# MARDIE SURFACE WATER ASSESSMENT

## Mardie salt project

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## Approval for issue

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# Contents

## Tables



## Figures

**(contained within report text)**



## Graphs

## Appendices

# 1 INTRODUCTION

## 1.1 Background

BCI's proposed Mardie Salt Project involves the production of 4 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 |

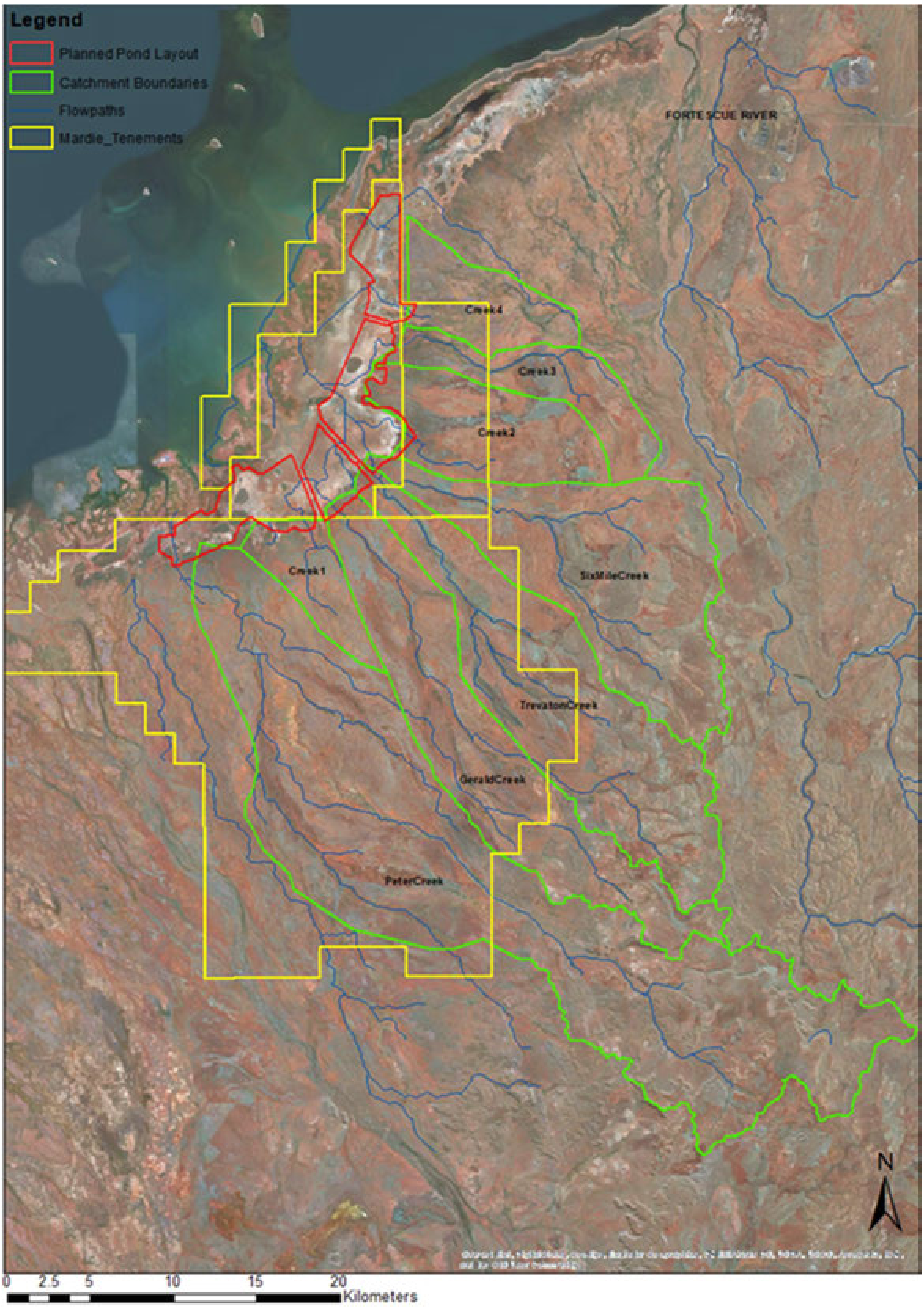



**Figure 2: Surface water catchments**

### 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 km²) 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 km² 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 110 m³/s, and the 100 year flow as 515 m³/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.

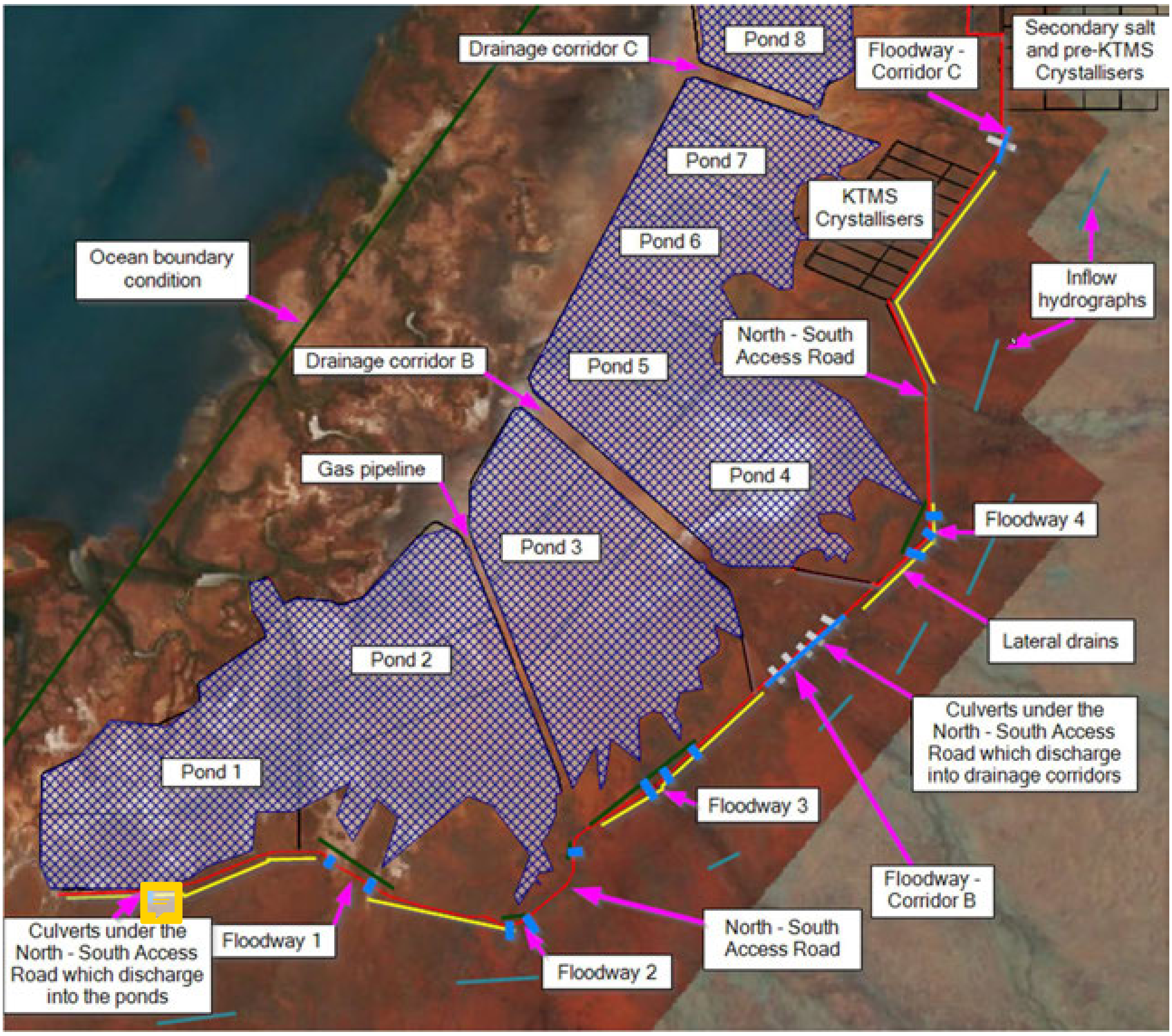



**Figure 3: Site overview**

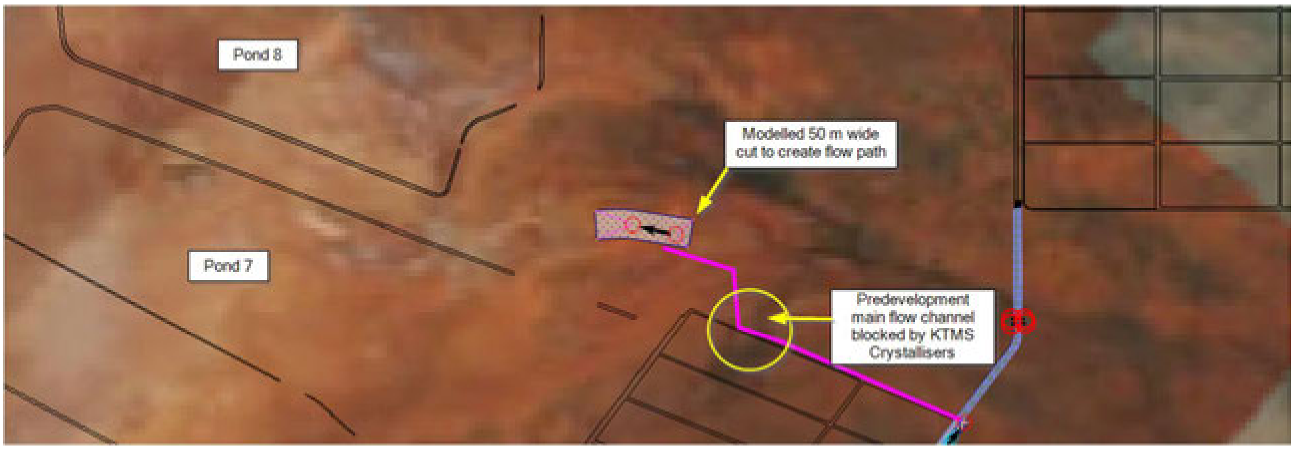



**Figure 4: KTMS Crystallisers interface with flow channel**

## 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.

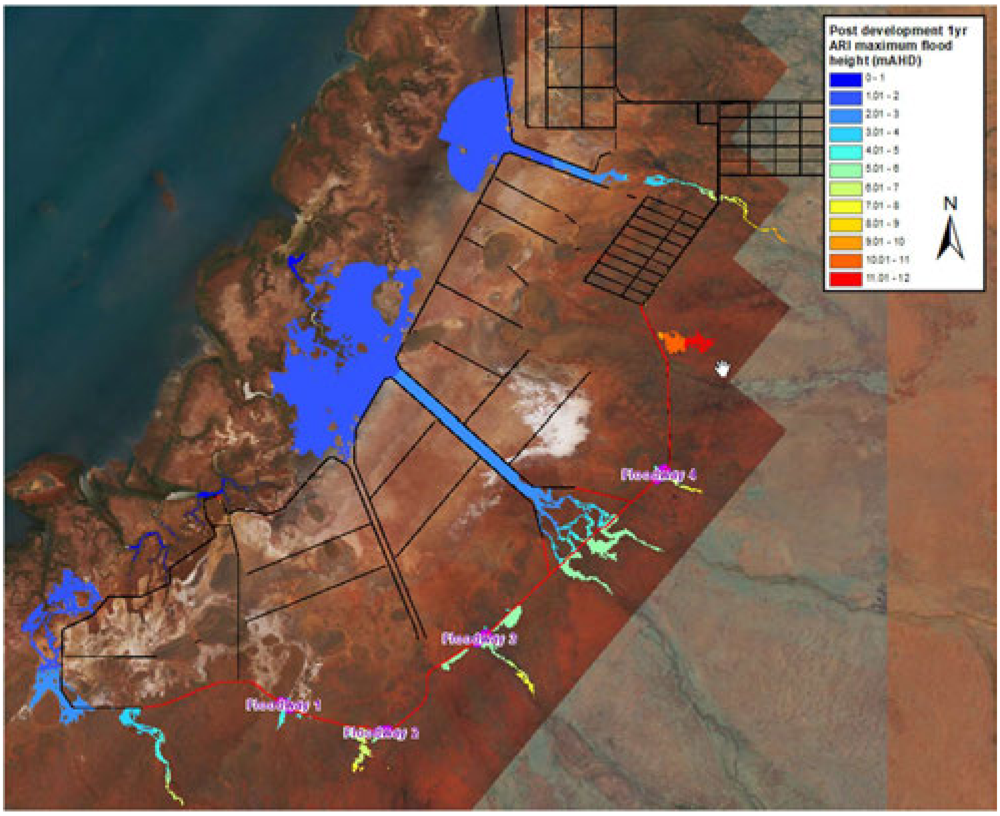



**Figure 5: Post development 1 yr ARI maximum flood height (mAHD)**

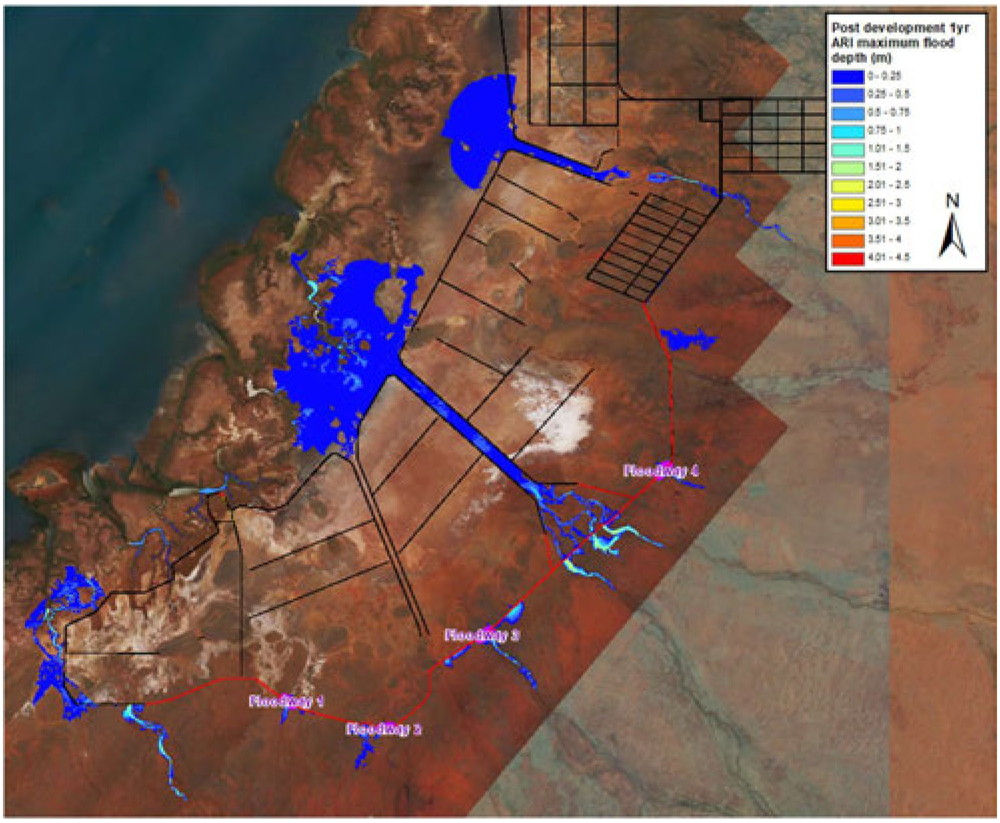



**Figure 6: Post development 1 year ARI maximum flood depth (m)**

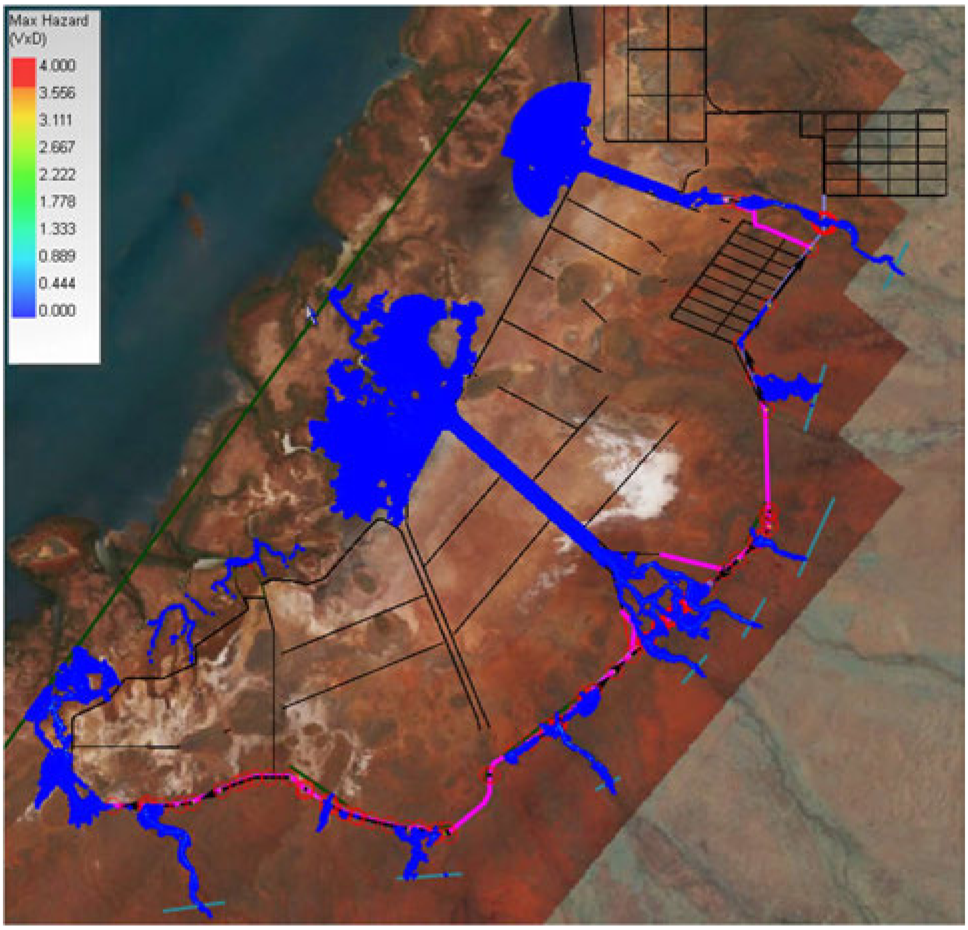



**Figure 7: Post development 1 year ARI maximum hazard (velocity x depth) ($m^2/s$)**

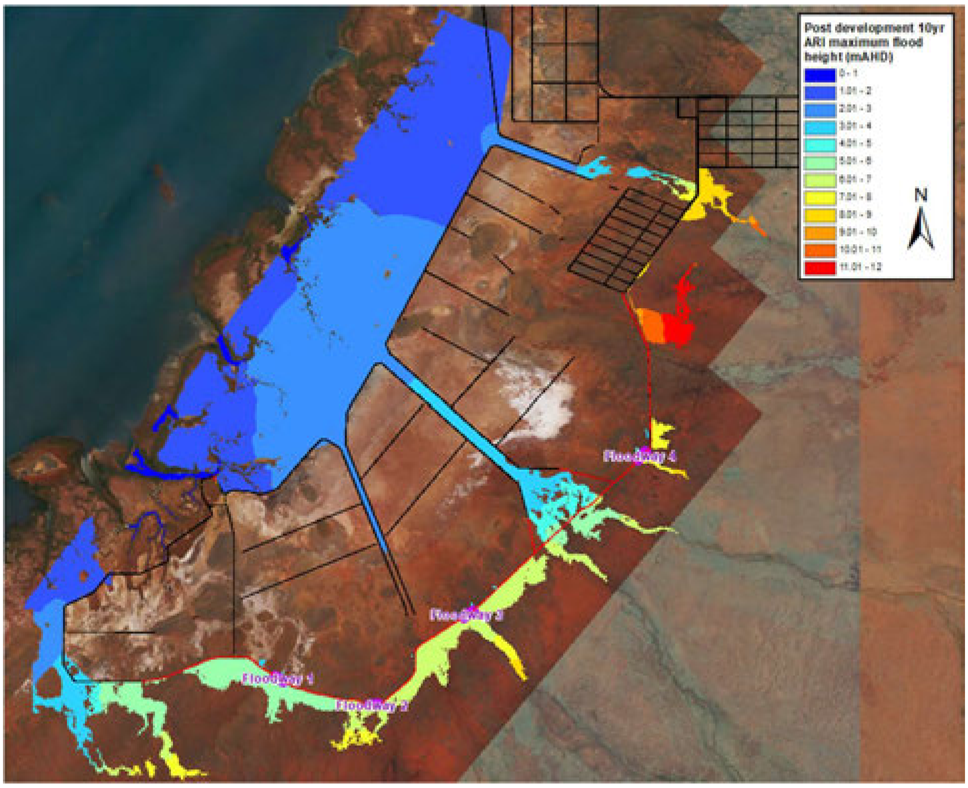



**Figure 8: Post development 10 year ARI maximum flood height (mAHD)**

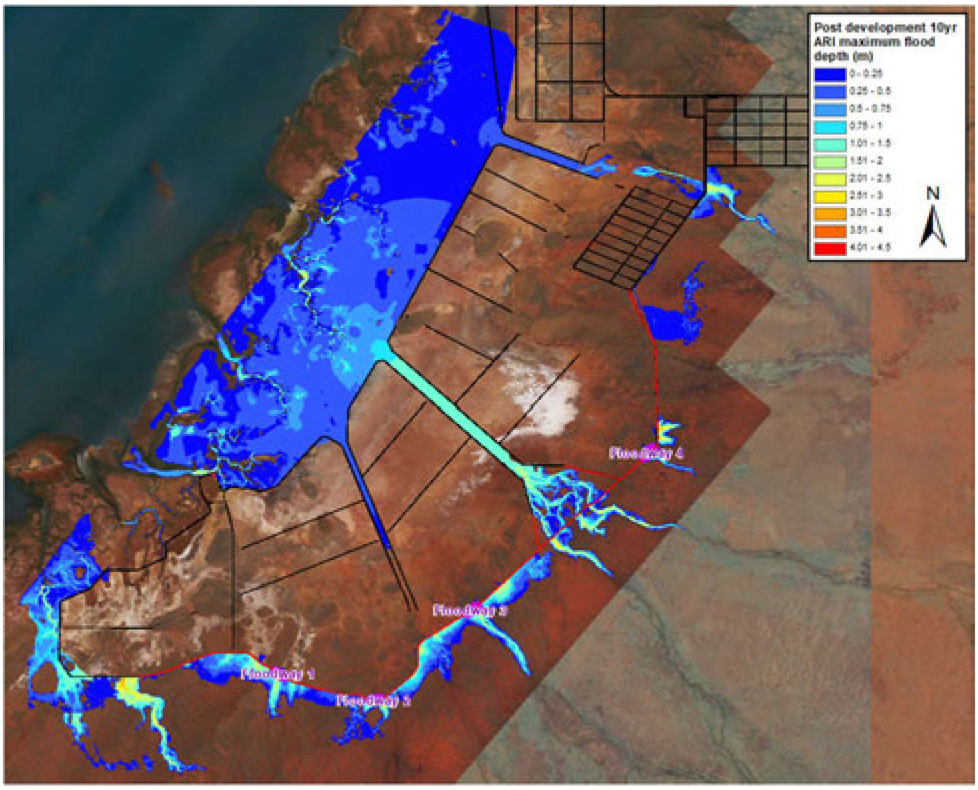



**Figure 9: Post development 10 year ARI maximum flood depth (m)**

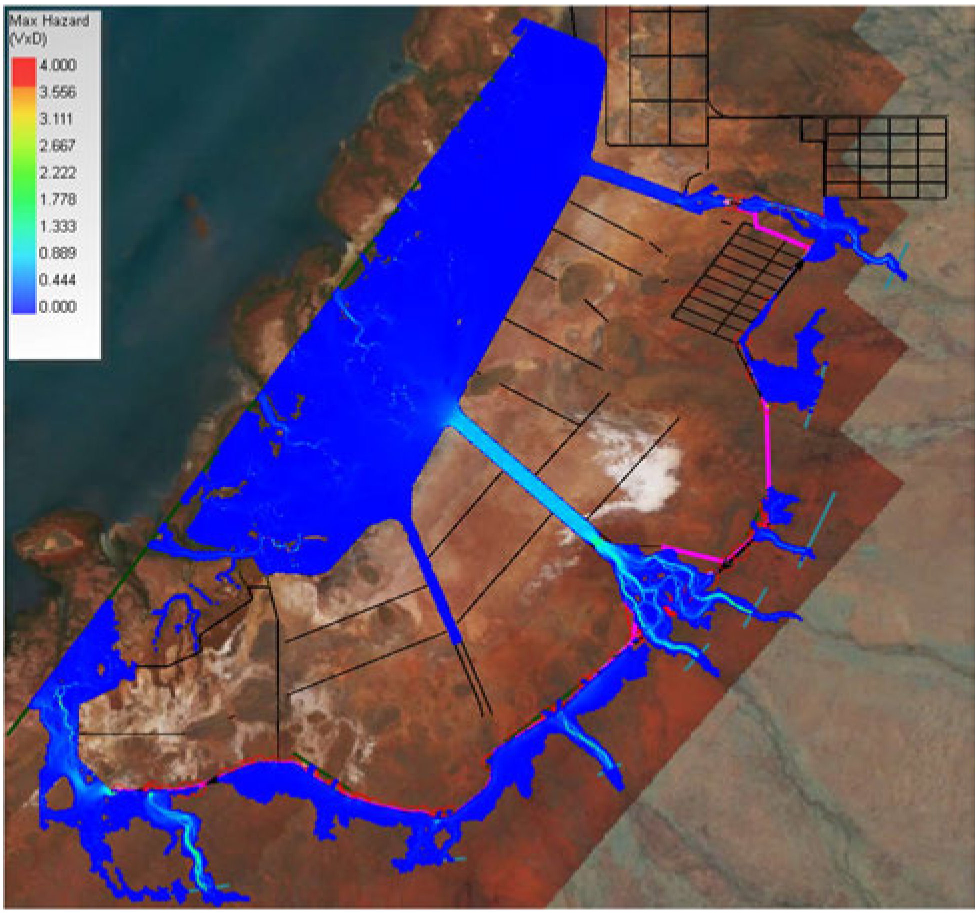



**Figure 10: Post development 10 year ARI maximum hazard (velocity x depth) ($m^2/s$)**

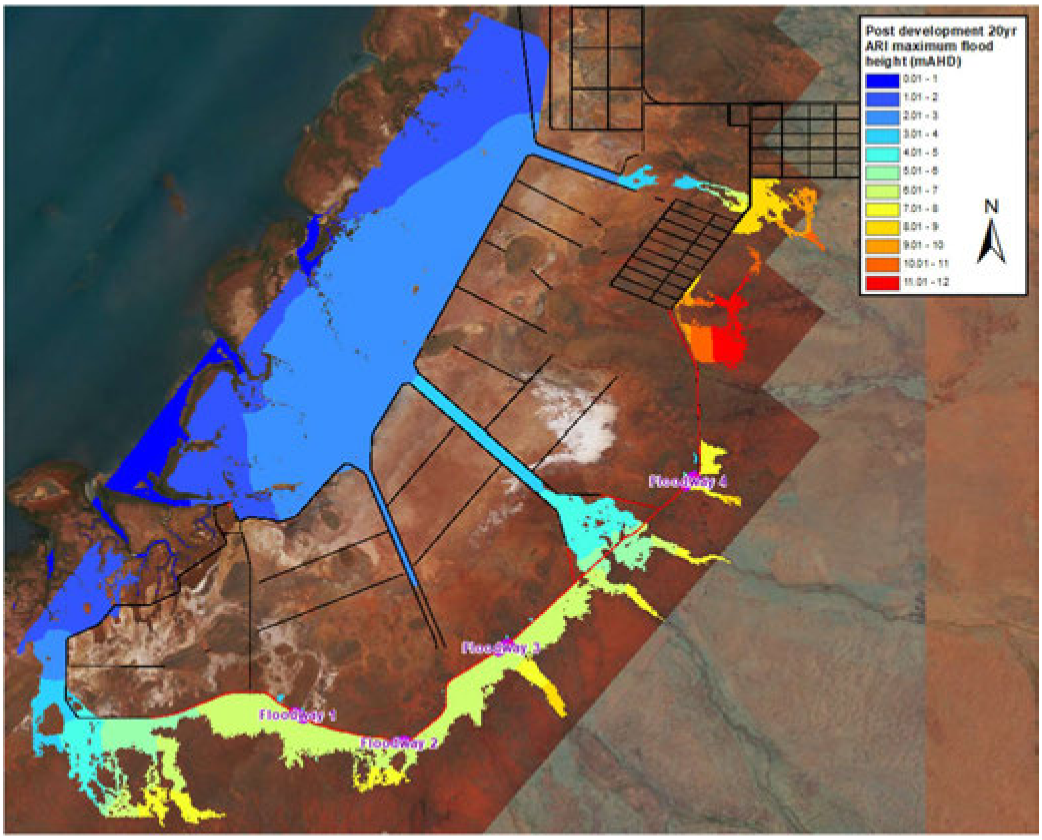



**Figure 11: Post development 20 year ARI maximum flood height (mAHD)**

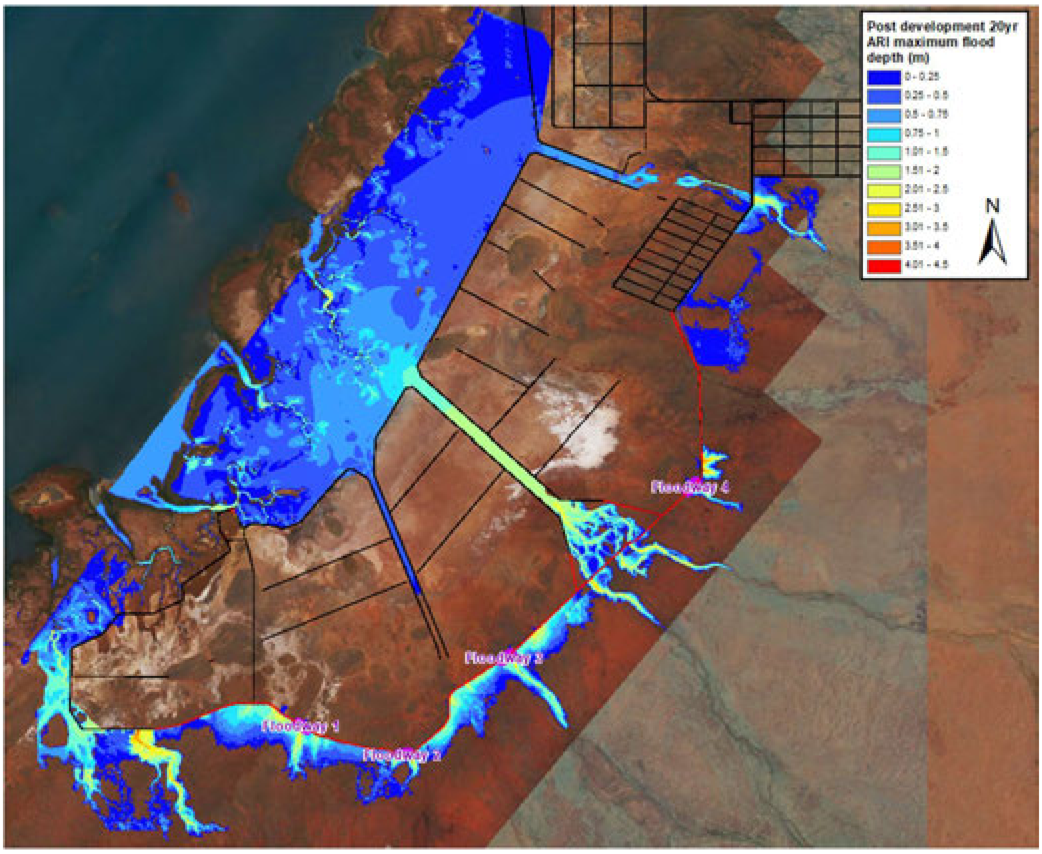



**Figure 12: Post development 20 year ARI maximum flood depth (m)**

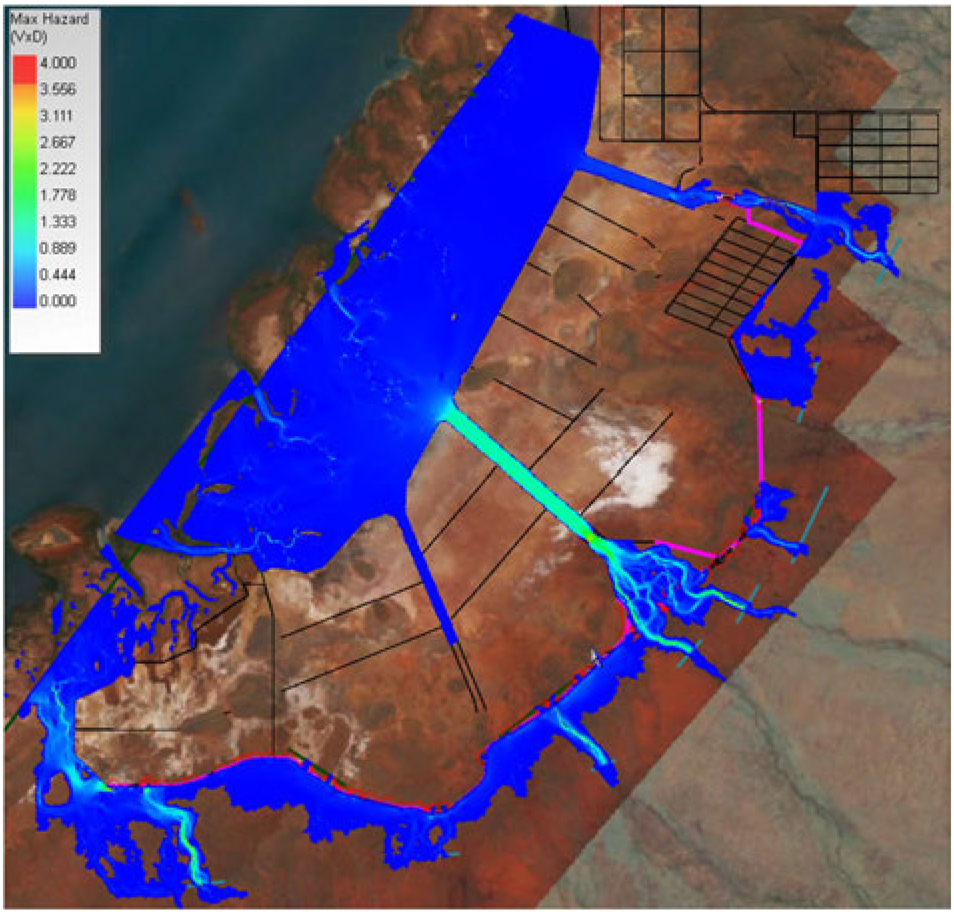



**Figure 13: Post development 20 year ARI maximum hazard (velocity x depth) ($m^2/s$)**

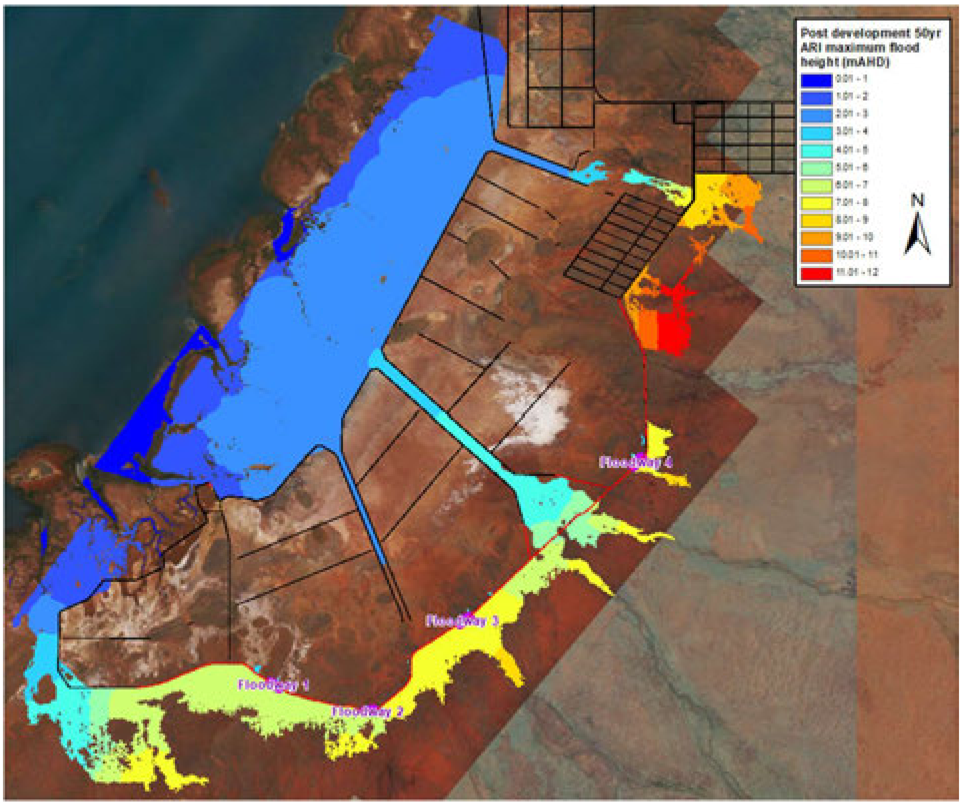



**Figure 14: Post development 50 year ARI maximum flood height (mAHD)**

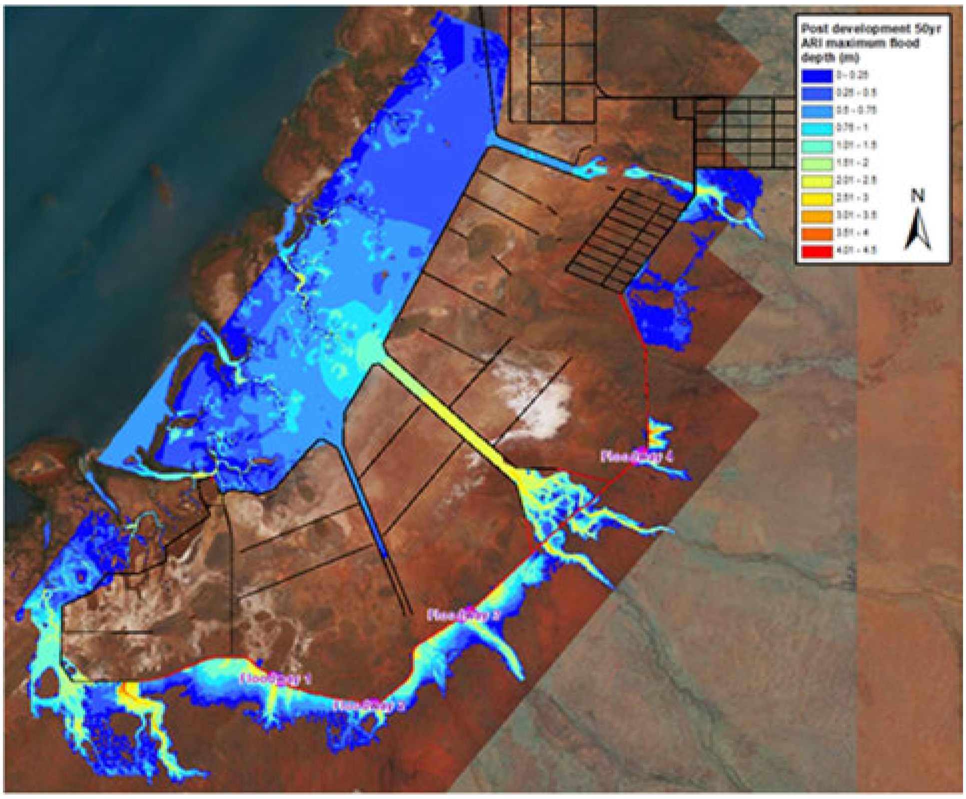



**Figure 15: Post development 50 year ARI maximum flood depth (m)**

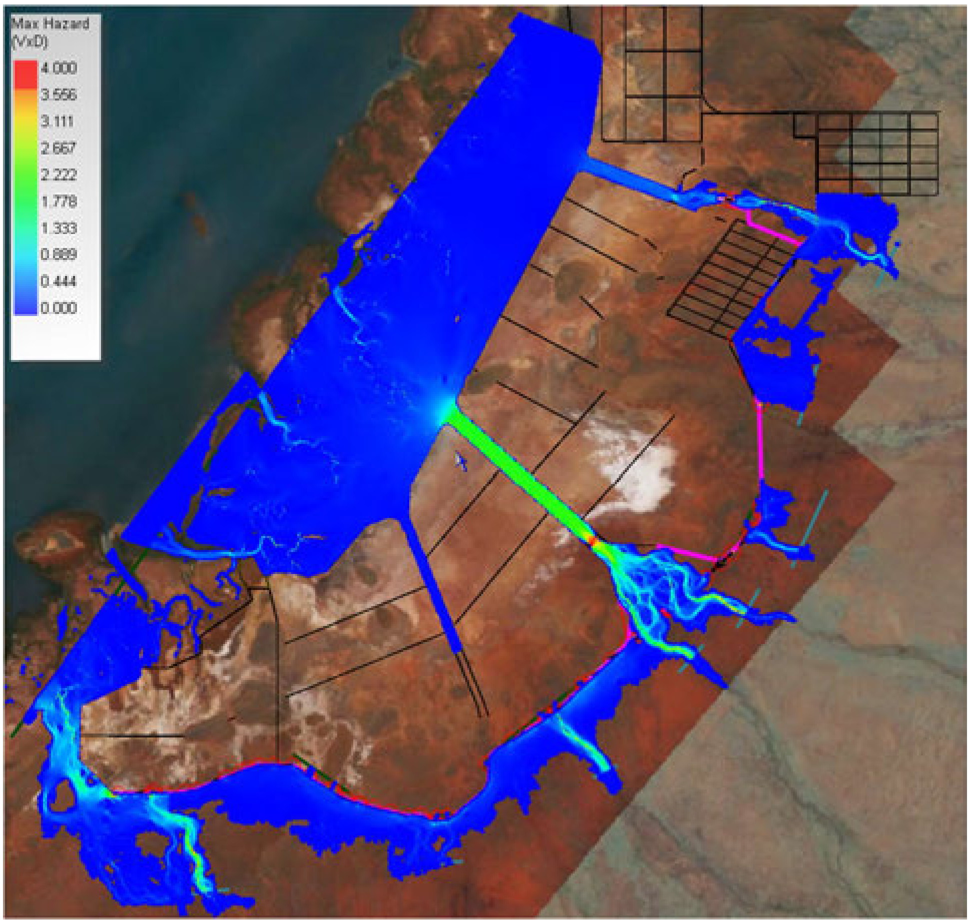



**Figure 16: Post development 50 year ARI maximum hazard (velocity x depth) ($m^2/s$)**

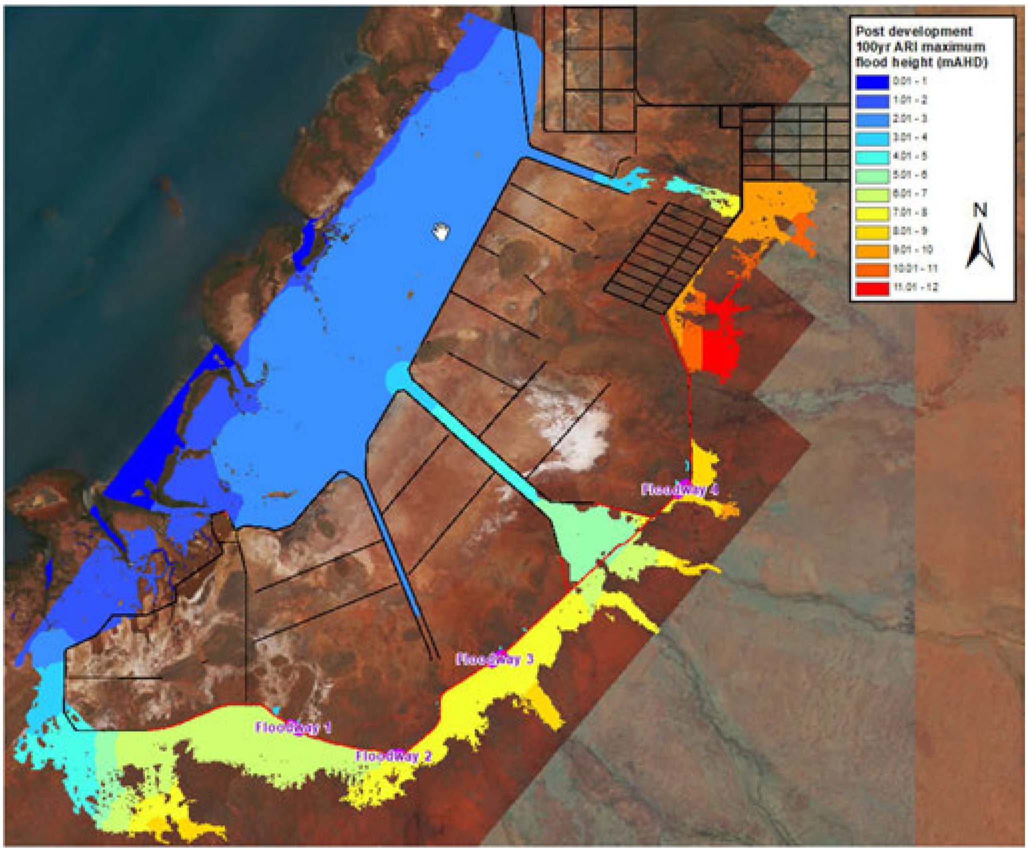



**Figure 17: Post development 100 year ARI maximum flood height (mAHD)**

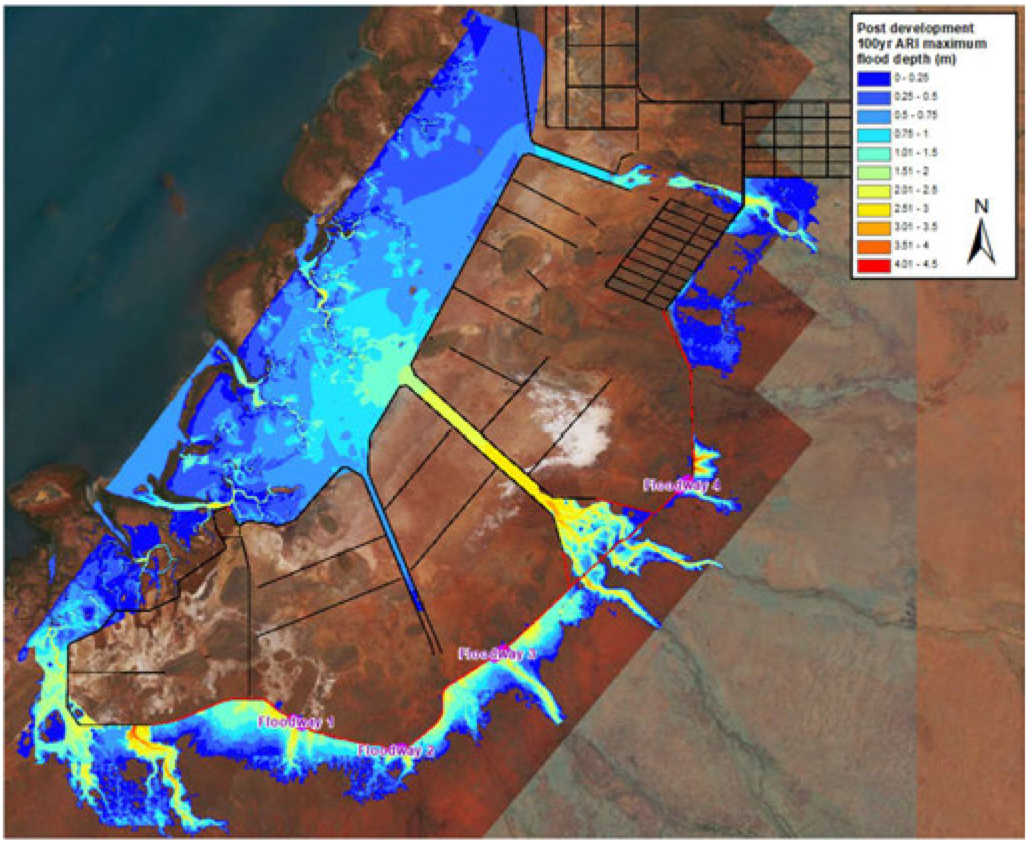



**Figure 18: Post development 100 year ARI maximum flood depth (m)**

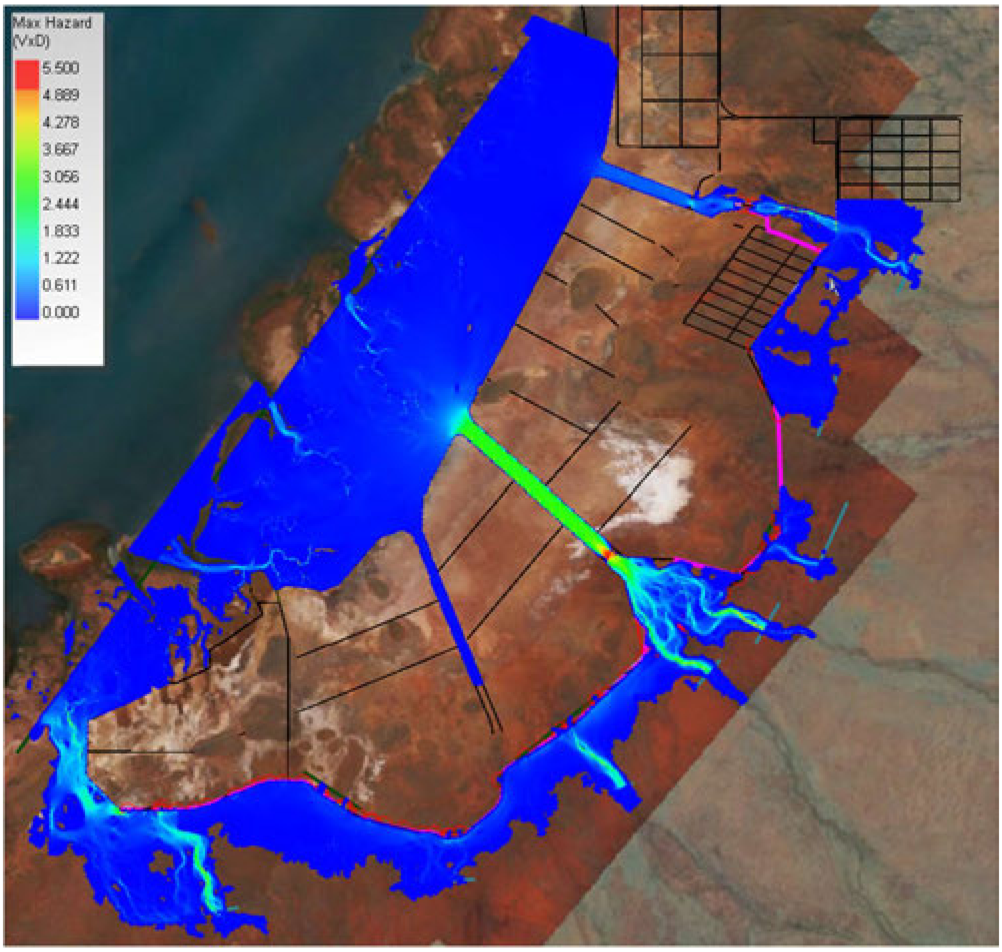



**Figure 19: Post development 100 year ARI maximum hazard (velocity x depth) ($m^2/s$)**

## 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).

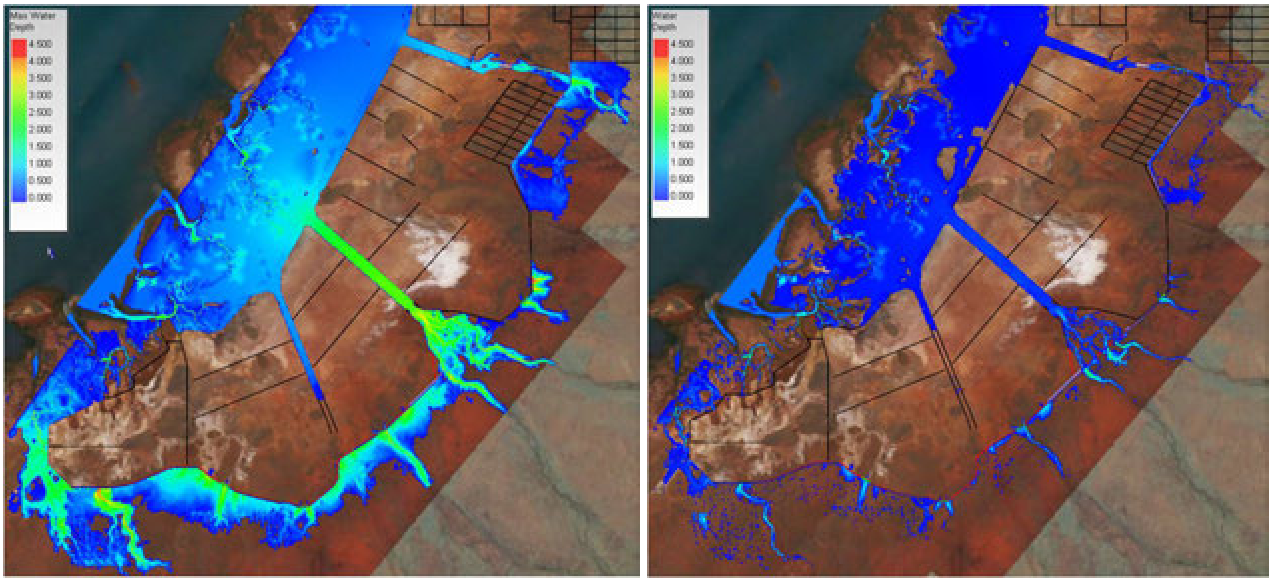


**Figure 20: Extent of inundation, during 100 year ARI storm peak (left image) and in trapped low areas following storm (right image)**

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 |

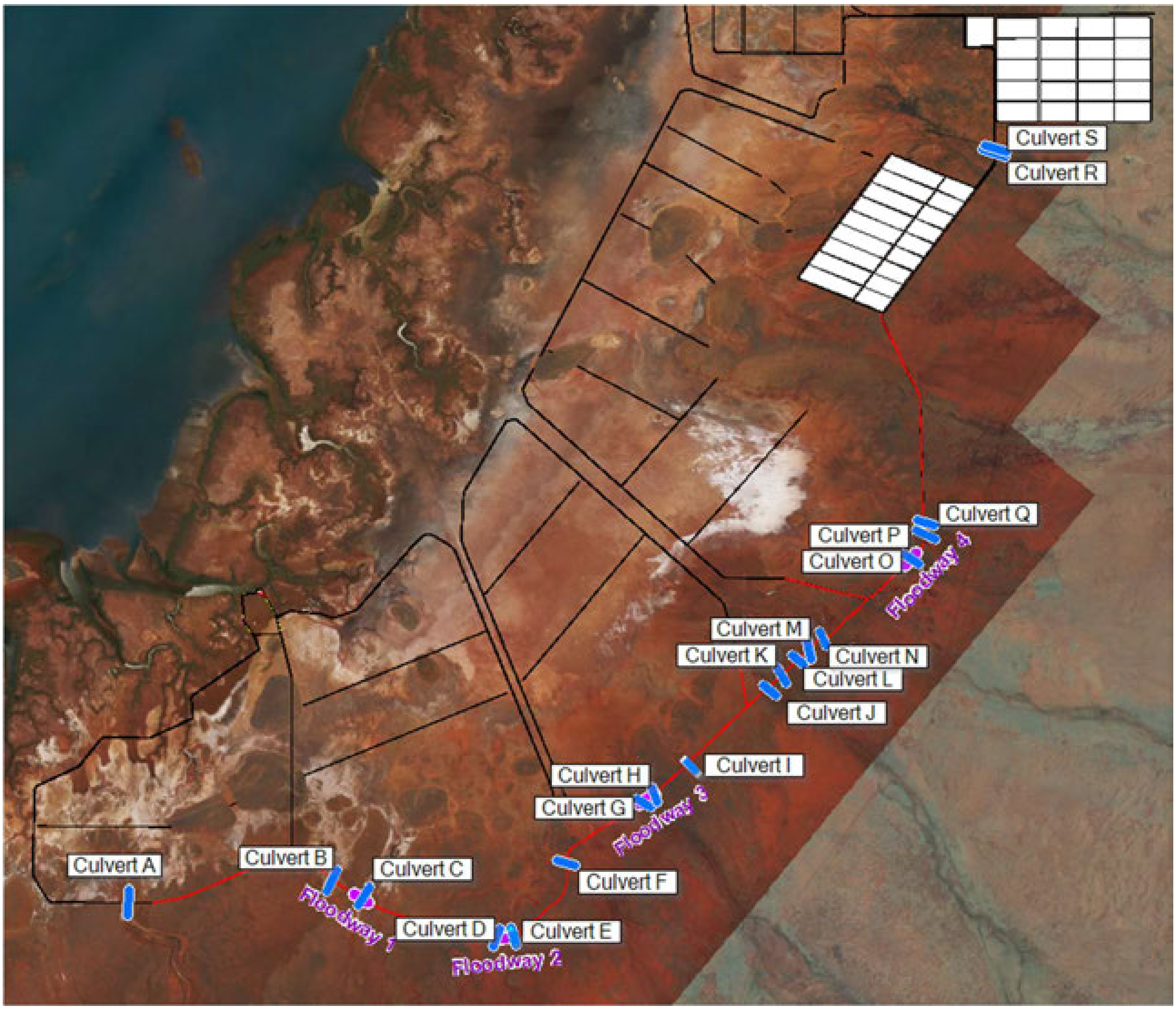



**Figure 21: Culvert locations**

## 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.

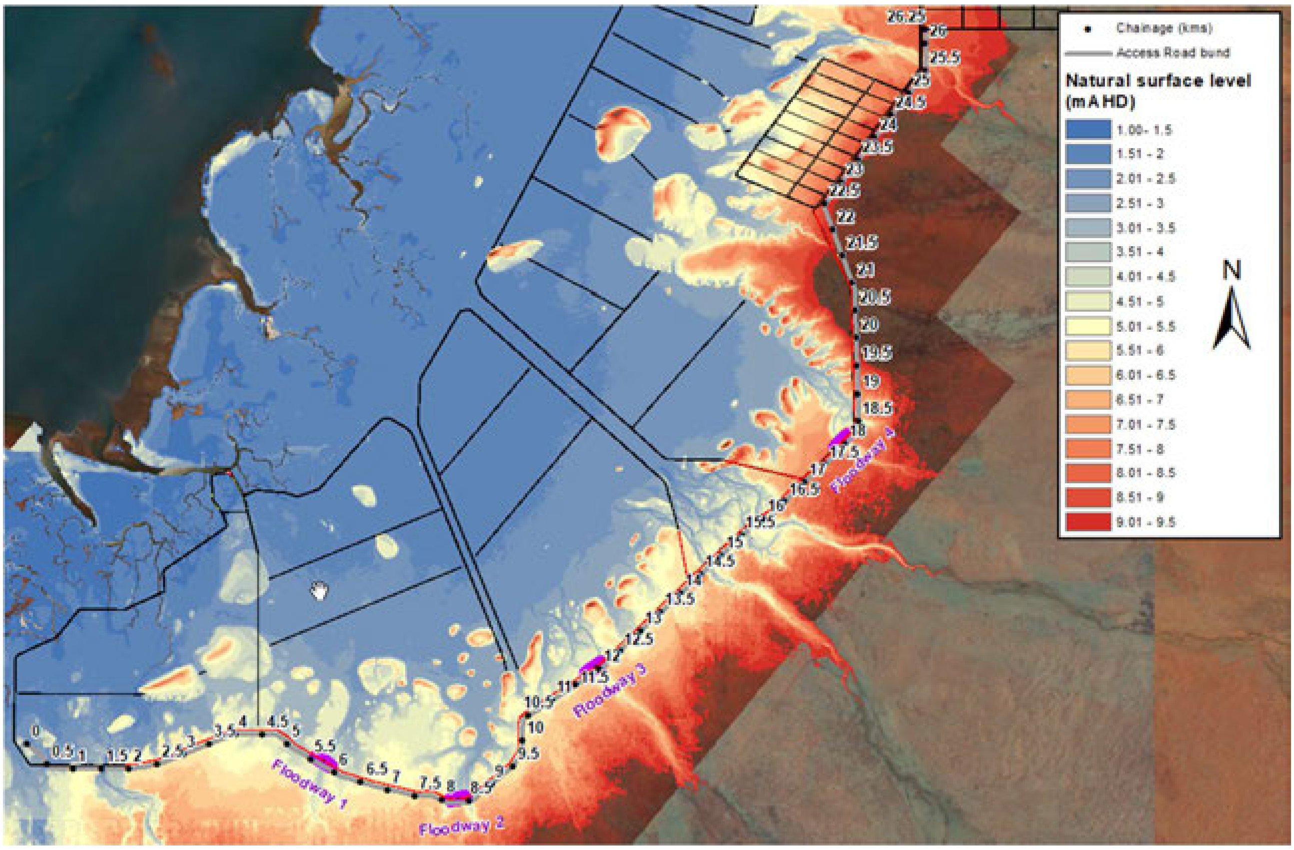
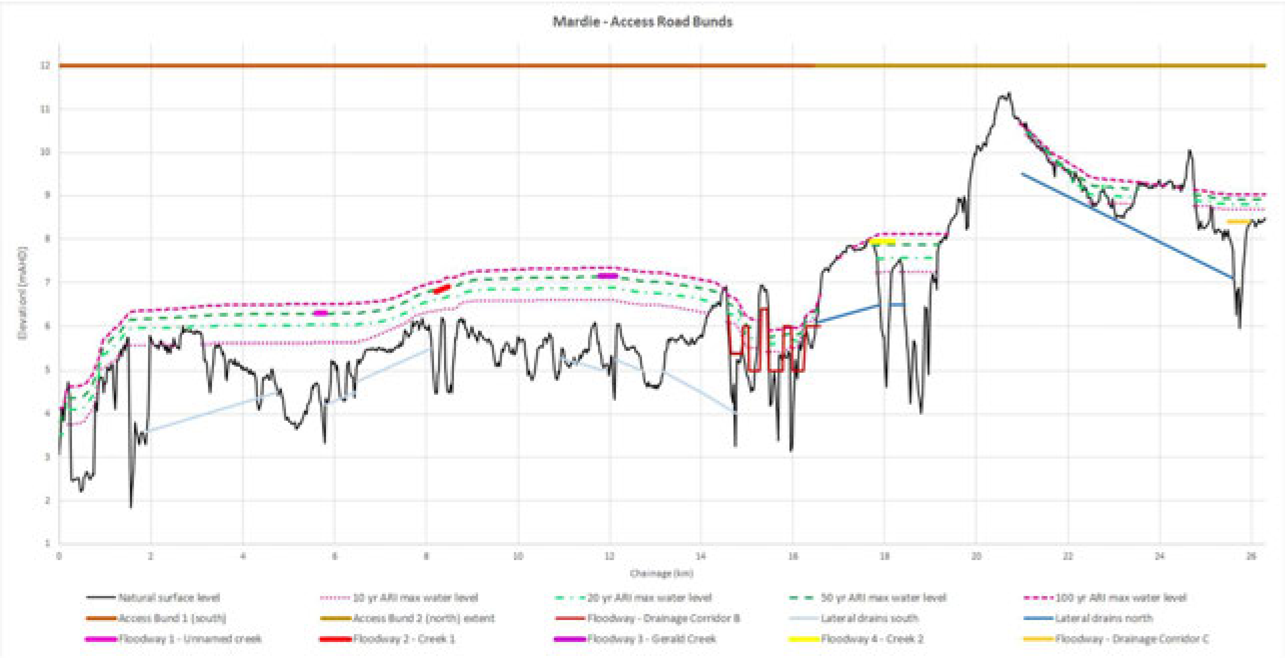



**Figure 22: Access bund 1 and 2 chainage**




**Graph 1: Access bund 1 and 2 long section**

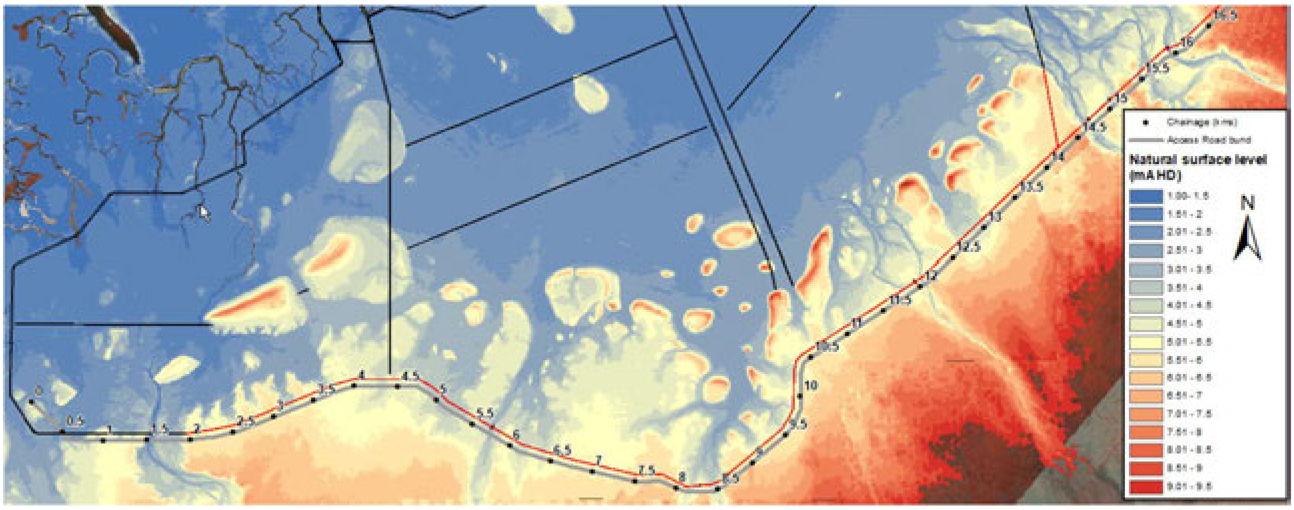
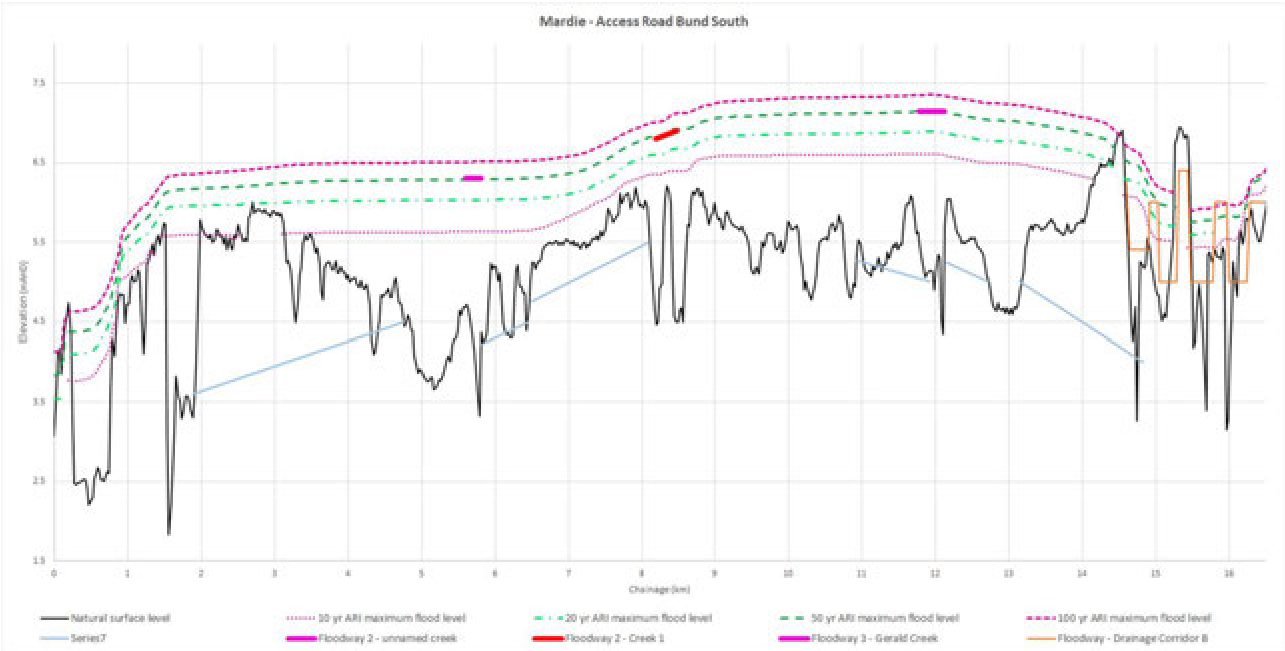



Figure 23: Access bund 1 chainage




Graph 2: Access bund 1 long section

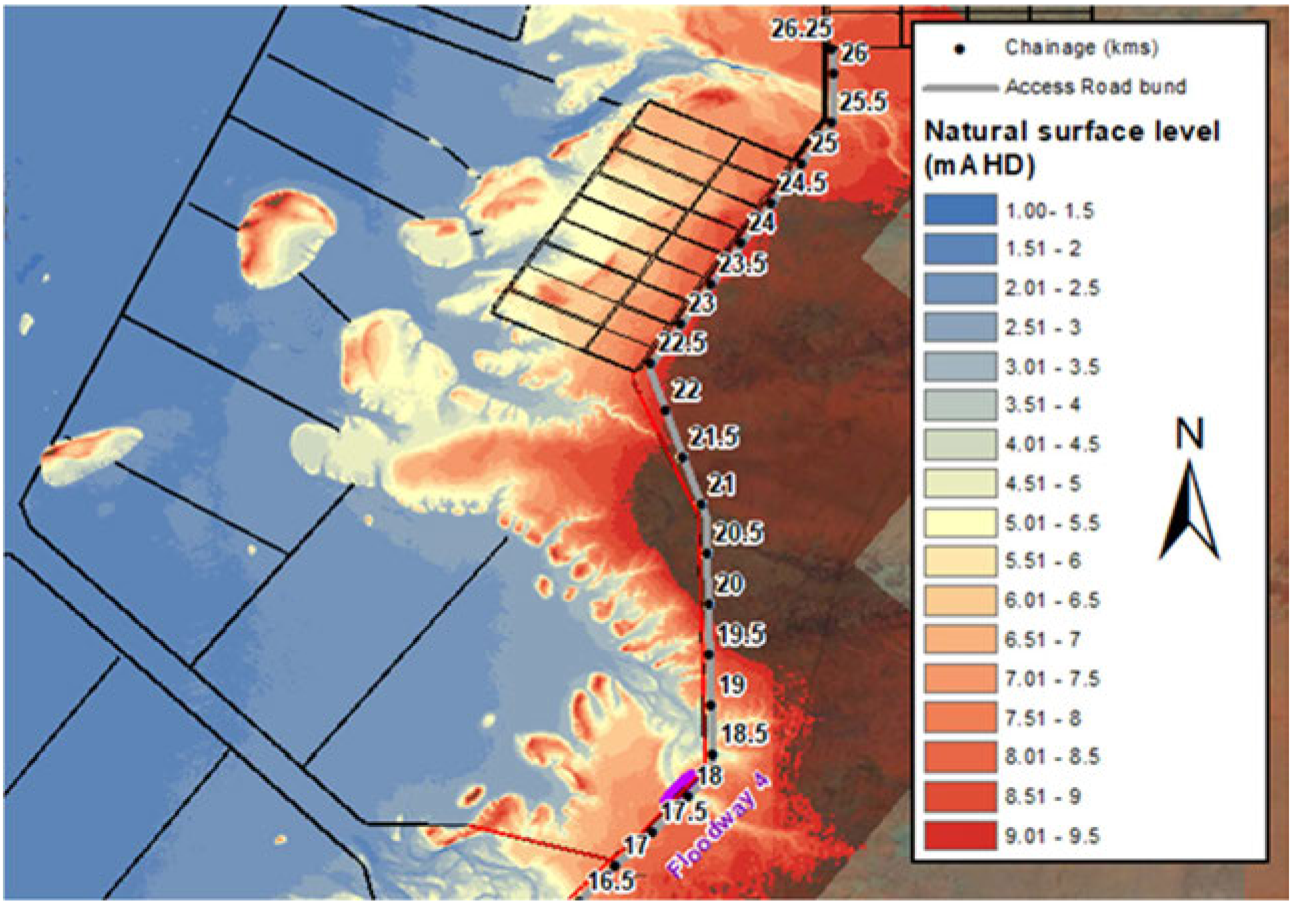
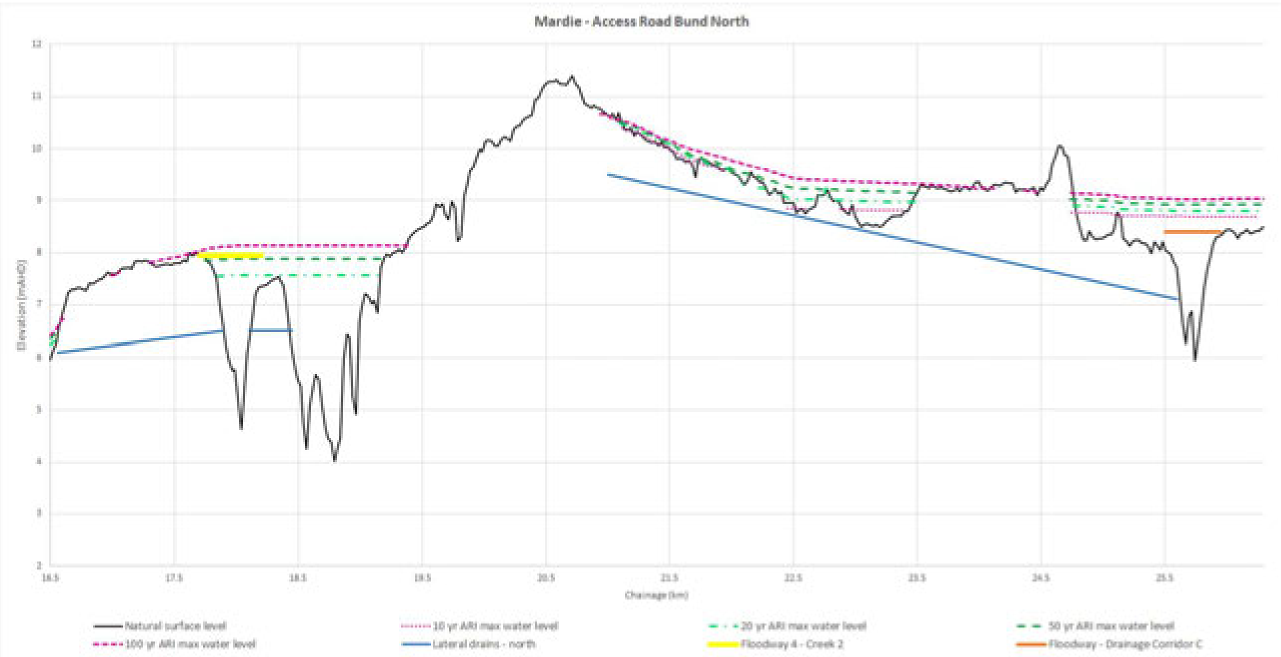



**Figure 24: Access bund 2 chainage**




**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).

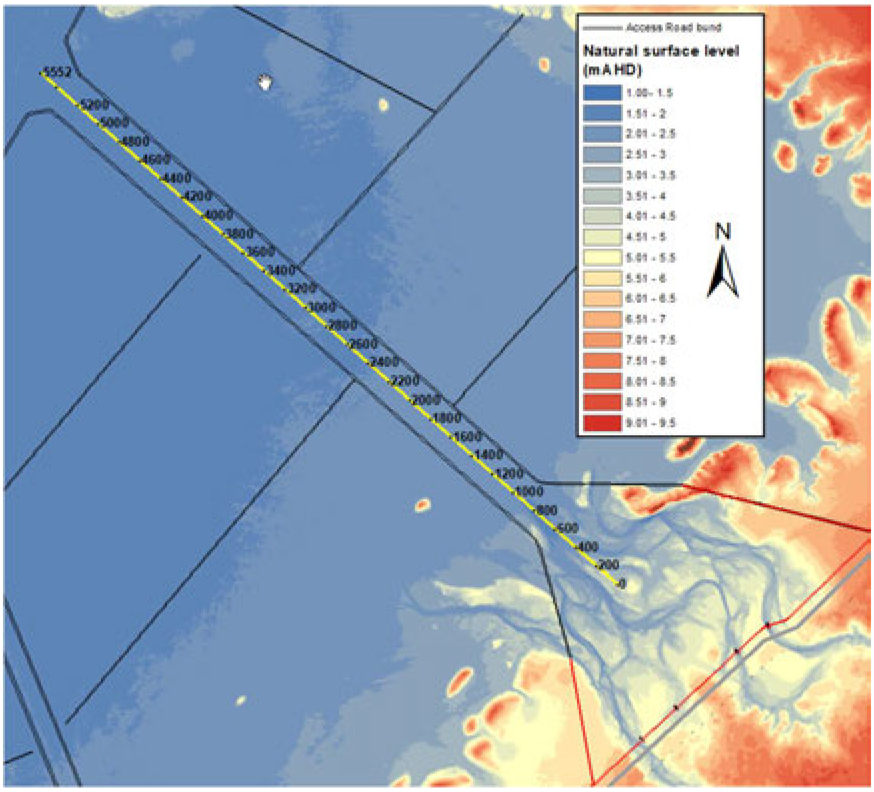
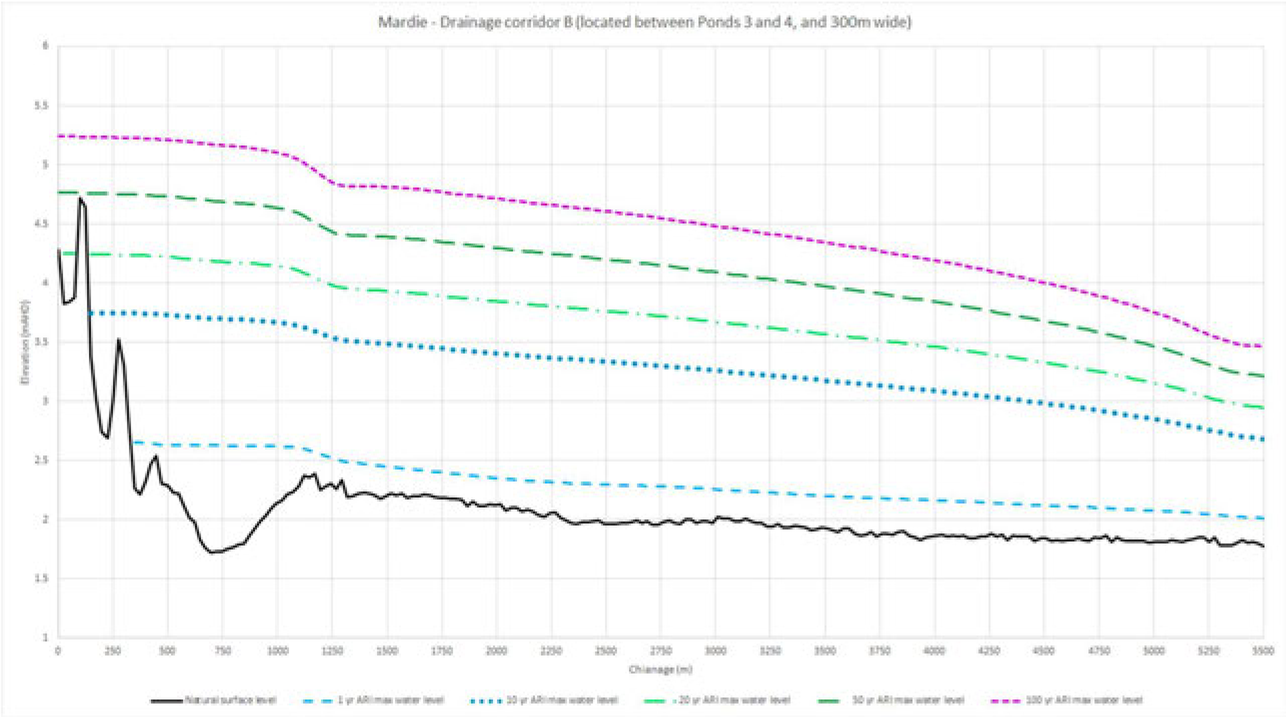



**Figure 25: Chainage (kms) along Drainage Corridor B**




**Graph 4: Flood levels within Drainage Corridor B**

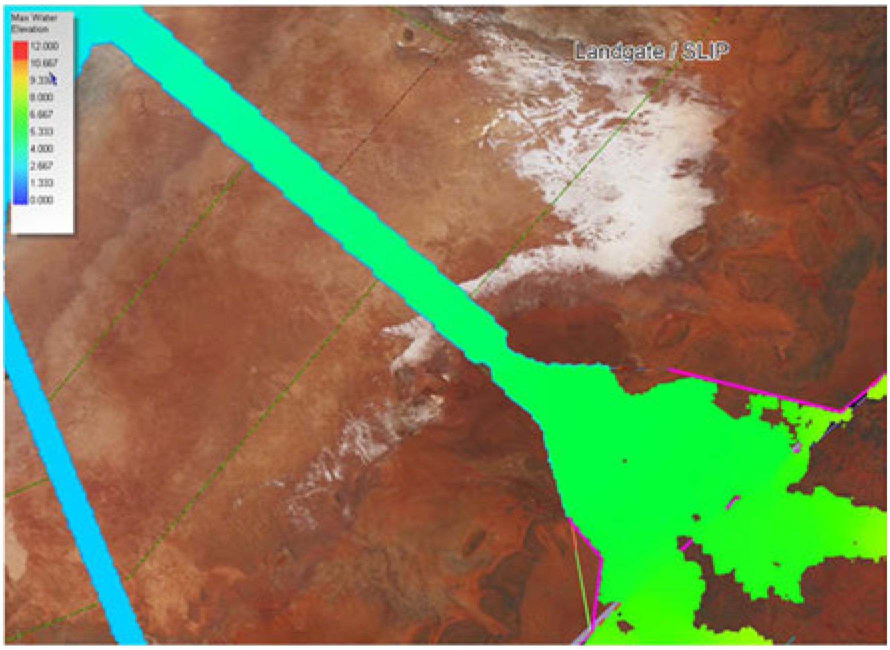



**Figure 26: Maximum flood elevation (mAHD) in the 100 year ARI storm within Drainage Corridor B**

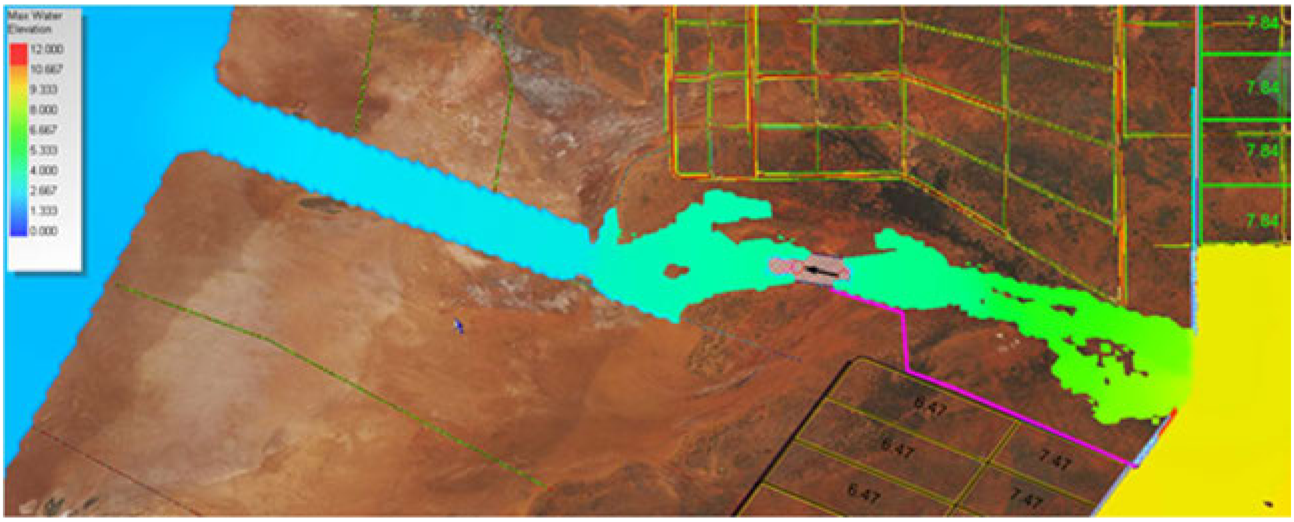



**Figure 27: Maximum flood elevation (mAHD) in the 100 year ARI storm within Drainage Corridor C**

## 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.

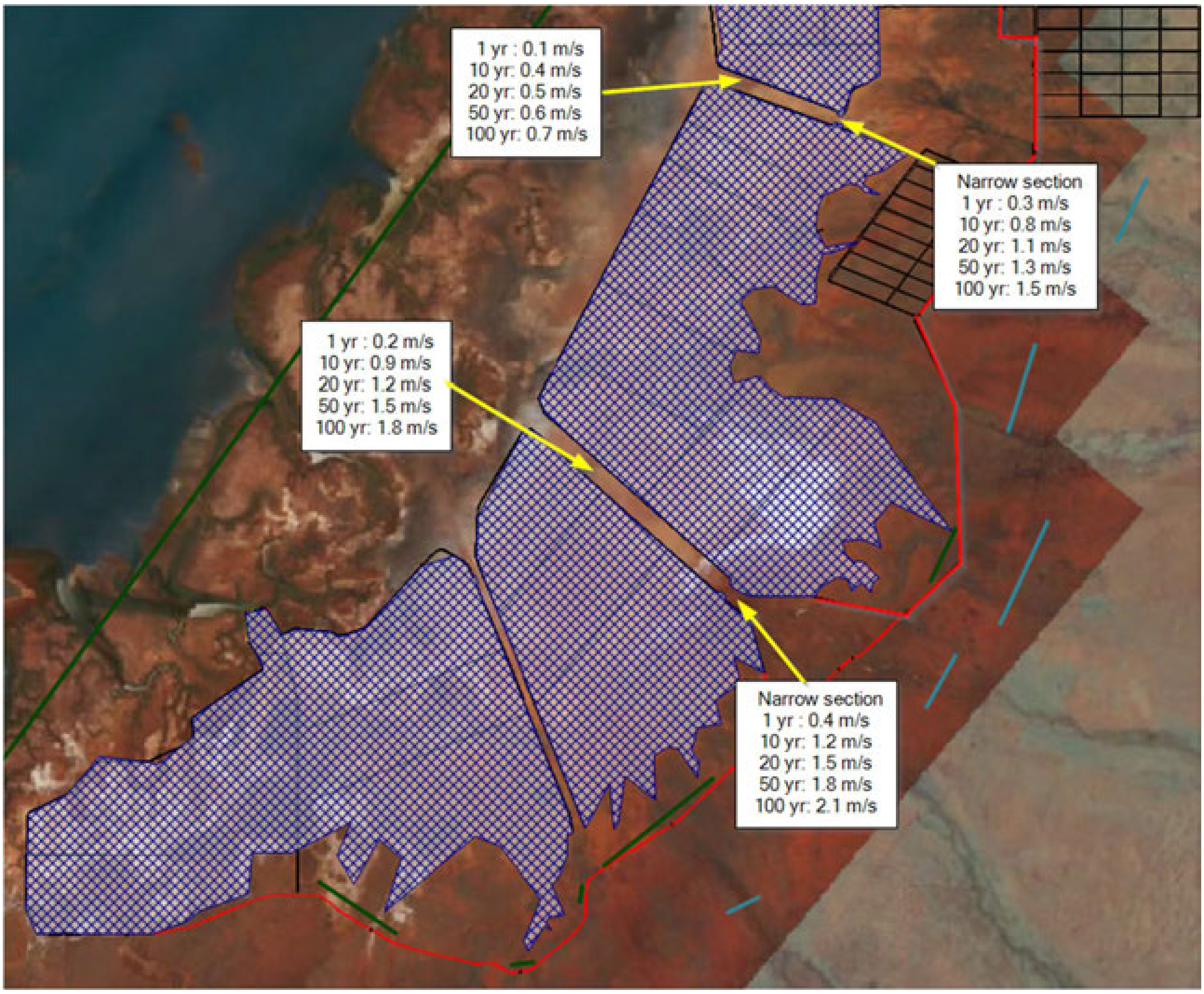



**Figure 28: Modelled peak velocities**

### 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.

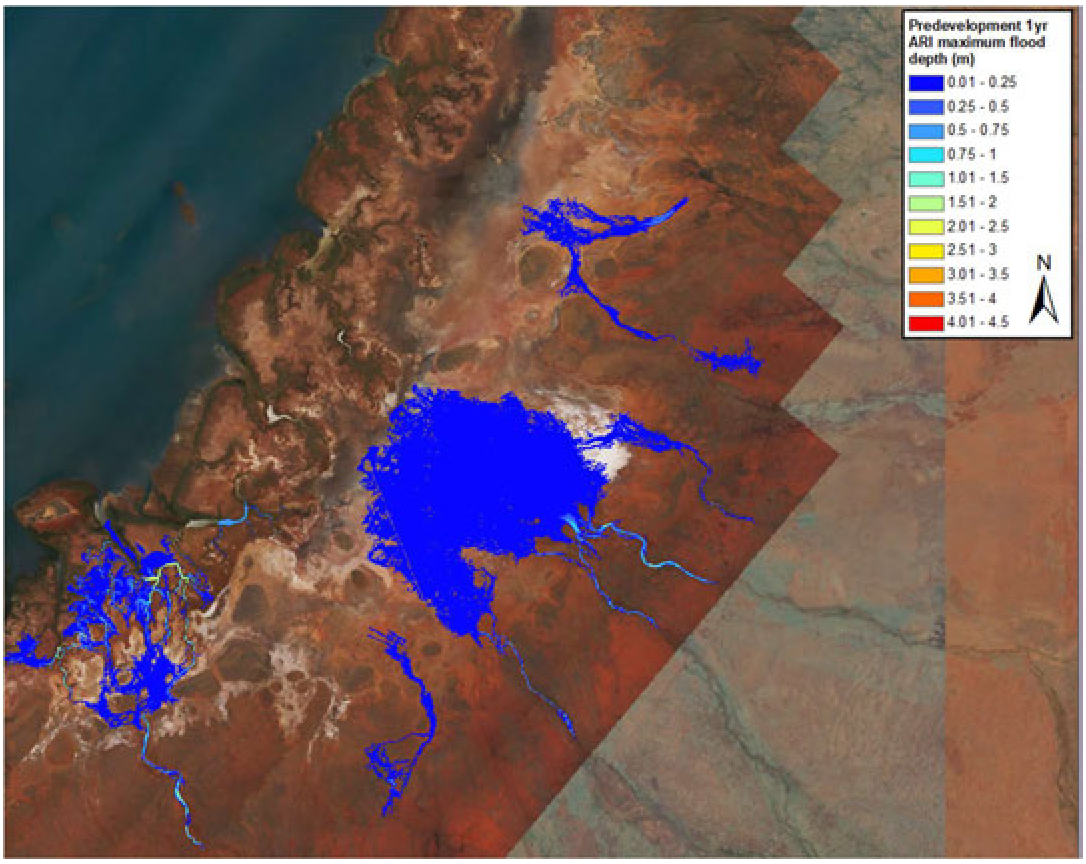



**Figure 29: Predevelopment 1 year ARI maximum flood depth (m)**

### 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 opposed 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 did 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.

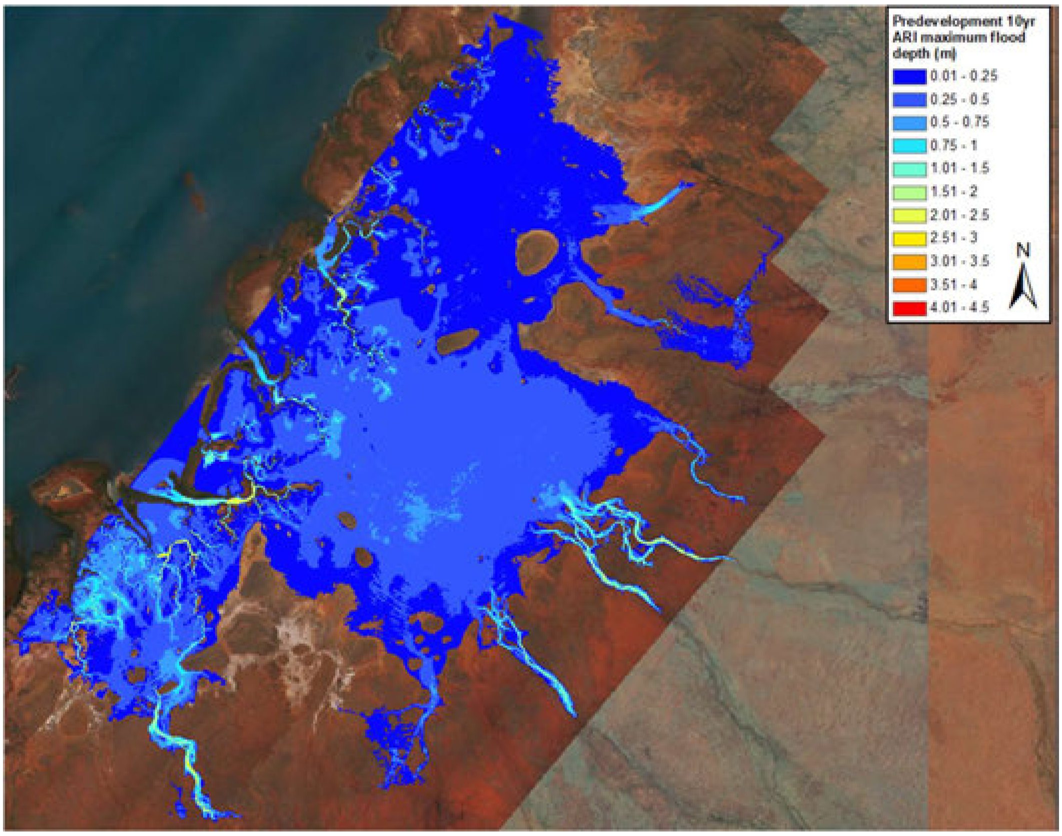



**Figure 30: Predevelopment 10 year ARI maximum flood depth (m)**

## 5.3 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;

- ~~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; and~~
- 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.

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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)

# Appendix B

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

# Appendix C

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

# Appendix D

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

# Appendix E

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