GRANTS LITHIUM PROJECT

Environmental Impact Statement

Appendix H

Surface water modelling reports

Prepared by EcOz

October 2018

Core Exploration Ltd, Cox Peninsula

Project 1: Existing hydrological condition and hydrology model calibration

Report ECA-HA-0004-01

Core Exploration Ltd, Cox Peninsula

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Aug 2018

EnviroConsult Australia Pty Ltd

Executive Summary

The report addresses the following:

1. Inspections of the Core Exploration Ltd ML 31726 site were conducted to assess hydrological conditions pre-mining, and
2. Calibration of the HEC-HMS hydrology model to regional catchments for use in accessing the site water balance and changes in stream flow and inundation caused by mining.

Existing site hydrology

The lease is intersected by the catchment boundary of Darwin Harbour West Arm catchment to the north and Charlotte River/Bynoe Harbour catchment to the south. Four major catchments were delineated that affect the lease. Catchments 5 and 2 flow through the large culverts to West Arm and Catchment 1 and 4 flow to Bynoe harbour. Most of the streamlines to the south of Cox Peninsula Road are shallow swales filled with black soils, which become more incised once they coalesce downstream of the culverts where the banks and channels consist of sandy sediment. The main channel in Catchment 4 is incised for a considerable distance before entering the tidal channels to Bynoe Harbour. All streams on ML 31726 are ephemeral. Stream Catchments 1, 2, 4 and 5 range in area from 6.27km^2 to 10.7km^2 with the dam catchment having an area of 1.65km^2. The peak catchment discharges for a 1%AEP rainfall event range from 98.2m^{3}s^{-1} to 124m^{3}s^{-1} based on Regional Flood Frequency Estimations.

Cox Peninsula Road runs through ML 31726 and there are two main culverts draining the lease which flow toward Darwin Harbour. Based on the probable design, these culverts will cause inundation on the lease during a flood event >0.11EY.

Numerous tracks are present on the site and some had become conduits for flow and have the potential to become drainage lines.

Old mine sites are present. The ones inspected were filled with water and bunded for safety.

The existing dam has a water surface area of 15.5ha at the spillway level of ≈30mAHD. The available 2-m digital elevation model (DEM) reflected the water surface level and not the underlaying terrain of the dam and based on that DEM a volume of only 0.25ML storage was calculated. Therefore, the volume of the dam at the spillway level was determined by estimating the original contours and the amount of cut required to build the dam wall. This gave water volumes at the spillway level of ≈364ML. There are numerous mature woody trees on the dam walls and although the dam is generally in good condition, these trees may cause structural damage and regular inspection is important.

There are no baseline hydrological data for the site but, data are available from the Bureau of Meteorology (rainfall) and the Northern Territory Government's water portal (rainfall, stream discharge) sites in the region. Analysis of rainfall from the Bureau of Meteorology sites indicated that regional rainfall is similar between sites and can be used in analysis for ML 31726.

Because of the steep terrain in the Bynoe Harbour catchments, it is unlikely storm surge will have an impact. In the Darwin Harbour catchment, the outlets of Catchments 1 and 2 is at ≈4.5mAHD, lower than the estimated 2010 1%AEP and 0.1%AEP surge inundation levels (4.7 and 5.3 mAHD) so storm

surge may have an impact. This is investigated further in Project 3: Inundation Risk Modelling of Large Rainfall Events.

Calibration of the HEC-HMS hydrology model

Since there are no site rainfall or stream discharge data, hydrological modelling will be used for water balance modelling and inundation modelling. The HEC-HMS hydrology model was calibrated to 3 offsite catchments where rainfall and/or discharge data are available.

These catchments are:

1. The Cullen Hill catchment which, although a considerable distance to the south, has similar surface morphology to ML 31726,
2. The Gulungul Creek catchment is located on a similar latitude but further to the east. This site has an excellent data set with 6-min interval rainfall for several locations and continuous discharge data recorded at the upstream inlet and the downstream outlet of the catchment, and
3. The Carawarra Creek catchment located near to ML 31726 and to the north. Rainfall data were not available for the site, but stream discharge data were. This catchment was calibrated using nearby BOM high resolution rainfall data.

The calibration of the model to these catchments showed that there was no significant difference between simulated and observed weekly flow volume and cumulative flow volume. Gulungul Creek, which has a very high-quality data set had the best fit between simulated and observed peak discharges. Gulungul Creek model had the highest efficiency coefficients, indicating predictive power, of 0.66, 0.95 and 1 for the hydrograph, weekly flow volume and cumulative flow volume respectively.

Although there are some differences in fitted and observed peak discharge for the 3 calibration catchments, fitted and observed flow volumes are good fits and this is the most important aspect of water balance modelling. The results were validated using Gulungul Creek and a different Wet Season's rainfall. Good fits were obtained confirming the parameter values obtained during the calibration process. This gives confidence that the calibrated model is an appropriate tool to determine catchment flows, water balances and inundation levels in later studies.

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1 Introduction

The Core Exploration Ltd Lithium resource is located at 695000m East, 8596000m North, GDA94/MGA52, near Berry Springs, Northern Territory. This report focuses on ML 31726 (*Figure 1*) but also includes the current dam catchment (*Figure 1*) which, although not located on the ML 31726, is to be used as a water supply.

EcOz Environmental Consultants commissioned EnviroConsult Australia Pty Ltd to undertake a hydrological and water balance assessment of the proposed mine site, both before mining and with proposed mine infrastructure in place. The assessment is divided into three projects:

1. Site visits, hydrological model calibration;
2. Catchment hydrology model/water balance; and
3. Inundation risk modelling of large rainfall events.

The report describes the result of Project 1: Site visits and hydrological model calibration.

1.1 Methodology

The steps undertaken to perform Project 1 were:

1. Aerial photo interpretation and field trips to confirm catchments and drainage features;
2. Collation and description of existing hydrological baseline data for the Cox Peninsula region and ML 31726 area;
3. Calibration of the hydrological model HEC-HMS to regional catchments using recorded rainfall and runoff data for those catchments for use in water balance and inundation modelling.

2 Site Description

Desk-top analysis of aerial imagery provided an understanding of the stream catchments in the area and the general channel network. This was followed with: 1) site inspections to confirm the nature and location of the existing channel networks, and 2) further desk-top analysis to adjust initial interpretations as required.

2.1 Stream Catchments

The two major watersheds dividing ML 31726 are the Darwin Harbour West Arm catchment and the lower reaches of the Charlotte River draining into Bynoe Harbour (*Figure 2*). Most of the natural drainage on the lease drains north towards West Arm.

There is a small amount of drainage on the lease toward the southwest into Bynoe Harbour.

There are four stream catchments intersected by ML 31726 (*Figure 2*, Catchments 1, 2, 4 and 5). A fifth catchment, Catchment 3, which is located outside the boundaries of ML 31726, contains the existing dam.

Catchments 2 and 5 form the headwaters of the Darwin Harbour West Arm Catchment. Catchment 1, 3 and 4 are in the Bynoe Harbour/lower reaches of the Charlotte River catchment. Details of the catchments are given in *Table 1*. All channels are ephemeral apart from the lower tidal reaches for Catchments 1 and 4.

Catchments 1 and 4 have the steepest thalweg slope and have a distinct break in the slope (*Table 1*). These channels rise at higher elevations than Catchment 2 and 5, and debouch into Bynoe Harbour through tidal, mangrove-lined channels, approximately 1850m and 2005m long respectively. After the slope break, the thalweg in Catchment 4 has a similar grade to Catchments 2 and 5 (West Arm), which rise at lower elevations than the Bynoe Harbour catchments.

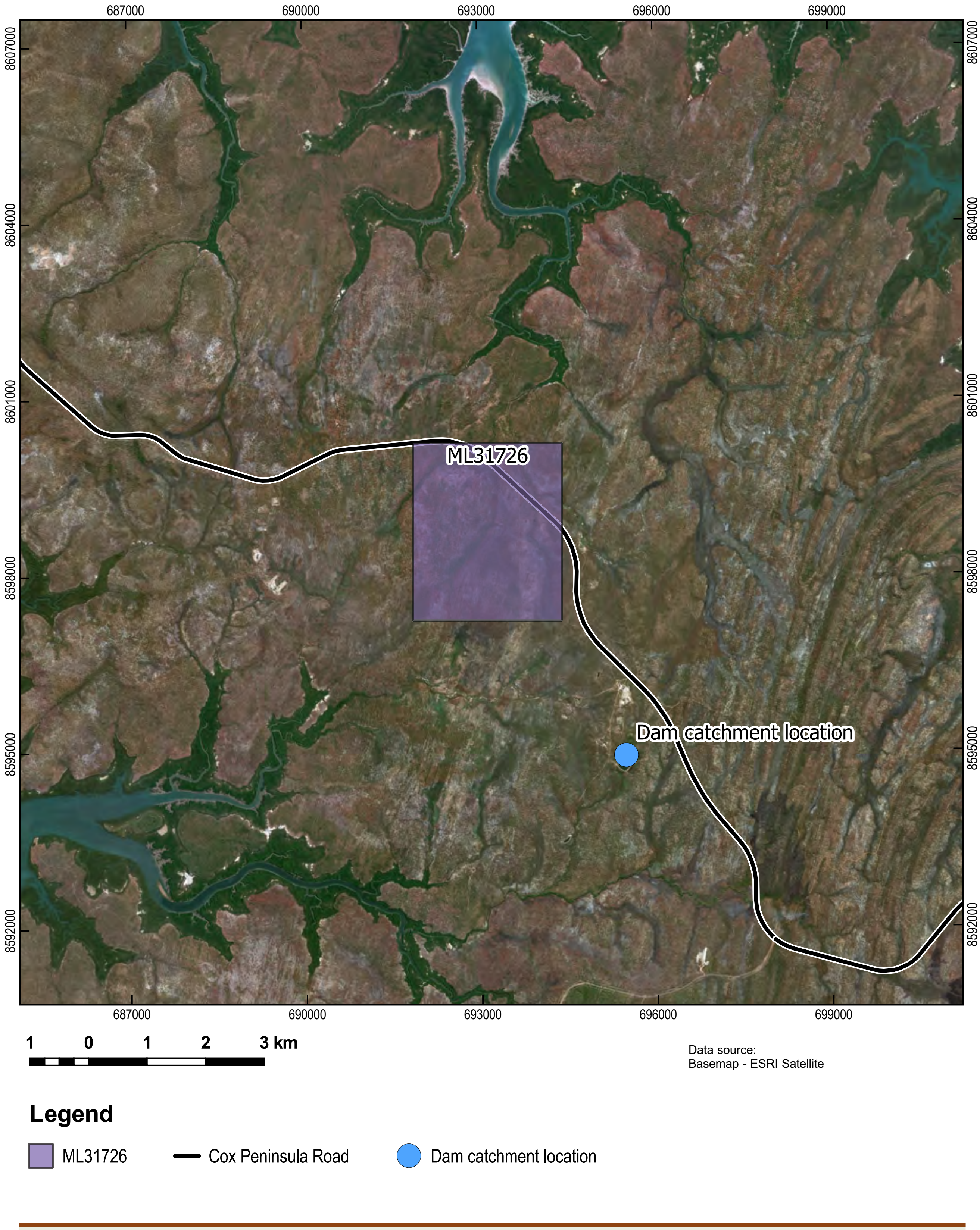
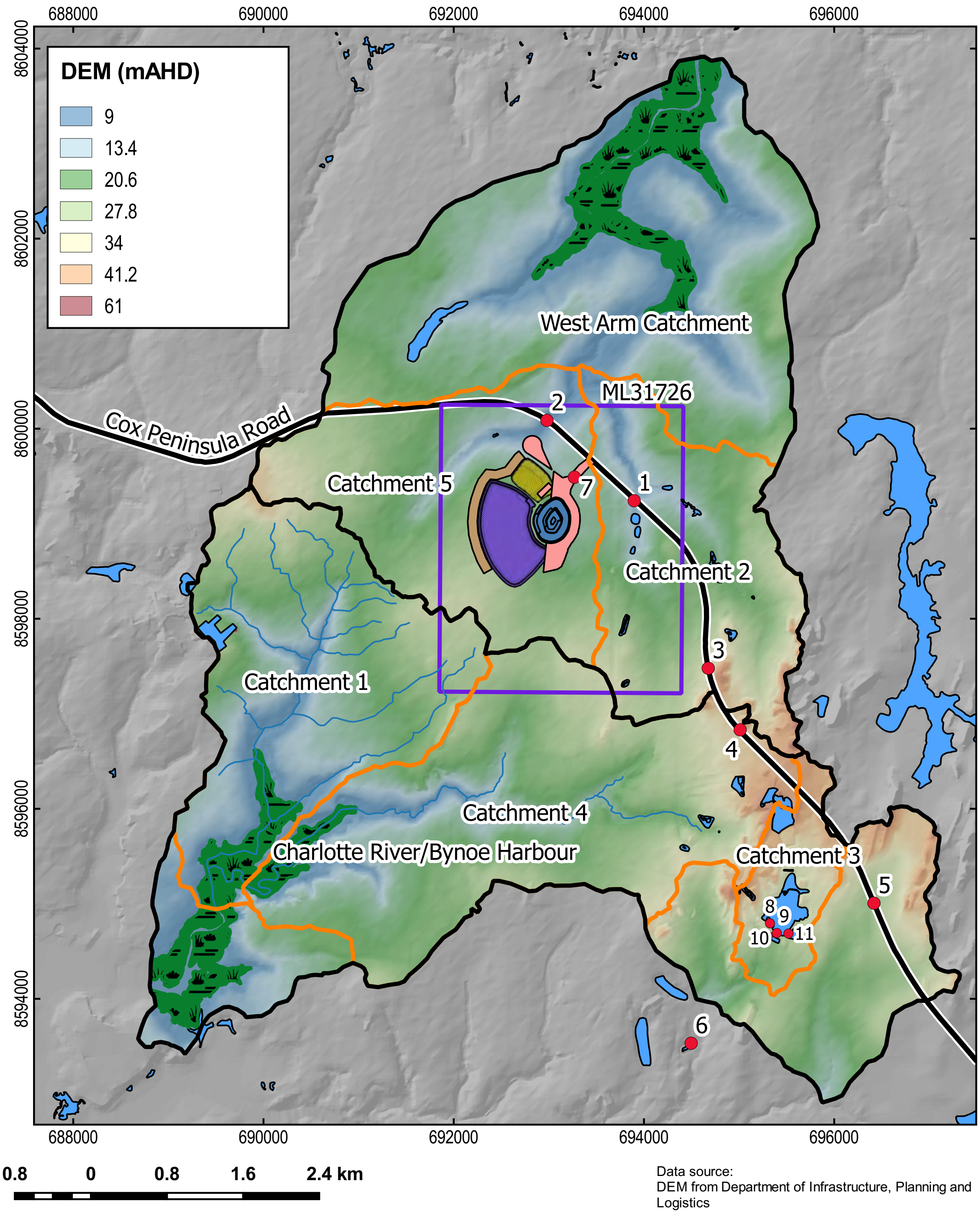


Figure 1. Location of ML31726 and dam catchment

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52



Data source:
DEM from Department of Infrastructure, Planning and Logistics

Legend

- ML31726
- Mine Pit
- Mine Dump Area
- Run-of-mine Pad
- Top soil pile
- Flood Diversion Bund
- West Arm/Bynoe Harbour Catchment
- Catchments
- Mangroves
- Water bodies and abandoned mines
- Inspection sites
- Cox Peninsula Road
- Streamlines

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Figure 2. Layout of the catchments delineated for this study

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

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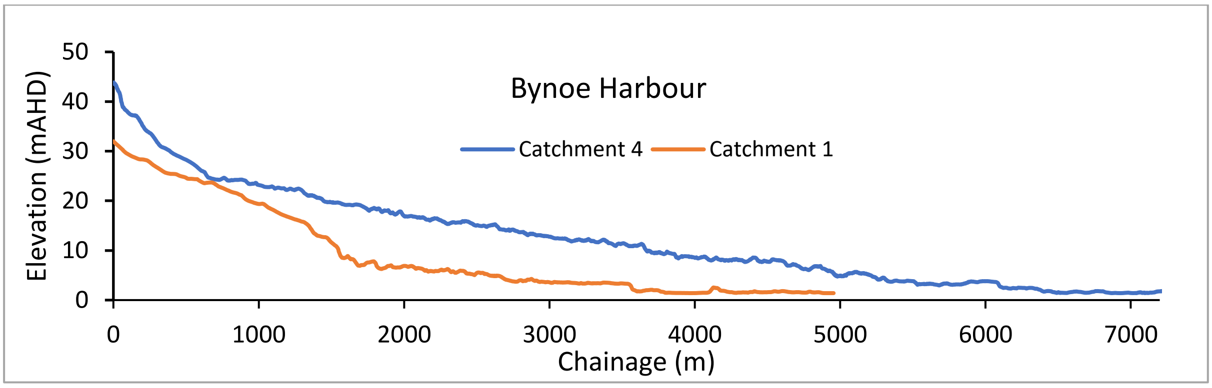
Table 1. Catchment dimensions, locations (based on MGA Zone 52) and RFFE peak discharges (Section 2.3.3 Design Rainfall Events).

Catchment	Area (km^2)	Stream slope (%)	Catchment Outlet		Catchment Centroid		RFFE
			Easting (m)	Northing (m)	Easting (m)	Northing (m)	Peak Q 1% AEP (m^3s^{-1})
1	7.85	0.8	689399	8595418	690486	8597417	100
2	6.27	0.6	689491	8594946	694459	8598489	98.2
3	1.65	NA dam impacted	695354	8594041	695525	8595204	43.8
4	10.70	0.6	689836	8595229	692746	8595951	124
5	7.27	0.5	693329.7	8600669	692144	8599234	109



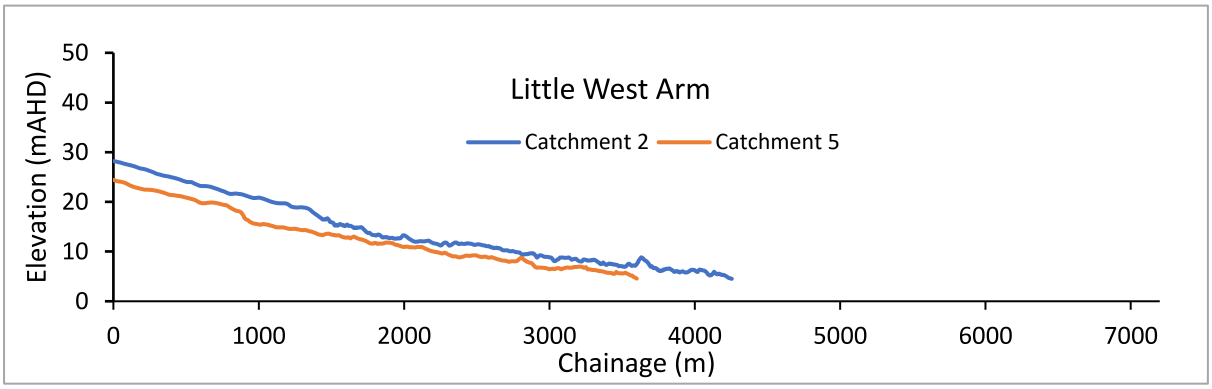


Figure 3. Long-section of the thalwegs of the catchments intersected by ML 31726.

3 Existing Site Hydrology

Information available for this project is listed below:

- GIS-based topographic information, which provided information on waterways and waterbodies, the location of proposed infrastructure and ML 31726,

- A 2m digital elevation model was purchased from the Northern Territory Government (NTG) and this allowed delineation of catchments and streamlines,
- ESRI Satellite (ArcGIS/World Imagery) aerial photography accessed through Quantum GIS,
- Daily rainfall records for nearby sites available for downloading from the Bureau of Meteorology (BOM), and 15-minutes interval rainfall from the NTG water portal,
- Regional stream flow data available from the NTG water portal, and
- Online storm surge maps for Darwin Harbour and Bynoe Harbour.

3.1 Data Requirements

Ideally, an assessment of surface water hydrology would be able to draw on the following data:

- Data from at least one continuously recording rainfall gauging station (pluviograph) on or near the site, capable of giving rainfall at sub-daily time intervals (hourly, minutes). These data are used for estimating flood behaviour and design of drainage facilities,
- Data from one or more stream flow gauging station on or near the site. These data are used for estimating flood flows and would assist in assessing long-term water availability for water supply purposes, and
- Details of historical flooding at the proposed site including the extent of inundation and frequency of flooding.

None of the data described above were available from this site, however, good regional data did allow calibration of a hydrology model that can be used to predict site discharges, water availability and inundation.

3.2 Regional Rainfall and Evaporation

The open source Bureau of Meteorology (BOM) daily rainfall records were considered suitable for use in assessing regional rainfall quality for this analysis (*Figure 4*). There are no rain gauges located near ML 31726.

A frequency distribution for annual rainfall was derived and is shown in *Figure 5*. Four sites (Dum In Mirrie Airstrip, Darwin River Dam, Wagait Beach and Territory Wildlife Park), represented by large dots in *Figure 5*, had similar annual rainfall distributions. The other sites, represented by small dots in *Figure 5*, had quite dissimilar distributions. The line of best fit and confidence limits were refined using the four similar sites. Each of these similar sites had at least 20 years of data on record. It is considered that any one of the similar sites is representative of daily rainfall on the ML 31726.

Figure 6 shows the similarity in mean monthly rainfalls at the four similar sites. Mean monthly evaporation data at the Darwin River Dam are also shown[1]. The distinct Wet and Dry season rainfall and the relationship to evaporation are obvious.

[1] Paiva J 1993. Darwin River Dam Safe Yield Improvement. Hanna's Pool Diversion. Power and Water Authority Water Resources Division. Report 26/93

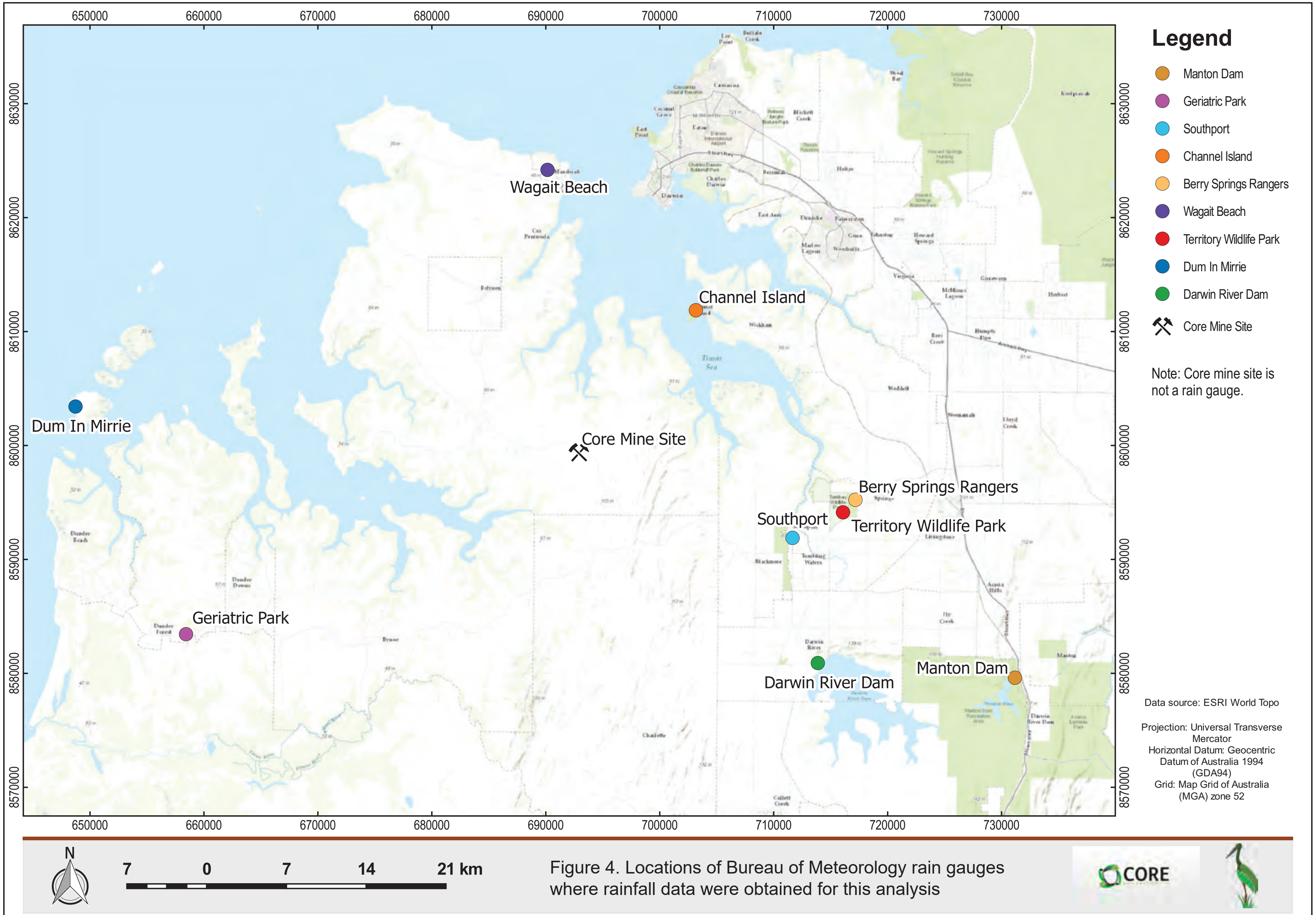


Figure 4. Locations of Bureau of Meteorology rain gauges where rainfall data were obtained for this analysis

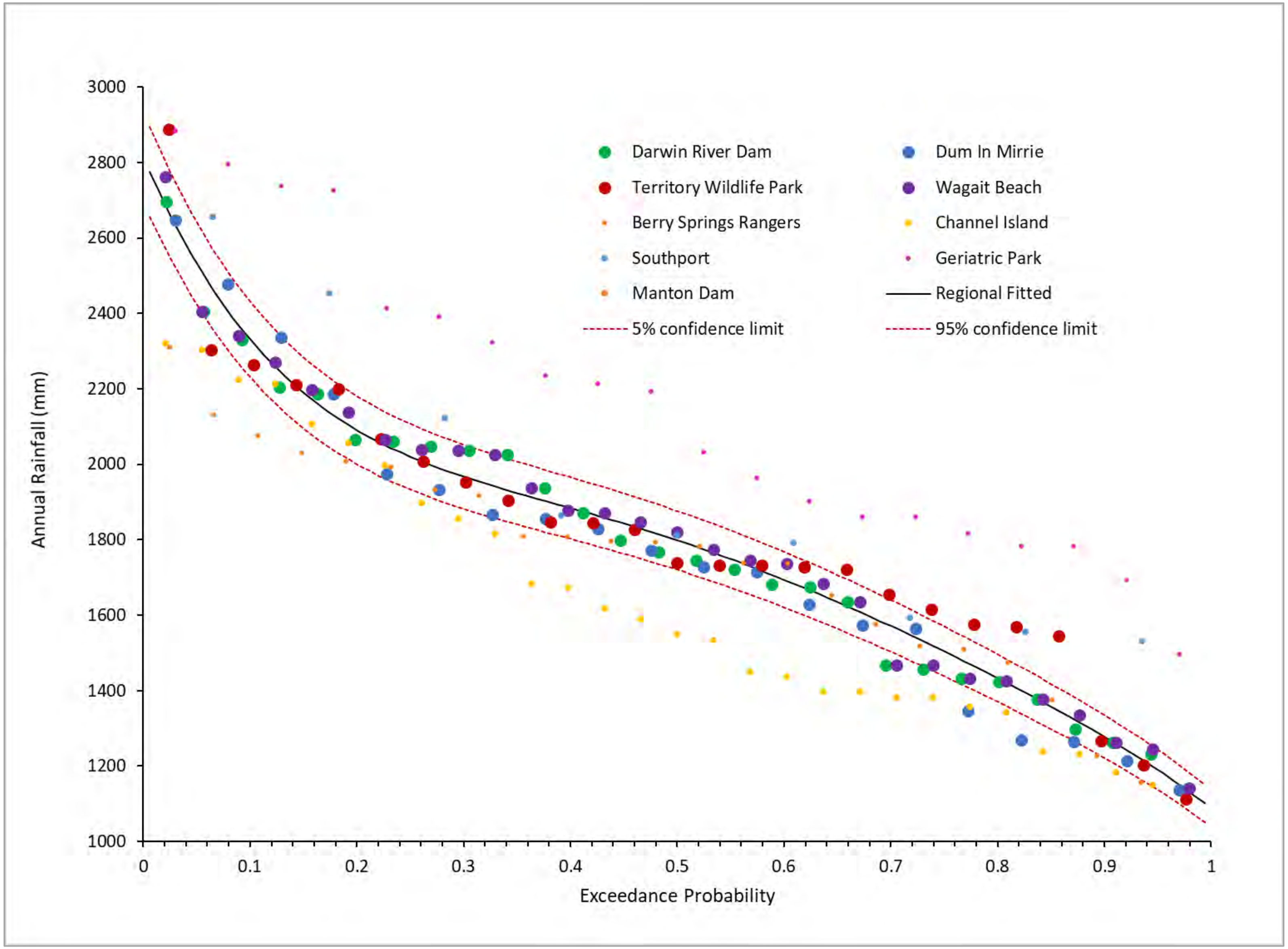


Figure 5. Probability of exceedance for annual rainfall recorded at the BOM gauges.

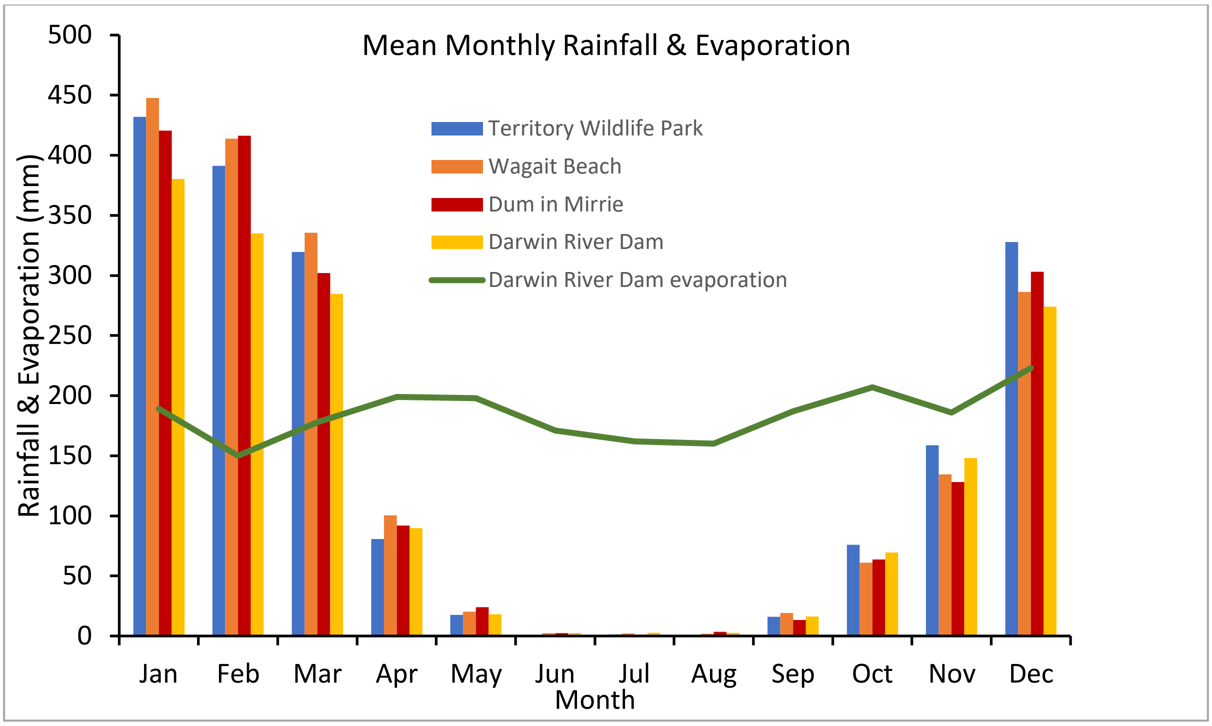


Figure 6. Mean monthly rainfall at the four sites with similar annual exceedance probabilities. Monthly evaporation at the Darwin River Dam is also shown.

3.3 Design Rainfall Events

In Project 3: Inundation risk modelling of large rainfall events, 1%AEP and 0.1%AEP[2] rainfall events will be used in the modelling. The 2016 Intensity–Frequency–Duration (IFD) design rainfalls for the site are provided by the BOM for use in conjunction with the 2016 edition of Australian Rainfall and Runoff (ARR2016, http://arr.ga.gov.au/arr-guidelines) to derive the probable rainfall runoff characteristics of such events required for the inundation risk modelling in Project 3.

Figure 7 shows the more frequent event IFD curves and *Figure 8* shows rare event IFD curves for ML 31726.

The Regional Flood Frequency Estimate (RFFE) method is an online tool developed by Engineers Australia to estimate design flood peak discharge (Q). Inputs are catchment outlet, centroid co-ordinates and catchment area. RFFE was used to determine the peak discharge (peak Q) at the outlet of the catchments for 1%AEP rainfall event (*Table 1*). The peak Q values are used as a calibration/validation tool for the inundation risk modelling.

The Bynoe Harbour Catchments 1 and 4 are the largest in surface area and have the highest RFFE-derived 1%AEP peak Q values (*Table 1*).

[2] This is the current terminology recommend by Engineers Australia for the return interval of rainfall events. 1%AEP is 1 in 100 or Q100 and 0.1%AEP is 1 in 1000 or Q1000.

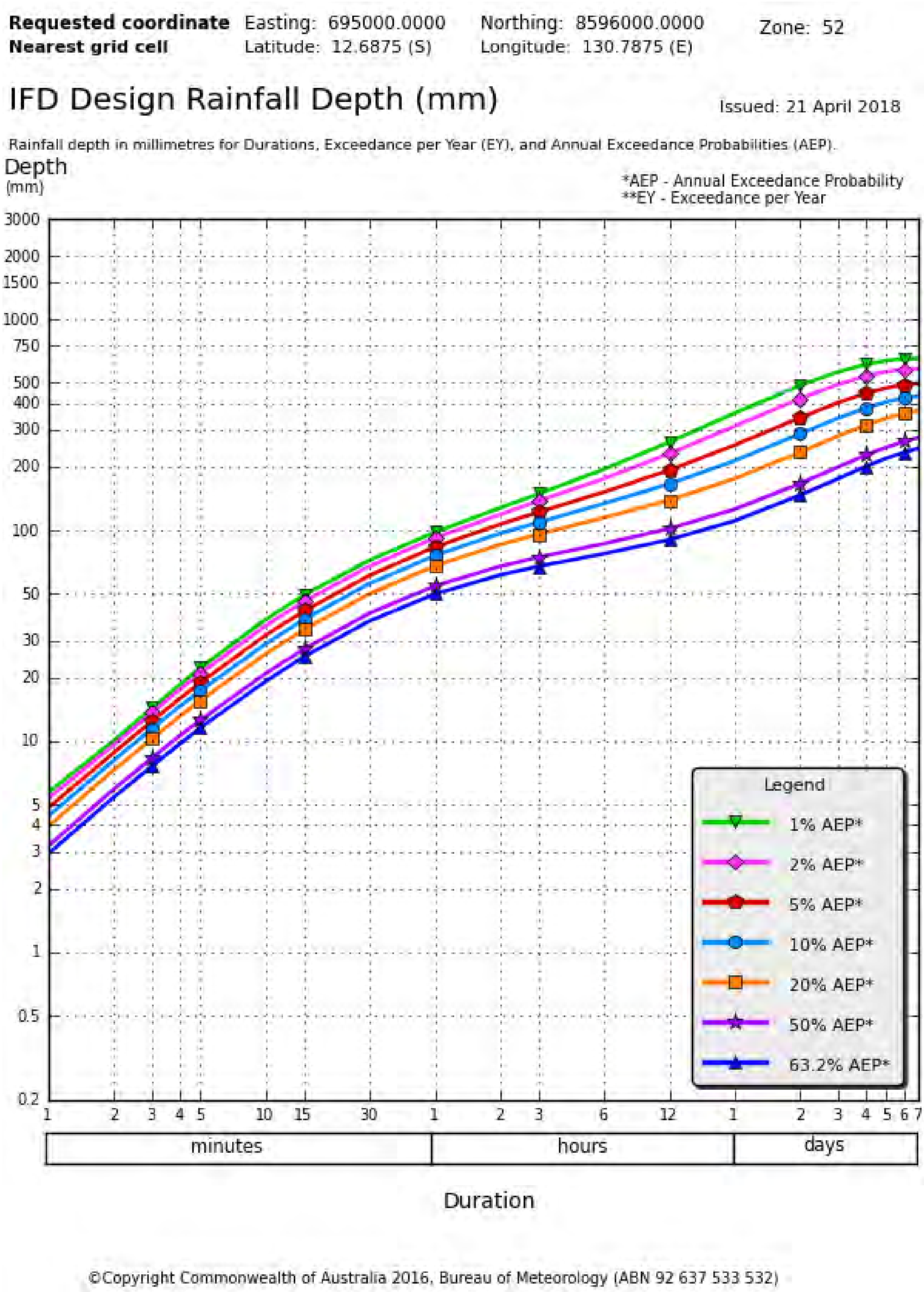


Figure 7. BOM intensity, frequency, duration curves for design rainfall up to a 1%AEP event at ML 31726.

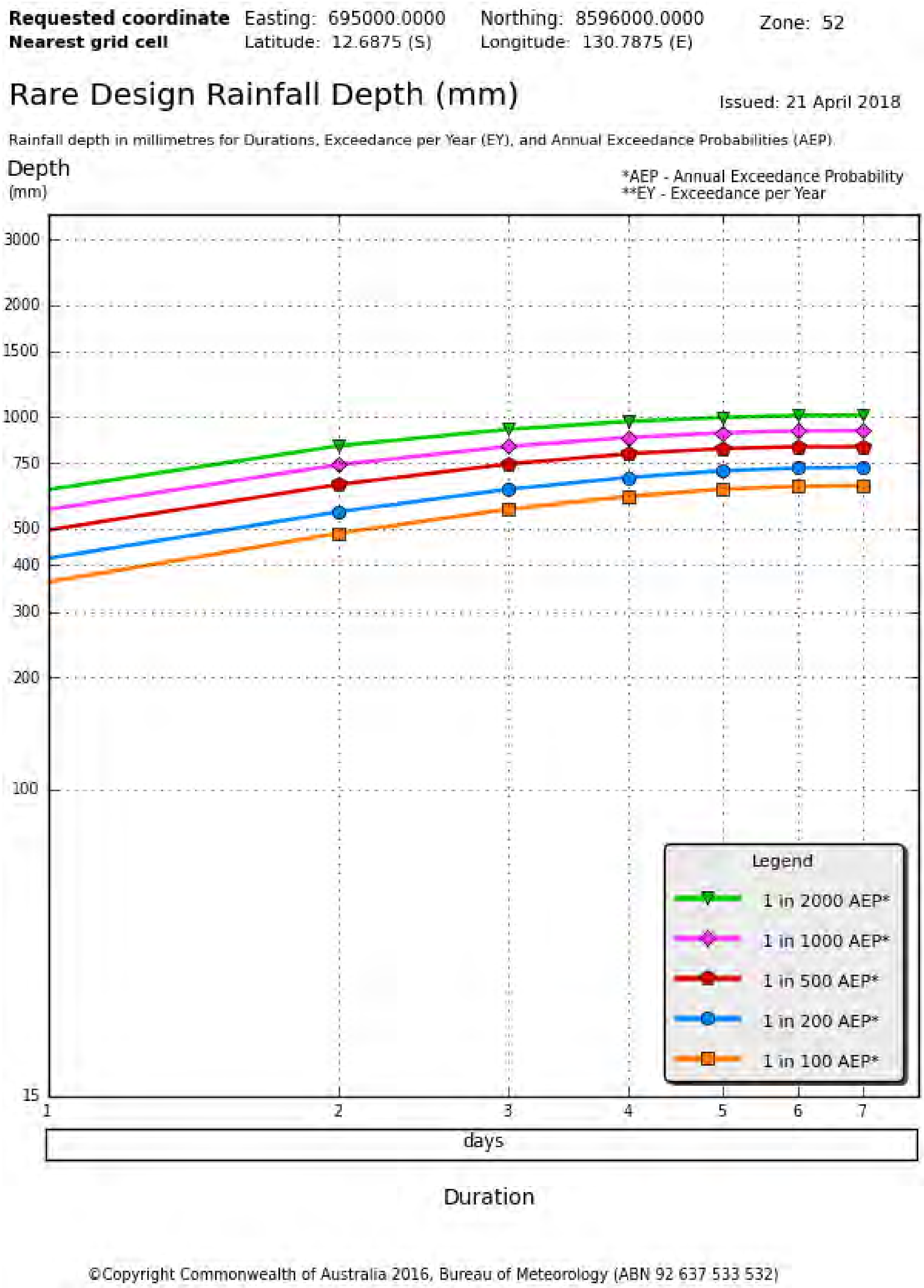


Figure 8. BOM intensity, frequency duration curves for rainfall events from 1%AEP to 0.05%AEP.

3.4 Site inspection

The site is located on the boundary of the Darwin Harbour West Arm catchment and the lower tidal reaches of the Charlotte River (*Figure 2*). The majority of ML 31726 is located south of Cox Peninsula Road.

Features inspected were road drainage and tracks, general drainage and streamlines (wherever accessible), old mine sites, and the existing dam.

3.4.1 Road drainage

The main access road to the site is the Cox Peninsula Road, a two-lane sealed carriageway rural road. Five culverts along Cox Peninsula Road were inspected (*Figure 2*). Three of these culverts lie within the boundary of ML 31726, draining the site toward West Arm, while the other two are outside the lease but are relevant to the existing dam. The two culverts outside ML 31726 drain toward the Charlotte River and Bynoe Harbour. It is likely that all five culverts inspected were designed for a 0.11 EY[3] flood event (1 in 10 y), although this observation is unconfirmed.

Culvert 1, on the lease, (*Appendix A 1*, *Appendix A 2*, *Appendix A 3*, *Appendix A 4*, *Figure 2*, Site 1), is a three-section reinforced concrete box culvert (RBC). At the time of inspection, the recessional tail of the hydrograph, resulting from the monsoon that occurred in the region in late January – early February 2018, was flowing with an average velocity of about 0.35ms^{-1}. The stream flows toward West Arm and it becomes sinuous as it enters the upper tidal reaches of West Arm. In this location the channel is lined with mangroves extending across wide overbank flood plains.

Culvert 2, also on the lease, is the other major culvert draining the lease that was inspected (*Appendix A 5*, *Appendix A 6*, *Appendix A 7*, *Figure 2*, Site 2). This culvert comprises two RBC culverts with erosion and scour control at the downstream outlet. Culvert 2 is a left-bank tributary of the stream that flows through Culvert 1, and they join about 650m downstream of Culvert 2. The flow velocity was 0.75ms^{-1} At the time of inspection on 23 February 2018.

Downstream of Culverts 1 (*Appendix A 4*, *Figure 2*, Site 1) and 2 (*Appendix A 7*, *Figure 2*, Site 2), plunge pools have developed.

Of the smaller culverts, Culvert 3 drains toward West Arm (*Appendix A 8*, *Figure 2*, Site 3) while Culvert 4 (*Appendix A 9*, *Figure 2*, Site 4) and Culvert 5 (*Appendix A 10*, *Appendix A 11*, *Figure 2*, Site 5) drain toward Bynoe Harbour.

Culverts will play a major role in site inundation during large floods, since they are most likely designed to flood during >0.11EY events.

3.4.2 Tracks

There are numerous vehicle tracks throughout the study area (*Figure 2*). The larger tracks provide access to old mine workings and drilling sites. Smaller tracks are probably caused by 4-wheel drive and other off-road vehicles.

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[3] This is the current terminology recommend by Engineers Australia for the return interval of more frequent rainfall events. 0.11EY is approximately 1 in 10 or Q10 event.

Across the Top End of the NT, the skeletal soils of the Koolpinyah Surface are quite fragile, and it is well recognised that vehicle ruts can quickly become gullies during the Wet Season. Therefore, these tracks are expected to become drainage controls and develop into streamlines during monsoons.

Some tracks have been regraded for the current drilling programme and temporary PVC pipe drainage has been put in place (*Appendix A 12*, *Figure 2*, Site 7).

3.4.3 Old Mines

There are several old open cut mines in the area. These are now filled with water and have been bunded to prevent access by people and animals. (*Appendix A 13*, *Figure 2*, Site 6)

3.5 The Existing Dam

The existing dam is in Catchment 3 draining towards Bynoe Harbour (*Figure 9*). The dam wall is of earth construction, approximately 10m wide at the top.

Ponding of water occurs at the downstream toe of the dam wall (*Appendix A 15*, *Figure 9*, Site 9) and there is a large gully developing at what appears to be the line where the constructed dam wall abuts natural terrain (*Appendix A 16*, *Figure 9*, Site 10).

A second gully, on the downstream side of the dam wall incising the top of the wall, was observed (*Appendix A 17*, *Figure 9*, Site 11). This gully is very small, and the dam wall appears to be stable in this area but ponding at the toe can cause saturation and this, combined with gullying, could over time cause sloughing or hydraulic failure in the wall. Gullies should be regularly repaired.

A visual inspection of the integrity of the dam wall was made. No geotechnical testing was conducted. The top of the dam wall grades down to a low point of ≈31.7mAHD from the western end (right abutment), at about the position of the gully in *Appendix A 17* (*Figure 9*, Site 11). The top of the dam wall then increases upgrade to the east. This level was taken from the DEM which is inaccurate in this area but was measured as ≈1.7m above the water level which was about 30mAHD at spillway level. The gully position (at the low point of the wall) indicates that the dam may overtop at this location during rare rainfall events and that runoff water from the top of the dam wall may accumulate here. The downstream toe of the wall is at a level of about 24.5mAHD and a downstream wall batter has a slope of about 25%. Elevations were taken using a hand-held GPS and checked on a 2m digital elevation model (DEM). The water surface was at spillway level at the time of inspection. The dam water surface area at spillway level is about 15.5ha.

In general, the dam wall appears to be in good condition with no major cracks or gullies apart from those mentioned above. However, there are many woody trees on the dam which can be detrimental to the wall structure. Sudden uprooting of trees can cause the displacement of a relatively large amount of embankment material, thereby lowering the dam crest, reducing the effective width of the dam, or facilitating seepage. Another potential danger associated with roots is their providing seepage paths through the dam which can, in time, penetrate deep into the dam embankment. This can result in piping and the subsequent removal of large volumes of embankment material. Excessive trees also hinder visual inspection and prevents the growth of grassy vegetation which is the preferred vegetation coverage.

Removal of the trees is a major task and the actual removal may cause damage. We recommend that a geotechnical assessment is conducted to assess the status of the wall and a dam wall inspection and maintenance schedule developed to prevent these issues becoming serious.

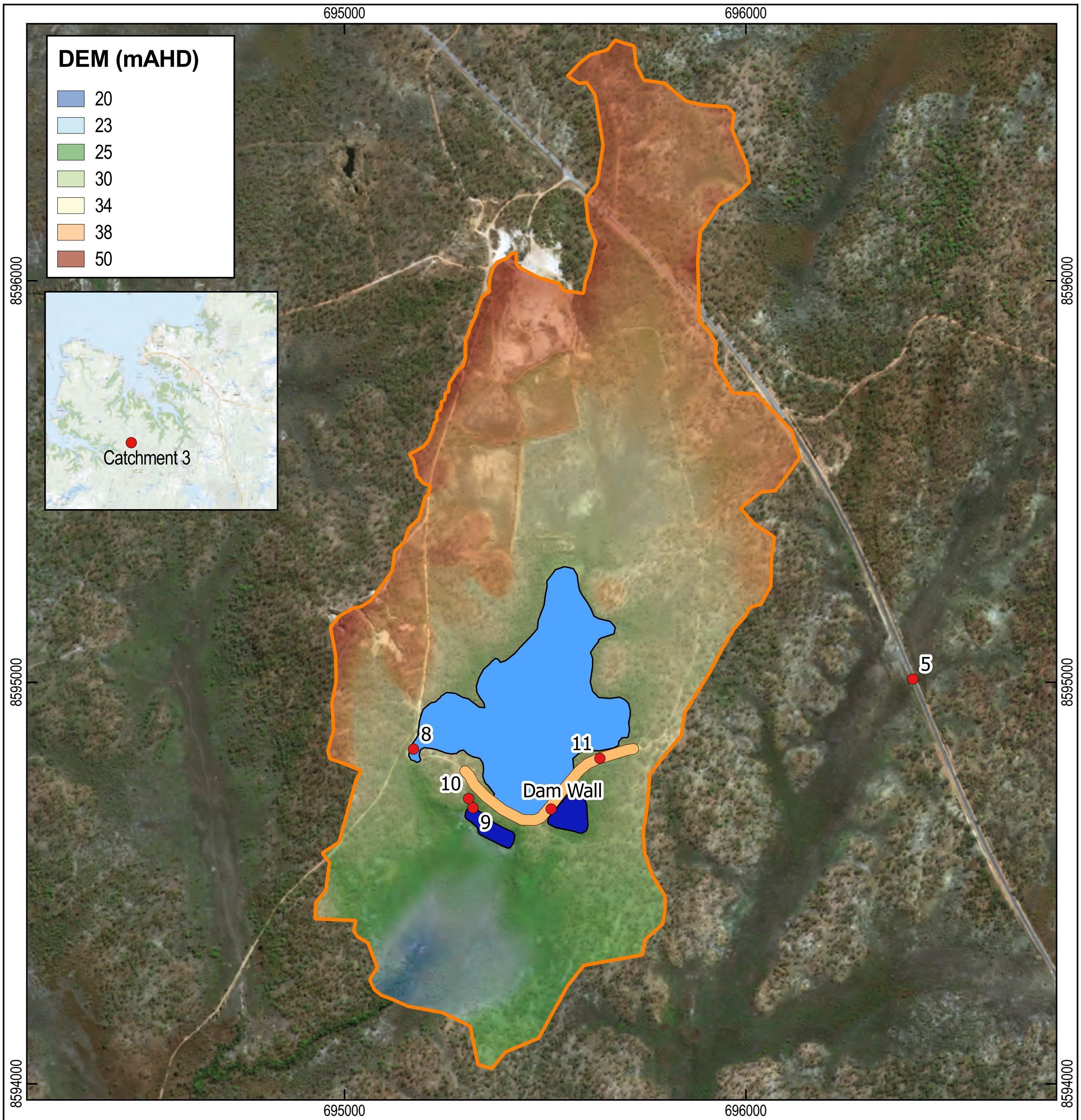
No bathymetric data were available for the existing dam.

A high quality 2m-DEM was purchased from the NTG to assist with hydrological analysis. This DEM appeared to reflect only the water surface at the spillway level (≈30mAHD) of Dam C3, rather than the true surface terrain underlaying the dam in that area, as the profile upstream of the dam wall was much flatter than expected. Furthermore, when QGIS software was used to calculate the volume of the DEM beneath the water surface at the 30mAHD spillway level, thus providing an estimate of dam water volume, the results showed that the volume of water in Dam C3 was only 0.25ML (*Table 2*).

The correct volume was estimated by recreating the bathymetry of the dam based on surrounding terrains and estimated cut and fill information. The upstream toe of the dam wall was assumed to be 24.6mAHD, similar to the downstream toe of the dam wall. The volume of earth that was cut to build the dam was estimated to achieve the wall at its current levels. The recreated bathymetry was then blended to the DEM of catchment 3, which is shown in *Figure 10*. A volume of ≈364 ML was estimated for Dam C3 at a spillway level of 30mAHD using the recreated DEM. As a validation exercise we determined the volume at the 30m contour of a neighbouring catchment of similar dimensions. The volume for this catchment, including excavation volumes for the wall was ≈342ML with a water surface area of 15.4ha, similar to the recreated bathymetry volume and surface area of Dam C3.

Table 2. Physical properties of the existing dam C3 and the recreated dam C3 (Figure 10).

Dam	Water surface Area(ha)	Volume at spillway level of 30m(AHD) (ML)
Existing	15.5	0.25
Recreated	14.1	364



0.1 0 0.1 0.2 0.3 km

Data source:
DEM from Department of Infrastructure, Planning and Logistics
Zoomed-out map from Australia - Whole Country - National Map Colour Topographic Base Map

Legend

- Cox Peninsula Road
- Catchment 3
- Dam
- Water surface at spillway level
- Inspection Sites
- Streamlines
- Footwall Ponds

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Figure 9. Catchment 3 which contains the water storage dam

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

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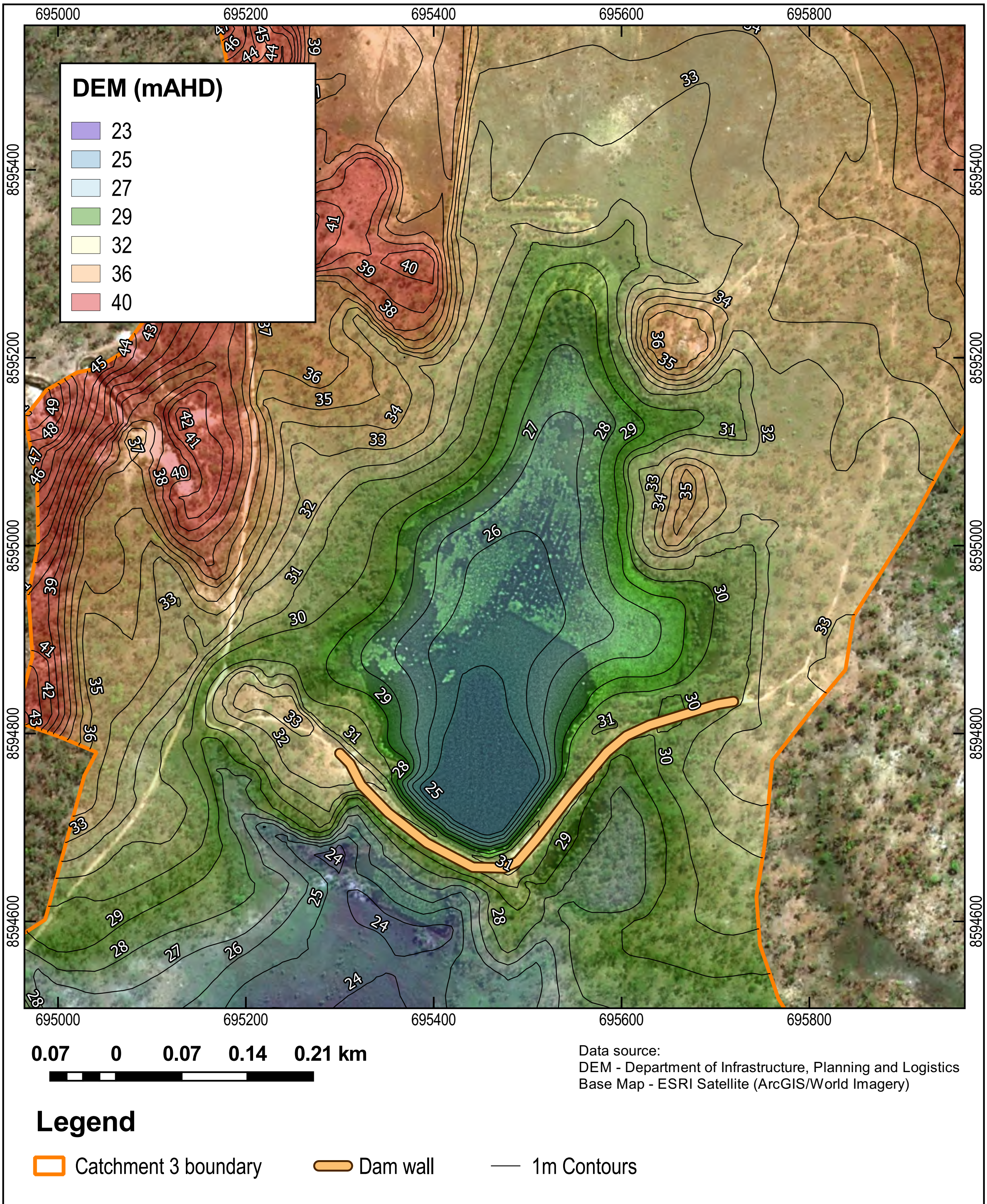


Figure 10. Catchment 3 DEM which contains the recreated bathymetry of water storage dam

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

3.6 Storm Surge

Storm surge data are available from the NTG for Darwin Harbour and parts of Bynoe Harbour. The Bynoe Harbour data were reported by CO_2 Australia and Seafarms, in their environmental assessment report to the NTEPA for commercial development. Storm tide heights for the primary surge 1%AEP and secondary surge 0.1%AEP events are shown in *Table 3*

Table 3. Storm surge levels in Darwin and Bynoe Harbours.

Year	Darwin Harbour		Bynoe Harbour	
	Primary surge (mAHD)	Secondary surge (mAHD)	Primary surge (mAHD)	Secondary surge (mAHD)
2010	4.7	5.3	3.7	4.7
2050	5.1	5.7	4.1	5.6
2100	5.6	6.5	4.6	6.2

Due to the steep topography of the Bynoe Harbour catchments (Catchments 1 and 4) (*Figure 3*) upstream of the slope break, it is unlikely these catchments will be affected by storm surge apart from a small amount in the lower tidal reaches. However, the outlets of the Darwin Harbour Catchments 2 and 5 are at ≈4.5mAHD, lower than the estimated 2010 1%AEP and 0.1%AEP surge inundation levels (4.7 and 5.3 mAHD) for Darwin Harbour (*Figure 11*), so storm surge may have an impact. This is investigated further in Project 3: Inundation Risk Modelling of Large Rainfall Events.

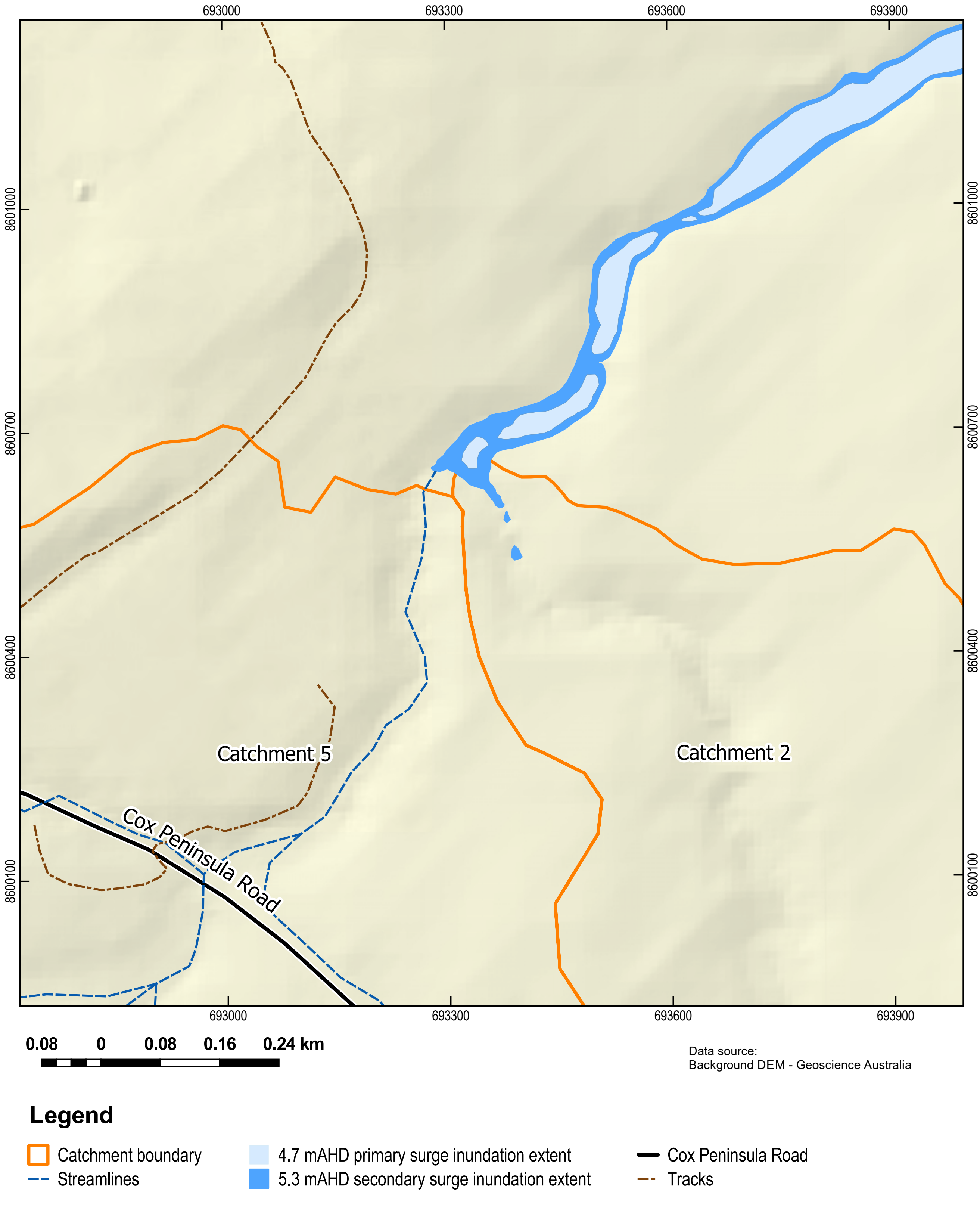


Figure 11. Estimated 2010 1%AEP and 0.1%AEP surge inundation extent near the outlets of Catchments 2 and 5.

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

4 HEC-HMS Hydrological modelling

4.1 Introduction

No site rainfall or stream flow data that can be used to estimate an annual water budget is available for the Core Exploration project ML 31726 site. An alternative to site data is the application of a well-calibrated hydrology model that can be used to simulate flow volumes resulting from applied rainfall, measured at another location, at various catchments of the site. In this study HEC-HMS is used (Appendix B1)

Without site data it is important that a model is calibrated to a nearby site/s. Additionally, long-term high-resolution rainfall and stream discharge are required. 15-min rainfall data and stream discharge data records from the NTG water portal were used to calibrate the HEC-HMS hydrology model against three similar catchments in the region:

1. The Cullen Hill catchment which, although a considerable distance to the south, has similar surface morphology to ML 31726,
2. The Gulungul Creek catchment, located on a similar latitude but further to the east. This site has an excellent data set with 6-min interval rainfall for several locations and continuous discharge data recorded at the upstream inlet and the downstream outlet of the catchment, and
3. The Carawarra Creek catchment located near to ML 31726 and to the north. Rainfall data were not available for the site, but stream discharge data were. This catchment was calibrated using nearby BOM high resolution rainfall data.

There is relative uniformity in land surface morphology across the NT's Koolpinyah Surface, which gives confidence that calibration to these relatively undisturbed catchments is appropriate.

4.2 Calibration of HEC-HMS models for Carawarra Creek, Gulungul Creek and Cullen Hill catchments

4.2.1 Calibration procedures

Model calibration is an important part of HEC-HMS modelling process to determine catchment water balance. During calibration, model parameters which cannot be assigned precise values based on available information are systematically adjusted until model results achieve an optimal fit with observed data. A common approach for model calibration is to fit the predicted hydrograph to the hydrograph observed by stream gauging within the modelled catchment. However, stream gauging data were not available on site. Therefore, the alternative approach of calibrating the model based on similar alternate catchments was used instead.

The three catchments used in calibration lie within a 250 km radius around ML 31726 (*Figure 13*) and have similar morphology and landcover types to the ML 31726 catchments. The Cullen Hill catchment and the Gulungul Creek catchment have previously been inspected in other studies and the morphology and hydrology is well understood. The Carawarra Creek catchment is discussed in greater detail in *Section 4.2.2*.

The optimal parameter values determined during the calibration process are expected to be applicable to the ML 31726 catchments. The locations of the three calibration catchments are shown in *Figure 13* and their physical properties are presented in *Table 4*.

Table 4. Properties of the calibration catchment.

Catchment name	Distance to mine (km)	Area (km^2)	Main channel length (km)	Main channel slope (%)
Carawarra Creek	15.7	51.54	13.4	0.15
Gulungul Creek	228.4	27.02	8.0	0.20
Cullen Hill	183.2	449.3	36.8	0.11

The Cullen Hill catchment was calibrated first, with the calibrated parameter values from the Cullen Hill catchment model then used as initial parameter values for the calibration of the models for the other 2 sites. It was expected that using the same parameter values in the other two models should also achieve a satisfactory fit between model results and observed data.

The details of the calibrated models are shown in Appendix B. Calibrated models.

4.2.2 Carawarra Creek

With respect to the Carawarra Creek catchment, there were no rainfall data at 15min resolution available. However, this catchment was chosen because of its proximity to ML 31726 and the good quality discharge data available. It was considered important to have at least a partial calibration from a site close to the mine. It could also be used to assess how well the parameter values fitted in the other calibrations, predicted discharge at the ML 31726. Therefore, 15-min rainfall from the NTG water portal Winnellie site was used. Although the site is some distance from ML 31726, the annual exceedance probability is somewhat similar to the regional, fitted exceedance probability (*Figure 12*).

Although it is unlikely there will be similarities between small localised events at Winnellie and ML 31726, widespread Monsoonal conditions may have similar rainfall. The aim of this exercise was to assess if the fitted parameters would produce predicted total runoff volumes similar to those measured at Carawarra Creek. This is also a form of validation.

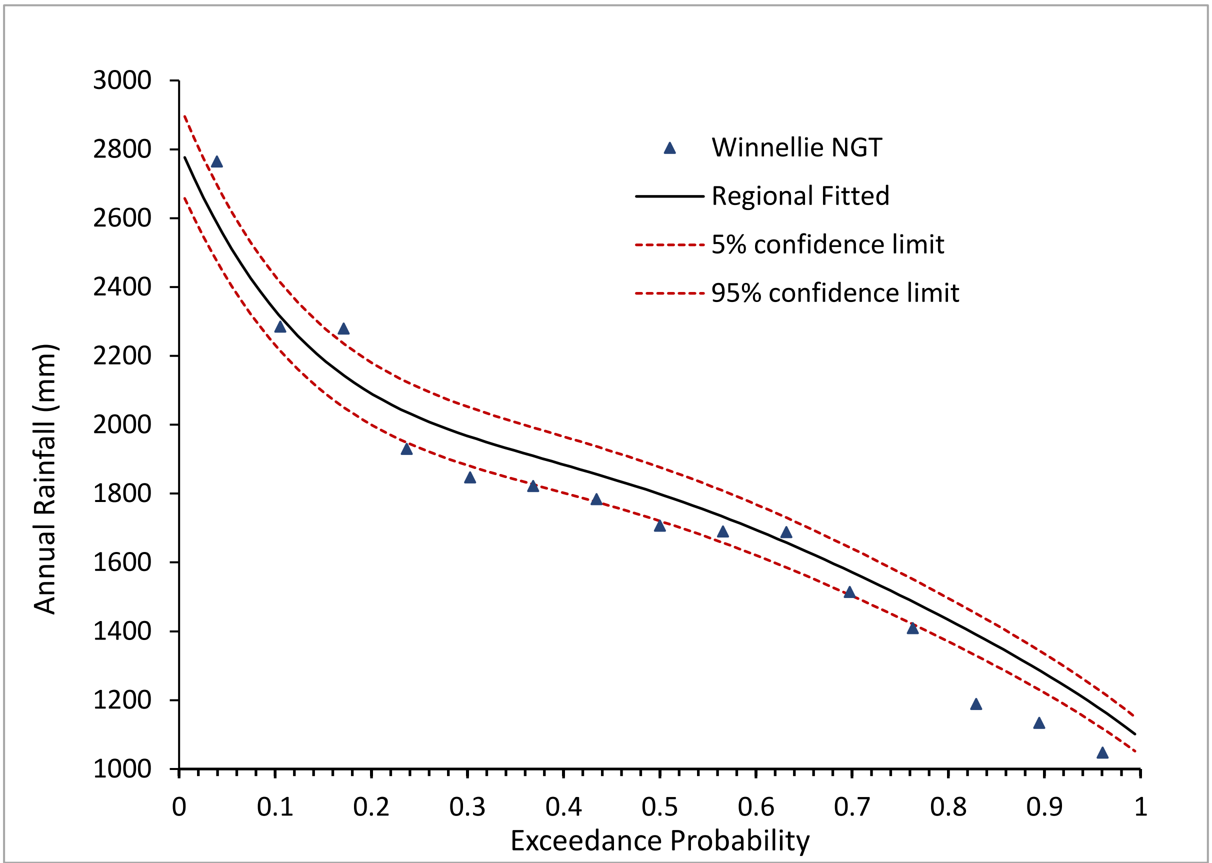
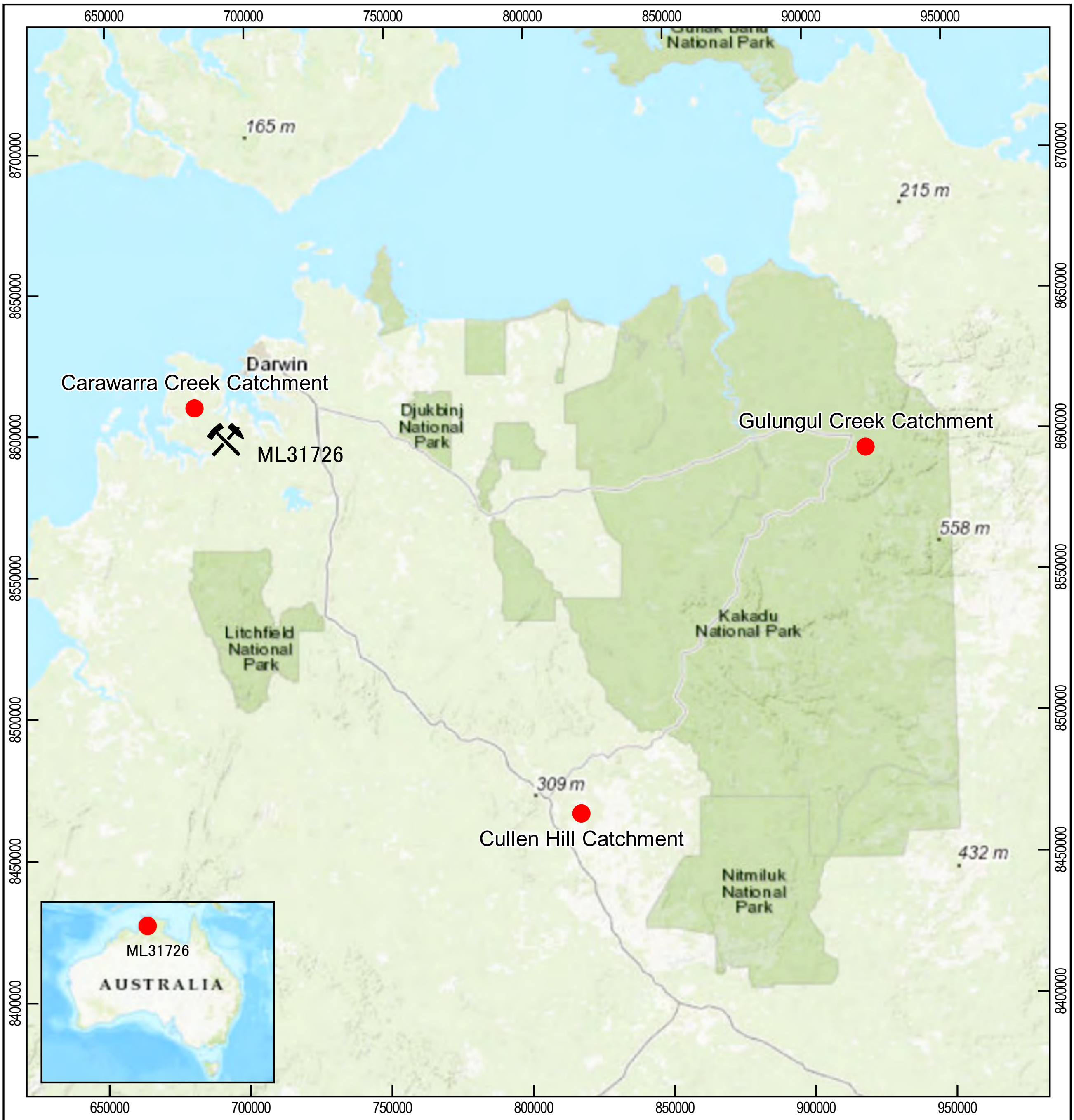


Figure 12. Regional annual exceedance probability for ML 31726 with Winnellie NTG data plotted.



20 0 20 40 60 km

Data source:
Basemap - Esri World Topographic Map
Zoomed out map - Esri World Topographic Map

Legend

ML31726

Calibration catchment location

N

Figure 13. locations of the calibration catchments

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94) Grid: Map Grid of Australia (MGA) zone 52

4.2.3 Calibration results

The fitted parameter values for each model are shown in *Table 5*.

Table 5. Calibrated parameter values

Catchment	Loss model parameter			Base flow model parameter		
	Initial (mm)	Constant (mmhr^{-1})	Impervious (%)	Initial discharge (m^{3}s^{-1})	Recession constant	Ratio to peak
Carawarra Creek	400.0	4.4	5%	0.00	0.90	0.05
Gulungul Creek	400.0	2.4	5%	0.00	0.90	0.01
Cullen Hill	400.0	3.0	5%	0.00	0.90	0.05

Calibration results are presented as graphs of the following:

- Fitted and observed hydrographs (recorded every 15 minutes) (*Figure 14*),
- Fitted and observed weekly flow volumes(ML) (*Figure 15*), and
- Fitted and observed cumulative flow volumes (ML) (*Figure 16*).

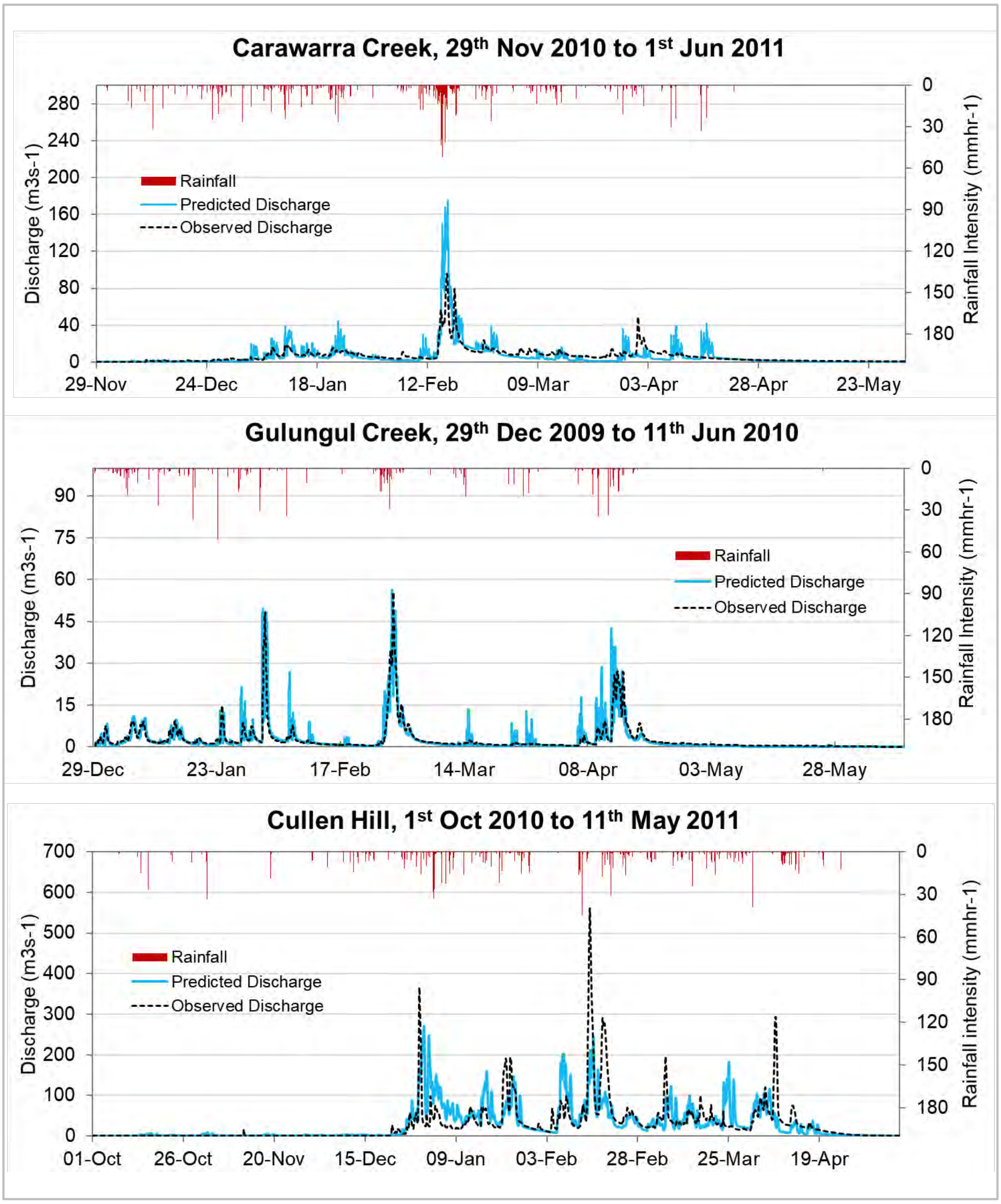


Figure 14. Graphs of fitted and observed hydrographs (15-minute intervals). Gulungul Creek data was provided by the Environmental Research Institute of the Supervising Scientist.

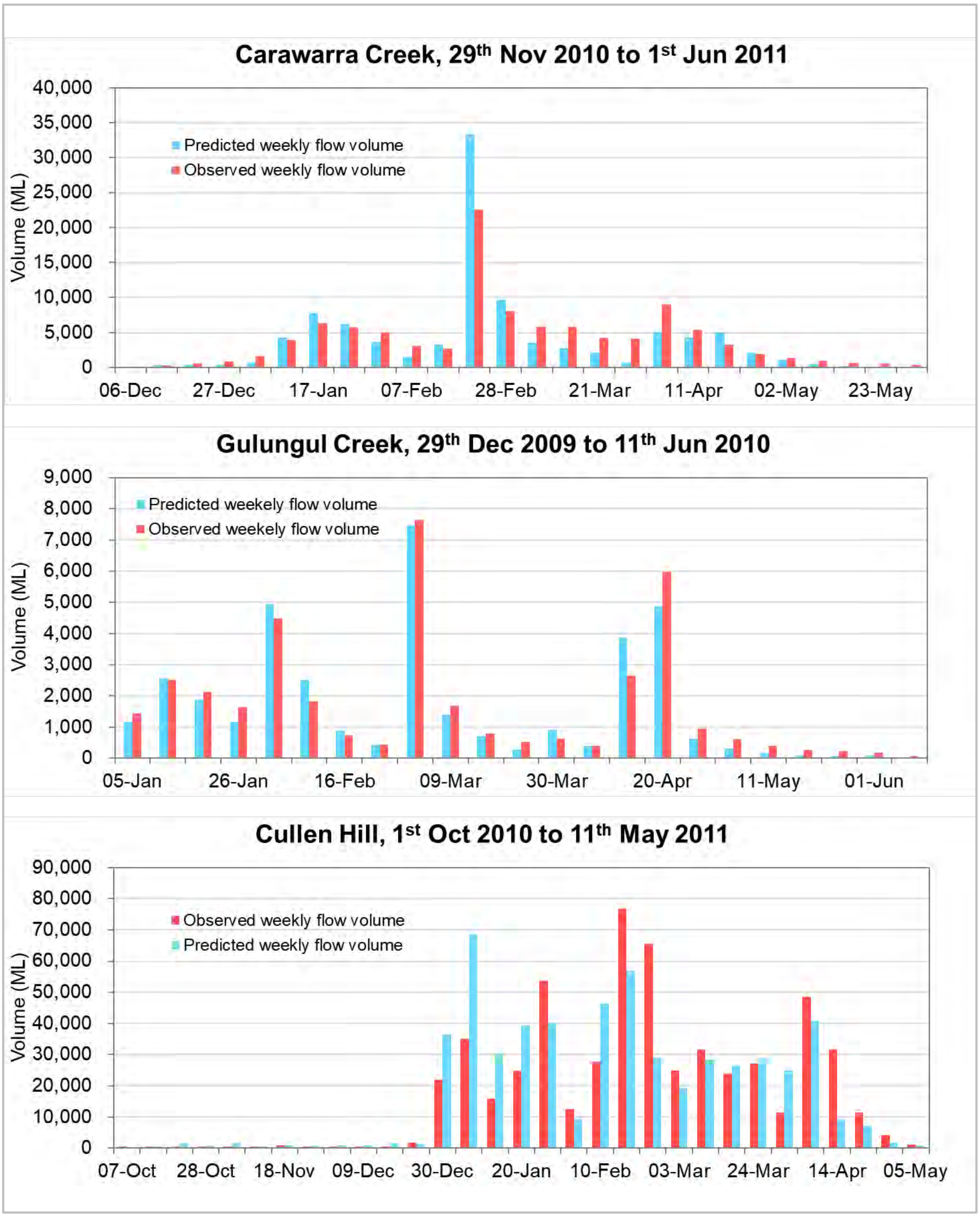


Figure 15. Fitted and observed weekly flow volumes(ML)Figure. Graphs of fitted and observed hydrographs (15-minute intervals).

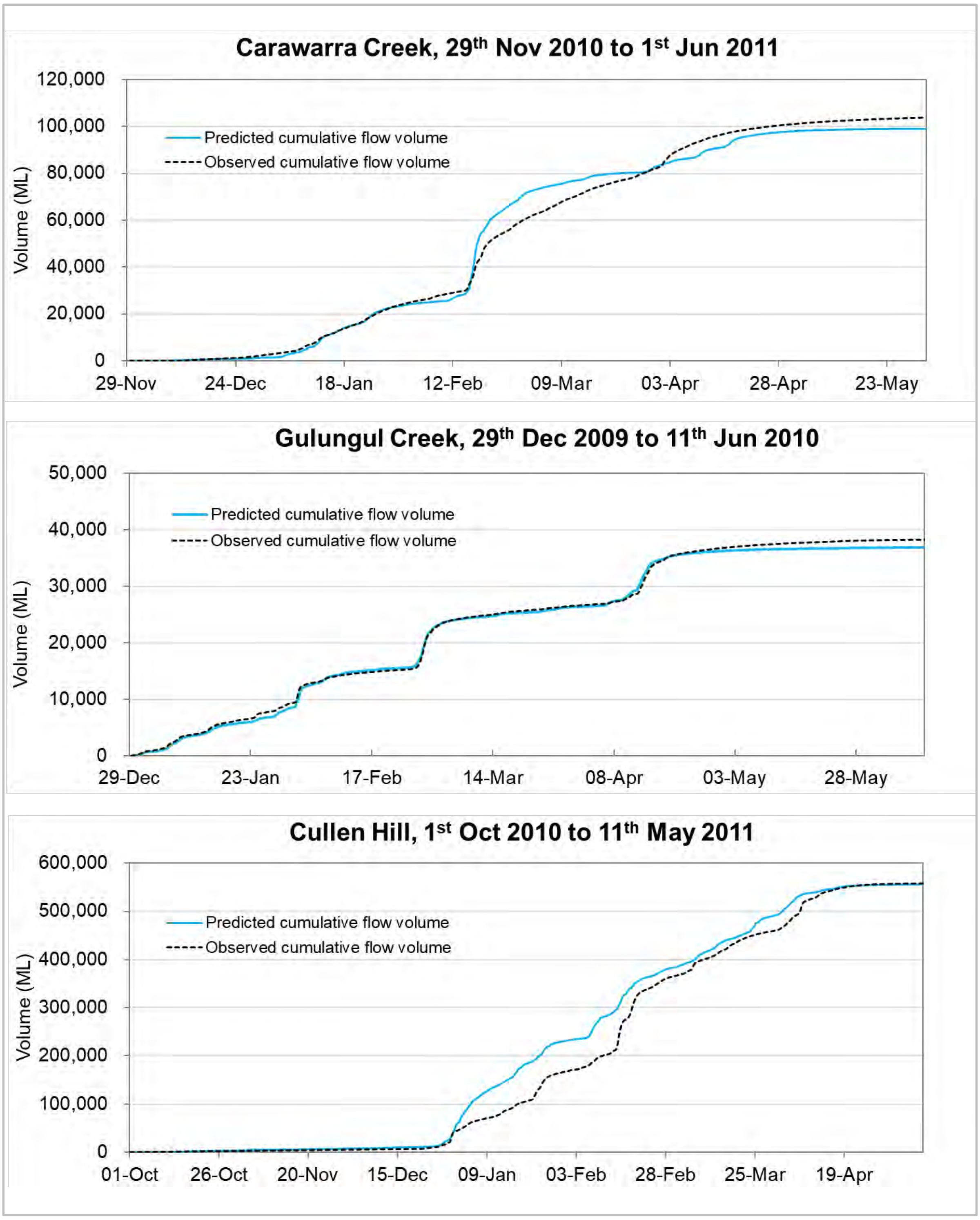


Figure 16. Fitted and observed cumulative flow volumes (ML).

4.2.4 Statistical analysis

Goodness of Fit was determined using Pearson's Correlation (R^2) and Nash-Sutcliffe model efficiency coefficient (NSE).

Pearson's R-Square (R^2) indicates the fit between model output and observed data with a coefficient of 0 indicating a poor fit and a coefficient of 1 indicating a perfect fit.

Nash-Sutcliffe model efficiency coefficient (NSE) is a measure of predictive power of a hydrological model varying from $-\infty$ to 1, with 1 indicating perfect accuracy. Satisfactory results are generally 0.5 or higher.

The values of R^2 and NSE for each calibrated catchment model are shown in *Table 6*.

Table 6. R^2 and NSE values for modelling results

Catchment	Statistic	Hydrograph	Weekly flow volume	Cumulative flow volume
Carawarra Creek	R^2	0.63	0.90	0.99
	NSE	-0.11	0.56	0.98
Gulungul Creek	R^2	0.69	0.95	1.00
	NSE	0.66	0.95	1.00
Cullen Hill	R^2	0.38	0.69	0.99
	NSE	0.35	0.68	0.98

4.2.5 Validation of calibrated parameter values

Although a model can be fitted to observed data via the calibration process, a single set of data may not be enough to prove the reliability of a model. The parameter values determined by fitting the model to a single set of data may be biased. To further prove the reliability of calibrated parameter values, a common practice is to re-run the model with calibrated parameter values and a second set of data as input. By comparing the new model output against observed data, the model can be validated.

In this study, the calibration of Gulungul Creek was validated as it has the best data set[4] and had the best calibration results. Wet Season rainfall and upstream discharge data from 2015 to 2016 were used to simulate discharge at the downstream site i.e. outlet of the catchment. The results showed that there were some differences in predicted and observed peak discharge values, but weekly flow volumes and cumulative flow volumes were well-predicted with good statistical comparison (*Table 7*) (*Figure 17*).

[4] Gulungul Creek data were provided by the Environmental Research Institute of the Supervising Scientist.

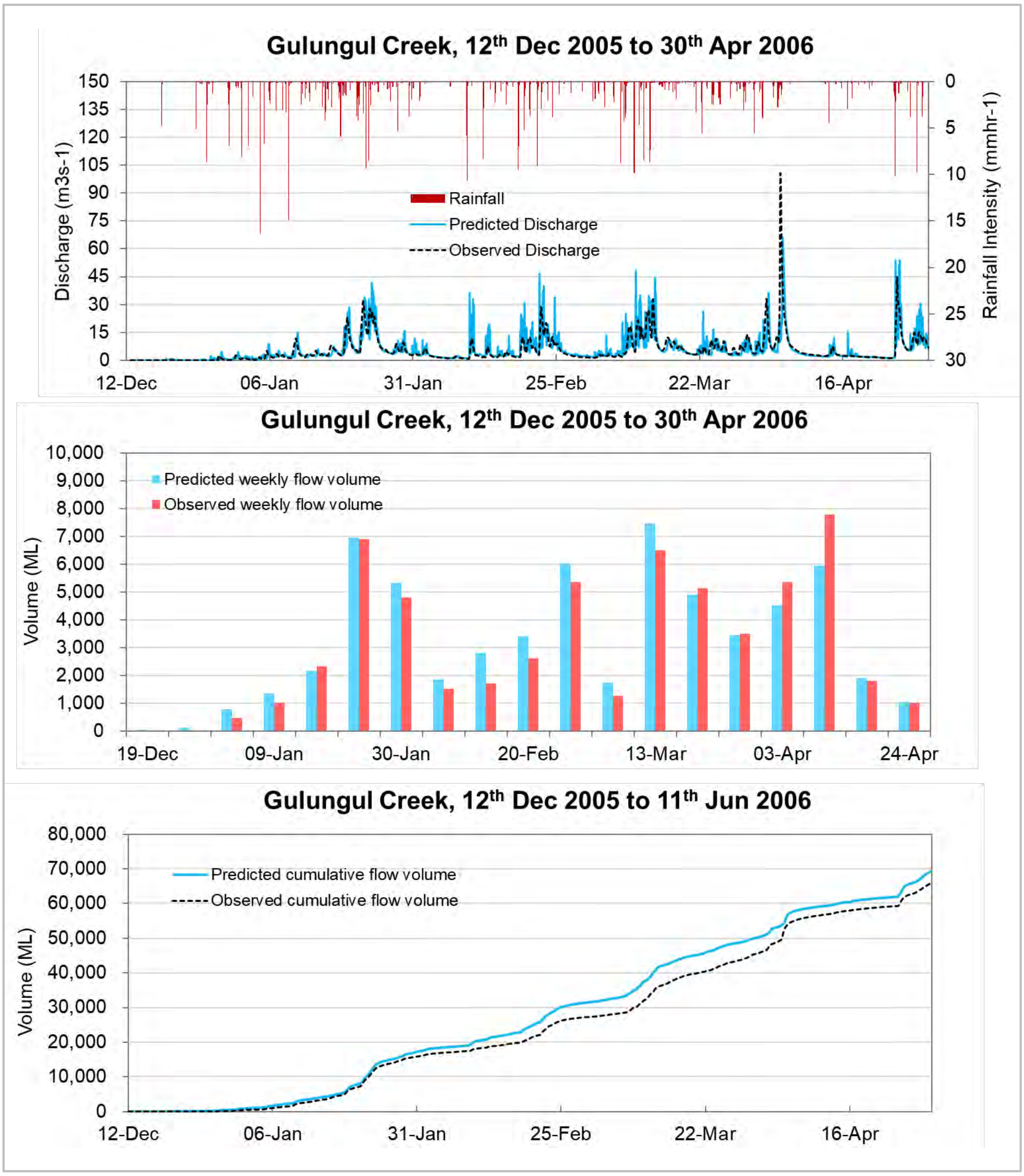


Figure 17. Comparison between fitted and observed for the 2015-2016 Wet Season at Gulungul Creek.

Table 7. R^2 and NSE values for modelling results

Catchment	Statistic	Hydrograph	Weekly flow volume	Cumulative flow volume
Gulungul Creek	R^2	0.49	0.93	1.00
	NSE	0.41	0.93	0.98

4.2.6 Discussion

It can be observed from *Figure 14, Figure 15, Figure 16*, that the modelling results generally fitted well with observed data. The timing of fitted hydrographs generally matched the observed hydrographs for all three sites, with some over-prediction and some under-prediction of event peak Q. All three models showed excellent fit for cumulative weekly flow volume and cumulative flow volume. Validation of Gulungul Creek confirm the calibrated parameter values (*Figure 17*).

Although the values of R^2 (0.63) for the predicted hydrograph for Carawarra Creek catchment indicates a satisfactory fit between observed data and predicted data, the low NSE value (-0.11) suggests relatively poor predictive value. This is caused by over-prediction of peak discharges in the predicted hydrograph (*Figure 14*). However, the durations of those over-estimated peak discharges are very short, and the excessive flow volume caused by these discharges are relatively low when considering cumulative flow volumes in total. This is supported by the statistics for the predicted weekly flow volumes: both R^2 (0.9) and NSE (0.56) values indicate a very good fit with observed data. For Gulungul Creek and Cullen Hill catchments, the performance of the two calibrated models is satisfactory in calculating both the instantaneous hydrograph and weekly flow volumes, which is reflected by the values of R^2 and NSE.

For all three calibration catchments, the values of R^2 and NSE for cumulative volumes are very close to 1.0, which indicates an almost perfect fit between observed and predicted data. This gives confidence that the calibrated model can accurately predict real-time total volume of flow over a long period, which is very important in water balance studies.

The fitted parameter values (*Table 5*) are similar across all three calibrated models, although the Gulungul Creek model used smaller ratio to peak (0.01) and continuing loss (2.4mmh^{-1}) parameters than the other two models. The reason was that Gulungul Creek had higher input discharges from upstream catchments, which caused the peak output discharges to be much larger than the peak discharges generated only from the rainfall runoff within the Gulungul Creek catchment itself. The Gulungul Creek model was calibrated to partial catchment flow only, and since these higher peak discharges were used by the model to calculate base flow, the ratio to peak parameter was reduced to avoid overestimation of base flow. A ratio to peak parameter of 0.05 (as determined from calibration of the Cullen Hill and Carawarra Creek catchments) was therefore deemed to be suitable for the mine site catchments. The values of constant loss were slightly different for the three models. Taking a conservative approach, a higher constant loss (4.4 mmhr^{-1}) was used for Carawarra Creek to make the predicted cumulative flow volume greater than the observed volume (*Figure 16*). Since the Carawarra Creek catchment is close to ML 31726, this constant loss (4.4 mmhr^{-1}) will be used for the proposed mine site catchments in the water balance study, to obtain conservative total flow volumes.

4.2.7 Conclusion

The calibration results show the fitted parameters produce very good results for predicted total volumes, demonstrating their suitability for water balance modelling. The selected parameter values that will be used for modelling the ML 31726 catchments are shown in *Table 8*.

With respect to the partial validation using the Carawarra Creek catchment, the fitted parameters over-predict the total volume slightly even though statistically the results are significantly similar. An over-prediction of volume is conservative for engineering design, but non-conservative for water balance as the available water may be less than the predicted volumes.

Table 8.Values of loss method parameters and base flow method parameters for mine catchment.

Parameter	Loss method parameter			Base flow method parameter		
	Initial (mm)	Constant ($mmhr^{-1}$)	Impervious (%)	Initial discharge (m^3s^{-1})	Recession constant	Ratio to peak
Value	400.0	4.4	5%	0.00	0.90	0.05

Appendix A. Site visit record

The coordinates in this Appendix are based on the projection: GDA 94/MGA zone 52.

<table>
<tr><td>

Appendix A 1

</td><td>

Site: 1

Easting (m), Northing (m):

693909.7, 8599243

Culvert number: 1

Water depth catchment inlet (m):

0.14

Description:

This comprises 3 RBC culverts draining toward West Arm. This view is from upstream with a road drain entering from the left-bank. At the time of inspection there was flow only in the left-bank culvert.

23 February 2018

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Appendix A 2

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Site: 1

Easting (m), Northing (m):

693910, 8599240

Culvert number: 1

Water depth catchment inlet (m):

0.14

Description:

Flow from Upstream and left bank road drain entering.

23 February 2018

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</table>

Appendix A 3 	**Site: 1** **Easting (m), Northing (m):** 693866.3, 8599288 **Culvert number:** 1 **Water depth catchment inlet (m):** 0.14 **Description:** Looking SE floodway culvert 1. **23 February 2018**
Appendix A 4 	**Site: 1** **Easting (m), Northing (m):** 693908.6, 8599260 **Culvert number:** 1 **Water depth catchment inlet (m):** 0.13 **Description:** Downstream view at culvert 1. **23 February 2018**

<table>
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Appendix A 5

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Site: 2

Easting (m), Northing (m):
692948.9, 8600106

Culvert number:
2

Water depth catchment inlet (m):
0.5

Description:
Upstream culvert inlet road drain 2 box culvert.

23 February 2018

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Appendix A 6

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Site: 2

Easting (m), Northing (m):
692917.4, 8600126

Culvert number:
2

Water depth catchment inlet (m):
0.5

Description:
Culvert 2 floodway looking E-SE.

23 February 2018

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<tr>
<td>Appendix A 7
</td>
<td>Site: 2
Easting (m), Northing (m):
692951.3, 8600122

Culvert number:
2
Water depth catchment inlet (m):
0.5

Description:
Downstream culvert 2 comprises 2 RBC culverts draining toward West Arm. This view is from downstream with scour control in place below the outlet. A plunge pool has developed.

23 February 2018</td>
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<td>Appendix A 8
</td>
<td>Site: 3
Easting (m), Northing (m):
694684.3, 8597461

Culvert number:
3

Water depth catchment inlet (m):
0

Description:
Culvert 3 Upstream view flow N to S.

23 February 2018</td>
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<td>Appendix A 9
</td>
<td>Site: 4
Easting (m), Northing (m):
695009.8, 8596831

Culvert number:
4

Water depth catchment inlet (m):
0

Description:
Culvert 4 Upstream view flow N to S.

23 February 2018</td>
</tr>
<tr>
<td>Appendix A 10
</td>
<td>Site: 5
Easting (m), Northing (m):
696418.4, 8595018

Culvert number:
5

Water depth catchment inlet (m):
0.16

Description:
Floodway NE culvert 5.

23 February 2018</td>
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Appendix A 11

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Site: 5

Easting (m), Northing (m):

696416.2, 8595007

Culvert number:

5

Water depth catchment inlet (m):

0.16

Description:

Culvert 5 Upstream view flow SE to NW.

23 February 2018

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Appendix A 12

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Site: 7

Easting (m), Northing (m):

693267.5, 8599486

Description:

Onsite track drainage pipes general drainage toward northwest.

23 February 2018

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Appendix A 13 	**Site: 6** **Easting (m), Northing (m):** 694471.6, 8593530.0 **Description:** Old mine pit in the north of the dam. **23 February 2018**
Appendix A 14 	**Site: 8** **Easting (m), Northing (m):** 695218, 8594870 **Elevation (mAHD):** ≈31 **Description:** Dam outlet spillway **23 February 2018**

Appendix A 15	**Site: 9**
	**Easting (m), Northing (m):** 695319.8, 8594687 **Description:** Water body at toe of downstream dam wall. **24 April 2018**

Appendix A 16	**Site: 10**
	**Easting (m), Northing (m):** 695319.8, 8594687 **Description:** Gully forming near western end of dam wall along the line where the dam abuts natural terrain. The gully flows to the footwall pond. The water bottle is a scale reference in the picture. **24 April 2018**

Appendix A 17 	**Site: 11** **Easting (m), Northing (m):** 695615.3, 8594825 **Elevation (mAHD):** ≈32 **Description:** Gullying downstream side dam wall some drainage from road. Dam could over flow here looks like low point in the wall. **23 February 2018**

Appendix B. Calibrated models

B1. The HEC-HMS model

HEC-HMS (US Army Corps of Engineers) will be used to simulate rainfall-runoff processes in the catchments intersected by ML 31726 and the existing dam catchment in later Projects. The simulated hydrographs from HEC-HMS can be used to determine the flows from each catchment, changes in water storage in the existing dam, and how these may change with mining infrastructure present.

A HEC-HMS model consists of four major components: Basin model, Meteorological model, Control specification and Time-series data, which are described in *Table B1*.

Table B1-1 Description of major components of HEC-HMS models

Component	Description
Basin Model	The physical catchment is represented by the basin model. Sub-components of the catchment can be described by adding hydrologic elements including sub-catchment, reach, reservoir, junction, diversion, source, and sink. Each hydrologic element uses a mathematical model to describe the physical process involved in generating or transporting runoff.
Time-series Data	The observed time-series data is managed in the data manager in HEC-HMS. The data can be a precipitation graph, a water elevation graph and other kinds of time-series data such as evaporation. These stored data can then be applied to hydrologic elements or meteorological models as boundary conditions.
Meteorological Model	The meteorological model is used to define meteorological boundary conditions for sub-catchments. It links the input data to the Basin model and defines meteorological processes such as precipitation (including non-uniform spatial rainfall distributions), evaporation and solar radiation.
Control Specification	The control specification defines the start and end date and time of a simulation run and the computation time interval. If the specified computation time interval differs from that of the input data (precipitation and/or discharge data), the data will be linearly interpolated to fit the control specifications.

In this study, basin models were created for each calibration sub-catchment using field inspection data and the NTG 2m-DEM. The Time-series Data component, Meteorologic Model and Control Specification used available observed precipitation data from BOM sites and the NTG water portal. Initial values of model parameters were either set as the default values or estimated based on experience.

The Loss method and Base Flow method parameter values (*Table B1.1*) were iteratively fit through trial and error until the observed and predicted hydrographs reached an acceptable similarity. Between calibration runs, the goodness of fit between the predicted and observed hydrographs was visually evaluated based on similarity between:

1. Peak discharges,
2. Basic shape of the hydrograph, and
3. Total flow volumes (the most important for water balance assessment).

A number of additional parameters were also required to run the Transform method and Routing method (see *Table B1-2*.) mathematical components of the HEC-HMS model. These parameters did not require iterative fitting during calibration as they were calculated based on the catchment characteristics:

$$\text{Time of concentration - } T_c \text{ (hr)} = \frac{L}{A^{0.1}S^{0.2}} \qquad (1)$$

$$\text{Storage coefficient-R (hr)} = 3.74L^{0.342}S^{-0.79} \qquad (2)$$

$$\text{Lag time (hr)} = 166.7D \qquad (3)$$

Where

A = sub-catchment area (km^2);

L = length of the longest channel in the sub-catchment (km);

S = slope measured from 10% to 85% of the channel length from downstream to upstream (m/km); and

D = length of the channel segment represented by the HEC-HMS reach element

Table B1-2. The mathematic methods utilized in HEC-HMS models and relevant parameters

Method	Explanation	Parameter	Unit
Loss method "Initial and constant"	The method specifies the amount of initial infiltration that will be fulfilled by incoming precipitation before surface runoff begins and the continuing infiltration after the initial infiltration is satisfied.	Initial loss	mm
		Constant loss	$mmhr^{-1}$
		Impervious	%
Transform method "Clark unit hydrograph"	The method performs the calculation of runoff from precipitation. It transfers precipitation into a hydrograph based on a time-area curve built in the software and a linear reservoir algorithm that accounts for storage attenuation effects.	Time of concentration	hr
		Storage coefficient	hr
Base flow method "Recession"	This method performs the calculation of subsurface flow within a catchment. It approximates the typical behavior observed in a catchment when channel flow recedes exponentially after a storm event.	Initial discharge	m^3s^{-1}
		Recession constant	N/A
		Ratio to peak	N/A
Routing Method "Lag"	The method calculates the transport of flow in river channels. The lag time represents the travel time of flood waves in a reach element.	Lag time	min

B2. HEC-HMS Calibration model of Cullen Hill catchment

Basin model

The catchment was divided into 8 sub-catchments based on catchment morphology and streamlines. The basin model was then developed based on the these defined sub-catchments. The schematic basin model is shown in *Figure B2-1*. The map of Cullen Hill catchment *(Figure B2-2)* shows the sub-catchment boundaries and the location of the gauging stations.

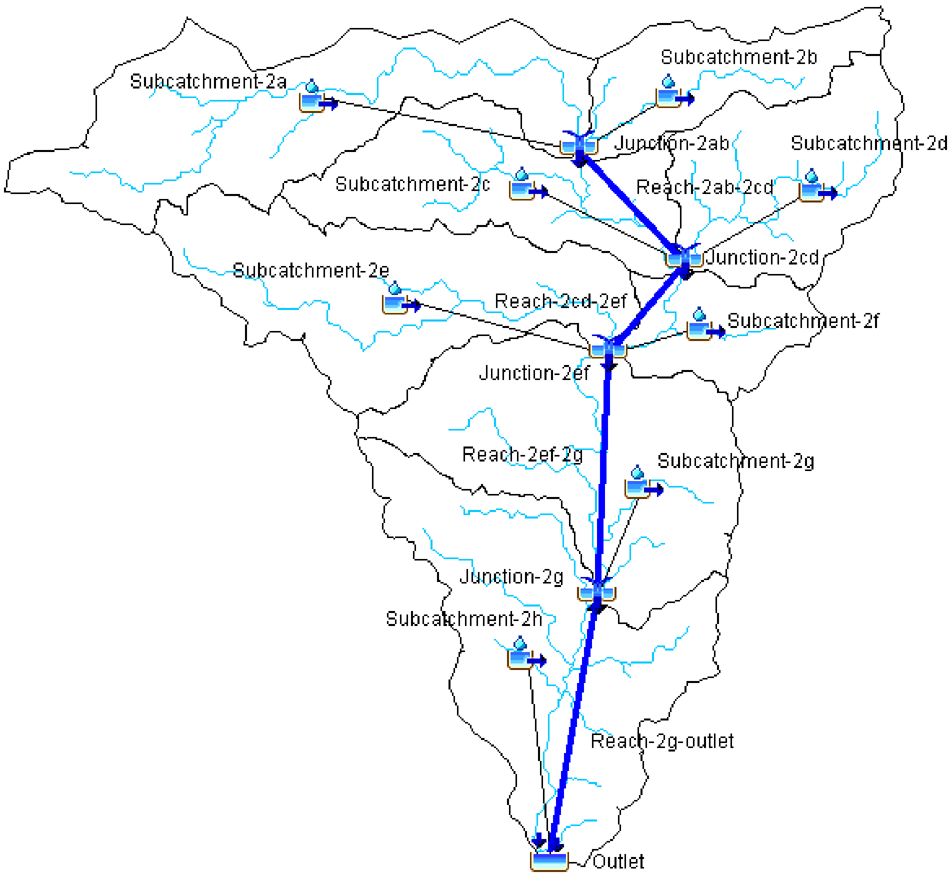

Figure B2-1. Schematic basin model of Cullen Hill catchment.

Time-series Data

The hourly rainfall intensity data from 1st Oct-2010 00:00 to 11th May 2011 00:00 measured at G8140060 gauging station were input as precipitation data for the model. The observed discharge time-series for the same period recorded at G8140060 gauging station were applied to the catchment outlet as observed. Predicted discharges were compared to predicted discharges until a good fit was obtained.

Meteorologic Model

Precipitation time-series data were used as the meteorologic model input and the "specified hyetograph" method was used. Under this method, precipitation observed by a single rain gauge (G8140060) was uniformly distributed to each sub-catchment during simulations.

Control Specification

The time window of the simulation was set to be from 1stOct-2010 00:00 to 11th May 2011 00:00 which is same with the period of input precipitation time series. The calculation time interval was set as 15 minutes which normally gives a reasonable simulation result.

Calibrated parameter values

The calibrated parameter set, including fitted and fixed parameters, is shown in *Table B2-1*.

Table B2-1. Calibrated parameter set for HEC-HMS model of Cullen Hill catchment.

Sub-basins	Area (km^2)	Loss method parameters			Transform method parameters		Base flow method parameters		
		Initial (mm)	Constant (mmhr-1)	Impervious (%)	Time of concentration (hr)	Storage coefficient (hr)	Initial discharge (m^3s^{-1})	Recession constant	Ratio to Peak
Subcatchment-2a	76.1	400	3.0	5.0	12.55	5.10	0.00	0.90	0.05
Subcatchment-2b	28.9	400	3.0	5.0	6.80	3.38	0.00	0.90	0.05
Subcatchment-2c	48.9	400	3.0	5.0	9.33	5.12	0.00	0.90	0.05
Subcatchment-2d	45.6	400	3.0	5.0	7.11	3.75	0.00	0.90	0.05
Subcatchment-2e	69.7	400	3.0	5.0	11.17	5.53	0.00	0.90	0.05
Subcatchment-2f	21.8	400	3.0	5.0	4.54	3.44	0.00	0.90	0.05
Subcatchment-2g	76.4	400	3.0	5.0	7.48	8.16	0.00	0.90	0.05
Subcatchment-2h	82.8	400	3.0	5.0	5.46	7.70	0.00	0.90	0.05

Reaches	Routing method parameter
	Lag time (min)
Reach-2ab-2cd	1800
Reach-2cd-2ef	650
Reach-2ef-2g	650
Reach-2g-outlet	650

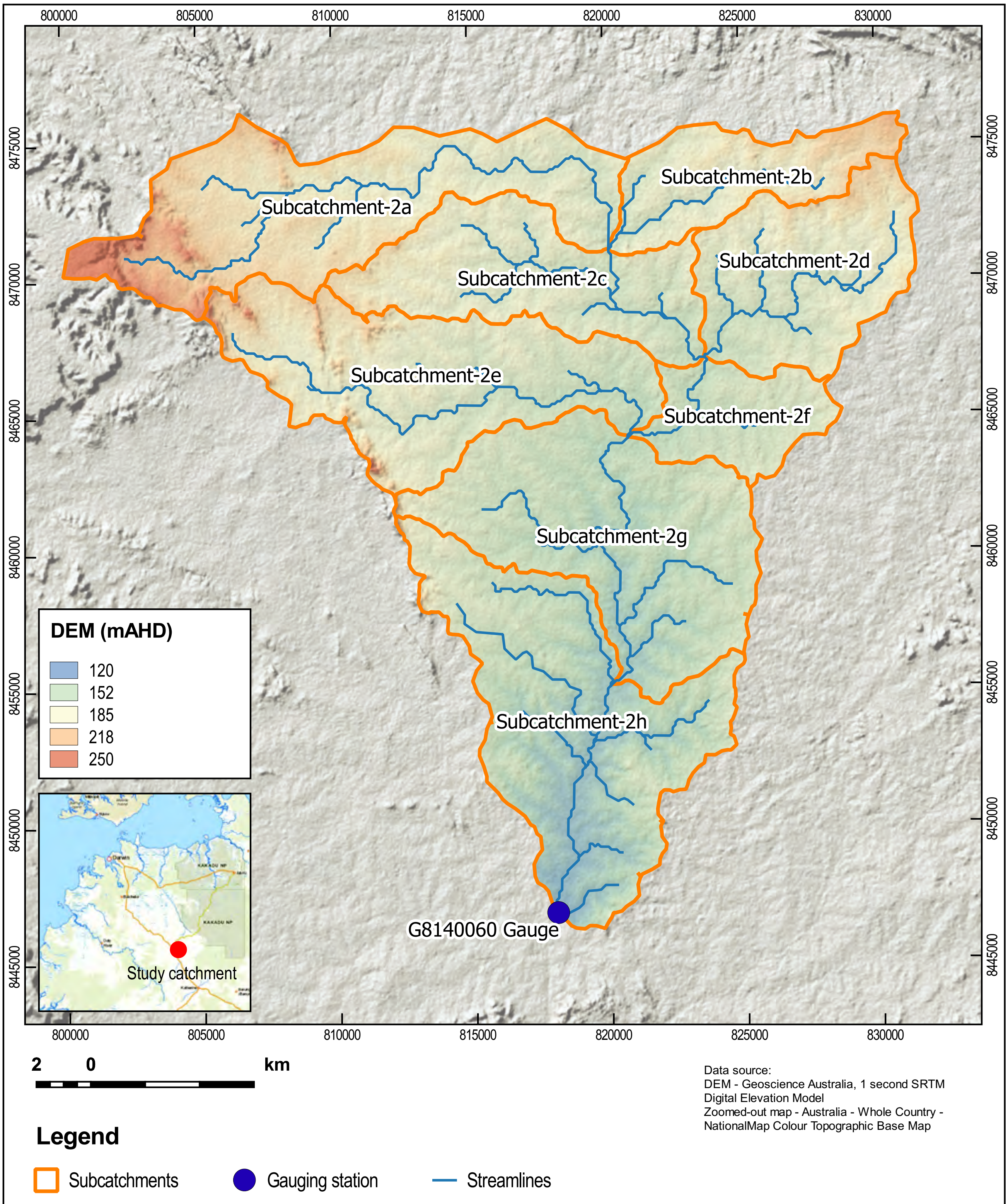

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Figure B2-2. Cullen Hill study catchment

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

B3. HEC-HMS Calibration model of Carawarra Creek catchment

Basin model

The catchment was divided into 6 sub-catchments based on catchment morphology and streamlines. The basin model was then developed based on the sub-catchments defined. The schematic basin model is shown in *Figure B3-1*. The map of Carawarra Creek which shows the sub-catchment boundaries and the location of gauging station is shown in *Figure B3-2*.

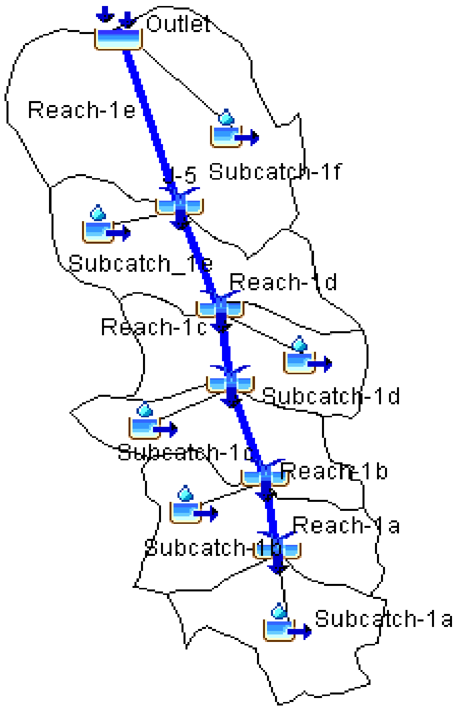

Figure B3-1. Schematic basin model of Carawarra Creek catchment.

Time-series Data

The hourly precipitation data from 29th Nov 2010 17:00 to 1st Jun 2011 00:00 measured by the G8150016 gauging station were used as input to the model. The observed discharge time-series for the same period recorded by the G8150096 gauging station were applied to the catchment outlet as observed data for comparison with fitted data.

Meteorologic Model

For this model, only precipitation time-series data was used as Meteorologic model input and the "specified hyetograph" method were used. Under this method, precipitation observed by a single rain gauge (G8150096) was uniformly distributed to each sub-catchment during simulations.

Control Specification

The time window of the simulation was set to be from 29th Nov 2010 17:00 to 1st Jun 2011 00:00 which is same with the period of input precipitation time series. The calculation time interval was set as 15 minutes which normally gives a reasonable simulation result.

Calibrated parameter values

The calibrated parameter set, including fitted and fixed parameters, is shown in *Table B3-1*

Table B3-1. Calibrated parameter set for the HEC-HMS model of Carawarra Creek catchment

Sub-basins	Area (km^2)	Loss method parameters			Transform method parameters		Base flow method parameters		
		Initial (mm)	Constant ($mmhr^{-1}$)	Impervious (%)	Time of concentration (hr)	Storage coefficient (hr)	Initial discharge (m^3s^{-1})	Recession constant	Ratio to Peak
Subcatchment-1a	6.08	400	4.4	5.0	1.64	2.02	0.00	0.90	0.05
Subcatchment-1b	7.62	400	4.4	5.0	1.51	2.17	0.00	0.90	0.05
Subcatchment-1c	6.45	400	4.4	5.0	1.54	5.19	0.00	0.90	0.05
Subcatchment-1d	7.00	400	4.4	5.0	1.59	4.98	0.00	0.90	0.05
Subcatchment-1e	8.20	400	4.4	5.0	1.69	1.19	0.00	0.90	0.05
Subcatchment-1f	16.23	400	4.4	5.0	2.19	3.01	0.00	0.90	0.05

Reaches	Routing method parameter
	Lag time (min)
Reach-1a	259
Reach-1b	315
Reach-1c	279
Reach-1d	236
Reach-1e	862

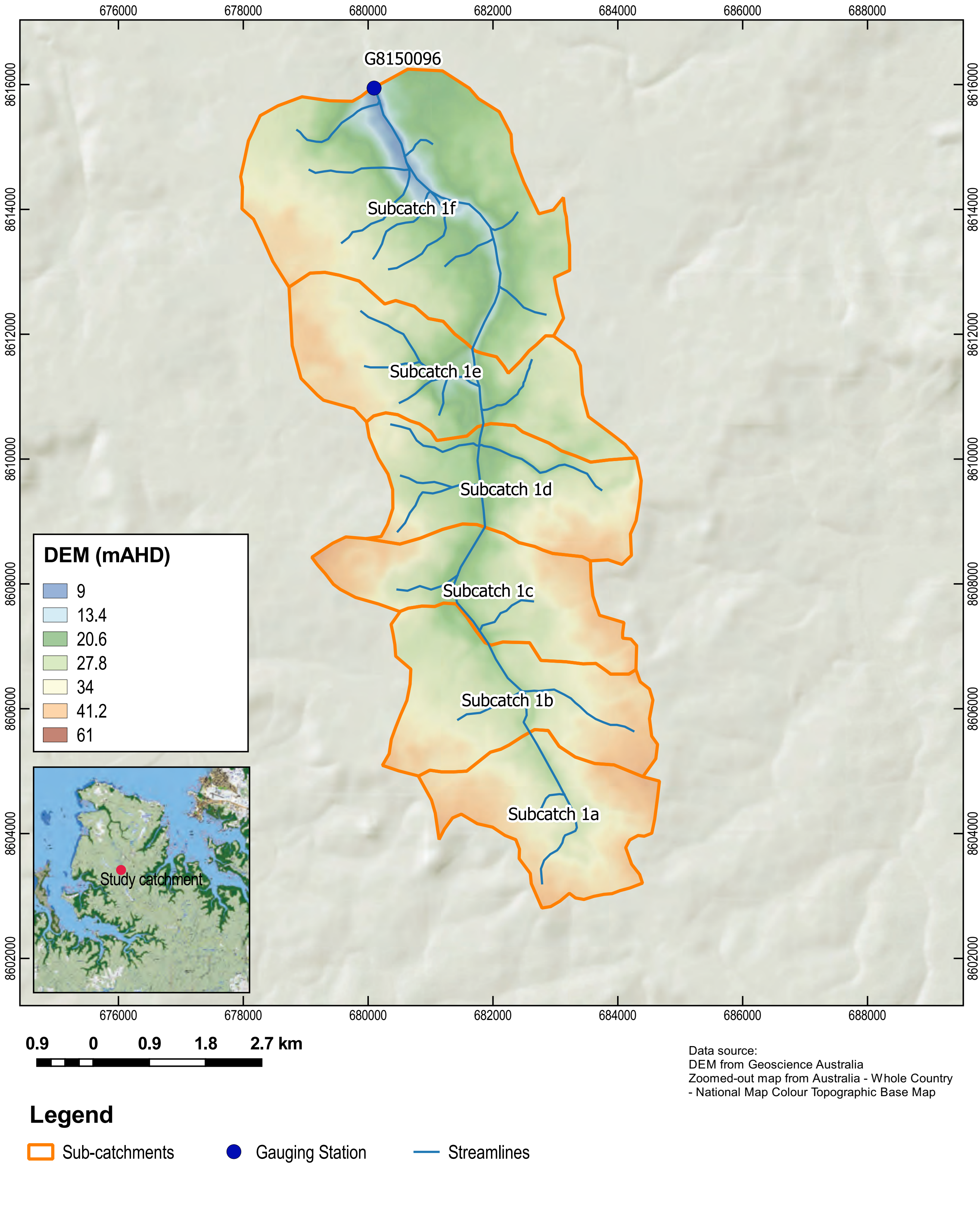

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Figure B3-2. The map of Carawarra Creek catchment

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

B4. HEC-HMS Calibration model of Gulungul Creek catchment

Basin model

The catchment was divided into 13 sub-catchments based on catchment morphology and streamlines. The basin model was then developed based on the sub-catchments defined. The schematic basin model is shown in *Figure B4-1*. The map of Gulungul Creek which shows the sub-catchment boundaries and the location of gauging station is shown in *Figure B4-2*.

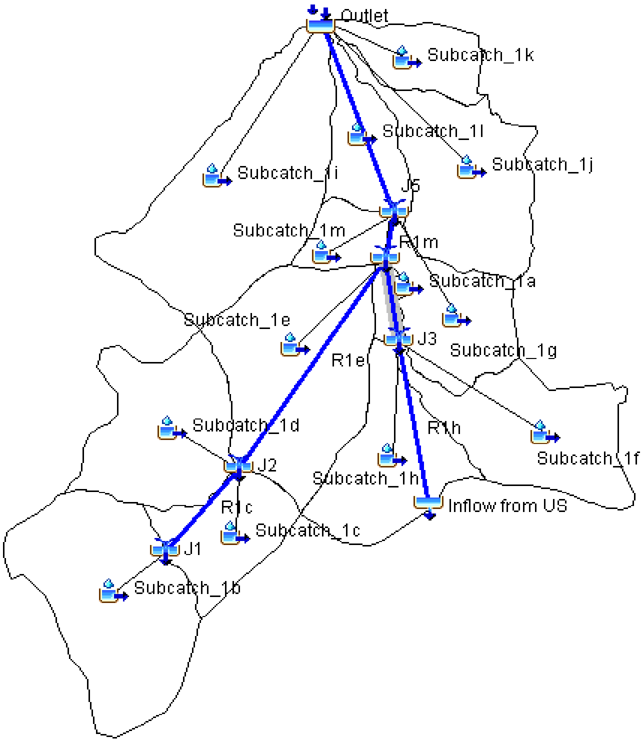

Figure B4-1. Schematic basin model of Gulungul Creek catchment.

Time-series Data

The hourly precipitation data from 29th Dec 2009 11:00 to 11th June 2010 19:00 measured by the Gulungul Creek downstream gauging station and upstream gauging station were input into the model as precipitation input to the model. The observed discharge time-series for the same period recorded by the Gulungul creek upstream gauge were applied to the upstream inlet of the catchment as input hydrograph to the model. The observed discharge time-series for the same period recorded by the Gulungul creek downstream gauging station were applied to the catchment outlet as observed data for comparison with the fitted hydrograph.

Meteorologic Model

For this model, only precipitation time-series data were used as Meteorologic model input and the "gauge weights" method was used. Under this method, the observed precipitation data from the two gauging stations (downstream and upstream) were distributed to each sub-catchment according to the weights set to each gauge. In this study, the weights were determined based on the distance from each gauge to the catchment centroid to represent their effects to the catchment.

Control Specification

The time window of the simulation was set to be from 29th Dec 2009 11:00 to 11th June 2010 19:00 which is the period of the input precipitation time series. The calculation time interval was set as 15 minutes which normally gives a reasonable simulation result.

Calibrated parameter values

The calibrated parameter set, including fitted and fixed parameters, is shown in *Table B4-1.*

Table B4-1 Calibrated parameter set for Gulungul Creek catchment.

Subcatchment	Area (km^2)	Loss method parameter			Transform method parameter		Base flow method parameters		
		Initial (mm)	Constant (mmhr-1)	Impervious (%)	Time of concentration (hr)	Storage coefficient (hr)	Initial discharge (m^3s^{-1})	Recession constant	Ratio to Peak
Subcatchment-1a	0.25	400	2.4	5.0	0.76	1.35	0.00	0.90	0.01
Subcatchment-1b	3.56	400	2.4	5.0	0.72	0.54	0.00	0.90	0.01
Subcatchment-1c	1.41	400	2.4	5.0	0.72	0.58	0.00	0.90	0.01
Subcatchment-1d	2.43	400	2.4	5.0	0.89	0.49	0.00	0.90	0.01
Subcatchment-1e	3.55	400	2.4	5.0	1.59	1.16	0.00	0.90	0.01
Subcatchment-1f	2.41	400	2.4	5.0	1.56	0.78	0.00	0.90	0.01
Subcatchment-1g	1.65	400	2.4	5.0	1.06	1.34	0.00	0.90	0.01
Subcatchment-1h	2.07	400	2.4	5.0	1.20	1.81	0.00	0.90	0.01
Subcatchment-1i	4.29	400	2.4	5.0	1.93	1.54	0.00	0.90	0.01
Subcatchment-1j	2.65	400	2.4	5.0	1.50	0.92	0.00	0.90	0.01
Subcatchment-1k	1.00	400	2.4	5.0	1.04	0.86	0.00	0.90	0.01
Subcathcment-1l	1.17	400.0	2.4	5.0	1.27	0.94	0.00	0.90	0.01
Subcatchment-1m	0.60	400.0	2.4	5.0	0.37	0.41	0.00	0.90	0.01

Reaches	Routing method parameter
	Lag time (min)
Reach-1a	146
Reach-1c	210
Reach-1e	446
Reach-1h	279
Reach-1l	335
Reach-1m	101
Reach-J6	27
Reach-J7	23

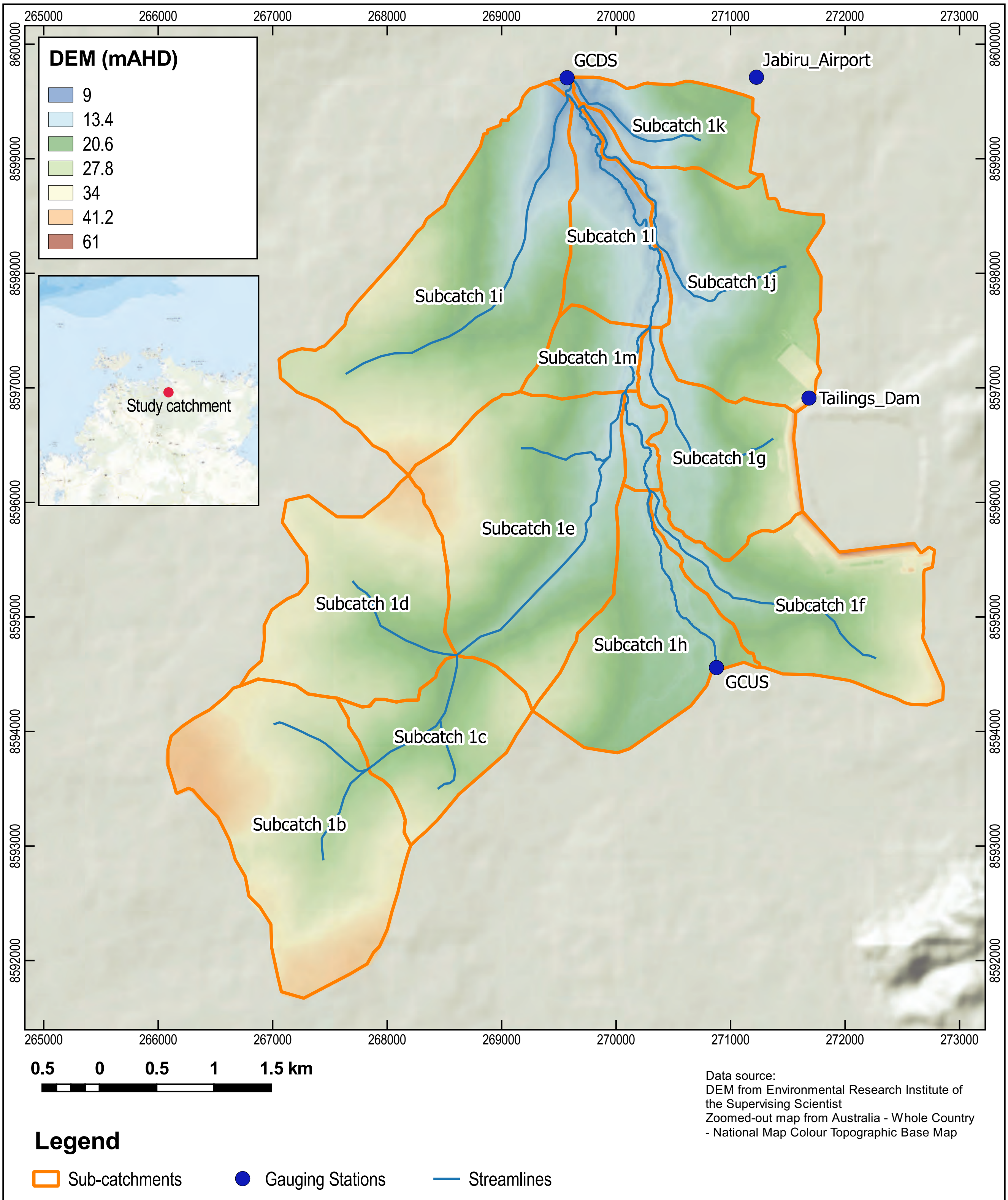

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Figure B4-2. The map of Gulungul Creek catchment

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 53

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This study, report and analyses have been based on the information available to and the assumptions made by EnviroConsult Australia Pty Ltd at the time of preparation. EnviroConsult Australia Pty Ltd does not take responsibility for errors and omissions in the conclusions, report and maps produced by this study due to incorrect information or any error in, or omission in connection with assumptions. No chemical analysis, groundwater hydrology, water quality or contaminant export studies of any kind were conducted. Any comment made in the report with respect to the above are speculative based on the surface hydrology analysis and should not be relied upon as fact.

Description	Author	Reviewer	Approved for Issue		
			Name	Signature	Date
Revision 0	Ken Evans Mike Liu Rojita Thapa	Ken Evans	Ken Evans	KG Evans	01/05/2018
Revision 1	Ken Evans Mike Liu	Ken Evans Mike Liu Aiden Campbell	Ken Evans	KG Evans	16/05/2018
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Core Exploration Ltd, Cox Peninsula

Project 2: Mining Lease 31726 and Observation Hill Dam Water Balance

Report ECA-HA-0004-02

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Project 2: Mining Lease 31726 and Observation Hill Dam Water Balance

Report ECA-HA-0004-02

Aug 2018

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EnviroConsult Australia Pty Ltd

Executive Summary

This report addresses the following:

1. The application of a calibrated hydrology model to compare the pre-mining and post-mining water balance for Core Exploration Ltd ML(A) 31726 site,
2. Observation Hill dam yield analysis, and
3. The assessment of alternative water storage facilities to secure mining water use requirements.

Pre- and post-mining water balance

The pre-mining baseline water balance was derived using HEC-HMS simulating a low, average and high rainfall year based on 15-min rainfall collected from Winnellie NT.

The Core mining lease straddles the Darwin Harbour and Boyne Harbour catchment boundary. Two catchments were defined for Darwin Harbour (catchments 2 and 5) and annual catchment outflows for the low, average and high years were 8725ML, 16560ML, and 28482ML respectively. Two catchments were defined for Bynoe Harbour (catchments 1 and 4) and combined annual catchment outflows for the low, average and high years were 13743ML, 25577ML, and 42702ML respectively.

Catchments 2 and 5 are where mine infrastructure will be constructed and will remain after closure. This infrastructure comprises waste rock dump, topsoil store, pit, bund, ponds and run-of-mine pad. This may also include a water storage dam to meet water requirements.

For Darwin harbour catchment with infrastructure only, reduction in stream flow at catchment outlet were about 7.2% of the pre-mine catchment outflow and most of this reduction was due to the retention of water in and evaporation from the sediment ponds and pit.

When the dam is included in catchment 5, reduction to catchment outflow due the dam and the infrastructure is about 8.8% of the pre-mine outflow for the average rainfall year, 8.5% for the high rainfall years and about 11.7% for the low rainfall year. So, for an average year the dam itself, is responsible for a reduction of only 1.6%. The increases in reduction in stream flow are due to evaporation of water from storage in the dam. These higher reductions can be reduced post-mining by lowering the dam spillway level.

Observation Hill dam yield analysis

Simulations indicated the mine-site water usage and evaporation and/or seepage losses deplete the Observation Hill Dam (OHD) water volume to different levels and at different rates depending on the pump rate scenarios applied. The storage capacity for the current dam at a spillway level of 30mAHD is ≈364 ML which is insufficient to meet the pump rate scenarios provided by Core. Therefore, additional water may be required for mining and 2 wall-lift scenarios which would increase the volume of OHD were simulated to address this requirement. One with a spillway level of 31.5mAHD giving a capacity of ≈628ML, and a second with a spillway level of 33.6mAHD with a capacity of ≈1191ML.

The simulations for different pump rates and wall lift scenarios indicated that for constant pump rates of 2.02 MLd^{-1} and 1.2 MLd^{-1}, for a 5-year scenario, there was a water deficit for the low rainfall year for each of the lift scenarios. Overall, simulations indicate variable deficits of water for mine applications ranging from of 18ML to 366ML. Economies of water usage, such as no dust suppression in the Wet Season, de-watering of the pit allow the dam to re-fill to capacity may address the deficit. These economies were assessed for an average rainfall year using variable pump rates of 1.07 MLd^{-1} in wet, 2.02 MLd^{-1} in dry and 0.64 MLd^{-1} in wet, 1.2 MLd^{-1} in dry the only deficit occurred for the higher rates with the 30m spillway level scenario. Other spillway levels supported requirements.

Additionally, the impact of pumping and wall lift on downstream flows were investigated. For the spillway out flow only, the total volume reduced by 36.8% when the pumping rate is 2.02 MLd^{-1} through an average rainfall year. For the wall lift scenarios, the reduction in spillway flow volume was up to 100% for 33.6 mAHD spillway scenario. However, as OHD is a small part of a much larger catchment, the maximum reduction in flow volume in the watershed where it enters the first tidal Boyne Harbour tributary downstream was considered; and the reduction in flow volume at this point, due to ODH, was 2.6%.

Alternative water storage facilities

Apart from a wall lift or reduction in water usage, an alternative to achieve sufficient water storage may be the construction of a second smaller dam.

Two sites were assessed, 1 in catchment 5 and 1 in catchment 4. Both sites have similar catchment sizes as the OHD. The simulations show that either site is suitable for ancillary water storage for the worst-case scenario of an annual average deficit of 258ML.

The impact of dam, with a spillway level of 16.93mAHD in catchment 5, on downstream flows were assessed at 7 locations. The impact of the dam on downstream flows reduces progressively downstream from the catchment 5 outlet to the outlet of the watershed draining to the Darwin Harbour. When the impact of the dam, without mine infrastructure, is simulated, the reduction in total flows due to the dam are 8.7% at the outlet of catchment 5 and 0.9% at the watershed outlet. When mine infrastructure is considered, the reductions are 37% and 0.9%. The 37% reduction in total flow is mainly a result of changes in stream lines due to the mine infrastructure.

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1 Introduction

The Core Exploration Ltd (Core) Lithium resource (Grants Project) is located at 695000m East, 8596000m North, GDA94/MGA52, near Berry Springs, Northern Territory. This report focuses on ML(A) 31726 but also includes the current Observation Hill Dam (OHD) catchment which, although not located on the ML(A) 31726, is to be used as a water supply (*Figure 1*).

EcOz Environmental Consultants commissioned EnviroConsult Australia Pty Ltd to undertake three projects relating the surface water hydrology of Grants Project:

- Project 1: Description of hydrological conditions of site and calibration of hydrological model,
- Project 2: Application of hydrological model to complete a hydrological assessment and water balance, and
- Project 3: Complete inundation modelling.

Project 1 has been completed by EnviroConsult Australia Pty Ltd. This report describes Project 2

1.1 Methodology

The steps undertaken to perform Project 2 were:

1. Delineate the sub-catchments for Darwin and Bynoe harbours using the Quantum Geographic Information System,
2. Apply the calibrated hydrology model, HEC-HMS (Project 1) to the mining lease and environs to determine rainfall runoff volumes for the ML(A) 31726 for the pre-mining topographic condition for a low, average and high rainfall year,
3. For the same rainfall scenarios, apply HEC-HMS to the post-mining topography i.e. including the pit, waste rock dump, bund/haul road and run-of-mine pad and possible alternate dam,
4. Use HEC-HMS, to assess the adequacy of water storage in the Observation Hill dam for mine applications, and
5. Identify 2 sites that may be suitable for dam construction to meet water requirement shortfalls.

1.2 Water Balance Equation

The water balance equation used is:

$$f(P) \propto f(R)\, f(L),$$

where P = precipitation, R = stream flow, and L = catchment storage and losses.

HEC-HEM was used to determine the water balance for each of the catchments intersected by ML31726 as described in the project 1 report.

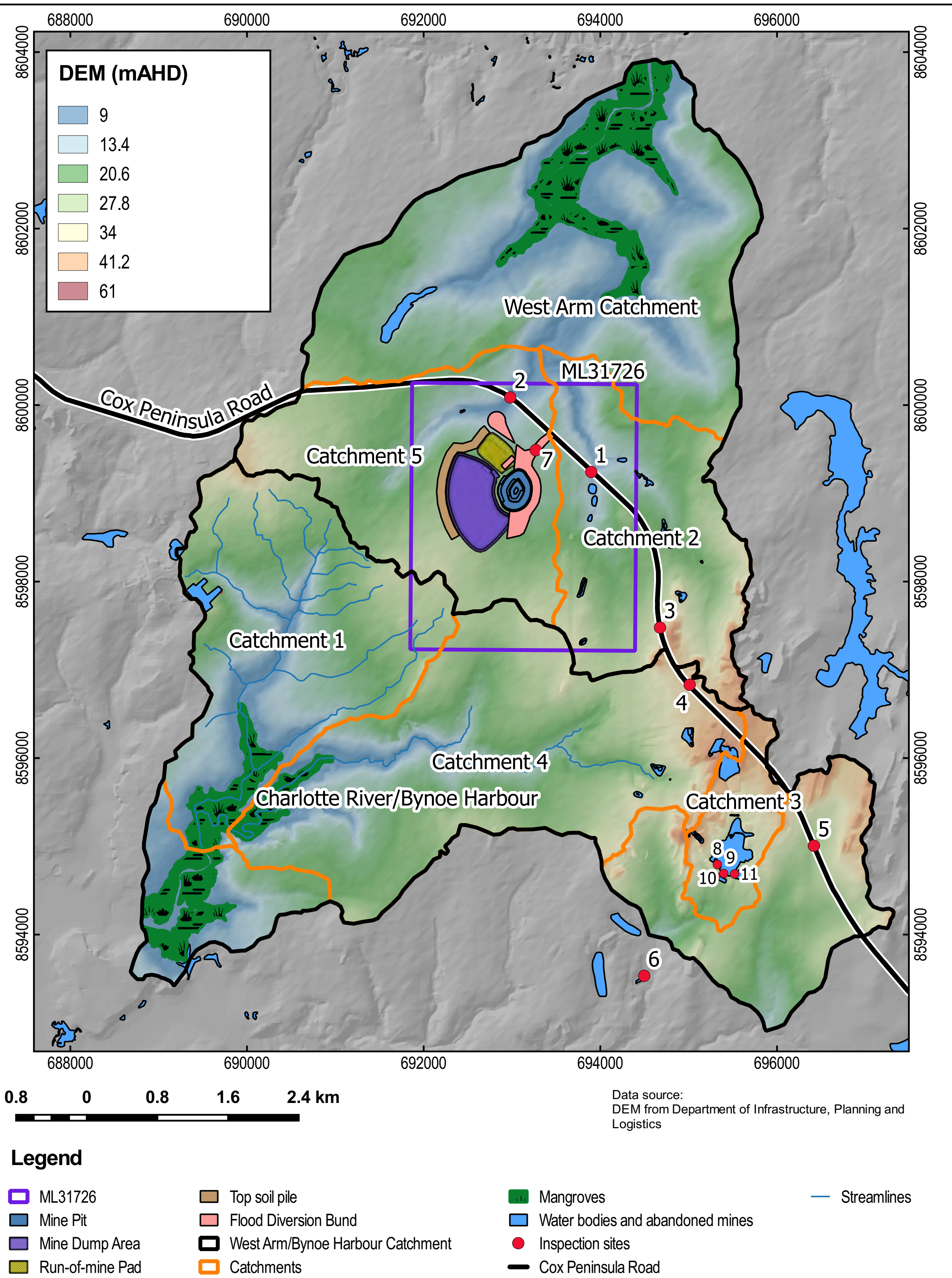

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Figure 1. Location of ML(A) 31726 and the stream catchments analysed in this project.

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

CORE

2 Low, average and high rainfall year scenarios for HEC-HMS modelling

Low, average and high rainfall-year scenarios for HEC-HMS modelling in this project were determined based on annual rainfall from 1st July to 30th June next year.

A 15-min rainfall record from the Northern Territory Government's (NTG) Winnellie site was used for simulations and the annual exceedance probability for the annual rainfall record was determined (Report 1). The lowest recorded annual rainfall is 992mm. An equal or lower annual rainfall depth has a 30% chance of occurring in 5 years (chose to exceed the expected life of mine). The average rainfall is 1792mm. An equal or lower annual rainfall depth has a 98% chance of occurring in 5 years. The highest is 2810mm. The chance of occurrence of an equal or lower rainfall depth in 5 years is almost 100% (*Table 1*).

Table 1. Selected low, average and high rainfall years.

Rainfall scenario		Wet season annual rainfall depth (mm)	Annual Exceedance Probability	Probability of an equal or lower annual rainfall depth occurs in a 5-year period
Low	2008-2009	992	0.93	0.30
Average	2006-2007	1792	0.47	0.98
High	2010-2011	2810	0.07	1.00

3 Pre-Mining baseline water balance

The calibrated HEC-HMS was used to simulate rainfall discharge for Darwin Harbour catchments and the Bynoe Harbour catchment intersected by ML(A) 31726 (*Figure 1*). The catchment model components were defined in QGIS and are shown in Appendix A.

The results are presented in the following sub-sections.

3.1 Darwin Harbour catchments

Four sub-catchments were delineated in Darwin Harbour Catchment 2 and 5 (*Figure 2*). The Darwin Harbour HEC-HMS model is shown in Appendix A. The results of the simulations are in Table 2 and the simulated hydrographs relative to rainfall are in *Figure 3*. For the scenarios used, the main rainfall occurs in February and March.

Table 2. Results of pre-mining water balance modelling for Darwin Habour Catchments 2 and 5.

Catchment	Area (km^2)	Low rainfall year		Average rainfall year		High rainfall year	
		Streamflow (ML)	Losses (ML)	Streamflow (ML)	Losses (ML)	Streamflow (ML)	Losses (ML)
Sub-catchment 5a	4.8	3375	1426	6318	2355	10697	2903
Sub-catchment 5b	2.4	1550	791	2934	1295	5004	1628
Sub-catchment 2a	3.4	1893	1310	3674	2112	6533	2540
Sub-catchment 2b	3.0	1907	1082	3634	1765	6248	2218
Total	**13.6**	**8725**	**4609**	**16560**	**7527**	**28482**	**9289**

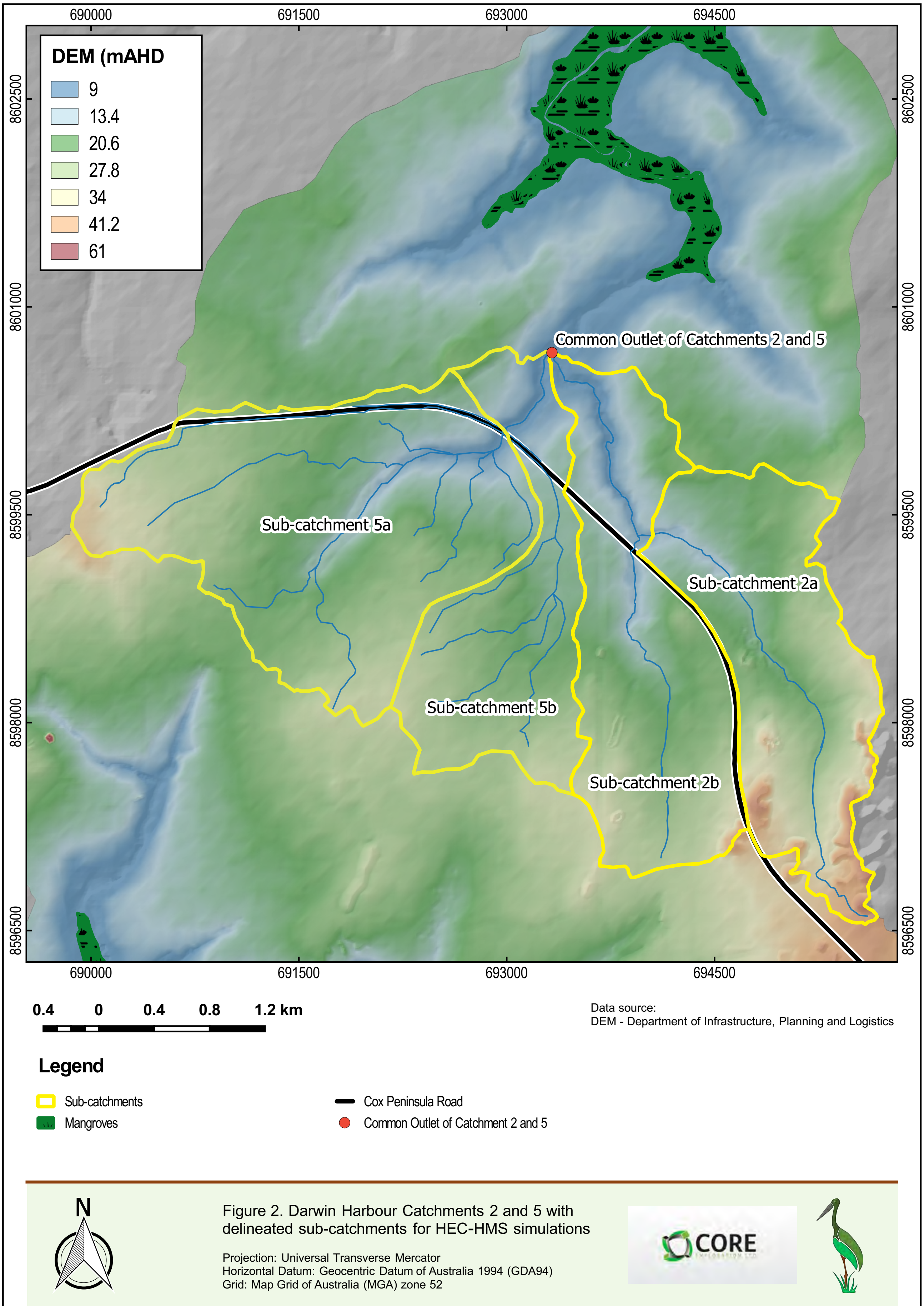


Figure 2. Darwin Harbour Catchments 2 and 5 with delineated sub-catchments for HEC-HMS simulations

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

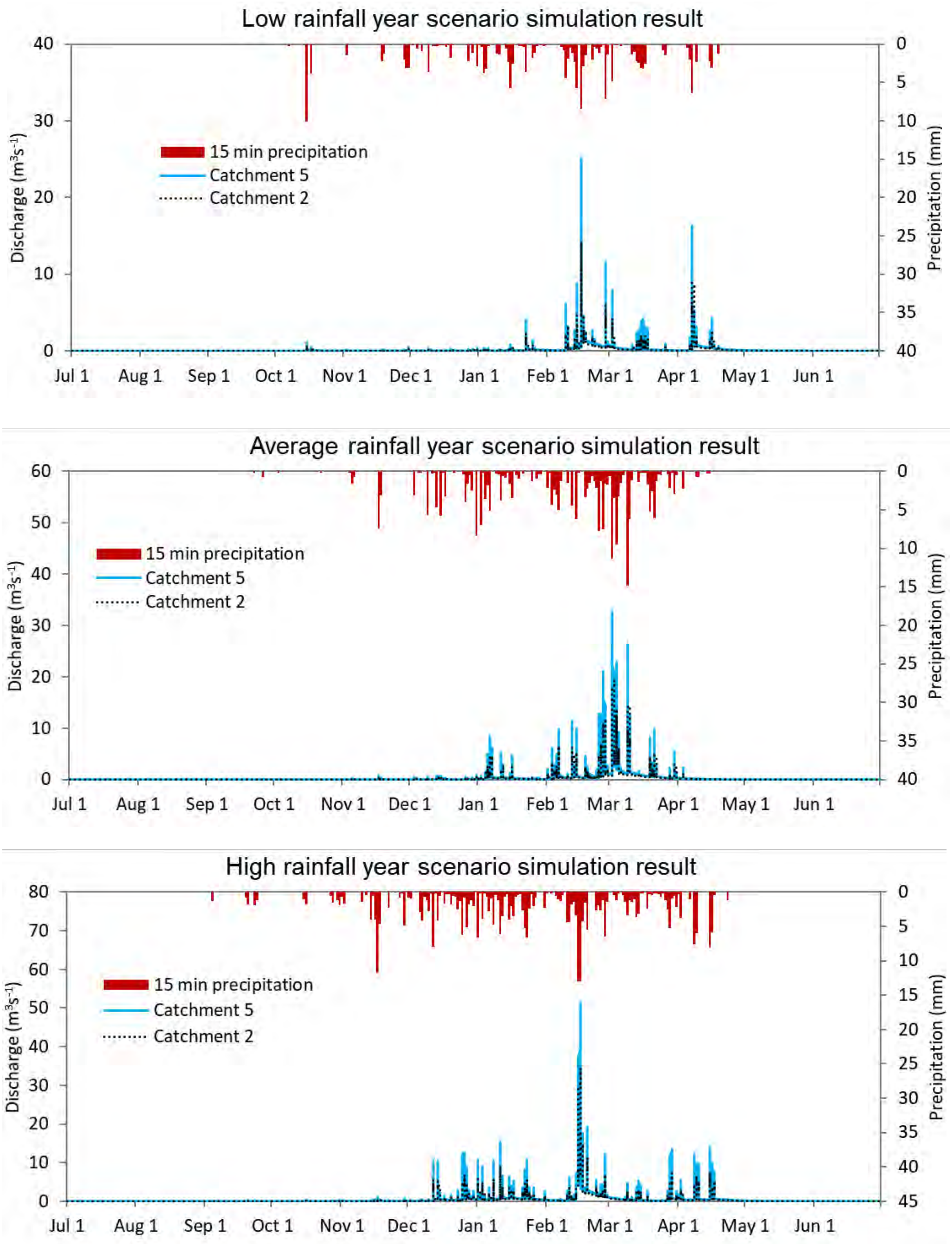


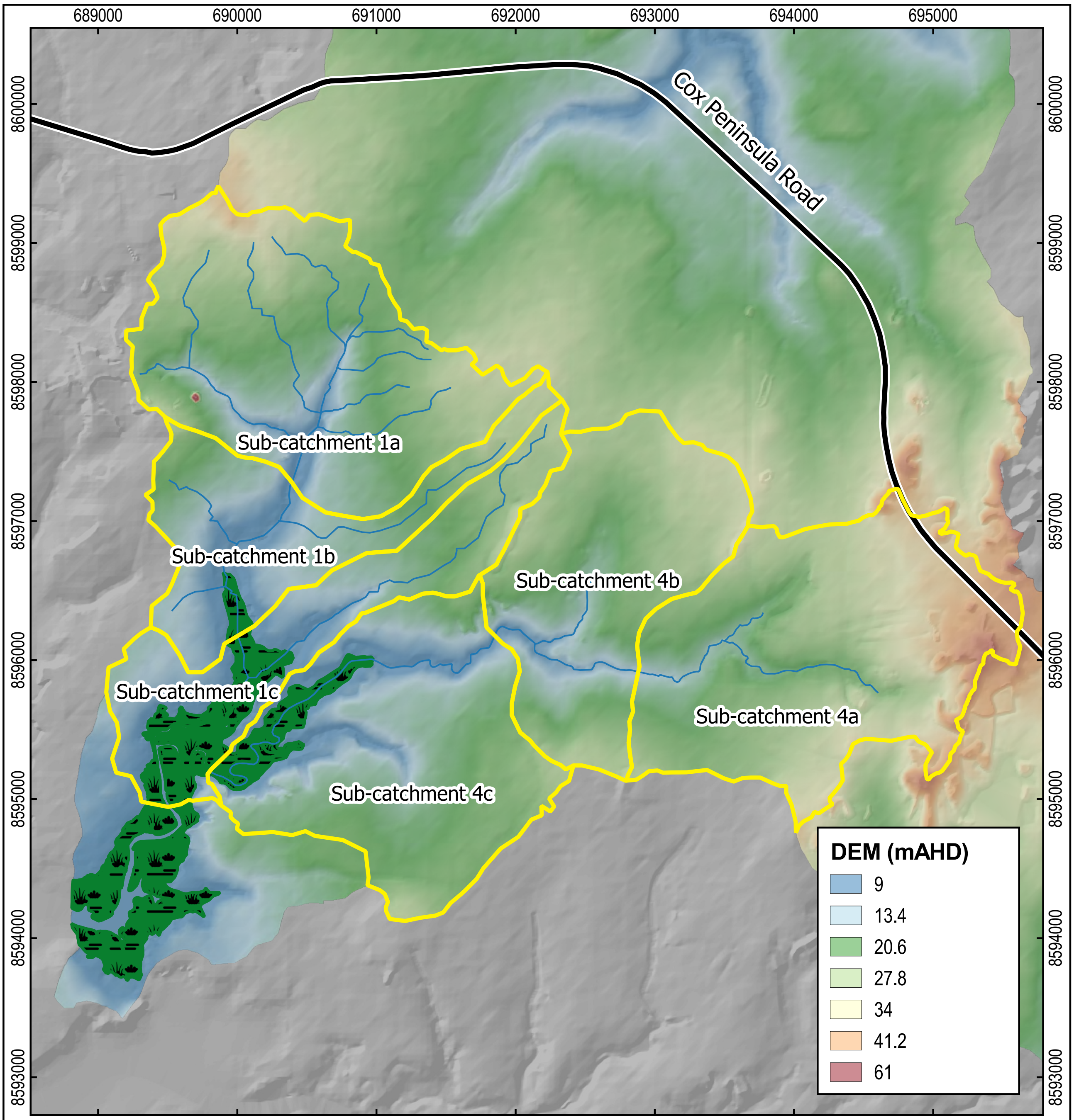
Figure 3. The variation of simulated discharge relative to a low, average and high rainfall year for the Darwin Harbour catchments (Catchments 2 and 5).

3.2 Bynoe Harbour catchments

Six sub-catchments were delineated in Catchments 2 and 4 (*Figure 4*). The Bynoe Harbour HEC-HMS model is shown in Appendix A. The results of the simulations are in Table 3 and the simulated hydrographs relative to rainfall are in *Figure 5*.

Table 3. Results of pre-mining water balance modelling for Bynoe Harbour Catchments 1 and 4.

Catchment	Area (km^2)	Low rainfall year		Average rainfall year		High rainfall year	
		Streamflow (ML)	Losses (ML)	Streamflow (ML)	Losses (ML)	Streamflow (ML)	Losses (ML)
Sub-Catchment 1a	3.93	2996	903	5514	1530	9118	1928
Sub-Catchment 1b	1.96	1304	644	2460	1059	4190	1329
Sub-Catchment 1c	2.26	1307	931	2541	1502	4540	1799
Sub-Catchment 4a	4.20	3004	1160	5551	1972	9412	2384
Sub-Catchment 4b	2.94	2672	239	4898	361	7609	638
Sub-Catchment 4c	3.58	2460	1094	4613	1808	7833	2235
Total	**18.87**	**13743**	**4971**	**25577**	**8232**	**42702**	**10313**



0.5 0 0.5 1 1.5 km

Data source:
DEM - Department of Infrastructure, Planning and Logistics
Base Map - ESRI Satellite (ArcGIS/World Imagery)

Legend

Sub-catchments | Streamlines | Cox Peninsula Road | Mangroves

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Figure 4. Bynoe Harbour catchments 1 and 4 with delineated sub-catchments for HEC-HMS modelling

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

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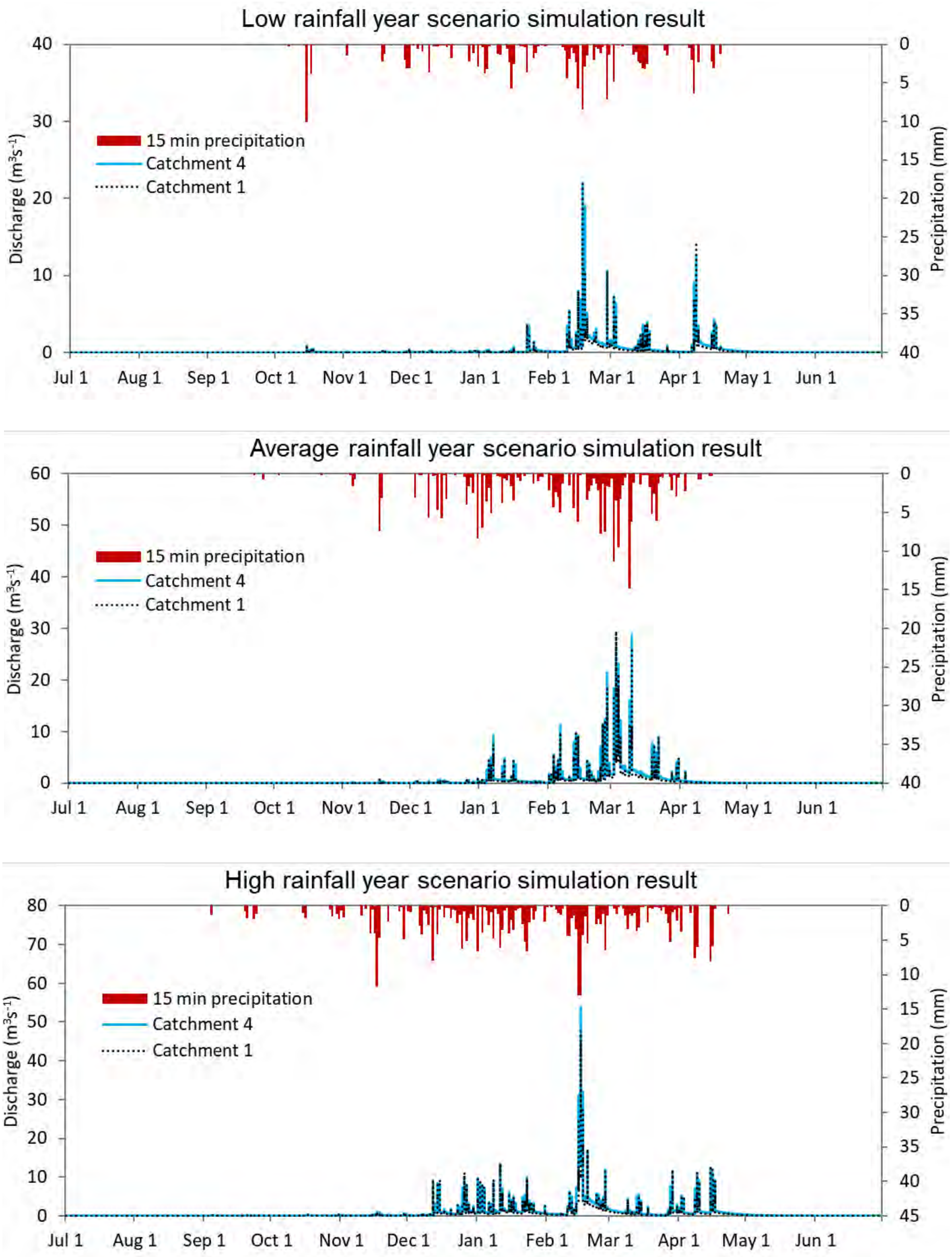


Figure 5. Simulated discharge relative to a low, average and high rainfall year for the Bynoe Harbour catchments (Catchments 1 and 4).

4 Post-mining water balance – Catchments 2 & 5

Simulations for the post-mining condition were run for the Darwin Harbour catchments (Catchments 2 and 5) only, as these are the only catchments where infrastructure is proposed at this stage.

4.1 Mine infrastructure

To produce the post-mining digital elevation model (DEM), the DEM for the mine infrastructure: pit, waste rock dump, topsoil store, bund, ponds and the run-of-mine (RoM) pad, were developed based on the digital drawings supplied by Core. The design of the bund and the ponds was not final at the time of this analysis. In the drawing, the part of the bund housing the access road is about 15m lower than the existing ground surface which is obviously not realistic, so the DEM for this part of the bund was developed as a slope linking the bund top and the existing ground surface. In addition, the unfinished structure to the northeast of the RoM pad was not considered in the simulations. The ponds had bottom elevation of 0mAHD which again may not be the final design. The DEM of mine infrastructure was then merged with the 2m-DEM (*Figure 6*).

Cross sections through merged and original DEMs are shown in *Figure 7*. The total area covered by infrastructure is only about 10% of catchments 2 and 5. The post-mining HEC-HMS model was developed based on the merged DEM and run for the low, average and high rainfall years assuming that at the commencement of simulations, the ponds and the dam were full of water.

Section 5 below assesses water storage in the Observation Hill dam (OHD) and its sufficiency to support mining requirements. The assessment showed that ancillary water sources may be required depending on pump rates used OHD spillway level. Therefore, 2 other possible sites were assessed for water storage. One of those locations is in Catchment 5 and is referred to as Dam C5 (*Figure 6*). For hypothetical scenarios, Dam C5, with a spillway elevation of 16.93 mAHD, was added to the post-mining (infrastructure added) HEC-HMS model (Appendix A) and simulations were run for the low, average and high rainfall years.

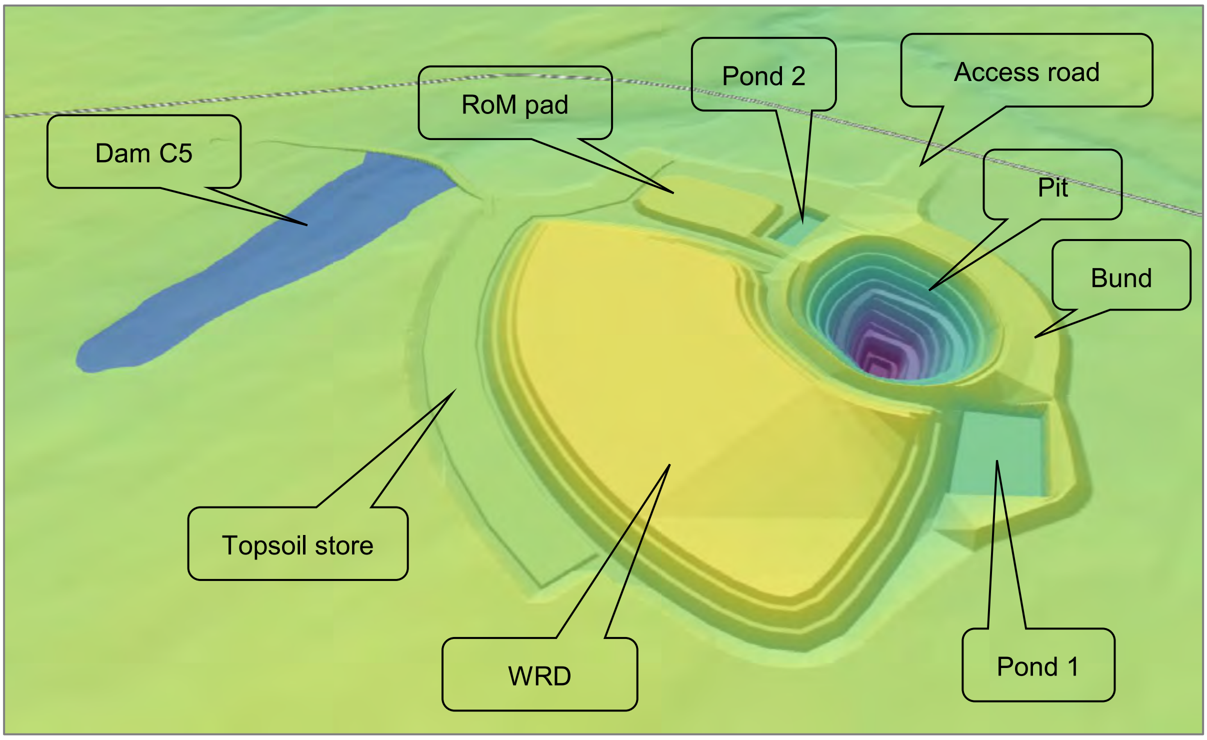


Figure 6. 3D view of proposed mining infrastructure DEMs merged with the NTG 2m DEM with hypothetical dam C5.

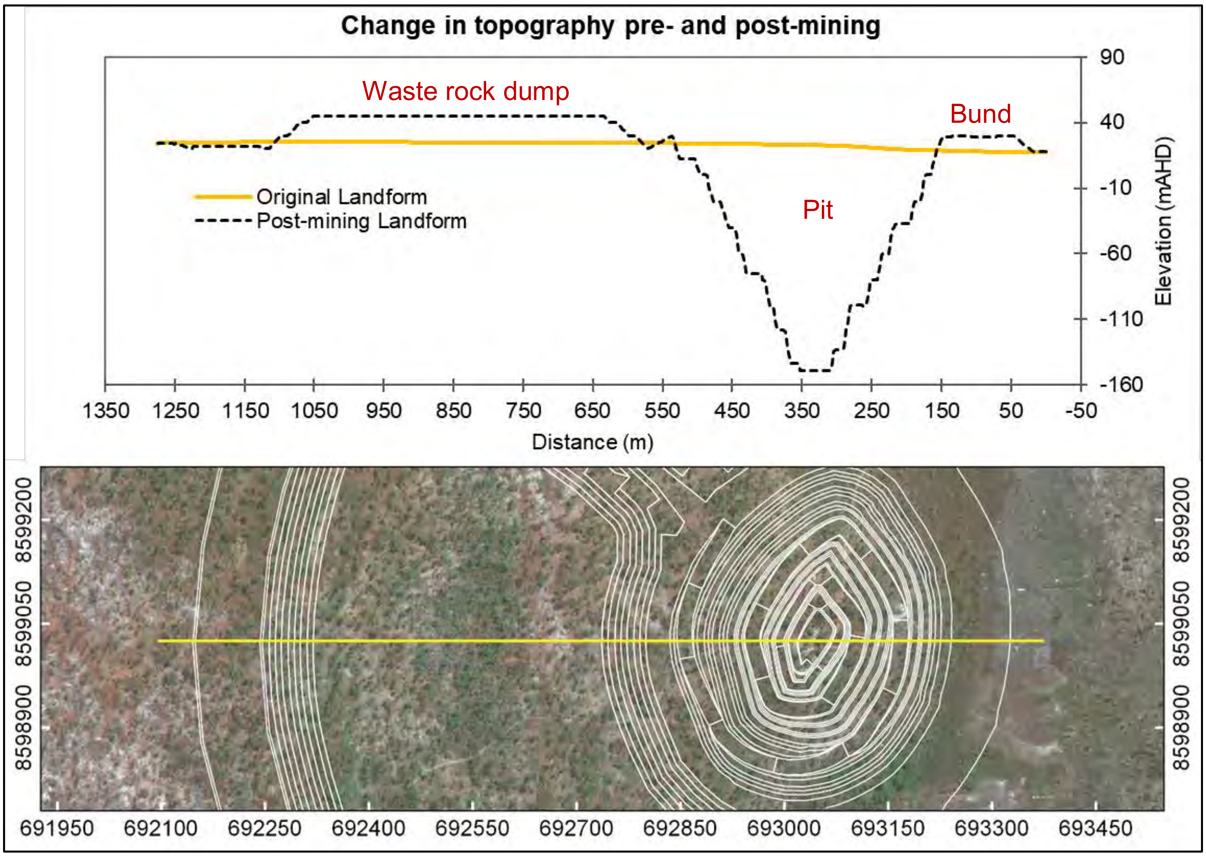


Figure 7. Cross sections through the merged and original DEMs showing the change in topography post-mining (GDA94/MGA52).

The water balance for Darwin Harbour catchment 2 and 5 with pre- and post-mining conditions are shown in *Table 4*. Simulated weekly flows at the catchment outlet are shown in *Figure 8*. The data were taken at the common outlet of the two catchments (*Figure 2*).

Table 4. Water balance for catchment 2 and 5 pre-mining, with post-mining infrastructure: pit, waste rock dump, topsoil store and run-of-mine-pad, and ponds, and hypothetical dam C5. The flows are taken at the outlets of the catchments.

Rainfall scenario	Pre-mining conditions		Post-mining conditions (with mine infrastructure)					Post-mining conditions (with mine infrastructure and hypothetical dam C5)					
	Streamflow (ML)	Losses (ML)	Streamflow (ML)	Losses (ML)	Storage in pit (ML)	Storage in Pond 1 (ML)	Storage in Pond 2 (ML)	Streamflow (ML)	Losses (ML)	Storage in pit (ML)	Storage in Pool 1 (ML)	Storage in Pond 2 (ML)	Storage in dam C5 (ML)
Low rainfall year	8738	4432	8000	5126	62	503	156	7716	5492	62	503	156	308
Average rainfall year	16592	7441	15743	8158	153	501	155	15547	8453	153	501	155	291
High rainfall year	28503	9088	26232	11123	256	503	156	26092	11342	256	503	156	309

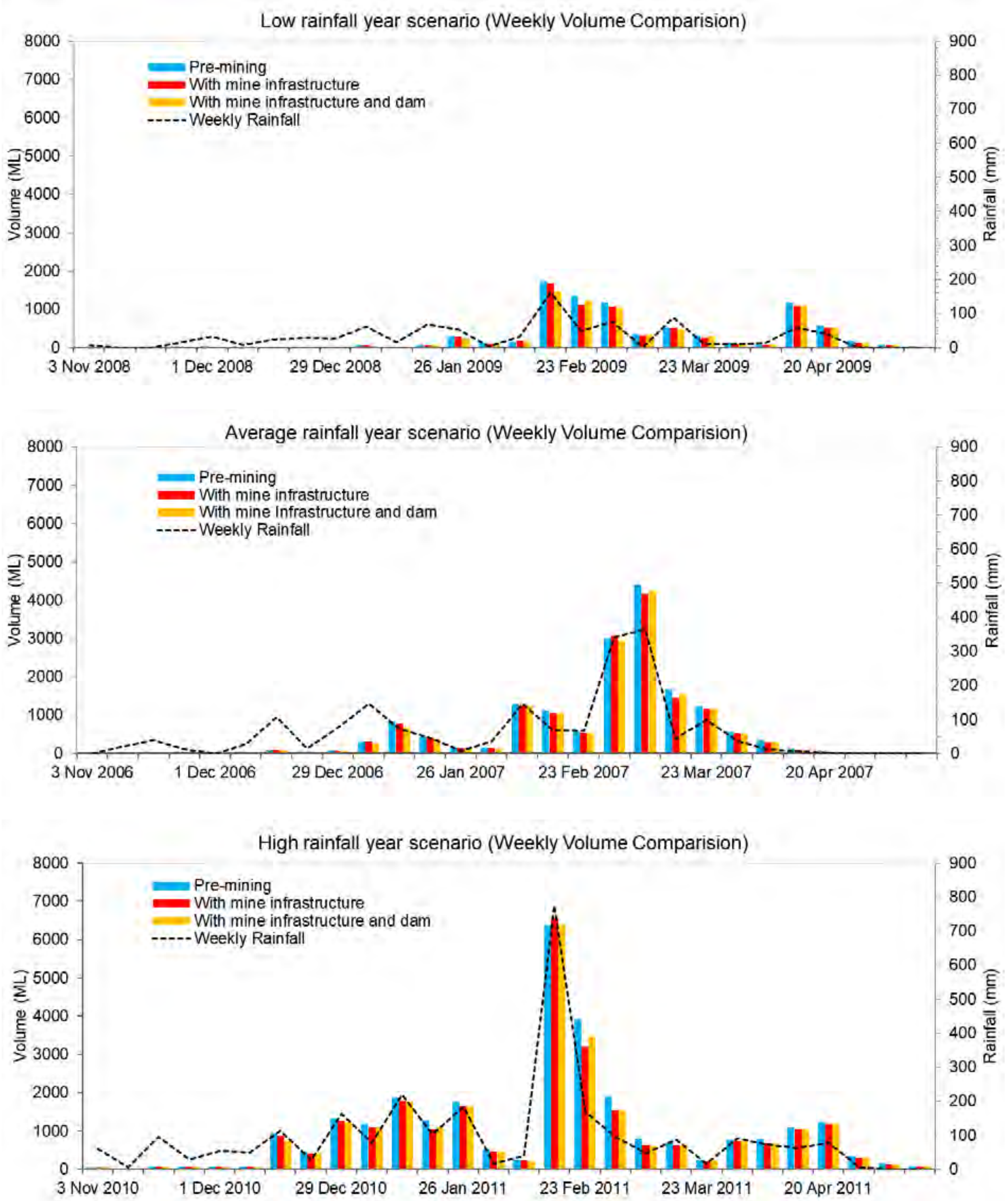


Figure 8. Simulated weekly catchment outflows relative to rainfall for catchments 2 and 5 for pre- and post-mining conditions and post-mining with Dam C5.

These simulations show that for the post-mining conditions, without the dam, total catchment outflow decreases by about an average of 7.2%. With the dam the decrease in outflow is 8.5% for a high rainfall year 6.3% for average rainfall year and 11.7% for the low rainfall year. Most of this water is stored in the dam and ponds with increased losses to evaporation.

Figure 9 shows Darwin Harbour catchment flow accumulation derived using QGIS before mining without infrastructure (pit, waste rock dump, topsoil store, RoM pad, bund and ponds) and Darwin Harbour catchment post-mine with infrastructure.

4.2 Summary of results

The changes in flow accumulation show how the infrastructure changes the original surface drainage of the catchment. The modelling shows reduction in flow to the outlet of catchments 2 and 5 by and average about 7.2% for the high, low and average years when only infrastructure is considered. Most of this loss is from the increased water loss due to retention of water in and evaporation from the sediment ponds and the pit. When the dam is included reduction to catchment outflow 6.3% of the pre-mine outflow for the average rainfall years, 8.5% for the high rainfall years and about 11.7% for the low rainfall year. The average reduction due to the dam is only about 1.6% of the total pre-mine outflow. This small increase in flow reduction is due to evaporation of water from the dam.

The increased reduction in stream flow can be reduced by resizing the dam if the option of a dam in this catchment is used and reducing the spillway level during rehabilitation.

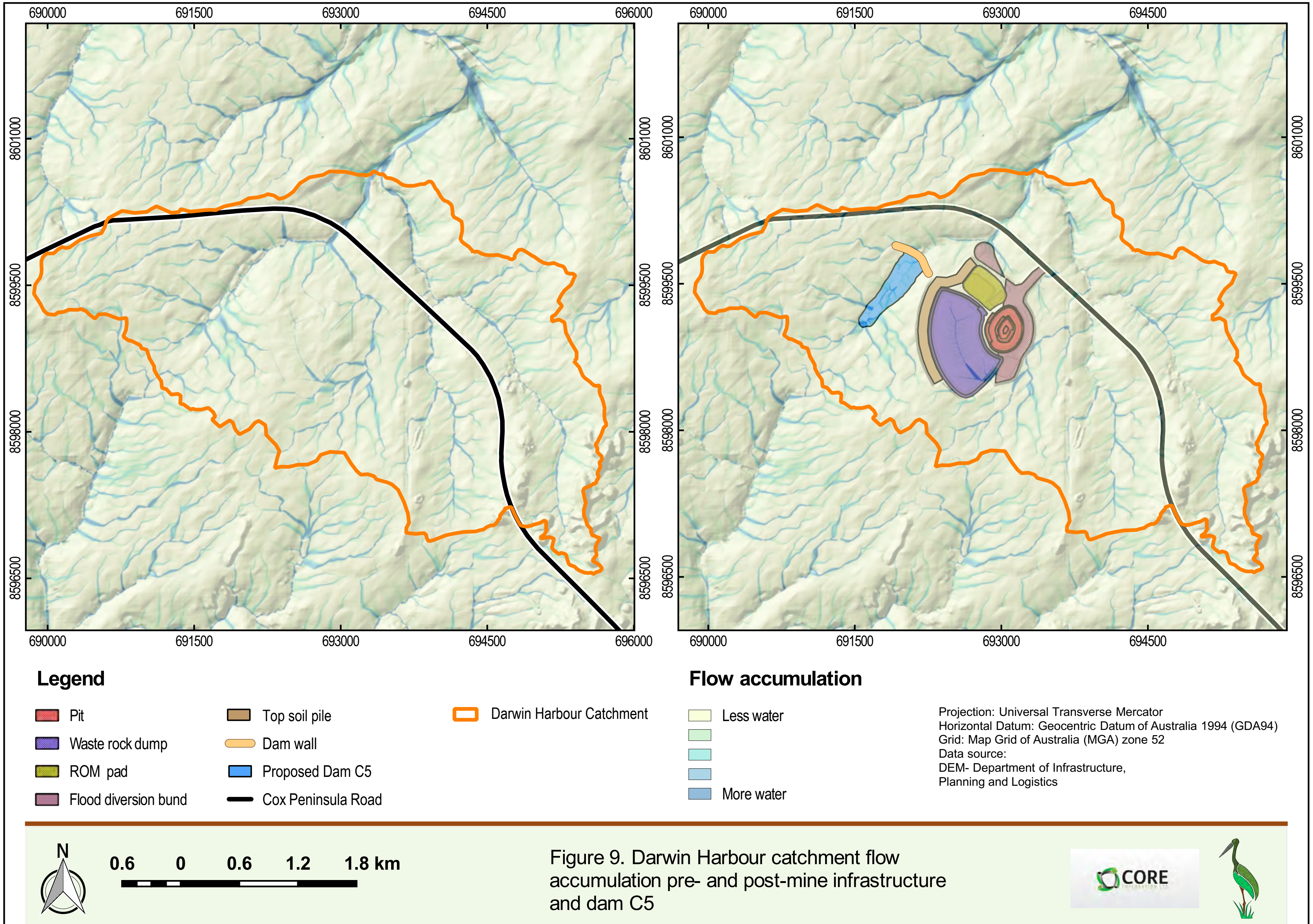


Figure 9. Darwin Harbour catchment flow accumulation pre- and post-mine infrastructure and dam C5

5 Observation Hill dam yield assessment

Observation Hill dam (OHD) is the main water storage facility near the mining lease and the stored water will be used for mining operations.

5.1 Catchment hydrology

The sub-catchments and dam water retention component of the OHD catchment were delineated and drainage lines determined for the HEC-HMS model. The model was run for the 3 annual rainfall scenarios (low, average and high rainfall years) and the total volume of direct rainfall and catchment run-off input to OHD and the peak rate of the run-off inflow determined (*Table 5*).

Table 5. Results of the HEC-HMS model of the sub-catchments draining to OHD.

Rainfall scenario	Total Rainfall (mm)	Total Inflow (ML)	Peak Inflow Rate (m^3s^{-1})
Low rainfall year	992	599	6.3
Average rainfall year	1792	1228	8.4
High rainfall year	2810	1980	10.8

5.2 Storage capacity

OHD bathymetry was not represented on the 2m DEM used in this study. Therefore, the existing excavated bathymetry was recreated (as described in Project 1 report) and used for simulation of a low, average and high rainfall year to assess volumetric changes due to rainfall, evaporation, seepage and mine applications.

A volume of ≈364 ML was estimated for OHD at a spillway level of ≈30mAHD using the recreated DEM (*Figure 11*). A preliminary assessment showed that the stored volume may not be sufficient for sustaining mine requirements, so the OHD storage capacity with wall lift scenarios were also assessed.

The maximum spillway elevation can be about 40 mAHD at which level Cox Peninsula Road will be impacted during flooding (*Figure 12*, 40 mAHD water surface area). A volume-elevation (V-E) curve OHD (*Figure 10*) was derived from the recreated DEM (*Figure 11*). This curve does not consider increased dam volume resulting from excavations to construct the wall lifts.

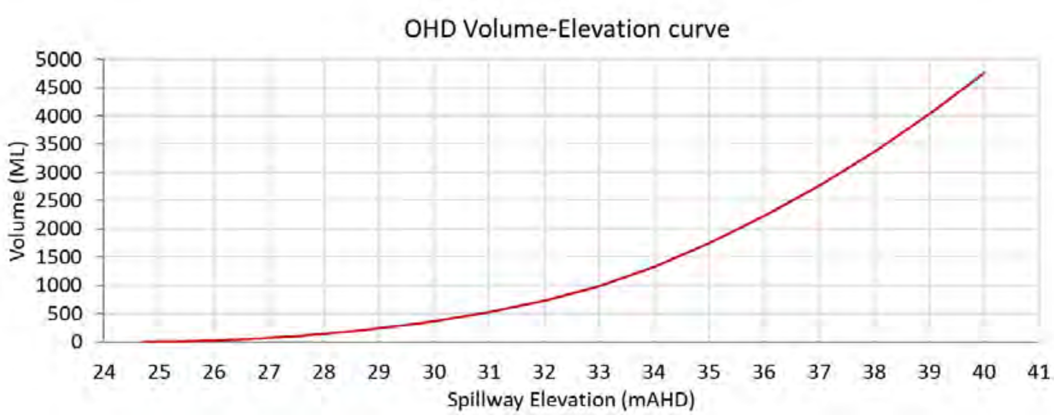


Figure 10. The volume-elevation curve of OHD developed based on the recreated DEM

For wall lift scenarios, it was assumed that the top of the dam wall is 1.5 m higher than the spillway elevation. This assumption is based on the common design practice for earth dams to prevent overtopping. A more accurate spillway level and volume storage should be determined by detailed engineering analysis during the design phase of the wall lift.

Based on the annual inflow volume to OHD in *Table 5*, the V-E curve in *Figure 10* and an assessment of the existing topography along the centre line of proposed wall lifts (*Figure 12*), 2 wall lift scenarios were selected for volume calculations using HEC-HMS modelling. Storage capacity for the lift scenarios and the existing dam are shown in *Table 6*.

Table 6. OHD spillway elevation scenarios for HEC-HMS modelling

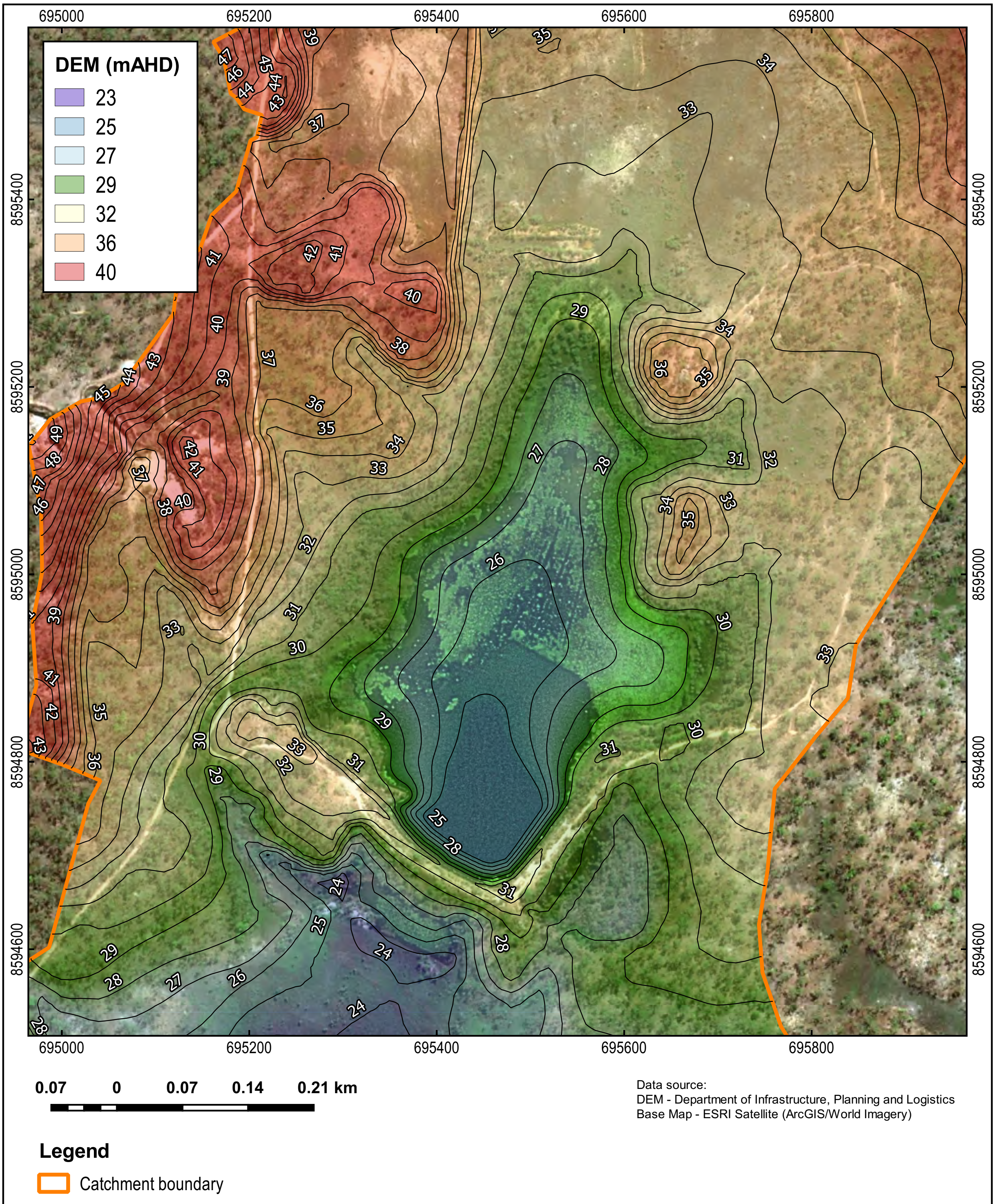
Spillway elevation scenario	Spillway elevation (mAHD)	Top of dam wall (mAHD)	Storage capacity (ML)
1. Existing	30.0	31.7 at lowest point	364
2. Wall lift 1	31.5	33.0	628
3. Wall lift 2	33.6	35.1	1191

In “wall lift 1” scenario, without considering evaporation, the dam can hold approximately 51% of the total inflow volume of an average rainfall year (*Table 5*). This scenario involves relatively less earthworks.

In “wall lift 2” scenario, without considering evaporation, the dam can hold approximately 97% of the total inflow volume of an average rainfall year (*Table 5*). This scenario involves a large amount of earthworks. The existing dam wall needs to be raised by up to 3.4 m (*Figure 13*).

The water surface area at spillway elevation and the sections needing earthworks under “wall lift 1” and “wall lift 2” scenarios are shown in *Figure 12*.

The profiles of the dam wall along the centre line of proposed wall raises for the 3 scenarios are shown in *Figure 13*



Data source:
DEM - Department of Infrastructure, Planning and Logistics
Base Map - ESRI Satellite (ArcGIS/World Imagery)

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Figure 11. DEM showing estimated bathymetric surface of OHD

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

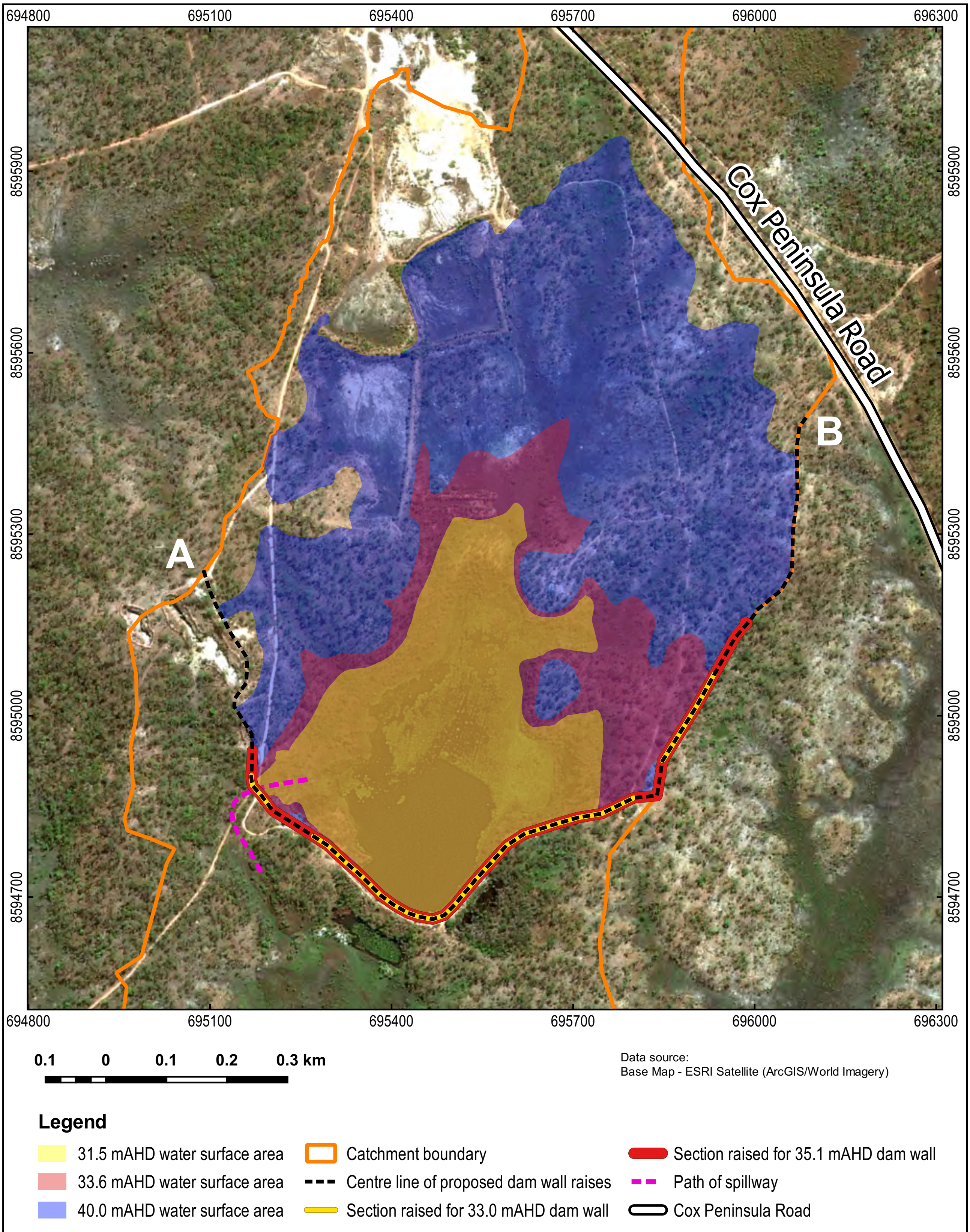

Figure 12. Water surface area for different spillway elevations and sections need to be raised for different dam wall elevations

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

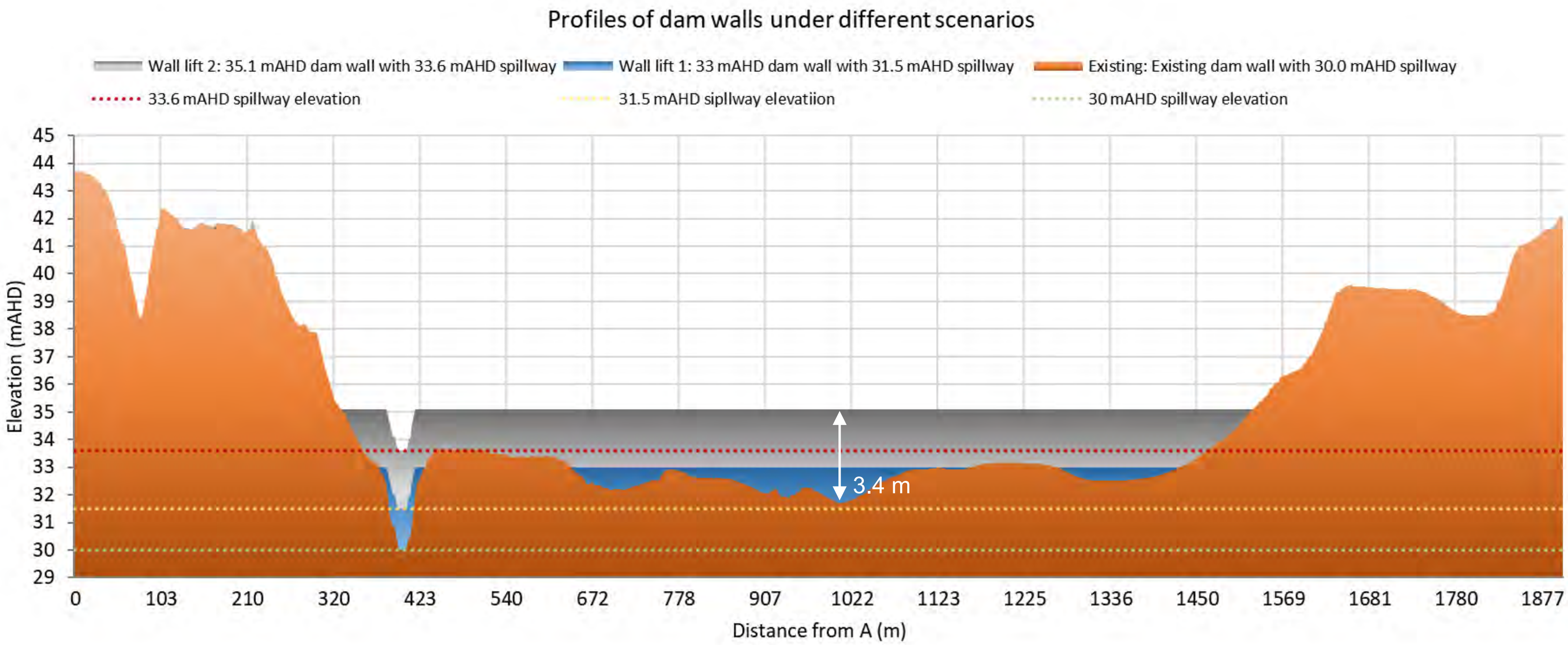

Figure 13. Profiles for the existing dam wall and raised dam walls along the centre line of proposed dam wall raises from A to B in Figure 12. The existing dam wall profile was taken from the 2-m DEM.

5.3 Yield analysis

The HEC-HMS model was used for a yield analysis for the dam wall height and rainfall scenarios.

5.3.1 OHD HEC-HMS model setup

The OHD was modelled according to the following specifications:

- The volume of the dam component in the HEC-HMS model was defined based on the V-E curve in *Figure 10*.
- The spillway elevation was set as 30, 31.5 or 33.6 mAHD for different wall height scenarios,
- An integrated pump element was used to simulate the mine site water use. The pump will turn off when water depth is <0.1m and turn on again when water depth is >1.9m,
- Monthly evaporation rates (*Table 7*) for Darwin River[1] dam were used for this model,
- A seepage of 2.3 mm per day applied. This seepage rate is derived from a study of 136 irrigation dams in Australia[2], and
- A groundwater component wasn't applied because at the time of writing this report data were not yet available.

Table 7. Monthly evaporation (mm) of Darwin river dam.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Evaporation	232	187	197	223	207	181	193	174	204	231	226	262

5.3.2 Simulation scenarios

Since pit dewatering may be used and dust suppression may not be required during the wet season, 4 water use scenarios (*Table 8*) were considered.

Table 8. Water use scenarios considered for water balance modelling.

Water use scenario	Water use from OHD
1. All water is extracted from OHD	2.02 MLd^{-1}
2. Volume extracted from OHD discounts return from Pit Dewatering	1.2 MLd^{-1}
3. All water is extracted from OHD but no dust suppression in wet	1.07 MLd^{-1} in wet, 2.02 MLd^{-1} in dry
4. Volume extracted from OHD discounts return from Pit Dewatering and no dust suppression in wet	0.64 MLd^{-1} in wet, 1.2 MLd^{-1} in dry

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[1] Paiva J 1993. Darwin River Dam Safe Yield Improvement. Hanna's Pool Diversion. Power and Water Authority Water Resources Division. Report 26/93

[2] Wigginton D 2011. Storage Seepage and Evaporation, Final Summary of Results, Cotton Storages Project 2011. Cotton Catchment Communities CRC and National Water Commission. Department of Natural Resources and Mine.

Each of the 4 water use scenarios (*Table 8*) were applied to each of the 3 spillway elevation scenarios (*Table 6*) for the following rainfall scenarios:

- Water use scenario 1 and 2 (Table 8) were run for a 5-year continuous sequence: Average, Low, Average, High and Average rainfall years.
- The models for water use scenario 3 and 4 (Table 8) were run for an average rainfall year.
- The initial storage of the dam in the simulations was determined by running the model with water at the spillway level (end of Wet Season) with the pump off through a dry season applying only evaporation and seepage. That is, pump extraction started at the start of the next Wet Season.
- It was assumed the pump extraction started at the beginning of the hypothetical 2018 wet season (October) of average rainfall.

5.3.3 OHD water balance simulation results

The modelling result for each water use scenario under the 30, 31.5 and 33.6 mAHD spillway elevation scenarios are shown in *Figure 14*, *Figure 15* and *Figure 16*[3]. The deficit of water for different scenarios are shown in *Table 9, Table 10* and *Table 11*. Note: in these tables the average annual deficits were not calculated for some 5-year simulations which have deficit only caused by a low rainfall year.

[3] Where the figures show the pump is off, this is due a lack of water rather than the project not requiring water to be pumped during this period.

Table 9. Simulated deficits for 30.0 mAHD spillway level scenario for average, low, average, high, average rainfall years. Only simulations for an average rainfall year were conducted for the variable pump rates.

Water use scenario	2.02 MLd^{-1} for a 5-year simulation				
Year (1st April to 1st Oct)	Average	Low	Average	High	Average
No. of days in deficit	49	181	141	114	153
Water deficit (ML)	100	366	285	230	309
Average annual deficit (ML)	258				
Water use scenario	1.2 MLd^{-1} for a 5-year simulation				
Year (1st April to 1st Oct)	Average	Low	Average	High	Average
No. of days in deficit	0	124	73	78	71
Water deficit (ML)	0	149	88	94	85
Average annual deficit (ML)	83				
Water use scenario	1.07 MLd^{-1} in wet, 2.02 MLd-1 in dry 1-year average rainfall simulation				
Year (1st April to 1st Oct)	Average	-	-	-	-
No. of days in deficit	41	-	-	-	-
Water deficit (ML)	83	-	-	-	-
Average annual deficit (ML)	83				
Water use scenario	0.64 MLd^{-1} in wet, 1.2 MLd-1 in dry 1-year average rainfall simulation				
Year (1st April to 1st Oct)	Average	-	-	-	-
No. of days in deficit	0	-	-	-	-
Water deficit (ML)	0	-	-	-	-
Average annual deficit (ML)	0				

Table 10. Simulated deficits for 31.5 mAHD spillway level scenario for average, low, average, high, average rainfall years. Only simulation for an average rainfall year were conducted for the variable pump rates.

Water use scenario	2.02 MLd^{-1} for a 5-year simulation				
Year (1st April to 1st Oct)	Average	Low	Average	High	Average
No. of days in deficit	0	156	123	57	21
Water deficit (ML)	0	315	248	115	42
Average annual deficit (ML)	144				
Water use scenario	1.2 MLd^{-1} for a 5-year simulation				
Year (1st April to 1st Oct)	Average	Low	Average	High	Average
No. of days in deficit	0	30	76	0	0
Water deficit (ML)	0	36	91	0	0
Water use scenario	1.07 MLd^{-1} in wet, 2.02 MLd-1 in dry 1-year average rainfall simulation				
Year (1st April to 1st Oct)	Average	-	-	-	-
No. of days in deficit	0	-	-	-	-
Water deficit (ML)	0	-	-	-	-
Average annual deficit (ML)	0				
Water use scenario	0.64 MLd^{-1} in wet, 1.2 MLd-1 in dry 1-year average rainfall simulation				
Year (1st April to 1st Oct)	Average	-	-	-	-
No. of days in deficit	0	-	-	-	-
Water deficit (ML)	0	-	-	-	-
Average annual deficit (ML)	0				

Table 11. Simulated deficits for 33.6 mAHD spillway level scenario for average, low, average, high, average rainfall years. Only simulation for an average rainfall year were conducted for the variable pump rates.

Water use scenario	2.02 MLd^{-1} for a 5-year simulation				
Year (1st April to 1st Oct)	Average	Low	Average	High	Average
No. of days in deficit	0	81	113	0	0
Water deficit (ML)	0	164	228	0	0
Water use scenario	1.2 MLd^{-1} for a 5-year simulation				
Year (1st April to 1st Oct)	Average	Low	Average	High	Average
No. of days in deficit	0	0	15	0	0
Water deficit (ML)	0	0	18	0	0
Water use scenario	1.07 MLd^{-1} in wet, 2.02 MLd-1 in dry 1-year average rainfall simulation				
Year (1st April to 1st Oct)	Average	-	-	-	-
No. of days in deficit	0	-	-	-	-
Water deficit (ML)	0	-	-	-	-
Average annual deficit (ML)	0				
Water use scenario	0.64 MLd^{-1} in wet, 1.2 MLd-1 in dry 1-year average rainfall simulation				
Year (1st April to 1st Oct)	Average	-	-	-	-
No. of days in deficit	0	-	-	-	-
Water deficit (ML)	0	-	-	-	-
Average annual deficit (ML)	0				

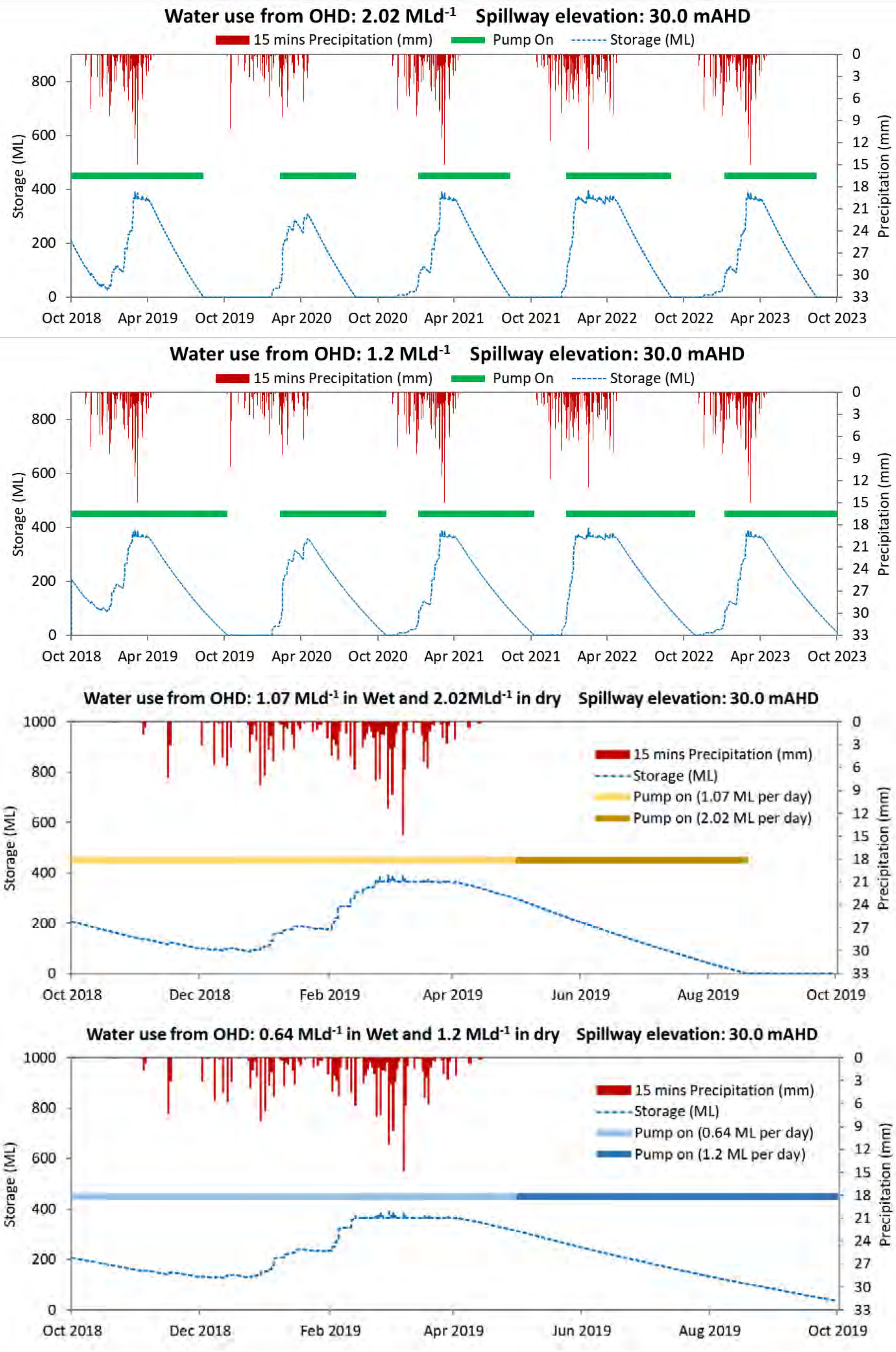


Figure 14. HEC-HMS modelling result for 30 mAHD spillway elevation scenario.

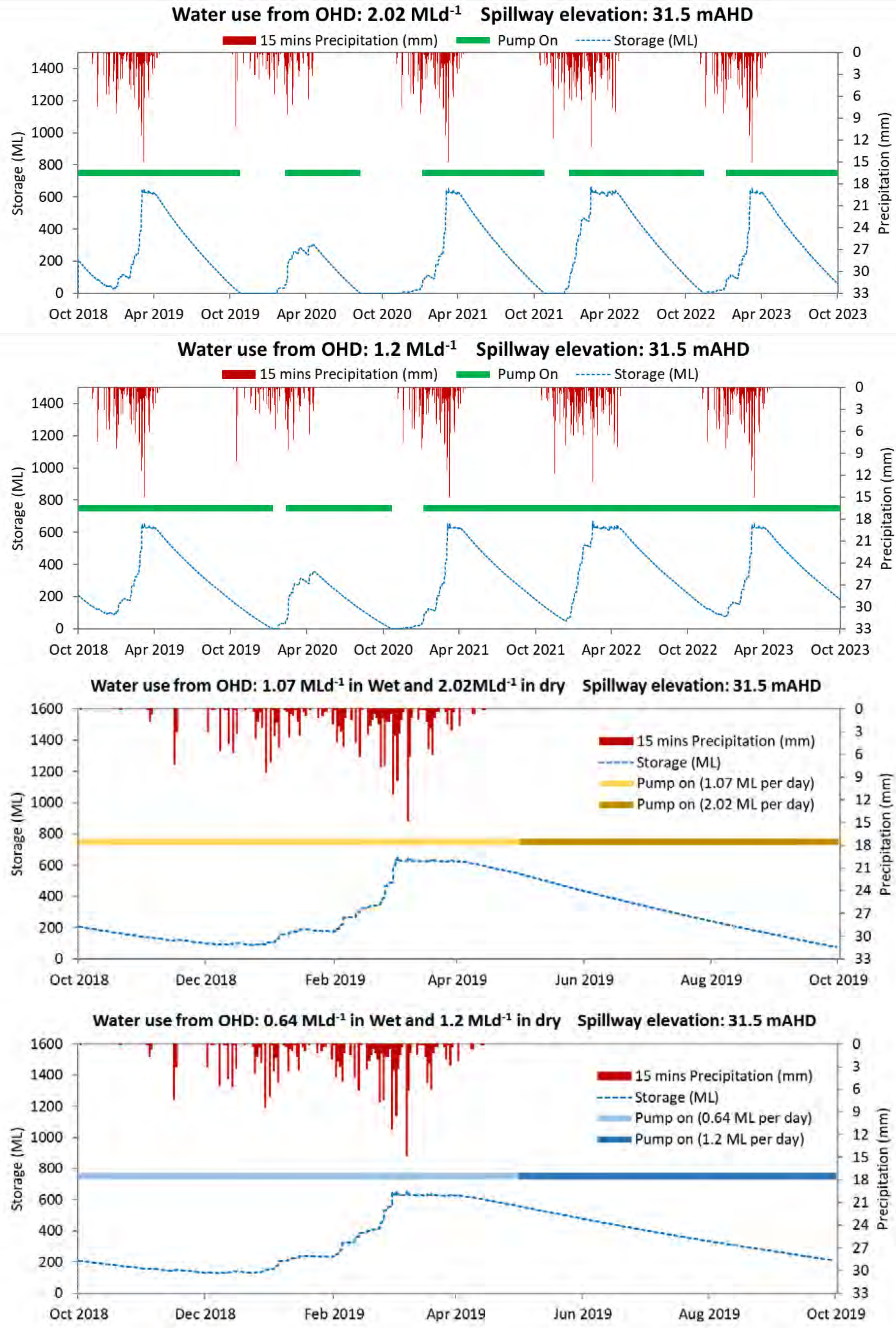


Figure 15. HEC-HMS modelling result for 31.5 mAHD spillway elevation scenario.

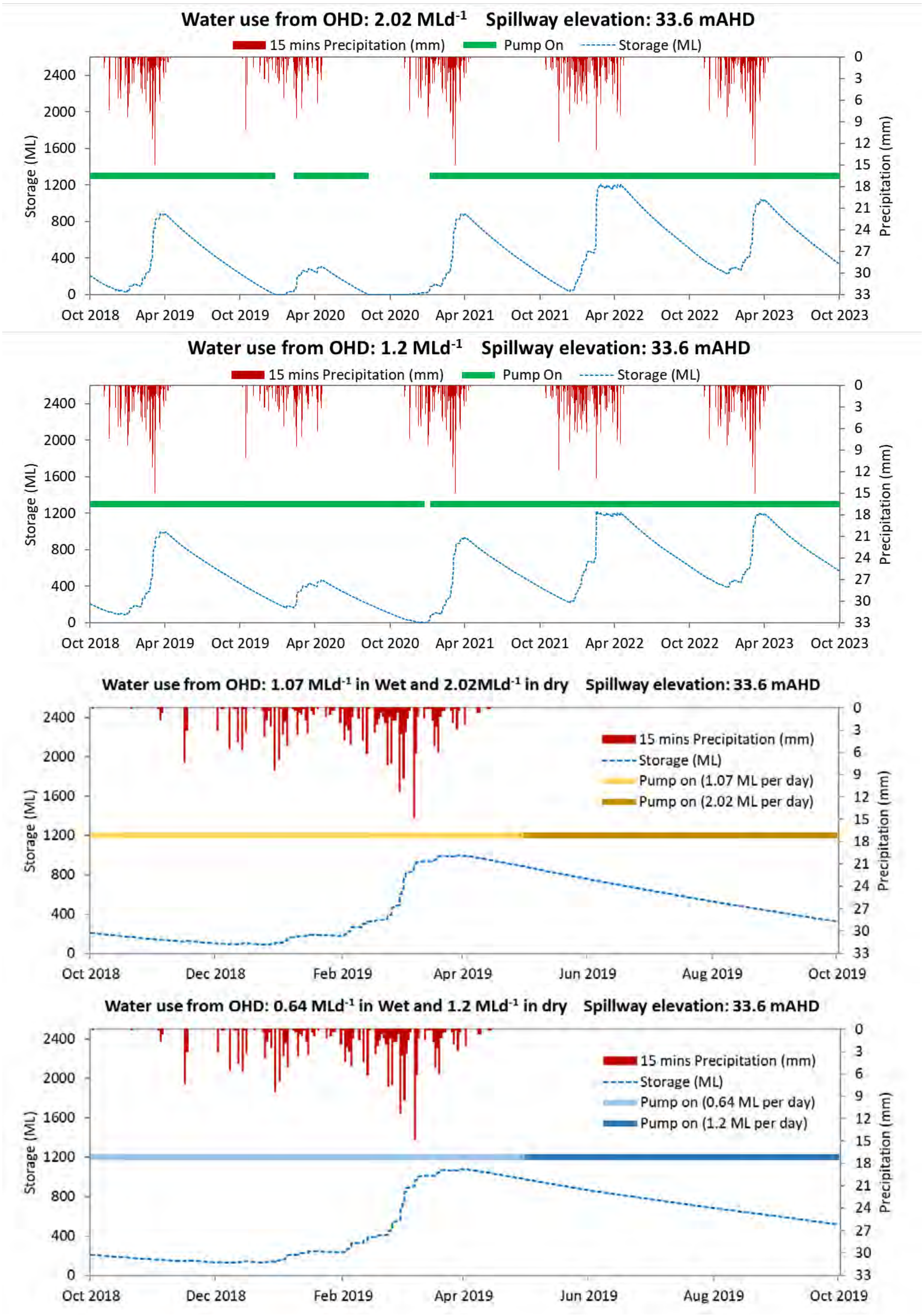


Figure 16. HEC-HMS modelling result for 33.6 mAHD spillway elevation scenario.

5.4 Influence of pumping and wall lift on downstream flows

The pumping and/or raises in dam wall height could cause changes in downstream flows; these flows can be important to environmental values in downstream areas. The change in downstream flows is mainly caused by the change of spillway outflow of OHD. This because water retention and discharge from in the OHD is the main influence on change of downstream discharge and this is caused by spillway height.

To investigate the effect of spillway height on flow at the OHD spillway (spillway flow), the model was run for an average rainfall year under different spillway height scenarios. Only the maximum pumping scenario (2.02 MLd^{-1} through the whole year) was investigated as this would provide a worst-case scenario for changes in stream flow. The hydrographs of the spillway flow under different spillway elevation scenarios are shown in *Figure 17*. The reductions in total annual spillway flow volume for OHD with different spillway elevations are in *Table 12*.

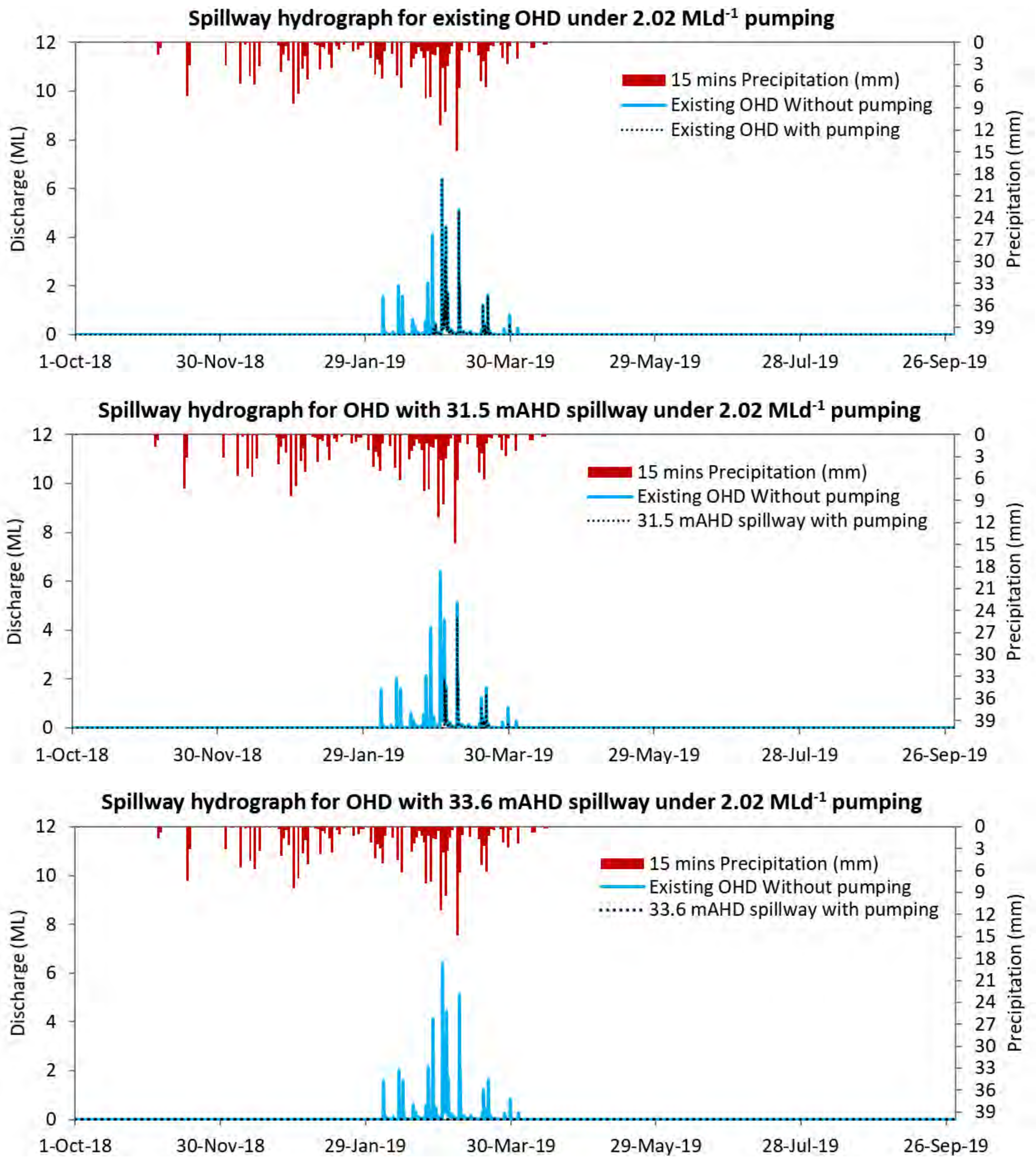


Figure 17. Compare spillway hydrographs for different spillway elevations under pumping with spillway hydrograph of existing spill without pumping

Table 12. The reduction in total annual spillway flow volumes for different spillway elevations.

Scenarios	Annual spillway flow volume (ML)	Reduction In flow volume
Existing OHD without pumping	863	N/A
Existing OHD under 2.02 MLd^{-1} pumping	545	36.8%
OHD with 31.5 mAHD spillway under 2.02 MLd^{-1} pumping	267	69.1%
OHD with 33.6 mAHD spillway under 2.02 MLd^{-1} pumping	0	100%

The OHD spillway outflow is only one of many sources of inflows to the downstream area. The relative effect of spillway level on downstream catchment flows will decrease at different downstream points, reducing progressively further downstream as other sub-catchments enter the system increasing total catchment area and discharge. That is, spillway effect it is inversely proportional to downstream catchment size. Therefore, to investigate the effect of flow changes on a wide range of vegetation communities including mangroves, it will be more appropriate to investigate the change of flow to the larger sub-catchment of the Charlotte River where it enters the tidal reaches and mangrove communities (*Figure 19*).

To investigate the flow changes for the watershed, the HEC-HMS model of OHD was expanded to include other catchments of the watershed shown in *Figure 19*. The model was run for an average annual rainfall year under different spillway height scenarios for OHD using a pumping rate of 2.02MLd^{-1}. The hydrographs of the flow at the outlet of the watershed (shown on Figure 19) under different spillway elevation scenarios are shown in *Figure 18*. The reductions in total annual flow volume at the watershed outlet for OHD with different spillway elevations under pumping are shown in *Table 13*.

Table 13. The reduction in total annual flow volume at the watershed outlet for different spillway elevations.

Scenarios	Annual flow volume at watershed outlet (ML)	Reduction in flow volume
Existing OHD without pumping	33215	N/A
Existing OHD under 2.02 MLd^{-1} pumping	32898	0.95%
OHD with 31.5 mAHD spillway under 2.02 MLd^{-1} pumping	32619	1.79%
OHD with 33.6 mAHD spillway under 2.02 MLd^{-1} pumping	32352	2.6%

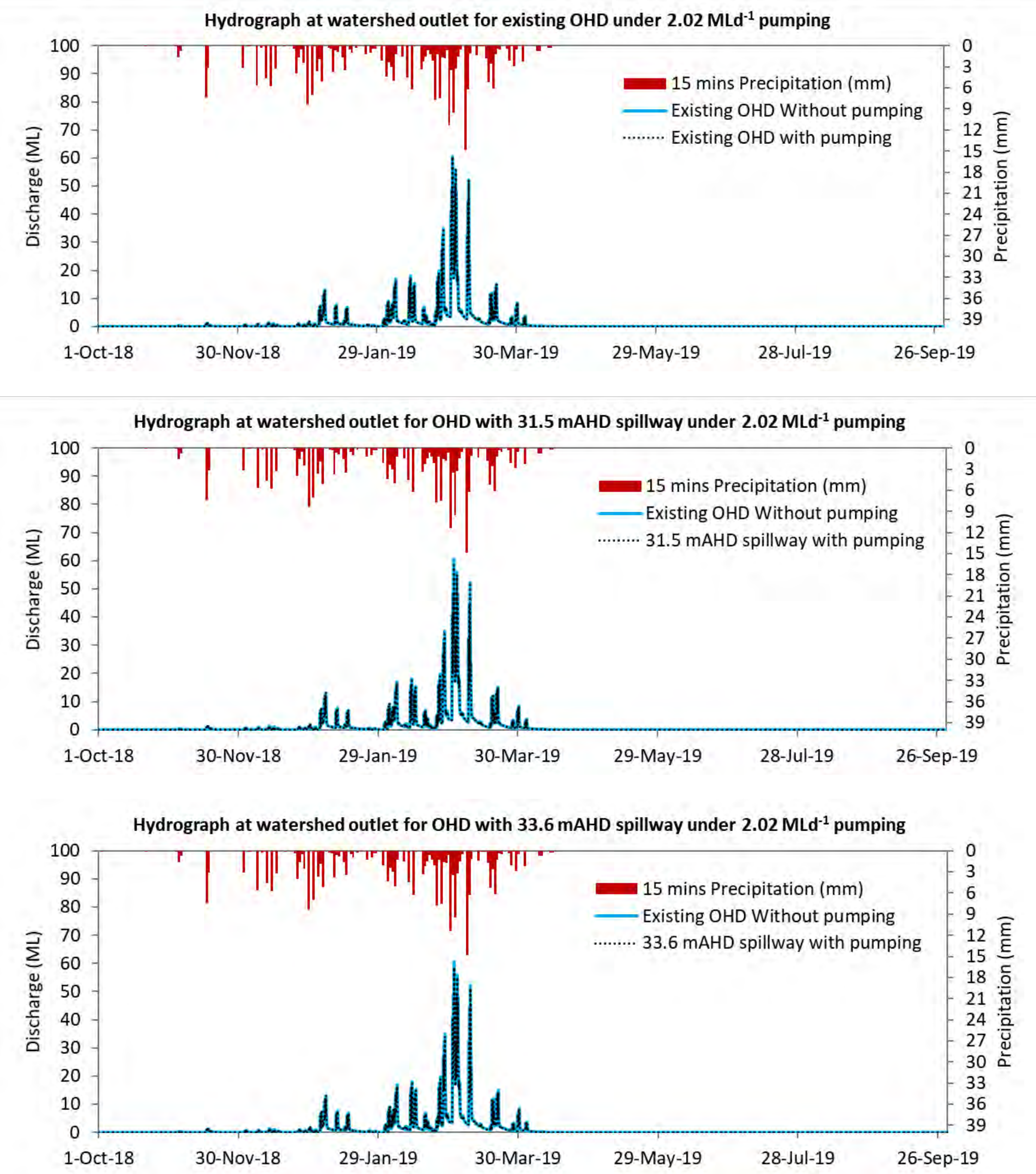


Figure 18. Compare hydrographs at the watershed outlet for different OHD spillway elevations under pumping with the hydrograph at the watershed outlet for existing OHD without pumping

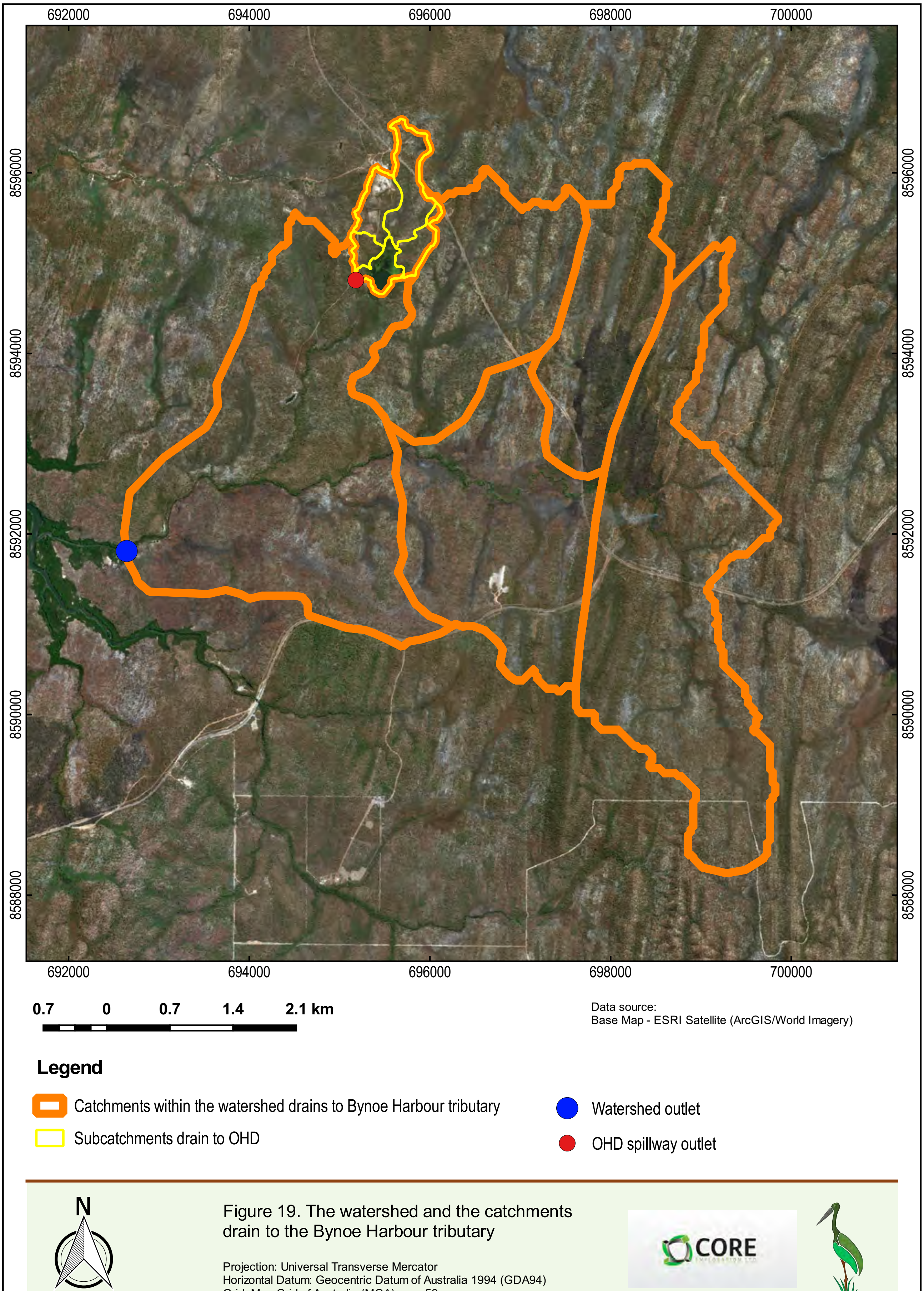


Figure 19. The watershed and the catchments drain to the Bynoe Harbour tributary

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

5.5 Summary

Simulations indicated the mine-site water usage and evaporation and/or seepage losses deplete OHD water volume to different levels and at different rates depending on the water use scenarios applied.

The 5-year simulations, with constant pump rates, indicate that for all spillway levels, should a low rainfall year occur during mining, there will be a deficit of water for mine applications. For the exiting OHD scenario, annual deficits range from 85 ML to 366ML. The 1-year simulation, for an average rainfall year, for the existing OHD indicates that for a pump rate of 0.64 MLd^{-1} in the wet, 1.2 MLd^{-1} in in the dry, water storage will be sufficient for mining operations, however, this does not take into consideration the effect of lower than average rainfall years. Apart from a wall lift or reduction in water usage, or in addition to these strategies, an alternative to secure mine application water requirements may be the construction of a second dam.

With respect to reduced flows downstream of the dam, spillway flow volume is reduced 36.8% when the pumping rate is 2.02 MLd^{-1} through an average rainfall year. The reduction in spillway flow volume was 100% for 33.6 mAHD spillway scenario and 69.1% of the 31.5 mAHD spillway scenario. However, for the larger sub-catchment that contains OHD, the maximum reduction in flow volume to the upper tidal reach of the Boyne Harbour was only 2.6% this was under the 33.6 mAHD spillway scenario.

6 Alternate water storage

Locations of ancillary water storage dams were assessed to enable continuous water supply for mining applications. The maximum required annual yield from an ancillary dam, based on the assessment of the OHD capacity, is 258 ML. The storage capacity needs to be larger than this number because the deficit normally happens in the dry season when no water enters the dam and water losses due to evaporation, and seepage occur. To estimate the required storage capacity of the alternate water storage, simulations were conducted for the existing OHD (364 ML) under different pumping rates. The simulations started in April with the dam full. The evaporation and seepage rates are the same as the models in Section 5. The simulations stopped when the water in the dam ran out. Pump extraction volumes and evaporation and seepage losses simulated in each simulation are shown in *Table 14*.

Table 14. Pump extraction volumes and evaporation and seepage losses during dry season for existing OHD

Pumping rate in dry season	Evaporation and seepage losses (ML)	Pump extraction volume (ML)	Ratio of evaporation and seepage losses to pump extracted volume
2.02 MLd^{-1}	93	271	0.34
1.2 MLd^{-1} (with pit dewatering)	122	242	0.50

According to the results (*Table 14*), the required storage capacity of an alternate dam is taken as 1.5 times 258 ML if the total deficit is made up by the water from ancillary dams. Therefore, under the worst-case scenario, the required storage capacity in an alternate dam is 387 ML.

6.1 Location and potential storage capacity of 2 alternate dams: Dam C4 and Dam C5

The analysis of catchments in the study area indicated 2 suitable catchments (Catchment C4 or Catchment C5) for building an ancillary dam (*Figure 20*).

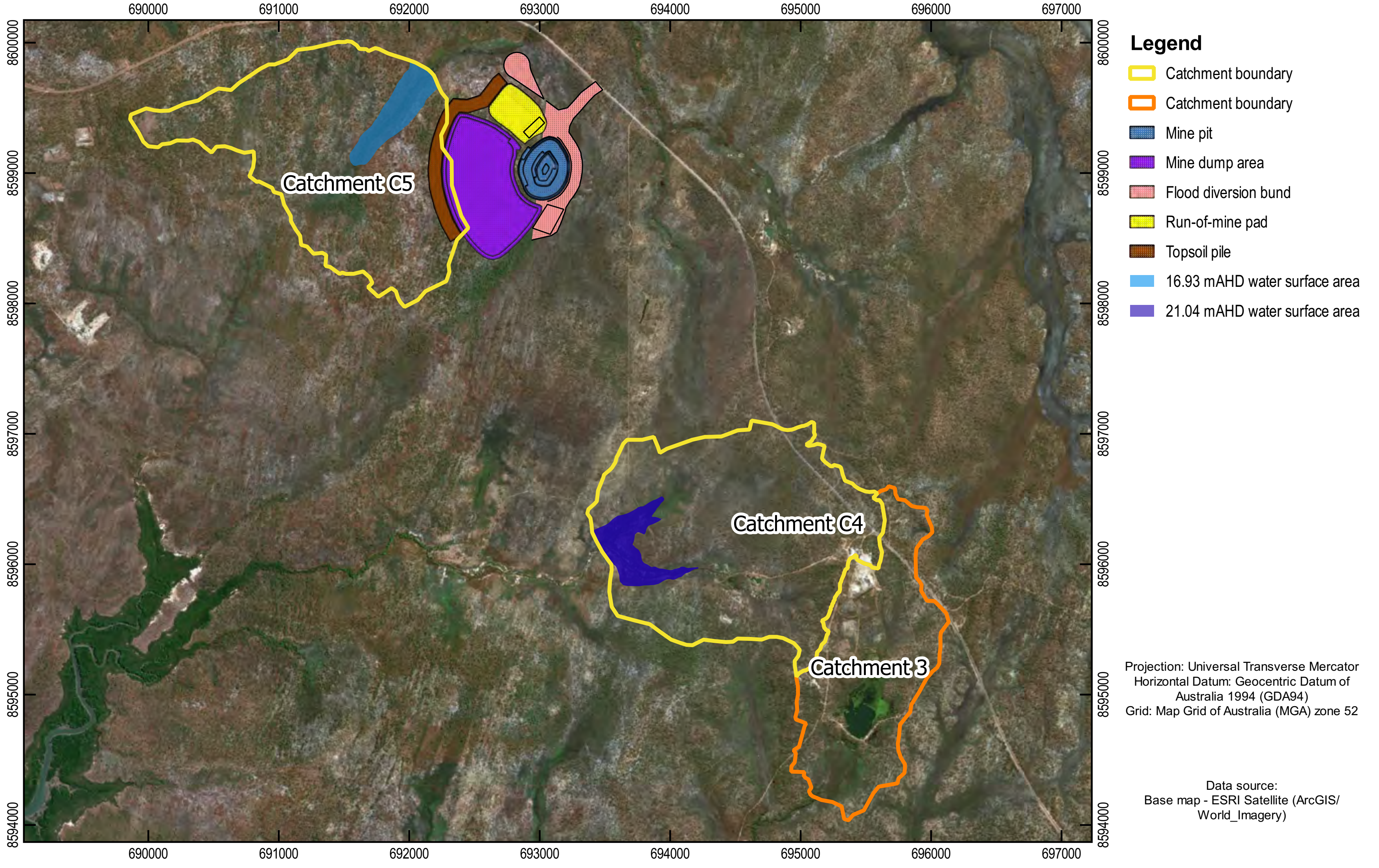


Figure 20. Location of alternate dams C4 and C5

The relationship between storage capacity and spillway level for the alternate dam C4 and C5 are shown in *Figure 21*.

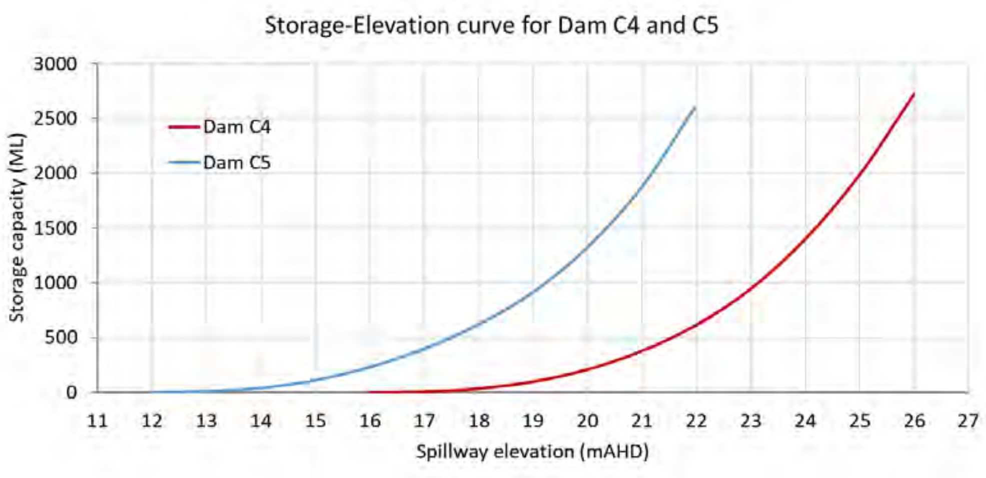


Figure 21. Variation of storage capacity with spillway level for the alternate dam C4 and C5

Based on the required storage capacity (387 ML) for an alternate dam. The minimum spillway levels for dam C4 and C5, to meet the deficit of water, are shown in *Table 15*. The water surface area for these spillway levels of dam C4 and C5 are shown in *Figure 20*.

Table 15. Minimum spillway levels for dam C4 and C5 to meet the deficit of water under the worst-case scenario.

Dam	Minimum spillway level to meet the required storage capacity of 387 ML (mAHD)
C4	21.04
C5	16.93

6.2 Dam C4 and C5 catchment inputs

HEC-HMS modelling was conducted to determine the amount of runoff draining to the dam C4 and C5 in low, average and high rainfall years. The results are shown in *Table 16*.

Table 16. The total volume of inflow to Dam C4 and C5 for low, average and high rainfall year scenarios.

Scenario	Total Inflow (ML)	
	Dam C4	Dam C5
Low rainfall year	2220	1601
Average rainfall year	3913	3397
High rainfall year	6617	5583

The simulations show that either site receives enough annual inflow to fill the proposed dam to the spillway level in a single wet season under the worst-case scenario of requiring the ancillary dam to provide 387 ML. This analysis is based on the existing OHD 30mAHD spillway. If a dam lift is used to increase the capacity OHD, a smaller ancillary dam could be used.

6.3 Influence of alternate Dam C5 on downstream flows

If an alternate Dam is constructed in catchment 5, the retention of surface flow and pumping could cause changes in downstream flows; these flows can be important to environmental values in downstream areas, especially where catchment outlets meet mangroves.

To investigate the impact of Dam C5 on downstream flows, the downstream catchments draining to Darwin Harbour tributaries under the influence of Dam C5 were determined (*Figure 23*). The existing HEC-HMS models for pre-mining and post mining catchments 2 and 5 were modified by adding the additional downstream catchments shown in *Figure 23*. A dam component with a spillway level of 16.93mAHD was also added to the post-mining models to simulate the Dam C5. In summary, 3 models were developed for the following scenarios:

1. Pre-mining: Undisturbed catchments. No dam component in the model.
2. Dam C5, no mine infrastructure: Undisturbed catchments. Dam component was added into the model.
3. Dam C5 and mine infrastructure: Catchments changed due to the change of drainage lines caused by mine infrastructure. Some runoff is retained in on-site sediment ponds and pit. The dam component was added into the model.

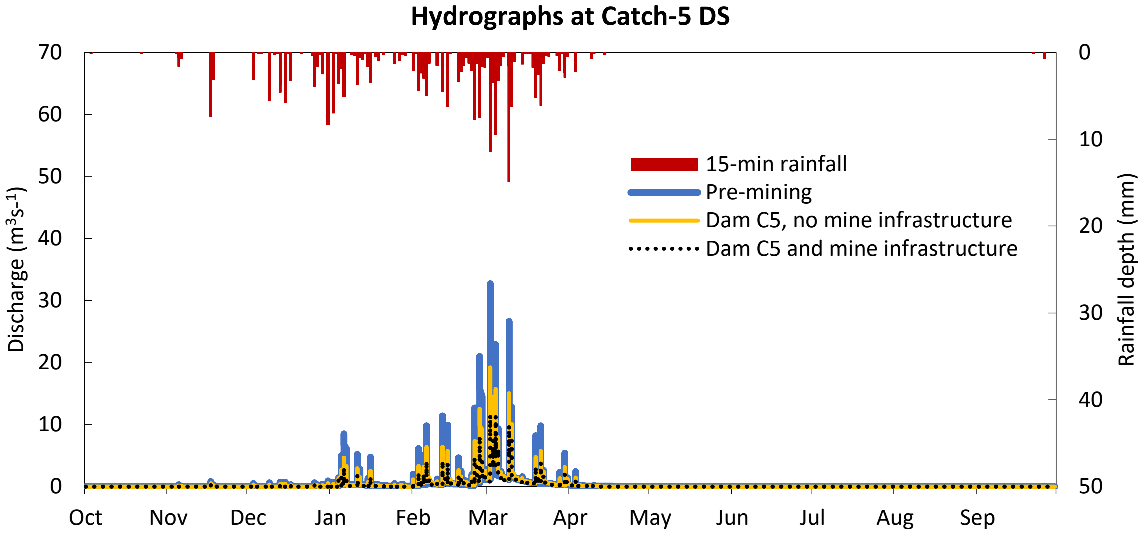
The models were run for an average rainfall year (a wet season followed by a dry season). Only the maximum pumping scenario (2.02 MLd^{-1} through the whole year) was investigated as this would provide a worst-case scenario for changes in stream flow. It was assumed the water in the dam was fully consumed during the last dry season, so the simulations started with no water in the dam.

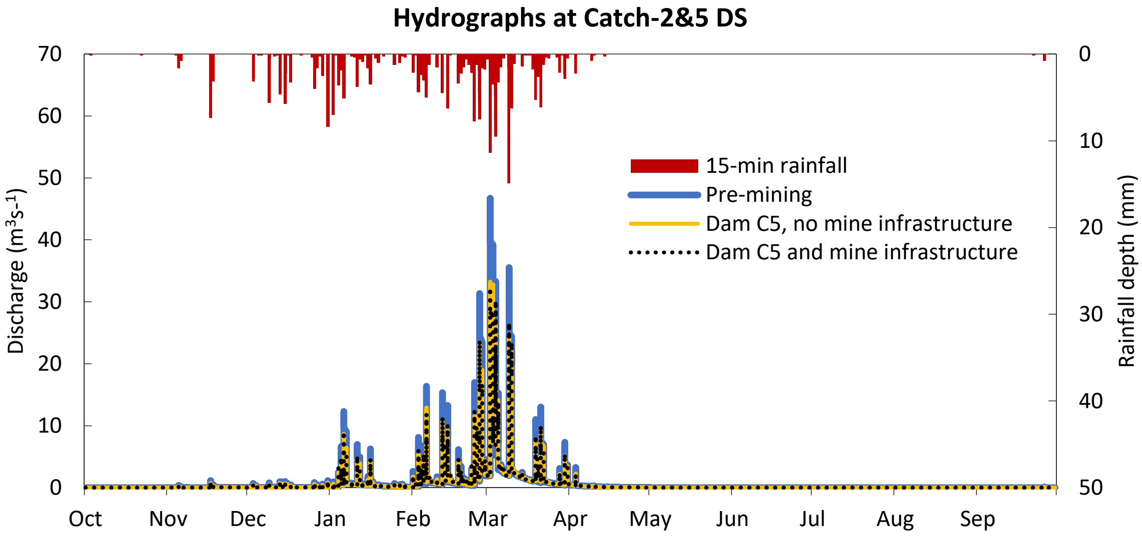
The reductions in total annual flow volume at the catchment 2 outlet, common outlet of catchment 2, and 5 and 5 other locations where catchment outlets meet mangroves (*Figure 23*) are in *Table 17*. The hydrographs at Catch-5 DS, Catch-2&5 DS and DS 5 (*Figure 23*) are in *Figure 22*. The hydrographs at locations further downstream are not shown since the difference between plots were too small to display.

Table 17 shows when the impact from mine infrastructure is not considered, the percentage reduction in downstream flows reduce progressively from 8.7% to 0.9% as other sub-catchments enter the system increasing total catchment area and discharge. When mine infrastructure is considered, the percentage reductions are from 37% to 1.4%. The 37% reduction in total flow at catchment 5 outlet is mainly a result of changes in stream lines due to the mine infrastructure (*Figure 9* shows some runoff is redirected from to catchment 5 to catchment 2 due to mine infrastructure).

Table 17. The total flow volumes at 7 locations downstream from proposed Dam C5 (Figure 23) and the reduction in total flow volume compared to the pre-mining condition.

Location	Pre-mining	Dam C5, no mine infrastructure		Dam C5 and mine infrastructure	
	Total flow (ML)	Total flow (ML)	Reduction in total flow (%)	Total flow (ML)	Reduction in total flow (%)
Catch-5 DS	9267	8460	8.7	7016	37.0
Catch-2&5 DS	16578	15771	4.9	15406	7.5
DS 5	22720	21913	3.6	21602	5.5
DS 4	33795	32988	2.4	32677	3.7
DS 3	60128	59321	1.3	59010	2.1
DS 2	66984	66177	1.2	65866	1.9
DS	89562	88756	0.9	88445	1.4





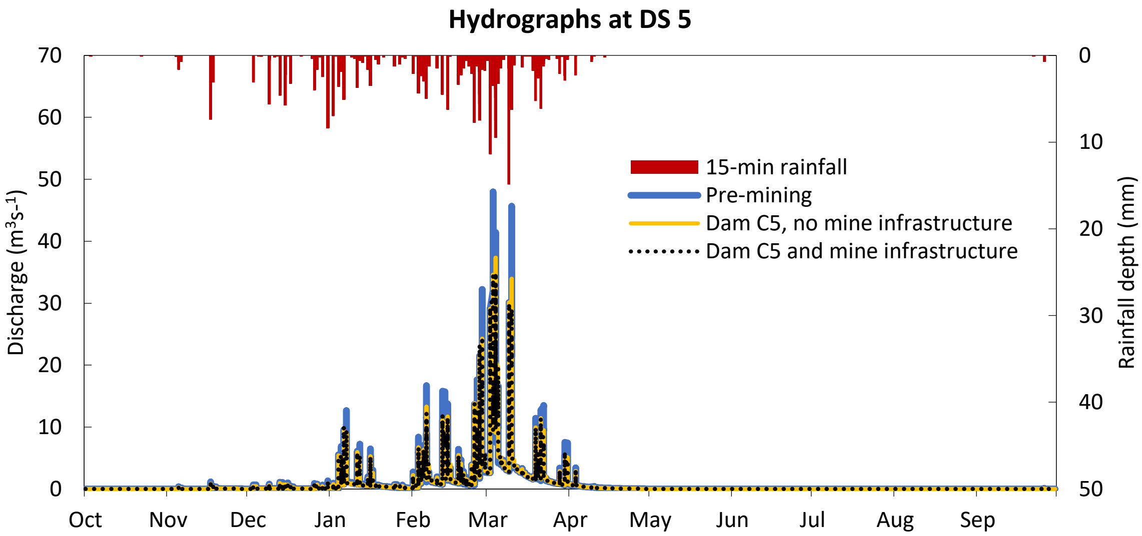
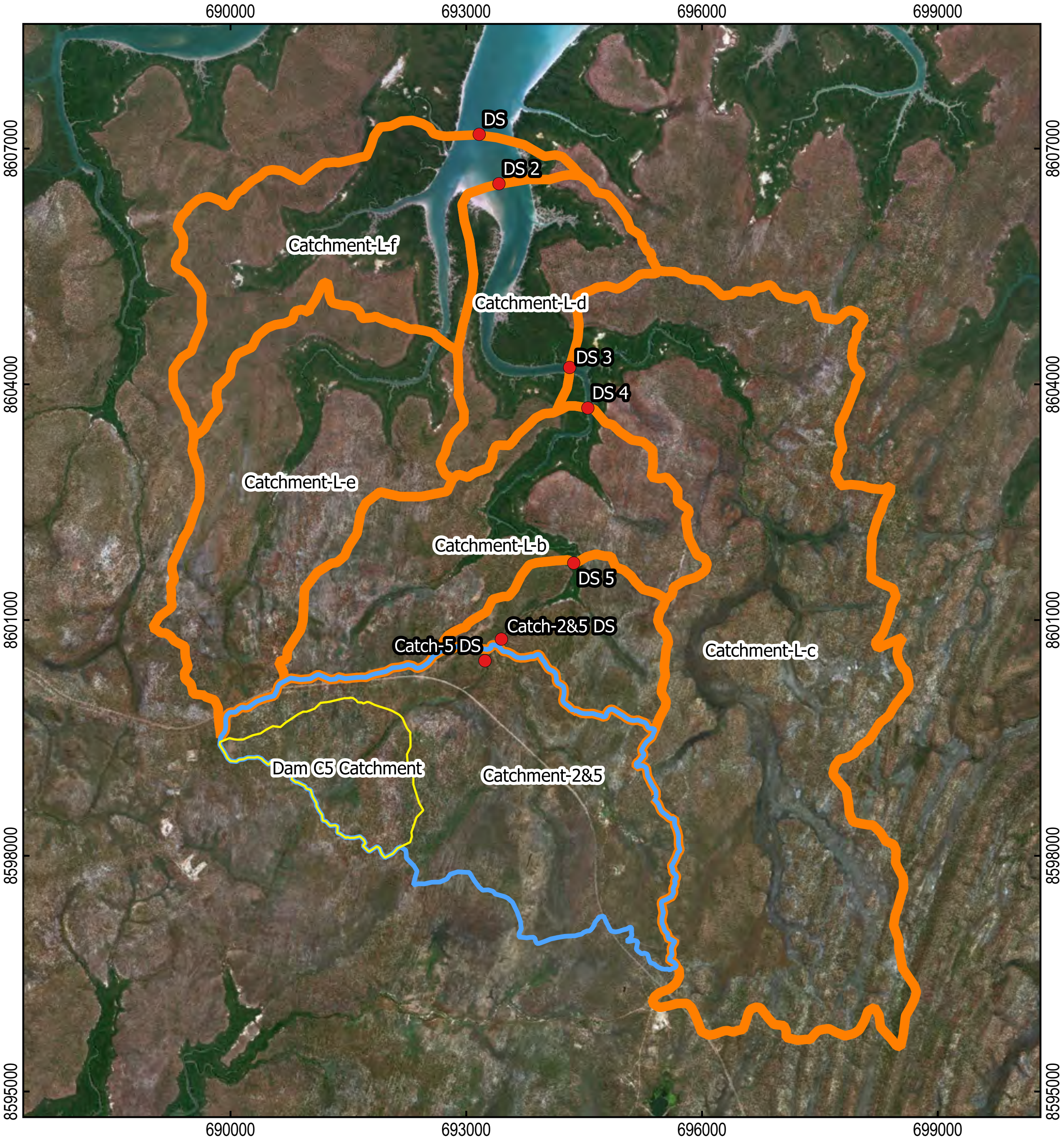


Figure 22. Hydrographs at Catch-2, Catch-2&5 and DS 5.



1 0 1 2 3 km

Data source:
Base Map - ESRI Satellite (ArcGIS/World Imagery)

Legend

- Catchments draining to Darwin Harbour tributary
- Darwin harbour catchments 2 and 5
- Catchment draining to Dam C5
- Location where total flow is calculated

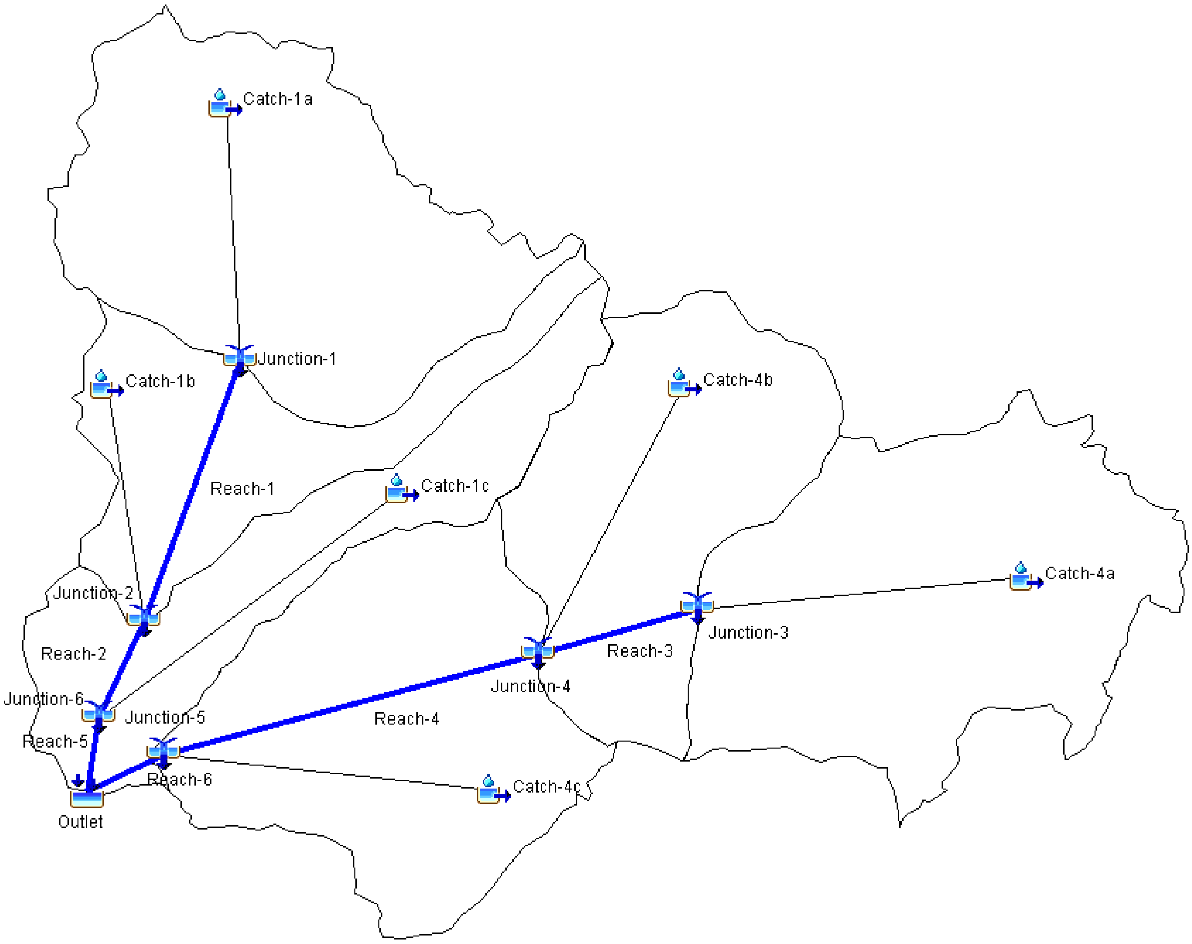
N

Figure 23. The watershed and the catchments draining to Darwin Harbour tributaries

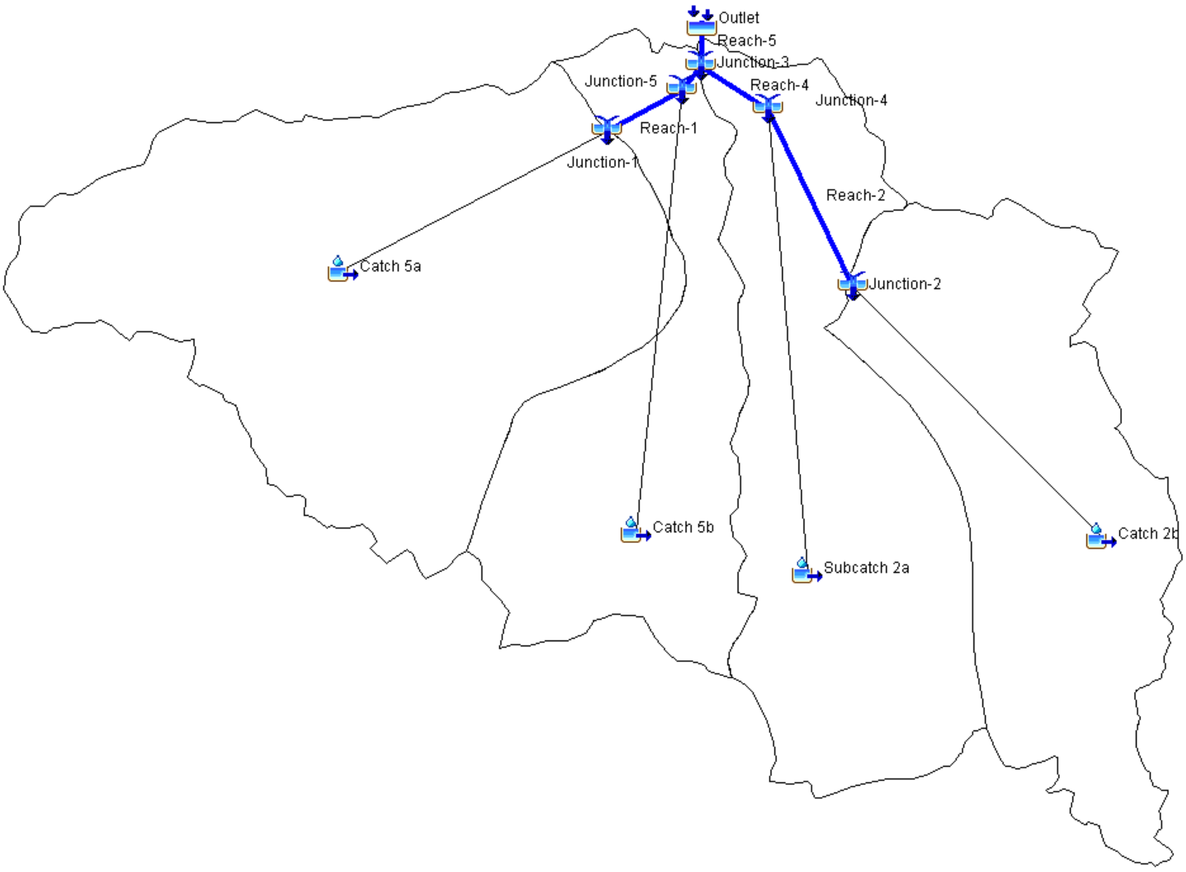
Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

Appendix A

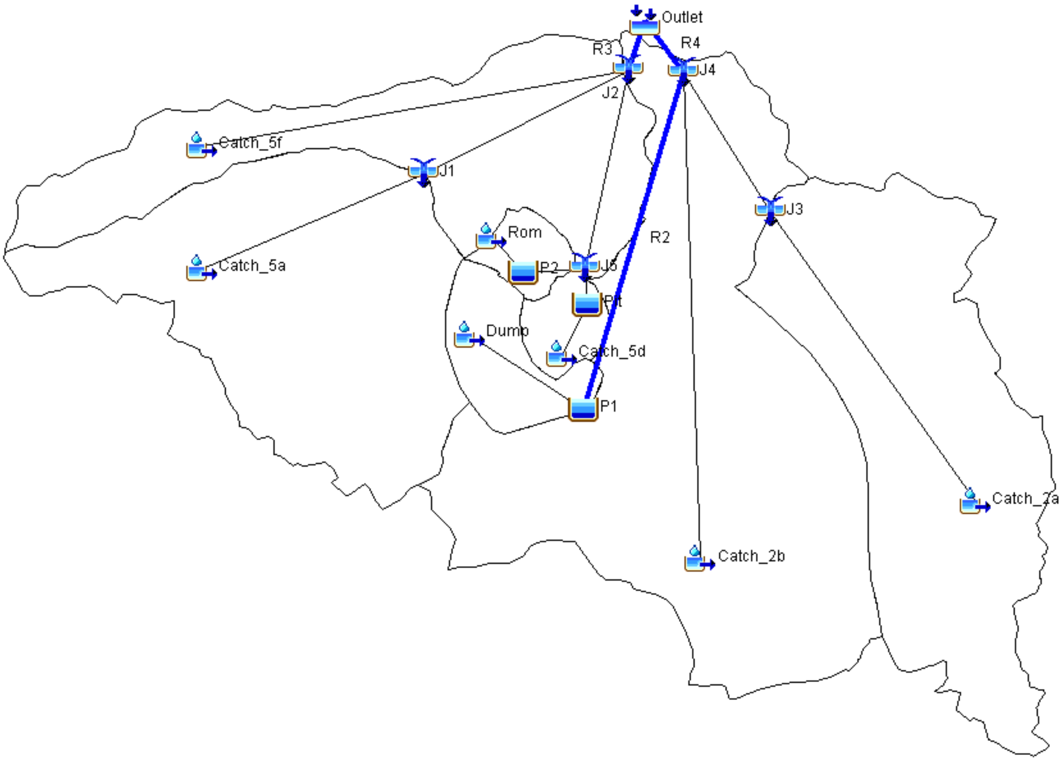
A1. Bynoe Harbour HEC-HMS model



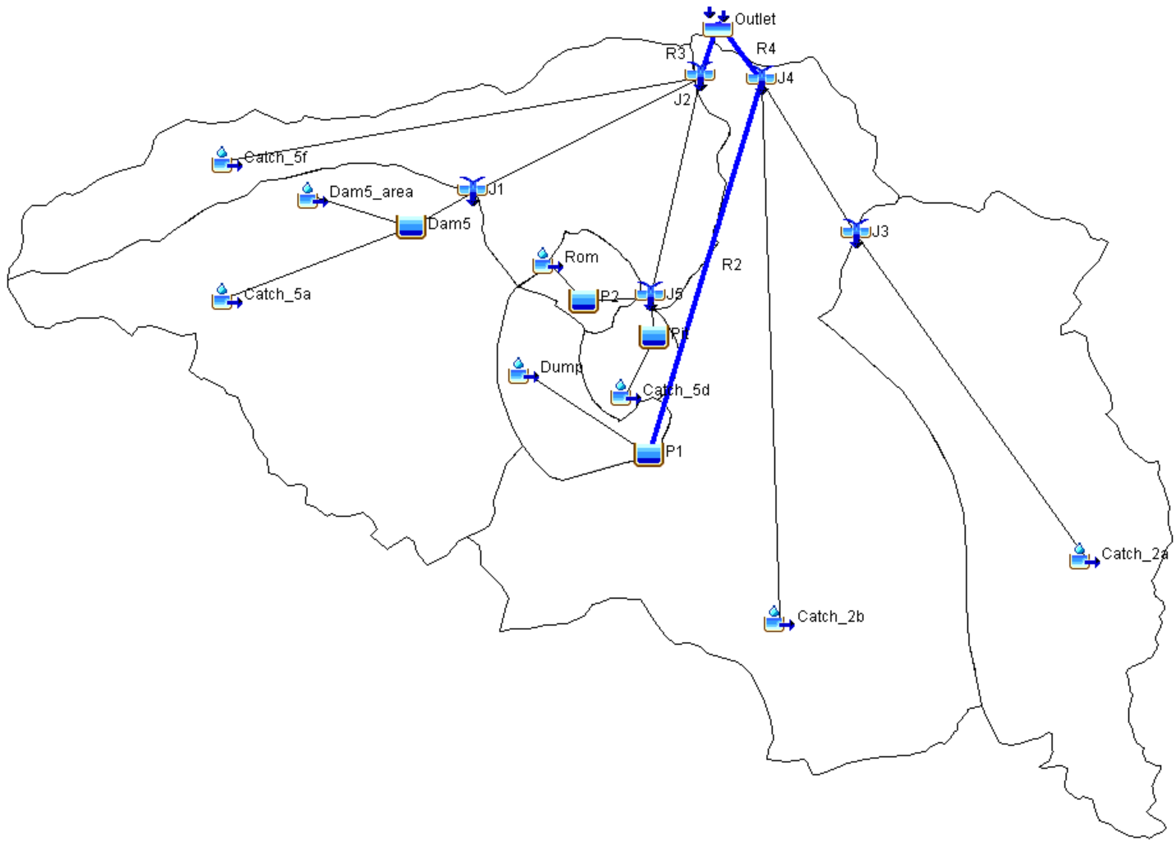
A2. Darwin Harbour pre-mining HEC-HMS model



A3. Darwin Harbour catchment post-mining HEC-HMS model (without dam C5)



A4. Darwin Harbour catchment post-mining HEC-HMS model (with dam C5)



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RELIANCE, USES and LIMITATIONS

This study, report and analyses have been based on the information available to and the assumptions made by EnviroConsult Australia Pty Ltd at the time of preparation. EnviroConsult Australia Pty Ltd does not take responsibility for errors and omissions in the conclusions, report and maps produced by this study due to incorrect information or any error in, or omission in connection with assumptions. No chemical analysis, groundwater hydrology, water quality or contaminant export studies of any kind were conducted. Any comment made in the report with respect to the above are speculative based on the surface hydrology analysis and should not be relied upon as fact.

Description	Author	Reviewer	Approved for Issue		
			Name	Signature	Date
Revision 0	Ken Evans Mike Liu Rojita Thapa	Ken Evans	Ken Evans		01/06/2018
Revision 1	Ken Evans Mike Liu Rojita Thapa	Ken Evans Aiden Campbell	Ken Evans		19/06/2018
Revision 2	Mike Liu	Ken Evans Aiden Campbell	Ken Evans		11/07/2018
Final			Ken Evans		01/08/2018

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Core Exploration Ltd, Cox Peninsula

Project 3: Mining Lease 31726 Flood Inundation Study

Report ECA-HA-0004-03

Core Exploration Ltd, Cox Peninsula

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Report ECA-HA-0004-03

Aug 2018

EnviroConsult Australia Pty Ltd

Executive Summary

This study assessed how mine infrastructure at the Core site, Cox Peninsula NT, might affect flooding in the area resulting from a 1%AEP rainfall event. This was done by assessing the inundation resulting from a 1%AEP rainfall runoff event in catchment 2 and 5 of the Darwin Harbour before mining and after mining with mine infrastructure of ponds, waste landform, pit, run of mine pad, drainage, bunds and haul road embankment in place.

Hydrological modelling

The rainfall and hydrograph for the 1%AEP model simulations was determined probabilistically using the Monte Carlo simulation feature of the RORBwin hydrology model. The simulations gave a critical rainfall duration of 6h for the event and the probable maximum peak discharge for the pre-mining condition as 118.9m^{3}s^{-1} and 115.0m^{3}s^{-1} for the post-mining condition, a change of 3%. For total Q there was a drop of 6% between the pre- and post-mining conditions. The change in peak discharge caused by the mine infrastructure is due to the ponds and the pit which are water retaining structures and, although the final depths of the ponds have not been designed, do not contribute to the total discharge under post-mining conditions.

Flood inundation

Flood inundation was modelled using HEC-RAS 2D with the rainfall and runoff hydrographs for node inputs derived using RORBwin.

The following 4 scenarios were simulated:

1. Local inundation in the pre-mine catchments 2 and 5 for the 1%AEP rainfall event,
2. Local inundation in the post-mine catchment 2 and 5 for the 1%AEP rainfall event,
3. Local inundation in the pre-mine catchment for the 1%AEP flood event occurring at the same time as primary storm surge in Darwin Harbour, and
4. Local inundation in the post-mine catchment for the 1%AEP flood event occurring at the same time as primary storm surge in Darwin Harbour

For local inundation only, the area of inundation is less for the post-mining scenario because of the pit and pond.

The simulations indicate, that near culvert 1, Cox Peninsula road is inundated for a shorter period for post-mine conditions (6.1 hrs) than for the pre-mine conditions (7.0 hrs). The maximum water depth above the road surface at the location of this culvert is 0.38m for pre-mine and 0.34m for the post-mine scenario. At culvert 2, simulations indicate that Cox Peninsula Road is inundated for a shorter period for post-mine conditions (3.4 Hrs) than for the pre-mine conditions (3.5 Hrs). The maximum water depth above the road surface at the location of this culvert is 0.29m for pre-mine and 0.27m for post-mine scenarios.

For local inundation occurring at the same time as primary storm surge in Darwin harbour there is negligible effect of the surge on inundation levels for both the pre- and post-mining conditions. The results indicate that water availability for ecological processes on the downstream floodplains is not affected. Changes in water quality has not been assessed.

The initial design of the haul road embankment did not have a culvert beneath the road so, the final assessment was a simulation of the post-mining catchment with a culvert in the haul road bund. The haul road culvert was simulated using the same dimensions as culvert 2 but had not been designed. The purpose of this simulation was to assess how the culvert prevented water ponding at the toe of the haul road embankment. Simulations showed no real change in maximum inundation extent, but ponding areas were reduced.

In summary the mine infrastructure has very little effect on inundation in the affected catchments and reduces the time of inundation on Cox Peninsula Road during floods.

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1 Introduction

The Core Exploration Ltd (Core) Lithium resource (Grants Project) is located at 695000m East, 8596000m North, GDA94/MGA52, near Berry Springs, Northern Territory. This report focuses on ML(A) 31726 and possible inundation in the catchments where the proposed mine is to be located (*Figure 1*).

EcOz Environmental Consultants commissioned EnviroConsult Australia Pty Ltd to undertake three projects relating the surface water hydrology of Grants Project:

- Project 1: Description of hydrological conditions of site and calibration of hydrological model,
- Project 2: Application of hydrological model to complete a hydrological assessment and water balance, and
- Project 3: Inundation modelling of the site.

Projects 1 and 2 have been completed by EnviroConsult Australia Pty Ltd. This report describes Project 3

Project 3 assesses flood inundation of the site pre-mining and post-mining and focuses on Darwin Harbour catchments 2 and 5.

A 1%AEP (Annual Exceedance Probability) rainfall event was used. two models are used for this analysis. The first is RORBwin hydrology model which uses Monte Carlo simulations to determine rainfall and runoff for the event causing the peak discharge (Q) for the studied AEP event. The second is the HEC-RAS hydrodynamic model which uses the RORBwin output hydrographs to simulate flood inundation modelling.

1.1 Methodology

The steps undertaken to complete Project 3, based on the results of Projects 1 and 2, are listed in the following.

1. Determine the hydrology of the mine infrastructure catchments for a 1%AEP rainfall event,
2. Determine the hydrology of the undisturbed catchments for a 1%AEP rainfall event,
3. Compare the extent of inundation caused by a 1%AEP event in the pre- and post-mining catchments, and
4. Determine the effect of local inundation on the site should a 1%AEP rainfall event occur in the mine infrastructure catchments at the same time as a primary storm surge in the downstream Darwin Harbour estuaries.

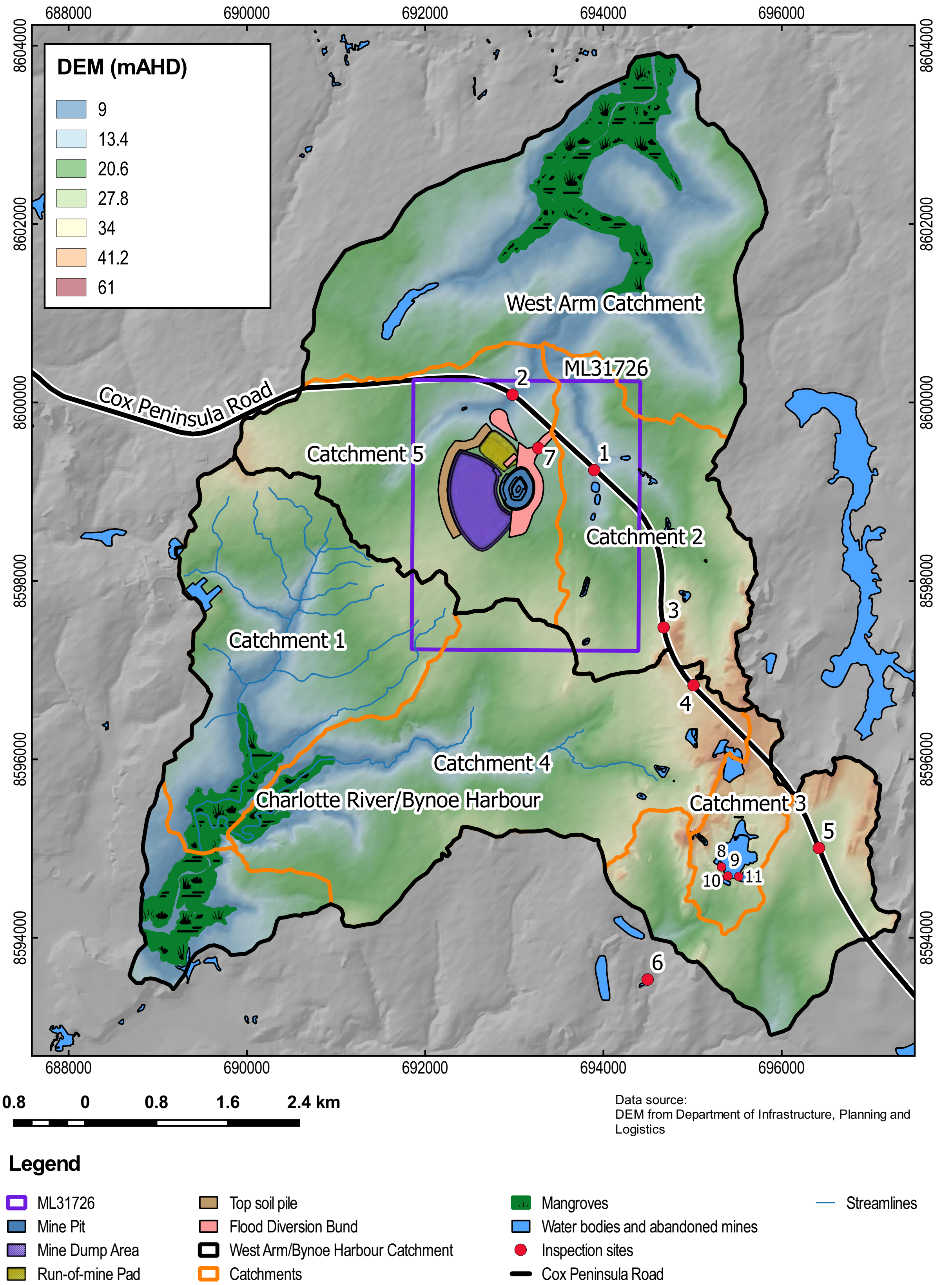


Figure 1. Location of ML(A) 31726 and the stream catchments analysed in this project.

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

2 Methods

2.1 Digital Elevation Model

A digital elevation model (DEM) for the mine infrastructure comprising pit, waste landform (WL), ponds, run of mine pad (RoM), and drainage was provided by Core Exploration Ltd. This was previously merged with the existing Cox Peninsula 2m DEM in Project 2.

2.2 Derivation of the 1%AEP hydrograph

A 1%AEP (annual exceedance probability) rainfall event is a recommended design event for major developments and roadways. For ungauged catchments a probabilistic method to determine storm duration and intensity is recommend. RORBwin uses Monte Carlo simulations to determine critical duration and temporal distribution of the rainfall event causing the probable peak Q for the design event in the catchment of interest.

2.2.1 RORBwin

RORBwin was used to determine the hydrograph for a 1%AEP rainfall event at the various locations in catchments 2 and 5. This hydrograph was used as an input for the HEC-RAS 2D model to determine the inundation caused by the event.

RORBwin is a rainfall excess model which generates flow with the application of excess rainfall over defined sub-catchment areas. The significant parameters in RORBwin are 1) Initial Loss (IL), which is abstracted from the beginning of the rainfall event; 2) Continuing Loss (CL), which is applied at a constant rate over the duration of the rain event, 3) the routing parameter k_c; and 4) the hydrograph exponent m (recommended value 0.8). These parameters' values are fitted during a calibration process.

Values for k_c can be derived empirically using the following equation for humid zones, north of latitude 15ºS, where Se is equal slope (m/km), and A is area (km^2):

$$k_c=1.8(A/Se^{0.5})^{0.55} \qquad (1)$$

RORBwin, uses Bureau of Meteorology intensity frequency duration curves for the site for the Monte Carlo simulations.

The model was calibrated to Gulungul Creek monitoring data using the method described in Report 1.

To create the RORBwin catchment model, the mine infrastructure catchments, catchments 2 and 5 as described in Projects reports 1 and 2 were divided in to 33 sub-catchments for the pre-mining condition and 25 sub-catchments for the post-mining condition (**Appendix A1.** RORBwin model for Catchments 2 and 5: pre-mining and **Appendix A2.** RORBwin model for Catchments 2 and 5: post-mining). The lesser number of sub-catchments in the post-mining model is due to the internal drainage in the mine infrastructure. In this model the waste landform (WL), Run of Mine pad (RoM), ponds and pit are not considered as the WL and RoM drain to ponds and flow doesn't exit the site. The pit receives direct rainfall which is retained in the pit.

The 1%AEP rainfall event was run for the pre- and post-mining model and hydrographs produced at the nodes (*Figure 2* and *Figure 3*) required for HEC-RAS 2D modelling.

2.3 Inundation modelling

HEC-RAS was used to model flood inundation on the site. The following 4 scenarios were simulated:

1. Local inundation in the pre-mine catchments 2 and 5 for the 1%AEP rainfall event,
2. Local inundation in the post-mine catchment 2 and 5 for the 1%AEP rainfall event,
3. Local inundation in the pre-mine catchment for the 1%AEP flood event occurring at the same time as primary storm surge in Darwin Harbour, and
4. Local inundation in the post-mine catchment for the 1%AEP flood event occurring at the same time as primary storm surge in Darwin Harbour

The model meshes were constructed from available topographic and bathymetric data. The mesh for post-mining scenarios were constructed from the pre-mining DEM merged with mining infrastructure created from the design drawings supplied by Core. Catchment hydrographs from the RORBwin Monte Carlo simulations were used as inputs. The HEC-RAS model meshes are mainly made up of 15m×15m squares which uses vertexes to capture ground elevations. Other model components and model inputs at chosen locations for each simulation are described in following sub-sections.

2.3.1 HEC-RAS model mesh and model inputs for 1%AEP rainfall event pre- and post-mine

The HEC-RAS meshes are developed for pre- and post-mine scenarios to cover the expected inundation extent. The input hydrographs at selected nodes are generated by the RORBwin model. The model mesh and input nodes for pre-mine scenario is shown in *Figure 2*. The input hydrographs for each node in *Figure 2* are shown in *Figure 8*. The model mesh and input locations for post-mine scenario is shown in *Figure 3*. The input hydrographs for each node in *Figure 3* are shown in *Figure 9*.

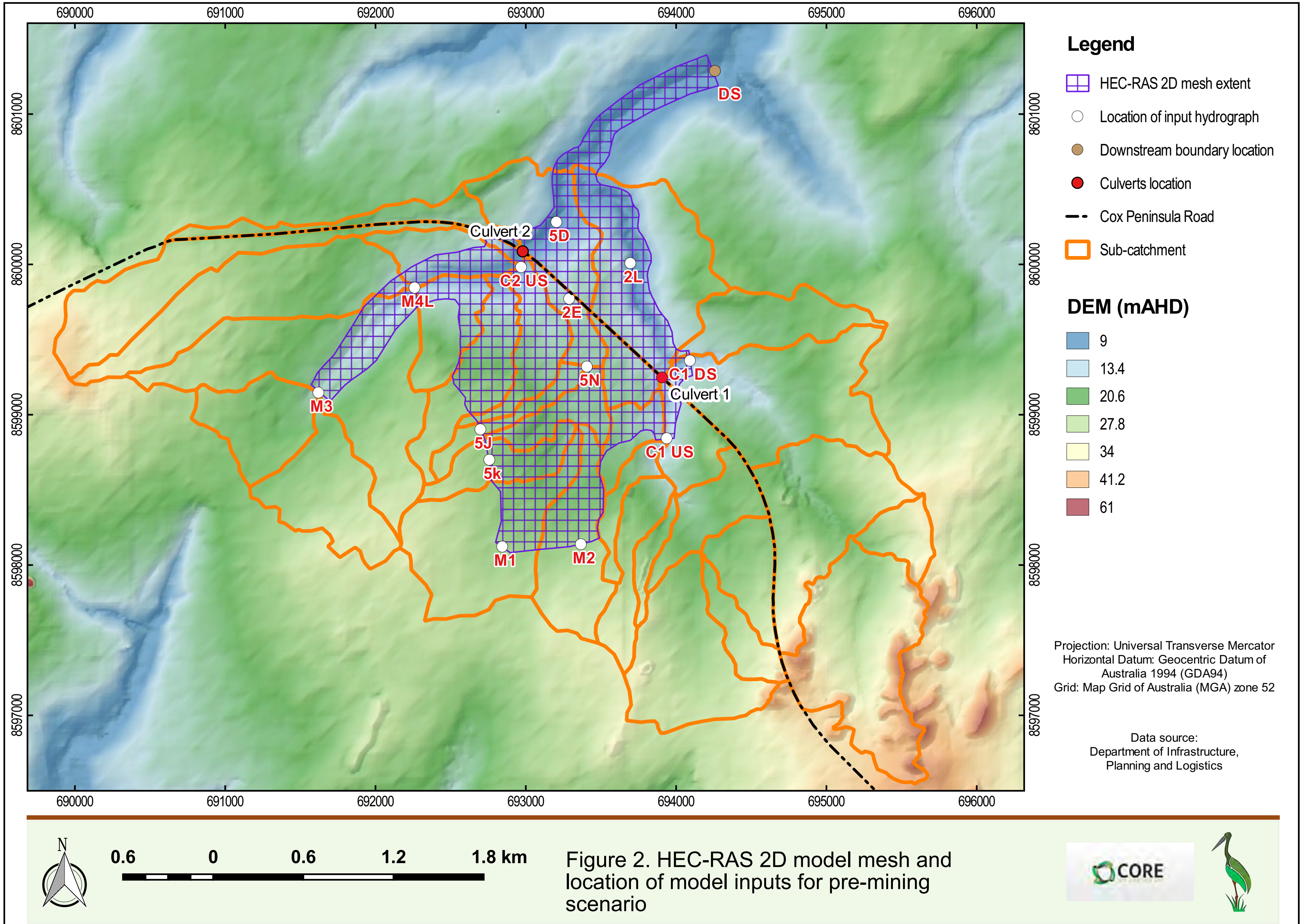


Figure 2. HEC-RAS 2D model mesh and location of model inputs for pre-mining scenario

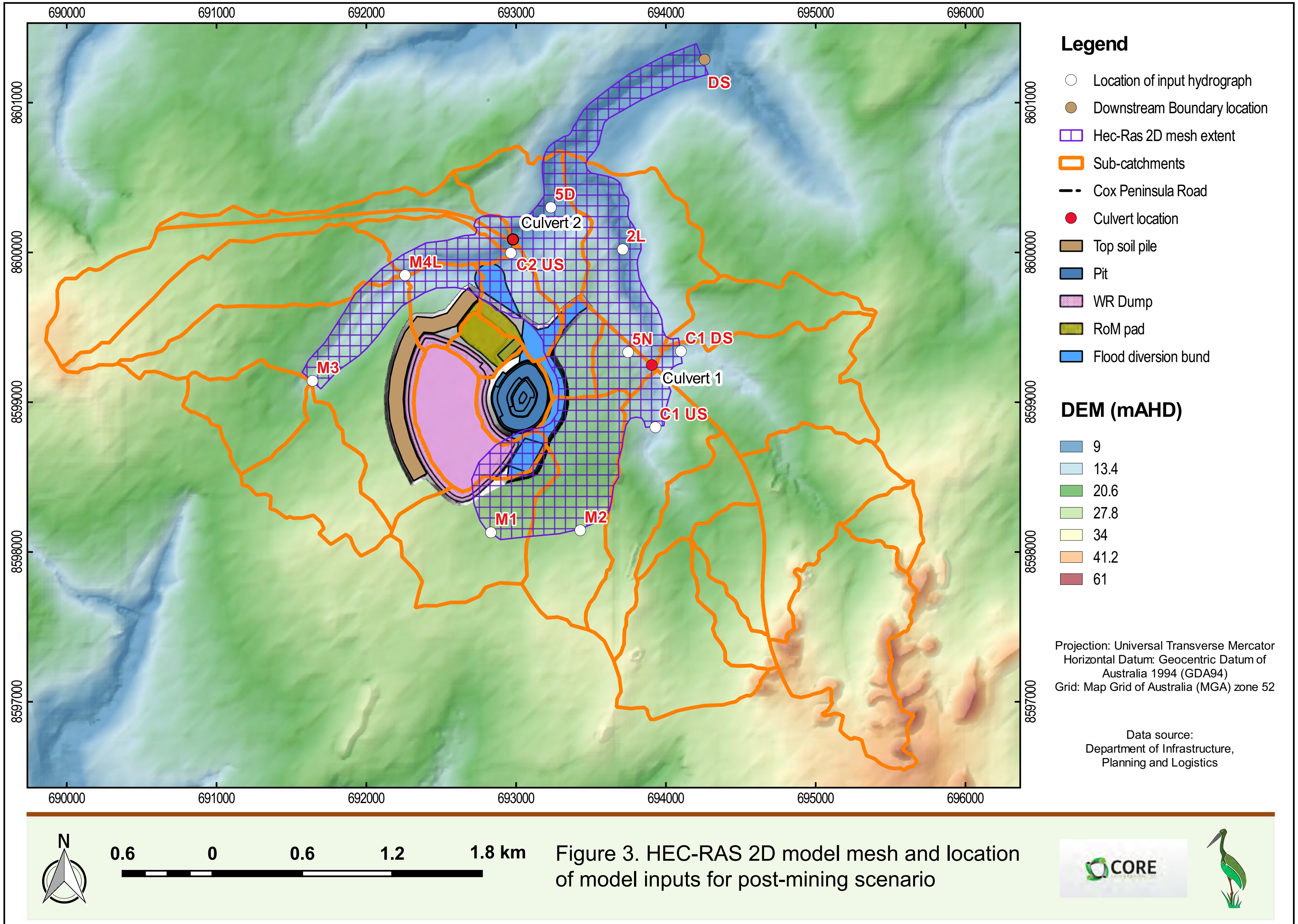


Figure 3. HEC-RAS 2D model mesh and location of model inputs for post-mining scenario

2.3.2 Hydraulic structures – culvert 1 and 2.

The culverts affect mine site inundation are culvert 1 and 2 (*Figure 1*). Culvert 1 (**Appendix B1**) is where catchment 2 intersects the Cox Peninsula Road (It comprises 3 box culverts 920mm high and 1230mm wide. Culvert 2 (**Appendix B2**) is where catchment 5 intersects the Cox Peninsula Road. It comprises 2 box culverts 1820mm high and 2420mm wide. These two culverts were integrated into each model mesh using the culvert component of HEC-RAS (*Figure 4*). The water levels before and after and flow through each culvert were simulated for inundation analysis.

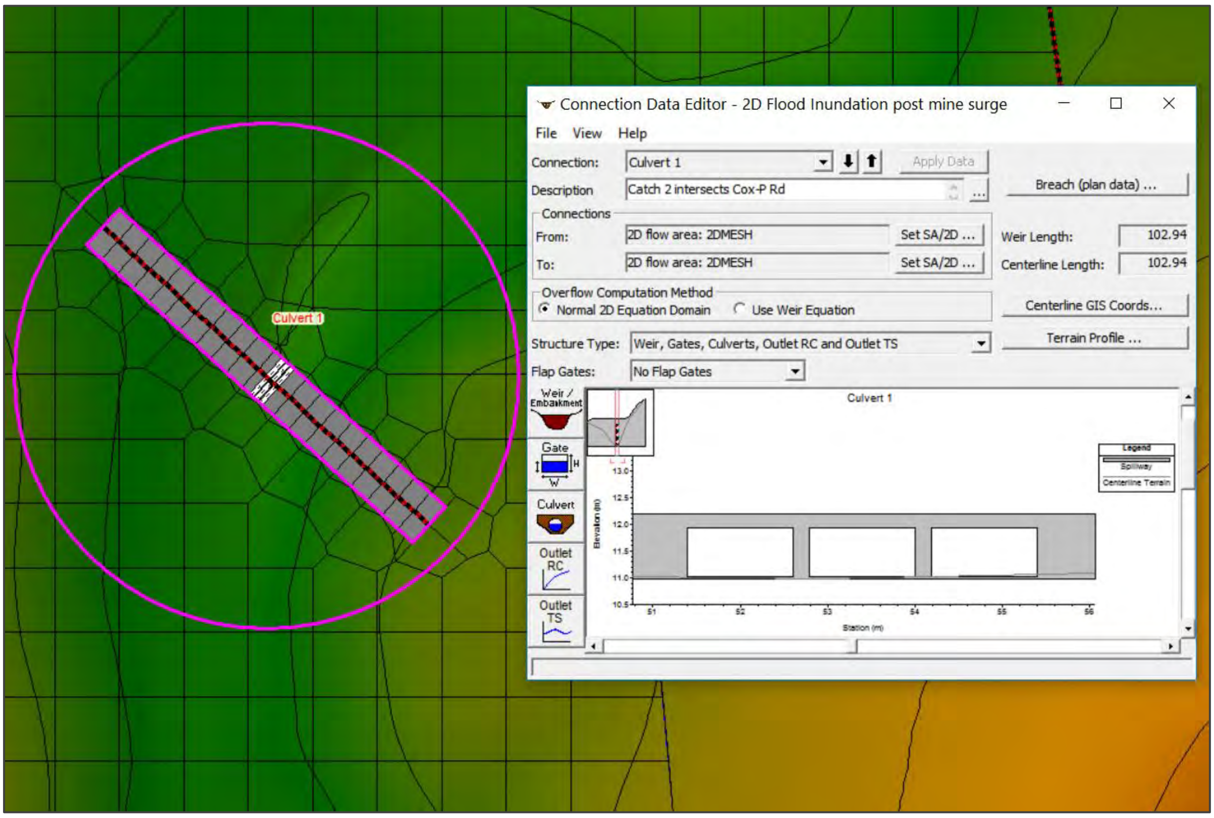


Figure 4. Culvert component in HEC-RAS.

2.3.3 Modification to model meshes and inputs for the 1% AEP flood occurring at the same time as primary storm surge in Darwin Harbour

To simulate the inundation area caused by 1% AEP flood occurring at the same time as primary storm surge in Darwin Harbour, the downstream boundary of the pre- and post-mining meshes described above were extended to the mouth of the tributary linked to the Darwin harbour. To simulate the condition that the flood peak coincides with the storm surge peak, the downstream boundary condition was set as a constant water surface elevation of 4.7 mAHD which is the estimated peak level of primary storm surge in Darwin Harbour. No changes were made to the inputs at other locations. The modified model mesh and input locations are shown in *Figure 5* and *Figure 6*.

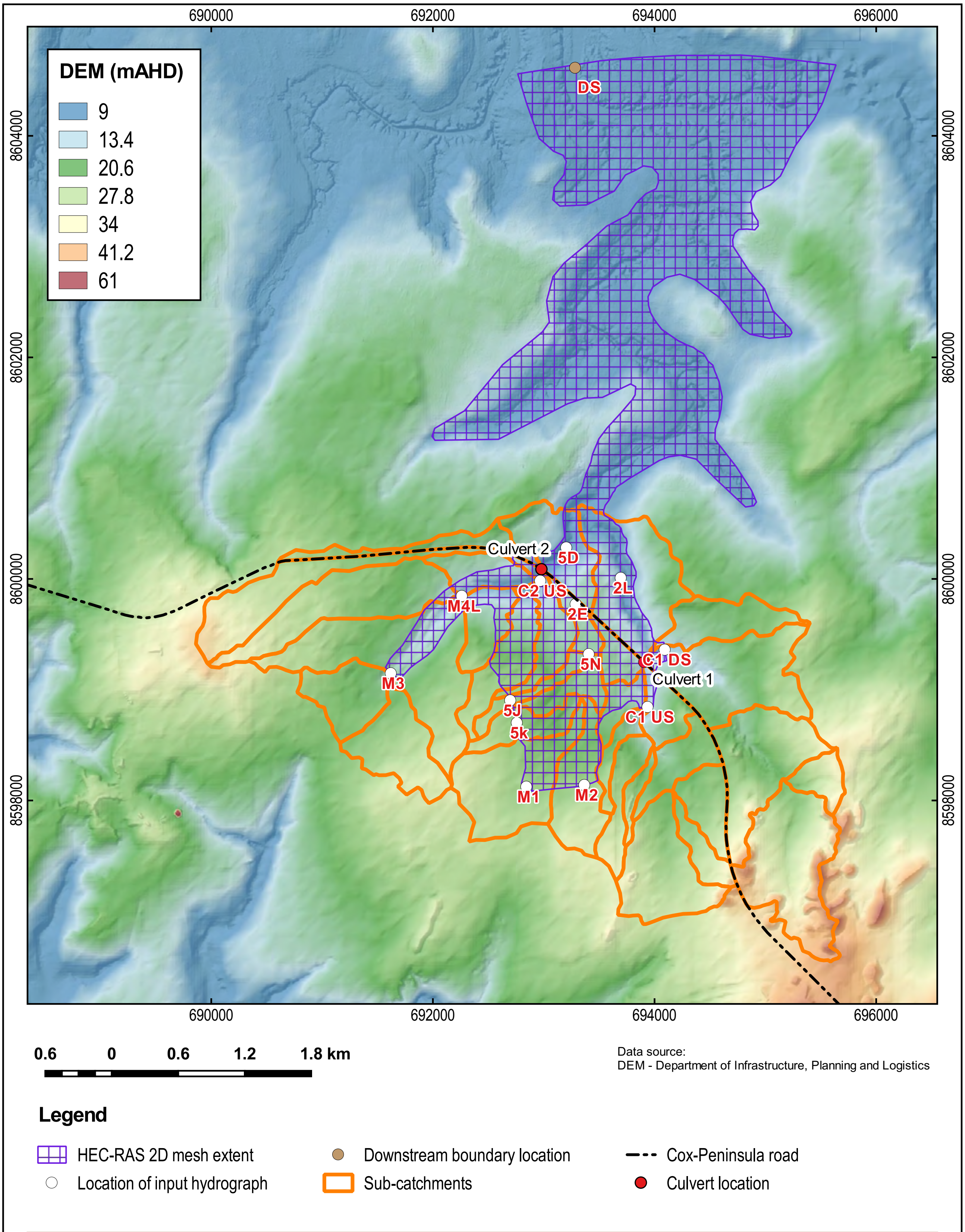


Figure 5. Modified pre-mining model mesh for incorporating the influence of primary storm surge

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

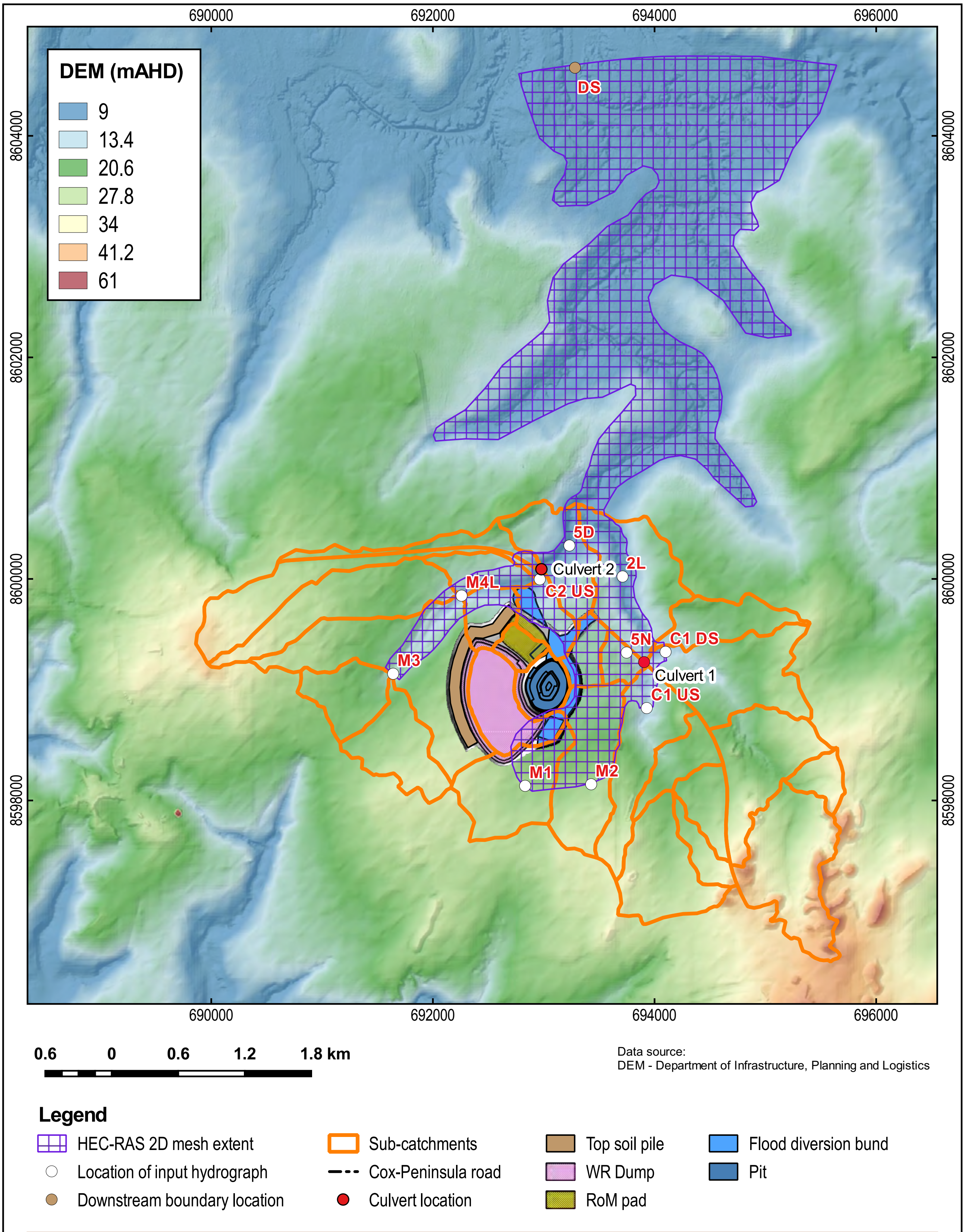


Figure.6 Modified post-mining model mesh for incorporating the influence of primary storm surge

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

2.3.4 Model roughness parameter

The roughness parameter describes the ground surface condition which affects the behaviour (velocity and depth) of flow in the model. A previous study has proved that using 2 roughness values to represent the roughness of channel and floodplain is sufficient for HEC-RAS flood inundation modelling[1]. The roughness parameter used in HEC-RAS is Manning's n. The spatial distribution of Manning's n values in HEC-RAS is represented by inputting a land cover map with Manning's n zones delineated using satellite photography. The land cover map is shown in *Figure 7*. The Manning's n values are set as 0.06 for channel and channel bank and 0.10 for floodplain and other regions. These values were selected from the typical ranges of 2D roughness parameters provided in Australian Rainfall and Runoff (ARR) report[2].

[1] Zhu L, Venkatesh M & Keighobad J 2018. Investigating the role of model structure and surface roughness in generating flood inundation extents using 1D and 2D hydraulic models. *Journal of Flood Risk Management.*

[2] ARR 2012. Project 15: Two-Dimensional Modelling in Urban and Rural Floodplains. Engineers Australia. Report P15/S1/009.

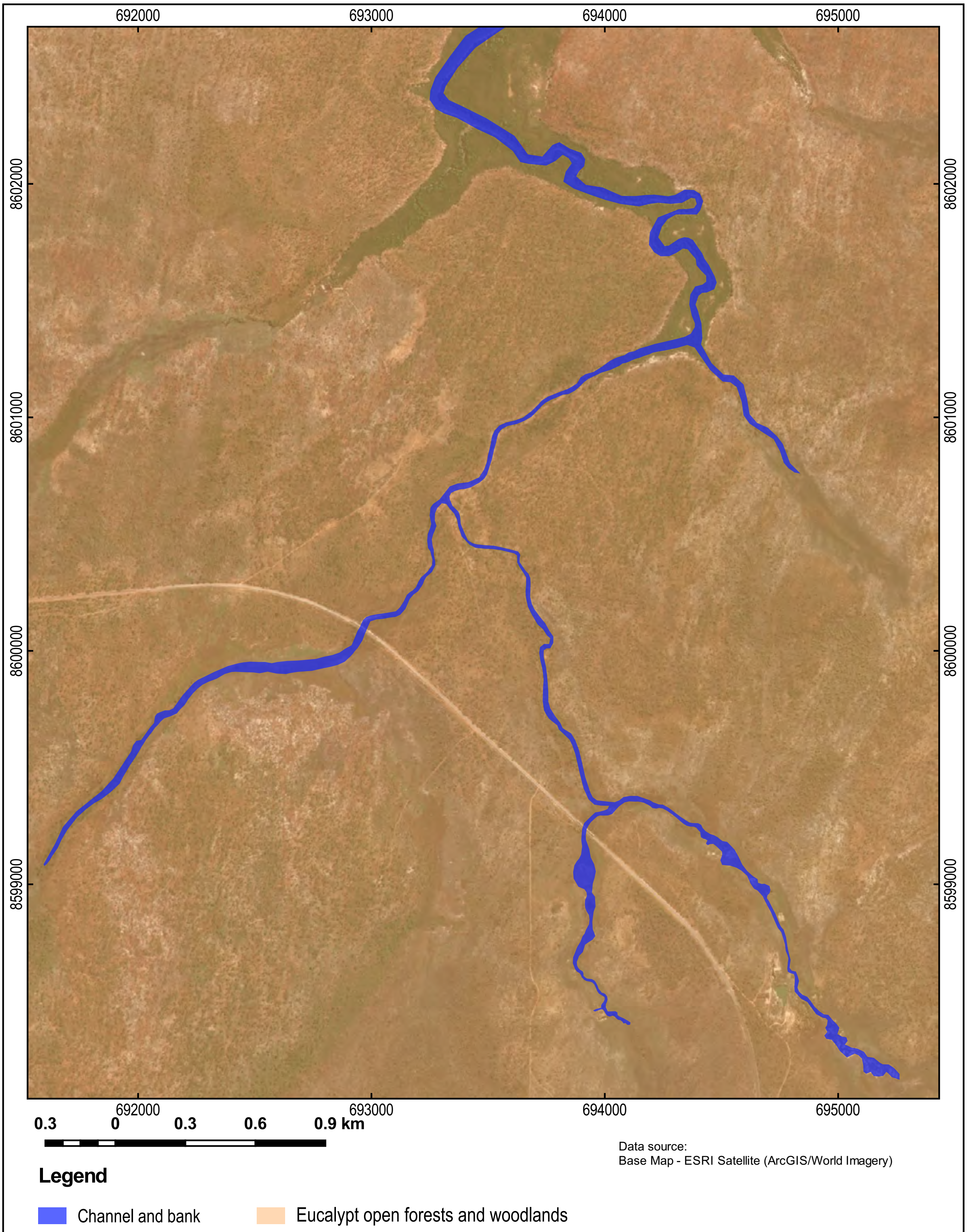


Figure 7. Land cover map used by HEC-RAS

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94) Grid:
Map Grid of Australia (MGA) zone 52

3 Results

The results of the RORBwin and HEC-RAS modelling are represented in the following.

3.1 RORBwin modelling

The calibrated RORBwin parameter values were IL = 15mm, CL = 3.1mmh^{-1}, k_c = 4.22, and m = 0.8. The empirically derived Se value was 8.15mkm^{-1}.

The RORBwin simulated 1%AEP event peak Q and total Q for the HEC-RAS 2D nodes for pre- and post-mining conditions are given in *Table 1*. The differences in pre- and post-mine peak discharges for the same nodes because the mine infrastructure affects the area of sub-catchments draining through those nodes.

The RORBwin Monte Carlo simulations gave the critical rainfall duration of 6h for the 1%AEP event. RORBwin simulated peak Q at the DS node (*Figure 2* and *Figure 3*) as 118.90m^{3}s^{-1} for pre-mine scenario, and 115.0m^{3}s^{-1} for post-mine scenario, a change of 3%, and a time to peak Q as approximately 2h. Total Q at the outlet of catchments 2 and 5 for the 1%AEP event is 2090ML for pre-mine scenario and 1970ML for the post-mine scenario, a drop of 6% between the pre- and post-mining condition.

The RORBwin simulated rainfall hyetographs and their resulting hydrographs for sub-catchments as they combine downstream for the 1%AEP event are shown in *Figure 8* and *Figure 9*. The upper hyetograph is the rainfall depth per 15-min interval and the continuous hydrograph are those simulated by RORBwin Monte Carlo simulations for the probable peak Q of the event.

These hydrographs are used as input to the HEC-RAS 2D inundation model to assess local inundation as a result of 1%AEP rainfall event and the 1%AEP rainfall event occurring at the same time as primary storm surge.

Table 1. RORBwin simulated total Q and peak Q for the pre- and post-mining HEC-RAS 2D nodes for the 1%AEP event.

HEC-RAS 2D Node	Pre-mining			Post-mining		
	Area (km^2)	Peak Q (m^3s^{-1})	Total Q (ML)	Area (km^2)	Peak Q (m^3s^{-1})	Total Q (ML)
M1	0.741	11.16	114	0.634	10.83	97
M2	0.459	6.78	70	0.727	13.24	112
M3	0.876	16.36	138	0.865	16.88	133
M4L	1.985	32.58	314	2.100	41.58	392
5D	0.885	16.85	102	0.996	18.92	119
5J	0.134	5.28	21	-	-	-
5K	0.235	5.24	36	-	-	-
5N	0.185	7.28	28	0.126	3.31	19
2E	0.149	3.94	23	-	-	-
2L	0.837	15.03	128	0.844	15.92	177
C1 US	2.188	26.53	355	1.999	20.89	307
C1 DS	3.046	32.16	467	3.029	29.22	410
C2US	1.917	28.54	294	1.328	18.62	204
DS	13.637	118.90	2090	12.648	115.0	1970

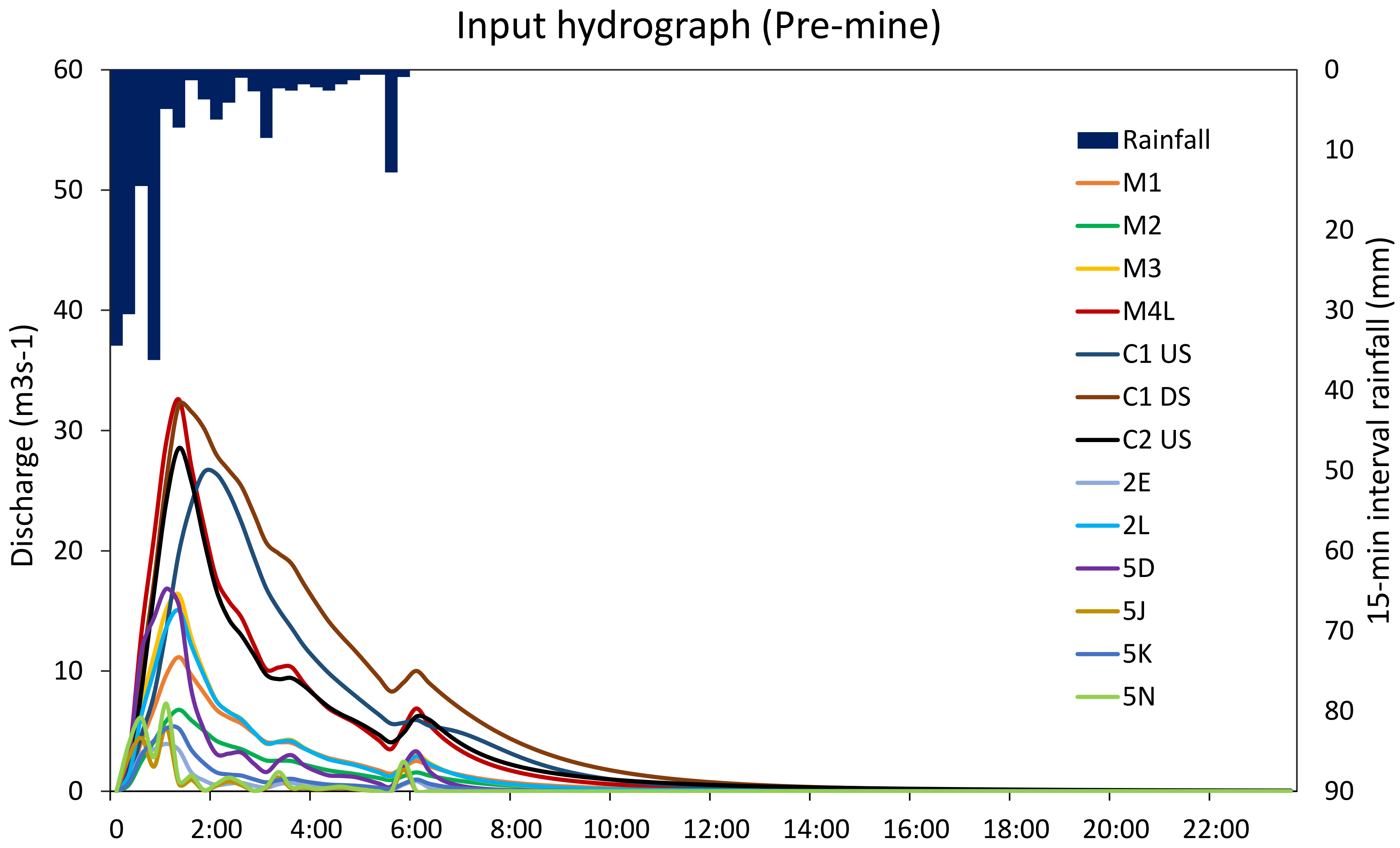


Figure 8. Input hydrographs from RORBwin for pre-mining scenario

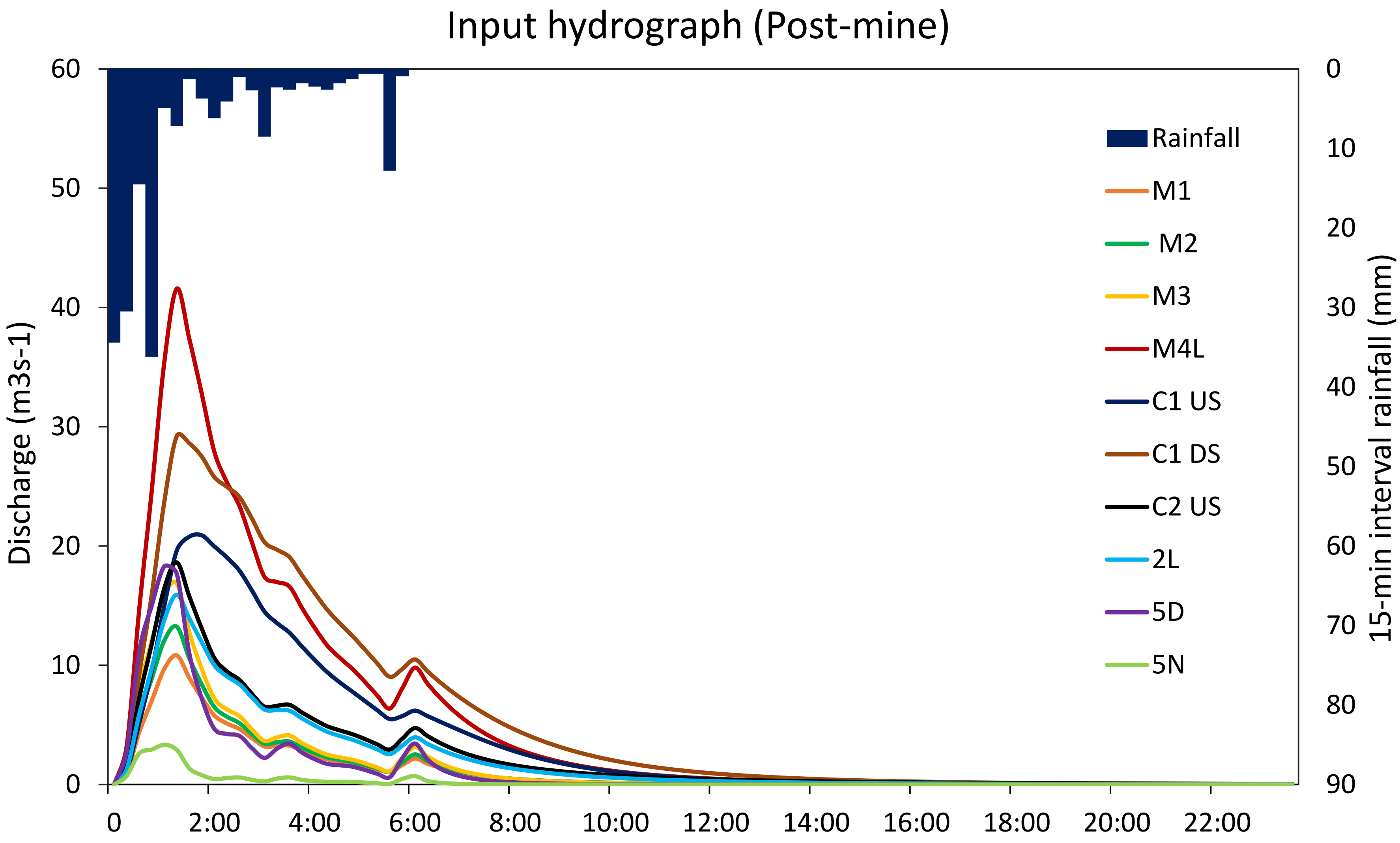


Figure 9. Input hydrographs from RORBwin for post-mining scenario

3.2 Flood Inundation Modelling

The results of inundation modelling for a 1%AEP event allowed the assessment of the following:

1. What impact will inundation have on mine infrastructure,
2. How has the mine infrastructure affected flooding of the Cox Peninsula Road at the culverts where the road intersects catchments 2 & 5, and
3. What effect will primary storm surge in Darwin Harbour have if it occurs simultaneously with the 1%AEP rainfall event.

3.2.1 Catchment inundation

Figure 10 shows the pre-mine flood inundation for the 1%AEP rainfall event for catchment 2 and 5. *Figure 11* shows the post-mine flood inundation for the 1%AEP rainfall event for catchments 2 and 5. The post-mine inundation area is less than the pre-mine area because of the flow paths between the upper streams of culvert 1 and 2 is blocked by the mine infrastructure and the mine haul road embankment (*Figure 12*). There is less inundation of the Cox Peninsula Road (particularly around culvert 2) with the mine infrastructure in place because the surface area of the pit and ponds do not contribute water, and direct rainfall into those areas is retained on site. This slightly increases the inundation area to the east of the mine. This could also cause long-term ponding with a depth up to 1.3m within the low-lying area along the flood diversion bund and the haul road embankment after most of the flood water recedes (highlighted by the red ellipse in *Figure 13*). This can be prevented by installing a culvert to the haul road embankment, which is discussed in Section 0. In addition, there is one location on the mine that if the bund just extends to the position indicated by the drawing supplied, water could enter one of the ponds (highlighted by the red circle in *Figure 11*).

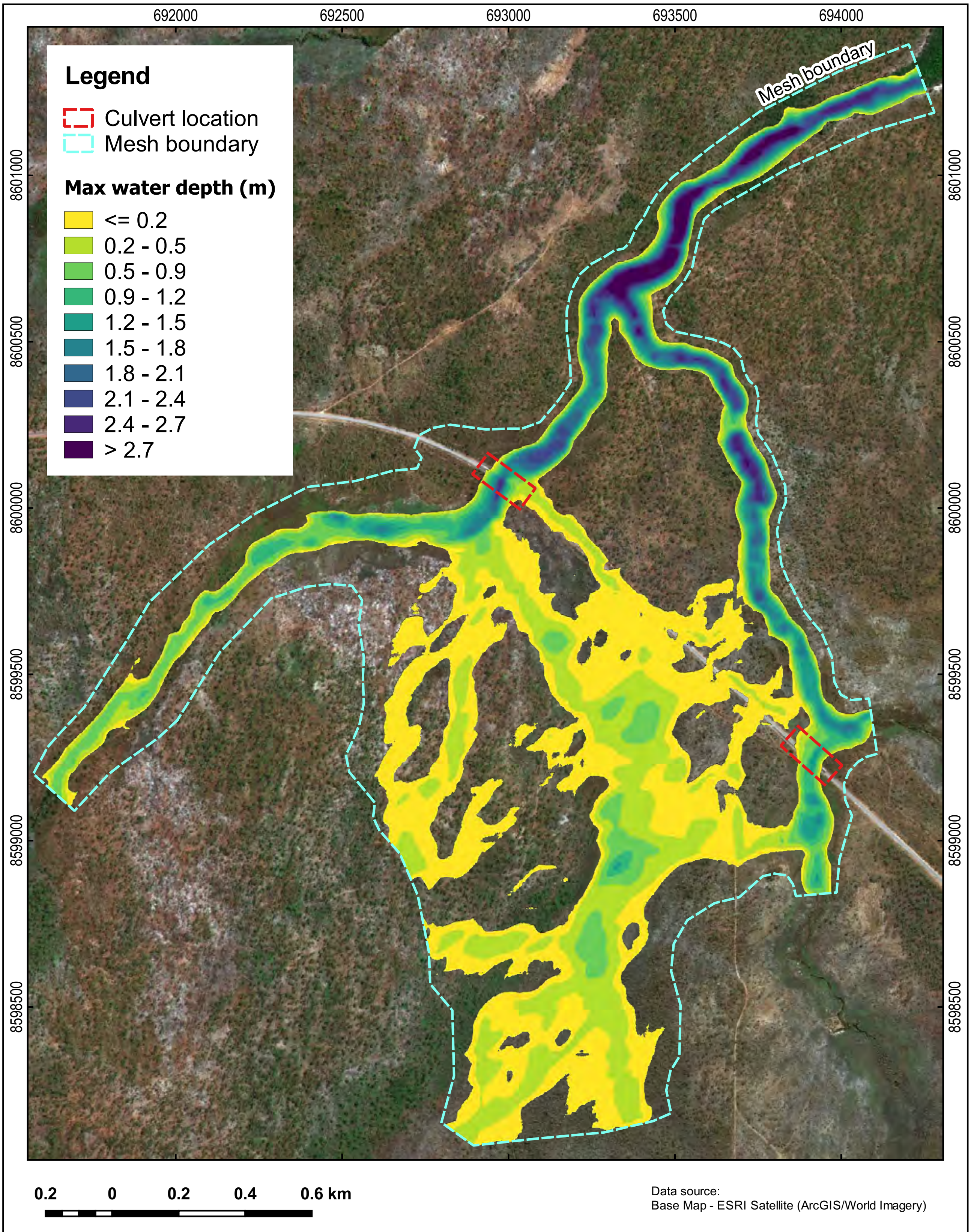
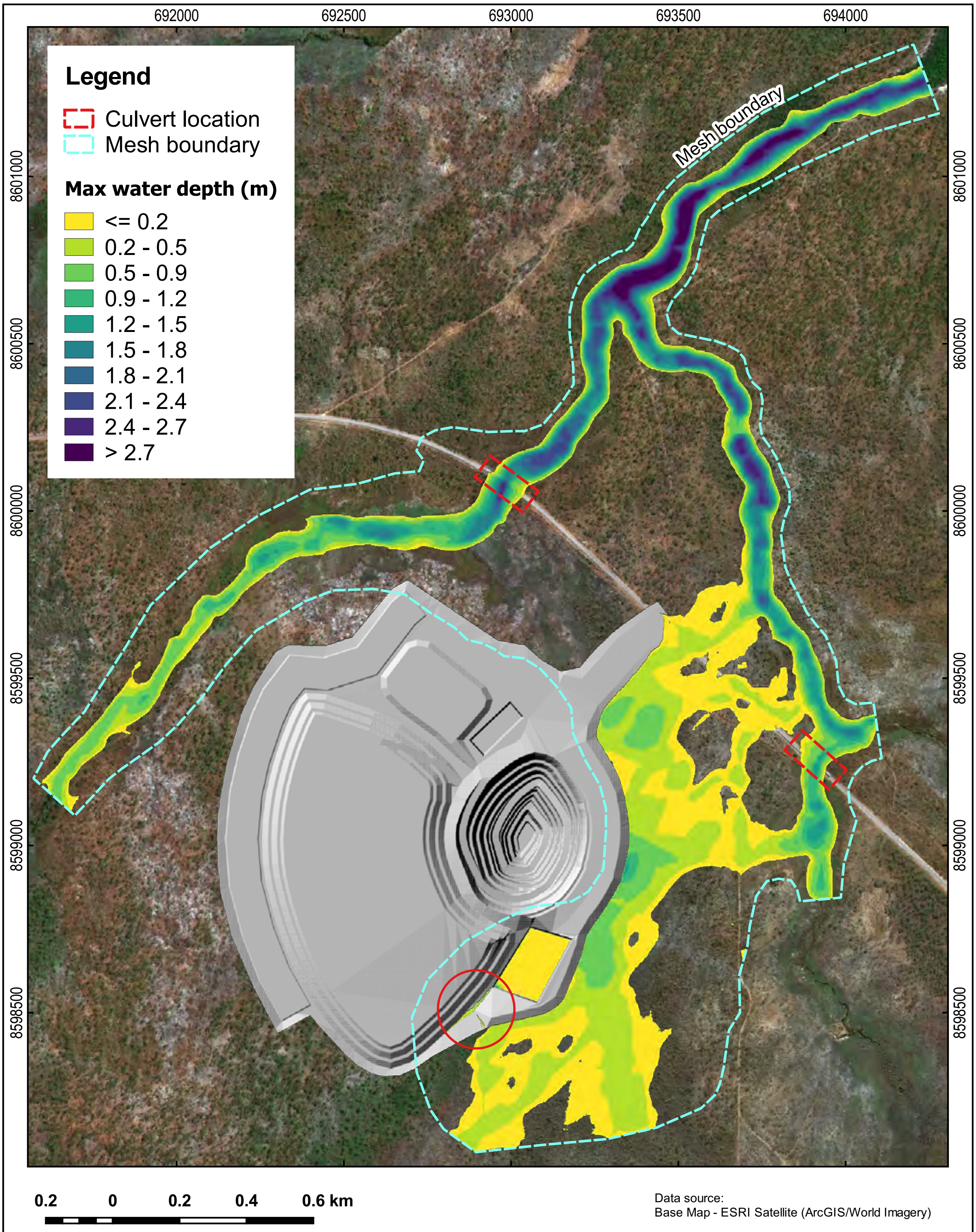


Figure 10. Pre-mine 1%AEP rainfall inundation extent

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52



Data source:
Base Map - ESRI Satellite (ArcGIS/World Imagery)

Figure 11. Post-mine 1%AEP rainfall inundation extent

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

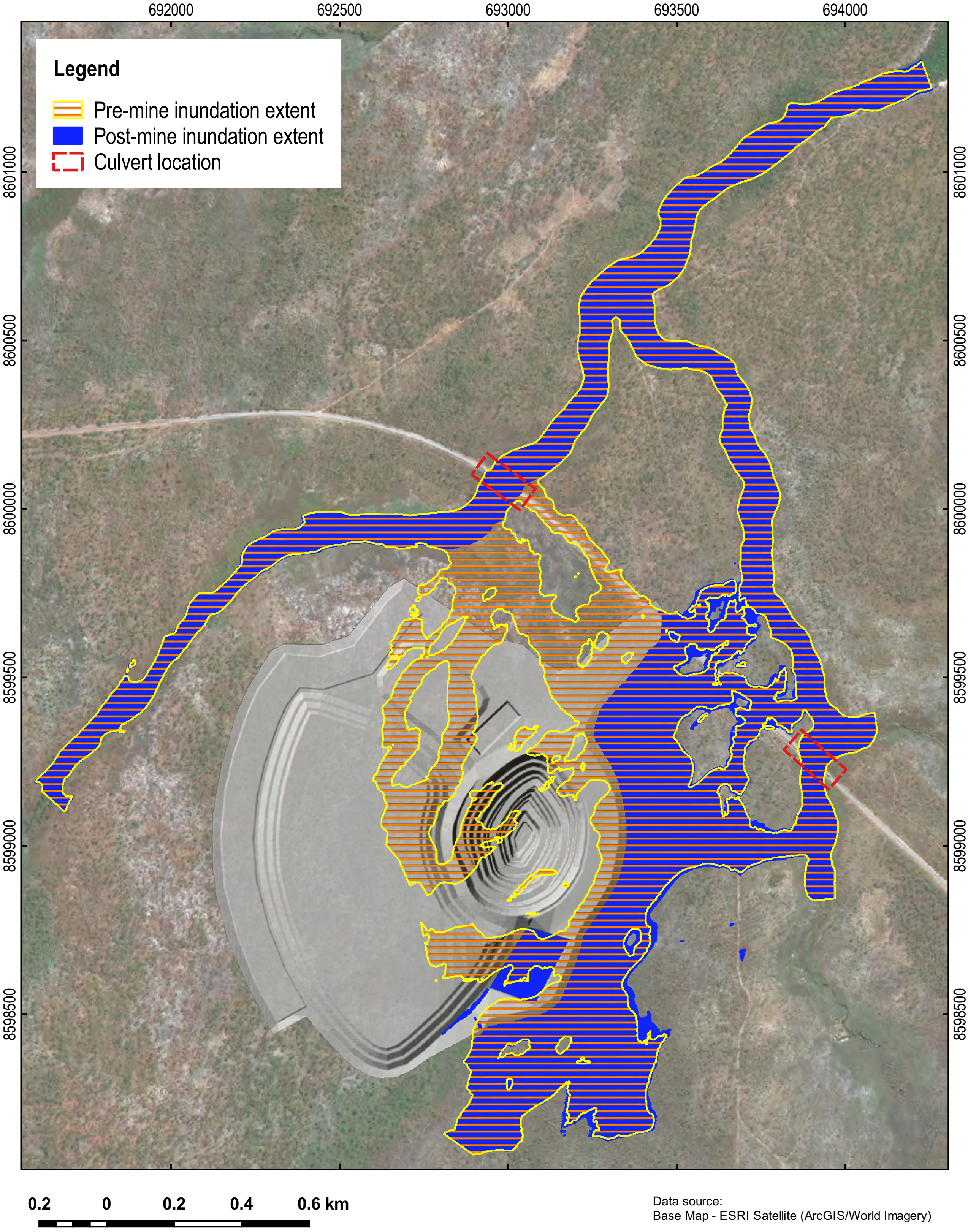
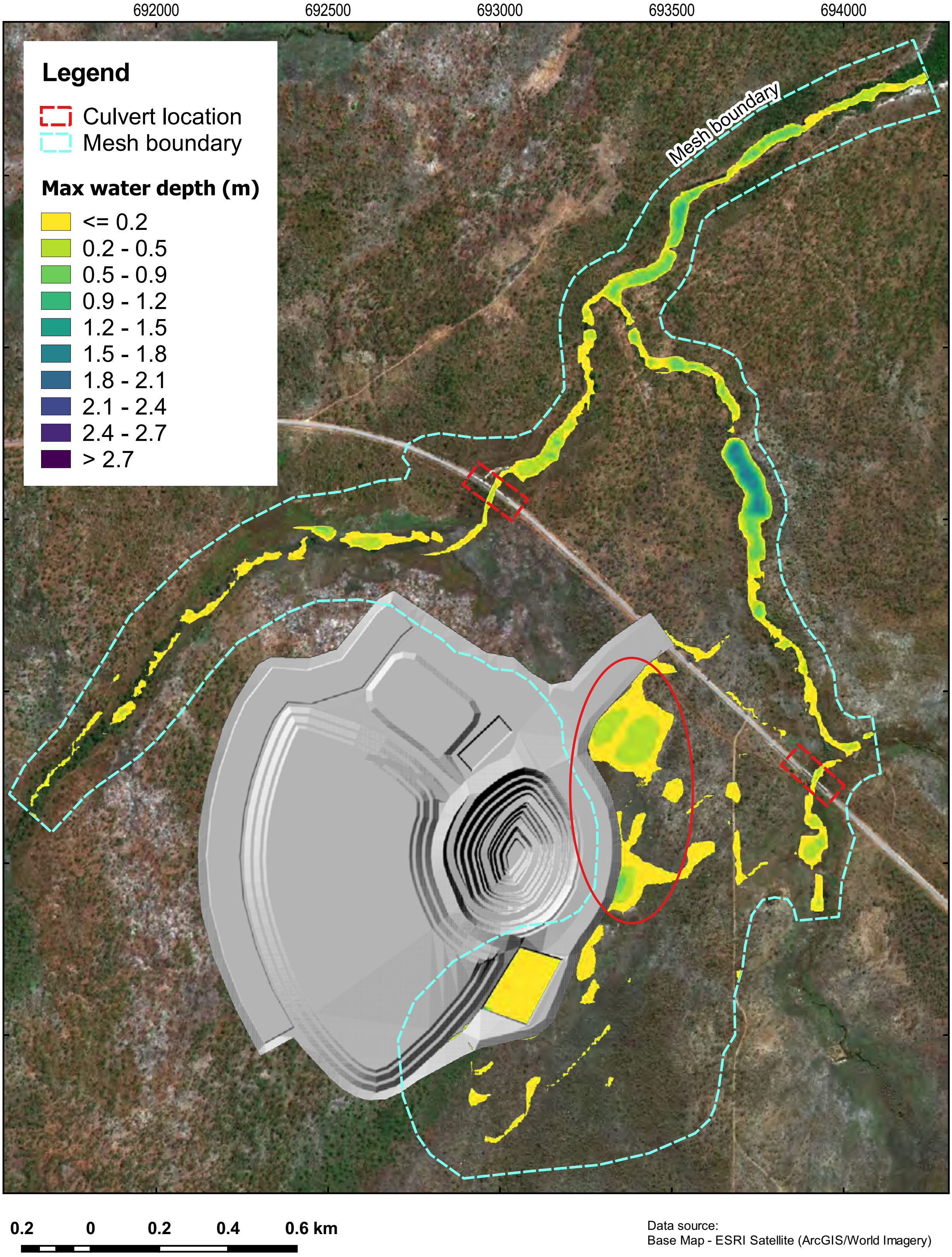


Figure 12. Pre- and post-mine 1%AEP rainfall inundation extents

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52



Data source:
Base Map - ESRI Satellite (ArcGIS/World Imagery)

Figure 13. Post-mine 1%AEP flood water depth 18 hours after cease of rainfall

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

3.2.2 Culverts 1 and 2

The simulated flow components at the culverts (Head Water (HW) height, Tail Water (Height) and Flow (include over flow)) are illustrated in *Figure 14*. When HW height is higher than the road surface, the road is regarded as being inundated.

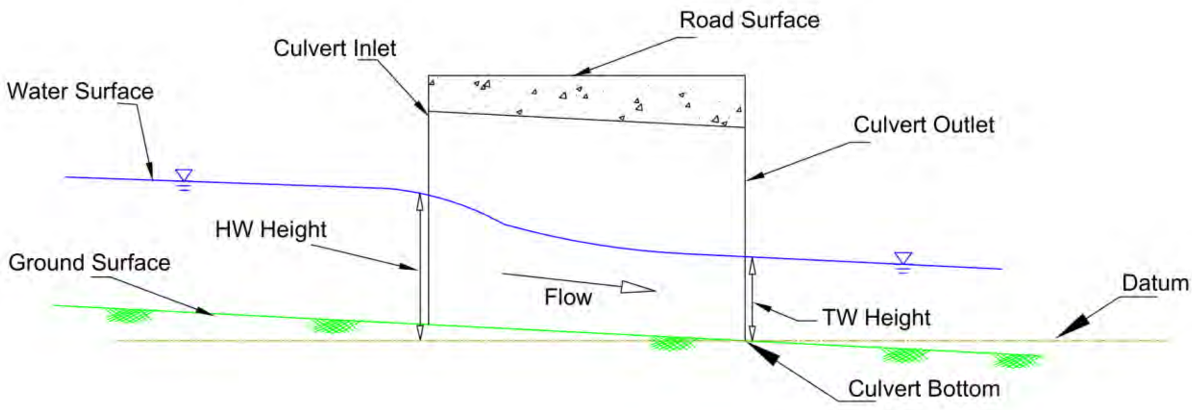


Figure 14. Schematic diagram of simulated flows for culverts.

The simulation results for the 1%AEP flood for pre- and post-mine conditions for Culvert 1 are shown in *Figure 15.*

The simulations indicate that Cox Peninsula road is inundated for a shorter period for post-mine conditions (6.1 hrs) than for the pre-mine conditions (7.0 hrs). The maximum water depth above the road surface at the location of this culvert is 0.38m for pre-mine and 0.34m for post-mine scenarios.

The derived simulation results for the 1%AEP flood for pre- and post-mine conditions for Culvert 2 are shown in *Figure 16*. Again, the simulations indicate that Cox Peninsula Road is inundated for a shorter period for post-mine conditions (3.4 Hrs) than for the pre-mine conditions (3.5 Hrs). The maximum water depth above the road surface at the location of this culvert is 0.29m for pre-mine and 0.27m for post-mine scenarios.

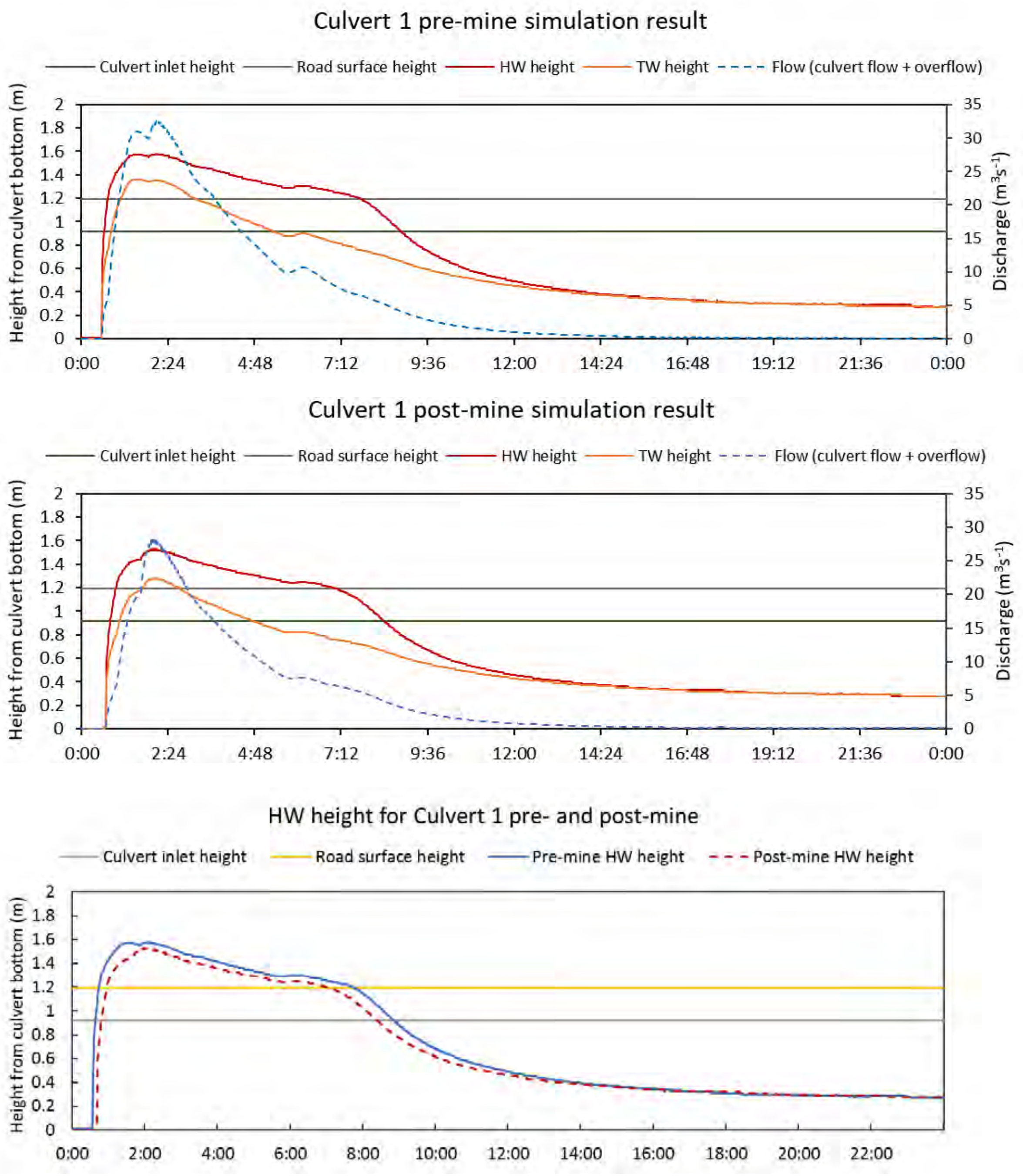


Figure 15. Culvert 1 pre- and post-mine simulation results.

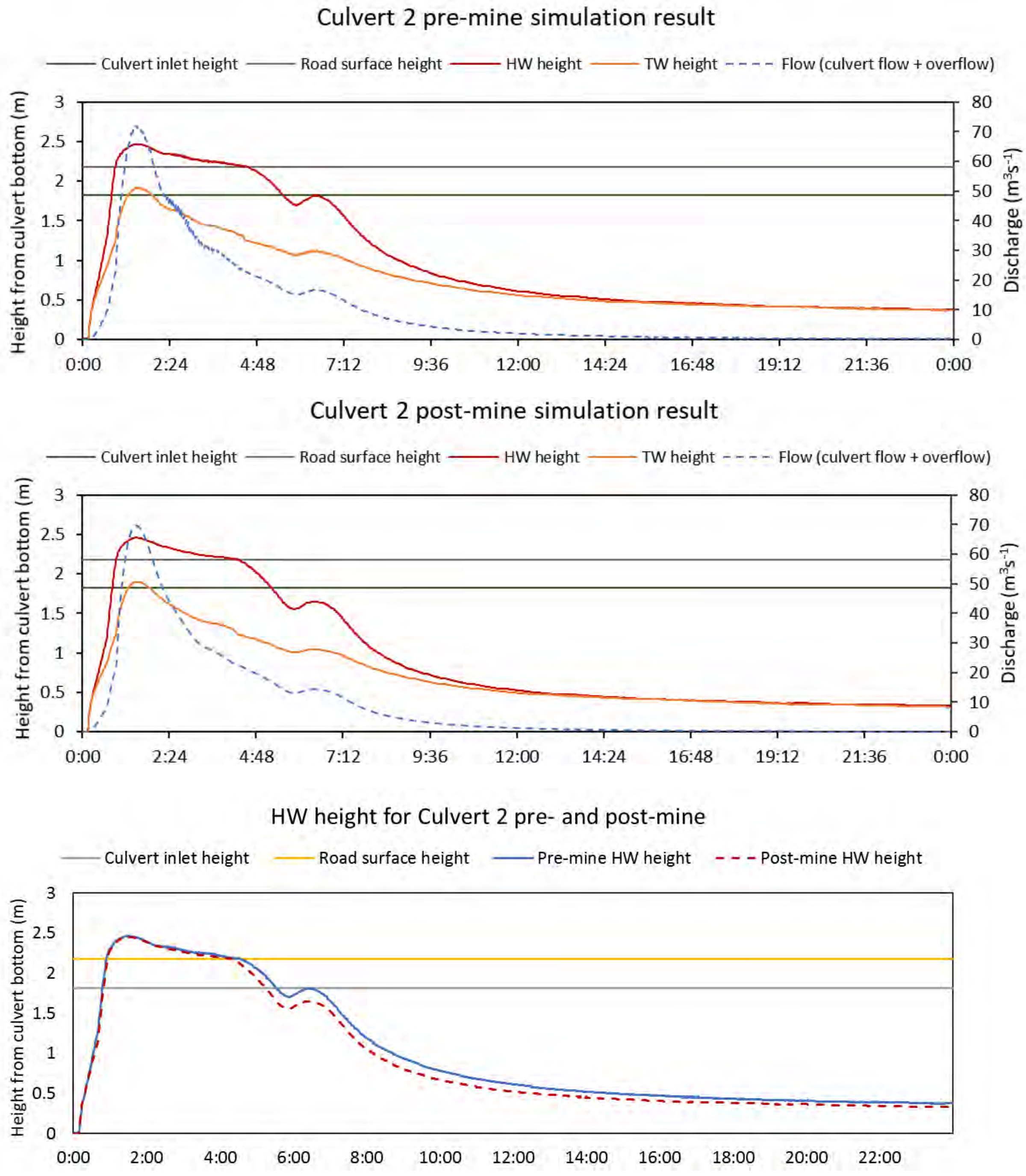
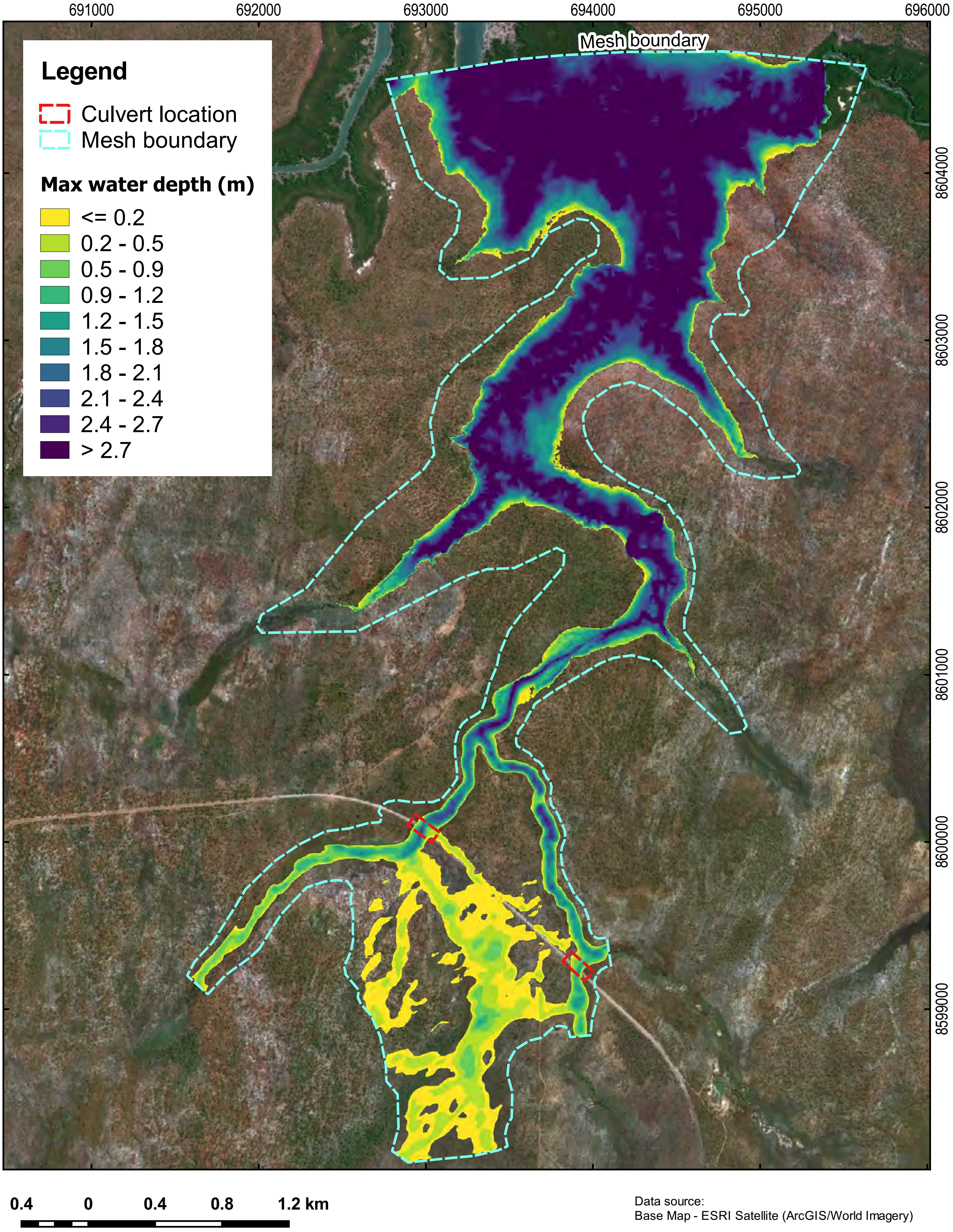


Figure 16. Culvert 2 pre- and post-mine simulation results.

3.2.3 The effect of primary storm surge in Darwin Harbour

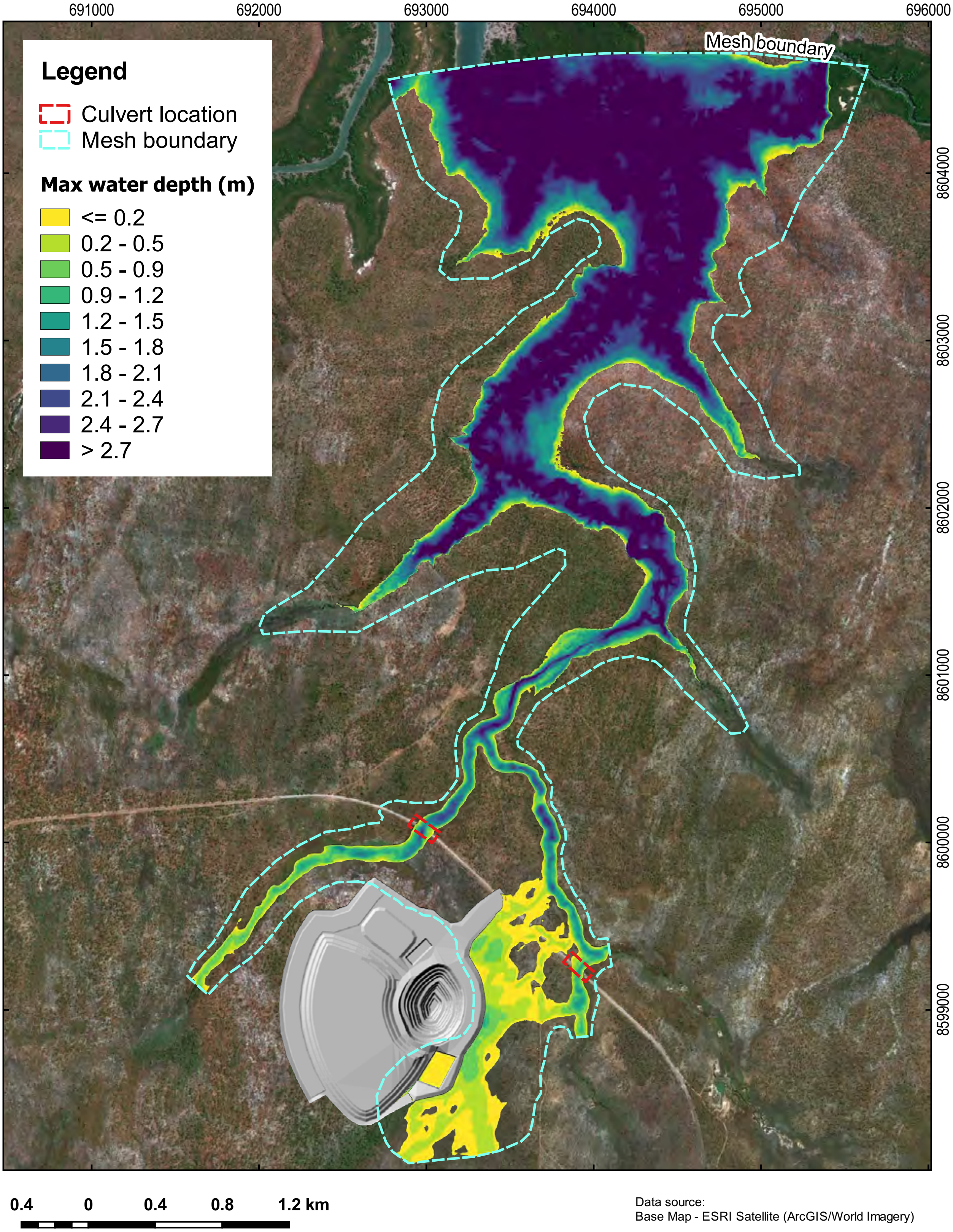
Figure 17 shows the pre-mine flood inundation for the 1%AEP rainfall event occurring at the same time as primary storm surge in Darwin Harbour for catchment 2 and 5. *Figure 18* shows the post-mine flood inundation for the 1%AEP rainfall event occurring at the same time as primary storm surge in Darwin Harbour for catchment 2 and 5. For the upstream area for culverts 1 and 2, the inundation extent for 1%AEP rainfall and 1%AEP rainfall with primary storm surge are visually the same for both pre- and post-mining scenarios (*Figure 19* and *Figure 20*). *Figure 21* also shows the primary storm surge in Darwin Harbour has negligible impact on the HW height of culverts 1 and 2 under either pre- or post-mining scenarios. Based on this assessment it appears, that there is an insignificant difference in floodplain inundation and therefore little change to water availability for ecological processes. Change in water quality have not been addressed in this study.



Data source:
Base Map - ESRI Satellite (ArcGIS/World Imagery)

Figure 17. Pre-mine inundation extent for the 1%AEP rainfall occurring at the same time as primary storm surge

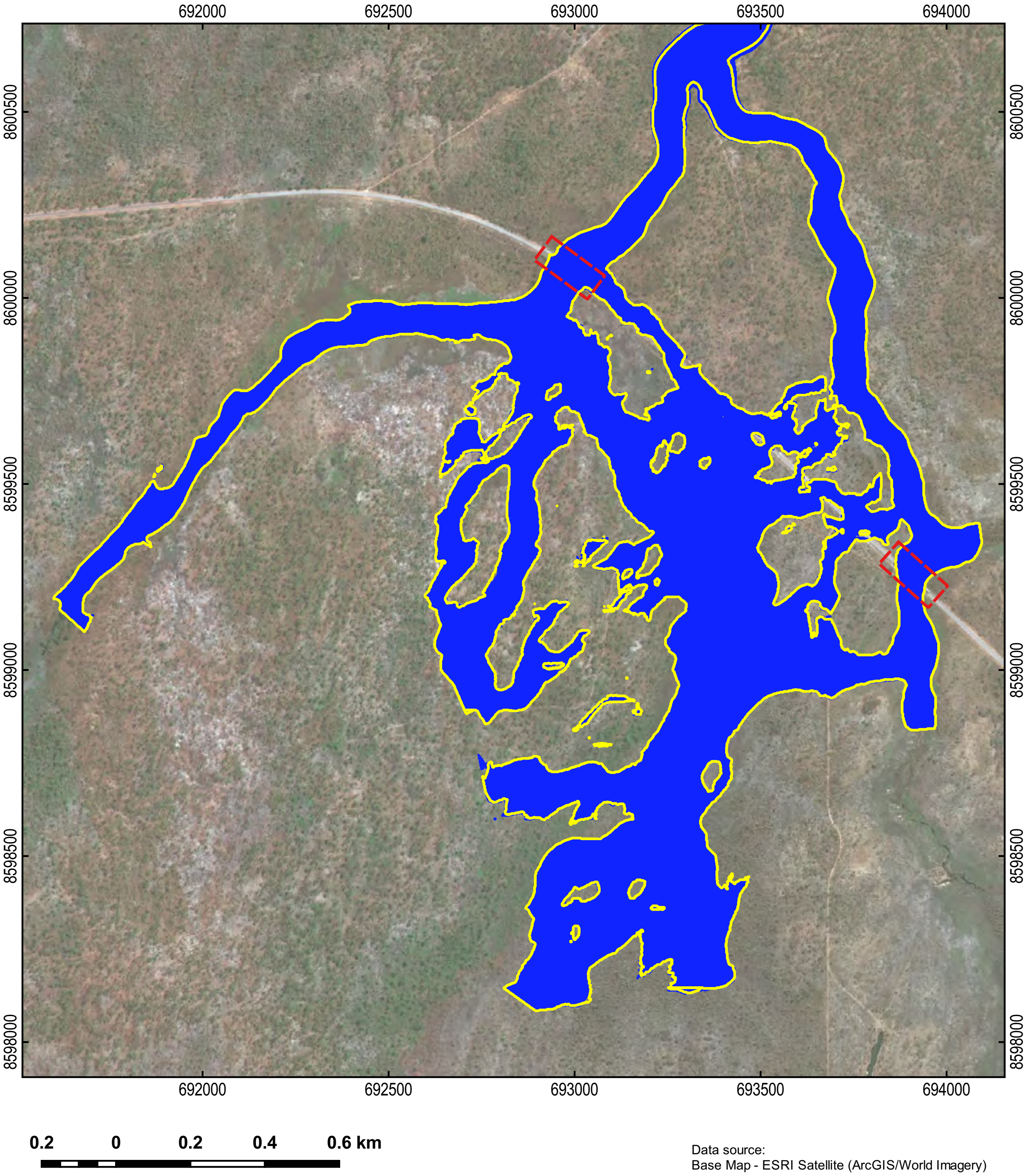
Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52



Data source:
Base Map - ESRI Satellite (ArcGIS/World Imagery)

Figure 18. Post-mine inundation extent for the 1%AEP rainfall occurring at the same time as primary storm surge

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52



Data source:
Base Map - ESRI Satellite (ArcGIS/World Imagery)

Legend

1%AEP rainfall inundation extent

1%AEP rainfall with primary surge inundation extent

Culvert location

N

Figure 19. Pre-mine inundation extents for 1%AEP rainfall and 1%AEP rainfall occurring at the same time as primary surge

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

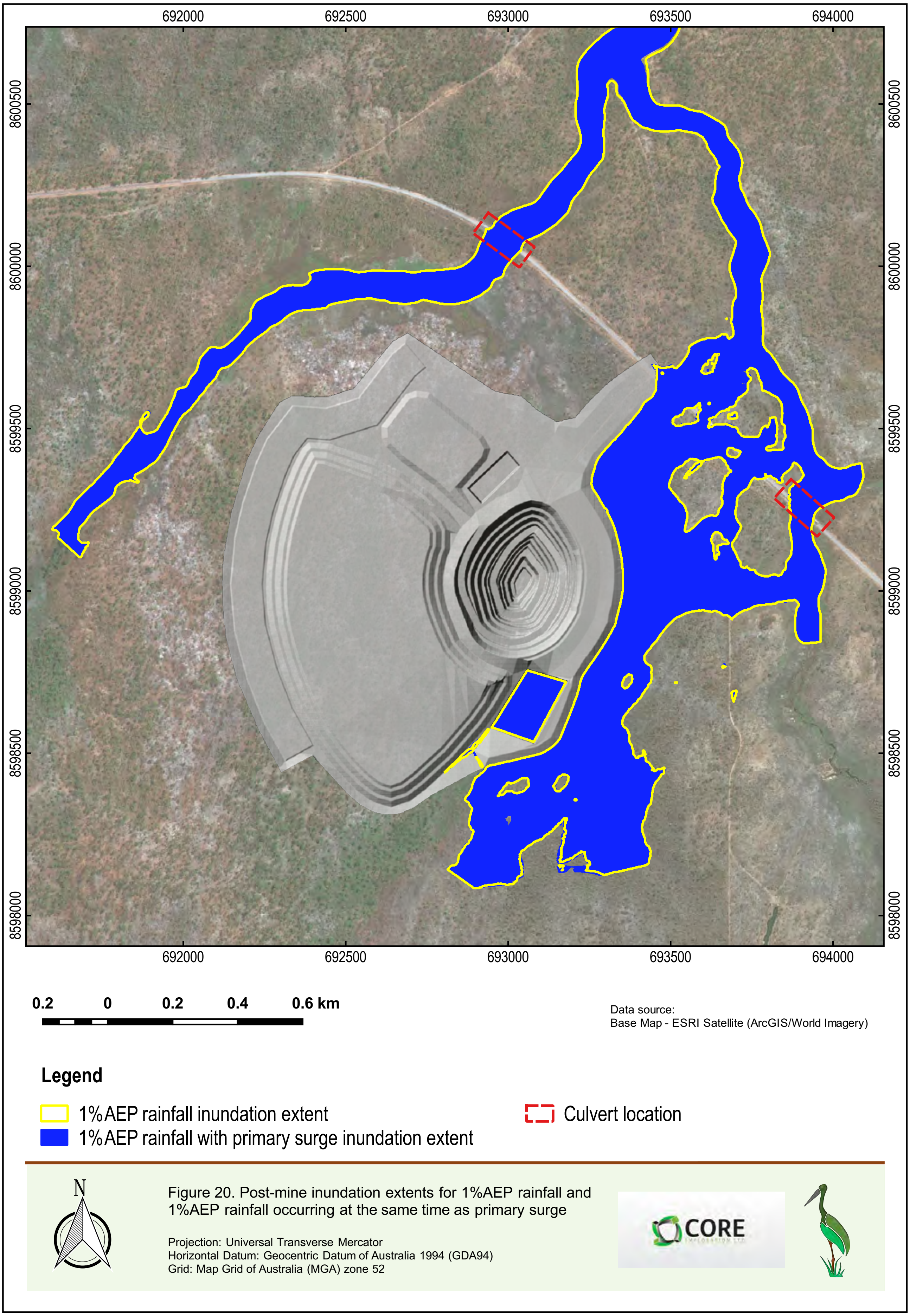
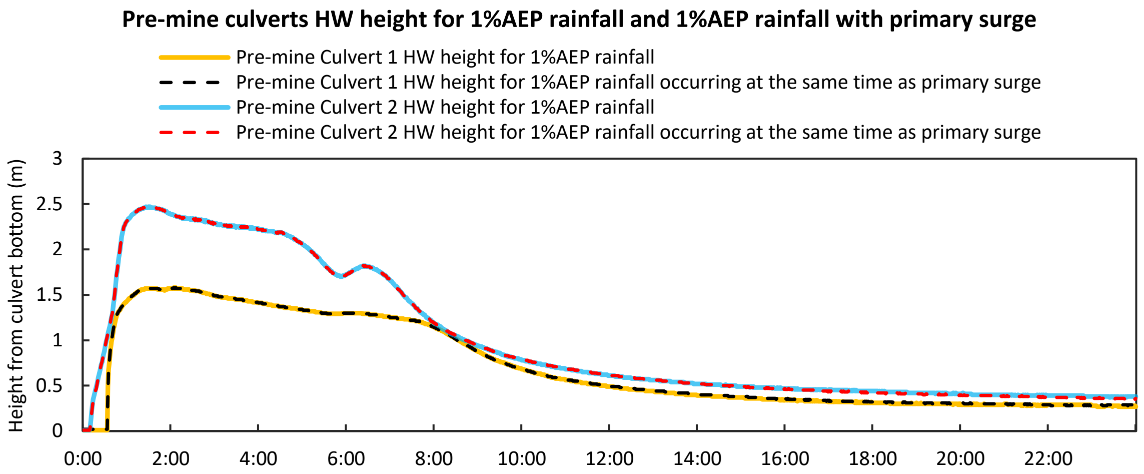


Figure 20. Post-mine inundation extents for 1%AEP rainfall and 1%AEP rainfall occurring at the same time as primary surge

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52



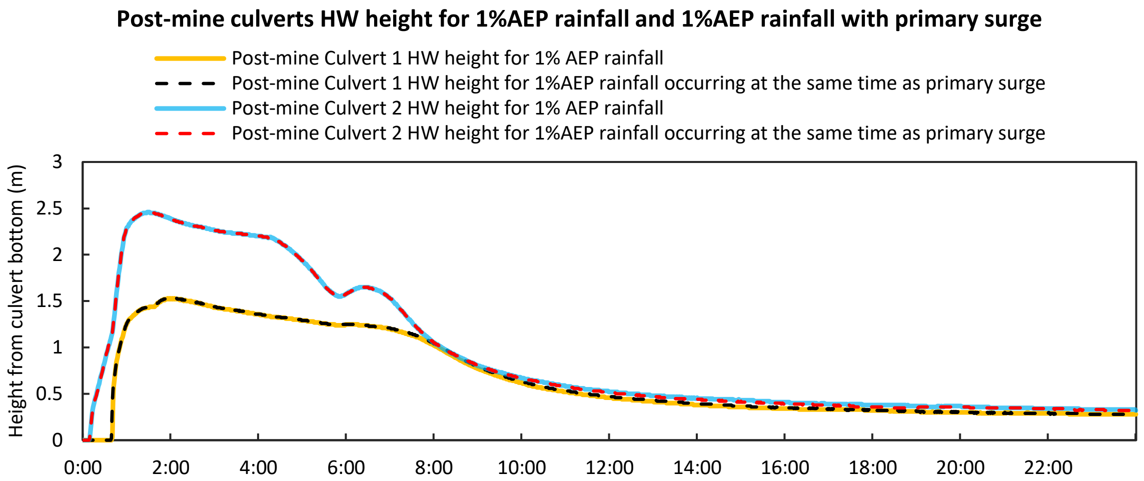


Figure 21. Culverts HW height pre- and post-mining conditions for 1%AEP rainfall and 1%AEP rainfall occurring at the same time as primary surge

3.2.4 Additional culvert

The ponding near the mine infrastructure can be prevented if a culvert is in the location shown in *Figure 23*. The DEM and the HEC-RAS model were modified to incorporate the culvert. The culvert was set as the same size as Culvert 2 (**Appendix B2**) which comprises 2 box culverts 1820mm high and 2420mm wide. These sizes were not determined using engineering design methods and they are expected to be oversized. The modelling aims to provide a general indication of the change of flow behaviour should drainage structure be in place to prevent ponding.

The inundation extent for the modified model is shown in *Figure 24*. *Figure 25* shows the inundation extent 18 hours after the cease of rainfall. It can be seen from *Figure 25* that water was able to be drained to Culvert 1 via the haul road culvert. The simulated hydrograph of flow through the haul road culvert is shown in *Figure 22*.

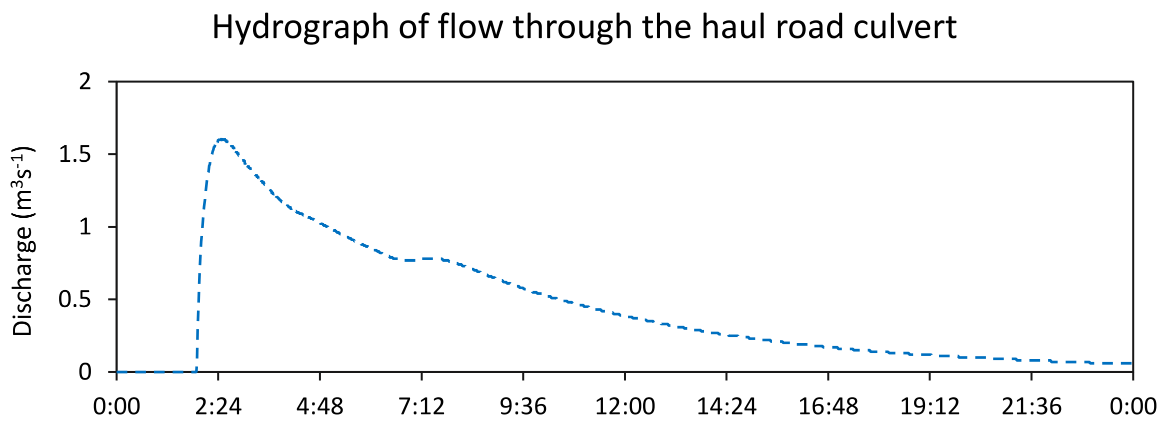


Figure 22. Simulated hydrograph of flow through the additional culvert.

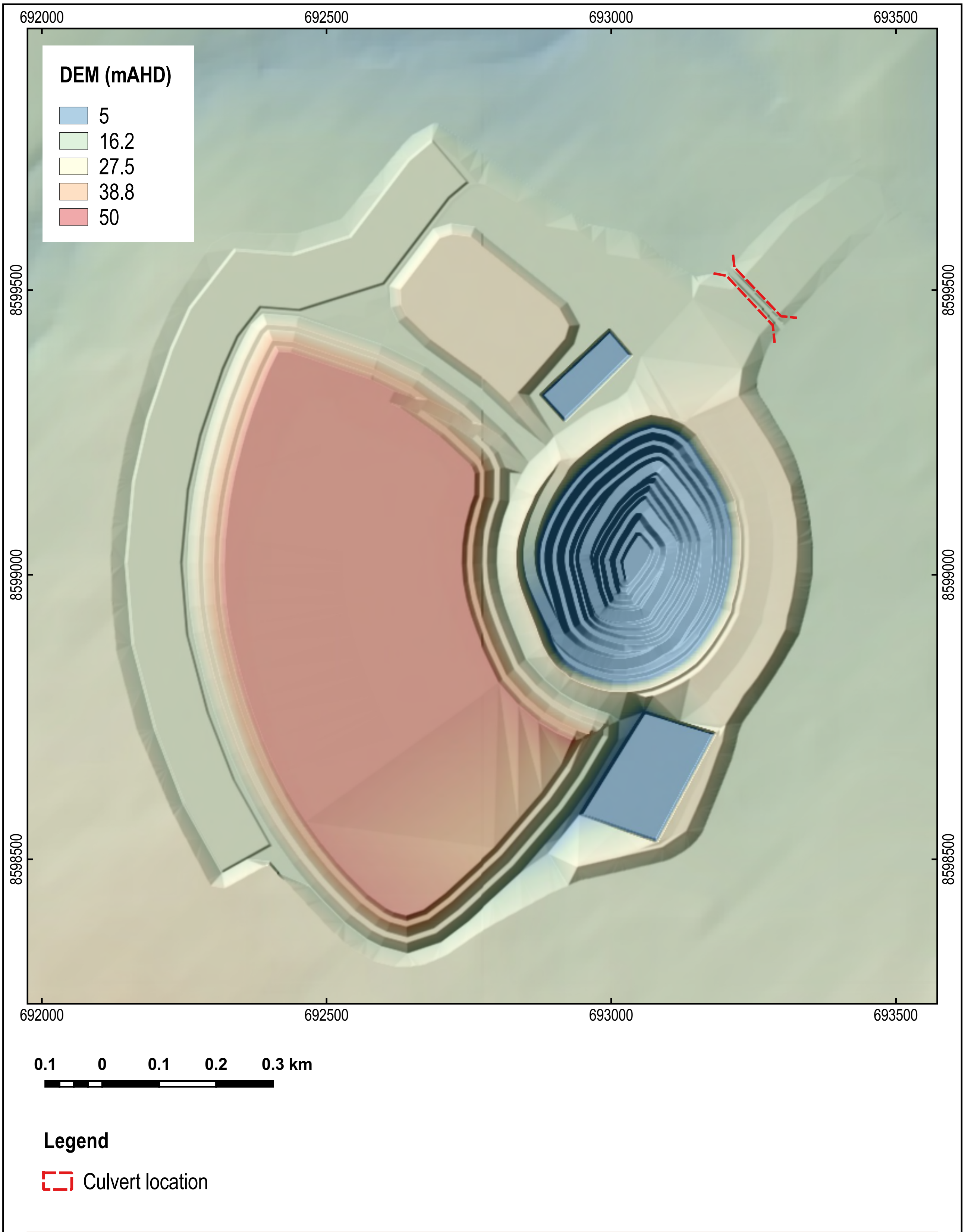


Figure 23. Location of haul road culvert

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

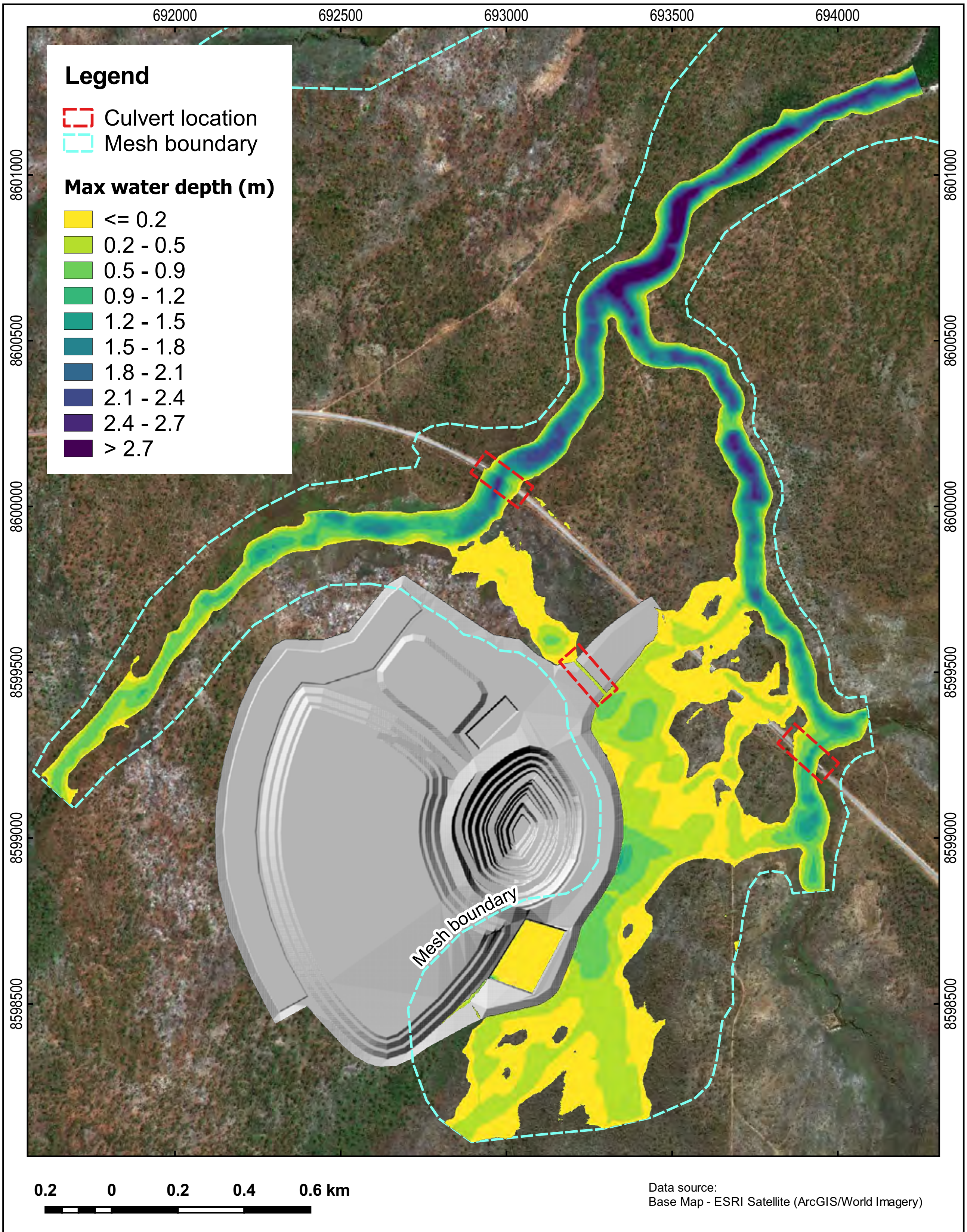
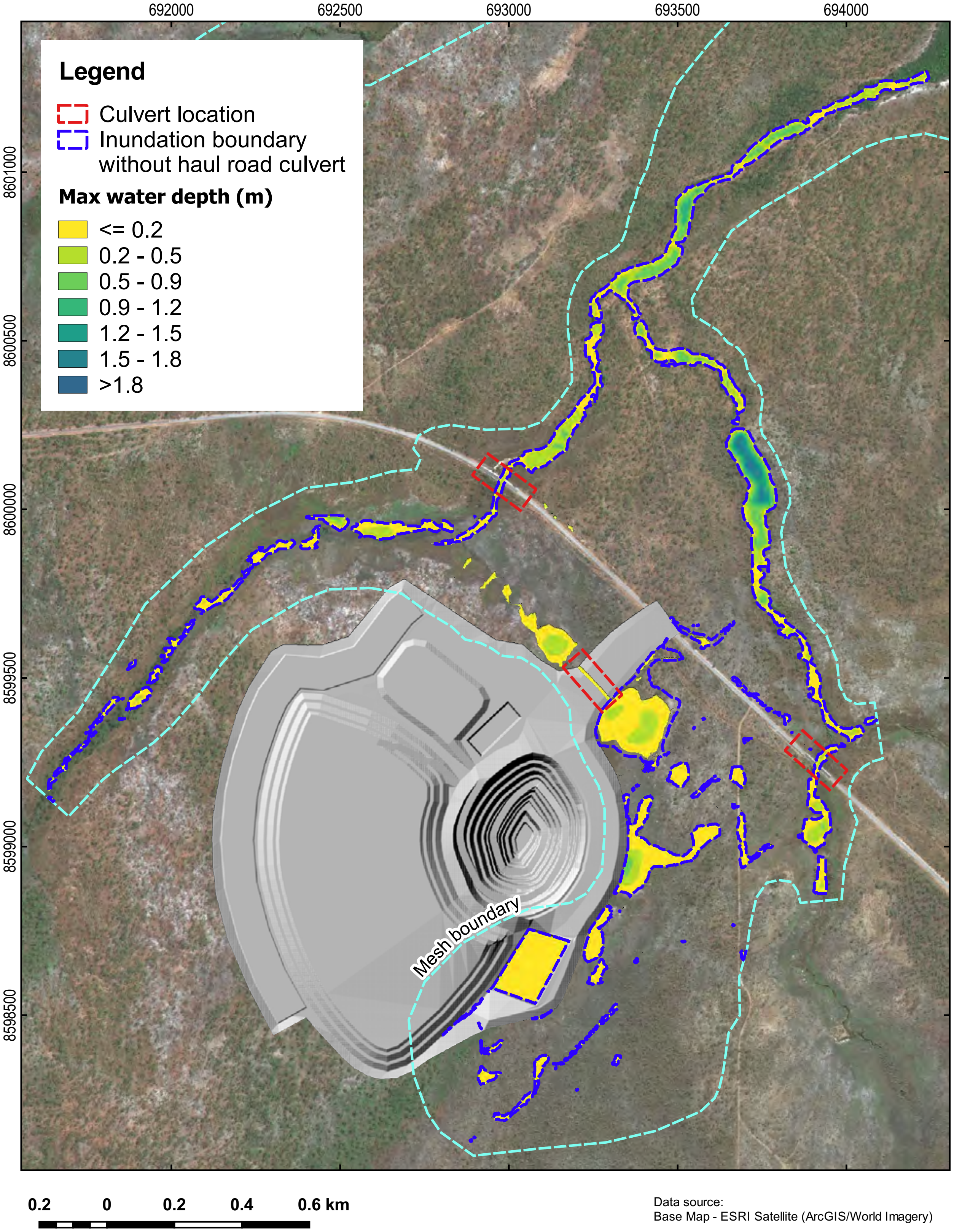


Figure 24. Post-mine 1%AEP flood inundation extent for mine with haul road culvert

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52



N

Figure 25. Post-mine 1%AEP flood water depth 18 hours after cease of rainfall when haul road culvert installed

Projection: Universal Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia 1994 (GDA94)
Grid: Map Grid of Australia (MGA) zone 52

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4 Summary

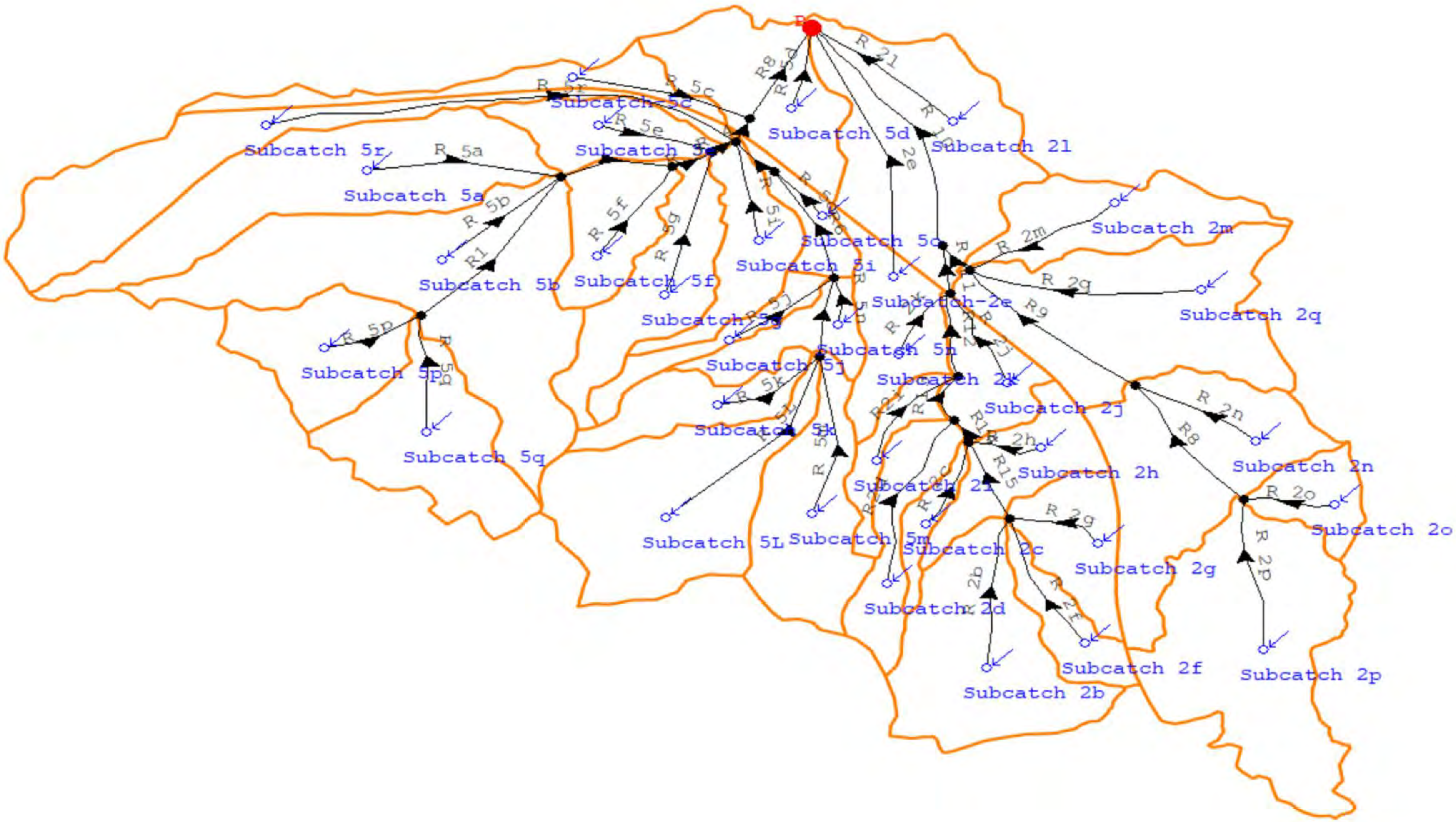
The RORBwin hydrology modelling shows little change in the peak Q (-3%) and total Q at the outlet of catchments 2 and 5 for (-6%) for pre-mining and post-mining conditions of the catchments for probabilistic 1%AEP rainfall runoff event.

When the RORBwin results are used in flood inundation modelling there is little difference in inundation areas between pre- and post-mining apart from an area to the northeast of the infrastructure around Cox Peninsular road. The addition of a culvert under the haul road increases temporary inundation in this area.

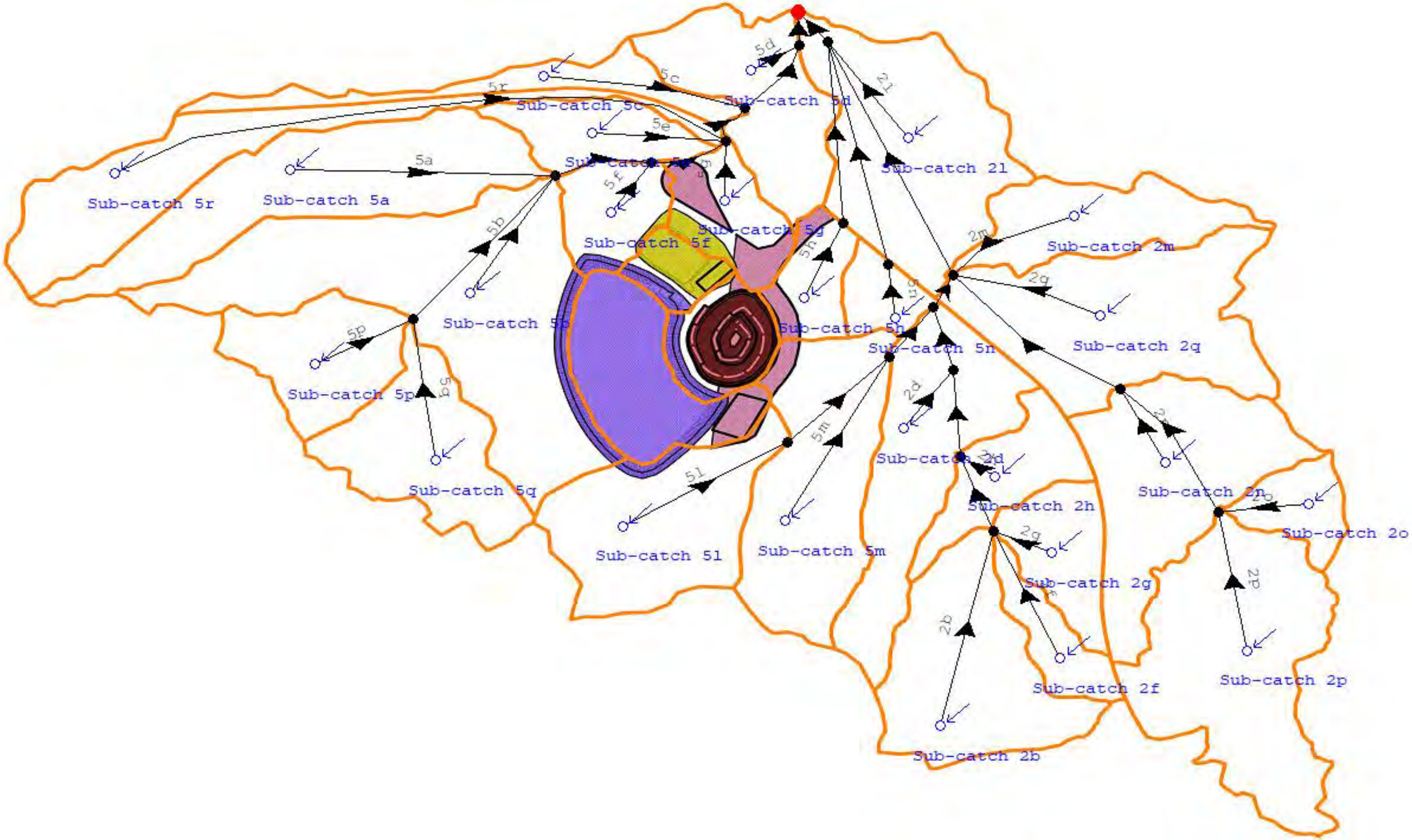
For floodplain inundation, further downstream, there is little negligible difference between the 4 scenarios modelled, pre- and post-mining with and without contemporaneous primary storm surge in Darwin harbour. Indicating that the availability of water for ecological processes on the mangrove tidal floodplains will not change for the 1%AEP event. This assessment does not consider water quality.

Appendix A

A1. RORBwin model for Catchments 2 and 5: pre-mining

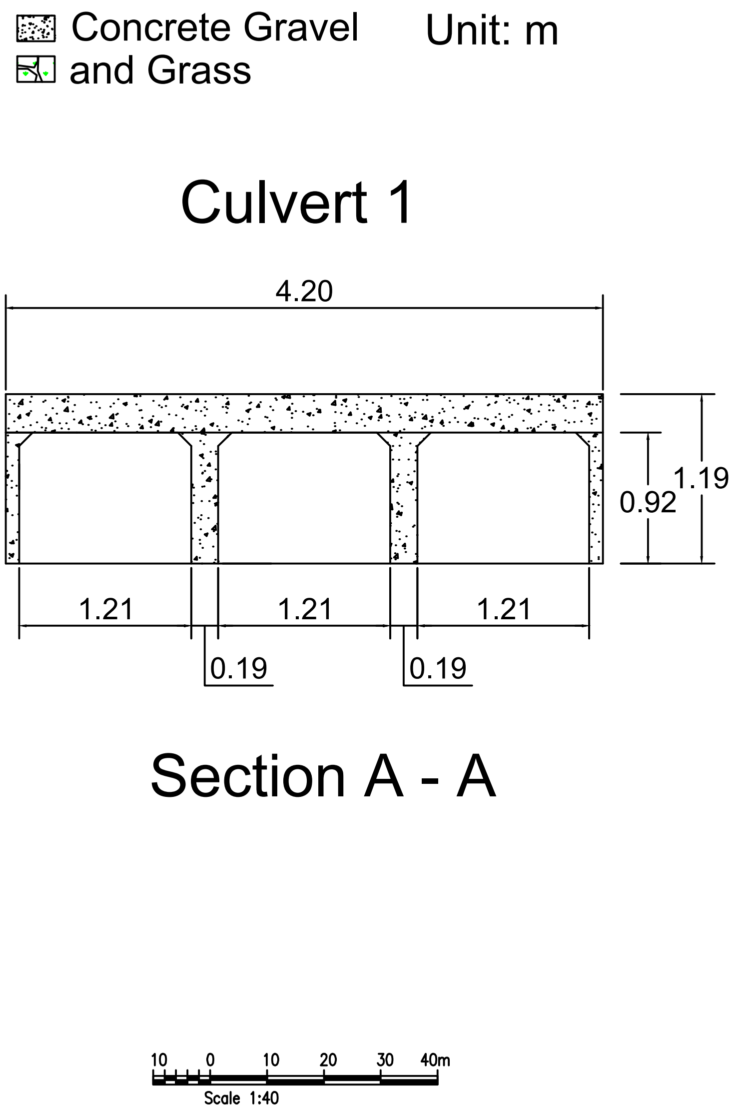


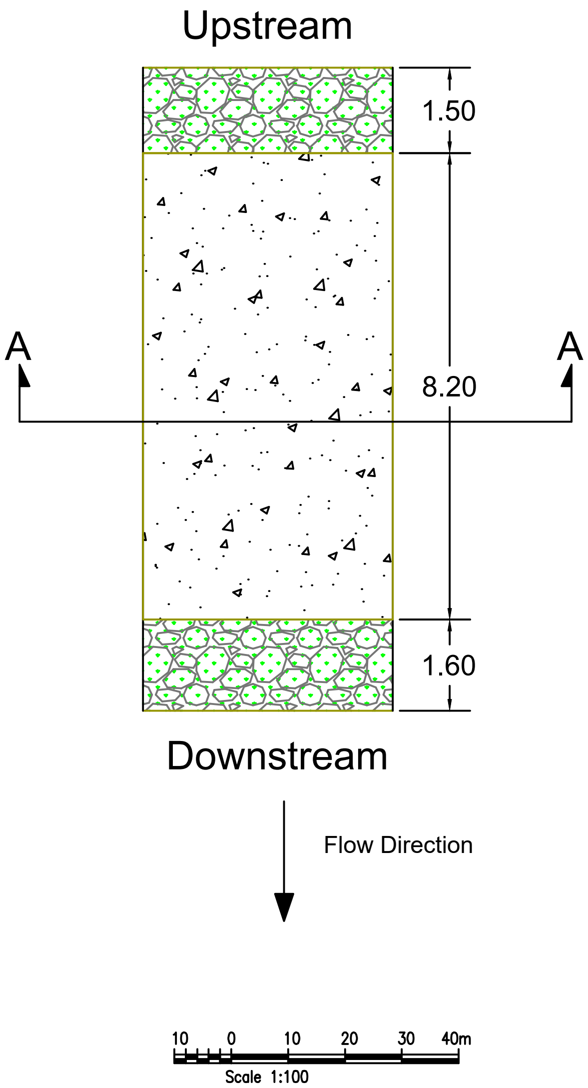
A2. RORBwin model for Catchments 2 and 5: post-mining



Appendix B

B1. Drawing of Culvert 1



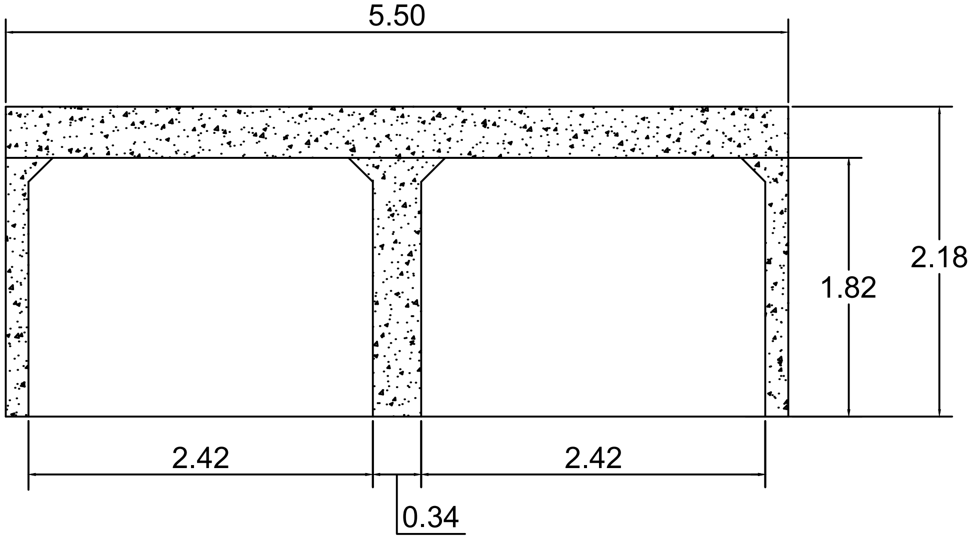


B2. Drawing of Culvert 2

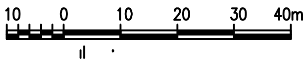
Concrete
Gravel and Grass

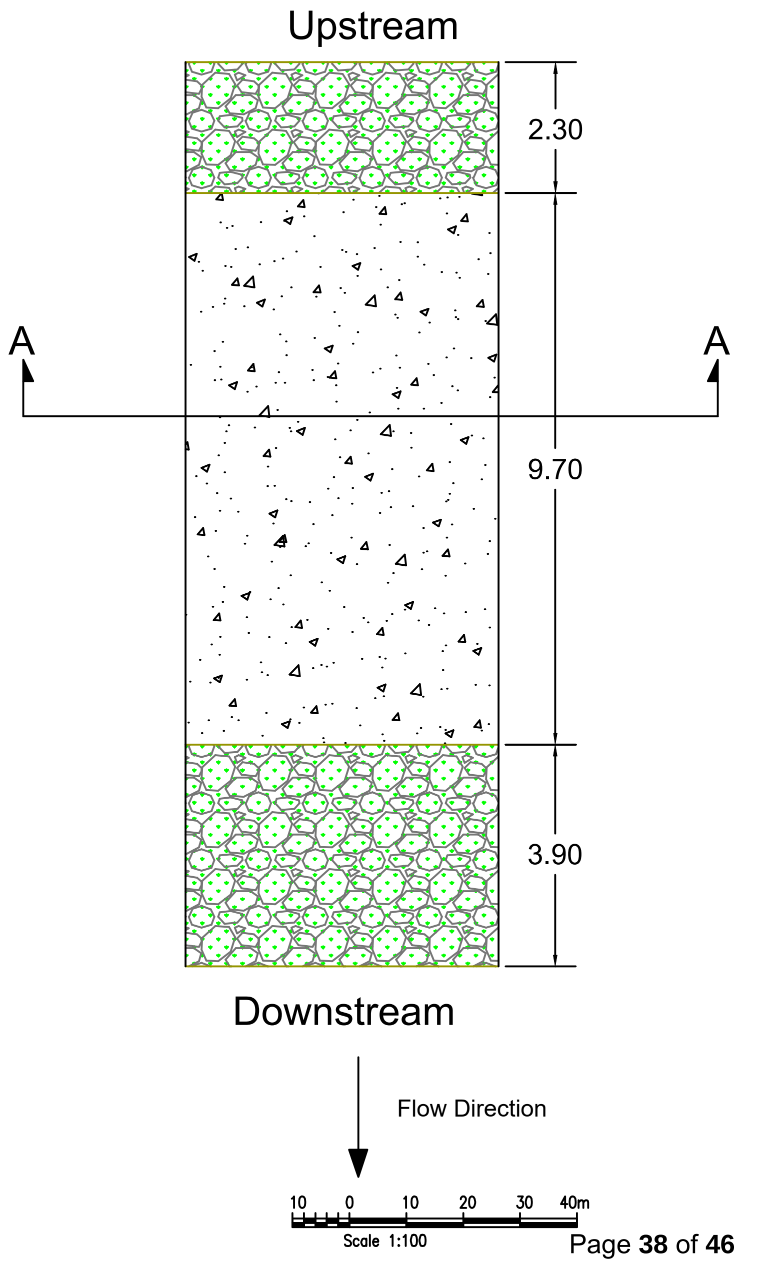
Unit: m

Culvert 2



Section A - A





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RELIANCE, USES and LIMITATIONS

This study, report and analyses have been based on the information available to and the assumptions made by EnviroConsult Australia Pty Ltd at the time of preparation. EnviroConsult Australia Pty Ltd does not take responsibility for errors and omissions in the conclusions, report and maps produced by this study due to incorrect information or any error in, or omission in connection with assumptions. No chemical analysis, groundwater hydrology, water quality or contaminant export studies of any kind were conducted. Any comment made in the report with respect to the above are speculative based on the surface hydrology analysis and should not be relied upon as fact.

Description	Author	Reviewer	Approved for Issue		
			Name	Signature	Date
Revision 0	Ken Evans Mike Liu Rojita Thapa	Ken Evans Aiden Campbell	Ken Evans	KG Evans	20/07/2018
Final			Ken Evans	KG Evans	01/08/2018

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