BHP

Stantec

Ministers North Trucking

Surface Water Impact Assessment Report

Prepared for BHP WAIO

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Prepared for BHP WAIO

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Executive Summary

Stantec Australia Pty Ltd was engaged by BHP Western Australia Iron Ore (WAIO) to undertake an updated surface water assessment for the development of a new mining area consisting of two pits, a transport corridor connecting Ministers North to Yandi, and new mine site infrastructure.

Stantec previously undertook a Surface Water Environmental Impact Assessment (EIA) in 2018 in support of the Part IV environmental approvals. Since the completion of the assessment, the proposed layout of the Ministers North mine has changed. BHP are conducting the updated EIA utilising the outputs from this Surface Water Assessment Report.

Three hydrological models for three catchments were developed in RORB in accordance with Australian Rainfall and Runoff (ARR2019). RORB results were used to provide inflow hydrographs for major catchments in the HEC-RAS 2D hydraulic model and to provide validation of hydraulic parameters such as Manning's roughness applied to the HEC-RAS 2D rain-on-grid coverage area.

The three catchments include:

1. Yandicoogina Creek
2. Central Tributary to Marillana Creek, covering the central part of the development envelope
3. Mungadoo Tributary to Marillana Creek, previously referred to as "Northern" in 2018 study as it related to Yandicoogina Creek

Comprehensive sensitivity analyses were undertaken for the hydrological catchments to ensure the defensibility of the adopted parameters and peak flow rates when compared to six regional estimation validation methods. The measured data from two local BHP gauges illustrated that very frequent events, such as the events on the 14th of January 2024 (estimated 3EY and 4EY) and 24th to 25th January 2024 (estimated 1EY to 2EY) produced a runoff response from the catchment.

The resulting 1% AEP peak flow rates at the outlet for the Yandicoogina Creek Catchment, Central Tributary and Mungadoo Tributary Catchments are 830 m^3/s, 392 m^3/s, and 275 m^3/s, respectively.

The 2D HEC-RAS model was set with external inflow hydrographs applied as time series boundary conditions for major catchment areas, and direct precipitation applied to the 2D flow area. Excess precipitation hyetographs generated in RORB were applied to the Rain-on-grid (ROG) model based on representative catchment sizes. The adopted roughness coefficient was adjusted to provide the best match for runoff volume, peak discharge rate, and response time at selected validation nodes.

Hydraulic modelling was developed for the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 1% AEP, and 1 in 10,000 AEP (with climate change uplift) flood events under the following three scenarios:

1. *Baseline condition*, as reflected in the 2024 digital elevation model.
2. *Life of mine condition (LoM)*, with the trucking route, mine pits, overburden storage areas (OSAs), stockpiles, and associated infrastructure in place.
3. *Closure condition*, with the trucking route and stockpiles removed.
4. *Rail closure condition,* with the BHP railway removed.

This report includes depth, velocity, and water surface elevation afflux results for the 50% AEP, 1% AEP and 1 in 10,000 AEP events. The remaining events are available in the accompanying electronic files. In general, flow depths are increased upstream of the proposed trucking route embankment, OSAs, and stockpiles and decreased downstream.

Where landforms block flow paths, ponded water would be subject to infiltration and evaporation, which is not modelled. Where culvert crossings are provided, the upstream afflux is temporary as the flows recede by the end of the simulation.

The loss in regional catchment area based on the pits is approximately 0.35% for the Weeli Wolli Creek catchment and 0.64% for the Marillana Creek catchment.

Five sediment basin locations have been indicatively provided to intercept sediment from any water which is pumped out of the pits (with an assumed discharge rate of 100L/s), as well as capture any runoff from disturbed surfaces (within reason) before the runoff discharges to the creeks.

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

1 Introduction

Stantec was engaged by BHP Western Australia Iron Ore (WAIO) to undertake an updated surface water impact assessment for the proposed mine and transport corridor development of the Ministers North Trucking project.

1.1 Study Objectives

The key study objectives are to assess the flood impacts between the Baseline and Life of Mine (LoM) conditions for the 50%, 20%, 10%, 5% and 1% events, as well as the Closure scenarios for the 1 in 10,000 Annual Exceedance Probability (AEP) event inclusive of climate change uplift estimates.

The revised mine layout required additional hydrological modelling and an extended hydraulic model with updated infrastructure within the entire development envelope. Sensitivity assessment of the hydrological and hydraulic models was undertaken to provide confidence in the model parameters considering the uncertainty and limited gauge information in the local catchment areas.

Impacts to the study area were considered in relation to loss of regional catchment area, changes in flood levels, velocity, and flow rates to inform the future BHP environmental impact assessment.

1.1.1 Consultation

Progress meetings were held between the project team and BHP throughout the duration of the project. Meeting minutes for key project decisions and the comments register addressing BHP comments on earlier revisions of the figures and report were supplied in addition to this report.

1.1.2 Limitations

The following scope limitations have been adopted for the study:

1. The environmental impact assessment was excluded from Stantec's engagement following inception of the project. It is understood this is being undertaken by BHP internally.
2. Assessment of the Marillana Creek influences is excluded from this study. The peak discharge rates for Marillana Creek were based on a flood frequency assessment included in the Marillana Creek Baseline Hydrology Report (Advisian, 2023).
3. An assessment of cyclonic activity is not included in this study.
4. Climate change uplift is based on draft updated guidance as of March 2023 and was considered best available information at the time of project commencement. Updates to Australian Rainfall and Runoff in August 2024 impacted these assumptions and have been included as sensitivity analysis. Future design assessments are strongly recommended to adopt the updated ARR 4.2 guidance.
5. The Life of Mine condition included in this report is subject to change and will need to be updated for the final configuration of mine infrastructure, including (but not limited to) road design surfaces, haul roads, access roads and culverts. Stantec is unable to provide guarantee that the surface water modelling results based on the "snapshot in time" design inputs at September 2024 can be relied on for ultimate surface water impact extents.
6. Loss of catchment area due to borrow pits have not been included in this assessment. Proposed mitigation measures are outlined in Section 9 to outline future borrow pit considerations.
7. High-level mitigation measure and sediment management recommendations have been provided. Alignment of these measures with BHP discharge licences are to be confirmed.

1.2 Site Description

The area of interest illustrated in Figure 1-1 is located to the south of the BHP Yandi hub in the Pilbara region of Western Australia.

The Ministers North Trucking project is intended to sustain WAIO production at 310 Mtpa (base plan) and support future growth to 330Mtpa (target plan). Ministers North is an above water table (AWT) Brockman ore deposit located 15 km south-east of the Yandi hub that proposes to utilise latent processing and rail capacity. It has an ore reserve of approximately 250 Mt; LoA24 included mining MN at 15M tpa over a 10-year mine life.

The principal changes to the layout from the assessment previously conducted by Stantec in 2018 are that the previously proposed West Pit, East Pit and overburden storage area (OSA) East are no longer being considered. An updated model extent was required to encompass the entire development envelope that extends to the Yandi hub.

1.3 Hydrological Setting

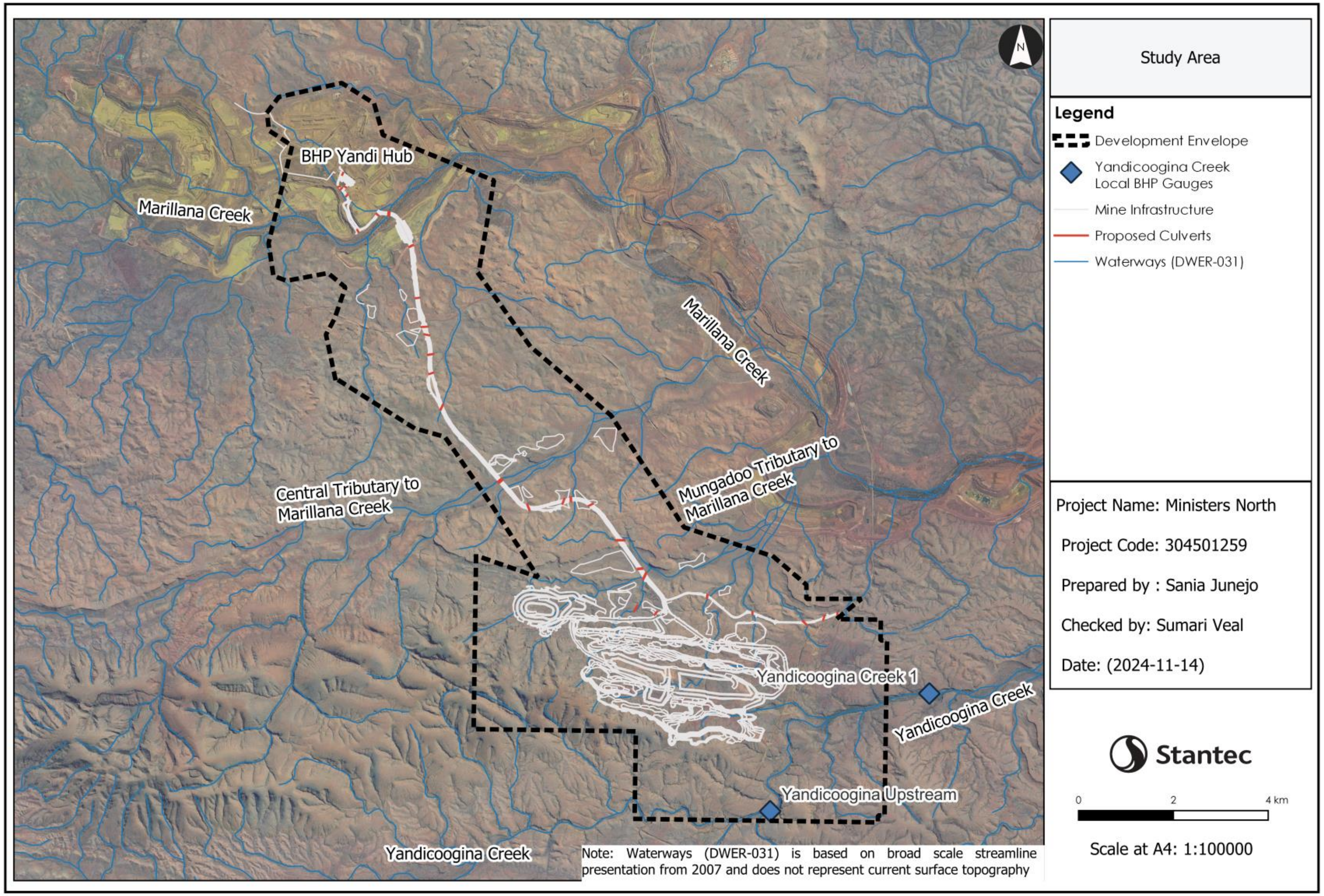
The proposed development site typically grades to the east and intersects lower Marillana Creek, several tributaries of Marillana Creek (including those named Central Tributary and Mungadoo Tributary for the purposes of this study), and Yandicoogina Creek. The Marillana Creek headwaters rise from the high relief areas of Hamersley Range, draining into the Munjina Claypan upstream of the study area.

As for most parts of the Pilbara, the watercourses are non-perennial. Creek flow is ephemeral, occurring only after significant and intense rainfall events.

Two local BHP gauges are located within and downstream of the development envelope as shown in Figure 1-1.

The study area is in an area that is subject to an arid grassland climate with very hot summers and mild winters.

Temperature variations in the region can be large with variable rainfall., mostly occurring in the summer wet season. The mean annual evaporation rate is approximately 2,000 mm/year, exceeding the mean annual rainfall.



Note: Waterways (DWER-031) is based on broad scale streamline presentation from 2007 and does not represent current surface topography

This document has been prepared based on information provided by others as cited in the data sources. Stantec has not verified the accuracy and/or completeness of this information as shall not be help responsible for any errors or omissions which may be incorporated herein as a result. Stantec assumes no responsibility for data supplied in electronic format, and recipient accepts full responsibility for verifying the accuracy and completeness of the data.

Figure 1-1 Site Boundary

1.4 Supplied Data and Projection

The key files supplied by BHP which have been incorporated as the existing surface and hydrological features for modelling include:

1. **1m xyz topographical data** (merged and reprojected into 2024_MN_Merge_YAN94.tif):
    a. MNT_AOI1_Extracts.xyz
    b. MNT_AOI4B_Extracts.xyz
    c. MNT_AOI5A_Extracts.xyz
    d. MNT_AOI5b_Extracts.xyz
    e. MNT_AOI6a_Extracts.xyz
    f. MNT_AOI6b_Extracts.xyz
    g. MNT_AOINEA_Extracts.xyz
    h. MNT_AOINEB_Extracts.xyz
    i. MTD_AOI2_Extracts.xyz
    j. MTD_AOI4_Extracts.xyz

Positional uncertainty is horizontal 0.3m and vertical 0.1m, with capture dates ranging from July 2018 to May 2024. Earlier survey in May 2013 makes up only a minor section of the study area and has a positional uncertainty of 1m.

2. **1m GeoTIFF 2013 to January 2024 LiDAR** (merged and used as a secondary topographical layer for areas not covered by 2024 updated xyz files)
    a. MNT_WestNorth_TIFF
    b. MNT_East_TIFF
    c. MNT_West_South_TIFF_tif
    d. MNT_DE_Central_TIFF
    e. MNT_DE_North_TIFF
    f. MNT_DE_South_TIFF
    g. Yandicoogina_Gorge.tif

3. **Aerial imagery**
    a. MNT_AerialImagery_YAN94_231103 (does not cover extent of Yandicoogina Creek Rocks gauge)
4. **Development Layers**
    a. MN_DevelopmentEnvelope_MGA50
    b. Bechtel_YANDI_MN_Haul_Road_Yan94
    c. IO MAP EFE EARTHWORKS.dxf
    d. IO MAP LV SME ROAD.dxf
    e. IO MAP LV YANDI ROAD.dxf
    f. IO MAP PIPELINE CORRIDOR.dxf
    g. IO MAP W5 XING.dxf
    h. MN_Haul_Roads_Yan94.dxf
    i. MN_New_Stockpiles_Yan94.dxf
    j. MN_Temporary Access Roads_MGA50
    k. Future_Max_Disturbance_MGA50
    l. MN_Power_Alignment_MGA50
    m. MN_132kV Powerline Diversion_MGA50
    n. MN_Additional Topsoil Storage_MGA50
    o. MN_Batch Plant_MGA50

    p. MNT_TFBorrowAreas_MGA50_Filtered
    q. MN_Construction Water Pipeline_MGA50
    r. MN_EOY0 PCF Communications Points_MGA50
    s. MN_EOY0 PCF Communications Tracks_MGA50
    t. MN_EOY1 Communications Points_MGA50
    u. MN_EOY1 Communications Tracks_MGA50
    v. MN_EOY0 PCF WTS Infrastructure Locations_MGA50
    w. OHP3_STOCKYARD_TLO
    x. RT_BHP_220kV_re-route_20230203_YAN94
    y. SPS_GI_YANDI_COORDS_MGA50
    z. Stockyard_Stockpiles
    aa. MN_Explosives Storage_MGA50
    bb. MN_Laydown Buildings_MGA50
    cc. MN_Modules Transport Route_MGA50 (polylines only – insufficient for modelling purposes)
    dd. MN_NPI Accommodation TTP Proposed_MGA50
    ee. MN_Preferred Alternative Disturbance_MGA50
    ff. MN_Topsoil Storage Areas Construction_MGA50
    gg. MN_Topsoil Storage Areas_MGA50
    hh. MN_Turkeys Nest_MGA50
5. **Operational Pit and OSA/WRD Outlines**
    a. MN_Pits_Yan94.dxf
    b. MN_Waste_Dumps_Yan94

6. **Design Surfaces (0.5m grid dem exports from 12D)**
    a. NPI Ewks and Roads 240912.dem
    b. PIPELINE CORRIDOR SURFACE 240912.dem
    c. ALPHA STOCKPILE DIVERSION ROADS 240912.dem (replaces IO MAP LV YANDI ROAD.dxf)
    d. EFE OVERALL DESIGN 240912.dem (replaces IO MAP EFE EARTHWORKS.dxf)
    e. LV SME RD 240912.dem (replaces IO MAP LV SME ROAD.dxf)
    f. PIPELINE CORRIDOR SURFACE 240912.dem (replaces IO MAP PIPELINE CORRIDOR.dxf)
    g. RIO 220KV DIVERSION TRACKS.dem
    h. MARILLANA CREEK CROSSING 240912.dem (replaces IO MAP W5 XING.dxf)
    i. W5 CROSSING 240912.dem

The following design files were noted as superseded by BHP and therefore excluded from the latest modelling:

a. MN_PitDesignPPENorth_MGA50
b. MN_PitDesignPPESouth_MGA50
c. MN_OSADesignCentral_MGA50
d. MN_OSADesignWest_MGA50
e. MN_OSAOutlines_MGA50
f. MN_PitDesignAWTOnlyNorth_MGA50
g. MN_PitDesignAWTOnlySouth_MGA50
a. 15MTPA_DESIGN_COMBINED_EWKS_SHP_YAN94
b. EFE_Area
c. MARILLANA_CREEK_8m_MGA50 (Bridge)
d. MN_Borrow Areas_MGA50
e. MN_Stockpile BG SPS_MGA50
f. MN_Stockpile HG SPS_MGA50
g. MN_Haul Road 001 SPS_MGA50
h. MN_Haul Road 002 003 SPS_MGA50
i. MN_NPIPrecinct_LV_AccessRoads_MGA50
j. MN_PowerDesign_MGA50
k. MN_Access Tracks_MGA50 (polylines only – insufficient for modelling purposes)
h. MN_Stockpile BG SPS_MGA50
i. MN_Stockpile HG SPS_MGA50

A temporary construction stage surface was provided (ref: CONSTRUCTION FILL 240912.dem) but excluded from the final LoM model. Separate staging works were assessed as part of the road design scope of works by others.

The stockpile layers adjacent to the SV LME road and haul roads near the pits (ref: MN_New_Stockpiles_Yan94.dxf) were included in the developed Life of Mine (LoM) scenario but excluded from the closure scenario as outlined in Section 5.10.

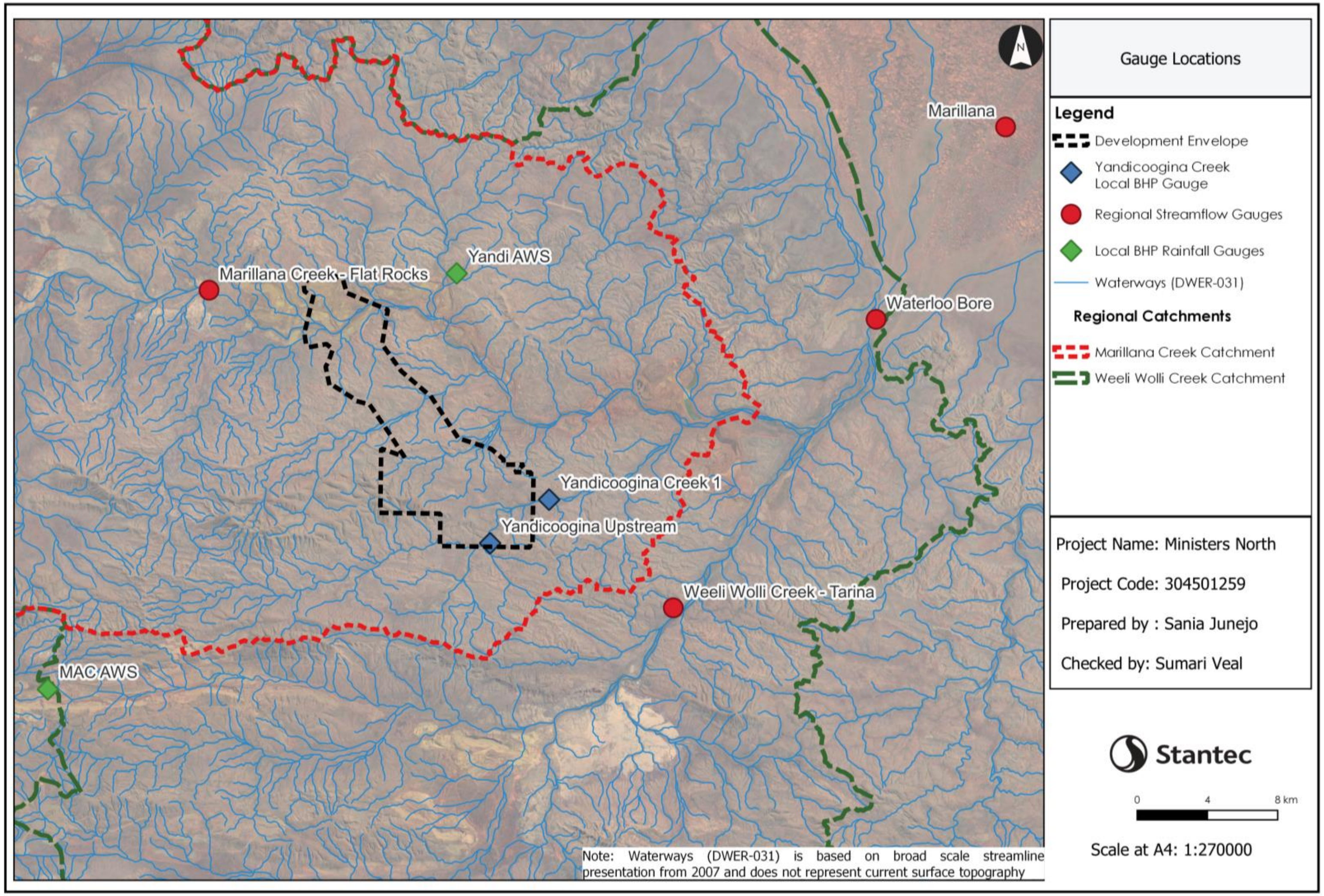
Shapefile and dxf data was received in MGA 94 Zone 50 in 2023, with the latest 2024 design updates being received in YAN94.

Additional culvert datasets were retained from previous work completed on behalf of BHP, Rio Tinto (RTIO) and Bechtel.

The existing BHP and RTIO railway lines were captured in the LiDAR survey and therefore form part of the baseline condition surface.

1.5 Gauge Data

There are three regional Department of Water and Environmental Regulation (DWER) and two local BHP stream gauges in proximity to the site as illustrated in Figure 1-2. There are two local BHP rainfall gauges that are also shown which supported streamflow validation.



Project Name: Ministers North

Project Code: 304501259

Prepared by : Sania Junejo

Checked by: Sumari Veal

Note: Waterways (DWER-031) is based on broad scale streamline presentation from 2007 and does not represent current surface topography

Figure 1-2 Gauge Locations

1.5.1 Regional Streamflow Gauge Data

The nearest regional Department of Water and Environmental Regulation (DWER) streamflow and rainfall gauges are outlined below:

1. **Marillana Creek - Flat Rocks**
    a. Located upstream of the proposed development in the Marillana Creek catchment
    b. Streamflow gauge (708001) with data range 1967-2024
    c. Rainfall gauge (505011) with data range 1972-2024
    d. Upstream catchment area: 1369 km^2
2. **Tarina**
    a. Located in the neighbouring catchment, Weeli Wolli Creek prior to the confluence with Marillana Creek
    b. Streamflow gauge (708014) with data range 1984-2024
    c. Rainfall gauge (505040) with data range 1984-2024
    d. Upstream catchment area: 1509 km^2
3. **Waterloo Bore**
    a. Located downstream of the proposed development and Weeli Wolli and Marillana Creek confluence
    b. Streamflow gauge (708013) with data range 1984-2024
    c. Rainfall gauge (505041) with data range 1984-2024
    d. Upstream catchment area: 3988 km^2

The annual total discharge and rainfall plots for the Flat Rocks, Tarina, and Waterloo bore gauges are illustrated in Figure 1-3, Figure 1-4 and Figure 1-5.

It has been noted in numerous historical studies of the Flat Rocks station that the catchment upstream exhibits hydrological characteristics inconsistent with regional trends, likely due to the Munjina Claypan. The Flat Rocks Flood Frequency Analysis undertaken by Advisian (2023) was incorporated in this study as validation of the inflows from Marillana Creek to the upstream portion of the development envelope.

The Waterloo Bore gauge is influenced by hydrological behaviour from both Weeli Wolli and Marillana Creek catchments and may not represent the local catchment behaviour.

The Environmental Protection Authority of Western Australia (EPA) previously commissioned calibration of the Tarina catchment 1987 Australian Rainfall and Runoff (ARR1987) RORB model using data from the Tarina streamflow gauge (708014) as part of the publicly available Baby Hope Hydrology and Hydraulic Assessment (2014).

Analysis of annual rainfall indicates that the Marillana Creek catchment is generally wetter than the Weeli Wolli Creek catchment, receiving on average 6% more rainfall on an annual basis. This suggests a spatial variation in rainfall, which is commonly experienced across the Pilbara.

As such, regional loss and routing parameter estimations determined from analysis of the Tarina gauge (EPA, 2014) have been considered more suitable for validation in this study as detailed in Appendix A but are not suitable for direct flow comparison.

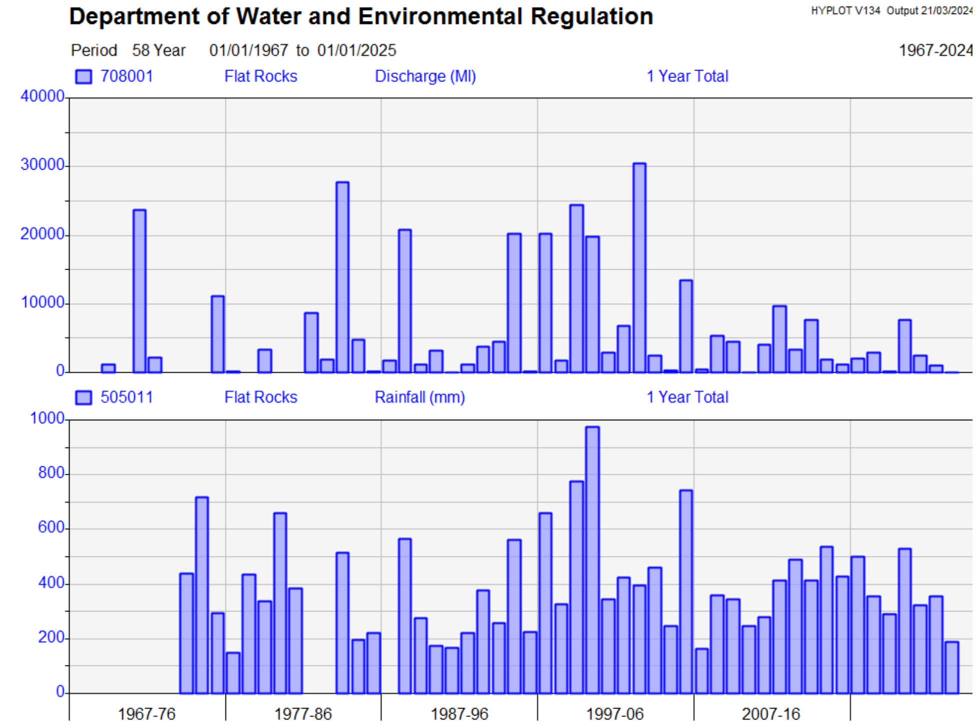


Figure 1-3 Flat Rocks Gauge Total Annual Rainfall (mm) and Discharge (ML)

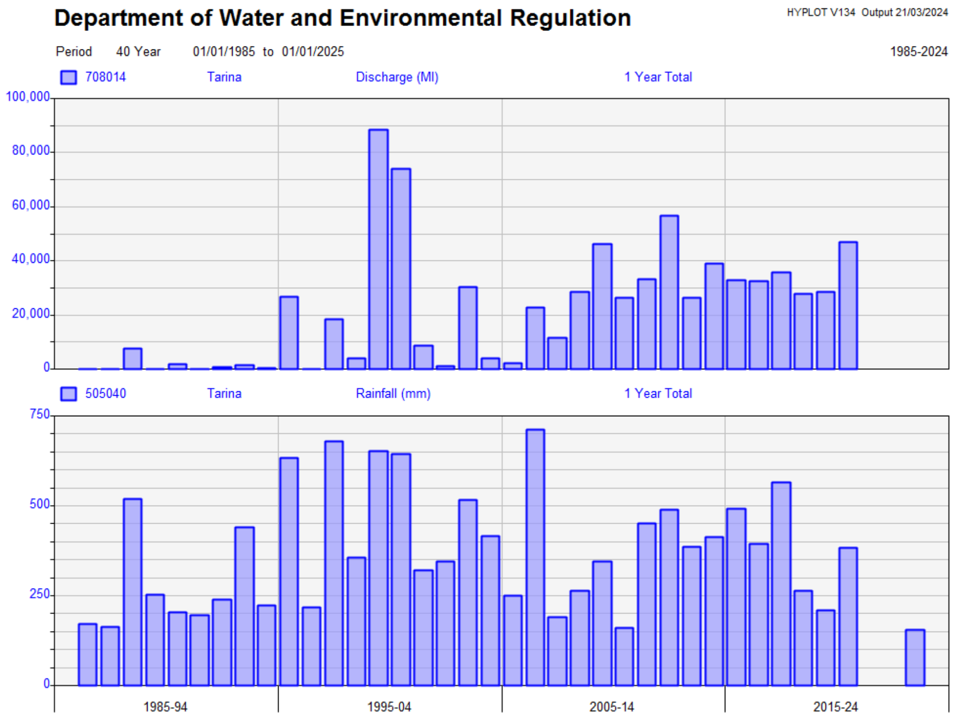


Figure 1-4 Tarina Gauge Total Annual Rainfall (mm) and Discharge (ML)

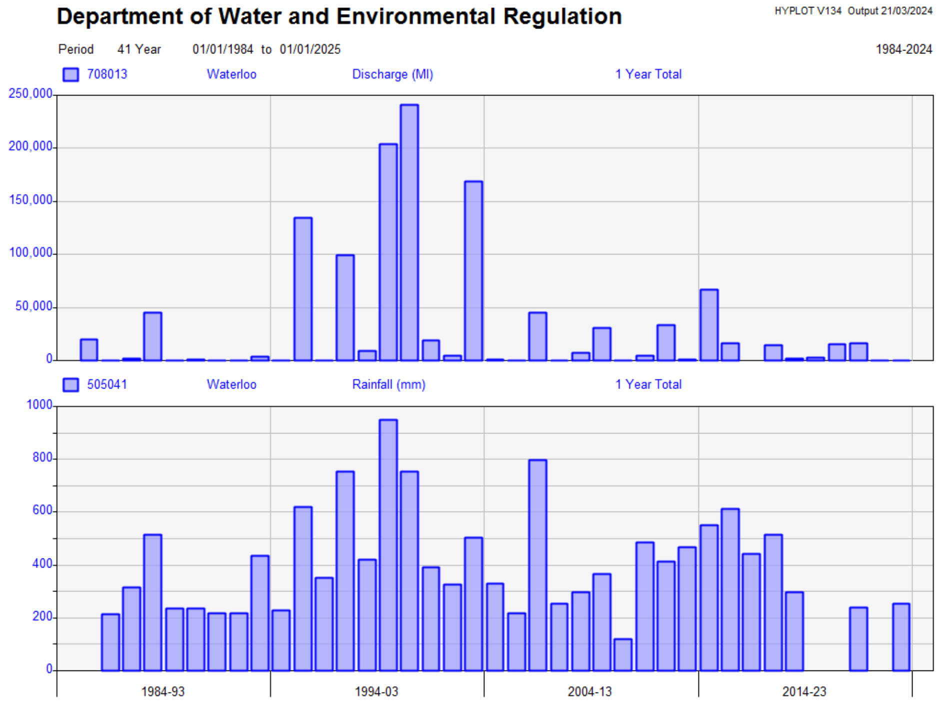


Figure 1-5 Waterloo Bore Gauge Total Annual Rainfall (mm) and Discharge (ML)

1.5.2 Local Rainfall Station Data

Daily rainfall from the Yandi Automatic Weather Station (MCWS001 Yandi AWS) and Mining Area C AWS (ACWS001) was supplied for January 2020 to June 2024, with 7 days of missing data in March 2024 at Yandi AWS.

Sub-daily rainfall was supplied from the beginning of July 2022 to end of January 2024. The maximum recorded hourly rainfall within the data range was 18.8 mm on the 25th of March 2023.

1.5.3 Local Streamflow and Temperature Gauge Data

The Yandicoogina Creek Upstream and Yandicoogina Creek 1 local gauge temperature and water level data were provided by BHP to enable validation of local parameters. Data ranges are outlined below:

- **Yandicoogina Creek Upstream (YC US)**
  - water levels and temperature data from December 2020 to start of June 2024.
- **Yandicoogina Creek 1 (YC 1)**
  - water level data from December 2020 to April 2024
  - temperature data from January 2024 to February 2024

The water levels are higher at the Yandicoogina 1 gauge which is indicative of the ponding and flow attenuation occurring at that location. The readings at Yandicoogina 1 (YC 1) gauge have been included for context and a point of comparison, however, the focus of this assessment is the Yandicoogina Upstream (YC US) gauge readings. The results of this analysis have been incorporated into the loss estimates for the local catchments as the Flavell (2012) initial loss assumption of 40 mm results in zero streamflow in frequent events, which is clearly not supported by the findings at the local gauge. Non-neutral initial losses of between 22 mm for the 63.2% AEP to 57 mm for the 5, 2 and 1% AEP, with fixed 6 mm/h continuing losses were adopted as detailed in Appendix A.

An approximate 72 to 96-hour event with 96 mm of rainfall was observed on the 9th to 12th of December 2020 with only one dry day between the previous ~22 mm rainfall event on the 6th and 7th, and ~68 mm on the 2nd to 4th. Correlation with the BoM IFDs determined this event may have corresponded to between a 50% AEP and 0.5EY event. Only air pressure was measured at the Yandicoogina 1 gauge for this date and no records are available at the Yandicoogina Upstream gauge.

There were 1.3 m and 0.9 m maximum water levels recorded on the 12th of February 2021 at Yandicoogina 1 and Yandicoogina Upstream gauges.

This correlated to 64 mm in 48 to 72 hours for a likely 2EY to 1EY event, proceeded by a potential 0.5EY to 20% AEP event on the 31st of January to 6th February 2021.

The 8th of February 2022 experienced 1.5 m in maximum water level at Yandicoogina Upstream and 2.4 m at Yandicoogina 1 after only ~19 mm of rainfall for a 72-hour event between the 6th to 8th, proceeded by 9 dry days. Similarly, on the 30th of March 2022 the gauge recorded 0.8 m in maximum water level from only 24 mm in rain within 48 hours.

These events likely correlate to a 6EY to 4EY and 4EY to 3EY event comparative to BoM IFDs, respectively. These high water levels do not appear to correlate directly to the supplied rainfall as clearly as the January 2024 events and likely more reflective of a 10% AEP water level for an approximate 200m^3/s flow rate.

An approximate 24 to 48-hour event with 77 mm of rainfall was observed on the 3rd to 4th of September 2022. Correlation with the BoM IFDs determined this event may have corresponded to between a 1EY and 0.5EY event. The associated water level reading for this event at Yandicoogina Upstream was only 0.07m with only an air pressure reading at Yandicoogina 1. This highlights, whilst this event was initially assumed to have been more readily utilised to validate the hydraulic model levels, the rainfall must not have been occurring directly at the gauged catchment and therefore provides no validation value.

There was a further total of 23 mm over a period of 8 hours on the 23rd February 2023 for which no water level reading is available at either gauge. This indicates that a 3EY to 2EY event proceeded by previous wet days can occur with no material runoff or the rainfall must not have been occurring directly at the gauged catchment and therefore provides no validation value.

At 6pm on the 14th of January 2024, the highest hourly rainfall of 11 mm was observed for any month within the sub-daily data range provided with water levels at a water level of 0.08 m. This instantaneous one-hour (or lesser) event was preceded by 5 virtually dry days. Comparison of the rainfall depth to the Bureau of Meteorology (BoM) Intensity Frequency Duration (IFD) data indicates this rainfall event may have corresponded to between a 4EY to 3EY event.

An approximate 12-hour event with 43 mm of rainfall was observed on the 24th to 25th of January 2024 with 7 dry days between the previous ~3 mm rainfall event on the 17th, ~3 mm on the 16th and ~11 mm on the 14th. Correlation with the BoM IFDs determined this event may have corresponded to between a 1EY and 2EY event.

The events on the 12th of February 2021, 14th of January 2024 and 24th January 2024 are of significance as all events resulted in increased stream level readings correlating to a rainfall event.

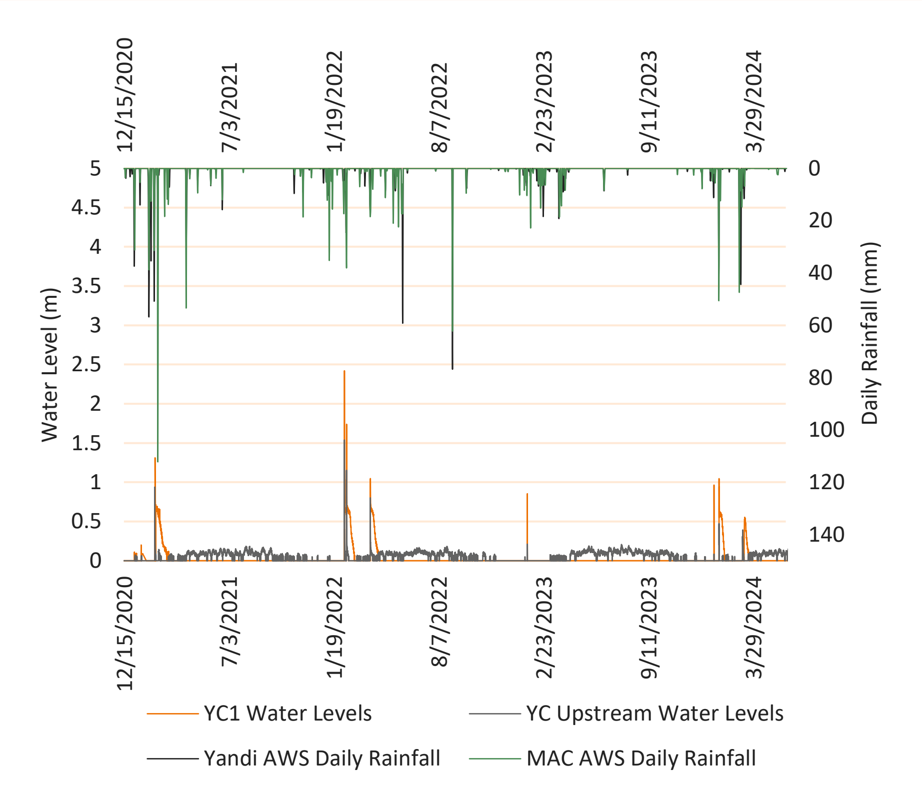


Figure 1-6 Water Level and Rainfall Readings at Local Gauges

The reduced temperature readings validate the presence of stream flow at the Yandicoogina 1 and Yandicoogina Upstream gauge locations, such as the extract in Figure 1-8 showing the temperature reduction for the events recorded on the 14th of January 2024 and 24th January 2024.

It is noted that the low-level noise in the Yandicoogina Upstream gauge readings is not stream flow and is only a reflection of the need for barometric compensation in the device.

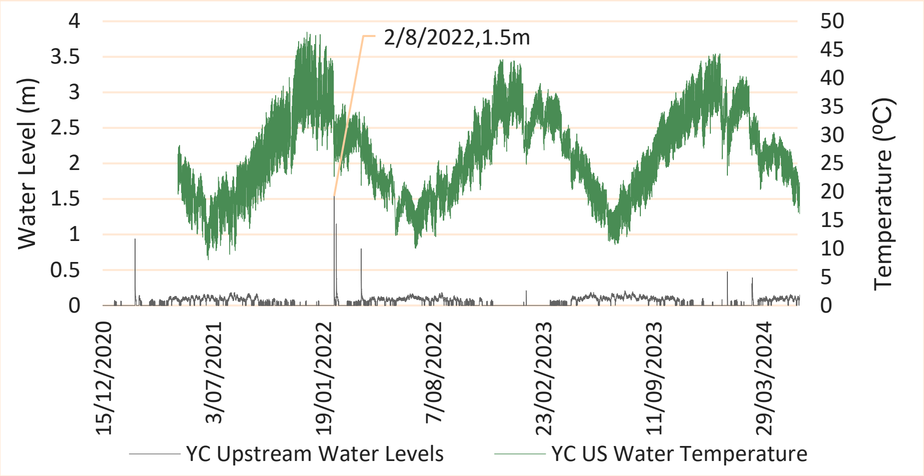


Figure 1-7 Water Level and Temperature Readings at Yandicoogina Upstream

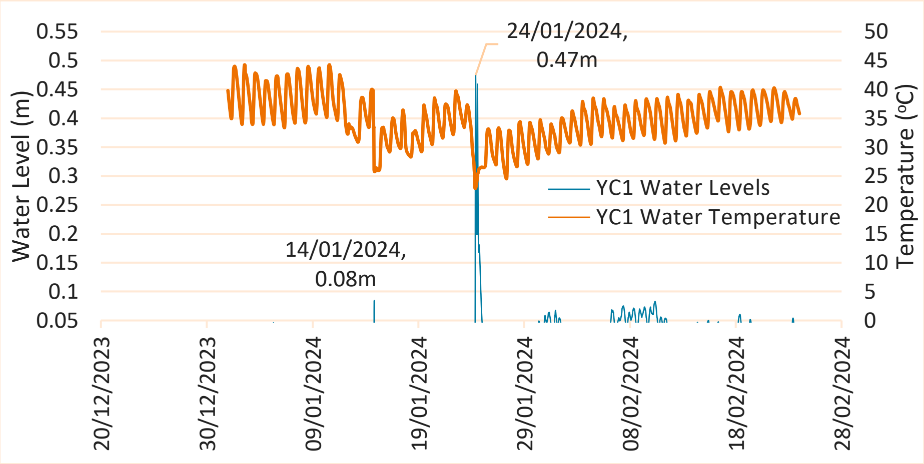


Figure 1-8 Water Level and Temperature Readings at Yandicoogina 1

2 Project Background

Several background studies have been reviewed as part of this project. The applicable conclusions from key studies have been summarised below.

2.1 Marillana Creek Baseline Hydrology (Advisian, 2023)

Advisian undertook flow estimates for Marillana Creek to support the transition from Yandi Operations to South Flank operations in accordance with regulatory approval for the Marillana Creek (Yandi) Mining Operations, outlined in Ministerial Statement 1039 (MS 1039), BHP "shall ensure that the proposal is decommissioned and rehabilitated in an ecologically sustainable manner".

This supersedes the previous assessment of design flows at the Yandi mine by GHD in 2012 and 2014 due to changes in Australian Rainfall and Runoff guidance and design rainfall estimates. A rainfall-runoff model (RORB) was developed for the entire Marillana Creek catchment to the downstream extent of the BHP mine lease at the BHP rail crossing. Reach-storage parameter validation was undertaken to 2D hydraulic modelling and derived Flood Frequency Analysis quantiles at Flat Rocks to inform empirical loss estimates.

Peak flows at Flat Rocks were comparable to those in the previous study (GHD, 2014) for events up to the 1% AEP event due to the similar FFA quantiles derived between the two studies at this reference location. Minor differences between studies were noted at the lower end of the mine lease for more frequent events.

Peak 1 in 10,000 AEP design flows at Flat Rocks were predicted to be marginally higher than the previous study at 7,244 m^3/s (a 2% increase on the previous estimate). The 1 in 10,000 AEP inflow rate downstream of the Lamb Creek confluence was adopted, with additional updates to the climate change uplift outlined in Section 3.5 for this study.

2.2 Previous Ministers North Hydrological Studies

Three previous surface water assessments have been undertaken specifically for the Ministers North Trucking Project. This includes:

1. Ministers North Surface Water Environmental Impact Assessment, Stantec 2018
2. Ministers North OLC Hydraulic Modelling Report Preliminary, Surface Water Solutions, May 2019
3. WAIO Project - Yandi Ministers North Trucking SPS Hydrology Investigations Report, Surface Water Solutions 2023

Additional details from these previous studies relevant to this project are summarised below.

2.2.1 Ministers North Surface Water Environmental Impact Assessment (Stantec, 2018)

The 2018 Surface Water Assessment was limited to the Indicative Development Envelope and included three overburden storage areas (OSAs) and the Ministers North mining pits in the southern section of the overall development envelope as illustrated in Figure 2-1.

Two catchment areas, Yandicoogina Creek and a validation catchment (comparable to Mungadoo Tributary catchment for this study) were modelled in RORB. A TUFLOW hydraulic model was undertaken for the pit and OSA areas only.

The key differences between the 2018 assessment and the current 2024 study include:

1. The previously proposed West Pit, East Pit and OSA East are no longer being considered in the 2024 assessment.
2. The 2018 assessment adopted 40 mm initial and 5 mm/hr continuing losses in RORB based on Flavell (2012) as opposed to the 2024 updated non-neutral losses with an upper initial loss limit of 57 mm and continuing loss of 9 mm/hr based on local gauge analysis.
3. HEC-RAS 2D was used for the hydraulic analysis following the 2018 Stantec TUFLOW scope of modelling which was only for a small model extent focussed on the southern pits and OSAs.

4. Comprehensive sensitivity analysis was conducted for the 2024 study.

Both 2018 and 2024 studies adopted Pearcey et. al (2014) k_c RORB assumptions.

The peak flow rates determined at the outlets to the Yandicoogina Creek and validation catchment (comparable to Mungadoo Tributary catchment for this study) are outlined in Table 2-1.

Table 2-1 Peak rainfall and discharge rates for Ministers North catchments (2018)

AEP	Ministers North (Validation)			Yandicoogina Creek		
	Critical duration (hr)	Rainfall depth (mm)	Peak flow (m^3/s)	Critical duration (hr)	Rainfall depth (mm)	Peak flow (m^3/s)
50%	24	72	**2**	24	72	**5**
20%	6	71	**32**	12	92	**90**
10%	6	89	**56**	12	116	**223**
5%	6	107	**86**	12	143	**321**
2%	6	133	**108**	12	181	**453**
1%	6	155	**141**	24	271	**574**
1:1,000	2	142	**341**	6	235	**1,540**
1:10,000	2	208	**553**	6	340	**3,076**

The updated 2024 study in this report presents an increase in flows from those reported above in 2018.

This was due to more comprehensive sensitivity analysis leading to the adoption of non-neutral losses, local gauge information providing evidence of frequent event flow occurrences and validation to six regional methods.

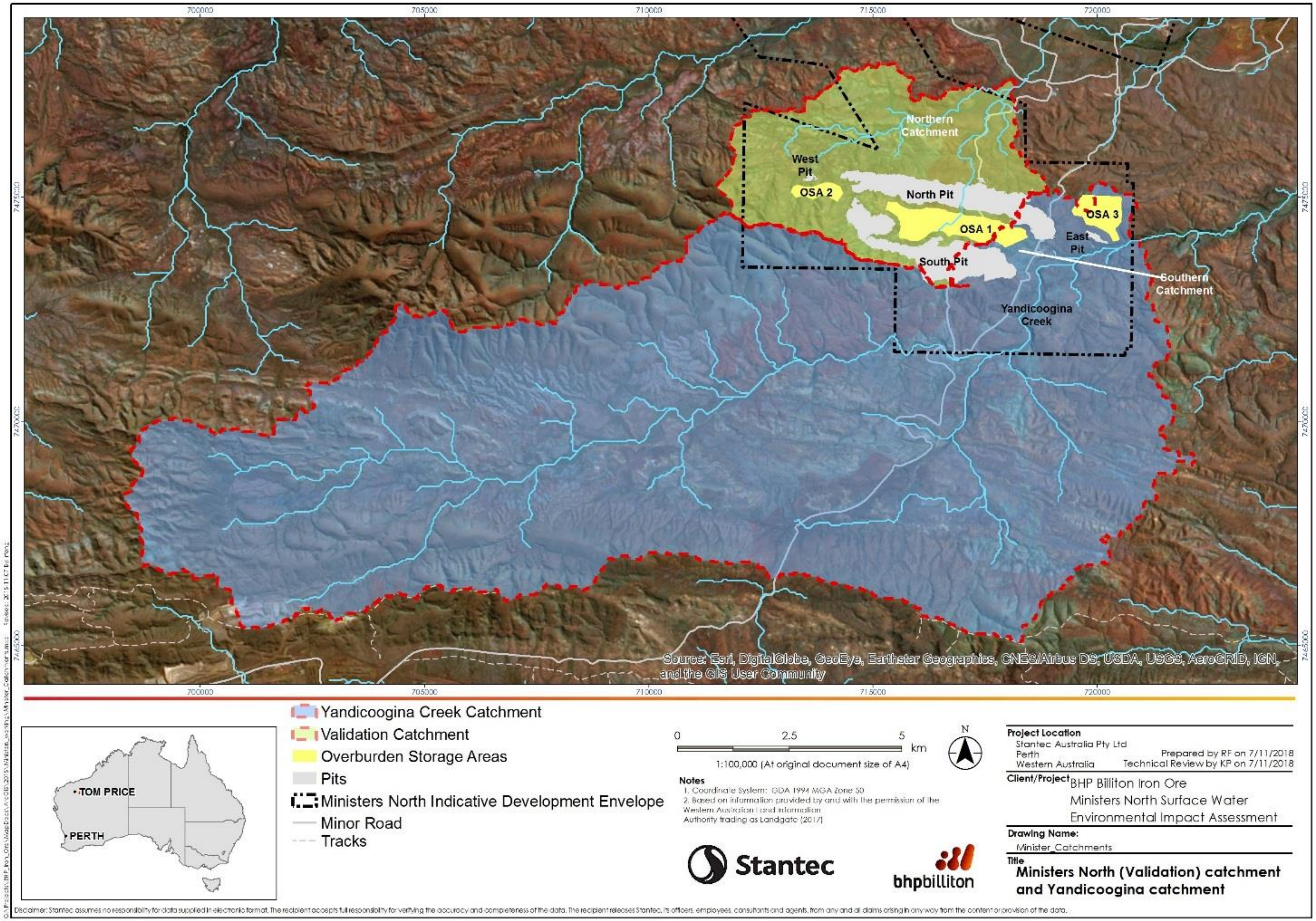


Figure 2-1: Ministers North Validation Northern Catchment* and Yandicoogina Catchment (Stantec, 2018)

*Mungadoo Tributary catchment for this study

2.2.2 Yandi Ministers North Trucking SPS Hydrology Investigations, (SWS, 2019-2023)

Surface Water Solutions (SWS) undertook two hydrological and hydraulic investigations for the Yandi Ministers North Trucking Project in 2019 and 2023 to support culvert sizing to meet roadway serviceability and scour recommendations.

Marillana Creek inflows were adopted as per the GHD (2014) Yandi Flood Study, which included calibrated loss rates that accounted for the upstream claypan as well as other unapplicable factors. The local catchment areas considered as part of the assessment are illustrated in Figure 2-2. This excluded consideration of Yandicoogina Creek flows as these were determined not to contribute to the objectives of the project.

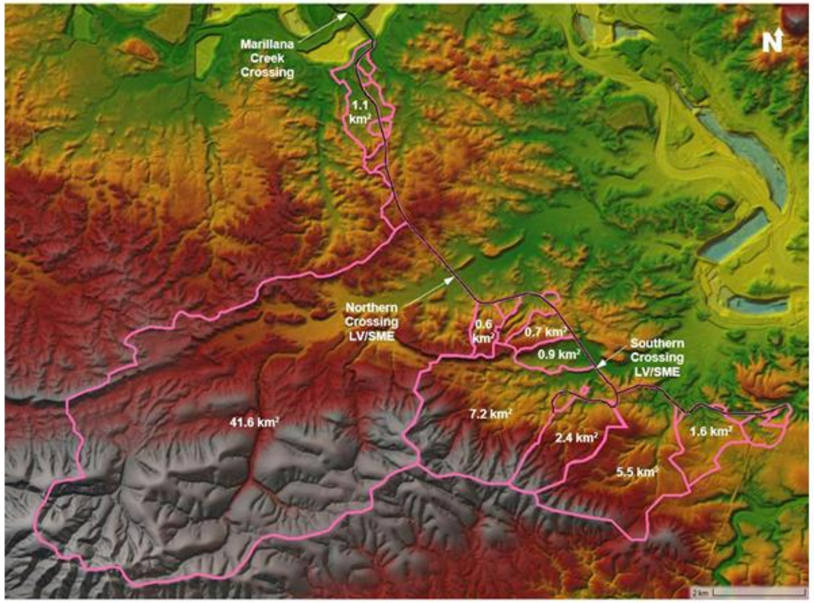


Figure 2-2 Contributing Catchment Areas (SWS, 2023)

For the Northern Crossing Catchment (comparable to the Central Tributary to Marillana Creek Catchment in this study), the peak flows adopted from the 2018 Stantec analysis were included as external inflow boundary conditions with the remaining catchments being modelled using rain-on-grid method.

Peak flows were assessed in 2019 based on a wide range of previous assessments such as RORB, rain-on-grid modelling tools such as TUFLOW, MIKE and HEC-RAS and the Regional Flood Frequency Estimation Model (RFFE, 2019).

The 2023 SWS assessment utilised HEC-RAS to create a two-dimensional rain-on-grid hydraulic model to analyse flood levels in the 20%, 10%, 5%, and 1% AEP events. IFD data was used to create centrally loaded, nested frequency storms used as precipitation inflow boundary conditions for a 6-hour synthetic storm with peak rainfall intensity occurring 3 hours after the storm. This was based on the time it took the largest catchment runoff to reach the proposed infrastructure upgrades.

An initial loss of 40 mm and a continuing loss rate of 5 mm/hr was applied to the catchments for consistency with the 2018 Stantec assessment.

Table 2-2 summarises the model parameters used for the HEC-RAS rain-on-grid model.

Table 2-2 Summary of model input parameters (2023)

Model Parameter	Value
DEM Resolution	0.5m to 5m
Inflow	2014 GHD Values for Marillana Creek (Figure 3-1) 2018 RORB Values for Northern Catchment* (Figure 2-1)
Precipitation	Nested 6-hour Frequency Storm
Outflow Boundary Condition	0.3% - 1% normal depth energy gradient
Computational Time Step	0.5 – 5 seconds
Simulation Window	8 hours
Computational Mesh Size	5-20 metres
Catchment Roughness	0.12
Channel Roughness	0.04
Culvert Roughness	0.015
Equation Set	Full momentum shallow water equation

*The Northern Catchment referenced in the 2018 study has been renamed as Mungadoo Tributary catchment for the 2024 study.

The results showed overtopping occurred in the 1% AEP between chainages 5000 to 5250. This is similarly demonstrated in this study report and considered acceptable due to the large number and size of culverts required to provide 1% AEP dry serviceability.

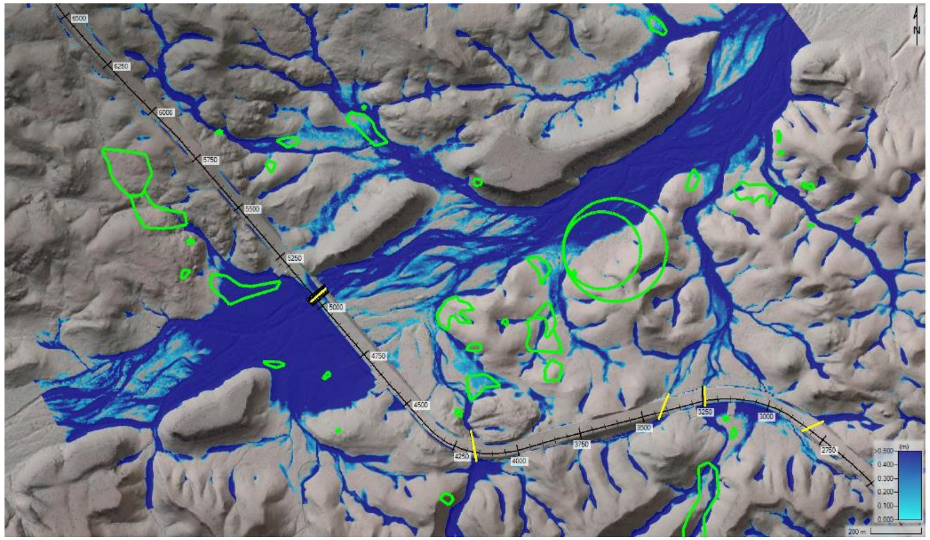


Figure 2-3 1% AEP Flood Depths at Centre of Study Area from Yandi Ministers North Trucking SPS Hydrology Investigations (SWS, 2023)

3 Hydrological Analysis

3.1 Approach

The overall hydrological approach for the study is summarised below:

1. The Marillana Creek catchment was not hydrologically modelled. The peak discharge rates for Marillana Creek have been adopted from the Marillana Creek Baseline Hydrology Report (Advisian, 2023). A hydrological model was not required for this assessment as the Advisian model provides relatively recent, calibrated data that is transferable to this assessment.

2. Hydrological analysis for the local catchments impacting the development envelope was undertaken using RORB Model Version 6.45. Ensemble analysis of all ten temporal patterns for each duration between 10 minutes to 72 hours was used to determine the critical duration and associated median temporal pattern for design event selection.

3. As discussed in Section 1.5, there are three regional gauges and two local gauges that have been reviewed for this assessment. The regional gauges were deemed inappropriate for transposition to local analysis, however, the flows from the Marillana Creek are incorporated based on the Flat Rocks gauge FFA (Advisian, 2023).

4. Comprehensive hydrological sensitivity analysis was undertaken of model parameters to ensure defensible hydraulic model validation and inflows. This is outlined in Appendix A.

5. Non-neutral losses were adopted for local catchments with initial loss values being scaled from 22 mm for the 63.2% AEP (1 EY) Event through to 57 mm for the 5% AEP using the ratio of consecutive IFD rainfall depths. The static initial loss of 57 mm was adopted for the 5% to 1% AEP events.

6. A continuing loss of 6 mm/h was adopted based on the outcomes of the local BHP rainfall and stream level gauge downstream of the Yandicoogina Creek inflow to the development site.

7. The k_c estimate for each sub-catchment was adopted based on the Pearcey et al (2014) method with a k_c/d_{av} ratio of 0.59. This was validated against calibration undertaken at the Tarina gauge (EPA, 2014).

8. Validation of hydrological flows at local catchment outlets has been undertaken using six regional methods.

9. The intention of the hydrological modelling is to provide inflows to the HEC-RAS model for large upstream areas (Central Tributary) and excess rainfall hyetographs for local representative rain-on-grid catchments. This matches the approach of the 2023 SWS study.

3.2 Catchment Delineation

There are four catchments that transect the Development Envelope (Figure 3-1). The largest watercourse crossing the proposed trucking alignment is Marillana Creek, with a contributing catchment area of approximately 2,000 km^2. The Development Envelope is completely encompassed by the Weeli Wolli catchment.

Three local catchments were delineated to determine inflows to the development footprint and validate hydraulic results within the development footprint. The local Yandicoogina Creek, Mungadoo Tributary and Central Tributary catchments are illustrated in Figure 3-1 in relation to the larger Marillana Creek and Weeli Wolli Creek catchments.

A summary of the catchment sizes and characteristics is outlined in Table 3-1. It is noted that the Marillana Creek catchment is a sub-catchment of the Weeli Wolli Catchment.

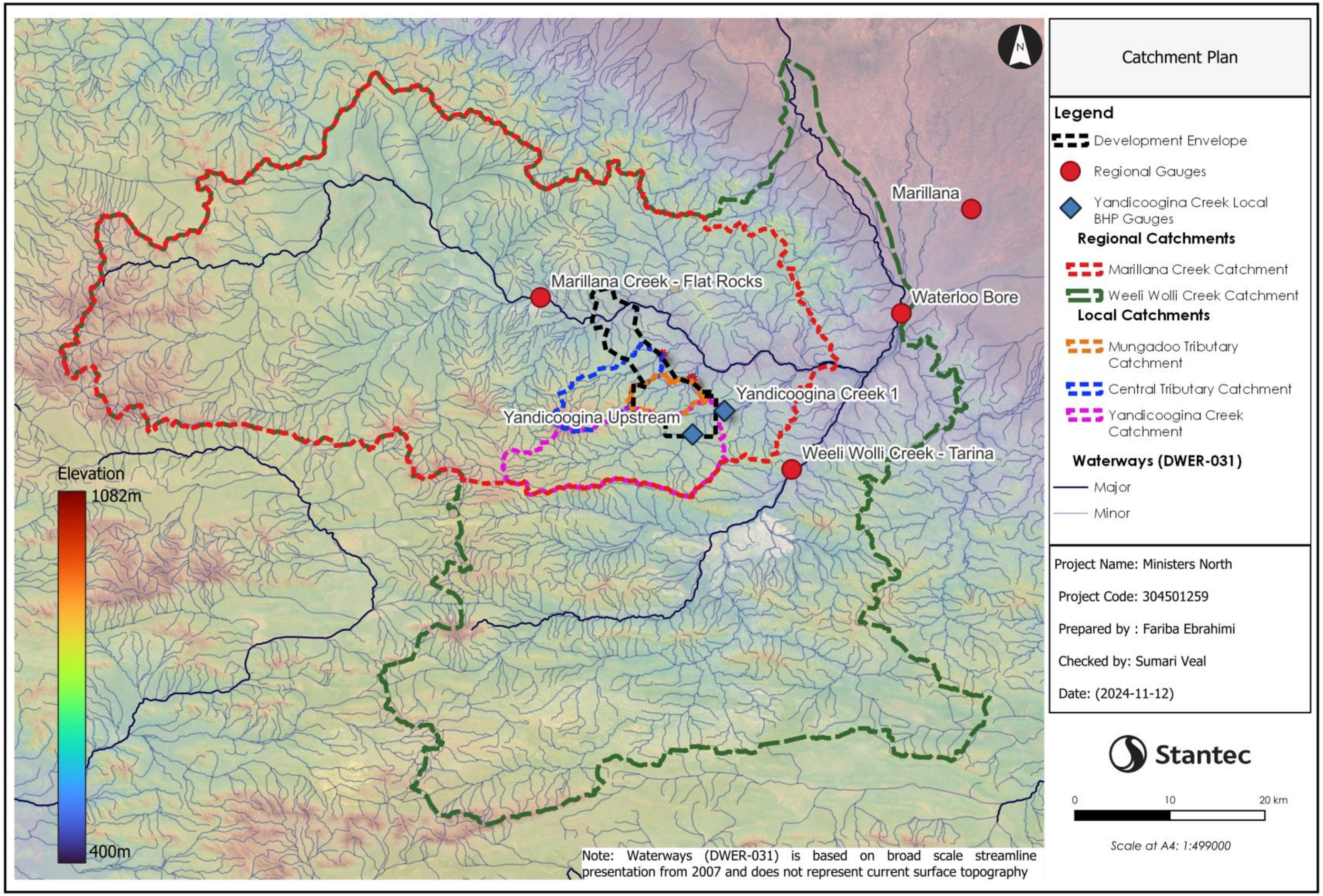
Due to the shorter flow paths and concentration times, peak flows from minor tributaries that drain across the Ministers North site tend to reach peak flows prior to the arrival of the peak flow from Yandicoogina and Marillana Creek and larger tributaries such as the Central Tributary catchment defined in this study.

Pits are located high in the catchment and do not receive runoff from a large upstream catchment. The Development Envelope crosses into the Yandicoogina Creek floodplain but the primary flow path is not impacted by the developed area.

Catchments have not been explicitly modelled in RORB to account for the location of pits and OSAs. Pit and OSA locations do not impact the catchments upstream where the inflows are being applied from the hydrological model. The hydrological inflows at these upstream locations for the LoM and Closure models will be the same as the Baseline. The rain-on-grid model is designed to account for the location of the pits and OSAs.

Several proposed culverts in the Ministers North development boundary are downstream of small single subarea catchments in the LoM condition. These catchments have also been split in the Baseline condition by the proposed road alignment to avoid additional unnecessary hydrological runs.

The subareas within each local catchment, relative to the proposed mine infrastructure, is illustrated in Figure 3-2.



Note: Waterways (DWER-031) is based on broad scale streamline presentation from 2007 and does not represent current surface topography

This document has been prepared based on information provided by others as cited in the data sources. Stantec has not verified the accuracy and/or completeness of this information as shall not be help responsible for any errors or omissions which may be incorporated herein as a result. Stantec assumes no responsibility for data supplied in electronic format, and recipient accepts full responsibility for verifying the accuracy and completeness of the data.

Figure 3-1 Regional Catchment Plans in Relation to Development Envelope

Table 3-1 Catchment Overview

Catchment	Area (km^2)	Average Subarea (km^2)	Elevation Range (m AHD)	Reach* Slope Length (km)	Slope (%)	Critical Duration
Marillana Creek	2,228	Not delineated	485-1170	~80	~1	24-36 hours
Central Tributary	50	1.6	568-1080	10 for reach 0.9 for subarea average	0.8 for reach 2-26 for subarea (average 5%)	4.5-12 hours outlet 2-4.5 hours subarea
Mungadoo Tributary	24	1.1	550-940	5 for reach 0.6 for subarea average	0.7 for reach 2-40 for subarea (average 5%)	4.5-12 hours outlet 2-4.5 hours subarea
Yandicoogina Creek	150	6.2	563-1060	20 for reach 1.4 for subarea average	1 for reach 1-9 for subarea (average 2%)	6-12 hours outlet 2-6 hours subarea
Weeli Wolli Creek	4,150	Not delineated	495-1160	~60 (upstream of Marillana confluence)	~1 (upstream of Marillana confluence)	12-24 hours (upstream of Marillana confluence)

*Reach length for the main flow path is taken from the upstream subarea within each catchment

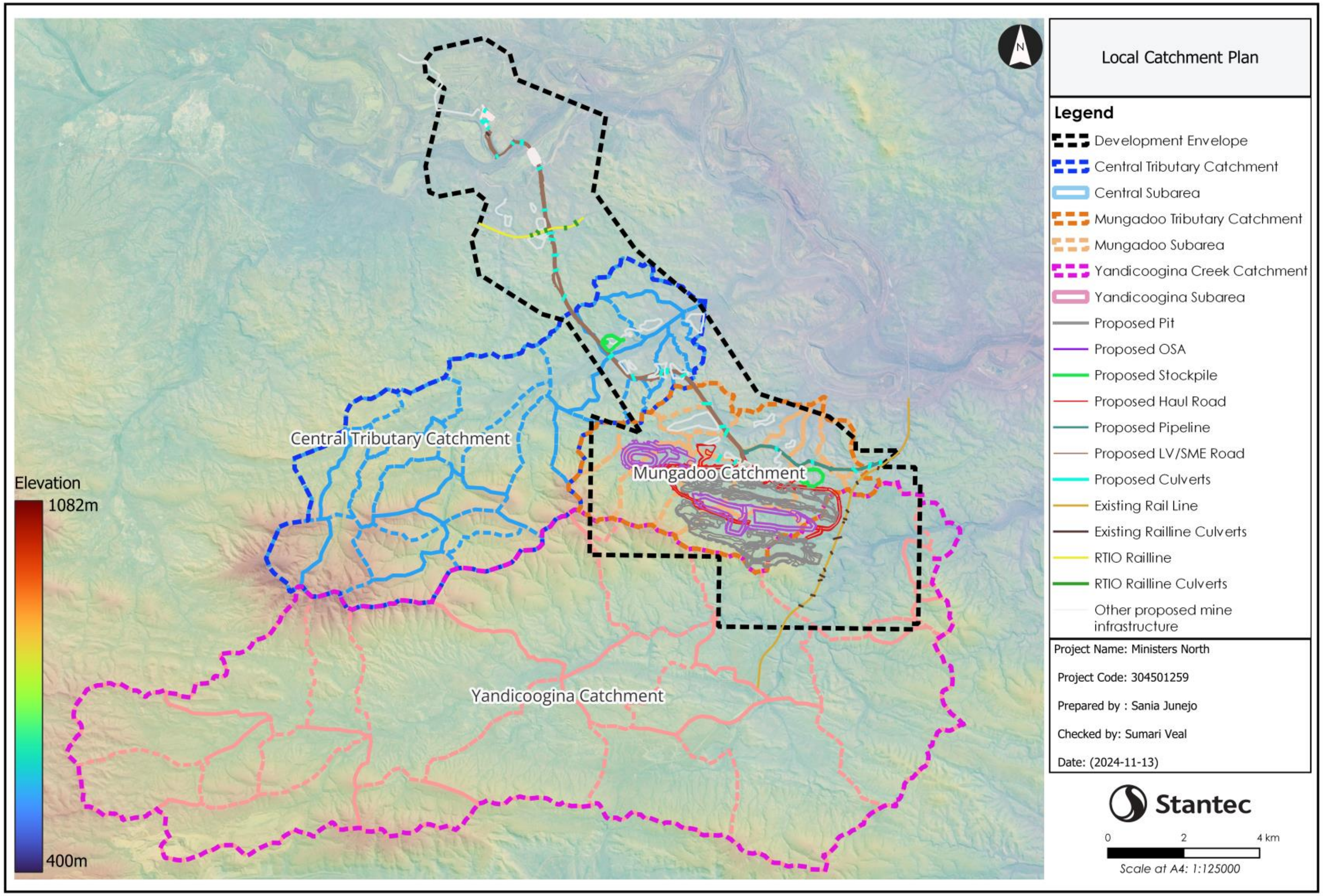


Figure 3-2 Local Catchment Subareas in Relation to Proposed Development

3.3 Design Rainfall Data Sources

The design rainfall data sources for the various local modelled scenarios are summarised in Table 3-2.

The IFD data, associated temporal patterns and losses were downloaded from the ARR2019 QGIS ARR plugin tool and validated against the ARR data hub for the specified coordinates (-22.8125, 119.1125).

Table 3-2: Source of Design Rainfalls for Ministers North

Storm Duration (hr)	Event		
	50% AEP to 1 in 1,000 ARI	1 in 10,000 ARI	PMP
< 1	2016 Bureau of Meteorology IFD data		
1		Estimated from extrapolation procedure outline in ARR 2019. Climate change uplift applied in line with draft updated guidance (Engineers Australia & DCCEEW, 2023)	The Generalised Short Duration Method (GSDM)
2			
3			
6			
12			Extrapolated
24			Revised Generalised Tropical Storm Method (GTSMR)
48			
72			

3.4 Key Parameter Selection

Details of the parameter selection for the hydrological modelling, including comprehensive sensitivity analysis, are outlined in Appendix B. A summary of the adopted parameters is outlined in Table 3-3.

Table 3-3 Summary of model input parameters

Model Parameter	Value		
	Yandicoogina Creek	Central Tributary	Mungadoo Tributary
k_c	8.51 (Pearcey Method)	4.94 (Pearcey Method)	2.66 (Pearcey Method)
Initial Loss	Between 22 mm for the 63.2% AEP to 57 mm for the 5, 2 and 1% AEP		
Continuing Loss	6 mm/h		
Temporal Patterns	Point	Point	Point
Areal Reduction Factors	Northern Coastal ARF ARR data hub values based on total catchment areas		
Preburst	Median as per ARR data hub		
Filtered Embedded Bursts	Filtered through built-in RORB tool		
Simulation	Ensemble		

3.5 Extreme Event Estimates

The extreme event analysis has been undertaken to determine the Probable Maximum Precipitation (PMP) event and thereby interpolate to the 1 in 10,000 AEP event.

The Generalised Short Duration Method (GSDM) (BoM, 2003) was used to estimate PMP values up to 6 hours for use in the interpolation. As Yandicoogina Creek catchment exceeds 75km², the spatial distribution of short duration rainfall depths was developed by overlaying and positioning consecutive ellipses on the Yandicoogina Creek catchment plan such that the best fit by the smallest possible ellipse was obtained. No spatial variation was considered for the Central and Mungadoo Tributary.

The design temporal distribution was adopted in line with Table 1 of *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method* (BoM, 2003).

Durations between 12 to 120 hours were estimated using the Generalised Tropical Storm Method Revised Method (GSTMR) (BoM, 2005) for the Coastal Zone. An example of the Yandicoogina Creek PMP depth envelope plot is illustrated in Figure 3-3.

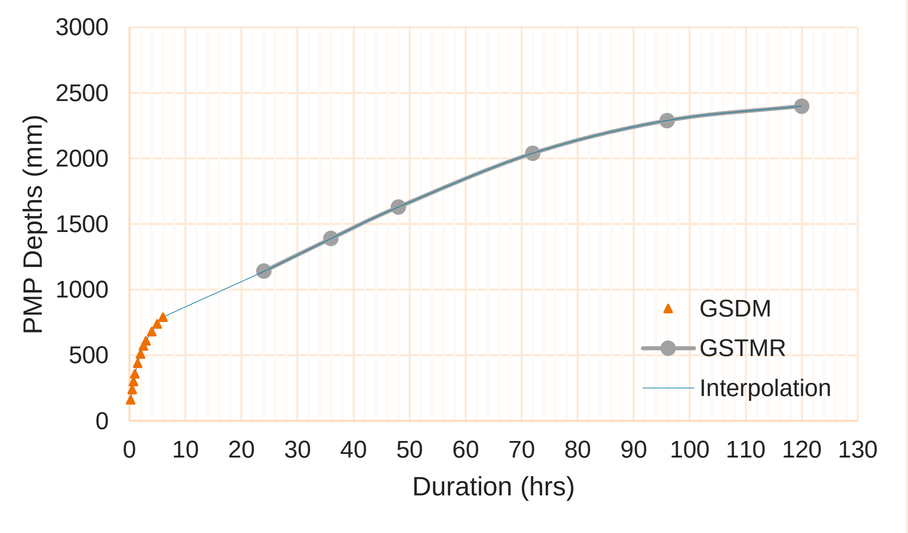


Figure 3-3 Final Permissible Maximum Precipitation Envelope for Yandicoogina Creek

A frequency curve was created using a log-normal scale through the derived rainfall totals (i.e. 1 in 200 AEP, 1 in 500 AEP, 1 in 1,000 AEP, 1 in 2,000 AEP and the PMP). The 1 in 10,000 AEP rainfall depth was visually determined from the resulting curve.

3.6 Climate Change Uplift

The 1 in 10,000 AEP closure conditions analysis includes a Climate Change uplift consideration and was adopted as per the *Draft Updates to the ARR2019 Climate Change Guidelines* (Engineers Australia and DCCEEW, 2024). BHP confirmed the adoption of the Shared Socioeconomic Pathways SSP2 (RCP4.5) emissions scenario, which corresponded to a 2.7°C increase by the end of the century (2090), relative to the baseline adopted for the 2016 IFDs. This correlated to a 14.1% increase in rainfall depths for the catchments according to Equation 1 (ARR2019) and included no changes to loss parameters.

This was the best available information at the commencement of the project and remains appropriate the scope of this assessment. Sensitivity analysis was conducted in September 2024 following the release of the August 2024 updated guidance in Section 4.

3.7 Model Schematisation

Three hydrological models for the two local Marillana Creek tributary catchments (Central and Mungadoo) and Yandicoogina Creek catchment were developed in RORB in accordance with Australian Rainfall and Runoff (ARR2019).

These models were built to provide inflow hydrographs to the HEC-RAS 2D hydraulic model and provide validation of hydraulic parameters such as manning's roughness. The model schematics for Yandicoogina Creek, Mungadoo Tributary and Central Tributary Catchments are illustrated in Figure 3-4.

The RORB models designate nodes at subarea centroids and connect via reaches to junctions on the main flow path reach. This is to ensure no duplication of routing is incurred with potential dampening of runoff impacts.

Reach type 1 has been applied for natural waterways where the slope is less than 2%. In areas of steeper grade than 2%, Reach type 2 has been adopted to better simulate the more intense runoff response. This is in keeping with best practice industry standards and consideration for the limitations of RORB where reach slopes of greater than 2% are not considered to be within the original development intent of the model.

Subarea sizes range from 0.01 to 16 km^2 based on required print out locations and overall catchment size. All catchments were adopted as pervious areas.

A summary of the catchment data was outlined in Section 3.2.

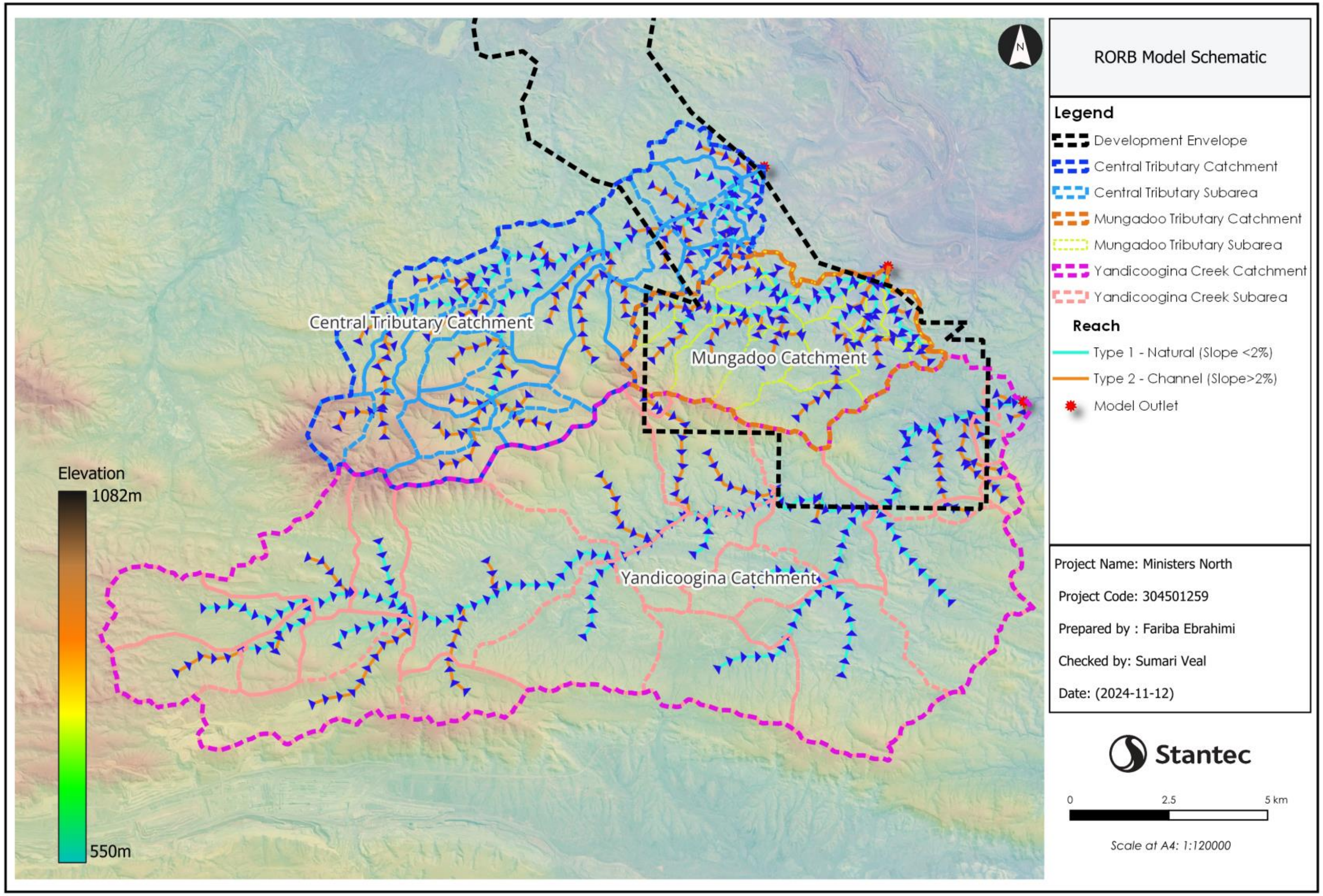


Figure 3-4 Local RORB Model Schematics

3.8 Peak Flow Summary

3.8.1 Baseline Climate Scenario

The resulting baseline peak flow rates at the outlet of each of the catchments are outlined in Table 3-4. The critical duration and median temporal pattern (TP) for the design peak flow are stipulated in brackets. The ARR2019 data hub includes a set of ten possible design temporal patterns which define the timeseries over which a proportion of the rainfall event is distributed. The temporal patterns are consistent between the 50% to 20%, 10% to 5% and 2% to 1% AEP events. There is only one available temporal pattern for each of the extreme rainfall event durations.

Table 3-4 Hydrological Peak Flow Summary at Outlet

AEP	Peak Flow (m^3/s) (critical duration, median temporal pattern)		
	Central Tributary	Mungadoo Tributary	Yandicoogina Creek
Area (km^2)	50	24	150
50%	25 (12 hour, 6TP)	22 (4.5 hour, 8TP/12 hour, 6TP)	36 (12 hour, 6TP)
20%	70 (12 hour, 5TP)	52 (12 hour, 6TP)	115 (12 hour, 6TP)
10%	102 (6 hour, 18TP)	73 (12 hour, 17TP)	184 (12 hour, 16TP)
5%	141 (6 hour, 18TP)	99 (12 hour 14TP)	280 (12 hour, 16TP)
1%	392 (4.5 hour, 29TP)	275 (6 hour, 27TP)	830 (6 hour, 30TP)
1 in 10 000	1622 (2 hour)	1120 (1 hour)	3570 (3 hour)

The critical duration trends are reflective of the critical duration becoming shorter in less frequent events as the high rainfall intensity results in faster flows and shorter travel times to the outlet.

3.8.2 Climate Change Closure Scenario

The resulting 1 in 10,000 AEP climate change RCP4.5 peak flow rates at the outlet of each of the catchments are outlined in Table 3-5.

Table 3-5 Hydrological Peak Flow Summary at Outlet

AEP	Peak Flow (m^3/s) (critical duration)		
	Central Tributary	Mungadoo Tributary	Yandicoogina Creek
Area (km^2)	50	24	150
1 in 10 000 RCP 4.5	1889 (2 hour)	1301 (1 hour)	4181 (3 hour)

The peak flow at the outlet of each catchment for the 50% AEP, 1% AEP and 1 in 10,000 AEP climate change (SSP)2-4.5, representing the distribution of flows are illustrated in Figure 3-5 to Figure 3-7.

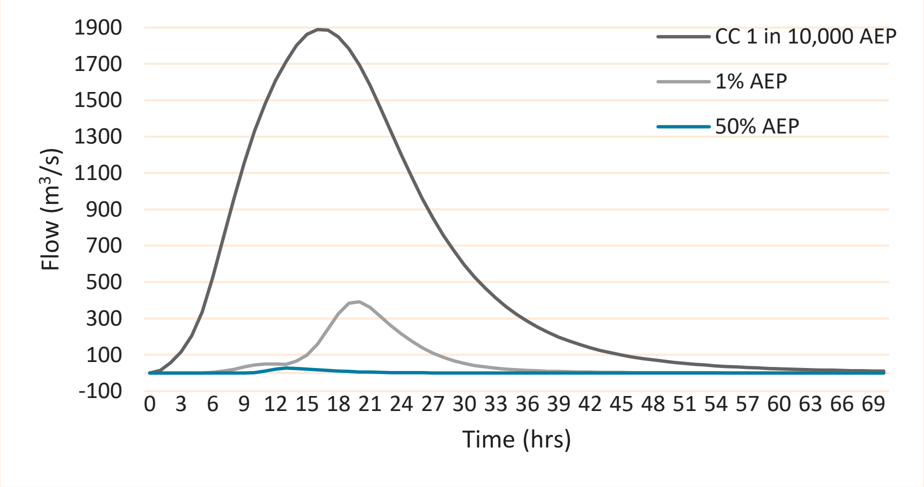


Figure 3-5 Central Tributary Outlet Flows

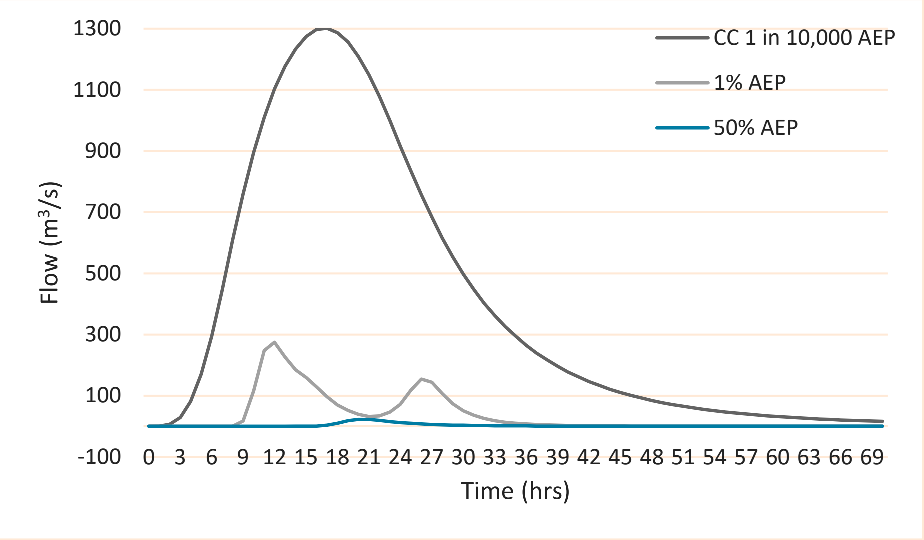


Figure 3-6 Mungadoo Tributary Outlet Flows

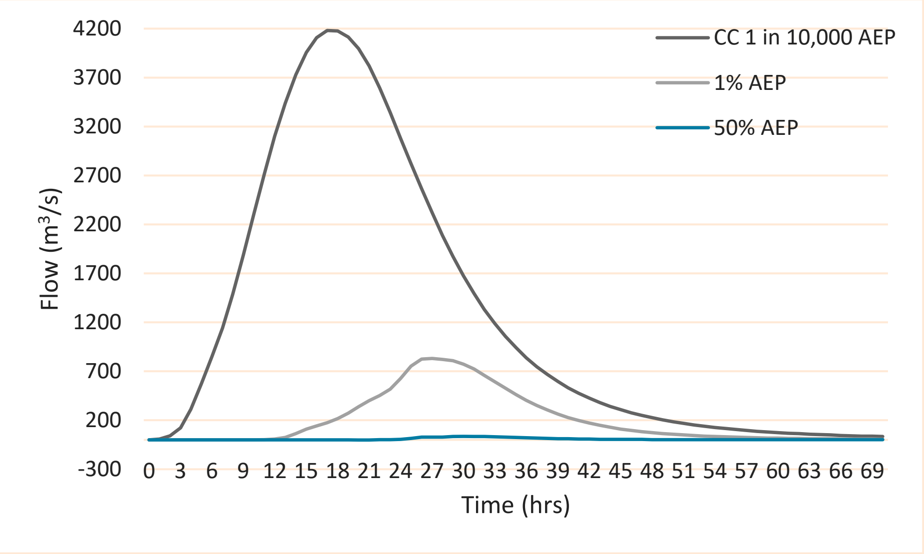


Figure 3-7 Yandicoogina Creek Outlet Flows

3.9 Validation Summary

The nearest streamflow gauges are located at Flat Rocks, Tarina and Waterloo Bore as per Section 1.5. The flows from these gauges were not transposed for validation of the peak flows of the assessed catchments.
It has been noted in numerous studies of the Flat Rocks and Waterloo Bore gauging stations that the catchment upstream exhibits hydrological characteristics inconsistent with regional trends, likely due to the Munjina Claypan.

The limited timeseries of water level data at the local BHP Yandicoogina Creek Rocks gauge and lack of flow data allowed for validation of the loss assumptions but not specific flow rates.

As such, regional loss and routing parameter estimations determined from analysis of the Tarina gauge (EPA, 2014) have been considered more suitable for validation in this study but are not suitable for direct flow comparison given the difference in catchment shape of the Weeli Wolli Creek catchment to the local catchments and evidence of storage effects not exhibited in the local catchments.

Six regional estimation methods have therefore been used to produce confidence bands for modelled flow rates.

These include:

1. Regional Flood Frequency Procedure (**RFFP**) (Flavell, 2012) – Pilbara Region (**2000** and **2006** Methods)
2. ARR87 Index Flood Method **(IFM) (**IEA**,**1987) – Catchments in Northwest Pilbara Region
3. Pilbara and Gascoyne Regional Flood Frequency Analysis (**RFFA**) (Davies and Yip, 2014)
4. Regional Flood Frequency Estimation Model (**RFFE**) (ARR, 2019)
5. ARR87 Regional **Rational Method**

The method background and details are further outlined in Appendix A.

The peak flows from all three catchments are within an average of 20-40% difference to the median of all six regional estimation validation methods and approximately -20% of the average of all six regional estimation validation methods. This has been deemed a suitable fit as illustrated in Figure 3-8 to Figure 3-10.

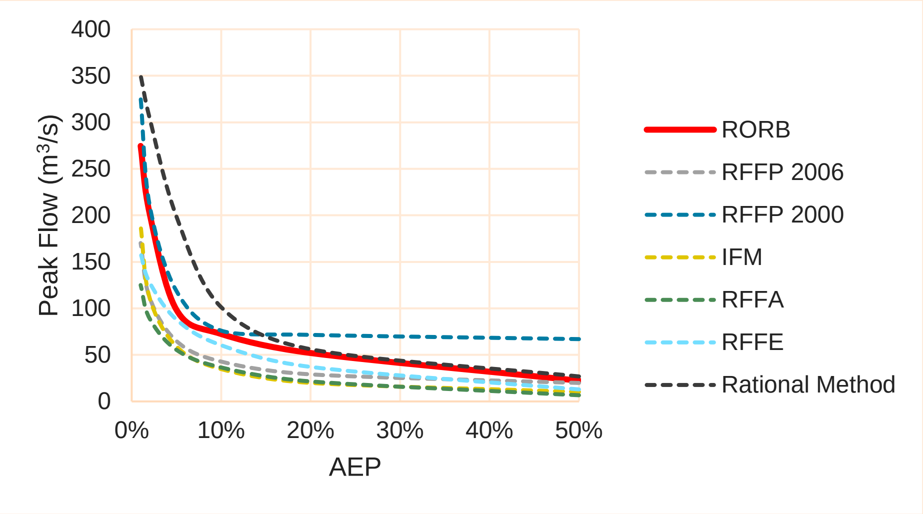


Figure 3-8 Mungadoo Tributary Catchment RORB Validation to Regional Estimates

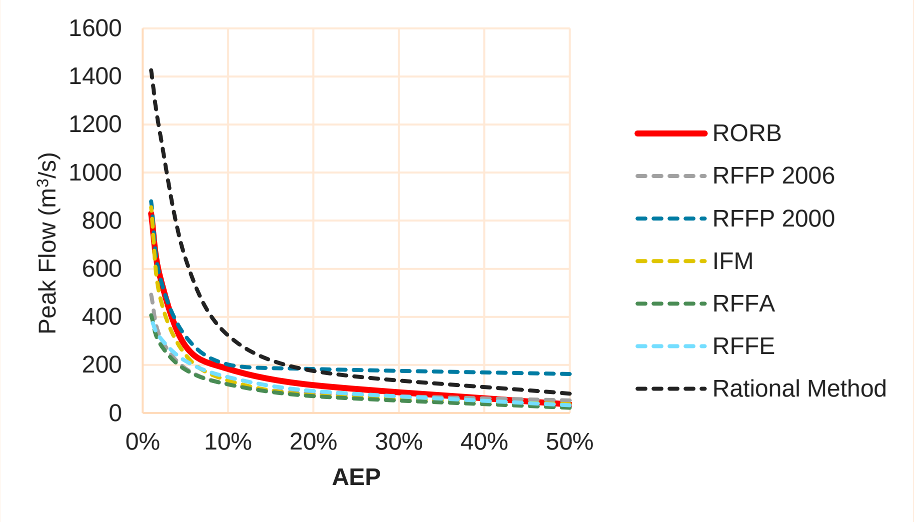


Figure 3-10 Yandicoogina Creek Catchment RORB Validation to Regional Estimates

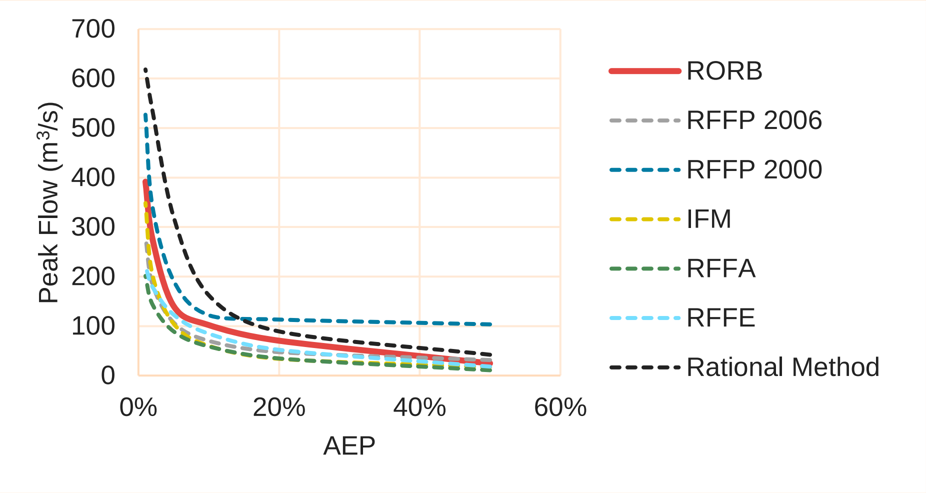


Figure 3-9 Central Tributary Catchment RORB Validation to Regional Estimates

4 Updated ARR Sensitivity Analysis

Following the release of new Australian Rainfall and Runoff guidance in August 2024 (ARR 4.2), Stantec undertook sensitivity analysis of the potential change to hydraulic model inflows for the Central Tributary Catchment for the Baseline (2030) and Closure (2100) scenarios. All RORB schematisation, including reaches, nodes, and catchments, and the ratio between non-neutral losses for each AEP were maintained.

The Shared Socioeconomic Pathways SSP2-4.5 was adopted for this sensitivity analysis in line with previous BHP risk allowance. Table 4-1 presents the global mean surface temperature projections for SSP2-4.5.

Table 4-1 Global mean surface temperature projections for SSP2 relative to 1961- 1990. The 90% uncertainty interval is provided in parentheses (DCCEEW, 2023)

Climate Scenario	Terminology used in the current document	SSP2-4.5
Current and near- term (2021-2040) (°C)	Uplifted Baseline scenario	1.2 (0.9-1.5)
Medium- term (2041-2060) (°C)	NA	1.7 (1.3-2.2)
Long- term (2081-2100) (°C)	Uplifted Closure scenario	2.4 (1.8-3.2)

4.1 Key Parameter Selection

A summary of the parameters adopted for the ARR4.2 analysis is provided in Table 4-2. Several hydrology parameters remain unchanged.

Table 4-2 Summary of RORB model input parameters for ARR4.2 analysis

Model Parameter	Uplifted Baseline scenario	Uplifted Closure scenario
	Central Tributary	
k_c	4.94 (No Change)	
Initial Loss	Between 23 mm for the 63.2% AEP to 60 mm for the 5%, 2% and 1% AEP	Between 24.4 mm for the 63.2% AEP to 63.4 mm for the 5%, 2% and 1% AEP
Continuing Loss	6.42 mm/h	6.84 mm/h
Temporal Patterns	Point (No Change)	
Areal Reduction Factors	Northern Coastal ARF ARR data hub values based on total catchment areas (No Change)	
Preburst	Proportionate to uplifted rainfall	
Filtered Embedded Bursts	Built-in RORB function (No Change)	

4.2 Uplifted Peak Flow Summary

4.2.1 ARR4.2 Uplifted Versus ARR2019 Baseline Climate Scenario

Following recommendations in ARR4.2, year 2030 was selected as the uplifted Baseline scenario for this study ("Current and near-term" Climate Scenario). The uplifted Baseline (SSP2-4.5) peak flow rates at the outlet of Central Tributary catchment are outlined in Table 4-3. The frequent events show up to approximately 50% increase in flow rates and reduced critical duration timing. This is reflective of the known behaviour of short intensity burst events which are likely to be exacerbated by climate change.

Table 4-3 Uplifted Baseline Peak Flow at the Outlet of Central Catchment

AEP	ARR4.2 Peak Flow (m^3/s)	ARR2019 Peak Flow (m3/s)	% Increase	Change in Critical Duration
50%	36.62	25	46%	Reduced to 6hour from 12hour
20%	96.7	70	38%	Reduced to 6hour from 12hour
10%	136.8	102	34%	6hour- No Change
5%	185	141	31%	6hour- No Change
2%	348.4	276	26%	Reduced to 4.5hour from 6hour
1%	489.4	392	25%	4.5hour- No Change

4.2.2 ARR4.2 Uplifted Versus ARR2019 Closure Scenario

Calculations for the uplifted closure scenario was performed based on temperature projection and rates of change for the year 2100 under SSP2-4.5 scenario. The resulting uplifted 1 in 10,000 AEP peak flow rates and critical duration at the outlet of the Central catchment are outlined in Table 4-4.

Table 4-4 1 in 10,000 AEP Peak Flow Summary at Outlet of the Central Catchment

AEP	ARR4.2 Peak Flow (m^3/s)	ARR2019 Peak Flow (m3/s)	% Increase	Change in Critical Duration
1 in 10,000	2263.2	1889 (with 14.1% uplift)	20%	2hour- No Change

The ARR4.2 uplifted peak flow at the outlet of the Central catchment for the 50% AEP, 1% AEP and 1 in 10,000 AEP under SSP2-4.5 scenario, representing the distribution of flows are illustrated in Figure 4-1.

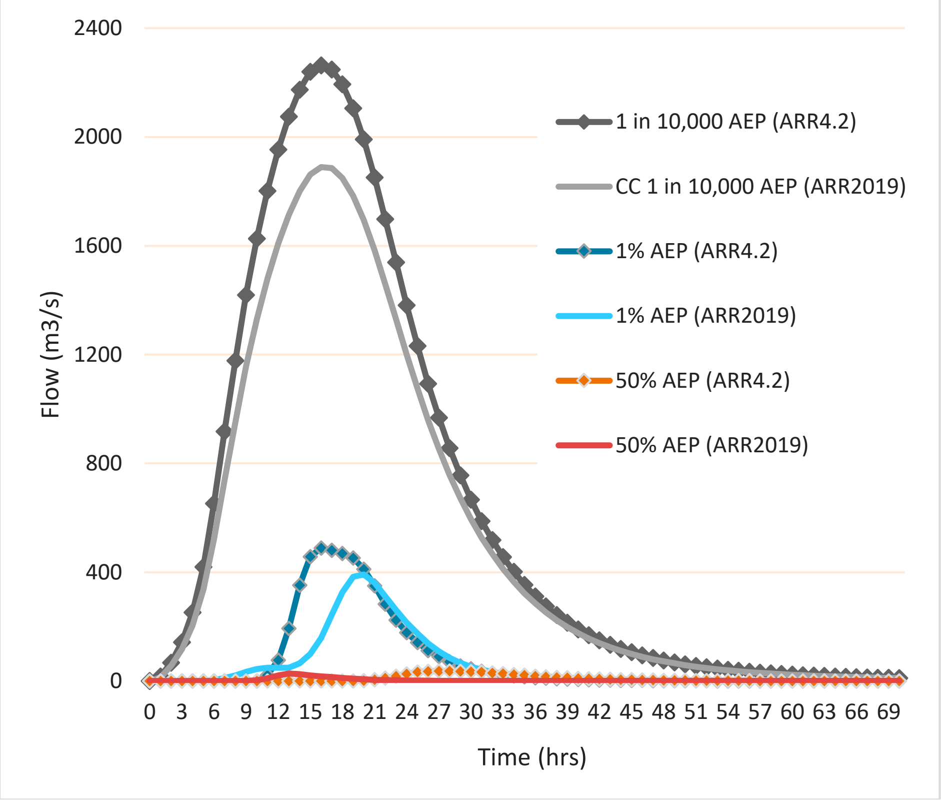


Figure 4-1 Central Tributary Outlet Flows (ARR4.2 Estimation)

4.3 Conclusion

The ARR 4.2 sensitivity analysis provides a valuable insight into the impacts of the updated guidance which was published in the final stages of this project.

As the hydrologic inputs have been validated based on recent creek flow data in the Yandicoogina Creek, any additional uplifts due to climate change (ARR 4.2 current and near-term Climate Scenario for the years 2021 – 2040 would likely be over-conservative. The 1 in 10,000-year AEP for the Closure scenario has already considered a climate change uplift based on the draft climate change guidelines available at the initiation of this study.

BHP WAIO has an awareness of the risk profile associated with the increased rainfall intensity predicted in future climate scenarios and will consider the updated guidance in any future designs and assessments.

5 Hydraulic Model Development

A hydraulic model of the Ministers North area was developed in HEC-RAS Version 6.6 software (USACE 2024) using the following modelling parameters.

5.1 Approach

The HEC-RAS model was set up using a two-dimensional (2D) rain on grid (ROG) approach, with external inflow hydrographs applied as time series boundary conditions for major catchment areas, and direct precipitation applied to the 2D flow area.

5.2 Hydrological Inputs

5.2.1 Hydrograph Inflows

The largest watercourse crossing the development area is Marillana Creek, with a contributing catchment area of approximately 2,000 km^2. The Yandi Flood Study (GHD 2014) peak discharge rates for Marillana Creek at Flat Rocks were previously adopted in the 2023 SWS assessment. An updated Flat Rocks FFA was developed for the Marillana Creek Baseline Hydrology Study (Figure 5-1, Advisian, 2023). Peak discharge rates for Marillana Creek at Lamb Creek confluence from the Advisian report were applied as external inflow boundary conditions for the rain-on-grid hydraulic model. A 14.1% uplift for climate change was applied to the 1 in 10,000 AEP discharge rates.

Given the modelled reach of Marillana Creek is relatively short and storage factors are not significant, the peak discharge rates were applied at a constant flow rate throughout the hydraulic modelling simulation window (Figure 5-3). The 50% AEP and 20% AEP peak discharge rates were not presented for the Lamb Creek confluence in the Advisian report. As such, these flows were estimated based on the Flat Rocks FFA. Table 5-1 summarises the discharge rates applied to the hydraulic analysis.

Table 5-1 Marillana Creek Inflow

Design Storm	Peak Discharge Rate (m^3/s)
50% AEP	100
20% AEP	300
10% AEP	508
5% AEP	929
2% AEP	1,573
1% AEP	2,129
1 in 10,000 AEP (includes 14.1% uplift for climate change)	9,103

Hydrographs for Yandicoogina Creek and the Central Tributary were extracted from the RORB results presented above and applied as external time series boundary conditions (Figure 5-3).

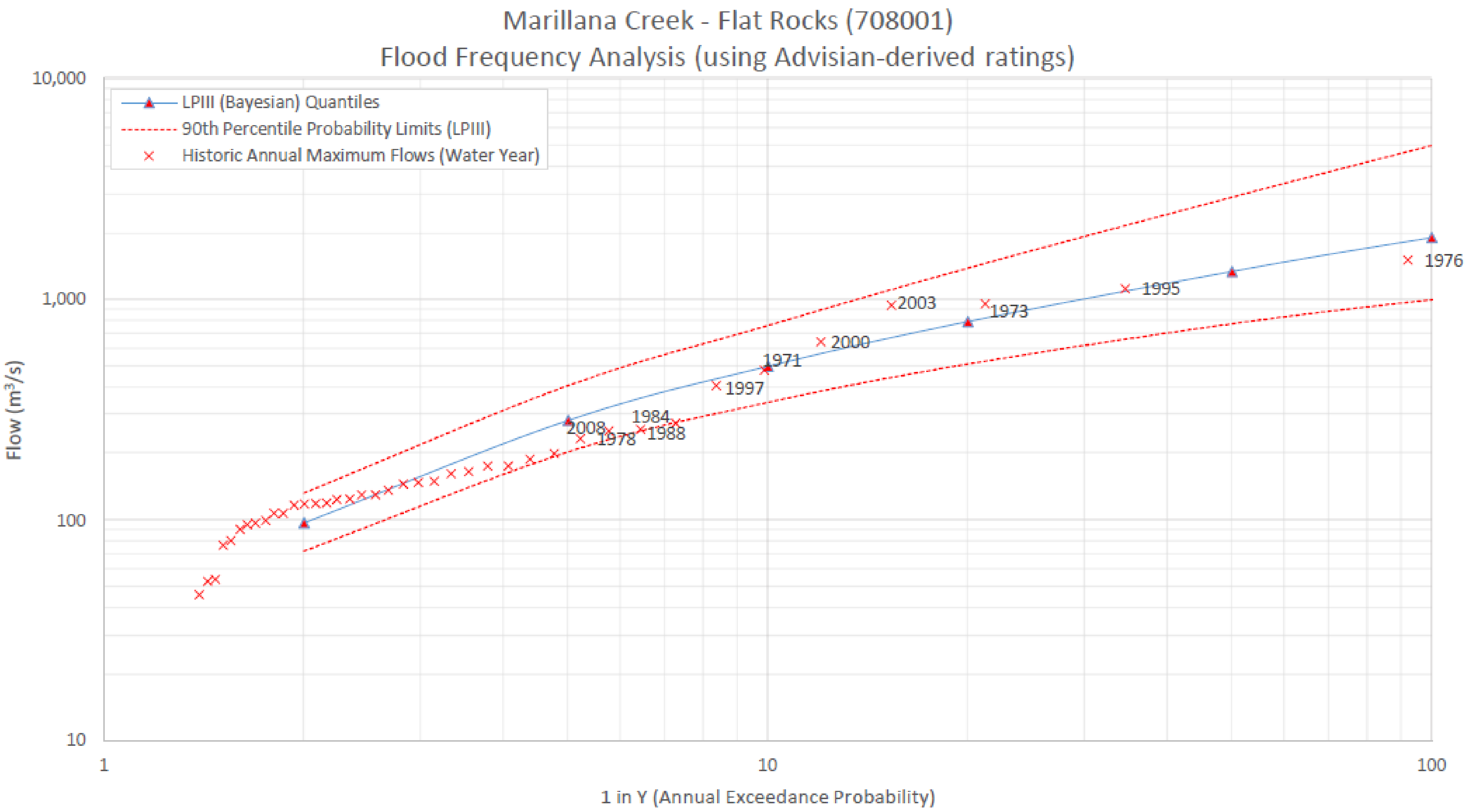


Figure 5-1 Flat Rocks Flood Frequency Analysis (Advisian, 2023)

5.2.2 Direct Excess Precipitation

Excess precipitation hyetographs generated in RORB were applied to the ROG model based on representative catchment sizes.

Six precipitation nodes were placed around and inside the 2D Flow Area as shown in Figure 5-2. Critical duration temporal patterns from the RORB validation catchments presented in Section 7 were applied to Nodes 1, 2, 3, 4, and 6.

For Node #5, located near the largest local contributing catchment area, longer-duration hyetographs were applied. The applied hyetographs for Node #5 match the temporal patterns that generated the adopted critical discharge rate in the RORB model at the Mungadoo Tributary outlet. Node #5 is schematically shown to indicate longer duration events being applied in the upstream parts of the model extents to tributaries, whereas the shorter duration events are applied closer to the model extent downstream boundary to represent smaller culvert catchments.

The Central Catchment is generally outside of the rain-on-grid area, and inflow hydrographs are adopted from the RORB model rather than from direct precipitation, so the spatial variation is less critical. The rain-on-grid area covers the Mungadoo Tributary, which has longer catchment response times than the local culvert catchments and benefits from this spatial variation.

A spatially varying precipitation layer was generated from the precipitation nodes in HEC-RAS for application of excess rainfall. Figure 5-2 shows an example of the spatial variation in accumulated precipitation depth during a 50% AEP event simulation.

Hyetographs were applied for the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 1% AEP, and 1 in 10,000 AEP flood events. The 1 in 10,000 AEP condition precipitation values and inflow hydrograph scenarios include uplift values based on interim ARR climate change guidance as outlined in Section 3.6.

5.3 Digital Elevation Model (DEM)

The underlying terrain for the HEC-RAS 2D Flow Area was compiled from the same 1-m x 1-m resolution digital elevation model used for the RORB analysis. The DEM was created by merging multiple surface tiles provided by BHP in xyz and geoTIFF format as per Section 1.4.

The DEM was applied without alteration for the existing conditions models. For the Life of Mine conditions, the DEM was modified to incorporate project features based on the design files listed in Section 1.4, including the LV SME road, haul roads, AWT pits, OSAs and stockpiles.

For the Closure condition, the trucking route and stockpiles were removed, with AWT pits and OSAs retained. The Marillana Creek floodway was removed, and the land bridge south of the floodway was retained.

An additional Rail Closure condition removes the BHP rail embankment.

5.4 Computational Mesh

A 2D flow area was applied to cover the development area. The 2D perimeter is shown in Figure 5-3. The total model area covers approximately 65 km^2. Areas with proposed development were assigned a mesh spacing of 10 m in all model runs for consistent comparisons between existing and developed scenarios.

Breaklines with a resolution of 5 m were added along selected features. The flow areas extend a minimum of 500 metres downstream of development features to capture changes in the surface water regime. Upper catchment areas were assigned a computational mesh spacing ranging from 20 m to 40 m.

5.5 Hydraulic Structures

The Baseline conditions model includes existing culverts at the Marillana Creek crossing, along the Rio Tinto rail line, and along the BHP rail line. Existing culverts are shown in blue in Figure 5-3.

The Life of Mine model applies the updated preliminary culvert schedule developed by Bechtel as illustrated in red in Figure 5-3.

The Closure conditions model removes culverts along the trucking route, including the Marillana Creek culverts. The existing rail culverts are retained in the closure conditions model.

An additional Rail Closure condition removes the BHP culverts.

5.6 Outflow Boundary Conditions

Normal depth outflow boundary conditions were assigned along the external perimeter of the 2D flow area, with energy gradients matching the bed slope in the downstream channel. The following energy gradients were applied:

- Marillana Creek: 0.3%
- Yandicoogina Creek: 0.5%
- Central Tributary: 0.8%
- Mungadoo Tributary: 0.8%

A 1% outflow energy gradient was applied along the remainder of the 2D Flow Area to allow the conveyance of runoff outside the development catchment areas.

5.7 Computational Time Step

An adaptive time step was applied in the model using a maximum Courant Number threshold of 2.0. The model adjusts the time step based on flow velocities and computational mesh size. The resulting time steps varied between 2 and 15 seconds. A simulation window of 12 hours was applied to the model to capture the arrival and recession of peak flows for all critical durations.

5.8 Roughness

Roughness coefficients were adjusted to provide the best match for runoff volume, peak discharge rate, and response time at selected validation nodes as described below. Areas with channelised flow depths exceeding 0.5 m in the 10% AEP event were assigned a roughness value of 0.04. Contributing catchment areas with sheet flow were assigned a roughness value varying between 0.04 and 0.08. Culverts were assigned a roughness coefficient of 0.023.

5.9 Computational Options

The full momentum shallow water equation was applied to the HEC-RAS model runs. Except where otherwise noted, program defaults have been applied to all remaining coefficients, options, tolerances, and model settings.

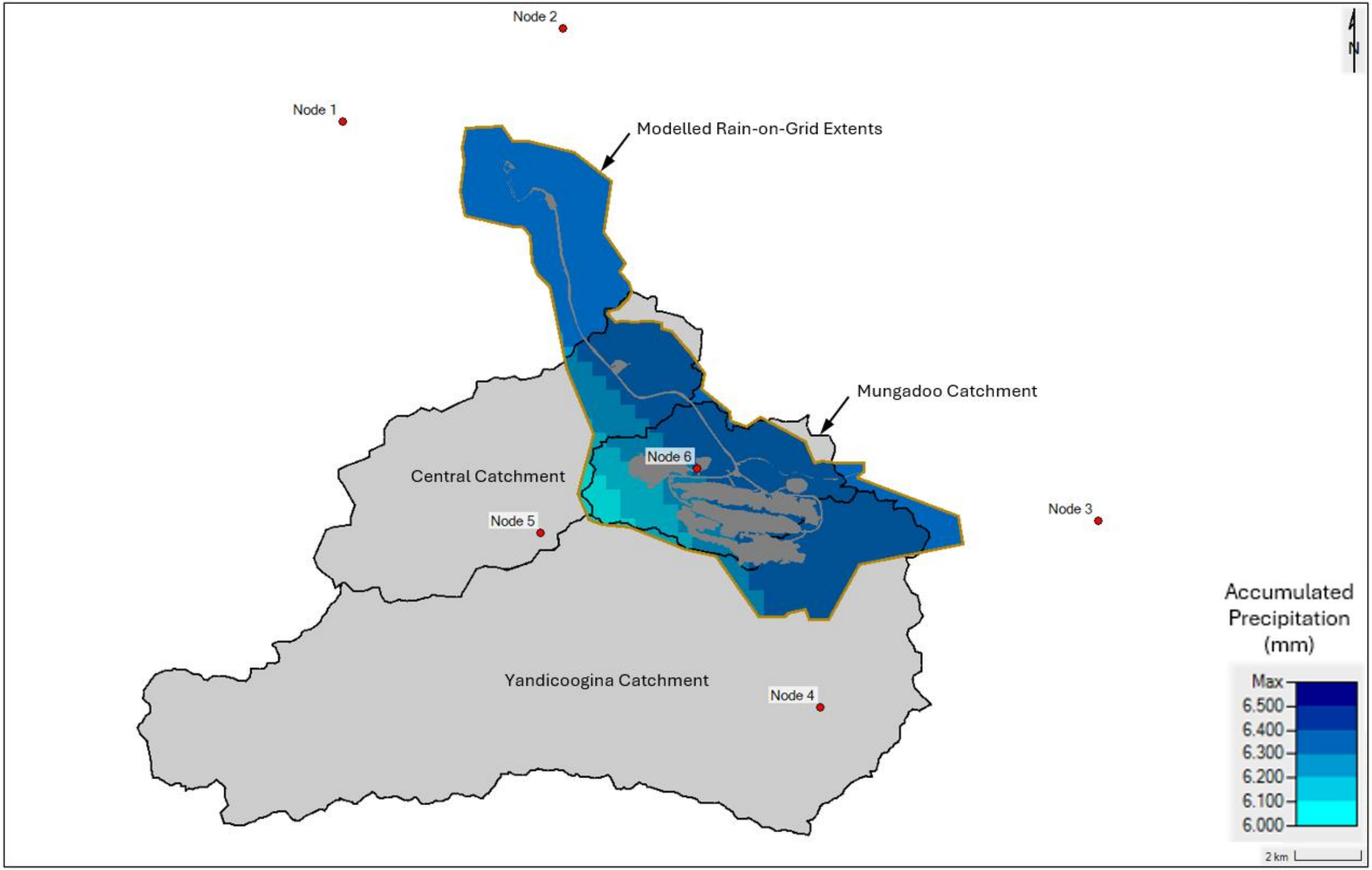


Figure 5-2 Spatially varying precipitation example

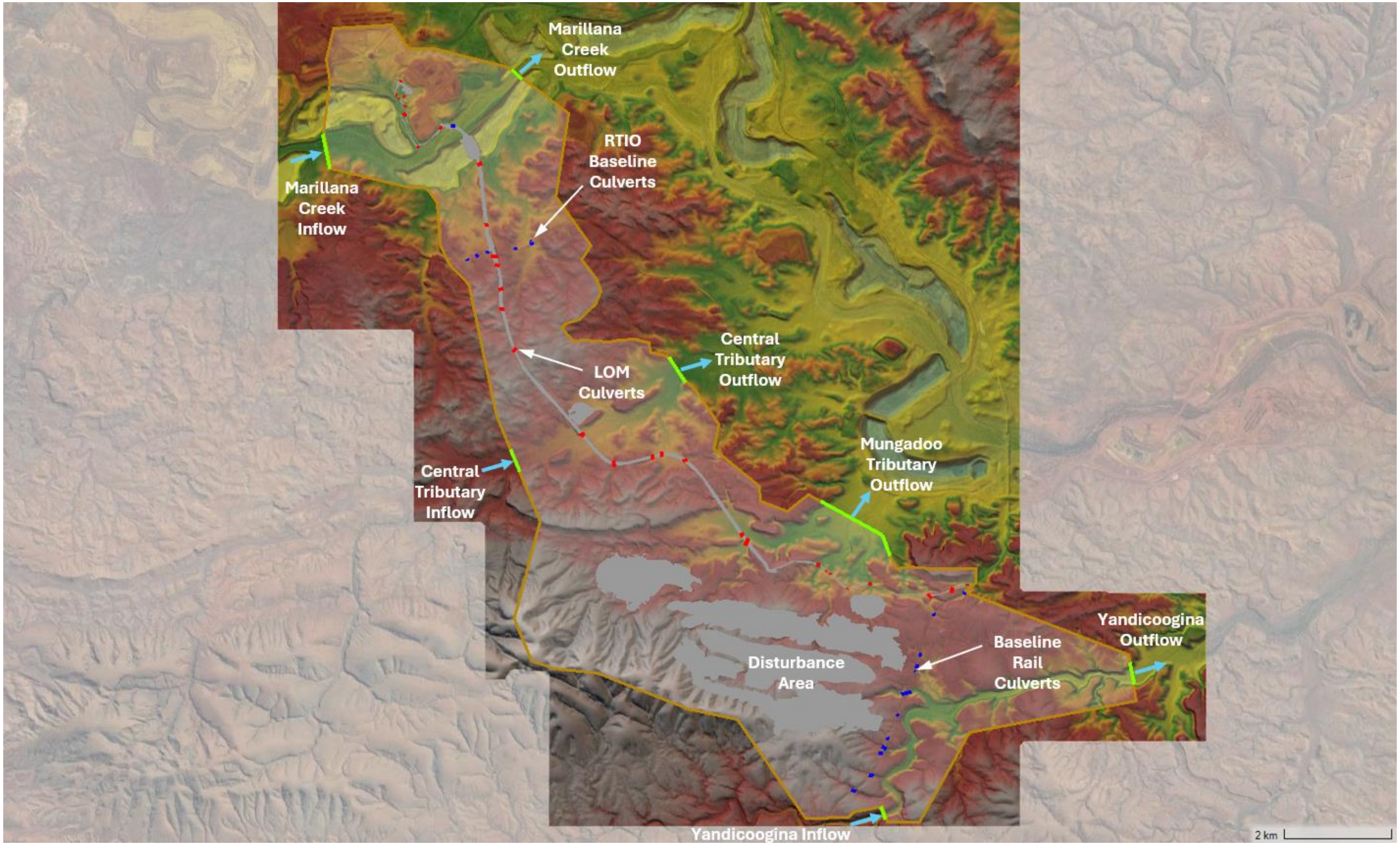


Figure 5-3 Hydraulic Model Setup

5.10 Model Summary

Table 5-2 summarises the model parameters used for the existing, developed, and closure conditions model runs. Additional sensitivity runs were applied to selected models to optimise model parameters such as the computational mesh size and computational time step and to estimate confidence bands around the presented results.

Table 5-2 Summary of model input parameters

Model Parameter	Value
DEM Resolution	1m x 1m
Inflow	GHD constant peak flow rates for Marillana Creek RORB hydrograph values for major catchments
Precipitation	RORB excess hyetographs
Outflow Boundary Condition	0.3% - 1% normal depth energy gradient
Computational Time Step	2 – 15 seconds
Simulation Window	12 hours
Computational Mesh Size	5-20 metres
Catchment Roughness	0.04 - 0.08
Channel Roughness	0.04
Culvert Roughness	0.023
Equation Set	Full momentum shallow water equation

Table 5-3 summarises the model runs along with the underlying terrain features and flood events applied to each model run.

Table 5-3 Summary of model inputs

Scenario	Terrain	Roads	Structures
Baseline	Merged 1m GeoTIFFs and xyz files with capture dates ranging from May 2013 to May 2024 (ref: 2024_MN_Merge_YAN94.tif) BHP Rail Embankment (surveyed) RTIO Rail Embankment (surveyed)	Existing Yandi Hub Roads	Existing BHP rail culverts Existing RTIO rail culverts Existing Marillana Creek Culverts and Floodway Land bridge south of the floodway
LoM	2024_MN_Merge_YAN94.tif AWT Operational Pits Overburden Storage Areas Stockpiles Topsoil Storage Areas Turkeys Nest BHP Rail Embankment (surveyed) RTIO Rail Embankment (surveyed)	Existing Yandi Hub Roads Ministers North LV SME Trucking Route (Bechtel Design) Access and Haul Roads as provided in CAD files	Existing BHP rail culverts Existing RTIO rail culverts Existing Marillana Creek Culverts and Floodway LV SME Trucking Route Culverts (Bechtel Design) Land bridge south of the floodway
Ministers North Closure	2024_MN_Merge_YAN94.tif AWT Pit Shells Overburden Storage Areas BHP Rail Embankment (surveyed)	All roadways removed	Existing BHP rail culverts Existing RTIO rail culverts Existing RTIO Rail Embankment (surveyed) Land bridge south of the floodway
Rail Closure	Backfilled Pit Shells BHP Rail Embankment Removed RTIO Rail Embankment (surveyed)	All roadways removed	BHP Culverts removed Existing RTIO rail culverts Land bridge south of the floodway

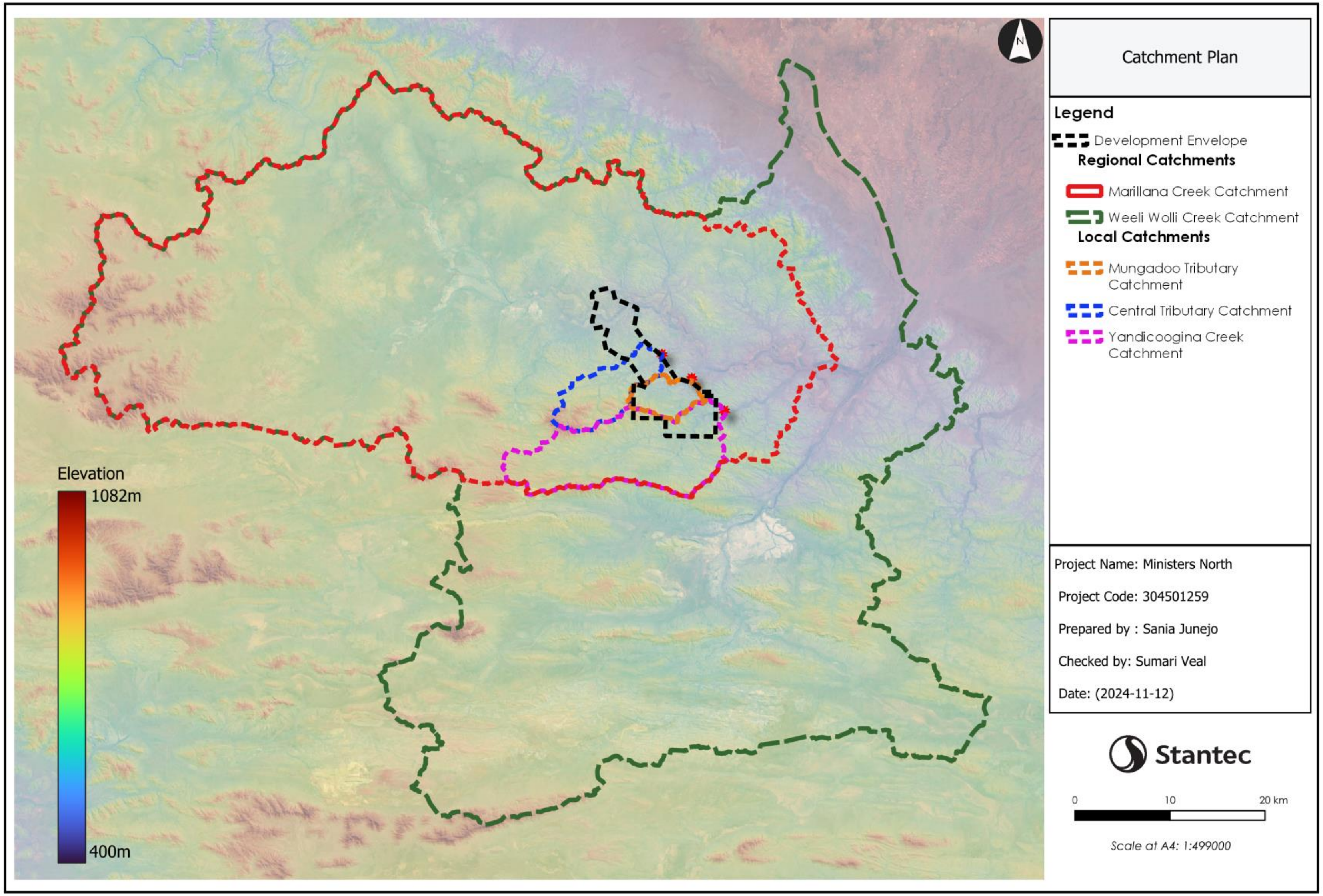
6 Catchment Area Reduction

Table 6-1 summarises the reduction in Marillana and Weeli Wolli Creek catchments in the Life of Mine condition due to the proposed pit, OSA, stockpiles and mine haul road development. For the Marillana Creek Catchment (2,228 km^2), an upper limit percentage reduction of 0.64% was observed, whereas for the Weeli Wolli Creek Catchment (4150 km^2), an upper limit percentage reduction of 0.35% was noted.

A 70% loss factor was applied to the OSAs and stockpiles.

Please note that the Marillana Creek catchment is a sub-catchment of the Weeli Wolli Catchment as shown in Figure 6-1. The local catchments with respect to the pit and OSA development is shown in Figure 6-2.

The catchment reduction specific to the Ministers North SV LME Trucking Road and other roads distributed across the development envelope have not been explicitly included in this section due to uncertainties related to the preliminary design resolution, however, overall reduction in catchment outflows and volumes are documented in Section 8.3 to inform environmental approvals.



Project Name: Ministers North

Project Code: 304501259

Prepared by : Sania Junejo

Checked by: Sumari Veal

Date: (2024-11-12)

Figure 6-1: Regional Catchments Relative to Local Catchments Areas with Reductions

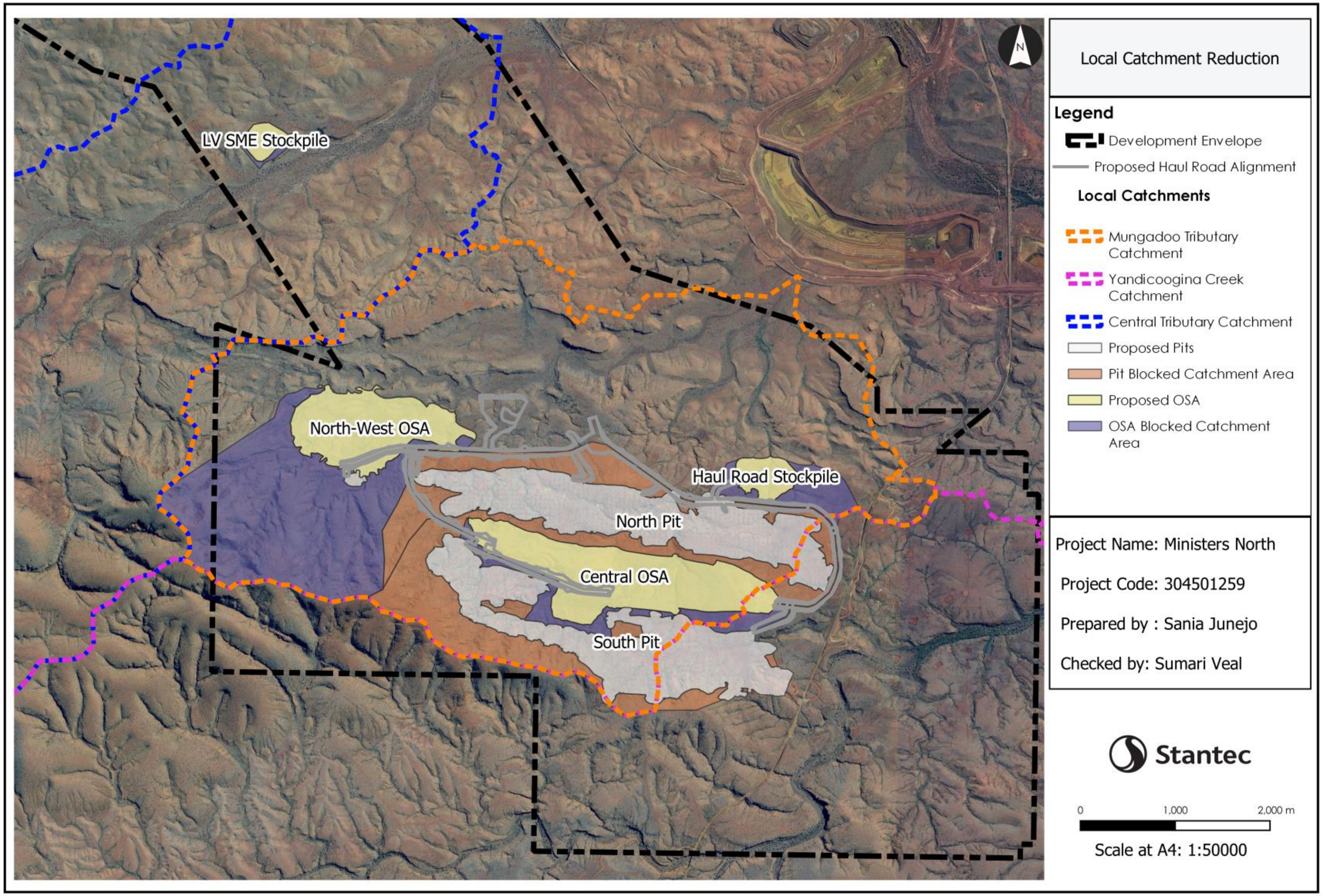


Figure 6-2: Local Catchments Relative to Development Pits, OSAs and Mine Haul Road

Table 6-1: Catchment Reduction

Local Catchment ID	Central Tributary Catchment	Mungadoo Tributary Catchment	Yandicoogina Catchment	Total
Pit area (ha)	0	345	100.3	445.3
Pit catchment (ha)	0	276	32.2	308.2
OSA area (ha)	12.6	295.3	7.9	315.8
OSA catchment (ha)	5.2	408.4	11.8	425.4
Reduction in Catchment Area (ha)	14.0	1236.1	149.8	1399.9
Marillana Creek Catchment Reduction	0.01%	0.55%	0.07%	0.63%
Weeli Wolli Creek Catchment Reduction	0.003%	0.30%	0.04%	0.34%

7 Hydraulic Model Validation

The adopted roughness coefficient was adjusted in the hydraulic model to provide a reasonable match between the RORB and HEC-RAS model results at selected validation points, including runoff volume, peak discharge rate, and response time within each area. To avoid exaggerating losses, depression storage within the adopted DEM was filled using pre-start precipitation. A minimum roughness coefficient of 0.04 was applied for the validation exercise.

Figure 7-1 shows the location of the validation points, and Table 7-1 summarises the comparison of Baseline peak flow rates at the selected points. Overall, the ROG model peak discharge rates are within 25% of the peak flow across the RORB validation points for the 1% AEP. Shallower flows are more sensitive to the differences in computational methods for routing flows between rainfall-runoff models and rain-on-grid hydraulic models. Flow rates for the 50% AEP differ more substantially between the model runs, with an average variation of 40%.

This discrepancy is explained by the RORB values shown in Table 7-1 being based on critical duration temporal patterns for individual subcatchments under Baseline conditions and do not account for the direct precipitation distribution in the hydraulic model, complexity of the highly undulating topography and existing hydraulic controls such as existing railways. The contributing catchment areas at individual validation points vary in the LoM and closure conditions.

Detailed hydrologic modelling of these areas may result in different temporal patterns for each scenario, however, as the afflux mapping comparison of Baseline, LoM, and Closure conditions requires consistent precipitation input, a single, gridded temporal pattern was applied for all scenarios. In some cases, the ROG results underestimate the critical duration peak discharge rates at individual subcatchments in favour of a closer match at tributary outlets. This is considered appropriate for this assessment.

Peak discharge rates in major tributaries match more closely between the RORB and ROG results, with comparison hydrographs at tributary outlets shown in Appendix D. These comparative hydrographs were only run to the peak flow to allow comparison of peaks as considered appropriate for the scope of works. Comparison of the Marillana Creek flows were not undertaken as a hydrological model was not required specific to this study.

Table 7-1 Comparison of RORB and Rain on Grid results at Selected Validation Points

Point	Peak Flow (m^3/s)					
	50% AEP RORB	50% AEP ROG	Diff (%)	1% AEP RORB	1% AEP ROG	Diff (%)
C5	1.9	1.0	51%	9.5	9.2	3%
C6	0.5	0.3	41%	2.6	2.7	4%
C7	1.3	0.6	55%	6.5	5.3	19%
M3	0.6	0.3	58%	2.6	2.1	19%
M4	0.1	0.1	100%	0.2	0.2	27%
M5	9.2	3.0	67%	42.3	30.3	28%
M7	2.4	1.1	53%	11.3	10.2	10%
M8	1.4	1.0	26%	12.9	9.9	23%
M9	13.0	6.3	51%	83.3	60.5	27%
M10	0.1	0.1	29%	0.6	0.1	83%
M11	6.3	2.2	65%	27.2	20.8	24%
M12	1.7	0.7	62%	7.4	4.7	37%
M13	0.4	0.1	67%	1.6	1.2	27%
Average			**40%**			**25%**

Table 7-2 Comparison of RORB and Rain on Grid results at Outlets

Point	Peak Flow (m^3/s)					
	50% AEP RORB	50% AEP ROG	Diff (%)	1% AEP RORB	1% AEP ROG	Diff (%)
Central Tributary	25	26	4%	392	358	-9%
Mungadoo Tributary	22	22	0%	275	221	-20%
Yandicoogina Creek	36	47	31%	830	842	1%
Average			**12%**			**-9%**

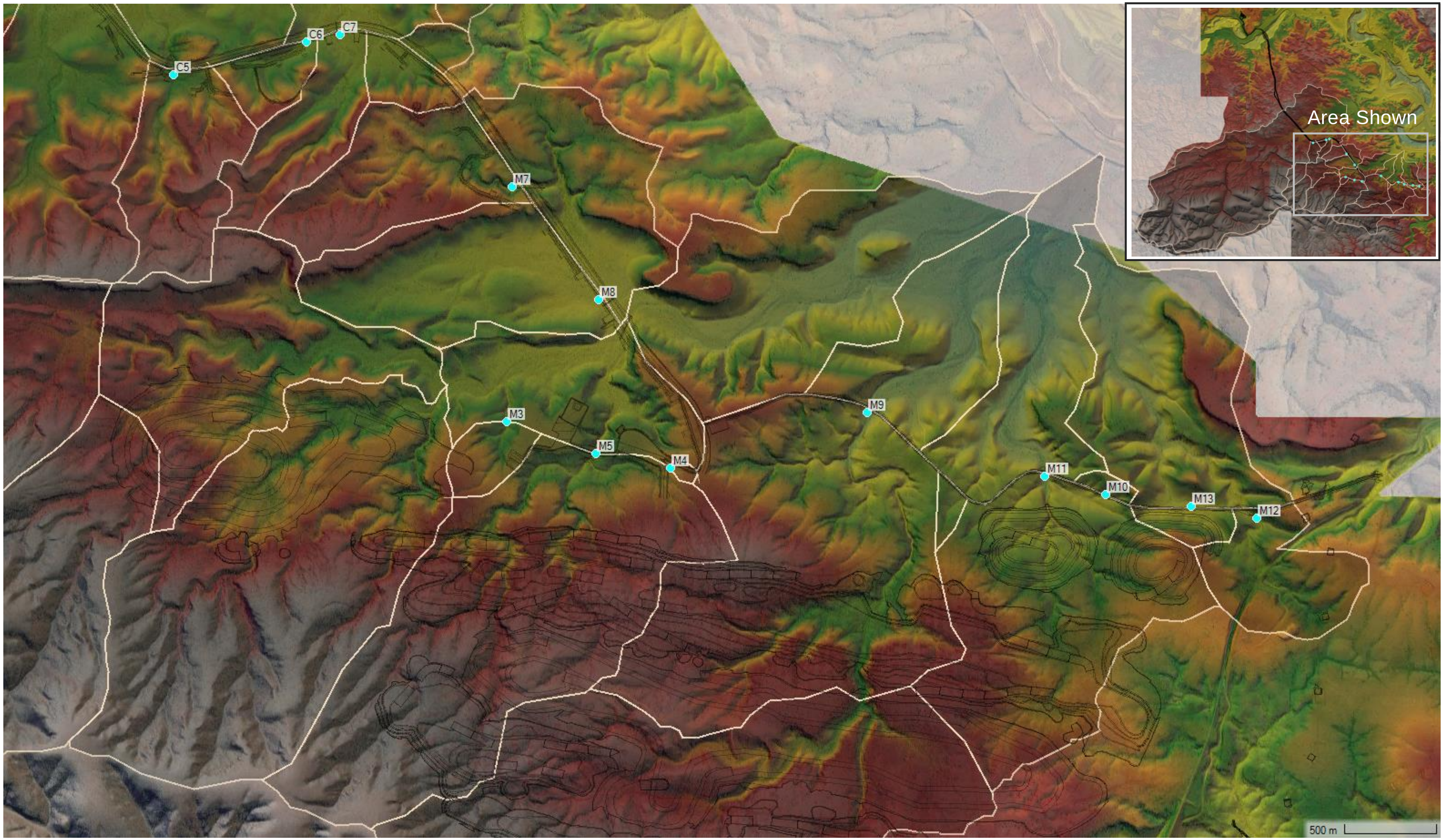


Figure 7-1 Location of Validation Points

8 Hydraulic Results

This report includes selected depth, velocity, and water surface elevation afflux results for the 50% AEP, 1% AEP and 1 in 10,000 AEP (with climate change uplift) events. Results for the remaining events are available in the accompanying electronic files.

Five figure viewports have been generated along the study area alignment to enable adequate interrogation of flood results. Table 8-1 shows the figures included in this report (5 for each scenario). Data for the remaining scenarios are available in the accompanying electronic files.

Table 8-1 Scenarios included in this report

AEP	Depth			Velocity			WSEL Afflux	
	Appendix B						Appendix C	
	Baseline	LOM	Closure	Baseline	LOM	Closure	LOM	Closure
50%	✓						✓	✓
20%								
10%								
5%								
1%	✓	✓	✓	✓	✓	✓	✓	✓
1 in 10,000 CC uplift			✓				✓	✓

The Rail Closure results are shown in Appendix E.

8.1 Flood Depths and Velocities

Maximum flood depths and velocities for the 50%, 1% and 1 in 10,000 AEP (with climate change uplift) have been included in Appendix B.

The flood depths in the Baseline condition figures highlight the presence of the existing Marillana Creek floodway to the south of the Yandi Hub. This is removed in the Closure condition, reflected in the reduced upstream ponding relative to the LoM.

As shown in Figure Set 3 at the centre of the site, the trucking route design contains a sacrificial section of road overtopping in the 1% AEP event. This is consistent with the previous study (SWS, 2023) and industry-level standard design criteria.

In general, velocity reductions are shown upstream of earthworks that impede flow, including the inlet side of culvert crossings along roadway formations. These areas have the potential for sediment deposition that should be monitored during operations.

Localised increases in velocities occur in the immediate vicinity of culvert outlets. Scour mitigation is to be incorporated as part of the operational design, with monitoring and maintenance implemented as required.

High velocities are shown along steep slopes of proposed pits and landforms. Detailed drainage designs would need to accommodate any concentrated flow paths with high energy.

Recommendations for the management of sediment and velocities are included in Section 9.

8.2 Change in Flood Levels (Afflux)

Water Surface Elevation (WSEL) Afflux maps comparing the baseline condition to the life of mine and closure conditions are included in Appendix C.

In order to generate the afflux mapping, peak flood levels were exported in raster grid format for each condition. WSEL values were then subtracted from each other to generate a set of difference maps in raster grid format.

As shown in the difference mapping, the largest potential impact is ponded water that accumulates in low areas against the landforms, with the potential formation of breaches where overflows occur.

Along the perimeter of the western landform and on the upstream side of the haul road embankment shown in Figure 8-4, for example, increased water levels are shown. The potential for erosion can be mitigated in these areas with additional culvert or drains.

Reduced water levels are shown downstream of pits, landforms, and flow constrictions that intercept upstream catchments or attenuate peak flows. The 1% AEP Life of Mine afflux maps are illustrated in Figure 8-1 to Figure 8-5, with the remaining scenarios shown in Appendix C.

In some areas of the model, apparent afflux is shown where no changes are proposed due to computational differences between the model runs. In general, the adopted filter of +/- 0.2 m eliminates these areas from display; however, some areas outside of these thresholds are shown in the figures. This is not considered a concern for environmental approval purposes.

Afflux is generally limited to the range of -1.0 - +2.0 m; however, there are some areas where values are outside of these ranges. Areas where the afflux are significantly exceeded are called out with annotation in the figures.

Figure 8-1 1% AEP Life of Mine Afflux (Location 1 of 5)

Figure 8-2 1% AEP Life of Mine Afflux (Location 2 of 5)

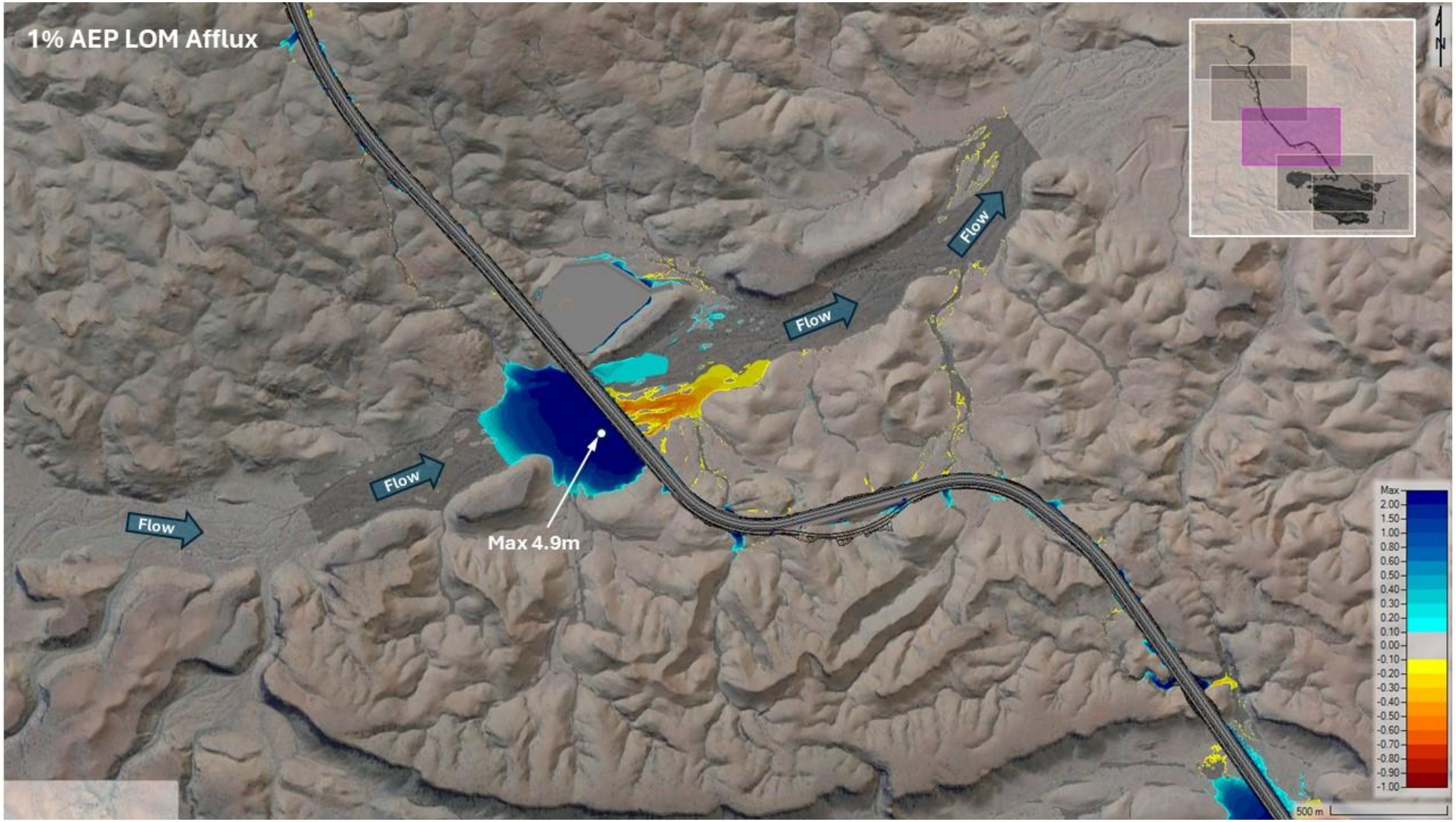


Figure 8-3 1% AEP Life of Mine Afflux (Location 3 of 5)

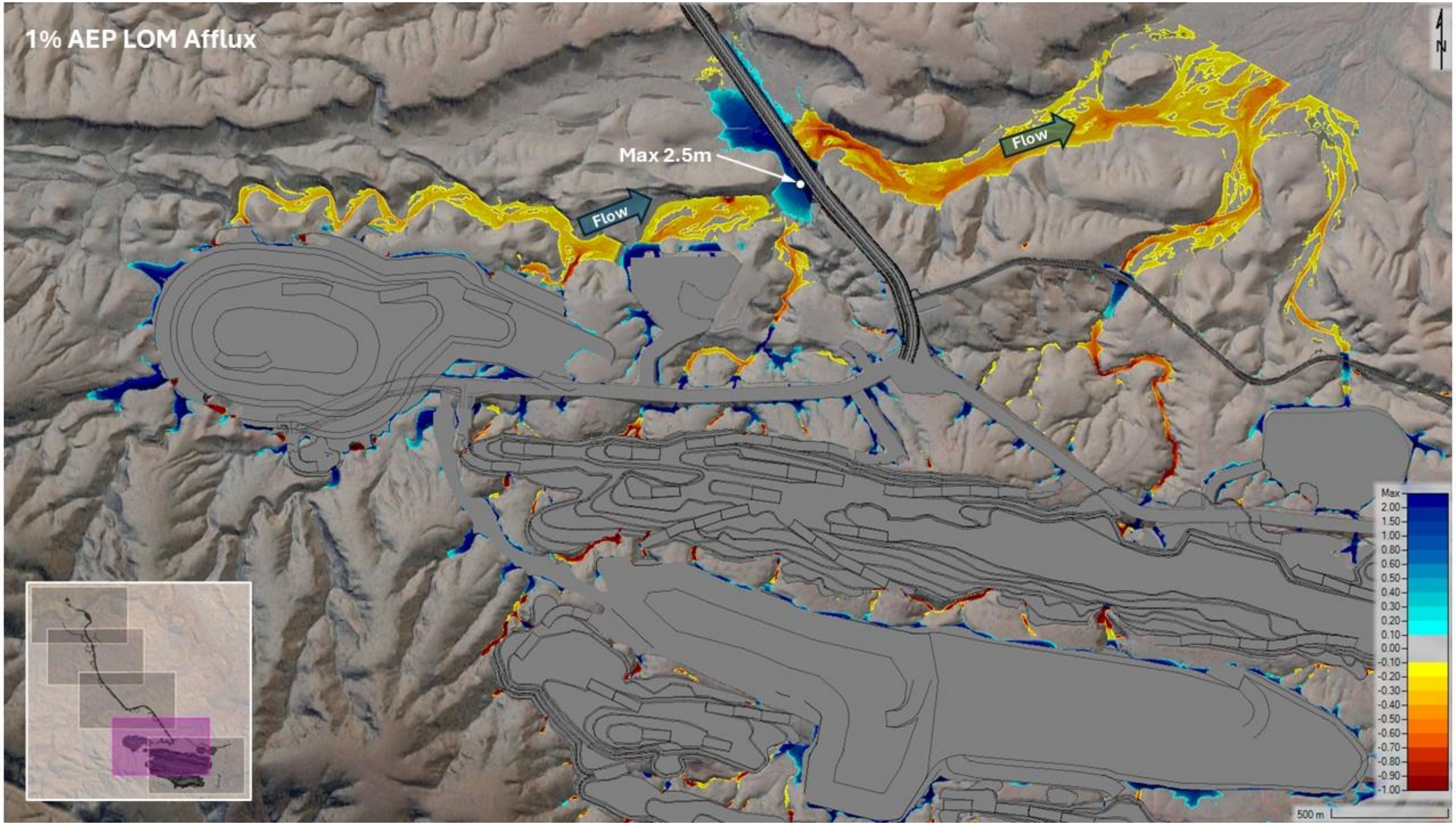


Figure 8-4 1% AEP Life of Mine Afflux (Location 4 of 5)

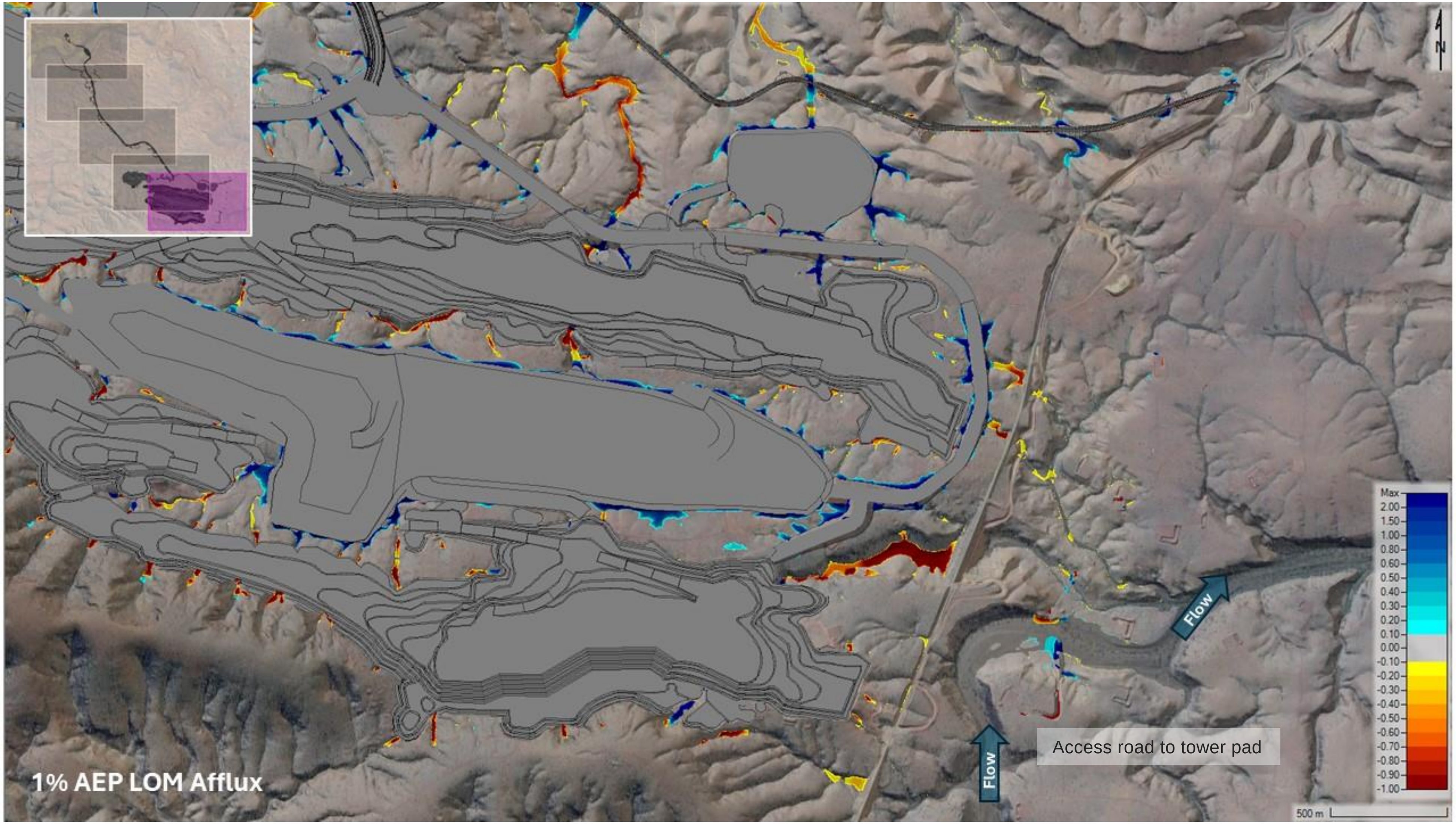


Figure 8-5 1% AEP Life of Mine Afflux (Location 5 of 5)

8.3 Comparative Peak Flows and Volumes

Flow hydrographs were extracted along outflow locations placed at the downstream extent of the key catchment areas. The hydrographs are shown in Appendix D.

As shown in the hydrographs, both the peak flow rate and the time to peak flow are affected by the development. The total flow volume (as represented by the area under the hydrographs in Appendix D) is similarly affected.

The change in total cumulative volumes at the Yandicoogina Creek outlet is negligible between Baseline, LoM and Closure in the 1% AEP. There is approximately 20,000m^3 difference in the 50% AEP Baseline to LoM and Closure due to the inclusion of the pits which equates to a volume reduction of approximately 2%.

The Central Tributary 50% Baseline and Closure cumulative volume differences are negligible. The total 50% AEP LoM cumulative volume reduction is approximately 5% relative to the baseline volume due to the inclusion of the LV SME Trucking haul road and stockpile.

The cumulative 1% AEP flow volume in the Mungadoo Tributary is approximately 1,660,000 m^3. This volume is reduced by more than half in the LoM and Closure scenarios. Similarly, the 50% AEP cumulative volumes for the LoM and Closure are less than half the 230,000 m^3 cumulative baseline volume.

The changes in peak flow rate, time to peak, and flow volume are not considered to be significant on a regional scale in specific relation to the Minister's North development. Lost catchment area specific to the pit, OSA, stockpile and mine haul roads is specified in Section 6.

Table 8-2 summarises the peak discharge rates at the primary tributary outlets in the frequent 50% AEP (approximately 1 in 2-year ARI) and rare 1% AEP events (approximately 1 in 100-year ARI).

Table 8-2 Comparison of peak discharge rates at outlets

Outlet	Peak Flow (m^3/s)					
	50% AEP Baseline	50% AEP LOM	50% AEP Closure	1% AEP Baseline	1% AEP LOM	1% AEP Closure
Central Tributary	26.3	22.3	26.3	370.1	349.3	370.1
Difference to Baseline		**-15%**	**0%**		**-6%**	**0%**
Mungadoo Tributary	22.2	6.4	9.8	218.6	64.4	94.9
Difference to Baseline		**-71%**	**-56%**		**-71%**	**-57%**
Yandicoogina Creek	47.7	46.9	47.2	841.8	840.8	841.0
Difference to Baseline		**-2%**	**-1%**		**-0.1%**	**-0.1%**
Marillana Creek	**103**	**103**	**103**	**2162**	**2162**	**2162**
Difference to Baseline		**0%**	**0%**		**0%**	**0%**

The changes in the Mungadoo Tributary catchment peak flows and volumes are due to the pits and OSAs consisting of 30% of the local catchment area. The highly disturbed Yandicoogina Oxbow RTIO mine site is immediately downstream of the Mungdaoo Tribuary catchment. As the pits (backfilled) and OSAs are maintained in the closure scenario, the Mungadoo closure changes are also to be expected.

The changes in the Marillana Creek peak flows are negligible given the steady inflow condition.

9 Mitigation Measures

9.1 Overview

Erosion, sediment transport and sediment deposition are key areas requiring mitigation as part of future mine planning phases.

Velocity reductions are shown upstream of earthworks that impede flow, including the inlet side of culvert crossings along roadway formations and these areas have the potential for sediment deposition that should be monitored during operations.

Localised increases in velocities occur in the immediate vicinity of culvert outlets. Scour mitigation is to be incorporated as part of the operational design, with monitoring and maintenance implemented as required.

High velocities are shown along steep slopes of proposed pits and landforms. In these areas where concentrated flow enters the pits, some erosion would be expected. The construction of rock chutes, down-drains, or other measures may be required to prevent erosion of the pit wall and crest at concentrated inflow locations. Energy dissipation at the toe of the slope or at periodic intervals along the flow path may be warranted to reduce erosive forces. Detailed drainage designs would need to accommodate any concentrated flow paths with high energy.

During construction in particular, an increase in sediment runoff and scour may occur as a result of the additional ground disturbance and vegetation removal. Runoff resulting from direct rainfall on landforms is assumed to be contained onsite. Where feasible, sediment-laden runoff from landform slopes are to be collected in toe drains and diverted to downstream sediment ponds. The high-level locations and sizing of the sediment ponds are outlined in the section to follow.

When the pits are closed, the waste rock dumps are to have an armoured rock exterior of inert material to protect against sediment runoff in the long term.

9.2 Recommended Diversions and Bunding

Diversion of localised drainage paths around proposed pits or controlled inflow measures may be required in some locations where concentrated flow paths intercept pits and landforms.

Additional areas where bunding and/or minor diversions may be required include the proposed batch plant, explosives storage area and top soil storage areas.

There are several turkeys nest areas designated throughout the development envelope to reduce loss of water and to stop groundwater contamination.

9.3 Borrow Pit Management

Borrow pits have been designated across the development site and are subject to further confirmation. The location of these borrow pits are subject to environmental considerations outside the scope of this assessment. Adequate buffers, silt fences, sediment traps, and other erosion control measures are to be utilised to minimise impact to receiving waters. Mine rehabilitation at closure is to restore vegetation cover to stabilise soil and reduce erosion after the borrow pits are decommissioned.

9.4 Indicative Sediment Basins

During construction staging and the Life of Mine, sediment basins will be implemented to retain sediment-laden runoff to capture the 10% AEP 72-hour event and trap sediment-laden rainfall runoff to allow for the settling of sediments before runoff enters the environment, minimising the risk of sediment levels exceeding natural conditions. Any excess water that passes through or around sediment ponds is assumed to have been attenuated sufficiently to avoid impacting the predicted peak flood extents. There is particular concern for ensuring that sediment is removed before any runoff reports to Yandicoogina Gorge.

Sediments in surface water within the mine pits should be contained in pit floor sumps for sedimentation retention time before being pumping to the proposed mine-affected sediment basins to avoid high concentrations of sediment in the dewatering discharge. The proposed 100L/s discharge is to be released into stable, rock protected ponds to limit localised system erosion.

A high-level assessment has been undertaken based on existing topography, infrastructure locations and distance to creeks. Five sediment basins are proposed, one for each pit, OSAs and stockpiles. For the North pit, there are two potential locations. The indicative sediment basin locations are within the development envelope and greater than 50m away from the 1% AEP LoM flood extent (Figure 9-1).

Table 9-1 provides the high-level design summary of the indicative sediment basins. The sediment basins were designed for the 10% AEP 72-hour design event, with 0.7 runoff coefficient for the OSA and stockpiles to attain a minimum 70% removal of very fine sand particles. A 4:1 length to width ratio has been assumed to minimise short circuiting.

Table 9-1: High-level Design Summary of Sediment Basins

Characteristic	North Pit	South Pit	North-West OSA	Haul Rd Stockpile	LV SME Stockpile
Catchment Area (ha)	443	486	471	65	18
Flow Rate (m^3/s)	0.1 (pump)	0.1 (pump)	3	0.4	0.1
Target Sediment Particle Size (µm)	125				
Settling Velocity (mm/s)	11				
Surface Area (m^2)	64	64	576	64	64
Depth (m)	1.5	1.5	1.5	1.5	1.5
Volume (m^3)	96	96	864	96	96

*Please note that the OSA 1 runoff ultimately reports to the North and South Pit. This has been included in the catchment area for the North and South pits, respectively.

The following limitations for the high-level sediment basin design are noted:

- Basins have been sized based on a desktop analysis of catchment areas and expected runoff behaviour.
- It has been assumed that diversion drains will be utilised to divert flows around the OSA to the sediment basin. Therefore, the design of sediment basins receiving runoff from OSAs includes both the OSA area and the area of the catchment reporting to the OSA.
- Sediment basin sizing has not been undertaken with regard to cut and fill requirements, overflow weir dimensions, decant structures, erosion protection design of pond outfalls, batters, maintenance ramps or safety design measures. These will need to be incorporated in future design phases.

During a flood greater than the 10% AEP capacity of the sedimentation pond volume, sediment may pass through the detention ponds with lower rates of settlement. In a naturally occurring scenario, sediment loads during large events are high, so the impact of releasing water from the sediment ponds during these events should be minimal on a regional scale. The percentage of runoff water and sediment from the disturbed ground within the mine operation will also only form a small percentage of the runoff from the upstream and adjacent natural catchment areas and consequently is not expected to have a significant impact on the overall catchment downstream of the mine site. The natural background suspended sediment load in the receiving waters will be high due to a major rainfall event, and overflows from a sedimentation pond would be diluted and dissipated quickly.

A larger-than-design flow event could, however, flush out sediments from the sediment ponds, causing a localised concentration of silts into the downstream drainage line. Pond designs should therefore include overflow provisions to ensure the containment embankment remains stable during a large event. Sediment flushing may be minimised by regular maintenance of the pond to remove and dispose of sediments into the body of the waste rock dump before large events occur. Even if full, provided the sediment pond embankments are sound, the water that passes through the pond is expected to have some settlement and associated improvement to discharge water quality due to residence time in the sediment pond, even in a larger-than-design event.

It is important to manage sediment basins during operations to ensure the basin is functioning as designed. It is recommended to incorporate sediment basin maintenance and cleaning into the site’s maintenance schedule to mitigate risk. After a large storm event, inspection and maintenance of the sediment basin is required to assess damage and identify excessive erosion. Rock protection will be required downstream of the sediment basin for erosion reduction and sediment control.

Following closure, however, sediment ponds would fill over time and eventually become ineffective, with the intention that exposed slopes would have been rehabilitated and vegetation established while the sediment ponds are still in use. All rock protection will need to be removed, and the affected area restored to its natural surface level.

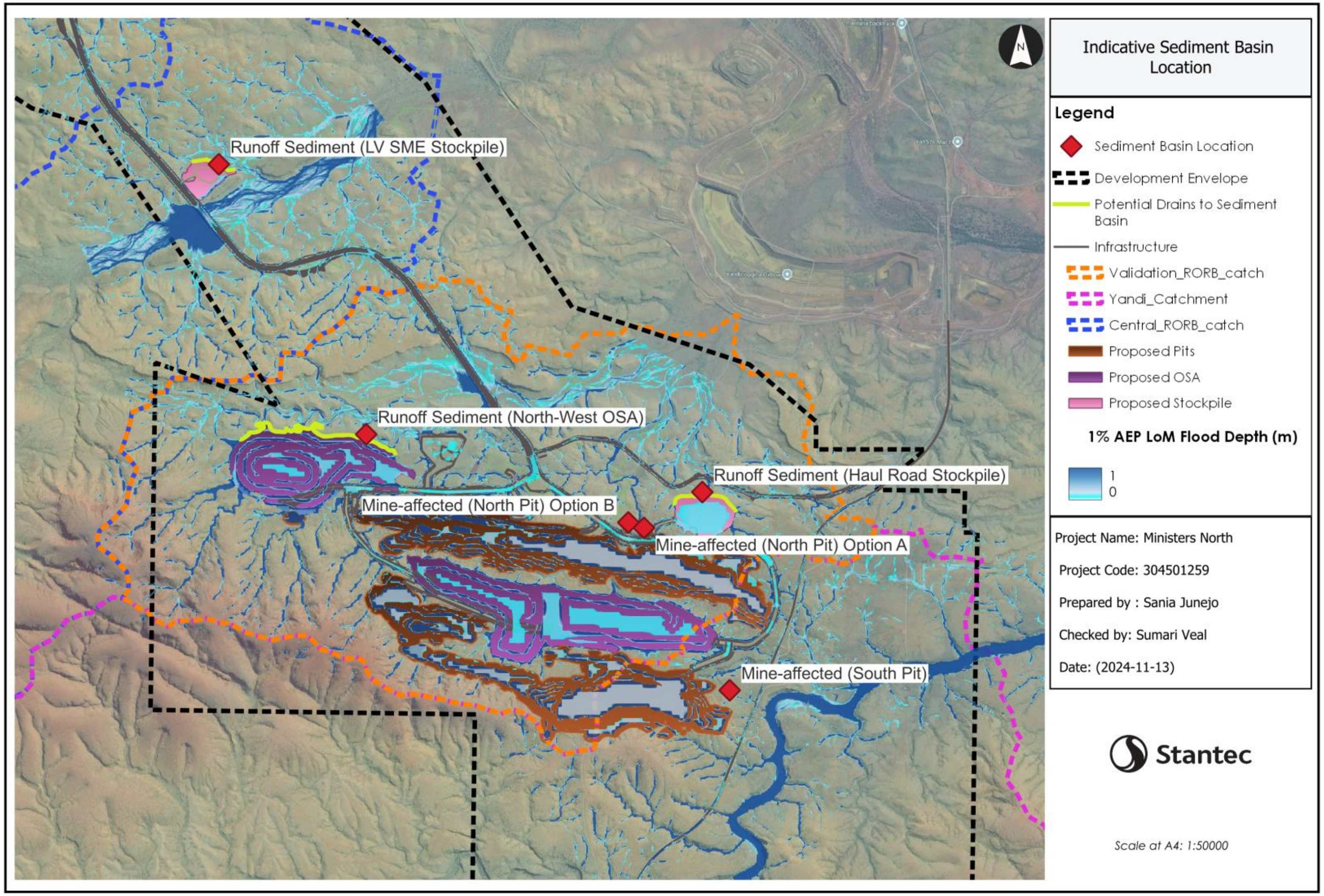


Figure 9-1: Indicative Sediment Basin Locations

10 Conclusion

The proposed landforms and pits are generally located on higher ground near the catchment divide with relatively small contributing catchment areas; therefore, the risk of significant pit inundation and creek capture is lessened, and the operations can be protected from flooding with relatively standard surface water management features.

Rainfall and runoff have been assessed over the Development Envelope, and design rainfall events have been derived for the 50%, 20%, 10%, 5%, 1%, and 1:10,000 AEP events using RORB, and HEC-RAS 2D models.

The afflux maps in this report highlight areas where flow depths will be increased or decreased in the Life of Mine and Closure condition. In general, flow depths are increased upstream of the proposed trucking route embankment, OSAs, and stockpiles. The figures compare maximum water levels, which typically coincide with peak flow conditions. The life of mine and closure conditions models do not include diversion drains or levees to route flow around proposed features.

Where landforms block flow paths, ponded water would be subject to infiltration and evaporation, which is not modelled and provides a conservative estimate of catchment loss. Where culvert crossings are provided, the upstream afflux is temporary as the flows recede by the end of the simulation. Along the perimeter of the western OSA landform and on the upstream side of the haul road embankment, increased water levels are shown. The potential for erosion can be mitigated in these areas with additional culvert or drains. Reduced water levels are shown downstream of pits, landforms, and flow constrictions that intercept upstream catchments or attenuate peak flows.

For the Marillana Creek Catchment (2,228 km^2), an upper limit percentage reduction of 0.64% was observed, whereas for the Weeli Wolli Creek Catchment (4150 km^2), an upper limit percentage reduction of 0.35% was noted. The changes in peak flow rate, time to peak, and flow volume are not considered to be significant on a regional scale.

Five sediment basin locations have been indicatively provided to intercept sediment from any water which is pumped out of the pits (with an assumed discharge rate of 100L/s), as well as capture any runoff from disturbed surfaces (within reason) before the runoff discharges to the creeks

The findings of this report will be utilised by BHP to undertake an environmental impact assessment in considering the key environmental factors relating to the Environmental Protection Authority (EPA) guidance on Inland Waters, the objective for which is to maintain the hydrological regimes and quality of groundwater and surface water so that environmental values are protected.

11 References

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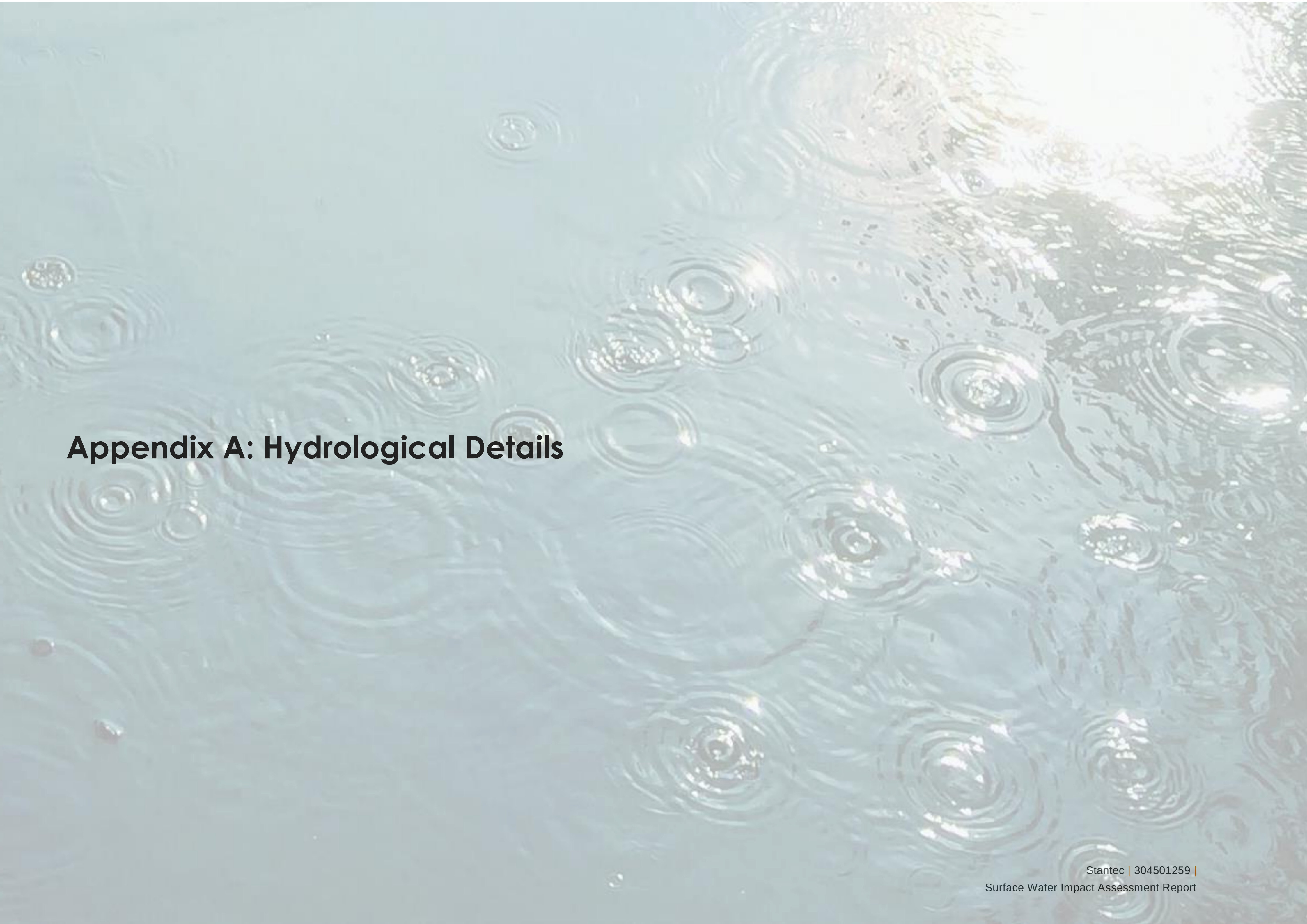


Appendix A: Hydrological Details

A.1 Hydrological Details

The following sections outline key inputs to the hydrological modelling.

A.1.1 Intensity Frequency Duration (IFD) Data

The IFD data, associated temporal patterns and losses were downloaded from the ARR2019 QGIS ARR plugin tool and validated against the ARR data hub for the specified coordinates (-22.8125, 119.1125).

Standard and non-standard durations rainfall depths outlined in Table A-1 were used, ranging from 10 minutes to 72 hours.

A-1 Design Rainfall Depths for Ministers North

	Annual Exceedance Probability (AEP)						
Duration	50%	20%	10%	5%	2%	1%	0.1%
10 min	11.5	16.5	20	23.4	27.9	31.3	47.2
15 min	14.6	20.9	25.3	29.6	35.3	39.6	59.7
20 min	16.9	24.2	29.3	34.2	40.7	45.6	68.7
25 min	18.7	26.8	32.4	37.8	44.9	50.4	75.8
30 min	20.2	29	35	40.8	48.5	54.4	81.8
45 min	23.7	33.9	40.9	47.7	56.8	63.8	95.7
1 hour	26.3	37.6	45.4	53	63.3	71.2	107
1.5 hour	30.1	43.3	52.5	61.5	73.8	83.4	125
2 hour	33.1	47.9	58.2	68.5	82.7	93.9	141
3 hour	37.8	55.3	67.7	80.3	98	112	169
4.5 hour	43.1	64	79.3	95	117	135	204
6 hour	47.3	71.2	88.9	107	133	155	234
9 hour	53.9	82.6	104	127	160	187	283
12 hour	59	91.5	116	143	181	212	321
18 hour	66.4	104	134	167	211	248	374
24 hour	71.7	114	147	183	231	271	407
30 hour	75.7	120	155	194	245	286	430
36 hour	78.8	125	162	202	254	297	443
48 hour	83.3	132	171	213	266	308	454
72 hour	89	140	180	224	275	315	457

A.1.2 Spatial Variability

The spatial variability of rainfall has been assessed to determine applicability of point IFD data given the catchment size exceeds 20km^2. The four locations across the three local catchments checked for spatial variance include:

1. -22.729175, 119.040846
2. -22.755320, 119.051532
3. -22.796500, 119.077430
4. -22.809980, 119.112475

The rainfall depth from the IFD data for each event and duration at each location were compared against one another. This assessment showed that there was a coefficient of variance (CV) between the relevant rainfall depths of less than 3% and so it can be concluded that this catchment does not require the application of spatially variant rainfall.

A.1.3 Temporal Patterns

Temporal patterns for the Rangelands West zone were utilised for the hydrological assessment. Areal temporal patterns were initially tested for the Yandicoogina Creek catchment which exceeds 75km^2.

However, given areal temporal patterns are only available for the 12-hour to 72-hour events and critical durations for the local catchments are typically below 12 hours, point temporal patterns were adopted for all local models.

A.1.4 Filtered Embedded Bursts

Embedded bursts within temporal patterns that have an AEP rarer than the burst as a whole have been removed for all sensitivity analyses other than those based on the Tarina FFA in the Baby Hope EPA study via the built-in RORB filtering technique that has been developed by *Scorah et al.* (2019).

A.1.5 Losses

There were no regional losses available from the ARR data hub. As such, a sensitivity analysis on a range of loss assumptions was undertaken.

The Initial Loss/Continuing Loss (IL/CL) model methodology has been applied in keeping with ARR2019 guidance. Given the catchment is entirely pervious in the predeveloped scenario and the proportion of proposed impervious areas in the LoM condition is dwarfed by the remaining pervious catchment areas, there was no variability in effective impervious areas applied.

There is limited local gauge information available downstream of the development envelope for the Yandicoogina Creek catchment as outlined in Section 1.5.2. The regional gauges outlined in Section 1.5.1, other than the Tarina gauge are influenced by the Munjina Claypan.

As such, three loss methodologies including consideration of Tarina gauge calibration data was tested in attempt to improve parameter defensibility. These include:

1. **Flavell (2012) loss estimation with a constant 40mm initial loss and 5mm/h continuing loss** as per 2018 Stantec assessment parameters summarised in Table A-2 below.
    a. The key limitation of this approach is that validating against regional methods provides limited defensibility and modifying k_c alone does not improve correlation at each event.

A-2 Adopted loss rates for Flavell Method in 2018 Ministers North Catchments

AEP	Initial Loss (mm)	Continuing Loss (mm/h)
50%	40	5
20%	40	5
10%	40	5
5%	40	5
2%	40	5
1%	40	5
0.1%	5	1.5
0.01%	1	0.4

2. **Baby Hope EPA study (2014) Tarina FFA non-neutral initial loss rates and a constant continuing loss of 9mm/h** as per Table 4-12 of the report. An extract is illustrated below.
    a. The key limitation of this study is the use of CRC-Forge data and ARR1987 rainfall and temporal patterns in the calibration, as well as the fitting to peak flow rates for calibration of loss values only despite hydrograph fitting of historical events for k_c determination.
    b. As initial loss varies with antecedent conditions it was considered the most appropriate parameter to vary for the calibration of design rainfall to the FFA. This results in underestimated frequent events and overestimated infrequent events when compared to regional validation methods.

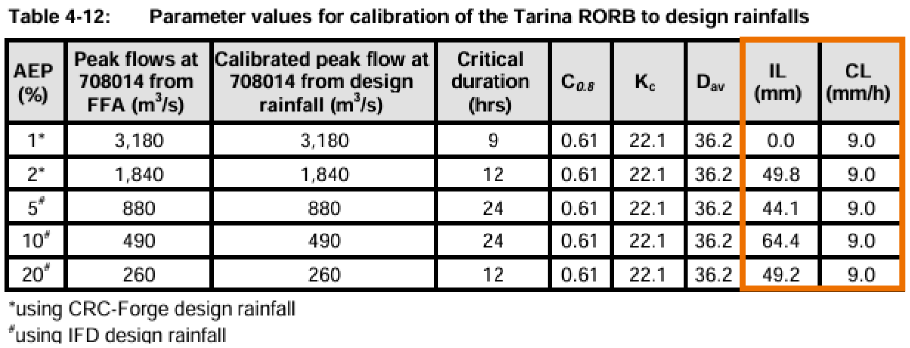


Table 4-12: Parameter values for calibration of the Tarina RORB to design rainfalls

AEP (%)	Peak flows at 708014 from FFA (m^3/s)	Calibrated peak flow at 708014 from design rainfall (m^3/s)	Critical duration (hrs)	$C_{0.8}$	K_c	D_{av}	IL (mm)	CL (mm/h)
1*	3,180	3,180	9	0.61	22.1	36.2	0.0	9.0
2*	1,840	1,840	12	0.61	22.1	36.2	49.8	9.0
5#	880	880	24	0.61	22.1	36.2	44.1	9.0
10#	490	490	24	0.61	22.1	36.2	64.4	9.0
20#	260	260	12	0.61	22.1	36.2	49.2	9.0

*using CRC-Forge design rainfall
#using IFD design rainfall

Baby Hope EPA Tarina FFA Calibration Parameters 2014 (Table 4-12)

3. **<u>Adopted Loss Approach:</u> Non-neutral initial loss values based on IFD ratios, continuing loss of 6 mm/h based on local gauge frequent event validation and peak flow correlation to median value of six regional estimation methods outlined in Section 3.9. Adopted values are summarised in the Table A-4 opposite.**
    a. The key limitation of regional estimate validation is the fitting to peak flow rates and not hydrographs and the large variability between various regional estimation methods. This limitation was addressed by further validation of RORB results to HEC-RAS 2D results.
    b. An initial sensitivity analysis was conducted on varying continuing loss values (between 5-9 mm) as well as initial losses to match the median value of six regional estimation methods. This was then fixed to 6 mm/h based on the outcomes of the local Yandicoogina Creek Rocks gauge analysis in Section 1.5.2.
    c. An initial sensitivity analysis was conducted on varying initial loss values based on regional estimate comparisons only. This resulted in the overall range of tested initial loss values of between 0 to 90 mm.
    d. Initial loss values were then scaled by IFD depth ratios outlined below for consecutive event loss based on discussion with BHP, which resulted in between 20 to 25 mm being adopted for the 63.2% AEP (1 EY) event, with a range of 70 to 90 mm for the 1% AEP.

A-3 IFD Ratios to Inform Non-Neutrality of Initial Loss

AEP	Consecutive IFD Ratio	Resulting Factor to 63.2% AEP	Adopted Initial Loss
63.30%	0	1	22
50%	0.17	1.17	26
20%	0.49	1.74	38
10%	0.24	2.16	48
5%	0.2	2.59	57
2%	0.21	3.14	69
1%	0.14	3.58	79

    e. Additional sensitivity analysis was conducted in line with the assumption that initial loss peaks at the 10% AEP and reduces to zero for the 1% AEP (EPA Tarina Calibration, 2014). This produced 1% AEP event peak flow rates significantly higher than the regional estimates. Considering also the RORB results are on average 10% higher than the HEC-RAS 2D results outlined in Section 6, <u>it appears reasonable not to reduce initial losses based on antecedent conditions till the 1 in 1,000 AEP for this catchment</u>.
    f. The loss value for the 1 in 10,000 AEP climate change scenario is unable to be informed by the current *Draft* ARR2019 guidance. As such, saturated antecedent soil conditions are assumed for initial loss conservatism.

A-4 Variable IL based on IFD ratio and CL based on local gauge validation

	Yandicoogina Creek		Mungadoo Tributary		Central Tributary	
AEP	IL (mm)	CL (mm/h	IL (mm)	CL (mm/h	IL (mm)	CL (mm/h
63.2%	22	6	22	6	22	6
50%	26	6	26	6	26	6
20%	38	6	38	6	38	6
10%	48	6	48	6	48	6
5%	57	6	57	6	57	6
2%	57	6	57	6	57	6
1%	57	6	57	6	57	6
1 in 10,000	0	6	0	6	0	6
PMP	0	6	0	6	0	6

A.1.6 Pre-burst

Application of median pre-burst rainfall was tested as part of the sensitivity analysis as per the process outlined in ARR 2019 using the in-built functionality of RORB. Median pre-burst data downloaded from the ARR 2019 Data Hub was read into RORB and applied in a single increment prior to the design storm. Median pre-burst values were adopted as they are the most defendable values without any additional guidance currently available in literature.

The time increments were manually set to be compatible with the imported temporal pattern increments. Given that pre-burst ratios for durations of less than one hour are not available from the ARR 2019 Data Hub, ratios for the one-hour duration were adopted for all durations of less than one hour.

The two scenarios tested for pre-burst include those tested with and those tested without. The application of pre-burst increased the peak flow rates as is to be expected. It is also decreased the critical duration by a single duration in some instances, representative of an earlier peak.

The adopted pre-burst ratios for the catchments is outlined below.

A-5 Adopted Pre-burst Ratios for Sensitivity Analysis

Duration	50%	20%	10%	5%	2%	1%
10 min	2.3	2.6	2.7	2.7	3.4	3.7
15 min	2.3	2.6	2.7	2.7	3.4	3.7
20 min	2.3	2.6	2.7	2.7	3.4	3.7
25 min	2.3	2.6	2.7	2.7	3.4	3.7
30 min	2.3	2.6	2.7	2.7	3.4	3.7
45 min	2.3	2.6	2.7	2.7	3.4	3.7
1 hour	2.3	2.6	2.7	2.7	3.4	3.7
1.5 hour	2.2	2.3	2.3	2.3	3.1	3.6
2 hour	4	2.5	2	1.6	2.9	3.7
3 hour	0.1	2.5	3.3	3.8	8.5	10.9
4.5 hour	0	2.1	2.8	3.2	16.9	23.9
6 hour	0	2.1	2.8	3.2	16.9	23.9
9 hour	0	2.6	3.5	3.9	6.8	8.2
12 hour	0	2.6	3.5	3.9	6.8	8.2
18 hour	0	0.9	1.2	1.3	5.1	6.9
24 hour	0	0.6	0.7	0.8	3	4
30 hour	0	0	0	0	0.9	1.3
36 hour	0	0	0	0	0.9	1.3
48 hour	0	0	0	0	0	0
72 hour	0	0	0	0	0	0

A.1.7 Areal Reduction Factor

Areal Reduction Factors (ARF) account for the fact that design rainfall intensities at a point are not representative of the areal average rainfall intensity across the catchment. ARF is the ratio between the design values of areal average rainfall and point rainfall, computed for the same duration and AEP.

This allows for the fact that larger catchments are less likely than smaller catchments to experience high intensity storms simultaneously over the whole of the catchment area.

Northern Coastal ARF Parameters were downloaded from the ARR data hub and applied for the total catchment area of the local RORB models.

A.1.8 K_c Determination

In RORB modelling, k_c is a parameter that is used to model the runoff generation and flow routing of a catchment. It is considered a fixed parameter but has been found to exhibit a high degree of variability.

Due to the additional uncertainty in modelling ungauged catchments, sensitivity analysis has been conducted for three k_c methods (where m=0.8) including:

1. K_c/d_{av} ratio as per Pearcey et al. (2014) = 0.59
2. K_c/d_{av} ratio as per Tarina FFA calibration by EPA (2014) = 0.61
3. K_c as per Dyer (1994) Pearse et al (2012) calculation = $1.14*d_{av}$

The regional relationship developed for Pilbara catchments by Pearcey et al. (2014) and McMahon and Muller (1983) which showed that k_c is directly proportional to d_{av} by a ratio of $C_{0.8}$. This method was used for each of the catchments for the sensitivity analysis.

The final adopted k_c parameters are summarised below.

A-6 Kc Parameters

Catchment	Average Reach Length (d_{av})	K_c
Yandicoogina Creek	14.43	8.51
Central Tributary	8.38	4.94
Mungadoo Tributary	4.5	2.66

A.2 Regional Methods Background

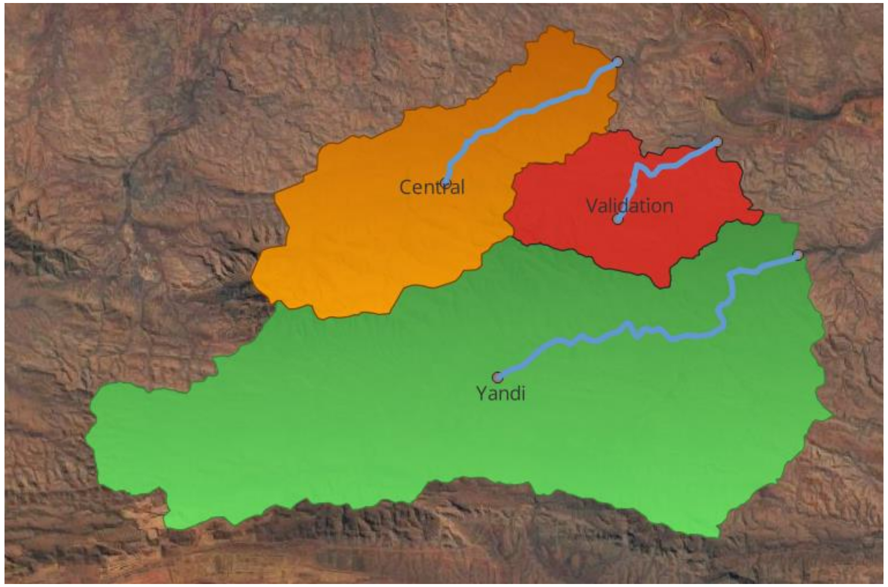
Several methods of peak flow estimation specific to the Pilbara region were compared for validation as summarised in Section 3.9. The methods are described in the sections to follow.

A.2.1 Regional Methods Catchment Data

The catchment data required to calculate the regional estimation peak flows are shown below and opposite.

A-7 Catchment Characteristics

ID	Area (km2)	Long (centroid)	Lat (centroid)	Length from Centroid (m)	Slope (m/Km)
Yandicoogina	148.43	119.0600058	-22.86290576	12837.23	7.41
Validation	24.26	119.0960391	-22.8174392	4887.51	20.3
Central	50.39	119.0430136	-22.80769676	7173.01	15.8



A-8 Regional Estimation Catchments

A.2.2 ARR87 Index Flood Method (IEA, 1987) – Catchments in Northwest Pilbara Region

The index flood method (IFM) was developed based on regression analysis of flood frequency curves derived for many locations across WA with the assumption of homogeneous regions. Most of the regressions for the runoff coefficient for the IFM are based on 2 or 5-year AEP flood data.

Section 1.4.7 of ARR87 Book 4 notes that the Index Flood Method for the Northwest Pilbara Region of WA with the peak discharge for AEP of 5 years (Q5) as the index variable.

A.2.3 ARR87 Regional Rational Method (IEA, 1987) – Catchments in Northwest Pilbara Region

The Regional Rational Method (RRM) uses a probabilistic or statistical method in estimating peak flow of selected ARIs from an average rainfall intensity of the same AEP derived from the IFD design curves for any location in Australia as described in ARR87 Book 2 Section 1. The RRM incorporates a runoff coefficient, and the catchment and rainfall characteristics in the design peak flow estimation.

Section 1.4.7 of ARR87 Book 4 notes that the Rational Method for the Northwest Pilbara Region of WA with the runoff coefficient for AEP of 2 years (C2) as the index variable. The RRM is most conservative of the regional estimates.

A.2.4 Regional Flood Frequency Procedure (Flavell, 2012) – Pilbara Region

The Regional Flood Frequency Procedure (RFFP) is based on streamflow data for approximately twice the length of record of that used to develop the methods in ARR87. The RFFP has been checked against flows estimated from flood debris levels and recorded rainfall data and found be within an acceptable order of magnitude.

The RFFP was initially developed for the Pilbara prior to 2000 and was redeveloped in 2006 for 15 catchments with longer stream flow records available for validation of the procedure.

The RFFP 2000 method has been recommended as the preferred procedure for catchments in the Pilbara as it provides a more conservative estimate for the events commonly used for the design drainage structures and includes a shape factor, comparative to the 2006 method.

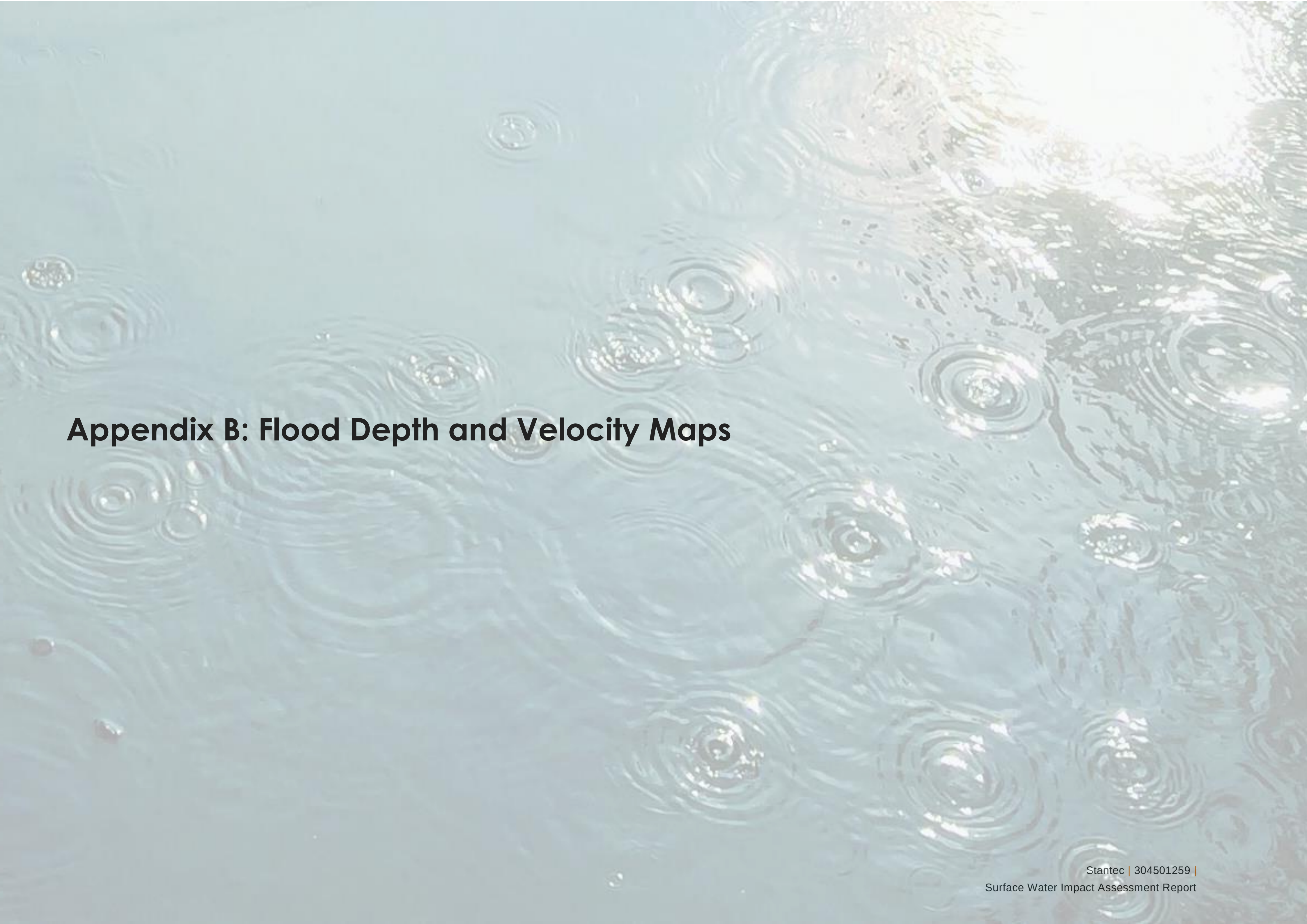
A.2.5 Regional Flood Frequency Estimation Model (RFFE) (ARR, 2019)

The Regional Flood Frequency Estimation Model (RFFE) model provides peak discharge rates by location based on historical gauge records and assigned catchment input values (Engineers Australia, 2016). The Tarina, Weeli Wolli Springs, Flat Rocks, and Waterloo Bore gauges have catchment areas outside the range of applicability for the RFFE model (1,000 km^2 maximum catchment area) and are excluded from the analysis.

The nearest gauge applied in the model is located approximately 150 km west of the site, with a contributing catchment area of approximately 250 km^2 (Hardey River, Gauge 706207). RFFE results include maps of the gauges used in the RFFE analysis along with discharge versus area relationships for gauged catchments in the region.

A.2.6 Pilbara and Gascoyne Regional Flood Frequency Analysis (RFFA) (Davies and Yip, 2014)

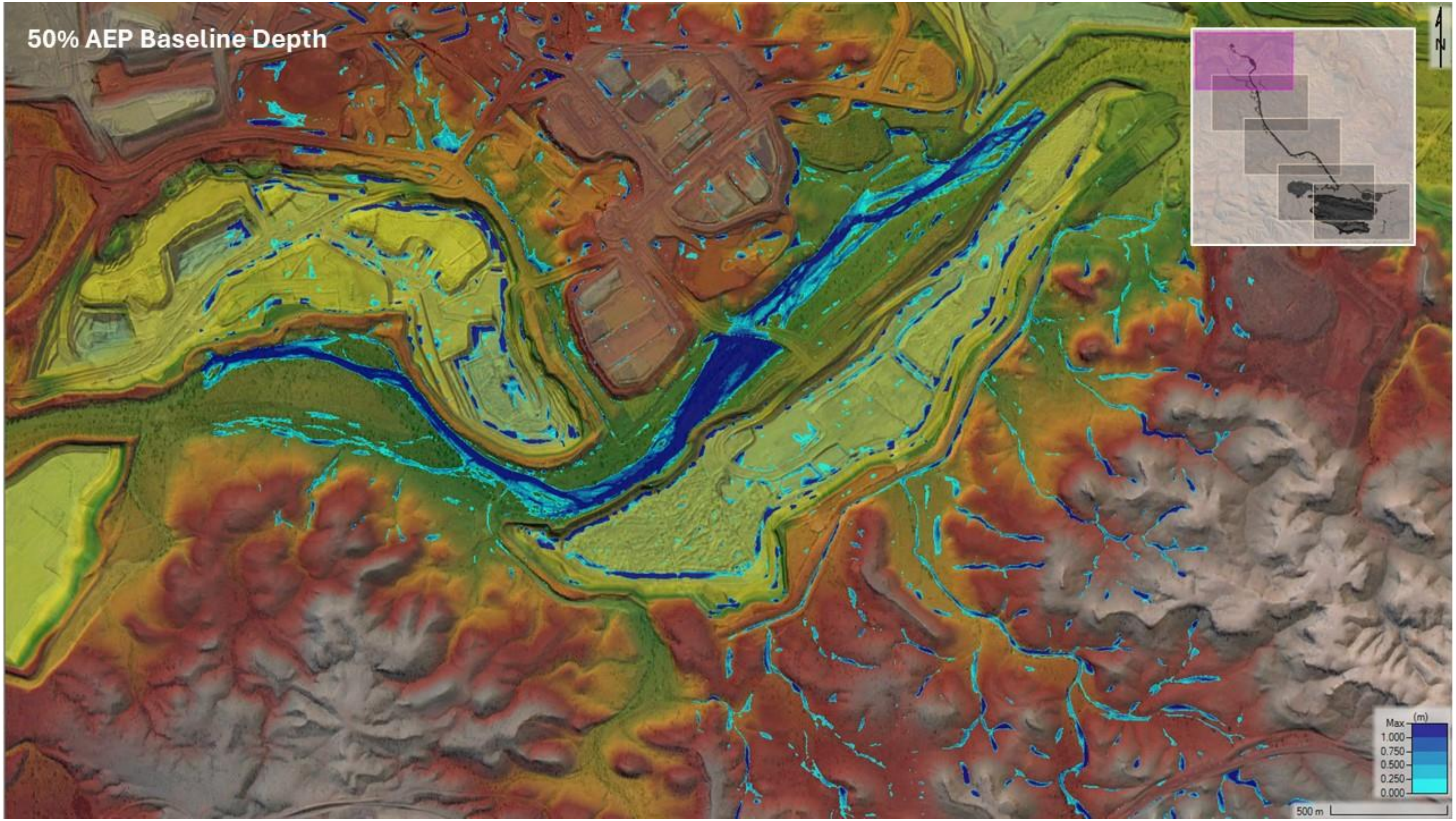
Regional flood frequency analysis (RFFA) is a commonly adopted technique to derive design flood estimates on ungauged catchments (Rahman et.al. 2012) in the Pilbara. The RFFA method attempts to transfer flood characteristic information from a group of gauged catchments to an ungauged catchment of interest. Comparative to other regional estimates, the RFFA is the least conservative.



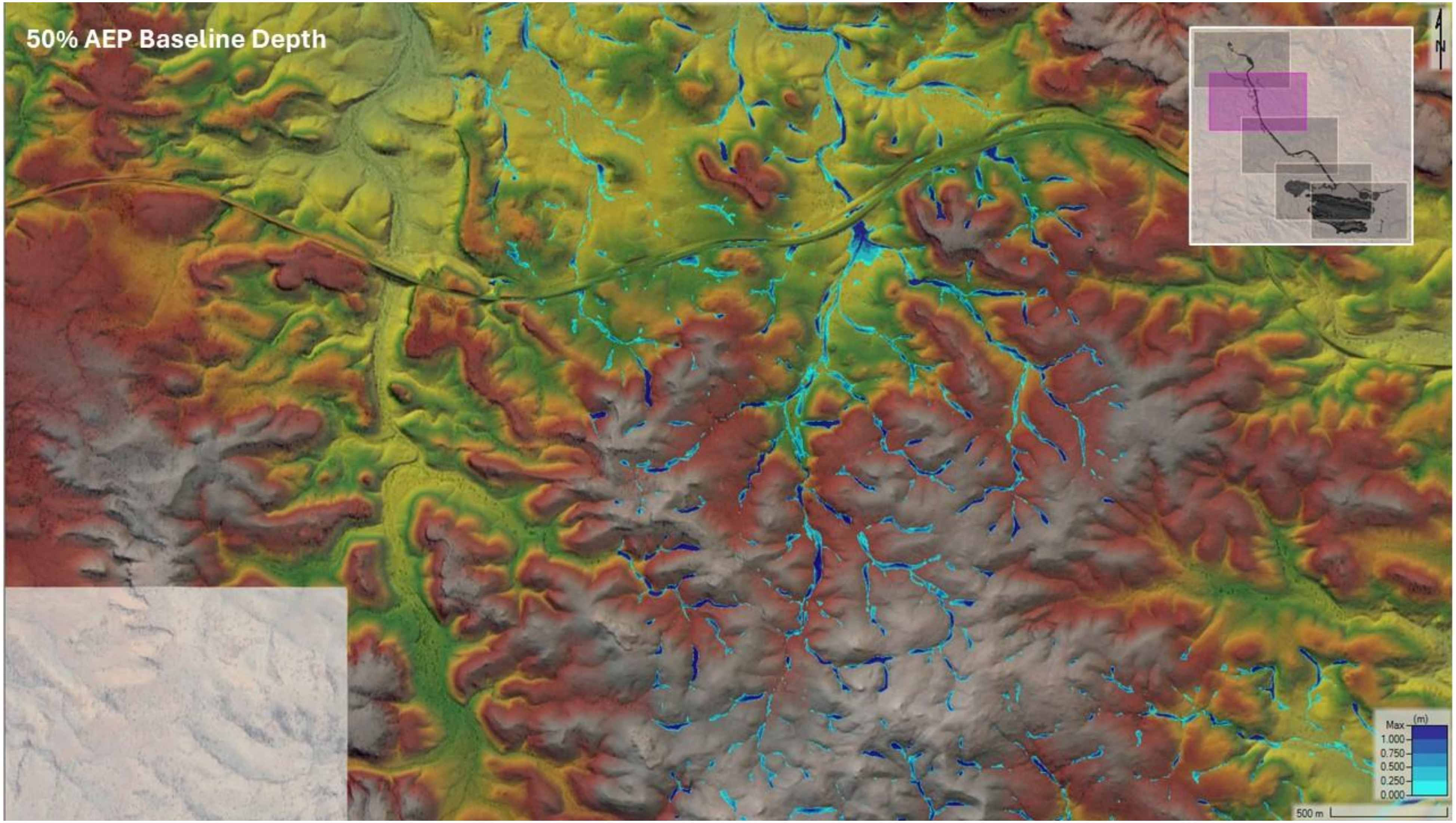
Appendix B: Flood Depth and Velocity Maps

AEP	**Depth**			**Velocity**		
	Appendix B					
	Baseline	LOM	Closure	Baseline	LOM	Closure
50%	Figure B-1 to B-5					
20%						
10%						
5%						
1%	Figure B-6 to B-10	Figure B-11 to B-15	Figure B-16 to B-20	Figure B-21 to B-25B-22	Figure B-26 to B-30	Figure B-31 to B-35
1 in 10,000 CC uplift			Figure B-36 to B-40			

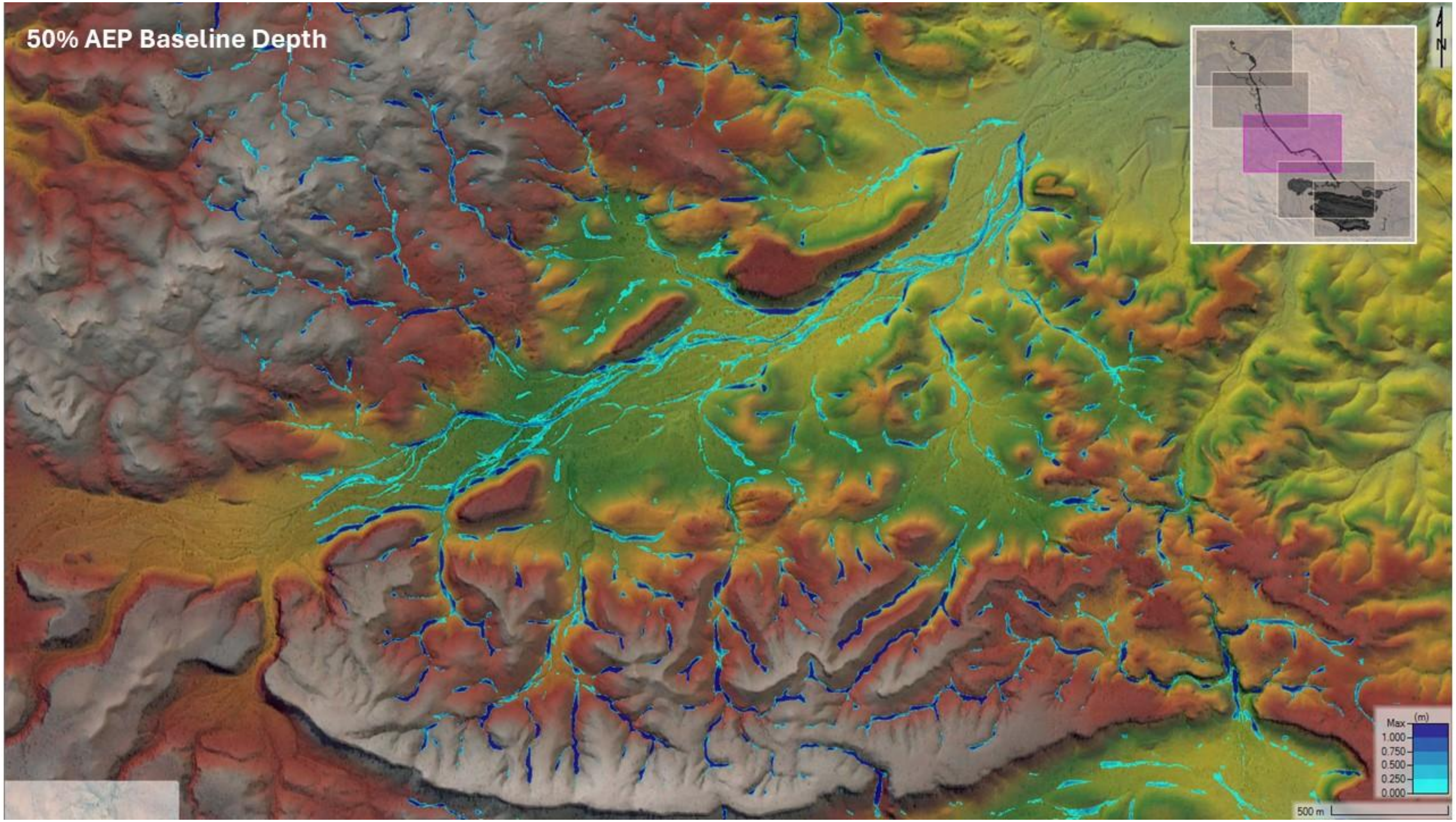
Note: Remaining figures available in accompanying electronic files



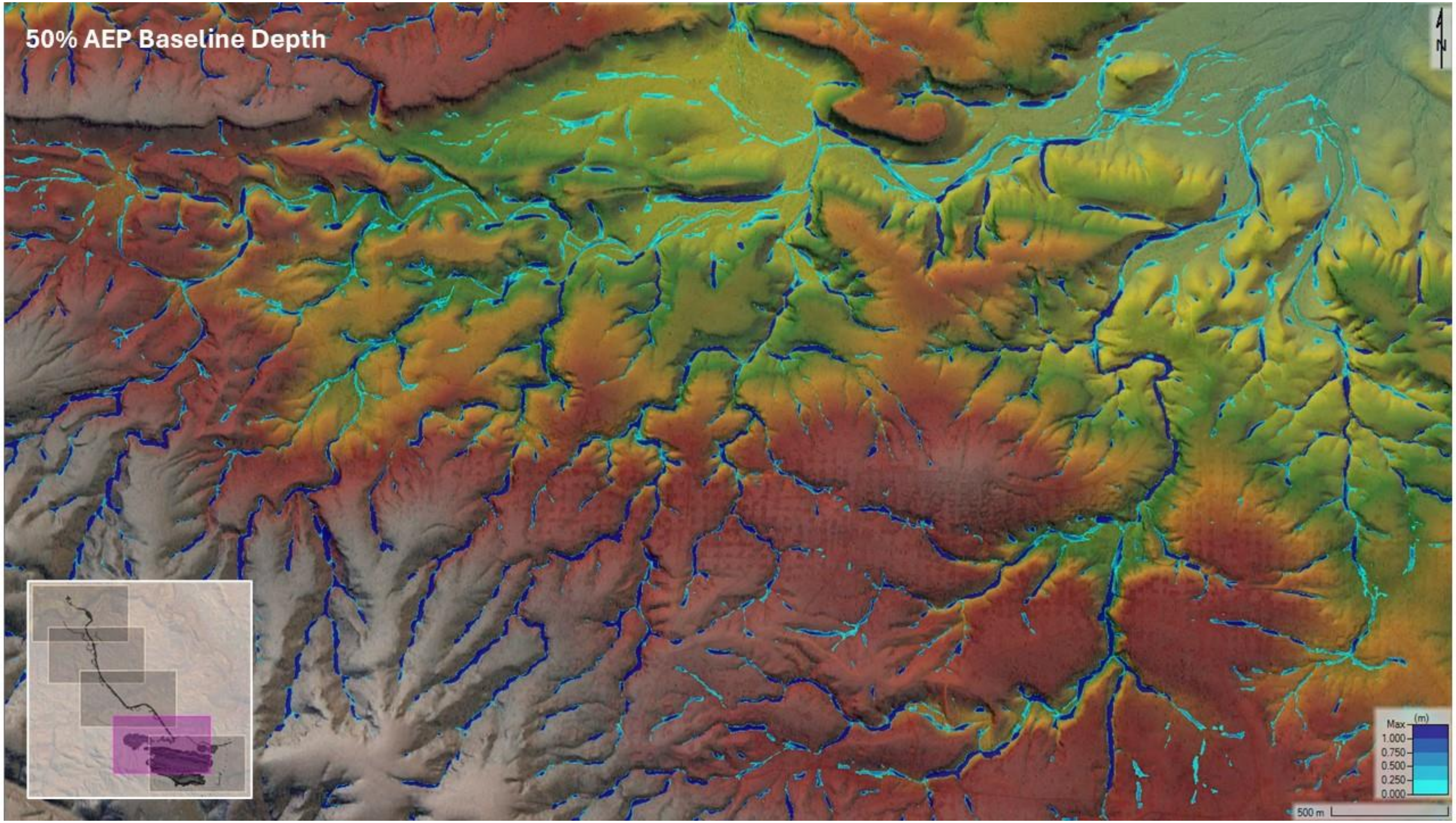
B-1 50% AEP Baseline Depth (Location 1)



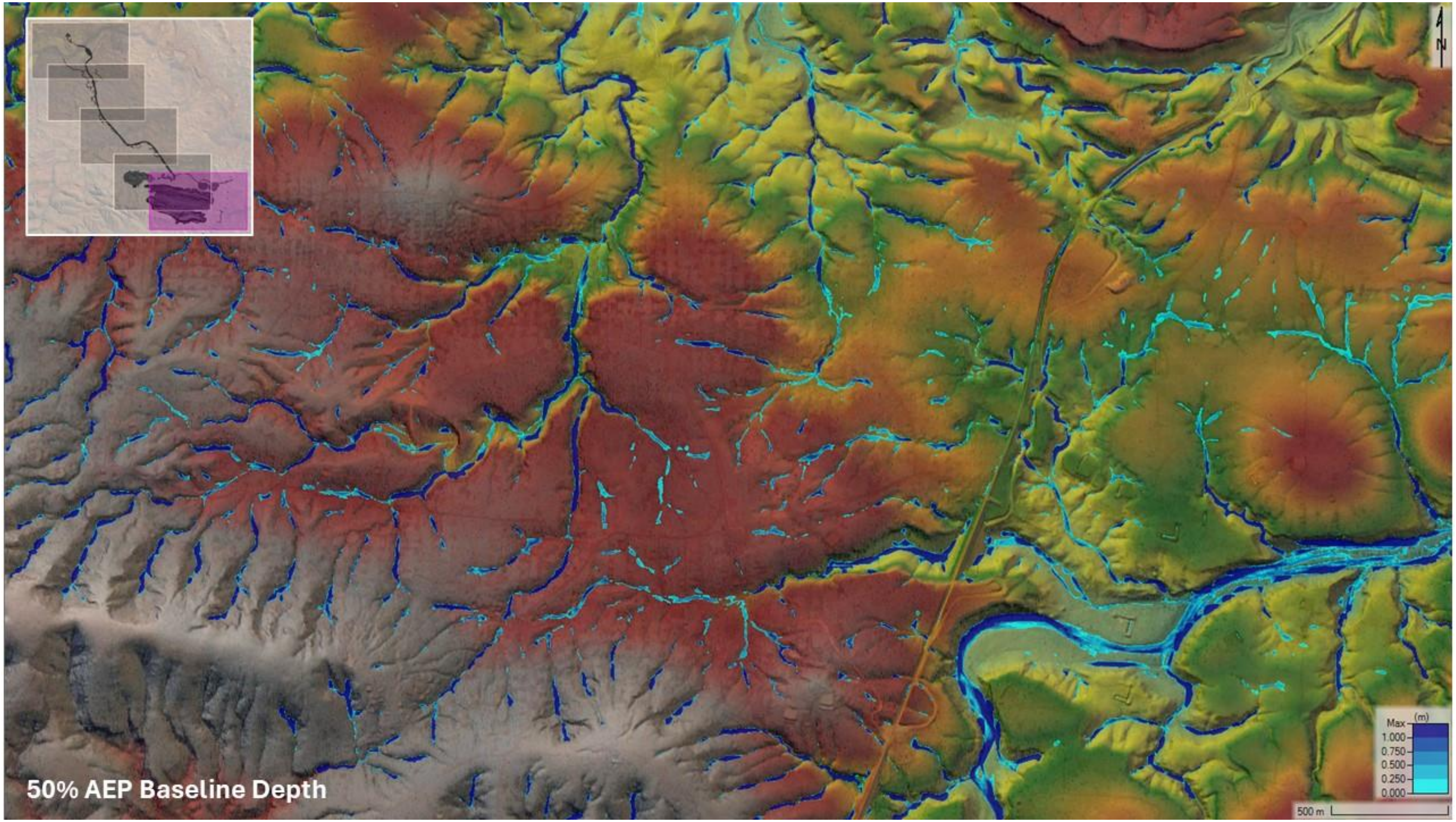
B-2 50% AEP Baseline Depth (Location 2)



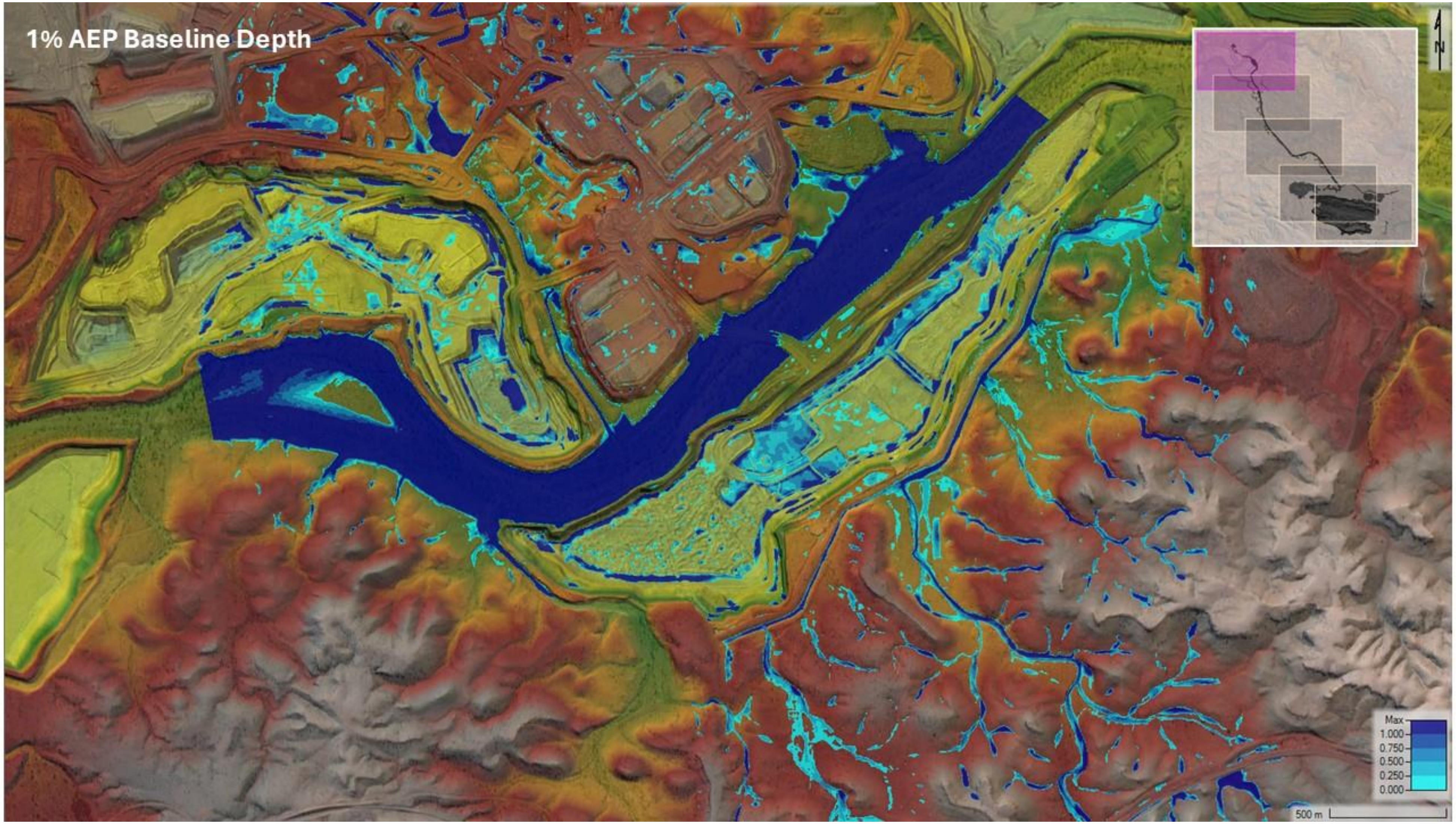
B-3 50% AEP Baseline Depth (Location 3)



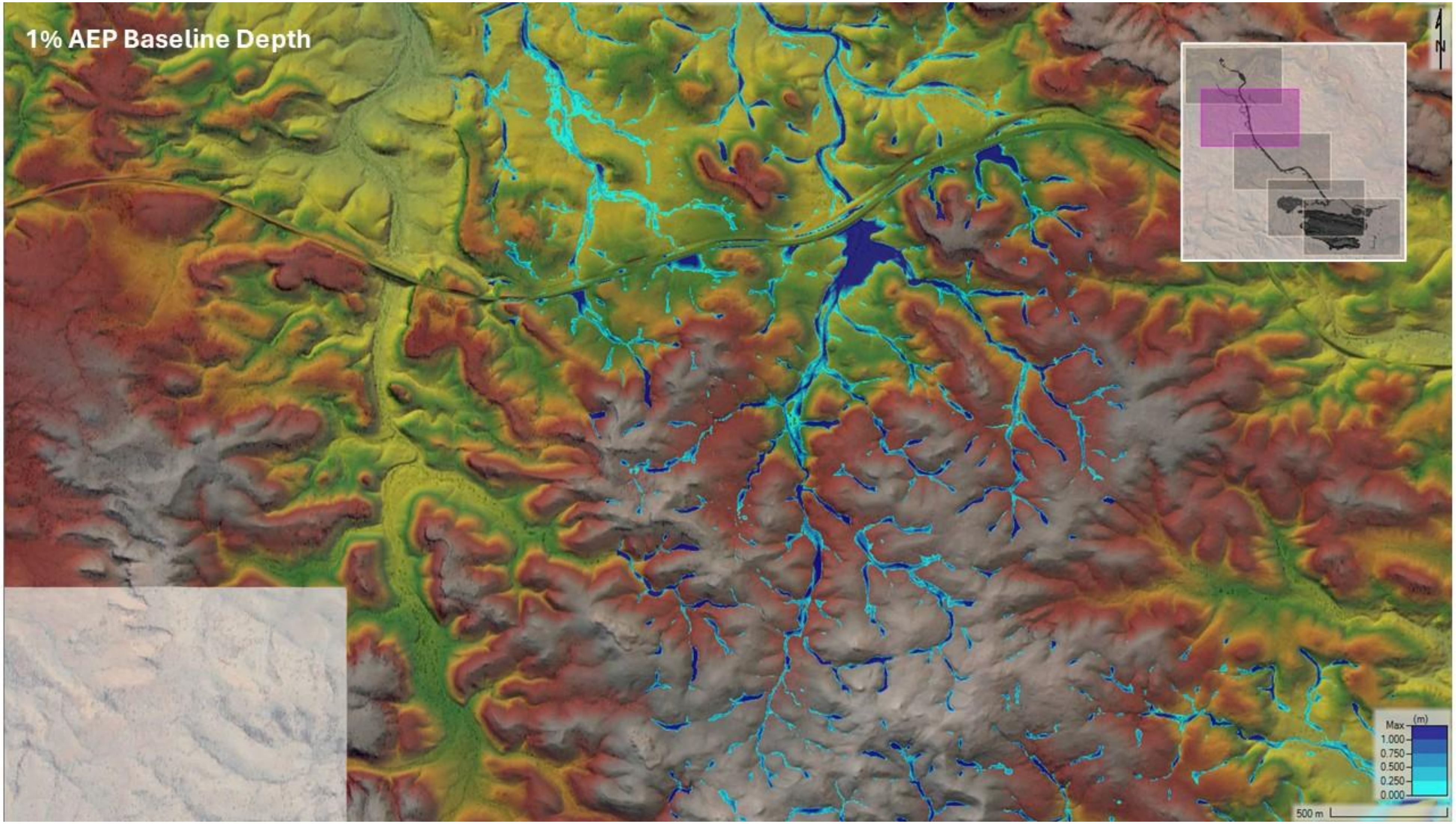
B-4 50% AEP Baseline Depth (Location 4)



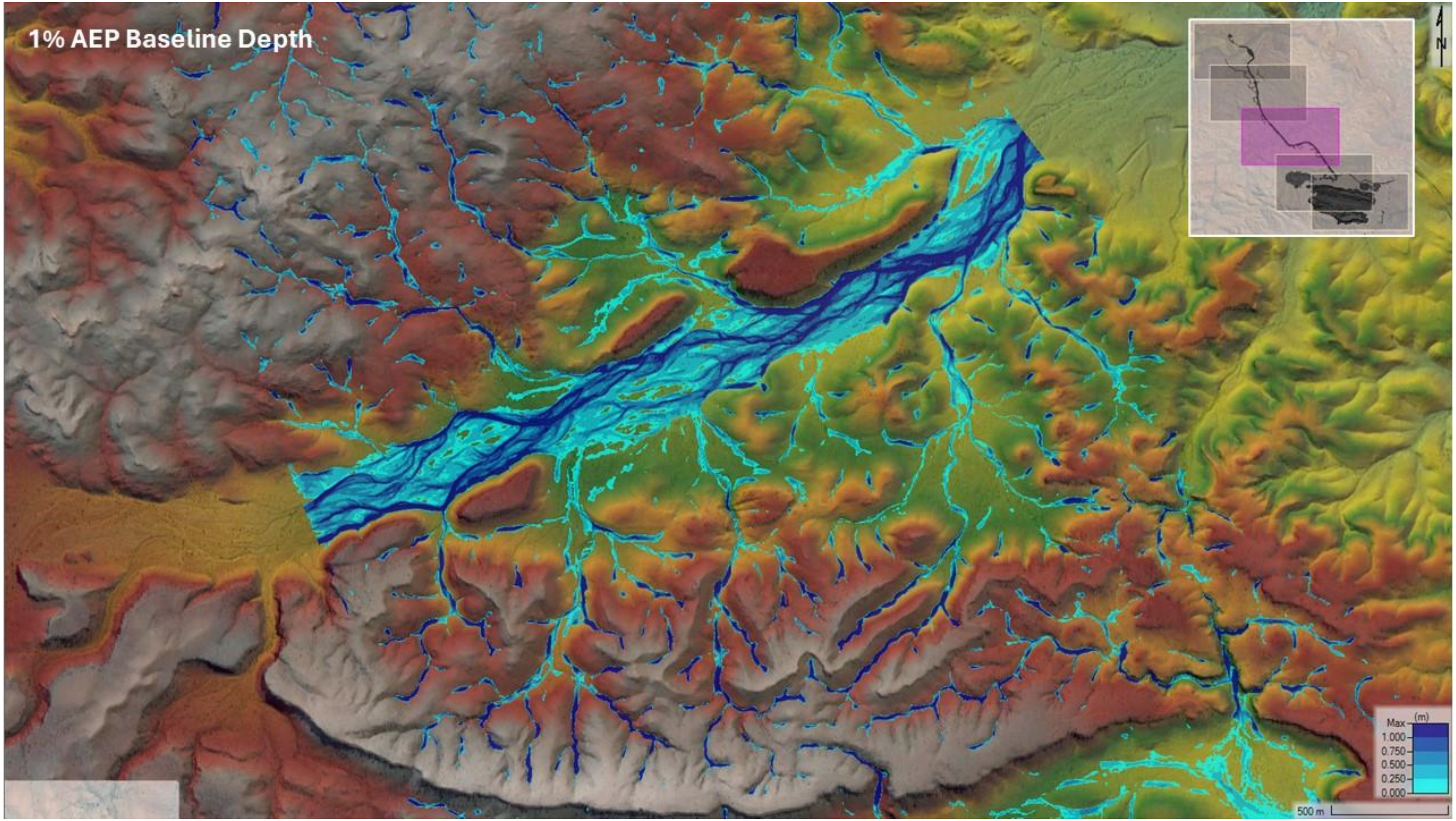
B-5 50% AEP Baseline Depth (Location 5)



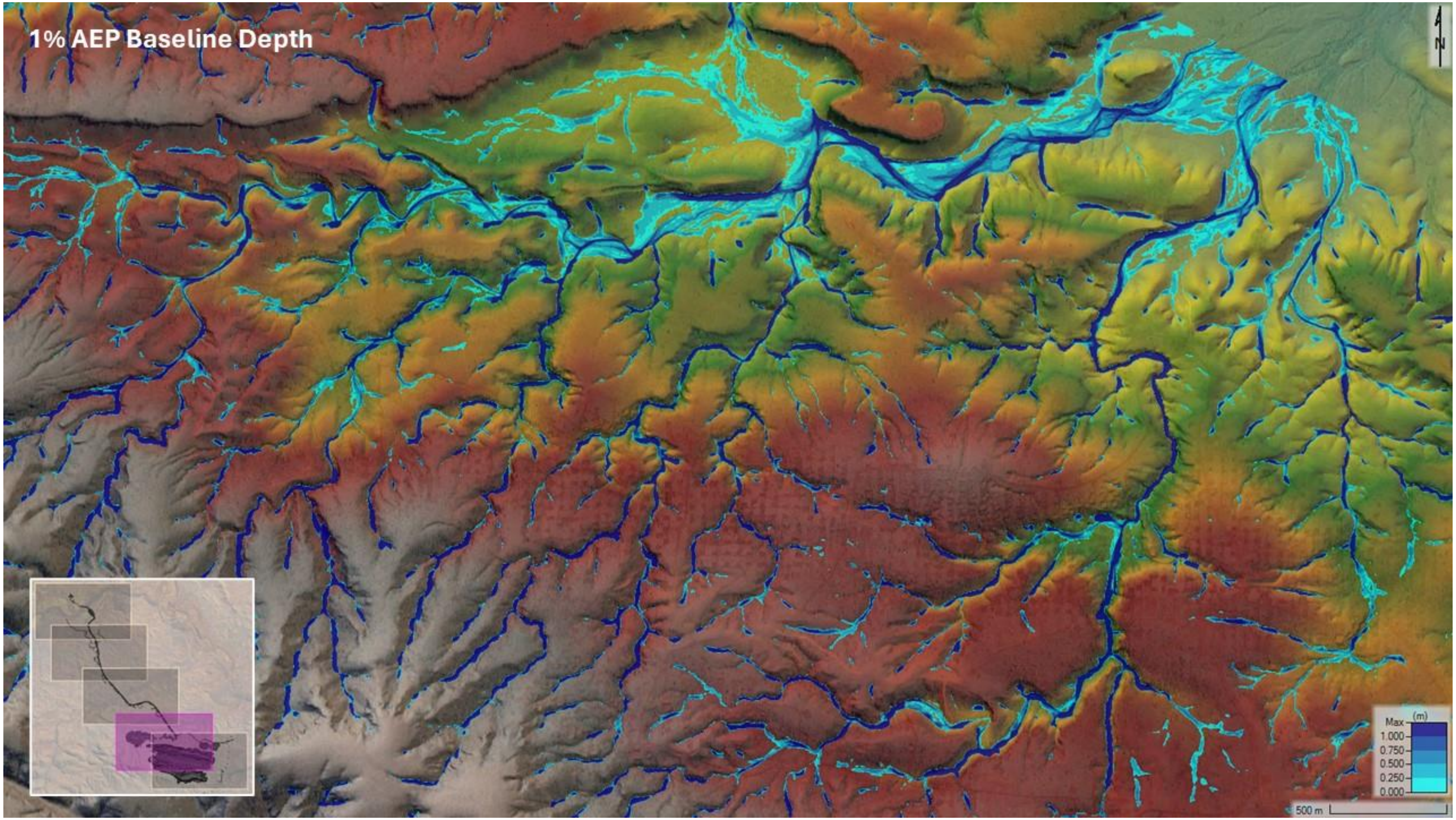
B-6 1% AEP Baseline Depth (Location 1)



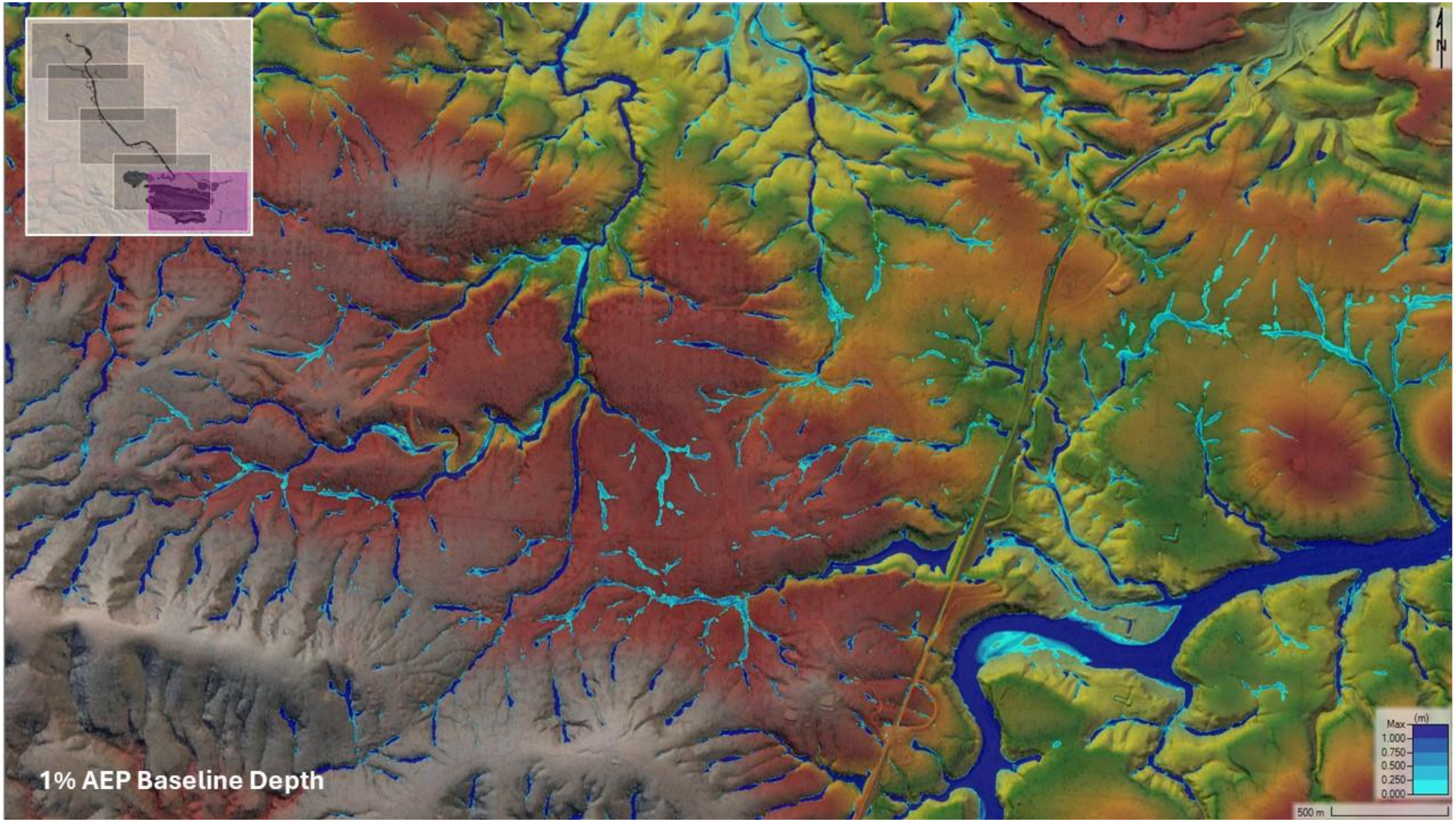
B-7 1% AEP Baseline Depth (Location 2)



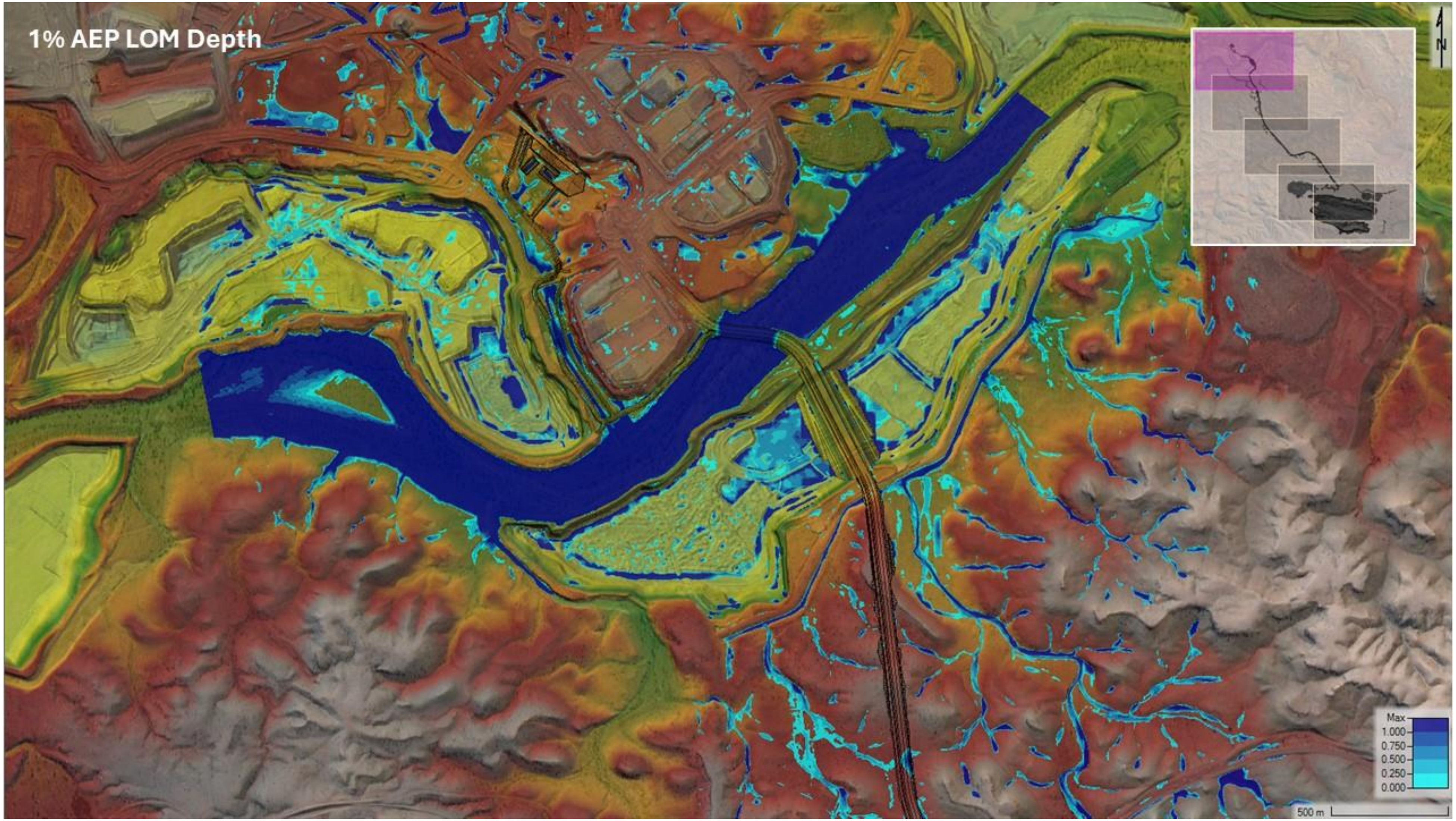
B-8 1% AEP Baseline Depth (Location 3)



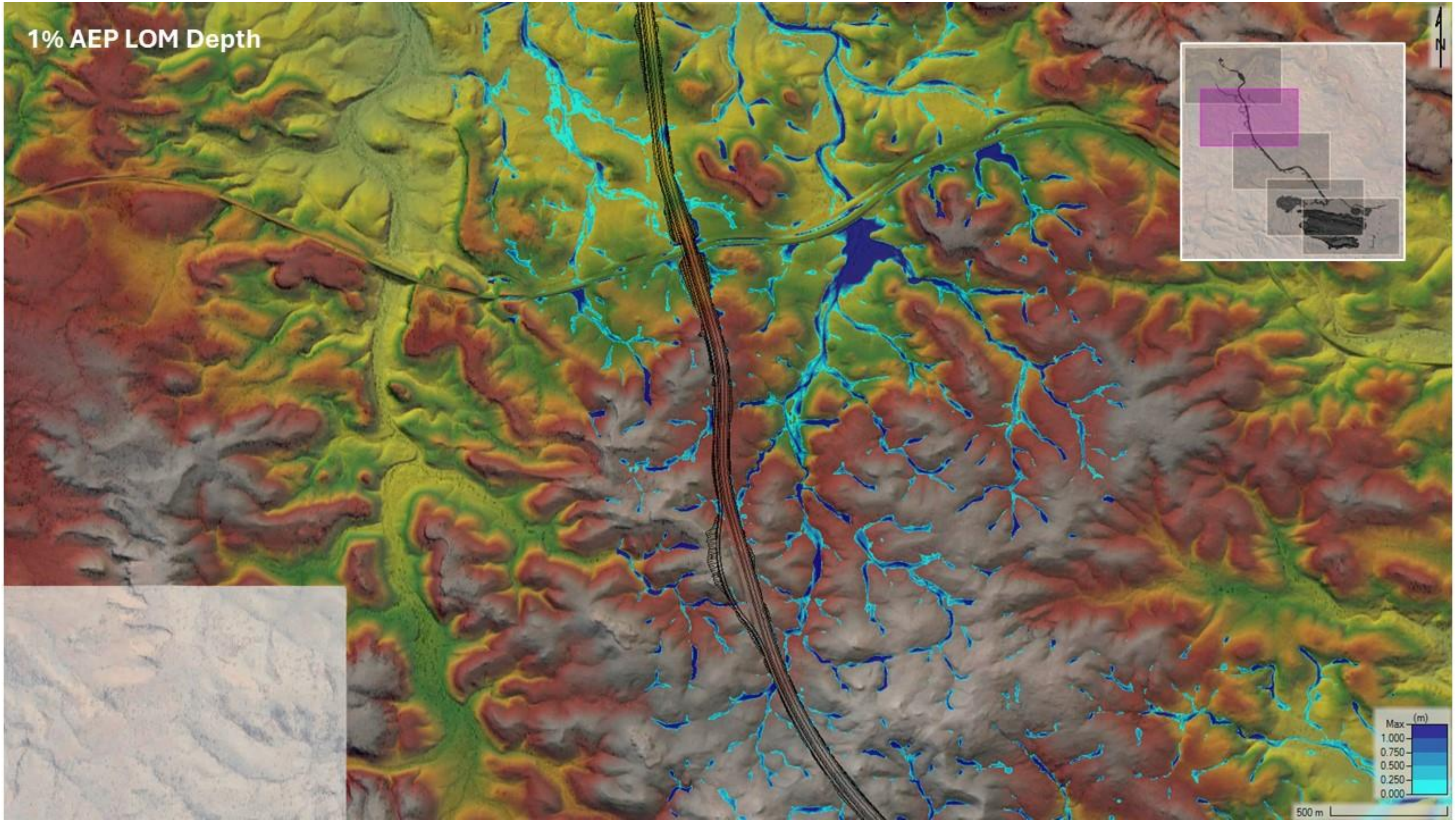
B-9 1% AEP Baseline Depth (Location 4)



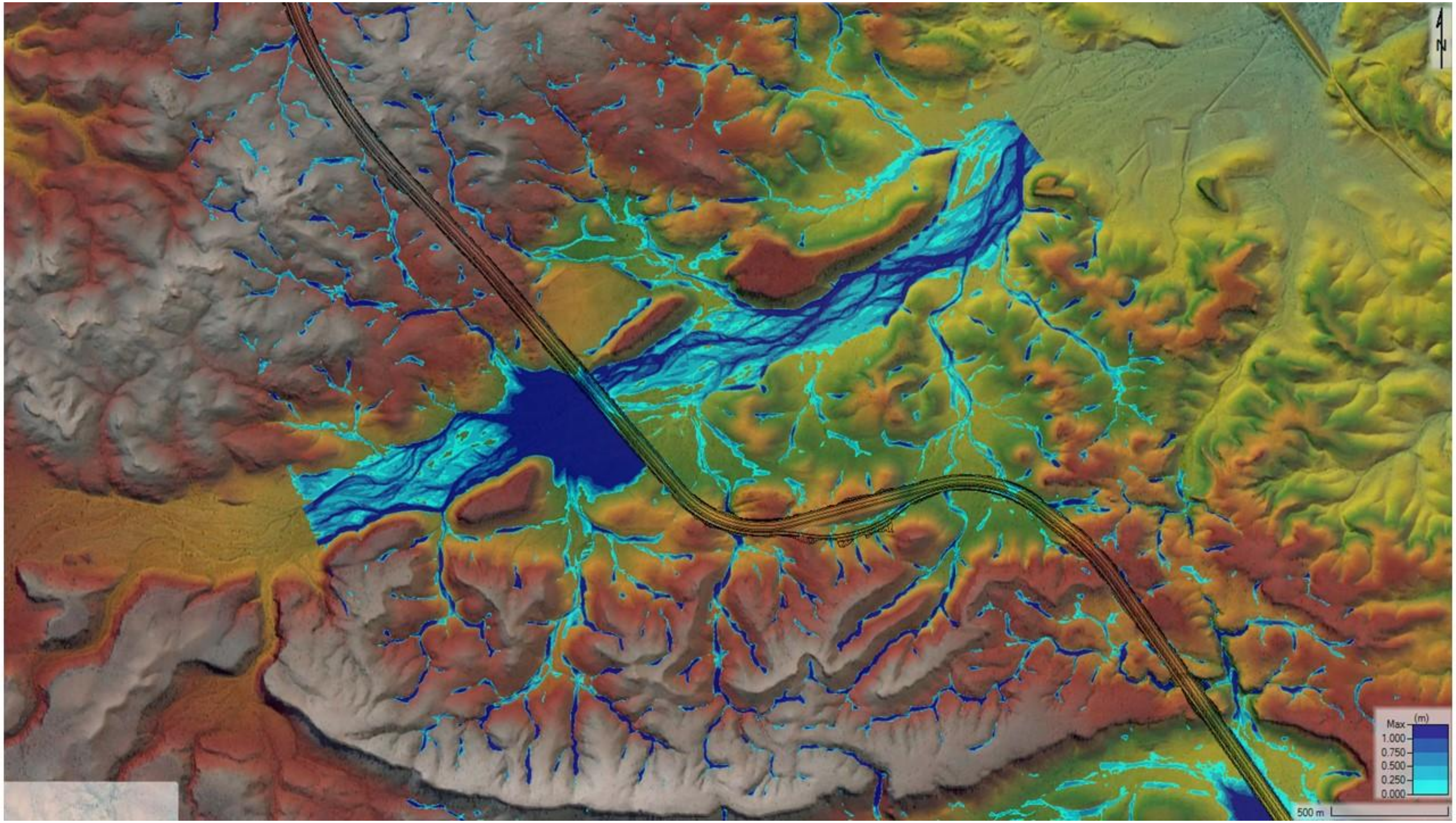
B-10 1% AEP Baseline Depth (Location 5)



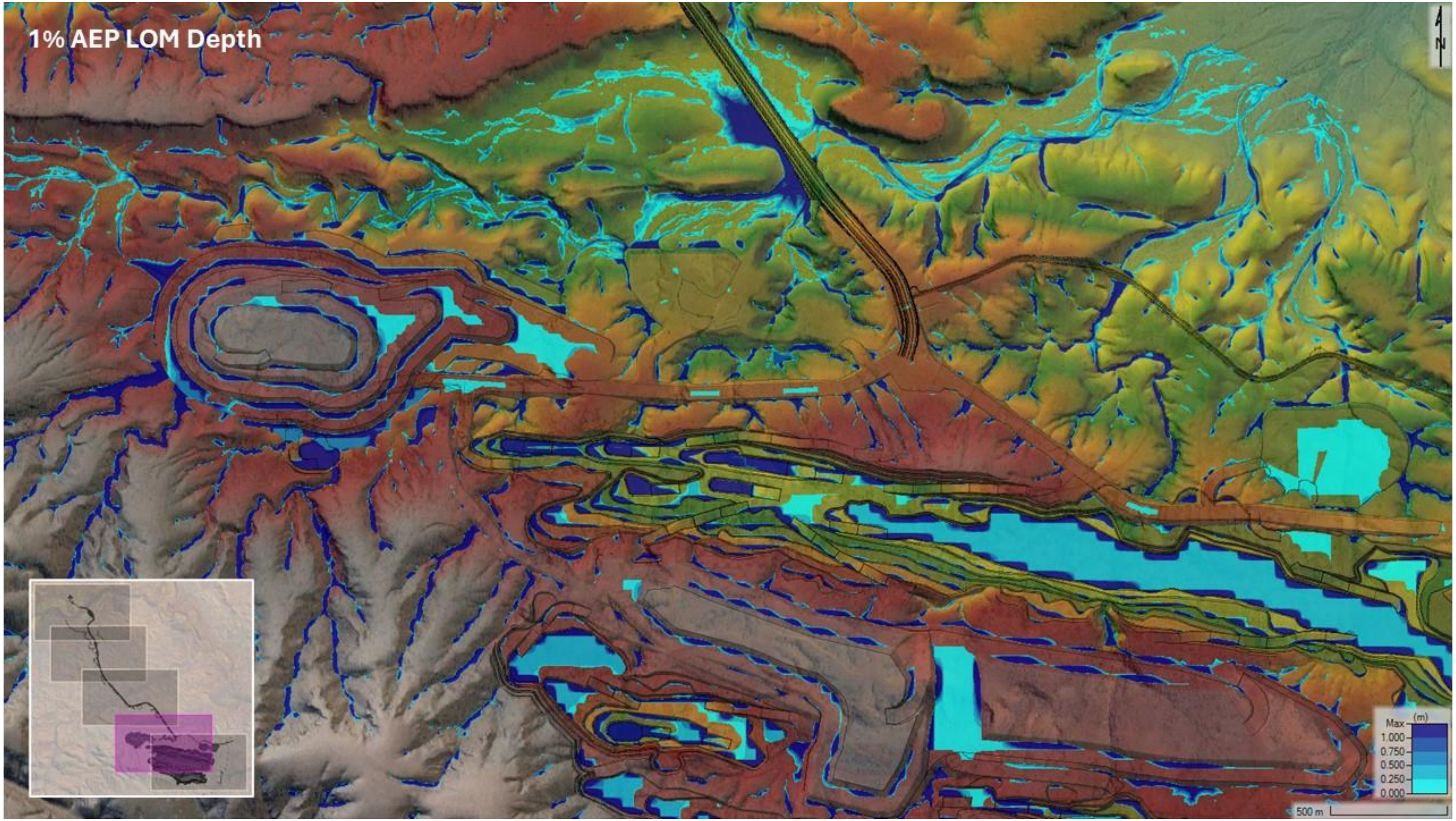
B-11 1% AEP LOM Depth (Location 1)



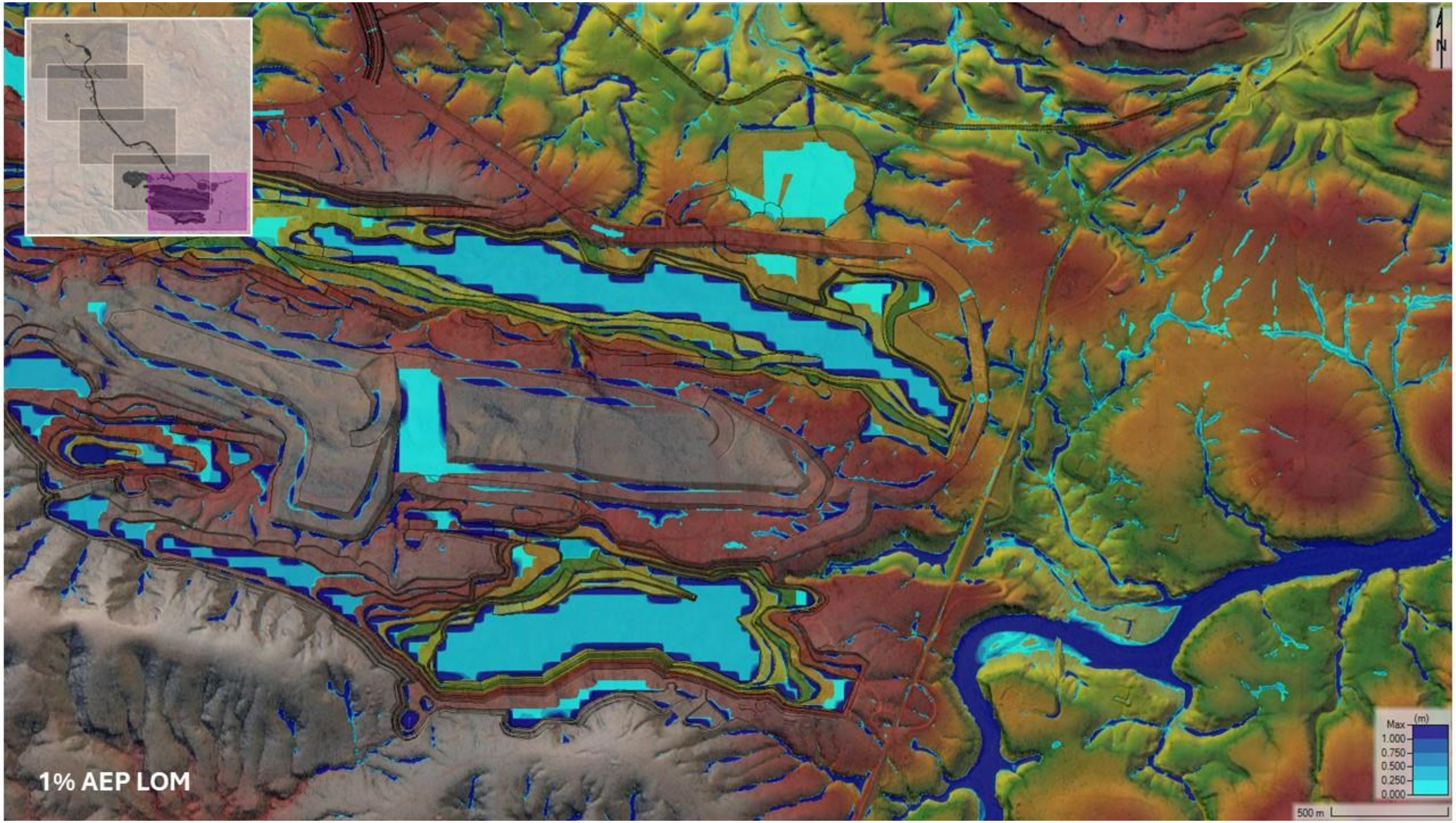
B-12 1% AEP LOM Depth (Location 2)



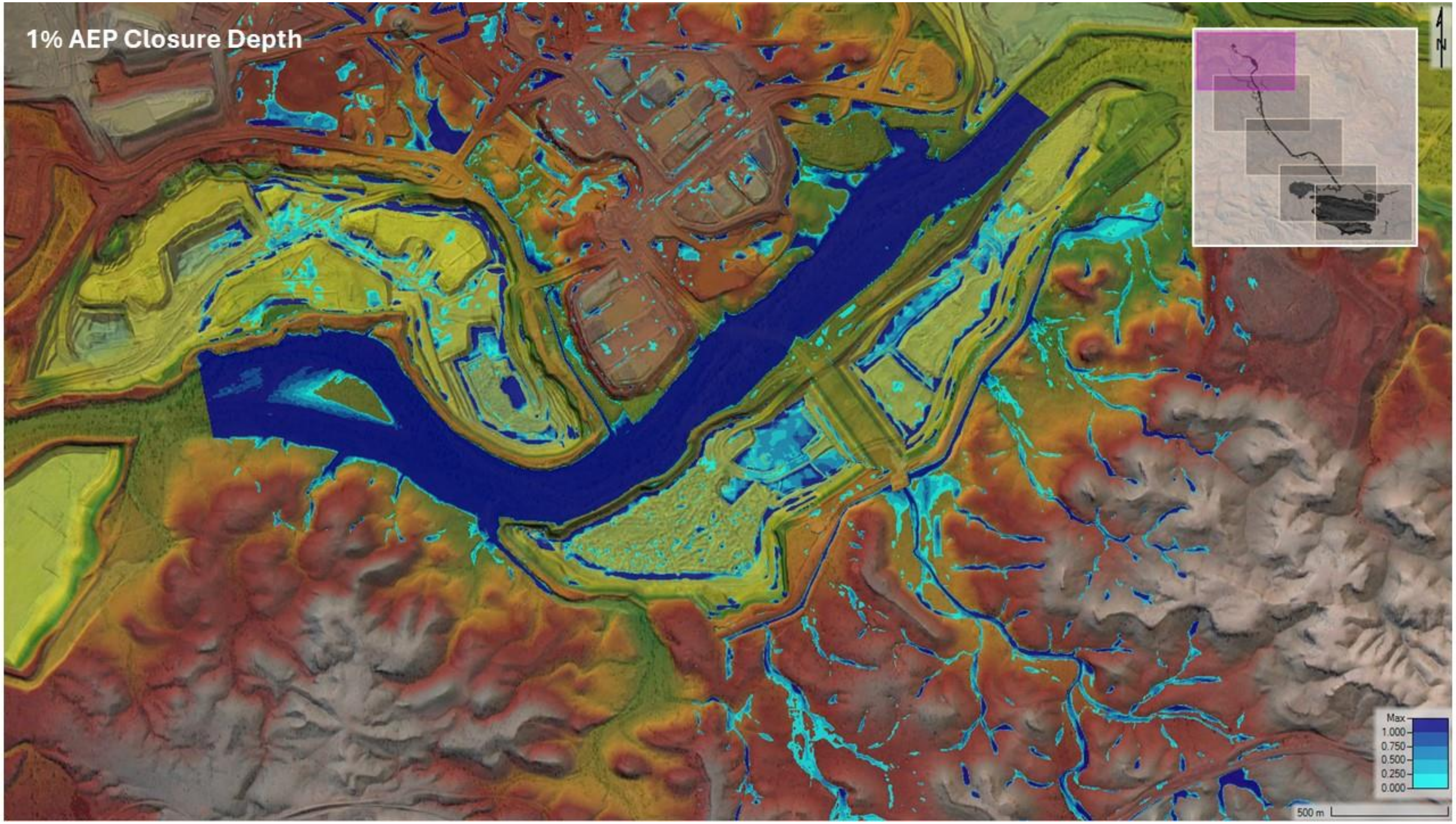
B-13 1% AEP LOM Depth (Location 3)



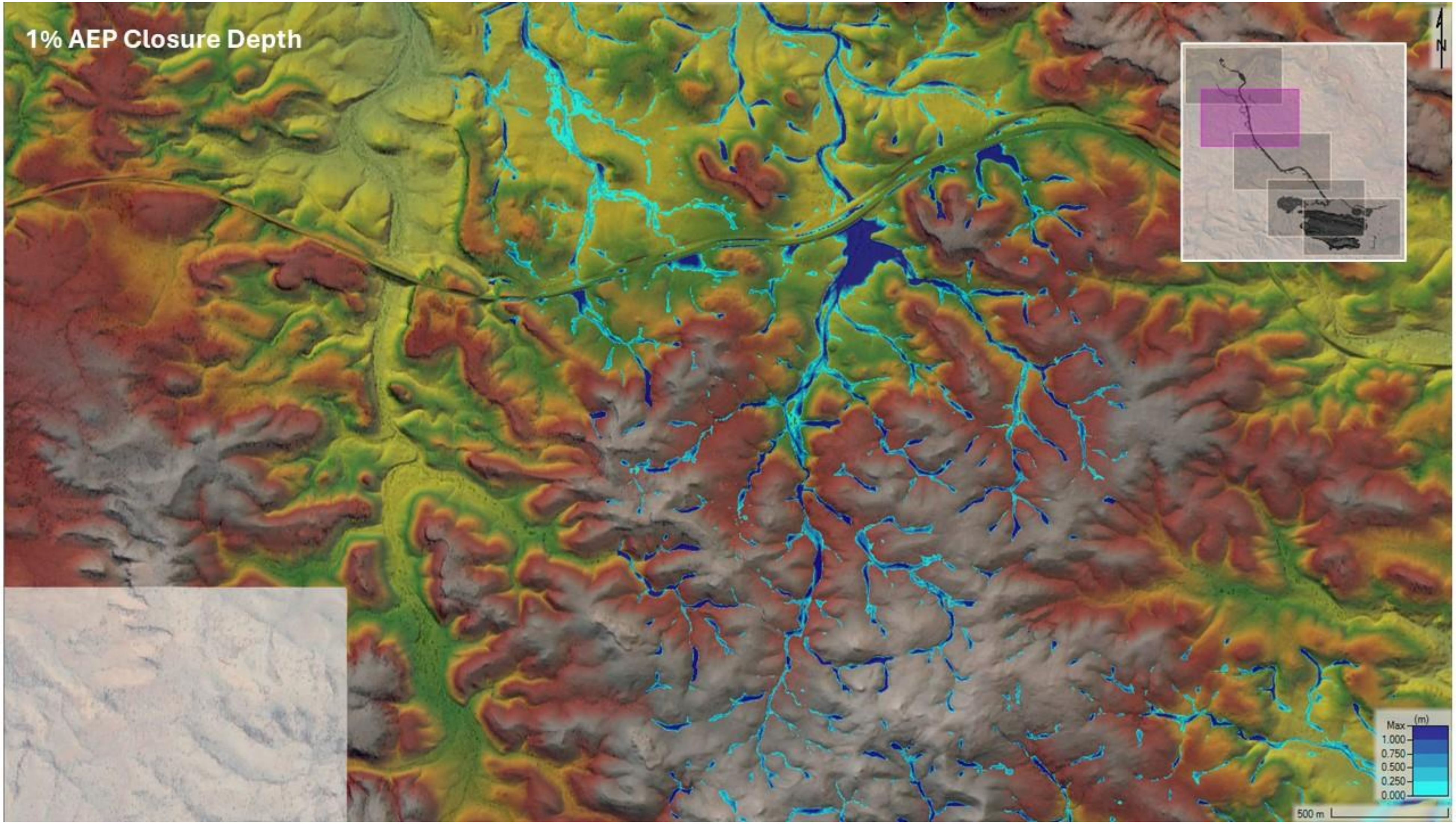
B-14 1% AEP LOM Depth (Location 4)



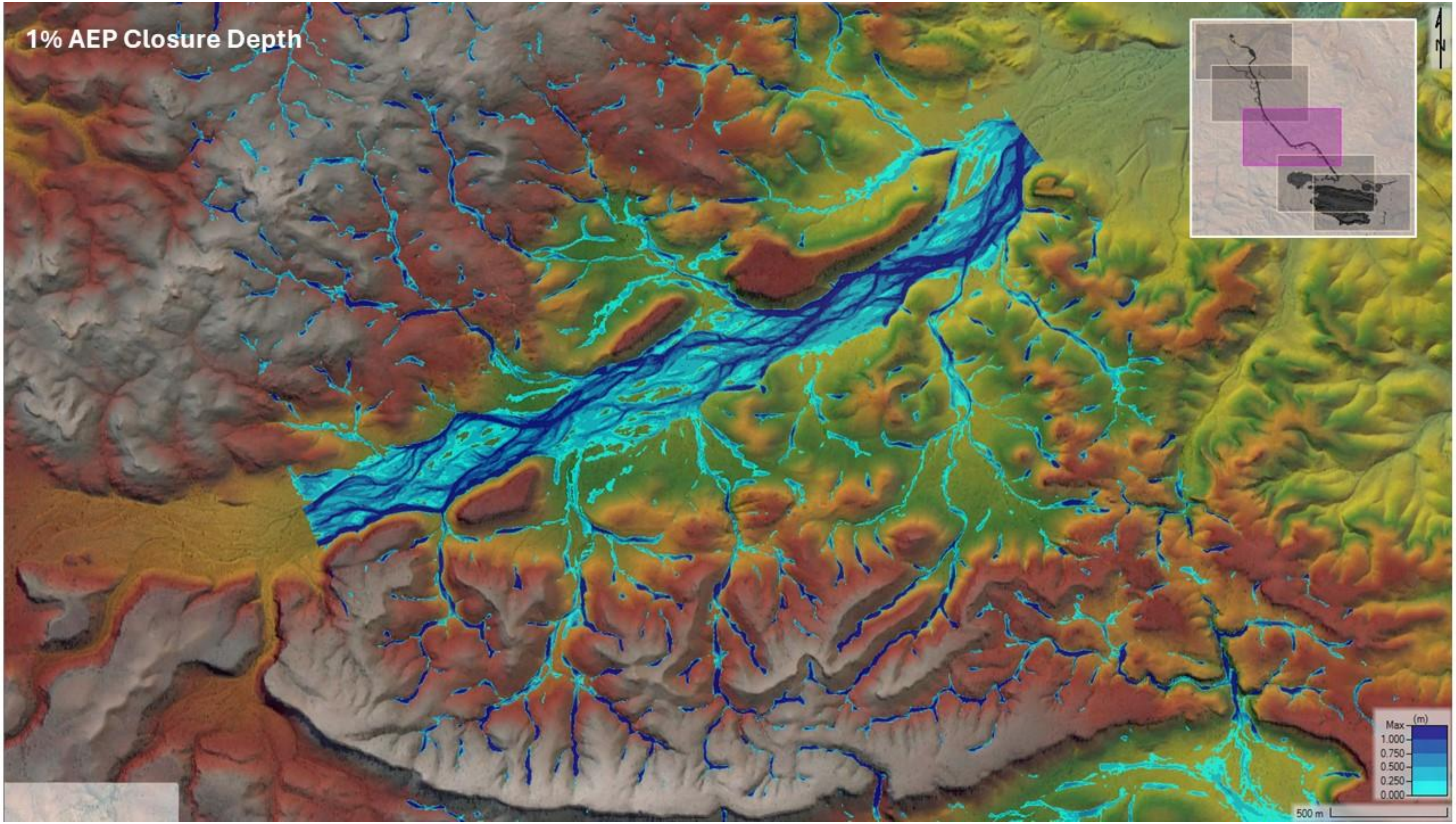
B-15 1% AEP LOM Depth (Location 5)



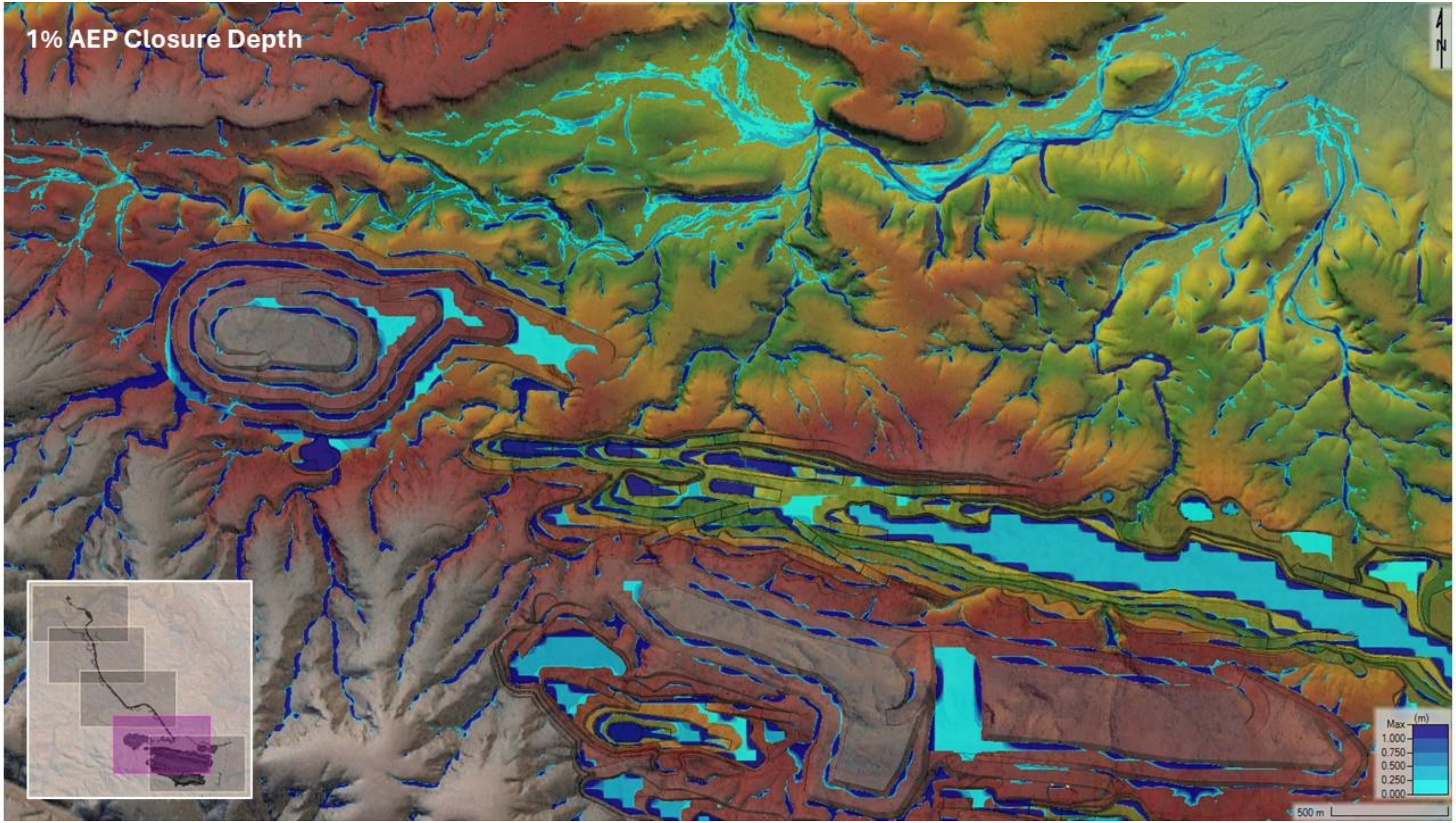
B-16 1% AEP Closure Depth (Location 1)



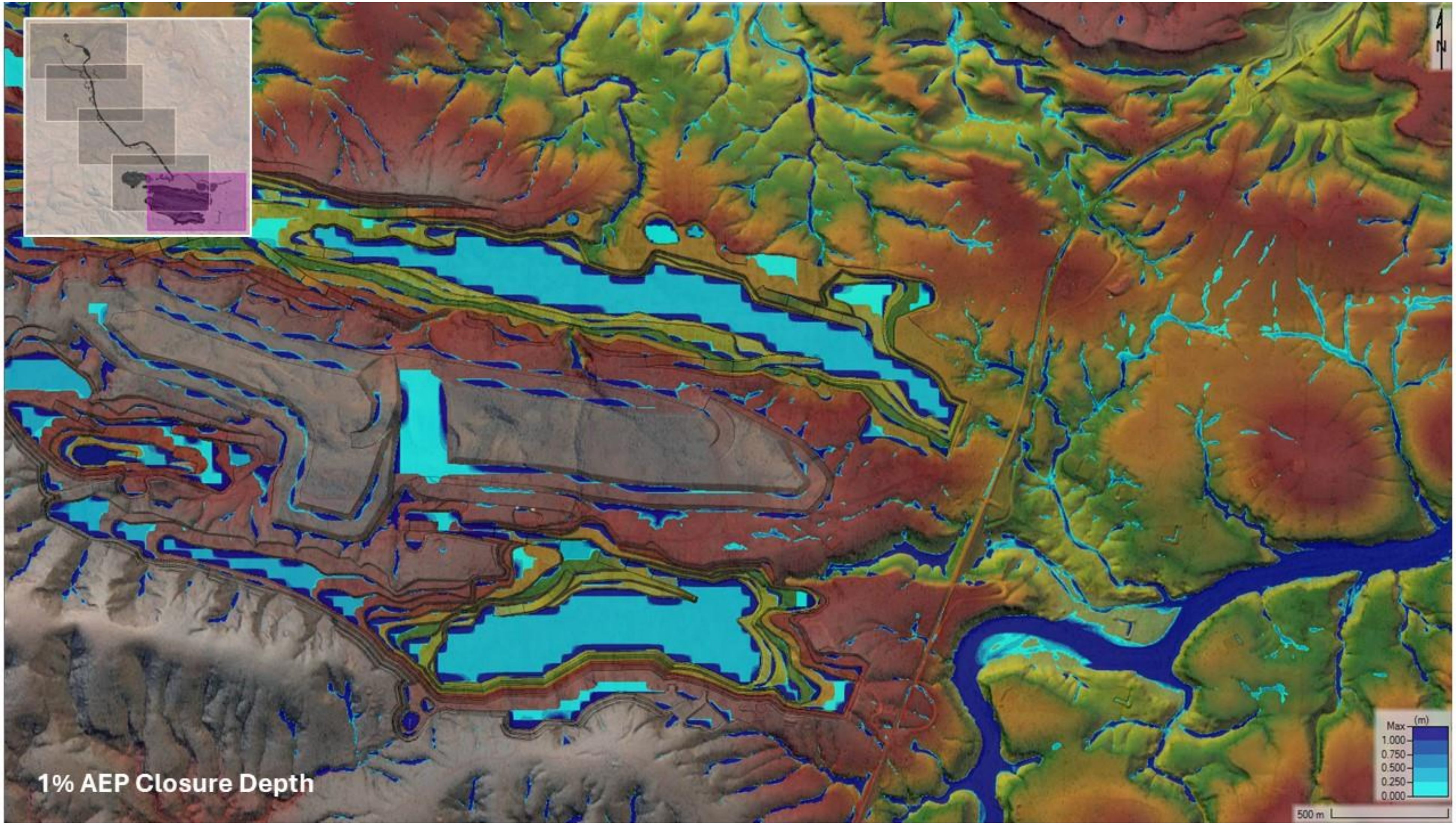
B-17 1% AEP Closure Depth (Location 2)



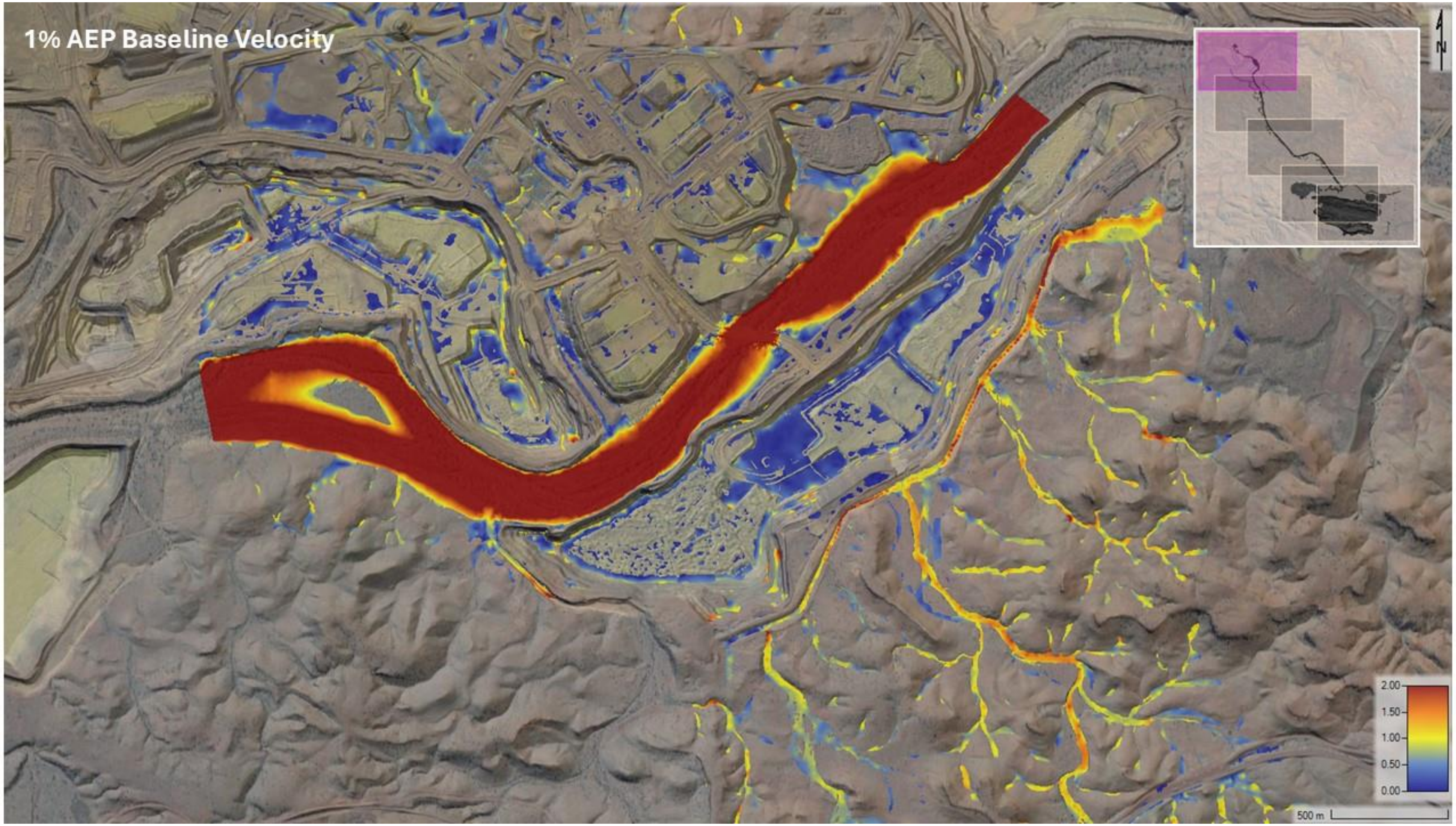
B-18 1% AEP Closure Depth (Location 3)



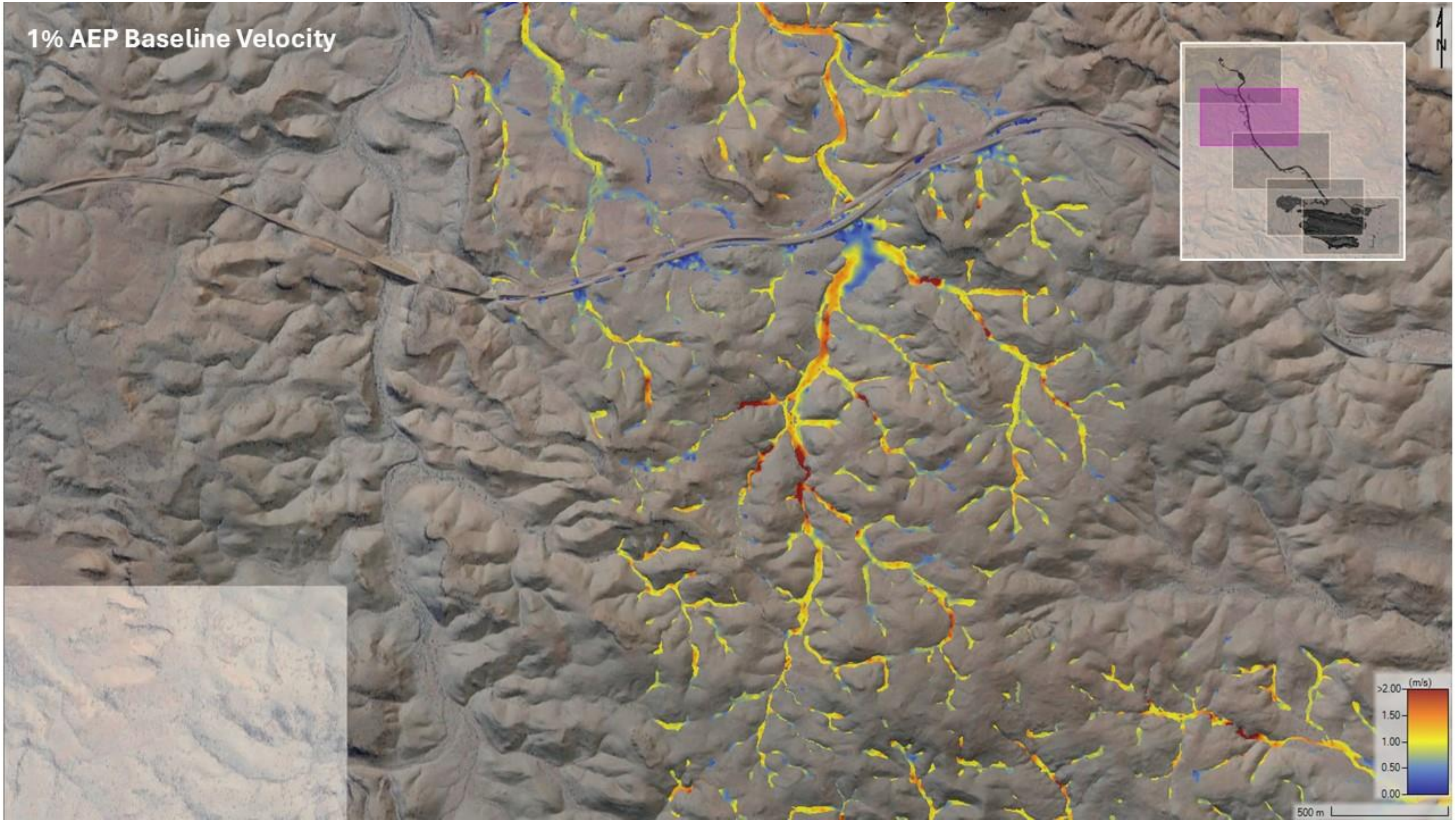
B-19 **1% AEP Closure Depth (Location 4)**



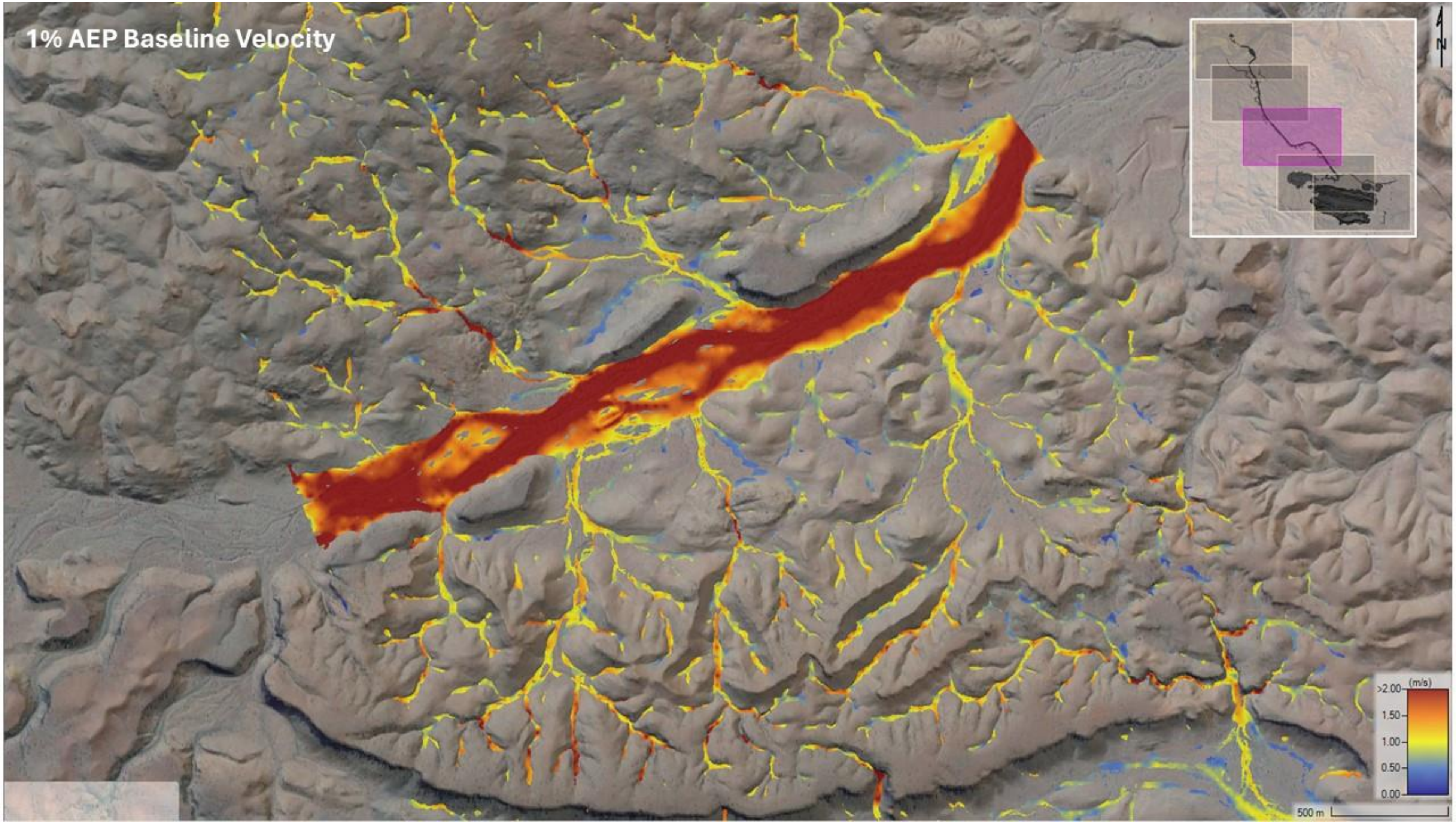
B-20 **1% AEP Closure Depth (Location 5)**



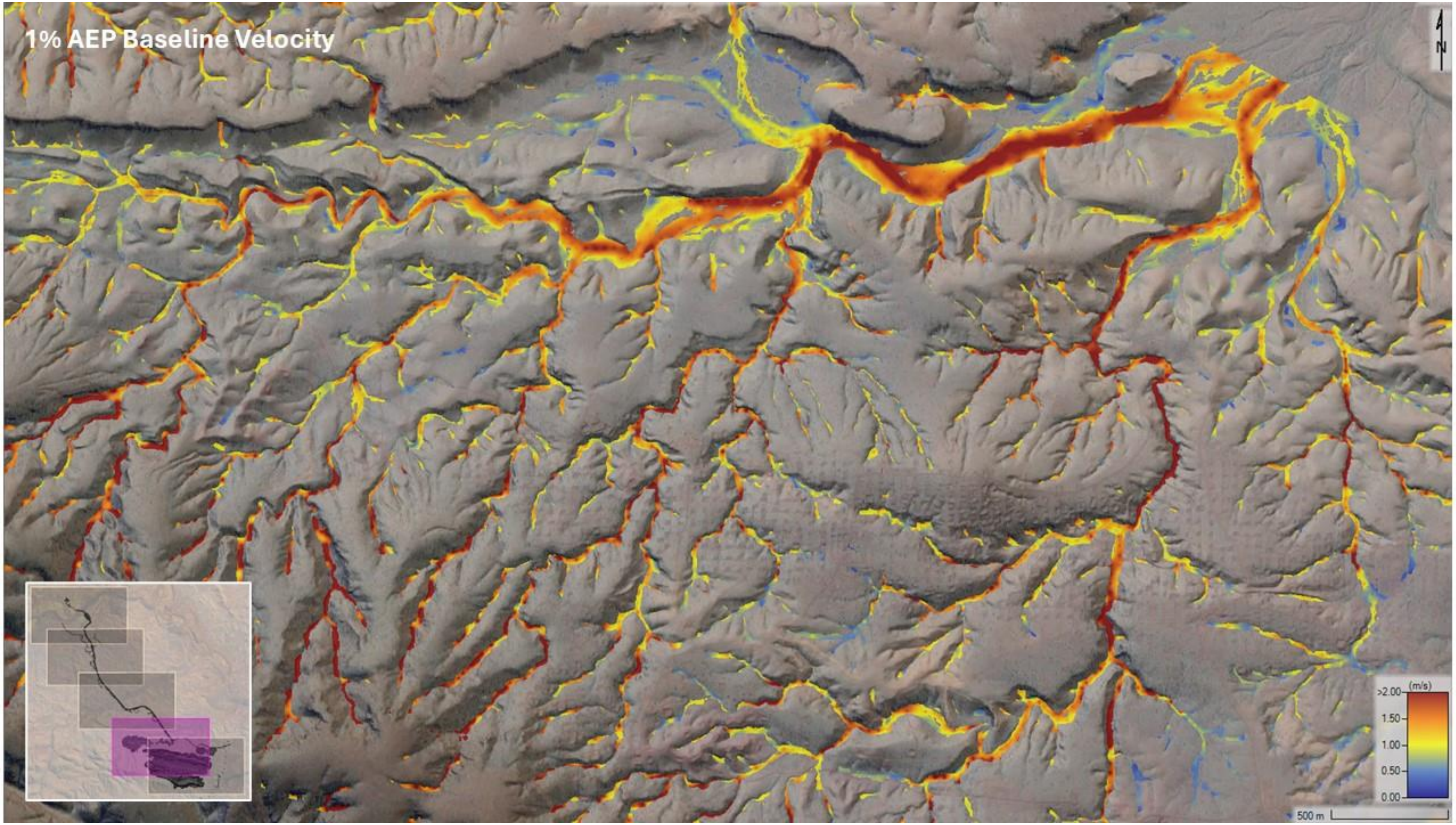
B-21 1% AEP Baseline Velocity (Location 1)



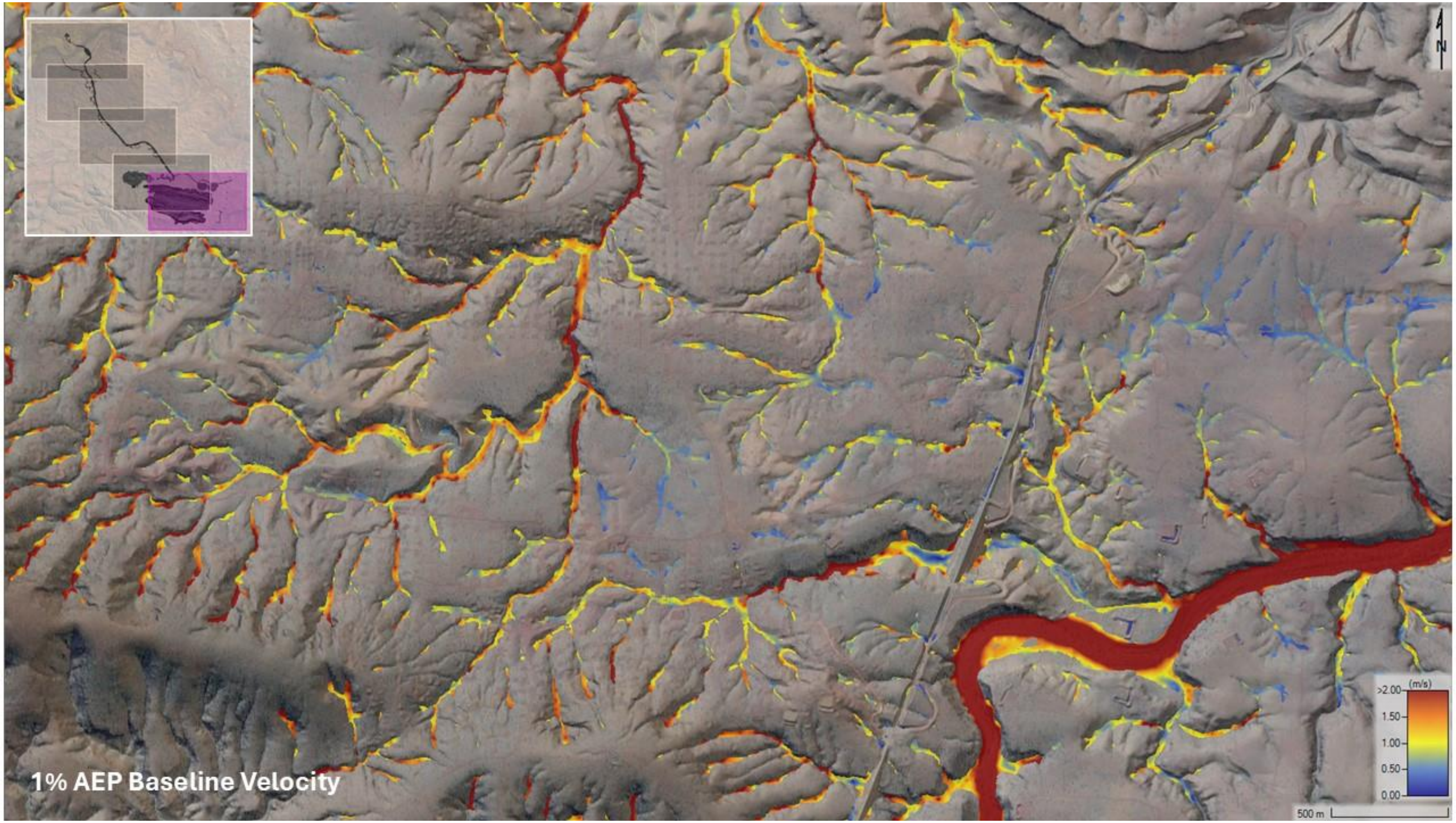
B-22 1% AEP Baseline Velocity (Location 2)



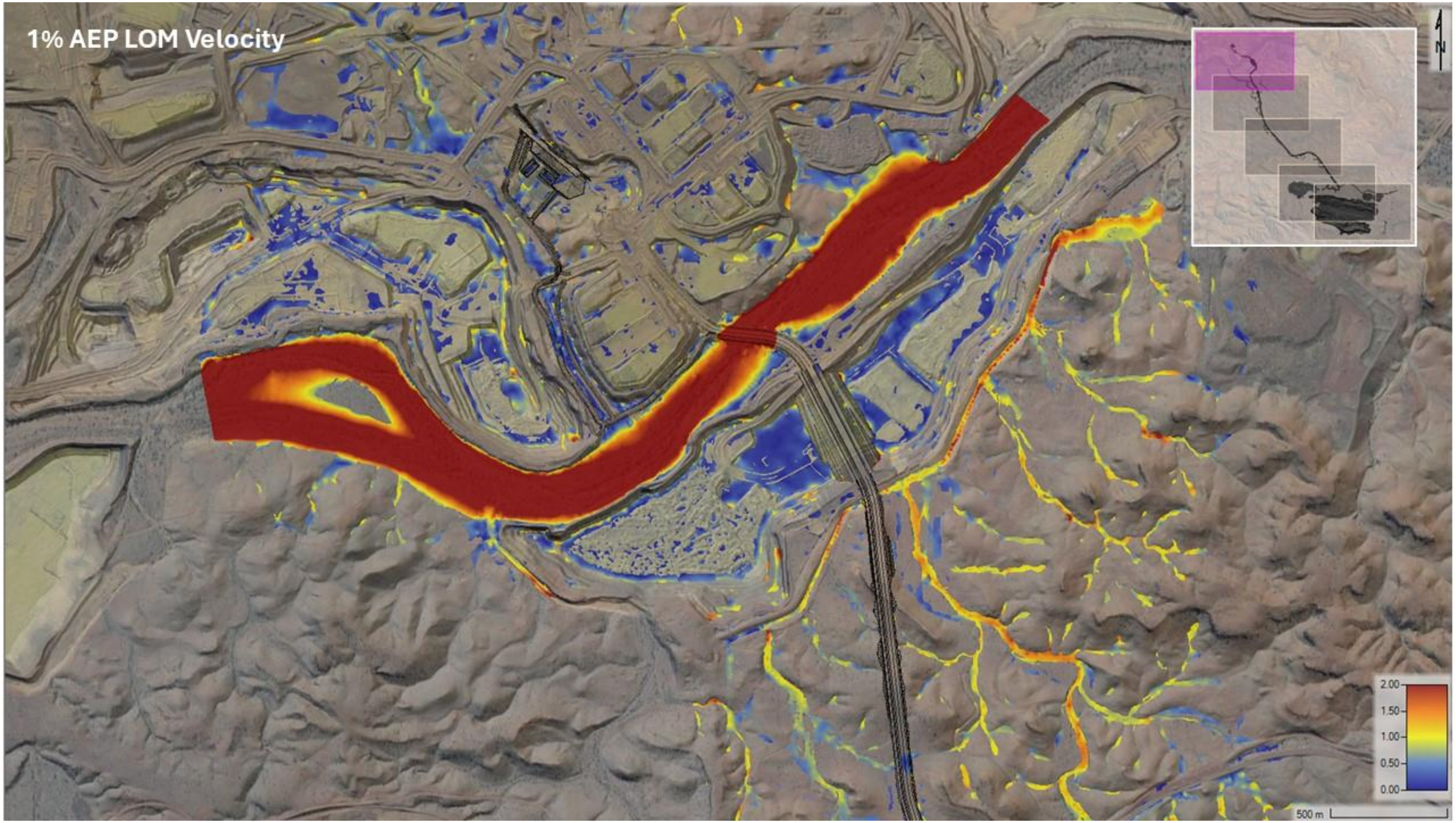
B-23 1% AEP Baseline Velocity (Location 3)



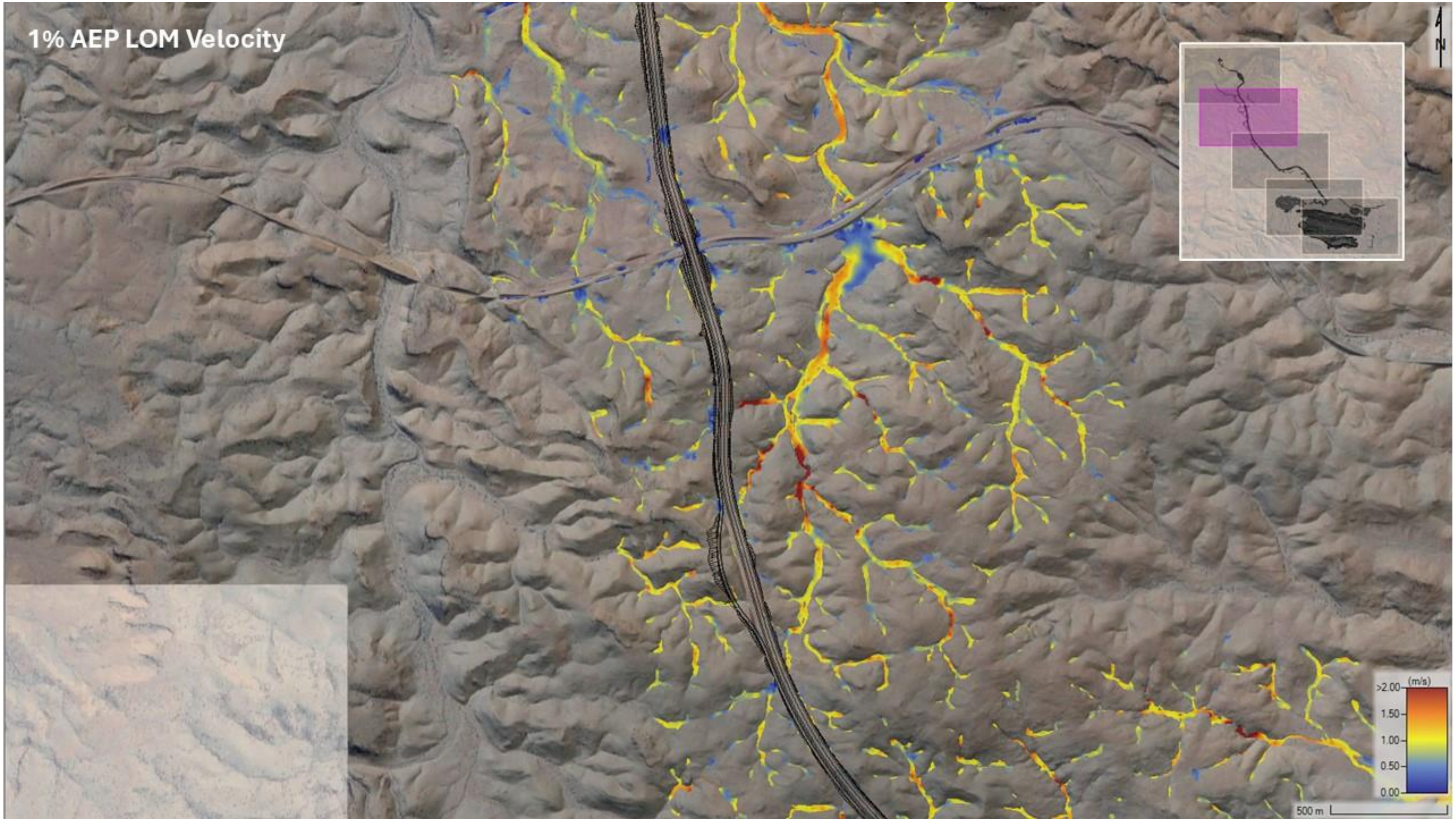
B-24 1% AEP Baseline Velocity (Location 4)



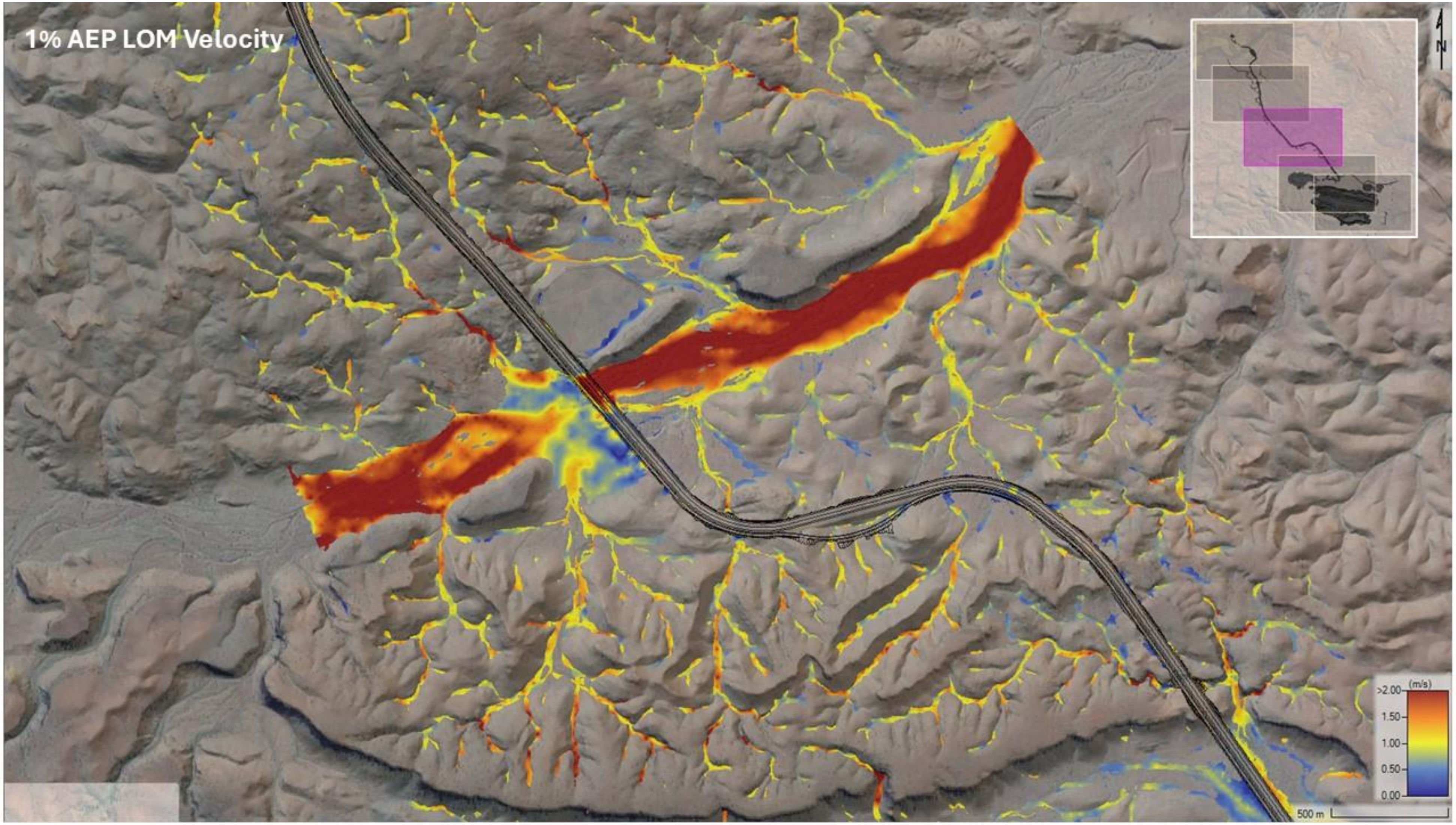
B-25 1% AEP Baseline Velocity (Location 5)



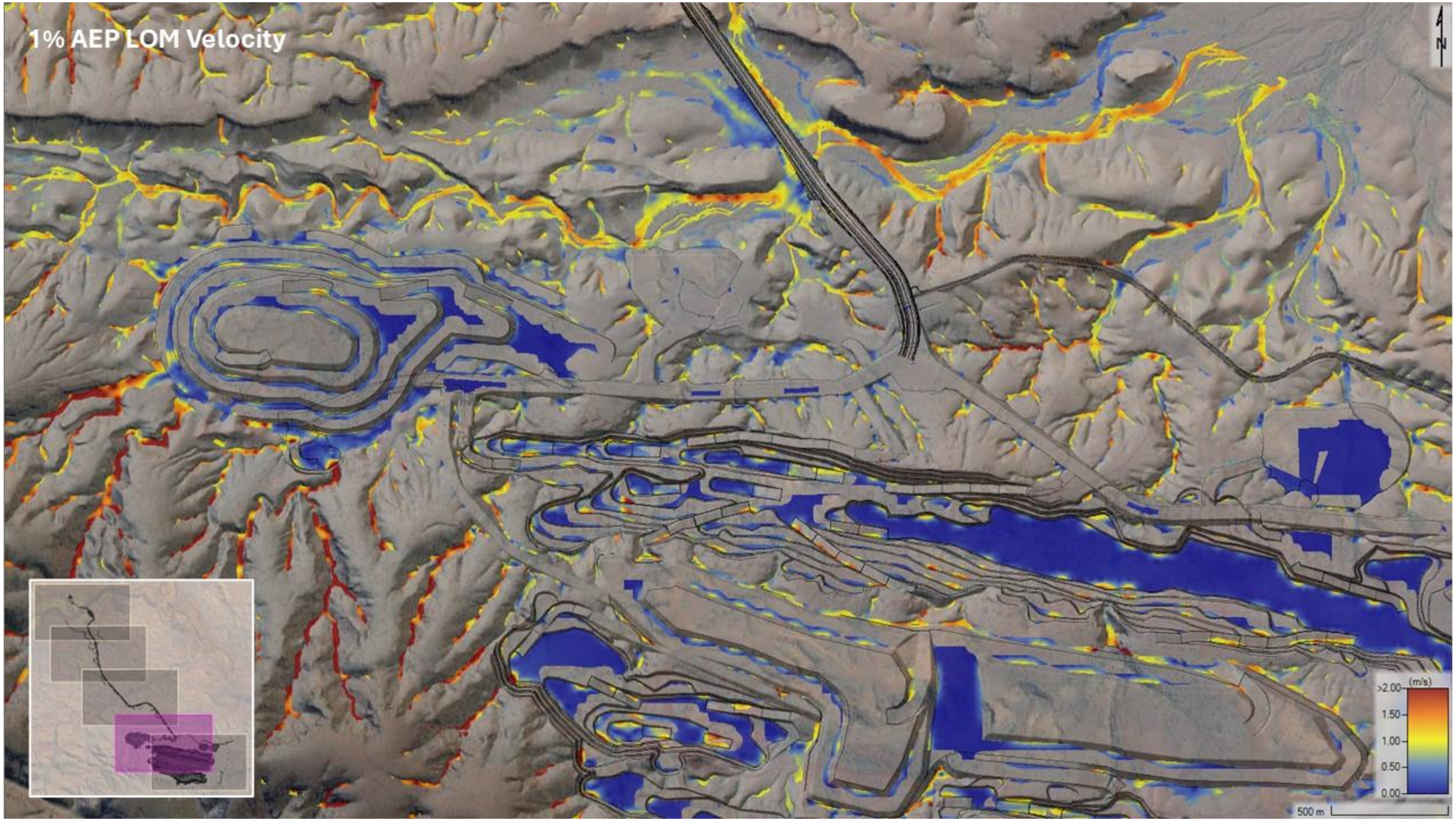
B-26 1% AEP LOM Velocity (Location 1)



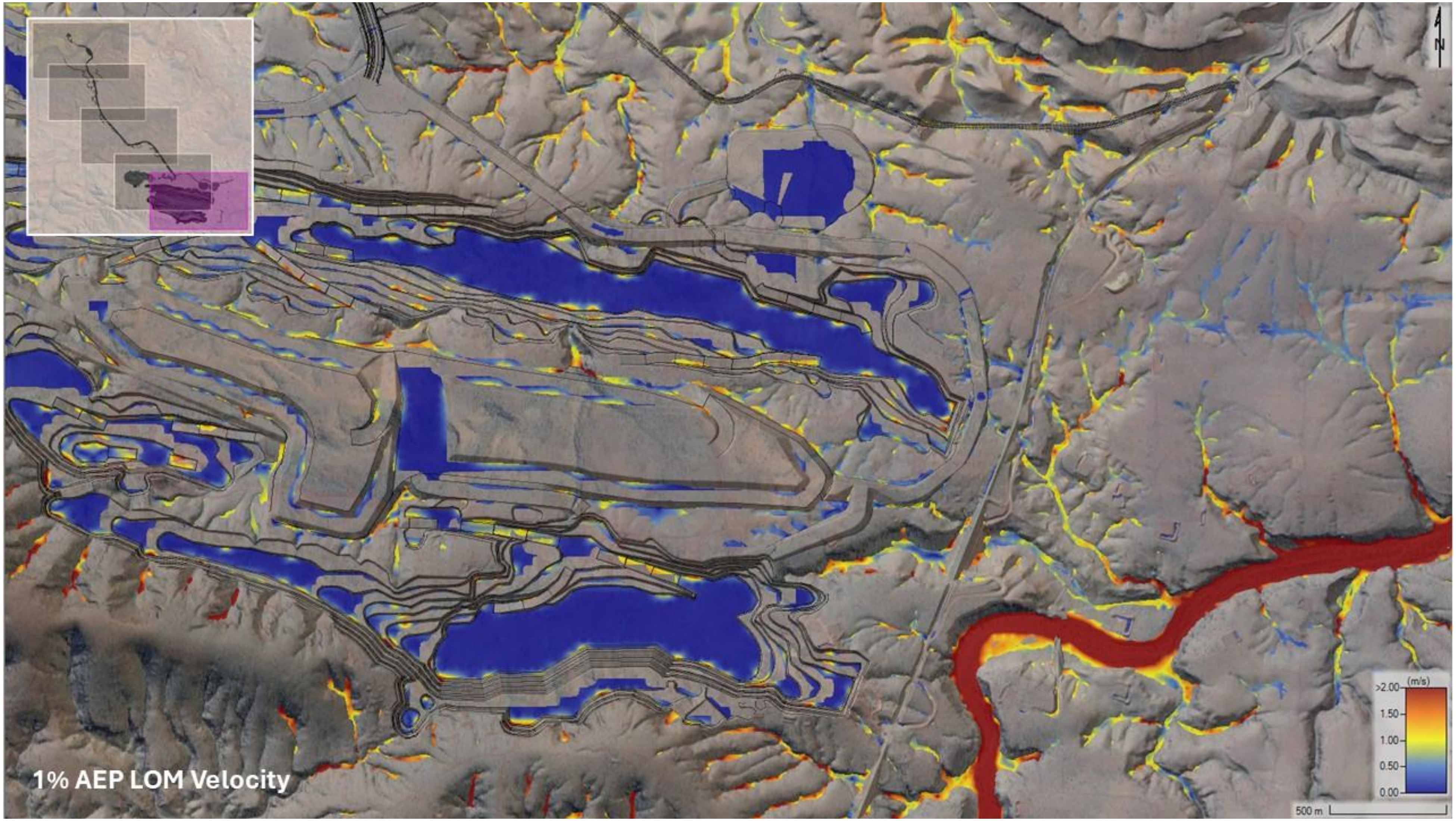
B-27 1% AEP LOM Velocity (Location 2)



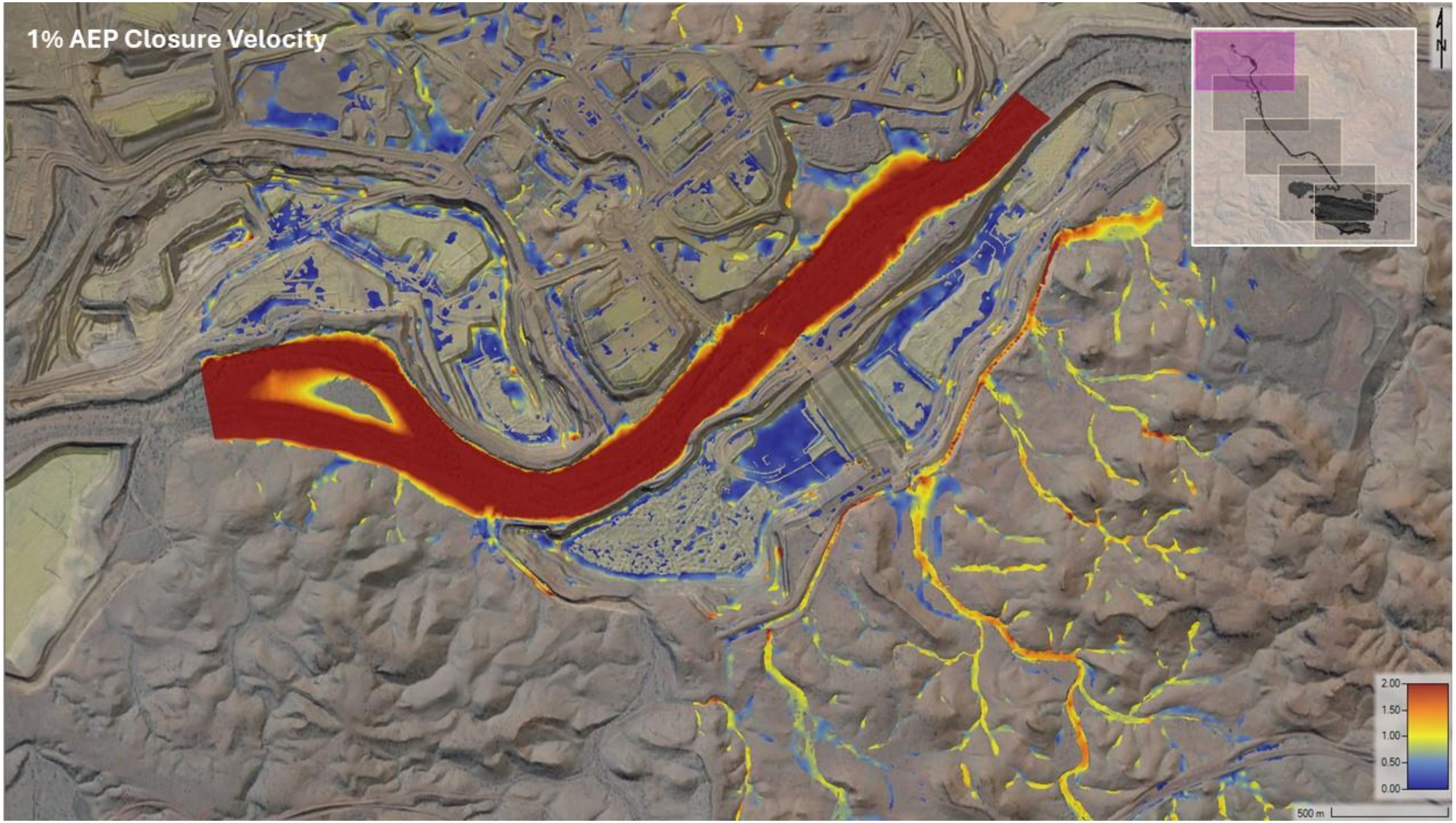
B-28 1% AEP LOM Velocity (Location 3)



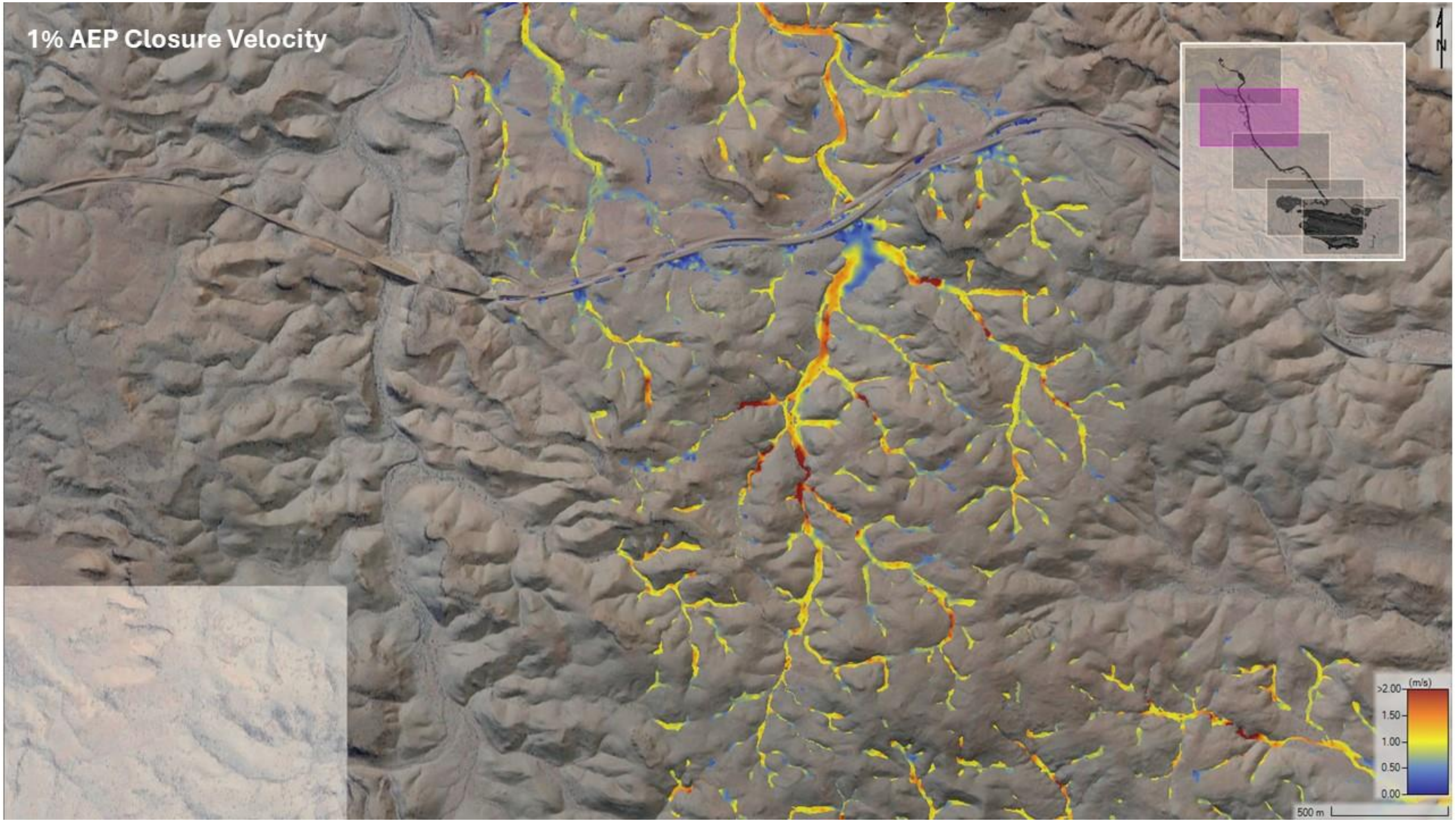
B-29 1% AEP LOM Velocity (Location 4)



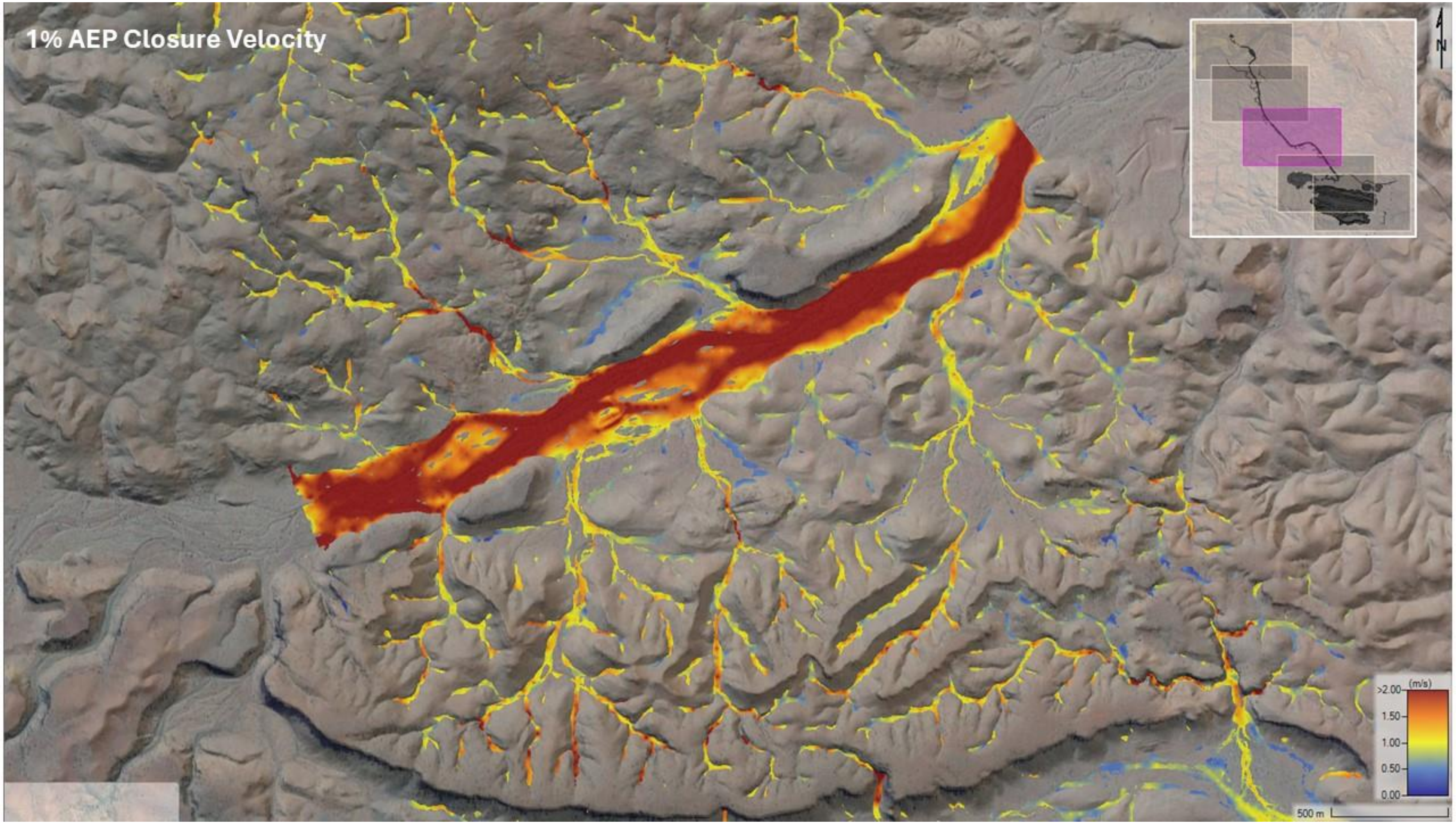
B-30 1% AEP LOM Velocity (Location 5)



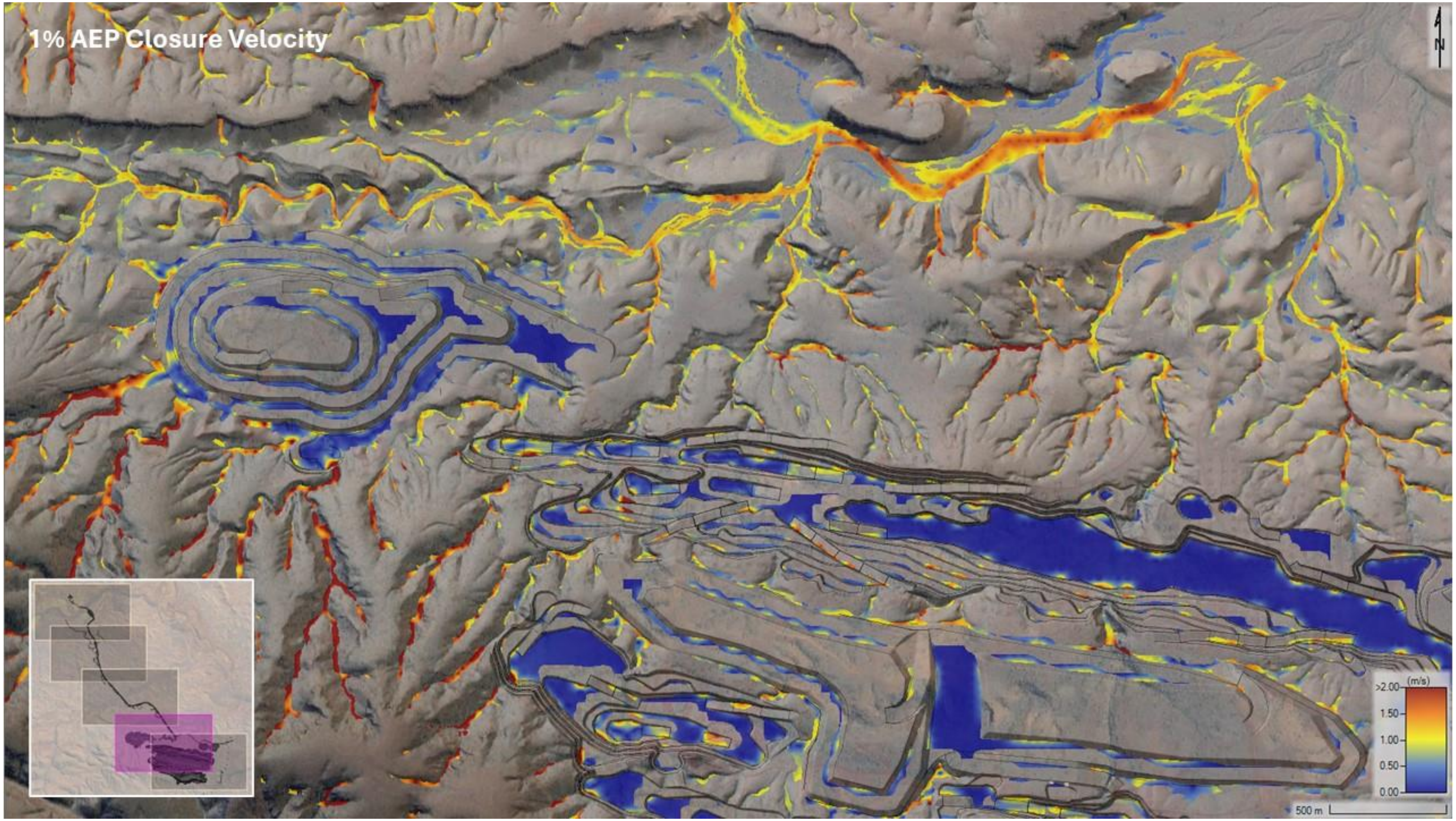
B-31 1% AEP Closure Velocity (Location 1)



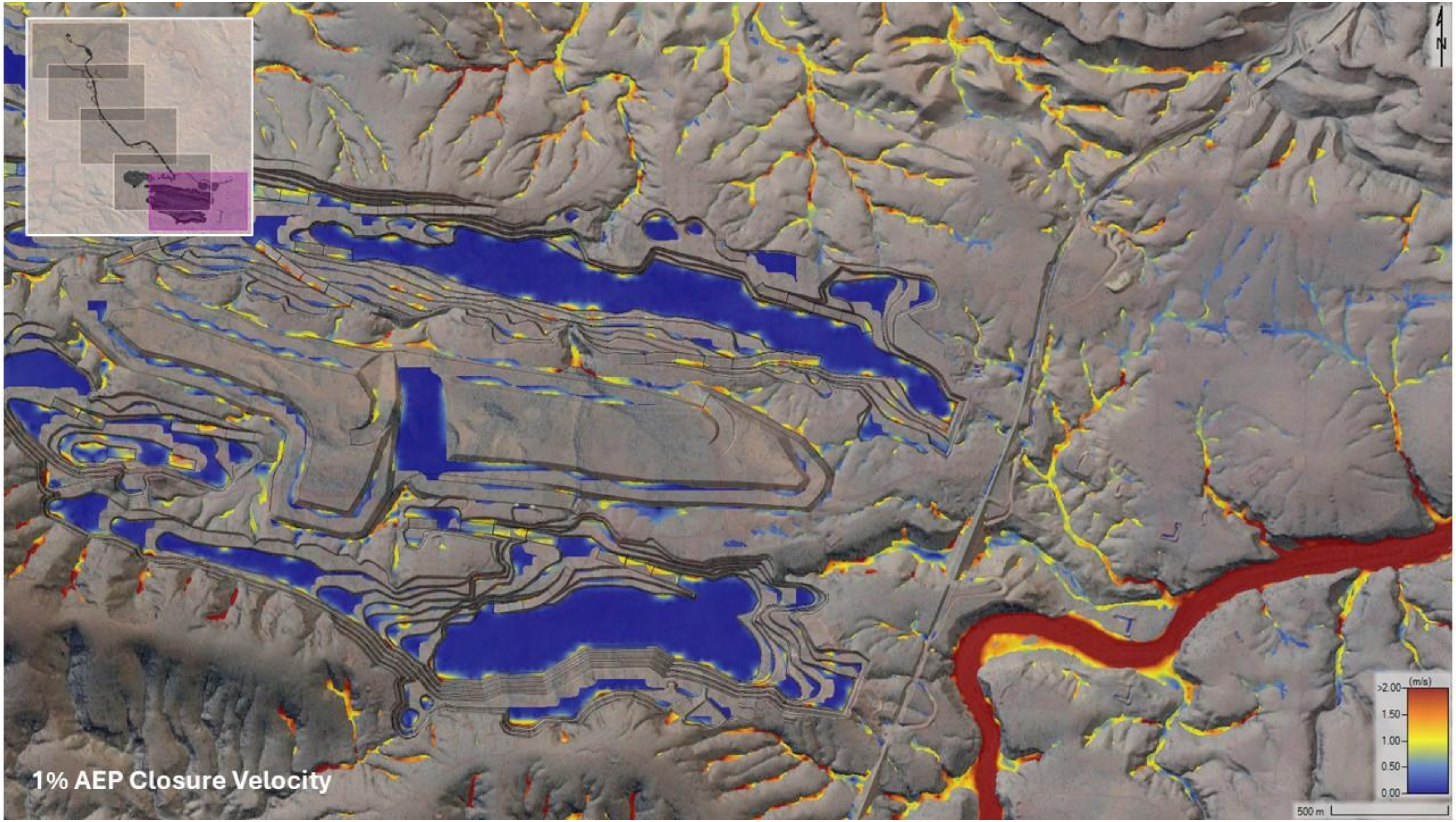
B-32 1% AEP Closure Velocity (Location 2)



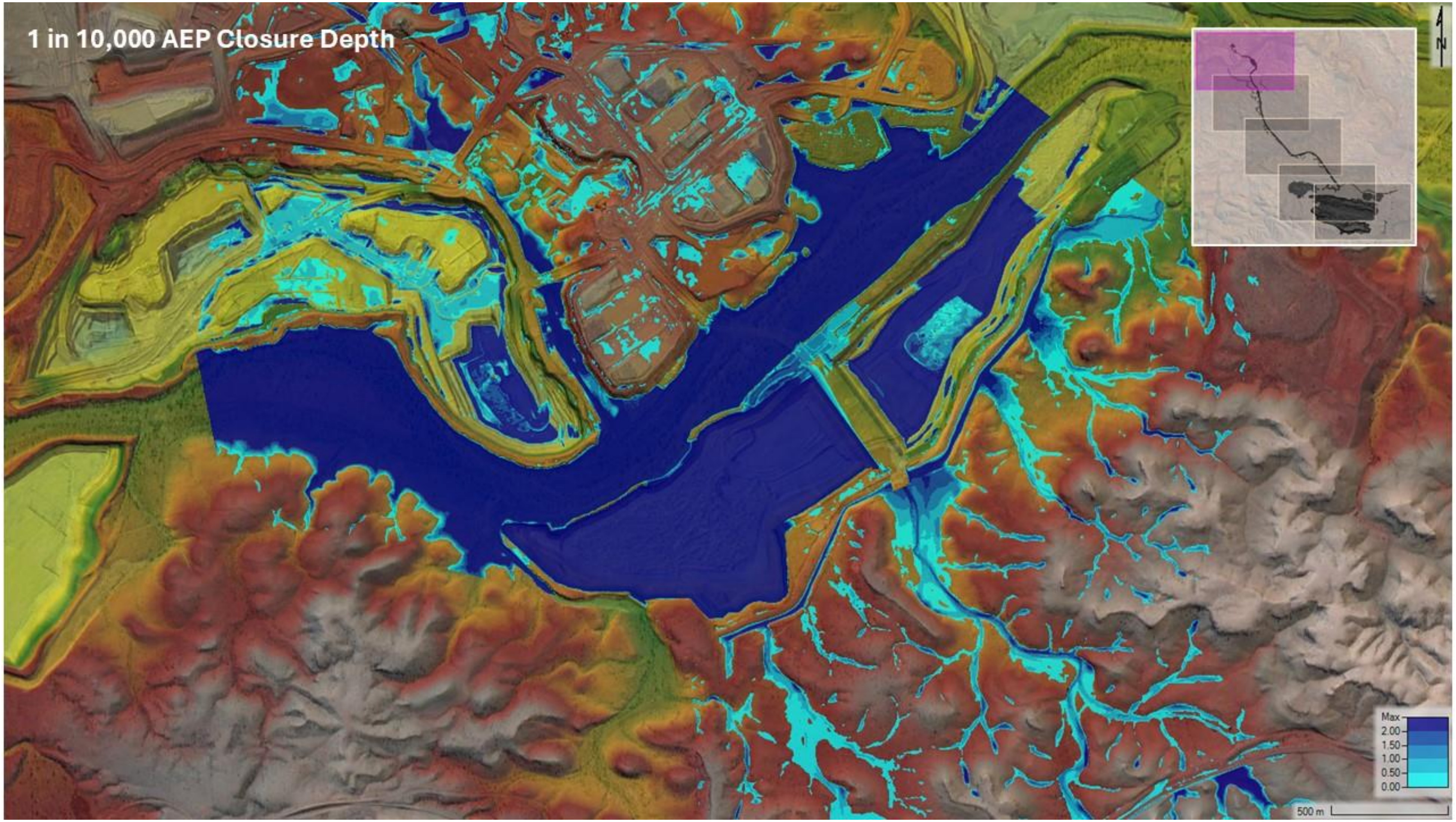
B-33 1% AEP Closure Velocity (Location 3)



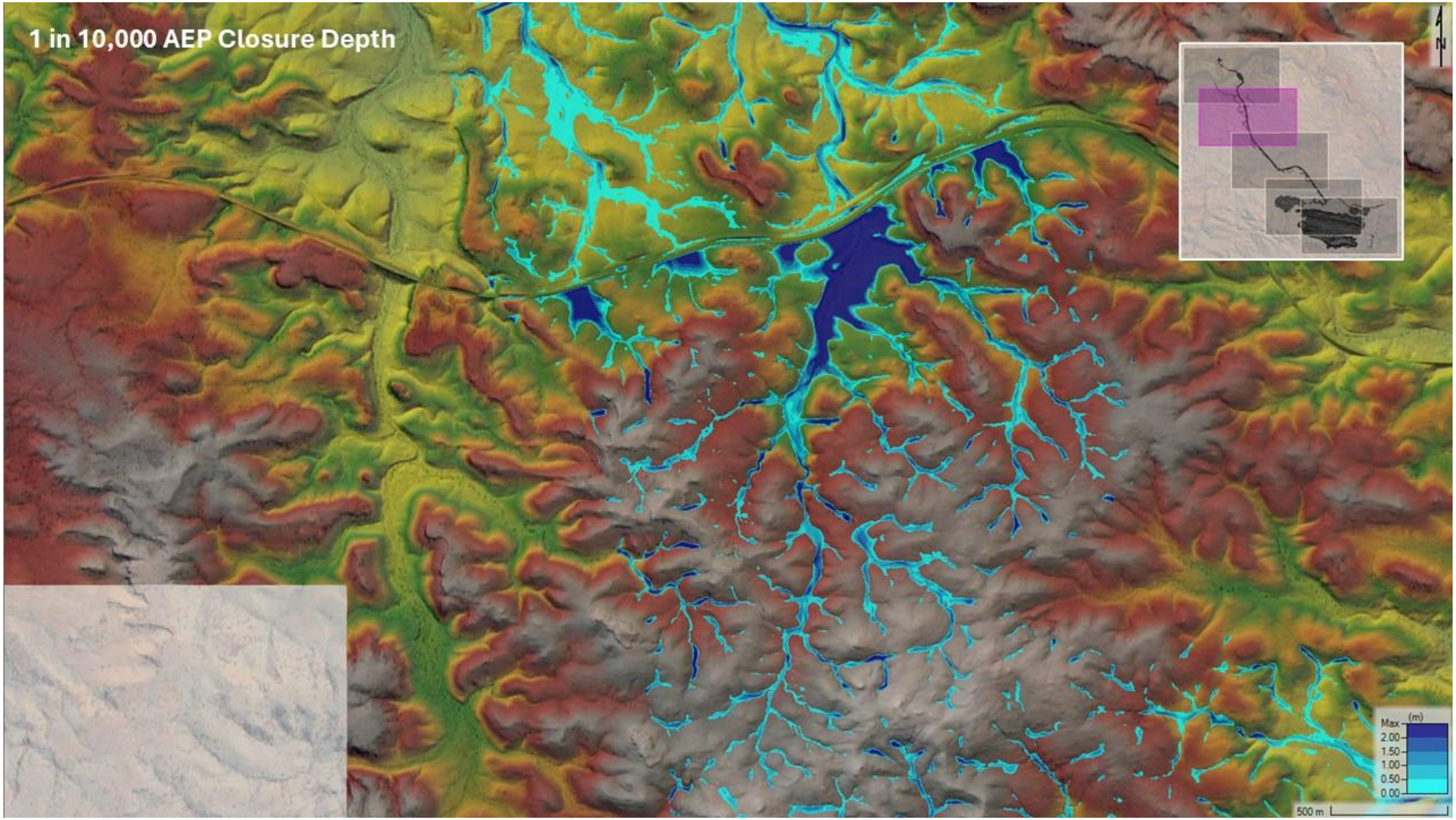
B-34 1% AEP Closure Velocity (Location 4)



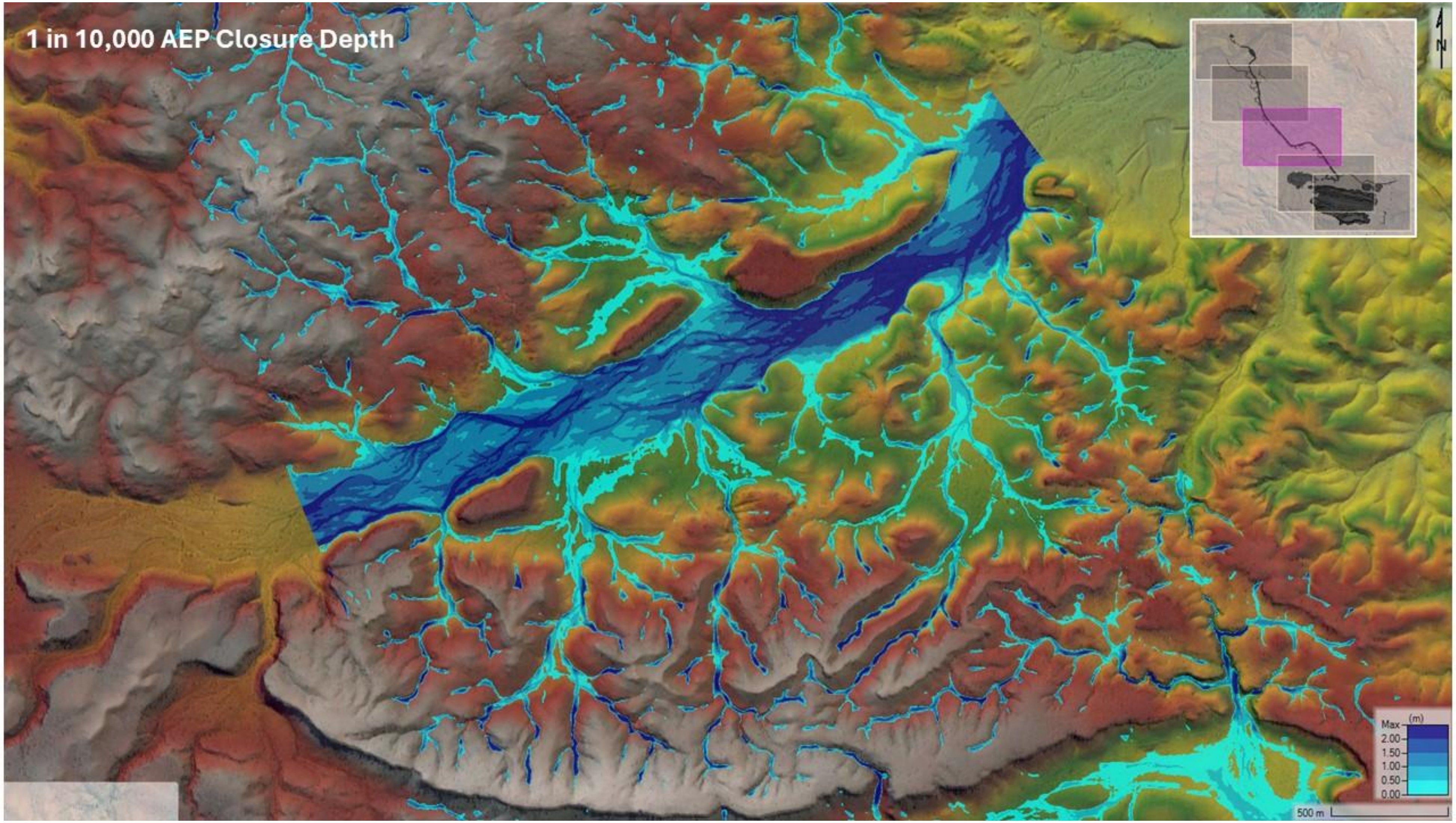
B-35 1% AEP Closure Velocity (Location 5)



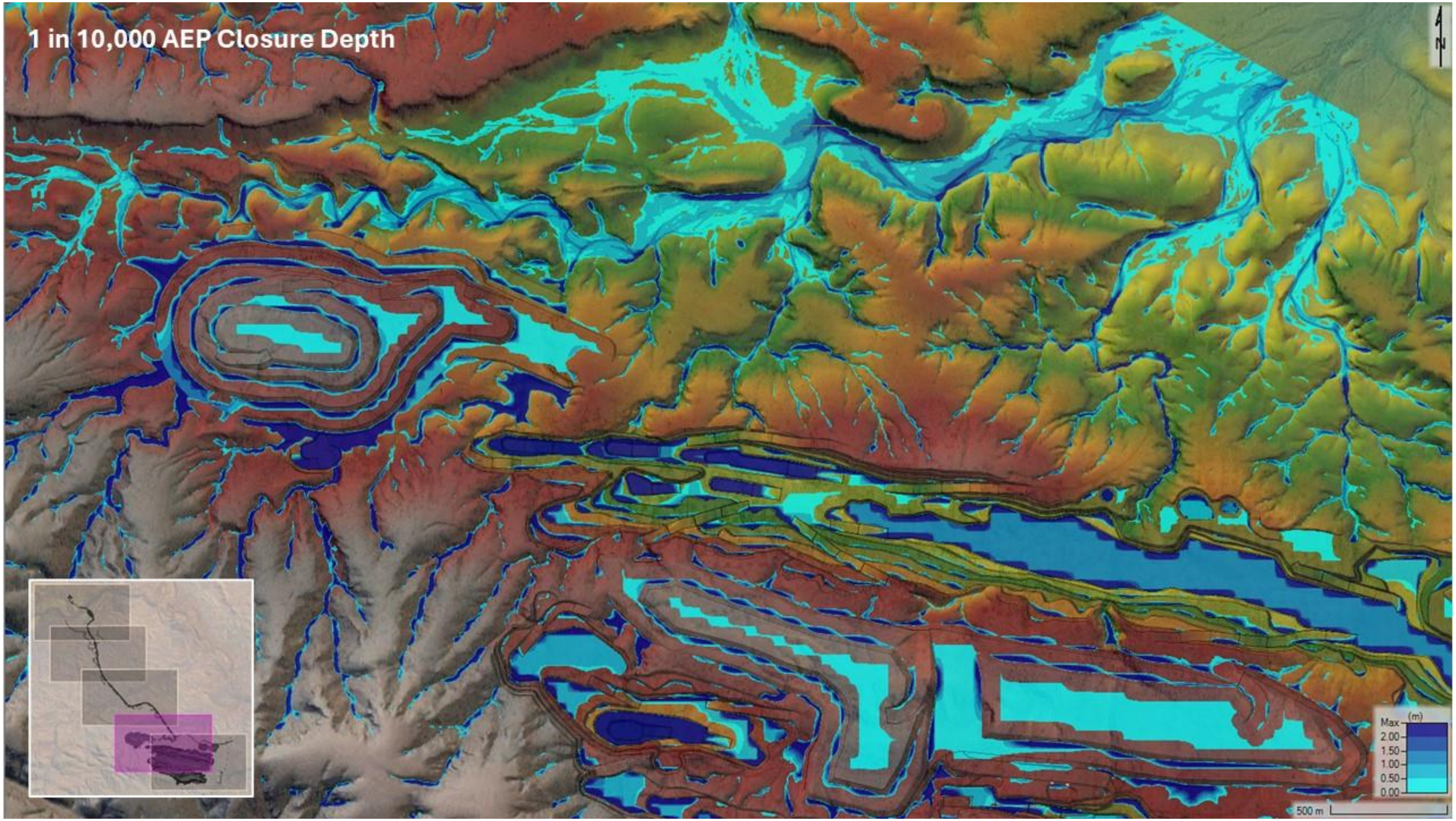
B-36 1 in 10,000 AEP Closure Depth (Location 1)



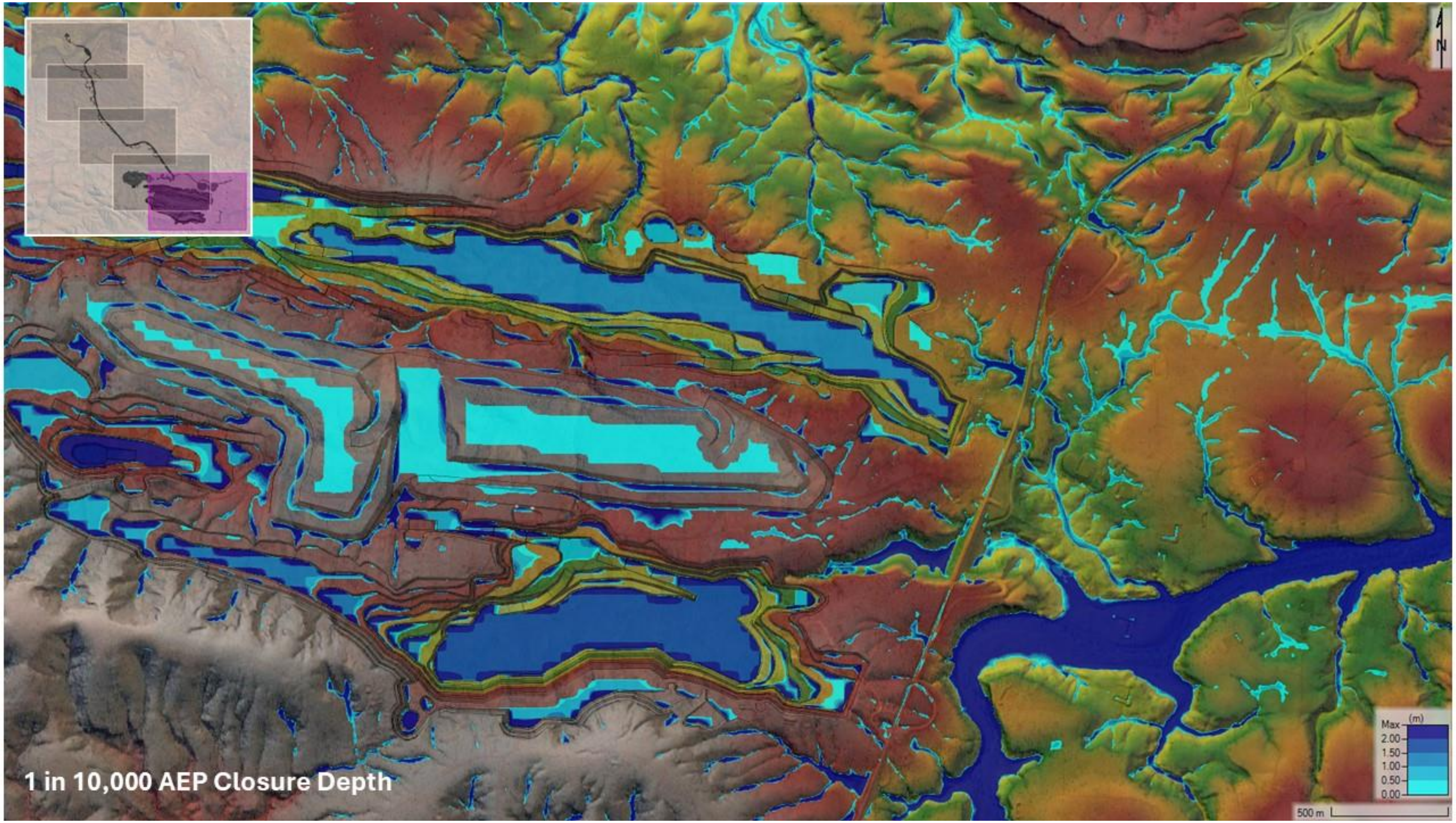
B-37 1 in 10,000 AEP Closure Depth (Location 2)



B-38 1 in 10,000 AEP Closure Depth (Location 3)



B-39 1 in 10,000 AEP Closure Depth (Location 4)



B-40 1 in 10,000 AEP Closure Depth (Location 5)

Appendix C: Flood Afflux Maps

Note: Life of mine 1% AEP Afflux Map shown in Section 8 of the report body.

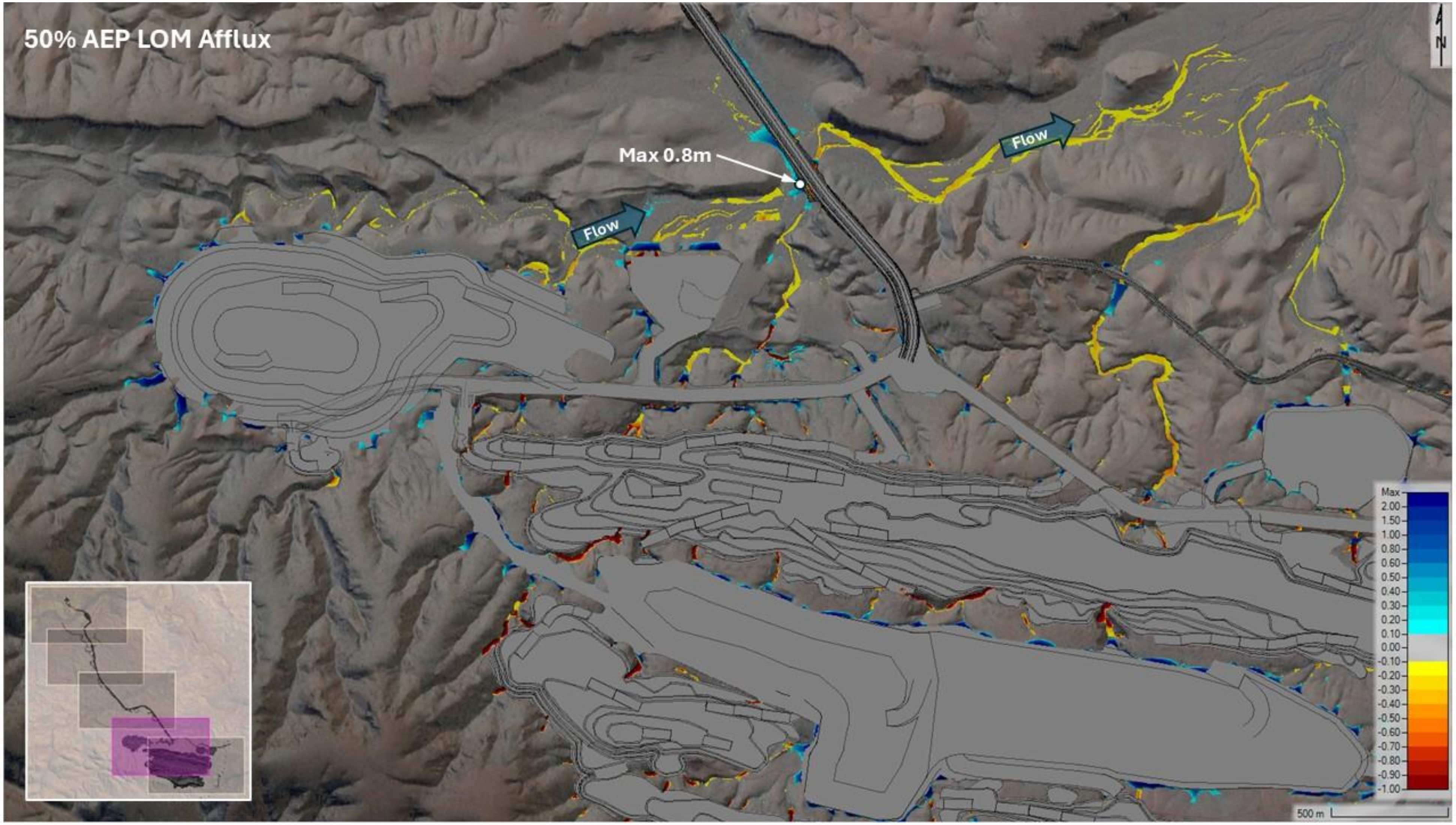
AEP	WSEL Afflux	
	Appendix C	
	LOM	Closure
50%	Figure C1-C5 A.1.1.1C-1	Figure C6-C10
20%		
10%		
5%		
1%	Figure 8-1 - Figure 8-5 (main report)	Figure C11-C15
1 in 10,000 CC uplift		Figure C16-C20

Note: Remaining figures available in accompanying electronic files

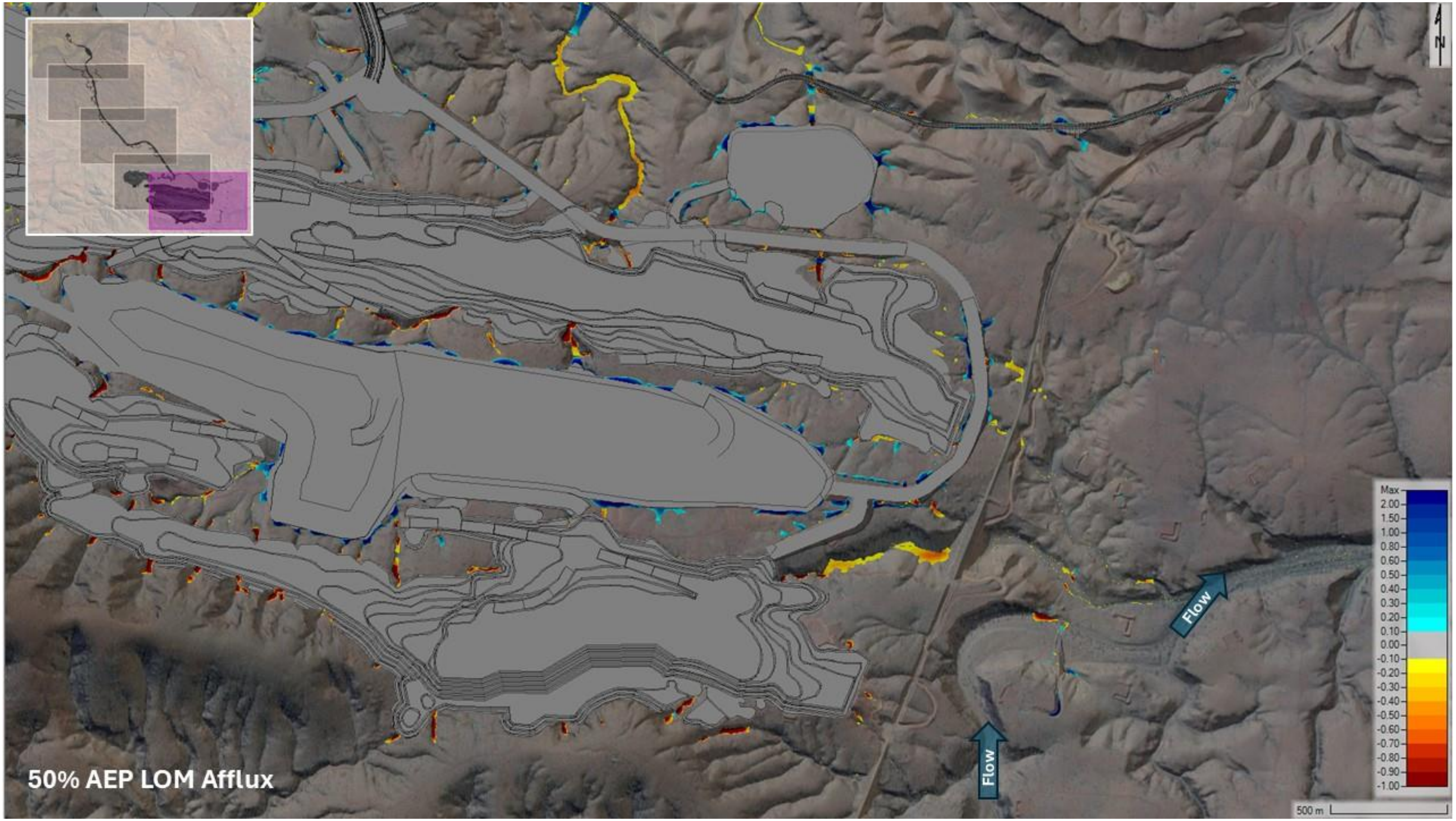
C-1 50% AEP LOM Afflux (Location 1)

C-2 50% AEP LOM Afflux (Location 2)

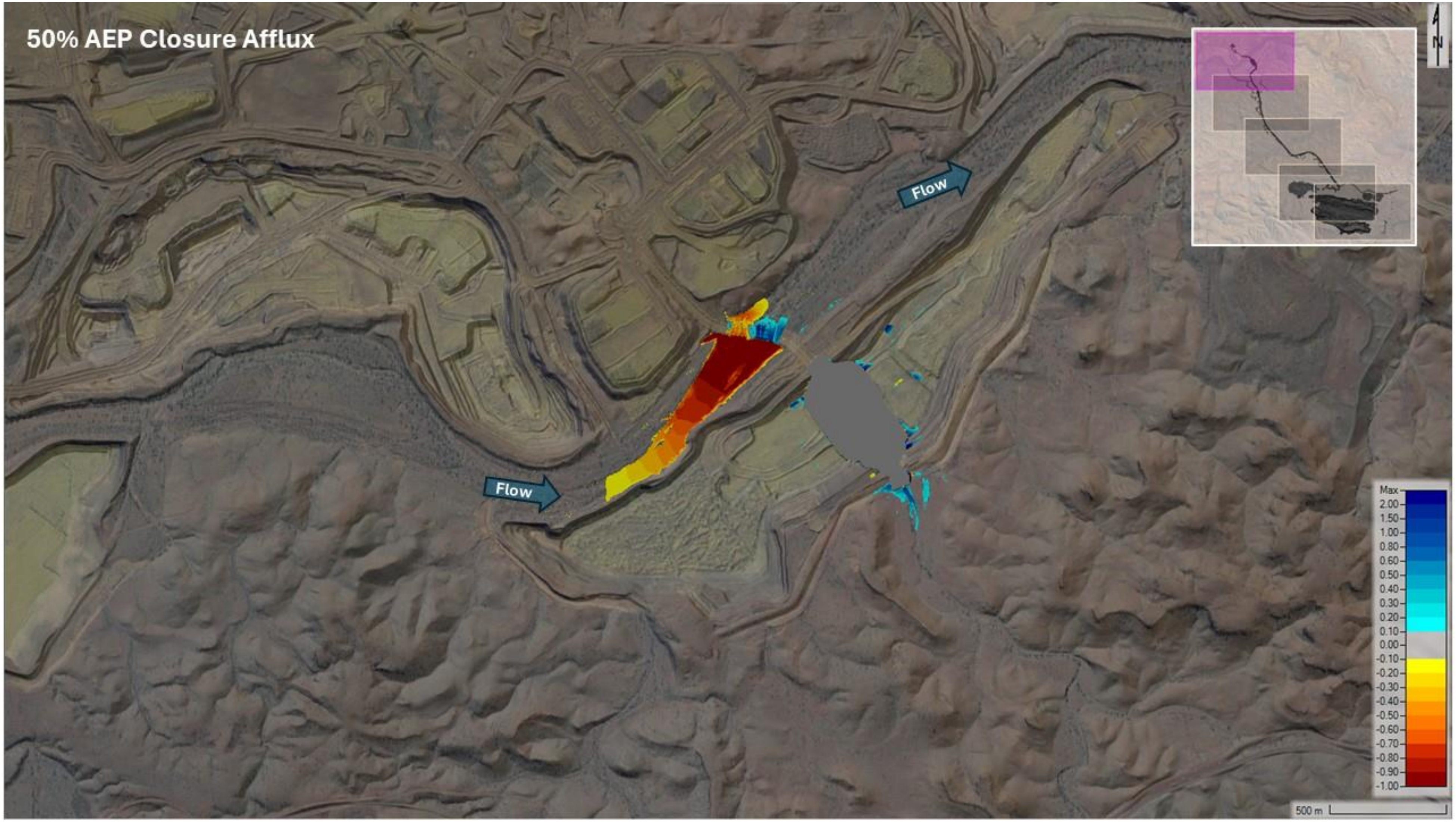
C-3 50% AEP LOM Afflux (Location 3)



C-4 50% AEP LOM Afflux (Location 4)



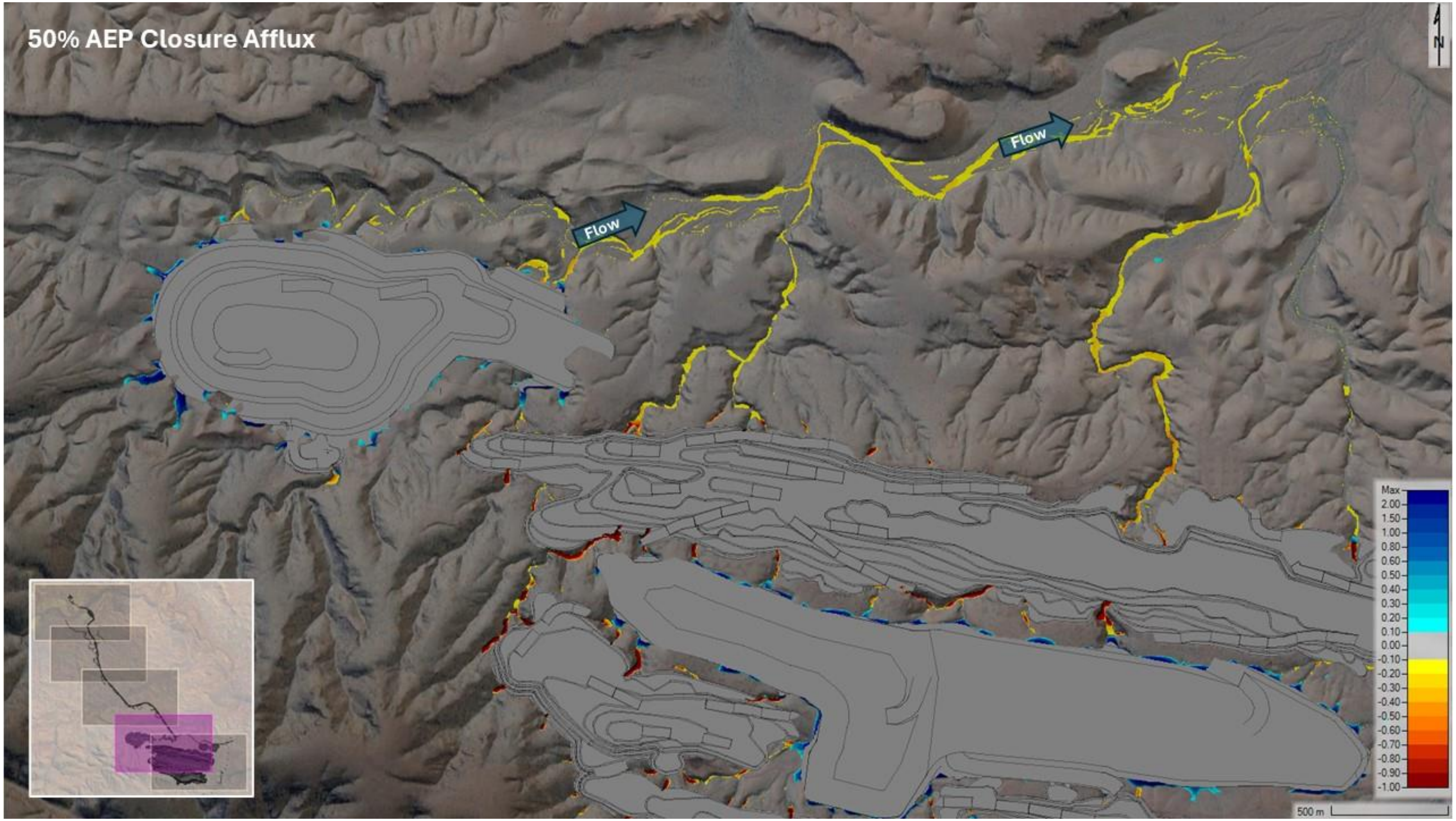
C-5 50% AEP LOM Afflux (Location 5)



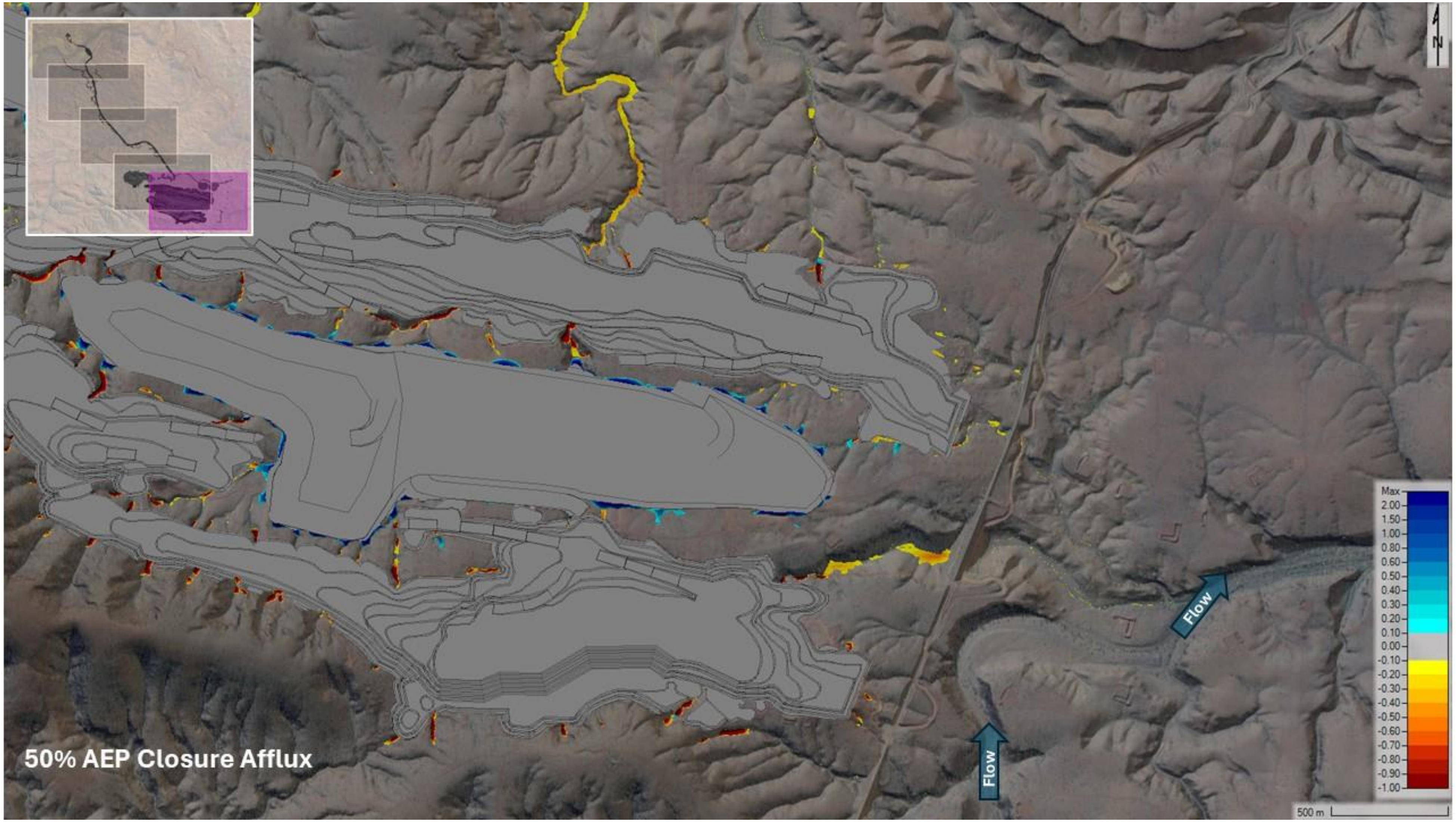
C-6 **50% AEP Closure Afflux (Location 1)**

C-7 50% AEP Closure Afflux (Location 2)

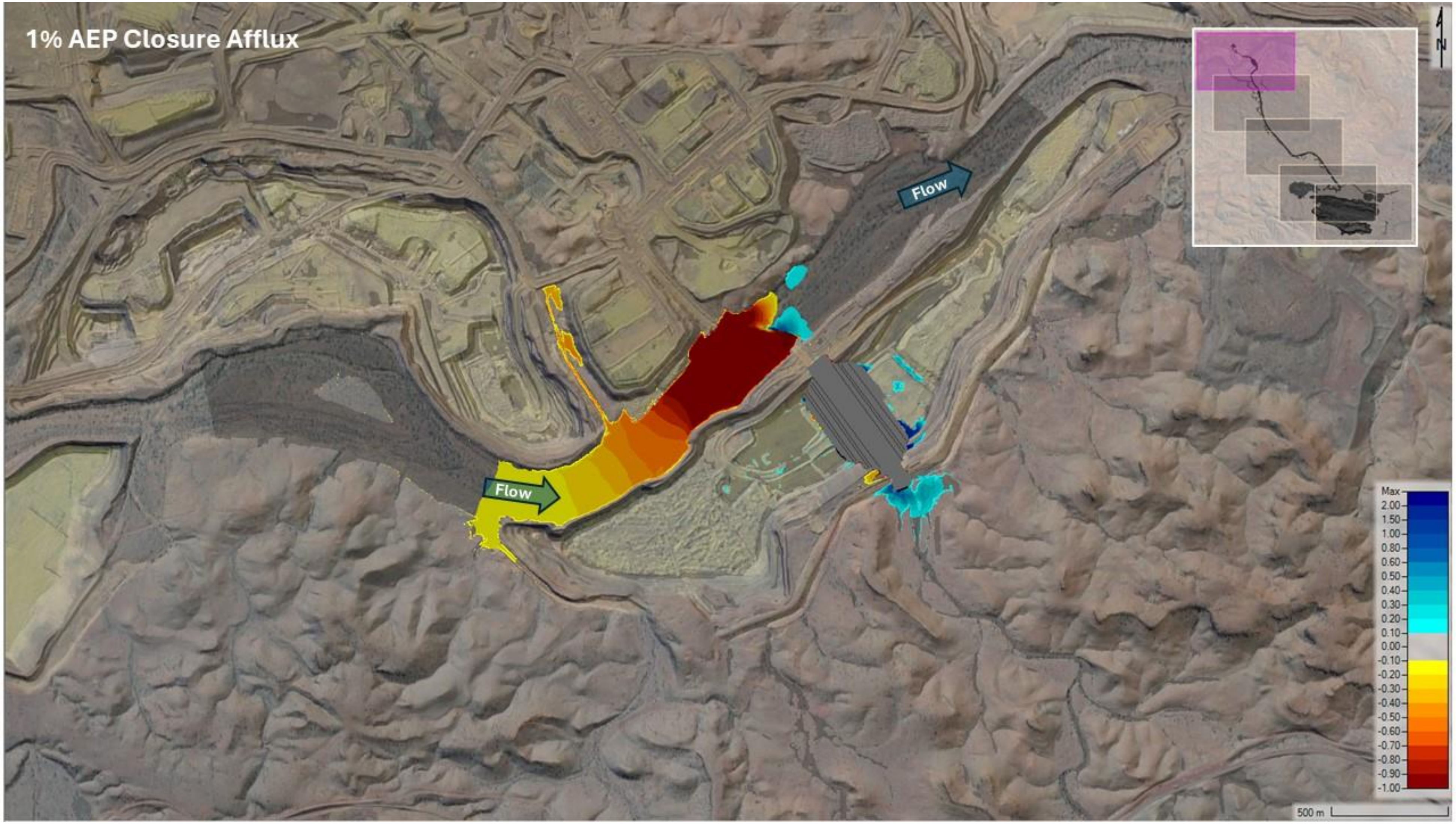
C-8 50% AEP Closure Afflux (Location 3)



C-9 50% AEP Closure Afflux (Location 4)



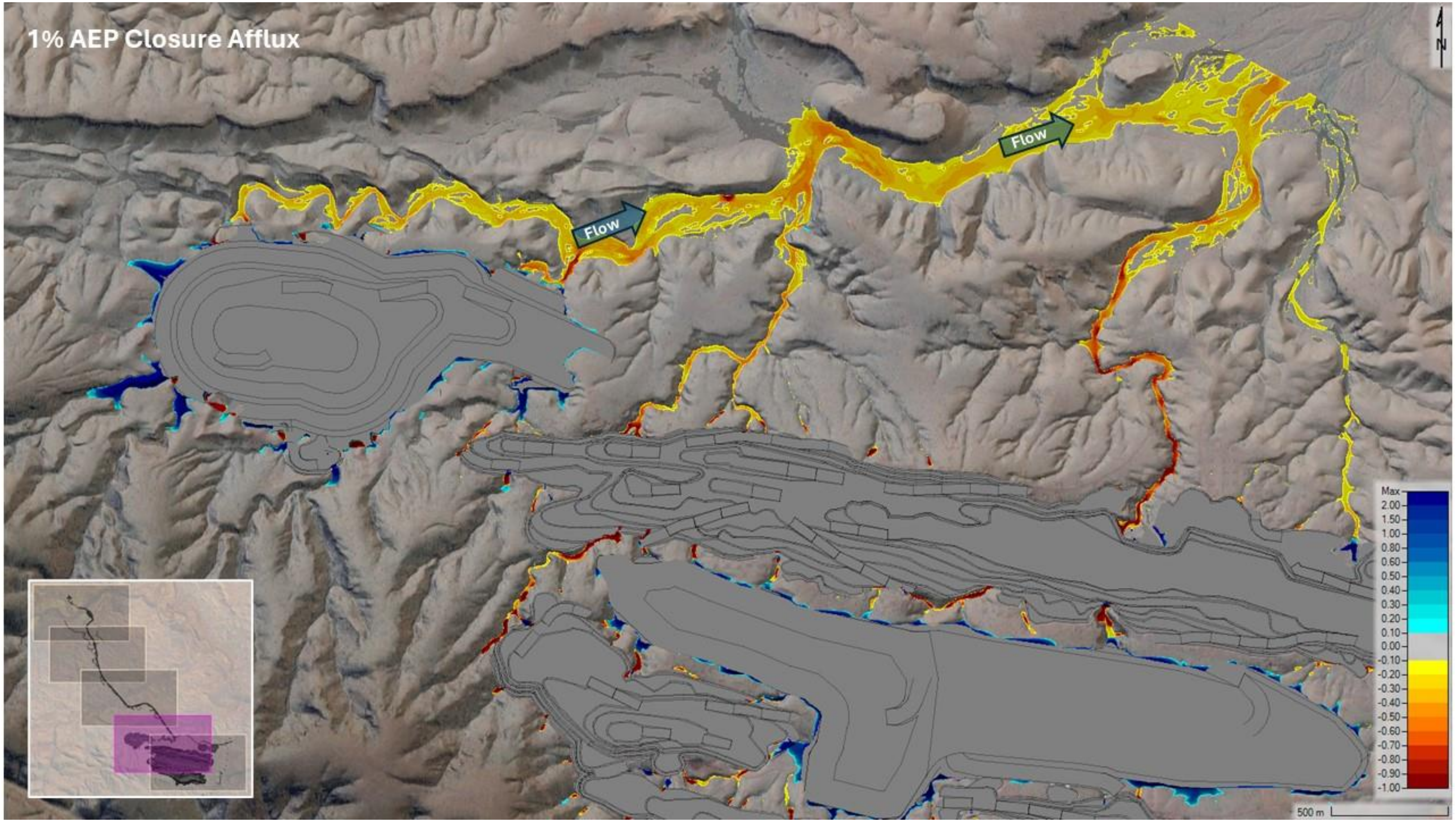
C-10 50% AEP Closure Afflux (Location 5)



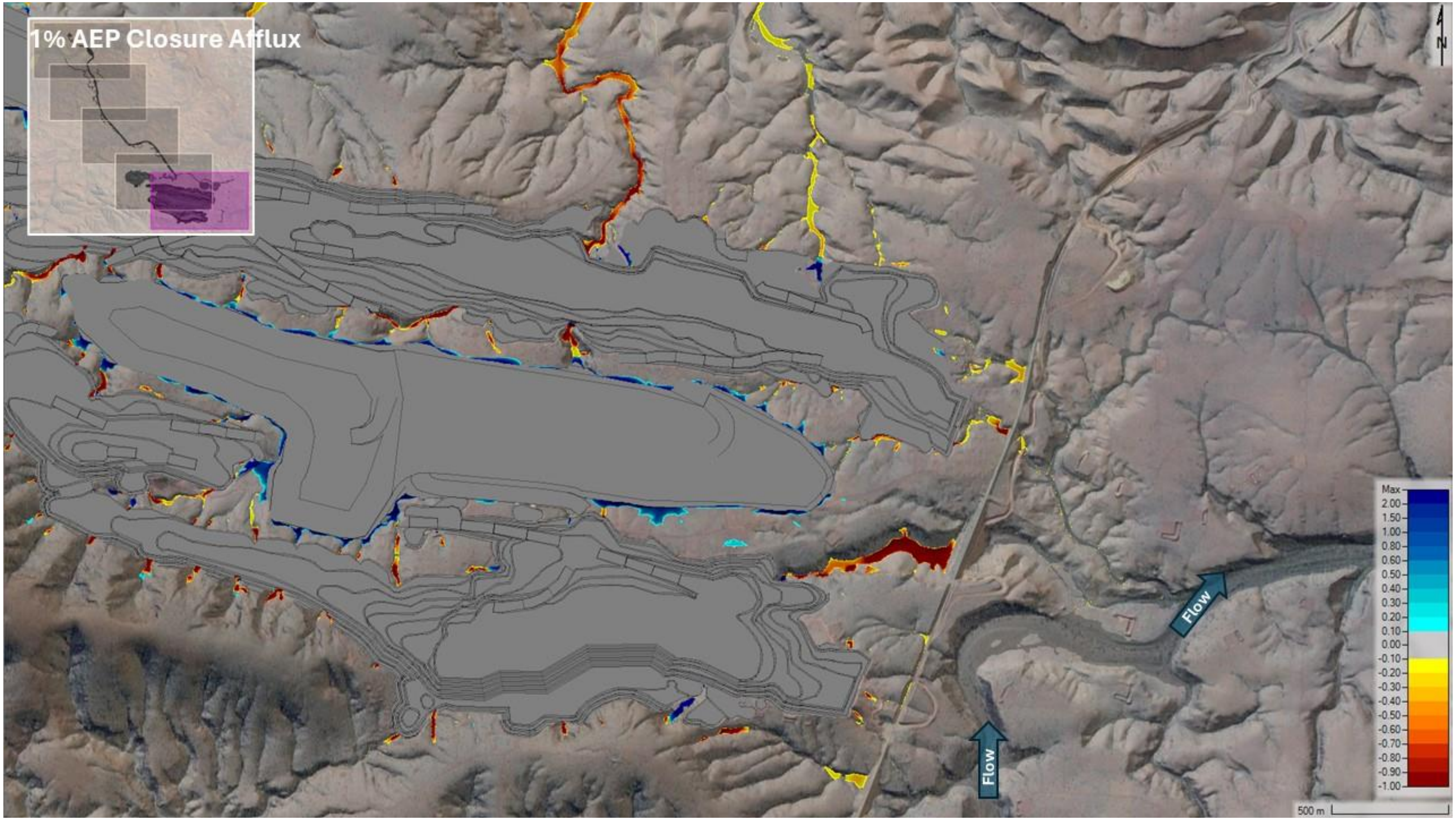
C-11 1% AEP Closure Afflux (Location 1)

C-12 1% AEP Closure Afflux (Location 2)

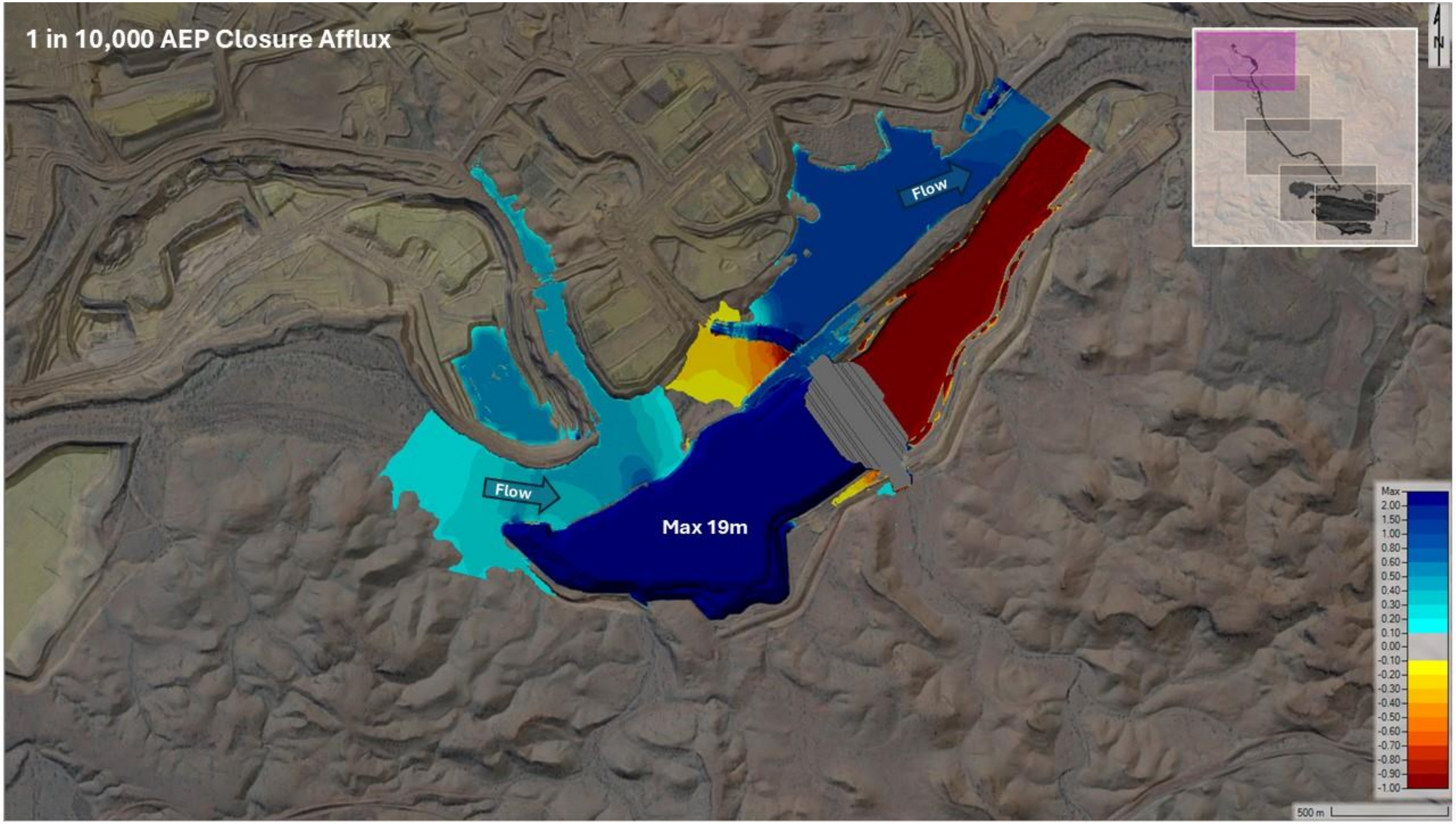
C-13 1% AEP Closure Afflux (Location 3)



C-14 1% AEP Closure Afflux (Location 4)



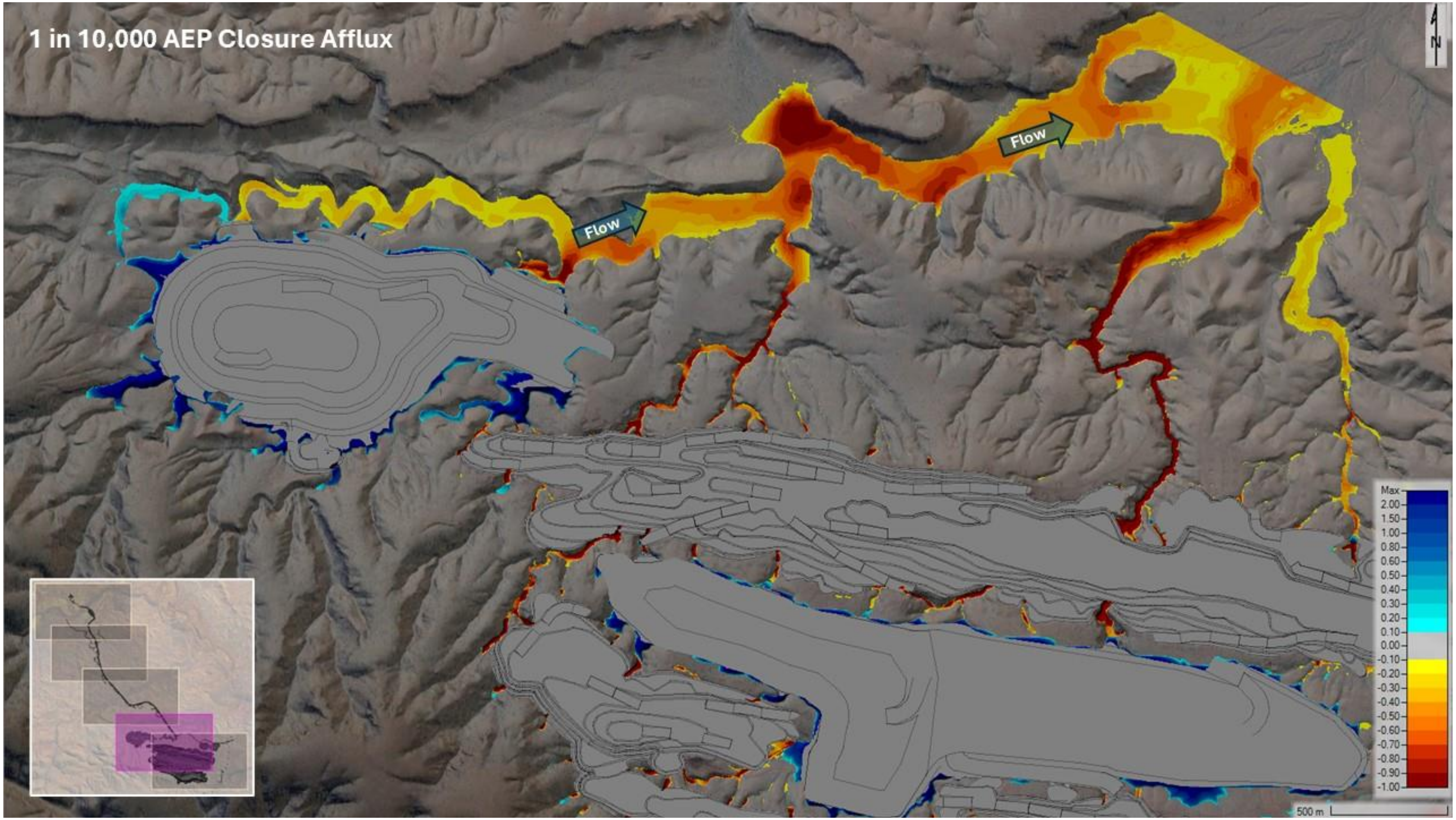
C-15 1% AEP Closure Afflux (Location 5)



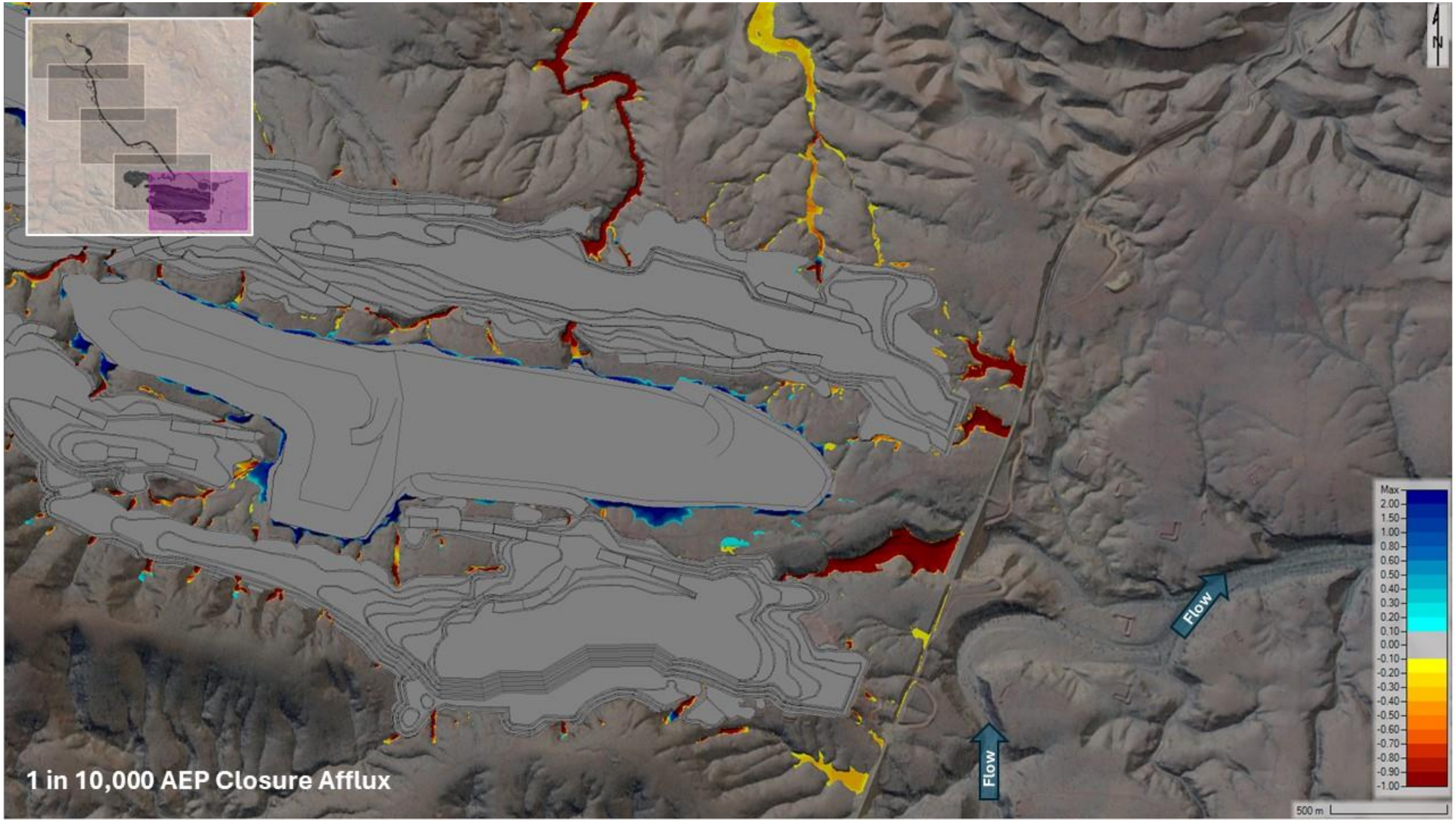
C-16 1 in 10,000 AEP Closure Afflux (Location 1)

C-17 1 in 10,000 AEP Closure Afflux (Location 2)

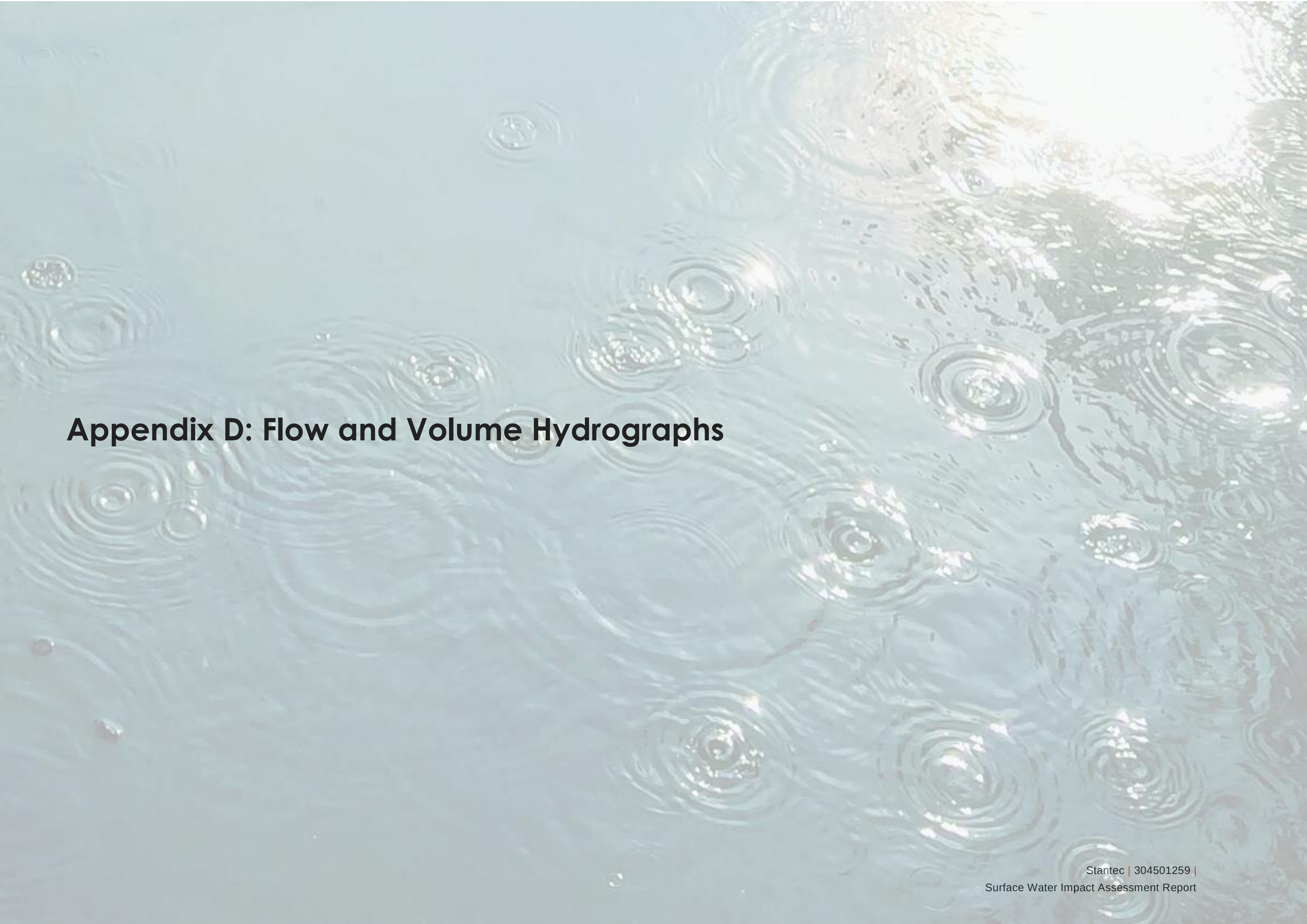
C-18 1 in 10,000 AEP Closure Afflux (Location 3)



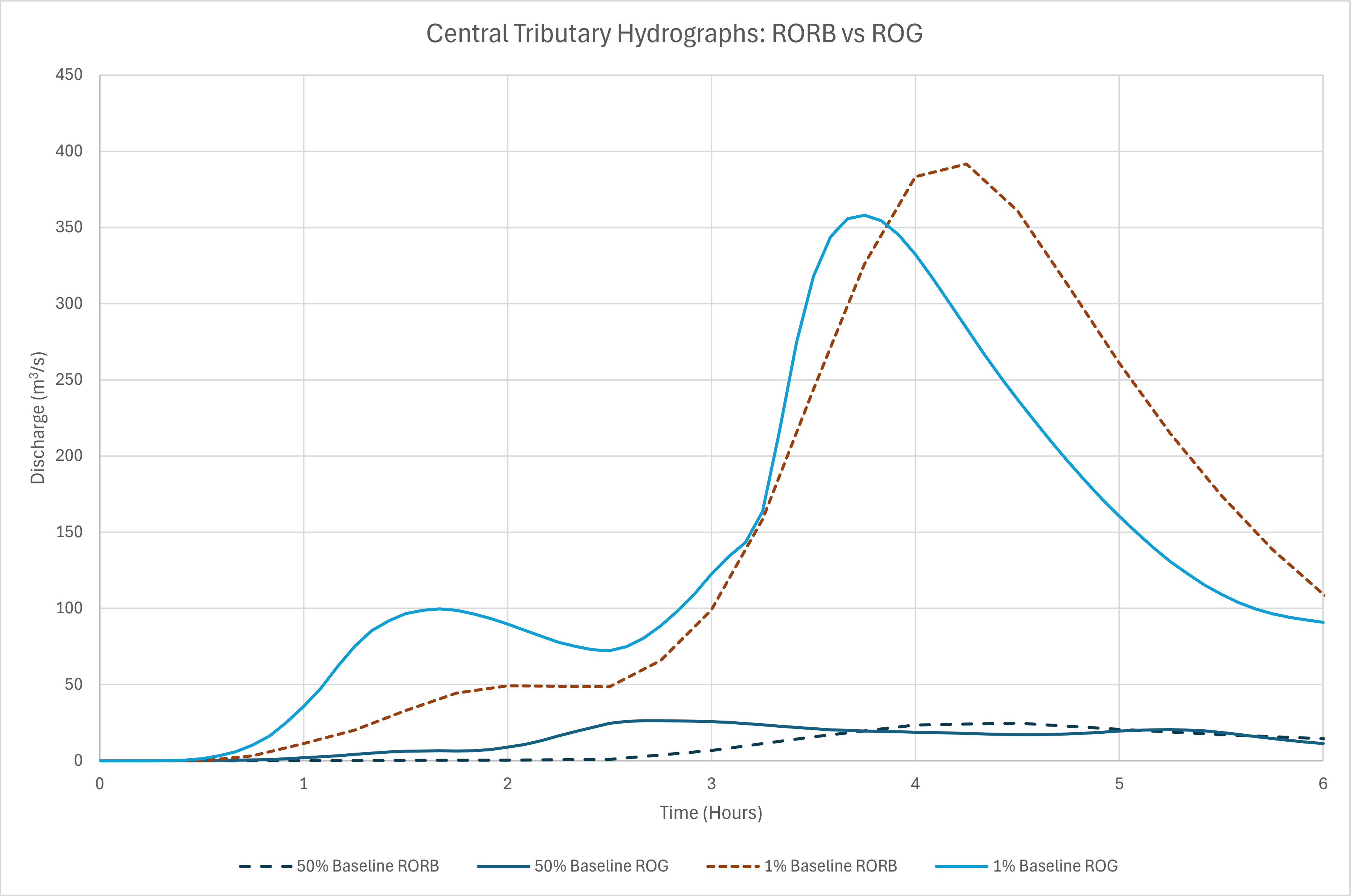
C-19 1 in 10,000 AEP Closure Afflux (Location 4)



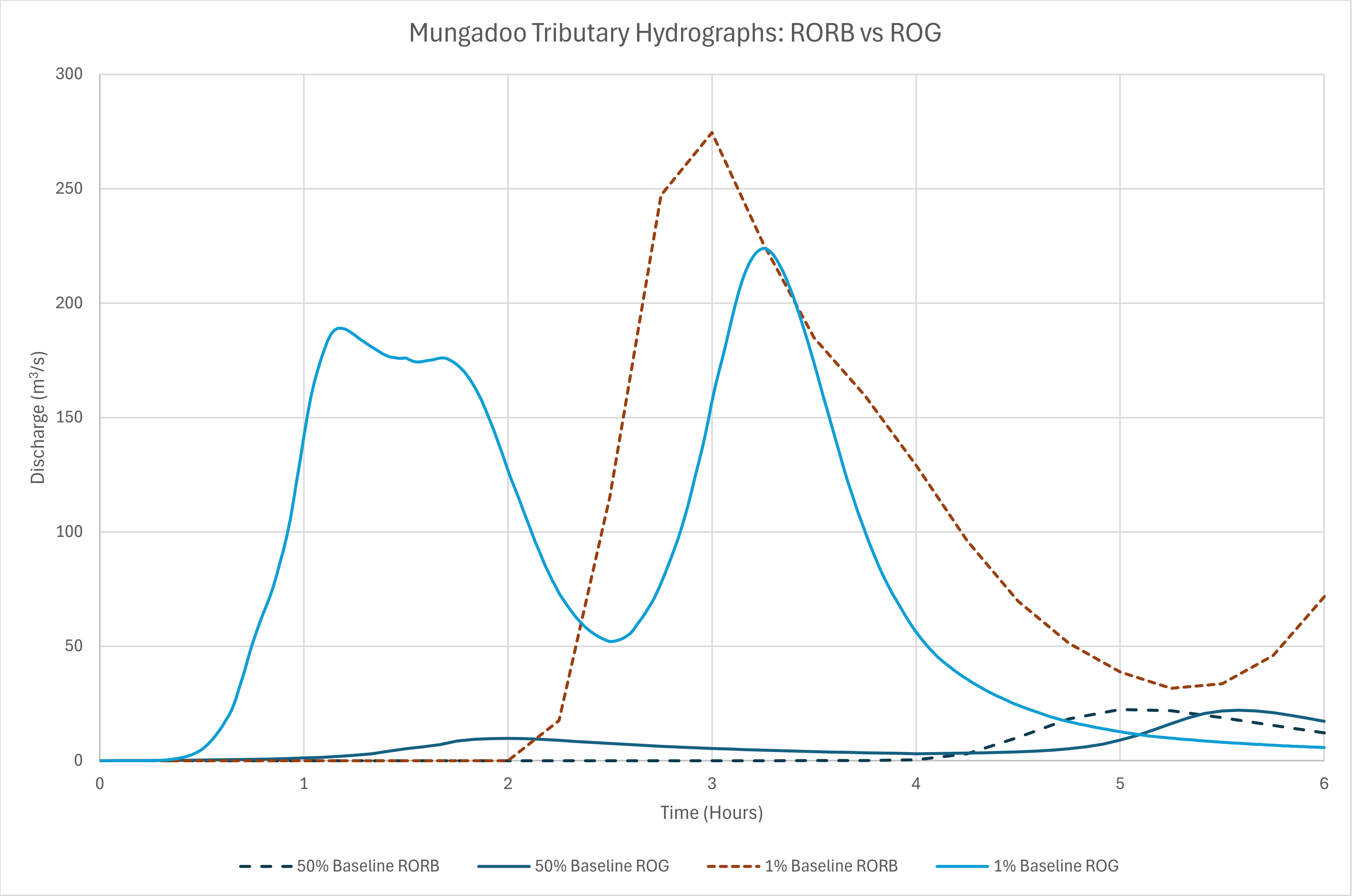
C-20 1 in 10,000 AEP Closure Afflux (Location 5)



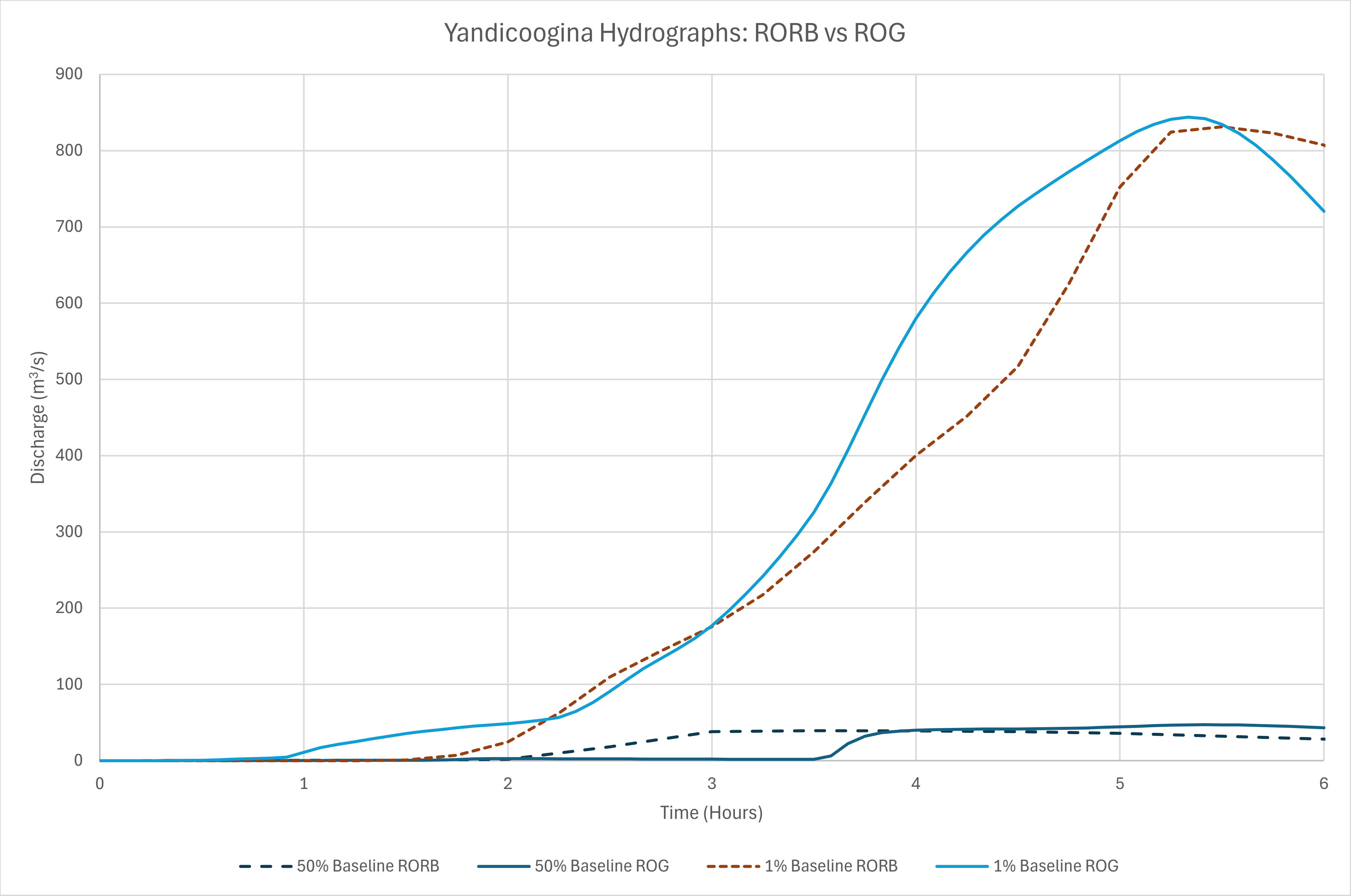
Appendix D: Flow and Volume Hydrographs



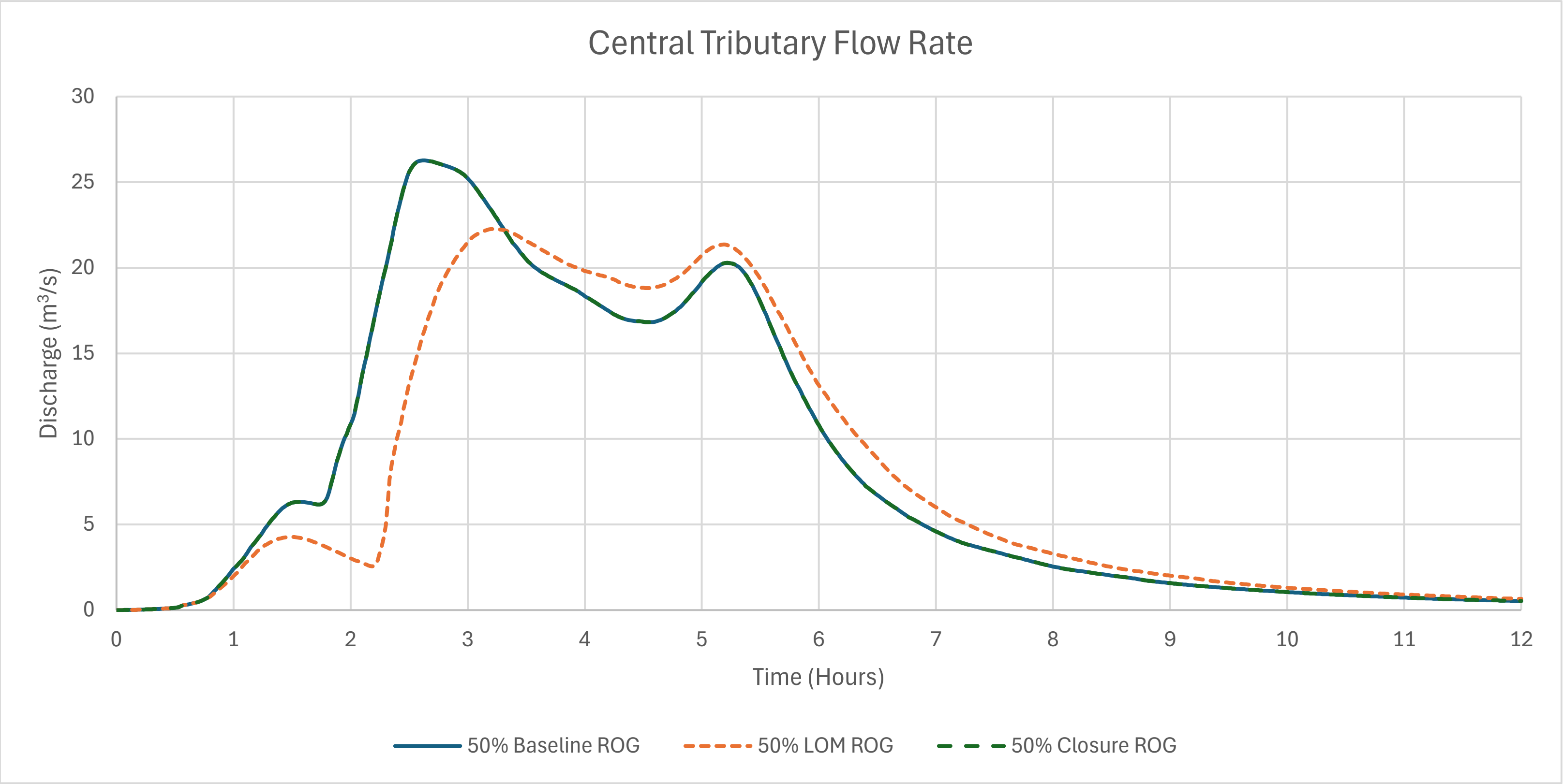
D-1 Central Tributary Hydrographs (RORB vs ROG)



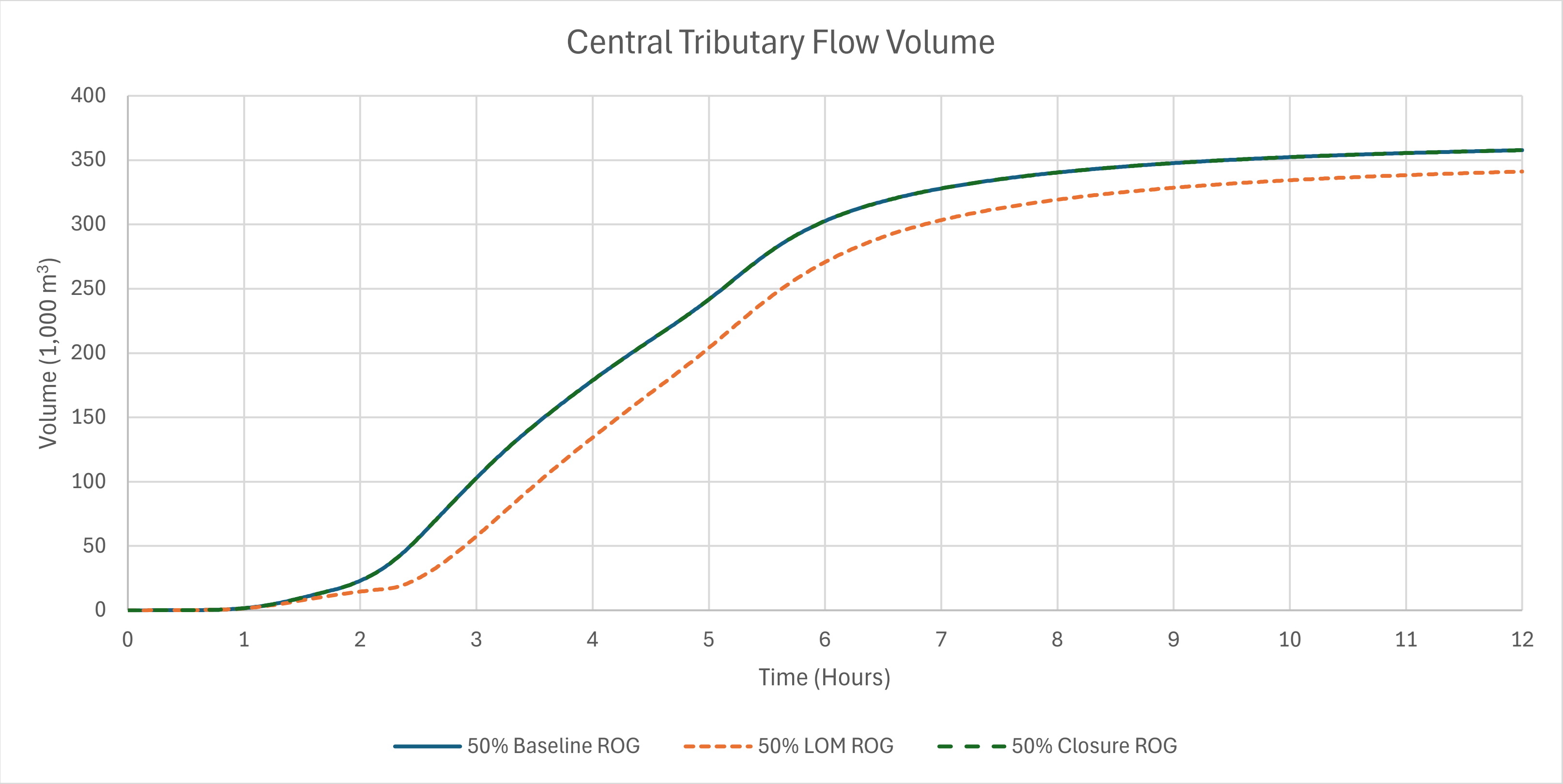
D-2 Mungadoo Tributary Hydrographs (RORB vs ROG)



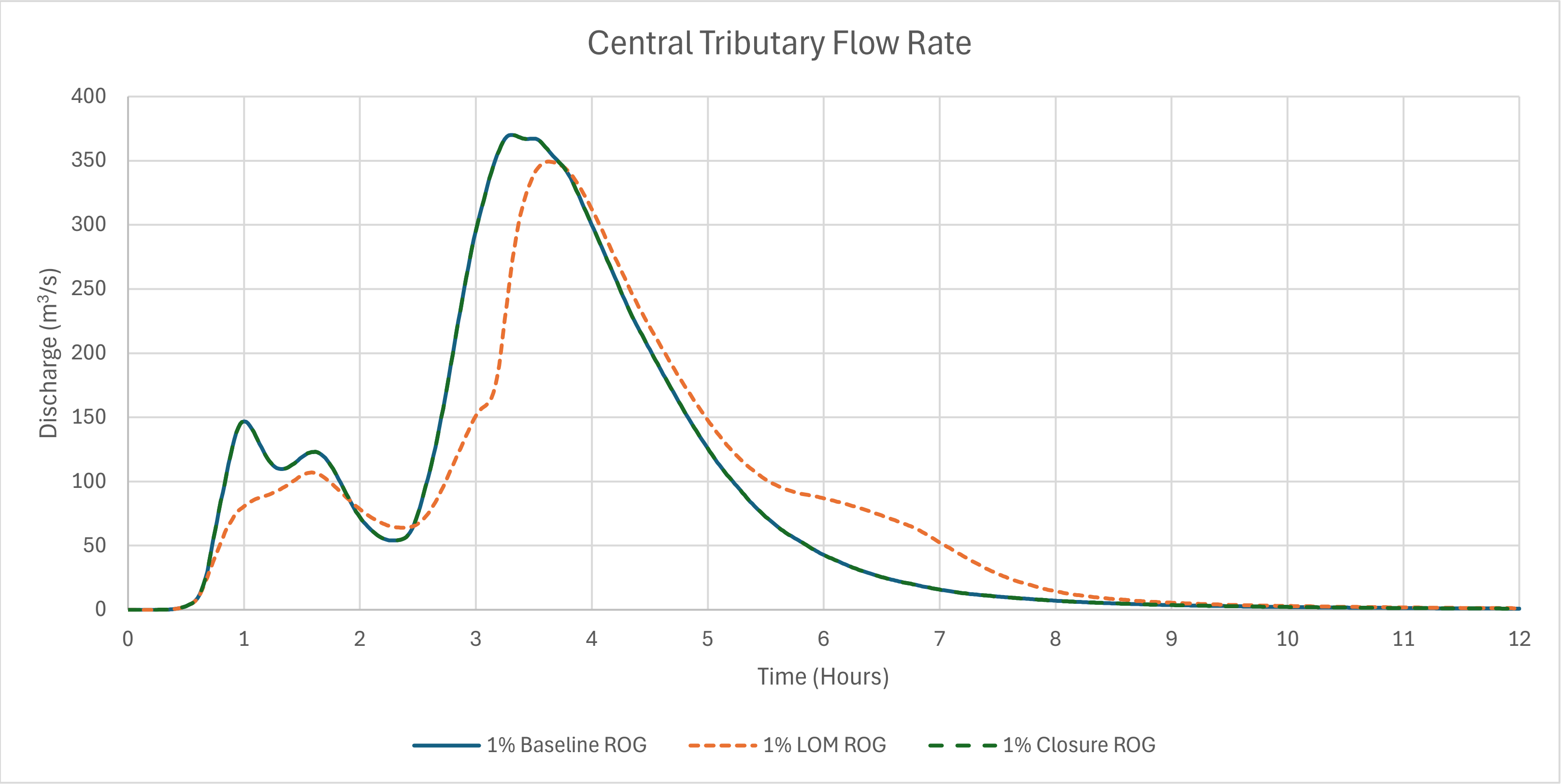
D-3 Yandicoogina Hydrographs (RORB vs ROG)



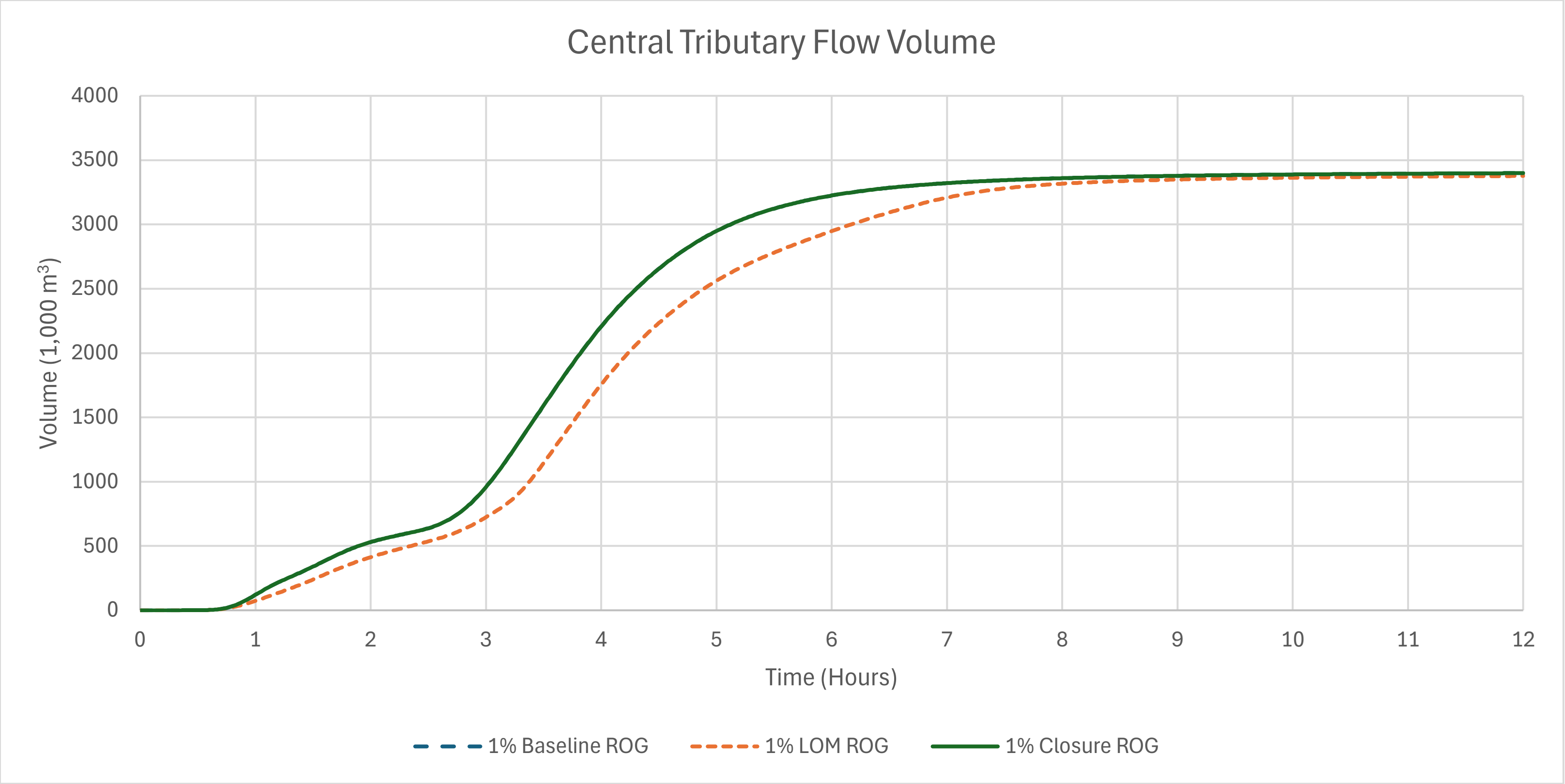
D-4 **Central Tributary 50% AEP Flow Hydrographs (Baseline, LOM and Closure)**



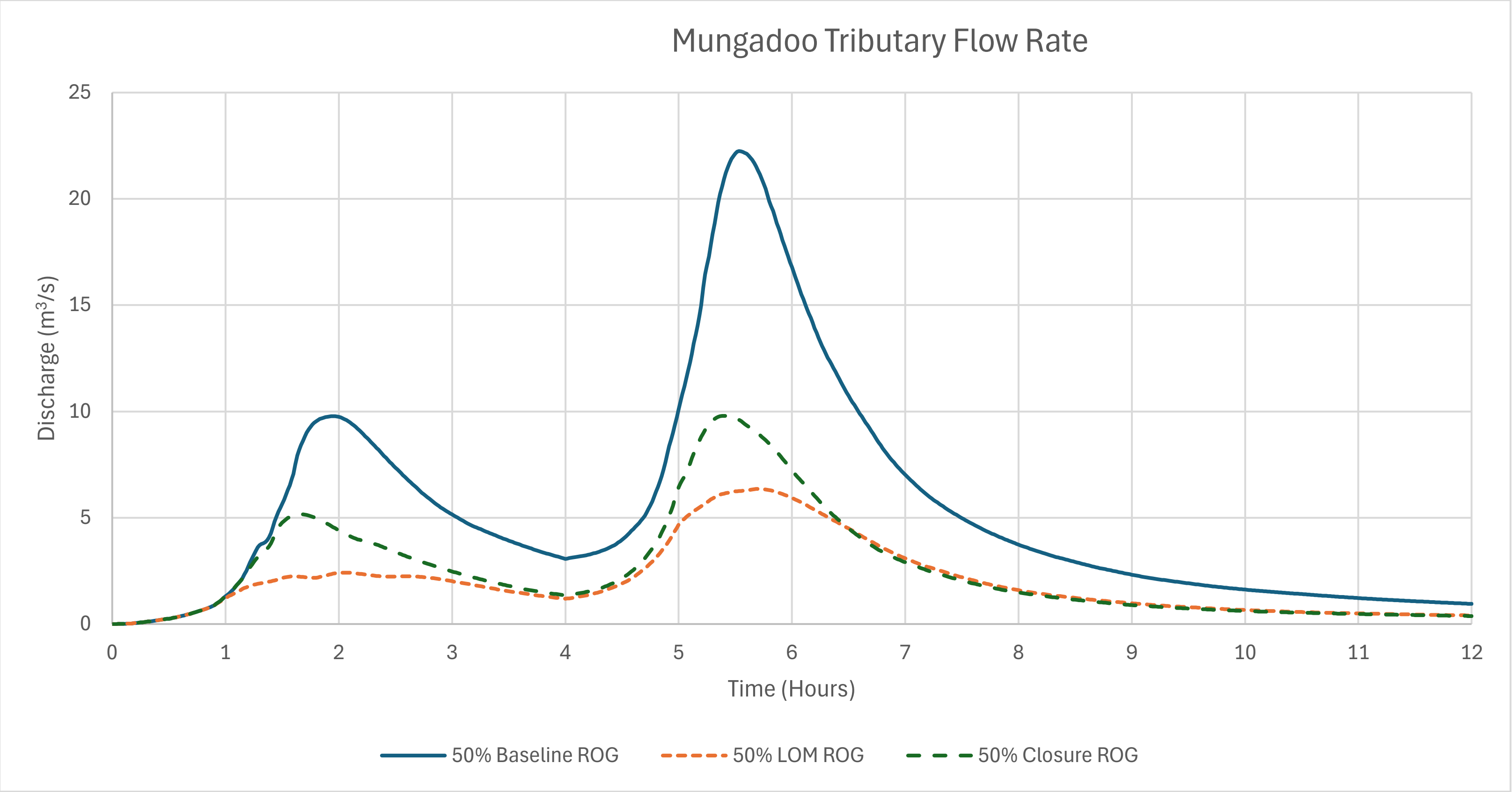
D-5 Central Tributary 50% AEP Cumulative Flow Volume (Baseline, LOM and Closure)



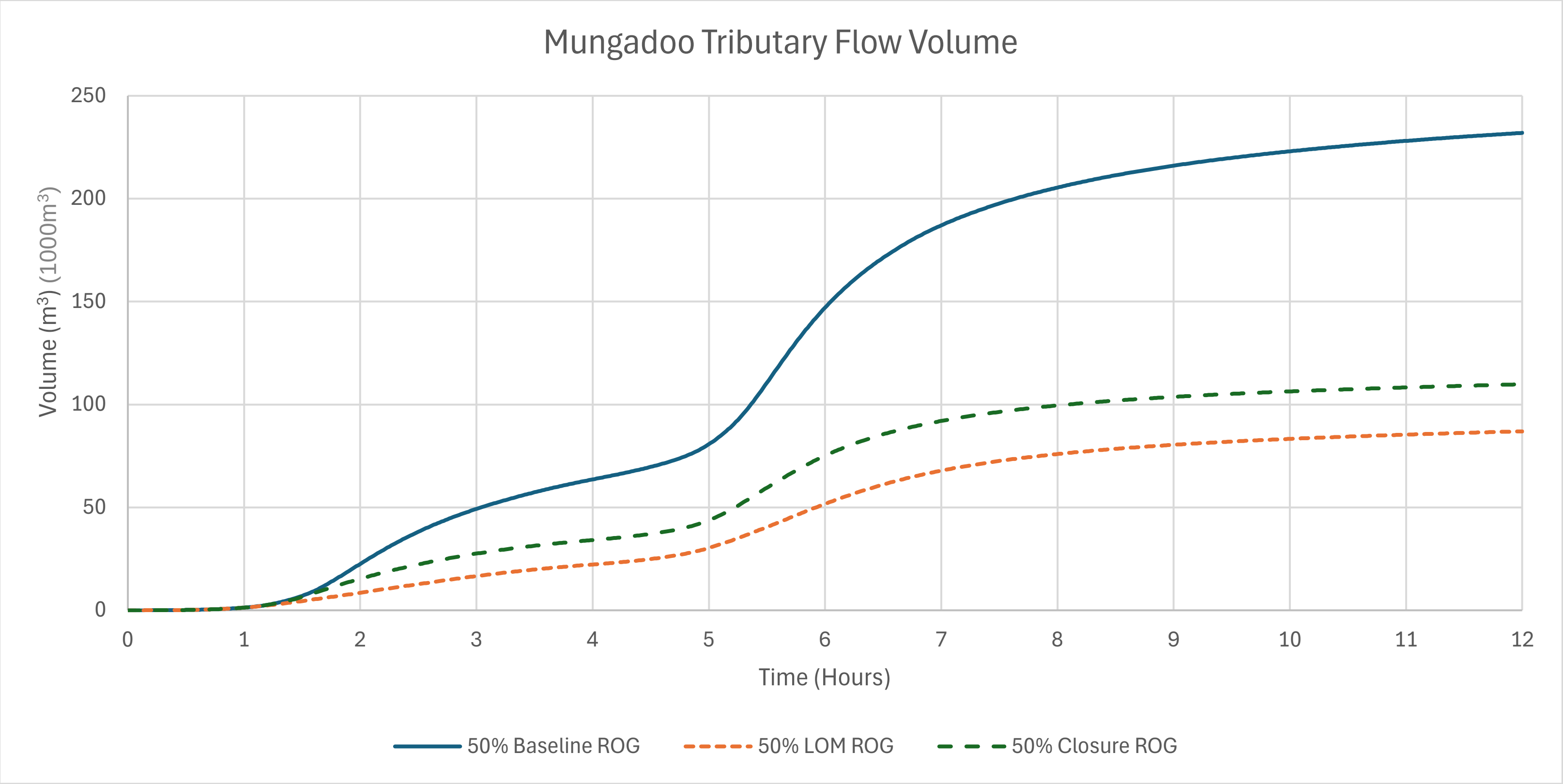
D-6 Central Tributary 1% AEP Flow Hydrographs (Baseline, LOM and Closure)



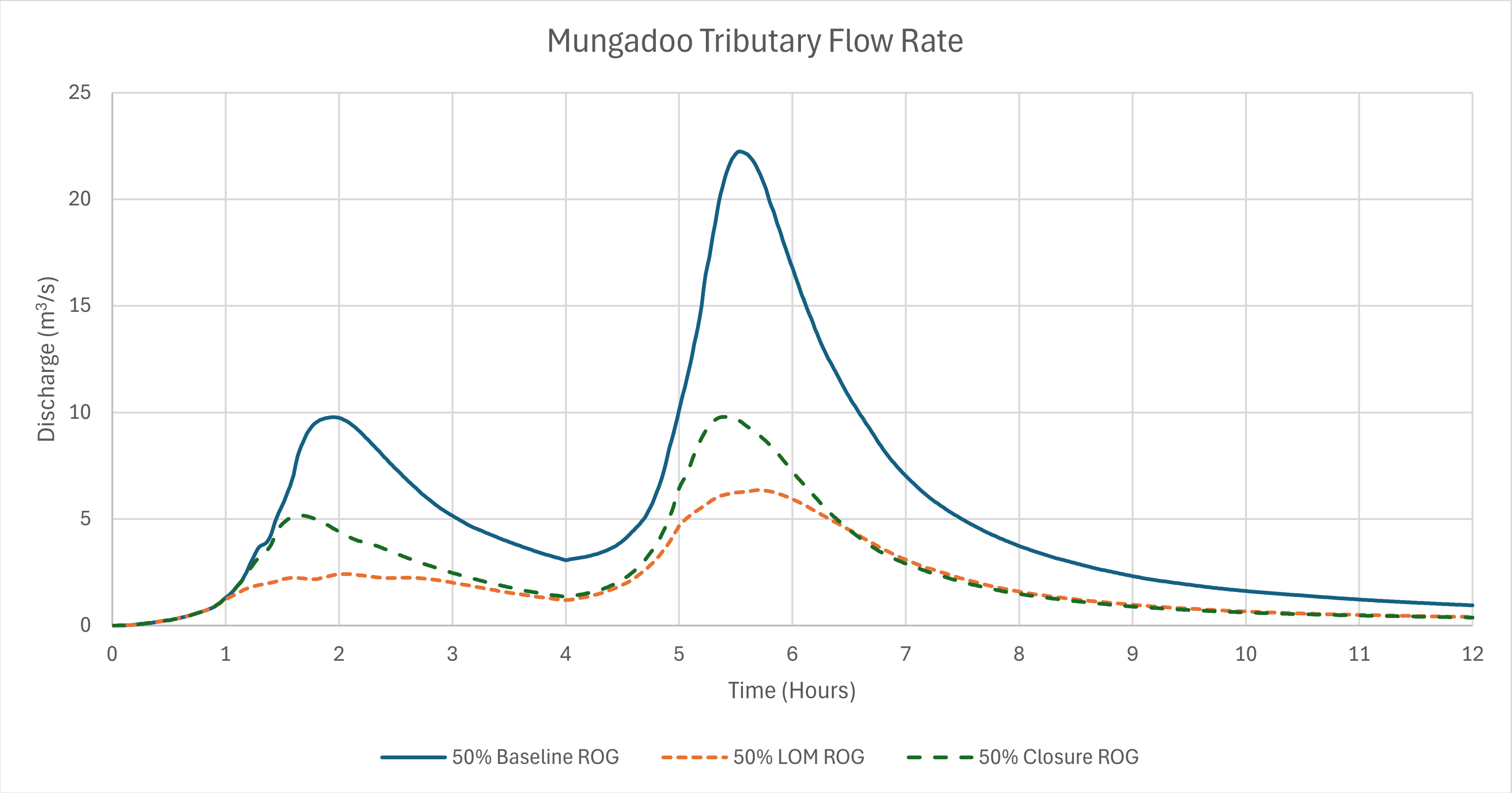
D-7 Central Tributary 1% AEP Cumulative Flow Volume (Baseline, LOM and Closure)



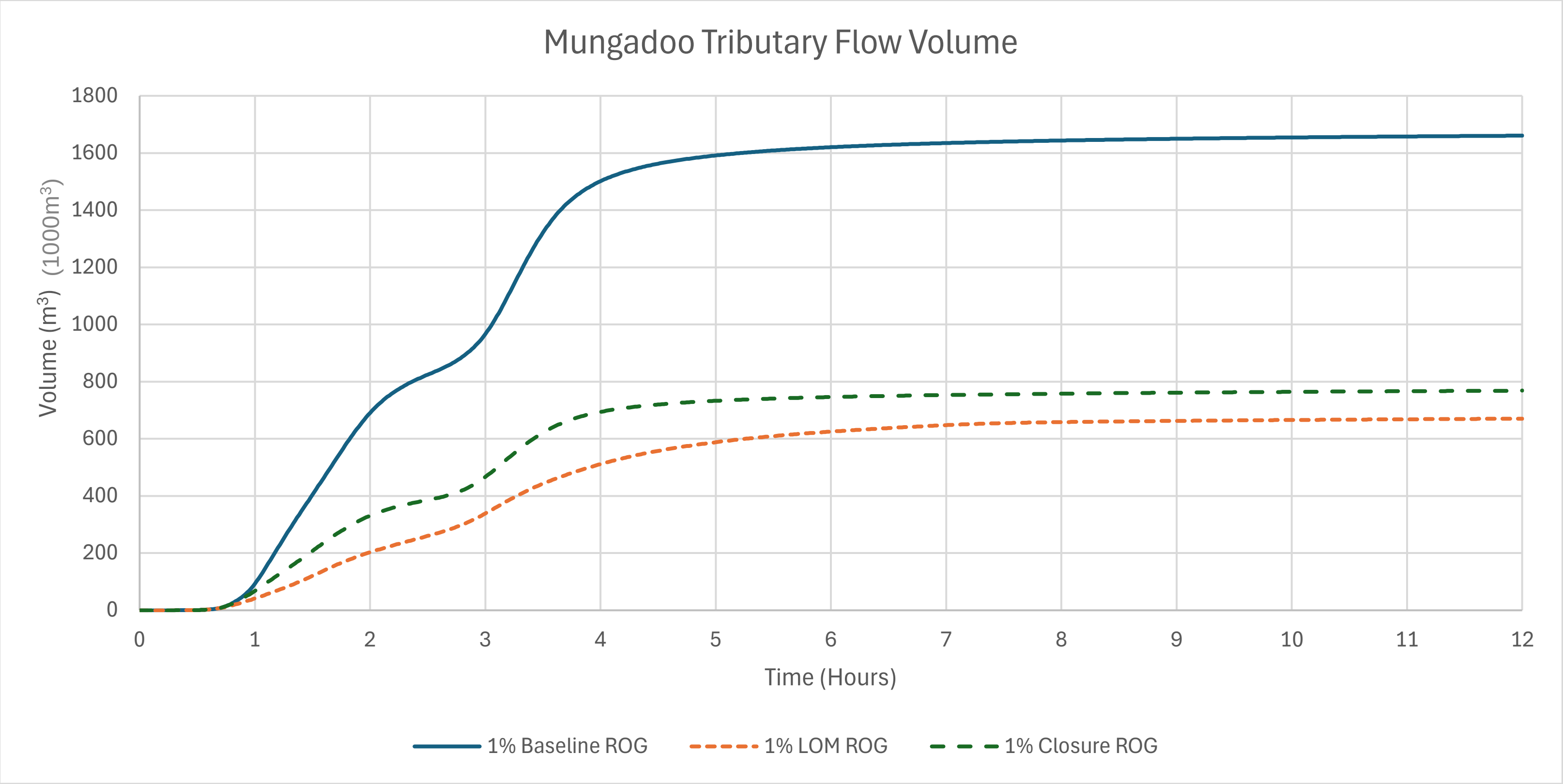
D-8 Mungadoo Tributary 50% AEP Flow Hydrographs (Baseline, LOM and Closure)



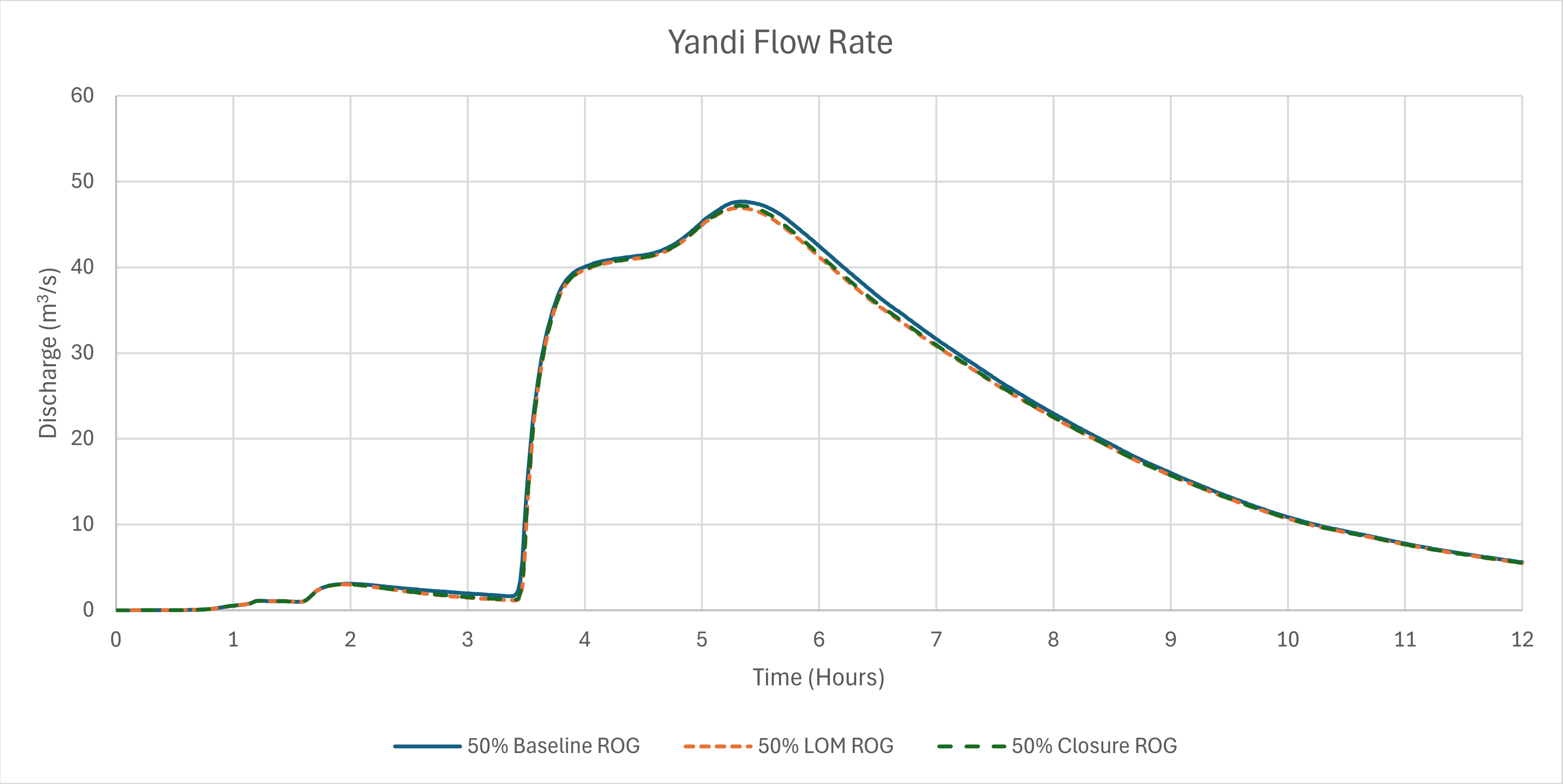
D-9 Mungadoo Tributary 50% AEP Cumulative Flow Volume (Baseline, LOM and Closure)



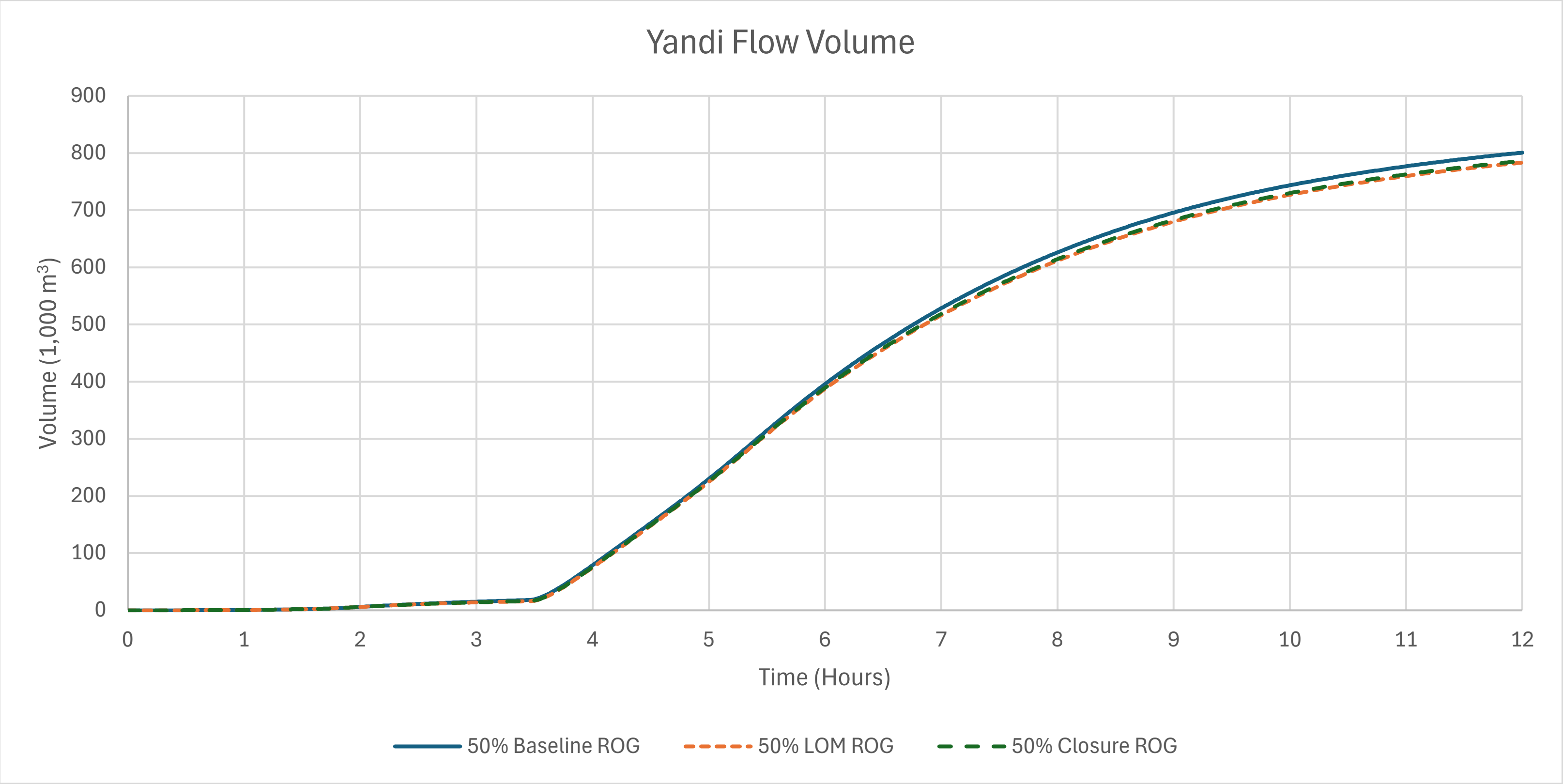
D-10 Mungadoo Tributary 1% AEP Flow Hydrographs (Baseline, LOM and Closure)



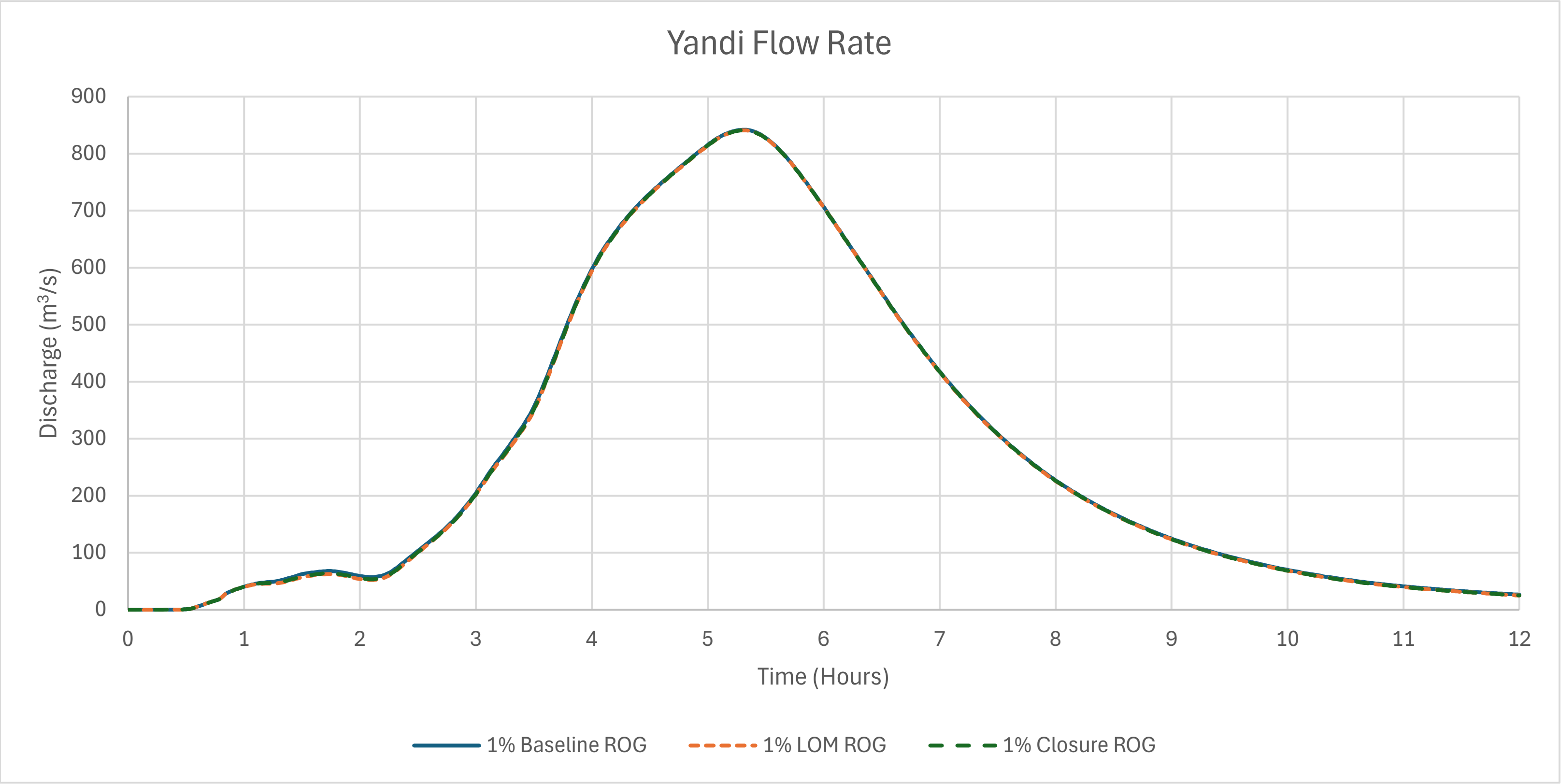
D-11 Mungadoo Tributary 1% AEP Cumulative Flow Volume (Baseline, LOM and Closure)



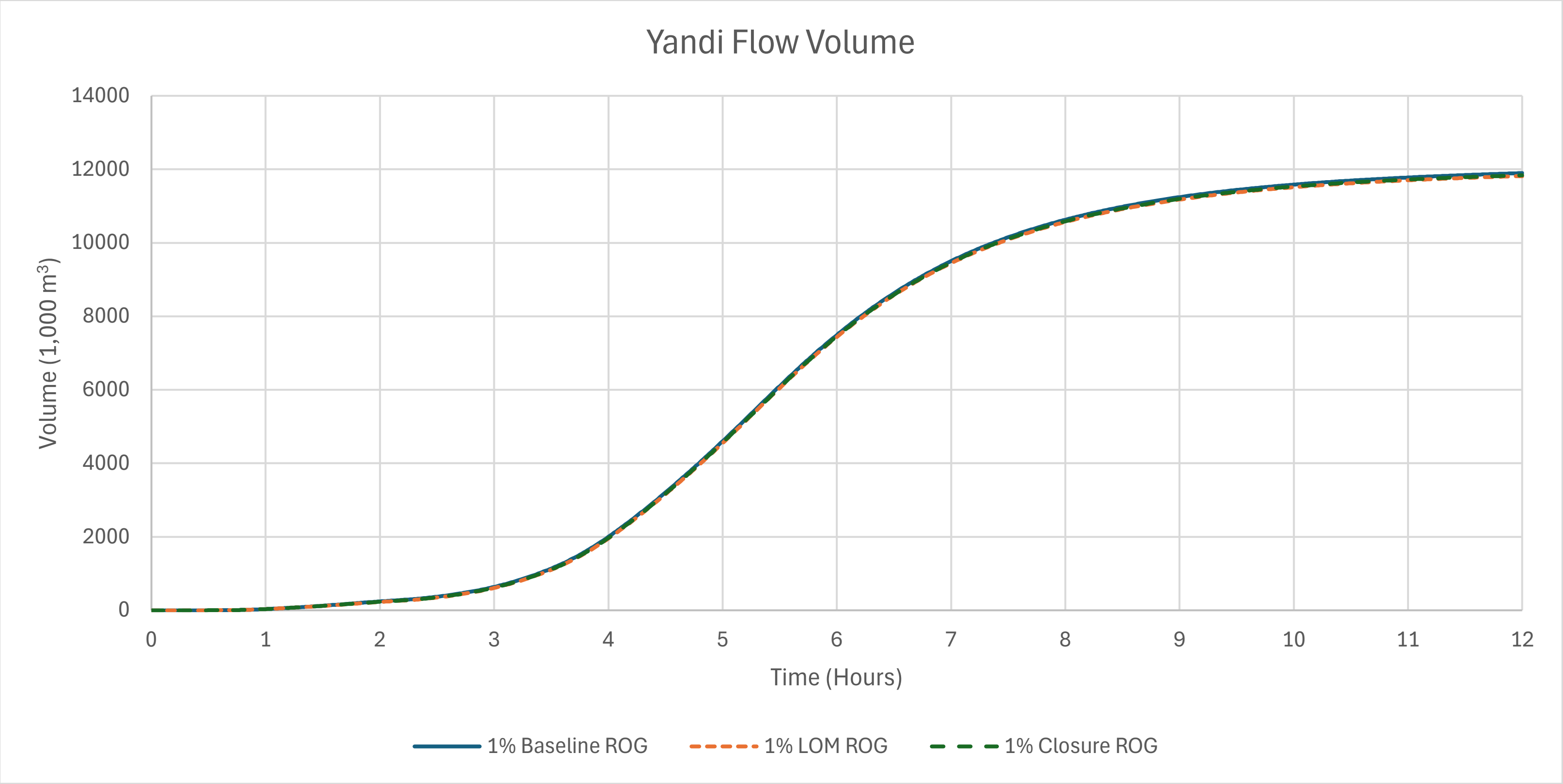
D-12 Yandicoogina 50% AEP Flow Hydrographs (Baseline, LOM and Closure)



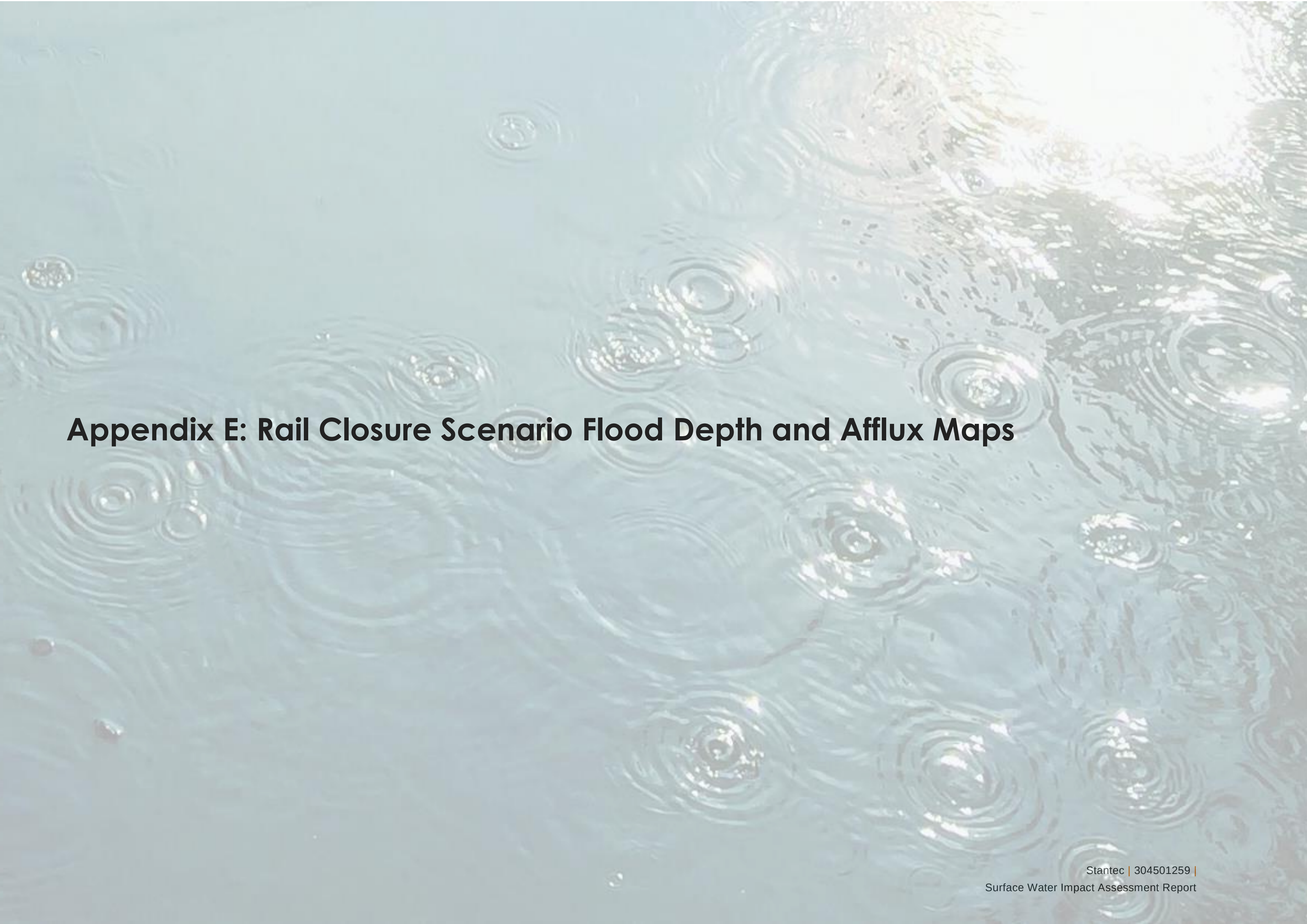
D-13 Yandicoogina 50% AEP Cumulative Flow Volume (Baseline, LOM and Closure)



D-14 Yandicoogina 1% AEP Flow Hydrographs (Baseline, LOM and Closure)



D-15 Yandicoogina 1% AEP Cumulative Flow Volume (Baseline, LOM and Closure)

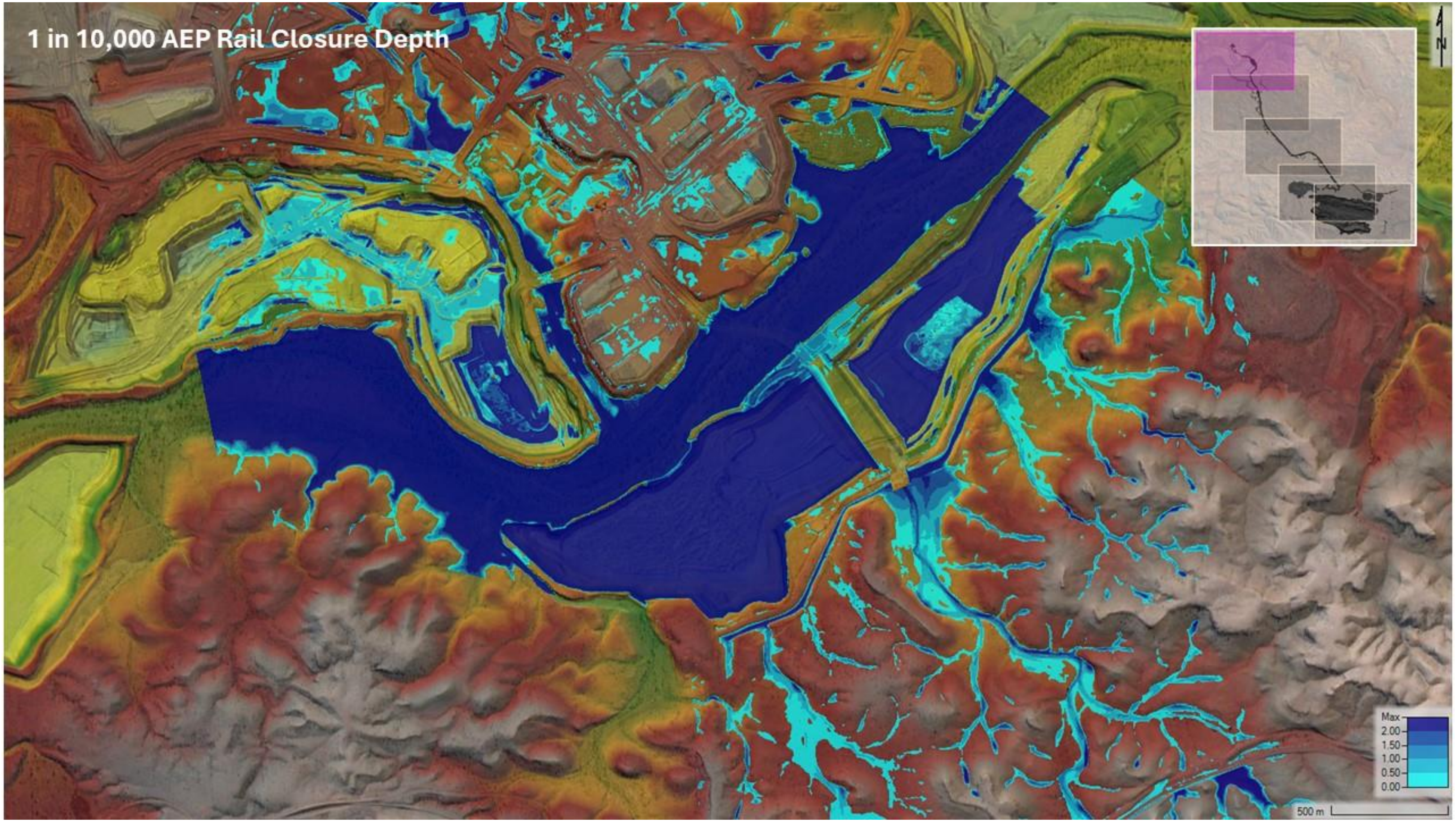


Appendix E: Rail Closure Scenario Flood Depth and Afflux Maps

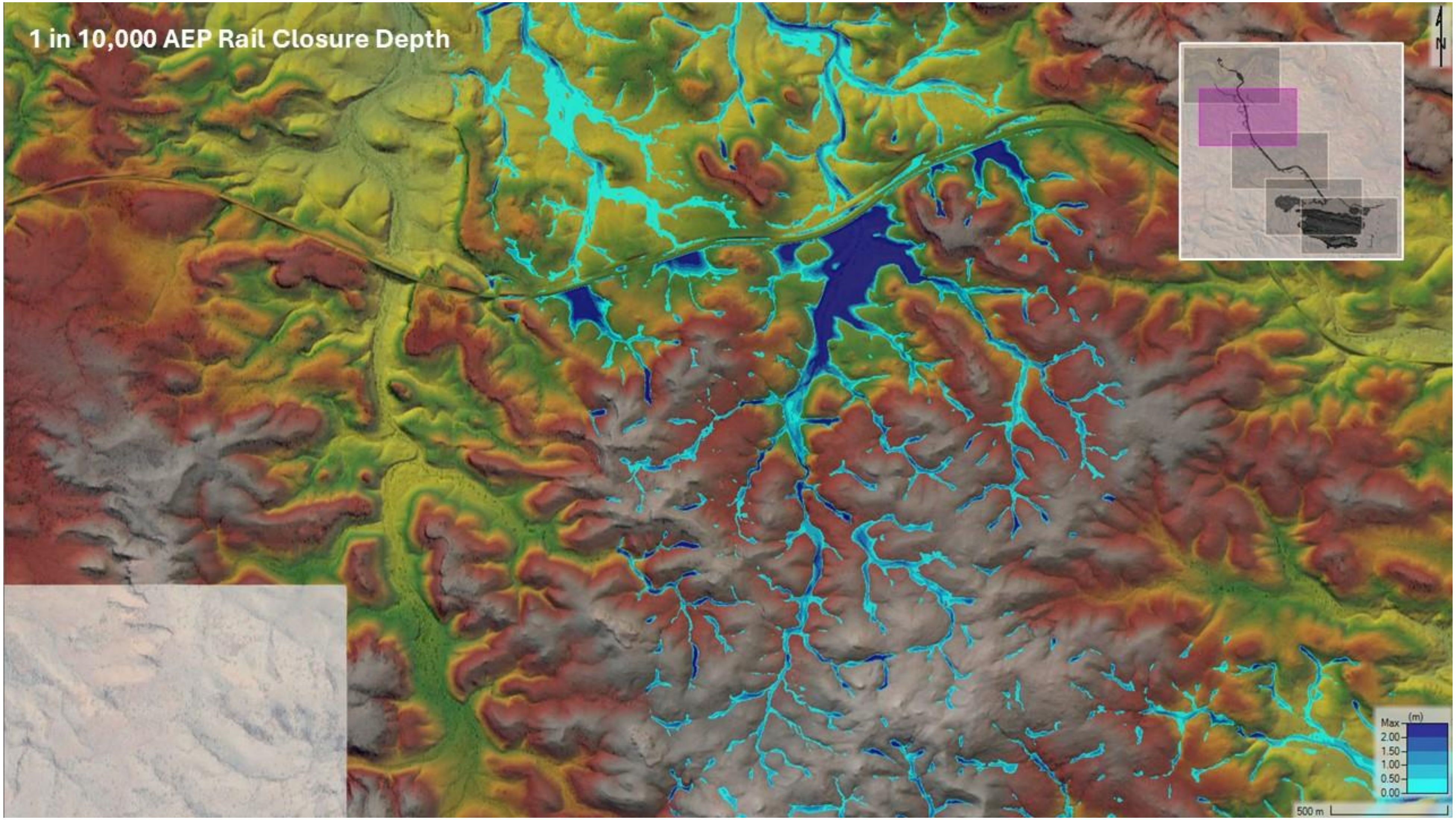
AEP	**Appendix E**	
	Depths	WSEL Afflux
50%		
20%		
10%		
5%		
1%		
1 in 10,000 **CC uplift**	Figure E1-E5	Figure E6-E10

Note: Remaining AEPs not relevant to Closure

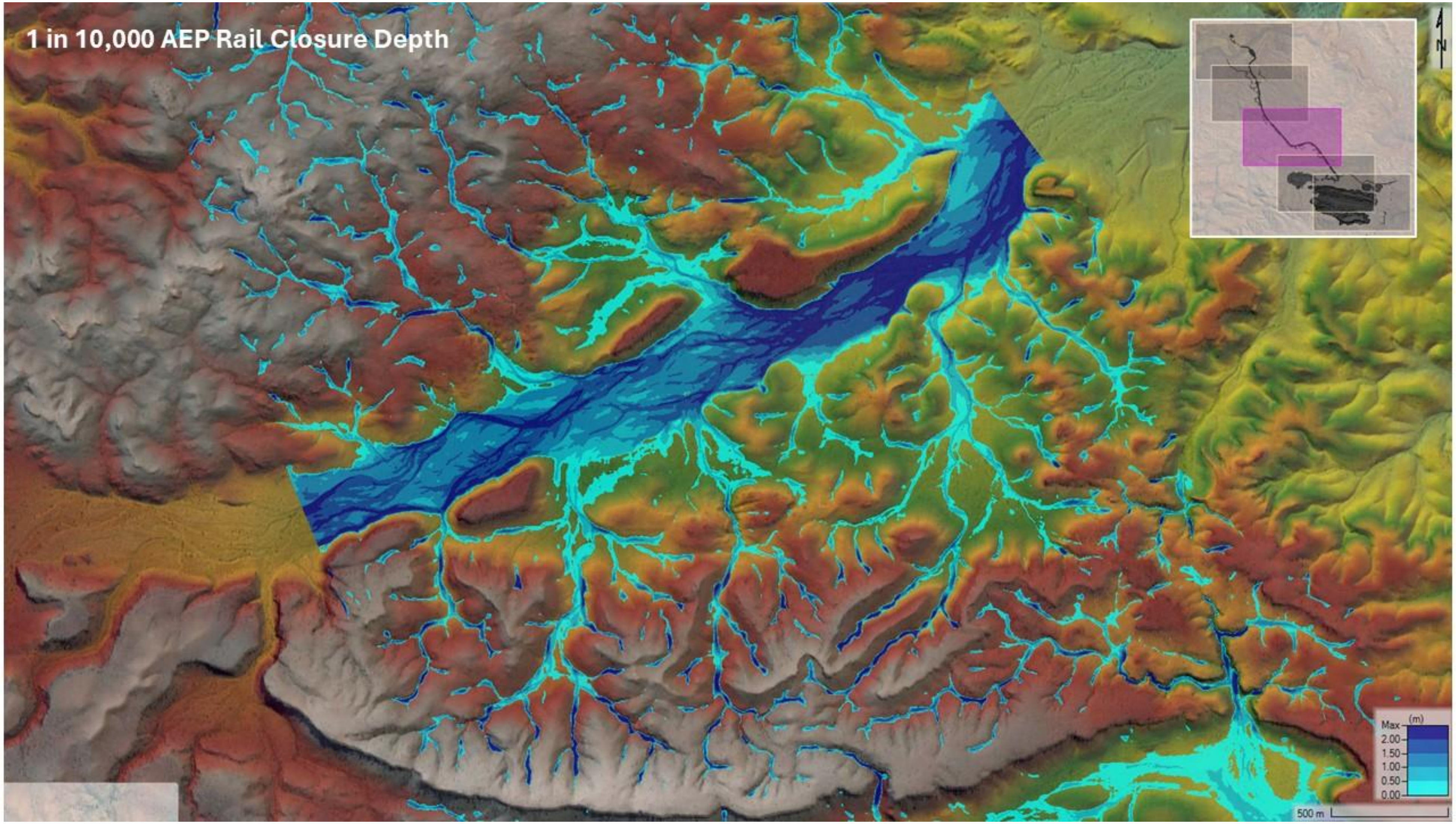
.



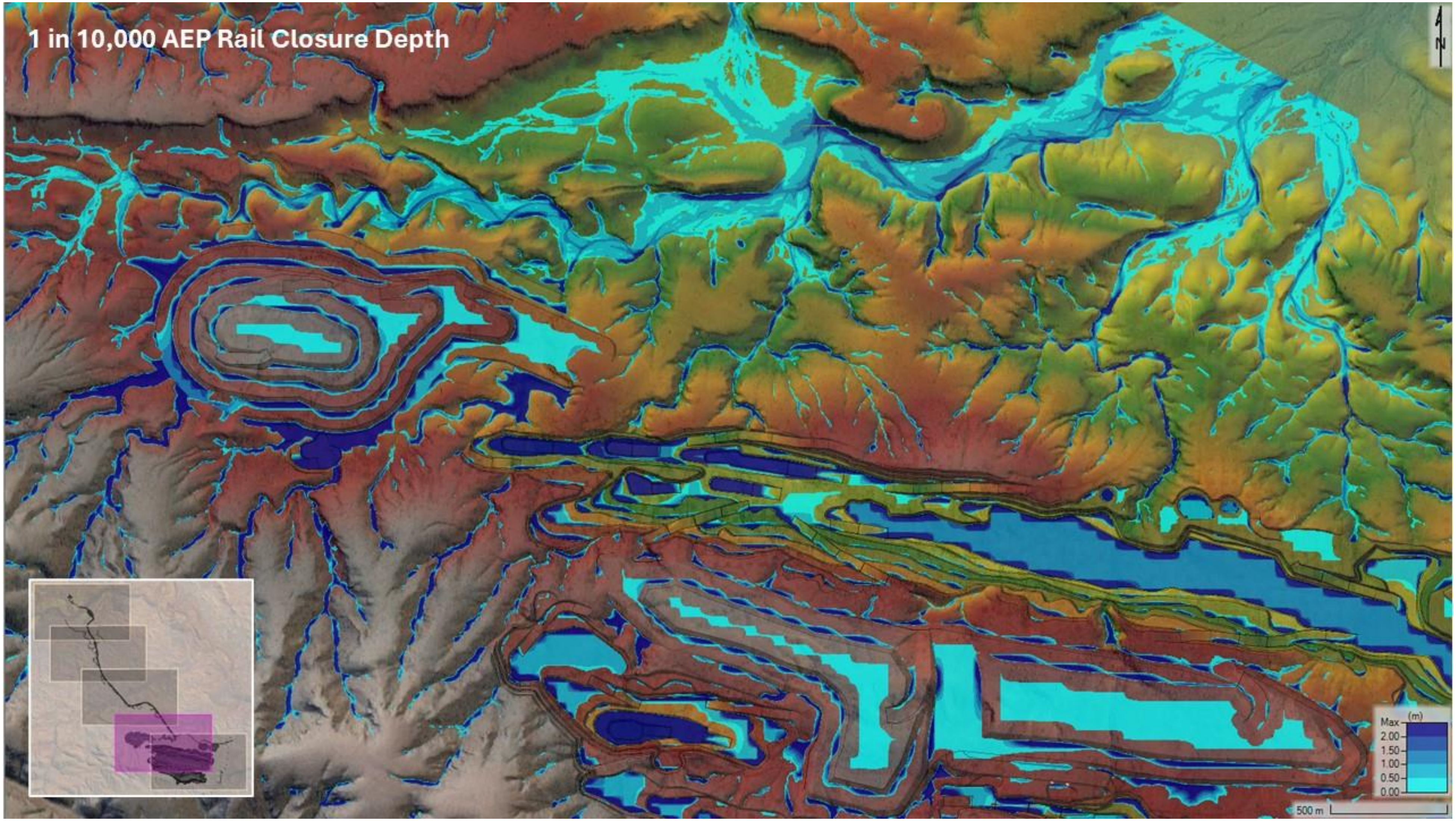
E-1 1 in 10,000 AEP Rail Closure Depth (Location 1)



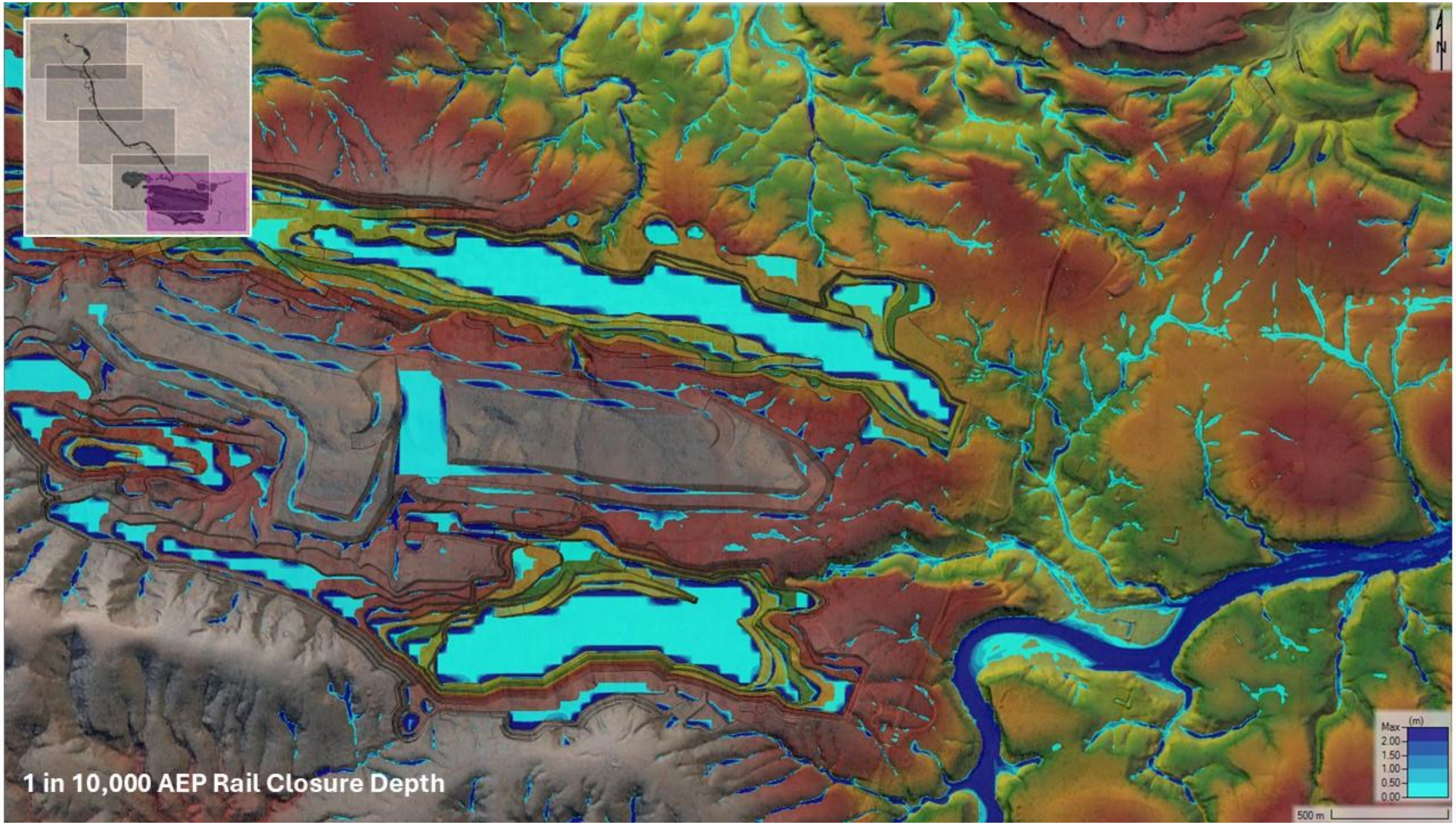
E-2 1 in 10,000 AEP Rail Closure Depth (Location 2)



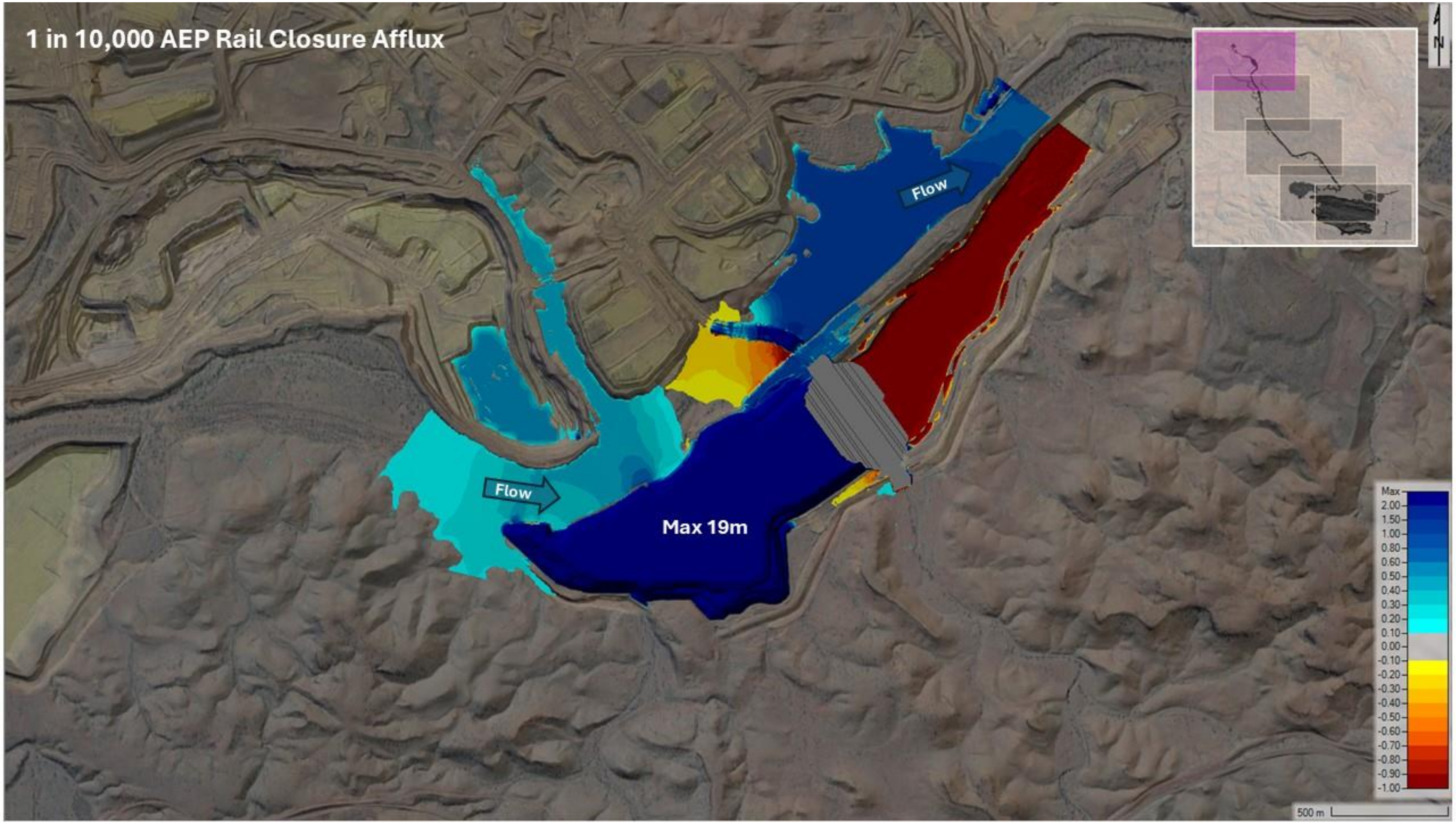
E-3 1 in 10,000 AEP Rail Closure Depth (Location 3)



E-4 1 in 10,000 AEP Rail Closure Depth (Location 4)



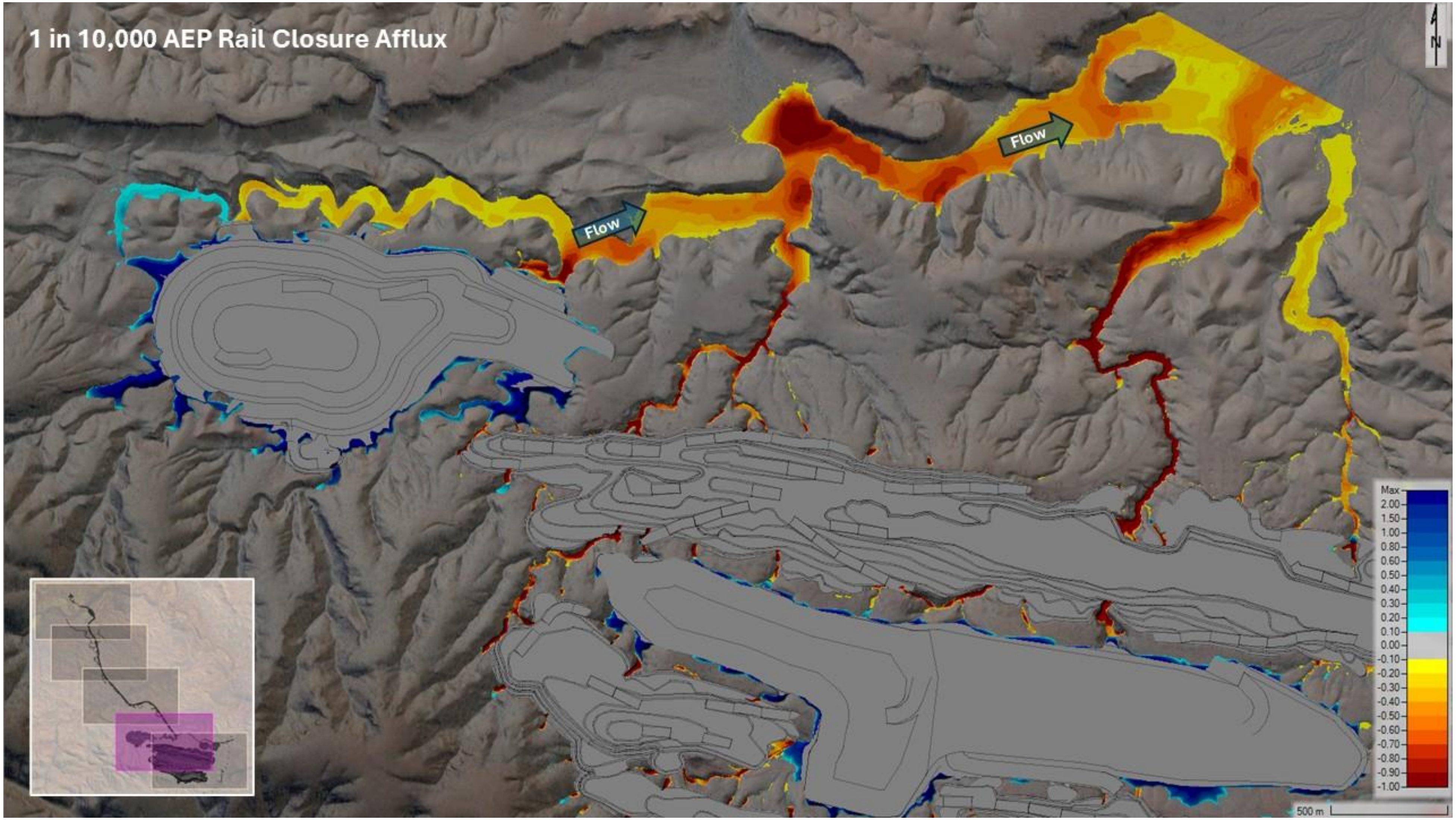
E-5 1 in 10,000 AEP Rail Closure Depth (Location 5)



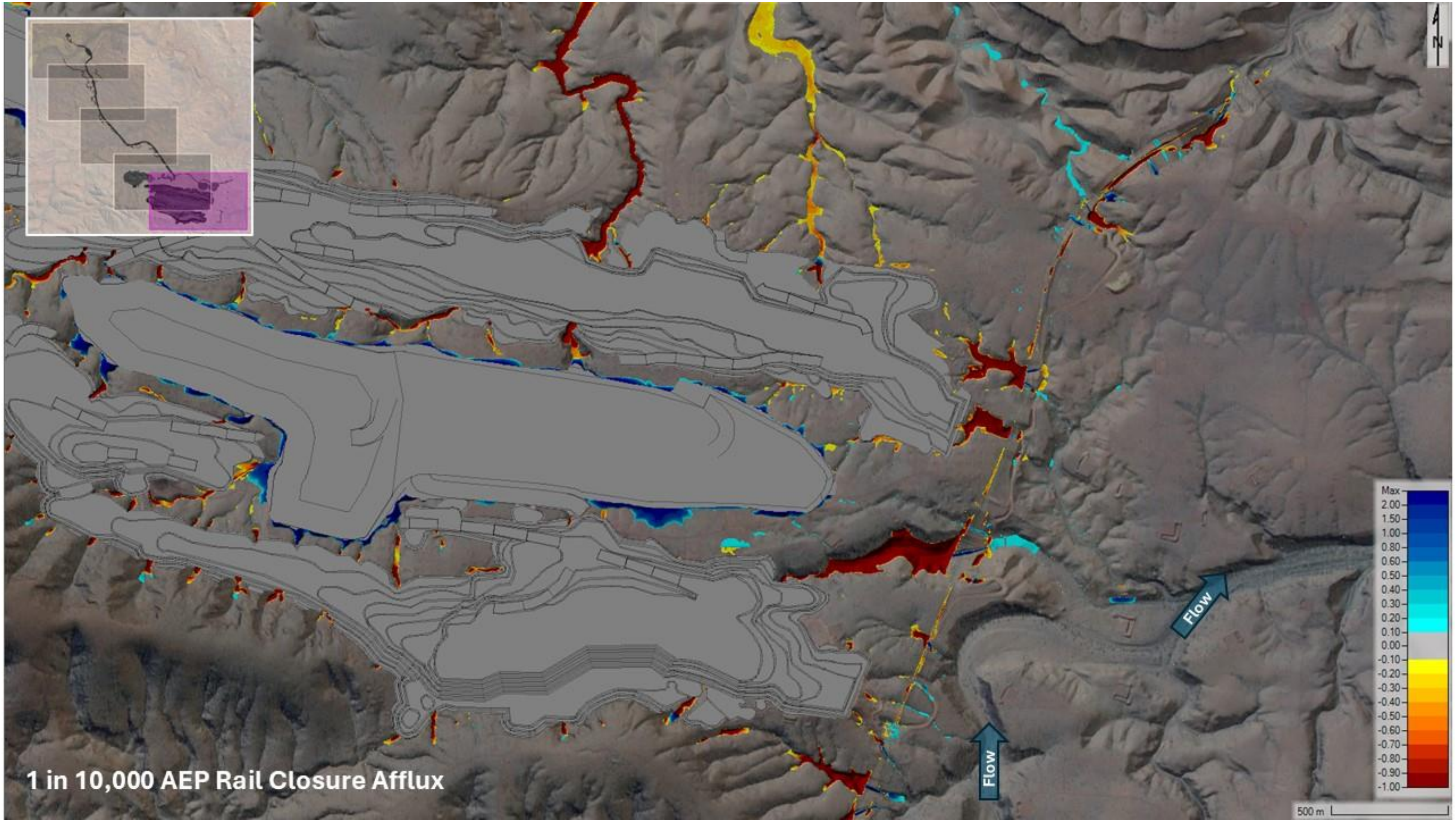
E-6 1 in 10,000 AEP Rail Closure Afflux (Location 1)

E-7 1 in 10,000 AEP Rail Closure Afflux (Location 2)

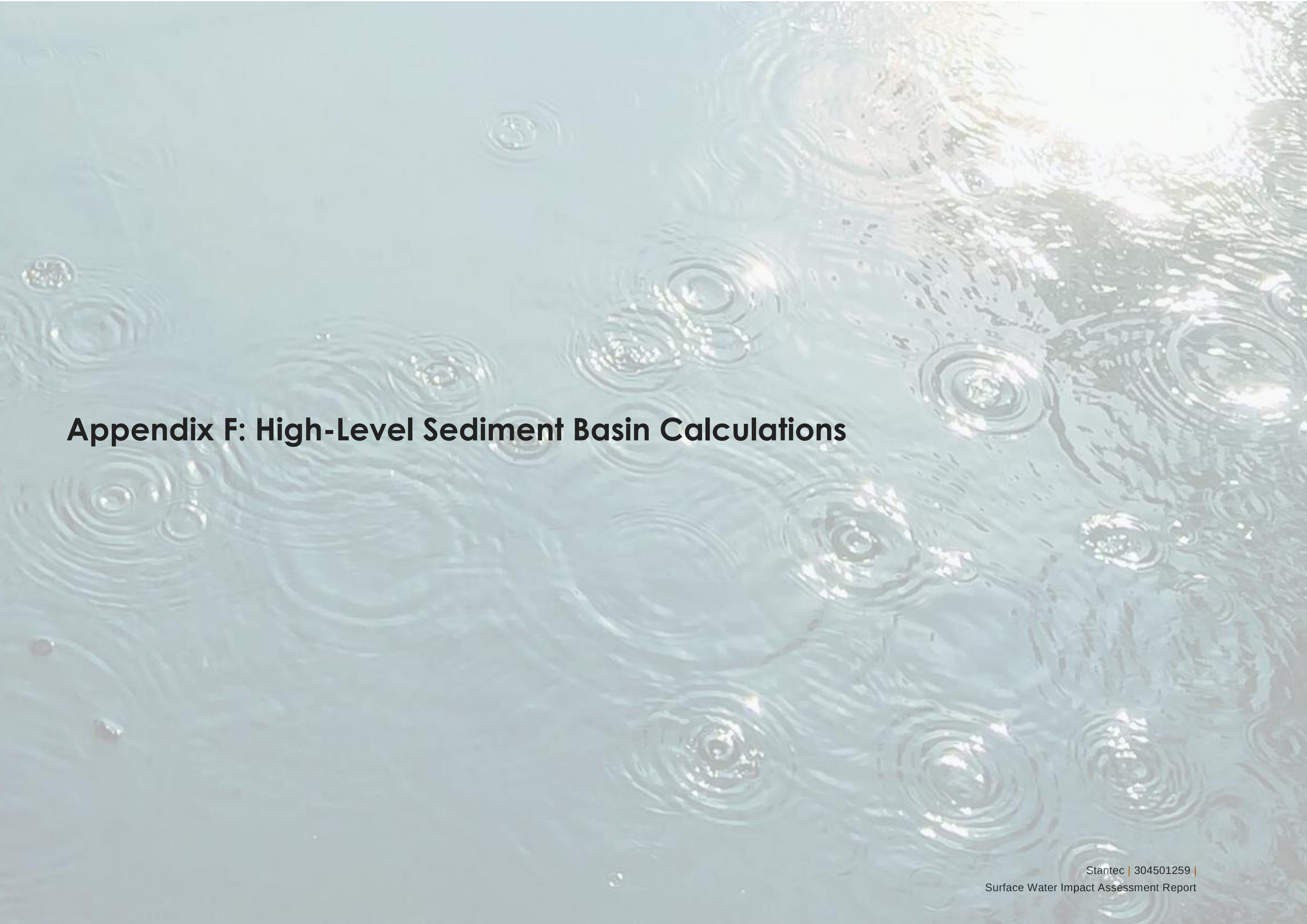
E-8 1 in 10,000 AEP Rail Closure Afflux (Location 3)



E-9 1 in 10,000 AEP Rail Closure Afflux (Location 4)



E-10 1 in 10,000 AEP Rail Closure Afflux (Location 5)

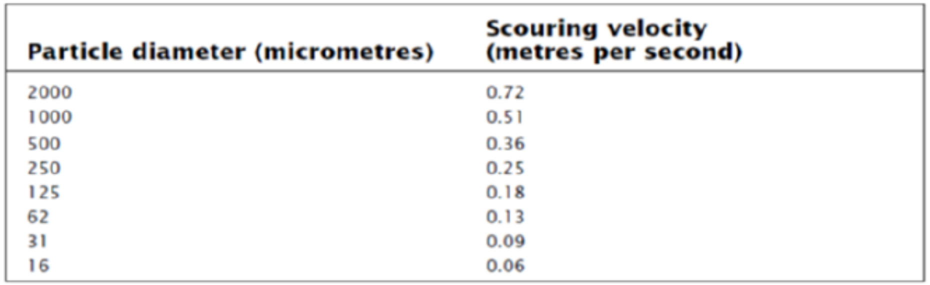


Appendix F: High-Level Sediment Basin Calculations

North & South Pit Sediment Basin	*As per Urban Stormwater: Best Practice Environment Management Guidelines (CSIRO 2009)	
Settling Velocity (mm/s)	11	For very fine sand, 125 micron
Settling Velocity (m/s)	0.011	
Peak Runoff Flow (m3/s)	0.1	*Pumped Discharge of 100 L/s
Based on theoretical Barnes Et Al		
Surface Area of Pond (m2)	9	Equation 1 $v_s = \frac{Q}{A}$
Depth of permanent storage (m)	0.50	*Assuming pond is not empty
Depth of extended storage (m)	0.35	
Permanent storage volume (m3)	5	Based on 1.5m depth
Extended storage volume (m3)	3	
Length of Pond (m)	6.0	Equation 2 (assumes length/width ration of 4:1) $L = \left(\frac{rQ}{v_s}\right)^{0.5}$
Width of Pond (m)	1.5	
Proportion of target particle removed	0.975	Equation 3 Ok Must be >0.7 $R = 1 - \left(1 + \frac{V_s[S_p + S_e]}{Qd_e}\right)^{-n}$
Check Scour Velocity (Depth=1.5m)	0.044	Must be less than 0.18m/s

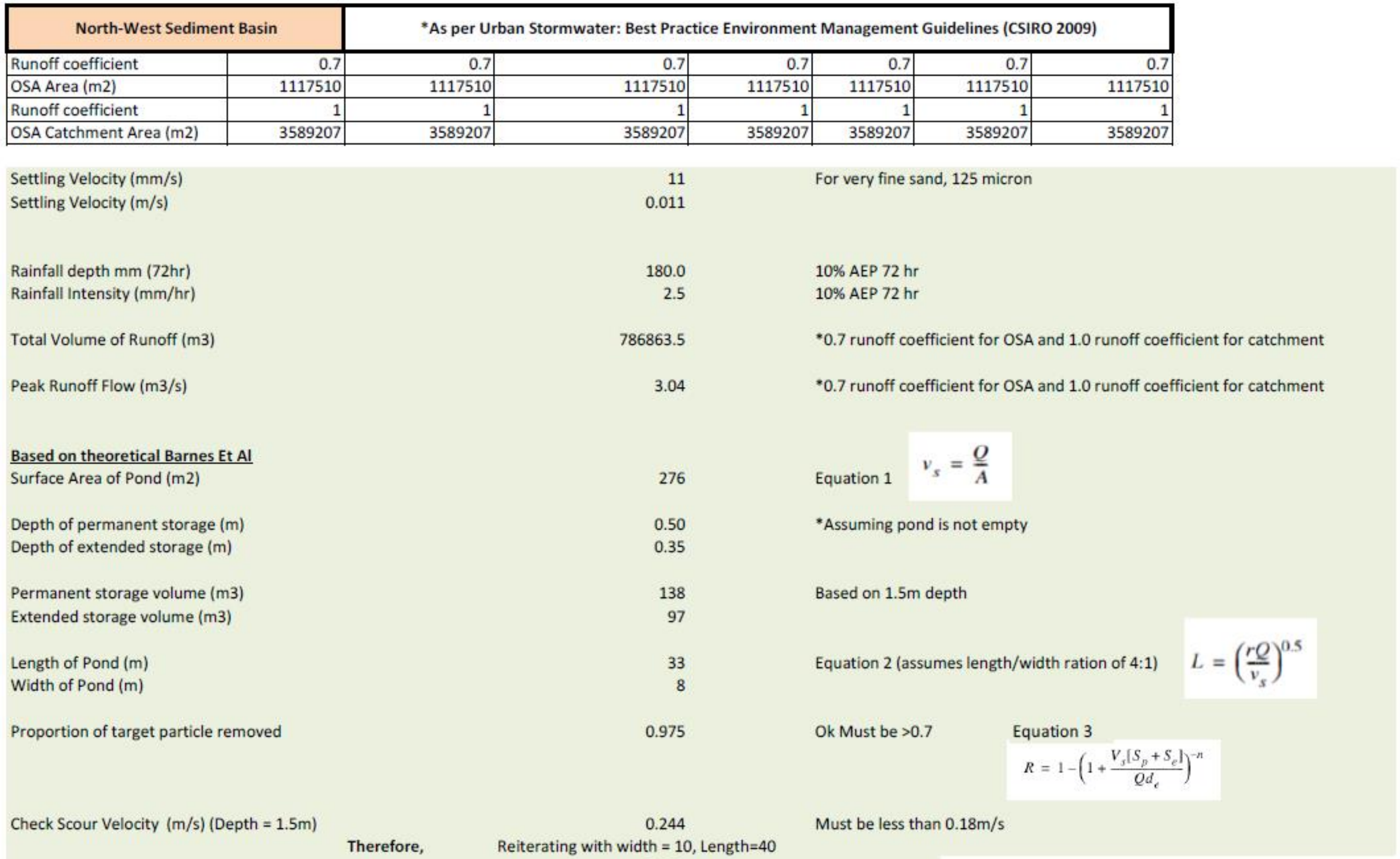
Assuming minimum width = 4m, Propose 16x4x1.5

Sediment Basin Surface Area (m2)	64	m2
Sediment Basin Volume (m3)	96	m3



Particle diameter (micrometres)	Scouring velocity (metres per second)
2000	0.72
1000	0.51
500	0.36
250	0.25
125	0.18
62	0.13
31	0.09
16	0.06

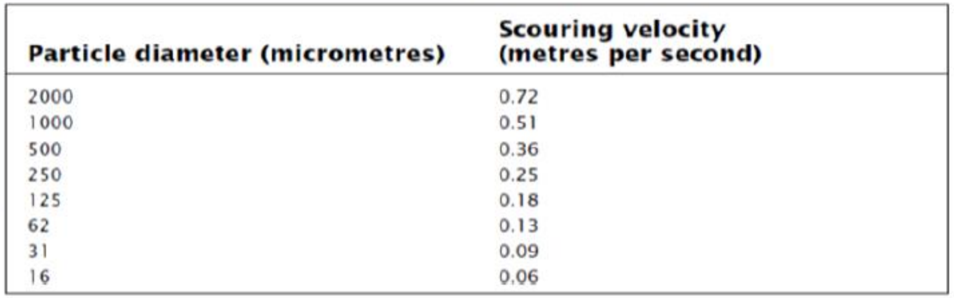
Table B2 Estimated scouring velocities (after Metcalf and Eddy 1991).



North-West Sediment Basin	*As per Urban Stormwater: Best Practice Environment Management Guidelines (CSIRO 2009)						
Runoff coefficient	0.7	0.7	0.7	0.7	0.7	0.7	0.7
OSA Area (m2)	1117510	1117510	1117510	1117510	1117510	1117510	1117510
Runoff coefficient	1	1	1	1	1	1	1
OSA Catchment Area (m2)	3589207	3589207	3589207	3589207	3589207	3589207	3589207

Settling Velocity (mm/s)		11	For very fine sand, 125 micron
Settling Velocity (m/s)		0.011	
Rainfall depth mm (72hr)		180.0	10% AEP 72 hr
Rainfall Intensity (mm/hr)		2.5	10% AEP 72 hr
Total Volume of Runoff (m3)		786863.5	*0.7 runoff coefficient for OSA and 1.0 runoff coefficient for catchment
Peak Runoff Flow (m3/s)		3.04	*0.7 runoff coefficient for OSA and 1.0 runoff coefficient for catchment
Based on theoretical Barnes Et Al			
Surface Area of Pond (m2)		276	Equation 1 $v_s = \frac{Q}{A}$
Depth of permanent storage (m)		0.50	*Assuming pond is not empty
Depth of extended storage (m)		0.35	
Permanent storage volume (m3)		138	Based on 1.5m depth
Extended storage volume (m3)		97	
Length of Pond (m)		33	Equation 2 (assumes length/width ration of 4:1) $L = \left(\frac{rQ}{v_s}\right)^{0.5}$
Width of Pond (m)		8	
Proportion of target particle removed		0.975	Ok Must be >0.7 Equation 3 $R = 1-\left(1+\frac{V_s[S_p+S_e]}{Qd_e}\right)^{-n}$
Check Scour Velocity (m/s) (Depth = 1.5m)		0.244	Must be less than 0.18m/s
	Therefore,	Reiterating with width = 10, Length=40	
		0.202	
		Reiterating with width = 12, Length=48	
		0.169 < 0.18	OK
	Therefore,		

Sediment Basin Surface Area	576	m2
Sediment Basin Volume	864	m3



Particle diameter (micrometres)	Scouring velocity (metres per second)
2000	0.72
1000	0.51
500	0.36
250	0.25
125	0.18
62	0.13
31	0.09
16	0.06

Table B2 Estimated scouring velocities (after Metcalf and Eddy 1991).

Haul Road Stockpile	***As per Urban Stormwater: Best Practice Environment Management Guidelines (CSIRO 2009)**						
Runoff coefficient	0.7	0.7	0.7	0.7	0.7	0.7	0.7
Stockpile Area(m2)	225423	225423	225423	225423	225423	225423	225423
Runoff coefficient	1	1	1	1	1	1	1
Stockpile Catchment Area (m2)	424666	424666	424666	424666	424666	424666	424666

Settling Velocity (mm/s)	11	For very fine sand, 125 micron	
Settling Velocity (m/s)	0.011		
Rainfall depth (mm)	180.0	10% AEP 72 hr	
Rainfall Intensity (mm/hr)	2.5	10% AEP 72 hr	
Total Volume of Runoff	104843.18	*0.7 runoff coefficient for OSA and 1.0 runoff coefficient for catchment	
Peak Runoff Flow (m3/s)	0.4	*0.7 runoff coefficient for OSA and 1.0 runoff coefficient for catchment	
Based on theoretical Barnes Et Al			
Surface Area of Pond (m2)	37	Equation 1	$v_s = \frac{Q}{A}$
Depth of permanent storage (m)	0.50	*Assuming pond is not empty	
Depth of extended storage (m)	0.35		
Permanent storage volume (m3)	18	Based on 1.5m depth	
Extended storage volume (m3)	13		
Length of Pond (m)	12	Equation 2 (assumes length/width ration of 4:1)	$L = \left(\frac{rQ}{v_s}\right)^{0.5}$
Width of Pond (m)	3	<4 min so assume W=4, L=16	
Proportion of target particle removed	0.975	**Ok Must be >0.7**	Equation 3 $R = 1 - \left(1 + \frac{V_s[S_p + S_e]}{Qd_e}\right)^{-n}$
Check Scour Velocity (m/s) (Depth = 1.5m)	0.089	Must be less than 0.18m/s	**OK**

Assuming minimum width = 4m, Propose 16x4x1.5

LV SME Stockpile		***As per Urban Stormwater: Best Practice Environment Management Guidelines (CSIRO 2009)**					
Runoff coefficient	0.7	0.7	0.7	0.7	0.7	0.7	0.7
Stockpile Area(m2)	125547	125547	125547	125547	125547	125547	125547
Runoff coefficient	1	1	1	1	1	1	1
Stockpile Catchment Area (m2)	51858	51858	51858	51858	51858	51858	51858

Settling Velocity (mm/s)	11	For very fine sand, 125 micron		
Settling Velocity (m/s)	0.011			
Rainfall depth (mm)	180.0	10% AEP 72 hr		
Rainfall Intensity (mm/hr)	2.5	10% AEP 72 hr		
Total Volume of Runoff (m3)	25153.362	*0.7 runoff coefficient for OSA and 1.0 runoff coefficient for catchment		
Peak Runoff Flow (m3/s)	0.097	*0.7 runoff coefficient for OSA and 1.0 runoff coefficient for catchment		
<u>Based on theoretical Barnes Et Al</u>				
Surface Area of Pond (m2)	9	Equation 1	$v_s = \frac{Q}{A}$	
Depth of permanent storage (m)	0.50	*Assuming pond is not empty		
Depth of extended storage (m)	0.35			
Permanent storage volume (m3)	4	Based on 1.5m depth		
Extended storage volume (m3)	3			
Length of Pond (m)	6	Equation 2 (assumes length/width ration of 4:1)	$L = \left(\frac{rQ}{v_s}\right)^{0.5}$	
Width of Pond (m)	1.5	<4 min so assume W=4, L=16		
Proportion of target particle removed	0.975	Equation 3	$R = 1 - \left(1 + \frac{V_s[S_p + S_e]}{Qd_e}\right)^{-n}$	Ok Must be >0.7
Check Scour Velocity (m/s) (Depth = 1.5m)	0.044	Must be less than 0.18m/s	OK	

Assuming minimum width = 4m, Propose 16x4x1.5

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