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Memo

To	Fabian Goddard	Company	CZR Resources
From	Tamar Haviv	Job No.	385M
Date	29 November 2024	Doc No.	039b
Subject	CZR Robe Mesa – Robe Mesa MOC and Mungarathoona Creek Surface Water Modelling		

Fabian,

Please find below our technical memo report outlining the predicted impacts to the hydrological characteristics of Mungarathoona Creek due to the proposed haul road required for CZR Resources' Robe Mesa project.

1. INTRODUCTION

CZR Resources' Robe Mesa Iron Ore Project is located approximately 140km southwest of Karratha and 32km west of the Rio Tinto township of Pannawonica. The proposed mine site lies adjacent to, and on the south side of the Robe River, between the Rio Tinto-owned Mesa A and Mesa J iron ore mines (and immediately north of Mesa F). Processed iron ore from Robe Mesa is expected to be transported via a new 11 km haul road connecting the mine site and the North West Coastal Highway (NWCH). The road alignment runs from the Mine Operations Centre (MOC) southwards, on the east side of Mesa F, crossing Mungarathoona Creek, before running west to NWCH. The proposed haul road crosses several ephemeral creek systems which can experience significant flow during heavy rainfall.

A layout plan of the proposed haul road, MOC development (northern orange area) and camp (southern orange area) is provided in Figure 1-1.

The Robe River is a significant river system in the region and drains east to west through the high relief areas of the Hamersley Ranges, then between Mesa formations on the Southern Peneplain, and onto the gently sloping coastal plain prior to discharging into the ocean (Ruprecht, 2000 and Beard, 1975). The river passes across the NWCH bridge / causeway and Yarraloola gauging station. The river has a minor ocean outlet with discharge to a marsh flat on the coastal plain (Ruprecht, 2000). At the junction of Mungarathoona Creek and the Robe River, but within the Mungarathoona Creek channel, a seasonal pool forms following runoff events (as referred to by DEMIRS).

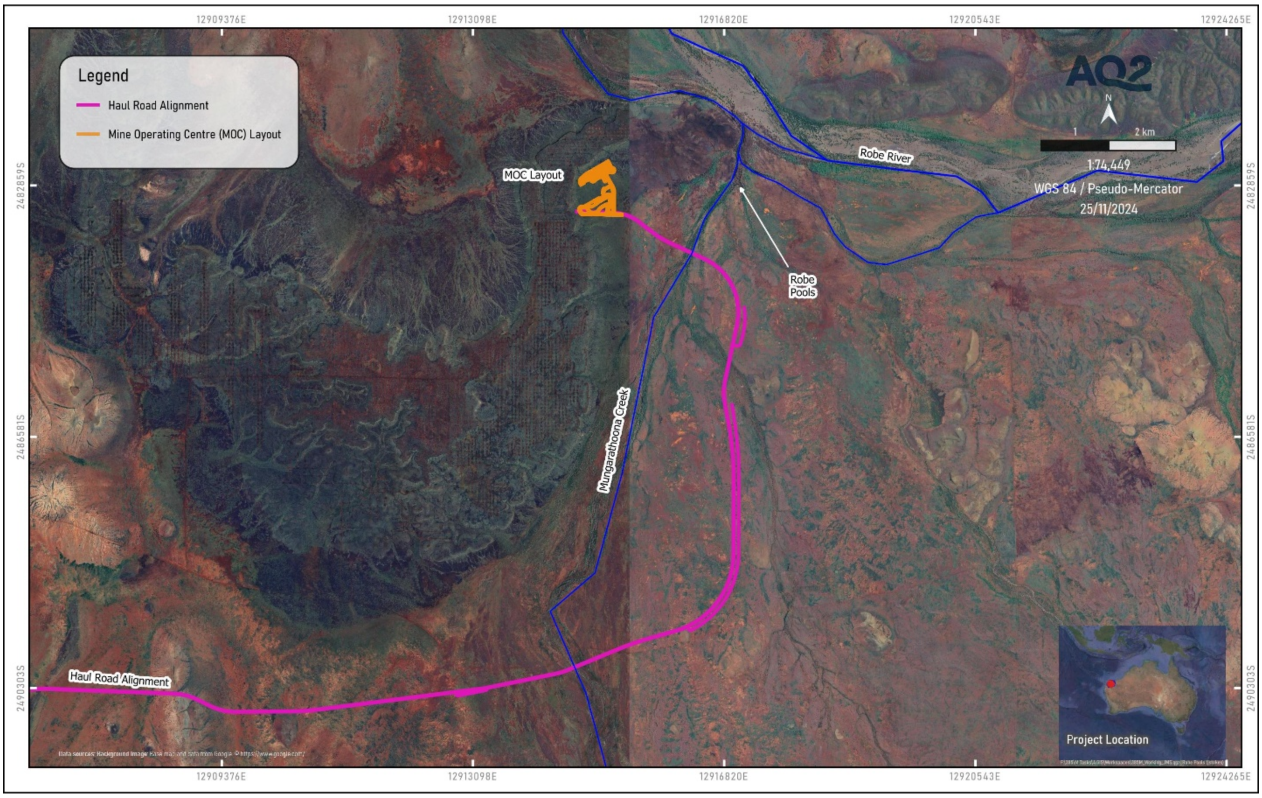


Figure 1-1 Project Layout

AQ2 has previously provided surface water associated with the project in 2021 and 2022. The haul road alignment has altered since previous 2D flood modelling assessments were completed and the MOC layout has been revised. CZR received comments from the Department of Energy, Mines, Industry Regulation and Safety (DEMIRS). The comments are as follows:

1. *"It is noted that the location of infrastructure and haul road were different during the modelling (AQ2, 2021) than the current proposed locations. Consequently, modelled creek crossings are further south than the current proposed crossing, which occurs after the confluence of Mungarathoona / Red Hill creek with an unnamed creek. Flood depths and volume of water traversing the area will be higher than what was modelled. Based on Figure 5 (AQ2, 2021) flood depths will be in the range of 4-8m rather than 3-4m.*

2. *It is recommended that the surface water modelling be repeated with the current proposed footprint and envelope. This will better inform management options for the creek crossings and thus what impacts there may be to downstream hydrology and its effects on vegetation.*

3. *Additionally, the southern crossing was modelled for peak volume, but modelled flood depth or velocity (AQ2, 2022). This crossing should be modelled when the surface water studies are repeated and details on culverts and / or crossing management should be provided. Modelled flow rates and capacity will inform if the installation of culverts at either crossing will maintain surface hydrology downstream to Robe Pool."*

Responses to these comments are required as part of the Mining Proposal and Native Vegetation Clearing Permit.

An assessment has been completed and documented within the following report to address the comments. The assessment has included a detailed flood model that reflects the updated haul road alignment and culvert configurations which is used to compare pre and post-development hydrological conditions. The proposed layout of the MOC has been compared with predicted flood extents from Mungarathoona Creek.

2. FLOOD MODELLING

The setup and results from a 2D flood model are discussed further in the document, but the 2D flood model has been used to:

- Predict flood conditions in a 1% AEP flood event to define the flood risk to the MOC.
- Simulate predicted hydrological conditions before and after the construction of the proposed haul road from a nominal runoff event which would occur relatively frequently (in this case, the 10% AEP event).

2.1 Catchment Definition

The Mungarathoona Creek sub-catchments to the proposed haul road have been defined in previous reports and are outlined in Figure 2-1.

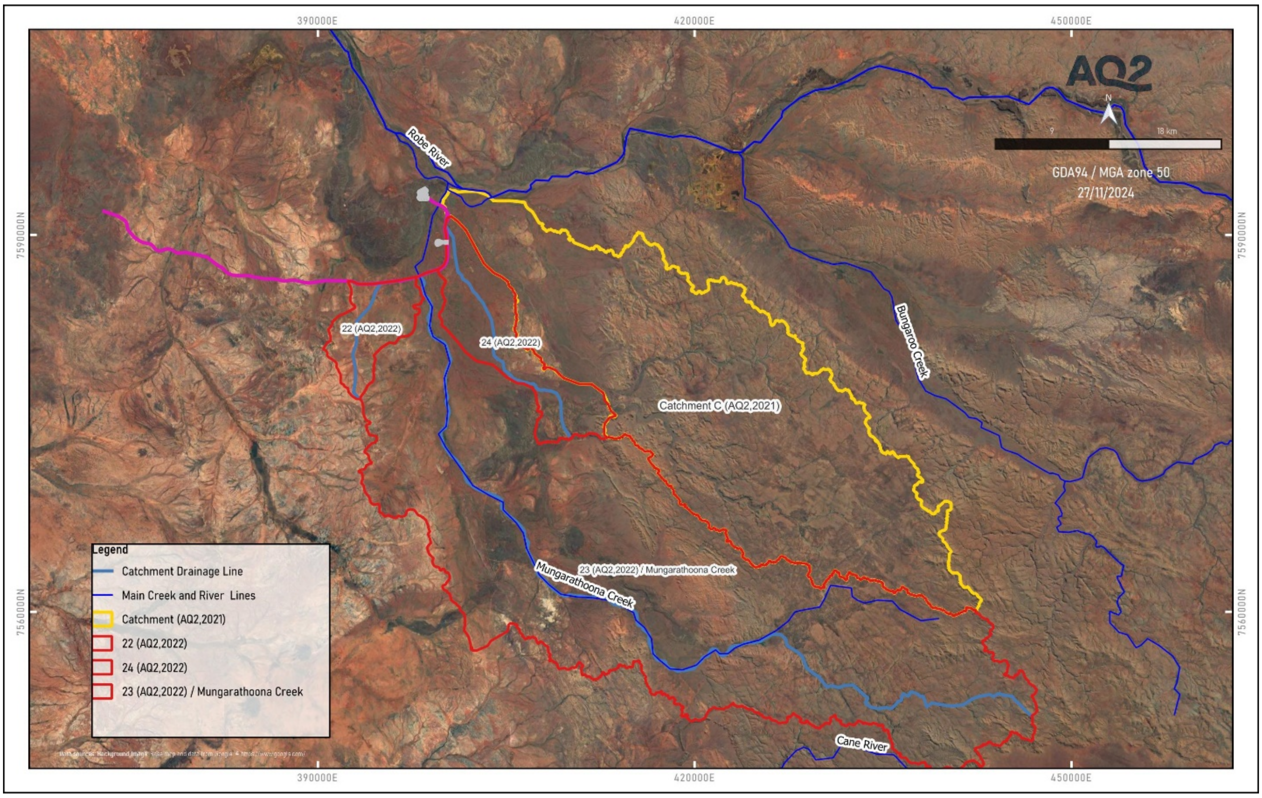


Figure 2-1 Mungarathoona Creek Sub-Catchments Flood Model Setup

A 2D flood model was developed using HEC-RAS V6.6 modelling software. The 2D model was set up to cover the extent the haul road within the Mungarathoona Creek catchment and the MOC. Upstream of the haul road, the model covers the area where LiDAR elevation data was provided. The model extends along the Robe River downstream of the Robe Mesa project area a sufficient distance to define flood levels within the Robe River at the junction with Mungarathoona Creek, without any model backwater effects impacting flood level estimates in the project area. SRTM elevation data was required to allow the model to be extended downstream.

The general model build details are as follows:

- 50 x 50m grid with orientation refined by "breakline" alignments along the river and tributary channels.
- Model inflows at the upstream model boundary (as documented in the following sub-sections).
  - Note that due to the nature of the drainage at the upstream model boundary, where multiple flow paths tend to occur, estimated inflows were spread along a wide inflow boundary and allowed to distribute across the channels.
  - The model only simulates runoff from within the drainage lines passing through the model extent. Local runoff, such as runoff from the mesa behind the MOC, is not simulated.
- A Roughness manning's 'n' value varied according to flood depth (0.05 for depth>1m, 0.08 for depths 0.2 to 1m, and 0.12 to simulate sheet flow less than 0.2m deep).
- Outflow along the Downstream Boundary using normal depth calculations and a downstream slope with an average hydraulic grade line (0.002, assumed to be equal to the average slope of the creek bed).
- Variable timestep calculated internally in the model using a maximum Courant Number of 2.

To create the 2D HEC-RAS model, it was necessary to integrate two separate terrain datasets to cover the full model extent. The first dataset consisted of high-resolution LiDAR data, while the second was derived from coarser SRTM data. At the interface of these terrain sets, a discrepancy in elevation was observed and the SRTM data was lowered to join the terrain. The SRTM was not detailed enough to define the creek lines, such that the terrain had to be modified to include a nominal channel to allow flow to continue across the SRTM patch of data. This has been indicated on the flood model result figures in Section 3 and the resulting flood characteristics within this area are not considered to be valid.

A layout of the 2D flood model is shown in Figure 2-2. Note that the model does not simulate runoff from the flanks of the mesa to the north and west of the MOC area and only assesses the flood risk due to the Mungarathoona Creek.

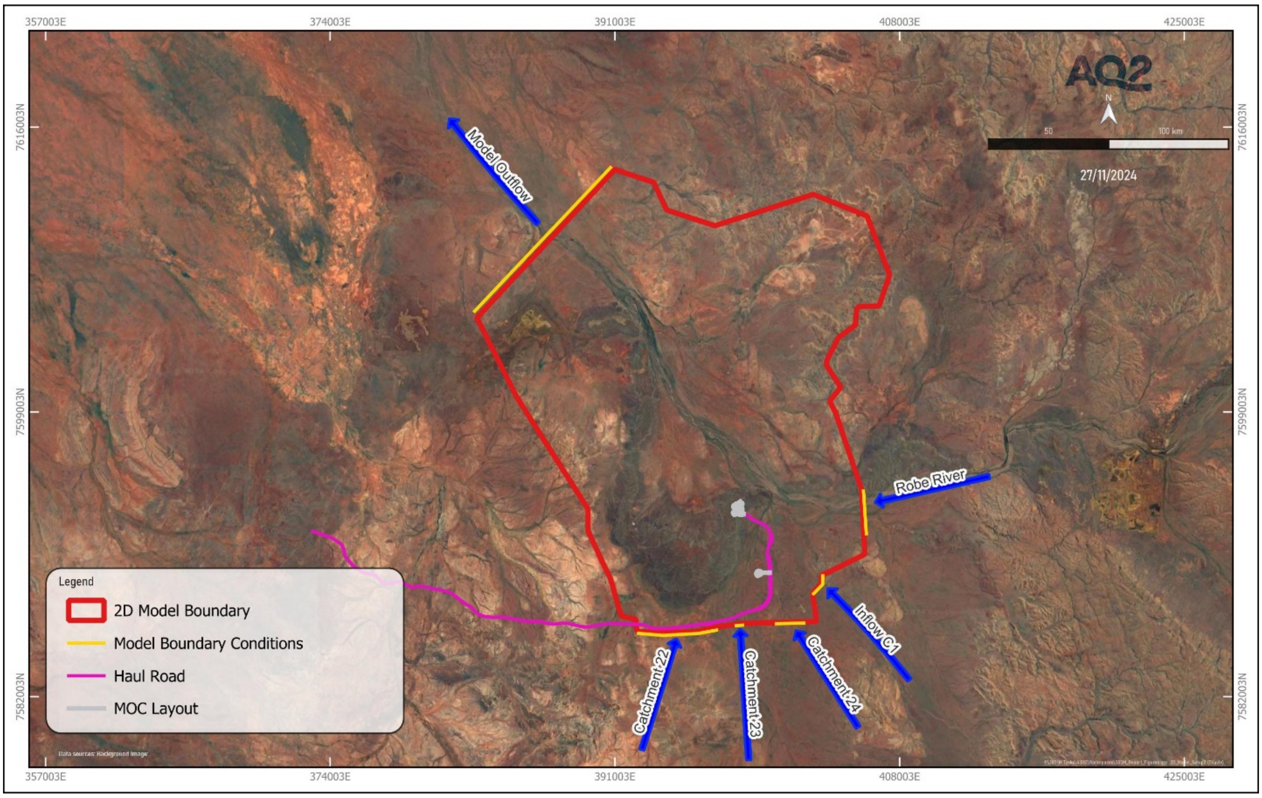
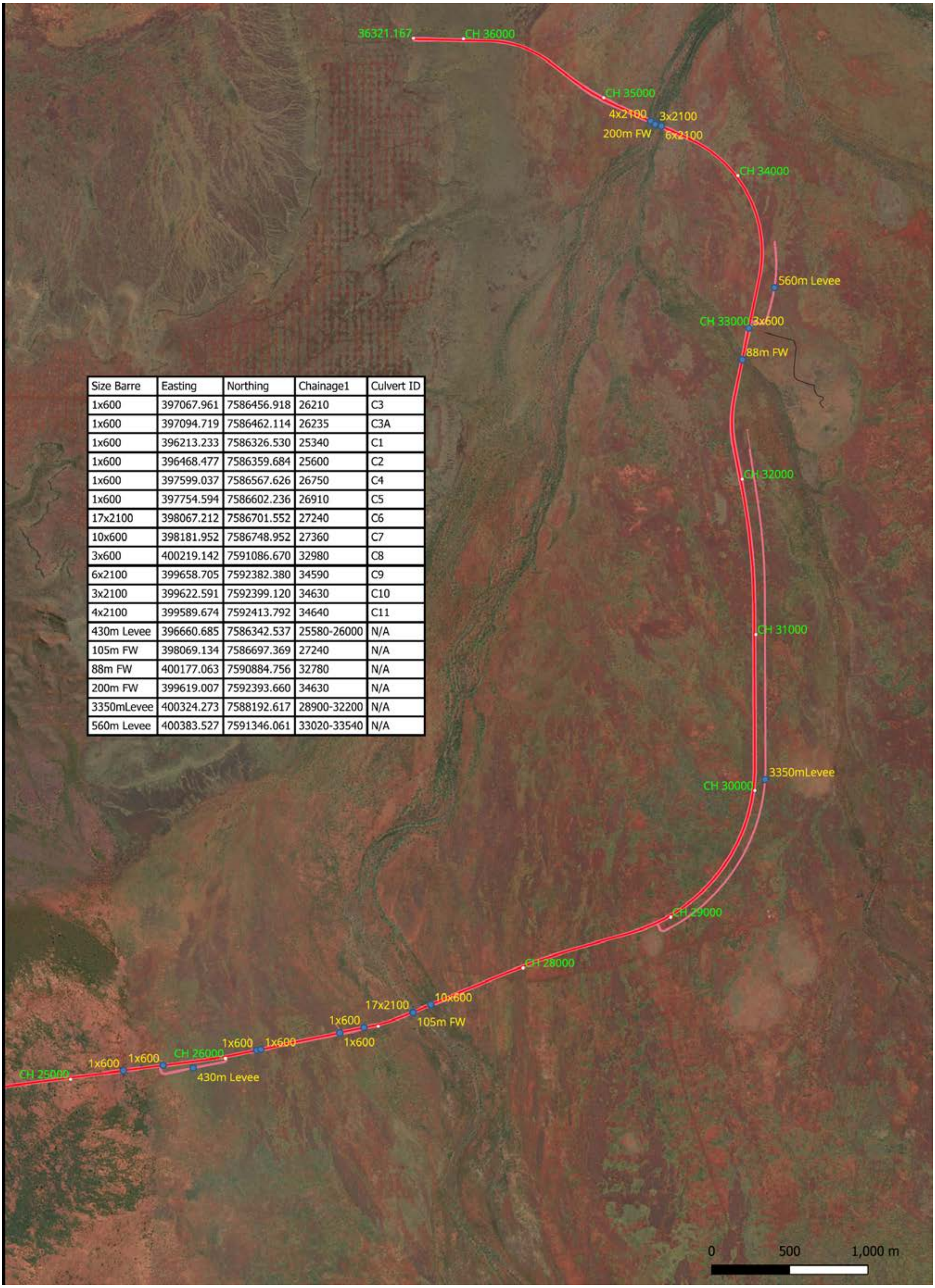


Figure 2-2 2D Model Layout

To develop the post-development flood model, the terrain was altered to include the proposed embankment heights, levees and culvert sizes for the haul road. The road embankment civil design surface and culvert sizes and located were provided by Shawmac. These were designed from preliminary flood assessment work completed by AQ2.

The proposed haul road is typically 20m wide (embankment crest width) and 11 banks of culverts are proposed. The details of the proposed banks of culverts are shown in Figure 2-3 (as provided by Shawmac).

Breaklines within the model were aligned with the haul road to allow refined model cells covering the haul road and the terrain immediately to each side of the haul road.



Size Barre	Easting	Northing	Chainage1	Culvert ID
1x600	397067.961	7586456.918	26210	C3
1x600	397094.719	7586462.114	26235	C3A
1x600	396213.233	7586326.530	25340	C1
1x600	396468.477	7586359.684	25600	C2
1x600	397599.037	7586567.626	26750	C4
1x600	397754.594	7586602.236	26910	C5
17x2100	398067.212	7586701.552	27240	C6
10x600	398181.952	7586748.952	27360	C7
3x600	400219.142	7591086.670	32980	C8
6x2100	399658.705	7592382.380	34590	C9
3x2100	399622.591	7592399.120	34630	C10
4x2100	399589.674	7592413.792	34640	C11
430m Levee	396660.685	7586342.537	25580-26000	N/A
105m FW	398069.134	7586697.369	27240	N/A
88m FW	400177.063	7590884.756	32780	N/A
200m FW	399619.007	7592393.660	34630	N/A
3350mLevee	400324.273	7588192.617	28900-32200	N/A
560m Levee	400383.527	7591346.061	33020-33540	N/A

Figure 2-3 Haul Road Design Details (Supplied by Shawmac)

2.2 Robe River Peak Flow Assessment

The main Robe River channel lies to the north of the MOC. Potential flooding of the MOC and haul road may occur due to flooding of the Robe River backwatering up Mungarathoona Creek, and water levels within the Robe River have the potential to impact flood levels in Mungarathoona Creek within the vicinity of the haul road and MOC.

To estimate Robe River flow rates for various AEP flow events, a Flood Frequency Analysis was undertaken on the Yarraloola gauging station flow data (1989-2021), which is located ~20km downstream of the project area. The assessment which was completed is outlined in AQ2 2021 and the results (which have been used for this study) are summarised in Table 2-1. These Robe River flow rates were used within the 2D flood model, with a constant flow rate adopted as the inflow boundary condition for the Robe River.

Table 2-1 Robe River Design Peak Flow Estimates (m^3/s)

Catchment	10% AEPF	1% AEP
Robe River- to Yarraloola	3,000	17,100

2.3 Mungarathoona 1% AEP Catchment Runoff Estimates

Design flow rates for different AEP events have been estimated for each of the Mungarathoona Creek sub-catchments which were shown in Figure 2-1. These sub-catchments were defined in previous AQ2 studies and are summarised as follows:

- Catchment C has been divided into two sub-catchments, C1 and C2. Sub-catchment C1 contributes flows as a tributary to the Mungarathoona Creek catchment, while sub-catchment C2 directs its flows into the Robe River system (refer AQ2 2021).
- Catchment 24, a tributary of Mungarathoona Creek, flows northeast of the main catchment and merges with the primary Mungarathoona Creek close to the MOC (refer AQ2 2022).
- Catchment 23 covers the main Mungarathoona Creek channel, a major tributary of the Robe River, flows through the Project Tenement area from south to north and around the northern part of the Mesa. (AQ2,2022).
- Catchment 22, a tributary of Mungarathoona Creek, flows west of the main catchment and converges with the primary Mungarathoona Creek downstream of the haul road crossing. (AQ2,2022).

The peak flows for each of these catchments were estimated (AQ2 2021) using the Pilbara Regional Flood Frequency Procedure (RFFP2000) (Flavell 2012) and are shown in Table 2-2. These flow rates were applied to the inflow boundaries of the 2D flood model as constant flow rates. The constant inflow rate approach was adopted as a conservative approach to estimating the flood levels within the 2D flood model for the 1% AEP event (for the purpose of defining flood risk at the MOC) as it assumes that the peak flows in each of these catchments occurs concurrently.

Table 2-2 Upstream Inflows to the Mungarathoona Creek Catchments 1% AEP (m^3/s)

Catchment C1 Inflows	Catchment 24 Inflows	Catchment 23 Mungarathoona Creek Inflows	Catchment 22 Inflows
1,170	216	1,370	140

2.4 Mungarathoona 10% AEP Flow Hydrograph

To assess the potential environmental impact of the proposed haul road on the hydrological environment, a smaller magnitude, more frequent event has been selected as it will predict hydrological characteristics for events more appropriate for ecosystem (vegetation) function. As such, a 10% AEP event was nominally selected for the impact assessment. Further, the use of constant inflow rates to the 2D model boundary is likely to produce invalid results when assessing the potential impact of the haul road on the hydrological conditions as the effect of flow attenuation upstream of the haul road will not be simulated when assessing peak flow rate, flow depth and flow velocity criteria.

A RORB hydrology model was created for the main Mungarathoona Creek catchment (Catchment 23) to produce a nominal hydrograph for a circa 10% AEP event.

The RORB model adopted the following input parameters for the 10% AEP runoff event:

- Rainfall initial loss: 62 mm.
- Rainfall continuing loss: 6.9mm/hr.
- Kc routing parameter: 32 (using ARR regional formula for Kc).

As per ARR guidelines, the RORB model passed an ensemble of design storms for different storm durations over the modelled catchment. The median flows from the ensemble of storms were estimated for each storm duration. The storm duration with the highest median flow was selected as the critical duration event, and a flood hydrograph which approximated the median flow rate from the critical duration was extracted for use in the 2D model. The results from the RORB modelling are presented in Table 2-3.

Table 2-3 10% AEP – Mungarathoona Creek (Catchment 23) RORB Flow Estimates

Catchment Characteristics	RORB Peak Flows (m3 /s)
Area (km2)	10% AEP
789	270

It is noted that the peak flow and shape of the hydrograph adopted is not critical to the assessment, as long as the same flow hydrograph is used for the pre-development and post-development flood models to allow impacts to be quantified.

To prepare flow hydrographs for catchments C1, 24 and 22 for application to the 2D model boundary, nominal 10% AEP flow hydrographs were produced by scaling the Catchment 23 hydrograph by the peak flow proportions of the catchments derived using the Regional Flood Frequency Procedure (refer AQ2 2022).

3. FLOOD MODEL RESULTS AND DISCUSSION

3.1 1% AEP Flood Risk

The 1% AEP event flood depth predictions from the pre-development model relative to the proposed MOC layout are shown in Figure 3-1 & Figure 3-2. The predictions indicate that during a 1% AEP flood event, the majority of the MOC area has been positioned outside of the flood extents of Mungarathoona Creek. However, a portion of the southeast section of the MOC may be prone to inundation during large flood events. Depending on the nature of the infrastructure proposed in this area, flood protection measures may be required.

Note that local runoff from the mesa to the north and west of the MOC has not been simulated in the model and small scale diversions within the MOC area may be required to manage this runoff.

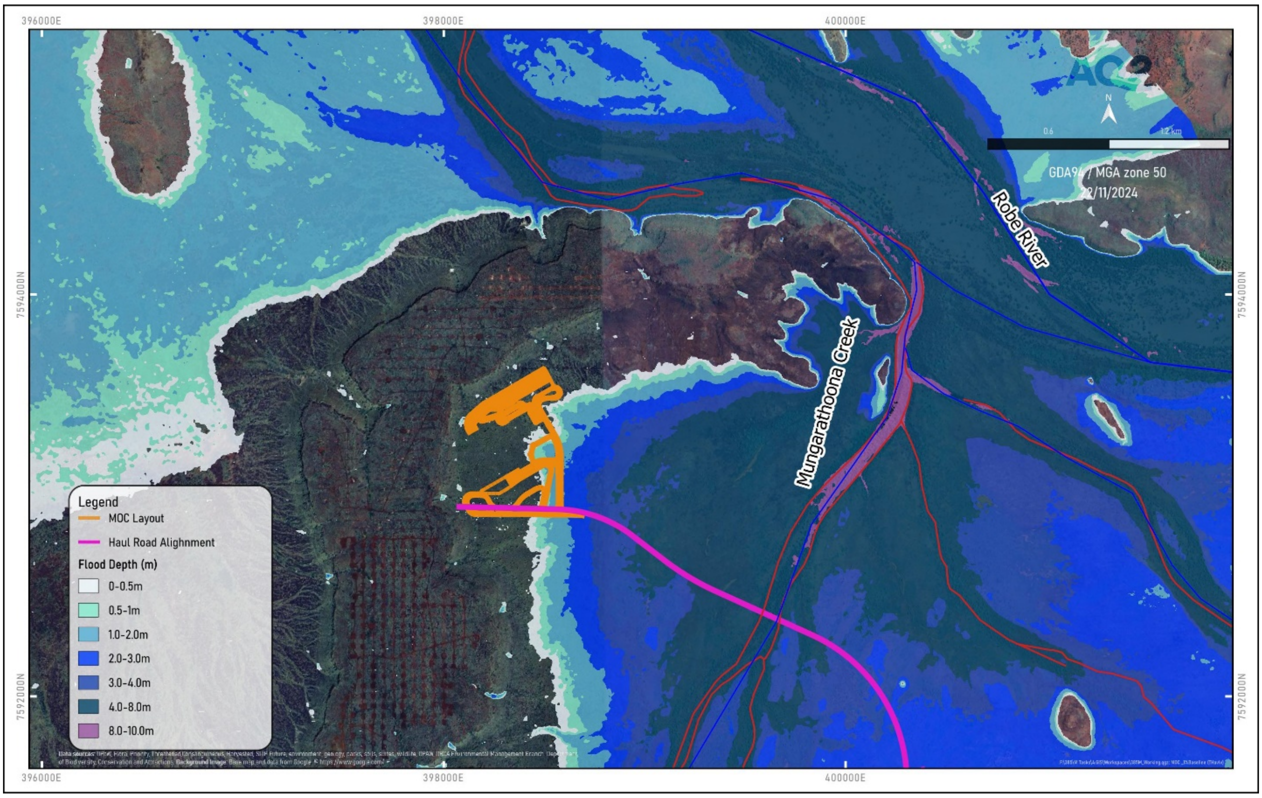


Figure 3-1 1% AEP Pre-Development Flood Map at MOC

The flooding at the proposed camp location has been reassessed with the updated 1% AEP runoff model, with results shown in Figure 3-2. The model shows that the camp footprint is generally located in an area that is predicted to be impacted by shallow sheet flow runoff (<0.5m deep) with some deeper inundation potential around the western and norther side of the camp. The runoff could be managed by creating a defined drainage path through the camp area, by raising the camp buildings above the flood level or be some nominal bunding to deflect the upstream runoff around the eastern and western sides of the camp.

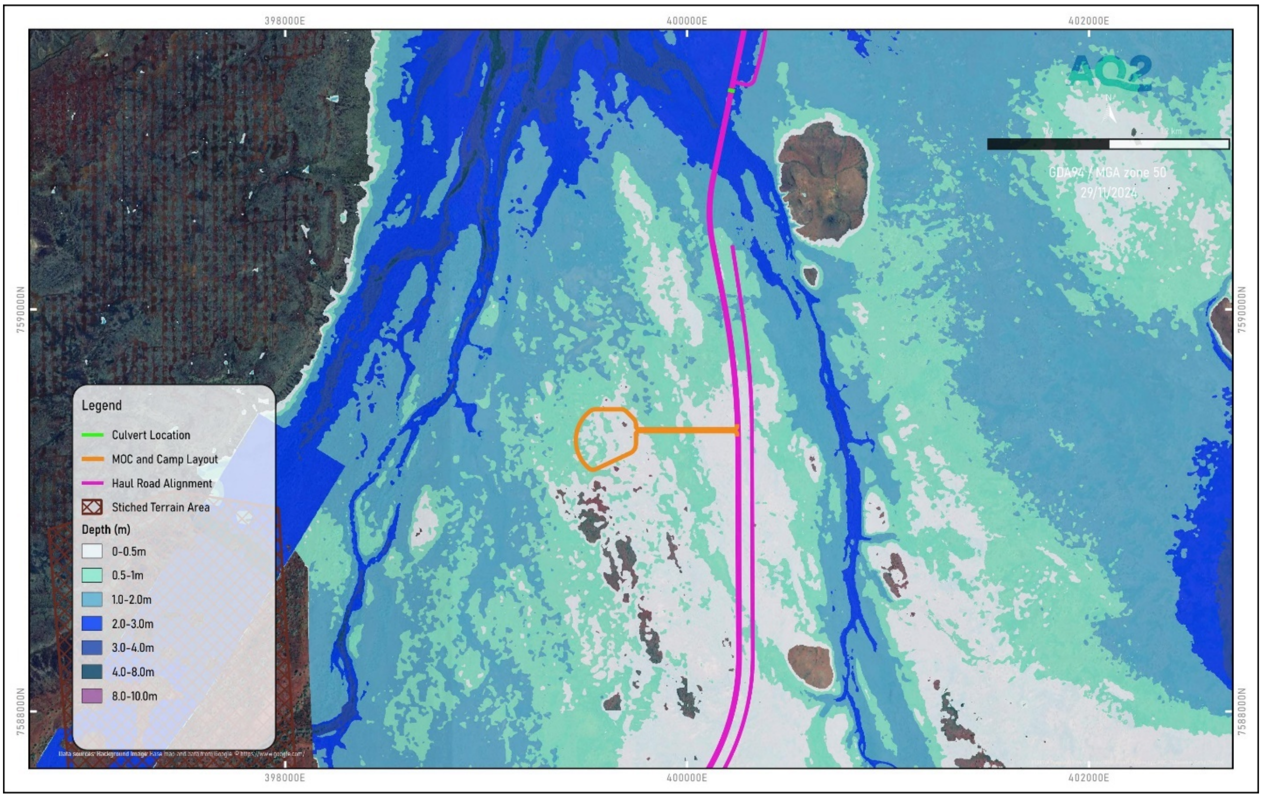


Figure 3-2 1% AEP Pre-Development Flood Map at Camp

3.2 Hydrological Impact of Mungarathoona Creek

The 2D flood model was run for the pre-development and post-development scenarios using nominal inflow hydrographs that are in the order of a 10% AEP event (as discussed in the preceding sections). The result predictions from the model scenarios are presented in the following figures:

- Pre-development maximum flood depth maps – Figure 3-3
- Pre-development maximum flood velocity maps – Figure 3-4
- Post-development maximum flood depth maps – Figure 3-5
- Post-development maximum flood velocity maps – Figure 3-6
- Difference flood depth maps (post-development max depth minus pre-development max depth) – Figure 3-7
- Difference flood velocity maps (post-development max velocity minus pre-development max velocity) – Figure 3-8
- Pre and post development flood hydrograph comparison on Mungarathoona Creek downstream of haul road – Figure 3-9
- Pre and post development cumulative flow volume comparison on Mungarathoona Creek downstream of haul road - Figure 3-10
- Examples of trapped/ponding water against haul road - Figure 3-11 and Figure 3-12

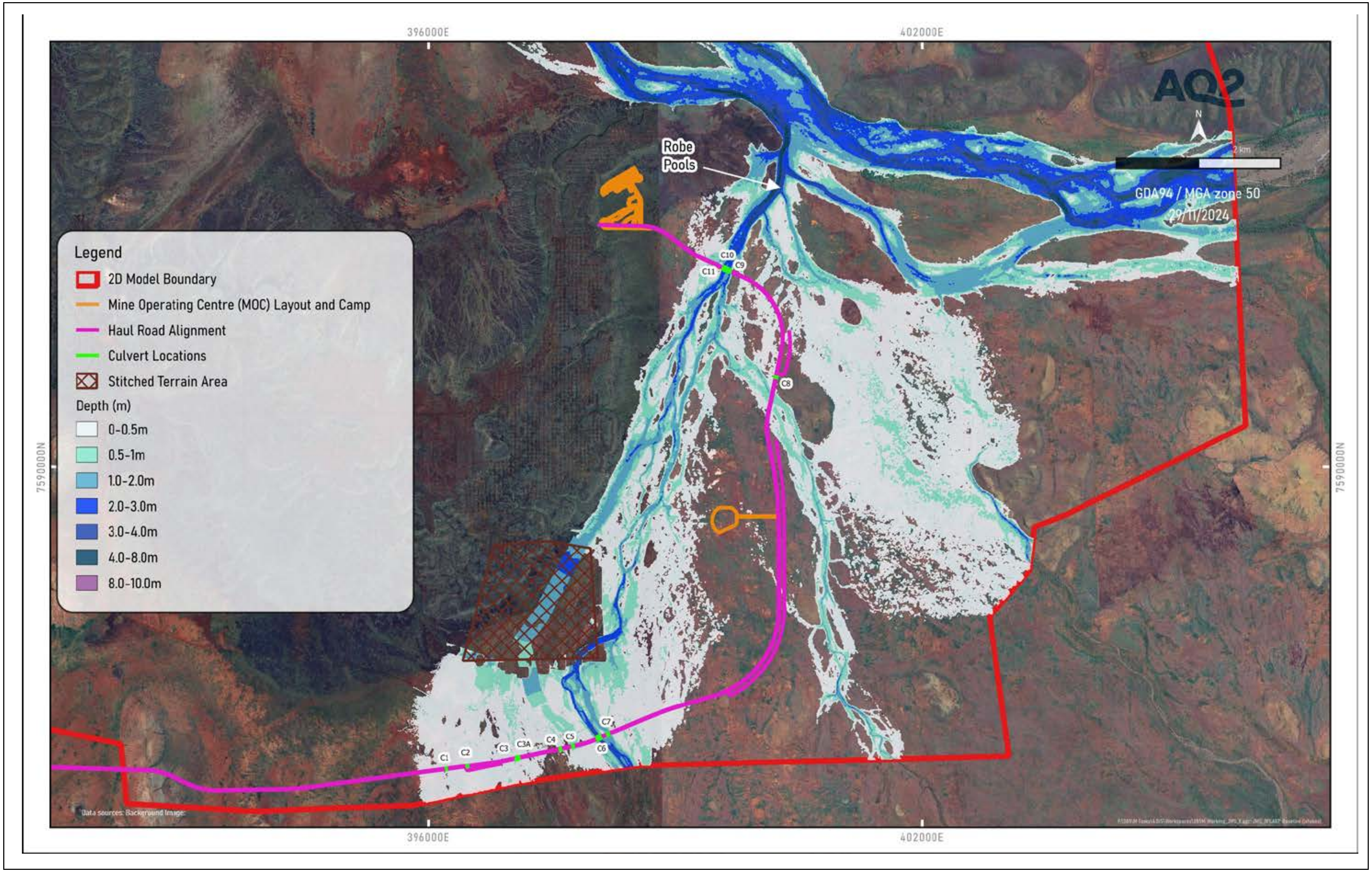


Figure 3-3 10%AEP Pre-Development Flood Depth

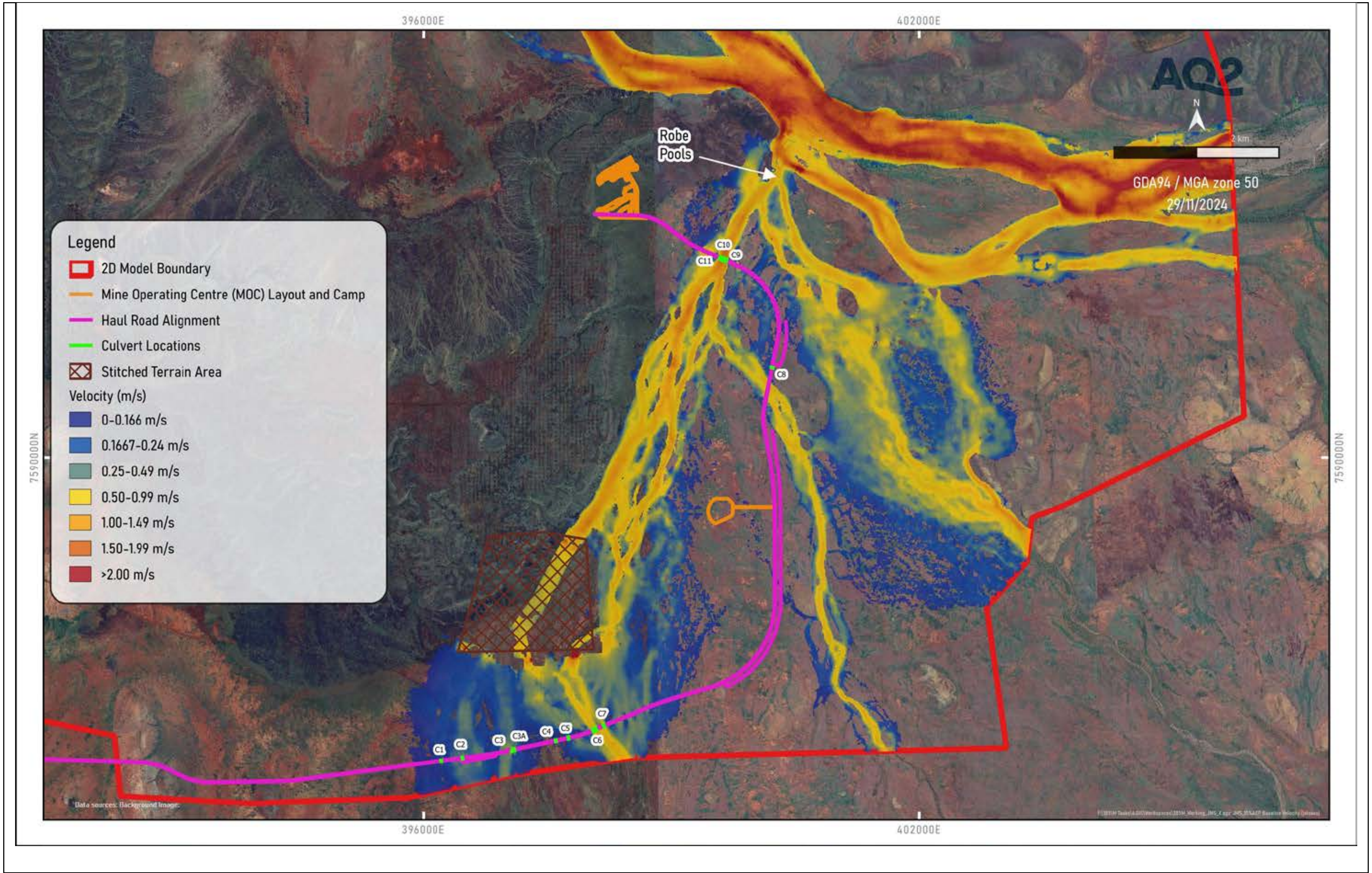


Figure 3-4 10% AEP Pre-Development Flood Velocity

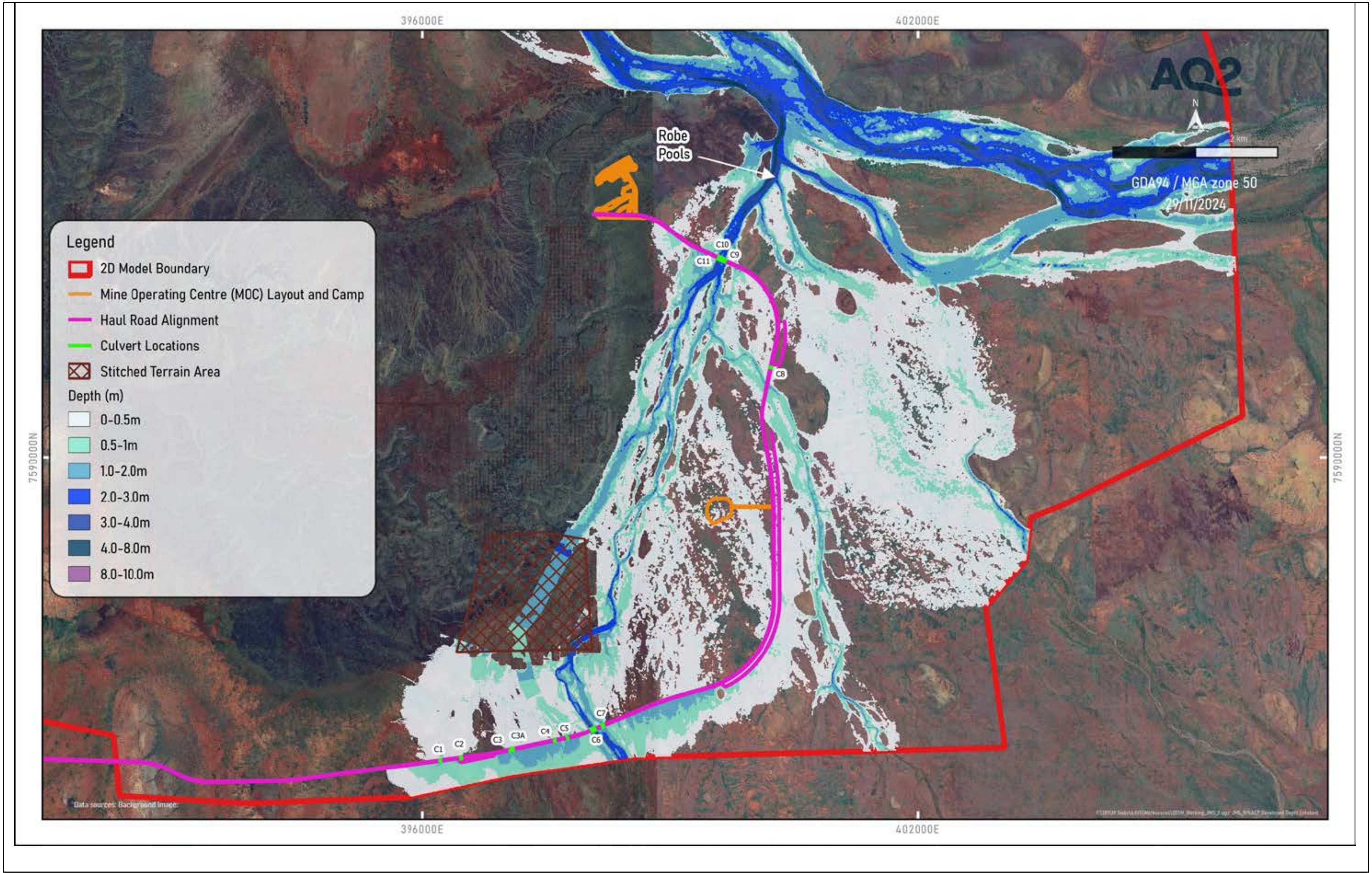


Figure 3-5 10% AEP Post-Development Flood Depth

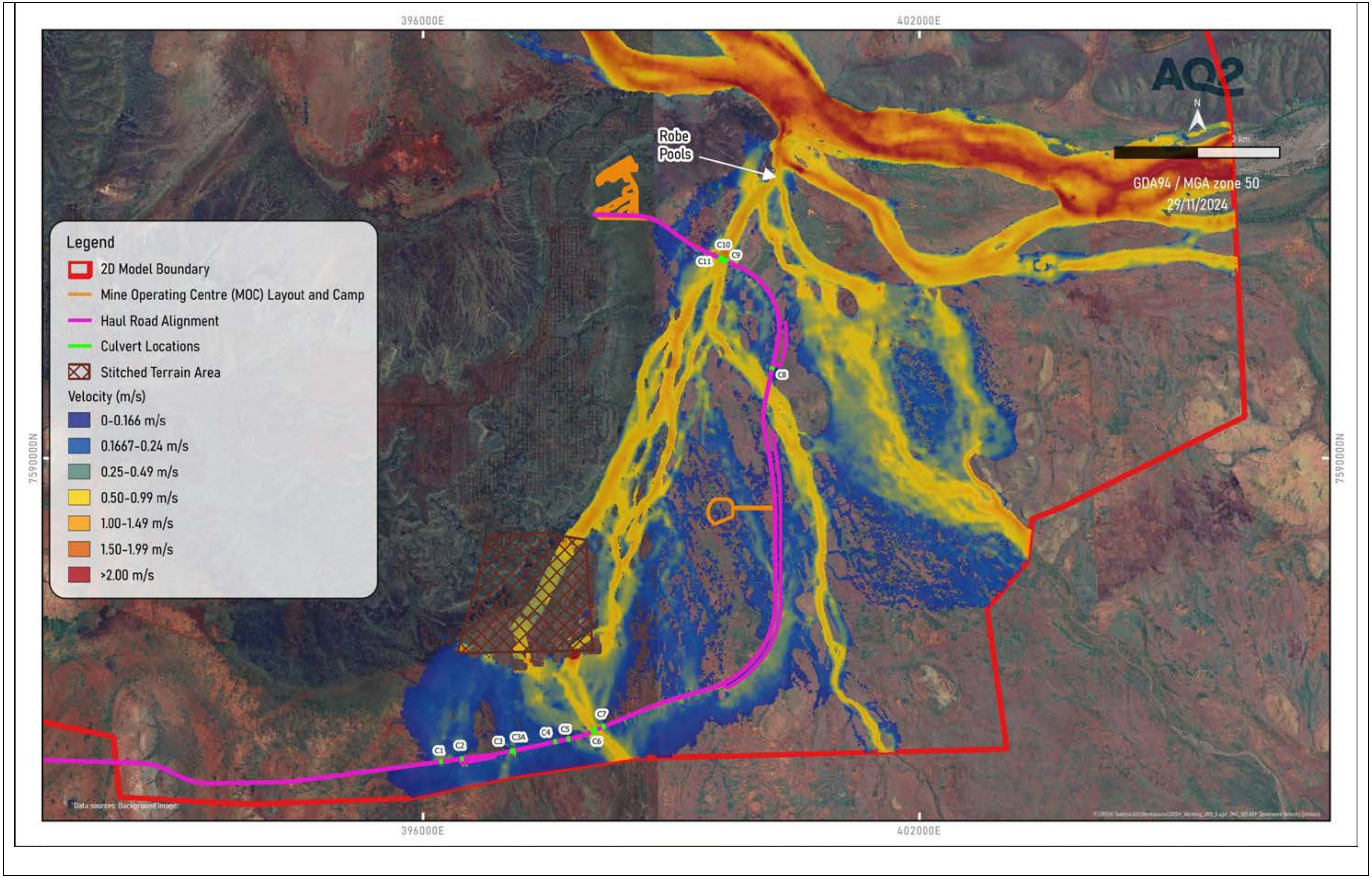


Figure 3-6 10% AEP Post-Development Flood Velocity

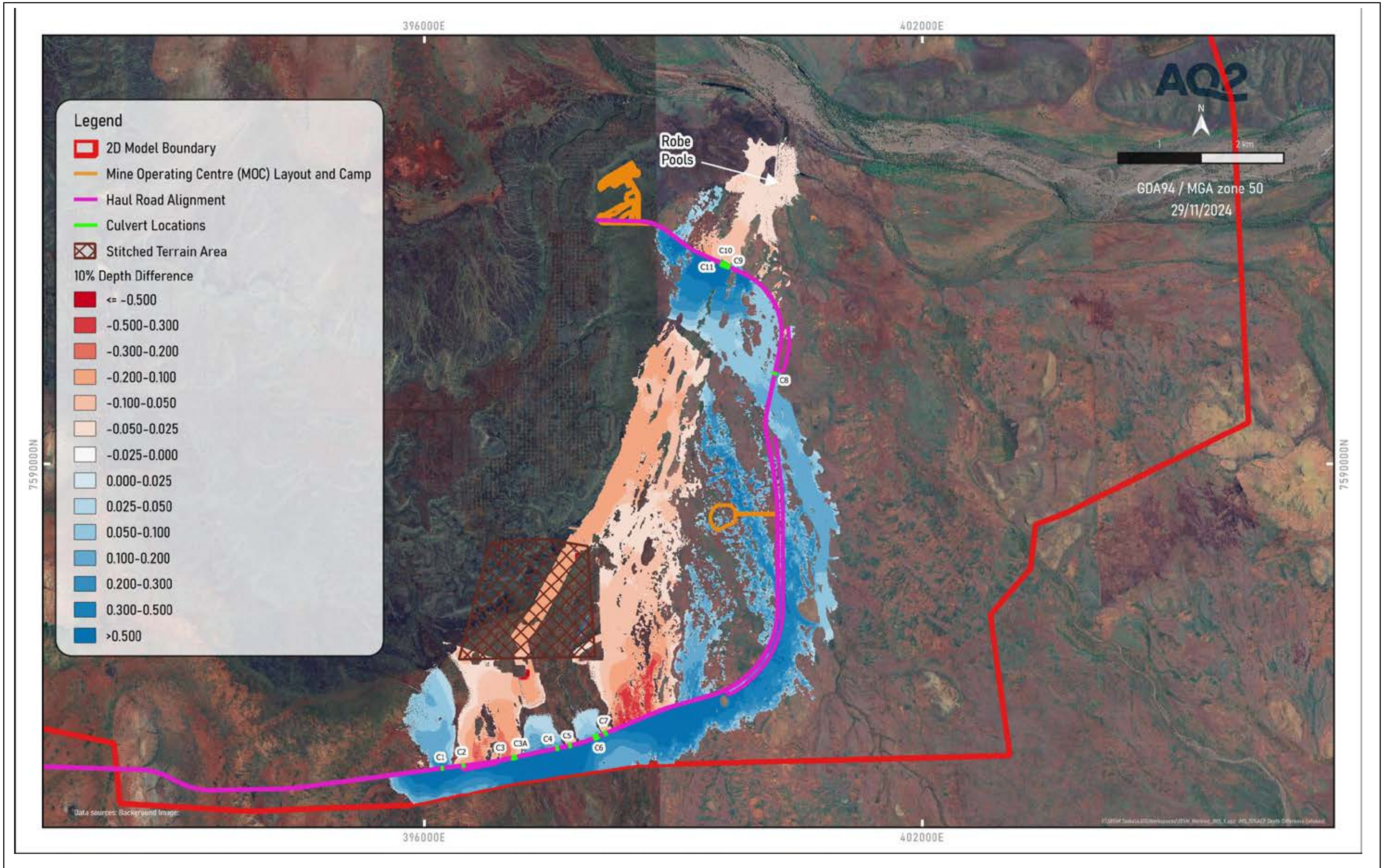


Figure 3-7 10% AEP Flood Depth Difference Map

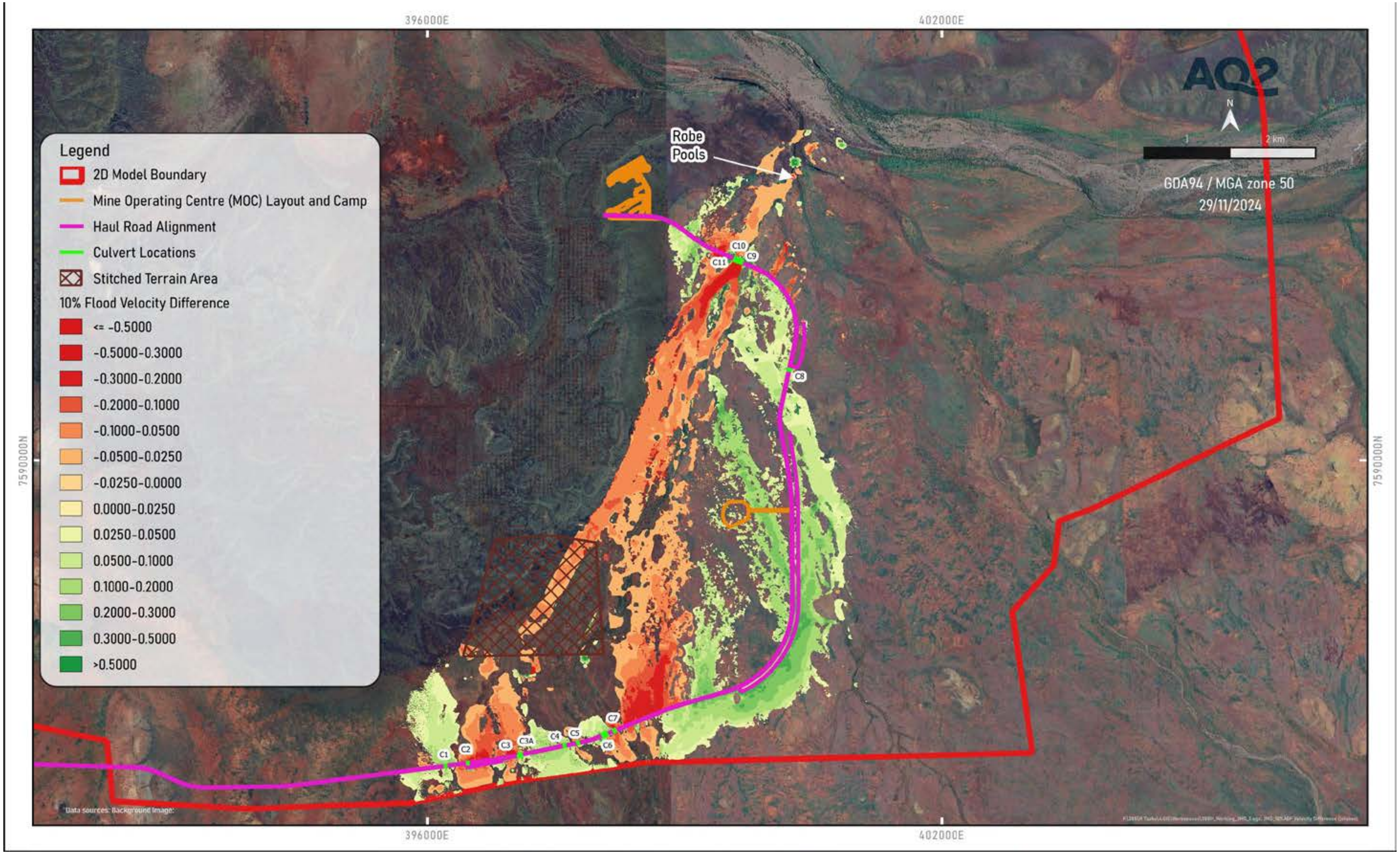


Figure 3-8 **10% AEP Flood Velocity Difference**

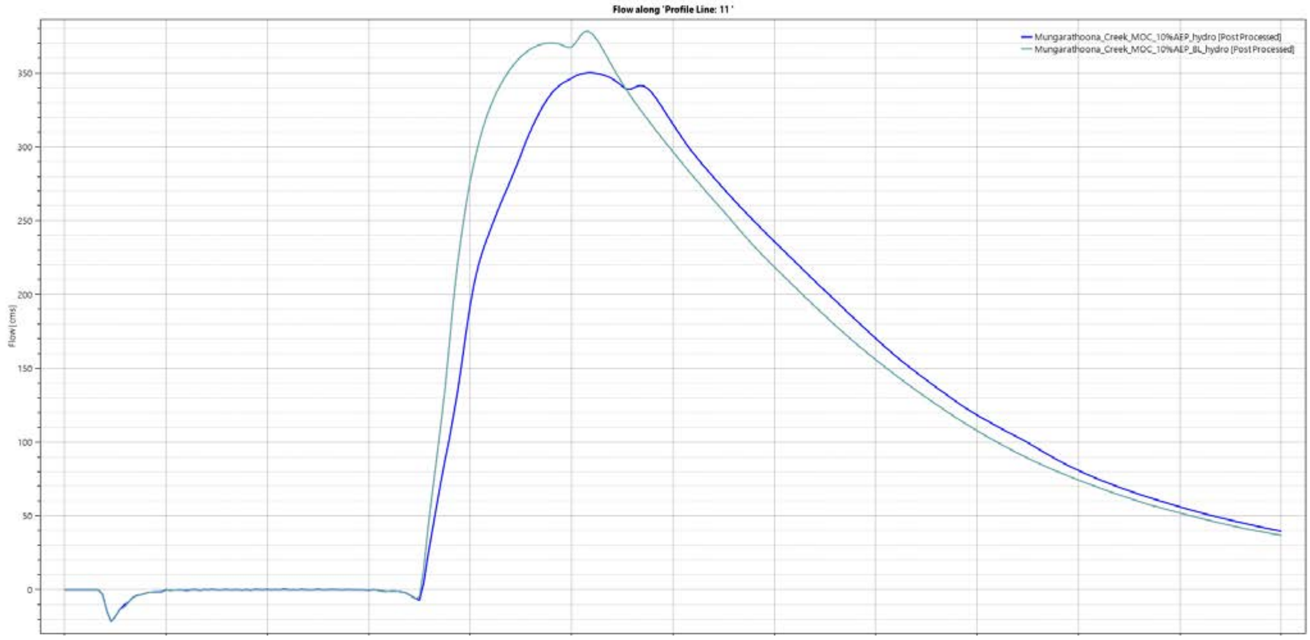


Figure 3-9 Pre-Development (Green) vs Post-Development (Blue) Flood Hydrographs between Haul Road and Robe Pools

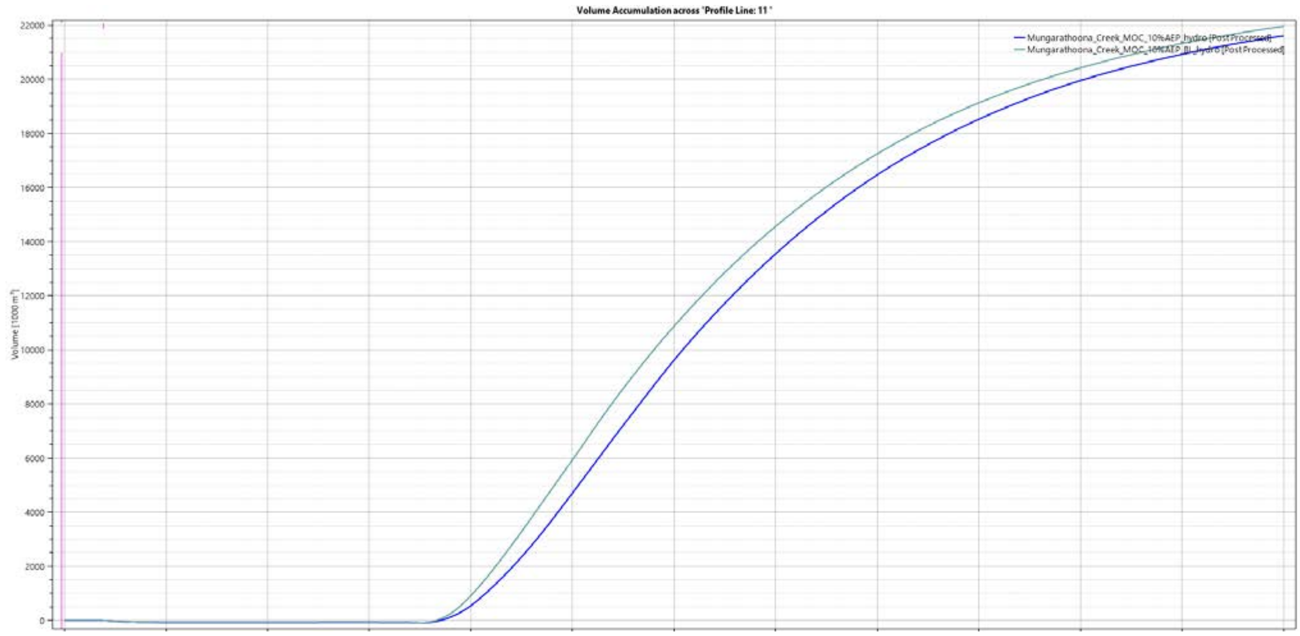


Figure 3-10 Pre-Development (Green) vs Post-Development (Blue) Flow Volume between Haul Road and Robe Pools

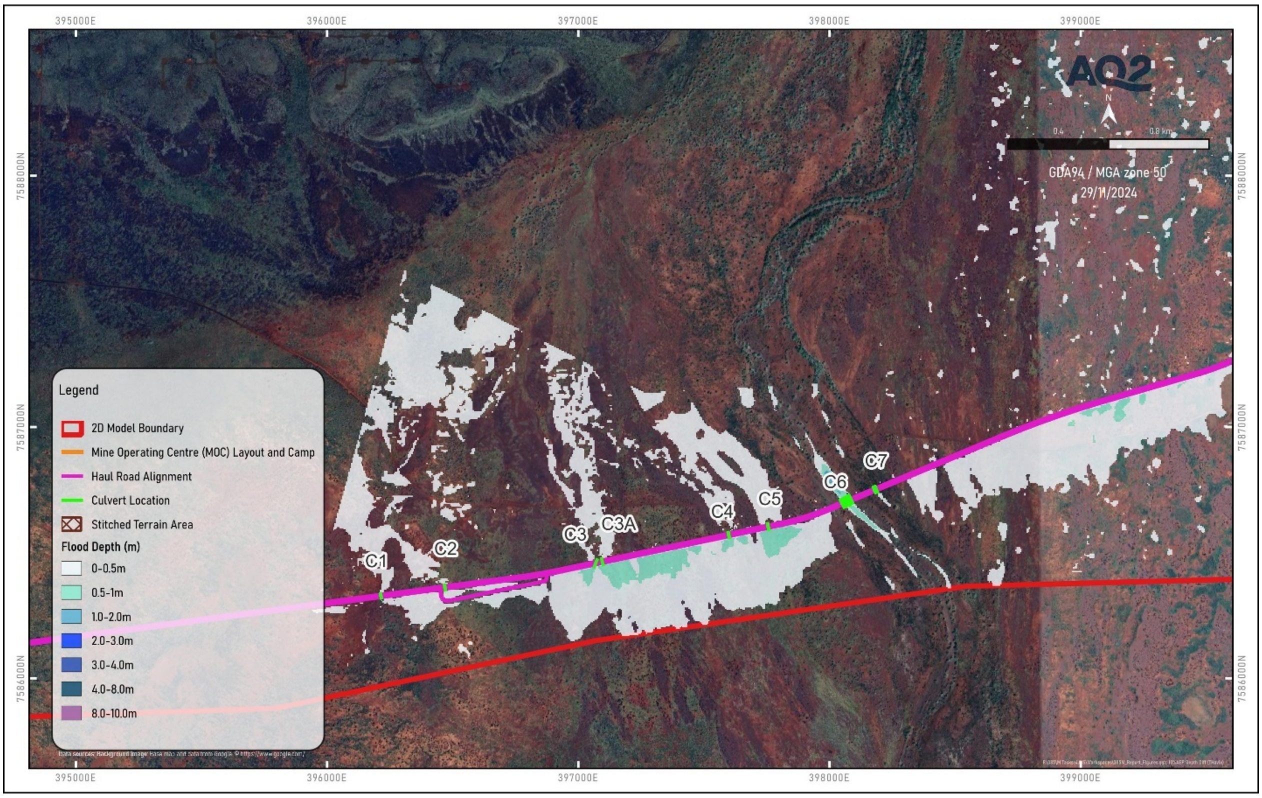


Figure 3-11 Ponding Location (Southern Haul Road Area)

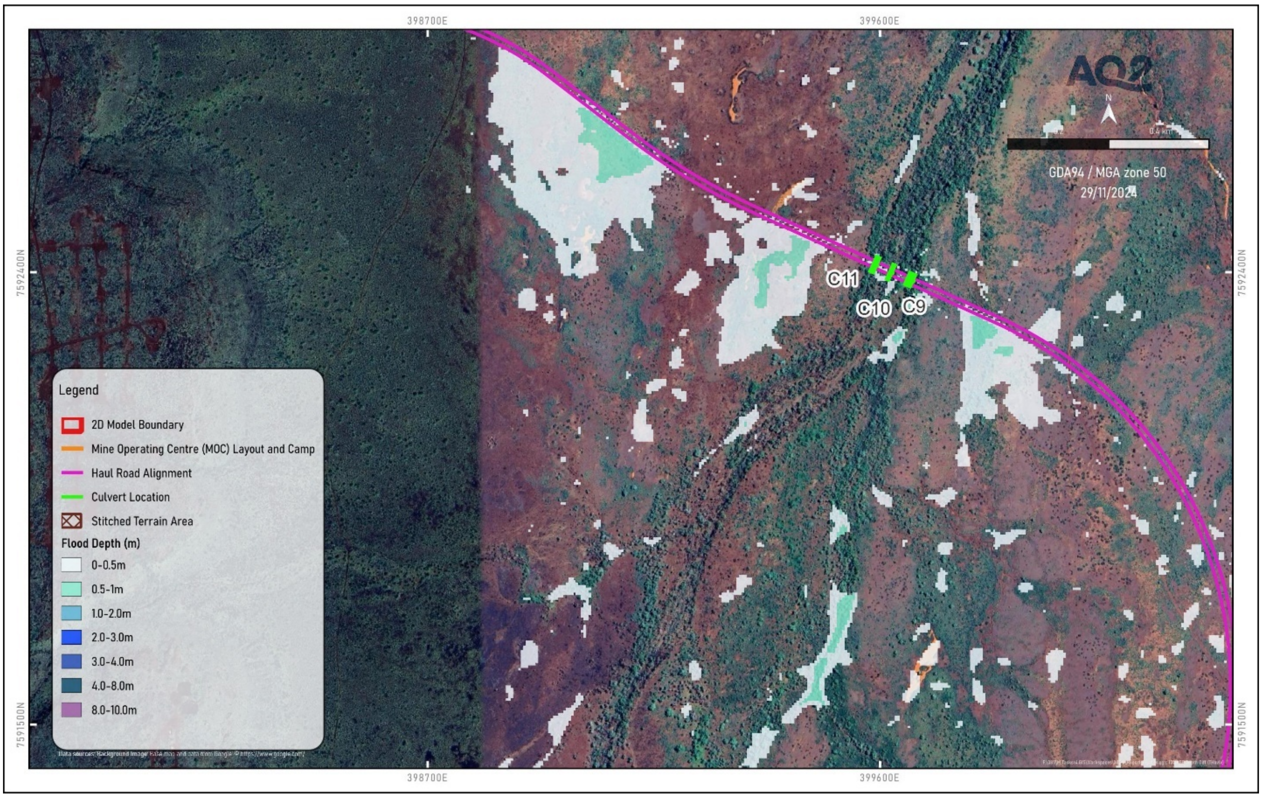


Figure 3-12 Ponding Location (Northern Creek Crossing)

The following observations are made in relation to the result predictions:

- Southern Mungarathoona Creek Crossing:
  - Increased flood depths are predicted along the upstream side of the haul road, as the road embankment creates a barrier to flow, with flow passing to the downstream side of the road via culverts or where flooding is predicted to overtop the road embankment. The predicted flood depth increases upstream of the haul road are, in places, greater than 0.5m, and instances are greater than 1.35 m.
  - The haul road attenuates the peak flows from the runoff event, such that maximum flood levels on the downstream side are generally reduced.
  - In some locations, the redistribution of flow due to the size and positioning of culverts and the points where flooding overtops the road creates areas where flooding is increased on the downstream side of the road.
  - The buildup of water behind the road at the southern crossing of Mungarathoona Creek causes water to flow parallel to the haul road eastwards and then to the north, causing increased flood depths through these areas (including the proposed camp location).
  - The changes to the inundation areas on the upstream side of the road will be smaller in smaller, more frequent runoff events.
  - The flow velocity difference map shows large areas upstream of the haul road predicted to have an increase in flow velocity. This seems counter-intuitive but represents the diversion of flow into areas which were not predicted to be inundated in the pre-development model (hence the increase in velocity predicted).
  - Note that, in reality, much of the areas which were predicted to be dry in the pre-development flood model may have shallow sheet flow runoff occurring which isn't simulated in the (Mungarathoona focused) modelling.
  - Water is predicted to pond against the haul road in areas where culverts aren't installed. An example is shown in Figure 3-11, where residual ponding of up to 0.8m is predicted to occur.
- Northern Mungarathoona Creek Crossing:
  - At the northern haul road crossing of Mungarathoona Creek (and upstream of Robe Pools), flooding is predicted to overtop the road.
  - Water is predicted to build up behind the haul road and spill to the west and into a local low point (drainage line) where overtopping of the haul road is also predicted to occur.
  - A reduction if maximum flood depth of 0.115m is predicted on the downstream side of the road crossing within Mungarathoona Creek.
  - Figure 3-9 compares the peak flow rates in Mungarathoona Creek between the Pre-Development and Post-Development model run at a point downstream of the northern road crossing and Robe Pools. The predictions indicate that the impact of the road on the flow rates downstream will be marginal (an unlikely to affect the Robe Pools).
  - Figure 3-10 compares the Pre and Post-Development cumulative flow volumes through a section between the haul road and Robe Pools. The predictions indicate that the reduction in flow volume will be negligible.
  - Although the flood modelling adopts a conservatively high flow rate through the Robe River, the modelling predicts that the Robe River flood level in the 10% AEP event is higher than the Mungarathoona Creek bed at the haul road crossing such that flow from the Robe River to the Robe Pools is likely to occur (independent to any changes to Mungarathoona Creek).
  - Predicted flow velocities are lower on the upstream and downstream side of the road crossing. However, there is likely to be some higher flow velocities where flood flows exit the culverts

and overtop the road (which aren't simulated in the model) and adequate erosion protection will be required in these areas.

  - Some pooling is predicted to occur on the upstream side of the haul road. In particular, on either side of creek crossing local low points are predicted to inundate during flood flows but will be unable to completely recede with the flooding.

4. SUMMARY

The 2D flood modelling for the CZR Robe Mesa has been revised to account for the revised MOC location and the haul road alignment. Post-development flood modelling has been based on preliminary civil designs for the haul road, which has included floodways, culverts and flood levees.

The flood model has been used to:

- Assess the flood risk to the revised MOC location and camp from the 1% AEP flood event.
- Quantify predicted changes to hydrological conditions within Mungarathoona Creek due to the construction of the haul road. A nominal flood hydrograph approximating the 10% AEP flood event has been used to quantify the potential impacts.

The assessment indicates that the impact of the haul road on the Mungarathoona Creek and the Robe Pools will be minimal.

We trust that this assessment meets your requirements. Please contact us should you require any further information.

Regards,

Tamar Haviv	Mark Nicholls
Water Resources Engineer	**Consulting Water Resources Engineer**

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