Ashburton Salt Project

Marine and Coastal Assessment and Modelling

K+S Salt Australia Pty Ltd

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CONTENTS

1 INTRODUCTION 7

2 HYDRODYNAMIC MODELLING PACKAGE 9
2.1 Modelling Package 9
2.1.1 Model Mesh 9
2.2 Model Inputs and Parametrisation 10
2.2.1 Bathymetry 10
2.2.2 Development Layout and Model Implementation 16
2.2.3 Eddy Viscosity/diffusivity 21
2.2.4 Bed Roughness 21
2.3 Boundary Conditions 22
2.3.1 Water Levels and Currents 22
2.3.2 Water Temperature and Salinity 22
2.3.3 Meteorological Inputs 22
2.4 Model Calibration 24
2.4.1 Water Levels 24
2.4.2 Temperature and Salinity 31
2.5 Currents 32
2.6 Limitations and Assumptions 32

3 COASTAL PROCESSES MODELLING 35
3.1 Hydrodynamic Module 35
3.2 Sand Transport Module 36
3.3 Model Results 36
3.3.1 Modelled Changes to Inundation of Overall Project Layout 37
3.3.2 Modelled Hydrodynamic Changes to Urala Creek South 45
3.3.3 Modelled Hydrodynamic Changes at Locker Point (Jetty Site) 57
3.4 Key Findings 60
3.4.1 Pre- and Post-development Submergence Curves in Urala Creek South 60
3.4.2 Disturbance to Tidal Discharges in Urala Creek South 61
3.4.3 Modification to Tidal Inundation 63
3.4.4 Impacts to Urala Creek South Morphology 69
3.4.5 Impacts to Shoreline at Locker Point 69

4 BITTERNS DISCHARGE MODELLING 73
4.1 Study Approach 73
4.2 Bitterns Characterisation 74
4.2.1 Bitterns Generation 74
4.2.2 Laboratory Testing 74
4.2.3 Bitterns Discharge Characteristics 76
4.3 Setting of Environmental Quality Criteria 78
4.3.1 Regulatory Framework: Physical and Chemical Stressors 78
4.3.2 Regulatory Framework: Toxicants 79
4.3.3 Approach for this Project 79

5196-10_R04_v04_Coastal_&_Marine_Modelling_EPAUpdates.docx

4.3.4 Salinity – Proposed Interim EQGs 79
4.3.5 Metals (Toxicants) – Proposed Interim EQGs 81
4.3.6 EPA Guidelines for Low and Moderate LEP Zone Sizes 82
4.4 Receiving Water Characteristics 82
4.4.1 Water Depths 82
4.4.2 Currents 83
4.4.3 Ambient Salinity 83
4.5 Diffuser Design Approach 83
4.5.1 Near-Field Approach 84
4.5.2 Far-Field Approach 84
4.6 Near Field Modelling 84
4.6.1 Design Parameters 85
4.6.2 Assimilative Capacity 86
4.6.3 Preliminary Assessment 86
4.6.4 Summary of Considerations 88
4.6.5 Nearfield Modelling System 88
4.6.6 Near Field Modelling Results 88
4.6.7 Diffuser Design Concept 91
4.6.8 Diffuser Layout Optimisation 92
4.7 Bitterns Discharge Far-Field Modelling 92
4.7.1 Far-field Modelling System 92
4.7.2 MIKE 3 FM Model Settings 93
4.7.3 Monthly Bitterns Discharge 95
4.7.4 Model Scenarios 96
4.7.5 Salinity Results 98
4.7.6 Toxicant (Metal) Results 107
4.7.7 Dilution Analysis 109
4.8 Summary 113

5 DREDGING PLUME MODELLING 115
5.1 Introduction 115
5.2 Dredge Conceptualisation 116
5.2.1 Sediment Composition 116
5.2.2 Dredging Volume 117
5.2.3 Dredging Method 118
5.2.4 Sediment Release 118
5.3 Modelling Package 119
5.3.1 Hydrodynamic Model 119
5.3.2 Mud Transport Module 119
5.4 Model Setup 119
5.4.1 General Settings 119
5.4.2 Background TSS 119
5.4.3 Representative Seasonal Scenarios 120
5.5 Model Results 121
5.5.1 Impact Zone Scheme and Results Analysis 121

5.5.2 Dredging Plume Results 122
5.5.3 Sediment Deposition Results 128
5.5.4 Duration of Plume 129
5.5.5 Maintenance Dredging 129
5.6 Summary and Discussion 130

6 REFERENCES 131

LIST OF FIGURES

Figure 1-1 Project Location 8
Figure 1-2 Proposed Project Layout 8
Figure 2-1 Regional Model Mesh 9
Figure 2-2 Local Model Mesh 10
Figure 2-3 Bathymetry data sources and extents 11
Figure 2-4 LiDAR banding and ridges in Salt Flats 13
Figure 2-5 Tidal Inundation (Colour denotes water level) Modelled - 15 April 2020 15
Figure 2-6 Sentinel Satellite Image - 15 April 2020 16
Figure 2-7 Intake facilities (upper panel) and embankments Pile Supported Jetty and conveyor (lower panel) 17
Figure 2-8 Existing (left) and developed (right) model mesh 19
Figure 2-9 Intake Design (Source: Vortex Australia, December 2020) 20
Figure 2-10 Outfall configuration 21
Figure 2-11 Modelled and Measured Water Levels at Locker Point (Proposed bitterns discharge location) 25
Figure 2-12 Modelled and Measured Water Levels at Urala Creek south (Intake – upper panel) and Locker Point (discharge – lower panel) 26
Figure 2-13 Seasonal Water Level simulated at Locker Point in 2015 27
Figure 2-14 Seasonal Water Level from Onslow Tidal records (orange box highligts model period) 27
Figure 2-15 Tidal Inundation Frequency (Mar-Apr) 28
Figure 2-16 Tidal Inundation Frequency (Aug-Sep) 28
Figure 2-17 Tidal Inundation (COlor denotes water level) Modelled - 15 April 2020 29
Figure 2-18 Sentinel Satellite Image - 15 April 2020 30
Figure 2-19 Modelled and measured Temperature & salinity 31
Figure 2-20 Modelled current magnitude in Urala Creek North 33
Figure 2-21 Cumulative percentile distributions in north Exmouth Gulf: meaured (GHD, 2018), and modelled (Water Technology) 33
Figure 3-1 Model Framework 36
Figure 3-2 Sand Transport Model over 1 year - Existing Pre-development scenario 55
Figure 3-3 Sand Transport Model over 1 year – Post-Development scenario 56
Figure 3-4 1-year Sand Transport Model Result - existing & post-development Difference 56
Figure 3-5 submergence curves at Urala creek south pre- and post-development 2015 61
Figure 3-6 Tidal discharge extraction locations and current speeds during a flood tide 62
Figure 3-7 Tidal discharges at q1, q2 AND q3 locations 63
Figure 3-8 Tidal Inundation Time (Percentage of time) in Year 2015 - Existing Conditions 65
Figure 3-9 Tidal Inundation Time (Percentage of time) in Year 2015 – Post-Development SCENARIO 66
Figure 3-10 Difference of Tidal Inundation Time due to project development in Year 2015 67

Figure 3-11 Difference of Tidal Inundation Time due to projet development in Year 2015 (near the intake channel embankment) 68
Figure 3-12 Proposed jetty abutment (extract from K+S draft design drawing RP21032-0000-MA-DG-0006) 71
Figure 3-13 Coastal erosion hazard zone for 50-year design life, as per SPP2.6 methodology 72
Figure 4-1 Project layout and jetty 73
Figure 4-2 Salinity Statistics at Locker Point Based on Monthly Sonde Measurements 80
Figure 4-3 Existing seabed Elevations Near The Diffuser (m MSL) 83
Figure 4-4 Indicative Plot Of Bitterns Jet 85
Figure 4-5 Dilution Rate Estimated By Different Concept Options 90
Figure 4-6 Maximum Jet Height Predicted By Various Concept Options 91
Figure 4-7 Jetty and DIffuser Layout Optimization 92
Figure 4-8 Water Quality Modelling Framework 93
Figure 4-9 Jetty And Diffuser Layouts 94
Figure 4-10 Measured Solar Radiation, Air Temperature And Relative Humidity At Onslow Airport 97
Figure 4-11 Modelled LEPA and MEPA Zones (Yearly Average, about 58% maximal discharge) 105
Figure 4-12 Modelled LEPA and MEPA Zones (November, Worse Case, Maximal Discharge) 105
Figure 4-13 Modelled lepa and Mepa Zones (June, Best Case, Minimal Discharge (about 15% maximal discharge)) 106
Figure 4-14 November 2015 Dilution Contour for chromium (38:1 dilution) 108
Figure 4-15 Predicted 50th percentile dilution at the surface (left) and bottom (right) 111
Figure 4-16 Predicted 80th percentile dilution at the surface (left) and bottom (right) 111
Figure 4-17 Predicted 90th percentile dilution at the surface (left) and bottom (right) 112
Figure 4-18 Predicted 99th percentile dilution at the surface (left) and bottom (right) 112
Figure 4-19 Predicted 80th percentile dilution at the surface (left) and bottom (right) for the month of June 113
Figure 4-20 Predicted 80th percentile dilution at the surface (left) and bottom (right) for the month of November 113
Figure 5-1 Proposed Berth Pocket 115
Figure 5-2 PSD diagram of Sediments sampled at Sed03 location (shown as red star) 117
Figure 5-3 Timeseries of Wind Speed and Direction at Onslow for 2010 (top), and 2015 (bottom). model periods are highlighted by red 121

LIST OF TABLES

Table 2-1 June August Calibration statistics at Locker Point 14
Table 2-2 Model Implementation (existing versus development) 18
Table 2-3 Roughness Values 21
Table 2-4 Selected TC events within 5% of the indicated Tide+Surge WL 23
Table 2-5 March/April Calibration statistics at Locker Point 24
Table 2-6 June August Calibration statistics at Locker Point 25
Table 2-7 Tidal Constituents Comparison 26
Table 3-1 Model scenarios 35
Table 3-2 Modelled Inundation on Tidal Flat 40
Table 3-3 Modelled Water Level at Urala Creek South 47
Table 3-4 Simulated Current Speed 53

Table 3-5	Modelled High Tide Water Near Locker Point	58
Table 3-6	Comparison of impacts to tidal discharge rates at q1, q2 and q3 locations	62
Table 4-1	Bitterns Constituents of Laboratory Created Bitterns and Expected Levels	74
Table 4-2	Summary of Laboratory Test Results	75
Table 4-3	Monthly Discharge Properties	77
Table 4-4	Proposed Interim Metals EQG's and Dilutions Required	82
Table 4-5	Near Field Model Inputs	86
Table 4-6	Model Results	89
Table 4-7	Monthly Bitterns Effluent Properties For Far Field Modelling	96
Table 4-8	summary of modelled impacts to salinity	98
Table 4-9	Modelled Impacts To Salinity Field	100
Table 4-10	Supplied recommended guideline dilution values (%)	109
Table 5-1	Derived possible and probable coral mortality thresholds (Fisher, 2019) from the dredging science node of WAMSI	122
Table 5-2	Modelled Impact zones from proposed dredging	124
Table 5-3	Modelled deposition thickness from proposed dredging	128

1 INTRODUCTION

K+S Salt Australia (K+S) is proposing to construct a solar salt evaporation facility (the Ashburton Salt Project) approximately 40 km southwest of Onslow.

The location of the proposed Project is presented in Figure 1-1 and Figure 1-2.

The facility will require a range of infrastructure to be constructed including a seawater intake and hypersaline wastewater (bitterns) outfall, as well as a jetty and berthing pocket to allow for export of the salt product.

The operational layout will be constructed on existing salt flat areas that are located inshore from the coast.

K+S commissioned Water Technology to undertake a Marine and Coastal Assessment and Modelling Study to understand the magnitude of impacts to coastal processes, coastal habitats and marine water quality. This understanding will guide the analysis and determination of coastal environmental and water quality impacts and mitigation measures. Overall, this study:

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

This report focuses on the description of the coastal and marine modelling, specifically, it includes:

- Development of a calibrated hydrodynamic model across marine, nearshore and coastal areas.
- Simulation of a range of typical and extreme events and description of existing coastal environment across the development area.
- Evaluation of the residual impacts to hydrodynamics, coastal processes and tidal inundation resulting from the proposed development.
- Development of a concept for the nearfield diffuser, modelling the bitterns discharge and determining the extent of the near-field mixing.
- Bitterns discharge modelling and assessment of bitterns plume impact zones.
- Dredging plume modelling and assessment of dredging plume impact zones.
- Technical details to support the Environmental Impact Assessment (EIA).

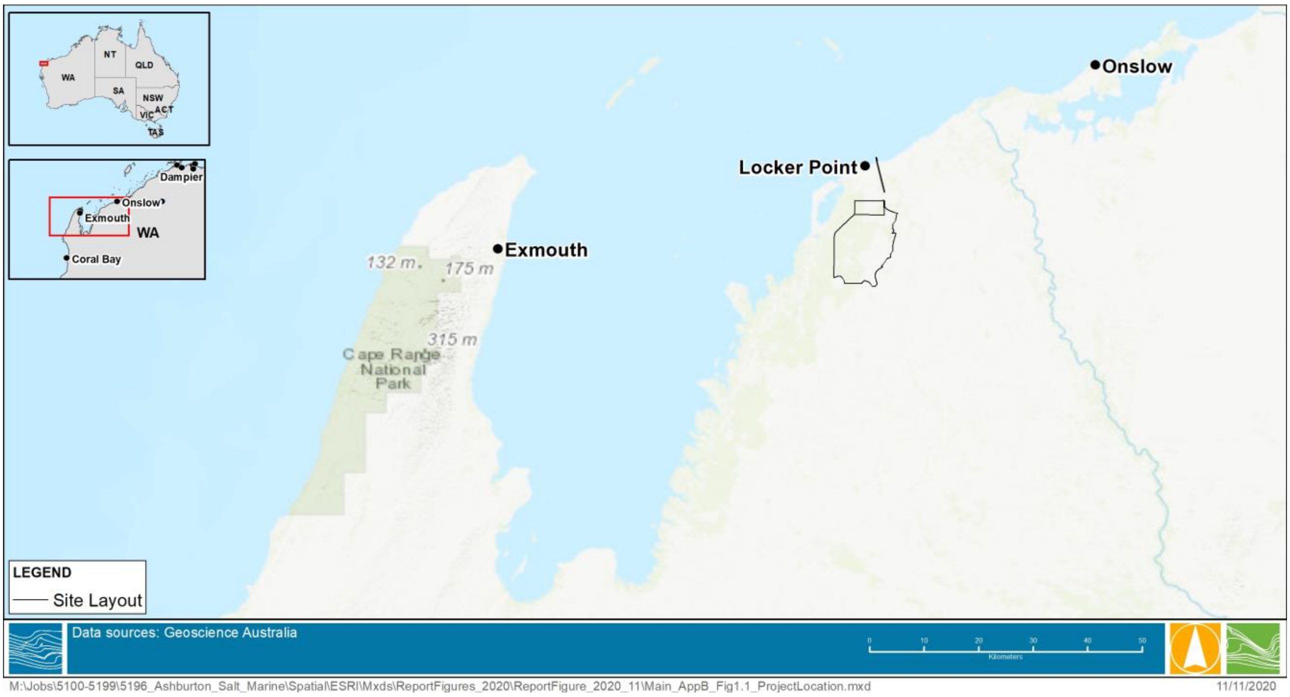

FIGURE 1-1 PROJECT LOCATION

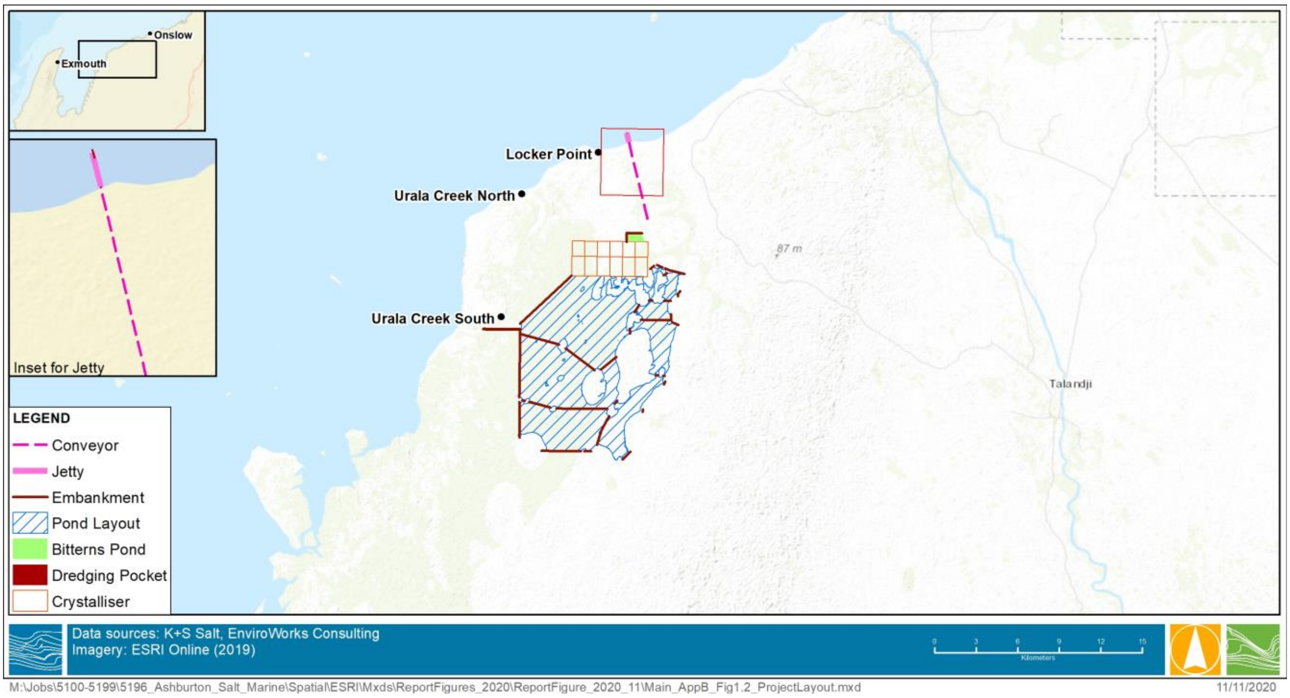

FIGURE 1-2 PROPOSED PROJECT LAYOUT

2 HYDRODYNAMIC MODELLING PACKAGE

2.1 Modelling Package

The numerical modelling approach is utilised to evaluate the change in hydrodynamics associated with the proposed development. The DHI MIKE modelling package has been used which is industry standard software used for hydrodynamic simulations.

The DHI MIKE FM Hydrodynamic model (HD) has been used for this study. It is a general modelling system for simulation of flows in oceans, estuaries, bays and coastal areas. The model simulates unsteady three-dimensional water flows driven by density variations, bathymetry as well as external forcing from meteorologic influences, tide, ocean current and river inflows.

2.1.1 Model Mesh

The model is built on an unstructured flexible mesh and uses a finite volume solution technique. Horizontally, the mesh is comprised of triangle and quadrilateral elements. This approach enables variation of the horizontal resolution of the mesh within the model area and tailoring of finer mesh in selected sub-areas. The regional model mesh is displayed in Figure 2-1.

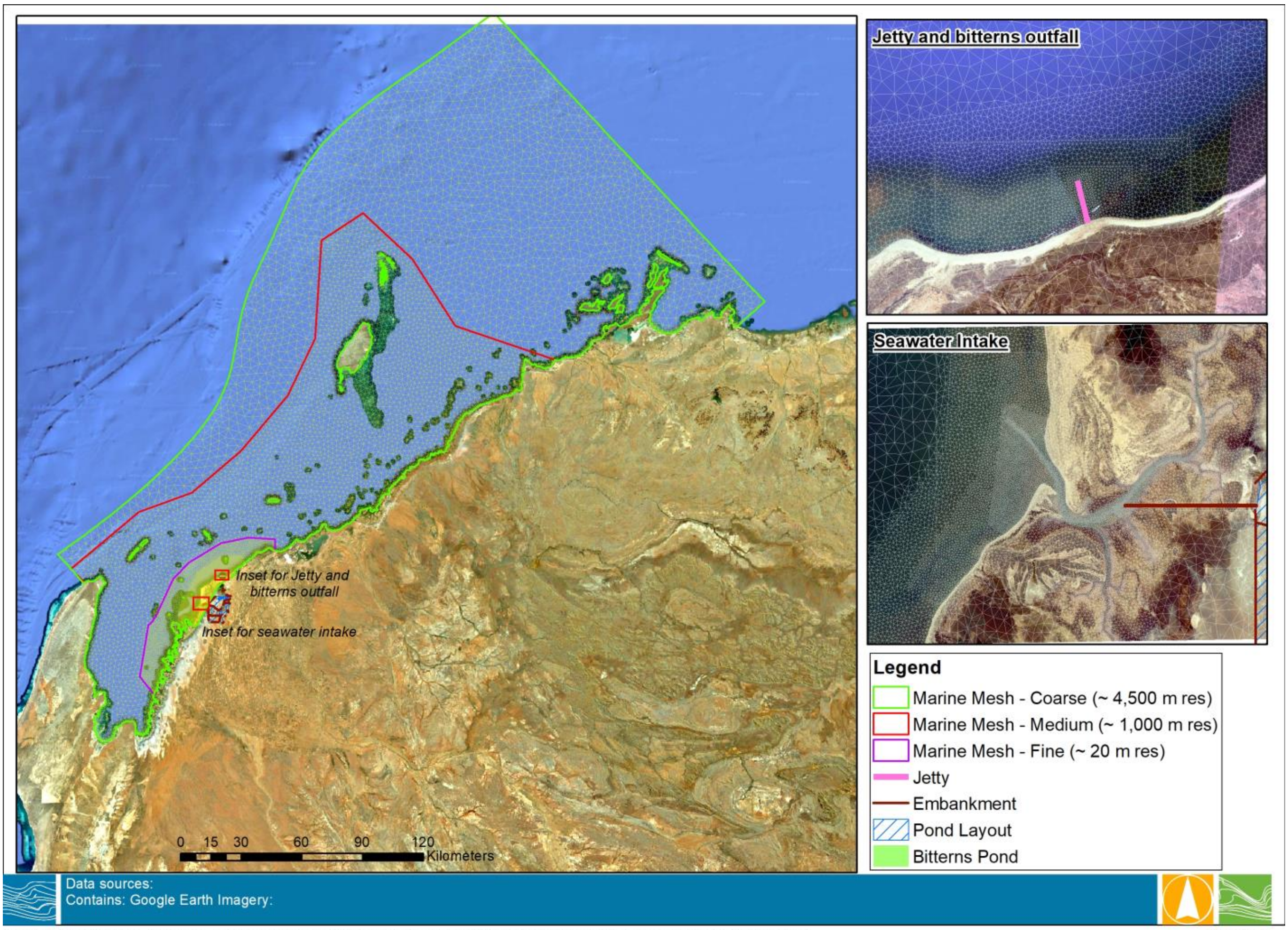

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FIGURE 2-1 REGIONAL MODEL MESH

The model northern boundary extends offshore to the edge of the continental shelf, whereas the western boundary starts from north of Ningaloo Peninsula. This is to ensure the model includes circulation resulting from the complex offshore regional current system.

The computational triangular mesh of the model is comprised of sufficiently small cells capable of resolving the detailed conditions in the study area. Model resolution varies between 4,500 m at the boundary to less than 20 m in Urala Creek South.

Local Mesh

For tidal inundation analysis, a refined local mesh was nested into the main mesh with additional resolution in creeks, intertidal zones and supratidal areas. A mixture of triangular and quadrilateral elements was used. In the coarser offshore region, element lengths were up 1 km and in the nearshore region were as small as 5 m.

FIGURE 2-2 LOCAL MODEL MESH

2.2 Model Inputs and Parametrisation

2.2.1 Bathymetry

Model bathymetry and topography was derived from a range sources, the extents of which are displayed in **Error! Reference source not found.**. A description of each data source is provided below:

- RPS DEM: coarse offshore 200 m gridded Digital Elevation Model (DEM) developed by RPS (2017). Accuracy unknown.

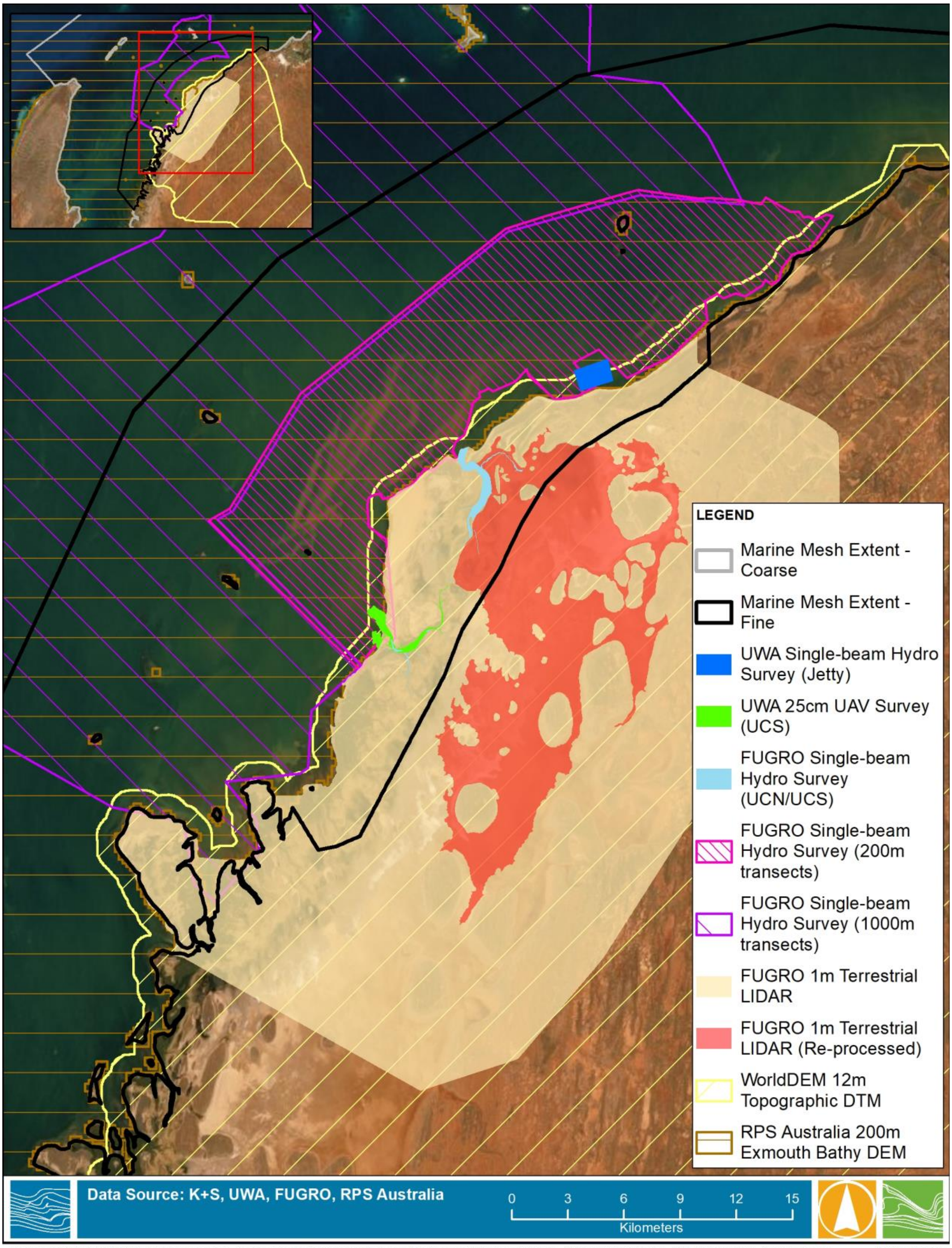

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FIGURE 2-3 BATHYMETRY DATA SOURCES AND EXTENTS

- Geoscience Australia: 250 m resolution gridded bathymetry to fill offshore data gaps not covered by RPS data (not shown on image). Varying degrees of accuracy depending on the region.
- WorldDEM DTM: 12 m-resolution DEM for nearshore areas.
- Fugro LiDAR: 1 m-resolution DEM for terrestrial areas. Stated accuracy ±0.2 m. Collected in 2017.
- Fugro Sonar Surveys: Single beam sonar surveys in Urala Creek North and Urala Creek South. In nearshore waters transects were completed 200 m apart, and further offshore transects were 1000 m apart. This data was converted to a DEM. . Collected in 2017.
- University of Western Australia (UWA) Survey: 25 cm-resolution DEM derived from an underwater autonomous vehicle (UAV) survey of Urala Creek South. No metadata provided but accuracy estimated to be ±0.2 m. Collected in 2018.
- K+S Single Beam Sonar Survey: Single beam sonar survey at Locker Point/Jetty site. No metadata provided, accuracy unknown. Collected in 2020.

Review of the Fugro 1m Terrestrial LiDAR showed some ridges and banding errors along the flight paths (refer Figure 2-4 for example) and some accuracy issues in mangrove habitat and salt flat areas, which was observed during model sensitivity testing. The following post-processing was undertaken to smooth the ridges and prepare the elevation model for the inundation modelling:

- Each 1 m grid cell was set with the minimum value from the surrounding 50 x 50 m window, using the ArcGIS Spatial Analyst tools including the Block Statistics tool. The reprocessing area is shown in **Error! Reference source not found.**.
- Compensate tilt of elevation model levels with known elevated high ground.
- Apply discrete local corrections to creek channels using mapping by Seashore Engineering.
- Apply suitable local smoothing to consider satellite photographs of tidal inundation.
- Implement a 0.2 m reduction of DEM in intertidal zone, as suggested by low tide inundation extents from satellite photographs. This was also supported by ground truthing undertaken by UWA in 2019.

The discrepancy between the UWA elevation data collected to ground truth the Fugro 1m Terrestrial LiDAR, found an offset of 0.2 m. Consequently, local adjustments to the LiDAR DEM were made and checked with satellite images showing tidal inundation. Additional LiDAR survey and further ground truthing of site benchmarks would improve the DEM for detailed engineering, however the DEM in its current form is suitable for use in the environmental impact assessment modelling.

A range of bathymetric data sources were used and all those in the nearshore region were collected recently for this project throughout 2017 and 2020. This is considered a suitably up to date and detailed bathymetric dataset. As the model development progressed, bathymetric data gaps and uncertainties were identified, and additional data was collected or ground truthing was undertaken. This ensured that there was a comprehensive, robust and recent bathymetric dataset that was fit for purpose.

The hydrodynamic model calibration demonstrates that the bathymetric data is representative of the creeks and intertidal areas. Bathymetry is a pivotal model component and if it is incorrect will lead to a poor calibration result. Careful consideration was given to the post-processing; data interpolation and checks were performed for spurious values to provide confidence that the data correctly represents the features of interest. Without good underlying bathymetric data, the task of trying to calibrate a hydrodynamic model would be extremely difficult, especially in the complex coastal and estuarine areas across the study site.

Model calibration of water levels was performed over one month in June/August 2019 on the local model mesh. Modelled water levels were compared to measured data at Urala Creek South (intake location) and Locker Point (bitterns discharge location).

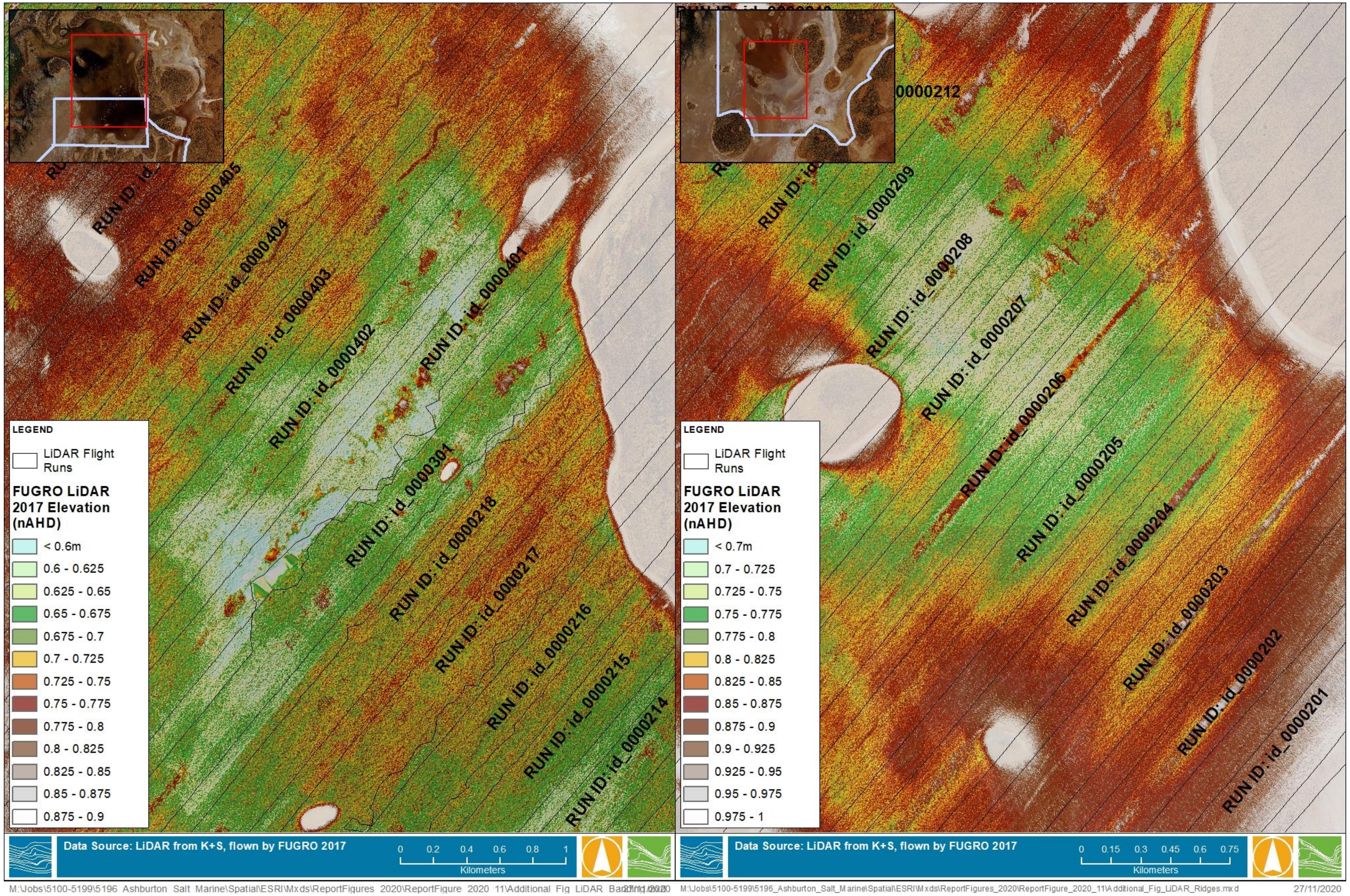


FIGURE 2-4 LIDAR BANDING AND RIDGES IN SALT FLATS

The modelled water levels provided a good representation of water level variations throughout the tidal range and phases. Statistics for Locker Point are presented in Table 2-1. The WS, RMSE and bias of modelled water levels were 0.98, 0.12 m and -0.01 respectively at Locker Point. Similar model performance was observed in Urala Creek South.

TABLE 2-1 JUNE AUGUST CALIBRATION STATISTICS AT LOCKER POINT

Measure	Statistic
WS	0.98
RMSE	0.12
Bias	-0.01

The tidal flats connected to Urala Creek North and South have a typical elevation in the range from 0.2 to 1 m AHD. These low gradient intertidal and supratidal areas, which consist of mangrove, algal mat and bare salt flat habitats, experience cyclical tidal inundation. Flooding into the algal mats and salt flats areas occurs through the mangroves zone and other areas of lower elevation when water levels are greater than MHWS (~0.8 m). Under these water level conditions a shallow tidally induced sheet flow submerges expansive areas, particularly surrounding Urala Creek North.

The area of tidal inundation is sensitive to the modelled water level as well as to the accuracy of LiDAR topography data. The model calibration run in April 2020 (the month which experiences the highest seasonal tidal inundation) was compared to Sentinel-2 satellite images during the same period, which showed expansive inundation as shown in Figure 2-5 and Figure 2-6. The comparison shows that the model is capable of simulating tidal inundation over the complex intertidal zone in the project area and also confirms that the topography and bathymetry in the model is fit for purpose.

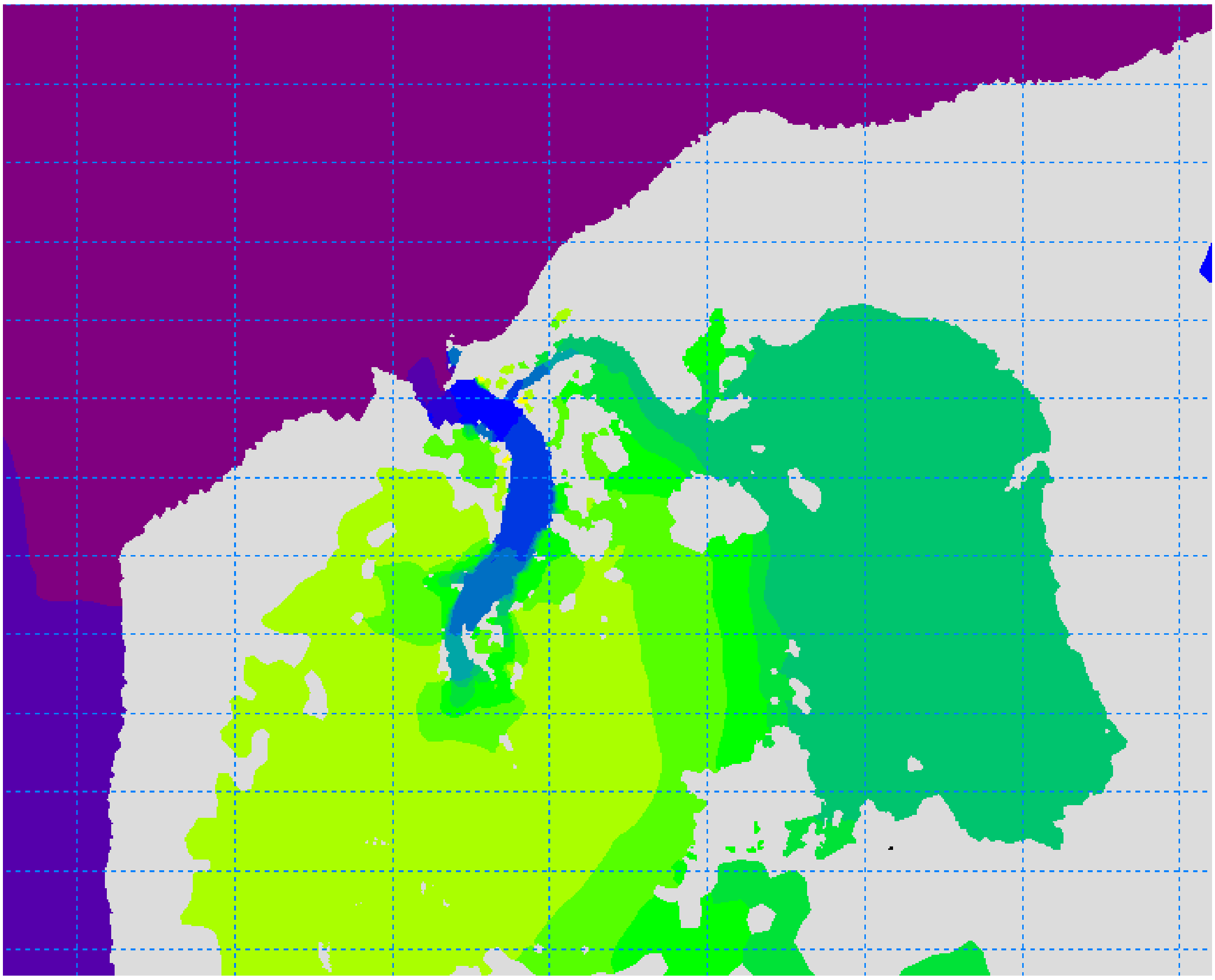


FIGURE 2-5 TIDAL INUNDATION (COLOUR DENOTES WATER LEVEL) MODELLED - 15 APRIL 2020

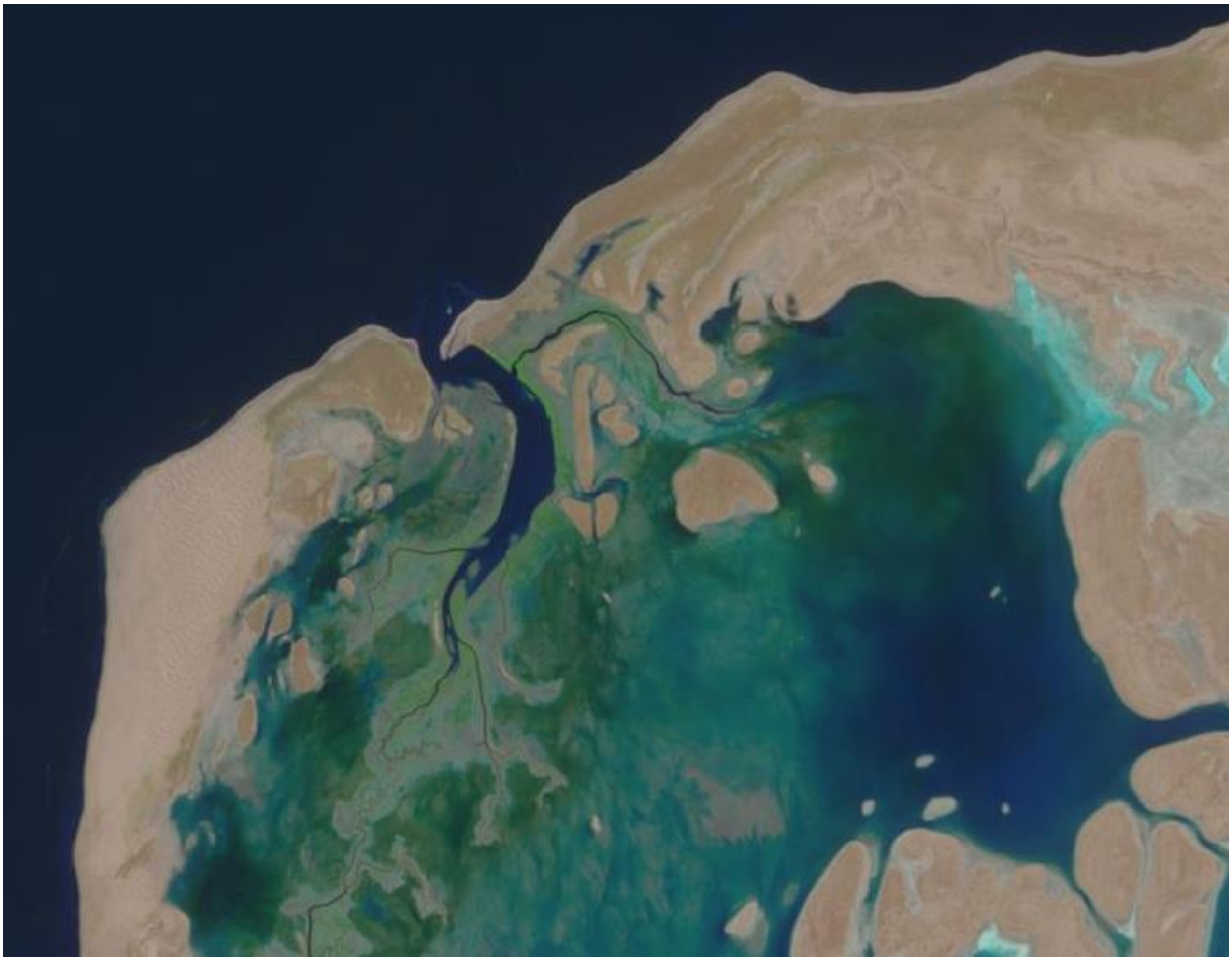


FIGURE 2-6 SENTINEL SATELLITE IMAGE - 15 APRIL 2020

2.2.2 Development Layout and Model Implementation

The proposed development consists of land-based structures such as evaporation and crystalliser ponds, bitterns dilution pond, roads, embankments, as well as marine structures such as the seawater intake, jetty, bitterns discharge diffuser and berthing pocket. Structures were incorporated into the model mesh as shown in Figure 2-7 .

Within the model, the following key structures were integrated into the modelling platform:

- The intake was implemented as a negative source point with a constant inflow rate of 10.97 m^3/s.
- A dike was used to conceptualise the solid structures (e.g., apron, revetment) near the 4-intake pump station.
- The outfall was implemented as a positive source points with a constant total discharge of 0.98 m^3/s.
- A friction map developed for the catchment modelling was utilised which had higher friction in areas with vegetation growth/mangrove habitats and lower friction over flat seabed or land surfaces (refer to Section 2.2.4).

The model uses a high-resolution local mesh with refined areas in Urala Creek South to resolve flow pathways along the narrow creek branches and intertidal areas. It has over nine grid points across the main creek channel and one to two grid points to resolve the narrow upper channels (see Section 2.1.1)

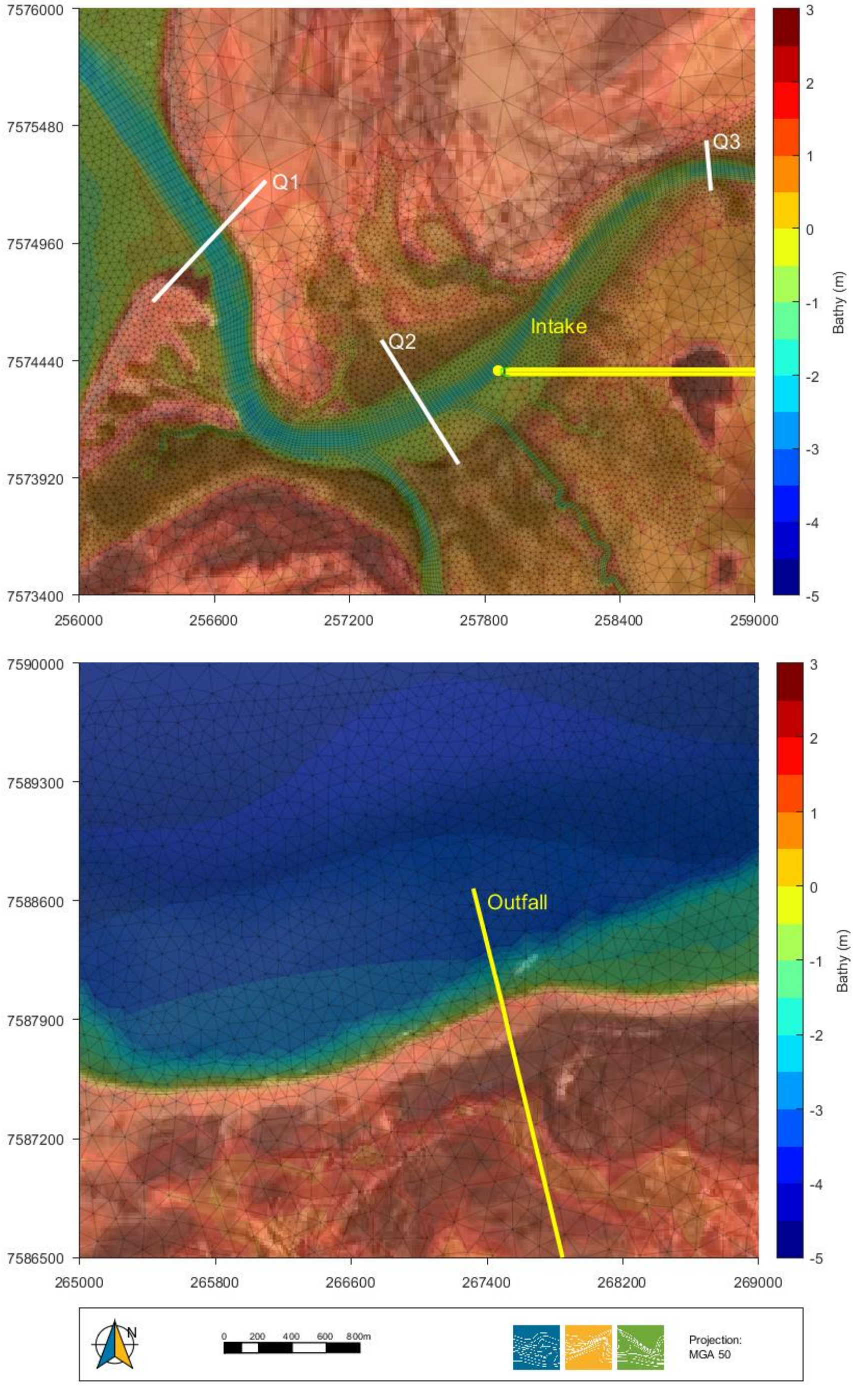


FIGURE 2-7 INTAKE FACILITIES (UPPER PANEL) AND EMBANKMENTS PILE SUPPORTED JETTY AND CONVEYOR (LOWER PANEL)

A summary of key differences between the existing and development model mesh are provided in Table 2-2. As shown in Figure 2-8, the same model mesh was used, but the development mesh had the elevations from the site layout incorporated. Using the same model mesh ensures inundation patterns will remain the same in areas that are unaffected by the development layout.

TABLE 2-2 MODEL IMPLEMENTATION (EXISTING VERSUS DEVELOPMENT)

Settings	Existing	Development
Bathymetry	Existing bathymetry data (see Section 2.2.1)	Modified bathymetry: ■ Elevated road/embankments to design water levels ■ Intake flow channel connecting to the creek (bottom elevation 0 m AHD, embankment wall about 2 m AHD)
		Elevated road/embankments to design water levels
		Intake flow channel connecting to the creek (bottom elevation 0 m AHD, embankment wall about 2 m AHD)
		Abutment and other solid structures conceptualised as a dike with crest elevation of 4.5 m AHD
Intake	No intake	Pre-Feasibility Intake Design, negative source point
Outfall	No discharge	Concept Outfall Design – refer to Section 4 for details, positive source point

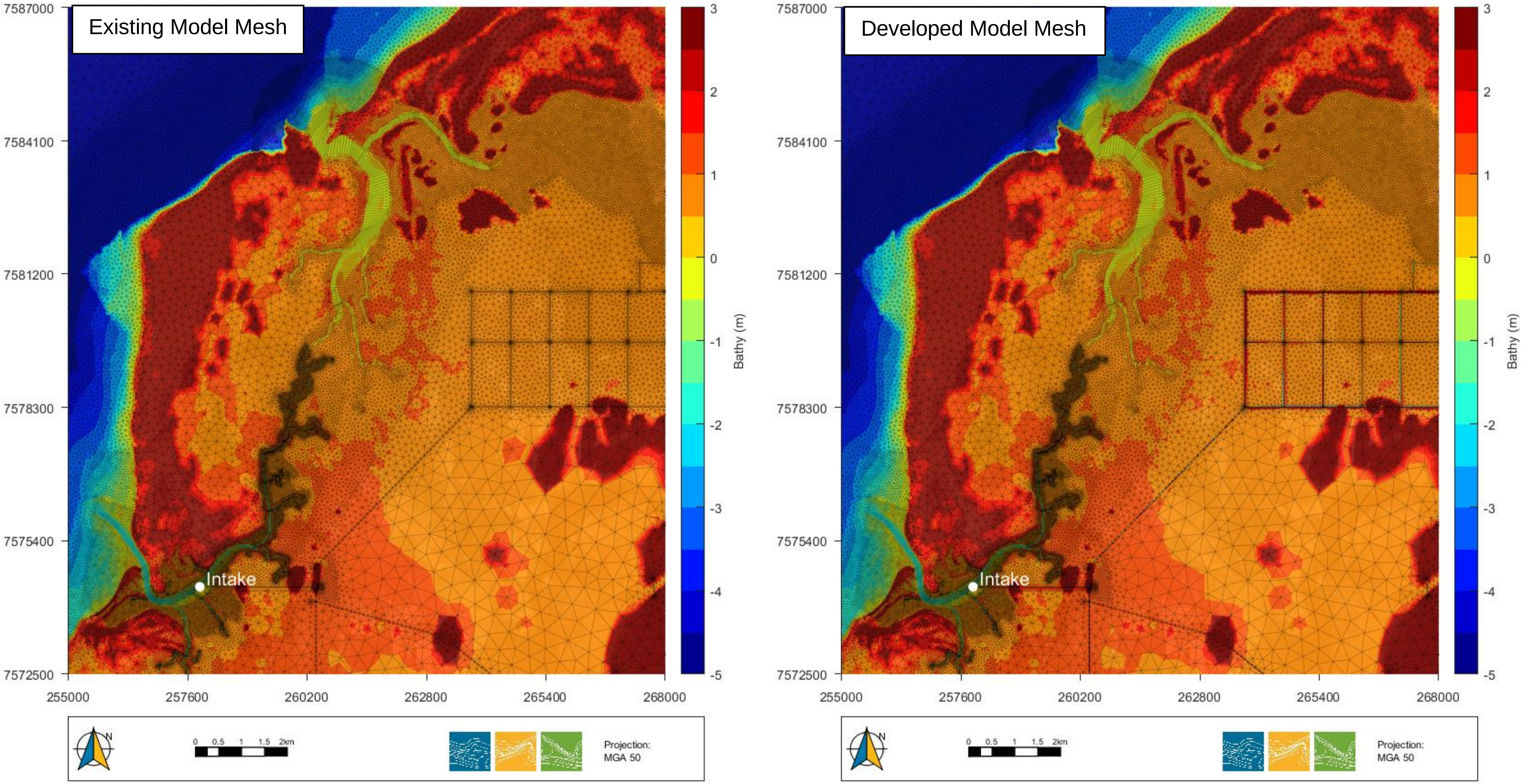


FIGURE 2-8 EXISTING (LEFT) AND DEVELOPED (RIGHT) MODEL MESH

Pre-Feasibility Intake Design

The preliminary design of the seawater intake consists of multiple pumps with openings facing downward at 0.5 m above the bottom of the intake pond (floor level at -4 m AHD elevation, as shown in Figure 2-9). The total intake rate is approximately 10.97 m^3/s representing the highest monthly intake in November (~29 GL). The seawater intake is conceptualised as a negative source point in the hydrodynamic model.

Modelling has been configured on a conservative basis by representing the seawater intake as continuous pumping in both low and high tide conditions. However, it is understood that pumping will not occur at low tide.

The seawater intake in the model is approximately 200 m upstream of the final location adopted in the EIA. The model remains suited for the purposes of the EIA, as the hydrodynamic impacts modelled on the main channel are unlikely to be susceptible to such a localised adjustment.

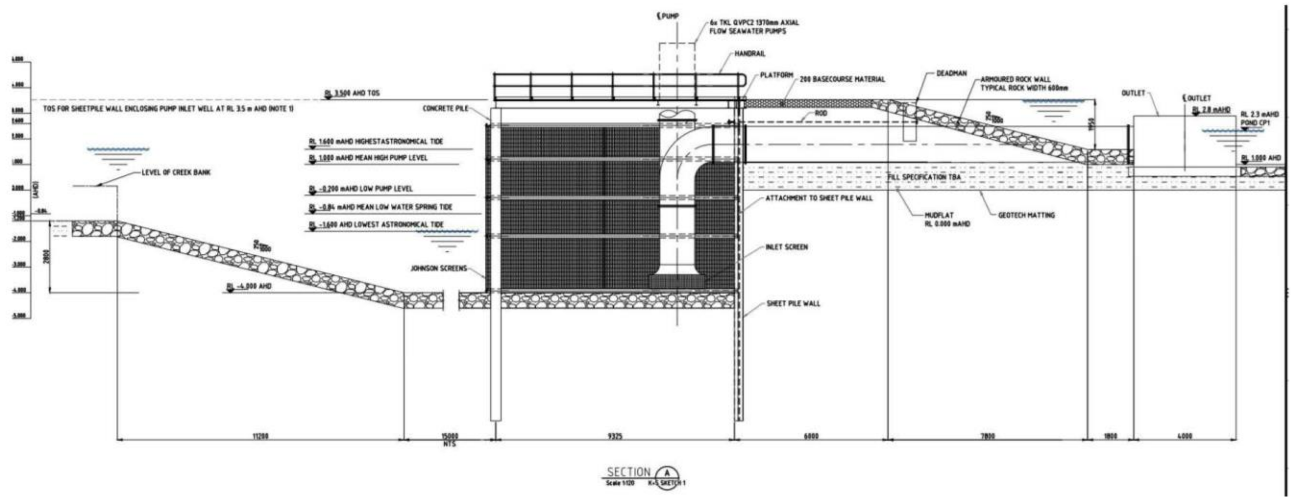


FIGURE 2-9 INTAKE DESIGN (SOURCE: VORTEX AUSTRALIA, DECEMBER 2020)

Preliminary Bitterns Outfall Design

Preliminary design of the bitterns outfall consists of a long diffuser along the last 400 m section of the jetty discharging at a maximal rate of 0.98 m^3/s representing the highest production month in November. The bitterns will be discharged from the middle depth of the water column upwards to maximalise near-field dilution. Refer to Section 4 for a more detailed description of the diffuser concept design (also see Figure 2-10 for a conceptual illustration of the diffuser design).

The bitterns outfall is conceptualised as positive source points in the hydrodynamic model.

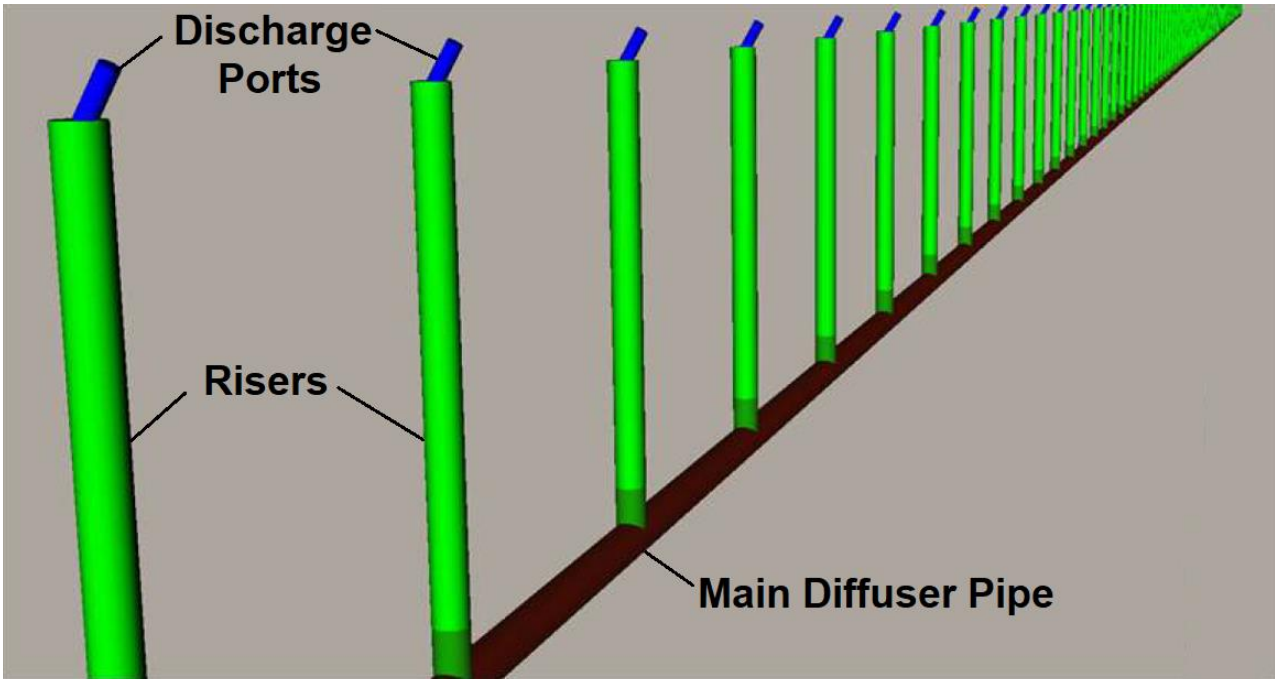


FIGURE 2-10 OUTFALL CONFIGURATION

2.2.3 Eddy Viscosity/diffusivity

Sub-grid scale horizontal eddy viscosity was parameterised as a function of the local grid resolution using a Smagorinsky formulation. This expresses the effects of sub-grid scale turbulence by an effective eddy viscosity related to a characteristic length scale and the local spatial current variations.

2.2.4 Bed Roughness

A bed resistance map was developed in accordance with intertidal and terrestrial features such as mangrove habitats (see Table 2-3). It was used as a key parameter to calibrate tidal levels.

TABLE 2-3 ROUGHNESS VALUES

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

2.3 Boundary Conditions

The location of the model offshore is presented in Figure 2-1.

2.3.1 Water Levels and Currents

The model is driven by a Flather boundary condition (Flather, 1976) at its open boundaries which takes account of both tidal and subtidal components. Water levels and currents were modelled as follows:

- Tidal levels were extracted from Global Tide Model developed by Denmark Technical University (DTU) Space. The model is available on a 0.125° x 0.125° resolution grid for the major 10 constituents in the tidal spectra. The model utilises the latest 17 years' multi-mission measurements from TOPEX/Poseidon, Jason-1 and Jason-2 satellite altimetry for sea level residual analysis. The constituents consider the semidiurnal M2, S2, K2, N2 the diurnal S1, K1, O1, P1, Q1 and the shallow water constituents M4.
- Subtidal water levels were extracted from Hybrid Coordinate Ocean Model (HYCOM) global operation model. This is to represent any seasonal variation of water levels in the region.
- HYCOM currents were applied at the model boundary in addition to water levels. The HYCOM currents are required for the Flather type boundary condition to force subtidal flow into hydrodynamic model.

HYCOM is an operational ocean model system from National Oceanic and Atmospheric Administration (NOAA). The model assimilates satellite altimeter observations, satellite and in situ sea surface temperature, as well as available in situ vertical temperature and salinity profiles from XBTs, ARGO floats, and moored buoys. It is a proven product providing high precision hydrodynamic predictions at a global coverage and with a resolution ranging from 1/12-degree world-wide to 1/25-degree at Gulf of Mexico. The model has over 40 vertical layers while most layers are located near the surface to resolve the stratification near the surface boundary layer.

2.3.2 Water Temperature and Salinity

HYCOM temperature and salinity data were utilised to drive the model on the boundary and more importantly to assist initialising the model in the absence of adequate spatially variable temperature and salinity measurements.

It was noticed that the in-situ salinity records are higher than concurrent HYCOM predictions nearshore and that the HYCOM grid resolution is too coarse to initialise the salinity field nearshore. Additional data collected in the field such as conductivity, temperature, depth (CTD) probe measurements, and temperature/salinity records from other loggers have been used to amend the boundary/initial conditions wherever appliable. The data collected for this Project is described further in '*Marine, Coastal and Surface Water Data Collection'*, (Water Technology 2021).

2.3.3 Meteorological Inputs

2.3.3.1 Evaporation and Precipitation

Due to a lack of reliable measured data, the annual mean evaporation from Learmonth Airport weather station was used within the model. For precipitation, measured data from Onslow Bureau of Meteorology (BoM) weather station was used.

2.3.3.2 Surface Heating

The model is driven by spatial air pressure and cloud coverage from Climate Forecasting System Reanalysis (CFSR). Other empirical surface heat flux inputs are specified where appropriate.

2.3.3.3 Non-cyclonic winds

Wind data was sourced from the National Centre for Environmental Prediction (NCEP) Climate Forecast System Reanalysis (CFSR) model. The CFSR wind model includes observations from many data sources; surface observations, upper-atmosphere air balloon observations, aircraft observations and satellite observations. The model has a resolution of 0.25° at the equator and 0.5° globally and is capable of accurately representing the interaction between the earth's oceans, land and atmosphere (Saha et al. 2010). CFSR derived spatially varying wind and pressure was applied throughout the model domain.

2.3.3.4 Cyclonic winds

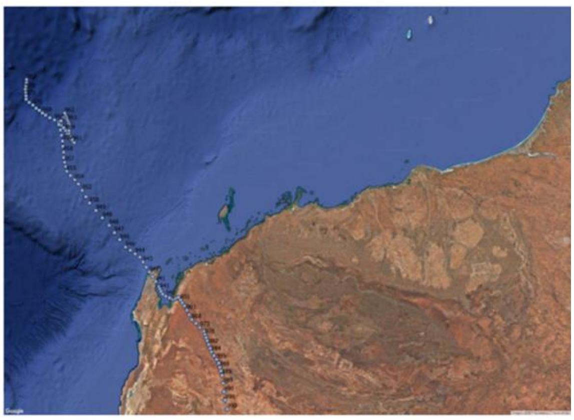
SEAsim (https://www.systemsengineeringaustralia.com.au/models.htm, Harper B A, (1997)) is a discrete Monte-Carlo statistical model employing tide generation and a parametric hurricane surge model, which can be applied to coastal areas. The model simulated 10,000 years of synthetic Tropical Cyclone (TC) events at a fixed Mean Sea Level (MSL), then assigns a random start year between 1950 and 2050 to enable a tide to be generated and combined with the surge response, as well as making a small non-linear tide-surge interaction adjustment. The model then randomly re-samples the tide a further four times for each event by adjusting the start time, constraining it to lie within a 14-day period to maintain seasonal tidal behaviour. This results in an equivalent 50,000-year statistical sample.

The SEAsim model retains a summary of each TC event that is modelled, which enables post-processing to select those events generating peak water levels at specific coastal points that bracket a specific Average Recurrence Interval (ARI) level. In this case the selection is based on TC events that have generated levels within 5% of the 100-year ARI and 500-year ARI combined Tide+Surge Level respectively at the study site. Several hundred events are available. The events with the lowest error statistics have been considered for hydrodynamic modelling simulation. These events are summarised in Table 2-4.

TABLE 2-4 SELECTED TC EVENTS WITHIN 5% OF THE INDICATED TIDE+SURGE WL

ARI (year)	Parameters when near the Study Site					Track Starting (date-time)
	p_0 (hPa)	Distance (km)	V_{fm} (m/s)	R (km)	θ_{fm} (°)	
100	855	34	7.5	20	215	06/03/1980 19:00
500	840	52	3.0	31	150	25/01/1986 14:00

100y ARI event track



500y ARI event track

The 100-year event approaches on a steady track from the northeast and passes 34 km west of the study site. The 500-year event is slightly more intense and approaches from the northwest, passing 52 km to the west of the study site. Both events have occurred near high tides. It can be noted that the track of TC Vance in 1999 was more northerly and essentially bisected these tracks. It was also a more intense storm (BoM 1999) and had a very significant impact.

2.4 Model Calibration

Model calibration consisted of an iterative process of adjusting modelling parameters to arrive at a reasonable comparison between modelled data and recorded measurements. These parameters include but are not limited to bathymetry, boundary and initial conditions, bed resistance and other model constants.

Water levels were calibrated to evaluate model performance in simulating water level variations driven by tides, air pressure, winds and other metrological and oceanographic forces in the model domain and specifically in the vicinity of the Project site.

The evaluation of model performance measures was undertaken to demonstrate the model's ability to accurately replicate natural processes and characteristics of the region of interest. The following statistical measures were used:

- Willmott Skill (WS) – measure of model predictive skill that ranges from 0 to 1 (Willmott 1981). The highest value of 1 means perfect agreement between model and observation.
- Root mean square error (RMSE) – standard way to measure the error of a model in predicting quantitative data standard deviation of the residuals. Lower values indicate a better fit.
- Bias - mean time-averaged difference. Lower values indicate better model performance.

In addition to statistical evaluation of model accuracy, an assessment of tide constituents, seasonal water level variations and a visual comparison to satellite imagery was undertaken.

2.4.1 Water Levels

2.4.1.1 Regional Model Calibration - March to April 2019

The regional model was calibrated in March/April 2019 using the regional mesh (lower resolution in Urala Creek and tidal flat), during a time period when water levels are generally elevated. The results of the calibration are displayed in Figure 2-11 and statistics for Locker Point are presented in Table 2-5. The WS, RMSE and bias (of modelled water levels) were 0.99, 0.09 m and 0.01 respectively. The results at Locker Point are displayed in in Figure 2-11 which shows very good representation of tidal range and phases

TABLE 2-5 MARCH/APRIL CALIBRATION STATISTICS AT LOCKER POINT

Measure	Statistic
Model skill	0.99
RMSE	0.09
Bias	0.01

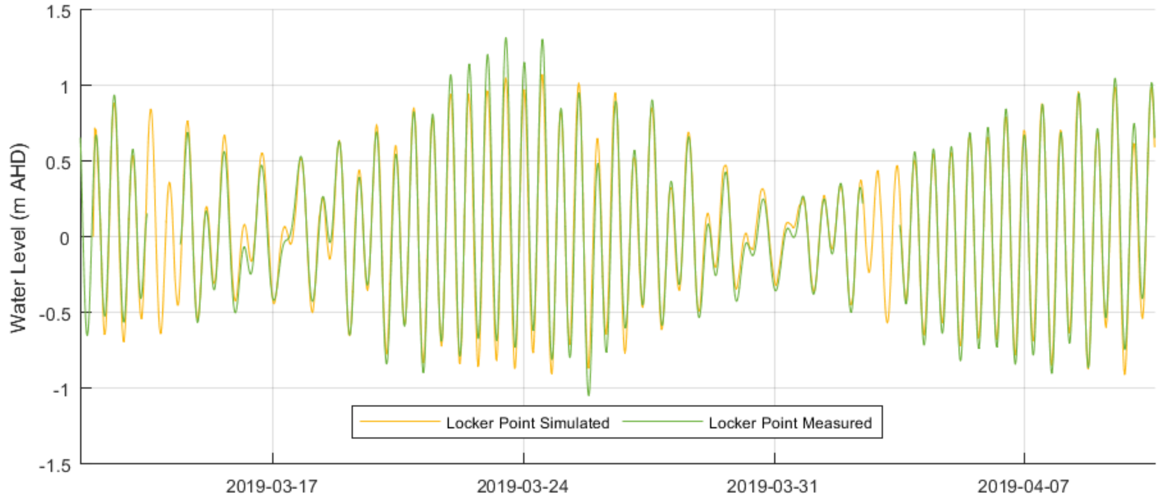


FIGURE 2-11 MODELLED AND MEASURED WATER LEVELS AT LOCKER POINT (PROPOSED BITTERNS DISCHARGE LOCATION)

2.4.1.2 Local Model Calibration - June to August 2019

Model calibration was performed over one month in June/August 2019 on the local model mesh. Modelled water levels were compared to measured data at Urala Creek South (intake location) and Locker Point (bitterns discharge location).

The modelled water levels provided a good representation of water level variations throughout the tidal range and phases. The results of the calibration are displayed in Figure 2-12 and statistics for Locker Point are presented in Table 2-6. The WS, RMSE and bias of modelled water levels were 0.98, 0.12 m and -0.01 respectively at Locker Point. Similar model performance was observed in Urala Creek South.

Despite the different modelling periods and mesh resolution, the model has shown consistent performance in simulating water levels by including model inputs from DTU tide prediction, air pressure and winds from CFS and subtidal water levels from HYCOM.

TABLE 2-6 JUNE AUGUST CALIBRATION STATISTICS AT LOCKER POINT

Measure	Statistic
WS	0.98
RMSE	0.12
Bias	-0.01

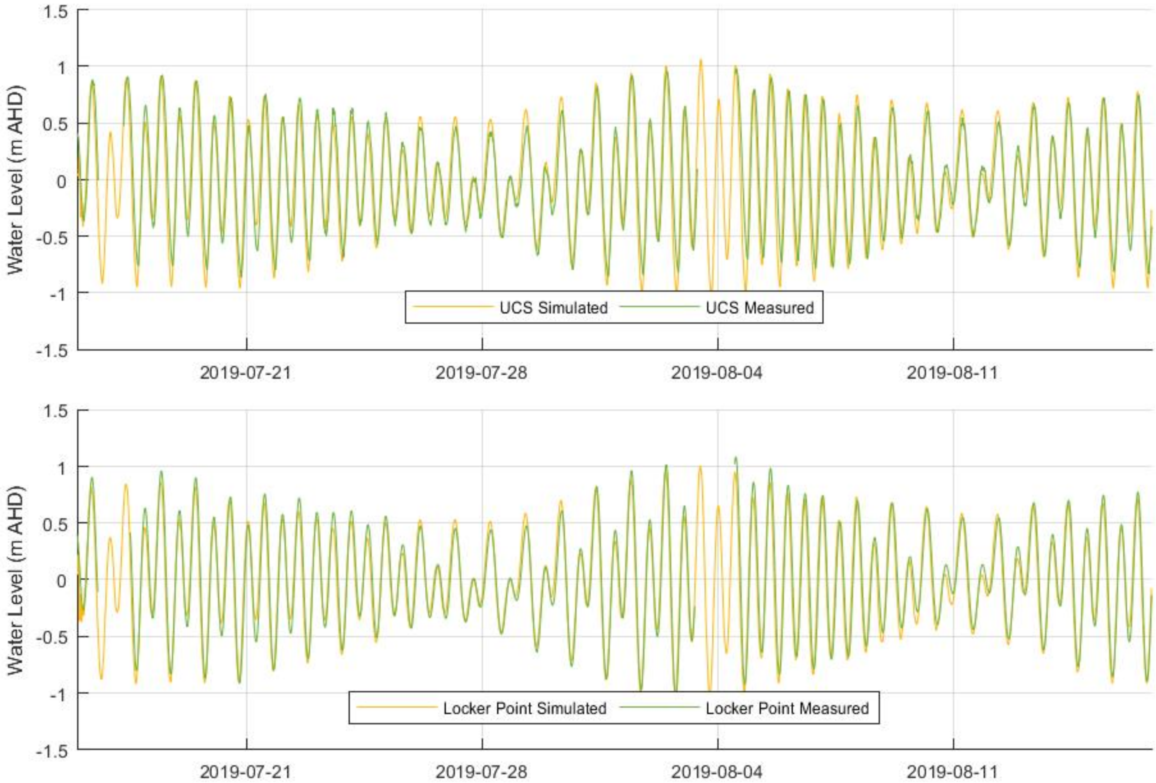


FIGURE 2-12 MODELLED AND MEASURED WATER LEVELS AT URALA CREEK SOUTH (INTAKE – UPPER PANEL) AND LOCKER POINT (DISCHARGE – LOWER PANEL)

2.4.1.3 Modelled Tidal Constituents

Review of modelled water levels at Locker Point showed consistent tidal amplitude and range of variation between the model prediction and in-situ measurements. Primary tidal constituents were simulated with reasonable accuracy as displayed in Table 2-7.

Locker Point has a lower tidal range when compared to tidal levels observed at Onslow and Exmouth tidal gauges due to local transformation/frictional impacts. Note the analysis was based on a full year of model simulation in 2015, and tidal measurements obtained at Locker Point during July 2019 and June 2020 (measurements referred to MSL for each deployment periods).

TABLE 2-7 TIDAL CONSTITUENTS COMPARISON

Tide Constitutes (m)	Model Results (2015)	Locker Points Measurements (2019)	Exmouth	Onslow	RPS Prediction
O1	0.14	0.11	0.14	0.13	0.13
K1	0.22	0.20	0.21	0.21	0.21
M2	0.46	0.47	0.58	0.60	0.51
S2	0.23	0.23	0.31	0.32	0.27
N2	0.09	0.07	0.10	0.11	0.09
K2	0.07	0.06	0.09	0.09	0.08

2.4.1.4 Modelled Seasonal Water Level Variation

Seasonal water levels were simulated through usage of HYCOM inputs on model boundaries, as described in 2.3. The seasonal water level component is shown in Figure 2-13. There is a ±0.2 m variation from the lowest month in August to the highest month in March. Review of tidal gauge records at Onslow (low-pass filtered) showed similar amplitude of variation as presented in Figure 2-14 (orange box highlighted the modelled period). We thus obtained confidence that boundary forces were properly implemented with sufficient consideration applied to seasonal water level impacts.

This seasonal water level variation results in higher spring tides in March and April than in August and September and leads to infrequent widespread tidal inundation of the bare salt flat areas each year by seasonal high tide water.

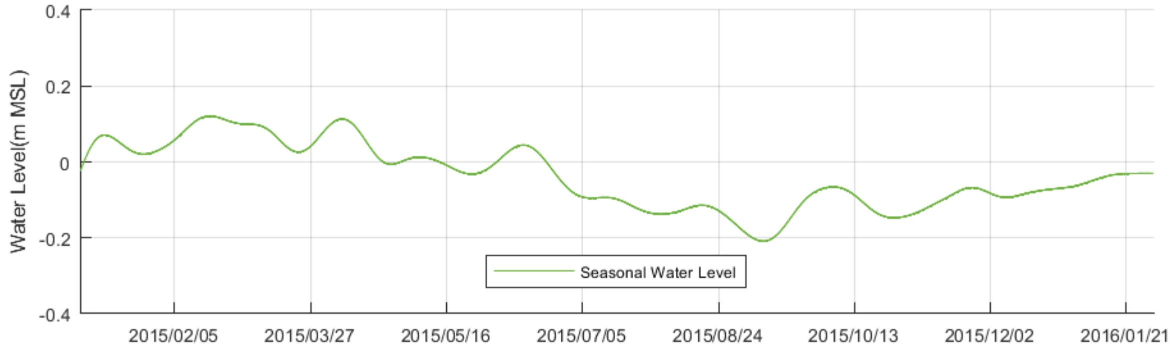


FIGURE 2-13 SEASONAL WATER LEVEL SIMULATED AT LOCKER POINT IN 2015

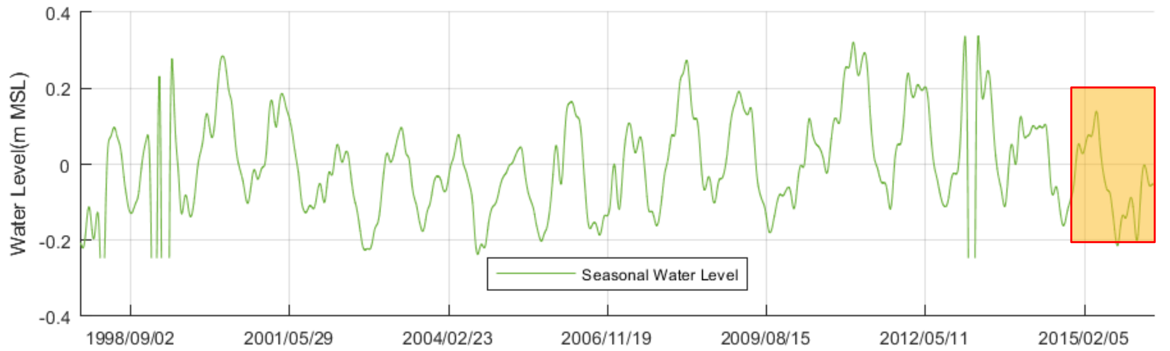


FIGURE 2-14 SEASONAL WATER LEVEL FROM ONSLOW TIDAL RECORDS (ORANGE BOX HIGHLIGTS MODEL PERIOD)

2.4.1.5 Validation of Seasonal Variation in Tidal Inundation

Water level analysis shows the region has about ±0.2 m seasonal water level variation and about ±1 m tide range. This, combined with the flat terrain in the intertidal zone, contributes to substantial differences in inundation frequency throughout the year. The phenomenon was successfully simulated by our model through inclusion of seasonal water levels from HYCOM. The results are presented in Figure 2-15 and Figure 2-16 which compare the inundation time (in percentage) between March and April (end of summer) and August to September (end of winter). Similar results are presented in Seashore Engineering (2021) indicating that the high tide plane could be inundated 10 times more frequently in March than in August.

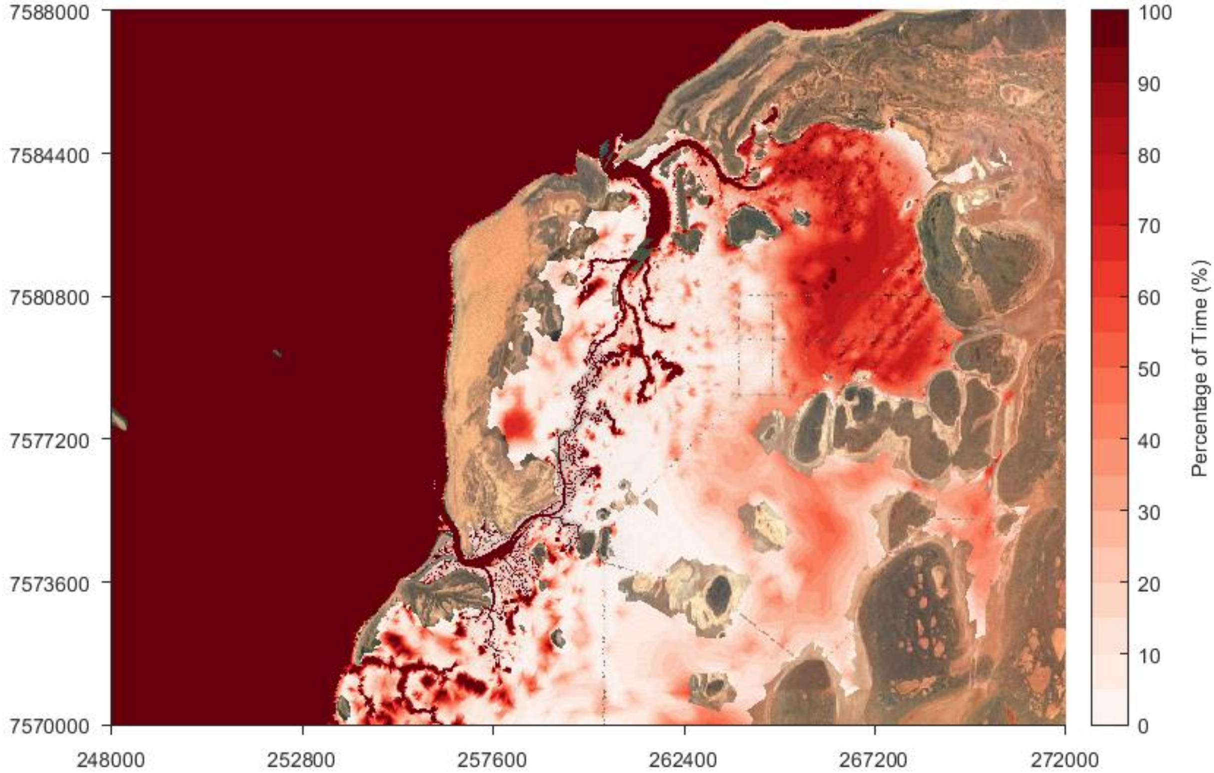


FIGURE 2-15 TIDAL INUNDATION FREQUENCY (MAR-APR)

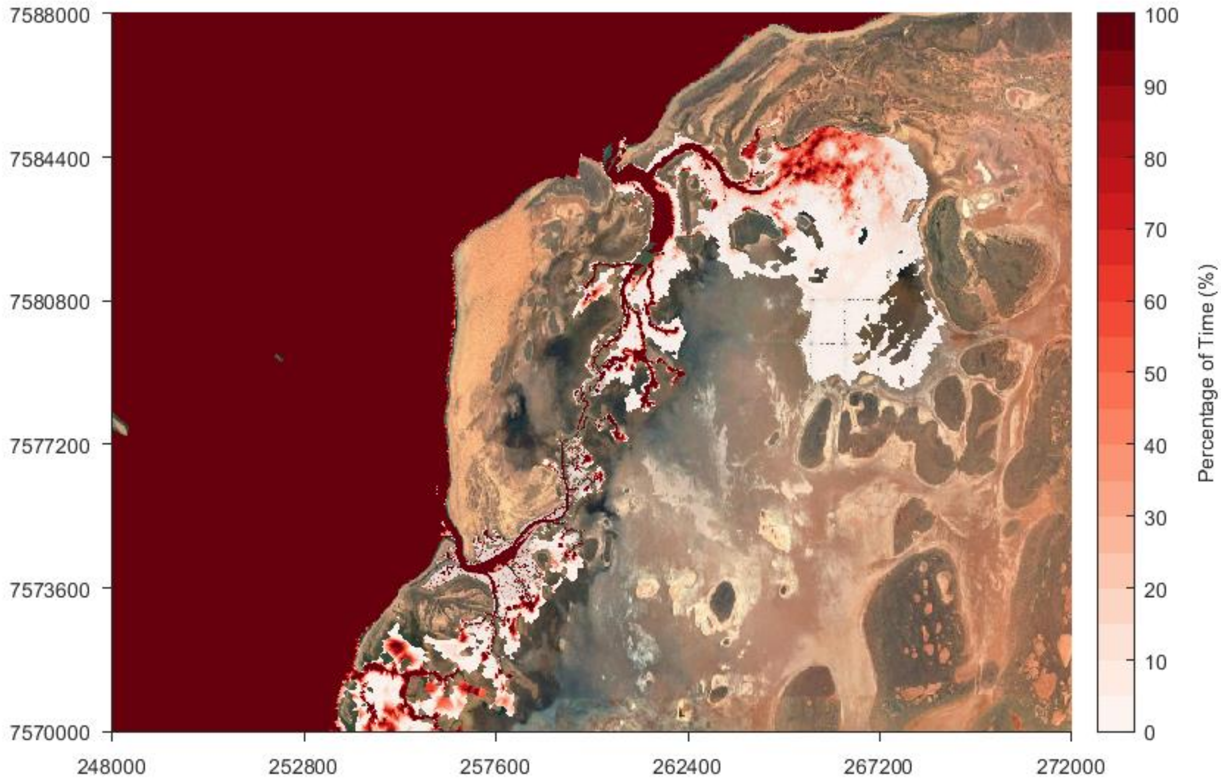


FIGURE 2-16 TIDAL INUNDATION FREQUENCY (AUG-SEP)

2.4.1.6 Validation of Peak Tidal Inundation (April 2020)

The tidal flats connected to Urala Creek North and South have a typical elevation in the range from 0.2 to 1 m AHD. These low gradient intertidal and supratidal areas, which consist of mangrove, algal mat and bare salt flat habitats, experience cyclical tidal inundation. Flooding into the algal mats and salt flats areas occurs through the mangroves zone and other areas of lower elevation when water levels are greater than MHWS (~0.8 m). Under these water level conditions a shallow tidally induced sheet flow submerges expansive areas, particularly surrounding Urala Creek North.

The area of tidal inundation is sensitive to the modelled water level as well as to the accuracy of LiDAR topography data. The model calibration run in April 2020 (the month which experiences the highest seasonal tidal inundation) was compared to Sentinel-2 satellite images during the same period, which showed expansive inundation as shown in Figure 2-17 and Figure 2-18. The comparison shows that the model is capable of simulating tidal inundation over the complex intertidal zone in Project area.

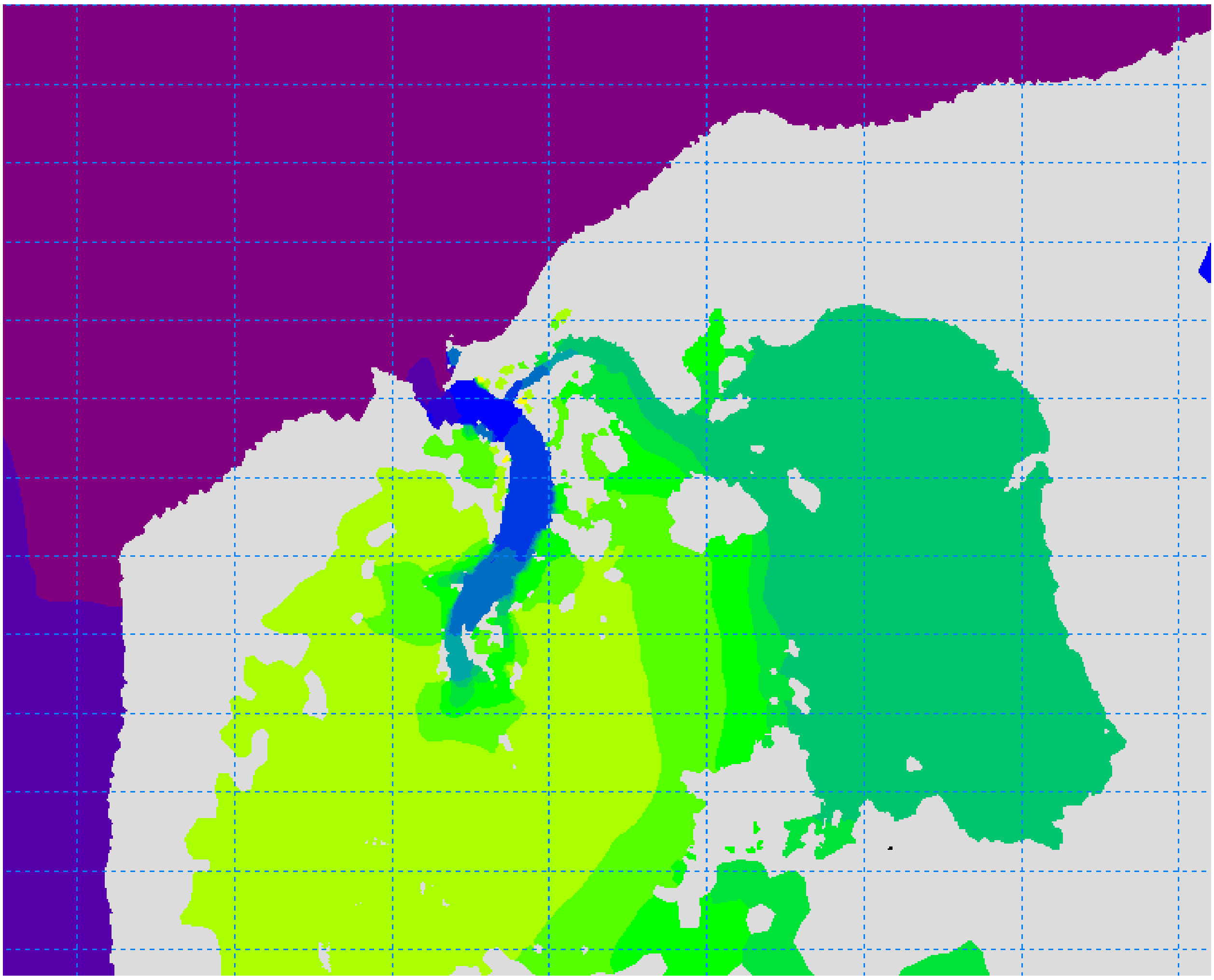


FIGURE 2-17 TIDAL INUNDATION (COLOR DENOTES WATER LEVEL) MODELLED - 15 APRIL 2020

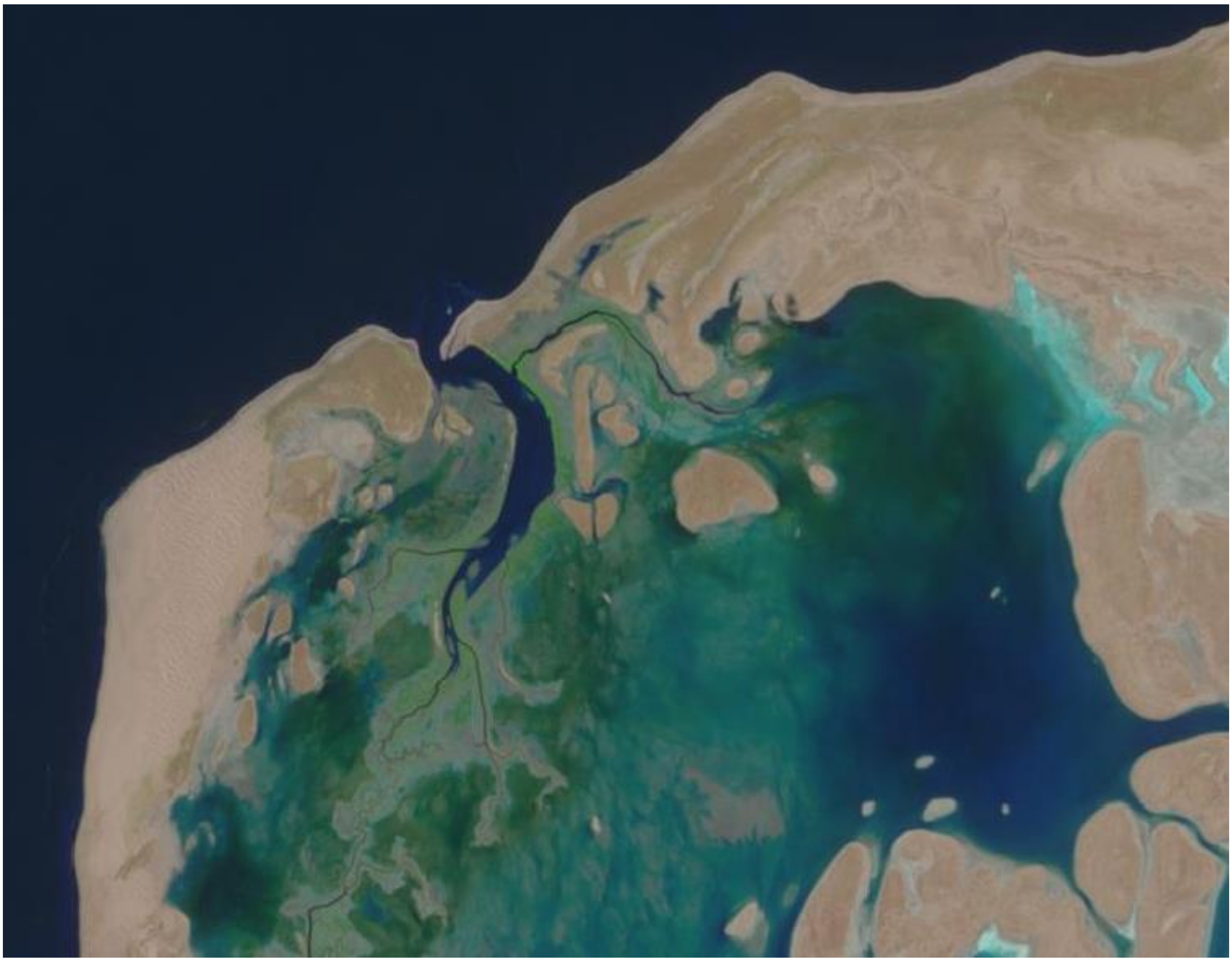


FIGURE 2-18 SENTINEL SATELLITE IMAGE - 15 APRIL 2020

2.4.2 Temperature and Salinity

The modelled temperature and salinity results were compared to measured data at Urala Creek South (seawater intake) and Locker Point (bitterns discharge), as illustrated in Figure 2-19. There was good agreement at both locations, with the model representing the daily temperature and salinity fluctuations in Urala Creek South well, however it did slightly underpredict the daily salinity range. At Locker Point (discharge/jetty site), there was significantly less daily variation, and the model captured the stable temperature and salinity well.

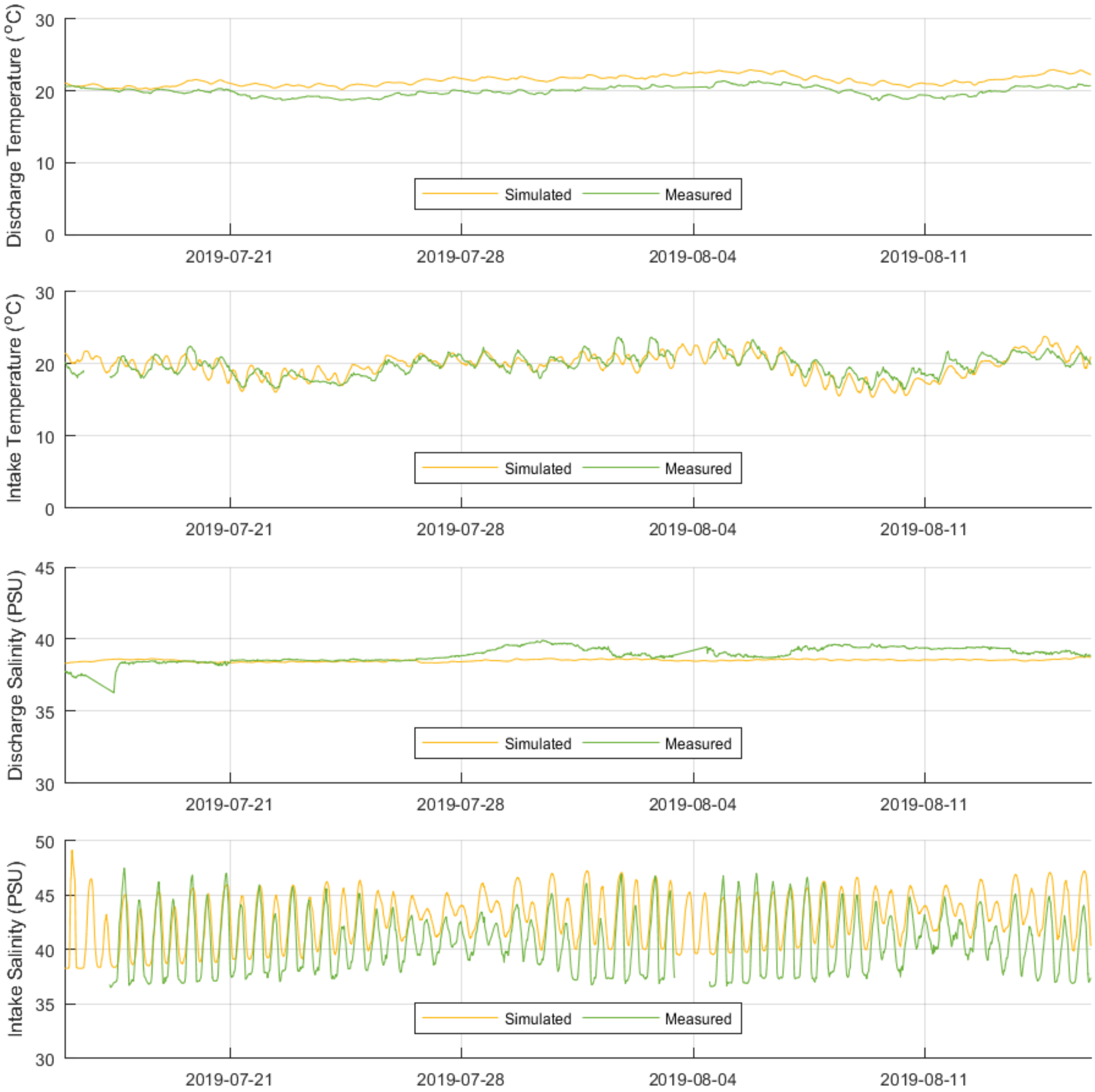


FIGURE 2-19 MODELLED AND MEASURED TEMPERATURE & SALINITY

2.5 Currents

Model calibration focused on water level, temperature and salinity as time series of nearshore currents were not available. However, a basic coastal data collection program was carried out in the vicinity of the Ashburton Salt Project site between September 2017 and November 2017. Acoustic Doppler Current Profiler (ADCP) point current data were recorded in Urala Creek North on the 13th of September 2017. Spot current magnitudes are displayed in Table 2-8 below. Visually comparing this data to the results of the 6-month hydrodynamic simulation in Urala Creek North (Figure 2-20) indicates an adequate comparison.

Table 2-8 ADCP point data collected in Urala Creek North, 13th of September 2017

Site	Transect/ Point Name	Average Velocity (m/s)
UC_N_ADCP_7	7b	0.22
UC_N_ADCP_7	8b	0.23
UC_N_ADCP_7	9b	0.33
UC_N_ADCP_7	10b	0.37
UC_N_ADCP_7	11b	0.36
UC_N_ADCP_7	12b	0.46

A field campaign undertaken by GHD in 2018 within Exmouth Gulf was utilised as an additional validation of the model results. Figure 2-21 presents a visual comparison of the cumulative current magnitude distributions at a location in the north of the Exmouth Gulf. The median current magnitudes are similar at approximately 0.2m/s, as are the 80th percentiles at approximately 0.3 m/s.

As discussed in Section 2.2.1 and 2.4.1, modelled water levels were compared to measured data at both Urala Creek South (intake location) and Locker Point (bitterns discharge location). The modelled water levels provided a good representation of water level variations. In addition, the model is capable of simulating tidal inundation over the complex intertidal zone in the project area. Given all of the above, the model is considered appropriate for use.

2.6 Limitations and Assumptions

The model study was undertaken utilising information and data available at the time. Much of the data used in this report has been captured as part of the physical monitoring program undertaken the Ashburton Salt Project. In some instances, there are limitations associated with data/information and various assumptions that have been made. These are summarised below:

- **Topography survey** – high resolution LiDAR data has been captured for this study. Review of LiDAR data shows compromised data accuracy between different flight paths and areas with dense vegetation growth, which is common for this data type. The topography survey data was corrected using a combined method of minimum correction of the above DEM inaccuracies and calibration of the tidal inundation model against available satellite imagery showing consistent maximum tidal inundation. The correction was kept to a minimum while still producing reasonable model results. In particular, the model generates inundation areas at high tide which are consistent with actual inundation observed in high tide satellite imagery (see Section 2.4).
- **Bathymetric survey** – Bathymetry surveys underwater were primarily focused on Urala Creek South main channel and Locker Point Jetty site. Sensitivity testing was used to understand the potential implications and overall, it was considered suitable for the purposes of this study. It is almost impossible to survey the numerous secondary tidal creeks and flow paths, and this is considered of limited benefit because their hydrodynamic contribution is minor with respect to the primary model outcomes.

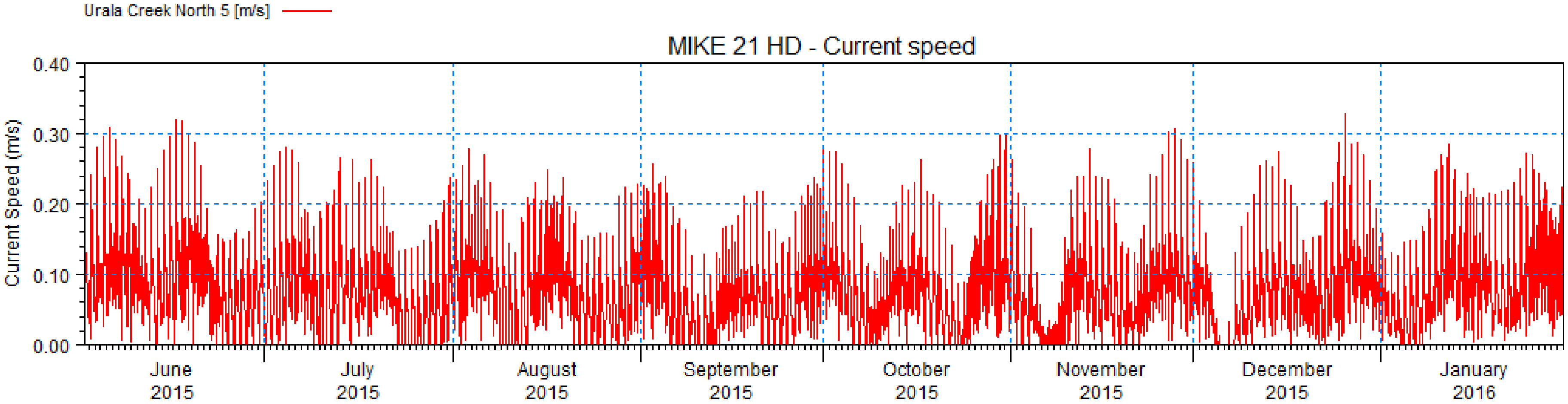


FIGURE 2-20 MODELLED CURRENT MAGNITUDE IN URALA CREEK NORTH

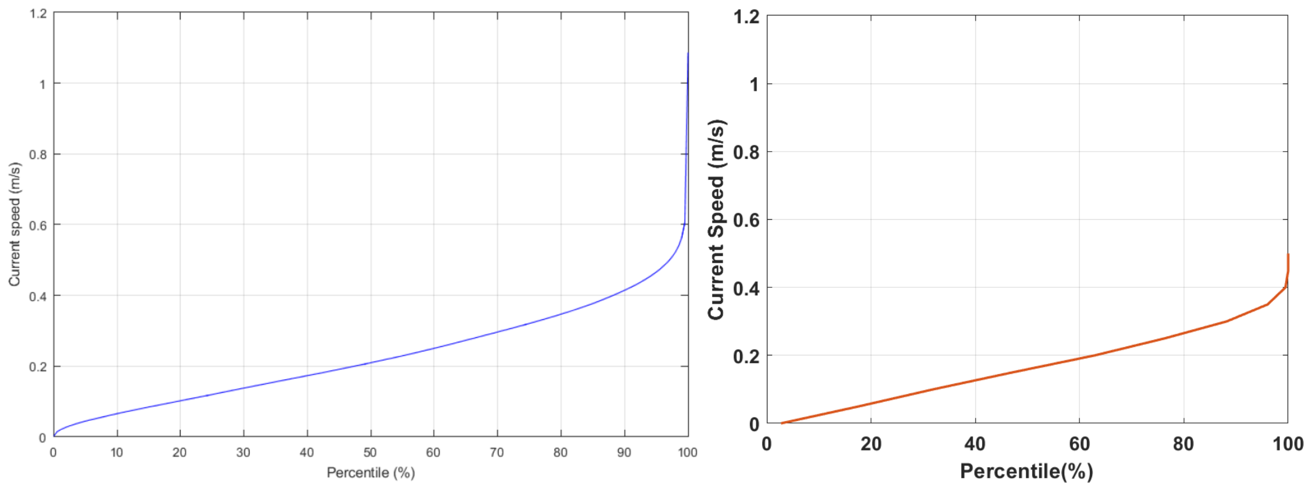


FIGURE 2-21 CUMULATIVE PERCENTILE DISTRIBUTIONS IN NORTH EXMOUTH GULF: MEAURED (GHD, 2018), AND MODELLED (WATER TECHNOLOGY)

- **Climate data** – Long term weather and climate data are available at two nearby BoM weather stations (Learmonth and Onslow) and are not directly available at the Ashburton Project site. Weather station data used in this study is considered to be representative of typical conditions on site, given the scale of the study area and the spatial variability of rainfall and weather conditions.
- **Water level and site-specific data** – water level measurements taken on site were collected to assist the calibration of the numerical models and to improve understanding of the coastal environmental conditions. The data was collected for a limited duration and as such does not represent all conditions that can occur on site. It is considered suitable for the purpose of model calibration and environmental impact assessment. It is however recommended that further benchmarking of the data should be undertaken if it is to be used for detailed engineering design or vessel navigation purposes as these studies require understanding of a wider range of conditions.
- **Currents** – Acoustic Doppler Current Profiler (ADCP) deployments were undertaken in 2017 at Urala Creek South, Urala Creek North and in the vicinity of the proposed bitterns outfall location at Locker Point. These records showed very low current flow conditions at Locker Point that has also been simulated by our Hydrodynamic Model. Time series of nearshore currents were not available. Model calibration therefore focused on water level, temperature and salinity.
- **Sediment sampling** – some sediment sampling was undertaken in Urala Creek South and other locations, and this has enabled a basic understanding of sediment properties for input into the sediment transport model. Therefore, the results of the sediment transport model are supported by some site specific information which is considered suitable for environmental impact assessment. However, a more detailed sediment sampling program and further modelling are recommended prior to detailed design of the seawater intake pumping station.

The accuracy of the outputs from a numerical model is a function of the accuracy of the underlying information, such as the topography or bathymetry and the adopted model parameters. Calibration of the model aims to minimise any uncertainty in the modelled outputs by comparing model results to measured information. The calibration process consisted of comparison between the observed and modelled parameters where possible, as well as model sensitivity testing where measured information was not available or where there was uncertainty in the accuracy of the measured data.

Overall, despite the above limitations, the calibrated model can represent the water levels, hydraulic conditions, inundation areas as well as salinity and temperature observations.

3 COASTAL PROCESSES MODELLING

3.1 Hydrodynamic Module

The DHI MIKE modelling package has been used for hydrodynamic simulations. The DHI Flow model is capable of simulating hydraulic and environmental phenomena in oceans, lakes, estuaries, bays and coastal areas.

For the coastal processes modelling all simulations were two-dimensional. This modelling is based on the numerical solution of the two-dimensional shallow water equations - the depth-integrated incompressible Reynolds averaged Navier-Stokes equations. Thus, the model consists of continuity, momentum, temperature, salinity and density equations. The hydrodynamic model is described in detail in Section 2.

To assess the impacts of the seawater intake on water levels and tidal inundation, six representative scenarios were modelled:

- Spring tide: seasonal high spring tide conditions.
- Spring tide plus sea level rise (SLR): ambient spring tide plus 0.4 m sea level rise.
- 2-year ARI: Storm event with a 2 year return interval.
- 20-year ARI: Storm event with a 20-year return interval.
- 500-year ARI Cyclone: Synthetic cyclone with a 500-year return interval so as to comply with State Planning Policy No. 2.6 – State Coastal Planning Policy (SPP2.6, WAPC, 2013).

The two spring tide scenarios were simulated to evaluate potential impacts to tidal inundation for both present-day conditions and for a sea level rise scenario of 0.4 m, which is representative of the 50-year design period. It also aligns with the 2070 planning period in SPP 2.6 (WAPC 2013). This was to assess the most critical 'day to day' inundation condition that may affect the distribution of benthic habitats. The spring tide period simulated (April 2020) was used for validation as described in Section 2.

In addition to the spring tide scenarios, three representative storm events (i.e., 2-year, 20-year and 500-year ARI), were simulated to evaluate the potential changes to inundation of mangrove habitats and salt flats from storm events. The frequent storms (i.e., 2-year and 20-year ARI) were identified as critical to assessing potential impacts to benthic communities, whilst the 500-year storm is to comply with SPP 2.6 (WAPC 2013).

The representative year (2015) was used to simulate the duration of tidal inundation, creek tidal discharge, account for seasonal variation of water levels (see Section 2.4.1.4) as well as residual impacts from the development at a yearly time scale.

TABLE 3-1 MODEL SCENARIOS

Scenarios	Existing	Development
Spring Tide	April 2020	April 2020
Spring Tide +SLR	April 2020	April 2020
2-year ARI	2012 April storm	2013 Jan storm
20-year ARI	2013 Jan storm	2013 Jan storm
500-year ARI Cyclone	Synthetic Cyclone 1985	Synthetic Cyclone 1985
Representative Year	2015 Jan to 2015 Dec	2015 Jan to 2015 Dec

3.2 Sand Transport Module

DHI MIKE hydrodynamic module was coupled with the Sand Transport (ST) module to undertake sediment and coastal morphology modelling. DHI's MIKE ST module is a state-of-art model tool capable of simulating the erosion, transportation and deposition of sediments in the marine environment. The ST model was used to investigate morphology evolution in Urala Creek South only. The jetty site is driven by surf zone coastal processes such as littoral drift, which cannot be accurately estimated by ST model.

The overall modelling framework is conceptualised below.

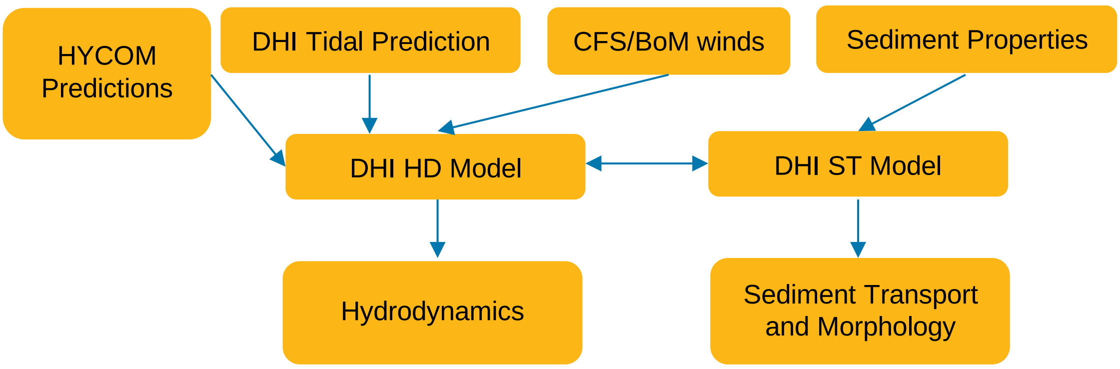


FIGURE 3-1 MODEL FRAMEWORK

The ST module is driven by currents with consideration of helical flow which is generated via bends in channels. Wave forcing is not considered, given the negligible wave energy in the creek.

The erosible layer thickness is the bed material thickness applied throughout the model domain. This thickness must be large enough for erosion to occur but not so great that unrealistic bottom erosion is predicted near model boundaries and along intertidal areas.

Sand grain sizes are determined based on the assumption that the bottom shear stress exceeds the critical shear stress of sand (shield diagram) at medium flood tide. A minimal grain size of 0.2 mm and a maximal grain size of 2 mm are applied to limit the uncertainty of calculation.

The year 2015 was identified as a representative year and was used to simulate the yearly fluvial morphology evolution in Urala Creek South.

3.3 Model Results

The model results are presented in three sections:

- Tidal Inundation: This section focuses on the changes to water levels and inundation due to the development layout, in particular inundation changes to intertidal and supratidal areas around Urala Creek North and South.
- Urala Creek South: This section focuses impacts related to the seawater intake (including pump station and embankments), and residual impacts from the development layout.
- Locker Point: This section focuses on impacts on coastal processes related to the proposed jetty.

3.3.1 Modelled Changes to Inundation of Overall Project Layout

The inundation pattern over the tidal flat was simulated for both existing and developed conditions for the overall project layout for a yearly spring tide maximal inundation (April 2020) as well as 2, 20 and 500 Average Recurrence Interval (ARI) storm events including a combination storm surge and rainfall. Results are presented in Table 3-2:

- Left column for existing conditions.
- Middle column for developed conditions.
- Right column for the difference plot between existing and developed conditions.

Existing Conditions

During yearly spring tide maximal inundation, water floods into the low intertidal areas through Urala Creek North and Urala Creek South. The area of tidal inundation increases while the water level approaches high tide along the course of the creeks. It takes a few hours for the water to move through the creek sub-channels and flood onto the tidal flat. The maximum area inundated over the tidal flat is observed at about 4 hours after high tide, showing a clear delay relative to the timing of high tide. The landward extent of the flooded area is limited by the volume of water flow into the creek (which is dependent on the tide level).

As the tide drops, water levels nearshore and in the creeks are lowered and water begins to drain back off the tidal flats via multiple drainage channels (sub-creeks). Approximately 10 hours after high tide most water has drained back during the receding tide, although remnants of water may be temporarily retained in isolated temporary ponds which are subject to high levels of evaporation.

Inundation levels over the tidal flat are somewhat lower than the water level nearshore. This can be attributed to the land barrier on western side of Urala Creek North and South and head loss through roughness of the terrain.

The coastal terrain consists of high grounds (dunes) which are not inundated even during severe cyclones e.g., a 500-year cyclone event (about 3 m MSL storm tide) and large areas of low-lying flat land which are inundated by more regular storms and tides. Simulation results indicate that water inundates the tidal flat through multiple pathways:

- A tidal channel to the north of and connecting to Urala Creek North (subject to regular tidal inundation).
- Water flooded out of the creeks from multiple low points along the course of both Urala Creek South and Urala Creek North and onto the surrounding land (inundated at high tide and by regular storms).
- Multiple secondary tidal channels (sub-creeks) and their terminal ends on the tidal flats (inundated by tides and storms).
- Flood water from further south flushed towards the Project site (inundated only during extreme storms).

Post-development Conditions

Model results show no significant changes in the natural pattern of tidal inundation under seasonal spring high tide for post-development conditions.

Most of the project embankment walls are positioned beyond or near the upper limit of tidal inundation. The embankments have only minor predicted impacts within or proximal to the proposed development footprint.

In particular, the crystalliser ponds and bitterns pond are located in an area which is inundated during extreme spring tides (infrequent seasonal high water in March and April) for a very small depth of typically 30 cm or less. The construction of the embankment walls in these areas will result in some minor backing up of tidal inundation water in extreme spring tides with a localised depth difference of approximately 5 cm for a short period of only 4 hours.

Similarly, the western (seaward side) evaporation pond embankments are located in an area which is inundated in extreme spring tides (infrequently in March and April) with a depth of typically 50 cm or less. The construction of these evaporation pond embankment walls will result in some minor backing up of tidal inundation water in extreme spring tides with a localised depth difference of approximately 5 cm.

The seawater intake channel embankments show some limited impacts primarily confined to backing up of water adjacent to the embankments. It is also expected that water pumping could marginally lower the water level and modify flow condition in Urala Creek South particularly at low tide which is discussed in more detail in the Section 3.3.2 and Section 3.4.2. The intake channel embankment may cause temporary ponding of seawater against the physical barrier it creates. This ponding of water may last over the duration of the full-tidal cycle however could be mitigated by natural processes such as evaporation and infiltration as well as drainage paths (either natural or constructed).

Whilst the changes mentioned above in water levels are for extreme spring tides (in March and April each year), it should be noted that smaller changes are expected during regular spring tides for the remainder of the year.

The crest level (3.5 m AHD) of embankments is above the storm tide level of a 500-year cyclone. The area within the pond embankments will no longer be affected by external tide and storm water.

For storm events (including rainfall) a change of about 10 cm is predicted for a 2-year event, about 20 cm during a 20-year event and about 50 cm during a 500-year event.

Overall, inundation over the tidal flat is only weakly affected by the project, primarily in areas in close vicinity of the embankment walls with shallow and temporary ponding immediately adjacent to the embankments.

For the main habitats such as the algal mat and mangroves, the predicted changes in water level are in order of centimetres which is unlikely to affect the existing inundation pattern and duration of these habitats.

In summary:

- Under extreme spring tidal conditions (March/April) the modelling predicts some localised ponding on tidal flats immediately next to embankment walls. The duration of ponding may vary by site and ponding is of a temporary nature rather than permanent. Also, the model may overpredict the ponding as water infiltration and fine scale drainage paths were not simulated.
- Under extreme spring tidal conditions (March/April) there are no predicted changes to inundation in the mangroves and the majority of algal mat areas with the exception of some minor (and temporary) ponding in localised algal mat areas immediately adjacent to embankment walls.
- Under normal spring tidal conditions (months other than March/April) impacts are expected to be smaller than those in extreme spring tidal conditions.

Sea Level Rise

Sea level rise has significant impacts to the tidal inundation pattern of the area. With future sea level rise (a 0.4 m sea level rise is projected during the project life of 50 years, according to WACP 2013), the entire tidal flat will be inundated at high tide.

Under the sea level rise scenario, at lower tide, some inundated areas will be drained while main tidal channels will be inter-connected resulting in a wider channel when compared to present day conditions.

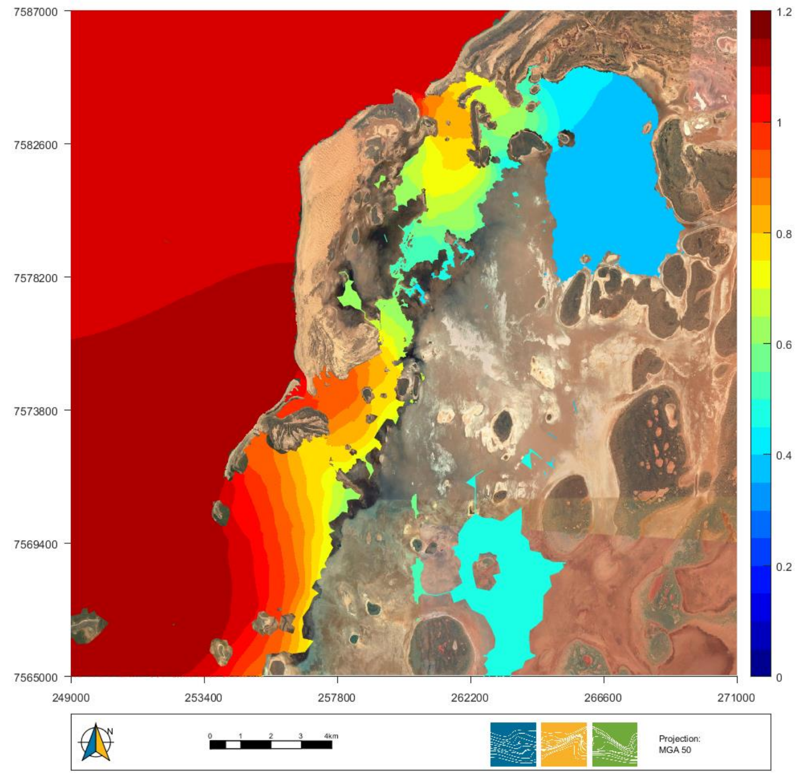
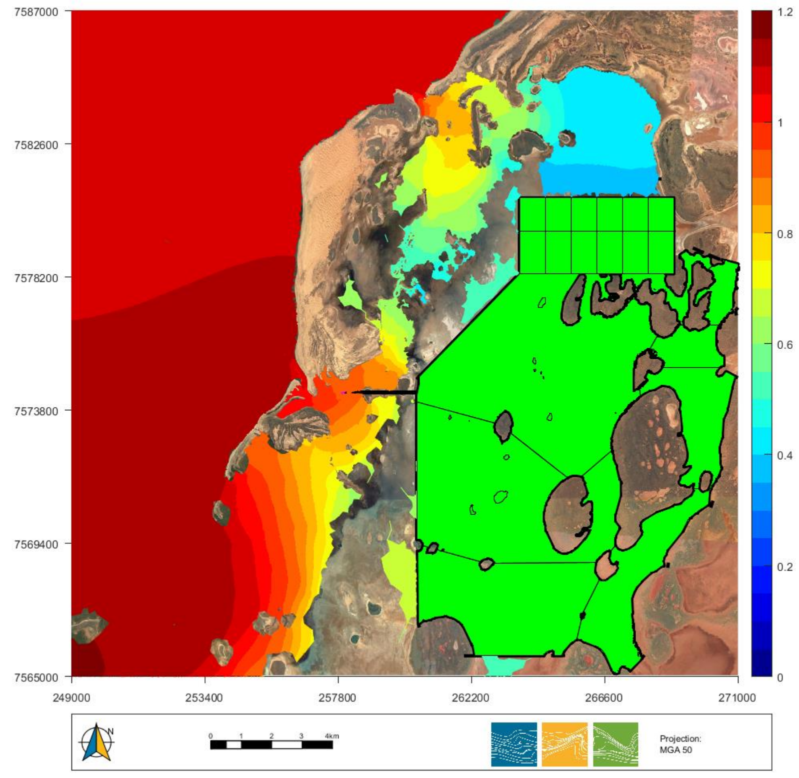
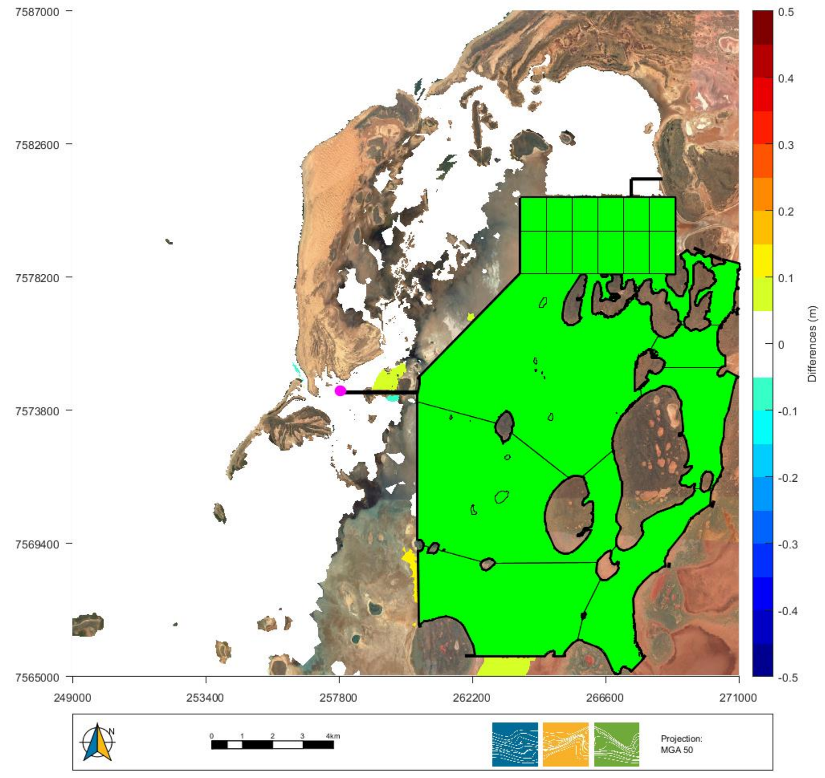
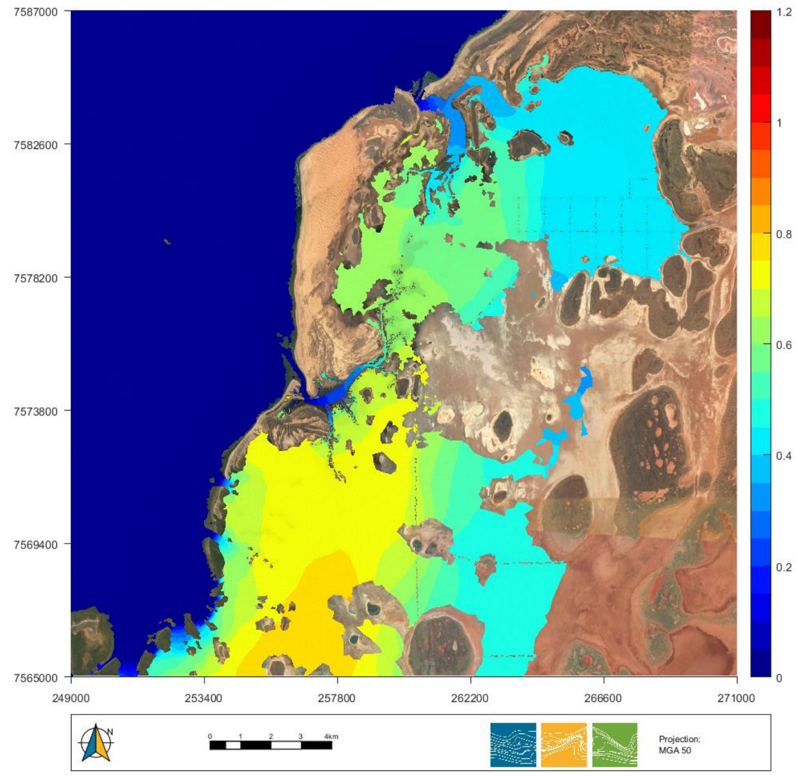
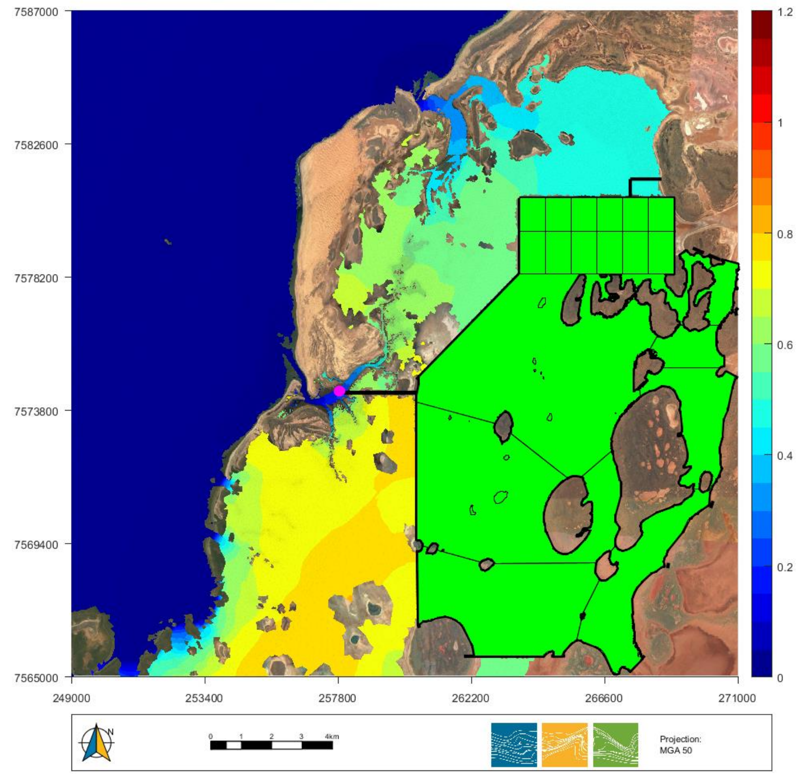
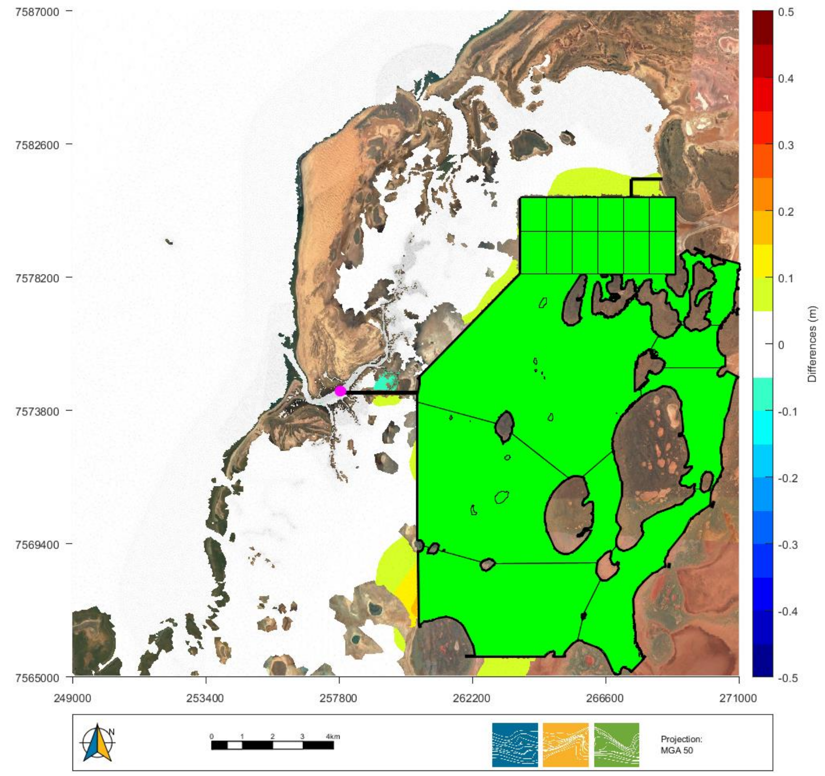
For the sea level rise scenario, the proposed development contributes to up to 10 cm water level increase over the tidal flat which is similar in magnitude to the spring tide scenario.

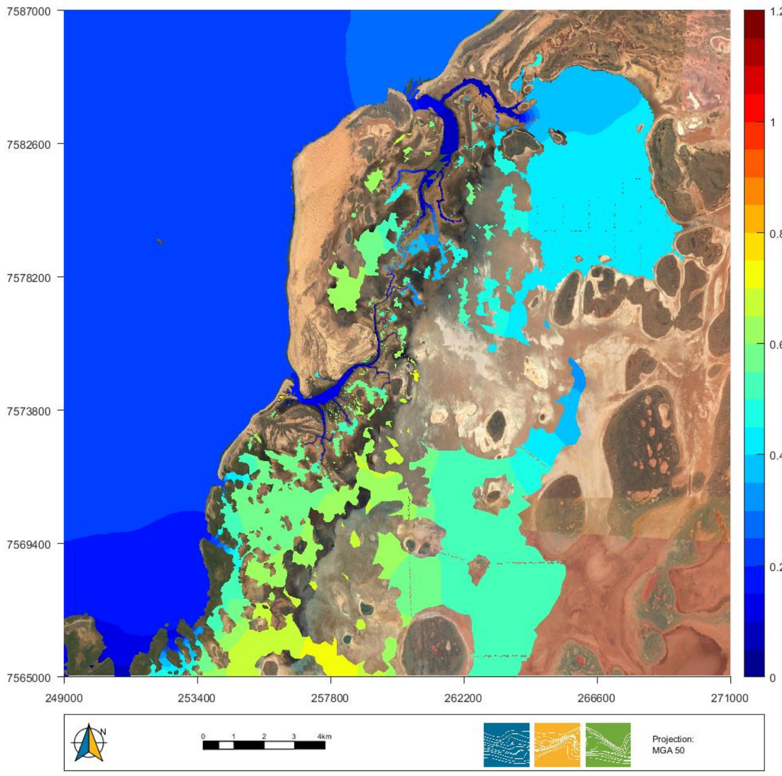
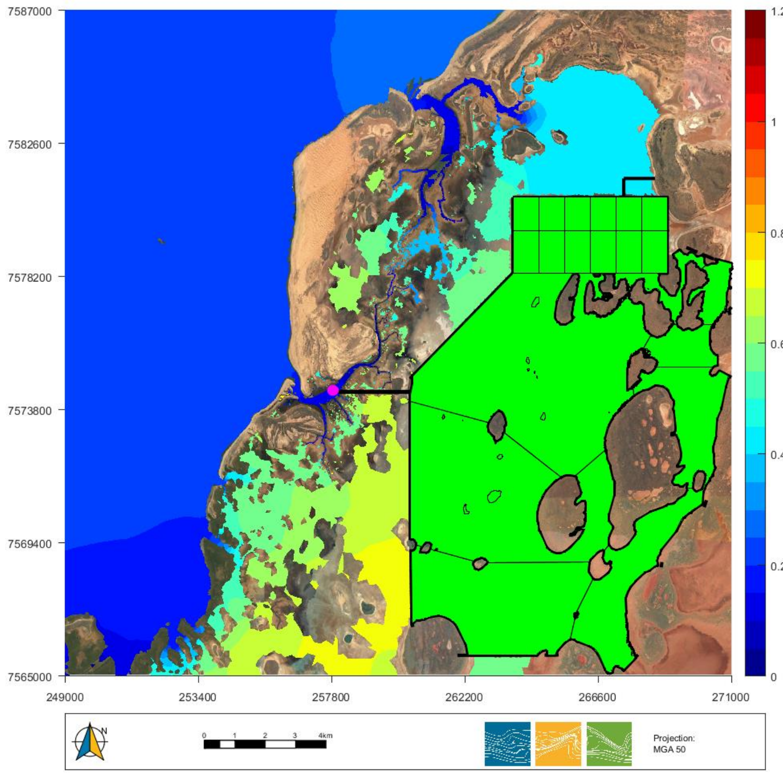
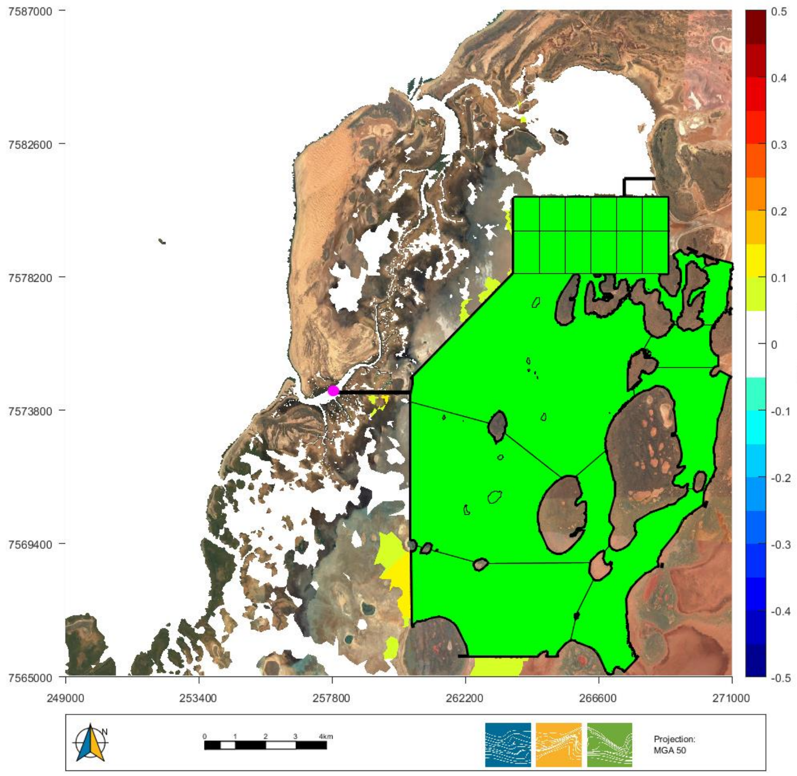
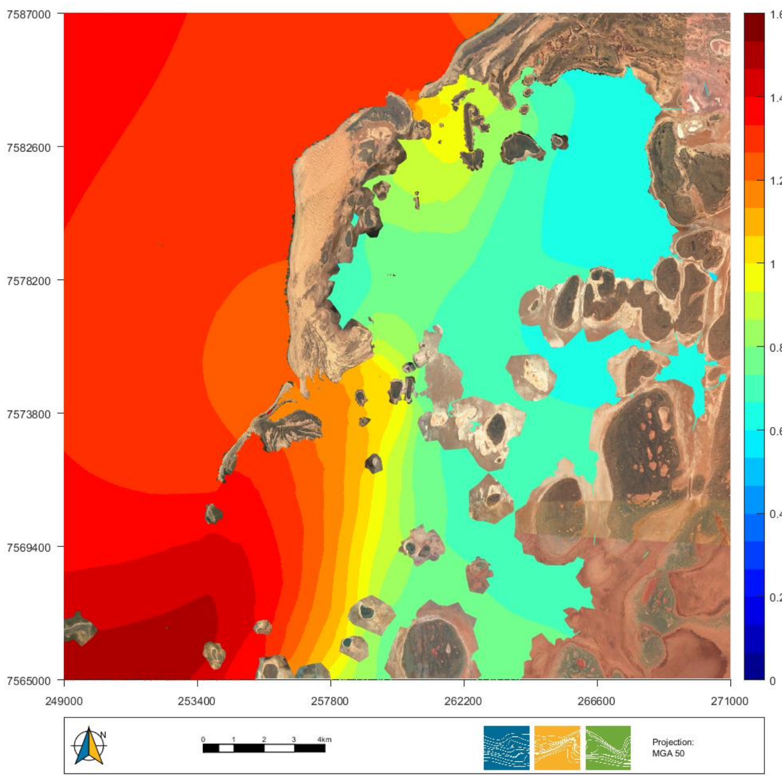
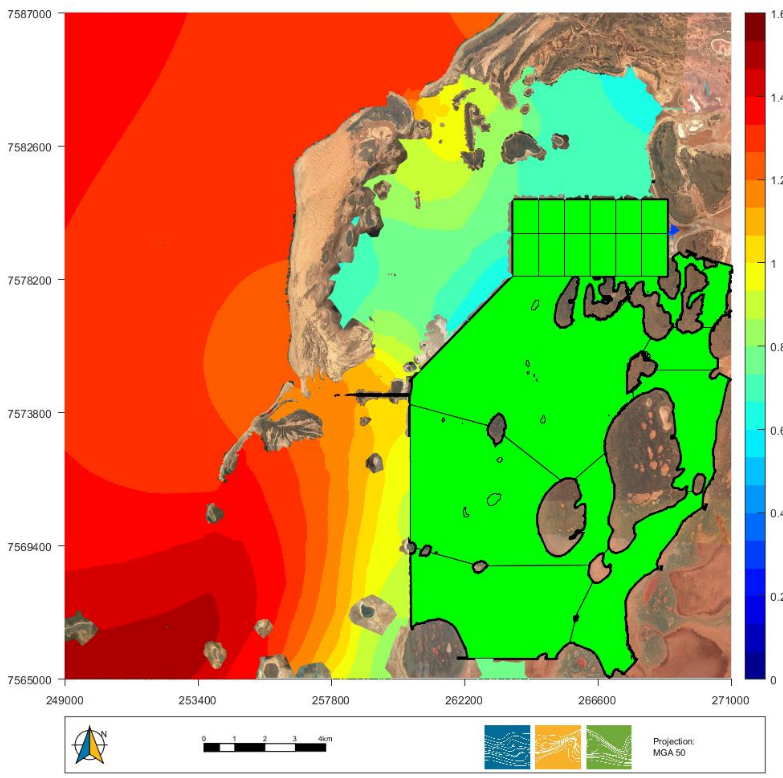
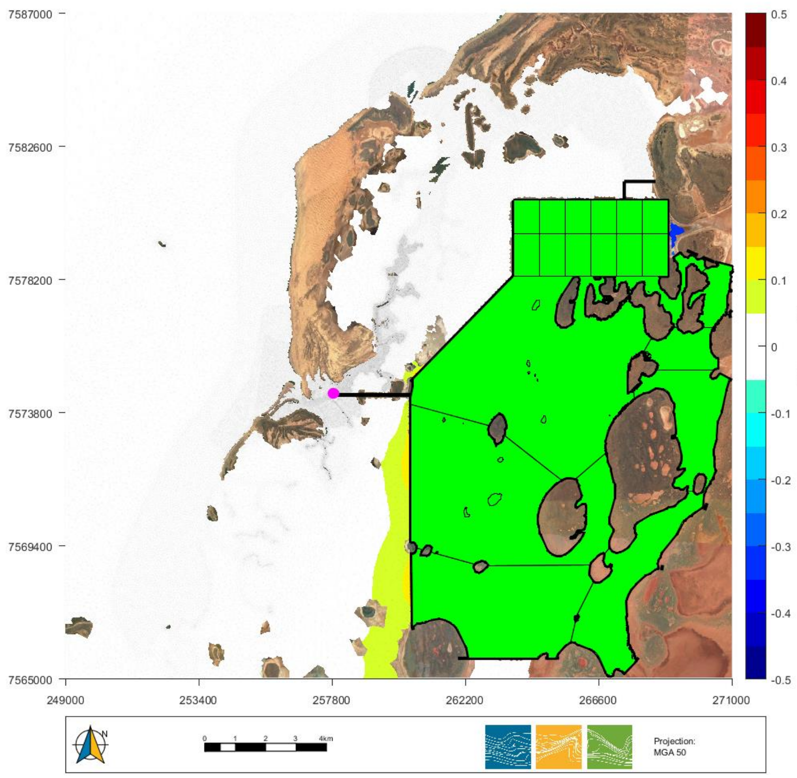
The impacted area is somewhat larger due to greater impacts from the embankment walls which are located within the reach of tidal flows which reach further inland due to sea level rise.

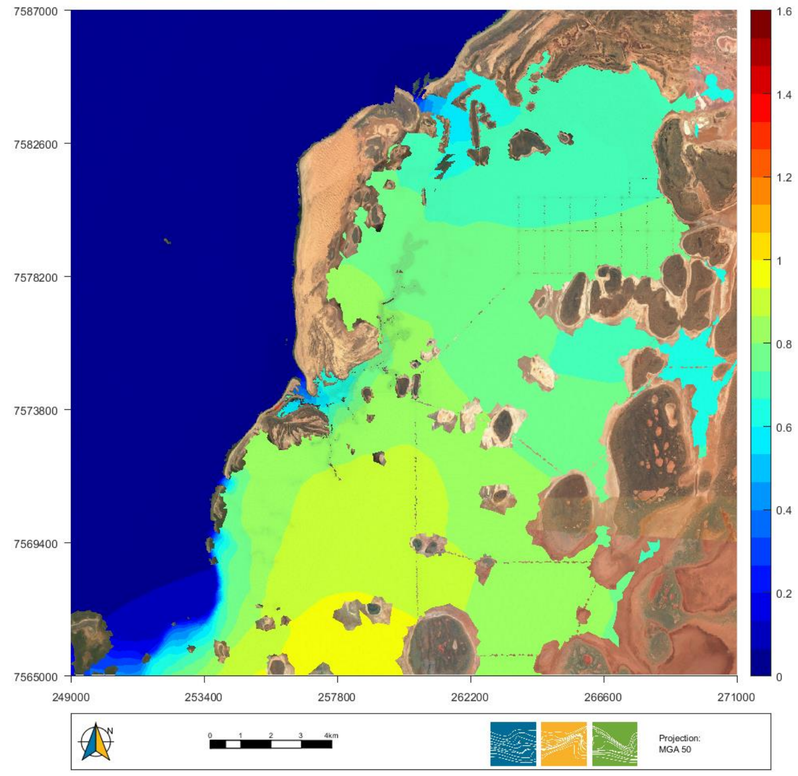
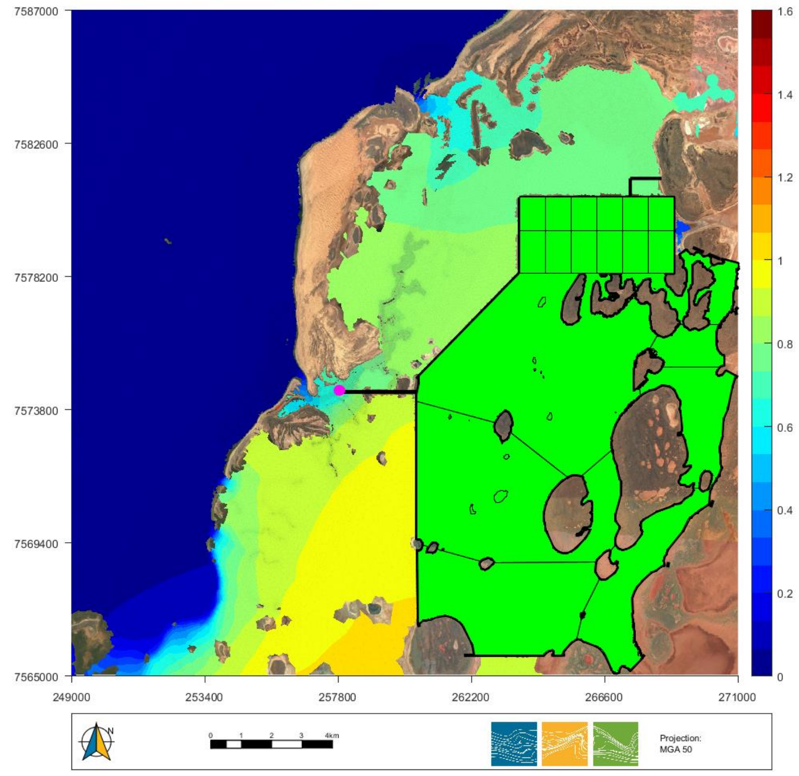
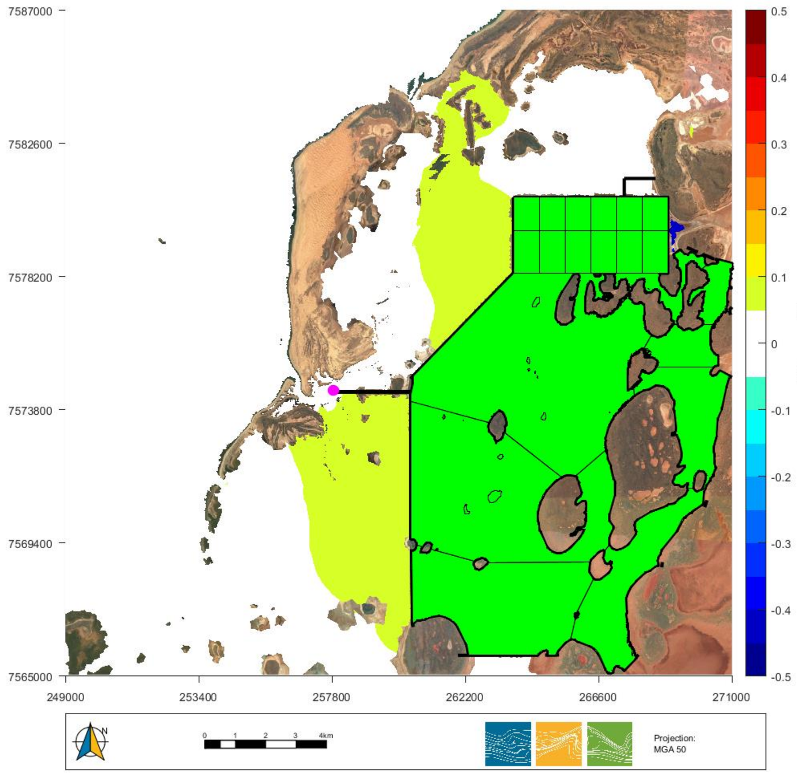
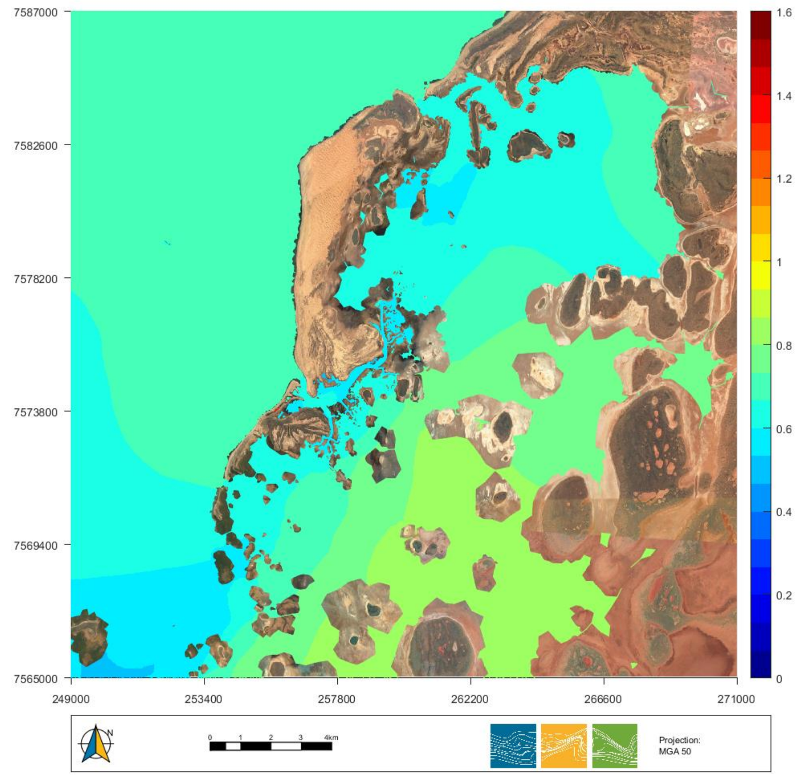
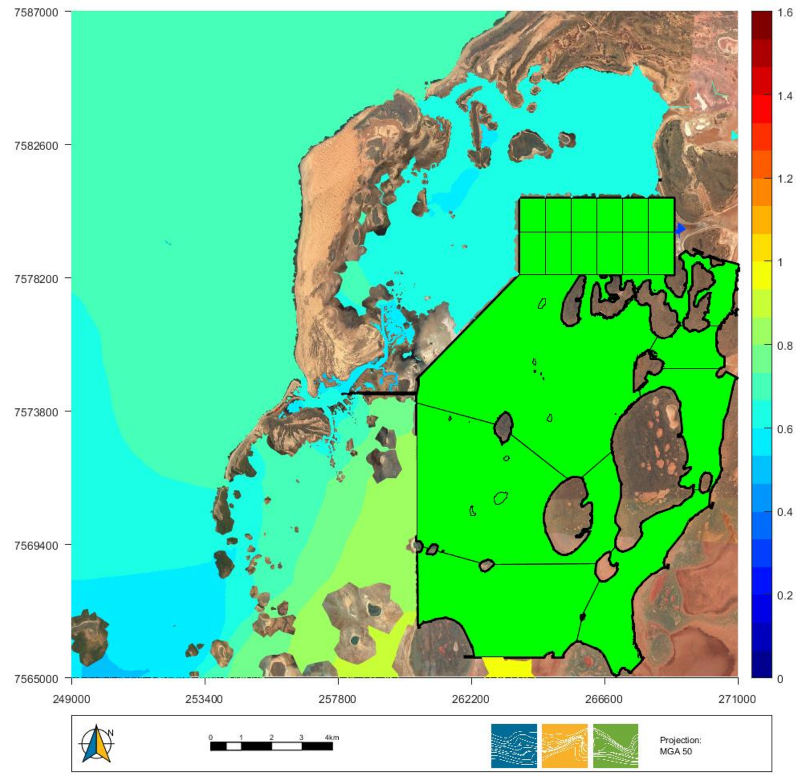
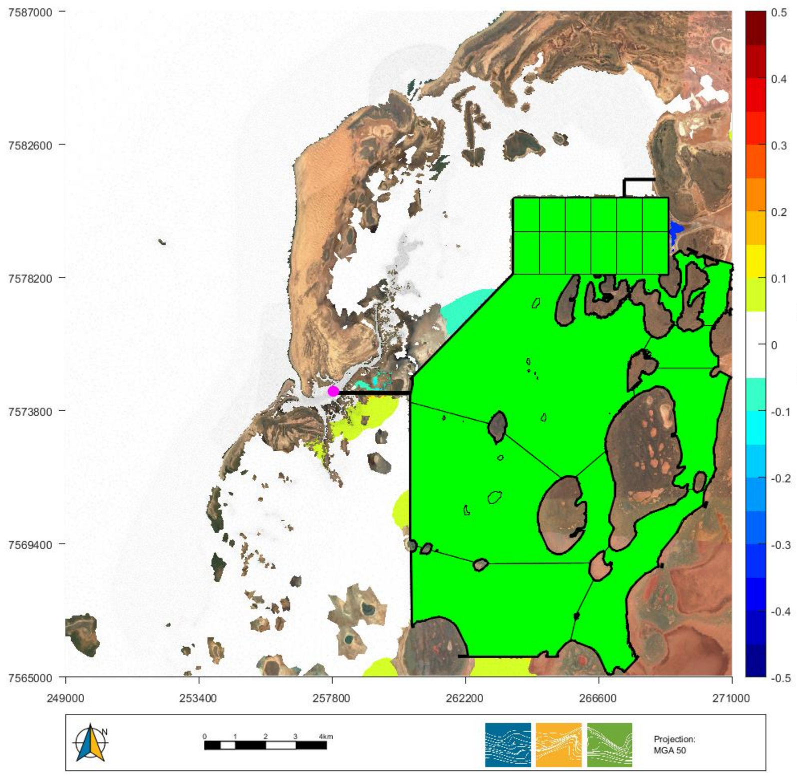
Assessment of model results suggests sea level rise appears to alleviate the impact from the seawater intake while weakly enhances the impact from embankment walls.

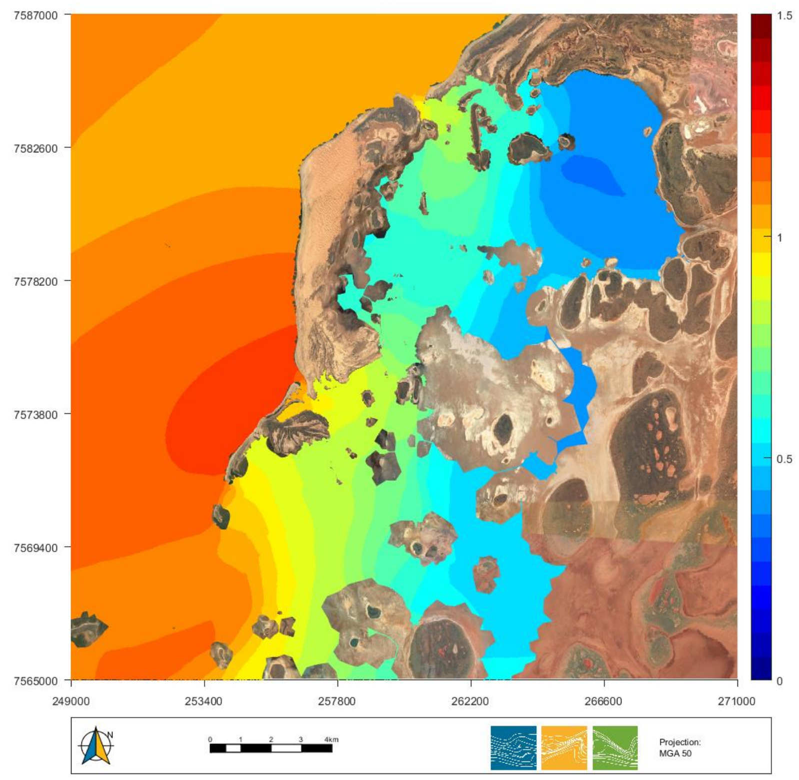
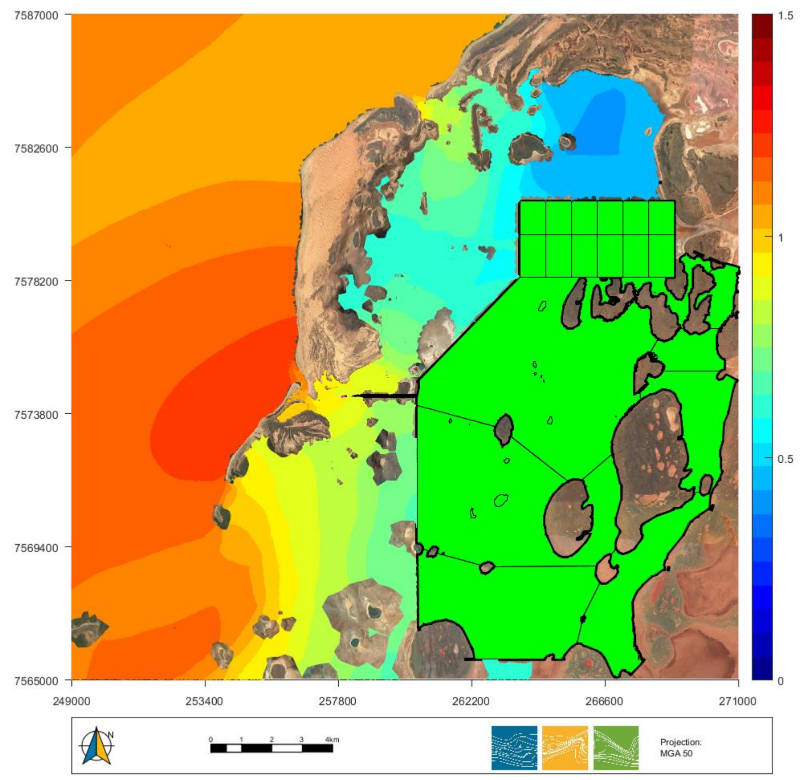
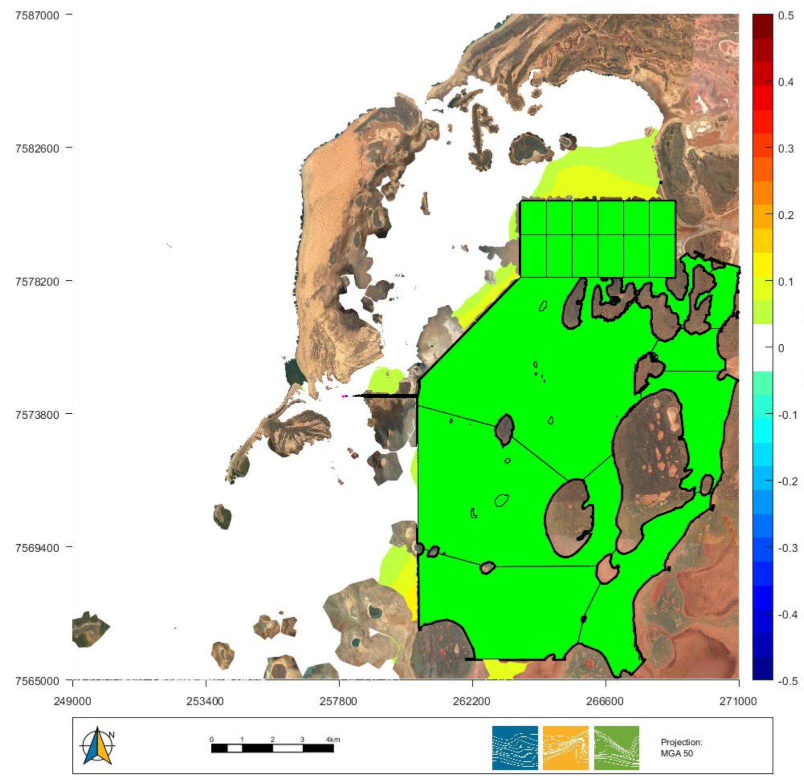
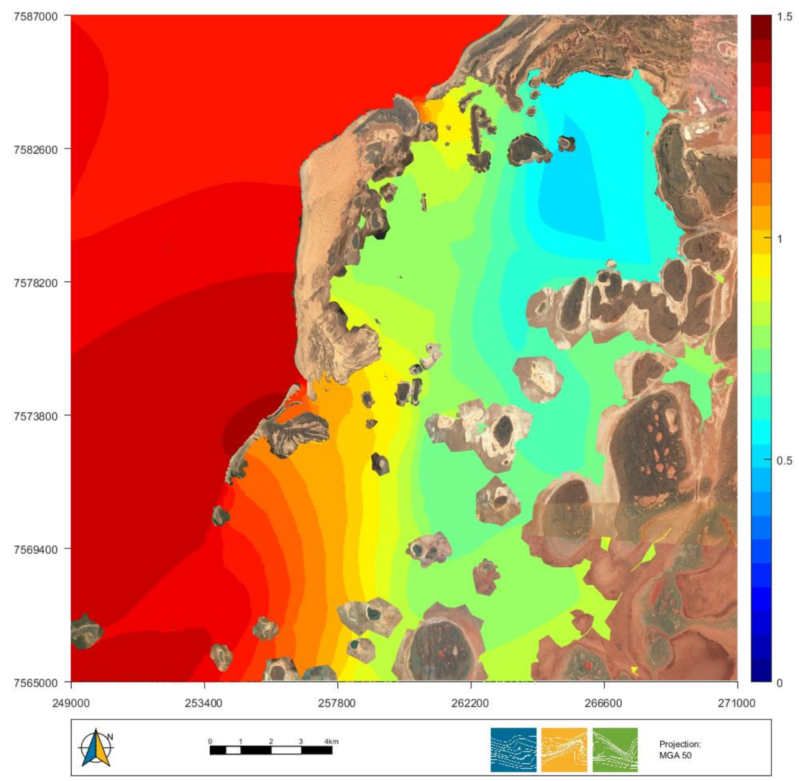
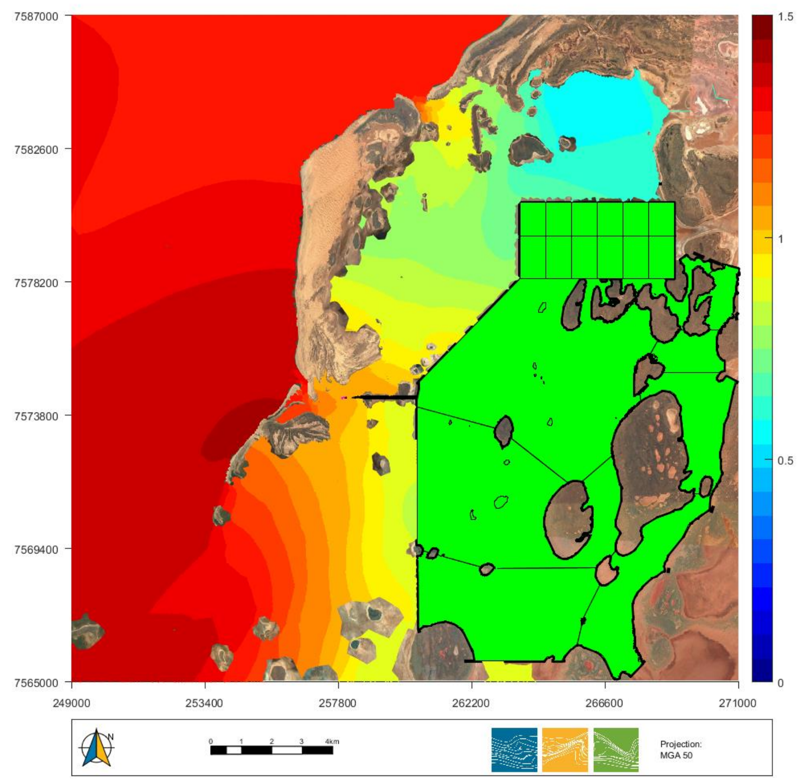
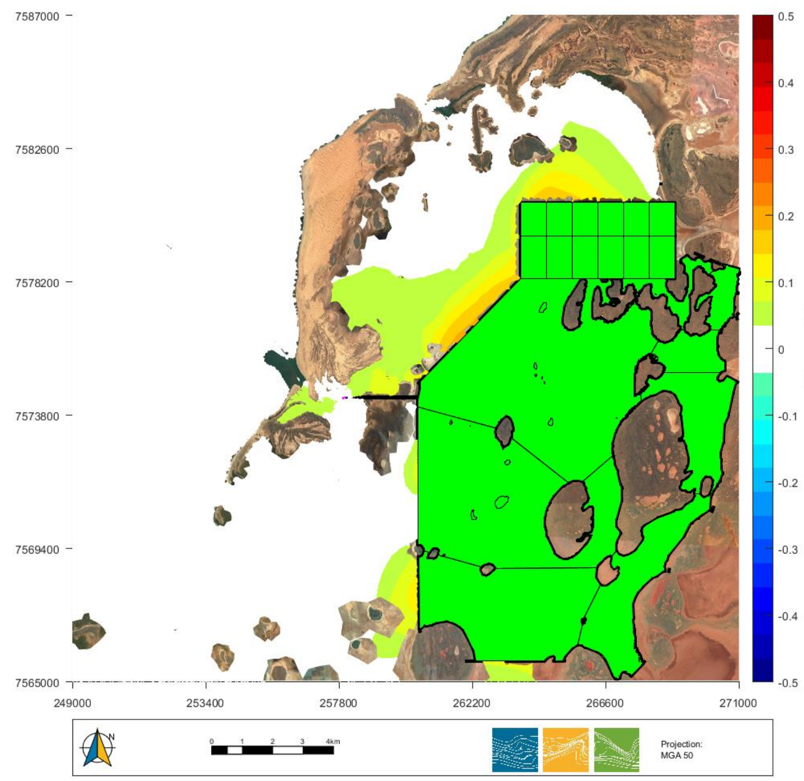
Sea level rise itself also has far greater impact to the existing coastal environment (40 cm water level increase over the entire region) than the proposed development (less than 10 cm localised water level increase near the embankment walls).

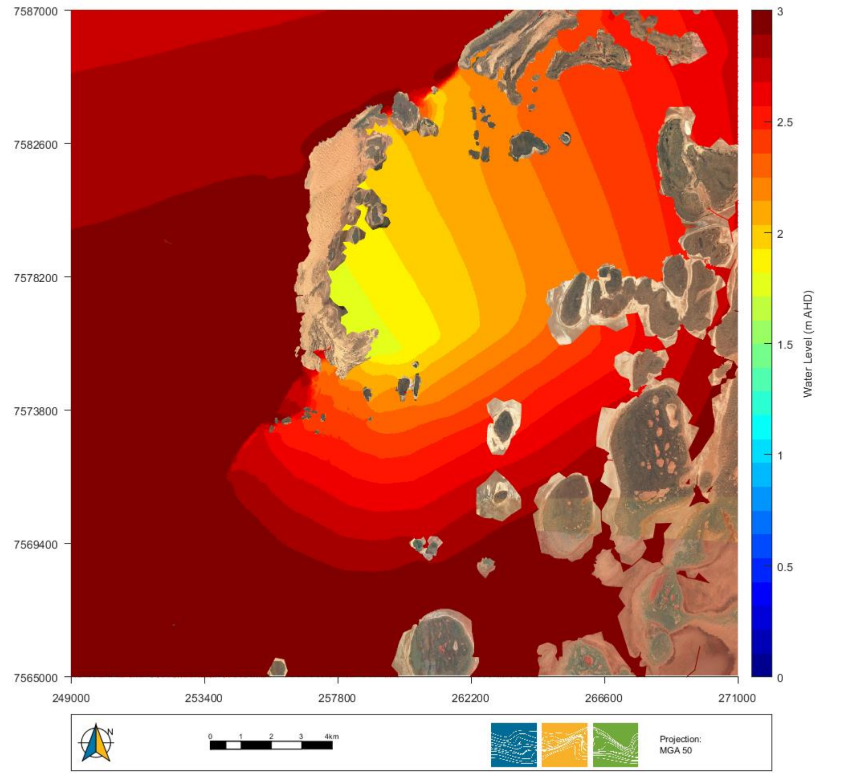
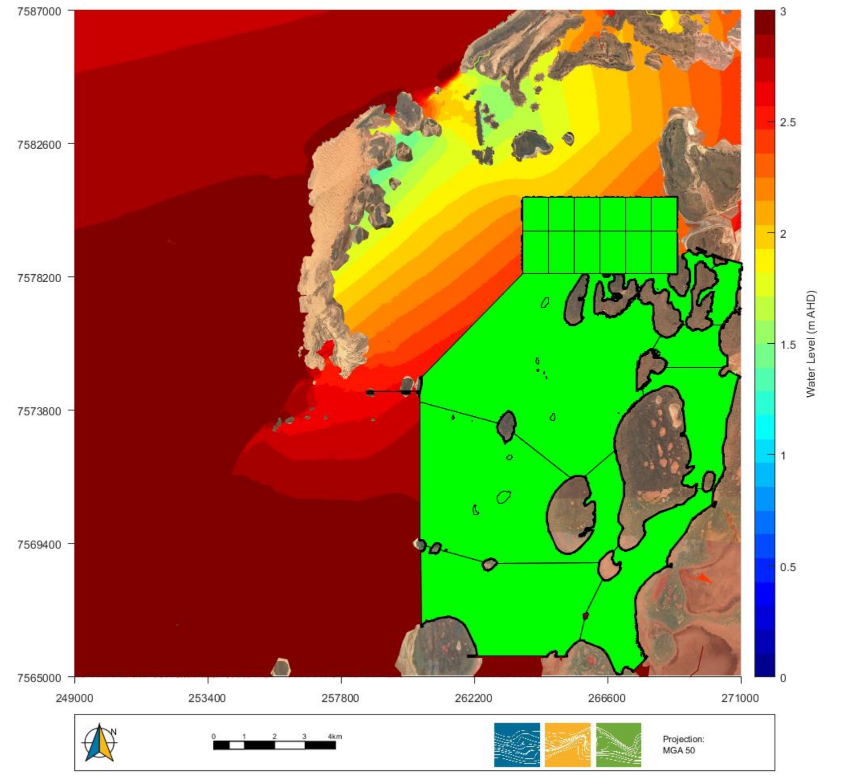
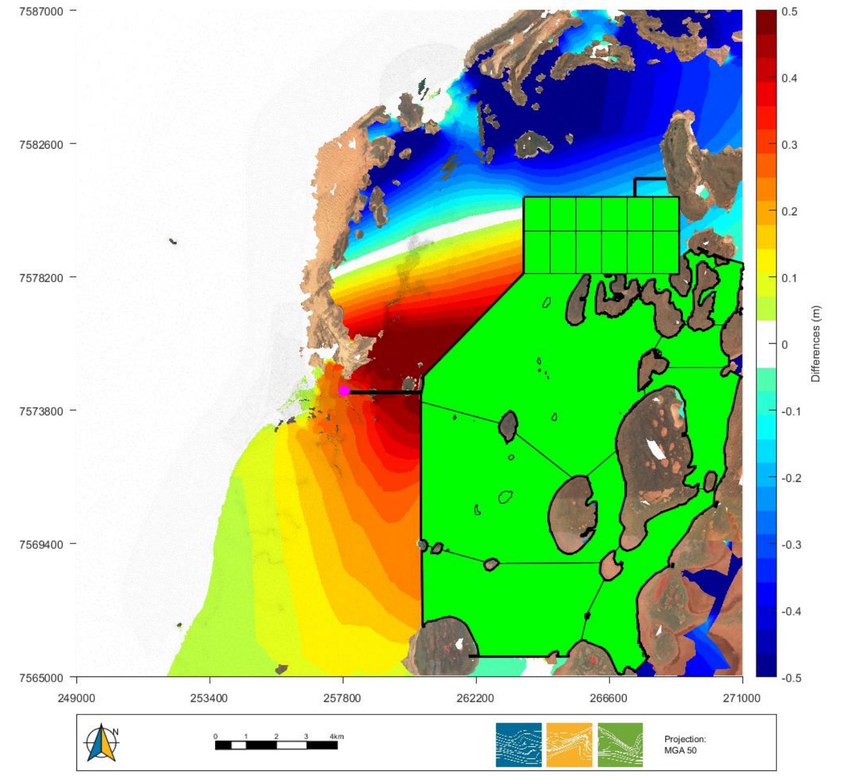
TABLE 3-2 MODELLED INUNDATION ON TIDAL FLAT

Scenarios	Existing	Development	Differences (Developed – Existing)
Spring Tide (2020-4-11 4:30)			
4 hours after high tide (2020-4-11 8:30)			

Scenarios	Existing	Development	Differences (Developed – Existing)
8 hours after (2020-4-11 12:30)			
Spring Tide (2020-4-11 4:30) + SLR			

Scenarios	Existing	Development	Differences (Developed – Existing)
4 Hours After High Tide (2020-4-11 8:30) + SLR			
8 Hours After High Tide (2020-4-11 12:00) + SLR			

Scenarios	Existing	Development	Differences (Developed – Existing)
Maximal Inundation 2-year ARI Storm			
Maximal Inundation 20-year ARI Storm			

Scenarios	Existing	Development	Differences (Developed – Existing)
Maximal Inundation 500-year ARI Storm			

3.3.2 Modelled Hydrodynamic Changes to Urala Creek South

For Urala Creek South water level and tidal current modelling, a high level of conservatism has been adopted by modelling a worst case scenario as follows:

- The extreme spring tide scenario (representative of March/April) was modelled to capture the highest tidal inundation
- The highest seawater intake pumping rate (which occurs in November/December due to highest evaporation levels) was assumed to occur concurrently with the extreme spring tide scenario even though pumping rates would be significantly lower in March/April than in November/December
- It was assumed that the seawater intake would pump water at a constant rate throughout the tidal cycle (at both high and low tide), however it is planned to cease pumping at low tide

For the fluvial morphology (sand transport) modelling, the variable intake rate predicted over the year was used (given the sand transport model was required to run for the full year). However, the assumption was still conservative given constant pumping at both high and low tide, whereas the project does not plan to pump at low tide.

This high level of conservatism has been employed to determine if the modelled impacts of the project are acceptable on a conservative modelling basis, as this will enable assessment of environmental risk.

3.3.2.1 Water Level

The spatial distribution of modelled high and low water, including differences between pre- and post-development, are presented in Table 3-3:

- Left column for existing conditions.
- Middle column for developed conditions.
- Right column for the difference plot between existing and developed conditions.

Existing Conditions

For the extreme high spring tide scenario in April, the main tidal channel, mangrove habitats and tidal flat in Urala Creek South are flooded. The inundation reaches further upstream until connecting to Urala Creek North.

For the April low tide, water level in the main tidal area downstream is drained to be lower than -1 m Mean Sea Level (MSL). The model predicts some small tributaries to be dry and disconnected from the main creek channel, which remains inundated by a shallow depth.

Greater inundated areas were simulated during more extreme storm events when almost all areas with a low gradient are inter-connected by floodwater.

Post-development Conditions

The maximum intake rate in November was used to determine changes to Urala Creek South water levels.

The seawater intake shows minimal impact to April (extreme) high tide levels in the Urala South Creek channel itself for all modelled scenarios. The predicted changes were in the order of centimetres (less than 5 cm during an extreme April spring tide and less than 20 cm during all tidal phases).

The seawater intake primarily affects Urala Creek channel water level during a spring low tide. The model prediction shows a moderate impact (approximately 10 cm to 20 cm decrease in water level) during the spring low tide levels in Urala Creek South channel. These changes are observed directly above and downstream of the intake.

Inundation/flooding over the tidal flat is affected mainly by embankment walls as discussed in Section 3.3.1. According to the model results, up to a 10 cm water level increase is anticipated against the seaside embankment walls and along the intake channel embankments connecting the evaporation pond to the seawater intake pumping station, during an extreme (April) spring high tide and a 2-year storm.

At April spring high tide, there is backing up of water on the northern side of the channel in the order of 10 cm and a reduction in water level on the southern side of the channel also in the order of 10 cm. This is due to the physical barrier effect of the seawater intake channel embankments which impede the flow of tidal waters in a southerly direction during the incoming tide.

Conversely, at April spring low tide, there is a backing up of water on the southern side of the seawater intake channel in the order of 10 cm and a reduction in water level on the northern side of seawater intake channel within a sub-creek which drains the tidal flat into Urala Creek South in the order of 10 cm. This is due to the physical barrier effect of the seawater intake channel embankments which impede the flow of tidal waters in a northerly direction during the receding of tide.

This barrier effect of the seawater intake channel is localised and temporary given daily evaporation rates in the area are high, and any ponding evaporates within days. New drainage paths may be formed by natural adjustment of morphology due to tidal flushing along the toe of embankment wall.

The impact to water level may exceed 20 cm for a 20-year storm and 60 cm for a 500-year storm indicating greater impact at higher water.

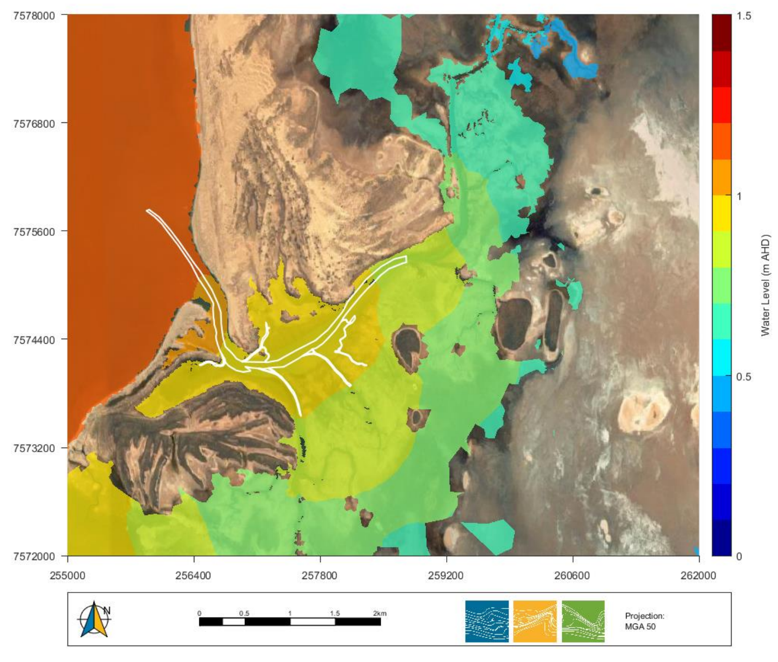
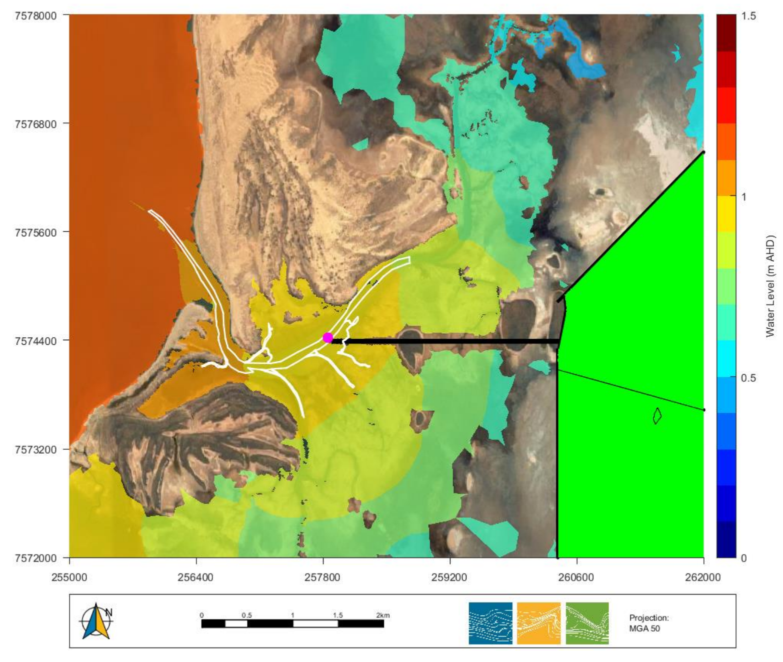
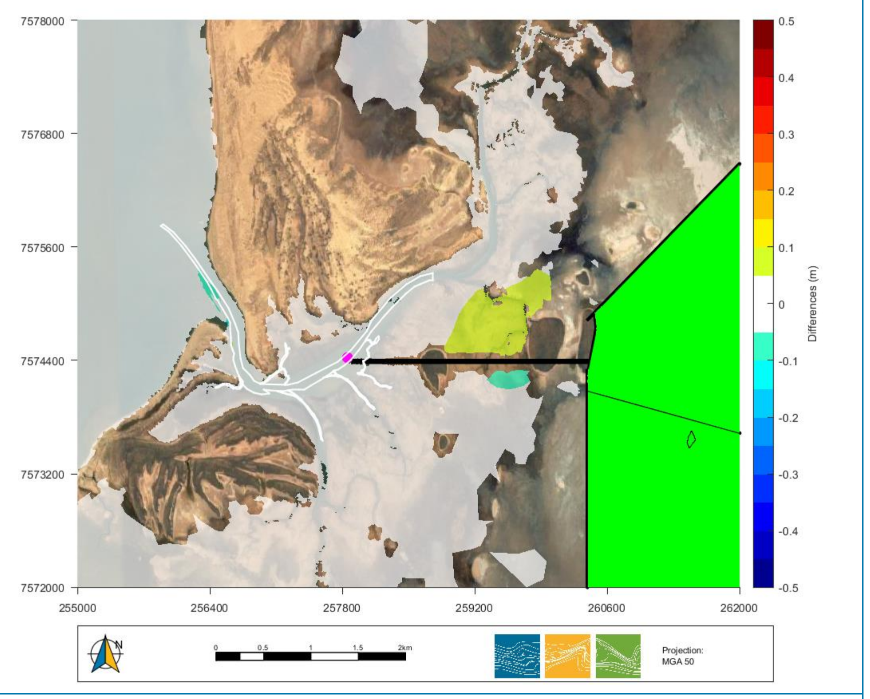
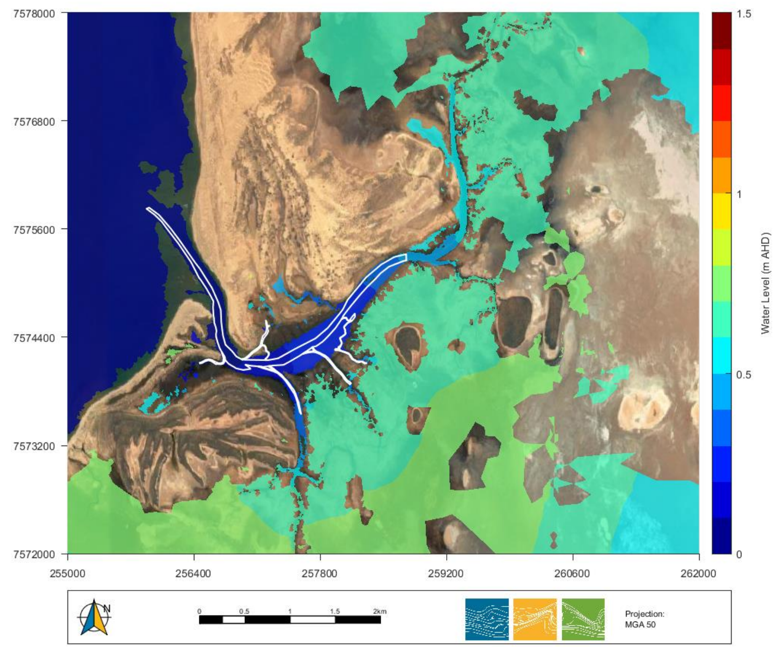
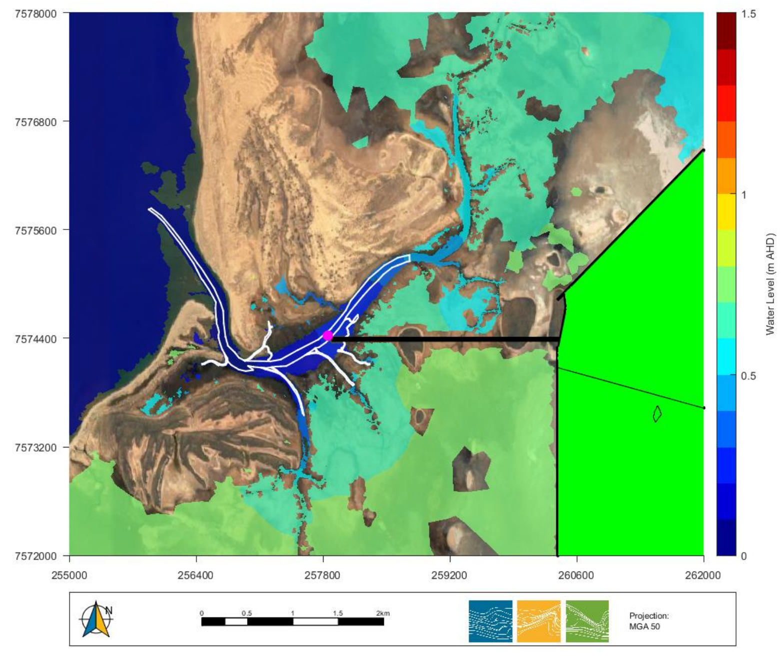
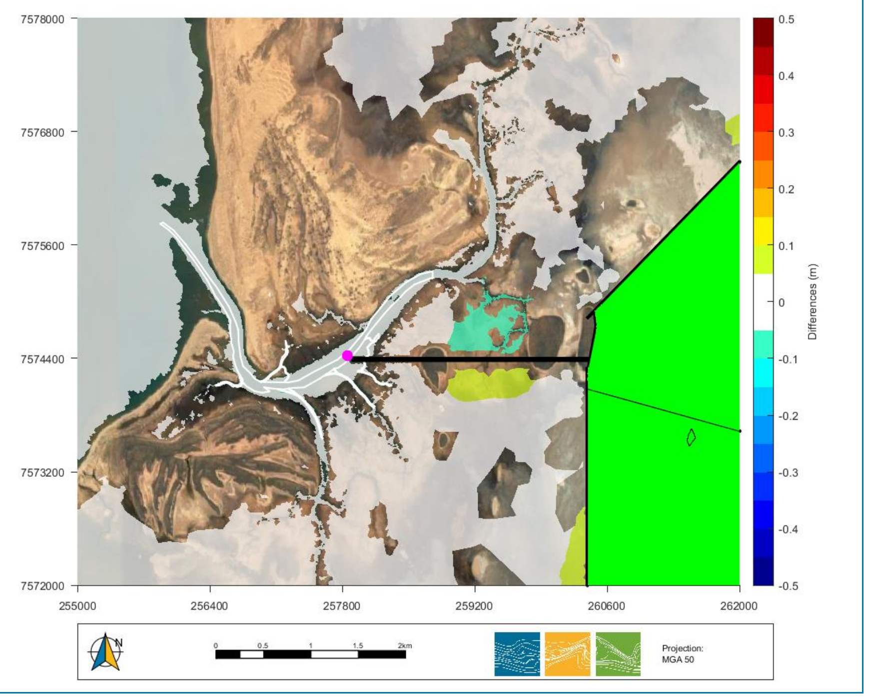
Overall, inundation is only weakly affected by the project. In summary:

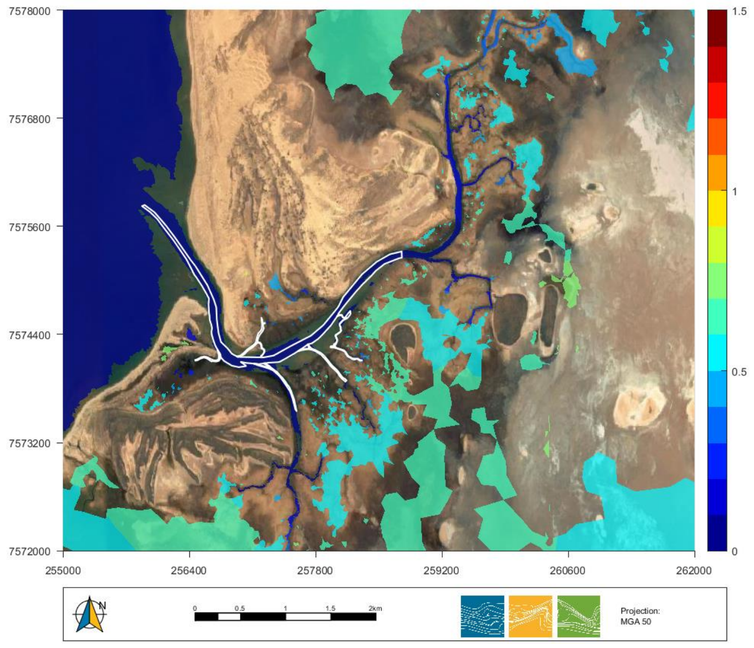
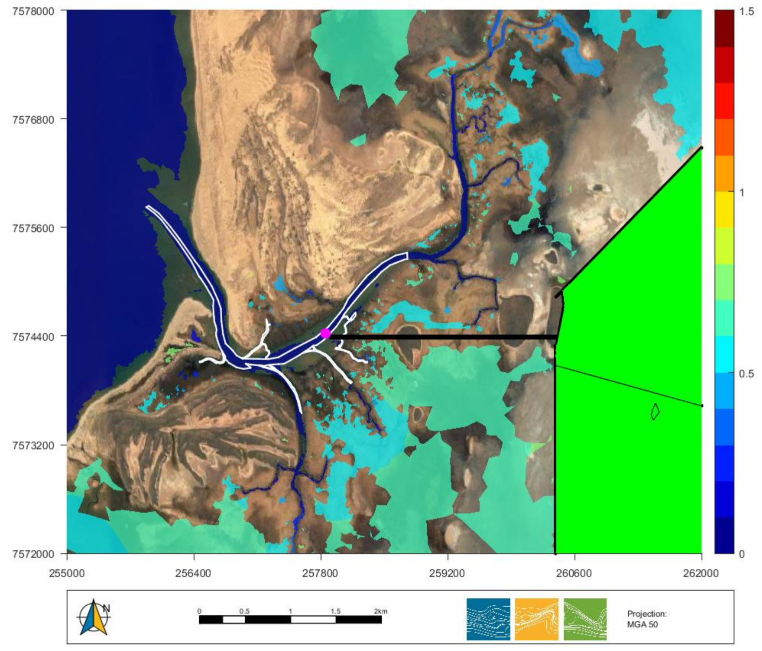
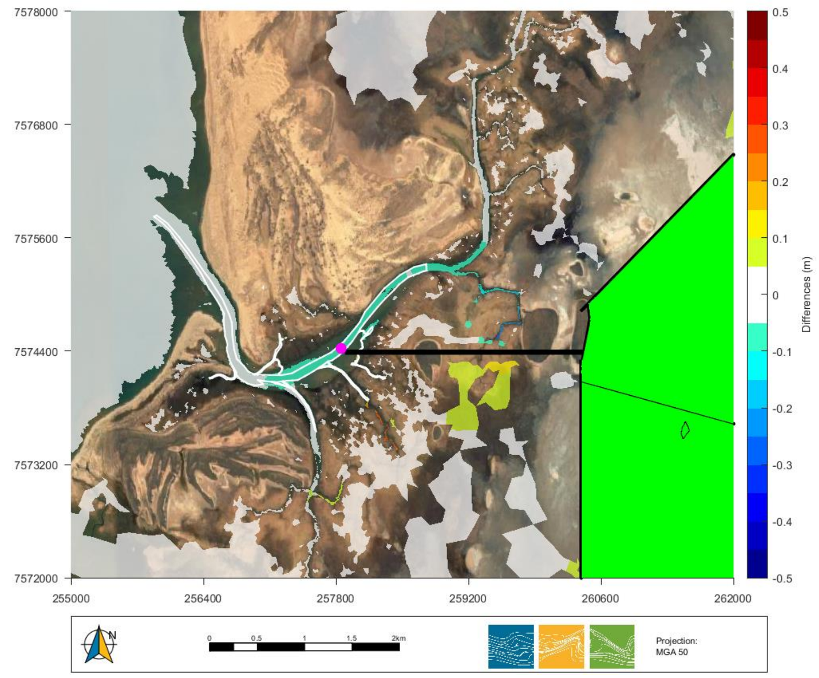
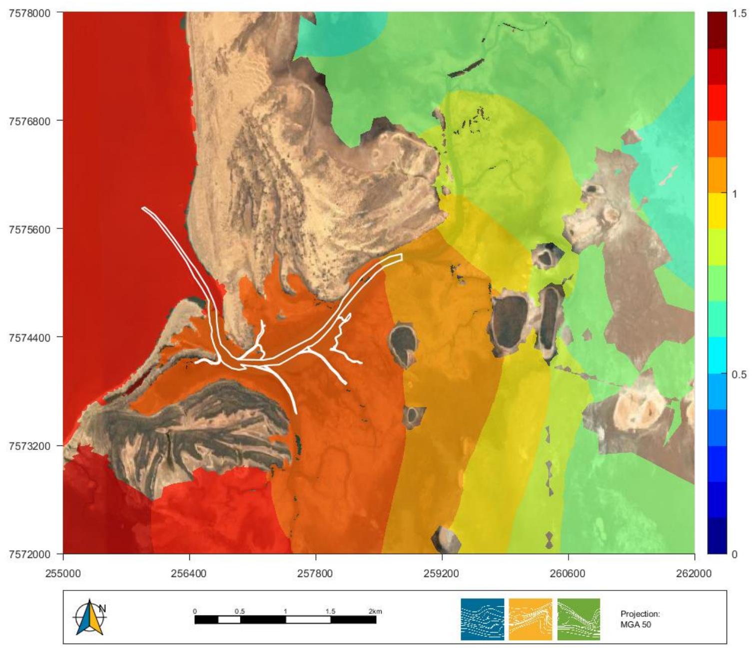
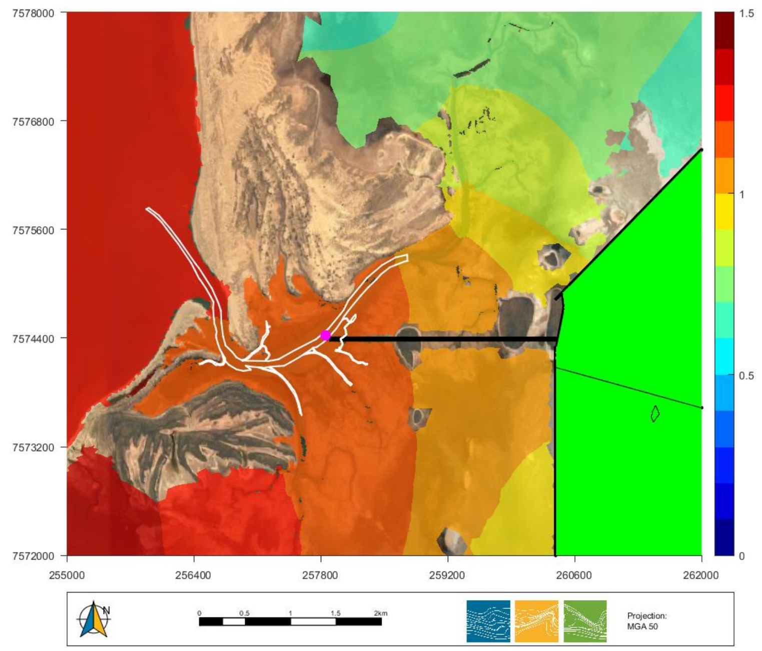
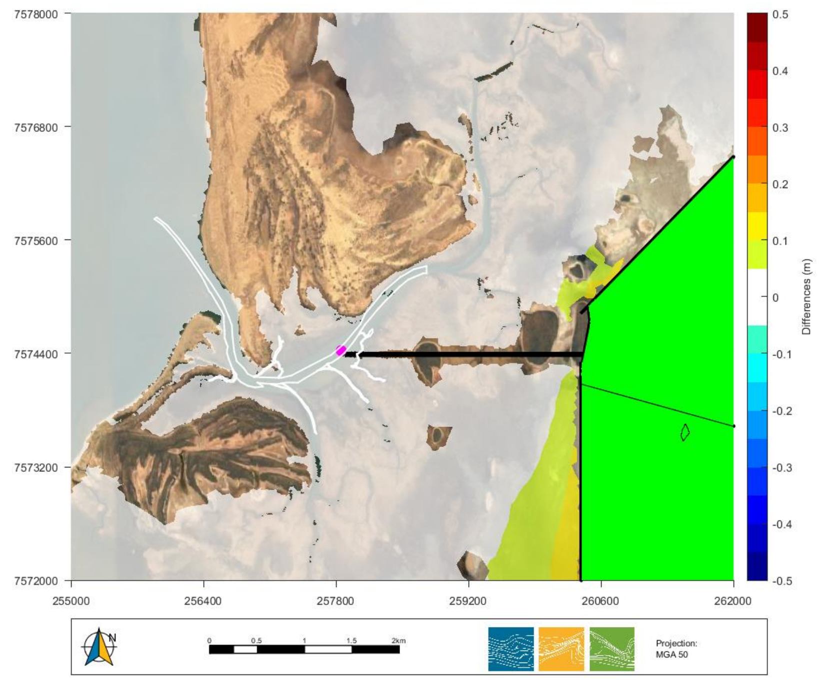
- Under extreme spring tidal conditions (March/April) the modelling predicts some localised and temporary backing up of water to the North of the seawater intake channel and a reduction of water level to the South of the channel (due to the physical barrier effect). This change in water level is small (in the order of 10 cm) and temporary.
- For the main habitats such as the algal mat and mangroves, the predicted changes in water level are small and temporary, therefore likely to have no material effect on the existing inundation pattern and duration for these habitats.
- Under normal spring tidal conditions (months other than March/April) impacts are expected to be even smaller than those in extreme spring tidal conditions.

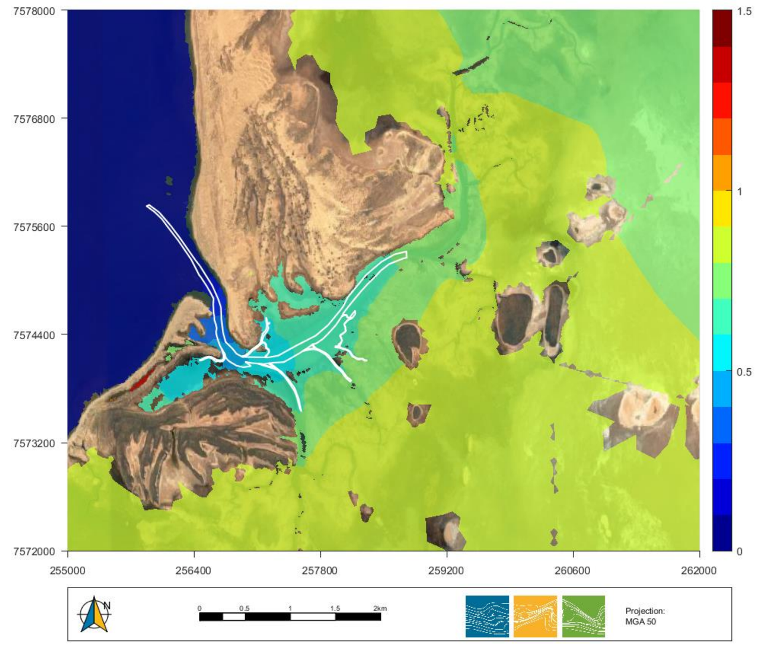
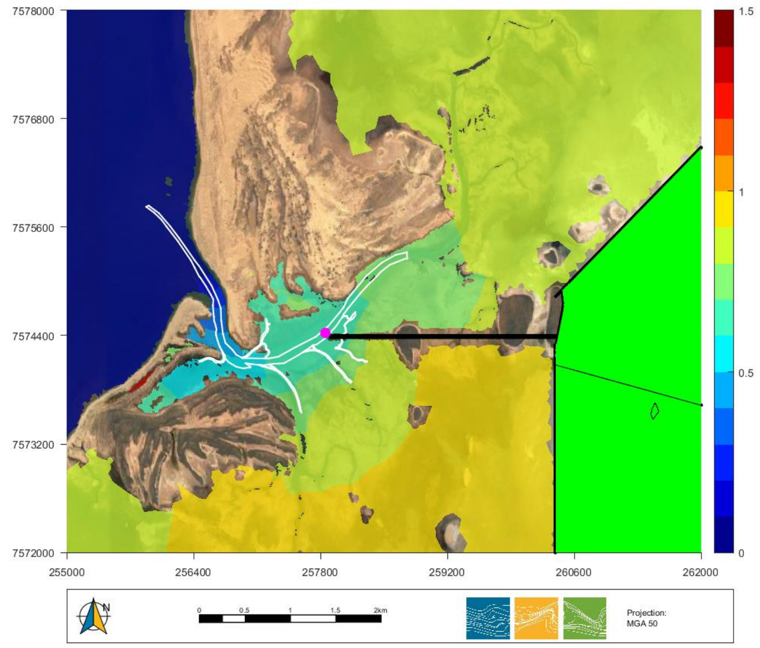
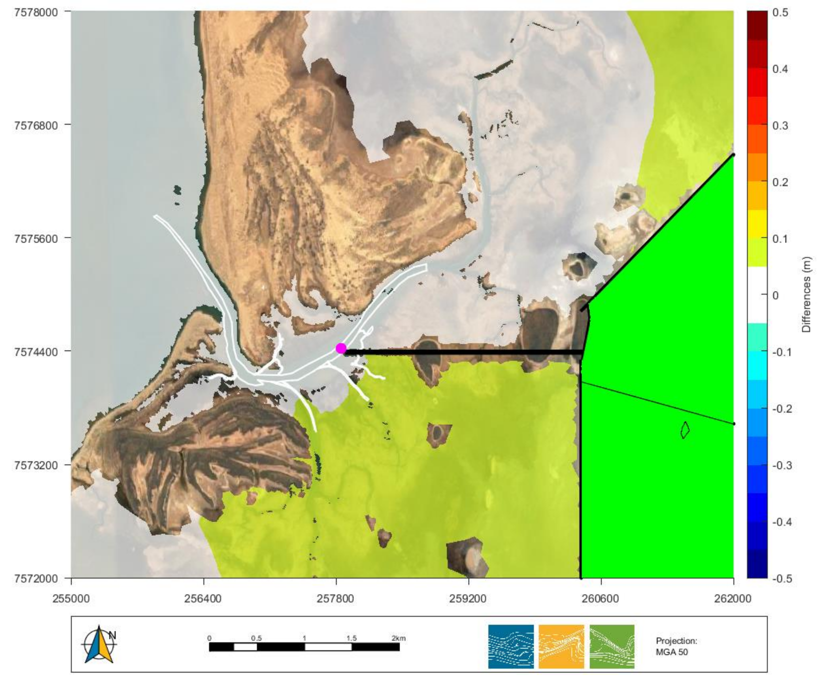
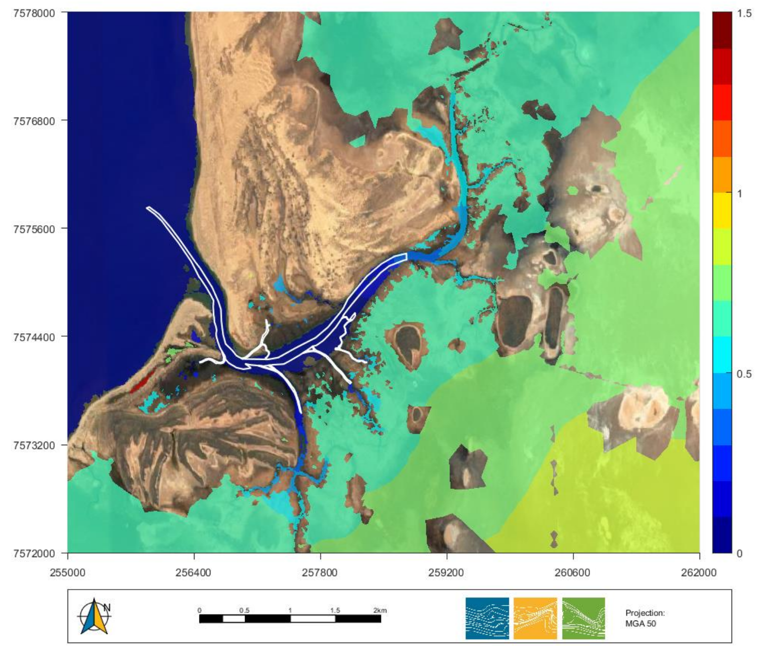
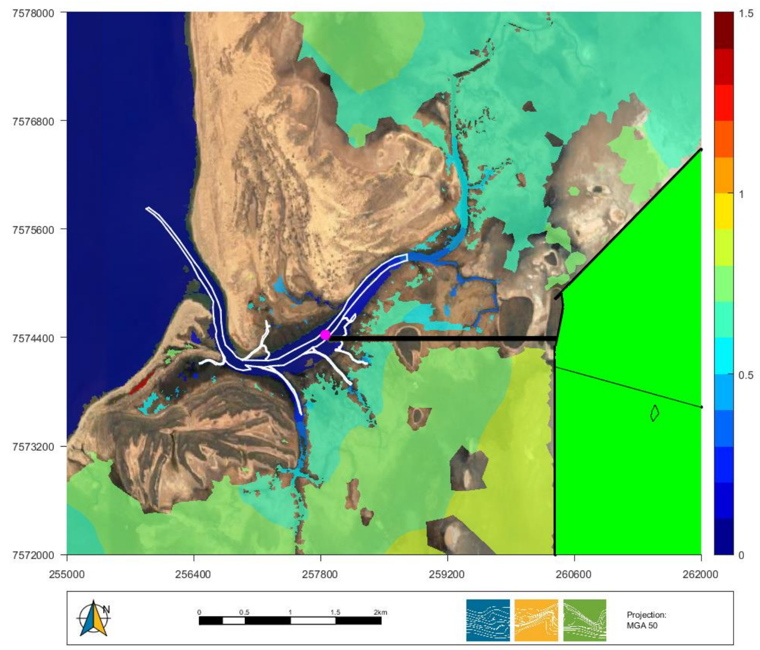
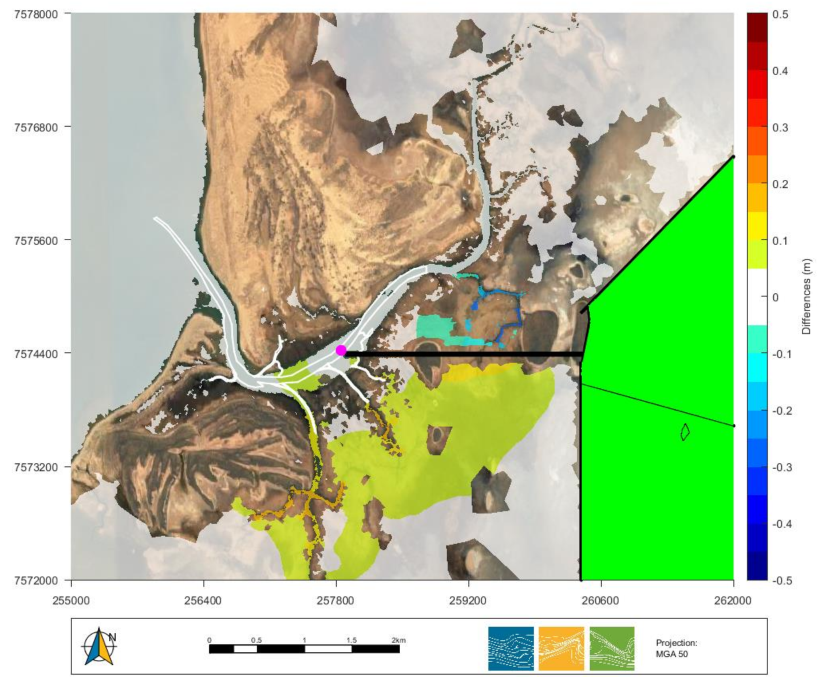
Sea Level Rise

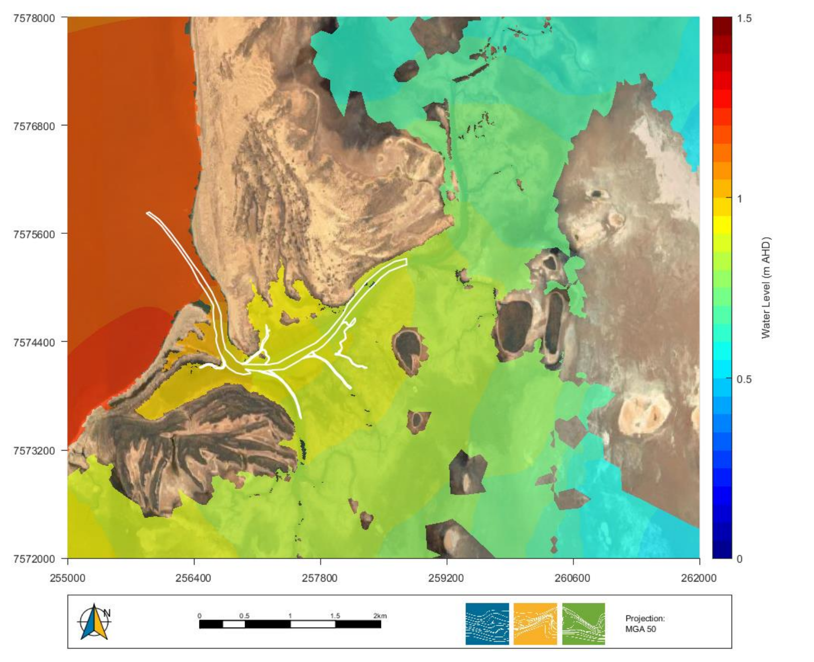
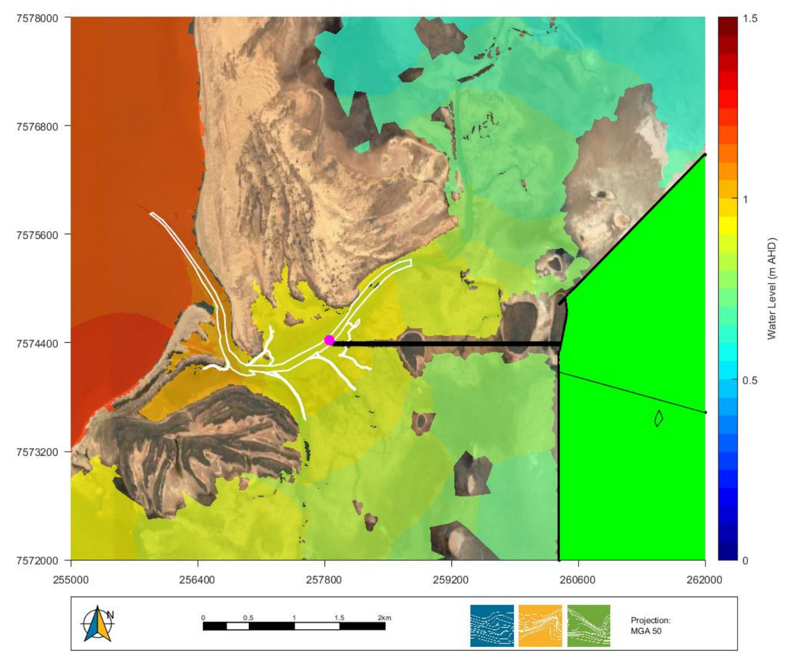
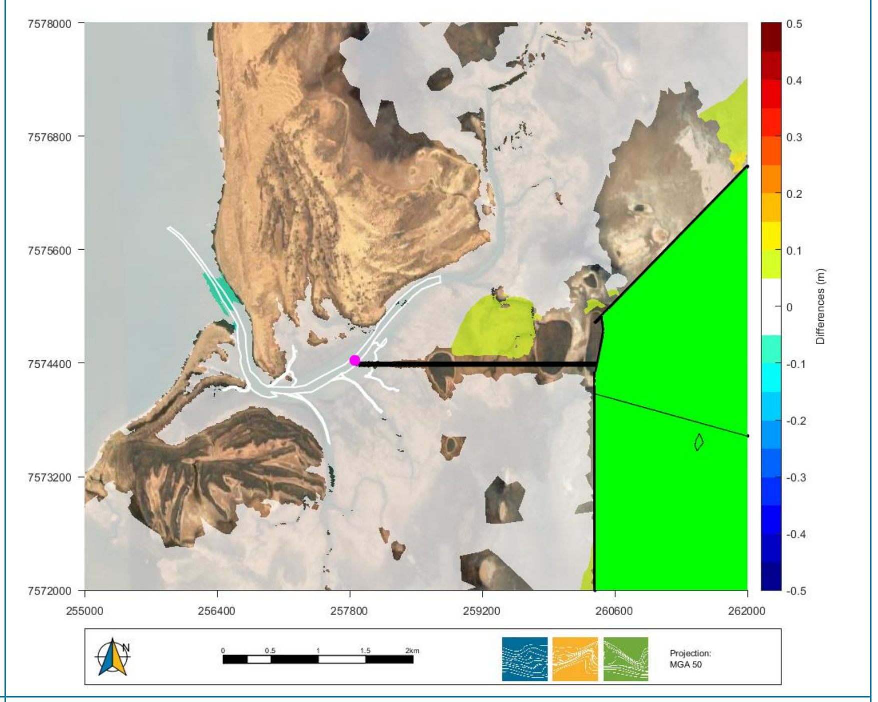
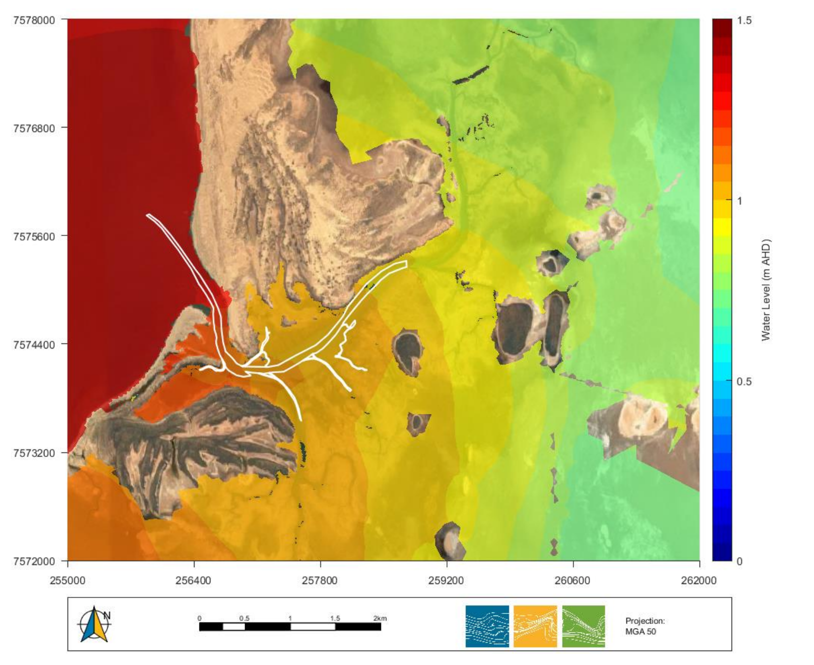
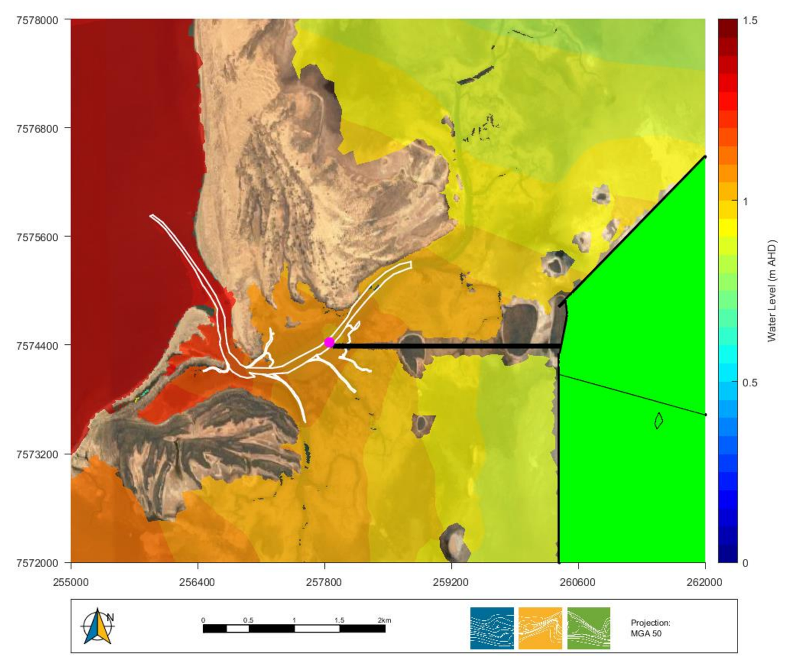
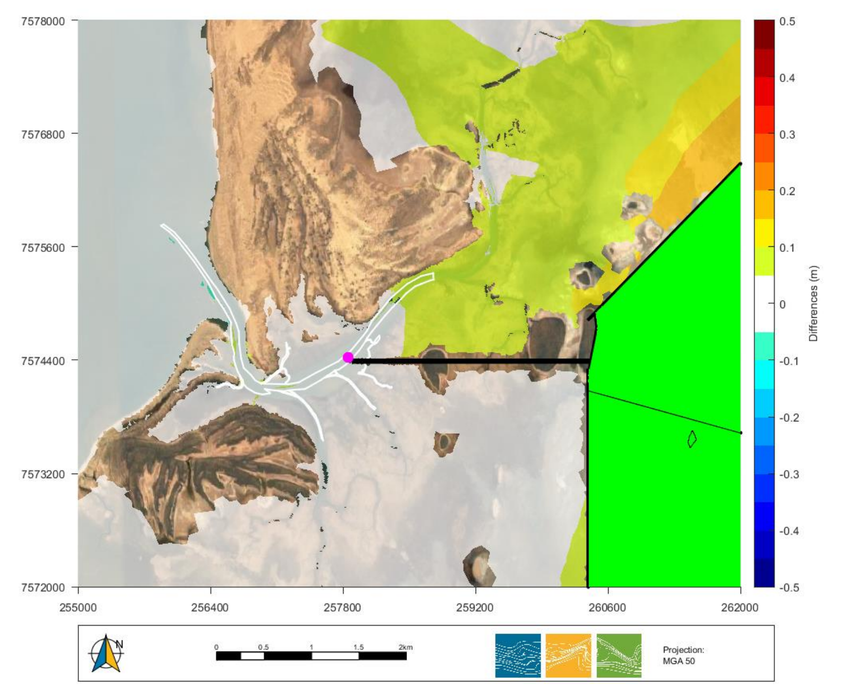
The pattern of impact to tidal inundation for seasonal high spring tides (April/March) (i.e., about 10 cm water level increase against the embankment walls) would remain the same for sea level rise scenarios. Slightly more simulated impact can be attributed to the greater extent (closer to the toe of embankment wall) of the intertidal zone and larger tidal prism under future sea level rises.

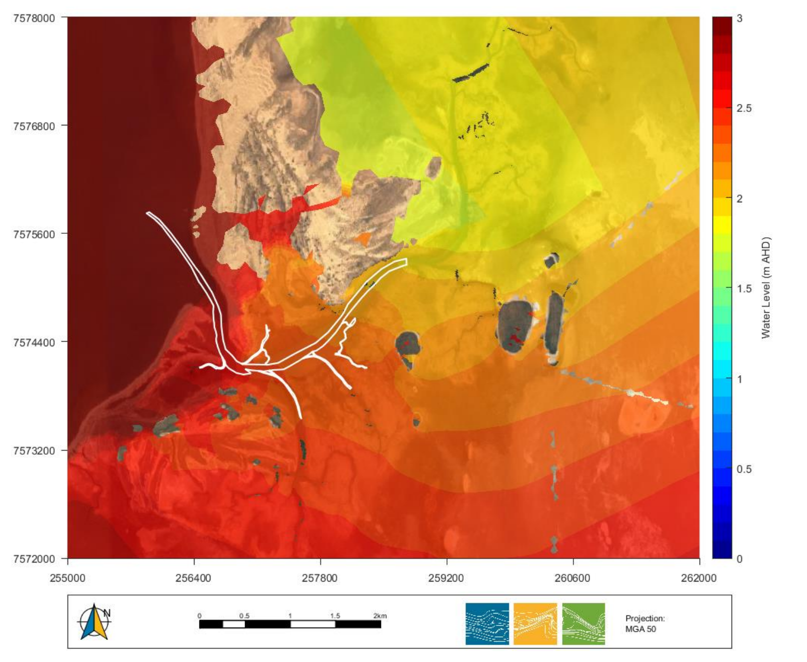
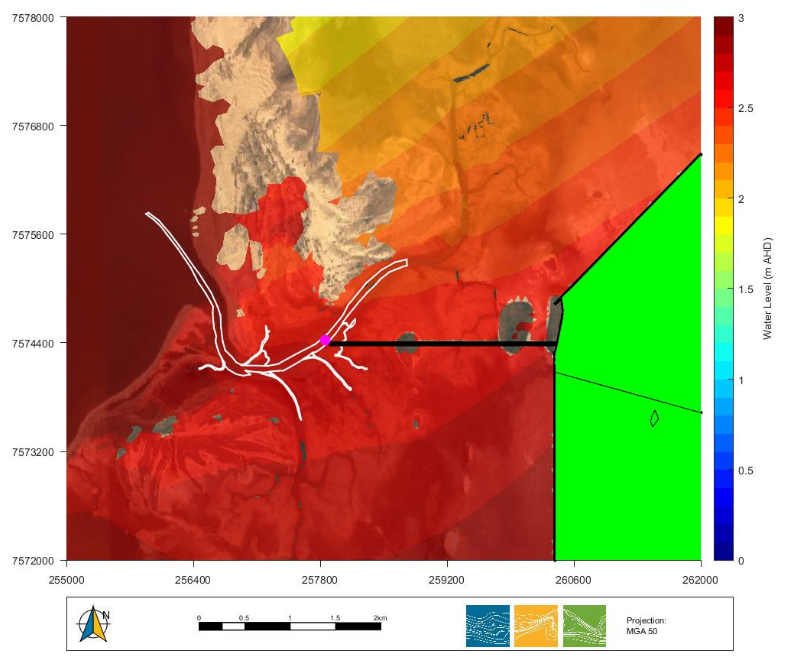
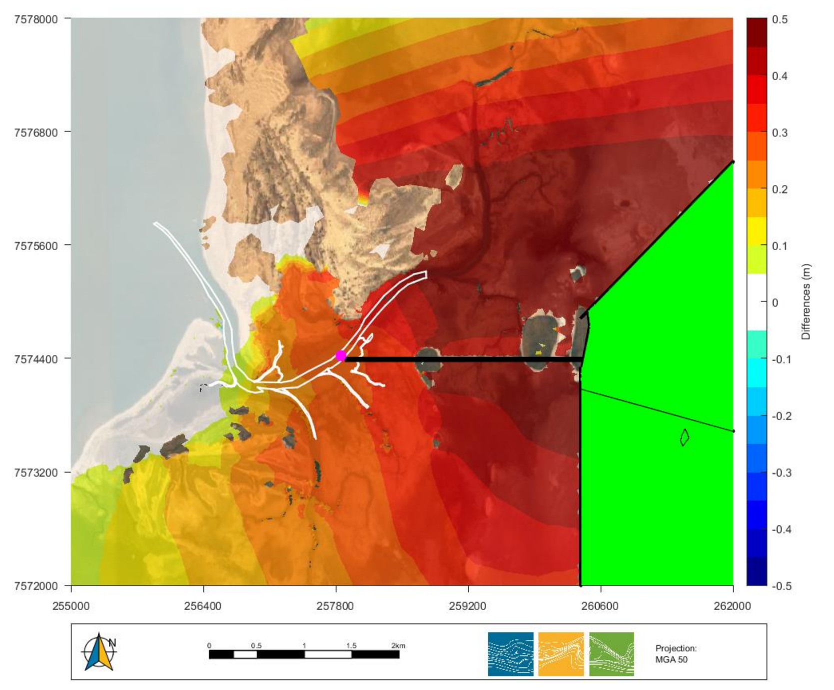
TABLE 3-3 MODELLED WATER LEVEL AT URALA CREEK SOUTH

Scenarios	Existing	Development	Differences (Developed – Existing)
Spring High Tide (2020-4-11 4:30)			
Spring High Tide +4 hours (2020-4-11 8:30)			

Scenarios	Existing	Development	Differences (Developed – Existing)
Spring High Tide +8 hours (2020-4-11 12:30)			
Spring High Tide (2020-4-11 4:30) + SLR			

Scenarios	Existing	Development	Differences (Developed – Existing)
Spring High Tide +4 hours (2020-4-11 8:30) + SLR			
Spring High Tide+8 hours (2020-4-11 12:30) + SLR			

Scenarios	Existing	Development	Differences (Developed – Existing)
2-year ARI Storm			
20-year ARI Storm			

Scenarios	Existing	Development	Differences (Developed – Existing)
500-year ARI Storm			

3.3.2.2 Tidal Current in Urala Creek South

The spatial distribution of modelled peak flood and ebb tidal current speeds including differences between pre- and post- development are presented in Table 3-4:

- Left column for existing conditions.
- Middle column for developed conditions.
- Right column for the difference plot between existing and developed conditions.

Existing Conditions

Extreme spring tide (March/April) conditions were modelled. For the pre-development scenario, the main Urala Creek South tidal channel is filled with seawater at peak flood tide before draining with the ebb tide. Flood flows gradually converge near the entrance of Urala Creek South and forms a channel flow in the lower sections of the creek. The strongest flood current speeds occur downstream along the lower ~2 km of the creek and may exceed 0.5 m/s. Current speeds upstream and in the small tributaries are much weaker (usually <0.3 m/s). The spatial distribution of peak flood currents is similar for all simulated scenarios.

During the extreme spring tides (March/April) peak ebb flow, high current speeds extend to about 2 km offshore and form a "jet" type flow feeding into nearshore water, which has a background speed of around 0.1 m/s. Ebb flow during an extreme peak (March/April) spring tide may exceed 0.8 m/s which is stronger than flood flow.

Current speeds are lower than the above during less extreme spring tides (outside of March/April) and much lower during neap tides.

Post-development Conditions

The maximum pumping rates for the seawater intake occur in November/December when evaporation is highest. Therefore, to model the impacts of pumping in a conservative manner, the maximum intake rate in November was used to determine changes to Urala Creek South current speeds if such pumping occurred during the peak flow months of March/April.

The seawater intake increases the flood flow while decreases the ebb flow in Urala Creek South channel. There is also some predicted impact on water flow speed (current) over the tidal flat due to presence of embankment walls however this is minor and localised.

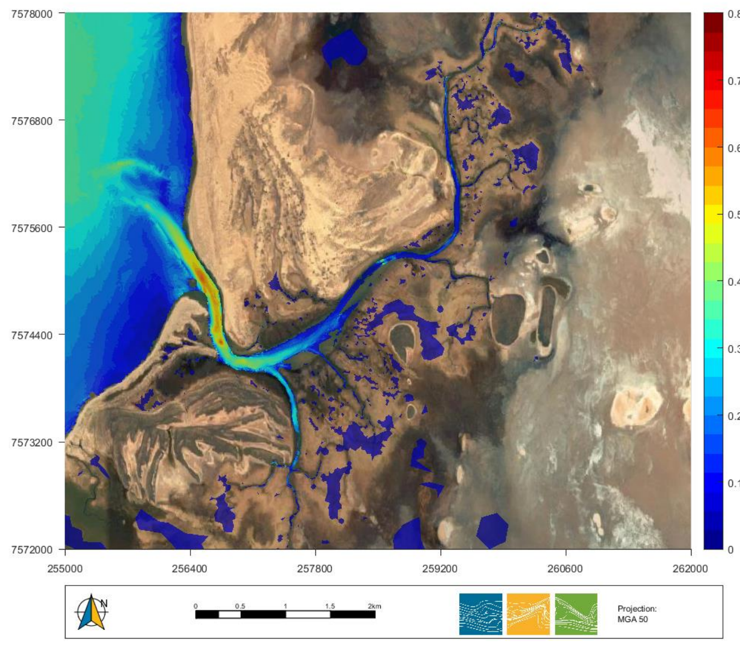
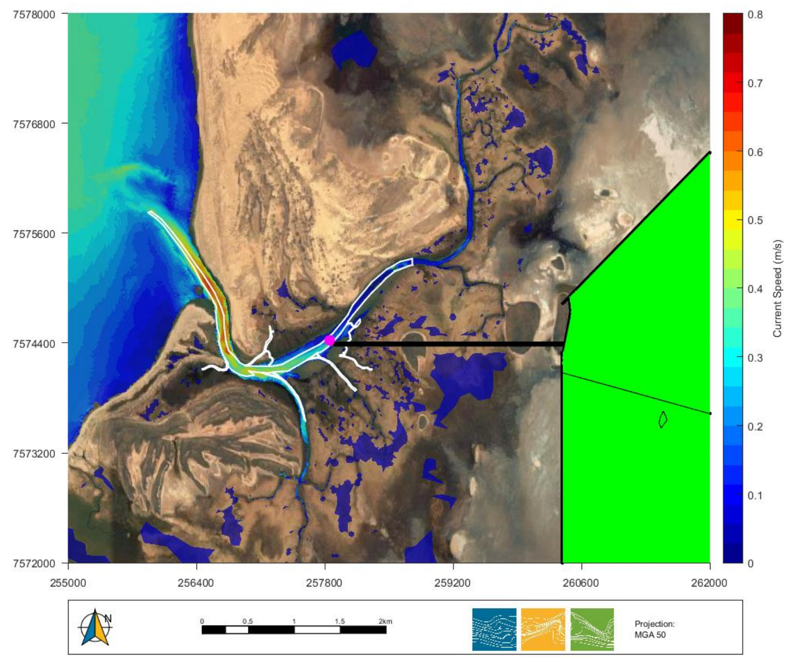
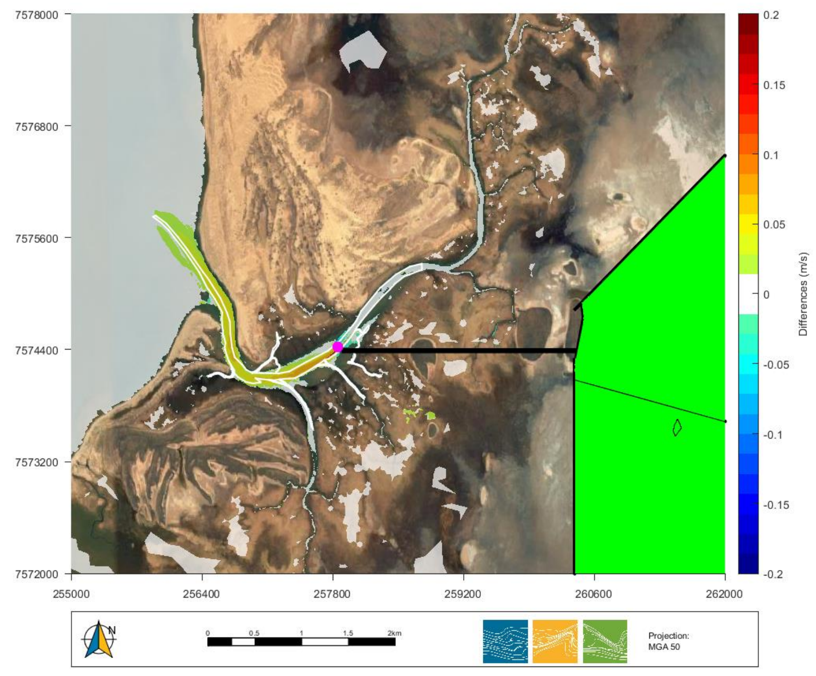
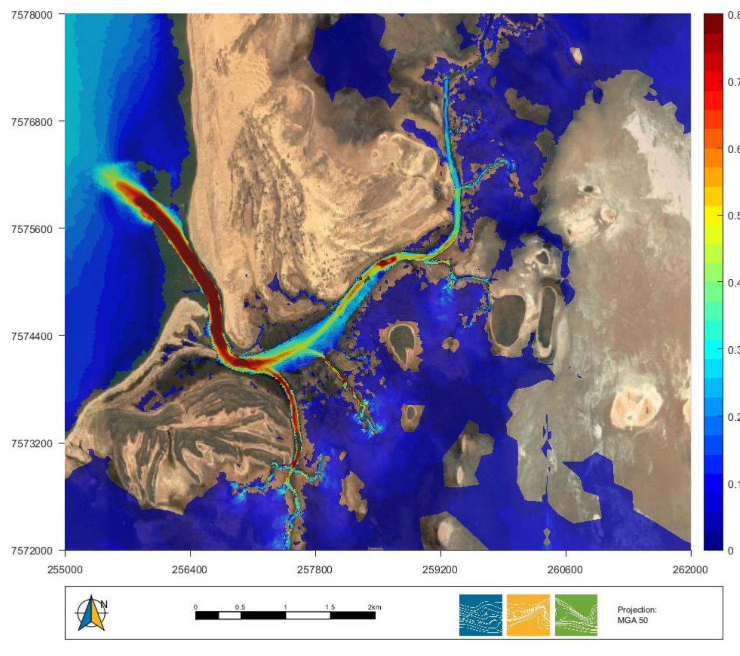
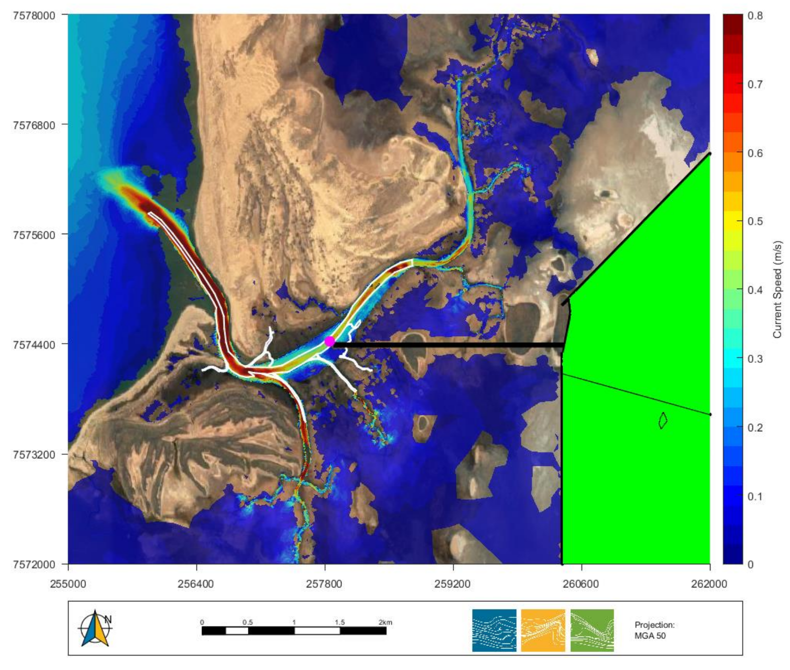
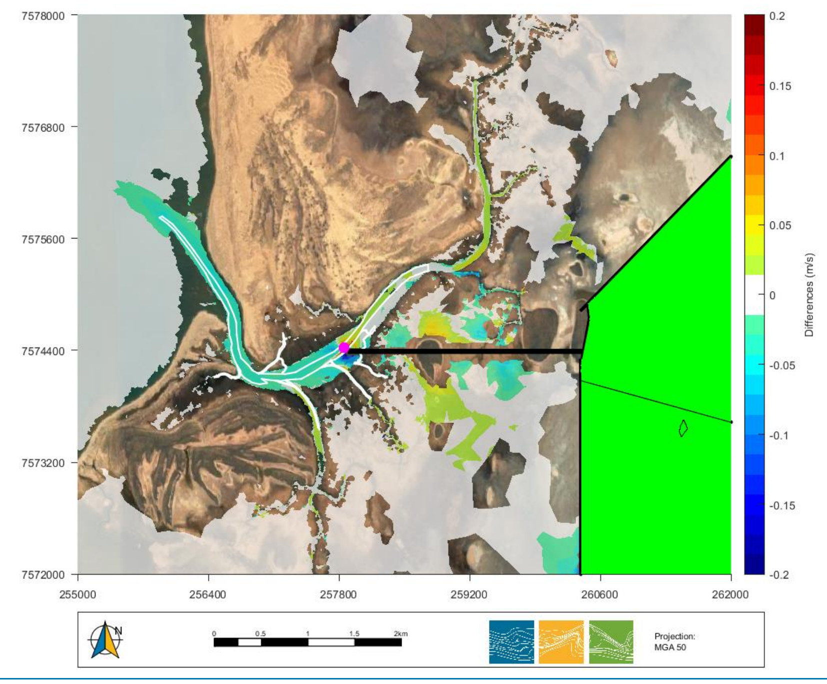
During extreme spring tides (April/March) predicted impacts using maximum (November) pumping rates are as follows:

- During the peak flood tide, current speeds increase (<5 cm/s) in the portion of Urala Creek west from the seawater intake and marginally decrease upstream of the seawater intake (<2 cm/s). This is due to the additional water flow being caused by the intake pumps.
- During the peak ebb tide, current speeds decrease (<5 cm/s) in the portion of Urala Creek South from the mouth to the seawater intake and marginally decrease upstream of the seawater intake (<2 cm/s). This is due to the additional water flow into the creek against the ebb tide flow. It should be noted that this impact is conservative as continuous pumping during high and low tide has been modelled, whereas it is not planned to pump at low tide.

Sea Level Rise

For the sea level rise scenario, the maximum currents in the creek will become stronger due to a larger tidal prism. Impacts from the seawater intake will be alleviated, since the intake rate will become relatively smaller when compared to a stronger tidal discharge. Due to the changes in the area of the intertidal zone, model results show some additional impact from the embankment walls as shown in Table 3-4.

TABLE 3-4 SIMULATED CURRENT SPEED

Scenarios	Existing	Development	Differences (Developed – Existing)
Peak Flood Tide (April)			
Peak Ebb Tide (April)			

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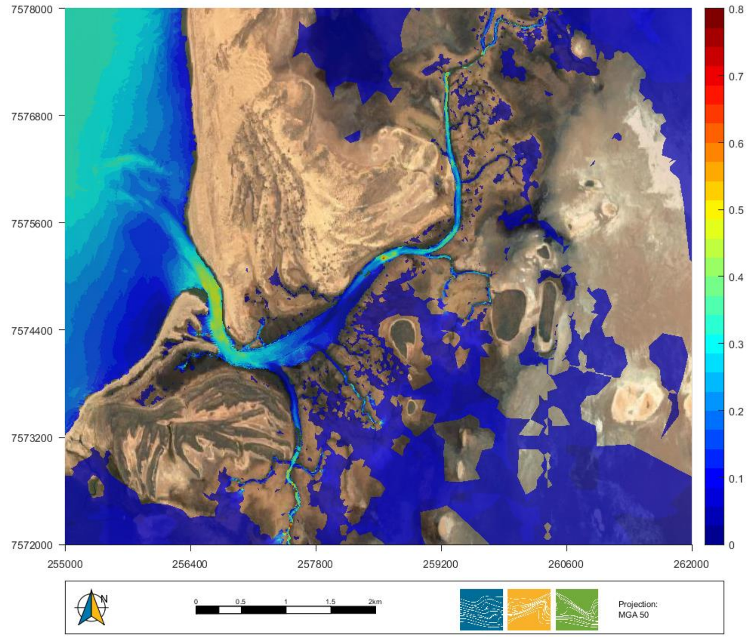
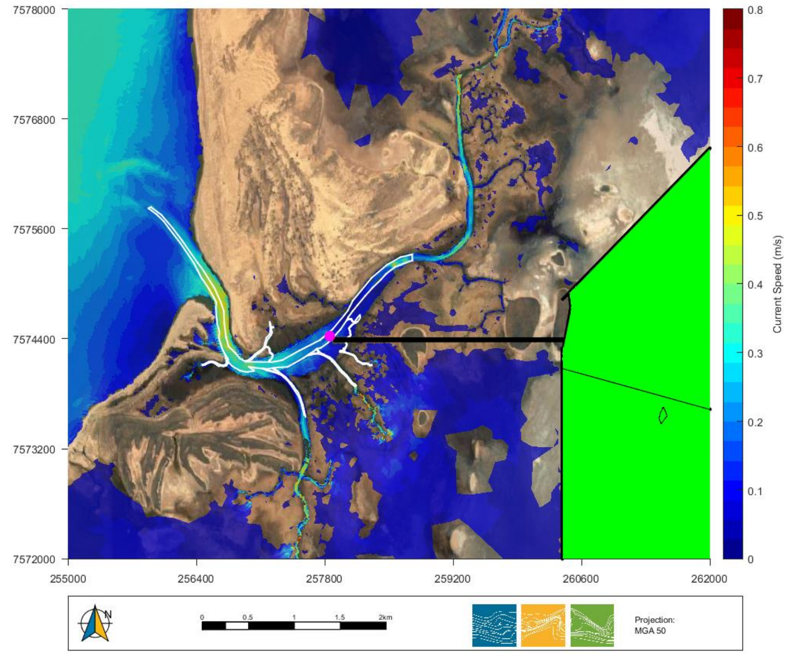
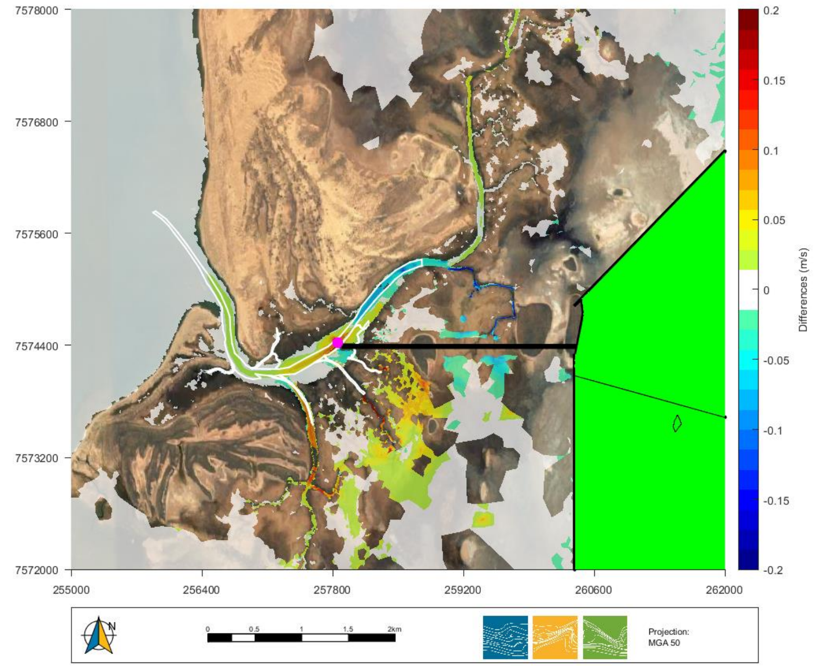
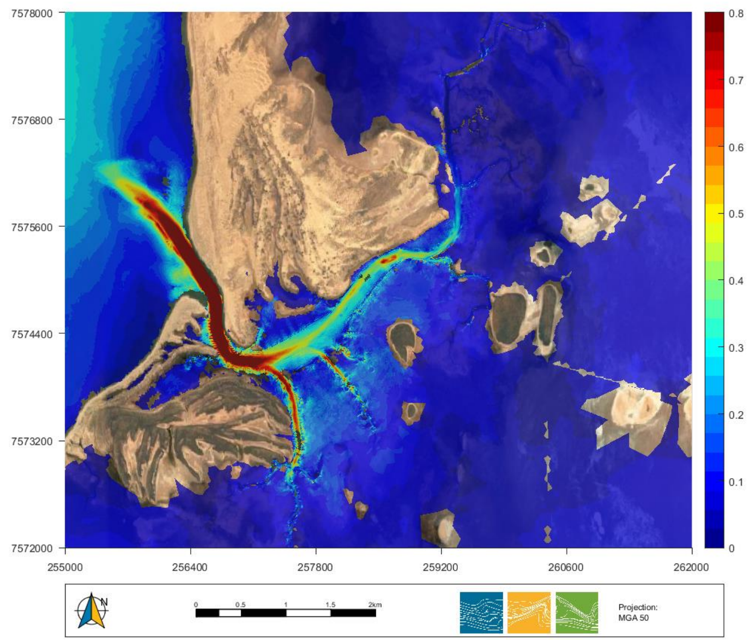
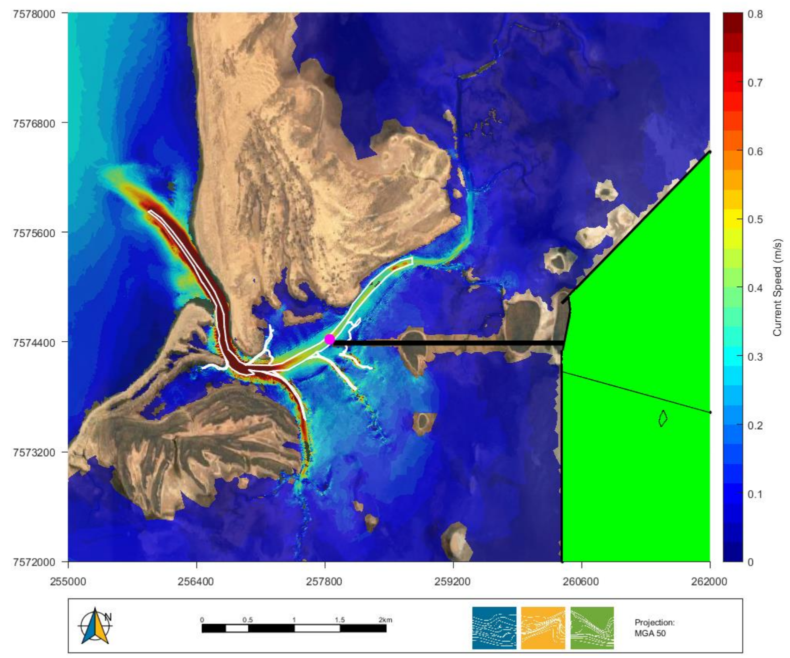
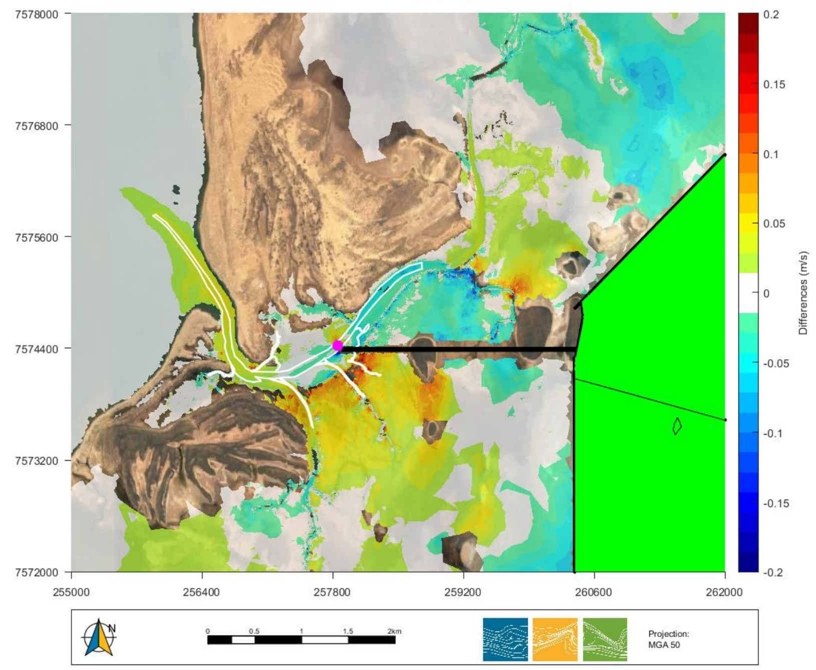
Scenarios	Existing	Development	Differences (Developed – Existing)
Peak Flood Tide +SLR (April)			
Peak Ebb Tide+SLR (April)			

3.3.2.3 Fluvial Morphology

Urala Creek South morphology has been modelled by MIKE 21 Sand transport (ST) module with results presented in Figure 3-2 to Figure 3-4.

For existing versus developed conditions, the model predicts very small bed level changes in the main channel and weak bed level variations near the entrance over a year of morphology modelling. The morphology changes predicted over the year, which are in the order of 10 cm, are low as the creek has already attained a dynamic equilibrium state due to historical sediment transport and vegetation growth and is also consistent with review outcomes from historic satellite images and conclusions by Seashore Engineering (2021).

For developed conditions, at the creek mouth and an intertidal area immediately seaward of the creek mouth, there is reduced sediment deposition. This is likely due to changes in tidal flows downstream of the intake.

Overall, the predicted impact post-development is low with relatively small and localised changes predicted.

Furthermore, these predictions may be conservative, given pumping is not expected to be undertaken at low tide, whilst modelling has used a constant pumping regime at both high and low tide.

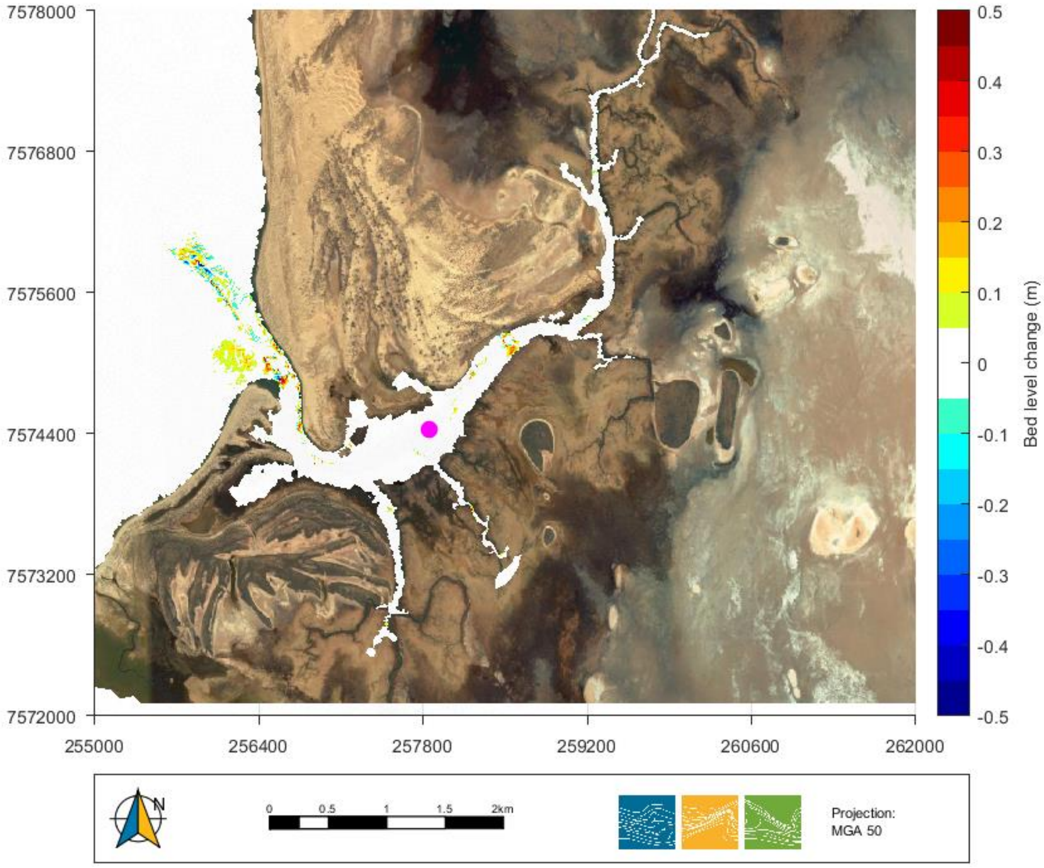


FIGURE 3-2 SAND TRANSPORT MODEL OVER 1 YEAR - EXISTING PRE-DEVELOPMENT SCENARIO

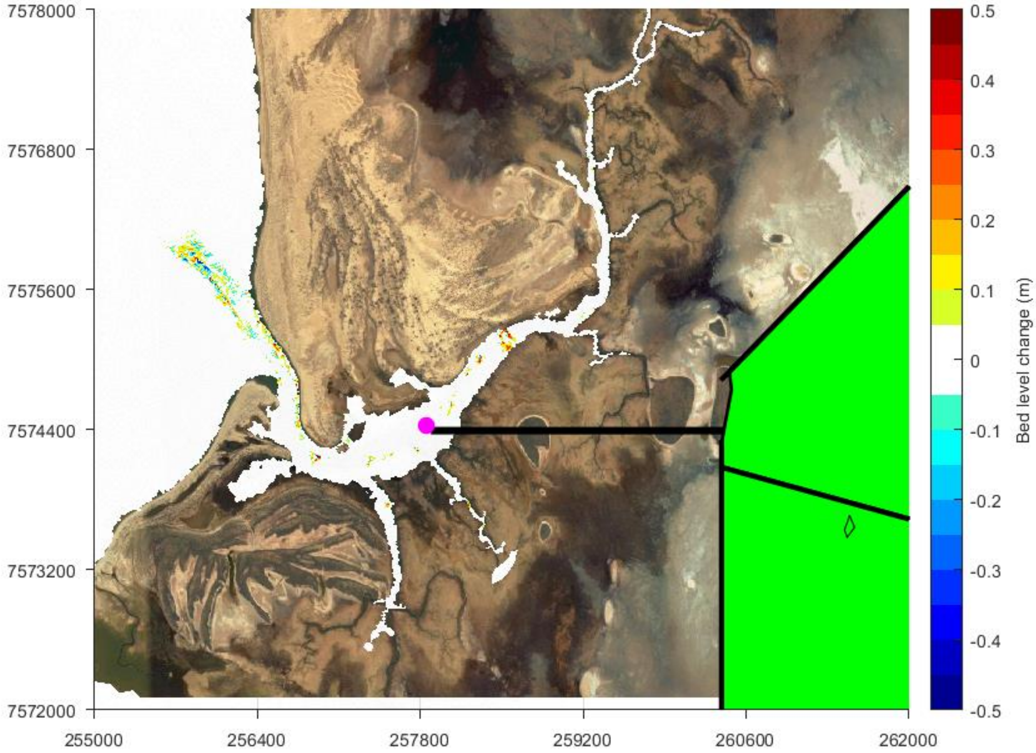


FIGURE 3-3 SAND TRANSPORT MODEL OVER 1 YEAR – POST-DEVELOPMENT SCENARIO

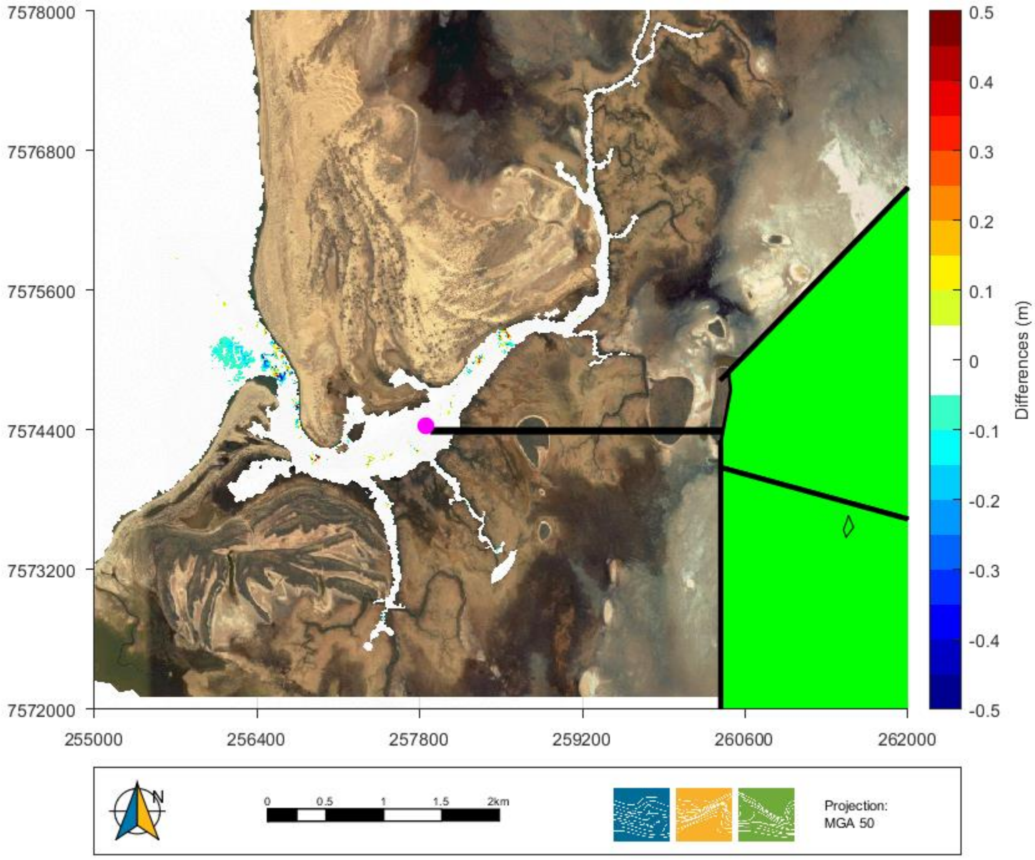


FIGURE 3-4 1-YEAR SAND TRANSPORT MODEL RESULT - EXISTING & POST-DEVELOPMENT DIFFERENCE

3.3.3 Modelled Hydrodynamic Changes at Locker Point (Jetty Site)

3.3.3.1 Water Level

The spatial distribution of modelled high and low water, including differences between pre- and post-development, are presented in Table 3-5:

- Left column for existing conditions.
- Middle column for developed conditions.
- Right column for the difference plot between existing and developed conditions.

Existing Conditions

At Locker Point, the modelled high water level is about 1 m AHD for an extreme (March/April) spring high tide, about 1.2 m AHD for a 2-year event, about 1.4 m AHD for a 20-year event (see Table 3-5), and about 3 m AHD for a 500-year cyclone event.

Post-development Conditions

The model prediction shows negligible impact from proposed jetty, diffuser and berthing pocket to nearshore water levels as expected, given the jetty design on pylons which is not expected to alter water levels due to its high transmissivity.

3.3.3.2 Current and Waves

The spatial distribution of currents or waves has not been depicted visually given the negligible impacts of the development on these aspects.

Existing Conditions

Fieldwork undertaken as described in *Marine, Coastal and Surface Water Data Collection* (Water Technology, 2021) and modelling indicate the area in the vicinity of jetty site experiences very low current speeds. The modelled flood/ebb current speeds are in the order of 0.05-0.1 m/s for a typical tide which is weaker than tidal flow (in order of ~0.2 m/s) to the west.

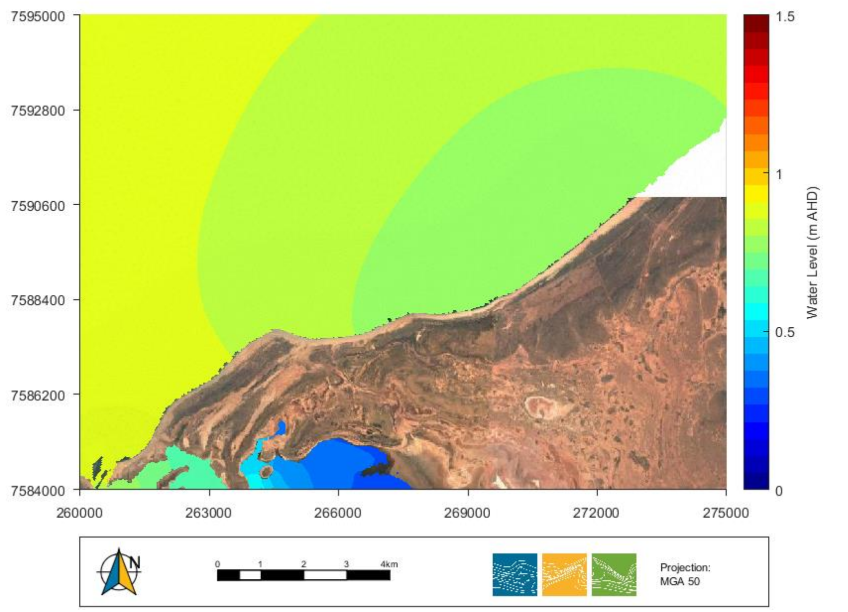
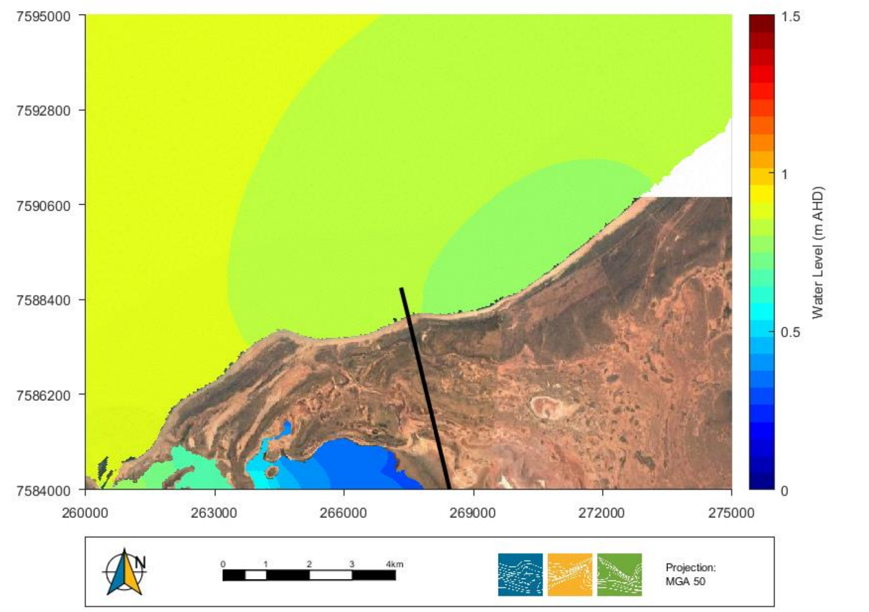
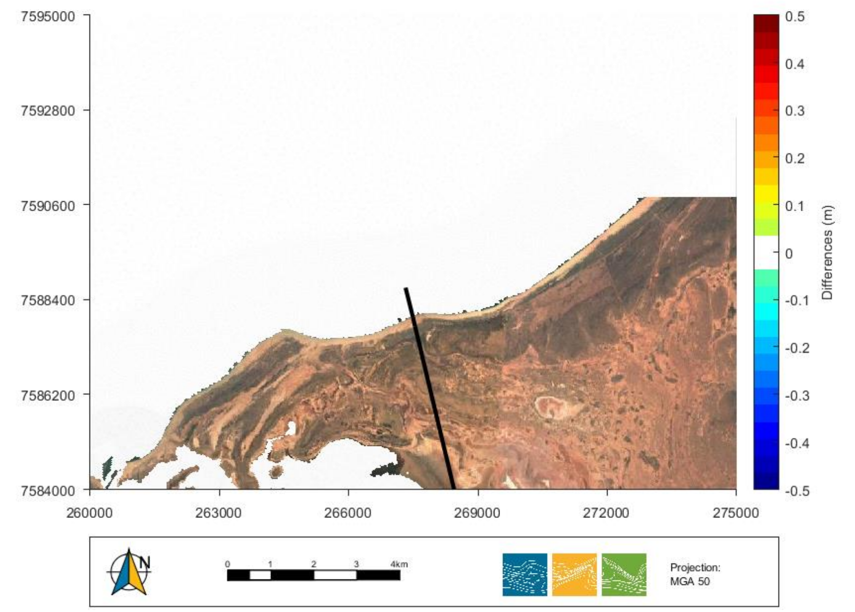
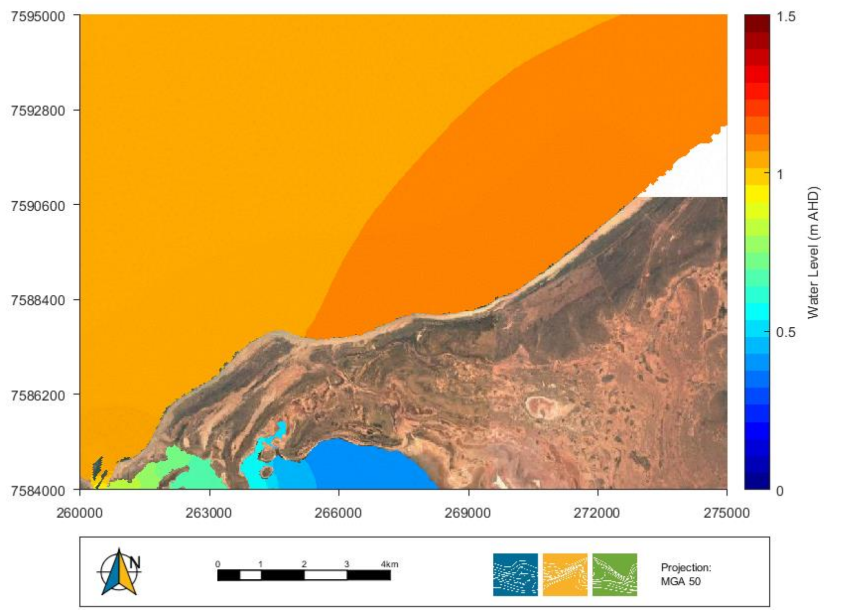
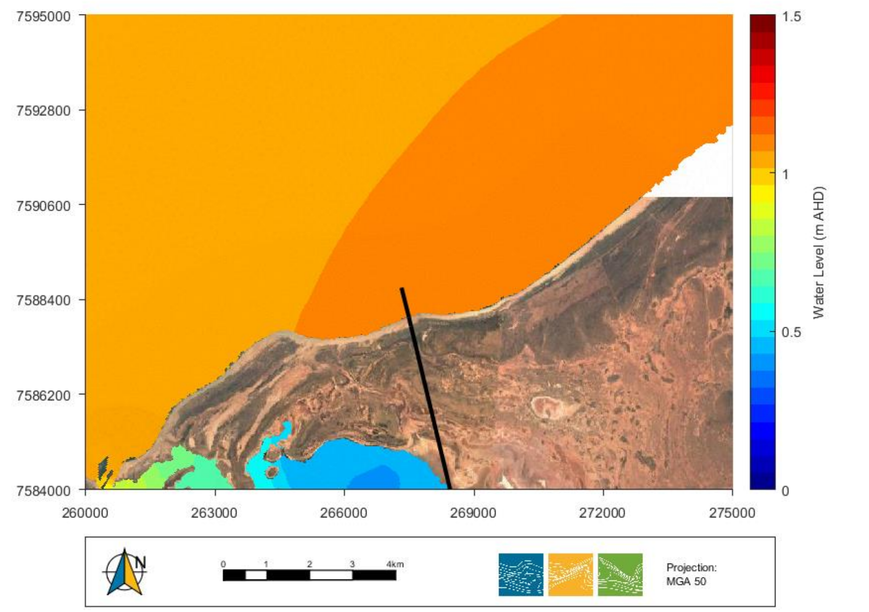
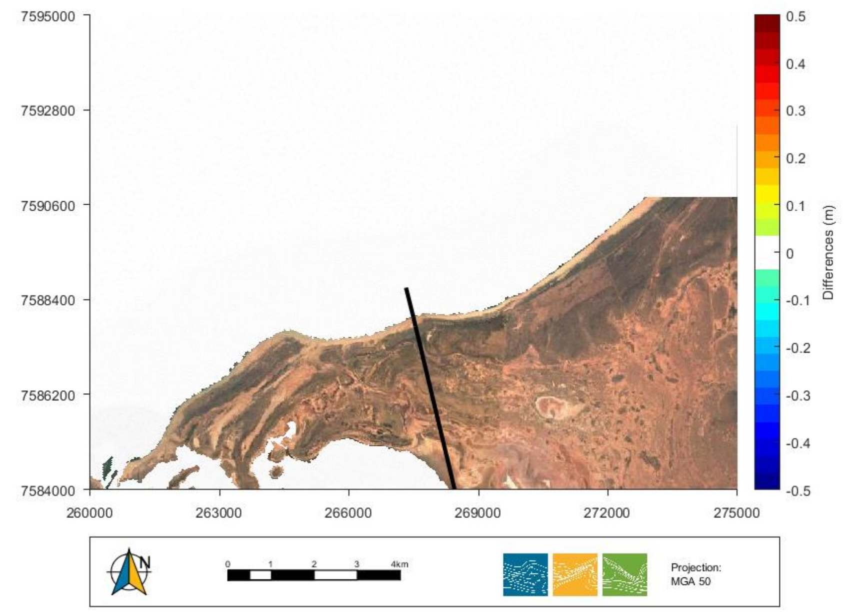
Wind driven waves and currents can be much stronger than ambient tidal flow which is not only correlated with wind speed but also affected by incoming wind direction and duration.

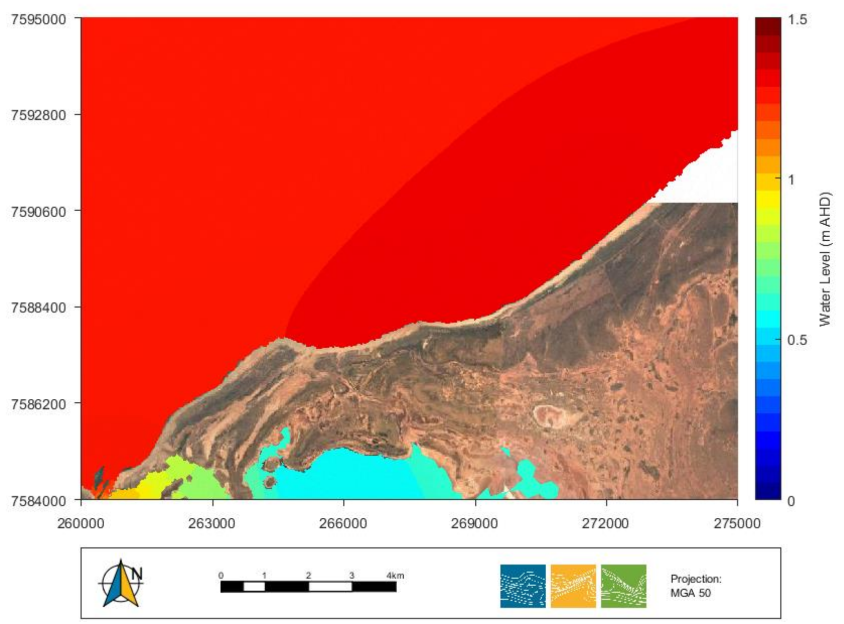
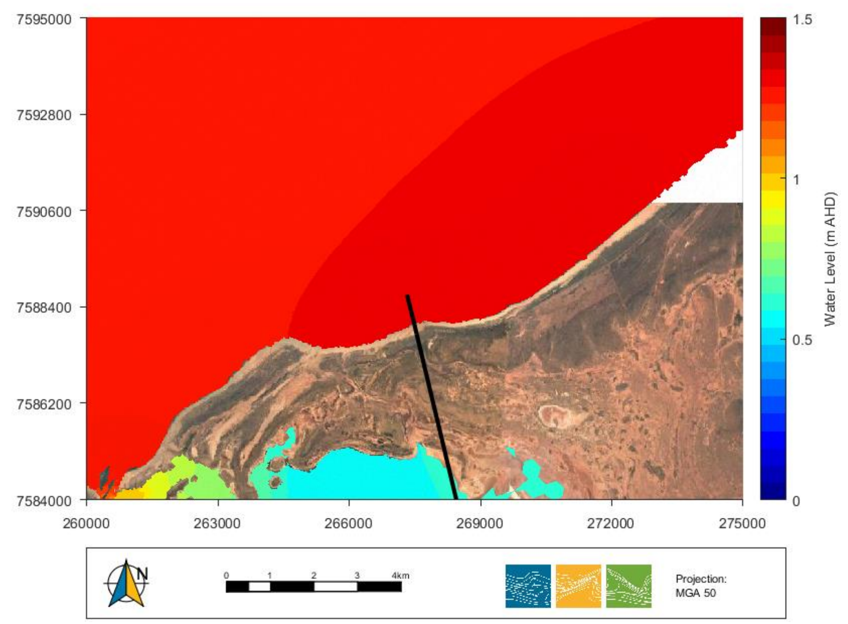
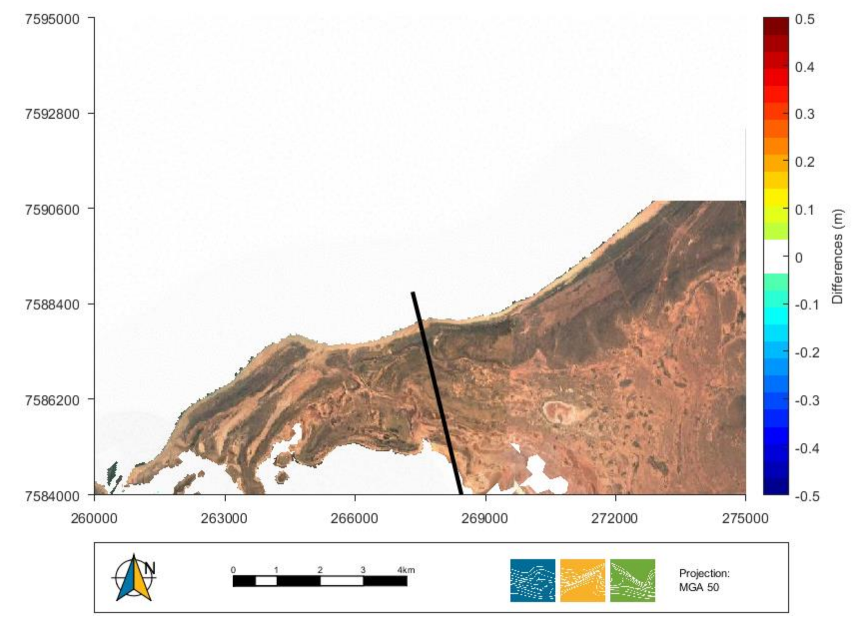
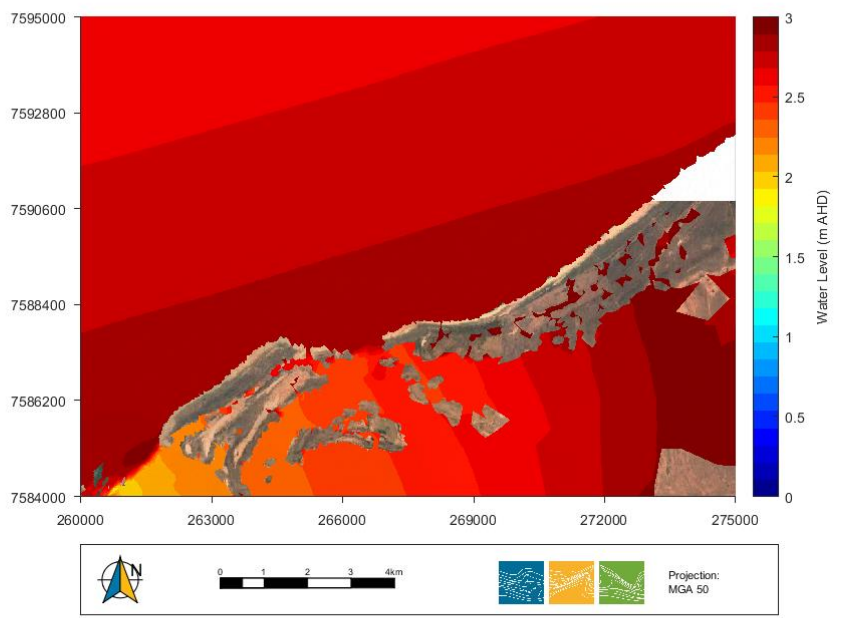
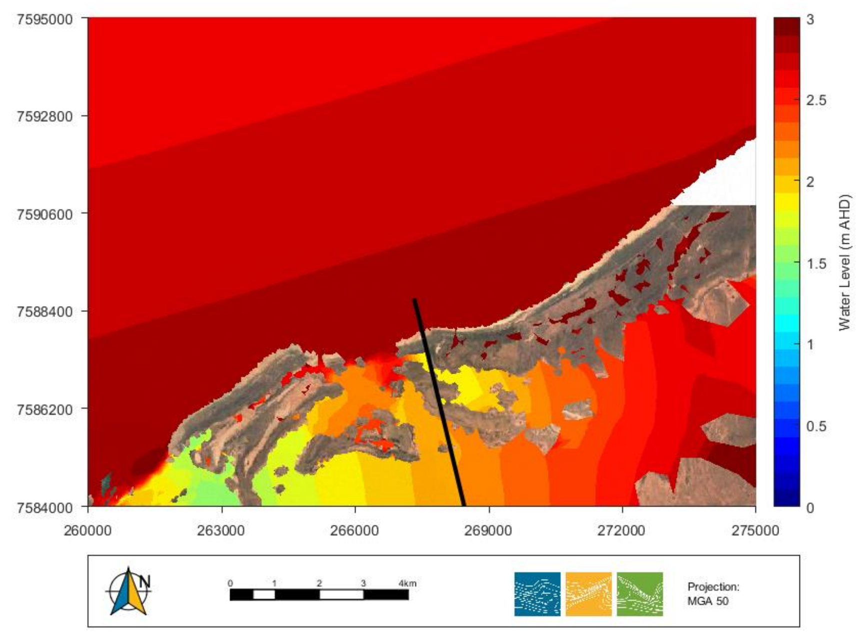
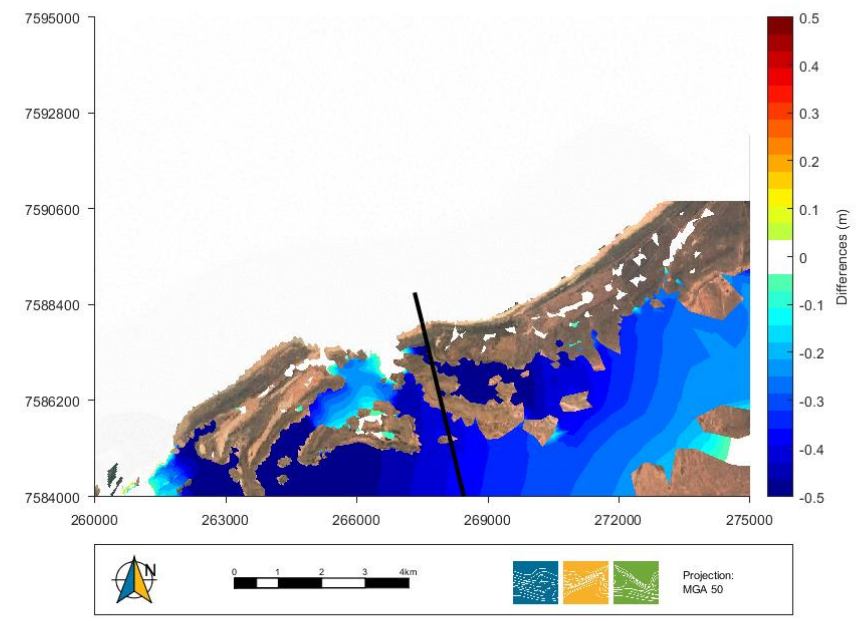
Post-development Conditions

The predicted impacts to currents and waves at Locker Point post-development are negligible which can be attributed to:

- The pile-supported jetty structure which is transmissive to water flows and therefore does not affect currents significantly.
- The small bitterns discharge rate relative to the existing tidal current (water flow rate) across the diffuser.
- The footprint of proposed development (the jetty piers are too small to generate any material impact to nearshore wave/hydrodynamic conditions in such a low energy environment).

TABLE 3-5 MODELLED HIGH TIDE WATER NEAR LOCKER POINT

Scenarios	Existing	Development	Differences (Developed – Existing)
Spring Tide (April)			
2-year ARI Storm			

Scenarios	Existing	Development	Differences (Developed – Existing)
20-year ARI Storm			
500-year ARI Storm			

3.4 Key Findings

3.4.1 Pre- and Post-development Submergence Curves in Urala Creek South

Figure 3-5 presents a comparison of submergence curves of modelled pre-development and post-development water levels at the Urala Creek South intake location for the year 2015. The submergence curve comparison highlights the following:

- There is negligible impact to the percentage time submerged at high tide. The pre- and post-development submergence curves show negligible difference for the Highest Astronomical Tide (HAT), Mean High Water Spring (MHWS) tide and Mean High Water Neap (MHWN) tide with the submergence curves practically overlapping.
- There is a very small impact (a few centimetres difference in submergence pre- and post-development) when the water level is at Mean Sea Level (MSL) or lower with the submergence curves aligned adjacent to one another.
- The model indicates a moderate impact to submergence at low tide. The modelled impact increases gradually from ~5 cm difference in submergence pre- and post-development at Mean Low Water Neap (MLWN) to over 20 cm difference in submergence at Lowest Astronomical Tide (LAT) with the submergence curves moving gradually apart. This reduction in level would only affect the main creek channel as the mangroves are naturally exposed (not submerged) at this part of the tidal cycle. Furthermore, it should be noted that this predicted impact would only occur if pumping actually occurred during low tide as has been modelled, however the project plans to cease pumping at low tide and therefore this modelled impact is unlikely to occur.
- In general, there are negligible impacts to intertidal areas inhabited by mangroves and algal mats due to their elevations above Mean Sea Level (MSL). Impacts can be described by difference in percentage time submerged for these habitats as follows:

  Mangroves

  - At the lower elevation of the mangrove zone (~**0 above MSL**) there is a predicted reduction in time submerged from ~**50% to ~48%.**
  - At the upper elevation of the mangrove zone (**~0.7 m above MSL**) there is no predicted reduction in time submerged **(with ~5% both pre- and post-development).**

  Algal Mats

  - At the lower elevation of the algal mat zone (**~0.8 m above MSL**) there is no predicted reduction in time submerged **(with ~3% both pre- and post-development).**
  - At the upper elevation of the algal mat zone (**1.1 m above MSL**) there is no predicted reduction in time submerged **(with ~1% both pre- and post-development).**

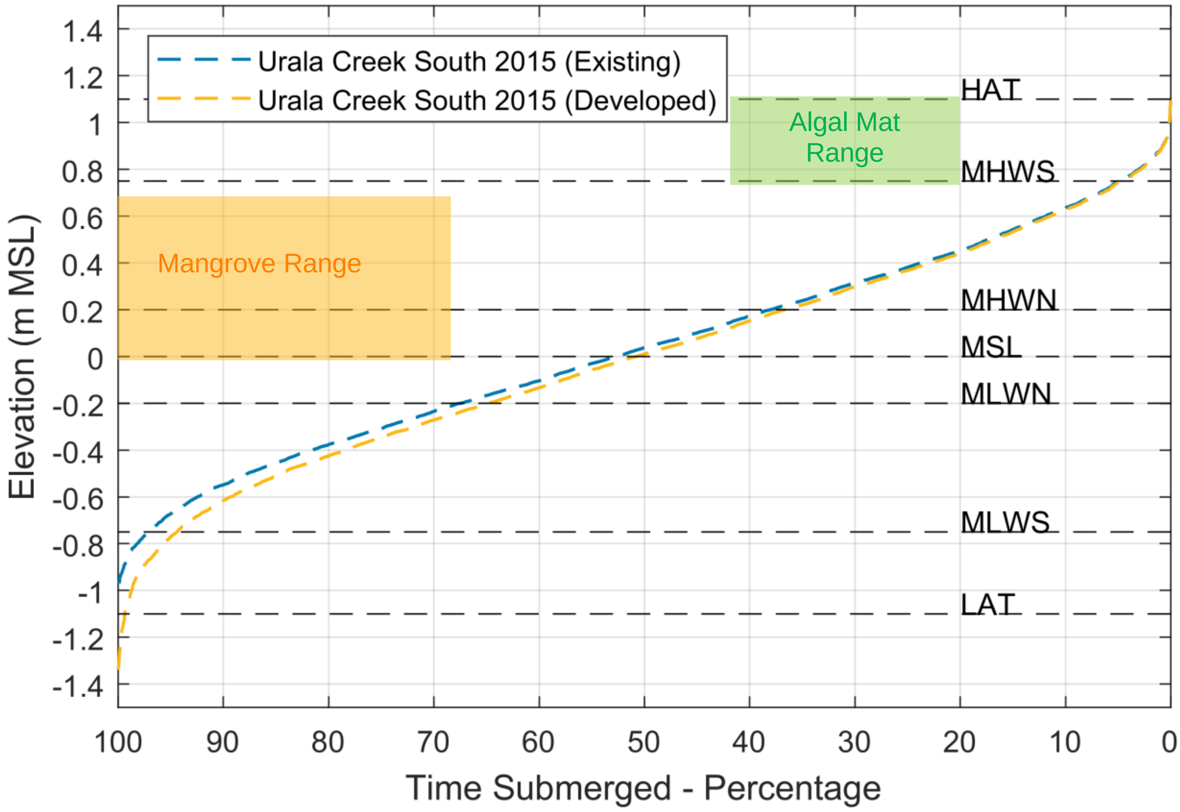


FIGURE 3-5 SUBMERGENCE CURVES AT URALA CREEK SOUTH PRE- AND POST-DEVELOPMENT 2015

3.4.2 Disturbance to Tidal Discharges in Urala Creek South

Tidal discharge through the Urala Creek South was assessed by extracting the discharge rate (m^3/s) across three cross-sections. The locations of the cross-sections and current speeds on a flood tide are presented in Figure 3-6 whilst the pre- and post-development tidal discharge rate (m^3/s) throughout the modelled year (2015) is compared graphically for each cross-section in Figure 3-7.

Table 3-6 shows comparison of 98th (flood flow) and 2nd percentile (ebb flow) discharge rates between pre- and post-development for each defined cross-section. Data was extracted from the 12-month model simulation in year 2015.

The seawater intake increases the incoming tidal flux during the flood tide while decreases the discharge tidal flux on the ebb tide. The results demonstrate about a 4 to 7 percent increase in flood tidal fluxes and a 5 to 11 percent decrease in ebb tidal fluxes. At two downstream cross-sections Q1 and Q2, the relative difference is about 8 m^3/s which generally agrees with the 10.97 m^3/s intake flow rate. The upstream section Q3 shows fewer absolute impacts (2.5 m^3/s) while comparable relative impacts (percentage). These differences in tidal discharges are proportionally small and unlikely to significantly impact key habitats (as described in Section 3.4.1) or creek morphology (as described in Section 3.4.4 below).

At present, the embankment walls are positioned well beyond the regular tidal inundation zone at Urala Creek South. The primary impact to flood and ebb flow in Urala Creek South channel is associated with the seawater intake volumes rather than the effect of embankment walls.

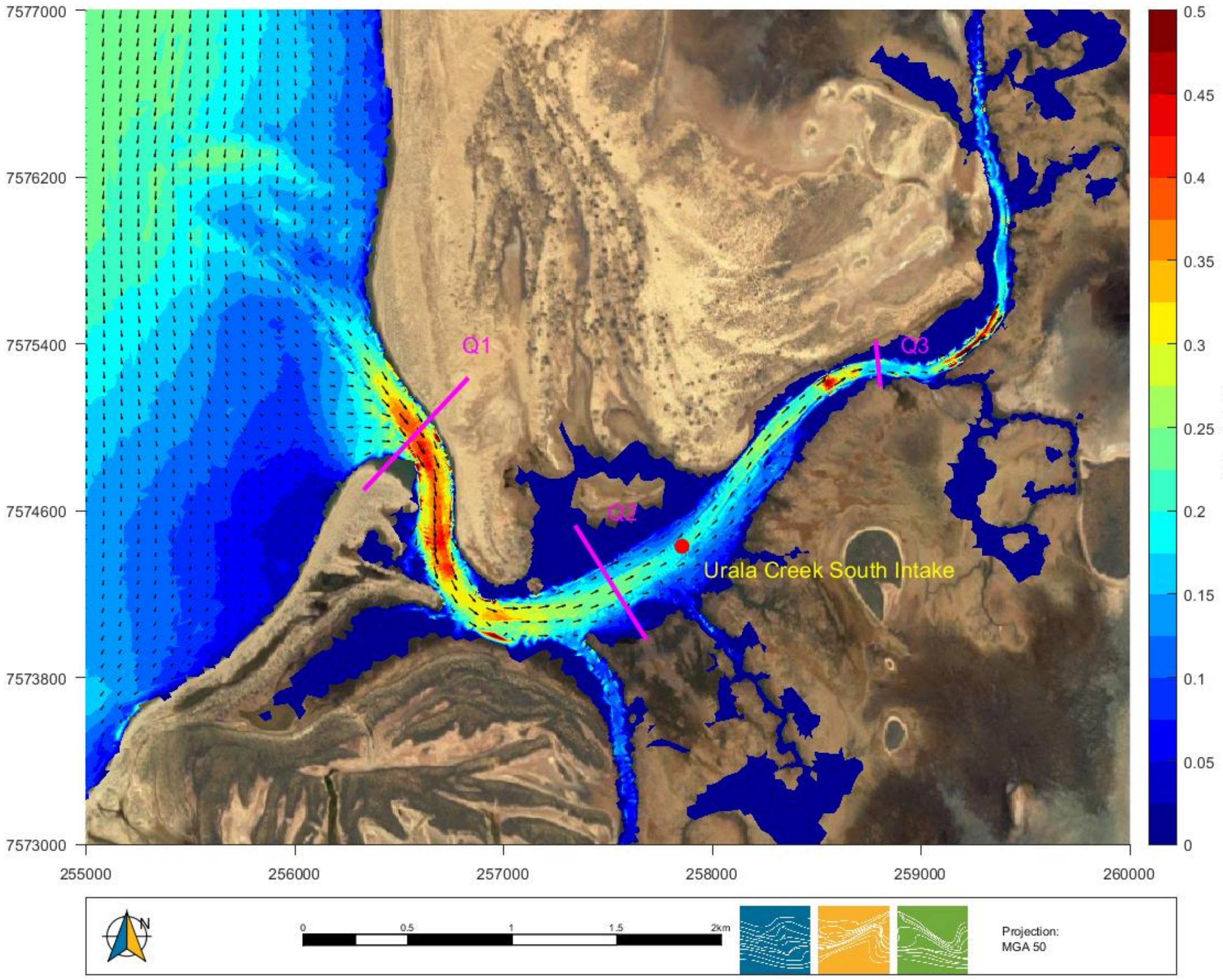


FIGURE 3-6 TIDAL DISCHARGE EXTRACTION LOCATIONS AND CURRENT SPEEDS DURING A FLOOD TIDE

TABLE 3-6 COMPARISON OF IMPACTS TO TIDAL DISCHARGE RATES AT Q1, Q2 AND Q3 LOCATIONS

Scenario	Cross Section	Existing, tidal discharge rate (m3/s)	Development, tidal discharge rate (m3/s)	Change in Discharge Rate
Flood Tide (98th Percentile)	Q1	178.2	185.1	4%
	Q2	103.0	110.0	7%
	Q3	35.7	38.2	7%
Ebb Tide (2nd Percentile)	Q1	-169.2	-160.8	5%
	Q2	-92.7	-82.6	11%
	Q3	-32.2	-28.7	11%

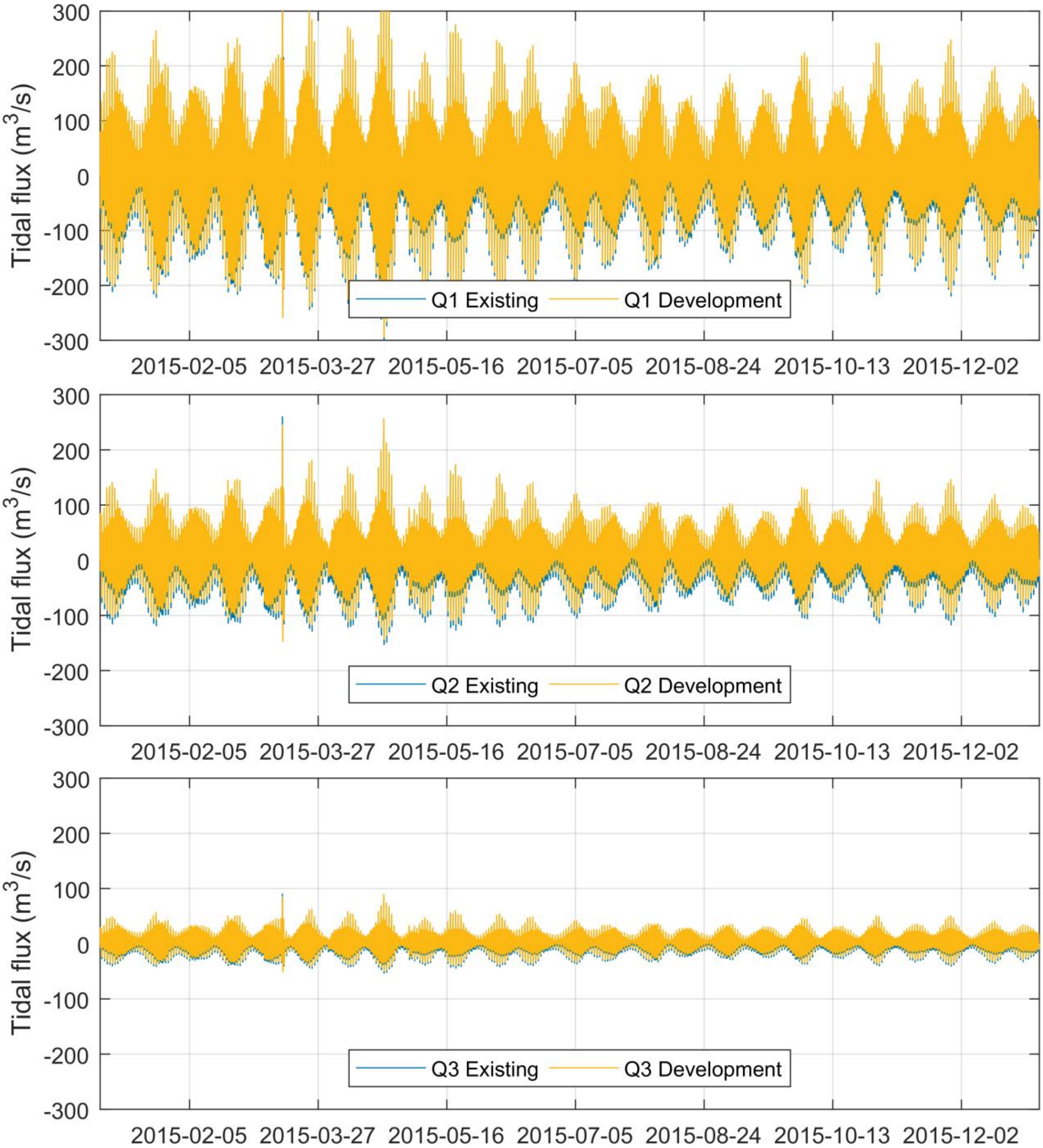


FIGURE 3-7 TIDAL DISCHARGES AT Q1, Q2 AND Q3 LOCATIONS

3.4.3 Modification to Tidal Inundation

The one-year simulation of tidal inundation with both the pond and seawater intake channel embankment walls in place, as well as seawater intake pumping from Urala Creek South, shows some predicted impacts on water movements along the seaward facing embankment walls and adjacent to the intake channel.

For neap, normal and spring tides outside of March/April, the pond embankment walls were located well beyond the reach of high tide water thereby showed very limited impact to the landward movement of water.

The water over the tidal flat connecting to Urala Creek North is so shallow that it was almost entirely driven by tidal movements and evaporation. The embankment walls showed very limited boundary impact to block the water flow as the flow is extremely weak near the upper limit of tidal inundation. This was evidenced by model results in Table 3-2 where water level was only marginally increased in this region even during an extreme (April/March) spring tide.

Due to the multiple flow paths causing inundation, there are no apparent effects of the embankment walls on the wetting of land on the seaward side of the walls. Except within the ponds themselves, none of the areas that flooded in existing scenarios will be dried out in the post-development scenarios.

The effect from embankment walls becomes somewhat more apparent when the water level is higher (e.g., during April spring high tide or storms). The impact can be expressed as a barrier effect to the water that would normally flow further inland into the proposed development area. This wall barrier effect elevates the water level against the embankment walls at peak storm tide however will not last once the high tide passes. According to model results shown in Table 3-2, this has led to about 4 km width water level increase (over 10 cm) along the northwest pond walls for a 20-year storm and even greater impacts (over 50 cm) during a 500-year cyclone event.

The impact was investigated further through comparison of inundation (% time inundated) between existing and development scenarios for the year 2015 (see Figure 3-8, Figure 3-9, Figure 3-10 and Figure 3-11).

Existing Conditions

- The duration of inundation is reduced by land elevation and also by distance from the sea.
- The main creek channels of Urala Creek North and South were inundated for about 100% of time due to their lower elevation. The channel profiles typically extend from approximately MSL (top of the creek bank) to below low tidal levels in the central sections of the channel.
- The tidal flat was partially inundated, and the duration of inundation varied substantially in spatially and seasonally. The tidal flat connecting to Urala Creek North showed the longest duration of inundation in particular at the north end, however this was skewed by seasonal inundation in March and April when water levels are significantly higher (see Section 2.4.1.4). Modelling in August and September showed almost no inundation of the salt flats.

Post-development Conditions

- The proposed development showed very minimal impacts to duration of tidal inundation. A marginal increase of inundation time (about 5%) was observed against the seaside embankment walls and over the tidal flat (in either salt flat or algal mat habitats) connecting to Urala Creek North.
- The greatest impact was found over a small area adjacent to the southern embankment of the seawater intake channel where water was retained due to blockage of a minor drainage path (refer to Figure 3-11 which is an enlargement of Figure 3-10 in this area).
- There is no predicted change to % time inundated for all mangrove habitat surrounding Urala Creek North and South, due to the large set back between the seaward embankments and the mangrove zone.
- Algal Mat is only inundated by spring high tide and during storms for a very shallow depth (~ 30 cm over the tidal flat connected to Urala Creek North and even shallower (<10 cm) for higher ground and area further in land). Impact to Algal Mat was mainly caused by physical barriers created by embankment walls which generates some additional inundation time from water ponding while in reality the impact would be even less due to water infiltration and microscale drainage paths not resolved by the model. The scale of impact is rather small, especially when compared to the overall footprint of the development.

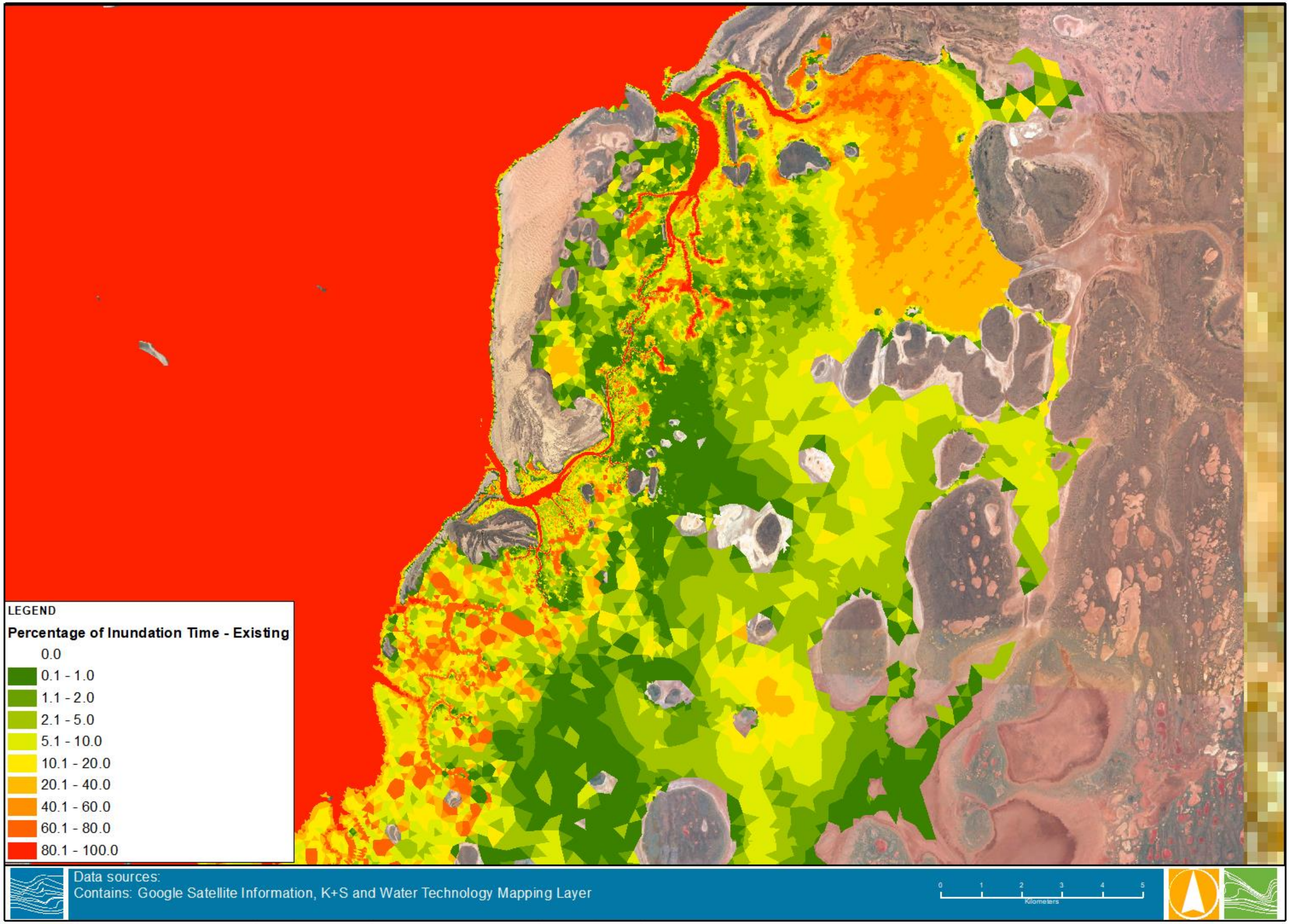


FIGURE 3-8 TIDAL INUNDATION TIME (PERCENTAGE OF TIME) IN YEAR 2015 - EXISTING CONDITIONS

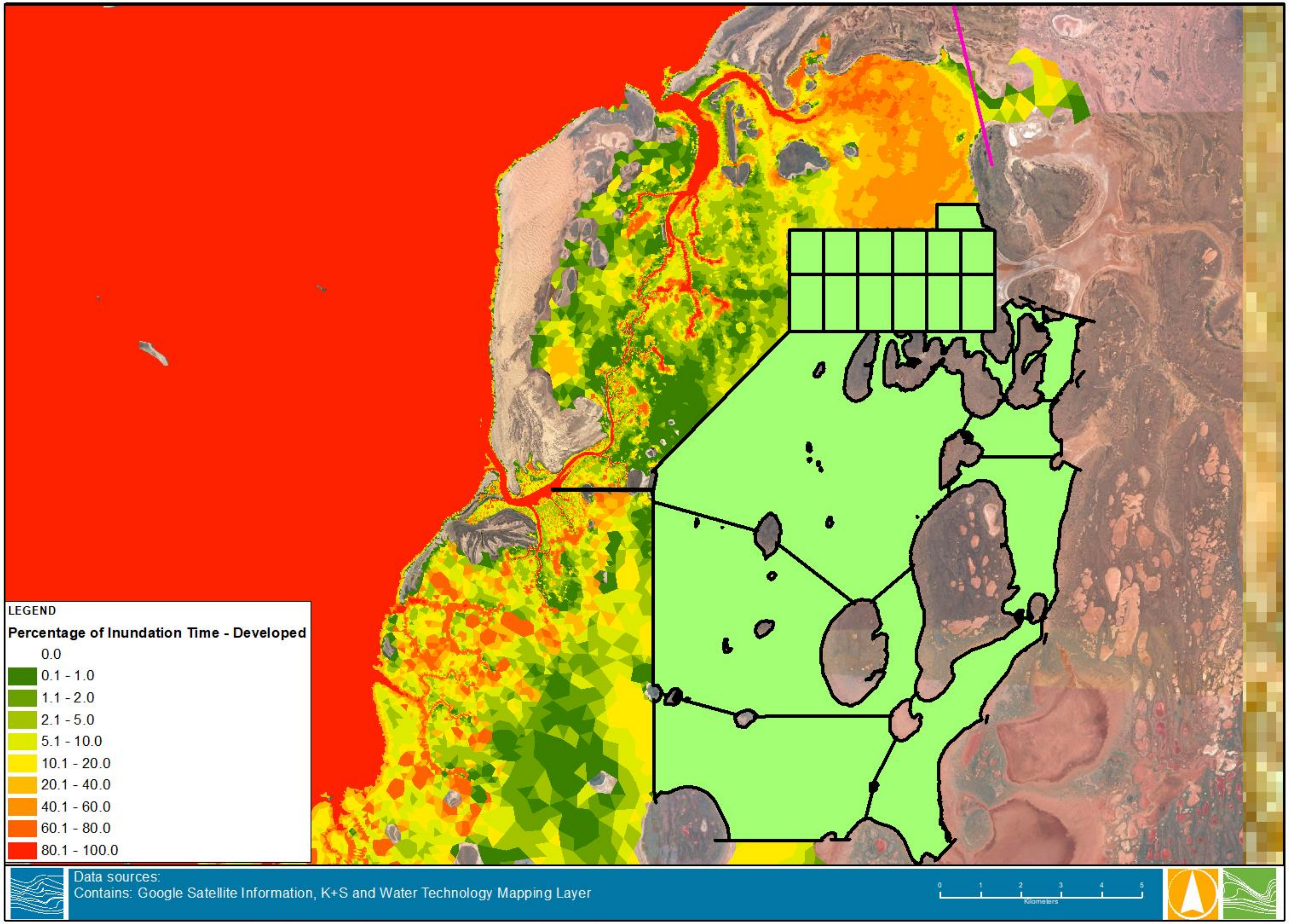


FIGURE 3-9 TIDAL INUNDATION TIME (PERCENTAGE OF TIME) IN YEAR 2015 – POST-DEVELOPMENT SCENARIO

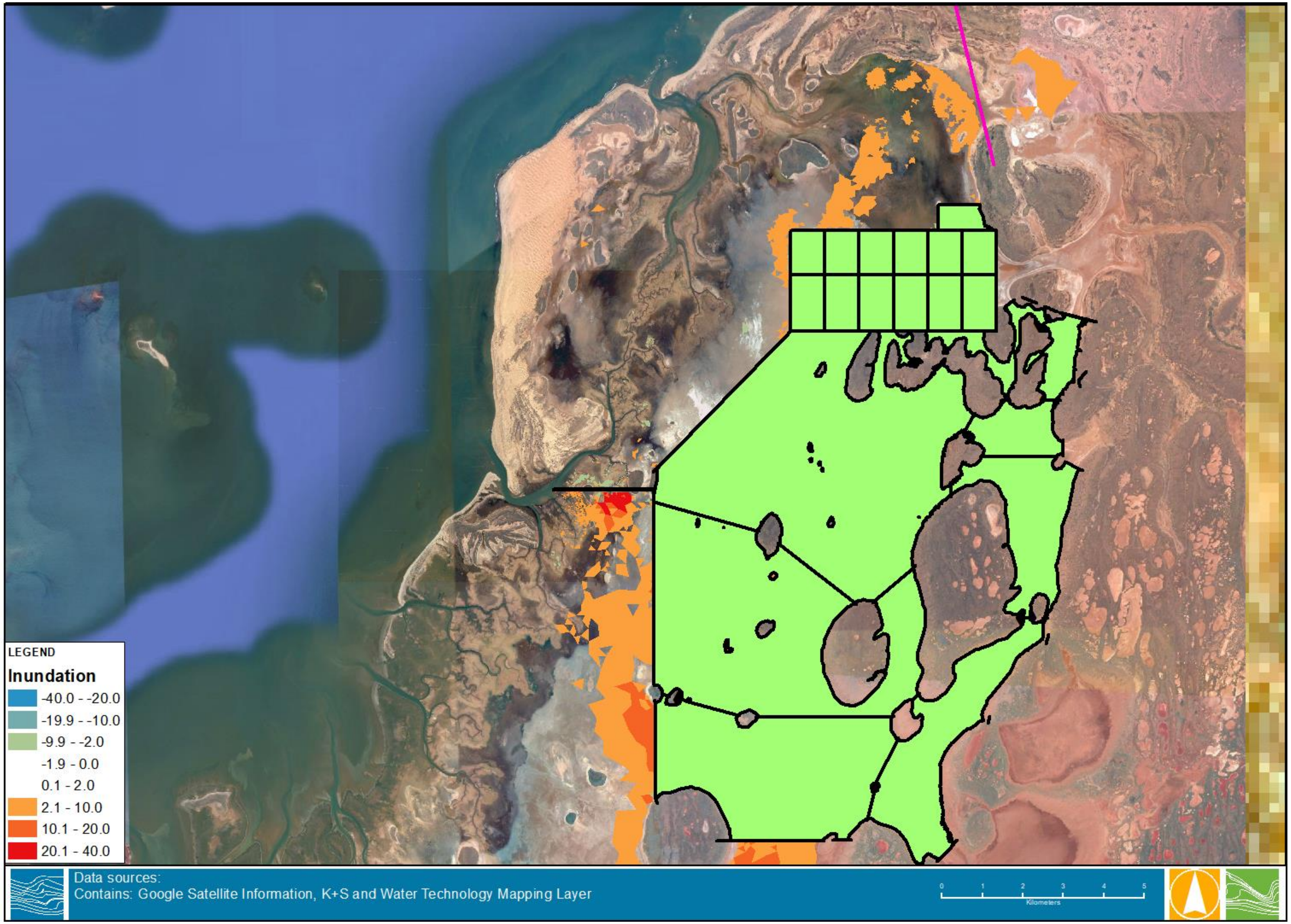


FIGURE 3-10 DIFFERENCE OF TIDAL INUNDATION TIME DUE TO PROJECT DEVELOPMENT IN YEAR 2015

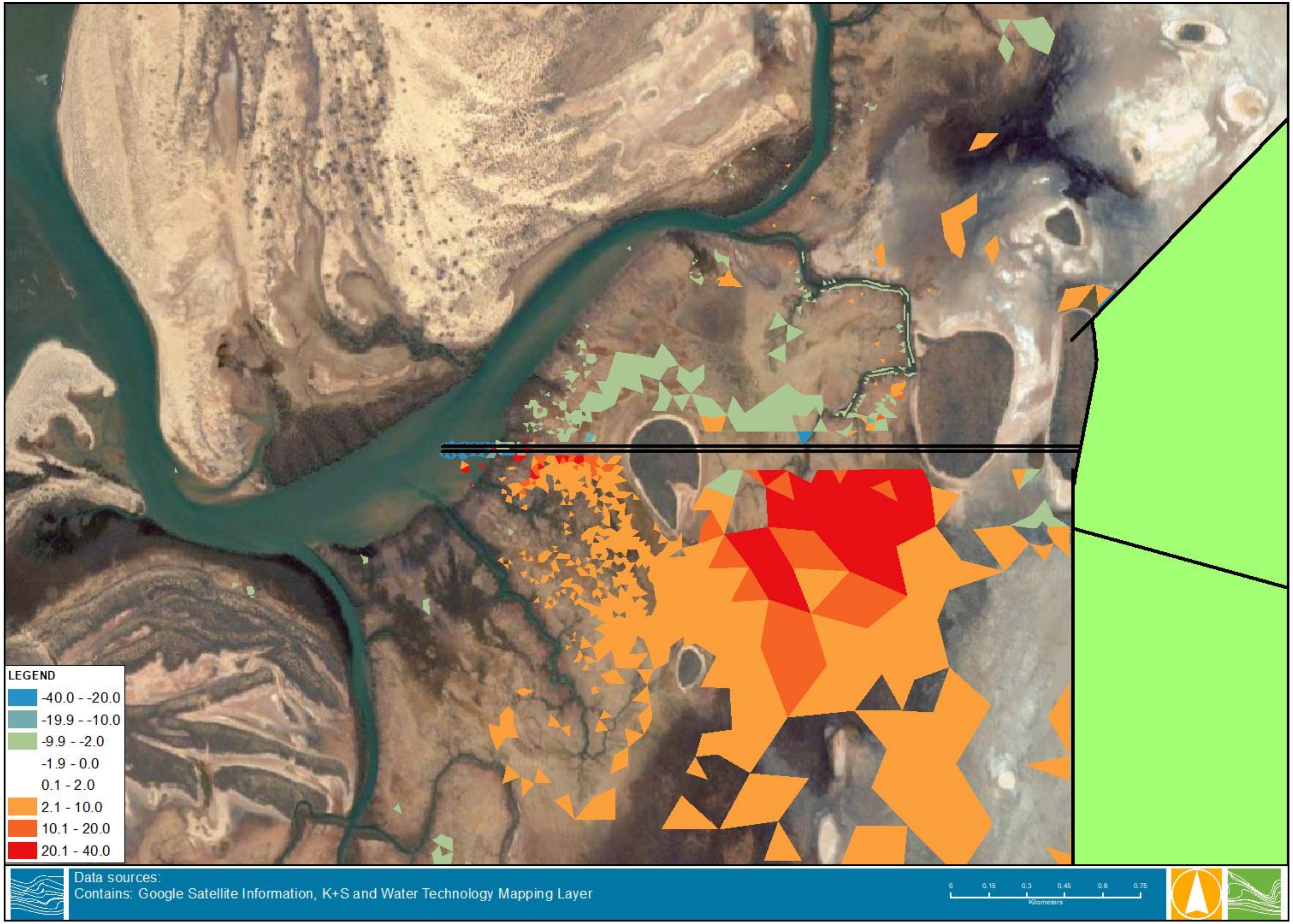


FIGURE 3-11 DIFFERENCE OF TIDAL INUNDATION TIME DUE TO PROJET DEVELOPMENT IN YEAR 2015 (NEAR THE INTAKE CHANNEL EMBANKMENT)

3.4.4 Impacts to Urala Creek South Morphology

Fluvial morphology in Urala Creek South is tide dominated and is occasionally affected by tropical storms. The creek is comprised of a main channel which shows very minimal changes in bank alignment since 2001 as outlined in *Marine, Coastal and Surface Water Existing Environment* (Water Technology, 2021). Mangrove habitats have stabilised the riverbank sediments within the dense mangal growth. The oscillatory tidal flow has attained a dynamic equilibrium showing no clear trend of bed level or shoreline evolution based on modelling and a literature review of existing coastal processes including historical satellite images. This is consistent with a geomorphic report by Seashore Engineering (2021), which notes that tidal channels across the area are remarkably stable.

Additional flood flows and reduced ebb flows related to the intake could potentially impact the current morphological equilibrium, however the modelling indicates these changes will be minimal (refer to Section 3.3.2.3).

Changes to creek hydrodynamics such as the increased flood current speeds predicted by the model have a small risk of riverbed scouring. A review of fluvial morphology factors and Lane's relation (a qualitative expression to relate water discharge, channel slope, sediment load, and a representative bed sediment size for a river reach under dynamic equilibrium conditions) indicates this will likely induce the same level of impact to sediment transport and river morphology, as predicted by our model. This evaluation however relies on riverbed materials and has limited applications to oscillatory tidal flow conditions. In the natural environment, fluvial scouring is affected by the sediment properties of erodible layers (grain sizes through depth), consolidation/sorting of bed materials and impacts from vegetation growth (e.g., mangrove root systems) which are all difficult to model. Notwithstanding this, given the current relatively stable state of the creek morphology, it is considered that the current model predictions provide a reasonable prediction of the likely small and localised morphological changes that may result from the project development suitable for environmental impact assessment purposes (refer to Section 3.3.2.3). Further sediment assessment and modelling is recommended prior to detailed design of the seawater intake pumping station.

The entire terrain is very flat and therefore inundation processes are sensitive to changes in mean sea level. A 0.4 m sea level rise (predicted SLR for 50 years – the project design life) will significantly increase the tidal prism and volume of water discharging through the creek channels. This will likely result in greater hydrodynamic impacts on the morphology of Urala Creek South, than the proposed seawater water intake. Tidal network adjustment (changes in creek and sub-creek morphology) is one of the potential responses to sea level rise expected between high energy sandy coasts and muddy wetland coasts (Rossington et al. 2009; Alizad et al. 2018; Leuven et al. 2019) and this will likely result in a natural adjustment of tidal flat level and formation of channel banks in some areas. All these changes are part of natural geomorphological response to future climate change which are not directly related to the proposed development.

3.4.5 Impacts to Shoreline at Locker Point

The jetty site at Locker Point is located in a weak energy environment with typical tidal currents of 0.1 m/s or less and significant wave heights (H_s) less than 0.6 m for the majority (99%) of the time. The ambient tides and waves would have a limited contribution to variations in the coastal morphology, which is also evidenced by a desktop review of historical shoreline variations as outlined in *Marine, Coastal and Surface Water Existing Environment* (Water Technology, 2021). The proposed development layout including construction of a pile-supported jetty, minor dredging at the berthing pocket, installation of discharge diffuser and bitterns discharge (<1 m^3/s) is predicted to have a limited influence on the hydrodynamic regime of the site since:

- The pile-supported jetty is a transmissive structure and therefore is predicted to provide no direct interruption to hydrodynamics, longshore sediment transport/littoral drift process.
- It is predicted there will only be marginal impacts to wave propagation across the berthing pocket due to its limited footprint of dredging offshore.

Sedimentation may occur during tropical storms when the bottom shear stress will be enhanced by storm waves. This will likely drive resuspension/transportation/deposition of seabed materials and periodic morphology variations. The shoreline impacts from the proposed jetty and berth facilities are expected to be minimal because the jetty is open to incoming waves and the piles won't obstruct the wave propagation towards the shore.

The proposed shore crossing of the infrastructure is presented in Figure 3-12. The jetty piles will cross the beach face; however, these are not anticipated to impact the coastal processes significantly as sediment will still be free to move between the piles. The proposed jetty abutment is an armoured structure. As per Figure 3-12, this is proposed to be located well behind the beach face, within the primary dune. Under present day sea level conditions, it is not anticipated that the jetty abutment impact coastal processes as it is beyond the limit of storm activity. Revegetation around the structure, combined with monitoring, is recommended to avoid any dune blowouts caused by wind action.

The predicted coastal hazard zone was calculated for the 50-year design life (Figure 3-13) in the vicinity of the abutment, as per the method outlined in Western Australian Planning Commission's State Planning Policy No. 2.6: State Coastal Planning Policy (WAPC, 2013, herein referred to as "SPP2.6"). The coastal hazard zone includes allowances for predicted short-term storm-induced erosion, historical shoreline movement and long-term shoreline retreat as a result of sea level rise. The jetty abutment protrudes approximately 45m seaward of the predicted 2070 coastal hazard line (refer Figure 3-13). It is noted this hazard zone is not a prediction of the 2070 shoreline, rather an area of potential erosion hazard by 2070. During the operational phase, if the present design were selected, coastal monitoring would be recommended to ensure longshore transport processes were not impacted as a result of the abutment. In addition, rock armour of the abutment should be designed to withstand wave impact and run-up, to combat the risk of shoreline retreat and potential exposure to wave action. It is noted the method of SPP2.6 is designed with conservatism in mind.

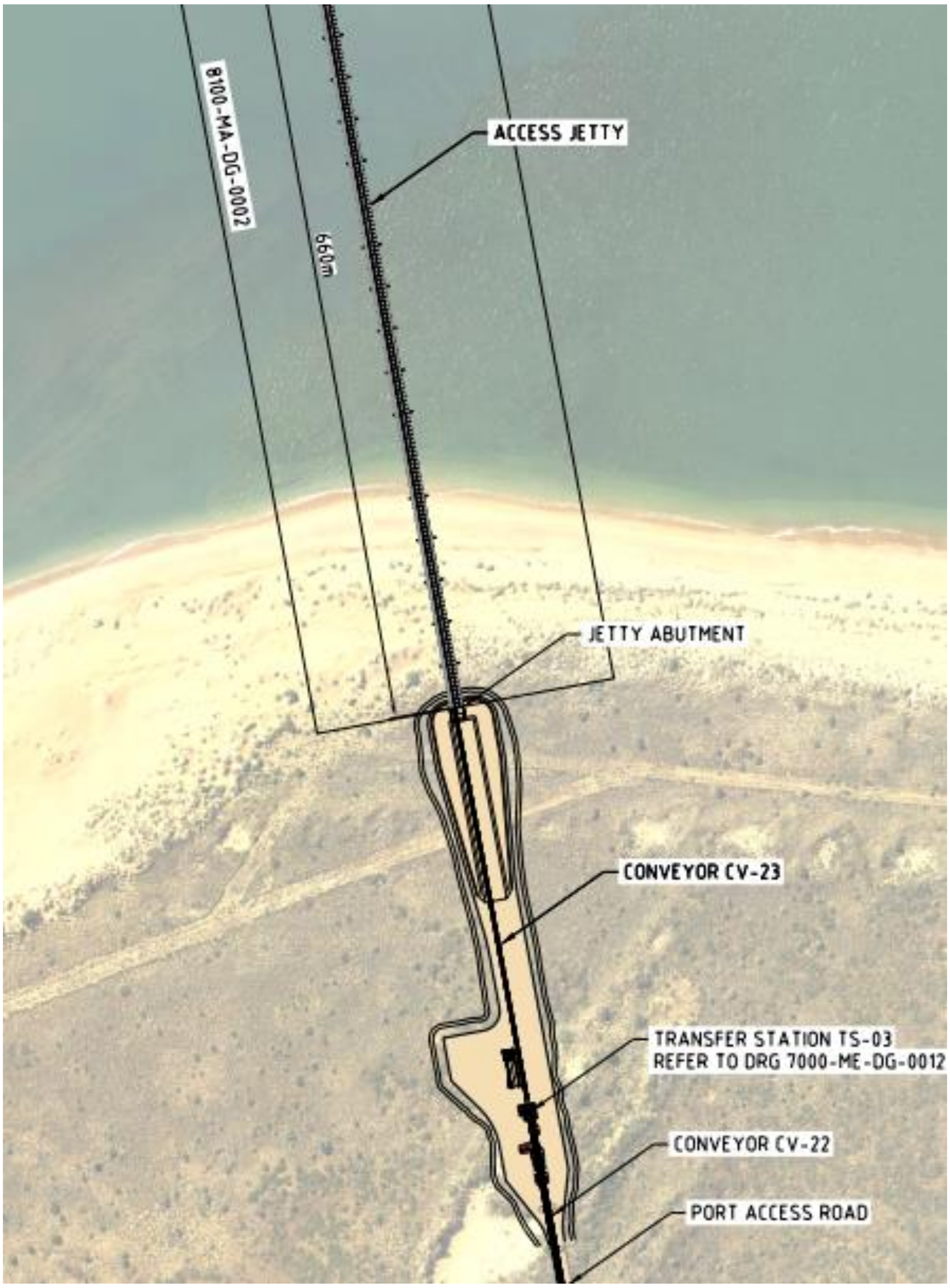


FIGURE 3-12 PROPOSED JETTY ABUTMENT (EXTRACT FROM K+S DRAFT DESIGN DRAWING RP21032-0000-MA-DG-0006)

FIGURE 3-13 COASTAL EROSION HAZARD ZONE FOR 50-YEAR DESIGN LIFE, AS PER SPP2.6 METHODOLOGY

4 BITTERNS DISCHARGE MODELLING

4.1 Study Approach

The Ashburton Salt Project will produce a hypersaline wastewater stream (bitterns). The bitterns are to be discharged from a diffuser located along the outer end of the proposed export jetty at Locker Point, as shown in Figure 4-1.

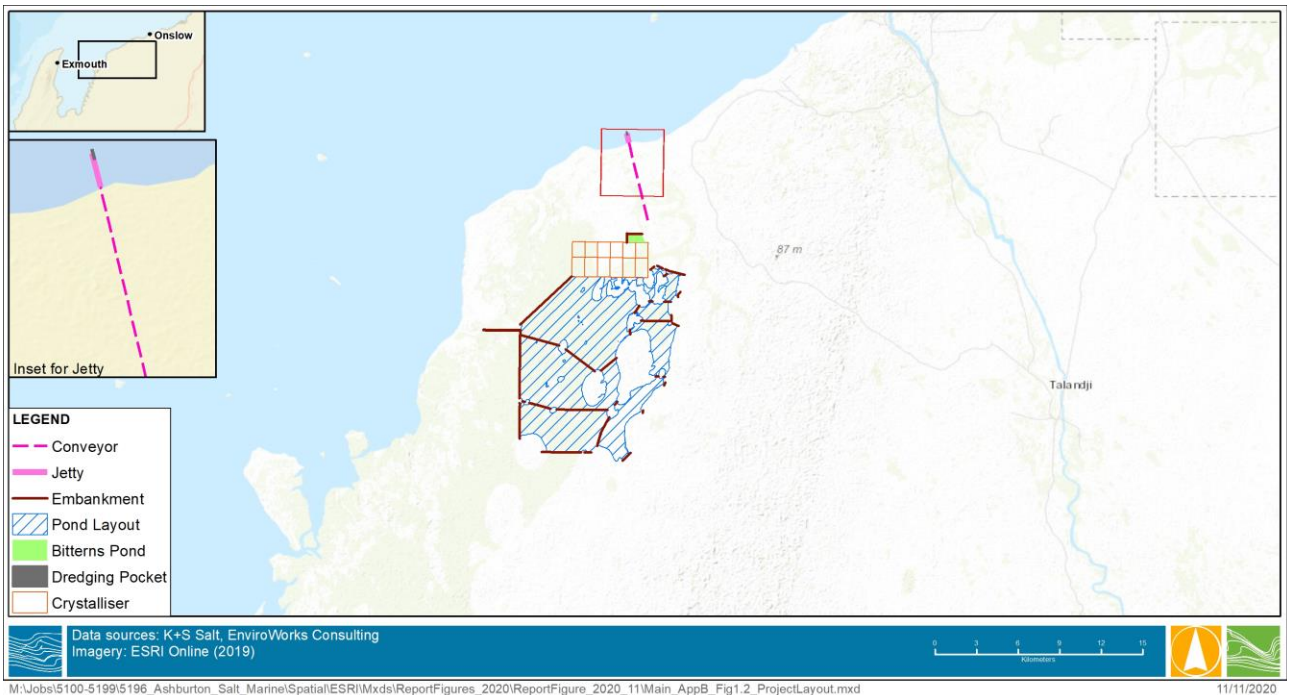


FIGURE 4-1 PROJECT LAYOUT AND JETTY

The bitterns solutions generally have a salinity of around 290 PSU and a density of 1,250 kg/m^3. They are markedly denser than the local seawater, which in the area has natural range of 35.0 to 53.5 PSU and a corresponding range in density of 1,027 to 1,041 kg/m^3.

Being denser than the receiving seawater (negatively buoyant), the bitterns discharge will behave in a similar manner to the wastewater discharge from a desalination plant. The bitterns are, however, significantly more saline and denser than the wastewater from a desalination plant which, typically, may have a salinity of around 70 PSU and a density of 1,050 kg/m^3.

The high salinity of bitterns discharges makes it extremely challenging to achieve adequate dilution within a relatively small mixing zone. This typically requires the use of either:

- Larger mixing zones than for other less saline brine discharges; and/or
- Novel approaches to achieve greater mixing, such as pre-dilution, longer diffusers, or multiple diffusers.

Water Technology has worked together with K+S to develop a design concept for a diffuser that will mitigate the effects that the bitterns discharge will have on the water quality of receiving environment. This work has included:

- Characterising the chemical composition and discharge properties of the bitterns (Section 4.2).
- Setting Environmental Quality Guidelines (EQGs) for evaluating the performance of the diffuser (Section 0).

- Defining the receiving water physical and chemical characteristics (Section 4.4).
- Development of an outfall diffuser concept design, through:
  - Nearfield modelling to optimise the mixing of individual discharge ports (Section 4.6).
  - Far field modelling to evaluate the overall performance of the diffuser design (Section 4.7).
- Use of the far-field model to develop mixing zone contours (Section 4.7.5.2).

4.2 Bitterns Characterisation

Whilst bitterns generated by solar salt projects generally exhibit the same characteristics at a macro level, the constituents can vary at a micro level due to varying constituents in the source seawater. In order to understand the chemical characteristics of the bitterns that will be produced by the Ashburton Salt Project, a bitterns sample was generated from a sample of local seawater and subsequently analysed.

4.2.1 Bitterns Generation

A 30 L sample of local seawater (from the location of the proposed seawater intake) was collected by AECOM and provided to Analytical Reference Laboratory (ARL). ARL concentrated the sample using evaporation, to mimic the process by which bitterns are created. Salt was precipitated (crystallised) and removed. The evaporation process was continued until the remaining solution reached a density typical of bitterns (1,248 kg/m^3). The bitterns sample was then tested for levels of expected chemical composition to confirm it was representative of bitterns constituents at expected levels (based on known constituent levels of bitterns analysed from other salt projects).

The comparison of the main constituents of the bitterns is presented in Table 4-1. This shows that the chemical composition of the bitterns sample generated is representative of the expected composition of bitterns, with some minor variations as expected given each bitterns sample generated using different source seawater will be slightly different.

TABLE 4-1 BITTERNS CONSTITUENTS OF LABORATORY CREATED BITTERNS AND EXPECTED LEVELS

Main Constituent	Laboratory Created Bitterns (mg/L)	Average Level Expected of Bitterns based on other Solar Salt Projects (mg/L)
Na	91,000	80,000
Mg	37,000	41,000
K	8,800	10,000
Ca	210	350
Cl	220,000	230,000
SO_4	44,000	48,000
TDS	450,000	450,000
Density (g/cm^3)	1.248	1.248

4.2.2 Laboratory Testing

ARL carried out physical and chemical testing of the laboratory generated bitterns sample. The results are presented in Table 4-2. From an analysis of these results, two key water quality parameters were identified as needing to be assessed and regulated. These were:

- Salinity: a physical/chemical (PC) stressor

- Metals: toxicants

TABLE 4-2 SUMMARY OF LABORATORY TEST RESULTS

Parameter	Unit	PQL	Result	Parameter	Unit	PQL	Result
pH	pH units	0.1	6.8	Aluminium - Total	mg/L	0.01	0.06
Conductivity	mS/cm	0.01	190	Manganese - Total	mg/L	0.01	0.04
Total Dissolved Solids	mg/L	5	450,000	Manganese - Dissolved	mg/L	0.01	0.04
Alkalinity	mg $CaCO_3$/L	5	490	Tin - Total	mg/L	0.01	<0.01
Bicarbonate	mg $CaCO_3$/L	5	490	Tin - Dissolved	mg/L	0.01	<0.01
Carbonate	mg $CaCO_3$/L	5	<5	Vanadium - Total	mg/L	0.01	0.01
Hydroxide	mg $CaCO_3$/L	5	<5	Zinc - Total	mg/L	0.005	0.024
Chloride	mg/L	5	220,000	Arsenic - Total	mg/L	0.001	0.009
Sulfate	mg/L	1	44,000	Chromium - Total	mg/L	0.001	0.001
Filterable Reactive Phosphorus	mg/L	0.01	0.01	Cobalt - Total	mg/L	0.001	<0.001
Ammonia-N	mg/L	0.02	0.17	Cobalt - Dissolved	mg/L	0.001	<0.001
Nitrate-N	mg/L	0.01	0.4	Copper - Total	mg/L	0.001	0.015
NOx-N	mg/L	0.01	0.44	Copper - Dissolved	mg/L	0.001	0.015
Nitrite-N	mg/L	0.01	0.24	Lead - Total	mg/L	0.001	<0.001
Bromide	mg/L	0.1	3,600	Lead - Dissolved	mg/L	0.001	<0.001
Total Nitrogen	mg/L	0.2	0.6	Nickel - Total	mg/L	0.001	0.007
Total Kjeldahl Nitrogen	mg/L	0.2	<0.2	Nickel - Dissolved	mg/L	0.001	0.005
Total Phosphorus	mg/L	0.01	0.16	Cadmium - Total	mg/L	0.0001	0.0005
Sodium - Total	mg/L	0.1	91,000	Mercury - Total	mg/L	0.0001	<0.0001
Sodium - Dissolved	mg/L	0.1	91,000	Mercury - Dissolved	mg/L	0.0001	<0.0001
Calcium - Total	mg/L	0.1	210	Selenium - Total	mg/L	0.001	<0.001
Calcium - Dissolved	mg/L	0.1	210				
Magnesium - Total	mg/L	0.1	37,000	Note: PQL = Practical Quantitation Limit			
Magnesium - Dissolved	mg/L	0.1	37,000				
Potassium - Total	mg/L	0.1	8,800				
Potassium - Dissolved	mg/L	0.1	8,800				

4.2.3 Bitterns Discharge Characteristics

The bitterns to be discharged from the diffuser will comprise bitterns, as described above, and a smaller amount of washwater. Here, washwater is clean seawater that has been used to wash the harvested salt to remove any residual bitterns, and to remove any Potassium Chloride crystals that may have grown during transport. The washwater will have similar chemical characteristics to the bitterns, described above, but at a somewhat lower concentration. There will be no additional chemicals or organic material.

Bitterns and washwater generation will vary with seasonal variations in evaporation and salt production. The monthly variations in the main properties of the proposed bitterns discharges were provided by K+S. These were based on the targeted seasonal production rates. Details of the main bitterns and washwater discharge properties are summarised in Table 4-3. The highest bitterns discharge will be in November, which is the month with the highest production rate. This is likely to be the "worst case" scenario and has been used in the assessment of diffuser performance.

K+S proposed two main discharge scenarios to be considered in the preliminary design of outfall diffuser:

a. No predilution: In this scenario, bitterns (with an average density of around 1.25 kg/m^3) would be combined with a smaller amount of washwater (with an average density of 1.19 kg/m^3) and discharged directly from the outfall diffuser. The combined effluent would have an average density of about 1,235 kg/m^3, with discharge rates ranging from about 0.08 m^3/s in June to about 0.54 m^3/s in November.

b. Prediluted by seawater: In this scenario, the bitterns would be diluted 1:1 with an equal amount of seawater before being combined with the washwater and discharged from the diffuser. The resulting effluent will have an average density of about 1,135 kg/m^3, with discharge rates ranging from about 0.14 m^3/s in June to about 0.98 m^3/s in November.

TABLE 4-3 MONTHLY DISCHARGE PROPERTIES

Discharge (no pre-Dilution)							
Month	Temp (°C)	Bitterns (m^3/s)	Washwater (m^3/s)	Seawater (m^3/s)	Total (m^3/s)	Salinity (PSU)	Density (kg/m^3)
January	27	0.4	0.08	0	0.48	287.1	1235
February	27	0.15	0.03	0	0.18	287.1	1235
March	28	0.32	0.07	0	0.39	287.1	1235
April	27	0.17	0.05	0	0.23	287.1	1235
May	25	0.1	0.03	0	0.13	287.1	1235
June	23	0.06	0.02	0	0.08	287.1	1235
July	22	0.13	0.03	0	0.16	287.1	1235
August	21	0.18	0.05	0	0.23	287.1	1235
September	22	0.29	0.07	0	0.36	287.1	1235
October	23	0.4	0.09	0	0.49	287.1	1235
November	24	0.44	0.09	0	0.54	287.1	1235
December	25	0.42	0.09	0	0.52	287.1	1235
Discharge (with pre-Dilution)							
Month	Temp (°C)	Bitterns (m^3/s)	Washwater (m^3/s)	Seawater (m^3/s)	Total (m^3/s)	Salinity (PSU)	Density (kg/m^3)
January	27	0.4	0.08	0.4	0.88	174.5	1135
February	27	0.15	0.03	0.15	0.33	174.5	1135
March	28	0.32	0.07	0.32	0.71	174.5	1135
April	27	0.17	0.05	0.17	0.4	174.5	1135
May	25	0.1	0.03	0.1	0.23	174.5	1135
June	23	0.06	0.02	0.06	0.14	174.5	1135
July	22	0.13	0.03	0.13	0.29	174.5	1135
August	21	0.18	0.05	0.18	0.41	174.5	1135
September	22	0.29	0.07	0.29	0.65	174.5	1135
October	23	0.4	0.09	0.4	0.89	174.5	1135
November	24	0.44	0.09	0.44	0.98	174.5	1135
December	25	0.42	0.09	0.42	0.94	174.5	1135

4.3 Setting of Environmental Quality Criteria

The regulatory framework for the proposed discharge of bitterns is contained within:

- Australian and New Zealand Guidelines for Fresh and Marine Water Quality (ANZG, 2018)
- Technical Guidance – Protecting the Quality of Western Australia's Marine Environment (EPA, 2016).

According to EPA (2016), the predicted concentration of constituents at the point of discharge will be compared to Environmental Quality Criteria (EQC) which are scientifically based limits of acceptable change to an environmental quality indicator.

A fundamental requirement of EQC is that they should be clear, readily measurable, and auditable. Wherever possible there should be a standardised approach to measurement of the indicator and for comparison of the resulting data against the EQC (EPA, 2016).

EQC are divided into relatively simple and easy to measure environmental quality guidelines (EQG) and more robust environmental quality standards (EQS) described below (EPA, 2016):

- **Environmental Quality Guidelines (EQG)** are threshold numerical values or narrative statements which if met indicate there is a high degree of certainty that the associated environmental quality objective has been achieved. If the guideline is not met, then there is uncertainty as to whether the associated environmental quality objective has been achieved and a more detailed assessment against an environmental quality standard is triggered. This assessment is risk-based and investigative in nature (EPA, 2016).
- **Environmental Quality Standards (EQS)** are threshold numerical values or narrative statements that indicate a level which if not met indicates there is a significant risk that the associated environmental quality objective has not been achieved and a management response is required. The response would normally focus on identifying the cause (or source) of the exceedance and then reducing loads of the contaminant of concern (i.e., source control) and may also require in-situ remedial work to be undertaken (EPA, 2016).

The sections below outline proposed EQCs in the form of Environmental Quality Guidelines (EQGs).

4.3.1 Regulatory Framework: Physical and Chemical Stressors

ANZG (2018) state that an unnatural change in salinity (potentially leading to a change in biological diversity) is a PC stressor. Salinity is the key PC stressor identified from the proposed bitterns discharge for the Ashburton Salt Project.

The ANZG (2018) default approach for developing guideline values for PC stressors uses reference data. In other words, the recommended approach to derive guideline values is to calculate an appropriate percentile of measured reference site water quality data. This is the same approach supported by the WA EPA which recommends that PC stressors should remain within the 80^{th} percentile of natural background for a high Level of Environmental Protection. ANZG (2018) regard this approach as inherently conservative, but often a good starting point when field based biological-effects data is not available (ANZG, 2018).

Guideline values for PC stressors are rarely derived using laboratory data, with a historical and current preference for using a referential approach. Guideline values for physical and chemical stressors can be derived and refined using field effects data. That is obtaining ecological-effects data through site or ecosystem-specific field monitoring studies and experiments. Locally relevant published scientific literature on ecosystem tolerance can be used where available. The key benefit of field-effects, particularly when multiple lines of evidence are being considered, is greater environmental realism. However, trade-offs exist in acquiring field effects data, including logistical issues, costs and confounding by variables other than the contaminant of interest (ANZG, 2018).

4.3.2 Regulatory Framework: Toxicants

Given no additives are introduced during the solar salt production process, the only toxicants that exist in the bitterns wastewater are naturally occurring elements of seawater (specifically metals). Metal toxicity or metal poisoning is the toxic or poisonous effect of certain metals in certain forms and doses on living organisms. Metals can bioaccumulate in the marine environment, contributing to their potential toxicity.

ANZG (2018) provides default guideline values for assessing a range of toxicants in marine waters. Specifically, for metals in marine waters ANZG (2018) default guideline values are provided for Aluminium, Antimony, Arsenic, Boron, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Mercury, Molybdenum, Nickel, Selenium, Silver, Thallium, Tributyltin, Uranium, Vanadium and Zinc.

Laboratory effects data from single-toxicant and single-species ecotoxicity laboratory tests underpin most of the information used by ANZG (2018) to derive toxicant water quality default guideline values. Species sensitivity distributions (SSDs) of chronic laboratory ecotoxicity data have been used by ANZG to derive default guideline values that will protect 80, 90, 95 or 99% of species. The percent level of protection adopted is then applied according to the current or desired ecosystem condition and associated level of protection (ANZG, 2018).

The use of the ANZG default guideline levels is supported by the WA EPA which recommends that 99% species protection levels are adopted for a high Level of Environmental Protection with the exception of cobalt where 95% species protection levels are recommended (EPA, 2016).

4.3.3 Approach for this Project

For the Ashburton Salt Project, Environmental Quality Guidelines (EQGs) have been proposed as recommended by ANZG (2018) and EPA (2016) as follows:

- For the PC stressor salinity, a referential approach has been followed, involving the collection of 20 months of baseline data at the proposed bitterns discharge location (Locker Point). This has enabled the development of a baseline salinity dataset, enabling the calculation of percentiles of the dataset.
- For toxicants (metals), ANZG (2018) default guideline values for appropriate species protection levels have been proposed.

EPA (2016) defines an Environmental Quality Plan (EQP) as a plan that identifies the environmental values that apply to an area and spatially maps the zones where the Environmental Quality Objectives (EQOs including Levels of Ecological Protection - LEP) should be achieved. The EPA has already established an EQP for the Pilbara coastal waters which assigns a “High” LEP to the proposed bitterns discharge area at Locker Point (DoE, 2006).

4.3.4 Salinity – Proposed Interim EQGs

The salinity of receiving environment at the proposed bitterns discharge location (Locker Point) changes throughout the year. The monitoring data presented in *Marine, Coastal and Surface Water Data Collection* (Water Technology, 2021) shows salinities at Locker Point varying typically from around 36 PSU, up to about 42 PSU. There is, however, no clear seasonal pattern as the salinity can vary significantly from one year to another. For instance, the in-situ water quality monitoring probe (sonde probe) data presented in Water Technology (2021) shows salinities of around 37 PSU during November 2018 increasing to around 41 PSU in November 2019. Salinity profiles at Locker Point show minimal vertical variations in salinity, indicating well-mixed seawater near the diffuser location.

In addition to the in-situ (sonde) monitoring data collected, samples were also collected and sent to a NATA accredited laboratory for water quality analysis. As a quality control check, an analysis was undertaken of laboratory data and the in-situ (sonde) field salinity data collected over 20 months at Locker Point. Quality controlled Total Dissolved Solids (TDS) results obtained by laboratory gravimetric analysis of water samples were similar to the in-situ TDS measurements, providing reasonable confidence in the baseline in-situ results (laboratory TDS averaged 39,211 mg/L whilst in-situ results averaged 38,096 mg/L).

To account for natural variation, the bitterns EQG salinity trigger is prescribed as an allowable level above background. The accepted method of calculating the allowance above background is determining the difference between the appropriate percentile and the median of the baseline dataset as this represents the range of natural variation. The appropriate percentile and median are calculated as each new data point is collected and a rolling 12-month average of the selected percentile minus the median is established.

The WA EPA recommends that PC stressors such as salinity should remain within the 80th and 95th percentile of natural background for a High and Moderate Level of Ecological Protection (LEP) respectively. In addition, EPA recommends no EQG's should apply for Low LEPs (EPA, 2016).

The rolling 12-month average of the 80th and 95th percentile minus the median has been calculated for the in-situ baseline salinity dataset at the bitterns discharge reference site (Locker Point) as shown in Figure 4-2. On this basis:

- The High LEP salinity EQG was calculated as 1.6 PSU above background (being the 12-month rolling average of the difference between the rolling 80th percentile and rolling median of the dataset).
- The Moderate LEP salinity EQG was calculated as 2.2 PSU above background (being the 12-month rolling average of the difference between the rolling 95th percentile and rolling median of the dataset).

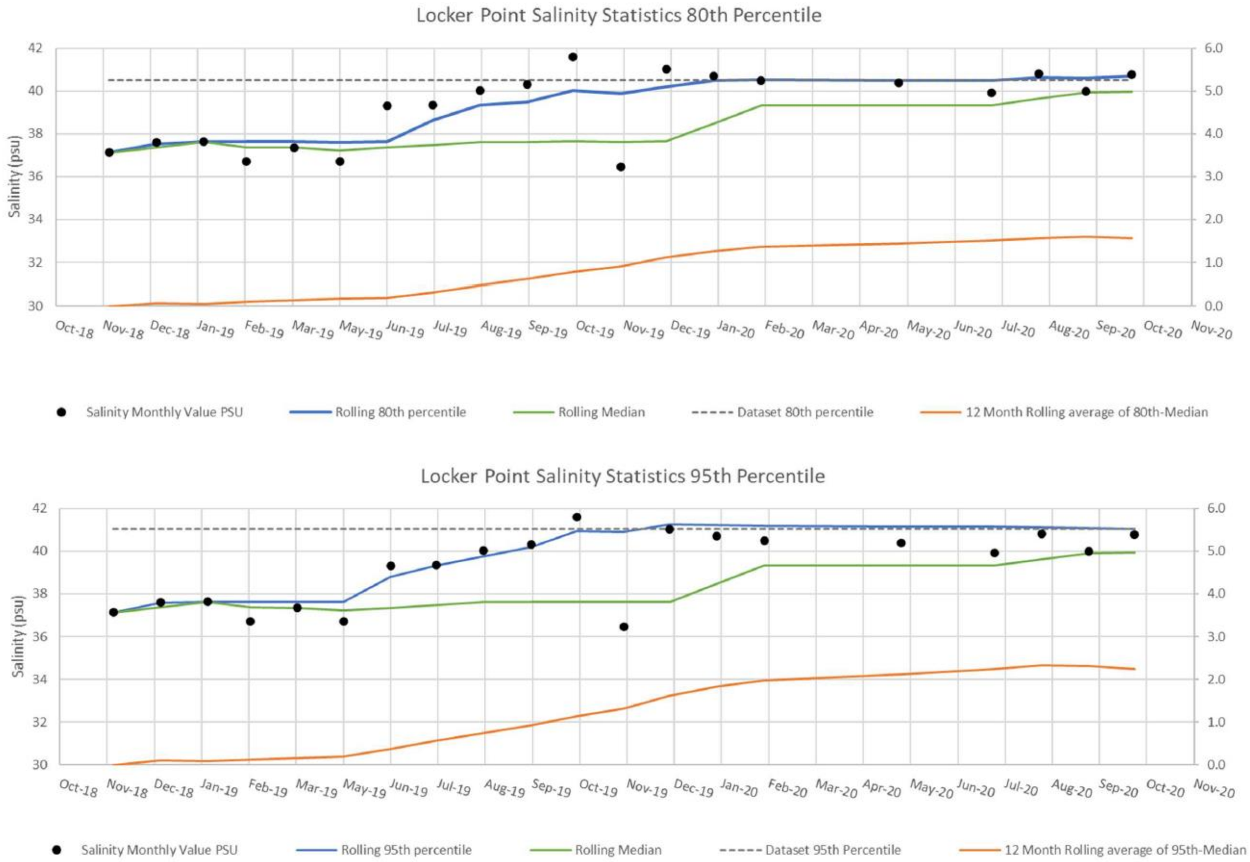


FIGURE 4-2 SALINITY STATISTICS AT LOCKER POINT BASED ON MONTHLY SONDE MEASUREMENTS

4.3.5 Metals (Toxicants) – Proposed Interim EQGs

ANZG (2018) provides default guideline values for assessing a range of toxicants (including metals) in marine waters. The use of these ANZG default guideline levels is recommended by the WA EPA (2016) which states that:

- For a High LEP: 99% species protection levels are adopted with the exception of cobalt where 95% species protection levels are recommended (EPA, 2016).
- For a Moderate LEP: 90% species protection levels are adopted (EPA, 2016).

For the Ashburton Salt Project, metals EQGs have been formulated by AECOM (the project's marine science advisor) using ANZG (2018) default guideline values (DGVs) with some minor variations as described below:

- At present there is no default guideline specified for aluminium in ANZG (2018). A low reliability screening level is presented in ANZG (2018) however, it is noted that this is not based on sufficient data such that it should be considered a trigger level (Wenziker et. al. 2006). A more recent publication presents a study combining chronic biological effects data generated over several years with toxicity data from the open literature to construct species sensitivity distributions (SSDs) which enabled the computation of revised water quality guidelines for aluminium (van Dam et. al. 2018). A guideline concentration of 0.002 mg/L was derived for a 99% species protection level in tropical waters and this guideline concentration has been recommended as appropriate for this project by AECOM the marine science advisors for the Ashburton Salt Project.
- Monitoring shows that baseline copper levels at Locker Point regularly exceed the ANZG (2018) default 99% guideline value of 0.0003 mg/L. On this basis use of a High LEP EQG based on a less stringent 95% species protection ANZG (2018) default guideline level has been recommended as appropriate for this project by AECOM the marine science advisors for the Ashburton Salt Project.

The proposed metals EQGs are outlined below in Table 4-4.

AECOM have also calculated dilutions of the bitterns plume required in order to achieve the proposed EQGs, based on pre-dilution with 1 part seawater and the measured metals levels within the bitterns sample that was created from local seawater as described in Table 4-2. These dilutions are also outlined below in Table 4-4.

TABLE 4-4 PROPOSED INTERIM METALS EQG'S AND DILUTIONS REQUIRED

Metal	High LEP Interim EQGs			Moderate LEP Interim EQGS			
	Proposed EQG (mg/L)	% Species Protection Level (ANZG, 2018)	Dilution Required in Plume	Proposed EQG (mg/L)	% Species Protection Level (ANZG, 2018)	Dilution Required in Plume	Notes
Aluminium	0.002	99	17.7	0.051	90	0*	EQG based on van Dam et al (2018)
Manganese	0.08	99	0*	NL	90	0*	
Vanadium	0.05	99	0*	NL	90	0*	
Zinc	0.007	99	2.3	0.023	90	0*	
Arsenic	0.0023	99	6.7	N/A	90	N/A	Lower EQG for As III applied
Chromium	0.0001	99	38	0.02	90	0*	Lower EQG for Cr VI applied
Cobalt	0.001	95	0*	NL	90	0*	
Copper	0.0013	95	19.7	0.003	90	1.55	
Lead	0.0022	99	0*	NL	90	0*	
Nickel	0.007	99	0*	NL	90	0*	
Cadmium	0.0007	99	0*	NL	90	0*	
Mercury	0.0001	99	0*	NL	90	0*	
Selenium	0.003	99	0*	NL	90	0*	

Table Notes
* concentration of the metal in the pre-diluted bitterns is less than the ANZG (2018) default guideline
NL Not listed because dilution contours not required to be provided *
N/A Not applicable. Low reliability screening level in ANZG 2018 for As III is 0.0023 mg/L (same as High LEP level)
Blue text denotes modelling outputs provided

4.3.6 EPA Guidelines for Low and Moderate LEP Zone Sizes

EPA (2016) states that:

- A Low Ecological Protection Area (LEPA) should be considered around a wastewater discharge. The zone should be as small as possible usually extending no more than 70 m from the diffuser, often tied to the zone of initial near-field dilution.
- A Moderate Ecological Protection Area (MEPA) may be applied to relatively small areas within inner ports or adjacent to heavy industrial premises. The MEPA typically extends up to 250 m from ship turning basins and berths.

4.4 Receiving Water Characteristics

The bitterns are to be discharged from the location of the salt export jetty at Locker Point. Key properties of the receiving waters in this vicinity have been summarised as follows.

4.4.1 Water Depths

A bathymetric survey was used to determine the water depths in the area of the proposed export jetty. The results of the survey are presented in Figure 4-3. An all-tide berthing pocket for the salt transhipper is to be located along the outer 150 m of the eastern side of the export jetty. This will require depths dredged to about 7.5 m relative to Mean Sea Level (MSL). The bitterns outfall diffuser is to be located along the outer 400 m of the north-east side of the jetty. Figure 4-3 shows that water depths along this section of the jetty are in the range of 4.0 m to 5.2 m, with an average of about 5.0 m relative to MSL.

The tidal range in the vicinity of the outfall varies from -0.2 m during neap tides to over 1.8 m during spring tides. As a result, the available depth for nearfield mixing will range from about 4.0 m at low water spring tides to over 4.5 m at low water neap tides.

FIGURE 4-3 EXISTING SEABED ELEVATIONS NEAR THE DIFFUSER (M MSL)

4.4.2 Currents

Hydrodynamic modelling has predicted that tidal currents in the area are in the order of 0.05 m/s. Current directions are aligned with the coastline and alternate with the phase of the tide. Non-tidal, residual currents appear to be dominated by the seasonal wind climate, showing an eastward flow in summer and a westward flow in winter.

In the nearfield modelling, the effects of tidal and seasonal wind-driven currents were excluded for conservatism of the assessment.

4.4.3 Ambient Salinity

Based on the monitoring data, an ambient salinity of 40 PSU (corresponding to about 1,027 kg/m^3 water density) was selected for modelling of near-field mixing. This is conservative as it is slightly higher than the median salinity. Due to the distinct difference in salinity between ambient seawater and the bitterns, small variations in background salinity levels will have a negligible impact on nearfield mixing.

4.5 Diffuser Design Approach

The bitterns outfall diffuser design has been developed through a combination of near-field and far-field modelling. The approach has been to maximise the initial dilution achieved by the diffuser the near-field in order to maximise the subsequent dispersion in the far-field. These concepts are discussed in more detail below.

4.5.1 Near-Field Approach

The near-field region is located in the immediate vicinity of the discharge location. It is characterised by high initial mixing at small spatial scales (metres) and short temporal scales (minutes). Mixing in this region is dominated by the initial properties of the discharge. These include the initial momentum and density difference of the discharge, its physical dimensions and its position in the water column.

To enhance the opportunities for subsequent mixing in the far-field, the aim of the diffuser was to provide sufficient initial dilution such that the density of the diluted bitterns was only about 1.0 kg/m^3 above the background density of the surrounding seawater.

Near-field modelling has been used in the present study to investigate a range of diffuser options, and to develop a preferred concept design to meet the 1.0 kg/m^3 density difference objective.

4.5.2 Far-Field Approach

The far-field region involves the subsequent mixing of the now diluted bitterns within the broader receiving waters. It occurs over much larger spatial scales (100s of metres to kilometres) and longer temporal scales (hours and days). Mixing in this region is dominated by larger-scale ambient conditions. These include the effects of tides and wind-driven currents.

For the present study, the key factors affecting the far-field dispersion are the tidal and wind-driven currents, and the local bathymetry.

4.5.2.1 Currents

As the currents in the area are aligned with the coastline, maximum dispersion of the diluted bitterns will be achieved by aligning the diffuser at right angles to the coast.

4.5.2.2 Bathymetry

The bathymetry in the vicinity of the diffuser and loading jetty is presented in Figure 4-3. This shows that the bathymetry slopes downwards from the coast to about the -5.0m contour (to MSL). Beyond the -5.0m contour the seabed becomes relatively flat and there appears to be a shallow depression extending northwards from the end of the jetty.

There is to be no dredged channel to provide barge access to and from the jetty. Instead, there is to be a deeper all-tide berthing pocket at the eastern end of the jetty. Transhipment barges will then make use of the tide to provide adequate under-keel clearance when sailing over the natural seabed to the north. As such, using the diffuser to direct the diluted bitterns to a deeper departure channel is not an option with this project.

As an alternative, the aim has been to achieve significant initial dilution with the diffuser such that the resulting diluted bitterns will be only marginally more saline, and hence marginally (of order 1.0 kg/m^3) denser than the surrounding receiving waters. As such, there will be tendency for the diluted bitterns to gradually drift offshore with the seabed contours as it mixes further with the tidal and wind-driven currents.

Far-field modelling has been used to evaluate the overall performance of the diffuser design and to develop a range of mixing zone contours.

4.6 Near Field Modelling

Nearfield dilution begins immediately on discharge from the outfall pipe. The principal mechanism associated with initial dilution is the mixing that occurs as the stream of effluent exiting the discharge port entrains surrounding seawater due to turbulence generated by its initial momentum and buoyancy effects. For a negatively buoyant effluent (such as bitterns, see Figure 4-4), an initially upwards discharge will create a rising negatively buoyant jet with an ascending trajectory in which the negative buoyancy opposes the initial vertical momentum. At a certain distance from the port, the vertical momentum reduces to zero and the jet will reach its maximum height. Beyond this point, additional dilution is generated by turbulence as the flow descends and spreads over the seafloor.

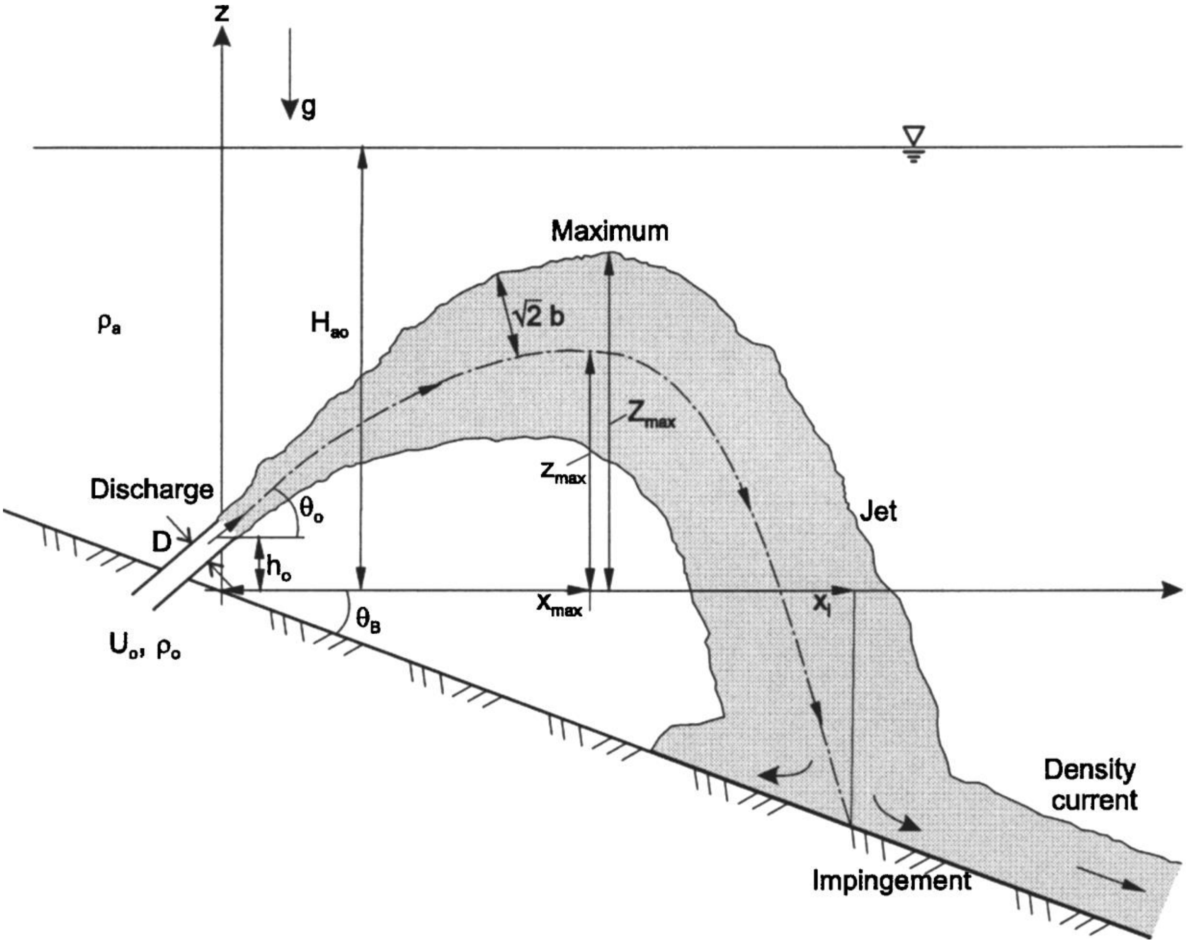


FIGURE 4-4 INDICATIVE PLOT OF BITTERNS JET

The initial dilution achieved by an outfall and associated diffuser is dependent on a range of parameters. These include:

- The main properties of the discharge: port size, and initial velocity, density and direction; and.
- The receiving water characteristics: water depth and velocity.

In this study, a preliminary nearfield mixing assessment was carried out using empirical relationships, while a more detailed assessment of preferred design concepts was then carried out using numerical modelling.

4.6.1 Design Parameters

The main design parameters used in the nearfield evaluation are presented in Table 4-5. In this study:

- The discharge rate in November was selected as the design scenario for analysis of near field mixing. It represents the month when maximum flow of bittern will be discharged into the receiving environment.
- To maximise the mixing, the discharge was assumed to be from a diffuser located along the outer 400 m of the 700 m long salt export jetty.
- A mean water depth of 5.0 m (MSL) was assumed at the diffuser in accordance with the bathymetry survey outcomes. The model did not include the deepening of the berth pocket, which is conservative with respect to mapping the extent of the near field mixing zone because less water column will be available for mixing.

Also, the berth pocket covers only half the length of the diffuser thereby does not represent the natural seabed condition along the diffuser.

TABLE 4-5 NEAR FIELD MODEL INPUTS

Bitterns Effluent (Base Cases)		
Variable	**No Predilution**	**Predilution**
Seawater Temperature (ºC)	24	24
Seawater Salinity (PSU)	40	40
Seawater Ambient Water Density (kg/m^3)	1027	1027
Bitterns Flow Rate (m^3/s)	0.444	0.444
Washwater Flow Rate (m^3/s)	0.093	0.093
Seawater Dilution Flow Rate (m^3/s)	0.0	0.444
Overall Discharge Flow Rate (m^3/s)	0.537	0.981
Bitterns Discharge Salinity (PSU)	287.1	174.5
Bitterns Discharge Density (kg/m^3)	1235	1136
Water Depth (m)	5	
Length of diffuser (m)	400	

4.6.2 Assimilative Capacity

The shallow water depth and weak tidal flow are the primary constraints that limit the diffuser layout options. A brief analysis suggests that for a 400 m long diffuser discharging into 5 m deep water with an ambient current of about 0.05 m/s, there will be an ambient flow rate of around 100 m^3/s across the line of the diffuser. This will provide maximum dilutions of around:

- 185:1 for a discharge of 0.54 m^3/s for bitterns without pre-dilution; and
- 100:1 for a discharge of 0.98 m^3/s for the pre-diluted bitterns.

For the discharge concentrations given in Table 4-3, these initial dilutions will result in minimum bitterns concentrations of around 1.6 to 1.7 PSU above background levels at the diffuser. Although some temporary accumulation of higher salinity levels could be expected during periods of slack water, it is likely that, with appropriate diffuser design, the mixing zone criteria of 1.6 and 2.2 PSU above background levels will be met in close proximity to the diffuser.

The main purpose of the nearfield investigations is therefore to develop a diffuser concept that would provide the maximum initial mixing throughout the water depth. This would then provide the optimal basis for the subsequent far-field mixing (as discussed in Section 4.7).

4.6.3 Preliminary Assessment

To maximise nearfield mixing, it will be necessary for the bitterns discharge to mix as much as possible throughout the full depth of water available along the line of diffuser. Two initial concepts were considered, as described below. These were:

- Horizontal discharge from just below the water surface; and
- Upwards discharge from the seabed.

The results of these investigations led to the development of a third "hybrid" concept:

- Upwards discharge from part-way up the water column.

Each of these options is discussed below.

4.6.3.1 Horizontal surface discharge

Horizontal discharge from just below the sea surface would provide mixing from the surface layer through to the seabed. To ensure the diffuser ports are underwater at all tidal conditions, the diffuser ports would need to be fixed just below Lowest Astronomical Tide (LAT). This would provide only around 3.5 m to 4 m of water depth for nearfield mixing.

Preliminary surface discharge calculations were carried out using standard empirical nearfield relationships (e.g., Cederwall, 1968). The results showed that to achieve the level of dilution discussed above it would require a very high number of impractically small diameter discharge ports. It was found that, with a horizontal discharge velocity of around 4.0 m/s, it would be necessary to have in excess of 1,000 discharge ports with a diameter of around 13 mm to achieve an initial dilution of around 185:1 for the undiluted bitterns. For the diluted bitterns, over 800 discharge ports with a diameter of around 20 mm would be required to achieve an initial dilution of around 100:1.

4.6.3.2 Upwards discharge from the seabed.

Many desalination outfalls dispose of their hyper-saline wastewater via diffusers mounted on or just above the seabed. This approach allows for dilution of the negatively buoyant brine during both the ascending phase (as it travels upwards toward the sea surface due to its initial momentum) and the descending phase (as it travels downwards due to negative buoyancy). Preliminary seabed discharge calculations were carried out using empirical nearfield relationships developed by Roberts et al. (1987, 1997). For these calculations, it was assumed that discharge ports were inclined at a 60º angle upwards from the seabed. This angle has been found to provide optimal mixing for negatively buoyant jets (e.g., Zeitoun et al, 1970). Calculations were carried out for a range of port numbers and diameters.

Discharge velocities used were 6.0 m/s (that used in the Sydney desalination plant outfall) and 8.0 m/s (that used by the Adelaide desalination plant outfall). The results showed that the high initial density of the bitterns resulted in a significantly lower height of rise of the jet relative to that which would be expected from a desalination plant outfall having the same configuration. For the undiluted bitterns, mixing throughout the depth could only be achieved with a relatively small number of large diameter ports: 4 by 170 mm diameter ports for a 6 m/s discharge velocity; and 8 by 100 mm diameter ports for an 8 m/s discharge velocity. The resulting nearfield dilutions when the plume had returned to the seabed were less than 20:1 for the 6 m/s discharge and less than 30:1 for the 8 m/s discharge. These were well below the required dilutions discussed above.

For the diluted bitterns, mixing throughout the depth could be achieved by a somewhat larger number of smaller ports: 20 by 100 mm diameter ports for a 6 m/s discharge velocity; and 40 by 100 mm diameter ports for an 8 m/s discharge velocity. The resulting nearfield dilutions of around 30:1 for the 6 m/s discharge and 50:1 for the 8 m/s discharge would provide a significant improvement but were still well below the levels required.

The initial dilutions that could be achieved through upwards discharge from the seabed could be increased by using a larger number of smaller diameter ports. As the port diameter was reduced, however, the initial height of rise of the jet was also reduced. This meant that mixing became limited to the lower part of the water column and the overall initial dilution at the seabed was less than for a surface discharge with an equivalent number of ports.

4.6.3.3 Hybrid solution

To maximise the initial dilution, it was then proposed to investigate the use of a 60º upwards discharge from part-way up the water column. With this approach, the level of the discharge would be located above seabed so that the discharge jet would rise to just below the sea surface. This would effectively combine the mixing effects of the two concepts considered above. It would provide initial mixing on the upward path, followed by further mixing throughout the water depth as the resulting diluted near-surface jet then descended to the seabed.

4.6.4 Summary of Considerations

The concept design of the diffuser depends largely on the effluent properties, discharge location (e.g., water depth) and ambient receiving water conditions. As a result of the preliminary assessment, it was concluded that the preferred system should include the following:

- Pre-dilution to maximise the efficiency of the nearfield dilution, as it requires only half the near-field dilution to meet the same criteria when compared to the non-predilution case.
- A "hybrid" solution with an upwards discharge from part-way up the water column as the most effective way of maximising the nearfield dilution and ensuring that mixing occurs throughout the full water depth.
- Relatively high discharge velocities and small port diameters to achieve the level of nearfield mixing required (recognising that higher discharge velocities will increase the head losses, thereby requiring more pumping power).

Additionally, it was found that even with a relatively large number of small diameter ports, the 400 m length of the diffuser and shallow depth meant that there would be no direct interaction between jets from adjacent discharge ports. This means that the mixing characteristics of the discharge ports can be analysed individually.

4.6.5 Nearfield Modelling System

Commercial models have been widely used as prediction tools in assessment of diffuser performance. These models were initially developed for positively buoyant discharges and were later adapted to also consider negatively buoyancy effluents (e.g., brine/bitterns discharges). One of the most notable commercial software nearfield modelling packages is CORMIX (Cornell Mixing Zone Expert System). This comprises a jet integral model based on the conservation of the mass and momentum. It uses a flow classification system based on hydrodynamic criteria that refer to length scale analysis and empirical relationships obtained from laboratory and field experiments.

CORMIX can simulate discharge of effluents with positive, negative and neutral buoyancy under different discharge and ambient conditions. There are, however, several limitations to the operation of the system. It is a steady-state model and, as such, each model run can only simulate a single discharge scenario. Further, Palomar (2012) found that:

- There can be an unrealistically sharp transition between the near-field solution and subsequent mid-field mixing provided by the spreading layer approximation; and
- If the model detects the jet impacting the surface, flow is homogenised throughout the water column, and the model applies semi-empirical formulas resulting in unrealistically higher dilutions than for the case where the jet does not impact the surface.

To avoid any potential issues relating to the overall operation of CORMIX, the modelling was focussed on the use of CORJET which is the integral jet model component of CORMIX that analyses the performance of discharges from individual diffuser ports. This was considered to be appropriate as the preliminary assessment found that there would be no direct "interference" between the discharges from each of the ports within the diffuser.

4.6.6 Near Field Modelling Results

A wide range of model tests were carried out to investigate the effects of changes in the depth of the diffuser, the port diameter, and the port discharge velocity. Model results were interrogated to provide predictions of dilution and excess salinity concentrations after the completion of near-field mixing (i.e., the point at which the resulting plume reached the seabed).

The results from some selected sensitivity tests are presented Table 4-6, and in Figure 4-5 and Figure 4-6. Only the results of tests involving diluted bitterns have been considered. Figure 4-5 shows initial dilution as a function of port diameter, discharge velocity and discharge depth. Figure 4-6 shows the corresponding maximum heights of rise of the different jets. With respect to the last point, it is noted that the height of rise was calculated using the Roberts equation, as this resulted in a slightly higher (conservative) value than that calculated by CORJET. The findings are summarised as follows:

- For all test cases, it was found that nearfield mixing was completed within 10 m of the diffuser;
- Initial dilution increased with decrease in port diameter. There is, however, a lower limit on port size that can be practically used;
- Initial dilution increased with discharge velocity. The model results showed that increasing the jet velocity from 6 m/s to 8 m/s only resulted in increases in initial dilution in the order of 5% to 10%;
- The maximum height of rise increased with jet velocity. Increasing the jet velocity from 6 m/s to 8 m/s resulted in increases in the height of rise of 30% or more; and
- Increasing the port height results in a corresponding increase in the height of rise of the jet.

With respect to the last two points, it is noted that the height of rise of the jet is limited to the position of the sea surface. This in turn is governed by the tide level above the diffuser opening and mixing will be reduced for jets that reach the sea surface at low water.

TABLE 4-6 MODEL RESULTS

Model Scenarios	Port Size (mm)	Jet speed (m/s)	Froude Num	Port Height (m MSL)	Dilution	Salinity Exceedance (PSU)	Maximum Height Of Jet (m MSL)
					CORJET	CORJET	Roberts
Case 1	20	6	41.5	-3	107.7	1.2	-1.3
Case 2	25	6	37.2	-3	82.0	1.6	-1.1
Case 3	30	6	33.9	-3	68.6	2.0	-0.9
Case 4	35	6	31.4	-3	60.5	2.2	-0.7
Case 5	40	6	29.4	-3	52.3	2.6	-0.6
Case 6	50	6	26.3	-3	40.7	3.3	-0.3
Case 7	20	6	41.5	-2.5	132.5	1.1	-0.8
Case 8	25	6	37.2	-2.5	97.3	1.4	-0.6
Case 9	30	6	33.9	-2.5	80.7	1.8	-0.3
Case 10	20	6	41.5	-2	157.3	0.9	-0.3
Case 11	25	6	37.2	-2	113.6	1.2	-0.1
Case 12	30	6	33.9	-2	92.2	1.5	0.1
Case 13	35	6	31.4	-2	77.4	1.7	0.3
Case 14	40	6	29.4	-2	68.5	2.0	0.4
Case 15	50	6	26.3	-2	55.9	2.4	0.7
Case 16	20	8	55.4	-3	117.8	1.1	-0.7
Case 17	25	8	49.5	-3	95.3	1.4	-0.5
Case 18	30	8	45.2	-3	78.2	1.7	-0.2
Case 19	35	8	41.9	-3	66.3	2.0	0.0
Case 20	40	8	39.2	-3	59.1	2.3	0.2

Model Scenarios	Port Size (mm)	Jet speed (m/s)	Froude Num	Port Height (m MSL)	Dilution	Salinity Exceedance (PSU)	Maximum Height Of Jet (m MSL)
					CORJET	CORJET	Roberts
Case 21	50	8	35.0	-3	48.9	2.8	0.6
Case 22	20	8	55.4	-2.5	140.1	1.0	-0.2
Case 23	25	8	49.5	-2.5	109.8	1.3	0.0
Case 24	30	8	45.2	-2.5	89.0	1.5	0.3
Case 25	20	8	55.4	-2	162.4	0.8	0.3
Case 26	25	8	49.5	-2	124.4	1.1	0.5
Case 27	30	8	45.2	-2	99.9	1.3	0.8
Case 28	35	8	41.9	-2	87.0	1.5	1.0
Case 29	40	8	39.2	-2	74.0	1.8	1.2
Case 30	50	8	35.0	-2	58.2	2.3	1.6

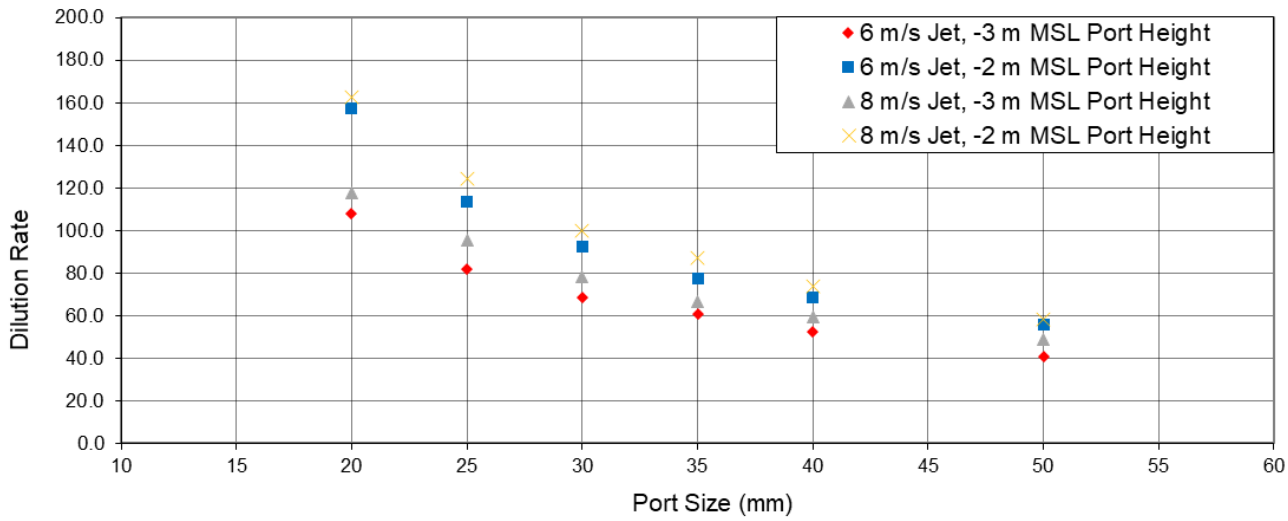


FIGURE 4-5 DILUTION RATE ESTIMATED BY DIFFERENT CONCEPT OPTIONS

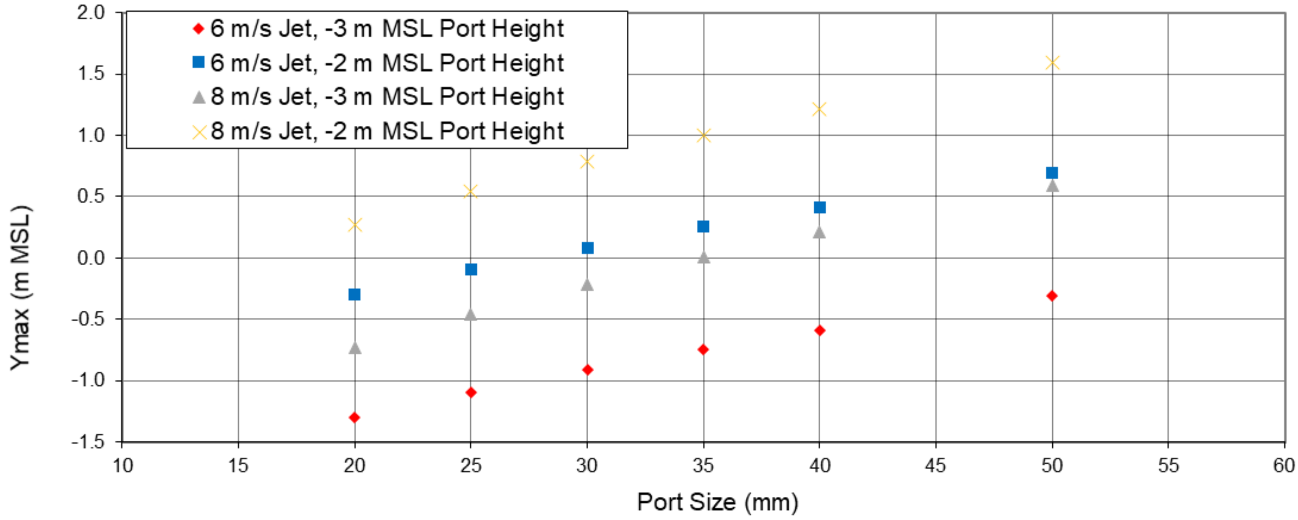


FIGURE 4-6 MAXIMUM JET HEIGHT PREDICTED BY VARIOUS CONCEPT OPTIONS

4.6.7 Diffuser Design Concept

The diffuser concept design was developed based on the following considerations:

- The target nearfield dilution would be about 100:1 relative to the pre-diluted bitterns effluent, which is almost equivalent to the assimilative capacity (absorption capability) of tidal flows across the line of the diffuser.
- Pre-dilution would be required to maximise the overall initial dilution.
- A discharge velocity of 6m/s would be used.
- The maximum height of rise should be around -0.5m MSL. This may result in the jet breaking the surface for short periods during low water but would provide improved nearfield mixing at other tidal phases.

Of the options presented in Table 4-4, Case 8 was considered to be the most promising for further development in the concept design, as it:

- Provides maximum mixing on the way up (to approximately mean low water) and then on the way down again.
- Uses a high (6 m/s) but not impractically high discharge velocity.
- Provides the minimum number of largest possible diameter ports for practicality.
- Meets the mixing zone dilution requirements in the nearfield (noting that some local and temporary accumulation may occur during periods of slack water).

This results in a diffuser concept design with 333 in total 25 mm diameter ports located at about 1.2 m spacings along a 400 m long diffuser. The current 25 mm diameter ports are already very small. Further reduction of the diameter of the ports may not be practical, given the potential adverse impact from marine growth, sedimentation, and requirement of pumping pressure. The diffuser is to be located along the north-east side of the outer 400 m of the export jetty with the ports discharging pre-diluted bitterns at 60º upwards from horizontal from 2.5 m below MSL (approximately mid-depth in the water column).

4.6.8 Diffuser Layout Optimisation

The diffuser layout was optimised during the concept design phase to increase the bitterns dilution. In particular, during the modelling process it was determined that increased bitterns dilution could be achieved by placing the diffuser in the deepest water available locally (to increase mixing within the deeper water column) as well as orienting it perpendicular to prevailing tidal currents (to increase mixing by tidal water flows).

As a result, the following layout changes were made to the diffuser and the jetty to which it will be fixed:

- Moving the jetty and diffuser 200 m to the north-west into water which was approximately 0.2 m deeper than the initial location.
- Rotating the jetty and diffuser by approximately 10 degrees to be perpendicular to tidal currents.

FIGURE 4-7 JETTY AND DIFFUSER LAYOUT OPTIMIZATION

4.7 Bitterns Discharge Far-Field Modelling

Following the identification of a diffuser concept design to maximise the near-field dilution of the bitterns, far-field modelling was then undertaken. The modelled near-field dilution for the diffuser concept design is approximately 100:1 corresponding to about a 1.4 Practical Salinity Units (PSU) increase above background salinity. Near-field modelling however does not simulate the further far-field dilution of the bitterns into the surrounding marine environment by ocean currents. This requires additional investigation through far-field modelling of the bitterns plume advection/diffusion at larger scales (100’s of meters to a few kilometres).

4.7.1 Far-field Modelling System

Far-field mixing was investigated through the DHI MIKE 3 FM modelling software. MIKE 3 FM HD is based on numerical representation of the three-dimensional incompressible Reynolds averaged Navier-Stokes equations, invoking the assumptions of Boussinesq and hydrostatic pressure. The model consists of continuity, momentum, temperature, salinity and density equations and is closed by k-epsilon turbulent closure scheme.

The spatial discretisation of the equations is performed using a cell-centred finite volume approach. In the horizontal plane an unstructured grid is used, while in the vertical domain a structured mesh is applied.

MIKE 3 FM HD was coupled with the Advection-Dispersion module to simulate the diffuser configuration. The model calculates the transport (advection) of materials based on the flow conditions found in the hydrodynamic calculations while dispersion is calculated in the horizontal and vertical planes within the hydrodynamic model and scaled according to model calibration. The framework for the far-field water quality modelling is conceptualised in Figure 4-8.

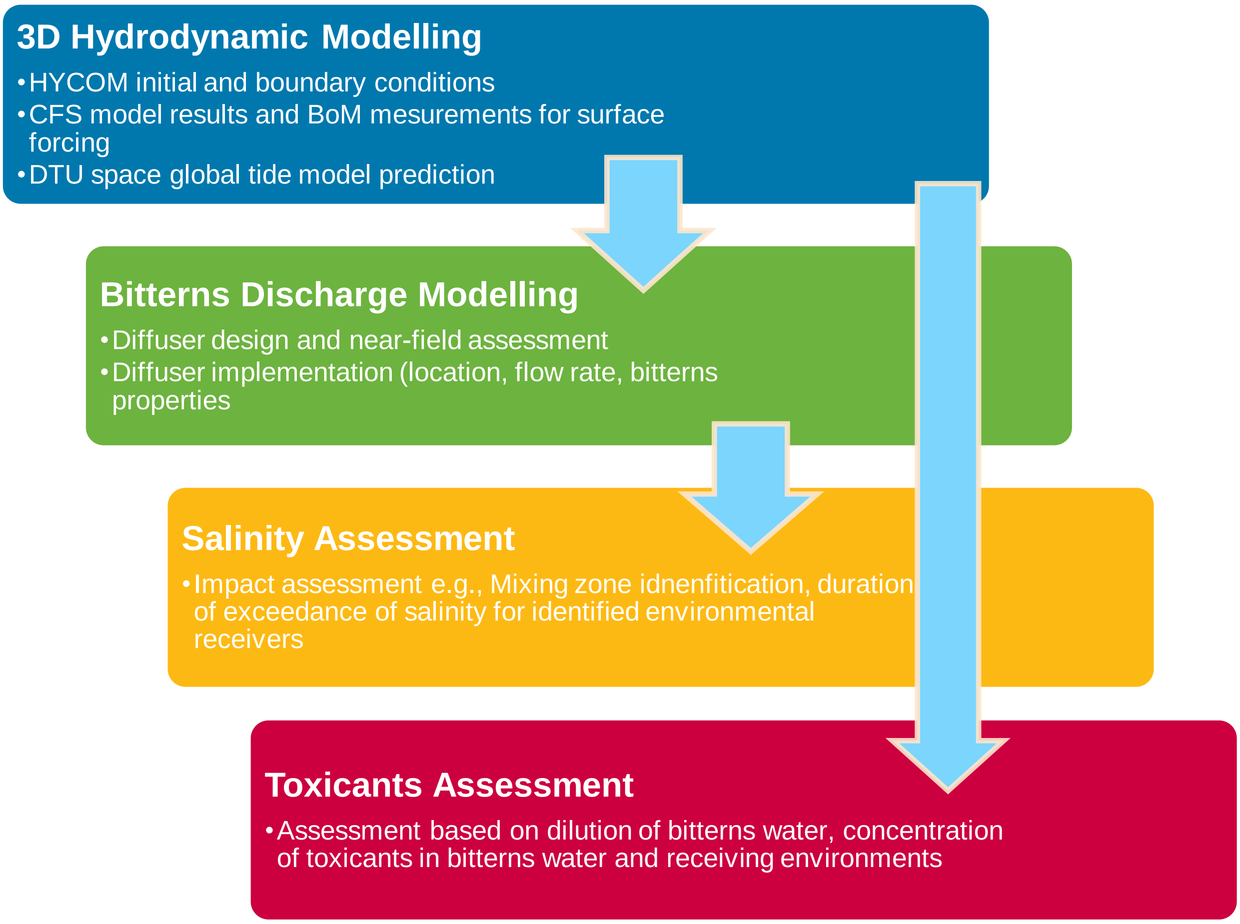


FIGURE 4-8 WATER QUALITY MODELLING FRAMEWORK

4.7.2 MIKE 3 FM Model Settings

4.7.2.1 Modelling Mesh

The model is built on an unstructured flexible mesh (Figure 2-1) and uses a finite volume solution technique. The northern model boundary is located offshore near the 250 m contour, whereas the western boundary starts from the northwest of Exmouth. The model mesh as presented in Section 2.1.1 was utilised.

The computational triangular mesh of the model is made with sufficiently small cells to resolve the detailed conditions in the study area, especially around the Ashburton Salt Project site. The model mesh (Figure 4-9) was further refined in the vicinity of the diffuser to facilitate the introduction of the bitterns discharge, and to minimise the impact of numerical mixing.

Vertically the model is resolved by a hybrid of sigma (terrain following) coordinates focussed near the surface to provide a good representation of the shallow terrain as well as z-coordinates underneath to allow more accurate approximation of baroclinic pressure across model layers.

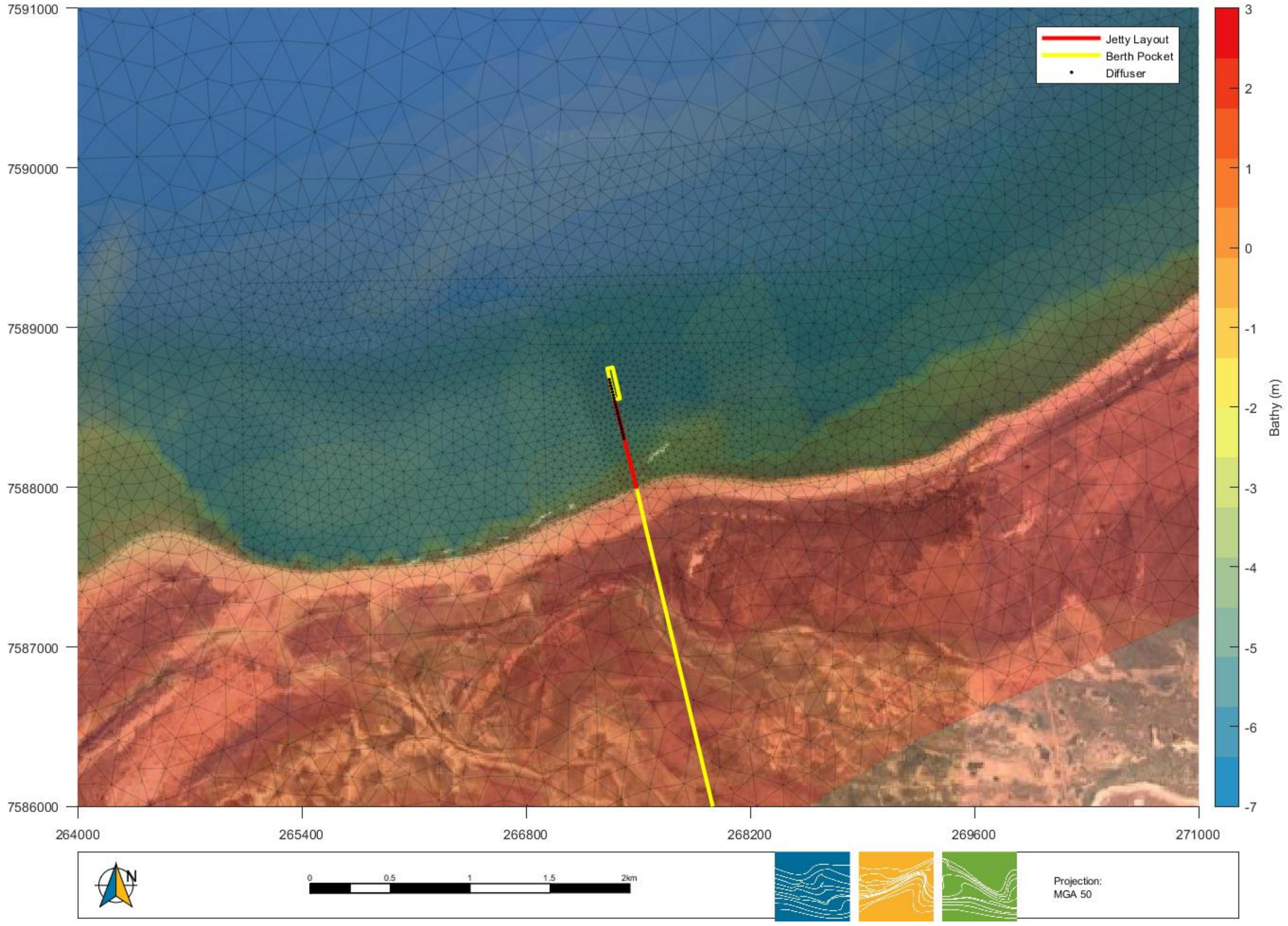


FIGURE 4-9 JETTY AND DIFFUSER LAYOUTS

4.7.2.2 Seasonal Forcing

Seasonal dynamic forcing is incorporated through inclusion of seasonal water level/temperature/salinity variations from Hybrid Coordinate Ocean Model (HYCOM), seasonal wind and air pressure from Climate Forecast System Analysis CFSv2, and seasonal surface heating, rainfall and evaporation from Onslow Airport Bureau of Meteorology (BoM) weather station. The model is configured to retrospectively simulate the full hydrodynamic processes over the year 2015 so that seasonal impacts that would affect the far-field mixing can be represented by the model.

4.7.2.3 Eddy Viscosity/diffusivity

Sub-grid scale horizontal eddy viscosity was parameterised as a function of the local grid resolution using a Smagorinsky formulation. Vertical eddy viscosity (which characterises the three dimensionality of flow structures under impacts of vertical stratification) was simulated by k-ε formulation which is a two-equation turbulent closure scheme tested in numerous 3D model applications.

A scaling factor was applied (based on Prandtl Number) to approximate eddy diffusivity as a function of eddy viscosity which is a reasonable assumption applicable for a wide range of flow conditions.

4.7.2.4 Precipitation and Evaporation

Rainfall data was sourced from the BoM weather station at Onslow Airport.

Evaporation was estimated internally through MIKE heat exchange module. Evaporation data sourced from BoM Learmonth Airport weather station was used to validate the adopted model settings.

The available measured data from nearby BoM weather stations is described further in *Marine, Coastal and Surface Water Existing Environment* (Water Technology 2021).

4.7.2.5 Heat Exchange Parameters

Air-sea heat fluxes make a significant contribution to ocean heat budgets (temperature) and loss of water from surface evaporation. For the relatively shallow water in the Exmouth Gulf and the surrounding coastline, their effects can be more profound when compared to those in deeper open ocean areas.

Short wave radiation, or solar radiation, measured at Onslow Airport (in watts/m^2) was applied to the model (see Figure 4-10). Short wave radiation is the most critical thermal energy input for the ocean heat budget, and measured data shows strong seasonal and diurnal patterns.

Air temperature and relative humidity data were used to calculate the latent heat flux which affect the water temperature and evaporation. For a given air temperature, an increase in humidity may contribute to lower evaporation. The data from Onslow Airport were obtained from BoM and included within the heat exchange module.

Due to the lack of cloud cover data, a constant cloud cover of 80% clear skies was applied over the local model domain.

4.7.2.6 Vertical Approximation

The water column was resolved by a hybrid mesh including four sigma layers near surface and nine z-layers towards the seabed. This configuration provided at least four vertical layers with layer thicknesses of 25% of the water depth throughout the shallow nearshore areas.

4.7.2.7 Bitterns Discharge

The bitterns discharge is conceptualised as source points along the diffuser with parameters specified in accordance with near-field model design. Unlike the near-field model which evaluated the “worst case” scenario, the far-field model considered the impact from the varying discharge rate on a monthly basis.

4.7.3 Monthly Bitterns Discharge

The discharge scheme has been reviewed extensively during the concept design stage. Without predilution the design of diffuser the would be very challenging (requiring extra small port openings and higher pumping pressure). As such the prediluted scenario was used in the far-field model simulations as this was considered to be the most appropriate due to the practicalities of the diffuser design. The monthly variations in bitterns discharge properties are presented in Table 4-7.

For the bitterns far-field assessment the dilution rate was calculated as a ratio against prediluted bitterns with a salinity of 174.5 PSU.

TABLE 4-7 MONTHLY BITTERNS EFFLUENT PROPERTIES FOR FAR FIELD MODELLING

Month	Temperature (°C)	Bitterns Discharge (m^3/s)	Salinity (PSU)	Density (kg/m^3)
January	27	0.88	174.5	1135
February	27	0.33	174.5	1135
March	28	0.71	174.5	1135
April	27	0.4	174.5	1135
May	25	0.23	174.5	1135
June	23	0.14	174.5	1135
July	22	0.29	174.5	1135
August	21	0.41	174.5	1135
September	22	0.65	174.5	1135
October	23	0.89	174.5	1135
November	24	0.98	174.5	1135
December	25	0.94	174.5	1135

4.7.4 Model Scenarios

Figure 4-10 presents an example of observed key meteorological conditions at Onslow Airport. The representative year, 2015, was selected to investigate the cumulative impacts of the bitterns discharge in a dynamic receiving environment as well as to assess potential seasonal discharge variations. The year 2015 was selected due to its average meteorological climate. Throughout the year the weather conditions are relatively calm which is suitable to evaluate the cumulative impacts from bitterns discharge in a conservative manner.

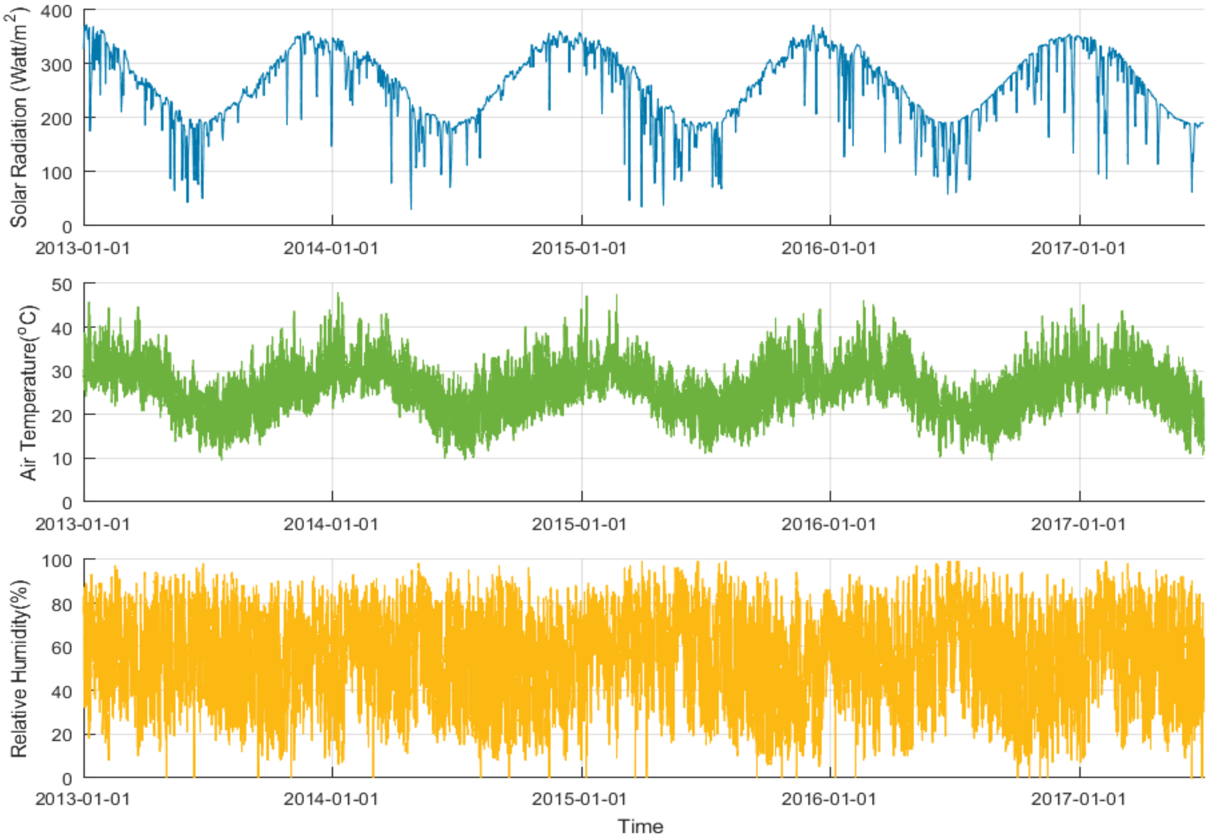


FIGURE 4-10 MEASURED SOLAR RADIATION, AIR TEMPERATURE AND RELATIVE HUMIDITY AT ONSLOW AIRPORT

4.7.5 Salinity Results

4.7.5.1 Modelled Impacts to Salinity

Model results show ambient pre-development salinity may increase from about 38 PSU off the coast to over 42 PSU nearshore, which is within the range of natural salinity that has been measured in the locality. The higher salinity towards the shore is driven by high surface evaporation and salt concentration which has greater effects in shallower waters.

The residual impact to water quality was investigated through plots of modelled differences between existing and post-development predictions. Model results have been interrogated to illustrate the extents of residual impact for different durations of exceedances. Potential impacts relative to background salinity are presented in Table 4-8 and Table 4-9.

TABLE 4-8 SUMMARY OF MODELLED IMPACTS TO SALINITY

Percentile	Percent of the time	Increase in Salinity Around the Diffuser (after initial nearfield <10m mixing)	Increase in Salinity Further from the Diffuser
50th	50%	>0.5 PSU and <1 PSU within 200 m from the diffuser	<0.5 PSU not shown
80th	20%	>1.5 PSU and <2 PSU 200 m - 50 m from the diffuser > 2 PSU and < 2.5 PSU within 50 m from the diffuser	>1 PSU up to 500 m from the diffuser
90th	10%	>1.5 PSU and <2 PSU 400 m - 100 m from the diffuser > 2 PSU and < 3 PSU within 100 m from the diffuser	>1 PSU up to 1.5 km from the diffuser
99th	1%	> 2 PSU and < 3 PSU 500 m -100m from the diffuser > 3 PSU and < 4.5 PSU within 100 m from the diffuser	>1 PSU up to 2 km from the diffuser >1.5 PSU up to 1 km from the diffuser

The observed impacts are summarised as follows:

- There is a significant salinity difference between surface and bottom water. Model results show spreading of high salinity bitterns water towards the seabed. Under a worse case condition (99th percentile or 1% of the time), the modelled salinity near the seabed could be about 4.5 PSU higher than ambient salinity in the immediate vicinity of the diffuser. The corresponding salinity at the surface would be less than 3 PSU above background. This is expected given the dense nature of the diluted bitterns which will cause it to settle due to negative buoyancy relative to the surrounding ambient water.
- Predicted impacts at the seabed (lowest model layer) for the 50th, 80th, 90th and 99th percentile salinity exceedance plots over 12 months are as follows (also see Table 4-8):
  - For 50% of the time (50th percentile), the model shows salinity increases of greater than 0.5 PSU while less than 1 PSU 200 m from the diffuser. Further away from the diffuser salinity increases were less than 0.5 PSU.
  - For 20% of the time (80th percentile), the predicted salinity exceeds the background salinity by 1 PSU over an area up to 500 m from the diffuser. Slightly higher impacts (greater than 1.5 PSU but less than 2.5 PSU) were predicted within 200 m of the diffuser.
  - For 10% of the time (90th percentile), the predicted salinity exceeds the background salinity by 1 PSU over an area with a diameter up to 1 km from the diffuser. Slightly higher impacts (greater than 1.5 PSU but less than 3 PSU) were predicted within 400 m of the diffuser.

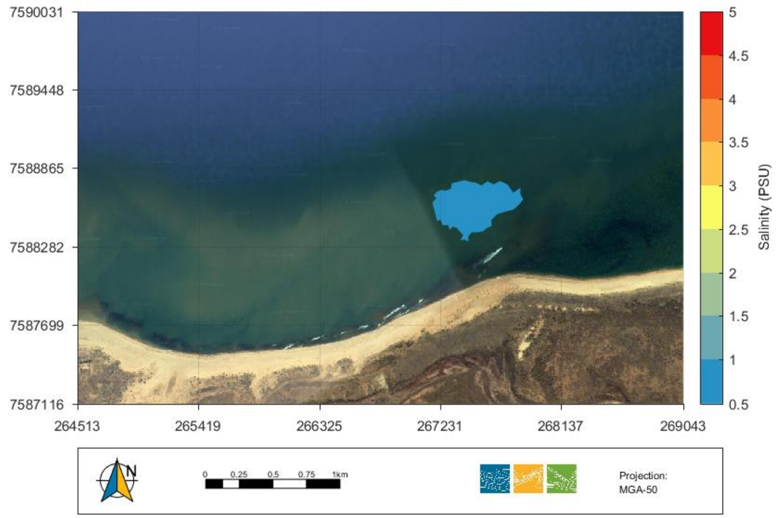
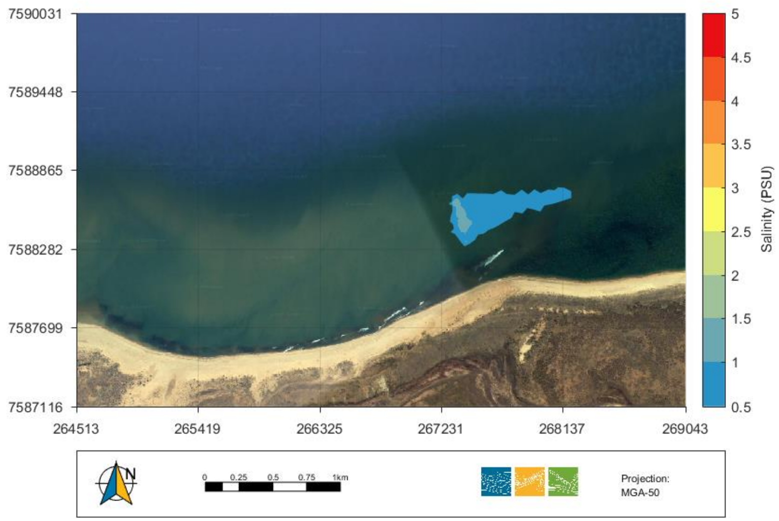
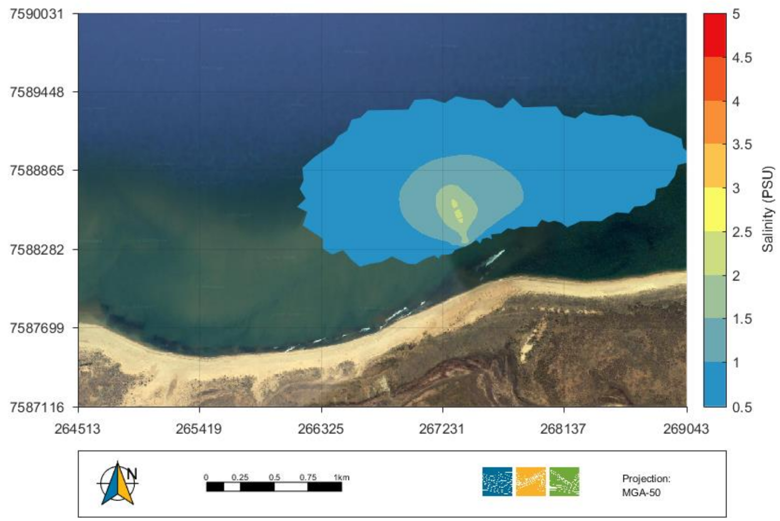
- For 1% of the time (99th percentile), the predicted salinity exceeds the background salinity by 3 to 4.5 PSU over a narrow (~50 m in width) zone around the diffuser. 1 PSU and 1.5 PSU salinity increases were predicted 2 km and 1 km from the diffuser respectively.

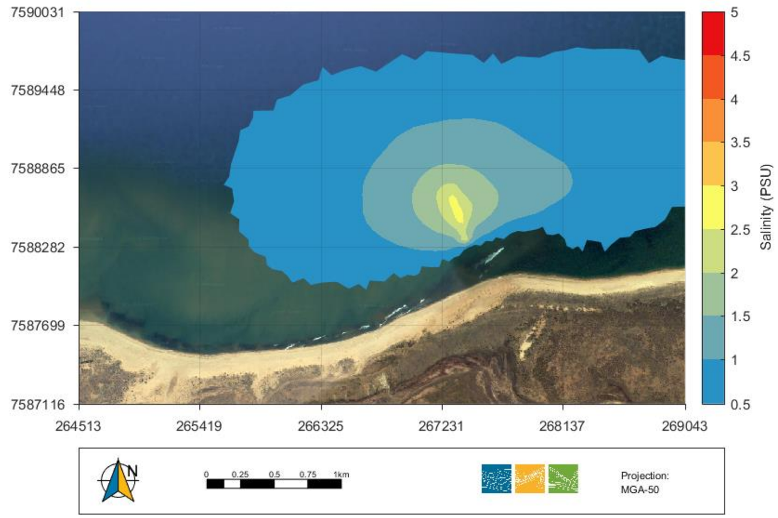
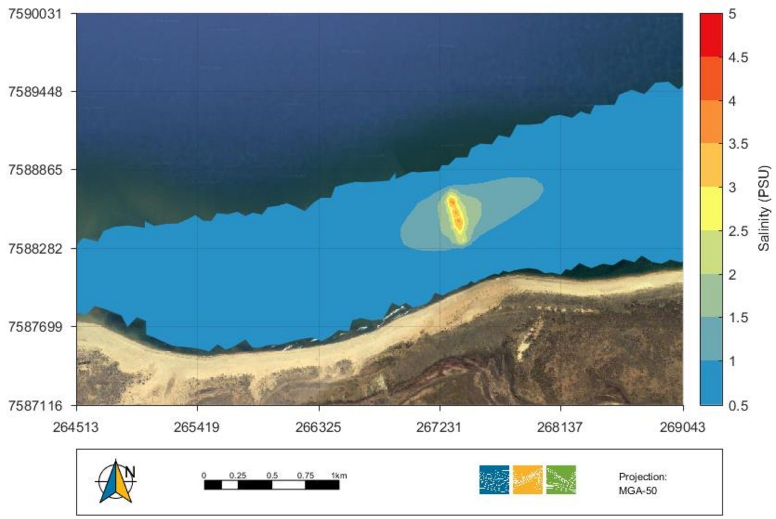
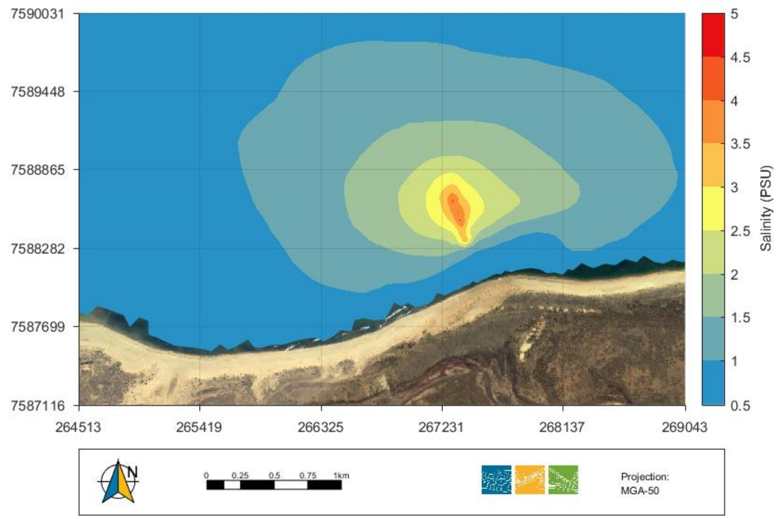
- For 99% of the time the predicted salinity increases at the seabed are reasonably low at 1 to 2 km distance from the diffuser, with salinity increases being in the order of 1 to 1.5 PSU above background. This level of salinity increase is within the range of natural variation measured at the locality as described in *Marine, Coastal and Surface Water Data Collection* (Water Technology, 2021).
- Whilst higher salinity increases are observed closer to the diffuser itself, these are only observed for smaller percentages of time. For example, up to 2.5 PSU within 50 m of the diffuser for 20% of the time, up to 2.5 PSU within 100 m of the diffuser for 10% of the time and up to 4.5 PSU within 50 m of the diffuser for 1% of the time. These higher salinity increases closer to the diffuser for smaller time percentages are expected for a dense discharge of this nature and represent the best dilutions that can be achieved with the optimised concept design of diffuser for the given receiving environment.
- Model results in Table 4-9 also show significant seasonal variation in predicted impacts which can be summarised as follows:
  - Maximum impact was observed in November which corresponds with the maximum bitterns discharge rate (due to highest evaporation rates). For 20% of the time (80th percentile), the predicted salinity exceeds the background salinity by 1.5 PSU over an area up to 400 m from the diffuser.
  - More moderate impacts were observed in months with lower bitterns discharge rates, such as 0.41 m^3/s in August when the predicted area of impact reduces to about half the size of the predicted impact area in November.
  - Minor impact was observed in months with the minimum bitterns discharge rate, such as 0.14 m^3/s in June. Salinity impact in June were insignificant even in the close vicinity of the diffuser.
  - Secondary influences from seasonal residual currents were predicted. In summer there was an eastward trend driving the bitterns plume direct to the east, whilst in winter the water became more stagnant with the current slightly reversed re-shaping the bitterns plume westward. This is driven by the prevailing wind climate of the region.

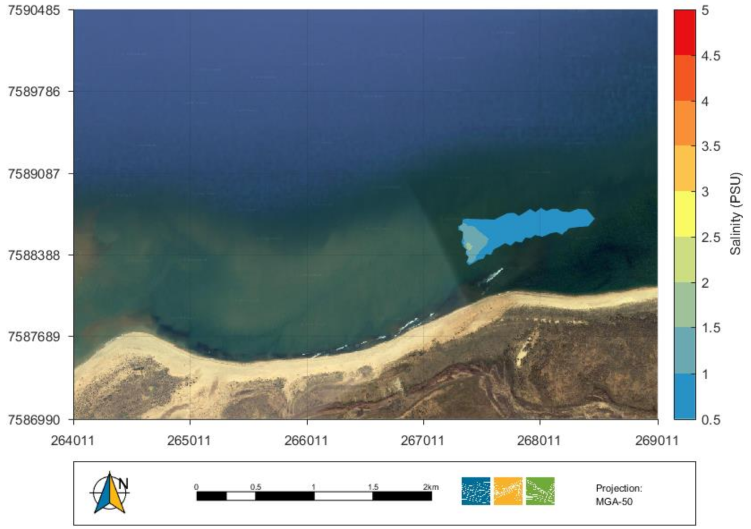
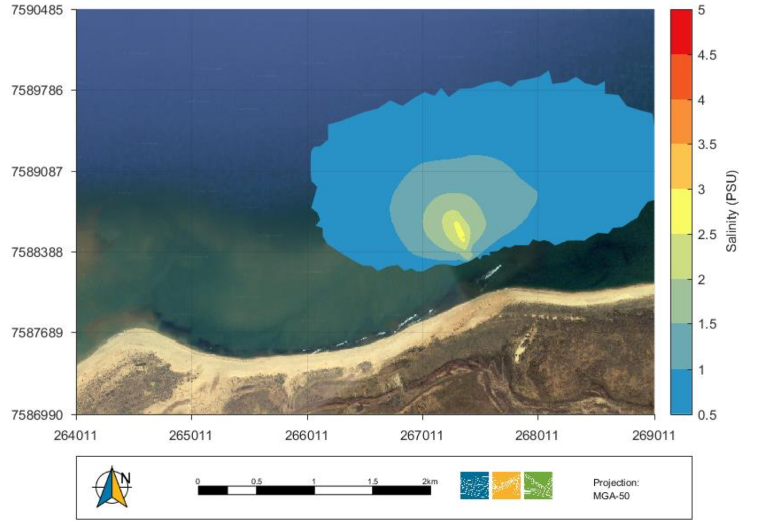
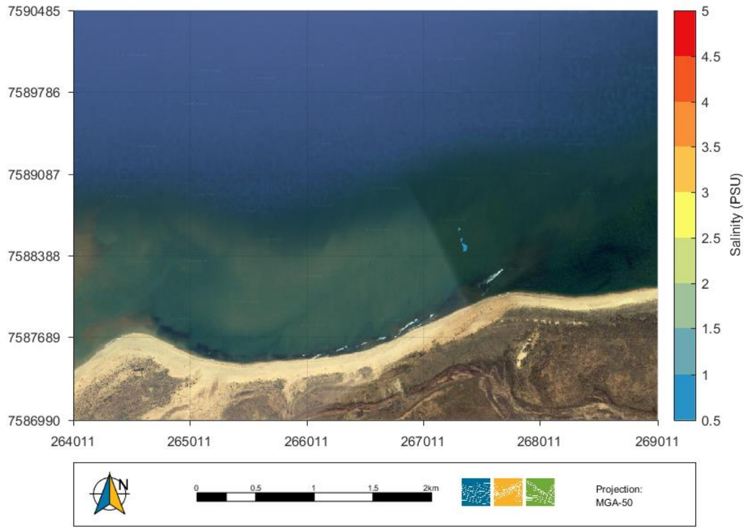
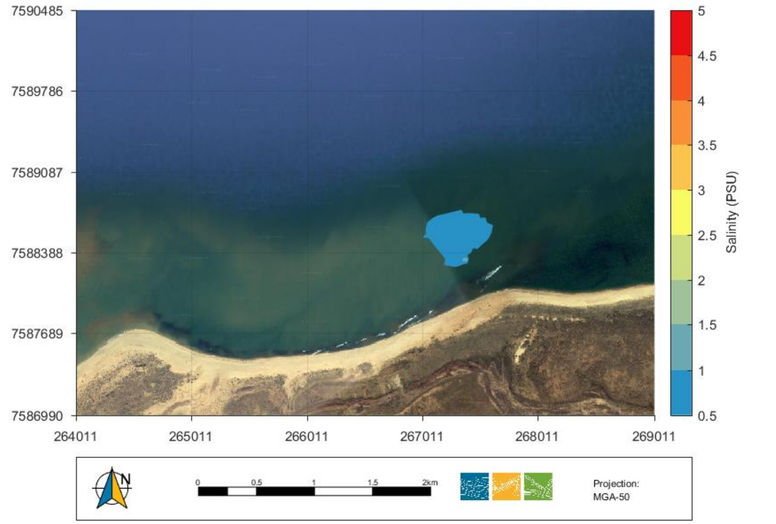
In summary:

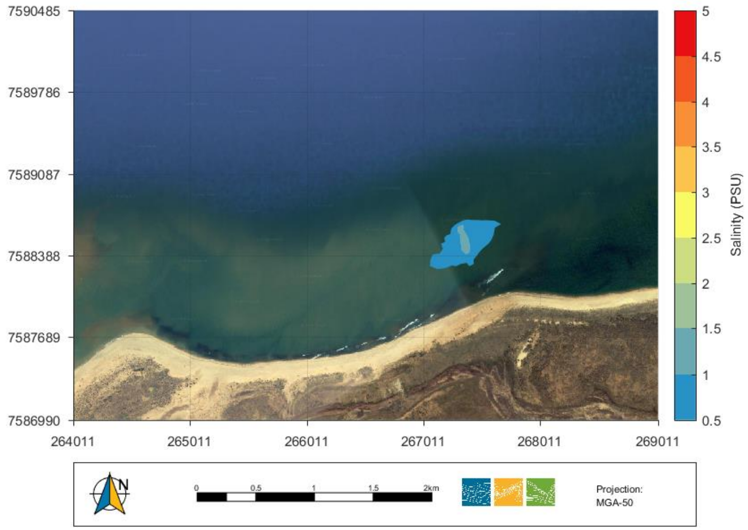
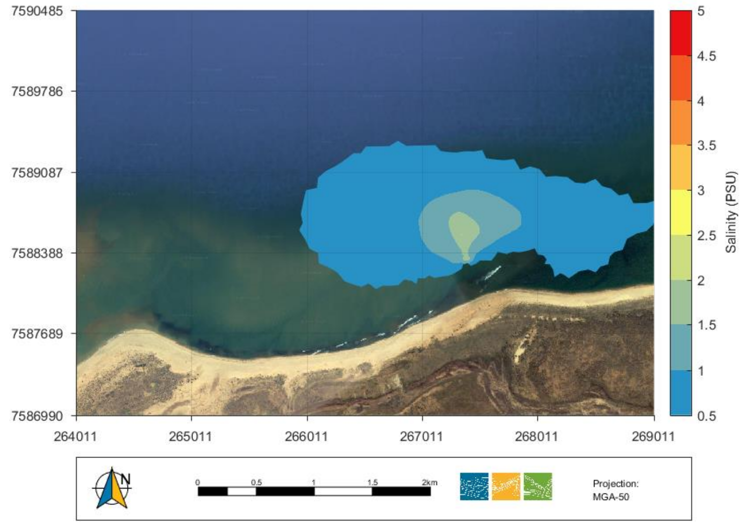
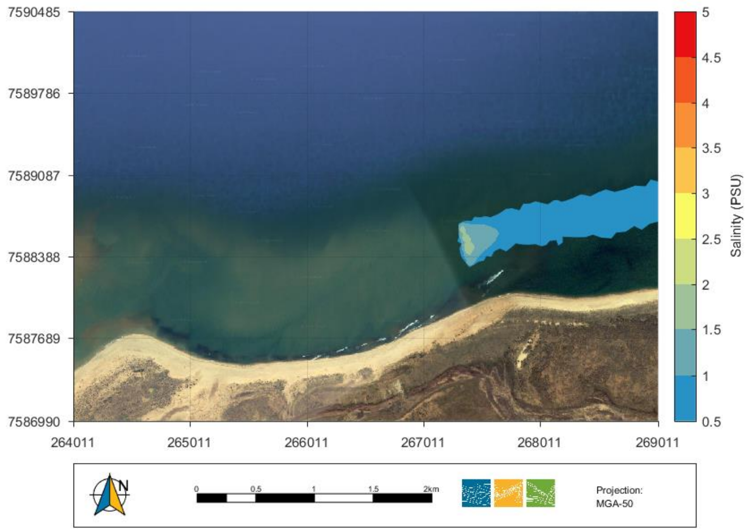
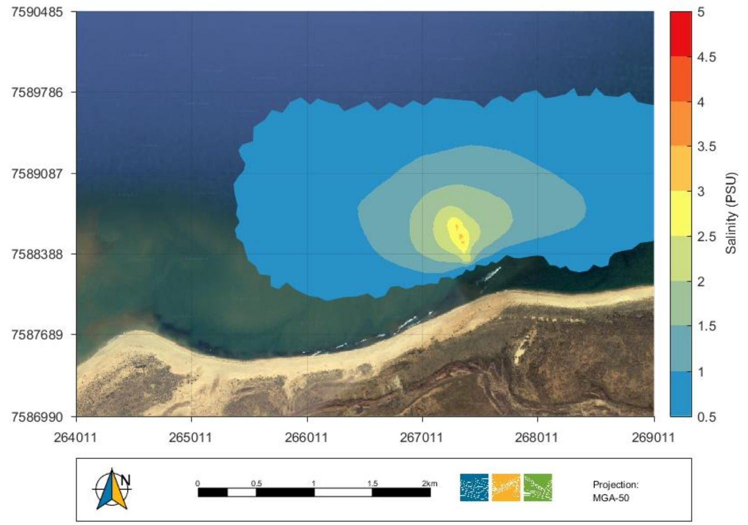
- the magnitude of the modelled increases in salinity is closely related to the rate of bitterns discharge, which varies considerably month by month according to evaporation rates with highest discharge rates in summer and lowest discharge rates in winter.
- The far-field mixing diffuses the discharged bitterns in both alongshore and cross-shore directions.
- The modelled salinity increases represent the best level of dilution that can be achieved with an optimised diffuser design and layout:
  - The diffuser design optimisation process is outlined in Section 4.5 to 4.6.
  - The jetty and diffuser layout has been optimised to take advantage of greater water depth (moving the jetty 200 m to the north-west).
  - The diffuser alignment was adjusted to be perpendicular to tidal currents (rotating the jetty and diffuser by approximately 10 degrees) as outlined in Section 4.6.8.

TABLE 4-9 MODELLED IMPACTS TO SALINITY FIELD

<table>
<tr><th>Scenarios</th><th>Differences – Surface</th><th>Differences – Bottom</th></tr>
<tr><td colspan="3">2015 Yearly Statistics</td></tr>
<tr><td>50th Percentile (2015) – 50% of the time</td><td>
</td><td>
</td></tr>
<tr><td>80th Percentile (2015) – 20% of the time</td><td>
</td><td>
</td></tr>
</table>

<table>
<tr><th>Scenarios</th><th>Differences – Surface</th><th>Differences – Bottom</th></tr>
<tr><td>90th Percentile (2015) – 10% of the time</td><td></td><td></td></tr>
<tr><td>99th Percentile (2015) – 1% of the time</td><td></td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
</table>

Scenarios	Differences – Surface	Differences – Bottom
Monthly Statsitics		
80th Percentile (Jan 2015, 0.88 m^3/s discharge)		
80th Percentile (Jun 2015, 0.14 m^3/s discharge)		

Scenarios	Differences – Surface	Differences – Bottom
80th Percentile (Aug 2015, 0.41 m^3/s discharge)		
80th Percentile (Nov 2015, 0.98 m^3/s discharge)		

Note preceding pages have been removed as they relate to the calculation of ecological protection areas. These calculations are more accurately discussed in AECOM (2022a) *Assessment of Benthic Communities and Habitats. Ashburton Salt Project.*

4.7.7 Dilution Analysis

As per Section 4.7.3, the prediluted bitterns discharge scenario was used in the far-field model simulations as this was considered to be the most appropriate due to the practicalities of the diffuser design. Section 4.7.5 presents the potential impacts of these prediluted bitterns relative to background salinity. In response to EPA comments, an additional presentation of potential impacts relative to overall bitterns dilution is undertaken in this section.

K+S determined bitterns dilution guideline values to inform species protection levels, based on ecotoxicity analysis. The dilution guideline percentages are presented in the far right hand column of Table 4-10. These correspond to species protection levels in the far left hand column of the table.

TABLE 4-10 SUPPLIED RECOMMENDED GUIDELINE DILUTION VALUES (%)

Species Protection Level (%)	LEP	Estimated Dilution Ratio	Dilution Guideline (%)
99	High	417	0.24
95	-	-	0.33
90	Moderate	263	0.38
80	Low	227	0.44

The bitterns discharge modelling results were analysed using the following steps to apply these guideline values:

- As per the modelling presented in Section 4.7.5, the hydrodynamic model was simulated for 1-year (2015) for two scenarios: existing conditions and the proposed bitterns discharge.
- As per the modelled results presented in Section 4.7.5, the existing conditions salinity at each timestep across the model domain was then subtracted from the proposed bitterns discharge scenario, providing a spatial timeseries of predicted "**salinity exceedance**" due to the proposed development.
  - The above two steps are the same as previously presented in Section 4.7.5
- To present these results in terms of overall bitterns dilution, relative to the raw bitterns, the dilution was then calculated according to the following formula:
  - $Dilution\ (\%) = \frac{Modelled\ Salinity\ Exceedance}{Raw\ Bitterns\ Concentration} \times 100$
  - Where the raw bitterns concentration is 291.3 PSU, as supplied by K+S.

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- The median, 80th, 90th and 99th percentile dilutions in each element of the model was then calculated for the full 1-year simulation, and also for the months of June (lowest discharge rate) and November (highest discharge rate).

The resultant dilution spatial plots for the full-year analysis are presented in Figure 4-15 to Figure 4-18 below. The spatial distribution of the potential impact zones is similar to the plots presented in Table 4-9, as expected given it is the same modelling, with just a different method applied to interpret the potential impact. The plots show that, over a year:

- 50% of the time:
  - No impacts are predicted to species at the surface.
  - At the seabed, the impact is predicted to be limited to the 99% species protection level, extending approximately 170m from the jetty.
- 20% of the time, it is predicted that:
  - The 80% species protection level is exceeded within a 350m radius of the jetty location on the seabed, and not exceeded at all at the surface.
  - The 99% species protection level is exceeded within a 1.1km radius of the jetty location on the seabed, and 300m at the surface.
- 10% of the time, it is predicted that:
  - The 80% species protection level is exceeded within a 530m radius of the jetty location on the seabed, and within 60m at the surface.
  - The 99% species protection level is exceeded within a 1.6km radius of the jetty location on the seabed, and 450m at the surface.
- 1% of the time:
  - The 80% species protection level is exceeded within a 1.4km radius of the jetty location on the seabed, and within 350m at the surface.
  - The 99% species protection level is exceeded within a 3.5km radius of the jetty location on the seabed, and 2.5km at the surface.

The discharge rate is expected to vary throughout the year (refer Table 4-3), to account for variable levels of production. Figure 4-19 and Figure 4-20 present the 80th percentile dilutions at the surface and bottom for the months of June (minimum discharge rate) and November (maximum discharge rate) respectively. These results show that:

- In June:
  - The 80% species protection level is not predicted to be exceeded at all.
  - The 99% species protection level is exceeded within a 140m radius at the seabed, and not exceeded at all at the surface.
- In November:
  - The 80% species protection level is exceeded within a 730m radius of the jetty location on the seabed, and within 180m at the surface.
  - The 99% species protection level is exceeded within a 2km radius of the jetty location on the seabed, and 1.1km at the surface.
  - The results sit somewhere between the 90% and 99th percentile plots for the full year, as expected given November has the maximum discharge rate.

It should be noted that the dilution statistics are calculated in each model element individually, so the spatial plots represent the dilution independent of time. That is, the extent of impacts will not necessarily match this full extent at any one time, but instead the figures represent the greatest spatial footprint of the development over the course of the simulated timeframe.

FIGURE 4-15 PREDICTED 50TH PERCENTILE DILUTION AT THE SURFACE (LEFT) AND BOTTOM (RIGHT)

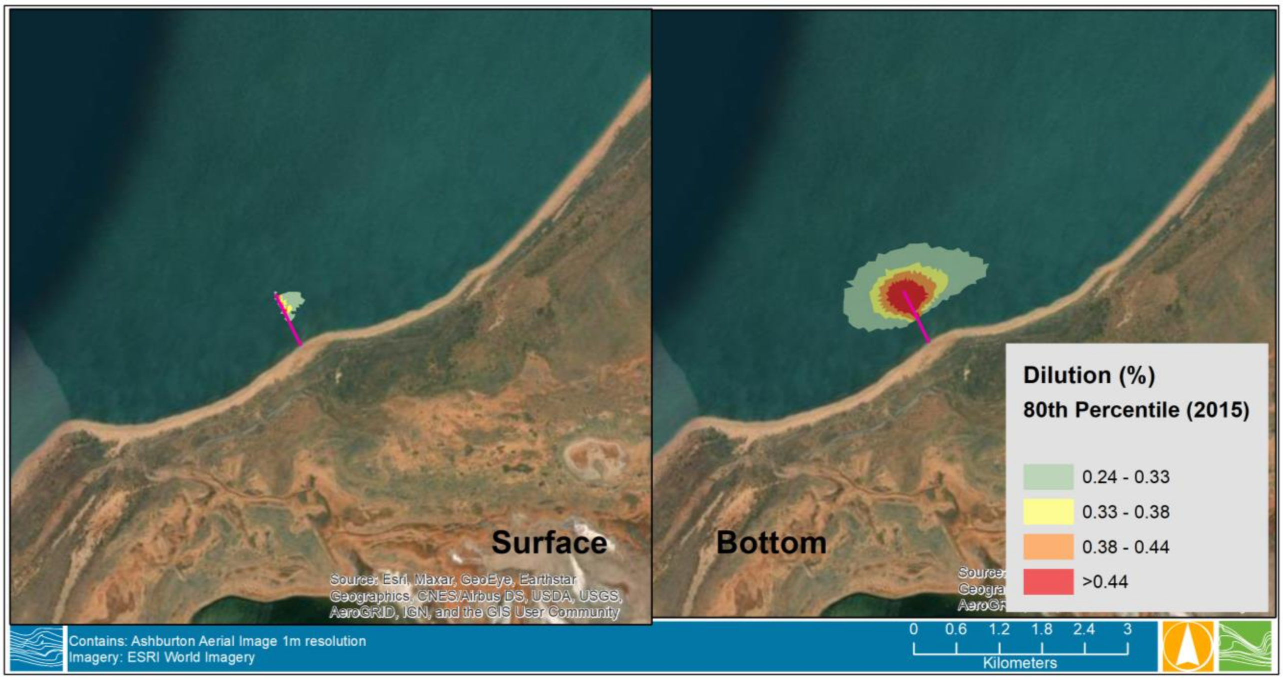


FIGURE 4-16 PREDICTED 80TH PERCENTILE DILUTION AT THE SURFACE (LEFT) AND BOTTOM (RIGHT)

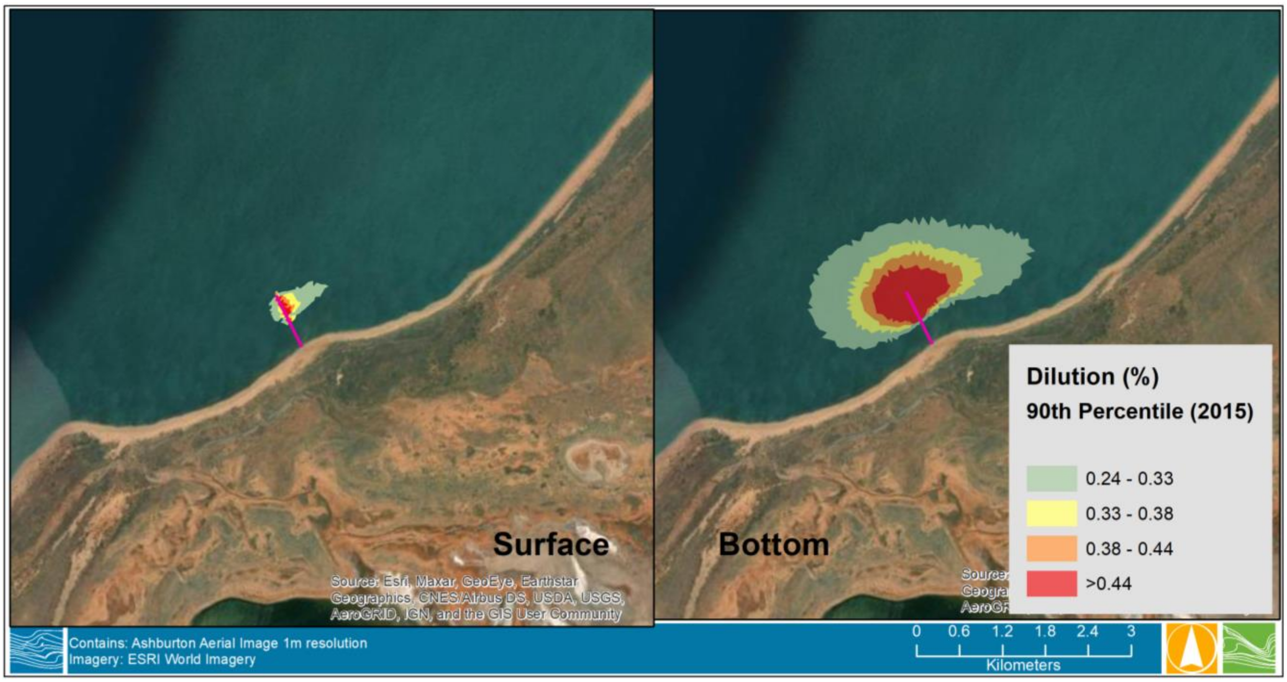


FIGURE 4-17 PREDICTED 90TH PERCENTILE DILUTION AT THE SURFACE (LEFT) AND BOTTOM (RIGHT)

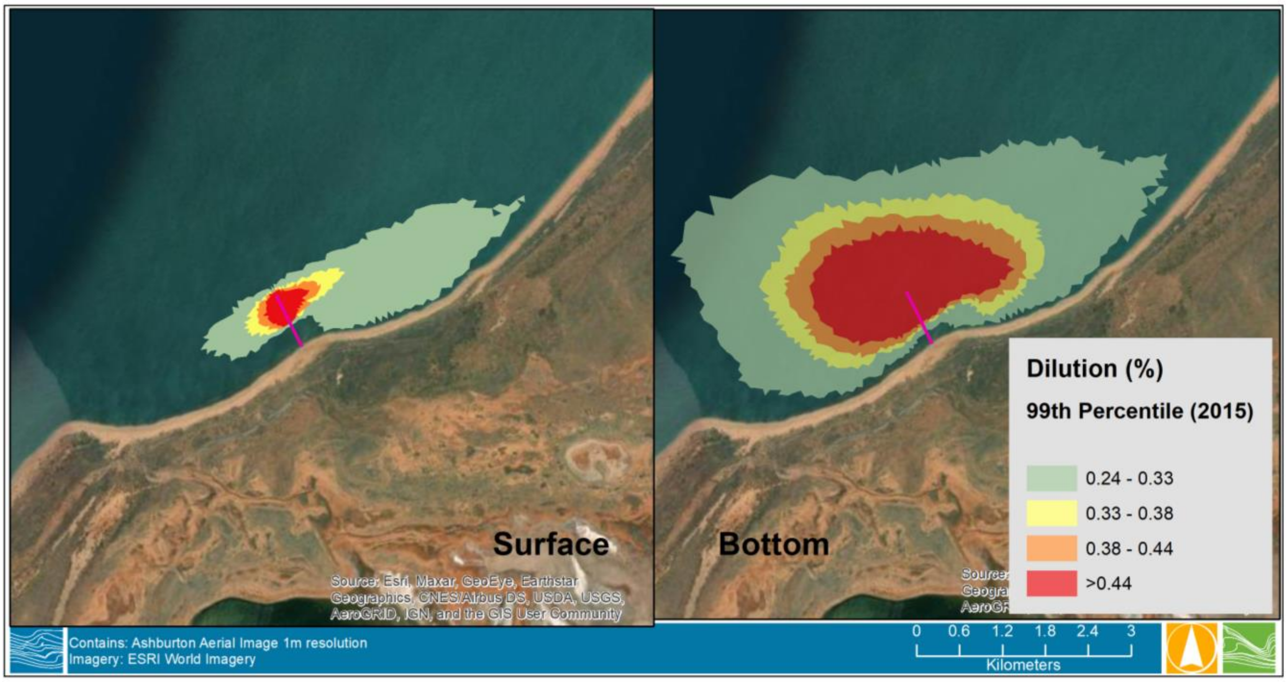


FIGURE 4-18 PREDICTED 99TH PERCENTILE DILUTION AT THE SURFACE (LEFT) AND BOTTOM (RIGHT)

FIGURE 4-19 PREDICTED 80TH PERCENTILE DILUTION AT THE SURFACE (LEFT) AND BOTTOM (RIGHT) FOR THE MONTH OF JUNE

FIGURE 4-20 PREDICTED 80TH PERCENTILE DILUTION AT THE SURFACE (LEFT) AND BOTTOM (RIGHT) FOR THE MONTH OF NOVEMBER

4.8 Summary

Bitterns discharge has been investigated in two parts:

1. Nearfield analysis to develop a practical and suitable diffuser design and achieve dilution of the bitterns at a rate of approximately 100:1 in the immediate vicinity (less than 10 m) of the diffuser.

2. Far-field modelling to investigate water quality impacts on the receiving environment surrounding the diffuser.

Near-field analysis was undertaken through both analytical and numerical (CORMIX) approaches. Results showed that the mixing forces surrounding the diffuser are dominated by port sizes and water depths above the port opening. The negative buoyancy of the bitterns leads to the settling of bitterns in the water column and as such adverse impacts can be mitigated through a proper design of the diffuser.

It was found that positioning the port openings at middle depth in the water column and discharging upwards at an angle of 60° with usage of a proper jet speed and port size achieved the required near-field dilution (about 100:1 within 10 m of the diffuser, corresponding to a 1.4 PSU salinity increase above background which is within the range of natural variation at the site). The recommended 25 mm port diameter is already very small, and usage of any smaller port size will be challenging in terms of both practicality and reliability.

In addition, the jetty and diffuser layout has been optimised to take advantage of greater water depth as well as an adjusted diffuser alignment perpendicular to tidal currents to maximise far field dilution.

The proposed bitterns discharge design and layout has shown good performance in terms of both near-field and far-field dilution. The targeted near-field dilution rate (100:1) is consistent with the assimilative capacity (or absorption capability of receiving water) of ambient water flow and thereby provides the most practical and optimal diffuser design.

Far-field impacts were investigated using a MIKE 3D hydrodynamic model incorporating seasonality of dynamic forcing (e.g., winds, subtidal water levels and currents) and discharge rates. Analysis of ecological protection area (mixing zone) sizes was carried out based on EPA guidelines and according to EQGs derived from water quality monitoring data, ANZG (2018) and relevant research. All mixing zone contours were generated based on pre-diluted bitterns with one part seawater.

5 DREDGING PLUME MODELLING

5.1 Introduction

The marine facility was designed to load the Ashburton Salt Project salt onto a transhipment barge to an offshore ocean-going vessel. The marine facility includes a piled jetty, loading platform, dolphins, berthing pocket, and navigational aids to facilitate docking and loading.

Dredging work is required to deepen the water depth in the berthing pocket so transhipment vessels can remain docked in the berth at low tide and tranship offshore at higher tide. According to the marine facility's pre-feasibility study and concept design, the proposed transhipment barge will have a ballast draft of 3.5 m and 5 m once fully loaded. This requires about 2.5 m dredging from the current seabed level (about 5.2 m AHD). The proposed location of the berthing pocket and survey contours are shown in Figure 5-1.

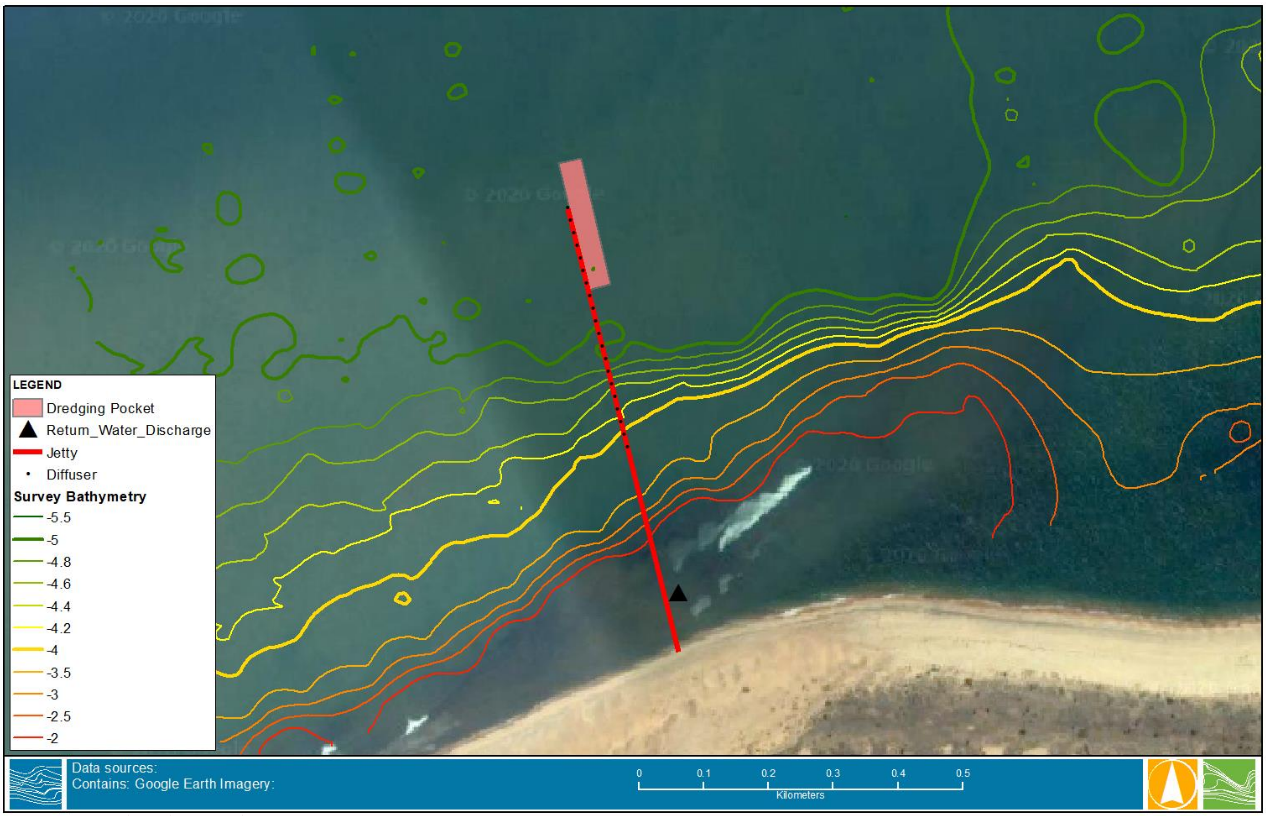


FIGURE 5-1 PROPOSED BERTH POCKET

Water Technology acquired information from dredge suppliers to facilitate conceptualisation of the dredging program. Based on the water depth requirement and a geotechnical survey of bed material, a small to medium sized cutter suction dredge will likely be suitable to undertake the proposed dredging works. Based on this advice, K+S have confirmed the use of a cutter suction dredge for this project. All dredged material will be:

- Disposed in a bunded area on land.
- Retained for an adequate time to achieve settlement of suspended solids and achieve required water quality standards.

After the above treatment and water quality standards have been met, the decant water will be released back to the nearshore marine environment. It is however difficult to accurately estimate this turbidity without knowing the design of the dewatering pond. In order to provide a more conservative assessment of the turbidity plume generated by the dewatering process, it is assumed that the decant water with a constant turbidity of 100 NTU will be discharged directly into the nearshore water. This is a conservative estimate generated by a review of existing dredging projects undertaken in similar environments in the Pilbara region.

According to 'Technical Guidance - Environmental Impact Assessment of Marine Dredging Proposals' (EPA 2016), dredging activities and associated sediment plumes have the potential to influence larger areas beyond the footprint of the development boundary. Environmental impact assessment is thereby required to be based on the prediction of the extent, severity and duration of environmental impacts.

This aim of this dredging plume modelling study was to predict the potential dredging plume impact. More specifically, Water Technology has proposed a preliminary dredging plan and conceptualised this plan in our model with sufficient allowances to account for any known uncertainties. Details of the dredging plume modelling and outcomes are presented in subsequent sections.

5.2 Dredge Conceptualisation

The proposed dredging operation was conceptualised to represent the activity in the numerical model. The dredge conceptualisation phase involves the characterisation of the following key components, which are detailed below:

- Sediment composition (Section 5.2.1).
- Dredging volumes (Section 5.2.2).
- Dredging method (Section 5.2.3).
- Sediment release (Section 5.2.4).

5.2.1 Sediment Composition

A basic description of sediment layers in bore logs was provided in a geotechnical report (GHD 2020). In general, there are four layers of seabed material in the area:

- Soft clayey silt, typically down to 0.4 m below the seabed (but as low as 0.7 m).
- Medium density clayey sand, typically down to 0.4 to 1.1 m below the seabed (lower level ranges from 1.0 to 2.7 m).
- Stiff Sandy clay, typically down to 2.2 m below the seabed (lower level ranges from 1.3 to 2.7 m).
- Hard Rock, typically 2.2m below the seabed.

It should be noted that the GHD (2020) bore hole sampling was done along the previously proposed Jetty location. The updated Jetty location is 200m to the east, however It has been assumed the new location has the same geotechnical conditions. It is subject to confirmation whether these values should be adopted during the detailed design stage.

Review of the dredging depth requirement (i.e., 6.0 m depth at LAT) indicates about 2 m to 2.5 m of bed material is required to be removed from within the dredging pocket. This requires dredging work through the three top layers: the clayey silt top layer, the underlying clayey sand layer and a stiff sandy clay layer. Dredging through the rock layer is likely to be unnecessary.

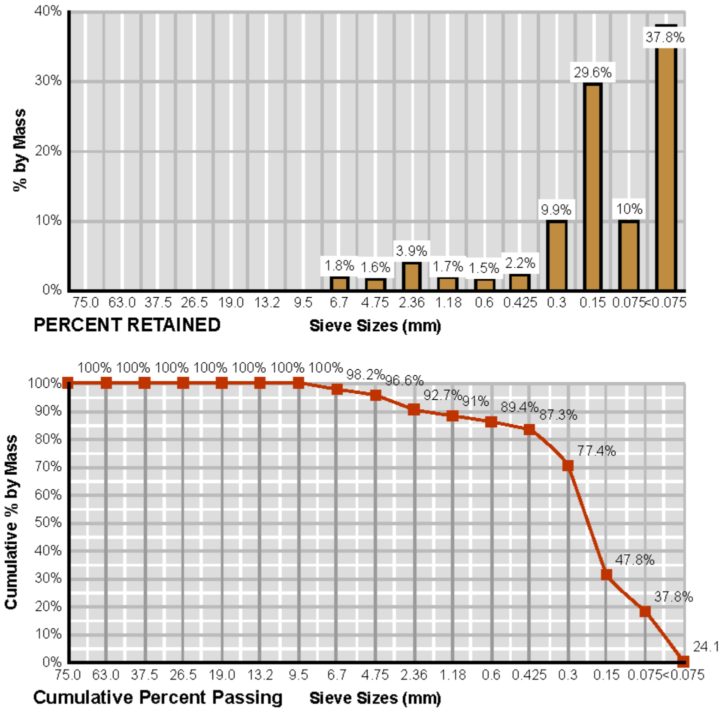
The sediment Particle Size Distribution (PSD) for each layer is not available from the GHD geotechnical report. A PSD was available for one site, as shown in Figure 5-2. Primary sediments are medium to fine sands while there is a significant portion (about 19%) of clay with particle sizes less than 0.002 mm. The PSD data is consistent with the "clayey sand" second layer described by GHD in their geotechnical report.

SUMMARY
Analysed by SESL Australia Pty Ltd, NATA
Results only requested.

D VALUES	
D_{95}:	0.504
D_{90}:	0.467
D_{85}:	0.429
D_{60}:	0.164
D_{50}:	0.124
D_{15}:	0.002
D_{10}:	0.001
D_{5}:	0.001

PERFORMANCE FACTORS	
Gradation Index (D_{90}/D_{10}):	470.00
Coefficient of Uniformity: (D_{60}/D_{10})	160.00

PARTICLE SIZE DISTRIBUTION GRAPH



PARTICLE SIZE ANALYSIS

Sieve (mm)	Fraction	% Retained by mass	% Passing by mass
75.0	Cobbles	0	100
63.0	Very Coarse Gravel	0	100
37.5	Coarse Gravel	0	100
26.5	Coarse Gravel	0	100
19.0	Medium Gravel	0	100
13.2	Medium Gravel	0	100
9.5	Medium Gravel	0	100
6.7	Fine Gravel	1.83	98.17
4.75	Fine Gravel	1.62	96.55
2.36	Fine Gravel	3.85	92.7
1.18	Very Coarse Sand	1.72	90.98
0.6	Medium Sand	1.54	89.44
0.425	Medium Sand	2.15	87.29
0.3	Medium Sand	9.89	77.4
0.15	Fine Sand	29.6	47.8
0.075	Fine Sand	9.99	37.81
FINE PARTICLES (Hydrometer calculated)			
0.02	Very Fine Sand	13.73	24.08
0.002	Silt	5.14	18.94
<0.002	Clay	18.94	N/A

FIGURE 5-2 PSD DIAGRAM OF SEDIMENTS SAMPLED AT SED03 LOCATION (SHOWN AS RED STAR)

5.2.2 Dredging Volume

Dredging of the berthing pocket along the eastern end of the export jetty is required to allow adequate water depth for the transhipper to remain within the berthing pocket without tidal restriction. The berthing pocket is required to be of sufficient depth, length and width to allow the loaded transhipper sufficient under keel clearance to remain in the berthing pocket under all tide conditions.

K+S have indicated the berthing pocket will be about 200 m in length (i.e., 150 m berth length plus 50 m extension beyond the jetty), 35 m in width and 6 m in depth at low tide. The area has an approximate water depth of around 5.2 m MSL. The berthing pocket requires a dredge depth of about 2.5 m, subject to variation of bed levels along the jetty. Based on these specifications the total volume of material to be removed is estimated to be around 17,000 m^3.

5.2.3 Dredging Method

A small to medium sized cutter suction dredger will likely be used to carry out the proposed dredging work. For example, Cooper Dredging (http://www.cgcgroup.com.au/equipment-specifications/mudlark-i.html) has a cutter suction dredger with the following specifications:

- 7 m dredging depth.
- 300 mm diameter discharge pipe.
- Hydraulic cutter up to 9 kw bucket wheel or crown head options.
- Designed to be transported on road trailer that meet the road transport regulation.
- Self-contained for launching and retrieval by specially designed trailer.

It is intended to achieve a target production rate of about 1,200 m^3 per day so the dredging campaign can be finished within 14 days (assuming 8 hours operation per day).

Dredge spoil disposal will occur onshore. The dredged slurry will be pumped to a bunded area. The bunded area will have a sufficient volume such that the material can be retained for a long enough time period to achieve necessary water quality standard. The return water would be expected to have a typical turbidity of about 100 NTU or total suspended solid (TSS) of about 100 mg/L. This is three times higher than the 98th percentile turbidity observed at Locker Point in the monitoring program conducted (*Marine, Coastal and Surface Water Data Collection*, Water Technology 2021).

5.2.4 Sediment Release

The dredging operation will have the following characteristics:

- In total about 17,000 m^3 of dredge volume is to be removed (200 m x 35 m x 2.5 m).
- The target slurry flow rate will be 1,020 m^3 per hour, with a production rate of 15% solids.
- 14 days dredging time is required based on 8 working hours per day from 8 am to 4 pm.
- The cutter head releases about 1.5 kg/s of suspended solids during dredging.
- 0.007 kg/s of suspended solid from return water is estimated.
- The dredger will move through the berthing pocket.

From the above assumptions, two sediment sources are implemented in the model:

- Dredge Total Suspended Solids (TSS) source:
  - Eight locations along the berthing pocket.
  - Operation during 8 am and 4 pm.
  - 15 days spin up and continuous plume generation of 1.5 kg/s for 8 hours per day over 30 days. Note that the required time of dredging is only 14 days, but the simulation period has been expanded in order to assess the potential impacts under a wider range of wind conditions.
  - Spill of discharged material near the bottom.
- Return water TSS source:
  - One discharge location near the beach.
  - 15 days spin up and continuous plume generation of 0.028 kg/s over 30 days of model simulation.
  - Spill of discharged material throughout the water column.

The model simulated a continuous dredging period of one month (8 hours per day). This has provided over 100% allowance for the required dredging volume to be removed, which is sufficient to account for future variations of the dredging plan and uncertainty of surveyed bathymetry. This is a conservative approach and is expected to overestimate potential sediment plume impacts.

5.3 Modelling Package

5.3.1 Hydrodynamic Model

MIKE 3 FM HD was developed for applications across oceanographic, coastal and estuarine environments. It is the basic computational component of the remaining MIKE FM modelling systems, providing the three-dimensional hydrodynamic conditions for the other modules.

The hydrodynamic module is based on the numerical solution of the three-dimensional incompressible Reynolds averaged Navier-Stokes equations, invoking the assumptions of Boussinesq and hydrostatic pressure. The model consists of continuity, momentum, temperature, salinity and density equations and is closed by a turbulent closure scheme. The spatial discretisation of the equations is performed using a cell-centred finite volume approach. In the horizontal plane an unstructured grid is used, while in the vertical domain a structured mesh is applied.

The hydrodynamic model is described in detail in Section 2 and Section 4.7.2.

5.3.2 Mud Transport Module

MIKE Mud Transport (MT) Model was coupled with the hydrodynamic model to investigate the transport and fate of sediments released into the water column throughout the dredging program. The MT module is designed to simulate suspension, transportation and deposition of non-cohesive and cohesive sediments under combined coastal processes. The model can be run under either coupled or decoupled mode so the hydrodynamic results can be reused to simulate different dredging scenarios.

Total suspended solid (TSS) or equivalently suspended solid concentration (SSC) was modelled and used as an indicator to evaluate the impacts of the dredging plume.

5.4 Model Setup

5.4.1 General Settings

- High order accuracy.
- Sediment specific density 2650 kg/m^3.
- Critical shear stress for deposition 0.1 N/m^2.
- Critical shear stress for erosion 0.25 N/m^2.
- Layer thickness was configured with an erosible layer in the dredging pocket and 0 m thickness of this layer in other areas. This is to eliminate the impacts from erosion of the existing seabed not affected by dredging operation. Note 0 m layer thickness still allows for resuspension and deposition of plume material in other areas.

5.4.2 Background TSS

The dredging plume model is executed with no background TSS. The model results represent sediment concentrations above the background TSS.

Natural background TSS varies significantly with tidal phases and seasons. The median background turbidity at Locker Point is about 5 NTU. The background turbidity levels of the existing environment regularly exceed

10 NTU at ebb tide and occasionally exceed 50 NTU in bad weather. The 98th percentile turbidity observed around 2% of the time at Locker Point in water quality monitoring is approximately 25 NTU (*Marine, Coastal and Surface Water Data Collection*, Water Technology 2021).

5.4.2.1 Sediment Settling Velocity

A key parameter in sediment plume modelling is sediment fractions and associated settling velocity. A literature review shows caution against the practice of deriving settling velocity based on laboratory analysis of sediment core data or water samples alone, as the dredged material would likely be different from the natural suspended particles.

Van Rijn (1990) has shown that the settling rates of fine silts and clays are dependent upon flocculation. At high concentrations, there is a greater amount of flocculation, and the larger flocs that form settle out much faster than individual sediment particles. At lower concentrations, there is less opportunity for large flocs to form, and the sediment settles out significantly more slowly. Rather than using a relatively small number of discrete fractions, each with its individual settling velocity, a concentration dependent settling rate has been used. With this approach, a continuous spectrum of settling velocities is used where, at any given time and place, the settling rate is determined by the sediment concentration and a pre-determined concentration vs settling rate relationship. It is considered that this approach provides a better reproduction of the natural processes occurring in nature.

For the present study, the concentration / settling rate relationship was determined from large scale settling tube analyses carried out on silty clay material (McCowan and Kahl, 2006), with reference to van Rijn (1990). With this relationship, the settling rates reduce from 1.0 mm/s at a relatively high TSS concentration of 1,000 mg/l to 0.1 mm/s at a moderate TSS concentration of 100 mg/l. These rates are consistent with the faster settling rates described by WAMSI (2020) for resolving *“the initial spreading of the passive plume and resuspension events”*. At lower concentrations, the settling rate would further reduce down to 0.01 mm/s at a relatively low concentration of 10 mg/l. This slowest settling rate is consistent with slow settling rates described by WAMSI *“to reproduce low concentration background TSS”*.

Even at a very low settling rate of 0.01 mm/s, WAMSI noted that, in their case study, *“a low background TSS concentration (of 1-2 mg/L) was not reproduced by the model”*. As a result, WAMSI suggests the contribution of slow settling sediments to the background TSS could be estimated empirically. For the present study, only the dredge sediment concentrations above background concentrations have been simulated. The overall TSS occurring in the vicinity of the dredge site can then be approximated by adding in the median background TSS in the area of 9 mg/l, as reported in Water Technology (2021), the Marine, Coastal and Surface Water Data Collection Report.

The range of settling velocities simulated (0.01 mm/s to 1 mm/s) covers suspended sediments from dredging of the silt and clay material at the Project site. Coarser materials (e.g., coarse silt and sand) will settle much faster and would have a very limited effect on the assessment results. A sensitivity test was carried out to ensure that the dispersion model was operating consistently with the applied inputs.

5.4.3 Representative Seasonal Scenarios

Model periods were selected to represent the typical seasonal climate of the site. The year 2015 was previously identified as having average meteorological conditions. Within that the year the following two periods were identified:

- A summer season during January and February 2015 (see Figure 5-3). This represents a high wind (non-cyclonic) energy season with prevailing wind blowing from southwest.
- A winter season during June and July 2015 (see Figure 5-3). This represents a low wind (non-cyclonic) energy season with wind from various directions.

Review of wind data shows occurrence of a tropical cyclone in March and this month was excluded.

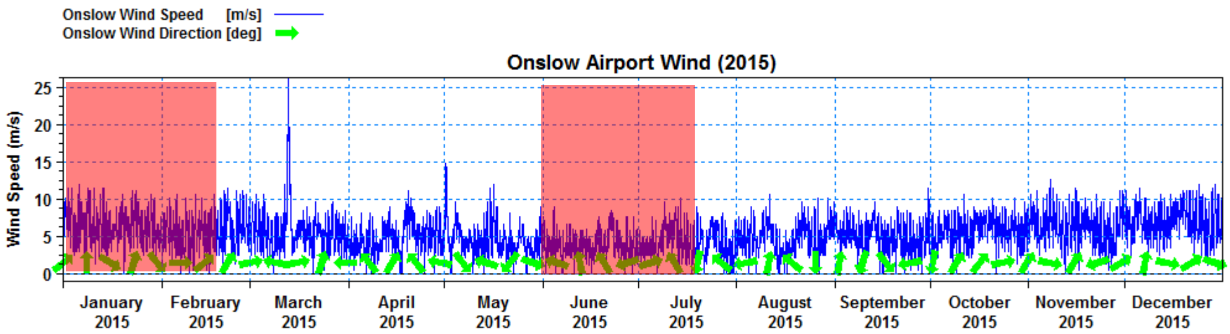


FIGURE 5-3 TIMESERIES OF WIND SPEED AND DIRECTION AT ONSLOW FOR 2010 (TOP), AND 2015 (BOTTOM). MODEL PERIODS ARE HIGHLIGHTED BY RED

5.5 Model Results

5.5.1 Impact Zone Scheme and Results Analysis

The EPA has developed a spatially based zonation scheme for proponents to use as a common basis to describe the predicted extent, severity and duration of impacts associated with dredging proposals. The scheme consists of three zones to represent different levels of impacts.

- **Zone of High Impact (ZoHI)** is the area where impacts on benthic communities or habitats are predicted to be irreversible. The term irreversible means 'lacking a capacity to return or recover to a state resembling that prior to being impacted within a timeframe of five years or less'. Areas within and immediately adjacent to proposed dredge and disposal sites are typically within zones of high impact.
- **Zone of Moderate Impact (ZoMI)** is the area within which predicted impacts on benthic organisms are recoverable within a period of five years following completion of the dredging activities. This zone abuts, and lies immediately outside of, the zone of high impact. The outer boundary of this zone is coincident with the inner boundary of the next zone, the Zone of Influence.
- **Zone of Influence (ZoI)** is the area within which changes in environmental quality associated with dredge plumes are predicted and anticipated during the dredging operations, but where these changes would not result in a detectible impact on benthic biota. These areas can be large, but at any point in time the dredge plumes are likely to be restricted to a relatively small portion of the Zone of Influence.

In dredging plume assessments, cumulative probability and running mean methods are often used to identify the zone of impact. Calculation of each zone is based on analysis of either cumulative probability (% days) or running mean TSS against possible coral mortality thresholds shown in Table 5-1. It should be noted that natural background turbidity levels at Locker Point regularly exceed some of these thresholds. Both methods are used to consolidate our findings.

The following methodology was used to define each zone:

- ZoHI is determined by thresholds corresponding to a high probability of observing non-zero coral mortality (see permissive thresholds in Table 5-1, SSC/TSS>6.9 mg/L for 20% cumulative probability/80th percentile, SSC/TSS>13.2 mg/L for 28 days running average).
- ZoMI is determined as the area that encompasses the region immediately outside the ZoHI up to distances where thresholds are indicative of only possible coral mortality (see strict thresholds in Table 5-1, SSC/TSS>5 mg/L for 20% cumulative probability/80th percentile, SSC/TSS>9.3 mg/L for 28 days running average).

- ZoI, where changes to water quality may occur, but not to an extent that constitutes a hazard to any underlying coral communities. It is determined up to distances where modelled exceedance of TSS is above 2 mg/L. Note 2 mg/L represents a typical lower range (5th percentile) of TSS measured at Locker Point.

TABLE 5-1 DERIVED POSSIBLE AND PROBABLE CORAL MORTALITY THRESHOLDS (FISHER, 2019) FROM THE DREDGING SCIENCE NODE OF WAMSI

Thresholds type		Possible effects (*strict*)			Probable effects (*permissive*)		
		>NTU	>SSC	<DLI	>NTU	>SSC	<DLI
Cumulative probability (% days)	**90%**	0.7	1.3	7.4	0.7	1.3	6.6
	80%	1.0	1.9	6.3	1.2	2.1	5.4
	70%	1.3	2.3	5.5	1.6	2.8	4.6
	60%	1.5	2.8	4.9	1.9	3.4	4.0
	50%	1.8	3.2	4.4	2.3	4.1	3.4
	40%	2.0	3.7	3.8	2.6	4.8	2.8
	30%	2.3	4.2	3.2	3.1	5.6	2.3
	20%	2.8	5.0	2.6	3.8	6.9	1.7
	10%	3.5	6.3	1.7	5.1	9.1	1.0
Running mean (days)	**1 d**	15.5	27.9	0.4	32.4	58.3	0.1
	3 d	**10.8**	**19.4**	**1.1**	**19.9**	**35.7**	**0.3**
	7 d	**8.2**	**14.7**	**1.8**	**13.6**	**24.5**	**0.6**
	10 d	7.3	13.1	2.2	11.6	20.9	0.9
	14 d	**6.5**	**11.7**	**2.5**	**10.0**	**18.0**	**1.1**
	17 d	6.1	11.0	2.7	9.2	16.5	1.3
	21 d	5.7	10.2	2.9	8.3	15.0	1.5
	28 d	**5.2**	**9.3**	**3.1**	**7.3**	**13.2**	**1.8**
	30 d	5.1	9.1	3.1	7.1	12.8	1.9

The following analysis and post-processing were performed on the model results:

- Extraction of TSS for the bottom and surface sigma layer for both modelled seasons.
- Estimation of the 80th percentile and running average statistics for the periods when dredging operation is undertaken (30 days modelled).
- Mapping of TSS exceeding the ZoHI, ZoMI and ZoI thresholds. Note different thresholds are applied to different cumulative probabilities.

5.5.2 Dredging Plume Results

Model results are presented in Table 5-2. The ZoHI, ZoMI and ZoI are highlighted by red, yellow and light green contours, respectively.

In the summer simulation, the prevailing south-westerly wind drove a slow current eastward. This resulted in an eastward trajectory of the dredging plume as shown by the left column of Table 5-2. The following can be concluded from the summer dredge plume modelling:

- Other than the dredging footprint itself, which is directly impacted by the suction cutter dredge, the 20% cumulative probability shows no ZoHI area. The modelled ZoMI area is about 1,000 m in length and about 100 m in width for both the bottom and surface layer. The ZoI zone may extend a few kilometres to the east at a width of about 300 m.
- The running average shows a very small ZoHI area at the bottom layer (about 50 m in width and 200 m in length within the berthing pocket). The modelled ZoMI is about 150 m in width and 300 m in length which covers only the area in the vicinity of dredging pocket. The ZoI zone extends about 2 kilometres to the east at a width of about 400 m. Very limited impacts are found at the surface layer.

For the winter simulation, the absence of a prevailing south-westerly wind has led to a weak westward current and a westward trajectory of the dredging plume as shown by the right column of Table 5-2. The model results can be summarised as follows:

- Other than the dredging footprint itself, which is directly impacted by the suction cutter dredge, the 20% cumulative probability shows no ZoHI near the surface and a 200 m diameter of ZoHI near the bottom. The modelled ZoMI is about 400 m diameter near bottom, and about 100 m diameter near the surface. ZoI extends a few kilometres to the west at a width of about 1000 m. A small (<200 m) ZoHI/ZoMI is modelled near the location where the decant water is discharged from the sediment settling pond.
- The running average shows very similar ZoHI and ZoMI zones as in summer i.e., about 50 m X 200 m for ZoHI and about 150 m X 300 m for ZoMI. The ZoI extends for about 1.5 kilometres to the west at a width of about 500 m. Very limited impacts are found at surface layer.

Overall, for the 20% cumulative probability (67 hours of exposure in 14 days dredging), the model has identified a 200 m ZoHI zone (<0.05 km^2) in winter when the wind/current is low. The shape of the ZoMI may change in different seasons, showing a stretched narrow zone in summer (100 m X 1000 m) and roundish zone in winter (400 m diameter). The area of ZoMI zone is less than 0.2 km^2 for both seasons which likely envelopes the maximum area of reversible impact. The ZoI outlines an area (green contour in Table 5-2) of small disturbance that is lower than the median background TSS with no detectable impacts.

There are small vertical differences in the ZoI, as the lower concentration plumes have lower settling rates and tend to be well mixed through the water column during the transportation and dispersion process. Larger vertical differences are found in ZoMI and ZoHI where the higher concentration plumes settle more quickly.

The duration of exposure corresponding to the 20% cumulative probability is about 67 hours over 14-days and 144 hours (6 days) over a 30- day dredging campaign. The trigger value (Fisher, 2019) is based on a typical dredging period which is 4-10 weeks in WA and 2-4 weeks in Queensland. These are both longer than the proposed dredging plan (about 2 weeks). The estimates presented are thereby conservative, given the extra 100% (additional 14 days) duration of dredging period simulated by the model.

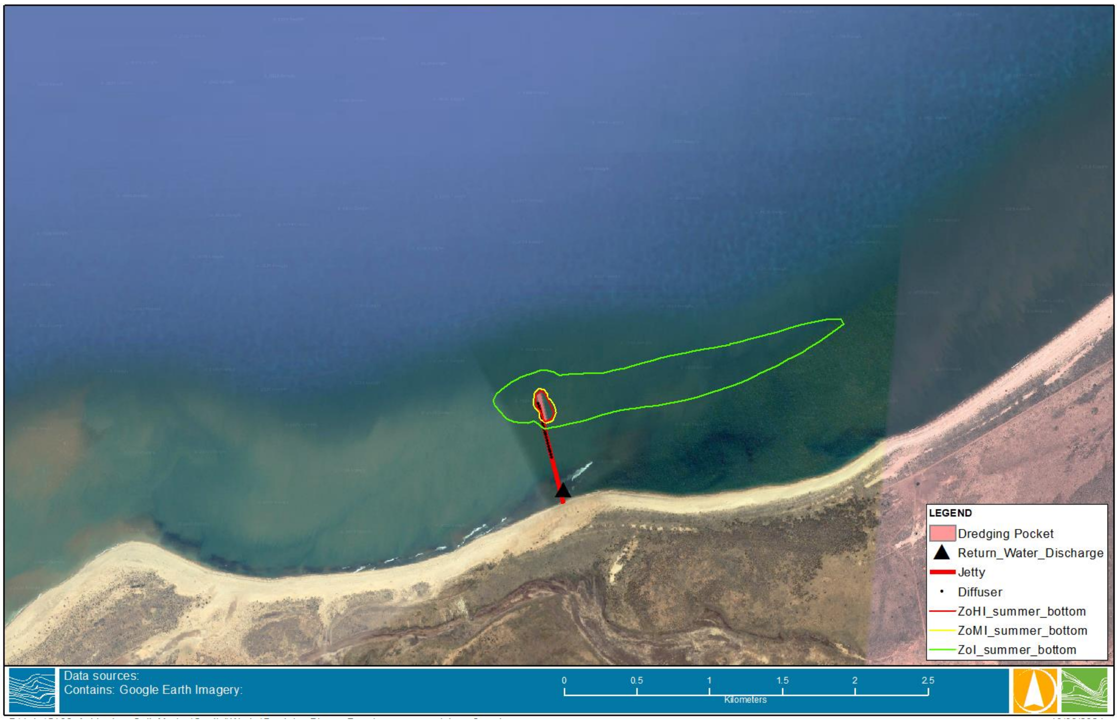
From Table 5-1, 14 days of duration has a trigger TSS/SSC of 10.0 mg/L for ZoHI and 18.0 mg/L for ZoMI, both significantly higher than the trigger value used for this assessment. We are thus confident that both running average and 20% cumulative probability results are very conservative.

The current model has simulated the high turbidity return water (100 NTU) discharged at a very shallow depth (~0.5 m depth) onshore. Model results show very limited turbidity impact from the discharge of decant water (from the sediment settling pond) primarily in winter season when the water become more stagnant. The assumed turbidity was derived based on a conservative assumption of typical turbidity of decant water discharged from sediment settling pond. If the turbidity of final discharge is lower than 100 NTU, the actual environmental impact would be less than the predicted impact.

TABLE 5-2 MODELLED IMPACT ZONES FROM PROPOSED DREDGING

	Dredging Plume impact Zones
20% duration Bottom - Summer	
20% duration Surface - Summer	

<table>
<tr><th></th><th>Dredging Plume impact Zones</th></tr>
<tr><td>20% duration
Bottom - Winter</td><td>
</td></tr>
<tr><td>20% duration
Surface - Winter</td><td>
</td></tr>
</table>

	Dredging Plume impact Zones
28 days Running Average, Bottom - Summer	
28 days Running Average, Surface - Summer	

	Dredging Plume impact Zones
28 days Running Average, Bottom - Winter	
28 days Running Average, Surface - Winter	

5.5.3 Sediment Deposition Results

Sediment deposition thickness is another indicator of dredging potential impact to the surrounding environment. As shown in Table 5-3, for a distance over 500 m from the dredging pocket, the modelled deposition layer is less than 0.1 mm thick. The area with over 1 mm deposition is found within a 100 m buffer zone from the dredging pocket. Overall, the modelled impact is small (less than 0.1 mm) for most of the area around the dredging site. Bed levels within the dredging pocket will be reduced due to the material removed by the dredger.

Note that the model only simulates deposition of silt and clay. Coarser sediments, e.g., fine sands may also be suspended during the dredging process. These could be expected to increase the level of deposition in the close vicinity of the dredging pocket. An empirical estimate has found that these coarser materials are likely to settle well within a 100 m radius of the dredge site.

TABLE 5-3 MODELLED DEPOSITION THICKNESS FROM PROPOSED DREDGING

	Sediment deposition thickness (mm)
Summer	

	Sediment deposition thickness (mm)
Winter	

5.5.4 Duration of Plume

The modelled turbidity plume was generated during the dredging operation and started to reduce within hours after the completion of dredging and took up to 3 days to settle under calm weather. Therefore, as per the dredging plan, the plume will not last for more than 17 days (including 14 days dredging time). The potential impact to the environment is therefore transient and much less than dredging projects with much longer durations (often over 4 weeks).

5.5.5 Maintenance Dredging

The base of the berthing pocket is expected to consist of stiff sandy clay. This will effectively form a hard base that will resist erosion under the effects of waves, currents and propellor wash from the transhipment barge. By comparison, the natural surface of the surrounding seabed consists of soft clayey silt that can be more readily put into suspension.

A series of water samples were taken from near the location for the end of the jetty at roughly monthly intervals from November 2018 through to February 2020. Samples were taken from the sea surface and near the seabed. These samples had a mean suspended sediment concentration (TSS) of just under 10 mg/l. The maximum measured TSS was 32 mg/l, and the 80^{th} percentile was 16 mg/l. These suspended sediments would be expected to consist of fine silt and clay particles consistent with the surrounding seabed. If sediment at these concentrations was allowed to settle continuously in the berthing pocket, it would be expected to result in sediment build-up on the bed of the berthing pocket of just under 1 cm per year. In the absence of other factors, and allowing for the effects of consolidation, this could be expected to result in net sedimentation of only around 0.3m over the 50 year life of the facility.

Further, the transhipment barge is expected to carry out an average of 318 transhipment operations per year. Each barge transhipment cycle is expected to take an average of approximately 21 hours and will involve two manoeuvring operations within the berthing pocket: one for departure when laden, and one for mooring when in ballast. It is expected that propellor wash from these manoeuvring operations would help keep the berthing pocket clear of any unconsolidated fine sediment and empty what may have settled out between each mooring operation.

Overall, it is unlikely that maintenance dredging will be required over the operational life of the facility. If, as a result of tropical cyclones or some other unforeseen circumstance, the rate of sedimentation is higher than expected, then the total volume of material to be removed by maintenance dredging will be significantly less than that for the original capital works dredging, considered above. Similarly, any environmental effects would correspondingly be significantly less.

5.6 Summary and Discussion

The potential impacts of dredging of the berthing pocket have been investigated by modelling the generation and movement of the resulting turbidity plume, on the basis of information provided by K+S. This has included dredge pocket geometry and geotechnical information. Dredging was assumed to have been carried out using a small cutter suction dredge.

The predicted sediment plume dispersion, based on conservative modelling assumptions, was found to be localised and of short duration. This was as expected, given the rather low production rate from a small cutter suction dredge, the relatively small volume of material to be removed (17,000 m^3) and the proposed onshore disposal and settling method. Further, the background turbidity levels of the existing environment regularly exceed 10 NTU at ebb tide and occasionally exceed 50 NTU in bad weather (*Marine, Coastal and Surface Water Data Collection*, Water Technology 2021). This background turbidity is significantly higher than the derived turbidity criteria for the definition of impact zones referred to in Table 5-1.

Additionally, the model results are conservative as the dredging will only be undertaken over a period of 14 days and the model simulated dredging over a full month. The modelled impact is only temporary over the duration of the dredging operation. Once the dredging operation completed, the dredging plume will likely disappear within a few days after the project completion. As such, the predicted turbidity increases from the dredging program should have no material impacts to the background turbidity of this region in the long term. The predicted overall impacts from the dredging program are low in terms of both impact area and duration.

It is unlikely that maintenance dredging will be required over the 50 year operational life of the facility.

6 REFERENCES

Alizad K, Hagen SC, Medeiros SC, Bilskie MV, Morris JT, Balthis L & Buckel CA. 2018, ‘Dynamic responses and implications to coastal wetlands and the surrounding regions under sea level rise’, *PloS One*, vol. 13, no. 10, e0205176

ANZECC & ARMCANZ (2000) Australia and New Zealand Guidelines for Fresh and Marine Water Quality (Australia and New Zealand Environment and Conservation Council & Agriculture and Resource management Council of Australia and New Zealand (ARMCANZ).

ANZG 2019. Australian and New Zealand Guidelines for Fresh and Marine Water Quality .

Bureau of Meteorology 1999, Tropical Cyclone Vance report, (http://www.bom.gov.au/cyclone/history/pdf/vance.pdf).

Chaojiao Sun, K. S. 2016, *Numerical modelling of dredge plumes: A review*. Perth: WAMSI.

CEDERWALL, K. 1968 Hydraulics of marine waste disposal. Chalmers Institute of Technology, Hydraulics Division, Report no. 42, Göteborg, Sweden

Department of Environment 2006, Pilbara Coastal Water Quality Consultation Outcomes – Environmental Values and Environmental Quality Objectives, Marine Series Report No.1, Department of Environment (DoE), Government of Western Australia.

Department of Environment 2006, Pilbara Coastal Water Quality Consultation Outcomes – Environmental Values and Environmental Quality Objectives, Marine Series Report No.1, Department of Environment (DoE), Government of Western Australia.

Environmental Protection Authority 2016, Environmental Factor Guideline: Statement of Environmental Principles, Factors and Objectives, EPA, Western Australia

EPA 2016. Environmental Assessment Guideline for Protecting the Quality of WA Marine Environment

Environmental Protection Authority 2016, Environmental Factor Guideline: Statement of Environmental Principles, Factors and Objectives, EPA, Western Australia

Environmental Protection Authority 2016, Environmental Factor Guideline: Marine Environmental Quality, Office of the Environmental Protection Authority, Western Australia, Perth

Environmental Protection Authority 2016, Technical Guidance - Environmental Impact Assessment of Marine Dredging Proposals, Office of the Environmental Authority, Western Australia, Perth

Fisher, R. J.-B. 2019. *Effects of dredging and dredging related activities on water quality: Impacts on coral mortality and threshold development*. Perth: WAMSI.

Flather, R. A. (1976), A tidal model of the northwest European continental shelf, Mem. Soc. R. Sci. Liege, 10(6), 141–164

GHD 2020, Borehole Log. Report prepared for K+S.

Harper B A 1997, *Numerical Modelling of Extreme Tropical Cyclone Winds*, Proceedings from the 4th Asia Pacific Symposium on Wind Engineering, Gold Coast, July.

Hansen, SR and Associates 1994, *Cargill Napa Disposal Evaluation Study*. Prepared for Cargill Salt by S.R. Hansen and Associates, January 1994.

Leuven JR, Pierik HJ, van der Vegt M, Bouma TJ & Kleinhans MG. 2019, 'Sea-level-rise-induced threats depend on the size of tide-influenced estuaries worldwide', *Nature Climate Change*, vol. 9, no. 12, pp. 986-992.

P. Palomar a, J. L. 2012, Near field brine discharge modelling part 1: Analysis of commercial tools. *Desalination*, pp. 14-27.

Parsons Brinkerhoff 2006, *Straits Ecotoxicity Project: Direct Toxicity Assessment of Bitterns*. Report by Parsons Brinkerhoff for Straits Salt Ltd. February 2006. Ref.: 2140169B_PR2_1479C.

Roberts, P.J.W., & Toms, G. 1987. Inclined Dense Jets in Flowing Current. Journal of Hydraulic Engineering, 113(3), 323–341.

Roberts, P.J.W., & Sternau, R. 1997. Mixing zone analysis for coastal wastewater discharge. Journal of Environmental Engineering, 123(12), 1244–1250

Rossington K, Whitehouse RJS & Spearman J. 2009, 'Morphological modelling of intertidal profiles in estuaries with strong tidal currents', *Rivers, Coastal and Estuarine Morphodynamics*, pp. 941-946.

RPS Australia West Pty Ltd (2017). *Ashburton Salt Project – Exmouth Gulf Region Bathymetric Data and Tide Levels*, Technical Note No: 100-CN-TNT-0830.Rev0, report prepared for K+S Salt Australia

Setinel 2020. USGS EROS Archive - Sentinel-2 <https://www.usgs.gov/centers/eros/science/usgs-eros-archive-sentinel-2?qt-science_center_objects=0#qt-science_center_objects>

Suranjana Saha; Shrinivas Moorthi; Xingren Wu; Jiande Wang; Sudhir Nadiga; Patrick Tripp; David Behringer; Yu-Tai Hou; Hui-ya Chuang; Mark Iredell Michael Ek; Jesse Meng; Rongqian Yang; Malaquías Peña Mendez; Huug van den Dool; Qin Zhang; Wanqiu Wang; Mingyue Chen; Emily Becker 2014, 'The NCEP Climate Forecast System', *Version 2: J. Climate*, vol. 27, no. 6, pp. 2185–2208.

Saha, S., et al. (2010) The NCEP Climate Forecast System Reanalysis. Bulletin of American Meteorological Society, 91, 1015-1057.

Seashore Engineering 2021, *Ashburton Salt Response to Sea Level Rise*. Perth.

Van Rijn, L.C., 1990, 'Sediment Transport in combined Currents and Waves', *Euromech 262*, United Kingdom, Wallingford.

Van Dam, Trenfield, Streten, Hardfard, Parry and van Dam. 2018. Water quality guideline values for aluminium, gallium, molybdenum in marine environments. s.l.: Environmental Science and Pollution Research25, 26592 - 26602, 2018.

Water Technology 2021. Suite of Reports Prepared for the Ashburton Salt Project. Reports prepared for K+S:

- *Marine, Coastal and Surface Water Data Collection*
- *Marine, Coastal and Surface Water Existing Environment*
- *Surface Water Assessment and Modelling*
- *Nutrient Pathways Assessment and Modelling*
- *Marine and Coastal Assessment and Modelling.*

Willmott, C. J. 1981, 'On the validation of models', *Physical Geography*, vol. 2, no. 2, pp. 184–194.

Wenziker, McAlpine, Apte and Mansini. 2006. Background quality for coastal marine waters of the North West Shelf, WA. s.l.: North West Shelf Joint Environmental Management Study - Technical Report, 2006

Western Australian Planning Commission (WAPC) 2013, *State Planning Policy No. 2.6 – State Coastal Planning Policy*. Government of Western Australia, Perth.

Western Australian Planning Commission 2017, *WA Coastal Zone Strategy*, Government of Western Australia, Perth.

YSI Incorporated 2017, *EXO User Manual: Advanced Water Quality Monitoring Platform*. Ohio: YSI Incorporated.

Zeitoun, M.A. 1970. Conceptual Designs of Outfall Systems for Desalting Plants. Tech. rept.

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