MARDIE SURFACE WATER ASSESSMENT

Mardie salt project

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1 INTRODUCTION

1.1 Background

BCI's proposed Mardie Salt Project involves the production of 4.0 million tonnes per annum of sodium chloride salt from a seawater intake and series of solar evaporation ponds.

A scoping study has been completed to prove the technical and economic feasibility of the project, as well as a pre-feasibility study. This report provides additional modelling details and assessment based on the preferred design.

The proposed evaporation ponds are located on mud flats on the landward side of the coastal mangrove areas and stretch over 25 km of coastline. Several creeks flow through the area that will be occupied by the evaporation ponds.

Among the potential impacts to the pond areas are changes to the surface water hydrology, and the requirement for flood protection (bund walls, diversion drains, scour protection measures, etc). Mangroves, fringing mudflats and algal mats are sensitive habitats, and the project will need to demonstrate that impact to these habitats can be minimised or mitigated through appropriate design.

As part of this assessment, RPS has undertaken hydraulic modelling of stormwater flows to support the preliminary engineering design. This report focuses particularly on the drainage corridors which run between the ponds and the rear-of-pond locations where it proposed that flood protection infrastructure (e.g. bund associated with the access road alignment, floodways and lateral drains) will be provided to mitigate flood impacts to the salt ponds and gas pipeline infrastructure. It also investigates the impact to flow regimes and flooding that the proposed development will have on the tidal creeks and mangrove areas located downstream of the ponds.

1.2 Scope of services

A desktop surface water assessment was undertaken to assess the options and requirements for preferred surface water management design for the proposed salt ponds. The objective was to develop the relevant surface water scenarios and provide preliminary information on hydraulic and engineering parameters associated with surface water management infrastructure, as well as potential hydrological impacts to the sensitive downstream environment.

The report addresses the following:

- Characterise and describe the existing surface water environment, including climate, location and size of catchments, existing drainage conditions and flow directions;
- Identify key surface water management issues and hydrological risks associated with the proposed development, particularly potential impacts from local creek lines affecting the proposed salt pond infrastructure;
- Estimation of catchments and associated flood flows at key locations throughout the site;
- A conceptual flood mitigation design to provide the required level of flood protection including details of the flood bunds, diversion drains, floodways and drainage corridors; and
- A preliminary assessment of the potential hydrological impacts from the project on downstream sensitive environments (algal mats, mangroves etc).

1.3 Definitions

100 year ARI flood - the flood event having an average recurrence interval (ARI) of 100 years. It has a 1% chance of occurring or being exceeded in any one year (e.g. a 1% Annual Exceedance Probability, AEP).

The 50 year ARI flood has a 2% chance of being exceeded in any one year (i.e. 2% AEP), a 10 year ARI flood has a 10% chance of being exceeded in any one year (i.e. 10% AEP), and so on.

Floodplain - The portion of a river valley adjacent to the river channel which is covered with water when the river overflows its banks during floods.

1.4 Location

BCI's proposed Mardie Salt Project is located between the Robe River and Fortescue River mouths in the north-west of Western Australia (Figure 1).

Figure 1: Location

2 HYDROLOGY

2.1 Seasonal rainfall and evaporation

WA has three broad climate divisions - the south-west corner of WA with a Mediterranean climate, with long hot summers and wet winters; the central eastern areas of WA with arid land or desert climates and the area of interest, the dry tropical northern part of the State, receiving summer rainfall.

The average annual rainfall at nearby Mardie Station is 277 mm (BOM, Site number 5008) as measured over a 129 year period (1885 - 2017), but annual rainfall is highly variable with a minimum of 9mm recorded, and a maximum of 886mm.

The majority of rainfall occurs January-June (38-63 mm average monthly rainfall), and July-December is typically drier (average monthly rainfall 1-9 mm).

There is limited evaporation data available, but the annual Class A pan evaporation at Mardie, as estimated by BCI, is about 3,250 mm per annum, varying from 12 mm/day in summer to 5mm/d in winter.

2.2 Intensity frequency duration (IFD)

Intensity-Frequency-Duration (IFD) data is required to characterise the storm intensity in the area under consideration. This is generally provided by techniques in ARR (Australian Rainfall and Runoff), a national guideline for the estimation of design flood characteristics in Australia, published by the Institution of Engineers Australia. New IFD design rainfalls were produced in 2016.

Typical IFD data for this area is as follows:

Table 1: IFD Data (rainfall depth in mm)

ARI	1 year (mm)	2 year (mm)	5 year (mm)	10 year (mm)	20 year (mm)	50 year (mm)	100 year (mm)
1 hour	23	41	41	50	59	73	83
2 hour	29	51	51	64	77	95	109
6 hour	39	70	75	95	117	149	174
12 hour	47	87	95	124	155	198	233
24 hour	57	106	119	156	196	251	296
72 hour	72	134	148	192	238	301	354

Information on storms exceeding the 100 year ARI event is not (readily) available in ARR, but by extrapolation, estimates can be made. The 1000 year ARI and Probable Maximum Precipitation (PMP) rainfalls are in the order of 1.7x and 3.3x the 100 year rainfalls respectively.

The (sliding, not calendar day) 24 hour rainfalls are estimated as:

- 2 year ARI 106 mm
- 5 year ARI 120 mm
- 10 year ARI 160 mm
- 20 year ARI 200 mm
- 50 year ARI 250 mm

- 100 year ARI 300 mm
- 1000 year ARI 500 mm
- PMP (Probable Max. Precipitation) 1,000 mm

2.3 Flood flow estimation

2.3.1 Regional context

The catchment details for the development area are shown in Figure 2. The project area itself is located on terrain gently sloping from the North West Coastal Highway to the north west at a low 0.15-0.20% gradient. whilst the proposed salt ponds are located on very flat terrain associated with the tidal mud flats.

Based on local rainfall and runoff trends for the area, the flood flows (as a proportion of the 100 year ARI flood) would typically be:

Table 2: Typical Presumptive Flood Flows as Proportion of the Q100 Flood

ARI (years)	Fraction of Q100 flood
2	0.05
5	0.15
10	0.28
20	0.45
50	0.73
100	1.0
1000	~2.1
PMF	~6.3

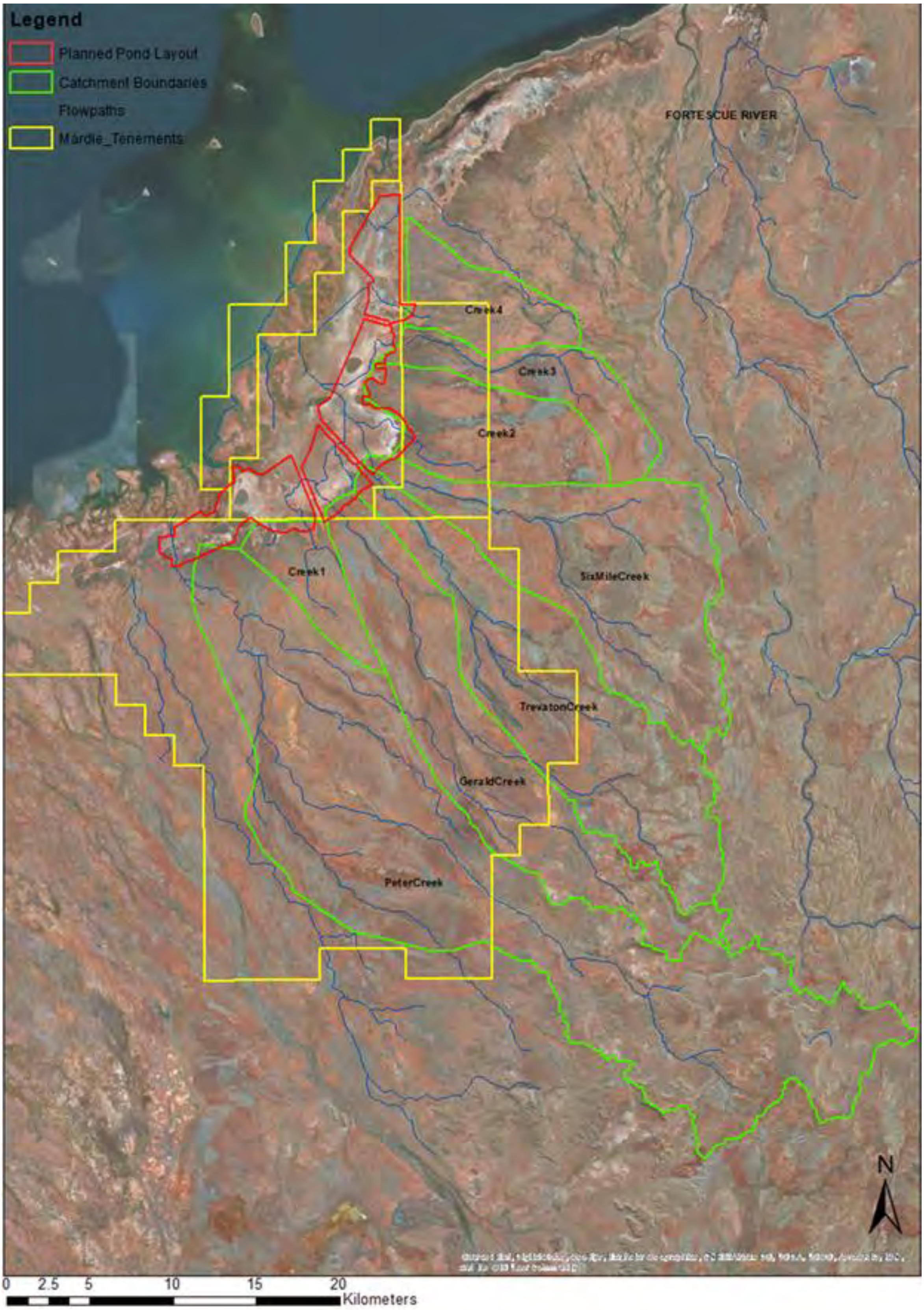


Figure 2: Surface water catchments

2.3.2 Hydrological modelling

There are no relevant streamflow gauging data / gauged catchments from which flood estimates may be made directly. Flood estimation therefore relies on Australian Rainfall and Runoff (ARR) flood estimation methods for ungauged catchments, or an individual customised rainfall runoff model for each catchment.

In this case the RAFTS nonlinear rainfall / runoff program has been used. RAFTS uses design rainfall data derived from ARR. RAFTS requires customising for each catchment (parameters include terrain slopes, roughness, local rainfall data and rainfall losses). The catchments for the relevant creeks were divided into sub-catchments, with routing links between. The program calculates flood flows (hydrographs) by simulating rainfall over a catchment with time, removing losses to calculate the rainfall excess runoff, and then routing this runoff through the model reaches. The RAFTS 'pern' or surface roughness factor affects the storage factor and was set at 0.045.

2.3.3 Peak flow estimates

The RAFTS hydrological model provided flood flow hydrographs for a range of design rainfall IFDs. For each ARI, the critical rainfall event duration (producing the highest peak flow rate) was identified and used for subsequent hydraulic modelling. The peak flow rate for each ARI is provided in Table 3.

Table 3: Estimated Flood Flows (m^3/s)

ARI (years)	Ac (km^2)	2 ARI (yrs)	5 ARI (yrs)	10 ARI (yrs)	20 ARI (yrs)	50 ARI (yrs)	100 ARI (yrs)	1000 ARI (yrs)	PMF
Peter Ck	422	27	80	149	240	389	533	1,119	3,356
Gerald Ck	153	16	49	91	146	236	324	679	2,038
Trevarton Ck	172	18	55	103	165	268	367	771	2,314
6 Mile Ck	164	19	56	104	167	271	372	780	2,341
Fortescue River	18,360	1,090	2,850	5,000	8,080	13,500	20,000	42,000	126,000

The 100 year ARI local flows may be generally estimated as Q100 = 36 x Ac0.45 (where Ac = catchment area in km^2) based on typical RAFTS estimates.

2.3.4 Haul Road corridor hydrology

A previous study by RPS was carried out on a haul road from the proposed Bungaroo South mine to the Cape Preston area for Iron Ore Holdings (ref: Buckland Project Haul Road Corridor Hydrology, April 2014, RPS 1488T/003a). The haul road route initially headed west across hilly terrain in the Hamersley ranges to the North West Coastal Highway. From the highway, the road route continued north in undulating terrain, generally paralleling the highway alignment to the west, and following the Dampier Bunbury Natural Gas Pipeline alignment.

Some of the haul road creek crossings flow north west and through the current Mardie Salt area of interest. These include:

- Robe River – the river has a catchment area of 7,100 km^2 at the Yarraloola gauging station installed near the North West Coastal Highway bridge. The largest flows recorded (Cyclone Monty in Feb/Mar 2004 and the tropical depression over the Pilbara in February 2009 overtopped the Robe highway bridge.

- Peter Creek (Catchment 95) – bridge at the highway, with a catchment area of 188km^2. The 10 year flood was estimated as 110 m^3/s, and the 100 year flow as 515 m^3/s;

- Gerald Creek – not included in the previous study, as this creek does not cross the haul road per se, but forms downstream, possibly gaining flow in very flat terrain from breakout flows from Trevarton Creek, and less likely Peter Creek;

- Trevarton Creek (Catchment 100) - floodway at the highway, with a catchment area of 96 km². The 10 year flood was estimated as 71 m³/s, and the 100 year flow as 324 m³/s;
- 6 Mile Creek (Catchment 109) - small culvert at the highway, with a catchment area of 53km². The 10 year flood was estimated as 48 m³/s, and the 100 year flow as 216 m³/s;
- Various unnamed creeks north of 6 Mile Creek –not included in the previous study, as they do not cross the haul road. These creeks form downstream, as local runoff, but also potentially as part of the channel system draining the broader Fortescue River floodplain (west of the main channel); and
- Fortescue River (Catchment 117) - 400m long, high level bridge at the highway, with a catchment area of 18,360 km². The 10 year flood was estimated as ~5,000 m³/s and the 100 year flow as ~20,000 m³/s. The haul road crossing is on a new alignment, crossing the river on a floodway downstream of the road bridge. The river flows were measured at the "Jimbegnyinoo Pool" just upstream of the road bridge, and now at "Bilanoo" at the road bridge gauging station. The same rain events that overtopped the highway at the Robe River bridge also overtopped the Fortescue River bridge.

2.3.5 Fortescue River break-out

Part of the Mardie Salt site is potentially impacted by “breakout” flows from the Fortescue River during major flood events.

Upstream from the North West Coastal Highway, the Fortescue River is generally contained between ridges. However, downstream of the highway, the topography becomes less pronounced and the river flow path less constrained. On the west side of the main river channel, there is a noticeable north-south ridge line at about RL30-40m elevation. The river floodplain at this point is generally 5 km wide, with numerous smaller flow channels developed, discharging in the same general direction as the main channel.

However, during large flood events, river flows can “break-out” from the main floodplain. There is a significant “break-out” area between the north end of the ridgeline and Coolangara Hill (a small hill 15 km north of the highway, elevation ~RL45 m) which encroaches into the main floodplain and redirects high level flood water upstream away from the main river channel system. The floodplain east of the hill then reduces to about 4 km wide.

Break out flows generally head north-westerly towards the coast 25km away. Flows eventually exit to the ocean, at anywhere up to 25 km west of the Fortescue River mouth.

A significant volume of flow would be diverted away from the main Fortescue channels in the largest floods. The Department of Water and Environmental Regulation (then WRC) previously estimated a 100 year ARI flood flow of 9,220 m³/s, with around 1,200 m³/s of that flow (i.e. 13%) following channels north and north-west to the sea, west of the main channels. It is not possible however to estimate the quantum of break out flows without 2D hydraulic modelling over a very large area.

For a now estimated 100 year flow of about 20,000 m³/s, the break out flows may be assumed as up to 20%, or 4,000 m³/s, “lost” from the Fortescue River system. This high flow is spread over a very large area, and the direct impact at any location (other than in a flow channel) would be anticipated as relatively low. The impact at the coast in the larger Fortescue flows would probably be north of 6 Mile Creek and has not been included in the design scenario modelling in this study.

A sensitivity scenario was modelled as part of this study which included breakout flow, to assess the potential impact behind the salt ponds. A breakout flow of 1,000 m3/s was simulated as impacting the project area (most of the breakout flow is likely to occur further north of the project area). The impact on 100 year ARI flood levels along the rear bund when including the breakout flow was minor (<0.1 m) for all locations, except at the very northern end of the site, where flood levels increased by up to 0.3 m. The results indicate that breakout flow from the Fortescue River is not likely to play a major role in the flood mitigation design for the project.

2.4 Coastal inundation

2.4.1 Sea levels

Normal tidal variations cause inundation over the coastal flats. Satellite and time-lapse photography indicate that flood overflow of the tidal creeks starts at around 1.1 – 1.2 m above mean sea level (MSL), with the tide level varying over the area.

Mean neap tide levels vary around +/-0.5 m from MSL, and spring tide levels vary around +/-1.8 m (and up to 2.2 m in the far north during king tides). The highest and lowest astronomical tides (HAT to LAT), which are the highest and lowest tidal levels which can be predicted to occur under average meteorological conditions, vary by approximately +/-2.4 m from mean sea level.

Under abnormal meteorological conditions, greater variations in the tidal range are possible, and actual still water sea levels are produced by the interaction of astronomical tides, storm surges and wave set-up.

The Pilbara coast cyclone season runs from December to April, peaking in February and March. Potentially the most destructive phenomenon associated with cyclones that make landfall, is storm surge, a raised mound of seawater typically some 50 km across, and up to several metres higher than the normal tide. The worst scenario arises when a severe cyclone crosses a coastline with a gently sloping seabed, at or close to high tide.

RPS undertook a Metocean analysis as part of the Pre-feasibility Study to provide estimates of still water level for various return periods. The estimated 100 year still water sea level is RL4.2 - 4.3 m, about 2 m higher than HAT. The 10 year sea level is 3.5 - 3.7 m, 1.3 m higher than the HAT. These sea levels would flood the coastline inland for several km from the mean sea level (RL00 m) location.

2.4.2 Flood level joint probability estimates

The evaporation ponds will be impacted both by creek flooding and coastal inundation, and any flood works should account for both.

The flood level in the ocean is an end / downstream condition which is required when hydraulic modelling flood flows in the various creeks - a joint probability situation. Flooding of infrastructure located near the coast can be impacted either by creek flooding from the inland side, or high sea surge levels from the ocean side.

In this regard, it is noted that the largest river floods in the Fortescue River, and ocean storm surges both occur as a result of tropical cyclone activity. Generally, a cyclone related flood in the river would occur sometime after any associated abnormal sea level (the height of which can vary greatly), as the cyclone tracked across the coast and moved inland. Hence significant storm surge and river flooding are not dependent, and do not generally occur simultaneously.

The creeks of interest are much smaller than the Fortescue River, and the smaller catchments near the coast are likely to increase the degree of dependence a little between the two flood mechanisms.

A common way of handling this joint probability between the two flood mechanisms is provided in, for example, the “Flood Risk Management Guide” (NSW Department of Environment, Climate Change & Water 2010/759, August 2010). This approach adopts a probability ratio for the two flood mechanisms of 1:5, i.e. assuming 20 year ARI catchment flooding in conjunction with 100 year sea levels, or 100 year catchment flooding in conjunction with 20 year sea levels. The “Karratha Coastal Vulnerability Study” (JDA, August 2012) studied the joint probability between river flood levels and storm surge in the Karratha area and found no obvious correlation; that study therefore adopted the 100 year catchment flood flow in conjunction with the 20 year sea level (estimated as RL3.9 m) as the downstream boundary condition.

3 SURFACE WATER IMPACTS

3.1 Overview

Regional stream flow in the Pilbara is ephemeral, related to intense rainfall from cyclonic activity or localised thunderstorms. Stream flow decays rapidly once rainfall has ceased, with negligible base flow.

The proposed infrastructure is comprised primarily of salt evaporation ponds which extend along approximately 25 km of coastline. The project area is situated at the downstream end of several creek system catchments, at the point of creek discharge to the coastal mudflats. The relevant creek catchments range between 33 - 422 km^2 in size. The salt pond design will need to facilitate drainage of these creeks through, or around, the salt ponds to the ocean.

The terrain on which the salt ponds are proposed to be built is extremely flat, with surface slopes in the order of 0.01% (1 in 10,000).

3.2 Infrastructure impact on surface water

Based on the layout of the proposed salt ponds (Figure 3) in relation to surface water:

- The salt ponds occupy a significant proportion of the coastal mudflats to which the local creek systems discharge;
- The overall footprint of the salt ponds intercepts four named creeks (Peter Ck, Gerald Ck, Trevarton Ck and Six Mile Creek), as well as several smaller creeks;
- The salt ponds will need to provide drainage corridors to convey flows through or around the salt ponds to the ocean;
- The current salt pond layout provides two drainage corridors through the salt ponds which are generally aligned with the larger creek systems; lateral diversion drains will also be required to intercept the other smaller creeks and isolated catchments to convey these flows to the drainage corridors, or to the north or south of the salt ponds; and
- The drainage corridors will have the effect of concentrating flood flows to fewer points of discharge to the coastal mudflats, which will also be in closer proximity to the algal mats, tidal creeks and mangroves that comprise the downstream environment.

3.3 Surface water impact on infrastructure

The local creek systems convey flows in a north-westerly direction towards the rear (landward) boundary of the proposed salt ponds (refer Figure 3).

- The four named creeks have an estimated 100 year flow of 324 – 533 m^3/s, and a PMF of about ~2,000 – 3,300 m^3/s;
- The creek channels discharge to the coastal mudflats which are situated just within the proposed salt pond footprint, the salt ponds will need to be protected from freshwater inflow by diversion drains and levees; and
- The larger creeks are expected to flow at least 3 – 4 m deep in the 100 year flood, and the smaller creeks around 1 - 2 m deep, based on the results of hydraulic modelling.

4 FLOOD MITIGATION CONCEPT

4.1 Post development hydraulic modelling

The hydrological assessment methodology and results (e.g. flow hydrograph estimation) has previously been described in the Pre-Feasibility Surface Water Assessment report. The same flow hydrographs for various design events (e.g. 10, 20, 50, 100 year ARI) were used in this modelling exercise and input to the upstream boundary of the 2D model at the location of each major creek – Peter Creek, Gerald Creek, Trevarton Creek, 6 Mile Creek and the three other creek catchments identified in the hydrology assessment which flow into the project area from the southeast.

The same XPSWMM hydraulic model as used in the Pre-Feasibility Surface Water Assessment (RPS 2018, Appendix D) was used for this scope of works. A12.5 m grid cell size was used for the predevelopment hydraulic modelling. The post development modelling was initially run with a 12.5 m grid size, however it this was changed to a 25 m grid size due to negligible changes in results, faster run time and greater model stability. This is consistent with sensitivity testing reported in previous reports

The 20, 50 and 100 year ARI models adopted a low sea level boundary condition (0.5 mAHD) in order to simulate "worst case" flow velocities along bunds and through drainage corridors etc (i.e. a high sea level state boundary condition could possibly result in higher tailwater conditions and thus lower flow velocities). The 10 and 1 year ARI model results presented are based on a sea level boundary condition of 0 mAHD. Sensitivity testing undertaken in earlier modelling (appended) identified that modelling with a higher downstream tailwater condition only affected the modelled flood levels at the downstream side of the ponds where flood levels are governed by the assumed tidal / storm surge; the tailwater level does not have a significant impact on upstream flood levels.

The model was run with:

- A bund/wall between the North-South access road and the ponds, modelled with a height greater than the 100 year ARI event (refer to Figure 3). Floodwater can overtop the access road bund and flow into the ponds at modelled floodway locations or flow along the bund to the points of discharge which are via the western-most extent of the ponds or via the Drainage Corridors B and C (between ponds 3 and 4, and between Ponds 7 and 8);

- "lateral drains" which are nominal 28 m base width drains alongside the flood bund at selected elevated locations (i.e. where lateral flow behind the flood bund is likely to be improved by the inclusion of the drains). A nominal 28 m base width for the lateral drains was adopted on the basis of preliminary advice from the project engineers regarding the cut-to-fill balance as well as sensitivity modelling reported in previous reports (appended) which investigated the relationship between the lateral drain width and the effectiveness of the drains for reducing flood levels behind the flood bund. Lateral drains were modelled with an invert graded between the low points in the natural surface along the rear salt pond alignment (i.e. the base of the drain coincides with natural surface level at the low-lying areas (such as creek beds) and is cut into the terrain in the more elevated areas). The depth of the drain is therefore highly variable, depending on the surrounding ground levels. These lateral drains are modelled as 1D "links" for greater accuracy;

- "floodways" where floodwater can overtop the bund via 300 m long floodways at selected low points in the terrain and flow into the rear of the salt ponds. The floodway scenario was modelled with floodway elevations set at the 50 year ARI top water level, which means that discharge into the ponds will only occur in flood events of greater magnitude than 50 year ARI; and

- Drainage corridors B and C have been modelled as 300 m and 250 m wide, respectively, with the adjacent pond levels set as infinite walls (as the final design of pond walls will provide freeboard to flood levels). The 250 - 300 m corridor width was adopted on the basis of sensitivity modelling reported in previous reports (appended) which investigated the relationship between drainage corridor width and flood heights within and upstream of the drainage corridor.

An overview of the key site attributes and their relative locations is provided in Figure 3. Figure 4 shows where the proposed wall alignment for KTMS Crystalliser Ponds blocks the main flow channel of the creek that discharges via that location; for the purpose of this scope of works a 50 m wide cut was modelled to maintain a flow path to the drainage corridor. The 50 m wide cut drain resulted in a modelled peak velocity through the cut of 1.5 m/s in the 100 year ARI event (1.1 m/s in the 20 year ARI event). A wider cut may be required depending on the scour potential of the material and the scour protection requirements.

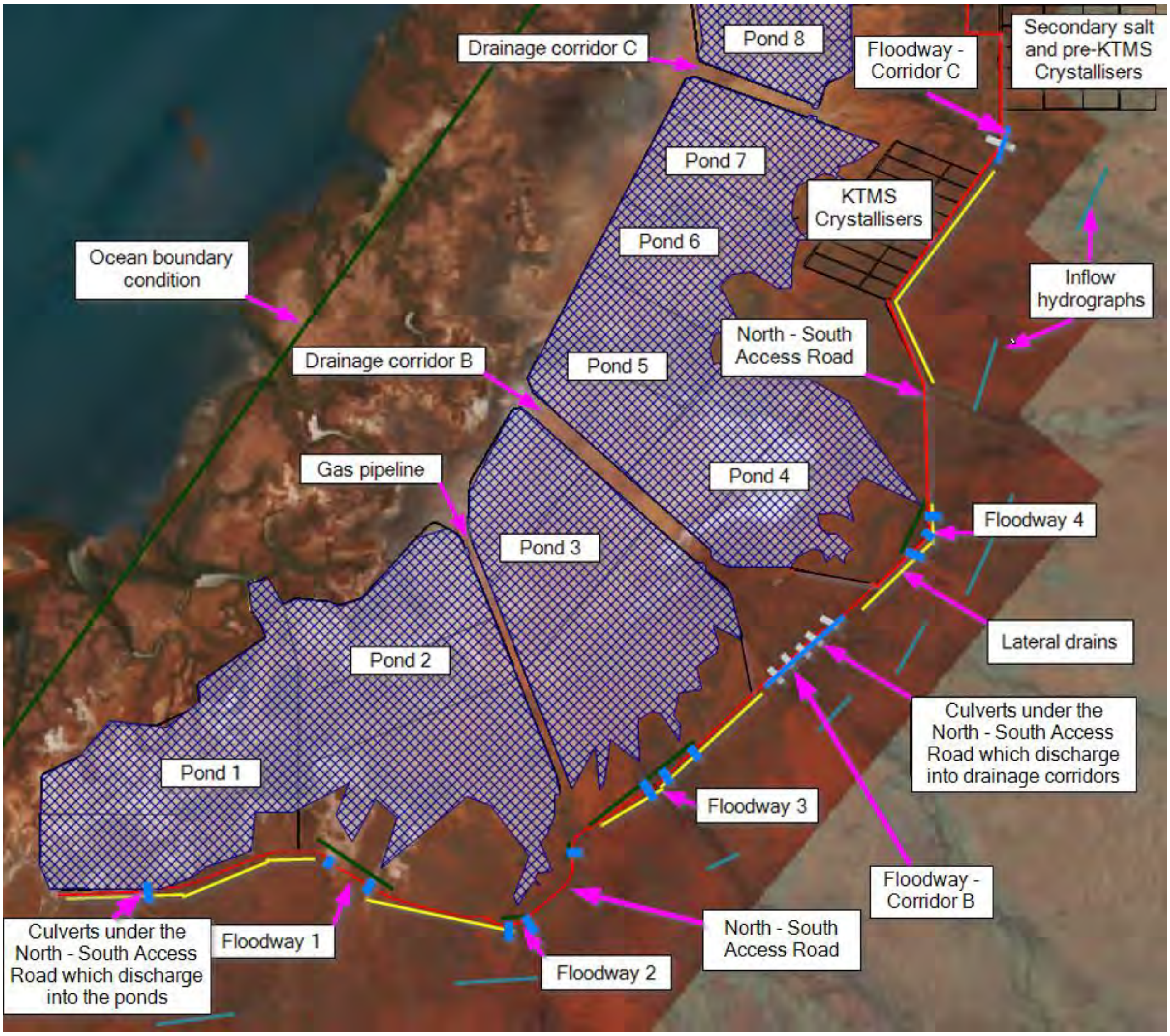


Figure 3: Site overview

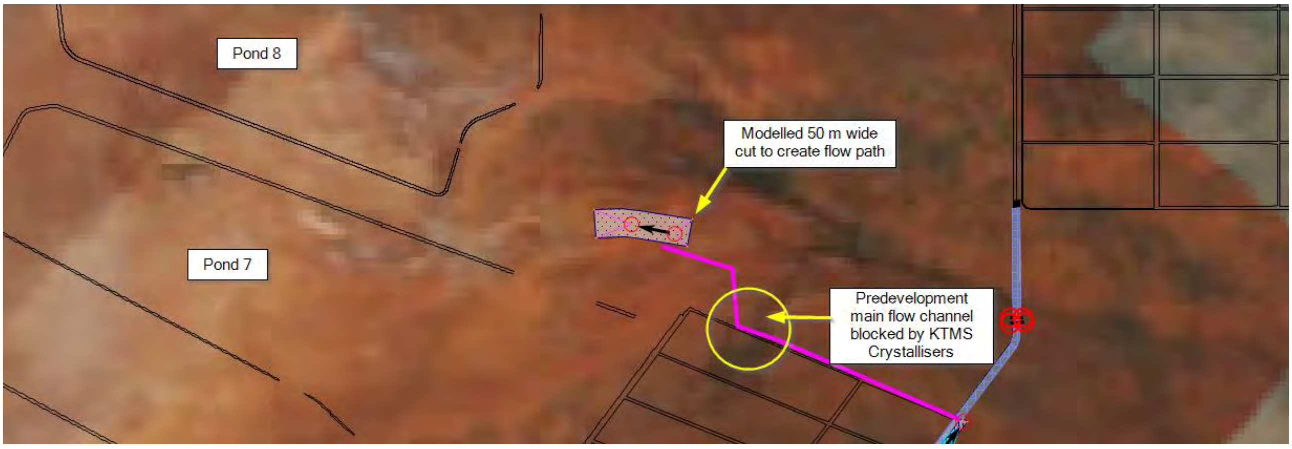


Figure 4: KTMS Crystallisers interface with flow channel

4.2 Post development flood depth, height and hazard modelling results

Post development maximum flood depths, flood elevations and maximum hazard (velocity x depth) figures are presented below for the 1, 10, 20, 50 and 100 year ARI events.

The maps show that the maximum flood depths (in the creek channels upstream of the flood bund) in the 1, 10 and 100 year ARI event are approximately 1, 2.5 and 4.5 m respectively, with lower depths occurring downstream of the bunds within the drainage corridors and downstream of the ponds.

Flood heights upstream of the flood bund are variable along the length of the bund, with the highest levels occurring midway along the bund corresponding with more elevated terrain, and lower levels occurring at the drainage corridors or southern end of the bund where floodwater discharges to the downstream mudflats. The flood heights at Drainage Corridor C are higher than those at Drainage Corridor B due to the former being located in naturally more elevated terrain, as well as the design of the Corridor C floodway which is elevated to facilitate larger culverts beneath the floodway (the larger culverts at Corridor C have been incorporated to convey small rainfall beneath the floodway thus avoiding disruption to the brine transfer pipe/channel which runs along the floodway alignment). The 100 year ARI flood height is generally approximately 0.5-1.0 m higher than the 10 year ARI flood height.

Hazard (the velocity-depth product) mapping results show that the velocity-depth product is generally low (~0.3 m^2/s) outside of the main flow paths. Corridor B has a notably higher hazard risk with a value of around 1 m^2/s in the 10 year ARI event and 3 m^2/s in the 100 year ARI event (and as high as 5.5 m^2/s through the narrow section / services crossing). These values reflect the increased depth and velocity of flow that is necessary to convey the converging floodwaters from Gerald, Trevarton and Six Mile Creeks.

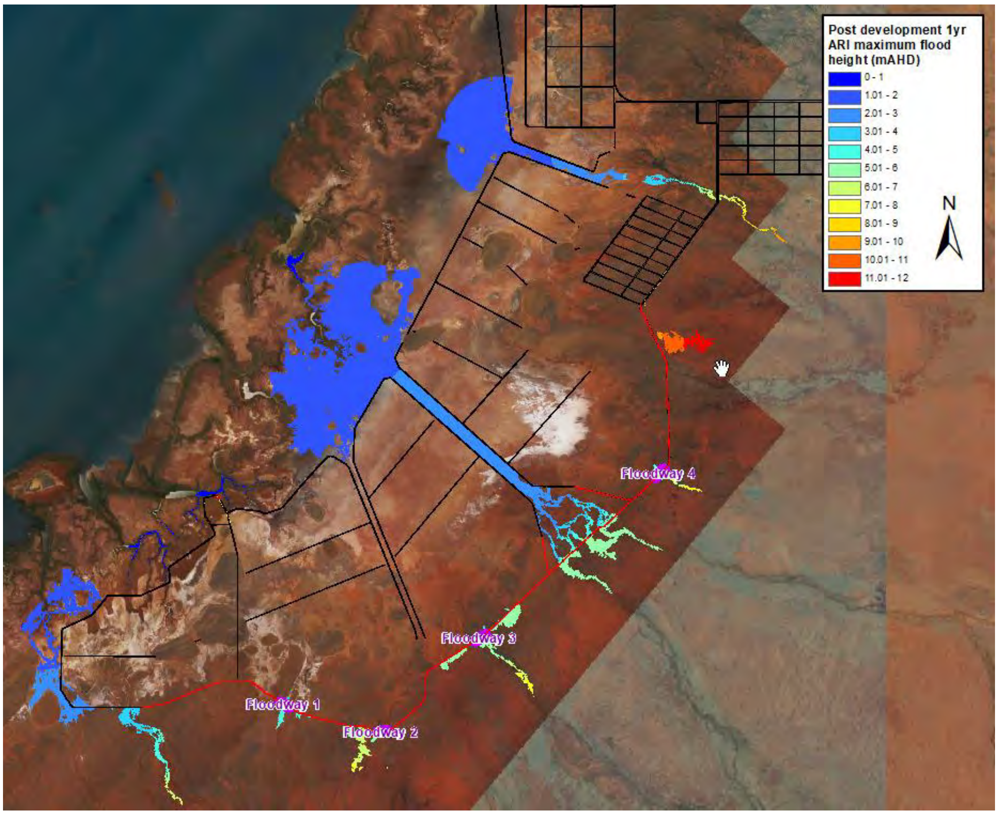


Figure 5: Post development 1 yr ARI maximum flood height (mAHD)

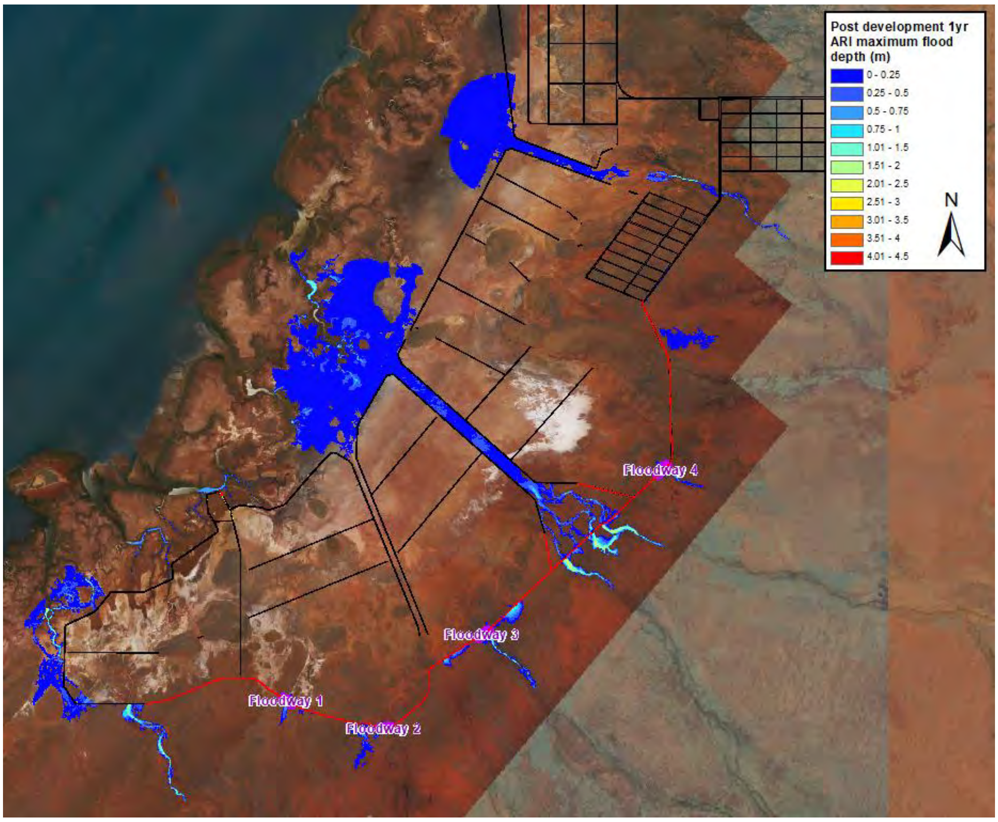


Figure 6: Post development 1 year ARI maximum flood depth (m)

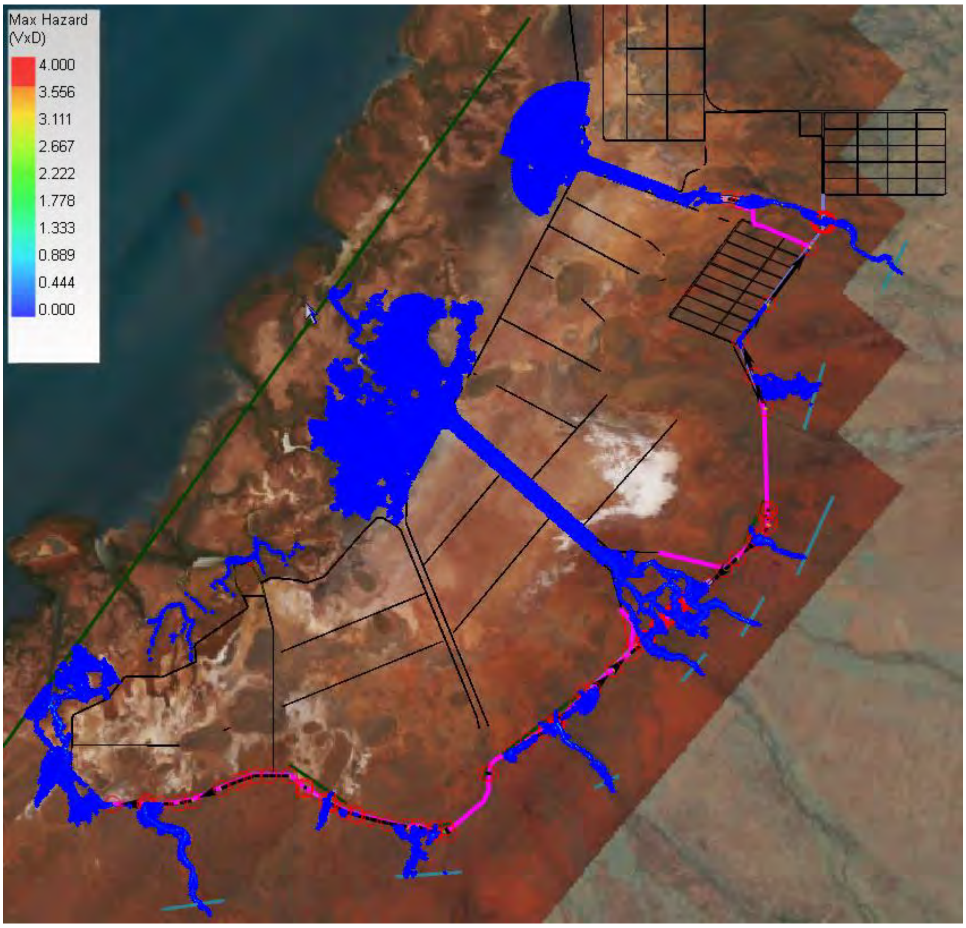


Figure 7: Post development 1 year ARI maximum hazard (velocity x depth) (m^2/s)

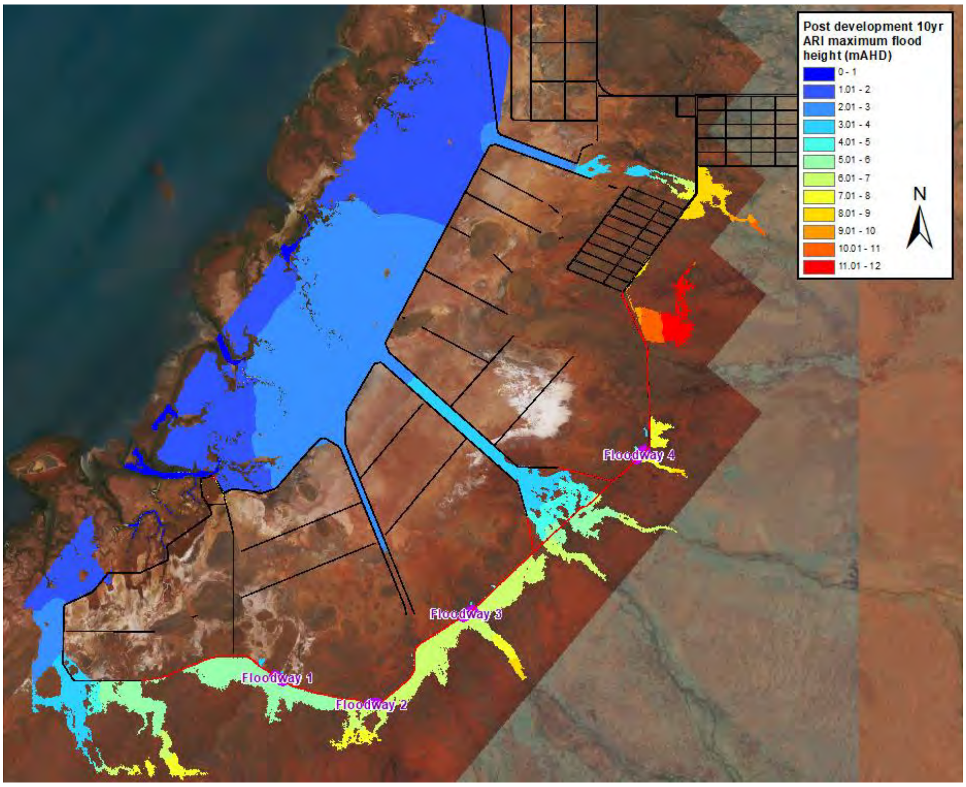


Figure 8: Post development 10 year ARI maximum flood height (mAHD)

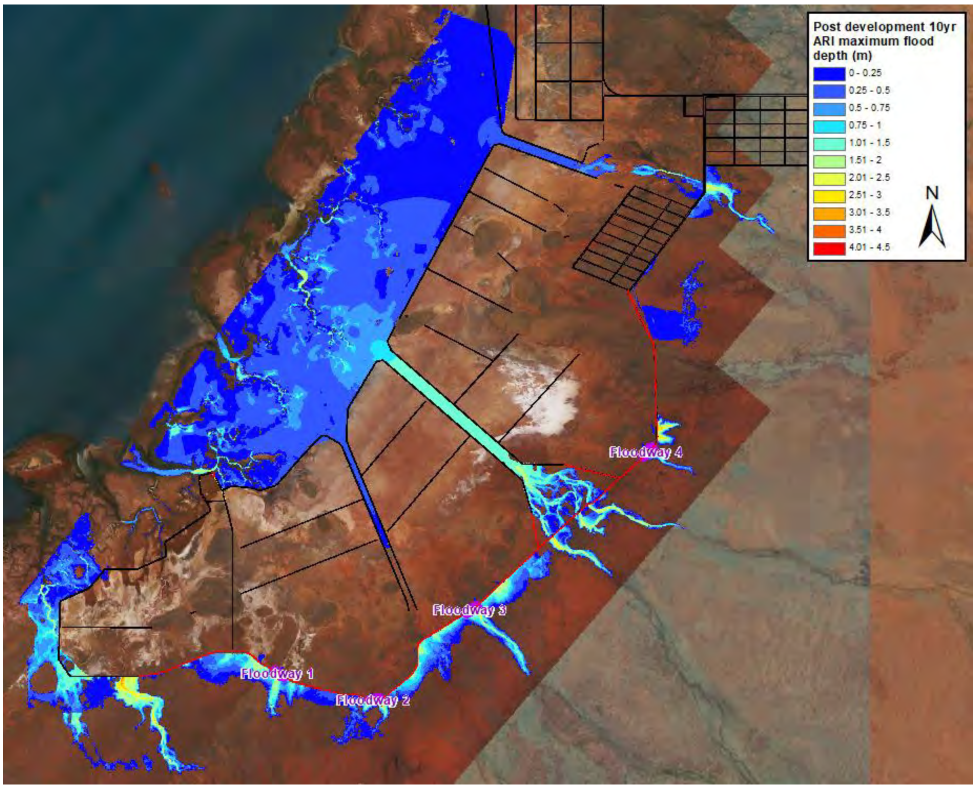


Figure 9: Post development 10 year ARI maximum flood depth (m)

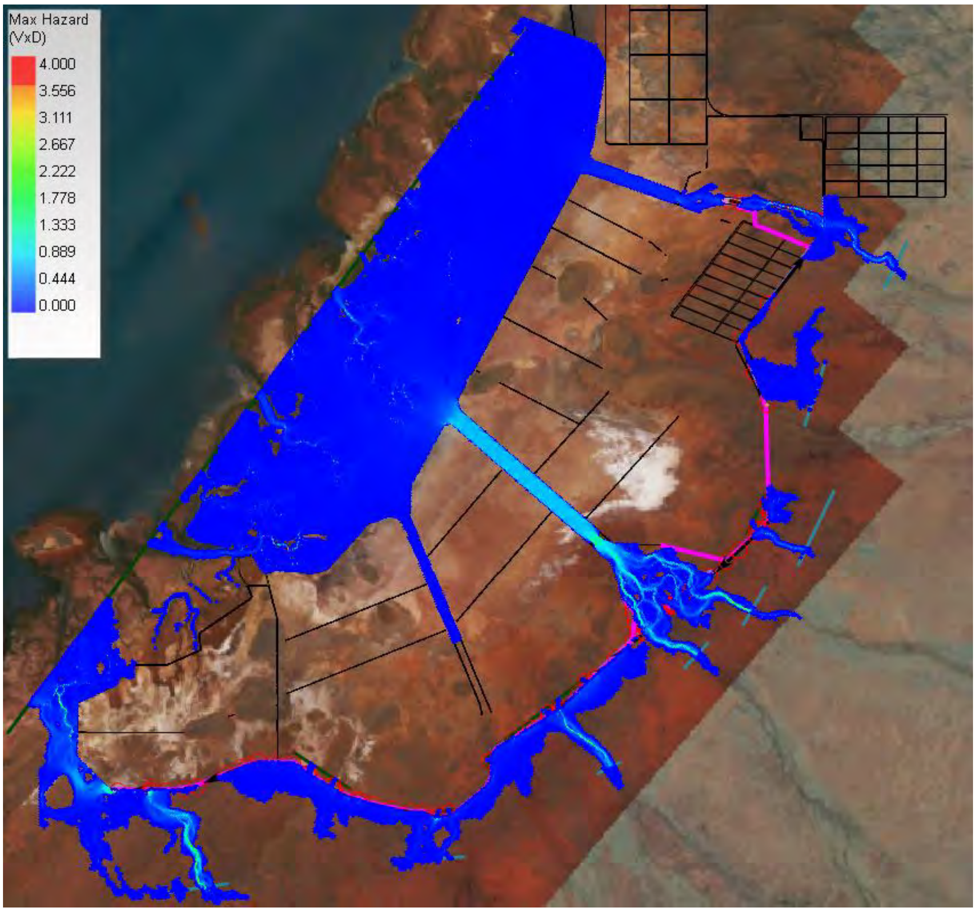


Figure 10: Post development 10 year ARI maximum hazard (velocity x depth) (m^2/s)

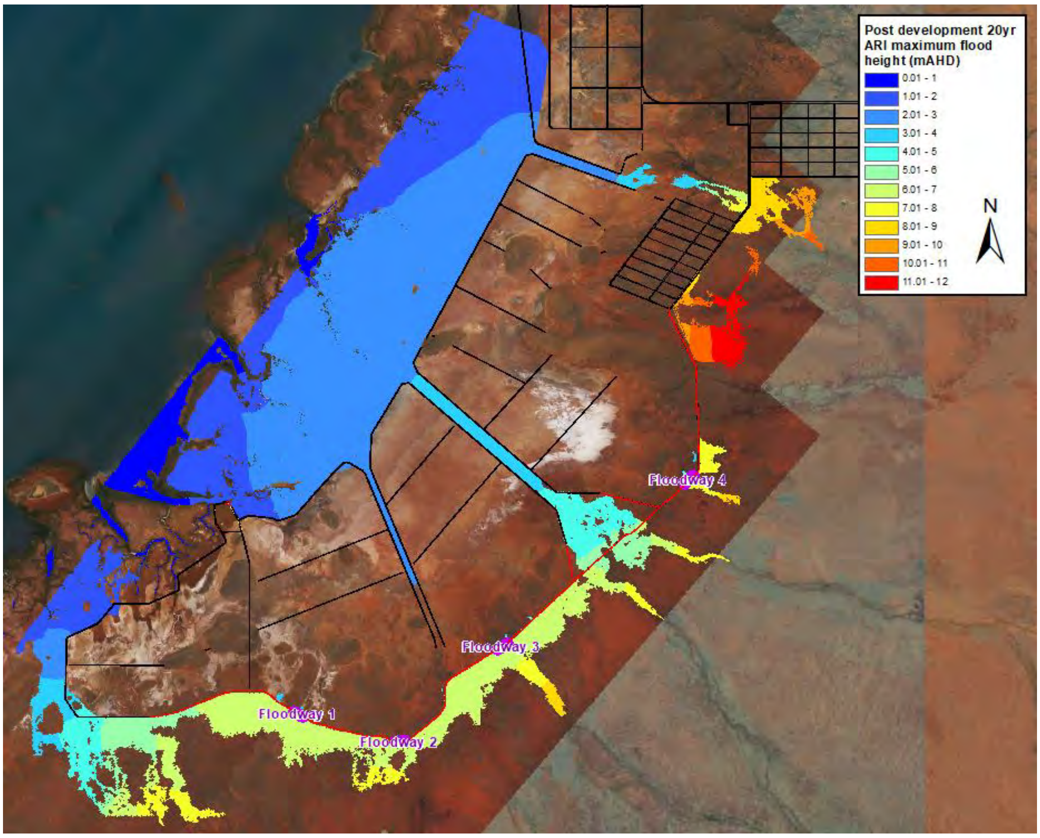


Figure 11: Post development 20 year ARI maximum flood height (mAHD)

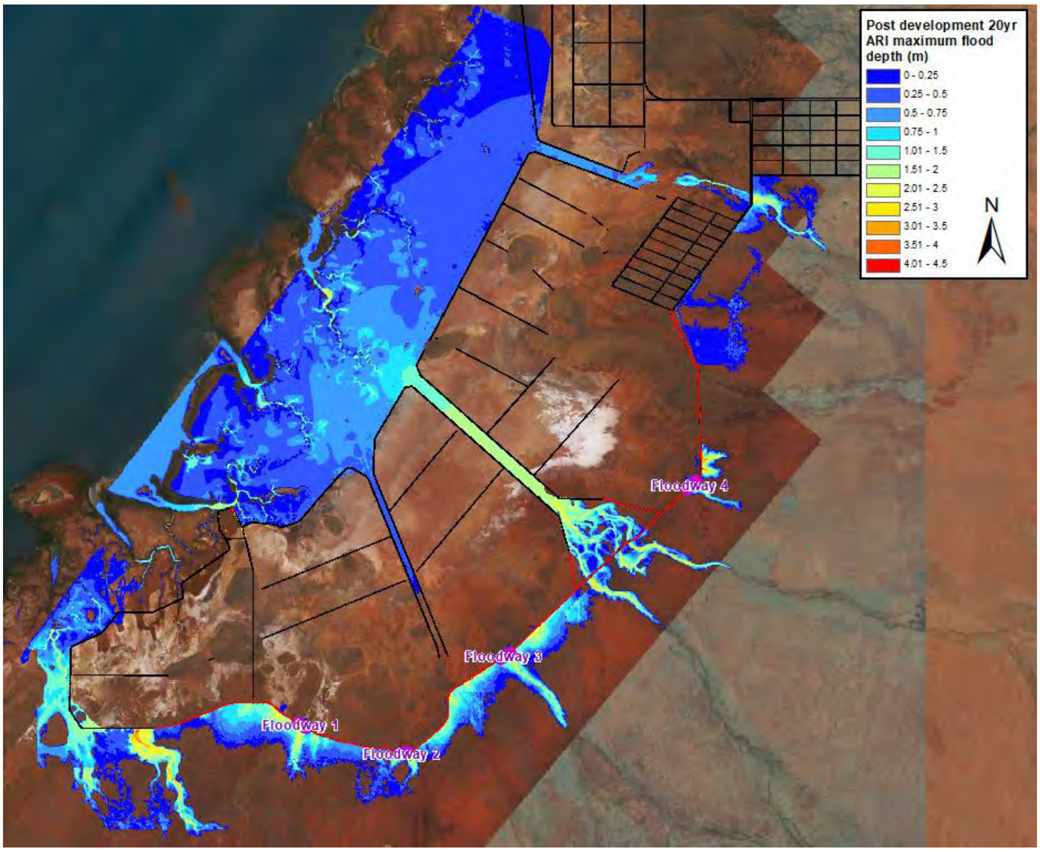


Figure 12: Post development 20 year ARI maximum flood depth (m)

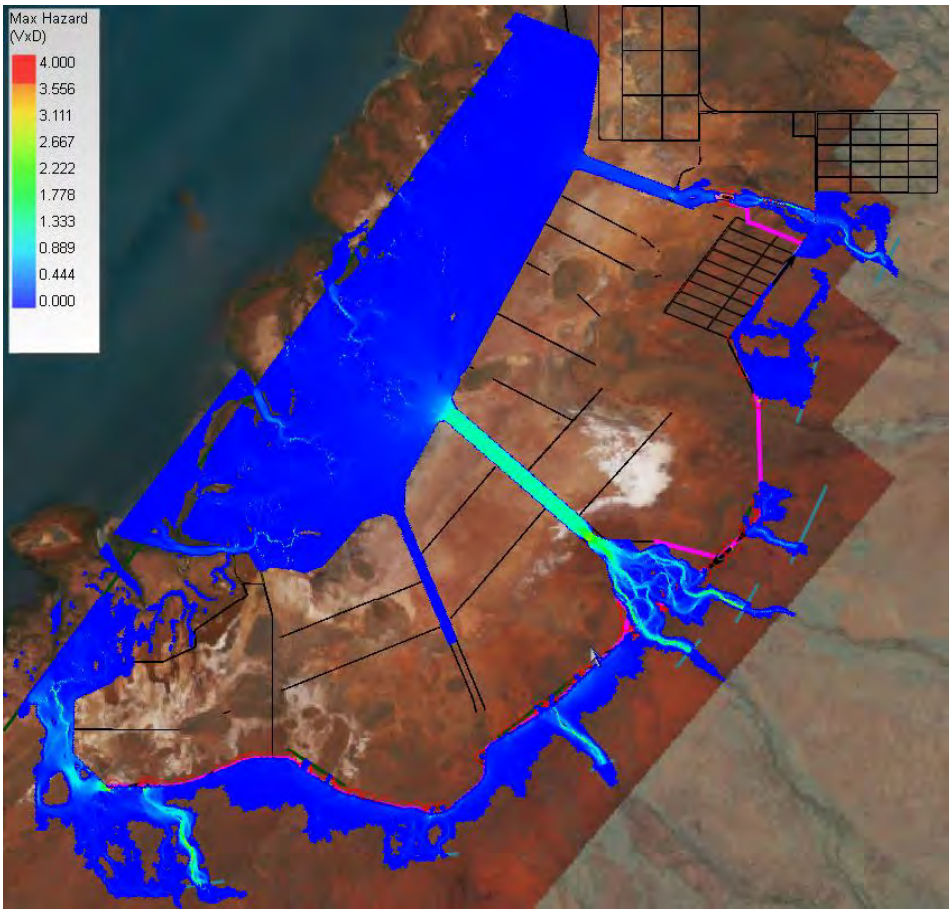


Figure 13: Post development 20 year ARI maximum hazard (velocity x depth) (m^2/s)

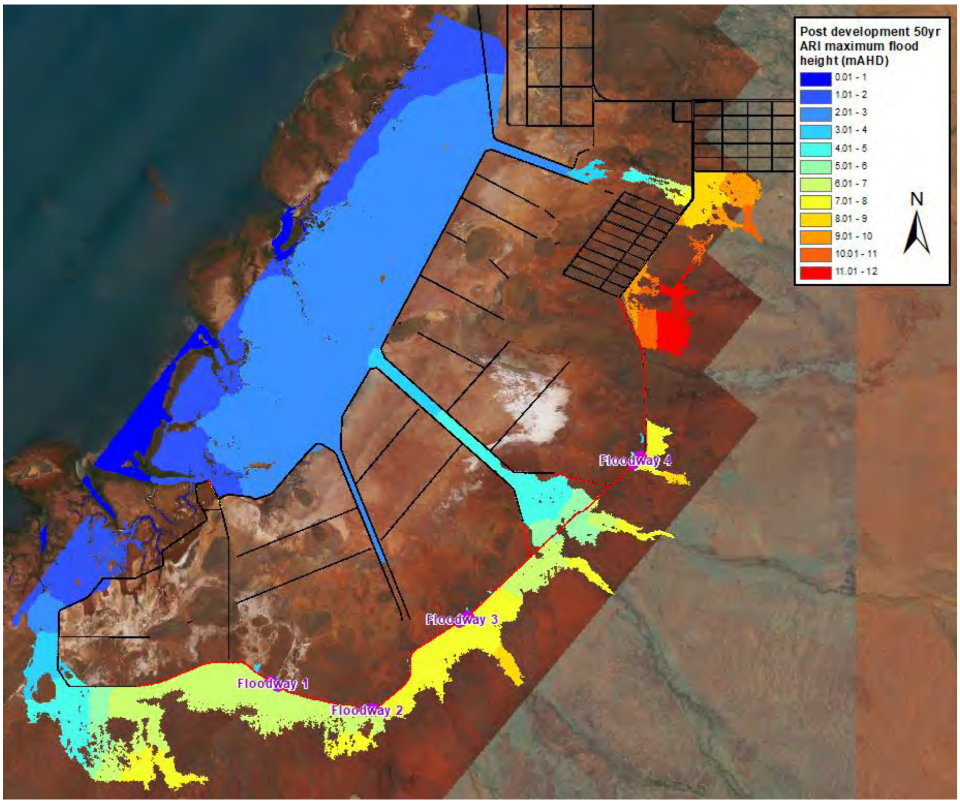


Figure 14: Post development 50 year ARI maximum flood height (mAHD)

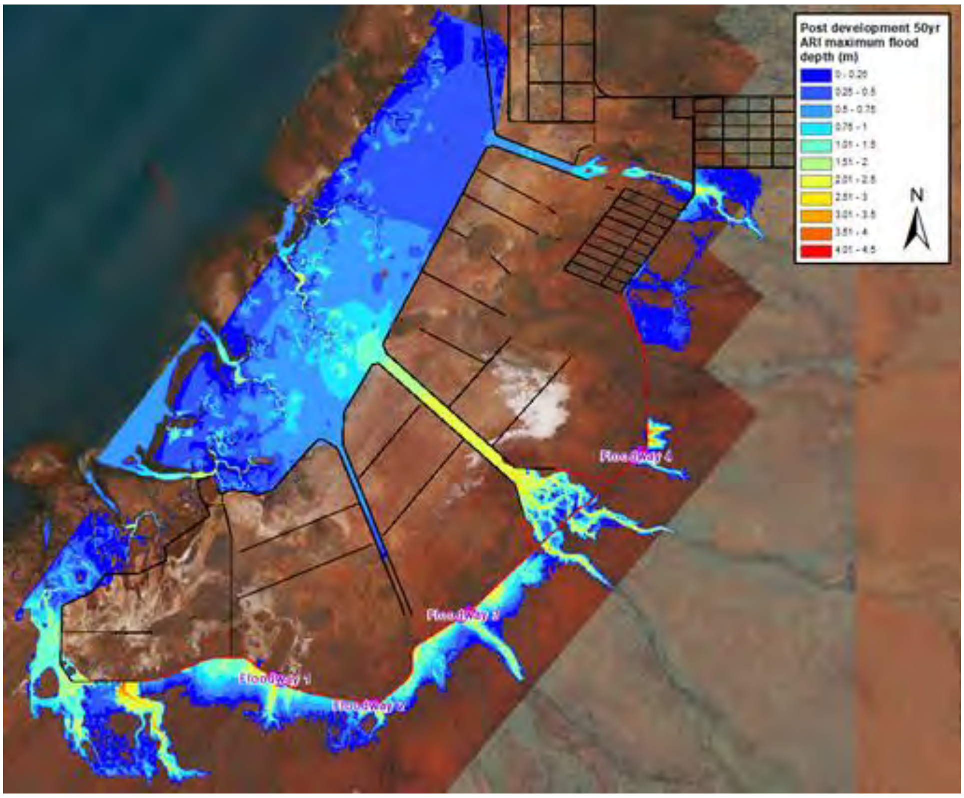


Figure 15: Post development 50 year ARI maximum flood depth (m)

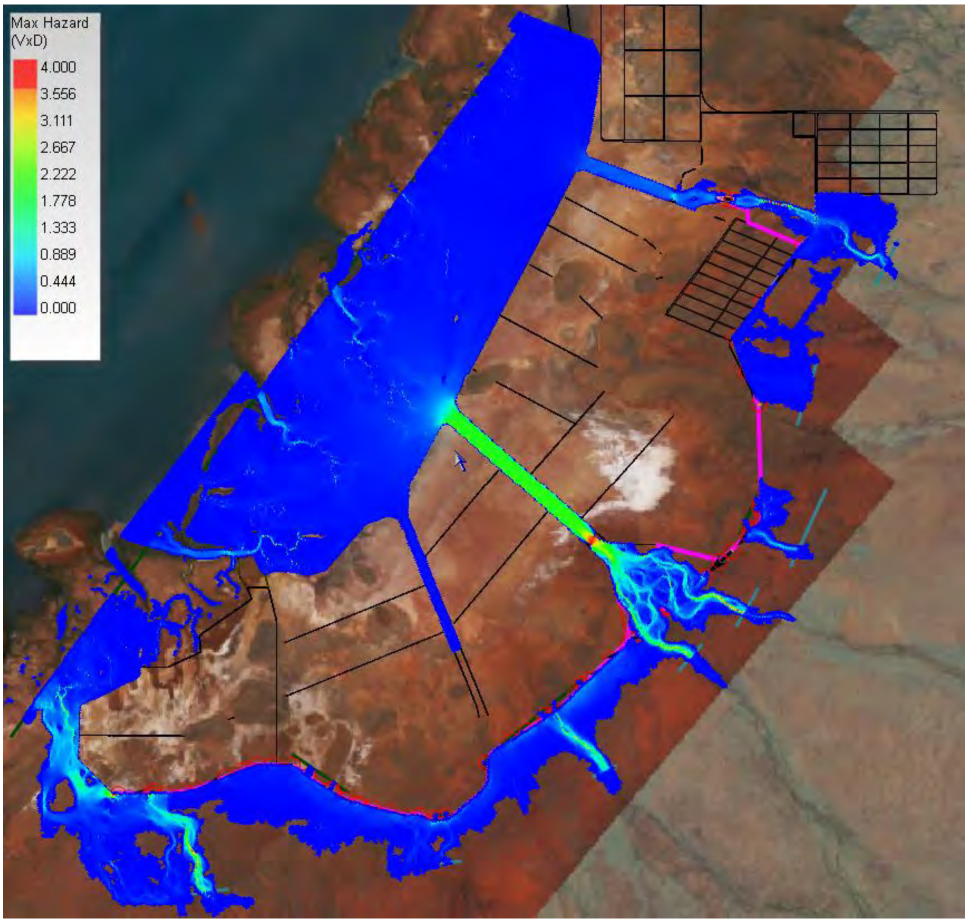


Figure 16: Post development 50 year ARI maximum hazard (velocity x depth) (m^2/s)

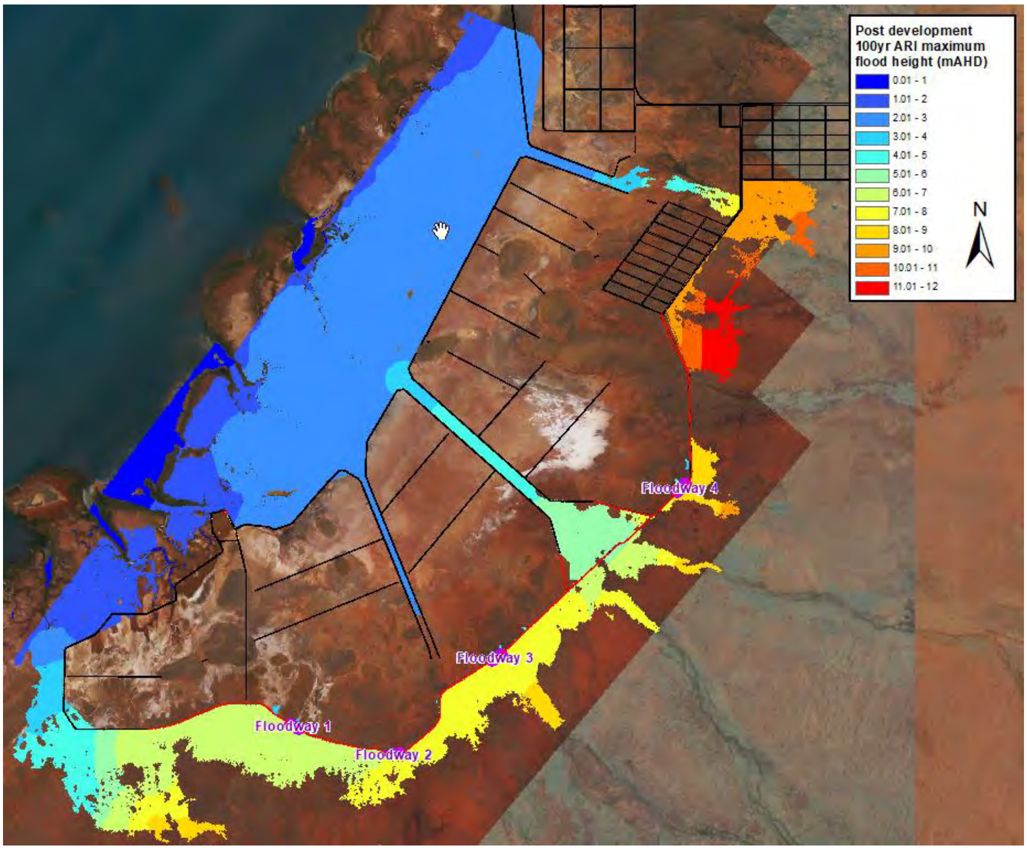


Figure 17: Post development 100 year ARI maximum flood height (mAHD)

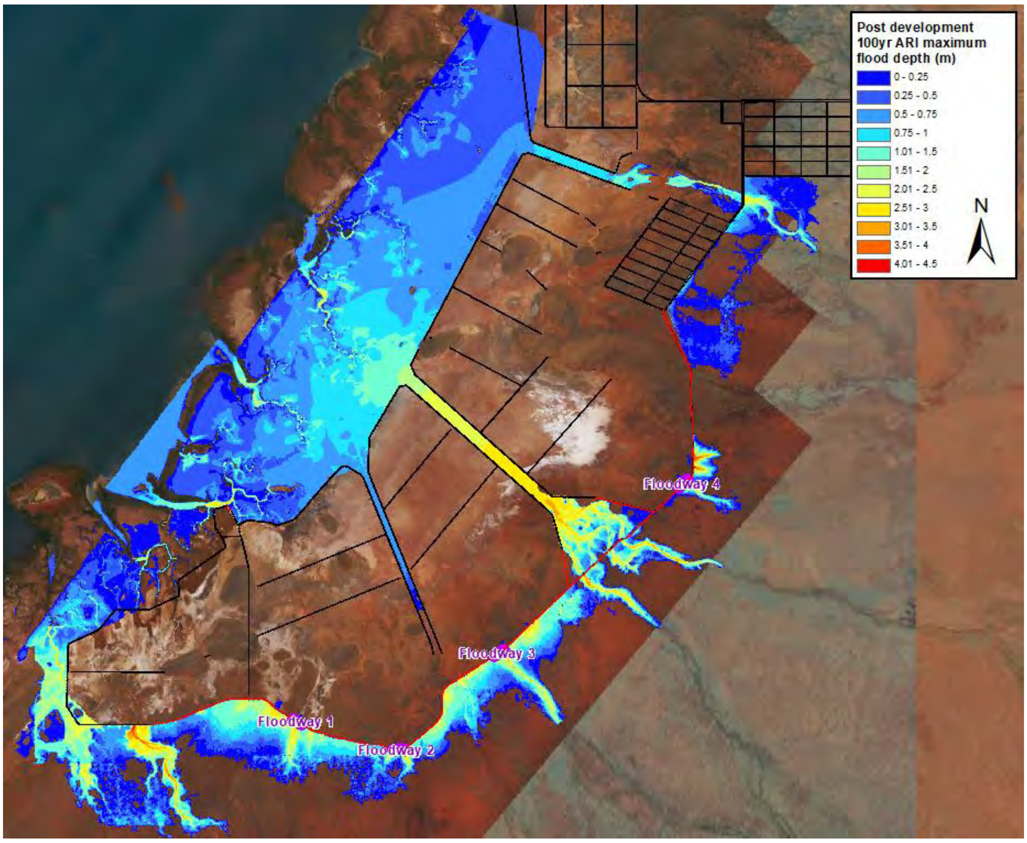


Figure 18: Post development 100 year ARI maximum flood depth (m)

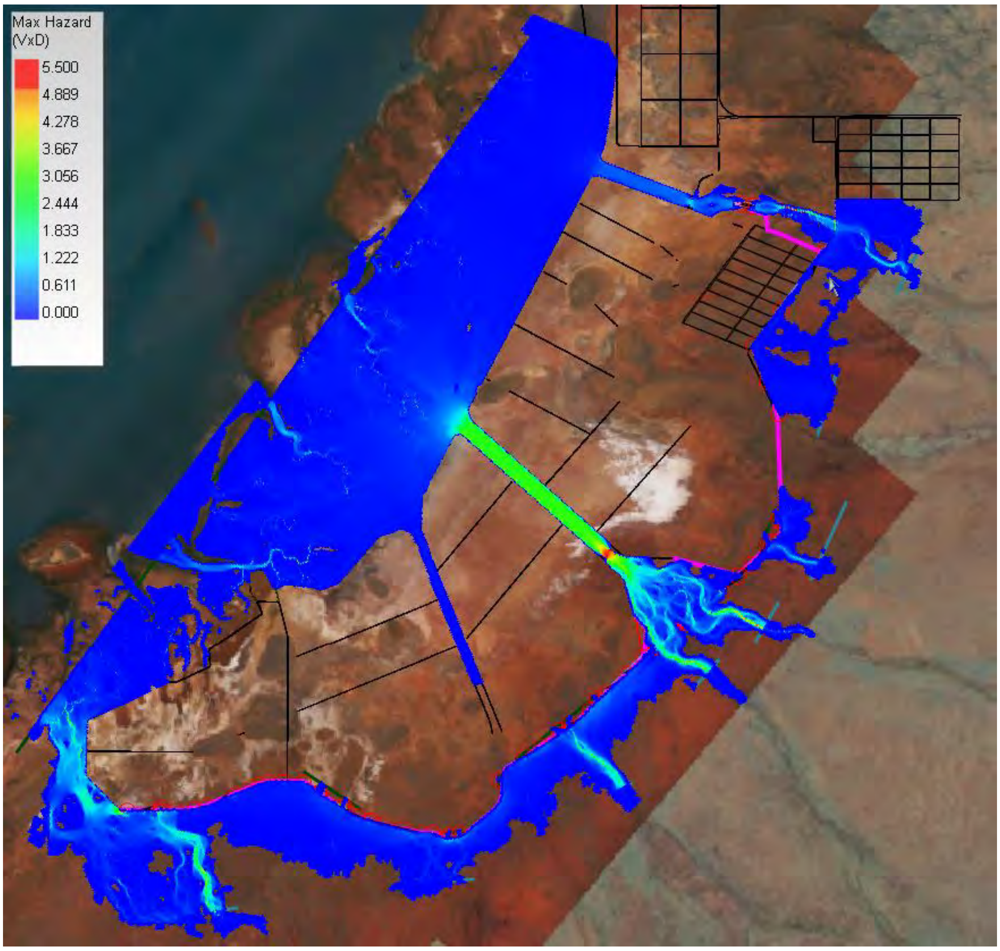


Figure 19: Post development 100 year ARI maximum hazard (velocity x depth) (m^2/s)

4.3 Floodways and culverts modelling results

As previously described, floodways to allow some discharge into the salt ponds during major flood events are proposed at four locations (refer to Figure 3 for locations). Various floodway heights have been previously modelled and reported to ascertain the impact on flood levels. A floodway set at a level equivalent to the 50 year ARI flood elevation has been adopted to minimise the potential for large flows discharging into the ponds (ie. only events rarer than 50 year ARI will flow overtop the floodways and discharge into the ponds). Tables 4 and 5 presents the peak flow and flood volume over the floodways for the various ARI storm events.

Previous modelling (RPS 2019, Appendix E) assessed the likely volumes of discharge into the salt ponds from both the floodways and the relief culverts that will drain trapped low areas. With floodways set at the 50 year ARI flood level, the modelled 100 year ARI discharge over the four floodways ranged from 150 ML to 500 ML (with a total of 1,340 ML). The corresponding water/brine level rise in the receiving salt pond for each of the respective floodways was estimated as between 0.02 m and 0.04 m.

In addition to the four floodways that discharge to the salt ponds (only during >50 year ARI events), the design incorporates an additional two floodways where floodwater will overtop the north-south access road and flow through Drainage Corridors B and C. Discharge over these floodways does not impact the salt ponds. The preliminary design level of these two floodways are illustrated on Graphs 1-3 in the following section.

Table 4: Floodway discharge volumes (floodway set at 50 year ARI TWL)

			100 year ARI event		50 year ARI event		20 year ARI event	
Floodway	Receiving pond	Pond area (km2)	Floodway discharge volume (ML)	Pond water level rise (m)	Floodway discharge volume (ML)	Pond water level rise (m)	Floodway discharge volume (ML)	Pond water level rise (m)
1	2	15.4	500	0.04	0	0	0	0
2			149		0		0	
3	3	15.5	380	0.02	0	0	0	0
4	4	9.4	310	0.03	0	0	0	0

Estimated water level rise is based on modelled discharge volume over floodway; excludes direct rainfall on ponds.

Table 5: Peak discharge rate over floodways (m^3/s)

	Spillway at 50 year ARI TWL		
Floodway	100 year ARI event	50 year ARI event	20 year ARI event
1	19	0	0
2	8	0	0
3	20	0	0
4	17	0	0

Figure 20 below illustrates the inundation extent during the peak of the 100 year ARI storm event (shown on the left) and also the inundation extent following recession of the floodwaters when water remains pooled in trapped low areas (shown on the right). The duration of inundation during the peak of the storm (on the left) is in the order of several hours, following which floodwater recedes over the course of approximately 1 - 2 days until water remains only in trapped low areas (shown on the right). The extent of inundation in trapped low areas will ultimately be determined by the elevation at which relief culverts and lateral drains are constructed to partially drain these areas; the extents shown in Figure 20 below are based on the lateral drain elevations that were assumed in the modelling (refer to Graphs 1 - 3 in the next section).

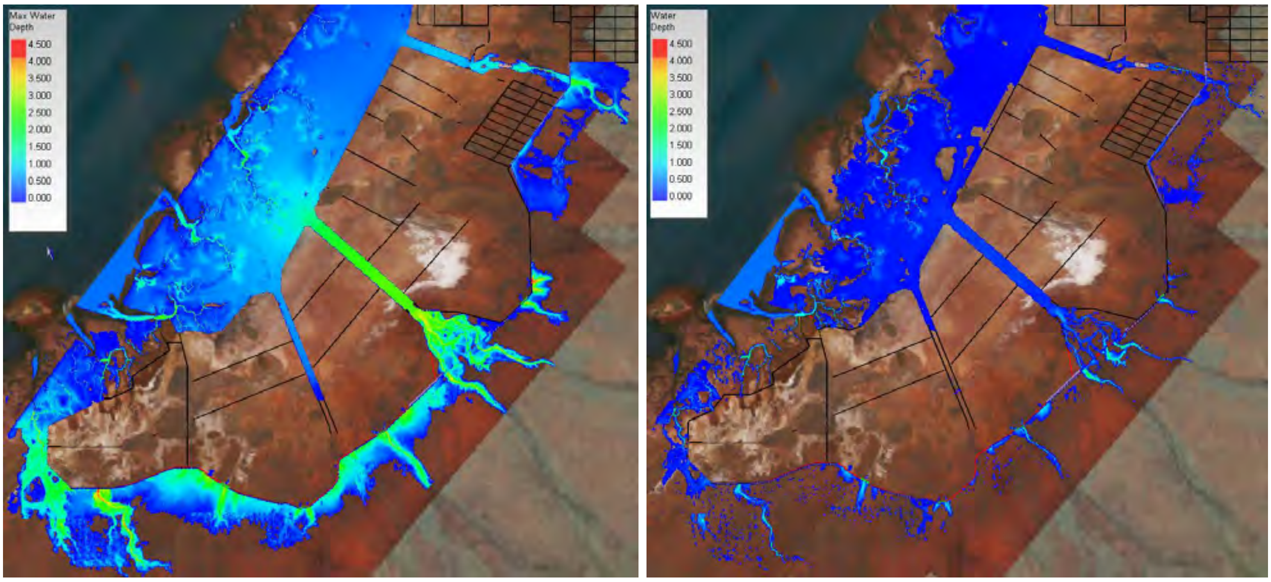


Figure 20: Extent of inundation, during 100 year ARI storm peak (left image) and in trapped low areas following storm (right image)

Culverts have been modelled under the flood bund and floodways to allow for low flow discharge to reduce the duration of ponded water in trapped low areas. Culvert locations, modelled culvert schedules and peak discharge rates are detailed in Figure 21 and Table 6 below.

The volume of water estimated to be held in the identified trapped low areas was 530 ML in total (noting that this estimation is dependent on the elevation at which the lateral drains are set and will subsequently allow the flood level to recede to). It was also estimated that the volume of water which would discharge via the relief culverts during a flood event would be of a similar magnitude to the amount of trapped water that would discharge following a flood event (i.e. 1-2 days of rainfall / flooding followed by 1-2 days to discharge the trapped water). Therefore, the amount of water that is estimated to discharge to the salt ponds via the relief culverts during a 100 year ARI event (~1,000 m^3) is similar to the volume that is estimated to discharge to the salt ponds via the floodways during a 100 year ARI event.

Table 6: Culvert details

Culvert details					
Culvert ID	**Discharges into**	**Culvert size (mm)**	**Peak flow (m3/s) in 100 year ARI event**	**Peak flow (m3/s) in 10 year ARI event**	**Peak flow (m3/s) in 1 year ARI event**
A	Pond 1	450	0.30	0.26	0.1
B	Pond 2	450	0.32	0.26	0
C	Pond 2	450	0.40	0.33	0.19
D	Pond 2	450	0.31	0.27	0.18
E	Pond 2	450	0.32	0.27	0
F	Pond 2	450	0.34	0.29	0
G	Pond 3	450	0.45	0.37	0.25
H	Pond 3	450	0.46	0.39	0.32
I	Pond 3	450	0.34	0.29	0.22
J	Drainage channel B	600	0.57	0.45	0.01
K	Drainage channel B	600	0.30	0.25	0.02
L	Drainage channel B	600	0.25	0.13	0.01
M	Drainage channel B	600	0.66	0.27	0.01

Culvert details

N	Drainage channel B	600	0.61	0.33	0.02
O	Pond 4	450	0.42	0.35	0.30
P	Pond 4	450	0.38	0.33	0
Q	Pond 4	450	0.37	0.32	0
R	Drainage channel C	1000	4.17	4.16	0.30
S	Drainage channel C	1000	4.17	4.16	0.78

* Note that the culvert locations depicted are indicative only based on trapped low locations and the requirement and location of culverts will be confirmed through detailed design.

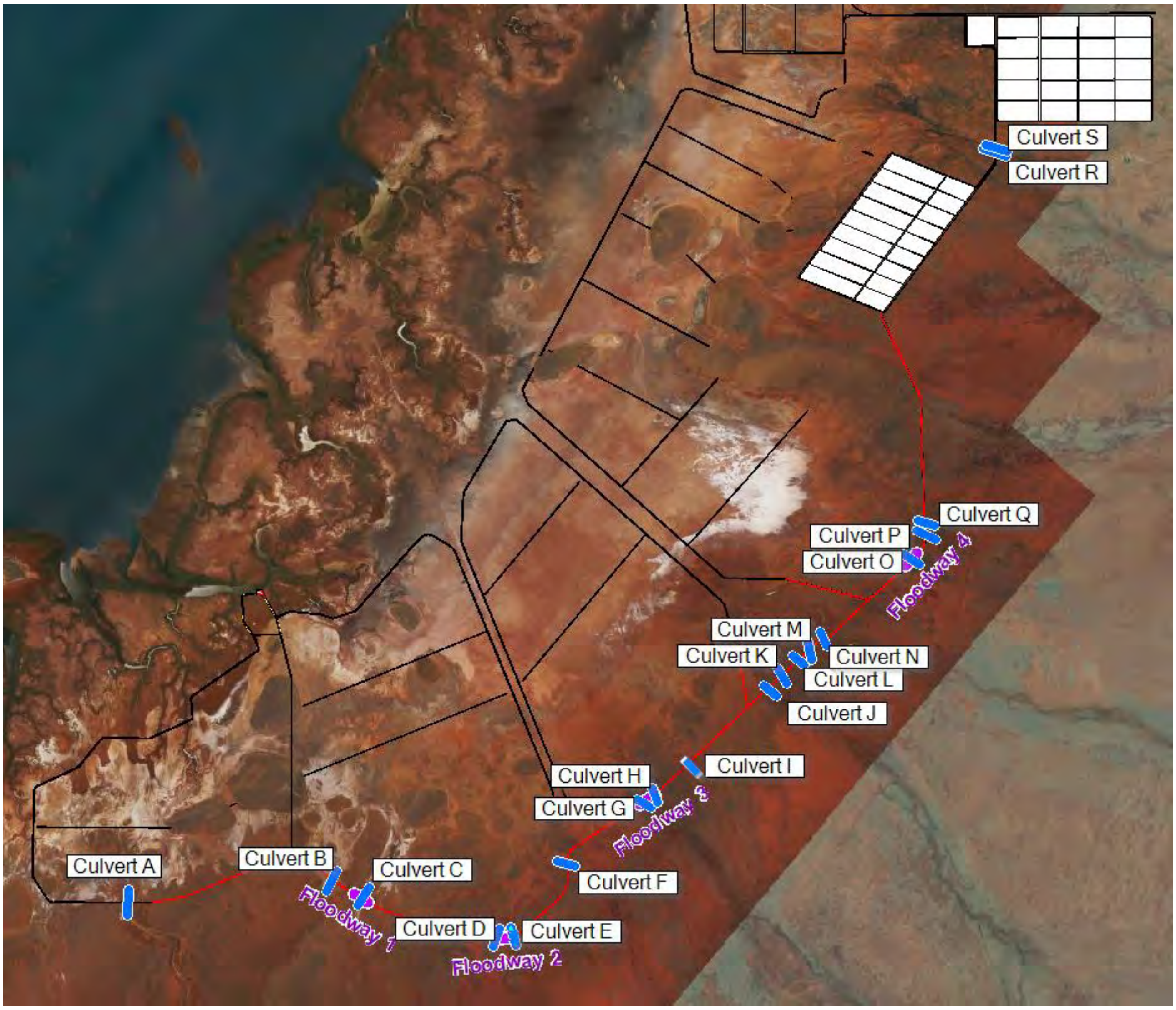


Figure 21: Indicative culvert locations

4.4 Flood bunds and lateral drain modelling results

Figure 22 shows the chainage along the back of the pond area which includes the flood bunds, floodways and corridors B and C. Graphs 1 to 3 show the long sections along the chainage including natural surface level, the elevation of lateral drains and floodways and the modelled top water levels.

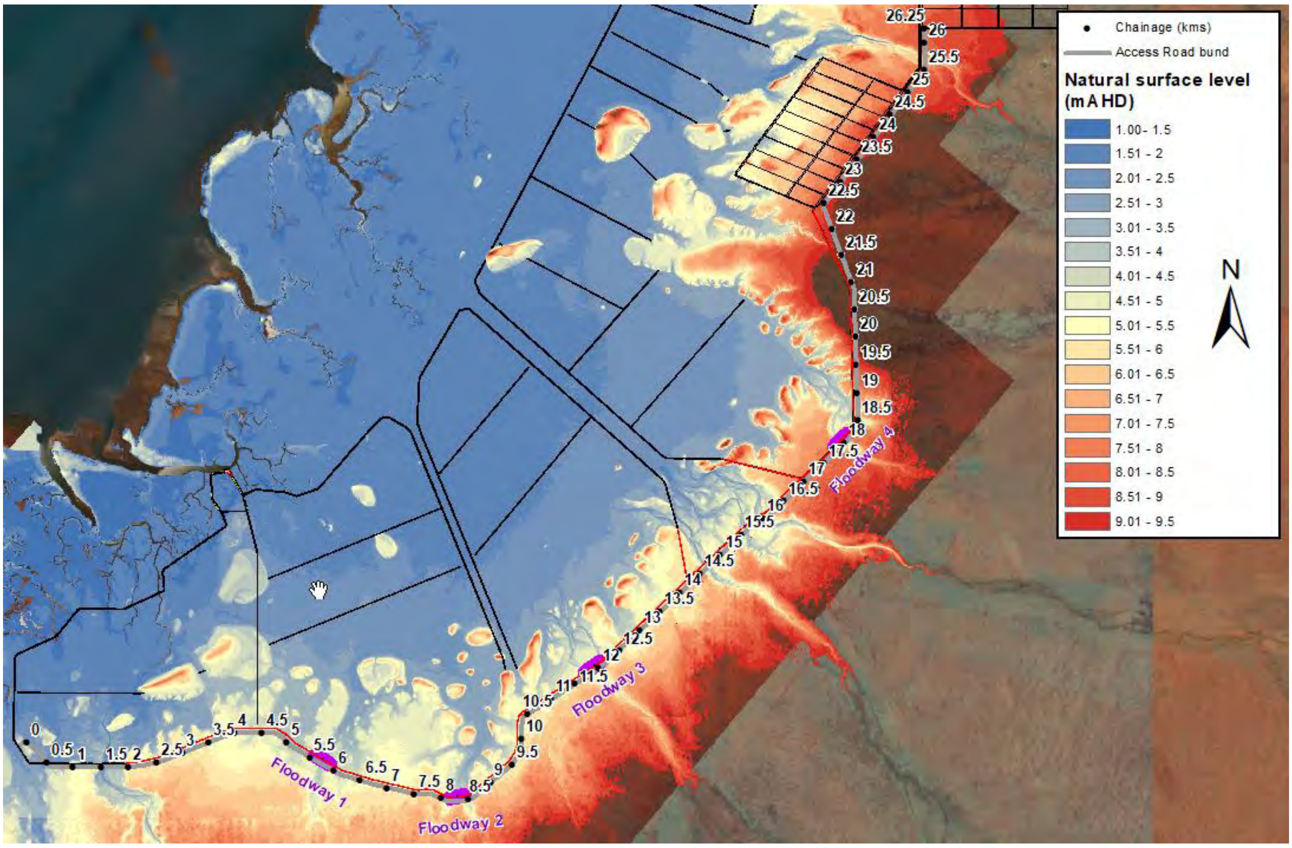
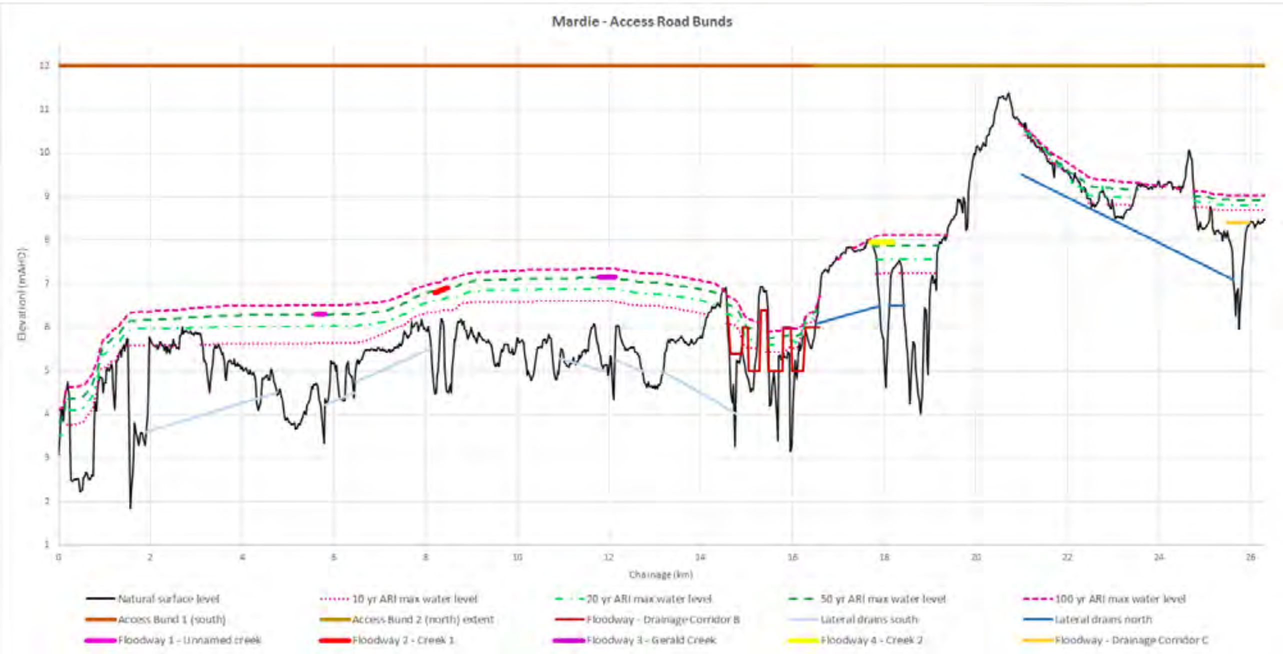


Figure 22: Access bund 1 and 2 chainage



Graph 1: Access bund 1 and 2 long section

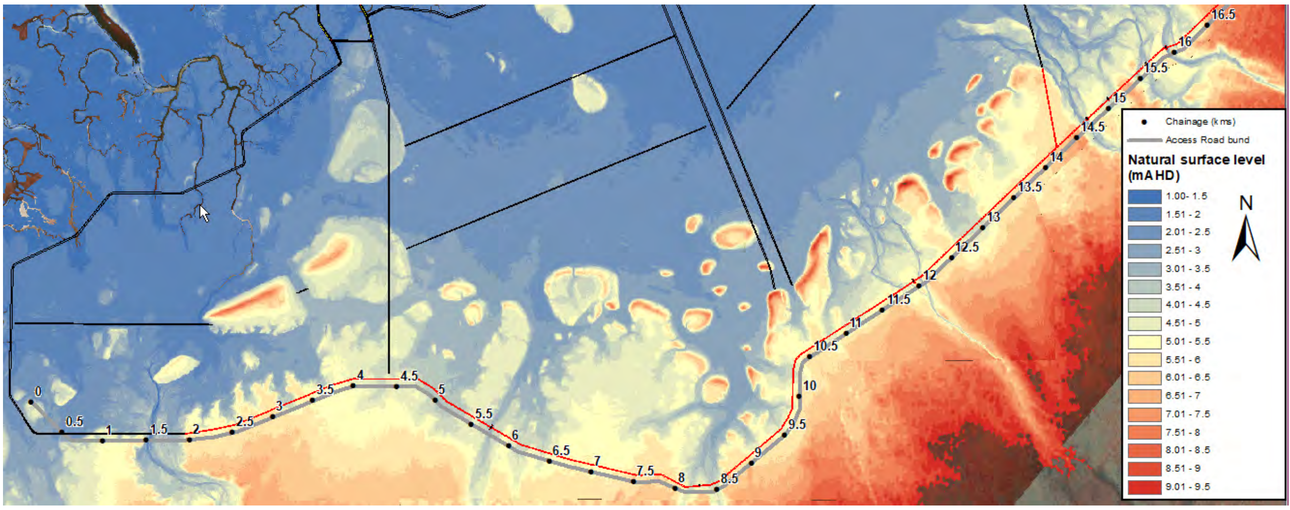
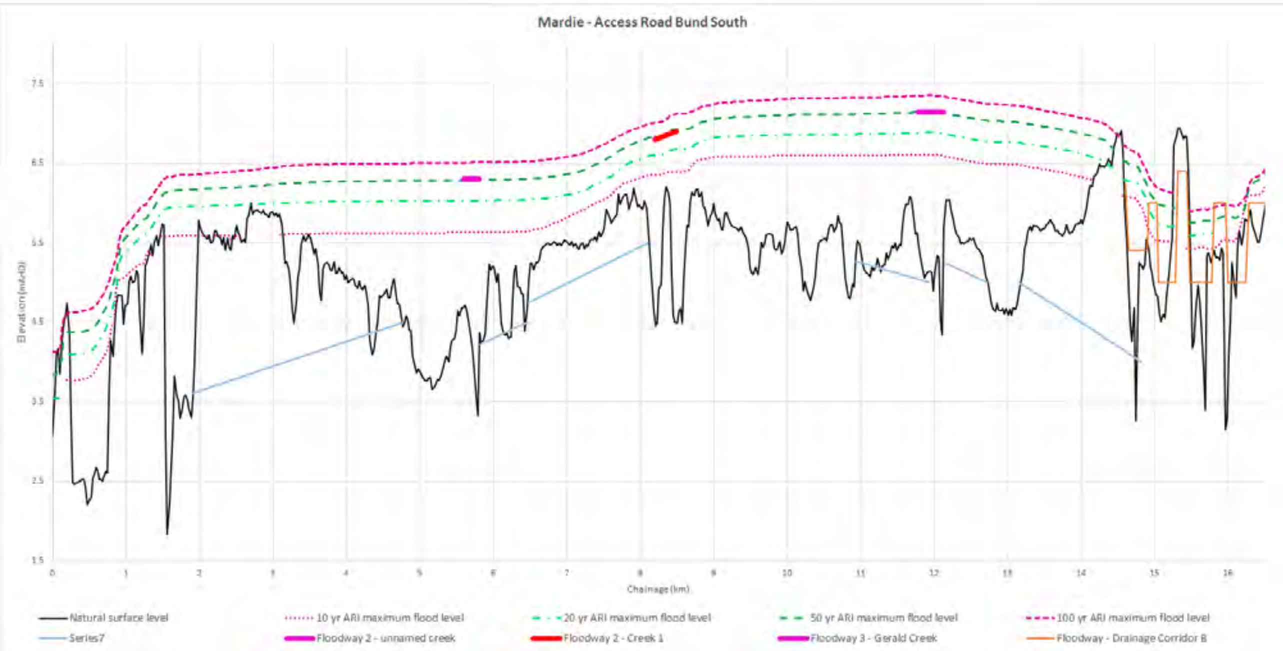


Figure 23: Access bund 1 chainage



Graph 2: Access bund 1 long section

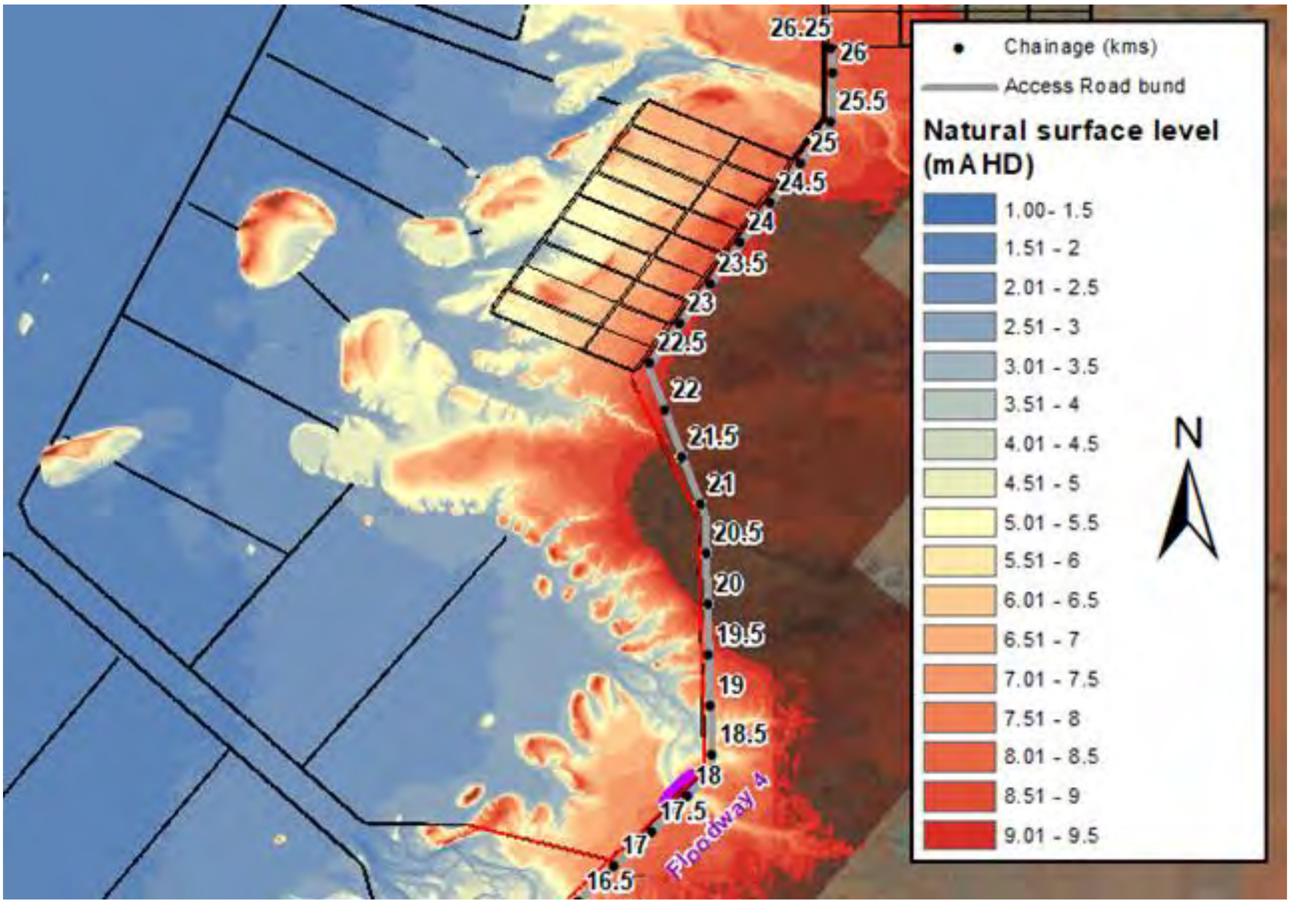
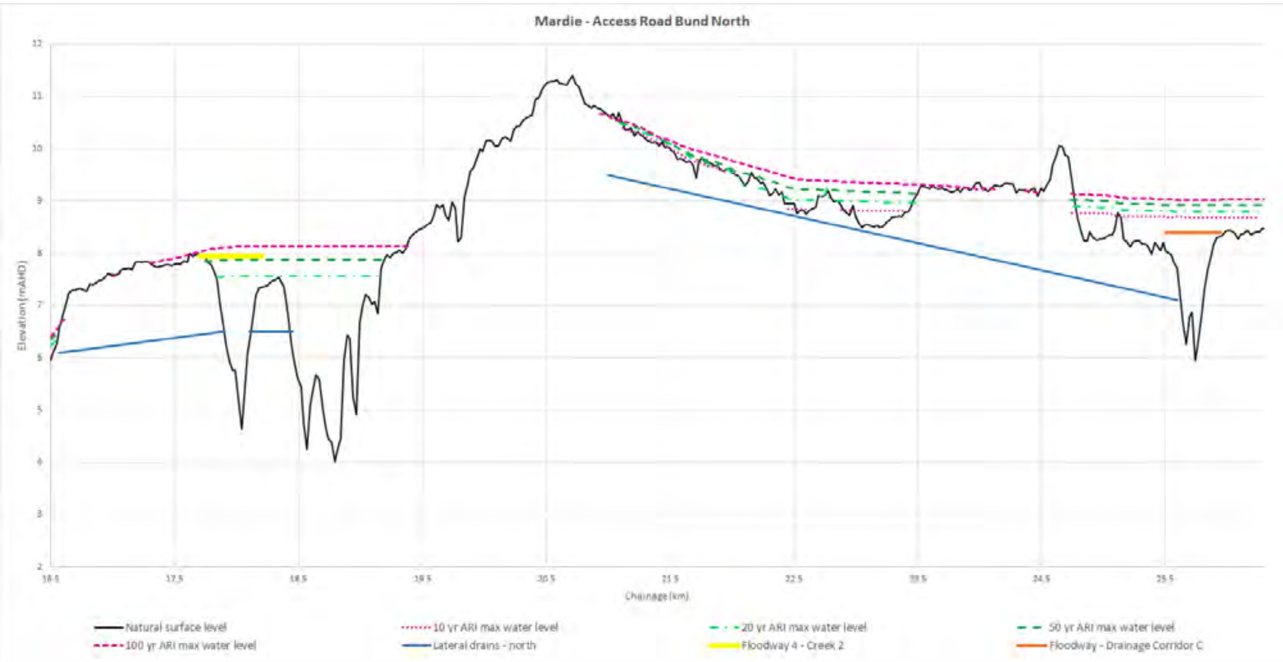


Figure 24: Access bund 2 chainage



Graph 3: Access bund 2 long section

4.5 Drainage corridors

Drainage Corridor B and C are required to carry floods from the rear of the ponds though to the front of the ponds and the sea. Corridor A is not proposed to be used for drainage in order to protect the existing gas pipeline infrastructure that traverses Corridor A. Various corridor drain widths have been previously modelled and reported to ascertain the impact on flood levels (RPS 2017c, Appendix C).

The final design has settled on drainage corridor widths of 250-300 m, with narrow sections of 100-150 m width to facilitate services crossing. The chainage and modelled flood levels for each modelled event within the Drainage Corridor B are shown in Figures 25 and 26, and Graph 4. The maximum flood level in the Drainage Corridor B in the 100 year ARI event is less than 5.25 mAHD, which is below the below the height of the upstream floodway located on the north-south access road and greater than the estimated 100 year ARI storm surge level of 4.3 mAHD.

The maximum flood depth in Drainage Corridor C is approximately 3 mAHD and is well below the 4.3 mAHD storm surge level (Figure 27).

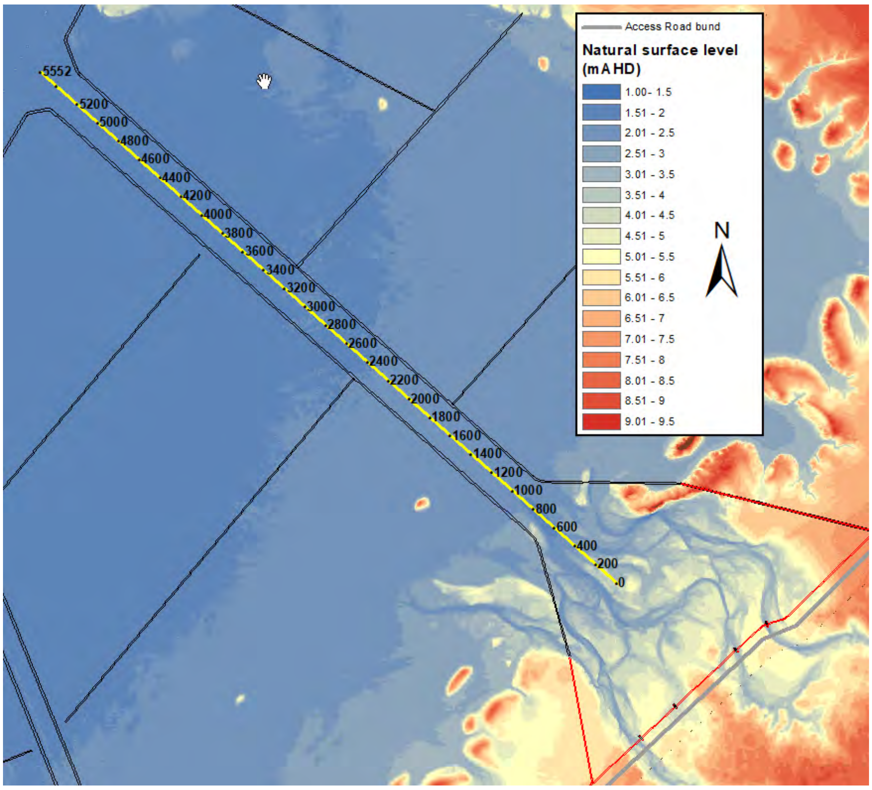
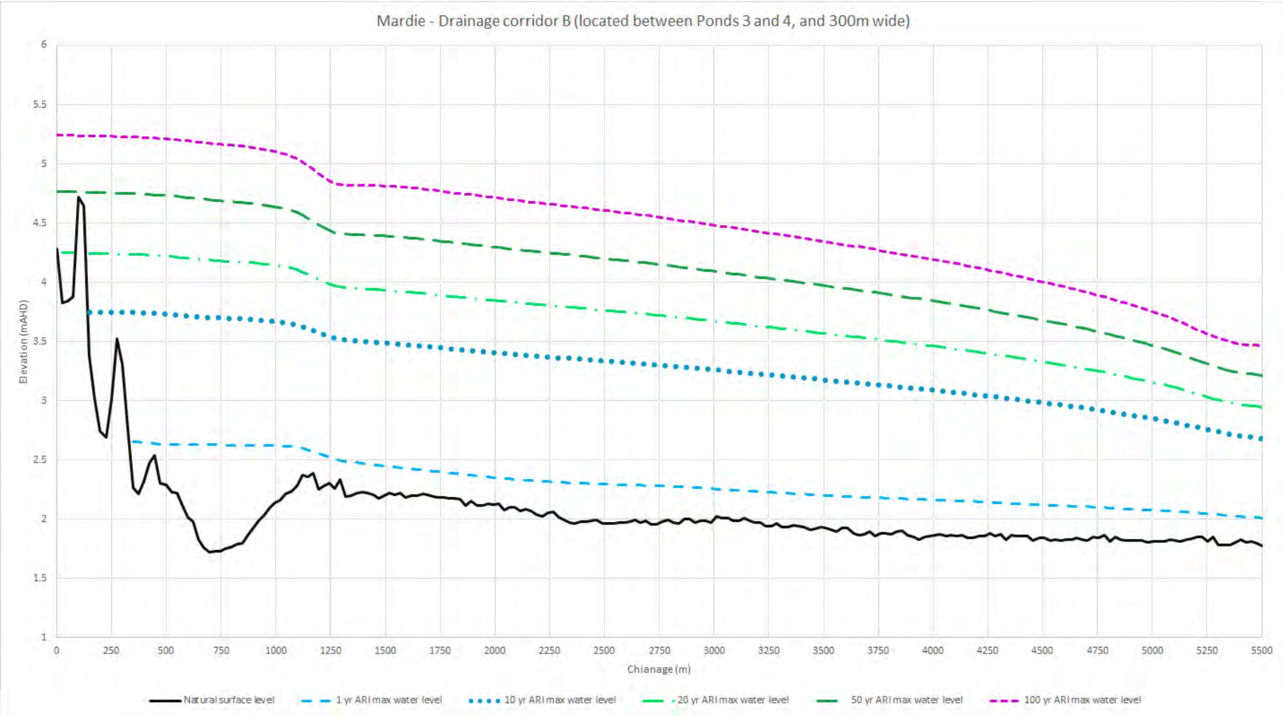


Figure 25: Chainage (kms) along Drainage Corridor B



Graph 4: Flood levels within Drainage Corridor B

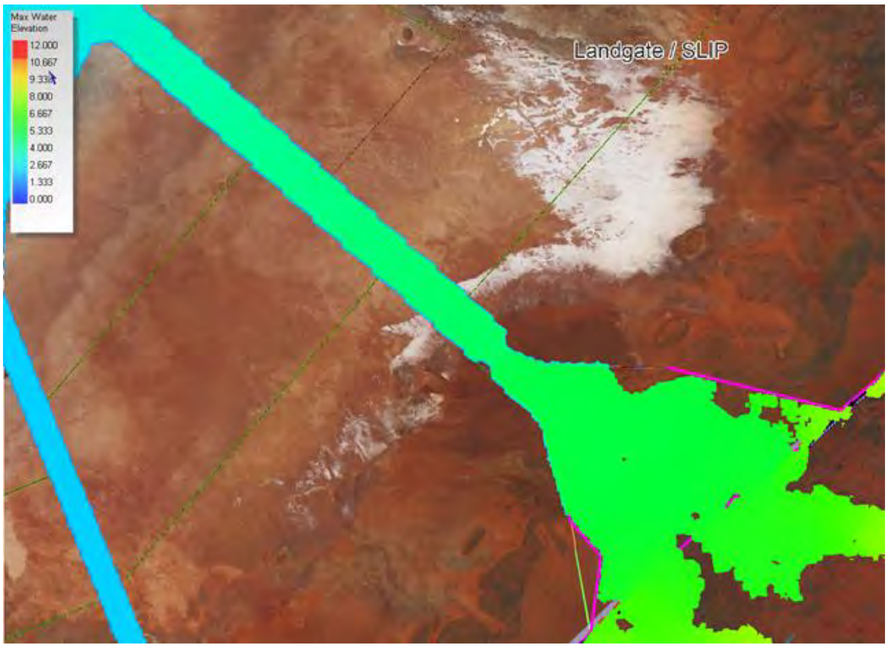


Figure 26: Maximum flood elevation (mAHD) in the 100 year ARI storm within Drainage Corridor B

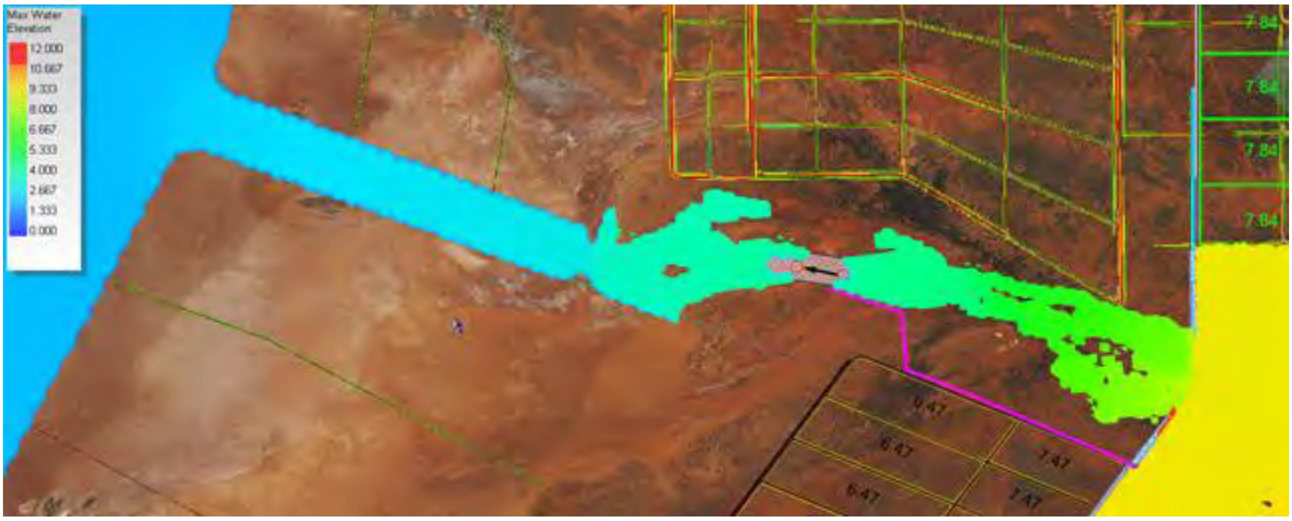


Figure 27: Maximum flood elevation (mAHD) in the 100 year ARI storm within Drainage Corridor C

4.6 Design flood levels

Table 7 summarises the design flood level estimates at the front and rear of the salt ponds, for the modelled 28 m lateral diversion drains, floodway invert levels set at the 50 year ARI event top water level and 250-300 m drainage corridor widths. The design flood levels at the front of the salt ponds are dominated by sea levels (tide, storm surge, wave setup, etc) and have been taken as the storm surge levels estimated by the preliminary RPS metocean analysis. The design flood levels at the rear of the salt ponds are dominated by the surface flood levels and are highly variable along the flood bund (depending on the natural surface levels and the magnitude of flood flows being intercepted).

Table 7: Design Flood Level Estimates (mAHD)

Return Period (ARI)	Flood (Sea) Level (Front of Ponds)	Flood Level (Levees Upstream of Ponds)
10	3.6	3.7 – 8.8
20	3.9	4.0 – 9.2
50	4.1	4.2 – 10.4
100	4.3	4.3 – 10.6

4.6.1 Level of protection

The flood bunds are proposed to be set at a level above the 100 year ARI event and the floodways (excluding the crossings at Corridors A and B) will be set at the 50 year ARI event top water level. This means that the ponds will only receive discharge from the upstream catchments in events greater than the 50 year ARI via the floodways. A suitable freeboard is added to flood levels to design the bund heights to cater for the various associated uncertainties (as described previously, RPS 2017b, Appendix B).

Table 8 below describes the probability of exceedance for various ARIs and project life.

Table 8: Probability of exceedance versus project life

Project Life (Years)	Average Recurrence Interval				
-	10 Year ARI	20 Year ARI	50 Year ARI	100 Year ARI	500 Year ARI
10	65%	40%	18%	10%	2%
20	88%	65%	33%	18%	4%
50	99%	92%	64%	40%	10%
100	100%	99%	87%	63%	18%

4.7 Engineering construction

4.7.1 Bund material

Soil materials may be characterised to ensure suitability, but the performance requirements for temporary water storage are not specific. The embankment would typically use the most suitable available material at the site, e.g. waste material or diversion excavations, and be constructed homogeneously (i.e. not zoned).

Flood bunds are generally watertight for stability reasons and some clay content is required - materials range from clayey gravels and sands (preferred), through to poorly graded sands (least preferred), and preferably no rock particles >75 mm.

4.7.2 Erosion protection

Scour in unprotected soils will typically occur when maximum velocities reach about 1.2 - 2 m/s for clays, up to about 1.5 m/s for sand, and higher for rocky material.

Rock armour can be used to protect earthworks against scouring and erosion, and can be applied where problems occur, or in the long term where permissible velocities may be exceeded. Generally, it is not considered necessary to rock armour an operational embankment or channel against velocities <2m/s for the design flood event (subject to operational experience).

Figure 28 below provides the modelled flow velocities within the drainage corridors. The peak velocity in the 10 year ARI event is 0.9 m/s or less, except at the narrow section within drainage corridor B where it is in 1.2 m/s. The peak velocity in the 100 year ARI event is 1.8 m/s or less except the narrow section of corridor B where it is 2.1 m/s. Further assessment and optimisation of drainage corridor B may be required at detailed design to confirm flow velocities and appropriate protection measures.

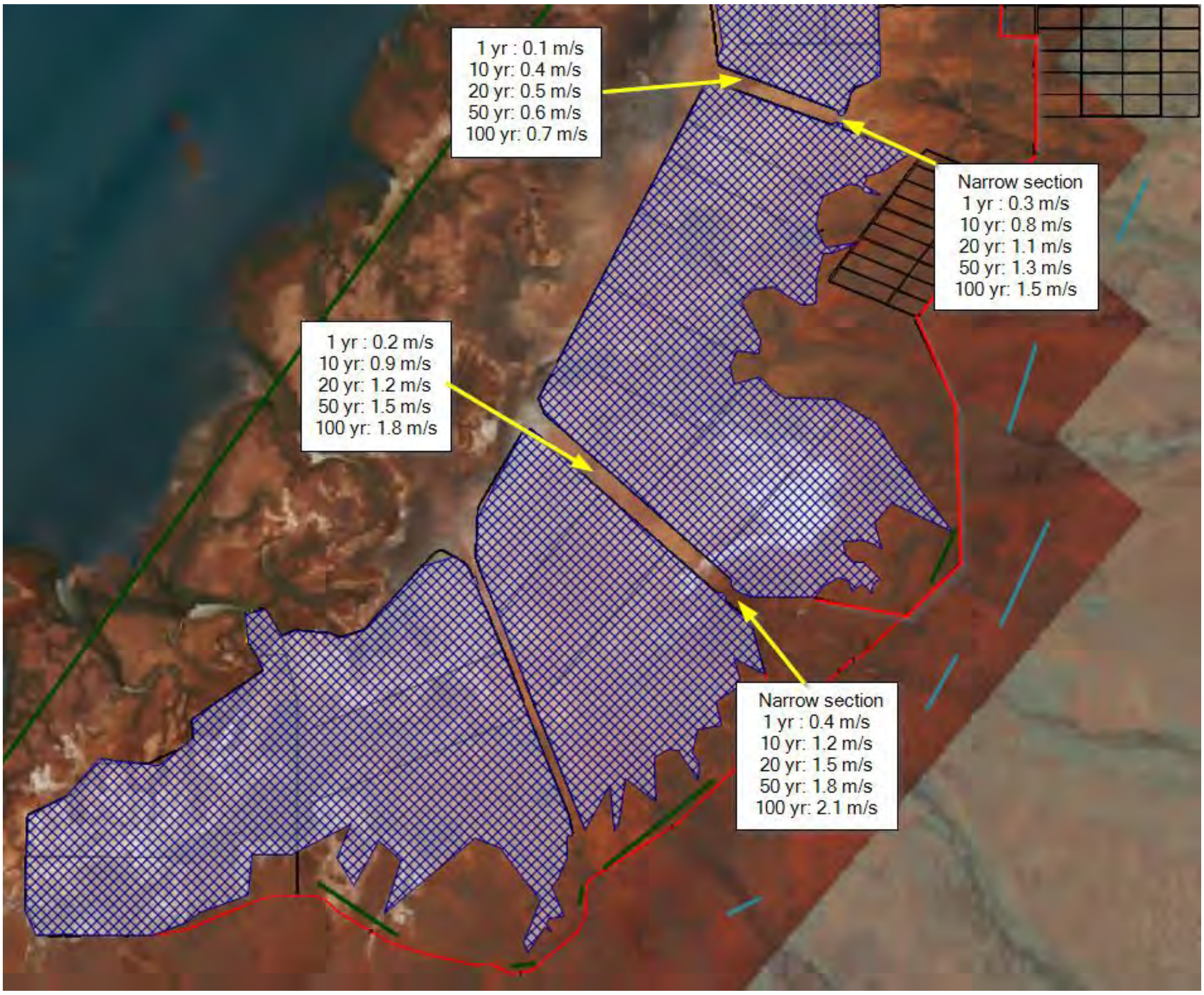


Figure 28: Modelled peak velocities

4.7.3 Construction

Earthworks (bund) construction requirements typically entail:

- Excavate to strip depth, scarify the base in preparation for construction of an embankment;
- Maintain moisture content in the embankment material at optimum (which allows the maximum density to be achieved by the compaction equipment in use);
- Place and compact material in layers as specified (e.g. 95% SMDD (Standard Maximum Dry Density); or 92% MMDD (Modified Standard Maximum Dry Density); and
- Control batter slopes to line and level.

5 HYDROLOGICAL IMPACTS TO ENVIRONMENT

5.1 Overview

The algal mats and mangroves located in the intertidal zone downstream of the proposed salt ponds have been identified as sensitive ecological systems which will require protection using appropriate design measures to mitigate and minimise potential impacts from the project.

To simulate the hydrological impacts of the salt ponds, the pre-development scenario (existing conditions) was modelled alongside the post-development scenario (with salt ponds) for the 1 and 10 year ARI events. The 1 year ARI event was modelled to represent a relatively frequent event which is considered more likely to be significant in terms of playing a role in ecological function (e.g. water and/or nutrient cycling). The 10 year ARI event was modelled to represent a more significant flood event with greater potential to impact the environment through physical means such as scour etc.

The 1 year ARI event was modelled with a 0 mAHD downstream boundary condition, i.e. approximately mean sea level and no tidal inundation of the mudflats downgradient of the salt ponds. The 10 year ARI event was also modelled with a 0 mAHD downstream boundary condition. Note that the 10 year ARI model was also run with a 0.5 mAHD water level and it was shown to make minimal difference in terms of flooding extent as a result of discharge from the upstream catchments.

5.2 Modelled impacts

The 1 and 10 year ARI predevelopment maximum flood depths are presented in Figures 29 to 30 below. The 1 and 10 year ARI post development maximum flood depth, flood height and hazard are presented in Figures 5 to 10.

Post-development flood hazard results have been presented as it provides an indication of likely impact to the environment as it is calculated as the product of flood depth multiplied by velocity. For example, a peak velocity of 1 m/s coinciding with a depth of 0.3 m gives a hazard value of 0.3 m^2/s. Similarly, a hazard value of 0.3 m^2/s would result from a velocity of 2 m/s coinciding with a depth is 0.15 m.

5.2.1 1 year ARI scenario

The impact of the proposed development on maximum water depth in the 1 year ARI event (Figure 5) is variable across the project area. The areas of most significant change as compared to predevelopment (Figure 29) are:

- Additional flooding areas downstream of Drainage Corridor B with modelled depths of generally <0.25m (other than within tidal creek channels where greater depth of flow may occur). However, it is noted that the scale of the model does not represent the small tidal creeks very well which is potentially overestimating the extent of impact in this area (i.e. floodwater may in fact dissipate more readily through the network of small tidal creeks which are not explicitly modelled);
- Additional flooding downstream of Drainage Corridor C to a maximum depth of 0.25 m;
- In the creek channels and diversion drains immediately upstream of the access bunds where maximum depths increase by up to approximately 0.75 m over a relatively small area; and
- Downstream of Pond 1 where there is a slight reduction in flooding extent due to the redistribution of flow from Peter Creek around the southern boundary of the salt ponds.

While the modelling did show some flood depth differences in the pre and post development 1 year ARI event, the majority of the additional flooding is less than 0.25 m deep.

In the post development scenario, the maximum hazard map (Figure 7) reports a value of less than 0.3 m^2/s across the site. This means that the risk of erosion and impact to vegetation due to water levels and peak flows are likely to be minimal given that a value of less than 0.3 m^2/s represents a fairly low energy flow environment. This is supported by previous modelling (RPS 2018, Appendix D) which indicated that the impact on velocities and water depths downstream of the ponds (from the concentration of flow into the drainage corridors) is limited to a short distance downstream of the drainage corridors and that there was negligible difference between pre-development and post-development water depths and velocities further downstream.

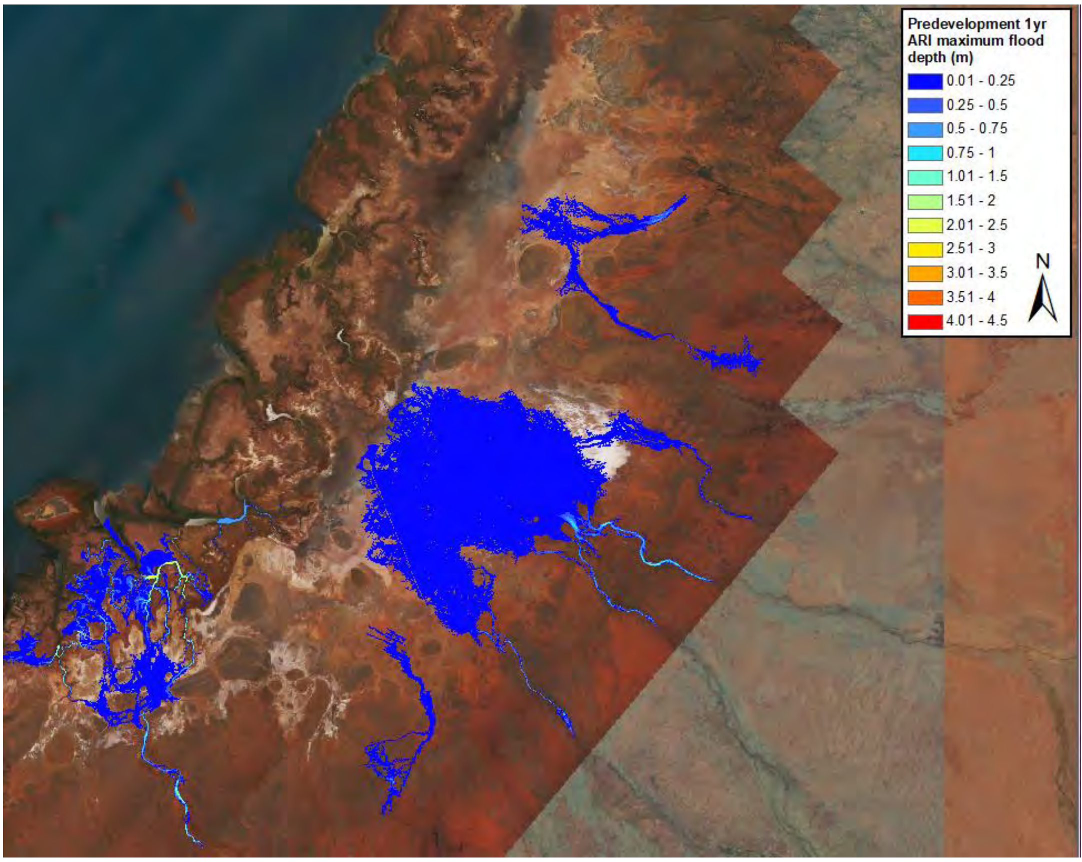


Figure 29: Predevelopment 1 year ARI maximum flood depth (m)

5.2.2 10 year ARI scenario

The impact of the proposed development on maximum water depth in the 10 year ARI event (Figure 8) as compared to the predevelopment levels (Figure 30) is more notable than in the 1 year ARI event. The areas of most significant change are:

- Addition flooding depth downstream of Drainage Corridor B over an extent of several hundred metres. Post-development flood depths immediately downstream of the corridor are approximately 0.5-0.75 m compared to pre-development depths of 0.25-0.5 m;
- Additional flooding depth for a small extent downstream of Drainage Corridor C by up to 0.25 m;
- In the creek channels and diversion drains immediately upstream of the access road flood bund where maximum depths increase by over 1 m in some locations. However, this is over a relatively small extent; and
- Downstream of Pond 1 there is a reduction in flooding extent in some areas and an increase in flooding depth (by up to approximately 0.75 m) in other locations. This is related to the redistribution of flow from Peter Creek around the southern boundary of the salt ponds.

While the modelling does show the proposed development will result in increased flood levels in certain areas (downstream of the drainage corridors and where Peter Creek discharges at the southern extent of the salt ponds), the post development hazard map (Figure 10) shows a velocity-depth value of less than 0.3 m^2/s

across most of the inundated area downstream of the ponds. Therefore, the risk in terms of increased erosion and impacts to vegetation from water levels and peak flows is likely to be low.

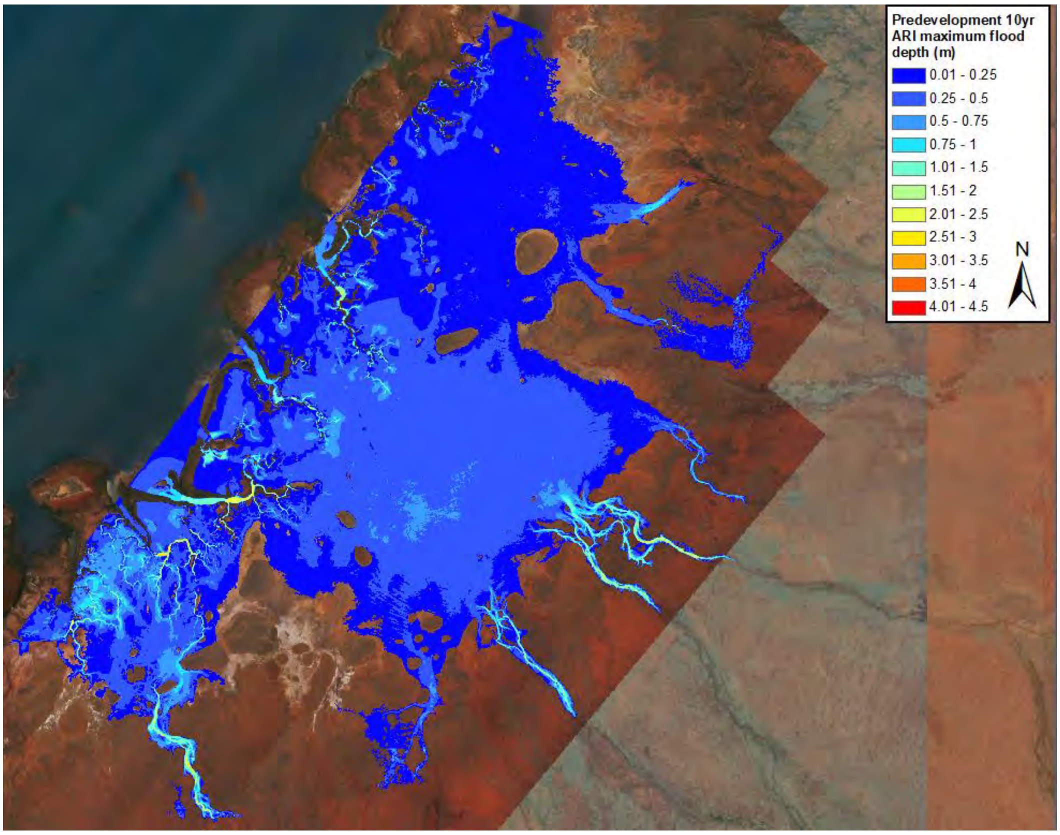


Figure 30: Predevelopment 10 year ARI maximum flood depth (m)

5.2.3 100 year ARI scenario

Similar to the 10 year ARI event, the post development 100 year ARI event maximum flood depths (Figure 17) downstream of the drainage corridors are greater than the predevelopment maximum depths (Figure 31).

However the maximum hazard (velocity-depth) maps for both the 100 year ARI predevelopment (Figure 19) and post development (Figure 32) scenarios show a value of approximately 0.3 m^2/s across most of the inundated area downstream of the ponds. This indicates that there is a relatively low risk of increased erosion due to the proposed development.

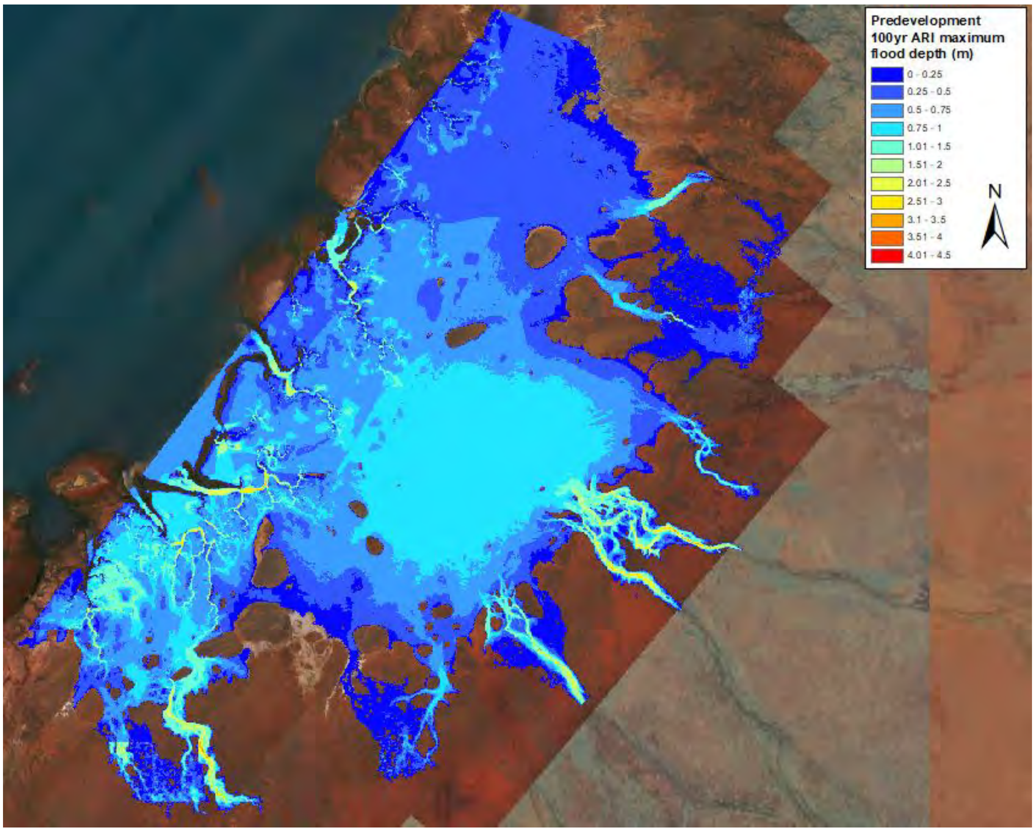


Figure 31: Predevelopment 100 year ARI maximum flood depth (m)

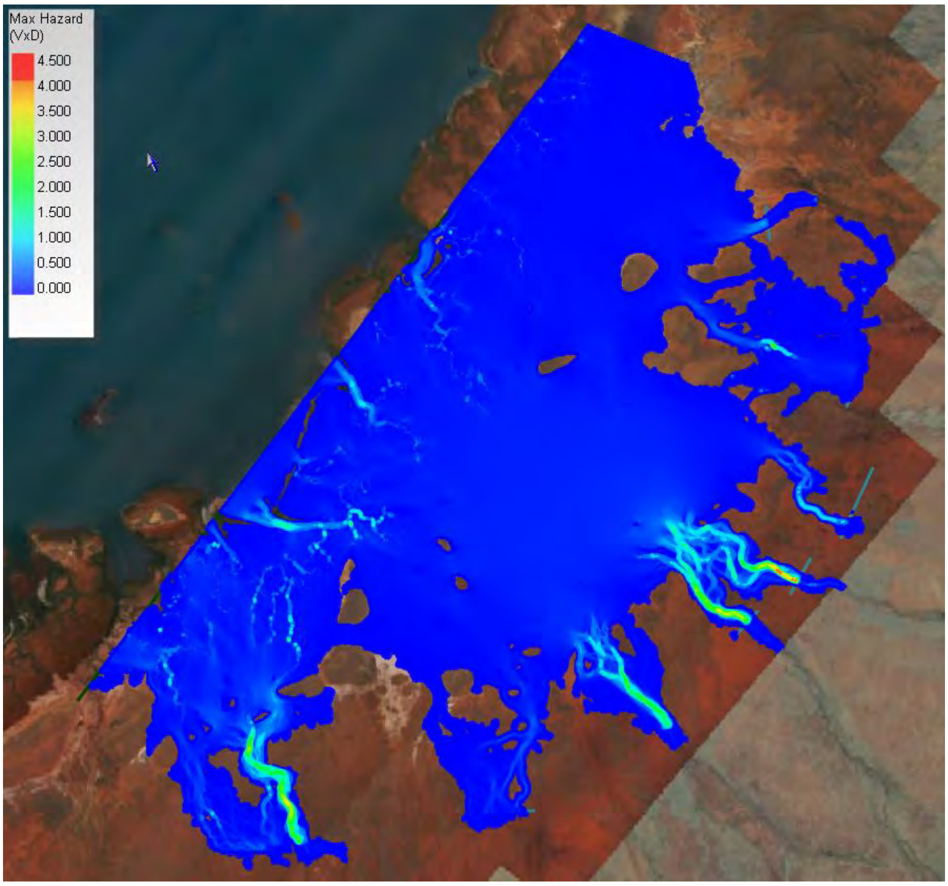


Figure 32: Predevelopment 100 year ARI maximum hazard (velocity x depth) (m^2/s)

5.3 Summary of hydrological impacts

5.3.1 Flow depth and velocity

The hydrological impacts of the project in terms of water levels and velocities are generally limited to immediately upstream and downstream of the ponds and based on hazard maps the potential for impacts related to scour or erosion is likely to be low due to the low flow velocities.

It should be noted that this simulation assumed a low sea level; during higher sea level conditions when the intertidal zone is inundated water levels will be controlled by the sea level thus further reducing the potential for impact from flood discharges to the coast.

5.3.2 Inundation time, sediment loads and currents/ jets

Impacts from the salt ponds on inundation times is expected to be minor. The flood flows discharge to coastal mudflats which in turn are flooded and discharge to the sea via tidal creeks. The salt ponds do not affect the tidal creeks or the mechanism by which flood flows discharge to the sea, except for the removal of some of the mudflat area. The impact of this is likely to be a reduction in inundation time due to a reduction in the available flood storage area/ capacity of the mudflats. However, the dominant process that will determine the inundation regime/ duration is the coincident tidal conditions during flood events.

The channelisation of flows discharging to the mudflat via the drainage corridors has the potential to create jets or currents. Higher velocity flow (such as occur in a culvert) can retain its jet-like character for some distance as it passes downstream. However, it is noted that the modelled flow velocities within the drainage corridors are relatively low in the 1 year ARI event and the estimated extent of velocity impacts is limited to a small area downstream of the drainage corridors, indicating that there is limited potential for this to occur.

The concentration of flood flows into drainage corridors also has the potential to impact sediment loads and distribution. Whilst the salt ponds and drainage corridors are not expected to generate significant sediment loads (due to the low gradients and flow velocities) the natural sediment loads of the upstream catchments will be concentrated, via the drainage corridors, to the points of discharge to the mudflats. Here sediment laden floodwaters will mix with sea water and be discharged to the sea via tidal processes. The potential impact of redistribution of sediment loads on downstream habitats has not been assessed.

5.3.3 Freshwater flows to environment

Another potential hydrological impact related to the altered drainage flow paths (i.e. the concentration of flood flows into drainage corridors through the salt ponds) is the potential effect on ecological processes downstream that rely on freshwater inflows.

Under existing conditions flood flows discharge via creeks to the coastal flats at about the location of the upstream boundary of the proposed salt ponds. Here the flood flows inundate a wide expanse of mudflats before ultimately draining via the tidal creeks, i.e. there are no defined flow paths connecting the upgradient creek systems to the tidal creeks.

Under post-development conditions, the flood flows from the upgradient creeks will still discharge to the coastal flats, but via constructed drainage corridors to the downstream boundary of the salt ponds adjacent to the tidal creeks. As the discharge locations will be confined to the drainage corridors the freshwater discharge will be more concentrated than predevelopment, with some areas receiving reduced freshwater discharge. Flood flows will still discharge to the ocean via inundation of the coastal flats and subsequent discharge via the tidal creeks during lower tide conditions. This process is expected to somewhat limit the impact of the mine infrastructure on the location and distribution of freshwater inputs to the intertidal ecosystems.

The ecological importance of freshwater inputs to the environment from the fluvial (freshwater) flooding regime is beyond the scope of this study. However, it is noted that the occurrence of fluvial floods is highly variable and infrequent compared to the very regular tidal inundation that occurs across the mangrove and algal mat areas. Therefore, it is anticipated that these habitats are not likely to be freshwater dependent.

6 CONCLUSION

6.1 Summary

This surface water assessment was undertaken to support the definitive feasibility study and most current design layout and provides an overview of the predevelopment and proposed post-development hydrology as well as concept level flood management design.

The project area is traversed by several creeks, with catchment sizes ranging from 33 to 422 km^2. The salt ponds are situated on very flat terrain at the point where the upgradient creeks discharge to the coastal mudflats. Drainage corridors are required to carry flood water runoff from the rear of the ponds though to the front of the ponds adjacent the ocean, as without them flows from upstream catchments may be dammed against the rear of the salt ponds.

A hydraulic model was used to simulate flood flows through the drainage corridors provided by the proposed pond layout. The model incorporated details of the flood bunds, floodways and lateral drains.

The bund heights have been set above the 100 year ARI flood elevation, with floodways set at the 50 year ARI flood elevation to allow for some discharge into the ponds in larger events and reduce flood levels at the rear of the ponds.

With floodways set at above the 50 year ARI top water level and with relief culverts, the estimated pond water level rise resulting from discharge to the salt ponds during a 100 year ARI event is expected to be minor (<0.1m).

Estimated post development maximum flood depths occur upstream of the flood bunds within the existing creek channels and are generally below 1, 2.5 and 3.5 m in the 1, 10 and 100 year ARI events respectively. Maximum flood depths are lower downstream of the flood bund within the drainage corridors and downstream of the ponds.

Hazard mapping results show that there is relatively low risk outside of the creek channels and drainage corridors with velocity-depth product values generally less than 0.3 m^2/s. The velocity-depth product in Drainage Corridor B is notably higher, with a value of around 1 m^2/s in the 10 year ARI event and 3 m^2/s in the 100 year ARI event (up to 5.5 m^2/s where the corridor narrows for services crossing).

The hydraulic model was used to compare flow depths and velocities for existing conditions and post-development conditions. The impact of the salt ponds on peak flood levels and velocities was found to be generally minor and limited to areas immediately upstream and downstream of the salt ponds. Other potential hydrological impacts have been identified, such as changes to the distribution of freshwater and sediment loads to the downstream habitats, however these have not been defined in this study as they are largely driven by tidal processes. It is recommended that the hydrological impacts of the mine infrastructure on the downstream environment be assessed in the context of the sensitive algal mat and mangrove habitats that occur in the intertidal zone immediately downstream of the ponds.

In general, the hydrological impacts from the salt ponds are not anticipated to present a significant physical or ecological risk to the downstream environments; however it should be noted that this study has not included any investigation of mangrove or algal mat sensitivity to the hydrological factors that have been discussed.

6.2 Further studies

As the project progresses through subsequent design and regulator assessment phases, further detailed hydrological investigations will be required. This may include the following:

- Refinement of hydraulic model to include greater level of infrastructure design detail;
- Further assessment of flow velocities, scour potential, appropriate erosion protection measures, sedimentation basin design etc;

- Post-closure design planning (to demonstrate that surface and groundwater hydrological patterns and quality reflect original conditions etc).

7 REFERENCES

RPS. 12 October 2017a. Pre-feasibility surface water assessment, Mardie Salt project. Prepared for BC Iron Limited. West Perth, Western Australia.

RPS. 27 October 2017b. Pre-feasibility hydraulic assessment, Mardie Salt project. Prepared for BC Iron Limited. West Perth, Western Australia.

RPS. 10 November 2017c. Pre-feasibility hydraulic assessment, Mardie Salt project. Prepared for BC Iron Limited. West Perth, Western Australia.

RPS. 21 May 2018. Pre-feasibility surface water assessment, Mardie Salt project. Prepared for BCI Minerals. West Perth, Western Australia.

RPS. 16 August 2019. Hydraulic modelling for rear-of -pond flood levels, Mardie Salt. Prepared for GR Engineering Service

Appendix A

Mardie Salt Project – Pre-feasibility surface water assessment (RPS 2017a)

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Attn: Les Purves
Senior Environmental Advisor
BC Iron Limited
Level 1, 15 Rheola Street West Perth WA 6005

Dear Les

RE: Mardie Salt Project - Pre-Feasibility Surface Water Assessment

Please find an initial Surface Water Assessment for the Mardie Salt Project. The objective of this assessment is to quantify the potential flood flows impacting the project site from the inland creek catchments to inform preliminary engineering design (e.g. required bund heights, drainage corridor widths etc).

This report provides the results of the initial hydrological assessment, namely the estimated flood flows impacting the site. Due to the extremely flat terrain of the site, the significant influence of detention storage behind the pond walls and the interaction of flood flows with sea levels, a 2D hydraulic model will be required to assess the interaction of flood flows with the salt ponds (i.e. determination of flood levels). Hydraulic modelling will be undertaken by RPS as the next stage of this assessment to inform flood protection and drainage channel design considerations.

1.1 Background

BCI's proposed Mardie Salt Project is located between the Robe River and Fortescue River mouths in the north-west of Western Australia (refer Figure 1). The project involves the production of 3.0-3.5 million tonnes per annum of sodium chloride salt from a seawater intake and series of solar evaporation ponds.

A scoping study has been completed to prove the technical and economic feasibility of the project, and now additional pre-feasibility studies are required to further inform the technical and approvals feasibility of the project, including the requirement for a baseline hydrological assessment.

The proposed evaporation ponds are located on mud flats on the landward side of the coastal mangrove areas and stretch over 20km of coastline. Several creeks flow through the area that will be occupied by the evaporation ponds.

Among the potential impacts to the pond areas are changes to the surface water hydrology, and the requirement for flood protection (bund walls, diversion drains, scour protection measures, etc). Mangroves, fringing mudflats and algal mats are sensitive habitats, and the project will need to demonstrate that impact to these habitats can be minimised or mitigated through appropriate design.

1.2 Definitions

100 year ARI flood - the flood having an average recurrence interval (ARI) of 100 years. It has a 1% chance of occurring or being exceeded in any one year, and a 50% chance of being experienced at least once in any average life span of a person.

The 10 year ARI flood has a 10% chance of being exceeded in any one year, the 2 year ARI flood has a 50% chance of being exceeded in any one year.

Floodplain - The portion of a river valley adjacent to the river channel which is covered with water when the river overflows its banks during floods.

1.3 Hydrology

WA has three broad climate divisions - the south-west corner of WA with a Mediterranean climate, with long hot summers and wet winters; the central eastern areas of WA with arid land or desert climates and the area of interest, the dry tropical northern part of the State, receiving summer rainfall.

The average annual rainfall at nearby Mardie Station is 277mm (BOM, Site number 5008) as measured over a 129 year period (1885-2017), but annual rainfall is highly variable with a minimum of 9mm recorded, and a maximum of 886mm.

The majority of rainfall occurs January-June (38-63mm average monthly rainfall), and July-December is typically drier (average monthly rainfall 1-9mm).

There is limited evaporation data available, but the annual Class A pan evaporation at Mardie, as estimated by BCI, about 3250mm pa, varying from 12mm/d in summer to 5mm/d in winter.

1.4 Haul Road Corridor Hydrology

1.4.1 General

A study was carried out on a haul road from the proposed Bungaroo South mine to the Cape Preston area for Iron Ore Holdings (ref: Buckland Project Haul Road Corridor Hydrology, April 2014, RPS 1488T/003a). The haul road route initially headed west across hilly terrain in the Hamersley ranges to the North West Coastal Highway. From the highway, the road route continued north in undulating terrain, generally paralleling the highway alignment to the west, and following the Dampier Bunbury Natural Gas Pipeline alignment.

Some of the haul road creek crossings flow north west and through the current Mardie Salt area of interest. These include:

- Robe River – the river has a catchment area of 7,100km² at the Yarraloola gauging station installed near the North West Coastal Highway bridge. The largest flows recorded (Cyclone Monty in Feb/Mar 2004 and the tropical depression over the Pilbara in February 2009 overtopped the Robe highway bridge;
- Peter Creek (Catchment 95) – bridge at the highway, with a catchment area of 188km². The 10 year flood was estimated as 110m³/s, and the 100 year flow as 515m³/s;
- Gerald Creek – not included in the previous study, as this creek does not cross the haul road per se, but forms downstream, possibly gaining flow in very flat terrain from breakout flows from Trevarton Creek, and less likely Peter Creek;
- Trevarton Creek (Catchment 100) - floodway at the highway, with a catchment area of 96km². The 10 year flood was estimated as 71m³/s, and the 100 year flow as 324m³/s;
- 6 Mile Creek (Catchment 109) - small culvert at the highway, with a catchment area of 53km². The 10 year flood was estimated as 48m³/s, and the 100 year flow as 216m³/s;
- Various unnamed creeks north of 6 Mile Creek –not included in the previous study, as they do not cross the haul road. These creeks form downstream, as local runoff, but also potentially as part of the channel system draining the broader Fortescue River floodplain (west of the main channel);

- Fortescue River (Catchment 117) - 400m long, high level bridge at the highway, with a catchment area of 18,360km². The 10 year flood was estimated as ~5,000m³/s and the 100 year flow as ~20,000m³/s. The haul road crossing is on a new alignment, crossing the river on a floodway downstream of the road bridge. The river flows were measured at the "Jimbegnyinoo Pool" just upstream of the road bridge, and now at "Bilanoo" at the road bridge gauging station. The same rain events that overtopped the highway at the Robe River bridge also overtopped the Fortescue River bridge.

1.5 Intensity-Frequency-Duration (IFD)

Intensity-Frequency-Duration (IFD) data is required to characterise the storm intensity in the area under consideration. This is generally provided by techniques in ARR (Australian Rainfall and Runoff), a national guideline for the estimation of design flood characteristics in Australia, published by the Institution of Engineers Australia. New IFD design rainfalls were produced in 2016.

Typical IFD data for this area is as follows:

Table 1 IFD Data - Mardie

ARI	1 year (mm)	2 year (mm)	5 year (mm)	10 year (mm)	20 year (mm)	50 year (mm)	100 year (mm)
1 hour	23	41	41	50	59	73	83
2 hour	29	51	51	64	77	95	109
6 hour	39	70	75	95	117	149	174
12 hour	47	87	95	124	155	198	233
24 hour	57	106	119	156	196	251	296
72 hour	72	134	148	192	238	301	354

The 1 hour rainfall is typically about 0.3-0.4x the 24 hour rainfall. The 72 hour rainfall is typically about 1.20-1.25x the 24 hour rainfall.

Information on storms exceeding the 100 year ARI event is not (readily) available in ARR, but by extrapolation, estimates can be made. The 1000 year ARI and Probable Maximum Precipitation (PMP) rainfalls are in the order of 1.7x and 3.3x the 100 year rainfalls respectively.

The (sliding, not calendar day) 24 hour rainfalls are estimated as:

- 2 year 106mm
- 5 year 120mm
- 10 year 160mm
- 20 year 200mm
- 50 year 250mm
- 100 year 300mm
- 1000 years 500mm
- PMP (Probable Max. Precipitation) 1,000mm

1.6 Flood Estimation - Creeks

1.6.1 General

The catchment details for the development area are shown in Figure 2. The project area itself is located on terrain gently sloping from the North West Coastal Highway to the north west at a low 0.15-0.20% gradient.

Based on local rainfall and runoff trends for the area, the flood flows (as a proportion of the 100 year ARI flood) would typically be:

Table 2 Typical Presumptive Flood Flows as Proportion of the Q100 Flood

ARI (years)	Fraction of Q100 flood
2	0.05
5	0.15
10	0.28
20	0.45
50	0.73
100	1.0
1000	~2.1
PMF	~6.3

1.6.2 Peak Stream Flow Estimation

There are no relevant streamflow gauging data / gauged catchments from which flood estimates may be made directly. Flood estimation therefore relies on Australian Rainfall and Runoff (ARR) flood estimation methods for ungauged catchments, or an individual customised rainfall runoff model for each catchment.

In this case the RAFTS nonlinear rainfall / runoff program has been used (as used for the previous Haul Road Corridor Hydrology study).

RAFTS uses design rainfall data derived from ARR. RAFTS requires customising for each catchment (parameters include terrain slopes, roughness, local rainfall data and rainfall losses). The catchments for the relevant creeks were divided into sub-catchments, with routing links between. The program calculates flood flows (hydrographs) by simulating rainfall over a catchment with time, removing losses to calculate the rainfall excess runoff, and then routing this runoff through the model reaches. The RAFTS 'pern' or surface roughness factor affects the storage factor and was set at 0.045.

1.6.3 Rainfall Runoff Results

The relevant estimated 100 year ARI flood flows, impacting the project are shown in Table 3.

Table 3 Estimated Flood Flows (m³/s)

ARI (years)	Ac (km²)	2 ARI (yrs)	5 ARI (yrs)	10 ARI (yrs)	20 ARI (yrs)	50 ARI (yrs)	100 ARI (yrs)	1000 ARI (yrs)	PMF
Peter Ck	422	27	80	149	240	389	533	1,119	3,356
Gerald Ck	153	16	49	91	146	236	324	679	2,038
Trevarton Ck	172	18	55	103	165	268	367	771	2,314
6 Mile Ck	164	19	56	104	167	271	372	780	2,341
Fortescue River	18,360	1,090	2,850	5,000	8,080	13,500	20,000	42,000	126,000

The 100 year ARI local flows may be generally estimated as $Q_{100} = 36 \times Ac^{0.45}$ (where Ac = catchment area in km²) based on typical RAFTS estimates.

1.7 Flood Estimation - Sea

Normal tidal variations cause inundation over the coastal flats. Mean neap tide levels vary around +/-0.5m from mean sea level, and spring tide levels vary around +/-1.8m. The highest and lowest astronomical tides, which are the highest and lowest tidal levels which can be predicted to occur under average meteorological conditions, vary by approximately +/-2.4m from mean sea level. The Highest Astronomical Tide (HAT) in the area is RL2.4m.

Under abnormal meteorological conditions, greater variations in the tidal range are possible, and actual still water sea levels are produced by the interaction of astronomical tides, storm surges and wave set-up.

The Pilbara coast cyclone season runs from December to April, peaking in February and March. Potentially the most destructive phenomenon associated with cyclones that make landfall, is storm surge, a raised mound of seawater typically some 50km across, and up to several metres higher than the normal tide. The worst scenario arises when a severe cyclone crosses a coastline with a gently sloping seabed, at or close to high tide.

RPS have recently undertaken a metocean analysis to provide estimates of still water level for various return periods. The estimated 100 year still water sea level is RL4.2-4.3m, about 2m higher than HAT. The 10 year sea level is 3.5-3.7m, 1.3m higher than the HAT. These sea levels would flood the coastline for several km from the mean sea level (RL00m) location.

1.8 Flood Level Estimation - Joint Probability

1.8.1 General

The evaporation ponds will be impacted both by creek flooding and sea surge, and any flood consideration works should account for both.

The flood level in the ocean is an end downstream condition which is required when hydraulic modelling flood flows in the various creeks - a joint probability situation. Flooding of infrastructure located near the coast can be impacted either by creek flooding from the inland side or high sea surge levels from the ocean side.

In this regard, it is noted that the largest river floods in the Fortescue River, and ocean storm surges both occur as a result of tropical cyclone activity; generally a cyclone related flood in the river would occur sometime after any associated abnormal sea level (the height of which can vary greatly), as the cyclone tracked across the coast and moved inland. Hence significant storm surge and river flooding are not dependent, and do not generally occur simultaneously.

The creeks of interest are much smaller, and the smaller catchments near the coast are likely to increase the degree of dependence a little between the two flood mechanisms.

A common way on handling this joint probability between the two flood mechanisms is provided in for example, the “Flood Risk Management Guide” (NSW Department of Environment, Climate Change & Water 2010/759, August 2010). This adopts an approach using (a) 20 year ARI catchment flooding with 100 year sea levels, and (b) 100 year flooding with 20 year sea levels. The “Karratha Coastal Vulnerability Study” (JDA, August 2012) used the 100 year flood flow in conjunction with 20 year sea levels (estimated as RL3.9m) as the downstream boundary condition (the joint probability between river flood levels and storm surge was studied, but no obvious correlation was found).

1.8.2 Required Level of Protection

The selection above of the 100 year criterion is often used (particularly for public infrastructure).

A design return period (Average Recurrence Interval) and associated flood level is required upon which to assess the potential for flooding of the project area. The appropriate ARI needs to be selected upon which to base the flooding assessments. A 20% chance of exceedance in the project life is a common design assumption.

For a 20 year life of project, there is an 18% chance that a 100 year flow event (or higher) may occur (10 year event 86%, 20 year 63%, 50 year 33% chance of occurring). Commonly, structures are designed to withstand an event with the probability of exceedances of 20% over the life of the operation. However this value may be increased or reduced after considering costs associated with additional flood protection of larger events, environmental impact of failed structures and the impact on project operations of reinstating the structure after flooding. Thus the accepted level of protection may be refined at the discretion of the designer.

If a lower level of protection is required, e.g. the 50 year ARI protection level, then the joint probability might be 50/10 year flooding (~4.1/3.6m sea levels); the 20 year protection level as joint 20/4 year flooding (~3.9/3.1m sea level); and 10 year protection level as joint 10/2 year flooding (3.6/2.6m or about the HAT sea level).

1.9 Fortescue River Break-out

Part of the Mardie Salt site is potentially impacted by “breakout” flows from the Fortescue River during major flood events.

Upstream from the North West Coastal Highway, the Fortescue River is generally contained between ridges.

However, downstream of the highway, the topography becomes less pronounced and the river flow path less constrained. On the west side of the main river channel, there is a noticeable north-south ridge line at about RL30-40m elevation. The river floodplain at this point is generally 5km wide, with numerous smaller flow channels developed, discharging in the same general direction as the main channel.

However during large flood events, river flows can “break-out” from the main floodplain. There is a significant “break-out” area between the north end of the ridgeline and Coolangara Hill (a small hill 15km north of the highway, elevation ~RL45m) which encroaches into the main floodplain and redirects high level flood water upstream away from the main river channel system. The floodplain east of the hill then reduces to about 4km wide.

Break out flows generally head north-westerly towards the coast 25km away. Flows eventually exit to the ocean, at anywhere up to 25km west of the Fortescue River mouth.

A significant volume of flow would be diverted away from the main Fortescue channels in the largest floods. The Department of Water (then WRC) previously estimated a 100 year ARI flood flow of 9,220m³/s, with around 1,200m³/s of that flow (i.e. 13%) following channels north and north-west to the sea, west of the main channels. It is not possible however to estimate the quantum of break out flows without 2D hydraulic modelling over a very large area.

For a now estimated 100 year flow of about 20,000m³/s, the break out flows may be assumed as up to 20%, or 4,000m³/s, "lost" from the Fortescue River system. This high flow is spread over a very large area, and the direct impact at any location (other than in a flow channel) would be anticipated as relatively low. The impact at the coast in the larger Fortescue flows would probably be north of 6 Mile Creek.

1.10 Drainage Through the Proposed Evaporation Ponds

The proposed evaporation pond layout provides east-west channels for storm surface water flow to travel to the ocean following the larger of the natural creek lines.

The interaction of the evaporation ponds, extremely flat terrain, and varying sea levels requires the application of detailed 2D hydraulic modelling to determine design flood levels. However relevant parameters and considerations associated with the surface water through channels include:

- The channel bed roughness as Manning n of 0.045, associated with a typical natural alluvial creek bed form;
- A suitable freeboard can be applied to the resulting flood levels behind the eastern walls or channel flow, to cater for the storage behind the ponds, the risk in regard to the flood flow estimates, the hydraulic modelling uncertainties, uncertainty in topographical data, roughness factors selected in the model, wave action, non-uniform flow patterns, and settlement and erosion which may reduce the bund height.
- Temporary ponding / storage of water behind the evaporation pond wall, to control the flow through the channel. Due to the flat terrain, very large temporary storage volumes are anticipated behind the eastern walls, and this storage will significantly reduce the peak flows generated by the impacting creek;
- Concurrent sea levels will also play a major role in the determination of flood levels.

We trust that this meets your current requirements, however please contact the undersigned if you require any additional information.

Yours sincerely
RPS

Dan

Dan Williams
Supervising Scientist

Rhod

Rhod Wright
Principal Civil / Water Resources Engineer

N

0 200 400

Kilometers

Scale: 1:10,000,000 @A3

GDA 1994 Zone 50

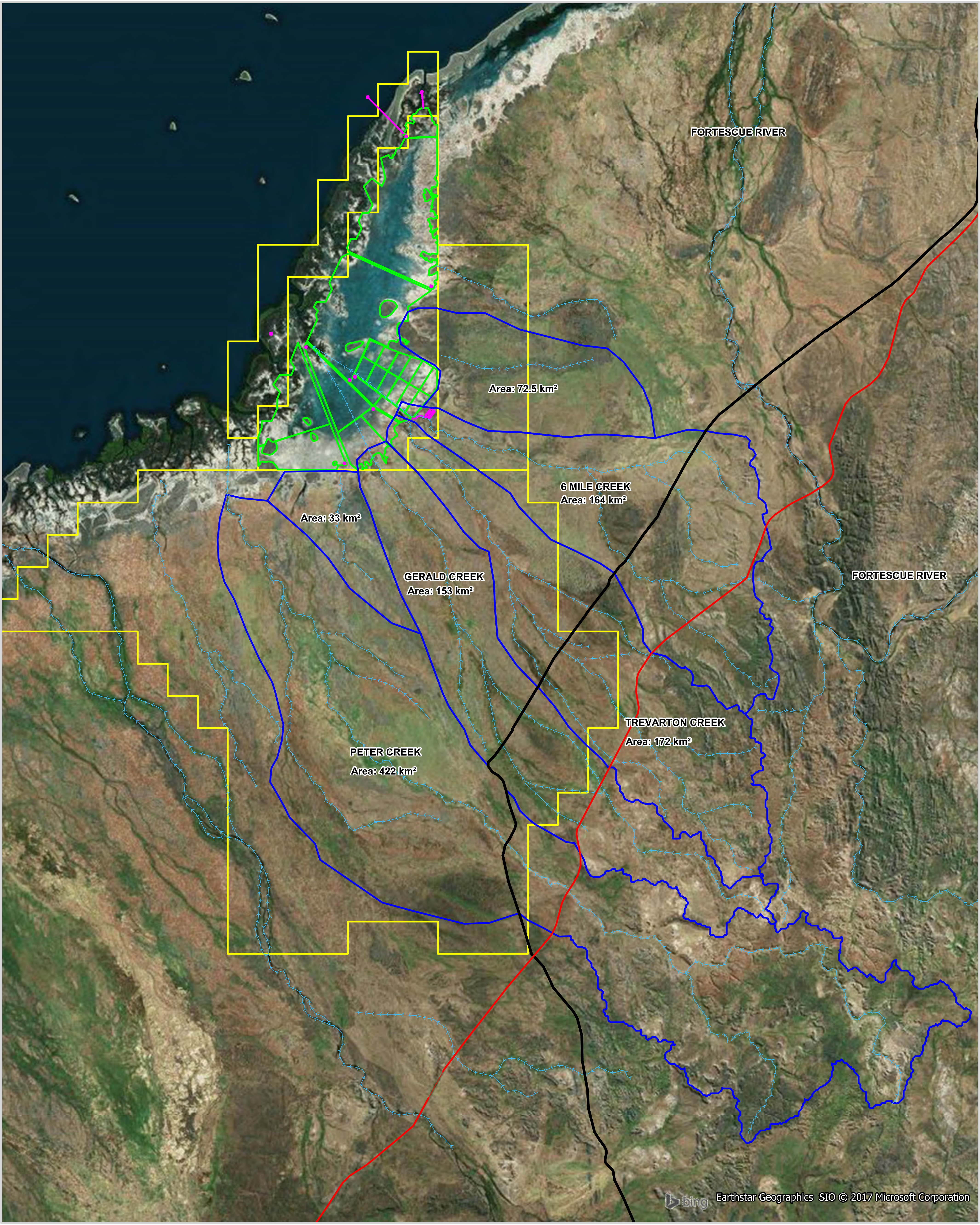
AUTHOR:	AB	REPORT NO:	003a
DRAWN:	AB	REVISION:	A
DATE:	12/010/2017	JOB NO:	EWP72667

RPS

FIGURE 1

REGIONAL LOCATION PLAN

Location: F:\Jobs\EWP72667 - BCI Mardi Salt\Spatial_Data\MapInfo\Workspaces\EWP72667_Figure1



AUTHOR:	AB	REPORT NO:	003a
DRAWN:	AB	REVISION:	A
DATE:	12/10/2017	JOB NO:	EWP72667

LEGEND

- Mardie Tenement Boundary
- Pond Layout
- Catchment Boundary
- Flow Paths
- Proposed Haul Road Alignment
- North West Coastal Highway

RPS

FIGURE 2

SURFACE WATER CATCHMENTS

Location: F:\Jobs\EWP72667 - BCI Mardi Salt\Spatial_Data\MapInfo\Workspaces\EWP72667_Figure1

Appendix B

Mardie Salt Project – Pre-feasibility hydraulic assessment: Preliminary results (RPS 2017b)

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Date: 27 October 2017 **Direct Dial:** 08 9211 3510

Attn: Les Purves
Senior Environmental Advisor
BC Iron Limited
Level 1, 15 Rheola Street West Perth WA 6005

Dear Les

RE: Mardie Salt Project - Pre-Feasibility Hydraulic Assessment: Preliminary Results

1.1 Background

RPS previously completed a hydrological assessment for the Mardie Salt site to quantify design criteria for the creek catchments impacting the proposed salt ponds. The outcomes of that assessment were provided to BC Iron Limited in a letter dated 12 October 2017.

The design flood flows estimated therein have subsequently been used in hydraulic modelling to estimate flood levels at the site and potential design level requirements for flood protection. This letter provides the outcomes of the hydraulic modelling completed for the Existing Conditions and the Post Development Base Case Scenarios.

As discussed at the end of the report, further modelling is currently in progress to provide flood levels for:

- A more realistic design scenario (the assessment herein focuses on the preliminary pond layout provided by BC Iron Limited and identifies some required modifications)
- Lower magnitude flood events (e.g. 10 and 20 year ARI).

1.2 Hydraulic Assessment Methodology

A 2D hydraulic model was developed to cover the salt ponds and drainage channels, the creek channels and floodplain area immediately upstream of the salt ponds, and the coastal margin downstream of the salt ponds. The 2D grid is based on the 5m resolution DEM provided by BC Iron and has a 15m cell size for pre-development (Existing Conditions) simulations and a 10m cell size for post-development simulations (a smaller grid size is achievable for post-development simulations due to the large salt pond areas being made inactive in the 2D model).

Post-development simulations also use 1D “links” within the model to represent the proposed drainage channels through the salt ponds. This allows the drainage channels to be modelled with greater accuracy.

Design flood hydrographs from the hydrological assessment were input to the upstream boundary of the 2D model at the location of each major creek – Peter Creek, Gerald Creek, Trevarton Creek, 6 Mile Creek and the three other creek catchments identified in the hydrology assessment which flow into the project area from the southeast. Figure 1 illustrates the model setup.

The downstream boundary condition for the 2D model is a tidal sea water level. The assessment included simulations without storm surge (nominal 1 m AHD downstream boundary level) and with surge (3.9 m AHD downstream boundary level to represent the 20 year ARI storm surge level).

This report provides modelled flood levels for the 100 year ARI flood event which represents the likely upper range of design flood magnitude for engineering design and flood protection considerations. It is acknowledged that a lower design criterion (e.g. 20 year or 50 year ARI) may be acceptable for the project and these flood events are currently being modelled, with this information to be provided shortly.

1.3 Existing Conditions

The existing conditions were modelled to provide baseline flood levels for comparison with post-development scenarios. The 100 year ARI flood was modelled for the existing conditions with and without storm surge. Maximum flood water depths and levels for the existing conditions are presented in Figure 2 (without storm surge) and Figure 3 (with storm surge) whilst Figure 4 provides a difference map showing the increase in flood levels when storm surge is included.

The water depth in the creek channels immediately upstream of the planned ponds is approximately 1~2 m in the 100 year ARI event whilst. the large mudflat areas to be occupied by the planned ponds experience depths between 0.1~1.4m as broad sheet flows.

The difference map (Figure 4) illustrates the extent of the influence of storm surge. It can be seen that the area of influence is mainly at the near-coast area due to its low elevation ground level (1.5~2.5m AHD) where tidal inundation will dominate during a storm surge. The influence of storm surge diminishes further inland areas where it becomes minimal (estimated 100mm ~150mm water depth increase) at the upstream boundary of the project footprint.

1.4 Post Development – Base Case

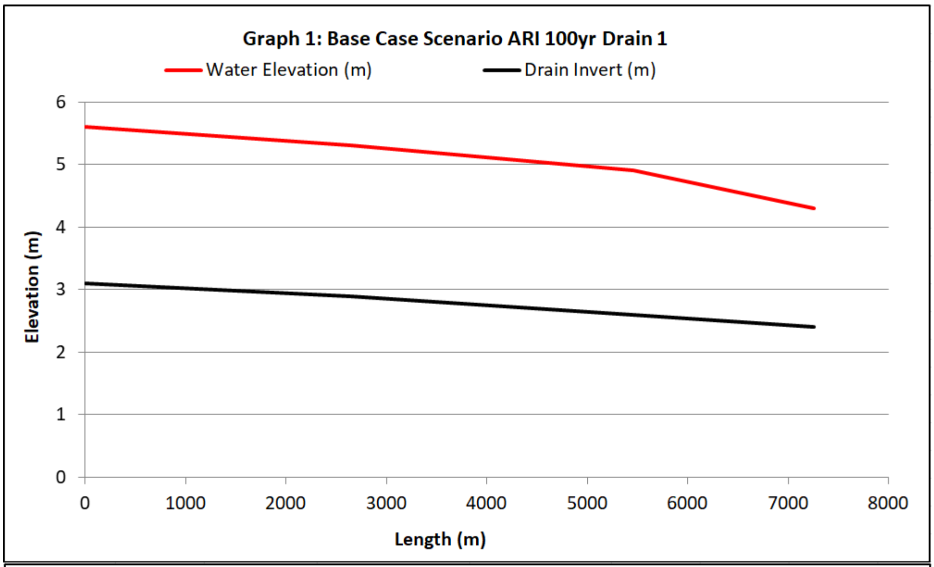
The proposed salt pond layout as provided by BC Iron was modelled as the base case post-development scenario. Drain widths of 30m, 60m and 30m were assumed for Drains 1, 2 and 3, respectively based on the relative magnitude of flood flows for their respective contributing catchment areas (drains are numbered from south to north as shown on Figure 1).

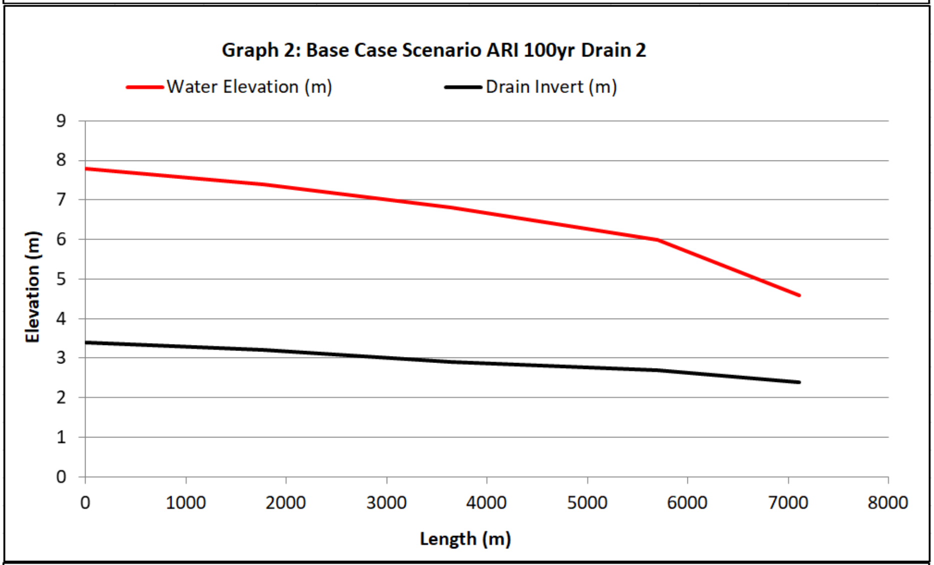
The 100 year ARI flood levels for the base case are presented on Figure 5. A difference map is also provided in Figure 6 showing the difference in 100 year ARI flood levels between the existing conditions and the base case scenario. It shows that flood levels increase by up to ~3m behind (upstream of) the salt ponds. The areas of most significant difference are associated with locations where the proposed salt pond layout has isolated creek flow paths from the low-lying mudflats, effectively damning the catchment flows and causing significant ponding.

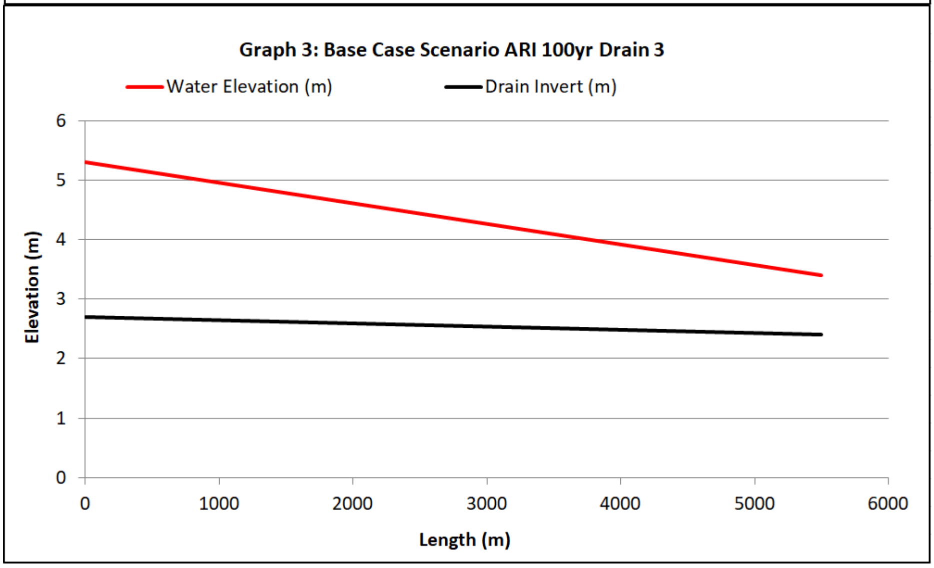
The design will need to facilitate flow from these creek catchments through the drainage corridors to the coast. This will be achieved with lateral diversion drains behind the ponds and/or by retreating the landward walls of the salt ponds further into the mudflats to provide connectivity between the creek outlets and the drainage corridors.

Graphs 1 to 3 below provide long-sections of the 100 year ARI hydraulic gradeline within Drains 1, 2 and 3 to provide an indication of potential flood protection levels (i.e. required bund heights). As shown on the graphs, maximum water depth estimated in Drain 1, Drain 2 and Drain 3 for the ARI 100 years flood event are 2.5m, 4.4m and 2.6m respectively.

Further modelling of the various design level floods (e.g. 10 years, 20 years ARI etc) will be undertaken for a more realistic design scenario (i.e. one which does not result in daming of creek catchments behind the ponds) to determine how the selection of design criteria for flood protection impacts on engineering requirements and costs.







Graphs 1-3: 100 Year ARI Hydraulic Gradeline for Drains 1, 2 & 3 – Base Case

1.5 Base Case with Storm Surge

The 100 years ARI flood levels coinciding with the 20 years ARI storm surge for the Base Case scenario are presented in Figure 7. Impact of the storm surge on water levels is demonstrated on Figure 8 which provides a difference map of flood levels with and without storm surge. It shows that the impact of storm surge on modelled flood levels ranges between approximately 0.2~1.8m on the downstream side of the salt ponds where flood levels are storm surge dominated to 0.05~0.15m on the upstream side of the salt ponds where storm surge has minimal influence.

Further modelling will include simulation of the 100 year ARI storm surge in conjunction with the 20 year ARI catchment flood.

1.6 Post Development – Modified Drains Scenario

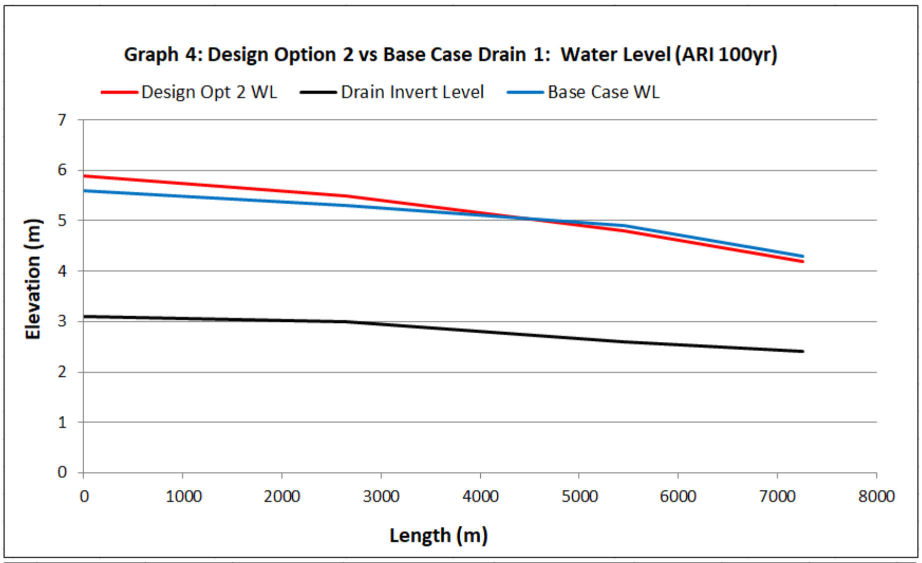
As described above, the preliminary salt pond layout provided by BC Iron Limited cuts off several creek flow paths causing a daming affect and ponding of floodwater behind the salt ponds. To address this, two alternative developed scenarios are proposed for testing with the model. These include one or both of the following modifications; using lateral diversion drains behind the salt ponds to intercept creek flows, and; increasing the drain widths to facilitate better drainage of the floodwater buildup behind the ponds. The improved drain scenarios are illustrated on Figure 9 and summarised as:

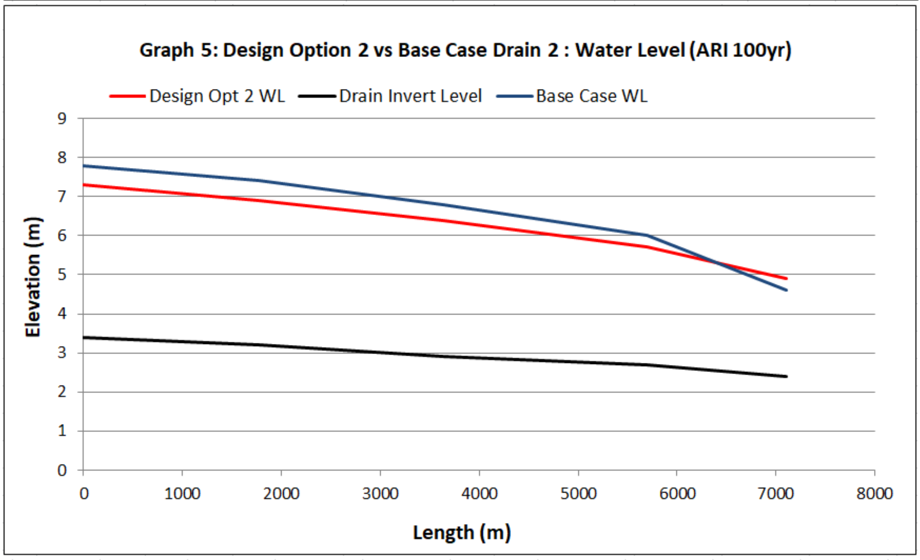
- Design Option 1 – lateral diversion drains
- Design Option 2 – lateral diversion drains and drain widths increased.

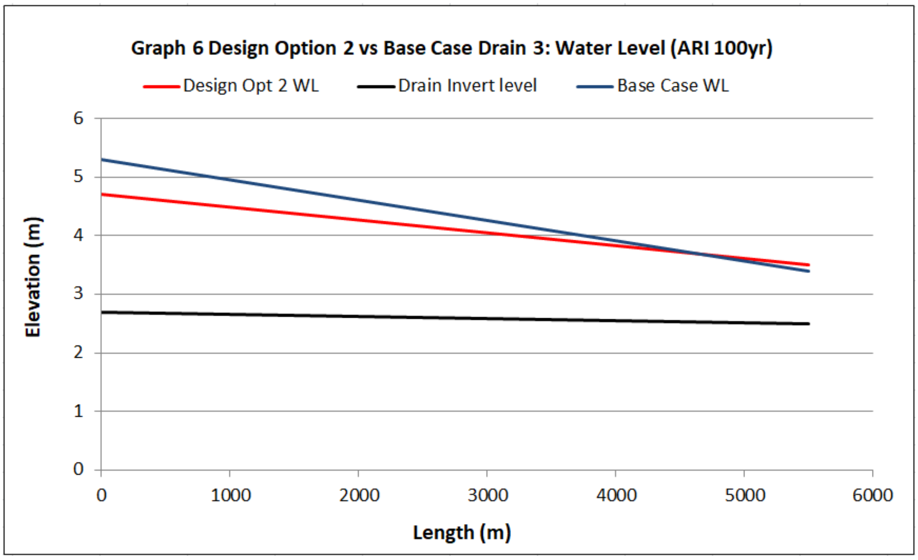
The modelling of the improved drain scenarios is still under progress and the full results are not yet available. However, the 100 year ARI flood has been simulated for Design Option 2 with the results provided in Figure 10. A difference map is also provided in Figure 11 showing the difference in flood levels between the Developed Base Case and Design Option 2 which shows that the inclusion of lateral diversion drains in Design Option 2 reduces flood levels behind the salt ponds by up to ~2m in areas that previously experienced significant localised ponding.

Also provided in Graphs 4 to 6 below are long-sections showing the hydraulic gradeline in the drains for the Developed Base Case versus the Design Option 2. It shows that the modified drain design causes the hydraulic gradeline within the drains to decrease at the upstream end in Drains 2 and 3, whilst it increases in Drain 1.

The slightly increased water level in Drain 1 is due to the increased flow that results from the inclusion of lateral diversion drains to intercept the creeks. The reduction in water level in Drains 2 and 3 is due to the increased flow capacity of the widened drainage corridors which reduces the amount of water backing up behind the ponds.







Graphs 4-6: 100 Year ARI Hydraulic Gradeline: Base Case versus Design Option 2

1.7 Final Remarks

Further modelling is currently underway to assess the relative benefits of the above drain design options (lateral diversion drains and increased drain width), and to provide flood levels for lower magnitude flood events, e.g. 10, 20 and 50 years ARI.

The information herein (along with the results of the additional modelling which will be summarised in further correspondence when available) is provided to inform preliminary project planning and engineering design.

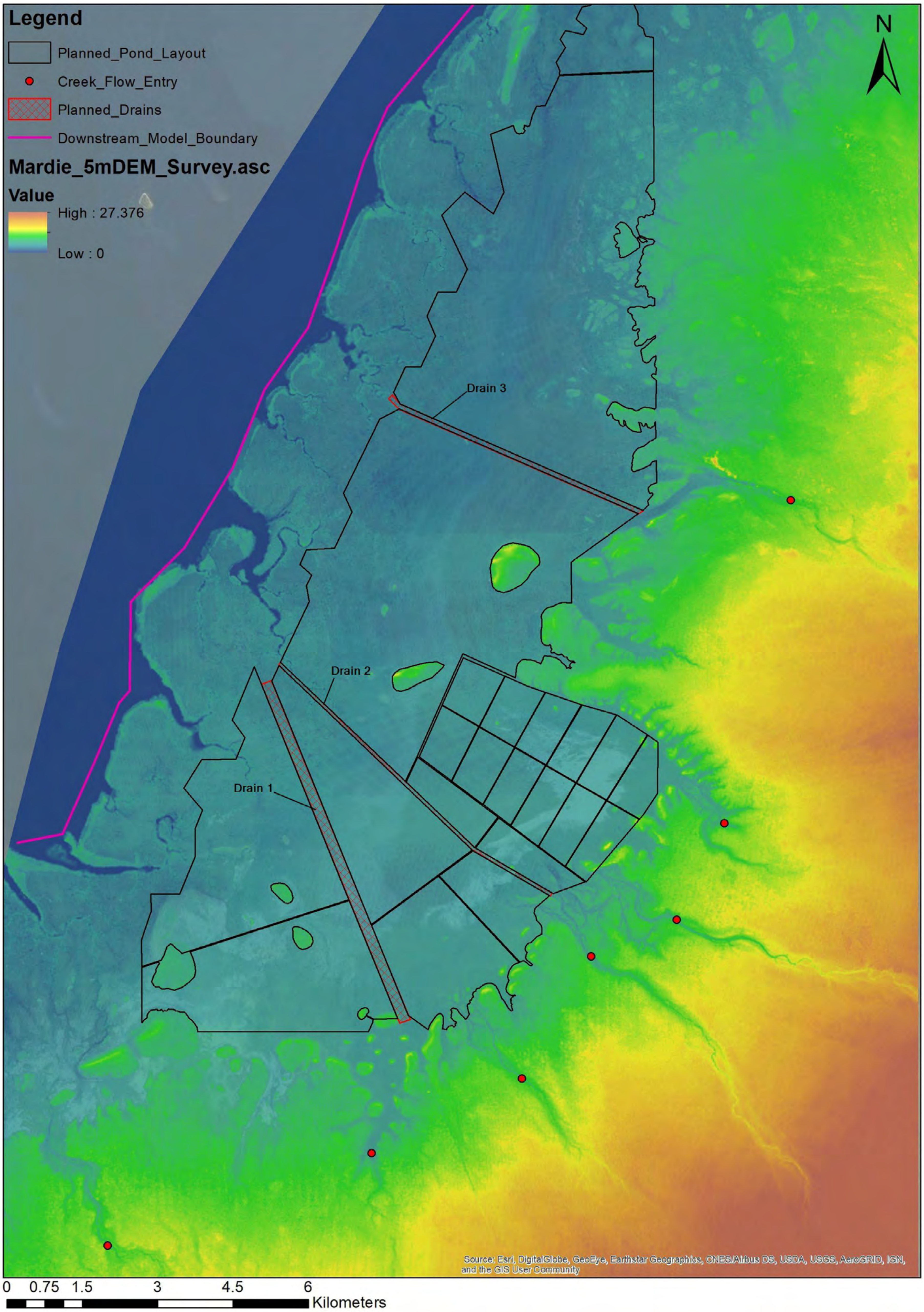
Yours sincerely
RPS

Dan

Dan Williams
Supervising Scientist

Rhod

Rhod Wright
Principal Civil / Water Resources Engineer



HYDRAULIC MODEL BOUNDARIES FIGURE 1

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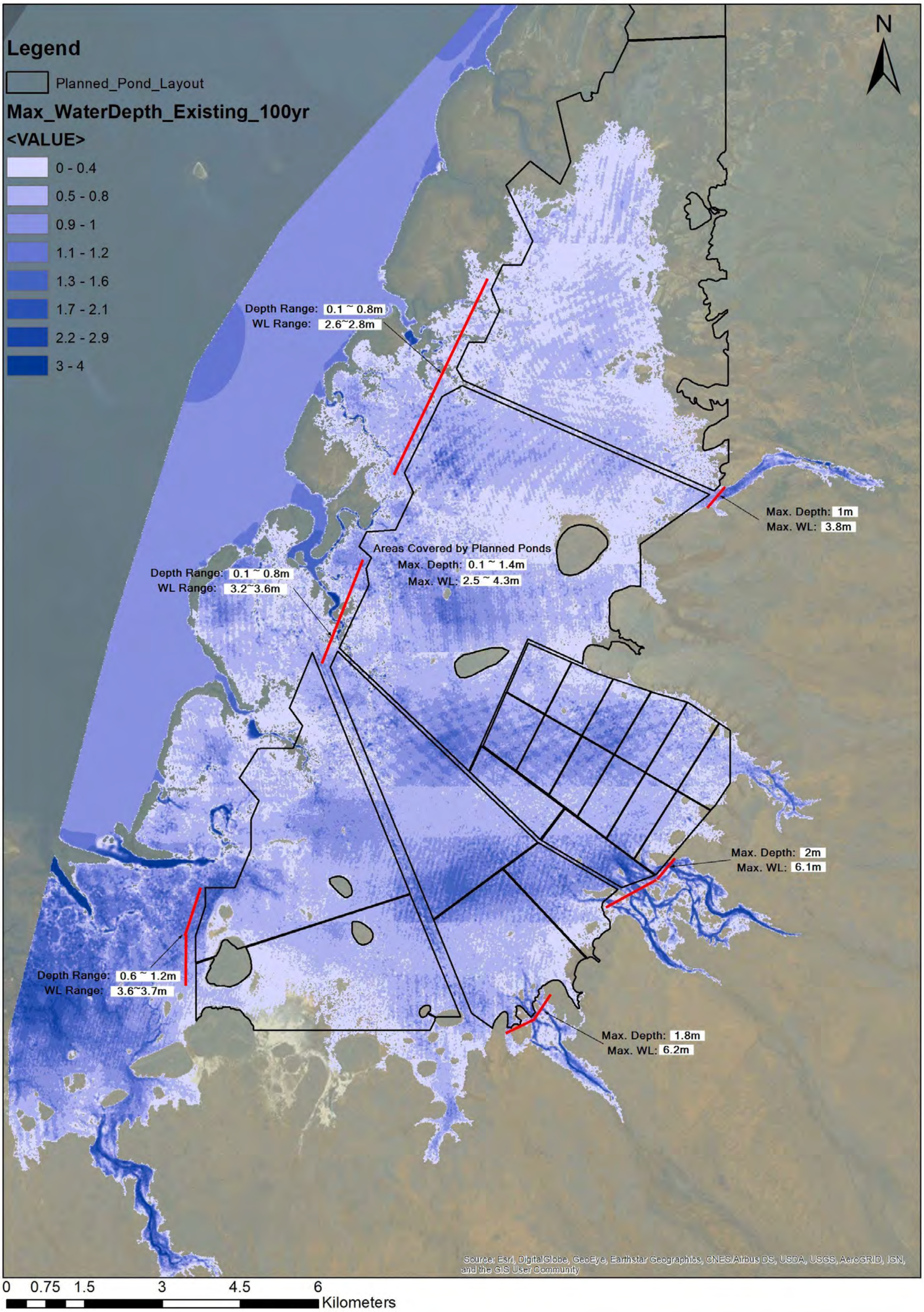

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MAX. WATER DEPTH – EXISTING CONDITIONS 100 YEAR ARI FLOOD FIGURE 2

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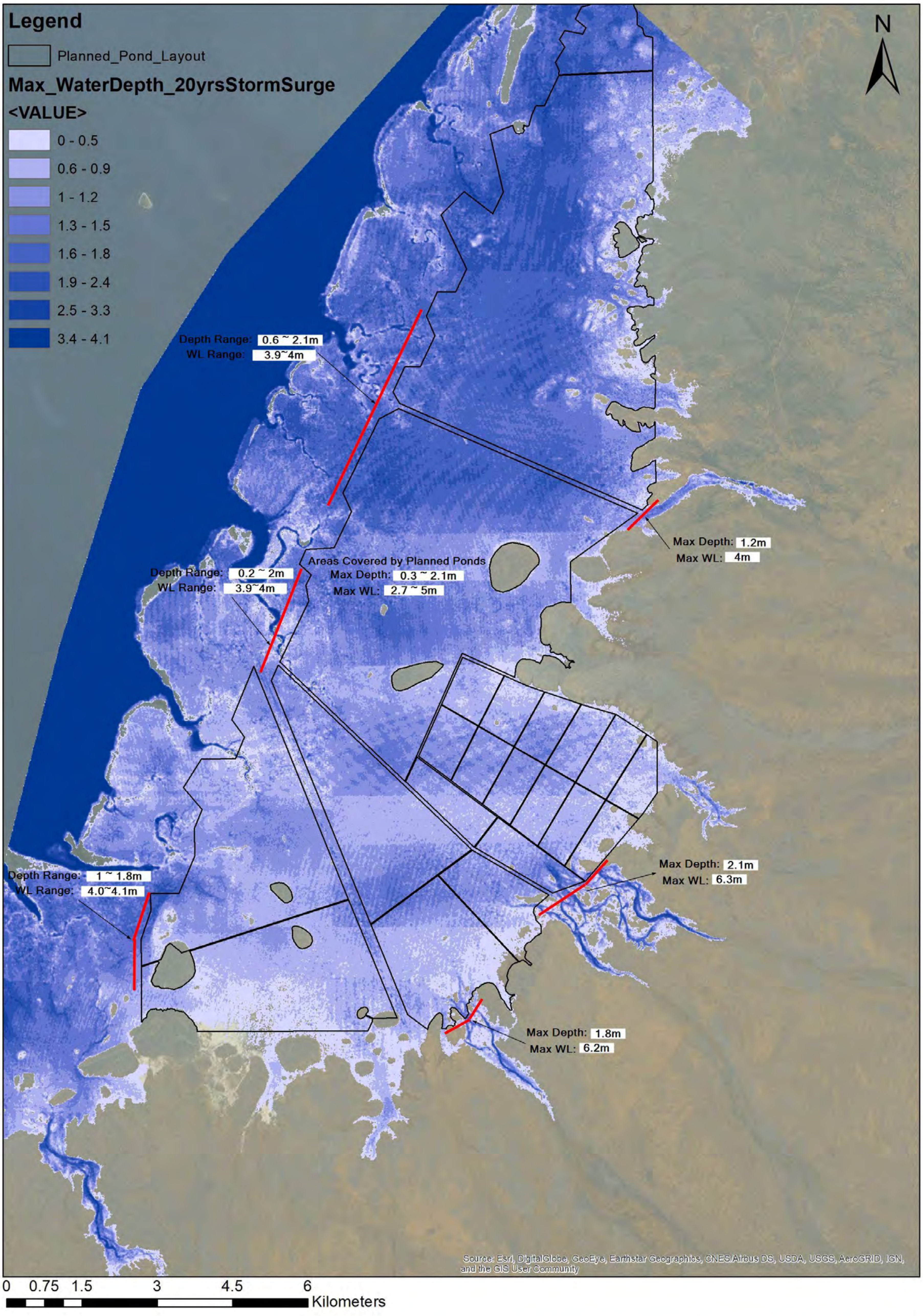

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MAX. WATER DEPTH – EXISTING CONDITIONS 100 YEAR ARI FLOOD WITH 20 YEAR ARI STORM SURGE FIGURE 3

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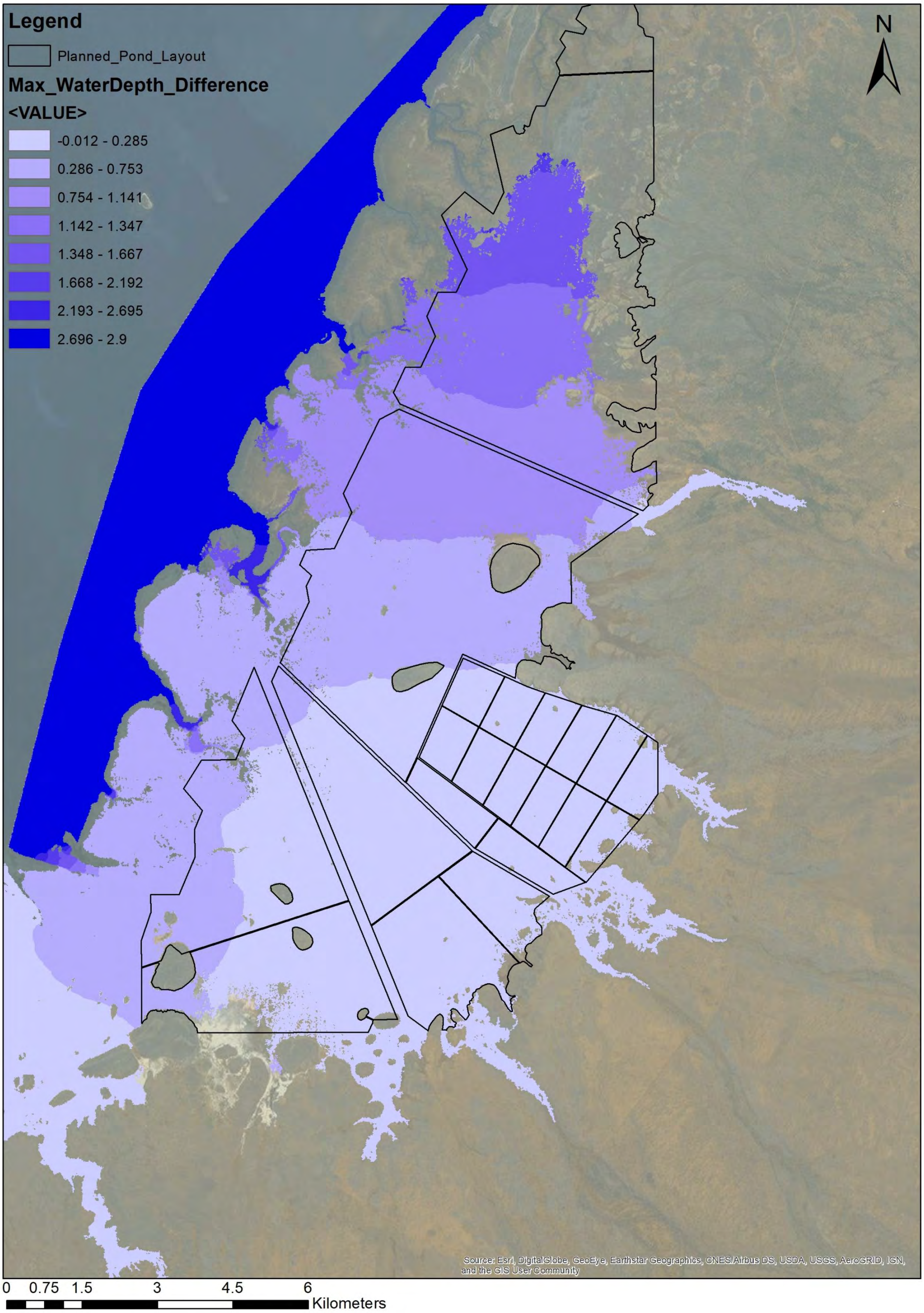

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MAX. DEPTH DIFFERENCE: EXISTING CONDITIONS 100 YEARS ARI: WITH VS WITHOUT STORM SURGE FIGURE 4

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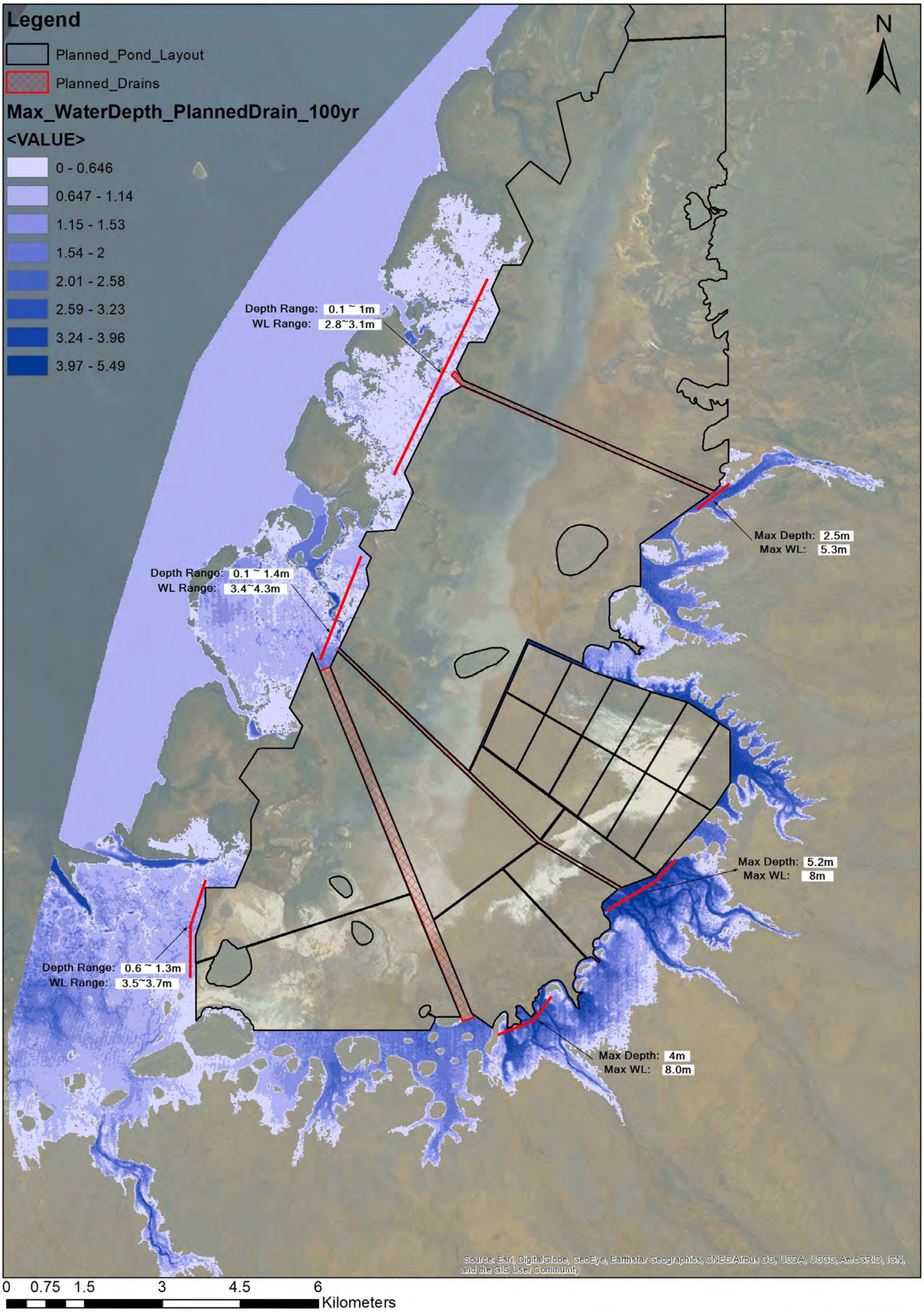

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MAX. WATER DEPTH – BASE CASE 100 YEAR ARI FLOOD FIGURE 5

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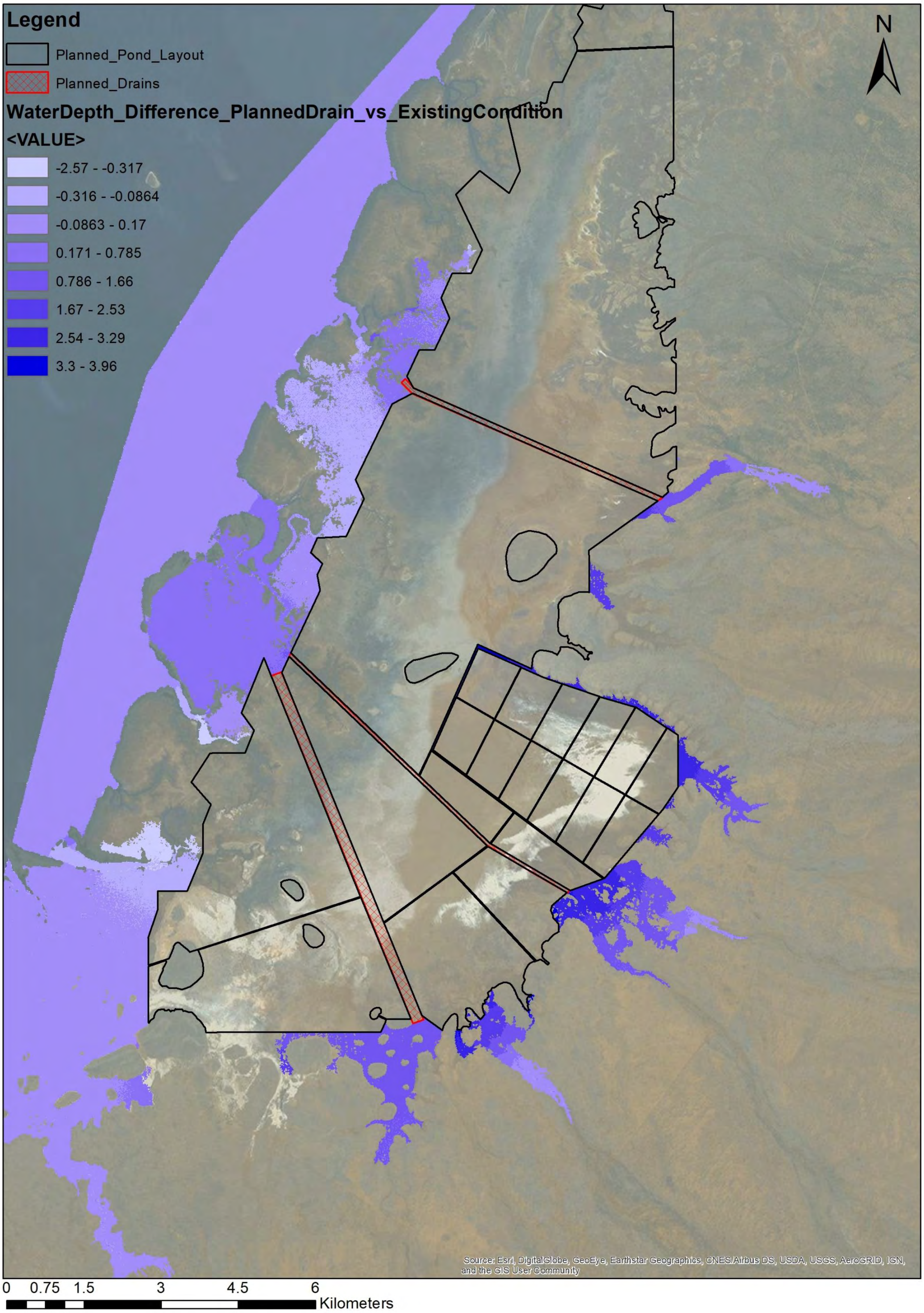

RPS

MAX. DEPTH DIFFERENCE: 100 YEAR ARI FLOOD: BASE CASE VS EXISTING CONDITIONS FIGURE 6

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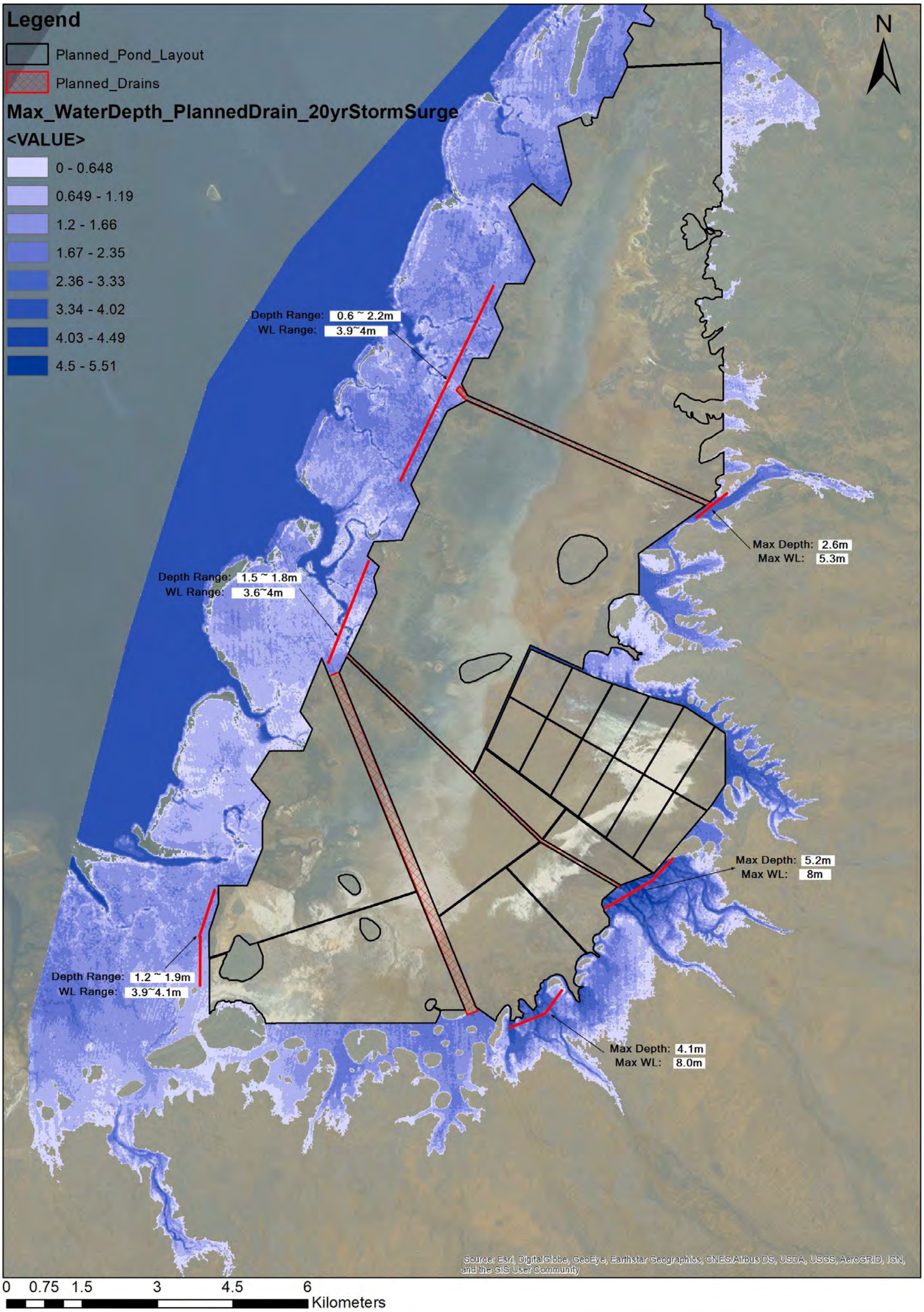
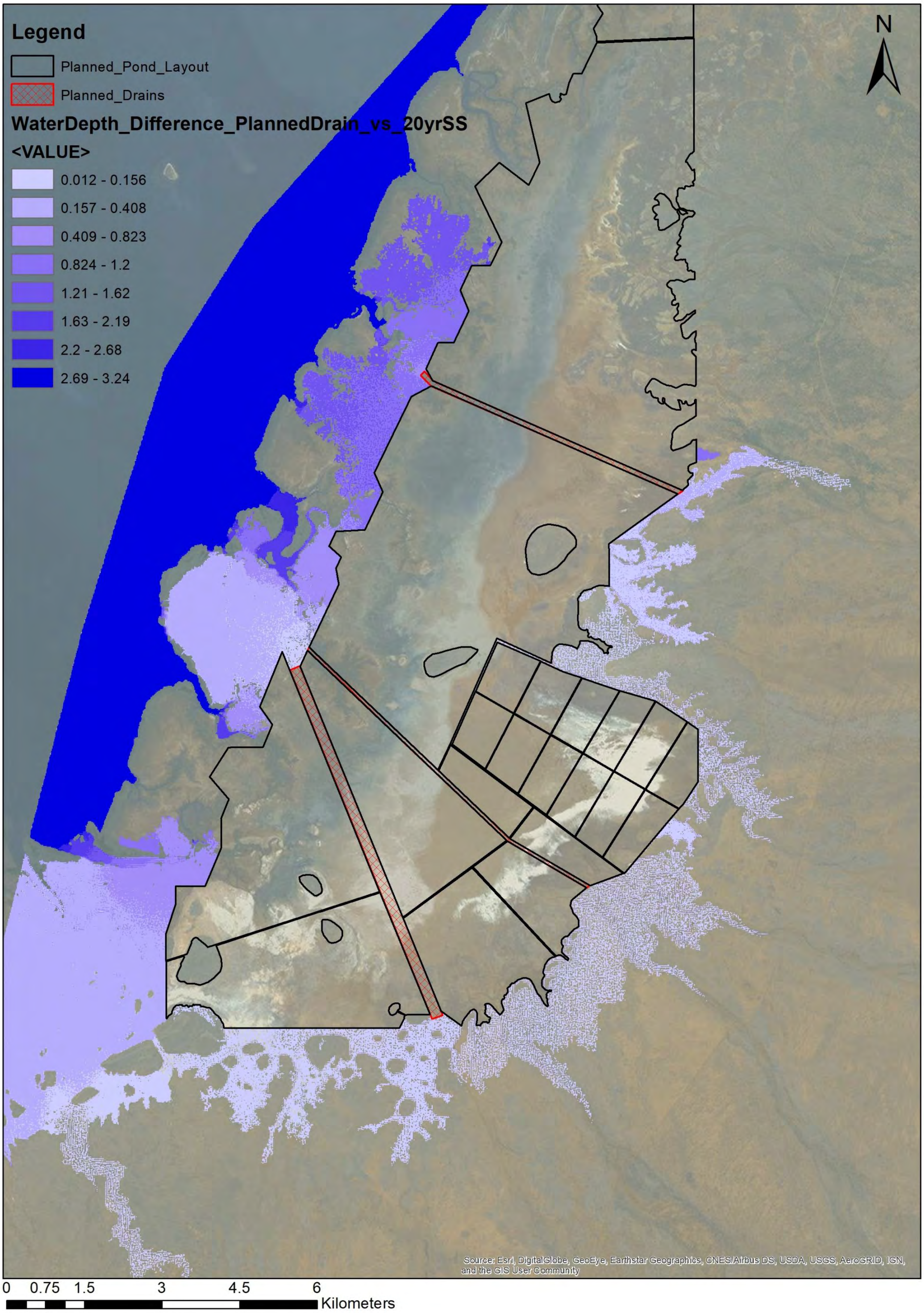


MAX. WATER DEPTH – BASE CASE 100 YEAR ARI FLOOD WITH 20 YEAR ARI STORM SURGE FIGURE 7



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MAX. DEPTH DIFFERENCE: BASE CASE 100 YEAR ARI FLOOD: WITH VS WITHOUT STORM SURGE FIGURE 8

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ALTERNATIVE DRAIN DESIGN SCENARIOS FIGURE 9

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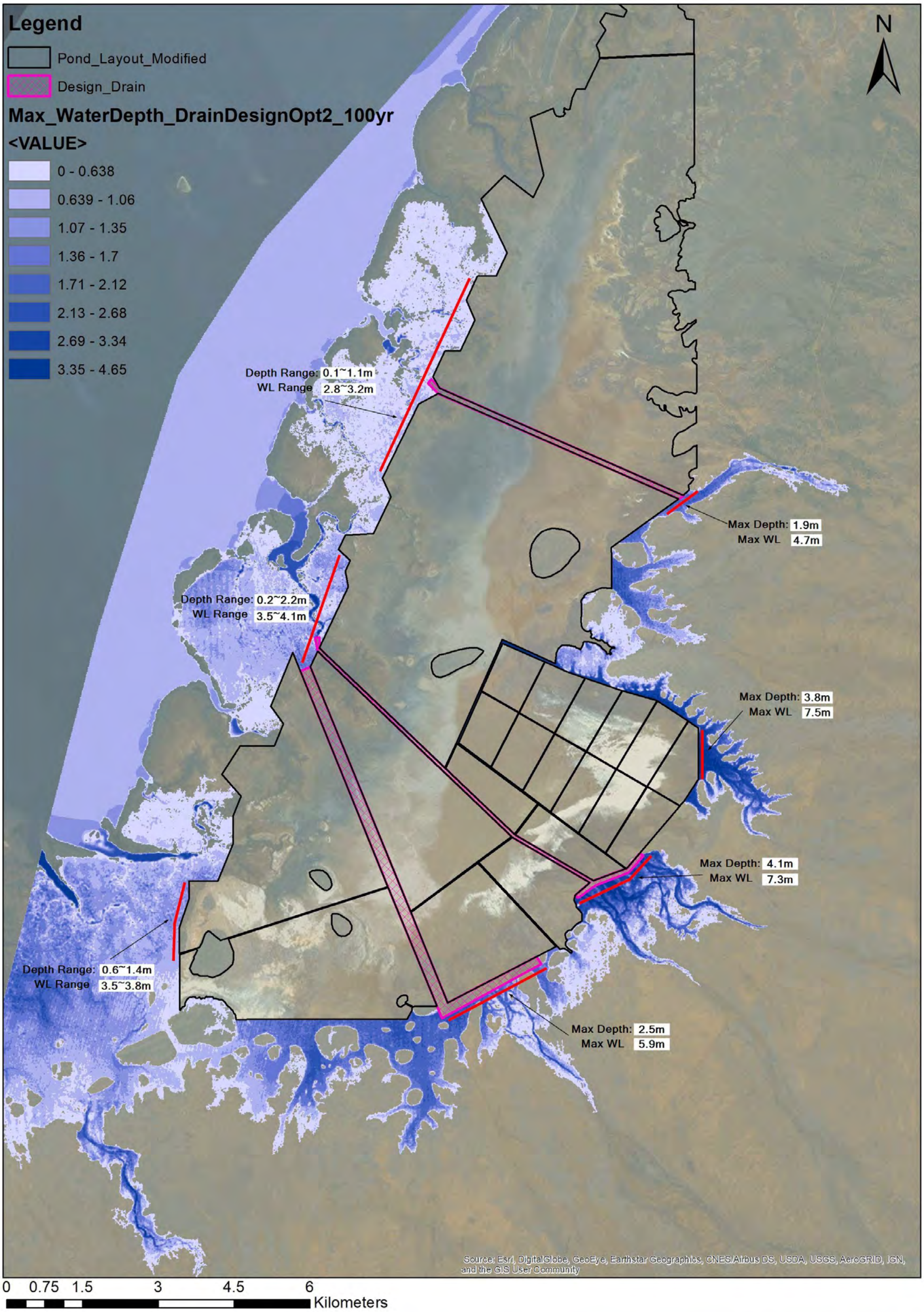

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MAX. DEPTH: 100 YEAR ARI FLOOD: DRAIN DESIGN OPTION 2 FIGURE 10

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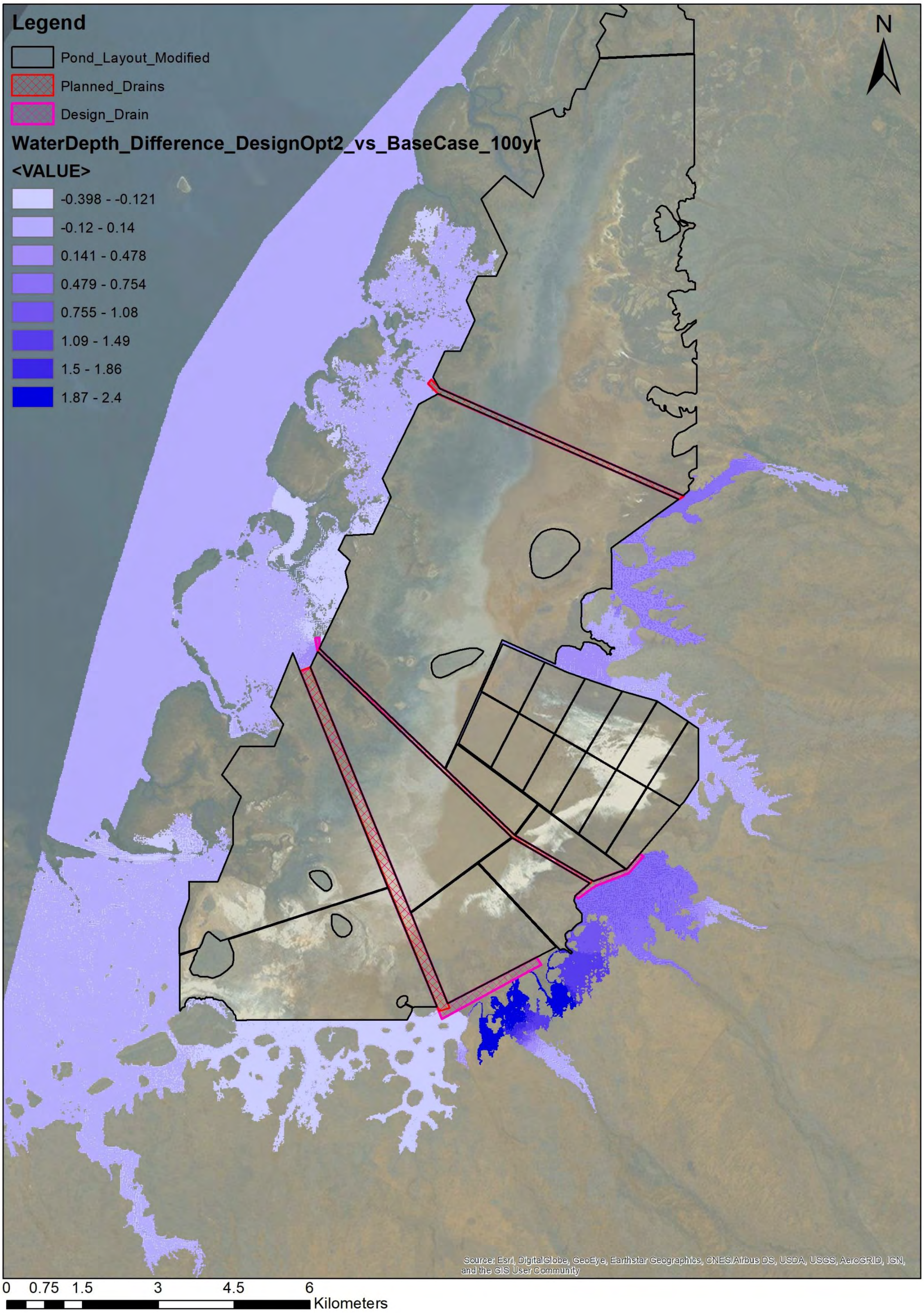

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MAX. DEPTH DIFFERENCE: 100 YEAR ARI FLOOD: BASE CASE VERSUS DESIGN OPTION 2 FIGURE 11

EWP72667

Appendix C

Mardie Salt Project – Pre-feasibility hydraulic assessment: Preliminary results (RPS 2017c)

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Date:	10 November 2017	**Direct Dial:**	08 9211 3510

Attn: Les Purves
Senior Environmental Advisor
BC Iron Limited
Level 1, 15 Rheola Street, West Perth WA 6005

Dear Les

RE: Mardie Salt Project - Pre-Feasibility Hydraulic Assessment: Preliminary Results

1.1 Background

RPS previously completed a hydrological assessment for the Mardie Salt site to quantify design criteria for the sea levels and creek catchments impacting the proposed salt ponds. The outcomes of that assessment were provided to BC Iron Limited in a letter dated 12 October 2017. Correspondence dated 27 October 2017 provided details of preliminary hydraulic modelling undertaken to estimate flood levels for the site.

Further hydraulic modelling has now been undertaken to characterise the interaction of flood flows with the proposed salt ponds, and to provide an understanding of how potential drainage infrastructure may influence design flood levels (and pond bund design levels).

1.2 Hydraulic Assessment Methodology

The modelling methodology used for this assessment was as generally described previously (RPS, 27 October 2017). The previous modelling for the existing conditions and preliminary assessment of the salt pond layout was based on a finer (10-15m) grid cell size. The modelling described herein was undertaken with a coarser (25m) grid cell size (to increase model speed), following testing to confirm that the model results are relatively insensitive to the grid cell size (refinement of the model resolution may be appropriate at the detailed design stage).

1.3 Lateral Diversion Drains

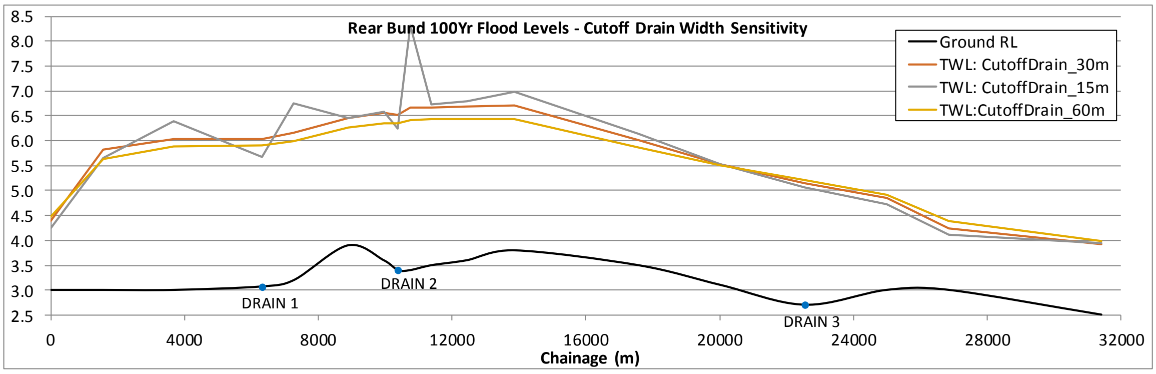
The salt ponds lie on very flat coastal terrain, but the eastern side of the ponds in the preliminary layout provided by BC Iron lies close to elevated terrain. As previously, this creates isolated catchments which are unable to drain due to no or limited lateral connectivity to the proposed drainage corridors through the ponds from rear to front. This would cause significant localised depths of floodwater against the rear of the salt ponds, thereby increasing the required height of the bund.

Some form of lateral diversion drainage is typically required, and the hydraulic model has been modified to simulate a lateral diversion drain along the entire length of the rear bund. This diversion drains all the minor catchments and depressed areas which would otherwise be dammed against the rear of the salt ponds.

The diversion drain was modelled with a drain invert graded between the low points in the natural surface along the rear bund alignment (i.e. the base of the drain coincides with natural surface level at the low-lying areas (such as creek beds) and is cut into the terrain in the more elevated areas). The depth of the drain is therefore highly variable, depending on the surrounding ground levels.

The lateral drain was modelled with uniform base widths of 15m, 30m and 60m to test the sensitivity of flood levels to drain width. The results of this sensitivity analysis are presented in Graph 1, which depicts a long-section of the rear of the salt ponds (with left to right representing south to north).

The results show that the 15m wide drain results in significantly steeper hydraulic gradients and higher flood levels, localised where large creeks impact the ponds, and there is insufficient drain width and lateral flow capacity along the rear of the ponds towards the through drains. The 30m and 60m wide drain widths have a smoother hydraulic gradient and lower flood levels, indicating improved lateral drainage capacity. Therefore, a nominal 30-50m wide lateral diversion drains are indicated along the rear of the salt ponds (possibly smaller in areas where lateral drainage is less important, to be resolved at detailed design).



Graph 1: Sensitivity of Flood Levels to Lateral Diversion Drain Width

The depth of floodwater behind the ponds is estimated as about 3m in the 100 year ARI event, with a maximum water level of ~RL6.5m for the 30m and 60m wide lateral drain scenarios. This demonstrates a significant improvement compared to the previous modelling in the absence of lateral diversion drains, in which flood levels up to RL8.0m were estimated.

In this regard, it is noted that if the rear side of the salt ponds is retreated further towards the coast, leaving more flatter ground between the ponds and the elevated terrain, then the required lateral drainage may largely be accommodated in this flatter area (although some drain construction would still be required through high points and ensure effective drainage).

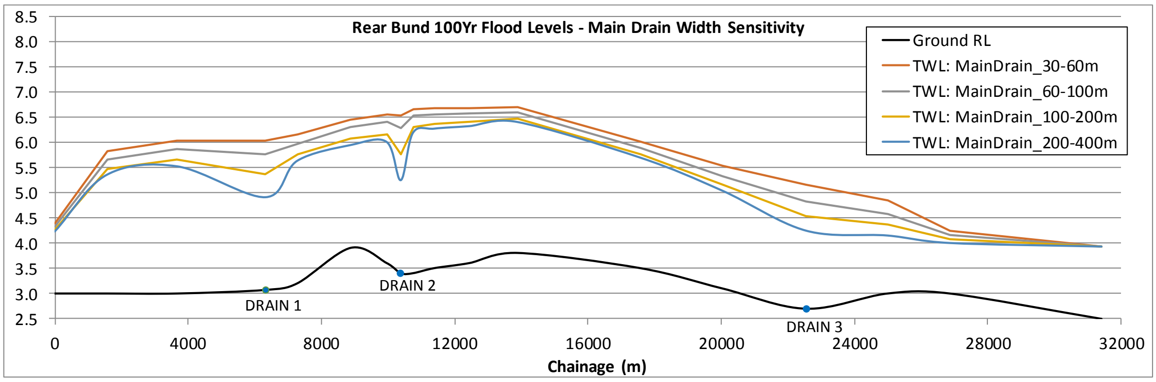
1.4 Drainage Corridors

Drainage corridors are required to carry floods from the rear of the ponds though to the front of the ponds and the sea. Various corridor drain widths were simulated to ascertain the impact on flood levels.

As described previously (RPS, 27 October 2017), nominal drain widths of 30m, 60m and 30m were initially modelled for Drains 1, 2 and 3, respectively, proportionate to the magnitude of flood flows impacting each drain location.

The sensitivity modelling for drain width was undertaken by doubling these drain widths in subsequent scenarios up to 200m, 400m and 200m (Drains 1, 2 and 3). The results are presented in Graph 2 which depicts a long section at the rear bund of the ponds (with left to right representing south to north). The flood level at the upstream end of the drainage corridors is reasonably sensitive to the drain width, with the flood level reducing by approximately 0.25-0.5m each time the drain width is doubled. However, the effect is localised to the drainage corridor and a small adjacent section of the rear bund. The drain width has relatively minor influence on rear flood levels between the drains.

This suggests that the benefit of increasing drainage corridor width is constrained by the lateral drainage capacity at the rear of the salt ponds, and to significantly lower flood levels would require significantly wider lateral drains; or alternatively retreating the salt ponds further seaward, away from the elevated terrain, to enhance the effective lateral drainage behind the salt ponds.



Graph 2: **Sensitivity of Flood Levels to Main Drain Width**

1.5 Flood Protection Criteria

1.5.1 Required Level of Protection

A design ARI (Average Recurrence Interval) and associated flood level is required upon which to assess the potential for flooding of the project area. The appropriate ARI needs to be selected upon which to base flooding assessments and design decisions. A 20% chance of exceedance during the project life is a common design assumption, and for a long project life (>20 years), a 100 year design criterion is suggested. However if the risk of damage is low should overtopping occur, then a <100 year flood level criterion may be more appropriate. A suitable freeboard is added to flood levels to design the bund heights to cater for the various associated uncertainties as describe previously (RPS, 12 October 2017).

Table 1 below describes the probability of exceedance for various ARIs and project life.

Table 1: **Probability of Exceedance versus Project Life**

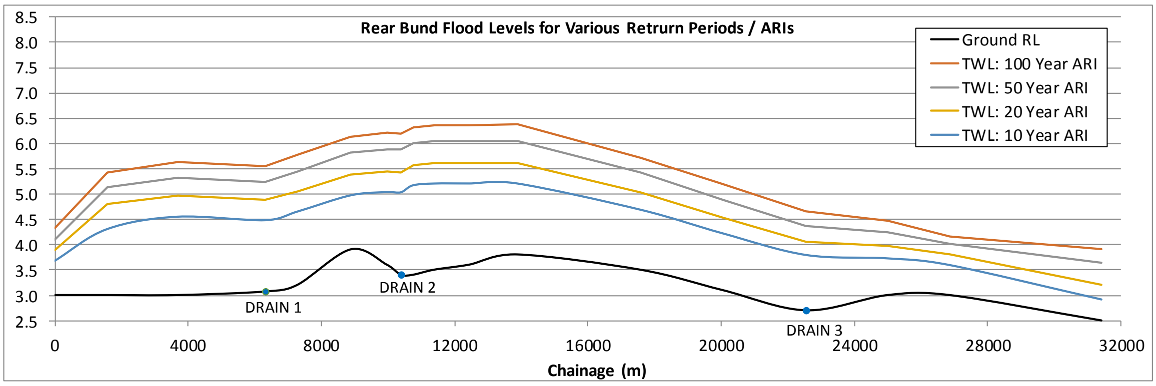
Project Life (Years)	Average Recurrence Interval				
-	10 Year ARI	20 Year ARI	50 Year ARI	100 Year ARI	500 Year ARI
10	65%	40%	18%	10%	2%
20	88%	65%	33%	18%	4%
50	99%	92%	64%	40%	10%
100	100%	99%	87%	63%	18%

1.5.2 Modelled Flood Levels – 10 to 100 Years ARI

Sensitivity analysis (comparison between design criteria) was then carried out as follows:

- Lateral diversion drains fixed at a nominal 50m width;
- Drainage corridors fixed at a nominal 100m width;
- Flood levels modelled for the 10, 20, 50 and 100 year ARI events;
- Downstream boundary water level based on a sea level ARI of 20% the corresponding fluvial flooding event (e.g.. 100 year fluvial flooding with 20 year sea level).

Graph 3 below presents the estimated flood levels along the rear bund of the salt ponds.



Graph 3: Rear Bund Flood Levels for Various Return Periods (ARIs)

The difference in flood levels is fairly uniform along the bund alignment, and are shown in Table 2, as follows.

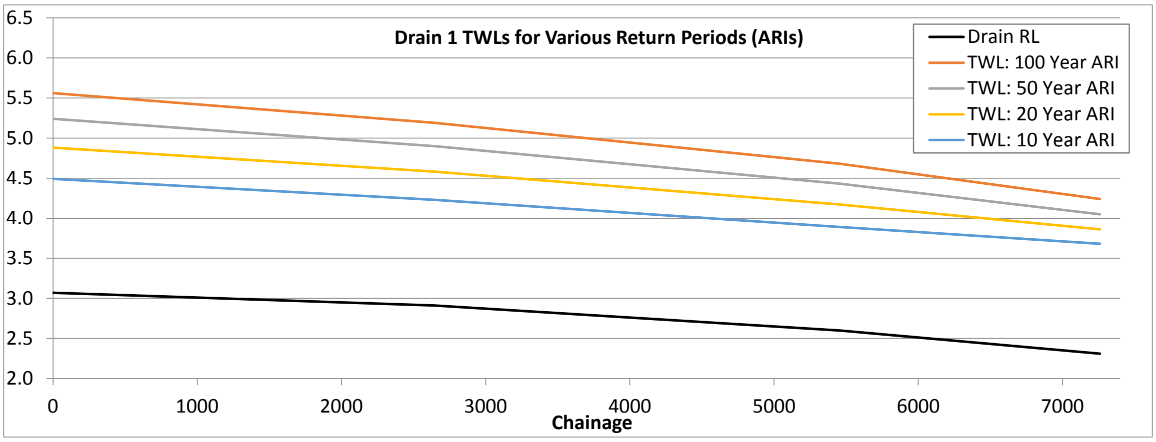
Table 2: Design Criterion (ARI) versus Flood Level (Rear of Ponds)

ARI (years)	Flood Level Difference (m)
100	0.00 (as base case)
50	-0.30
20	-0.70
10	-1.05

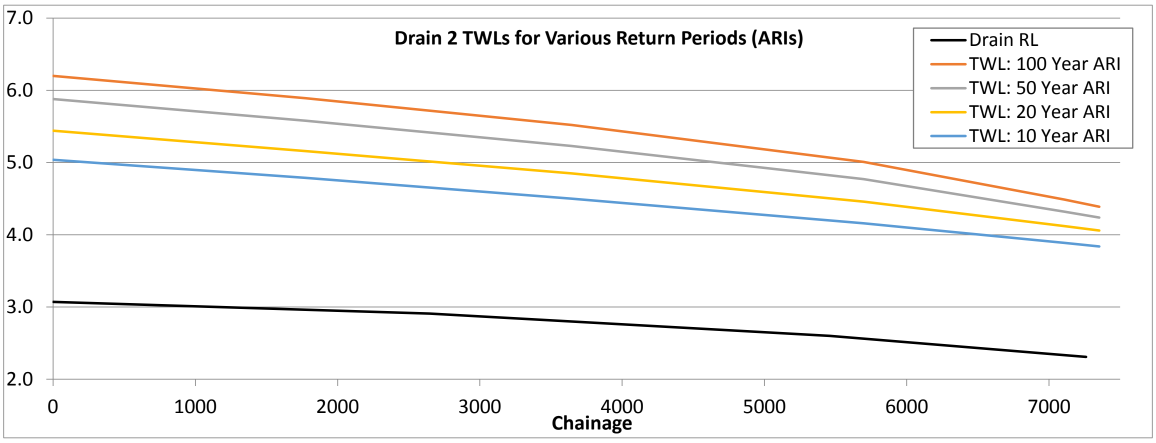
The 5 year flood would be about 1.3-1.4m lower, and the 500 year flood about 0.75m higher, than the 100 year flood levels.

Graphs 4 to 6 provide long-sections of drain inverts and flood levels for each of the drain corridors.

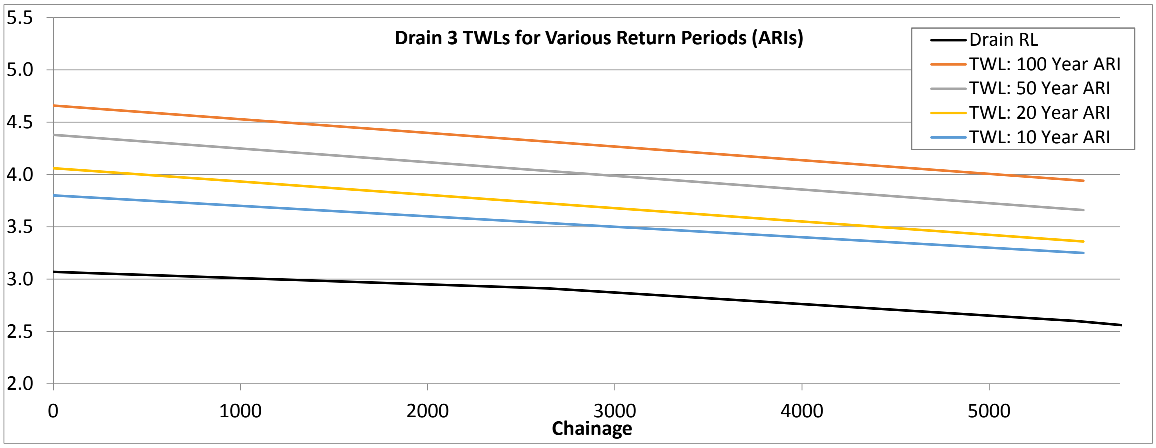
The flood level at the downstream end of the drains is slightly higher than the design sea level that was applied to the model as a boundary condition. For example, the 100 year downstream boundary water level is RL4.2m compared with the sea level RL3.9m. However this effect is very local and can be removed by expanding the drainage corridor width near the end. The flood level at the sea side of the ponds can be adopted as the design sea level.



Graph 4: Drain 1 Flood Levels for Various Return Periods (ARIs)



Graph 5: Drain 2 Flood Levels for Various Return Periods (ARIs)



Graph 6: Drain 3 Flood Levels for Various Return Periods (ARIs)

1.5.3 Design Flood Levels

Table 2 summarises the design flood level estimates at the front and rear of the salt ponds, for the nominal lateral and corridor drain widths. The design flood levels at the front of the salt ponds are dominated by sea levels (tide, storm surge, wave setup, etc). The design flood levels at the rear of the salt ponds are dominated by surface flood levels.

Table 2: Design Flood Level (RLm) Estimates

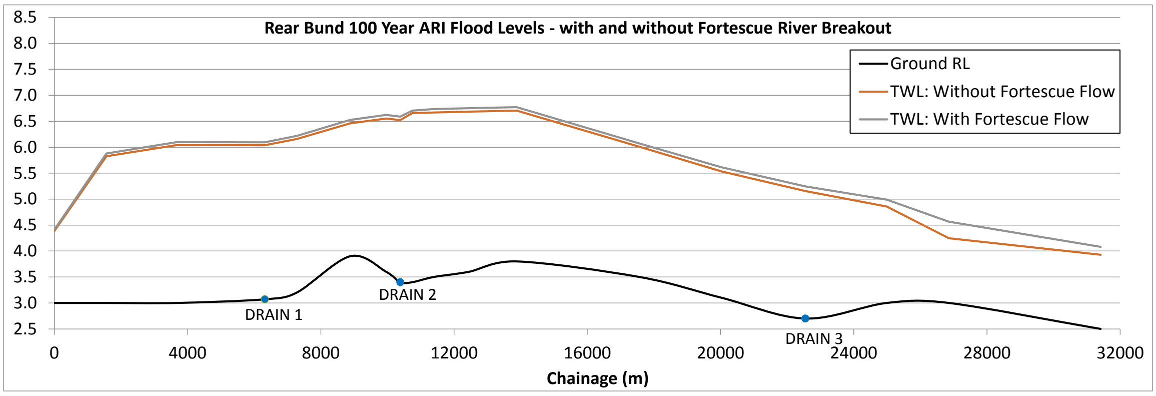
Average Recurrence Interval	Flood (Sea) Level (Front of Ponds, RLm)	Flood Level (Rear of Ponds, RLm)
10 Years	3.6	3.6 – 5.2
20 Years	3.9	3.9 – 5.6
50 Years	4.1	4.1 – 6.0
100 Years	4.3	4.3 – 6.4

1.6 Flood Impact from Fortescue River

As discussed in the hydrological assessment (RPS, 12 October 2017), breakout flow from the Fortescue River into the project area is likely to occur during major (e.g. 100 year ARI) flood events. The river breakout occurs downstream of the North West Coastal Highway, where the topography is flatter.

The direct impact of breakout flow on the project is likely to be relatively low with flows spread over a very large area. A scenario was modelled which included breakout flow, to assess the potential impact behind the salt ponds. Based on the information summarised in the hydrological assessment report, a breakout flow of 1,000m^3/s was simulated as impacting the project area (the majority of the breakout flow is likely to occur further north of the project area).

The results of this simulation are presented in Graph 7 which compares the 100 year ARI flood levels along the rear bund, with and without breakout flow from the Fortescue River included. The difference in flood levels is minor (<0.1m) for all locations except at the very northern end of the site, where flood levels increased by up to 0.3m. The results indicate that breakout flow from the Fortescue River is not likely to play a major role in the flood mitigation design for the project.



Graph 7: Effect of Fortescue River Breakout on Flood Levels at the Site

1.7 Final Remarks

The information provided herein characterises how flood flows are likely to interact with the proposed salt pond infrastructure and provides a high level assessment of how the drainage design (e.g. drain widths) may impact the post-development flood regime, and therefore the design flood levels for a range of design criterion.

Preliminary design flood levels are provided for the front of the ponds (sea levels) and the rear of the ponds (surface flooding) for assumed conditions - 50m wide lateral diversions and 100m wide drain corridors.

The rear of the ponds flood levels will rise / fall with narrower / wider lateral drains respectively. The variations in flood level are relatively minor except where a narrow drain and confining elevated surrounding terrain in combination significantly constrains lateral flow capacity.

The rear of the ponds flood levels will rise / fall with narrower / wider corridor drains respectively (again relatively minor variations).

Following further deliberation on the salt pond and drainage layout by BC Iron, the hydraulic model can be used to further assess or refine the flood level estimates. The model can also be used to assess how the drainage design may impact sensitive coastal habitats (mangroves, mudflats etc.) via changes to aspects of the hydrology such as flow velocities, and extent of inundation.

Yours sincerely
RPS

Dan

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Supervising Scientist

Rhod

Rhod Wright
Principal Civil / Water Resources Engineer

Appendix D

Mardie Salt – Pre-feasibility surface water assessment (RPS 2018)

REPORT

Mardie Salt

Pre-Feasibility Surface Water Assessment

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Approved:	
No.:	EWP72667
Version:	0
Date:	21/05/2018

Document Status

Version	Purpose of Document	Approved by	Reviewed by	Review Date
0	Issued for Review	Dan Williams	Rhod Wright	21/05/2018

Approval for issue

Name	Signature	Date

Contents

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Figures

(contained within report text)

(compiled at rear of report)

Graphs

1 Introduction

1.1 Background

BCI Minerals' proposed Mardie Salt Project is located between the Robe River and Fortescue River mouths in the north-west of Western Australia (refer Figure 1). The project involves the production of 3.0-3.5 million tonnes per annum of sodium chloride salt from a seawater intake and series of solar evaporation ponds.

A scoping study has been completed to prove the technical and economic feasibility of the project, and now additional pre-feasibility studies are required to further inform the technical and approvals feasibility of the project, including the requirement for a baseline hydrological assessment.

The proposed evaporation ponds are located on mud flats on the landward side of the coastal mangrove areas and stretch over 25km of coastline. Several creeks flow through the area that will be occupied by the evaporation ponds.

Among the potential impacts from the project are changes to the surface water hydrology, and the requirement for flood protection infrastructure (bund walls, diversion drains, scour protection measures, etc). Mangroves, fringing mudflats and algal mats are sensitive habitats, and the project will need to demonstrate that impact to these habitats can be minimised or mitigated through appropriate design.

1.2 Scope of Services

A desktop surface water assessment was undertaken to assess the options and requirements for surface water management for the proposed salt ponds. The objective was to develop the relevant surface water scenarios and provide preliminary information on hydraulic and engineering parameters associated with surface water management infrastructure, as well as potential hydrological impacts to the sensitive downstream environment.

The report addresses the following:

- Characterise and describe the existing surface water environment, including climate, location and size of catchments, existing drainage conditions and flow directions;
- Identify key surface water management issues and hydrological risks associated with the proposed development, particularly potential impacts from local creek lines affecting the proposed salt pond infrastructure;
- Estimation of catchments and associated flood flows at key locations throughout the site;
- A conceptual flood mitigation design to provide the required level of flood protection including conceptual location of bunding and diversion drains, indicative dimensions of diversion infrastructure, scour control considerations etc;
- A preliminary assessment of the potential hydrological impacts from the project on downstream sensitive environments (algal mats, mangroves etc).

1.3 Definitions

100 year ARI flood - the flood having an average recurrence interval (ARI) of 100 years. It has a 1% chance of occurring or being exceeded in any one year, and a 50% chance of being experienced at least once in any average life span of a person.

The 10 year ARI flood has a 10% chance of being exceeded in any one year, the 2 year ARI flood has a 50% chance of being exceeded in any one year.

Floodplain - The portion of a river valley adjacent to the river channel which is covered with water when the river overflows its banks during floods.

2 Hydrology

2.1 Seasonal Rainfall and Evaporation

WA has three broad climate divisions - the south-west corner of WA with a Mediterranean climate, with long hot summers and wet winters; the central eastern areas of WA with arid land or desert climates and the area of interest, the dry tropical northern part of the State, receiving summer rainfall.

The average annual rainfall at nearby Mardie Station is 277mm (BOM, Site number 5008) as measured over a 129 year period (1885-2017), but annual rainfall is highly variable with a minimum of 9mm recorded, and a maximum of 886mm.

The majority of rainfall occurs January-June (38-63mm average monthly rainfall), and July-December is typically drier (average monthly rainfall 1-9mm).

There is limited evaporation data available, but the annual Class A pan evaporation at Mardie, as estimated by BCI Minerals, is about 3250mm pa, varying from 12mm/d in summer to 5mm/d in winter.

2.2 Intensity-Frequency-Duration (IFD)

Intensity-Frequency-Duration (IFD) data is required to characterise the storm intensity in the area under consideration. This is generally provided by techniques in ARR (Australian Rainfall and Runoff), a national guideline for the estimation of design flood characteristics in Australia, published by the Institution of Engineers Australia. New IFD design rainfalls were produced in 2016.

Typical IFD data for this area is as follows:

Table 1: IFD Data (rainfall depth in mm)

ARI	1 year	5 year	10 year	20 year	50 year	100 year
1 hour	23	41	50	59	73	83
2 hour	29	51	64	77	95	109
6 hour	39	75	95	117	149	174
12 hour	47	95	124	155	198	233
24 hour	57	119	156	196	251	296
72 hour	71	148	192	238	301	354

Information on storms exceeding the 100 year ARI event is not (readily) available in ARR, but by extrapolation, estimates can be made. The 1000 year ARI and Probable Maximum Precipitation (PMP) rainfalls are in the order of 1.7x and 3.3x the 100 year rainfalls respectively.

The (sliding, not calendar day) 24 hour rainfalls are estimated as:

2 year -	106mm
5 year -	120mm
10 year -	160mm
20 year -	200mm
50 year -	250mm
100 year -	300mm
1000 year -	500mm
PMP -	1,000mm

2.3 Flood Flow Estimation

2.3.1 Regional Context

The catchment details for the development area are shown in Figure 2. The general project area is located on terrain gently sloping from the North West Coastal Highway to the north west at a low 0.15-0.20% gradient, whilst the proposed salt ponds are located on very flat terrain associated with the tidal mud flats.

Based on local rainfall and runoff trends for the area, the flood flows (as a proportion of the 100 year ARI flood) would typically be:

Table 2 Typical Presumptive Flood Flows as Proportion of the Q100 Flood

ARI (years)	Fraction of Q100 flood
2	0.05
5	0.15
10	0.28
20	0.45
50	0.73
100	1.0
1000	~2.1
PMF	~6.3

2.3.2 Hydrological Modelling

There are no relevant streamflow gauging data / gauged catchments from which flood estimates may be made directly. Flood estimation therefore relies on Australian Rainfall and Runoff (ARR) flood estimation methods for ungauged catchments, or an individual customised rainfall runoff model for each catchment.

In this case the RAFTS nonlinear rainfall / runoff model has been used (as used for the previous Haul Road Corridor Hydrology study). RAFTS uses design rainfall data derived from ARR and customised catchment parameter input for each catchment (including terrain slopes, roughness, local rainfall data and rainfall losses). The catchments for the relevant creeks were divided into sub-catchments, with routing links between. The model calculates flood flows (hydrographs) by simulating rainfall over a catchment with time, removing losses to calculate the rainfall excess runoff, and then routing this runoff through the model reaches. The RAFTS 'pern' or surface roughness factor affects the storage factor and was set at 0.045.

2.3.3 Peak Flow Estimates

The RAFTS hydrological model provided flood flow hydrographs for a range of design rainfall IFDs. For each ARI, the critical rainfall event duration (producing the highest peak flow rate) was identified and used for subsequent hydraulic modelling. The peak flow rate for each ARI is provided in Table 3.

Table 3: Estimated Flood Flows (m^3/s)

ARI (years)	Ac (km^2)	2 yr ARI	5yr ARI	10yr ARI	20yr ARI	50yr ARI	100yr ARI	1000yr ARI	PMF
Peter Ck	422	27	80	149	240	389	533	1,119	3,356
Gerald Ck	153	16	49	91	146	236	324	679	2,038
Trevarton Ck	172	18	55	103	165	268	367	771	2,314
6 Mile Ck	164	19	56	104	167	271	372	780	2,341
Fortescue River	18,360	1,090	2,850	5,000	8,080	13,500	20,000	42,000	126,000

The 100 year ARI local flows may be generally estimated as $Q_{100} = 36 \times Ac^{0.45}$ (where Ac = catchment area in km^2) based on typical RAFTS estimates.

2.3.4 Fortescue River Breakout Flow

Part of the Mardie Salt site is potentially impacted by "breakout" flows from the Fortescue River during major flood events.

Upstream from the North West Coastal Highway, the Fortescue River is generally contained between ridges. However, downstream of the highway, the topography becomes less pronounced and the river flow path less constrained. On the west side of the main river channel, there is a noticeable north-south ridge line at about RL30-40m elevation. The river floodplain at this point is generally 5km wide, with numerous smaller flow channels developed, discharging in the same general direction as the main channel.

However, during large flood events, river flows can "break-out" from the main floodplain. There is a significant "break-out" area between the north end of the ridgeline and Coolangara Hill (a small hill 15km north of the highway, elevation ~RL45m) which encroaches into the main floodplain and redirects high level flood water upstream away from the main river channel system. The floodplain east of the hill then reduces to about 4km wide.

Break out flows generally head north-westerly towards the coast 25km away. Flows eventually exit to the ocean, at anywhere up to 25km west of the Fortescue River mouth.

A significant volume of flow would be diverted away from the main Fortescue channels in the largest floods. The Department of Water (then WRC) previously estimated a 100 year ARI flood flow of 9,220m^3/s, with around 1,200m³/s of that flow (i.e. 13%) following channels north and north-west to the sea, west of the main channels. It is not possible however to estimate the quantum of break out flows without 2D hydraulic modelling over a very large area.

For a now estimated 100 year flow of about 20,000m³/s, the break out flows may be assumed as up to 20%, or 4,000m³/s, "lost" from the Fortescue River system. This high flow is spread over a very large area, and the direct impact at any location (other than in a flow channel) would be anticipated as relatively low. The extent of impact at the coast for the larger Fortescue flows would probably be limited to an area north of 6 Mile Creek.

A scenario was modelled as part of this study which included breakout flow, to assess the potential impact behind the salt ponds. A breakout flow of 1,000m³/s was simulated as impacting the project area (most of the breakout flow is likely to occur further north of the project area). The impact on 100 year ARI flood levels along the rear bund when including the breakout flow was minor (<0.1m) for all locations, except at the very northern end of the site, where flood levels increased by up to 0.3m. The results indicate that breakout flow from the Fortescue River is not likely to play a major role in the flood mitigation design for the project

2.4 Coastal Inundation

2.4.1 Sea Levels

Normal tidal variations cause inundation over the coastal flats. Satellite and time-lapse photography indicates that flood overflow of the tidal creeks starts at around 1.1m – 1.2m above mean sea level (MSL), with the tide level varying over the area.

Mean neap tide levels vary around +/-0.5m from MSL, and spring tide levels vary around +/-1.8m (and up to 2.2m in the far north during king tides). The highest and lowest astronomical tides (HAT to LAT), which are the highest and lowest tidal levels which can be predicted to occur under average meteorological conditions, vary by approximately +/-2.4m from mean sea level.

Under abnormal meteorological conditions, greater variations in the tidal range are possible, and actual still water sea levels are produced by the interaction of astronomical tides, storm surges and wave set-up.

The Pilbara coast cyclone season runs from December to April, peaking in February and March. Potentially the most destructive phenomenon associated with cyclones that make landfall, is storm surge, a raised mound of seawater typically some 50km across, and up to several metres higher than the normal tide. The worst scenario arises when a severe cyclone crosses a coastline with a gently sloping seabed, at or close to high tide.

RPS have recently undertaken a Metocean analysis to provide estimates of still water level for various return periods. The estimated 100 year still water sea level is RL4.2-4.3m, about 2m higher than HAT. The 10 year sea level is 3.5-3.7m, 1.3m higher than the HAT. These sea levels would flood the coastline inland for several km from the mean sea level (RL00m) location.

2.4.2 Flood Level Joint Probability

The evaporation ponds will be impacted both by creek flooding and coastal inundation, and any flood works should account for both.

The flood level in the ocean is an end / downstream condition which is required when hydraulic modelling flood flows in the various creeks - a joint probability situation. Flooding of infrastructure located near the coast can be impacted either by creek flooding from the inland side, or high sea surge levels from the ocean side.

In this regard, it is noted that the largest river floods in the Fortescue River, and ocean storm surges both occur as a result of tropical cyclone activity. Generally, a cyclone related flood in the river would occur sometime after any associated abnormal sea level (the height of which can vary greatly), as the cyclone tracked across the coast and moved inland. Hence significant storm surge and river flooding are not dependent, and do not generally occur simultaneously.

The creeks of interest are much smaller than the Fortescue River, and the smaller catchments near the coast are likely to increase the degree of dependence a little between the two flood mechanisms.

A common way on handling this joint probability between the two flood mechanisms is provided in, for example, the “Flood Risk Management Guide” (NSW Department of Environment, Climate Change & Water 2010/759, August 2010). This approach adopts a probability ratio for the two flood mechanisms of 1:5, i.e. assuming 20 year ARI catchment flooding in conjunction with 100 year sea levels, or 100 year catchment flooding in conjunction with 20 year sea levels. The “Karratha Coastal Vulnerability Study” (JDA, August 2012) studied the joint probability between river flood levels and storm surge in the Karratha area and found no obvious correlation; that study therefore adopted the 100 year catchment flood flow in conjunction with the 20 year sea level (estimated as RL3.9m) as the downstream boundary condition.

3 Surface Water Impacts

3.1 Overview

Regional stream flow in the Pilbara is ephemeral, related to intense rainfall from cyclonic activity or localised thunderstorms. Stream flow decays rapidly once rainfall has ceased, with negligible base flow.

The proposed infrastructure is comprised primarily of salt evaporation ponds which extend along approximately 25km of coastline. The project area is situated at the downstream end of several creek system catchments, at the point of creek discharge to the coastal mudflats. The relevant creek catchments range between 33 and 422 km^2 in size. The salt pond design will need to facilitate drainage of these creeks through, or around, the salt ponds to the ocean.

The terrain on which the salt ponds are proposed to be built is extremely flat, with surface slopes in the order of 0.01% (1 in 10,000).

3.2 Infrastructure Impact on Surface Water

Based on the layout of the proposed salt ponds (Figure 2) in relation to surface water:

- The salt ponds occupy a significant proportion of the coastal mudflats to which the local creek systems discharge;
- The overall footprint of the salt ponds intercepts four named creeks (Peter Ck, Gerald Ck, Trevarton Ck and Six Mile Creek), as well as several smaller creeks.
- The salt ponds will need to provide drainage corridors to convey flows through or around the salt ponds to the ocean;
- The current salt pond layout provides three drainage corridors through the salt ponds, which are generally aligned with the larger creek systems; lateral diversion drains will also be required to intercept the other smaller creeks and isolated catchments to convey these flows to the drainage corridors, or to the north or south of the salt ponds;
- The drainage corridors will have the effect of concentrating flood flows to fewer points of discharge to the coastal mudflats, which will also be in closer proximity to the algal mats, tidal creeks and mangroves that comprise the downstream environment.

3.3 Surface Water Impact on Infrastructure

The local creek systems convey flows in a north-westerly direction towards the rear (landward) boundary of the proposed salt ponds (refer Figure 2).

- The four named creeks have an estimated 100 year flow of 324 - 533m^3/s, and a PMF of about ~2,000 – 3,300 m^3/s;
- The creek channels discharge to the coastal mudflats which are situated just within the proposed salt pond footprint, the salt ponds will need to be protected from freshwater inflow by diversion drains and levees;
- The larger creeks are expected to flow at least 3-4m deep in the 100 year flood, and the smaller creeks at least 2m deep, based on the results of hydraulic modelling.

4 Flood Mitigation Concept

4.1 Hydraulic Assessment Methodology

A 2D hydraulic model was developed to cover the salt ponds and drainage channels, the creek channels and floodplain area immediately upstream of the salt ponds, and the coastal margin downstream of the salt ponds. The 2D grid is based on the 0.25m interval topographic contours provided by BCI Minerals and supplemented (for the southern project area which is not covered by the contours) by the 5m horizontal resolution DEM, also provided by BCI Minerals. The hydraulic model utilised a cell size of 25m for most of the simulations. A smaller 10m cell size was used for the pre- versus-post development simulations (1 and 10 year ARI) to resolve a higher level of detail around the small coastal creeks. The effect of cell size on modelled water levels behind the salt ponds was tested and found to be negligible.

Post-development simulations use 1D “links” within the model to represent the proposed drainage channels through the salt ponds, as well as diversion drains along the rear of the salt ponds. This allows the drainage channels to be modelled with greater accuracy.

Design flood hydrographs from the hydrological assessment were input to the upstream boundary of the 2D model at the location of each major creek – Peter Creek, Gerald Creek, Trevarton Creek, 6 Mile Creek and the three other creek catchments identified in the hydrology assessment which flow into the project area from the southeast. Figure 3 illustrates the model setup.

The downstream boundary condition for the 2D model is a tidal sea water level, with the boundary elevation set at the estimated storm surge sea level for return period ratio of 1:5, e.g. a 20 year ARI storm surge in conjunction with a 100 year ARI fluvial flood.

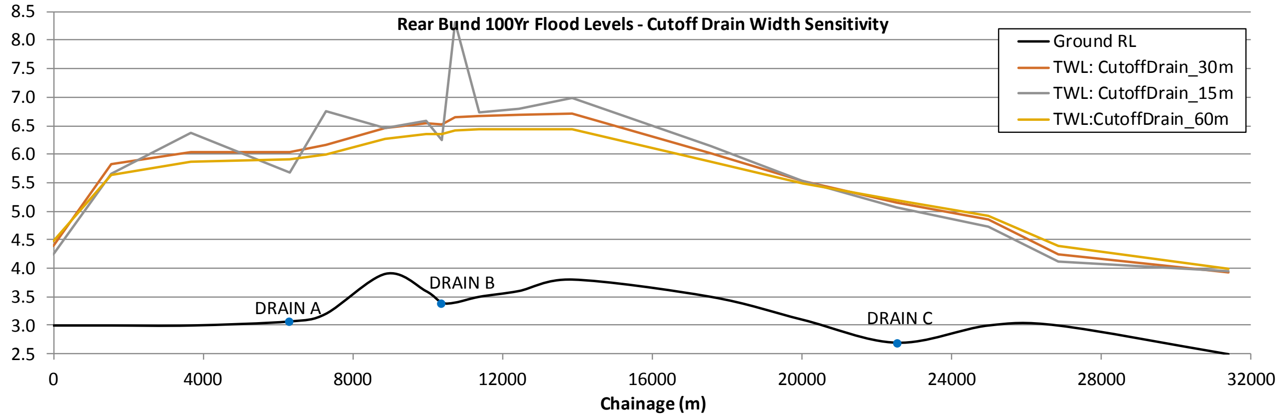
4.2 Lateral Diversion Drains

The salt ponds lie on very flat coastal terrain, but the eastern side of the ponds lies close to elevated terrain. This creates isolated catchments which are unable to drain due to no or limited lateral connectivity to the proposed drainage corridors through the ponds from rear to front. This would cause significant localised floodwater depths against the rear of the salt ponds, thereby increasing the required height of the bund.

On this basis, some form of lateral diversion drainage is typically required to drain all the minor catchments and depressed areas which would otherwise be dammed against the rear of the salt ponds.

The hydraulic model was used to simulate a lateral diversion drain along the entire length of the rear bund in the first instance. The diversion drain was modelled with a drain invert graded between the low points in the natural surface along the rear salt pond alignment (i.e. the base of the drain coincides with natural surface level at the low-lying areas (such as creek beds) and is cut into the terrain in the more elevated areas). The depth of the drain is therefore highly variable, depending on the surrounding ground levels.

The lateral drain was modelled with uniform base widths of 15m, 30m and 60m to test the sensitivity of flood levels to drain width. The results of this sensitivity analysis are presented in Graph 1, which depicts a long-section of the rear of the salt ponds, with left to right representing south to north.



Graph 1: Sensitivity of Flood Levels to Lateral Diversion Drain Width

The results show that the 15m wide drain results in significantly steeper hydraulic gradients and higher flood levels, localised where large creeks impact the ponds, and there is insufficient drain width and lateral flow capacity along the rear of the ponds towards the through drains. The 30m and 60m wide drain widths have a smoother hydraulic gradient and lower flood levels, indicating improved lateral drainage capacity. Therefore, a nominal 30-50m wide lateral diversion drain is indicated along the rear of the salt ponds (possibly smaller in areas where lateral drainage is less important, to be resolved at detailed design).

Subsequent simulations therefore adopted a uniform diversion drain width of 50m. Rather than assume a single continuous diversion drain, individual drains were subsequently modelled as and where required according to the topography and location of dammed catchments. The modelled diversion drain alignments are shown in Figure 3.

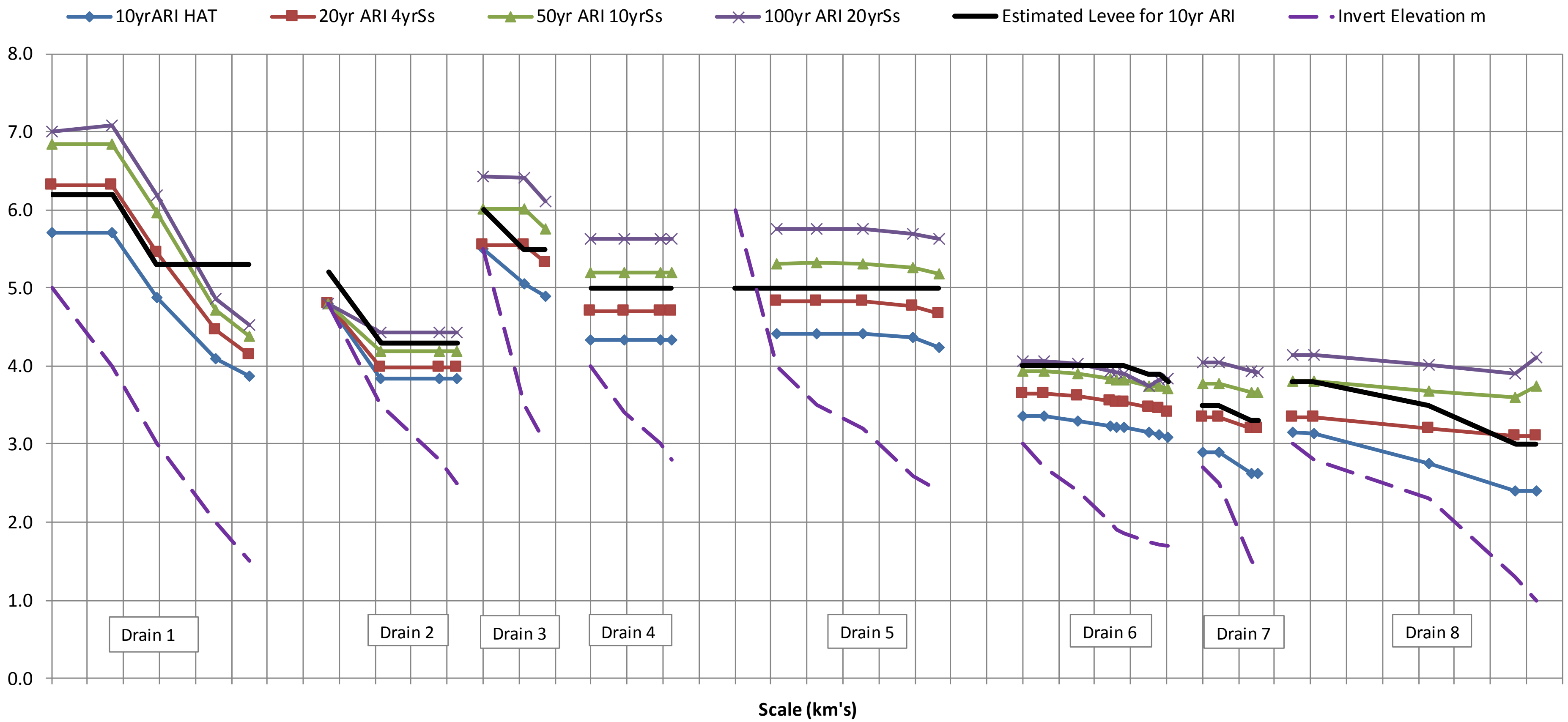
4.3 Levees

The lateral diversion drains will require a levee to contain flood flows, the height of which will be variable according to the surrounding ground levels (i.e. a higher levee will be required where diversion drains intercept lower-lying areas such as creeks).

Levees were initially modelled as infinite height to determine maximum potential water levels for each ARI event. Graph 2 presents long-sections of the levee alignments displaying the modelled TWL along the levees for the 10, 20, 50 and 100 year ARI events.

It is understood from preliminary discussions with BCI Minerals that the desired level of protection required for the salt ponds from freshwater flooding is likely to be around 10 years ARI. Levees have therefore been modelled with a variable height corresponding to the 10 year ARI water level, plus a freeboard of approximately 500-800mm. Graph 2 also shows the levee height that was modelled for subsequent simulations.

RPS

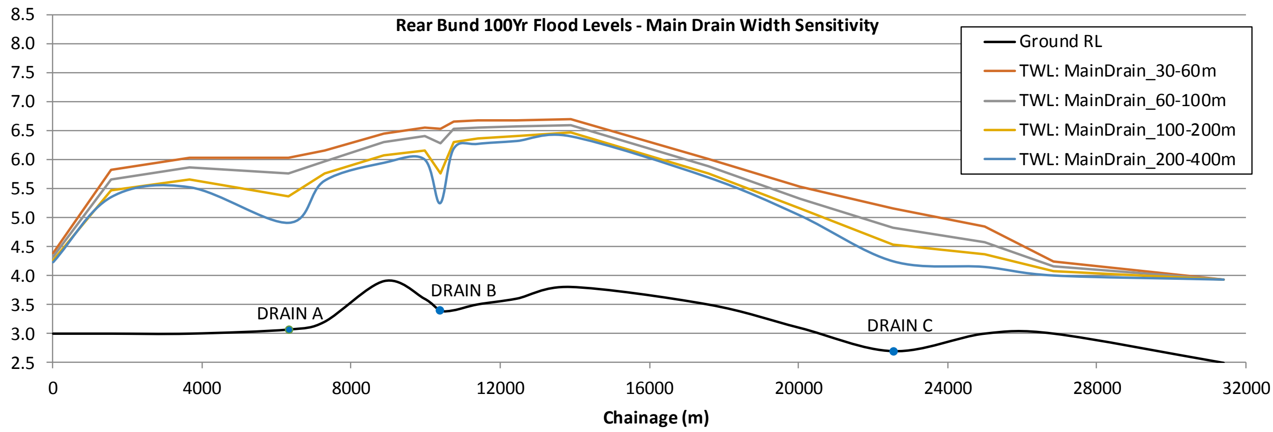


Graph 2: Levee Alignments with Modelled Flood Levels (assuming infinite levee height)

4.4 Drainage Corridors

Drainage corridors are required to carry floods from the rear of the ponds though to the front of the ponds and the sea. Various corridor drain widths were simulated to ascertain the impact on flood levels.

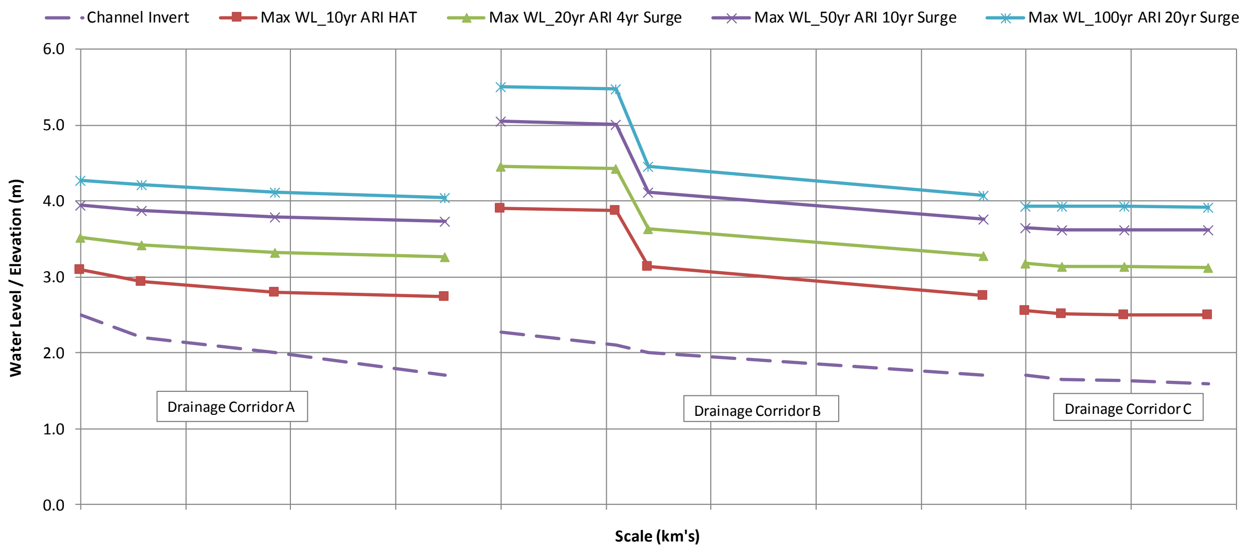
As described previously (RPS, 27 November 2017), a sensitivity analysis for drain width was undertaken by doubling these drain widths in subsequent scenarios from nominal drain widths of 30m, 60m and 30m (for drainage corridors A, B and C, respectively) up to 200m, 400m and 200m. The results are presented in Graph 3 which depicts a long section at the rear bund of the ponds (with left to right representing south to north).



Graph 3: Sensitivity of Flood Levels to Drainage Corridor Width

The flood level at the upstream end of the drainage corridors is reasonably sensitive to the drain width, with the flood level reducing by approximately 0.25-0.5m each time the drain width is doubled. However, the effect is localised to the drainage corridor and a small adjacent section of the rear bund. The drain width has relatively minor influence on rear flood levels between the drains, suggesting that the benefit of increasing drainage corridor width is constrained by the lateral drainage capacity at the rear of the salt ponds.

Following this earlier modelling, BCI Minerals provided a revised pond layout which provided for 300m wide drainage corridors (besides narrow sections of 100m width to facilitate services crossing). The modelled flood levels within the current pond and drainage corridor layout is shown in Graph 4 which presents long-sections of each drainage corridor. A significant steepening of the hydraulic gradeline occurs in Drainage Corridor B associated with the section of narrowed (100m width) drain, indicating that flood flows are constrained at this location and impacting the upstream flood levels. It is likely that further optimisation of this section of drainage corridor during detailed design could facilitate lower flood levels upstream of Drainage Corridor B.



Graph 4: Flood Levels within Drainage Corridors

4.5 Design Flood Levels

Table 4 summarises the design flood level estimates at the front and rear of the salt ponds, for the modelled lateral diversion drain and drainage corridor widths. The design flood levels at the front of the salt ponds are dominated by sea levels (tide, storm surge, wave setup, etc). The design flood levels at the rear of the salt ponds are dominated by the surface flood levels and are highly variable within the different diversion drains (depending on the natural surface levels and the magnitude of flood flows being intercepted).

Table 4: Design Flood Level Estimates (RLm)

Return Period (ARI)	Flood (Sea) Level (Front of Ponds)	Flood Level (Levees Upstream of Ponds)
10	3.6	3.6 – 5.7
20	3.9	3.9 – 6.3
50	4.1	4.1 – 6.9
100	4.3	4.3 – 7.1

4.5.1 Required Level of Protection

A design ARI (Average Recurrence Interval) and associated flood level is required upon which to assess the potential for flooding of the project area. The appropriate ARI needs to be selected upon which to base flooding assessments and design decisions. A 20% chance of exceedance during the project life is a common design assumption, and for a long project life (>20 years), a 100 year design criterion is suggested. However, if the risk of damage is low should overtopping occur, then a <100 year flood level criterion may be more appropriate. A suitable freeboard is added to flood levels to design the bund heights to cater for the various associated uncertainties (as describe previously, RPS, 12 October 2017).

RPS

Table 5 below describes the probability of exceedance for various ARIs and project life.

Table 5: Probability of Exceedance versus Project Life

Project Life (Years)	Average Recurrence Interval				
-	10 Year ARI	20 Year ARI	50 Year ARI	100 Year ARI	500 Year ARI
10	65%	40%	18%	10%	2%
20	88%	65%	33%	18%	4%
50	99%	92%	64%	40%	10%
100	100%	99%	87%	63%	18%

4.6 Engineering Construction

4.6.1 Bund Material

Soil materials may be characterised to ensure suitability, but the performance requirements for temporary water storage are not specific. The embankment would typically use the most suitable available material at the site, e.g. waste material or diversion excavations, and be constructed homogeneously (i.e. not zoned).

Flood bunds are generally watertight for stability reasons and some clay content is required - materials range from clayey gravels and sands (preferred), through to poorly graded sands (least preferred), and preferably no rock particles >75mm.

4.6.2 Erosion Protection

Scour in unprotected soils will typically occur when maximum velocities reach about 1.2-2m/s for clays, up to about 1.5m/s for sand, and higher for rocky material.

Rock armour can be used to protect earthworks against scouring and erosion, and can be applied where problems occur, or in the long term where permissible velocities may be exceeded. Generally, it is not considered necessary to rock armour an operational embankment or channel against velocities <2m/s for the design flood event (subject to operational experience).

Figure A below provides the modelled flow velocities within the drainage corridors. The 10 year ARI peak velocity is 0.5 m/s or lower except through the narrow section of drainage corridor B where it is 1.1 m/s. The 100 year ARI velocity is 0.9 m/s or lower except the narrow section of corridor B where it is ~2 m/s. Further assessment and optimisation of drainage corridor B may be required at detailed design to decrease flow velocities.

It should be noted that the velocities presented below are inclusive of the downstream boundary condition based on storm surge level with a 1:5 probability ratio. Peak velocities should be further investigated at detailed design for a larger range of downstream boundary (sea level) conditions; however it is not anticipated that peak velocities significantly higher than those described below would be predicted given the extremely flat grade.

RPS

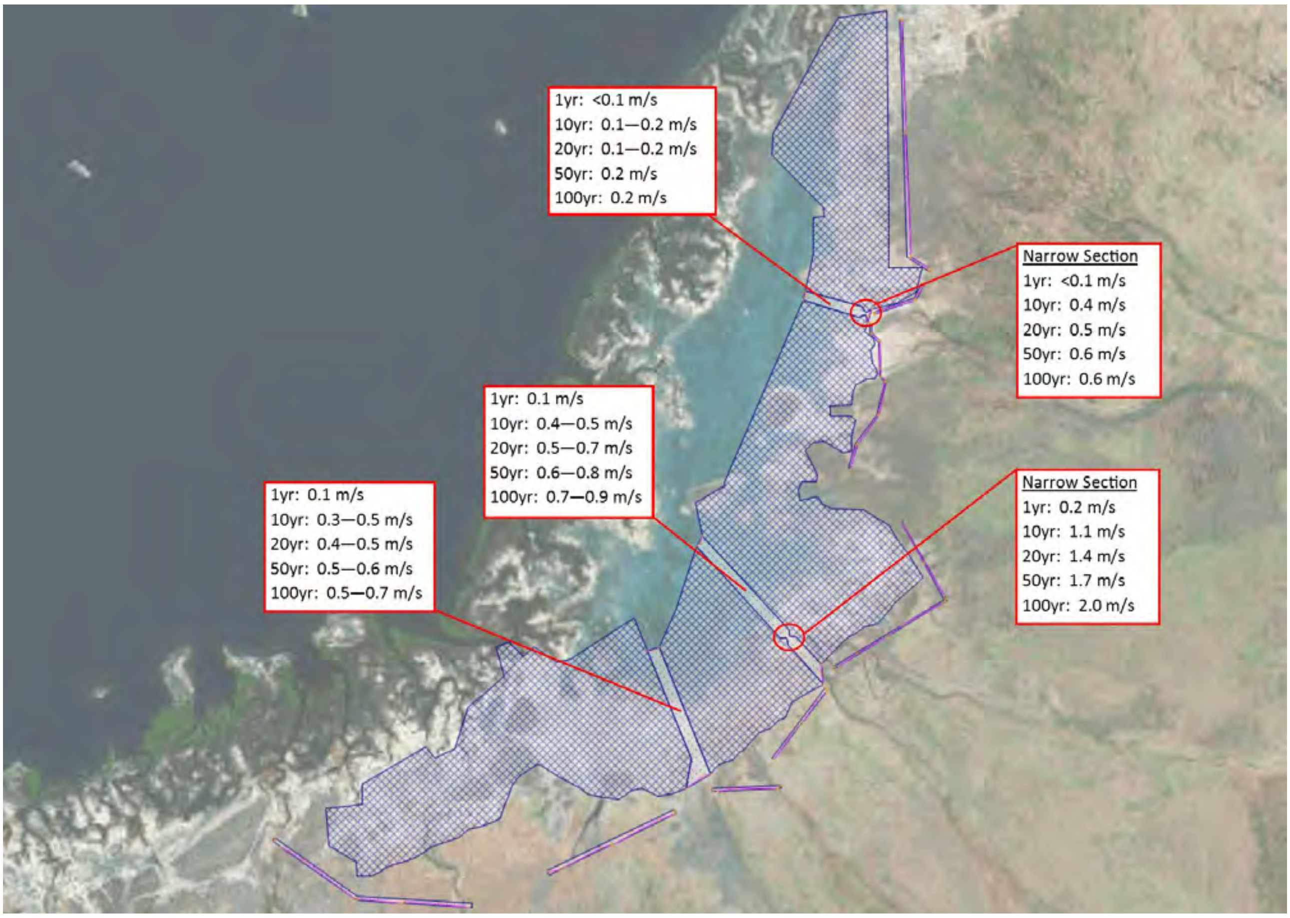


Figure A: Modelled Peak Velocities

4.6.3 Construction

Earthworks (bund) construction requirements typically entail:

- Excavate to strip depth, scarify the base in preparation for construction of an embankment;
- Maintain moisture content in the embankment material at optimum (which allows the maximum density to be achieved by the compaction equipment in use);
- Place and compact material in layers as specified (e.g. 95% SMDD (Standard Maximum Dry Density); or 92% MMDD (Modified Standard Maximum Dry Density);
- Control batter slopes to line and level.

5 Hydrological Impacts to Environment

5.1 Overview

The algal mats and mangroves located in the intertidal zone downstream of the proposed salt ponds have been identified as sensitive ecological systems which will require protection using appropriate design measures to mitigate and minimise potential impacts from the project.

To simulate the hydrological impacts of the salt ponds, the pre-development scenario (existing conditions) was modelled alongside the post-development scenario (with salt ponds) for the 1 and 10 year ARI events. The 1 year ARI was modelled to represent a relatively frequent event which is considered more likely to be significant in terms of playing a role in ecological function (e.g. water and/or nutrient cycling). The 10 year ARI event was modelled to represent a more significant flood event with greater potential to impact the environment through physical means such as scour etc.

The 1 year ARI event was modelled with a 0 RLm downstream boundary condition, i.e. approximately mean sea level and no tidal inundation of the mudflats downgradient of the salt ponds. The 10 year ARI event was modelled with a 2 RLm downstream boundary condition, i.e. a spring high tide with associated inundation of the mudflats).

5.2 Modelled Impacts

The results of the pre-versus-post development comparisons are provided in Figures 4 to 7 at the rear of the report which present difference maps for maximum water depth and velocity. Time series data has also been extracted from the model at the locations shown in Figure B below and discussed in the following sections.

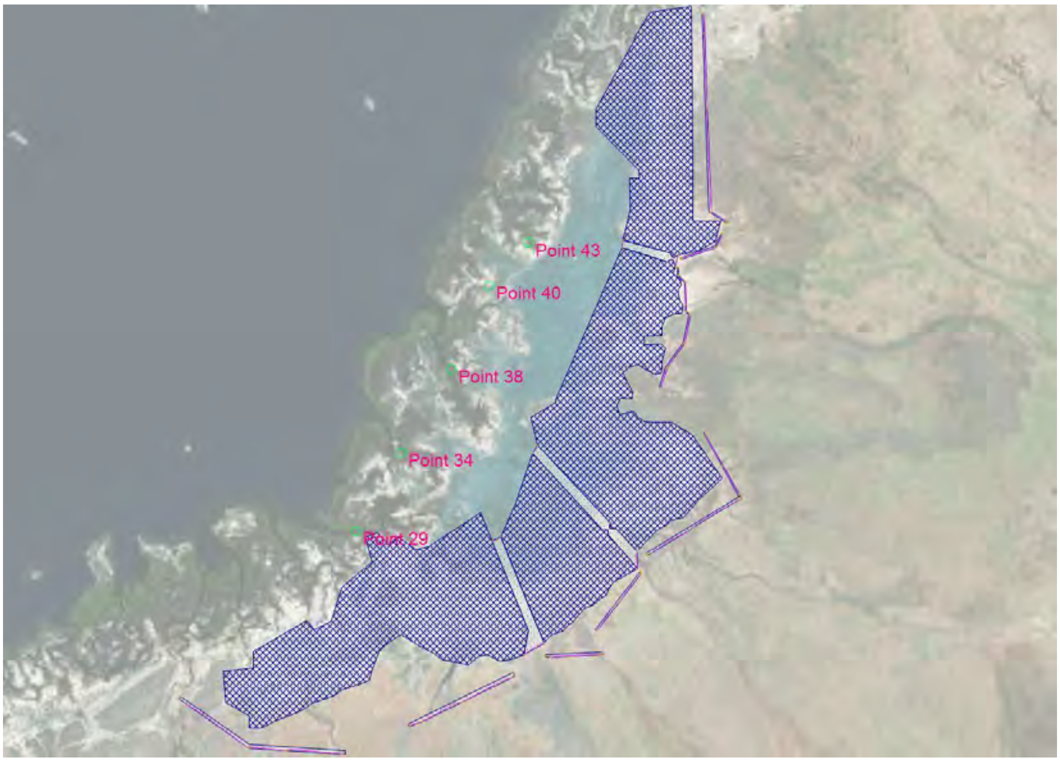


Figure B: Water Level Time Series Locations

RPS

5.2.1 1 Year ARI Scenario

The impact of the salt ponds on maximum water depth for the 1 year ARI event (Figure 4) is variable across the project area, with localised depth increases up to 0.5m. The areas of most significant change are:

- In the creek channels immediately upstream of the levees where maximum depths increase by up to approximately 0.5m.
- At the discharge location of Diversion Drain 9 (to the northern boundary of the salt ponds) where maximum water depths increase by up to approximately 0.5m.
- Immediately downstream of the main drainage corridors where maximum depths increase by up to approximately 0.2m.

The change to water depth is significant where the rear diversion drains and levees intercept creek channels upstream of the salt ponds (which is to be expected). The increased water depth is also significant at the discharge location of the diversion drain at the northern boundary of the salt ponds. However, this is related to the model configuration whereby the discharge from the 1D model channel to the 2D model grid occurs close to the tidal creek channels. Discharge from the diversion drain to the coastal flats would actually occur almost 2km further upstream of this point where the surrounding terrain becomes low and flat; this in turn would result in a reduction in flow depth and velocity as the flows spread across the flats.

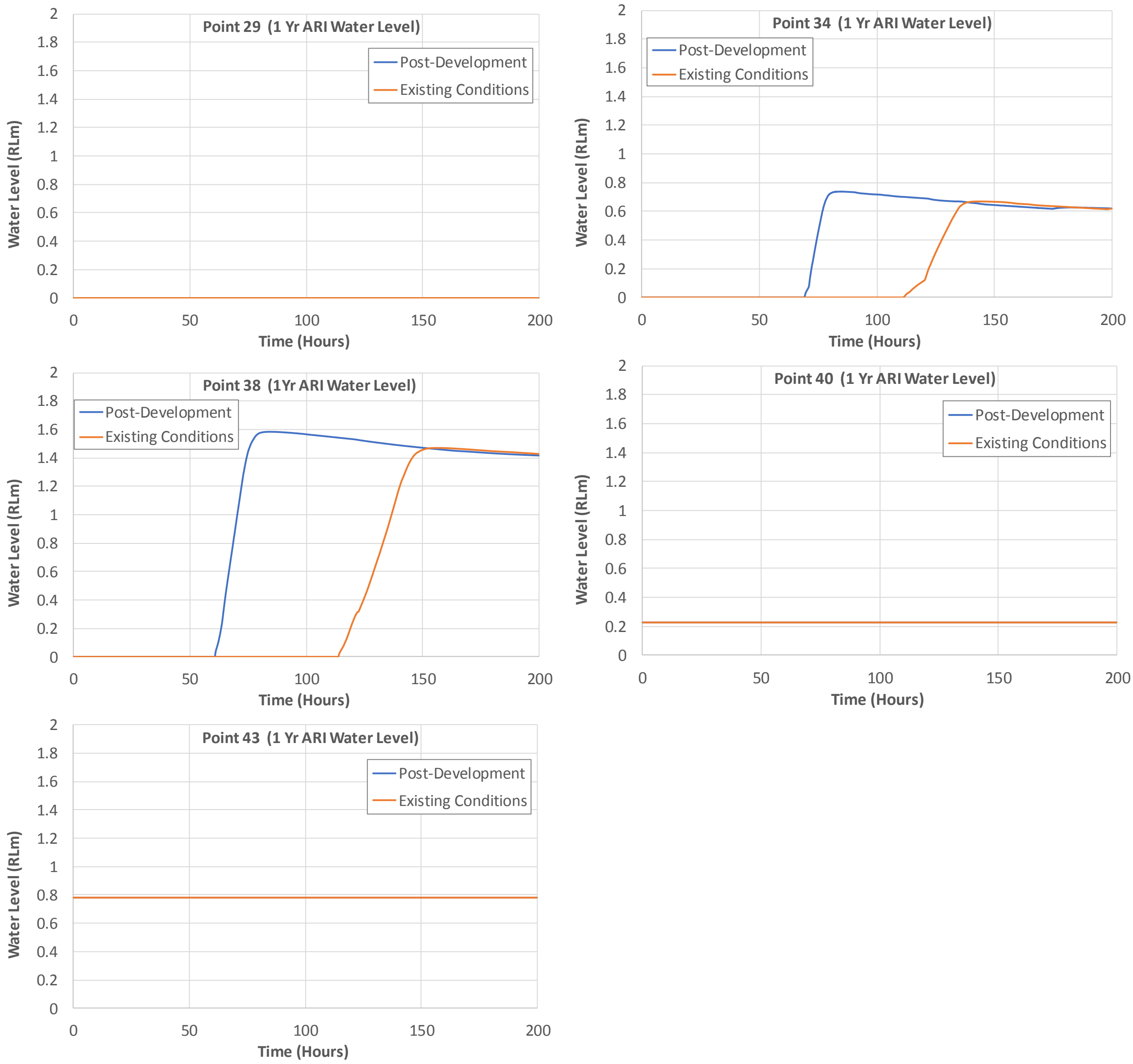
Figure 4 also suggests that the maximum water depth increases significantly (up to approximately 0.5m) at the location of discharge from the diversion drain to the southern boundary of the salt ponds. Whilst an increase in flows and depths at this location is to be expected (due to the diversion of the significant Peter Creek catchment to this location) the modelled results are potentially overestimating the degree of impact. This is because the pre-development simulation did not account for flows within the unnamed creek that discharges at this location. The impact from diverted Peter Creek flows to this location will need to be further defined during the next phase of investigations.

The change to water depth is up to approximately 0.2m at the end of the drainage corridors; however, this is over a very limited areal extent. The change in maximum water depth in general is much less than this for the vast majority of the area downgradient of the ponds.

Graphs 5a-e provide the modelled water level time series for existing and post-development conditions at several locations (which are illustrated in Figure B). The maximum depth within the tidal creeks adjacent to the two main salt pond drainage corridors (Points 34 and 38) increases by approximately 0.05-0.1m in the post-development simulation.

The graphs show that several locations (Points 29, 40 and 43) were not inundated during the modelled 1 year ARI event, either with or without the salt ponds. This suggests that during minor rainfall events, the flows generated by the creeks will likely be insufficient to contribute significant freshwater flow to all the tidal creeks and mangrove areas.

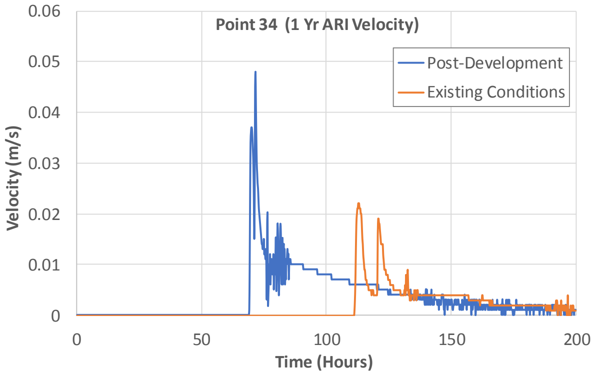
RPS



Graphs 5a-e: 1 Year ARI Water Level Time Series – Tidal Creek Locations

Figure 5 presents the mapped difference in peak velocity between the pre-development and post-development simulations. The most significant change in maximum velocity occurs immediately downstream of the drainage corridors where the concentrated flood flows discharge to the mudflats. Peak velocities increase by up to approximately 0.2m/s at this location but only over a very limited areal extent.

By comparison, the change in velocity further downstream of the two major drainage corridors (at Point 34) is very minor, increasing from 0.02 m/s to 0.05 m/s (Graph 6 below)



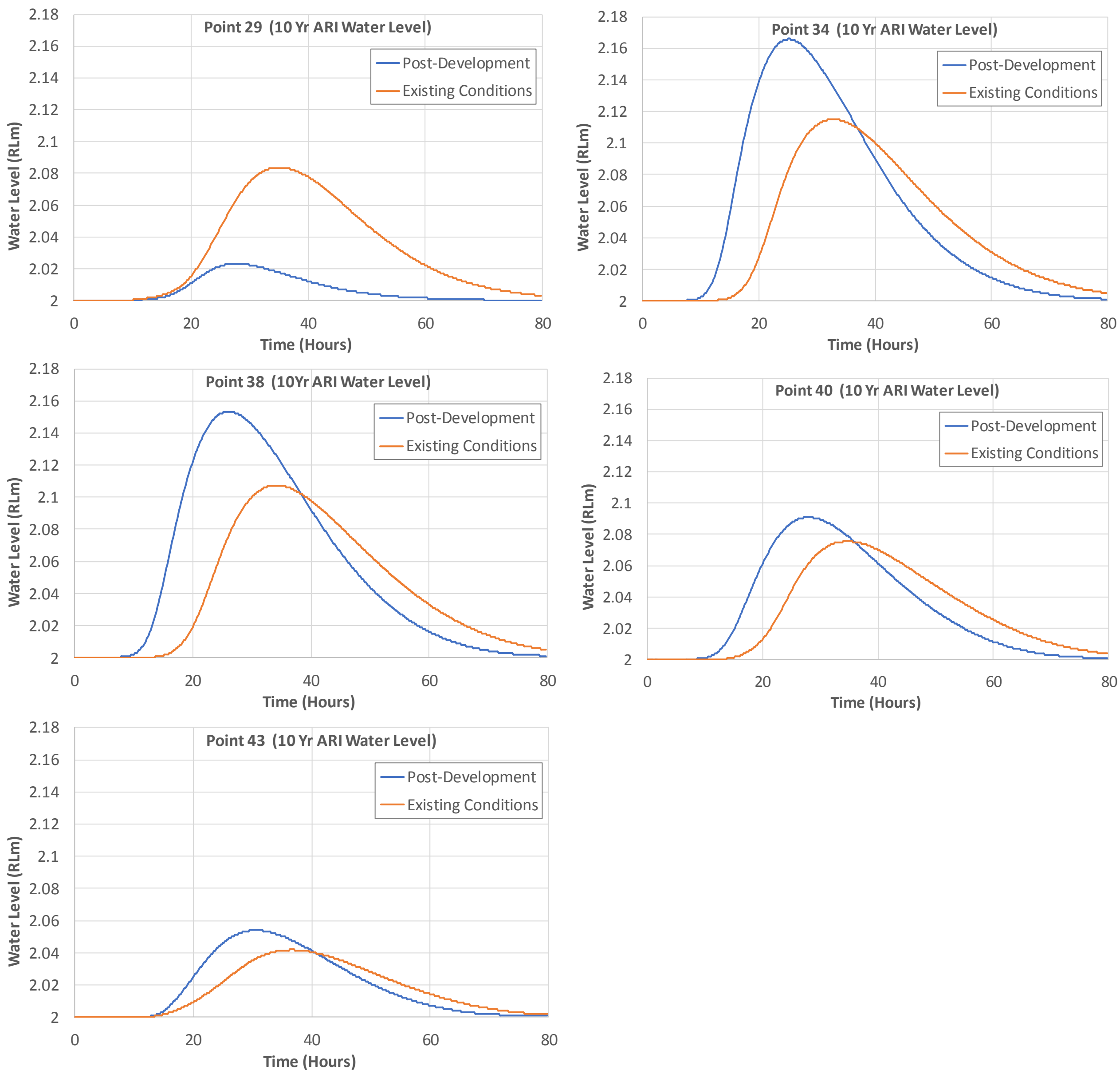
Graph 6: 1 Year ARI Flow Velocity Time Series – Point 34

5.2.2 10 Year ARI Scenario

The impact of the salt ponds on maximum water depth for the 10 year ARI event (Figure 6) is variable across the project area. The most significant change is immediately downgradient of the drainage corridors where maximum water depth increases by more than 0.3m over a very limited areal extent. However, again, the change in maximum water depth is much less than this for the vast majority of the area downgradient of the ponds.

Graphs 7a-e provide the modelled water level time series for existing and post-development conditions at several locations (which are illustrated in Figure B). The change in maximum depth within the tidal creeks downstream of the salt ponds ranges between -0.06m (at Point 29 which is located away from the drainage corridors and thus receives decreased flood flows post-development) and 0.05m (at Points 34 and 38 which are located closer to the two larger drainage corridors).

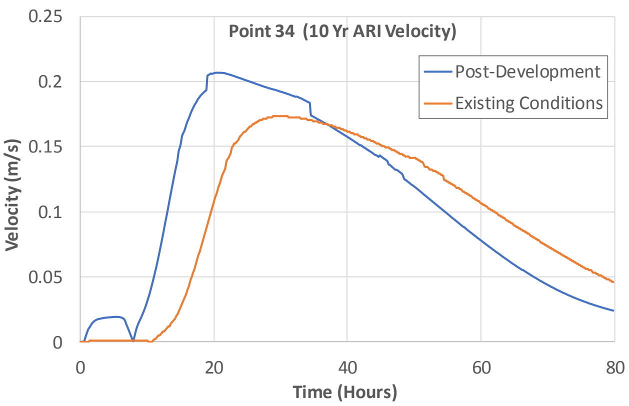
Figure 6 suggests that the maximum water depth increases significantly (up to approximately 0.5m) at the location of discharge from the diversion drain to the southern boundary of the salt ponds. However, as discussed in Section 5.2.1, the degree of impact at this location ids potentially being overestimated by the model. This is because the pre-development simulation did not account for flows within the unnamed creek that discharges at this location. The impact from diverted Peter Creek flows to this location will need to be further defined during the next phase of investigations.



Graphs 7a-e: **10 Year ARI Water Level Time Series – Tidal Creek Locations**

Figure 7 shows the change in maximum flow velocity for the 10 year ARI event. The most significant change in maximum velocity occurs immediately downstream of the drainage corridors where the concentrated flood flows discharge to the mudflats. Peak velocities increase by more than 0.3m/s at this location but only over a limited areal extent.

By comparison, the change in velocity further downstream of the two major drainage corridors (at Point 34) is only minor, increasing from 0.17 m/s to 0.21 m/s (Graph 8 below).



Graph 8: 10 Year ARI Flow Velocity Time Series – Point 34

5.3 Summary of Hydrological Impacts

5.3.1 Flow Depth and Velocity

The hydrological impacts of the project in terms of water levels and velocities are generally limited to immediately upstream and downstream of the ponds and have been estimated as low.

The estimated increases to peak 1 year ARI flow depth and velocity at the end of the drainage corridors is 0.2m and 0.2m/s respectively. However, these impacts are very localised to the area immediately downstream of the drainage corridors, with the increases to peak water depth and velocity further downstream (within the tidal creeks) being estimated at <0.1m and <0.05m/s respectively.

The estimated increases to peak 10 year ARI flow depth and velocity at the end of the drainage corridors is 0.3m and 0.3m/s respectively. However, again, these impacts are very localised to the area immediately downstream of the drainage corridors, with the increases to peak water depth and velocity further downstream (within the tidal creeks) being estimated at <0.1m and <0.05m/s respectively.

The impact to flow depths and velocities at the downstream end of the northern and southern diversion drains (i.e. at the northern and southern extents of the salt ponds) is also estimated as high, up to approximately 0.5m for the 1 year ARI event. It should be noted that this simulation assumed a low sea level; during higher sea level conditions when the intertidal zone is inundated (as modelled with the 10 year ARI event) water levels will be controlled by the sea level thus reducing the impact of the salt ponds. It should also be noted that the model is likely to be overestimating the impact at both of these locations for the reasons discussed in Section 5.2.1.

5.3.2 Inundation Time, Sediment Loads and Currents/Jets

Impacts from the salt ponds on inundation times is expected to be minor. The flood flows discharge to coastal mudflats which in turn are flooded and discharge to the sea via tidal creeks. The salt ponds do not affect the tidal creeks or the mechanism by which flood flows discharge to the sea, except for the removal of some of the mudflat area. The impact of this is likely to be a reduction in inundation time due to a reduction in the available flood storage area / capacity of the mudflats. However; the dominant process that will determine the inundation regime / duration is the coincident tidal conditions during flood events.

The channelization of flows discharging to the mudflat via the drainage corridors has the potential to create jets or currents. Higher velocity flow (such as occur in a culvert) has the ability to retain its jet-like character for some distance as it passes downstream. However, it is noted that he modelled flow velocities within the drainage corridors are low and the estimated extent of velocity impacts is limited to a small area downstream of the drainage corridors, indicating that there is limited potential for this to occur.

The concentration of flood flows into drainage corridors also has the potential to impact sediment loads and distribution. Whilst the salt ponds and drainage corridors are not expected to generate significant sediment loads (due to the low gradients and flow velocities) the natural sediment loads of the upstream catchments will be concentrated, via the drainage corridors, to the points of discharge to the mudflats. Here sediment laden floodwaters will mix with sea water and be discharged to the sea via tidal processes. The potential impact of redistribution of sediment loads on downstream habitats has not been assessed.

5.3.3 Freshwater Flows to Environment

Another potential hydrological impact related to the altered drainage flow paths (i.e. the concentration of flood flows into drainage corridors through the salt ponds) is the potential effect on ecological processes downstream that rely on freshwater inflows.

Under existing conditions flood flows discharge via creeks to the coastal flats at about the location of the upstream boundary of the proposed salt ponds. Here the flood flows inundate a wide expanse of mudflats before ultimately draining via the tidal creeks, i.e. there are no defined flow paths connecting the upgradient creek systems to the tidal creeks.

Under post-development conditions, the flood flows from the upgradient creeks will still discharge to the coastal flats, but via constructed drainage corridors to the downstream boundary of the salt ponds and closer to the tidal creeks. Therefore, flood flows will still discharge to the ocean via inundation of the coastal flats and subsequent discharge via the tidal creeks during lower tide conditions. This process is expected to somewhat limit the impact of the mine infrastructure on the location and distribution of freshwater inputs to the intertidal ecosystems.

The ecological importance of freshwater inputs to the environment from the fluvial (freshwater) flooding regime is beyond the scope of this study. However, it is noted that the occurrence of fluvial floods are highly variable and infrequent compared to the very regular tidal inundation that occurs across the mangrove and algal mat areas. Therefore, it is anticipated that these habitats are not likely to be freshwater dependent.

6 Conclusion

6.1 Summary

This surface water assessment was undertaken to inform the pre-feasibility phase of the project and provides an overview of the pre-development and post-development hydrology as well as concept level flood management design (diversion drain alignments, levee heights, estimated flood levels etc).

The project area is traversed by several creeks, with catchment sizes ranging from 33 to 422 km^2. The salt ponds are situated on very flat terrain at the point where the upgradient creeks discharge to the coastal mudflats. Drainage corridors are required to carry floods from the rear of the ponds though to the front of the ponds and the sea. Some form of lateral diversion drainage is typically required to drain all the minor catchments and depressed areas which would otherwise be dammed against the rear of the salt ponds.

A hydraulic model was used to simulate flood flows through the drainage corridors provided by the proposed pond layout. The model incorporated indicative alignments for lateral drainage diversions and associated levees. Approximate levee heights have been estimated based on the 10 year ARI flood level plus a nominal freeboard of approximately 500-800mm.

Estimated flood levels for the 10 year ARI event are approximately 3.6 mRL at the front of the ponds where flood level is dominated by sea level (storm surge etc) and up to approximately 5.7 mRL along the diversion levees at the rear of the ponds where flood flows from upgradient catchments are intercepted.

Flow velocities within the drainage corridors are ~0.5 m/s or less during a 10 year ARI event and ~1 m/s or less during a 100 year ARI event. The narrow section of drainage corridor B appears to significantly constrain flows causing velocity at that location to increase to ~1 m/s and ~2 m/s for the 10 and 100 year ARI events, respectively.

The hydrological impacts of the mine infrastructure on the downstream environment need to be assessed in the context of the sensitive algal mat and mangrove habitats that occur in the intertidal zone immediately downstream of the ponds. The hydraulic model was used to compare flow depths and velocities for existing conditions and post-development conditions. The impact of the salt ponds on peak flood levels and velocities is generally minor and limited to areas immediately upstream and downstream of the salt ponds. Other potential hydrological impacts have been identified, such as changes to the distribution of freshwater and sediment loads to the downstream habitats, however these have not been defined in this study as they are driven by tidal processes.

In general, the hydrological impacts from the salt ponds are not anticipated to represent a significant physical or ecological risk to the downstream environments; however it should be noted that this study has not included any investigation of mangrove or algal mat sensitivity to the hydrological factors that have been discussed.

6.2 Further Studies

As the project progresses through subsequent design and regulator assessment phases, further detailed hydrological investigations will be required. This may include the following:

- Refinement of hydraulic model to include greater level of infrastructure design detail;
- Refinement of hydraulic model to provide more detailed assessment of potential hydrological impacts to downstream environments, e.g. to inform ecological assessment, to assess potential impacts to particular areas of concern etc;

- Further assessment of flow velocities, scour potential, appropriate erosion protection measures, sedimentation basin design etc;
- Baseline water monitoring program (possibly groundwater and surface water) to further define existing environment and determine criteria and targets for monitoring during life of mine;
- Groundwater assessment with regards to salt pond impacts on local hydrogeology, potential saline seepage etc;
- Post-closure design planning (to demonstrate that surface and groundwater hydrological patterns and quality reflect original conditions etc).

Figures

N

0 200 400
Kilometers

Scale: 1:10,000,000 @A3

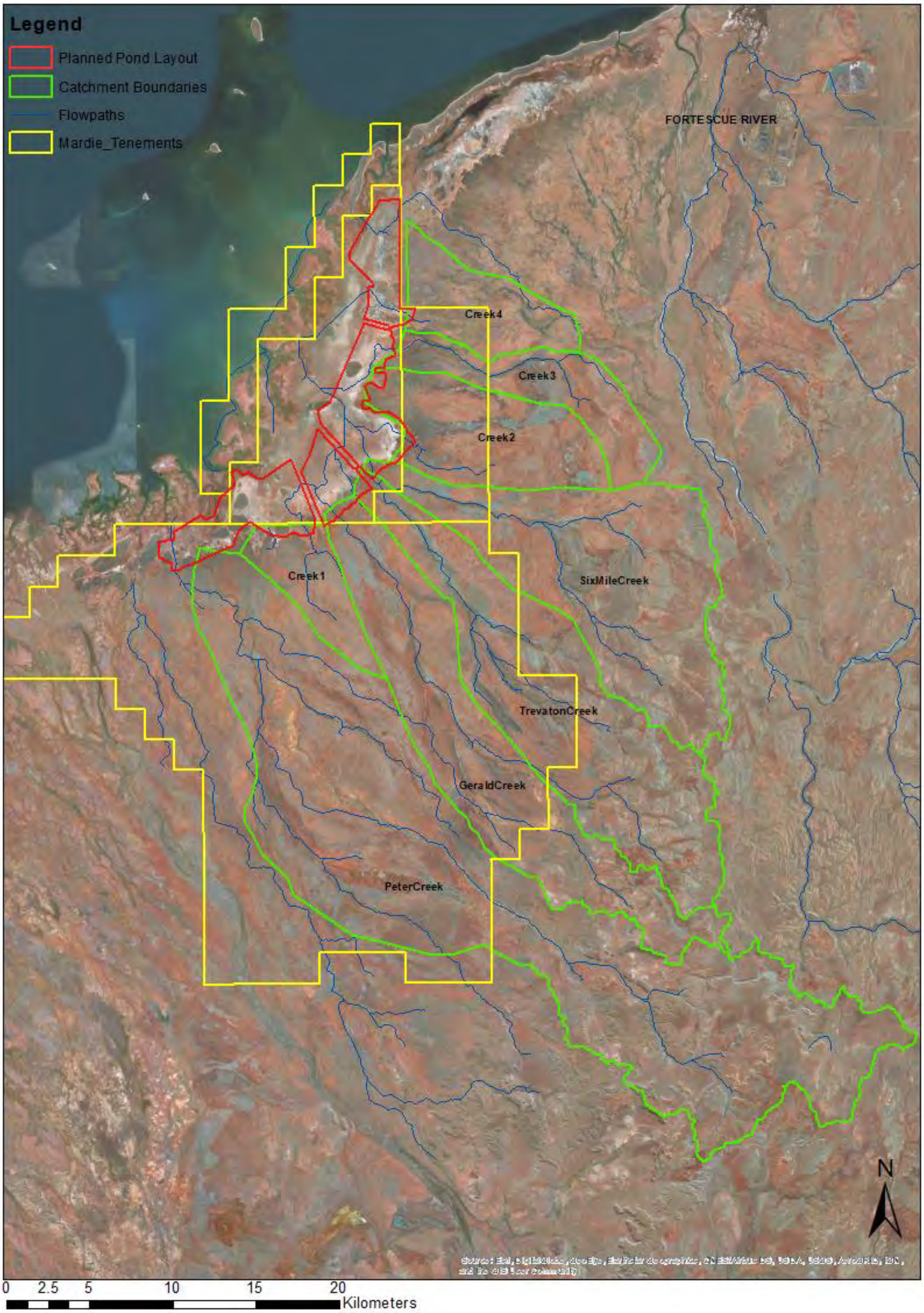
GDA 1994 Zone 50

AUTHOR:	AB	REPORT NO:	003a
DRAWN:	AB	REVISION:	A
DATE:	12/010/2017	JOB NO:	EWP72667

FIGURE 1

REGIONAL LOCATION PLAN

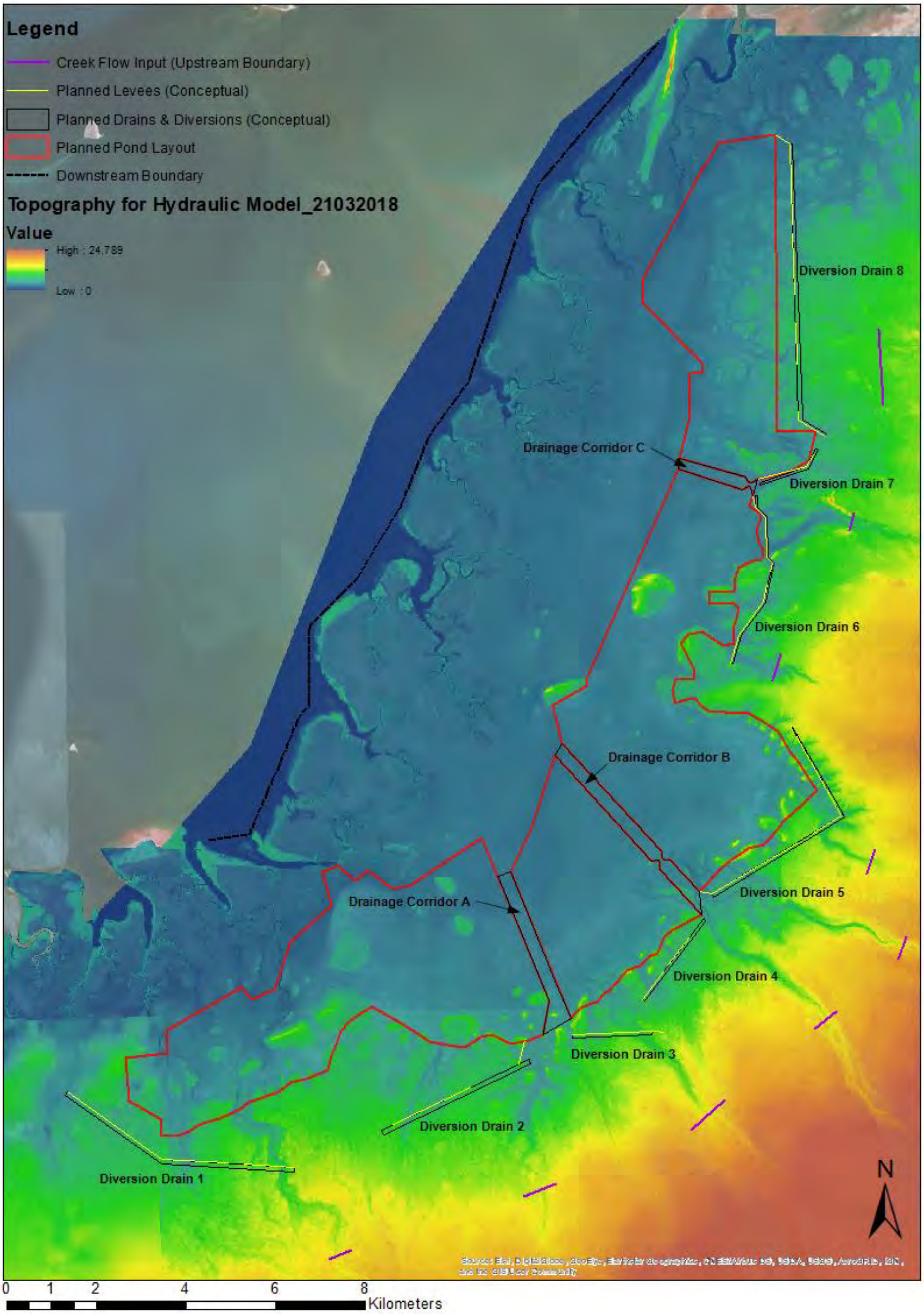
Location: F:\Jobs\EWP72667 - BCI Mardi Salt\Spatial_Data\MapInfo\Workspaces\EWP72667_Figure1



RPS

SURFACE WATER CATCHMENTS FIGURE 2

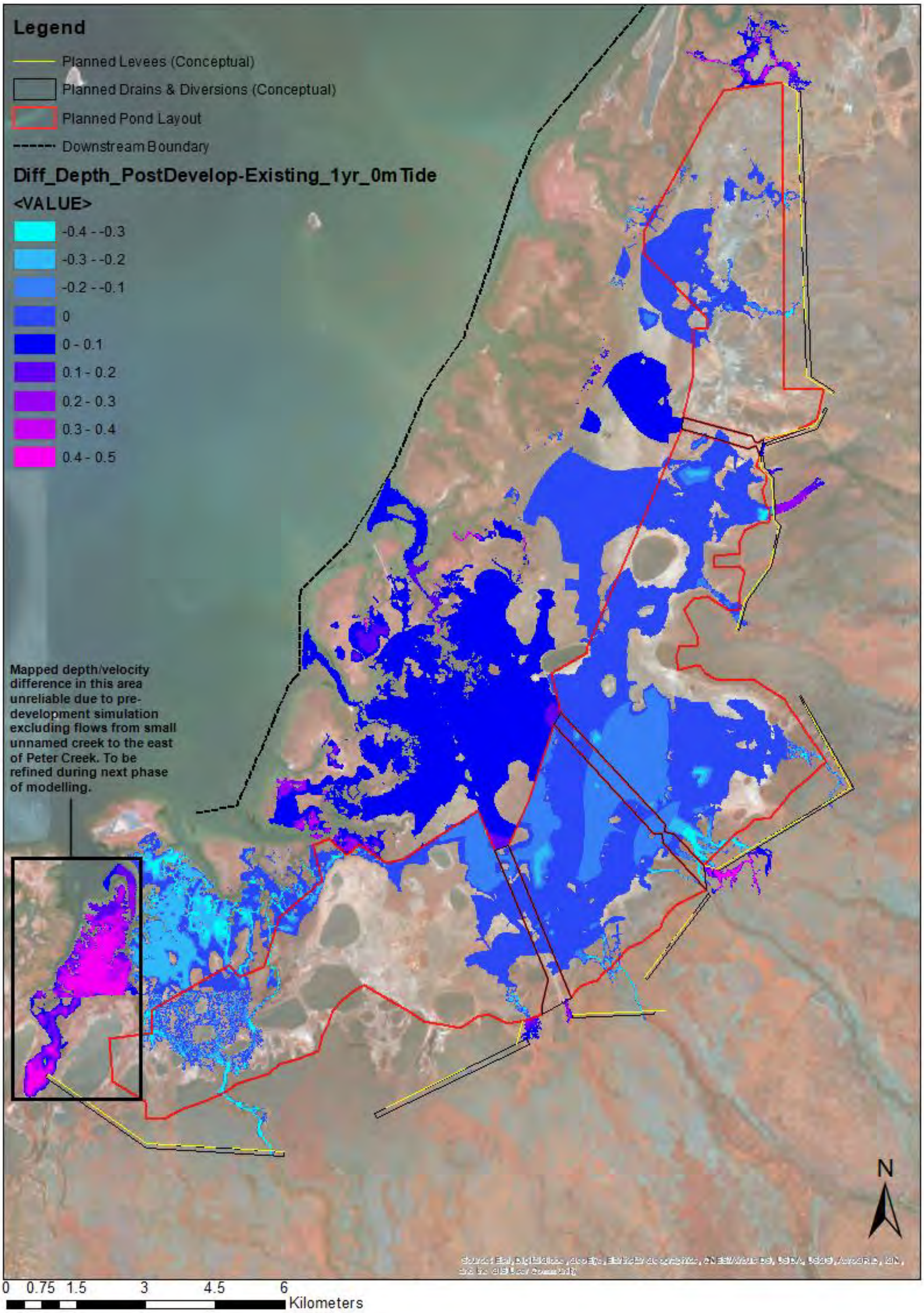
EWP72667



RPS

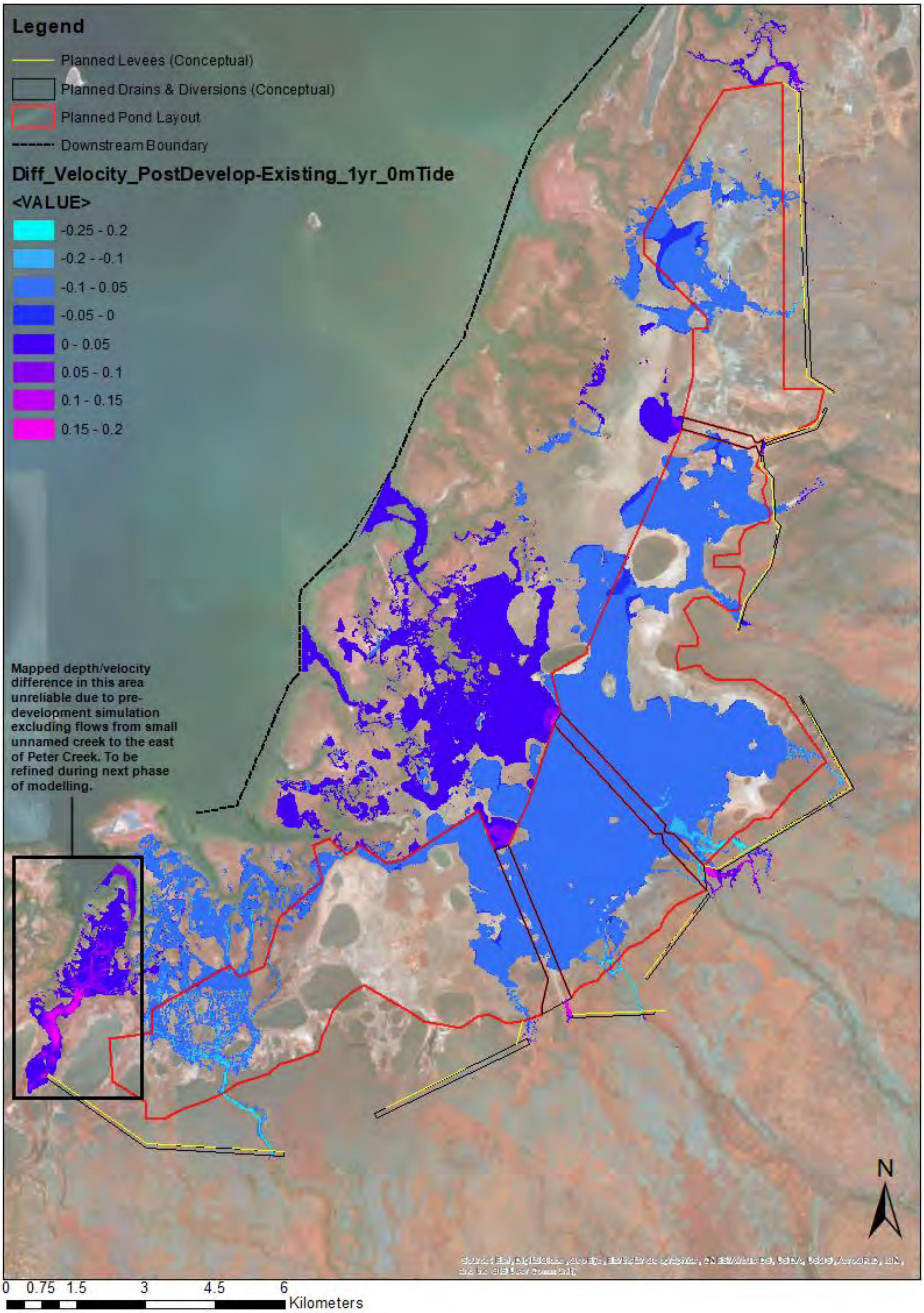
HYDRAULIC MODEL SETUP FIGURE 3

EWP72667



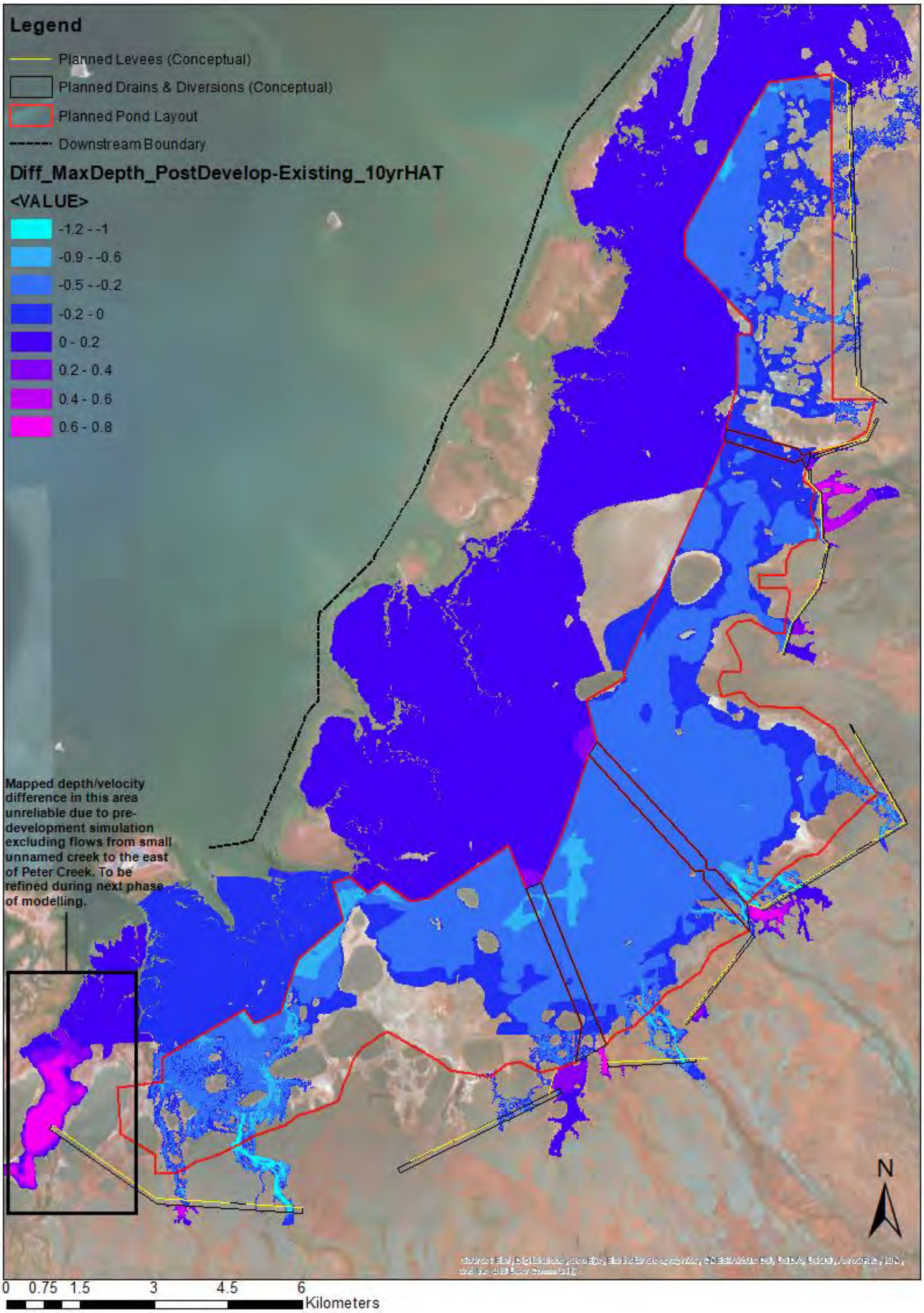
DIFFERENCE MAP – 1 YEAR ARI MAXIMUM DEPTH: POST-DEVELOPMENT MINUS EXISTING CONDITION FIGURE 4

EWP72667



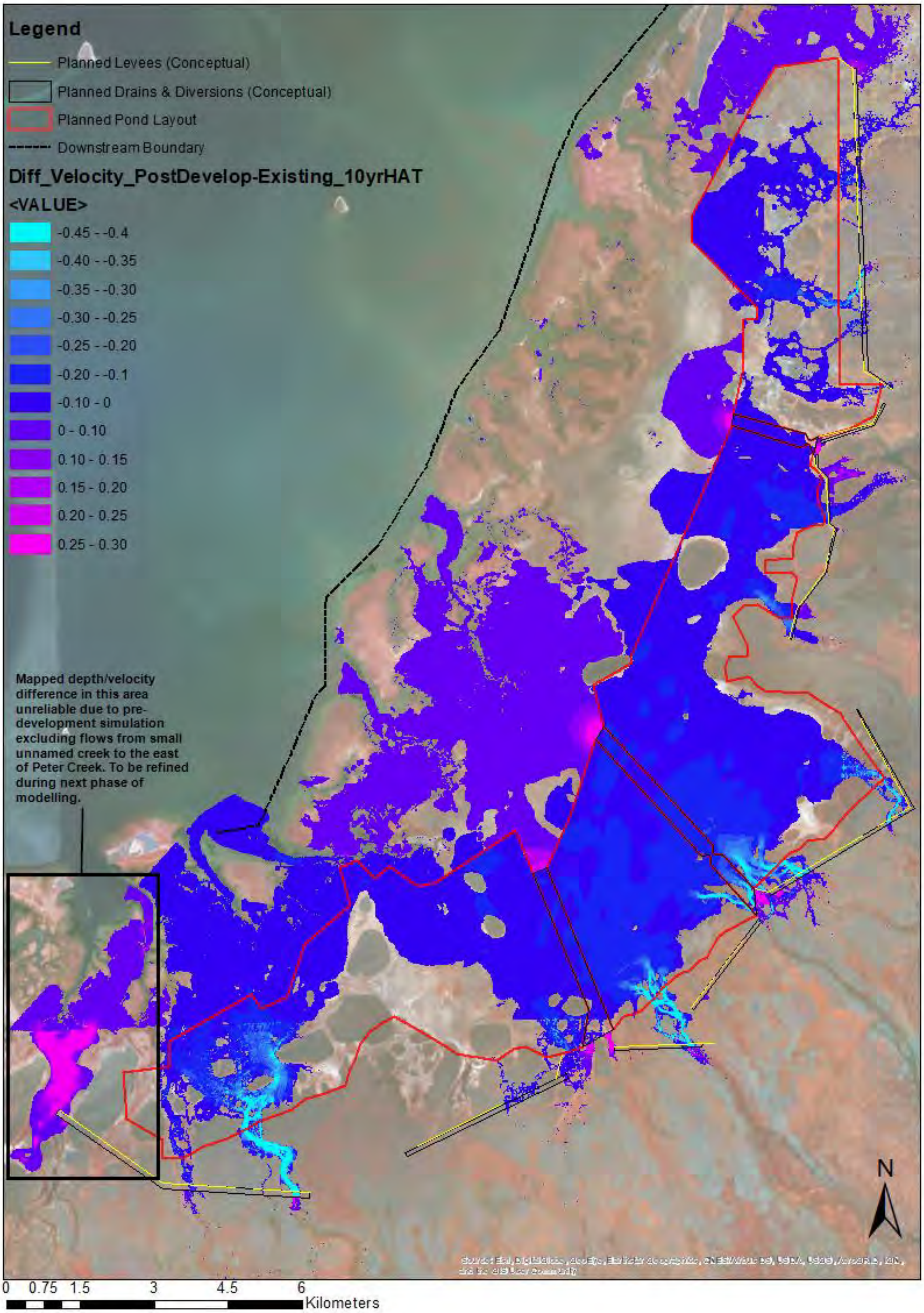
DIFFERENCE MAP – 1 YEAR ARI MAXIMUM VELOCITY: POST-DEVELOPMENT MINUS EXISTING CONDITION FIGURE 5

EWP72667



DIFFERENCE MAP – 10 YEAR ARI MAXIMUM DEPTH: POST-DEVELOPMENT MINUS EXISTING CONDITION FIGURE 6

EWP72667



DIFFERENCE MAP – 10 YEAR ARI MAXIMUM VELOCITY: POST-DEVELOPMENT MINUS EXISTING CONDITION FIGURE 7

EWP72667

Appendix E

Mardie Salt – Hydraulic modelling for rear-of -pond flood levels (RPS 2019)

MAKING COMPLEX EASY

Our ref: EWP72667.002

Level 2, 27-31 Troode Street
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T +61 8 9211 1111

Date: 16 August 2019

Rob Sceresini
GR Engineering Services Limited
71 Daly Street
ASCOT WA 6104

Dear Rob,

Mardie Salt – hydraulic modelling for rear-of-pond flood levels

Background

RPS has undertaken hydraulic modelling of stormwater flows to support GR Engineering Services' preliminary engineering design. This scope of works focused particularly on the rear-of-pond locations where it proposed that flood protection infrastructure (e.g. bund associated with the access road alignment) will be provided to mitigate flood impacts to the salt ponds and gas pipeline infrastructure.

RPS provided the results of some initial modelling to GRES via email on 9th May 2019 which focused on the flood levels in the vicinity of the gas pipeline corridor between Ponds 2 and 3. The results provided herein include a larger portion of the salt ponds and access road alignment and supersede the information provided on 9th May.

Note regarding topographic data sources

Various topographic data sources have been sourced from the client (BCIM) or public datasets since RPS commenced working on this project in 2017, including:

- Publicly available SRTM (satellite) topographic data
- Landgate sourced photogrammetric Digital Elevation Model (DEM), provided by BCIM
- Field-surveyed control points (spot levels measured by Land Surveys at various locations across the project site)
- 0.25m contours provided by BCIM (of unknown origin but understood to be sourced from "LiDAR")
- Photogrammetric survey captured by Aerometrex in 2019.

RPS makes the following observations regarding the above data sources:

- The SRTM data is coarse and of relatively low vertical accuracy; this data was only used for catchment delineation and not hydraulic modelling.
- The Landgate-sourced DEM covers a large and useful area (it is the only one of the site-specific data sources which extends significantly inland of the access road to include some of the areas relevant to

the flood modelling). However, the Landgate data has significant accuracy/quality issues. There are “seams” through the dataset along which large vertical shifts (up to approximately 2m) occur in the data.

- When compared to the field-surveyed control points, the Landgate DEM was between 0.12 and 1.74m higher, confirming that the error in the Landgate DEM is highly spatially variable.

- The 0.25m contours provided by BCIM appear to match well with aerial photography and field captured photography of tidal fluctuations. The contours were adjusted by +0.47m to better match field surveyed spot levels. The RPS team also manually adjusted the contours to include greater detail around the tidal creeks, based on aerial photography and field surveyed spot levels.

- The recently captured (by Aerometrex) photogrammetric survey data is detailed and accurate, however it only covers a portion of the area relevant to the flood modelling (it does not extend far enough seaward to capture all of the tidal creeks, nor far enough inland to capture some of the floodplain areas important to the flood modelling).

- Due to the data limitations/issues summarised above, a composite DEM has been produced by RPS for this flood modelling scope of works. The composite DEM includes the Aerometrex survey and is supplemented by the BCIM-provided contours for areas downstream of the Aerometrex survey, and the Landgate data (with an adjustment/correction included) for areas upstream of the Aerometrex survey.

Overview of modelling scope

The hydrological assessment methodology and results (e.g. flow hydrograph estimation) has previously been described in the Pre-Feasibility Surface Water Assessment report. The same flow hydrographs for various design events (e.g. 10, 20, 50, 100 year ARI) were used in this modelling exercise.

The same XPSWMM hydraulic model as used in the Pre-Feasibility Surface Water Assessment was used for this scope of works. A 25m grid cell size was used for the hydraulic modelling to provide a reasonable balance between detail and simulation times. As detailed in previous reports, sensitivity testing has indicated that cell sizes smaller than 25m result in negligible changes to the results.

This modelling exercise adopted a low sea level boundary condition (0.5m AHD) in order to simulate “worst-case” flow velocities along bunds and through drainage corridors etc (i.e. a high sea level state boundary condition could possibly result in higher tailwater conditions and thus lower flow velocities).

The following scenarios were modelled:

- “<u>base-case</u>” which assumes an infinite wall along the modelled “bund1” and “bund 2” alignments (refer to Figure 1). In this scenario, all floodwater is forced to flow around the bunds to the points of discharge which are via the western-most extent of the ponds or via the two drainage corridors (between ponds 3 & 4, and between ponds 7 & 8).

- “<u>lateral drains</u>” which is as per the base-case scenario but with the addition of nominal 28m base width drains alongside the flood bunds at selected elevated locations (i.e. where lateral flow behind the flood bund is likely to be improved by the inclusion of the drains.

- “<u>floodway</u>” scenarios where floodwater is allowed to overtop the bunds via 300m long floodways at selected low points in the terrain and flow into the rear of the salt ponds. The floodway scenario was modelled with floodway elevations set at various heights (based on the 10, 20 and 50 year ARI top water level as modelled in the “lateral drains” scenario.

An overview of the key site attributes and their relative locations is provided in Figure 1. Figure 2 shows where the proposed wall alignment for Pond 7 blocks the main flow channel of the creek that discharges via that location; for the purpose of this scope of works a 50m wide cut was modelled to maintain a flow path to the drainage corridor. The 50m wide cut drain resulted in a modelled peak velocity through the cut of 1.6 m/s in the 100 year ARI event (1.2 m/s in the 20 year ARI event). A wider cut may be required depending on the scour potential of the material and the scour protection requirements.

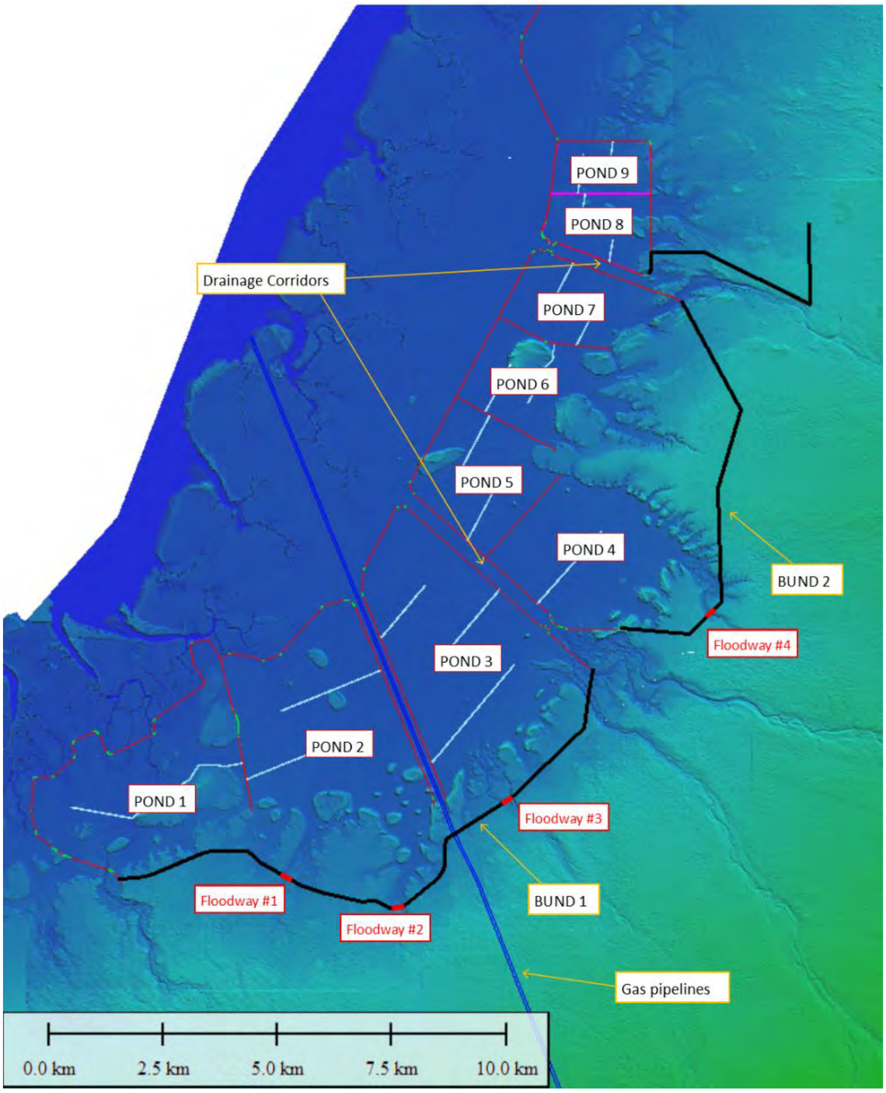


Figure 1: Site overview

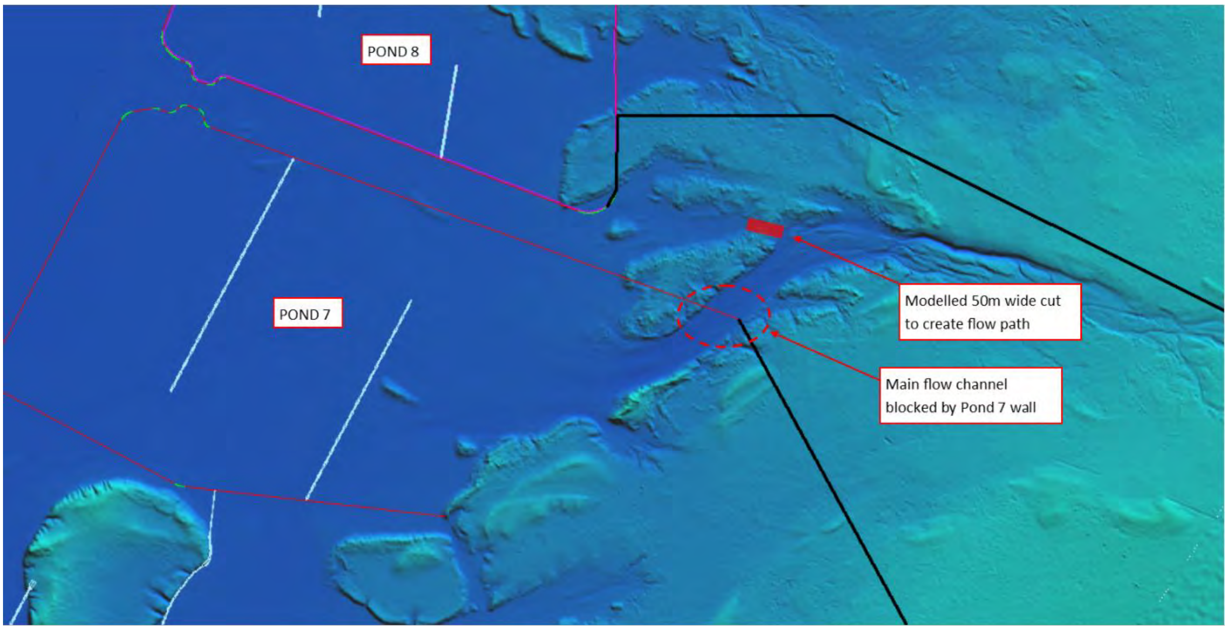


Figure 2: Pond 7 interface with flow channel

Results

Graphs 1 to 6 below provide the results for each scenario, presented as long-sections showing the natural surface level, the elevation of lateral drains and floodways and the modelled top water levels. The long-sections demonstrate the reduction in flood level that results from the inclusion of lateral drains and floodways to enhance drainage of floodwater around and/or into the salt ponds.

Also provided below in Tables 1 to 3 are the total discharge volumes into the salt ponds via the floodways, along with an estimation of the resulting water level rise in the receiving pond. It should be noted that the reported values are specifically for the design storms that were modelled (which were the critical storm durations resulting in peak flow rates, generally the 12 to 24 hour duration storm). It is possible that larger volumes of discharge to the salt ponds could occur as a result of longer duration storm events.

Table 4 presents the peak flow over the floodways for the various ARI storm events and floodway heights.

Table 1: Floodway discharge volumes (floodway set at 10 year ARI TWL)

			100 year ARI event		50 year ARI event		20 year ARI event	
Floodway	Receiving pond	Pond area (km2)	Floodway discharge volume (ML)	Pond water level rise (m)	Floodway discharge volume (ML)	Pond water level rise (m)	Floodway discharge volume (ML)	Pond water level rise (m)
1	2	15.4	1907	0.22	1040	0.12	463	0.05
2			1514		824		292	
3	3	15.5	4830	0.31	2561	0.17	806	0.05
4	4	9.4	2179	0.23	1252	0.13	486	0.05

Estimated water level rise is based on modelled discharge volume over floodway; excludes direct rainfall on ponds.

Table 2: Floodway discharge volumes (floodway set at 20 year ARI TWL)

			100 year ARI event		50 year ARI event		20 year ARI event	
Floodway	Receiving pond	Pond area (km2)	Floodway discharge volume (ML)	Pond water level rise (m)	Floodway discharge volume (ML)	Pond water level rise (m)	Floodway discharge volume (ML)	Pond water level rise (m)
1	2	15.4	1262	0.13	443	0.04	0	0
2			698		222		0	
3	3	15.5	1944	0.13	575	0.04	0	0
4	4	9.4	1157	0.12	440	0.05	0	0

Estimated water level rise is based on modelled discharge volume over floodway; excludes direct rainfall on ponds.

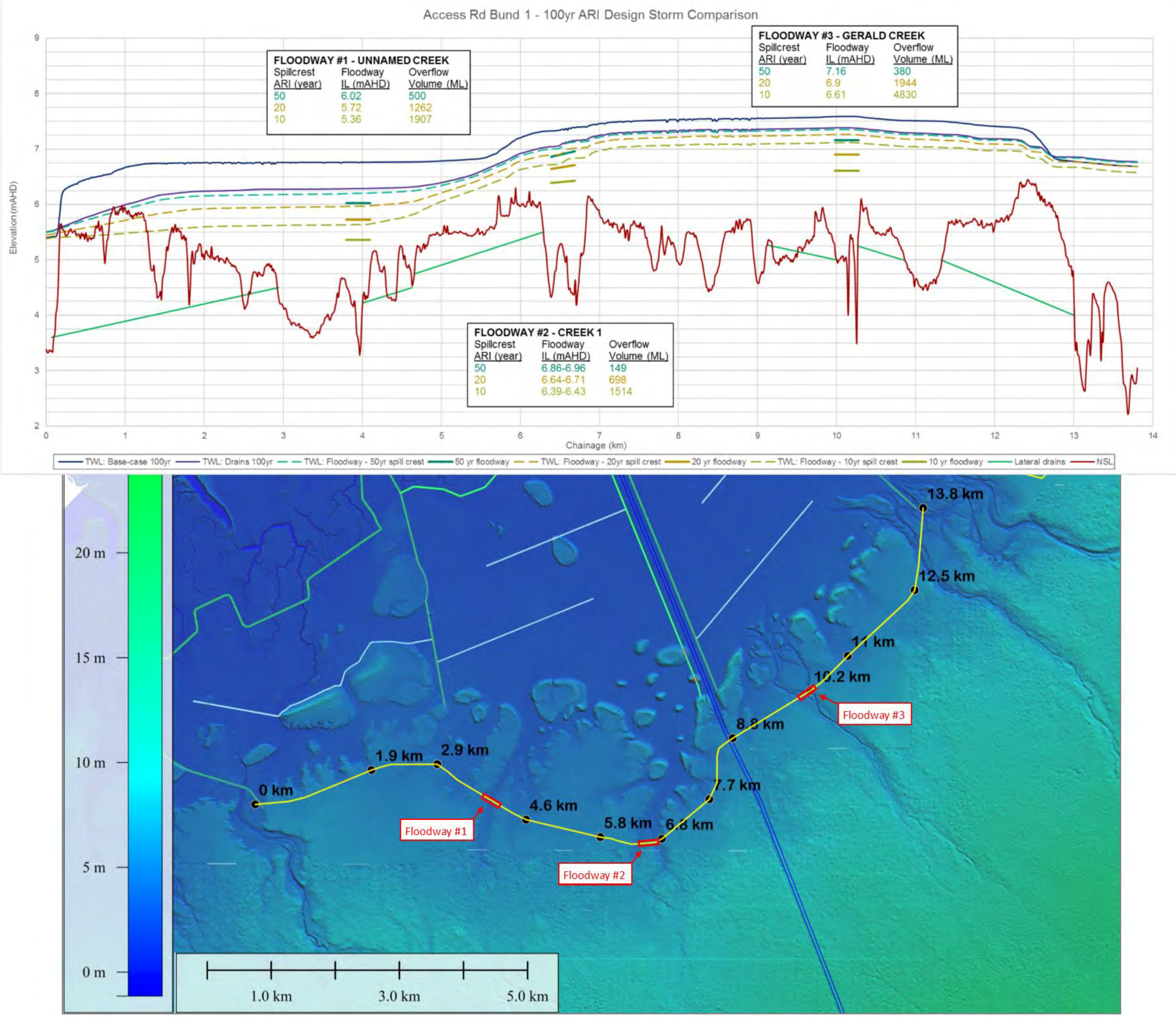
Table 3: Floodway discharge volumes (floodway set at 50 year ARI TWL)

			100 year ARI event		50 year ARI event		20 year ARI event	
Floodway	**Receiving pond**	**Pond area (km2)**	**Floodway discharge volume (ML)**	**Pond water level rise (m)**	**Floodway discharge volume (ML)**	**Pond water level rise (m)**	**Floodway discharge volume (ML)**	**Pond water level rise (m)**
1	2	15.4	500	0.04	0	0	0	0
2			149		0		0	
3	3	15.5	380	0.02	0	0	0	0
4	4	9.4	310	0.03	0	0	0	0

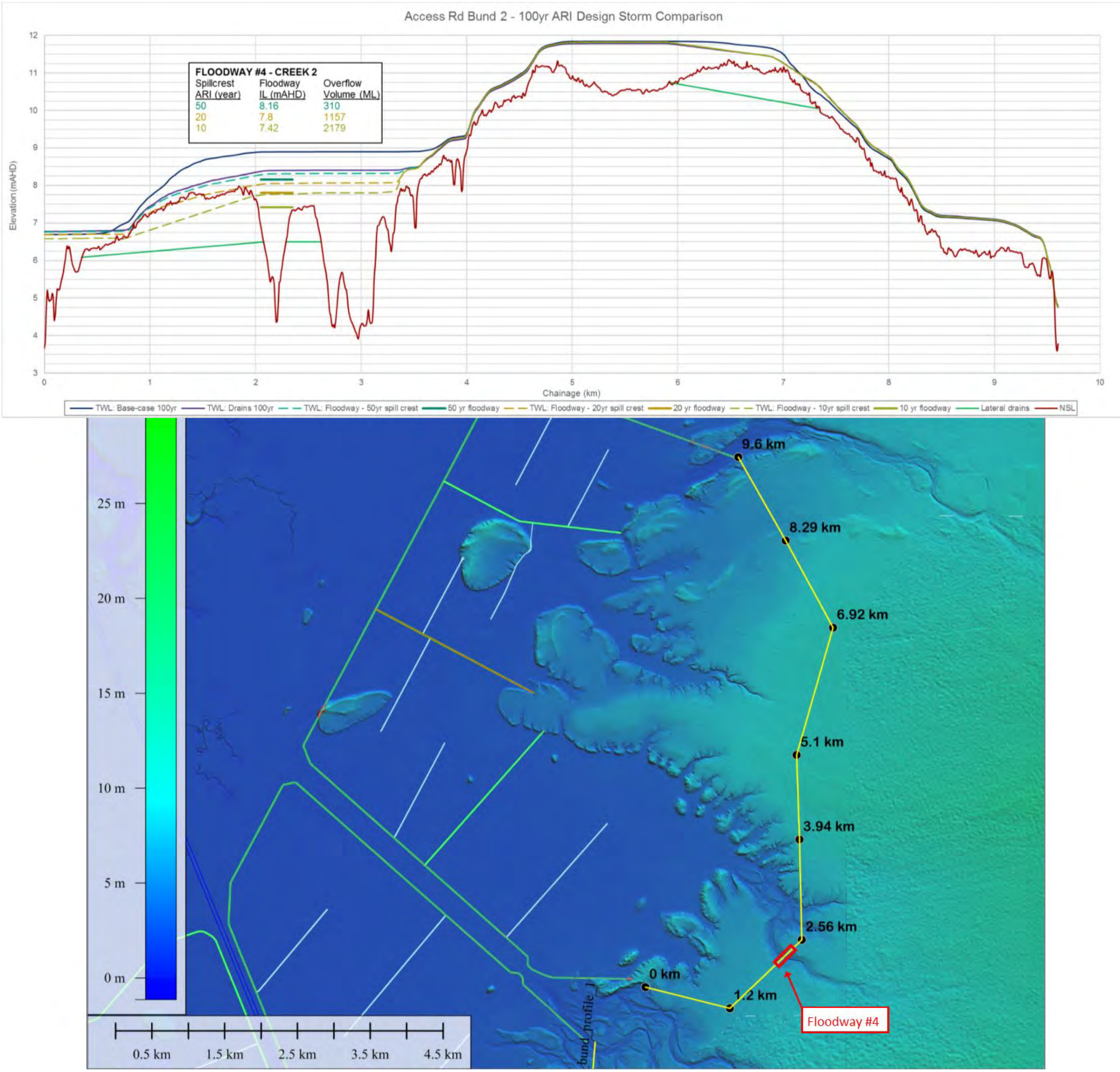
Estimated water level rise is based on modelled discharge volume over floodway; excludes direct rainfall on ponds.

Table 4: Peak discharge rate over floodways (m^3/s)

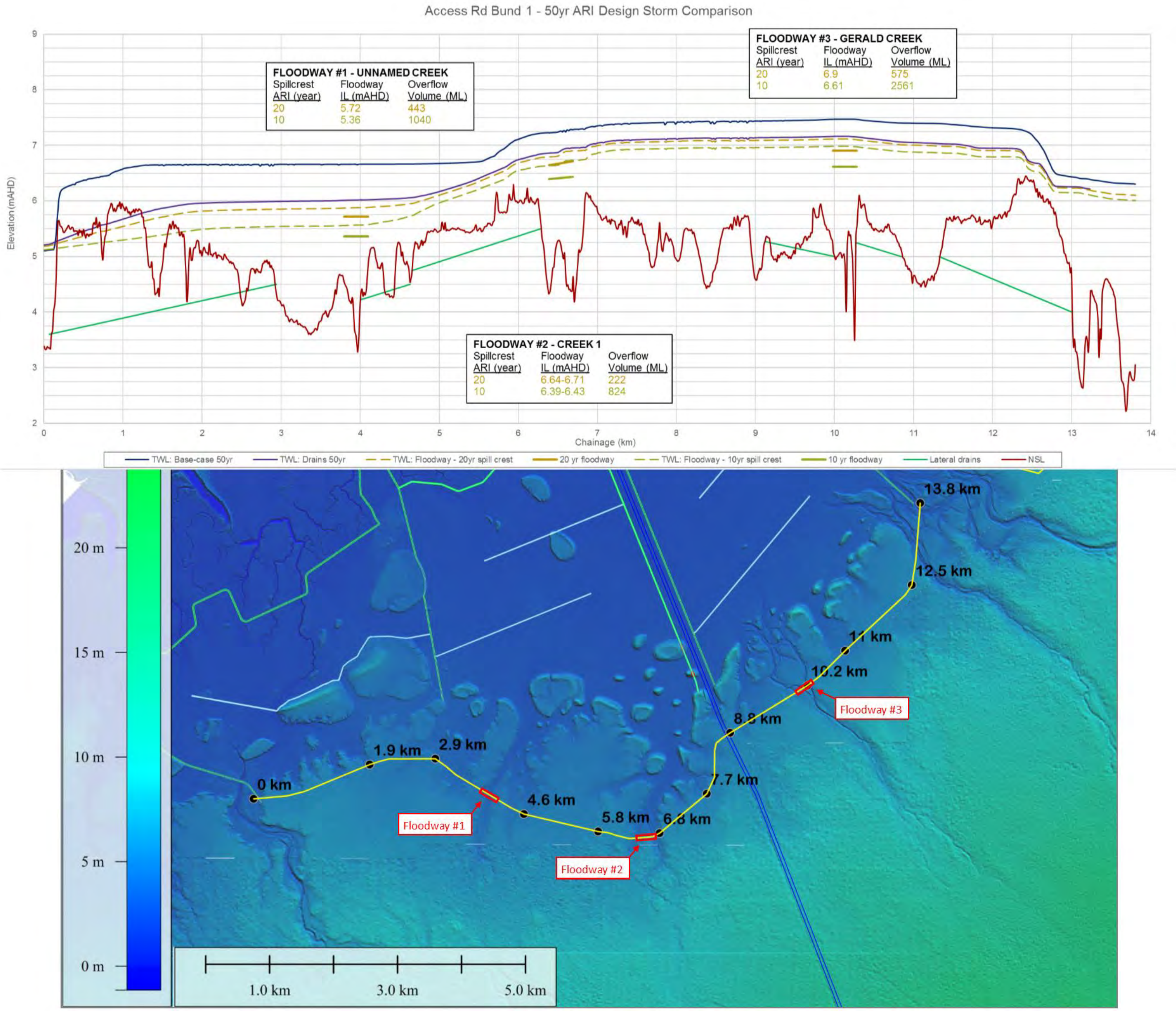
	Spillway at 10 year ARI TWL			Spillway at 20 year ARI TWL			Spillway at 50 year ARI TWL		
Floodway	**100 year ARI event**	**50 year ARI event**	**20 year ARI event**	**100 year ARI event**	**50 year ARI event**	**20 year ARI event**	**100 year ARI event**	**50 year ARI event**	**20 year ARI event**
1	42	24	11	34	15	0	19	0	0
2	40	24	9	24	10	0	8	0	0
3	125	71	24	67	25	0	20	0	0
4	58	39	16	42	20	0	17	0	0



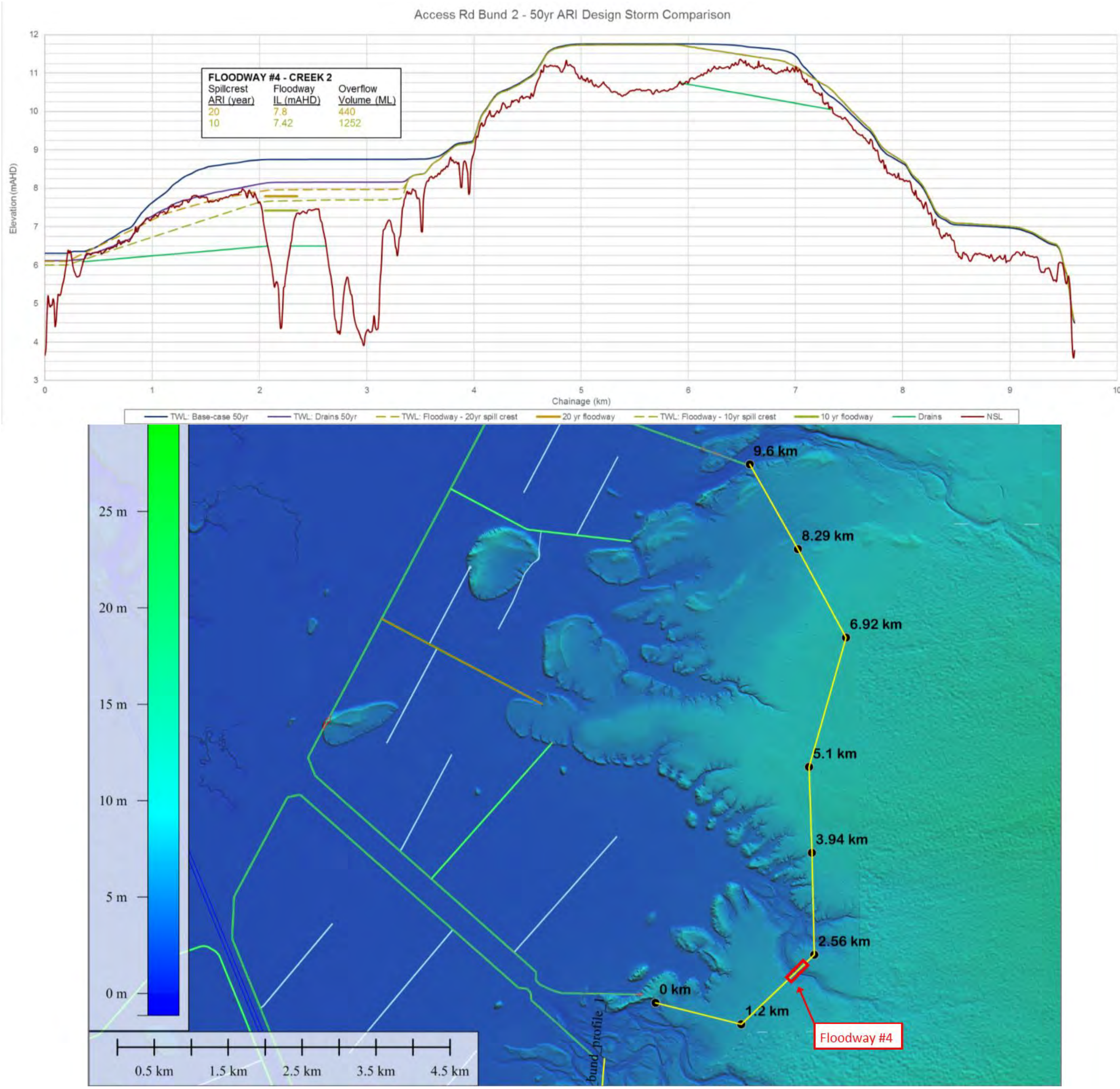
Graph 1: Bund 1 results – 100 year ARI



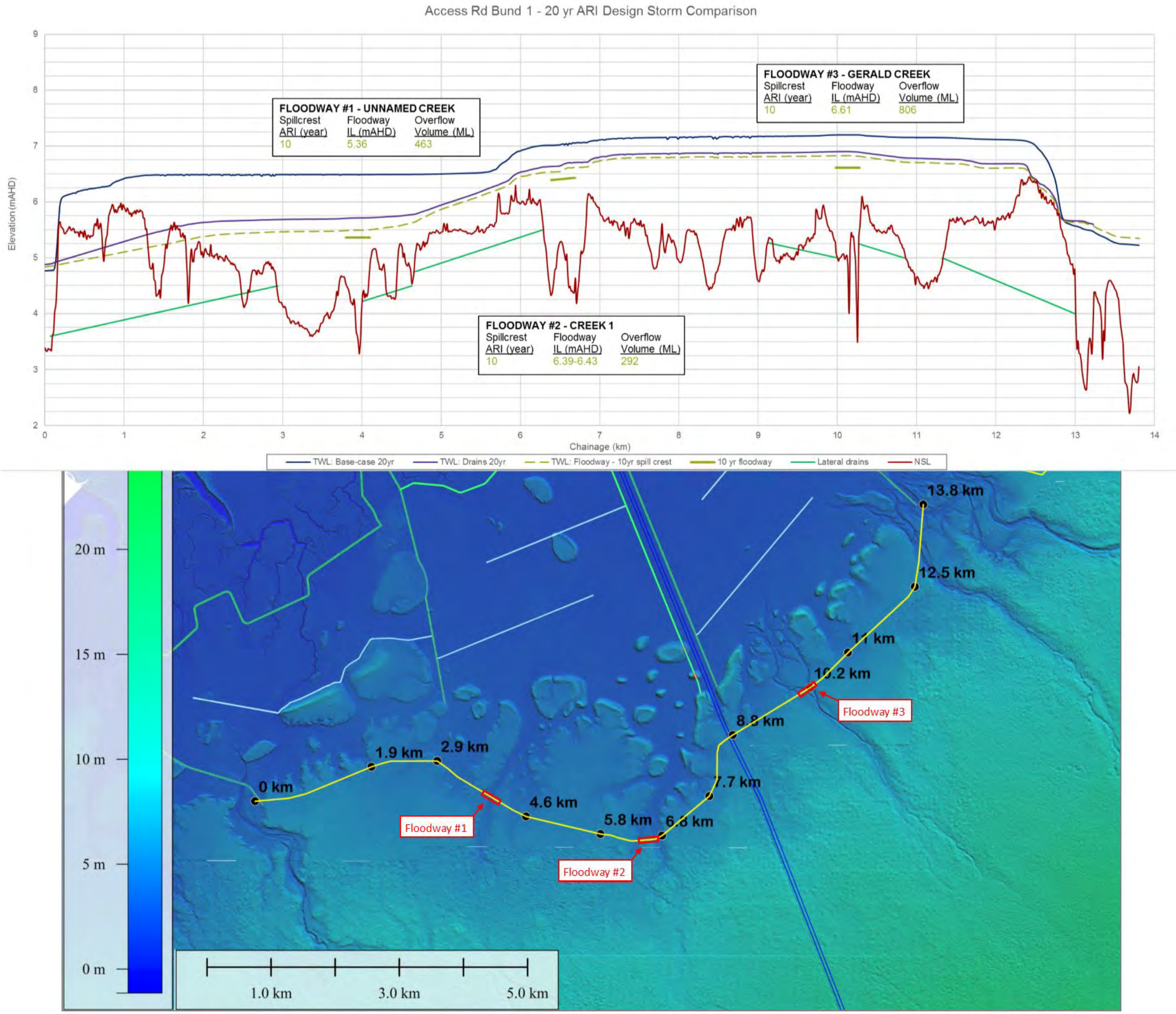
Graph 2: Bund 2 results – 100 year ARI



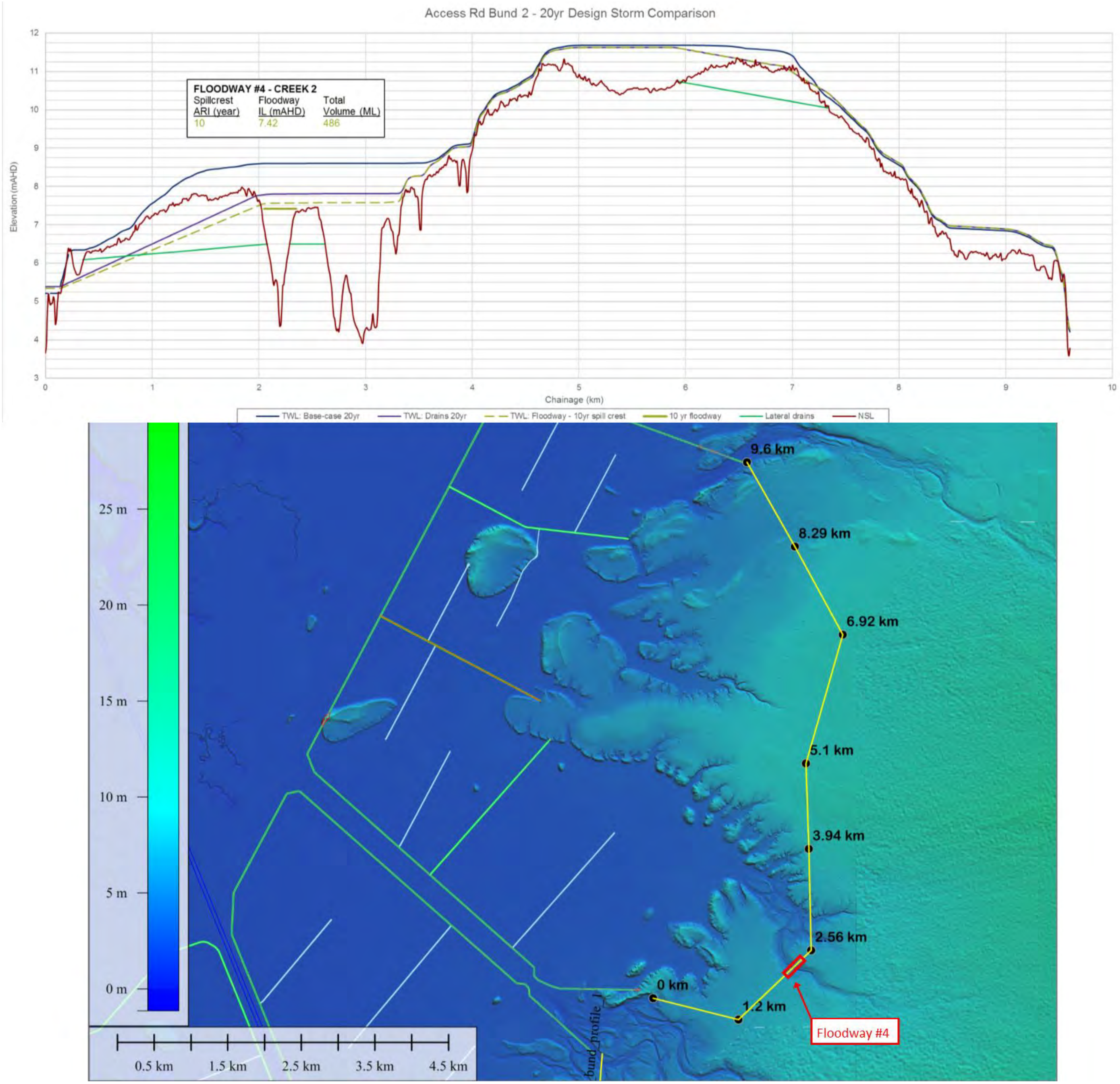
Graph 3: Bund 1 results – 50 year ARI



Graph 4: Bund 2 results – 50 year ARI



Graph 5: Bund 1 results – 20 year ARI



Graph 6: Bund 2 results – 20 year ARI

Figure 3 below illustrates the inundation extent during the peak of the 100 year ARI storm event (shown on the left) and also the inundation extent following recession of the floodwaters when water remains pooled in trapped low areas (shown on the right). The duration of inundation during the peak of the storm (on the left) is in the order of several hours, following which floodwater recedes over the course of approximately 1-2 days until water remains only in the trapped low areas (shown on the right). The extent of inundation in trapped low areas will depend on the elevation at which drains are constructed to partially drain these areas; the extents shown in Figure 3 below are based on the drain elevations that were assumed in the modelling (refer to Graphs 1-6 above).

It is assumed that any trapped low areas could be drained by small relief culverts into the rear of the ponds. Based on the expected discharge rate of a nominal 450mm diameter culvert, the trapped low areas would be expected to drain away within several days. Without relief culverts, any water remaining within trapped low areas would slowly dissipate via infiltration and evaporation which could take weeks to months.

The volume of water trapped in these low areas has been calculated as between approximately 20 and 180 ML for each of the eight trapped low points that were identified, with a total volume of approximately 530 ML for all eight of the areas. Again, this estimate is based on the drain elevations that have been assumed in the model; lower drains would result in less trapped water and higher drains would result in more trapped water. The additional volume of water that would discharge via the relief culverts during the storm event is similar in scale to the volume of trapped water that would be discharged after the storm event (i.e. there would likely be several days of discharge during the storm event followed by several days of discharge of trapped floodwater after the storm event).

The above volume estimate indicates that the volume of floodwater that would be collected in trapped low areas and discharged into the ponds via relief culverts is about an order of magnitude smaller than the volumes that would discharge into the ponds via floodways (if the floodways were set at the 10 year ARI top water level). As another point of comparison, the volume of direct rainfall onto ponds 2, 3, 4 and 6 (the receiving ponds for potential discharge from the trapped low areas) during a 100mm rainfall event (equivalent to a 5 year ARI-12 hour rainfall event) would be approximately 4,600ML, which is also an order of magnitude higher than the volume of water trapped in low points.

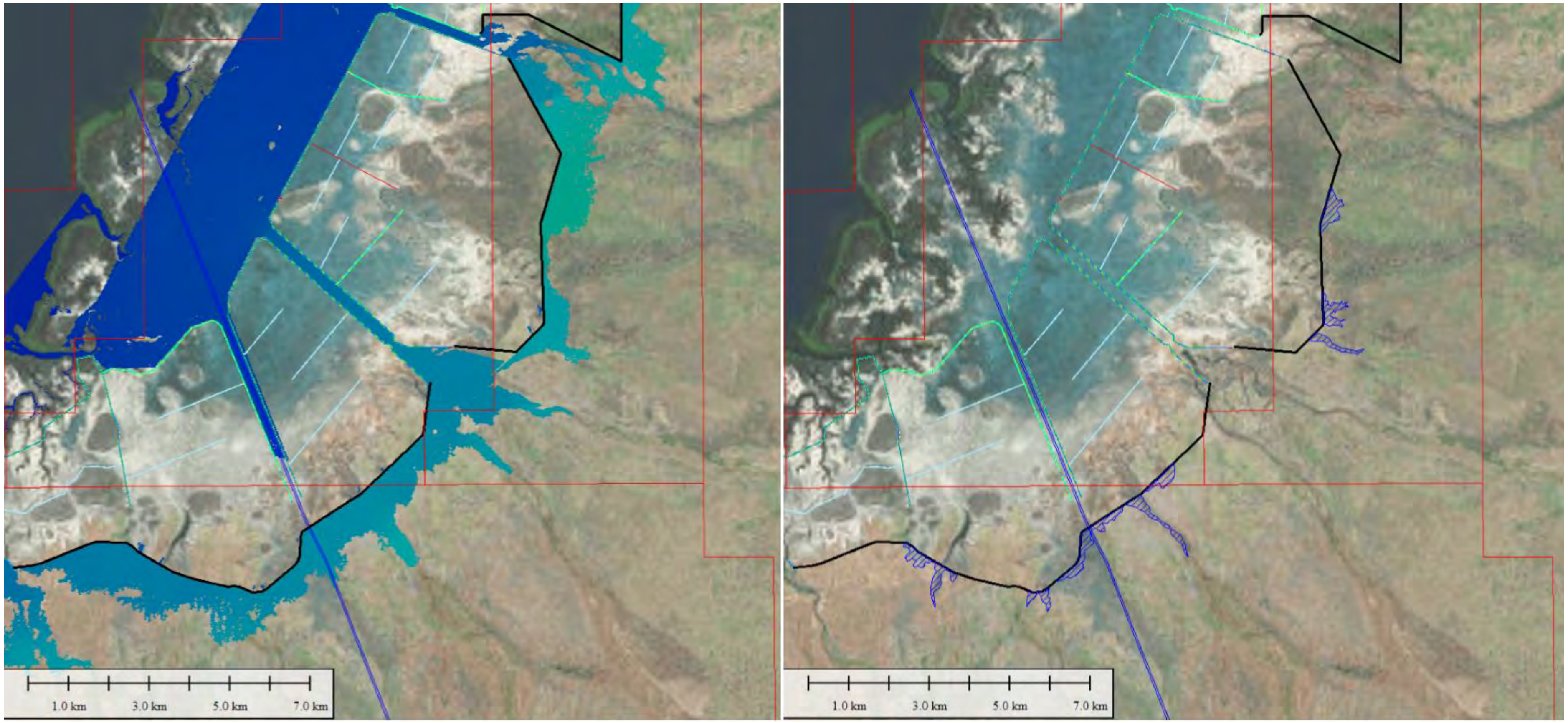


Figure 3: Extent of inundation, during storm peak (left image) and in trapped low areas following storm (right image)

Conclusions and recommendations

The modelling results indicate that flood levels are significantly lowered (by up to approximately 0.5m) in some locations by the inclusion of lateral drains to enhance lateral flow of floodwater behind the bund. The results also indicate that the inclusion of floodways (to allow some discharge of floodwater into the rear of the salt ponds) further reduces flood levels in some locations. The effectiveness of the floodways depends on the elevation at which they are set, with lower floodways resulting in lower flood levels and the trade-off being floodwater discharging into the salt ponds more frequently and in greater volumes. If the floodways are set at the 10 year ARI top water level (e.g. only events rarer than 10 year ARI will discharge into the ponds) then the flood levels decrease by a further 0.25-0.5m compared to the scenario with lateral drains but no floodways.

The volumes of floodwater to discharge into the ponds via floodways has been estimated, along with the resulting rise in salt pond water level. For the most “aggressive” scenario where floodways are set at the 10 year ARI top water level, the estimated pond water level rise due to floodwater discharged over the floodways is ~0.05m in a 20 year ARI event and ~0.2-0.3m in a 100 year ARI event.

It is noted that the additional flow through the Ponds 3/4 drainage corridor (as a result of diverting Gerald Creek to that corridor to avoid flow along the pipeline corridor) is likely to have increased the flood levels within and upstream of the Ponds 3/4 drainage corridor relative to what was modelled previously in the Pre-Feasibility Surface Water Assessment. It is expected that the impact of these higher flood levels is limited to within and immediately upstream of the drainage corridor and that there is negligible impact on flood levels along the more elevated segments of the access road / flood bund. However, further assessment of flood levels within the Ponds 3/4 drainage corridor (including how these are influenced by the corridor width and downstream ocean/tidal levels) may be warranted.

Yours sincerely,
for RPS Australia West Pty Ltd

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