Ashburton Salt Project

Surface Water Assessment and Modelling

K+S Salt Australia Pty Ltd

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1 INTRODUCTION

1.1 Background

K+S Salt Australia (K+S) is proposing to construct a solar salt evaporation facility (the Ashburton Salt Project) approximately 40 km south west of Onslow. The facility will be constructed on existing salt flat areas that are located inshore from the coast. The Project will require a range of infrastructure to be constructed including a seawater intake and hypersaline wastewater (bitterns) outfall structures, as well as a jetty and berthing pocket to allow for export of the salt product. The location of the Project is presented in Figure 1-1.

K+S commissioned Water Technology to undertake a Surface Water Assessment and Modelling to support the preparation of an Environmental Review Document (ERD) for the project.

This study:

- Defines existing catchment, surface water and inland water quality conditions relevant to the Project;
- Develops and validates an appropriate suite of numerical models to enable hydrological processes to be understood and simulated reliably, as well as impacts to hydrological processes assessed;
- Undertakes relevant environmental impact assessment and recommend mitigation measures; and
- Provides engineering advice to support the design of all key elements of site infrastructure.

1.2 Project Description

The Ashburton facility seeks to harvest seawater salt through solar evaporation. The infrastructure necessary for the Project includes a seawater intake, solar evaporation ponds, crystalliser ponds, an outfall for the discharge of hypersaline water, and a salt export jetty.

Figure 1-2 shows the general arrangement of the facility studied in this report. This is the 8th layout for the Project and many revisions underpin the iterative nature of the design process and the work carried out since the ESD to manage environmental impacts.

An overview of the infrastructure required for the Project follows:

- **Seawater Intake** - The seawater intake is located in Urala Creek South, and has an annual intake estimated to be 250 gigalitres (GL).

  A peak monthly intake of 29 GL per month is anticipated to occur in October to December, when solar evaporation rates are highest. This intake volume includes all seawater required for the entire project including evaporation ponds, wash plant and bitterns dilution water.
- **Solar Evaporation Ponds** - Seawater will be pumped from Urala Creek South into a series of eight evaporation (salt concentration) ponds. As seawater passes through the pond system, water evaporates, thereby producing a progressively denser brine with an increasing concentration of dissolved salts. Calcium salts precipitate out of the brine at an early stage, initially as calcium carbonate, then as calcium sulphate (i.e., gypsum). As the calcium salts settle to the pond floors, the ponds become less and less permeable.
- **Crystalliser Ponds** - Twelve crystalliser ponds are located immediately north of the solar evaporation ponds. They are laid out in two rows of six ponds. Their purpose is to perform the final crystallisation process to create the salt product.

  The saturated brine enters the crystalliser ponds where water is evaporated by solar energy until salt crystals (predominantly sodium chloride) are precipitated. Once the brine reaches a particular specific gravity, most of the remaining calcium will have been precipitated.

- **Export jetty** – Salt will be carried by the conveyor to the jetty where it will be conveyed along the 700m long jetty to a shiploader to an self-unloading transhipment vessel. Dredging of a berthing pocket at the end of the jetty (on the northern side) is required to allow the laden transhipper adequate water depth to remain within the berthing pocket without tidal restriction. The dimensions of the berthing pocket are 200m x 35m x 2.5m seabed depth (6 m water depth at low tide). The volume of material to be dredged for the berthing pocket is estimated to be 17,000m^3. Dredging would be carried-out by a cutter suction dredge. Dredge spoil disposal will occur on land within the project development envelope with appropriate management in place.
- **Sea Outfall -** The remaining hypersaline wastewater left from the crystallisation process is called bitterns. This concentrated salt solution flows from the crystalliser ponds into a bitterns dilution pond. The bitterns dilution pond will be located directly to the north of the northern set of crystalliser ponds. Seawater will be pumped into the bitterns dilution pond to dilute the bitterns prior to being discharged in the sea. The diluted bitterns will be pumped via a pipeline to the jetty for disposal offshore via an outfall equipped with a diffuser. The pipeline overland route will follow the salt conveyor route and will extend offshore along the export jetty. A bitterns pump station will provide the pumping requirements to transport the bitterns to the coast. A multi-port diffuser will be installed at the end of the pipeline to mix discharged bitterns with seawater.
- **Access Road** – An access road travelling in a north-easterly direction from the north-east tip of the crystalliser ponds towards the Ashburton River is required to allow access to the site from Onslow. Currently the road is in the preliminary design phase and as such road inverts and widths have not been supplied. Water Technology recommends the road is designed to the 5% annual exceedance probability (AEP) event with all events >5% overtopping via floodways.
- **Conveyor -** A 5 km long conveyor and adjacent access road are proposed to move the salt brine from the crystalliser to the jetty. The conveyor and its associated access road will travel in a westerly direction towards the jetty. The conveyor will be built on an embankment with culverts underneath to convey water flows.

1.3 Supporting Studies

The preparation of this report included physical data collection, a detailed review of scientific documentation as well as numerical modelling investigations. Supporting studies include:

- **Marine, Coastal and Surface Water Data Collection, Ashburton Salt Project, Water Technology 2021** - A physical data collection program was undertaken for this study which included the deployment of water level, wave and water quality data loggers. It also included the collection of water quality monitoring data, bathymetric data and current transects to assist in characterising the physical coastal environment. The data collected has been used to support the development and calibration of numerical models.
- **Marine, Coastal and Surface Water Existing Environment, Ashburton Salt Project, Water Technology 2021** - An extensive literature review and interpretation of field data was undertaken to document existing catchment, coastal and marine conditions within the local and regional environment. This report describes the existing environment based on desktop analyses and field data collection.

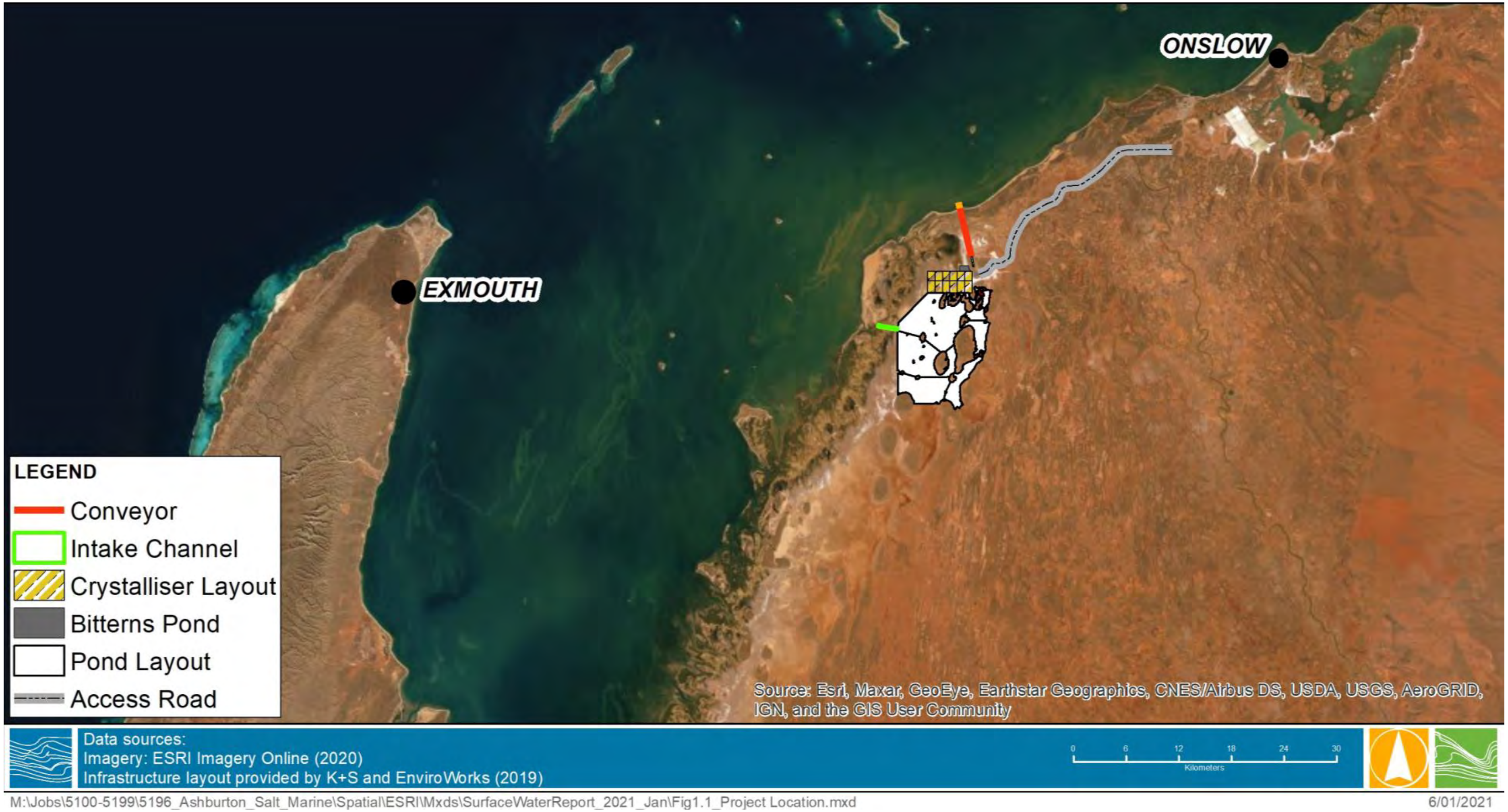

FIGURE 1-1 PROJECT LOCATION

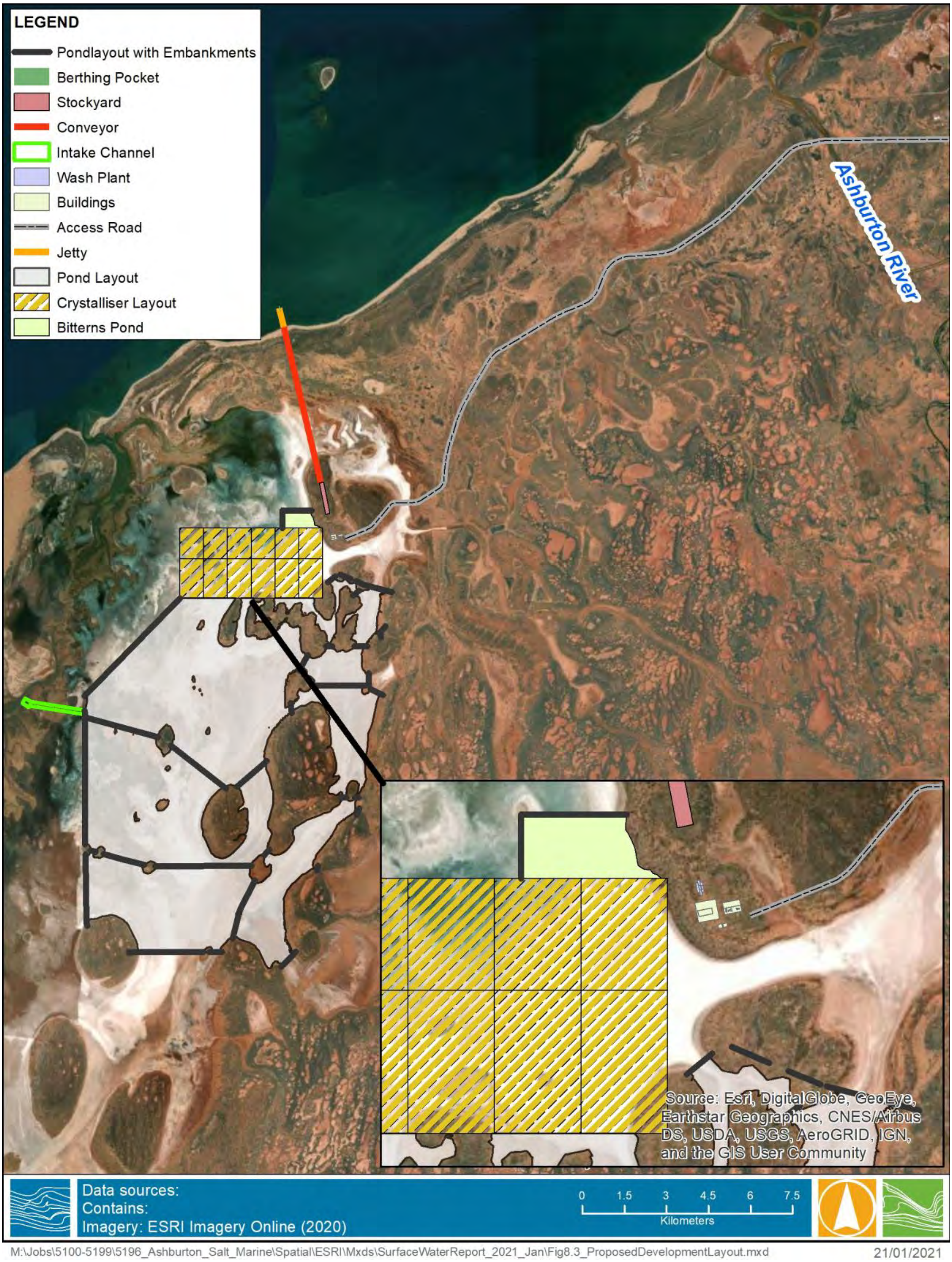

FIGURE 1-2 PROPOSED DEVELOPMENT LAYOUT

2 ENVIRONMENTAL OBJECTIVES AND TARGETS

2.1 Environmental Objectives

This study addresses the following EPA environmental objectives:

- To maintain the hydrological regimes of groundwater and surface water so that environmental values are protected; and
- To maintain the quality of groundwater and surface water so that environmental values are protected.

This report focuses on the surface water component of the objectives. Groundwater is addressed in a separate study.

2.2 Regulatory Framework

The Western Australian Environmental Protection Authority (EPA) has determined that the project is to be assessed under Part IV of the Environmental Protection Act 1986 (EP Act). The Australian Department of Agriculture Water and the Environment (DAWE) has determined that the proposal will be assessed under the Environment Protection and Biodiversity Conservation Act 1999 (EPBC Act) as a controlled action, via an accredited process. The requirements of both the EP Act and the EPBC Act are thus to be addressed.

In line with the Environmental Scoping Document submitted to the EPA, this report covers modelling work associated with the following environmental factors for inclusion in the environmental review:

- Hydrological Processes
- Inland Waters Environmental Quality

This study has been undertaken in accordance with the following regulatory frameworks:

- Environmental Factor Guideline: Inland Waters (EPA 2016a)
- Environmental Factor Guideline: Hydrological Processes (EPA 2016b)
- Australian and New Zealand Guidelines for Fresh and Marine Water Quality 2000 (Commonwealth) (ANZECC and ARMCANZ 2000);
- State Water Quality Management Strategy Document No. 6 (DoW, 2004);
- National Water Quality Management Strategy (ARMCANZ and ANZECC 1995).

3 EXISTING SURFACE WATER ENVIRONMENT

The existing environment at both a local and regional scale is described comprehensively in '*Marine, Coastal and Surface Water Existing Environment'*, Water Technology (2021). A brief summary of key features and processes considered relevant to this study are described below.

3.1 Meteorology

The climate at Ashburton is classified as hot, semi-arid with rainfall occurring from January through to July. The dry season occurs from late August through to December. There is a tropical cyclone season that runs from the middle of December to April with a peak occurring in the wet months of February and March.

Key climatic drivers are presented in Figure 3-1, presented by the Bureau of Meteorology (BoM, 2010). Along the Pilbara coast, the Indian Ocean Dipole, West Coast Troughs and Northwest Cloudbands dominate climatic conditions. In addition to this, the position of the subtropical ridge influences the seasonal change as the ridge shifts to the south in summer and to the north in winter, resulting in contrasting wet and dry seasons, respectively.

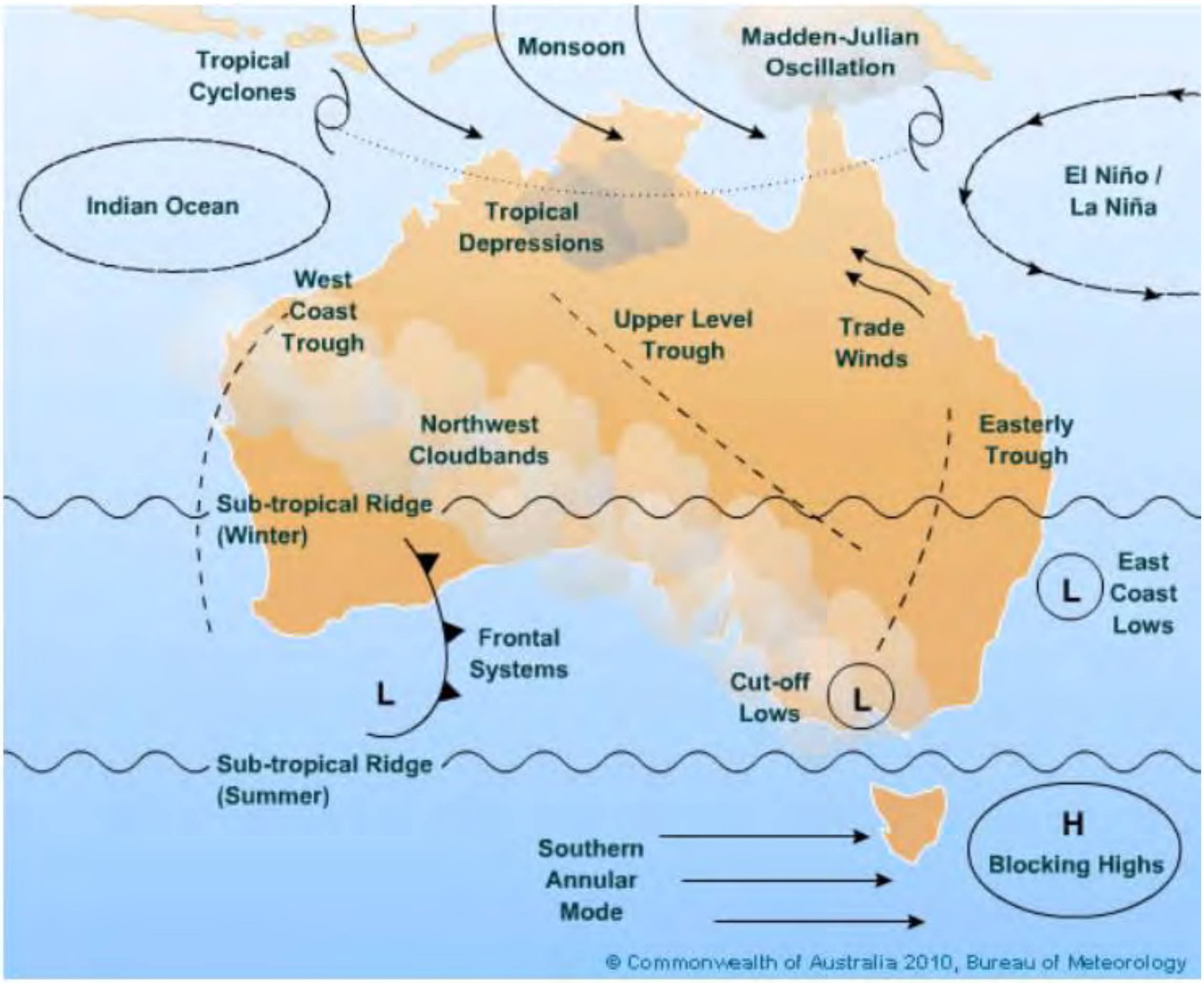

FIGURE 3-1 AUSTRALIAN CLIMATE DRIVERS (BOM 2010)

The project area is located within the Australian Southern Semi-arid Pasture Region land use zone. Due to the sparse and highly variable rainfall in this region, surface runoff is usually only generated during extreme weather conditions, typically associated with tropical cyclones (Blandford & Associates 2005).

3.2 Catchment Description

3.2.1 Overview

The proposed project is located approximately 40 km south west of Onslow Western Australia, between the Ashburton and Yannarie Rivers. Relevant external surface water catchments are shown in Figure 3-2. The red boundary denoted as the 'Hydraulic Model Extent' in Figure 3-2 represents local surface water catchment relevant to the proposed project.

The Ashburton River is the largest waterway in the vicinity the project site. It has a catchment area of approximately 71,000km^2 and has a defined waterway all the way to the coast. The river is perched between natural levee banks, and any flood waters that escape from the channel tend to fan out across the floodplain, both to the west and east. The floodplain comprises a range of landforms and when flood waters from the river reach the outwash plain inland of the project area, they inundate interdunal basins and claypans. Much of the water that reaches these storages is eventually lost through evaporation and to a lesser extent through infiltration. There is no direct connection of the Ashburton River to the project site, however there are some overland flow paths across the floodplain to the west of the main Ashburton River channel, which direct flows towards the salt flats and intertidal areas, including those near the project site.

The Yannarie River lies approximately 50 km to south east of the project site. It has a catchment area of approximately 4,300 km^2, and a stream length of 185 km. The channel becomes poorly defined where it reaches the outwash plain inland of the project site and its flood waters spread out across the outwash plain and dune field. Similarly, the adjacent Rouse Creek which has a catchment area of 1,700 km^2 and a stream length of 75 km has no defined channel once it reaches the outwash plain (Blandford et al 2005). As with Ashburton River flows, when waters from these systems reach the outwash plain, they flood interdunal basins and claypans, where much of the water is eventually lost through evaporation and to a lesser extent through infiltration. During significant flood events, water from these systems can enter the salt flats and intertidal areas to the west of the project area via overland flow paths.

Local rainfall across the local catchment also contributes to runoff toward the project area, during significant rainfall events.

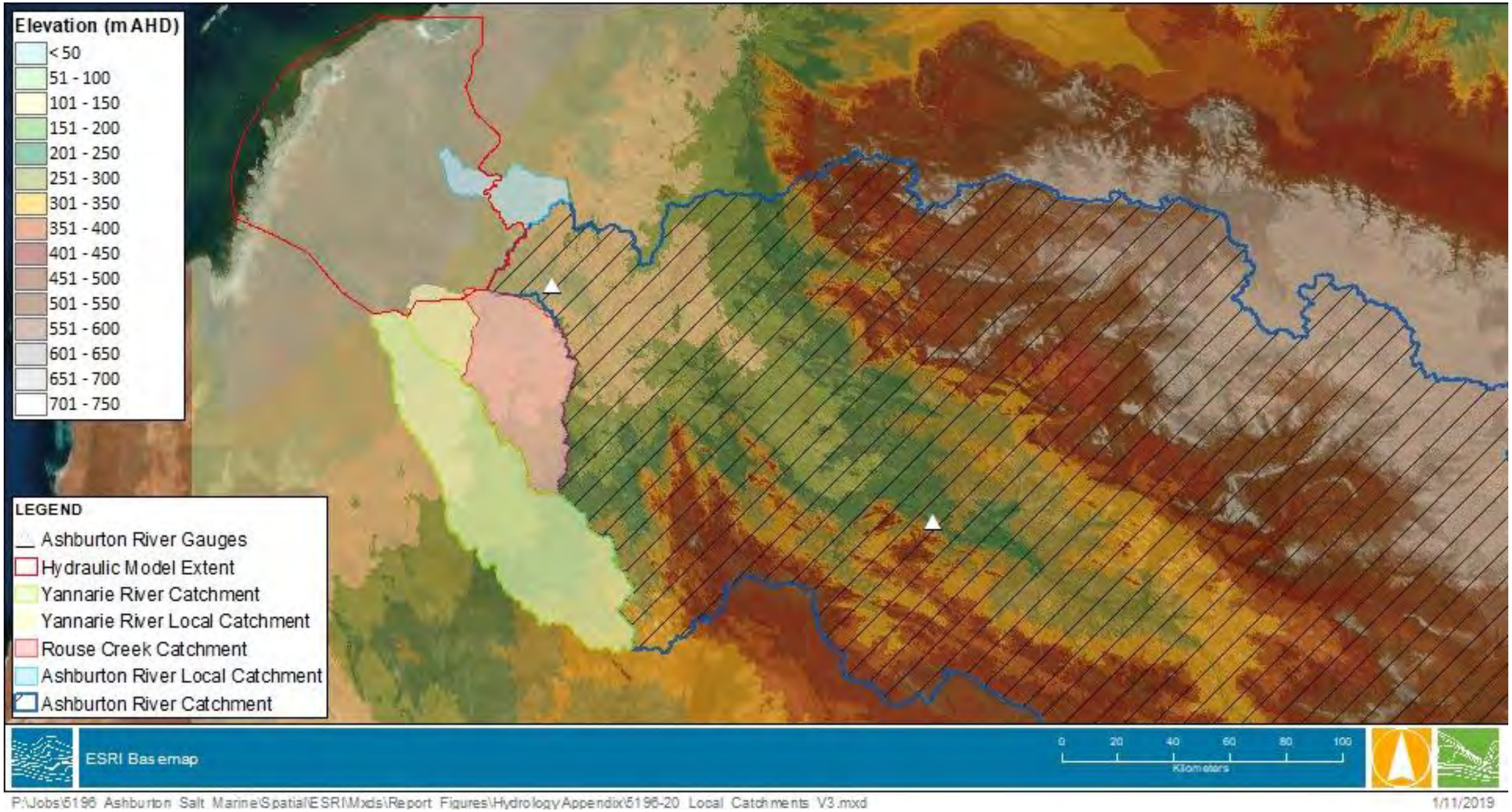

FIGURE 3-2 LOCAL CATCHMENT LOCATIONS

3.2.2 Catchment Geomorphology

The Ashburton River catchment exhibits high topographic relief with waterways typically remaining channelised upstream of Nanutarra. Downstream of this location, the topography becomes much flatter, and numerous possible flow breakouts and extensive floodplains occur, with most of this depositional and erosional zone classified as outwash plains (Blandford et al 2005).

The outwash plain landscape consists of alluvial and colluvial sediments. The alluvial sediments enter the plain from overland flows during flood events on the Ashburton and Yannarie Rivers, and consist of finer sediments such as clay, silt and fine sand particles. The colluvial sediments consist of coarser particles which include coarse sands and gravel (Blandford et al 2005).

Further downstream, overland flows traverse the remnant dune field (Dune Land System). The dune field begins 15 km inland from the coast and runs parallel to the coastline, covering an area of approximately 3,225 km^2. The dunes are predominantly orientated north to south and were formed by aeolian transport. Vegetation cover on the dunes is abundant, indicating that they are relatively stable. The rows of dunes display longitudinal depressions or swales between them, allowing water to flow between and around the dunes, and sometimes act as significant storages where water can pond (Blandford et al 2005). There are also several defined overland flow paths across this area.

Salt flats (part of the Littoral Land System) located on the seaward side of the dune field are typically inundated during extreme tide or storm events. During flood events on the Ashburton River, the area acts as an outlet for catchment flow paths. Given the low topographic gradient of the area, overland flows usually consist of shallow sheet flow across the area, with no clearly defined channels. The flats run from Sandalwood Peninsula to the mouth of the Harding River, covering an area of approximately 555km^2 (Blandford et al 2005).

The coastal fringe separates the salt flats and the coastline. The coastal fringe is comprised of beach systems, sand sheets and limestone outcrops, and is the final outlet for overland flows. Tidal creeks, such as Urala Creek North and Urala Creek South, are abundant over the landscape and provide mangrove habitat.

3.2.3 Key Flow Paths

Surface water flow paths in and around the project site is a complex interaction between watercourses including the Ashburton River, Yannarie River and Rouse Creek and the wide outwash plain, salt flats and dune fields adjacent to the coast.

Catchment inflows to the project area have been modelled within subsequent sections of this report. The generalised flow paths identified from this modelling are mapped below in Figure 3-3.

Breakout overland flows from Yannarie River and Rouse Creek typically enter the coastal system 35 km to the south of the proposed project. Yannarie River itself is located 50 km south east of the proposed project, whilst Rouse creek is located approximately 75 km to the south east of the proposed project.

Breakout overland flows from the Ashburton River combined with local runoff create sheet flow conditions across the catchment and flows that pass through the inland dune field and claypan system.

Overland flows from the hinterland dune field immediately to the east of the project enter the salt flats via large local basins adjacent to the eastern boundary of the proposed salt evaporation ponds.

To the immediate north and south of the proposed project local flows are conveyed along more defined local flow paths, specifically ‘Chinty Creek’ to the north and an unnamed flow path to the south (Figure 3-3).

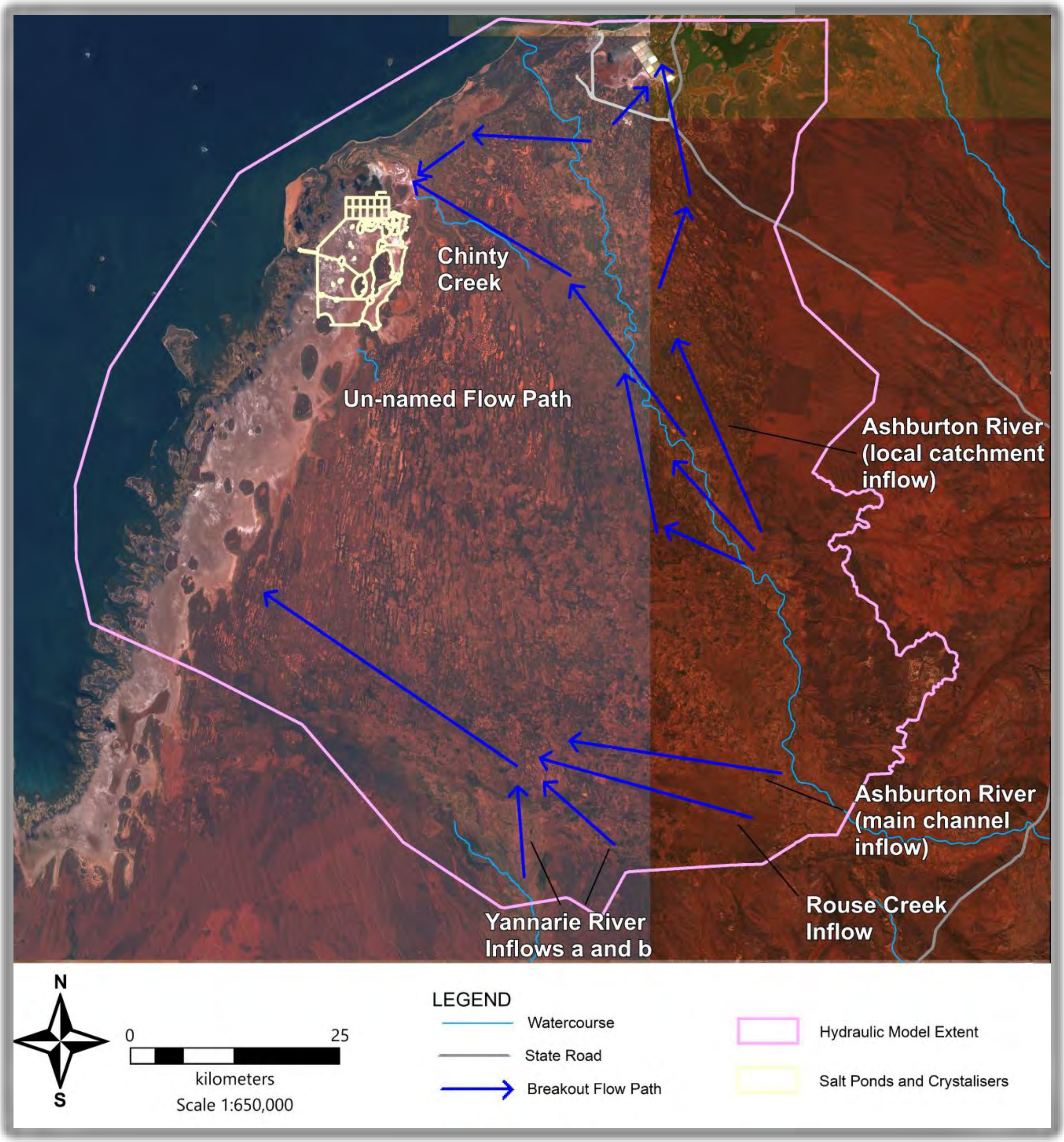

FIGURE 3-3 GENERALISED INFLOWS TO PROJECT AREA AND FLOW PATHS

3.3 Inland Surface Water Quality

3.3.1 Regional Surface Water Quality

The Ashburton River is located approximately 25 km north east of the proposed salt ponds. The Ashburton River is generally fresh, with Total Dissolved Solids (TDS) (a measure of salinity) being around 133 mg/L (Ruprecht and Ivanescu, 2000). This is similar to other rivers in the Pilbara region (TDS range 50 - 1,000 mg/L).

Salinity in the Ashburton River, and all Pilbara region rivers, generally decreases with increasing flow and becomes more saline during times of low flow (URS, 2010b).

Total Suspended Solids (TSS) and turbidity in the Ashburton River are generally low, and generally increase with increasing flow. The turbidity of the Ashburton River ranges from less than 10 NTU over a range of flows, from 30 m3/sec to 250 m3/sec, to 3,300 NTU at a flow rate of around 250 m3/sec. The flow weighted turbidity for Ashburton River is 1,705 NTU, which is higher than other Pilbara river sites, which range from 10 - 587 NTU (Ruprecht and Ivanescu, 2000).

3.3.2 Local Surface Water Quality

Due to the low frequency of significant rainfall events resulting in surface water flows or flooding, limited local surface water quality data is available. Two significant rainfall events have occurred in the project area since 2019 which have allowed K+S to sample the flooded salt flat areas (one rainfall event of 44 mm in April 2019 and another of 79.5 mm in March 2021). The data from this sampling is presented in *Marine, Coastal and Surface Water Data Collection Report* (Water Technology, 2021). The results show:

- Total Dissolved Solids (TDS) measurements indicate that surface water is saline to hypersaline on the salt flats with TDS in salt flat samples ranging from 45,000 mg/L to 120,000 mg/L.
- pH across the salt flats and inland flow paths ranged from neutral to slightly alkaline (pH range 7.3 – 8.6).
- Total Suspended Solids (TSS) varied significantly with lower levels on the salt flats (<5 – 87 mg/L) and higher levels inland of the salt flats (up to 19,000 mg/L). Levels within an inland flow path were extremely high (resembling a slurry) at 510,000 mg/L.
- Levels of chlorophyll-a were low in all samples (<0.001 to 0.006 mg/L) except that from the overland flow sample which resembled a slurry (0.32 mg/L).
- The mean total nitrogen concentration across nine sites (excluding the high sediment sample) was 1.1 mg/L. The high sediment sample was excluded as it was more a sediment slurry, as opposed to a representative surface water sample.
- Samples were comprised of predominantly dissolved organic nitrogen (ranging from <.0.2 mg/L to 1.7 mg/L).
- The overland flow sample which resembled a slurry and had high total nitrogen content (120 mg/L), representative of nitrogen within the sediments from overland flows.
- Phosphorus was highest at the most inland sites and largely particulate at these locations. The sites with the high phosphorus also corresponded to sites with the highest TSS. This is the result of phosphorus adsorption to sediment. This observation adds further confidence to the assertion that the environment is nitrogen limited, as there is phosphorus readily available in soils across the site.
- Nitrogen in the water ponding on the bare salt flats is low compared with the other samples, particularly those received as suspended solids in overland flows (such as the highly turbid water from overland flows entering the salt flats). The data shows that the bare salt flats do not generate comparatively large amounts of nitrogen in ponded water, even after inundation with rainfall, compared with turbid overland flows/ponding from the hinterland.
- High levels of total dissolved solids (TDS) in the samples from the bare salt flats indicate that the surface salt crust was dissolving into the ponded water on the bare salt flats, but there are comparatively low levels of nitrogen in this dissolved salt crust compared with overland flows.

4 SURFACE WATER MODELLING APPROACH

4.1 Overall Approach

An extensive literature review was conducted to document existing hydrological conditions and available rainfall and streamflow data at the site and within upstream contributing catchments. This literature review identified available data, desktop analyses, and the rainfall-runoff modelling (i.e., RORB) used to inform the numerical flood modelling.

A suite of numerical models was developed to enable the simulation of surface water flows upstream and through the study area:

- The Danish Hydraulic Institute's (DHI) MIKE Modelling suite was used to assess the conditions and impacts of the development regionally.
- The MIKE model utilised inflows generated using RORB.
- The 2D TUFLOW hydraulic modelling software package utilised inflows from the regional MIKE model and was developed to assess the impacts of the proposed development more locally and at a finer resolution.

4.2 Extent and Resolution

The extent and resolution of the regional MIKE model is presented in Figure 4-1 whilst the local 2D TUFLOW model is displayed in Figure 4-2.

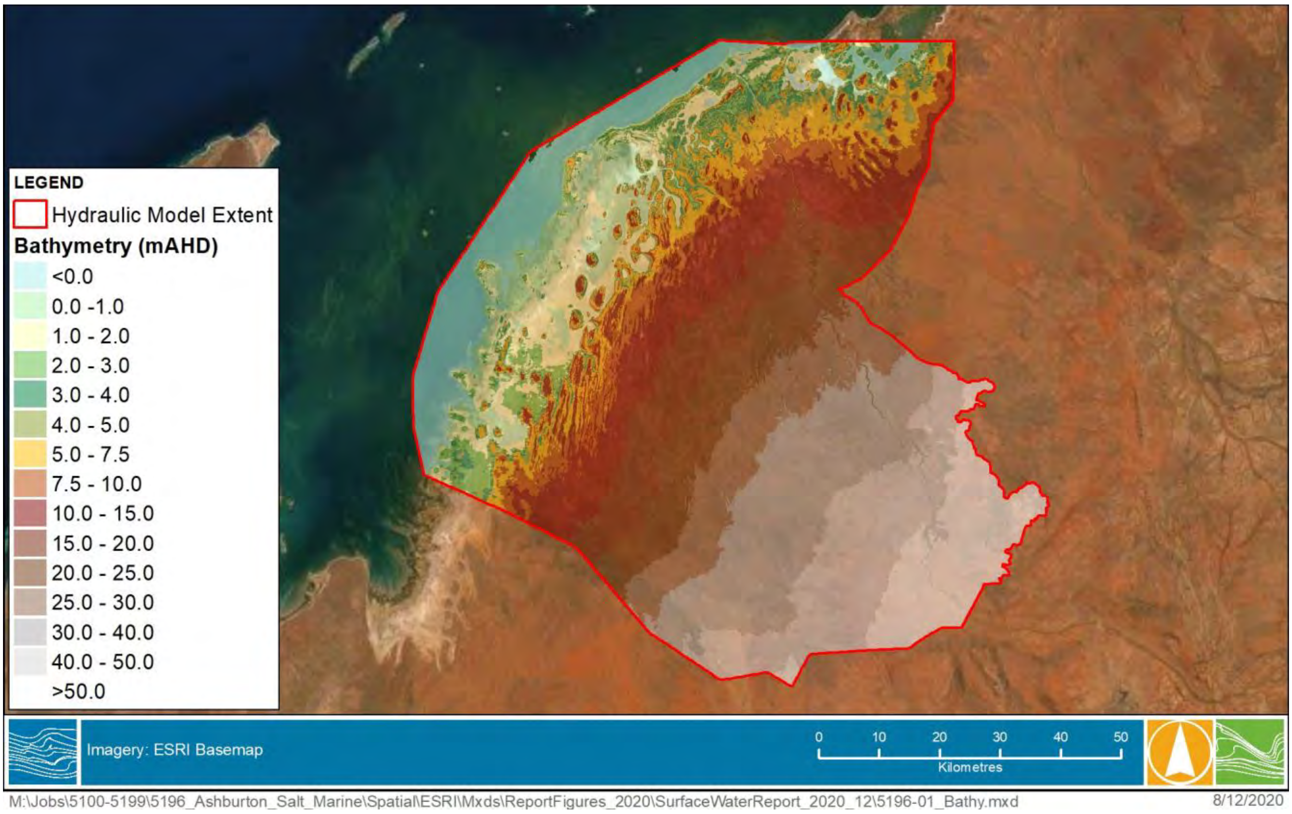

FIGURE 4-1 MIKE-21 MODEL BOUNDARY AND TOPOGRAPHY

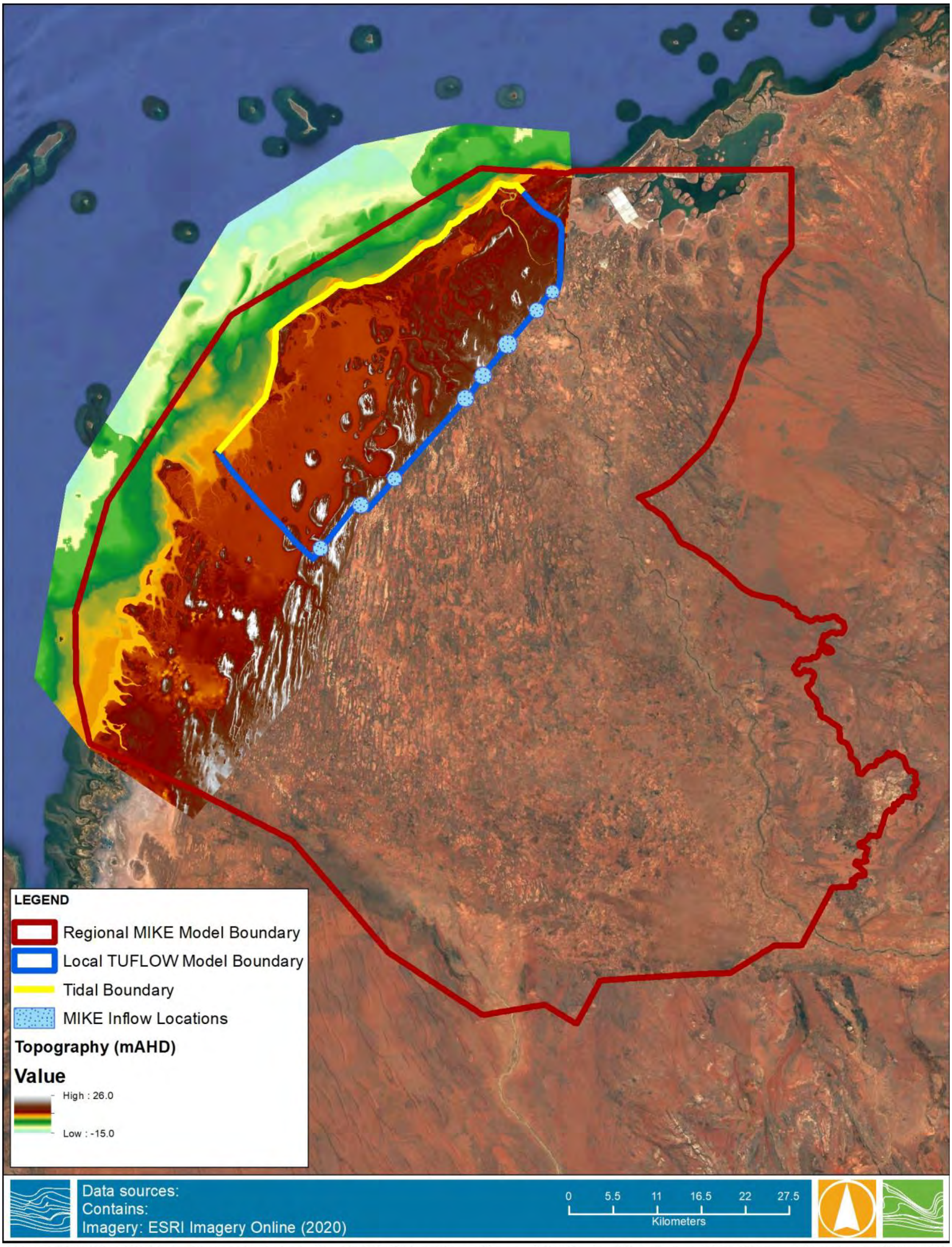

FIGURE 4-2 TUFLOW LOCAL HYDRAULIC MODEL BOUNDARY AND INFLOW LOCATIONS

4.3 Inflow Generation

Due to the proximity and high-quality gauging data of the Ashburton River at Nanutarra gauge, this gauge was used to provide direct input into the hydraulic model for calibration.

For the generation of design event flows, a flood frequency analysis was undertaken on this gauge, and the largest flow on record (February 1997) was scaled to the design event flows.

There are also four ungauged catchments that are included as boundaries into the hydraulic model, and to generate hydrographs for these catchments a RORB rainfall-runoff hydrology model was used.

Measured rainfall and best practice parameter estimates were utilised to give hydraulic model inflows for the calibration event at the ungauged catchments.

For the design events, the flows were compared to both the Flood Frequency Analysis (FFA) and Regional Flood Frequency Estimation (RFFE) methods.

Parameters for the RORB model were selected based on these comparisons and consistency with recommendations made in ARR2016, including those from Pearcey et al (2014).

Further details regarding the above, are provided within the Sections below.

5 LITERATURE REVIEW INFORMING MODELS

A literature review of previous studies conducted in the vicinity of the project was undertaken as described below, to inform the suite of models developed.

5.1 Yannarie Salt Project

In 2006, Parsons Brinckerhoff undertook an assessment of the 'Yannarie Salt Project' which included hydraulic and hydrologic studies in support of the development. These studies incorporated validation of hydrology-generated flows using software named AFFLUX (a package developed by Main Roads Western Australia) and modelling the hydraulics of the surrounding floodplain using the EXTRAN modelling package. This project's modelling was mainly focussed on specified bridge locations along the Yannarie River and along Rouse Creek and was predominantly used to compute velocities and head losses at those specific bridge structures.

Some basic details were provided regarding the hydraulic model in that it requires upstream inflows from the hydrologic model and that it produces downstream outflows at several key locations. More specifically, routing within the model uses upstream inflows, downstream outflows and a node to represent floodplain storage. Of note, this model specifically ignores continuing losses such as infiltration and evaporation and as such is considered conservative for flow derivation. The model was stated as being 'calibrated' to Tropical Cyclone Vance (March 1999) by modifying outflow inverts until one of the model outflows stopped flowing after 3 days of the storm, which was observed in satellite imagery (Parsons Brinckerhoff, 2006).

5.2 Macedon Gas Development

In 2010, URS was engaged to model the potential hydraulic impacts of the proposed Macedon Gas Plant development. Hydraulic modelling of this site was undertaken using MIKE FLOOD (software developed by the Danish Hydraulics Institute (DHI)), which coupled two-dimensional and one-dimensional modelling suites together to represent the floodplain. The model used a fixed square grid with cell size of 40 m to represent the floodplain and was based on LiDAR survey and Geoscience Australia topography data available at the time. The model however only represents the far downstream part of the Ashburton River near Onslow and structures (such as culverts/bridges) were represented in the 1D domain, whilst the floodplain was represented in the 2D domain. Hydrographs from a validated hydrology model were applied to the hydraulic model and the downstream boundary was set to open ocean water levels. No calibration data was available at the time of development, so the model was not calibrated. Design events were simulated representing scenarios with and without the Gas Development to estimate potential floodplain impacts due to the development. This model was also utilised to establish freeboard to flood levels on different parts of the development (URS, 2010a).

5.3 RORB kc Parameter for Ungauged Catchments in the Pilbara

A study was undertaken by Pearcey et al (2014) to summarise the hydrologic model RORB routing parameters calibrated for rivers in the Pilbara region. The routing parameters calibrated in RORB were the non-linearity exponent m and the coefficient kc. The exponent m is usually set to 0.8 and this was concluded by the authors to be appropriate in north-west Australia. It was found that the routing coefficient k_c could be estimated from the d_{av}, the average flow distance in the channel network from the centroid of each sub-area to the catchment outlet, by the following equation:

$K_c = C * d_{av}$

The mean value of C for calibrated Pilbara catchments was found to be 0.59, and values of ± one standard deviation were 0.71 and 0.48 respectively. The C value was constant with catchment area and not subject to a spatial trend. There was some evidence that a lower C value may be appropriate for steeper catchments, but there was insufficient data to make strong conclusions.

6 ASHBURTON RIVER INFLOW GENERATION FOR MODELLING

6.1 Flow data availability and quality

Flow data is available for two gauging stations in the Ashburton River catchment:

- 706003 Ashburton River – Nanutarra; and
- 706209 Ashburton River – Capricorn Range.

The flow gauging data indicates a strong seasonal pattern in stream flows with a wet season from December to June and a dry season from July to November. Details of the two gauges are provided in Table 6-1.

TABLE 6-1 GAUGING STATION DETAILS FOR ASHBURTON RIVER GAUGES

	706003 Ashburton River – Nanutarra	706209 Ashburton River – Capricorn Range
Catchment area (km^2)	71,387	43,098
Period of record	21/06/1972 – 05/12/2020	09/01/1968 – 21/04/2020
Years	48	53
Highest gauged stage	18.03 m on 7/02/1997	16.28 m on 25/01/1973
Highest recorded stage	18.128 m on 7/02/1997	22.284 m on 12/12/1975
Highest recorded flow (m^3/s)	12,612	8,314
Significant gaps	Various minor gaps up to 66 days	12/02-28/07/1972, 31/01-08/05/1997, and various minor gaps up to 73 days
Site photo		

The Nanutarra gauge is the furthest downstream and the closest to the study area, and has good period of record, coverage and rating curve quality. Furthermore, the major events at each gauge are different, likely because they were driven by different rainfall patterns. Flow time series for the Nanutarra gauge is shown in Figure 6-1. The February 1997 event was the highest flow on record in the lower catchment. The gauge record indicates that this event was driven entirely by rainfall downstream of the Capricorn Range gauge. Given the study area is at the downstream end of the system, the hydrological analysis was undertaken on data from the Nanutarra gauge (706003) in the lower catchment.

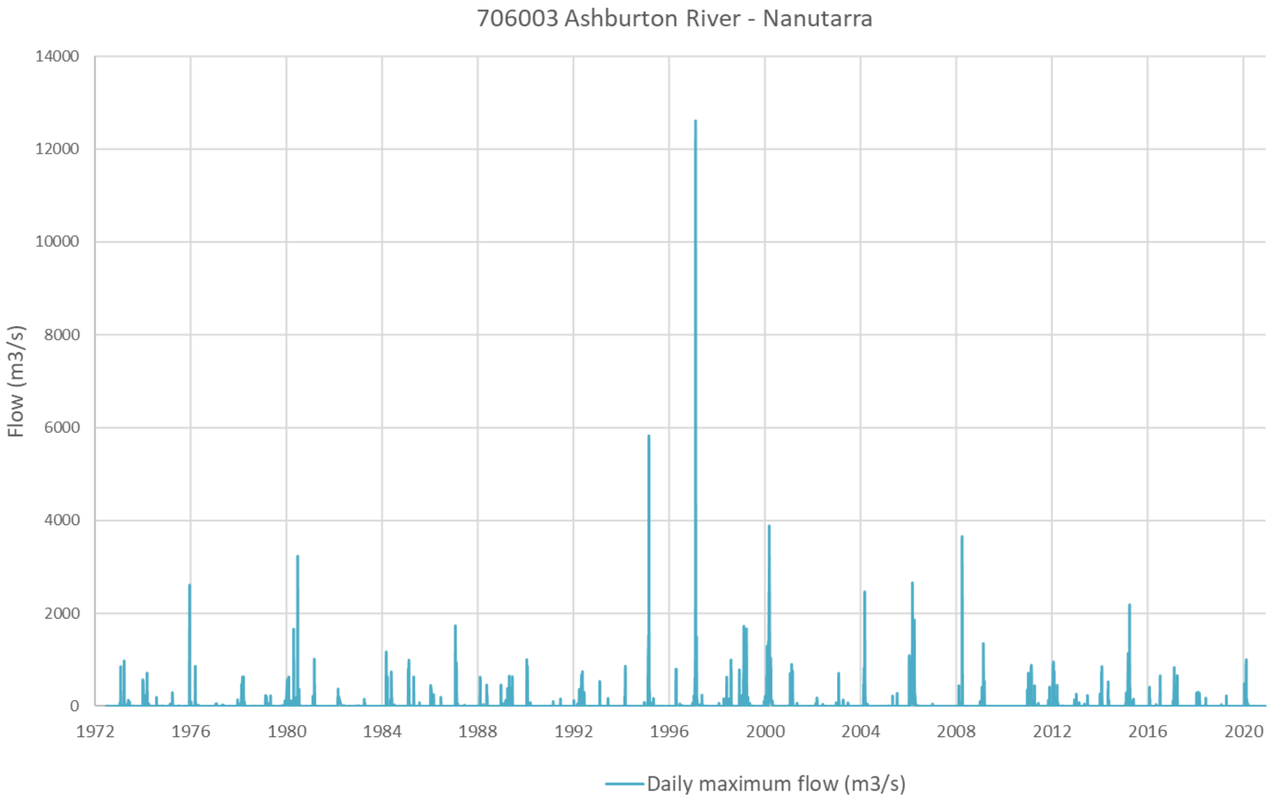

FIGURE 6-1 DAILY MAXIMUM FLOW TIME SERIES FOR 706003 ASHBURTON RIVER - NANUTARRA

6.2 Flood Frequency Analysis – Peak Flows

A Flood Frequency Analysis (FFA) was undertaken at Nanutarra (706003) to provide estimates of design flow magnitude. An annual series was compiled of the maximum flow in each flow year, with the flow year defined from October to September. This flow year was used instead of a calendar year because the wet season spans the December-January period and maximum flows in consecutive calendar years may not be independent.

The FFA was undertaken on the annual flow series from the gauge records using the analysis program FLIKE, as recommended in the updated ARR guidelines. FLIKE uses a Bayesian approach to parameter fitting to the records in order to assess the return period of different magnitude flows. There are a number of probability distributions which can be used to undertake an FFA, including the Log Pearson III, Log-Normal, Generalised Pareto, Generalised Extreme Value and Gumbel distributions and a selection of these are applied in the analysis, with the 'best fit' distribution adopted in the final assessment.

The models were fitted without prior information on parameters, as the catchment area is well outside the limit of application of current regional parameter estimation methods. The Rahman et al. (2015) Regional Flood Frequency Estimation (RFFE) method described in Chapter 3, Book 3 of Australian Rainfall and Runoff (ARR) is applicable only up to catchment areas of 1,000 km^2.

6.2.1 Peak Flows at Ashburton River – Nanutarra

An annual series was developed for the flow year ending September 1973 to the flow year ending September 2016 (44 years), as shown in Table 6-2. The gauge record started in June 1972 but there was not sufficient data to include the 1972 flow year in the series. The 2016 flow year was also incomplete, but the high flow period was mostly present, and it was considered unlikely a higher flow would have occurred in the missing period from mid-June to September 2016. Other flow years with missing data were inspected and it was considered that the peak flow was likely to be missing in the 1986 flow year but present in all other years. The 1986 peak flow was included as censored peaks above a threshold. The 2010 flow year had zero flow recorded for the whole year. The flows were not flagged as missing data or any other quality issue and were flagged as having low uncertainty, therefore the zero value for annual peak flow was considered accurate. Data gaps were inspected and were considered unlikely to contain any flows higher than those recorded as the annual maximum for that year.

TABLE 6-2 ANNUAL SERIES FOR 706003 ASHBURTON RIVER – NANUTARRA

Flow year ending 30 September	Maximum annual flow (m^3/s)	Comments
1972	-	incomplete year - removed
1973	977	
1974	716	
1975	294	
1976	2,609	
1977	59	
1978	633	
1979	233	
1980	3,232	
1981	1,013	
1982	373	
1983	153	Low flow censored
1984	1,171	
1985	995	
1986	449	peak flow likely missing. Included as censored peak > 449 m^3/s
1987	1,735	
1988	628	
1989	651	
1990	1,004	
1991	159	
1992	747	
1993	536	
1994	864	
1995	5,826	
1996	799	
1997	12,612	Highest recorded flow
1998	998	

Flow year ending 30 September	Maximum annual flow (m^3/s)	Comments
1999	1,724	
2000	3,887	
2001	903	
2002	180	
2003	712	
2004	2,467	
2005	279	
2006	2,660	
2007	51	Low flow censored
2008	3,658	
2009	1,351	
2010	0	Zero flow year - quality checked, Low flow censored
2011	885	
2012	954	
2013	262	
2014	857	
2015	2,186	
2016	412	Incomplete year – included

To prevent skewing of the data, low flows were censored using the Multiple Grubbs Beck Test, which resulted in the removal of the 3 lowest flows in the series. Censoring of low flows is significant due to the presence of non-flood years in the gauge annual series which can skew the analysis.

In addition to the low flow censoring, one annual peak was censored (for the flow year ending September 1986). The flow record was incomplete for that year and it was judged that the annual peak was likely not recorded. The record was removed from the annual series, and a single censored peak flood peak above the threshold of 449 m^3/s (the maximum recorded flow for that year) was included.

A Log Pearson III model was fitted to the annual series (Figure 6-2) and the resulting peak flow estimates are given in Table 6-3.

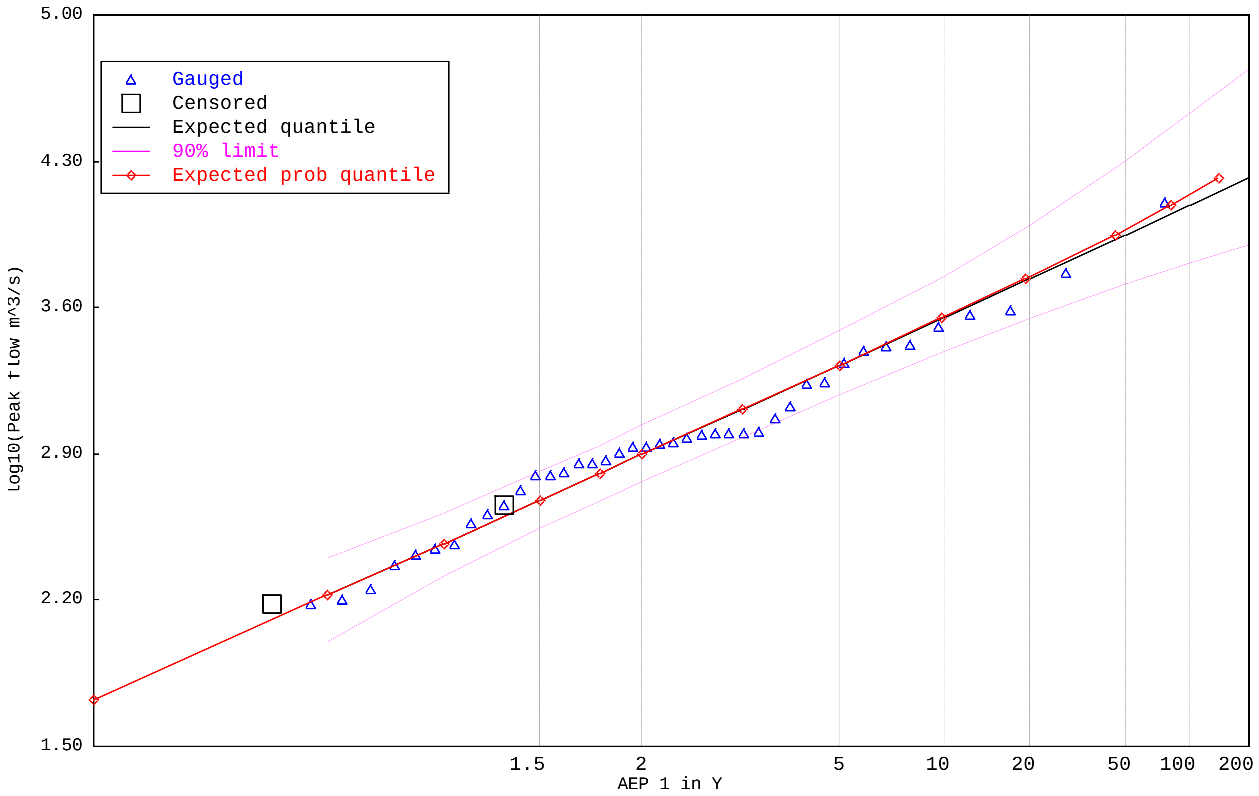

FIGURE 6-2 LOG PEARSON III DISTRIBUTION FITTED TO ANNUAL SERIES FOR 706003 ASHBURTON RIVER – NANUTARRA

TABLE 6-3 DESIGN PEAK FLOWS (M³/S) FOR 706003 ASHBURTON RIVER – NANUTARRA

AEP	LPIII Peak Flow Estimate (m^3/s)	5-95% Confidence Limits
10%	3,609	2,473 – 5,736
5%	5,534	3,562 – 9,870
2%	8,950	5,219 – 19,604
1%	12,327	6,580 – 31,926
0.5%	16,521	8,028 – 51,688

6.2.2 Volumes at Ashburton River – Nanutarra

Inspection of the recorded flow data showed that the larger flood events (10% AEP and greater) at Nanutarra tended to last from 4 to 20 days. Many of the largest events were compound events from multiple bursts, and when considering only the largest peak the duration was more consistent, ranging from 4 to 7 days with an average of 5 days. The volume FFA was therefore assessed using a duration of 5 days.

The 5-day volume was calculated across the entire time series (using hourly data) at 706003 Nanutarra.

An annual series was developed for the flow year ending September 1973 to the flow year ending September 2016 (44 years), as shown in Table 6-2.

TABLE 6-4 ANNUAL PEAK FLOW SERIES FOR 706003 ASHBURTON RIVER – NANUTARRA

Flow year ending 30 September	Maximum 5 day flow volume (GL)	Comments
1972	-	incomplete year - removed
1973	248	
1974	134	
1975	45	
1976	604	
1977	6	
1978	136	
1979	38	
1980	746	
1981	142	
1982	53	
1983	21	Low flow censored
1984	334	
1985	241	
1986	54	Peak flow likely missing. Included as censored peak > 54 m^3/s
1987	528	
1988	194	
1989	134	
1990	241	
1991	24	
1992	201	
1993	87	
1994	170	
1995	1,134	
1996	233	
1997	2,935	Highest recorded flow volume
1998	295	
1999	539	
2000	1,153	
2001	170	
2002	41	
2003	131	
2004	644	

Flow year ending 30 September	Maximum 5 day flow volume (GL)	Comments
2005	59	
2006	811	
2007	3	Low flow censored
2008	681	
2009	260	
2010	-	Zero flow year - quality checked Low flow censored
2011	284	
2012	340	
2013	60	
2014	187	
2015	393	
2016	76	Incomplete year – included

To prevent skewing of the data, low flow volumes were censored using the Multiple Grubbs Beck Test, which resulted in the removal of the 3 lowest flow volumes in the series.

In addition to the low flow censoring, one annual maximum flow volume was censored (for the flow year ending September 1986). The flow record was incomplete for that year and it was judged that the annual peak was likely not recorded. The record was removed from the annual series, and a single censored peak flood peak above the threshold of 54 m^3/s (the maximum recorded 5-day flow volume for that year) was included.

A Log Pearson III model was fitted to the annual series (Figure 6-3) and the resulting 5-day flow volume estimates are given in Table 6-5.

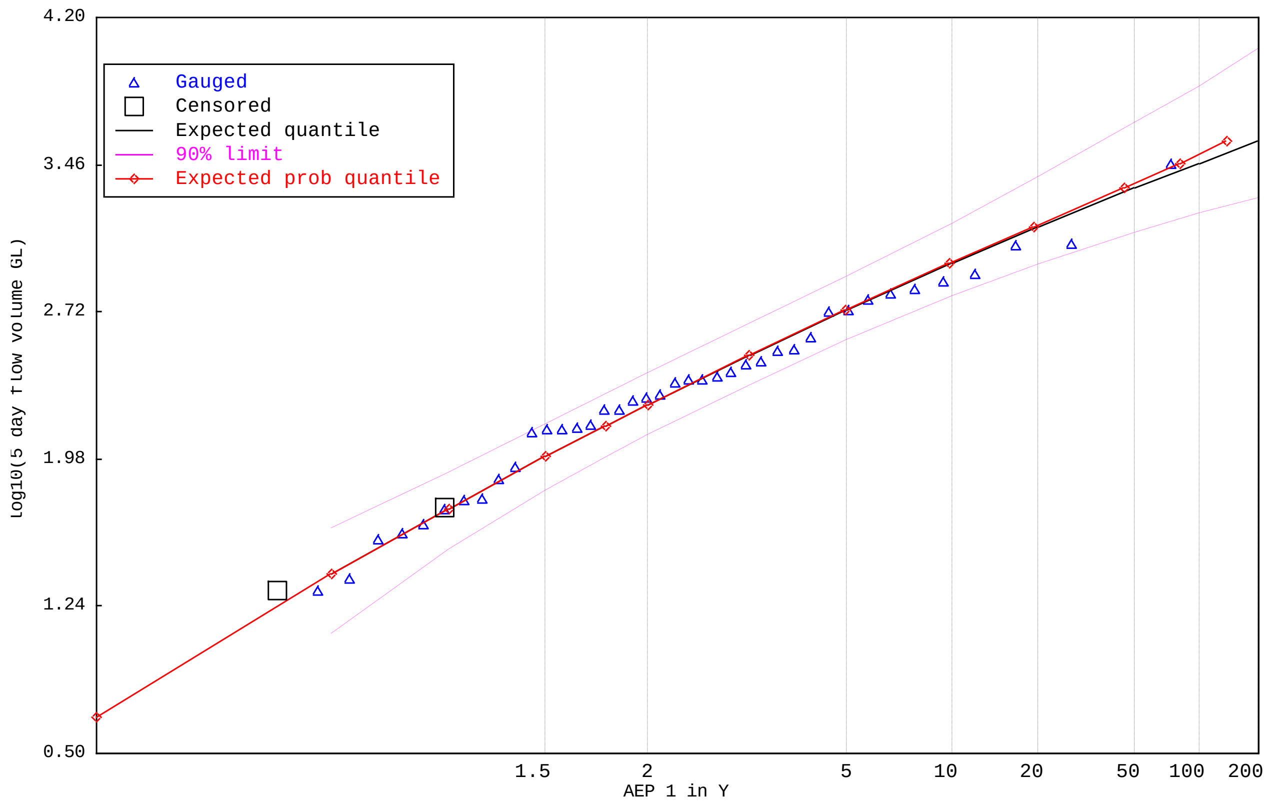

FIGURE 6-3 LOG PEARSON III DISTRIBUTION FITTED TO ANNUAL 5 DAY VOLUME SERIES FOR 706003 ASHBURTON RIVER – NANUTARRA

TABLE 6-5 DESIGN 5 DAY FLOW VOLUMES FOR 706003 ASHBURTON RIVER – NANUTARRA

AEP	LPIII 5 Day Flow Volume Estimate (GL)	5-95% Confidence Limits
10%	919	623-1,446
5%	1,398	906-2,472
2%	2,193	1,317-4,689
1%	2,923	1,641-7,135
0.5%	3,769	1,966-10,913

6.2.3 Comparison to Previous Design Flow Estimates

A flood frequency analysis was undertaken for the Ashburton River at Nanutarra in the Wheatstone Project Draft Environmental Impact Statement (URS, 2010b). The method used was not described in detail, and confidence limits were not reported. The approximate design flows presented on a flood frequency plot in that report are listed in Table 6-6. These previous estimates are lower than the current FFA results, particularly for rarer floods. Based on the FFA, the 1997 event was estimated to have an AEP of around 0.2%, whereas the current analysis indicates the AEP of this event is around 1%. The method used for the current FFA is likely more robust and has been reported in more detail than the previous work.

TABLE 6-6 DESIGN PEAK FLOWS FOR 706003 NANUTARRA (WHEATSTONE PROJECT, 2010)

AEP	LPIII Peak Flow Estimate (m^3/s) to nearest 1,000
10%	3,000
5%	4,000
2%	6,000
1%	7,000
0.5%	8,000

6.3 Calibration Data

6.3.1 Calibration Event

The March 1999 event was selected for calibration of the hydraulic model. The AEP of the 1999 event is thought to be slightly smaller than a 10% AEP event. The March 1999 event (Tropical Cyclone Vance) was chosen (despite not being the largest event) as LandSat imagery was available for validation of the hydraulic model flood extents for this event. Hydraulic model calibration of the event is further discussed in Appendix B.

Details of the selected calibration events are given in Table 6-7.

TABLE 6-7 RECORDED EVENTS

Calibration Event	Period	Peak flow at 706003 Nanutarra (m^3/s)	Approx. AEP at 706003 Nanutarra
March 1999	22/03/1999 – 5/04/1999	1,667	>10%

6.3.2 Calibration Event Rainfall Data

Pluviograph (rainfall intensity) data was available for the stations listed in Table 6-8. These pluviographs were used to give a temporal pattern to each local catchment, based on the sub-catchment proximity to each gauge for the 1999 calibration event modelling.

TABLE 6-8 PLUVIOGRAPH (RAINFALL INTENSITY) STATIONS IN AND AROUND CATCHMENT

Site	Name	Start	End	In catchment
5015	Mulga Downs	Sep-98	June-18	N
5069	Pannawonica	Nov-71	June-18	N
6072	Emu Creek Station	Jul-72	June-18	N
7019	Bulloo Downs	Jul-98	June-18	Y
7025	Wanna	Sep-98	June-18	N
7059	Mount Vernon	Jul-98	June-18	Y
7083	Turee Creek	Sep-98	June-18	Y
7152	Kumarina	Jun-98	June-18	N
7165	Mingah Springs	Jul-98	June-18	N

For spatial patterns for the 1999 Event, the Bureau of Meteorology's (BOM) Australian Water Availability Project (AWAP) gridded rainfall data was used.

6.4 Flow Data

The hydrograph for the 1999 calibration event at 706003 Nanutarra is shown in Figure 6-4.

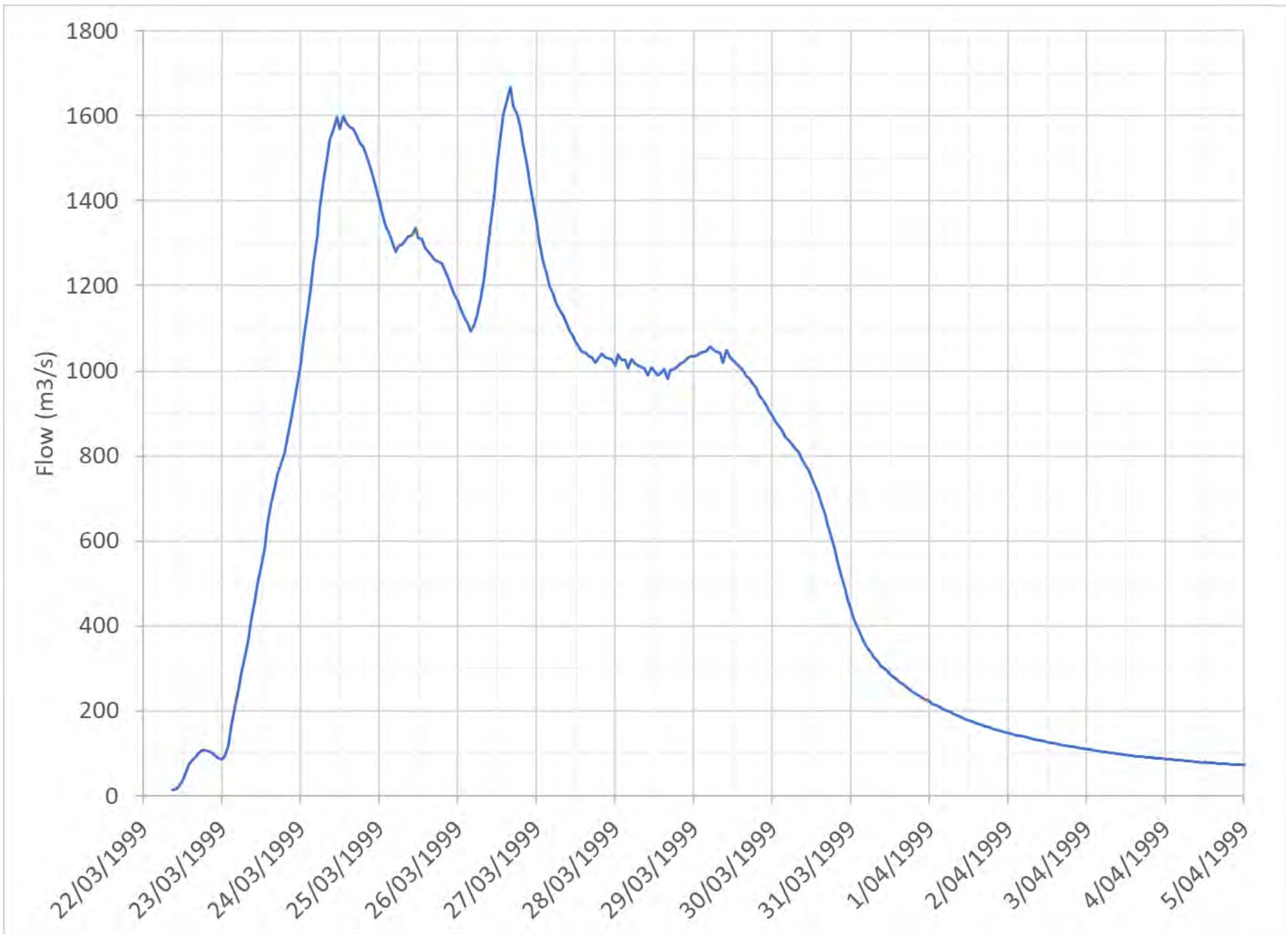

FIGURE 6-4 ASHBURTON RIVER AT NANUTARRA HYDROGRAPH FOR MARCH 1999 CALIBRATION EVENT

6.5 Design Flood Hydrographs

Design flood hydrographs were required for the Ashburton River and four local catchments for input to the hydraulic model, for the 0.5%, 1%, 2%, 5%, and 10% AEP.

A scaled recorded hydrograph from Nanutarra was used for the design flood events for the Ashburton River. The gauge captures over 99% of the catchment to the study area boundary. The four largest gauged events are summarised in Table 6-9. Of these events, the February 1997 event was selected as the most appropriate for scaling, as it has a single peak, its peak flow and volume have the same AEP, and it is within the range of required AEPs for design flows.

TABLE 6-9 GAUGED EVENTS AT 706003 NANUTARRA

Event	Period	Peak flow at 706003 Nanutarra (m^3/s)	Approx. AEP of peak flow	5 day flow volume at 706003 Nanutarra (GL)	Approx. AEP of volume
February 1995	25/02/1995-2/03/1995	5,826	5%	1,134	7%
February 1997	6/02/1997-11/02/1997	12,612	1%	2,935	1%
March 2000	8/03/2000-13/03/2000	3,887	10%	1,153	7%
March 2008	30/03/2008-4/03/2008	3,658	10%	681	16%

The 1997 hydrograph was scaled to the design peak flow and volume for each AEP, as shown in Figure 6-5.

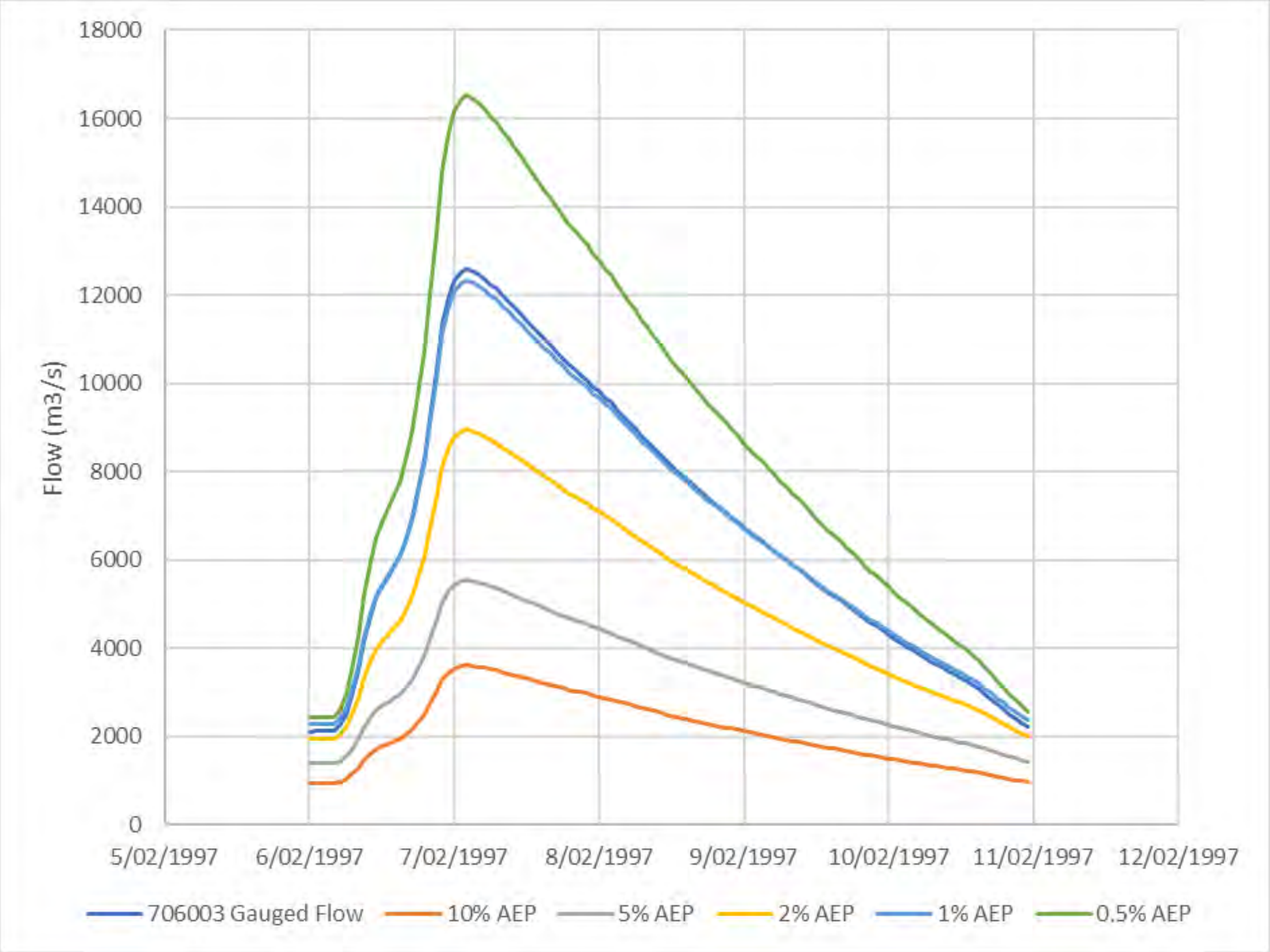

FIGURE 6-5 DESIGN EVENT HYDROGRAPHS FOR 706003 ASHBURTON RIVER AT NANUTARRA (SCALED FROM 1997 GAUGED HYDROGRAPH)

7 LOCAL CATCHMENT INFLOW GENERATION FOR MODELLING

7.1 Hydraulic Model

Other than the Ashburton river, four smaller rivers flow into the hydraulic model domain as shown in Figure 7-1. The runoff from these catchments was estimated using RORB, a rainfall-runoff program (Version 6.45). A RORB model was set up for each catchment.

RORB is a non-linear rainfall runoff and streamflow routing model for estimation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into subareas, connected by a series of conceptual reaches and storage areas. Observed or design storm rainfall is input to the centroid of each subarea. Specific initial and continuing proportional losses are then deducted, and the excess runoff is routed through the reach network.

The adopted methodology described below is based on current guidelines described in ARR (2019), which employs a Monte Carlo (MC) modelling approach. MC is a probabilistic approach whereby a large number of potential parameter combinations are randomly modelled to determine a range of design flow estimates, from which an appropriate design flow value is derived. In this instance 5,000 model runs were simulated for each storm duration, with varying initial losses and temporal patterns, to produce a probabilistic distribution of design flows. This allowed design peak flows for the range of design events to be derived from a flood frequency analysis of the simulated series of storm events.

There are no streamflow gauges within the local catchment to calibrate the RORB models, therefore parameter selection was based on regional parameter estimation formulas and design flows were validated to regional methods.

The design flows were then applied to the hydraulic model boundaries, and the resultant flows and water levels in the model compared to Landsat Imagery flood extents.

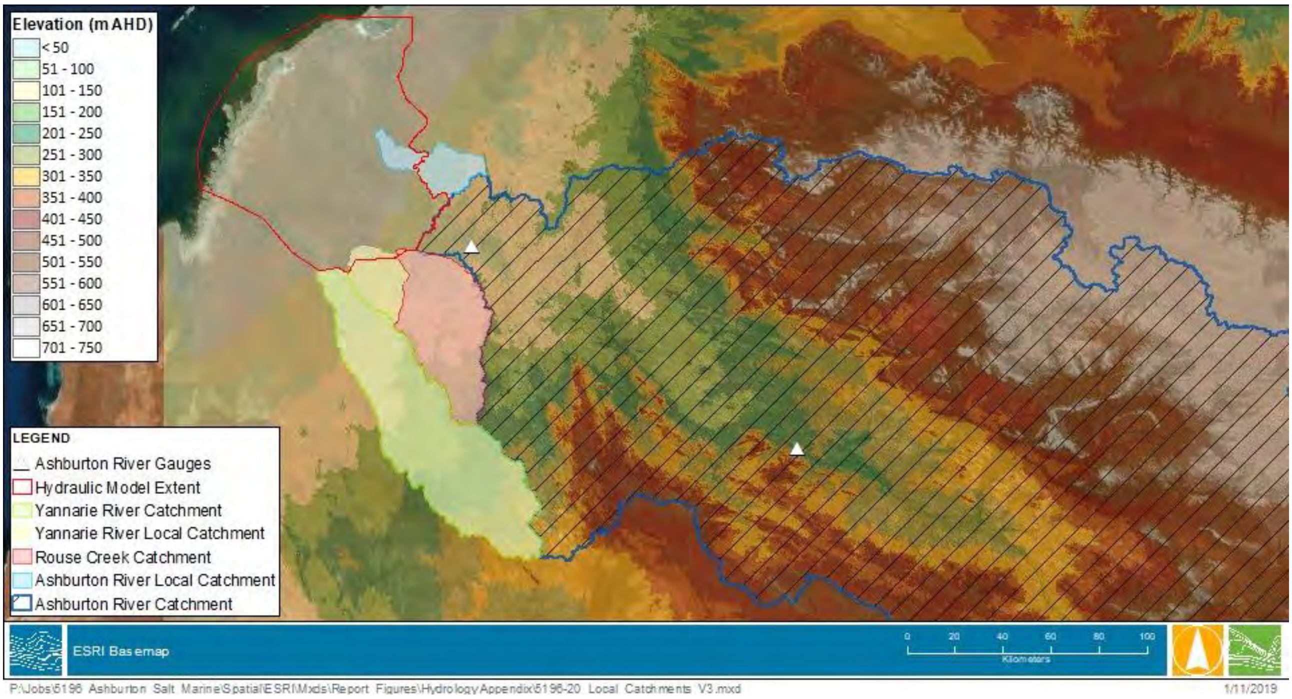

FIGURE 7-1 LOCAL CATCHMENT LOCATIONS

7.1.1 RORB Model Development

The catchments for each model were delineated using the Shuttle Radar Topography Mission (SRTM) datasets and are illustrated in Figure 7-2 to Figure 7-5. The catchment characteristics are sumamrised in Table 7-1.

TABLE 7-1 CATCHMENT CHARACTERISTICS

Catchment	Area (km^2)	Equal area slope (m/km)	Longest flow path (km)
A (Yannarie)	3,917	3.4	58.8
B (Yannarie Local)	528	3.3	45.4
C (Rouse)	1,870	2.3	94.7
D (Ashburton Local)	602	6.4	55.0

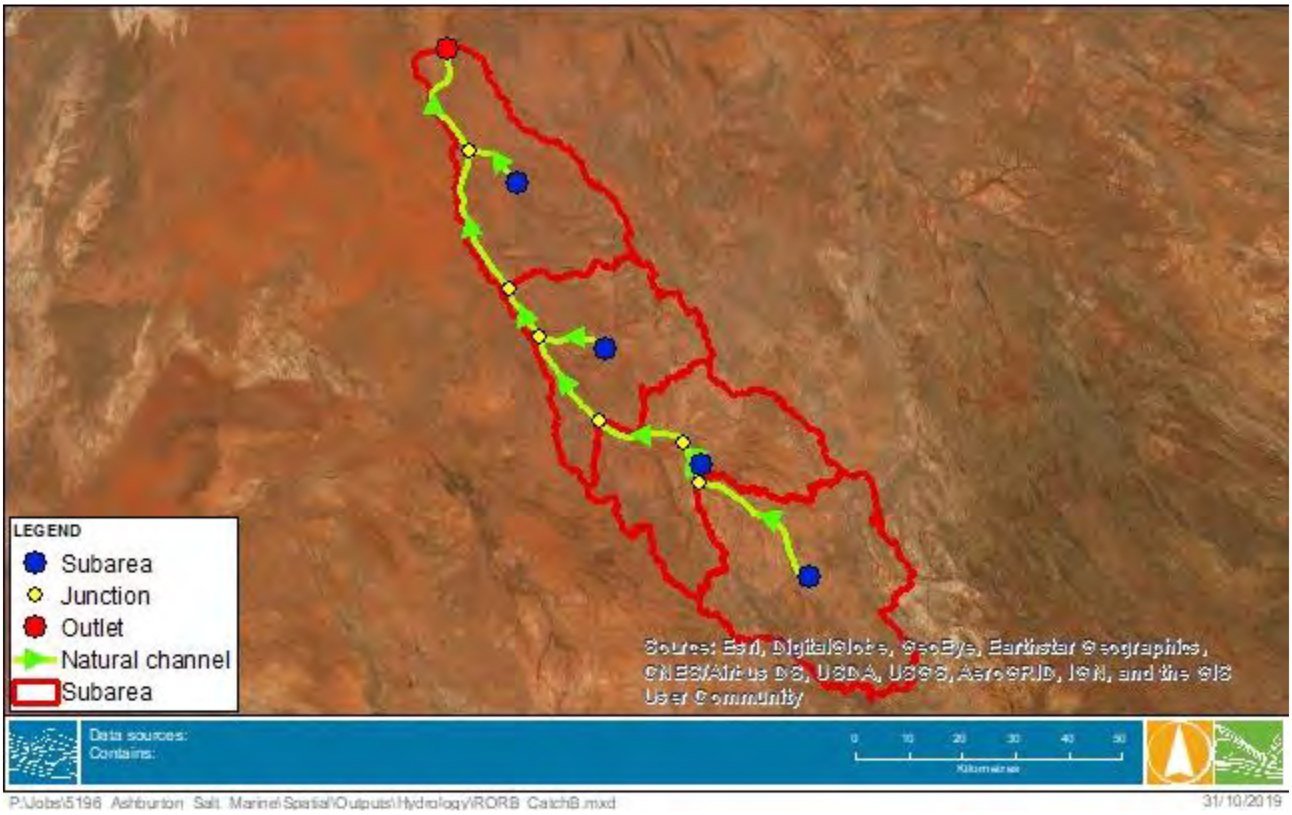

FIGURE 7-2 CATCHMENT A SUBAREA DELINEATION

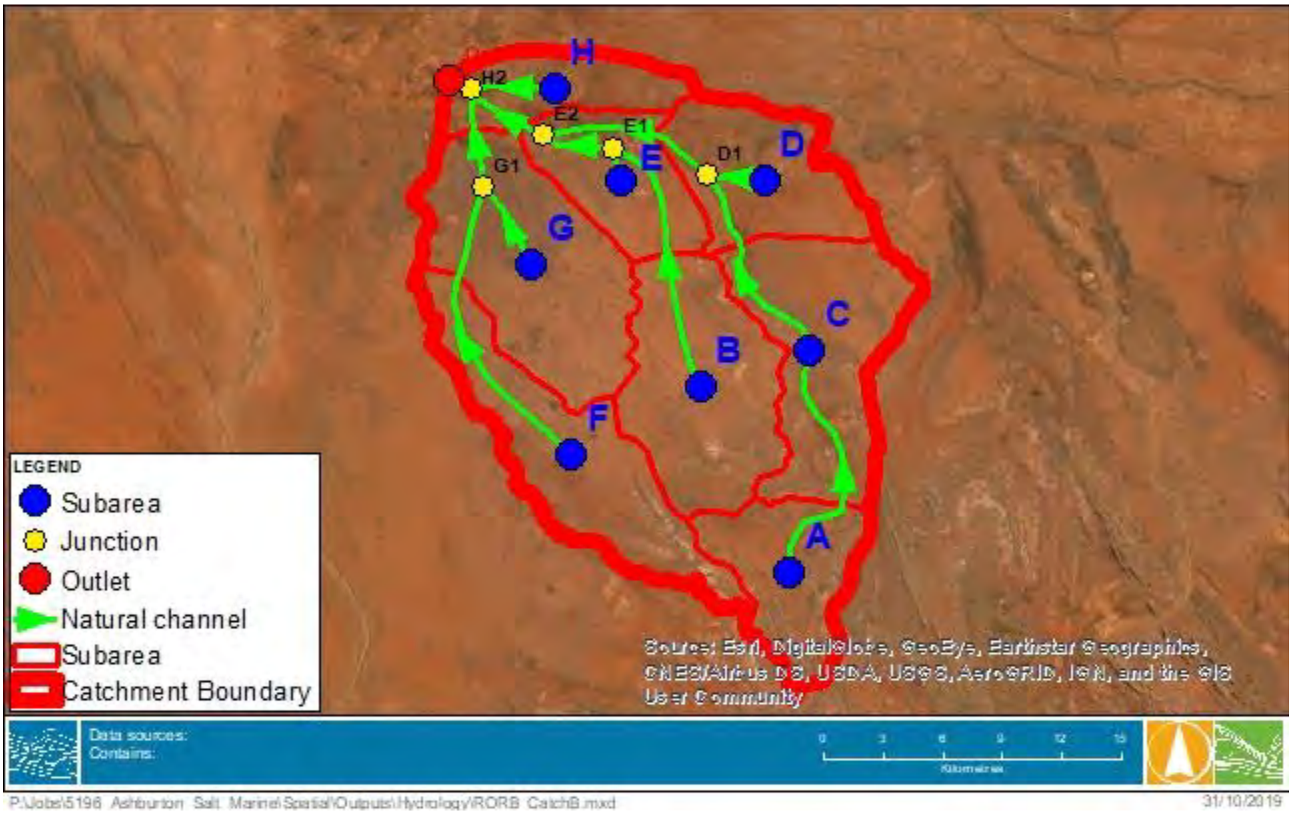

FIGURE 7-3 CATCHMENT B SUBAREA DELINEATION

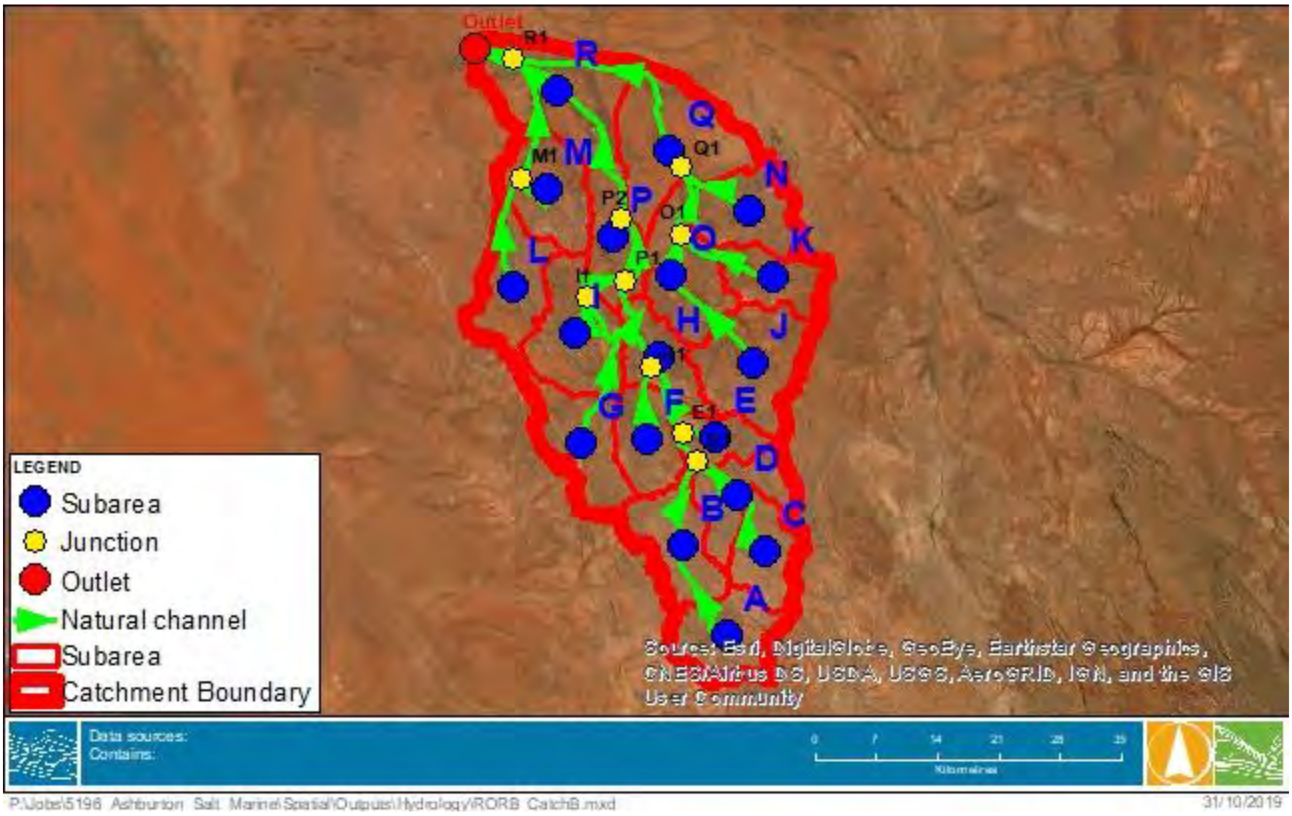

FIGURE 7-4 CATCHMENT C SUBAREA DELINEATION

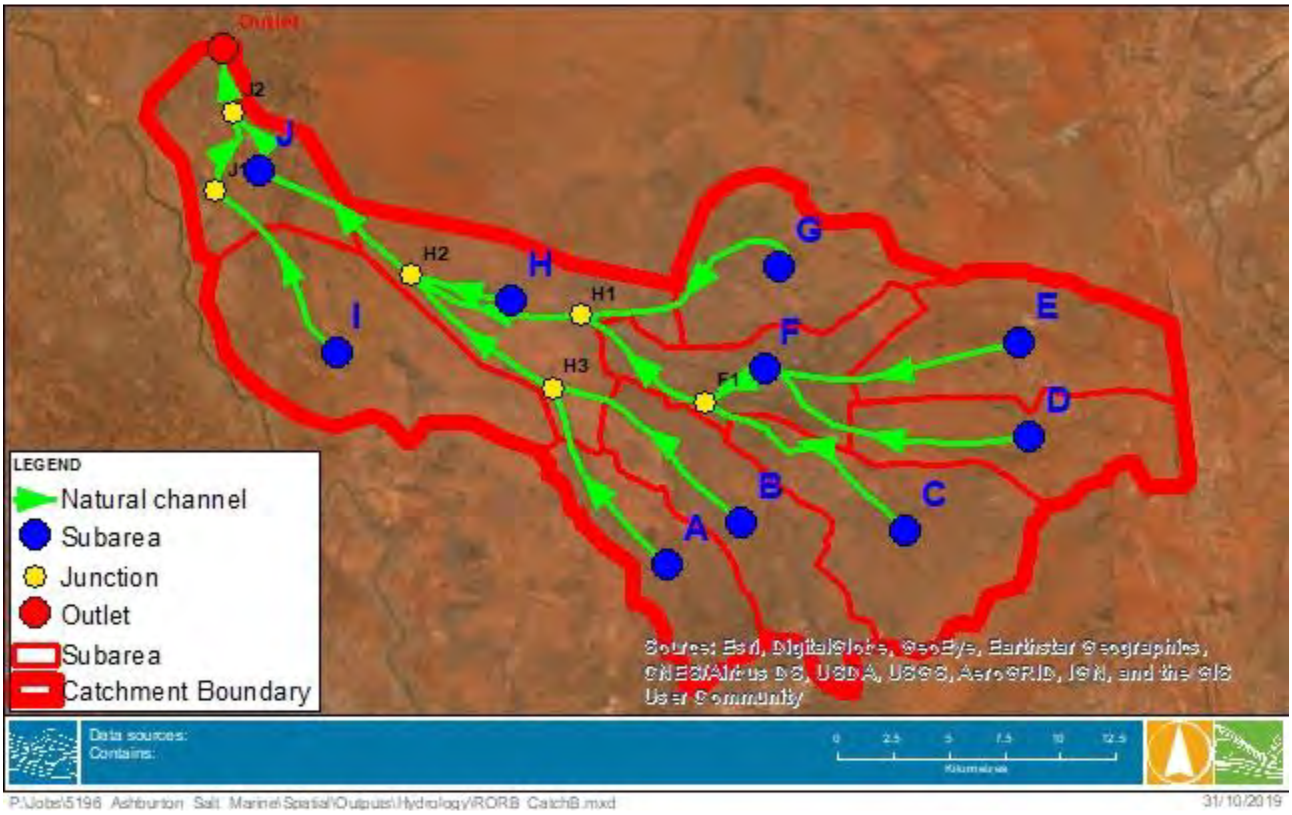

FIGURE 7-5 CATCHMENT D SUBAREA DELINEATION

7.1.2 RORB Model Parameters

In the absence of any calibration data, the Pearcey et al (2014) study summarising RORB routing parameters calibrated for rivers in the Pilbara region was used to estimate k_c values for the local catchments. The study found that the routing coefficient k_c could be estimated from the d_{av}, the average flow distance in the channel network from the centroid of each sub-area to the catchment outlet, by the following equation:

$K_c = C * d_{av}$

The mean value of C for calibrated Pilbara catchments was found to be 0.59, and values of ± one standard deviation were 0.71 and 0.48, respectively. A C value of 0.59 was adopted for all models and sensitivity analysis was conducted to assess the impact of C value to the flow estimates. This is discussed further in Section 7.3. K_c values adopted are shown in Table 7-2. These values were used for calibration and design events.

There is a lack of data in the study area to calibrate the RORB models. Flavell and Belstead (1986) recommends initial loss (IL) values between 40 to 50 mm and continuing loss (CL) of 5 mm/hr. A site visit to the Ashburton catchment was conducted. Soil infiltration tests were conducted at various locations downstream of the catchments. The tests found that soils were similar to the soils in the upstream catchments have a CL of approximately 5 mm/hr. For this study, an IL of 45 mm and a CL of 5mm/hr was adopted.

TABLE 7-2 ADOPTED RORB PARAMETERS

Catchment	K_c (RORB Routing Parameter)	Initial Loss (mm)	Continuing Loss (mm)
Catchment A	54.66	45	5
Catchment B	11.60	45	5
Catchment C	28.28	45	5
Catchment D	17.98	45	5

ARR (2019) recommends applying areal temporal patterns on catchments greater than 75 km^2. Thus, areal temporal patterns were adopted for all RORB models in this study. The study sites falls within the Rangeland West temporal pattern region.

7.2 Design Rainfall

Design rainfall depths for the centroid of each local catchment were extracted from BOM's intensity, frequency, duration (IFD) tool and are tabulated in Table 7-3, Table 7-4, Table 7-5 and Table 7-6.

TABLE 7-3 DESIGN RAINFALL DEPTH (MM) FOR CATCHMENT A

Duration (hr)	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
1	25.5	38.9	48.6	58.7	72.9	84.4
1.5	28.8	44.1	55.3	66.9	83.2	96.5
2	31.2	47.9	60.2	73.0	91.0	106
3	34.8	53.7	67.8	82.4	103	120
4.5	38.7	60.3	76.3	93.1	117	137
6	41.8	65.5	83.1	102	129	151
9	46.7	73.7	94.0	115	147	173
12	50.5	80.2	103	126	161	190
24	60.3	97.2	125	155	199	236
36	66.0	107	139	172	221	263
48	69.7	114	147	183	235	279
72	74.3	122	158	196	251	297
96	76.9	126	163	203	259	306

TABLE 7-4 DESIGN RAINFALL DEPTH (MM) FOR CATCHMENT B

Duration (hr)	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
1	28.8	43.5	53.8	64.3	78.6	90
1.5	32.5	49.5	61.5	73.7	90.4	104
2	35.2	54.0	67.3	80.9	99.6	115
3	39.1	60.7	76.3	92.1	114	132
4.5	43.2	68.2	86.3	105	132	153

Duration (hr)	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
6	46.4	74.0	94.3	116	146	170
9	51.3	83.0	107	132	168	198
12	55.0	89.9	116	145	185	219
24	64.8	108	141	177	229	272
36	70.6	118	155	195	252	299
48	74.5	125	164	206	265	314
72	79.6	133	174	218	278	327
96	82.6	137	179	224	283	331

TABLE 7-5 DESIGN RAINFALL DEPTH (MM) FOR CATCHMENT C

Duration (hr)	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
1	28.3	42.6	52.8	63.1	77.3	88.7
1.5	31.9	48.5	60.2	72.2	88.7	102
2	34.6	52.9	65.9	79.2	97.7	113
3	38.5	59.5	74.7	90.2	112	130
4.5	42.8	67	84.7	103	129	150
6	46.1	72.9	92.7	113	143	167
9	51.3	82.1	105	130	166	195
12	55.3	89.3	115	143	183	217
24	65.7	108	141	178	229	272
36	71.9	119	156	197	254	302
48	76.1	126	166	210	269	319
72	81.3	135	177	223	284	335
96	84.3	139	182	229	290	340

TABLE 7-6 DESIGN RAINFALL DEPTH (MM) FOR CATCHMENT D

Duration (hr)	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
1	30.5	45.4	55.8	66.1	80.2	91.3
1.5	34.6	51.9	64.0	76.0	92.5	106
2	37.5	56.8	70.3	83.8	102	117
3	41.8	64.3	80.2	96.3	119	137
4.5	46.5	72.7	91.6	111	138	160
6	50.1	79.3	101	123	154	179
9	55.7	89.8	115	142	180	211
12	59.9	97.8	126	157	200	235
24	70.9	118	155	194	249	294

Duration (hr)	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
36	77.3	130	170	214	274	324
48	81.4	136	179	224	287	339
72	86.3	144	188	234	297	351
96	88.9	148	191	237	300	354

7.3 Design Peak Flow Estimates

Peak flow estimates were produced for a range of AEPs and are presented in Table 7-7. For most of the catchments and AEPs, the critical duration is 24 hours. For the events where the critical duration is 12 hours, the peak flows between the 12 and 24 hour events is very close. Therefore, to simplify the input for the hydraulic model, the 24-hour storms were selected as the critical event for all catchments.

TABLE 7-7 PEAK FLOW ESTIMATES FROM RORB

AEP (%)	Peak Flow (m^3/s)			
	Catchment A	Catchment B	Catchment C	Catchment D
20	163.0	346.0	275.8	291.8
10	692.7	706.4	883.0	669.6
5	1491.1	1123.7	1655.0	1150.3
2	2912.2	1675.0	2962.4	1758.8
1	4165.8	2169.1	4008.8	2277.8

7.3.1 Sensitivity Analysis

A sensitivity analysis of the Kc value was conducted to investigate the impact of Kc on the peak flow estimates. The models were simulated with C value of 0.48 and 0.71 which are one standard deviation from the mean of 0.59 (Pearcy, Cheng, & Knoesen, 2014). Lower C values produce higher flow estimates and "peakier" flow hydrographs.

The percentage difference in peak flow for each AEP events is documented in Table 7-8. On average, peak flow increases by 18% when a C value of 0.48 is used whereas it declines by 15% when a C value of 0.71 is used. While the C value does affect the peak flow estimates, it does not cause a drastic difference.

TABLE 7-8 PERCENTAGE DIFFERENCE IN PEAK FLOW ESTIMATES BETWEEN C OF 0.59 WITH C OF 0.48 AND 0.71

AEP (%)	Catchment A		Catchment B		Catchment C		Catchment C	
	C = 0.48	C = 0.71	C = 0.48	C = 0.71	C = 0.48	C = 0.71	C = 0.48	C = 0.71
20	23%	-18%	8%	-18%	21%	-21%	21%	-17%
10	21%	-19%	7%	-16%	26%	-18%	25%	-15%
5	25%	-17%	5%	-11%	28%	-14%	16%	-12%
2	27%	-17%	6%	-9%	23%	-13%	14%	-13%
1	20%	-17%	6%	-9%	21%	-11%	9%	-13%

7.4 Regional Flood Frequency Estimation

Regional flood frequency estimation (RFFE) is a technique developed to provide peak flow estimates for ungauged catchments based on gauged data from nearby catchments. The accuracy of RFFE is largely dependent on the quantity and quality of data. Unfortunately, in the Pilbara there is generally a lack of recorded data. Nevertheless, the peak flow at each catchment outlet was derived using RFFE and the results are summarised in Table 7-9.

TABLE 7-9 RFFE PEAK FLOW ESTIMATES FOR LOCAL CATCHMENTS

AEP (%)	Catchment A	Catchment B	Catchment C	Catchment D
20	388	203	413	297
10	632	331	673	484
5	921	482	981	704
2	1340	700	1420	1020
1	1670	873	1780	1280

Comparisons between RFFE and RORB estimates (Figure 7-6 to Figure 7-9) indicate that RORB estimates are higher than RFFE for those events rarer than 10% AEP. However, estimates from RORB are still within the 95% confidence limit of RFFE. It is important to note that the estimates from RFFE are only used as a rough guide and the RORB models have not been calibrated to the RFFE estimates. This is because a high degree of uncertainty is associated with RFFE in the Pilbara region due to a lack of quality recorded data.

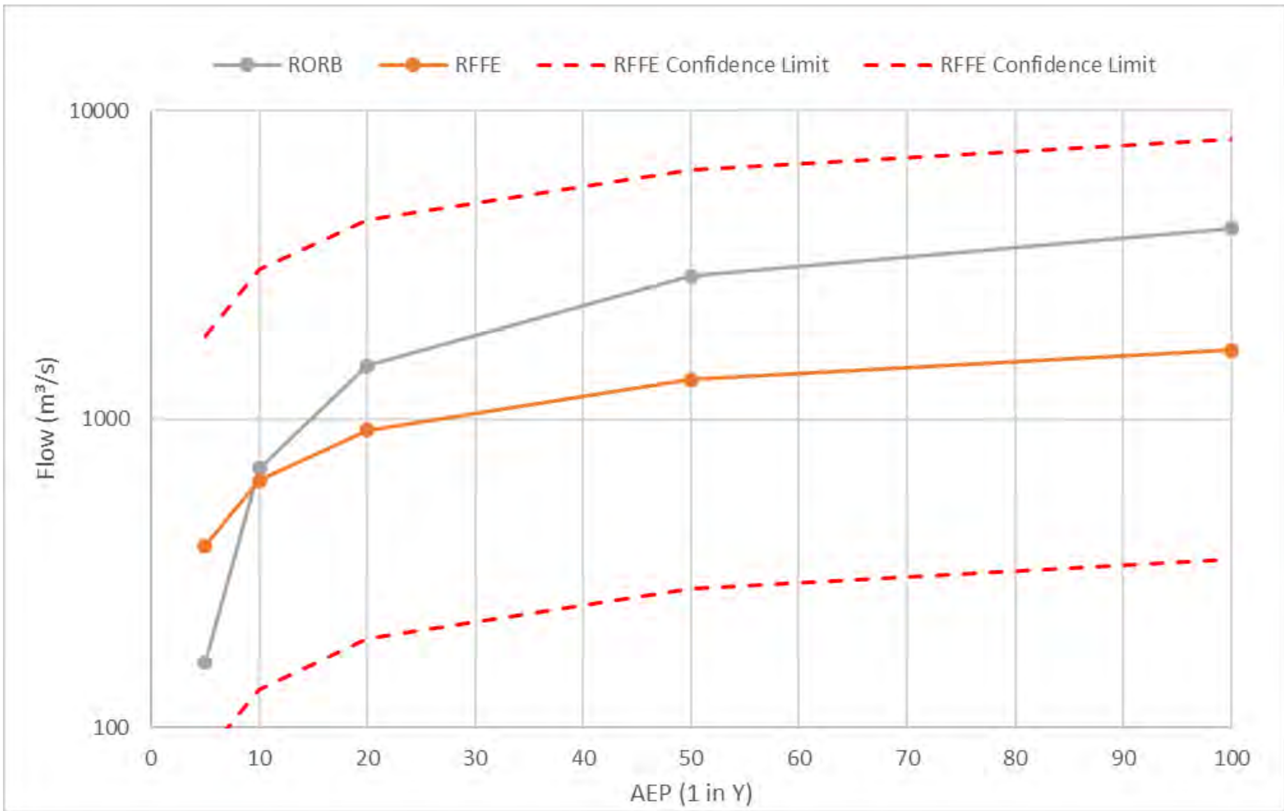

FIGURE 7-6 RORB PEAK FLOW COMPARED TO RFFE IN CATCHMENT A

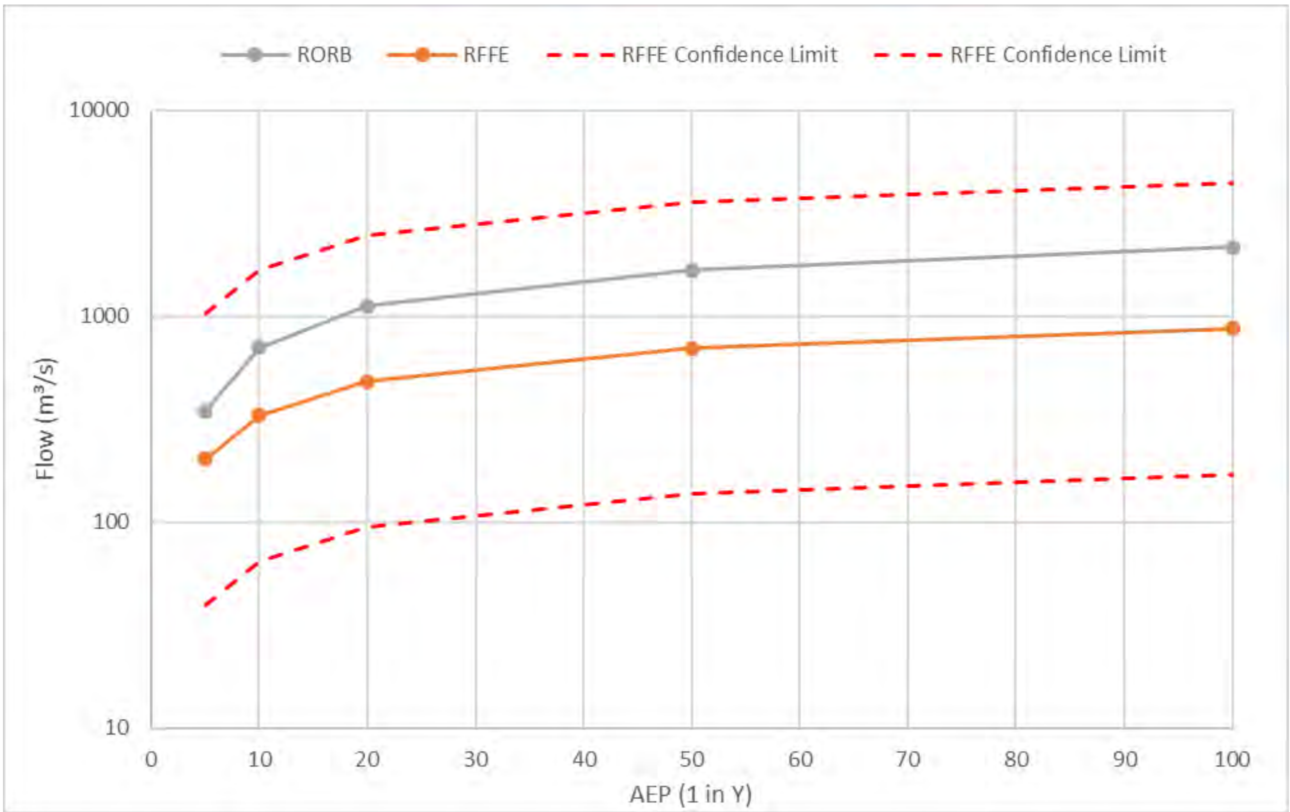

FIGURE 7-7 RORB PEAK FLOW COMPARED TO RFFE IN CATCHMENT B

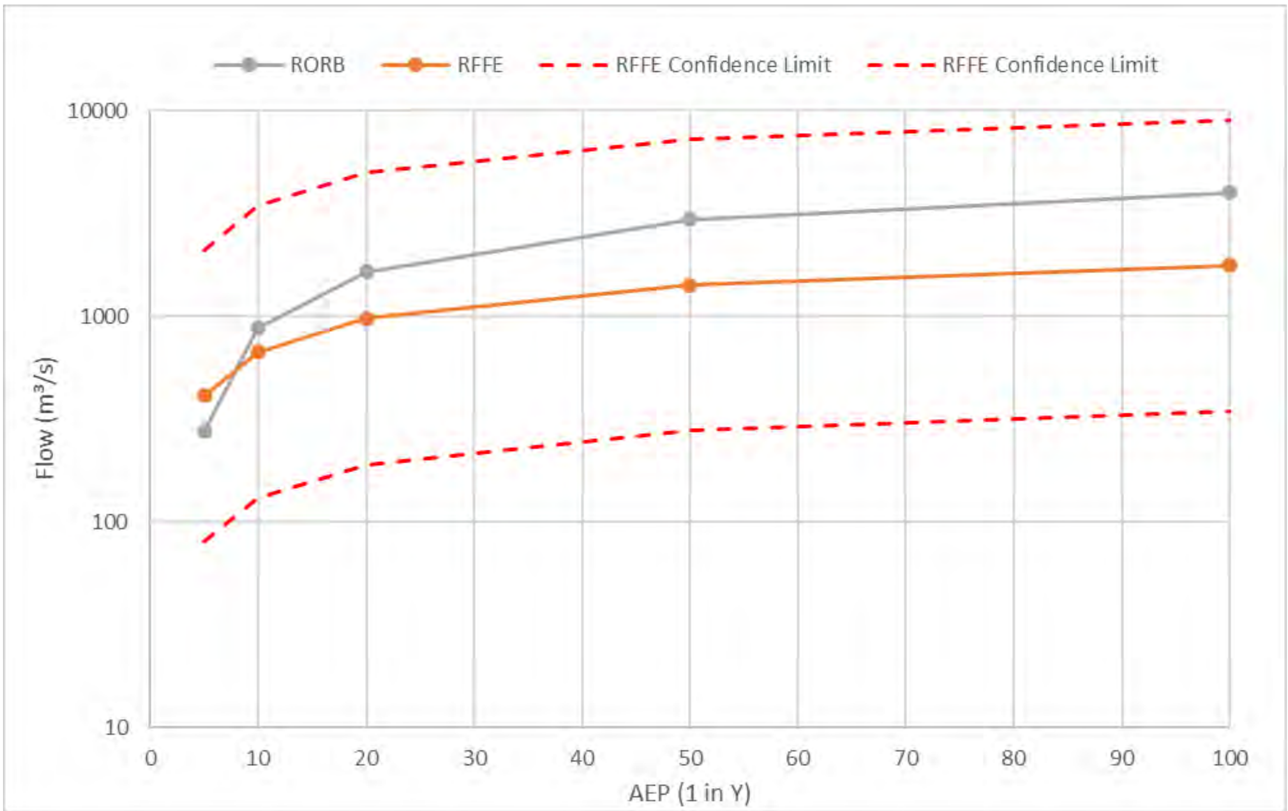

FIGURE 7-8 RORB PEAK FLOW COMPARED TO RFFE IN CATCHMENT C

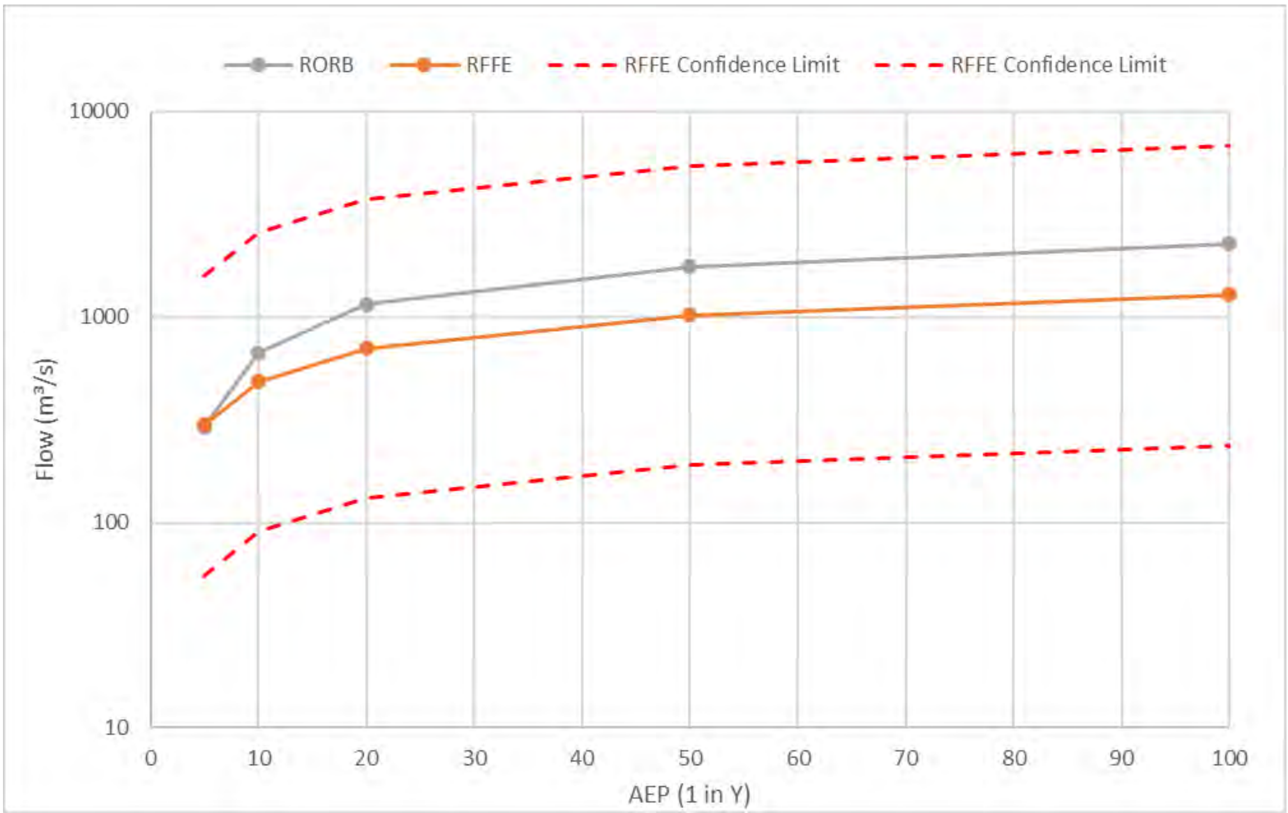

FIGURE 7-9 RORB PEAK FLOW COMPARED TO RFFE IN CATCHMENT D

7.5 Hydraulic Model Inflow Hydrographs

Inflow hydrographs inserted into the upstream boundary of the MIKE21FM model for the 10%, 5%, 2% and 1% AEP are shown in Figure 7-10, Figure 7-11, Figure 7-12 and Figure 7-13 respectively.

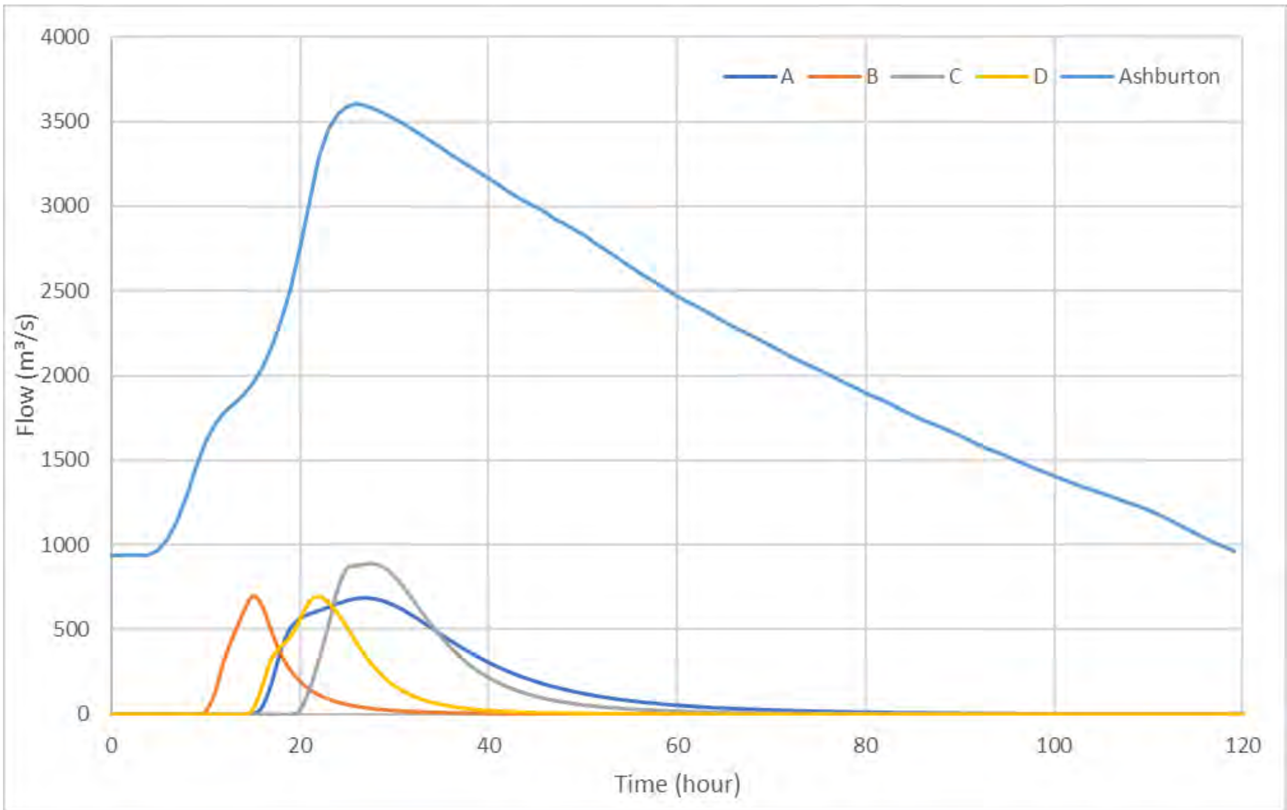

FIGURE 7-10 INFLOW HYDROGRAPHS FOR THE 10% AEP EVENT

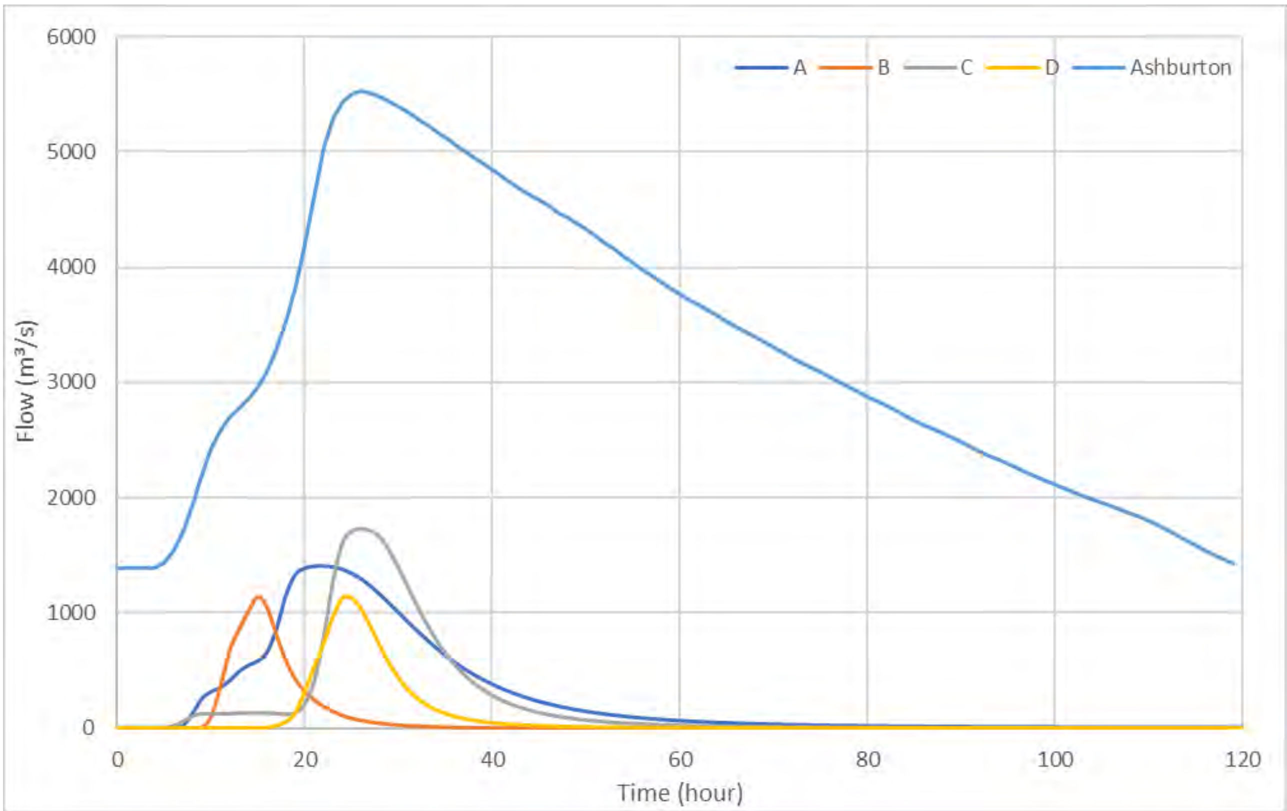

FIGURE 7-11 INFLOW HYDROGRAPHS FOR THE 5% AEP EVENT

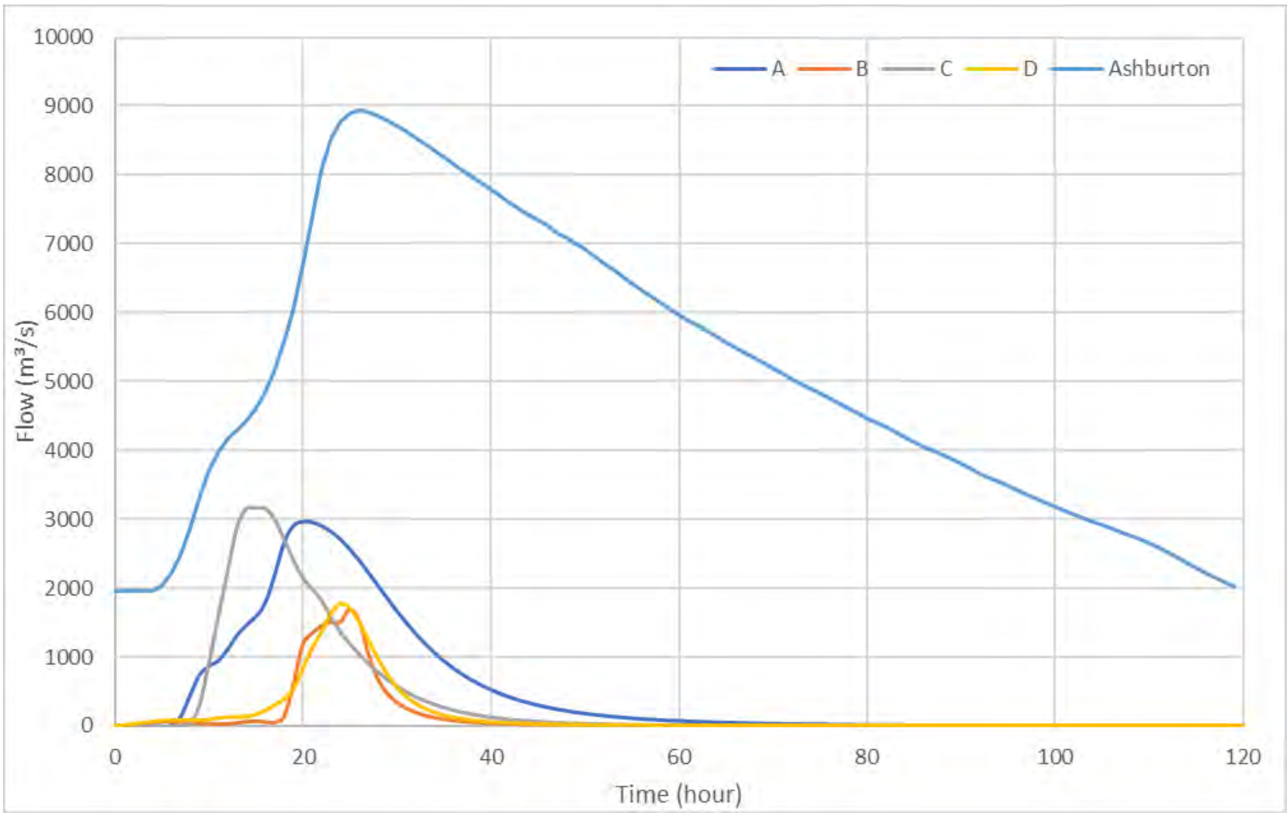

FIGURE 7-12 INFLOW HYDROGRAPHS FOR THE 2% AEP EVENT

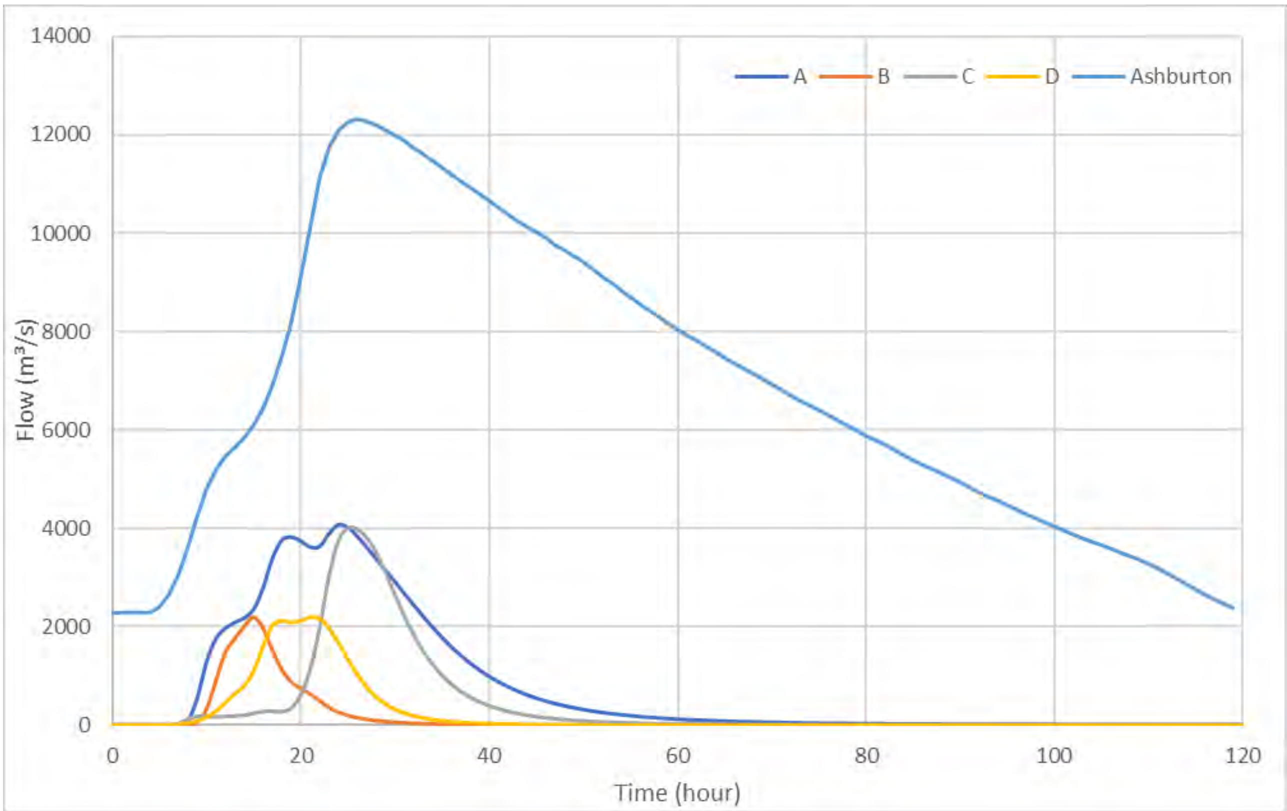

FIGURE 7-13 INFLOW HYDROGRAPHS FOR THE 1% AEP EVENT

8 HYDRAULIC MODEL SETUP

8.1 Hydraulic Model Schematisation

The modelling applied for this project utilised a Rain-on-Grid (RoG) approach. Two hydraulic modelling software packages were used to simulate the catchment to ensure accuracy at the site and the areas immediately surrounding the site infrastructure. Regional catchments were modelled using MIKE21FM, a two dimensional (2D) flexible mesh hydraulic model, whereas local catchments at the site were modelled using a 2D HPC TUFLOW hydraulic model. Both models are described below:

- **Regional Model** – utilised MIKE21FM GPU software (Mike by DHI) which is a 2D hydraulic model. Adopting a flexible mesh modelling approach allowed the hydraulic model to incorporate areas of importance in greater detail, yet larger element mesh sizes in less sensitive regions of the modelled area. The model boundary is shown in Figure 8-1.
- **Local Model** – utilised the 2D HPC TUFLOW hydraulic model which is a 2D hydraulic model. The modelling approach applied for this project utilised “rain on grid (ROG)” in which a design storm event is applied to a topographical Digital Elevation Model (DEM) and hydrodynamic equations are used to route flow through the system. The model boundary is shown in Figure 8-4.

8.2 Regional MIKE Model Setup

8.2.1 MIKE Model Domain and Topography

The model domain of this study extends from the coast adjacent to the proposed development site and its surroundings 80 km inland, capturing the entire lower Ashburton River, Yannarie River and Rouse Creek floodplains (Figure 8-1).

A 10 m grid digital elevation model was developed for the model from several data sources, essentially sourced from:

- Newly captured LiDAR, flown by FUGRO in May 2017;
- WorldDEM satellite DTM from Airbus Defence and Space; and
- SRTM satellite DTM from Geoscience Australia.

From the DEM, a flexible mesh was developed to represent the floodplain. The mesh comprises triangular elements of varying size. The mesh extent and bathymetry used for the regional hydraulic model is illustrated in Figure 8-1.

The mesh resolution in the Ashburton River is approximately 255 m^2, this approximate resolution was also used in the development site. The resolution of the mesh reduces towards the floodplain and offshore, with a median element size throughout the model of 2,000 m^2 and a maximum element size of 100,000m^2. Spatially varying the resolution allows model run times to be optimised whilst maintaining element sizes needed to adequately map the study area. The MIKE-21 model was originally run to encompass the entire development site to assess potential impacts of the development. However, it was determined that TUFLOW performed better at higher resolutions at the development site, so the MIKE-21 model was subsequently used for pre-development purposes at a regional scale (see below for greater detail). Both models were developed to simulate flood inundation for the 1%, 2%, 5% and 10% per cent Annual Exceedance Probability (AEP) flood events for a range of scenarios.

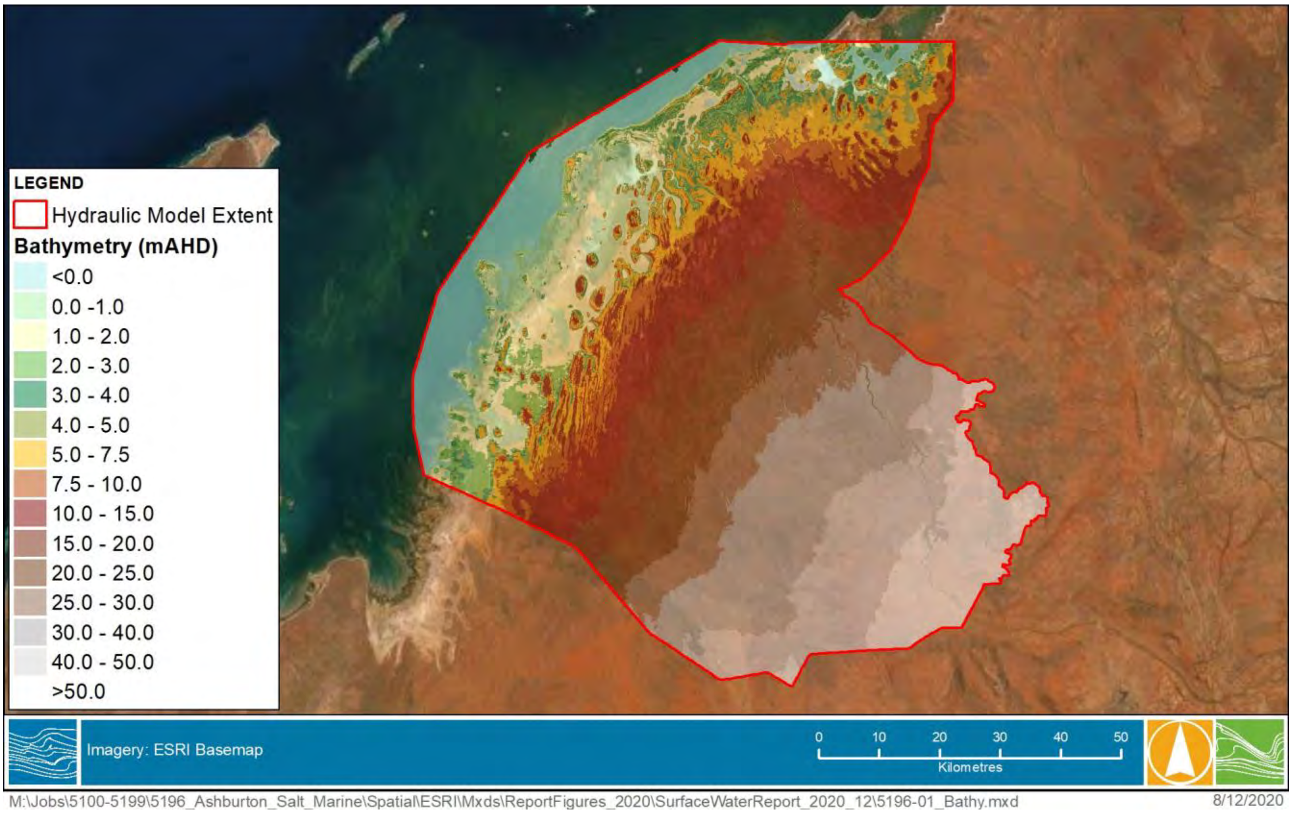

FIGURE 8-1 MIKE-21 MODEL BOUNDARY AND TOPOGRAPHY

8.2.2 MIKE Model Boundary Conditions

8.2.2.1 Inflow Boundaries

Ashburton River, Yannarie River and Rouse Creek inflows, along with 3 other local catchment inflows, were applied at the upstream end of the hydraulic model. Of these catchments, only the Ashburton River is gauged, and further details on the calibration process for the upstream flows are provided in Section 6. The Ashburton River boundary is applied directly downstream of the Nanutarra gauge.

8.2.2.2 Direct Rainfall Boundary

Due to the extensive size of the regional hydraulic model domain, internal sub-catchments of the hydraulic model can generate significant runoff. Therefore, the entire model has rainfall applied to it during flood events to account for this effect. This rainfall is described in Section 7.2 and was based on measured rainfall for the calibration event and IFD parameters for design events.

8.2.2.3 Ocean Tidal Boundary Conditions

For the calibration event, the predicted tidal pattern was generated using tidal constituents at the proposed jetty location (*Marine and Coastal Assessment and Modelling*, Water Technology 2021) and applied as an ocean boundary. For design event scenarios, a series of design return period storm boundary conditions were developed using the marine model (*Marine and Coastal Assessment and Modelling*, Water Technology 2021) and applied to the ocean boundary.

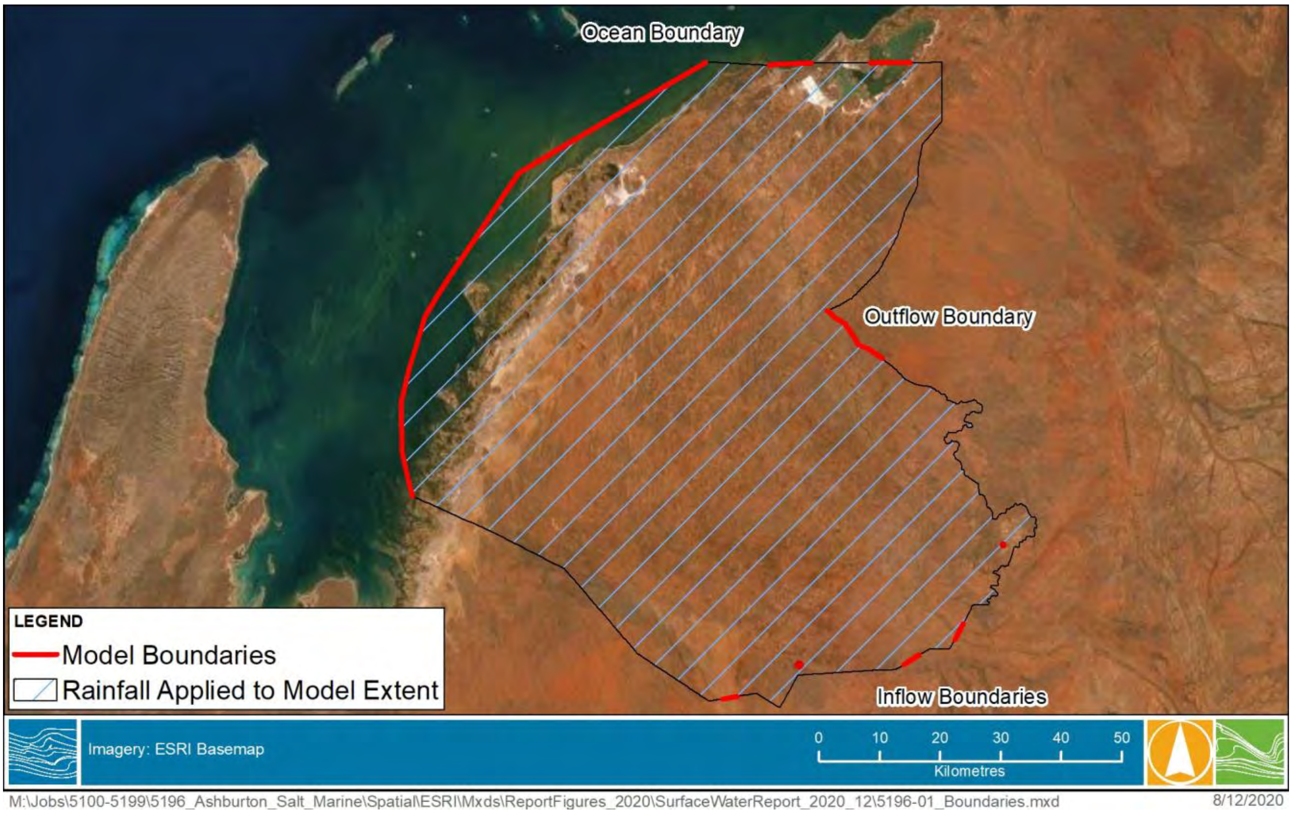

FIGURE 8-2 MIKE-21 MODEL BOUNDARIES

8.2.3 MIKE Model Roughness

Hydraulic roughness within the model was expressed as Manning's 'n' values. To estimate floodplain Manning's 'n' values, land use and vegetation cover from both aerial imagery and available mapping was assessed. Hydraulic roughness parameters were adopted using industry standard values that represent the channel and floodplain cover.

Hydraulic roughness values adopted are summarised in Table 8-1 and Figure 8-3.

TABLE 8-1 MIKE-21 MODEL ROUGHNESS VALUES

Land Use/Topographic Description	Manning's 'n'
Offshore	0.03
Sandy/Beach Areas	0.05
Salt Flats	0.05
Algal Mats	0.06
Light Vegetation	0.06
Heavy Vegetation	0.09
Mangrove	0.12

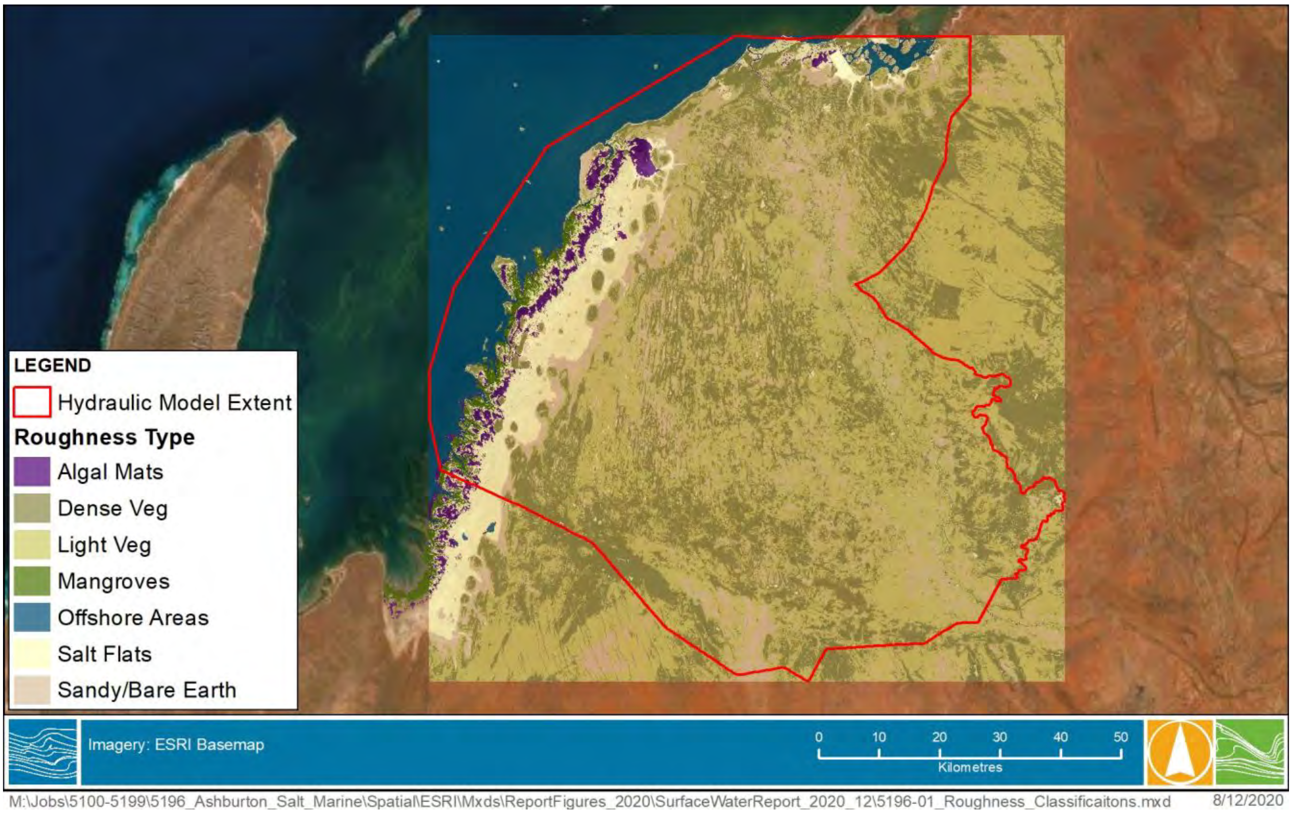

FIGURE 8-3 MIKE-21 MODEL ROUGHNESS

8.2.4 MIKE Model Rainfall

The design rainfall depths presented and discussed in Section 8.3.4 were applied to the model boundary for the 10%, 5%, 2% and 1% AEP storms events and a variety of rainfall intensities. See Section 8.3.4 for further detail.

8.2.5 MIKE Model Losses

Infiltration

Infiltration rates adopted within the model were based on soil landscape mapping covering Western Australia, Soil Landscape Mapping - Soil Sites (DPIRD-071) - Web Mapping Service - data.wa.gov.au, based on Tille (2006) as well as a range of soil and geotechnical data capture for the Yannarie project (summarised in Blandford and Associates, 2005).

Sensitivity testing was undertaken on the infiltration rates applied in the model and it was found that increasing the rates by a factor of 10 had negligible impact on modelled flood extents or depths.

Evaporation

Evaporation data observed at Learmonth Airport were used to define evaporation within the model. Evaporation is relatively consistent from year to year with notable seasonal variability. For model calibration, historical evaporation data was used to model Tropical Cyclone Vance. Evaporation rates range from 0-20 mm/day and therefore an average constant evaporation rate of 10mm/day was adopted within the model for design events.

TABLE 8-2 MIKE MODEL SOIL MOISTURE CAPACITY AND INFILTRATION RATES FOR DIFFERENT SOILS TYPES

Soil Type – from on Soil Landscape Mapping (Geoscience Australia)	Soil Capacity	Infiltration rate (mm/hr)
Active flood plains supporting coolabah woodlands and numerous tussock grasses.	10	1
Alluvial clay plains with gilgais, mixed open tussock grasslands and acacia tall shrublands.	0	0
Alluvial plains supporting acacia tall shrublands and tussock grasslands and sandy plains supporting hummock grasslands.	10	1
Alluvial plains supporting snakewood shrublands and minor tussock grasslands.	10	1
Alluvial plains supporting tall acacia shrublands and low eucalypt woodlands with prominent tussock grasses including buffel grass.	10	1
Alluvial plains with sandy and duplex soils supporting snakewood and other acacia shrublands often with buffel grass understorey.	10	1
Bare coastal mudflats (unvegetated), samphire flats, sandy islands, coastal dunes and beaches, supporting samphire low shrublands, sparse acacia shrublands and mangrove forests.	0	0
Broad sandy plains, pebbly plains and drainage tracts supporting hard and soft spinifex hummock grasslands with scattered acacia shrubs.	10	1
Dune fields supporting soft spinifex and minor hard spinifex grasslands.	5	1
Gently undulating stony plains supporting hard and soft spinifex grasslands and snakewood shrublands.	0	0
Granite hills, domes, tor fields and sandy plains supporting spinifex grasslands with scattered shrubs.	0	0
Low mesas and hills of sedimentary rocks supporting soft and hard spinifex shrubby grasslands.	0	0
Low plateaux, mesas and buttes of limonite supporting soft spinifex and occasionally hard spinifex grasslands.	0	0
Plains with dunes and numerous claypans, supporting soft spinifex and snakewood shrublands.	0	0
Rugged sandstone hills, ridges, stony footslopes and interfluves supporting low acacia shrublands or hard spinifex grasslands with scattered shrubs.	0	0
Sandy plains with linear dunes and broad sandy swales supporting hummock grasslands of hard and soft spinifex with scattered acacia shrubs.	0	0
Undulating sandplains, dunes and level clay plains supporting soft spinifex grasslands and minor tussock grasslands.	0	0

8.3 Local TUFLOW Modelling Setup

The local TUFLOW model was developed to simulate hydrological impacts of the development and to develop impact mitigation strategies for the site.

8.3.1 TUFLOW Model Domain and Topography

The model domain of this study covers the proposed development area and its upper catchments (see Figure 8-4). The model extent includes the upper catchments of the proposed development area to ensure that all runoff reporting to the evaporation ponds is appropriately accounted for.

As discussed in Section 8.2.1, a DEM was generated from three datasets to ensure rain-on-grid (ROG) modelling of the regional catchment was possible. In this instance, merging two datasets was required to capture the local catchments. The two merged datasets were the newly captured 5 m LIDAR flown by FUGRO (May 2017) and a 10 m WorldDEM satellite Digital Terrain Model (DTM) from Airbus Defence and Space. The datasets were merged to enable modelling of the access road which travels from the site north towards the Ashburton River and the bridge crossing of the Ashburton River. It should be noted that using data with varying grid resolutions can result in 'banding' across the terrain, which has the potential to alter flow patterns across the area of interest. To alleviate this effect, the area where the two datasets intersect was interpolated to ensure a smooth transition. It is important to note that in this instance the banding is situated 13 km to the north of the development envelope, and hence has not had a significant impact on flood levels and water movement within the project area.

The DEM discussed above formed the basis for development of the hydraulic model for both existing and proposed condition assessments. In the post development scenario, the proposed infrastructure footprint was represented in the DEM by embedding the embankments and crystalliser layout into the DEM. The remaining infrastructure areas (i.e., access road, conveyor, stockyard, buildings) were built into the model by artificially raising ground elevations to above the predicted 1% AEP water elevation. This is to simulate flood waters backing up behind the proposed infrastructure which will take place once the infrastructure in constructed.

8.3.2 TUFLOW Model Boundary Conditions

Three types of boundaries were applied to the TUFLOW model:

- Tailwater boundary using tidal levels for all design return periods (10% - 1%). A tailwater boundary is used to simulate the tidal level within the Exmouth Gulf (see Figure 8-4);
- Inflow hydrographs for all design events (10% - 1%) extracted from the calibrated regional MIKE model and input to the hydraulic model at eight locations on the upstream boundary. See Figure 8-4 for inflow hydrographs locations; and
- ROG boundary was used for all design events (10% - 1%). ARR2016 design rainfall depths extracted from the ARR data hub were applied to the model domain area represented in blue on Figure 8-4.

8.3.3 TUFLOW Roughness

Hydraulic roughness to represent different surface conditions across the model is defined using a Manning's 'n' value. In regional catchments, this is primarily driven by vegetation and degree of rock cover. In this instance, vegetation and rock cover is sparse across most of the catchment, with greater concentrations of low-lying vegetation occurring along the banks of the creeks and river channels. Based on previous experience in the region and the uniformity of the terrain, a Manning's 'n' of 0.04 was applied across the entire development area.

8.3.4 TUFLOW Rainfall

Intensity Frequency Durations (IFDs) are design rainfall intensities or design rainfall depths corresponding to selected probabilities. The IFD dataset was developed by the Bureau of Meteorology (BoM) using statistical analysis of historical rainfall. Current ARR2016 procedures included an update to the IFD's, with IFD information therefore representing the most updated dataset for application to design flood estimation. Aerial reduction factors were used to convert point rainfall to areal estimates and to account for the variation of rainfall intensities over a large catchment. ARR2016 areal reduction factors were extracted from the ARR data hub and applied to the catchment area. Design rainfall depths adopted in this study are summarised in Table 8-3.

As well as design rainfall depth, temporal patterns for each design event are required to define the pattern in which rainfall is distributed over storm durations. The full range of temporal patterns for the Pilbara region was adopted for this modelling exercise.

TABLE 8-3 DESIGN RAINFALL DEPTHS AND DURATIONS APPLIED TO TUFLOW MODEL DOMAIN

Duration (hours)	Rainfall Depth (mm)			
	10% AEP	5% AEP	2% AEP	1% AEP
3	61.4	72.4	82.2	98.4
6	89.6	109.0	135.0	155.0
9	111.2	137.0	171.8	200.4
12	127.9	158.4	199.7	233.8
18	153.7	191.3	243.6	285.9
24	173.7	216.8	277.1	326.2

8.3.5 TUFLOW Losses

Rainfall loss is defined as precipitation that does not translate to runoff and is dependent on the type of surface the rain lands on. The initial loss (IL)/continuing loss (CL) approach was adopted for this study whereby rainfall losses are subtracted from the model according to loss parameters. The model applies an IL at the beginning of a design rainfall event and a CL after the IL has been satisfied. The ARR2016 data hub does not provide an estimation of IL and CL at the study area due to a lack of recorded data in the Pilbara region. In the absence of any gauged data at the site, losses applied to the local hydraulic model were derived from the regional MIKE model (see Section 8.2.4). An initial loss of 1 mm and a continuing loss of 0.41 mm/hr were adopted for this study. The CL value incorporated evaporation which was based on an average evaporation rate of 10 mm/day; see Section 8.2.4 for greater detail.

8.3.6 Critical Duration

All events (10% - 1%) were simulated for the 3, 6, 9, 12, 24, 36- and 72-hours events to determine the critical duration for both regional and local (site) catchments. Peak flood levels from all durations were compared and critical durations determined; see Table 8-4 for respective durations.

TABLE 8-4 MIKE AND TUFLOW DESIGN EVENT CRITICAL DURATIONS

10%	5%	2%	1%
24 hr	24 hr	24 hr	24 hr

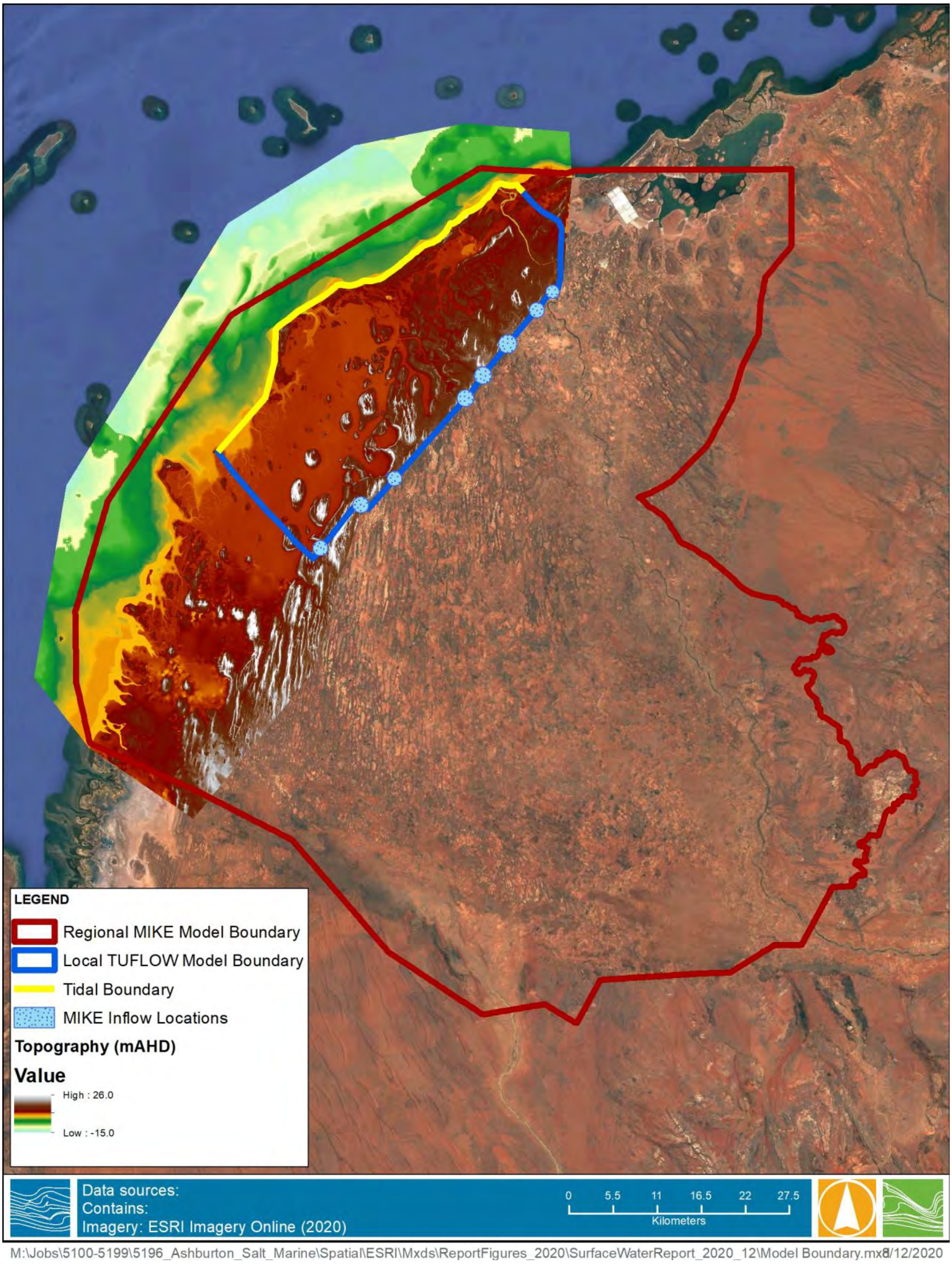

FIGURE 8-4 TUFLOW LOCAL HYDRAULIC MODEL SETUP

8.3.7 Model Assumptions and Limitations

Various assumptions were made in developing the TUFLOW model as follows:

- The height of the proposed infrastructure was not available at the time of this study due to its preliminary nature. In the absence of this information, the development height was represented in the model by raising the proposed footprint to an arbitrary level above the predicted 1% AEP flood level. Results from this modelling can be used to set suitable infrastructure heights; and
- The design of the bridge crossing was not available at the time of this study. In the absence of this information, the bridge was assumed to have 2 piers spanning 6.6 m in width and be 6 m high. The hydrodynamics at the base of the piers will need to be considered during future design stages to ensure scour impacts are considered.

Key adopted model parameters are summarised in Table 8-5 below.

TABLE 8-5 KEY TUFLOW MODEL PARAMETERS

Model Parameter	Description
Terrain data	■ LIDAR data merged with WorldDEM satellite DTM
Model type	TUFLOW HPC GPU
Model build	2020-01-AB-iSP-w64
Inflow source	■ Rain on grid ■ Inflow hydrographs (10% - 1% AEP events)
Outflow boundary	■ Tidal tailwater
Grid size	10 m
Roughness Mannings ‘n’	0.04
Losses	■ Initial Loss – 1 mm ■ Continuing Loss – 0.41 mm/hr
Model timestep	Adaptive timestep
Start time	0 hours
End time	130 hours

9 REGIONAL MODEL SCENARIOS

Table 9-1 summarises the hydraulic modelling scenarios undertaken for this project.

TABLE 9-1 MODELLED SCENARIOS

Scenario	Event Type	Flood Event (AEP)	Ocean Boundary Level (Average Recurrence Interval (ARI))
1	Calibration	1999 Cyclone Vance	Tidal (no measured ocean levels available)
2	Design	10% AEP	1 in 10 Year ARI
3	Design	5% AEP	1 in 20 Year ARI
4	Design	2% AEP	1 in 50 Year ARI
5	Design	1% AEP	1 in 50 Year ARI
6	Design	1% AEP	1 in 100 Year ARI
7	Design	1% AEP	1 in 500 Year ARI
8	Design	1% AEP	1 in 500 Year ARI plus mean sea level rise to 2070 (0.4 m)

9.1 Calibration Event (Scenario 1)

The most intense tropical cyclone ever recorded to cross the Australian coast, Tropical Cyclone Vance, passed over Exmouth Gulf in March 1999. TC Vance was a Category 5 cyclone with the highest ever recorded wind gust on the Australian mainland (267 km/hr) at Learmonth Airport on 22 March 1999 (Blandford & Associates, 2005). Cyclone Vance produced a storm surge at Exmouth of 3.6m and caused severe costal erosion. It also resulted in extreme rainfall events with rainfall totals of between 100 and 200 mm occurring with the Ashburton River catchment.

This event was selected for calibration of the catchment hydraulic model as it was the only event where satellite imagery was available illustrating the extent to which overland flooding occurred. Figure 9-1 shows a comparison between modelled and observed flood extents for Cyclone Vance. The modelled results compare favourably with satellite imagery across the catchment.

The lower lying and storm surge susceptible areas are not accurately resolved due to the lack of suitable model boundary conditions from water level records; however, this result confirms that coastal inundation for cyclonic events is limited to the tidal and supratidal areas.

The flood extent across the southern areas is also not clearly shown in the modelled results as the inflows from the Yannarie and Rouse River catchments are based on outputs from the hydrological model and the timing of these is unknown. Figure 9-2 shows the maximum inundation extents across the model for the entire event and the inflows from these southern catchments can clearly be seen.

A summary of flood behaviour during the event is provided in Figure 9-3 and Table 9-2. A series of breakouts from the Ashburton River occur at locations 2 and 3, and these breakouts are conveyed via overland flow towards the project area. Very little interaction is received from the Yannarie River, which is located approximately 50 km to the south east of the proposed project.

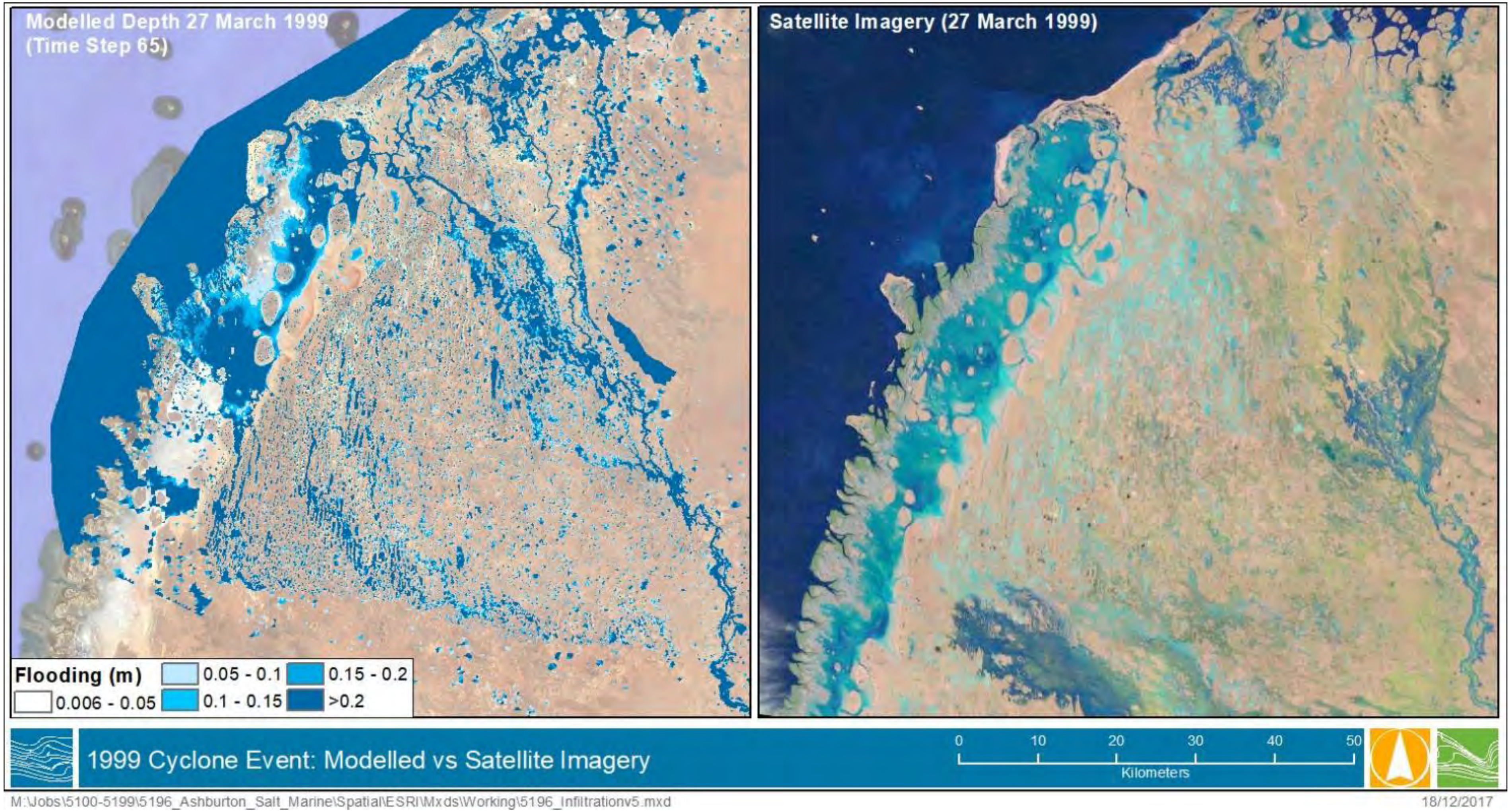

FIGURE 9-1 MODELLED CYCLONE VANCE INUNDATION EXTENT VS OBSERVED INUNDATION EXTENT CAPTURED VIA SATELLITE IMAGERY

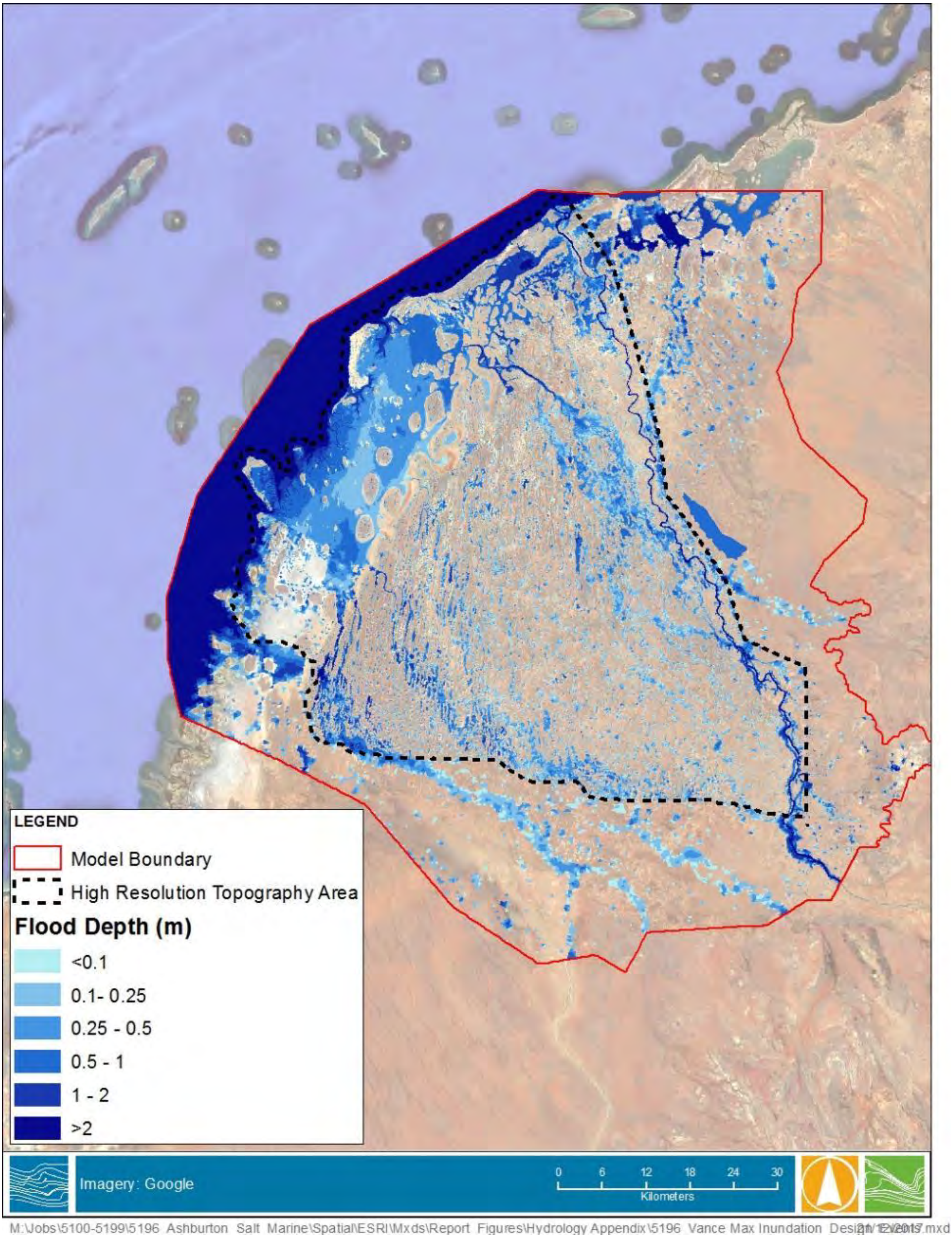

FIGURE 9-2 MAXIMUM MODELLED INUNDATION EXTENTS ACROSS THE STUDY AREA, CYCLONE VANCE

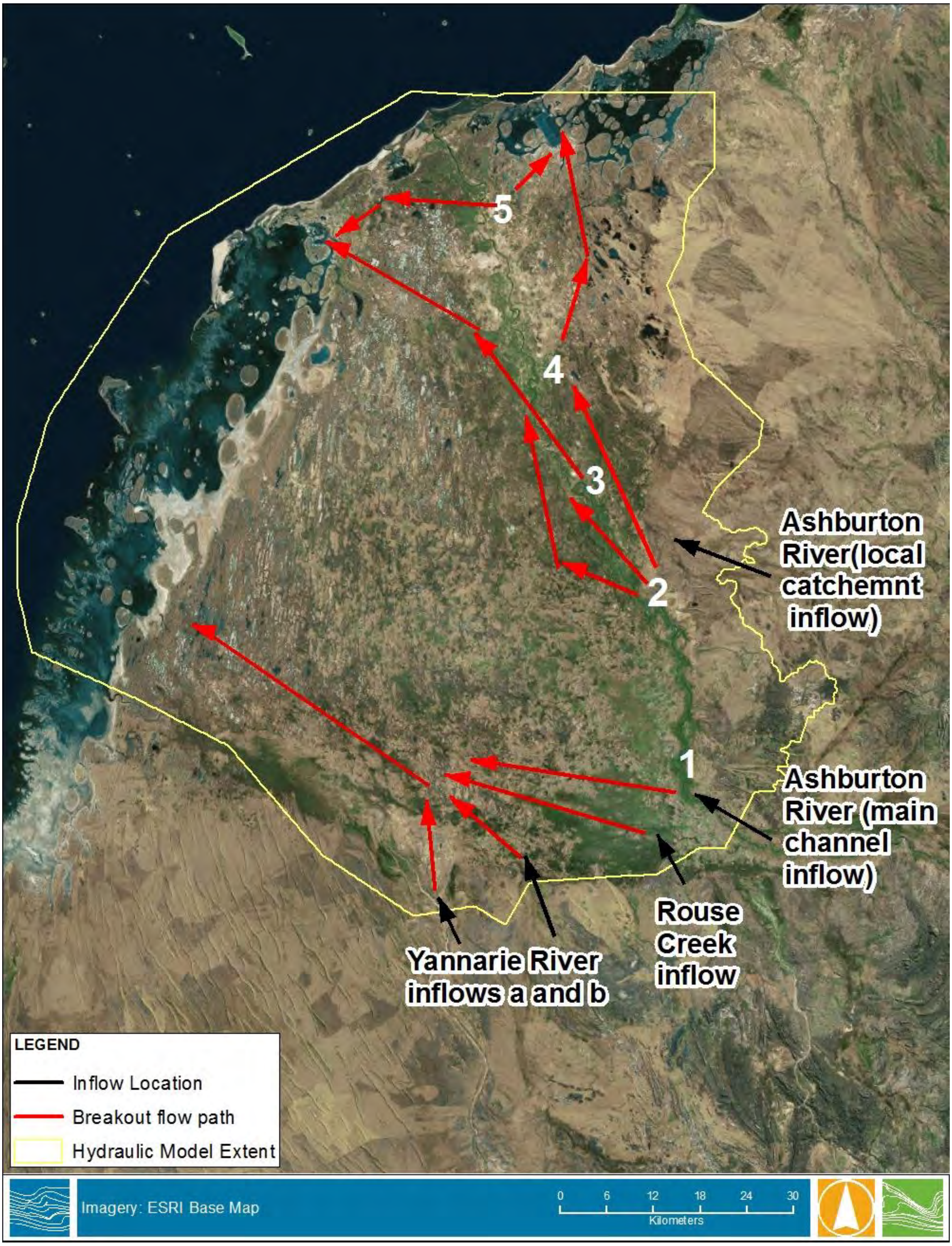

FIGURE 9-3 BREAKOUT LOCATIONS AND FLOOD FLOW PATHS MODELLED FOR CYCLONE VANCE

TABLE 9-2 CATCHMENT FLOOD BEHAVIOUR – CYCLONE VANCE

River	Event
Yannarie River and Rouse Creek	Flows from these catchments enter the southern section of the model, and take approximately 7 hours to reach the intertidal zone along the flow paths identified in Figure 9-3.
Ashburton River (main channel)	Flow through the main Ashburton River channel takes approximately 19 hours to reach the intertidal zone once it has entered the model at the Ashburton River main channel inflow boundary at the Nanutarra gauge.
Ashburton River (breakouts)	1. The first breakout from the Ashburton River main channel occurs at location 1. This flow path merges with flows from the Yannarie River and Rouse Creek. 2. The second breakout occurs at location 2 with flooding occurring on both sides of the main river channel. This is the most extensive breakout from the Ashburton River and provides a significant overland flood path westward towards the project area. 3. The third breakout occurs at location 3 with the westerly overland flow path merging with that of breakout 2. 4. The fourth breakout is to the east at location 4 and diverts overland flows towards Onslow. 5. The final breakout occurs at location 5 where coastal and riverine flooding both occur. The result is significant inundation on both sides of the river.

9.2 Design Flood Events (Scenarios 2 to 8)

Design flood events have been modelled to provide information on inundation due to riverine flooding and to understand overland flow behaviour across the project area.

The breakout locations indicated for the calibration case also occur in the design flood events. The two main overland flow paths observed for all design scenarios are highlighted in Table 9-3, which also describes the flood behaviour for each event.

TABLE 9-3 DESIGN EVENT FLOOD BEHAVIOUR

Design Event	Flood Behaviour Description
Scenario 2 – 10% AEP (Figure 9-5)	Flooding fans out from the first major right-hand bend downstream of the Ashburton River main channel inflow and results in overland flows to the west and east of this point. The western overland flows merge with the Yannarie River and the eastern overland flows move through the dune field parallel to the main channel of the Ashburton River. The flow through the Ashburton River results in breakouts at the same points identified within Figure 9-3. Both coastal and riverine flooding are observed at the downstream end of the Ashburton and Yannarie rivers with a maximum inundation level of 1 m observed within the salt flats (the average inundation level of salt flat is 0.50 m).

Design Event	Flood Behaviour Description
Scenario 3 – 5% AEP (Figure 9-6)	Flooding fans out from the first major right-hand bend downstream of the Ashburton River main channel inflow and results in overland flows to the west and east of this point. The western flows merge the Yannarie River flood path, however two distinct overland flow paths are observed through the dune field to the east of the main Yannarie river channel. Significant overland flows outside the main Ashburton River channel are observed within the dune field. Within the Ashburton River breakouts again occur in the same locations observed within Figure 9-3. Flooding of the supratidal zone is observed with a maximum inundation level of 1 m observed within the salt flats (the average inundation level of salt flat is 0.75 m).
Scenario 4 – 2% AEP (Figure 9-7)	Significant inundation extents originating at the inflow location of the Ashburton River main channel are observed. Overland flows observed within the dune field are significant and result in two distinct flow paths with widths of up to 20 km. Breakouts of the Ashburton River again occur in the same locations observed within Figure 9-3. Inundation within the coastal zone encompasses most of the salt flats with water levels reaching greater than 1 m in multiple locations (the average inundation level of salt flat is 1 m).
Scenario 5 – 1% AEP catchment, 1 in 50 Year ARI storm tide (Figure 9-8)	Again, significant inundation extents originating just downstream of the inflow point of the Ashburton River main channel inflow occur. Overland flows extend to the west and east along the Yannarie and Ashburton River channels. A significant volume of water flows through the dune system in between the Yannarie and Ashburton main river channels with only a small section of the dune system remaining dry during the flood event. Breakouts of the Ashburton River again occur in the same locations observed within Figure 9-3. Coastal and riverine inundation within the coastal zone encompasses the majority of the salt flats with water levels reaching between 1-2m in the majority of the salt flat region.
Scenario 6 – 1% AEP catchment, 1 in 100 Year ARI storm tide (see Figure 9-9)	Significant flooding originates at the inflow location of the Ashburton River main channel, before these overland flows branch off into two distinct overland flow paths, the flood width is in excess of 30 km. Two distinct overland flow paths are observed through the dune field and inundate the majority of the area. Breakouts of the Ashburton River again occur in the same locations observed within Figure 9-3. Flooding within the coastal region of the model is almost entirely between 1-2 m.
Scenario 7 – 1% AEP catchment, 1 in 500 Year ARI storm tide (Figure 9-10)	Flow patterns of the Ashburton and Yannarie Rivers are the same as in the 1% AEP catchment, 1 in 100 Year ARI storm tide scenario, with only a slight increase in coastal inundation extents and depth observed. Ashburton River breakouts again occur in the same locations observed within Figure 9-3. Inundation within the coastal zone encompasses the majority of the salt flats with water levels reaching greater than 2 m for multiple areas.
Scenario 8 – 1% AEP catchment, 1 in 500 Year ARI storm tide + mean sea level rise to 2070. (Figure 9-11)	Flow patterns of the Ashburton and Yannarie Rivers are the same as in the 1% AEP catchment, 1 in 500 Year ARI storm tide scenario. An increase in coastal inundation extents and depth are observed compared to the 1% AEP catchment, 1 in 500 Year ARI storm tide scenario. Ashburton River breakouts again occur in the same locations observed within Figure 9-3. Inundation within the coastal zone encompasses the majority of the salt flats with water levels reaching greater than 2 m in multiple large areas.

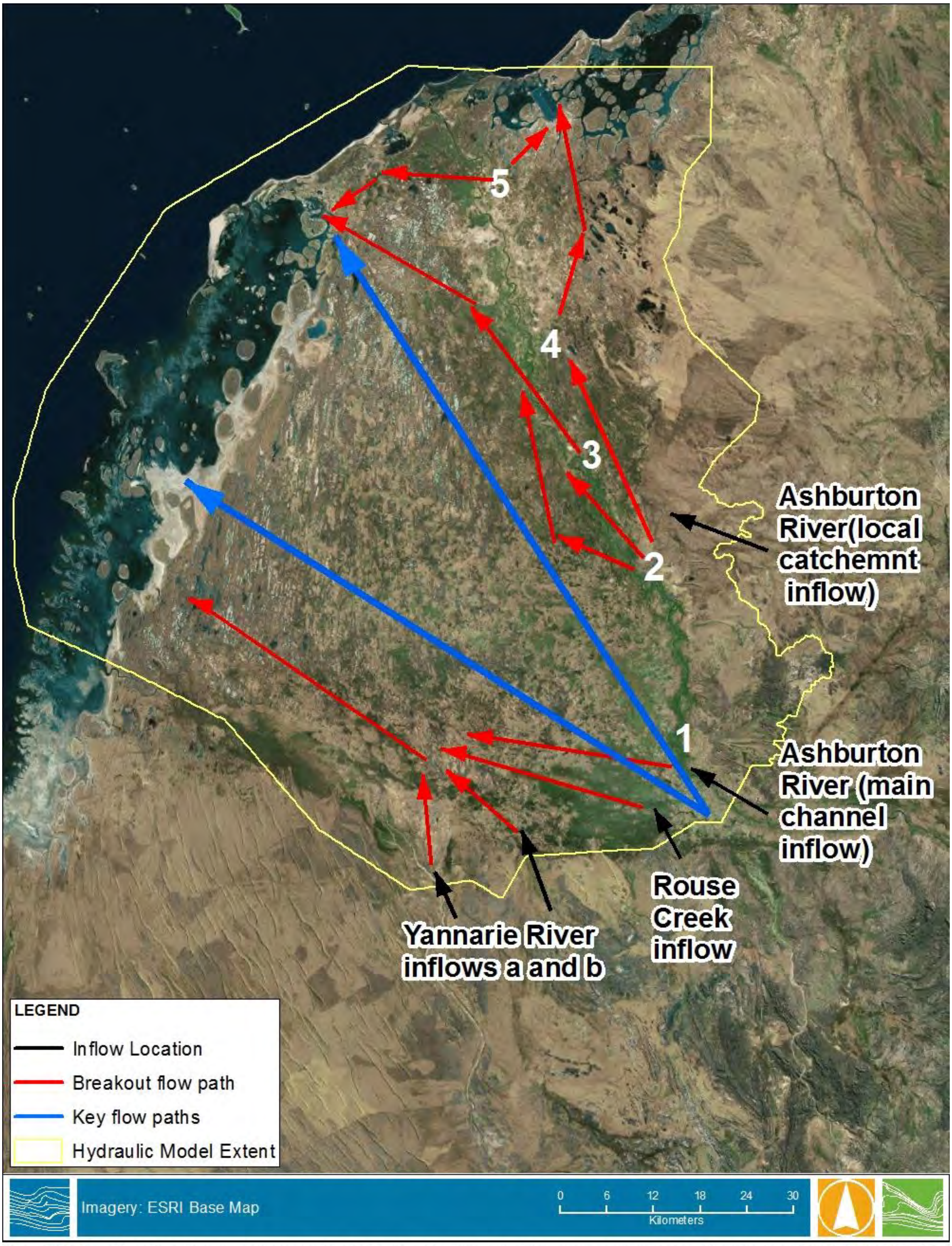

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FIGURE 9-4 DESIGN EVENT FLOW PATHS

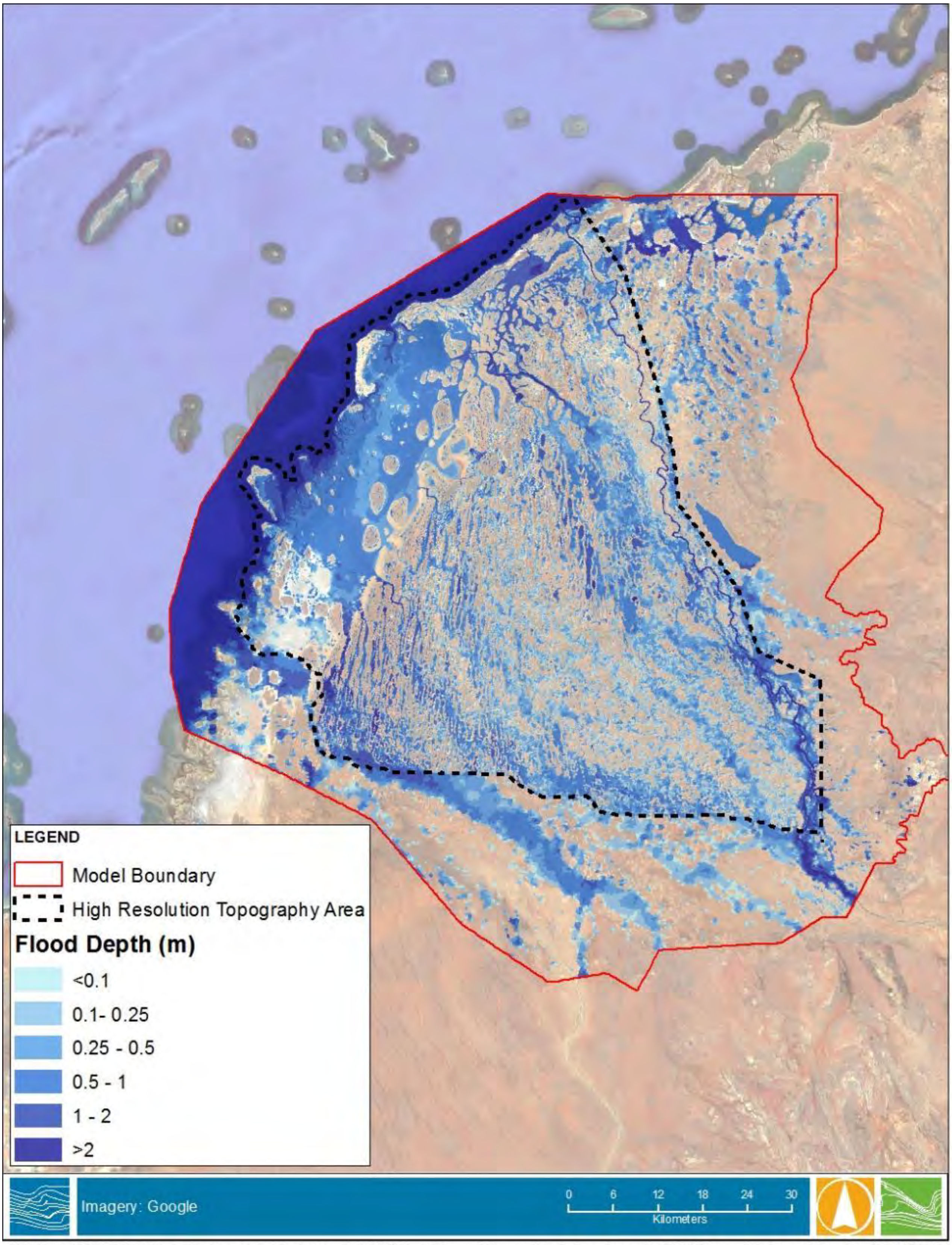

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FIGURE 9-5 MODEL SCENARIO 2: 10% AEP CATCHMENT FLOOD, MAXIMUM INUNDATION EXTENT

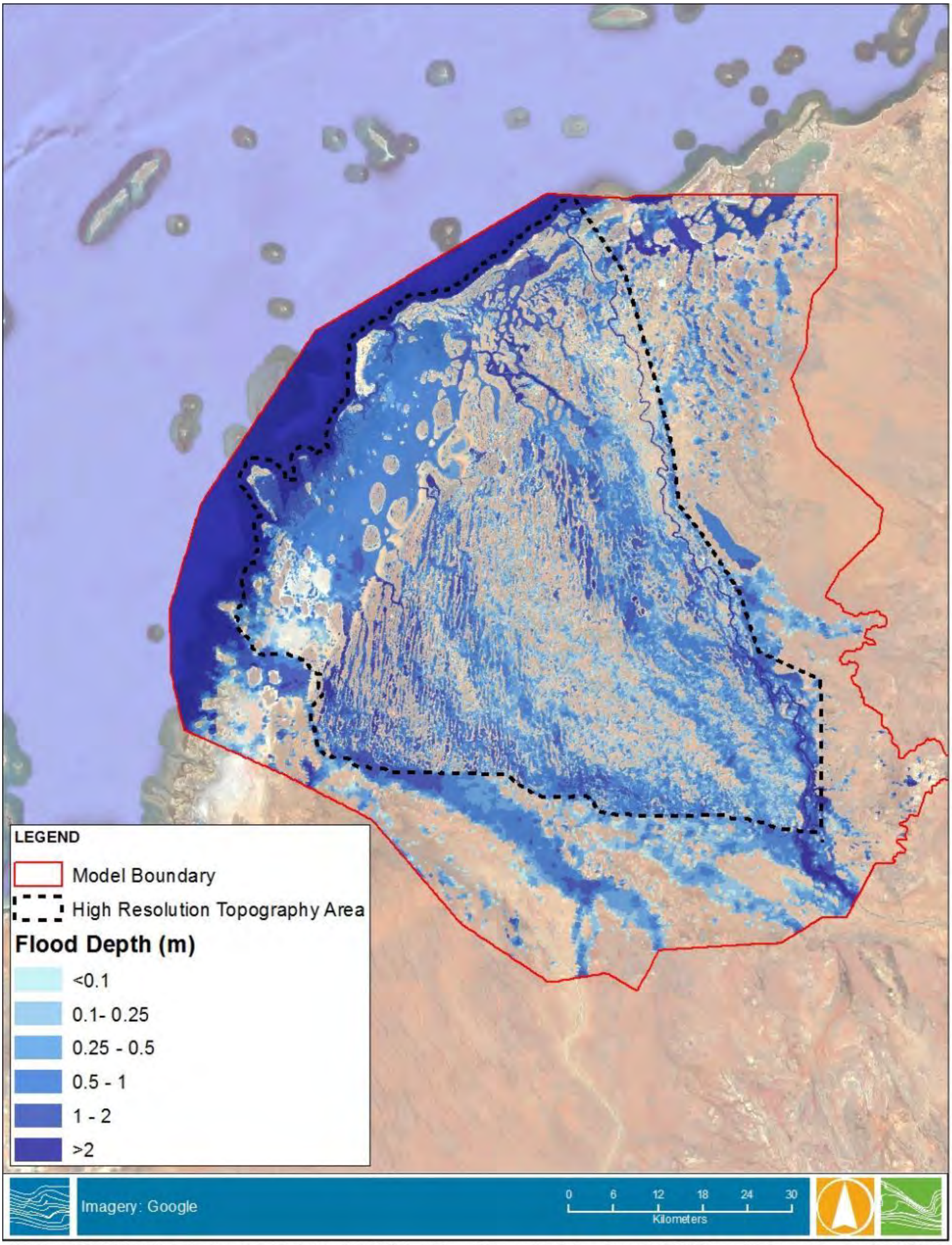

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FIGURE 9-6 MODEL SCENARIO 3: 5% AEP CATCHMENT FLOOD, MAXIMUM INUNDATION EXTENT

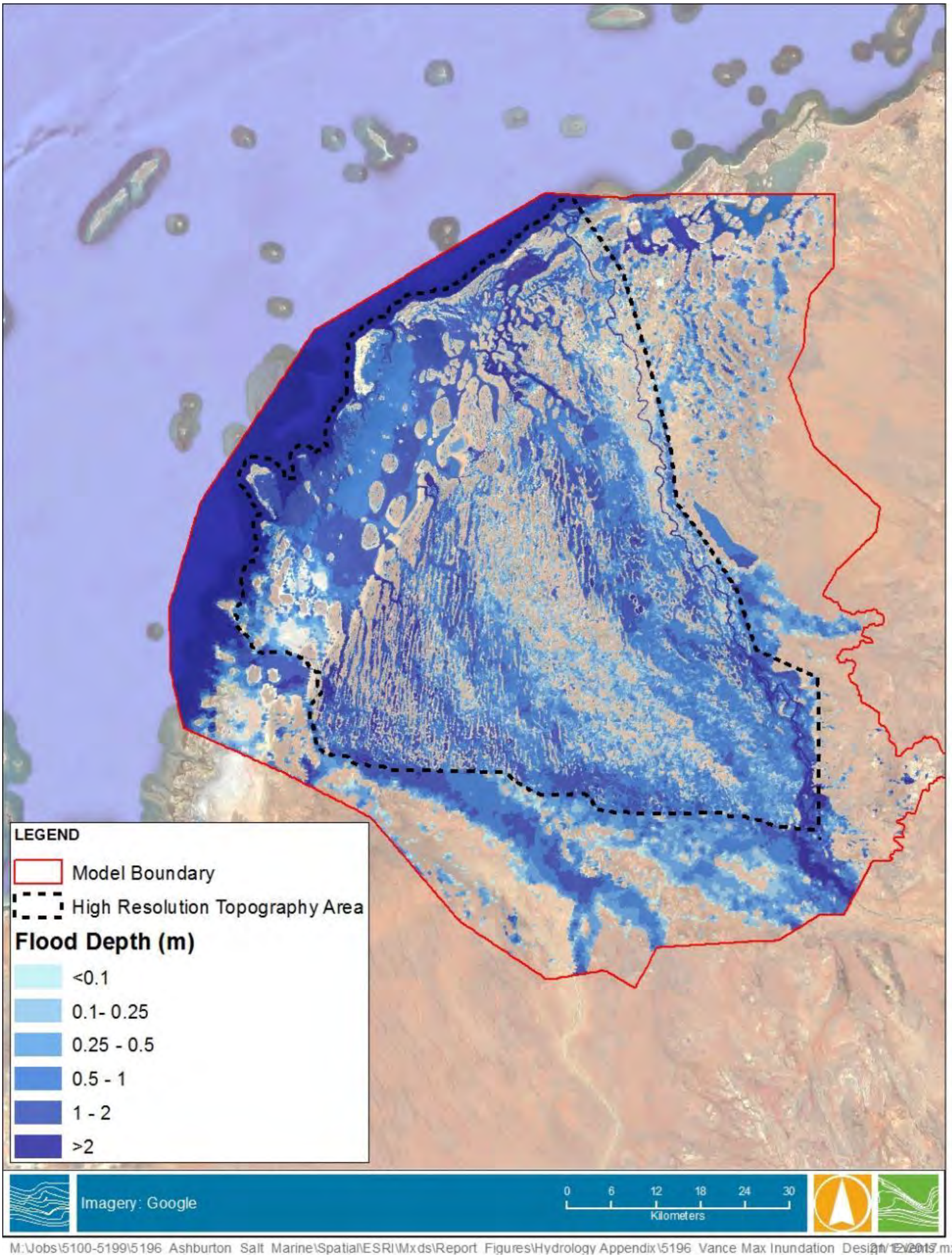

FIGURE 9-7 MODEL SCENARIO 4: 2% AEP CATCHMENT FLOOD, MAXIMUM INUNDATION EXTENT

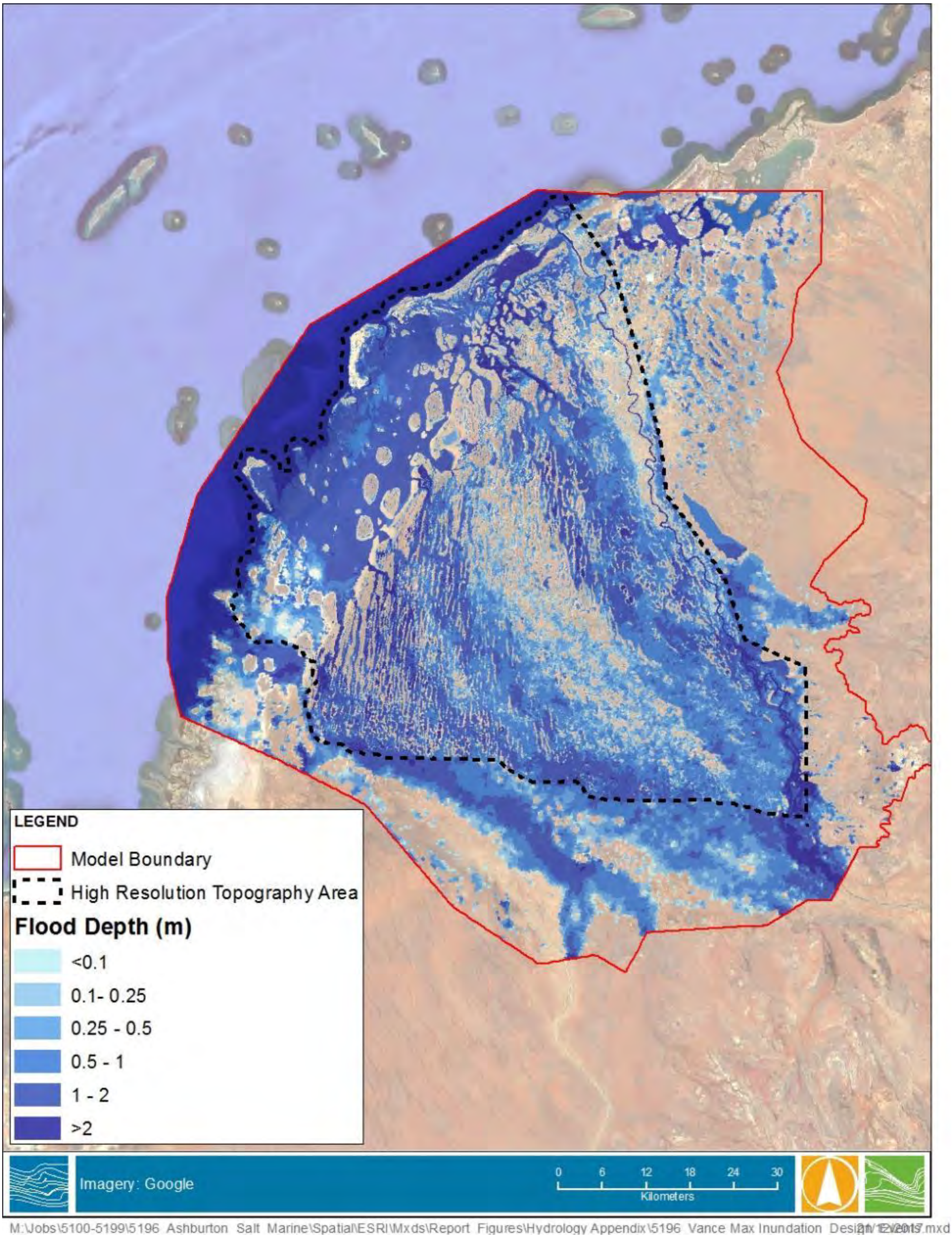

FIGURE 9-8 MODEL SCENARIO 5: 1% AEP CATCHMENT FLOOD AND 1 IN 50 YEAR ARI STORM TIDE, MAXIMUM INUNDATION EXTENT

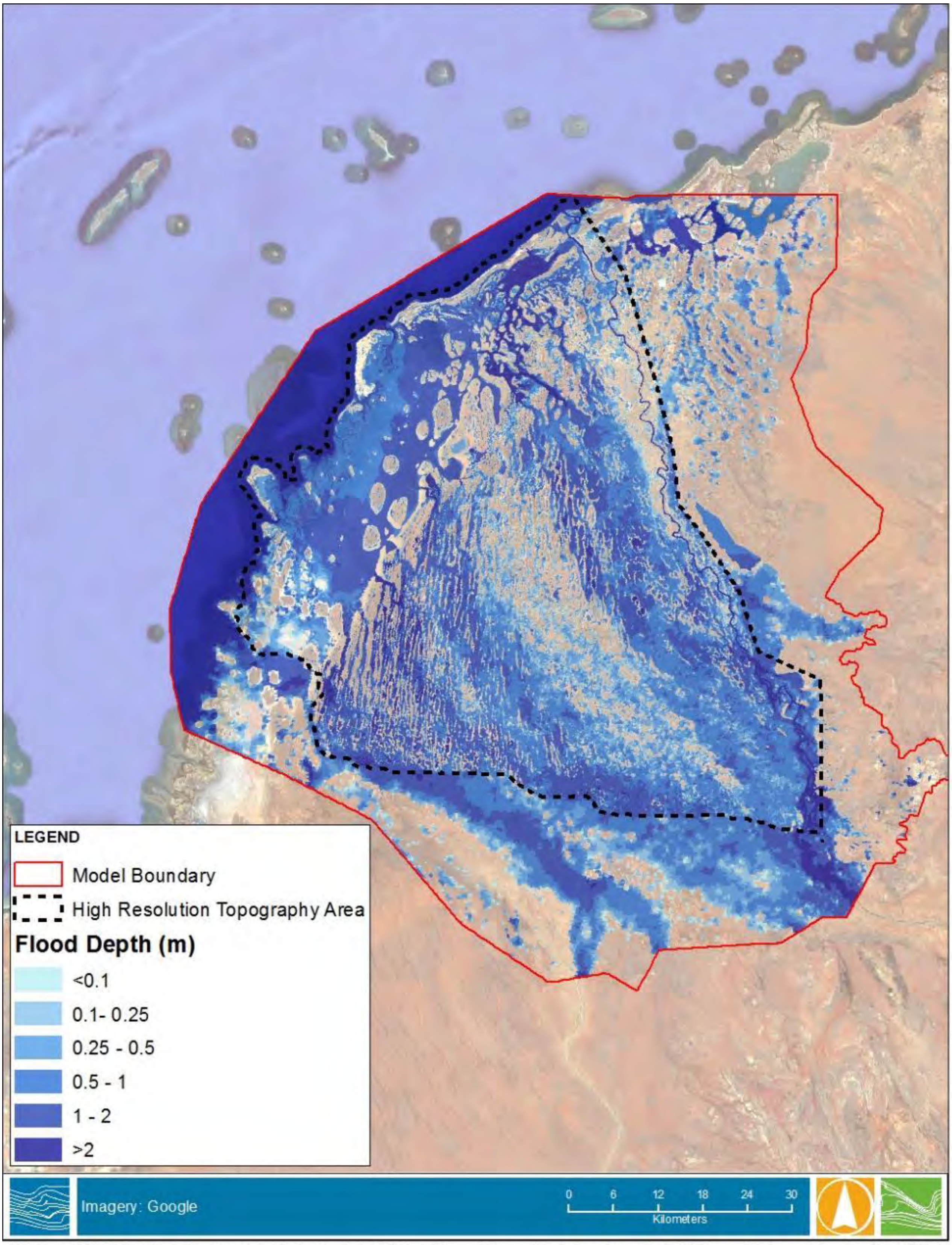

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FIGURE 9-9 MODEL SCENARIO 6: 1% AEP CATCHMENT FLOOD AND 1 IN 100 YEAR ARI STORM TIDE, MAXIMUM INUNDATION EXTENT

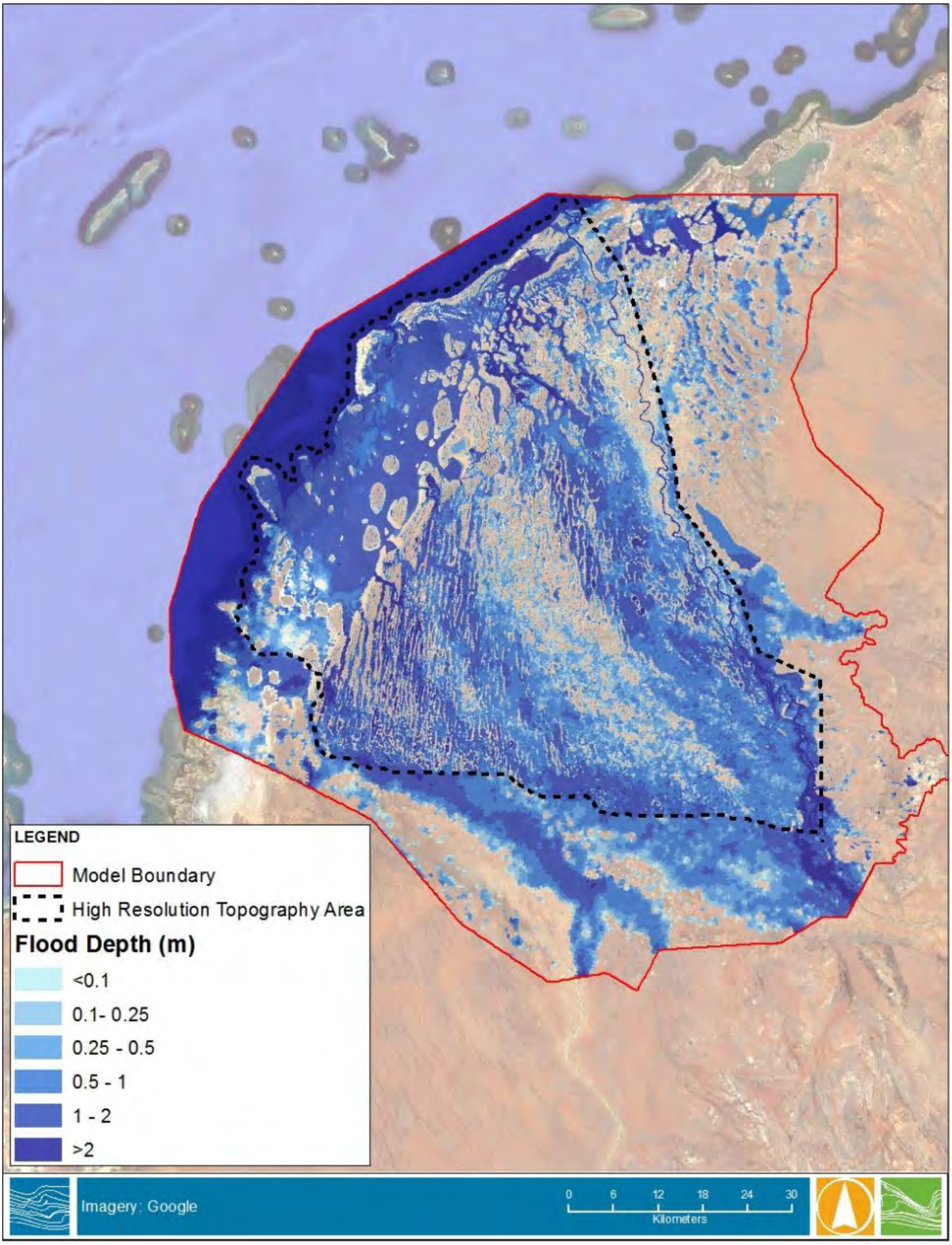

FIGURE 9-10 MODEL SCENARIO 7: 1% AEP CATCHMENT FLOOD, 1 IN 500 YEAR ARI STORM TIDE, MAXIMUM INUNDATION EXTENT

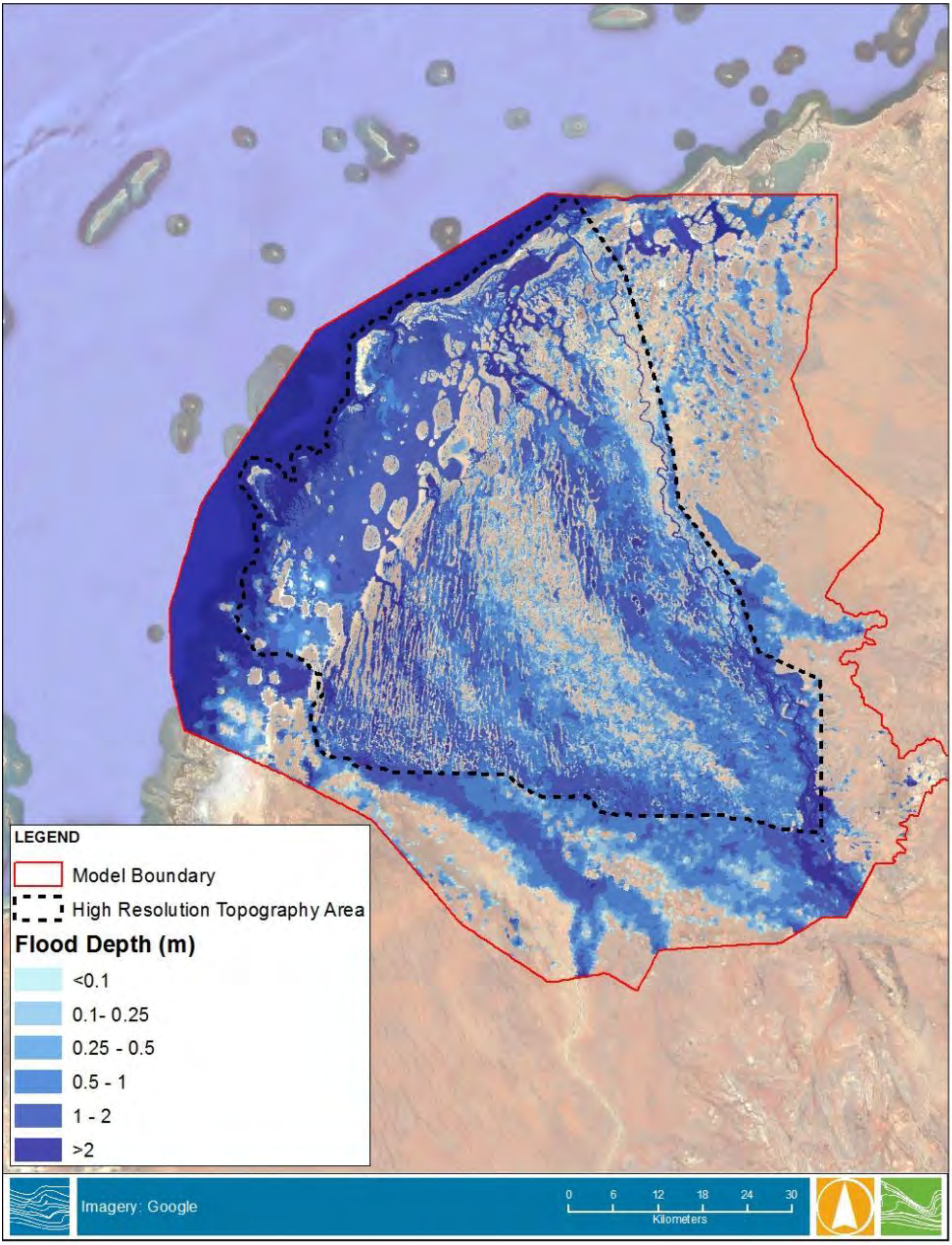

FIGURE 9-11 MODEL SCENARIO 8: 1% AEP CATCHMENT FLOOD, 1 IN 500 YEAR ARI STORM TIDE PLUS MEAN SEA LEVEL RISE TO 2070, MAXIMUM INUNDATION EXTENT

9.3 Salt Flats - Flooding Extent and Duration

Major (10% to 10%) AEP rainfall events (as described above) are of interest for infrastructure design and determination of hydrological impacts of major rainfall events. However, more frequent rainfall events (e.g. 50% to 20% AEP) are also of interest in characterising the environment and habitats which may become available for fauna use after flooding. For example, infrequently inundated areas such as salt flats can infrequently offer potential habitat for water birds, if significant extent and duration of flooding occurs.

To assist with habitat characterisation, limited modelling of minor rainfall events was conducted. An estimate of the duration of flooding within the salt flats was calculated for both minor and major rainfall events, based on average modelled flood depths and the evaporation rate of 12 mm per day (given evaporation causes removal of floodwaters). The adopted evaporation rate of 12 mm is the approximate average daily evaporation rate for the period November through to March, when the majority of rainfall occurs in the area.

Table 9-4 below has been prepared to summarise flooding extent and duration predicted to occur within the salt flats under both minor and major rainfall events, in order to assist with habitat characterisation being conducted as part of the ERD. These estimates are conservative given infiltration rates are not considered.

TABLE 9-4 ESTIMATED SALT FLAT FLOODING EXTENT AND DURATION FOR VARIOUS RAINFALL EVENTS

Rainfall Event (AEP)	Approx. ARI	Modelled Flood Extent and Depth of Salt Flats	Estimate of Salt Flat Flooding Duration (Days)
50%	2 year	Figure 9-12 – average flood depth in salt flats of 0.25 m.	20
20%	5 year	Figure 9-13 – average flood depth in salt flats of 0.35 m.	29
10%	10 year	Figure 9-5 – average flood depth in salt flats of 0.50 m.	41
5%	20 year	Figure 9-6 – average flood depth in salt flats of 0.75 m.	62
2%	50 year	Figure 9-7 – average flood depth in salt flats of 1 m.	83

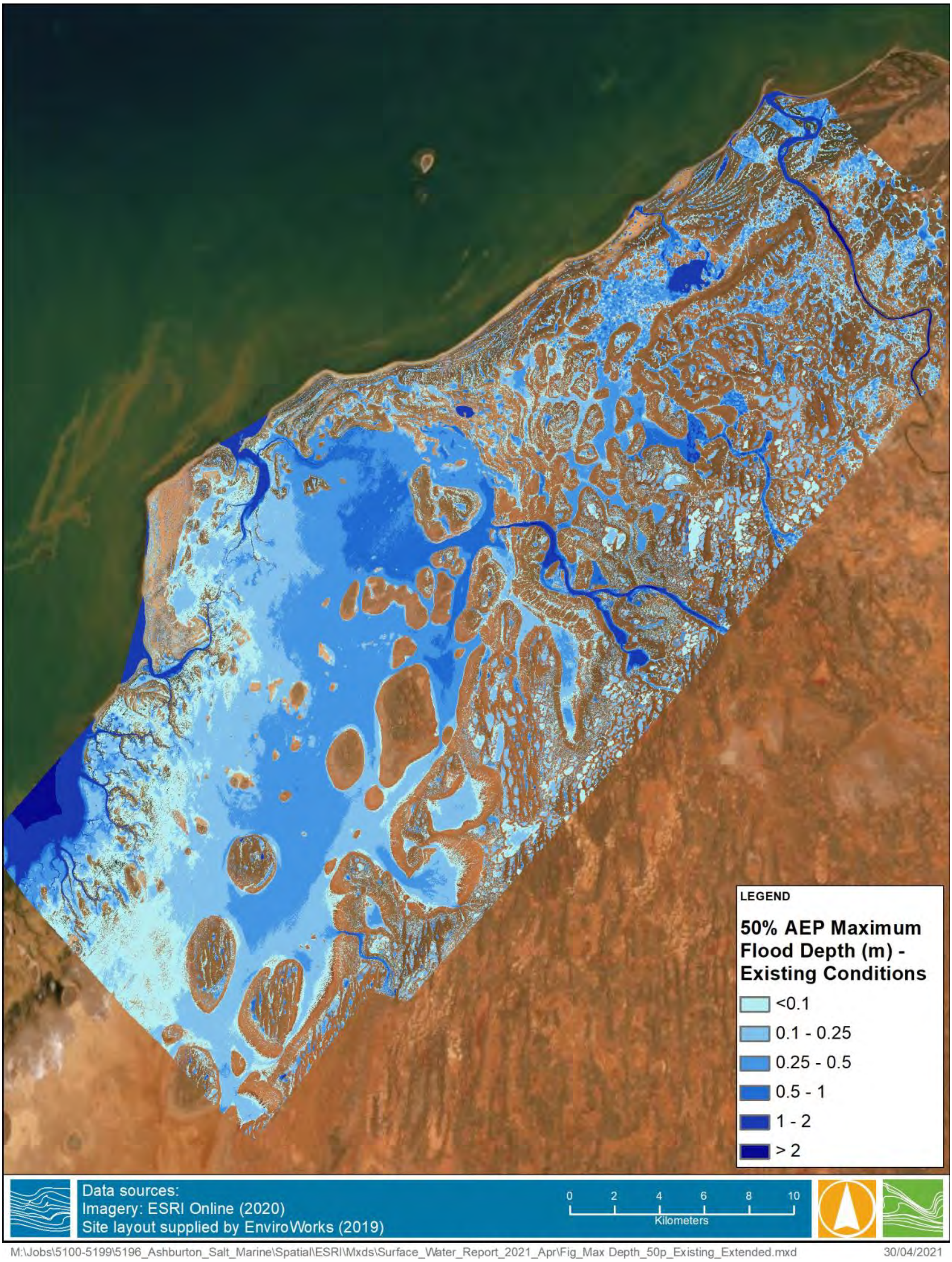

FIGURE 9-12 MODELLED 50% AEP CATCHMENT MODELLED FLOOD INUNDATION EXTENT

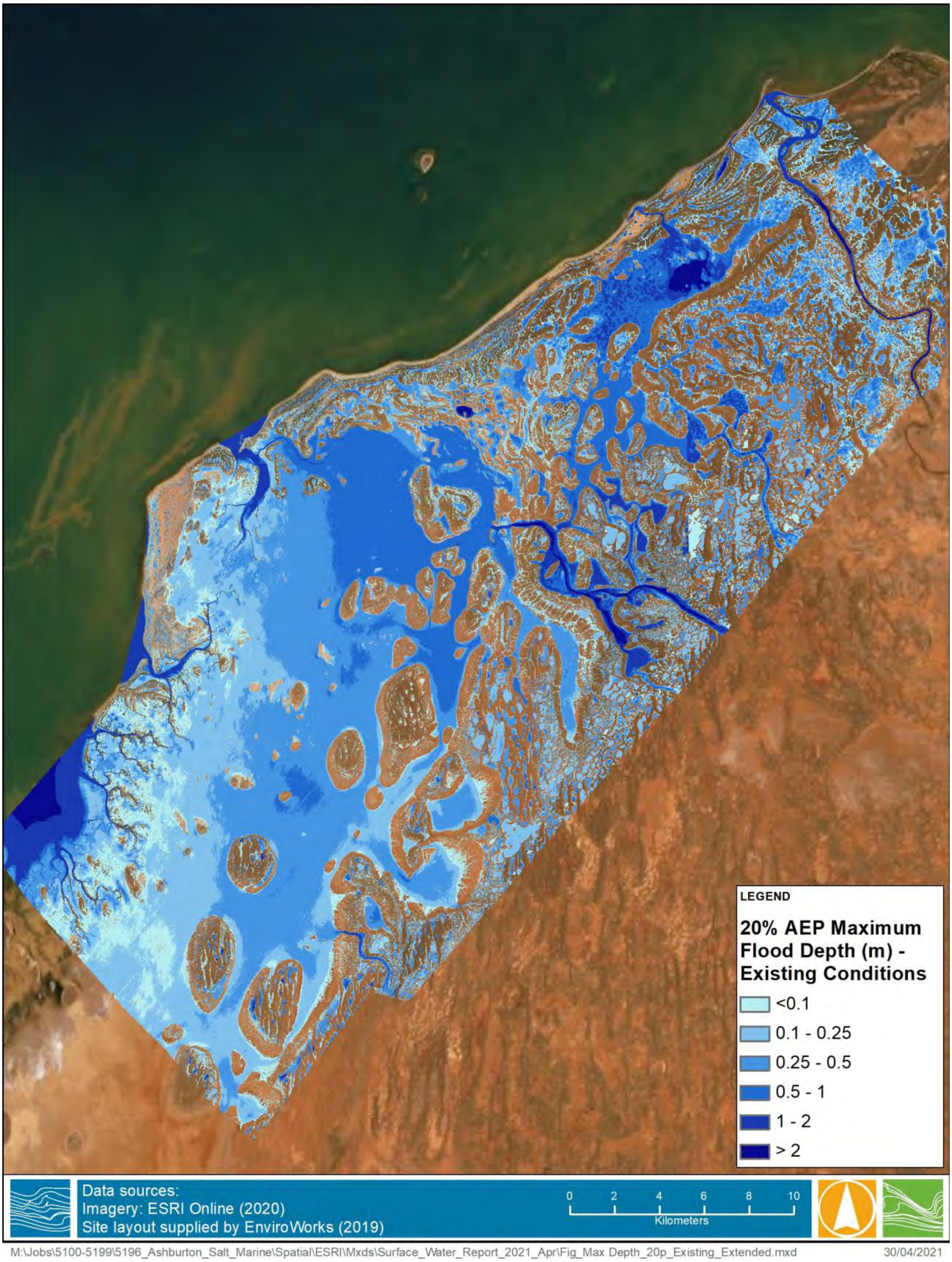

FIGURE 9-13 MODELLED 20% AEP CATCHMENT MODELLED FLOOD INUNDATION EXTENT

10 IMPACT ASSESSMENT

This section describes impact assessments conducted using the models described previously. Any impacts due to the development will be highlighted via afflux plots, vector velocity figures at select locations, flood depth and water level inundation illustrations. The local TUFLOW hydraulic model was used to simulate developed conditions at the site and to assess any impacts associated with the development.

10.1 Proposed Development

The proposed development comprises 8 salt ponds, a jetty, a stockyard, onsite buildings, conveyor, access road, bitterns pond, washdown bay and 12 crystalliser ponds. The site covers a large area and alterations to the topography will be undertaken to establish ponds for salt harvesting. The evaporation ponds comprise a series of embankment walls of varying heights. To ensure these were modelled accurately, embankment walls were incorporated into the DEM by raising elevations within the DEM from 3 to 6 m high, with a width of approximately 4 m. Figure 10-1 illustrates a cross-section of the modelled topographic changes whilst Figure 10-2 shows the location of the cross-section aerially. Figure 10-3 illustrates the proposed concept layout plan with all embankments raised in the model as depicted in this figure.

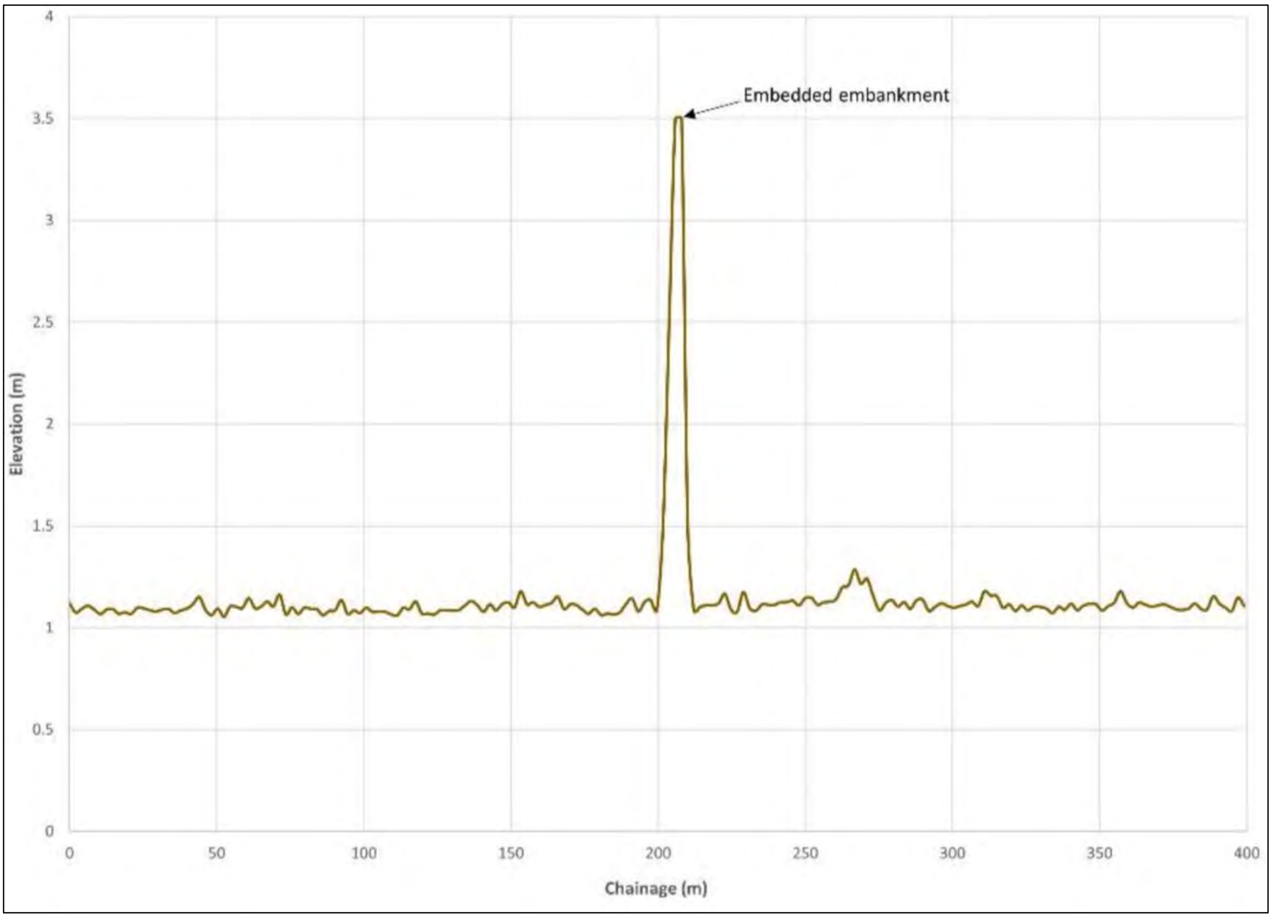

FIGURE 10-1 EXAMPLE EMBANKMENT X-SECTION

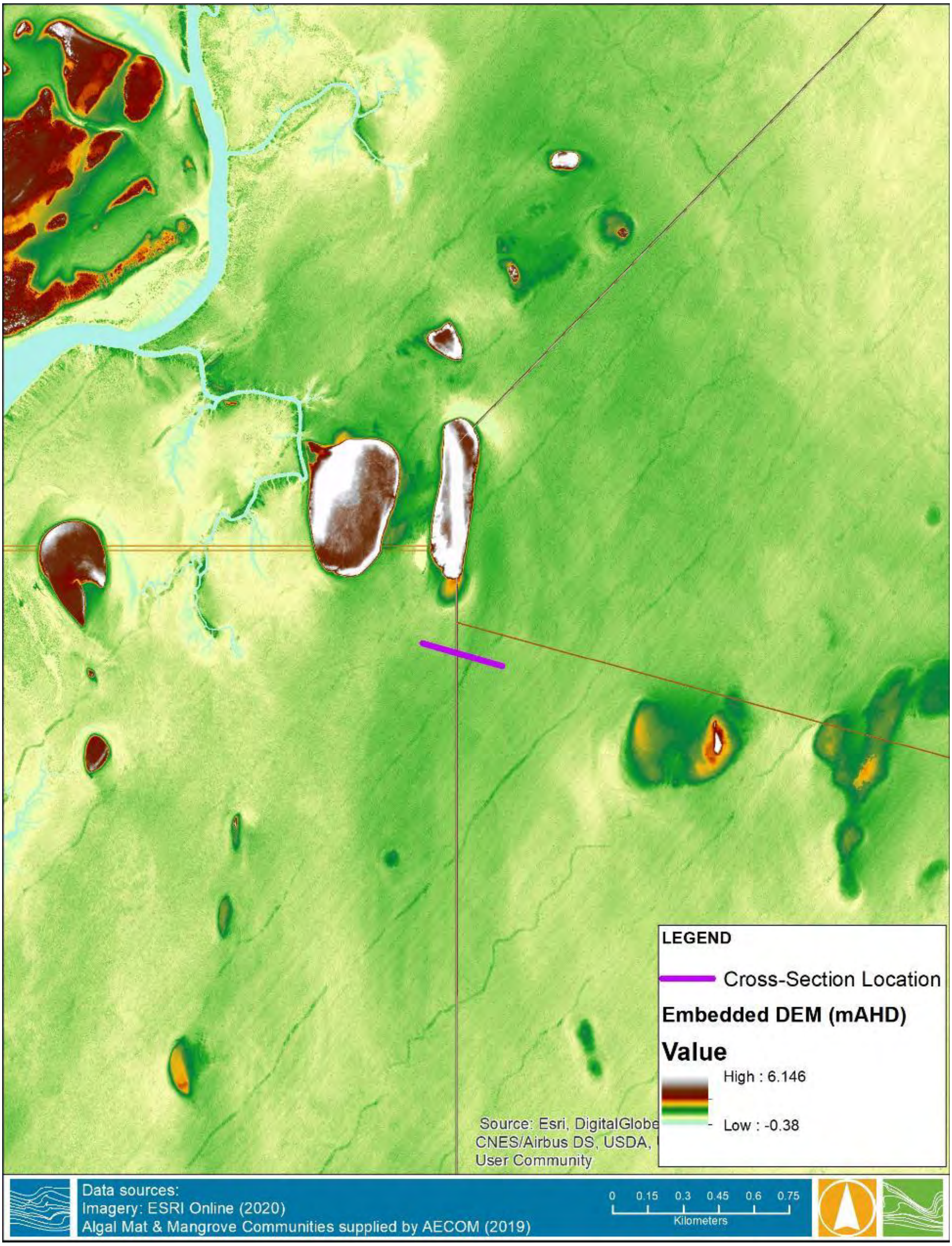

FIGURE 10-2 DEM SHOWING EMBEDDED EMBANKMENT

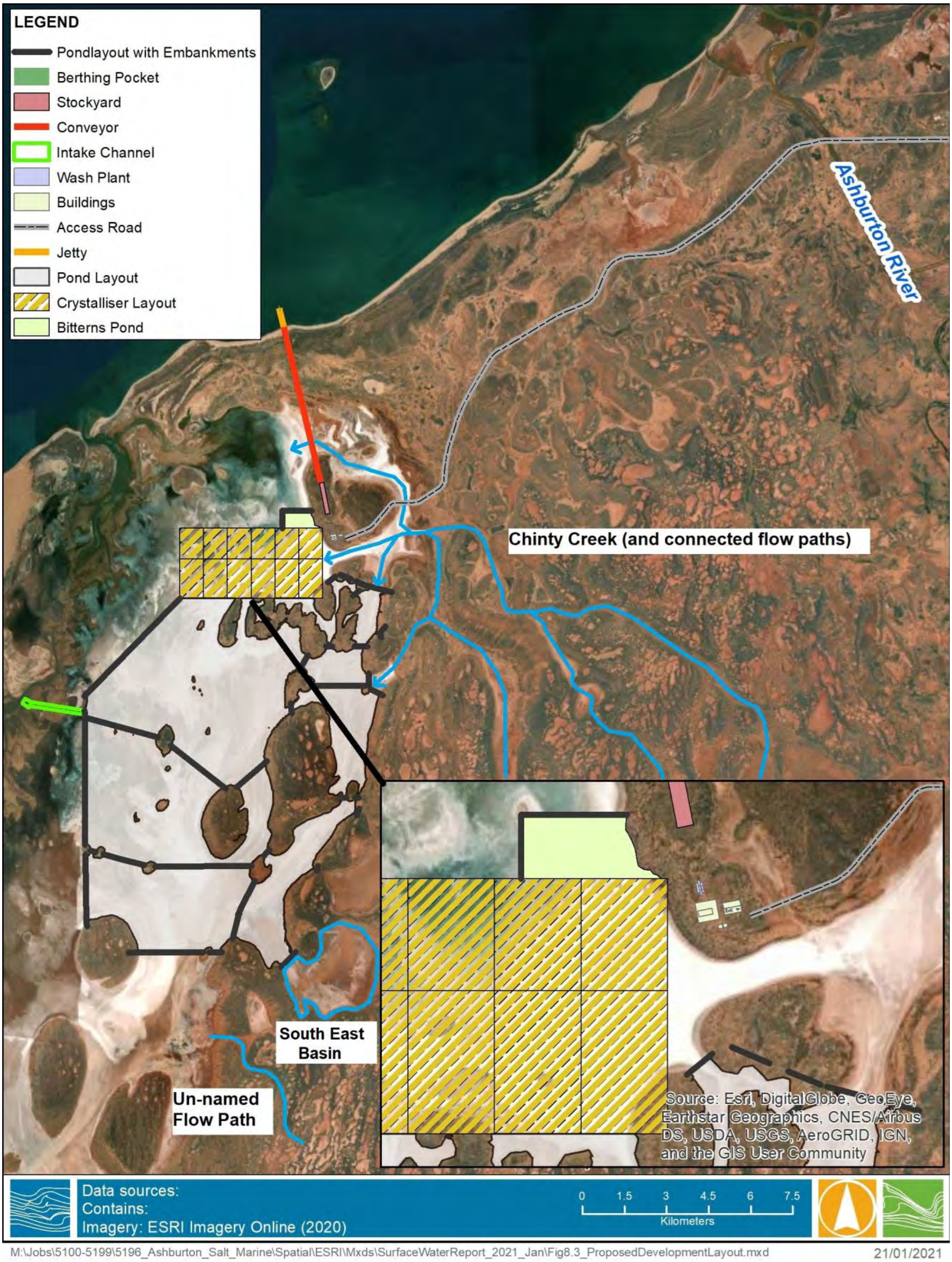

FIGURE 10-3 PROPOSED DEVELOPMENT LAYOUT AND INLAND FLOW PATHS

10.2 Overview

As specified in Section 8, a suite of design events (10% - 1%) was simulated to assess potential impacts of the proposed development. The following sections compare pre- and post-development scenarios. Maps are shown in Figure 10-5 to Figure 10-37 for the 10%, 5%, 2% and 1% AEP flood events. The figures show the following for pre- and post-development cases:

- Flood depths;
- Water levels;
- Flow velocities;
- Vector velocity maps (at key locations only); and
- Changes in water levels (i.e., afflux).

These data can be used to guide the design and protection of roads and infrastructure at the site during significant rainfall events. Results from the modelling were used to develop recommended flood mitigation strategies for the site.

10.3 TUFLOW Modelling Results

Key features taken from the hydraulic modelling are noted in Table 10-1 whilst a general summary of surface water movements through the development under both existing and post development conditions is provided in the following sections.

10.4 Existing Conditions

Interrogation of the TUFLOW modelling results (under existing conditions) suggests that rainfall generated surface water movement into the project area originates largely from inland flow paths. Chinty Creek and a series of interconnected overland flow paths exist in the to the north east of the proposed development resulting in overland flow into the salt flats. To the south of the salt ponds there is a smaller un-named overland flow path into the salt flats. In addition, a large basin (south east basin) exists adjacent to the proposed south eastern pond, which fills up with water after rainfall causing water to flow into the salt flats (Figure 10-3).

Pre-development water depths in these overland flow areas immediately upstream of the planned evaporation ponds are predicted to be approximately 1.0 – 2.5 m during the 2% AEP design event.

Once these flows discharge onto the salt flats, surface water moves uniformly as broad shallow sheet flow across the tidal flats where the evaporation ponds will be located.

Flood depths in the salt flats range from 0.1 to 1.4m (during the 2% AEP design event).

During high tides, surface water also travels across the tidal flats via several tidal creeks, namely Urala Creek South and North and an un-named tidal creek positioned approximately 15 km to the north of Urala Creek North. Modelling results suggest flow velocities (<0.2 m/s) are almost negligible across the entire site which was expected given the very flat grades.

10.5 Post-development Conditions (Unmitigated)

Under post-development conditions, modelling results suggest that water moving across the site from the existing overland flow paths is blocked by the access road, conveyor and pond embankments - resulting in significant water level increases (between 1 – 6 m) along the eastern margins of the site. Meanwhile, the western margins of the access road prevent tidal waters from travelling east. When considering flow velocities in the post-development scenario, velocities are comparable to existing conditions. The only area of minor variation are the slightly reduced flow velocities on the upstream side of the conveyor and crystalliser due to surface water flows backing up; see vector velocity maps for details of water movement at these sites (e.g. Figure 10-8 to Figure 10-11). A summary of modelled changes to water movement under unmitigated post-development conditions at each significant infrastructure location is provided below:

- ***Access Road*** - Along the north-south access road, water backs up and spreads laterally at the base of the road. Where the access road meets the crystalliser and evaporation ponds, flood waters converge and reach maximum flood depths of 4.0 m during the 2% AEP.
- ***Evaporation Pond Embankments*** - Along the embankment, flood waters back up and fill either the basins or the creek system upstream of the embankments before overtopping (during events >2% AEP) and travelling down gradient towards the most southern tip of the development; as shown in Figure 10-29.
- ***Conveyor*** - At the proposed conveyor site, tidal water movement from the north is impeded by the conveyor embankment resulting in an increase in water levels to the north and a reduction in water levels on the south side of the conveyor. Down gradient of the conveyor are the algal mat and mangrove communities which under existing conditions would receive tidal contributions from the north.

A more detailed description of modelled results for all events (10% - 1% AEP) under post development conditions is provided in Table 10-1.

TABLE 10-1 MODEL OUTPUTS KEY FEATURES – POST-DEVEOPMENT CONDITIONS (UNMITIGATED)

Rainfall Event	Figure Reference	Key Features
10% AEP	Figure 10-5 to 10-12	■ Flood depths in the development area range between 0.5 m and 4.0 m. Shallow depths are noted along the western margins of the site (<1.0 m) and maximum depths are noted at the ponds north-eastern corner (approximately 3.7 m). ■ Water depths on northern side of the conveyor range from 0.5 – 1.5 m. Maximum depths noted at the eastern ends of the alignment. ■ Flood depths at proposed Ashburton River bridge crossing reach 2.7 m. ■ Surface water backs-up and spreads laterally at the base of the north-south access road. Flood depths range between 0.1 m and 3.9 m on both the eastern and western edges of the road. Maximum water depths are noted at the southern end of alignment adjacent to the crystalliser. ■ Velocities across the site are negligible (<0.1 m/s) with maximum velocity (1.0 m/s) noted in the un-named creek to the south of the development.
5% AEP – Proposed Access Road Design Event	Figure 10-13 to 10-20	■ Flood depths in the development area range between 0.2 m and 4.5 m. Shallow depths noted along the western margins of the site (<1.0 m) and maximum depths noted at the ponds north-western corner (approximately 4.5 m). ■ Water depths on northern side of the conveyor range from 0.1 – 1.7 m, with maximum depths noted at the western ends of the alignment.

Rainfall Event	Figure Reference	Key Features
		■ Flood depths at proposed Ashburton River bridge crossing reach 3.8 m. ■ Surface water backing up behind the north-south access road ranges between 0.15 m and 3.9 m in depth on both the eastern and western edges of the road. Maximum water depths noted at the southern end of alignment (adjacent to the crystallisers) where depths reach 3.9 m. ■ Velocities across the site remain low (<0.15 m/s) with maximum velocity (2.2 m/s) noted in the un-named creek to the south of the development.
2% AEP – Embankment & bridge crossing design Event	Figure 10-21 to 10-29	■ Flood depths in the development area range between 1.0 m and 5.8 m. Shallow depths noted along the western margins of the site (<1.3 m) and maximum depths noted at the ponds south-eastern corner (approx.5.8 m). ■ Water depths increase on northern side of the conveyor. Depths range from 0.3 – 1.8 m. Maximum depths noted at the western ends of the alignment. ■ Flood depths at proposed Ashburton River bridge crossing reach 4.25 m. Velocities upstream and downstream of the bridge are <0.5 m/s. ■ Surface water backs-up and spreads laterally at the base of the north-south access road. Flood depths range between 0.30 m and 4.4 m on western edges of the road. Maximum water depths noted at southern end of alignment (adjacent to the crystallisers) where depths reach 4.4 m. ■ Velocities across the site remain low (<0.15 m/s) with maximum velocities of 1.0 m/s and 3.2 m/s noted in Chinty creek to the north and the un-named creek south of the development, respectively.
1% AEP	Figure 10-30 to 10-37	■ Flood depths in the development area range between 0.9 m and 5.8 m. Tidal driven surface water backs up behind the conveyor, access road and embankment resulting in significant flood depth across the site. Maximum flood depths are noted at the junction of the crystalliser and ponds (approximately 4.9 m) in the north-east and south-east corner (approximately 5.8 m). ■ Water depths remain similar to the 2% AEP along the conveyor alignment. Depths range from 0.2 – 1.9 m. Maximum depths noted at the far eastern and western ends of the conveyor alignment. ■ Flood depths at proposed Ashburton River bridge crossing reach 4.3 m whilst flow velocities remain low (0.5 m/s) upstream and downstream of the proposed bridge. ■ Surface water backs-up and spreads laterally at the base of the north-south access road. Flood depths range between 0.20 m and 4.9 m on the western edges of the road. Maximum water depths

Rainfall Event	Figure Reference	Key Features
		noted at southern end of the road alignment (adjacent to the crystallisers) where depths reach 4.9 m. ■ Velocities across the site remain low (<0.15 m/s) with maximum velocities of 1.2 m/s and 3.3 m/s noted in Chinty creek to the north and the un-named creek to the south of the development, respectively.

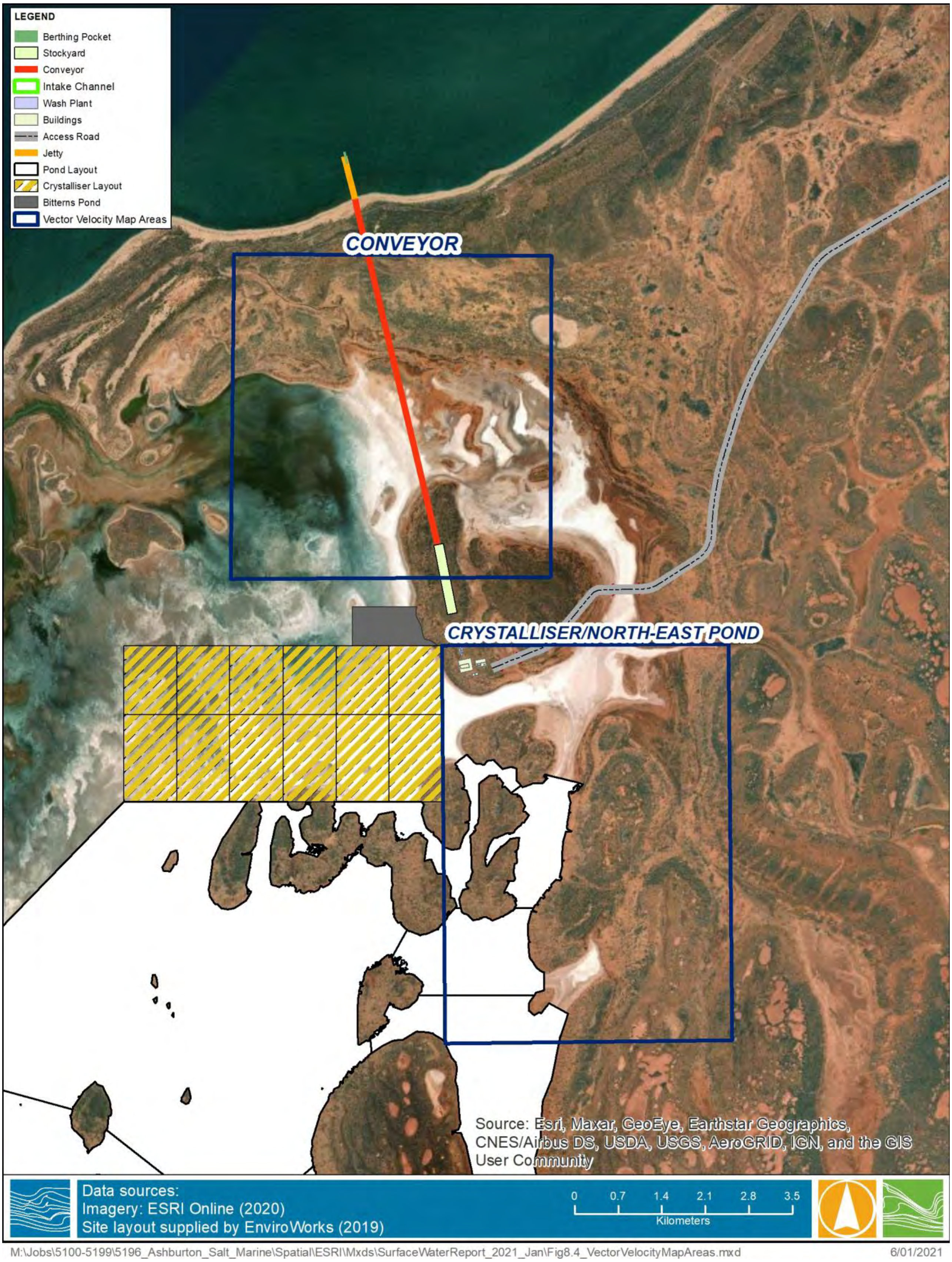

FIGURE 10-4 MODEL VECTOR VELOCITY LOCATIONS

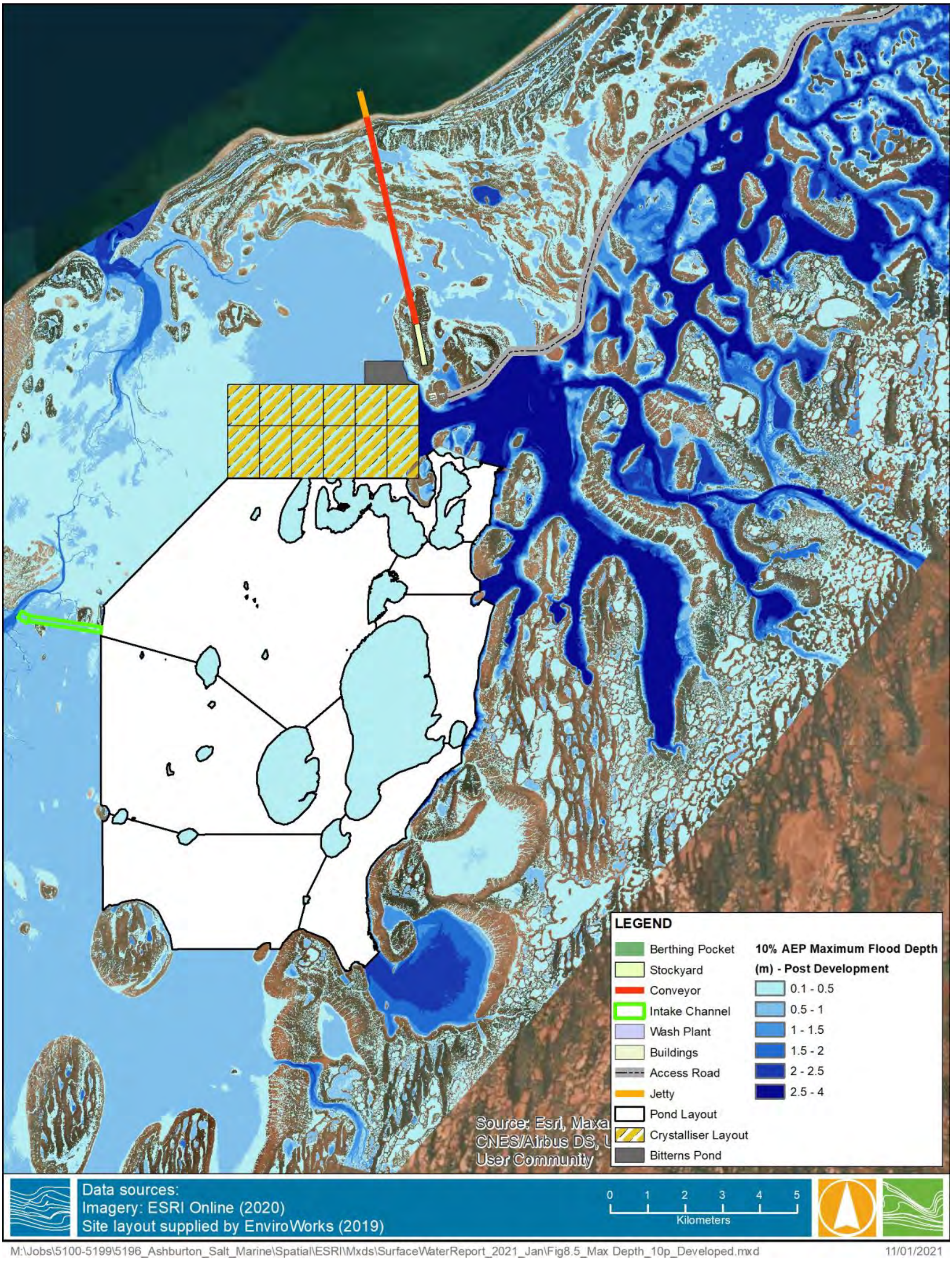

FIGURE 10-5 MODELLED 10% AEP MAXIMUM FLOOD DEPTH (M) – POST-DEVELOPMENT (UNMITIGATED)

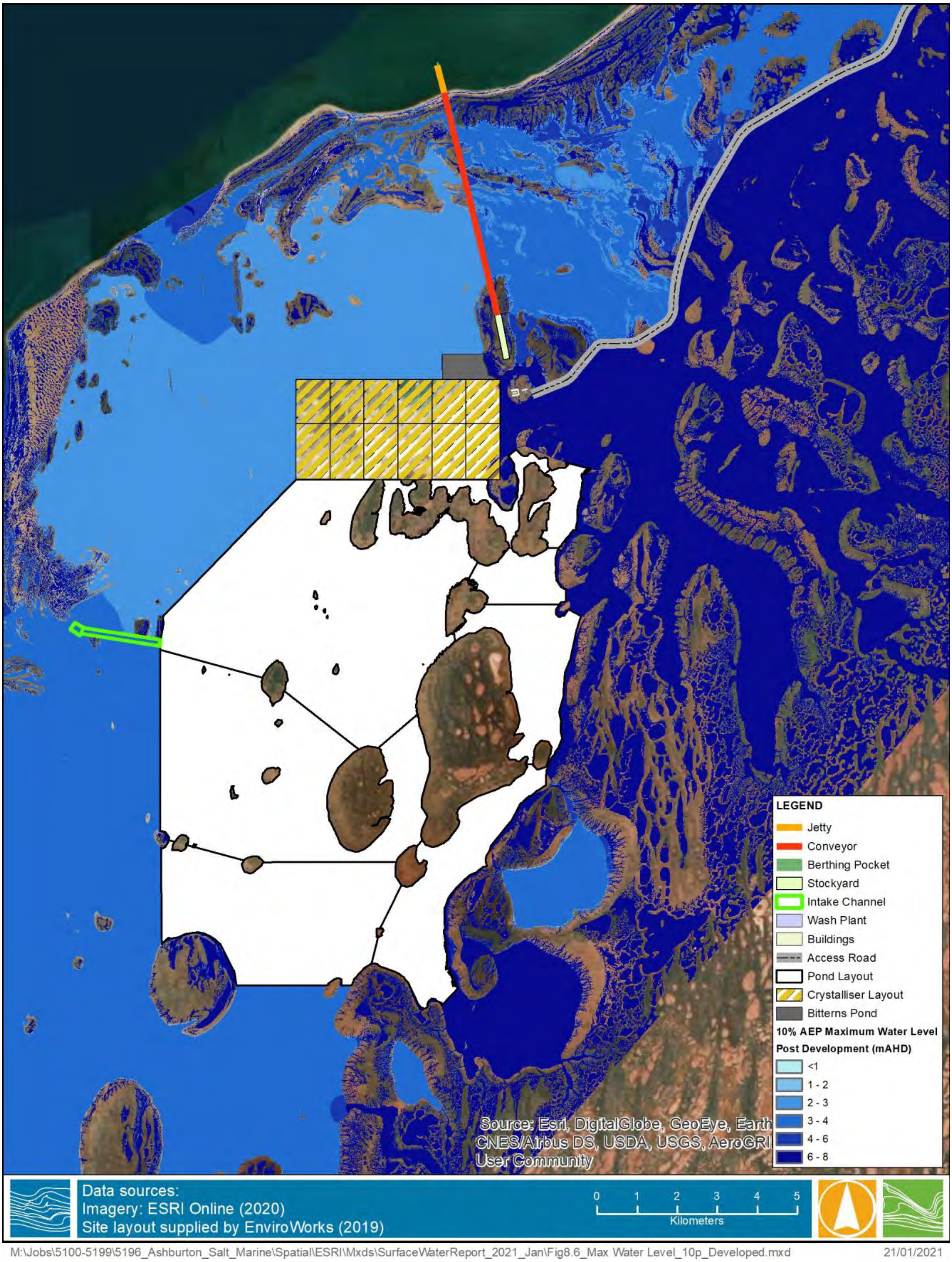

FIGURE 10-6 MODELLED 10% AEP MAXIMUM WATER LEVEL (M AHD) – POST DEVELOPMENT (UNMITIGATED)

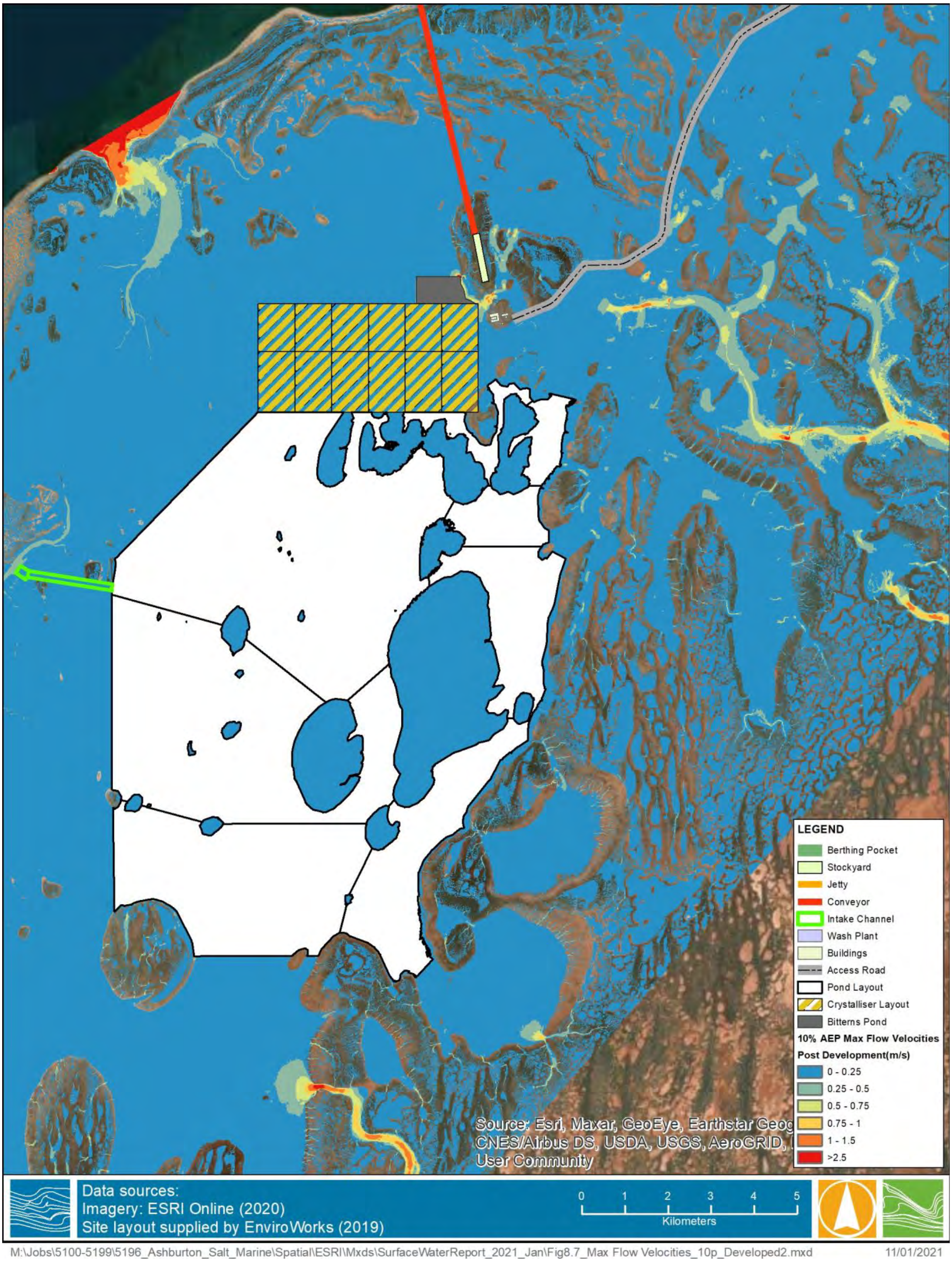

FIGURE 10-7 10% AEP MODELLED MAXIMUM FLOW VELOCITIES (M/S) – POST DEVELOPMENT (UNMITIGATED)

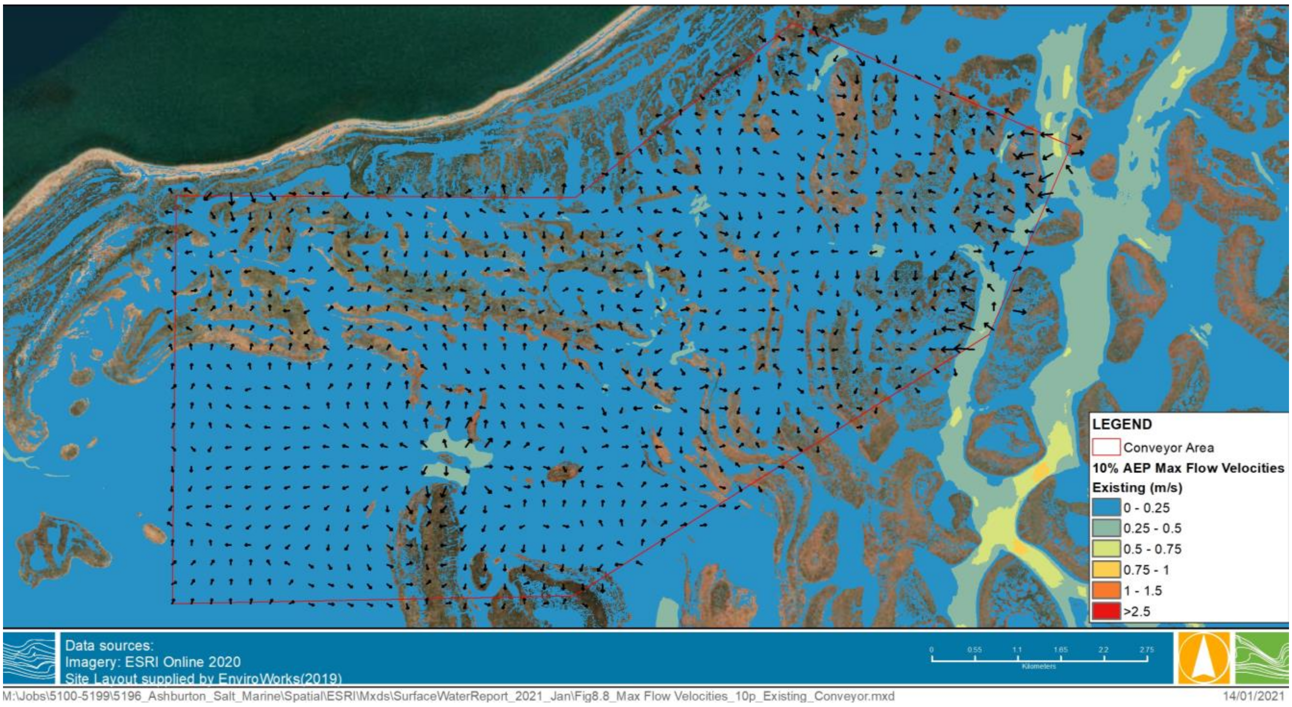

FIGURE 10-8 MODELLED 10% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CONVEYOR LOCATION – EXISTING CONDITIONS

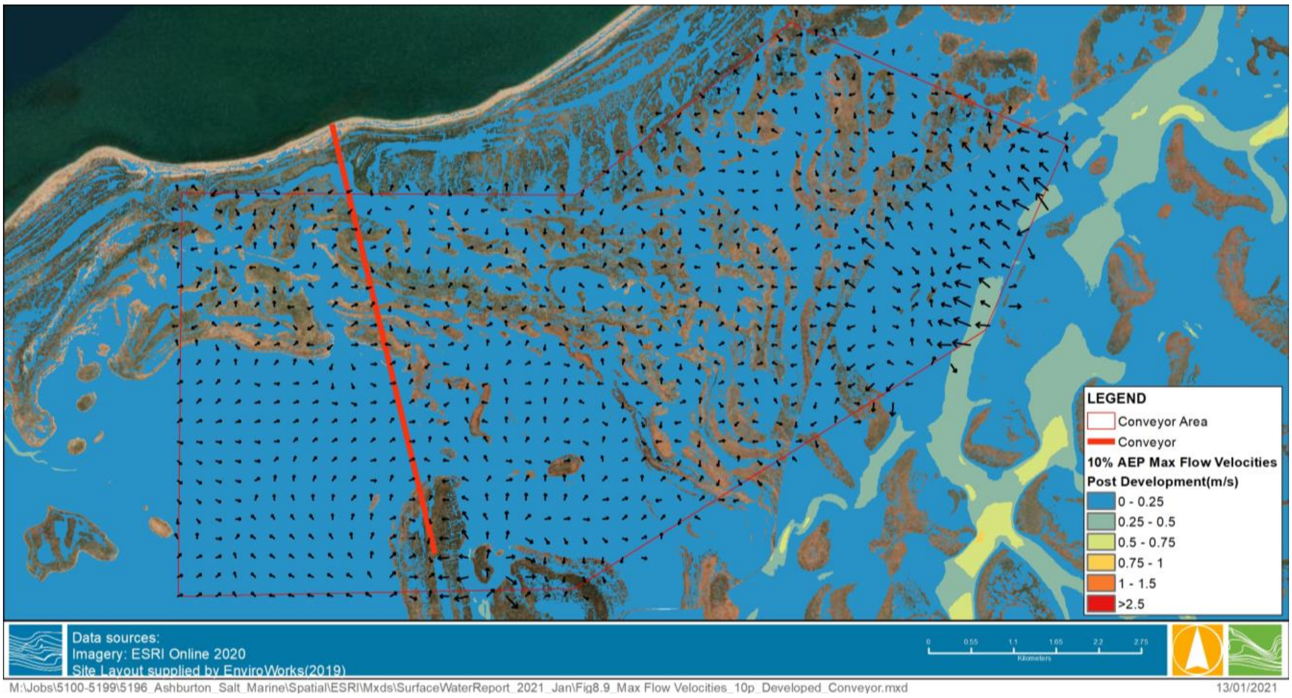

FIGURE 10-9 MODELLED 10% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CONVEYOR LOCATION – POST DEVELOPMENT (UNMITIGATED)

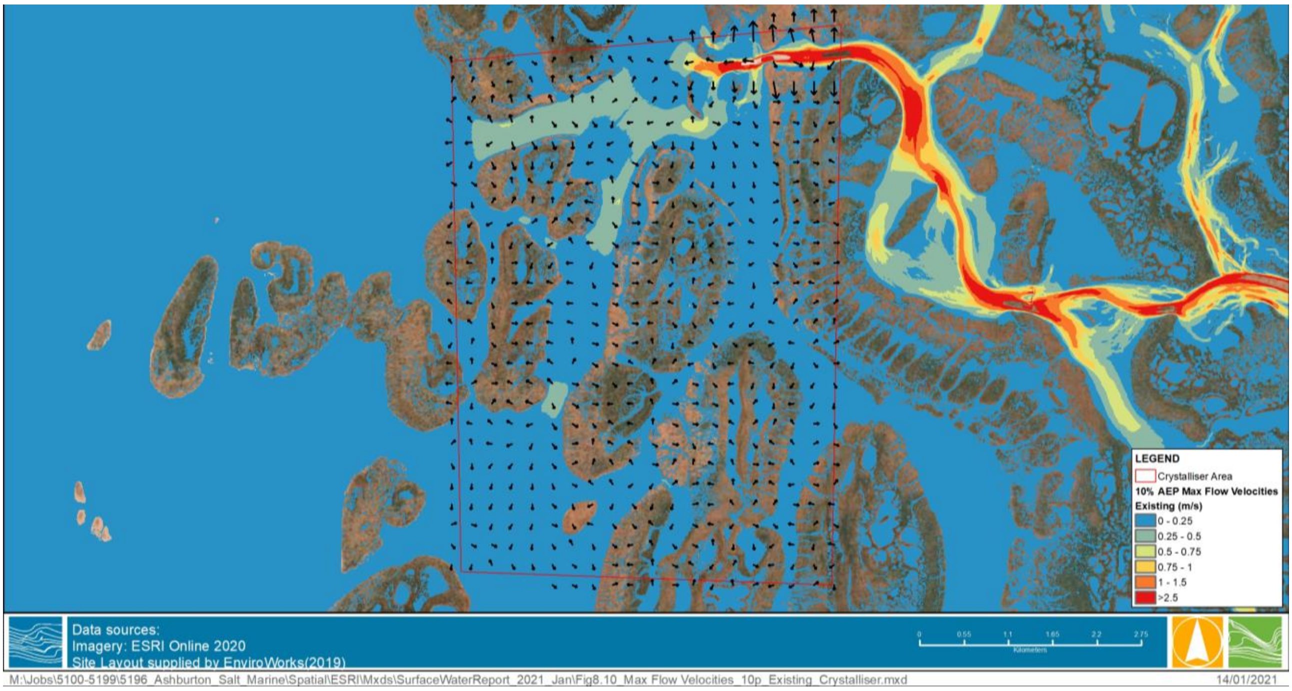

FIGURE 10-10 MODELLED 10% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CRYSTALLISER LOCATION – EXISTING CONDITIONS

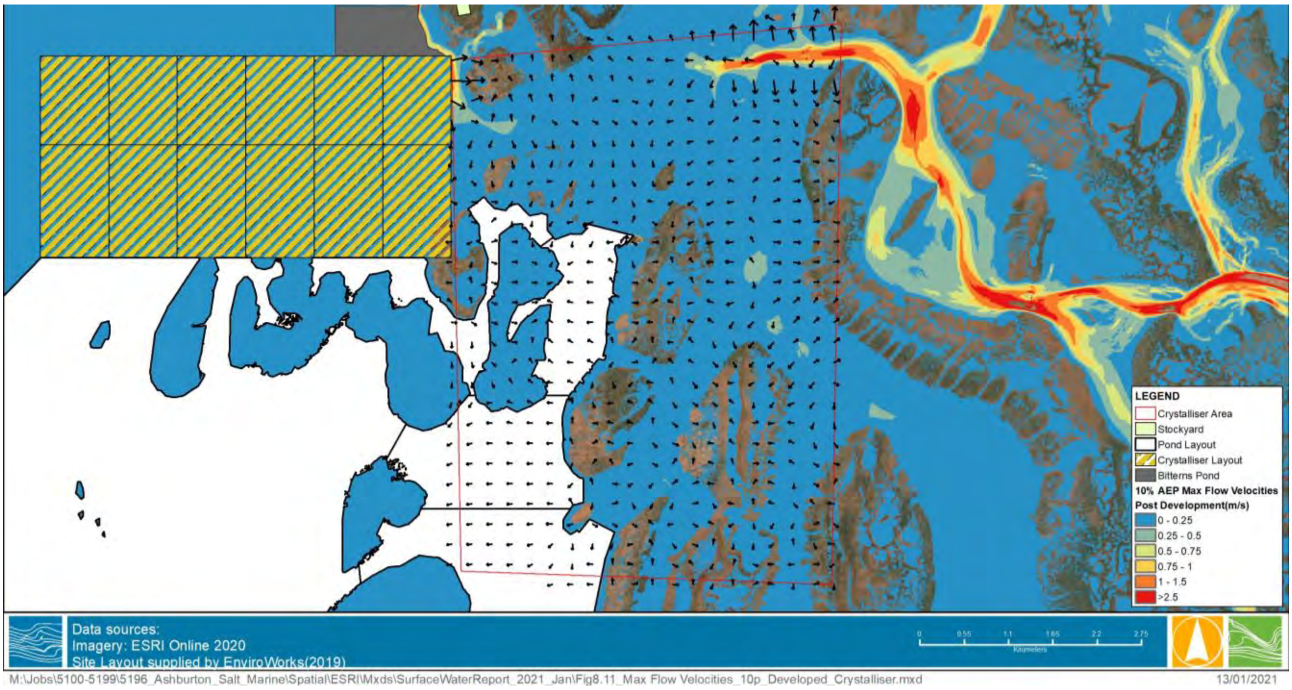

FIGURE 10-11 MODELLED 10% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CRYSTALLISER LOCATION – POST-DEVELOPMENT (UNMITIGATED)

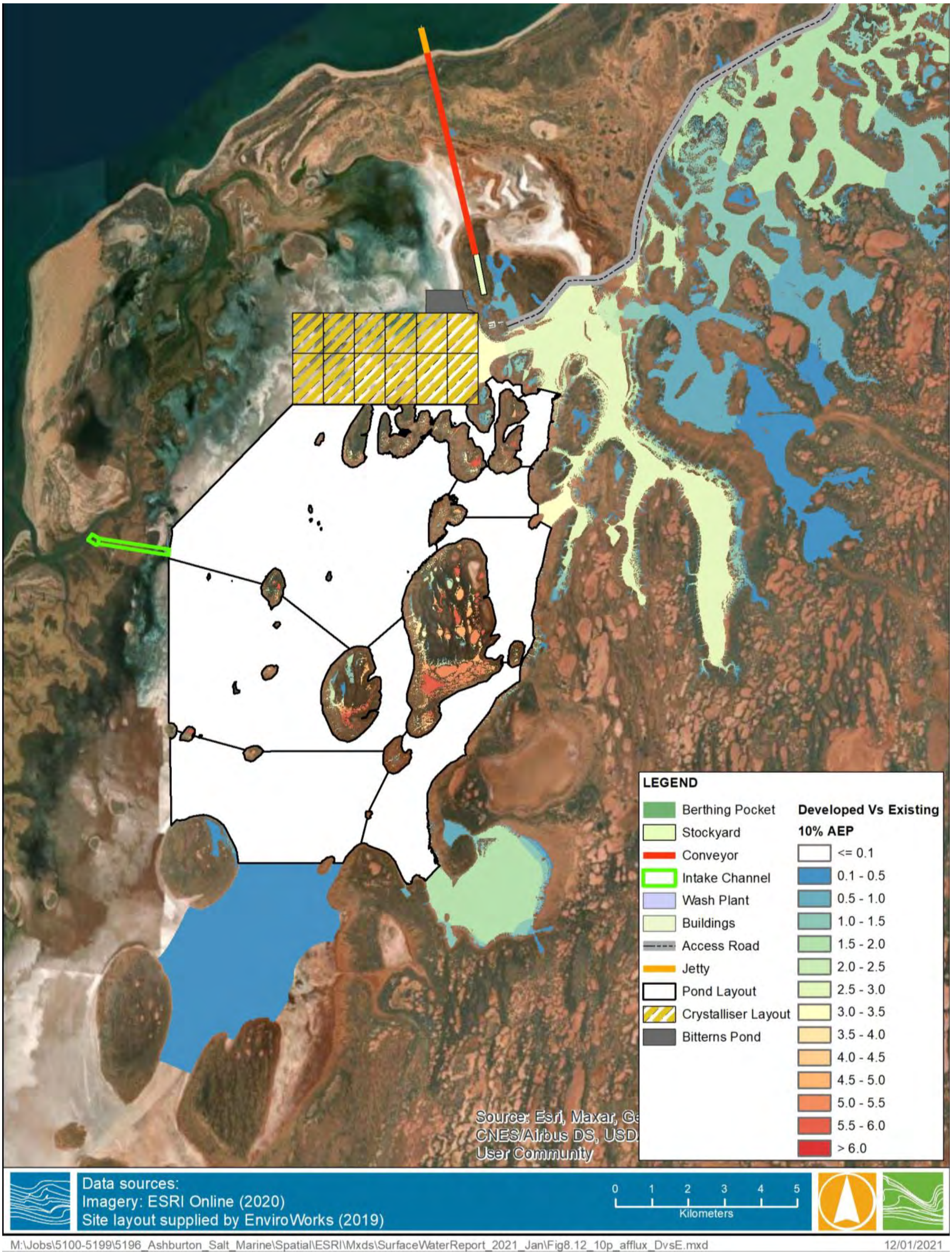

FIGURE 10-12 MODELLED 10% AEP AFFLUX (M) – EXISTING VS DEVELOPED (UNMITIGATED)

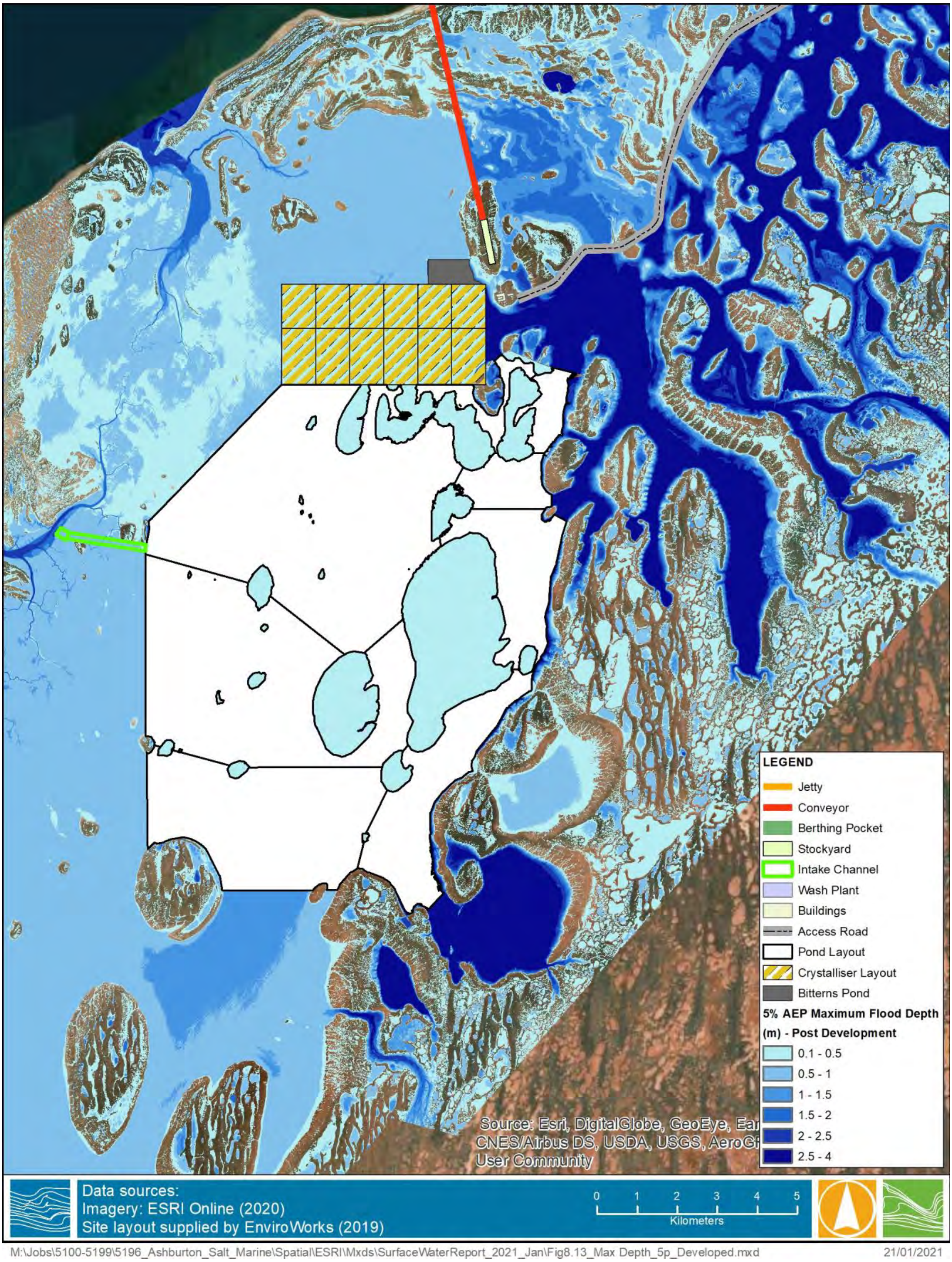

FIGURE 10-13 MODELLED 5% MAXIMUM FLOOD DEPTH (M) POST-DEVELOPMENT (UNMITIGATED)

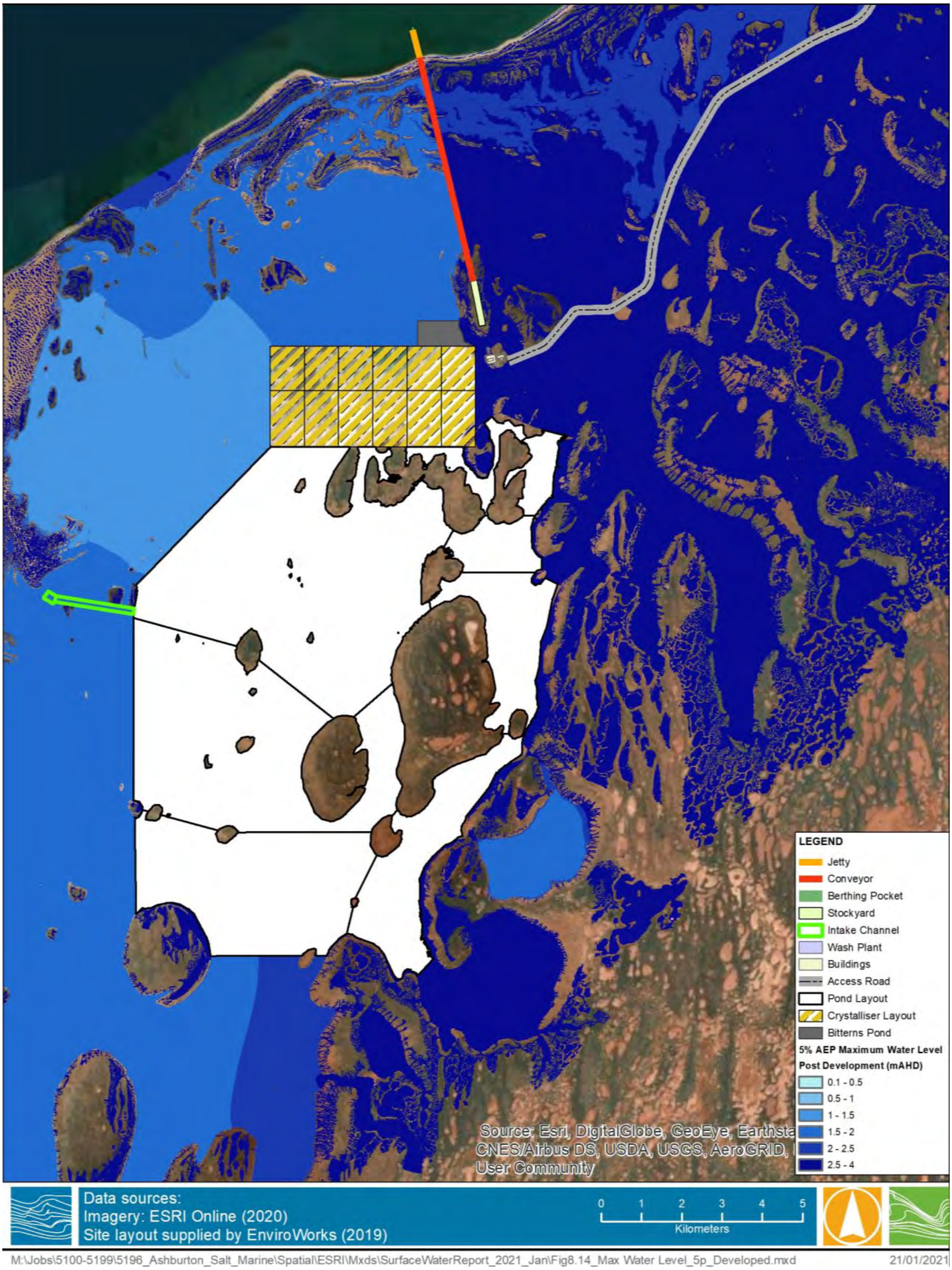

FIGURE 10-14 MODELLED 5% AEP MAXIMUM WATER LEVEL (M AHD) – POST DEVELOPMENT (UNMITIGATED)

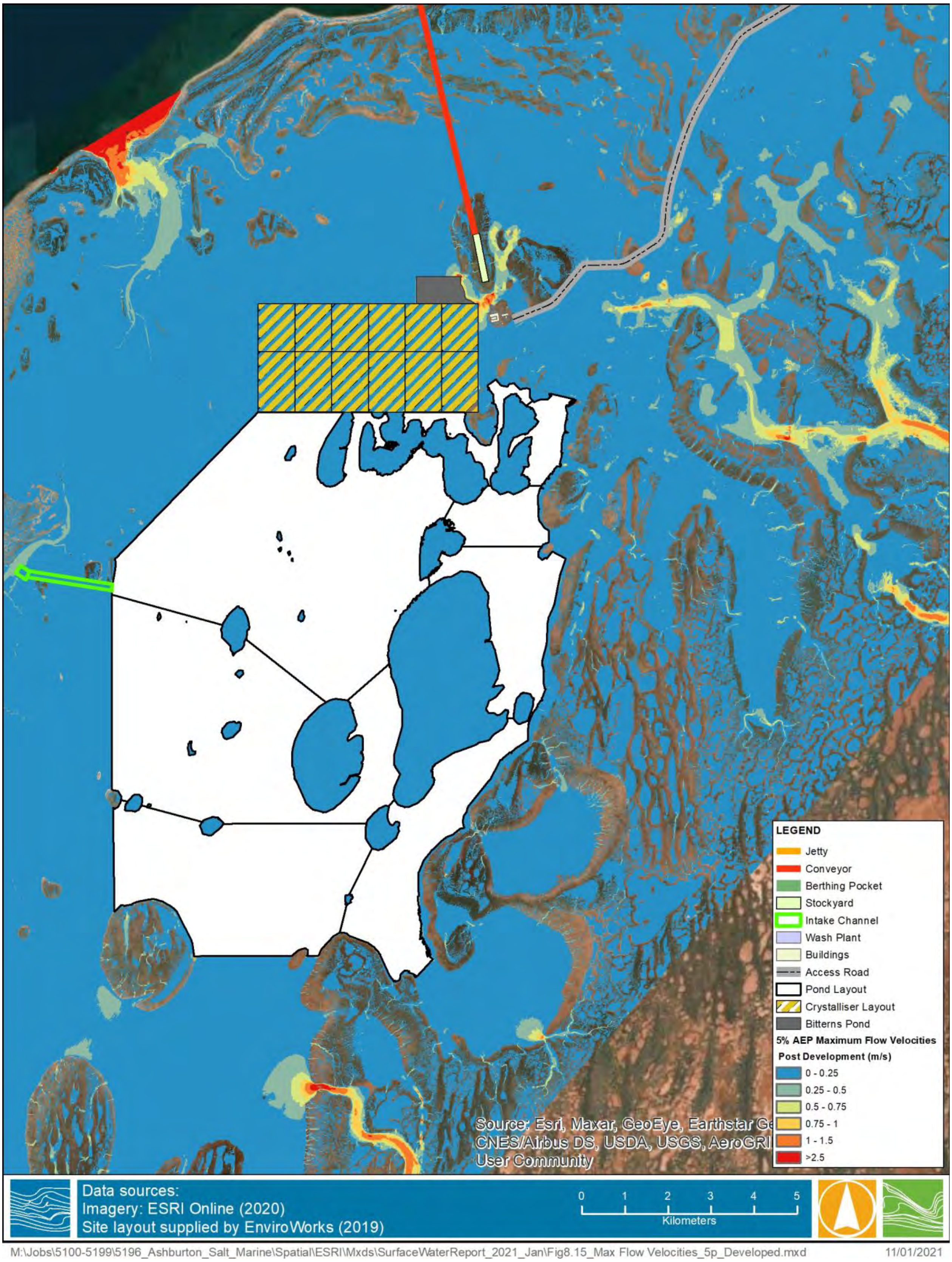

FIGURE 10-15 MODELLED 5% AEP MAXIMUM FLOW VELOCITIES (M/S) – POST DEVELOPMENT (UNMITIGATED)

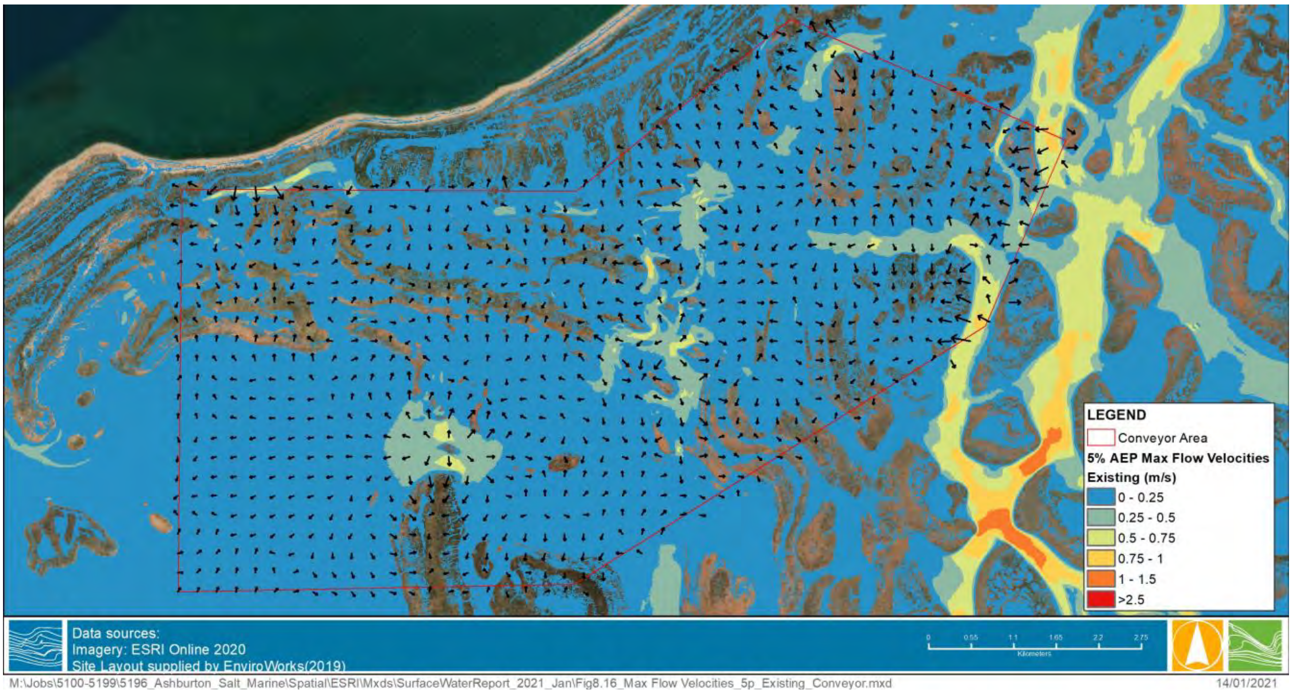

FIGURE 10-16 MODELLED 5% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CONVEYOR LOCATION – EXISTING CONDITIONS

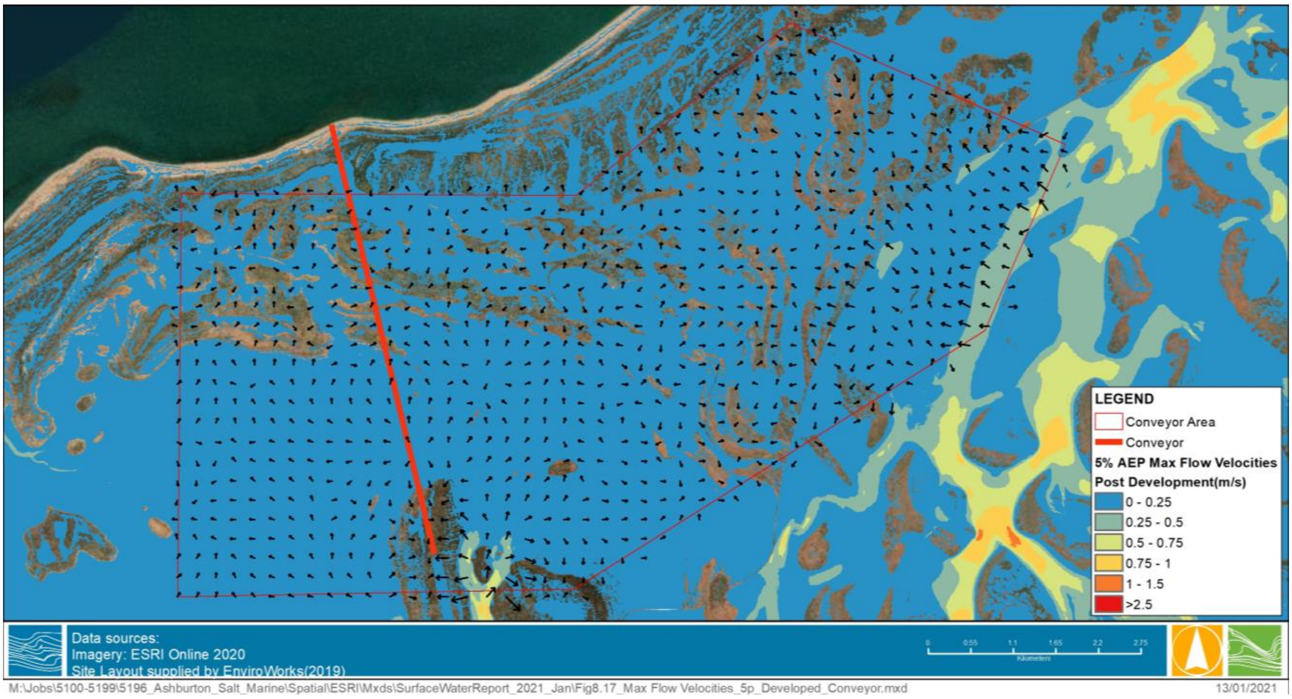

FIGURE 10-17 MODELLED 5% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CONVEYOR LOCATION – POST DEVELOPMENT (UNMITIGATED)

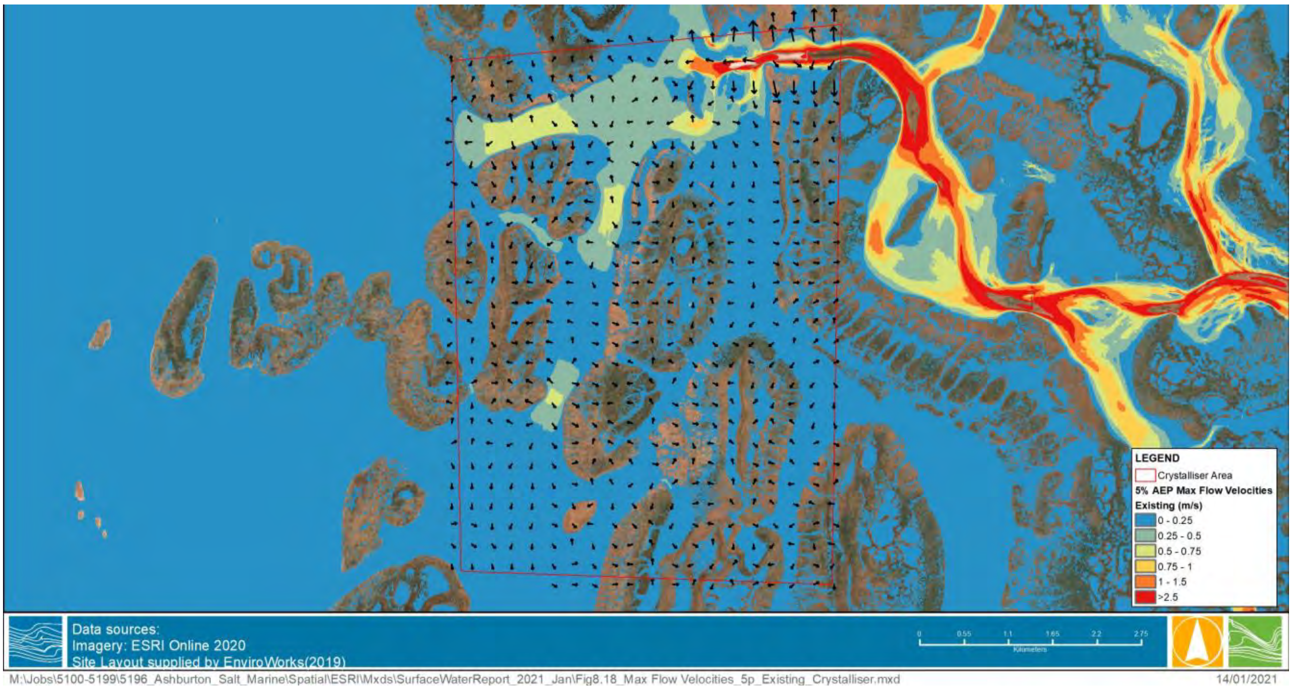

FIGURE 10-18 MODELLED 5% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CRYSTALLISER LOCATION – EXISTING CONDITIONS

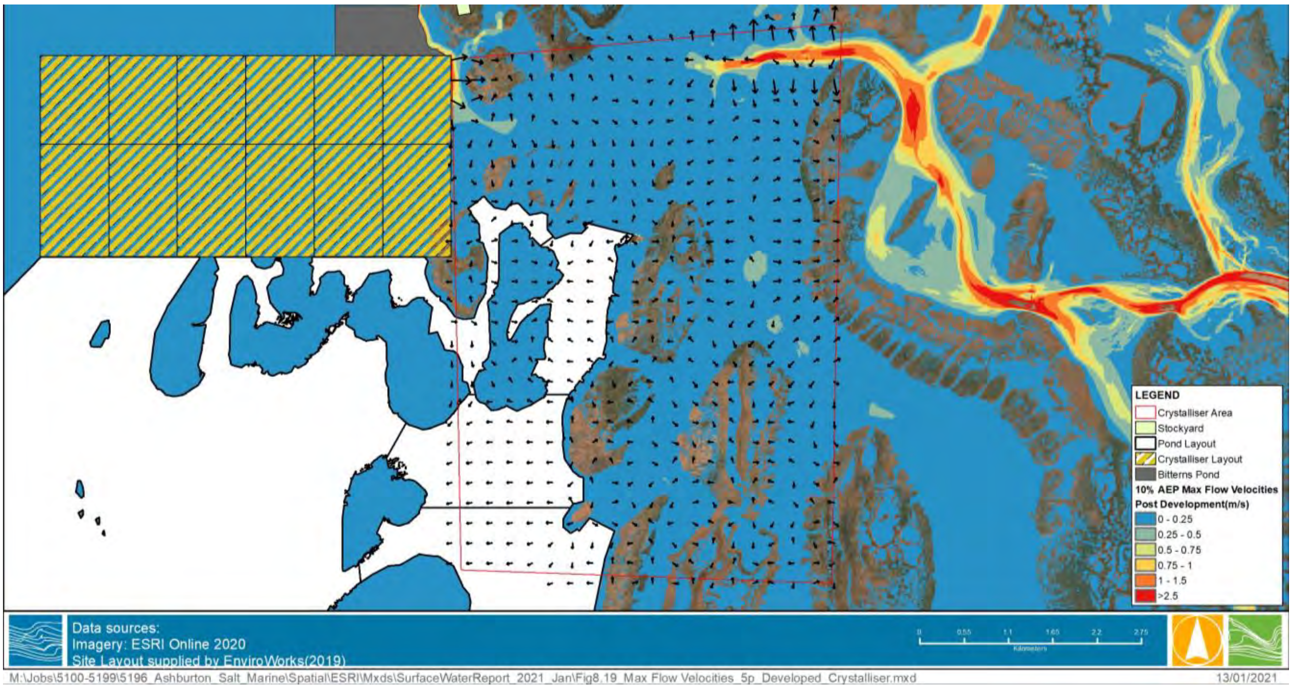

FIGURE 10-19 MODELLED 5% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CRYSTALLISER LOCATION – POST DEVELOPMENT (UNMITIGATED)

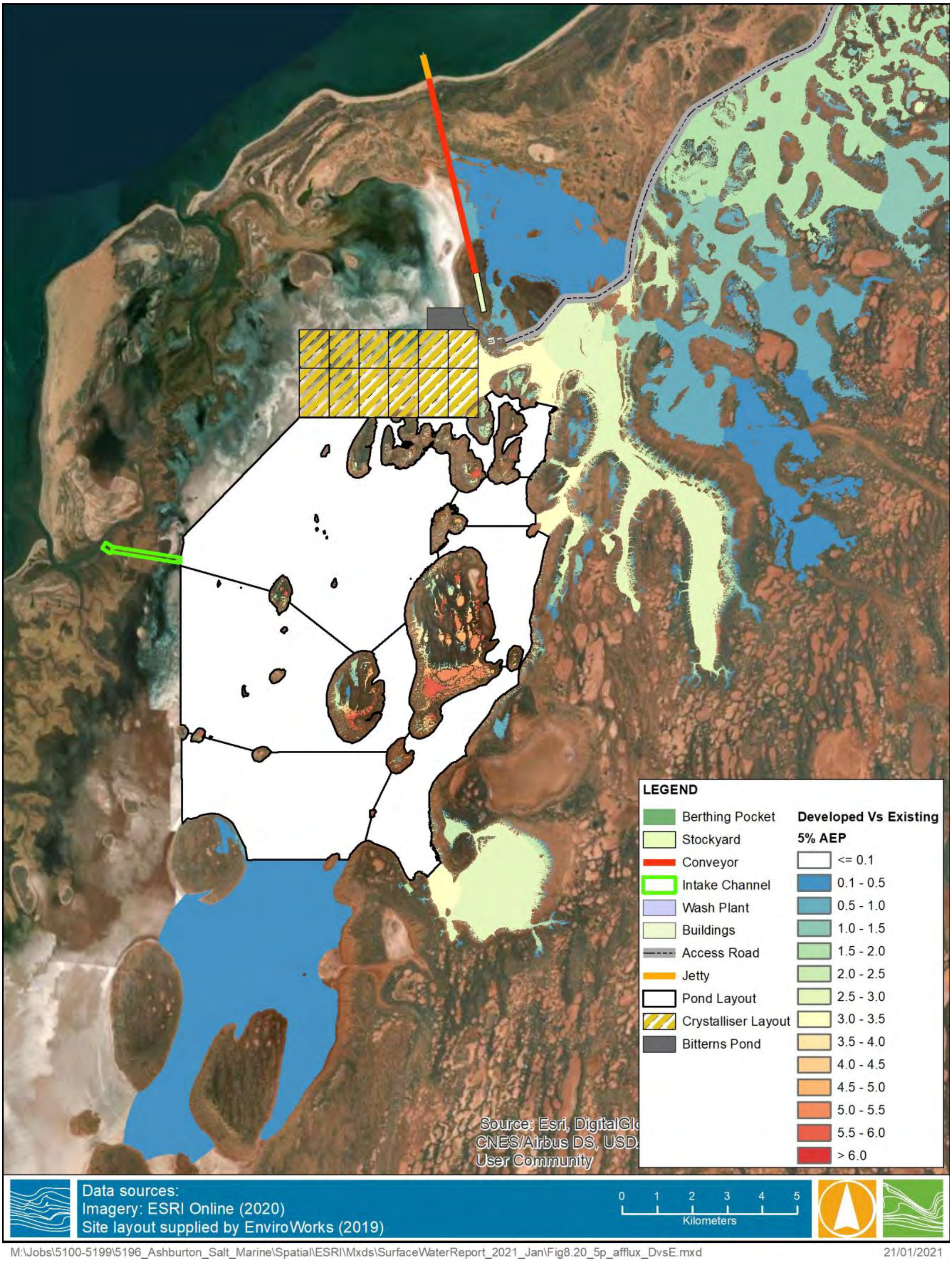

FIGURE 10-20 MODELELD 5% AEP AFFLUX (M) – EXISTING VS DEVELOPED (UNMITIGATED)

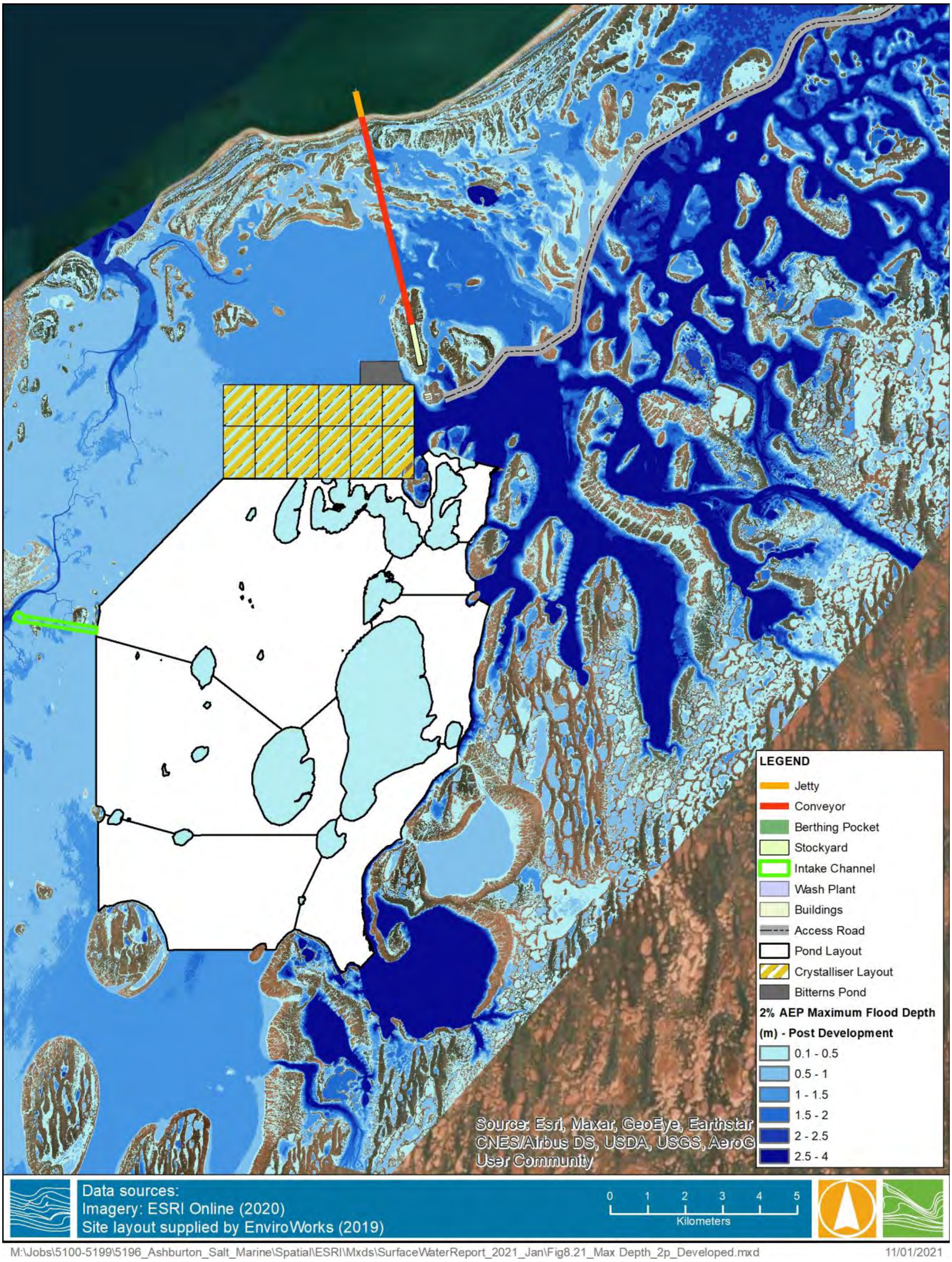

FIGURE 10-21 MODELLED 2% AEP MAXIMUM FLOOD DEPTH (M) – POST-DEVELOPMENT (UNMITIGATED)

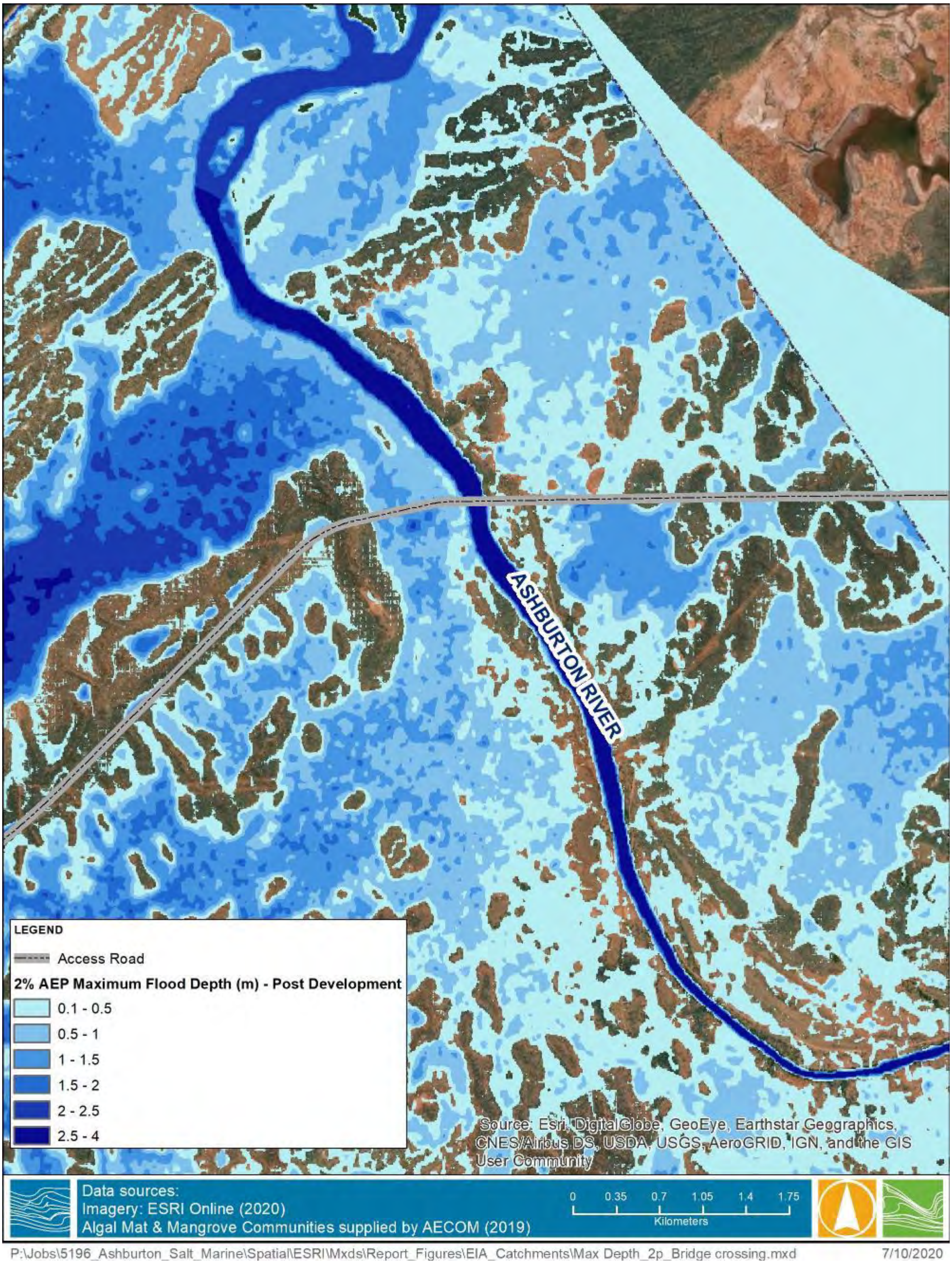

FIGURE 10-22 MODELLED 2% AEP MAXIMUM FLOOD DEPTH AT PROPOSED BRIDGE CROSSING

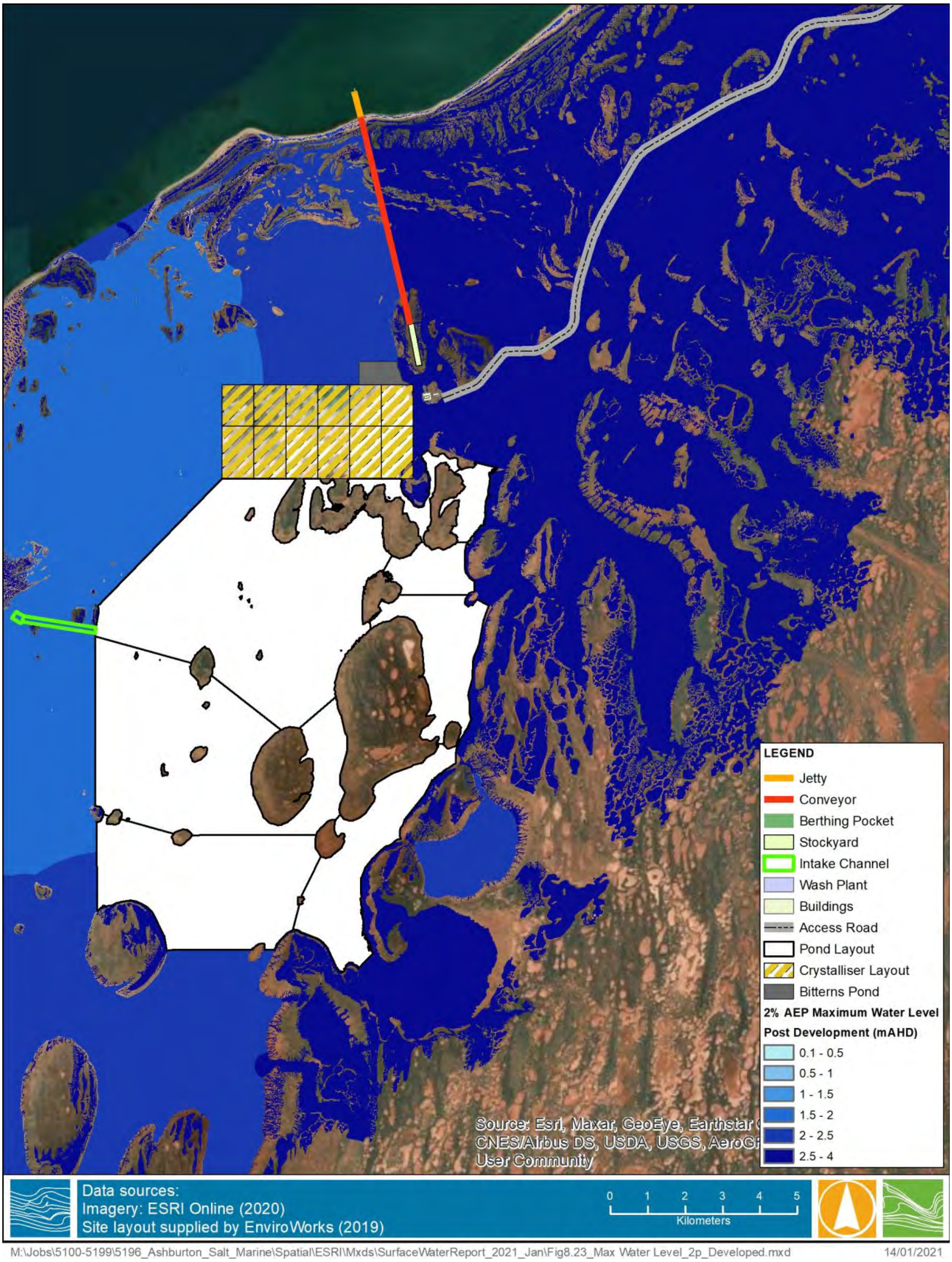

FIGURE 10-23 MODELLED 2% AEP MAXIMUM WATER LEVEL (M AHD) – POST DEVELOPMENT (UNMITIGATED)

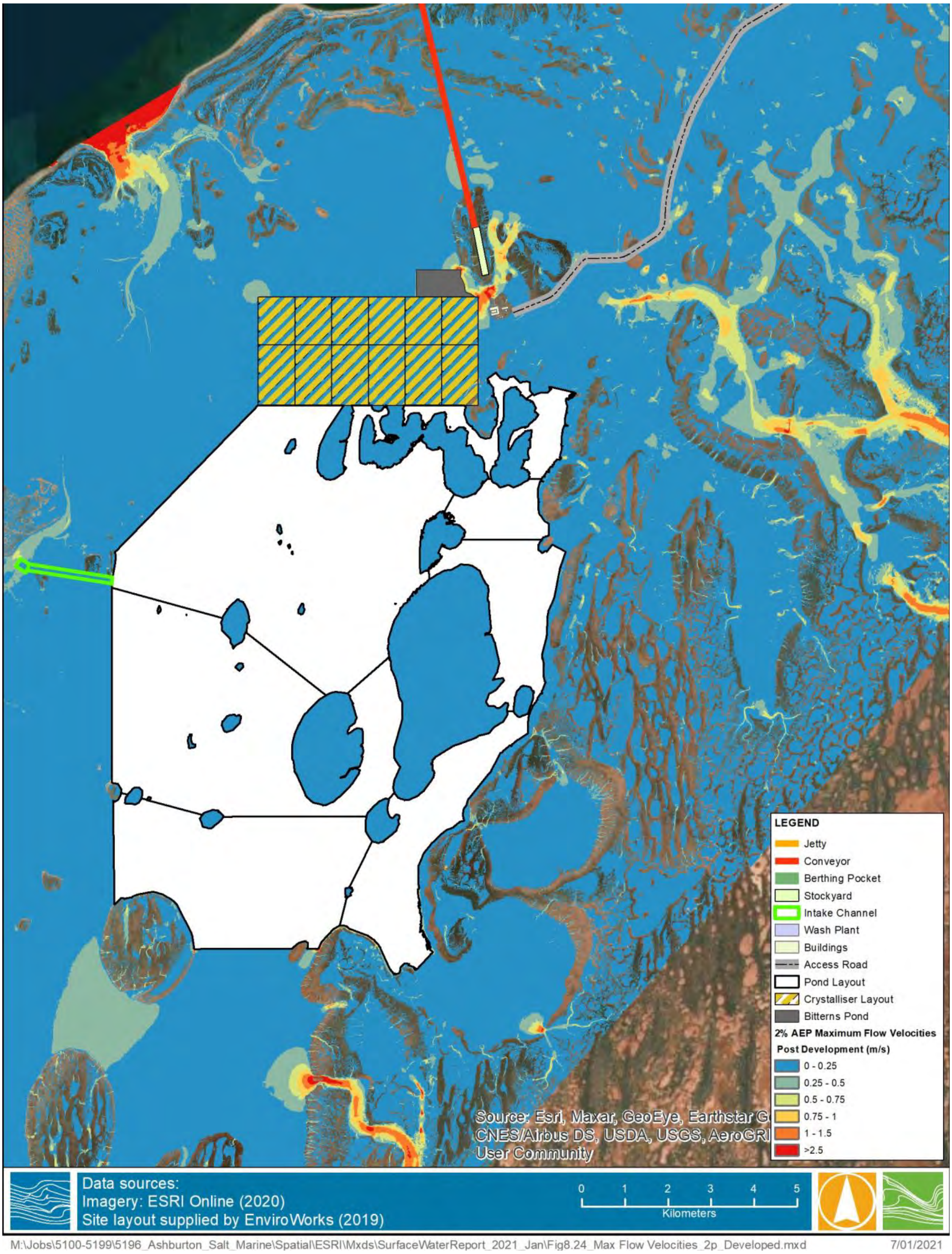

FIGURE 10-24 MODELLED 2% AEP MAXIMUM FLOW VELOCITIES (M/S) – POST DEVELOPMENT (UNMITIGATED)

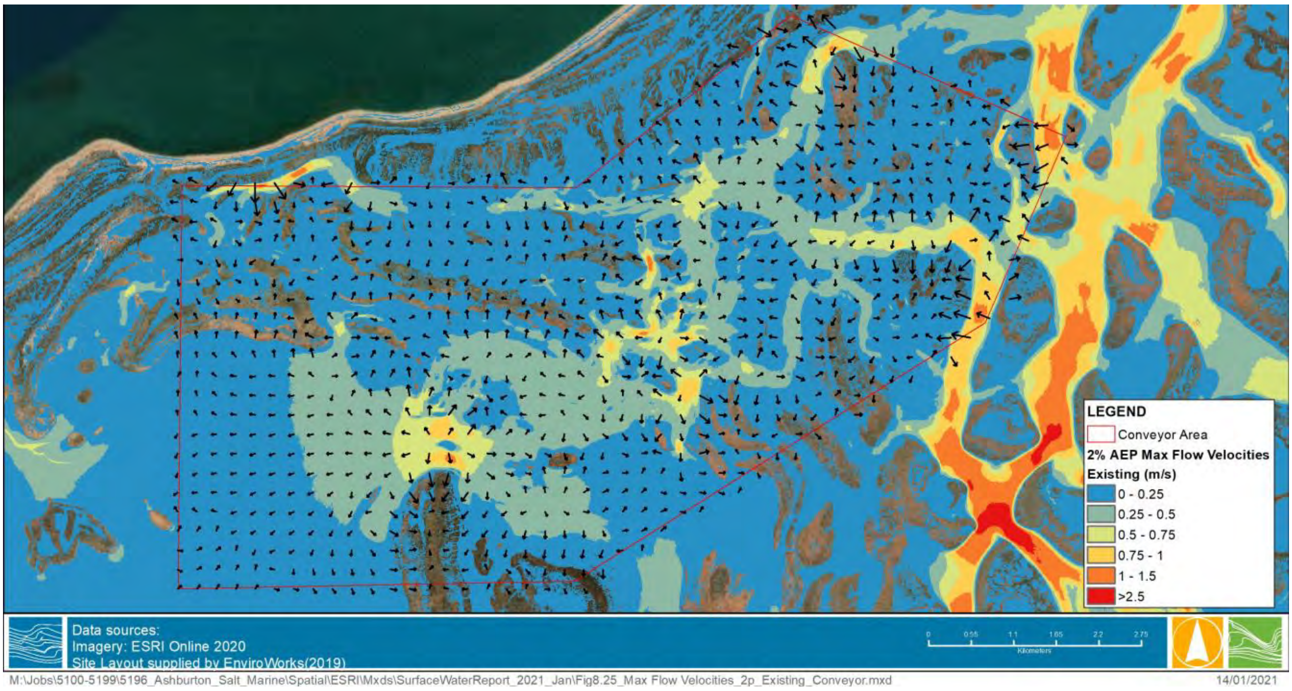

FIGURE 10-25 MODELLED 2% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CONVEYOR LOCATION – EXISTING CONDITIONS

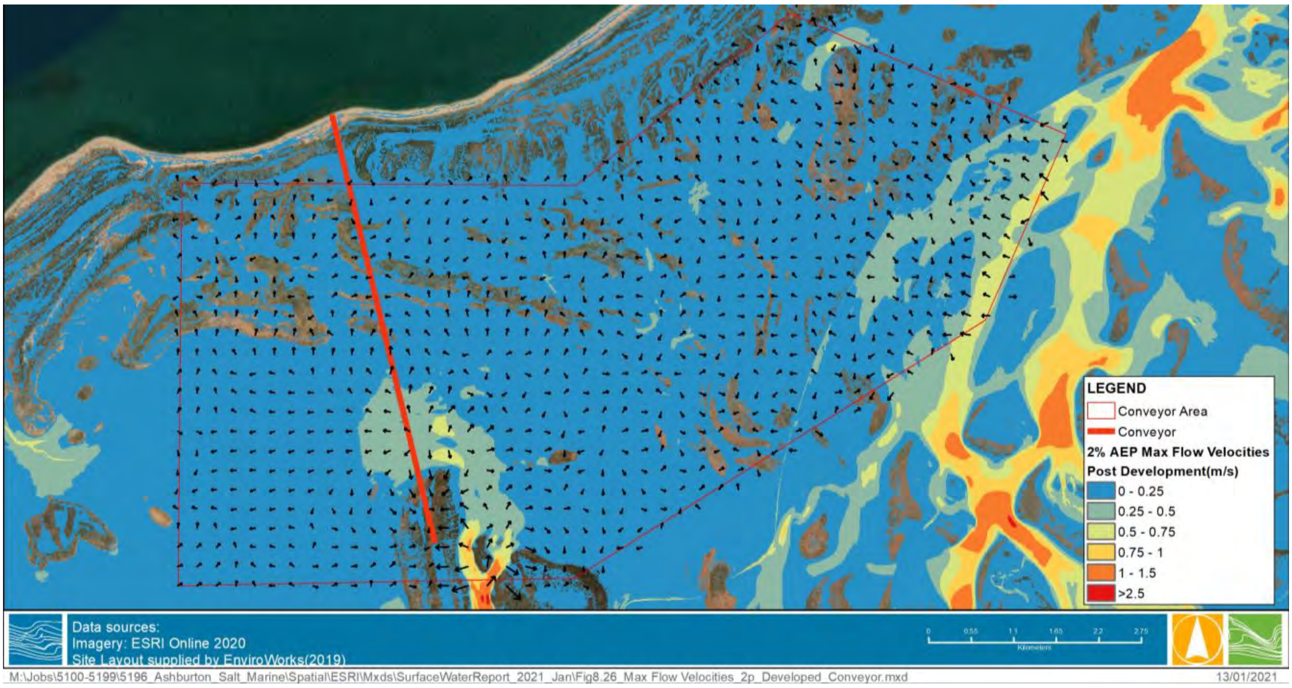

FIGURE 10-26 MODELLED 2% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CONVEYOR LOCATION – POST DEVELOPMENT (UNMITIGATED)

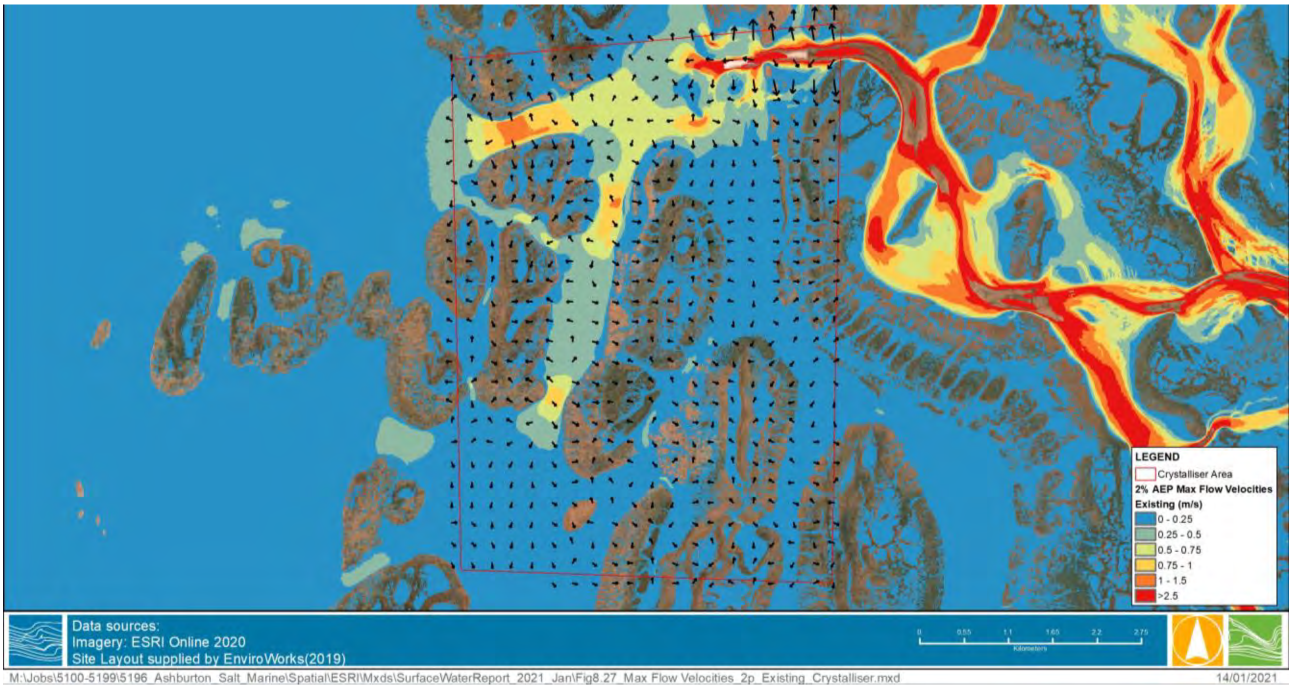

FIGURE 10-27 MODELLED 2% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CRYSTALLISER LOCATION – EXISTING CONDITIONS

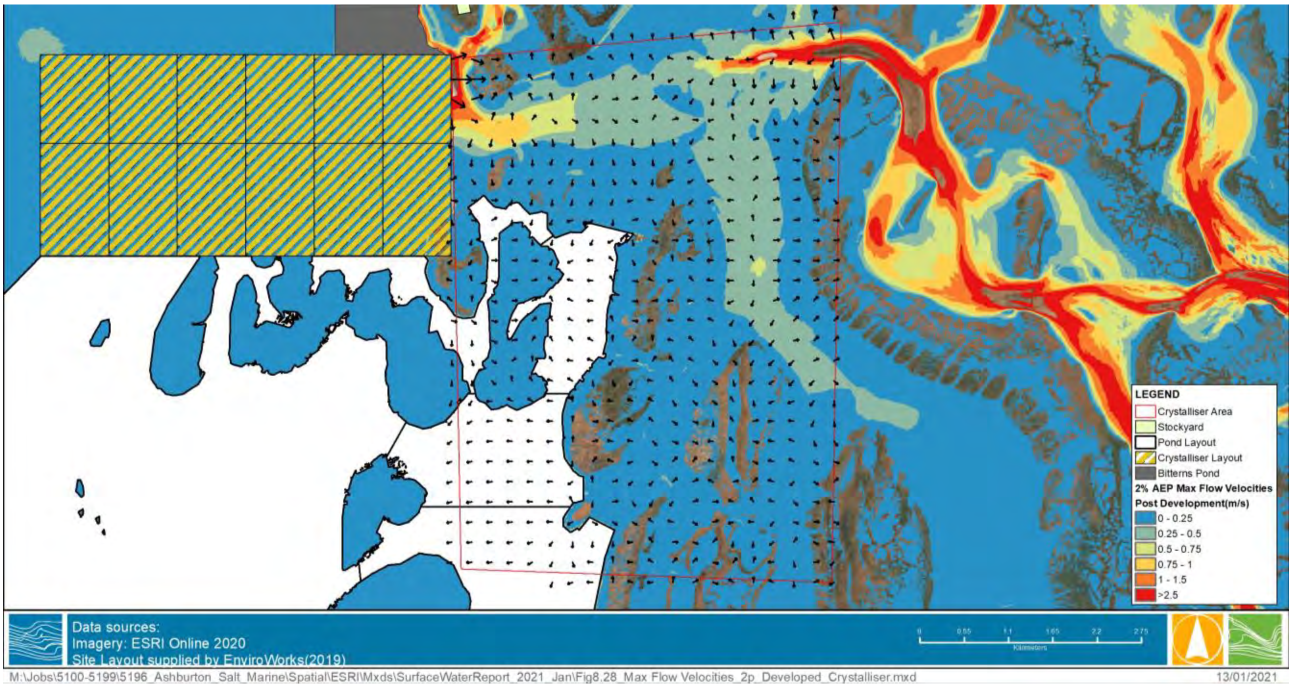

FIGURE 10-28 MODELLED 2% AEP MAXIMUM FLOW VELOCITIES (M/S) AT CRYSTALLISER LOCATION – POST DEVELOPMENT (UNMITIGATED)

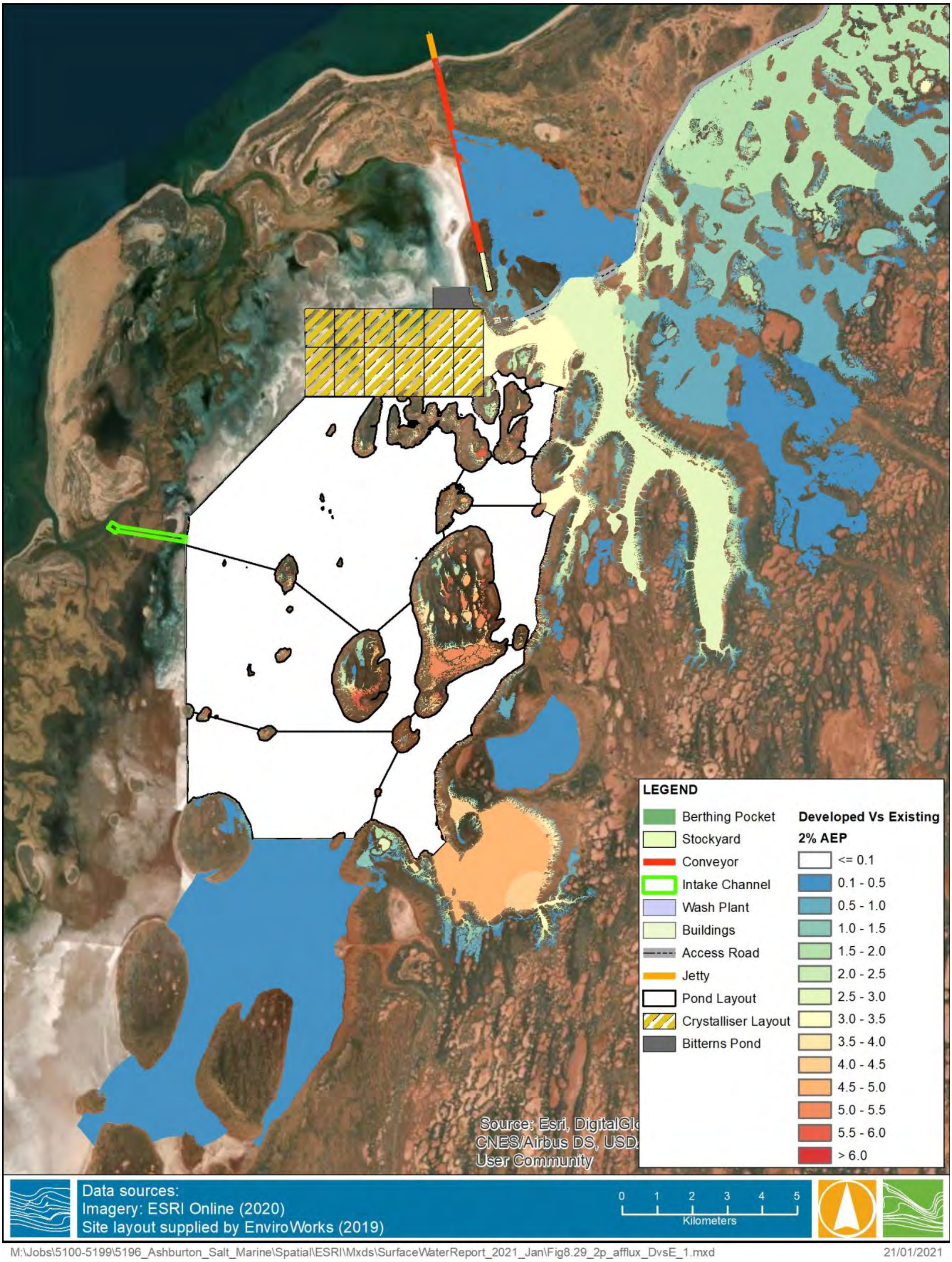

FIGURE 10-29 MODELLED 2% AEP AFFLUX (M) – EXISTING VS DEVLEOPED (UNMITIGATED)

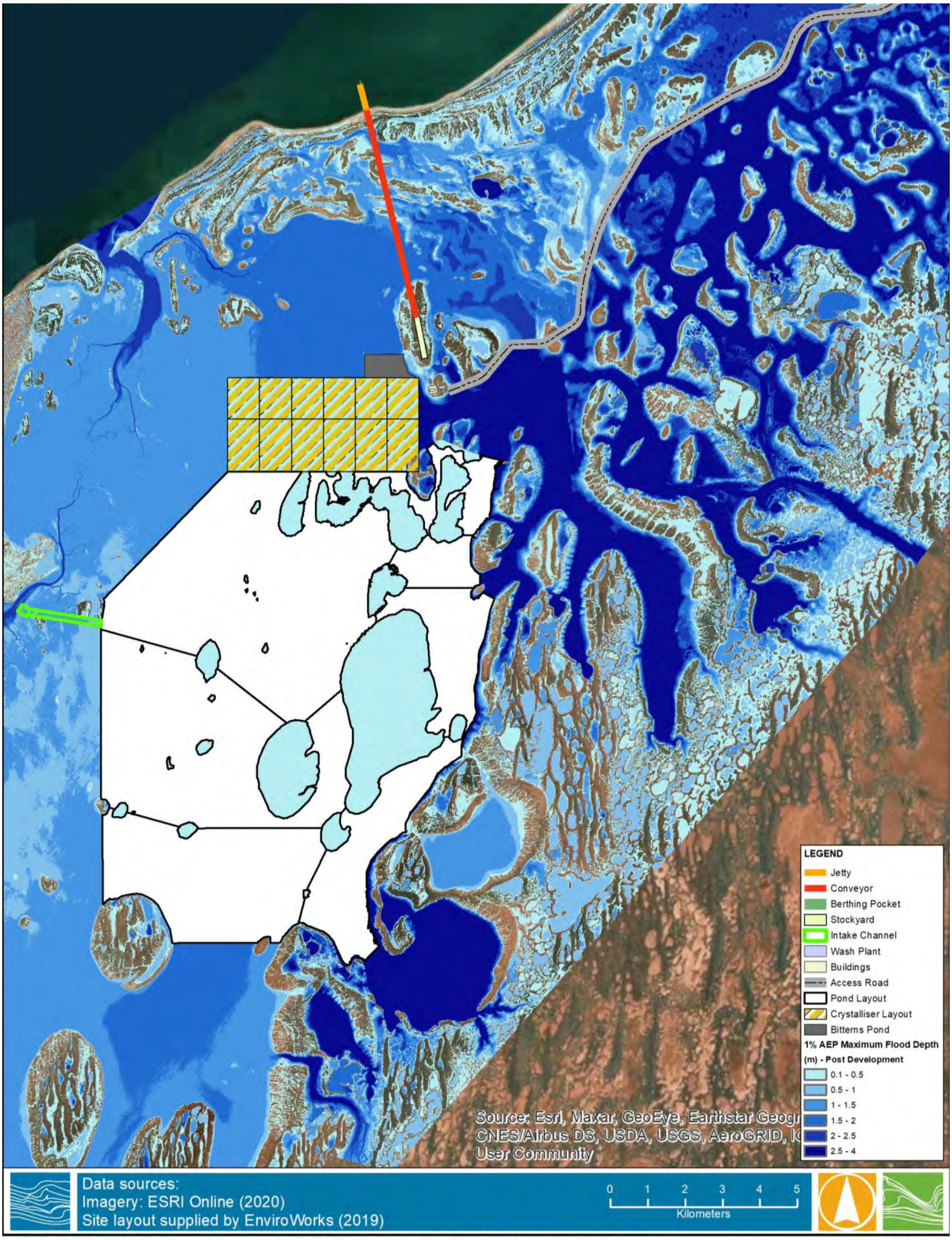

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FIGURE 10-30 MODELLED 1% AEP MAXIMUM FLOOD DEPTH (M) – POST DEVELOPMENT (UNMITIGATED)

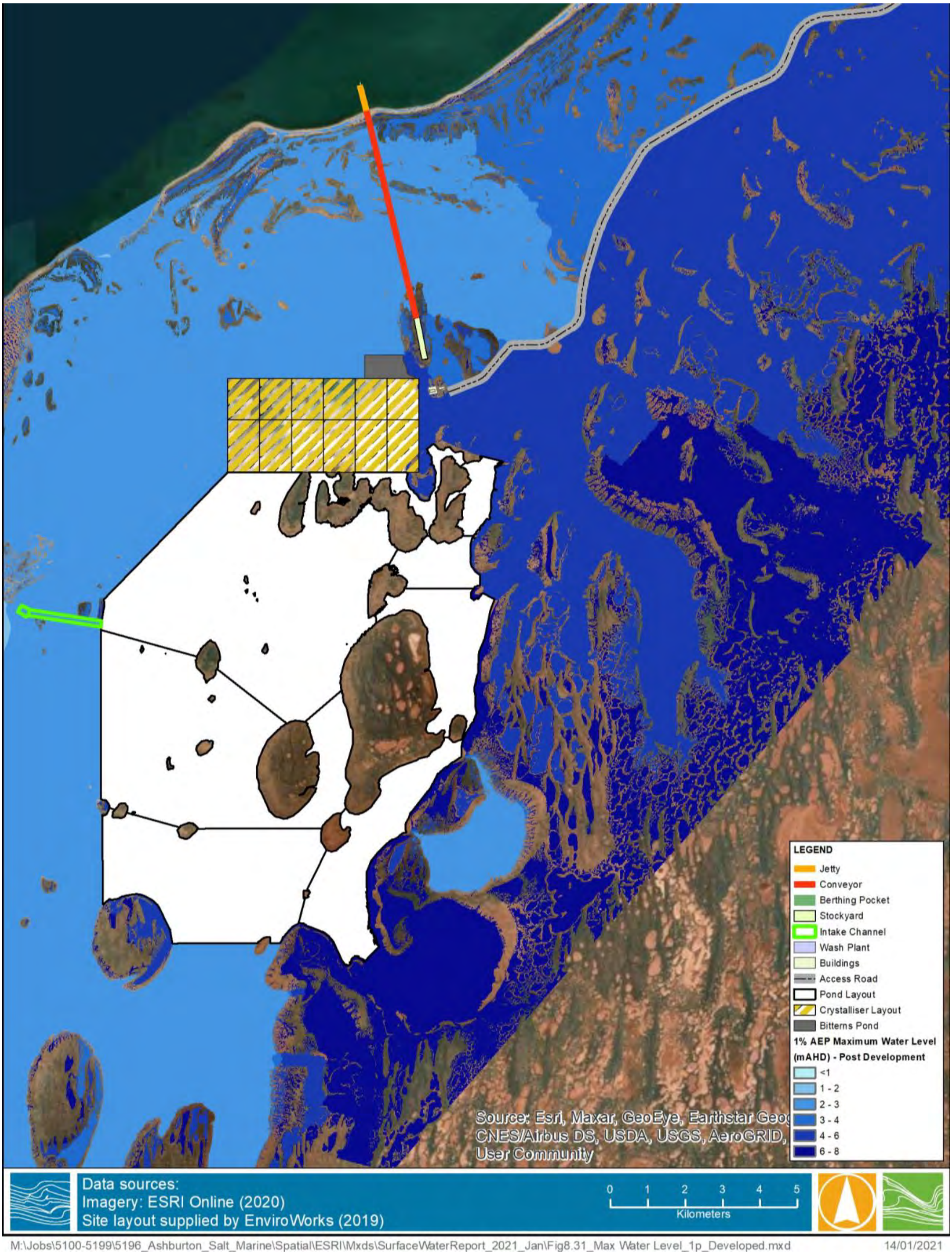

FIGURE 10-31 MODELLED 1% AEP MAXIMUM WATER LEVEL (M AHD) – POST DEVELOPMENT (UNMITIGATED)

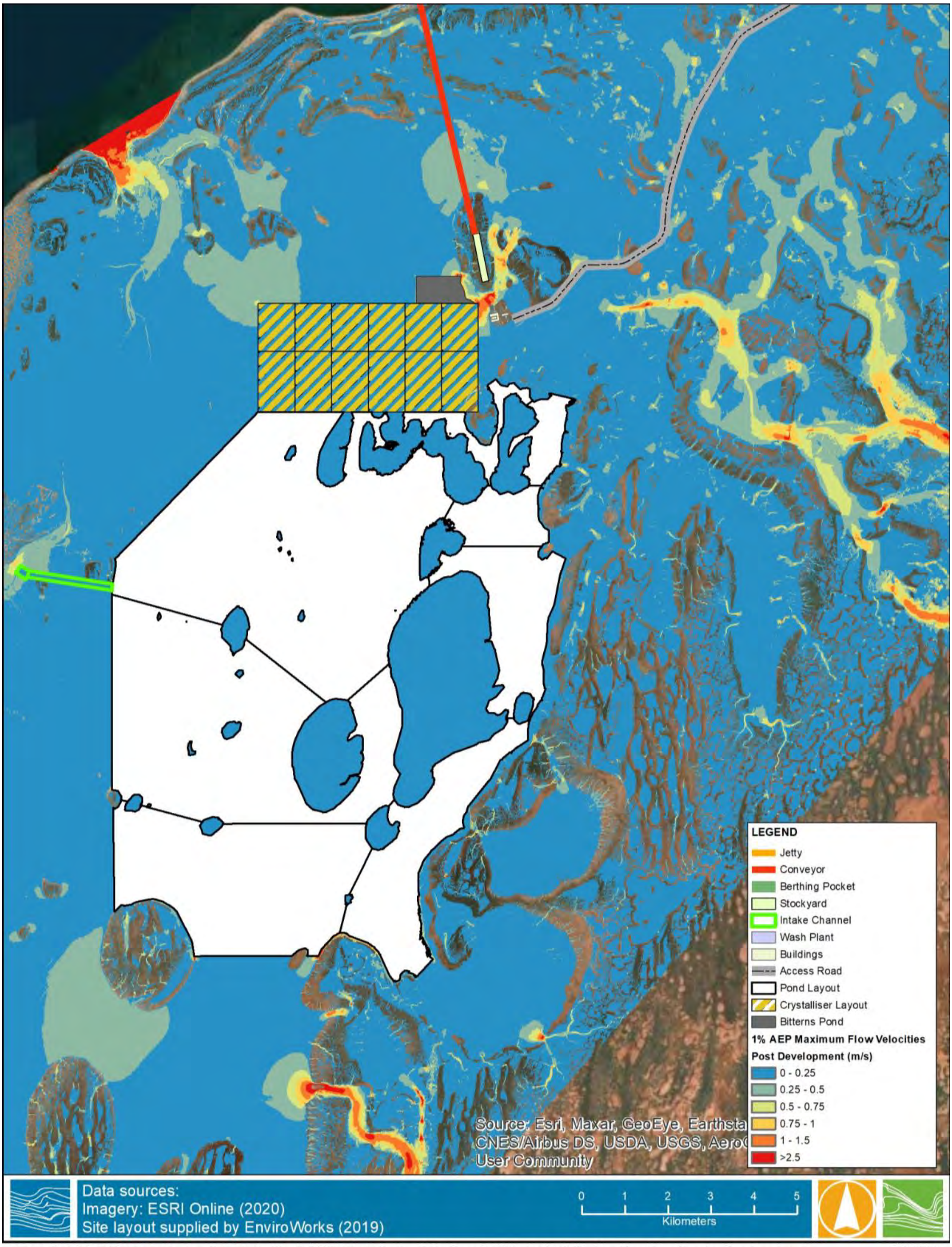

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FIGURE 10-32 MODELLED 1% AEP MAXIMUM FLOW VELOCITIES (M/S) – POST DEVELOPMENT (UNMITIGATED)

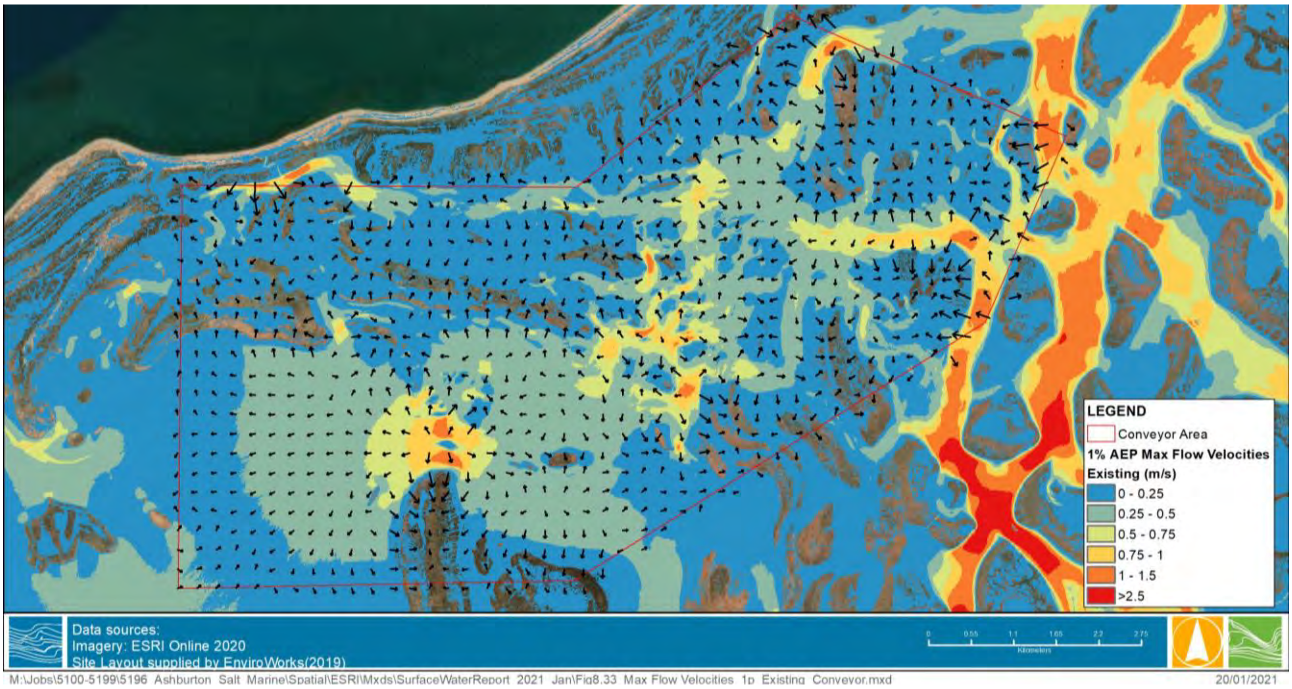

FIGURE 10-33 MODELLED 1% AEP MAXIMUM FLOW VELOCITIES AT CONVEYOR LOCATION – EXISTING CONDITIONS

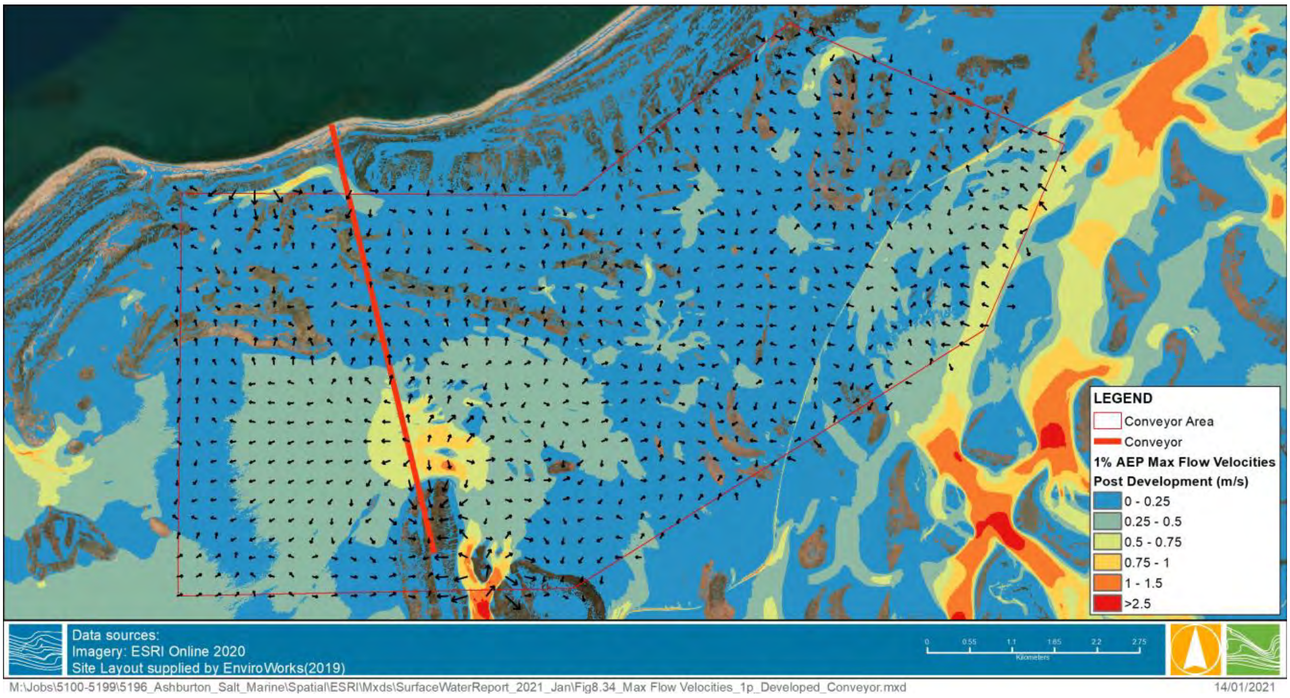

FIGURE 10-34 MODELLED 1% AEP MAXIMUM FLOW VELOCITIES AT CONVEYOR LOCATION – POST DEVELOPMENT (UNMITIGATED)

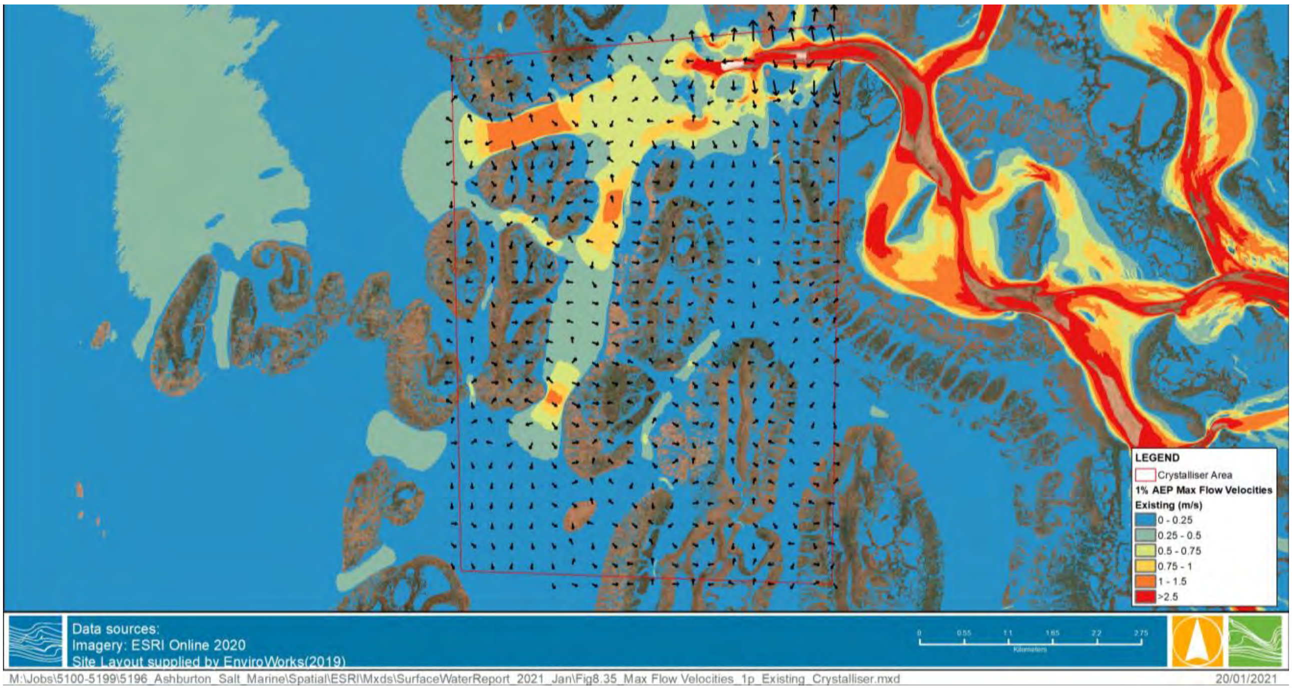

FIGURE 10-35 MODELLED 1% AEP MAXIMUM FLOW VELOCITIES AT CRYSTALLISER LOCATION – EXISTING CONDITIONS

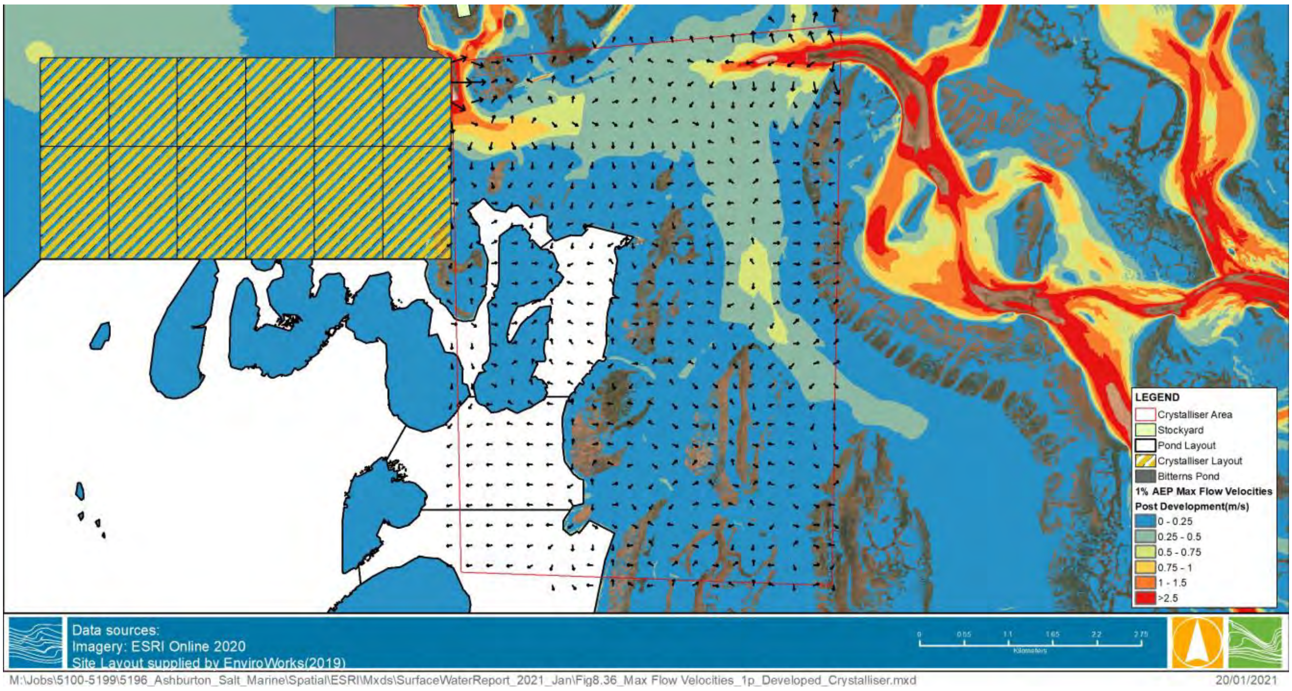

FIGURE 10-36 MODELLED 1% AEP MAXIMUM FLOW VELOCITIES AT CRYSTALLISER LOCATION – POST DEVELOPMENT (UNMITIGATED)

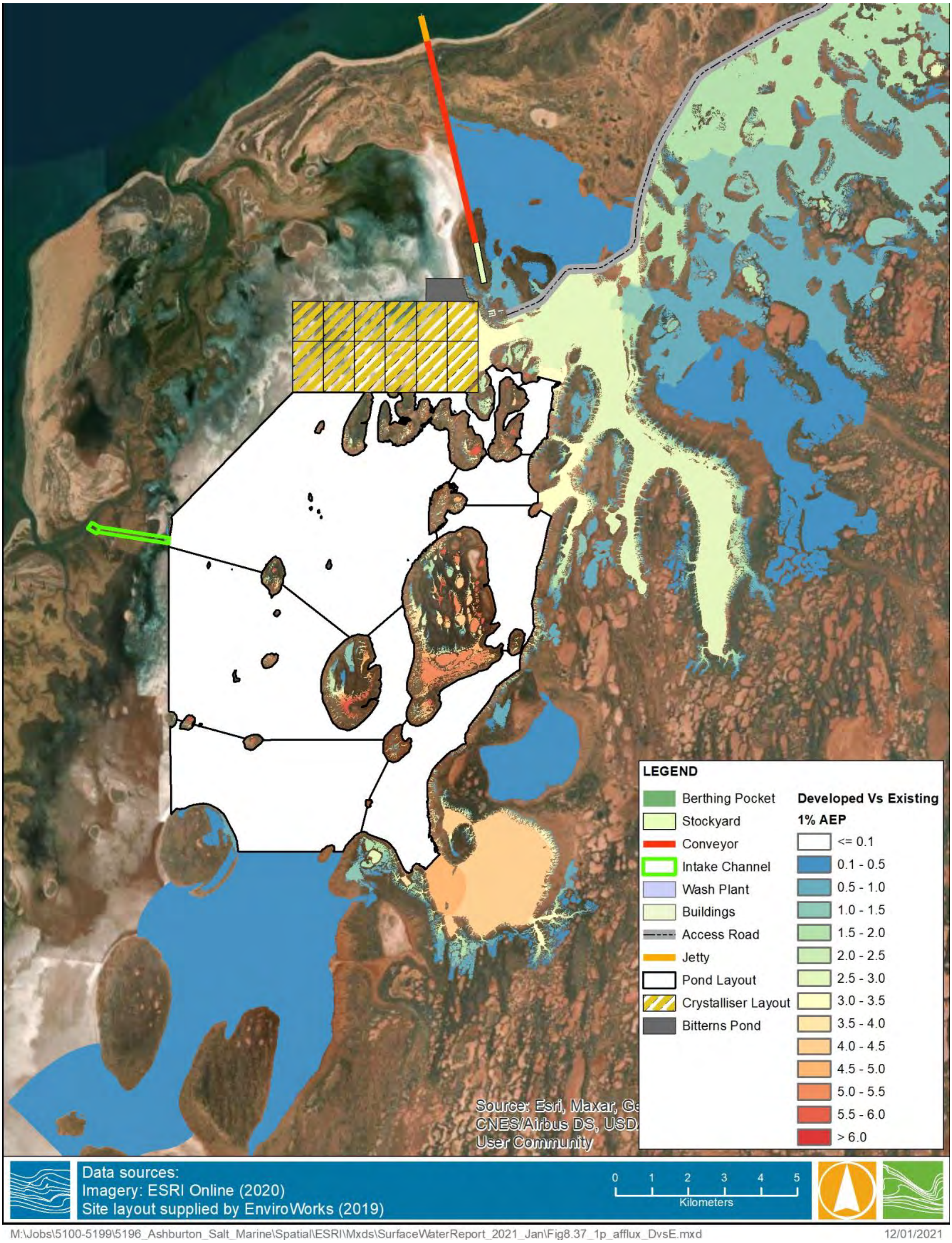

FIGURE 10-37 MODELLED 1% AEP AFFLUX - EXISTING VS DEVLEOPED (UNMITIGATED)

11 FLOOD MITIGATION STRATEGIES

11.1 Approach to Mitigation

Surface water modelling under post development conditions identified modifications to existing flow regimes with increases in water levels behind the proposed footprint, access roads and conveyor. To manage these impacts, the following mitigation strategies are recommended:

- Locate key infrastructure areas outside the 2% AEP flood zone, where possible;
- Divert flows around key infrastructure areas that intersect flow paths;
- Divert flows back onto natural flow paths;
- Ensure full conveyance of 10% surface water flows under the main access road into site;
- Ensure surface water flows into downstream receptors are not impeded by proposed infrastructure; and
- Protect infrastructure that falls outside of direct flow paths, but which is within the 2% AEP flood zone.

The following sections explain how these design strategies can be applied to the project site, given the results of the modelling completed for this study.

Based on hydraulic modelling of the 10%, 5%, 2% and 1% AEP flood events, the areas requiring flood mitigation protection have been identified and are shown in Figure 11-1.

11.2 Culvert Crossings

To accommodate catchment and tidal driven water moving across the site, the drainage control plan along the main access road and conveyor embankment should follow the following concepts:

- Ensure culverts at main access road are adequately sized to convey adopted design flows beneath the access road while avoiding adverse impacts on local watercourses. Water Technology suggests the main access road into the site convey the 10% AEP flood discharge;
- Table drains are recommended alongside the main access road to direct flow along the road to allow passage either through a floodway and/or culvert. All table drains/channels should be aligned with a grade sufficient to ensure no pooling of water. A minimum channel slope of 0.1% is recommended;
- Erosion protection should be provided up and down slope of the main access road where velocities are predicted to be >2 m/s; and
- Culverts at the conveyor are limited to an embankment fill height of no greater than 1 m in height with 0.5 m freeboard.

Water movement across the site is largely driven by tides, subsequently the surface water mitigation structures facilitating water moving back and forth underneath the access road and conveyor operate as balance pipes in most locations.

To ascertain preliminary culvert dimensions and numbers along the conveyor and access road, peak discharges were extracted from the developed hydraulic TUFLOW model. Culverts were then inserted into the TUFLOW model and via an iterative process the size and number of culverts were optimised to ensure conveyance. The proposed culvert locations along the access road and conveyor embankment are illustrated in Figure 11-1 and Figure 11-2, with their estimated discharge capacity summarised in Table 11-1. Since water movement across the site is broad and shallow and not within well-defined channels, the crossings were broken up into crossing areas (i.e., A, B, C, D, etc). Within each crossing area, multiple culvert locations are distributed to mimic natural surface water movement through the site. The modelled dimensions and quantities of culvert arrangements are presented in Table 11-2. It should be noted that this arrangement limits the use of diversion/table drains along the road to route flows to one central crossing location.

The recommended culverts were then inserted into the 2d hydraulic model using 1d_nwk and 2d_bc files to link the 1D culvert structures with the topography. The model was then run for the 10% and 5% AEP events to ascertain if conveyance under the road was possible without overtopping. At the conveyor, all culverts were limited to a 1 m height and 0.5 m freeboard whilst along the main access road culverts were sized for conveyance of the 10% event without overtopping. The results from the mitigated scenario hydraulic modelling simulations are presented as flood water level, depth and velocity maps in Section 11.5.

At the detailed design stage, Water Technology recommends detailed engineering drawings and survey at all road crossing locations to accurately size culverts and appropriate erosion protection requirements.

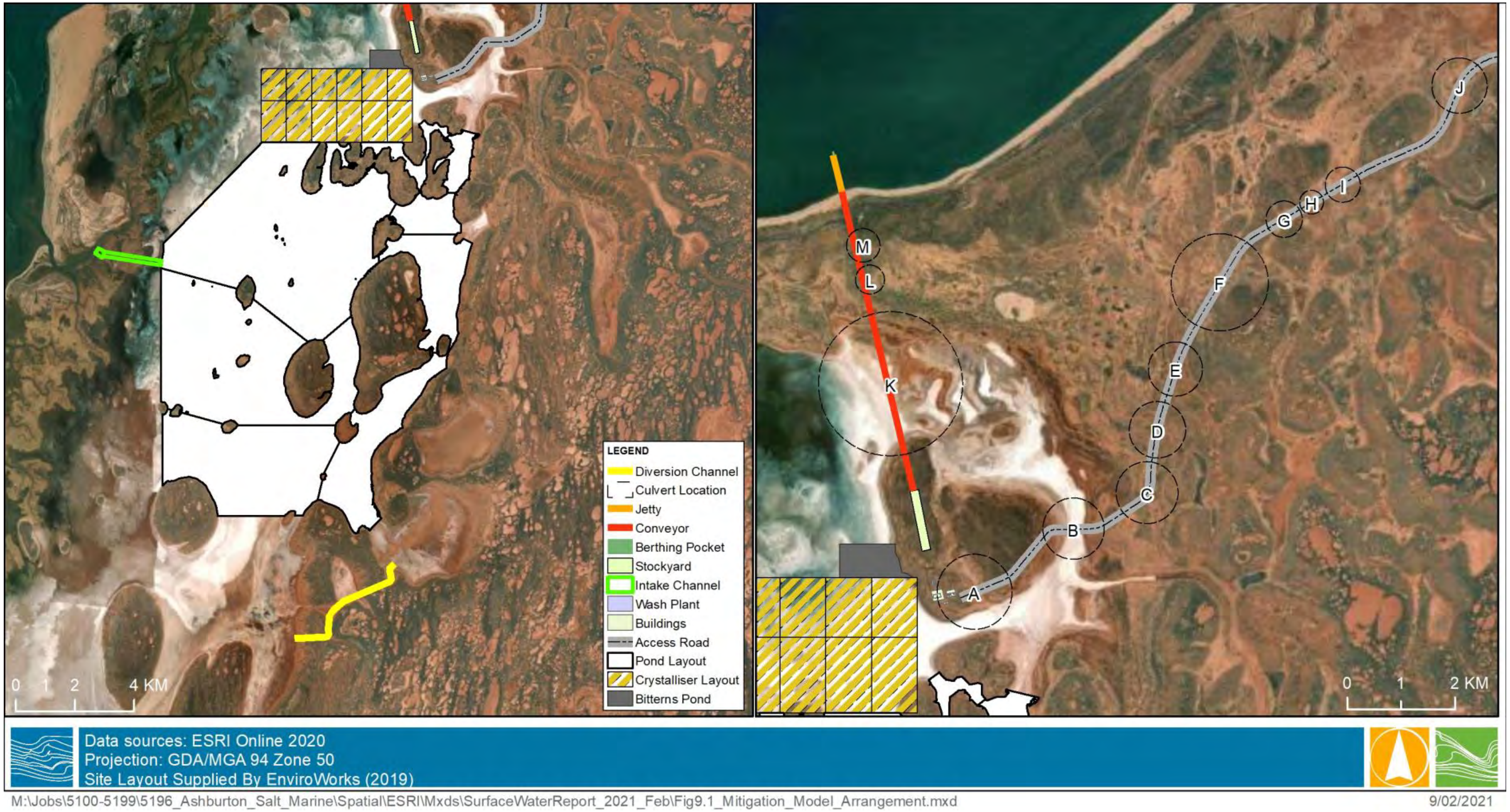

FIGURE 11-1 MITIGATION MODEL ARRANGEMENT

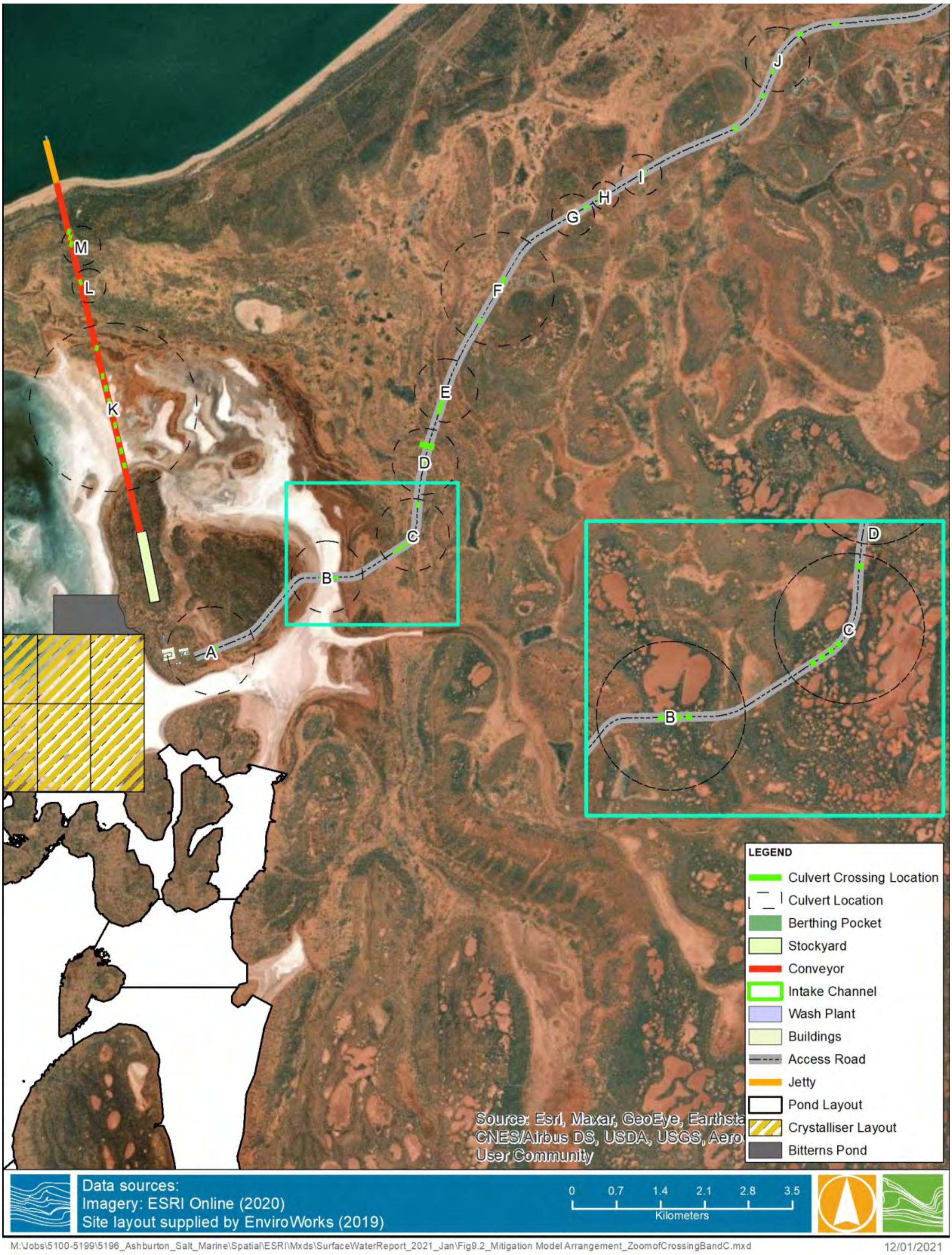

FIGURE 11-2 MITIGATION MODEL ARRANGEMENT (WITH INSET OF CROSSING B AND C)

TABLE 11-1 PROPOSED CULVERTS ESTIMATED DISCHARGE CAPACITY

Crossing Region	10% AEP Flow (m^3/s)	5% AEP Flow (m^3/s)	Flood Width at Crossing (m)
A	7.9	19	360
B	185	194	790
C	146	259	950
D	48	85	965
E	3.1	7	530
F	35	91	1400
G	38	59	570
H	10	41	230
I	232	378	759
J	187	251	3000
K	341	598	1600
L	75	90	440
M	101	143	600

TABLE 11-2 PROPOSED CULVERT ARRANGEMENT

Crossing Location	Culvert Crossing ID	Width or Depth (m)	Height (m)	Quantity	Total Quantity
A	A1	1.5	-	22	66
	A2	1.5	-	22	
	A3	1.5	-	22	
B	B1	1.5	-	24	96
	B2	1.5	-	24	
	B3	1.5	-	24	
	B4	1.5	-	24	
C	C1	2.1	-	34	136
	C2	2.1	-	34	
	C3	2.1	-	34	
	C4	2.1	-	34	
D	D1	1.8	-	24	72
	D2	1.8	-	24	
	D3	1.8	-	24	
E	E1	1.5	-	10	40
	E2	1.5	-	10	
	E3	1.5	-	10	
	E4	1.5	-	10	
F	F1	1.5	-	24	96
	F2	1.5	-	24	

Crossing Location	Culvert Crossing ID	Width or Depth (m)	Height (m)	Quantity	Total Quantity
	F3	1.5	-	24	
	F4	1.5	-	24	
G	G1	1.8	-	28	56
	G2	1.8	-	28	
H	H1	1.8	-	28	28
I	I1	3.3	3.3	40	40
J	J1	2.1	-	23	138
	J2	2.1	-	23	
	J3	2.1	-	23	
	J4	2.1	-	23	
	J5	2.1	-	23	
	J6	2.1	-	23	
K	K1	24	1	12	108
	K2	24	1	12	
	K3	24	1	12	
	K4	24	1	12	
	K5	24	1	12	
	K6	24	1	12	
	K7	24	1	12	
	K8	24	1	12	
	K9	24	1	12	
L	L1	24	1	17	17
M	M1	24	1	15	45
	M2	24	1	15	
	M3	24	1	15	

11.3 Diversion Channels and Levees

As shown in Figure 10-5 to Figure 10-37 (representing unmitigated post-development model outputs), surface water flows from upstream catchments and pools within basins on the eastern margins of the evaporation ponds. To alleviate flooding in this region, diversion channels were proposed and modelled to divert all events >2% AEP. Conceptually the diversion channels were designed to move water from one basin to the next in a southerly direction (to lower water levels) via three diversion channels. These structures were modelled for all events (i.e., 10%, 5%, 2% and 1% AEP), however they were only designed to the 2% AEP and above. This is due to the significant cuts required to capture and convey events <2%. See Figure 11-3 for diversion channel locations. Dimensions of the modelled diversion channels are presented in Table 11-3 and the channels depths are shown in Figure 11-4 and Figure 11-5. Results of the mitigation modelling are discussed in Section 11.5.

Nominal locations of levees to protect pond embankments from floodwaters within the eastern basins have also been provided in Figure 11-3. These levees are conceptual and engineering designs should be developed during the detailed design phase.

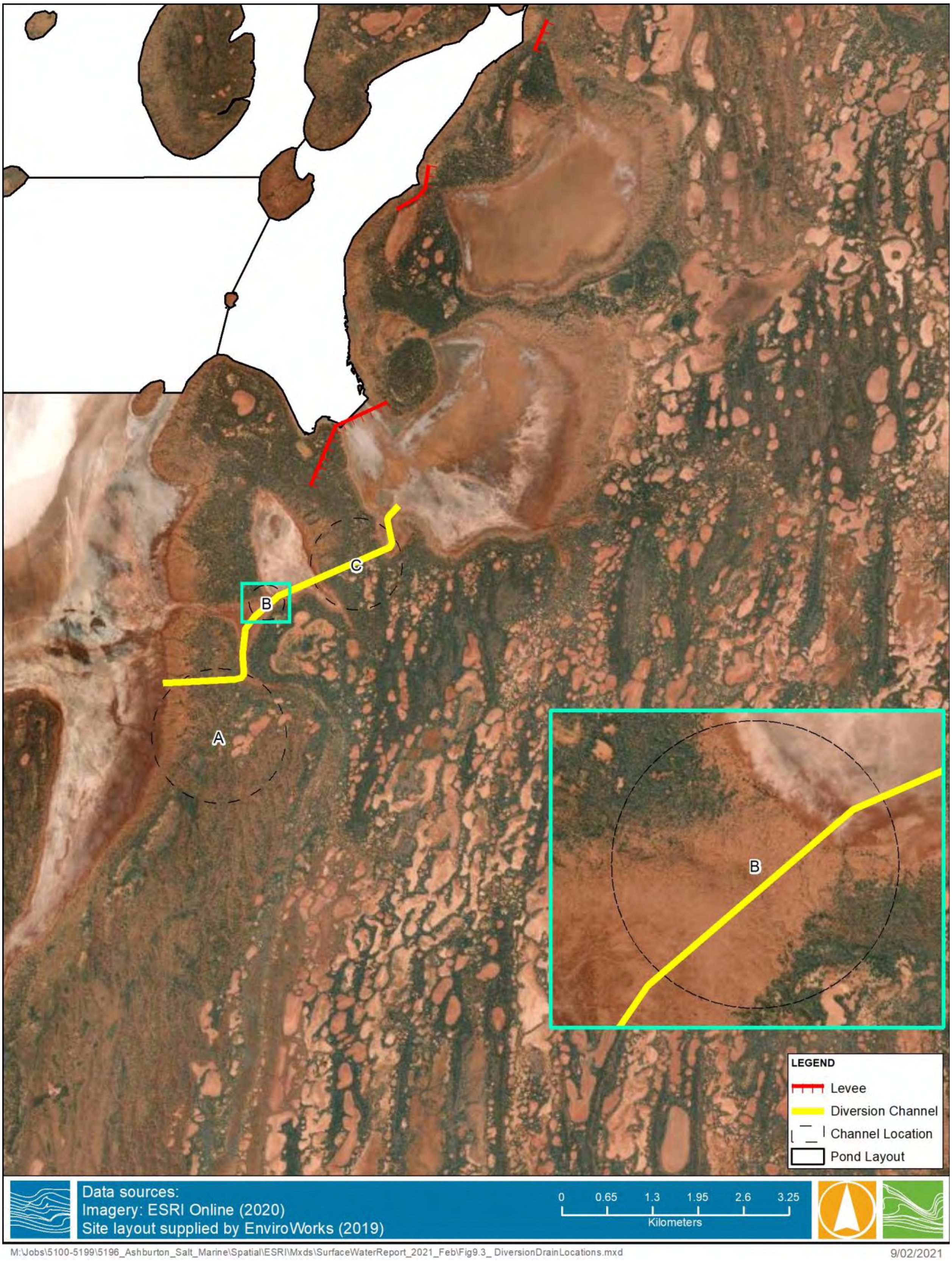

FIGURE 11-3 DIVERSION CHANNEL AND CONCEPTUAL LEVEE LOCATIONS (WITH INSET OF CHANNEL B)

TABLE 11-3 DIVERSION CHANNEL PARAMETERS

Diversion Channel ID	Length (m)	Width (m)	Elevation (m AHD)	Estimated Cut Volume (m^3)
A	880	150	3.9 – 4.2	329,900
B	150	150	4.8 – 4.9	31,800
C	950	150	5.3 – 5.6	138,500

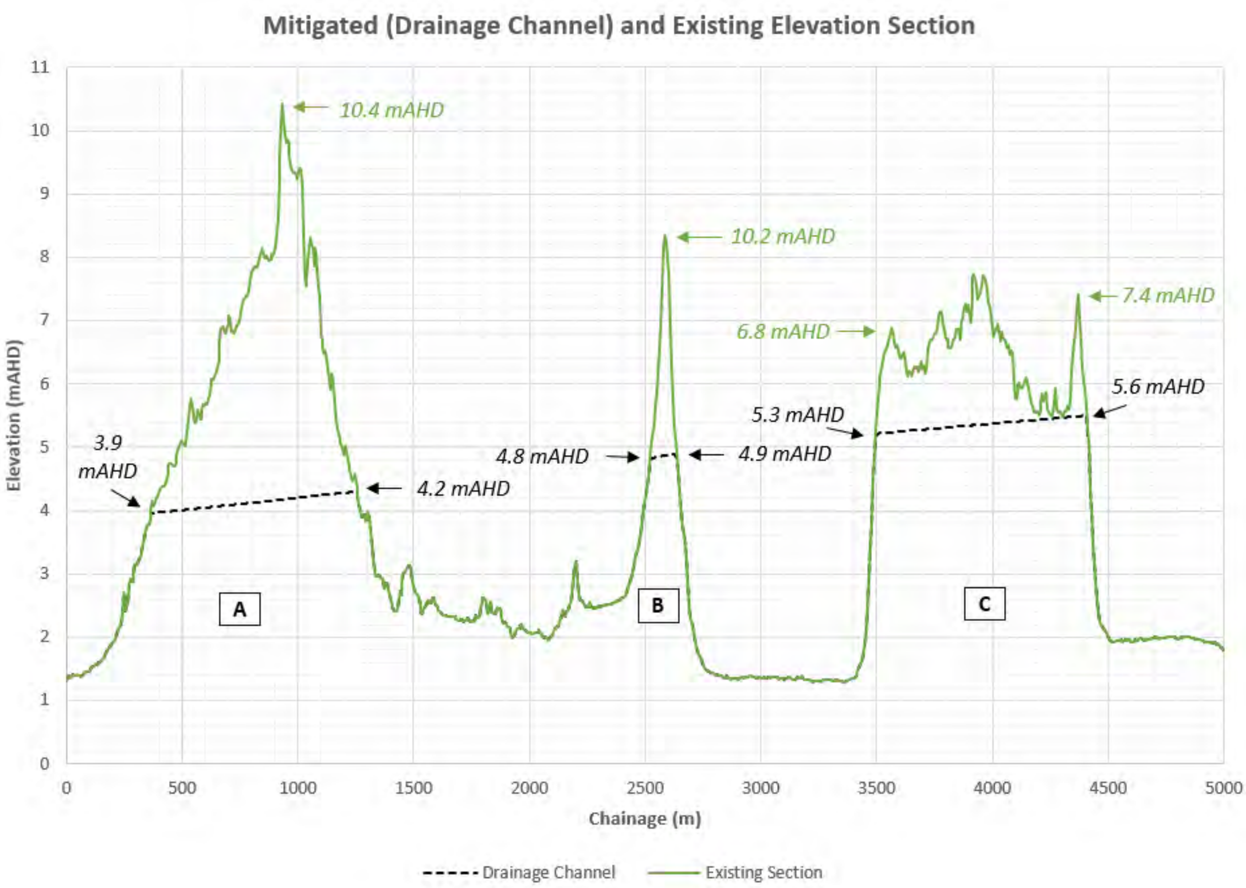

FIGURE 11-4 DIVERSION CHANNEL SECTION WITH ELEVATIONS

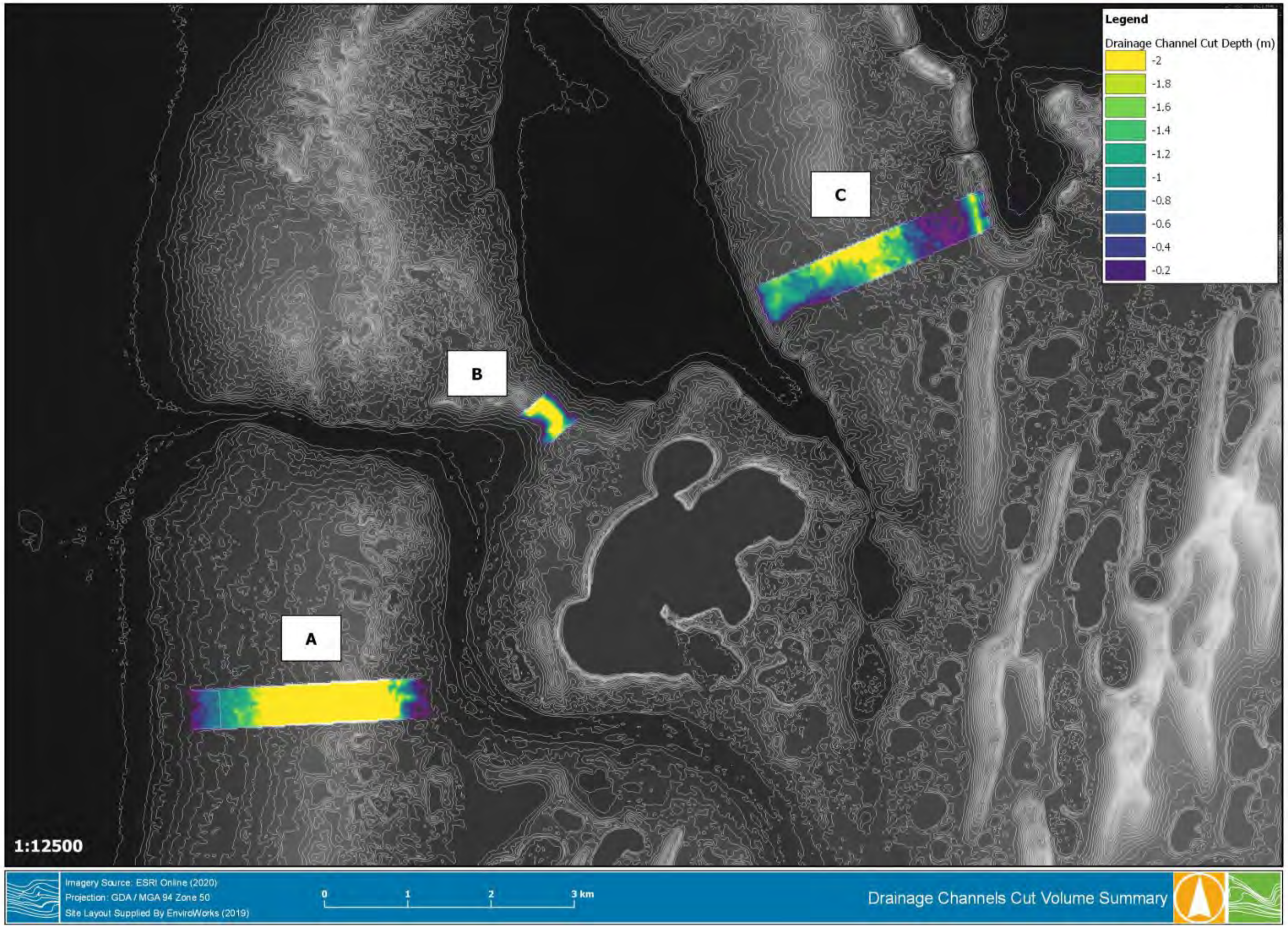

FIGURE 11-5 DIVERSION CHANNEL CUT DEPTHS

11.4 Ashburton River Bridge Crossing

Water Technology understands that a bridge crossing over the Ashburton River is required to allow access to the site from Onslow. The crossing location proposed by K+S is approximately 20 km from the development site/evaporation ponds. The bridge is being designed to the 2% AEP and is in its preliminary design stages. The results from this study will be used to set appropriate deck heights and to inform future more detailed hydraulic modelling.

Given the preliminary nature of this study, little design information was available and certain modelling assumptions were made; see Section 8.3.7 for design assumptions. Utilising all available information, the bridge was modelled in TUFLOW as a 'Layered Flow Constriction' (LFC) to represent the deck, piers and railings. Assessment of the modelling results suggests water levels, during the 2% AEP, reach a maximum level of 3.45 m AHD and flow velocities are low (<0.5%) both upstream and downstream of the bridge crossing. In the absence of detailed survey information, these values should be considered preliminary only. Once more detailed survey information is available, water levels and flow velocities can be further refined and the hydrodynamics at the base of the piers will also be considered to allow for scour impacts in the design.

11.5 Flood Mitigation Modelling Results

11.5.1 Mitigation Results – Flood Behaviour

Hydraulic modelling has been used to simulate the movement of flood waters during the 10%, 5%, 2% and 1% AEP design events under mitigated conditions (i.e. culverts, levees and diversion channels). The results of this assessment will identify the effectiveness of the culvert arrangements and diversion channels proposed in Section 11.2 of this report.

In undeveloped conditions (as discussed above in Section 10.4), water movement across the site is largely generated from discharge in a westerly direction onto the salt flat area from overland flow paths. During unmitigated developed conditions, the movement of surface water (in a westerly direction) becomes blocked by the main access road, conveyor, and pond embankments - resulting in significant water level increases (between 1 – 6 m) along the eastern margins of the site. However, a review of the hydraulic modelling results under mitigated conditions shows that the proposed culverts (along the access and conveyor roads) allow surface water (up to the 10% AEP) to move in a westerly direction under the access road and then south under the conveyor. Comparatively, flood depths under the mitigated modelling scenario vs the developed scenario are significantly reduced (1 – 4 m) on the eastern side of the access road.

Modelling of the mitigated post-development scenario indicates that some backing up or localised ponding of water will still occur adjacent to pond embankments, the access road and conveyor. However, ponding of water is localised and is a detailed engineering design consideration, rather than an environmental impact. Such localised ponding is unlikely to have any deleterious environmental effects, given it simply mimics the local flooding regime. However, it is recommended that this ponding is considered from an engineering perspective during detailed design, and if necessary, optimisation of mitigation strategies is undertaken to meet required engineering standards.

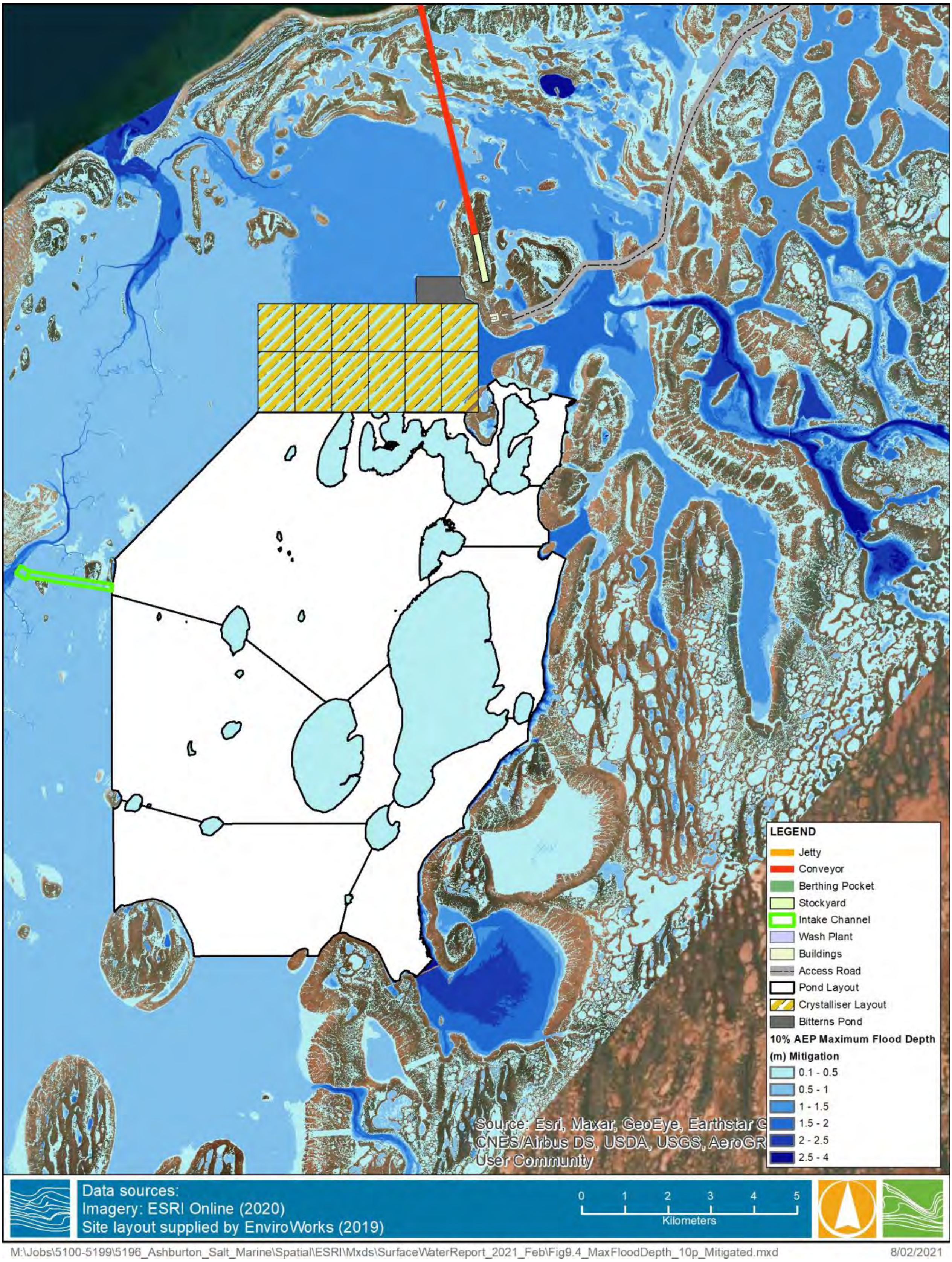

FIGURE 11-6 MODELLED 10% AEP MAXIMUM FLOOD DEPTH (POST-DEVELOPMENT MITIGATED)

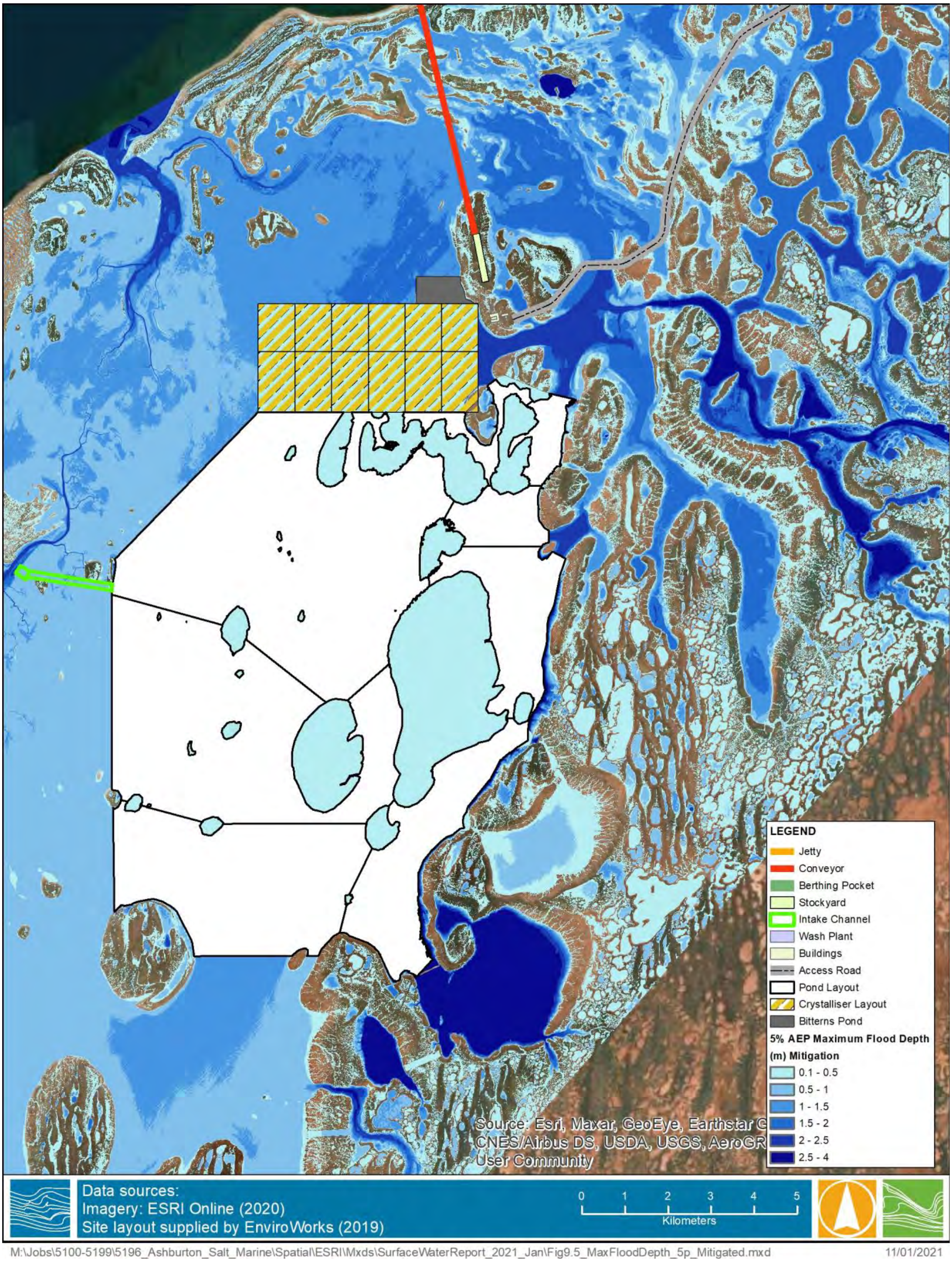

FIGURE 11-7 MODELLED 5% AEP MAXIMUM FLOOD DEPTH (POST DEVELOPMENT MITIGATED)

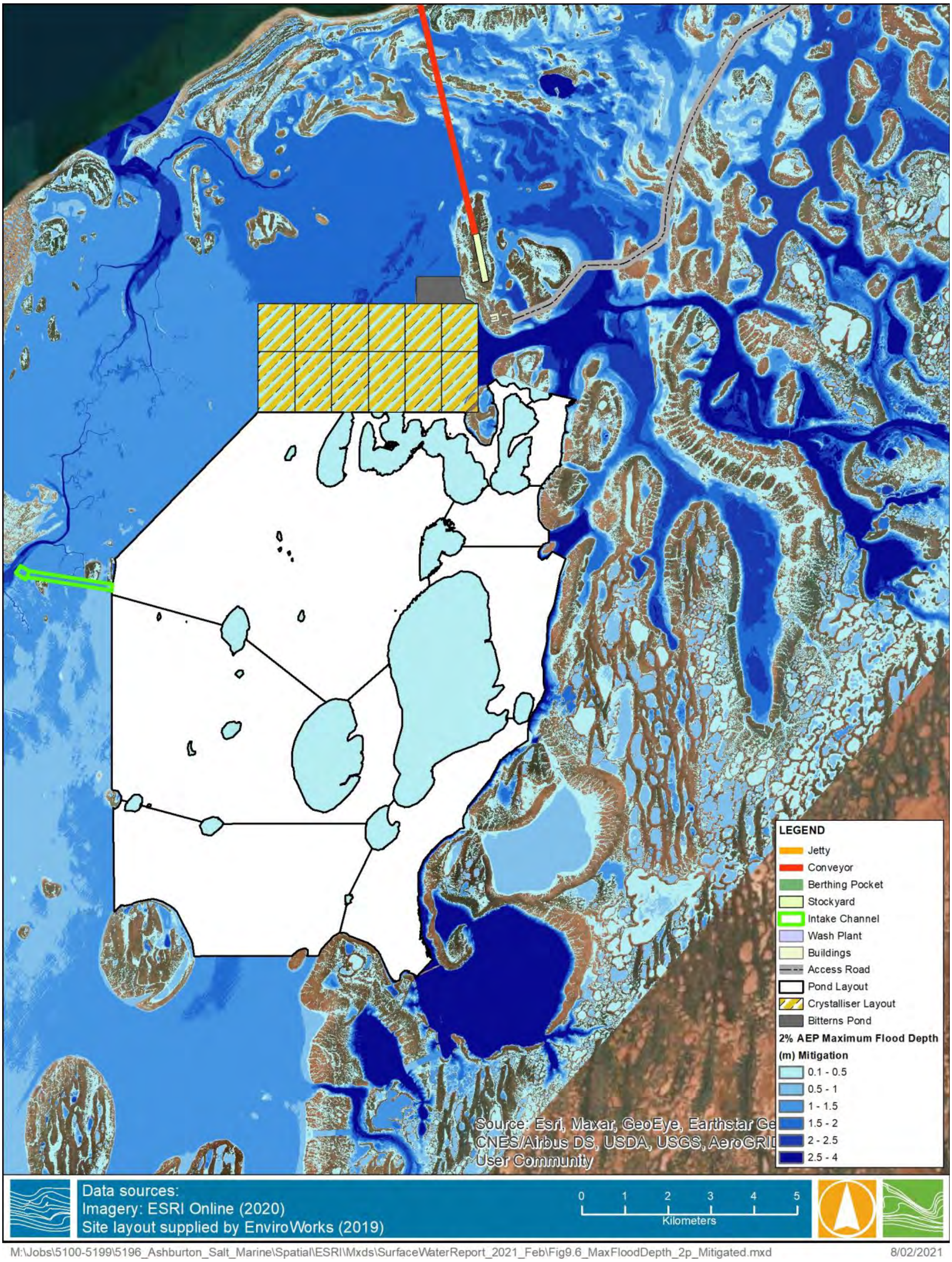

FIGURE 11-8 MODELLED 2% AEP MAXIMUM FLOOD DEPTH (POST DEVELOMENT MITIGATED)

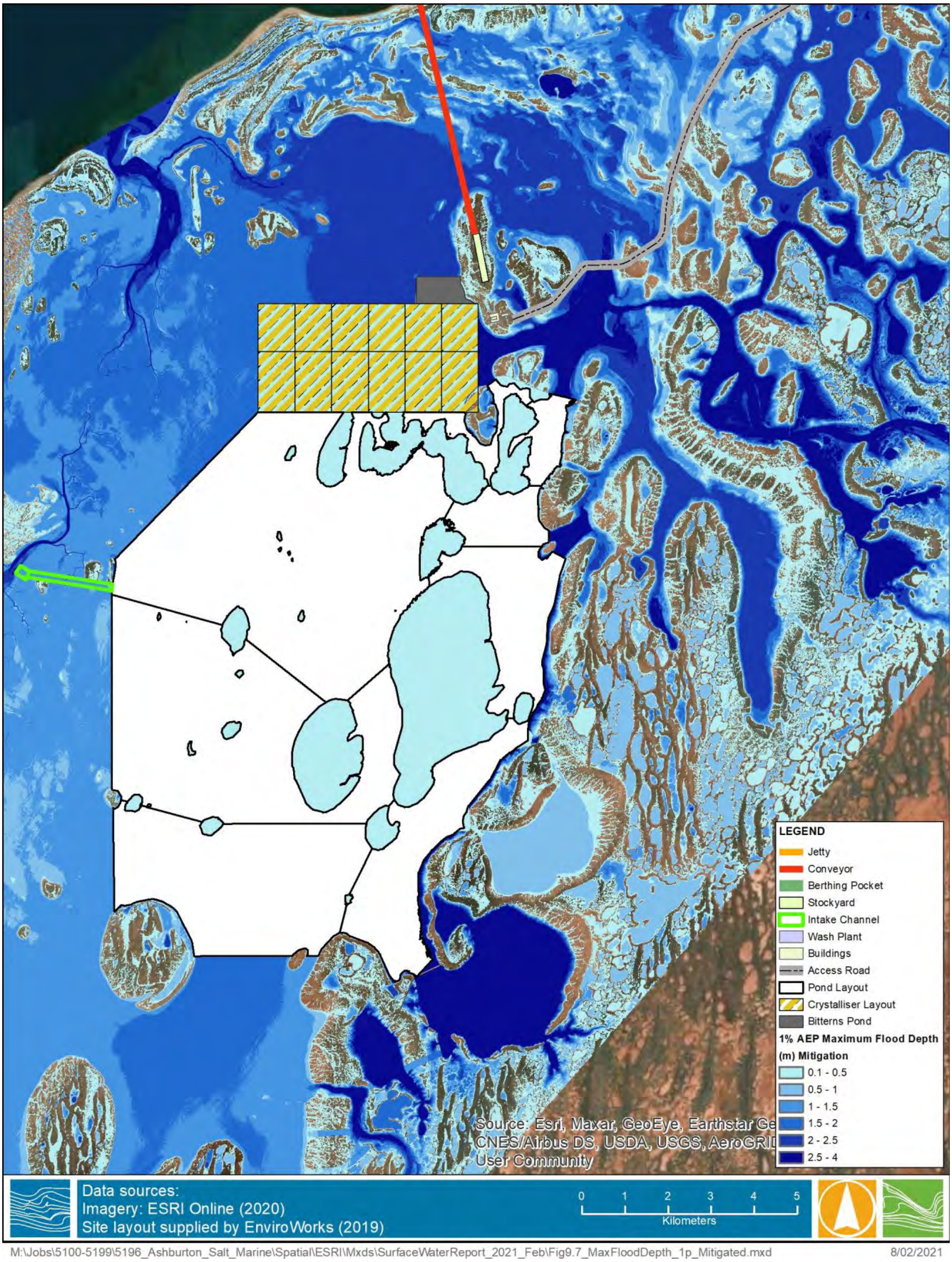

FIGURE 11-9 MODELLED 1% AEP MAXIMUM FLOOD DEPTH (POST DEVELOMENT MITIGATED)

11.5.2 Mitigated Results – Flood Level Afflux

Assessment of mitigated post-development modelling results against modelled pre-development conditions suggests that water levels are significantly reduced (following inclusion of culverts) on the upstream side of the main access road and conveyor embankment. However as shown by Figure 11-10, Figure 11-11, Figure 11-12 and Figure 11-13, water levels are still slightly higher (0.1 – 1.5 m) under mitigated conditions when compared to existing conditions. As mentioned, above this ponding of water is localised and unlikely to cause deleterious environmental impacts. However, it should be considered during detailed engineering design whether further optimisation of mitigation measures is required to meet engineering standards.

Similarly, the water level under post-development mitigated conditions in the south-east basin is reduced, however only modestly when compared to developed unmitigated conditions (0.1 m reduction). The mitigated conditions included diversion channels in the south-east corner (see Figure 11-3) designed to move water through the adjacent basins in a southerly direction for discharge onto the salt flats. However, modelling has shown that due to low grades in the area and more specifically within the salt flats, the diversion channels only cause a modest reduction in water levels within the south east basin. Ponding is localised to the existing basin which already floods due to rainfall. Increased water levels within the basin are unlikely to have a negative environmental impact – it will simply mean flooding duration is increased which may be environmentally beneficial given the flooded basins are habitat for waterbirds and invertebrate fauna. However, it should be considered during detailed engineering design whether further optimisation of the drainage diversions is required to meet engineering and production standards.

In addition, ponding within borrow pits planned to be constructed along the eastern margins of the site should also be considered during detailed design. The locations of the borrow pits were not available at the time surface water modelling for this project was conducted, however it is expected that any ponding of water within them will be an issue for engineering consideration, rather than an environmental concern, given flooding of the borrow pits will simply mimic the flooding regime of local basins (which may provide fauna habitat when flooded) and therefore may be environmentally beneficial.

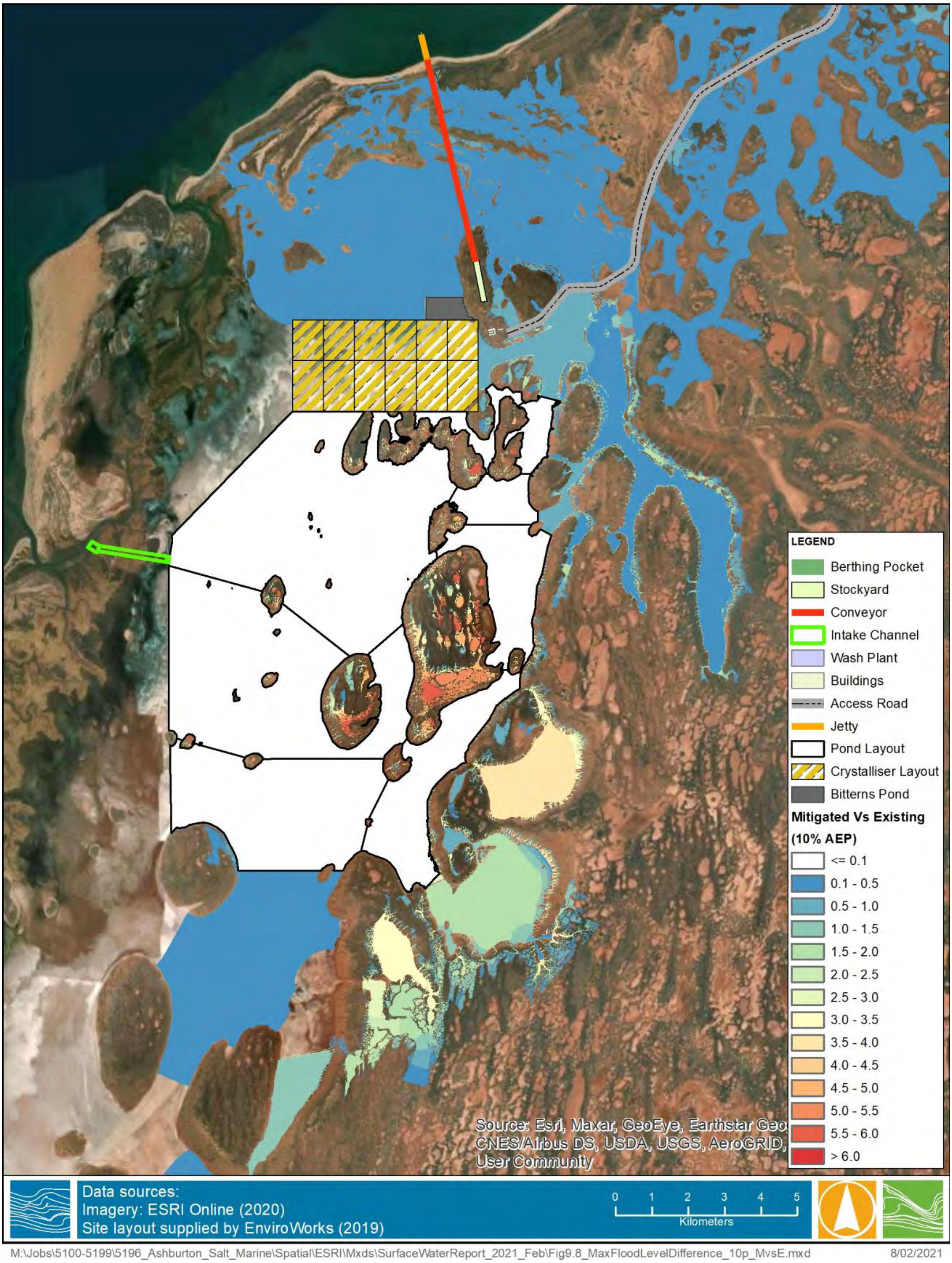

FIGURE 11-10 MODELLED 10% AEP AFFLUX (M) – EXISTING VS DEVELOPMENT (MITIGATED)

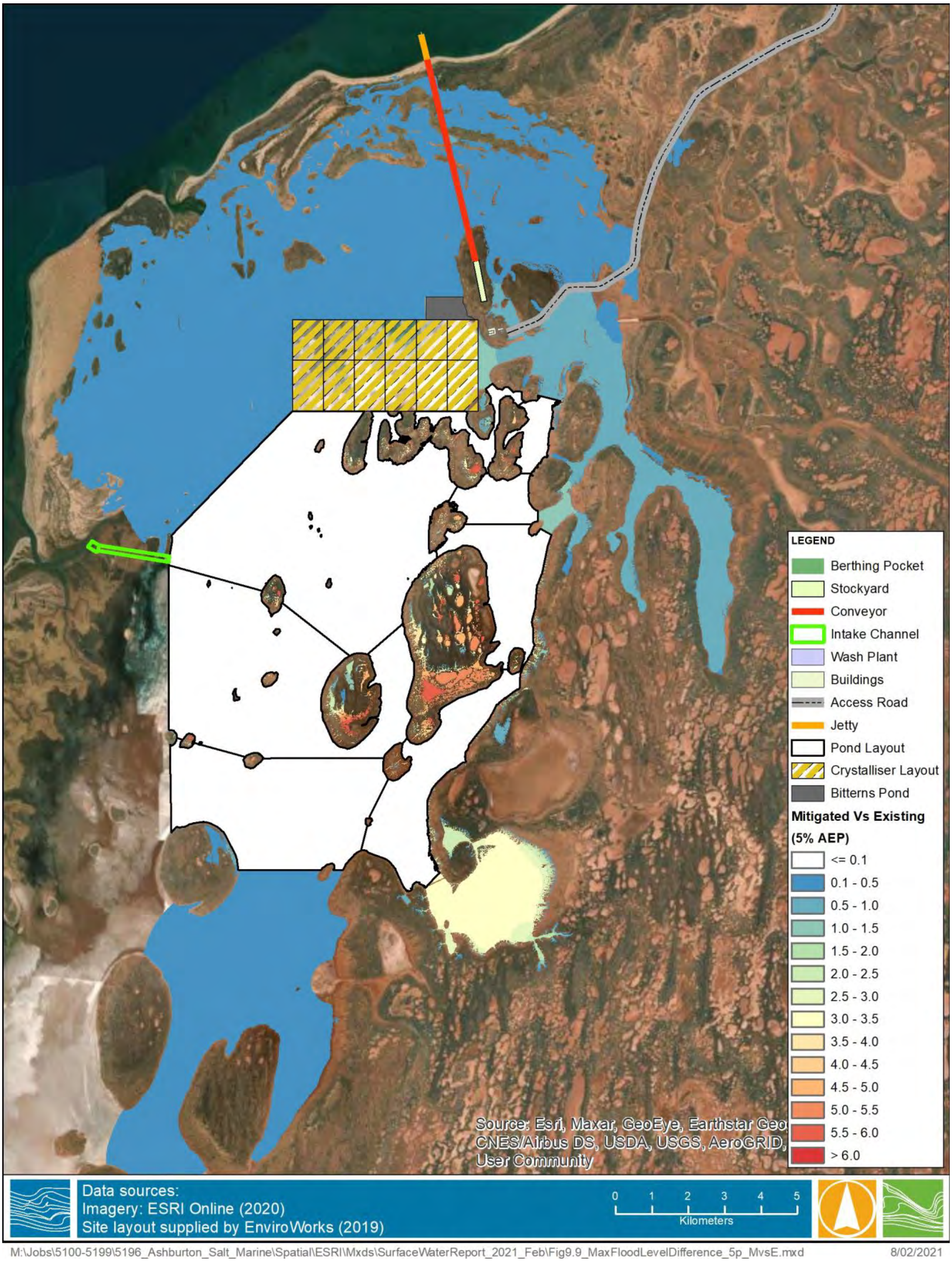

FIGURE 11-11 MODELLED 5% AEP AFFLUX (M) – EXISTING VS DEVELOPMENT (MITIGATED)

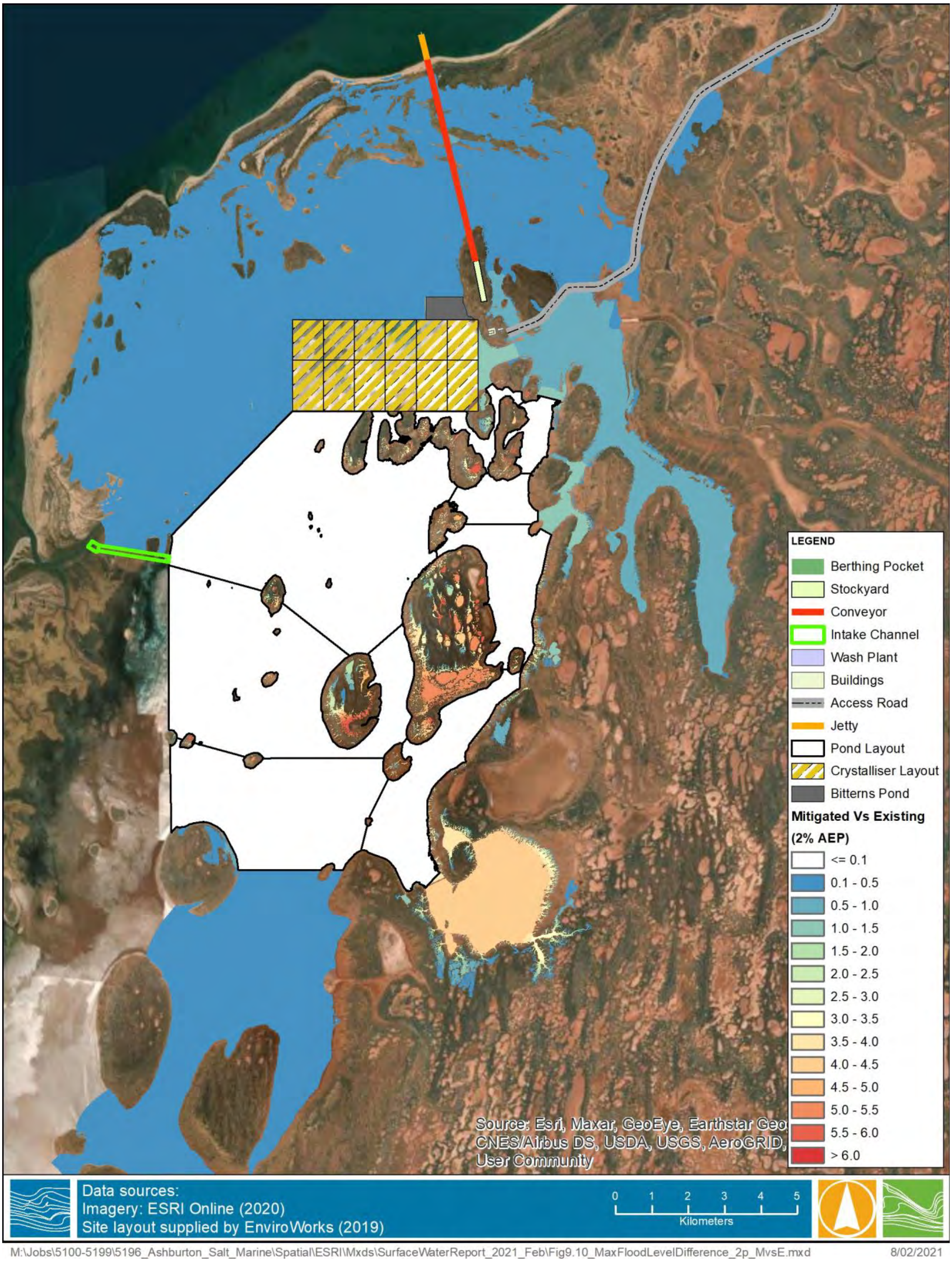

FIGURE 11-12 MODELLED 2% AEP AFFLUX (M) – EXISTING VS DEVELOPMENT (MITIGATED)

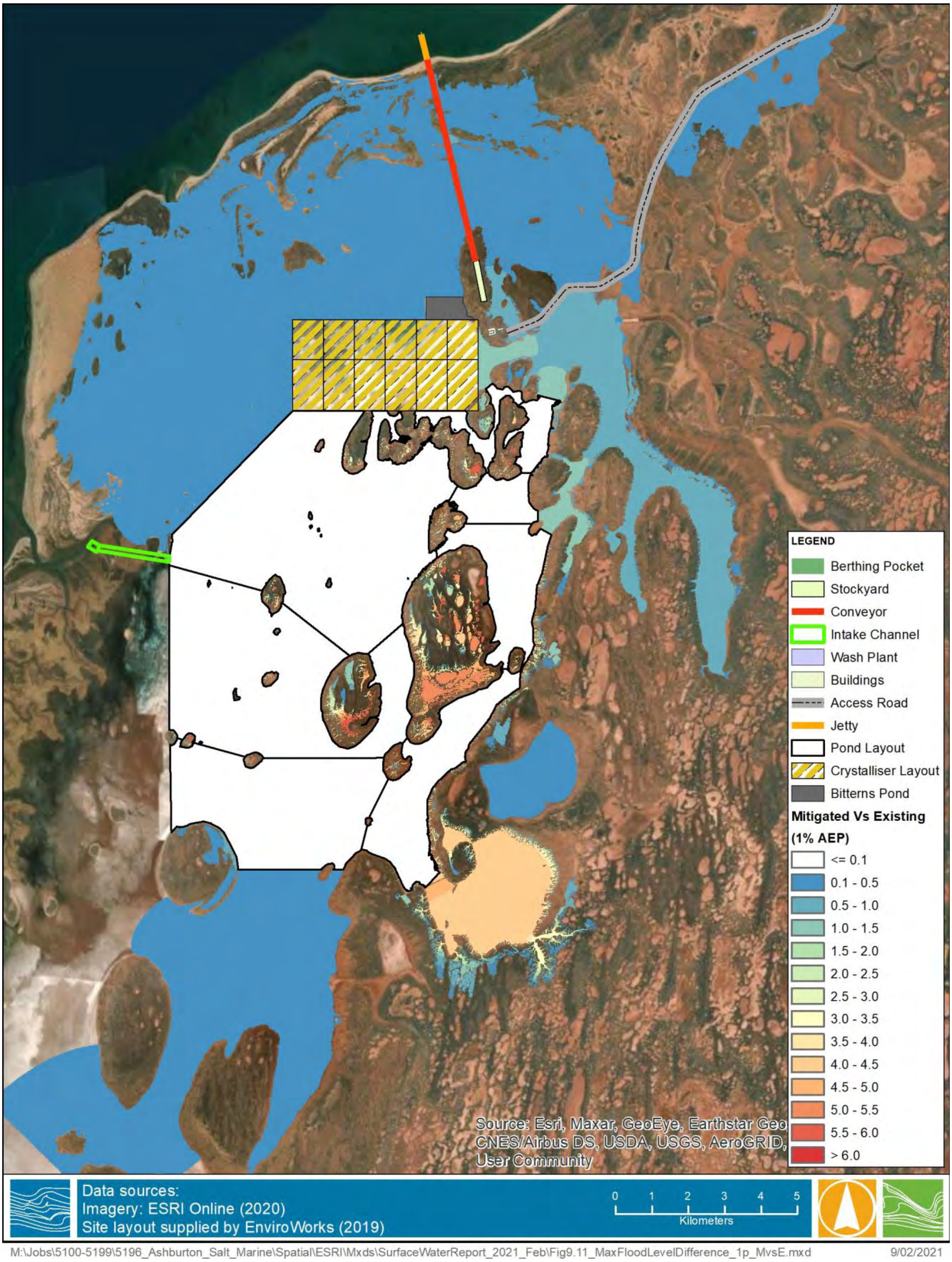

FIGURE 11-13 MODELLED 1% AFFLUX (M) – EXISTING VS DEVLOPED (MITIGATED)

Additionally, flood level changes at key locations throughout the model, under mitigated and developed scenarios and all events have been summarised in Table 11-4. The locations of these points are illustrated in Figure 11-14.

TABLE 11-4 MAXIMUM FLOOD LEVEL SUMMARY FOR ALL SCENARIOS AND EVENTS

Label	10% AEP Flood Level (m AHD)			5% AEP Flood Level (m AHD)			2% AEP Flood Level (m AHD)			1% AEP Flood Level (m AHD)		
	EXG	DEV	MIT	EXG	DEV	MIT	EXG	DEV	MIT	EXG	DEV	MIT
A	1.62	1.65	1.65	1.81	1.86	1.86	2.03	2.07	2.07	2.11	2.16	2.18
B	1.70	1.80	1.80	1.91	2.03	2.04	2.19	2.35	2.39	2.35	2.58	2.61
C	1.94	1.94	1.95	4.38	4.38	4.35	6.54	6.54	6.09	6.75	6.79	6.49
D	1.98	3.59	3.54	2.06	5.02	5.09	2.42	6.93	6.74	2.69	7.17	7.07
E	1.69	1.71	1.70	1.79	1.84	1.80	1.93	2.16	2.00	2.04	2.47	2.17
F	2.13	4.80	2.62	2.37	5.22	2.97	2.70	5.59	3.46	2.87	5.65	3.75
G	2.15	4.80	2.64	2.45	5.22	2.96	2.72	5.59	3.45	2.88	5.65	3.74
H	2.52	3.65	2.55	2.90	4.16	2.94	3.26	4.50	3.32	3.42	4.56	3.51
I	2.34	2.99	2.34	2.80	3.32	2.81	3.19	3.61	3.21	3.35	3.71	3.38
J	2.80	1.56	2.80	3.15	2.47	3.14	3.49	3.02	3.50	3.66	3.24	3.70
K	1.96	1.60	2.32	2.23	2.53	2.61	2.60	2.75	3.02	2.80	2.96	3.23
L	1.66	1.36	1.88	1.88	1.55	2.20	2.22	2.04	2.63	2.40	2.41	2.84
M	1.61	1.39	1.66	1.81	1.46	1.95	2.10	1.85	2.30	2.27	2.11	2.48
N	1.62	1.63	1.64	1.79	1.75	1.81	2.04	1.95	2.01	2.17	2.01	2.08

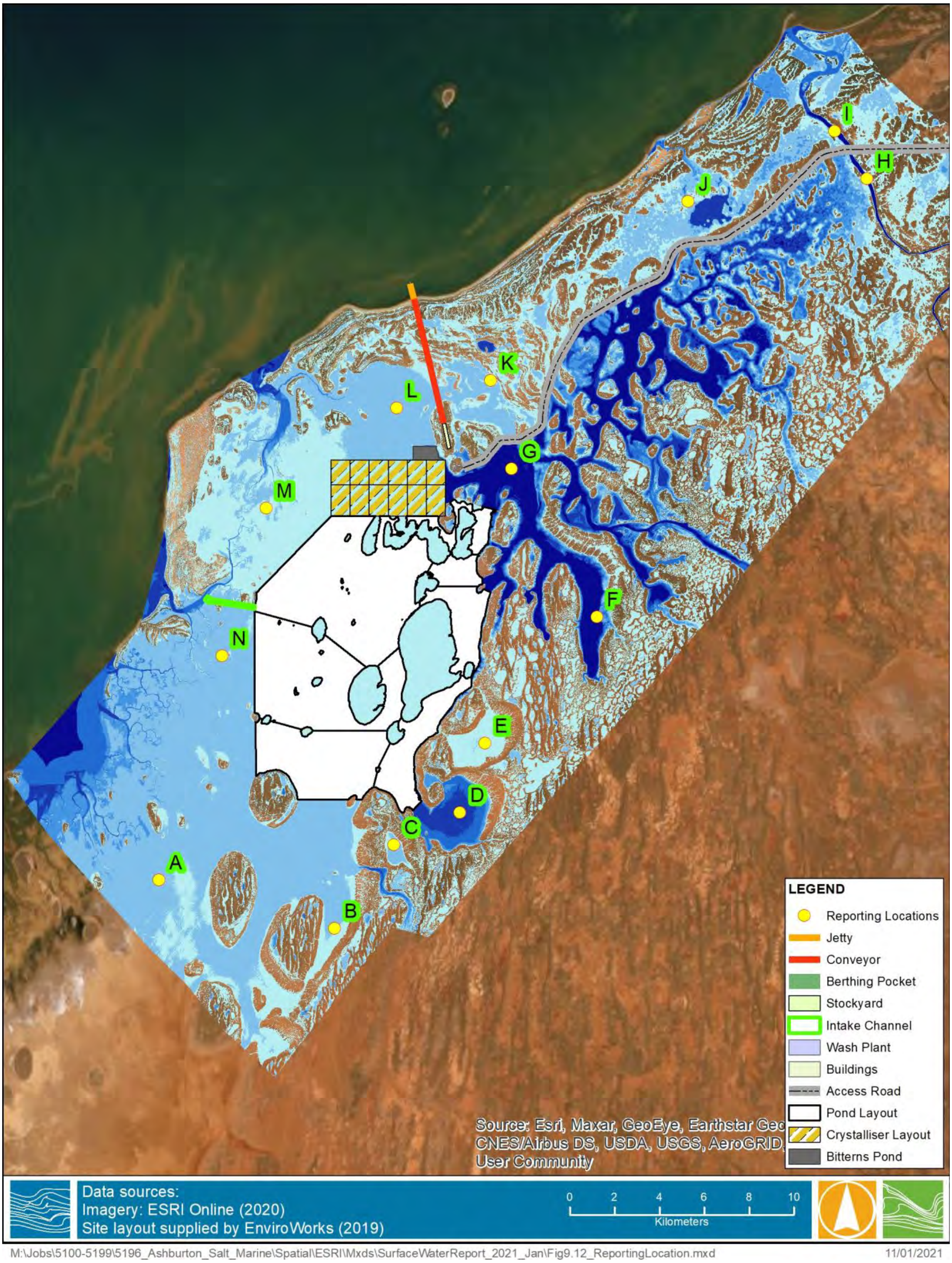

FIGURE 11-14 REPORTING LOCATIONS

11.5.3 Mitigated Results – Flood Velocity Difference

The hydraulic modelling indicates that floodwater velocities in the mitigated scenario are generally less than or equal to the existing scenario. This is particularly evident adjacent to the conveyor road and immediately upstream of the access road. Furthermore, there are notable decreases in flood velocity in the north-eastern corner of the hydraulic model and immediately downstream of the ponds. The results are illustrated in Figure 11-15 to Figure 11-18 which highlight the minimal flood velocity difference between the mitigated and existing scenarios for the 10%, 5%, 2% and 1% AEP design events.

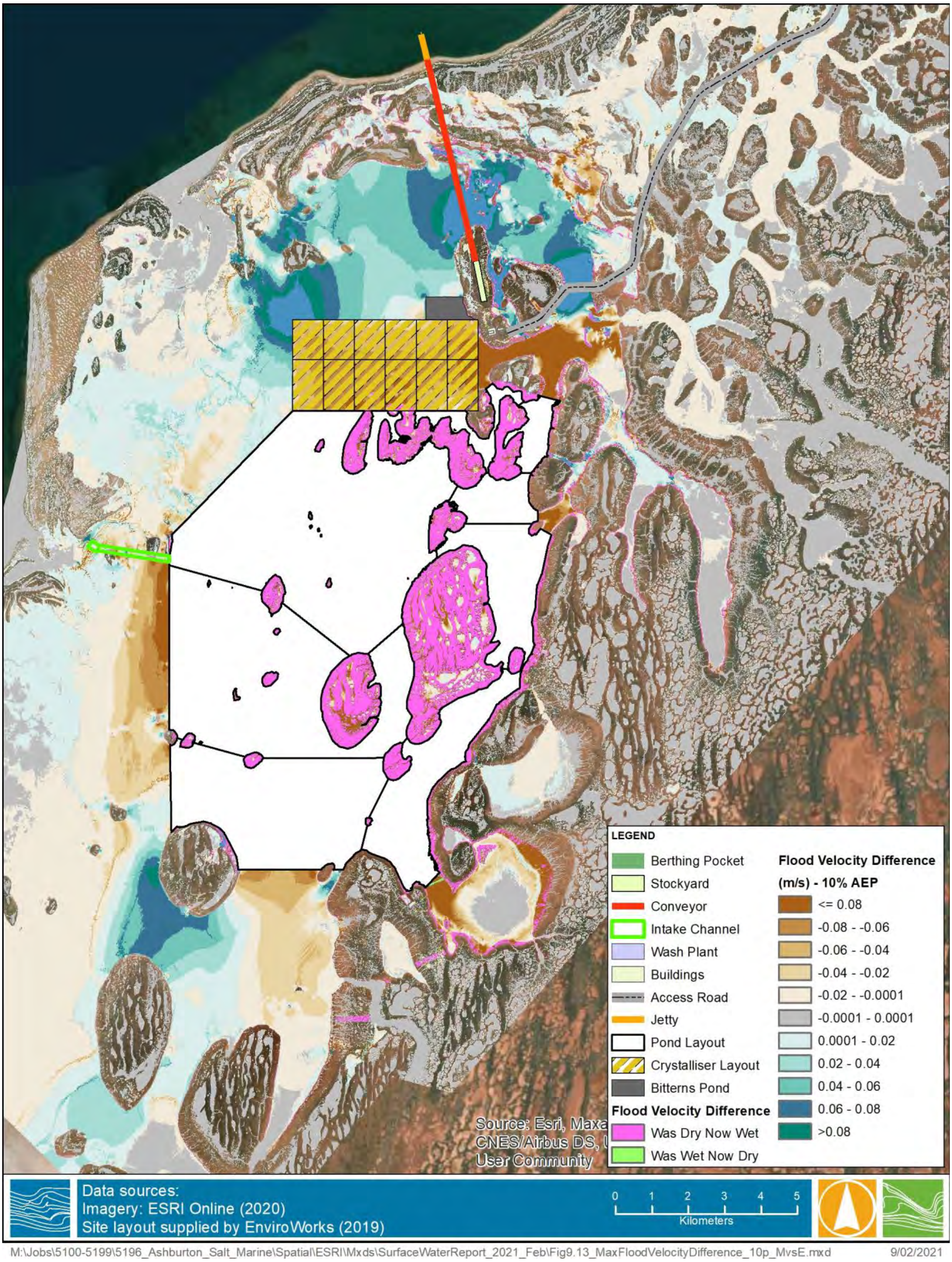

FIGURE 11-15 MODELLED 10% AEP MAXIMUM FLOOD VELOCITY DIFFERENCE (MITIGATED – EXISTING)

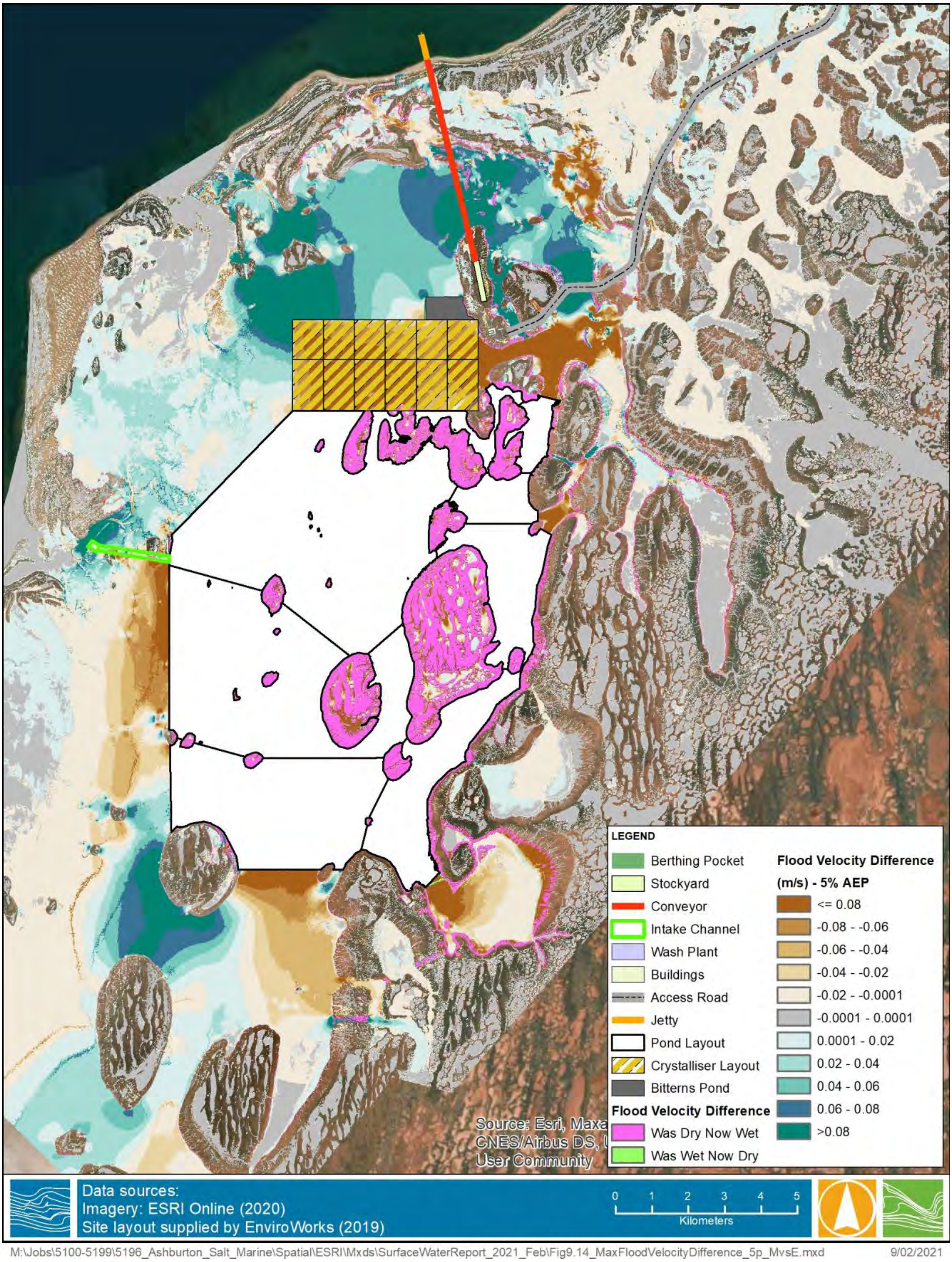

FIGURE 11-16 MODELLED 5% AEP MAXIMUM FLOOD VELOCITY DIFFERENCE (MITIGATED – EXISTING)

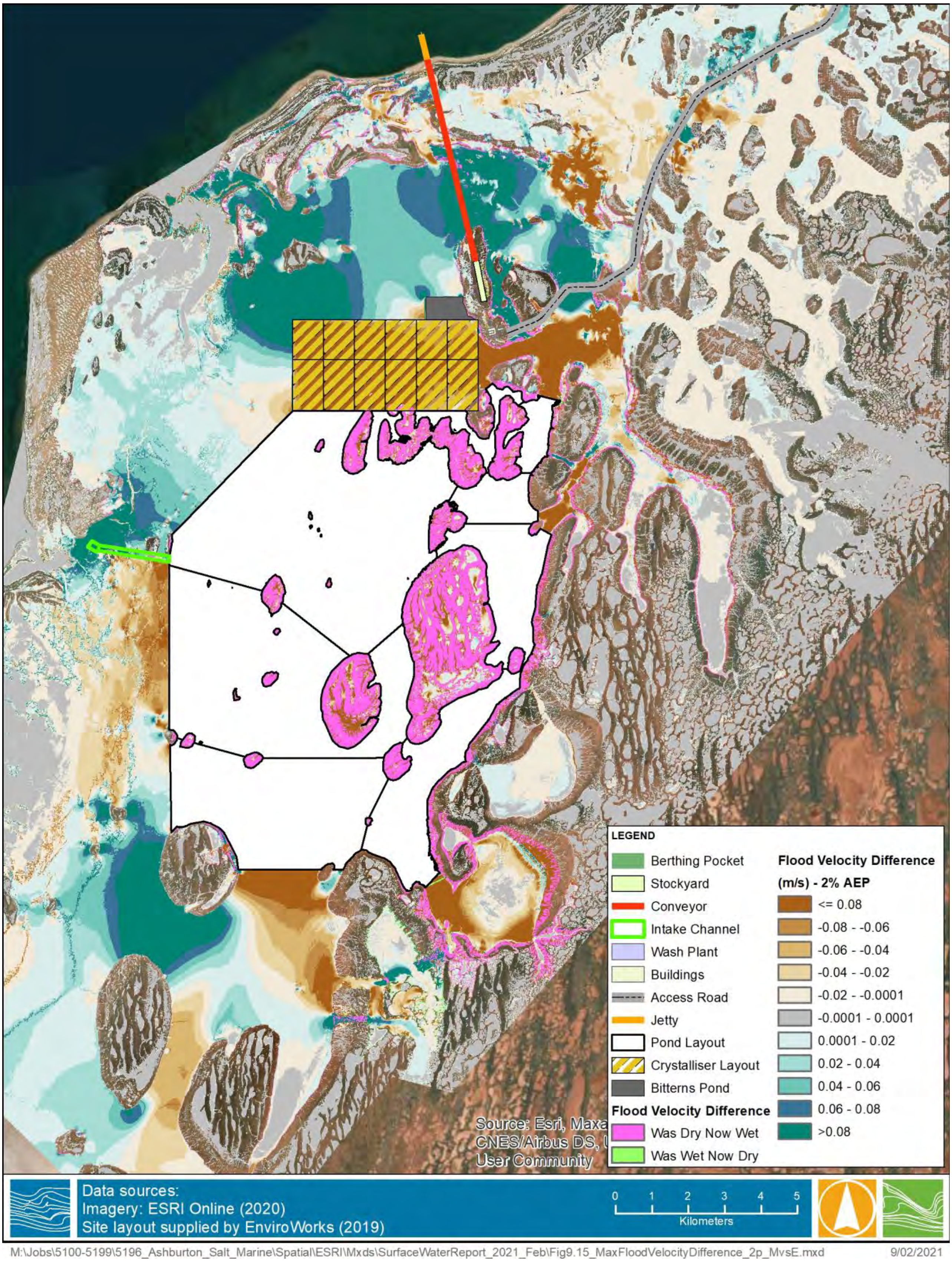

FIGURE 11-17 MODELLED 2% AEP MAXIMUM FLOOD VELOCITY DIFFERENCE (MITIGATED – EXISTING)

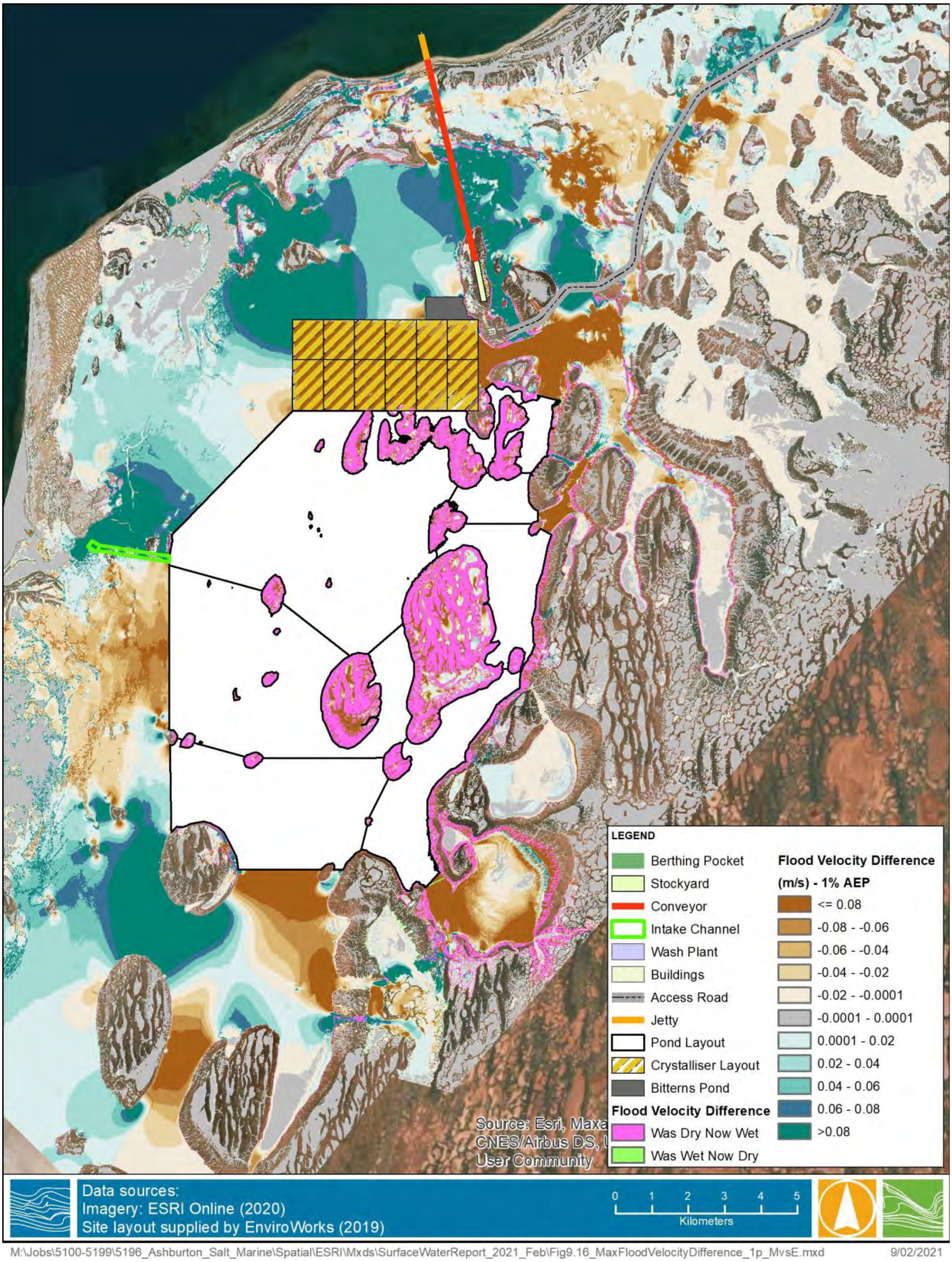

FIGURE 11-18 MODELLED 1% AEP MAXIMUM FLOOD VELOCITY DIFFERENCE (MITIGATED – EXISTING)

11.6 Erosion Protection Measures

The results of the analysis indicate flow velocities within the site are extremely low (<0.5 m/s) which is unlikely to result in erosion or subsequent downstream sedimentation. However, it is noted that during the 2% and 1% AEP events, velocities increase from <0.25 m/s to 0.5 m/s on the southern side of the conveyor alignment. Water Technology recommends re-visiting the conveyor once detailed designs of the conveyor are provided (i.e., elevations and culvert dimensions) to ensure erosion is unlikely to occur along this alignment. In the event that rip-rap is required (following further analysis), erosion protection specifications can be extracted from Table 11-5 which provides the required class of protection and the thickness of rock/rip-rap based on velocities at the area of concern. These erosion specifications are based on the Main Roads of Western Australia Floodway Design Guidelines (2006).

TABLE 11-5 EROSION PROTECTION SPECIFICATIONS

Velocity (m/s)	Class of Protection	Section of Thickness
<2	None	-
2 – 2.6	Facing	0.5
2.6 – 2.9	Light	0.75
2.9 – 3.9	¼	1.0
3.9 – 4.5	½	1.25
4.5 – 5.1	1.0	1.6

12 LIKELIHOOD OF EXTREME EVENTS

When considering management options, it is prudent to consider the likelihood that a particular size event (e.g., a 10% AEP event) will occur during the design life of the infrastructure. Table 12-1 summarises probabilities for several possible design scenarios, according to Equation 11.3 of ARR87.

Given an expected active operational period of 50 years, it is likely that a significant flood event will impact on the site. For example, the probability of a 1% AEP event occurring during the 10-year period is approximately 40% and the probability of a 10% AEP event occurring is 99%.

Current designs propose the height of the embankment bunds are set at the 2% AEP water level. According to Table 12-1, if the life of the project is 50 years the embankments have a 64% chance of being exceeded.

However, it should be noted that all solar salt projects are designed to be flooded at some point (by embankments being overtopped) given the costs of building extremely high embankments. The consequences of embankments being overtopped are a disruption to salt production, but environmental impacts of such flooding are unlikely given the huge volumes of water involved causing significant dilution of any saline water released from the ponds.

The likelihood of extreme events and flooding of the salt ponds should be considered further during detailed engineering design.

TABLE 12-1 PROBABILITY OF EXCEEDANCE VERSUS DESIGN LIFE

Project Life	Annual Exceedance Probability			
	10%	5%	2%	1%
10	65%	40%	18%	10%
20	88%	65%	33%	18%
50	99%	92%	64%	40%
100	100%	99%	87%	63%

13 CONCLUSION AND RECOMMENDATIONS

Water Technology conducted a catchment and flood assessment to estimate potential impacts associated with the proposed development and determine flood mitigation measures to reduce flooding impacts.

A detailed regional MIKE21FM hydraulic model and a refined local TUFLOW hydraulic model were developed of the study area and surrounding catchments (i.e., Yannarie River, Ashburton River and Rouse Creek) using the best available data. The intertidal areas and tidal flows of Urala Creek North and South were also included in the model domain.

The regional MIKE21FM hydraulic model was calibrated to satellite imagery from Tropical Cyclone Vance that passed across Exmouth Gulf in 1999. Design inflows from this regional model were then fed into the local TUFLOW 2D hydraulic model. The TUFLOW model was used to define flood movement through the proposed development site for a range of design events (10%, 5%, 2% and 1% AEP) to determine hydrological impacts at the site.

Overall, hydrological impacts are localised . The following trends were noted during all modelled flood events:

- During the modelled pre-development 10%, 5%, 2% and 1% AEP events, flooding is mainly confined to the existing overland flow paths and basins with the most significant increases in water levels (existing conditions) occurring along the north-eastern margins of the proposed project area.
- Modelled pre-development flood behaviour changes slightly along the eastern margins of the proposed project during events >2% with overtopping occurring in the basins resulting in water ponding to depths up to 6 m in the 1% AEP.
- Modelling of the unmitigated post-development scenario indicates that flow velocities in vicinity of the proposed site infrastructure are negligible (<1 m/s) and as such erosion protection is not required. Low velocities are due to gentle/flat gradients and the low energy tidal driven surface water movement across most of the site.
- Modelling of post-development conditions showed the need for culvert crossings at several locations along the main access road and conveyor embankment to allow movement of surface water back onto its natural flow path. Subsequently Water Technology sized culverts to convey surface water runoff under the main access road for the 10% AEP. At the conveyor the culvert size was limited to a fill height of 1.0 m and as such box culverts 1.0 m high were recommended. The recommended culvert locations, sizes and numbers were provided to K+S for the conveyor and main access road.
- Hydraulic modelling (under the 10%, 5% and 2% AEP events) of proposed culverts showed a significant reduction in water levels upstream of the main access road and the conveyor, with only localised ponding remaining. Such localised ponding is unlikely to have any deleterious environmental effects, given it simply mimics the local flooding regime. However, it is recommended that this ponding is considered from an engineering perspective during detailed design, and if necessary, optimisation of mitigation strategies is undertaken to meet required engineering standards.
- Hydraulic modelling of proposed drainage diversions adjacent to the south-eastern corner of the site showed modest reductions (0.1 m) to water levels within the south-east basin. Modelled post-development water levels within the south-eastern basin are unlikely to have a negative environmental impact – it will simply mean flooding duration is increased which may be environmentally beneficial given the flooded basins are habitat for waterbirds and invertebrate fauna. However, it should be considered during detailed engineering design whether post-development water levels within the basin have the potential to compromise engineering standards or production and if so, further optimisation of the drainage diversions is recommended during detailed design.
- In addition, ponding within borrow pits planned to be constructed along the eastern margins of the site should also be considered during detailed design. The locations of the borrow pits were not available at

the time surface water modelling for this project was conducted, however it is expected that any ponding of water within them will be an issue for engineering consideration, rather than an environmental concern, given flooding of the borrow pits will simply mimic the flooding regime of local basins (which may provide fauna habitat when flooded) and therefore may be environmentally beneficial.

- The Ashburton River bridge crossing was modelled in TUFLOW as a 'Layered Flow Constriction' (LFC). Modelling results from the 2% AEP suggest water levels will reach a maximum height of 4.25 m AHD and flow velocities will be low (<0.5%) both upstream and downstream of the bridge crossing. More detailed hydrodynamic modelling is recommended once survey information upstream and downstream of the bridge is made available.

Although the flood mitigation strategies discussed above are suitable for the preliminary design phase and environmental impact assessment to support the ERD, during the final engineering design phases Water Technology recommends further work to determine if optimisation of flood mitigation strategies are required to meet engineering standards.

It should also be noted that the modelling presented in this report depict a worst-case scenario of the proposed development and generally provides an overestimation of potential impacts. This is due to the model assuming catchment generated surface water flows and tidal driven surface water flows arrive at the same time; this is a conservative estimate.

This model can be refined in the future to incorporate the intricacies of the internal hydraulic infrastructure (i.e. culverts and diversion drains) contained within the site and expanded to include additional scenarios where tidal and catchment driven floodwaters do not arrive concurrently. By incorporating, and further enhancing the representation of the hydraulic structures (within the site) and modelling scenarios, it is expected that the impacts illustrated within this report will reduce. Therefore, it is recommended that a localised model of the development is considered during the detailed engineering phase.

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