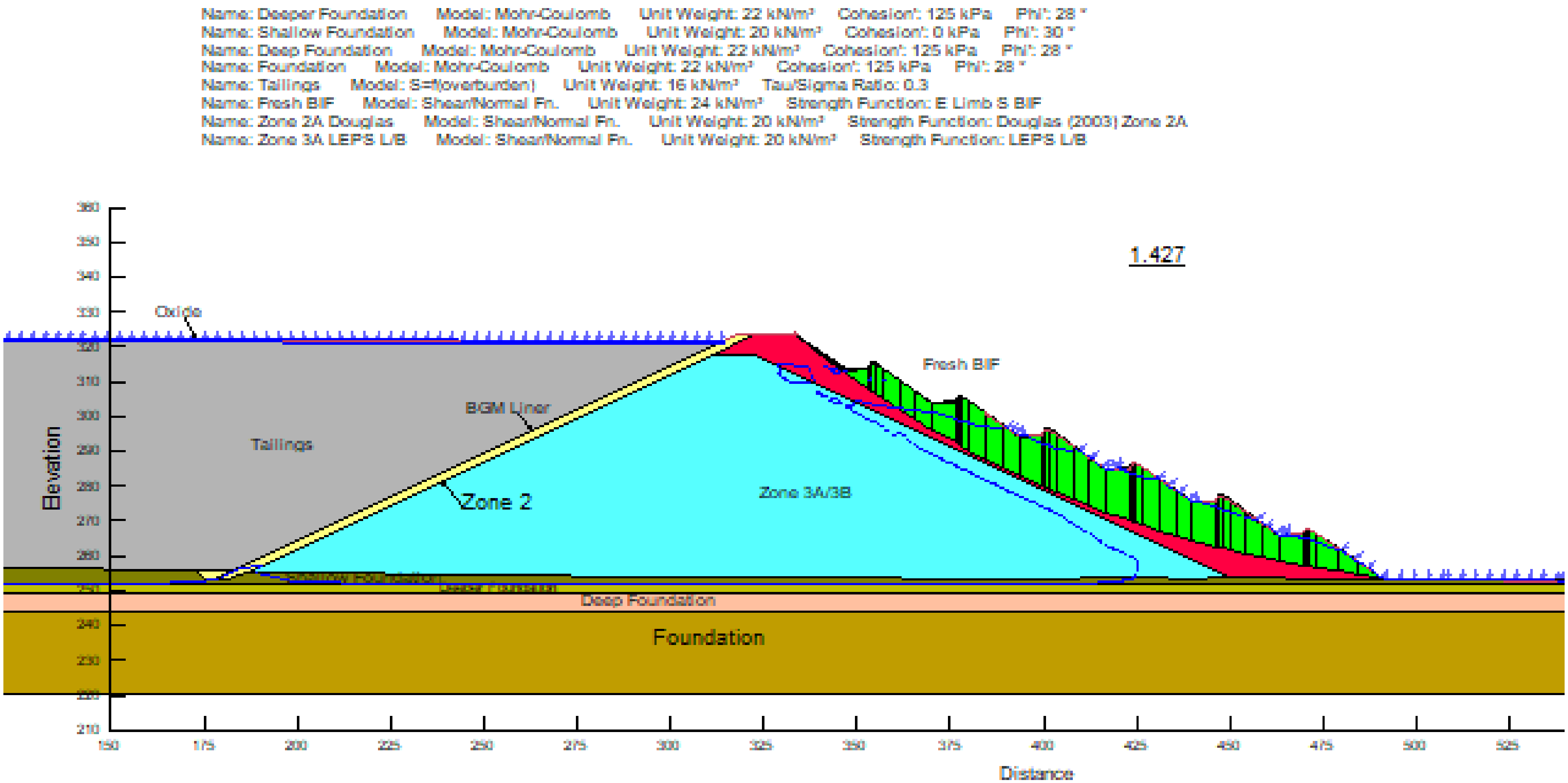
G:\Synergy\Projects\114\114185 North Star Magnetite Stage 2\15 TSF Closure Design\Data and Calcs\SeepW\Stability Results Rev 1.xlsx 1



IRON BRIDGE OPERATIONS PTY LTD				
NORTH STAR MAGNETITE - STAGE 2				
CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY				
Stability Analysis - Closure Face Fresh BIF Leps				
Date:	10-12-19	**Job No:**	114185.15	**FIGURE F4**

ATC Williams www.atcwilliams.com.au

G:\Synergy\Projects\114\114185 North Star Magnetite Stage 2\15 TSF Closure Design\Data and Calcs\SeepW\Stability Results Rev 1.xlsx 2



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NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY

Stability Analysis - Closure Face PMP 72 hr

Date:	13-12-19	**Job No:**	114185.15	**FIGURE F5**

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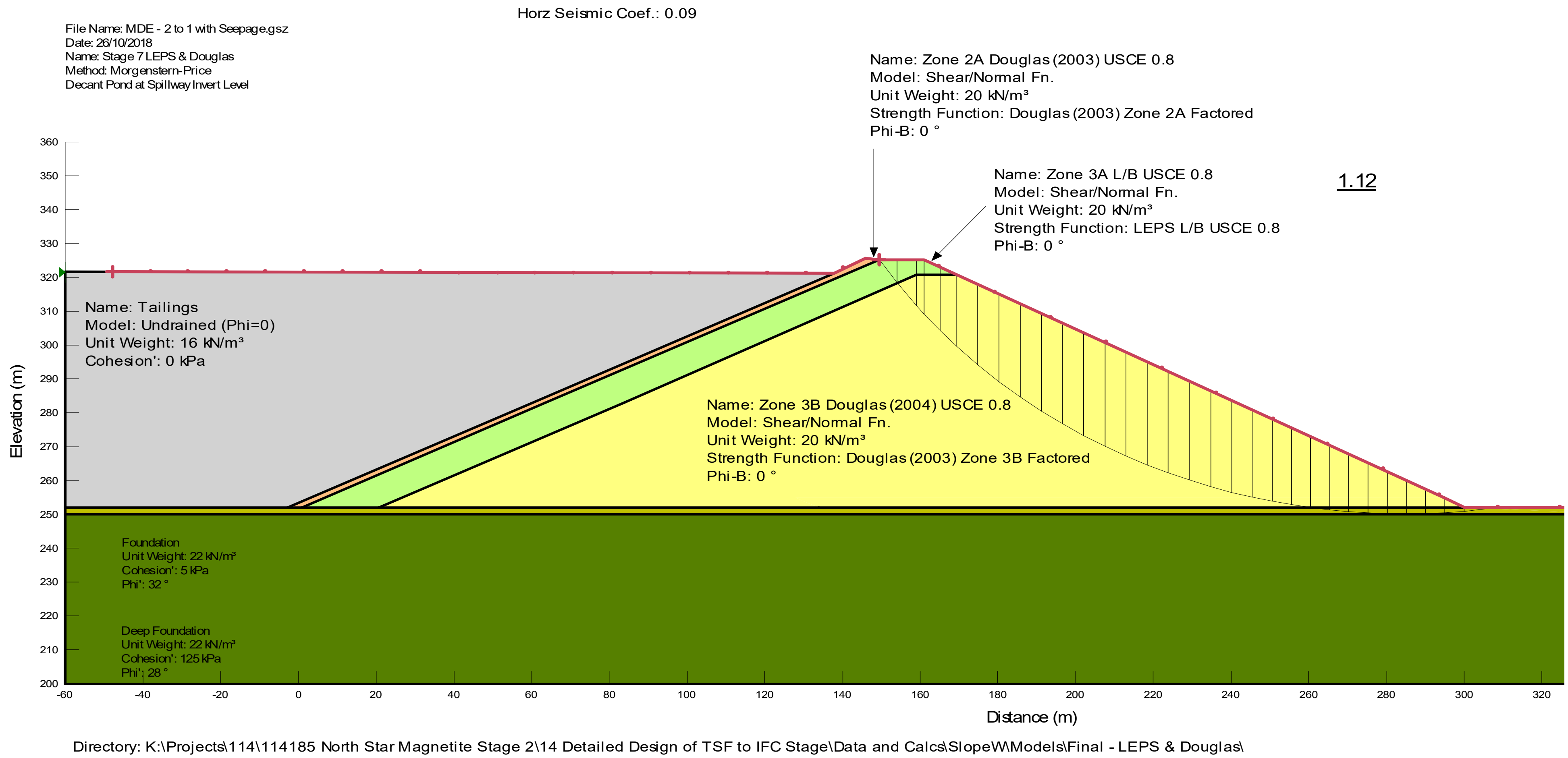

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NORTH STAR MAGNETITE - STAGE 2

DESIGN REPORT FOR TAILINGS STORAGE FACILITY

TSF Stage 7 - (2:1 D/S Slope) - MDE

Date:	14/02/2019	**Job No:**	114185.14	**FIGURE F6**

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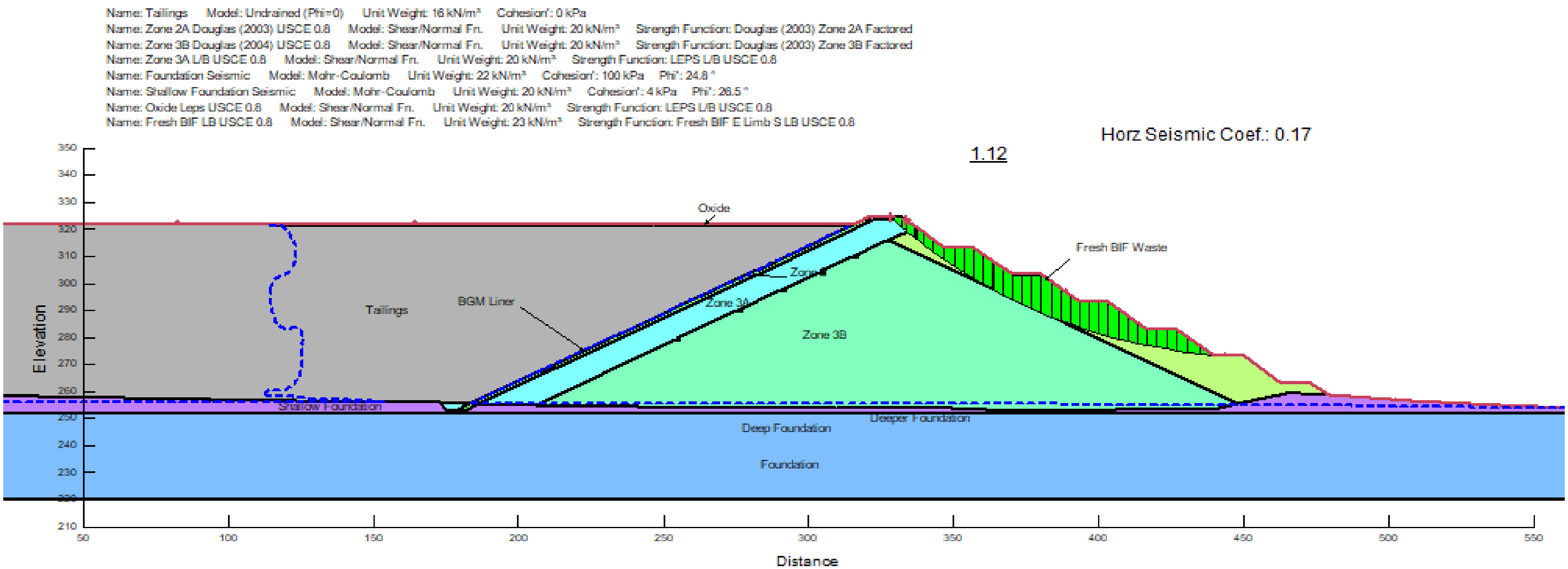

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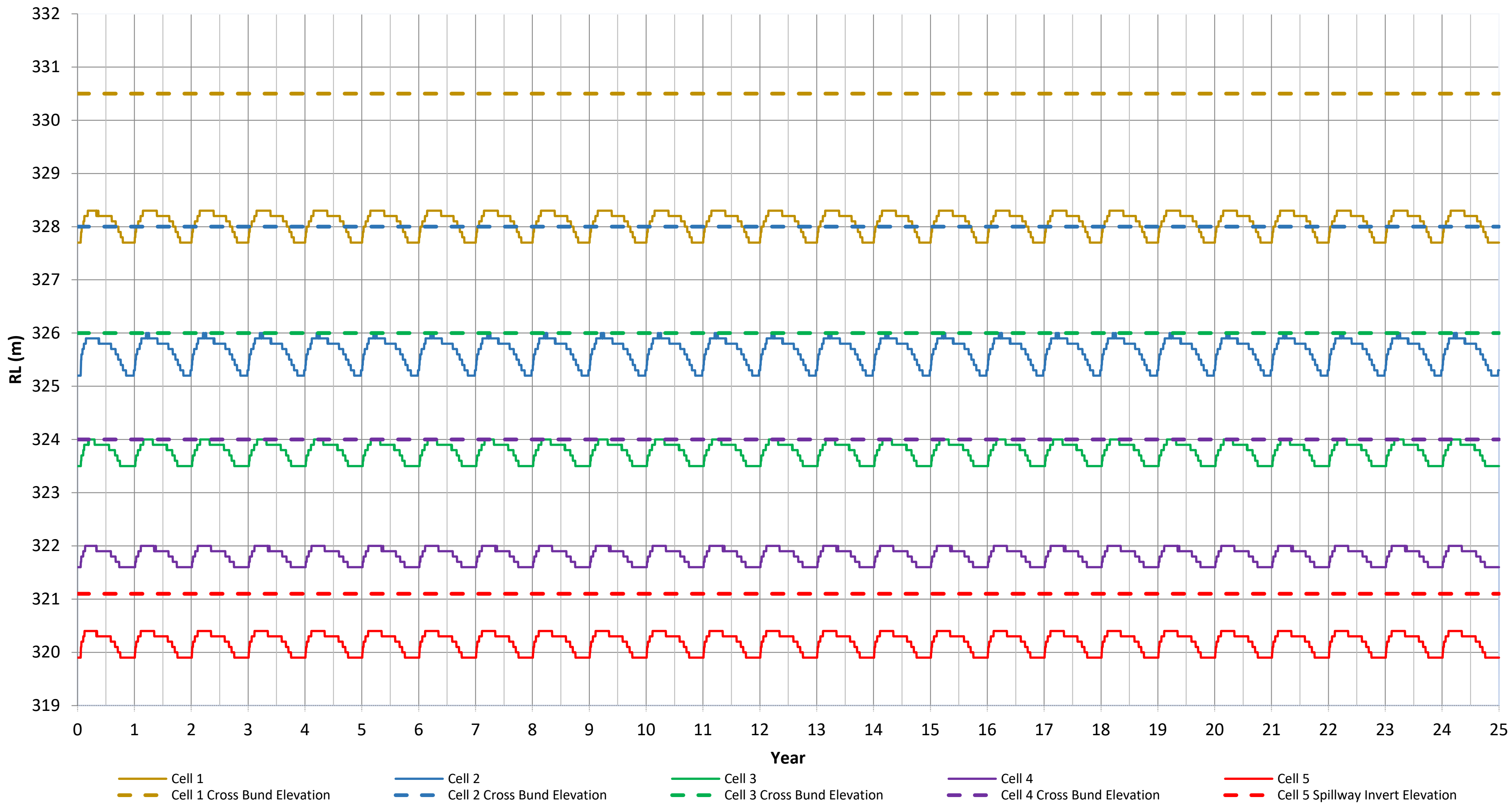
	NORTH STAR MAGNETITE - STAGE 2
	CLOSURE DESIGN FOR TAILINGS STORAGE FACILITY
	Stability Analysis - Closure Face MCE
ATC Williams www.atcwilliams.com.au	**Date:** 13-12-19 **Job No:** 114185.15 **FIGURE F7**

G:\Synergy\Projects\114\114185 North Star Magnetite Stage 2\15 TSF Closure Design\Data and Calcs\SeepW\Stability Results Rev 1.xlsx F7

APPENDIX G



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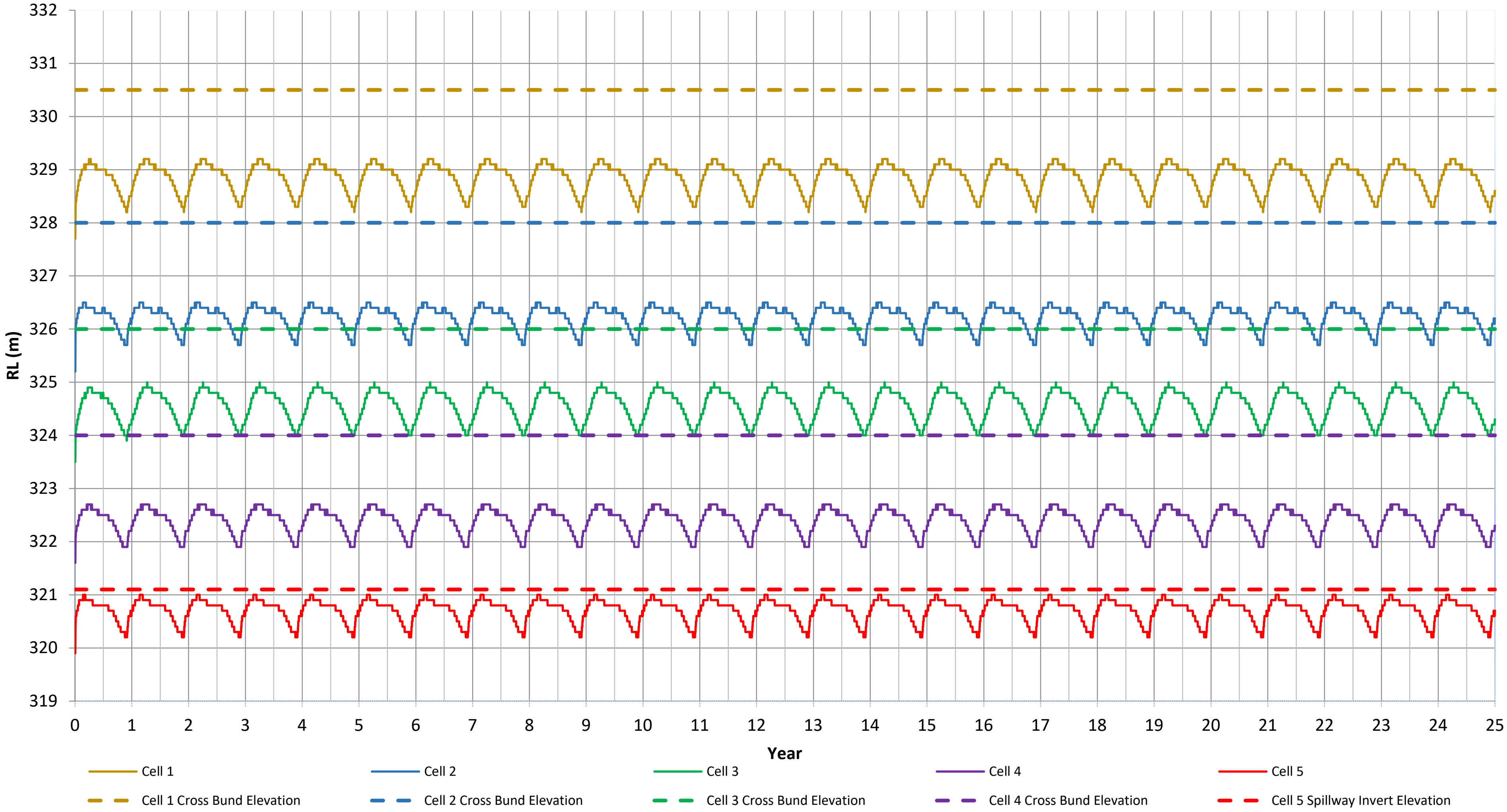
NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN WATER BALANCE

TSF Pond RL - Stochastic Outcome (50 Percentile)

Date:	18-04-19	**Job No:**	114185.15	**Figure G1**

G:\Synergy\Projects\114\114185 North Star Magnetite Stage 2\15 TSF Closure Design\Data and Calcs\Water Balance\Surface bunds case\Figures.xlsx Pond RL Graph 50



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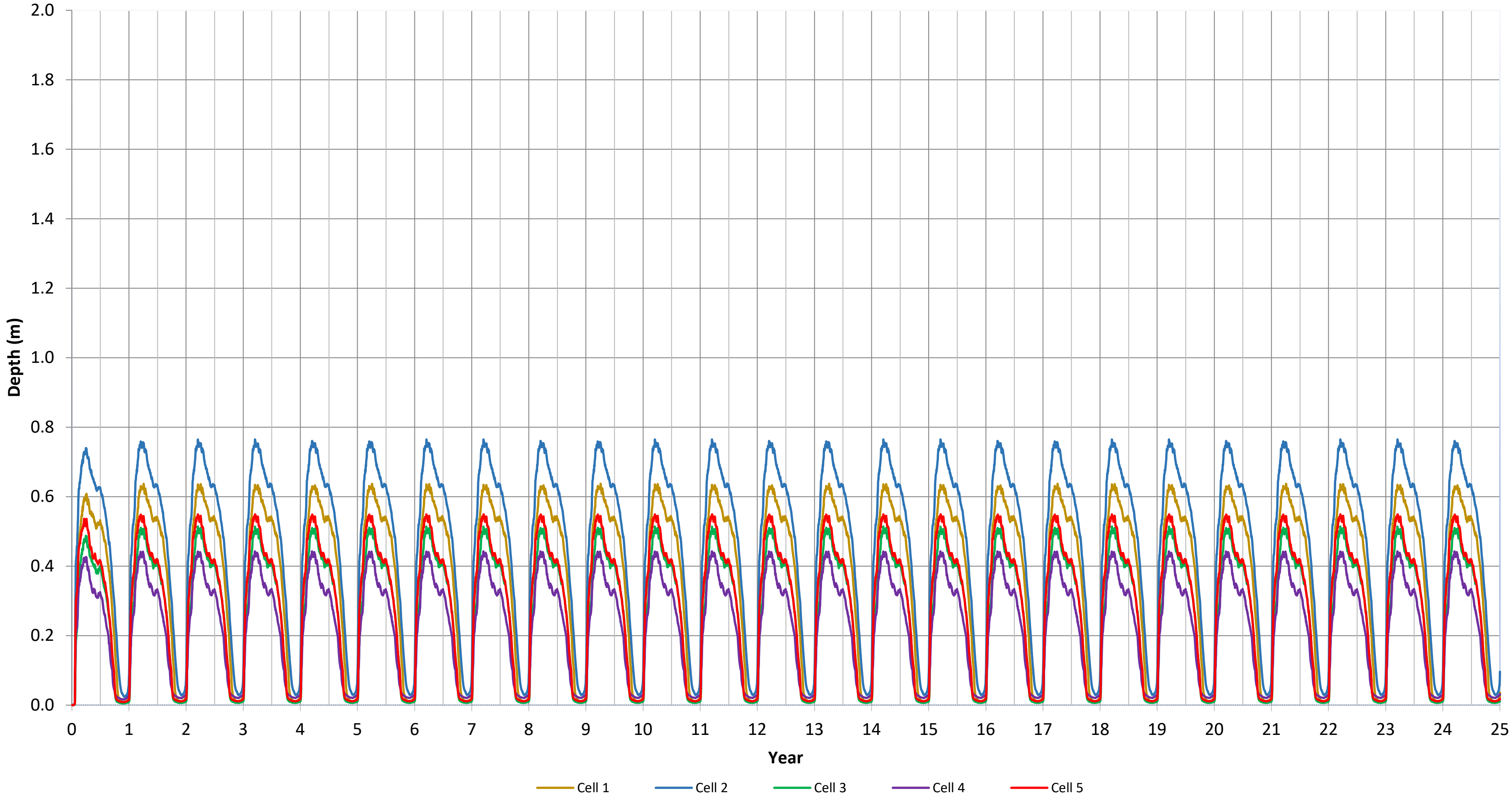
NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN WATER BALANCE

TSF Pond RL - Stochastic Outcome (95 Percentile)

Date:	18-04-19	**Job No:**	114185.15	**Figure G2**

G:\Synergy\Projects\114\114185 North Star Magnetite Stage 2\15 TSF Closure Design\Data and Calcs\Water Balance\Surface bunds case\Figures.xlsx Pond RL Graph 95



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NORTH STAR MAGNETITE - STAGE 2

CLOSURE DESIGN WATER BALANCE

TSF Pond Depth - Stochastic Outcome (50 Percentile)

Date: 18-04-19 **Job No:** 114185.15

Figure G3

G:\Synergy\Projects\114\114185 North Star Magnetite Stage 2\15 TSF Closure Design\Data and Calcs\Water Balance\Surface bunds case\Figures.xlsx Pond Depth Graph 50