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Flotation TSF Failure Impact Assessment

ARAFURA RESOURCES LIMITED

Nolans Project

Flotation TSF
Failure Impact Assessment

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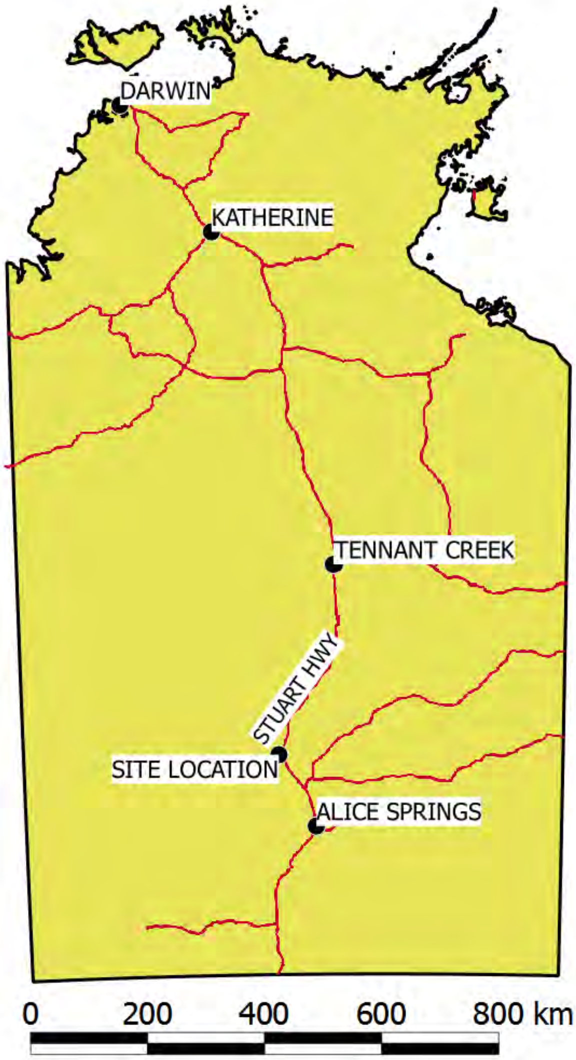
1 INTRODUCTION

Arafura Resources Limited (Arafura) has commissioned ATC Williams Pty Ltd (ATCW) to perform a Failure Impact Assessment (FIA) of the proposed Flotation TSF. The purpose of the FIA is to establish the potential risk to downstream residents from hypothetical dam-break scenarios, and thereby set a Consequence Category for the structure based upon the determined Population At Risk (PAR) and Potential Lives Lost (PLL).

Plate 1
Site Locality



1.1 Scope of Works

Dam safety legislation and guidelines within the Northern Territory have not been codified. As such, dam safety in the Northern Territory is currently self-regulated by dam owners. As a result, this FIA has been performed pursuant to the following industry and Queensland State Government Guidelines:

- Guidelines on the Consequence Categories for Dams (ANCOLD, 2012a)
- Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure (ANCOLD, 2012b)
- Guidelines for Failure Impact Assessment of Water Dams (DEWS, 2012)

The structure of the FIA is as follows:

Section 2	Provides a general description of the proposed Nolans Project, relevant to the Flotation TSF, as well as a summary of relevant regional data utilised in the assessment.
Section 3	Details the adopted approach and methodology for the Failure Impact Assessment.
Section 4	Outlines the input assumptions and outcomes of determining breach geometry
Section 5	Details the hydrological models and output hydrographs representing the dam break floodwave, natural rainfall events and runoff from upstream catchments.
Section 6	Details the development, assumptions and outcomes of a two-dimensional (2D) hydraulic model used to assess the extent of the Failure Impact Zone.
Section 7	Presents an assessment of the Population At Risk (PAR) as a result of the failure scenarios modelled.
Section 8	Presents an assessment of the Potential Lives Lost (PLL) as a result of the failure scenarios modelled.
Section 9	Consequence Category assessment.

2 BACKGROUND INFORMATION

Background information and concept design details for the Flotation TSF have been sourced from the following documentation:

- *Nolans Project – Tailings Storage Facilities Engineering Cost Study (Rev 0)*, Knight Piesold Consulting (February, 2014)
- *Nolans Project – Infrastructure Engineering Cost Study (Rev B)*, Lycopodium Minerals Pty Ltd (February, 2014)

2.1 Project Background

Arafura proposes to develop the Nolans Mine Site with an estimated life of mine of 43 to 45 years. The proposed site will incorporate a Flotation Tailings Storage Facility (FTSF) for the purpose of mine waste (tailings) disposal.

The Nolans Project is located some 140km to the north west of Alice Springs, as shown in . Infrastructure in the local vicinity includes the Stuart Highway 10km to the east, and a natural gas pipeline located some 5km to the south-east. At the closest point, this pipeline is located up-gradient of the site location.

The project is sited over the Nolans Bore Rare Earths Resource, and is expected to feature a mine, concentrator and rare earth intermediate plant. Site dams supporting mining and processing operations will include evaporation ponds, residue ponds and the FTSF, the latter of which is the subject of this assessment.

2.2 Flotation TSF

The FTSF will provide storage capacity for tailings disposal, and is to be sited adjacent to the concentrator. The FTSF will comprise a multi-cell configuration, with each cell providing deposition volume for a period of approximately 15 years. The life of mine is expected to be 45 years.

Proposed Dam Geometry, as provided by Arafura, was adopted as follows:

- Total Embankment Height 25m (per cell)
- Footprint area 33ha (per cell)
- Number of Cells 3

Embankment details were based upon the *Nolans Project – Tailings Storage Facilities Engineering Cost Study Rev 0 (*Knight Piesold, 2014).

Summary details of the geometry adopted for the FTSF are listed in **Table 1**:

Table 1 -
Proposed FTSF Details

Name of Dam		FTSF
Owner of Dam		Arafura Resources Limited
Status of Dam		Proposed (Concept Stage)
Purpose		Disposal of Tailings Waste
Hazard Category		TBD
Reference used for Consequence Category		ANCOLD (2012)
Date Dam Construction completed to current arrangement		N/A
Locality Description		Central Desert Regional Council, NT
Full Supply Level (FSL)		TBD
Dam Crest Level (DCL)		25.0m above embankment base
Total Maximum Storage Capacity		32.5 Mt (25×10^6 m^3)
Total Quantity of Tailings Deposited		Nil (Proposed)
Total Volume consumed by Tailings		Nil (Proposed)
Emergency Spillway Dimensions and Characteristics		No Spillway
Embankment Configuration	Type	Zoned Heterogeneous
	Total Height	25.0m
	Crest Length	2,400m (per cell)
	Crest Width	8.0m
Slope	Downstream	3:1 (H:V)
	Upstream	2.5:1 (H:V)
Slope Protection	Downstream	Compacted Rock Fill & Benched
	Upstream	Lined or Soil Liner

2.2.1 Embankment Construction Methods

Each cell will be raised using downstream construction methods, with five construction stages proposed per cell. Nominal embankment raise heights are as listed in **Table 2:**

Table 2 -
Proposed FTSF Raises

Stage	Nominal Raise Height	Raise Method
Starter	5.0m	N/A
Intermediate Stage 1	5.0m	Downstream
Intermediate Stage 2	5.0m	Downstream
Intermediate Stage 3	5.0m	Downstream
Ultimate	5.0m	Downstream

The design of the FTSF embankment incorporates a zoned earthen embankment. The embankment fill material predominantly comprised of Run-of-Mine waste material, with a 6.0m thick low permeability soil liner running along the upstream face of the dam. It is understood that testing of the FTSF impoundment footprint will determine the need to line the base of the storage.

2.2.2 Tailings & Process Characteristics

The tailings properties for the FTSF are expected to be silty sand with clay (Knight Piesold, 2014). Tailings tests indicate the slurry takes 6-8 days to achieve initial settlement, before consolidating to an air dry density of 1.37 t/ m^3. Arafura expect production rates and in-situ tailings properties to be as follows:

- Ore Throughput — 0.9 Mtpa
- Concentrate — 0.5 Mtpa
- Tailings Production Rate — 0.4 Mtpa (dry tonnes)
- Solids Content — 38.6% (wt/wt)
- Specific Gravity Tailings — 2.73 t/m^3
- Deposition Area (1 cell) — 33 ha
- Expected Field Dry Density — 1.3 t/m^3

The total tailings deposition volume available in the FTSF, adopted as per Arafura's request (Refer **Table 1)**, provides additional deposition volume than required for a production rate of tailings averaging 0.4 Mtpa over 45 years (i.e. 18.0 Mt of tailings). As per Arafura's request, this is to accommodate potential additional tailings deposition from mining as yet undefined mineral resources deeper than 215 m below surface at Nolans (subject to viability).

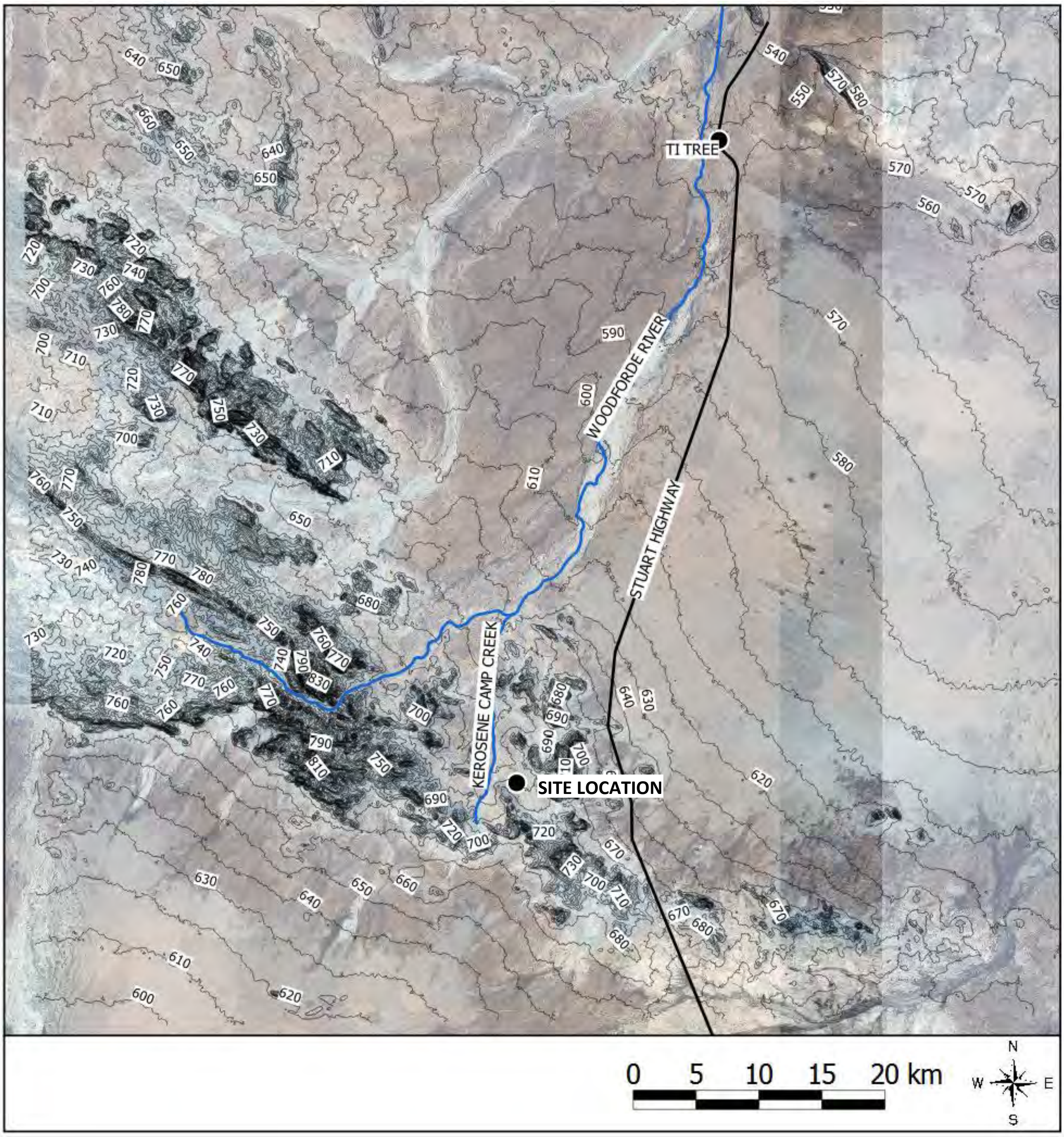
The proposed design includes decant water recycling measures and underdrainage. The rate of rise of tailings is expected to be in the range of 1.0 – 2.2 m/year. Based on the above, it is expected that the settled dry density of 1.3 t/m^3 is achievable.

2.3 Topography & Drainage

The proposed mine and concentrator area of the Nolans project is located on basement rocks of the Arunta Region in the catchment of the Wiso Basin in the Northern Territory. It is located in the headwaters of the Woodforde River at an elevation of some 700m. The Kerosene Camp Creek, to the west of the proposed site location, flows north to the Woodforde River. Drainages consist of low gradient, poorly defined ephemeral creeks, featuring sand beds which tend to widen as waters flow northwards.

A significant additional catchment (to the west) flows into the Woodforde some 10km north of the site, as is depicted in **Plate 2**:

Plate 2
Regional Topography and Drainage

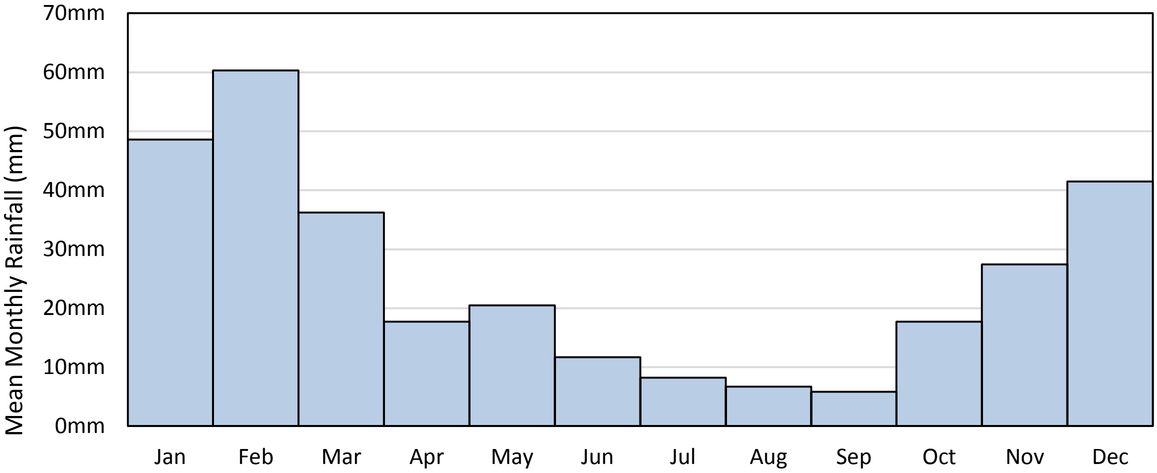


2.4 Climate & Rainfall Data

The site is located in an arid and dry region of the Northern Territory. Rainfall data was sourced from the Bureau of Meteorology's (BOM's) Aileron NT Station, located some 11km from the site location. On average, annual rainfall depths are only 300mm (arid), and the maximum recorded annual rainfall depth is 1,011mm.

Arid interior regions, such as the site location, experience lower intensity rainfall events than coastal regions. Furthermore, low gradient catchments which experience little rainfall tend to exhibit lower runoff rates, as a result of greater storage and retention. Average monthly rainfall rates are shown in **Plate 3:**

Plate 3
Monthly Mean Rainfall



Data sourced from Aileron NT Station (015543), BOM 2016

A stream gauge is located within the Woodforde Creek, some 25km north of the Nolans Project. The location of the gauge is depicted on **Plate 4**, with stream discharge data plotted on **Plate 5.**

Plate 4
Location of Stream Gauge

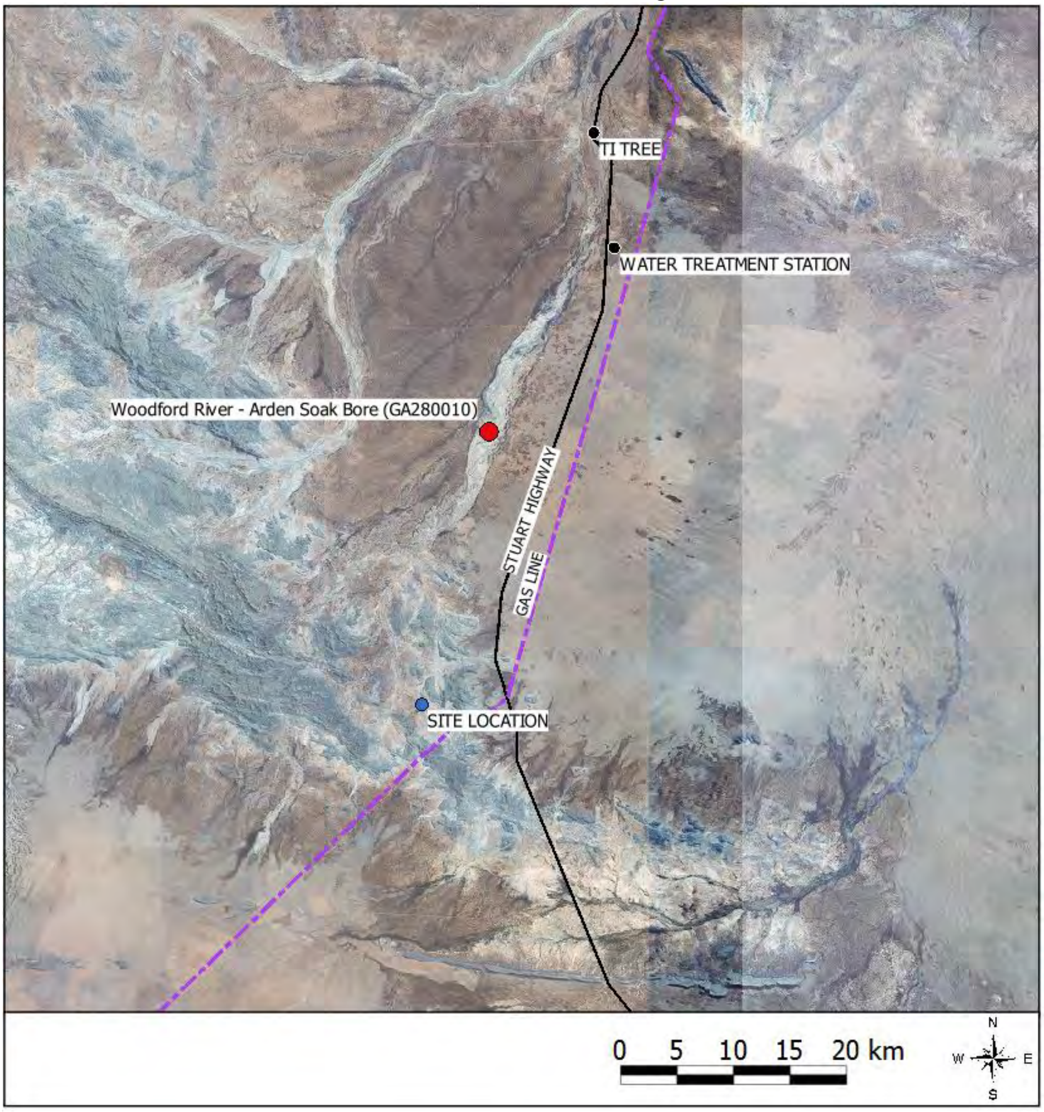
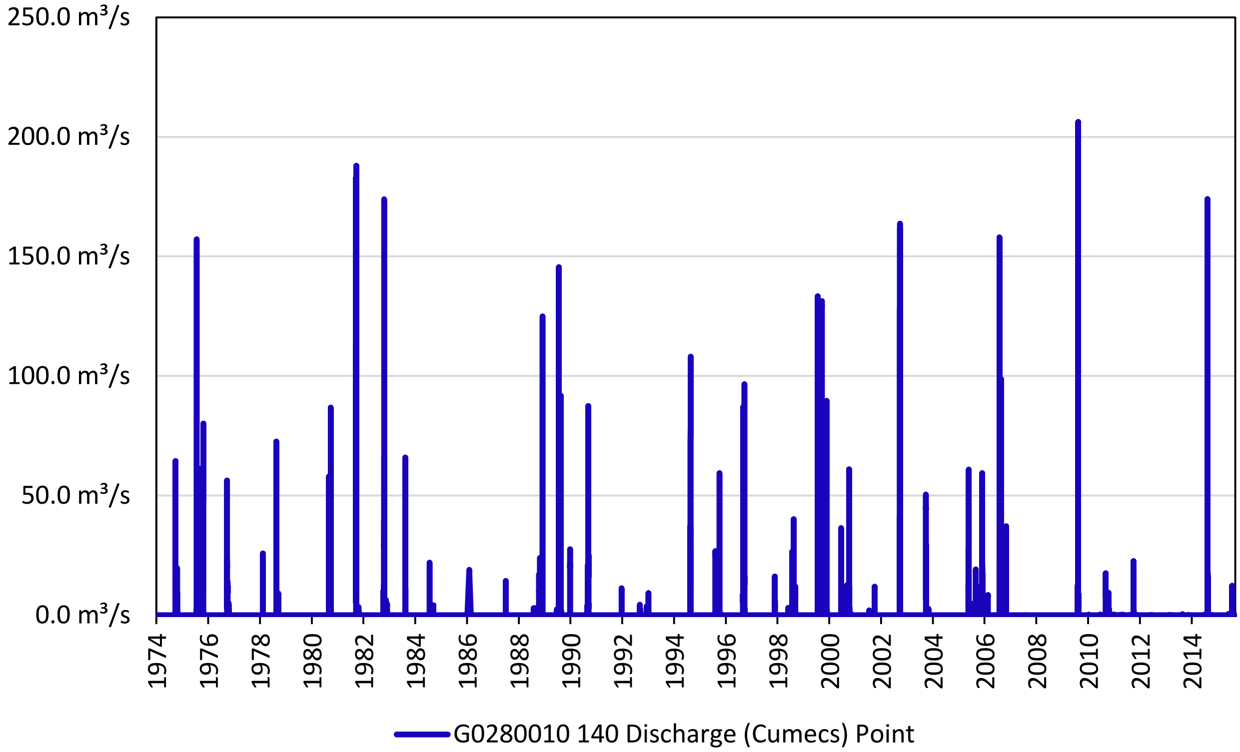
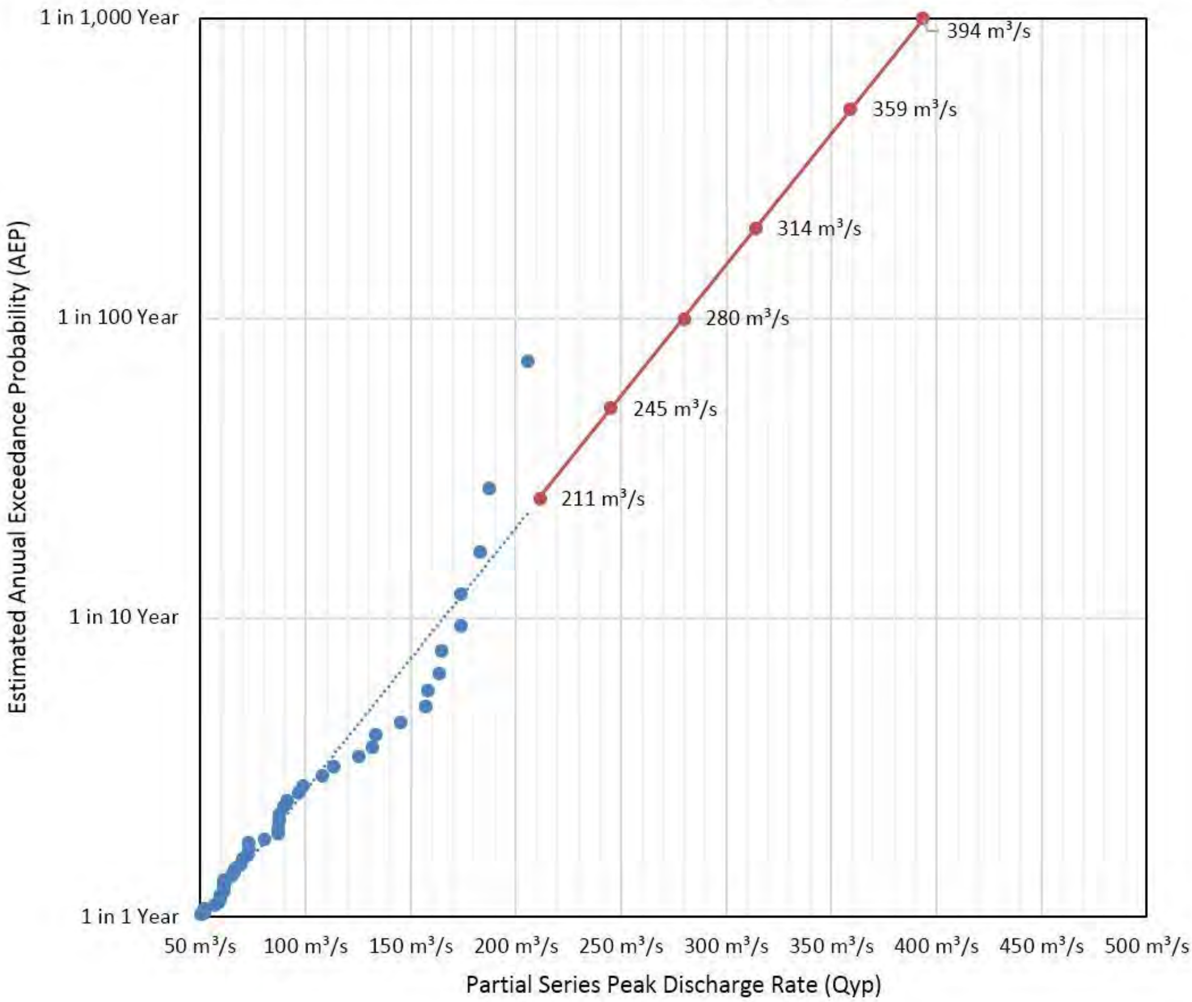


Plate 5
Stream Gauge Records



The data record available at the gauge is incomplete, with missing records and years. Consequently, it was not possible to perform a detailed analysis on the available discharge records. A partial series Flood Frequency Analysis was performed to estimate the *magnitude* of natural flow within the Woodforde Creek. The outcome of the analysis is plotted on **Plate 6**.

Plate 6
Partial Series Flood Frequency Analysis



The partial series analysis indicates that the expected magnitude of discharge from a 1 in 100 Year natural flooding event (at the gauge location) is on the order of 10^2 - 10^3 m^3/s.

2.5 Survey

Site survey and LiDAR was unavailable for the purposes of this assessment. Topographic data was sourced from the 1 Second SRTM program, which provides coarse resolution elevation data. The format of this data is 30m by 30m grid elevations, and was sourced from Geoscience Australia (2011).

A known issue with the 1 second SRTM program is the existence of noise error within the dataset. Resampling to a 10m grid resolution was performed, using a smoothing function to filter noise in the Woodforde Creek area.

2.6 Catchment Characteristics

Plate 7, **Plate 8** and **Plate 9** provide a photographic record of the catchment characteristics observed by Henning Boshoff during a site visit on 18 January 2016.

The Woodforde Creek catchment was fairly dry with minimal ground cover and with sparse shrubs and trees. Trees were primary along the edge of the dry creeks with very few trees found within the creeks. The creeks formed within large alluvial flood plains and geomorphic change is very likely.

Soils appear to be very sandy with some erosion found along the creek beds and small drainage features feeding into the creeks. Most of the catchment appear to be used for cattle grazing with a number of water holes observed.

Plate 7
Typical Surface Cover

It is expected sheet flow would drain in the general direction of the creeks and the flow occasionally hindered by windrows from road maintenance.

The overall terrain is fairly flat with some outcrops found along the catchment divide near the site location. The drainage features near these elevated areas show more signs of erosion and increased flow velocities, and would drain quicker into the creek.

Plate 8
Typical Drainage Path

Debris was observed in a number of locations, which were primarily debris caught in trees. It is unclear when the debris was caught and if all the observed debris were from the same event. However, it can be observed that flood waters generally do not rise much higher than the creek embankments and that large flood events would spread over the flood plains.

Plate 9
Debris Flood Height

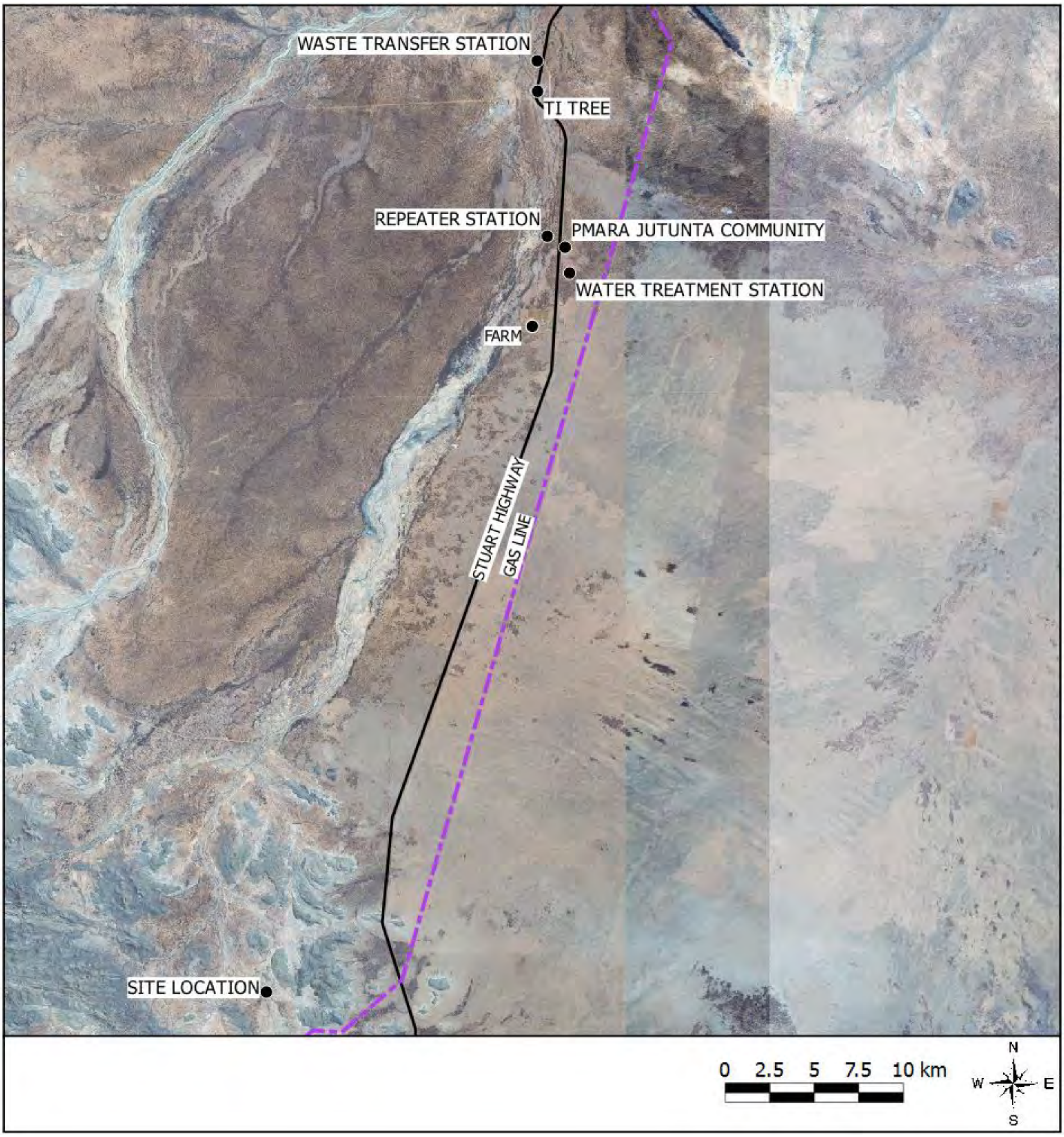
2.7 Occupancy & Infrastructure

Downstream occupancy was assessed, by examining the following data sources:

- Aerial imagery; and
- Northern Territory Residential Leases;

The main infrastructure of concern in the immediate downstream area is the Stuart Highway. Further downstream, several downstream residential areas were identified, and are depicted in **Plate 10:**

Plate 10
Downstream Occupancy



3 DAMBREAK METHODOLOGY

For this FIA, the *Failure Mode* and *Failure Scenario* are considered separately, as detailed in **Table 3**:

Table 3 -
FIA Definitions

Failure Mode	The initiation event causing the failure, such as Piping, Sliding, Overtopping, Overturning, Vandalism.
Failure Scenario	Sunny Day Failure or Flood Failure.
Sunny Day Failure (SDF)	Unexpected sudden failure, with little to no warning time provided to downstream occupants.
Flood Failure (FF)	Failure during a rare to extreme weather event. Warning of failure likely, with evacuation of flood prone areas expected.

3.1 Failure Scenarios

Two different failure scenarios have been considered for a breach of the FTSF. These events are characterised in **Table 4**:

Table 4 -
Failure Scenario

Modelled Failure Event	Failure Mode and Adopted Model Conditions
Sunny Day Failure (SDF)	• No Warning to Downstream Occupancy. • Piping Failure or Embankment Stability Failure initiated by any range of events. • Storage water level assumed to be elevated at time of event. • Downstream drainage environments assumed to be flowing at normal levels (as downstream creeks are ephemeral, creeks are assumed to be dry)
Flood Failure (FF)	• Warning to Downstream Occupancy. • As the FTSF is designed to accommodate the PMP without spillage, overtopping does not occur. • The failure is initiated by an extreme rainfall event over the FTSF extents. • Downstream drainage environments assumed to be flowing at levels consistent with a 1 in 100 year event (0.01 AEP).

3.2 Adopted Modelling Techniques

The adopted modelling techniques are summarised in **Table 5**. Discussion of the modelling and the results are summarised in **Sections 4**, **5** and **6**.

Table 5 -
Adopted Modelling Techniques

Model	Scenario	
	Sunny Day Failure	**Flood Failure**
Breach Geometry Model	Froehlich (2008)	Froehlich (2008)
Breach Hydrograph Model	DNRW (2008)	DNRW (2008)
Hydraulic Model	TUFLOW (BMT WBM, 2013)	TUFLOW (BMT WBM, 2013)
Upstream Hydrological Model (Concurrent Rainfall)	Nil	RORB (Monash University et al, 2012)
Direct Rainfall (Within Model Domain)	Nil	Rain-on-Grid

4 BREACH GEOMETRY

The breach model determines the geometry of the breach, based on Froehlich (2008). The breach model consists of estimator equations based on historical dam failure case studies. The model determines breach width and breach development time, and specifies breach geometry side slopes based upon the failure mode (piping or overtopping). The Froehlich (2008) equations are:

$$B_{AVE} = 0.27\ K_0 V_W^{0.32} h_b^{0.04}$$

$$t_f = 63.2 \sqrt{\frac{V_W}{g h_b^2}}$$

Where

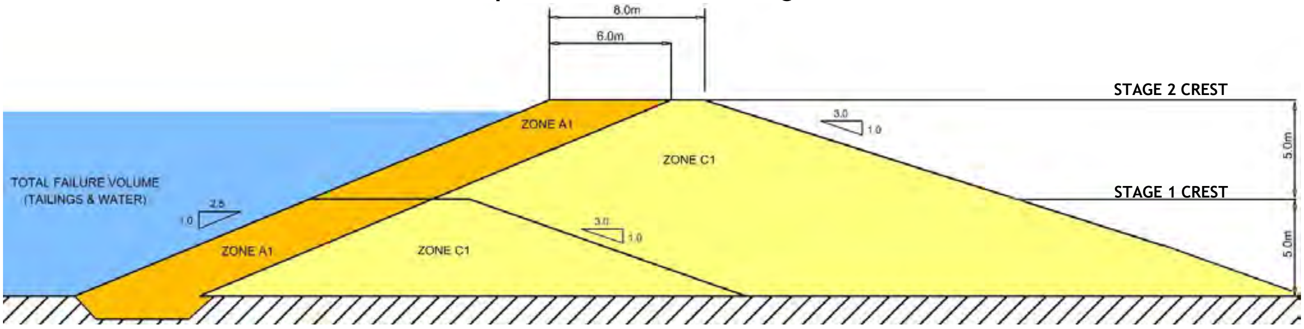
B_{AVE}	*Average Breach Width*	*m*
K_0	*Failure Mode Coefficient* *(K_{Piping} = 1.0, $K_{Overtopping}$ = 1.3)*	-
V_W	*Volume of Water Above Breach Level*	*m^3*
h_b	*Depth of Breach Measured From Embankment Crest Level*	*m*
t_f	*Breach Progression Time*	*hours*

The Froehlich equations require the total depth and volume of the failure as an input. As tailings dam failures are unlikely to erode into drained, consolidated and structurally competent tailings beaches, the failure scenarios assessed for the FTSF modelled the following assumed failure characteristics:

- Concurrent failure of two embankment raises; and
- FTSF Failure Area equal to 1.0 × Cell Area.

The assumed pre-failure configuration is depicted in **Plate 11:**

Plate 11
Conceptual Pre-Failure Configuration



Zone A1 – Low Permeability Soil Liner

Zone C1 – Run Of Mine (ROM) Waste Fill

Based on the assumed failure configuration, breach parameter outcomes are listed in **Table 6:**

Table 6 -
Breach Parameter Outcomes

Parameter	Breach Parameter Outcomes	
	Sunny Day Failure	Flood Failure
Assumed Depth of Failure h_b	7.5m	10.0m
Average Breach Width B_{AVE}	31.1m	34.6m
Breach Side Slope Z_{Breach}	0.7H : 1.0V	0.7H : 1.0V
Breach Development Time t_f	1.1 Hours	0.95 Hours

A comparison of the Froehlich (2008) Breach Parameter outcomes against historical case studies (Refer **Plate 12**, **Plate 13**) suggests that:

- The determined Breach Width (BAVE) is representative of the overall dataset; and
- The determined Breach Development Time (tf) is under-representative of the average progression times within the data.

As described in **Section 2.2.1**, breach characteristics were considered for all stages of the FTSF development, up to an ultimate embankment design height of 25.0m. Due to the expected downstream raise methods of embankment lifts, the critical case was determined to be a failure of the Ultimate and Intermediate Stage 3 lifts. This conservative breach depth of 10m resulted in full breach formation within 0.95 hours and a breach width of 34.6m for the Flood Failure scenario. The SDF was assessed as being smaller, and slower progressing than the FF scenario.

Plate 12
Breach Parameters: Width vs. Height

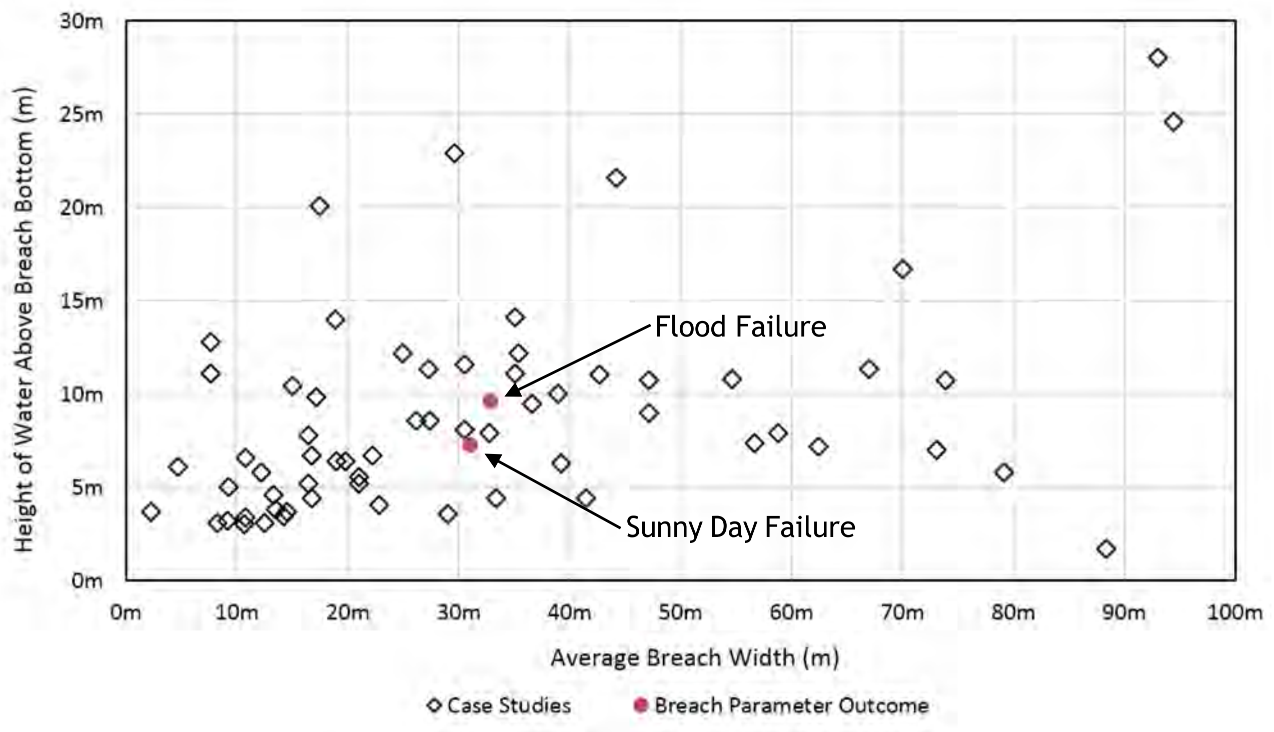
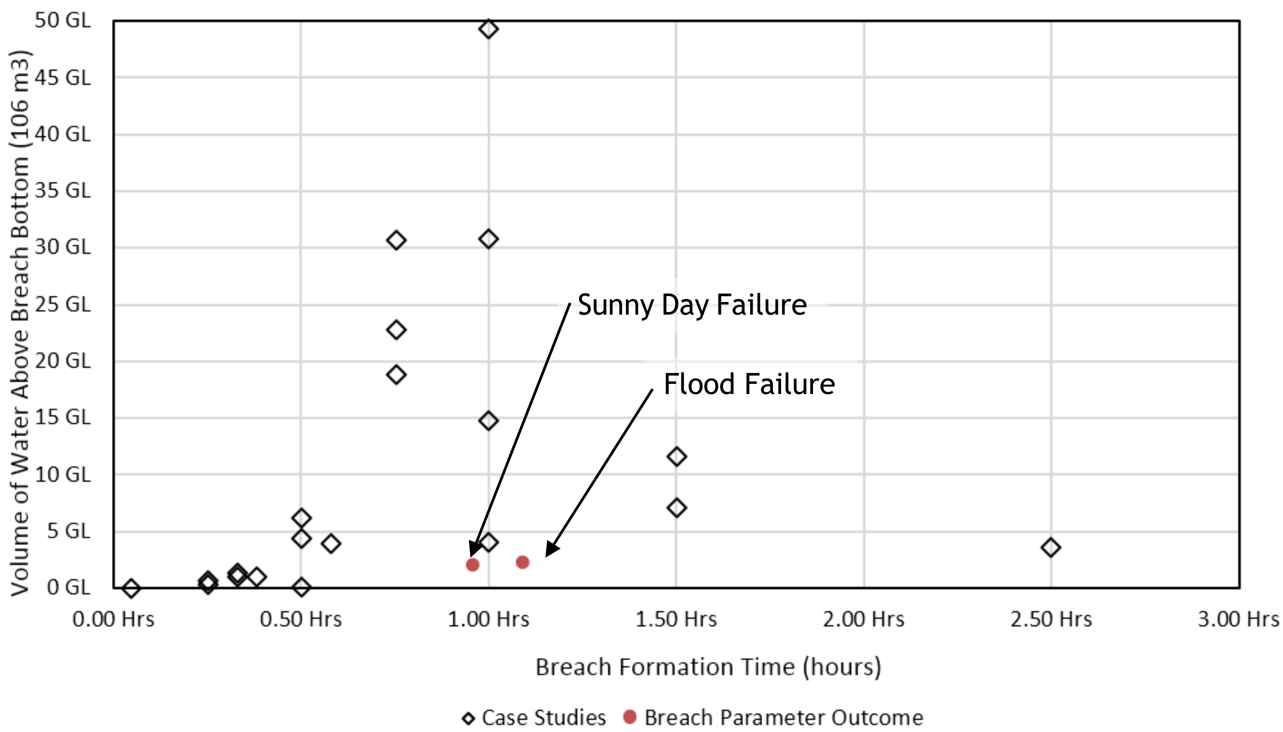


Plate 13
Breach Parameters: Volume vs. Breach Development Time



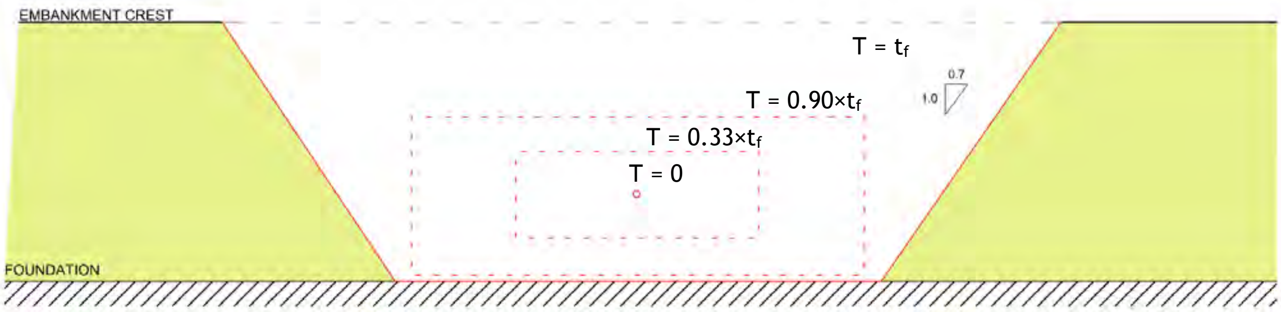
Consequently, the breach development time was reduced to 0.75 hours for the Sunny Day Failure case. This modification will result in a higher peak discharge and shorter duration dam-break hydrograph, consistent with a sudden failure occurring with little to no warning to downstream occupants.

The duration of the failure event was not changed in the Flood Failure case.

4.1.1 Sunny Day Failure (SDF)

The SDF breach geometry features an initial pipe forming within the embankment at the start of the failure (T=0). The dimensions of the pipe increase linearly over the Breach Formation Time (t_f) until the crest is intersected and the pipe failure becomes a full embankment failure.

Plate 14
SDF Breach Geometry

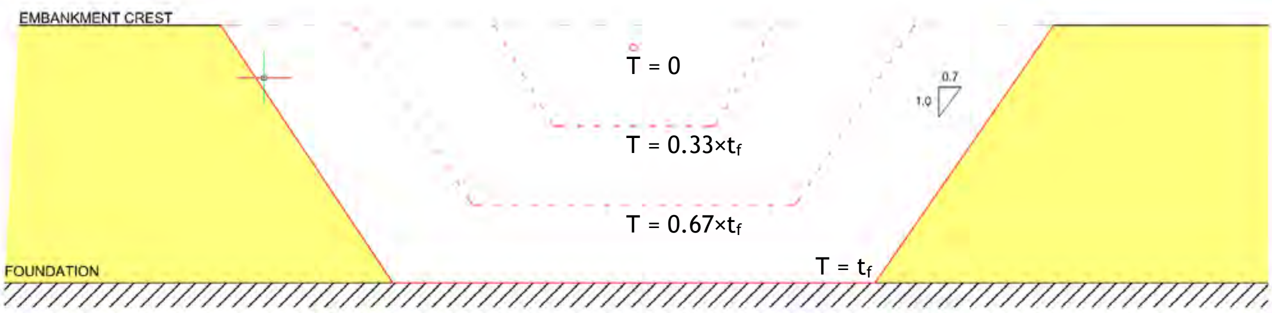


T = Time, t_f = Breach Development Time

4.1.2 Flood Failure (FF)

The FF breach geometry features an initial loss of embankment material near the crest of the embankment. Subsequent failure occurs as the water surface channels down into the embankment. The dimensions of the breach geometry increase linearly over the Breach Formation Time (t_f), and possess a trapezoidal shape.

Plate 15
FF Breach Geometry



T = Time, t_f = Breach Development Time

5 HYDROLOGICAL MODELS

The following models calculate input hydrographs and hyetographs for the TUFLOW 2D Hydraulic Model.

5.1 Rainfall

5.1.1 Rare Estimates (1 in 100 Year).

Rare estimates were sourced from the Bureau of Meteorology's Intensity-Frequency-Duration Program (BOM, 2013) for the site location, and are listed in **Table 7**:

Table 7 -
Rare Rainfall Estimates

Duration	Average Rainfall Intensity (1 in 100 Year)
10 mins	201.6 mm/hr
20 mins	159.7 mm/hr
30 mins	117.8 mm/hr
1 Hours	77.8 mm/hr
3 Hours	36.1 mm/hr
6 Hours	21.4 mm/hr
24 Hours	7.8 mm/hr
48 Hours	5.0 mm/hr

5.1.2 Probable Maximum Precipitation

The depth of the Probable Maximum Precipitation (PMP) was adopted as 1,009 mm for a 3-day event (Knight Piesold, 2014). As the dam is designed to accommodate the PMP event without spilling, it is assumed that the total failure volume (as depicted in **Plate 11)** includes the full depth of the 3-day PMP event.

5.1.3 Temporal Patterns

The adopted temporal pattern for the upstream hydrological model and rainfall-on-grid model was the GSDM pattern (BOM, 2003).

5.2 Breach Hydrograph Routing

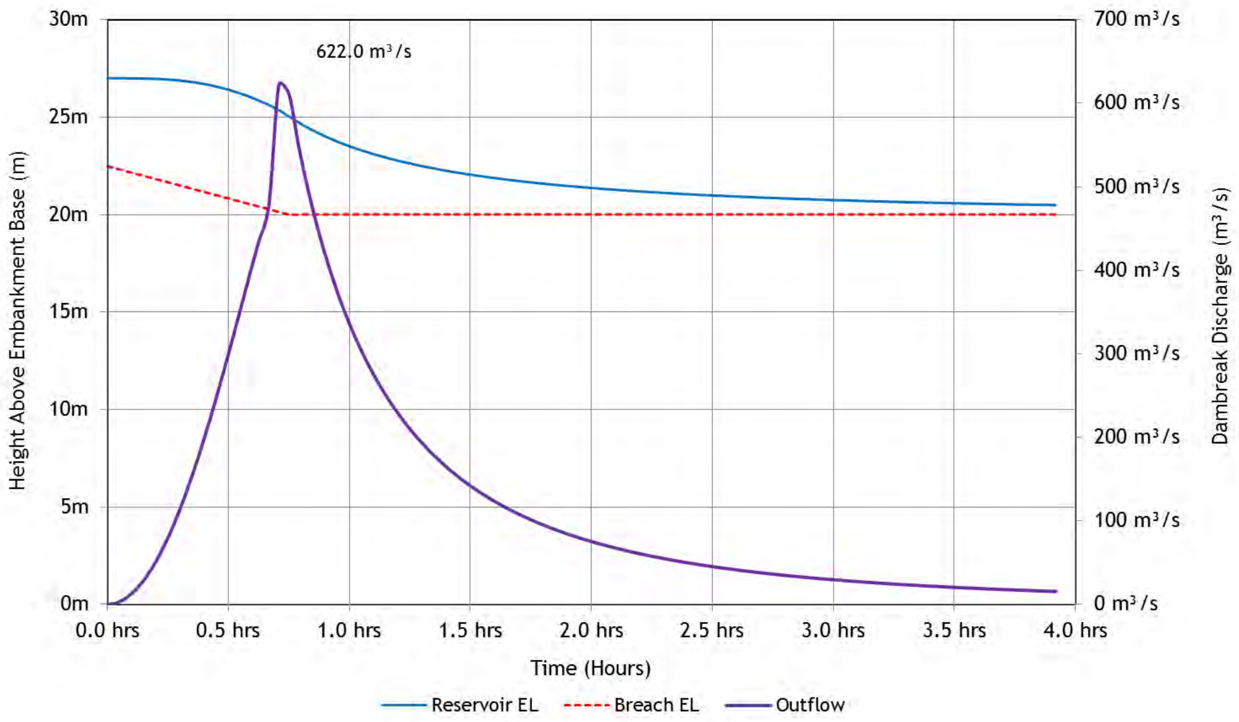
Breach hydrographs were determined by routing the assumed failure volume through the breach geometry determined in **Section 4.** Routing is performed in a spreadsheet specifically designed for modelling breach hydrographs (DNRW, 2008).

The spreadsheet initially models a piping failure, with discharge through the pipe calculated as pressure flow through an orifice. As the pipe enlarges, the water surface is drawn down below the top of the pipe. At this point full failure is assumed and discharge through the failure is calculated as weir flow.

5.2.1 Sunny Day Failure (SDF)

The SDF hydrograph reaches a peak discharge rate of some 622 m^3/s. The breach progression time is short, and peak discharge occurs 45 minutes after initial failure. These outcomes are depicted in **Plate 16**:

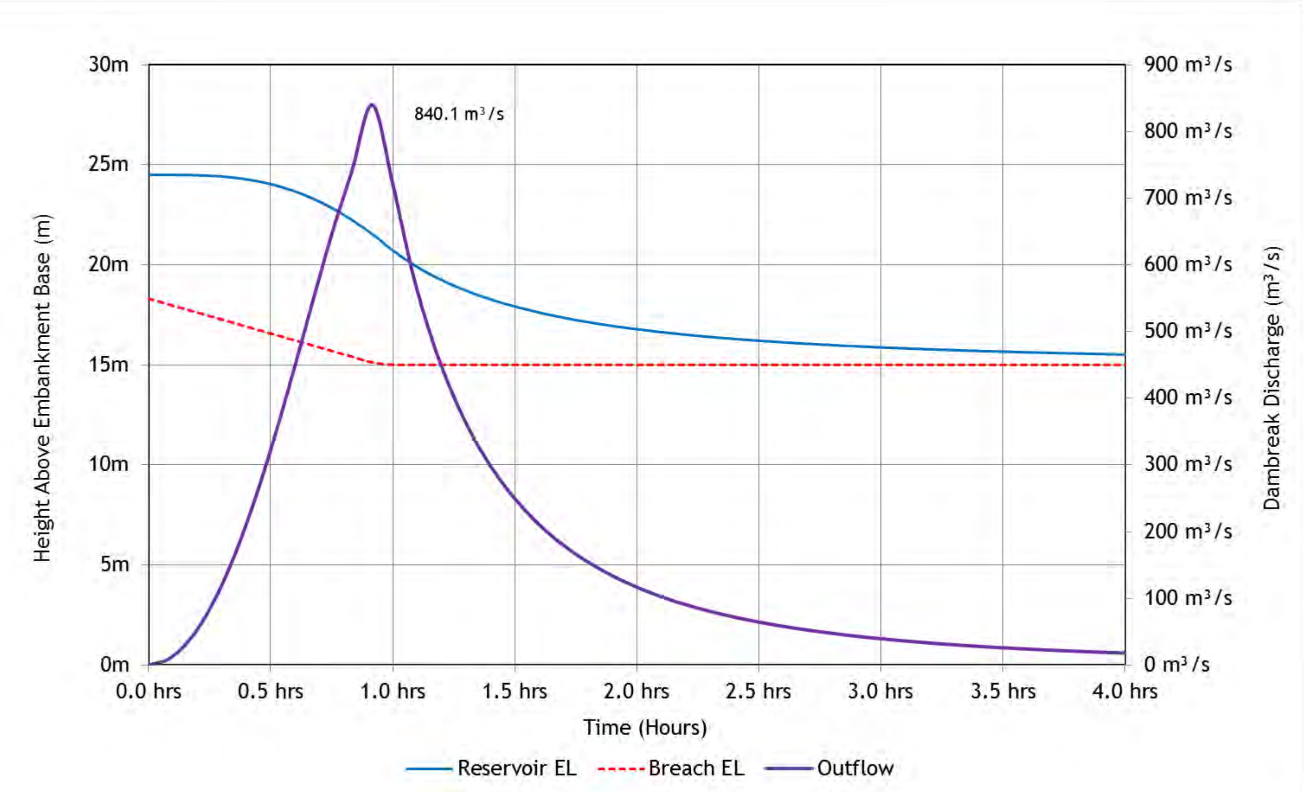
Plate 16
SDF Hydrograph



5.2.2 Flood Failure (FF)

The FF hydrograph reaches a peak discharge rate of 840.1 m^3/s. The breach progression time is longer, with peak discharge occurring at 0.95 hours after the start of the dam-break event. These outcomes are depicted in **Plate 17**:

Plate 17
FF Hydrograph



5.3 Upstream Hydrological Inputs

As part of the failure impact assessment FF scenario, hydrological modelling has been undertaken to characterise flood hydrographs within upstream hydrological catchments. As suggested by ANCOLD (2012a), the concurrent rainfall event for the FF Scenario is the 0.01 AEP rainfall event.

Hydrological modelling has comprised the following:

- RORB Hydrological Model for defined drainage pathways; and
- TUFLOW rain-on-grid approach for surface runoff within the model domain.

5.3.1 RORB Hydrological Model

5.3.1.1 Model Overview and Catchment Definition

RORB was used to model upstream tributaries of the Nolans Project that possess defined catchments. Rainfall events were applied to catchments, and catchment runoff response was assessed by considering water storage effects within the catchments and within drainage networks. The outcome of the upstream model is a set of time-delayed runoff hydrographs indicative of likely flows during the design rainfall events.

The catchment areas relevant are shown in **Plate 18**.

5.3.1.2 Losses & Storage Parameters

Recession (dampening) of hydrographs is influenced by assumed losses and storage parameters within the RORB model. Principal inputs include loss rates, which represents the potential infiltration losses that reduce the runoff generated from catchments, and Storage Coefficients (k_c), which represents the storage available within the hydrological basin.

In the flood failure scenario (refer **Section 3.1)**, the adopted rainfall event over the FTSF is the PMP event. It is assumed that a hypothetical PMP event over a small localised area is likely to coincide with significant concurrent and antecedent rainfall in the river basin. Consequently, the upstream catchment was assumed to be saturated at the initiation of the RORB model and available infiltration losses are low.

Storage parameters were estimated based upon the following:

- Drainage paths exhibiting low gradients; and
- Low annual rainfall; and
- Storage parameter (K_c) guidance provided in *Australian Rainfall and Runoff* (Pilgrim et al, 1997)

Parameters for the models are listed in **Table 8:**

Table 8
Hydrological Model Parameters

Parameter	Value
Initial Loss	0 mm
Continuing Loss	1.0 mm/hr
Storage Coefficient (k_c)	34.0
Linearity Coefficient (m)	0.8
Reach Coefficient (F_i)	1.0

The input catchment properties are detailed in **Plate 18**, **Table 9** and **Table 10**.

Plate 18
RORB Model Catchment Delineation

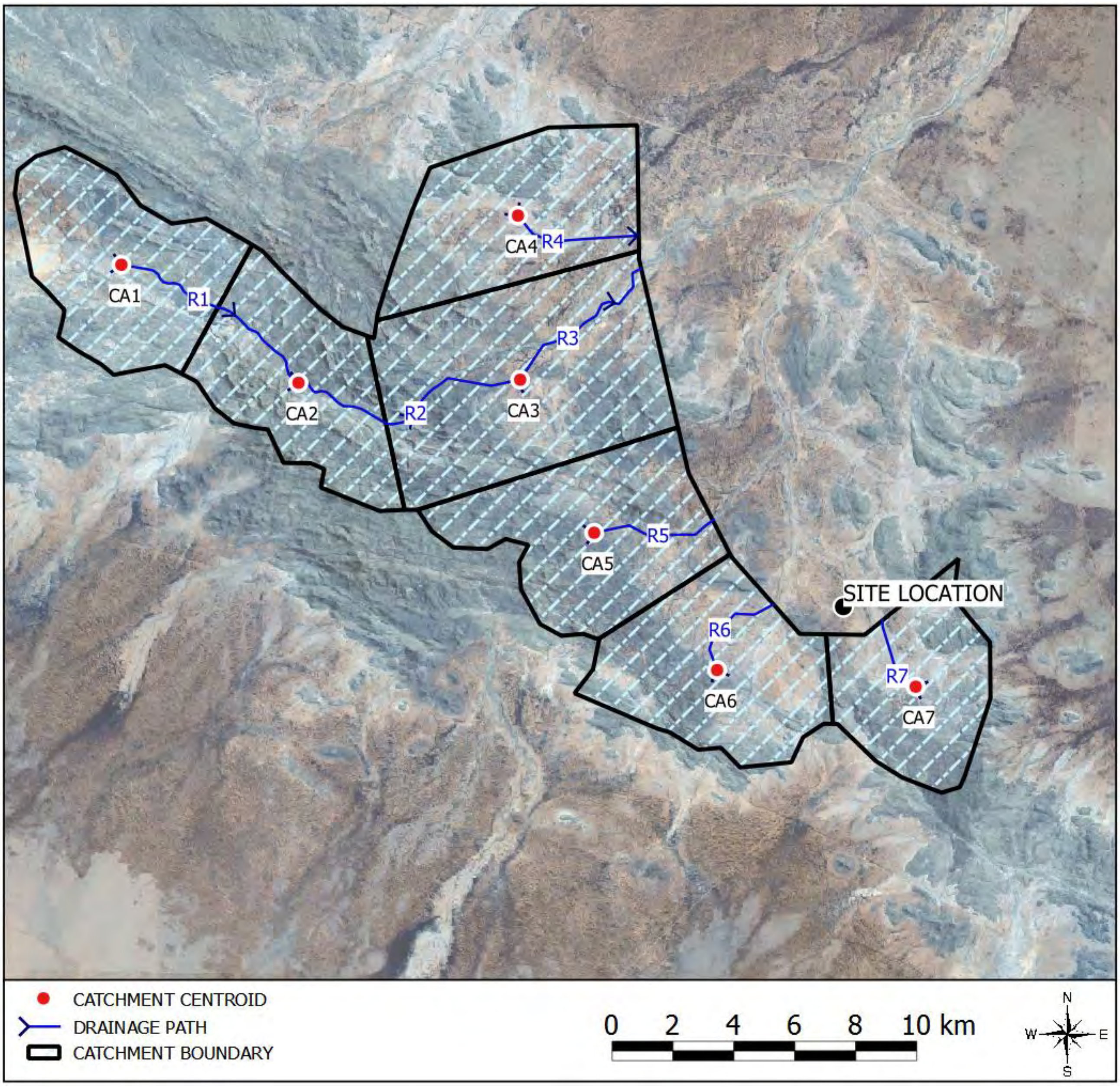


Table 9
RORB Model Catchment Areas

Catchment	Catchment Area
CA1	35.0 km^2
CA2	32.2 km^2
CA3	57.0 km^2
CA4	35.8 km^2
CA5	37.5 km^2
CA6	35.6 km^2
CA7	26.1 km^2

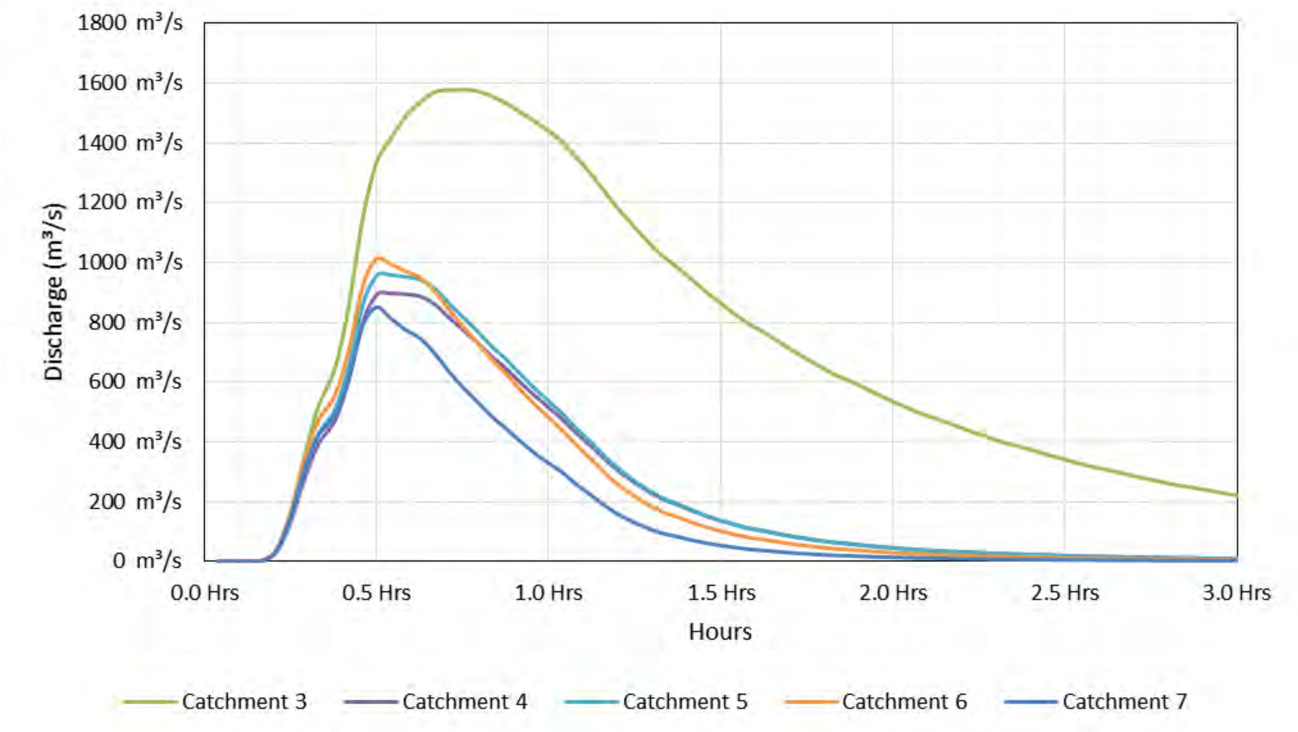
Table 10
RORB Reach Lengths

Reach ID	Length (km)	Grade (%)	Reporting Destination
R1	7.45 km^2	0.46 %	R2
R2	8.44 km^2	0.34 %	R3
R3	5.90 km^2	0.34 %	Woodforde River
R4	4.30 km^2	0.26 %	Woodforde River
R5	4.21 km^2	0.82 %	Kerosene Camp Creek
R6	3.53 km^2	0.80 %	Kerosene Camp Creek
R7	2.70 km^2	0.50 %	Kerosene Camp Creek

5.3.2 RORB Output Hydrographs

The results of the RORB hydrological model are shown in **Plate 19:**

Plate 19
RORB Hydrographs



5.3.3 TUFLOW Rain-on-Grid Module

Catchments comprising non-defined drainages were modelled through the TUFLOW rain-on-grid method. Rainfall depths are directly applied to the TUFLOW 2D mesh over each simulation time-step (one second time-step intervals was selected). The runoff generated in this method is allowed to flow according to topographic inclination. It is noted that this method considers storage and time-delay effects, however no losses were modelled.

6 HYDRAULIC MODELLING

6.1 TUFLOW

A two dimensional dam break model was developed using TUFLOW to simulate the effects of each breach. TUFLOW model development was based on a 10m × 10m gridded elevation raster model, resampled from available SRTM 30m × 30m data, to form a base topography.

No survey data was available to verify the SRTM data. As described in **Section 2.3**, drainage paths downstream of the proposed FTSF location comprise low-gradient sand bed channels varying in width from 5m-50m. The model resolution (30m, resampled to 10m) is considered reasonable for analysis of the main drainage path running north, however minor drainage paths less than 10m wide are unlikely to be fully represented within the topography. Consequently, flood outcomes should be verified against local drainage observations.

6.1.1 Model Domain

The model domain was selected as the catchment boundaries of the Woodforde River, extending past the town of Ti Tree to the north, excepting catchments modelled within the RORB hydrological model (Refer **Section 5.3.1**).

6.1.2 Surface Roughness

A single Manning's roughness coefficient (n) value was assigned to land use type within the modelled area, based on the reference data listed in **Table 11**. Selection of the coefficient was made considering the observed roughness of catchments observed during the site visit (Refer to **Plate 7** and **Plate 8,** and **Plate 9**):

- Drainage path beds comprised of predominantly coarse sandy material. Slightly winding, with limited in-stream obstructions. Exhibits low gradients. Bedding material is likely to be transient in modelled conditions.
- Drainage bank and over-bank sections. Medium coverage of medium sized trees. Extensive coverage of low grass. Nominal 1.0m vertical difference between bank and drainage bed levels.
- Overland flow paths, characterised by sparse coverage of medium trees, ground cover predominantly bare earth.

Table 11-
Reference Manning's N Values

Type of Channel and Description	Minimum	Normal	Maximum
1. Main Channels			
a. clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
b. same as above, but more stones and weeds	0.030	0.035	0.040
c. clean, winding, some pools and shoals	0.033	0.040	0.045
3. Floodplains			
c. Brush			
1. scattered brush, heavy weeds	0.035	0.050	0.070

(Reproduced from Chow V.T, 1959)

The majority of flow areas within the 2D Model are expected to centre around defined drainage paths. Notwithstanding this, as the discharge rate is high and the drainage path width is small, flow is expected to significantly overflow the banks. Consequently, as the resolution (grid sizing) of the model cannot effectively discriminate between creek-bed, overbank and overland flow areas, the adopted Manning's 'n' roughness selected is representative of all regions.

The adopted Manning's n values for modelling purposes are shown in **Table 12.**

Table 12
Adopted Manning's N Values for Each Land Use

Land use	Manning's n
TUFLOW Model Domain (Scattered Light Brush and Low Gradient Drainage Paths)	0.040

6.1.3 Boundary Conditions

Input boundary conditions modelled included the following:

- Dam-break Hydrograph;
- Upstream Runoff from Hydrological Model; and
- Rainfall over model domain.

A single exit boundary condition was applied at the downstream extent of the model, which removes water from the 2D model domain according to a normal flow calculation.

6.2 Results & Verification

The results of the model are depicted in **Figures 001** to **Figure 009.**

6.2.1 SDF Results

The maximum modelled Sunny Day Failure inundation depths are depicted in **Figure 001,** and maximum modelled velocities are depicted in **Figure 002.**

The SDF model was run for a total duration of 24 hours after the start of the dam-break failure. In this time, the dam-break wave travels 27km towards the north. By the end of the model duration, velocities (<1.0m/s) and depths (<300mm) are low, as the wave has dissipated as a result of storage and recession within the drainage path. As no losses have been modelled in the 2D model, it is considered that the dam-break is unlikely to flow as far as the occupants described in **Section 2.7.**

Inundation depths reach a maximum of 5.0m in the area immediate downstream of the failure location. Velocities approach 10 m/s in the immediate downstream area.

6.2.2 FF Results

The maximum modelled Flood Failure inundation depths are depicted in **Figure 003**, and maximum modelled velocities are depicted in **Figure 004.**

The FF model was run for a total duration of 48 hours after the start of the dam-break failure. Compared to the SDF scenario, downstream areas experience significant flooding. Maximum flood depths in the area of Ti Tree approach 5m, with velocities approaching 2.0 m/s.

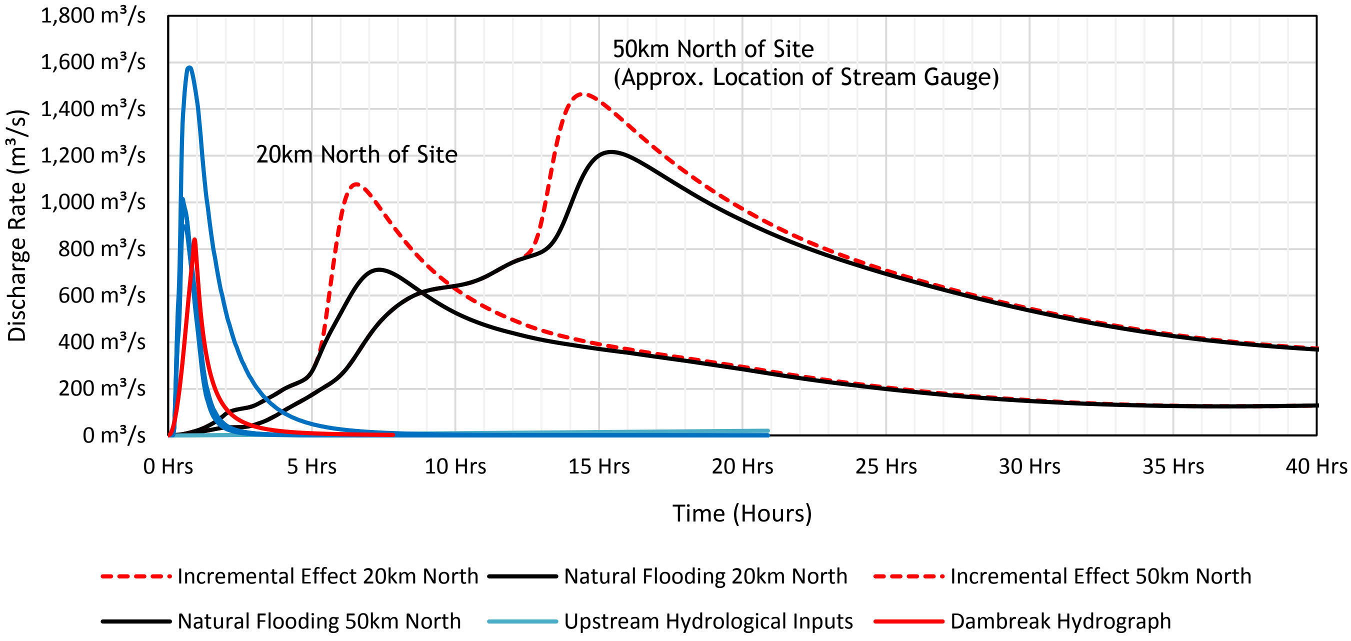
Almost all flooding within the model domain is caused by concurrent flooding, as a result of natural flooding in response to the 1 in 100 year rainfall event. To demonstrate the incremental effect of the dam-break failure, the Flood Failure model was rerun without dam failure. The results of this Natural Flooding Scenario are depicted in **Figures 005** and **006.**

The incremental effect is calculated by subtracting the Natural Flooding scenario inundation depths from the FF scenario inundation depths (Refer to **Figure 007**). In the area of Ti Tree, the incremental depth caused by the Dambreak is less than 10 cm.

Figures 008 and **009** depict cross sectional profiles of the incremental effect in the area of Ti Tree.

The effect is also depicted in **Plate 20** below. The impact of the dam-break hydrograph quickly recesses. The initial dam-break Q_{peak} corresponds to 840.1 m^3/s (solid red line), however this has reduced to approximately 500 m^3/s and 250 m^3/s after travelling 20km, and 50km, respectively (dotted red lines).

Plate 20
Hydrograph Recession

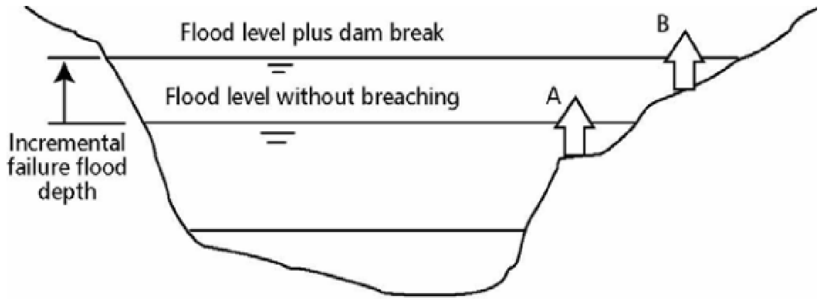


7 POPULATION AT RISK

Population At Risk (PAR) is the number of people expected to be within the failure impact zone in the event of a failure.

PAR has been determined for both the SDF and FF scenarios according to DEWS (2012), which defines "At Risk" as 300mm of flooding within occupied buildings. For the Flood Failure case, an incremental PAR is calculated. Incremental PAR only includes residents in the calculation if they are impacted by the *Incremental Failure Flood Depth*, which is depicted in **Plate 21**:

Plate 21
Incremental Flooding



Reproduced from DEWS (2012)

The assessment of PAR excludes site personnel. Non-itinerant occupants within the model domain were not considered. The Stuart Highway is not inundated in the model, however due to the remoteness of the region, small access tracks and dirt trails within the model domain were assumed unoccupied.

Flood inundation outcomes for the identified downstream occupancy are listed in **Table 13:**

Table 13
Flood Depth Outcomes

Location of Interest	SDF	FF			PAR
	Flood Depth	Total Flood Depth	Natural Flooding Only	Incremental Effect	
Ti Tree	Nil*	0.58m	0.52m	0.06m	Nil
Waste Transfer Station	Nil*	0.84m	0.75m	0.09m	Nil
Pmara Jutunta Community	Nil*	0.03m	0.03m	Nil	Nil
Water Treatment Station	Nil*	0.01m	0.01m	Nil	Nil
Repeater Station	Nil*	0.38m	0.28m	0.10m	Nil

*These locations are not inundated by the SDF dambreak, as the dambreak waters do not reach these locations within the model duration.

The modelled PAR, excluding site personnel, is less than one (PAR <1).

8 POTENTIAL LIVES LOST

Potential Lives Lost (PLL) is a measure of the expected fatalities arising from a failure scenario. In the context of dam-break, the PLL is calculated by applying fatality rates to the Population At Risk (PAR).

The most up to date method for calculating PLL is the Reclamation Consequence Estimating Methodology (USBR, 2014). However, as the Population At Risk (PAR) is less than 1, there are zero Potential Lives Lost (PLL = 0).

9 CONSEQUENCE CATEGORY

The assessment of a consequence category under the ANCOLD Consequence Guidelines (2012a) requires the consideration of PAR and severity of damages, as defined in **Table 14**. The recommended consequence category is the more significant of the PAR outcome and “Severity of Damage and Loss” outcome.

Table 14
Recommended Consequence Category

PAR	Severity of Damage and Loss			
	Minor	**Medium**	**Major**	**Catastrophic**
<1	Very Low	Low	Significant	High C
>1 to 10	Significant (Note 2)	Significant (Note 2)	High C	High B
>10 to 100	High C	High C	High B	High A
>100 to 1,000	(Note 1)	High B	High A	Extreme
>1,000		(Note 1)	Extreme	Extreme

Note 1: With a PAR in excess of 100, it is unlikely Damage will be minor. Similarly with a PAR in excess of 1,000 it is unlikely Damage will be classified as Medium.
Note 2: Change to "HIGH C" where there is the potential of one or more lives being lost. The potential for loss of life is determined by the characteristics of the flood area, particularly the depth and velocity of flow.

Reproduced from ANCOLD (2012)

Knight Piesold has previously performed an assessment of the "Severity of Damage and Loss" of the FTSF in *Nolans Project - Tailings Storage Facilities Engineering Cost Study* (February 2014, Rev 0), indicating a worst case impact of "Medium".

Considered in the context of a PAR of <1, the appropriate consequence category of the FTSF is "Low".

10 REFERENCES

[1] ANCOLD (2012a), *Guidelines on the Consequence Categories for Dams,* Australian National Committee on Large Dams Inc. October 2012a

[2] ANCOLD (2012b), *Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure*. 2012

[3] BOM (2013), Intensity-Frequency-Duration Program, Bureau of Meteorology, accessed February 2016 via http://www.bom.gov.au/water/designRainfalls/revised-ifd/

[4] Chow V.T (1959), *Open Channel Hydraulics*, McGraw Hill Book Company Inc, 1959.

[5] DEWS (2012), *Guidelines for Failure Impact Assessments of Water Dams,* Queensland Department of Energy and Water Supply, 2012

[6] Froehlich (2008), Embankment Dam Breach parameters and Their Uncertainties. ASCE, Journal of Hydraulic Engineering, Vol. 134, No. 12, December 2008, pages 1708-1721. ISSN 0733-9429

[7] Geoscience Australia (2011), *1 second SRTM Derived Products User Guide (Version 1.0.4)*. October 2011.

[8] Graham W. J., (1999), *A Procedure for Estimating Loss of Life Caused by Dam Failure,* US Department of Interior, September 1999, DSO-99-06

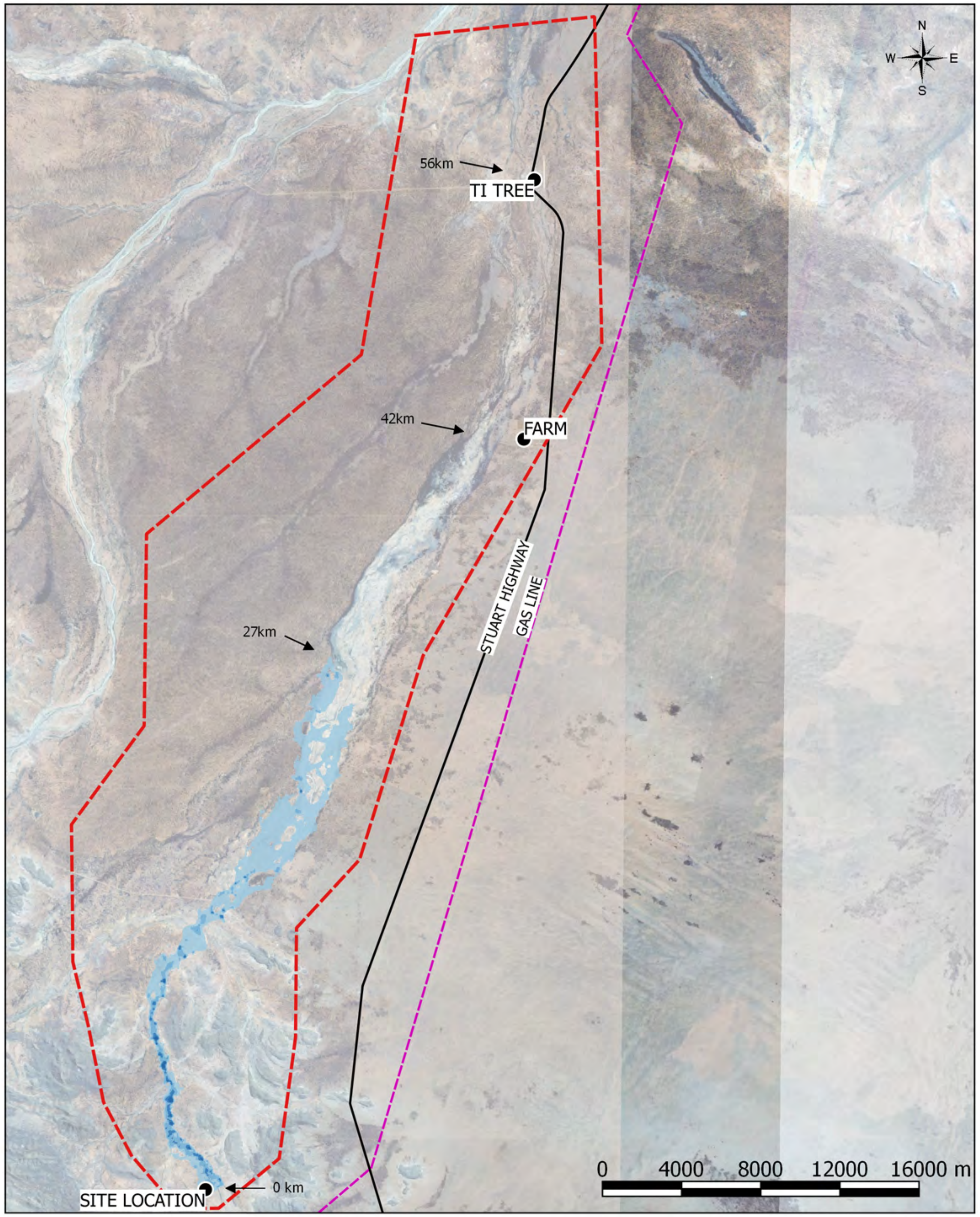
[9] Knight Piesold (2014), *Nolans Project – Tailings Storage Facilities Engineering Cost Study (Rev 0)*, Knight Piesold Consulting (February, 2014)

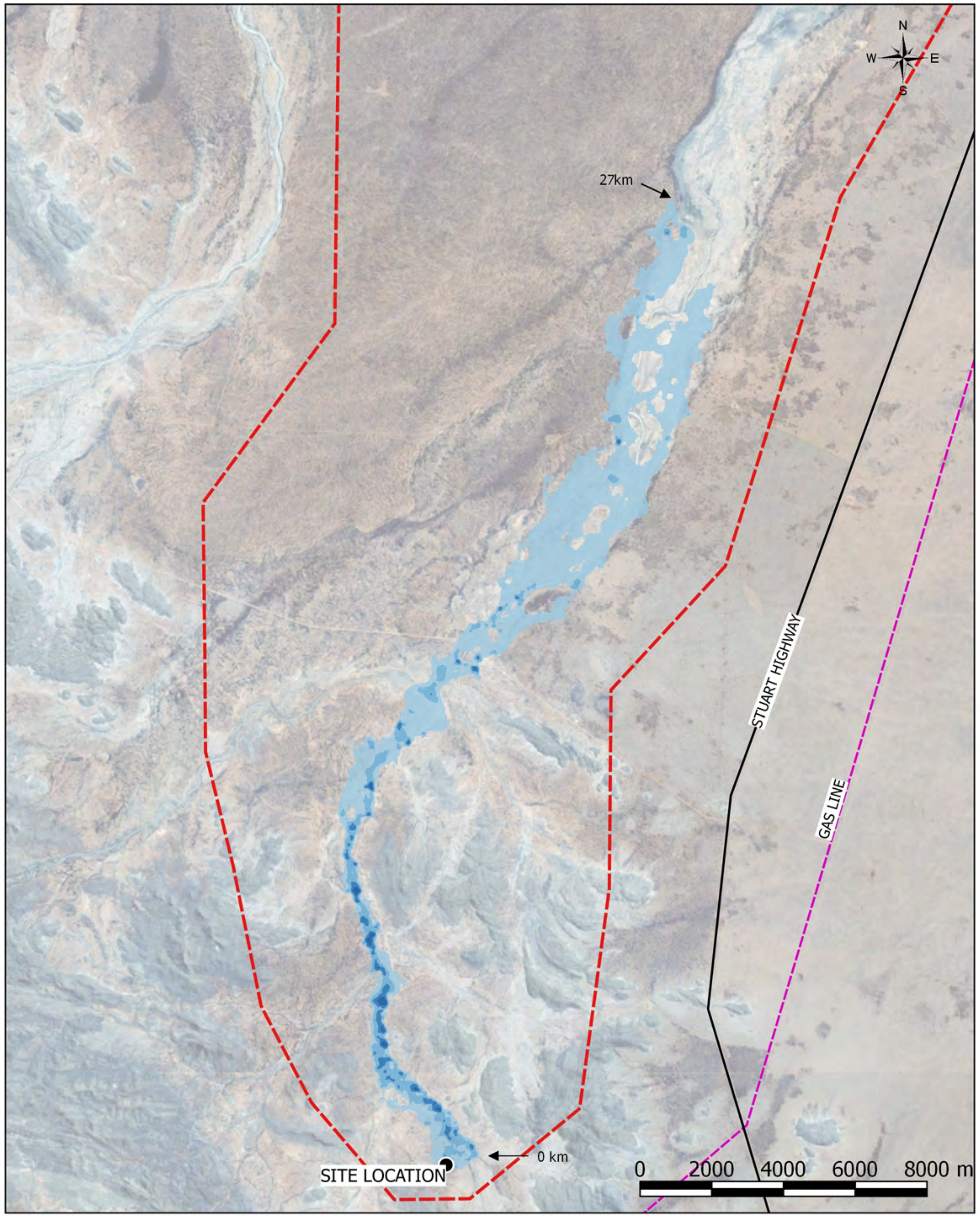
[10] Lycopodium (2014), *Nolans Project – Infrastructure Engineering Cost Study (Rev B)*, Lycopodium Minerals Pty Ltd (February, 2014)

[11] Pilgrim et al (1999), *Australian Rainfall and Runoff Volume 1*. Institute of Engineers Australia

[12] USBR (2014), *RCEM – Reclamation Consequence Estimating Methodology – Guidelines for Estimating Life Loss for Dam Safety Risk Analysis*, US Department of the Interior – Bureau of Reclamation, February 2014.

FIGURES





LEGEND

TUFLOW MODEL DOMAIN

DEPTH

- 0.0 - 0.3 m
- 0.3 - 0.5 m
- 0.5 - 1.0 m
- 1.0 - 1.5 m
- 1.5 - 3.0 m
- 3.0 - 5.0 m

FIGURE: 001

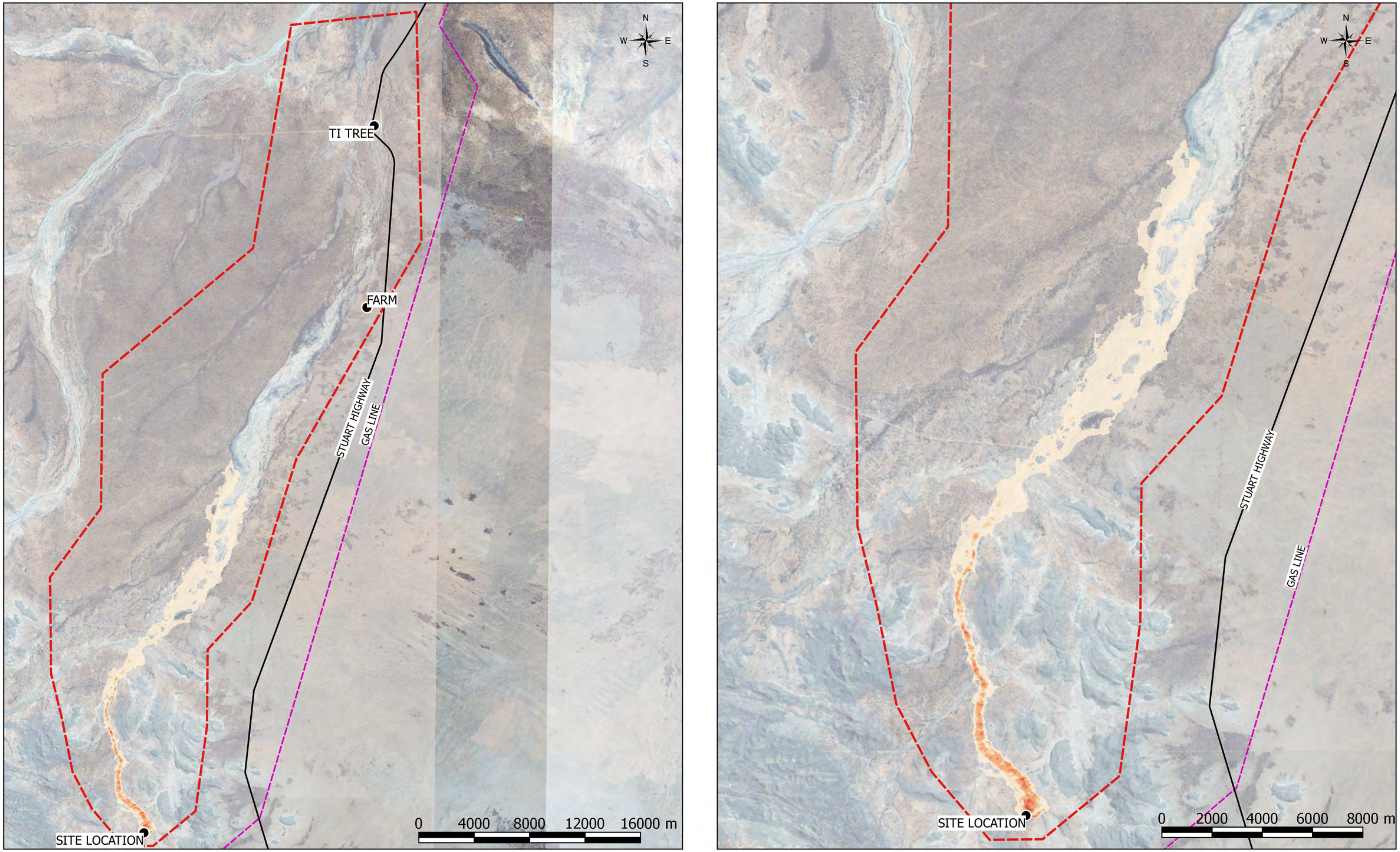
ARAFURA RESOURCES LTD - NOLANS PROJECT

FLOTATION TSF - FAILURE IMPACT ASSESSMENT

SUNNY DAY FAILURE

INUNDATION DEPTH

ATC Williams GROUNDED IN DESIGN



LEGEND

TUFLOW MODEL DOMAIN

VELOCITY

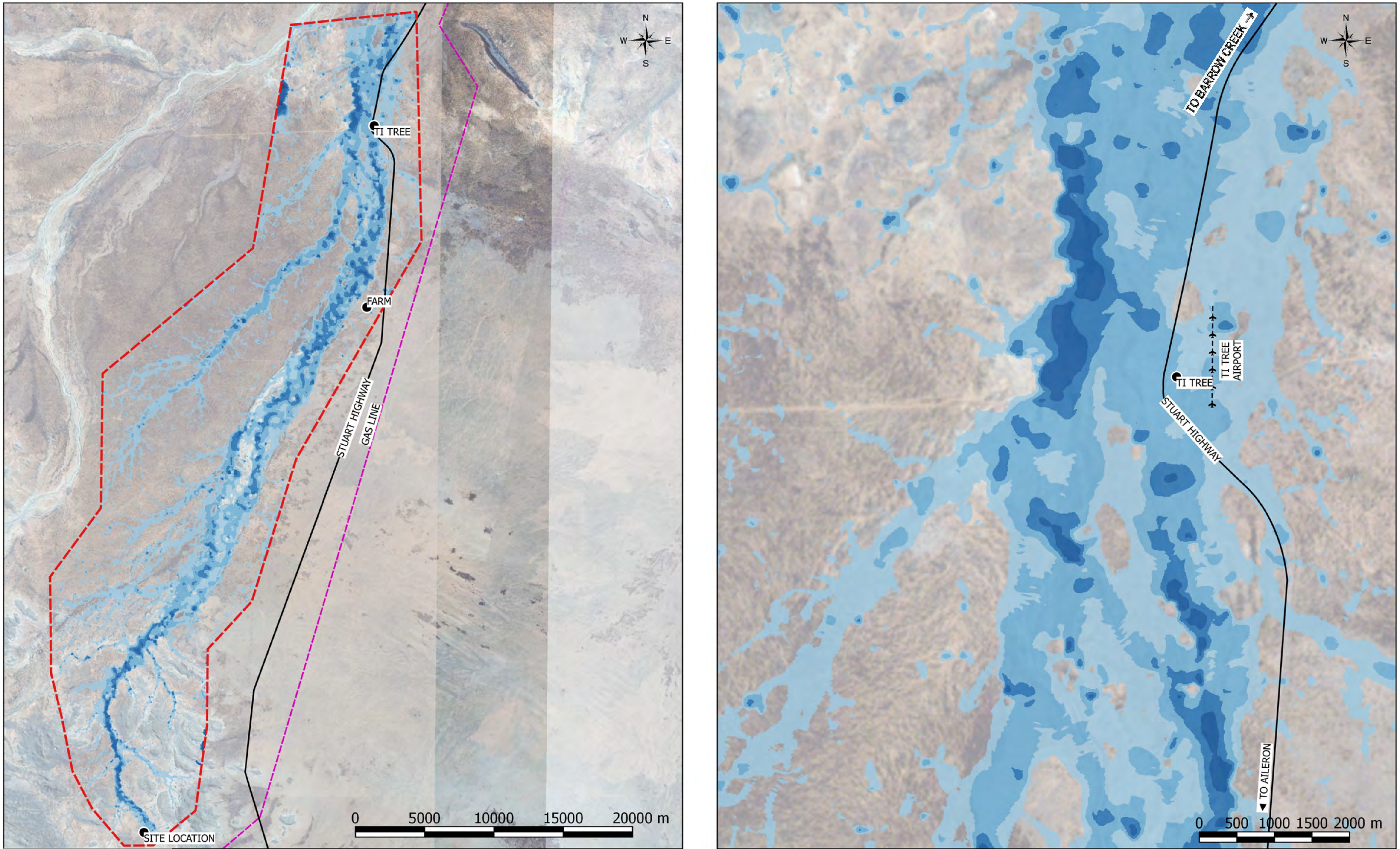
- 0.0 - 0.8 m/s
- 0.8 - 1.0 m/s
- 1.0 - 1.5 m/s
- 1.5 - 2.0 m/s
- 2.0 - 3.0 m/s
- 3.0 - 10.0 m/s

FIGURE: 002

ARAFURA RESOURCES LTD - NOLANS PROJECT
FLOTATION TSF - FAILURE IMPACT ASSESSMENT

SUNNY DAY FAILURE
VELOCITY

ATC Williams
GROUNDED IN DESIGN



Rev. B

LEGEND

TUFLOW MODEL DOMAIN

DEPTH

- 0.1 - 0.3 m
- 0.3 - 0.5 m
- 0.5 - 1.0 m
- 1.0 - 1.5 m
- 1.5 - 3.0 m
- 3.0 - 5.0 m

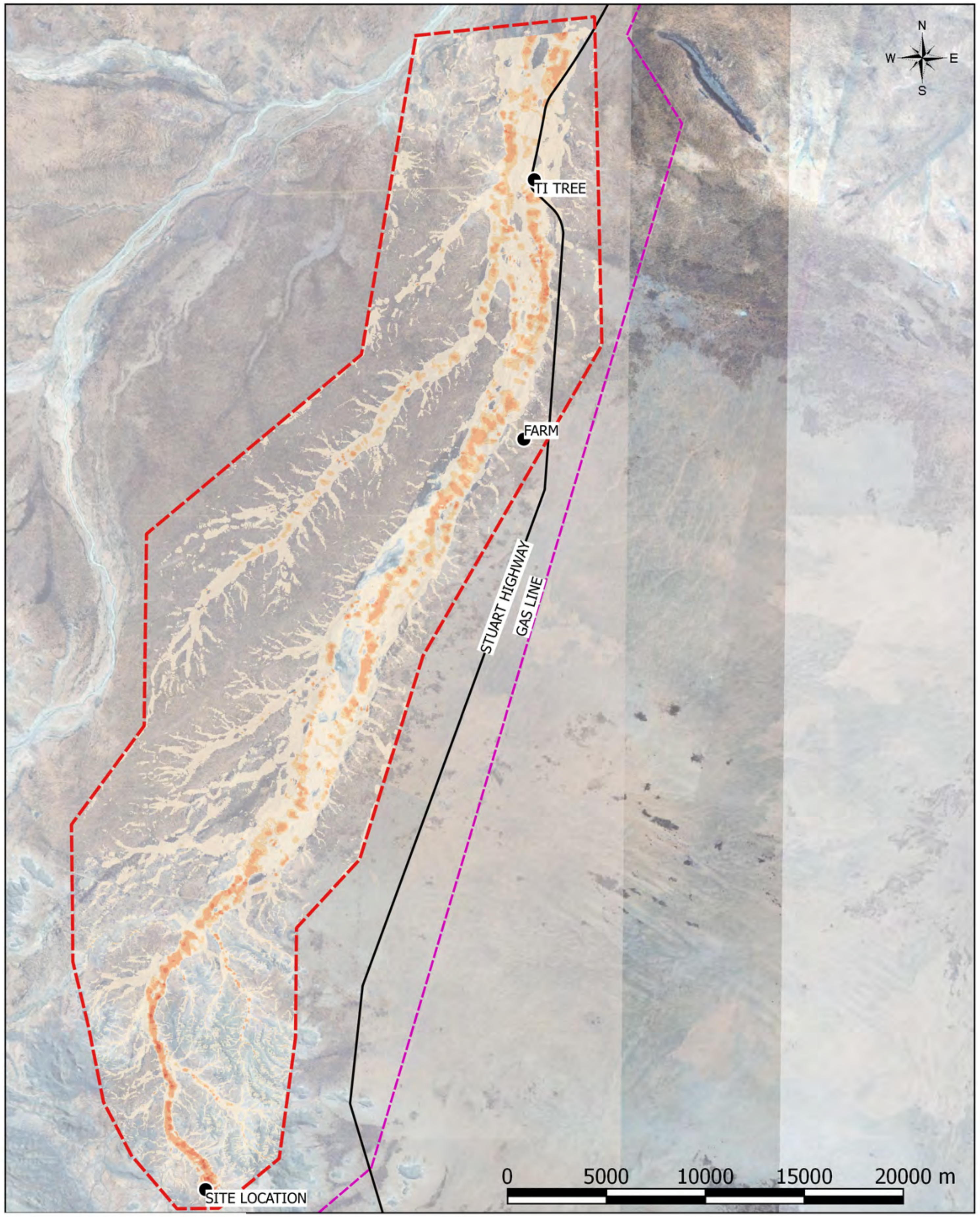
Note: Depths below 0.1m are not shown in this figure

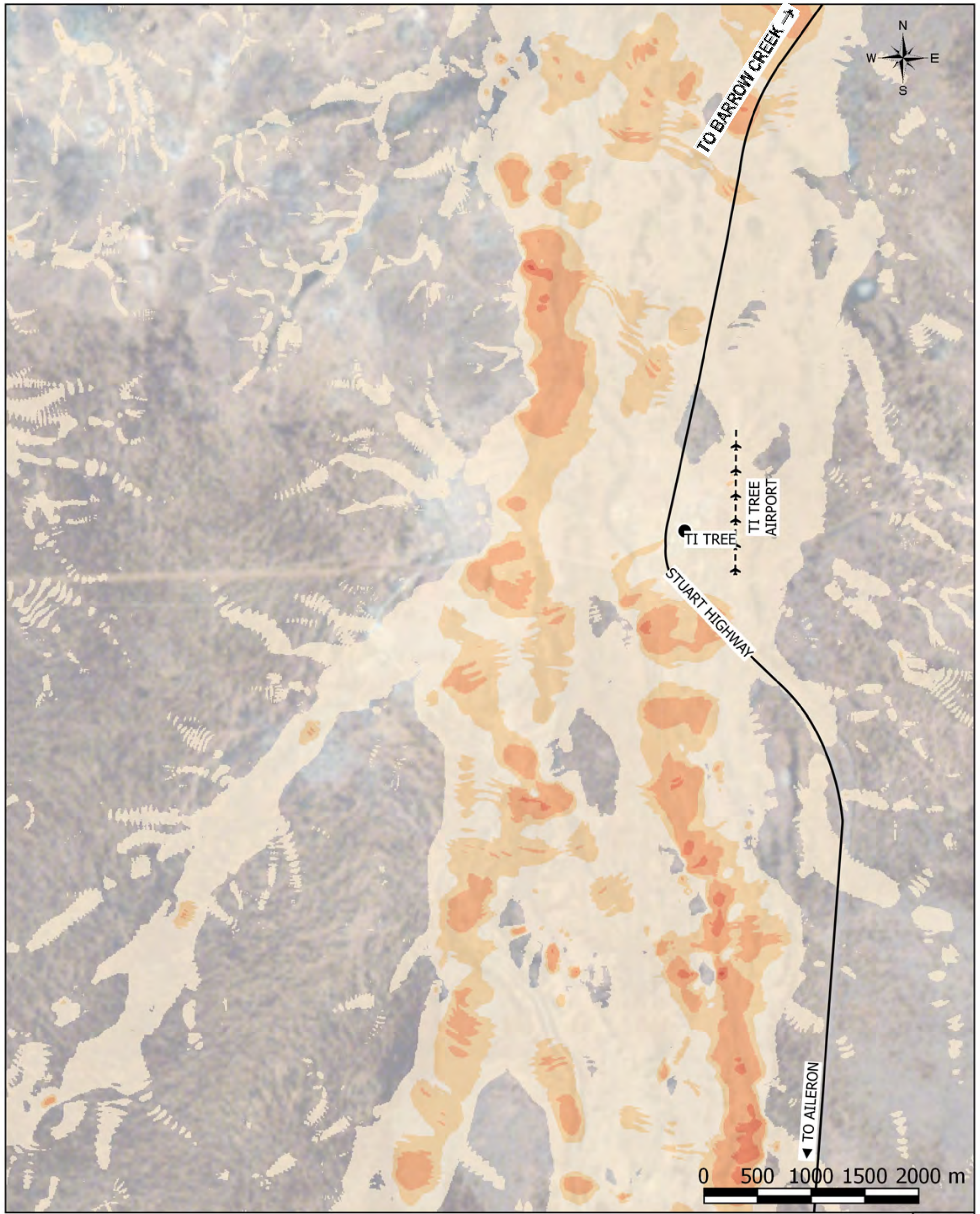
FIGURE: 003

ARAFURA RESOURCES LTD - NOLANS PROJECT
FLOTATION TSF - FAILURE IMPACT ASSESSMENT

FLOOD FAILURE
INUNDATION DEPTH

ATC Williams
GROUNDED IN DESIGN





Rev. B

LEGEND

TUFLOW MODEL DOMAIN

VELOCITY

- 0.25 - 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

Note: Velocities below 0.25 m/s are not shown in this figure

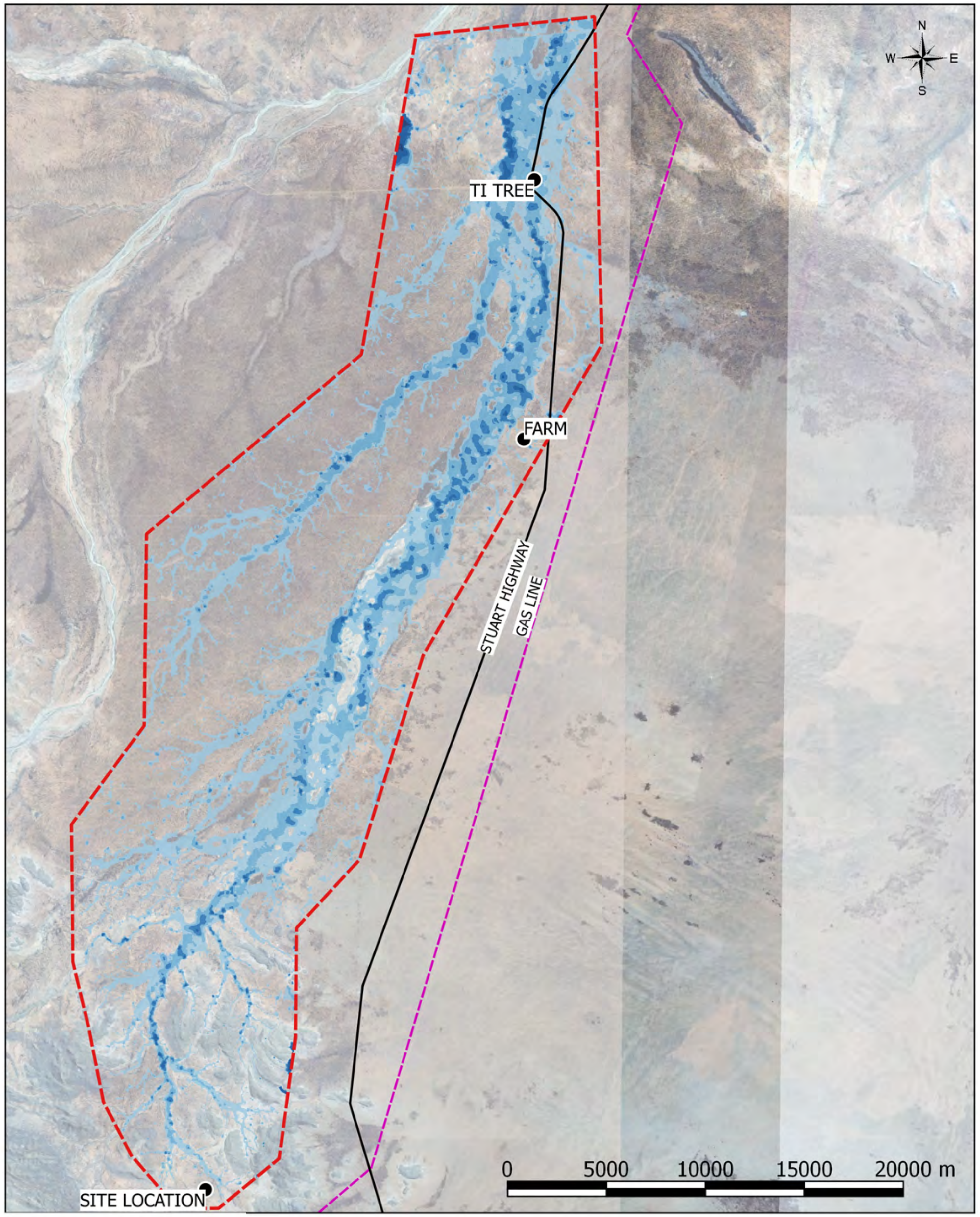
FIGURE: 004

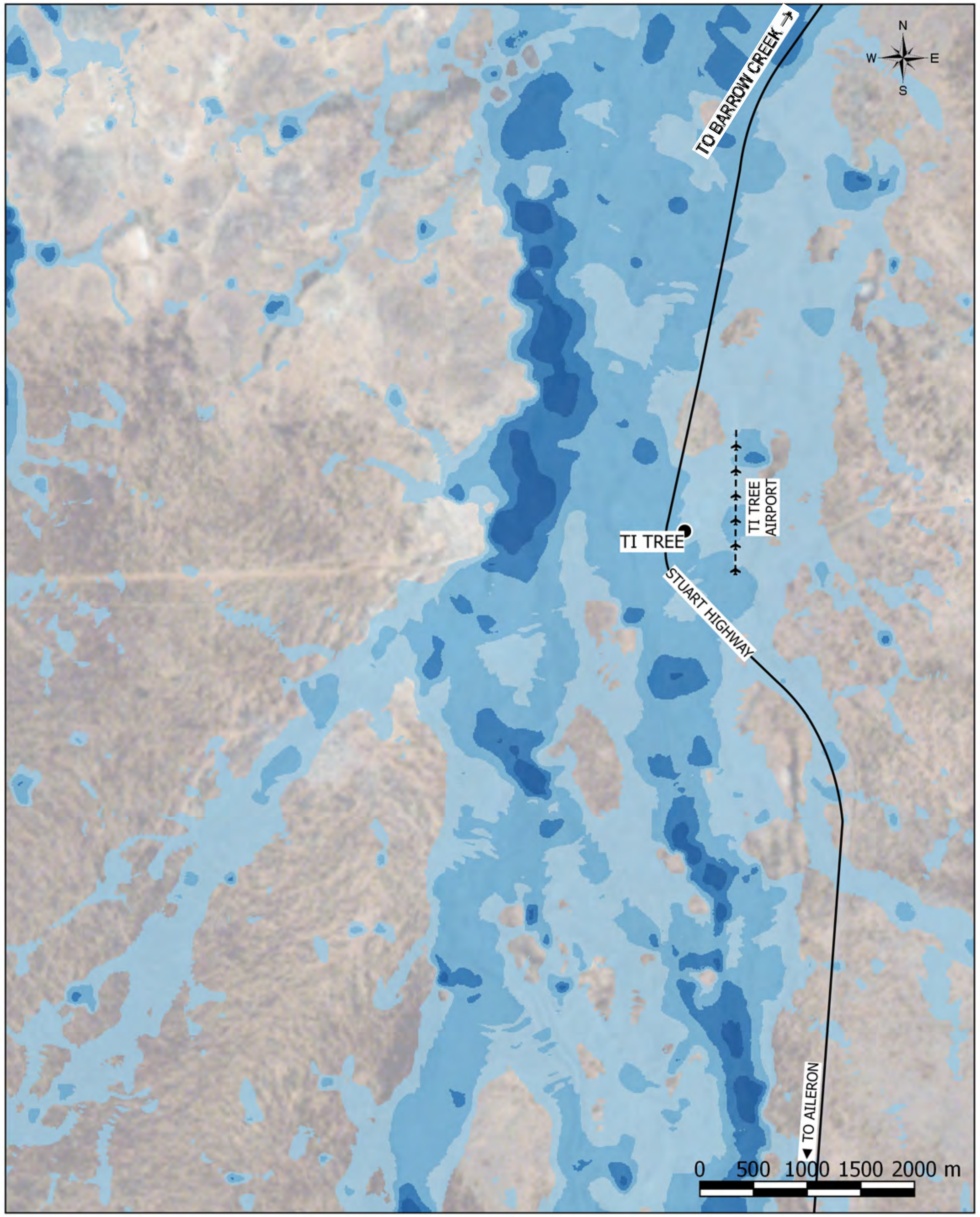
ARAFURA RESOURCES LTD - NOLANS PROJECT

FLOTATION TSF - FAILURE IMPACT ASSESSMENT

FLOOD FAILURE
VELOCITY

ATC Williams
GROUNDED IN DESIGN





LEGEND

TUFLOW MODEL DOMAIN

DEPTH

- 0.0 - 0.3 m
- 0.3 - 0.5 m
- 0.5 - 1.0 m
- 1.0 - 1.5 m
- 1.5 - 3.0 m
- 3.0 - 5.0 m

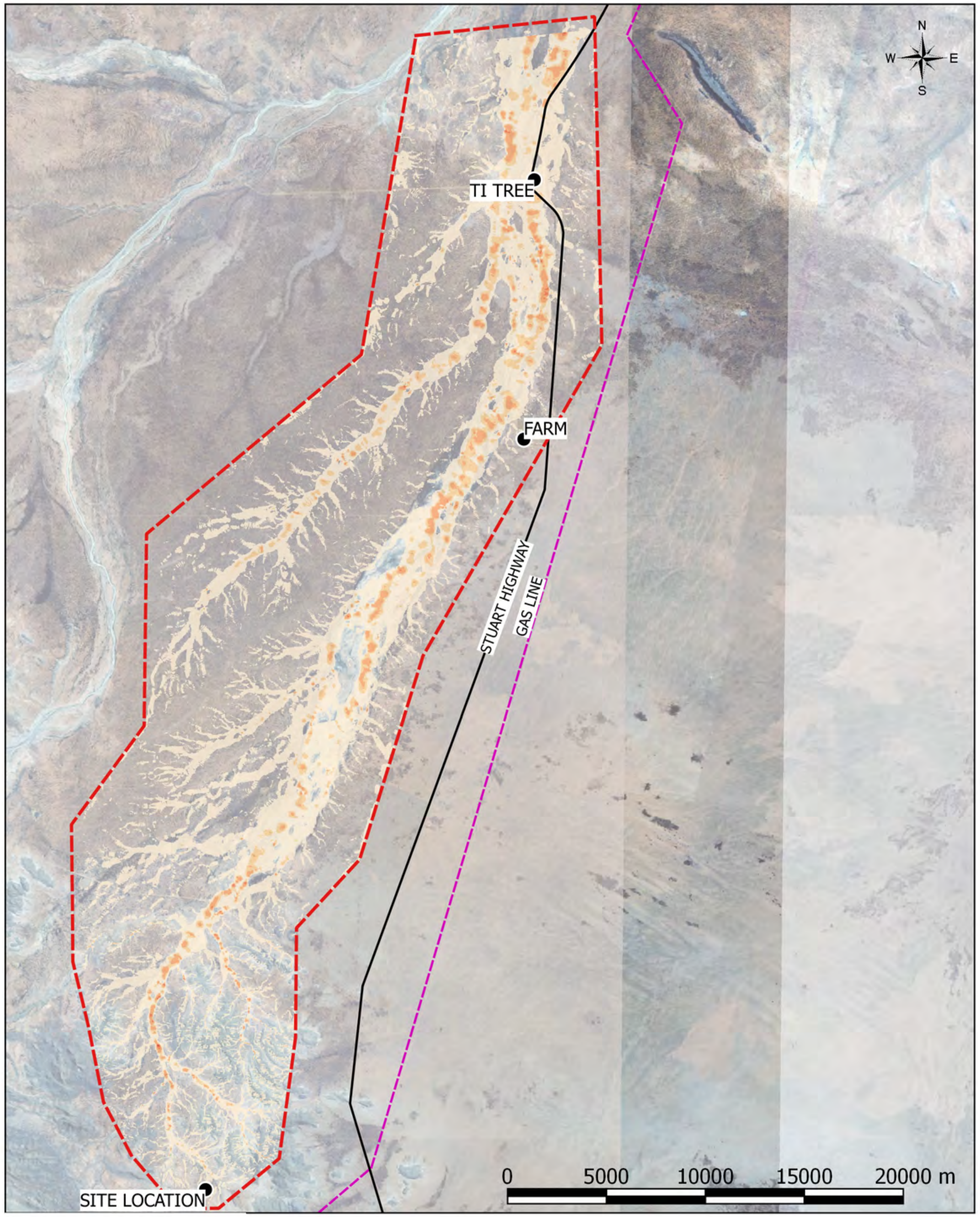
Note: Depths below 0.1m
are not shown in this figure

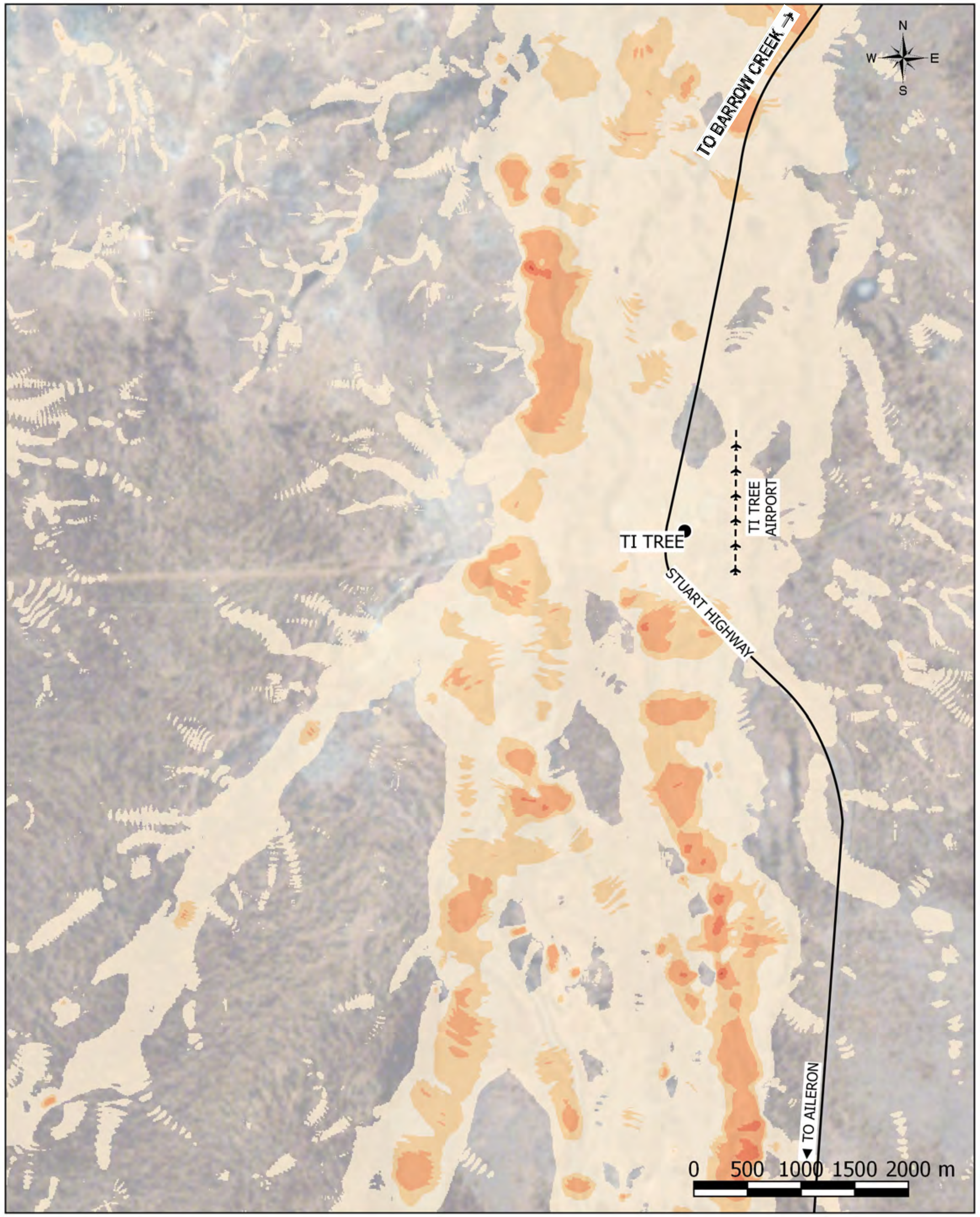
FIGURE: 005

ARAFURA RESOURCES LTD - NOLANS PROJECT
FLOTATION TSF - FAILURE IMPACT ASSESSMENT

1 IN 100 YEAR NATURAL FLOOD EVENT
INUNDATION DEPTH

ATC Williams
GROUNDED IN DESIGN





LEGEND

TUFLOW MODEL DOMAIN

VELOCITY

- 0.25 - 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

Note: Velocities below 0.25 m/s are not shown in this figure

FIGURE: 006

ARAFURA RESOURCES LTD - NOLANS PROJECT

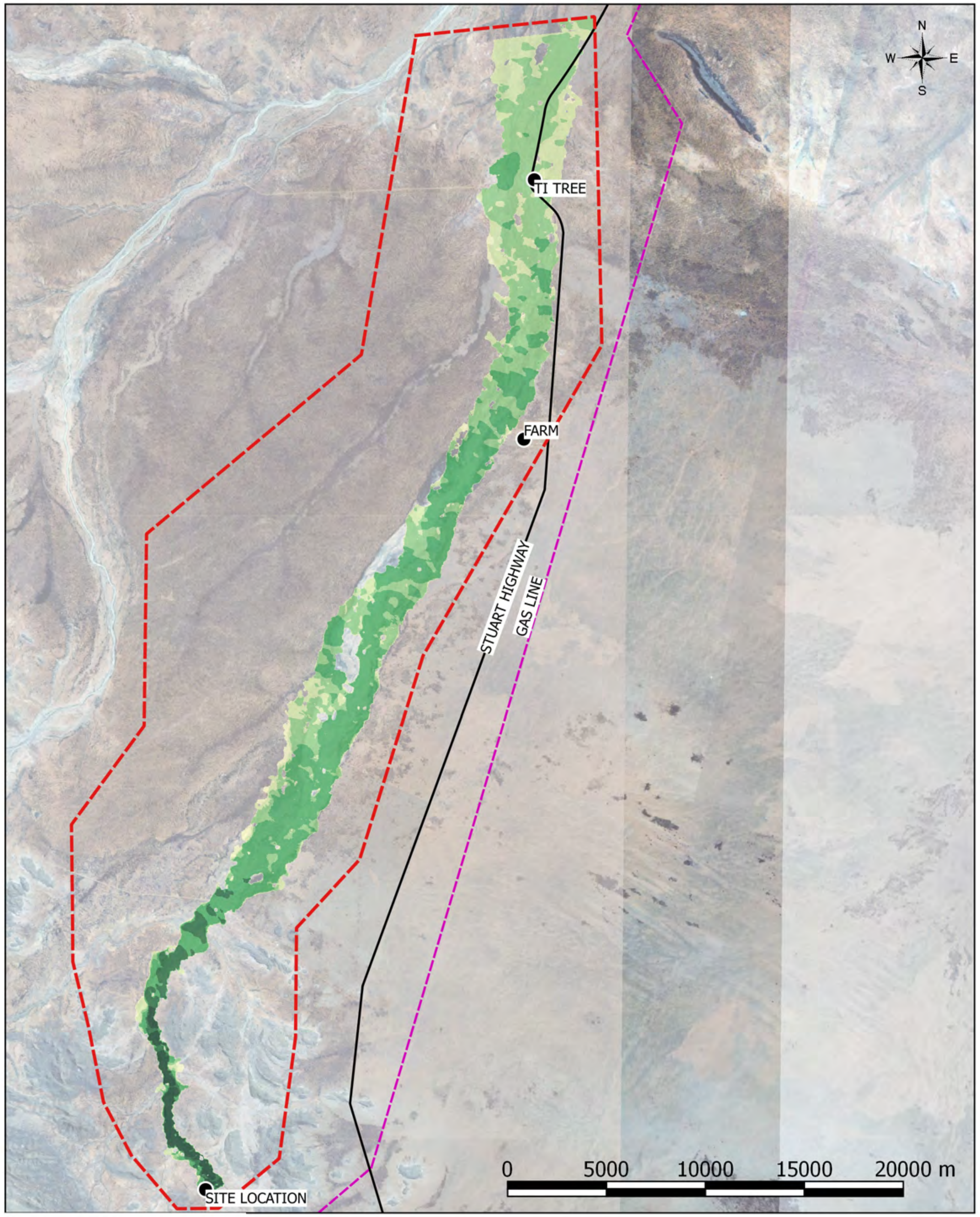
FLOTATION TSF - FAILURE IMPACT ASSESSMENT

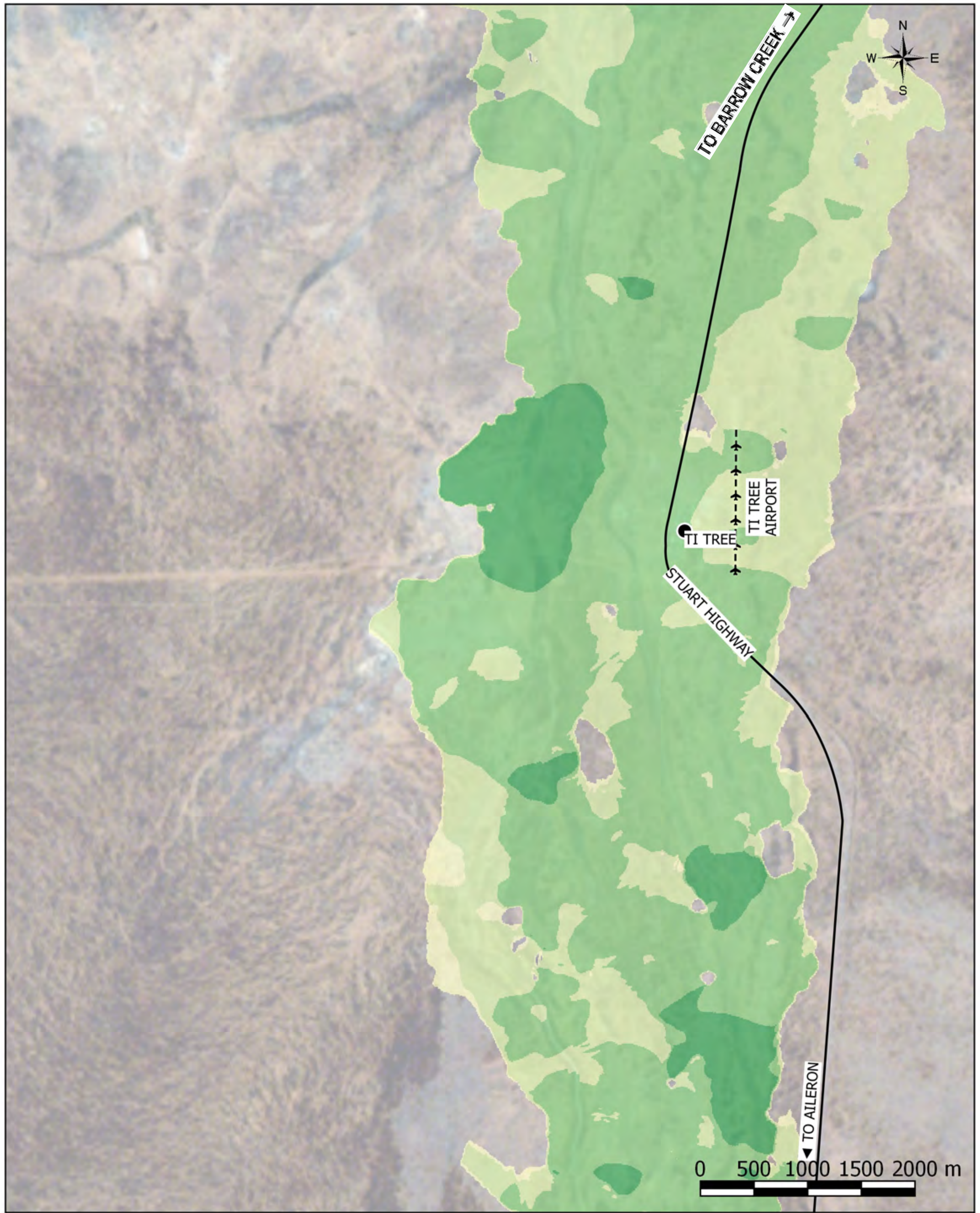
1 IN 100 YEAR NATURAL FLOOD EVENT

VELOCITY

ATC Williams

GROUNDED IN DESIGN





Rev. B

LEGEND

TUFLOW MODEL DOMAIN

INCREMENTAL DEPTH

- 0 - 0.01 m
- 0.01 - 0.05 m
- 0.05 - 0.1 m
- 0.1 - 0.3 m
- 0.3 - 0.5 m
- 0.5 - 1.0 m

FIGURE: 007

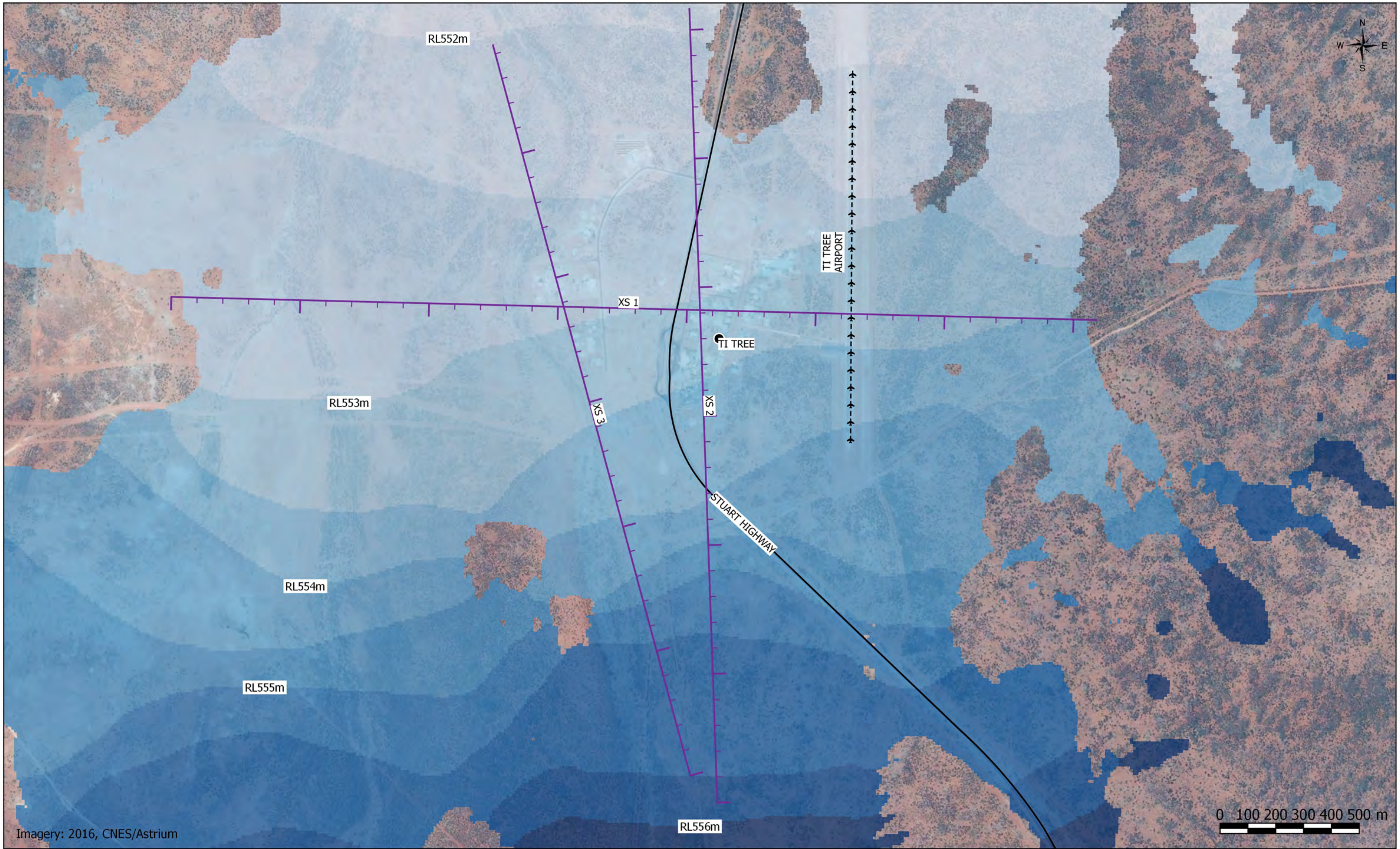
ARAFURA RESOURCES LTD - NOLANS PROJECT

FLOTATION TSF - FAILURE IMPACT ASSESSMENT

FLOOD FAILURE

INCREMENTAL EFFECT (DEPTH)

ATC Williams GROUNDED IN DESIGN



LEGEND

100m MARKER
500m MARKER

Rev. B

FIGURE: 008

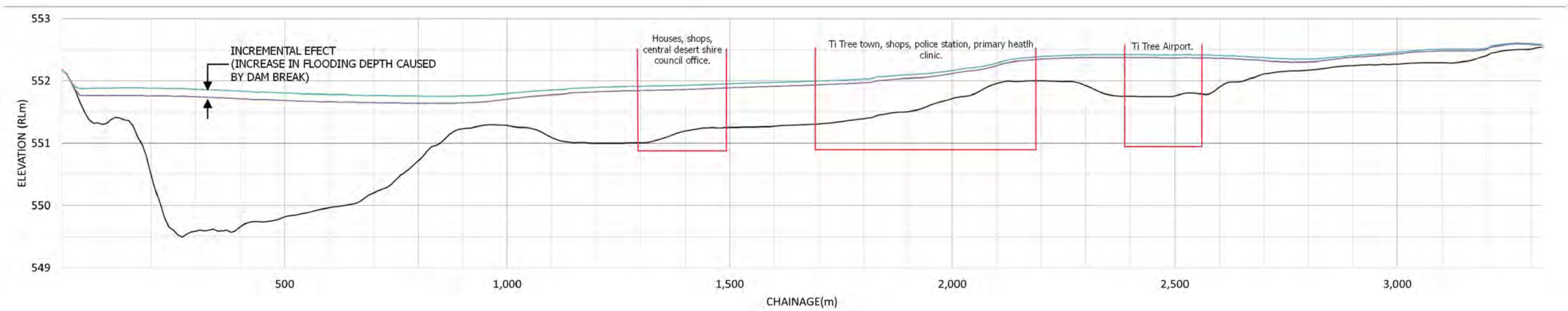
ARAFURA RESOURCES LTD - NOLANS PROJECT

FLOTATION TSF - FAILURE IMPACT ASSESSMENT

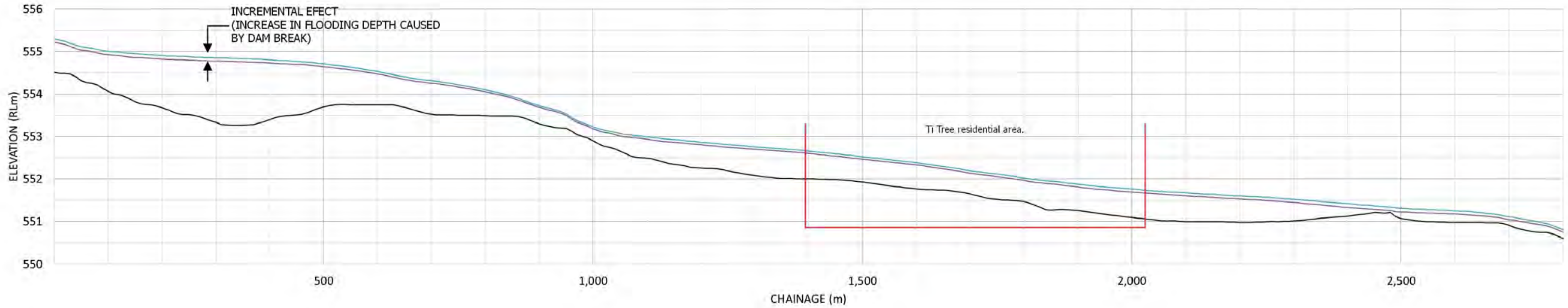
FLOOD FAILURE
LOCATION OF CROSS SECTIONS

ATC Williams
GROUNDED IN DESIGN

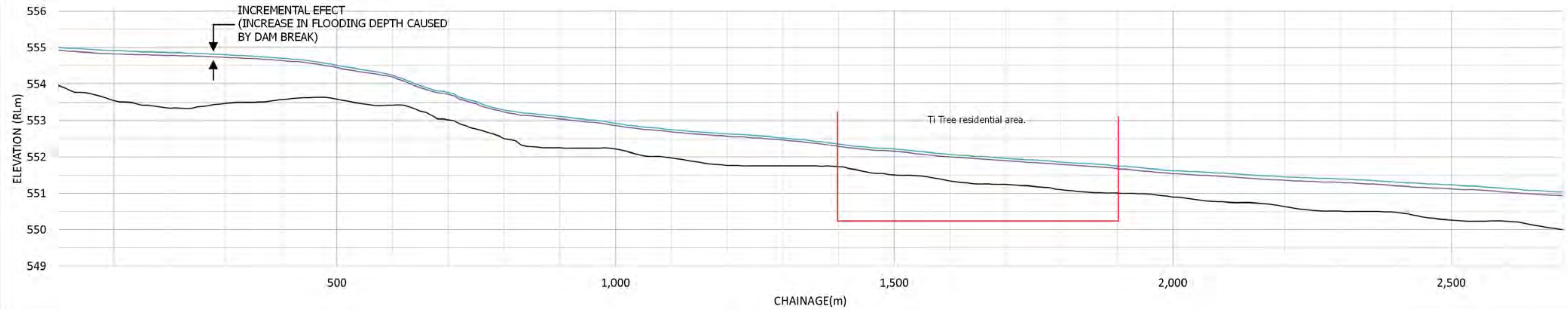
XS1



XS2



XS3



LEGEND

- NATURAL SURFACE
- NATURAL FLOOD EVENT
- FLOOD FAILURE

Rev. B

FIGURE: 009

ARAFURA RESOURCES LTD - NOLANS PROJECT

FLOTATION TSF - FAILURE IMPACT ASSESSMENT

FLOOD FAILURE

ELEVATION PROFILE

ATC Williams

GROUNDED IN DESIGN