Port Hedland Coastal Vulnerability Study
Final Report

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DISCLAIMER

The State Government and the Town of Port Hedland have a shared vision that will see Port Hedland transformed into Pilbara's Port City with a population of 50,000 by 2035.

Land availability, and in particular developable land above predicted future flood levels, is a clear constraint. In recognition of this the Royalties of Regions program has funded the preparation of the Port Hedland Coast Vulnerability Study. The Study considers the combined impact of flood from rainfall and storm surge. The Study was undertaken by Cardno Pty Ltd and was overseen by a Steering Committee that included the Department of Water, the Department of Transport and the Department of Planning.

The report is not a statutory document. However, the reports findings will be considered by statutory agencies in assessing future development proposals under the Town of Port Hedland's Town Planning Scheme and the WAPC's Statement of Planning Policy 2.6.

Developers and their consultants may use the report and its data for their own works but must undertake their own independent verification specific to their development proposal.

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ABBREVIATIONS AND GLOSSARY:

AHD	Australian Height Datum which is the standard vertical elevation datum for Australia. At Port Hedland, AHD is +3.902m above Chart Datum at the permanent tide gauge.
ARI	Average Recurrence Interval; relates to the probability of occurrence of a design event.
BoM	Bureau of Meteorology
Coastal Inundation	Flooding of coastal land due to inundation by ocean waters.
CPS	Coastal Processes Setback. As Defined in SPP 2.6.
Cross-shore Transport	Sediment transport occurring normal (or perpendicular) to the beach face.
CSIRO	Commonwealth Scientific and Industrial Research Organisation
CVS	Coastal Vulnerability Study
DCC	Australian Department of Climate Change
DEM	Digital Elevation Model
DoP	Department of Planning
DoT	Department of Transport Coastal Management
Erosion	Short-term erosion, typically associated with a specific storm event. May be referred to as storm bite. The beach will typically recover after an erosion event.
GIS	Geographical Information System
HAT	Highest Astronomical Tide
HSD	Horizontal Setback Datum
H_b	Breaking wave height.
H_{max}	Maximum wave height in a specified time period.
H_{mo}	Significant wave height (H_s) based on where is the zeroth moment of the wave energy spectrum (rather than the time domain $H_{1/3}$ parameter).
hPa	hecta-Pascal
H_s	Significant wave height is the average wave height of the highest third of a set of waves.
LAT	Low Astronomical Tide

LiDAR	Light Detection and Ranging
Longshore Transport	The movement of sand along the coastline caused by waves and a wave-caused current running parallel to the beach.
MHWM	Mean High Water Mark
MHWN	Mean High Water Neap
MHWS	Mean High Water Springs
MLWN	Mean Low Water Neap
MLWS	Mean Low Water Springs
MSL	Mean Sea Level
Shoreline Recession	The long-term (decadal plus) net landward movement of the shoreline/mean water line. Occasionally referred to as long-term erosion.
SPP 2.6	Refers to Statement of Planning Policy No. 2.6 which is referred to as the State Coastal Planning Policy which was Gazetted in June 2003.
SLR	Sea Level Rise
SLSC	Surf Life Saving Club
Storm surge	Elevation in water levels along the coastline caused by wind set-up and the inverse barometer effect.
T_p	Wave energy spectral peak period; that is, the wave period related to the highest ordinate in the wave energy spectrum.
T_z	Average zero crossing period based on upward zero crossings of the still water line. An alternative definition is based on the zeroth an second spectral moments.
ToPH	Town of Port Hedland
TSWL	Total Still Water Level - peak total water level including astronomical tide and the water level residual as a result oceanographic processes. In Western Australia where applicable, for example on the open coast, the TSWL normally includes shoreline wave set-up (see below).
USACE	United State Army Corps of Engineers
Water Level Residual	Water level difference between observed water level and the predicted (astronomical) water level.
Wave Height	The height between the top of the crest and the bottom of the trough.
Wave Length	The distance between two wave crests.
Wave Period	The time it takes for two successive wave crests to pass a given point.
Wave Run-up	The vertical distance between the maximum height that a wave runs up the beach (or a coastal structure) and the still water level, comprising tide and storm surge.
Wave Set-up	Wave set-up in included implicitly in wave run-up calculations.
WAPC	Western Australian Planning Commission
WRB	Wave Rider Buoy

EXECUTIVE SUMMARY

Cardno was commissioned by Landcorp to undertake a *Coastal Vulnerability Study* for the Port Hedland region to inform future planning and development decisions in the region. Port Hedland is a strategic regional centre in Western Australia and over the coming 15 years the population is forecast to increase from 19,500 to 40,000. The *Coastal Vulnerability Study* will be a critical study to identify development opportunities and constraints for the Port Hedland region to meet the infrastructure requirements as the population doubles over the coming 15 years. The *Coastal Vulnerability Study* provides inputs to a range of government departments including Landcorp, Department of Planning, Departments of Transport, Department of Water and the Town of Port Hedland.

The Port Hedland Coastal Vulnerability Study involves four inter-linked study components:-

1. Hydrodynamic ocean modelling to assess extreme water levels (cyclonic) at the Port Hedland study sites for nominated return periods - **see Section 6**.
2. Hydrologic assessment of the extreme rainfall conditions including hydraulic modelling to determine the design flows of rivers and watercourses within the study area for nominated return periods - **see Section 7**.
3. Hydraulic modelling of the combined effects of storm surge inundation from the sea and flooding from the land through a coupled hydrodynamic and coastal model system - **see Section 8**. This component has also required an assessment of the joint probability of ocean inundation and catchment flooding, including likelihood and phasing.
4. A shoreline stability assessment of the study area detailing response to short term events (storms) and reporting the long term trend of shoreline movement – recession or progradation - see **Section 9**. Based on this assessment, hazard lines (erosion and inundation) for future planning purposes are to be determined in the study area with appropriate allowance for sea level rise and climate change scenarios. The shoreline stability assessment has included analysis of photogrammetric survey data since 1948 along the shoreline surrounding Port Hedland. For this study, no geological site investigations have been undertaken for the assessment of setback components of State Planning Policy 2.6.

Each study component is detailed in technical appendices which outline the study approach, model calibration and outcomes from that phase.

Cardno has developed a comprehensive model system which can simulate wind, atmospheric pressure, tide, rainfall and overland flow to investigate ocean inundation and catchment flooding. The study has considered the impact of jointly occurring elevated ocean water levels and catchment flooding.

The study area is broad and covers the Port Hedland, Wedgefield and South Hedland region, together with two sites to the west of Port Hedland – Sites 1 and 2, and also the Shellborough site approximately 85km northeast of Port Hedland. The study area covers a range of land use including mining and industrial land use, together with residential and commercial development areas. A particular focus of the study area is to inform the design of two potential development areas at Port Hedland; East Port Hedland and the Spoil Bank Marina.

The outcomes of the study indicate that the Port Hedland region is vulnerable to tropical cyclone impacts from ocean and catchment flooding. The recent Pretty Pool development is outside of the inundation and coastal erosion hazard risk area. Several key infrastructure elements are at particular risk to damage or loss of function during cyclone events. At Port Hedland, a low elevation section along the main foreshore road provides a channel which directs open coast inundation towards the Port Hedland town centre and East Port Hedland. The present wastewater treatment plant is at particular risk and a severe cyclone could impact on operation of this plant for a considerable period of time following the cyclone. In South Hedland, the wastewater treatment plant may also be at risk of inundation from 100 and 500 year ARI events. Several key roads are also at risk from inundation during 100 and 500 year ARI events. The Shellborough and Site 2 locations are prone to extensive ocean inundation. At Shellborough, there are limited sites which are suitable for development.

Cardno have considered the potential impact of future development in the Port Hedland region on flood levels. Two sites have initially been considered - East Port Hedland and the Spoil Bank. For the present landform, along the coastline of the township, the 500-year ARI Ocean storm event for the present climate condition, and the 100-and-500 year ARI events for the 2110 climate condition, breach the dune system near Stevens Street on the open coast which can potentially flood levels at the East Port Hedland site to the east of the breach location. As a result of the infill development, the Pretty Pool estuary which previously provided a large storage area for water which came through the breach location near Stevens Street, no longer provides the same the storage capacity. Because of this the flood levels at the western edge of the East Port Hedland development increase. In the concept design for this site, the potential for ocean inundation from the open coast through the low-lying region near Stevens Street will need to be addressed.

Table I.1 presents a summary of the recommend design water levels for potential developments near the Spoil Bank and at East Port Hedland. The recommended design water levels are based on a 2110 planning period. If infill development is being considered, it is recommended that general fill levels be based on the acceptable risk level design criteria for a 2110 planning period. Based on the uncertainty in the modelling and in estimating long return period design levels, Cardno would generally advise that floor levels in any fill development be specified at least 0.5m above the required design water level. For the Spoil Bank region which has wave setup included in the design water levels in **Table I.1**, the potential inundation as a result of wave run-up and overtopping will also need to be considered when determining the crest level for any shoreline structures.

Table I.1: Summary of Design Peak Total Still Water Level (TSWL) for East Port Hedland and Spoil Bank Developments - Selected ARI's for 2110 climate scenario.

ARI (years)	Design Peak Total Still Water Level (mAHD)	
	East Port Hedland	Spoil Bank Area
2	4.4	4.4
10	4.9	5.7
20	5.0	5.8
50	5.3	6.1
100	5.9	6.8
200	6.0	7.0
500	6.6	7.8

1 INTRODUCTION

This report has been prepared by Cardno following engagement by LandCorp to undertake a *Coastal Vulnerability Study* to evaluate the combined effects of coastal inundation (flooding and storm surge) arising from cyclonic events for the Town of Port Hedland and the surrounding area, and to also assess shoreline stability over planning periods of up to 100-years (Year 2110).

Port Hedland is a strategic regional centre in Western Australia and over the coming 15 years the population is forecast to increase from 19,500 to 40,000. The *Coastal Vulnerability Study* will be a critical study to identify development opportunities and constraints for Port Hedland to meet the infrastructure requirements as the population doubles over the coming 15 years. The *Coastal Vulnerability Study* will provide inputs to a range of government departments including Landcorp, Department of Planning, Departments of Transport, Department of Water and the Town of Port Hedland.

The study area for the *Coastal Vulnerability Study* covers Port Hedland (town), a site 15km west of Port Hedland and Shellborough which is 85km northeast of Port Hedland. The study area also extends inland to cover major centres such as Wedgefield and South Hedland. **Figure 1.1** presents a locality plan of the study area. **Figures 1.2** and **1.3** present detailed plan views of the Port Hedland and Shellborough sites.

***Figure 1.1: Locality Plan** (Image Source: GoogleEarth).*

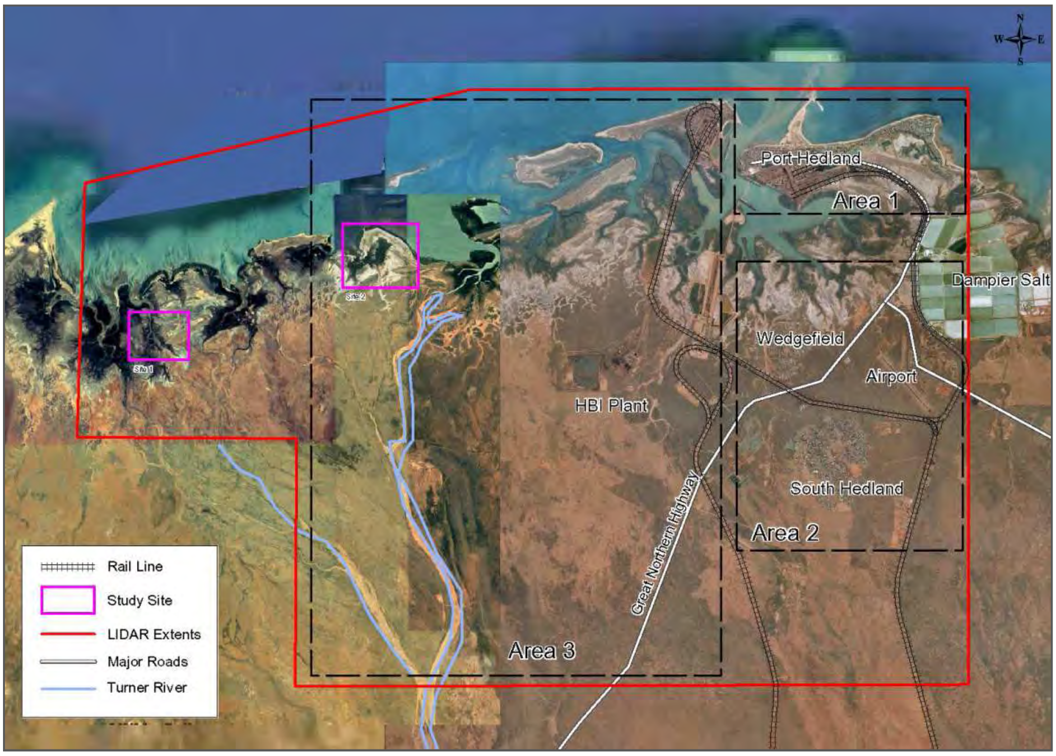

***Figure 1.2: Port Hedland Study Area** (Image Source: GoogleEarth).*

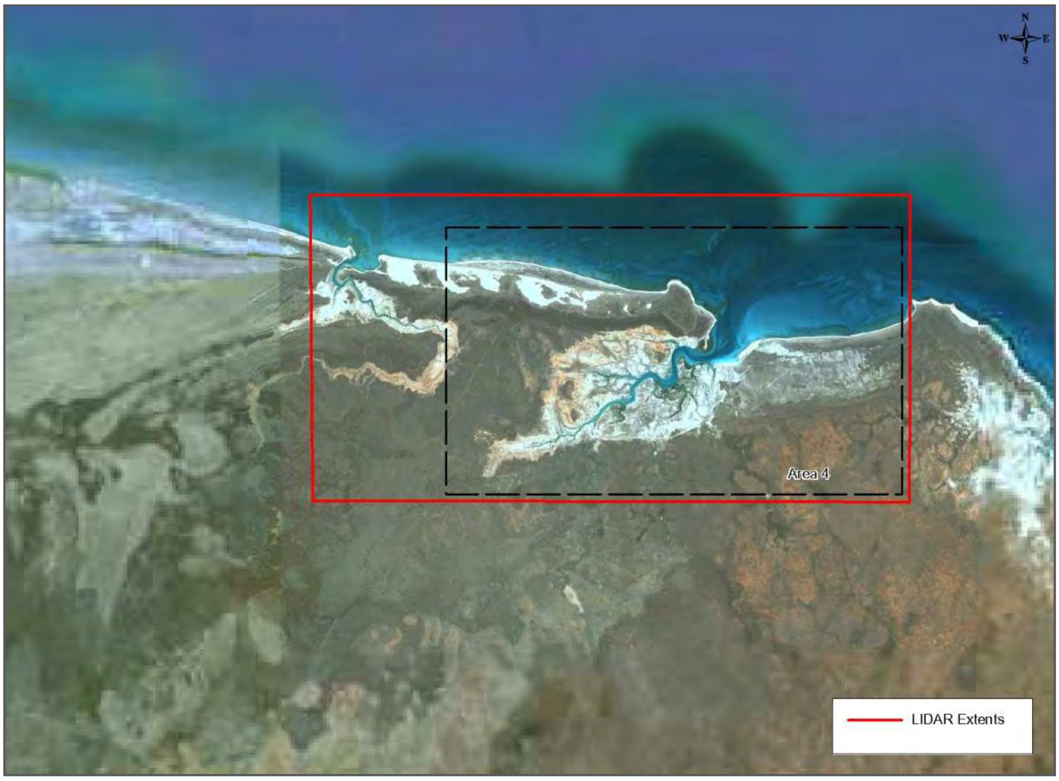

***Figure 1.3: Shellborough Study Area** (Image Source: GoogleEarth).*

Coastal inundation presents a significant constraint for the future expansion and development of Port Hedland as a Regional City. In order to facilitate the required city wide growth plan, it is important to be informed and prepared in the event of coastal flooding due to seasonal cyclonic heavy rainfall and associated storm surge. There is also an indication that predicted climate change in the Pilbara will result in increased intensity of cyclones and a rise in mean sea level. Depending on the geological nature of the area, gradual erosion of the shoreline over decades also needs to be considered. The impact of these predictions needs to be evaluated as to its consequences for the future development and expansion of Port Hedland. The projected shoreline erosion and coastal inundation level and its sensitivity to climate change scenarios over the next century are required to make informed decisions when allocating set back distances, assigning infrastructure corridors, preparing emergency response plan and improving the land value in any future development of Port Hedland.

The main industries in the region are iron ore processing and export, salt production from extensive evaporation ponds for export, shipping of manganese and other minerals and livestock production (mainly cattle). The Port is already one of the world's largest in tonnage terms, with over 100 million tonne of product worth more than $3 billion shipped each year and major expansions anticipated in the near future including a new $155m multi user Panamax berth at Utah Point. The two major business enterprises within the town are BHP Billiton, which operates a significant iron ore processing and shipping facility exporting iron ore and Dampier Salt, which produces over 3 million tonnes of industrial salt annually for export from solar salt ponds. Other minerals including manganese, copper and tantalum are also exported by various companies. Over the coming years, growth in iron ore exports from BHP Billiton, FMG and other smaller operators will be the primary driver to development in the region.

1.1 Data Conventions

In this report normal direction conventions have been adopted, that is:-

- Winds and waves – coming from; and
- Currents – flowing towards.

All directions are relative to True North (^{o}T). Time information is generally presented in Western Standard Time (WST, +8-hours UTC).

Vertical datum is normally Australian Height Datum (AHD). At Port Hedland, AHD is +3.902m above Lowest Astronomical Tide (LAT).

2 STUDY BRIEF

For the purposes of this study, the key outcome is the determination of the inundation levels and depths associated with cyclonic events in the Town of Port Hedland. The overall scope had a number of requirements including:-

1. Hydrodynamic modelling to assess extreme water levels (cyclonic) at the Port Hedland study sites for nominated return periods;
2. Hydrologic assessment of the extreme rainfall conditions including hydraulic modelling to determine the design flows of rivers and watercourses within the study area for nominated return periods;
3. Hydraulic modelling of the combined effects of storm surge inundation from the sea and flooding from the land through a coupled hydrodynamic and coastal model system. This component has also required an assessment of the joint probability of ocean inundation and catchment flooding, including likelihood and phasing.
4. A shoreline stability assessment of the study area detailing response to short term events (storms) and reporting the long term trend of shoreline movement – recession or progradation. Based on this assessment, hazard lines (erosion and inundation) for future planning purposes are to be determined in the study area with appropriate allowance for sea level rise and climate change scenarios. The shoreline stability assessment has included analysis of photogrammetric survey data since 1948 along the shoreline surrounding Port Hedland. For this study, no geological site investigations have been undertaken for the assessment of setback components of State Planning Policy 2.6.
5. Provision of detailed GIS datasets showing the combined impacts of storm surge and flooding by rainfall in the study area for nominated return periods.

The *Coastal Vulnerability Study* specifically investigates the inundation and the shoreline vulnerability for key locations within the study area which are:-

- Site 1 – Site 23 km west of Port Hedland.
- Site 2 – Site 15km west of Port Hedland and a potential future camp ground.
- South Hedland – Site 10km southwest of Port Hedland and a major population centre.
- Port Hedland – Key regional town which includes the Port Hedland Port.
- Shellborough – Site 85km northeast of Port Hedland and which is a potential future development area.

Figures 1.2 and **1.3** present a plan view of the detailed study sites.

The numerical ocean inundation and flood modelling undertaken for the Port Hedland *Coastal Vulnerability Study* has not been undertaken down to the local drainage scale. In the key flood prone area of South Hedland, the general drainage system is defined within the model system, however the main focus of this study was to identify flood extents and flows on a larger scale for the whole of the Port Hedland region. Future development and detailed assessment of potential flood levels on a lot-by-lot basis will require a local drainage study.

3 STUDY SITE DESCRIPTION

3.1 General

The purpose of this section is to describe the physical processes and environment of the Port Hedland region as these determine the overall conditions which affect the coastal vulnerability at the study sites. These processes are: -

- Oceanographic Environment;
- Meteorology and Climatology;
- Geomorphic Setting; and
- Historical Flooding and Cyclone Impacts.

3.2 Oceanographic Environment

The Northwest (NW) region is part of the Indo-Australian Basin, the ocean region between the Northwest (NW) coast of Australia and the Indonesian islands of Java and Sumatra. Dominant currents in the region include: the South Equatorial Current, the Indonesian Through-Flow (ITF); the Eastern Gyral Current, and the Leeuwin Current. **Figure 3.1** depicts the main surface currents of the region (from DEWHA, 2007). All of these current systems experience strong seasonal to interannual variations, which indicates that they are likely to be influenced by climate change over the coming decades.

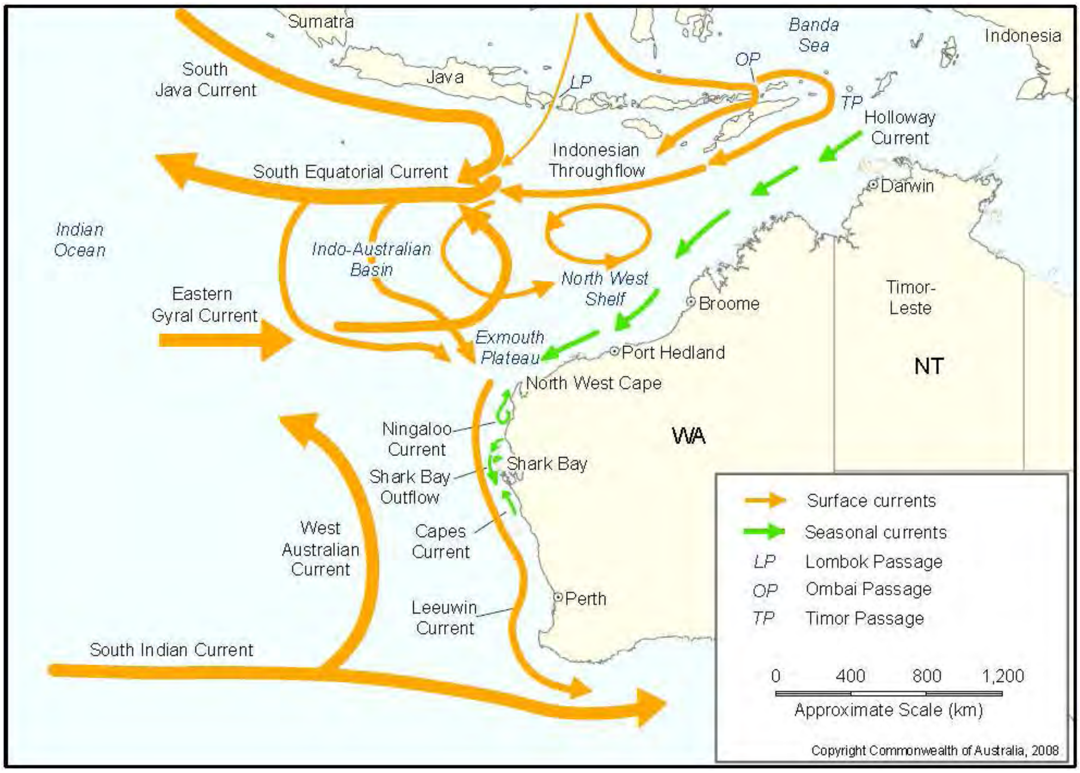

Figure 3.1 Regional Oceanography and Currents (DEWHA 2007).

The ITF is generated by the prevailing wind-fields over the Pacific Ocean, primarily the Trade Winds, which pile up water on the Western (W) side of the ocean creating a pressure gradient from the Pacific toward the Indian Ocean. While its net mass transport is moderate (10Sv), the current transports a significant amount of heat. Recirculation of ITF waters into the offshore Pilbara region largely contributes to surface flows off the shelf break, the slope and over the abyss. The origin and movement of shelf waters is not well understood, but it is believed that ITF waters flood the shelf via the offshore pathway (the so-called Eastern Gyral Current) and the Holloway Current. It is also likely that local eddies and internal tides affect cross-shelf transport and modify water properties through vertical mixing.

The Leeuwin Current (LC) is generated by the formation of a high pressure ridge in the upper ocean and the associated Eastward current (E) flowing toward Northwest (NW) Australia. The current turns Southward (S) approaching the coast and flows poleward along the whole length of the Western Australia coastline past Cape Leeuwin. The Leeuwin current is highly variable on seasonal and interannual timescales (Feng *et al* 2003, 2008). The interannual variation of the LC and its eddy field correlate to the El Niňo/Southern Oscillation (ENSO) phenomena, which is generally stronger during La Niňa years and weaker during El Niňo conditions.

The Holloway Current (HC) is a surface layer, poleward flowing ocean current that brings water, perhaps from as far north as the Band and Arafura seas, Southward (S) over the continental shelf of Northwest (NW) Australia at the end of the Northwest (NW) monsoon (D'Adamo *et al* 2009). A simple view of the generating mechanism is that the seasonal Southwesterly (SW) wind piles up water in the Arafura Sea and Gulf of Carpentaria during the peak monsoon and the current flows Southward (S) as the wind relaxes during the monsoon transition. Detailed analysis and modelling indicates that the impact of seasonal heating on sea level adds to the overall force balance to generate the full strength of the current.

Generally, there is no direct flow from the Timor Strait into the Leeuwin current - rather water recirculates Eastward (E) via the Eastern Gyral Current into the Northwest (NW) Marine Region. Their continued poleward flow is the dominant pathway by which Equatorial waters form the Leeuwin Current. However, on a seasonal basis, the Holloway Current does produce a direct pathway along the shelf, and could be partly responsible for the autumn surge in strength of the Leeuwin Current.

In the Southern (S) part of the region, shelf waters inshore of the Leeuwin Current are cooler and more saline. In summer these waters are driven northward by strong Southerly (S) winds giving rise to the seasonal Capes Current which, although extending into the Northwest (NW) Marine Region, originates in the Southwest (SW) region.

3.2.1 Water Levels

The coastal environment surrounding Port Hedland is subject to large, semi-diurnal tide forcing which has a major influence on water levels. At Port Hedland the tides are semi-diurnal with a spring neap cycle occurring over a two week (approximate) period. During neap periods a tidal range of approximately 2m is observed while during spring periods a range of up to 7.5m may occur. **Table 3.1** presents a summary of the published tidal plane for Port Hedland, and also selected tidal levels for Port Hedland and Shellborough based on the hydrodynamic model presented in **Section 6** and **Appendix A.** Note, there is no measured tidal data was available for the Shellborough region however **Table 3.1** indicates that the hydrodynamic model reproduces the typical tidal range at Port Hedland well.

Table 3.1: Tidal Plane Summary - Port Hedland and Shellborough Site

Tidal Plane	Tidal Plane Summary – m AHD		
	Port Hedland – AHO (2009)*	Port Hedland – Delft3D Model*	Shellborough – Delft3D Model#
HAT	3.6	-	-
MHWS	2.8	2.7	3.4
MHWN	0.7	0.7	0.7
MLWN	-0.6	-0.6	-0.7
MLWS	-2.7	-2.7	-3.4
LAT	-3.9	-	-

* At Port Hedland, AHD is +3.901m above Chart Datum

Based on assumption that at Shellborough AHD is approximately near mean sea level.

Water level residuals can result from a range of processes, most significantly from low pressure, tropical storm or cyclone events. Water level variations in the sea and at the coastline result from one or more of the following natural causes:-

- Eustatic and Tectonic Changes;
- Tides;
- Wind Set-up and the Inverse Barometer Effect;
- Wave Set-up;
- Wave Run-up;
- Tsunami;
- Greenhouse Effect; and
- Global Changes in Meteorological Conditions.

Eustatic sea level changes are long term world-wide changes in sea level relative to the land mass and are generally caused by changes to the polar ice caps. It is now generally accepted that eustatic changes will contribute to a rise in sea level over the next century. Tectonic changes are caused by movement of the Earth's crust; they may be vertical and/or horizontal. In comparison to the projected magnitude of eustatic sea level rise, the impact from tectonic changes are negligible. Additionally, land movement changes such as local subsidence of poorly consolidated coastal sediments is occurring, however quantifying this is not possible using data currently available (advice provided from DoT).

Tides are caused by the relative motions of the Earth, Moon and Sun and their gravitational attractions. While the vertical tidal fluctuations are generated as a result of these forces, the distribution of land masses, bathymetric variation and the Coriolis force determine the local tidal characteristics.

The physical processes which are most relevant to the *Coastal Vulnerability Study* are storm surge, wind driven currents and cyclone waves. Wind set-up and the inverse barometer effect are caused by regional meteorological conditions. When the wind blows over an open body of water, drag forces develop between the air and the water surface. The result is that a wind drift current is generated. This current may transport water towards the coast upon which it piles up causing wind set-up. Wind set-up is inversely proportional to depth and proportional to the square of wind speed. Additionally, southward flowing coast parallel currents at Port Hedland will be affected by the Coriolis force and cause some rise in water level at the coastline.

In addition, the drop in atmospheric pressure, which accompanies severe meteorological events such as tropical cyclones, causes water to flow from high pressure areas on the periphery of the meteorological formation to the low pressure area. This is called the 'inverse barometer effect' and results in water level increases up to 1cm for each hecta-Pascal (hPa) drop in central pressure below the average sea level atmospheric pressure in the area for the particular time of year, typically about 1010 hPa. The actual increase depends on the translation speed of the meteorological system and 1cm is only achieved if it is moving slowly. The phenomenon causes daily variations from predicted tide levels up to 0.1m. The combined result of wind set-up and the inverse barometer effect is called "storm surge" and has been included in design water level assessment at this site. The Pilbara coast is in one of the most cyclone prone areas of Australia and can be subject to large storm surges. The storm surges along the Pilbara coast can be particularly large due to the broad continental shelf which enhances wind set-up and the intense nature of cyclones which occur in this area. To some extent the large tide range at Port Hedland mitigates the storm tide effect, which is the combined astronomical tide plus storm surge, as it is less likely that large storm surges will coincide with spring tide high water conditions.

In this report, two key terms are commonly adopted in relation to ocean inundation levels. The first of these is the *water level residual.* This term is used to refer to the difference between the measured water level and the predicted astronomical tide water level for the corresponding period. The water level residual can be a result of large scale, regional oceanographic processes and/or from local meteorological forcing for example during a tropical cyclone. Along the Pilbara coastline, the magnitude of the water level residual from localised cyclone forcing is normally the most significant in relation to long-term planning levels. In the context of Port Hedland, the term *water level residual* has been adopted instead of *storm surge* due to the relationship between the magnitude of observed water level residuals during cyclone events and the phase of the astronomical tide. **Appendix A** of this report demonstrates the influence of the astronomical tide on the magnitude of the water level residual at Port Hedland. For a given cyclone track and intensity, the magnitude of the water level residual at Port Hedland will vary according to the phase of the astronomical tide relative to the cyclone track. In general, higher water level residuals are observed at Port Hedland when the peak cyclone forcing, which directs water towards the coast, occurs near low water on the ebb tide. Conversely, smaller water level residuals are observed near high water on the flood tide.

The second key term adopted in this report is *Total Still Water Level (TSWL)* which refers to the total still water level as a result of *water level residual* and the astronomical tide. In the context of Port Hedland, the joint occurrence of astronomical tide and the water level residual is critical in determining the potential for ocean inundation as the tidal range is significantly larger than the potential of the *water level residual* which may develop when a severe cyclone passes close to Port Hedland.

Wave related variations in water level also need to be considered when assessing ocean inundation levels. Two key wave related processes which need to be considered are *wave set-up* and wave run-up. *Wave set-up* refers to the increase in water surface elevation shoreward of the wave breaking zone as a result of the momentum flux in the surf zone. For locations on the open coast, *wave set-up* can be a significant factor in ocean inundation extents. *Wave run-up* refers to the maximum vertical level that a wave propagates to and is based on the wave conditions and also the shoreline structure or feature where the run-up occurs. *Wave run-up* levels are particularly important when considering the vulnerability of features directly on the coastline which are exposed to wave action.

Figure 3.2 presents an illustration of the key processes in respect to ocean inundation during a cyclone event.

Figure 3.2: Comparison of Coastal Water Levels during Typical Conditions (left) and during a Tropical Cyclone (right) (BoM, 2011).

Tsunamis are caused by sudden crustal movements of the earth and are commonly, but incorrectly, called 'tidal waves'. They are very infrequent and unlikely to occur during a storm and so have not been included in this study. Nevertheless, in the context of events having recurrence intervals in the order of 50 years, one should keep this issue in mind.

Global meteorological and oceanographic changes cause medium term variations in mean sea level. The former phenomenon may persist for a year or more. The causes are not properly understood, but analyses of long term data from tide gauges indicate that annual mean sea level may vary by up to 0.1m from the long term trend. General scientific consensus indicates that global warming of the Earth's atmosphere will lead to a rise in mean sea level. Predictions of global sea level rise due to the Greenhouse effect vary considerably.

The Western Australian government has released a revised sea level rise policy following the IPCC 2007 report. Following the release of this document, the revised sea level rise recommendations in SPP 2.6 were updated for use in planning periods of 50 and 100 years to +0.3m by 2060, and +0.9m by 2110 (WAPC, 2010). The BoM currently has a long term sea monitoring programme at sites around in Australia including Broome. Data from this programme which commenced in 1991 indicates that in the northern tropics, inter-annual sea level variations are very large and potentially the sea level rise is greater than temperate areas of Australia. Since 1991, the net relative annual rate of sea level change has been +8.9mm/year (NTC-BoM, 2009). Net relative change refers to the change in sea level after subtraction of vertical movements in the observing platform and the inverted barometric pressure effect.

3.2.2 Waves

Port Hedland is located on the north-west shelf of Western Australia. Waves in the region are generated by three principal mechanisms - Indian Ocean swell, locally generated sea and the occasional cyclone system. As a result wave conditions at the Port Hedland site are multi-modal, both in frequency and direction.

There is a strong seasonal character to the offshore wave climate at Port Hedland. During the winter months (June – September) the more easterly winds prevail due to low pressure systems over the North Australian basin. An underlying swell during this period remains predominantly west – north-west episodically influencing wave conditions at the study site. During the summer months the same west – north-westerly swell remains however the dominant winds in the region are from the west – south-west and tending north-west at the study site due to the local seabreeze effect. This effect is generated by the contrasting thermal responses of the land and water surface (Masselink and Pattiaratchi, 2001) and are typically diurnal in nature although can be persistent, depending on the location. Summer also sees infrequent cyclone activity, averaging approximately 3 cyclones a year.

Wave run-up is the vertical distance between the maximum height a wave runs up the beach or a coastal structure and the still water level, comprising tide plus storm surge. Additionally, run-up level varies with surf-beat, which arises from wave grouping effects. Wave run-up is an important consideration when assessing potential coastal hazards for sites which may be located above the peak still water level under a storm tide scenario but may be subject to inundation or damage as a result of wave run-up.

3.2.3 Currents

Currents in the Port Hedland region are caused by a range of phenomena, including: -

- Astronomical Tides;
- Winds;
- Nearshore Wave Processes; and
- Estuarine Discharges.

The astronomical tides are caused by the relative motions of the Earth, Moon and Sun. The regular rise and fall of the tide level in the sea causes a periodic inflow (flood tide) and outflow (ebb tide) of oceanic water to/from the Northwest (NW) shelf region. A consequence of this process is the generation of tidal currents. Based on current data recorded at this site, flood tide currents flow South-southeast (SSE) in the offshore area and North-northwest (NNW) on ebb tide, approximately. Stronger currents will occur in tidal inlets such as the inner harbour region of Port Hedland because of the restricted entrance width compared with the overall inter-tidal storage (tidal prism) within the harbour.

Wind forcing is applied to the water surface as interfacial shear; the drag coefficient and consequent drag force varying with wind speed. Momentum from the wind is gradually transferred down through the water column by vorticity, the maximum depth of this effect being termed the Ekman depth. At the surface, wind caused currents are in the direction of the wind, but in the Southern Hemisphere they gradually turn to the left of the wind direction until they flow in the opposite direction at the Ekman depth. Much of the region of the Northwest (NW) shelf included in the model system set up for this study is too shallow for this condition to develop fully and wind driven currents are affected more by the seabed boundary layer and physiographic features. Wind driven currents diminish with depth. Because wind forcing is applied at the water surface, the relative effect of wind speed is greater in shallow water where there is less water column volume per unit plan area. Therefore wind driven currents are often greater in more shallow areas. Maximum surface current speed is in the order of 1% to 3% of the wind speed, depending on water depth. Where water is piled up against a coastline by wind forcing, a reverse flow develops near the seabed.

The propagation of waves into the near shore region leads to wave breaking and energy dissipation. Where waves propagate obliquely to the shoreline this process leads to the generation of a longshore current in the surf zone, and to some extent seaward of that line.

3.2.4 Winds

The climate in the Pilbara region is tropical with wet and dry seasons. During the wet season (October to April), winds are generally Easterly (E) to Southeasterly (SE) in the morning and Westerly (W) to Northwesterly (NW) in the afternoon. During the dry season (May to September) winds are generally variable and from the Easterly (E) to East-southeasterly (ESE) directions in the morning and Westerly (W) to Northerly (N) in the afternoon.

Winds at Port Hedland are affected by local processes such as the sea breeze effect. Sea breezes are winds from the sea that develop near the coast. They are formed by increasing temperature differences between the land and water that create a pressure minimum over the land due to its relative warmth and forces higher pressure, cooler air from the sea to move inland. As a result sea breezes tend to develop in the afternoon. Along the Northwest (NW) West Australian coastline and near Port Hedland sea breezes are generally from a Westerly (W) to Northwesterly (NW) direction. Investigations into sea breeze systems along the West Australian coast have found that mean annual wind speeds are significantly less than wind speeds associated with sea breeze periods (Masselink and Pattiaratchi, 2001). At Karratha (~200km west of Port Hedland) and Broome (~400km northeast of Port Hedland), this difference was found to be around 25% and 53%, respectively. Annual mean wind speeds of 5.6m/s and 2.8m/s and a mean wind speed during sea breeze periods of 7.0m/s and 4.3m/s, occur at Karratha and Broome, respectively.

Port Hedland is located within Australia's cyclone belt along a stretch of coastline that is the most cyclone-prone area in Australia. According the Bureau of Meteorology the Pilbara coast experiences more cyclones than any other part of Australia. Since 1910 there have been 49 cyclones that have caused gale-force winds at Port Hedland. On average this equates to about one every two years. Winds associated with cyclone events may exceed 100 knots (10-minutes average) during the cyclone season that extends from November to May each year. Port Hedland has been severely affected by several severe tropical cyclones in the last thirty years. Between three and five cyclones might typically approach the Pilbara coast during the cyclone season from November to May each year and winds over 100 knots can be experienced together with associated rough sea conditions and elevated water levels. During this season it is not unusual to experience winds in excess of 20 knots daily, with winds in excess of 40 knots during the season's tropical storms. Large offshore waves and a storm surge are always possible in conjunction with any cyclone. One of the most damaging was Cyclone Joan in December 1975. Maximum wind speeds in Port Hedland reached 208km/h and that condition occurred when the centre of the cyclone crossed some 50km West (W) of the town.

3.3 Geomorphic Setting

The general coastal morphology of the Port Hedland region is a limestone barrier system which is typified by the low coastal cliff and rock formations along the shoreline. In general, there is limited mobile sediments present at the shoreline however the development of the Spoil Bank from dredge spoil since the 1960's has provided a source for mobile sediments around the shorelines of Port Hedland.

The coastline along the Pilbara coastline are identified as a mixture of Pleistocene and more recent overlying Holocene formations (Semeniuk, 1996). The large tide range at Port Hedland combined with wave forcing provides a highly energetic environment near the shoreline which results in limited mobile sediments in the nearshore zone. Sandy beaches in the Port Hedland region are normally perched on rock platforms or are constrained by rock formations. As a result of the underlying rock along much of the coastline, the mobility and erosive potential is limited under the combined effects of waves and currents. However, depending on the geomorphology it is worth considering that in some cases the presence of underlying rock could result in increased coastal recession with long term sea level rise. Mobile sediments in the nearshore zone generally have an eastwards movement although this can vary seasonally. The predominantly eastwards sediment transport in the nearshore zone at Port Hedland is illustrated by the movement of the Spoil Bank since its formation.

The major rivers in the Port Hedland region including the Yule and Turner Rivers are inactive deltas and the creeks which flow into the inner harbour including South and South-West Creek have low sediment supply. The De Gray River has an extensive delta system and a high sediment load.

3.4 Historical Flooding and Cyclone Impacts

Major rivers in the region of Port Hedland are the De Grey River to the east, and the Turner River and Yule River to the west. These river systems are largely seasonal conveying large amounts of runoff with the tendency to flood in extreme rainfall events. Whilst road and rail infrastructure connecting the town is at risk when these river systems flood, the town of Port Hedland itself is not directly threatened. Major flooding of the river systems surrounding Port Hedland can occur from events other than tropical cyclones. Rainfall totals greater than 100 mm are common with tropical lows that move over land. In February 2003 a tropical low moved over Port Hedland causing rainfall in excess of 300 mm in some parts and flooding the Yule River

There are small creeks that convey localised flows from the south of Port Hedland through and into the Port Hedland Inner Harbour region and these represent the greatest threat to low lying areas in South Hedland and Wedgefield. Combined with this is the possibility of elevated water levels from the estuarine areas as a result of the cyclone induced storm tide. The combination of elevated ocean water levels inside Port Hedland occurring close to peak catchments flows can increase flood extents, particularly in the area between Wedgefield and South Hedland.

Limited information is available regarding flooding events in the town of Port Hedland. A recent study completed by GHD in September 2010 reported on the existing drainage systems in South Hedland. Inundation mapping for the 5 to 100 year ARI events were modelled for the South Hedland area, along with a detailed review of the drainage network. The report showed the area was susceptible to flooding in low ARI events such as the 5 year event and tabled recommendations and costing for drainage network upgrades, maintenance and improvements. The inundation extents for these events are shown on **Figure 3.3** with the dark blue colour indicating the 5 year event and the lighter blue representing the 100 year events.

Figure 3.3: Flooding Impacts in South Hedland for the 5 and 100 Year ARI Events (GHD 2010).

Storm surge poses the greatest threat to the Port Hedland Township, its severity is determined by three key factors: -

- The magnitude of the tropical cyclone event;
- The proximity of the cyclone to the Town of Port Hedland (distance and heading); and
- The timing of the tidal cycle at the point of the cyclone approaching the coastline

Tropical cyclones are rated by their wind strength on a scale of 1 to 5 - a Category 5 is a severe tropical cyclone with wind strength of more than 280km/h. In the southern hemisphere winds rotate in a clockwise direction around a low pressure centre.

Cyclone movements are unpredictable so whilst the systems can be monitored, there is a degree of uncertainty as to the path a system will follow. Cyclones approaching the Port Hedland coastline can make landfall, track coast parallel along the coast or move back out to sea. During cyclone season on average, five tropical cyclones will form off the northwest coast of Australia from Broome to Exmouth, and of these two will cross the coast (BoM).

For Port Hedland the position of a tropical cyclone as it approaches the town is important, as this determines the wind direction over the water at the coast. Because winds move clockwise around the low pressure centre, a cyclone which tracks on the eastern side of the town will result in a wind direction predominantly offshore. In this situation the full effects of a storm tide are avoided as offshore winds drive the water away from the coast. Conversely, a cyclone which tracks to the west of the town results in the most severe storm tide effect, as water is driven ashore with the onshore wind and associated wind setup.

The proximity of the cyclone to the town of Port Hedland is also important, with cyclones making landfall at a distance greater than 200km from Port Hedland posing limited risk of dangerous storm tide effects.

The timing of the cyclone as it approaches the coast in relation to the tide phase is the crucial determinant of the storm tide magnitude. A coast crossing cyclone coinciding with the high tide is the worst possible scenario. In this instance the elevated water level induced by the storm surge on top of the astronomical tide combine to create a storm tide above the level of HAT causing damage to both the natural system and built environment. With the large tidal range of Port Hedland, the likelihood that the peak of the storm surge will coincide with the high tide level is significantly reduced.

The most severe cyclones affecting Port Hedland in the past 70 years, with wind gusts above 170km/h as reported by the BoM were :-

- January 1939
- March 1942
- Joan, December 1975
- Leo, March 1977
- Dean, February 1980
- Connie, January 1987
- George, March 2007

The tracks of these cyclones are shown on **Figure 3.4** with Port Hedland in the centre of the image.

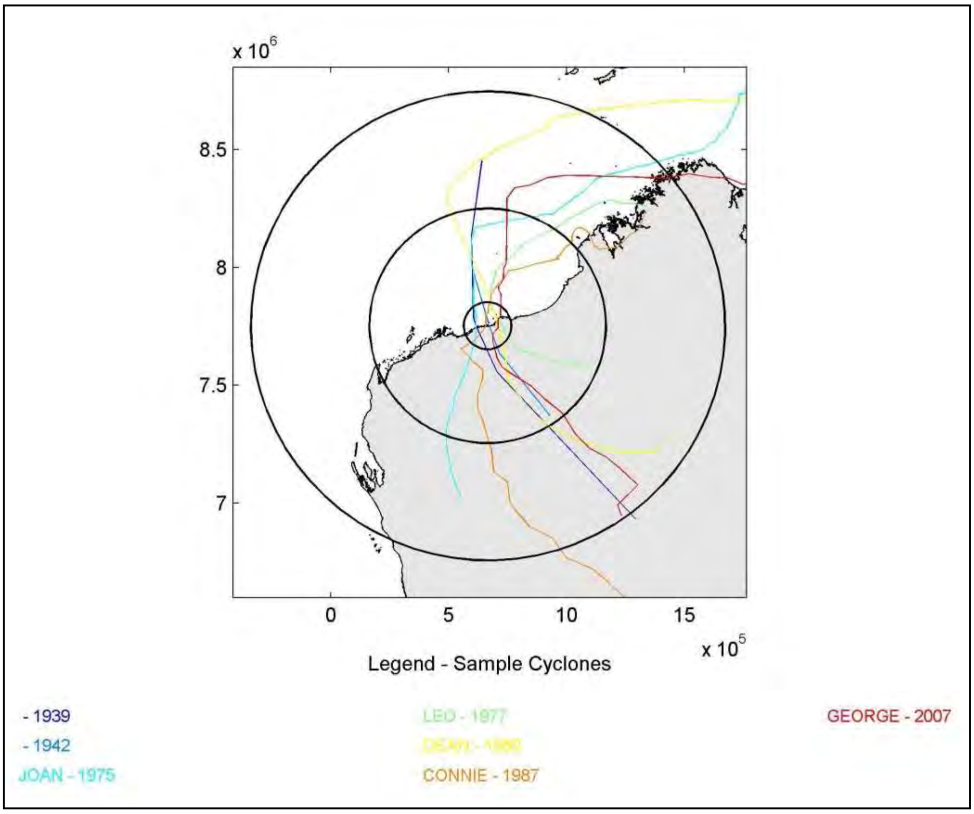

Figure 3.4: Plan view of the cyclone tracks from severe historical cyclone events.

The only one of these events which produced a significant storm tide level in Port Hedland was January 1939 where the storm surge coincided with high tide to cause an estimated storm tide level of 5.7m AHD flooding parts of the town at Airey Point as shown on **Figure 3.5**.

Figure 3.5: Storm Surge Port Hedland January 1939. GP Stockers Store Corner Anderson St and the Esplanade. (Port Hedland Historical Society).

It should be noted that the Port Hedland built environment around Airey Point in 1939 was significantly less developed than the present day and the flow regime in and around this area has since changed remarkably. It is probable that the recorded water level, which itself is uncertain, is likely to include a substantial contribution from shoreline wave setup due to the significant waves which are evident in **Figure 3.5**. For comparison purposes the 1949 and 2009 aerial photos of Airey Point are shown on **Figure 3.6** with the approximate field of vision for the photo shown in **Figure 3.5** indicated by the yellow arrows.

Figure 3.6: Aerial Photo Comparison Port Hedland Airey Point 1949 and 2009.

During the cyclone events of March 1942, Joan (1975), and Connie (1987) the town of Port Hedland was spared significant storm tide effects as these crossed the coast close to low tide resulting in a storm surge level less than HAT. Tropical cyclones Leo 1977, Dean 1980 and George 2007 passed to the east of town negating the magnitude of the storm surge. Some smaller cyclone events which resulted in storm tide impacts above HAT due to the timing of the storm surge with the high tide include March 1917 and TC Kerry (January 1973).

Cyclone Joan (8 December 1975) is rated by BOM as one of the most significant cyclones in WA's history. The cyclone crossed the coast 50km west of Port Hedland with a maximum recorded wind gust of 208km/h at Port Hedland Airport. The maximum storm surge was estimated as 3.25m, but the timing of the peak between the high and low tide was such that the resulting tide level stayed below HAT.

Eighty five percent of houses in Port Hedland were damaged to some degree. Along the ocean front all homes sustained some damage and many were unroofed, with sand piled up to depths of 1 to 2 metres. There was heavy rain inland and across the Hamersley Range with record rainfalls including the 596mm recorded at Marandoo over 60 hours. There was flooding along a number of streams flowing into the Fortescue River and serious flooding along the Yule River.

Severe Tropical Cyclone George, 8 March 2007, was the most destructive cyclone to affect the greater Port Hedland region since TC Joan. The cyclone crossed the coast close to the mouth of the De Grey River, approximately 40km east of Port Hedland with winds estimated at around 285km/h. Whilst there are no tide gauges in the area, modelling undertaken by the BoM estimated the cyclone crossed the coast shortly after low tide with a storm surge at the point of impact (near the Shellborough site) of approximately 4.8m. As a result the storm tide (storm surge plus astronomical tide) was below HAT. This estimation was backed up by reports from the shoreline adjacent the impact zone. The estimated wind gusts in Port Hedland as the cyclone crossed were 200 km/h, but storm surge was minimal as these winds were largely directed offshore due to the cyclones location to the east of the township.

4 STUDY METHODOLOGY

The Port Hedland Coastal vulnerability assessment comprises four main project elements which are:-

1. Setup, calibrate and undertake simulations with an ocean inundation model system which simulates wind, wave, tide and storm surge processes;
2. Setup, calibrate and undertake simulations with a hydrological modelling system to define catchment flows from design rainfall events;
3. Setup, calibrate and undertake simulations with a hydraulic modelling system which can simulate inundation from ocean and catchment processes; and
4. Undertake a shoreline stability assessment for key shoreline sites in the study area adopting a study methodology which is consistent with State Planning Policy 2.6 and considers a coastal hazard for the present condition, a 50-year planning period with a 0.3m Sea Level Rise (SLR) and a 100-year planning period with a 0.9m Sea Level Rise (SLR) in accordance with WAPC (2010).

The study components were undertaken concurrently, integrating the results across the studies. In accordance with advice from the Department of Transport, a risk-based methodology has been adopted which has involved simulation of a large number of discrete events to quantify the joint occurrence between astronomical tide and storm surge as it is this process that has the most significant influence on the ocean inundation for the Port Hedland region.

Currently two state planning policies, The State Coastal Planning Policy (SPP 2.6), and the Western Australian Natural Hazards and Disasters State Planning Policy (SPP 3.4), provide the policy framework for assessing ocean inundation and flooding in cyclone prone regions. In general, these two policy adopt a scenario based mechanism to assess inundation levels.

As part of ongoing revisions to SPP 2.6, the WA Department of Planning has been considering alternative assessment frameworks to assess inundation levels using a risk based assessment. Full risk assessment approaches are not commonly adopted in ocean inundation due to both the complexity of such studies and the difficulty in accurately quantifying the uncertainty associated with any investigation approach – whether numerical or empirical. The approach adopted in this study can be described as a safety standard based approach which has considered the potential for ocean and catchment inundation based on the simulation of a large number of discrete events, but the study has also adopted appropriate scenarios to define design rainfall conditions as well as future sea level, cyclone intensity and rainfall under climate change conditions.

5 DATA SOURCES

A range of data sets were utilised in this study and the key data sets are summarised in **Table 5.1**.

Table 5.1: Summary of key study data sets.

Name	Description	Source
Port Hedland LiDAR survey	0.5m contour data flown in November 2010 by AAM	LandCorp
Shellborough LiDAR survey	0.5m contour data flown in November 2010 by AAM	LandCorp
Hydrographic charts	Hydrographic charts relevant to model area AUS53, AUS325	Australian Hydrographic Office
Geoscience Australia digital bathymetry	1km digital bathymetric model	Geoscience Australia
Aerial Photography	Satellite photography coverage of most of the study area – excluding Shellborough	Department of Planning, WA
Rectified aerial photography and photogrammetric survey of Port Hedland region	Historical aerial photos from the years 1949, 1968, 1976, 1983, 1985, 1993, 1999, 2004 and 2009	Photogrammetric survey prepared by Survey Graphics. Images from Landgate and Geoscience Australia
Rectified aerial photography of Shellborough region	Historical aerial photos from the years 1949, 1968, 1971, 1972, 1976, 1993 and 2009	Images from Landgate and Geoscience Australia
Tide and wave data	Historical tide and wave data for specified time periods	DOT and PHPA
Meteorological Data (wind, pressure and rainfall)	Port Hedland Airport (4032) 1942-2010 Hillside Station (4015) 1999-2003	BoM
Stream Gauging Data	Boodarie (709012) 2000-2007 Pincunnah (709010) 1985-2010	DoW
Wedgefield Subdivision Design Levels	Drainage details and road levels from the Wedgefield subdivision	LandCorp
South Hedland Town Centre Flood Study	Details on the drainage network from the Wedgefield subdivision	LandCorp
Port Hedland Cadastre	Cadastre of Port Hedland in GIS format	Main Roads
Main Roads Survey of Hydraulics Structures	Survey / Design drawings for hydraulic structures which are part of Main Roads assets	Main Roads

5.1 LiDAR Survey Data

As part of the Port Hedland *Coastal Vulnerability Study*, LiDAR survey was commissioned and flown across the study area between the 3rd and 14th November 2010 by AAM. The completed LiDAR survey data was supplied to LandCorp in December 2010 and detailed the Port Hedland and Shellborough areas as 0.5m contours to a vertical accuracy of +/- 0.10m.

The Port Hedland LiDAR area and data is shown on **Figure 5.1** and **Figure 5.2** the Shellborough LiDAR area is shown on **Figure 5.3**

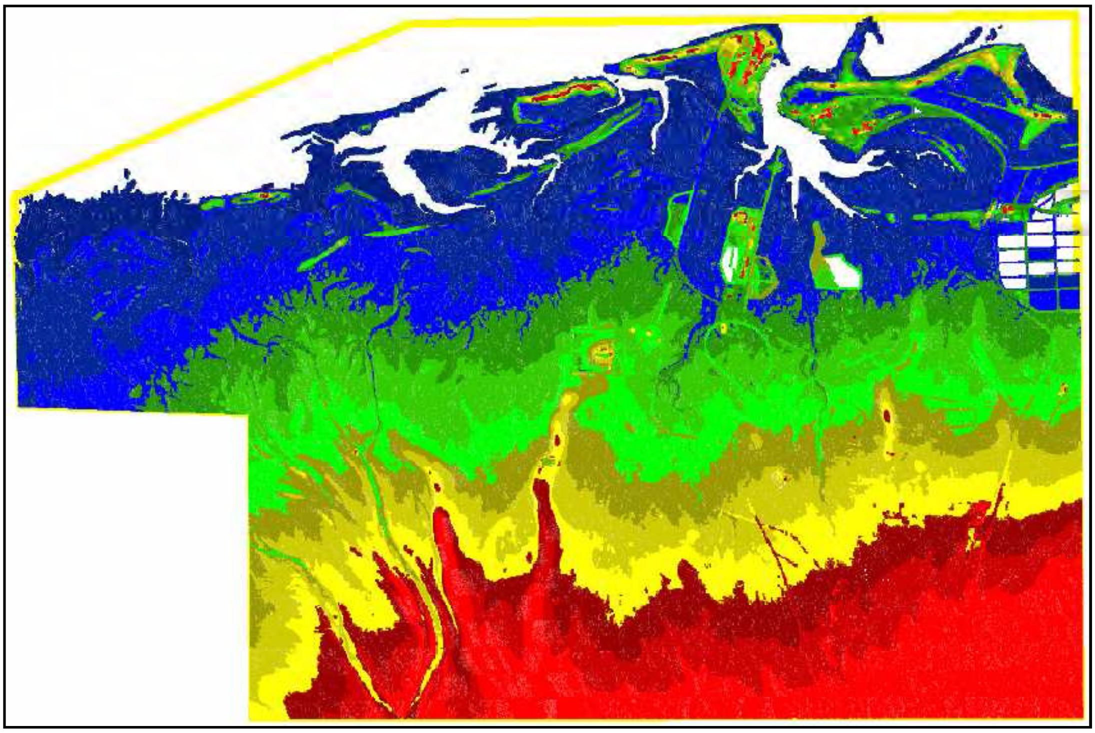

Figure 5.1: LiDAR Data Showing Ground Coloured By Elevation For Port Hedland (AAM 2010).

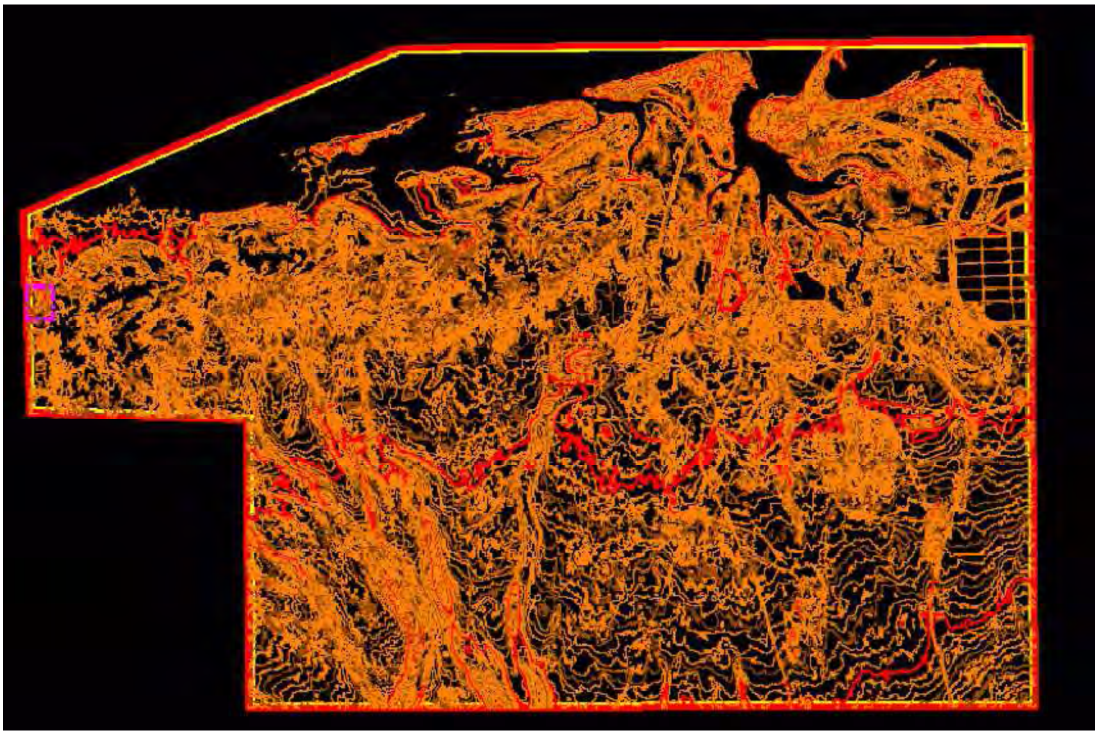

Figure 5.2: LiDAR Data Showing 0.5m Contour Information For Port Hedland (AAM 2010).

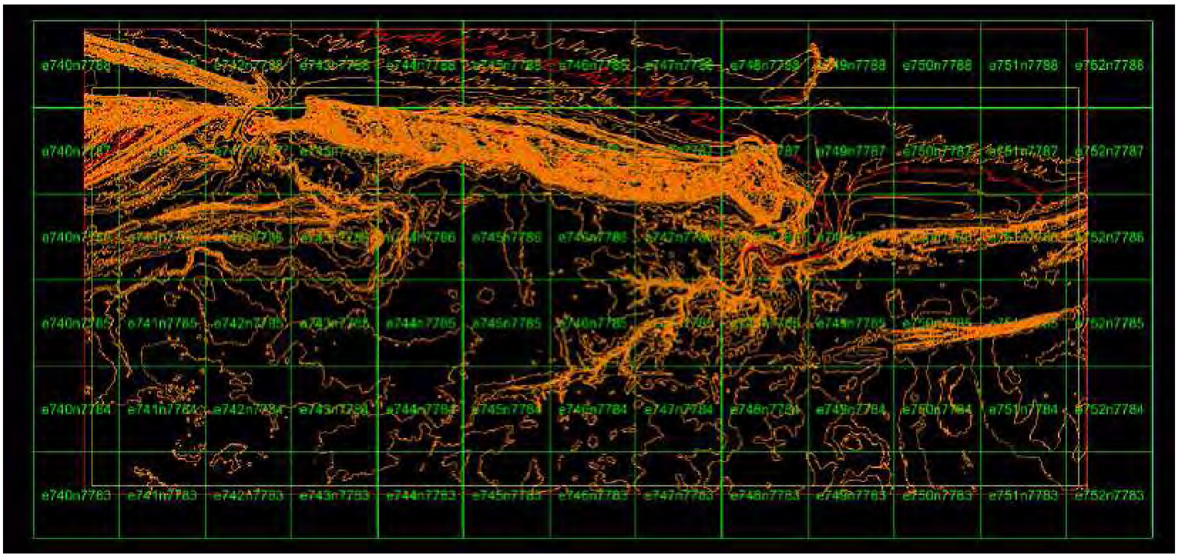

Figure 5.3: LiDAR Data Showing 0.5m Contour Information For Shellborough (AAM 2010).

6 OCEAN INUNDATION ASSESSMENT

6.1 Overview

The following sections present an overview of the methodology and outcomes from the ocean inundation assessment. **Appendix A** is a detailed technical document which presents a detailed description of the model system, model setup and calibration, and results from the design simulations.

6.2 Model Systems and Summary Description

The ocean inundation model system consists of three key components:-

- Wind field model to simulation wind and atmospheric pressure during cyclone events;
- Wave model (SWAN) which simulates cyclone waves which are generated up to 2000km from Port Hedland; and
- Hydrodynamic model system which can simulate tide and storm surge processes.

The wind field model adopted for this study is a modified version of the Holland (1980) parametric cyclone wind-field model - see Holland (1991). The Holland (1980) model has been combined with the wind field asymmetry model of Shapiro (1983) to define first-order wind-field asymmetry due to cyclone forward motion. The model also includes a parametric Radius to Maximum Wind (RMW) model to define this key cyclone wind field parameter. The RMW model which defines the distance (km) from the cyclone eye of the strongest wind speeds is based on the relationship described in Harper *et al* (1989 and 1993) for cyclones on the northwest shelf region. **Appendix A** presents a detailed description of the wind field model applied in this study.

A multi-domain SWAN wave model system has been developed for the Port Hedland *Coastal Vulnerability Study*. The model system is based on similar models which Cardno has developed for other projects in the Port Hedland region. In order to accurately simulate the wave conditions during cyclone events for this study, a large scale wave model domain that extends over 1000km north, west and east of Port Hedland has been developed to provide boundary conditions to the continental shelf and nearshore wave model (≈ 200km) which is coupled to the hydrodynamic tide and storm surge model. **Appendix A** presents a detailed description of the wave model system.

A 2D/3D numerical model of the Pilbara coastline centred around Port Hedland has been developed using the Delft3D modelling system. Cardno has been developing and refining this numerical model of the Port Hedland region since 2006. The current Delft3D model adopts a multi-domain Delft3D Domain Decomposition model configuration.

The model extends across a region extending approximately 200km east and west, and 180km north of Port Hedland. The model grid system was prepared with the main objective of providing high resolution in the Port Hedland nearshore and harbour, but it also needed to describe a large coastal area in order to accurately represent the tidal variations and propagation of the tidal wave throughout the model domain. The model also covers the Shellborough study area to the east of Port Hedland. **Figure 6.1** presents a plan view of the extent of the hydrodynamic model system including the extent of the sub-domains.

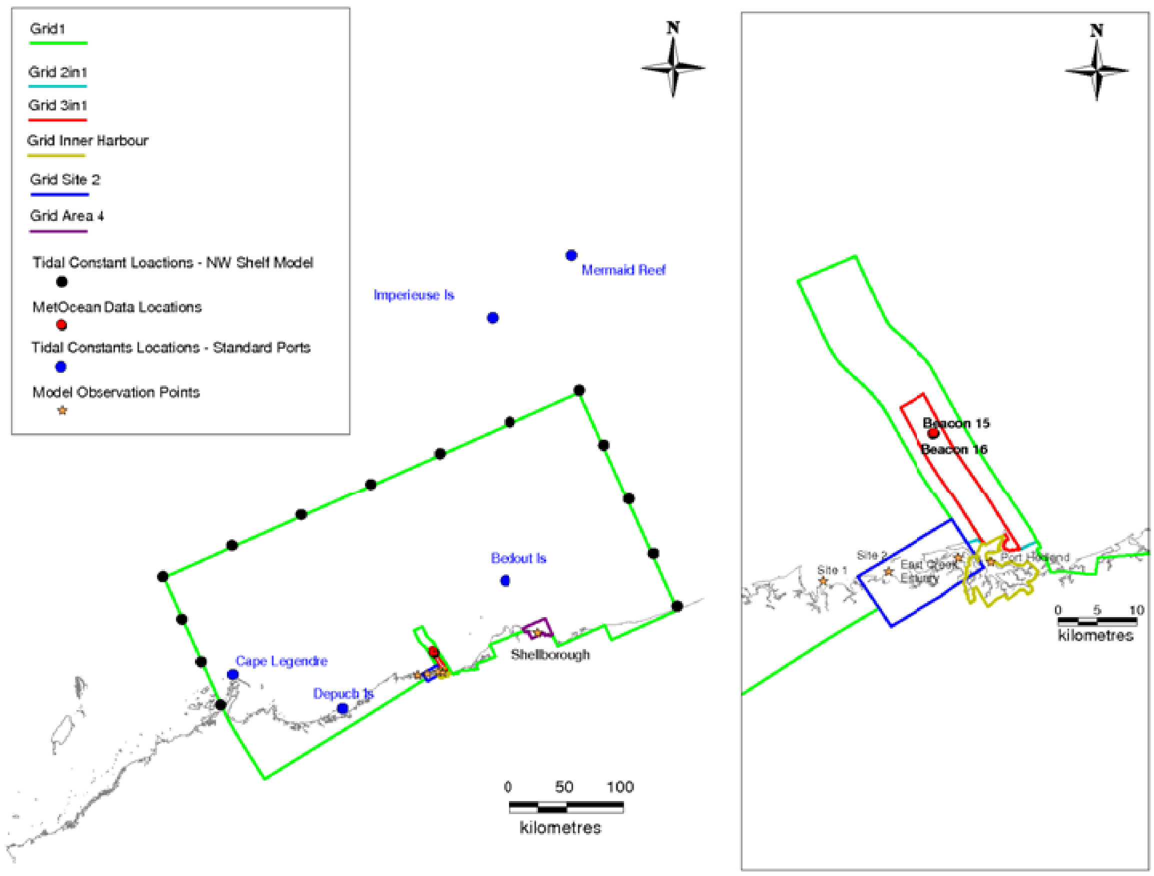

Figure 6.1: Plan View of the Extent of the Delft3D system adopted in this study including the detailed output study area.

6.3 Model Calibration

A detailed model calibration and validation exercise has been undertaken to confirm the accuracy of the combined wind, air pressure, surface wave and hydrodynamic model system. The calibration of the wind, wave and hydrodynamic model systems was undertaken using measured data from six historical cyclone events for which observed data is available at Port Hedland. Wind and air pressure observations were used to calibrate the wind field model. Recorded offshore surface wave data was used to calibrate the SWAN wave model. Water level and current data were utilised to calibrate the hydrodynamic model. The hydrodynamic model calibration process included both tidal and storm surge conditions.

Data available for calibration of the models varies through time. The six calibration events selected for this study included five cyclones which have occurred since 1999. The quality and resolution of the cyclone track data has improved since 1999 and also a wide range of concurrent water level, wind, air pressure and wave data is available since that time. An earlier cyclone event, Tropical Cyclone Connie, which occurred in 1988 is also included as a validation case because measured wind and surge data is available for this relatively severe event. The six calibration and validation events adopted for the ocean inundation model system are:-

1. Tropical Cyclone Connie – December 1988
2. Tropical Cyclone John – March 1999
3. Tropical Cyclone Monty – February 2003
4. Tropical Cyclone Clare – January 2006
5. Tropical Cyclone Daryl – January 2006
6. Tropical Cyclone George – March 2007

Model calibration was undertaken firstly for the wind field and pressure model as this provides the primary forcing for the wave and hydrodynamic model, followed by the wave and hydrodynamic models. The calibration process is summarised in the flow chart presented as **Figure 6.2**.

Overall the model system achieved a good degree of calibration for all model systems. In particular the agreement between modelled and measured parameters for the three most severe cyclone events, Tropical Cyclones Connie, John and George, is good. **Appendix A** presents a detailed description of the calibration process and outcome. Overall, the model is suitably accurate to be applied to a *Safety Standard* inundation risk assessment study – see **Section 4**.

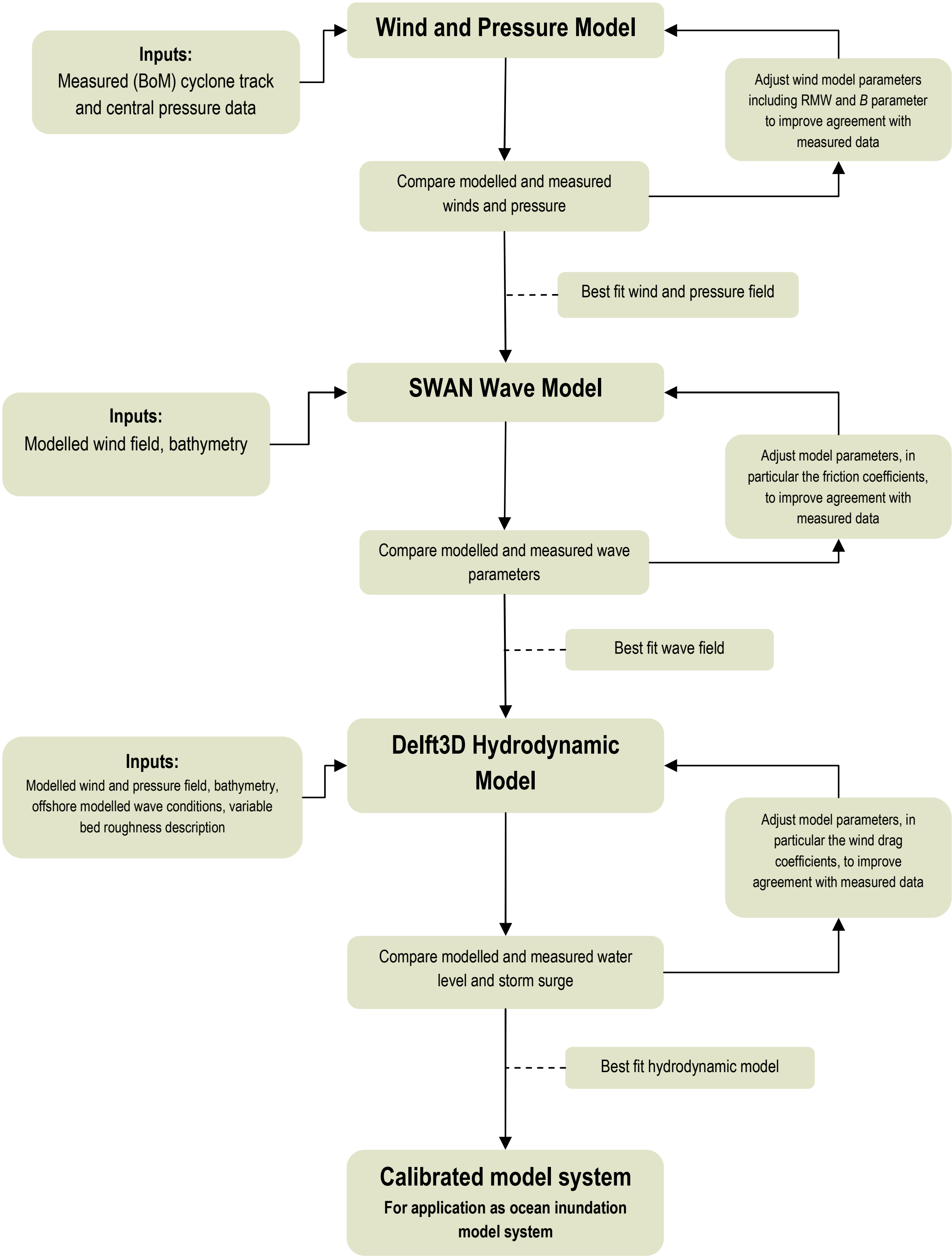

Figure 6.2: Flow chart of the calibration process for the ocean inundation model system.

6.4 Cyclone Hindcast Study

A hindcast model simulation study has been undertaken to define design storm tide levels in the study area based on the historical cyclone data. The Port Hedland region has a relatively high cyclone frequency (and intensity) which has resulted in a reasonability large measured data set of historical cyclone events which have generated significant storm surge at Port Hedland. The reliable cyclone record extends back to about 1960 when satellite imagery was adopted by the BoM. However the resolution, reliability and quality of more recent data has improved significantly when compared to the early data collected around 1960. For this study component, Cardno has adopted the post-1960 data set.

Following discussions between Cardno and DoT Coastal Management, due to the relatively high cyclone frequency for the Port Hedland region, it was considered that the hindcast study approach would provide a reasonable basis to assess design water levels up to approximately 50-years ARI. For 100-years ARI and beyond, this study has adopted a Monte Carlo modelling technique to define long-return period conditions based on a synthetic data set. The hindcast study provides a mechanism to assess the validity of the Monte Carlo at the 100-years ARI level.

Since 1960, Cardno identified a sample of 32 historical cyclones which passed within approximately 300km of Port Hedland and which had a central pressure below 980hPa. These events were selected for detailed model simulations to hindcast storm tide levels across the study area. **Figure 6.3** presents a plan view of the historical cyclone tracks investigated in this study. **Appendix A** presents further details on the methodology of the hindcast study and the characteristics of the hindcast events.

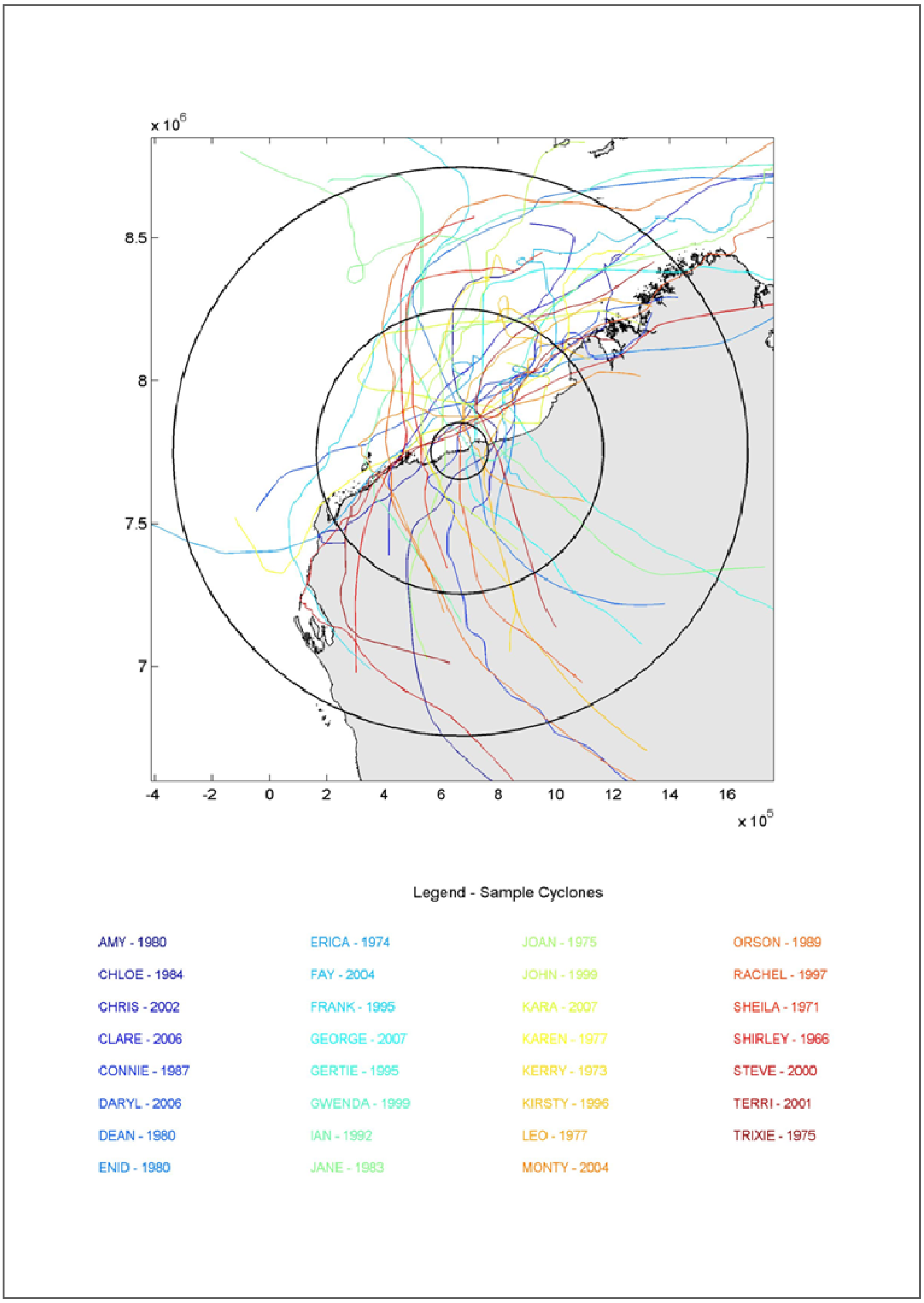

Figure 6.3: Cyclone Tracks of Selected Historical Events for Hindcast Study.

6.4.1 Hindcast Study Results

Appendix A presents a detailed summary of the results from each of the hindcast simulations at key study locations. A summary of the design levels calculated at key study locations is summarised in **Tables 6.1 to 6.4.** The design criteria presented in this report have been determined using Extreme Value Analysis (EVA) statistics. During the analysis of the model results, Type-1 (Gumbel) and Type-III (Weibull) distributions were considered. Overall, the Type-III distribution was determined to be preferable for this study as it is a more flexible distribution that can better accommodate data sets which have extreme outlying values which is the case for this study. The parameters for the EVA distribution were determined using a Maximum Likelihood technique as recommended by van Vledder *et al* (1993) and Goda (2000). Confidence intervals were determined using a boot-strapping procedure. Note, the level presented in **Tables 6.1** to **6.4** present Peak Total Still Water Levels and exclude additional factors such as non-cyclonic residual water level and shoreline wave setup (where applicable) which maybe in addition to these levels.

Table 6.1: Summary of Peak Total Still Water Level (TSWL) for Port Hedland (Berth-3 Tide Gauge) - Selected ARI's

ARI (years)	Port Hedland		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
2	3.1**	3.0	3.2
10	3.6	3.3	3.8
25	3.8	3.5	4.2
50	4.0	3.6	4.4
100	4.2	3.7	4.7

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

** Value below HAT at Port Hedland

Table 6.2: Summary of Peak Total Still Water Level (TSWL) for Site 1 - Selected ARI's

ARI (years)	Site 1		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
2	3.1**	3.0	3.2
10	3.6	3.4	3.9
25	3.9	3.5	4.3
50	4.1	3.6	4.6
100	4.4	3.7	5.0

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

** Value below HAT at Port Hedland

Table 6.3: Summary of Peak Total Still Water Level (TSWL) for Site 2 (Turner River Entrance)

ARI (years)	Site 2		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
2	3.1**	3.0	3.2
10	3.6	3.4	3.9
25	3.9	3.5	4.3
50	4.1	3.6	4.6
100	4.2	3.6	4.9

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

** Value below HAT at Port Hedland

Table 6.4: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough

ARI (years)	Shellborough		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
2	3.2**	3.1	3.4
10	4.0	3.9	4.2
25	4.4	4.2	4.5
50	4.6	4.4	4.8
100	4.7	4.5	5.0

* Peak Total Still Water Level based on modelled tide plane from the Delft3D model

** Value below HAT at Port Hedland

6.5 Monte Carlo Cyclone Model

A major limitation in developing long return period design criteria and planning levels for cyclone conditions is the limited reliable cyclone record. Within Australia, the reliable cyclone track data extends back to approximately 1960 when over-the-horizon radar and satellite data for cyclones began to be collected. At Port Hedland, there have been 77 cyclones with a central pressure below 980hPa that have passed within 500km of Port Hedland since 1960. Whilst this is a large number of cyclones in the Australian context, in order to appropriately develop extreme state design criteria, for example for the 100-years Average Recurrence Interval (ARI) or less frequent design events, this is a relatively small sample from a data period that is quite a short record relative to the required planning period criteria for this study. A widely adopted practice in oceanography and coastal engineering to investigate long return period cyclonic conditions is through the generation of much longer data records of synthetic cyclone tracks using a Monte Carlo modelling approach.

6.5.1 Monte Carlo Model Setup

Cardno has developed a Monte Carlo model system that is applied in the Port Hedland *Coastal Vulnerability Study*. A historical track database is populated and analysed to identify temporal and spatial relationships in key cyclone track parameters. Cyclones passing within a 500km radius with a central pressure below 980 hPa are incorporated into the database. Statistical distributions of the key parameters (time of origin, location of origin, central pressure, forward speed and cyclone heading) are utilised to drive a random walk process model that generates statistically and physically realistic synthetic cyclone tracks. **Figure 6.4** presents a plot of synthetic cyclone tracks for a 50 year duration equivalent to the reliable historical sample. Overall good agreement is demonstrated between the modelled and sample cyclones for the key track parameters. For further detail of the model setup and validation refer to **Appendix A.**

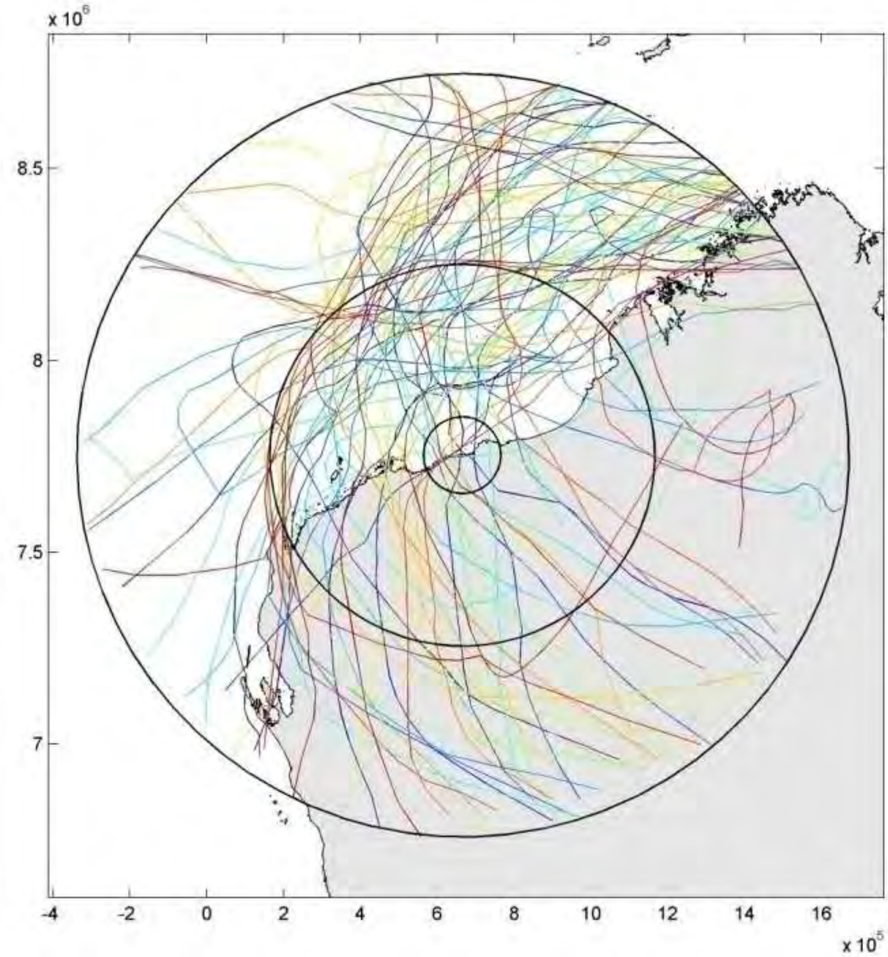

Figure 6.4: Plot of Modelled Synthetic Cyclone Tracks for a 50-year Duration Simulation (Equivalent to Reliable Track Record).

Through the Monte Carlo model system a data record of 10,000 years of synthetic cyclone tracks is generated. The storm surge and total water level associated with the synthetic cyclone tracks is assessed using a tiered ranking system that progressively identifies and models the design storm events with greater accuracy. Extreme value analysis is applied to calculate 100, 200 and 500 year ARI design still water levels and specific design events are identified. The final stage of the ranking process involves the application of the full process model for design events using the coupled wind, wave and hydrodynamic model system.

6.5.2 Design Event Simulation

Design events identified through the ranking of the 10,000 years of synthetic cyclone tracks were modelled using the coupled wind, wave and hydrodynamic model system. **Figure 6.5** presents a summary view of the top 1000 cyclone tracks for the Port Hedland area which were simulated by the Monte Carlo model.

Compared to the hindcast study results presented in **Tables 6.1 to 6.4**, the Peak TSWL at 100-years ARI are approximately +0.3m larger from the Monte Carlo modelling study near Port Hedland, and +0.5m higher at Shellborough. Design peak still water levels for the 100, 200 and 500 year ARI planning periods are calculated from the Monte Carlo modelling.

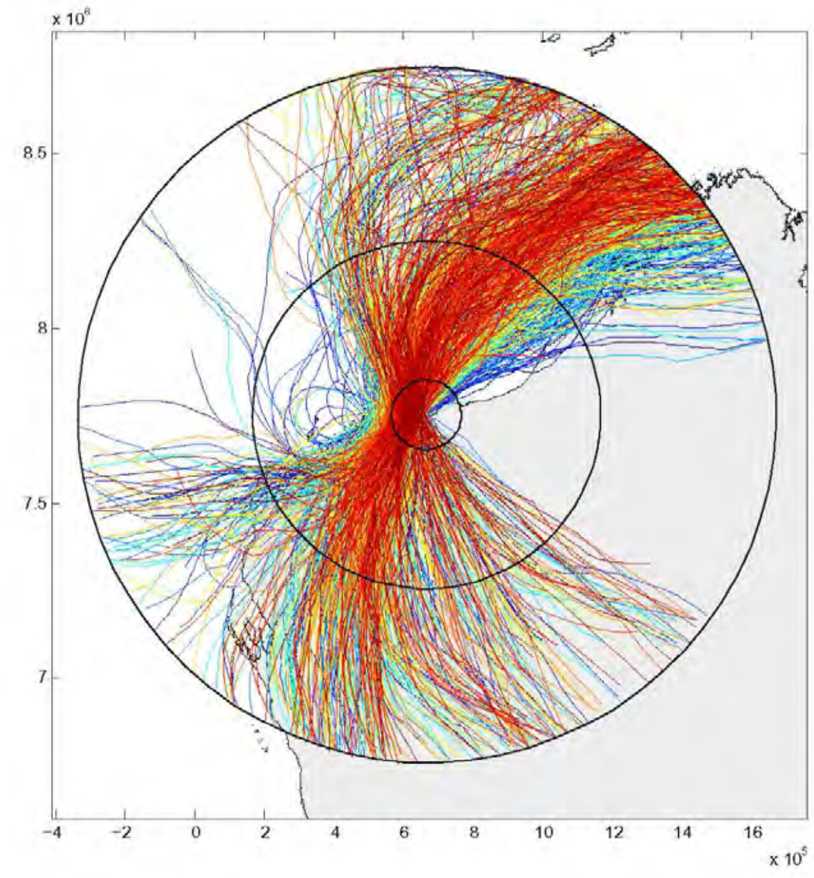

Figure 6.5: Plot of Top 1000 Modelled Synthetic Cyclone Tracks for a 10,000-year Duration Simulation.

Future Greenhouse enhanced global climate scenarios have the potential to affect the distribution, frequency and intensity of tropical cyclones. Estimates of changes to tropical cyclones under future climate change scenarios are uncertain due to the significant variability in the historical cyclone record and the large natural inter-decadal variations associated with processes such as ENSO and the Indian Ocean Dipole. The most comprehensive recent assessment of the potential changes in cyclone climatology undertaken by the CSIRO (2007) suggests that under future climate change scenarios there may be a reduction in frequency (perhaps up to 40%) across Northwest Australia, however, with an increase in severe Category 3 to 5 cyclone events. Based on recent cyclone studies undertaken by the CSIRO (2007) and Cardno Lawson Treloar (2009a and 2009b) a potential 10% increase in maximum cyclone wind speeds has been applied in the simulation of 2110 design criteria. This is also consistent with a recent paper regarding predicted changes to tropical cyclones due to climate change prepared by a number of leading researchers – Knutson *et al.* (2010).

Cardno has reviewed current data, estimates and policy regarding sea level rise so that an appropriate Sea Level Rise (SLR) allowance can be incorporated into the Port Hedland coastal vulnerability investigations. The *Western Australian Planning Commission (WAPC)* has recently released a position statement regarding sea level rise allowances for Western Australia. WAPC (2010) recommends that for a 2110 year planning period a SLR allowance of +0.9m should be accommodated and for a 2060 year planning period a SLR allowance of +0.3m should be accommodated. These levels have been adopted in this study.

6.6 Non Cyclonic Water Level Variations

Appendix A presents a description of data analyses undertaken by Cardno to determine non-cyclonic water level variations. For this study an allowance of +0.2m has been adopted to account for non-cyclonic water level variability. This represents inter-annual water level variability which occurs along the northwest shelf, a process which is discussed in greater detail in **Appendix A.6**.

6.7 Design Water Levels

Tables 6.5 to **6.9** present design water levels for selected locations within the study area for return periods between 2 and 500-years ARI. Design levels up to 50-years ARI are based on the hindcast study and for 100-years ARI and greater the results are from the Monte Carlo modelling. For the design water levels, an allowance of +0.2m has been added to account for the potential for a non-cyclonic residual water level when a cyclone occurs.

No measured water level or tidal plane data is available near Shellborough and the Delft3D model does not include all of the astronomical constituents to describe the full tide on the boundary of the model. As a result, an additional water level of +0.5m was added to the design level from the Shellborough site to account for the difference between the likely total tide range and the modelled tide. If water level data (minimum 2-months) were recorded near Shellborough it should be possible to combine the modelled storm surge results with a more accurate predicted (astronomical) tide for Shellborough and define the design water levels for the Shellborough region with greater confidence.

In the Port Hedland town region, the design water levels separate into three general regions within which the design water levels are broadly similar for a certain ARI. The first of these is the inner harbour area and for this study, design water levels have been provided relative to the Berth-3 permanent tide gauge. Overall spatial variations in modelled water inside the inner harbour are generally small, about 0.1m or less as the potential for increased wind setup inside the estuary is offset by frictional losses with distance upstream of the entrance.

Near the Port Hedland harbour entrance and on the western side of the Spoil Bank, there is a need to consider the potential for wave setup to increase the total water level at the shoreline for sites exposed to open coast swell waves during cyclones. The wave setup allowance is based on selected SBEACH simulations for a number of open coast profiles in the Port Hedland area – see **Appendix D**.

The presence of the Spoil Bank acts as a significant hydraulic control during design cyclone events with higher storm surges. As a consequence total water levels on the eastern side of the Spoil Bank, around to the Pretty Pool area are higher than the western side as this eastern area has significantly higher wind setup levels compared to the entrance of Port Hedland. Again, open coast locations also need to consider the impact of wave setup on design water levels. **Figure 6.6** delineates the general design water level zones defined in the ocean inundation modelling. Note, **Sections 8** and **9** define the inundation extents in more detail.

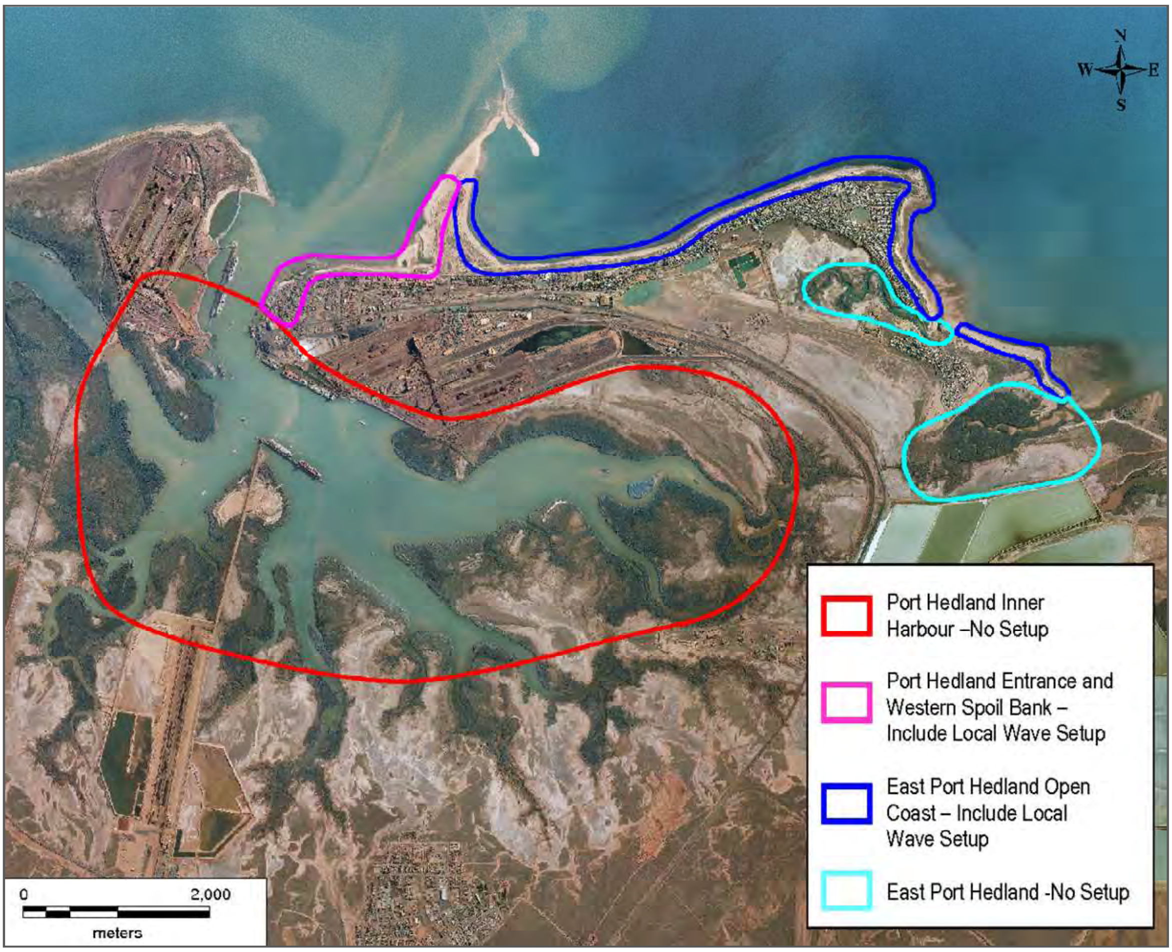

Figure 6.6: Plan view of the general Port Hedland regions where particular design levels are applicable.

Sites 1 and 2, which are to the west of Port Hedland, have similar design water levels at both sites based on results from the hindcast and Monte Carlo modelling investigations. As a result, a single set of design water level criteria have been presented for these sites. Open coast locations, for example Site 2 also need to consider the impact of wave setup on design water levels.

The Shellborough region has the highest design water levels in the study region. The combined effects of a larger tidal range, together with the potential for greater storm surge along this section of coast result in significantly larger design levels at this site compared to the Port Hedland region.

Table 6.5: Summary of Design Peak Total Still Water Level (TSWL) for Port Hedland Inner Harbour (Berth-3 Tide Gauge) - Selected ARI's for present climate scenario

ARI (years)	Port Hedland Inner Harbour (Berth-3)
	Peak TSWL (mAHD)*
2	3.3
10	3.8
20	3.9
50	4.2
100	4.7
200	4.9
500	5.1

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table 6.6: Summary of Design Peak Total Still Water Level (TSWL) for West of the Spoil Bank and Harbour Entrance - Selected ARI's for present climate scenario.

ARI (years)	Port Hedland Entrance and Western Spoil Bank		
	Peak TSWL (mAHD)*	Wave Setup – Ocean Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.3	0	3.3
10	3.8	0.8	4.6
20	3.9	0.8	4.7
50	4.2	0.8	5.0
100	4.7	0.9	5.6
200	4.9	1.0	5.9
500	5.1	1.2	6.3

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table 6.7: Summary of Design Peak Total Still Water Level (TSWL) for Eastern Spoil Bank to Pretty Pool - Selected ARI's for present climate scenario.

ARI (years)	East Port Hedland - Eastern Spoil Bank to Pretty Pool		
	Peak TSWL (mAHD)*	Wave Setup – Ocean Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.5	0	3.5
10	4.0	0.8	4.8
20	4.1	0.8	4.9
50	4.4	0.8	5.2
100	5.0	0.9	5.9
200	5.1	1.0	6.1
500	5.6	1.2	6.8

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table 6.8: Summary of Design Peak Total Still Water Level (TSWL) for Sites 1 and 2 - Selected ARI's for present climate scenario.

ARI (years)	Sites 1 and 2		
	Peak TSWL (mAHD)*	Wave Setup – Ocean Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.3	0	3.3
10	3.8	0.8	4.6
20	4.0	0.8	4.8
50	4.3	0.8	5.1
100	5.0	0.9	5.9
200	5.2	1.0	6.2
500	5.5	1.2	6.7

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009) and includes allowance for non-cyclonic variation in the mean water level.

Table 6.9: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough - Selected ARI's for present climate scenario.

ARI (years)	Shellborough		
	Peak TSWL (mAHD)#	Wave Setup – Ocean Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.9	0	3.9
10	4.7	0.7	5.4
20	5.1	0.7	5.8
50	5.3	0.8	6.1
100	5.9	0.9	6.8
200	6.4	1.0	7.4
500	6.6	1.0	7.6

Peak Total Still Water Level based on modelled tide and includes allowance for non-cyclonic variation in the mean water level (+0.2m) and also an allowance for the actual tide range (+0.5m) at the site which is not fully simulated in the Delft3D model.

6.8 2110 Design Water Levels

Tables 6.10 to **6.14** present design water levels for selected locations within the study area for return periods between 2 and 500-years ARI based on a 2110 climate scenario which has an increase in the mean sea level of +0.9m and an increase in cyclone intensity. The 2110 design water levels have been calculated by adding +0.9m to the design levels for the present climate scenario presented in **Section 6.9** for ARI's less than 100-years. For ARI's greater than 100-years, the design levels are based on results from simulations of the Monte Carlo model for a climate change scenario with a +0.9m increase in mean sea level and a 10% increase in cyclone intensity. The potential increase in maximum cyclone wind speeds of 10% has a relatively small influence on the design water level compared to the change in mean sea level. At the 500-year ARI level, this increase has only result in an additional +0.1m increase to the design water levels. For the design water levels, an allowance of +0.2m has been added to account for the potential for a non-cyclonic residual water level when a cyclone occurs.

Table 6.10: Summary of Design Peak Total Still Water Level (TSWL) for Port Hedland Inner Harbour (Berth-3 Tide Gauge) - Selected ARI's for 2110 climate scenario

ARI (years)	Port Hedland Inner Harbour (Berth-3)
	Peak TSWL (mAHD)*
2	4.2
10	4.7
20	4.8
50	5.1
100	5.6
200	5.8
500	6.1

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table 6.11: Summary of Design Peak Total Still Water Level (TSWL) for West of the Spoil Bank and Harbour Entrance - Selected ARI's for 2110 climate scenario.

ARI (years)	Port Hedland Entrance and Western Spoil Bank		
	Peak TSWL (mAHD)*	Wave Setup – Ocean Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.2	0	4.2
10	4.7	0.8	5.5
20	4.8	0.8	5.6
50	5.1	0.8	5.9
100	5.6	0.9	6.5
200	5.8	1.0	6.9
500	6.1	1.2	7.4

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table 6.12: Summary of Design Peak Total Still Water Level (TSWL) for Eastern Spoil Bank to Pretty Pool - Selected ARI's for 2110 climate scenario.

ARI (years)	East Port Hedland - Eastern Spoil Bank to Pretty Pool		
	Peak TSWL (mAHD)*	Wave Setup – Ocean Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.4	0	4.4
10	4.9	0.8	5.7
20	5.0	0.8	5.8
50	5.3	0.84	6.1
100	5.9	0.9	6.8
200	6.0	1.0	7.0
500	6.6	1.2	7.8

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table 6.13: Summary of Design Peak Total Still Water Level (TSWL) for Sites 1 and 2 - Selected ARI's for 2110 climate scenario.

ARI (years)	Sites 1 and 2		
	Peak TSWL (mAHD)*	Wave Setup – Ocean Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.2	0	4.2
10	4.7	0.8	5.5
20	4.9	0.8	5.7
50	5.2	0.8	6.0
100	5.9	0.9	6.8
200	6.1	1.0	7.1
500	6.5	1.2	7.7

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009) and includes allowance for non-cyclonic variation in the mean water level.

Table 6.14: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough - Selected ARI's for 2110 climate scenario.

ARI (years)	Shellborough		
	Peak TSWL (mAHD)#	Wave Setup – Ocean Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.8	0	4.8
10	5.6	0.7	6.3
20	6.0	0.7	6.7
50	6.2	0.8	7.0
100	6.8	0.9	7.7
200	7.3	1.0	8.3
500	7.6	1.0	8.6

Peak Total Still Water Level based on modelled tide and includes allowance for non-cyclonic variation in the mean water level (+0.2m) and also an allowance for the actual tide range (+0.5m) at the site which is not fully simulated in the Delft3D model.

The Spoil Bank is a dynamic section of the Port Hedland coastal foreshore and the historical development of this feature is discussed in greater detail in **Appendix D**. There is a degree of uncertainty regarding the future evolution and possible erosion of the Spoil Bank, and to assess its influence on storm tide impacts for the town a scenario in the 100 year planning period (2110) with the Spoil Bank removed from the Port Hedland foreshore has been investigated in **Appendix A.10**.

In summary, the presence of the Spoil Bank acts as a hydraulic control and its removal leads to significant change in the wave propagation in the vicinity of the Port Hedland Township and harbour entrance. The Spoil Bank induces a strong refractive process towards the Spoil Bank, diverting significant amounts of wave energy away from the harbour entrance and township shoreline to be dissipated on the Spoil Bank. The removal of the Spoil Bank results in an increase in wave height of approximately 0.6m and increases the potential total water level in the order of 0.3m for both of the selected representative 100 year and 500 year ARI cyclone events (refer **Figure 6.7**).

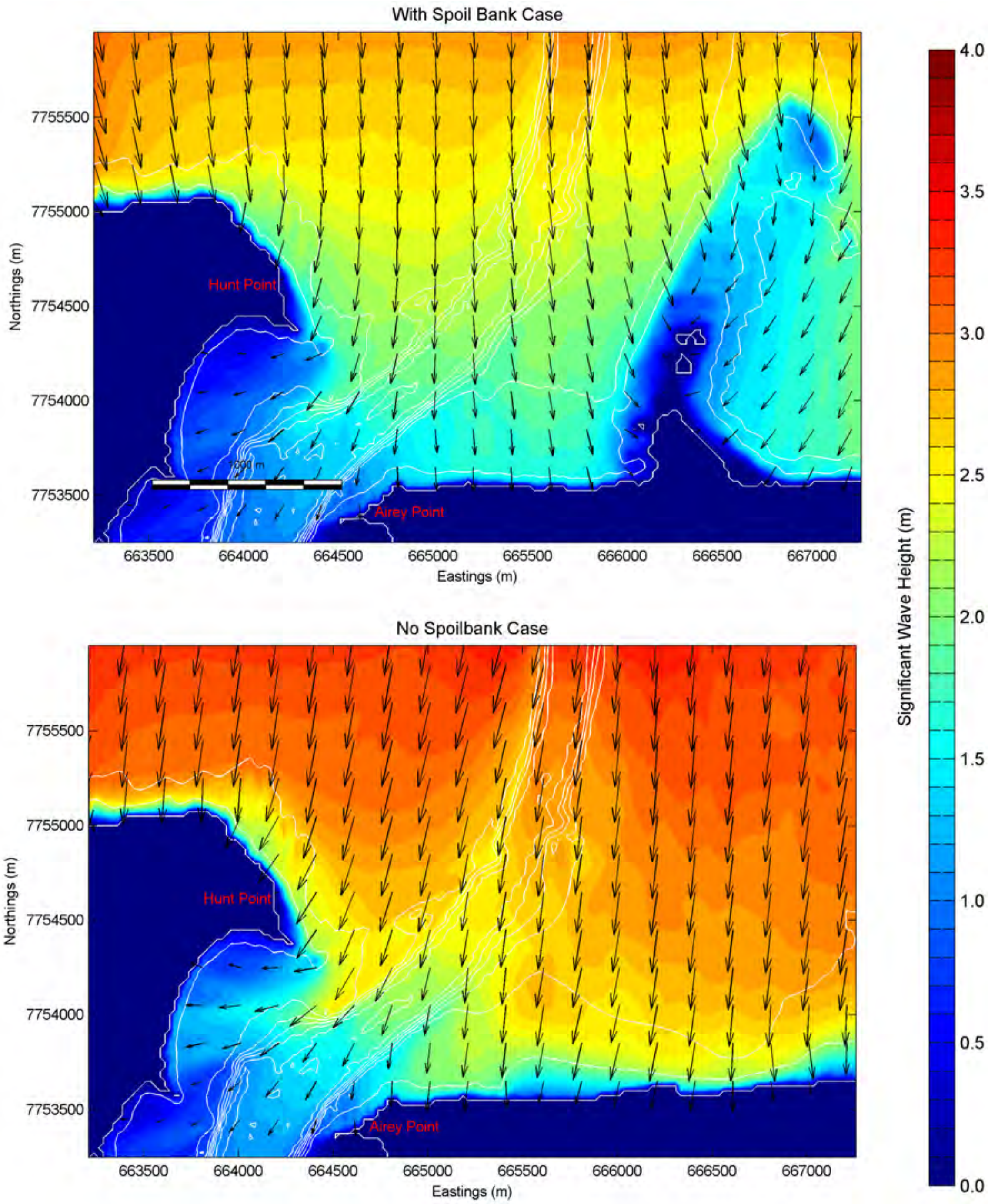

Figure 6.7: Wave Map Comparisons for the 100 Year ARI Cyclone with and without Spoil Bank at the entrance to Port Hedland Inner Harbour

6.9 Recommended Design Ocean Water Levels for East Port Hedland and Spoil Bank Developments

Table 6.15 presents a summary of the recommend design water levels for potential developments near the Spoil Bank and at East Port Hedland. The recommended design water levels are based on a 2110 planning period. If infill development is being considered, it is recommended that general fill levels be based on the acceptable risk level design criteria for a 2110 planning period. Based on the uncertainty in the modelling and in estimating long return period design levels, Cardno would generally advise that floor levels in any fill development be specified at least 0.5m above the required design water level. For the Spoil Bank region which has wave setup included in the design water levels in **Table 6.15**, the potential inundation as a result of wave run-up and overtopping will also need to be considered when determining the crest level for any shoreline structures.

Table 6.15: Summary of Design Peak Total Still Water Level (TSWL) for East Port Hedland and Spoil Bank Developments - Selected ARI's for 2110 climate scenario.

ARI (years)	Design Peak Total Still Water Level (mAHD)	
	East Port Hedland – Excluding Open Coast Locations	Spoil Bank Area
2	4.4	4.4
10	4.9	5.7
20	5.0	5.8
50	5.3	6.1
100	5.9	6.8
200	6.0	7.0
500	6.6	7.8

The inundation maps presented at the end of this report define the ocean inundation extents for various ARI between 2 and 500-years. Presently, ocean inundation design criteria is addressed in the SPP 2.6 which specifies that in cyclone prone areas, development should be set back sufficiently to avoid flooding from a "Category 5 cyclone tracking to maximise its associated storm surge". This design condition does not specifically relate to a defined probabilistic Average Recurrence Interval (ARI). It is understood that as part of the review of SPP 2.6, the specification of specific ARI's for cyclone inundation planning purposes is being considered.

Within Australia, for normal residential development it is common that in planning for ocean and/or catchment inundation, a 100-year ARI design condition is the normal standard. For new developments, a 100-year planning period is often considered so that the effects of climate change and sea level rise over a 100-year planning period are incorporated into the planning level. Higher risk infrastructure such as hospitals, evacuation routes and evacuation centres, are commonly designed for much longer return period conditions. For example in Queensland, evacuation centres have to be designed with floor levels above the 10,000-year ARI storm tide level.

The addition of a planning or design level freeboard of 0.5m is commonly adopted in Australia. The freeboard allowance is designed to cover uncertainty related to the design criteria and is often specified to the design criteria for finished floor levels. For example, a new development may have roads and lot levels designed to a 100-year ARI inundation level, but the finished floor levels will be specified as some height above the 100-year inundation level to cover uncertainty and to further reduce the risk of inundation of the dwelling. The freeboard allowance varies considerably depending on the uncertainty related to the calculation of design levels, and also due to the potential risk of an actual event being significantly more severe than the specified design ARI condition. In the context of this study, for the Port Hedland region the 95% confidence interval for the 100-year ocean water level is approximately +/- 0.5m.

The key issue regarding the future application of design levels defined by this study is the management of risk. It is recognised that there are varying levels of risk that may be deemed acceptable by planning authorities across a development site and a range of management options could be considered. For example it may be the case for certain areas that it is considered acceptable for emergency access corridors to be set at a certain elevation, with general roads and car parking areas at a higher elevation and habitable floor levels set higher again, provided good emergency management policies/procedures are in place and the higher risk is accepted.

7 HYDROLOGICAL MODELLING

7.1 Overview

The following sections present an overview of the methodology and outcomes from the hydrological modelling which has been undertaken to provide inflow data for the hydraulic modelling – see **Section 8**. **Appendix B** is a detailed technical document which presents a detailed description of the model system, model setup and calibration, and results from the design simulations.

7.2 Model Systems

For calculation of the surface water runoff from the catchment, a 1D model was created using the XPSWMM hydrologic and hydraulic modelling software package. The hydrologic component of the software uses the Laurenson non-liner runoff-routing method to simulate runoff from rainfall events. The Laurenson runoff-routing method assumes that runoff is proportional to slope, area, roughness, infiltration and percentage of imperviousness of a catchment. The model was calibrated to observed rainfall and runoff data throughout the catchment (**Appendix Section B.4**). The calibrated model was then used to run design rainfall events with the outputs from the model used as the inputs to the 2D hydraulic modelling.

7.3 Model Setup

Two hydrological models were created for the catchments surrounding Port Hedland and Shellborough respectively. **Appendix B** presents details on the catchment area. Hydraulic modelling was undertaken to identify the amount of flow runoff that is generated by the catchments in rainfall events. The Port Hedland catchment is divided into 101 sub-catchments which each flow toward the coast. The two main sub-catchments in the Port Hedland catchment are the Turner River sub-catchment and the South West Creek sub-catchment. The Port Hedland catchment spans a distance of 150km and an area of 498,000ha. The Shellborough catchment is smaller than the Port Hedland catchment as the 18 sub-catchments span a distance of 39km and an area of 44,000ha. The following sections outline the model setup, calibration and key outcomes from the hydrological modelling.

The sub-catchments were delineated from Satellite topography and LIDAR data. The LIDAR data was compiled after flights were undertaken in November 2010. AAM Pty Ltd undertook the LIDAR development providing 0.5m contours containing a vertical accuracy of +/-0.10m. Geological mapping units were identified from 1:250,000 geological mapping datasets. The sub-catchments were divided into two broad geological units, Sand and Rock. The infiltration rates were based on the geological units and calibrated to observed flow and stage data. The Rock geological unit is mainly located in the upper reaches of the Turner River catchment, whereas the remainder of the catchment is classified as Sand. **Appendix B** includes details on the sub-catchments, streamlines, gauging station locations and the LIDAR and 2D hydraulic model extent for both the Port Hedland and Shellborough regions.

The cross sections used for the 1D hydraulic links (i.e. river reaches) were generated from the Shuttle Radar Topography Mission (SRTM) satellite data from NASA. SRTM data has a resolution typically ranging between 45-60m (Smith & Sandwell, 2003) and a vertical accuracy of +/-8m (Rodriguez *et al.*, 2005). The cross section of the bridge where the stage height gauging station is located was identified from high resolution LIDAR data. The cross section of the bridge is important as it has a direct impact on calculating the stage height.

Rainfall data was sourced from the Port Hedland Airport and Hillside gauging stations. The Port Hedland Airport is located near the coast and the Hillside gauging station is at the top of the Turner River catchment (see **Figure 7.1**). No rainfall gauging stations were located in the Shellborough catchment. The Port Hedland gauging station began recording data from 1953 and contains a long and relatively accurate dataset. The Hillside gauging station began recording 1999 however contains variable quality of data. Both gauges record 6 minute pluvial rainfall data.

Table 7.1 outlines the gauging period and the agency responsible for its maintenance.

Table 7.1: Rainfall Gauging Station Details

Station Name	Type of Stream Gauging Station	Operational Period	Managing Agency
Port Hedland Airport	Meteorological data (wind, pressure and rainfall)	1942 – 2010	Bureau of Meteorology
Hillside	Meteorological data (wind, pressure and rainfall)	1999 – 2003	Bureau of Meteorology

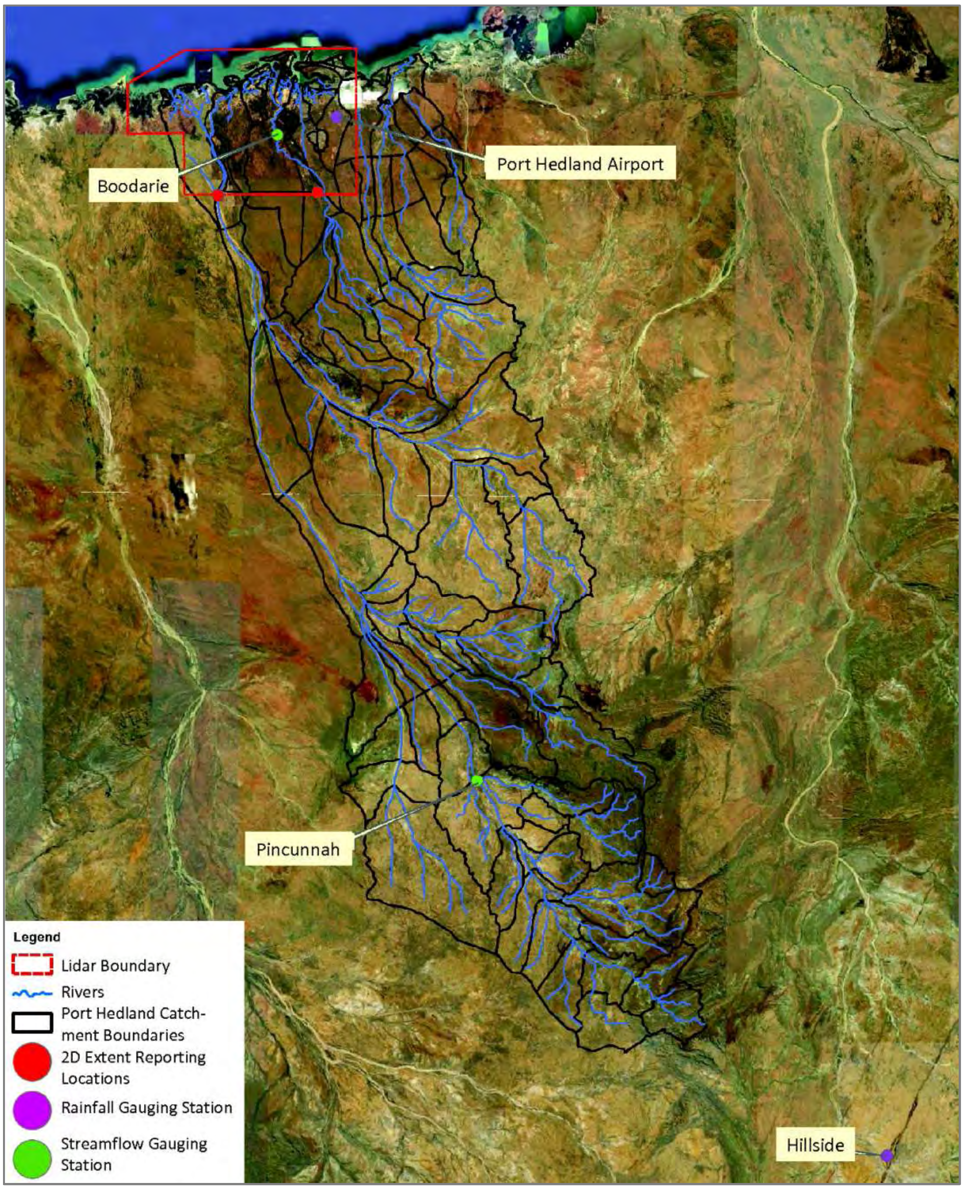

Figure 7.1: Modelled catchment area for the Turner River and Port Hedland including the locations of input and calibration data sets.

7.4 Model Calibration

The hydrological model was calibrated independently to the observed data for the associated rainfall event. This approach was used as each event behaved differently due to the antecedent conditions. In order to find the most consistent and expected catchment parameters for the predicted model hydrographs, the best calibration for each individual event was required to be undertaken. The two rainfall events in 2007 were modelled separately as there were two days between the rainfall periods of the cyclone event. As mentioned above, the antecedent conditions of a wetter catchment required a lower initial loss to be used in the second event.

The initial loss parameters adopted in this study are consistent with the mean initial loss values proposed for the North West Pilbara region in *Australian Rainfall & Runoff* which identify loam soils as having initial loss values ranging between 22-51mm (The Institute of Engineers, 2001).

Overall, the hydrological model has achieved a reasonable calibration given the limited available measured data. The focus on the calibration has been on simulating peak flows and total event volumes more accurately, with a lesser focus on the event phasing. The design simulations for the flood inundation extents (**Section 8**) adopt a prescribed relationship between peak catchment flows and ocean water levels and therefore the phasing from the hydrological model has little importance in the application of the hydrological flows in the hydraulic modelling.

Appendix B contains further details on the model calibration including time series and table comparisons of modelled and measured flow data.

7.5 Design Event Simulations

A design rainfall event is a probabilistic or statistical estimate which an average recurrence interval (ARI) or exceedance probability is attributed to (The Institute of Engineers, 1987). The design rainfall event simulations were generated to determine the extent, associated risk and potential impacts of design flood events. The design events were created using the following procedure:

- Creation of Intensity Frequency Duration (IFD) parameters and curves.
- Determine appropriate modelling parameters from the model calibration.
- A sensitivity analysis on the parameters chosen for the design rainfall events.
- A Flood Frequency Analysis (FFA) prediction based on observed streamflow gauging data.
- Creation of design ARI events.
- Comparison of design event results with Rational and Index Flood Method estimations.

Appendix B presents further details and outputs from the design event simulations.

7.6 Sensitivity Simulations

A sensitivity analysis was conducted on the 2 year, 100 year and 500 year ARI storm events in comparing the range of differences associated with the calibration parameters utilised. The sensitivity analysis aims to provide a better understanding of the range of uncertainty associated with the possible calibration parameters for the XPSWMM model, hence allowing a better understanding of the uncertainty and risk associated with the selected hydrology.

The sensitivity analysis identified significant variation in the peak flow rates with the different initial loss and Manning's roughness values used. The small rainfall event of the 2 year ARI design storm is severely impacted by the change in initial loss rates due to the low rainfall intensity of the storm event. The 100 year and 500 year design storms flow rates are affected by the changes in initial loss values more than the changes to the Manning's roughness coefficient.

For the 100 and 500-year ARI design cases, the sensitivity simulations produced variations in peak flow compared to the calibration case of between +10% to -26%. The results for the 2-year ARI sensitivity cases were much more variable with produced variations in peak flow compared to the calibration case of between +133% to -91%.

Appendix B presents further details and outputs from the sensitivity simulations.

7.7 Climate Change Scenario Investigations

An assessment of the peak flows and discharge volumes was undertaken for the climate changes scenarios for 2060 and 2110. Projections for changes in rainfall amounts in relation to cyclonic events range between +3% to +37%. The typical projected changes for rainfall at the end of the twenty-first century are about +20% within 100km of the storm centre (Knutson *et al.*, 2010). This is consistent with general guidance provided by the CSIRO (CSIRO, 2007). The design rainfall patterns for the 2060 and 2110 climate scenarios were maintained but the rainfall totals were adjusted to account for a 10% and 20% increase in rainfall across the two scenarios respectively.

Table 7.2 shows the changes in peak flows and discharge volumes in comparison to predicted design events.

Table 7.2: Climate Change Scenario Comparison of Peak Flow Rates and Discharge Volumes

Location	Peak Flow Rate (m^3/s)			Discharge Volume (GL)			Peak Flow Difference (%)		Volume Difference (%)	
	100yr 12hr	2060	2110	100yr 12hr	2060	2110	2060	2110	2060	2110
Pincunnah Station	4,059	4,680	5,318	198	225	251	15	31	14	27
Port Hedland 2D Model Extent (Turner River)	9,485	9,919	10,363	656	730	795	5	9	11	21

The change in rainfall intensity show increases of peak flows ranging between 5% and 31% and discharge volumes of 11% and 27% for the 2060 and 2110 respectively in comparison to the current ARI design storms.

7.8 Design Events – Peak Flows and Critical Durations

The peak flows and discharge volumes for the critical duration storm events at the 2D hydraulic model extents for Port Hedland and Shellborough regions which are modelled in **Section 8** are shown in **Table 7.3**, **Table 7.5** and **Table 7.5**.

Table 7.3: Design Storm Summary Results of Turner River Entering the Port Hedland 2D Model Extent

ARI Storm Event	2 year	10 year	100 year	200 year	500 year
Critical Storm Duration (hour)	36	36	12	18	12
Peak Flow (m^3/s)	263	2,793	9,485	10,365	11,561
Discharge Volume (GL)	31.2	314.6	655.9	913.4	940.4

Table 7.4: Design Storm Summary Results of South West Creek Entering the Port Hedland 2D Model Extent

ARI Storm Event	2 year	10 year	100 year	200 year	500 year
Critical Storm Duration (hour)	36	36	24	18	18
Peak Flow (m^3/s)	8	172	648	862	1,197
Discharge Volume (GL)	0.6	25.6	75.3	86.8	114.0

Table 7.5: Design Storm Summary Results Entering the Shellborough 2D Model Extent

ARI Storm Event	2 year	10 year	100 year	200 year	500 year
Critical Storm Duration (hour)	36	48	36	30	30
Peak Flow (m^3/s)	16	36	114	159	231
Discharge Volume (GL)	3.1	9.5	24.7	28.9	38.2

The hydrographs of the critical storm events for all catchments entering into and residing within the 2D model extent for both Port Hedland and Shellborough were used as inputs for the 2D hydraulic model. The critical storm events identified for the locations above all have long storm durations and hence, due to the lower intensity, the peak of the storm events occur well after the cyclone has passed. In order to determine the effectiveness of the 2D hydraulic model for short duration high intensity storm events, hydrograph data for the 1.5 hour duration of all modelled ARI storm events were used as the inputs. The peak flows and discharge volumes for a fully urban catchment (147.6ha) within the Port Hedland townsite are shown in **Table 7.6**.

Table 7.6: Design Storm Summary Results for Urban Catchment within the Port Hedland 2D Model Region

ARI Storm Event	2 year	10 year	100 year	200 year	500 year
Critical Storm Duration (hour)	1.5	1.5	1.5	1.5	1.5
Peak Flow (m^3/s)	20	54	88	103	126
Discharge Volume (ML)	43.0	96.7	192.4	226.0	273.8

8 HYDRAULIC MODELLING

8.1 Overview

The hydraulic modelling component of the Port Hedland Vulnerability study was undertaken to achieve two objectives. The first was to act as a hydraulic model to determine the inundation extents from the catchment flows simulated by the hydrological model described in **Section 7**. The second objective was to act as a hydraulic model to simulate in detail the ocean generated inundation for the design ocean inundation events described in **Section 6**. By adopting a single model system to determine the ocean and catchment flooding inundation extents also enables the combined ocean and catchment inundation extents from jointly occurring events to be developed.

The scale of the hydraulic modelling undertaken in this study is very large and the detailed modelling covers the LiDAR survey extents in the Port Hedland and Shellborough areas as presented in **Figures 5.2** and **5.3.** The hydraulic modelling does not extend down to the scale of local drainage, however for key areas such as South Hedland, the local drainage network is included in the modelling. It is envisaged that if assessment down to the local drainage scale is required, the model system developed in this project could be refined to include this detail within the area where the assessment is required.

A summary of the hydraulic modelling is presented in the following sections. **Appendix C** is a detailed technical report for the hydraulic modelling component.

8.2 Model Systems

The Deltares 1D2D modelling system, SOBEK, was used to undertake the hydraulic inundation modelling for the Port Hedland *Coastal Vulnerability Study*. The SOBEK model system uses the same hydraulic model engine as the Delft3D model which was applied in the ocean inundation modelling (**Section 6**) and is able to compute the channel (1D) and overland flow (2D) components of the study. SOBEK is a professional software package developed by Deltares , which is one of the largest independent hydraulic institutes in Europe (situated in The Netherlands) and is world-renowned in the fields of hydraulic research and consulting (WL|Delft, 2005).

This combined package allows for the computation of channel and pipe flow (including structures such as culverts, weirs, gates and pumps, and pipe details such as inverts, obverts, pipe sizes and pipe material) by the 1D module, which is then dynamically linked to the 2D overland flow module. The 1D and 2D domains are automatically coupled at 1D-calculation points (such as manholes) whenever they overlap each other. The model commences with the 1D component operating as the inflow increases until such time as the pipe or channel is full and overflows, with the flow then moving to the 2D domain. The 1D network and the 2D grid hydrodynamics are solved simultaneously using the robust Delft scheme that handles steep fronts, wetting and drying processes and subcritical and supercritical flows (Stelling, 1999).

The advantages of this system are that the channel/pipe system is explicitly modelled as a sub-system within the two-dimensional overland flow computation. This means that generalised assumptions regarding the capacity of the channel/pipe system are not required. This system employs a unique implicit coupling between the one and two-dimensional hydraulic components that provides high accuracy and stability within the computation.

8.3 Addressing Joint Occurrence between Catchment Flows and Ocean Water Levels in Design Events

The analysis of the 20-year rainfall and measured water level data from Port Hedland which is presented in **Appendix C** indicates that for this limited data set no discernable relationship between rainfall and ocean water level joint occurrence can be derived.

In the simulation of design flood events from either catchment or ocean inundation, it is prudent to adopt a risk averse approach and consider some form of joint occurrence between catchment flows (rainfall) and ocean water levels. In Western Australia, there are no guidelines to address the joint occurrence of ocean water levels and catchment flows for flood studies or in the preparation of water management plans. Joint occurrence between catchment flows and ocean water levels are addressed in consultation with Department of Water on a case by case basis.

For the Port Hedland area planning policies have been influenced by the Greater Port Hedland Storm Surge Study (GPHSS) undertaken for the Department of Planning in 2000 (GEMS, 2000). That study concluded that large ocean storm surge events tended not to be associated with peak catchment response. Within that study, in the assessment of flooding in streams and rivers, storm surge was not considered a hydraulic constraint on the water levels in the lower reaches of the streams and rivers. Similarly, recent flood studies and water management studies for South Hedland (GHD, 2010) and the Wedgefield extension (JDA, 2010) did not consider a downstream water level based on normal tidal or extreme event storm surge conditions. For both these studies, downstream water level conditions were derived from the design catchment flow conditions and the potential effects of high ocean water levels neglected.

Elsewhere in Australia and overseas, the joint occurrence of catchment flows and ocean water levels are addressed with a more formal approach, including with government guidelines for flood studies. For example, in NSW the Department of Environment, Climate Change and Water (DECCW) recommends an envelope approach is adopted to consider flooding from catchment and ocean. The typical scenarios involve considering the 100-year ARI catchment flows coinciding with a 20-year ARI ocean water level and vice-versa. In NSW, over the last 15-years flood studies by a range of consultants have adopted a range of joint occurrence scenarios including:-

- 100-year ARI catchment flows coinciding with a 5-year ARI ocean water level and vis-versa;
- 100-year ARI catchment flows coinciding with a 100-year ARI ocean water level and vis-versa; and
- 100-year ARI catchment flows coinciding with a 1% exceedence ocean water level which in NSW is normally just below HAT.

In the context of the large tide range at Port Hedland, this approach is likely to be conservative. In Queensland, whilst no government guideline exists, the joint occurrence of ocean water level and catchment flows have been investigated particularly in the Gold Coast region which is affected by East Coast Low storm systems as well as occasional tropical cyclones. CSIRO (2007) investigated the relationship between daily rainfall totals and residual ocean water levels over a 39-year period. The investigations indicated that for daily rainfalls totals greater than 300mm, there was a positive correlation with residual water levels in the Broadwater. The typical maximum residual in the Broadwater for events greater than 300mm total daily rainfall was about +0.3m. In 2010, Cardno undertook a literature assessment for the Gold Coast City Council into joint occurrence between catchment flows and ocean water levels which considered papers, guidelines and studies from around Australia and overseas. In that study, Cardno suggested a range of joint occurrence models for the Gold Coast Council area including:-

- Adopting a time varying tide with a peak ocean water level of MHWS occurring concurrently with peak catchment flows for large catchments with critical durations > 36-hours; and
- Adopting a time varying tide with a peak ocean water level equal to the 1% exceedence water level which is derived from long-term measured data. This approach may be particularly appropriate for small catchments with short critical durations which may experience a higher correlation between catchment flows and elevated ocean water levels.

Following a steering group meeting on 23 February 2011, and subsequent discussions between the Department of Transport and the Department of Water, an agreed model for the joint occurrence of catchment flows and ocean water levels has been developed specifically for this study. Based on this outcome, for the Port Hedland *Coastal Vulnerability Study*, Cardno have adopted the following joint occurrence model for the design event simulations:-

- Adopt 20-year ARI ocean water level in-conjunction with the 100-year ARI catchment flows. Ocean water levels should be simulated with a realistic time series with the peak ocean water level being coincident with the peak catchment flows. For 100-year ARI and more frequent design events, the ocean water level condition will be based on a design ocean water level with an ARI of one fifth the ARI of the catchment flows.
- For 200-year ARI event and less frequent design events, the ocean water level condition will be based on a design ocean water level with an ARI of one tenth the ARI of the catchment flow. For example, the 500-year catchment flows with the 50-year ocean water level.
- When design ocean water levels are below MHWS for a specific event, a time varying water level time series should be applied on the ocean boundary of the hydraulic model with a peak high water equal to MHWS occurring with the peak catchment flows. At Port Hedland, MHWS is +6.7m LAT (+2.8m AHD).

8.4 Model Setup

The hydraulic models consist of two main components:

- The channel network (1D); and
- 2D grids of the surface topography.

The 1D network aims to improve the performance of the hydraulic model in areas where the 2D model may not accurately represent the catchment. These areas can include townships and areas with defined channels and structures. As no formal drainage infrastructure exists in the Shellborough region, no 1D elements have been used in this area. In the Port Hedland area, 1D elements have been used to define bridges, culverts and channels, especially in the South Hedland and Wedgefield areas.

Within the South Hedland township, the drainage channels have been represented in the 1D network using estimated trapezoidal channels from engineering judgement and a site visit to the region. The structures in South Hedland have all been surveyed and included in the 1D network. The 1D network was defined in this region to improve the accuracy of the inundation mapping in the town.

For the remainder of the Port Hedland model the structures (bridges, culverts etc) have been included in the 1D network and no channels have been represented. The structures have been defined using engineering judgement and from the site inspection. The channels have not been included in the 1D network as the 2D grid adequately represents the wide channels in the region and no survey has been undertaken to capture the channel dimensions. Limited information was available from the Port Hedland Town Authority and Main Roads regarding the structures in the region. The Shellborough model has no 1D network for structures and channels as there are none present in this region.

The two-dimensional, overland flow component of the hydraulic model consists of a main grid that covers the entire study area and two smaller, nested grids covering sections of the South Hedland and Wedgefield township area. The main grid has a resolution of 40 m and the nested grids have a resolution of 10 m. For the general overland floodplain areas within the study area, the 40m grid resolution is considered adequate. However, in the South Hedland and Wedgefield area, where overland flow paths through urban areas were identified during preliminary model simulations, a finer resolution of 10 m was chosen. This provided a better definition of the detailed flood hydraulics through the township.

A nested grid covering the main town area of Port Hedland and extending around to Pretty Pool was modelled at 12m resolution, with varying ocean inundation boundary inputs applied as discussed in **Section 6.7**. This allowed detailed modelling of the flows through the constricted entrance to the Pretty Pool Estuary and good definition of flows across the town area. For the Shellborough region, a grid resolution of 10m was selected.

Figure 8.1 presents a plan view example of the setup for the Port Hedland region hydraulic model. **Appendix C** presents further details on the hydraulic model setup including the specification of spatial roughness values.

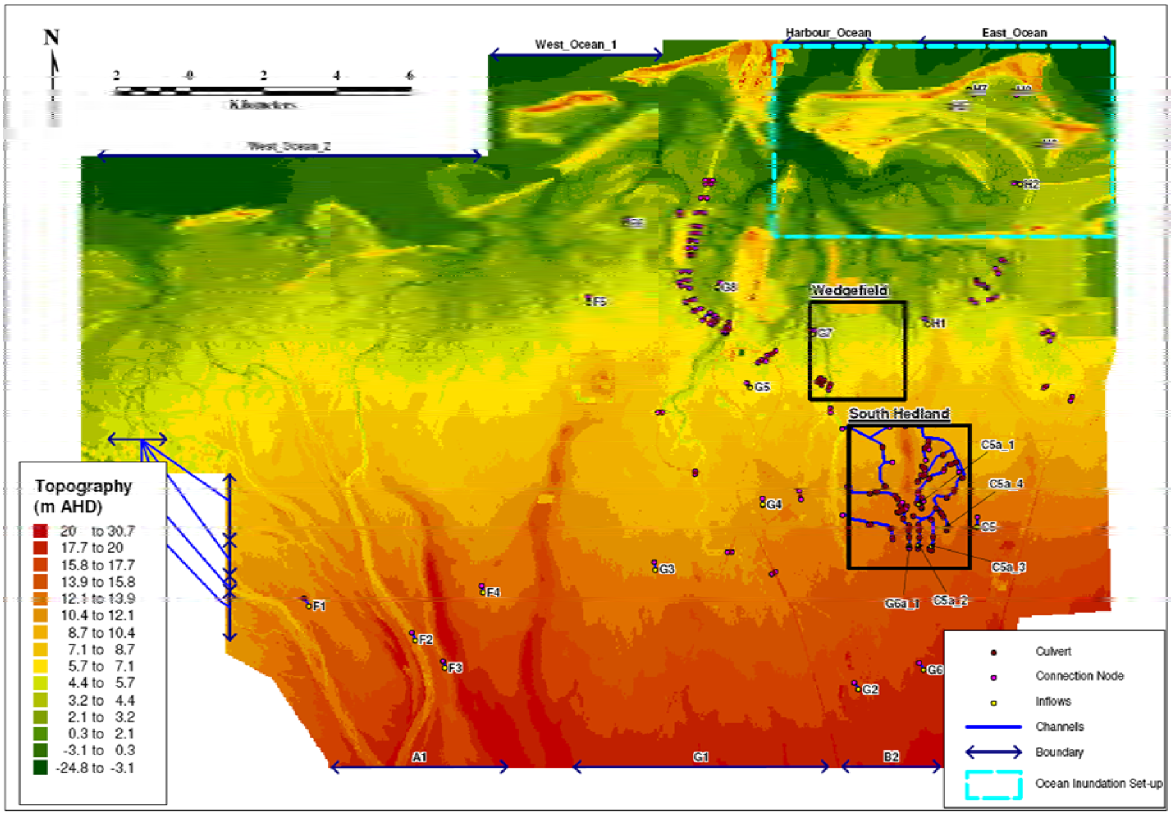

Figure 8.1: Hydraulic Model Setup – Port Hedland.

8.5 Model Calibration

Model validation was carried out for the Port Hedland model. Due to the lack of consistent stream flow information for both the Turner River and South West Creek gauges, a validation exercise was undertaken at the bridge structure located near the corner of Greater Northern Highway and Hamilton Road over South Creek as shown in **Figure 8.2**. Levels and flow data from an event in March 1989 recorded by main Roads was provided to Cardno by the Department of Planning (DoP). The event was estimated to be a 6 hour, 100-year ARI storm. Cardno is unable to verify the accuracy of this report as Main Roads was unable to provide documentation detailing the observations taken during the event. It should also be noted that Cardno did not receive any design plans of this structure from Main Roads and it was not surveyed as part of this project.

The information received from DoP for the March 1989 event indicated that the upstream and downstream water level at the structure was 7.770 m AHD and 7.580 m AHD respectively with flow rates estimated at 230 m^3/s. Cardno utilised the design flows for the 100-year ARI and 6-hour duration as inputs to the hydraulic model for validation purposes. Peak inflows for the major river and creek systems were:

- Turner River : 7252 m^3/s
- South Creek : 402 m^3/s
- South West Creek : 120 m^3/s

Table 8.1 shows the comparison between observed and modelled results.

Table 8.1: Calibration Results at Bridge Structure E_56

Parameters	Estimated 100-year, 6-hour Event – Main Roads	Modelled 6 hour 100-year ARI event
Upstream Water Level	7.77 m AHD	8.35 m AHD
Downstream Water Level	7.58 m AHD	7.70 m AHD
Flow at Bridge	230 m^3/s (estimated)	268 m^3/s

The validation assessment shows that the design 100-year, 6 hour event produces higher flows than the 1989 event. This is reflected in both the higher flow estimate at South Creek and the higher water levels. The flood model estimates that the flow capacity of the bridge at the soffit is in the order of 200 m^3/s. The soffit level is approximately 7.75 mAHD. Pressure flow effects are then experienced with a full flow capacity of approximately 250 m^3/s at the bridge before overtopping of the Great Northern Highway. The 200 m^3/s estimated at the bridge for a water level of 7.75 m is within 10% of the estimate of Main Roads for the 1989 event.

As the exact temporal and spatial distribution of the 1989 flood event cannot be reproduced, it is considered that the hydraulic model result is consistent with the known flood information.

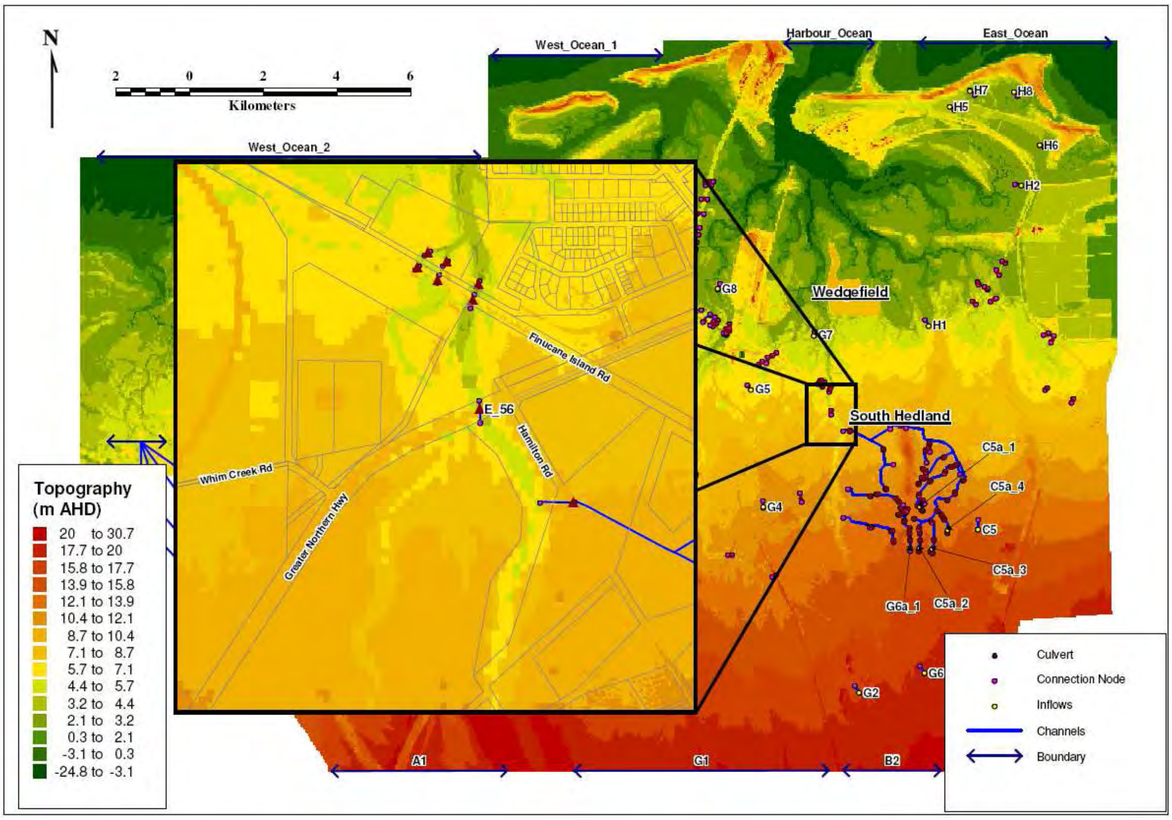

Figure 8.2: Hydraulic Model Calibration Site.

8.5.1 Summary of Flood Studies from the Port Hedland Region

A number of previous studies have been carried out in the Port Hedland region, however none have been completed that cover the total geographical area of this project. These studies have been undertaken for various projects in the vicinity of Port Hedland. Cardno has reviewed the following studies:-

- Fortescue Metals Group - Pilbara Iron Ore And Infrastructure Project Stage A Port And North-South Railway Surface Hydrology, undertaken by Aquaterra Consulting, 2004.
- Landcorp, Wedgefield Industrial Estate Extension, Port Hedland Local Water Management Strategy, undertaken by JDA Consultant Hydrologists, 2010.
- Town of Port Hedland - South Hedland Flood Study, undertaken by GHD 2010.
- Ministry of Planning, Greater Port Hedland Storm Surge Study, undertaken by GEMS, 2000.

These reports have all used the GEMS (2000) report as a baseline for comparison of their hydrological approaches. Most of the hydrological information is assessed for South Creek at the bridge crossing at the Great Northern Highway. JDA (2010) reports that the estimated flows at this location are between 269 and 777 m^3/s depending on the estimation method. The methods used for each estimate are relatively simple area based relationships and as such do not take into account cross-catchment flows, routing of flow and other hydraulic constraints, such as roads and channel conditions in their assessment. The JDA (2010) report adopted the 269 m^3/s level as the design flow rate, which was estimated using the Index Flood Method, in accordance with the procedures described in Australian Rainfall and Runoff (AR&R). The catchment modelling method used in this report is usually more accurate than the area based relationships according to AR&R. The JDA (2000) report also recommends that flood assessment be undertaken using a 2D model to obtain greater certainty in flood results.

The Greater Port Hedland Storm Surge Study (GEMS, 2000) estimated flows in South Creek as 383 m^3/s although the methodology used in this assessment is not clear. In the South Hedland Flood Study, GHD (2010) predicted a 100-year ARI flow at South Creek of 162 m^3/s, using a larger upstream catchment than that defined in the GEMS (2000) report. Although GHD (2010) considered the GEMS (2000) result overly conservative, they adopted it for use in their report. The Aquaterra (2008) report also adopts the flood flows reported in GEMS (2000), but cautions that for South Creek, the flows may be underestimated due to the impact of cross catchment inflows from South West Creek.

This report adopts a different approach to those defined above. A full hydrological model of the upstream catchments has been created and used as input to a large scale 2D model of the entire floodplain. Cross catchment flows are catered for in the hydraulic model as are the effects of bridges, culverts, roadways and other hydraulic constraints. The modelling undertaken by Cardno indicates that in the peak 100-year ARI event, model inflows to South Creek and South West Creek are 666 m^3/s and 212 m^3/s, respectively. There are additional local catchment inflows upstream of the Greater Northern Highway of 270 m^3/s. It should be noted that these flows do not occur simultaneously. The hydraulic model results from this study indicate that the peak flow at the Greater Northern Highway at South Creek is in the order of 410 m^3/s (290 m^3/s through the bridge and 120 m^3/s over the highway). However, catchment flows break out from the floodplain to the north of this area. The modelled flow rate from this study is close to the GEMS (2000) estimate of 383 m^3/s and accounts for the full range of cross catchment flows and floodplain storage. Modelled flood levels will vary from previous reports based on location but this report provides more realistic representation of the flood behaviour than previous studies.

8.6 Design Event Simulations

The Port Hedland and Shellborough hydraulic model has been run for Catchment Inundation (CI) and Ocean Inundation (OI). The modelling included assessment of the potential impacts of climate change at 2060 and 2110. The OI runs have an additional component which investigates the impact of wave setup on the coast.

Table 8.2 summaries the design event simulations undertaken for the project in the Port Hedland region including the reference to the appropriate **Flood Maps** at the end of this report. For the climate change scenario cases, 2060 and 2110 scenarios were simulated.

Table 8.2: Hydraulic Model Design Simulations - Port Hedland

Model Simulation	Catchment ARI Event	Ocean ARI Event	Climate Condition	Port Hedland Region Flood Maps
CI	500	50	2010	P1 - P2
CI	200	20	2010	P3 - P4
CI	100	20	2010	P5 - P6
CI	10	MHWS	2010	P7 - P8
CI	2	MHWS	2010	P9 - P10
CI	500	50	2060	P11 - P12
CI	100	20	2060	P13 - P14
CI	2	MHWS	2060	P15 - P16
CI	500	50	2110	P17 - P18
CI	100	20	2110	P19 - P20
CI	2	MHWS	2110	P21 - P22
OI*	-	500	2010	P23
OI*	-	200	2010	P24
OI*	-	100	2010	P25
OI*	-	10	2010	P26
OI*	-	500	2060	P27
OI*	-	100	2060	P28
OI*	-	500	2110	P29
OI*	-	100	2110	P30

* Shoreline wave set-up included for open coast shorelines in the Port Hedland township area

Table 8.3 summaries the design event simulations undertaken for the project in the Shellborough region including the reference to the appropriate **Flood Maps** at the end of this report. For the climate change scenario cases, 2060 and 2110 scenarios were simulated.

Table 8.3: Hydraulic Model Design Simulations - Shellborough Region

Model Simulation	Catchment ARI Event	Ocean ARI Event	Climate Condition	Shellborough Region Flood Maps
CI	500	50	2010	S1
CI	200	20	2010	S2
CI	100	20	2010	S3
CI	10	MHWS	2010	S4
CI	2	MHWS	2010	S5
CI	500	50	2060	S6
CI	100	20	2060	S7
CI	2	MHWS	2060	S8
CI	500	50	2110	S9
CI	100	20	2110	S10
CI	2	MHWS	2110	S11
OI*	-	500	2010	S12
OI*	-	200	2010	S13
OI*	-	100	2010	S14
OI*	-	10	2010	S15
OI*	-	2	2010	S16
OI*	-	500	2060	S17
OI*	-	100	2060	S18
OI*	-	2	2060	S19
OI*	-	500	2110	S20
OI*	-	100	2110	S21
OI*	-	2	2110	S22
OI	10	50	2010	S23
OI	10	50	2060	S24
OI	10	50	2110	S25

* Shoreline wave set-up included for open coast shorelines at the Shellborough site.

The two-dimensional, overland flow results are reported as depths and levels (m and m AHD) and flow velocities (m/s) for every grid cell at regular time intervals. Time series of water level, depth and flow velocity were also reported at specific locations. **Appendix C** contains more details and summaries of flood levels at specific locations.

It should be noted that the flood extents shown are a representation only of the actual flooding conditions in the catchment. The flood extents are based on the DEM developed for use in the project (**Section 2**) and do not include consideration of features such as minor piped drainage, localised flow obstructions (such as parked cars, telephone poles and small embankments) or other topographical features that are smaller than the grid cell definition.

8.6.1 Design Event Simulations Results: Port Hedland

Flood Maps P01 to **P30** at the end of this report describe the flood inundation extents and depths and water in the greater Port Hedland region for the design scenarios presented in **Table 8.2**. Note that all figures have been filtered to remove a flood depth less than 0.02 m. Water surface levels to mAHD are also shown at a number of key locations.

In the greater Port Hedland area, flood flows in the Turner River system do not interact with flood flows in the South Creek/South West Creek systems. There is significant cross catchment flows between South Creek and South West creek under all modelled flood events. The township of South Hedland is impacted by flows from South Creek and from local runoff. Various topographical features cause significant ponding of floodwater and provide barriers to flow conveyance, including the FMC railway line, the Greater Northern Highway and Willowbank Road. Significant flooding is noted in Wedgefield and South Hedland for the 100-year ARI flood event.

Flood inundation in the Port Hedland Township is primarily a result of ocean inundation events. Along the coastline of the township, the 500-year ARI Ocean storm event is expected to breach the dune system near Stevens Street. Fincuane Island Road is also overtopped in this event. In general, flood levels to the south of Wedgefield are controlled by ocean inundation events.

8.6.2 Design Event Simulations Results: Shellborough

Flood Maps S01 to **S24** at the end of this report describe the flood inundation extents and depths and water in the Shellborough region for the design scenarios presented in **Table 8.3**. Note that all figures have been filtered to remove a flood depth less than 0.02 m. Water surface levels to mAHD are also shown at a number of key locations.

During the 100 year ARI storm event with existing climate conditions, the majority of the Shellborough area is inundated. In the absence of a combined event, the 100 year tidal inundation results in far greater impact on the region. This is due to the majority of the Shellborough region being very low lying with little to no protection from ocean. Flood depths are significant in all modelled events. It is unlikely that any development could occur on the Shellborough Estuary, although parts of the dune system are not inundated in large storm events. The old Shellborough Township site remains free from flooding 100 year event; however no access to this area is available.

8.7 Development Area Analyses – East Port Hedland

Landcorp have provided a preliminary map of proposed development locations in Port Hedland - see **Figure 8.3**. This plan has been incorporated in the hydraulic model to assess the required planning levels for the development of lots. A worst case assessment has been made in the model, by raising all the land in the development area above the 2110 climate change ocean inundation with wave setup level. Using this methodology, the impact of the development on flooding can be assessed. In the area of development, peak levels are a result of the ocean inundation events and as such, only these events have been modelled. The 100 and 500 year events have been used for the assessment and include wave setup and results at presented as **Flood Maps P31** and **P32**. Climate change conditions at 2060 and 2110 (with wave setup) have also been assessed. **Maps P33** and **P34** show the 2110 inundations levels at the development.

Figure 8.3: East Port Hedland Infill Development Area.

Along the coastline of the township, the 500-year ARI Ocean storm event for the present climate condition, and the 100-and-500 year ARI events for the 2110 climate condition, breach the dune system near Stevens Street on the open coast which can potentially flood levels at the East Port Hedland site to the east. As a result of the infill development, the Pretty Pool estuary which previously provided a large storage area for water which came through the breach location near Stevens Street, no longer provides the same the storage capacity. Because of this the flood levels at the western edge of the East Port Hedland development increase. In the concept design for this site, the potential for ocean inundation from the open coast through the low-lying region near Stevens Street will need to be addressed.

8.8 Sensitivity Cases – Port Hedland

A number of sensitivity cases for combined catchment and ocean inundation were assessed and are presented on **Flood Maps P41** to **Map P45**. A sensitivity case showing the 100 year flood extent with a continuing loss is presented in **Flood Map P43** which is discussed in greater detail in **Appendix B**.

On **Flood Map P44** the outcome from the application of current hydrology combined with the 2110 ocean inundation level is presented. A difference map of this case against the 2110 hydrology and 2110 ocean condition is presented on **Flood Map P45.** The difference map shows negligible differences (< 0.05m) across the main Port Hedland town area and 0.05m to 0.1m difference through the South Hedland area.

Finally a difference map outlining the changes in inundation associated with the ARI500 and ARI100 events under the 2110 climate condition is shown on **Flood Map P42.** The difference map indicates that there are significant impacts (>0.5m) through the region of the main Port Hedland town area associated with the increased ARI condition.

9 SHORELINE STABILITY ASSESSMENT

9.1 Overview

The following sections present an overview of the methodology and outcomes from the shoreline stability assessment. **Appendix D** is a detailed technical document which presents a complete description of the investigation and analysis of the historical aerial datasets and the use of modelling systems.

9.2 State Coastal Planning Policy SPP2.6

The Western Australian Planning Commission (WAPC) released SPP 2.6 in June 2003 to assist land use planning and development issues specifically as they relate to the protection of the coast. **Schedule One** of SPP 2.6 provides direction for calculating the appropriate Coastal Processes Setback (CPS) distance for the new development on the Western Australian coast.

The intention of the CPS is to provide a buffer zone between the shoreline and development in which coastline changes in the short term (severe storms), the medium term (shoreline movement) and the longer term (sea level rise and fluctuation of natural processes) can occur.

The calculation of the CPS distance is based on the combined result of the following factors :-

1. (S1) Distance For Absorbing Acute Erosion (Extreme Storm Sequence)
2. (S2) Distance to Allow for Historic Trend (Chronic Erosion or Accretion)
3. (S3) Distance to Allow for Sea Level Change

SPP 2.6 states that the CPS should be defined on a case by case basis including S1, S2, S3 where relevant. Each of these factors is assessed in this report by the following methods :-

- S1. Selected profile modelling of storm erosion using a model system such as SBEACH;
- S2. Analysis of historical photogrammetric data and aerial photography to establish historical shoreline changes; and
- S3. Application of the Bruun Rule based on a vertical SLR of 0.30m in 2060 and 0.90m for the year 2110.

Additionally, regions north of latitude 30° are deemed cyclone prone areas under SPP2.6 and the CPS needs to take account of areas potentially inundated by storm surge associated with a Category 5 cyclone. The results from the detailed modelling undertaken in the Ocean Inundation Assessment **(Appendix A)** have been utilised to inform this process.

9.3 Study Methodology

9.3.1 Acute Storm Erosion (S1)

Short term erosion at the study sites was investigated using the SBEACH numerical model. SBEACH models the impact to the beach profile resulting from a severe storm event and shows the erosion and deposition of sand as large waves and elevated water levels reshape the shoreline. Consequent loss of beach area following large storm events is investigated to provide a measure of the beach area at risk from short term storm events (S1). A series of shoreline profiles were established across the study sites and subjected to design water levels and wave heights representative of a Category 5 cyclone. Full details of SBEACH profile locations and results are available in **Appendix D.**

9.3.2 Historical Shoreline Changes (S2)

Aerial datasets dating back 60 years were obtained for the study sites for assessment of the historical movement of the shoreline position.

For the Port Hedland site, nine aerial datasets flown in the period 1949 to 2009 were sourced from Landgate imagery. Photogrammetric analysis was undertaken on the ortho-rectified aerial images to measure relative changes in the shoreline position between each of the survey periods. The average rate of shoreline change was calculated to provide an annualised measure used as the basis for the historical trend component (S2) applied in assessment of the CPS.

For the Shellborough and Site 2 study areas, seven sets of aerial imagery across the years 1949 and 2007 were approximately ortho-rectified. At Shellborough the movement in the historical shoreline position was investigated over the available data period. The mangrove cover at Site 2 was mapped across the 60 years of historical data, to monitor changes to the overall appearance of the mangrove extents. Full details of the analysis of the historical datasets is available in **Appendix D**.

9.3.3 Sea Level Change (S3)

In 2010 the magnitude of sea level rise recommended for coastal setback planning in WA was updated in SPP2.6 for planning periods up to 100 years. For the 100 year planning timeframe (2010 to 2110) DOT recommended a vertical SLR of 0.9m be adopted, whilst in the 50 year planning timeframe 0.3m vertical SLR is appropriate.

SPP2.6 recommends the Bruun rule (Bruun 1962) be used for calculation of a setback distance based on the vertical SLR component. For sandy shores a multiplication factor of 100 is applied under the Bruun rule. For the 100 year planning timeframe a vertical SLR of 0.9m results in a horizontal setback distance (S3) of 90m. For the 50 year planning timeframe a setback distance (S3) of 30m is applicable.

For other shore types the setback distance S3 is assessed with regard to local geography. In the case of rocky shorelines, the S3 component is accounted for in the default CPS value of 50m.

9.3.4 Inundation Extent

Inundation levels resulting from extreme cyclonic events were investigated as part of the Ocean Inundation Assessment **(Appendix A).** The combined effect of elevated ocean levels caused by storm surge and extreme flooding from land based runoff were modelled to determine regions within the study areas at risk of inundation.

Inundation levels within the study areas were calculated based on the impact of a category 5 cyclone which represents a 500 year ARI case recommended under the guidelines in SPP 2.6. The lower likelihood 100 year ARI event was also modelled. The inundation levels were modelled for the current climate condition and the 2060 and 2110 condition based on prescribed climate change and sea level rise predictions.

9.4 Results

For Port Hedland the Coastal Processes Setback levels for the present climate condition and 2110 Planning Periods are summarised on **Figure 9.7** to **Figure 9.8.** Results for 2060 are available in **(Appendix D).**

Figure 9.1: Port Hedland Coastal Processes Setback Summary - Immediate Setback Level.

Figure 9.2: Port Hedland Coastal Processes Setback Summary -2110 Setback Level.

For the Shellborough Site the Inundation levels for the present climate condition and 2110 Planning Periods are summarised on **Figure 9.3** and **Figure 9.4.** Results for 2060 are available in **(Appendix D).**

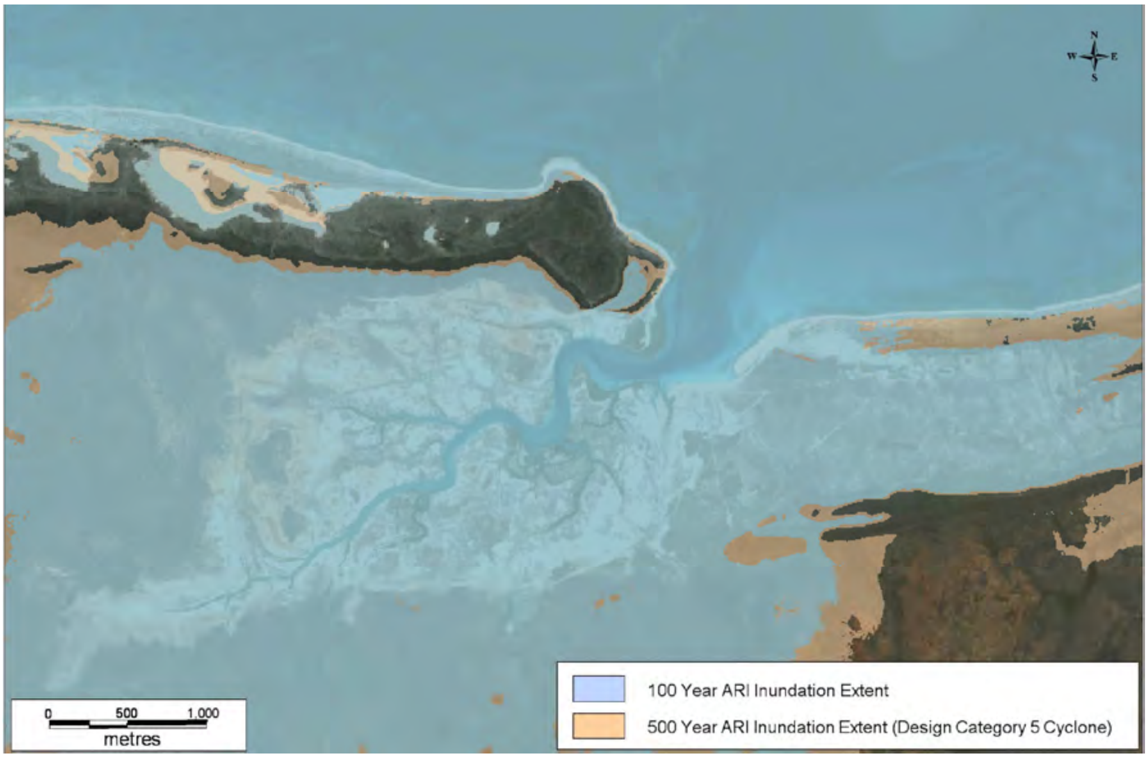

Figure 9.3: Shellborough Inundation Extents -Immediate Level (2010).

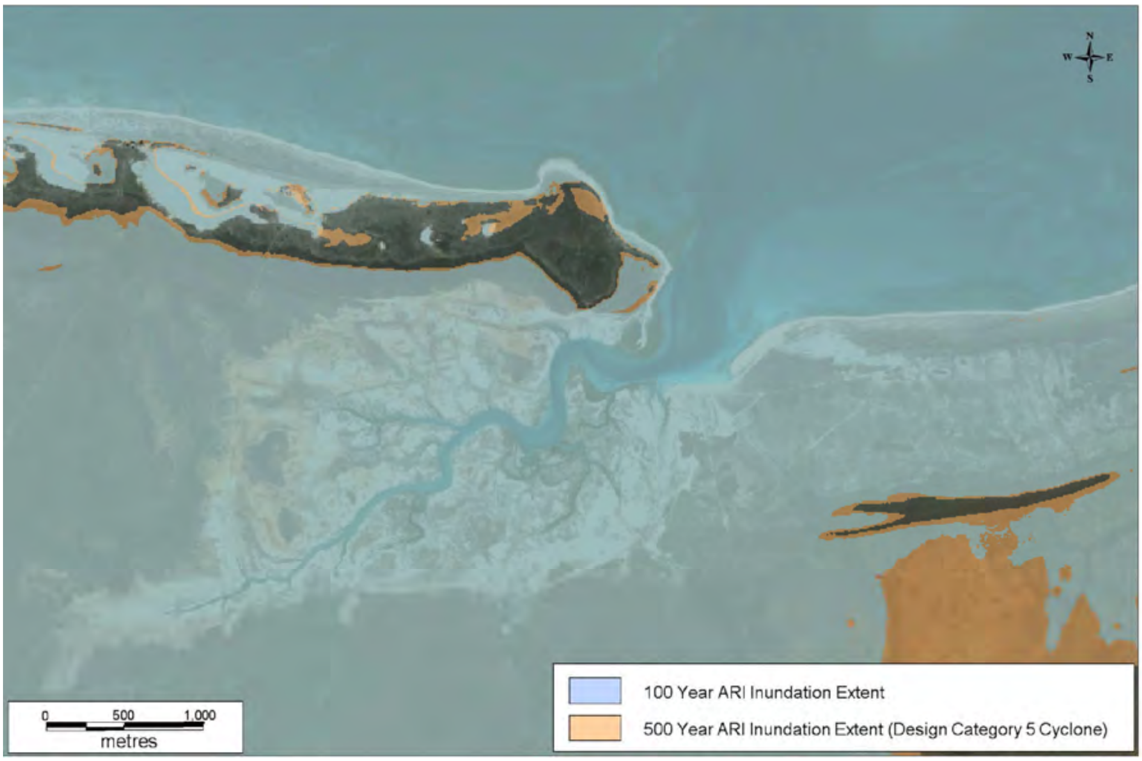

Figure 9.4: Shellborough Inundation Extents -100 Year Planning Period (2110).

The inundation extents for Site 2 are shown on **Figure 9.5** for the present climate condition with no SLR for a 100 year ARI event.

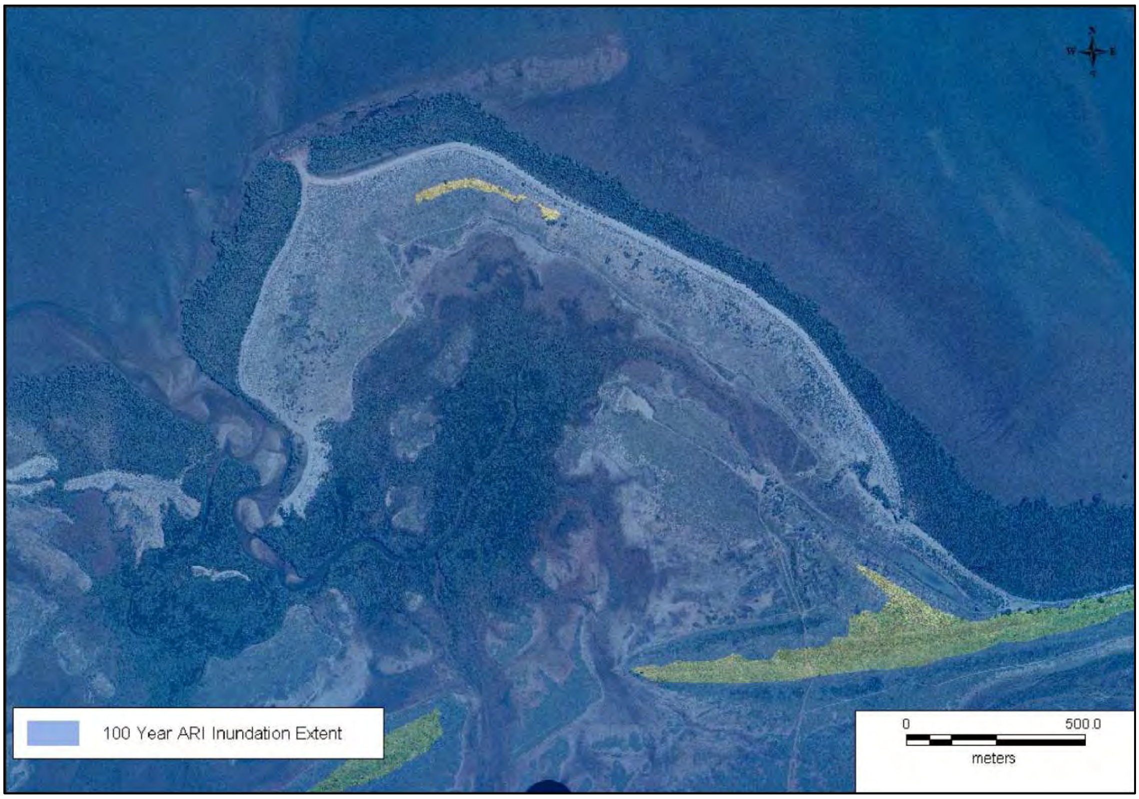

Figure 9.5: Site 2 100 Year ARI Inundation Extents (including shoreline wave setup) - Immediate Level (2010)

9.5 Conclusions

The following conclusions are made from the outcomes of this report:-

- There is a low lying section of shoreline east of the spoil bank (approximately 6 mAHD) which serves as a break through point for flows from the open coast which includes a shoreline wave setup contribution in extreme events. This break point directs water across currently existing properties and into lower lying land in East Port Hedland as well as the main business area of Port Hedland and increases the inundation area and depths in these areas.

- The Spoil Bank is inundated in both the 100 year and 500 year ARI event, with only a small portion left unaffected. Wave run-up results in even greater inundation potential for structures on the Spoil Bank.

- The region of East Port Hedland is at risk of inundation in the immediate term as flows from Pretty Pool estuary and the Four Mile creek Estuary flow into this low lying region. At the most extreme 500 year ARI event, flows are directed from the breakout point east of the spoil bank to further contribute to inundation levels. It should be noted however that even with the break through, the inundation water levels in East

Port Hedland are lower than on the open coast as a result of the limited volume of water which can flow through the break through channel.

- The Pretty Pool development remains above the 500 year inundation level (Category 5 Cyclone) in the immediate term as well as the 50 year and 100 year planning periods.

- For the 100 year planning period when adopting a sea level rise component of 0.9m and modelling increased storm intensity, there is a significant impact to the main business area of Port Hedland and near the East Port Hedland area with the present landform levels. For the 100 year ARI event the inundation level is such that the business area of Port Hedland and surrounding streets are completely inundated.

- In the calculation of potential shoreline recession due to SLR in this study, the potential presence of possible underlying rock which limits the landward movement of the beach profile has not been considered in this study. If more detailed setback distance analyses are required, for example for a particular parcel of land, it is recommended that a geotechnical site investigation be undertaken to determine the underlying sediment composition of the site and then assess the potential for shoreline recession due to SLR if underlying rock is present.

- Site 2 is at significant risk of inundation in the 100 year ARI event in present conditions, with only a small portion of the ridge above the water level. For the 500 year ARI design the site is completely inundated. In the 50 year and 100 year planning periods with additional sea level rise this site is completely inundated in all design storm events. Severe property and infrastructure damage would likely occur at this site under the design cyclone conditions.

- The Shellborough site is particularly low lying through the region adjacent Condon Creek, as well as the entire region east of the creek mouth and these areas are inundated in the 100 year ARI and 500 year ARI event. There is a high section of dune which extends from the mouth of the creek to the west and this remains above the 500 year ARI inundation extent in the present term and in the 100 year planning horizon (2110).

10 CONCLUSIONS

Cardno was commissioned by Landcorp to undertake a *Coastal Vulnerability Study* for the Port Hedland region to inform future planning and development decisions in the region. The *Coastal Vulnerability Study* is a critical study to identify development opportunities and constraints for the Port Hedland region to meet the infrastructure requirements as the population doubles over the coming 15 years. The *Coastal Vulnerability Study* will be utilised by a range of government departments including Landcorp, Department of Planning, Department of Transport, Department of Water and the Town of Port Hedland.

The study area is broad and covers the Port Hedland, Wedgefield and South Hedland region, together with two sites to the west of Port Hedland – Sites 1 and 2, and also the Shellborough site approximately 85km northeast of Port Hedland. The study area covers a range of land use including mining and industrial land use, together with residential and commercial development areas. A particular focus of the study area is to inform the design of two potential development areas at Port Hedland, East Port Hedland, and the Spoil Bank Marina. Design water levels have been specifically prepared for these two sites.

The Port Hedland Coastal Vulnerability Study involves four inter-linked study components which addressed ocean inundation (**Section 6**), catchment hydrology (**Section 7**) and combined catchment/ocean inundation (**Section 8**), and a shoreline stability assessment consistent with State Planning Policy 2.6 (**Section 9**). Flood maps for selected design scenarios between 2 and 500-years ARI are presented at the end of this report for the Port Hedland and Shellborough regions.

Cardno has developed a comprehensive model system which can simulate wind, atmospheric pressure, tide, rainfall and overland flow to investigate ocean inundation and catchment flooding. The study has considered the impact of jointly occurring elevated ocean water levels and catchment flooding. The whole of catchment hydrological modelling together with the large scale hydraulic modelling undertaken in this study has simulated a large volume of cross-catchment flow near South-West and South Creeks during the large design events (100-years ARI or greater).

Data gaps, in particular the limited stream flow, spatial rainfall data and near shore bathymetric data within the study area contributes to uncertainty in the modelling outcomes. In the case of the Shellborough site, the lack of long-term tidal data and reliable nearshore bathymetric data has led to the adoption of a +0.5m factor for design ocean water levels to address the specific uncertainty in determining ocean inundation levels at that site.

Climate change has been addressed in this study and appropriate Western Australian guidelines and recent scientific literature has been utilised to develop 2060 and 2110 climate scenarios which include changes to rainfall, cyclone wind speeds and sea level.

It should be noted that **Appendices A** to **D** also contain conclusions and recommendations specific to each component of the study. A large amount of spatial GIS data has been developed for this project including layers which define flood inundation and hazard (velocity x depth). Upon finalising this report, Cardno will provide a GIS data set to Landcorp which include flood and coastal hazard information to assist government agencies.

11 REFERENCES

Australian Hydrographic Service (2009). Australian National Tide Tables 2009.

BoM (2011). "Storm Surge Preparedness and Safety". Available at: http://www.bom.gov.au/cyclone/about/storm-surge/storm-surge.shtml. Accessed on 22 February 2011.

Cardno (2010): "Literature Review – Joint Probability". Prepared for Gold Coast City Council, May 2010. LJ8915.

CSIRO (2007). "Climate Change in Australia – Technical Report.". Prepared by CSIRO and Bureau of Meteorology. ISBN 9781921232947.

CSIRO (2007): The Impact of Climate Change on Extreme rainfall and Coastal Sea Levels over South-East Queensland – Phase 3: Storm Surge Modelling for Climate Change. Report Prepared for Gold Coast City Council.

Chow, (1973) *Open-Channel Hydraulics*, McGraw-Hill International Editions, Singapore.

D'Adamo N, Fandry C, Buchan S (2009). Northern sources of the Leeuwin Current and the "Holloway Current" on the North West Shelf. J Roy Soc West Aust (in press).

DECCW (2010). "Flood Risk Management Guide - Incorporating sea level rise benchmarks in flood risk assessments". Published by Department of Environment, Climate Change and Water. August 2010. ISBN 978 1 74232 921 5

Deltares, (2011) *Sobek Advanced Version 2.12.002.* Deltares, Delft, The Netherlands.

DEWHA (2007). A characterisation of the marine environment of the north-west marine region, summary of an expert workshop convened in Perth, Western Australia, 5-6 September, Department of the Environment, Water, Heritage and the Arts. Commonwealth of Australia, Barton. (http://www.environment.gov.au/coasts/mbp/publications/north-west/nwcharacterisation.html).

Egbert G.D. and Erofeeva S.Y. (2002). Efficient inverse modeling of barotropic ocean tides. Journal of Atmospheric and Oceanic Technology, vol.19, February 2002.

Feng, M, Meyers, G, Pearce, A & Wijffels, S (2003). Annual and interannual variations of the Leeuwin current at 32°S. Journal of Geophysical Research-Oceans, vol. 108, no. C11, 3355, doi: 10.1029/2002JC001763.

Feng, M, Biastoch, A, Böning, C, Caputi, N & Meyers, G (2008). Seasonal and interannual variations of upper ocean heat balance off the west coast of Australia. Journal of Geophysical Research-Oceans, vol. 113, C12025, doi:10.1029/2008JC004908.

Global Environmental Modelling Systems (2000) Greater Port Hedland Storm-surge Study. Final Report to WA Ministry for Planning and Port Hedland Town Council October 2000.

GHD (2010). "South Hedland Flood Study". Prepared for the Town of Port Hedland by GHD Pty Ltd.

Goda (2000): Random Seas and Design of Maritime Structures. *Advanced Series on Ocean Engineering – Volume 15*. World Scientific, Singapore. ISBN-13 978-981-02-3256-6.

JDA (2010). Wedgefield Industrial Estate Extension, Port Hedland – Local Water Management Strategy (LWMS). Prepared for Landcorp, July 2010.

The Institute of Engineers, Australia, 1987, *Australia Rainfall and Runoff: A Guide to Flood Estimation*, Volume 2, Ed. R.P. Canterford, The Institute of Engineers, Australia, Barton, ACT.

The Institute of Engineers, Australia, 2001, *Australia Rainfall and Runoff: A Guide to Flood Estimation*, Volume 1, Ed. D.H. Pilgrim, The Institute of Engineers, Australia, Barton, ACT.

Knutson, T.R., McBride, J.L., Chan, J., Emmanuel, K., Holland, G., Landsea, C., Held, I., Kossin, J.P., Srivastava, A.K. & Sugi, M., 2010, Tropical cyclones and climate change, *Natural Geoscience*, Vol. 3, pp. 157-163.

Masselink G. and Pattiaratchi C.B. (2001). Characteristics of the sea breeze system in Perth, Western Australia, and its effect on the nearshore wave climate. Journal of Coastal Research, 17, p173-187.

NTC-BOM (2009). "Australian Baseline Sea Level Monitoring Project: Annual Sea Level Data Summary Report – July 2008 to June 2009." Prepared by National Tidal Centre and Bureau of Meteorology for the Australian Greenhouse Office.

Rodriguez, E., Morris, C.S., Belz, J.E., Chapin, E.C., Martin, J.M., Daffer, W. & Hensley, S., 2005, *An assessment of the SRTM topographic products*, Technical Report JPL D-31639, Jet Propulsion Laboratory, Pasadena, California, 143 pp.

Smith, B. & Sandwell, D., 2003, Accuracy and resolution of shuttle radar topography mission data, *Geophys. Res. Lett.*, 30, doi: :10.1029/2002GL016643.

Stelling, G.S. Kernkamp, H.W.J and Laguzzii M.M., (1999) *Delft Flooding System - A Powerful Tool for Inundation Assessment Based Upon a Positive Flow Simulation*, *Hydroinformatics Conference, Sydney NSW.*

van Vledder G, Goda Y, Hawkes P, Mansard E, Martin M, Mathiesen M, Thompson E F and Peltier E (1993). "Case studies of Extreme Wave Analysis: a Comparative Analysis." *Ocean Wave Measurement and Analysis – Proceedings of the Second International Symposium.* New Orleans, Louisiana.

WAPC (2010): Position Statement – State Planning Policy No 2.6 – State Coastal Planning Policy Schedule 1 Sea Level Rise. Prepare by the WAPC and the Department of Planning, WA. September 2010.

Flood Maps

Port Hedland Region

Flood Maps

Shellborough Region

Appendix A

Ocean Inundation Modelling

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Final

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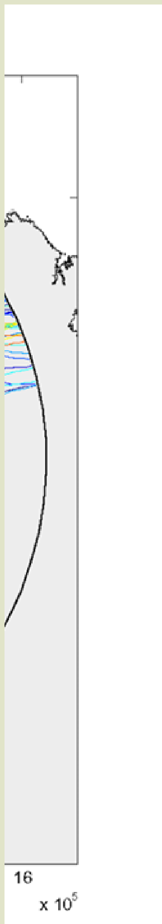

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DISCLAIMER

The State Government and the Town of Port Hedland have a shared vision that will see Port Hedland transformed into Pilbara's Port City with a population of 50,000 by 2035.

Land availability, and in particular developable land above predicted future flood levels, is a clear constraint. In recognition of this the Royalties of Regions program has funded the preparation of the Port Hedland Coast Vulnerability Study. The Study considers the combined impact of flood from rainfall and storm surge. The Study was undertaken by Cardno Pty Ltd and was overseen by a Steering Committee that included the Department of Water, the Department of Transport and the Department of Planning.

The report is not a statutory document. However, the reports findings will be considered by statutory agencies in assessing future development proposals under the Town of Port Hedland's Town Planning Scheme and the WAPC's Statement of Planning Policy 2.6.

Developers and their consultants may use the report and its data for their own works but must undertake their own independent verification specific to their development proposal.

Document Control: Port Hedland Coastal Vulnerability Study – Ocean Inundation Modelling

Version	Date	Author		Reviewer	
		Name	**Initials**	**Name**	**Initials**
1 – Preliminary Draft	14 February 2011	David Taylor	DRT	David van Senden	DvS
2 - Draft	16 March 2011	Paul Branson	PMB	David Taylor	DRT
3 - Draft	5 April 2011	Paul Branson	PMB	David Taylor	DRT
4 - Final	10 August 2011	Paul Branson	PMB	David van Senden	DRT

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ABBREVIATIONS:

ADCP	Acoustic Doppler Current Meter (Current and Wave Measurement Instrument)
AHD	Australian Height Datum which is the standard vertical elevation datum for Australia. At Port Hedland, AHD is +3.902m above Chart Datum at the permanent tide gauge.
ARI	Average Recurrence Interval (years)
AWAC	Acoustic Wave and Current Instrument (Current and Wave Measurement Instrument)
BoM	Bureau of Meteorology
CD	Commonly referred to water level datum which at Port Hedland is approximately equal to the Lowest Astronomical Tide (LAT) level.
DoT	Department of Transport
PHPA	Port Hedland Port Authority
SPP 2.6	Refers to Statement of Planning Policy No. 2.6 which is referred to as the State Coastal Planning Policy which was Gazetted in June 2003.
TSWL	Total Still Water Level

INTRODUCTION

The following report is a technical appendix document to the *Port Hedland Coastal Vulnerability Study Report*. This appendix specifically addresses technical aspects of the ocean inundation modelling including a description of the model systems, setup, calibration, methodology for the design simulations and results. The overall report for the *Port Hedland Coastal Vulnerability Study* provides an overview of the whole project including details on the scope, data sets, outcomes from the various modelling components of the study, and the coastal vulnerability assessment.

A.1 MODEL SYSTEMS

The ocean inundation assessment for the Port Hedland Coastal Vulnerability Study has adopted an integrated wind field, wave and hydrodynamic modelling system to simulate coastal water levels associated with the effects of cyclone wind, surface waves, tide and bathymetric forcing. The model system includes the Delft3D hydrodynamic model, SWAN wave model and a modified Holland cyclone wind and pressure model. Specific details on each of these model systems are presented in the following sections.

A.1.1 Wind field Model

The surface wind field under a moving cyclone and the associated energy transfer to the ocean has a significant impact on the wave conditions and also ocean water levels, particularly in shallower water. The wind field model that has been adopted for this study is a modified version of the Holland (1980) parametric cyclone wind-field model - see Holland (1991). The Holland (1980) model has been combined with the wind field asymmetry model of Shapiro (1983) to define first-order wind-field asymmetry due to cyclone forward motion.

The key inputs for the Holland wind-field model are:-

1. Ambient air pressure - p_∞
2. Central air pressure - p_0
3. Forward speed - V_{fwd}
4. Wind field 'peakedness' - B
5. Radius to maximum winds (RMW) - R_m

The ambient air pressure for calibration events has been based on data provided by the BoM. In the case of the Monte Carlo cyclone modelling system, ambient pressures have been based on the mean historical atmospheric air pressure from the month of generation for each synthetic event. The central pressure and forward speed parameters for model calibration have been obtained from analyses of the BoM cyclone track database.

The wind field 'peakedness' parameter, B, influences the radial extent of damaging winds. Small B values have strong winds extending a greater distance from the cyclone eye compared to larger values. Holland defined the realistic range for the B parameter to be between 1 and 2.6. Holland *et al* (1991) defined a potential parametric model for the B parameter in the form of **Equation A.1**.

$$B = 1.5 + \frac{(980 - P_0)}{120}$$

Equation A.1

As part of other cyclone modelling studies for the northwest shelf region, Cardno have applied a range of fixed *B* parameters and the parametric model described above have been tested to determine the most suitable *B* parameter. In general, Cardno have found that wind fields generated with **Equation A.1** did not provide as good calibration compared to wind fields with a fixed *B* parameter of 1. A fixed *B* parameter of 1 provided the best agreement between modelled and measured wind conditions when it was applied in conjunction with the RMW formula in **Equation A.2**.

The Radius to Maximum Wind (R_m) parameter can also be difficult to define. If there is sufficient density of wind measurement locations near the cyclone track, it is possible to calculate this parameter directly. However, this is rarely the case, particularly in the isolated northwest region of Australia. The detailed track data recorded by the BoM since 2002 provides more reliable estimates of the radius to maximum winds. This data, whilst it may be useful when hindcasting recent events, is not likely to be suitable for providing input for a long-term synthetic cyclone track generation model. This study has adopted a parametric radius to maximum wind model to define this parameter in both calibration (hindcast) and design event (synthetic track) simulations. Harper *et al* (1989 and 1993) describe a possible relationship for RMW (km) for cyclones on the northwest shelf region based on **Equation A.2**.

$$R_m(t) = \frac{R_c}{(p_\infty - p_0(t))} \qquad \textbf{Equation A.2}$$

In **Equation A.2**, values for R_c, which Harper adopted as a fixed constant for each hindcast event, ranged between 640 to 3000 with a mean value of 1850 were adopted. Harper *et al* provides no guidance on the distribution of the R_c coefficient and this has been adopted as a constant value of 1850 in this study.

Detailed cyclone track data provided by the BoM for events that have occurred since 2002 contain reasonable re-analysis estimates of the RMW. **Figure A.1.1** presents a scatter plot of the RMW estimates as a function of central pressure for cyclone events between 2002 and 2010. The relationship between central pressure and R_m, which is presented in **Equation A.2**, is evident. As a result, a constraint has been applied in the wind field model which sets the maximum RMW at 60km - when **Equation A.2** exceeds 60km the RMW is fixed to a value of 60km. **Figure A.1.2** presents a scatter plot of the BoM RMW estimates as a function of central pressure, together with the Rm formula which has been adopted in this study. Whilst the BoM re-analysis estimates of the RMW show a large degree of scatter in relation to RMW as a function of central pressure, the adjusted Harper model adopted in this study provides a reasonable description of the general trend. Note that, generally, more of the BoM RMW estimates lie below the line than above it.

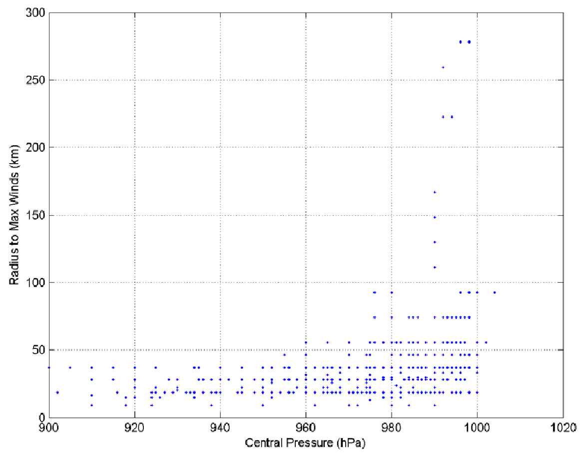


Figure A.1.1: Scatter Plot of Estimated Radius to Maximum Wind (R_m) and Central Pressure from BoM Data (2002-2010).

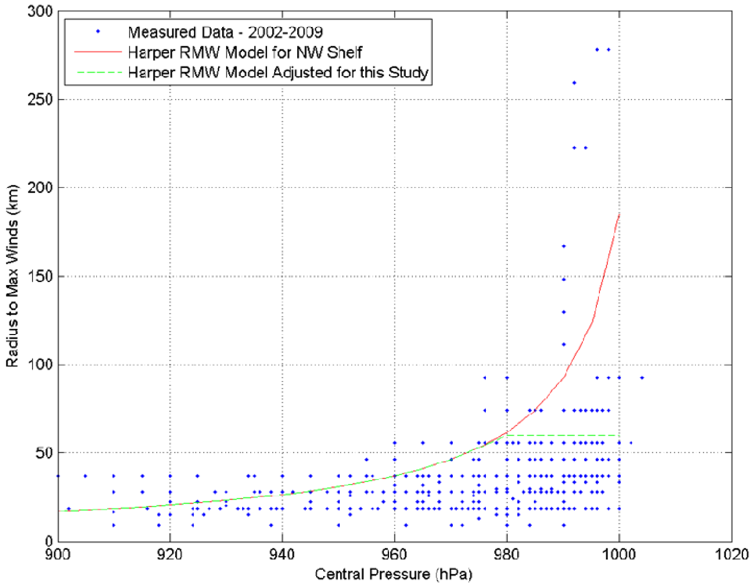


Figure A.1.2: Comparison of Estimated Radius to Maximum Wind (R_m) and Central Pressure from BoM Data (2002-2010) and the Parametric Model derived by Harper (1989 and 1993).

For the wind field model, cyclone track data for the calibration events was available from the BoM at 3-hourly, or coarser temporal resolution. In order to minimise any interpolation errors in the input wind fields, for all model simulations the cyclone track data was re-sampled to 1-hourly track data to provide a smooth wind field description with time. This process is particularly important for cyclones that track close to Port Hedland, when wind speed and direction can vary significantly within a period of three hours.

A.1.2 Wave Model

A multi-domain SWAN wave model system has been developed for the Port Hedland Coastal Vulnerability Study. The model system is based on similar models which Cardno has developed for other projects in the Port Hedland region.

In order to accurately simulate the wave conditions during cyclone events for this study, a large scale wave model domain that extends over 1000km north, west and east of Port Hedland has been developed to provide boundary conditions to the continental shelf and nearshore wave model (≈ 200km) which is coupled to the hydrodynamic tide and storm surge model – see **Section A.1.3**. The cyclone wave model developed for this study includes a large-scale non-stationary SWAN model extending over 1000km from Port Hedland on a 10km grid, which simulates cyclone waves for the duration of a cyclone event. Wind forcing is provided by the modified Holland model described in **Section A.1.1**. The large scale non-stationary SWAN model has the following key numerical parameter specifications:-

- Time step: 15-minutes;
- Numerical propagation scheme: 2nd order upwind scheme known as the S&L scheme; and
- Numerical diffusion specification: Wave-age coefficient of 24-hours.

The large-scale SWAN model is coupled to a continental shelf scale SWAN model system which couples directly with the tide and storm surge model presented in **Section A.1.3**. The northwest shelf scale SWAN model provides spectral boundary conditions to the continental shelf and nearshore scale model simulate waves crossing the continental shelf to the Port Hedland coastline on a fine grid. The shelf scale Port Hedland wave model also includes wind growth over the scale of the model domain. **Figure A.1.3** presents a plan view of the northwest shelf SWAN model extent. **Figure A.1.4** presents a plan view of the extent of the Delft3D and SWAN wave models which covers the area between the continental shelf and the Pilbara coastline.

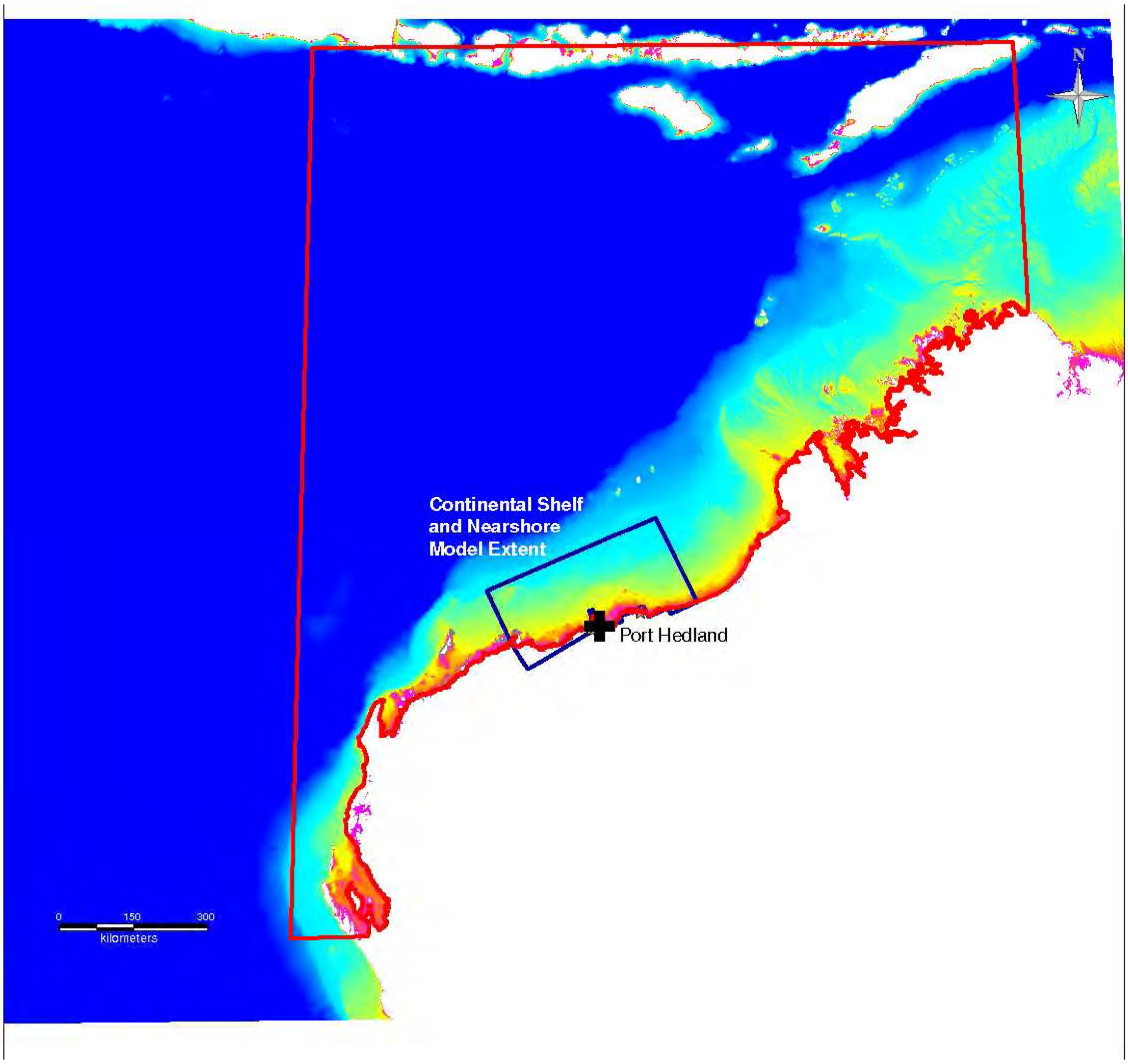


Figure A.1.3: Plan View of the Extent of the SWAN wave model system adopted in this study.

A.1.3 Hydrodynamic Model

A 2D/3D numerical model of the Pilbara coastline centred around Port Hedland has been developed using the Delft3D modelling system. Cardno has been developing and refining this numerical model of the Port Hedland region since 2006. The current Delft3D model adopts a multi-domain Delft3D Domain Decomposition model configuration.

The model extends across a region extending approximately 200km east and west, and 180km north of Port Hedland. The model grid system was prepared with the main objective of providing high resolution in the Port Hedland nearshore and harbour, but it also needed to describe a large coastal area in order to accurately represent the tidal variations and propagation of the tidal wave throughout the model domain. The model also covers the Shellborough study area to the east of Port Hedland.

To simulate the processes across the broad range of scales, the Delft3D model setup for this study adopts a domain decomposition configuration comprising of six (6) dynamically coupled grids. A coarse outer model grid (Grid 1) links to a 2D mid-scale grid (Grid 2). A refined Grid 3 covering the shipping channel is nested within Grid 2 and linked to a grid of the Port Hedland Inner Harbour area – Grid 4. The two other grids cover the estuary immediately west of Port Hedland (Study Sites 1 and 2) and a high-resolution grid (Grid 6) of the Shellborough site which is coupled directly to the overall Grid 1. This nested model system runs dynamically - time-step by time-step across all grids.

The grid domain and cell size for each grid are as follow:-

- Grid 1 extends 180km offshore (to water depths of 120 to 140m) and 200km east and west of Port Hedland. Variable grid size from 1500m x 1500m to 500m x 500m.
- Grid 2 extends about 45km offshore Port Hedland and about 4km on either side of the shipping channels. Grid cell size is 120m x 200m.
- Grid 3 (Shipping Channel) extends to about 20km offshore and about 1.8km on either side of the channel. Grid cell size is 40m x 100m with 40m resolution in the cross-shipping-channel direction.
- Grid 4 (Port Hedland Inner Harbour) covers the entire Inner Harbour area including the surrounding tidal flats and tributaries. Grid cell sizes at Inner harbour berth locations are in the order of 20m x 20m.
- Grid 5 (Sites 1 and 2) covers the study areas of Sites 1 and 2 in detail, as well as simulating the estuary area to the west of Port Hedland and the entrance to the Turner River. Grid cells size is in the order of 20m x 20m. This grid is directly coupled to Grids 2 and 3.
- Grid 6 (Shellborough) covers the Shellborough site area including the estuary inland of this site. Grid cells size is in the order of 20m x 20m. This grid is directly coupled to Grid 1.

Offshore tidal constants were extracted from the Oregon State University global model of ocean tides which uses along track averaged altimeter data from the TOPEX/Poseidon and Jason (on TOPEX/Poseidon tracks since 2002) satellites. The methodology of the global tide models is described in Egbert and Erofeeva (2002). TPXOv7.2 provides up to ten tidal constants on a 1/4 degree resolution full global grid. The tides are provided as complex amplitudes of earth-relative sea-surface elevation for eight primary (M_2, S_2, N_2, K_2, K_1, O_1, P_1, Q_1) and two long period (M_f, M_m) harmonic constituents. A regional model of Northern Australia (NAust) also provided M_2, S_2, K_1, O_1 and M_4 on a finer grid 1/24th degree of Northern Australia.

A total of fourteen (14) tidal boundary locations with separations of approximately 50km were selected along the western, northern and eastern boundaries of the coarse model grid. Eleven (11) tidal constituents (M_2, S_2, N_2, K_2, K_1, O_1, P_1, Q_1, M_m, M_f and M_4) were applied to the model boundary. In the Delft3D model, the tidal constants are used to generate water level time series along the model boundary at the specified input locations. Water levels between boundary points are interpolated linearly.

The time step for the model was 6-seconds over the whole model domain and each grid. This provides an accurate solution within the harbour and nearshore areas. The simulations presented in the following sections have adopted a two-dimensional (depth-averaged) model configuration.

As the Inner Harbour is currently undergoing significant dredging and port development, a recent bathymetric survey of the shipping channel (late 2009) and completed dredging for the Nelson Point, Harriet Point and Utah Point port facilities were included in the model. The model bathymetry also includes LiDAR data of intertidal areas which were captured during this survey and also detailed creek channel survey and contour data which was obtained from the Port Hedland Port Authority (PHPA). The model bathymetry of the nearshore region around the Shellborough site has the greatest uncertainty as no survey data set was available for that area. Bathymetric data was obtained from a number of sources including hydrographic charts AUS 54 and AUS 55, the Geoscience Australia 9-second DTM and a Geoscience Australia 1km x 1km DTM. For this study, the Geoscience Australia 1km x 1km DTM has been used to define the coastal bathymetry near the Shellborough site as this data set provided the best calibration of the hydrodynamic model for tidal and storm surge conditions near Port Hedland.

The available bathymetric and survey data sets for this study have been assembled into a Digital Terrain Model (DTM) which has been applied across the various wave, hydrodynamic and hydrological models systems developed for this study. The bathymetric data provided by FAST have been integrated into a Digital Terrain Model (DTM) using the 12D terrain modelling software (12d, 2009) of Port Hedland and surrounding nearshore region. The following datasets have been utilised in the DTM in their order of precedence (highest to lowest):-

1. LiDAR survey of Port Hedland area including the Shellborough site and studies areas to the west of Port Hedland;
2. PHPA 2009 channel and inner harbour survey;
3. Recent survey data of tidal creeks in Port Hedland (provided by PHPA)
4. Data from hydrographic charts available from the Australian Hydrographic Office; and
5. Data from a 1km digital bathymetric model available from Geoscience Australia.

Figure A.1.4 presents a plan view of the Delft3D model extent.

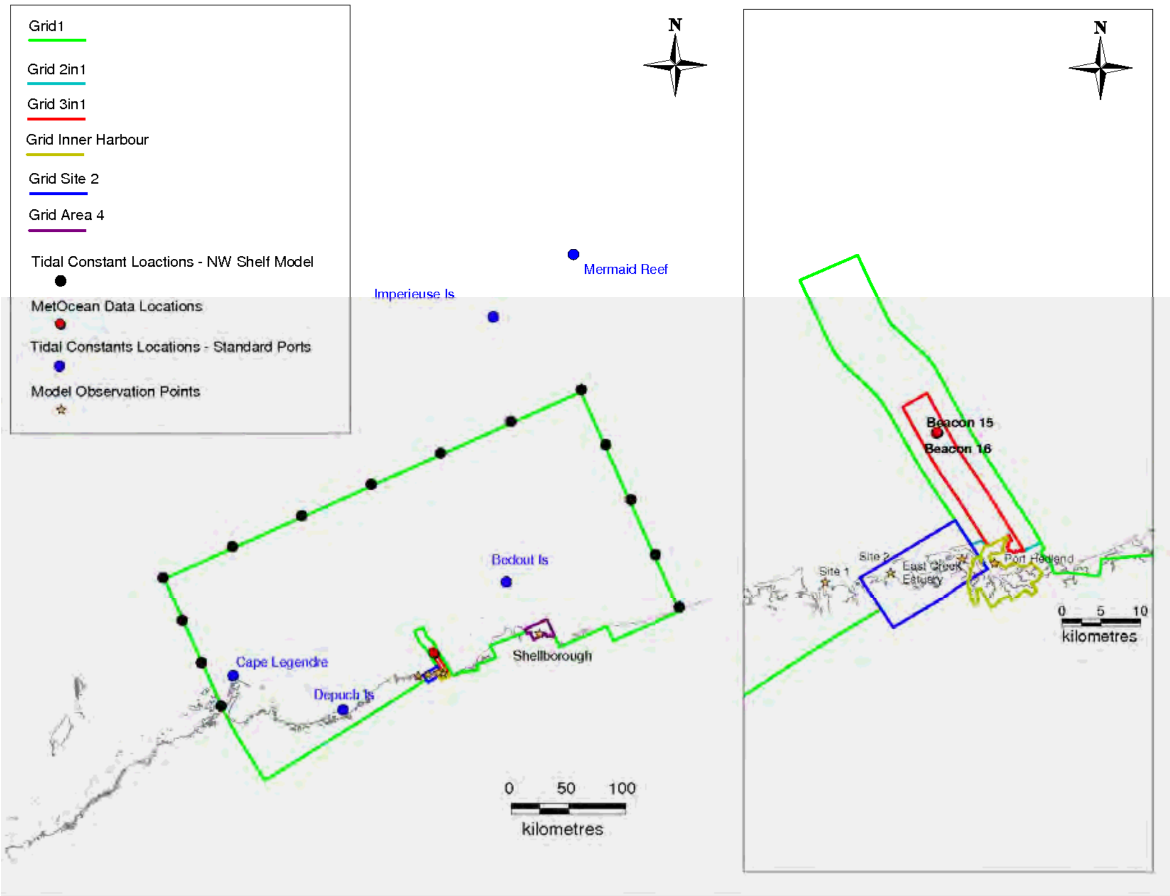


Figure A.1.4: Plan View of the Extent of the Delft3D system adopted in this study including the detailed output study area.

A.2 MODEL CALIBRATION

The following sections present details on the calibration of the wind, wave and hydrodynamic model systems using measured data from six historical cyclone events for which observed data is available at Port Hedland. Wind and air pressure observations were used to calibrate the wind field model. Recorded offshore surface wave data was used to calibrate the SWAN wave model. Water level and current data were utilised to calibrate the hydrodynamic model. The hydrodynamic model calibration process included both tidal and storm surge conditions – see **Section A.2.3**.

Data available for calibration of the models varies through time. Six calibration events selected for this study included five cyclones which have occurred since 1999 were selected for the model calibration exercise. The quality and resolution of the cyclone track data has improved since 1999 and also a wide range of concurrent water level, wind, air pressure and wave data is available since that time. An earlier cyclone event, Tropical Cyclone Connie, which occurred in 1988 is also included as a validation case because measured wind and surge data is available for this relatively severe event. The six calibration and validation events adopted for the ocean inundation model system are:-

1. Tropical Cyclone Connie – December 1988
2. Tropical Cyclone John – March 1999
3. Tropical Cyclone Monty – February 2003
4. Tropical Cyclone Clare – January 2006
5. Tropical Cyclone Daryl – January 2006
6. Tropical Cyclone George – March 2007

Track plots for each calibration event are presented in **Appendix A.1**. **Table A.2.1** summarises the proximity and intensity of the calibration events at Port Hedland. In terms of cyclone intensity near Port Hedland, Tropical Cyclone's Connie, John and George were relatively intense whereas Monty, Clare and Daryl were less intense and generally tracked further away from the coastline near Port Hedland.

Table A.2.1: Summary of Proximity and Intensity of the Calibration Events.

Cyclone Event	Duration of Simulation	Minimum Central Pressure (hPa)	Minimum Distance to Port Hedland (km)	Central Pressure at Minimum Distance (hPa)
Connie	7 days	950	8	950
John	6 days	915	114	950
Monty	7 days	935	124	981
Clare	6.75 days	960	165	960
Daryl	7 days	976	163	976
George	7 days	902	39	933

Figure A.2.1 presents a plan view of the wind, air pressure, wave, water level and current measurement locations utilised for the model calibration process.

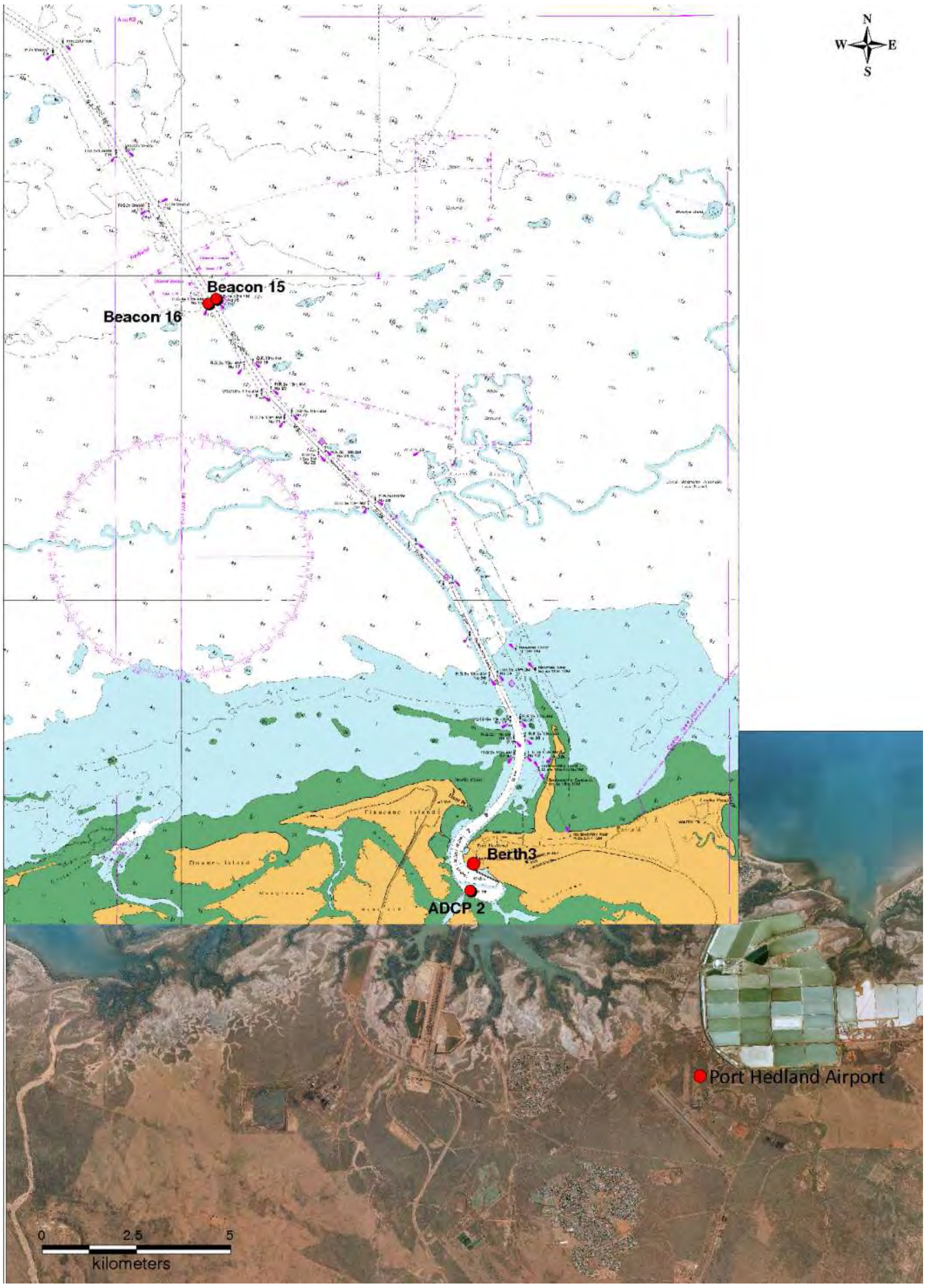


Figure A.2.1: Plan View of the Extent of the wind, air pressure, wave, water level and current measurement locations which has been utilised during the model calibration process.

The data sets from each of the measurement sites applied in the model calibration process are:-

- Beacons 15 and 16
  - Wind: Beacon 15 (15.7m above mean sea level) – 10-minute average records (01/05/2000 to 31/12/2009)
  - Waves:
    - Electromagnetic Tide and Wave Monitoring System (EWS): 1-hourly wave height and period data (2000 to 2007).
    - Acoustic Wave and Current (AWAC) instrument (Beacon 16): 1-hourly wave height, period and direction (July 2000 to 2007).
    - Directional Waverider Buoy (DWRB) (Beacon 15): 1-hourly wave height, period and direction (November 2006 to 2008).
  - Tide: Predicted and measured tides for selected events since 1999.

- Port Hedland Airport
  - Wind and atmospheric pressure: Bureau of Meteorology AWS (6.4m above ground) 3 hourly records (1942 to 2010) and 30-minute records (1993 to 2010).

- Inner Harbour (Berth-3)
  - Tide: Predicted and measured tides from the permanent tide gauge at Berth-3 for selected events since 1988.
- ADCP-2 Site - Inner Harbour (Berth-3)
  - Currents: Measured currents from a 3-week deployment of a seabed mounted Acoustic Doppler Current Profiler (ADCP) between December 2010 and January 2011.

Extensive cyclone track data have been obtained from the Bureau of Meteorology (BoM) to define the cyclone climate of the study site and also to provide data with which to populate the Monte Carlo model described in **Section A.4**. The key cyclone track data sets utilised in this study are:-

- Bureau of Meteorology (BoM) best track database of cyclones in the Australian region between 1906 and 2007;
- Bureau of Meteorology operational cyclone tracks between 2007 and April 2009; and
- Detailed cyclone track and parameter analyses prepared by the BoM for northwest Australian cyclones that have occurred since 2002.

Model calibration was undertaken firstly for the wind field and pressure model as this provides the primary forcing for the wave and hydrodynamic model, followed by the wave and hydrodynamic models. The calibration process is summarised in the flow chart presented as **Figure A.2.2**. The calibration results from each of the model components are presented in the following sections.

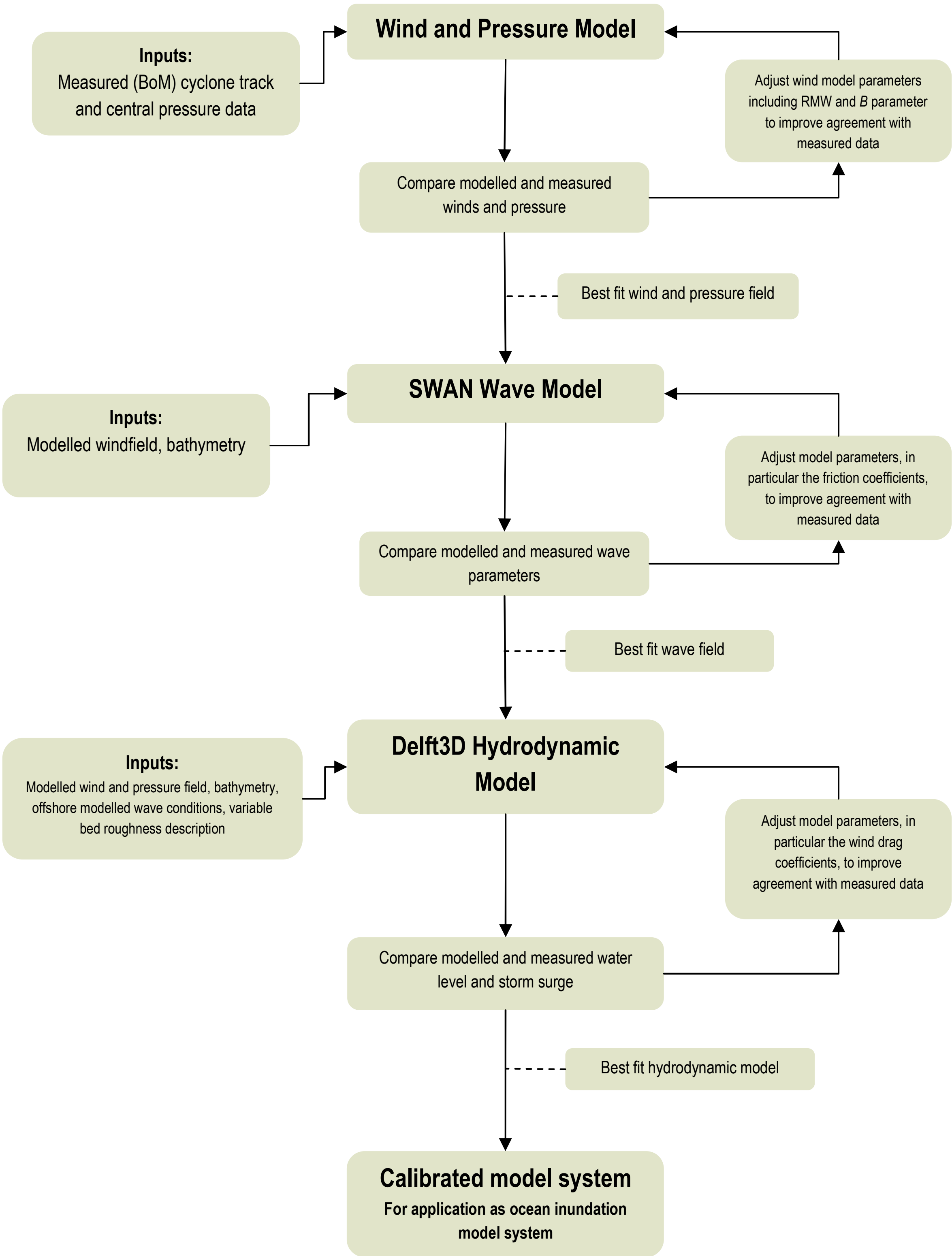


Figure A.2.2: Flow chart of the calibration process for the ocean inundation model system.

A.2.1 Wind field Model

Figures A.2.3 to **A.2.8** present time series plots of modelled and measured wind conditions at Port Hedland Airport and, where available, the Beacon 15 offshore site operated by PHPA for the six calibration events. The figures also include details of the estimated central pressure and the track relative to Port Hedland. A qualitative assessment indicates particularly for the three most severe events, Tropical Cyclones Connie, John and George, there is reasonable agreement between modelled and measured wind conditions.

The measured wind speeds in **Figures A.2.3** to **A.2.8** have been adjusted to +10m elevation. Wind measurements at Beacon 15 and the Airport wind measurements were adjusted using the power law method following Hsu *et al* (1994) as presented in **Equation A.3:-**

$$U_{10} = U_r (z_{10} / z_R)^P \qquad \textbf{Equation A.3}$$

where U_{10} is the wind speed at the standard 10m elevation (z_{10}) and U_R is the wind speed measured at the height z_R Although a value of *P*=1/7 is commonly used for wind data measured over land, a value of *P*=0.11 was empirically determined to be applicable most of the time over the ocean by Hsu et al (1994). The wind speed correction for Beacon 15 was based on the assumption that the wind measurements were taken at a constant elevation of 15.7m above the sea surface. The effect of tidal variations on the elevation appears to have a negligible effect on the winds speeds as determined through correlations between surface buoy mounted wind data and fixed elevation instrument data. Beacon 15 and Airport wind data were therefore adjusted to the 10m elevation standard using the power law method and an exponent *P* of 0.11 and 0.14 respectively.

For Tropical Cyclone Connie, the measured wind data from Port Hedland airport is only available at three hourly measurement intervals compared to 30-minute intervals for the later events. As a result, the measured data does not capture the 'lull' in wind conditions as the cyclone eye passes very close to Port Hedland as indicated by the low wind speeds persisting for only a short period of time before increasing significantly – see **Figure A.2.2**.

Overall, the agreement between modelled and measured wind conditions presented in **Figures A.2.3** to **A.2.8** is good. For the two least severe cyclone events which tracked the furthest distance from Port Hedland, TC's Clare and Daryl, the agreement between modelled and measured wind direction at the offshore Beacon 15 site is not as good. During both events, near the peak wind speed conditions, there is a 30 to 40–degree variation in wind direction between the wind field model and the measured data. In particular for Tropical Cyclone Clare, this variation in wind direction is likely to impact on the validation of the storm tide model – see **Section A.2.3.2**. The wind fields generated from cyclones with relatively low intensities such as TC Clare and Daryl are more likely to be influenced by the regional non-cyclonic atmospheric conditions which are not incorporated into the wind field model adopted in this study. Overall, the good agreement between modelled and measured wind field conditions for the three severe cyclones in the validation dataset (TC's Connie, John and George) provides confidence that the model system can simulate the wind field associated with severe cyclones to a reasonable level for the purposes of this study.

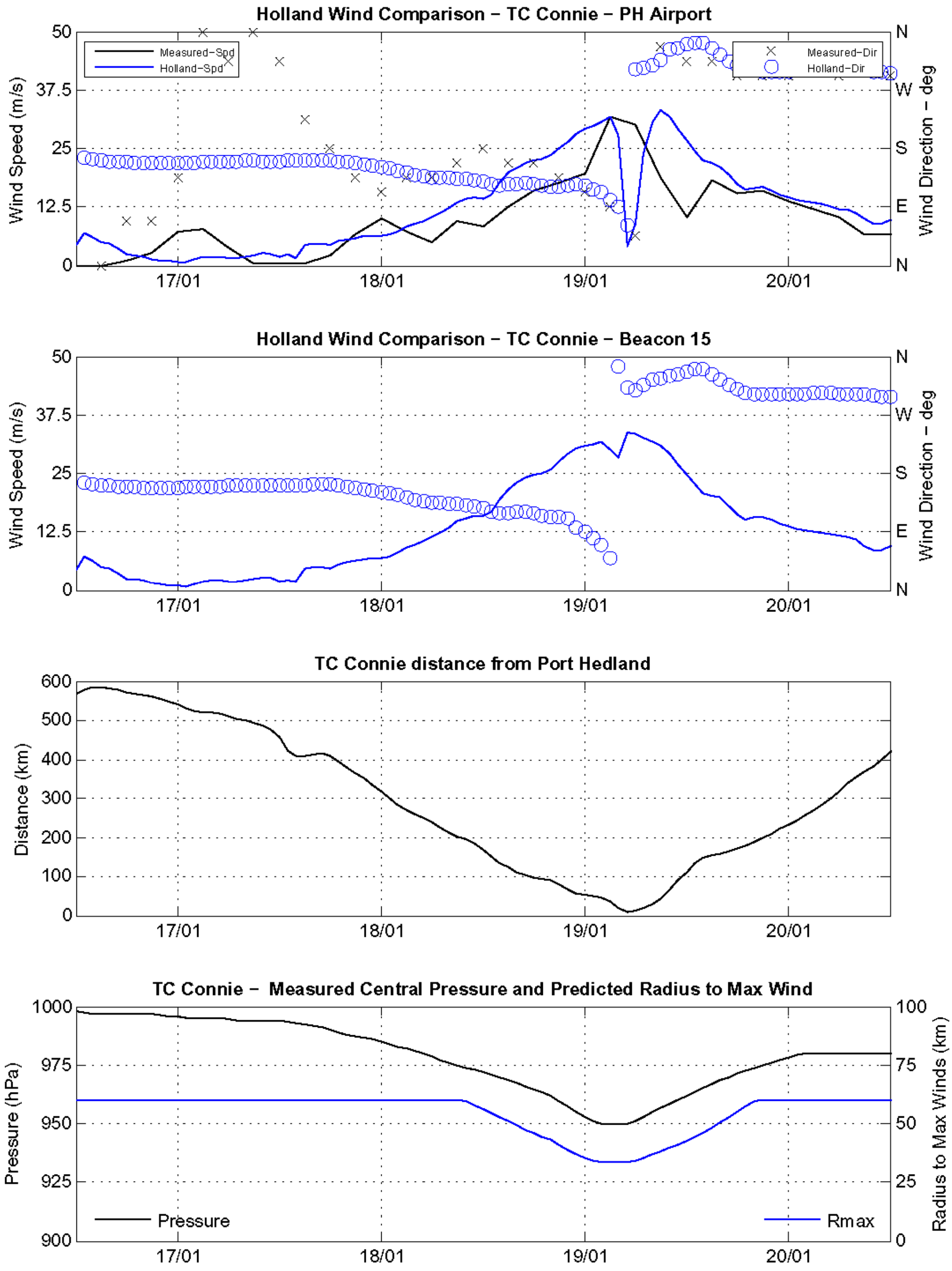


Figure A.2.3: Comparison of Modelled and Measured Wind Conditions near Port Hedland - TC Connie.

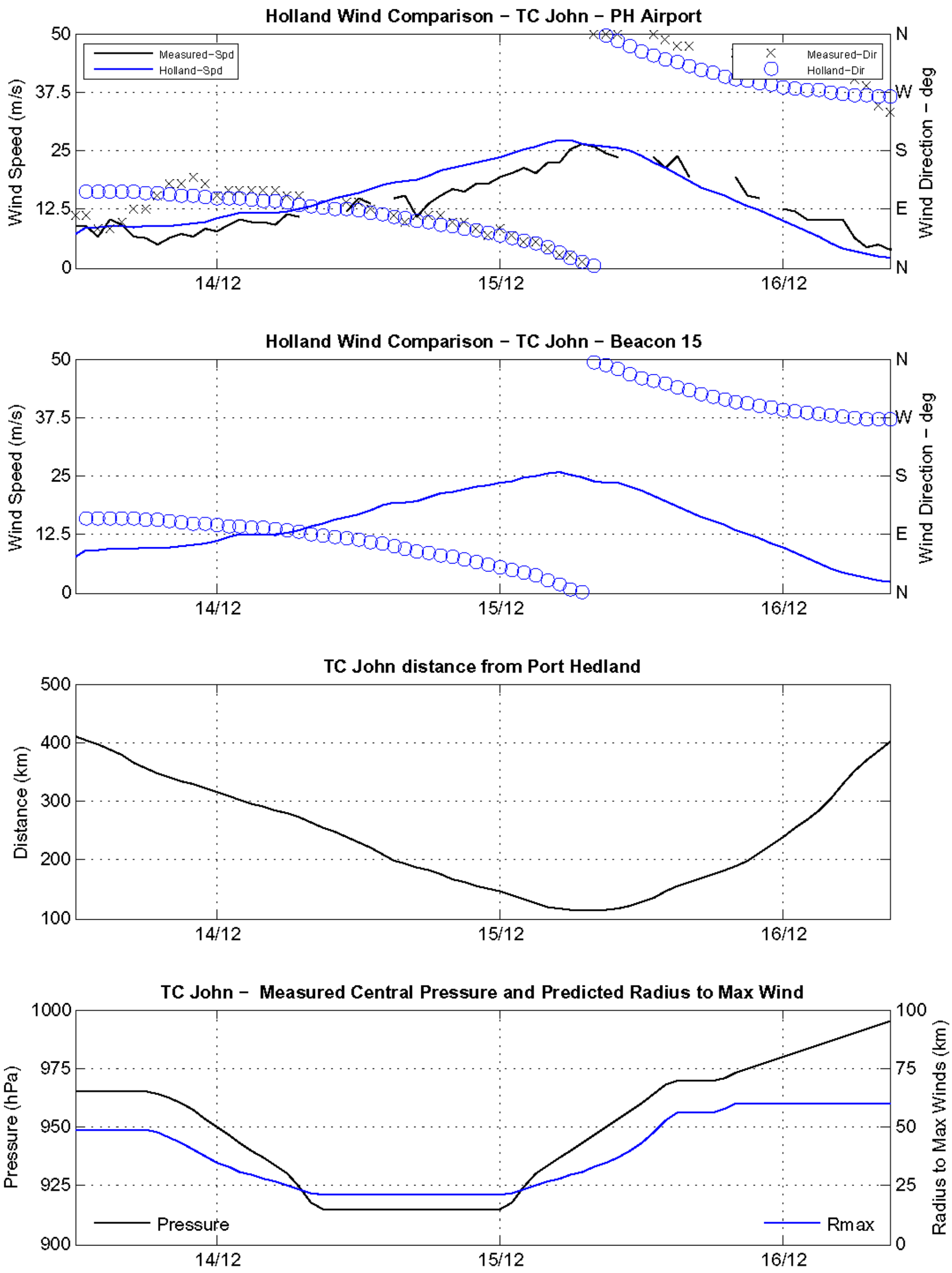


Figure A.2.4: Comparison of Modelled and Measured Wind Conditions near Port Hedland - TC John.

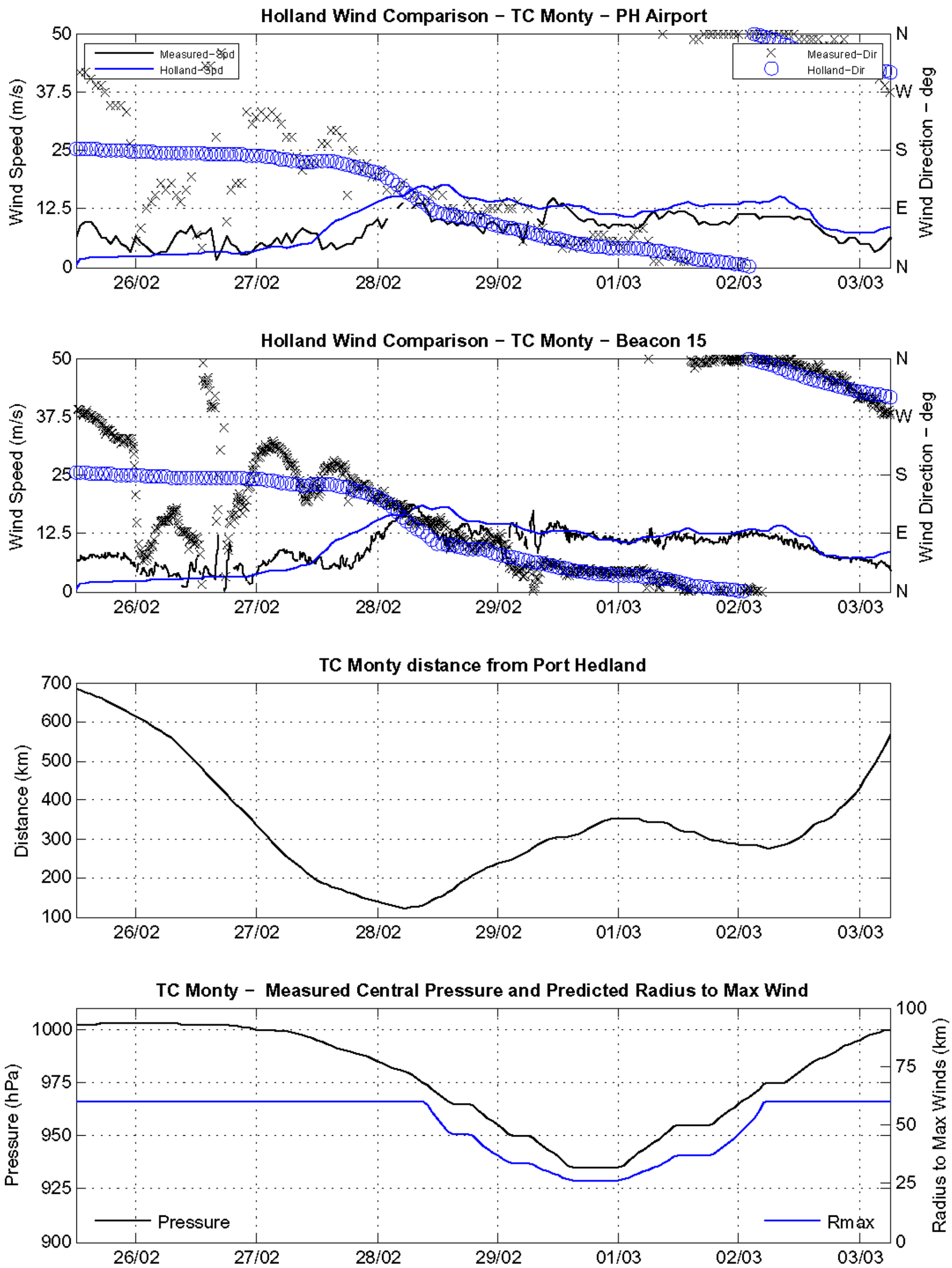


Figure A.2.5: Comparison of Modelled and Measured Wind Conditions near Port Hedland - TC Monty.

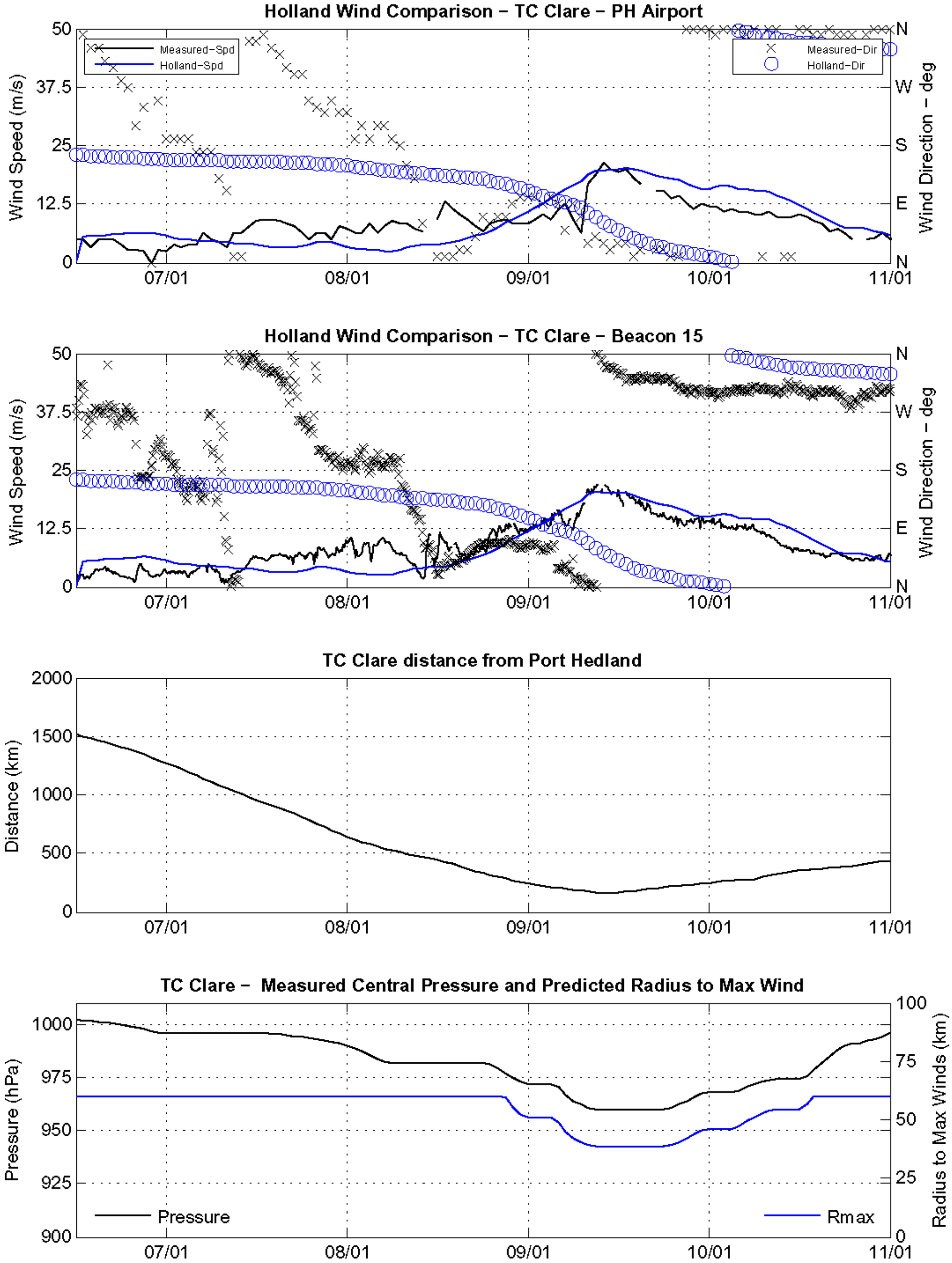


Figure A.2.6: Comparison of Modelled and Measured Wind Conditions near Port Hedland - TC Clare.

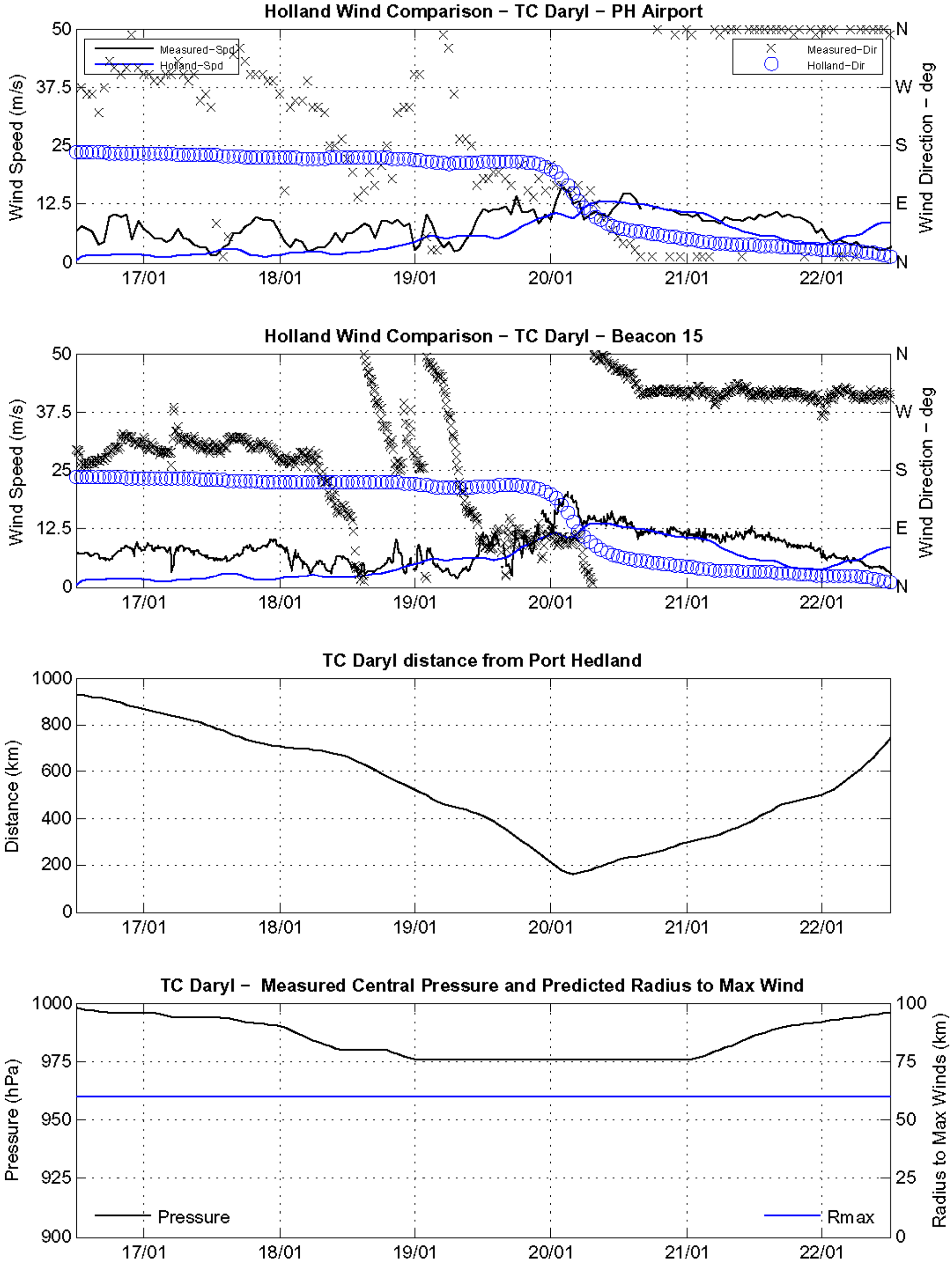


Figure A.2.7: Comparison of Modelled and Measured Wind Conditions near Port Hedland - TC Daryl.

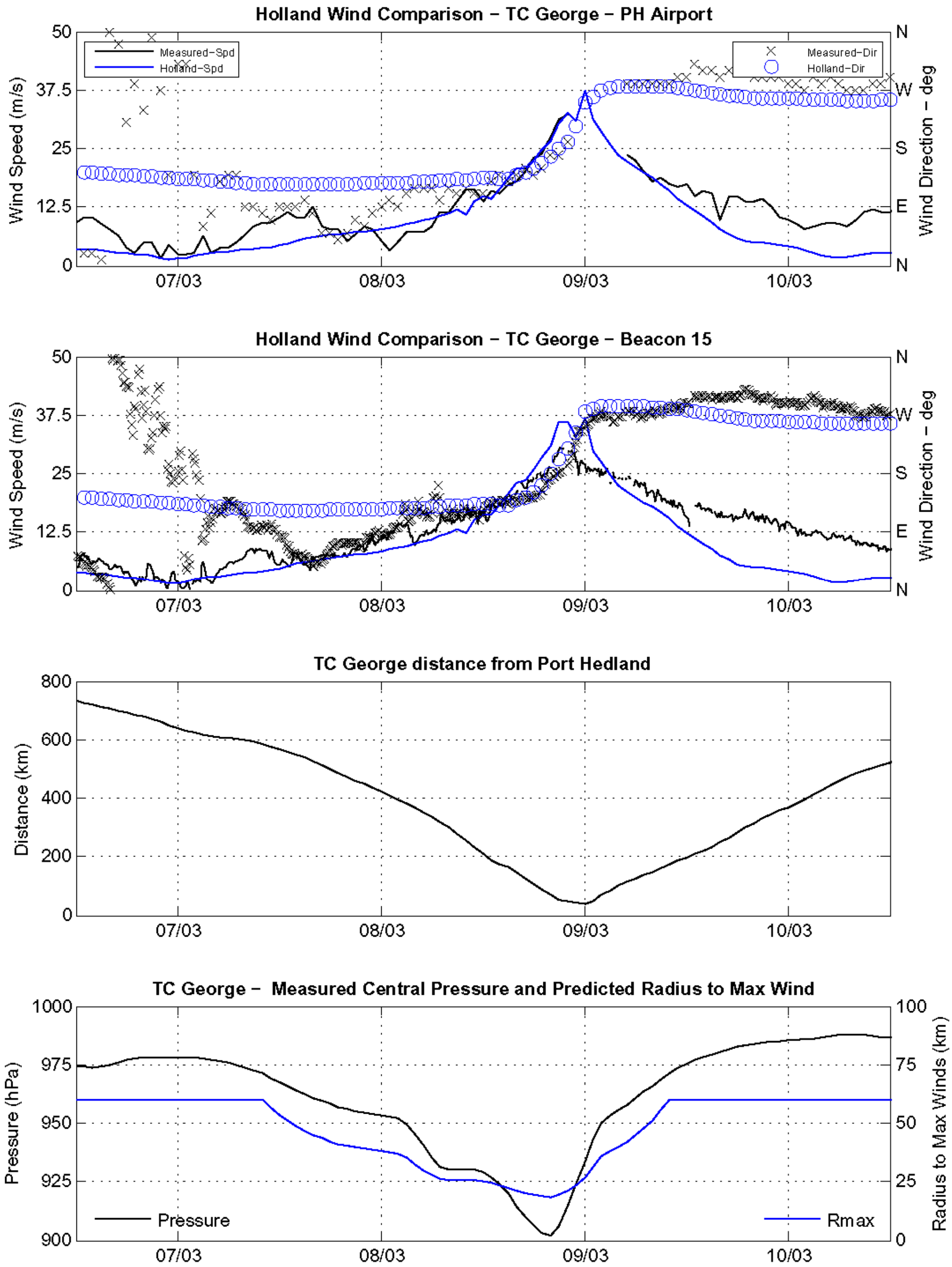


Figure A.2.8: Comparison of Modelled and Measured Wind Conditions near Port Hedland - TC George.

A.2.2 Wave Model

The wind field model presented in **Section A.1.1** has been applied to the SWAN model system to hindcast wave conditions in the Port Hedland region during each event. A description of the wave model system is presented in **Section A.1.2**.

A range of different wave data sets have been compared to the hindcast conditions from the SWAN model, depending on the event. **Table A.2.2** presents a summary of the wave measurement locations and instrument types adopted for each calibration event.

Table A.2.2 Wave Measurement Locations for Calibration Events

Cyclone Event	Measurement Location	Instrument Type
Connie	Unknown – offshore Port Hedland	Waverider buoy (non-directional)
John	Beacon 16	EWS – non directional
Monty	Beacon 16	EWS – non directional
Clare	Beacon 16	AWAC
Daryl	Beacon 16	AWAC
George	Beacon 15	Directional Waverider Buoy

Based on the wave model calibration simulations, the JONSWAP bed friction coefficient in the SWAN model has been adjusted to be $0.067m^2s^{-3}$. The value adopted in this study is consistent with the recommended value in the SWAN user manual for sea state conditions (Booij *et al*, 2004). The large cyclone waves at Port Hedland are dominated by sea rather than swell.

Figures A.2.9 to **A.2.14** present time series of modelled and, where available, measured wave conditions for the calibration events. Overall the wave model system simulates the extreme wave conditions reasonably well. The absence of a larger scale ambient wind field generally leads to the modelled wave conditions before and after the peak wave conditions being considerably lower than the measured data. Comparison between modelled and measured waves recorded during the peak of cyclone events is limited somewhat by the number of gaps in the wave measurements, particularly near the peak wave conditions of each event. However, for five out of the six cyclone events the modelled wave conditions for approximately +/- 12-hours near the peak of the cyclone conditions agree well with the measured data. A key outcome of the wave model calibration is that, although there can be +/- 0.5m variations between modelled and measured wave heights, the wave heights from the SWAN model do not appear to be biased. In addition, the objective of this study is to provide a reasonable representation of the wave field and its effects on coastal water levels and the SWAN model results presented in this section incorporated into the overall hydrodynamic model account for wave related variations in water level in the nearshore zone.

Table A.2.3 presents a summary of the SWAN wave model validation characteristics compared to the measured data for each event.

Table A.2.3 SWAN Model Validation Characteristics

Cyclone Event	Validation Characteristics
Connie	Hard copy plots of measured wave conditions recorded during this event are presented in WNI (1998) and show a peak wave height (H_s) of near 5.3m occurred around 18:00 hours on 19/1/1987. Comparing this measured data with the hindcast results in **Figure A.2.9** indicates that the modelled peak wave height of H_s=4.9m indicating reasonable agreement with the measured data.
John	The agreement between modelled and measured wave height and period for this event is very good leading up to the peak wave conditions. Following the peak wave height, the SWAN model result has lower wave heights than the measured data.
Monty	The wave model reproduces the general trend in wave conditions, however, the modelled wave height (H_s) is generally 0.5 to 1.0m larger than the recorded data. Wave period (peak) agrees well with measured data for the simulation with hydrodynamic processes included.
Clare	Overall, model wave height, direction and period agree reasonably well with the measured conditions from the AWAC instrument.
Daryl	The measured data features several small data gaps that restrict the ability to compare measured data with the modelled results. Overall model wave height, direction and period agree reasonably well with the measured conditions from the AWAC instrument.
George	The measured data features data gaps, which restrict the ability to compare measured data with the modelled results. Overall, modelled wave height, direction and period agree reasonably well with the measured conditions from the instrument.

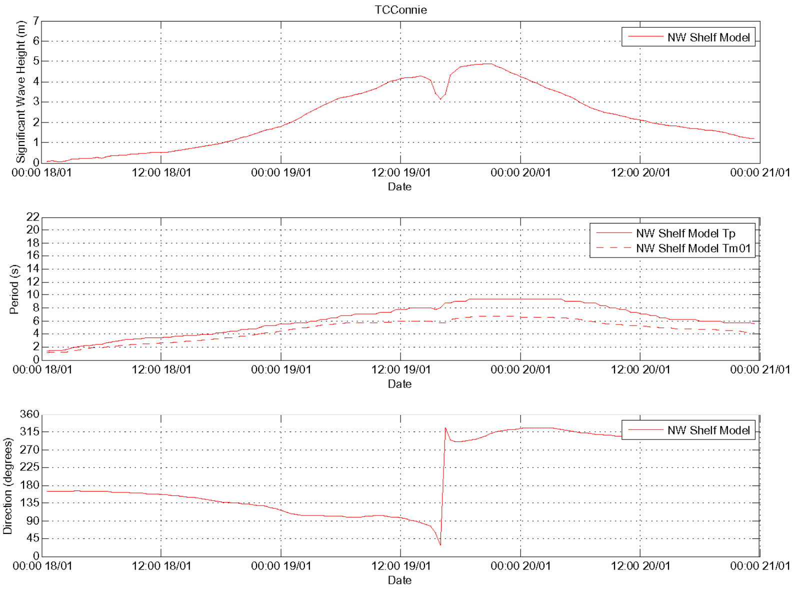


Figure A.2.9: Modelled Wave Conditions offshore Port Hedland (Beacon 15/16) - TC Connie.

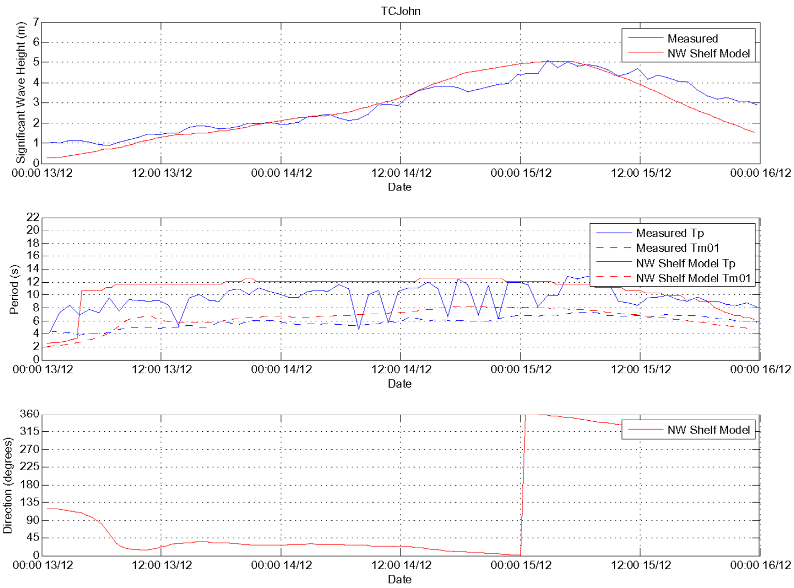


Figure A.2.10: Modelled and Measured Wave Conditions offshore Port Hedland (Beacon 15/16) - TC John.

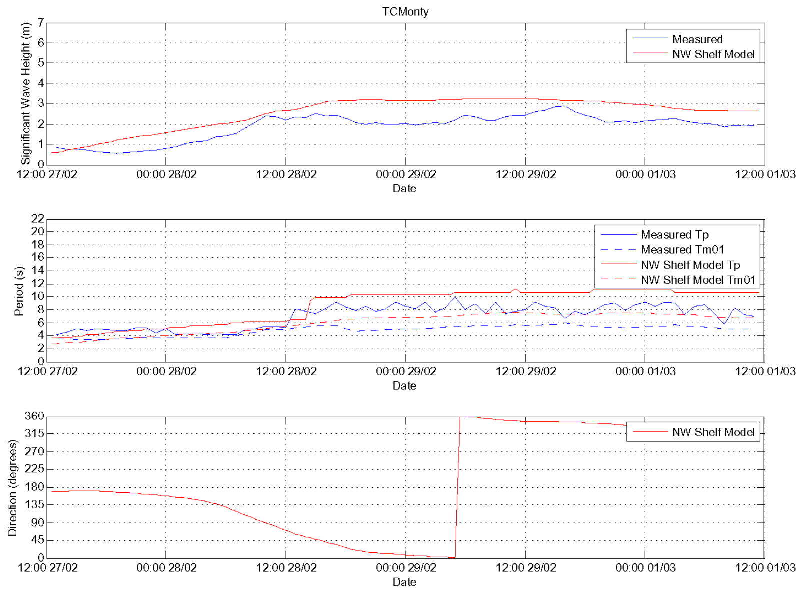


Figure A.2.11: Modelled and Measured Wave Conditions offshore Port Hedland (Beacon 15/16) - TC Monty.

Figure A.2.12: Modelled and Measured Wave Conditions offshore Port Hedland (Beacon 15/16) - TC Clare.

Figure A.2.13: Modelled and Measured Wave Conditions offshore Port Hedland (Beacon 15/16) - TC Daryl.

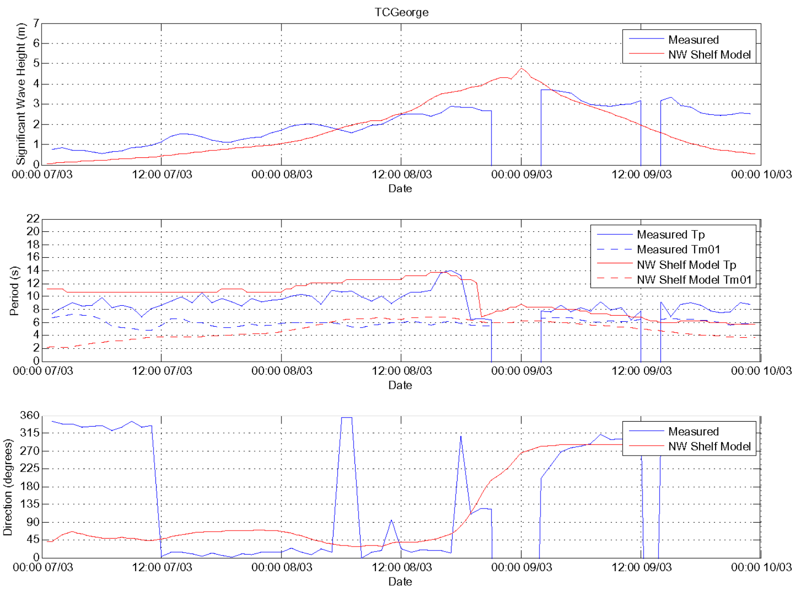


Figure A.2.14: Modelled and Measured Wave Conditions offshore Port Hedland (Beacon 15/16) – TC George.

A.2.3 Hydrodynamic Model System

A.2.3.1 Tidal Processes Model Validation

The Cardno Port Hedland Delft3D model system was first established in 2006 and has undergone a number of enhancements since. Since the original calibration of the model to measured water level and current data, the model has been subsequently validated as additional current data sets were obtained for the Inner Harbour, and also as port developments have proceeded over that time the changes to the harbour have been incorporated into the model. The most recent model validation exercise was undertaken based on data collected during the period from November 2009 to January 2010.

The roughness of the seabed and land within the Port Hedland estuarine system has a significant influence on tidal processes in the estuaries. Once the model bathymetry has been finalised, the primary model process coefficients which are adjusted during the tidal validation phase relate to bed roughness. Since the development of the Cardno Port Hedland model in 2006 there have been several enhancements to the roughness specifications for various areas of the model. Currently, the model calibration and validation process has led to four roughness coefficients being adopted, depending on the vertical level and vegetation of the seabed/inter-tidal area. The Delft3D model uses a Chezy bed roughness formulation with Chezy roughness coefficients (C) adopted for different depth ranges within the estuarine grids as follows:-

1.	<-3m AHD:	C=65$m^{1/2}$/s;
2.	>-3 & <0m AHD:	C=30 $m^{1/2}$/s;
3.	>0 & <3m AHD:	C=5 $m^{1/2}$/s;
4.	> 3m AHD:	C=20 $m^{1/2}$/s;

The third roughness category represents the roughness adopted within the mangrove areas and has been derived from the field and numerical modelling study presented in Wolanski *et al* (1980). For the mangrove roughness areas, the roughness map has been verified against aerial photography which defines the extent of mangrove vegetation areas at Port Hedland.

An ADCP instrument was deployed near Utah Point to measure currents and water levels over a 3-week period. The location of the ADCP instrument is presented in **Figure 6.4**. **Figure A.2.15** presents a comparison of modelled and measured water level, together with modelled depth-averaged current speed and direction. The modelled water level is in good agreement with the measured data and there is an offset in the MSL between the model and measured data of approximately 0.15m. The model current magnitude agrees very well with the measured data in terms of magnitude and phase. The modelled current direction indicates that there is an approximate 20-degree offset between the measured and modelled current direction although the phasing of the modelled current direction agrees well with the measured data. Whilst the model simulation presented in **Figure A.2.15** adopted a bathymetry which included a seabed survey from November 2009, between November 2009 and January 2010 when this data was collected, dredging near Utah Point associated with the Port expansion in that area was ongoing and based on available photography, it appears that over the 6 to 8 week period from the when the Inner Harbour survey was undertaken in 2009 to the ADCP measurement programme, the seabed surrounding the ADCP instrument had change considerably due to the dredging. This is likely to impact on the direction of current flow and it is the likely reason for the rotation in modelled current direction compared to the measured data.

The model comparison presented in **Figure A.2.15**, together with earlier model validation exercises (Cardno Lawson Treloar, 2007 and 2008a) indicate that the Cardno Delft3D model of Port Hedland shows good agreement with measured currents and water levels inside Port Hedland.

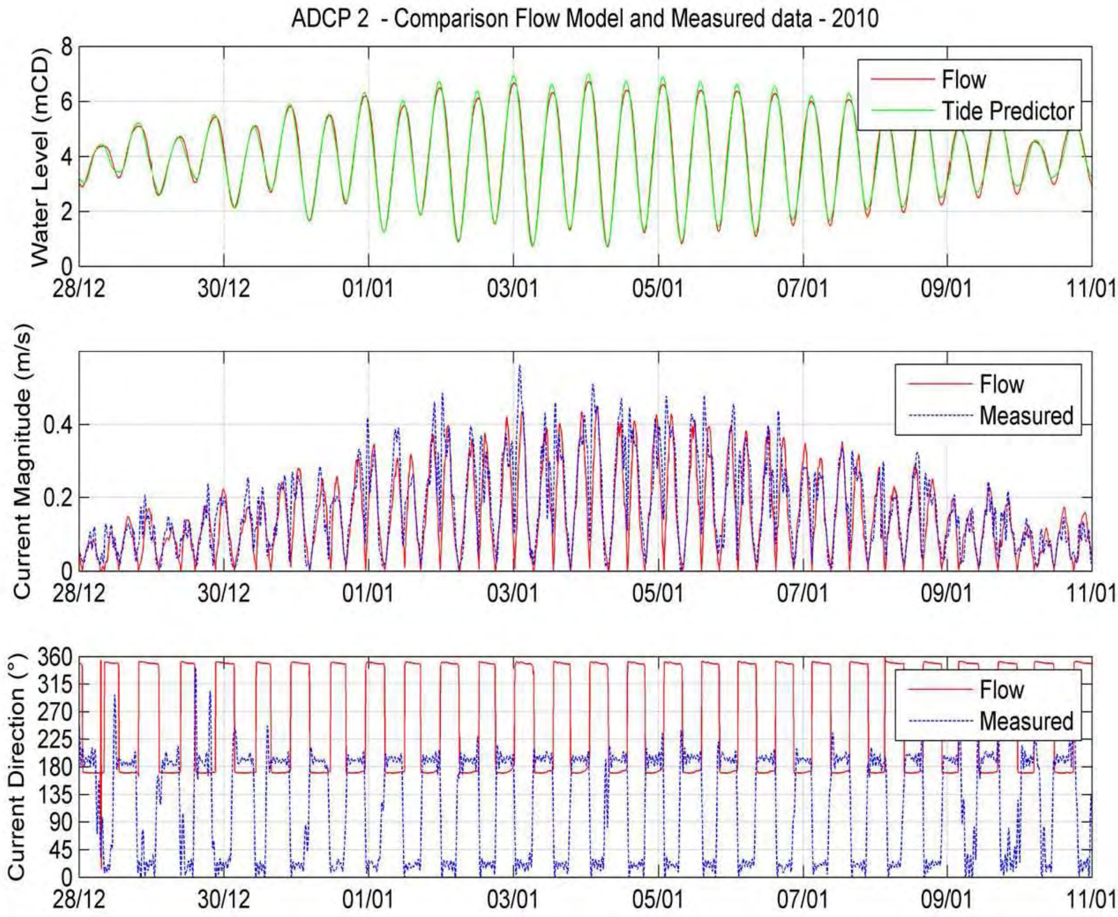


Figure A.2.15: Comparison of Modelled and Measured Tides and Currents (Depth-Averaged) near Utah Point.

A.2.3.2 Storm Tide Model Calibration

The wind field model presented in **Section A.2.3.1**, together with the wave model presented in **Section A.2.3.2**, have been coupled with the Delft3D hydrodynamic model to generate a comprehensive storm tide model. For the calibration simulations, the hydrodynamic model presented in Cardno Lawson Treloar (2010b) has been operated in 2D (depth-averaged) mode because storm tide is generally a 2D rather than 3D process. Simulations have also been undertaken for tide only conditions to generate the modelled predicted tide which can then be separated from the results of the full process model with wind, wave and pressure forcing to produce a modelled tidal residual. This allows a true comparison between measured and modelled residual water levels because the modelled astronomical tide will invariably differ slightly from the actual predicted tide (using tidal constants developed from water level records) at the site. For the calibration results presented in the following section, all simulations were undertaken with the following 'calibrated' wind drag coefficients:-

- $C_{d,1}$ = 0.001 @ 5m/s
- $C_{d,2}$ = 0.00325 @ 40m/s

These drag coefficient values were obtained from Bowden (1983). For wind speeds below 5m/s a constant drag coefficient of 0.001 is adopted, and above 40m/s a constant of 0.00325 is adopted. For wind speeds between 5m/s and 40m/s linearly interpolated drag coefficients are between these two values.

Figures A.2.16 to **A.2.21** present time series of modelled and measured water levels together with the residual water level for the calibration events.

Table A.2.4 Delft3D Model Validation Characteristics

Cyclone Event	Validation Characteristics
Connie	**Figure A.2.16** includes modelled and measured residual water levels for Tropical Cyclone Connie at the permanent Berth 3 tide gauge. The measured residual water heights record has been extracted from WNI (1998). The agreement between modelled and measured storm surge is excellent and this outcome is significant because Tropical Cyclone Connie is the most severe of all the selected calibration events and most resembles the conditions that would likely occur at the Quantum OHD site during a design event.
John	**Figure *A.2.17*** presents modelled and measured residual water levels for Tropical Cyclone John at the permanent Berth 3 tide gauge. The modelled and measure storm surge time series agree very well for this event which produced a moderately large surge at Port Hedland.
Monty	**Figure A.2.18** presents modelled and measured residual water levels for Tropical Cyclone Monty at the permanent Berth 3 tide gauge. For this event, the modelled and measured storm surge is relatively small and the modelled surge is generally +0.2 to 0.3m higher than the measured surge.
Clare	**Figure A.2.19** presents modelled and measured residual water levels for Tropical Cyclone Clare at the permanent Berth 3 tide gauge. In general, whilst the modelled peak storm surge magnitude agrees reasonably well, albeit higher, with the measured data, the phase agreement between modelled and measured storm surge is not as good.
Daryl	**Figure A.2.20** presents modelled and measured residual water levels for Tropical Cyclone Daryl at the permanent Berth 3 tide gauge. For this event, modelled and measured storm surge agree very well in magnitude and phase.
George	**Figure A.2.21** presents modelled and measured residual water levels for Tropical Cyclone George at the permanent Berth 3 tide gauge. For this event, modelled and measured storm surge agree very well. Tropical Cyclone George produced a classical cyclone residual water level profile for an intense event which tracks close to the study area. In this event a pronounced set-down is observed as the cyclone approaches Port Hedland followed by a rapid increase in the water level. The modelled and measured peak water level are approximately 105-minutes out of phase; however, it should be noted that the BoM data set only provides 3-hourly track estimates so that the exact location of the cyclone eye in-between data points is uncertain and can be important in influencing the storm surge phase.

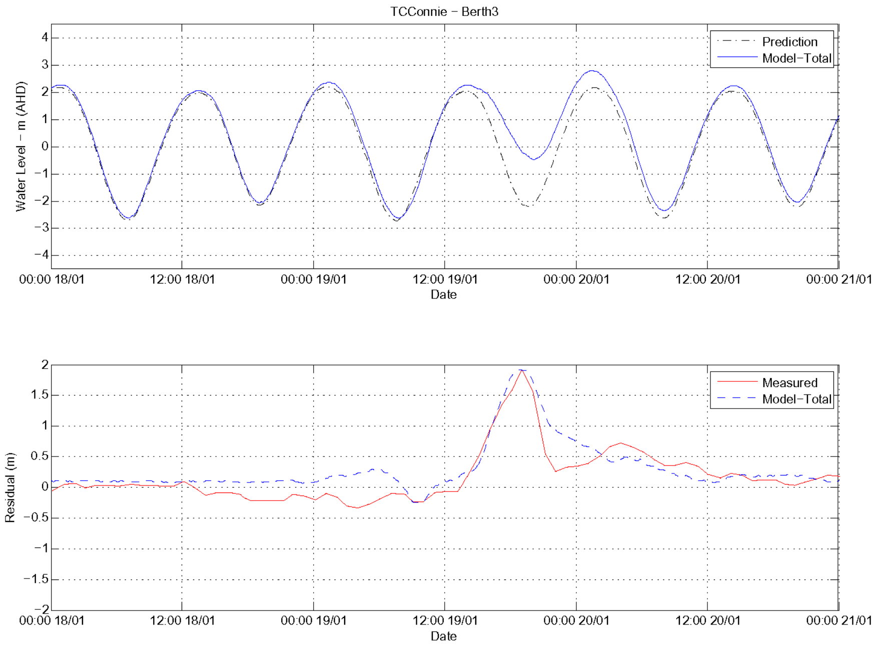


Figure A.2.16: Comparison of Modelled and Measured Storm Surge at Port Hedland – TC Connie.

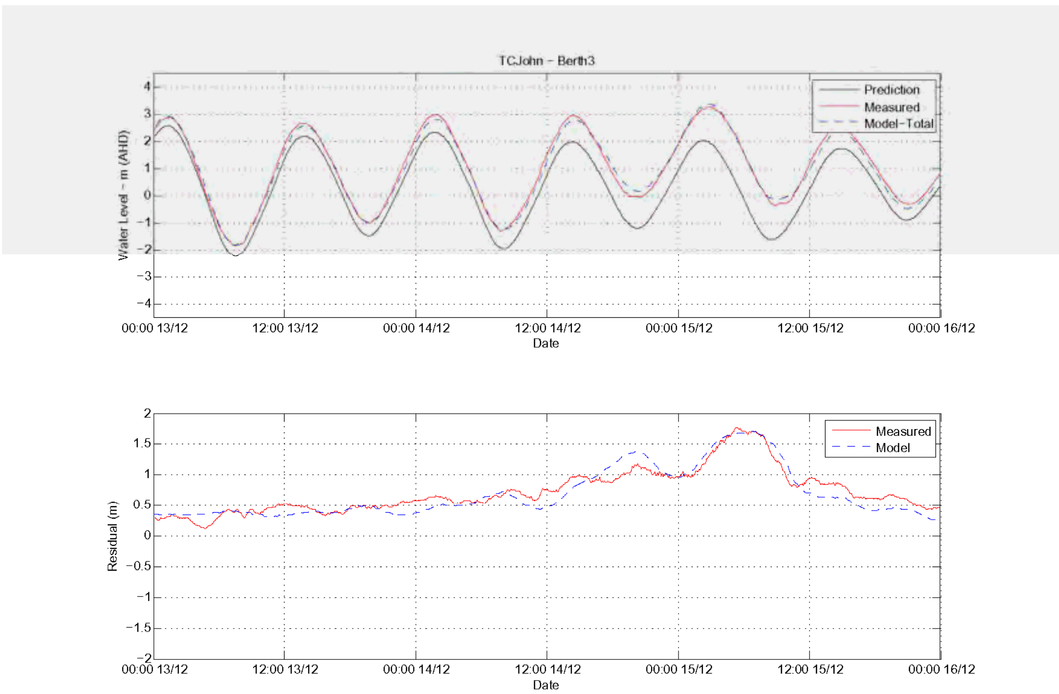


Figure A.2.17: Comparison of Modelled and Measured Storm Surge and Water Levels at Port Hedland – TC John.

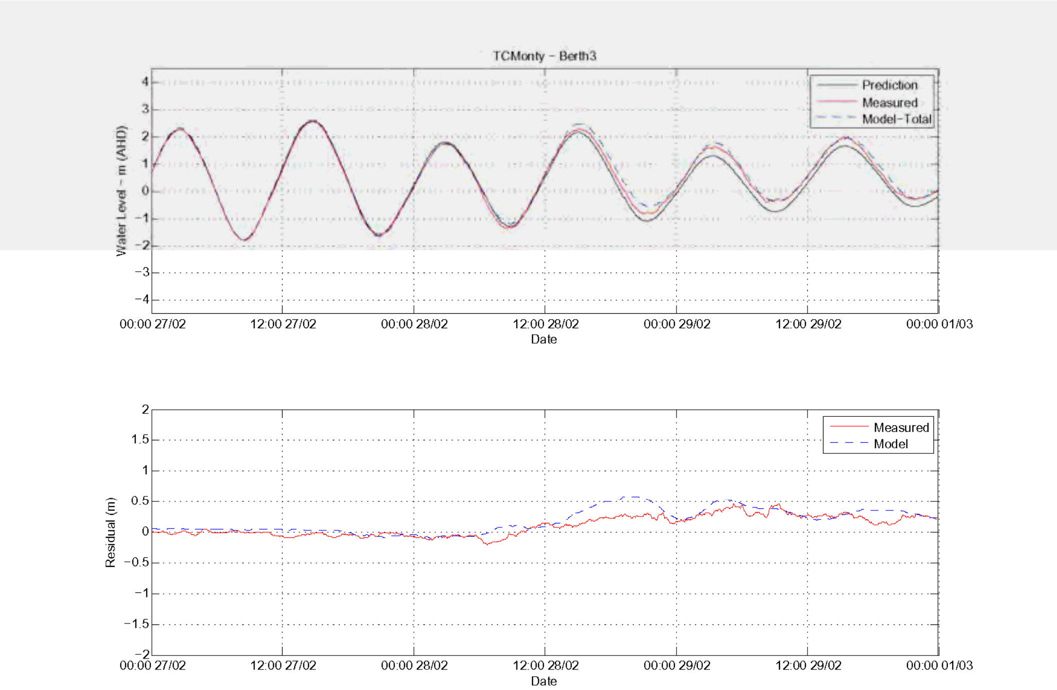


Figure A.2.18: Comparison of Modelled and Measured Storm Surge and Water Levels at Port Hedland – TC Monty.

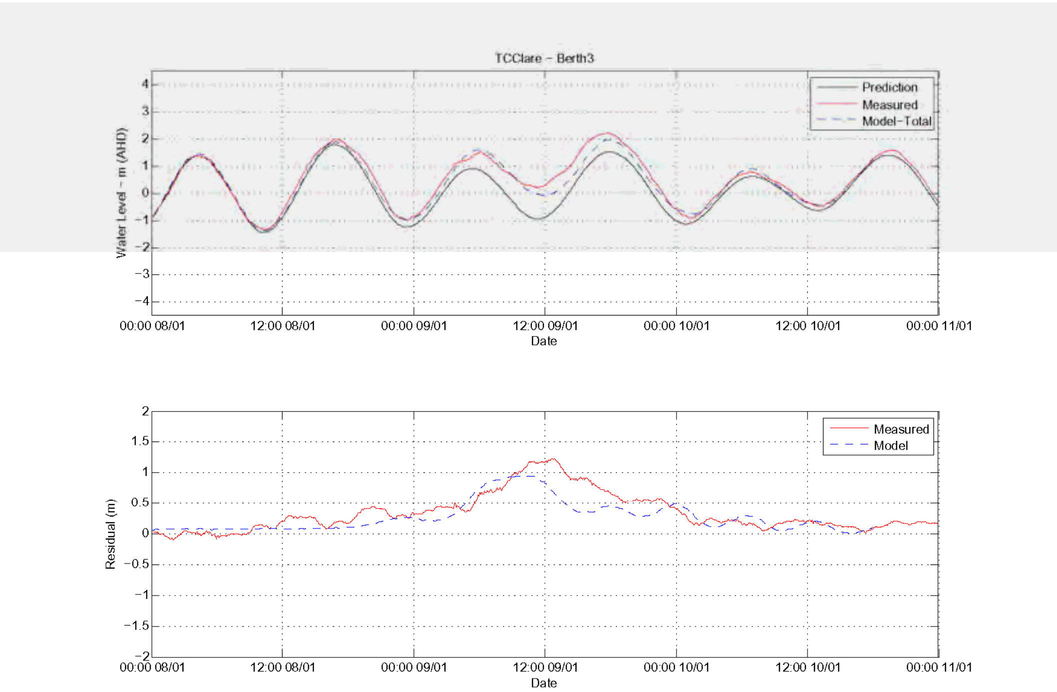


Figure A.2.19: Comparison of Modelled and Measured Storm Surge and Water Levels at Port Hedland – TC Clare.

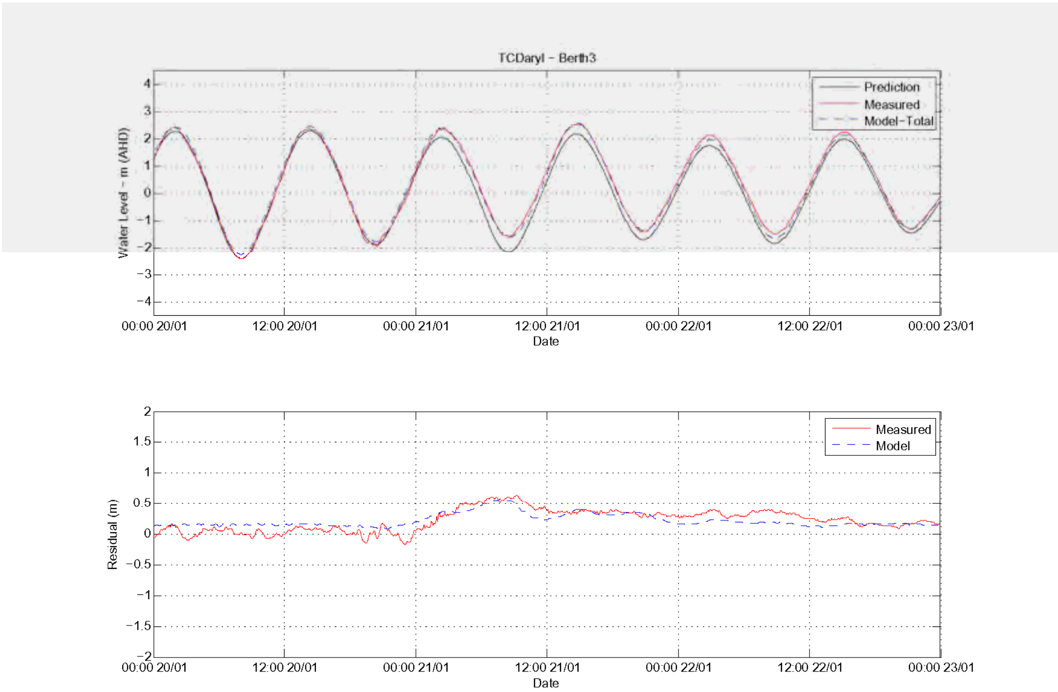


Figure A.2.20: Comparison of Modelled and Measured Storm Surge and Water Levels at Port Hedland – TC Daryl.

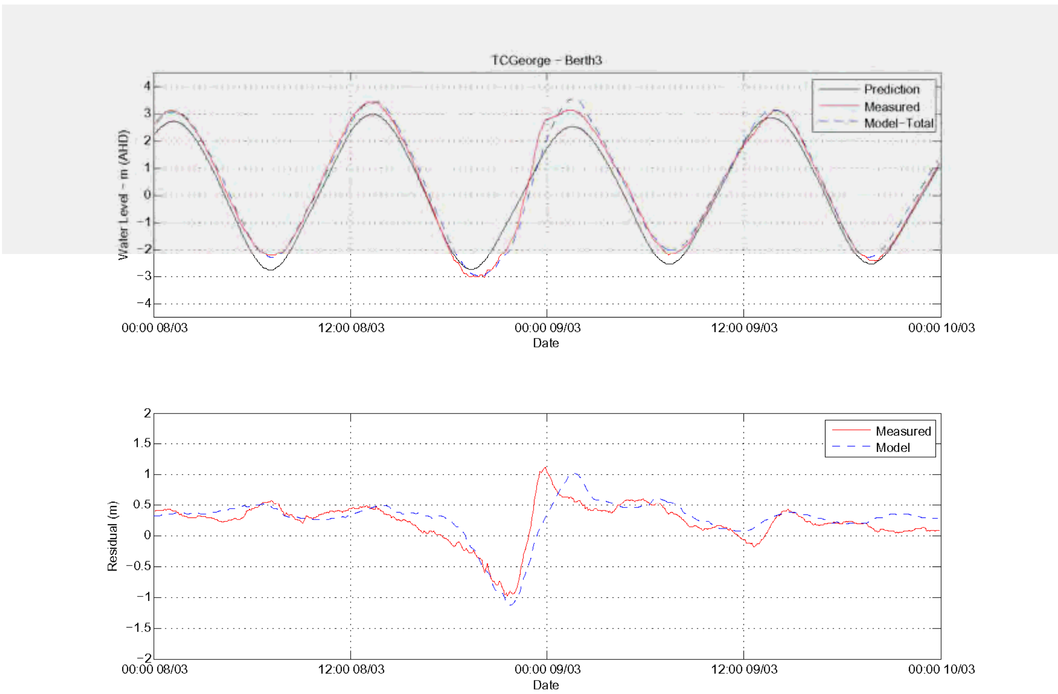


Figure A.2.21: Comparison of Modelled and Measured Storm Surge and Water Levels at Port Hedland – TC George.

Table A.2.3 presents quantitative validation metrics for modelled and measured residual water levels at the Port Hedland Berth-3 tide gauge. In general, modelled peak residual heights are within 0.1m of the measured event peaks. The correlation coefficient between modelled and measured residual water levels over the 48-hours period centred near the peak of each event is generally around 0.8, which indicates good correlation between modelled and measured residual water level s.

Overall, the ocean inundation model system has shown a good degree of calibration compared to measured wind, wave and water level data from six calibration events. It is reasonable to conclude that the calibrated model system provides a reliable tool to assist in defining ocean inundation planning levels.

Table A.2.5: Summary of Delft3D Surge Model Validation Metrics – Berth 3 Port Hedland

Cyclone Event	Maximum Water Level Residual (m)		Time of Maximum Residual		Minimum Water Level Residual (m)		Time of Minimum Residual		Correlation Coefficient +/- 24-hours of peak
	Measured	Modelled	Measured	Modelled	Measured	Modelled	Measured	Modelled	
John	1.77	1.72	05:20 15/12/99	06:50 15/12/99	-	-	-	-	0.94
Monty	0.45	0.58	09:20 29/02/04	19:20 28/02/04	-	-	-	-	0.59
Clare	1.23	0.95	12:50 09/01/06	10:00 09/01/06	-	-	-	-	0.81
Daryl	0.63	0.59	09:10 21/01/06	07:50 21/01/06	-	-	-	-	0.78
George	1.11	1.01	23:50 08/03/07	01:40 09/03/07	-0.94	-1.14	21:30 08/03/07	21:40 08/03/07	0.83

A.3 HINDCAST MODELLING STUDY

A hindcast model simulation study has been undertaken to define design storm tide levels in the study area based on the historical cyclone data. The Port Hedland region has relatively a high cyclone frequency (and intensity) which has resulted in a reasonability large measured data set of historical cyclone events which have generated significant storm surge at Port Hedland. The reliable cyclone record extents back to about 1960 when satellite imagery become adopted by the BoM. However the data resolution, reliability and quality of more recent data is higher compared to the early data collected around 1960. For this study component, Cardno has adopted the post-1960 data set.

A.3.1 Historical Sample Identification

Since 1960, Cardno identified a sample of 32 historical cyclones which passed with approximately 300km of Port Hedland and which had a central pressure below 980hPa. These events were selected for detailed model simulations to hindcast storm tide levels across the study area. **Figure A.3.1** presents a plan view of the historical cyclone tracks investigated in this study and **Table A.3.1** presents some details on each historical event.

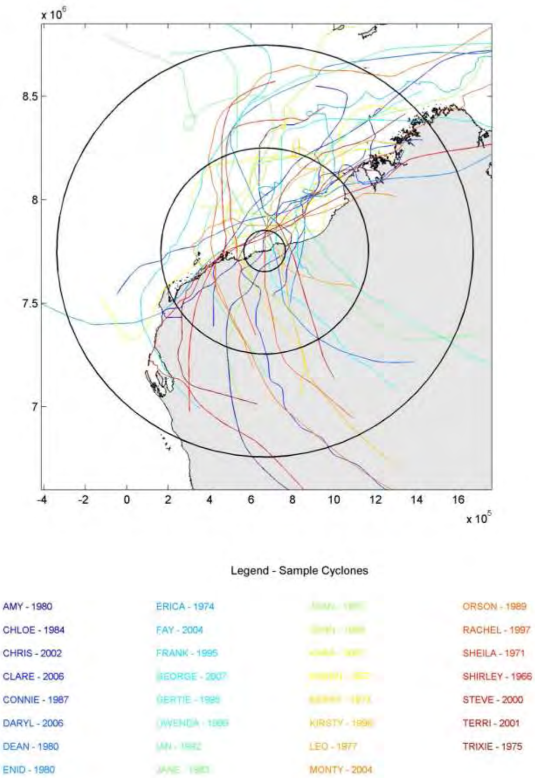


Figure A.3.1: Cyclone Tracks of Selected Historical Events for Hindcast Study.

Table A.3.1: Cyclone Track Characteristics for Hindcast Cyclone Events.

Cyclone Event	Date	Minimum Central Pressure (hPa)	Minimum Distance to Port Hedland (km)	Central Pressure at Minimum Distance (hPa)
SHIRLEY	Mar-1966	965	191	965
SHEILA	Feb-1971	925	134	925
KERRY	Jan-1973	960	115	963
ERICA	Dec-1973	977	137	977
TRIXIE	Feb-1975	925	79	945
JOAN	Dec-1975	915	51	915
KAREN	Mar-1977	970	98	973
LEO	Mar-1977	960	50	965
AMY	Jan-80	915	61	944
DEAN	Jan-80	930	46	931
ENID	Feb-80	930	176	947
IAN	Mar-82	930	308	960
JANE	Jan-83	947	112	947
CHLOE	Feb-84	955	83	955
FRANK	Dec-84	950	397	950
GERTIE	Jan-85	965	228	965
CONNIE	Jan-87	950	8	950
ORSON	Apr-89	905	246	918
KIRSTY	Mar-96	935	88	941
RACHEL	Jan-97	965	3	970
GWENDA	Apr-99	900	33	980
JOHN	Dec-99	915	114	943
STEVE	Mar-00	975	60	975
TERRI	Jan-01	975	116	980
CHRIS	Feb-02	915	126	947
MONTY	Feb-04	935	124	981
FAY	Mar-04	910	159	942
CLARE	Jan-06	960	165	960
DARYL	Jan-06	976	163	976
GEORGE	Mar-07	902	39	933
KARA	Mar-07	920	168	952
LEO	Mar-1977	960	50	965
AMY	Jan-1980	915	61	944
DEAN	Jan-1980	930	46	931
ENID	Feb-1980	930	176	947
IAN	Mar-1982	930	308	960
JANE	Jan-1983	947	112	947

A.3.2 Hindcast Study Methodology

The ocean inundation model system presented in **Sections A.1** and **A.2** has been applied to simulate the combined wind, wave and storm tide over the whole study area. For the hindcast study, the following modelling approach was adopted:-

1. Hindcast the wind field over the Northwest shelf using the modified Holland model;
2. Model overall wave conditions using the Northwest shelf scale SWAN model which provides boundary wave conditions for the continental shelf and nearshore scale model;
3. For the period of the cyclone simulation, undertake a simulation for each cyclone event of the modelled tide conditions over the whole model without wind, wave or air pressure forcing;
4. Undertake model simulations with pressure, wind, wave and tide forcing on the Delft3D model for each cyclone event;
5. For the Port Hedland region, calculate the modelled residual based on the difference between the tide only (Step 3) and full process simulation (Step 4);
6. Combine the modelled residual water level time series with the predicted tide for Port Hedland based on the published constants for the Port Hedland Standard Port (Australian Hydrographic Office, 2009);
7. Extract the maximum water level for each event from the output locations;
8. Apply appropriate Extreme Value Analyses to calculate total still water levels for specified ARI's.

A.3.3 Summary of Hindcast Results

Table A.3.2 presents a comparison of the hindcast peak total storm surge and also the peak total still water level from the model simulations at Port Hedland, Site 2 near the entrance to the Turner River, and also the Shellborough site for the 32 cyclone events. The results indicate that the hindcast surge and peak total still water levels for the Port Hedland and Site 2 locations are generally consistent from event to event. The Shellborough site which is located more than 90km east of Port Hedland has significantly different results for particular cyclone events. For the Port Hedland and Site 2 location, Tropical Cyclone Joan generated the largest storm surge. For the Shellborough site, Tropical George generated the largest hindcast surge.

A plan view of the key model output locations in presented in **Figure A.1.4**.

Table A.3.2: Summary of Hindcast Peak Surge and Total Still Water Level (TSWL) for Key Study locations

Cyclone Event	Port Hedland (Berth-3 Tide Gauge)		Site 2 (Turner River)		Shellborough	
	Peak Surge (m)	Peak TSWL (m AHD)*	Peak Surge (m)	Peak TSWL (m AHD)*	Peak Surge (m)	Peak TSWL (m AHD)#
Rachel	0.66	2.54	0.53	2.54	1.58	3.06
Orson	1.44	3.46	1.51	3.43	0.94	3.92
Monty	0.58	2.91	0.65	2.92	0.66	3.51
Leo	0.37	2.86	0.43	2.87	1.81	3.42
Kirsty	0.21	3.15	0.31	3.17	0.55	3.95
Kerry	1.59	4.09	1.60	4.24	0.93	4.36
Kara	0.48	3.05	0.59	3.05	0.96	3.75
Jane	0.38	2.09	0.43	2.11	1.70	2.59
Ian	0.62	2.61	0.70	2.62	0.62	3.13
George	0.81	3.53	0.60	3.42	5.33	4.36
Frank	0.90	3.12	1.03	3.14	0.83	3.67
Fay	0.17	3.26	0.18	3.27	0.33	4.00
Dean	0.77	2.86	0.87	2.85	3.07	3.62
Daryl	0.59	2.64	0.66	2.64	0.47	3.03
Connie	1.92	3.09	1.44	3.19	1.70	3.80
Clare	0.95	2.38	1.08	2.39	0.64	2.97
Chloe	1.25	2.83	1.33	3.00	1.17	2.65
John	1.52	3.37	1.70	3.58	0.92	3.45
Gertie	0.96	2.07	1.08	2.07	0.55	2.29
Sheila	1.52	3.25	1.81	3.27	0.98	3.97
Karen	1.23	3.64	1.21	3.67	1.09	4.29
Shirley	0.85	2.18	0.99	2.18	0.47	2.75
Joan	3.51	3.55	3.80	3.66	1.68	4.06
Steve	0.62	3.13	0.68	3.15	1.13	3.65
Trixie	1.30	3.02	1.45	3.13	1.52	3.49
Chris	0.12	2.49	0.13	2.50	0.36	3.07
Enid	0.14	3.25	0.22	3.22	0.27	4.02
Erica	0.89	2.64	1.01	2.63	0.79	3.01
Gwenda	1.27	2.83	1.39	2.84	1.58	3.41
Terri	0.18	2.71	0.22	2.73	0.91	3.16
Amy	0.47	2.34	0.57	2.37	1.15	3.07

*** Peak TSWL based on predicted tide using the astronomical constants published by AHS (2009).**
 # Peak TSWL based on Delft3D model.

The design criteria presented in this report have been determined using Extreme Value Analysis (EVA) statistics. During the analysis of the model results, Type-1 (Gumbel) and Type-III (Weibull) distributions were considered. Overall, the Type-III distribution was determined to be preferable for this study as it is a more flexible distribution that can better accommodate data sets which have extreme outlying values which is the case for this study. The parameters for the EVA distribution were determined using a Maximum Likelihood technique as recommended by van Vledder *et al* (1993) and Goda (2000). Confidence intervals were determined using a boot-strapping procedure.

Table A.3.3: Summary of Peak Total Still Water Level (TSWL) for Port Hedland (Berth-3 Tide Gauge) - Selected ARI's

ARI (years)	Port Hedland		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
2	3.1**	3.0	3.2
10	3.6	3.3	3.8
25	3.8	3.5	4.2
50	4.0	3.6	4.4
100	4.2	3.7	4.7

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

** Value below HAT at Port Hedland

Table A.3.4: Summary of Peak Total Still Water Level (TSWL) for Site 1 - Selected ARI's

ARI (years)	Site 1		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
2	3.1**	3.0	3.2
10	3.6	3.4	3.9
25	3.9	3.5	4.3
50	4.1	3.6	4.6
100	4.4	3.7	5.0

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

** Value below HAT at Port Hedland

Table A.3.5: Summary of Peak Total Still Water Level (TSWL) for Site 2 (Turner River Entrance)

ARI (years)	Site 2		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
2	3.1**	3.0	3.2
10	3.6	3.4	3.9
25	3.9	3.5	4.3
50	4.1	3.6	4.6
100	4.2	3.6	4.9

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

** Value below HAT at Port Hedland

Table A.3.6: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough

ARI (years)	Shellborough		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
2	3.2**	3.1	3.4
10	4.0	3.9	4.2
25	4.4	4.2	4.5
50	4.6	4.4	4.8
100	4.7	4.5	5.0

* Peak Total Still Water Level based on modelled tide plane from the Delft3D model

** Value below HAT at Port Hedland

A.4 MONTE CARLO CYCLONE MODEL

This section of the report describes the study methodology and set up of the Monte Carlo synthetic cyclone track model developed for the *Port Hedland Coastal Vulnerability Study* to generate stochastic, synthetic cyclone tracks and then rank their intensity in terms of coastal water levels at the study sites. The storm surge and total water level associated with the synthetic cyclone tracks were assessed using a tiered ranking system that progressively identifies and models the design storm events with greater accuracy. The final stage of the ranking process involves the application of the full process model for design events using the coupled wind, wave and hydrodynamic model system described in **Section A.3.**

A.4.1 Study Methodology

A major limitation in developing long return period design criteria and planning levels for cyclone conditions is the limited reliable cyclone record. Within Australia, the reliable cyclone track data extends back to approximately 1960 when over-the-horizon radar and satellite data for cyclones began to be collected. At Port Hedland, there have been 77 cyclones with a central pressure below 980hPa that have passed within 500km of Port Hedland since 1960. Whilst this is a large number of cyclones in the Australian context, in order to appropriately develop extreme state design criteria, for example, for the 100-years Average Recurrence Interval (ARI) or less frequent design events, this is a relatively small sample from a data period that is quite a short record relative to the required planning period criteria for this study.

There are two general methods available to address the limited historical records. Both methods require the application of Extreme Value Analyses (EVA) which, provided that a particular met-ocean parameter conforms with an applicable extreme value distribution, for example a Gumbel (Type-I) or Weibull (Type-III) distribution, EVA can be applied to reliably estimate design conditions which have an ARI 2 to 3 times greater than the record length of the measured data. In the case of cyclones at Port Hedland, hindcast event based EVA techniques are applicable for estimation of design criteria for the 100-years ARI design parameters. The hindcast method requires the prior hindcasting of a selected number of historical events - assuming that the met-ocean data has not been recorded. The limited number of events then affects the confidence limits of the design parameters developed from the data using the EVA.

An alternative approach to hindcasting is to generate a much longer 'synthetic' data record that can then be used to determine long return period conditions using EVA methods or the more basic approach of ranking the peak outputs from synthetic events. The synthetic data is usually developed based on analyses of the measured data record. The development of synthetic cyclone tracks to investigate long return period cyclonic conditions is a widely adopted practice in oceanography and coastal engineering. In the Australian context, some notable references on synthetic cyclone track modelling are Hardy *et al.* (2009) and the Department of Natural Resources and Mines (2001). In Queensland, which is subject to tropical cyclone risk along its entire coastline, it is policy for storm tide hazard studies to determine storm tide levels using a long duration synthetic cyclone modelling approach. Examples of two major Queensland storm tide hazard studies that include synthetic cyclone and Monte Carlo modelling include Hardy *et al.* (2004) and Cardno Lawson Treloar (2009a and b).

It should be noted that although the Monte Carlo modelling approach allows the generation of a long-term synthetic data set, the Monte Carlo model is still based on a relatively small sample of historical cyclones. As such, the Monte Carlo modelling approach still has limitations and uncertainty related to extrapolation.

For this study the Monte Carlo procedure was used to generate 10,000 years of synthetic cyclone tracks (approximately 16,000). The synthetic cyclone tracks were then ranked through the use of a simple regression model. From this ranking the top 1000 cyclones were then modelled using a regional storm tide model that incorporates the interaction of tide with pressure and wind surge processes. Finally, EVA is performed on the top 1000 events to identify specific design tracks for full process modelling.

A.4.2 Model Set Up

A.4.2.1 Cyclone Database

The BoM maintains a database of cyclone track data for the Australian region that includes track location, central pressure and, if available, maximum wind speed, gale extent and eye diameter, as well as various other cyclone track data. The complete cyclone database was filtered to extract cyclone events that have passed within a 500km radius of Port Hedland since 1960 and for which a recorded central pressure below 980hPa occurred, there by producing an appropriate cyclone database for Port Hedland. **Figure A.4.1** presents a plot of the historical cyclone events upon which the Monte Carlo model was established.

The historical track data was then analysed in terms of both space and time to identify temporal and spatial relationships for key cyclone parameters. Using this database, statistical distributions of key cyclone parameters have been estimated in order to develop a Monte Carlo cyclone track model in which statistically and physically realistic synthetic cyclone tracks can be produced, rather than idealized straight synthetic tracks or translated (and perturbed) historical tracks/parameters, which approach is commonly adopted in other Monte Carlo cyclone track models.

The key track parameters that have been analysed for cyclones that have passed within a 500km radius of Port Hedland and have a recorded a central pressure below 980hPa include:-

- Time of origin (adopted to be the time at which it entered a 1,000km radius from Port Hedland);
- Location of origin;
- Central pressure;
- Forward speed; and
- Cyclone heading.

Due to the variability present with respect to frequency of cyclone track information within the database, the historical cyclone tracks were normalised to a one hour sample interval using a cubic interpolation procedure. The data for each parameter has been analysed to establish statistical descriptions of initial values and their derivatives with respect to time over the course of the cyclone tracks.

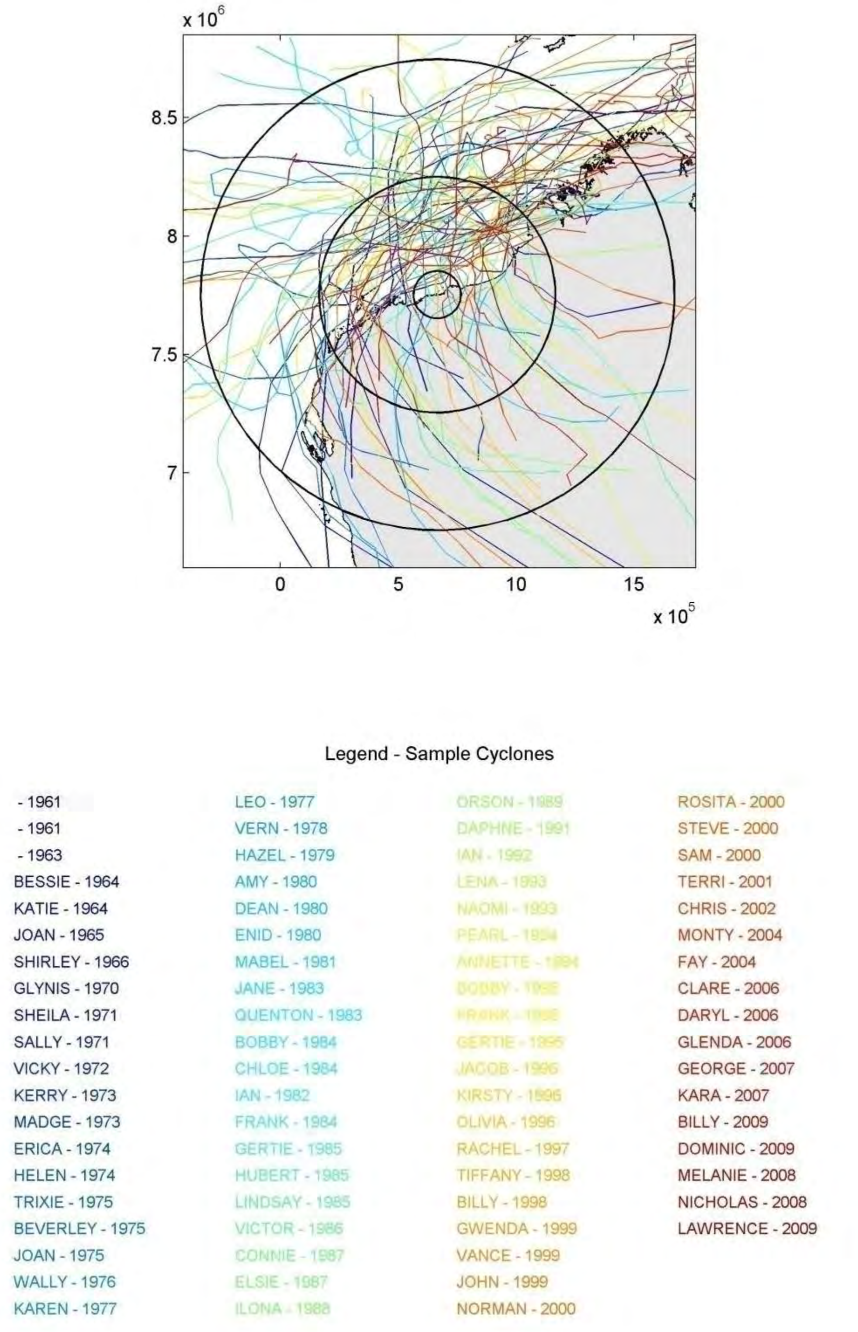


Figure A.4.1: Historical cyclone tracks used to populate the Monte Carlo track model database.

A.4.2.2 Distribution Grid

In the BoM cyclone track data set, strong spatial trends exist for key cyclone parameters such as the change in forward speed, cyclone heading and central pressure. In order to characterise the spatial variability, the major domain was divided into a 250km x 250km Cartesian grid. Within each grid cell, discrete distributions of the time rate-of-change for the key cyclone parameters were generated. In addition, in order to reproduce the intensification of cyclones that occurs just prior to land crossing, and the weakening just after land crossing, grid cells straddling the coast had their distributions separated into over-land and over-water cases. **Figure A.4.2** presents the 250km x 250km Cartesian grid used in the Monte Carlo synthetic cyclone track model with land/water grid cells shown in blue.

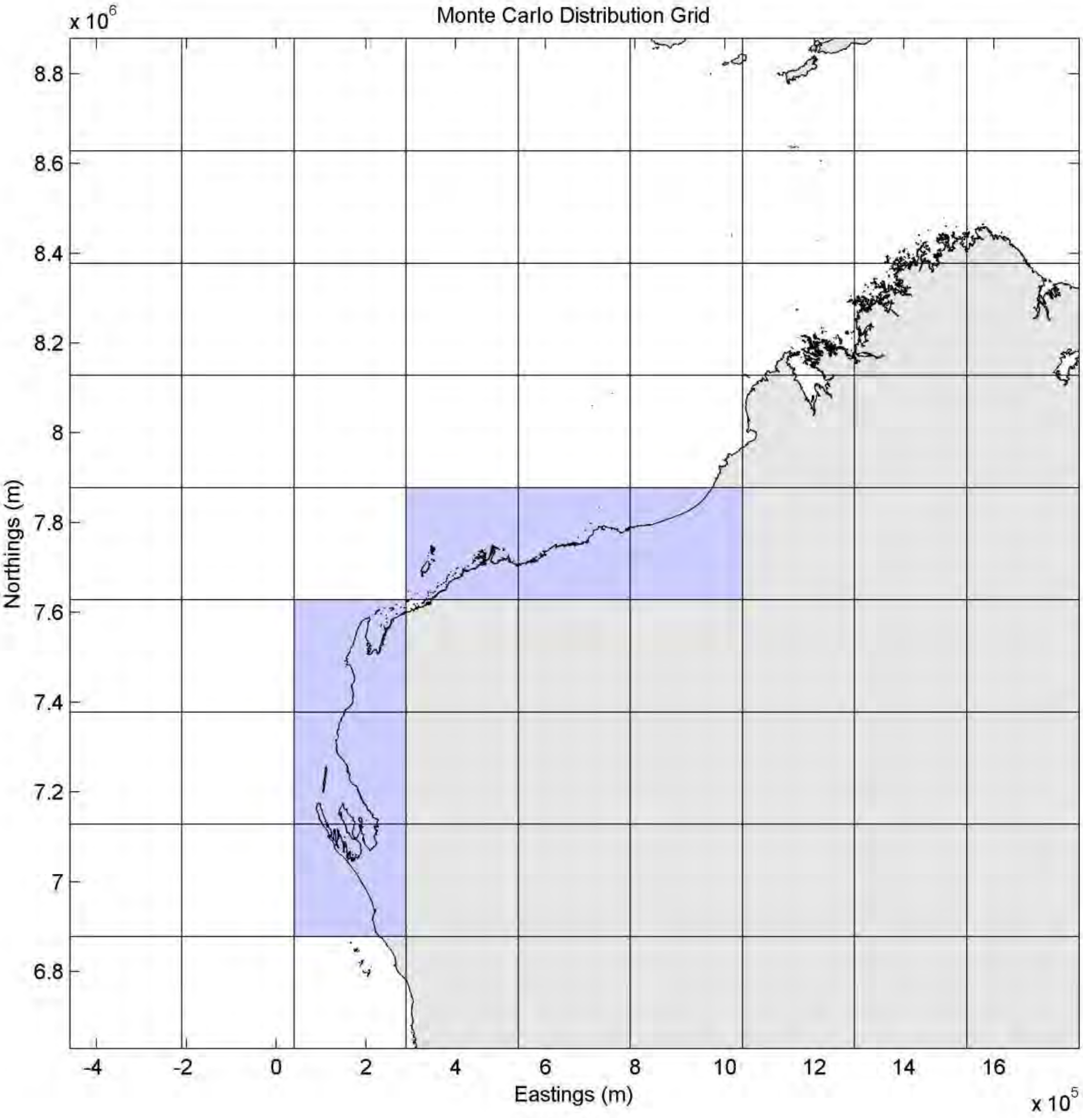


Figure A.4.2: Monte Carlo Cartesian distribution grid, cells defined as land/water cells are shaded.

A.4.2.3 Generalised Random Walk Process

Each cyclone is modelled as a generalised random walk process (**Equation A.4.1**) with the densities of Y_0 (initial parameter value) and delta Y_i (change in Y with time) estimated from the historical cyclone database.

$$Y = Y_0 + \sum_{i=1}^{n} Y_i \qquad \textbf{\textit{Equation A.4.1}}$$

Each synthetic cyclone is initiated at 1,000km range from Port Hedland with initial conditions for the time of generation, point of origin, central pressure, heading and speed chosen randomly from their respective (initial condition) distributions.

Table A.4.1 presents the monthly probability density for cyclone generation based on the sample cyclone database and the monthly mean atmospheric pressure calculated from the 3 hourly atmospheric pressure record at Port Hedland Airport. The initiation date and time are chosen by randomly selecting the month from the probability density shown below. The year is randomly chosen from a uniform distribution over the 18.6 years astronomical tide cycle. Finally the day (based on the month and year) and the hour of day for cyclone initiation are randomly selected.

Table A.4.1: Month of Origin for Cyclone Events and Associated Ambient Atmospheric Pressures

Month	% of Cyclones Entering Domain in Month	Monthly Mean Atmospheric Pressure (hPa)
November	0	1008.5
December	23	1006.4
January	46	1005.3
February	23	1005.4
March	7	1008.0
April	1	1011.4

Figure A.4.3 presents histograms of the initial bearing around the outer boundary of the model (relative to Port Hedland at the 1000km radii – see **Figure A.4.1**), track heading, speed and central pressure of the historical cyclones. **Figure A.4.4** presents histograms of the initial bearing, heading, speed and central pressure for each of the 16,000 synthetic cyclones produced by the Monte Carlo model together with similar data from the historical cyclone data base. Good agreement is shown between the sample population and synthetic cyclone initial conditions.

The Monte Carlo synthetic track simulations were performed with a 3-hours time step, consistent with the model sampling interval in the cyclone database. At each time step, based on the synthetic cyclone's current position in the Cartesian grid, a conditional probability is evaluated to determine the change in the key parameters for the next time step. **Equation A.4.2** provides a mathematical description of conditional probability, that is, the joint probability of two events occurring, normalised by the probability of a given event that is known to have occurred. In the Monte Carlo model the change in each key parameter per time step is conditional upon the modelled value of the parameter(s) at the previous time step.

$$P(Y = y \mid X = x) = \frac{P(X = x \cap Y = y)}{P(X = x)}$$ **Equation A.4.2**

In addition a boundary condition was implemented on the change in central pressure by evaluating the kernel smoothing density estimate of the central pressure distribution and scaling the probability of a decrease in central pressure by the probability of the cyclone having a lower central pressure than it currently does.

For qualitative comparison, **Figure A.4.5** presents the cyclone tracks produced by the Monte Carlo model for a simulation period equivalent to the historical sample. Overall the Monte Carlo model reproduced the major characteristics present in the sample set very well. Cyclone generation generally occurs in the north-east offshore from the Kimberley coast, with cyclones then propagating in a south-westerly direction before turning south-eastward and crossing the shoreline along the Pilbara coast. Some historical cyclones have continued out into the Indian Ocean and this characteristic also has been reproduced in the model.

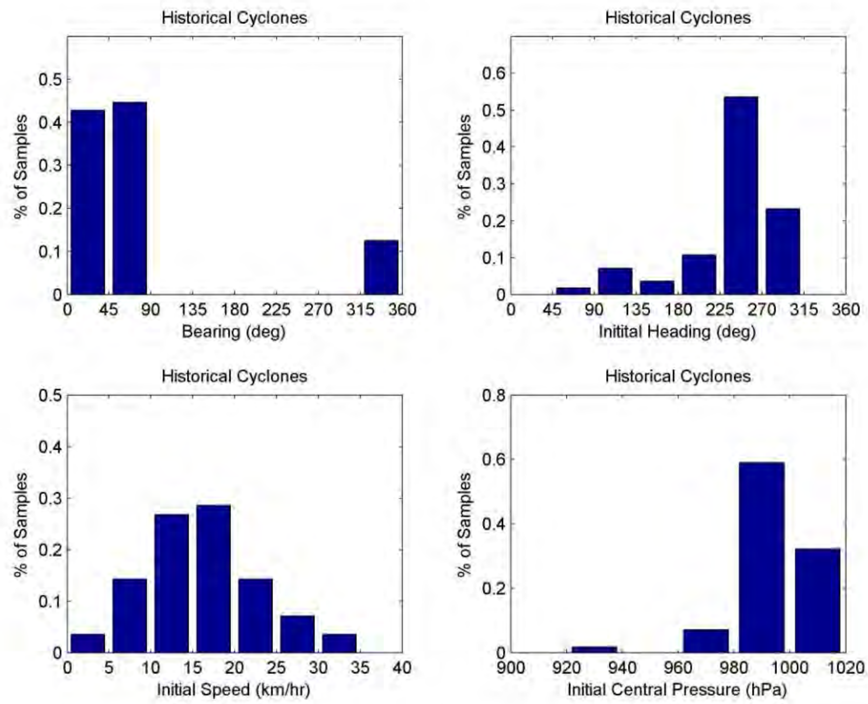


Figure A.4.3: Histograms of initial bearing, heading, speed and central pressure of sample cyclones.

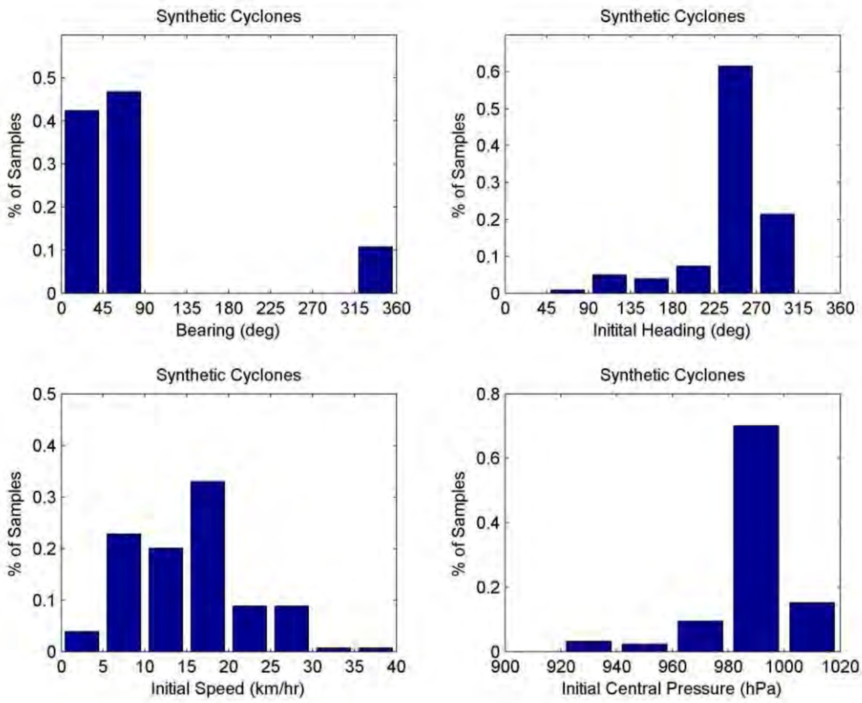


Figure A.4.4: Histograms of initial bearing, heading, speed and central pressure of synthetic cyclones.

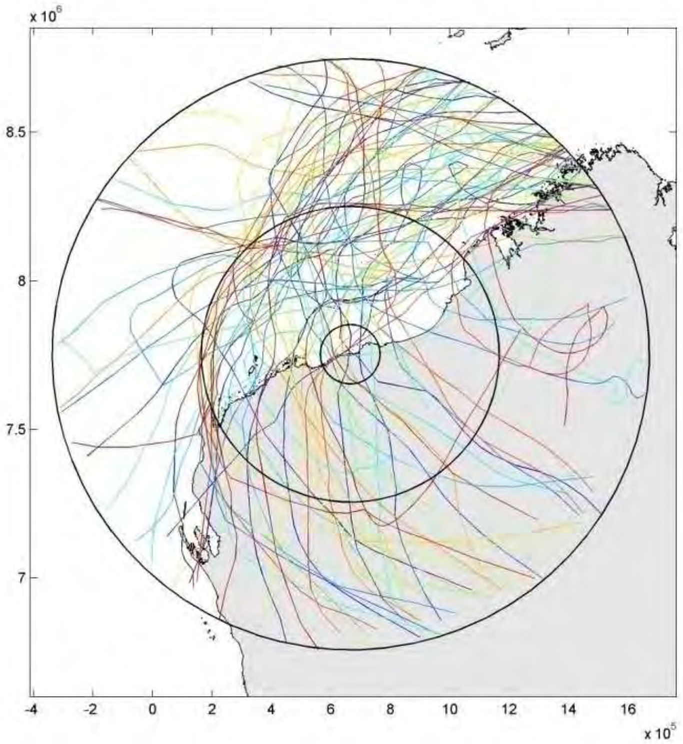


Figure A.4.5: Plot of synthetic cyclone tracks generated by the Monte Carlo cyclone model for an a duration equivalent to the historical sample.

Figure A.4.6 presents a scatter plot of (a) central pressure against latitude for the historical sample population and the equivalent duration Monte Carlo simulation and (b) a scatter plot of the change in central pressure with time against latitude. The major spatial trend present in the historical sample is the intensification north of 19°S and weakening south of 19°S. This is clearly demonstrated in by the scatter plot of change in central pressure with time against latitude. The Monte Carlo model reproduces the spatial trends of intensification and weakening present in the historical sample population.

Figure A.4.7 presents a scatter plot of (a) cyclone heading against latitude for the historical sample and the equivalent duration Monte Carlo simulation and (b) a scatter plot of the change in heading with time against latitude. The major spatial trend present in the historical sample is a south-westward heading in the north and south-eastward heading in the south. The Monte Carlo model reproduces the spatial trends in cyclone heading present in the historical sample population.

Figure A.4.8 presents a scatter plot of (a) cyclone forward speed against latitude for the historical sample and the equivalent duration Monte Carlo simulation and (b) a scatter plot of the change in speed with time against latitude. In general the historical cyclones travel slower in the north and accelerate once they cross the coast and head south-eastward. The Monte Carlo model reproduces the spatial trends in cyclone speed present in the historical sample population.

Figures A.4.9 to **A.4.12** present a comparison between the histograms of central pressure, heading and speed (respectively) between the historical cyclones and the 10,000 years Monte Carlo simulation. Good agreement is shown for these three key parameters.

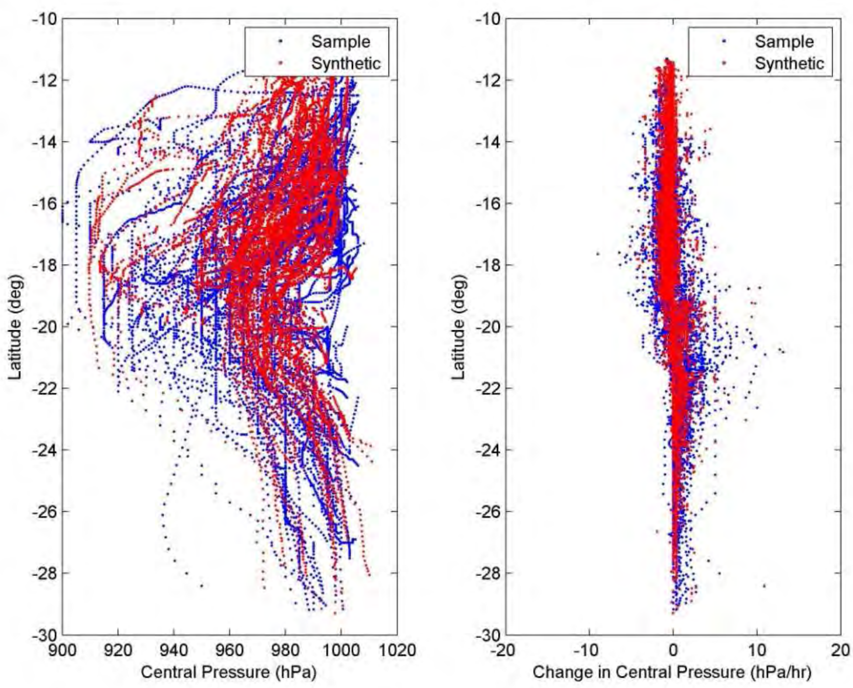


Figure A.4.6: (a) Scatter plot comparing the central pressure against latitude trend present in the sample and synthetic cyclone tracks and (b) a scatter plot of change in central pressure with time against latitude.

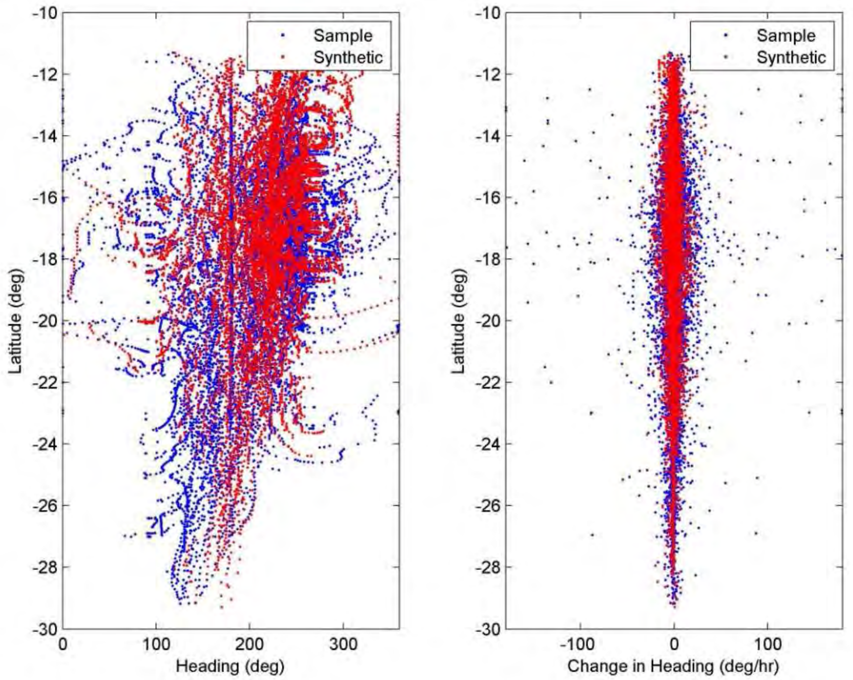


Figure A.4.7: (a) Scatter plot comparing cyclone heading against latitude for the sample and synthetic cyclone tracks and (b) a scatter plot of change in heading with time against latitude.

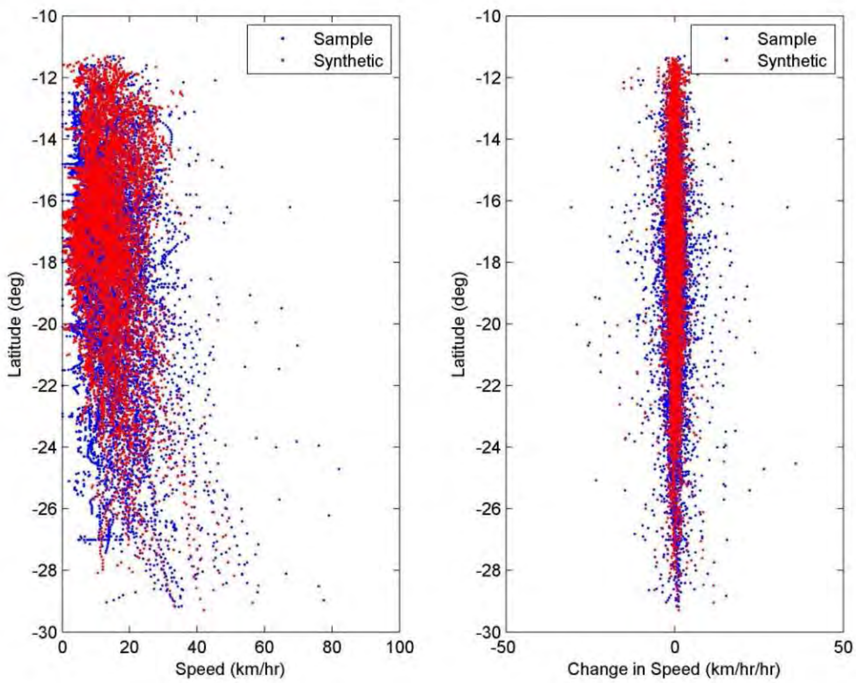


Figure A.4.8: (a) Scatter plot comparing cyclone speed against latitude for the sample and synthetic cyclone tracks and (b) a scatter plot of change in speed with time against latitude.

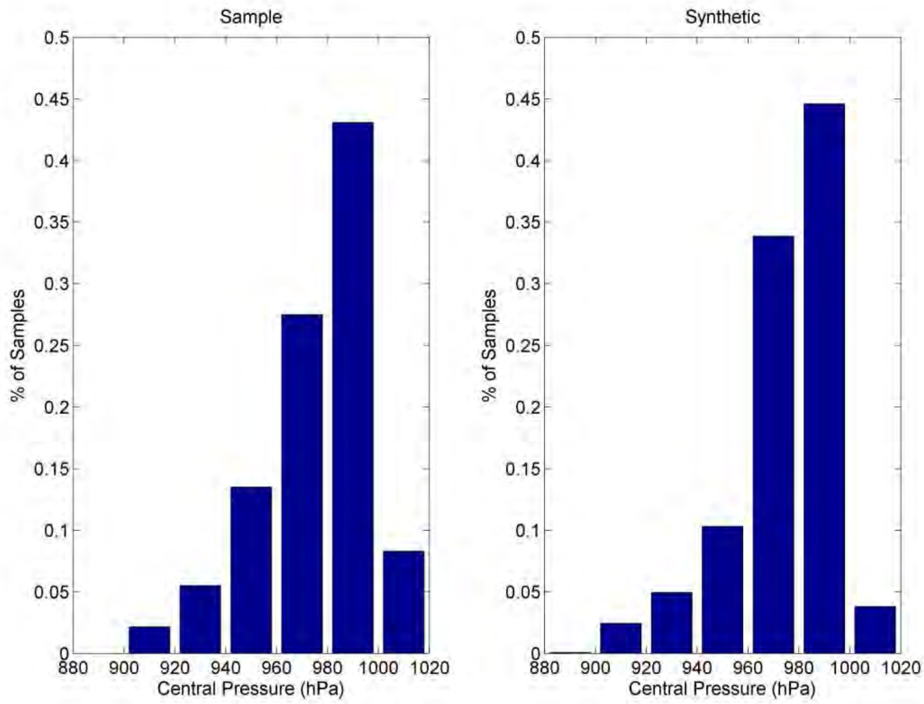


Figure A.4.9: Comparison of histograms of central pressure for sample and synthetic cyclones.

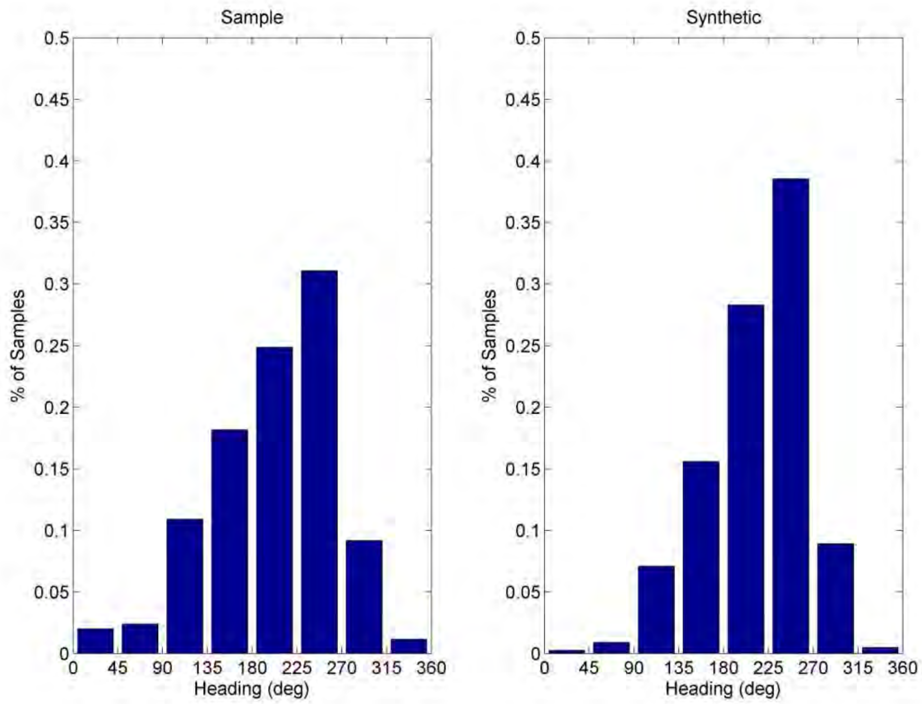


Figure A.4.10: Comparison of histograms of heading for sample and synthetic cyclones.

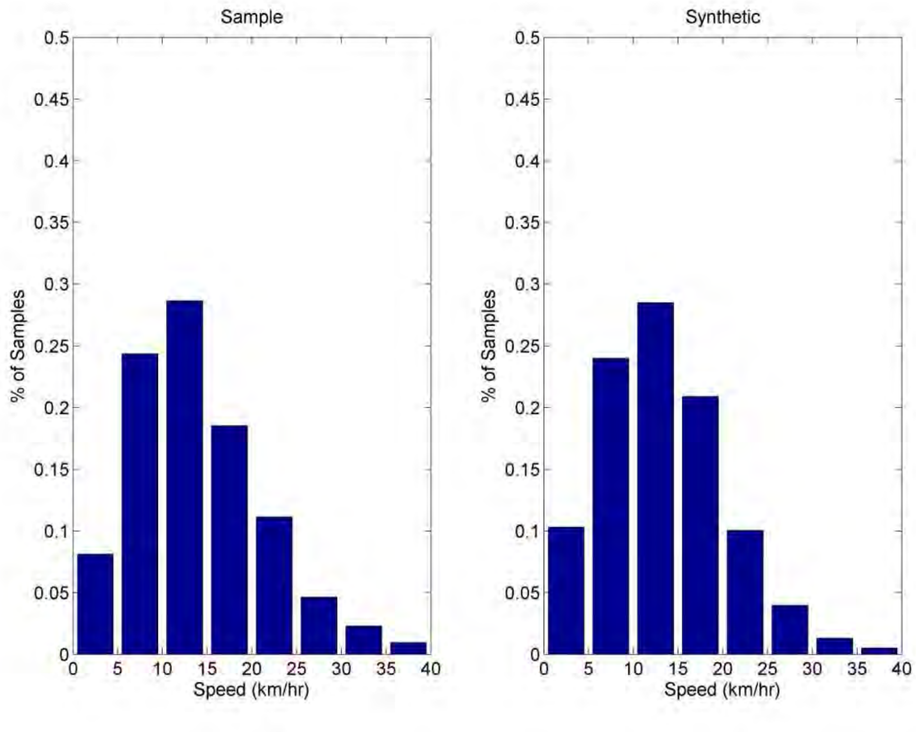


Figure A.4.11: Comparison of histograms of speed for sample and synthetic cyclones.

A.4.3 Initial Track Ranking

In order to effectively assess the potential of a given cyclone track (out of approximately 16,000) to cause elevated water levels at the study site an efficient first order estimate of the associated surge is necessary. Therefore it is necessary to estimate the surge based on track properties only. The surge associated with a given cyclone track can be assumed to be a function of the cyclone characteristics at the point at which it is closest to Port Hedland. The key properties are the minimum distance (D), central pressure at the minimum distance (CP) and the bearing (B) of the cyclone from Port Hedland at the minimum distance. These three parameters provide a proxy to three primary physical processes that determine the surge associated with a cyclone. Distance and central pressure (in interaction) govern the wind speed and inverse barometer, and bearing (due to the rotation of the wind field) acts as a proxy to the set-down or set-up associated with east and west crossing cyclones respectively.

Through the interaction of these three parameters a first order estimate of surge associated with a cyclone can be obtained by **Equation A.4.3** shown below where c_1 through c_4 are fit coefficients. The fit coefficients are determined through a non-linear fit based on the results of the hindcast study.

$$R = c_1 B.e^{c_2 D + c_3 CP} + c_4 \qquad \textbf{Equation A.4.2}$$

Figure A.4.12 shows a scatter plot of the full process model residual plotted against the regression model based on track properties. There is a clear underestimation of the surge for very intense cyclones as the non-linear characteristics of setup dominate. However, given the simplicity of the model, good discrimination between cyclones causing significant surge and cyclones causing little surge is demonstrated with an overall mean squared error of 0.1. Therefore it is an appropriate method to identify tracks likely to produce significant surge at the site.

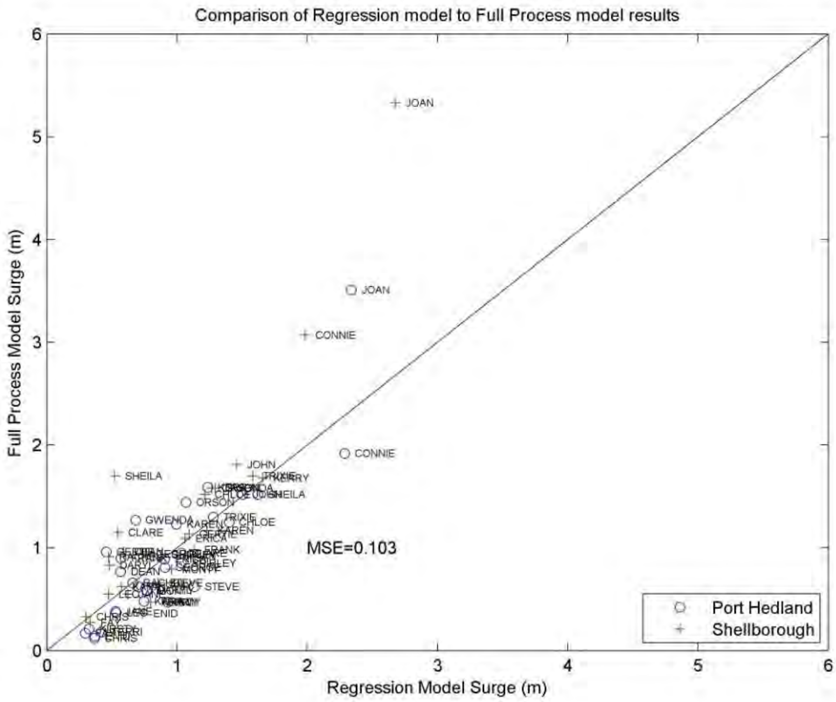


Figure A.4.12: Comparison of regression model to full process model results.

The 10,000 years of cyclone tracks were assessed for both the Port Hedland and Shellborough locations independently (due to the significant distance between the sites) to identify the 1000 tracks most likely to produce significant surge at each site. These 1000 tracks were then modelled with a regional hydrodynamic model to more accurately model the surge associated with each track and its interaction with tidal processes.

Design events identified through the ranking of the 10,000 years of synthetic cyclone tracks were modelled using the coupled wind, wave and hydrodynamic model system. **Figure A.4.12** presents a summary view of the top 1000 cyclone tracks for the Port Hedland area which were simulated by the Monte Carlo model.

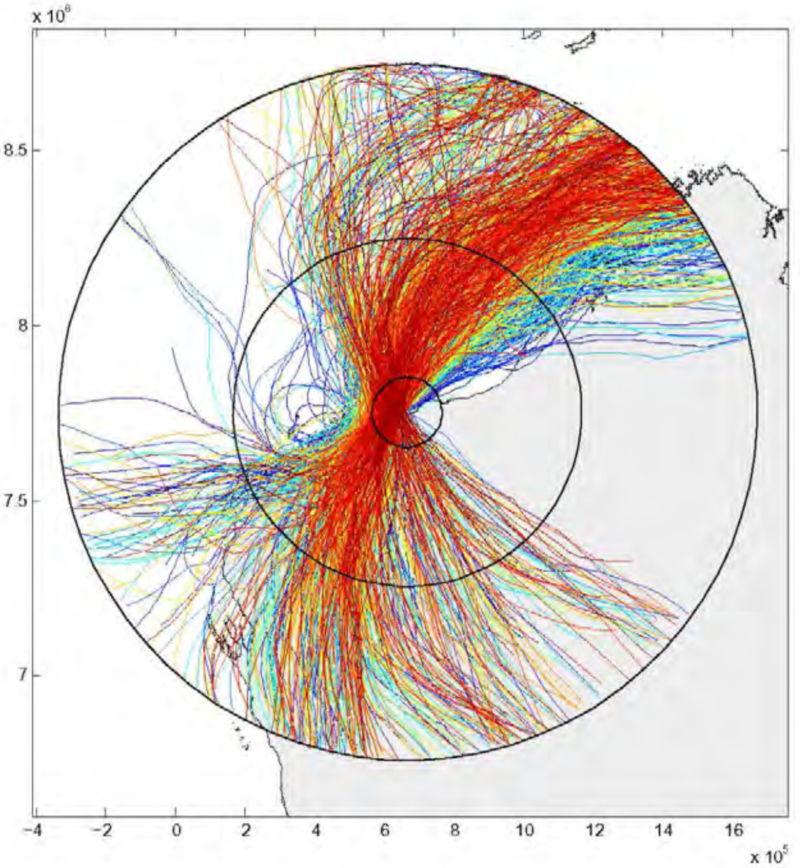


Figure A.4.13: Plot of Top 1000 Modelled Synthetic Cyclone Tracks for a 10,000-year Duration Simulation (Equivalent to Reliable Track Record).

A.4.4 Regional Track Ranking

A regional Delft3D hydrodynamic model was developed based on the continental shelf hydrodynamic grid utilised in the full process model (Grid 1) – see **Figure A.1.4**. The general model setup is the same as that described in **Section A.1.3**. The model had a variable grid size from 1500m x 1500m to 500m x 500m. The astronomical boundary condition as applied to the full process model was applied to the regional track ranking model. The Holland wind field model described in **Section A.1.2** was applied to the model and the model was executed with a time step of 5 minutes, sufficient to resolve tide and wind currents at the regional model grid resolution.

Validation of the regional track ranking model was undertaken for the Port Hedland and Shellborough sites by implementing the hindcast cyclone tracks in the regional model and comparing to the full process model results.

Figure A.4.14 presents a scatter plot of the regional model residual against the full process model residual for Port Hedland harbour. The regional model is expected to underestimate the residual due to the lower model resolution combined with the non-linearity of wind setup in shallow water. In addition the regional model output location was outside the Port Hedland, whilst the full process model output was inside the harbour. However excellent discrimination between cyclones causing significant surge and cyclones causing little surge is demonstrated by the model with approximately 90% of the variability present in the full process model surge accounted for by the regional model. **Figure A.4.15** presents a scatter plot of the regional model residual against the full process model residual for the Shellborough site. Excellent discrimination between cyclones causing significant surge and cyclones causing little surge is demonstrated with the regional model accounting for 90% of the variability present in the full process model.

Each set of the top 1000 tracks identified by the regression model is implemented in the Regional model. The water level time series at each site is analysed and the peak total water level extracted. Finally extreme value analysis (described in **Section A.4.5**) is undertaken to calculate the 100, 200 and 500 year ARI design still water levels. Based on these levels, design cyclone tracks are selected from the top 1000 for full process modelling in the model system described in **Section A.3**.

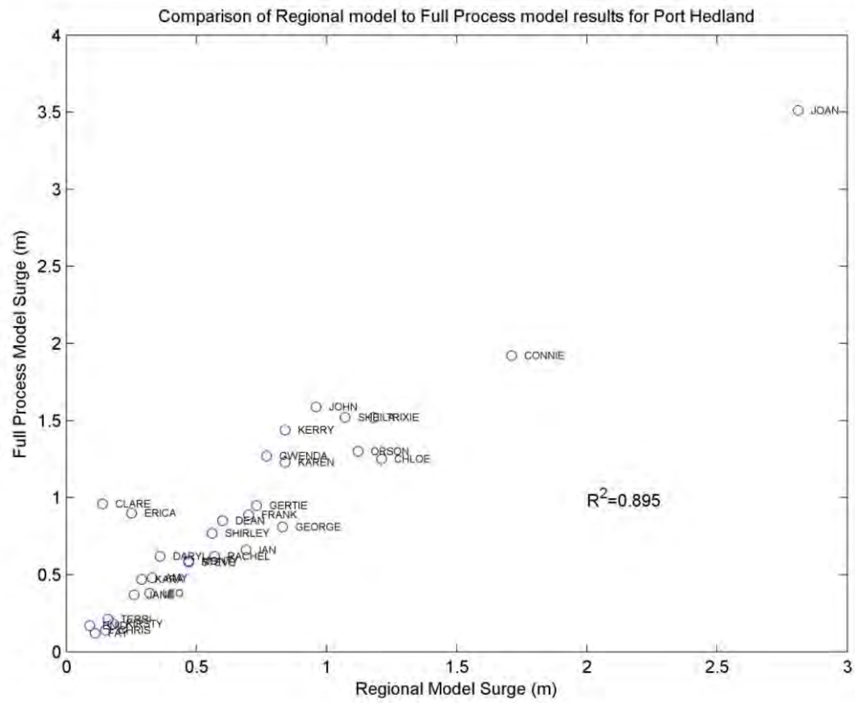


Figure A.4.14: Comparison of regional model to full process model results for Port Hedland harbour.

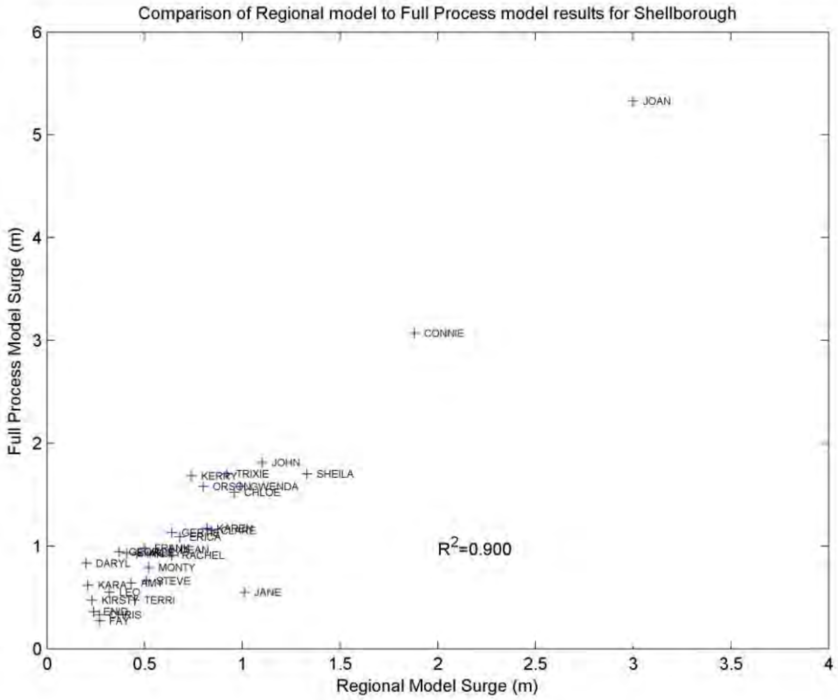


Figure A.4.15: Comparison of Regional model to full process model results Shellborough.

A.4.5 Extreme Value Analysis

The design criteria presented have been determined using Extreme Value Analysis (EVA) statistics. During the analysis of the model results, Type-1 (Gumbel) and Type-III (Weibull) distributions were considered. A peak over threshold value of approximately 0.7 was applied. Overall, the results from the Type-I and Type-III distributions agreed well due to the greater number of extreme events resulting from the Monte Carlo study approach as compared to the Hindcast Study. The Type-III distribution was determined to be preferable for this study due to the slightly more conservative results at higher average recurrence intervals. The parameters for the EVA distribution were determined using a Maximum Likelihood technique as recommended by van Vledder *et al* (1993) and Goda (2000). Confidence intervals were determined using a boot-strapping procedure. Based on the 100, 200 and 500 ARI still water levels established for the Regional model, design cyclone tracks are selected from the top 1000 for full process modelling in the model system described in **Section A.3** allowing for peak still water levels to be determined for each output location.

A.4.6 Summary of Monte Carlo Results

Tables A.4.2 through **A.4.5** presents the summary of results for the Port Hedland harbour, Site 1, Site 2 (Turner River) and Shellborough sites respectively. Peak total still water levels are highest at the Shellborough site as compared to locations closer to Port Hedland. Peak total still water levels at Site 1 and the Turner River entrance greater than the Port Hedland harbour. **Tables A.4.2** through **A.4.5** exclude wave set-up and run-up. Compared to the hindcast study results presented in **Tables 3.3** to **3.6**, the Peak TSWL at 100-years ARI are approximately +0.3m larger from the Monte Carlo modelling study near Port Hedland, and +0.5m higher at Shellborough.

Table A.4.2: Summary of Peak Total Still Water Level (TSWL) for Port Hedland (Berth-3 Tide Gauge) - Selected ARI's

ARI (years)	Port Hedland		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
100	4.5	4.0	5.1
200	4.7	4.0	5.3
500	4.9	4.1	5.6

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.4.3: Summary of Peak Total Still Water Level (TSWL) for Site 1 - Selected ARI's

ARI (years)	Site 1		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
100	4.9	4.2	5.6
200	5.2	4.3	6.0
500	5.4	4.4	6.5

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.4.4: Summary of Peak Total Still Water Level (TSWL) for Site 2 (Turner River Entrance)

ARI (years)	Site 2		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
100	4.8	4.1	5.5
200	5.0	4.2	5.8
500	5.4	4.3	6.4

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.4.5: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough

ARI (years)	Shellborough		
	Peak TSWL (mAHD)#	Lower 95% Conf. Int.	Upper 95% Conf. Int.
100	5.2	4.9	5.4
200	5.7	5.4	6.0
500	5.9	5.5	6.2

Peak Total Still Water Level based on modelled tide plane from the Delft3D model

Tables A.4.6 and **A.4.7** present tables of the key track properties for the selected design cyclone tracks for the Port Hedland and Shellborough sites respectively. Plots of the corresponding tracks are shown in **Figures A.4.16** and **A.4.17.** It should be noted that the total storm water level is the result of the combined astronomical tide and combined storm surge and although the 100-year ARI is not a particularly severe cyclone in its intensity, it coincided with a large spring tide.

Table A.4.6: Design cyclone track properties for Port Hedland.

Cyclone Event (ARI)	Minimum Central Pressure (hPa)	Minimum Distance to Port Hedland (km)	Central Pressure at Minimum Distance (hPa)
100	958.1	86.7	962.2
200	914.1	70.3	915.0
500	921.7	43.3	928.5

Table A.4.7: Design cyclone track properties for Shellborough.

Cyclone Event (ARI)	Minimum Central Pressure (hPa)	Minimum Distance to Shellborough (km)	Central Pressure at Minimum Distance (hPa)
100	920.5	89.4	938.9
200	919.5	25.3	942.5
500	918.4	27.8	942.7

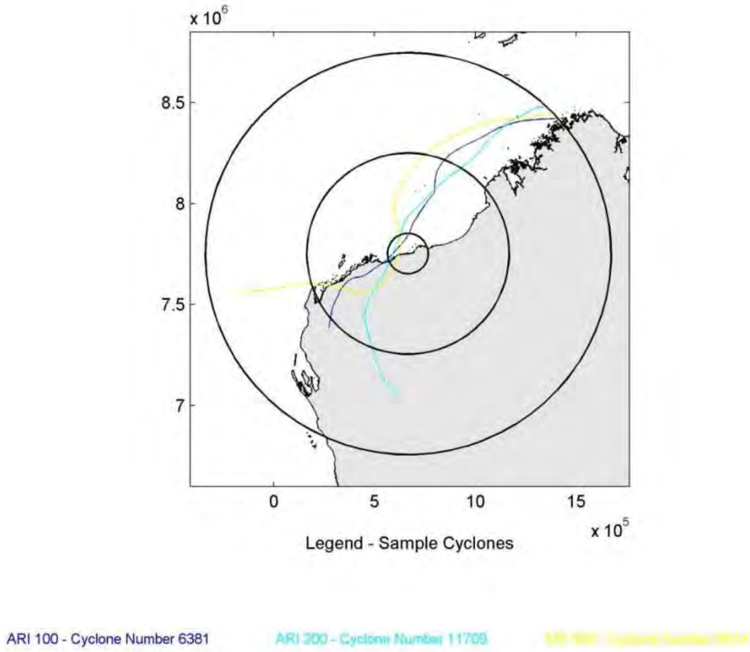


Figure A.4.16: Plot of ARI 100, 200 and 500 design cyclone tracks select in this study for Port Hedland.

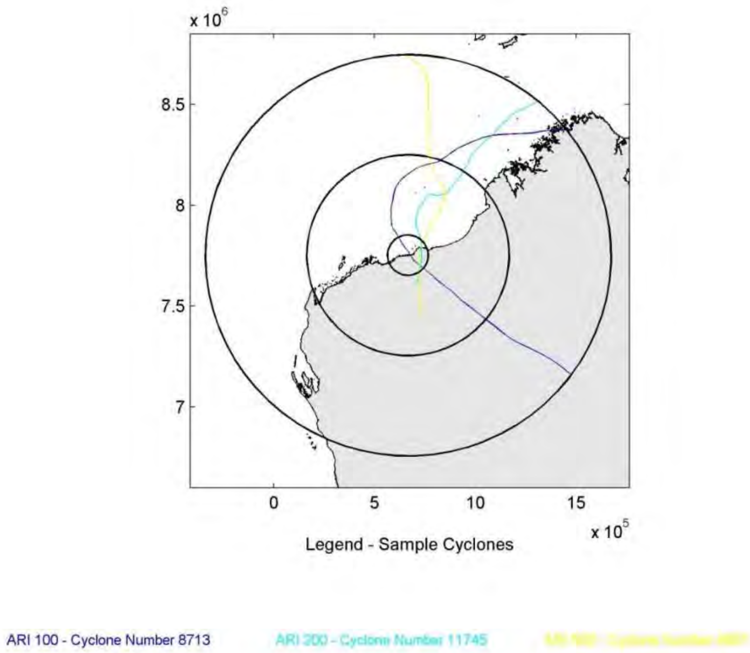


Figure A.4.17: Plot of ARI 100, 200 and 500 design cyclone tracks select in this study for Shellborough.

A.5 INFLUENCE OF TIDE PHASE ON SURGE MAGNITUDE

During the analysis of the hindcast and Monte Carlo modelling results, a general trend of larger modelled storm surge was observed when the peak wind forcing near Port Hedland occurred near low water. The phase of the tide plays an important role in governing the magnitude of observed water level residual for a storm of a given intensity. To demonstrate this process, a series of model runs were undertaken where the cyclone crossing time was shifted relative to the tide phase. A synthetic cyclone that passed within 43km of Port Hedland with a minimum central pressure of 925 hPa coincident with a high tide was moved in time from -6 hours to +6 hours at 2 hour intervals. **Figure A.5.1** provides a time series plot of the modelled tide, the residual for each scenario and the central pressure and distance of the cyclone for each scenario. Larger residual water levels are experienced during falling tide and low water than rising tide and high water.

Greater water level residuals occurs during falling tides and near low water due to the pressure gradient associated ebb tide water surface slope opposing the wind shear stress that is imparted into the water column which results in opposing potential energy sources which act to ‘hold back’ the astronomical tide on the continental shelf. The opposing wind shear stress and tidal forcing is subject to less energy losses compared to the alternative scenario when both the wind shear stress and tidal forcing is in the same direction which results in a greater proportion of kinetic energy which is subject to larger frictional and turbulence related energy losses. The lower water level also further amplifies the non-linearity of the wind setup process in shallow water resulting in larger surge events occurring near low tides.

The rising tide results in a tidal pressure gradient and associated water surface slope that acts in the same direction as the wind. This results in a reduction in surge and a phase shift in the peak as demonstrated by the +2hr scenario where the peak surge is lagged significantly behind the time the cyclone is closest to Port Hedland as compared to other scenarios as the surge wave is limited by the shallow water wave celerity along the coast.

The results from this sensitivity investigation highlights the importance of tidal processes on observed storm tides, particularly in regions with relatively large tide ranges. For other coastal vulnerability investigations on the North-Western coastline of Western Australia, the inclusion of astronomical tides is an important requirement for any numerical modelling or calculation of design ocean water levels.

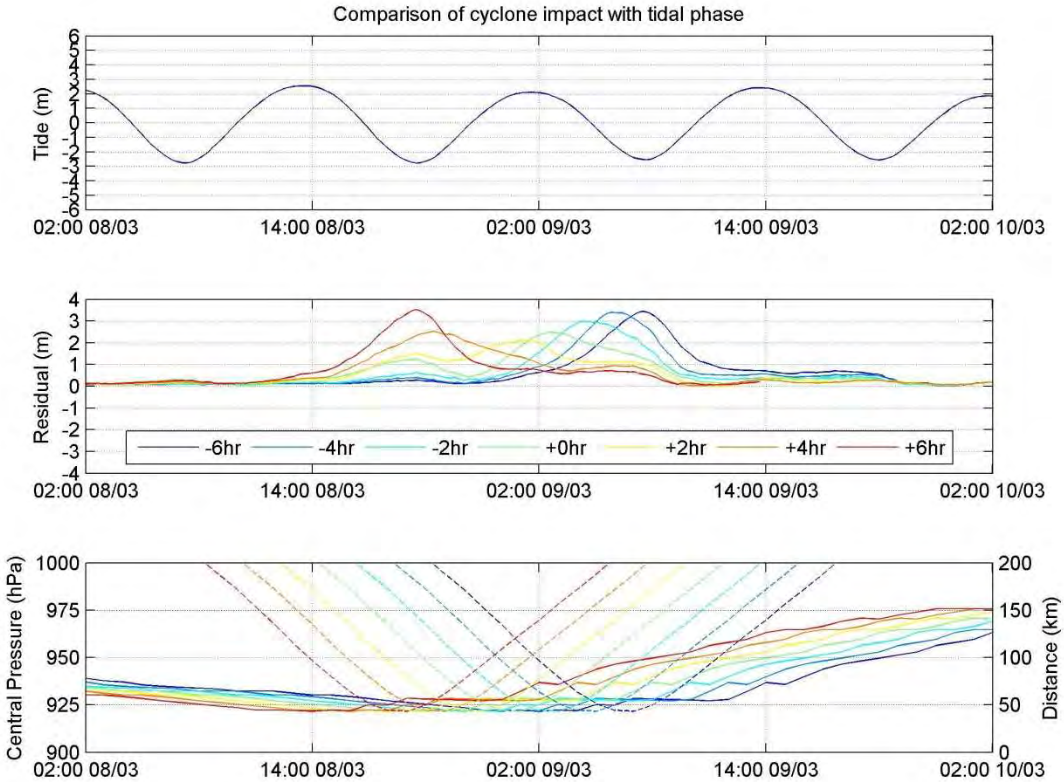


Figure A.5.1: Comparison of modelled residual water inside Port Hedland depending on cyclone and tide phasing for a specified cyclone event.

A.6 NON-CYCLONIC AND SEASONAL WATER LEVEL VARIABILITY

Cardno has undertaken an analysis of the long-term water level residual data set from Port Hedland to identify the potential non-cyclonic and seasonal variations in the water level residual. Cardno obtained measured water levels from the Port Hedland Berth 3 tide gauge for the periods shown in **Table A.6.1**. Measured data for each period was shifted to a consistent datum of LAT, corresponding to the measurement datum of the 2005 to 2009 dataset. LAT for this period is defined as 9.523m below PA26 and 4.0m below mean sea level (corresponding to AUS0054). A tidal prediction for the period was generated from the Published Tidal Constituents (AHO, 2009) for Port Hedland and subtracted from the (corrected) measured water levels in order to obtain the residual tide.

Table A.6.1: Measured water level records obtain for the Berth-3 tide gauge located in Port Hedland harbour and offset correction to consistent LAT datum.

Start	End	Offset (m)
03-Mar-1988	31-Dec-1994	-0.139
01-Jan-1996	31-Dec-1996	-0.239
16-Sep-1998	31-Dec-2004	-0.239
01-Jan-2005	31-Dec-2009	0

Table A.6.2 presents an exceedence table of the residual water level by month for the complete dataset from 1988 through to 2009 with known cyclonic periods removed from the record. **Figure A.6.1** provides a graphical representation of this table. A clear seasonal variation is present in the residual water level with greater residual tide evident in the wet season (December through April).

Table A.6.2: Probability of exceedence table of residual water level by month for the period 1988 through 2009.

	Probability of Exceedence								
	1%	5%	10%	20%	50%	80%	90%	95%	99%
Jan	-0.29	-0.22	-0.17	-0.12	-0.02	0.09	0.15	0.2	0.34
Feb	-0.32	-0.22	-0.18	-0.12	-0.01	0.1	0.17	0.23	0.37
Mar	-0.3	-0.21	-0.17	-0.11	0	0.12	0.18	0.23	0.4
Apr	-0.22	-0.16	-0.13	-0.09	0	0.1	0.16	0.2	0.31
May	-0.23	-0.17	-0.13	-0.09	-0.01	0.07	0.11	0.14	0.2
Jun	-0.23	-0.17	-0.13	-0.09	-0.01	0.08	0.12	0.15	0.22
Jul	-0.26	-0.18	-0.15	-0.1	-0.02	0.06	0.1	0.14	0.19
Aug	-0.27	-0.19	-0.15	-0.1	-0.02	0.05	0.1	0.13	0.2
Sep	-0.27	-0.19	-0.15	-0.11	-0.02	0.06	0.11	0.14	0.2
Oct	-0.26	-0.18	-0.14	-0.1	-0.01	0.08	0.12	0.16	0.22
Nov	-0.23	-0.17	-0.14	-0.1	-0.02	0.07	0.12	0.16	0.22
Dec	-0.26	-0.19	-0.15	-0.11	-0.02	0.08	0.14	0.19	0.35

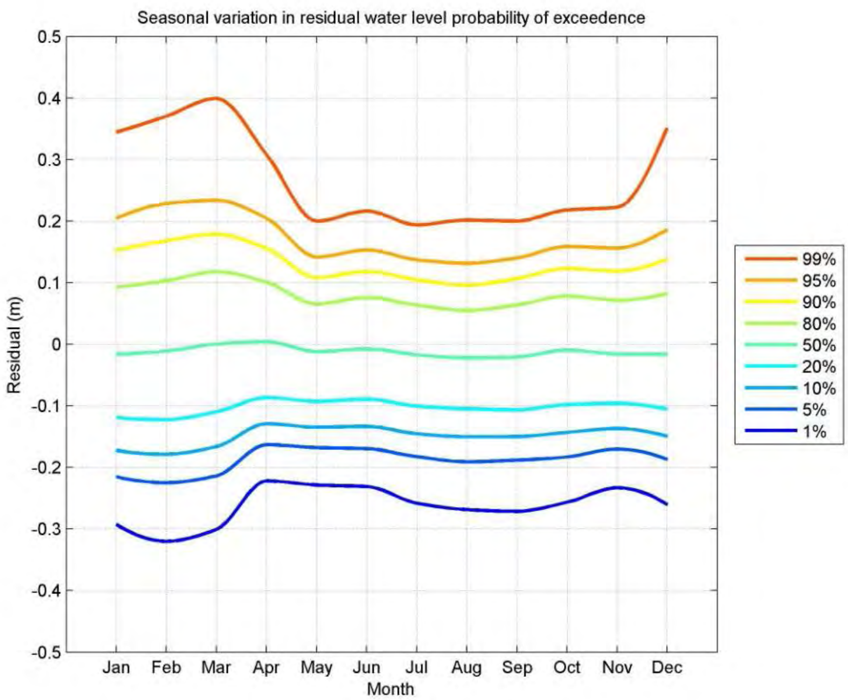


Figure A.6.1: Plot of probability of exceedence of residual water level by month for the period 1988 to 2009

To quantify the mean seasonal water level variation it is necessary to band pass filter the residual water level time series in order to quantify systematic mean sea level variations on seasonal timescales. This analysis must be undertaken on a continuous time series. The longest continuous record of measured water levels is from September 1998 through to December 2009. Water level variation on inter-annual timescales is known to be important on the North West shelf. Eliot (2010) demonstrated the importance of the 4.4 year lunar perigee sub-harmonic on water levels at inter-annual timescales. Ideal Butterworth filters were designed; a fourth order 30-day low pass filter was applied to extract variability in mean sea level and a third-order 2.8-year high pass filter was applied to remove inter-annual variability. **Figure A.6.2** presents a time series plot of the two filtered time series for the period 1998 through 2009. Clear seasonal and inter-annual variability is visible in the residual water level signal. The difference between the 30-day filter and 2.8-year filter is analysed on a monthly basis to calculate the season variation in mean sea level. The beginning and end 1.4 year periods were excluded from the analysis to minimise filter edge effects. **Table A.6.3** presents the mean variation in mean sea level by month for Port Hedland.

Table A.6.3: Average variation in Mean Sea Level by month for the period 1998 through 2009.

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.05m	0.07m	0.11m	0.12m	0.06m	-0.01m	-0.09m	-0.13m	-0.12m	-0.06m	-0.01m	0.02m

The seasonal variation in mean sea level has a magnitude of approximately 0.12m at Port Hedland. Eliot et al (2010) suggests that the signal range of the 4.4-year lunar perigee sub-harmonic is in the order of 0.1m, which is supported by **Figure A.6.2**. For this study an allowance of 0.2m has been added to design levels to account for the combined non-cyclonic residual water level as a result of short-term, seasonal and inter-annual water level variability.

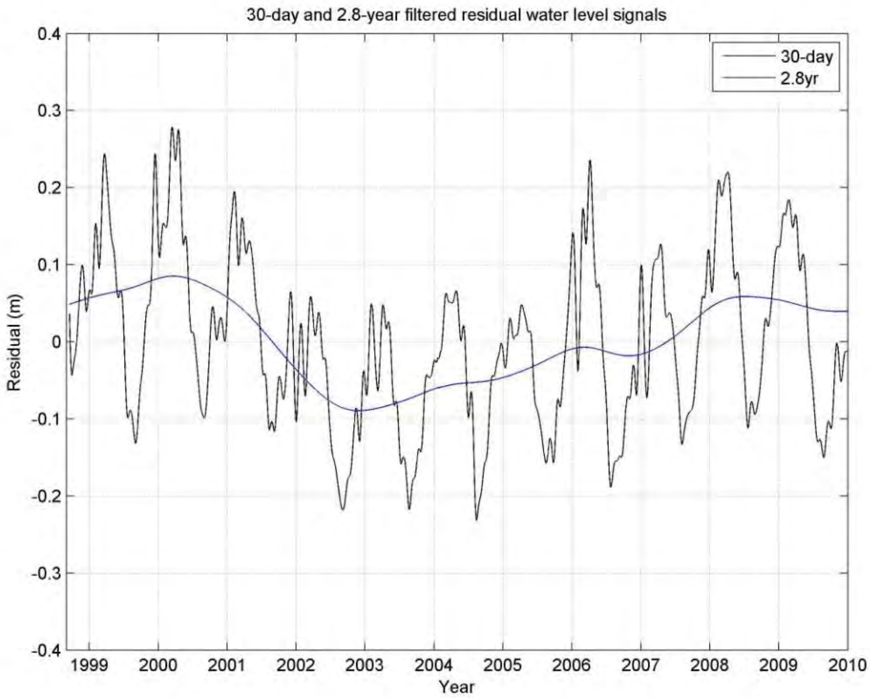


Figure A.6.2: Plot of 30-day filtered residual water level and 2.8-year filtered residual water level.

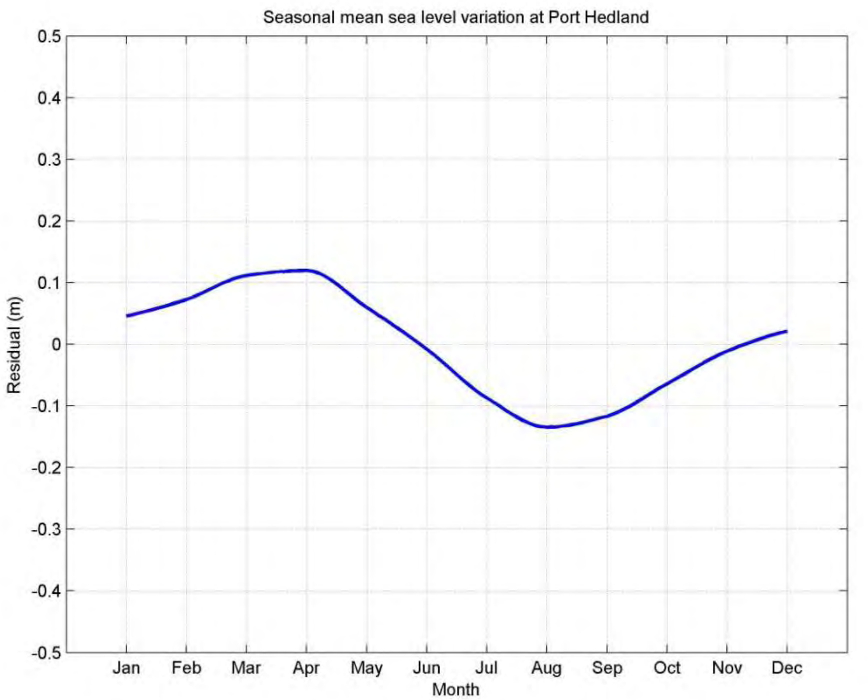


Figure A.6.3: Seasonal variability in mean sea level based on band pass filtering of residual water levels 1999 to 2009.

A.7 CLIMATE CHANGE SCENARIO INVESTIGATIONS

Future Greenhouse enhanced global climate scenarios have the potential to effect the distribution, frequency and intensity of tropical cyclones. Estimates of changes to tropical cyclones under future climate change scenarios are uncertain due to the significant variability in the historical cyclone record and the large natural inter-decadal variations associated with processes such as ENSO and the Indian Ocean Dipole.

The most comprehensive recent assessment of the potential change in cyclone climatology in Australia is presented in CSIRO (2007). Based on a number of global climate models, the consensus is that under future climate change scenarios, by 2070 there may be a significant reduction in the overall cyclone frequency, perhaps up to 40%, across Northwest Australia. Whilst overall cyclone frequency may decrease, there may be an increase in severe Category 3 to 5 cyclone events. All of these estimates are relatively uncertain and are based on global models which have a grid resolution in the same order of magnitude as a cyclone eye diameter and are, therefore, limited in their ability to resolve cyclone processes. A recent paper from a group of leading researchers also indicates the likelihood of reduction in the frequency of less severe cyclones, and a potential increase in the intensity of severe cyclones in the order of 10% (Knutson *et al.*, 2010).

There is no specific climate change scenario that is applicable to apply to Northwest Australian cyclones at this stage. Based on recent cyclone studies undertaken by the CSIRO (2007) and also by Cardno Lawson Treloar (2009a and 2009b) in Queensland, a potential 10% increase in maximum cyclone wind speeds has been applied in a simulation of the 2110 design criteria. The cyclone frequency in 2110 remained unchanged. The 2110 design criteria scenario presented here is likely to provide a conservative design outcome.

A.7.1 Sea Level Rise

Cardno has reviewed current data, estimates and policy regarding sea level rise so that an appropriate Sea Level Rise (SLR) allowance can be incorporated into the Port Hedland coastal vulnerability investigations. The best sea-level rise data set in Australia is collected by the National Tidal Centre, which is now part of the Bureau of Meteorology. The Australian Baseline Sea Level Monitoring Project has been running for the last 20-years and collects data from a number of specialised tide gauges located around Australia. The data monitoring includes measurements of earth movement to isolate relative sea level changes from actual sea level changes. Full details on the Australian Baseline Sea Level Monitoring Project can be found in National Tidal Centre (2009).

A monitoring station is located at Broome, which is 500km east of Port Hedland and located in a similar oceanographic environment. Data has been collected at this site since 1993, which is too short to infer long term changes in sea level, but does provide a good description of sea level change over the last 20-years. Since 1993, the net annual sea level rise at Broome has been 8.1mm, which is very high compared to other locations around Australia and overseas. For reference, the Perth monitoring station has averaged 8.6mm/year since 1993. The drivers for the sea level changes along the Western Australian coast are not well understood, but it is believed that inter-decadal oscillations in sea level are significant.

The *Western Australian Planning Commission (WAPC)* has recently released a position statement regarding sea level rise allowances for Western Australia. WAPC (2010) recommends that for a 2110 year planning period a SLR allowance of +0.9m should be accommodated and for a 2060 year planning period a SLR allowance of +0.3m should be accommodated. These planning levels are broadly consistent with SLR allowances in Queensland, New South Wales and Victoria. These states have Year 2100 SLR allowances of between 0.8m and 1.0m. For this study Cardno has adopted a 0.3m allowance for 2060 and a 0.9m allowance for 2110.

A.7.2 2110 Climate Scenario Results

Tables A.7.1 to **A.7.4** present 100, 200 and 500 year ARI TSWL values at selected sites for a 2110 climate scenario that incorporates a 10% increase in wind speed and +0.9m SLR allowance. For a 2060 climate scenario, a fixed value of 0.3m which represents the SLR allowance for that planning period was adopted for the design levels.

Table A.7.1: Summary of Peak Total Still Water Level (TSWL) for Port Hedland (Berth-3 Tide Gauge) - Selected ARI's for 2110 climate change scenario

ARI (years)	Port Hedland		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
100	5.3	4.7	6.0
200	5.6	4.8	5.3
500	5.7	4.8	6.7

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.7.2: Summary of Peak Total Still Water Level (TSWL) for Site 1 - Selected ARI's for 2110 climate change scenario

ARI (years)	Site 1		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
100	5.7	4.8	6.5
200	6.0	5.0	7.1
500	6.5	5.3	7.8

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.7.3: Summary of Peak Total Still Water Level (TSWL) for Site 2 (Turner River Entrance) - Selected ARI's for 2110 climate change scenario

ARI (years)	Site 2		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
100	5.6	4.8	6.4
200	5.9	4.9	6.8
500	6.4	5.1	7.6

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.7.4: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough- Selected ARI's for 2110 climate change scenario

ARI (years)	Shellborough		
	Peak TSWL (mAHD)*	Lower 95% Conf. Int.	Upper 95% Conf. Int.
100	6.0	5.7	6.3
200	6.7	6.4	7.0
500	6.9	6.5	7.3

* Peak Total Still Water Level based on modelled tide plane from the Delft3D model

A.8 DESIGN WATER LEVEL SUMMARY – OCEAN INUNDATION

A.8.1 Present Climate Condition (2010)

Tables A.8.1 to **A.8.5** present design water levels for selected locations within the study area for return periods between 2 and 500-years ARI. Design levels up to 50-years ARI are based on the hindcast study (**Section A.3**) and for 100-years ARI and greater the results are from the Monte Carlo modelling (**Section A.4**). For the design water levels, an allowance of +0.2m has been added to account for the potential for a non-cyclonic residual water level when a cyclone occurs.

In addition, no measured water level or tidal plane data is available near Shellborough and the Delft3D model does not include all of the astronomical constituents to describe the full tide on the boundary of the model. As a result, an additional water level of +0.5m was added to the design level from the Shellborough site to account for the difference between the likely total tide range and the modelled tide. If water level data (minimum 2-months) were recorded near Shellborough it should be possible to combine the modelled storm surge results with a more accurate predicted (astronomical) tide for Shellborough and define the design water levels for the Shellborough region with greater confidence.

The influence of water setup has been included as an additional item for sites which are exposed to the open coast. The wave setup has been calculated based on results from the SBEACH model presented in **Appendix D (Main Report)**.

In the Port Hedland town region, the design water levels separate into three general regions. The first of these is the inner harbour area and for this study, design water levels have been provided relative to the Berth-3 permanent tide gauge. Overall spatial variations in modelled water inside the inner harbour are generally small, about 0.1m or less as the potential for increased wind setup inside the estuary is offset by frictional losses with distance upstream of the entrance. Near the Port Hedland harbour entrance and on the western side of the Spoil Bank, there is a need to consider the potential for wave setup to increase the total water level at the shoreline for sites exposed to open coast swell waves during cyclones. The wave setup allowance is based on selected SBEACH simulations for a number of open coast profiles in the Port Hedland area – see **Appendix D (Main Report)**. The presence of the Spoil Bank acts as a significant hydraulic control during design cyclone events with higher storm surge, and consequently total water levels present on the eastern side of the Spoil Bank around to the Pretty Pool area as this area has significantly higher wind setup levels compared to the entrance of Port Hedland. Again, open coast locations also need to consider the impact of wave setup on design water levels. Further investigation of the influence of the Spoil Bank is presented in **Section A.9**.

Sites 1 and 2 which are to the west of Port Hedland have very similar design water levels based on results from the hindcast and Monte Carlo modelling investigations. A single set of design water level criteria have been presented for these two sites. Open coast locations, for example Site 2 also need to consider the impact of wave setup on design water levels.

The Shellborough region has the highest design water levels in the study region. The combined effects of a larger tidal range, together with the potential for greater storm surge along this section of coast result in significantly larger design levels at this site compared to the Port Hedland region.

Figure A.8.1 presents a plan view of the regions which are applicable for the different design conditions around Port Hedland.

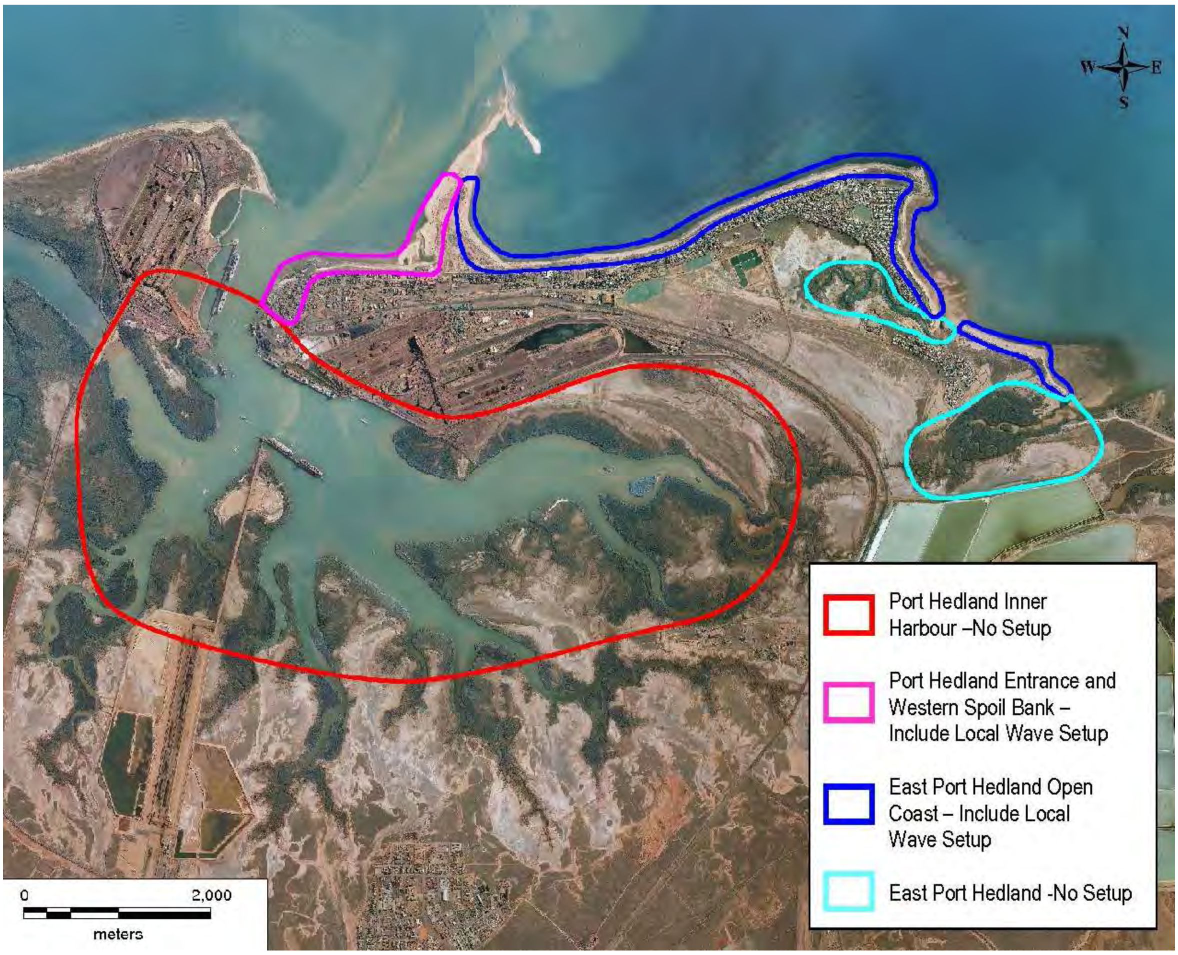


Figure A.8.1: Plan view of the general Port Hedland regions where particular design levels are applicable.

Table A.8.1: Summary of Design Peak Total Still Water Level (TSWL) for Port Hedland Inner Harbour (Berth-3 Tide Gauge) - Selected ARI's for present climate scenario

ARI (years)	Port Hedland Inner Harbour (Berth-3)
	Peak TSWL (mAHD)*
2	3.3
10	3.8
20	3.9
50	4.2
100	4.7
200	4.9

500	5.1

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.8.2: Summary of Design Peak Total Still Water Level (TSWL) for West of the Spoil Bank and Harbour Entrance - Selected ARI's for present climate scenario.

ARI (years)	Port Hedland Entrance and Western Spoil Bank		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.3	0	3.3
10	3.8	0.8	4.6
20	3.9	0.8	4.7
50	4.2	0.8	5.0
100	4.7	0.9	5.6
200	4.9	1.0	5.9
500	5.1	1.2	6.3

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.8.3: Summary of Design Peak Total Still Water Level (TSWL) for Eastern Spoil Bank to Pretty Pool - Selected ARI's for present climate scenario.

ARI (years)	East Port Hedland - Eastern Spoil Bank to Pretty Pool		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.5	0	3.5
10	4.0	0.8	4.8
20	4.1	0.8	4.9
50	4.4	0.8	5.2
100	5.0	0.9	5.9
200	5.1	1.0	6.1
500	5.6	1.2	6.8

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.8.4: Summary of Design Peak Total Still Water Level (TSWL) for Sites 1 and 2 - Selected ARI's for present climate scenario.

ARI (years)	Sites 1 and 2		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.3	0	3.3
10	3.8	0.8	4.6
20	4.0	0.8	4.8
50	4.3	0.8	5.1
100	5.0	0.9	5.9
200	5.2	1.0	6.2
500	5.5	1.2	6.7

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009) and includes allowance for non-cyclonic variation in the mean water level.

Table A.8.5: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough - Selected ARI's for present climate scenario.

ARI (years)	Shellborough		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.9	0	3.9
10	4.7	0.7	5.4
20	5.1	0.7	5.8
50	5.3	0.8	6.1
100	5.9	0.9	6.8
200	6.4	1.0	7.4
500	6.6	1.0	7.6

Peak Total Still Water Level based on modelled tide and includes allowance for non-cyclonic variation in the mean water level (+0.2m) and also an allowance for the actual tide range (+0.5m) at the site which is not fully simulated in the Delft3D model.

A.8.2 2060 Design Water Levels

Tables A.8.6 to **A.8.10** present design water levels for selected locations within the study area for return periods between 2 and 500-years ARI based on a 2060 climate scenario which has an increase in the mean sea level of +0.3m. The 2060 design water levels have been calculated by adding +0.3m to the design levels for the present climate scenario presented in **Section A.8.1**. For the design water levels, an allowance of +0.2m has been added to account for the potential for a non-cyclonic residual water level when a cyclone occurs.

Table A.8.6: Summary of Design Peak Total Still Water Level (TSWL) for Port Hedland Inner Harbour (Berth-3 Tide Gauge) - Selected ARI's for 2060 climate scenario

ARI (years)	Port Hedland Inner Harbour (Berth-3)
	Peak TSWL (mAHD)*
2	3.6
10	4.1
20	4.2
50	4.5
100	5.0
200	5.2
500	5.4

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.8.7: Summary of Design Peak Total Still Water Level (TSWL) for West of the Spoil Bank and Harbour Entrance - Selected ARI's for 2060 climate scenario.

ARI (years)	Port Hedland Entrance and Western Spoil Bank		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.6	0	3.6
10	4.1	0.8	4.9
20	4.2	0.8	5.0
50	4.5	0.8	5.3
100	5.0	0.9	5.9
200	5.2	1.0	6.2
500	5.4	1.2	6.7

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009).

Table A.8.8: Summary of Design Peak Total Still Water Level (TSWL) for Eastern Spoil Bank to Pretty Pool - Selected ARI's for 2060 climate scenario.

ARI (years)	East Port Hedland - Eastern Spoil Bank to Pretty Pool		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.8	0	3.8
10	4.3	0.8	5.1
20	4.4	0.8	5.2
50	4.7	0.8	5.5
100	5.3	0.9	6.2
200	5.4	1.0	6.4
500	5.9	1.2	7.1

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.8.9: Summary of Design Peak Total Still Water Level (TSWL) for Sites 1 and 2 - Selected ARI's for 2060 climate scenario.

ARI (years)	Sites 1 and 2		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	3.6	0	3.6
10	4.1	0.8	4.9
20	4.3	0.8	5.1
50	4.6	0.8	5.4
100	5.3	0.9	6.2
200	5.5	1.0	6.5
500	5.8	1.2	7.0

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009) and includes allowance for non-cyclonic variation in the mean water level.

Table A.8.10: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough - Selected ARI's for 2060 climate scenario.

ARI (years)	Shellborough		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.2	0	4.2
10	5.0	0.7	5.7
20	5.4	0.7	6.1
50	5.6	0.8	6.4
100	6.2	0.9	7.1
200	6.7	1.0	7.7
500	6.9	1.0	7.9

Peak Total Still Water Level based on modelled tide and includes allowance for non-cyclonic variation in the mean water level (+0.2m) and also an allowance for the actual tide range (+0.5m) at the site which is not fully simulated in the Delft3D model.

A.8.3 2110 Design Water Levels

Tables A.8.11 to **A.8.15** present design water levels for selected locations within the study area for return periods between 2 and 500-years ARI based on a 2110 climate scenario which has an increase in the mean sea level of +0.9m and an increase in cyclone intensity. The 2110 design water levels have been calculated by adding +0.9m to the design levels for the present climate scenario presented in **Section A.8.1** for ARI's less than 100-years. For ARI's greater than 100-years, the design levels are based on results from simulations of the Monte Carlo model (see **Section A.5**) for a climate change scenario with a +0.9m increase in mean sea level and a 10% increase in cyclone intensity. The potential increase in maximum cyclone wind speeds of 10% has a relatively small influence on the design water level compared to the change in mean sea level. At the 500-year ARI level, this increase has only result in an additional +0.1m increase to the design water levels. For the design water levels, an allowance of +0.2m has been added to account for the potential for a non-cyclonic residual water level when a cyclone occurs.

Table A.8.11: Summary of Design Peak Total Still Water Level (TSWL) for Port Hedland Inner Harbour (Berth-3 Tide Gauge) - Selected ARI's for 2110 climate scenario

ARI (years)	Port Hedland Inner Harbour (Berth-3)
	Peak TSWL (mAHD)*
2	4.2
10	4.7
20	4.8
50	5.1
100	5.6
200	5.8
500	6.1

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.8.12: Summary of Design Peak Total Still Water Level (TSWL) for West of the Spoil Bank and Harbour Entrance - Selected ARI's for 2110 climate scenario.

ARI (years)	Port Hedland Entrance and Western Spoil Bank		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.2	0	4.2
10	4.7	0.8	5.5
20	4.8	0.8	5.6
50	5.1	0.8	5.9
100	5.6	0.9	6.5
200	5.8	1.0	6.9
500	6.1	1.2	7.4

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.8.13: Summary of Design Peak Total Still Water Level (TSWL) for Eastern Spoil Bank to Pretty Pool - Selected ARI's for 2110 climate scenario.

ARI (years)	East Port Hedland - Eastern Spoil Bank to Pretty Pool		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.4	0	4.4
10	4.9	0.8	5.7
20	5.0	0.8	5.8
50	5.3	0.84	6.1
100	5.9	0.9	6.8
200	6.0	1.0	7.0
500	6.6	1.2	7.8

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009)

Table A.8.14: Summary of Design Peak Total Still Water Level (TSWL) for Sites 1 and 2 - Selected ARI's for 2110 climate scenario.

ARI (years)	Sites 1 and 2		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.2	0	4.2
10	4.7	0.8	5.5
20	4.9	0.8	5.7
50	5.2	0.8	6.0
100	5.9	0.9	6.8
200	6.1	1.0	7.1
500	6.5	1.2	7.7

* Peak Total Still Water Level based on Published Tidal Constituents (AHO, 2009) and includes allowance for non-cyclonic variation in the mean water level.

Table A.8.15: Summary of Design Peak Total Still Water Level (TSWL) for Shellborough - Selected ARI's for 2110 climate scenario.

ARI (years)	Shellborough		
	Peak TSWL (mAHD)*	Wave Setup – Open Coast (m)	Total Design Water Level for Open Coast (mAHD)
2	4.8	0	4.8
10	5.6	0.7	6.3
20	6.0	0.7	6.7
50	6.2	0.8	7.0
100	6.8	0.9	7.7
200	7.3	1.0	8.3
500	7.6	1.0	8.6

Peak Total Still Water Level based on modelled tide and includes allowance for non-cyclonic variation in the mean water level (+0.2m) and also an allowance for the actual tide range (+0.5m) at the site which is not fully simulated in the Delft3D model.

A.9 INFLUENCE OF THE SPOIL BANK ON OCEANIC INUNDATION LEVELS

It is possible that through time the Spoil Bank will progressively erode back to the natural shoreline over a 100 year planning period. To investigate the influence of the Spoil Bank on the modelled oceanic water levels for the coastal regions of the Port Hedland Township a model bathymetry was developed with the spoil bank removed and a bathymetry established offshore. A model run using the revised bathymetry with the 2110 climate change scenario, incorporating an increase in mean sea level of +0.9m and a 10% increase in cyclone wind speed, was conducted. A hydrographic survey predating the Spoil Bank (pre 1960) was obtained from the Department of Transport. A survey chart of Port Hedland from 1896 was geo-referenced to an aerial image of Port Hedland taken at low tide in 2001 as demonstrated in **Figures A.9.1** through **A.9.3** below.

Figure A.9.1: Port Hedland Aerial 2001

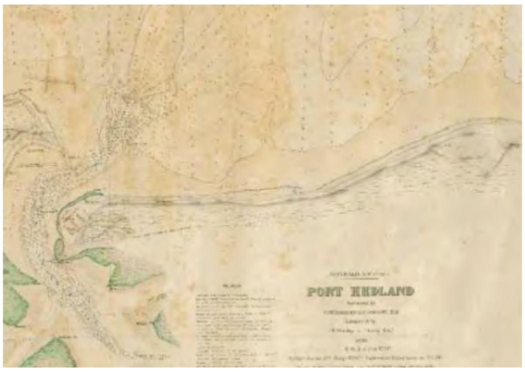


Figure A.9.2: Port Hedland Chart 1896 (Source DoT)

Figure A.9.3: Geo-referenced 1986 hydrographic chart overlaid with an aerial photo of Port Hedland from 2001.

Chart Datum is Low Water Springs (~ -2.7m AHD). The low water line (-2.7m AHD) is shown on the chart (**Figure A.9.2**) with good agreement to the aerial from Airey Point to Cooke Point except where the Spoil Bank is present.

The low water line and the survey points taken across the current region of the spoil bank were used as a basis for the "no spoil bank" scenario as shown in **Figure A.9.4**. A comparison of the resulting "no spoil bank" bathymetry and the present day bathymetry in the vicinity of the Spoil Bank is shown in **Figure A.9.5.**

Figure A.9.4: Chart Region showing where Coastline was redefined in red

A.9.1 Investigation of 100 Year ARI Event

The representative 100 year ARI design cyclone was simulated and the influence of the spoil bank on the wave conditions and water levels at the entrance to the harbour examined. The peak modelled wave and water level conditions at a location west of the northern end of the Spoil Bank in the shipping channel are summarised in **Table A.9.1.** At the peak of the event the absence of the Spoil Bank results in an increase in the water level of approximately 0.22m. **Figure A.9.6** presents a time series comparison of the water levels and wave conditions for a representative 100 year ARI cyclone. Due to the cyclonic wind pattern, cyclone events that result in high surge levels track to the west of Port Hedland where peak winds are directed onshore. This establishes a surge wave with a peak to the east of Port Hedland and that propagates to the west. The presence of the Spoil Bank acts as a hydraulic control to the propagation of the surge wave, causing a reduction in peak total water level and a lag in the phasing as the surge wave must refract around the Spoil Bank, losing energy in the process.

Table A.9.1: Comparison between ARI100 cases at peak wave condition in the shipping channel approximately 2km from the entrance of the Port Hedland Harbour.

Case	Peak Water Level (mAHD)	Peak Hs (m)	Tp (s)	Direction (deg TN)
With Spoil Bank	4.83	3.4	4.95	355
No Spoil Bank	5.05	3.5	6.68	9.8

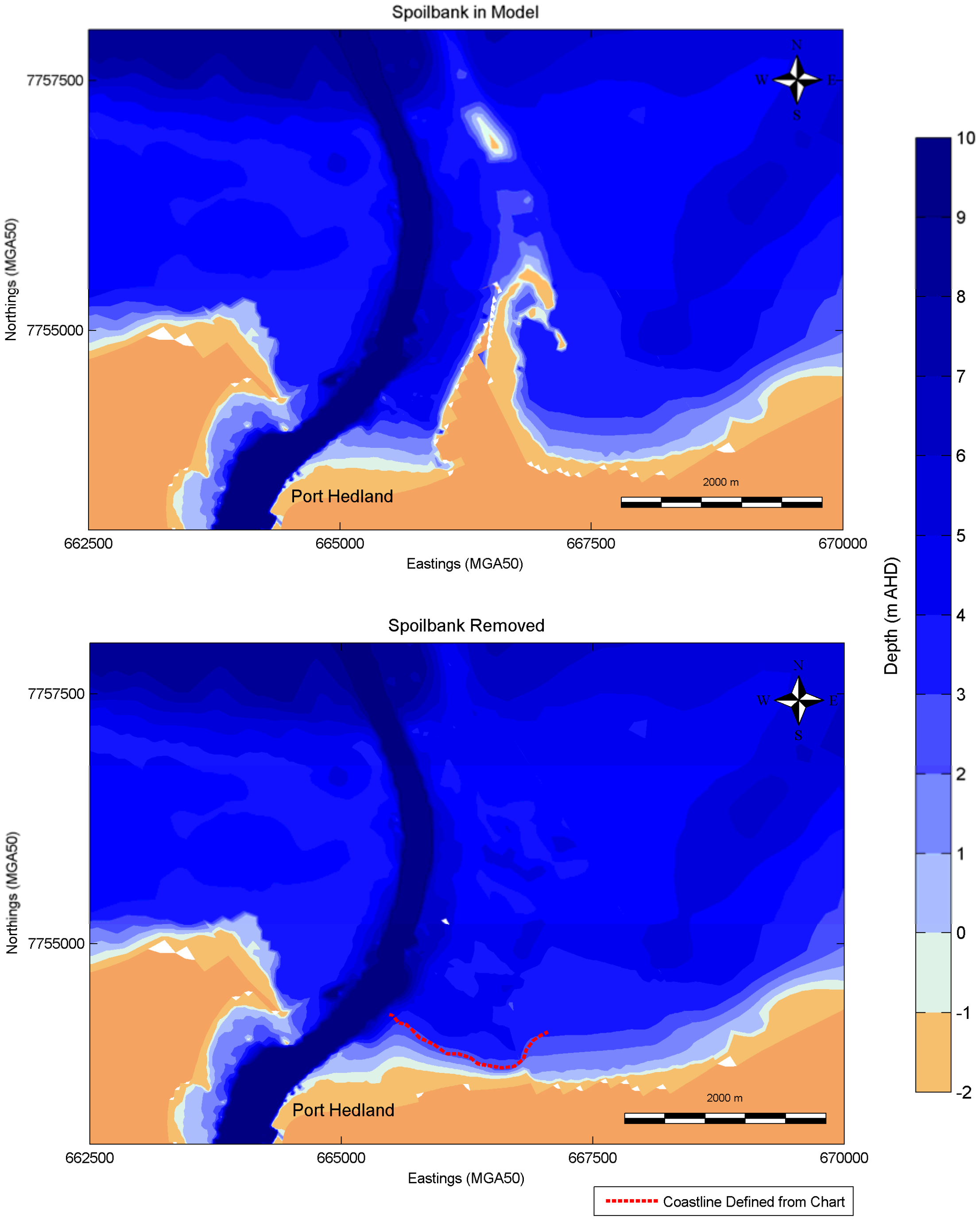


Figure A.9.5: Bathymetry Changes in the Delft3D Hydrodynamic Model Setup - With and Without Spoil Bank

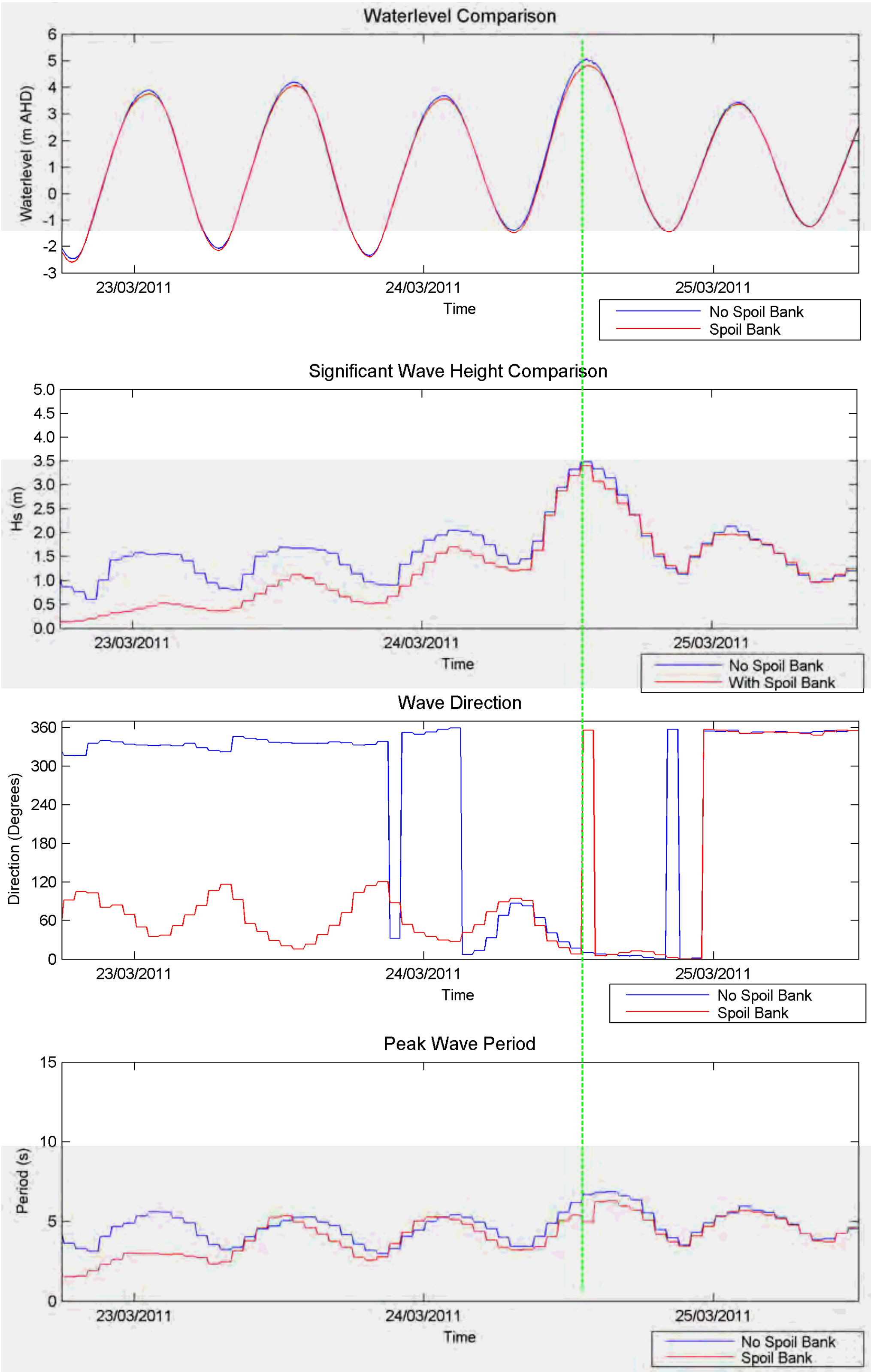


Figure A.9.6: Time series comparisons for a representative 100 Year ARI Cyclone with and without the Spoil Bank. Peak wave condition indicated by the green line.

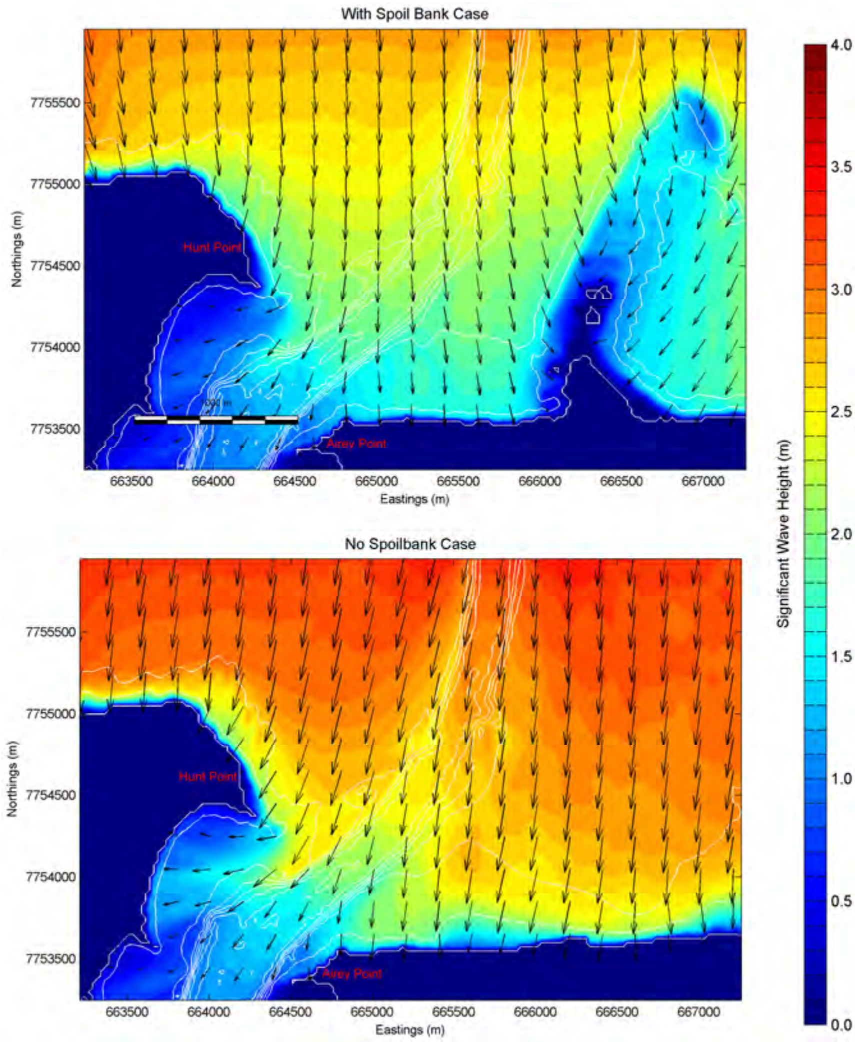


Figure A.9.7: Wave Map Comparisons for the 100 Year ARI Cyclone with and without Spoil Bank at the entrance to Port Hedland Inner Harbour

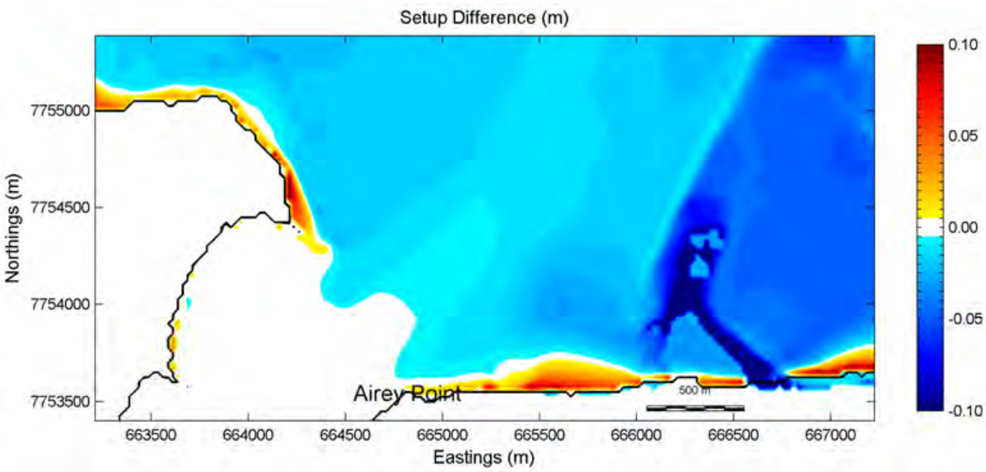


Figure A.9.8: ARI100 Wave Setup difference at the Port Hedland Shoreline as a result of removing the spoil bank

The Spoil Bank plays a major role in transforming the wave field along Port Hedland town shoreline. **Table A.9.1** demonstrates that there is a shift in the wave direction of about 15 degrees towards the Spoil Bank which is further demonstrated by **Figure A.9.6.** The peak wave period is slightly lower and the wave direction is more northerly with the spoil bank included in the model.

The results presented in **Table A.9.1** were applied as a boundary condition to a local SWAN model at 25m resolution covering the Port Hedland harbour entrance and Spoil Bank area. **Figure A.9.7** presents a spatial plot of wave height and direction across the model domain. The presence of the Spoil Bank establishes a strong refractive process towards the Spoil Bank resulting in a reduction in significant wave height between Airey Point and the Spoil Bank of approximately 0.6m. Each case was executed with the wave setup process activated to quantify the increase in water levels due to wave setup with the removal of the Spoil Bank. **Figure A.9.8** presents the spatial difference in wave setup resulting from the absence of the Spoil Bank. Wave setup is increased in the order of 0.05 m to 0.1 m along the shoreline of Port Hedland town with the absence of the Spoil Bank.

A.9.2 Investigation of 500 Year ARI Event

The representative 500 year ARI design cyclone was simulated and the influence of the spoil bank on the wave conditions and water levels at the entrance to the harbour examined. The peak modelled wave and water level conditions at a location west of the northern end of the Spoil Bank in the shipping channel (**Figure A.9.5**) are summarised in **Table A.9.2.** At the peak of the event the absence of the Spoil Bank results in an increase in the water level of approximately 0.22m. **Figure A.9.9** presents a time series comparison of the water levels and wave conditions for a representative 500 year ARI cyclone. Similar to the 100 year ARI event the presence of the Spoil Bank acts as a hydraulic control to the propagation of the surge wave, causing a reduction in peak total water level and a lag in the phasing as the surge wave must refract around the Spoil Bank, loosing energy in the process.

Table A.9.1: Comparison between ARI500 cases at peak wave condition in the shipping channel approximately 2km from the entrance of the Port Hedland Harbour.

Case	Peak Water Level (mAHD)	Peak Hs (m)	Tp (s)	Direction (deg TN)
With Spoil Bank	5.26	4.04	5.58	359
No Spoil Bank	5.44	4.07	6.09	21

The results presented in **Table A.9.2** were applied as a boundary condition to a local SWAN model at 25m resolution covering the Port Hedland harbour entrance and Spoil Bank area for bath bathymetries – with and without the spoil bank. **Figure A.9.10** presents spatial plots of wave height and direction across the model domain. Similar to the ARI100 case the presence of the Spoil Bank establishes a strong refractive process towards the Spoil Bank resulting in a reduction in significant wave height between Airey Point and the Spoil Bank of approximately 0.6m. Each case was executed with the wave setup process activated to quantify the increase in water levels due to wave setup with the removal of the Spoil Bank. **Figure A.9.11** presents the spatial difference in wave setup resulting from the absence of the Spoil Bank. Wave setup is increased in the order of 0.05 m to 0.1 m along the shoreline of Port Hedland town with the absence of the Spoil Bank.

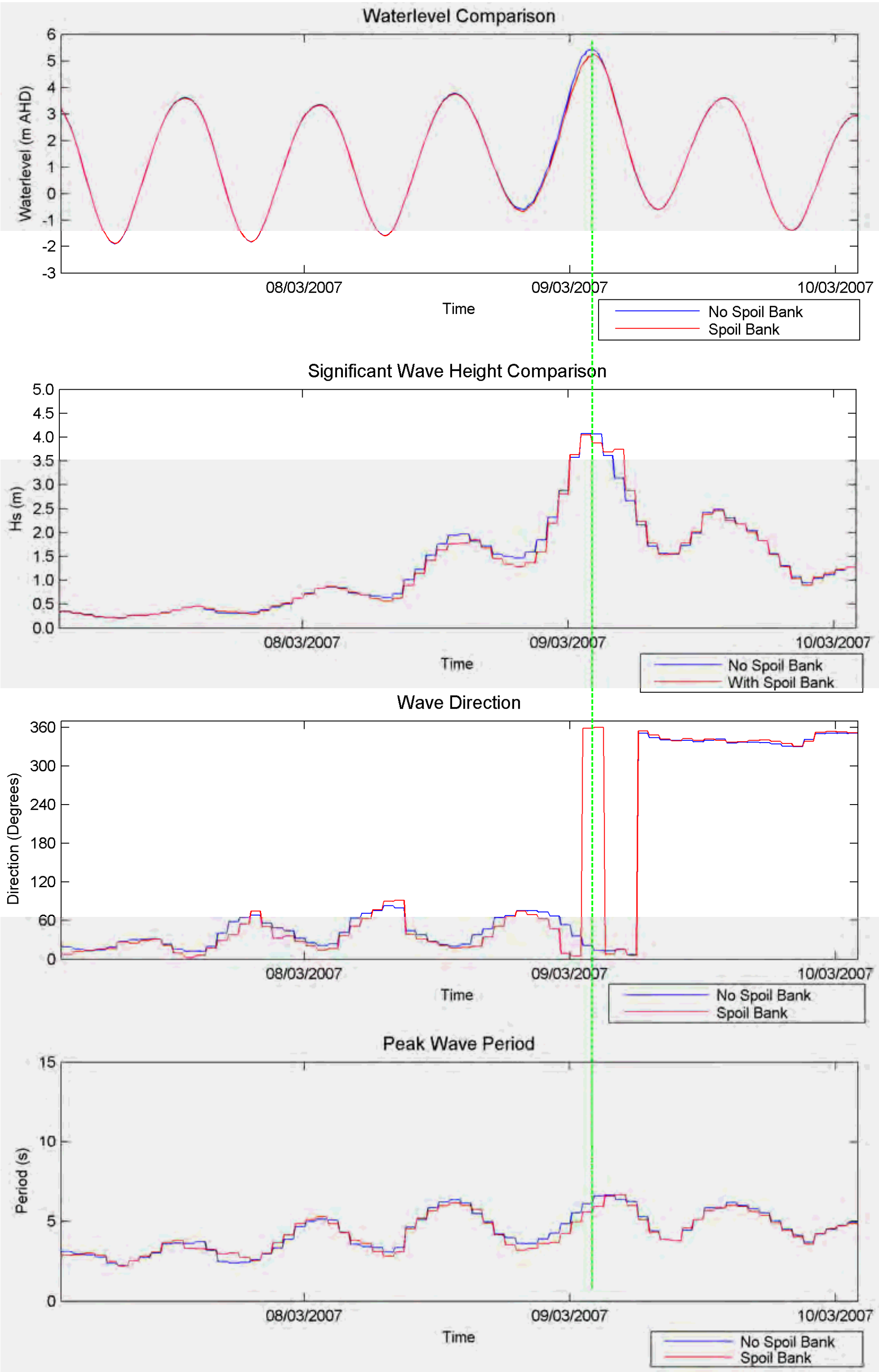


Figure A.9.9: Time Series Comparisons for the 500 Year ARI Cyclone with and without Spoil Bank. Peak wave condition indicated by the green line

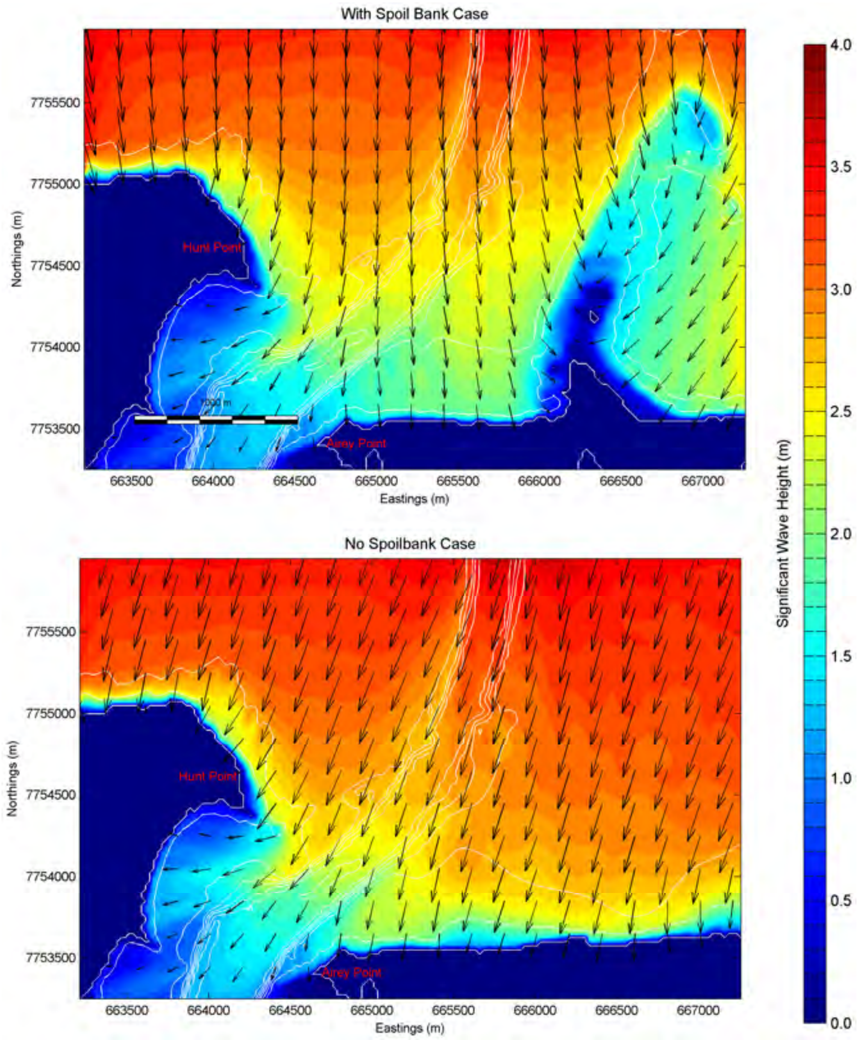


Figure A.9.10: Wave Map Comparisons for the 500 Year ARI Cyclone with and without Spoil Bank at the entrance to Port Hedland Inner Harbour

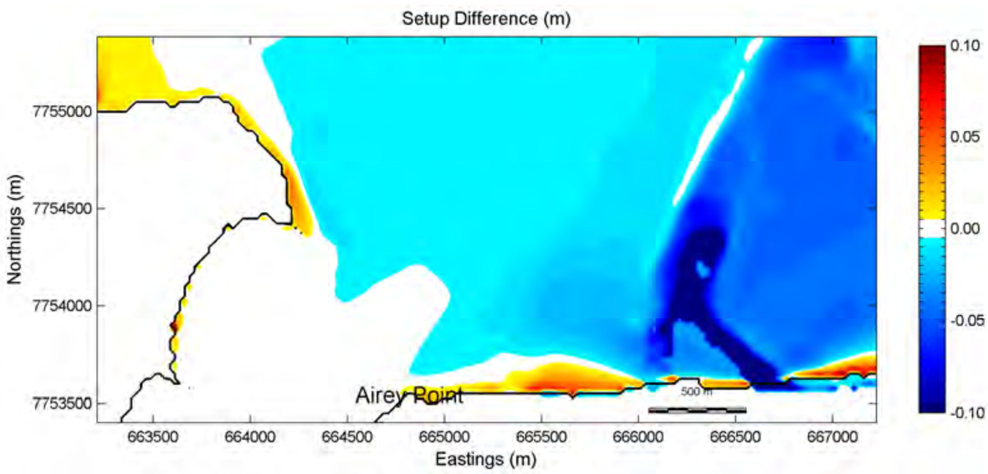


Figure A.9.11: ARI500 Wave Setup difference at the Port Hedland Shoreline as a result of removing the Spoil Bank.

A.9.3 Discussion

The removal of the Port Hedland spoil bank was modelled for a case representative of the 100 year ARI and 500 year ARI cyclone under climate change conditions to investigate the wave and water level conditions at the entrance to Port Hedland Harbour and along the Port Hedland township shoreline.

The removal of the Spoil Bank results in a significant change in the wave propagation in the vicinity of the Port Hedland Township and harbour entrance. The Spoil Bank induces a strong refractive process towards the Spoil Bank, diverting significant amounts of wave energy away from the harbour entrance and township shoreline to be dissipated on the Spoil Bank. The removal of the Spoil Bank results in an increase in wave height of approximately 0.6m and an increase in wave setup of approximately 0.1m at the 100 year and 500 year ARI.

In addition the Spoil Bank acts as a significant hydraulic control to the surge wave with a peak to the east of Port Hedland for large surge events under cyclonic conditions. The refraction of the surge wave around the Spoil Bank dissipates energy and results in a decrease in water level to the west of the Spoil Bank of approximately 0.2m when compared to the same location without the spoil bank.

Overall the removal of the Spoil Bank increases the potential total water level in the order of 0.3m for both of the selected representative 100 year and 500 year ARI cyclone events.

Over a 100 year planning period, the fate of the Spoil Bank is uncertain. In the event of the Spoil Bank being lost from the system, there is a likelihood of higher design water levels to the west of the current Spoil Bank, including the Port Hedland harbour. In the determination of habitable floor levels for development in the Port Hedland area, the uncertainty around the future of the Spoil Bank should be considered. The uncertainty and risk associated with the loss of the Spoil Bank can be accommodated through the use of an appropriate free board allowance. On the basis of this investigation the impact to water levels resulting from the loss of the Spoil Bank is within the +/-0.5m 95% confidence interval presented for the 100 year ARI ocean water level.

A.10 CONCLUSIONS

As part of the Port Hedland Coastal Vulnerability Study, a comprehensive assessment of design ocean inundation water levels has been undertaken based on analyses of measured data and application of a numerical model system. The numerical model system incorporates wind, air pressure, surface wave and astronomical tide forcing to simulate coastal water levels in the Port Hedland region.

All components of the model system have been calibrated to available measured data. The validation of the model system is generally good, in particular for the three most severe cyclones (TC Connie, John and George) in the six event calibration data set. The model system has been applied to two separate investigations to calculate design water levels at the study sites. The first of these investigations is a hindcast study has involved the simulation of the 32 most severe cyclones to impact the Port Hedland region since 1960. Outcomes from this investigation have been used to define design water levels up to 50-years ARI.

The second investigation approach has applied a validated Monte Carlo cyclone track model system which has been applied to simulate coastal water levels based on a synthetic cyclone track record of 10,000 years. This study approach is more appropriate for determining long return period water levels and has been used in this study to define design water levels for 100-years ARI and greater return periods. At the 100-years ARI design level, the Monte Carlo model produced higher design water levels compared to the hindcast study. Both the hindcast and Monte Carlo investigations adopted Extreme Value Analysis techniques to determine design water levels.

The Pilbara coastline is subject to non-cyclonic processes that influence coastal water levels on a daily, monthly and inter-annual basis. The effects of non-cyclonic water level residuals have been included in the design water level outcomes of this study. A non-cyclonic water level residual of +0.2m has been included in all design water levels and based on a measured data record during cyclonic months this level is only exceeded less than 5% of the time.

The hydrodynamic modelling system is able to reproduce coastal water levels and currents reasonably when compared to measured data, however due to the limited number of astronomical constants available along the boundary of the model, for large tide range periods which approach HAT, the numerical model underestimates the tide range. The design water levels for the Port Hedland region have been adjusted to account of the full tide range at the site based on the published tidal constants for the site (AHO, 2009). For Sites 1 and 2 which are to the west of Port Hedland, the full Port Hedland tidal plane has been used to define the design water levels.

The Shellborough site lies more than 90km east of Port Hedland and has a larger tidal range than Port Hedland. No measured tidal information is available for the Shellborough site so as a result, an additional allowance of +0.5m has been added to the Shellborough design water levels to account for the uncertainty in the modelled tidal conditions at this site. If more refined design water levels are required for the Shellborough region, it is recommended that a data collection exercise be undertaken to obtain water level data from the site which can be processed to calculate tidal constituents at the site. A minimum of 6-weeks of water level data should be collected to obtain sufficient tidal constituents to estimate the full tidal range at the site. In addition to the absence of any measured tidal planes at Shellborough, very limited bathymetric data is available for this section of the coastline and during the calibration process significant sensitivity was observed in the Port Hedland area with changes to bathymetry in the Shellborough region. If any major development with permanent settlement is planned for the Shellborough region, a data collection exercise to obtain bathymetric data for up to 20km offshore should also be undertaken as the modelled design water levels in the Shellborough region and to a lesser extent Port Hedland also, are influenced by the regional bathymetry which effects astronomical tide, wind set-up and wave propagation processes.

The outcomes from the Ocean Inundation Assessment have been incorporated into the hydraulic modelling presented in **Appendix C** which includes ocean inundation extents for selected ARI's. **Appendix C** includes ocean inundation flood mapping for a range of design conditions.

The summary report for *Port Hedland Coastal Vulnerability* Study includes discussion of appropriate design criteria for current and future development in the Port Hedland region. Presently, ocean inundation design criteria is addressed in SPP 2.6 which specifies that in cyclone prone areas, development should be set back sufficiently to avoid flooding from a "Category 5 cyclone tracking to maximise its associated storm surge". This design condition does not specifically relate to a defined probabilistic Average Recurrence Interval (ARI). It is understood that as part of the review of SPP 2.6, the specification of specific ARI's for cyclone inundation planning purposes is being considered.

Within Australia, for normal residential development it is common that in planning for ocean and/or catchment inundation, a 100-year ARI design condition is the normal standard. For new developments, a 100-year planning period is often considered so that the effects of climate change and sea level rise over a 100-year planning period are incorporated into the planning level. Higher risk infrastructure such as hospitals, evacuation routes and evacuation centres, are commonly designed for much longer return period conditions. For example in Queensland, evacuation centres have to be designed with floor levels above the 10,000-year ARI storm tide level.

The addition of a planning or design level freeboard is also commonly adopted in Australia. The freeboard allowance is designed to cover uncertainty related to the design criteria and is often specified to the design criteria for finished floor levels. For example, a new development may have roads and lot levels designed to a 100-year ARI inundation level, but the finished floor levels will be specified as some height above the 100-year inundation level to cover uncertainty and to further reduce the risk of inundation of the dwelling. The freeboard allowance varies considerably depending on the uncertainty related to the calculation of design levels, and also due to the potential risk of an actual event being significantly more severe than the specified design ARI condition. In the context of this study, for the Port Hedland region the 95% confidence interval for the 100-year ocean water level is approximately +/- 0.5m.

A.11 REFERENCES

Australian Hydrographic Service (2009): *Australian National Tide Tables 2009*.

Booij N., Haagsma Ij.G., Holthuijsen L.H., Kieftenburg A.T., Ris R.C., and Zijllema M. (2004). *Simulating Waves Nearshore (SWAN) Cycle III version 40.41– User Manual*. Delft University of Technology. December 24, 2004

Bowden (1983): "Physical Oceanography of Coastal Waters". Ellis Horwood. West Sussex, England.

Cardno (2010): "Literature Review – Joint Probability". Prepared for Gold Coast City Council, May 2010. LJ8915.

Cardno Lawson Treloar (2007) "Wave Climate Study and Assessment of Vessel Cyclone Moorings and Ship Interaction Study within the Port of Port Hedland" Report R2336 prepared for Port Hedland Port Authority.

Cardno Lawson Treloar (2008a) "Port Hedland Harbour – Harriet Point and Nelson Point BHPB Berths wave and Current Investigations" Report R2460 prepared for Sinclair Knight Merz.

Cardno Lawson Treloar (2008b) "Port Hedland - Effect of Armoured Revetment Construction – Numerical Hydrodynamic Modelling Investigations" Letter L1422 prepared for Sinclair Knight Merz.

Cardno Lawson Treloar (2009a): "Storm Tide Hazard Study, Moreton Bay Regional Council." Prepared for Moreton Bay Regional Council. Report LJ8824/R2461v4.

Cardno Lawson Treloar (2009b): "Storm Tide Hazard Study, Redland Shire and Logan City Councils." Report LJ8824/R2504v2.

CSIRO (2007). Climate Change in Australia. Published by CSIRO 2007. ISBN 9781921232947.

CSIRO (2010). "Sea Level Rise – Projections for the Australian Region". Prepared by the CSIRO for the Wealth from Ocean National Research Flagship. Available at: Projections - http://www.cmar.csiro.au/sealevel/sl_proj_regional.html.

CSIRO (2007): The Impact of Climate Change on Extreme rainfall and Coastal Sea Levels over South-East Queensland – Phase 3: Storm Surge Modelling for Climate Change. Report Prepared for Gold Coast City Council.

Department of Natural Resources and Mines (2001): Queensland Climate Change and Community Vulnerability to Tropical Cyclones: Ocean Hazards Assessment Stage 1 Review of Technical Requirements, March 2001. ISBN: 0 7345 1788 2.

DECCW (2010). "Flood Risk Management Guide - Incorporating sea level rise benchmarks in flood risk assessments". Published by Department of Environment, Climate Change and Water. August 2010.
ISBN 978 1 74232 921 5

Eliot, M (2010) Influence of interannual tidal modulation on coastal flooding along the Western Australian coast. Journal of Geophysical Research, 115, C11013.

Eliot, M., Haigh, I., Pattiaratchi, C. (2010) Tidal and Mean Sea Level Influences on WA Coast Flooding. WAMSI Node 6 symposium – November 2010. Available at: http://www.wamsi.org.au/2010/wamsi-node-6-symposium-ocean-science-offshore-and-coastal-engineering

Global Environmental Modelling Systems (2000) Greater Port Hedland Storm-surge Study. Final Report to WA Ministry for Planning and Port Hedland Town Council October 2000.

GHD (2010). "Town of Port Hedland – Report for South Hedland Flood Study - Draft". Prepared for Town of Port Hedland, September 2010.

Goda (2000): Random Seas and Design of Maritime Structures. *Advanced Series on Ocean Engineering – Volume 15*. World Scientific, Singapore. ISBN-13 978-981-02-3256-6.

Hardy T., Mason L., Astorquia A, and Harper T. (2004): Tropical Cyclone-Induced Water Levels and Waves: Hervey Bay and Sunshine Coast. Report Prepared for the Queensland Government, August 2004.

Harper, B.A., Lovell, K.F., Chandler, B.D. and Todd D.J. (1989): The Derivation of Environmental Design Criteria for Goodwyn 'A' Platform. Proceedings of the 9th Australian Conference on Coastal and Ocean Engineering, Institution of Engineers, Australia, Adelaide, December 1989.

Harper B.A., Mason L.B. and Bode L., (1993): Tropical Cyclone Orson – A Severe Test for Modelling. Proceedings of the 11th Australian Conference on Coastal and Ocean Engineering, Institution of Engineers, Australia. Townsville, August 1993.

Holland, G J (1980): An Analytical Model of the Wind and Pressure Profiles in Hurricanes. Monthly Weather Review, 108, pp 1212-1218.

Holthuijsen L. H. (2007) Waves in Oceanic and Coastal Waters. Cambridge University Press.

Hsu, S. A., Eric A. Meindl, and David B. Gilhousen, 1994: Determining the Power-Law Wind –Profile Exponent under Near-Neutral Stability Conditions at Sea, Applied Meteorology, Vol.33, No. 6, June 1994.

JDA (2010). Wedgefield Industrial Estate Extension, Port Hedland – Local Water Management Strategy (LWMS). Prepared for Landcorp, July 2010.

Knutson, T.R., McBride, J.L., Chan, J., Emmanuel, K., Holland, G., Landsea, C., Held, I., Kossin, J.P., Srivastava, A.K. & Sugi, M., 2010, Tropical cyclones and climate change, *Natural Geoscience*, Vol. 3, pp. 157-163.

Matsumoto, M and Nishimura, T (1998) Mersenne Twister: A 623-dimensiionally equidistributed uniform pseudorandom number generator. ACM Transactions on Modeling and Computer Simulations: Special Issue on Uniform Random Number Generation, 1998.

McConochie J.D., Mason L. B. and Hard T.A. (1999): A Coral Sea Cyclone Model Intended for Wave Modelling. Proceedings of Coasts and Ports 1999. Institution of Engineers, Australia, Perth, 1999.

National Tidal Centre (2009). "Australian Baseline Sea Level Monitoring Project – 2009 Annual Report". Prepared by the National Tidal Centre. Available at: http://www.bom.gov.au/oceanography/projects/ntc/ntc.shtml

Pawlowicz, R., B. Beardsley, and S. Lentz (2002). "Classical Tidal Harmonic Analysis Including Error Estimates in MATLAB using T_TIDE", Computers and Geosciences, 28, 929-937 (2002).

Shapiro L.J. (1983): The Asymmetric Boundary Layer Flow Under a Translating Hurricane. *Journal of Atmospheric Sciences*. Volume 40, pp 1984-1998.

USACE (1984). Shore Protection Manual (1984). Published by U.S. Army Corps of Engineers, Washington, DC.

van Vledder G, Goda Y, Hawkes P, Mansard E, Martin M, Mathiesen M, Thompson E F and Peltier E (1993). "Case studies of Extreme Wave Analysis: a Comparative Analysis." *Ocean Wave Measurement and Analysis – Proceedings of the Second International Symposium.* New Orleans, Louisiana.

WAPC (2010): Position Statement – State Planning Policy No 2.6 – State Coastal Planning Policy Schedule 1 Sea Level Rise. Prepared by the WAPC and the Department of Planning, WA. September 2010.

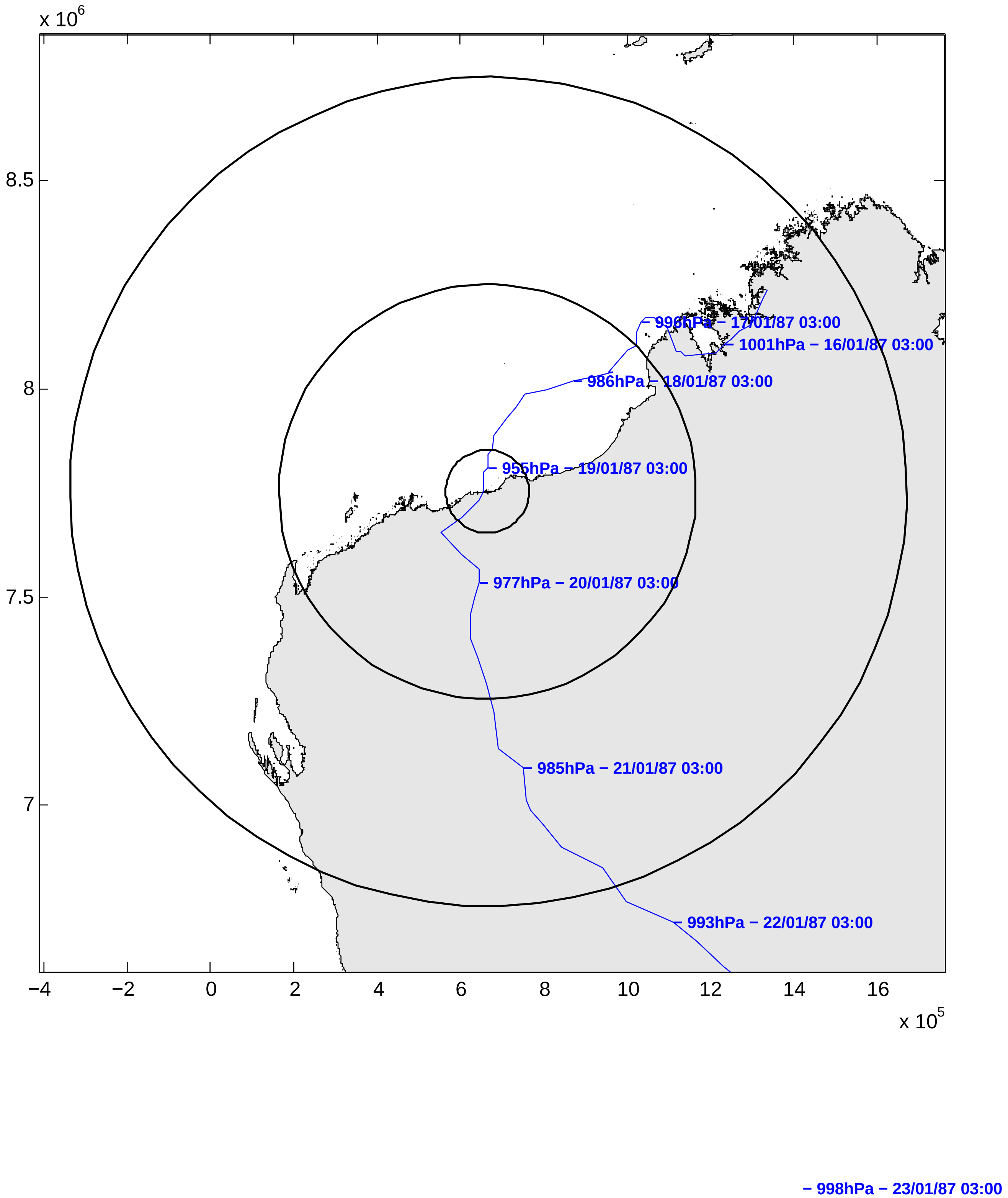
WNI (1998): Modelling of Cyclone Current and Wave Conditions off Port Hedland. Prepared for Halpern Glick Maunsell Pty Ltd. Ref: Job No. 2014, Report No R951.

Willmott, C.J., Ackleson, S.G., Davis, R.E,, Feddema, J.J., Klink, K.M., Legates, D.R., O'Donnell, J. and Rowe, C.M. (1985). Statistics for the Evaluation and Comparison of Models. Journal of Geophysical Research, Vol.90, No. C5, pages 8995-9905, September 20, 1985.

Wolanski E., Jones M., and Bun J.S. (1980). "Hydrodynamics of a Tidal Creek-Mangrove Swamp System". *Australian Journal of Marine and Freshwater Research*. Volume 31, pp431-450.

Appendix A.1

Cyclone Tracks for Calibration Events



Cardno Lawson Treloar

Port Hedland Coastal Vulnerability Study

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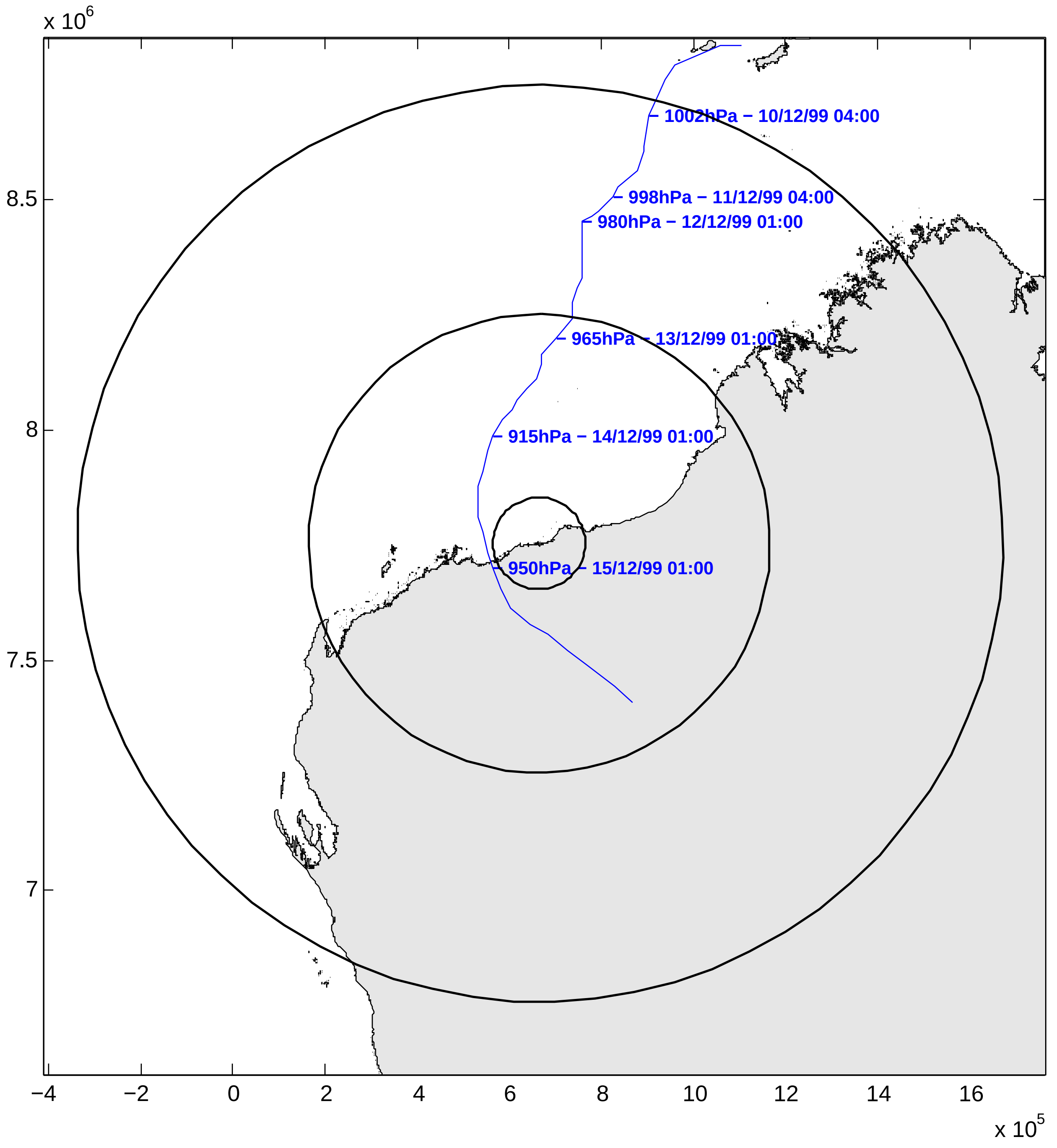

Port Hedland Coastal Vulnerability Study

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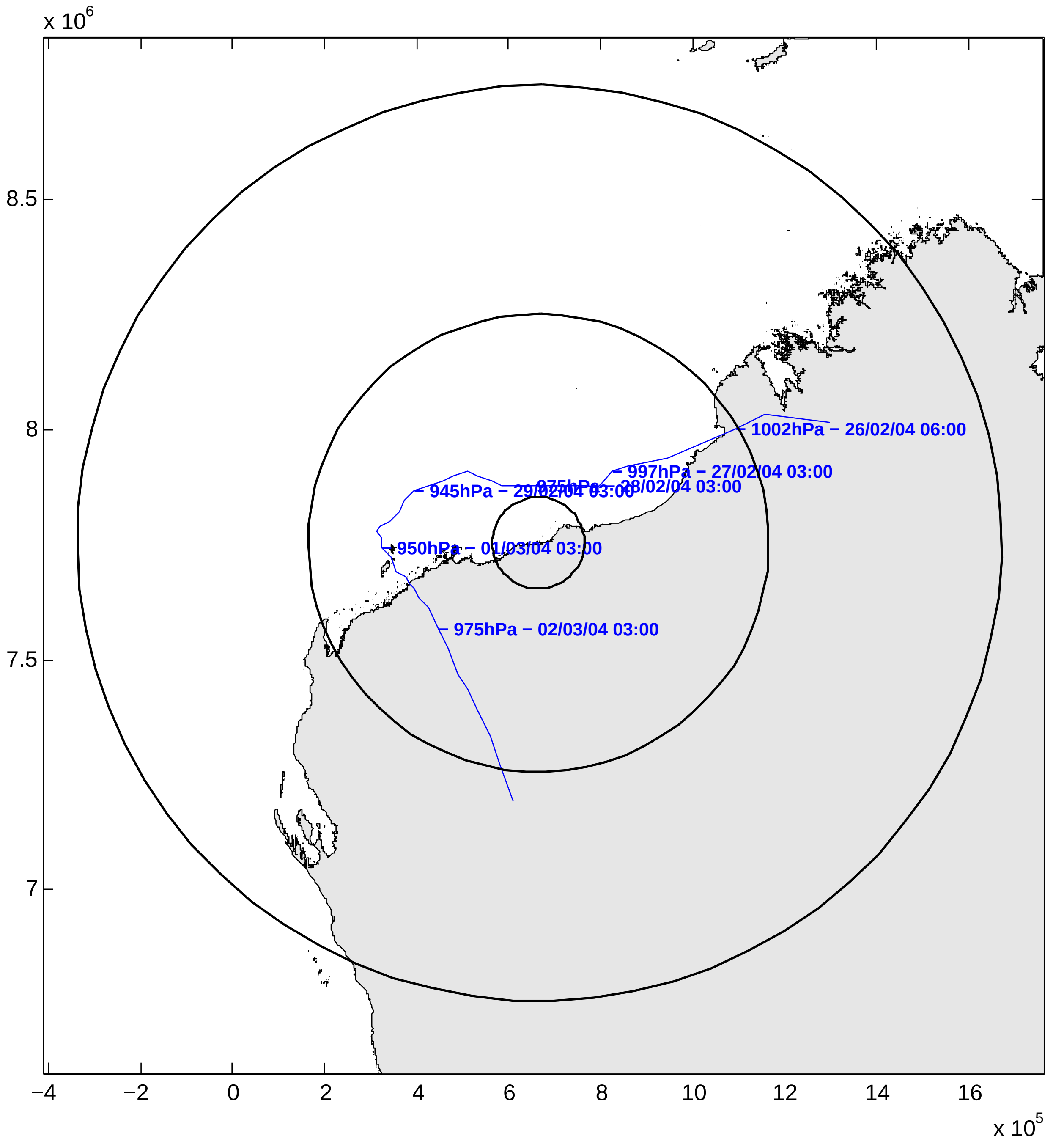

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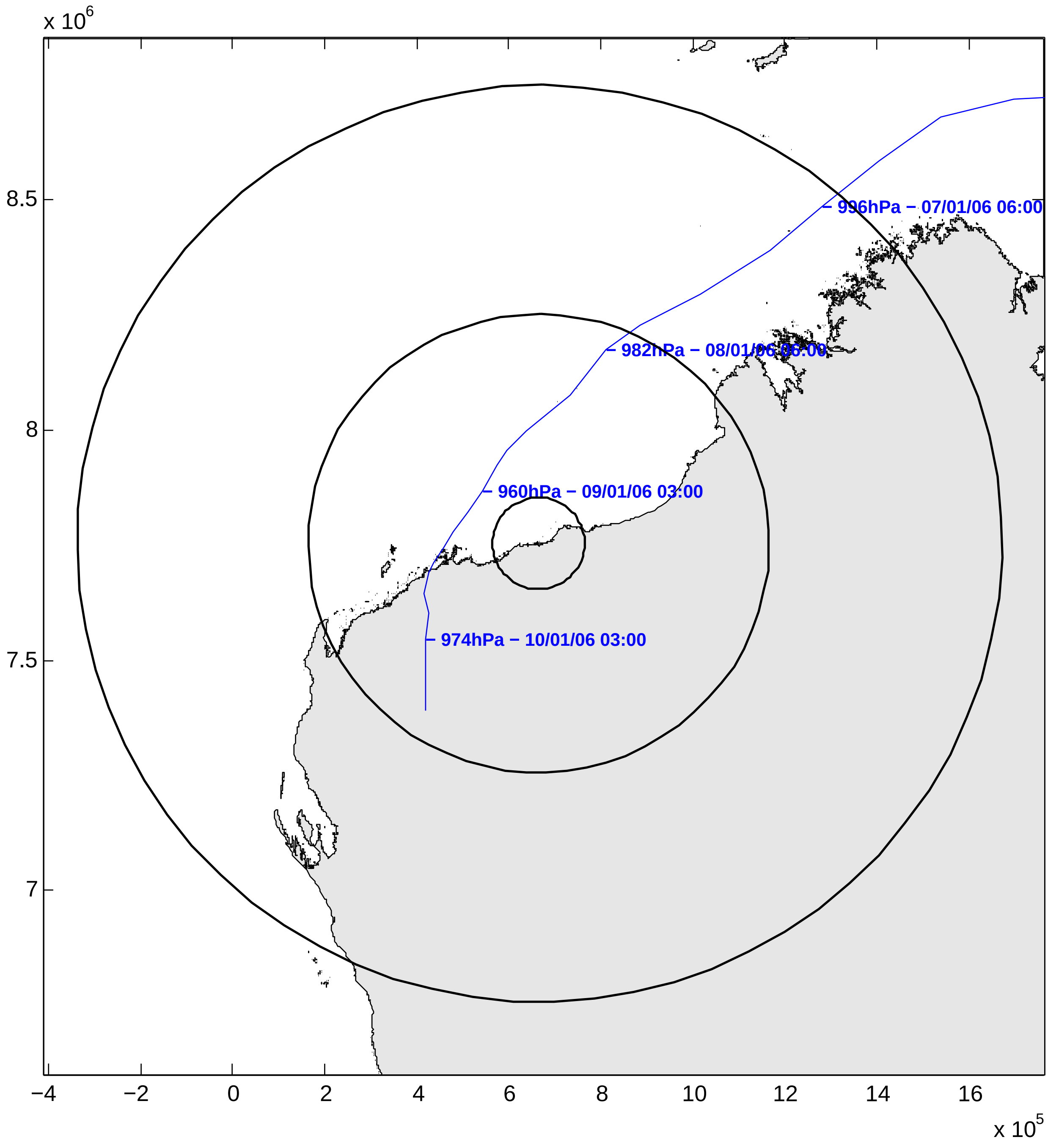

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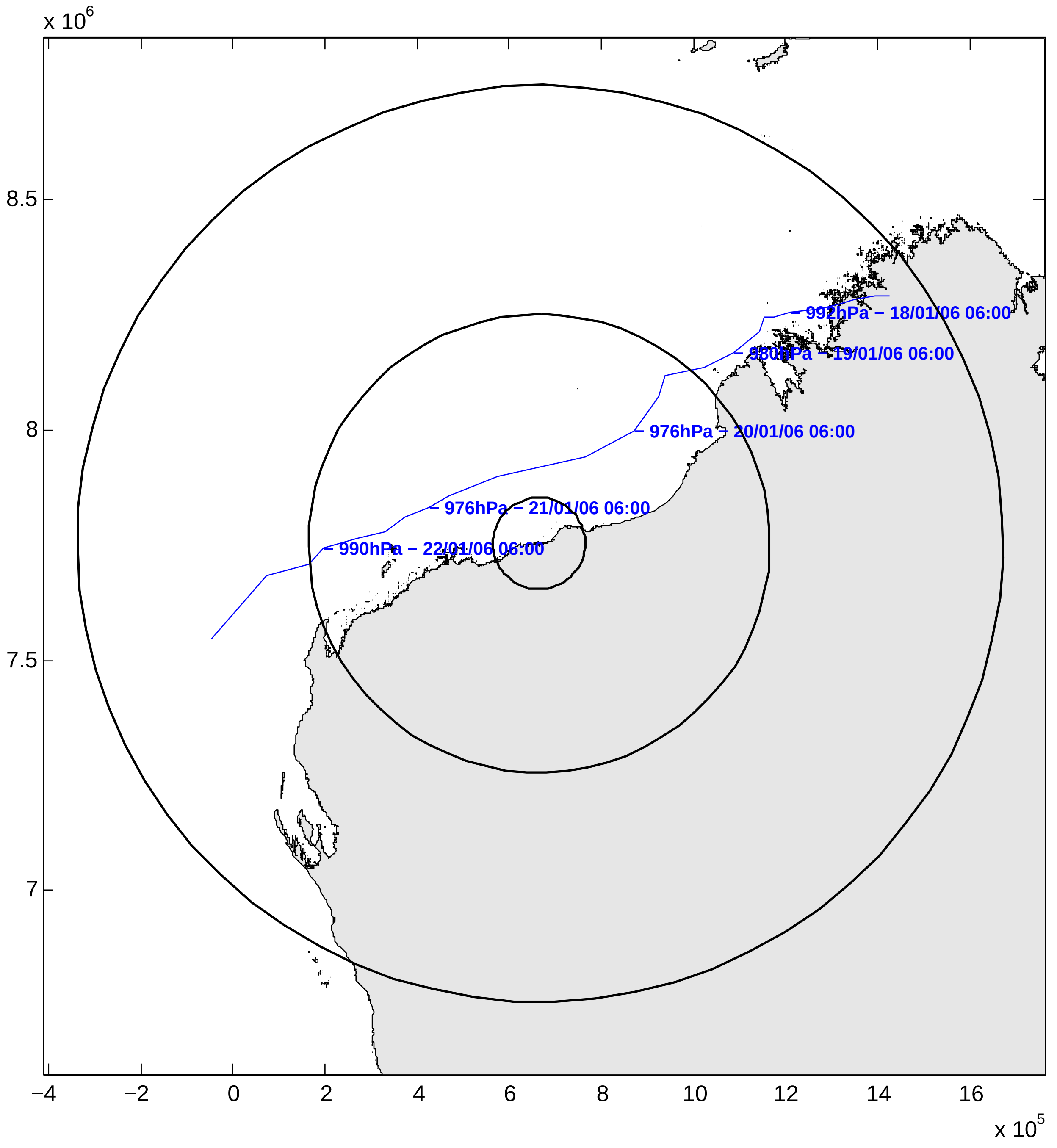

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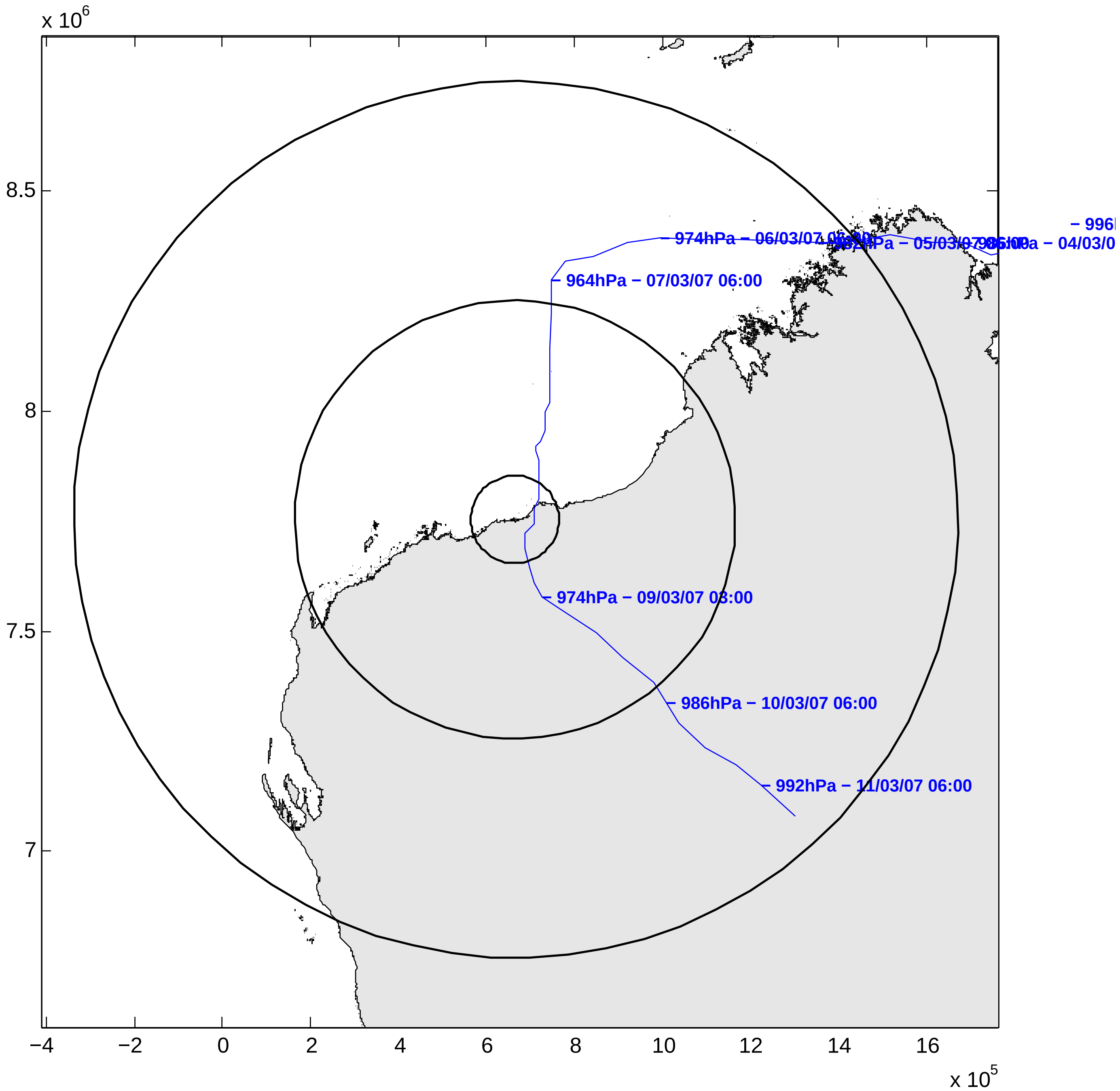

Port Hedland Coastal Vulnerability Study

TROPICAL CYCLONE GEORGE
27 Feb 2007 − 11 Mar 2007

LJ15014 November 2010
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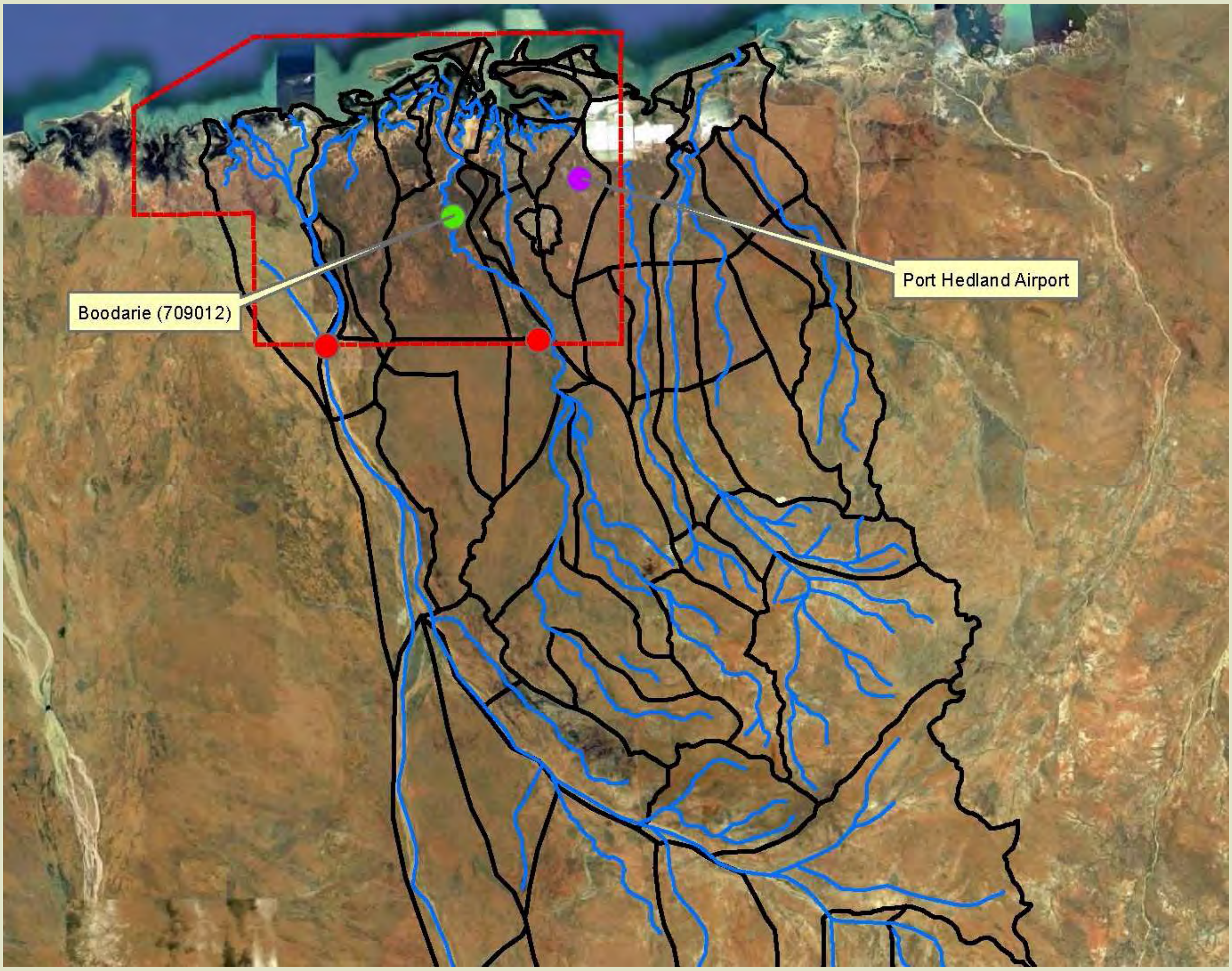
Appendix B

Hydrological Modelling

Port Hedland Coastal Vulnerability Study
Appendix B – Hydrological Modelling
Final

Job Number: LJ15014

Report Number: Rep1022p/Appendix B



Prepared for Landcorp

10 August 2011

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DISCLAIMER

The State Government and the Town of Port Hedland have a shared vision that will see Port Hedland transformed into Pilbara's Port City with a population of 50,000 by 2035.

Land availability, and in particular developable land above predicted future flood levels, is a clear constraint. In recognition of this the Royalties of Regions program has funded the preparation of the Port Hedland Coast Vulnerability Study. The Study considers the combined impact of flood from rainfall and storm surge. The Study was undertaken by Cardno Pty Ltd and was overseen by a Steering Committee that included the Department of Water, the Department of Transport and the Department of Planning.

The report is not a statutory document. However, the reports findings will be considered by statutory agencies in assessing future development proposals under the Town of Port Hedland's Town Planning Scheme and the WAPC's Statement of Planning Policy 2.6.

Developers and their consultants may use the report and its data for their own works but must undertake their own independent verification specific to their development proposal.

Document Control: Port Hedland Coastal Vulnerability Study – Hydrological Modelling Report

Version	Date	Author		Reviewer	
		Name	**Initials**	**Name**	**Initials**
1 – Preliminary Draft	8 March 2011	Samuel Cleary/ David Meyer	SLC / DLM	David Taylor / Heath Summerfield	DRT/HS
2 – Draft	4 April 2011	Samuel Cleary	SLC	David Taylor	DRT
3 – Final	10 August 2011	Samuel Cleary	SLC	David Van Senden	DVS

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B.1 OVERVIEW

The following report is a technical appendix document to the *Port Hedland Coastal Vulnerability Study Report.* This appendix specifically addresses technical aspects of the hydrological modelling program, including a description of the model systems, setup, calibration and the methodology for the design simulations. The overall report for the *Port Hedland Coastal Vulnerability Study* provides an overview of the whole project including details on the scope, data sets, outcomes from the various modelling components of the study, and the coastal vulnerability assessment.

Two hydrological models were created for the catchments surrounding Port Hedland and Shellborough respectively, the locations of which are shown in **Figure B.1** (end of report). Hydraulic modelling was undertaken to identify the amount of flow runoff that is generated by the catchments in rainfall events. The Port Hedland catchment is divided into 101 sub-catchments which each flow toward the coast. The two main sub-catchments in the Port Hedland catchment are the Turner River sub-catchment and the South West Creek sub-catchment. The Port Hedland catchment spans a distance of 150km and an area of 498,000ha (**Figure B.2** - end of report). The Shellborough catchment is smaller than the Port Hedland catchment as the 18 sub-catchments span a distance of 39km and an area of 44,000ha (**Figure B.3** - end of report). The following sections outline the model setup, calibration and key outcomes from the hydrological modelling.

B.2 MODEL SYSTEMS

For calculation of the surface water runoff from the catchment, a 1D model was created using the XPSWMM hydrologic and hydraulic modelling software package. The hydrologic component of the software uses the Laurenson non-liner runoff-routing method to simulate runoff from rainfall events. The Laurenson runoff-routing method assumes that runoff is proportional to slope, area, roughness, infiltration and percentage of imperviousness of a catchment. The model was calibrated to observed rainfall and runoff data available for a few locations within the catchment (**Section B.4**). The calibrated model was then used to simulate stream discharge in response to design rainfall events. The outputs from the hydrology model were then used as the inputs to the 2D hydraulic modelling of the lower catchment floodplain.

B.3 MODEL SETUP

B.3.1 Rainfall Data

Rainfall data was sourced from the Port Hedland Airport and Hillside gauging stations. The Port Hedland Airport is located near the coast and the Hillside gauging station is at the top of the Turner River catchment (see **Figure 2**). No rainfall gauging stations were located in the Shellborough catchment. The Port Hedland gauging station began recording data from 1953 and contains a long and relatively accurate dataset. The Hillside gauging station began recording 1999 however contains variable quality of data. Both gauges record 6 minute pluvial rainfall data. **Table B.3.1** outlines the gauging period and the agency responsible for its maintenance.

Table B.3.1: Rainfall Gauging Station Details

Station Name	Type of Stream Gauging Station	Operational Period	Managing Agency
Port Hedland Airport	Meteorological data (wind, pressure and rainfall)	1942 – 2010	Bureau of Meteorology
Hillside	Meteorological data (wind, pressure and rainfall)	1999 – 2003	Bureau of Meteorology

Four significant rainfall events were identified from the Port Hedland rainfall data for model calibration which are shown in **Table B.3.2**. The 1995, 1997, 2000 and 2007 events correspond to Cyclone Bobby, Cyclone Rachel, Cyclone Steve and Cyclone George respectively. The 2003 event was also caused by a cyclonic event but was unnamed by the Bureau of Meteorology (BOM). The cyclone tracks are shown in **Appendix B.1**. The Hillside gauging station, due to the variable quality of its recording processes, was only able to capture a single corresponding rainfall event as shown in **Table B.3.3**.

Table B.3.2: Significant Rainfall Events Observed at Port Hedland Airport Gauging Station

Start Date	End Date	Total Rainfall Time	Total Rainfall (mm)	Average Intensity (mm/hour)	Corresponding ARI Event
22/2/1995	24/2/1995	2 days, 9 hours	101.15	1.77	2 year 48 hour
7/1/1997	7/1/1997	23 hours	111.88	4.86	5 year 24 hour
5/3/2000	6/3/2000	1 day, 8 hours	98.02	3.06	2 year 30 hour
23/1/2003	25/3/2003	2 days	176.20	3.67	10 year 48 hour
08/3/2007	09/3/2007	1 day, 12 hours	122.46	3.40	5 year 36 hour
11/3/2007	12/3/2007	1 day, 5 hours	110.58	3.81	5 year 30 hour

Table B.3.3: Significant Rainfall Events Observed at Hillside Gauging Station

Start Date	End Date	Total Rainfall Time	Total Rainfall (mm)	Average Intensity (mm/hour)	Corresponding ARI Event
24/1/2003	26/1/2003	1 days, 23 hours	154.56	3.29	10 year 48 hour

A comparison of the rainfall patterns for the 2003 event is shown in **Figure B.3.1**. The two peak rainfall periods were observed approximately 5 hours apart from each other at the two gauging stations. With the cyclone event tracking inland, the average rainfall intensity recorded at Hillside gauging station decreased in comparison to rainfall recorded at the Port Hedland gauging station. The rainfall pattern changed as the cyclone moved through the catchment from north to south. However due to the limited data at Hillside, the change in rainfall patterns throughout the catchment were unable to be quantified.

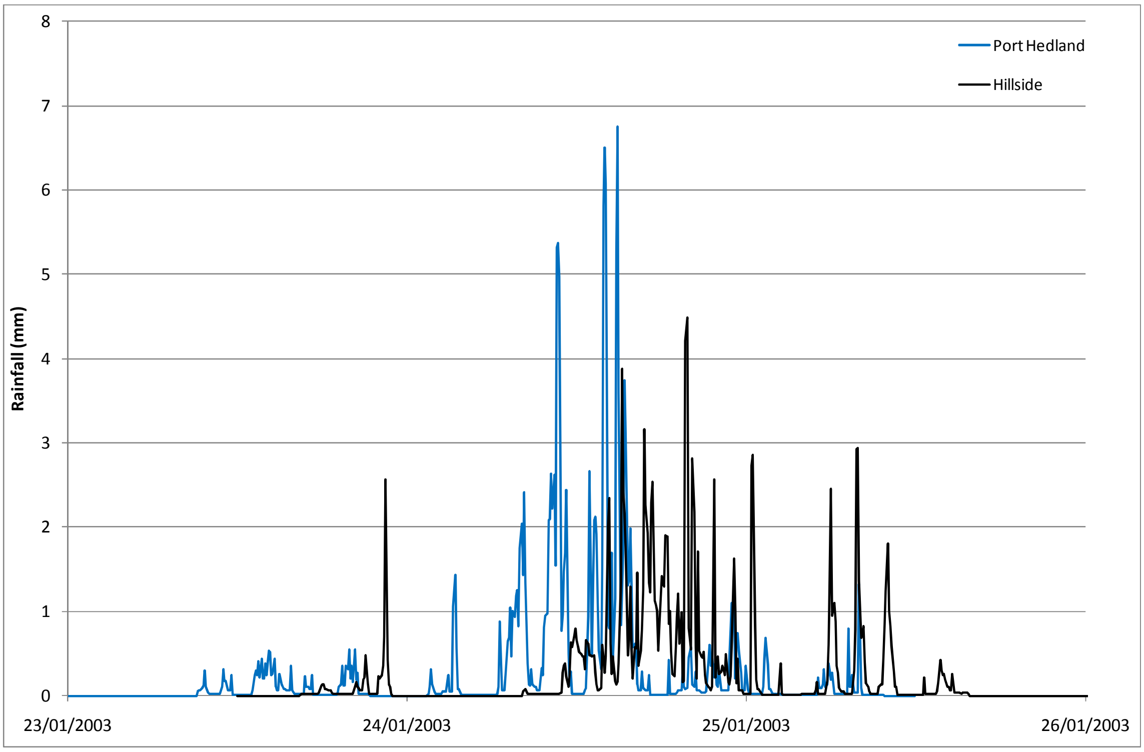

Figure B.3.1: Rainfall Patterns at Port Hedland and Hillside Gauging Stations

B.3.2 Catchments

The sub-catchments were delineated from Satellite topography and LIDAR data. The LIDAR data was compiled after flights were undertaken in November 2010. AAM Pty Ltd undertook the LIDAR survey providing 0.5m contours containing a vertical accuracy of +/-0.10m. Geological mapping units were identified from 1:250,000 geological mapping datasets. The sub-catchments were divided into two broad geological units, Sand and Rock. The infiltration rates were based on the geological units and calibrated to observed flow and stage data. The Rock geological unit is mainly located in the upper reaches of the Turner River catchment, whereas the remainder of the catchment is classified as Sand. **Figures B.2 and B.3** at the end of the report present the sub-catchments, streamlines, gauging station locations and the LIDAR and 2D hydraulic model extent for both the Port Hedland and Shellborough regions.

The cross sections used for the 1D hydraulic links (i.e. river reaches) were generated from the Shuttle Radar Topography Mission (SRTM) satellite data from NASA. SRTM data has a horizontal resolution typically ranging between 45-60m (Smith & Sandwell, 2003) and a vertical accuracy of +/-8m (Rodriguez *et al.*, 2005). The river cross section at the bridge where the stage height gauging station is located was identified from high resolution LIDAR data. The river cross section at the bridge is important as it has a direct impact on the calculation of the stage height.

B.4 MODEL CALIBRATION

The hydrological model was calibrated to the available observed flow and stage data in order to provide confidence that the model outputs give a reasonable representation of the real system. Calibration of the models to observed data provides confidence in future predictive simulations. Calibration was achieved by identifying significant rainfall events and adjusting the model infiltration and roughness parameters to the observed stage and flow data. The focus of the calibration was threefold: timing of the peak, overall volume of the event and shape of the event hydrograph, and the matching of the peak flow rate during the storm event.

It should be noted that this section specifically addresses calibration of the hydrological model. Further model calibration and comparative assessment with other models are presented in the hydraulic modelling report in **Appendix C (Main Report).**

B.4.1 Flow and Stage Data

Fifteen minute flow and stage data was sourced from the Department of Water (DoW) for the two gauging stations in the catchment. Pincunnah gauging station (709010) is situated on the Turner River in the upper reaches of the catchment. The gauging dataset provided by DoW for Pincunnah contains only flow data. Boodarie gauging station (709012) is located on South West Creek and only records stage height data. **Table B.4.1** outlines the gauging period and the agency responsible for its maintenance with the locations of the gauging stations shown in **Figure B.2**.

Table B.4.1: Streamflow Gauging Station Details

Station Name (Number)	Type of Stream Gauging Station	Operational Period	Managing Agency	Corresponding River	Catchment Area (ha)
Pincunnah (709010)	Flow	1985 – 2010	Department of Water	Turner River	96,774
Boodarie (709012)	Stage	2000 – 2007	Department of Water	South West Creek	48,140

Comparison of the DOW data recorded from Boodarie gauging station identified that these recorded data did not match the LIDAR data. The minimum recorded data was approximately 3m higher than the elevation of the channel under the bridge. The maximum recorded data was approximately 1.5m higher than the road elevation. Boodarie did not have a rating table and was known to have issues with fluctuating sand levels within the channel adjacent to the culverts that introduce errors to level recordings. The Boodarie gauging station was mainly used in citing access issues during and following flood events (pers comm. Kris Pascoe, Department of Water, 24 February 2011). Boodarie was shut down in 2007 after only seven years of operation. Because of the poor quality of data and limited records available at Boodarie, the stage height data was deemed inadequate for calibrating the hydrological model.

B.4.2 Calibration Parameters

Several hydrologic loss models were used in the calibration of the model including the "Initial Loss – Continuing Loss", "Initial Loss – Proportional Loss", Horton and GreenAmpt methods. Based on the review of the hydrologic models and the lack of accurate calibration data, the most appropriate model for the project was determined to be the "Initial Loss – Continuing Loss" infiltration model. The loss values were calibrated for the soil types identified in the catchment. The hydrologic model was applied for each of the significant rainfall events identified in **Table B.3.2**.

For each event, the model was calibrated independently to the observed data for the associated rainfall event. This approach was used as each event behaved differently due to the antecedent conditions. In order to find the most consistent and expected catchment parameters for the predicted model hydrographs, the best calibration for each individual event was required. The two rainfall events in 2007 were modelled separately as there were two days between the rainfall periods of the cyclone event. As mentioned above, the antecedent conditions of a wetter catchment required a lower initial loss to be used in the second event.

The infiltration loss and roughness parameters used to calibrate the significant rainfall events are summarised in **Table B.4.2**.

Table B.4.2: Calibrated Infiltration Loss Parameters for XPSWMM Hydrological Modelling

Soil Units	Rock			Sand		
Rainfall Events	Initial Loss (mm)	Continuing Loss (mm)	Manning's n	Initial Loss (mm)	Continuing Loss (mm)	Manning's n
22/2/1995	70	0	0.05	85	0	0.07
7/1/1997	30	0	0.07	50	0	0.09
5/3/2000	30	0	0.06	50	0	0.08
08/3/2007	70	0	0.05	85	0	0.07
11/3/2007	50	0	0.06	70	0	0.08

The initial loss parameters are consistent with the mean initial loss values proposed for the North West Pilbara region in *Australian Rainfall & Runoff* which identify loam soils as having initial loss values ranging between 22-51mm (The Institute of Engineers, 2001).

The 2003 event was identified as the largest flow event within the dataset and hence was identified as the most appropriate event to calibrate the model. Calibration of the hydrological model to the 2003 event was attempted but resulted in a faster modelled response time than was identified in the observed data. The DoW was contacted to determine if there were inaccuracies in the observed data. The DoW confirmed that the Pincunnha gauging station was vandalised and burnt in 2002 causing damage to the bubbleline resulting in the station being out of order between 6/6/2002 and 6/2/2003. Flow data from the 2003 rainfall event was recreated based on telemetered data from three surrounding rainfall gauging stations and observed flood marks. As this flow data is unreliable, the 2003 rainfall event was excluded from the calibration process.

B.4.3 Calibration Results

The rainfall events were modelled in XPSWMM and the flow data was calibrated to the observed records from Pincunnah station. The peak flow rates and discharge volumes of the observed and calibrated events reported at Pincunnah station are shown in **Table B.4.3**. The hydrographs are shown in **Figure B.4.1**, **Figure B.4.2**, **Figure B.4.3**, **Figure B.4.4** and **Figure B.4.5**.

Table B.4.3: Peak Flow and Discharge Volumes of Observed and Calibrated Modelled Events

Observed Event				Modelled Event				Difference		
Time of Peak Flow	Time to Peak (hrs)	Peak Flow (m^3/s)	Volume (GL)	Time of Peak Flow	Time to Peak (hrs)	Peak Flow (m^3/s)	Volume (GL)	Time to Peak (hrs)	Peak Flow (%)	Volume (%)
25/2/1995 09:15	67.9	231.0	27.22	24/2/1995 13:00	47.6	235.6	22.68	-29.8	2	-17
9/1/1997 00:45	48.8	196.5	10.59	8/1/1997 21:00	45.0	181.9	15.78	-7.7	-7	30
6/3/2000 20:15	33.9	499.1	46.53	6/3/2000 23:00	36.6	357.7	29.08	-8.1	-28	-38
9/3/2007 22:15	34.3	290.7	33.49	9/3/2007 12:45	24.8	452.4	35.02	-27.7	56	4
12/3/2007 00:00	16.7	625.2	51.72	12/3/2007 17:45	34.5	596.9	42.14	106.3	-5	-19

The shapes of the calibrated flow hydrographs, the peak flows and the discharged volumes are all similar to the observed values for the 1995, 1997 and the first 2007 storm events. The calibration hydrograph for the second rainfall event in 2007 responds significantly slower than the observed data. The peak flow rate for the modelled hydrograph matches well with the observed hydrograph indicating the initial infiltration loss rates are adequate. The general shape of the two hydrographs matches fairly well, although the observed data has two peak flows. The quick response and double peak for the observed hydrograph is attributed to the spatially variable rainfall patterns that are not captured in the model.

Rainfall gauged data within the catchment is limited in terms of both its quantity and quality. Temporal rainfall patterns were assumed to be even across the whole catchment as there is limited data in the Hillside gauging station to determine variations in rainfall patterns for the different calibration events. The second rainfall event recorded in 2007 shows a time lag in the modelled data which is attributed to the spatial- temporal rainfall variability throughout the catchment.

The quantity and quality of flow and stage data available to calibrate the model and associated infiltration parameters is limited. Pincunnnah station is the only streamflow gauging station on the Turner River. While the upper region of the catchment can be calibrated with some level of confidence, it is unknown if the mid to lower regions of the catchment are accurately calibrated. Additionally, the Boodarie station only contains stage height data and does not record flow data. This limits the ability to confirm that the flow and stage height data are coupled correctly.

Overall, the hydrological model achieves a reasonable calibration given the limited available measured data. The focus on the calibration has been on simulating peak flows and total event volumes more accurately, with a lesser focus on the event phasing. The design simulations for the flood inundation extents (**Appendix C**) adopt a prescribed relationship between peak catchment flows and ocean water levels and therefore the phasing from the hydrological model has little influence in the application of the hydrological flows in the hydraulic modelling.

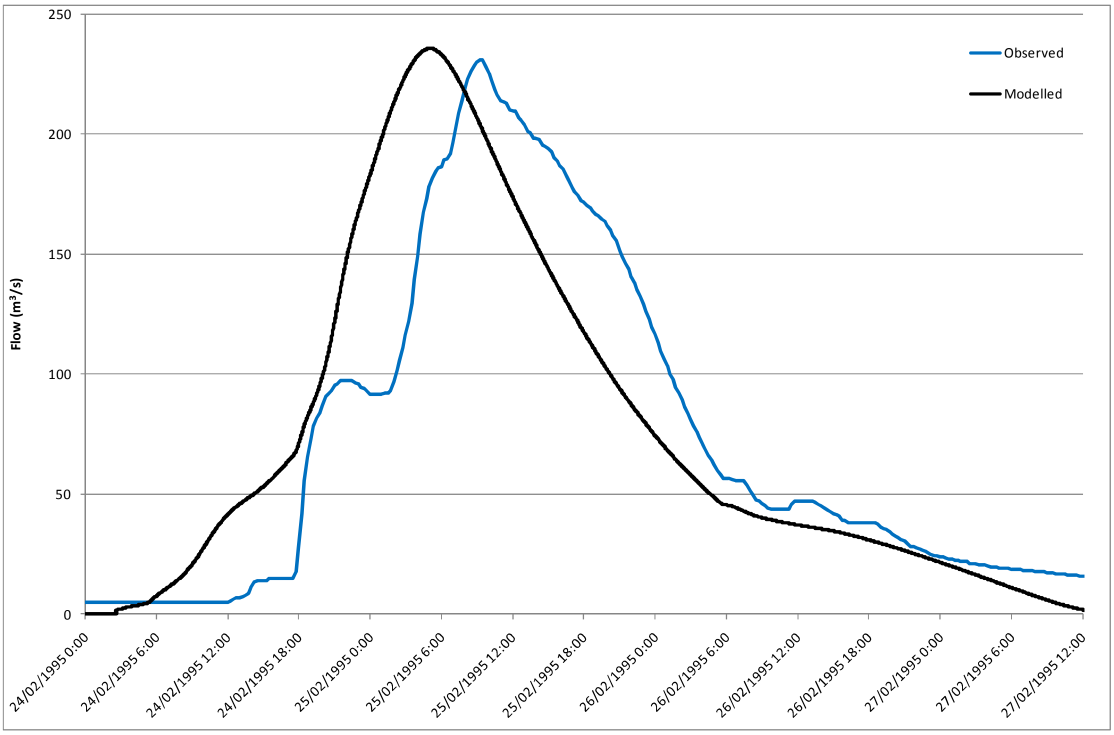

Figure B.4.1: Calibration of the 1995 Storm Event at Pincunnah Gauging Station (Turner River)

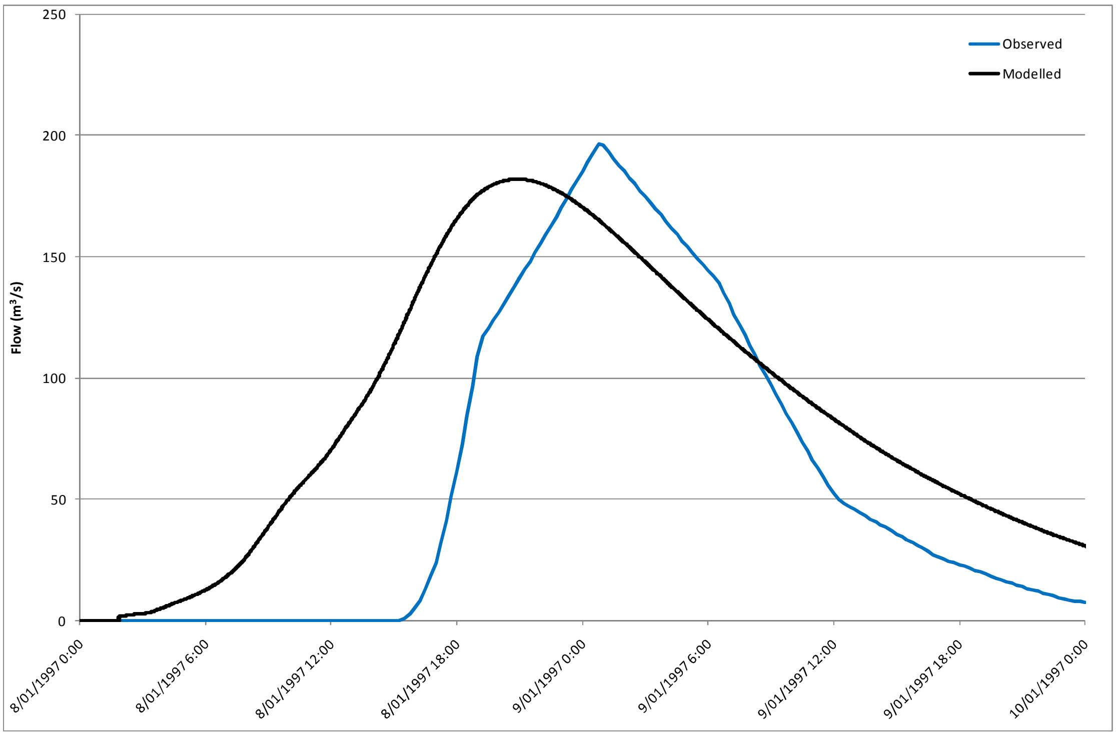

Figure B.4.2: Calibration of the 1997 Storm Event at Pincunnah Gauging Station (Turner River)

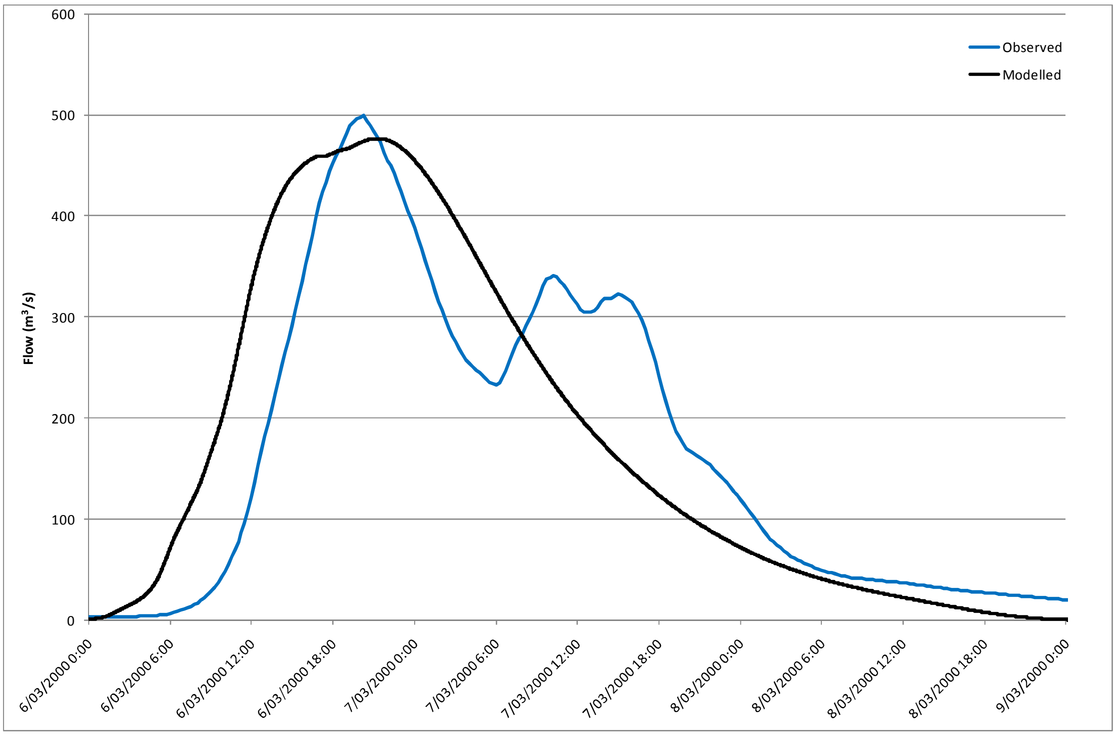

Figure B.4.3: Calibration of the 2000 Storm Event at Pincunnah Gauging Station (Turner River)

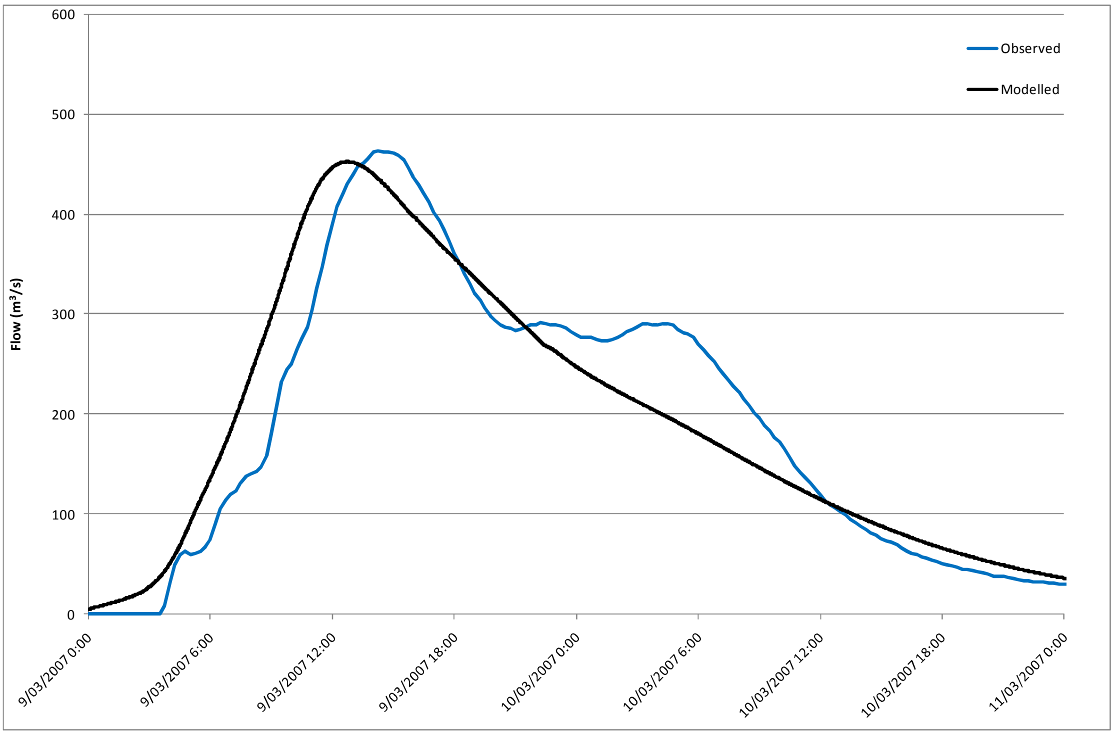

Figure B.4.4: Calibration of the first 2007 Storm Event at Pincunnah Gauging Station (Turner River)

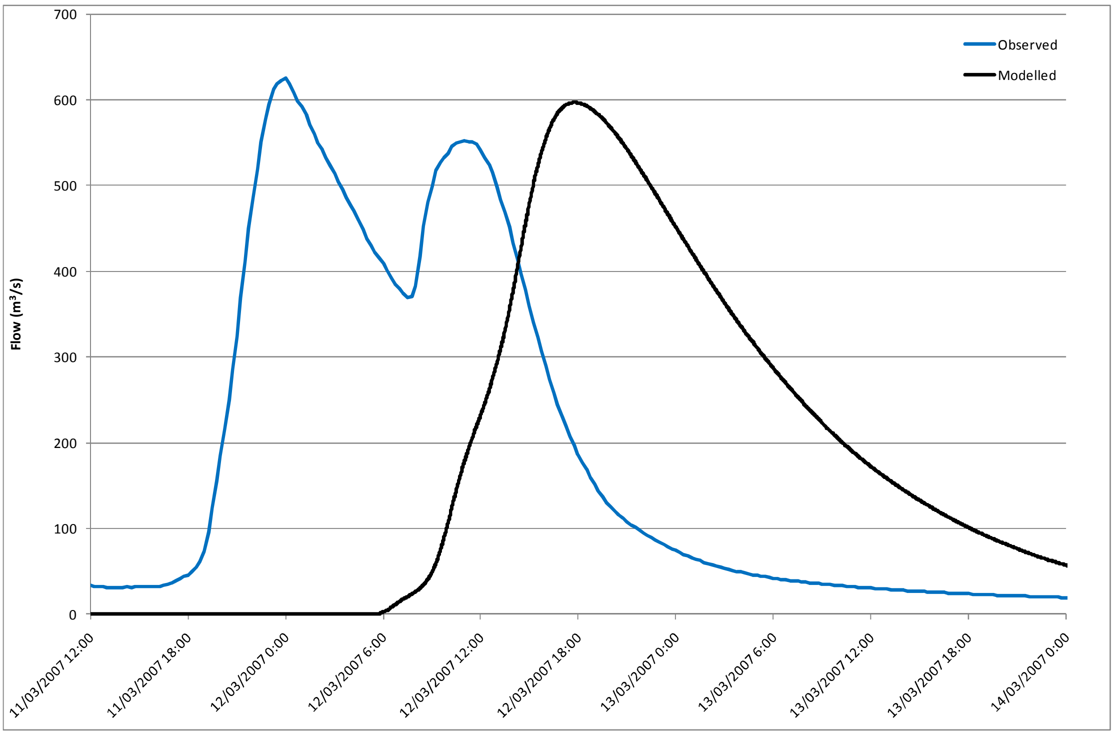

Figure B.4.5: Calibration of the second 2007 Storm Event at Pincunnah Gauging Station (Turner River)

B.4.4 Port Hedland Region Flood Studies

With the recent development in the Port Hedland region, a number of site specific drainage and flood studies have been undertaken in the region which in the absence of significant measured calibration data provides a benchmark for the hydrological and hydraulic models developed for the *Port Hedland CVS*. **Appendix C** (Main Report) presents a summary of the other flood studies from the Port Hedland region and a brief summary of the coverage area and whether there are any comparable flow locations and scenarios.

B.5 DESIGN EVENT SIMULATIONS

B.5.1 Introduction

A design rainfall event is a probabilistic or statistical estimate of a rainfall pattern with an average recurrence interval (ARI) or exceedance probability (The Institute of Engineers, 1987). The design rainfall event simulations were generated to determine the extent, associated risk and potential impacts of design flood events. The design events were created using the following procedure:

- Creation of Intensity Frequency Duration (IFD) parameters and curves.
- Determine appropriate modelling parameters from the model calibration.
- A sensitivity analysis on the parameters chosen for the design rainfall events.
- A Flood Frequency Analysis (FFA) prediction based on observed streamflow gauging data.
- Creation of design ARI events.
- Comparison of design event results with Rational and Index Flood Method estimations.

B.5.2 IFD Parameters

The IFD curves for the Port Hedland region were derived from the parameters identified in *Australia Rainfall and Runoff* (The Institute of Engineers, 1987) as shown in **Table B.5.1**. The F2, F50 and skewness parameters are equal to 4.13, 16.13 and 0 respectively.

Table B.5.1: IFD Parameters at Port Hedland (The Institute of Engineers, 1987)

Frequency (year)	Duration (hour)	Intensity (mm/hr)
2	1	35.47
2	12	5.22
2	72	1.30
50	1	104.39
50	12	19.04
50	72	4.74

The IFD curves were generated using the DESrain program created by M. Boyd at the University of Wollongong. The DESrain program was able to extrapolate design rainfall events out to 500 years ARI. The IFD Curves are shown in **Figure B.5.1**. The Zone 7 temporal rainfall patterns were used in the modelling of the ARI storm events (The Institute of Engineers, 1987).

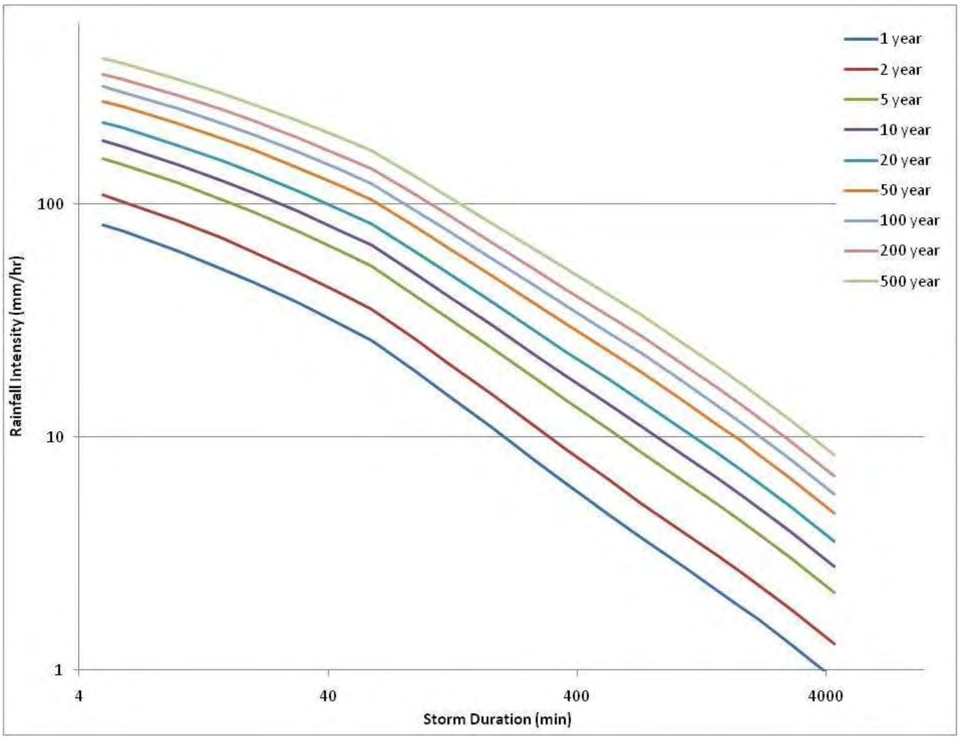

Figure B.5.1: IFD Curves for Port Hedland.

B.5.3 Modelling Parameters

The initial infiltration loss and Manning's roughness parameters chosen for the design storm simulations are listed below:-

Rock
- Initial loss = 50mm
- Manning's n = 0.05

Sand
- Initial loss = 70mm
- Manning's n = 0.07

The infiltration and roughness parameters chosen were within the mid range of values used for calibration (see **Section B.4.2**). These values were also identified as the most common out of the range of values used in the calibration process. No flow gauging stations are located within the Shellborough catchment and thus calibration of this area was not possible; however, considering the catchment is only 65km to the east of the Port Hedland catchment, the same calibration values were determined to be applicable for use in the Shellborough model as the soil types within the area are similar to those identified in the Port Hedland catchment.

B.5.4 Sensitivity Analysis

B.5.4.1 Calibrated Modelling Parameters

A sensitivity analysis was conducted on the 2 year, 100 year and 500 year ARI storm events in comparing the range of differences associated with the calibration parameters utilised. The sensitivity analysis aims to provide a better understanding of the range of uncertainty associated with the possible calibration parameters for the XPSWMM model, hence allowing a better understanding of the uncertainty and risk associated with the selected hydrology.

A combination of roughness coefficients (Manning's) and the initial infiltration loss based on the results identified in **Section B.4.2** were used to create a range of results. **Table B.5.2** shows the combination of the infiltration losses and Manning's roughness coefficients. As discussed in **Section B.5.3**, the "2a" parameter value set was identified as the most appropriate set of modelling parameters to be used as part of the design event simulations.

Table B.5.2: Combination of Initial Loss and Manning's n for Sensitivity Analysis

Manning's n	Initial Loss (mm)		
Rock (Sand)	30 (50)	50 (70)	70 (90)
0.05 (0.07)	1a	2a	3a
0.07 (0.09)	1b	2b	3b

An initial assessment of the design ARI events identified critical storm durations of 36, 12 and 12 hours for the 2, 100 and 500 year ARI events, respectively. The sensitivity analysis was conducted on these storm events with the peak flow rates reported at Pincunnah shown in **Table B.5.3** and the associated hydrographs in **Figure B.5.2**, **Figure B.5.3** and **Figure B.5.4**.

Table B.5.3: Peak Flow Rates for the Sensitivity Analysis Design Storm Events Reported at Pincunnah Station

Design Simulation	2 year 36 hour ARI		100 year 12 hour ARI		500 year 12 hour ARI	
	Peak Flow (m^3/s)	Difference from 2a (%)	Peak Flow (m^3/s)	Difference from 2a (%)	Peak Flow (m^3/s)	Difference from 2a (%)
1a	314	133	4,444	10	7,439	6
1b	245	83	3,545	-13	6,034	-14
2a	135	-	4,054	-	7,022	-
2b	103	-23	3,209	-21	5,669	-19
3a	17	-87	3,665	-10	6,613	-6
3b	12	-91	2,880	-29	5,315	-24

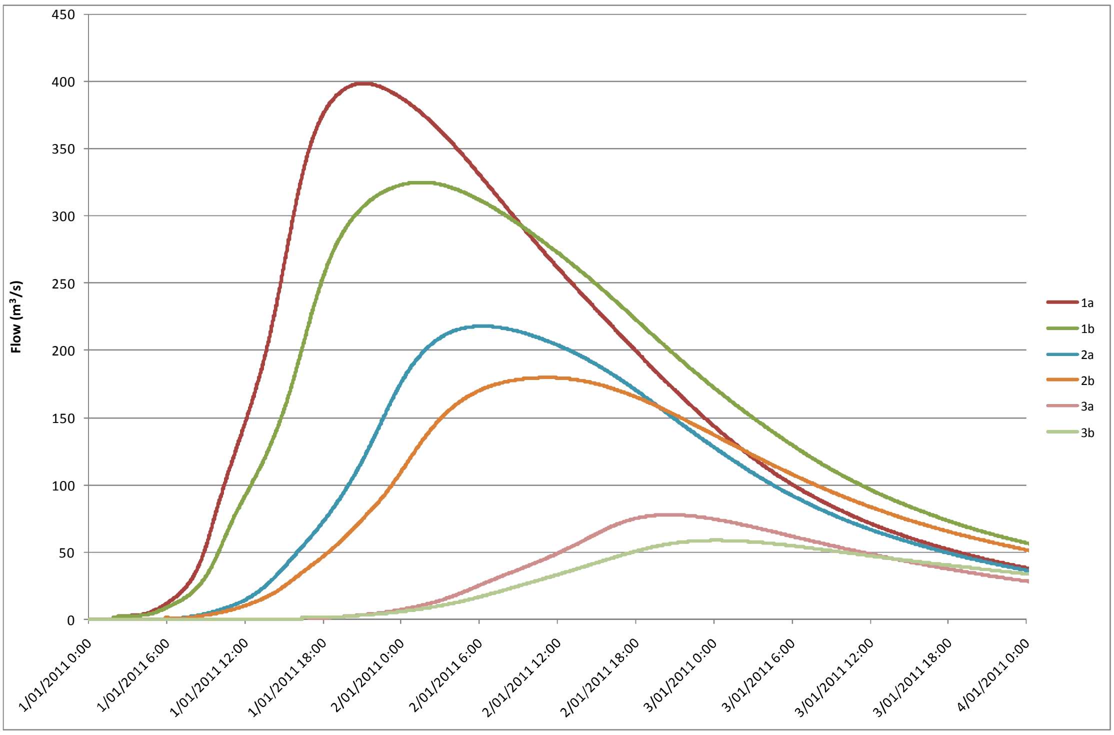

Figure B.5.2: Sensitivity Analysis 2 year Design Storm Hydrograph

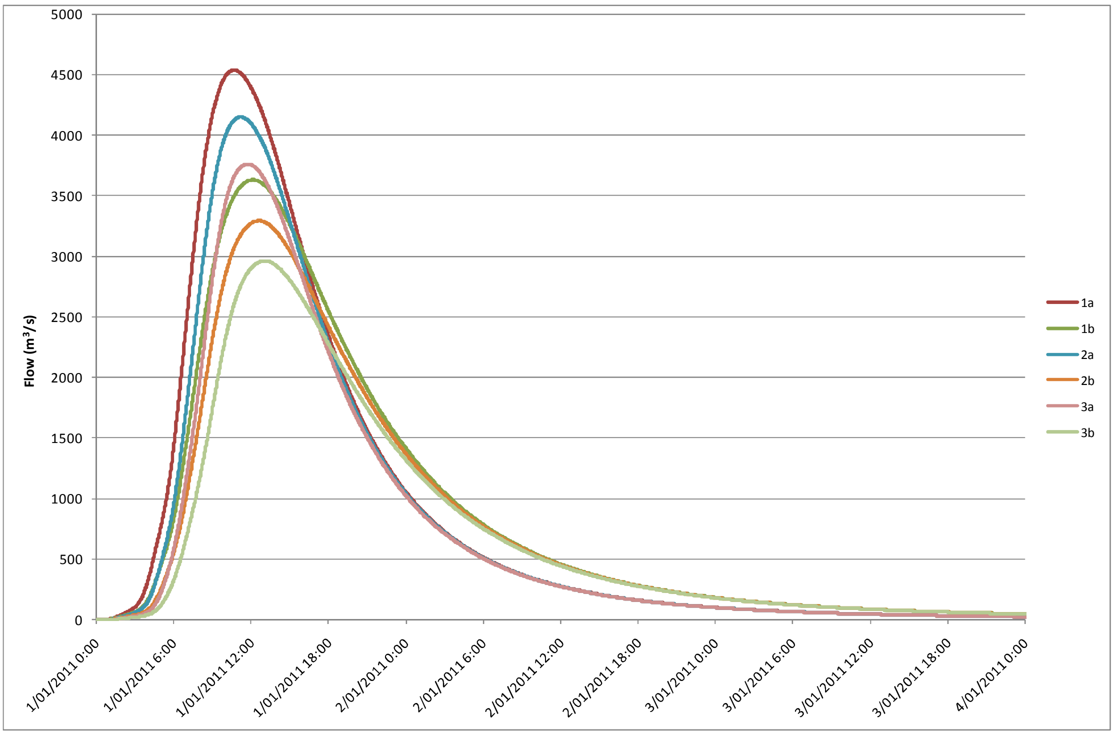

Figure B.5.3: Sensitivity Analysis 100 year Design Storm Hydrograph

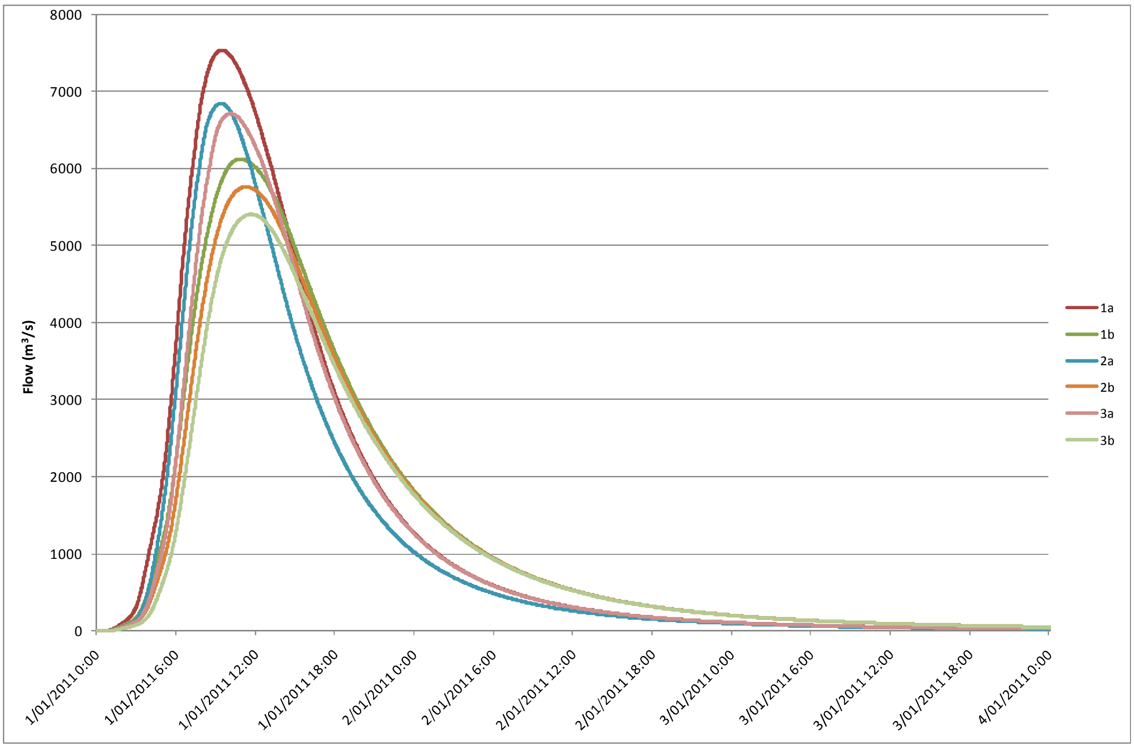

Figure B.5.4: Sensitivity Analysis 500 year Design Storm Hydrograph

The sensitivity analysis identified significant variation in the peak flow rates with the different initial loss and Manning's roughness values used. The small rainfall event of the 2 year ARI design storm is severely impacted by the change in initial loss rates due to the low rainfall intensity of the storm event. The 100 year and 500 year ARI design storms flow rates are affected by the changes in initial loss values more than the changes to the Manning's roughness coefficient.

B.5.4.2 Continuing Loss Parameters

While the calibration results indicate that zero continuing loss rates result in the best calibration to the available observed data, this is inconsistent with the advice provided in *Australian Rainfall and Runoff* (The Institute of Engineers, 2001) and intuitive understanding of the characteristics of the sand soil based on the geological mapping of the catchment. A sensitivity analysis was undertaken to determine the impact on incorporating a continuing loss rate in the calibration of the 100 year ARI design storm events. The 2000 and 2007 calibration events were reassessed using continuing loss rates as based on the values provided in *Australian Rainfall and Runoff*. The calibrated values and runs are outlined in **Table B.5.4** with the results of modelling recorded at Pincunnah station shown in **Figure B.5.5** and **Figure B.5.6** for the 2000 and 2007 calibration events respectively. The most appropriate Manning's n values identified in the sensitivity analysis above (i.e. 0.05 for rock and 0.07 for sand) were maintained for this analysis.

Table B.5.4: Combination of Initial and Continuing Loss for Sensitivity Analysis

Initial Loss (mm)	Continuing Loss (mm/hr)		
Rock (Sand)	0 (0)	3 (5)	4 (6)
2000 Calibration Event			
30 (50)	4a	4b	-
10 (20)	-	4c	-
2007 Calibration Event			
70 (85)	5a	-	5b
35 (55)	-	-	5c

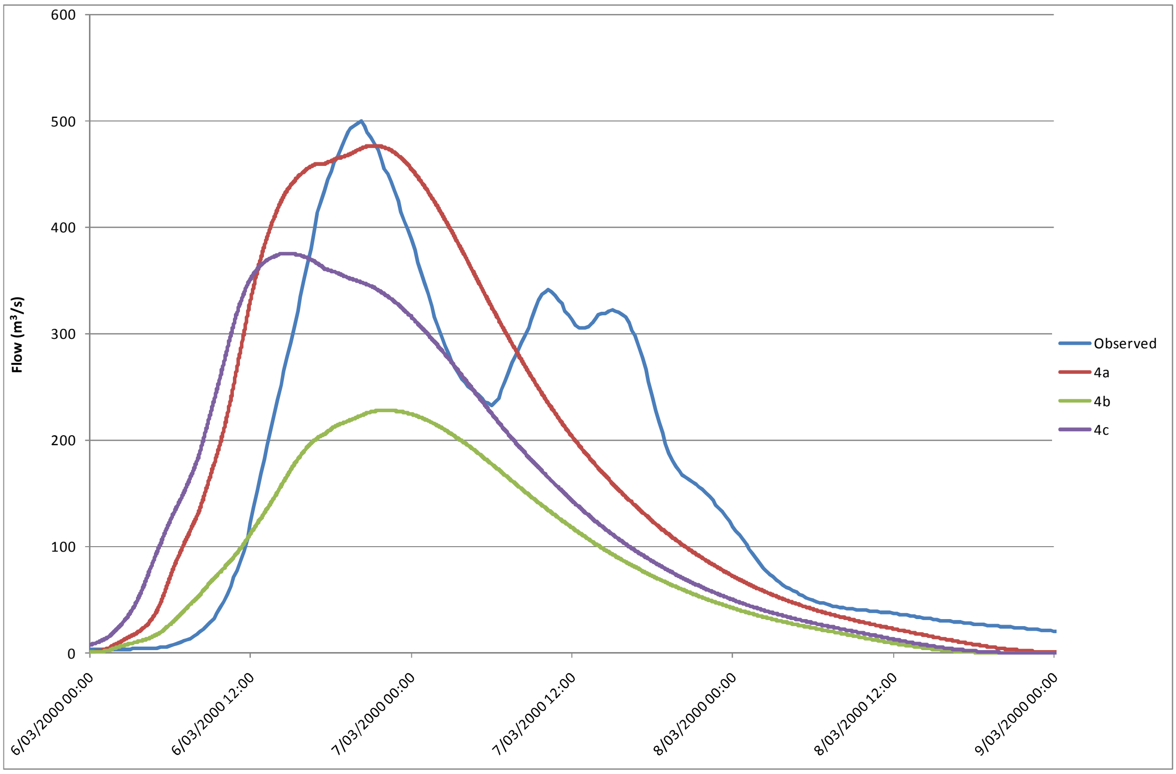

Figure B.5.5: Sensitivity Analysis 2000 Calibration Event

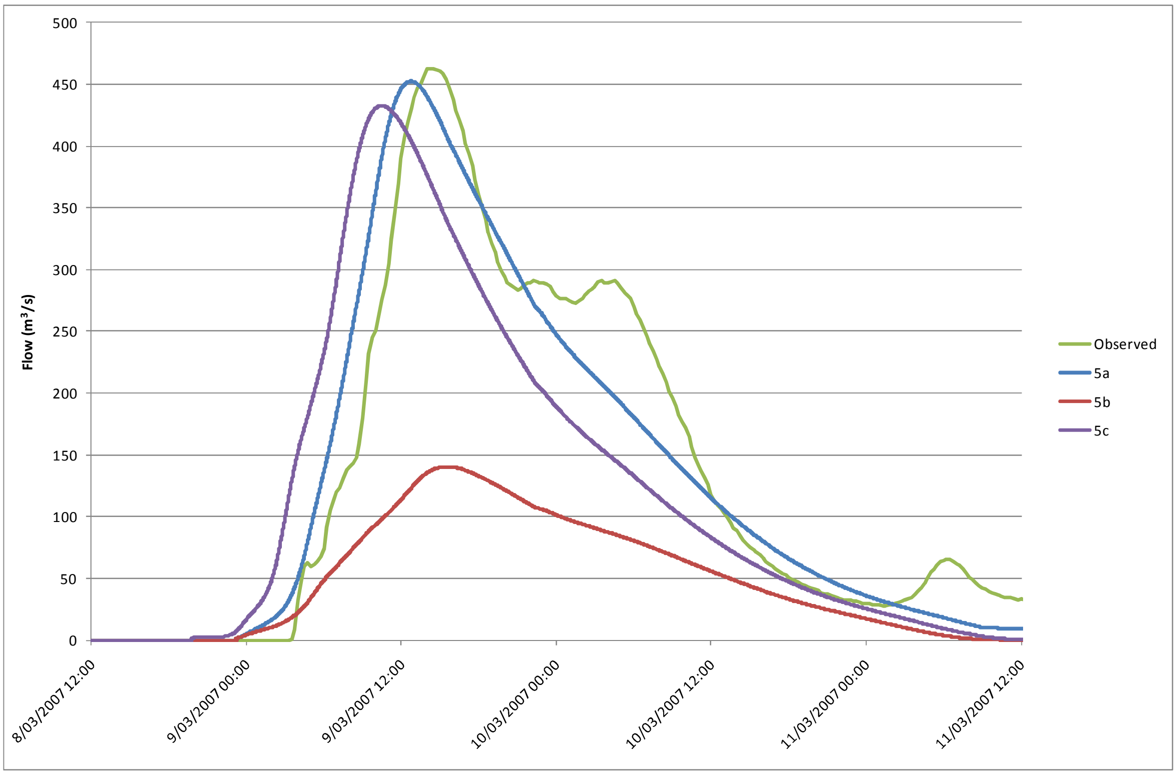

Figure B.5.6: Sensitivity Analysis 2007 Calibration Event

The inclusion of a continuing loss significantly reduced the flows and volumes in comparison to the calibrated parameters. The 2000 event resulted in a decrease of flow volumes by 52% whereas the 2007 event recorded a decrease of flow volumes by 64%. The initial loss values were altered in order to recalibrate the model. By decreasing the initial loss rates, the time to concentration has decrease by 11 hours and 4 hours in comparison to the observed data for the 2000 and 2007 calibration events respectively.

The parameter set "4c" was utilised to determine the critical storm duration for the 100 year ARI storm event. The 100 year critical duration storm event was 12 hours equating to peak flows of 9,580m^3/s on the Turner River and 590m^3/s on the South West Creek respectively at the Port Hedland 2D model extent. The hydrographs for the 100 year storm events is shown in **Appendix B**. **Figure B.5.7** shows the comparison of hydrographs for the parameter set "2a" and "4c" for flows on the Turner River at the Port Hedland 2D model extent. The difference between peak flows and volumes at various locations throughout the catchment are outlined in **Table B.5.5**. The hydrograph and peak flows do not differ significantly for the two calibrated model parameter sets as the peak flows differ by less than 5% at all locations. Given this outcome, the 2a calibrated parameter set is considered to be optimal for the development of the design storm events as a continuing loss parameter does not affect the peak flows from such a large catchment.

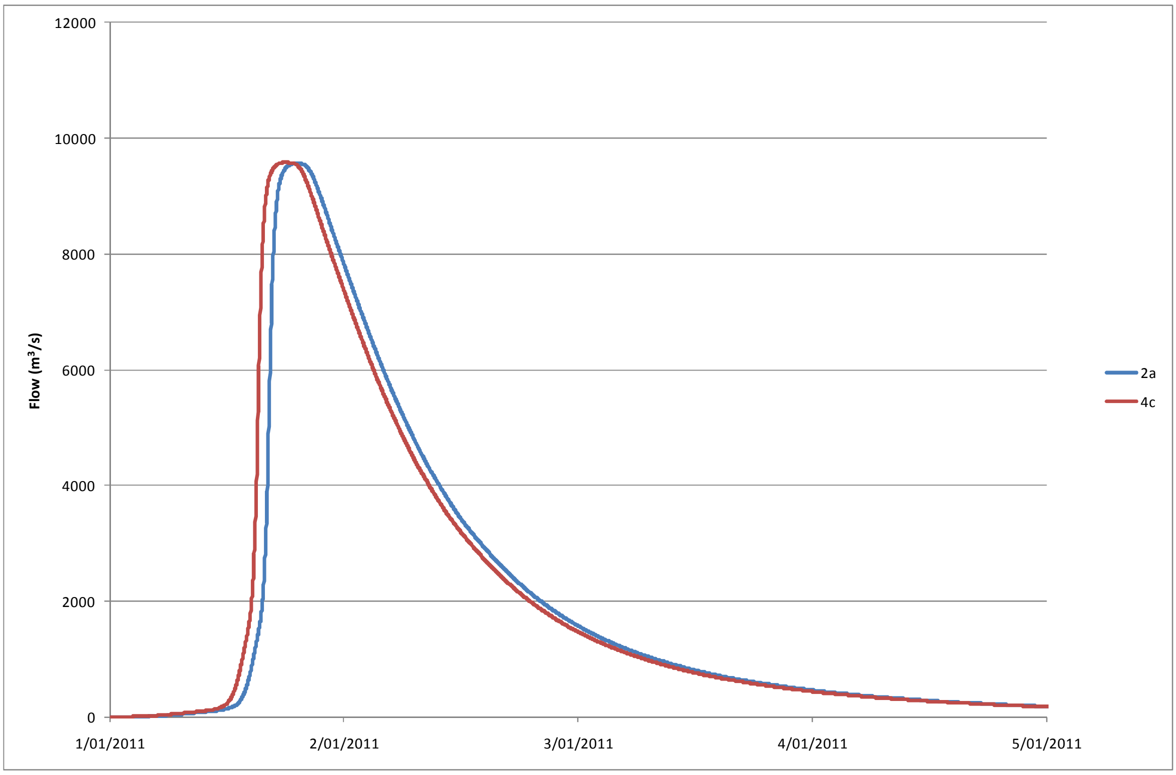

Figure B.5.7: Comparison of 100 year ARI Design Hydrographs on Turner River at Port Hedland 2D model extent

Table B.5.5: Peak Flow Rates for the Continuing Loss Sensitivity Analysis 100 year Design Storm Event

Design Simulation	Pincunnah Station		Turner River at 2D model extent		South West Creek at 2D model extent	
	Peak Flow (m^3/s)	Difference from 2a (%)	Peak Flow (m^3/s)	Difference from 2a (%)	Peak Flow (m^3/s)	Difference from 2a (%)
2a	4153.4	-	9564.4	-	607.2	-
4c	4285.4	3.2	9581.3	0.2	589.8	-2.9

B.5.5 Flood Frequency Analysis

A flood frequency analysis (FFA) was undertaken on the 15 minute flow data recorded at Pincunnah station. The FFA identifies and ranks peak flow events, assigns an associated ARI storm duration and is fitted to a predictive curve. The results of the FFA are shown in **Figure B.5.8**. The 17B curve in the figure is based on a Log Pearson Type III (LPIII) fitted curve. The curve has a low skew of -0.24 which shows that the annual peak events have been well represented by the LPIII distribution. Flow data from Pincunnah station began in 1985, providing 25 years worth of observed flow data. It is recognised that more data would provide a more accurate FFA and that the storm events greater than the 20 year event have been extrapolated from the limited dataset.

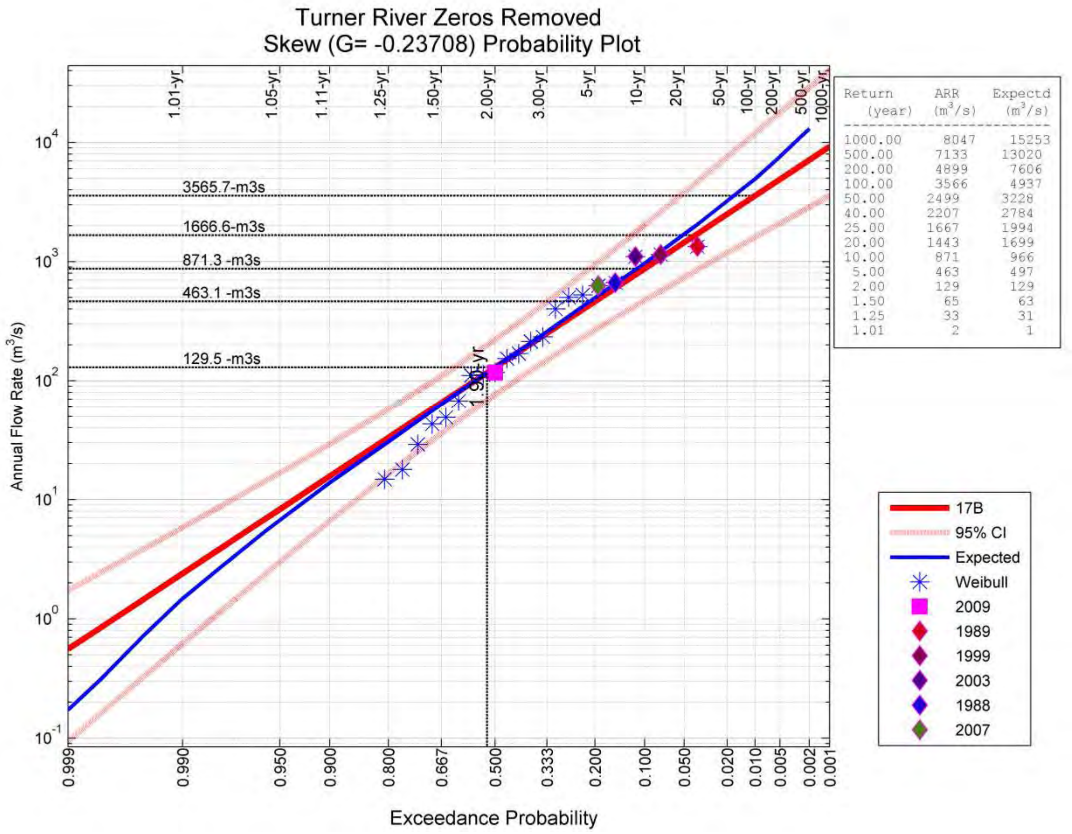

Figure B.5.8: Flood Frequency Analysis on Pincunnah Gauging Station Flow Data

A comparison of the peak flow events from the sensitivity analysis and the FFA using the "2a" parameter value set is shown in **Table B.5.6**. The peak flow rates match relatively well between the FFA and sensitivity analysis (2a) as shown in **Table B.5.6**. The full range of the sensitivity analysis results lie within the 95% confidence bands of the FFA, but it is important to note that the 95% confidence bands are extremely wide for the FFA due to the short data record for the streamflow gauge. Despite this the FFA validates the selected model parameters of the "2a" parameter set with consistent peak flow rates predicted. These results confirm that the "2a" parameter values set produces the most appropriate estimate of the modelling outputs in relation to the observed flow data.

Table B.5.6: Comparison of Peak Flow Rates of the Flood Frequency Analysis and Sensitivity Analysis

Design Simulation	2 year ARI (m^3/s)	100 year ARI (m^3/s)	500 year ARI (m^3/s)
High Estimate (1a)	314	4,444	7,439
Mid Estimate (2a)	135	4,054	7,022
Low Estimate (3b)	12	2,880	5,315
Flood Frequency Analysis	129	3,566	7,133
FFA (95% confidence)	68 – 202	1,638 – 12,670	3,015 – 31,771

Cardno have not undertaken an FFA in relation to the event flow volume over a defined time period using the data from Pincunnah Gauging Station. This type of analysis is not a standard approach outlined in The Institution of Engineers (2001) and peak event flows as well as volumes are already compared at the Pincunnah Gauging Station as part of the specific event calibration process - see **Section B.4.3**.

B.5.6 ARI Design Storms

A multi storm analysis was undertaken to identify the critical durations for the 2, 10, 100, 200 and 500 year ARI storm events. The "2a" parameter value set was used for the modelling. Design hydrographs were generated for the Turner River and South West Creek at the Port Hedland 2D hydraulic modelling extent and the river entering the Shellborough 2D hydraulic modelling extents (see **Figure B.2** and **Figure B.3**). Design hydrographs were also generated for a generic smaller catchment within the Port Hedland 2D hydraulic modelling region which identified a much shorter critical storm duration due to the reduce size of the catchment. All design hydrographs are shown in **Appendix B.2**. The critical storm durations and peak flow rates are shown in **Section B.7**.

B.5.7 Rational and Index Flood Method Comparisons

The Rational and Index Flood Methods (The Institution of Engineers, 2001) were used to compare the modelled results shown above in **Table B.7.1**. The Western Australian Pilbara region calculations and equations were used to compare against the modelled results. Only the 2 year and 10 year ARI storm events were calculated for the Rational and Index Flood Methods as there are no frequency factors greater than the 50 year event calculated for the Pilbara region. The Rational and Flood Index Method equations are shown in **Equations B.1**, **B.2** and **B.3**.

Rational Method

$$C_2 = 3.07 \times 10^{-1} \times L^{-0.2}$$ **Equation B.1**

$$Q_y = 0.278 \times C_2 \times y_y \times I_{t_c,y} \times A$$ **Equation B.2**

Where:
A = Drainage area (km^2)
L = Mainstream length (km)
C_2 = Runoff coefficient for 2 year ARI storm event
γ_y = Frequency factor (C_y / C_2)
$I_{tc,y}$ = Rainfall intensity for critical storm event and duration

Index Flood Method

$$Q_y = y_y \times 6.73 \times 10^{-4} \times A^{0.72} \times P^{1.51}$$ **Equation B.3**

Where:
y_y = Frequency factor (C_y / C_2)
A = Drainage area (km^2)
P = Average annual precipitation (mm)

The parameters and results for the Rational and Index Flood Method calculations at Pincunnah station are shown in **Table B.5.7** with the comparison of results of the three methods in **Table B.5.8**.

Table B.5.7: Rational and Index Method Calculation Parameters at Pincunnah Gauging Station

Rational Method					
ARI Storm Event	**A (km^2)**	**L (km)**	**y_y**	**C_2**	**Q (m^3/s)**
2 year 24 hour	958.65	50	1	0.14	117
10 year 24 hour	958.65	50	2.21	0.14	555
Index Flood Method					
ARI Storm Event	**A (km^2)**	**L (km)**	**P (mm)**	**y_y**	**Q (m^3/s)**
2 year 24 hour	958.65	50	300	0.48	249
10 year 24 hour	958.65	50	300	1.94	1,007

Table B.5.8: Comparison of Rational and Flood Index Methods to Modelled Flow Rates (m^3/s)

ARI Storm Event	Rational Method	Flood Index Method	XPSWMM Model
2 year 36 hour	117	249	134
10 year 30 hour	555	1,007	1,192

The flow rates from the XPSWMM model are comparable to the two calculation methods. The XPSWMM calibrated model is therefore considered to be an accurate model to use for the modelling of ARI storm events.

B.6 CLIMATE CHANGE SCENARIO INVESTIGATIONS

An assessment of the peak flows and discharge volumes was undertaken for the climate change scenarios for 2060 and 2110. Projections for changes in rainfall amounts in relation to cyclonic events range between +3% to +37%. The typical projected changes for rainfall at the end of the twenty-first century are about +20% within 100km of the storm centre (Knutson *et al.*, 2010). This is consistent with general guidance provided by the CSIRO (CSIRO, 2007). The design rainfall patterns for the 2060 and 2110 climate scenarios were maintained but the rainfall totals were adjusted to account for a 10% and 20% increase in rainfall across the two scenarios, respectively. **Table B.6.1** shows the changes in peak flows and discharge volumes in comparison to predicted design events.

Table B.6.1: Climate Change Scenario Comparison of Peak Flow Rates and Discharge Volumes

Location	Peak Flow Rate (m^3/s)			Discharge Volume (GL)			Peak Flow Difference (%)		Volume Difference (%)	
	100yr 12hr	2060	2110	100yr 12hr	2060	2110	2060	2110	2060	2110
Pincunnah Station	4,059	4,680	5,318	198	225	251	15	31	14	27
Port Hedland 2D Model Extent (Turner River)	9,485	9,919	10,363	656	730	795	5	9	11	21

The change in rainfall intensity show increases of peak flows ranging between 5% and 31% and discharge volumes of 11% and 27% for the 2060 and 2110 respectively in comparison to the current ARI design storms. **Figure B.6.2** shows the hydrographs of the climate change scenarios at Pincunnah gauging station and on the Turner River at the extent of the 2D hydraulic model boundary.

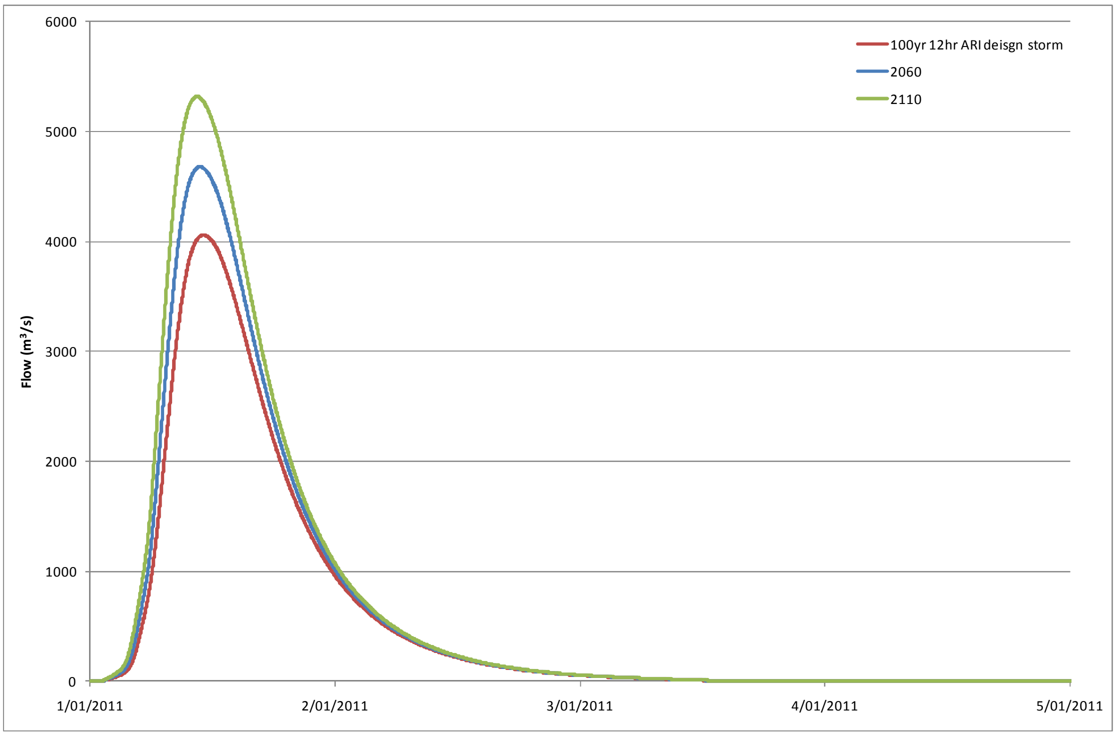

Figure B.6.1: Climate Change Scenario Hydrographs Reported at Pincunnah Gauging Station (Turner River)

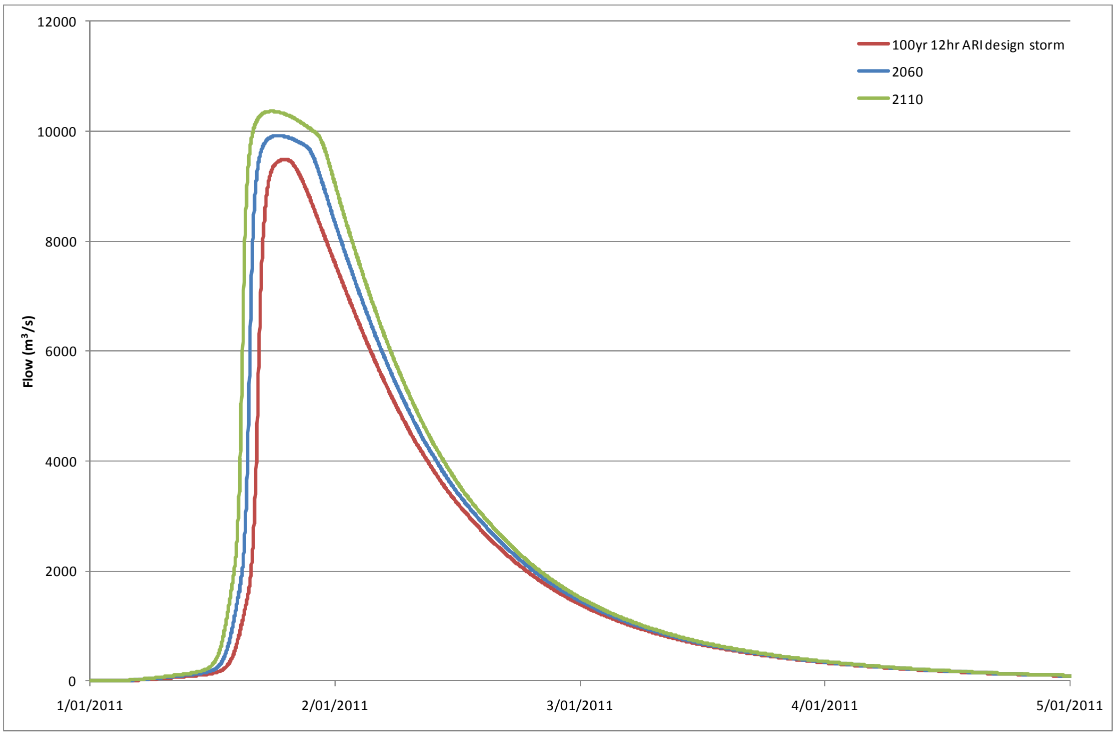

Figure B.6.2: Climate Change Scenario Hydrographs on at Turner River Reported at the Port Hedland 2D Model Extent

B.7 DESIGN EVENTS – PEAK FLOWS AND CITICAL DURATIONS

The peak flows and discharge volumes for the critical duration storm events at the 2D hydraulic model extents for Port Hedland and Shellborough are shown in **Table B.7.1**, **Table B.7.3** and **Table B.7.3**.

Table B.7.1: Design Storm Summary Results of Turner River Entering the Port Hedland 2D Model Extent

ARI Storm Event	2 year	10 year	100 year	200 year	500 year
Critical Storm Duration (hour)	36	36	12	18	12
Peak Flow (m^3/s)	263	2,793	9,485	10,365	11,561
Discharge Volume (GL)	31.2	314.6	655.9	913.4	940.4

Table B.7.2: Design Storm Summary Results of South West Creek Entering the Port Hedland 2D Model Extent

ARI Storm Event	2 year	10 year	100 year	200 year	500 year
Critical Storm Duration (hour)	36	36	24	18	18
Peak Flow (m^3/s)	8	172	648	862	1,197
Discharge Volume (GL)	0.6	25.6	75.3	86.8	114.0

Table B.7.3: Design Storm Summary Results Entering the Shellborough 2D Model Extent

ARI Storm Event	2 year	10 year	100 year	200 year	500 year
Critical Storm Duration (hour)	36	48	36	30	30
Peak Flow (m^3/s)	16	36	114	159	231
Discharge Volume (GL)	3.1	9.5	24.7	28.9	38.2

The hydrographs of the critical storm events for all catchments entering into and residing within the 2D model extent for both Port Hedland and Shellborough were used as inputs for the 2D hydraulic model. The critical storm events identified for the locations above all have long storm durations and hence, due to the lower intensity, the peak of the storm events occur well after the cyclone has passed. In order to determine the effectiveness of the 2D hydraulic model for short duration high intensity storm events, hydrograph data for the 1.5 hour duration of all modelled ARI storm events were used as the inputs. The peak flows and discharge volumes for a fully urban catchment (147.6ha) within the Port Hedland townsite are shown in **Table B.7.4**.

Appendix C (Main Report) presents a detailed summary of catchment inflows applied to the hydraulic model.

Table B.7.4: Design Storm Summary Results for Urban Catchment within the Port Hedland 2D Model Region

ARI Storm Event	2 year	10 year	100 year	200 year	500 year
Critical Storm Duration (hour)	1.5	1.5	1.5	1.5	1.5
Peak Flow (m^3/s)	20	54	88	103	126
Discharge Volume (ML)	43.0	96.7	192.4	226.0	273.8

B.8 REFERENCES

CSIRO (2007). “Climate Change in Australia – Technical Report.”. Prepared by CSIRO and Bureau of Meteorology. ISBN 9781921232947.

The Institute of Engineers, Australia, 1987, *Australia Rainfall and Runoff: A Guide to Flood Estimation*, Volume 2, Ed. R.P. Canterford, The Institute of Engineers, Australia, Barton, ACT.

The Institute of Engineers, Australia, 2001, *Australia Rainfall and Runoff: A Guide to Flood Estimation*, Volume 1, Ed. D.H. Pilgrim, The Institute of Engineers, Australia, Barton, ACT.

Knutson, T.R., McBride, J.L., Chan, J., Emmanuel, K., Holland, G., Landsea, C., Held, I., Kossin, J.P., Srivastava, A.K. & Sugi, M., 2010, Tropical cyclones and climate change, *Natural Geoscience*, Vol. 3, pp. 157-163.

Rodriguez, E., Morris, C.S., Belz, J.E., Chapin, E.C., Martin, J.M., Daffer, W. & Hensley, S., 2005, *An assessment of the SRTM topographic products*, Technical Report JPL D-31639, Jet Propulsion Laboratory, Pasadena, California, 143 pp.

Smith, B. & Sandwell, D., 2003, Accuracy and resolution of shuttle radar topography mission data, *Geophys. Res. Lett.*, 30, doi: :10.1029/2002GL016643.

Figures

DATE	No.	ACTIVITY - REVISION DESCRIPTION	DES	DRN	CHK'D	APPD	DATE	No.	ACTIVITY - REVISION DESCRIPTION	DES	DRN	CHK'D	APPD

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Scale: 1:400,000

0 5 10 20 30 40 50 60 70 80 Kilometres

PROJECT: Port Hedland Vulnerability Study
Hydrological Modelling Technical Report

DRAWING TITLE: FIGURE B1 : Site Locality

PRINCIPAL: Landcorp

Project Number: LJ15014

Original: A2

Drawing Number: SK01

Revision

Designed: SLC

Checked

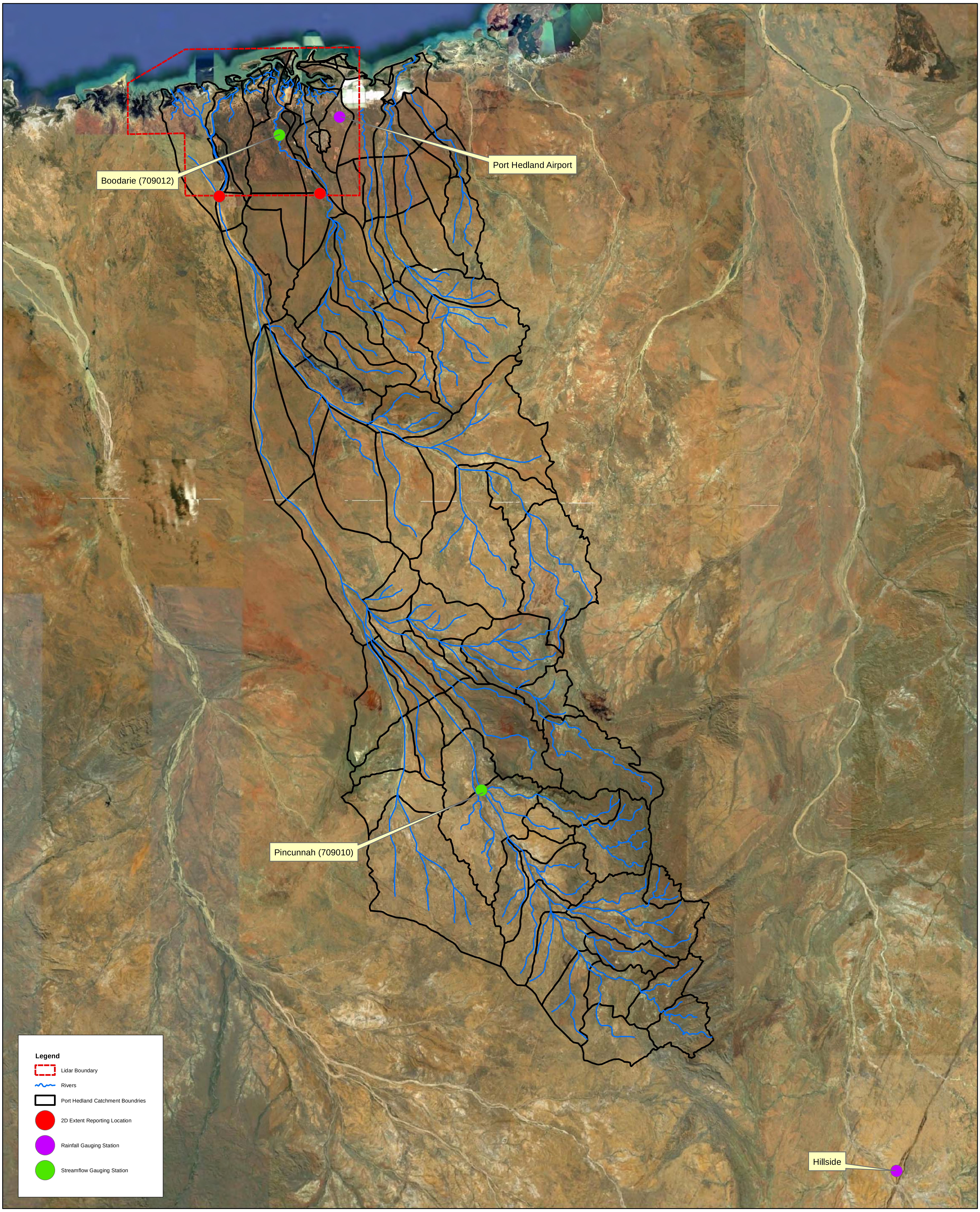
Drawn: MGW

Approved

Local Authority: Various

Sheet 1 of 1

Date: 10/03/2011



DATE	No.	ACTIVITY - REVISION DESCRIPTION	DES	DRN	CHK'D	APPD	DATE	No.	ACTIVITY - REVISION DESCRIPTION	DES	DRN	CHK'D	APPD

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Scale: 1:350,000

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PROJECT: Port Hedland Vulnerability Study
Hydrological Modelling Technical Report

DRAWING TITLE: FIGURE B2 : Port Hedland Catchment

PRINCIPAL: Landcorp

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Project Number: LJ15014
Original: A2
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Revision
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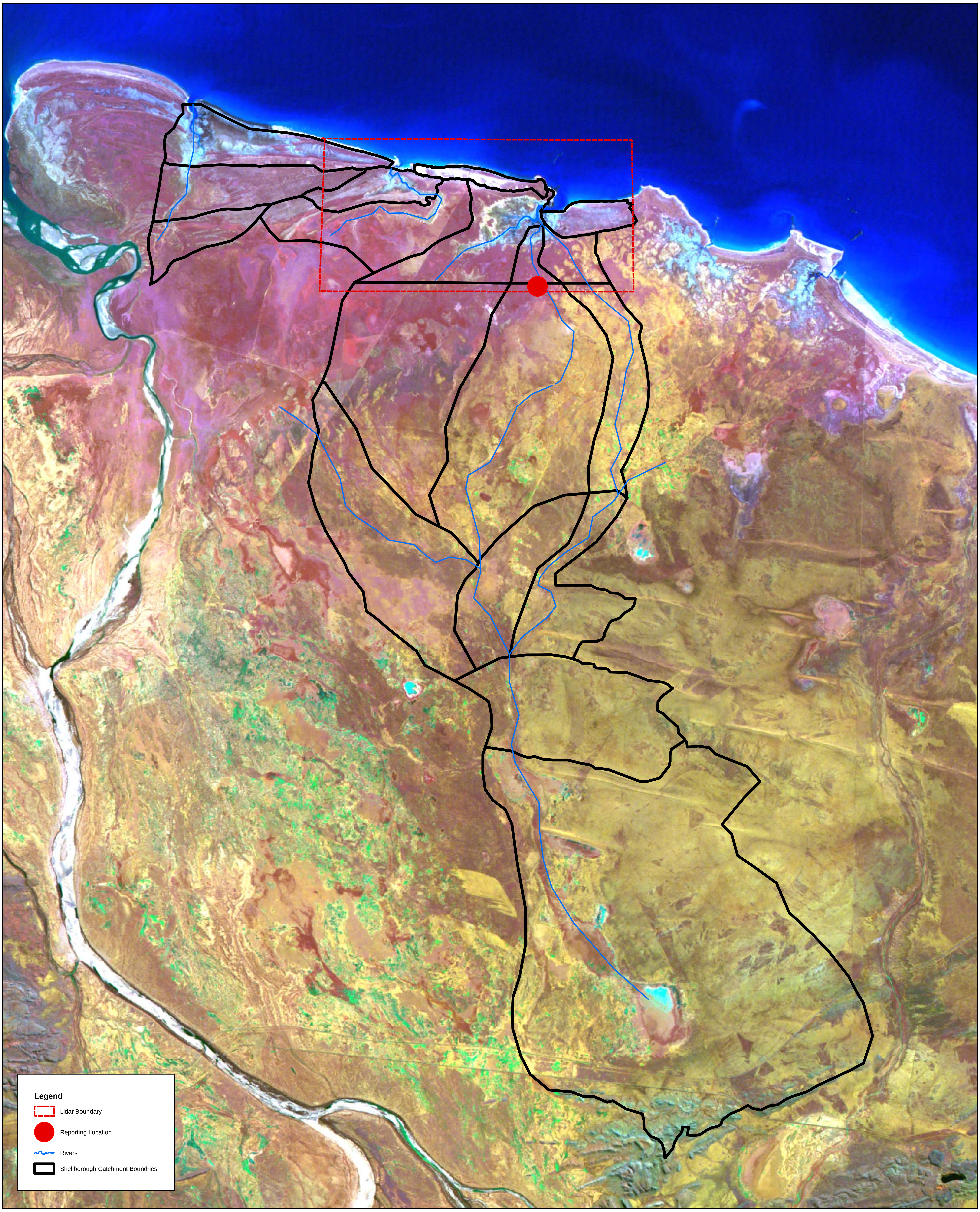
Local Authority: Various
Sheet 1 of 1
Date: 10/03/2011



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Scale: 1:100,000

0 2.5 5 10 15 20 Kilometres

PROJECT: Port Hedland Vulnerability Study
Hydrological Modelling Technical Report

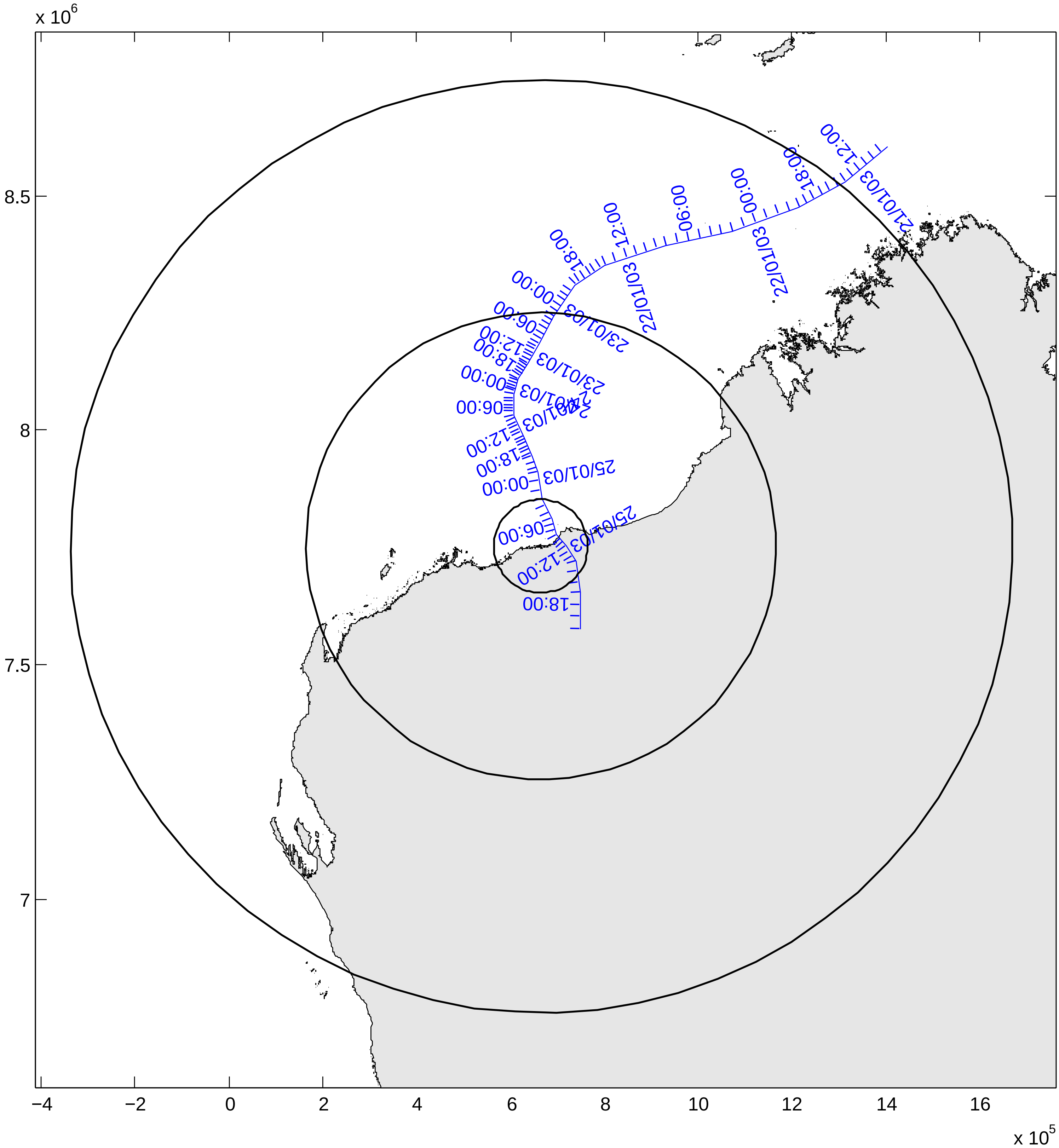
DRAWING TITLE: FIGURE B3 : Shellborough Catchment

PRINCIPAL: Landcorp

Project Number: LJ15014
Original: A2
Drawing Number: SK03
Revision
Designed: SLC
Checked
Drawn: MGW
Approved
Local Authority: Various
Sheet 1 of 1
Date: 10/03/2011

Appendix B.1

Cyclone Tracks for Calibration Events



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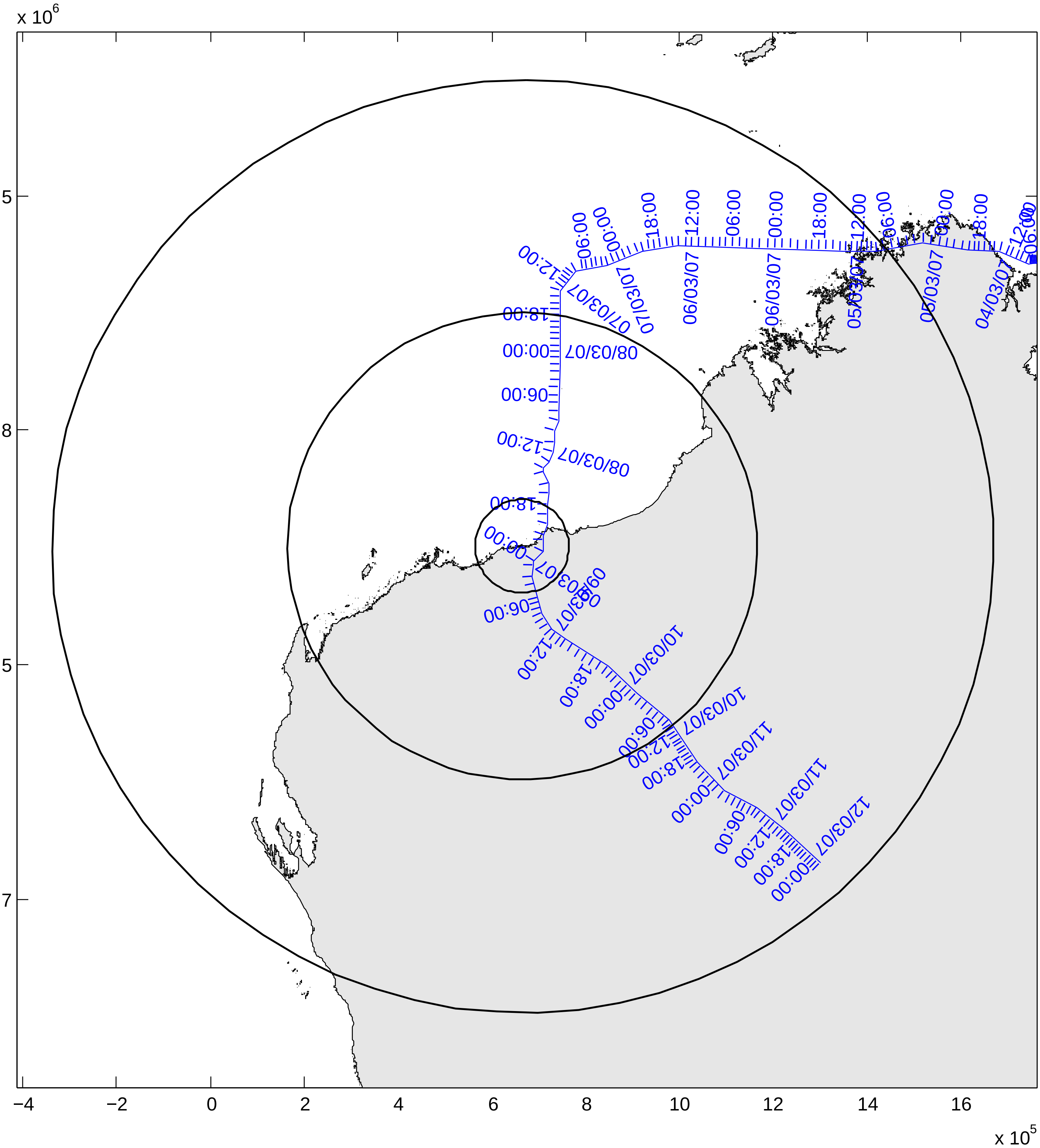

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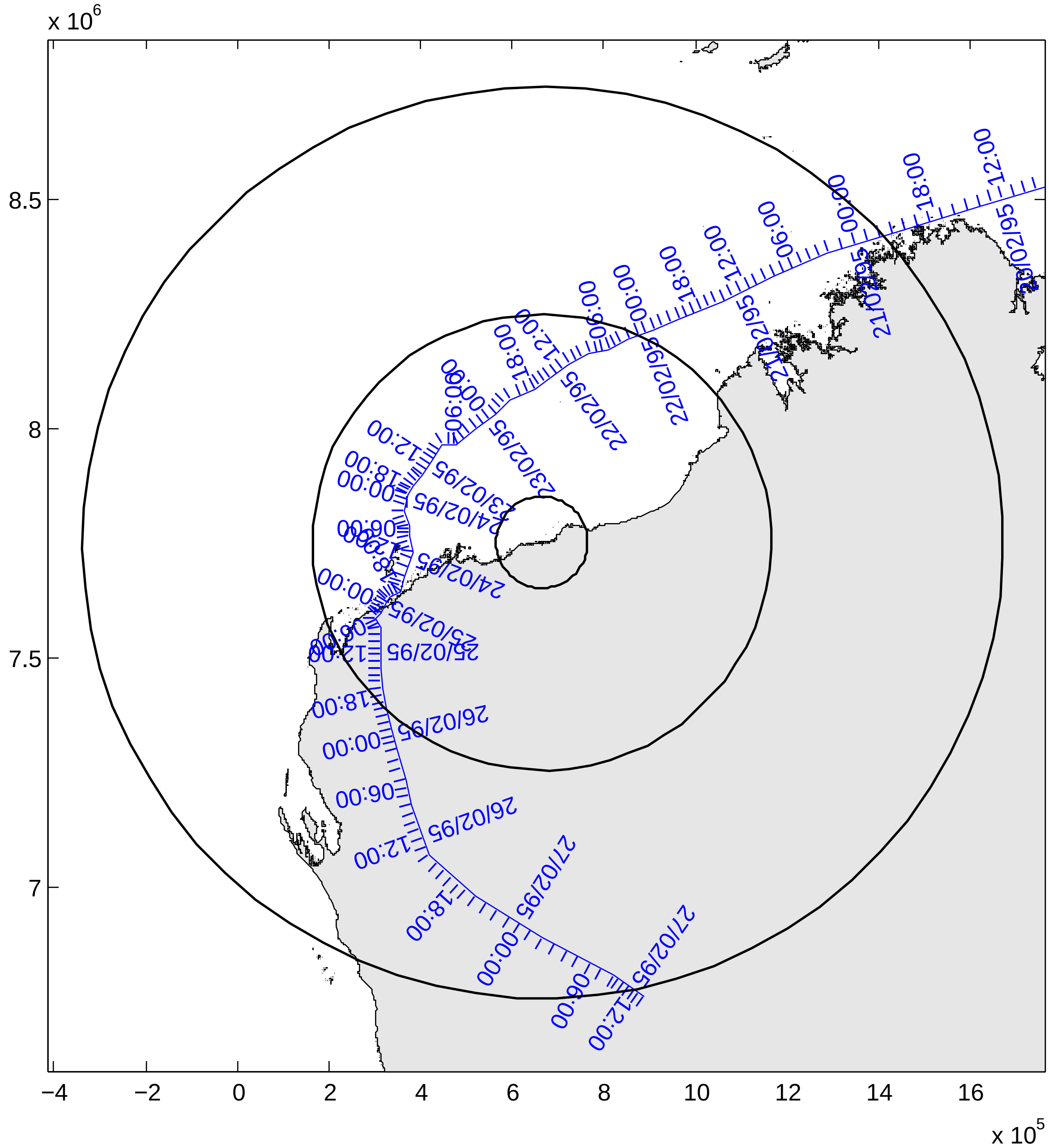

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Appendix B.2

Design Hydrographs

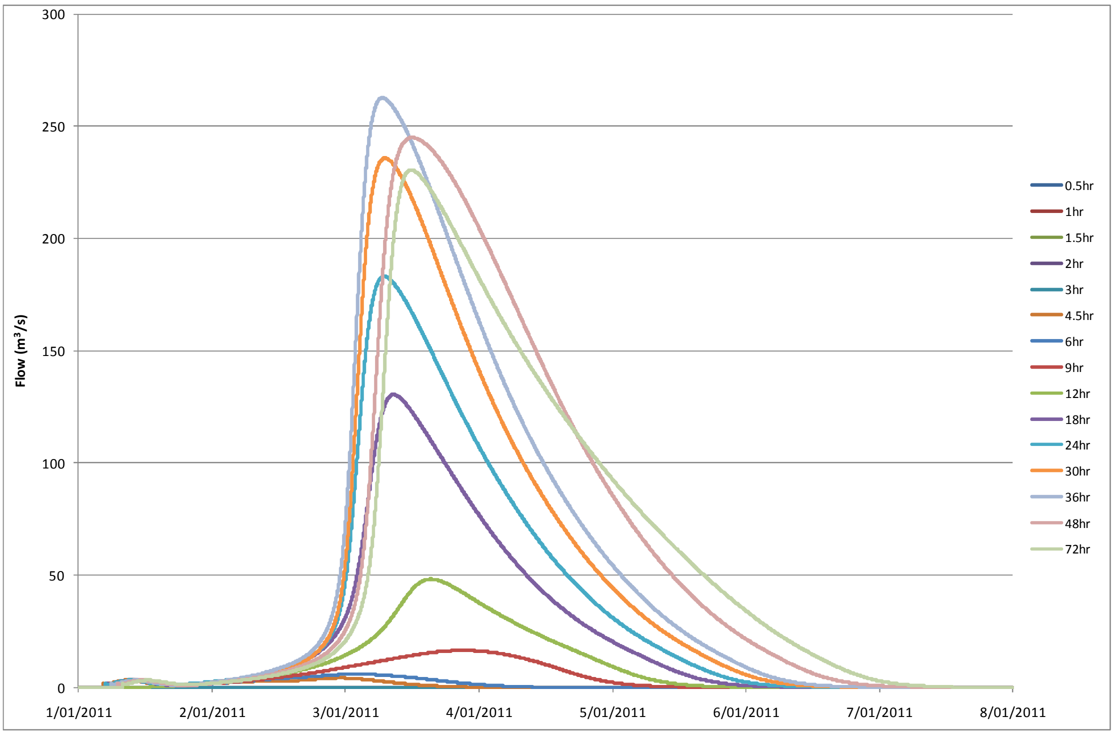

Figure B.B.1: 2 year ARI Design Hydrographs on Turner River at Port Hedland 2D Model Extent

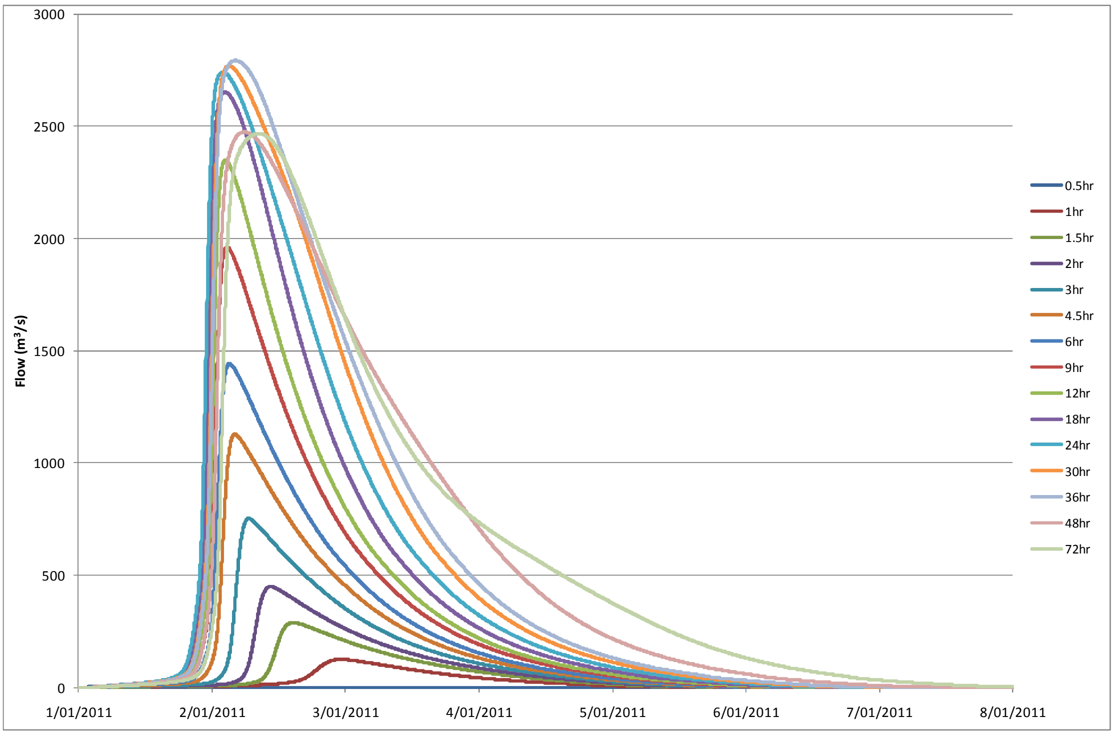

Figure B.B.2: 10 year ARI Design Hydrographs on Turner River at Port Hedland 2D Model Extent

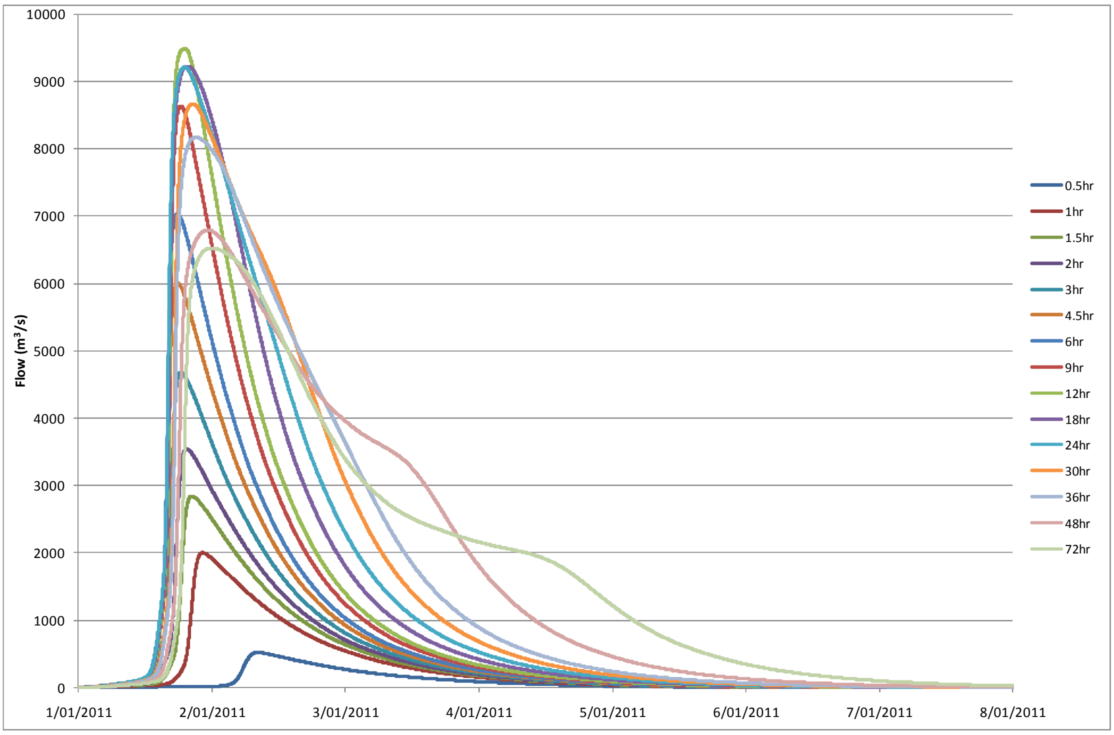

Figure B.B.3: 100 year ARI Design Hydrographs on Turner River at Port Hedland 2D Model Extent

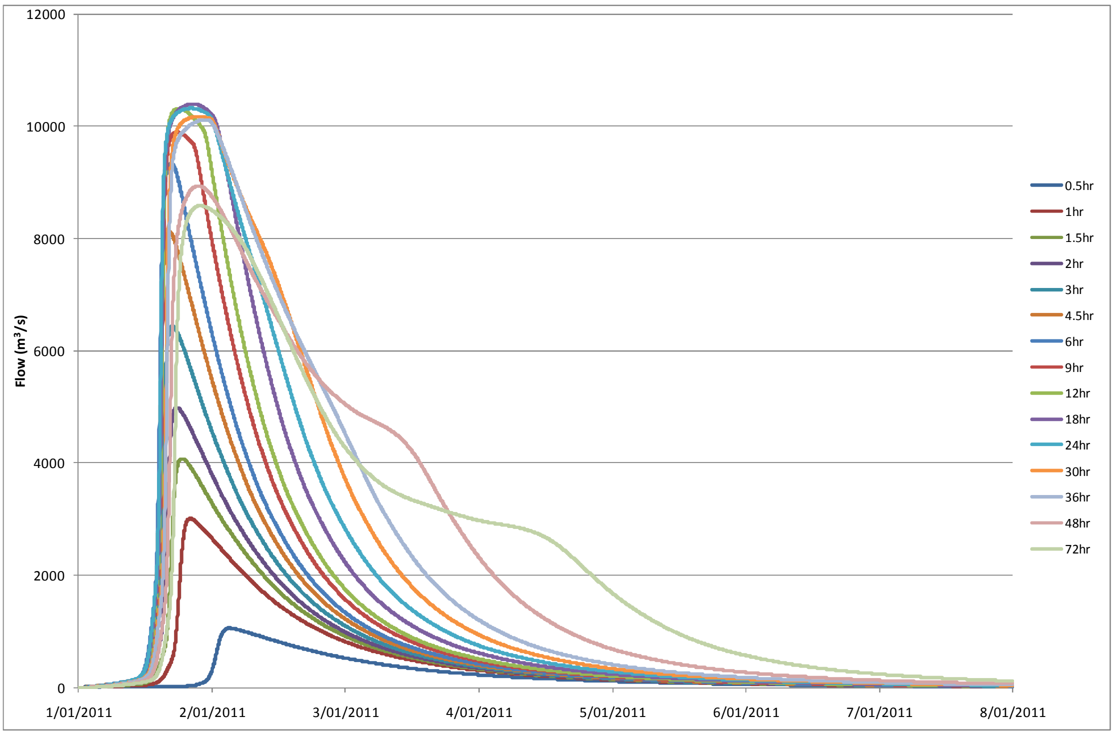

Figure B.B.4: 200 year ARI Design Hydrographs on Turner River at Port Hedland 2D Model Extent

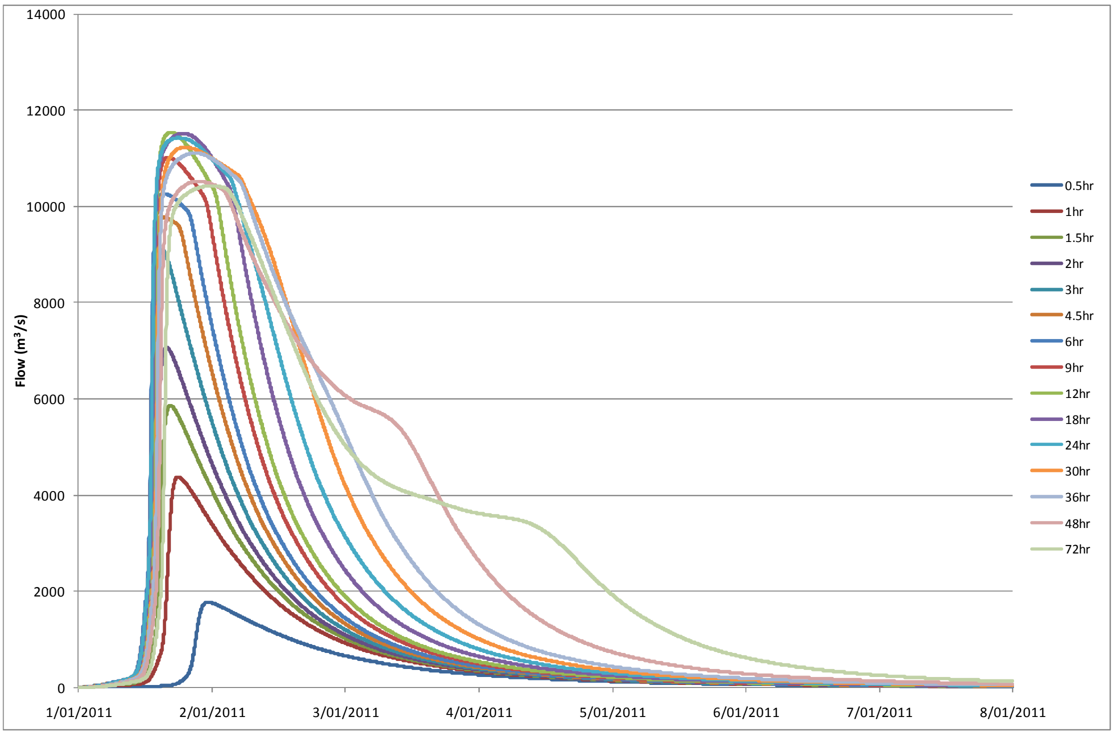

Figure B.B.5: 500 year ARI Design Hydrographs on Turner River at Port Hedland 2D Model Extent

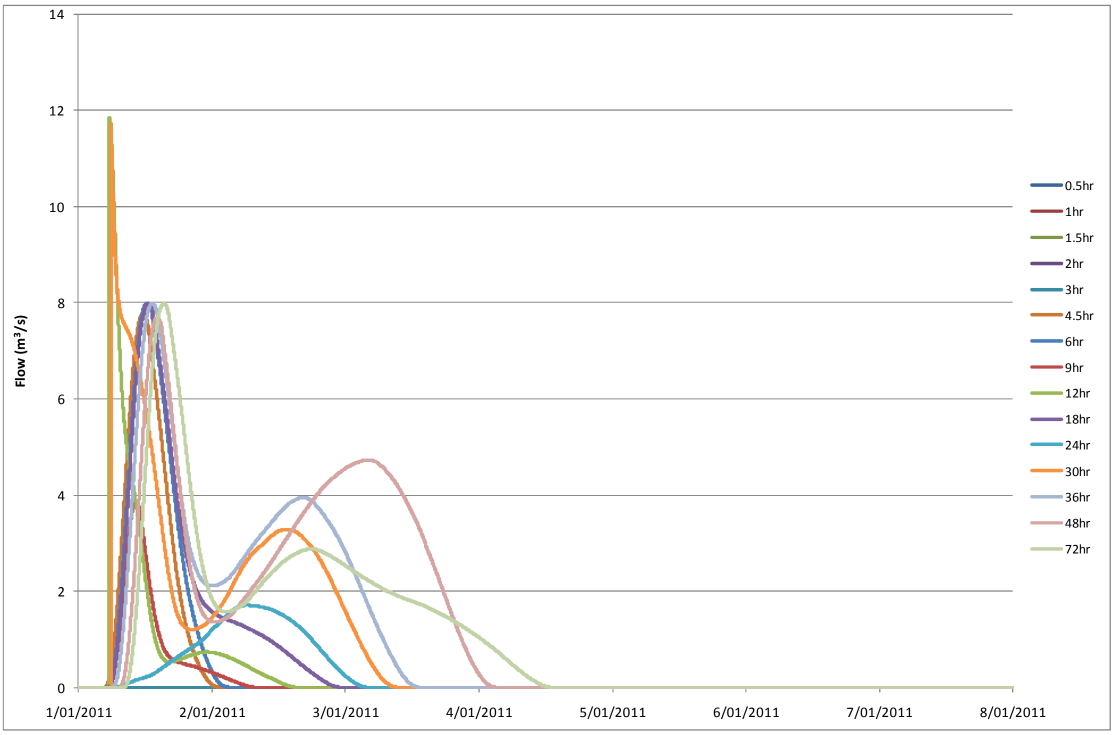

Figure B.B.6: 2 year ARI Design Hydrographs on South West Creek at Port Hedland 2D Model Extent

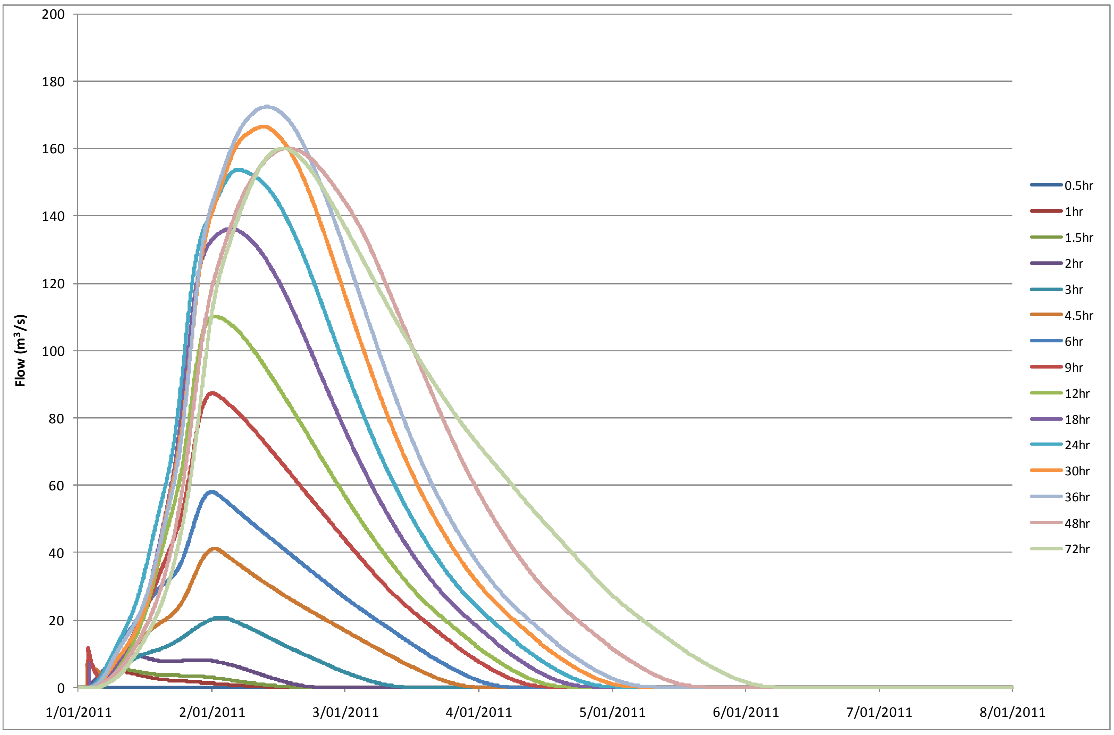

Figure B.B.7: 10 year ARI Design Hydrographs on South West Creek at Port Hedland 2D Model Extent

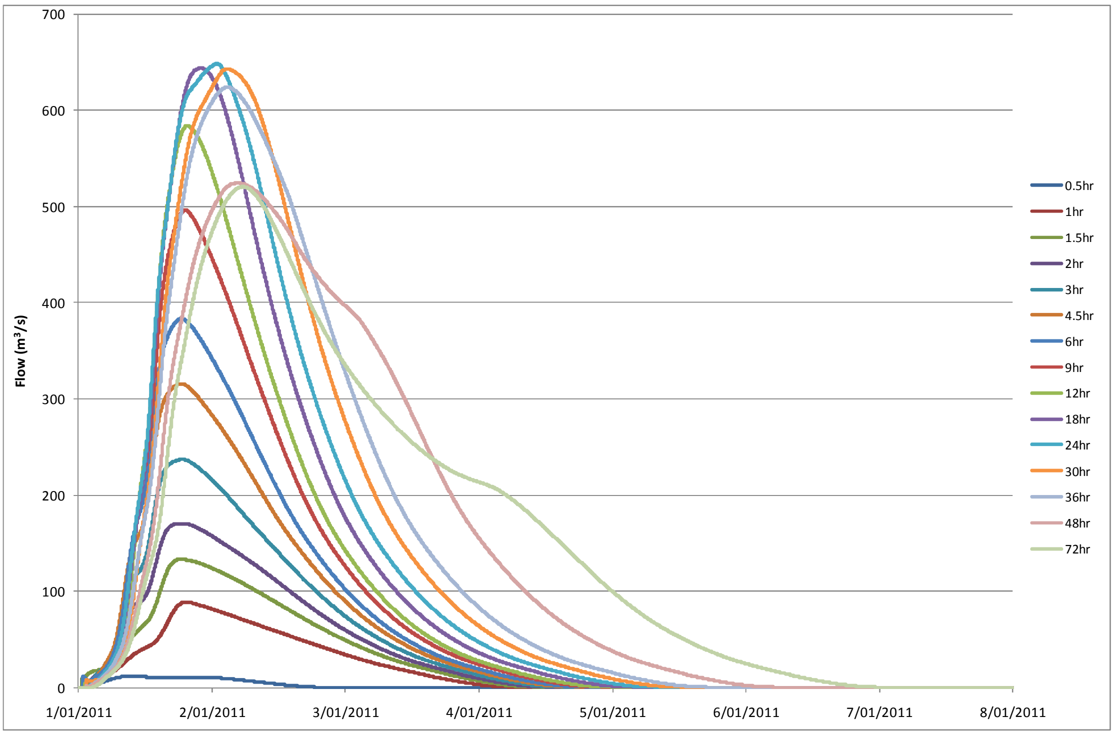

Figure B.B.8: 100 year ARI Design Hydrographs on South West Creek at Port Hedland 2D Model Extent

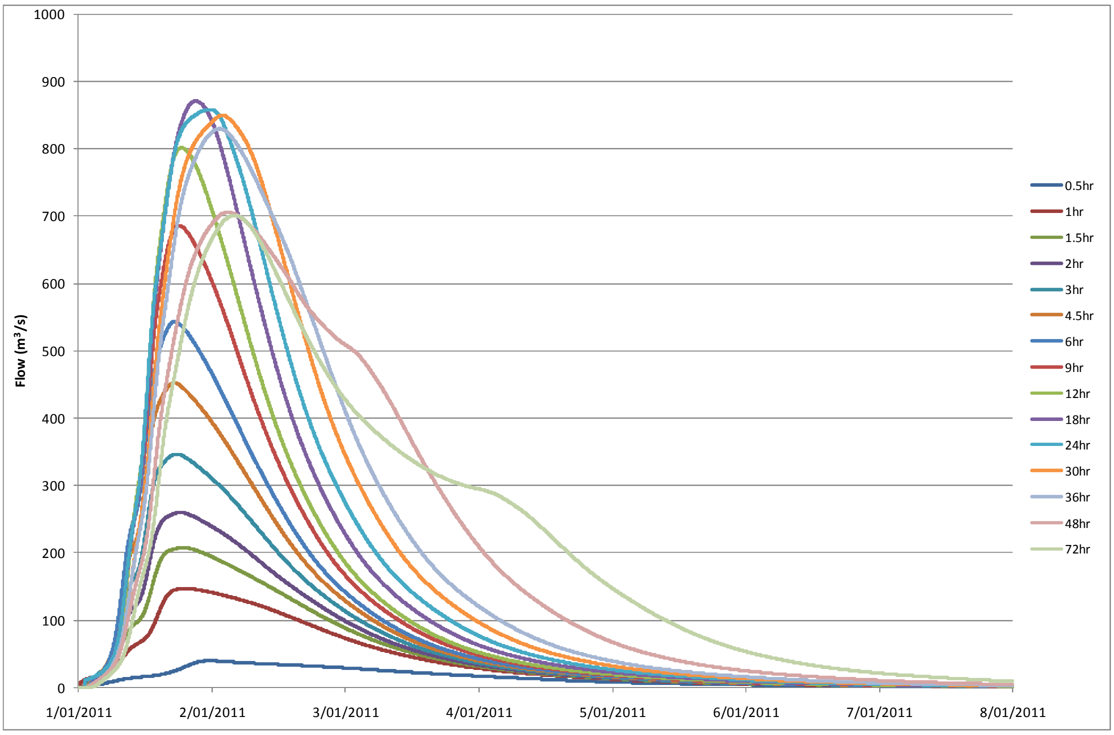

Figure B.B.9: 200 year ARI Design Hydrographs on South West Creek at Port Hedland 2D Model Extent

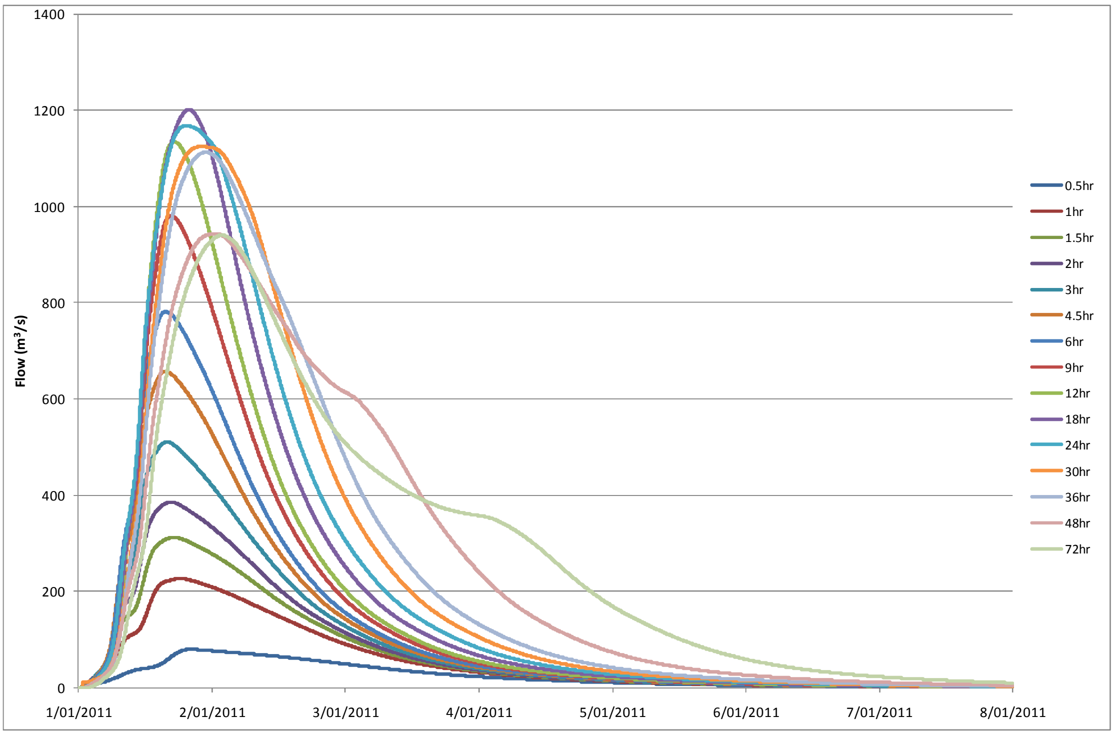

Figure B.B.10: 500 year ARI Design Hydrographs on South West Creek at Port Hedland 2D Model Extent

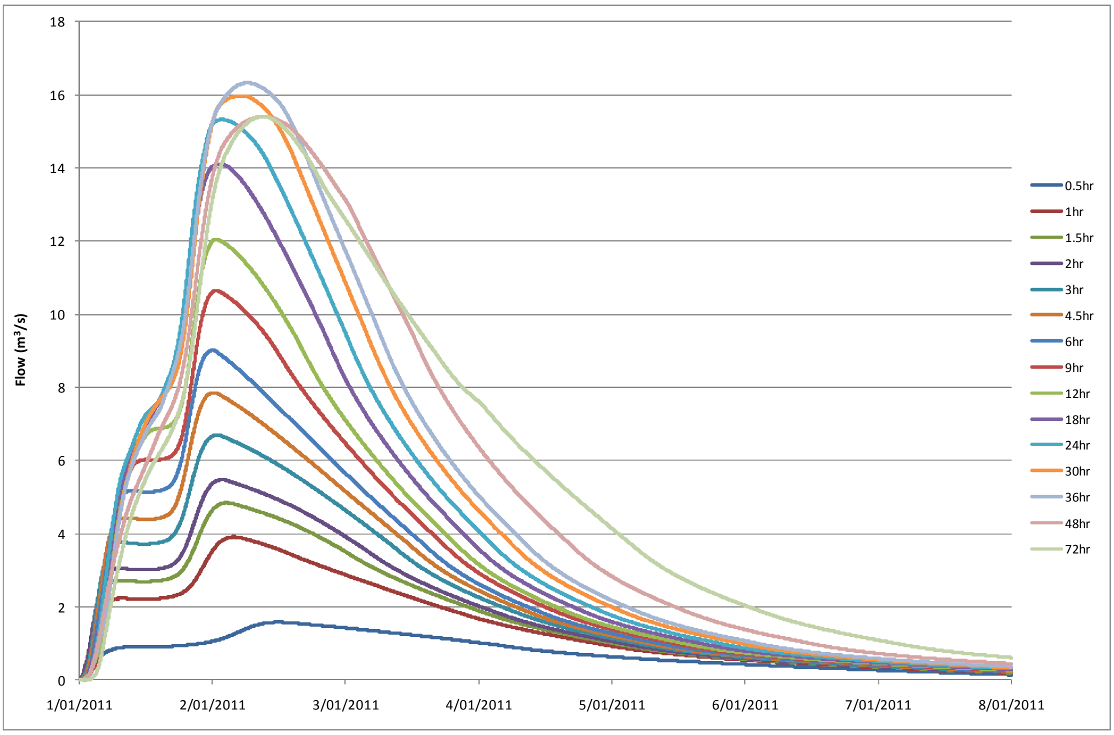

Figure B.B.11: 2 year ARI Design Hydrographs at Shellborough 2D Model Extent

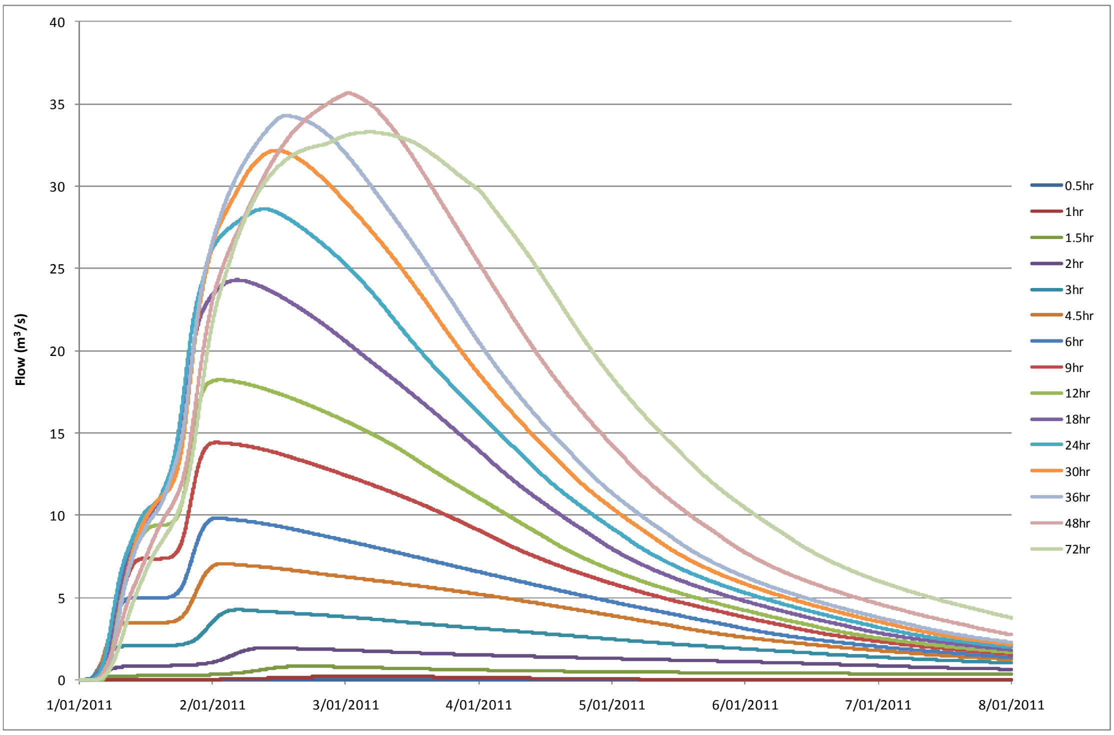

Figure B.B.12: 10 year ARI Design Hydrographs at Shellborough 2D Model Extent

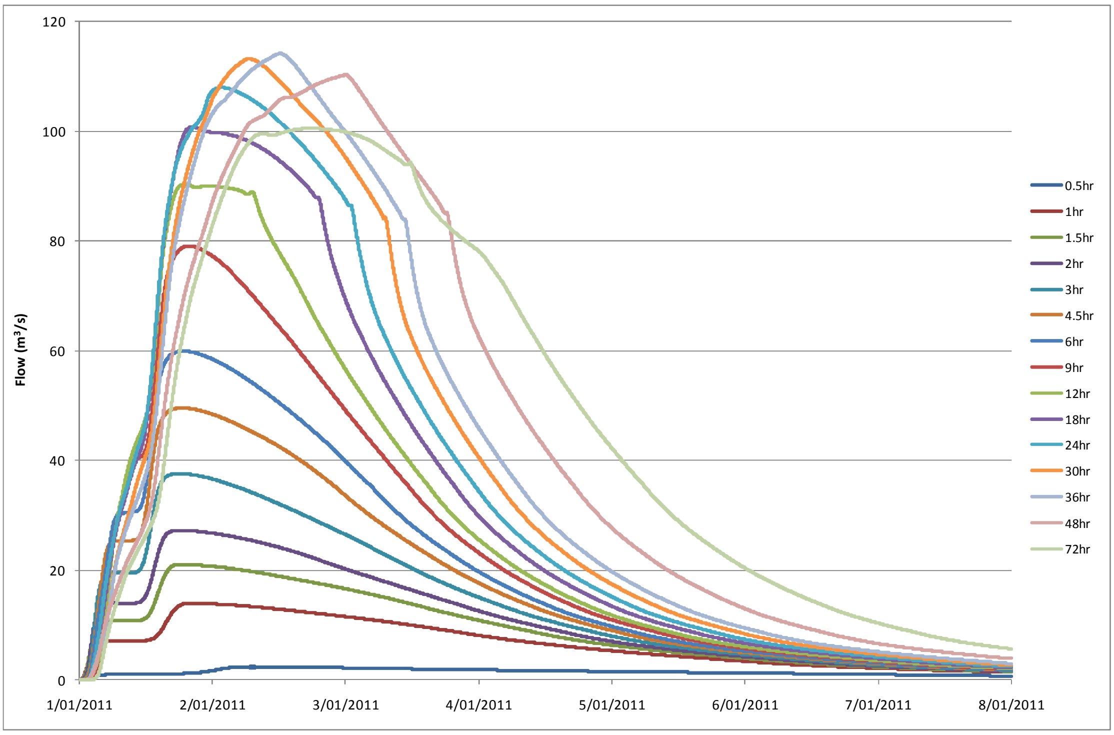

Figure B.B.13: 100 year ARI Design Hydrographs at Shellborough 2D Model Extent

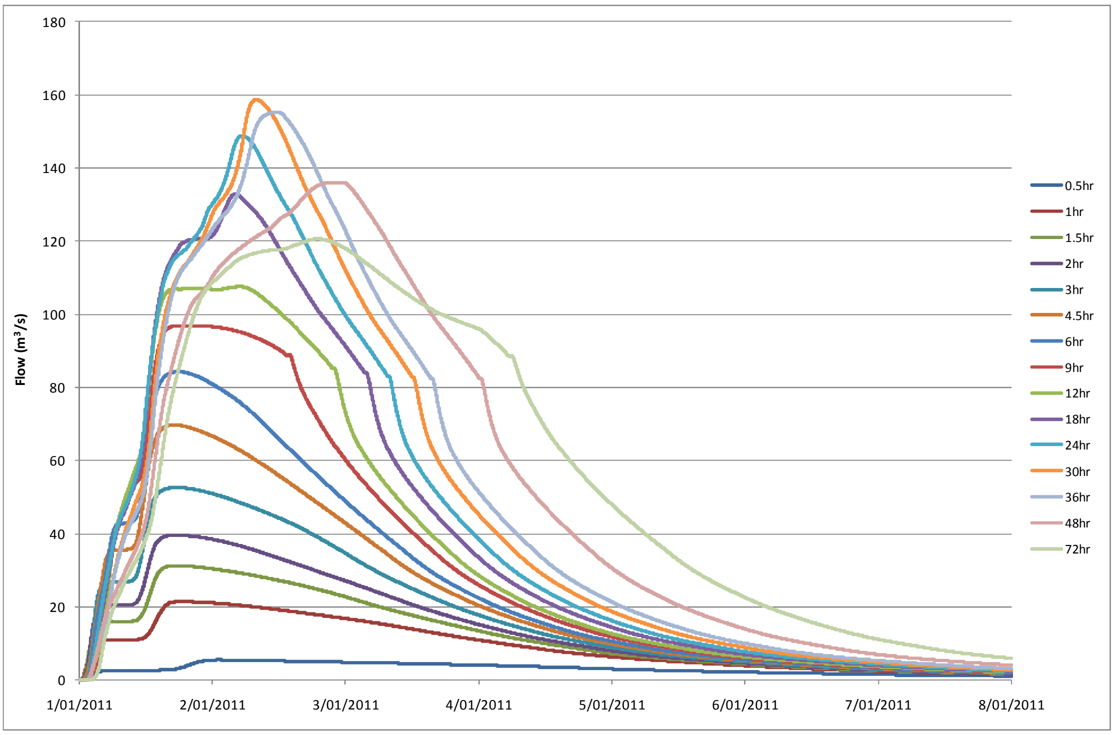

Figure B.B.14: 200 year ARI Design Hydrographs at Shellborough 2D Model Extent

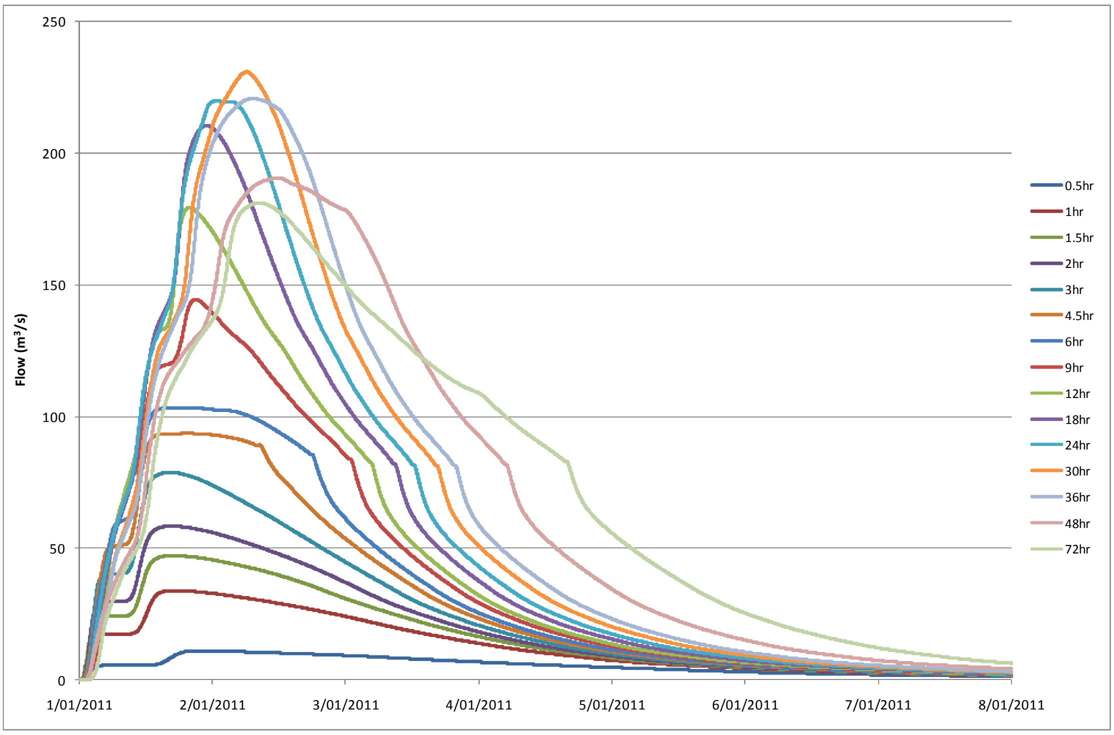

Figure B.B.15: 500 year ARI Design Hydrographs at Shellborough 2D Model Extent

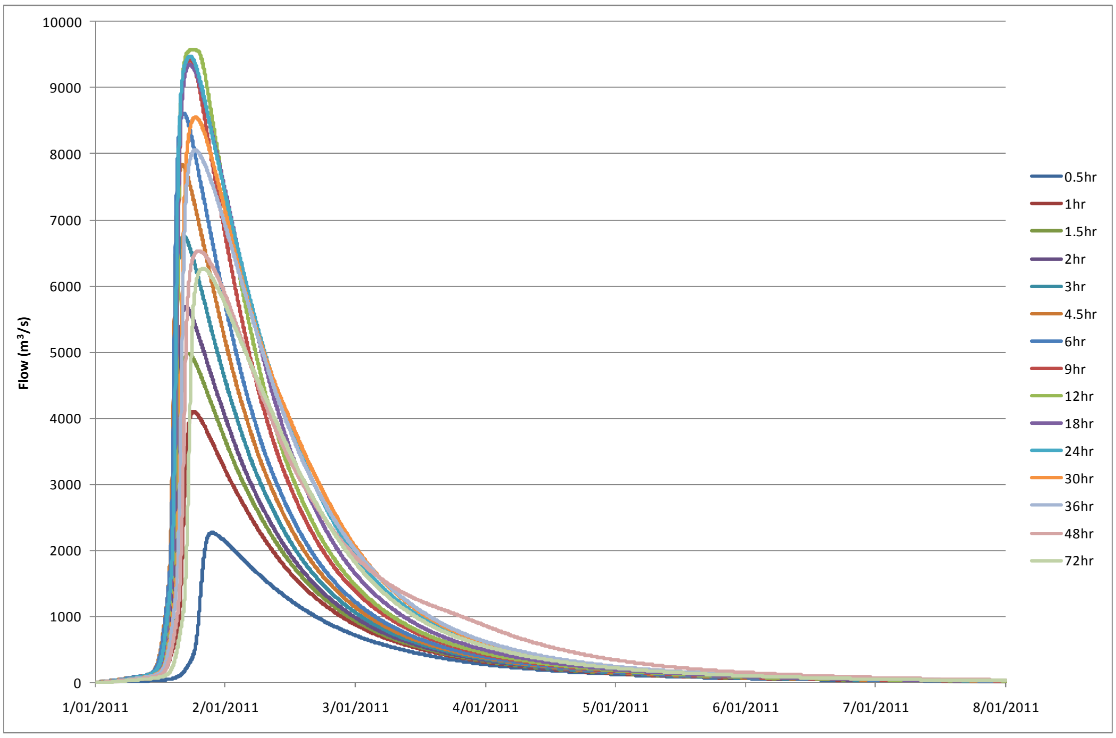

Figure B.B.16: 100 year ARI Design Hydrographs on Turner River at Port Hedland 2D Model Extent with Continuing Loss Parameters

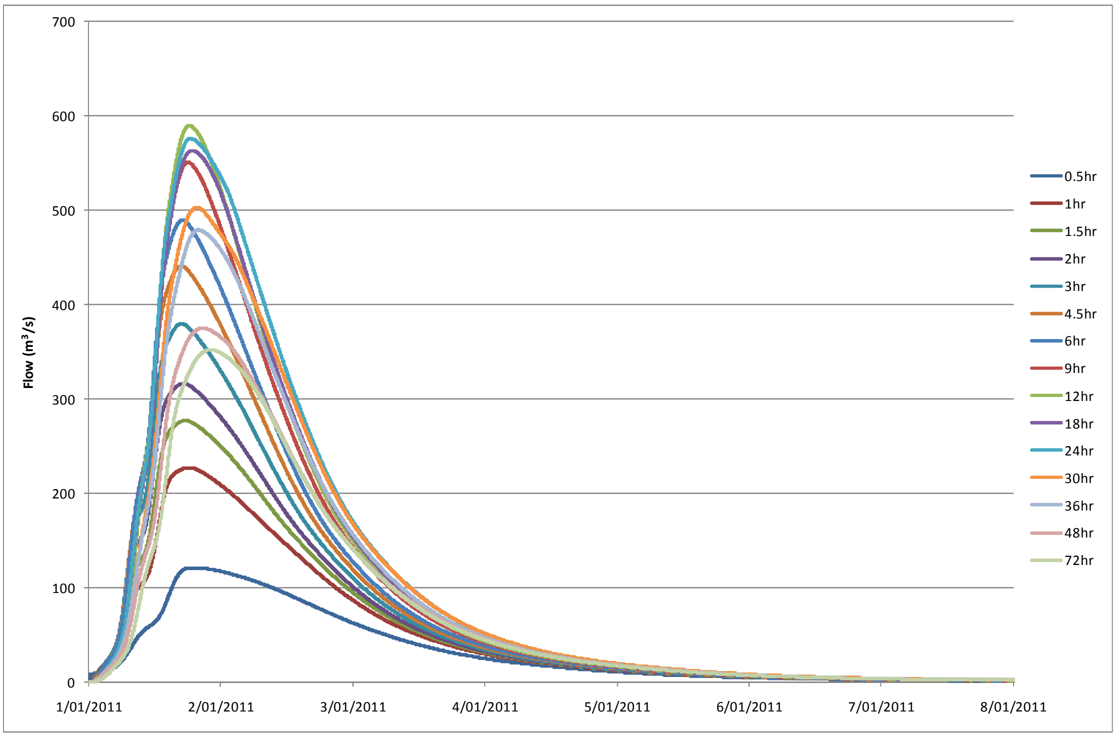

Figure B.B.17: 100 year ARI Design Hydrographs on South West Creek at Port Hedland 2D Model Extent with Continuing Loss Parameters

Appendix C

Hydraulic Modelling

Port Hedland Coastal Vulnerability Study
Appendix C – Hydraulic Modelling

Job Number: LJ15014

Report Number: Rep1022p/Appendix C

Prepared for Landcorp

12 August 2011

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DISCLAIMER

The State Government and the Town of Port Hedland have a shared vision that will see Port Hedland transformed into Pilbara's Port City with a population of 50,000 by 2035.

Land availability, and in particular developable land above predicted future flood levels, is a clear constraint. In recognition of this the Royalties of Regions program has funded the preparation of the Port Hedland Coast Vulnerability Study. The Study considers the combined impact of flood from rainfall and storm surge. The Study was undertaken by Cardno Pty Ltd and was overseen by a Steering Committee that included the Department of Water, the Department of Transport and the Department of Planning.

The report is not a statutory document. However, the reports findings will be considered by statutory agencies in assessing future development proposals under the Town of Port Hedland's Town Planning Scheme and the WAPC's Statement of Planning Policy 2.6.

Developers and their consultants may use the report and its data for their own works but must undertake their own independent verification specific to their development proposal.

Document Control: Port Hedland Coastal Vulnerability Study – Hydraulic Modelling Report

Version	Date	Author		Reviewer	
		Name	Initials	Name	Initials
1 – Draft	7 April 2011	Heath Sommerville / Rob Swan	RCS	Jo Garcia-Webb	JGW
2 - Final	12 August 2011	Heath Sommerville / Rob Swan	RCS	David Van Senden	DVS

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ABBREVIATIONS AND GLOSSARY:

AHD	Australian Height Datum which is the standard vertical elevation datum for Australia. At Port Hedland, AHD is +3.902m above Chart Datum at the permanent tide gauge.
ARI	Average Recurrence Interval; relates to the probability of occurrence of a design event.
BoM	Bureau of Meteorology
HAT	Highest Astronomical Tide
LAT	Low Astronomical Tide
LiDAR	Light Detection and Ranging
MSL	Mean Sea Level
SLR	Sea Level Rise
Water Level Residual	Water level difference between observed water level and the predicted (astronomical) water level.

C.1 HYDRAULIC MODEL SYSTEM

The Deltares 1D2D modelling system, SOBEK, was used to undertake the hydraulic inundation modelling for the Port Hedland Coastal Vulnerability Study. The SOBEK model system uses the same hydraulic model engine as the Delft3D model which was applied in the ocean inundation modelling (**Appendix A**) and is able to compute the channel (1D) and overland flow (2D) components of the study. SOBEK is a professional software package developed by Deltares , situated in The Netherlands, and is one of the largest independent hydraulic institutes in Europe. It is world-renowned in the fields of hydraulic research and consulting (WL|Delft, 2005).

The combined package allows for the computation by the 1D module of channel and pipe flow, including structures such as culverts, weirs, gates and pumps, and pipe details such as inverts, obverts, pipe sizes and pipe material, and is then dynamically linked to the 2D overland flow module. The 1D and 2D domains are automatically coupled at 1D-calculation points (such as manholes) whenever they overlap each other. The model commences with the 1D component computing the inflow levels until such time as pipe's or channels are full and overflows when the flow computation moves to the 2D domain. The 1D network and the 2D grid hydrodynamics are solved simultaneously using the robust Stelling numerical scheme that handles steep fronts, wetting and drying processes and subcritical and supercritical flows (Stelling, 1999).

The advantages of this system are that the channel/pipe system is explicitly modelled as a sub-system within the two-dimensional overland flow computation. This means that generalised assumptions regarding the capacity of the channel/pipe system are not required. This system employs a unique implicit coupling between the one and two-dimensional hydraulic components that provides high accuracy and stability within the computation.

C.2 HYDROLOGY AND OCEAN JOINT PROBABILITY

C.2.1 Data Analyses

In order to determine appropriate boundary conditions for the catchment and ocean inundation modelling, Cardno undertook a correlation analysis to assess the impact of rainfall upon measured water level residuals in the Port Hedland harbour. In the context of a coastal and estuarine environment, there are numerous physical processes that can contribute to the observed water level residual (observed water level minus the predicted astronomical tidal water levels) at any given time. It is expected that large rainfall events at Port Hedland, as a proxy for runoff, is strongly correlated with cyclone events. Tropical cyclones can generate large residual water levels at Port Hedland due to the effects of air pressure, wind stress and wave processes in the Port Hedland coastal environment. Cardno has undertaken investigations to identify if, from the measured rainfall and water level residual data sets for Port Hedland, there is any correlation between these two processes. In addition to this process, Cardno has also investigated the potential influence that rainfall runoff may have on water levels within the Port Hedland harbour.

The water level residual has been calculated by subtracting the predicted tide level based on the published tidal constants for Port Hedland (AHO, 2009) from the measured data set. The period of analysis for the water level record is from 9 December 1993 through to 31 December 2009 with 82.3% data coverage over this time. **Figure C.2.1** presents a scatter plot of daily rainfall to 9am recorded at Port Hedland Airport against the maximum residual water level measured in the Port Hedland harbour for the corresponding day. A statistically significant coefficient of determination of 0.15 is calculated for this dataset. **Figure C.2.2** presents a similar scatter plot of rainfall against residual water level for cyclonic periods only. A statistically significant coefficient of determination of 0.21 is calculated for this dataset. This suggests that during cyclonic conditions, there may be a minor correlation between rainfall at Port Hedland and the residual water levels in the harbour. This low correlation may be due to rainfall influencing water levels within the harbour, or more likely due to some weak correlation between effects of the cyclone on ocean water levels due to pressure, wind and wave forcing, and the rainfall generated by a cyclone. As part of these analyses, cross-correlation analyses with a time lag between ocean water levels and rainfall of up to 5-days was undertaken. No statistically significant correlation was observed for any lag up to five days.

Overall, the correlation between water levels and rainfall is very weak and at first glance it may indicate that the joint occurrence between rainfall and harbour water levels is not a significant factor for any flood study in the Port Hedland region. In addition, the large astronomical tide at the site is a significant influence on the design ocean inundation levels at Port Hedland and is a further independent variable which minimises the probability of large catchment flows occurring jointly with elevated harbour water levels. Also Port Hedland rainfall may not reflect runoff from the winter catchment

However, the data set applied in these analyses is limited in its duration (< 20-years) and in the context of a long-return period inundation assessment it may be prudent to adopt a conservative approach which assumes some degree of joint occurrence between water levels and catchment flows for design events. This approach is further discussed in **Section C.2.2.**

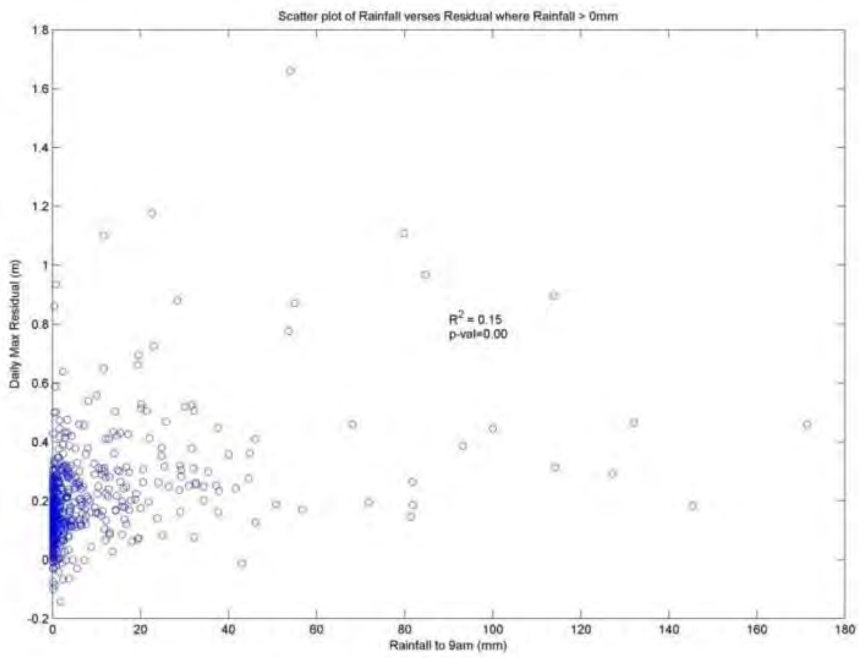


Figure C.2.1 - Scatter plot of rainfall to 9am at Port Hedland Airport versus maximum daily residual recorded at Port Hedland Harbour for days where rainfall was recorded (1993 through 2009)

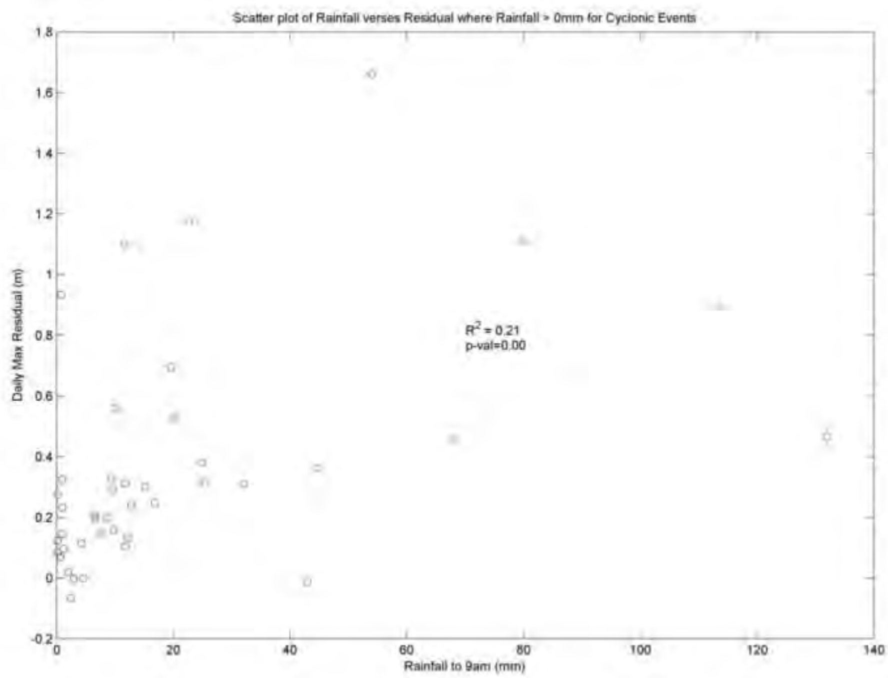


Figure C.2.2 - Scatter plot of rainfall to 9am at Port Hedland Airport versus maximum daily residual recorded at Port Hedland Harbour for cyclonic periods where rainfall was recorded (1993 through 2009)

C.2.2 Addressing Joint Occurrence between Catchment Flows and Ocean Water Levels in Design Events

The analysis of the 20-year rainfall and measured water level data from Port Hedland present in **Section C.2.1** indicates that for this limited data set no discernable relationship between rainfall and ocean water level joint occurrence can be derived. In the simulation of design flood events from either catchment or ocean inundation, it is prudent to adopt a risk averse approach and consider some form of joint occurrence between catchment flows (rainfall) and ocean water levels. In Western Australia, there are no guidelines to address the joint occurrence of ocean water levels and catchment flows for flood studies or in the preparation of water management plans. Joint occurrence between catchment flows and ocean water levels are addressed in consultation with the Department of Water on a case by case basis.

For the Port Hedland area planning policies have been influenced by the Greater Port Hedland Storm Surge Study (GPHSS) undertaken for the Department of Planning in 2000 (GEMS, 2000). That study concluded that large ocean storm surge events tended not to be associated with peak catchment response. Within that study, in the assessment of flooding in streams and rivers, storm surge was not considered a hydraulic constraint on the water levels in the lower reaches of the streams and rivers. Similarly, recent flood studies and water management studies for South Hedland (GHD, 2010) and the Wedgefield extension (JDA, 2010) did not consider a downstream water level based on normal tidal or extreme event storm surge conditions. For both these studies, downstream water level conditions were derived from the design catchment flow conditions and the potential effects of high ocean water levels were neglected.

Elsewhere in Australia and overseas, the joint occurrence of catchment flows and ocean water levels are addressed with a more formal approach, including with government guidelines for flood studies. For example, in NSW the Department of Environment, Climate Change and Water (DECCW) recommends an envelope approach is adopted to consider flooding from catchment and ocean. The typical scenarios involve considering the 100-year ARI catchment flows coinciding with a 20-year ARI ocean water level and vice-versa. In NSW, over the last 15-years flood studies undertaken by a number of consultants have adopted a range of joint occurrence scenarios including:-

- 100-year ARI catchment flows coinciding with a 5-year ARI ocean water level and vice-versa;
- 100-year ARI catchment flows coinciding with a 100-year ARI ocean water level and vice-versa; and
- 100-year ARI catchment flows coinciding with a 1% exceedence ocean water level which in NSW is normally just below HAT.

In the context of the large tidal range at Port Hedland, this approach is likely to be conservative. In Queensland, whilst no government guideline exists, the joint occurrence of ocean water level and catchment flows have been investigated particularly in the Gold Coast region which is affected by East Coast Low storm systems as well as occasional tropical cyclones. CSIRO (2007) investigated the relationship between daily rainfall totals and residual ocean water levels over a 39-year period. The investigations indicated that for daily rainfall totals greater than 300mm, there was a positive correlation with residual water levels in the Broadwater. The typical maximum residual in the Broadwater for events greater than 300mm total daily rainfall was about +0.3m. In 2010, Cardno undertook a literature assessment for the Gold Coast City Council into joint occurrence between catchment flows and ocean water levels which considered papers, guidelines and studies from around Australia and overseas. In that study, Cardno suggested a range of joint occurrence models for the Gold Coast Council area including:-

- Adopting a time varying tide with a peak ocean water level of MHWS occurring concurrently with peak catchment flows for large catchments with critical durations > 36-hours; and
- Adopting a time varying tide with a peak ocean water level equal to the 1% exceedence water level which is derived from long-term measured data. This approach may be particularly appropriate for small catchments with short critical durations which may experience a higher correlation between catchment flows and elevated ocean water levels.

Following a steering group meeting on 23 February 2011 and subsequent discussions between the Department of Transport and the Department of Water, an agreed guidelinel for the joint occurrence of catchment flows and ocean water levels was adopted specifically for this study. Based on this outcome, for the Port Hedland Coastal Vulnerability Study, Cardno are adopting the following joint occurrence model for the design event simulations:-

- Adopt 20-year ARI ocean water level in-conjunction with the 100-year ARI catchment flows. Ocean water levels should be simulated with a realistic time series with the peak ocean water level being coincident with the peak catchment flows. For 100-year ARI and more frequent design events, the ocean water level condition will be based on a design ocean water level with an ARI of one fifth the ARI of the catchment flows.
- For 200-year ARI event and less frequent design events, the ocean water level condition will be based on a design ocean water level with an ARI of one tenth the ARI of the catchment flow. For example, the 500-year catchment flows with the 50-year ocean water level.
- When design ocean water levels are below MHWS for a specific event, a time varying water level data set should be applied on the ocean boundary of the hydraulic model with a peak high water equal to MHWS occurring with the peak catchment flows. At Port Hedland, MHWS is +6.7m LAT (+2.8m AHD).

C.3 MODEL SETUP - PORT HEDLAND

The hydraulic models consist of two main components:-

- The channel network (1D); and
- 2D grids of the surface topography.

The establishment of these two components of the model is described in the following sections.

C.3.1 Channel System (1D)

The 1D network aims to improve the performance of the hydraulic model in areas where the 2D model may not accurately represent the catchment. These areas can include townships and areas with defined channels and structures. As no formal drainage infrastructure exists in the Shellborough region, no 1D elements have been used in this area. In the Port Hedland area, 1D elements have been used to define bridges, culverts and channels, especially in the South Hedland and Wedgefield areas.

Within the South Hedland township, the drainage channels have been represented in the 1D network using estimated trapezoidal channels based on engineering judgement and a site visit to the region. The structures in South Hedland have all been surveyed and included in the 1D network. The 1D network was defined in this region to improve the accuracy of the inundation mapping in the town.

For the remainder of the Port Hedland model the structures (bridges, culverts etc) have been included in the 1D network and no channels have been represented. The dimensions of structures have been defined using engineering judgement and from the site inspection. The channels have not been included in the 1D network as the 2D grid adequately represents the wide channels in the region and no survey has been undertaken to capture the channel dimensions. Limited information was available from the Town of Port Hedland and Main Roads regarding the structures in the region.

The Shellborough model has no 1D network for structures and channels as there are none present in this region.

C.3.2 Topography (2D)

The topography was defined using a Digital Terrain Model (DTM) of the region developed with the spatial data analysis package, 12D, using aerial survey data supplied by AAM.

C.3.2.1 Ocean and Catchment Inundation Hydraulic Modelling

The two-dimensional, overland flow component of the hydraulic model consists of a main grid that covers the entire study area and two smaller, nested grids covering sections of the South Hedland and Wedgefield township area.

The main grid has a resolution of 40 m and the nested grids have a resolution of 10 m. For the general overland floodplain areas within the study area, the 40m grid resolution is considered adequate. However, in the South Hedland and Wedgefield area, where overland flow paths through urban areas were identified during preliminary model simulations, a finer resolution of 10 m was chosen. This provided a better definition of the detailed flood hydraulics through the township.

The dimensions of the grids are summarised in Table C.3.1. The 2D model extent is shown in **Figure C.3.1** for the calibration and full model respectively.

Table C.3.1 – Topography grid size – Full Model

Parameter	Main Grid	South Hedland Nested Grid	Wedgefield Nested Grid
Cell size	40m x 40m	10m x 10m	10m x 10m
Grid Cells (x direction)	701 columns	258 columns	330 columns
Grid Cells (y direction)	494 rows	266 rows	390 rows

C.3.2.2 Wave Setup Inundation Modelling

The ocean wave and ocean response modelling is described in **Appendix A**. To assess the wave setup inundation, Cardno have focused the modelling on the Harbour region of Port Hedland. A 12m grid with and a nested 4m grid at the Pretty Pool area were used to assess the impact of wave setup on inundation in the Port Hedland township. Wave setup does not impact flood levels in the estuary and the results of the two models were combined in post processing to allow the derivation of a single continuous flood shape from ocean inundation. The properties of the grids used for the Port Hedland wave setup assessment is shown in **Table C.3.2** and the sub-grid location is shown in **Figure C.3.1.**

Table C.3.2 – Topography grid size – Ocean Inundation with Setup Model

	Parameter	Dimensions
Ocean Inundation 12 m Grid	Cell size	12m x 12m
	Grid Cells (x direction)	786 columns
	Grid Cells (y direction)	459 rows
Ocean Inundation 4 m Grid	Cell size	4m x 4m
	Grid Cells (x direction)	134 columns
	Grid Cells (y direction)	101 rows

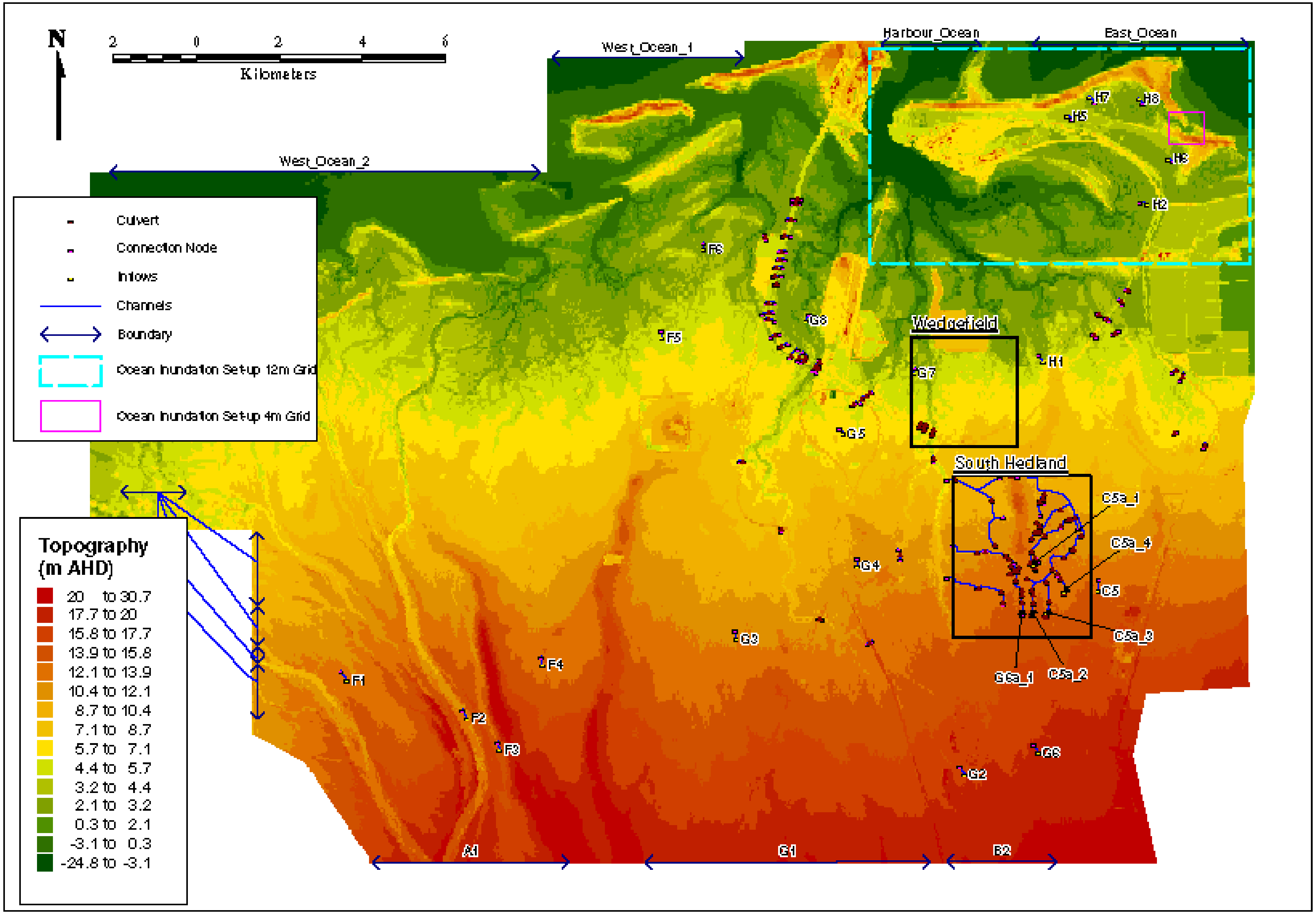


Figure C.3.1 – Model Setup – Port Hedland

C.3.3 Hydraulic Roughness

The hydraulic roughness for the overland flow model was described using a two-dimensional roughness map of Manning's "n" values. This was developed by digitising different land-use and vegetation zones from the digital aerial images within a GIS environment (MapInfo). The roughness values were set to the values as shown in **Table C.3.3** and the final spatial distribution of roughness is shown in **Figure C.3.2.**

The roughness parameters are consistent with the values specified by Chow (1973), the Mannings 'n' for the roads, residential and commercial are consistent with previous modelling experience and practices.

Table C.3.3 – Calibrated Roughness Parameters, Manning's 'n'

Parameter	Roughness Manning's 'n'
Roads	0.018
Light Vegetated Channel	0.0285
Ocean	0.03
Low Density Residential	0.05
Medium Vegetated Channel	0.0616
Rural	0.08
Heavy Vegetated Channel	0.0925
Mangrove	0.12
Urban	0.15
Industrial	0.5

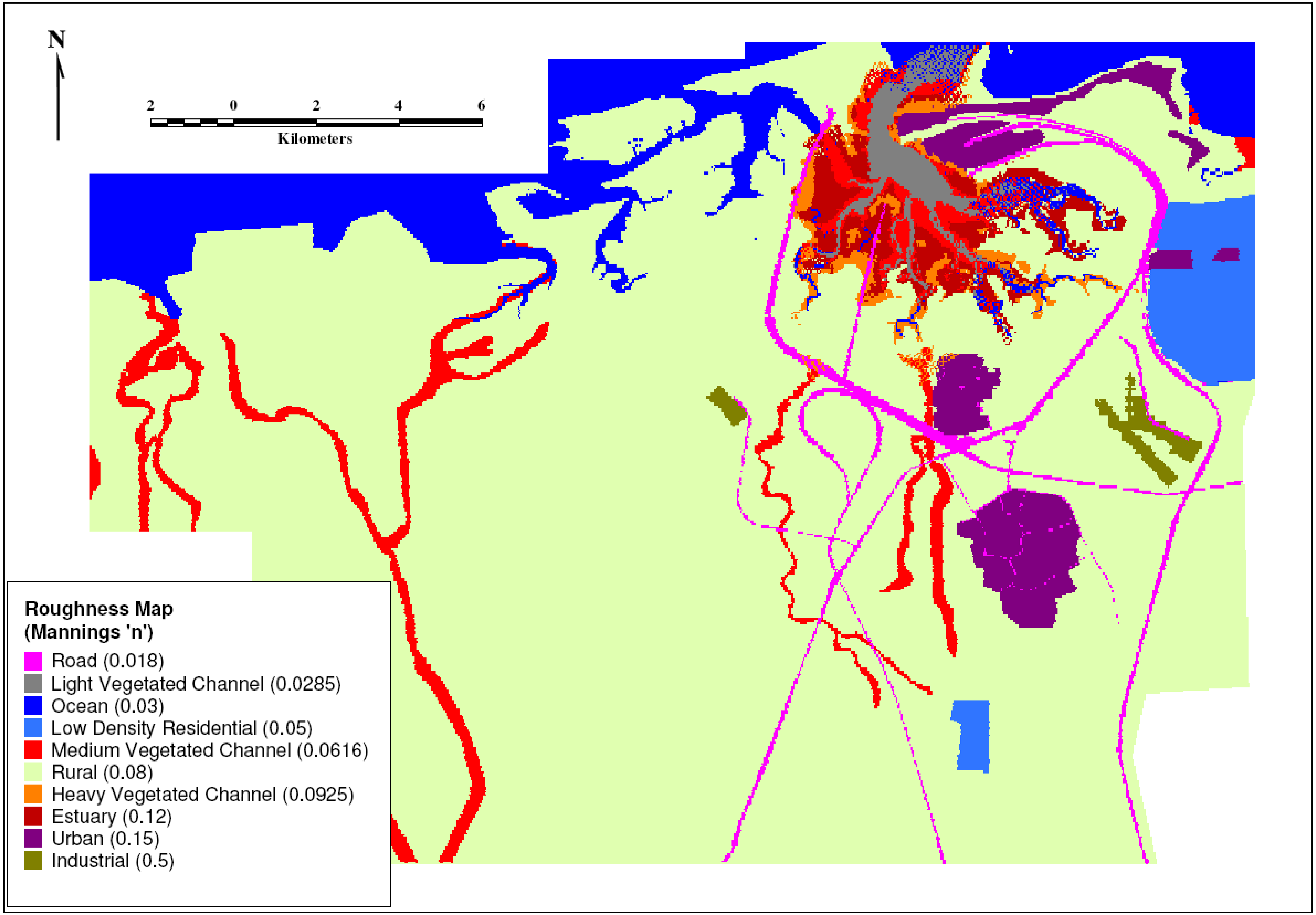


Figure C.3.2 – Roughness Map – Port Hedland

C.3.4 Ocean Boundary Conditions

C.3.4.1 Catchment Inundation Hydraulic Modelling

The downstream model boundary used a time varying water level, derived from the Delft3D ocean model results described in **Appendix A (Main Report)**. The four downstream boundaries, East Ocean, Harbour Ocean, West Ocean 1 and West Ocean 2, are shown in **Figure C.3.1**. The time of peak water level in the downstream boundaries has been matched to the time of peak flow in the river system at the ocean interface. **Table C.3.4** shows the relevant downstream condition for each individual ARI event.

Table C.3.4 – Downstream Boundary Conditions (Catchment Inundation)

Design ARI event	Downstream Level Condition/ARI
2	Mean High Water Springs
10	Mean High Water Springs
100	20-year ARI
200	50-year ARI
500	100-year ARI

C.3.4.2 Ocean Inundation Hydraulic Modelling

Ocean inundation hydraulic modelling was undertaken for a catchment wide storm surge ocean level and incorporated wave setup, for the Port Hedland area only. Time varying water level boundaries were applied at identical locations to the catchment wide hydraulic model. For the wave setup case, the Harbour Ocean and East Ocean boundaries were modified to include the wave setup component in the time series. **Table C.3.5** shows the modelled ocean inundation events.

Table C.3.5 – Downstream Boundary Conditions (Ocean Inundation)

Design Ocean Inundation ARI event	Wave Setup Required
2	No
10	Yes
100	Yes
200	Yes
500	Yes

C.3.5 Catchment Inflows

Flow is introduced into the hydraulic model through the use of a lateral inflow points (where the catchment centroid is in the 2D model domain) and through time varying flow boundaries. The stream inflow boundaries of the Turner River and South West Creek were set up as 2D line boundaries with inflow hydrographs representing the hydrology for the different events. Line boundaries distribute the inflow along the line based on the 2D topography, thereby maintaining the anticipated flow patterns. The upstream 2D line boundaries are shown in **Figure C.3.1**, represented by A1, G1 and B2 and the lateral inflow locations are also shown. The hydrographs at these boundaries were obtained from the hydrological model results.

Each design storm event contains different inflows at each location. Under existing conditions, multiple storms have been assessed that provide the peak inflow from both the local catchments and from the Turner River and South Creek catchments. For the climate change conditions, only the larger river catchment storms were assessed. The peak inflow for each design storm at each location is shown in **Table C.3.6** and **Table C.3.7**.

Table C.3.6 – Peak Inflow (m³/s) - Existing Conditions

Storm Event ARI	2 Year		10 -year		100 year		200 year		500 year	
Duration	1.5 Hr	36 hr	1.5 hr	36 hr	1.5 hr	24hr	1.5 hr	18 hr	1.5hr	18 hr
A1	0.0	494.7	406.9	3020.1	2959.2	9302.7	4079.1	10397	5857.6	11517
G1	0.0	3.9	1.0	62.3	40.6	212.0	60.6	277.4	91.2	382.0
B2	0.0	14.0	8.9	197.9	141.9	666.6	207.8	870.7	312.2	1201.7
F1	0.0	0.4	0.5	37.2	21.3	128.1	30.8	154.0	47.3	217.8
F2	0.0	0.2	0.0	3.1	2.3	10.6	3.4	14.1	5.3	19.1
F3	0.0	1.1	0.3	10.6	11.9	37.5	17.6	47.9	26.5	65.6
F4	0.0	0.8	0.4	29.0	17.9	103.0	26.0	126.7	40.3	177.9
F5	0.0	1.1	0.3	14.8	12.0	48.9	17.7	65.3	26.7	89.1
F6	0.0	0.4	0.1	5.1	4.6	17.4	6.8	22.1	10.5	30.3
G2	0.0	0.5	0.1	6.7	4.9	22.7	7.2	29.9	11.0	40.6
G3	10.8	4.5	25.2	35.8	48.2	126.7	58.2	152.6	73.4	214.7
G4	0.0	0.4	0.1	5.3	4.1	17.9	6.1	23.7	9.4	32.3
G5	0.0	0.3	0.1	3.3	3.3	11.5	5.1	14.7	7.9	20.0
G6	0.0	1.2	0.3	18.5	12.7	61.6	18.9	83.0	28.2	112.2
G6a_1	25.5	10.8	57.5	23.1	111.7	55.1	135.0	63.6	169.0	78.3
G7	21.2	8.4	54.0	18.0	96.9	48.0	115.0	57.9	141.6	75.3
G8	11.0	3.8	30.0	12.3	49.9	42.6	58.6	53.3	71.5	71.7
H1	7.5	2.5	20.2	14.2	33.1	48.5	39.2	60.7	47.4	84.2
H2	0.0	1.4	0.4	15.2	15.6	52.9	23.1	67.3	35.0	91.0
H3	20.0	6.6	53.6	14.0	87.8	33.4	103.5	38.5	125.6	47.4
H4	17.2	5.5	46.0	11.8	74.3	30.1	87.6	35.6	106.0	45.4
H5	3.5	0.9	9.2	2.4	13.0	7.7	15.2	10.4	18.2	13.3
H6	0.0	0.8	0.4	9.4	16.9	29.0	24.8	36.7	38.0	47.0
H7	3.4	0.8	8.3	2.2	11.7	6.3	13.6	8.2	16.3	11.8
H8	6.6	1.6	16.4	4.8	23.2	14.2	26.9	17.8	32.3	25.7
C5	0.0	1.0	0.2	16.1	10.6	56.0	15.6	73.0	23.9	99.9
C5a*	35.0	16.6	85.2	35.6	153.1	84.7	182.2	97.7	230.8	120.4

*catchment inflows from C5a are split into four inflow locations in the hydraulic model

Table C.3.7 – Peak Inflow (m^3/s) - Climate Change Conditions

Storm Event ARI	Climate Change 2060			Climate Change 2110		
Duration	2yr 2060	100yr 2060	500yr 2060	2yr 2110	100yr 2110	500yr 2110
A1	676.7	10026.3	12219.0	869.84	10432.70	12854.50
G1	7.6	247.2	448.1	12.00	279.78	506.84
B2	26.4	780.7	1411.1	40.54	885.96	1596.72
F1	3.9	156.3	261.3	6.37	179.98	302.43
F2	0.4	12.5	22.4	0.69	14.19	25.30
F3	1.7	43.8	77.2	2.44	50.21	87.67
F4	3.3	122.0	211.3	5.38	139.40	239.75
F5	2.3	58.4	103.5	3.31	67.94	116.64
F6	0.8	20.9	35.7	1.14	23.79	40.38
G2	0.9	26.7	47.4	1.44	30.34	53.54
G3	5.0	153.3	255.5	6.41	175.07	293.87
G4	0.8	21.1	38.0	1.20	24.21	42.79
G5	0.5	13.4	23.5	0.74	15.19	26.69
G6	2.4	73.7	130.4	3.69	84.03	147.17
G6a_1	11.9	60.6	86.1	12.99	66.08	93.95
G7	9.2	54.1	85.1	10.08	60.38	94.89
G8	4.1	49.2	83.8	4.50	55.49	95.55
H1	2.7	57.8	98.3	3.10	66.01	110.66
H2	2.4	62.0	106.7	3.38	69.95	121.38
H3	7.2	36.7	52.2	7.87	40.03	56.92
H4	6.0	33.5	50.9	6.60	37.04	56.36
H5	1.0	9.0	15.6	1.10	10.43	17.83
H6	1.3	33.8	54.4	1.86	38.86	63.52
H7	0.9	7.4	14.0	0.99	8.62	16.07
H8	1.8	16.9	30.4	1.95	19.26	35.33
C5	2.1	65.0	116.0	3.11	73.76	130.89
C5a*	18.3	93.1	132.4	19.97	101.58	144.44

C.4 MODEL SETUP - SHELLBOROUGH

Due to the undeveloped nature of the Shellborough area, the hydraulic model only requires a 2-dimensional model to accurately simulate inundation in the area. The establishment of this component of the model is described in the following section.

C.4.1 Topography (2D)

The topography was defined using a Digital Terrain Model (DTM) of the region developed with the spatial data analysis package, 12D, using aerial Lidar from AAM.

C.4.1.1 Ocean and Catchment Inundation Hydraulic Modelling

The two-dimensional, overland flow component of the hydraulic model consists of a main grid that covers the entire study area. The dimensions of the grids are summarised in **Table C.4.1.** The 2D model extent is shown in **Figure C.4.1** for the hydraulic model.

Table C.4.1– Topography grid size

Parameter	Dimensions
Cell size	10m x 10m
Grid Cells (x direction)	848 columns
Grid Cells (y direction)	541 rows

C.4.1.2 Wave Setup Inundation Modelling

A subset of the model grid was used to assess the impact of wave setup on inundation at Shellborough. Wave setup does not impact flood levels in the estuary and the results of the two models were combined in post processing to allow the derivation of a single continuous flood shape from ocean inundation. The properties of the grid used for the Shellborough wave setup assessment is shown in **Table C.4.2** and the sub-grid location is shown in **Figure C.4.1**.

Table C.4.2 – Topography grid size

Parameter	Dimensions
Cell size	10m x 10m
Grid Cells (x direction)	766 columns
Grid Cells (y direction)	347 rows

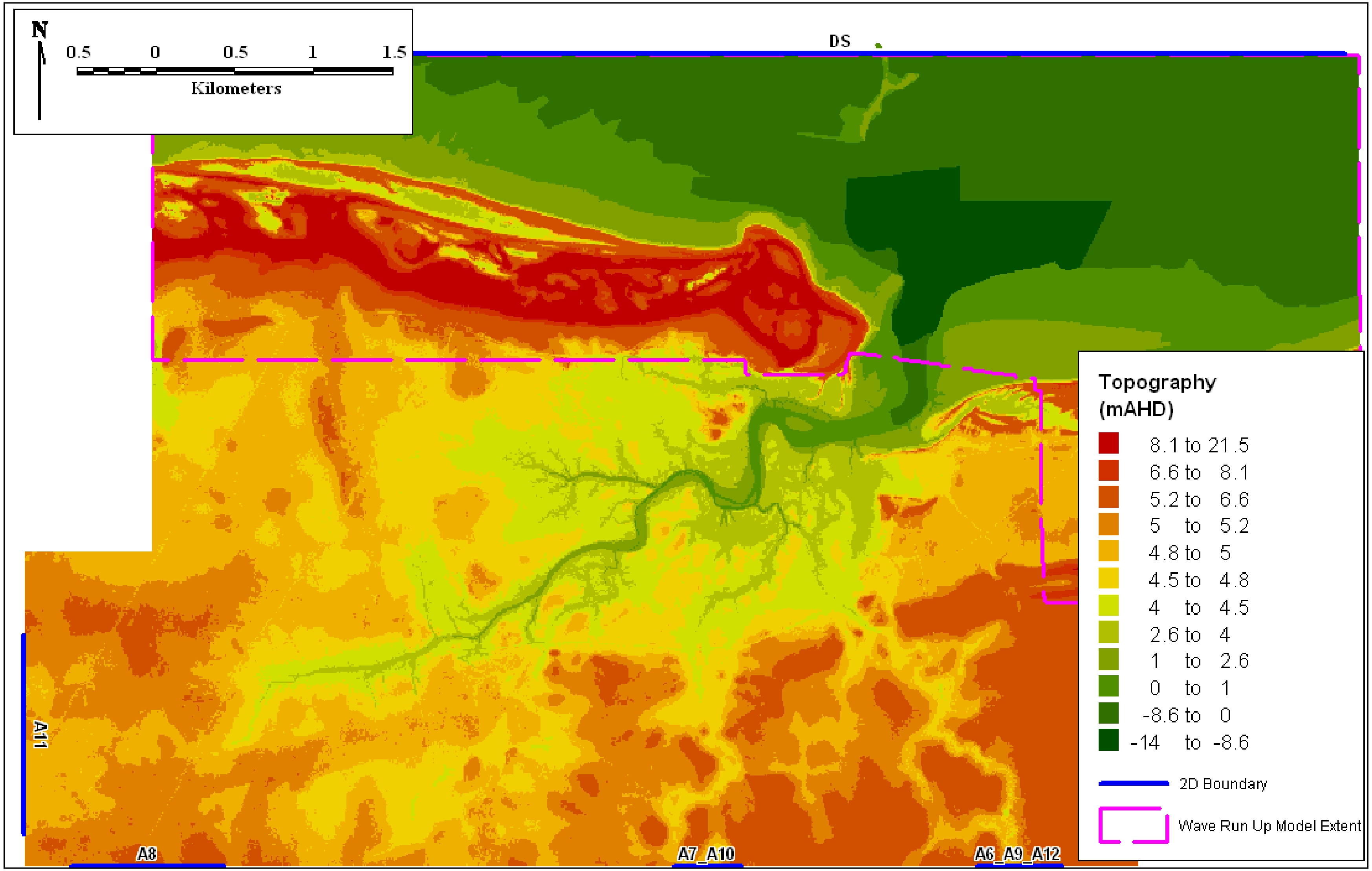


Figure C.4.1 – Model Setup – Shellborough

C.4.2 Hydraulic Roughness

Similar to Port Hedland Section 3.3 the roughness values for the Shellborough area were set to the values as shown in **Table C.4.3** and the spatial distribution of roughness is shown in **Figure C*.4.2***.

Table C.4.3 – Calibrated Roughness Parameters, Mannings 'n'

Parameter	Roughness Manning's 'n'
Ocean	0.03
Rural	0.08
Shellborough Estuary	0.12

C.4.3 Boundary Conditions - Ocean

C.4.3.1 Catchment Inundation Hydraulic Modelling

The downstream model boundary used a time varying water level, derived from the Delft3D ocean innundation model results (**Appendix A – Main Report).** The boundary location is shown in **Figure C*.4.1***. The time of peak water level at the downstream boundary has been matched to the time of peak flow in the river system at the ocean interface. **Table C.3.4** shows the relevant downstream condition for each individual ARI event.

C.4.3.2 Ocean Inundation Hydraulic Modelling

Ocean inundation hydraulic modelling was undertaken for a catchment wide storm surge ocean level and incorporated wave setup. Time varying water level boundaries was applied for each modelled case. For the wave setup case, the boundaries were modified to include the wave setup component in the time series. **Table C.3.5** shows the modelled ocean inundation events.

C.4.4 Catchment Inflows

Flow is introduced into the hydraulic model through the use of time varying flow boundaries. Boundary conditions were established at the upstream limit of the model, as 2D line boundaries with inflow hydrographs representing the hydrology for the different events. The upstream 2D line boundaries are shown in **Figure C.4.1**, represented by A11, A8, A7_A10 and A6_A9_A12. **Table C.4.4** shows the peak inflows for the Shellborough model.

Table C.4.4 – Peak Inflows (m³/s) - Shellborough

	ARI Event	A8	A11	A7_A10	A6_A9_A12
Existing	2	7.6	2.7	16.7	20.9
	10	16.2	5.8	36.7	44.6
	100	56.4	20.2	117.6	139.8
	200	73.1	26.2	162.6	181.3
	500	100.0	35.7	236.6	236.2
Climate Change 2060	2	8.8	3.1	19.3	23.9
	100	65.1	23.1	134.8	158.8
	500	114.3	41.3	276.0	265.2
Climate Change 2110	2	10.1	3.5	-	-
	100	73.7	26.0	161.9	178.1
	500	129.0	46.3	317.3	294.8

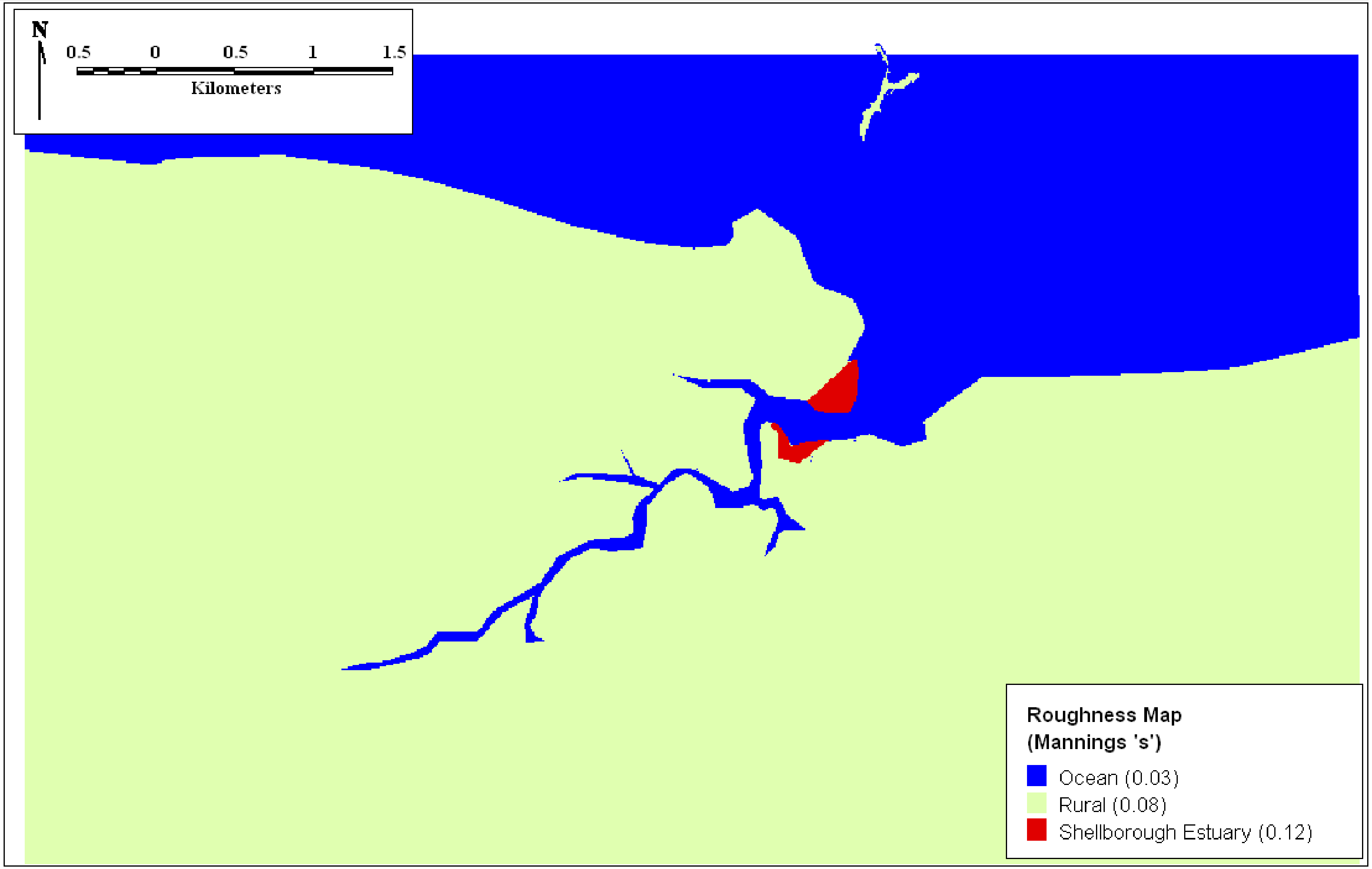

N
0.5 0 0.5 1 1.5
Kilometers
Roughness Map
(Mannings 's')
Ocean (0.03)
Rural (0.08)
Shellborough Estuary (0.12)

Figure C.4.2 – Roughness Grid – Shellborough

C.5 MODEL VALIDATION

Model validation was carried out for the Port Hedland model. Due to the lack of consistent stream flow information for both the Turner River and South West Creek gauges, a validation exercise was undertaken at the bridge structure located near the corner of Greater Northern Highway and Hamilton Road over South Creek as shown in **Figure C.5.1**. Levels and flow data from an event in March 1989 recorded by Main Roads was provided to Cardno by the Department of Planning (DoP). The event was estimated to be a 6 hour, 100-year ARI storm. Cardno is unable to verify the accuracy of this report as Main Roads was unable to provide documentation detailing the observations taken during the event. It should also be noted that Cardno did not receive any design plans of this structure from Main Roads and it was not surveyed as part of this project.

The information received from DoP for the March 1989 event indicated that the upstream and downstream water levels at the structure were 7.77 m AHD and 7.58 m AHD respectively with flow rates estimated at 230 m^3/s. Cardno utilised the design flows for the 100-year ARI, 6-hour duration event as inputs to the hydraulic model for validation purposes. Peak inflows for the major river and creek systems were estimated as:

- Turner River : 7,252 m^3/s
- South Creek : 402 m^3/s
- South West Creek : 120 m^3/s

Table C.5.1 shows the comparison between observed and modelled results.

Table C.5.1 – Calibration Results at Bridge Structure E_56

Parameters	Estimated 100-year, 6-hour Event – Main Roads	Modelled 6 hour 100-year ARI event
Upstream Water Level	7.77 m AHD	8.35 m AHD
Downstream Water Level	7.58 m AHD	7.70 m AHD
Flow at Bridge	230 m^3/s (estimated)	268 m^3/s

The validation assessment shows that the design 100-year, 6 hour event produces higher flows than the 1989 event estimates. This is reflected in both the higher flow estimate at South Creek and the higher water levels. The flood model estimates that the flow capacity of the bridge at the soffit is in the order of 200 m^3/s. The soffit level is approximately 7.75 m AHD. Pressure flow effects are then experienced with a full flow capacity of approximately 250 m^3/s at the bridge before overtopping of the Great Northern Highway. The 200 m^3/s estimated at the bridge for a water level of 7.75 m is within 10% of the estimate of Main Roads for the 1989 event.

As the actual temporal and spatial distribution of the 1989 flood event cannot be reproduced, it is considered that the hydraulic model result is consistent with the known flood information.

C.5.1 Summary of Flood Studies from the Port Hedland Region

A number of previous studies have been carried out in the Port Hedland region, however none have been completed that cover the total geographical area of this project. These studies have been undertaken for various projects in the vicinity of Port Hedland. Cardno has reviewed the following studies:

- Fortescue Metals Group - Pilbara Iron Ore And Infrastructure Project Stage A Port And North-South Railway Surface Hydrology, undertaken by Aquaterra Consulting, 2004.
- Landcorp, Wedgefield Industrial Estate Extension, Port Hedland Local Water Management Strategy, undertaken by JDA Consultant Hydrologists, 2010.
- Town of Port Hedland - South Hedland Flood Study, undertaken by GHD 2010.
- Ministry of Planning, Greater Port Hedland Storm Surge Study, undertaken by GEMS, 2000.

These reports have all used the GEMS report as a baseline for comparison of their hydrological approaches. Most of the hydrological information is assessed for South Creek to the bridge at the Great Northern Highway. JDA (2010) reports that the estimated flows at this location are between 269 and 777 m^3/s depending on the estimation method. The methods used for each estimate are relatively simple area based relationships and as such do not take into account cross-catchment flows, routing of flow and other hydraulic constraints, such as roads and channel conditions in their assessment. The JDA (2010) report adopted 269 m^3/s as the design flow rate, which was estimated using the Index Flood Method, in accordance with the procedures described in Australian Rainfall and Runoff (AR&R). The catchment modelling method used in this report is usually more accurate than the area based relationships according to AR&R. The JDA report also recommends that flood assessments be undertaken using a 2D model to obtain greater certainty in flood results.

The Greater Port Hedland Storm Surge Study (GEMS) (2000) estimated flows in South Creek as 383 m^3/s although the methodology used in this assessment is not clear. In the South Hedland Flood Study, GHD (2010) predicted a 100-year ARI flow at South Creek of 162 m^3/s, using a larger upstream catchment than that defined in the GEMS (2000) report. Although GHD (2010) considered the GEMS (2000) result overly conservative, they adopted it for use in their report. The Aquaterra (2004) report also adopts the flood flows reported in GEMS, but cautions that for South Creek, the flows may be underestimated due to the impact of cross catchment inflows from South West Creek.

This report adopts a different approach to those defined above. A full hydrological model of the upstream catchments has been created and used as input to a large scale 2D model of the entire floodplain. Cross catchment flows are catered for in the hydraulic model as are the effects of bridges, culverts, roadways and other hydraulic constraints. The modelling undertaken by Cardno indicates that in the peak 100-year ARI event, model inflows to South Creek and South West Creek are 666 m^3/s and 212 m^3/s respectively. There are additional local catchment inflows upstream of the Great Northern Highway of 270 m^3/s. It should be noted that these flows do not occur simultaneously. The hydraulic model results indicate that the peak flow at the Greater Northern Highway at South Creek is in the order of 410 m^3/s (comprised of 290 m^3/s through the bridge and 120 m^3/s over the highway), however, flows do break out from the floodplain to the north in this area. This flow rate is close to the GEMS (2000) estimate of 383 m^3/s and accounts for cross catchment flows and floodplain storage. Modelled flood levels will vary from previous reports based on location but the method adopted in this report provides greater definition of the flood behaviour than previous studies.

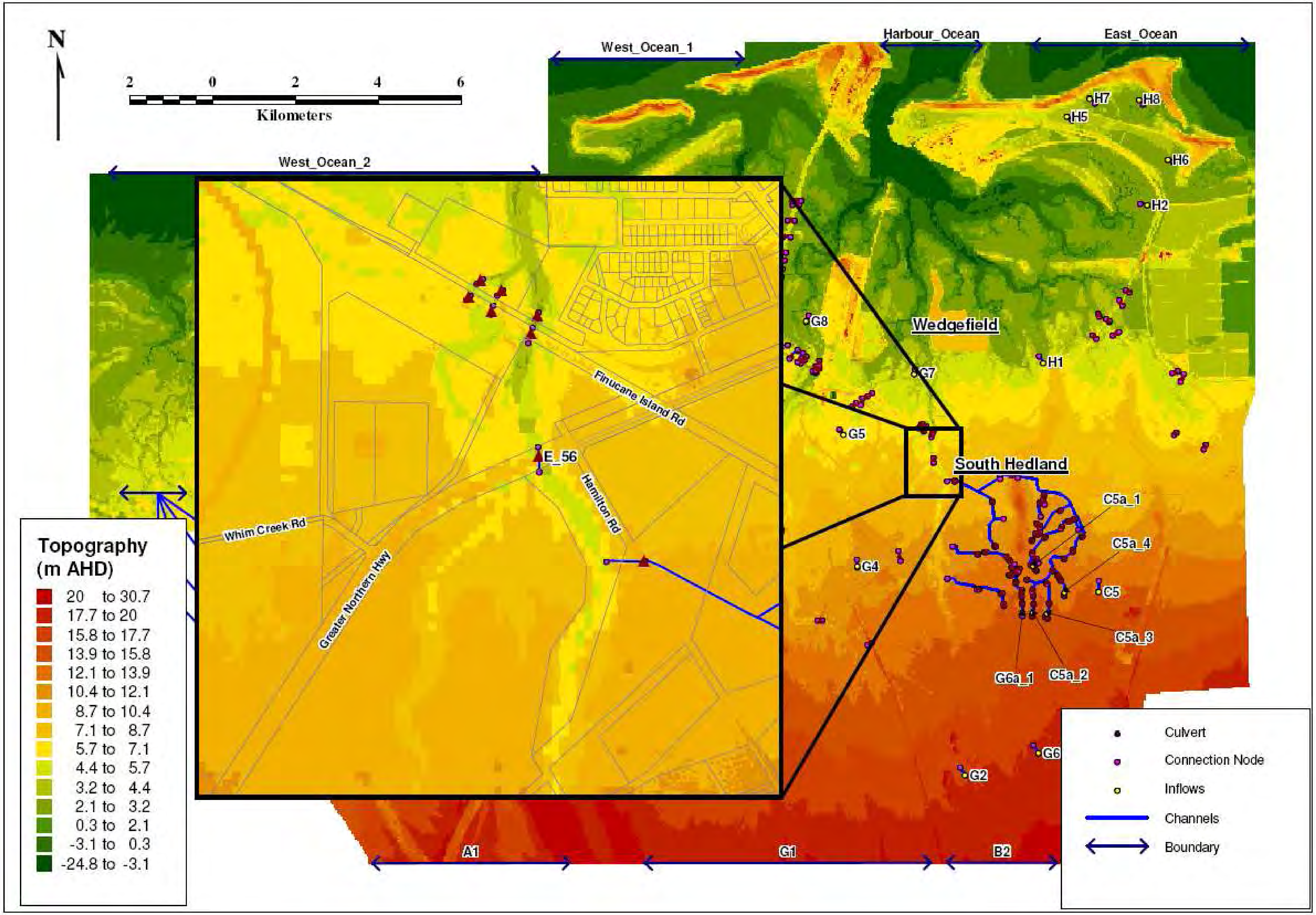


Figure C.5.1 – Calibration Location – Port Hedland

C.6 DESIGN EVENT SIMULATIONS

The Port Hedland and Shellborough hydraulic model has been run for Catchment Inundation (CI) and Ocean Inundation (OI). The modelling included an assessment of the potential impacts of projected sea level rise of +0.3m for 2060 and +0.9m for 2110. The OI runs have an additional component which investigates the impact of wave setup on the coast. **Table C.6.1** shows the hydraulic model runs undertaken for the project at Shellborough and Port Hedland.

Table C.6.1 – Hydraulic Model Runs

Model type	Catchment ARI Event	Ocean ARI Event	Climate Change	Wave Setup (additional runs)
CI	500	100	Yes	
CI	200	10		
CI	100	10	Yes	
CI	10	MHWS		
CI	2	MHWS	Yes	
OI	-	500	Yes	Yes
OI	-	200		Yes
OI	-	100	Yes	Yes
OI	-	10		Yes
OI	-	2	Yes	No
OI	10	50	Yes	

The two-dimensional, overland flow results are reported as water depths (m), water surface levels (m AHD) and flow velocities (m/s) for every grid cell at regular time intervals. Time series of water level, depth and flow velocity were also reported at specific locations.

It should be noted that the flood inundation shapes discussed in the following sections are a representation only of the actual flooding conditions in the catchment. The flood shapes are based on the DEM developed for use in the project (**Section C.3.2**) and do not include consideration of features such as minor piped drainage, localised flow obstructions (such as parked cars, telephone poles and small embankments) or other topographical features that are smaller than the grid cell definition.

C.6.1 Port Hedland

In the greater Port Hedland area, flood flows in the Turner River system do not interact with flood flows in the South Creek/South West Creek systems. There is significant cross catchment flows between South Creek and South West creek under all modelled flood events. The township of South Hedland is impacted by flows from South Creek and from local runoff. Various topographical features cause significant ponding of floodwater and provide barriers to flow conveyance, including the FMG railway line, the Greater Northern Highway and Willowbank Road. Significant flooding is noted in Wedgefield and South Hedland for the 100-year ARI flood event.

Flood inundation in the Port Hedland township is primarily a result of storm surge events. Along the coastline of the township, the 500-year ARI Ocean storm event is expected to breach the dune system near Stevens Street. Fincuane Island Road is also overtopped in this event. In general, flood levels to the south of Wedgefield are controlled by ocean inundation events.

A number of maps have been produced showing the maximum flood extents and depths for each modelled flood event. Note that all figures have been filtered to remove flood depths less than 0.02 m. Water surface levels to m AHD are also shown on the figures at key locations. **Table C.6.2** details the mapping outputs for the project. **Table C.6.3** summarises the maximum water levels at various locations for the catchment inundation events, shown in ***Figure C.6.1*** for each design storm. **Table C.6.34** summarises the maximum water levels at various locations (shown in **Figure C.6.2**) for the ocean inundation events. Note that for the climate change conditions, only the longer duration catchment wide storms have been assessed. For the short duration storms that may cause local impacts at Port Hedland under climate change, it is reasonable to assume a 20% increase in flood depth.

Table C.6.2 – Index of Flood Inundation Maps -Port Hedland

Map Index Numbers	Output Description
Map P01 – Map P10	Existing Conditions Catchment Inundation Maps
Map P11 – Map P16	2060 Climate Change Catchment Inundation Maps
Map P17 – Map P22	2110 Climate Change Catchment Inundation Maps
Map P23 – Map P26	Existing Conditions Ocean Inundation Maps
Map P27 – Map P28	2060 Climate Change Ocean Inundation Maps
Map P29 – Map P30	2110 Climate Change Ocean Inundation Maps
Map P31 – Map P34	East Port Hedland Developement Ocean Inundation Maps

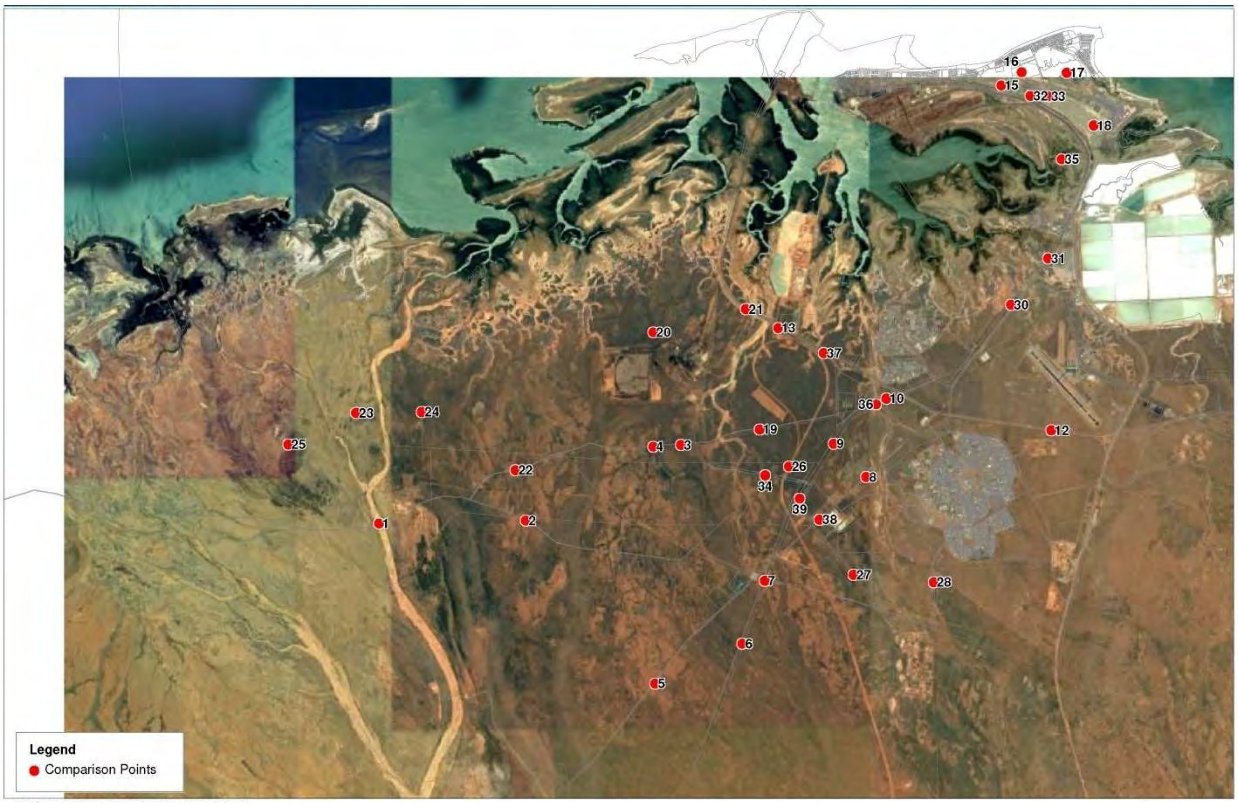


Figure C.6.1 – Catchment Inundation Water Level Reporting Locations– Port Hedland

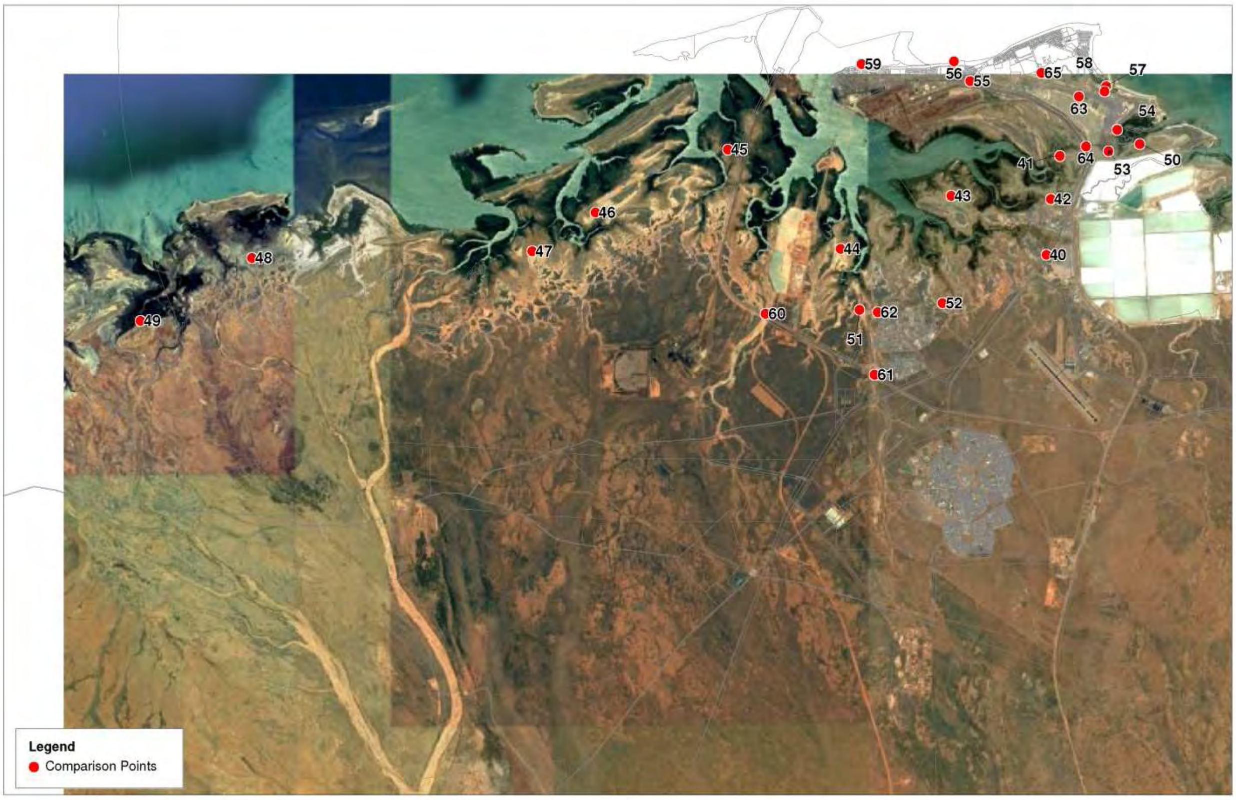


Figure C.6.2 – Ocean Inundation Water Level Reporting Locations– Port Hedland

Table C.6.3 – Port Hedland Area Peak Flood Levels (mAHD) - due to Catchment Inundation

	Existing Conditions					Climate Change 2060			Climate Change 2110		
Location	2y	10y	100y	200y	500y	2y	100y	500y	2y	100y	500y
1	10.21	12.26	12.80	12.91	13.03	10.53	12.88	13.10	10.81	12.92	13.16
2		10.62	10.79	10.84	10.94		10.83	10.99		10.87	11.03
3		8.75	9.26	9.40	9.61		9.36	9.73		9.44	9.82
4		9.48	9.70	9.78	9.91	9.31	9.76	9.99	9.34	9.81	10.06
5	15.01	15.26	15.46	15.53	15.64	15.04	15.50	15.70	15.07	15.53	15.75
6					15.44		0.00	15.45		15.39	15.46
7		12.51	12.60	12.66	12.79		12.63	12.88		12.66	12.96
8		10.37	10.68	10.82	10.99		10.76	11.09		10.83	11.16
9		0.00	9.81	9.85	9.91		9.83	9.94		9.85	9.98
10		0.00	8.00	8.11	8.26		8.07	8.33		8.13	8.40
12		10.01	10.06	10.08	10.10		10.07	10.12		10.08	10.13
13		0.00	6.12	6.24	6.36		6.23	6.42		6.32	6.60
15	3.75	3.97	4.09	4.12	4.16	3.80	4.34	4.94	3.84	5.07	5.76
16	2.85	3.45	3.66	3.72	3.81	2.97	4.38	4.92	2.99	5.06	5.75
17		2.67	3.52	3.44	3.79	2.79	4.47	4.91	3.27	5.05	5.74
18	2.75	2.96	3.41	3.35	3.68	2.84	4.68	5.21	3.29	5.26	5.88
19		8.40	8.49	8.56	8.70	8.31	8.54	8.78	8.33	8.58	8.86
20		0.00	6.63	6.68	6.76		6.66	6.80		6.69	6.84
21				6.17	6.28		6.17	6.36		6.26	6.57
22		8.78	8.97	9.01	9.10		9.00	9.15		9.03	9.20
23		8.24	8.77	8.86	8.94	7.35	8.83	8.99	7.38	8.86	9.03
24		7.91	8.48	8.58	8.66	0.00	8.55	8.71		8.58	8.76
25		6.94	8.62	8.78	8.90	0.00	8.73	8.97		8.78	9.04
26		10.39	10.50	10.54	10.59	0.00	10.53	10.62	10.23	10.54	10.66
27		13.93	14.27	14.36	14.48	0.00	14.33	14.54	13.45	14.37	14.59
28	14.42	14.60	15.04	15.15	15.30	14.43	15.11	15.38	14.44	15.16	15.45
30				5.17	5.39			5.61		5.70	6.14
31			4.70	4.71	4.75		5.26	5.56		5.63	5.99
32			3.59	3.61	3.64		4.61	5.05		5.13	5.76
33			3.36	3.32	3.59		4.64	5.13	3.23	5.18	5.79
34		9.70	9.92	10.02	10.19		9.99	10.29		10.05	10.37
35		8.42	8.73	8.82	8.93		8.78	8.99		8.82	9.03
37		6.40	6.74	6.80	6.86		6.78	6.88		6.81	6.91
38		12.41	12.59	12.63	12.70	12.21	12.61	12.74	12.25	12.64	12.77

* Locations indices shown in Figure C.6.1

Table C.6.4 – Port Hedland Area Peak Flood Levels (mAHD) due to Ocean Inundation

	Existing Conditions					Climate Change 2060			Climate Change 2110		
Location	2y	10y	100y	200y	500y	2y	100y	500y	2y	100y	500y
40			4.72	4.86	4.94		4.97	5.31		5.49	5.87
41	3.12	3.64	4.73	4.94	5.13	3.57	5.16	5.49	4.13	5.6	6.11
42	3.28	3.73	4.71	4.93	5.12	3.67	5.15	5.48	4.17	5.59	6.1
43	3.34	3.82	4.71	4.94	5.13	3.73	5.2	5.53	4.21	5.67	6.14
44	3.33	3.82	4.72	4.93	5.12	3.73	5.17	5.51	4.22	5.65	6.15
45	3.31	3.76	4.97	5.15	5.42	3.71	5.33	5.78	4.2	5.82	6.36
46	3.19	3.6	4.91	5.12	5.4	3.58	5.29	5.79	4.11	5.81	6.39
47	3.22	3.64	4.9	5.11	5.39	3.61	5.27	5.78	4.12	5.8	6.37
48	3.06	3.57	4.96	5.14	5.46	3.54	5.33	5.86	4.11	5.86	6.47
49		3.55	4.84	5.04	5.36	3.52	5.23	5.78	4	5.78	6.4
50	2.93	3.23	4.49	4.63	5.18	3.31	5	5.66	3.89	5.6	6.25
51		3.65	4.7	4.92	5.09	3.61	5.16	5.49	4.17	5.63	6.14
52	3.18	3.7	4.72	4.95	5.13	3.67	5.19	5.52	4.22	5.65	6.13
53	2.84	3.22	4.42	4.55	5.08	3.26	4.91	5.54	3.83	5.52	6.18
54	2.94	3.25	4.48	4.63	5.18	3.32	4.99	5.65	3.89	5.6	6.24
55					5.03			5.26		5.04	5.93
56	3.5	4.8	5.88	6.11	6.77	3.91	6.29	7.17	4.4	6.78	7.75
57		4.78	5.85	6.09	6.74		6.24	7.13	4.39	6.75	7.72
58	3.47	3.89	4.8	4.94	5.36	3.86	5.17	5.73	4.33	5.66	6.26
59	3.31	4.59	5.59	5.89	6.29	3.71	6.03	6.79	4.21	6.49	7.39
60			4.51	4.75	4.98		5.04	5.46	3.97	5.62	6.15
61			4.51	4.83	5.02		5.11	5.44	3.25	5.6	6.12
62		3.64	4.68	4.92	5.09	3.61	5.16	5.49	4.16	5.63	6.14
63				4.06	4.67		4.57	5.18		5.26	5.94
64		2.83	4.24	4.36	4.87	2.97	4.73	5.33	3.66	5.37	6.08
65		3.13	4.29	4.38	4.73	2.89	4.66	5	3.36	5.03	5.86

C.6.1.1 Duration of Inundation

Hydrograph plots, water level versus time for reporting locations near roads are shown below in **Chart C.6.1** to **Chart C.6.3**. The time of inundation above a specific level can be inferred from these charts. Where the charts do not fall back to the starting value, water levels have not yet receded from that location. The reporting stations of interest for the catchment inundation event are as follows, 1, 2, 3 10, 12, 21 & 26 (see **Figure C.6.1**). The reporting stations of interest for the ocean inundation events are 58, 60, 61, 62, 63, 64 & 65(see **Figure C.6.2**).

Charts Chart C.6.2 Chart **C.6.7** show the water level versus time for the catchment inundation scenario at seven reporting stations. The **Chart C.6.8** to **Chart C**.6.13 show the water level versus time for the ocean inundation scenarios at seven reporting stations.

Chart C.6.1 Catchment Inundation at Reporting Station 1

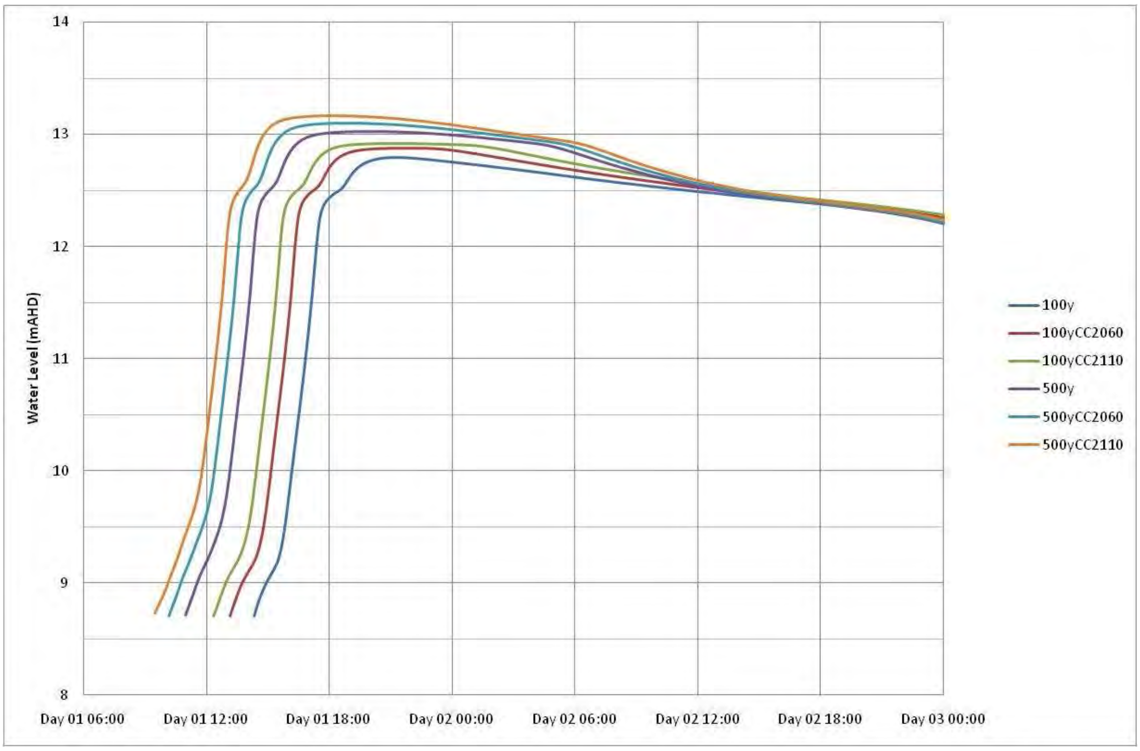


Chart C.6.2 Catchment Inundation at Reporting Station 2

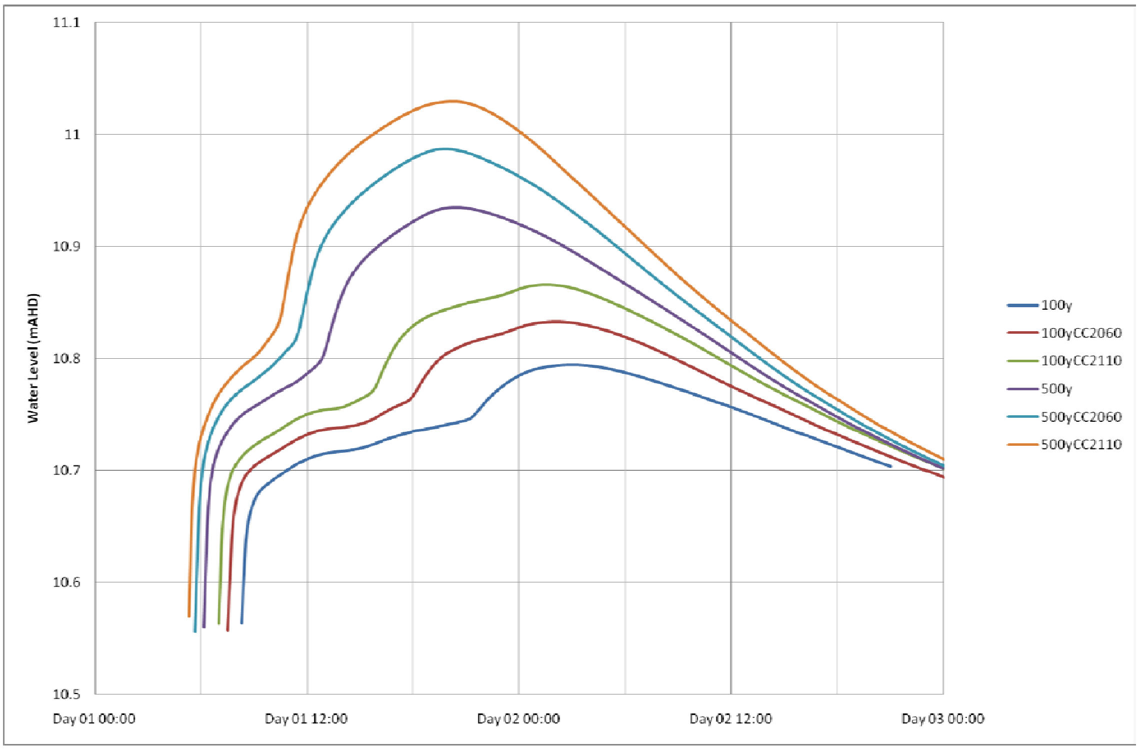


Chart C.6.3 Catchment Inundation at Reporting Station 3

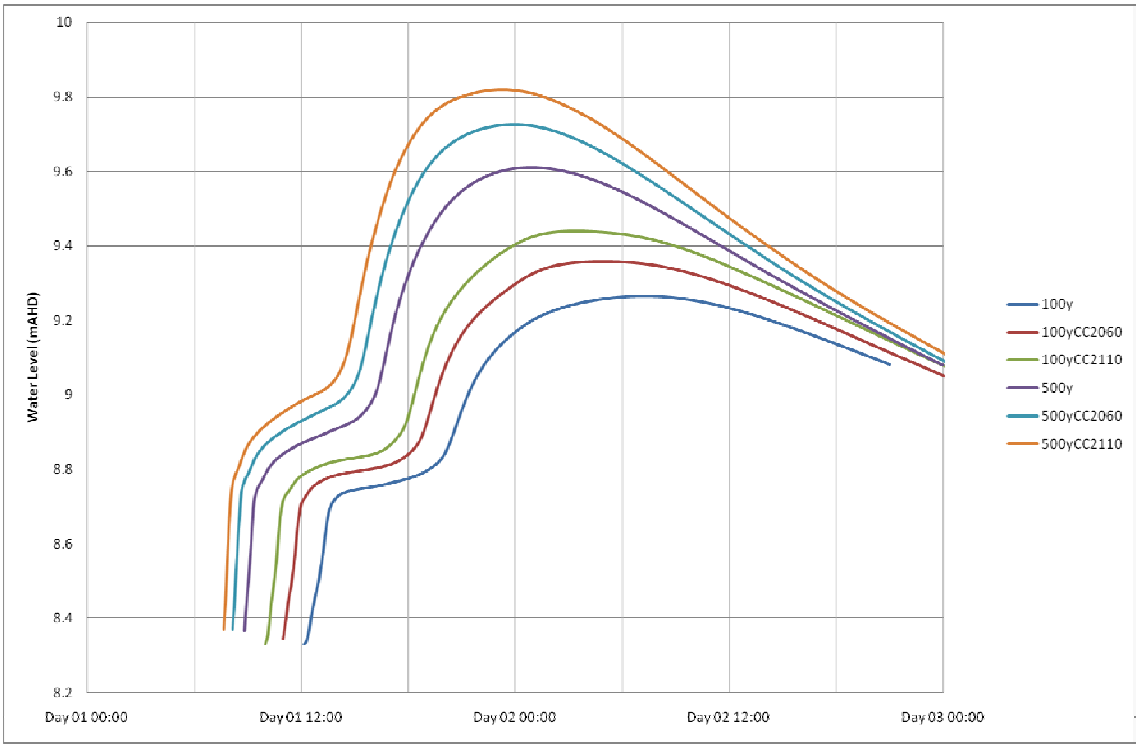


Chart C.6.4 Catchment Inundation at Reporting Station 10

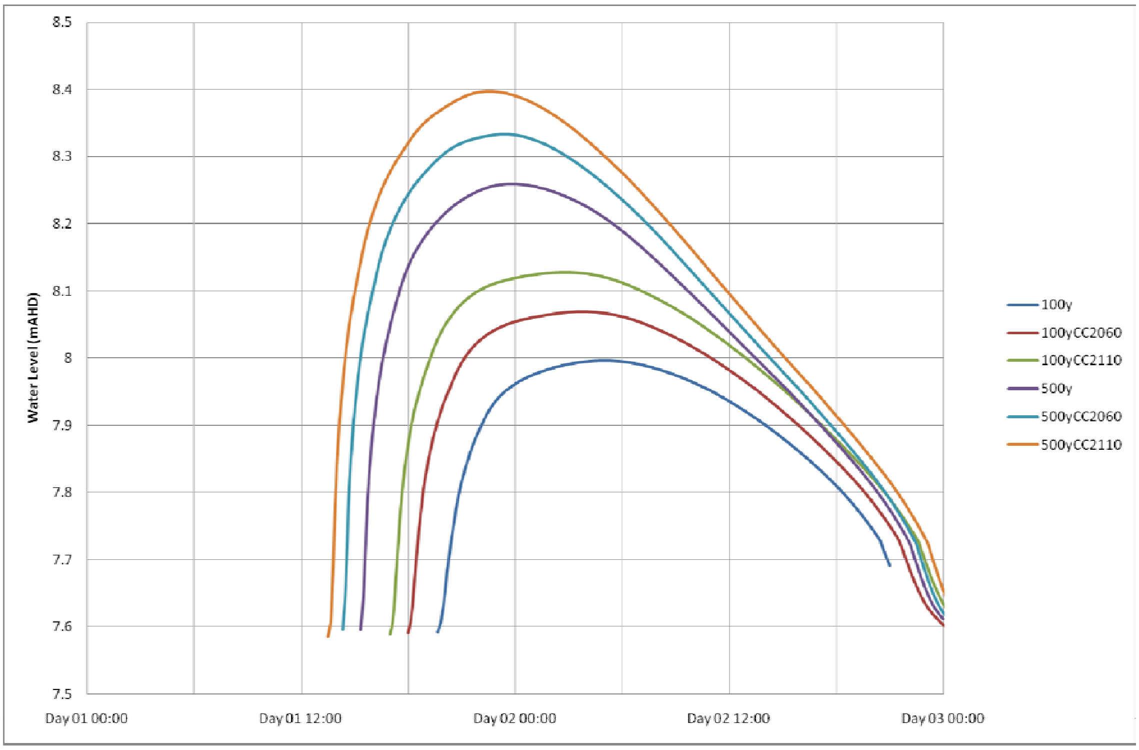


Chart C.6.5 Catchment Inundation at Reporting Station 12

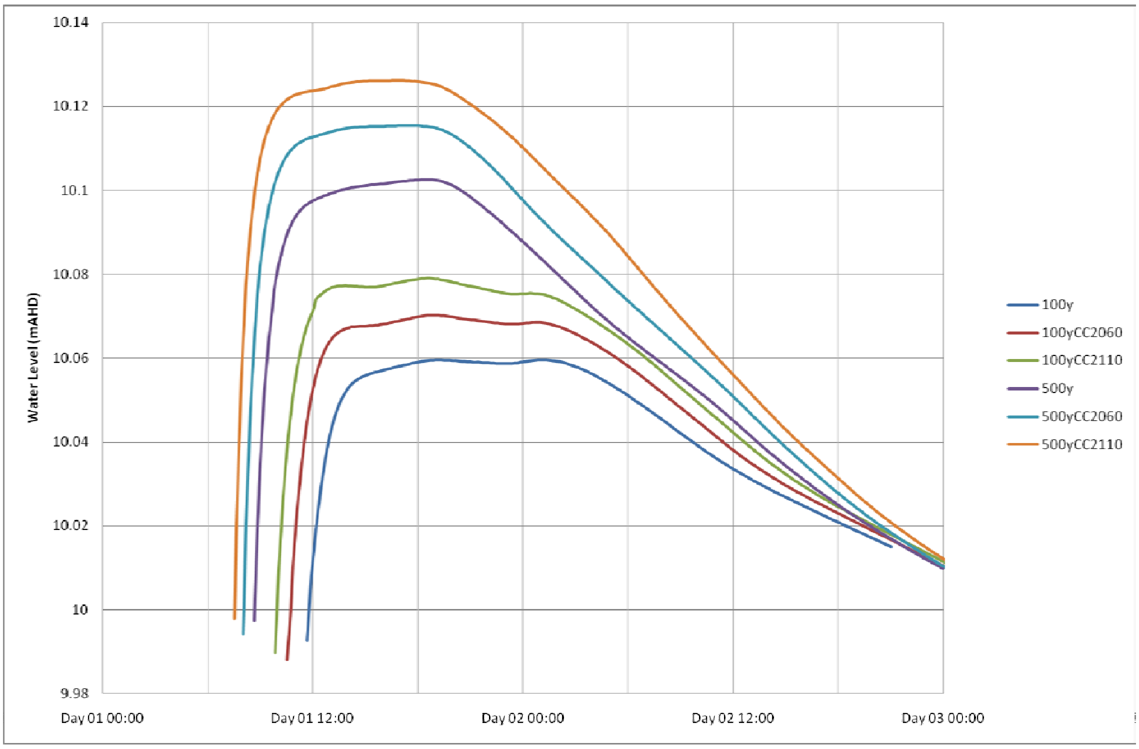


Chart C.6.6 Catchment Inundation at Reporting Station 21

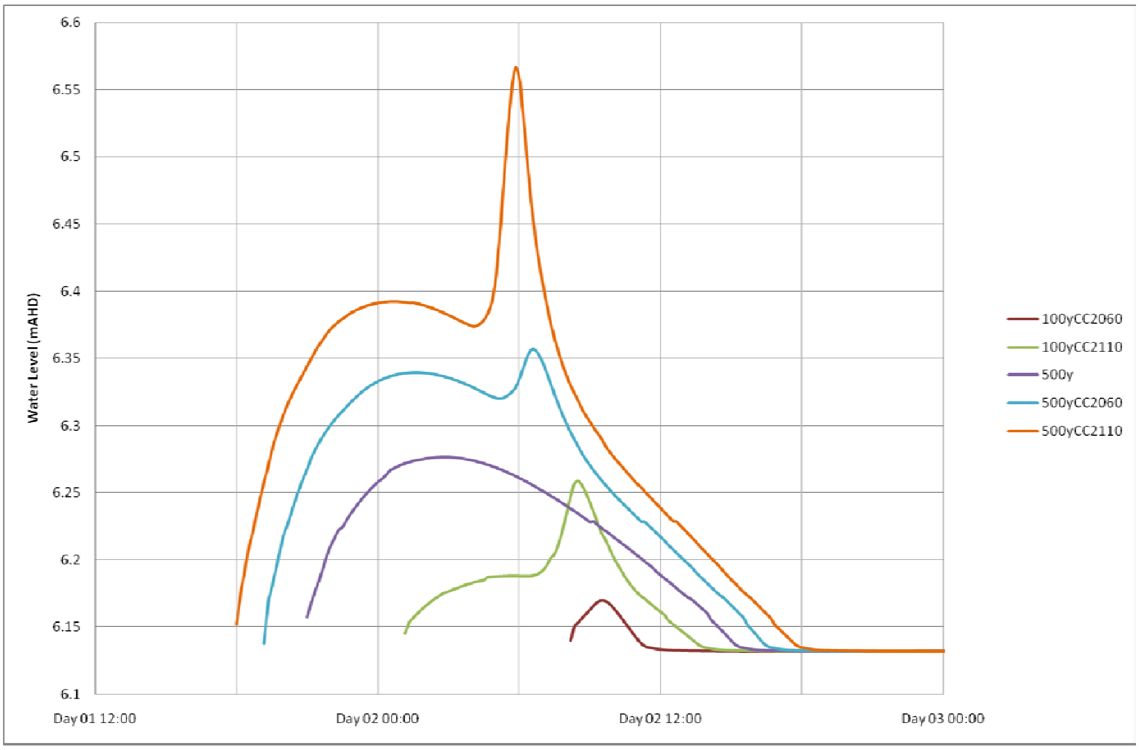


Chart C.6.7 Catchment Inundation at Reporting Station 26

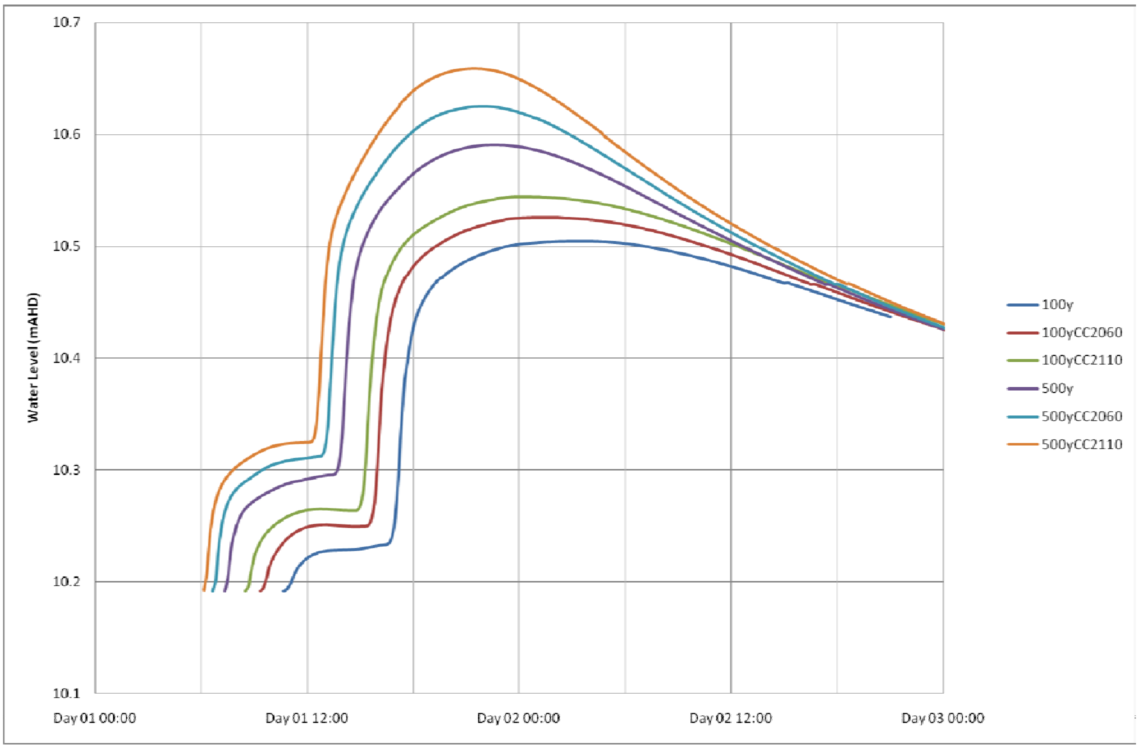


Chart C.6.8 Ocean Inundation at Reporting Station 60

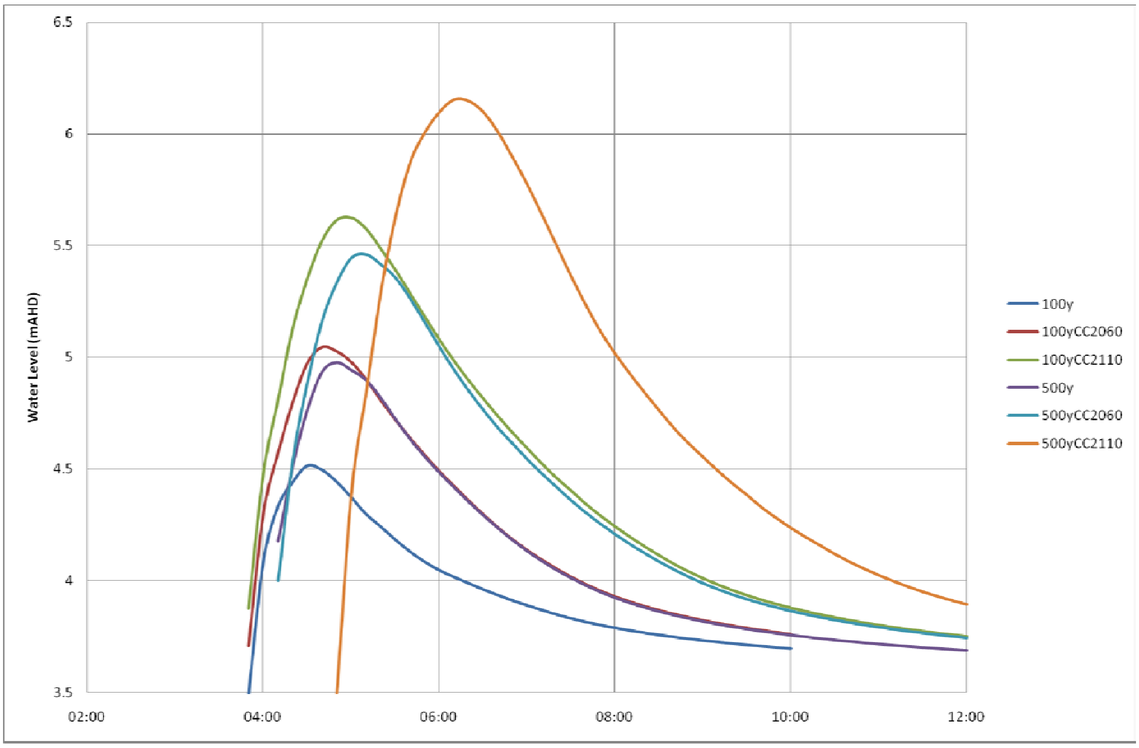


Chart C.6.9 Ocean Inundation at Reporting Station 61

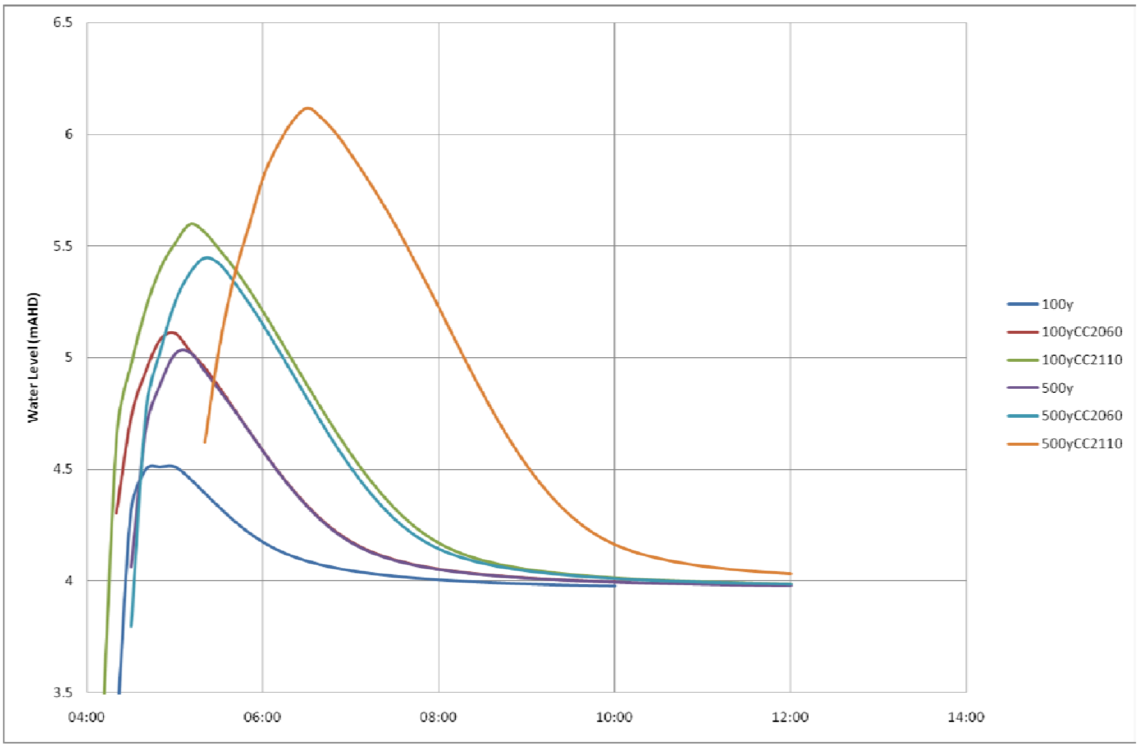


Chart C.6.10 Ocean Inundation at Reporting Station 62

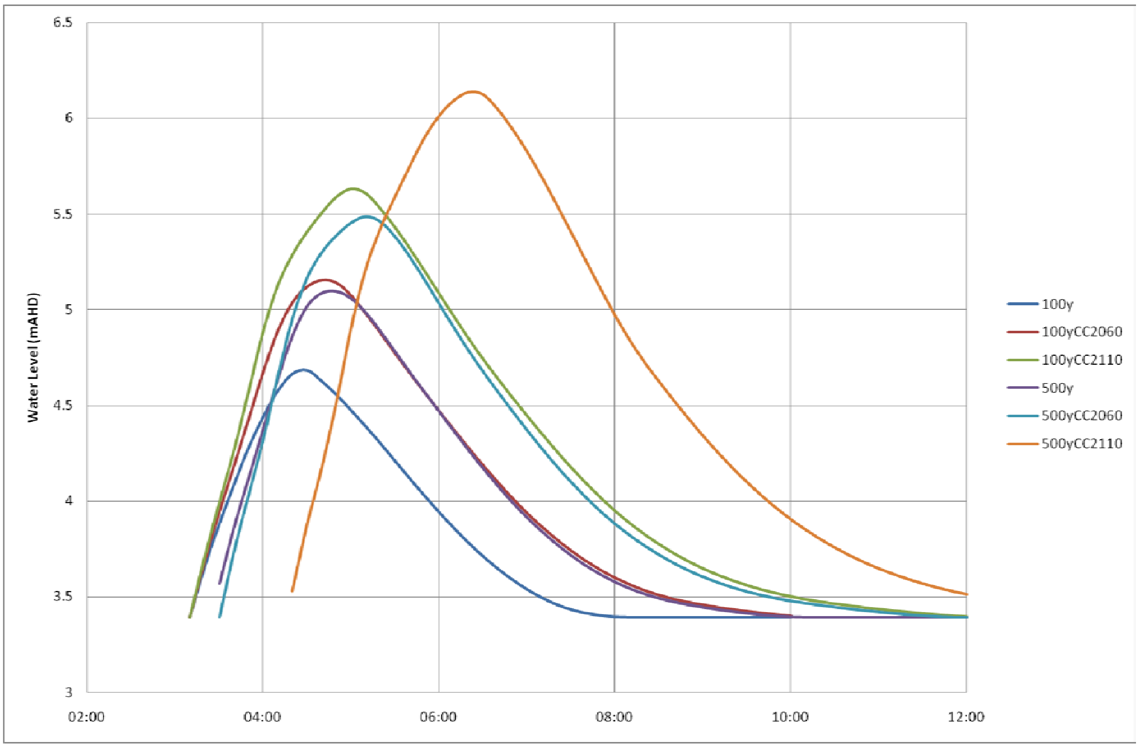


Chart C.6.11 Ocean Inundation at Reporting Station 63

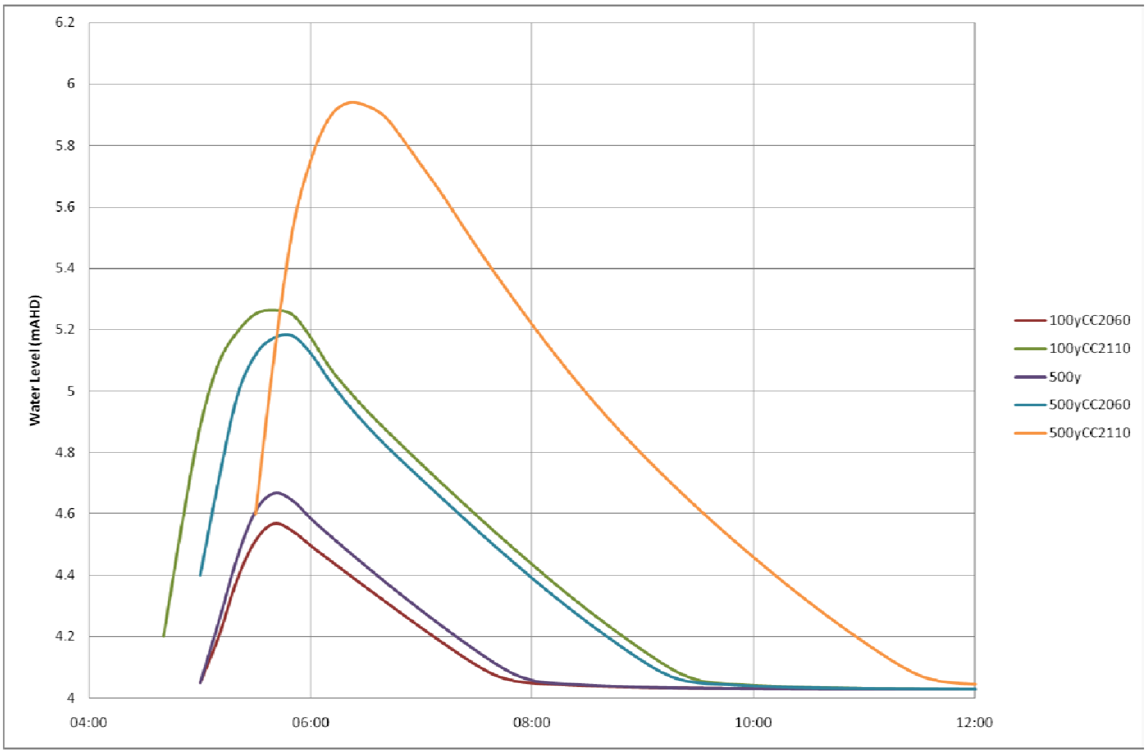


Chart C.6.12 Ocean Inundation at Reporting Station 64

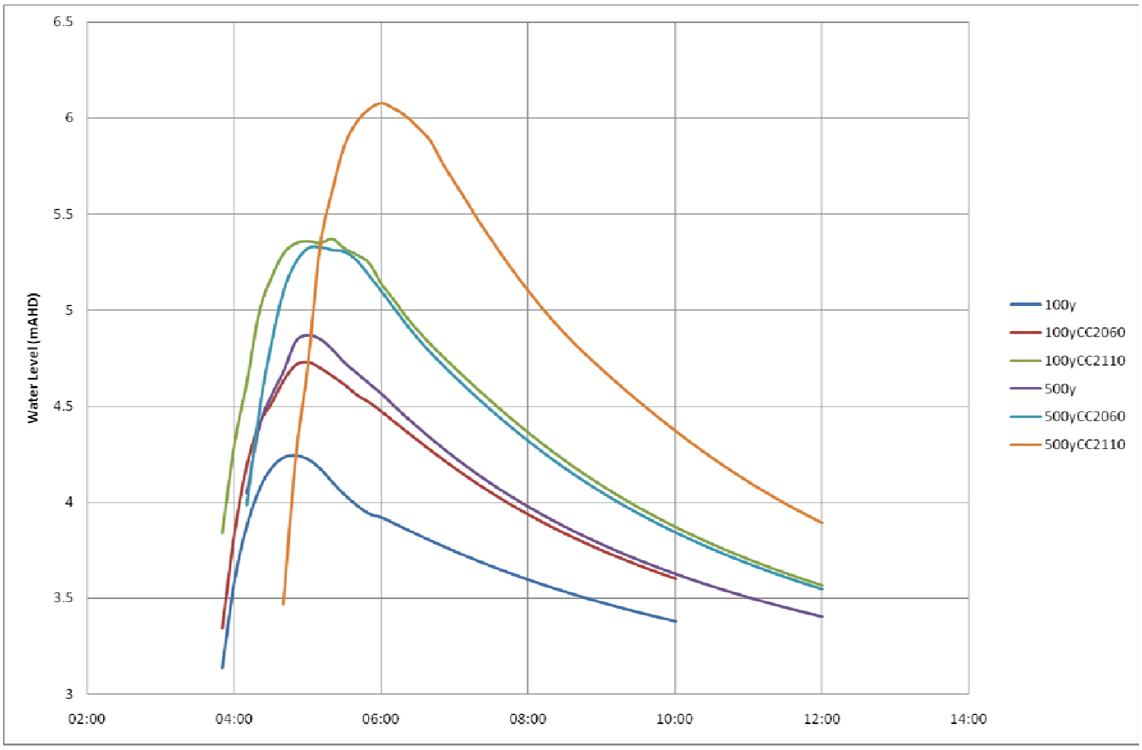
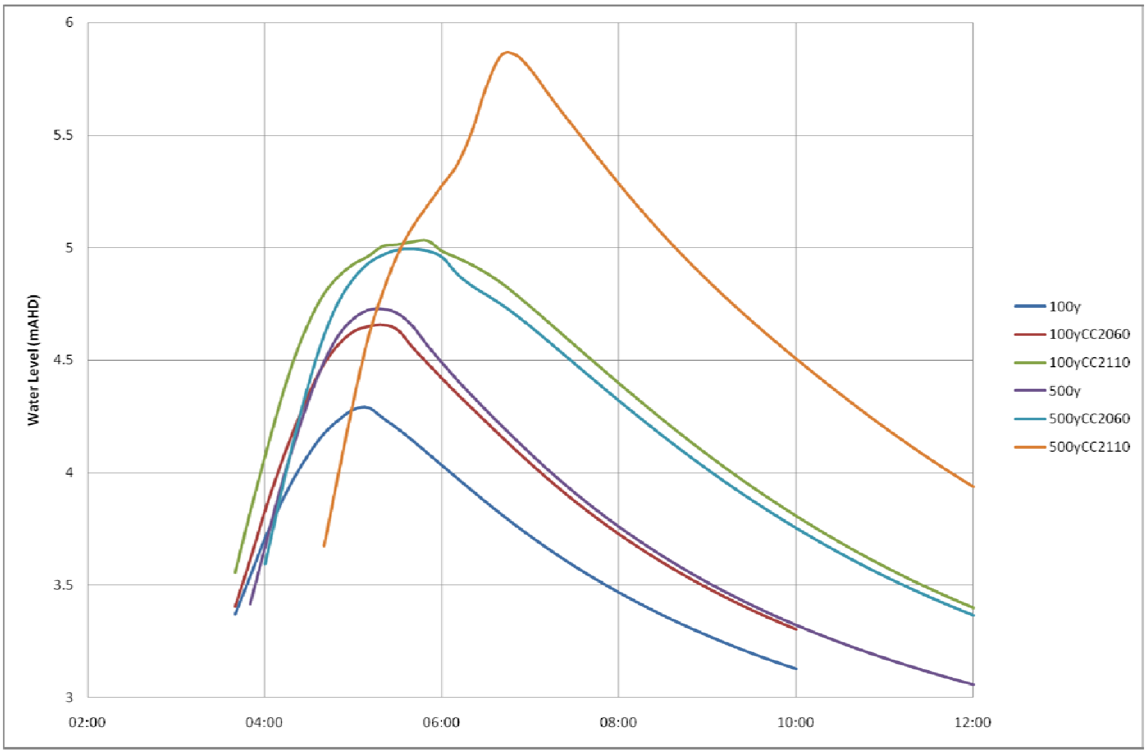


Chart C.6.13 Ocean Inundation at Reporting Station 65



C.6.2 Shellborough

During the 100 year ARI ocean storm event with existing climate conditions, the majority of the Shellborough area is inundated. In the absence of a combined event, the 100 year tidal inundation results in a far greater inundation extent when compared to the catchment scenarios modelled in this study. This is due to the majority of the Shellborough region being very low lying with little to no protection from the ocean, and relatively small catchment. Flood depths are significant in all modelled events. It is unlikely that any development could occur on the Shellborough Estuary, although parts of the dune system are not inundated in large storm events.

A number of maps have been produced showing the maximum flood extents and depths for each modelled flood event. Note that all figures have been filtered to remove flood depth less than 0.02 m. Water surface levels (m AHD) are also shown at a number of key locations.

Table C.6.5 details the mapping outputs for the project. **Table C.6.6** summarises the maximum water levels at various locations (shown in **Figure C.6.3**) for the catchment inundation events. **Table C.6.7** summarises the maximum water levels at the same locations for the ocean inundation events.

Table C.6.5 – Index of Flood Inundation Maps –Shellborough

Map Index Numbers	Output Description
Map S01 – Map S05	Existing Conditions Catchment Inundation Maps
Map S06 – Map S08	2060 Climate Change Catchment Inundation Maps
Map S09 – Map S11	2110 Climate Change Catchment Inundation Maps
Map S12 – Map	Existing Conditions Ocean Inundation Maps
Map S17 – Map S19	2060 Climate Change Ocean Inundation Maps
Map S20 – Map S22	2110 Climate Change Ocean Inundation Maps

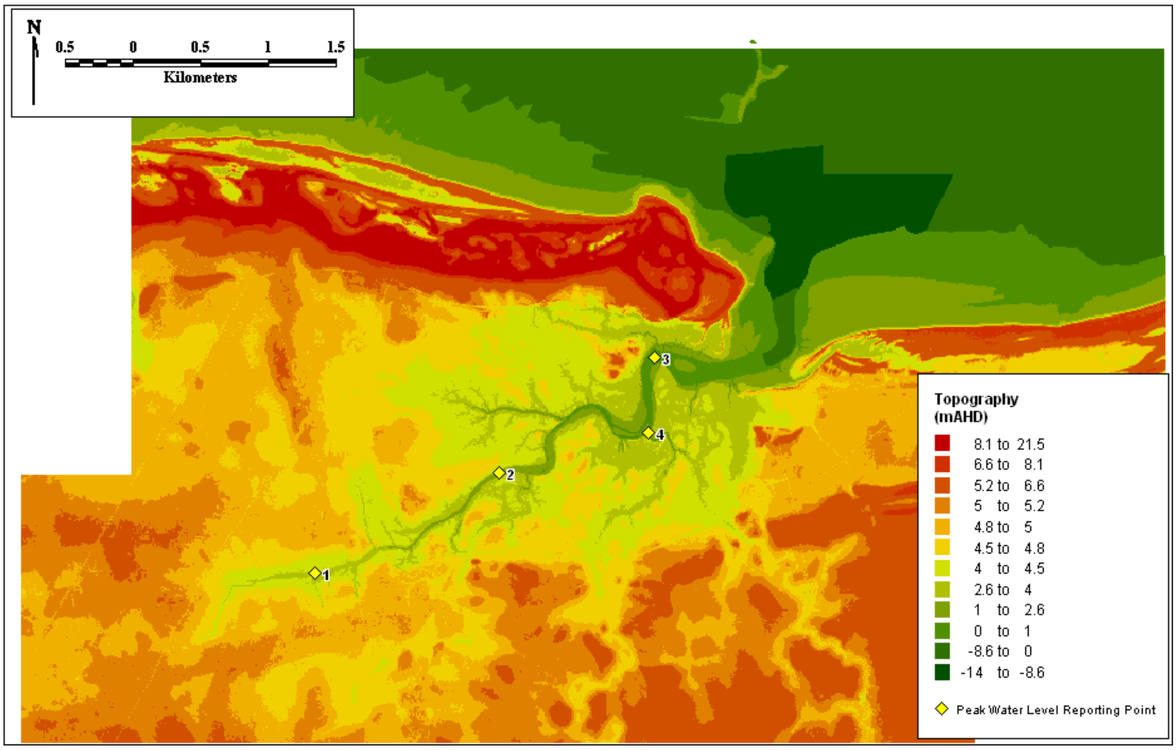


Figure C.6.3 – Water Level Reporting Locations– Shellborough

Table C.6.6 – Shellborough Peak Flood Levels (mAHD) — Catchment Inundation

	Existing Conditions					Climate Change 2060			Climate Change 2110		
Location	2y	10y	100y	200y	500y	2y	100y	500y	2y	100y	500y
1	4.11	4.45	5.19	5.28	N/A	4.15	5.41	5.70	4.45	5.72	6.02
2	3.50	3.58	5.04	5.08	N/A	3.88	5.45	5.69	4.36	5.84	6.07
3	3.45	3.45	5.02	5.02	N/A	3.85	5.50	5.71	4.34	5.96	6.15
4	3.47	3.48	5.02	5.03	N/A	3.85	5.49	5.71	4.35	5.93	6.13

Table C.6.7 – Shellborough Peak Flood Levels (mAHD) – Ocean Inundation

	Existing Conditions					Climate Change 2060			Climate Change 2110		
Location	2y	10y	100y	200y	500y	2y	100y	500y	2y	100y	500y
1	3.93	4.57	5.30	5.58	5.72	4.31	5.53	6.03	4.66	5.96	6.69
2	3.92	4.61	5.60	5.94	6.09	4.31	5.88	6.34	4.71	6.24	6.90
3	3.91	4.70	5.81	6.25	6.43	4.31	6.14	6.73	4.80	6.60	7.30
4	3.91	4.68	5.75	6.17	6.33	4.31	6.07	6.61	4.78	6.49	7.16

C.6.2.1 Duration of Inundation

Chart C.6.14 shows the water level versus time for the catchment inundation scenarios at reporting station 4 (shown in ***Figure C.6.3***). **Chart C.6.15** shows the water level versus time for the ocean inundation scenarios at reporting station 4 (shown in ***Figure C.6.3***). The location reported is influenced by tidal action during the catchment inundation event. The peak level represents the time at which the the catchment is fully contributing flows at the estuary.

Chart C.6.14 Catchment Inundation at Reporting Station 4

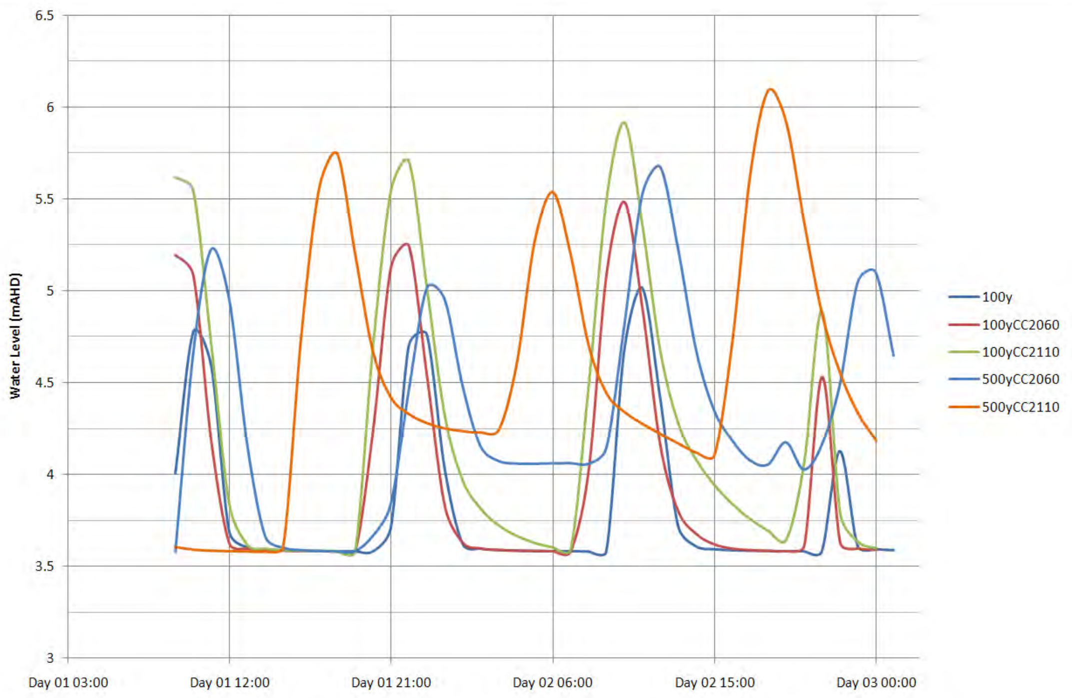
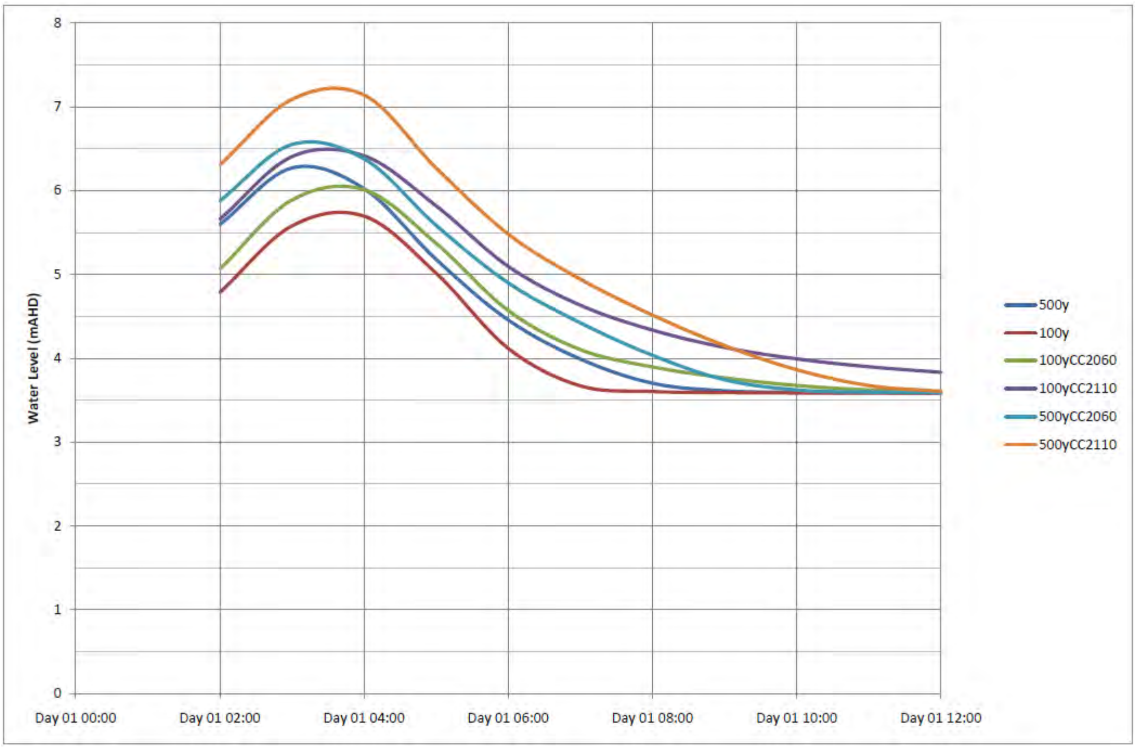


Chart C.6.15 Ocean Inundation at Reporting Station 4



C.7 DEVELOPMENT AREA ANALYSIS

Landcorp have provided a preliminary map of proposed development locations in Port Hedland (**Figure C.7.1**). This plan has been incorporated into the hydraulic model to assess the required planning levels for the development of lots. A worst case assessment has been made in the model, by raising all the land in the development area above the 2110 climate change ocean inundation with wave setup level. Using this methodology, the impact of the development on flooding can be assessed. In the proposed area of development, peak levels are a result of the ocean inundation events and as such, only these events have been modelled. The 100 and 500 year events have been used for the assessment and include wave setup. Climate change conditions at 2060 and 2110 (with wave setup) have also been assessed. Maps P33 and P34 show the 2110 inundation levels at the development.

Figure C.7.1 – Proposed Development Area – Port Hedland

Results of the modelling have been provided at the McGregor Road (Port Hedland), Pretty Pool and Four Mile Creek boundaries of the East Port Hedland proposed development area. **Table C.7.1** shows the expected water levels (m AHD) at the development site.

Table C.7.1 – Expected Water Levels (m AHD) at Development Sites

Location	Existing		Existing Developed		Climate Change 2110 Developed	
	500y	100y	500y	100y	500y	100y
Four Mile Creek	5.08	4.42	5.58	4.81	6.58	5.89
Pretty Pool*	6.74	5.85	6.82	5.89	7.79	6.82
Port Hedland	-	0.00	5.03	-	6.15	5.04

* Wave Setup Water Level incorporated in Pretty Pool water levels

C.8 CONCLUSIONS

There is significant inundation in the Port Hedland and Shellborough regions from both catchment and ocean derived flood events. The study has concluded:

- Cross catchment flows have a significant impact on flooding behaviour in the South Creek floodplain. Flows from South West Creek flow into the South Creek floodplain upstream of Shellborough and have an impact on the Shellborough Township.
- At Wedgefield, the dominant flood mechanism in the 100-year ARI event is catchment flooding. On the northern edge of the development, levels resulting from ocean inundation in the 100-year ARI event are approximately 0.7m lower than from the catchment inundation.
- At Port Hedland, ocean flooding is the dominant flood mechanism. The frontal dune is overtopped in the 500-year ARI ocean inundation event. This feature is consistent with anecdotal reports from previous storm surge events.
- Flows input to the model are generally consistent with those estimated in previous studies and due to the higher resolution 2D model the effect of cross catchment flows are explicitly included in this model. The use of time varying boundary conditions also properly accounts for storage effects behind roads, railways and other hydraulic features in the catchment.
- At Shellborough, the dominant flood mechanism is the ocean storm surge. Significant areas inland are inundated and access is likely to be severely restricted in this area under all flood conditions. The old Shellborough settlement site is not inundated in the 100-year ARI event.
- The impact of climate change on ocean and catchment flooding is significant, with increases in water level and time of inundation.

The results of the study and the model framework allow for future investigations into flooding in the greater Port Hedland region. This could include flood response planning, flood hazard analysis, flood mitigation options and development assessment. The model structure enables detailed sub-grids to examine individual areas in greater detail without recreating the entire model.

C.9 QUALIFICATIONS

The investigation and modelling procedures adopted for this study follow current best practice and considerable care has been applied to the preparation of the results. However, model set-up and calibration depends on the quality of data available.

This report should be read with regard to the following assumptions and qualifications:

- A number of assumptions have been made in the text of the document pertinent to modelling. The model is only valid for those assumptions stated in the text.
- Unless otherwise noted, the local drainage network has not been modelled as part of this study.
- The results of the study are based on the data sources listed in this document. The accuracy of the analysis undertaken as part of this project is dependent on the accuracy of these data sources. It should be noted that survey data of many hydraulic structures was not available and engineering judgement was used to estimate unknown data.
- Our analysis and methodology have been specifically designed for the particular requirements (as specified in the project brief) of this project. Study results should not be used for purposes other than those for which they were prepared without further input and advice from Cardno.

C.10 REFERENCES

Chow, (1973) *Open-Channel Hydraulics*, McGraw-Hill International Editions, Singapore.

Stelling, G.S. Kernkamp, H.W.J and Laguzzii M.M., (1999) *Delft Flooding System - A Powerful Tool for Inundation Assessment Based Upon a Positive Flow Simulation*, *Hydroinformatics Conference, Sydney NSW.*

Deltares, (2011) *Sobek Advanced Version 2.12.002.* Deltares, Delft, The Netherlands.

Pilgrim, (1987) Australian Rainfall and Runoff, Institute of Engineers Australia, Canberra

Appendix D

Shoreline Stability Assessment

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Final

Job Number: LJ15014

mber: Rep1022p/Appendix D

10 August 2011

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Document Control: Port Hedland Coastal Vulnerability Study – Shoreline Stability Assessment Report

Version	Date	Author		Reviewer	
		Name	Initials	Name	Initials
1 – Draft	1 April 2011	Jim Churchill	JWC	David Taylor	DRT
2 - Final	10 August 2011	Jim Churchill	JWC	David Van Senden	DVS

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TABLES:

FIGURES:

ABBREVIATIONS AND GLOSSARY:

AHD	Australian Height Datum which is the standard vertical elevation datum for Australia. At Port Hedland, AHD is +3.902m above Chart Datum at the permanent tide gauge.
ARI	Average Recurrence Interval (years)
BoM	Bureau of Meteorology
CD	Commonly referred to water level datum which at Port Hedland is approximately equal to the Lowest Astronomical Tide (LAT) level.
CPS	Coastal Processes Setback
HAT	Highest Astronomical Tide - at Port Hedland is 3.6m AHD
HSD	Horizontal Setback Datum
MSL	Mean Sea Level
MHWS	Mean High Water Springs at Port Hedland is 2.8m AHD
SLR	Sea Level Rise
SPP 2.6	Refers to Statement of Planning Policy No. 2.6 which is referred to as the State Coastal Planning Policy which was Gazetted in June 2003.
WAPC	Western Australian Planning Commission

D.1 INTRODUCTION

As part of the Port Hedland Coastal Vulnerability Study, a component of the study required that a shoreline stability assessment be undertaken for key sites within the study area. The three areas covered by this study are the shoreline surrounding the Port Hedland Township, 'Site 2' (also referred to as the 'Bus Stop') located on the coastal fringe 13km to the west of Port Hedland, and the Shellborough area approximately 90km northeast of Port Hedland.

Figure D.1.1 presents a plan view of the study area. Assessment of coastal processes affecting each of the three areas has been undertaken based on available data sources and methodology outlined in the WA Statement of Planning Policy No. 2.6: State Coastal Planning Policy (SPP 2.6).

The Western Australian Planning Commission (WAPC) released SPP 2.6 in June 2003 to assist land use planning and development issues specifically as they relate to the protection of the coast. **Schedule One** of SPP 2.6 provides direction for calculating the appropriate Coastal Processes Setback (CPS) distance for the siting of development on the Western Australian coast. The CPS provides a buffer zone between the shoreline and development in which coastline changes in the short term (severe storms), the medium term (shoreline movement) and the longer term (sea level rise and fluctuation of natural processes) can occur.

The calculation of the CPS distance is the combined result of the following factors :-

1. (S1) Distance For Absorbing Acute Erosion (Extreme Storm Sequence);
2. (S2) Distance to Allow for Historic Trend (Chronic Erosion or Accretion); and
3. (S3) Distance to Allow for Sea Level Change.

SPP 2.6 states that the CPS should be defined on a case by case basis including S1, S2, S3 where relevant. Each of these factors is assessed in this report by the following methods :-

S1. Selected profile modelling of storm erosion using the model system SBEACH;
S2. Analysis of historical photogrammetric data and aerial photography to establish historical shoreline changes; and
S3. Application of the Brunn Rule based on a vertical SLR of 0.30m in 2060 and 0.90m for the year 2110.

Additionally, regions north of latitude 30° are deemed cyclone prone areas under SPP2.6 and the CPS needs to take account of areas potentially inundated by storm surge associated with a Category 5 cyclone, tracking to maximise its potential. The results from the detailed modelling undertaken in the Ocean Inundation Assessment (**Appendix A**) have been utilised to inform this process.

The CPS is applied from a Horizontal Setback Datum (HSD), a fixed line which is defined based on the type of coastline being assessed. For sandy shorelines this datum is the landward limit of annual beach change which is identified by the toe of the erosion scarp on an eroding coast, or the seaward extent of ephemeral vegetation for an accreting coast. For rocky shorelines the HSD is taken as the normal landward limit of sea action (WAPC 2003).

The WAPC is currently reviewing SPP 2.6, and in September 2010 released a position statement with amended Sea Level Rise (SLR) estimates for the 100 year planning period up to 2110. These revisions have been adopted for this study and are +0.3m for a 50-year (2060) planning period and +0.9m for a 100-year (2110) planning period.

Figure D.1.1: Study Sites Port Hedland, Site 2 (The Bus Stop) and Shellborough.

D.2 STUDY SITE DESCRIPTION

D.2.1 Port Hedland

This study investigates the Port Hedland shoreline from Airey Point adjacent the entrance to the harbour, east along the main ocean front of the town to Cooke Point. Additionally the shoreline from Cooke Point going south to the Pretty Pool estuarine entrance is examined. **Figure D.2.1** highlights the extents of the study area.

Figure D.2.1: Port Hedland Shoreline Study Extents.

The town of Port Hedland has been established along a sand capped calcarenite barrier island at the mouth of Stingray Creek, a natural port which has been significantly modified to accommodate vessels associated with the regions iron ore operations (Short 2006). Most of the shoreline has been built upon with the development of the Port Hedland township, and a mixture of residential, community and commercial interests have made use of the ocean front areas.

The shoreline of the region is composed of a number of different beach types with varying physical characteristics. To differentiate these various sub regions, the study area has been divided into 14 zones in which common physical characteristics are found. The delineation separates sandy beaches from rocky areas, and natural features such as Cooke Point and the Pretty Pool estuary. The establishment of these zones provides the framework for the analysis of historical shoreline movement in sections to follow.

The 14 zones are shown on **Figure D.2.2** and **Figure D.2.3** with a general shoreline description provided for each of the zones on **Table D.2.1**.

Figure D.2.2: Port Hedland Study Area Zones 1 to 6.

Figure D.2.3: Port Hedland Study Area Zones 7 to 14.

Table D.2.1: Analysis Zones in the Port Hedland Town Study Area.

Zone	Shoreline Description
1	At the west of this zone a narrow sandy beach fronted by intermittent mangrove faces Northwest toward Hunt Point. There is a boat ramp and rock groyne, with rockwalls stabilising the carpark / launch area. To the east of the zone around Airey Point there is a hind-beach sand dune ridge and mangroves are present in the intertidal area which is fronted by low tide rocky flats.
2	Rocky back beach area with thin sandy deposits overlying a weathered-rock intertidal shelf. The intertidal region is generally flat extending down into rocky tidal flats.
3	A sandy beach in front of the Port Hedland Yacht Club is fronted by the protected marina basin leading around to the western side of the spoil bank. The steep shoreline of the spoil bank extends north-northeast for approximately 2km, with offshore depth determined by the navigation channel.
4	Protected sandy embayment running along the eastern side of the spoil bank which changes alignment from north to northwest at the base. The shoreline is not as steep as the western side of the spoil bank and is fronted by low tide sand flats
5	Cemetery Beach extends east from the connection point of the spoil bank to the shoreline adjacent Port Hedland Community Park. Calcareous sandstone cliffs dominate the high tide shoreline.
6	Relatively wide sandy beach fronted by sand / rock tidal flats and backed by region of vegetated low dunes with permanent vegetation. Width of dune area decreases toward the east of this zone as the sandy beach tends to a more rocky nature.
7	Shoreline is dominated by a rock platform with sand deposited at regions above the high water line extending up to a low vegetated dune at the rear. Sand / rock tidal flats extend offshore.
8	Shoreline similar to Region 7 with a much larger and vegetated dune system in the undeveloped region leading into Cooke Point.
9	Cooke Point, a major rocky headland that includes a seabed sill that projects northward and acts as a low seabed groyne that holds a low tide beach alignment
10	Narrow high tide beach aligned east and backed by a vegetated foredune. At low tide a wide expanse of ridged sand flats extend offshore with pockets of mangroves in the near shore zone.
11	Northern side of river estuary leading into the Pretty Pool tidal creek. Narrow sandy high tide beach backed by vegetated dune system and fronted by sand flats extending up to 700m offshore.
12	Southern shore of the tidal creek backed by the Pretty Pool subdivision. Sandy shoreline fronted by partial mangrove and backed by a vegetated bank above the high water mark to the development.
13	Southern side of Pretty Pool tidal creek entrance. Highly mobile sand spit dependent upon the alignment of the estuary channel and backed by vegetated dune system.
14	Beach is predominantly aligned to the northeast with a vegetated dune system more extensive than any previous zone. Tidal sand flats extend up to 700m offshore.

Photos taken during a site visit to the area are shown on **Figure D.2.4** and **Figure D.2.5**

Figure D.2.4: Photos Taken During Site Visit - From Top : Zone 1 Looking East at Airey Point, Zone 2 Looking East Toward Spoil Bank, Zone 3 Sheltered Section of Spoil Bank.

Figure D.2.5: Photos Taken During Site Visit - From Top : Zone 3 End of Spoil Bank Looking West, Zone 5 Looking East Toward Cooke Point, Zone 7 Cooke Point Looking West.

Whilst there is significant development along the Port Hedland shoreline, the calculation of the CPS for the region has been made based on the criteria for an undeveloped sandy shore outlined in **Section D.1**.

There is one exception to this method, where the CPS for **Zone 5** is determined based on the criteria in SPP2.6 for a rock shoreline. In SPP2.6 a shoreline is considered to be rock where the impact of sea action is in direct contact with lithified material. SPP2.6 recommends the CPS for rocky shoreline be determined by geotechnical survey which can account for possible erosion over a 100 year period. In the absence of detailed survey, a minimum CPS of 50m is recommended which incorporates the S1, S2 and S3 components of the CPS.

For the 100 year planning period the 50m default value has been applied. Over the 50 year planning period half this value has been adopted (25m).

Port Hedland Spoil Bank

One of the prominent features of the Port Hedland shoreline is the spoil bank which extends out from the coastline in **Zones 3** and **Zone 4** as a low sand spit parallel to the Goldsworthy channel. The spoil bank is a major man-made near shore feature that has developed as a result of dredge spoil from the development of Port Headland Inner Harbour and navigation channel.

Dredging commenced from the mid 1960's when the first iron ore berths were developed. Expansion of the inner harbour facilities and subsequent deepening to accommodate Cape Class vessels required major capital dredging programs in the years 1976 to 1977 (1.9 million m^3) and 1984 to 1987 (12.7 million m^3). Ongoing maintenance dredging has occurred at three to four year intervals with 600,000m^3 removed in each of the periods 1998 to 1999 and 2000 to 2001 (Paul, 2001).

The northern end of the spoil bank curves to the east suggesting that eastward sediment transport is the dominant sediment transport direction. The evolution of the spoil bank between the years 1949 to 2009 is shown on **Figure D.2.6**

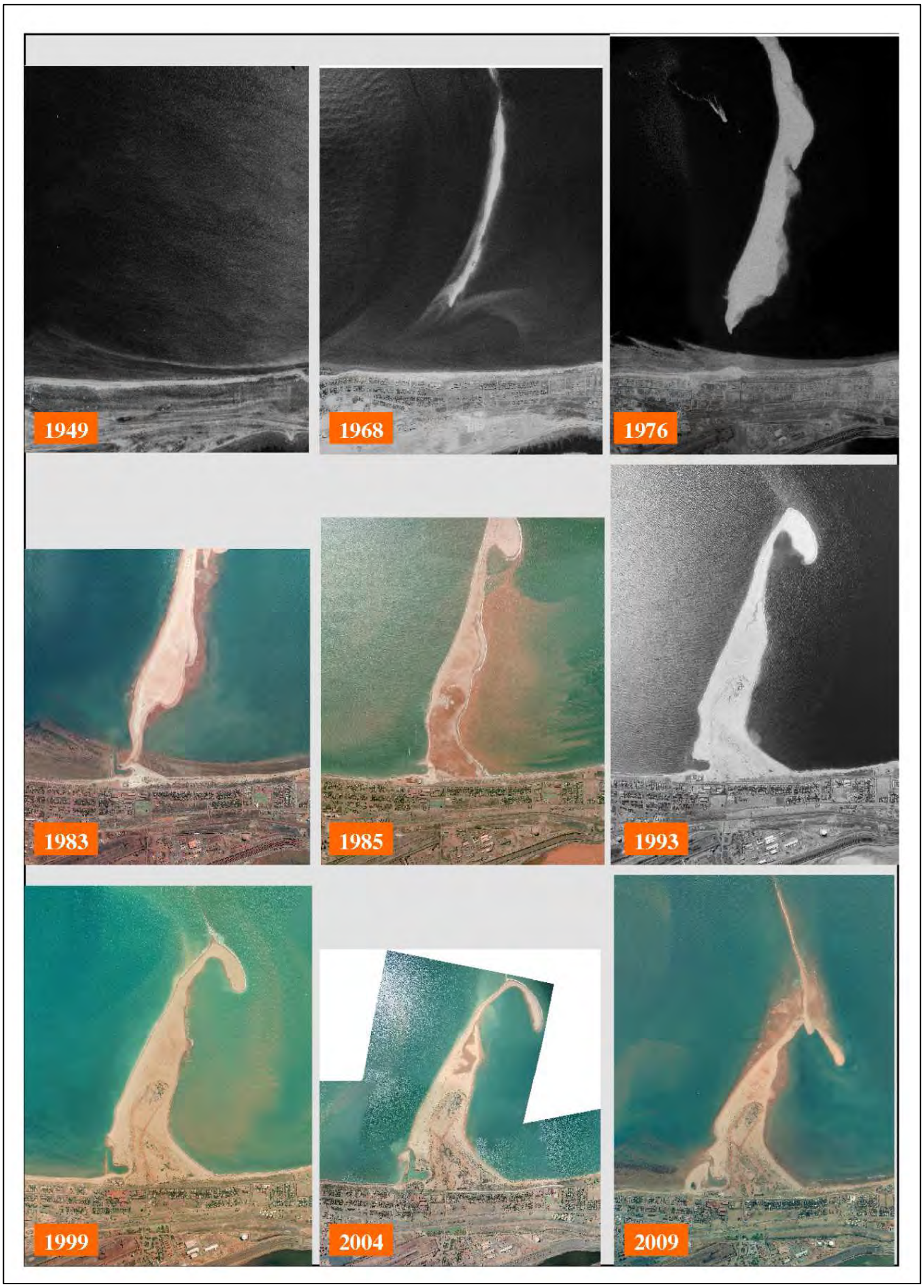


Figure D.2.6: Evolution of the Spoil Bank 1949 to 2009 - Port Hedland.

D.2.2 Study Site 2

Study Site 2 is marked in the release *Hedlands Future Today 2010* which outlines future development priorities for the Town of Port Hedland. The site is highlighted as a camping site called 'The Bus Stop' under the *Coastal Access and Managed Camping Project* which aims 'to improve access to selected recreational areas along the Pilbara coastline and provide infrastructure necessary to accomodate and manage visitors to these sites in the future'.

The site is approximately 13km west of the Town of Port Hedland and is shown on **Figure D.2.7.**

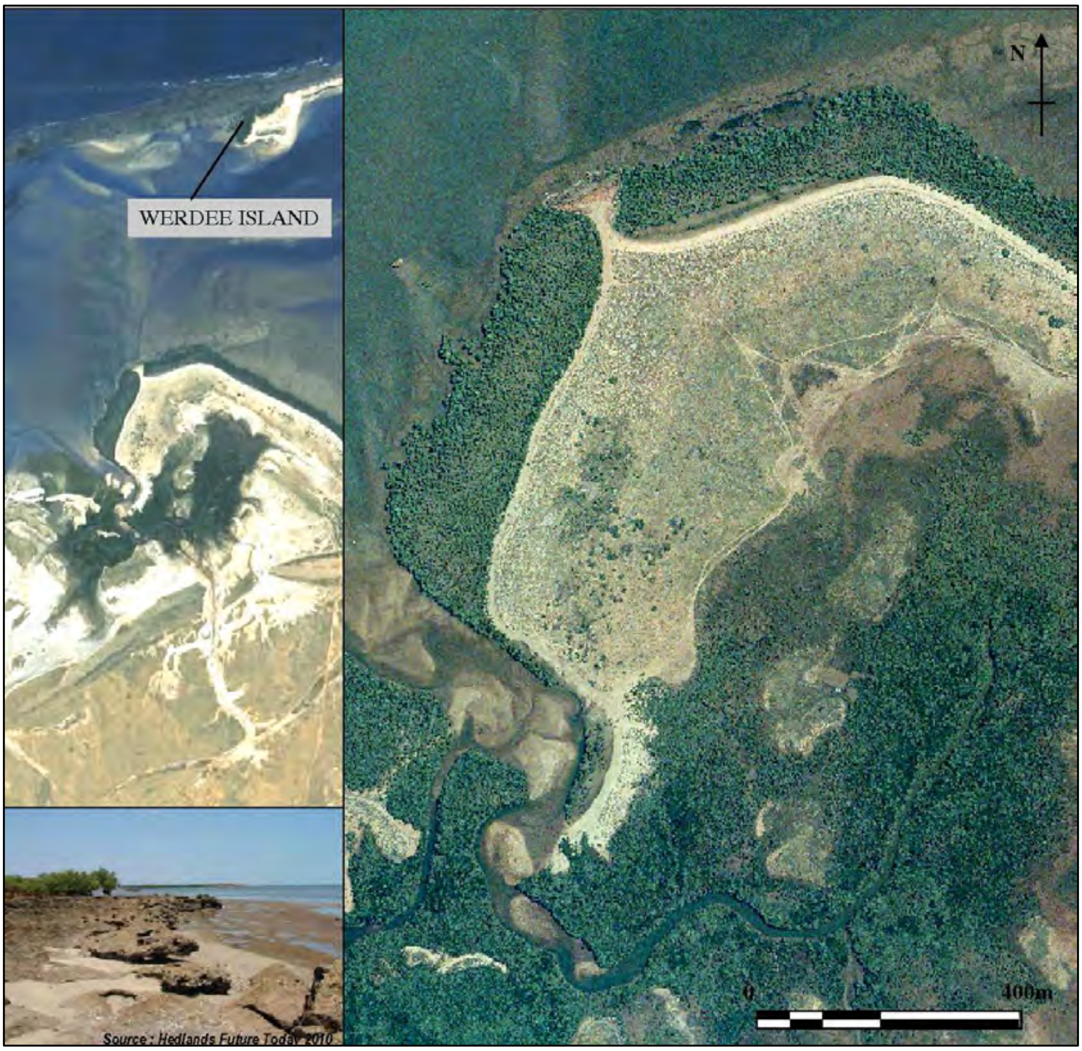


Figure D.2.7: Study Site 2 - 'The Bus Stop'.

The study site is located on a sand spit in the vicinity of the western branch of the Turner River. The site has vehicle access and is fronted by a narrow sandy beach which extends down to a well established mangrove region up to 200m wide. Vegetation above the HAT line provides a reasonable degree of cover across the ridge. A break in the mangrove at the northwest of the site leads to a rocky beach area with limited sand coverage (shown in the bottom left panel of **Figure D.2.7**). A weathered rock intertidal shelf extends into a shallow offshore zone. The region is provided some protection from wave action by Weerdee Island which itself has partial mangrove cover (top left panel of **Figure D.2.7**). At the rear of the site, low lying salt flats leads into thick mangrove covered estuary.

D.2.3 Shellborough Study Site

The Shellborough study site is another camping site marked for development in the release *Hedlands Future Today 2010* under the *Coastal Access and Managed Camping Project*. Presently the region is uninhabited and undeveloped, but the area is historically linked with Port Hedland.

The site is approximately 85kms northeast of Port Hedland and is shown on **Figure D.2.8**

Figure D.2.8: Study Site Shellborough.

The area once held the two late nineteenth century settlements of Shellborough and Condon, a community that numbered as many as 200 in the late 1800's. The site supported a large pearling fleet and served as a sea terminal for the supply of pastoral and mining provisions. With the establishment of Port Hedland as an alternative port, the Shellborough / Condon site was quickly abandoned, and by 1927 the telegraph station finished and the town ceased to exist (Garratt et al 1997). There is vehicle access to the site.

The Condon Creek flows out through a deep tidal channel, carrying flows from a localised catchment as shown on **Figure D.2.8.** Some rocky outcrops are found in the creek entrance and directly north and offshore of the Condon creek entrance is Miawuryguna Rocks.

From the eastern side of Condon Creek mouth a sandy beach continues to Cartaminia Point, fronted by tidal flats with partial mangrove cover and backed by vegetated dune aligned approximately north-northwest.

On the western side of the Condon creek entrance, there is shoreline aligned north and then northwest with moderate to thick mangrove cover leading into tidal flats. At the northernmost point a shoreline region dominated by calcarenite is present. Following the coastline westward there is open beach backed by degraded foredunes up to 20m high fronted by a varied density of mangrove cover that extends into tidal flats (Short 2006).

D.3 HISTORICAL CHANGES TO THE SHORELINE POSITION

This section describes the analysis of the shoreline position at the three study sites based on the photogrammetric survey data and ortho-rectified historical aerial photography between 1949 and 2009.

D.3.1 Measuring Historical Shoreline Changes in the Port Hedland Town study area

For the Port Hedland town site a total of nine aerial data sets were sourced from Landgate Imagery covering the 60 year period between 1949 to 2009 as shown on **Table D.3.1.** Major cyclones preceding each of the aerial datasets are shown.

Table D.3.1: Port Hedland Photogrammetric Sources

Date	Source	Resolution	Preceding Major Cyclone Event
1 August 1949	LANDGATE Imagery	1:50000	Unknown
2 June 1968	LANDGATE Imagery	1:80000	Shirley (Mar 1966)
15 November 1976	LANDGATE Imagery	1:34500	Joan (Dec 1975)
1 September 1983	LANDGATE Imagery	1:12000	Jane (Jan 1983)
7 July 1985	LANDGATE Imagery	1:25000	Chloe (Feb 1984)
4 August 1993	LANDGATE Imagery	1:50000	Connie (Jan 1987)
25 July 1999	LANDGATE Imagery	1:25000	Gwenda (April 1999)
1 September 2004	LANDGATE Imagery	1:7500	Monty (Feb 2004)
18 May 2009	LANDGATE Imagery	1:25000	George (Feb 2007)

As part of the Port Hedland Coastal Vulnerability Study Cardno engaged *Survey Graphics* to undertake photogrammetric analysis of the Port Hedland Shoreline between Airey Point and the Pretty Pool development. Aerial images were converted to 3D vector data by triangulating survey points within each of the aerial photography datasets. Following this the MHWS and HAT lines were determined within each of the datasets and used to assess shoreline movement.

- Mean High Water Springs MHWS = 2.8m AHD
- Highest Astronomical Tide HAT = 3.6m AHD

Across the Port Hedland site a total of 122 transects were cast at 100m intervals along the shoreline from Airey Point to the Pretty Pool development. **Figure D.3.1** shows the position of transects along the Port Hedland study area with every tenth transect numbered for reference.

Each of the transects was grouped into the 14 zones of the study site previously outlined in **Section D.2.1** and shown on **Figure D.2.2** and **Figure D.2.3.** The allocation of the transects to zones is shown on **Table D.3.2.**

Figure D.3.1: Port Hedland Transects Cast at 100m Intervals

Table D.3.2 Transects by Zones - Port Hedland Town Site

Zone	Transects Numbers	Zone	Transects Numbers
1	1 - 7	8	75 - 87
2	8 - 13	9	88 - 90
3	14 - 29	10	91 - 103
4	30 - 42	11	104 - 109
5	43 - 50	12	110- 114
6	51 - 61	13	115 - 118
7	62 - 74	14	119 - 122

D.3.1.1 Net Historical Shoreline Change at Port Hedland 1949 to 2009

The net change in the shoreline position was assessed over the 60 year period 1949 to 2009 based on the shoreline position in the first and last photogrammetric data sets. To measure change in shoreline position across the study site, each of the transect lines was measured to the intersection point of the MHWS contour for a given photogrammetric set.

The net shoreline change over the period is shown across the Port Hedland study site in **Figure D.3.2.**

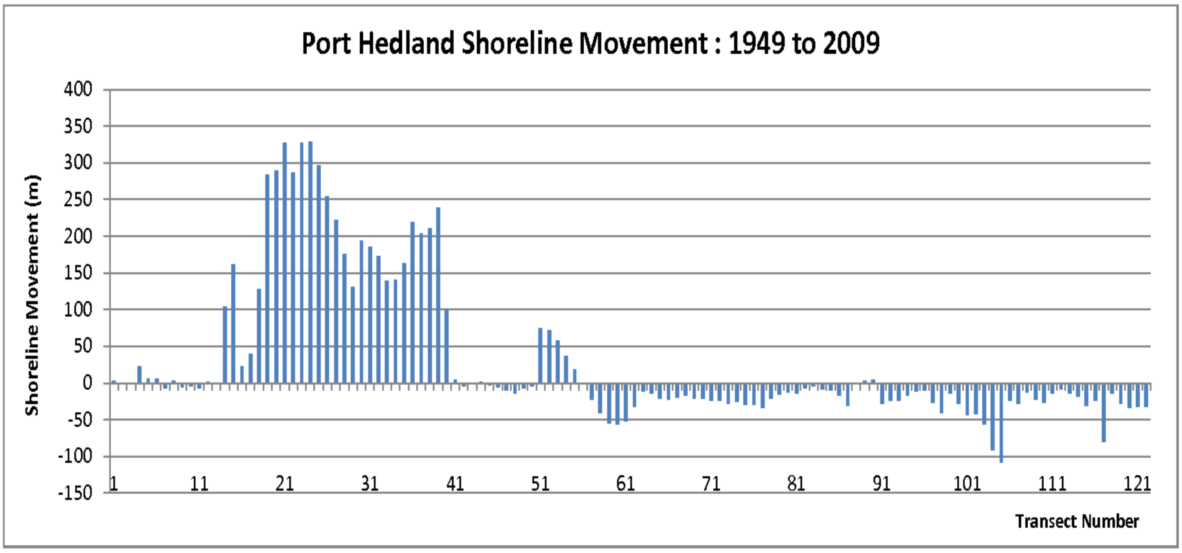


Figure D.3.2: Port Hedland Net Shoreline Changes 1949 to 2009

The very large positive shoreline accretion in Transects 14 to 39 represents the development of the spoil bank with significant shoreline accretion in this section between 1949 and 2009.

There is significant accretion in the shoreline position of Transects 51 to 55 followed by a similar level of erosion in Transects 56 to 62, representing the net change and reshaping of the sandy beach of Zone 6. In almost all transects following this zone there has been a recession of the shoreline over the 60 year period.

To further understand the timing and causes for the shoreline position changes, a breakdown of the shoreline position change into the years 1949 to 1983 and 1985 to 2009 is shown on **Figure D.3.3** and **Figure D.3.4.**

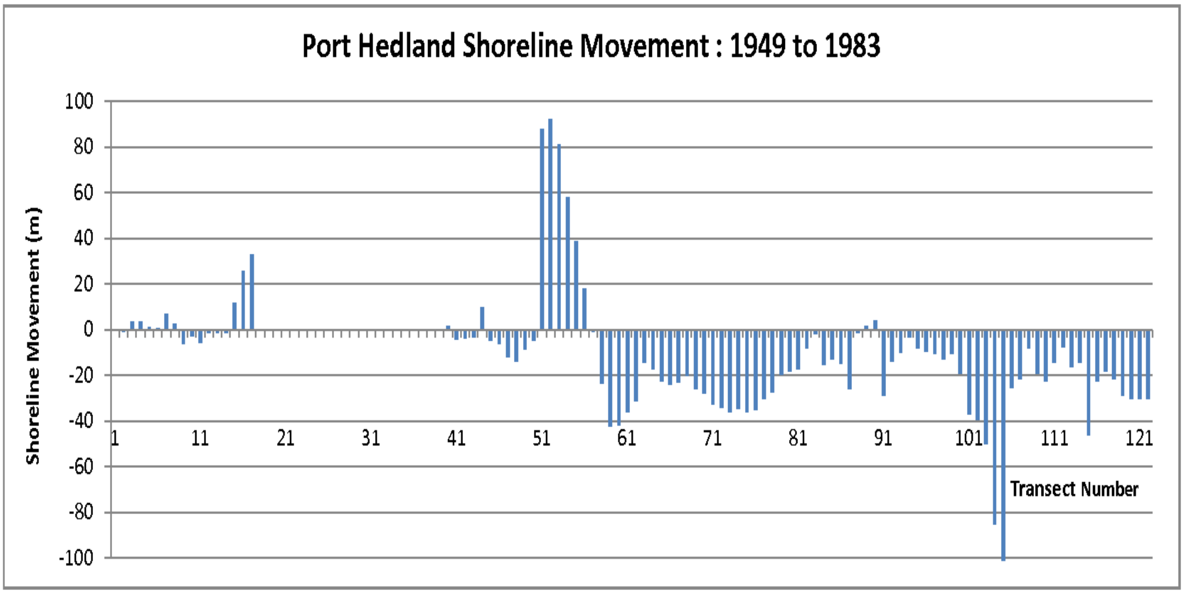


Figure D.3.3: Port Hedland Net Shoreline Changes 1949 to 1983.

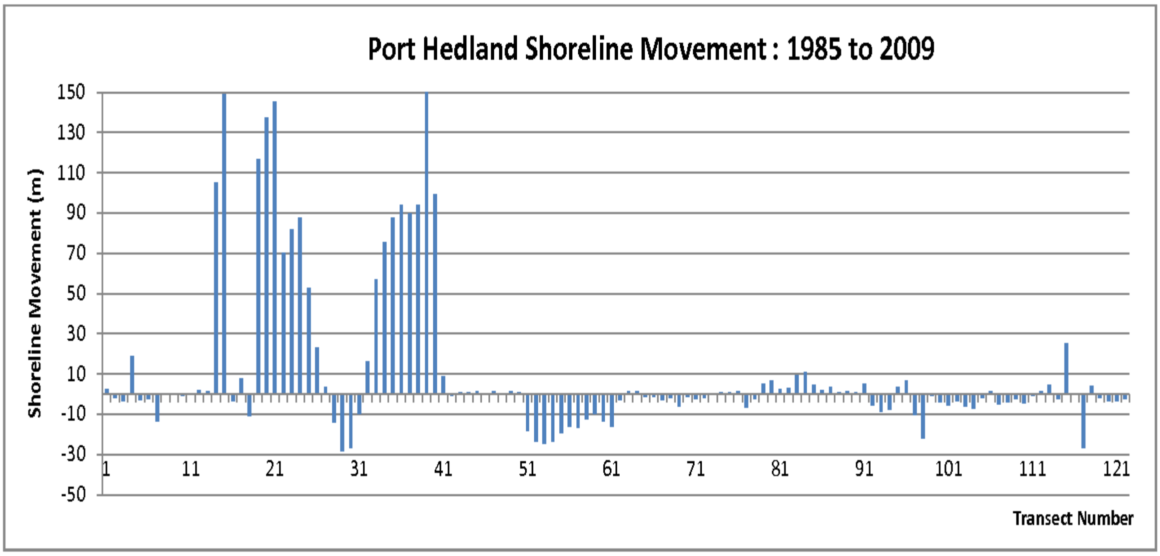


Figure D.3.4: Port Hedland Net Shoreline Changes 1985 to 2009.

In the 1949 to 1983 plot the recession of the shoreline from Transects 56 to 122 is evident. The reshaping of the sandy beach at **Zone 6** has largely occurred (Transects 51 to 62).

In the plot 1985 to 2009 the evolution of the spoil bank is clearly shown. The shoreline movement values run approximately highest to lowest across transects 14 to 29 (**Zone 3** Spoil Bank west side) and then lowest to highest in transects 30 to 39 (**Zone 4** Spoil Bank east Side). This shows the base of the spoil bank has grown at a greater rate than the outer regions.

The impact of Cyclone Joan (December 1975) to the shoreline position measured between 1968 and 1976 is shown on **Figure D.3.5**. Significant erosion occurred on the beach section through Transects 57 to 86 (**Zones 6, Zone 7, Zone 8**) with up to 50m shift in the shoreline position. There was also recession of the shoreline in **Zone 1, Zone 2** and **Zone 5** of up to 10m. Accretion was measured in Transects 51 to 55 (**Zone 6**) from 10 to 50m and at the estuarine entrance to the Pretty Pool region with accretion of 10m to 30m.

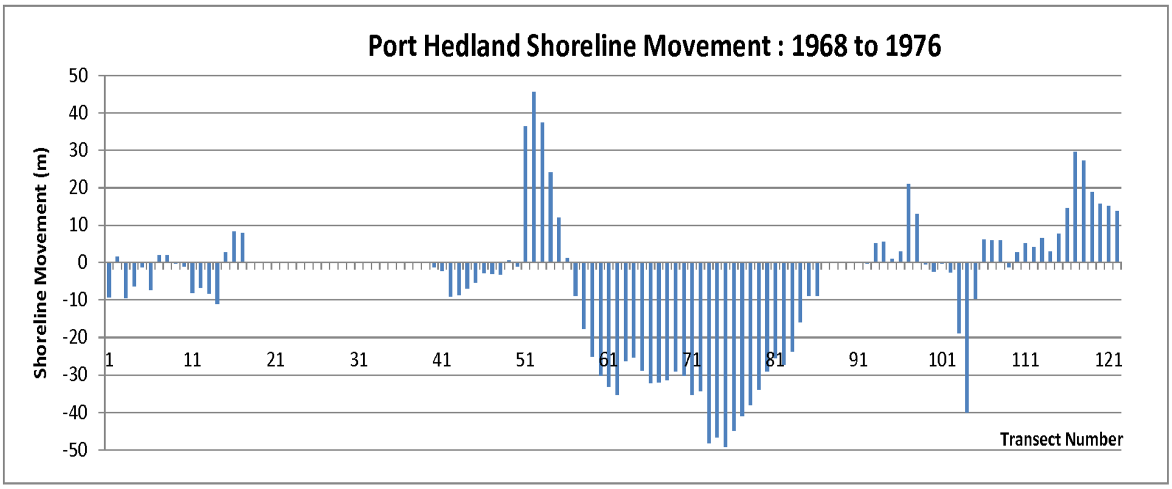


Figure D.3.5: Port Hedland Net Shoreline Changes 1968 to 1976.

The determination of future shoreline changes based on the historical data is complicated by the major change to the Port Hedland system that occurred with the advent of the spoil bank. Future forecasting of shoreline movements must consider whether this will remain a feature of the shoreline in 50-year and 100-year planning horizons.

D.3.1.2 Historical Shoreline Analysis of the Port Hedland Town study area by Zone

Changes in shoreline position across all survey years are summarised in this section by zone. For each of the years of photogrammetric data, the transect lines were measured to the intersection point of the MHWS contour. Shoreline position change was calculated as the difference in transect length relative to the 1985 dataset. The baseline year of 1985 was selected as this was the year in which the spoil bank was established as a connected landform on the Port Hedland shoreline (refer **Figure D.2.6**).

Port Hedland Zone 1

The shoreline position between 1949 and 2009 is shown on **Figure D.3.6.** The narrow beach at the west of Zone 1 (**Transects 1 and 2**) has undergone minor variation in shoreline position (<5m) across the study period. Accretion of 5 to 10m between the 1985 and 1993 periods was followed by a similar magnitude of reduction in the period immediately after, with the 1999 shoreline position assuming the general profile observed in all other years.

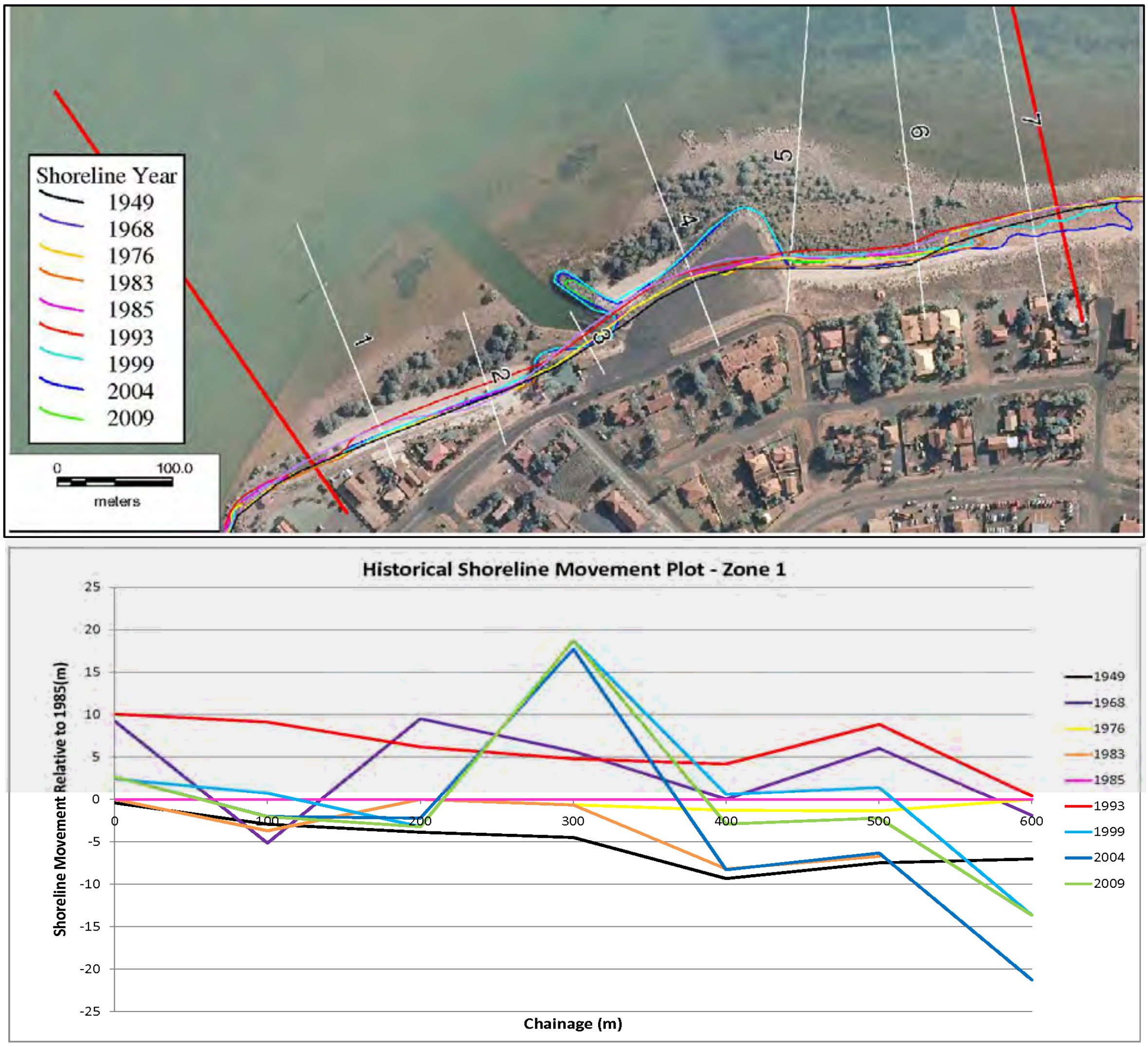


Figure D.3.6: Port Hedland Zone 1 Historical Shoreline Position Change.

Engineering works adjacent Captain Bert Madigan Park following 1993 which extended the shoreline to the north may have contributed to this reduction of the beach width. This part of the coastline was stabilised by rock wall protection with an extension to the car park adjoining the launch area at **Transects 3 and 4**. A fixed shoreline position in the years 1999 to 2009 is evident at chainage 200m to 300m.

Around Airey Point the hind beach shows variation in the shoreline position between 5 to 10m year around the 1985 baseline at **Transect 5 to Transect 7**. Shoreline erosion of 5 to 10m in the 1993 to 1999 period was followed by a similar level of erosion between 1999 and 2004. The 2004 to 2009 period showed accretion at these profiles back toward the baseline level.

Port Hedland Zone 2

Shoreline changes in this zone are shown on **Figure D.3.7 Error! Reference source not found.**across transects 8 to 13. In the earliest shoreline survey periods (1949, 1968) the shoreline position was up to 10m forward of the current and baseline position (1985). However, it should be noted that for these earlier datasets, the difference between these shoreline locations and the 1985 baseline are within the accuracy of the photogrammetric survey data.

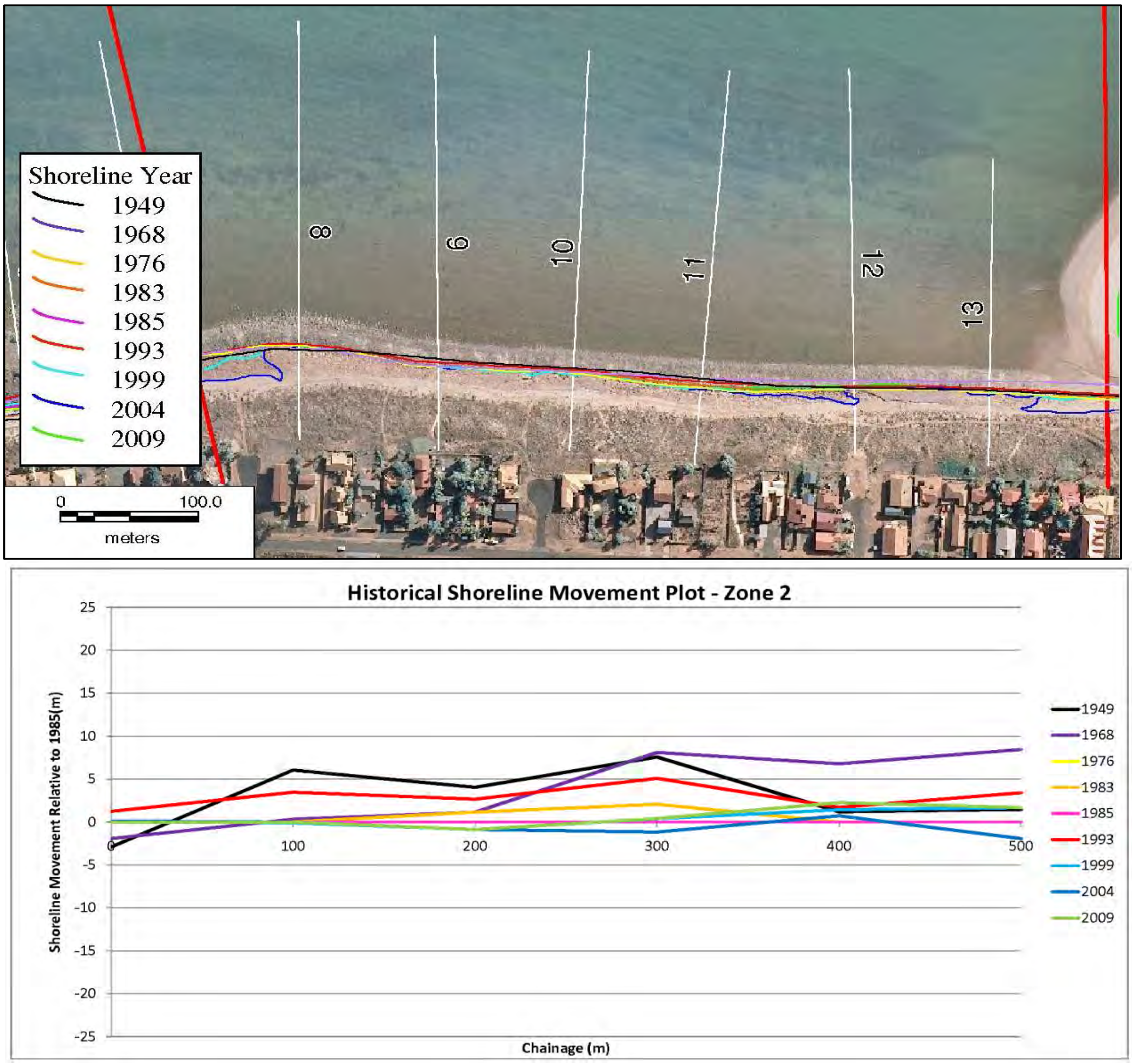


Figure D.3.7: Port Hedland Zone 2 Historical Shoreline Position Change.

Recession of the shoreline position measured in the 1976 period of 5 to 10m occurred following Tropical Cyclone Joan (December 1975). There has been very little change in shoreline position in recent periods (post 1976) and the rocky intertidal zone effectively limits any impacts from coastal processes. The accretion measured in the 1985-1993 period is likely influenced by the establishment of the spoil bank and the absence of major cyclones in the years preceding the aerial photo.

Port Hedland Zone 3

Changes in **Zone 3** are shown on **Figure D.3.8**. **Transects 14 to 17** have been measured across all data sets whilst **Transects 18 to 29** have been measured from 1985 to 2009 after the spoil bank connected to the mainland.

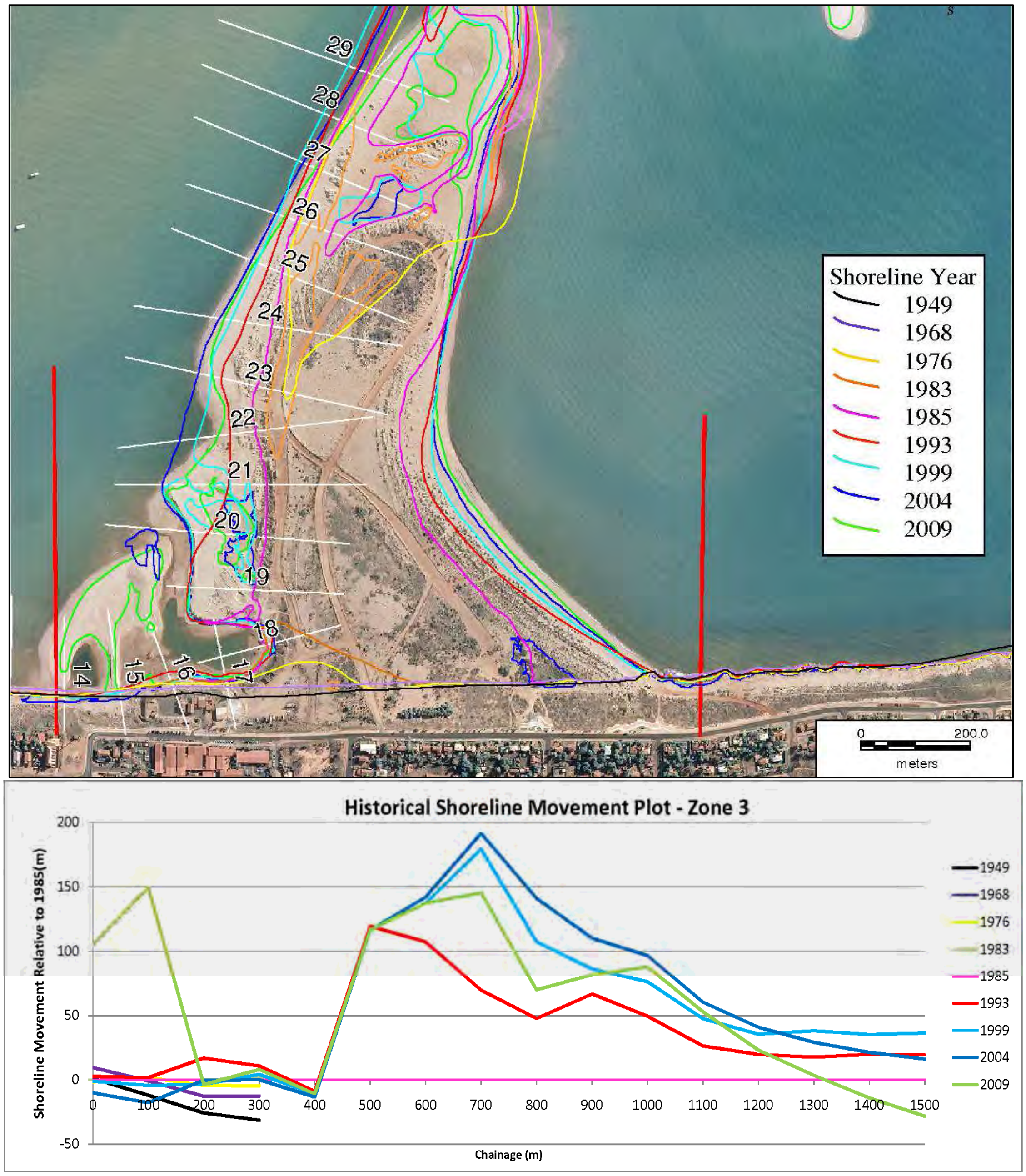
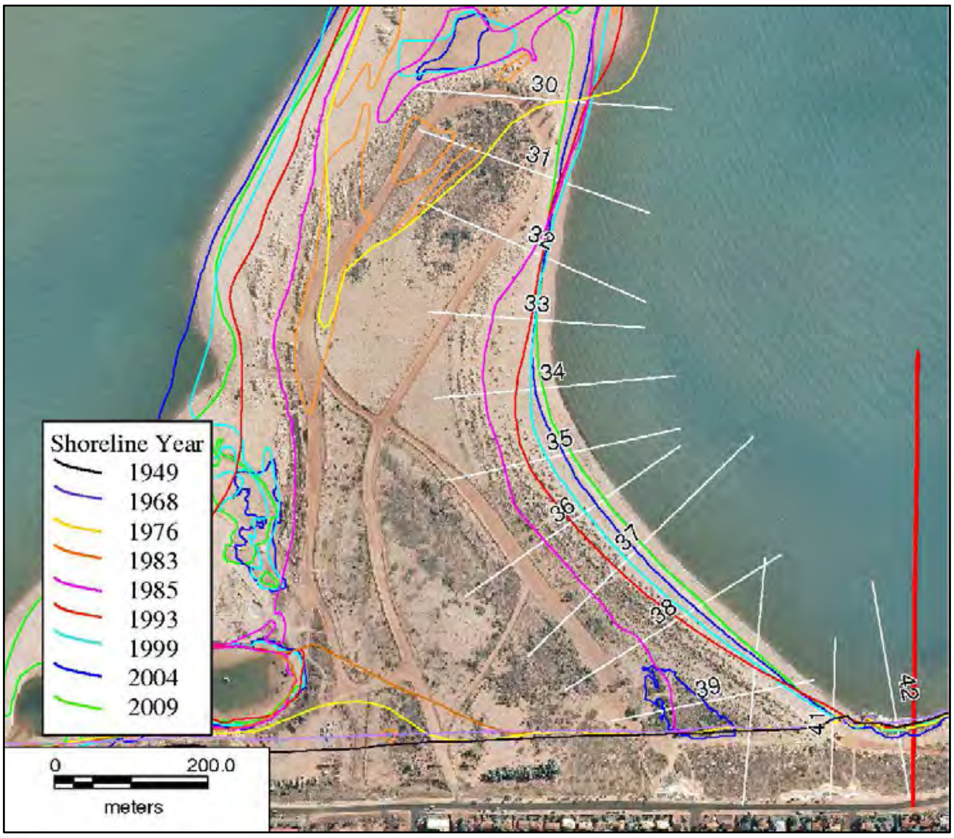


Figure D.3.8: Port Hedland Zone 3 Historical Shoreline Position Change.

Significant capital dredging in the years from 1960 contributed to the rapid establishment of this feature on the Port Hedland shoreline (refer **Figure D.2.6**). The connection of the spoil bank to the mainland of Port Hedland is clearly evident through this zone in the years 1985 onwards. The shoreline position of the sandy beach in front of the Port Hedland yacht club (**Transects 14 to 16**) advanced significantly between 2004 and 2009 as large deposits of sand were moved to develop the Port Hedland Marina.

Port Hedland Zone 4

Changes in this zone are shown on **Figure D.3.9**. **Transects 40 to 42** have been measured across all data sets whilst **Transects 30 to 39** have been measured from 1985 to 2009 after the spoil bank connected to the mainland.



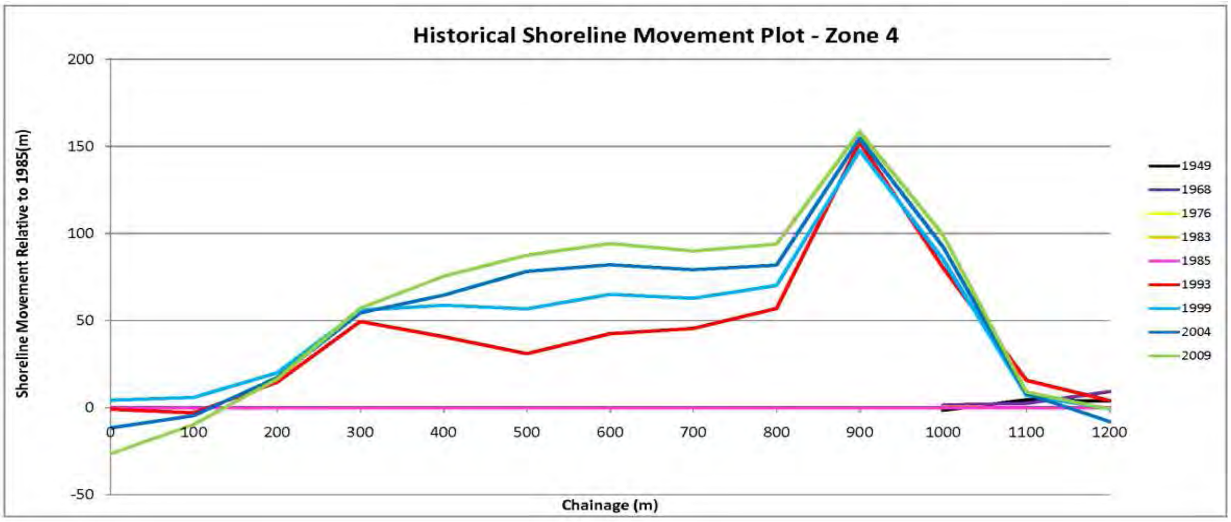


Figure D.3.9: Port Hedland Zone 4 Historical Shoreline Position Change.

Through most of this zone there is sustained accretion from the 1985 shoreline position, with the shoreline prograding in all subsequent surveys by 10-20m. As sand is transported east from the tip of the spoilbank it appears to build up along the beach area on the southeastern connection point.

Port Hedland Zone 5

Shoreline position changes in Zone 5 between 1949 and 2009 are shown on **Figure D.3.10**.

Figure D.3.10: Port Hedland Zone 5 Historical Shoreline Position Change.

The early shoreline position along this section in 1949 and 1968 was more advanced to the north than its present day position. There was a loss of sand from the beach system and consequent shoreline recession of 5 to 10m measured in the 1968 survey in **Transects 46 to 50**. However, it should be noted that for these earlier datasets, the difference between these shoreline locations and the 1985 baseline are within the accuracy of the photogrammetric survey data.

The 1968 aerial photos indicate the initial presence of the spoil bank as an offshore feature (see **Figure D.2.6**). There would have been a modification of the nearshore transport regime associated with this, which may account for some of the loss of sand in the Zone 5 region. Cyclone Shirley (March 1966) also occurred prior to the 1968 aerial survey.

From 1976 the shoreline position has remained largely unchanged as the cliffs along the shore maintain a barrier to extreme wind and wave action. The accretion measured in the 1985-1993 period corresponds with a time of major capital dredging works coupled with an abscence of major cyclones in the years preceding the 1993 survey.

Port Hedland Zone 6

The shoreline position in **Zone 6** between 1949 and 2009 is shown on **Figure D.3.11**.

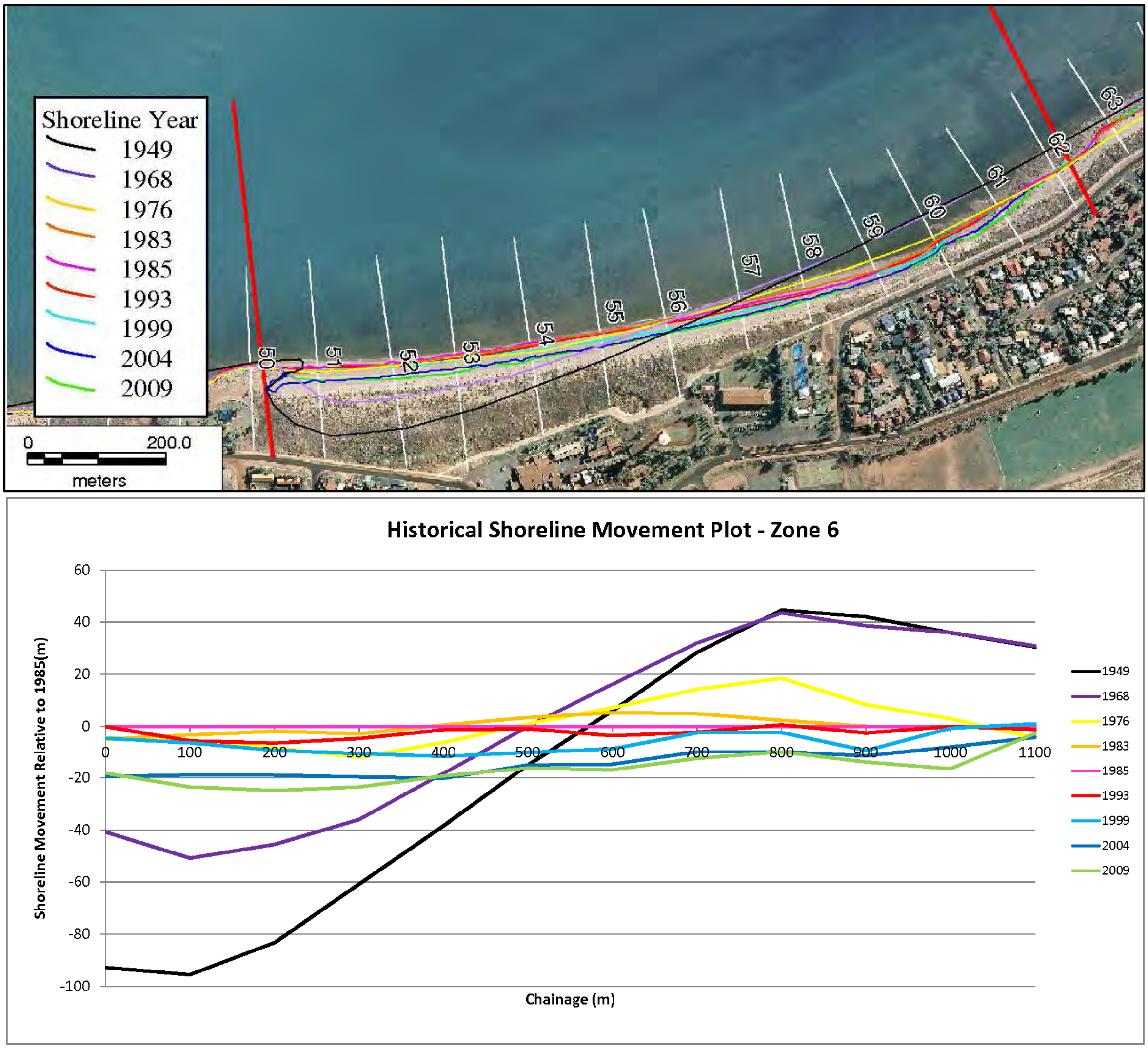


Figure D.3.11: Port Hedland Zone 6 Historical Shoreline Position Change.

A major reshaping of this beach occurred from the alignment of the earliest survey period (1949) to subsequent surveys in 1968 and 1976. Over this time the western side of Zone 6 showed significant accretion of up to 40m in each survey period. Between the 1968 and 1976 survey the eastern end of Zone 6 receded up to 40m.

The modification to the natural system with the development of the spoil bank as well cyclone Joan (December 1975) occurred across these surveys periods.

The 1976 survey which showed significant accretion in Transects 51 to 56 and erosion between transects 57 and 62 followed cyclone Joan. Erosion of up to 15m between the 1999 and 2004 shoreline position followed Tropical Cyclone Monty (February 2004).

Port Hedland Zone 7

Shoreline position for transects in **Zone 7** are shown on **Figure D.3.12**.

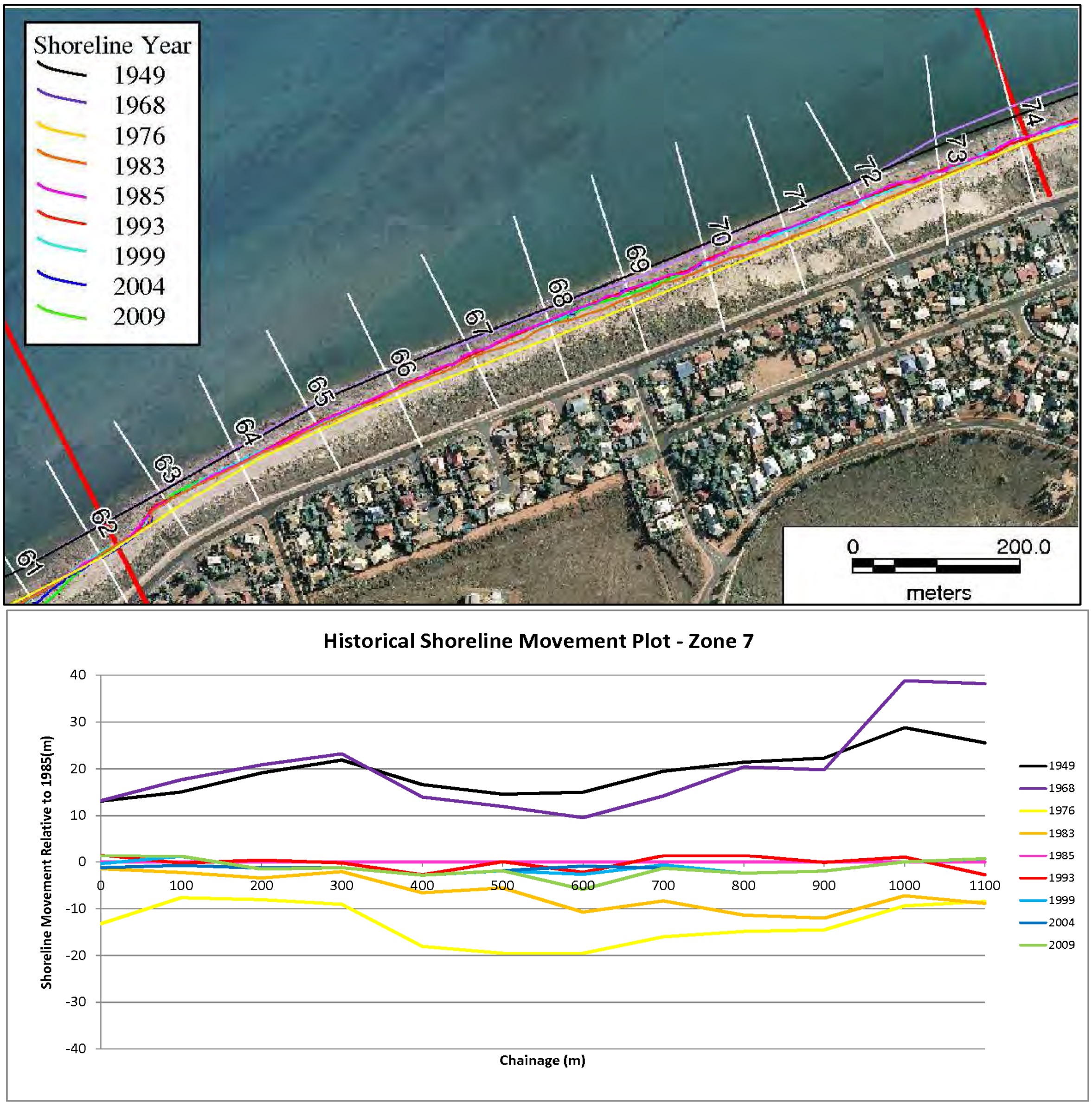


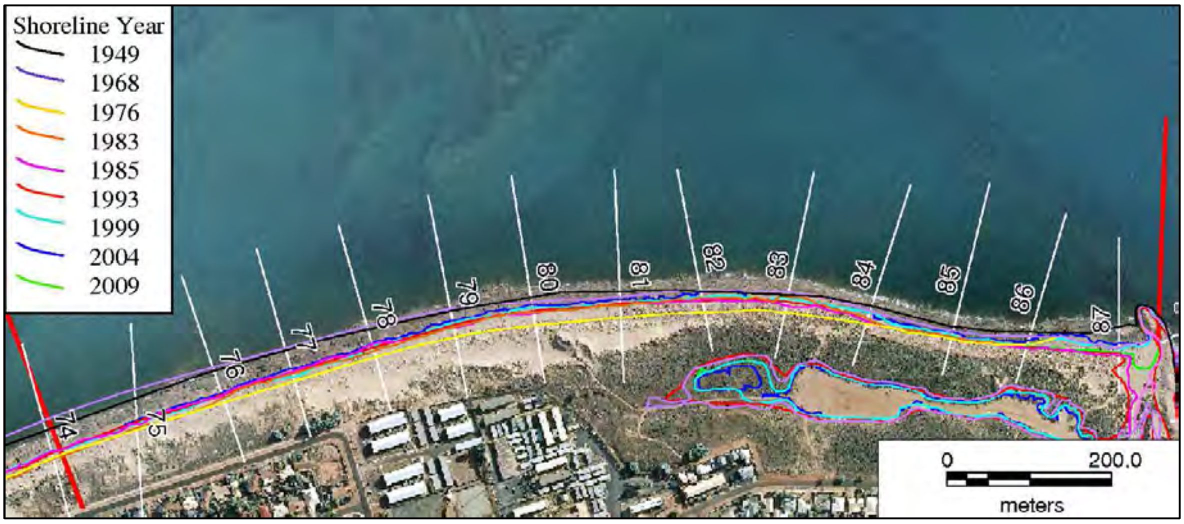
Figure D.3.12: Port Hedland Zone 7 Historical Shoreline Position Change.

Shoreline position shift from the earliest photogrammetric survey in 1949 indicate the present shoreline to have receded on average 20m. Review of the 1949 and 1968 aerial photos indicate the presence of a sandy beach at the mean water level along this section of the Port Hedland shoreline which has not returned in subsequent survey periods. The most dramatic recession of the shoreline occurred between the 1968 to 1976 surveys, as observed for the eastern end of **Zone 6**. As mentioned previously the modification to the natural system with the development of the spoil bank as well cyclone Joan (December 1975) occurred across these surveys periods.

The shoreline has remained largely unchanged through the recent period 1985-2009.

Port Hedland Zone 8

The shoreline positions from 1949 to 2009 in **Zone 8** are shown on **Figure D.3.13**.



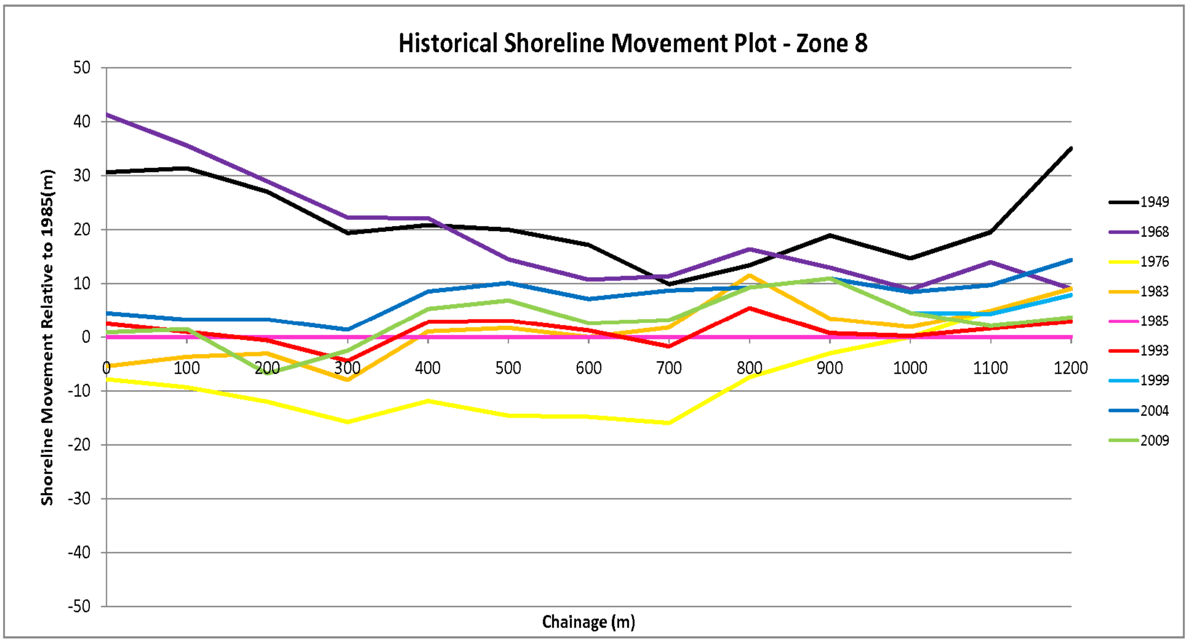


Figure D.3.13: Port Hedland Zone 8 Historical Shoreline Position Change.

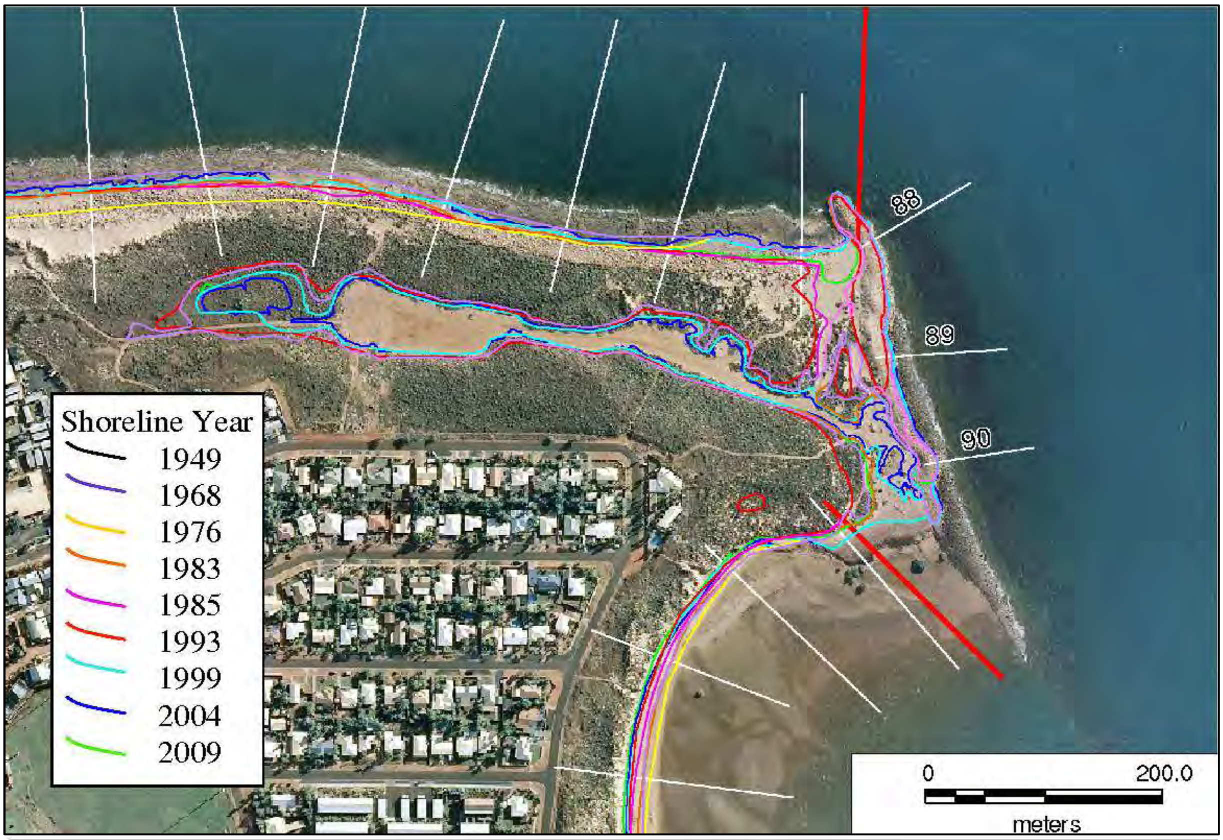
In this zone a recession of the shoreline position between 1968 and 1976 was observed as for the adjacent shoreline of **Zone 7**. As indicated for the previous zone a review of the aerial photos from 1949 and 1968 shows a sandy shoreline at the mean water level which has been modified in subsequent survey periods.

In the time period 1976 to 1983 the beach recovered approximately 10m through the western transects.

The eastern shoreline has experienced varied erosion and accretion of up to 10m a year since 1985, whilst the western end of Zone 8 has remained relatively stable.

Port Hedland Zone 9

Shoreline position measured between 1949 and 2009 for **Zone 9** is shown on **Figure D.3.14**.



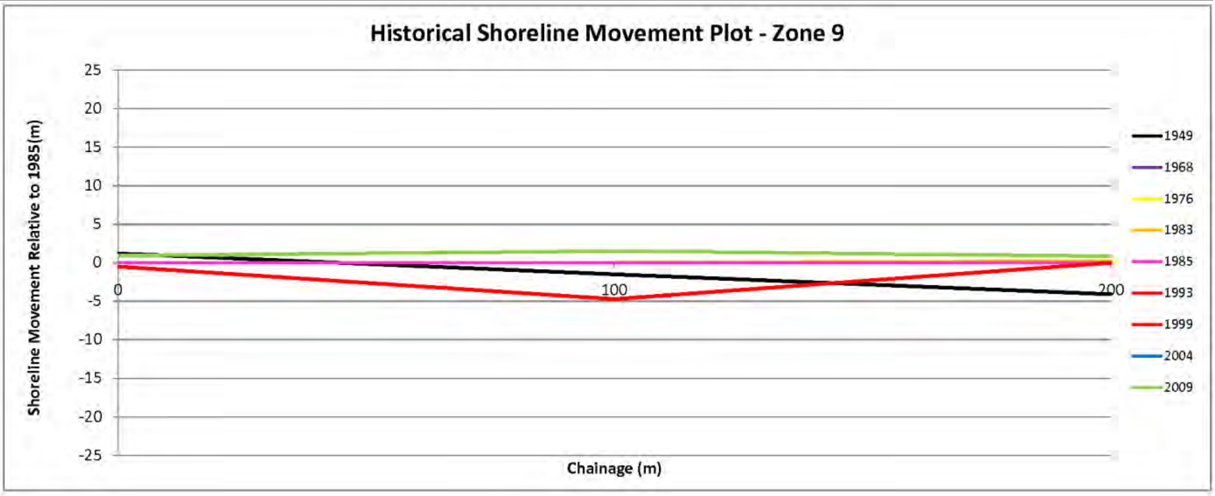


Figure D.3.14: Port Hedland Zone 9 Historical Shoreline Position Change.

Zone 9 is located across Cooke Point with the **Transects 88 to 90** across a very stable area. Very little to no change across the periods was measured at the shoreline. Behind the shoreline there is a low tidal region which has also remained fairly stable throughout the measured period.

Port Hedland Zone 10

The shoreline of **Zone 10** is shown between the years 1949 and 2009 on ***Figure D.3.15***.

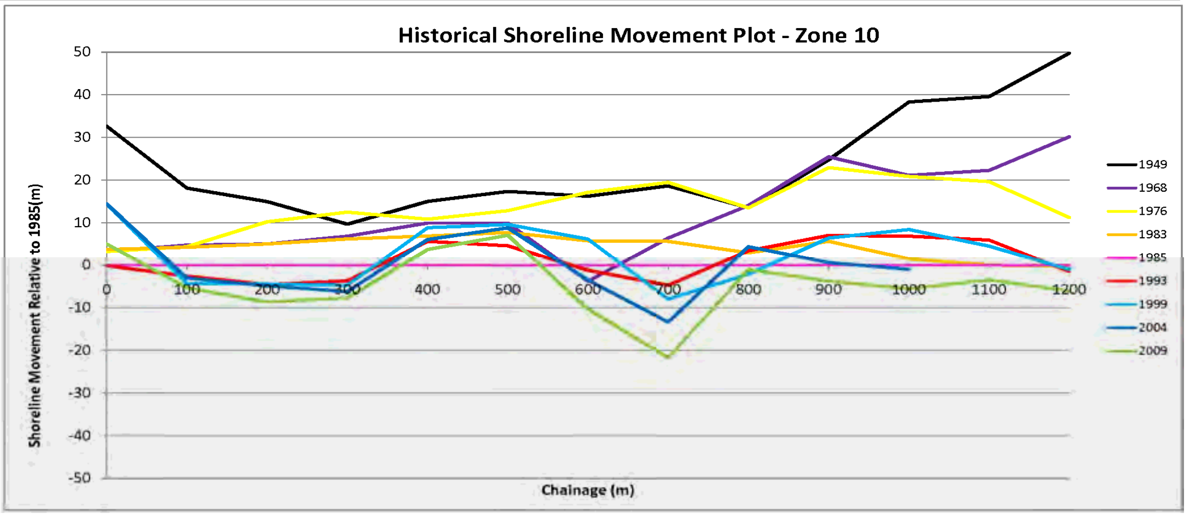
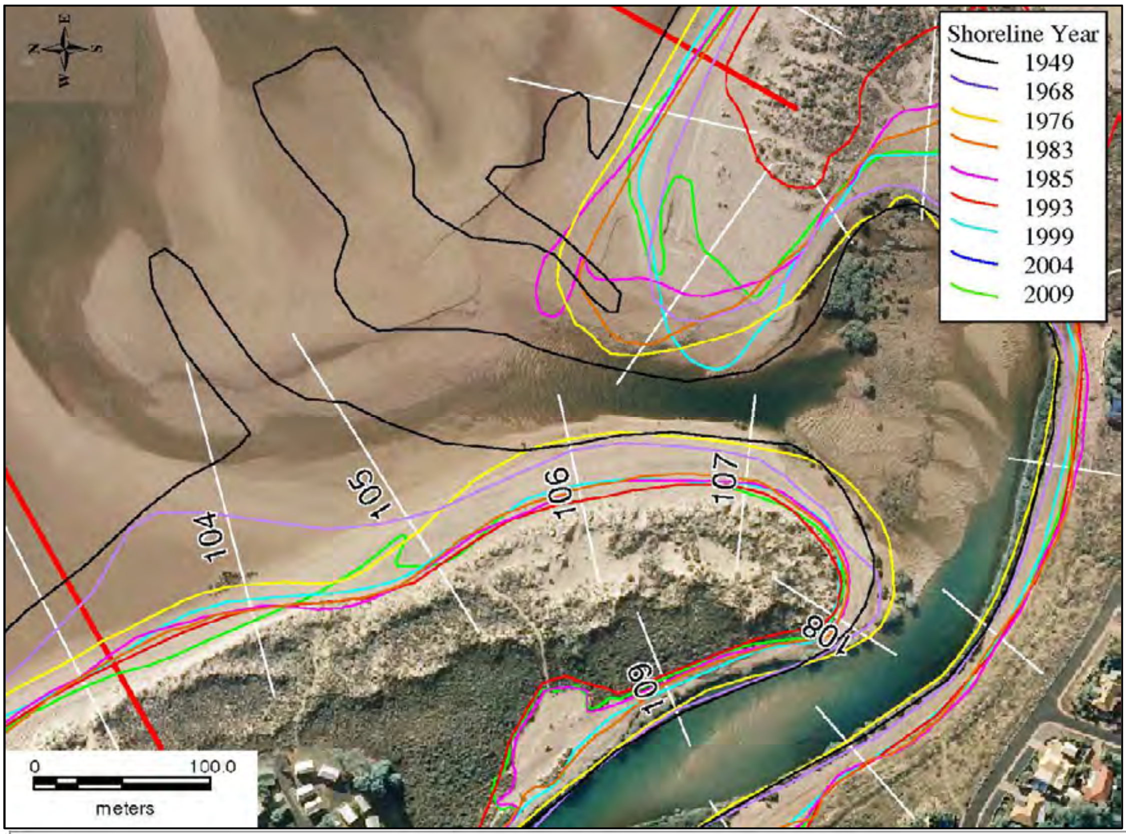


Figure D.3.15: Port Hedland Zone 10 Historical Shoreline Position Change.

Marked differences in the shoreline position prior to 1983 indicate greater sand coverage across the beach at this time. Following 1985, a distinct profile change occurs and is maintained across the zone with a sustained pattern of erosion between 1999 to 2009 most pronounced along the beachfront at **Transect 98**.

Port Hedland Zone 11

Transects 104 to 109 at the northern entrance to Pretty Pool are evaluated as part of **Zone 11** and shown on **Figure D.3.16**.



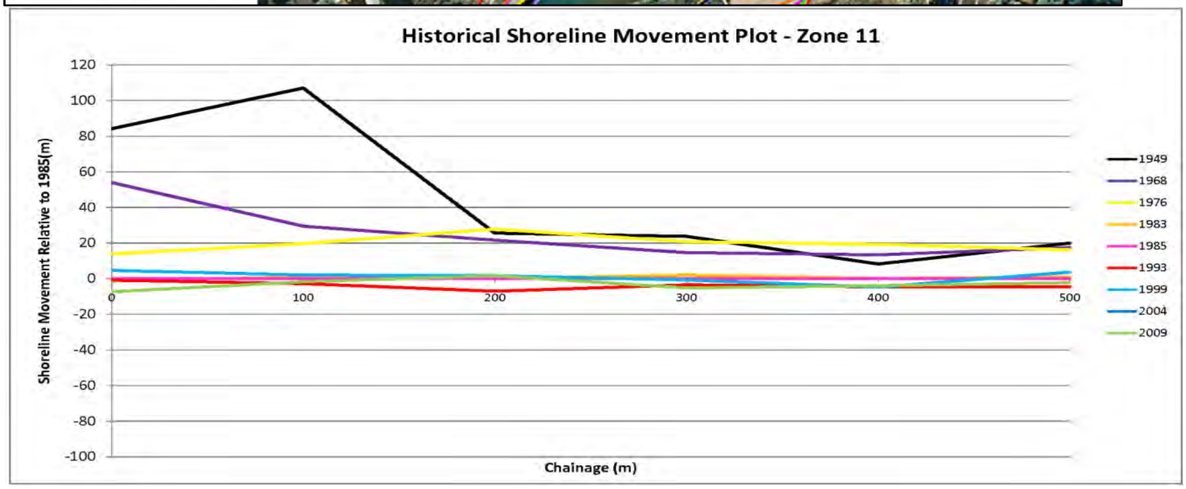
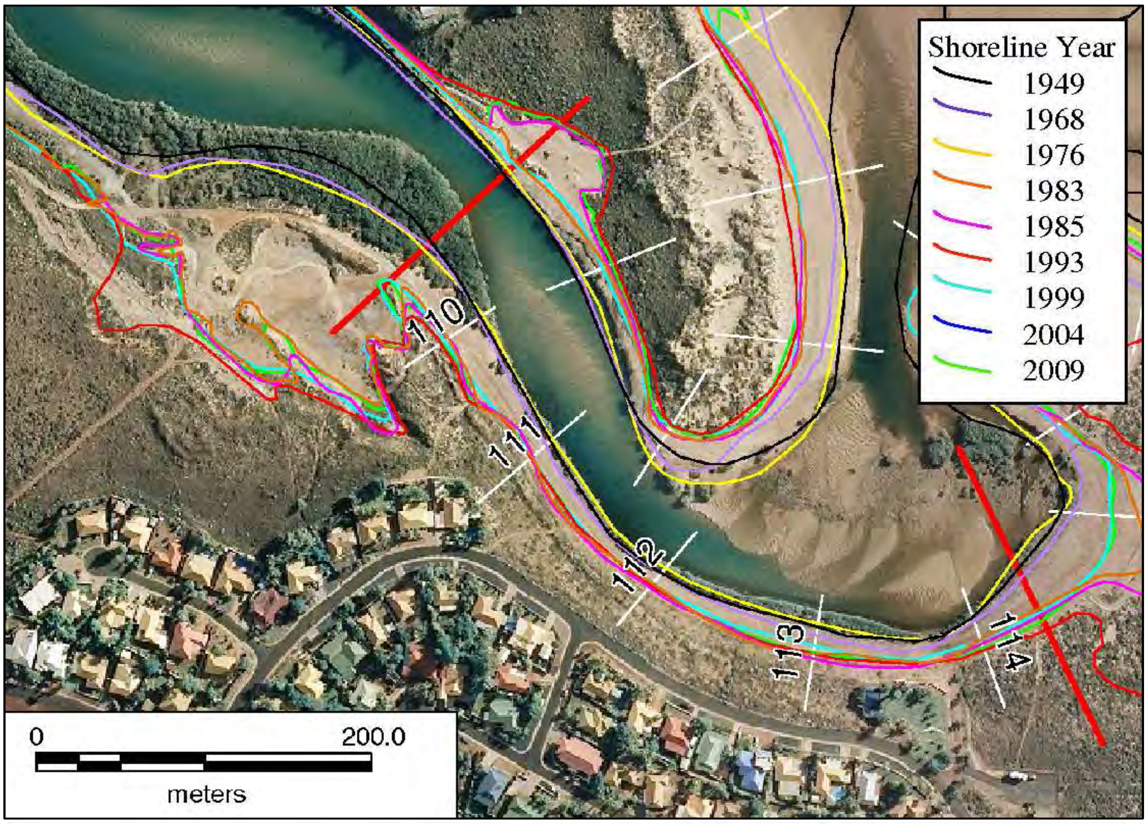


Figure D.3.16: Port Hedland Zone 11 Historical Shoreline Position Change.

The recession of the shoreline on the northern bank is evident when comparing the original position measured in 1949 against the present position. A shift back of 80m and 100m at **Transects 104 and 105** respectively from 1949 to current can be observed. Whilst the shoreline position receded in the survey periods 1968, 1976 and 1983 there has been very little movement in subsequent years.

Port Hedland Zone 12

The bank of the estuary on the northern side of the Pretty Pool estate is described by **Transect 110 to 114** and changes across **Zone 12** are shown on **Figure D.3.17**.



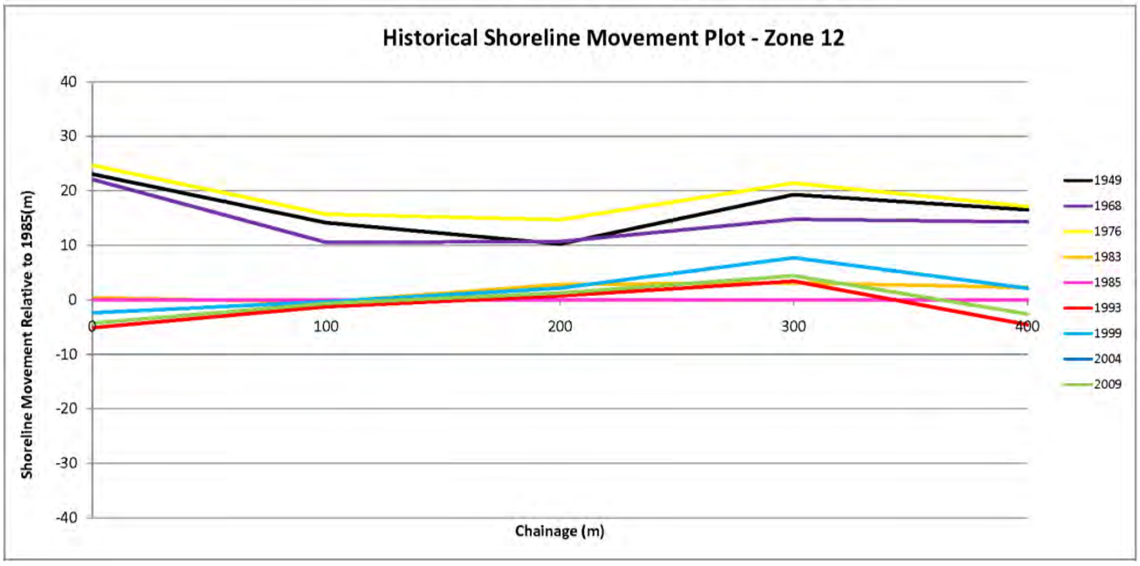
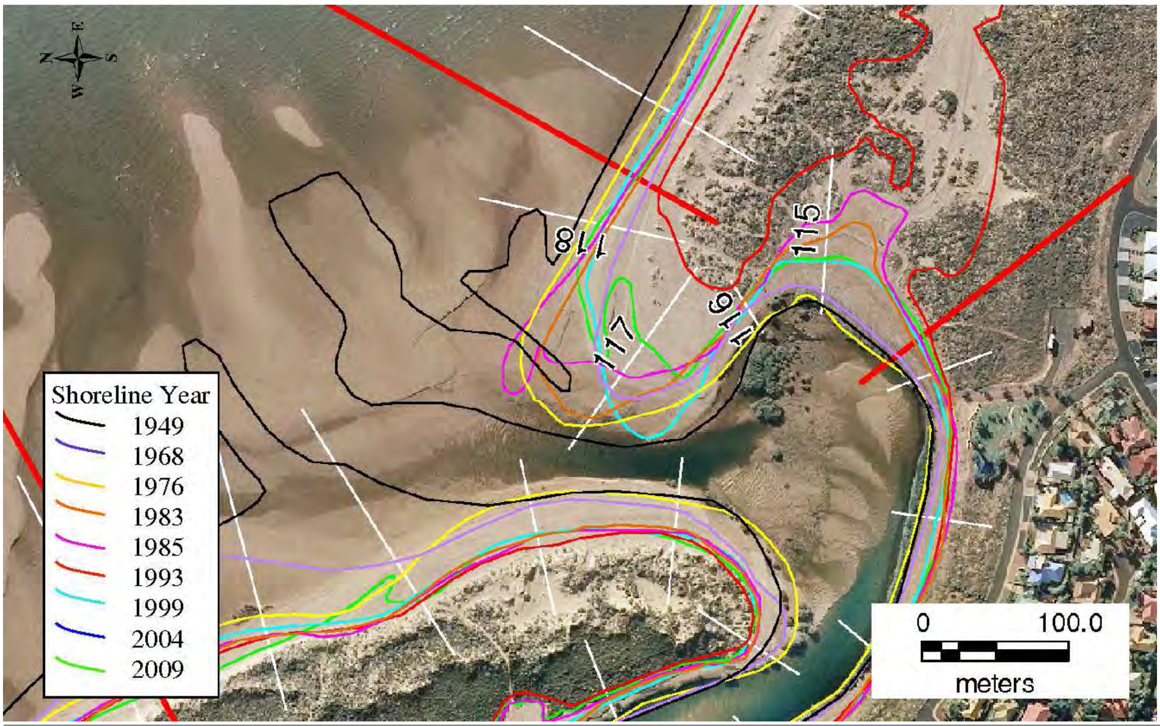


Figure D.3.17: Port Hedland Zone 12 Historical Shoreline Position Change.

The shoreline position measured in 1949, 1968 and 1976 indicates the estuary channel was aligned 10 to 20m north of the present line. Since the 1983 photogrammetry the shoreline position is largely unchanged.

Port Hedland Zone 13

The transects in **Zone 13** mark out the southern bank of the estuary entrance to Pretty Pool and are shown on **Figure D.3.18.**



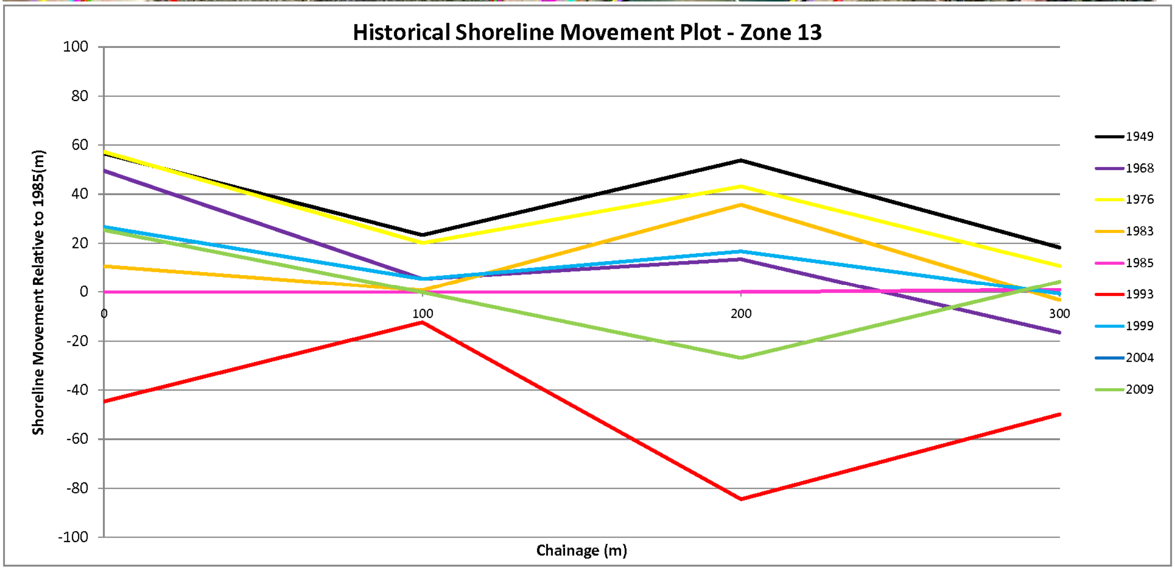


Figure D.3.18: Port Hedland Zone 13 Historical Shoreline Position Change.

As for **Zone 11** the shoreline position is extremely variable through the **Transects 115 to 118** particularly at **Transect 117** through the entrance point to the estuary. The shoreline in 1993 receded to the greatest extent across the study years, up to 80m from the 1985 position. In 1949, the southern side of the entrance was up to 60m from the baseline position.

Port Hedland Zone 14

Zone 14 is beach southeast of the estuary entrance shown on **Figure D.3.19** with **Transects 119 to 122.**

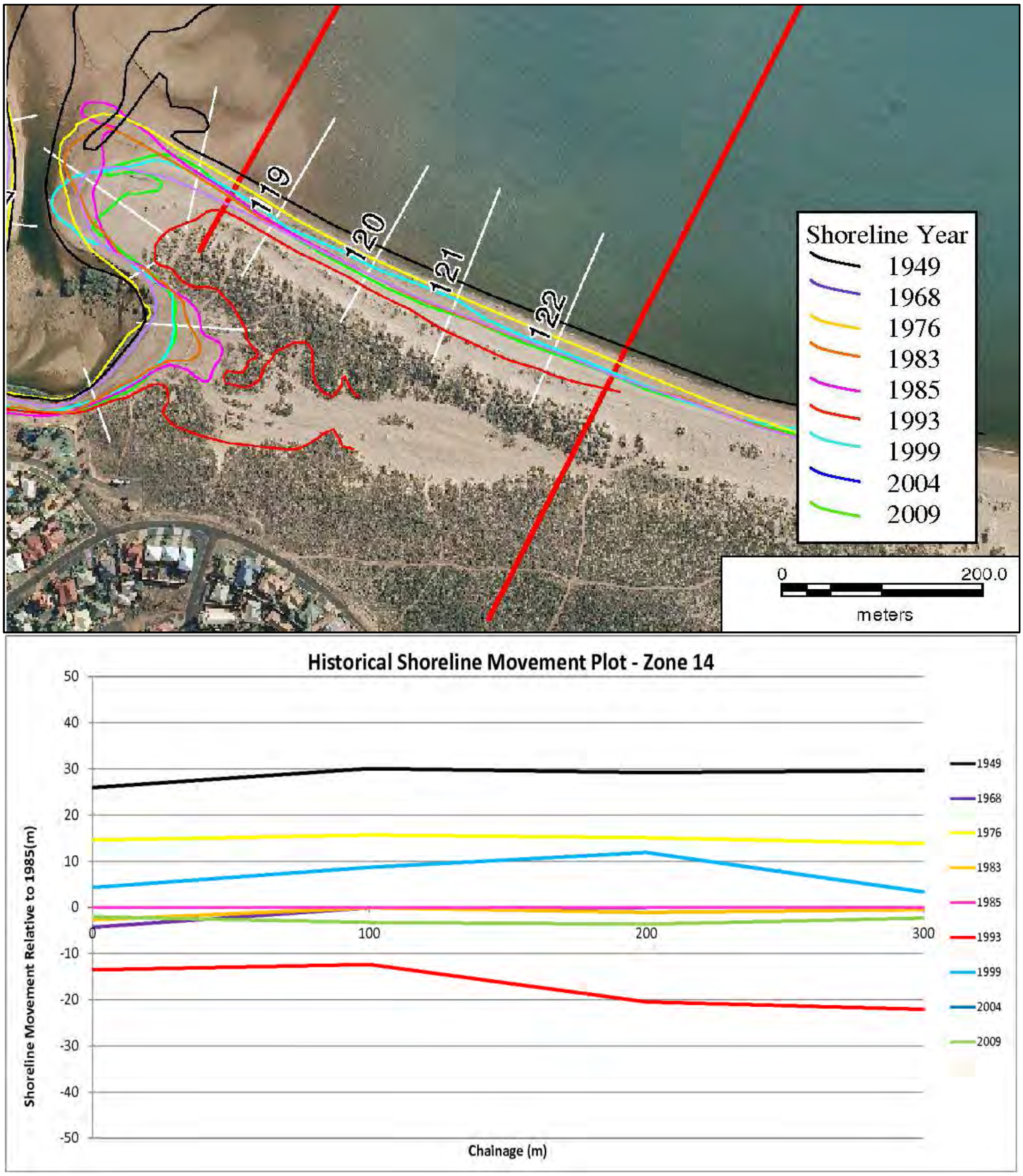


Figure D.3.19: Port Hedland Zone 14 Historical Shoreline Position Change.

This section of the shoreline is northwest facing and more exposed to wave action so has been susceptible to significant changes across the survey period. From the baseline position (1985) the shoreline has been up to 30m further forward in 1949 and up to 20m further back in 1993.

D.3.1.3 Port Hedland Historical Setback Component (S2)

The historical setback distance (S2) for each of the zones of the Port Hedland study site is summarised in **Table D.3.3**. The net shoreline change of the MHWS contour in two analysis periods is investigated - 1949 to 2009 and 1985 to 2009. This approach takes into account the formation of the Port Hedland Spoil Bank. For each analysis period, the most severe recession rate recorded across all transects within each respective zone was selected. The historic trend value is expressed in terms of annual change (m/yr) on **Table D.3.3** with recession rates shown as negative values. Zones in which all transects record accretion (eg **Zone 9**) result in positive values.

Assessment of the historical component S2 is based on the following methodology (adapted from SPP2.6) :-

1. The historical shoreline component (S2) is selected as the most severe recession value calculated across the two analysis periods - 1949 to 2009 and 1985 to 2009.
2. Historical rates of shoreline recession less than 20m across the 100 year planning period are assigned a default value of 20m.
3. Historical rates of shoreline recession less than 10m across the 50 year planning period are assigned a default value of 10m.
4. Historical rates of accretion greater than 0.2m/yr are assigned an S2 value of 0m.

Table D.3.3: Historical Setback (S2) Summary Port Hedland

Zone	Representative Transects	1949 - 2009 (m / yr)	1985 - 2009 (m / yr)	S2 Component 2060 (m)	S2 Component 2110 (m)
Zone 1	1,2,5,6	0.07	-0.05	10	20
Zone 2	8 - 13	-0.04	0.02	10	20
Zone 3					
Zone 4					
Zone 5					
Zone 6	51 - 62	0.00	-0.68	34	68
Zone 7	63 - 74	-0.35	-0.05	17	35
Zone 8	75 - 87	-0.30	0.13	15	30
Zone 9	88 - 90	0.04	0.05	10	20
Zone 10	91 - 103	-0.47	-0.19	24	47
Zone 11					
Zone 12	110 - 114	-0.28	-0.02	14	28
Zone 13					
Zone 14	119 - 122	-0.53	-0.12	27	53

Zone 5 is assessed as a rock shoreline. The uncertainty associated with the spoil bank through **Zone 3** and **Zone 4** make assessment of this feature for the 50 year and 100 year planning periods beyond the scope of this study. The mobile estuary entrance to Pretty Pool through **Zone 11** and **Zone 13** is not assessed for setback due to the mobility of this feature.

D.3.2 Measuring Historical Shoreline Changes in the Shellborough study area

For the Shellborough site, aerial photographs from the years 1949 to 2007 shown in **Table D.3.4** were approximately ortho-rectified by Cardno.

Table D.3.4: Shellborough Study Site Photogrammetric Sources

Date	Source
19 June 1949	LANDGATE Imagery
18 May 1968	LANDGATE Imagery
14 September 1971	LANDGATE Imagery
14 June 1972	LANDGATE Imagery
15 November 1976	LANDGATE Imagery
4 August 1993	LANDGATE Imagery
1 September 2007	LANDGATE Imagery

Changes to the shoreline position were mapped based on the movement of the Horizontal Setback Datum (HSD) across the years of photogrammetry. For the Shellborough study site which is largely sandy coast, the HSD line was assessed as the vegetation line along the fore dune as recommended in SPP2.6. Changes to the position of this line were investigated across all years of aerial photo data.

The Shellborough study area was divided into four zones as shown on **Figure D.3.20**.

Figure D.3.20: Shellborough Study Site by Zone.

D.3.2.1 Historical Shoreline Analysis of the Shellborough study area by Zone

Historical shoreline position is shown for **Zone 1** to **Zone 4** on **Figure D.3.21 to Figure D.3.24**.

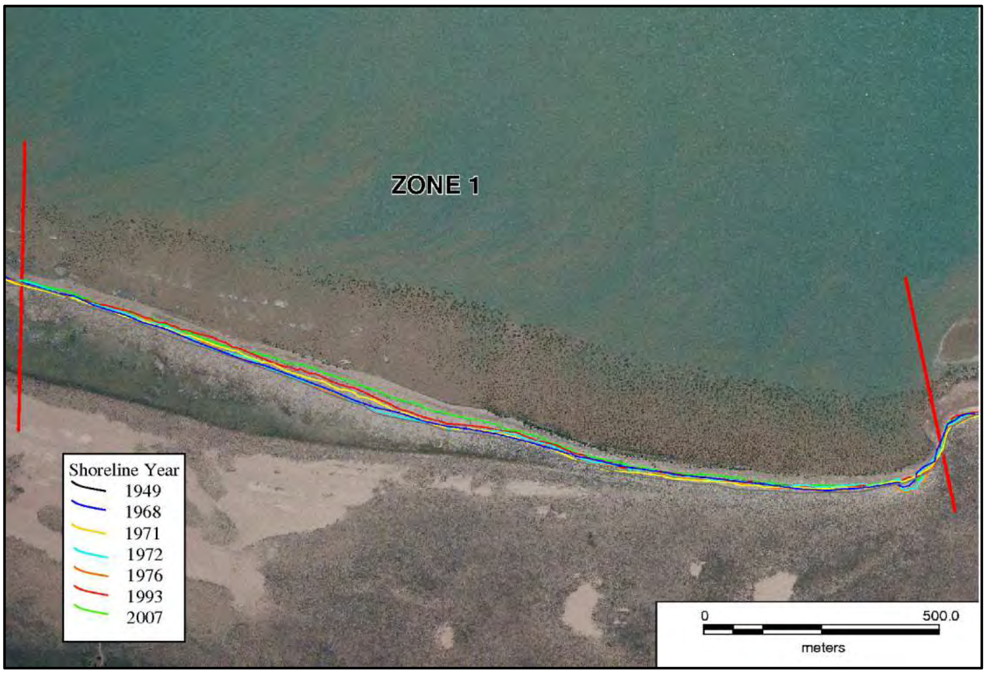


Figure D.3.21: Shellborough Study Zone 1.

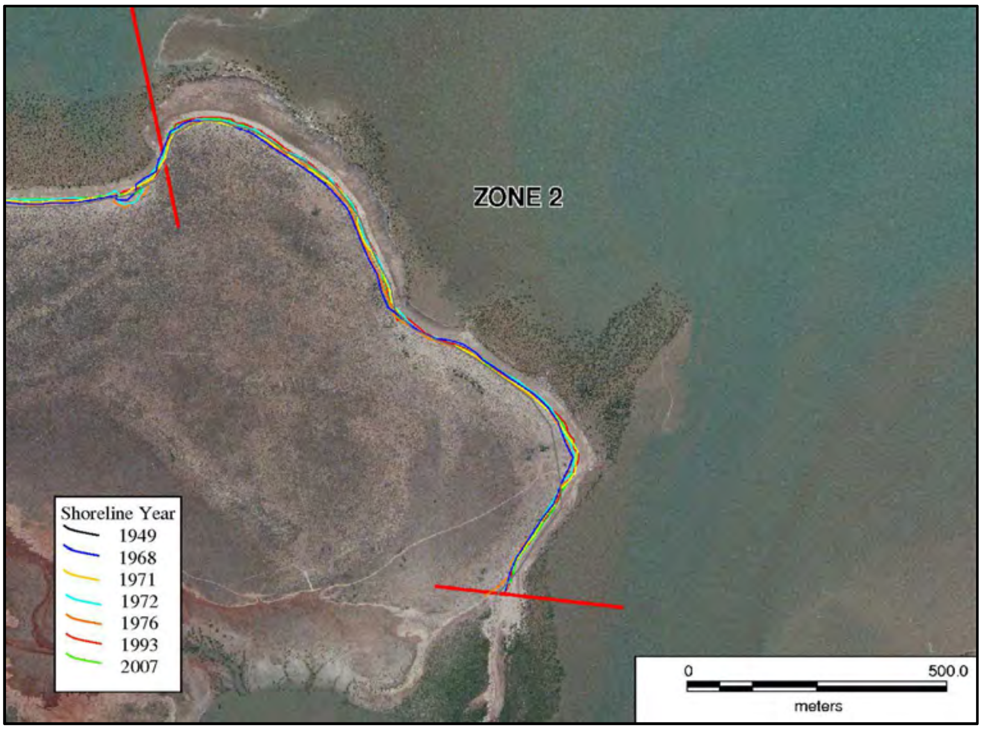


Figure D.3.22: Shellborough Study Zone 2.

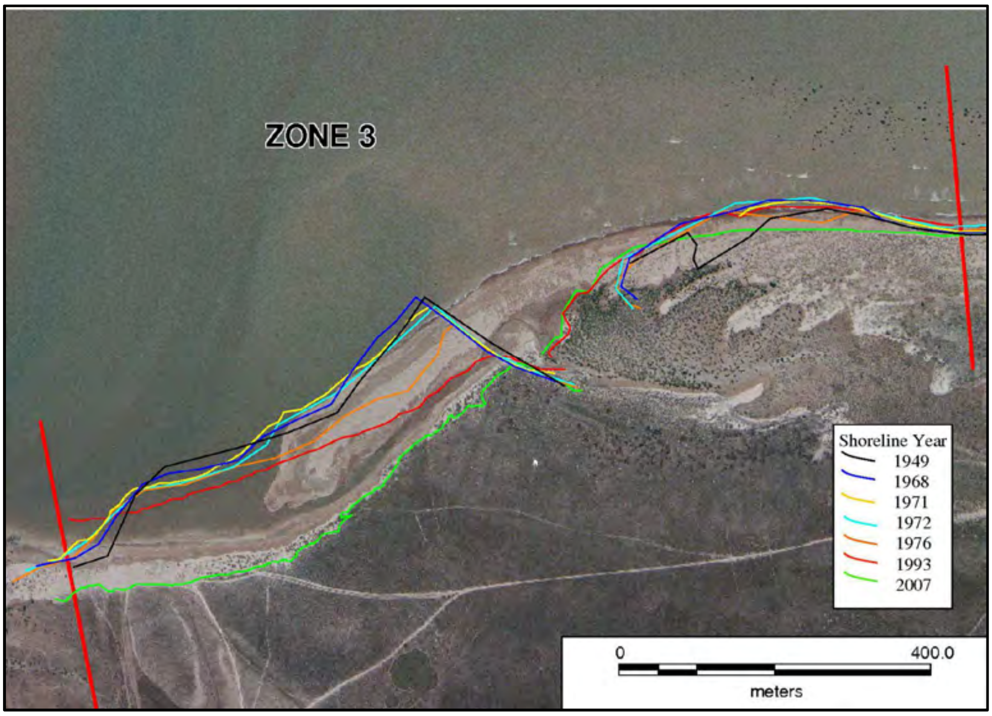


Figure D.3.23: Shellborough Study Zone 3.

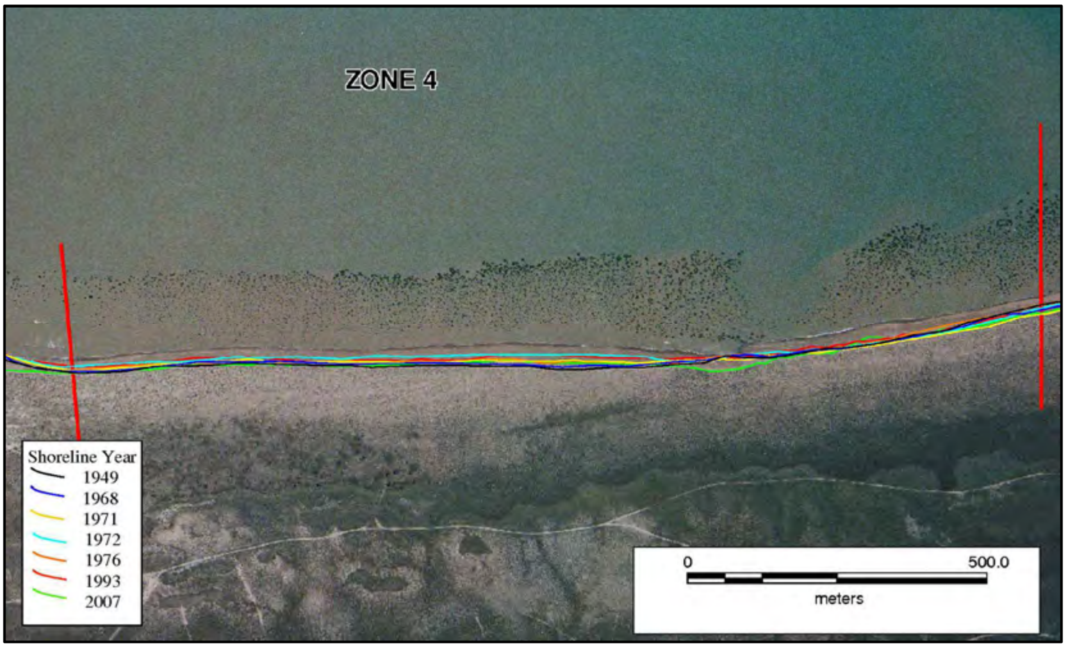


Figure D.3.24: Shellborough Study Zone 4.

In **Zone 1, Zone 2** and **Zone 4** there is very little movement of the HSD line across the historical data sets. **Zone 3** is a highly variable region which has shown significant shoreline recession of up to 160m over the 58 year period 1949 to 2007. For **Zone 3**, it is possible that Tropical Cyclone George in March 2007 may have influenced the erosion in **Zone 3**.

D.3.2.2 Shellborough Historical Trend Setback Distances (S2)

The historical setback component (S2) for the 2110 planning horizon in each of the zones of the Shellborough study site are summarised in **Table D.3.5**. The calculation of setback is based on the years 1949 to 2007 using the most severe recession rate of the HSD within each zone.

Table D.3.5: Historical Setback (S2) Shellborough Site.

Zone	1949 - 2007 (m / yr)	S2 Component 2110 (m)
Zone 1	- 0.09	20
Zone 2	- 0.15	20
Zone 3	- 2.75	275
Zone 4	- 0.20	20

D.3.3 Measuring Historical Shoreline Changes in the Site 2 study area

For Site 2, aerial photographs from the years 1949 to 2009 shown in **Table D.3.6** were ortho-rectified.

Table D.3.6: Study Site 2 Photogrammetric Sources.

Date	Source
4 July 1949	LANDGATE Imagery
2 June 1968	LANDGATE Imagery
12 September 1971	LANDGATE Imagery
13 June 1972	LANDGATE Imagery
15 November 1976	LANDGATE Imagery
15 August 1993	LANDGATE Imagery
16 May 2009	LANDGATE Imagery

The mangrove cover at Site 2 was mapped across the 60 years of historical data, to monitor changes to the overall appearance of the mangrove extents.

D.3.3.1 Results from Historical Shoreline Analysis of the Site 2

The mangrove coverage from the 1949 and 2009 aerial photography is shown on **Figure D.3.25**.

Overall only very minor changes have occurred to the shoreline in the Site 2 region across the 60 years of the historical photo data. The mangrove coverage has remained almost unchanged, with the mangrove extents in the 1949 and 2009 aerial data shown on **Figure D.3.25**. The mangrove cover has been constant throughout the 60 year period maintaining a natural defence along the shoreline in large wave events.

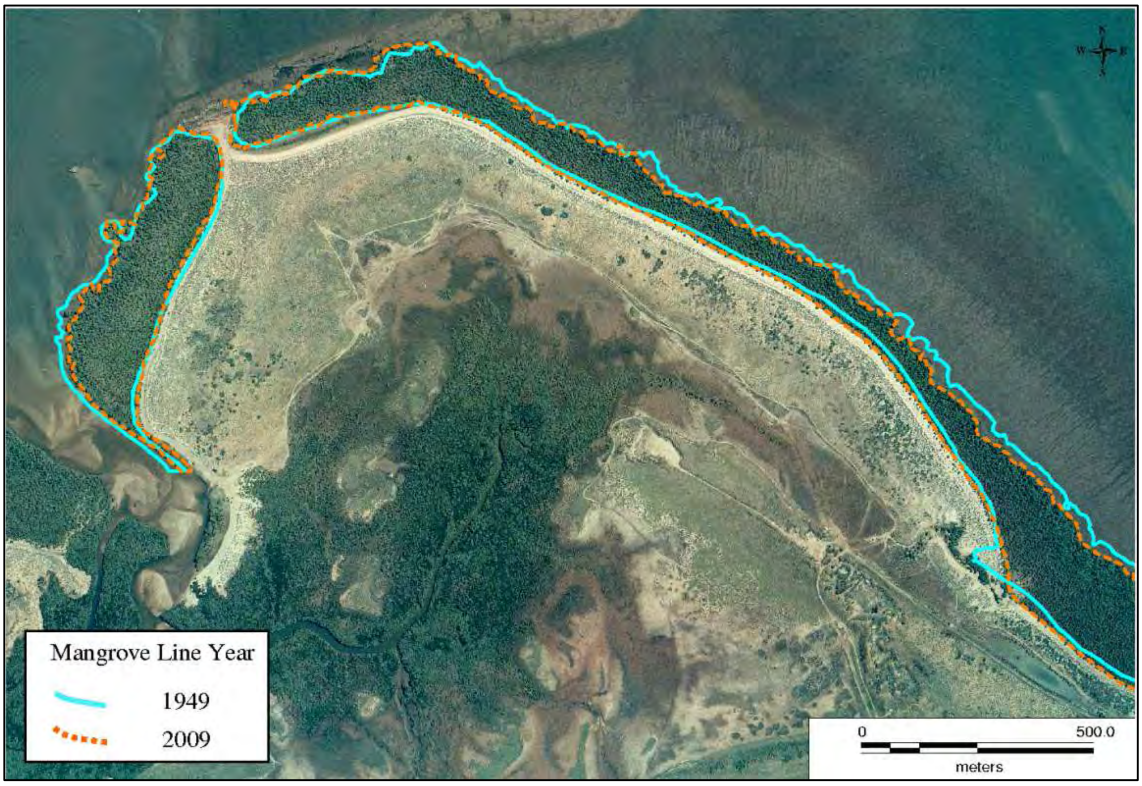


Figure D.3.25: Site 2 Mangrove Cover 1949 vs 2009.

Due to the historical stability of the study area the recommended historical setback distance for the purposes of S2 should be the default 20m.

D.4 SBEACH MODELLING

Short term acute (storm induced) erosion across the study sites was investigated using the SBEACH numerical model as recommended in SPP 2.6 for calculation of setback component S1. SBEACH (Storm-induced BEAch Change) was developed to calculate beach and dune erosion under storm wave action as described in Wise et al 1995.

Beach profiles were established at key locations in each of the study sites with the LIDAR dataset used to define the landward section of transects investigated in the SBEACH modelling. Each of the profile locations was chosen to be representative of the beach types within the study areas. For each profile the following was input to the SBEACH model:-

- Beach profile from the top of the fore dune to offshore based on the DTM (approx. +8m to -8m AHD);
- Depth of available sand below the beach profile;
- Depth of underlying rock strata (SBEACH 'hard bottom' feature);
- Sand Grain Size;
- Time series of Water Level for a Category 5 Cyclone Event (tide + storm surge);
- Time series of Wave Height (H_s) for a Category 5 Cyclone Event; and
- Time series of Peak wave Period (T_p) for a Category 5 Cyclone Event.

The response of the beach profile at each location was assessed for short term erosion based on design storms representative of a Category 5 Cyclone. The 500 year design storm was run three times consecutively for each SBEACH profile. The SBEACH model was also used to estimate the wave setup level for the design cyclone conditions along the selected profiles and the wave setup results were added to the ocean inundation modelling presented in **Appendix C** for open coast locations.

The SBEACH modelling adopted a medium grain size value of 0.30mm due to the absence of detailed site data. Sensitivity testing of the model based on sample sizes of 0.40mm and 0.20mm was undertaken with the following outcome:-

Sediment Grain Size	0.20mm	0.30mm	0.40mm
Impact to S1 Result	+ 20%	-	- 43%

The relative contribution of the SBEACH model component S1 to the total setback level (S1+S2+S3) is such that within the range of the model sensitivity the overall setback result is not greatly affected.

For almost all cases the major contributor to setback levels in the 100 year planning timeframe is Sea Level Rise (S3). On this basis the sediment grain size of 0.30mm is considered a reasonable approximation for the SBEACH modelling estimation of the S1 component.

D.4.1 SBEACH Modelling Port Hedland Town Site

Profiles for beach transects in the Port Hedland Study area were created at the locations shown on **Figure D.4.1** and **Figure D.4.2.** The change to the profile shoreline from the SBEACH model results were assessed at HAT (3.6m AHD) and at the vegetation / HSD line (4.2m AHD).

Due to the large tidal range of the Port Hedland region, the HAT line and vegetation line is impacted far more severely under design storm conditions than the Mean Sea Level Contour which is recommended under SPP2.6. The assessment of the setback component S1 is taken as the greater of the two impacts.

For comparison purposes, SBEACH modelling of Cyclone Connie and Cyclone John was additionally undertaken on each SBEACH profile and presented along with the 500 year design storm results. The results from the SBEACH model are presented in **Figure D.4.3** to **Figure D.4.9** and summarised in **Table D.4.1** below.

Table D.4.1 Results From SBEACH Modelling - Port Hedland Town Profiles

Port Hedland Profile	HSD Line (4.2m AHD)	HAT Line (3.6m AHD)	S1 (m)
Profile 1	*23m*	*1m*	23m
Profile 2	*28m*	*35m*	35m
Profile 3	*38m*	*21m*	38m
Profile 4	*40m*	*16m*	40m
Profile 5	*25m*	*10m*	25m
Profile 6	*19m*	*4m*	19m
Profile 7	*34m*	*13m*	34m

Figure D.4.1: Port Hedland SBEACH Model Transect locations 1 to 4.

Figure D.4.2: Port Hedland SBEACH Model Transect locations 5 to 7.

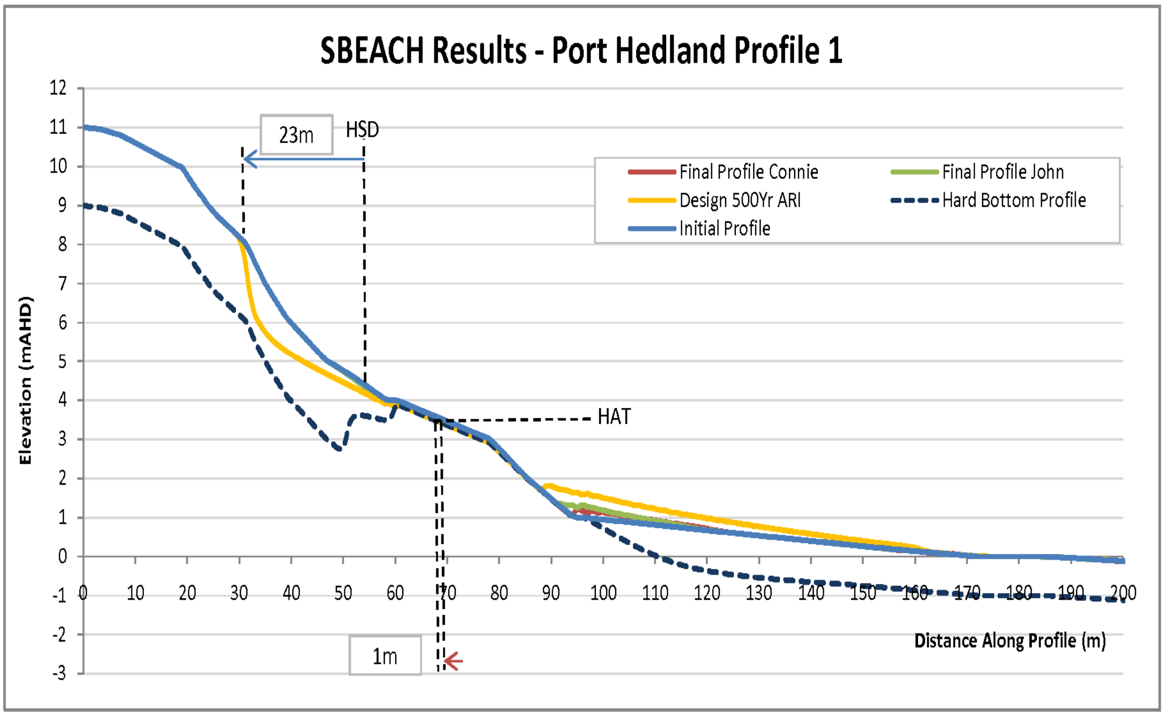


Figure D.4.3: Port Hedland SBEACH Model Results Transect 1.

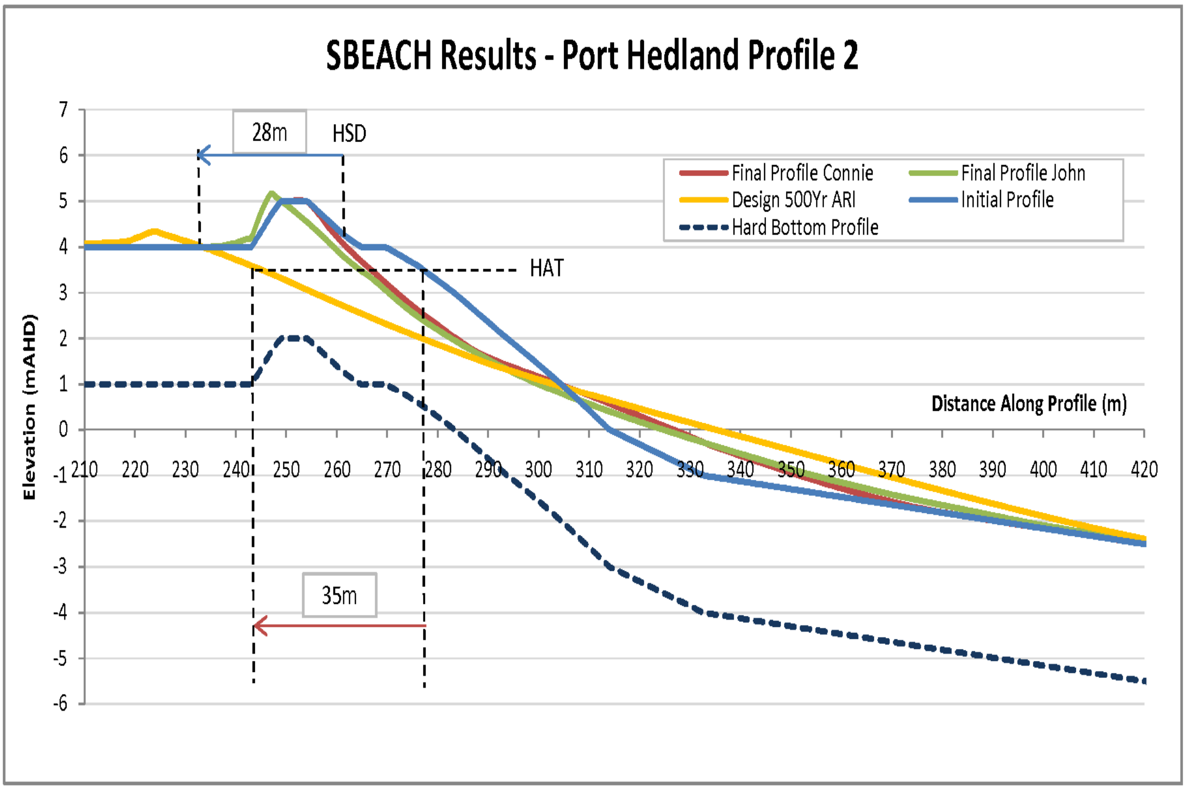


Figure D.4.4: Port Hedland SBEACH Model Results Transect 2.

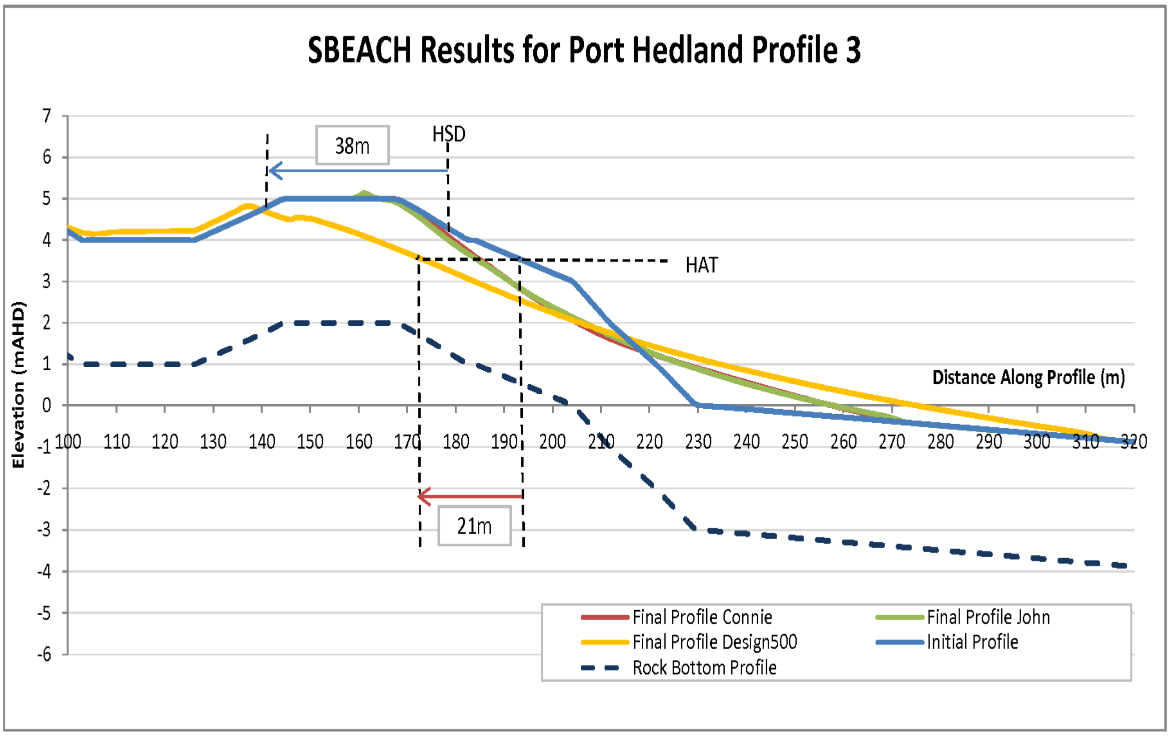


Figure D.4.5: Port Hedland SBEACH Model Results Transect 3.

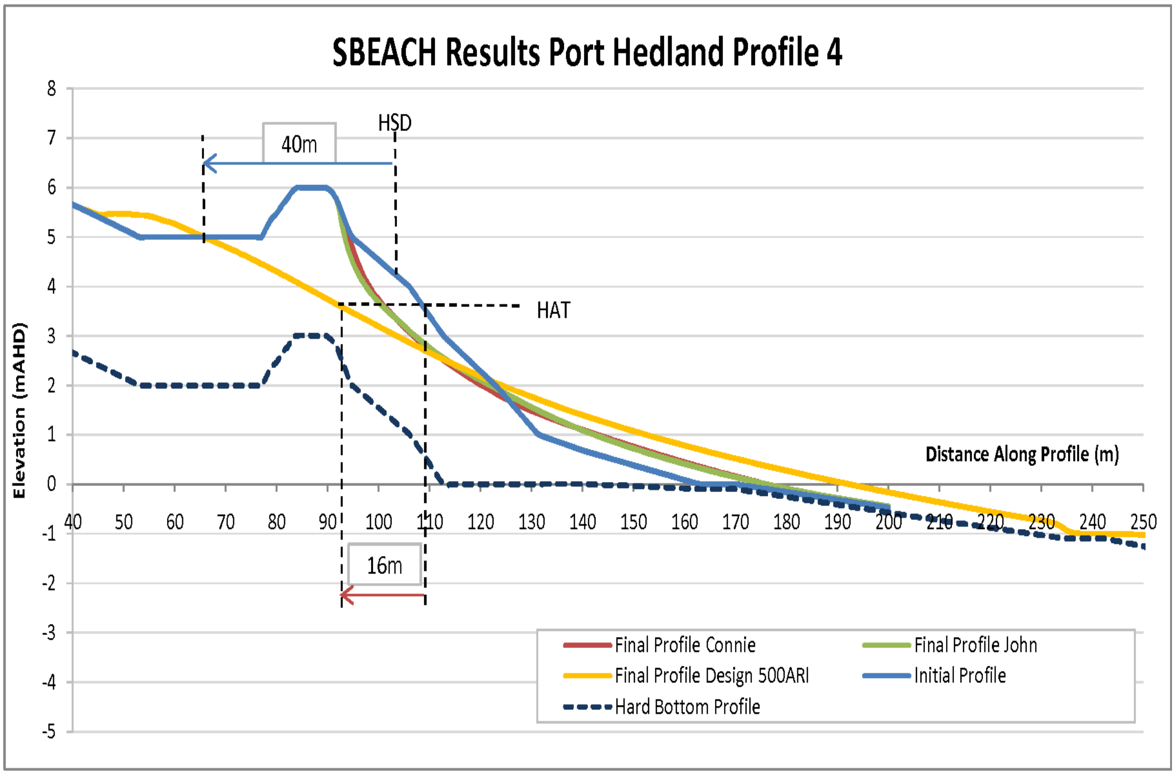


Figure D.4.6: Port Hedland SBEACH Model Results Transect 4

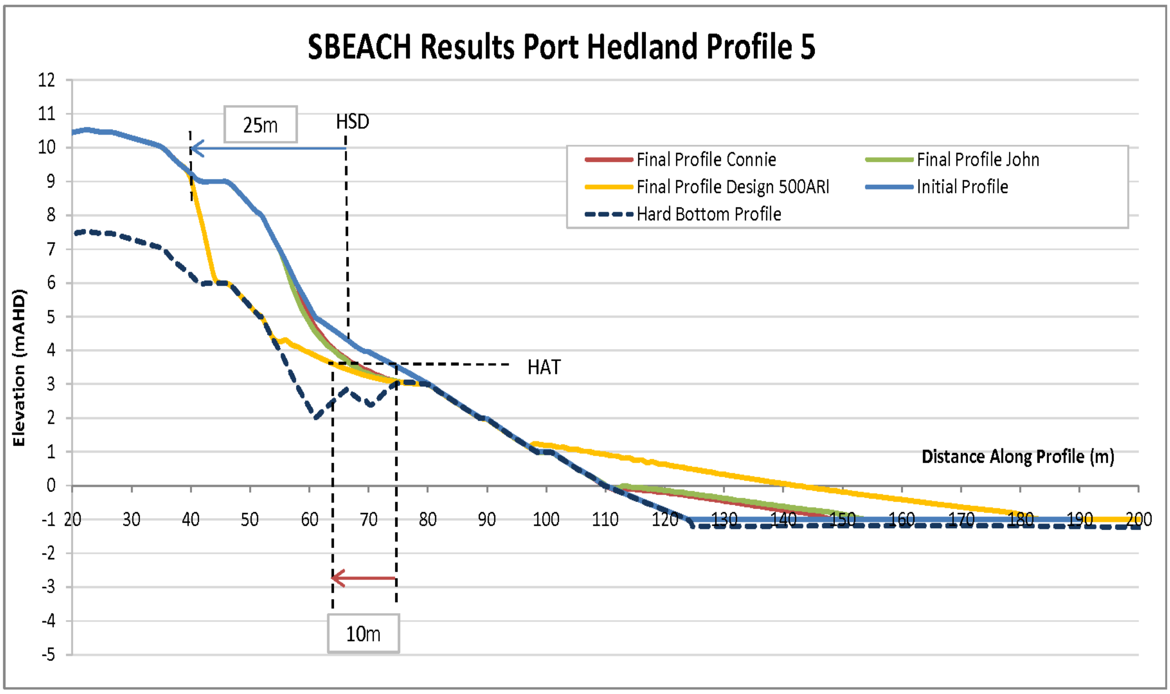


Figure D.4.7: Port Hedland SBEACH Model Results Transect 5.

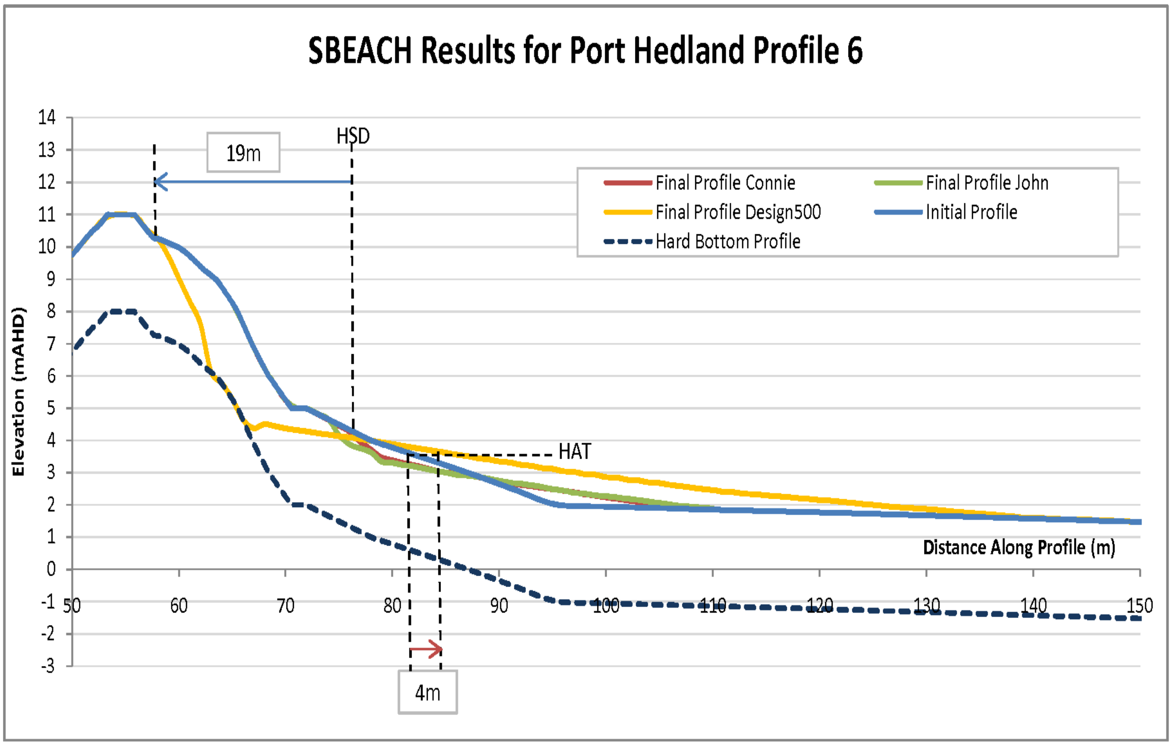


Figure D.4.8: Port Hedland SBEACH Model Results Transect 6.

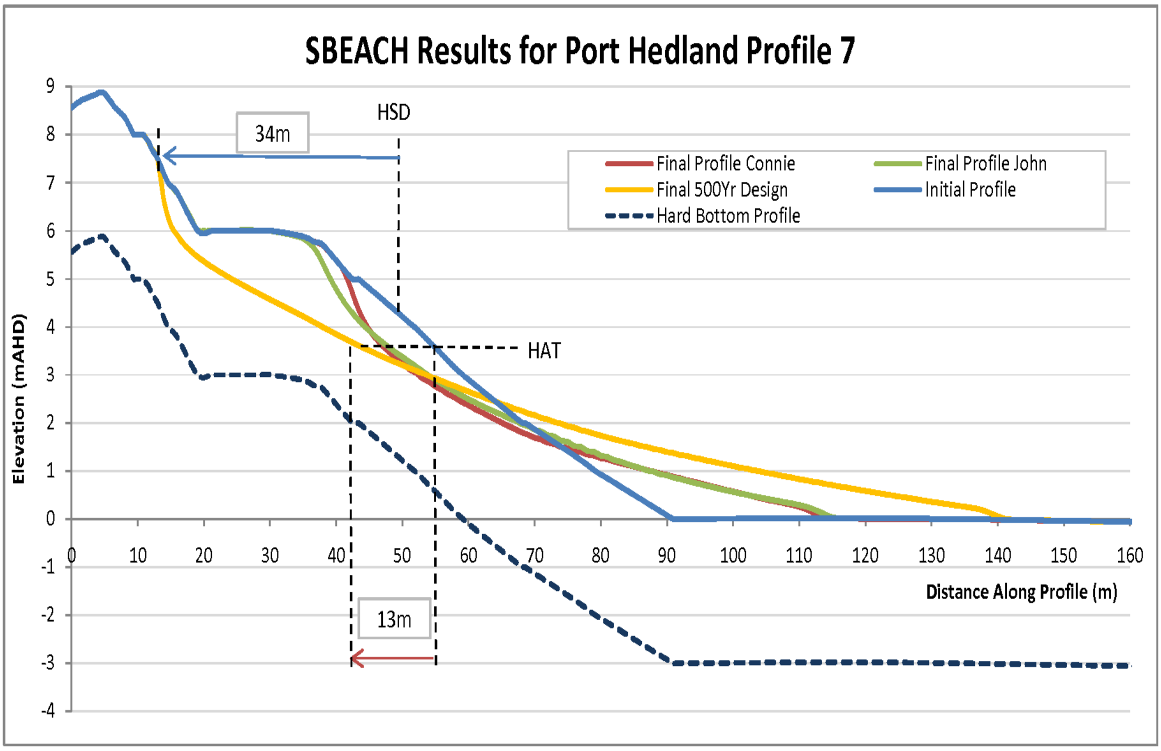


Figure D.4.9: Port Hedland SBEACH Model Results Transect 7.

D.4.2 SBEACH Modelling Shellborough Site

Profiles for beach transects in the Shellborough Study area were created at the locations shown on **Figure D.4.10.** The change to the profile shoreline from the SBEACH model results were assessed at HAT (3.6m AHD) and at the vegetation / HSD line (4.2m AHD). The assessment of the setback component S1 is taken as the greater of the two impacts.

The results from the SBEACH model are presented in **Figure D.4.11** to **Figure D.4.15** and summarised in **Table D.4.2** below.

Table D.4.2 Results From SBEACH Modelling - Shellborough Profiles

Shellborough Profile	HSD Line (4.2m AHD)	HAT Line (3.6m AHD)	S1 (m)
Profile 1	-44m	+11m	44m
Profile 2	-55m	+5m	55m
Profile 3	-51m	0m	51m
Profile 4	-35m	-6m	35m
Profile 5	-46m	+8m	46m

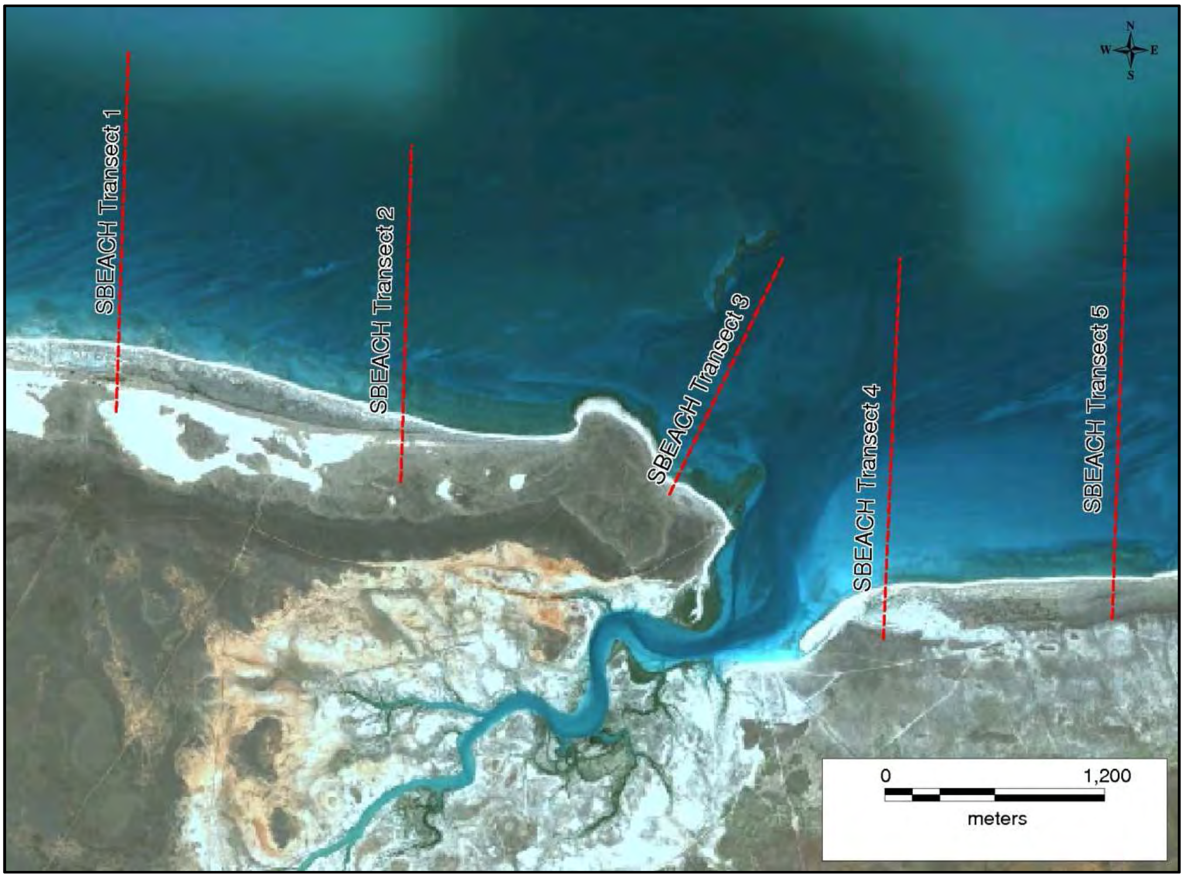


Figure D.4.10: Shellborough SBEACH Model Transect locations** 1 to **5.

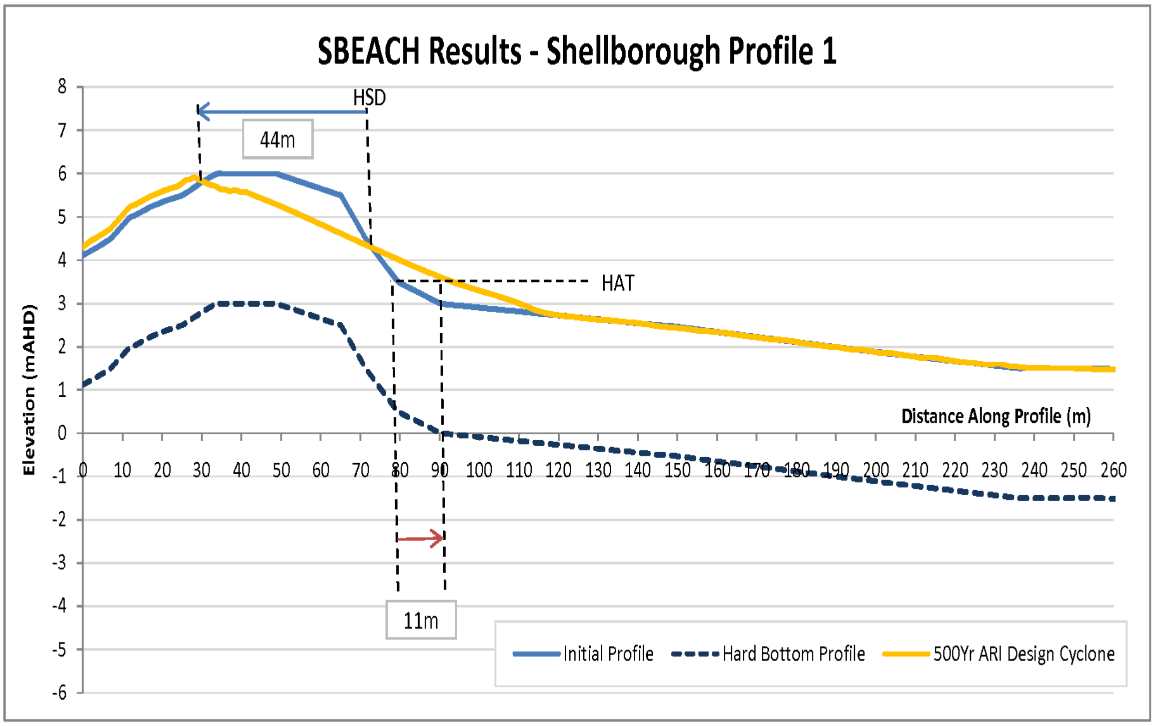


Figure D.4.11: Shellborough SBEACH Model Results Transect 1.

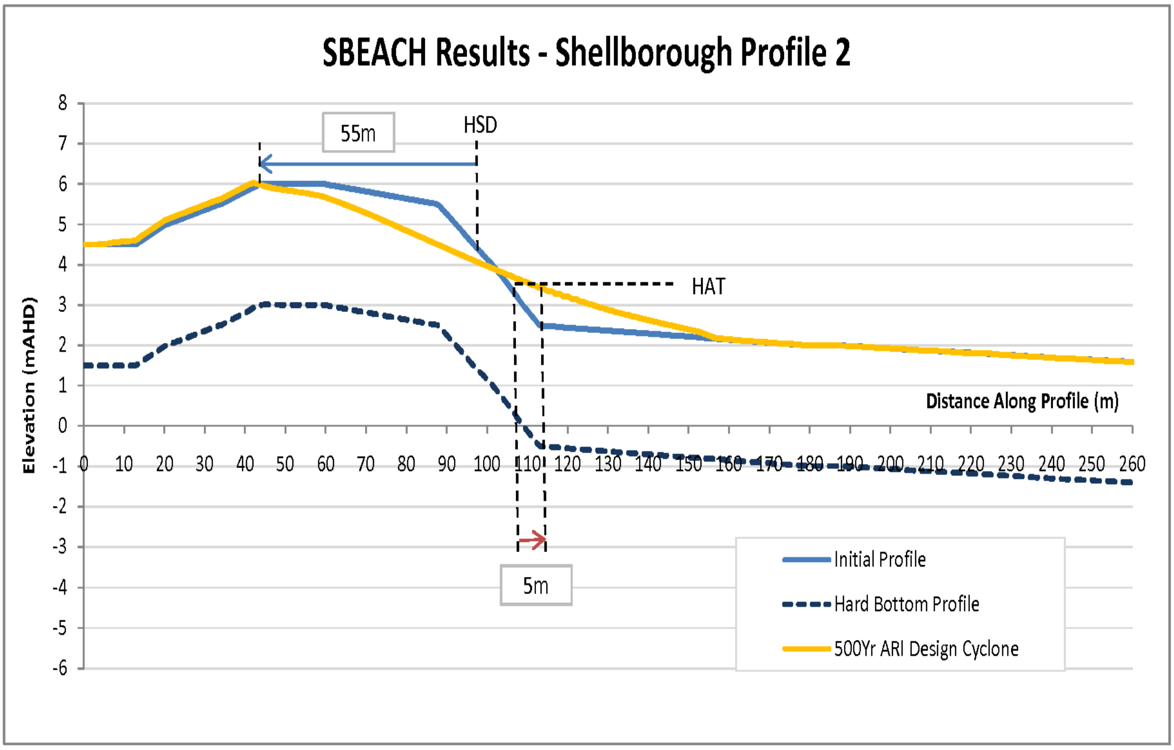


Figure D.4.12: Shellborough SBEACH Model Results Transect 2.

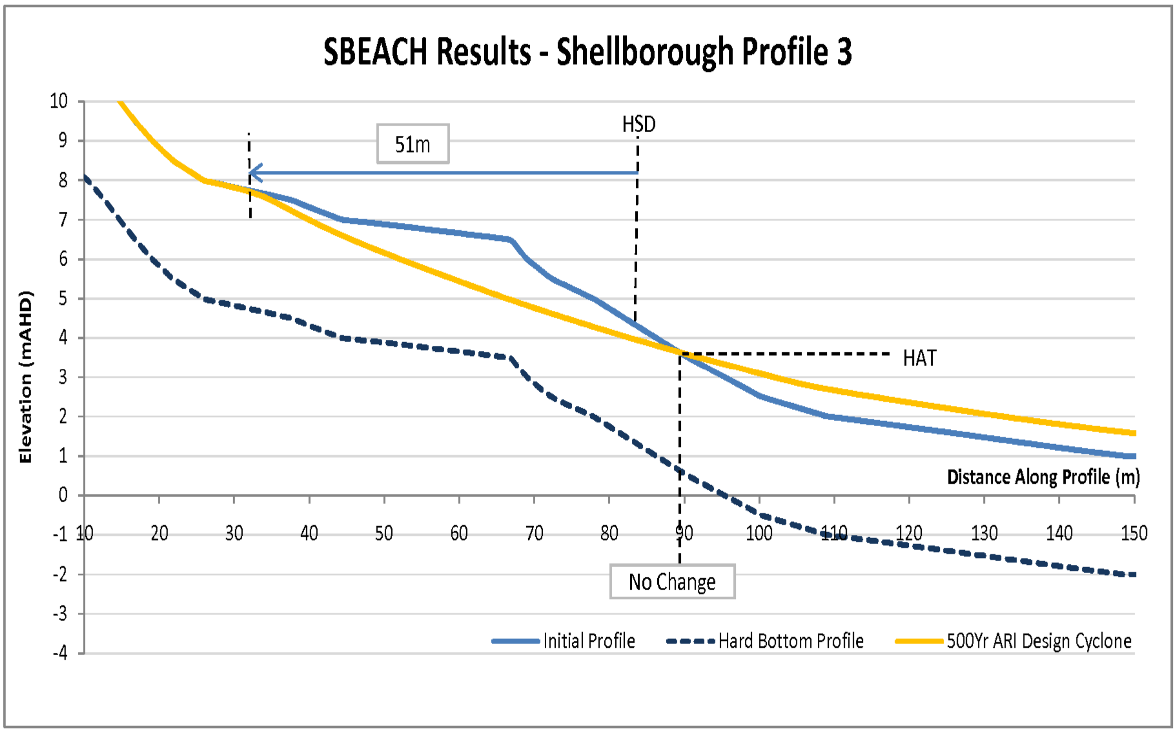


Figure D.4.13: Shellborough SBEACH Model Results Transect 3.

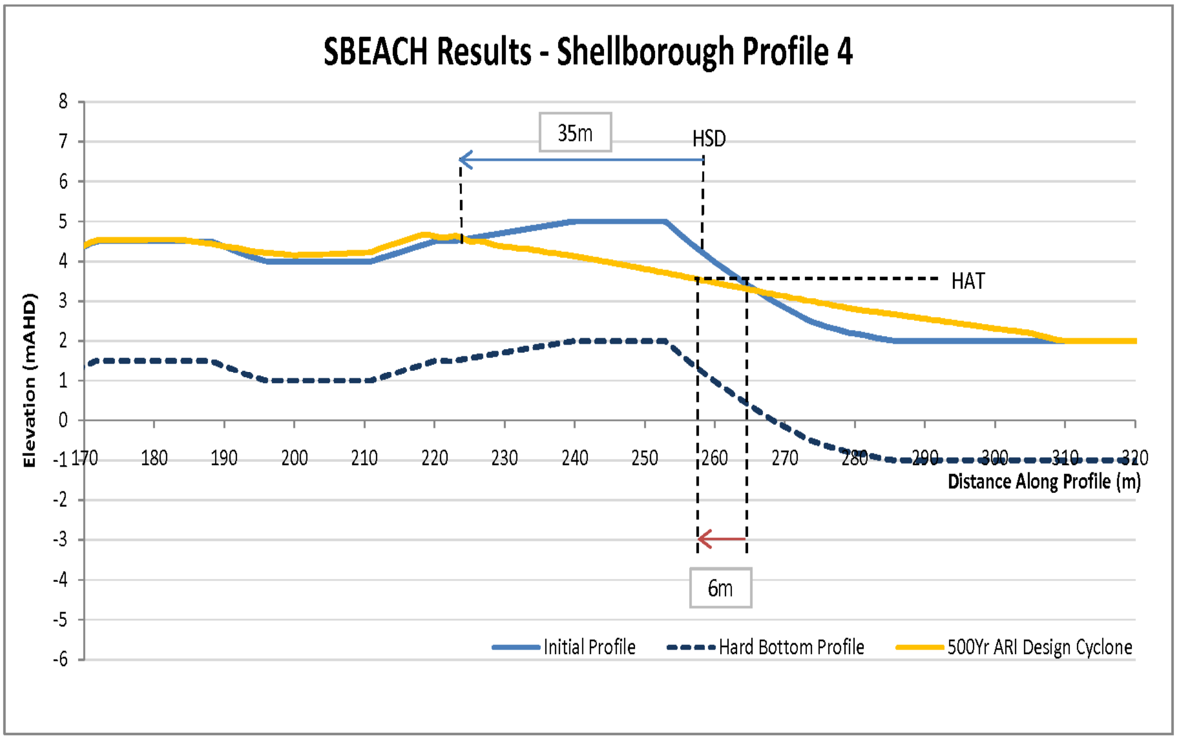


Figure D.4.14: Shellborough SBEACH Model Results Transect 4.

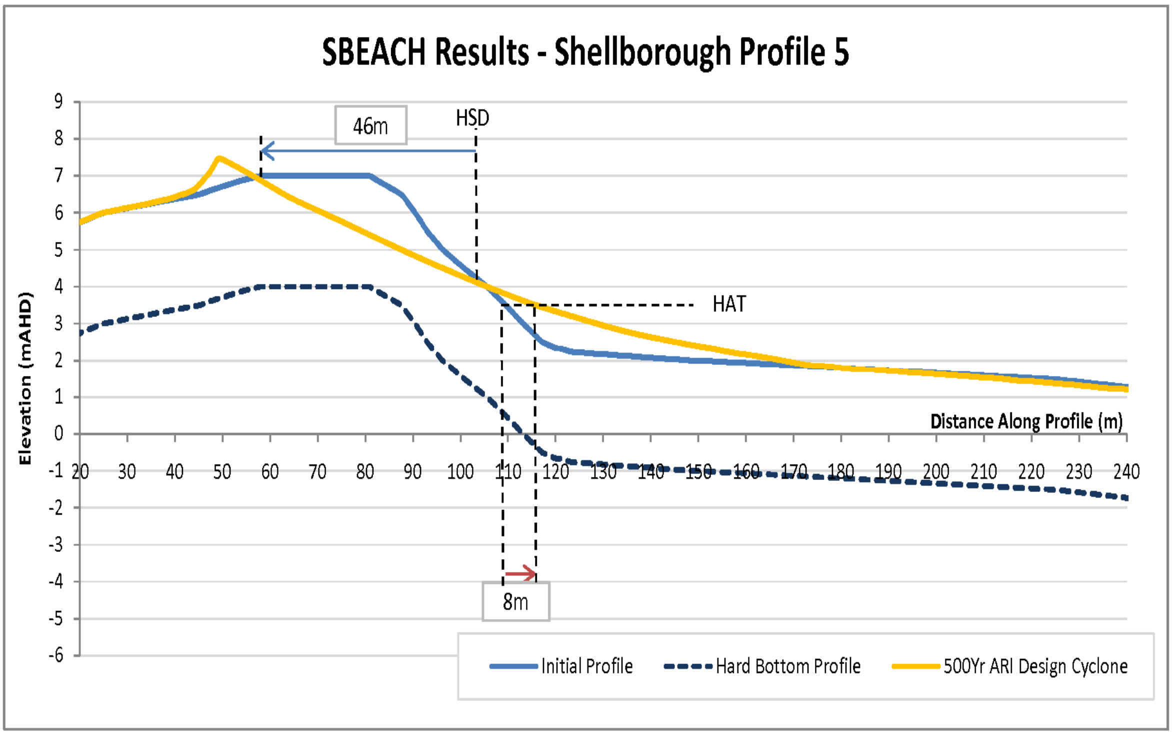


Figure D.4.15: Shellborough SBEACH Model Results Transect 5.

D.4.3 SBEACH Modelling Site 2

This site is surrounded by mangroves and cannot be effectively modelled using SBEACH (refer **SECTION D.3.3**) The mangrove cover has been in place across all the years of survey data with no discernible reduction, and it can be reasonably expected to continue to remain in place to provide an effective barrier against short term storm erosion. The small beach at the northwest of the site is a rocky platform, making the assessment of this small section unfeasible.

For setback purposes the S1 component is assigned as the default value of 40m, outlined in SPP2.6 for areas where detailed modelling is not undertaken.

D.5 SEA LEVEL RISE AND CLIMATE VARIATION

In 2010 the magnitude of sea level rise recommended for coastal setback planning in WA was updated in SPP2.6 for planning periods up to 100 years. This decision was guided by revised projections for global mean sea level from the Intergovernmental Panel on Climate Change Fourth Assessment Report (IPCC AR4, 2007) and studies completed by the CSIRO investigating local variations in mean sea level around the Australian coastline (CSIRO (2010).

For future planning in WA, the upper bound of the global sea level rise projections in IPCC AR4 were recommended for a climate scenario consistent with modelled timelines of SRES scenario A1F1 (Hunter 2009). The 95th percentile projection was adopted due to the uncertainty associated with the contributing factors to the climate change models and this scenario is shown on **Figure D.5.1.**

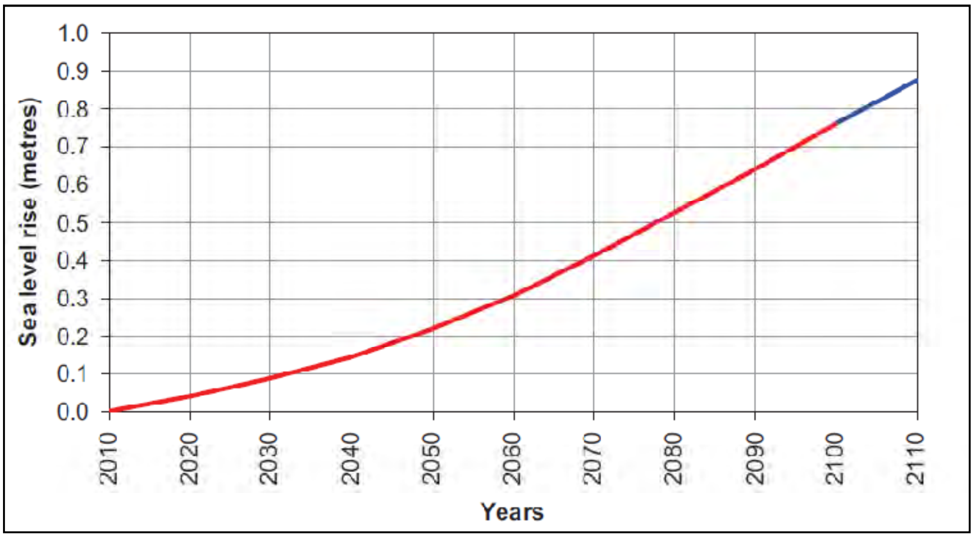


Figure D.5.1: Recommended Allowance for Sea Level Rise in Coastal Planning in WA (SRES scenario A1F1 95th Percentile after Hunter 2009)

For the 100 year planning timeframe (2010 to 2110) DOT recommended a vertical SLR of 0.9m be adopted, whilst in the 50 year planning timeframe 0.3m vertical SLR is appropriate. For planning timeframes beyond 100 years a vertical SLR of 0.01 m/year is added to the 2110 figure. These projections are due to be reviewed when the IPCC Fifth Assessment Report is completed (projected date of 2013).

SPP2.6 recommends the Bruun rule (Bruun 1962) be used for calculation of a setback distance based on the vertical SLR component. For sandy shores a multiplication factor of 100 is applied under the Bruun rule. For the 100 year planning timeframe a vertical SLR of 0.9m results in a horizontal setback distance (S3) of 90m. For the 50 year planning timeframe a setback distance (S3) of 30m is applicable. In the context of Port Hedland where underlying rock is present along most of the open coast shoreline, the direct application of the Bruun rule is likely to be a conservative assessment of the horiztonal recession due to SLR. If more detailed S3 setback distances are required, for example for a particular parcel of land, it is recommended that a geotechnical site investigation be undertaken to determine the underlying sediment composition of the site and then assess the potential for shoreline recession due to SLR if underlying rock is present.

For other shore types the setback distance S3 is assessed with regard to local geography. In the case of rocky shorelines, the S3 component is accounted for in the default setback value of 50m. This definition has been applied in this study for the Port Hedland Township in **Zone 5**.

DOT (2010) emphasised that new and existing development will need to consider the impact of future sea level rise to factors beyond the vertical component such as the increase in the frequency and potential severity of existing storm inundation events.

This has been considered in SBEACH testing of the Port Hedland profiles based on design storms modelled for the region under the 500 year ARI climate change scenario. This scenario includes sea level rise for the 100 year planning timeframe (+0.9m) as well as increased intensity in wind speed (and hence wave conditions) as summarised in the Ocean Inundation Modelling (**Appendix A**). For these modified outcomes the SBEACH results are presented on **Table D.5.1** below. The impact of the SLR on the storm erosion is most significant for profiles 2 and 3 which are on the Spoil Bank and have the most mobile sand on the beach profile compared to the other profiles where underlying and surface rock limits the degree of shoreline erosion.

Table D.5.1 Results From SBEACH Modelling - Port Hedland Town Profiles - SLR 0.90m

Port Hedland Profile	HSD Line (4.2m AHD)	HAT Line (3.6m AHD)	S1 (m)	Increase in S1 (m)
Profile 1	*25m*	*5m*	25m	2
Profile 2	*60m*	*59m*	60m	25
Profile 3	*85m*	*36m*	85m	47
Profile 4	*37m*	*32m*	37m	3
Profile 5	*30m*	*14m*	30m	5
Profile 6	*19m*	*4m*	19m	0
Profile 7	*40m*	*18m*	40m	6

From this it can be concluded there generally is an increase in the S1 setback component, which is most pronounced at the Port Hedland spoil ground transects of **Profile 2** and **Profile 3,** for the 0.9 SLR scenario. In all other cases the increase in the S1 component is negligible and the overall contribution from the SLR component (S3) represents a much greater proportion of the overall CPS (90m for the 2110 planning period).

D.6 COASTAL PROCESSES SETBACK SUMMARY

D.6.1 Port Hedland Study Site

The outcome of the setback investigations for the Port Hedland Township are summarised on **Table D.6.1** and **Table D.6.2** for the 50 year and 100 year planning period respectively. The immediate CPS hazard level for the region is defined by values in the Short Term (S1) column in **Table D.6.1.**

Table D.6.1: Port Hedland Site Coastal Processes Setback - 50 year Planning Period

Port Hedland Study Site					
Zone	**Short Term S1 (m)**	**Historical S2 (m)**	**Sea Level Rise S3 (m)**	**Total (m)**	**Comments**
1	23	10	30	**63**	Detailed geotechnical assessment, for erosion over 50 year planning period recommended. Seawall section is treated as rock shoreline *
2	23	10	30	**63**	Detailed geotechnical assessment, for erosion over 50 year planning period recommended
3	*28*	-	-	**-**	Spoil Bank Area - beyond Scope of this study
4	*38*	-	-	**-**	Spoil Bank Area - beyond Scope of this study
5	23	-	-	**25***	Default value for rocky shoreline
6	40	34	30	**104**	
7	25	17	30	**72**	Detailed geotechnical assessment, for erosion over 50 year planning period recommended
8	25	15	30	**70**	Detailed geotechnical assessment, for erosion over 50 year planning period recommended
9	25	10	30	**65**	Detailed geotechnical assessment, for erosion over 50 year planning period recommended
10	19	24	30	**73**	
11	-	-	-	**-**	Estuary Entrance
12	-	14	30	**44**	Estuary System - no S1 component
13	-	-	-	**-**	Estuary Entrance
14	34	27	30	**91**	

* The rock fronting Zone 5 and the seawall at Zone 1 create a barrier against erosion from short term events, whilst at the same time fixing the shoreline position during future events. The default total setback value for rocky shorelines of 25m for the 50 year period is adapted from the SPP2.6 recommendation of 50m for a 100 year planning horizon.

Table D.6.2: Port Hedland Site Coastal Processes Setback - 100 Year Planning Period

Port Hedland Study Site					
Zone	**Short Term S1 (m)**	**Historical S2 (m)**	**Sea Level Rise S3 (m)**	**Total (m)**	**Comments**
1	23	20	90	**133**	Detailed geotechnical assessment, for erosion over 100 year planning period recommended Seawall section is treated as rock shoreline *
2	23	20	90	**133**	Detailed geotechnical assessment, for erosion over 100 year planning period recommended
3	*28*	-	-	**-**	Spoil Bank Area - beyond Scope of this study
4	*38*	-	-	**-**	Spoil Bank Area - beyond Scope of this study
5*	23	-	-	**50***	Default value for rocky shoreline
6	40	68	90	**198**	
7	25	35	90	**150**	Detailed geotechnical assessment, for erosion over 100 year planning period recommended
8	25	30	90	**145**	Detailed geotechnical assessment, for erosion over 100 year planning period recommended
9	25	20	90	**135**	Detailed geotechnical assessment, for erosion over 100 year planning period recommended
10	19	47	90	**156**	
11	-	-	-	**-**	Estuary Entrance
12	-	28	90	**118**	Estuary System - no S1 component
13	-	-	-	**-**	Estuary Entrance
14	34	53	90	**177**	

* The rock fronting Zone 5 and the seawall at Zone 1 create a barrier against erosion from short term events, whilst at the same time fixing the shoreline position during future events. The default CPS value of 50m from SPP2.6 is applied.

Figure D.6.1 to **Figure D.6.3** present results for the immediate, 50 year and 100 year planning horizons in the Port Hedland study area. The following sets of data are shown on each figure :-

1. Coastal Processes Setback lines based on results summarised in **Table D.6.1** and **Table D.6.2**
2. Inundation levels associated with the 100 year ARI event and the 500 year ARI event as calculated in the Ocean Inundation Assessment (**Appendix A**). The 500 year ARI event is representative of a category 5 cyclone.

Figure D.6.1: Port Hedland Coastal Processes Setback - Immediate (2010) Inundation Extent

Figure D.6.2: Port Hedland Coastal Processes Setback - 50 year Planning Horizon (2060).

Figure D.6.3: Port Hedland Coastal Processes Setback - 100 year Planning Horizon (2110).

D.6.2 Shellborough Study Site

The outcome from the setback investigations for the Shellborough Site are summarised on **Table D.6.3** and presented graphically on **Figure D.6.4** and **Figure D.6.5.**

Table D.6.3: Shellborough Coastal Processes Setback - 100 Year Planning Period

Shellborough Study Site					
Zone	S1 (m)	S2 (m)	S3 (m)	Total (m)	Comments
1	55	20	90	**165**	
2	51	20	90	**161**	
3	35	275	90	**400**	Low Lying Shoreline Region
4	46	20	90	**156**	Low Lying Shoreline Region

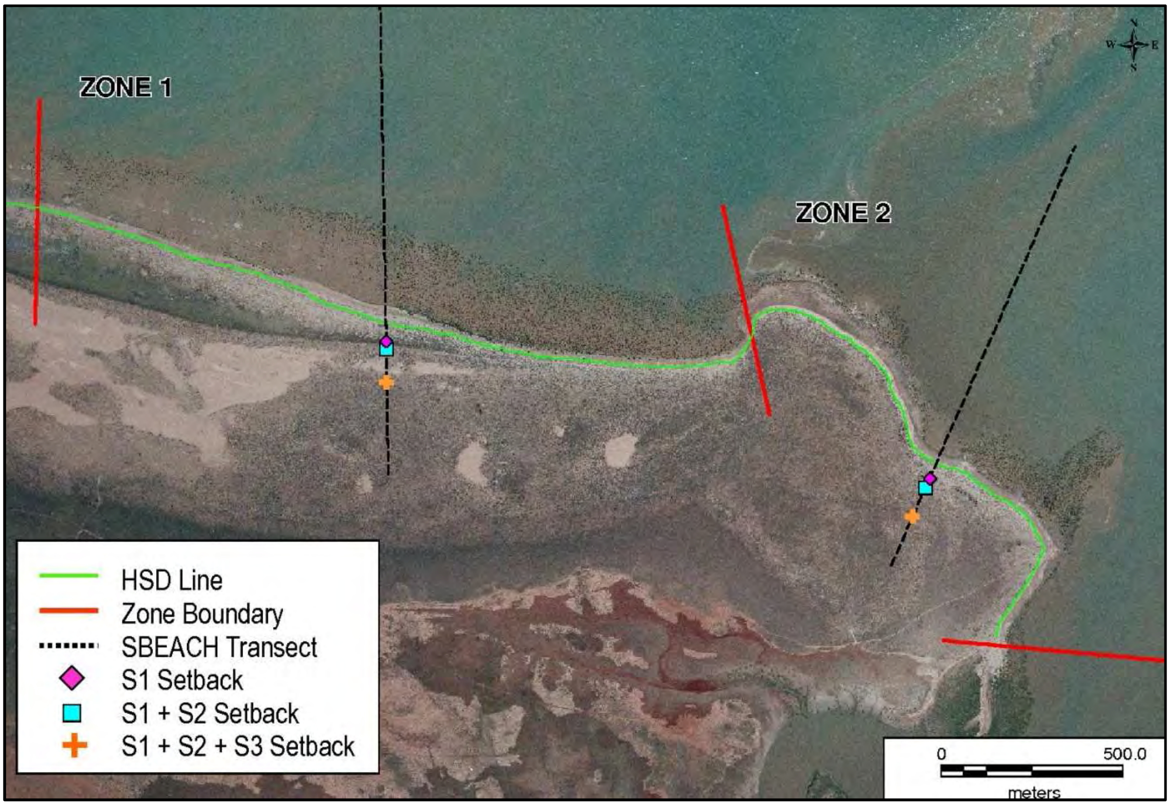


Figure D.6.4: Shellborough Coastal Processes Setback Summary Zone 1 and Zone 2

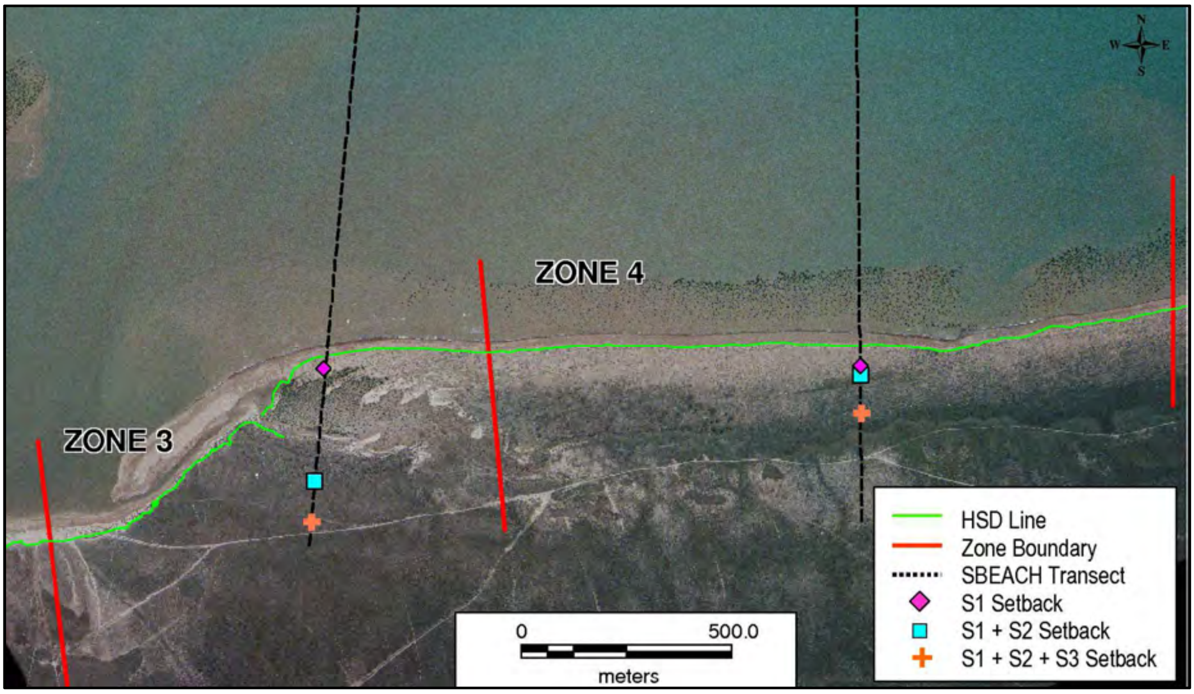


Figure D.6.5: Shellborough Coastal Processes Setback Summary Zone 3 and Zone 4

Figure D.6.6 to **Figure D.6.8** present inundation results for the immediate, 50 year and 100 year planning horizons in the Shellborough study area. Inundation levels associated with the 100 year ARI event and the 500 year ARI event as calculated in the Ocean Inundation Assessment (**Appendix A**) are shown.

Clearly the majority of the eastern side is of Condon creek is low lying and inundated in a 100 year ARI and 500 year ARI event. This effectively places the **Zone 3** and **Zone 4** regions outside the scope of development under the considerations of SPP 2.6 for cyclone risk areas.

On the western side of Condon creek entrance there is a high dune ridge behind **Zone 1** and **Zone 2** extending westward that is above the 500 year ARI inundation line in the present, 50 year and 100 year planning periods.

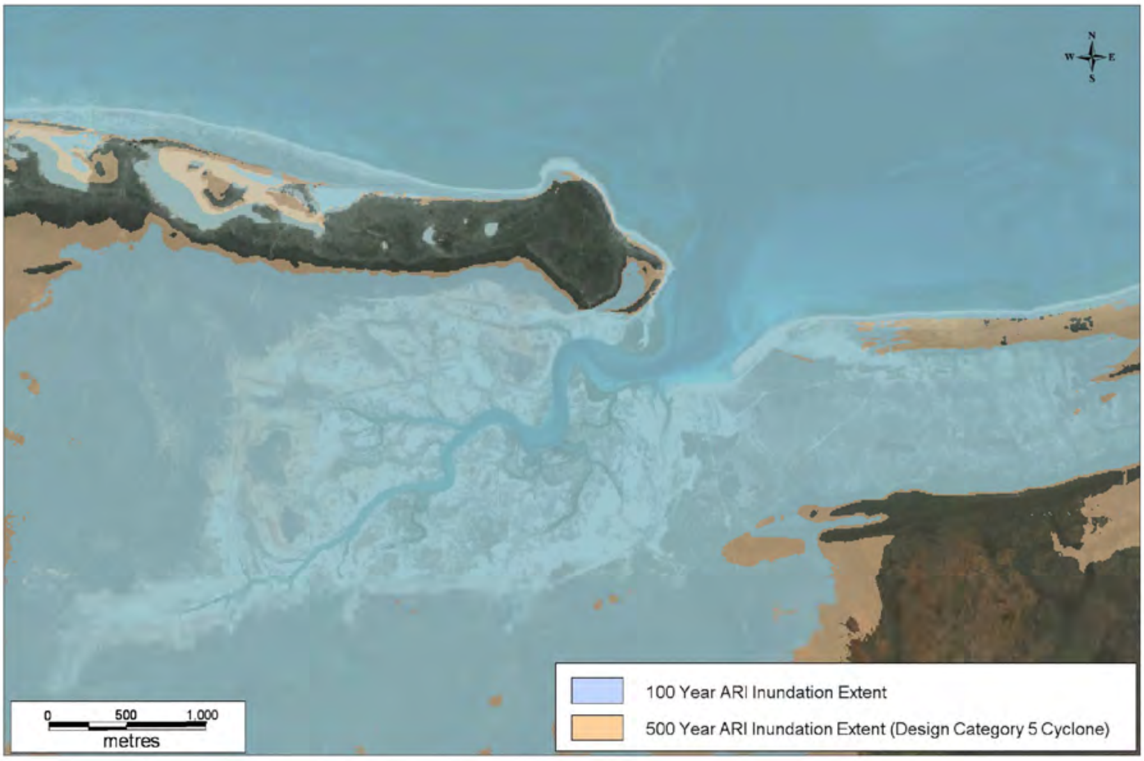


Figure D.6.6: Shellborough Inundation Extents - Immediate Level (2010).

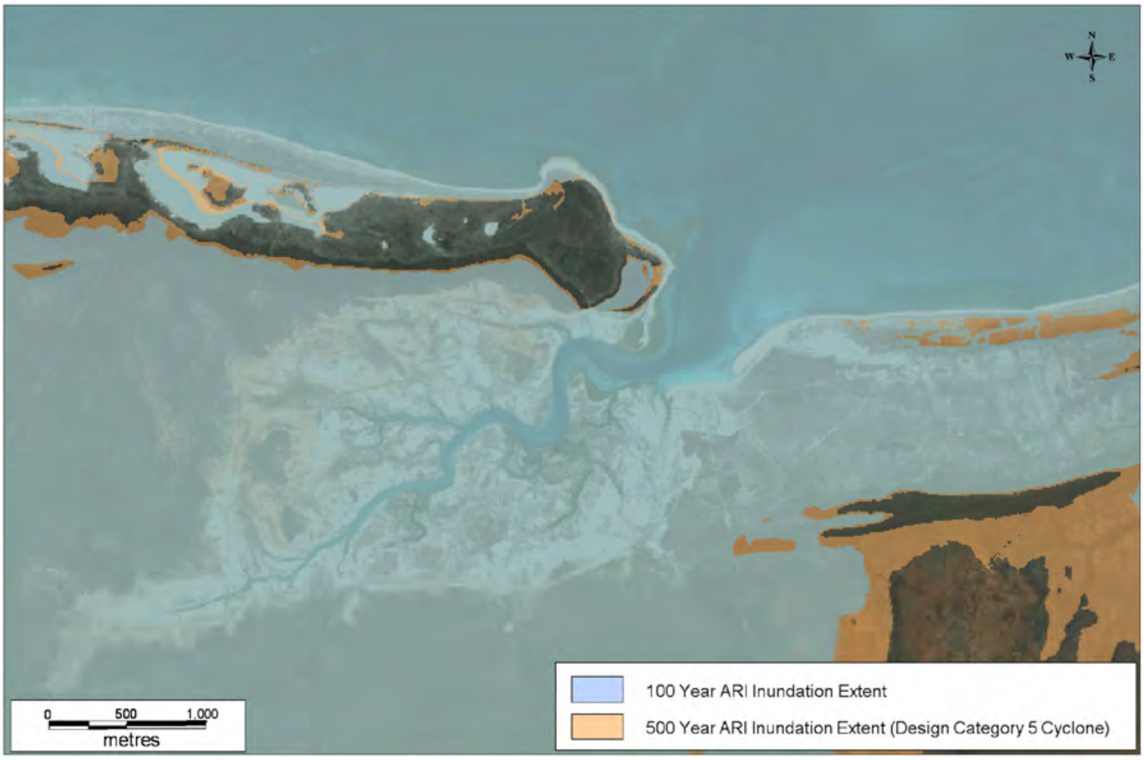


Figure D.6.7: Shellborough Inundation Extents - 50 Year Planning Period (2060).

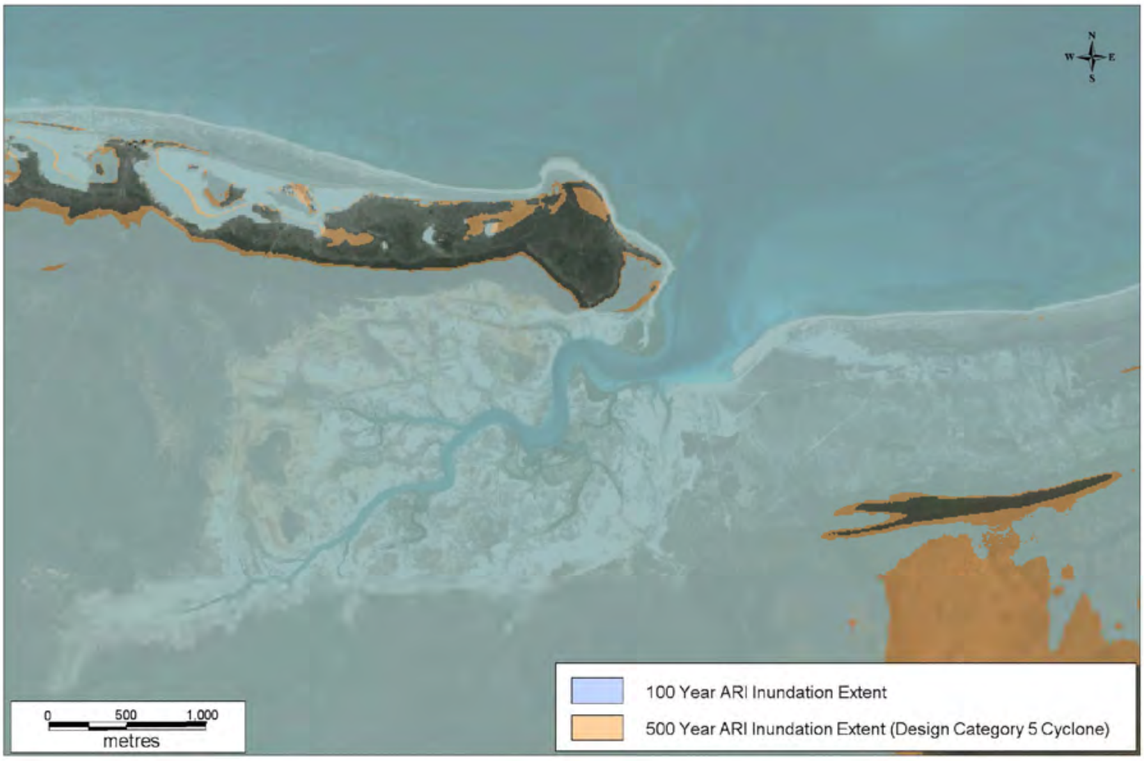


Figure D.6.8: Shellborough Inundation Extents - 100 Year Planning Period (2110).

D.6.3 Study Site 2

For Study Site 2, the CPS for the 100 year planning horizon is shown on **Table D.6.4**. Default values for the Short Term (S1) and Historical (S2) setback components have been applied in the absence of detailed modelling.

Table D.6.4: Site 2 ('The Bus Stop') Coastal Processes Setback - 100 Year Planning Period

Site 2 Study Area					
Zone	S1 (m)	S2 (m)	S3 (m)	Total (m)	Comments
1	40	20	90	**150**	Mangrove fronted shoreline which is dominated by the ocean inundation extents.

The inundation extents for Site 2 are shown on **Figure D.6.9** for the present climate condition with no SLR. This clearly indicates the majority of Site 2 is inundated in a 100 year ARI event. Site 2 is completely inundated in the 500 year event which is representative of a Category 5 cyclone, an outcome which over rides all calculations considered for the CPS at this location under the SPP 2.6 guidelines for cyclone risk areas.

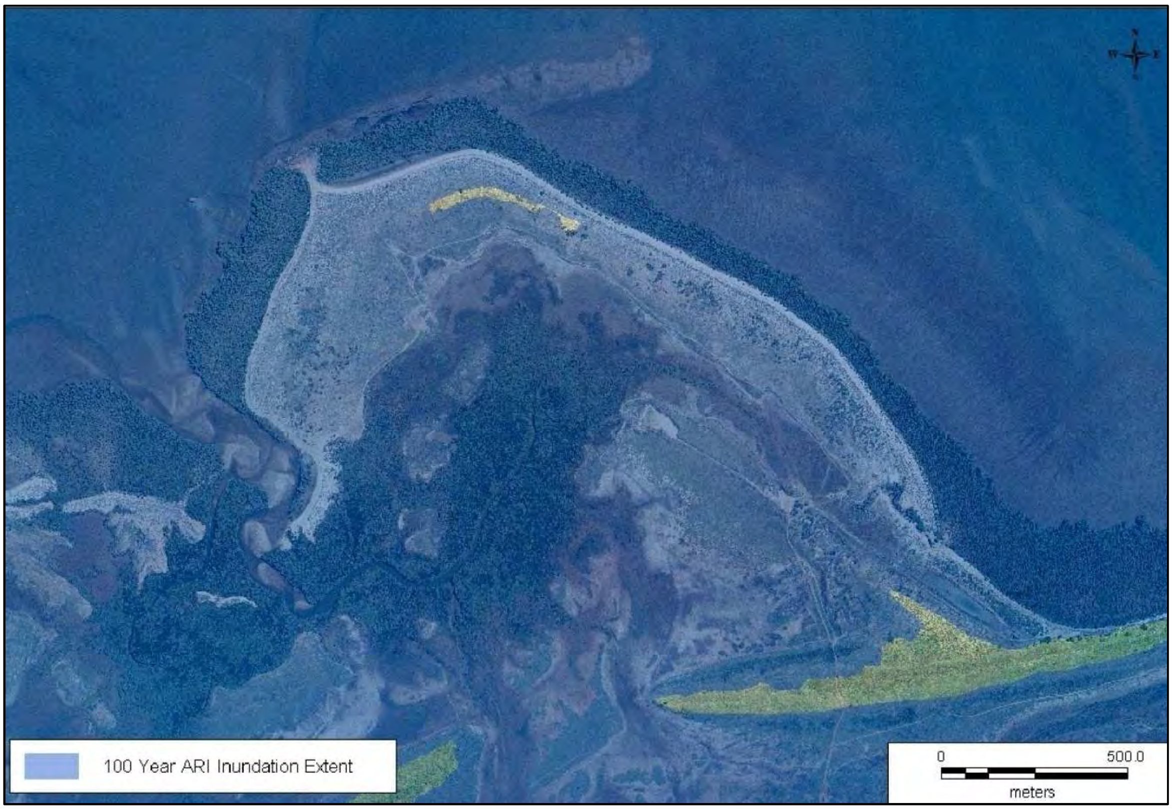


Figure D.6.9: Site 2 100 Year ARI Inundation Extents including Shoreline Wave Setup - Immediate Level (2010).

D.7 CONCLUSIONS

The shoreline stability assessment has been conducted based on the methodology described in SPP 2.6 for the treatment of shorelines in cyclone affected areas. Assessment of the coastal processes setback level has been completed based on the combination of the necessary allowances for:-

- Short term changes to the shoreline resulting from a category 5 cyclone (S1) ;
- Historical shoreline changes over approximately 60 years of aerial images (S2); and
- Allowance for shoreline recession associated with a rise in sea level projected at 0.3m for the 50 year planning period and 0.9m for the 100 year planning period (S3) .

The following conclusions are made from the outcomes of this report:-

- There is a low lying section of shoreline east of the spoil bank (approximately 6 mAHD) which serves as a break through point for flows from the open coast which includes a shoreline wave setup contribution in extreme events. This break point directs water across currently existing properties and into lower lying land in East Port Hedland as well as the main business area of Port Hedland and increases the inundation area and depths in these areas.

- The Spoil Bank is inundated in both the 100 year and 500 year ARI event, with only a small portion left unaffected. Wave run-up results in even greater inundation potential for structures on the Spoil Bank.

- The region of East Port Hedland is at risk of inundation in the immediate term as flows from Pretty Pool estuary and the Four Mile creek Estuary flow into this low lying region. At the most extreme 500 year ARI event flows are directed from the breakout point east of the spoil bank to further contribute to inundation levels. It should be noted however that even with the break through, the inundation water levels in East Port Hedland are lower than on the open coast as a result of the limited volume of water which can flow through the break through channel.

- The Pretty Pool development remains above the 500 year inundation level (Category 5 Cyclone) in the immediate term as well as the 50 year and 100 year planning periods.

- For the 100 year planning period when adopting a sea level rise component of 0.9m and modelling increased storm intensity, there is a significant impact to the main business area of Port Hedland and near the East Port Hedland area with the present landform levels. For the 100 year ARI event the inundation level is such that the business area of Port Hedland and surrounding streets are completely inundated.

- In the calculation of potential shoreline recession due to SLR in this study, the potential presence of possible underlying rock which limits the landward movement of the beach profile has not been considered in this study. If more detailed S3 setback distances are required, for example for a particular parcel of land, it is recommended that a geotechnical site investigation be undertaken to determine the underlying sediment composition of the site and then assess the potential for shoreline recession due to SLR if underlying rock is present.

- Site 2 is at significant risk of inundation in the 100 year ARI event in present conditions, with only a small portion of the ridge above the water level. For the 500 year ARI design the site is completely inundated. In the 50 year and 100 year planning periods with additional sea level rise this site is completely inundated in all design storm events. Severe property and infrastructure damage would likely occur at this site under the design cyclone conditions.

- The Shellborough site is particularly low lying through the region adjacent Condon Creek, as well as the entire region east of the creek mouth and these areas are inundated in the 100 year ARI and 500 year ARI events. There is a high section of dune which extends from the mouth of the creek to the west and this remains above the 500 year ARI inundation extent in the present term and in the 100 year planning horizon (2110).

D.8 REFERENCES

Bureau of Meteorology (2010), "Tropical Cyclones Affecting Port Hedland", Prepared by Australian Government Bureau of Meteorology Available at http://www.bom.gov.au/wa/cyclone/about/pthed/index.shtml

Bureau of Meteorology (2007), "Meteorological Aspects of Severe Tropical Cyclone Georges Impact on the Pilbara", Prepared by Australian Government Bureau of Meteorology. Available at http://www.bom.gov.au/cyclone/history/pdf/george.pdf

Brunn P., 1962. "Sea Level Rise as a Cause of Shore Erosion". Journal of Waterways Harbors Division, American Society of Civil Engineers, 88, 117-130

CSIRO (2007). "Climate Change in Australia". Published by CSIRO 2007. ISBN 9781921232947.

CSIRO (2010). “Sea Level Rise – Projections for the Australian Region”. Prepared by the CSIRO for the Wealth from Ocean National Research Flagship. Available at: Projections - http://www.cmar.csiro.au/sealevel/sl_proj_regional.html.

Department of Transport (2010) "Sea Level Change in Western Australia Application to Coastal Planning". Prepared by Coastal Infrastructure, Coastal Engineering Group

Garratt D, McCarthy M, Shaw R (1997) "Condon Maritime Heritage Site Inspection Report". Report by Department of Maritime Archeology, WA Maritime Museum, No. 128.

Hunter J (2009) "Estimating sea-level extremes under conditions of uncertain sea-level rise'. Climatic Change, DOI:10.1007/s10584-009-9671-6, published online at www.springerlink.com

IPCC (Intergovernmental Panel on Climate Change), 2007, "Climate Change 2007 : the Physical Science Basis", Contribution of Working Group 1 to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change, S Soloman, D Qin, M Manning, Z Chen, M Marquis, KB Averyt, M Tignor and HL Miller (eds), Cambridge University Press, Cambridge, United Kingdom &d New York, USA

Paul M. J. (2001). "Review of Silting Rates and Calculations of Dredge Volumes in the Port Hedland Harbour Channel, Turning Basin and Berths". Prepared for the Port Hedland Port Authority.

Short A. D. (2006) "Beaches of the Western Australian Coast : Eucla to Roebuck Bay" Sydney University Press

USACE (1984). "Shore Protection Manual (1984)". Published by U.S. Army Corps of Engineers, Washington, DC.

WAPC(2003) "Statement of Planning Policy No 2.6 – State Coastal Planning Policy". Prepared by the WAPC and the Department of Planning, WA.

WAPC (2010): "Position Statement – State Planning Policy No 2.6 – State Coastal Planning Policy Schedule 1 Sea Level Rise". Prepared by the WAPC and the Department of Planning, WA. September 2010.

Wise, R. A.,Smith, S J & Larson, M. 1995 "SBEACH: Numerical Model for Simulating Storm Induced Beach Change; Report 4 Cross Shore Transport Under Random Waves and Model Validation with SUPERTANK and Field Data". Technical Report CERC-89-9 rept 4. Coastal Engineering Research Centre, Vicksburg, MS

The State Government and the Town of Port Hedland have a shared vision that will see Port Hedland transformed into Pilbara's Port City with a population of 50,000 by 2035.

Land availability, and in particular developable land above predicted future flood levels, is a clear constraint. In recognition of this the Royalties of Regions program has funded the preparation of the Port Hedland Coast Vulnerability Study. The Study considers the combined impact of flood from rainfall and storm surge. The Study was undertaken by Cardno Pty Ltd and was overseen by a Steering Committee that included the Department of Water, the Department of Transport and the Department of Planning.

The report is not a statutory document. However, the reports findings will be considered by statutory agencies in assessing future development proposals under the Town of Port Hedland's Town Planning Scheme and the WAPC's Statement of Planning Policy 2.6.

Developers and their consultants may use the report and its data for their own works but must undertake their own independent verification specific to their development proposal.

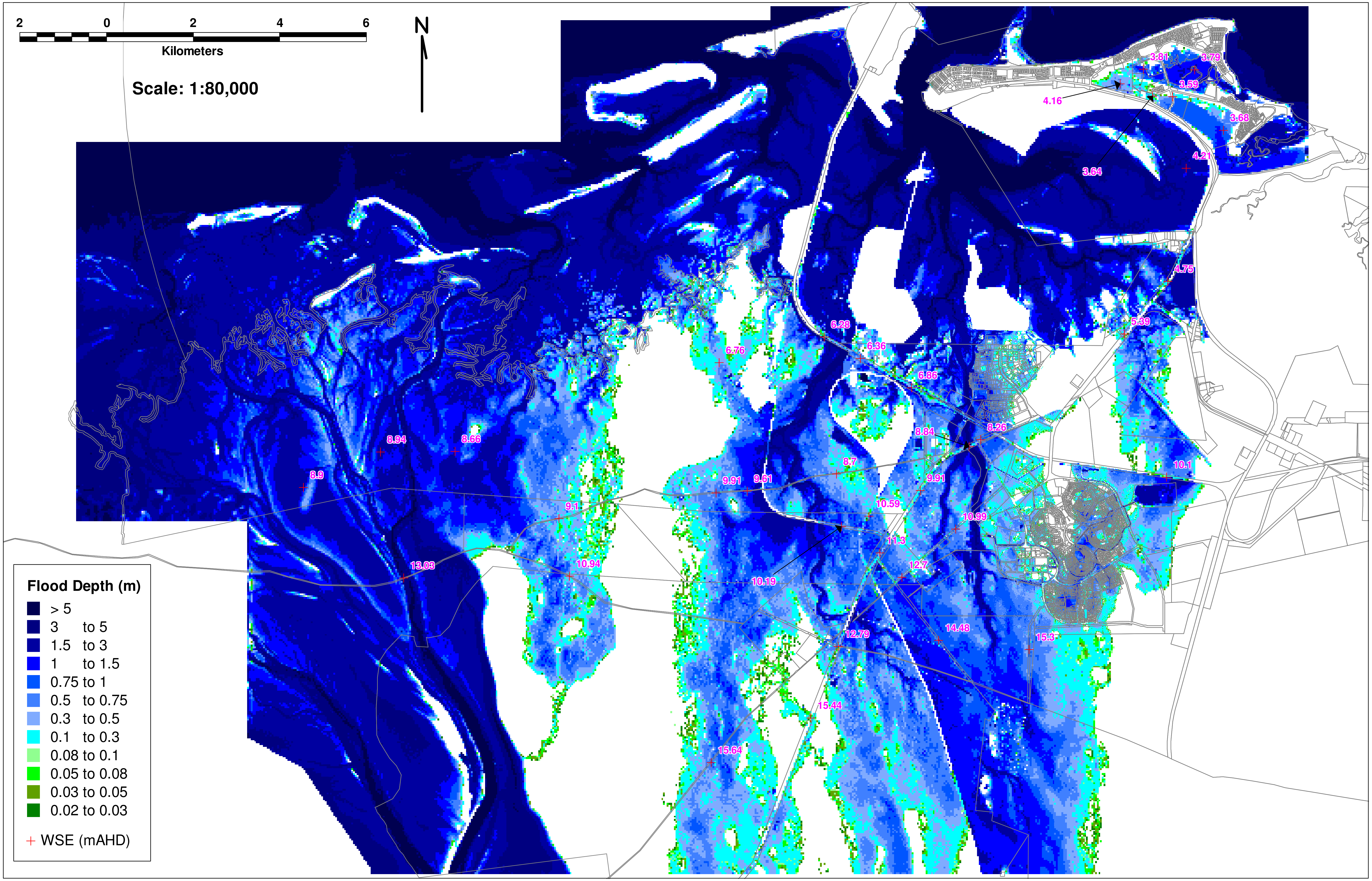
Cardno
Shaping the Future

Port Hedland Coastal Vulnerability Study
- Flood Extents - Climate Change 2110 Condition

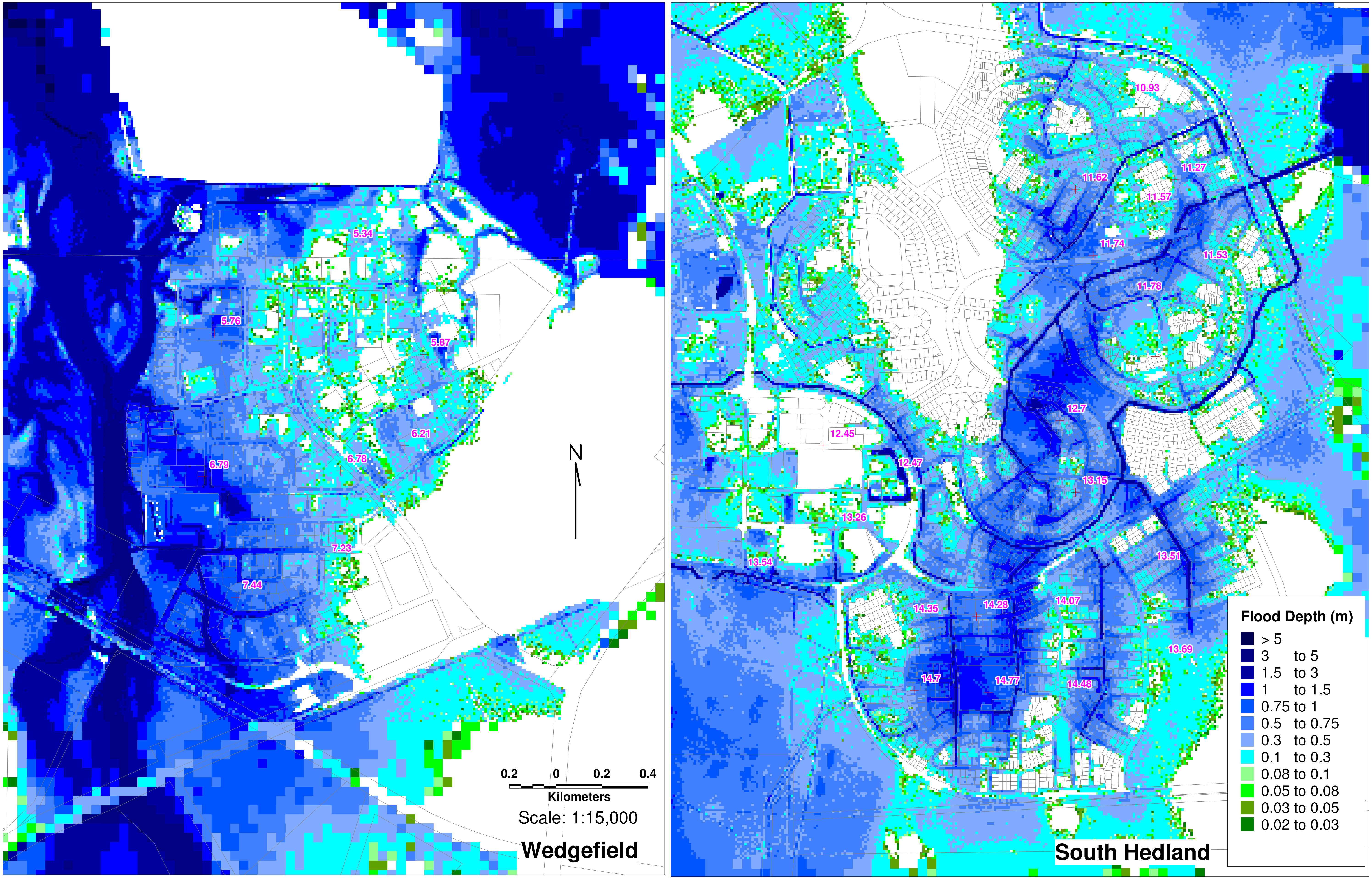
Landcorp and Department of Planning
Port Hedland Coastal Vulnerability Study
Final V1.0

ROYALTIES FOR REGIONS

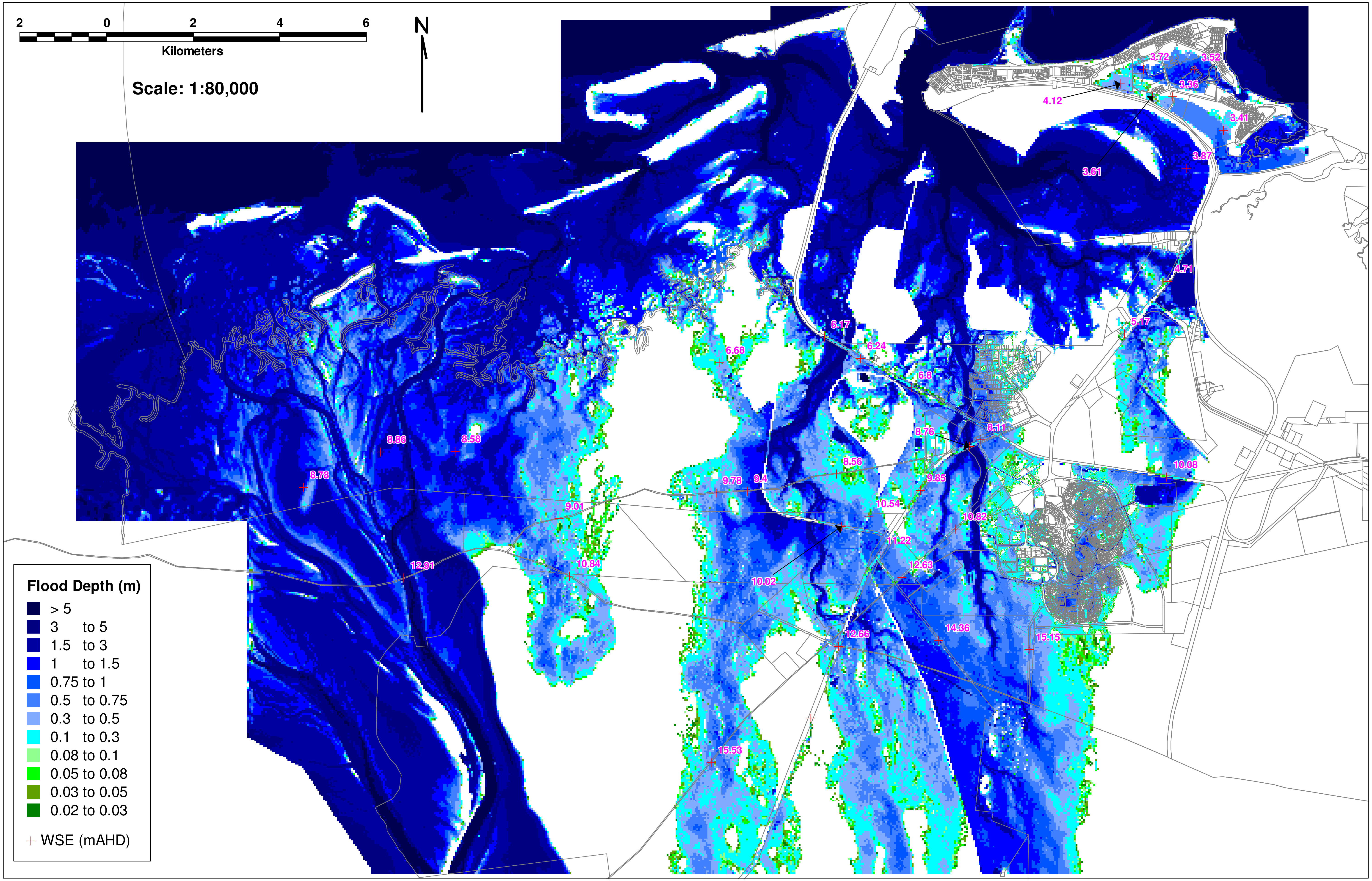
LANDCORP



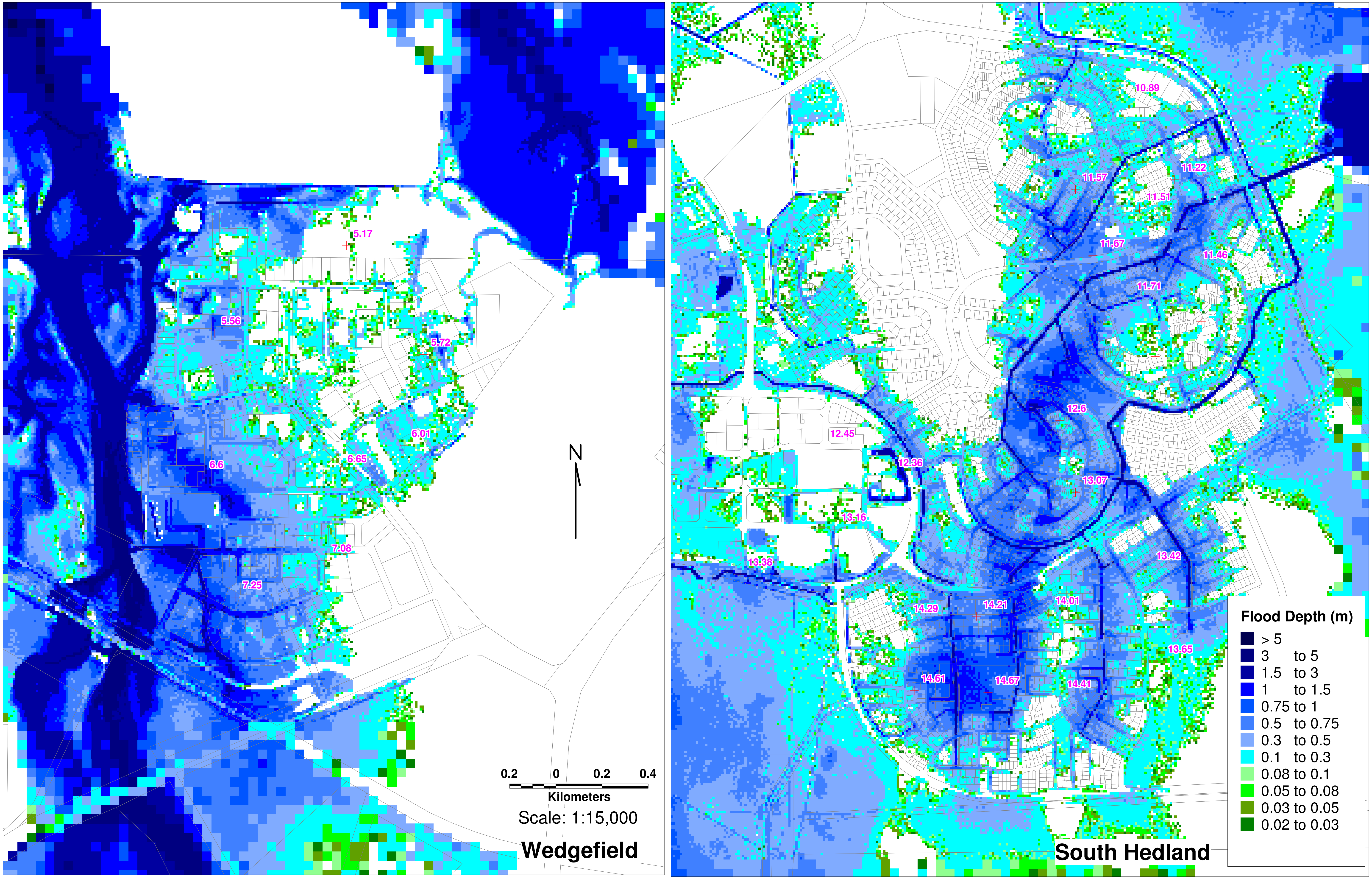
MAP P01 - 500-year Flood Depth - Existing Conditions - 500-year Catchment Flow & 50-year Ocean Water Level - Port Hedland



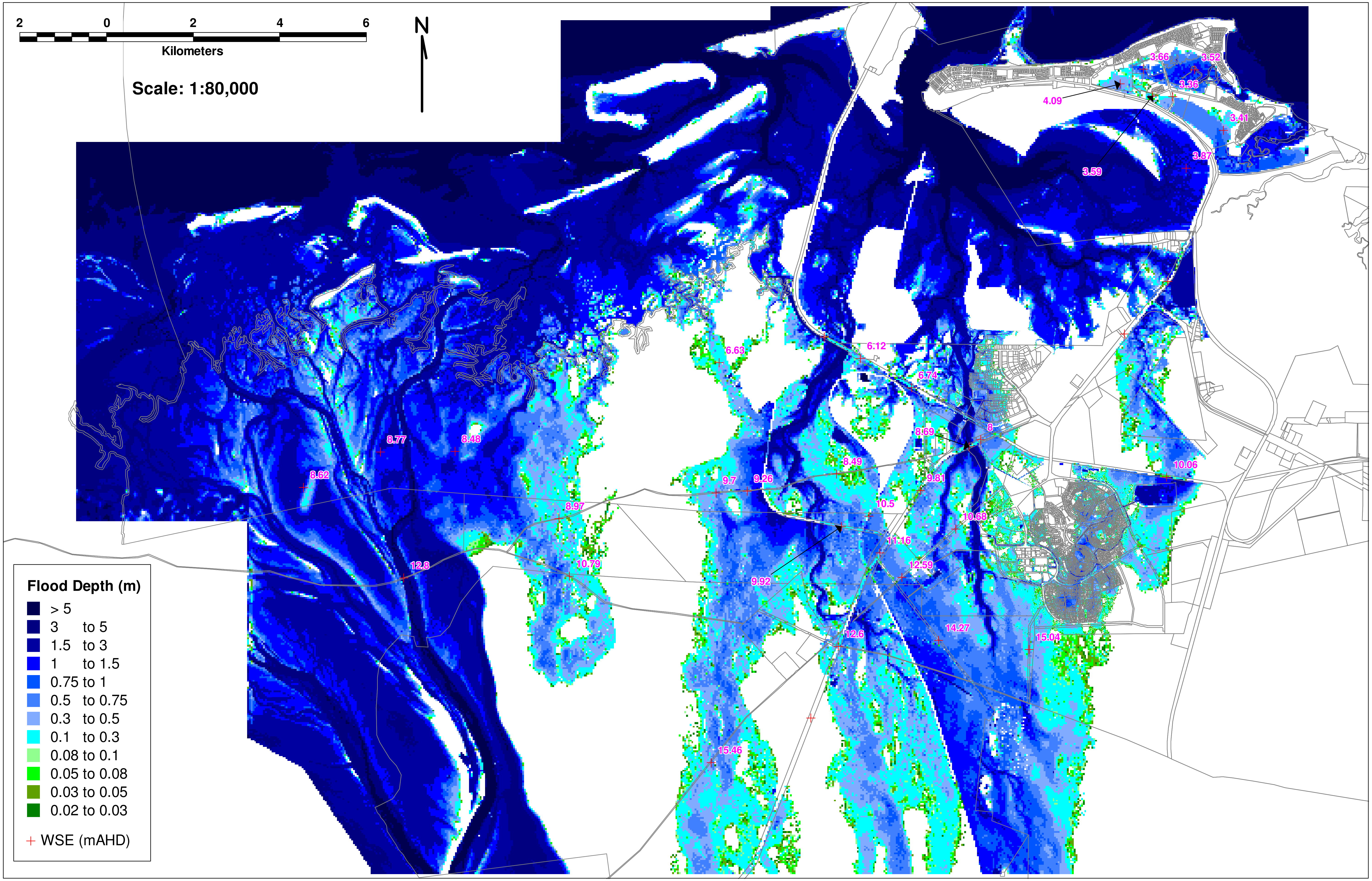
MAP P02 - 500-year Flood Depth - Existing Conditions - 500-year Catchment Flow & 50-year Ocean Water Level - Wedgefield & South Hedland



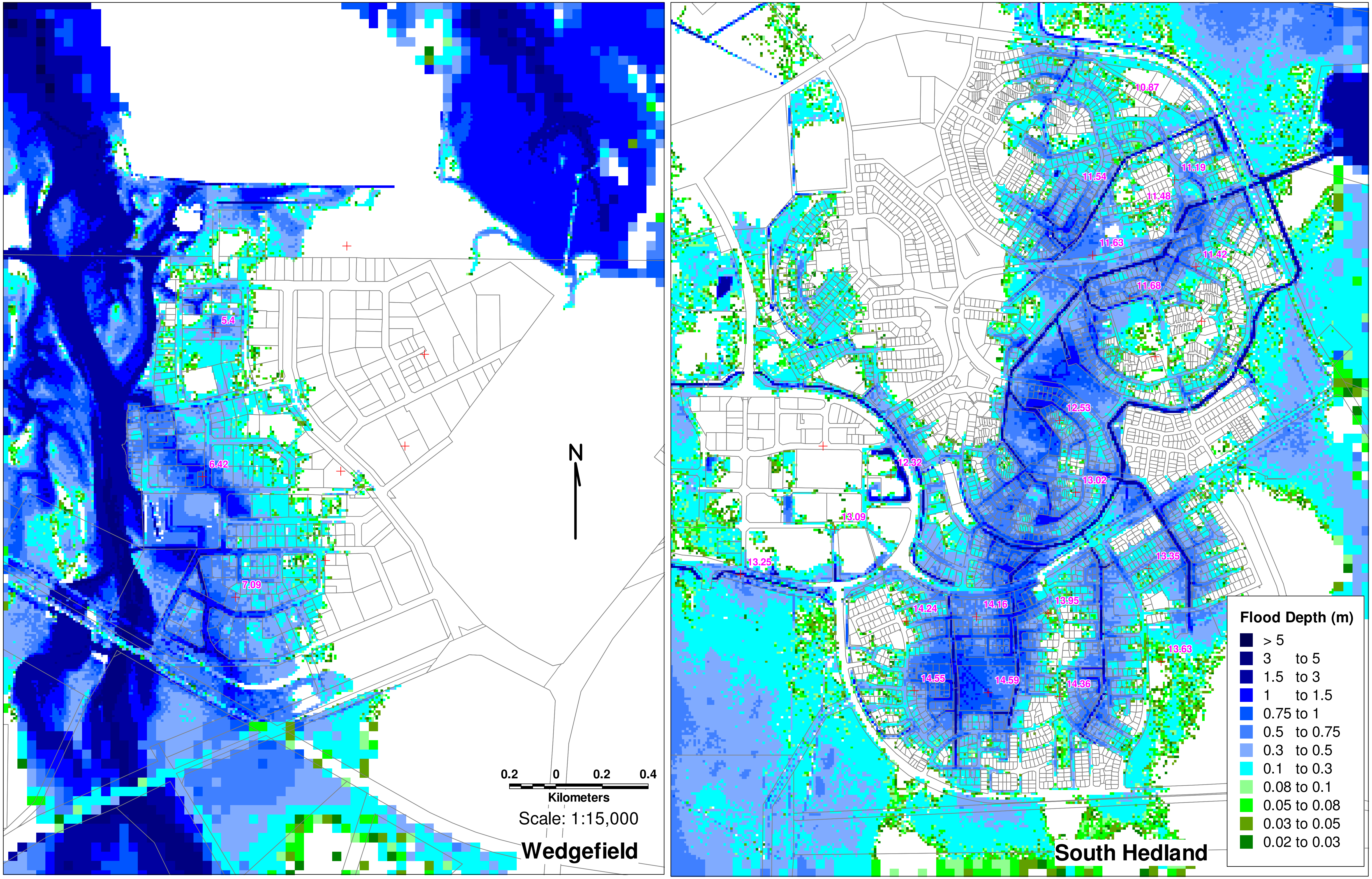
MAP P03 - 200-year Flood Depth - Existing Conditions - 200-year Catchment Flow & 20-year Ocean Water Level - Port Hedland



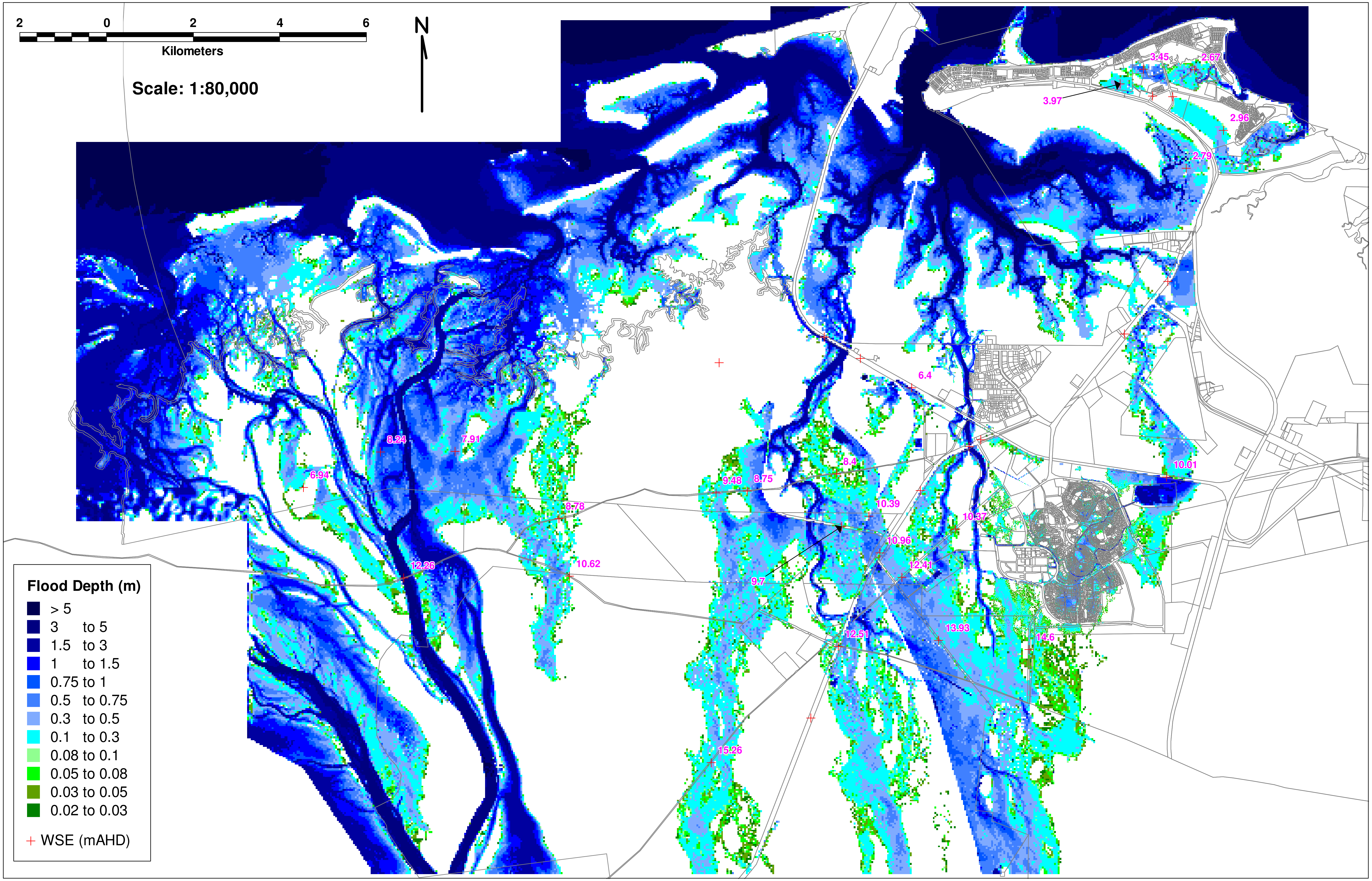
MAP P04 - 200-year Flood Depth - Existing Conditions - 200-year Catchment Flow & 20-year Ocean Water Level - Wedgefield & South Hedland



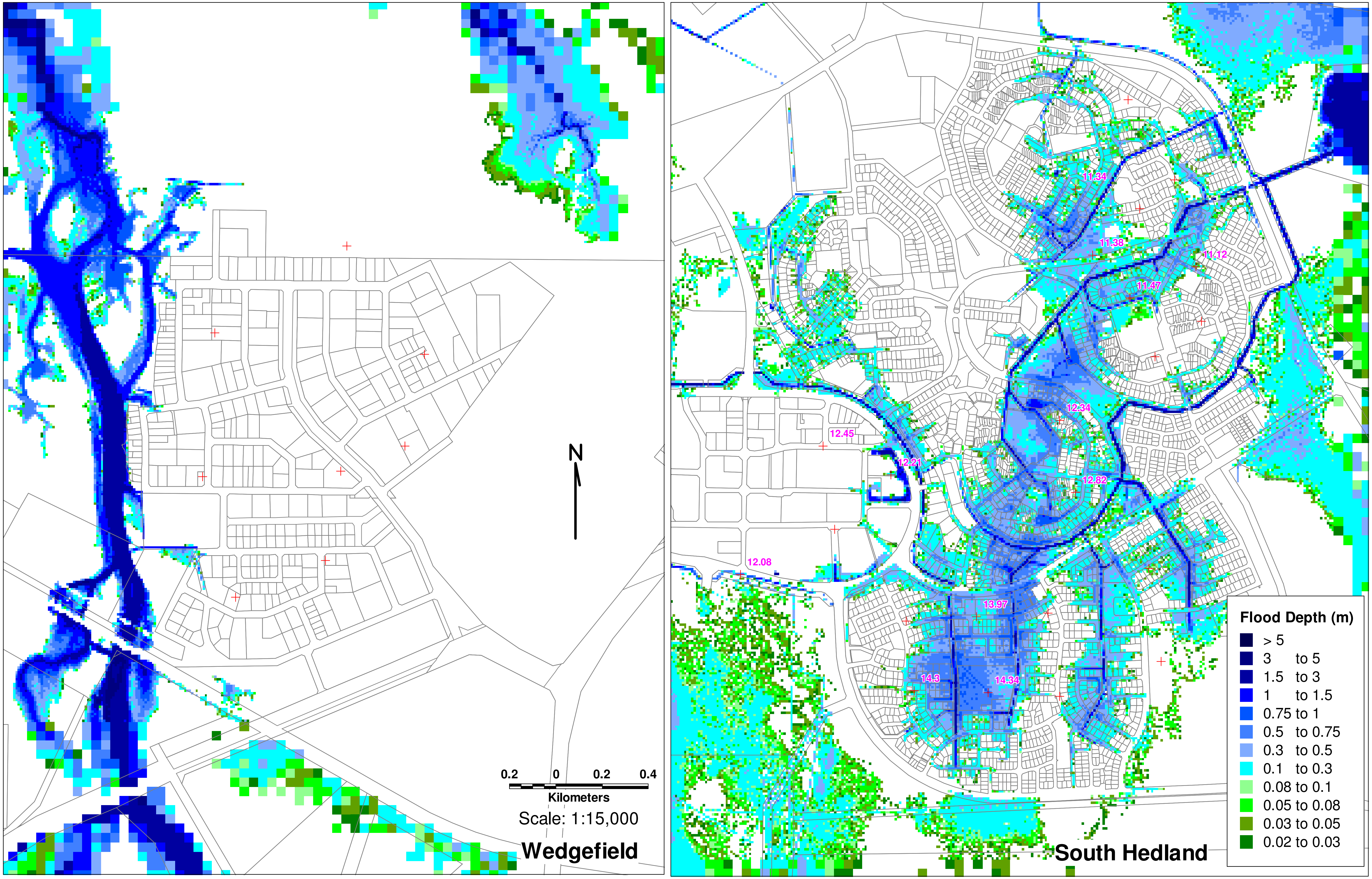
MAP P05 - 100-year Flood Depth - Existing Conditions - 100-year Catchment Flow & 20-year Ocean Water Level - Port Hedland



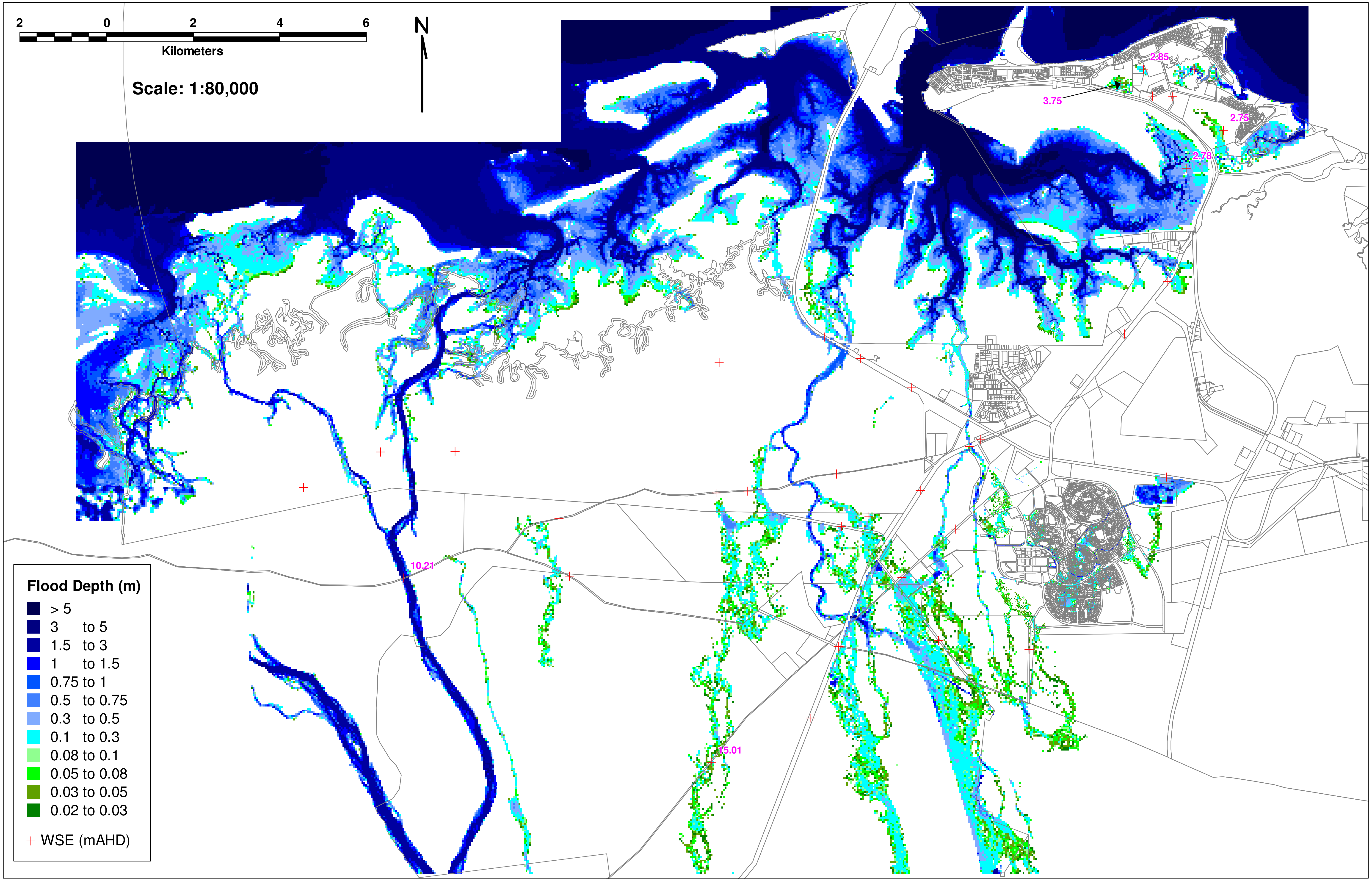
MAP P06 - 100-year Flood Depth - Existing Conditions - 100-year Catchment Flow & 20-year Ocean Water Level - Wedgefield & South Hedland



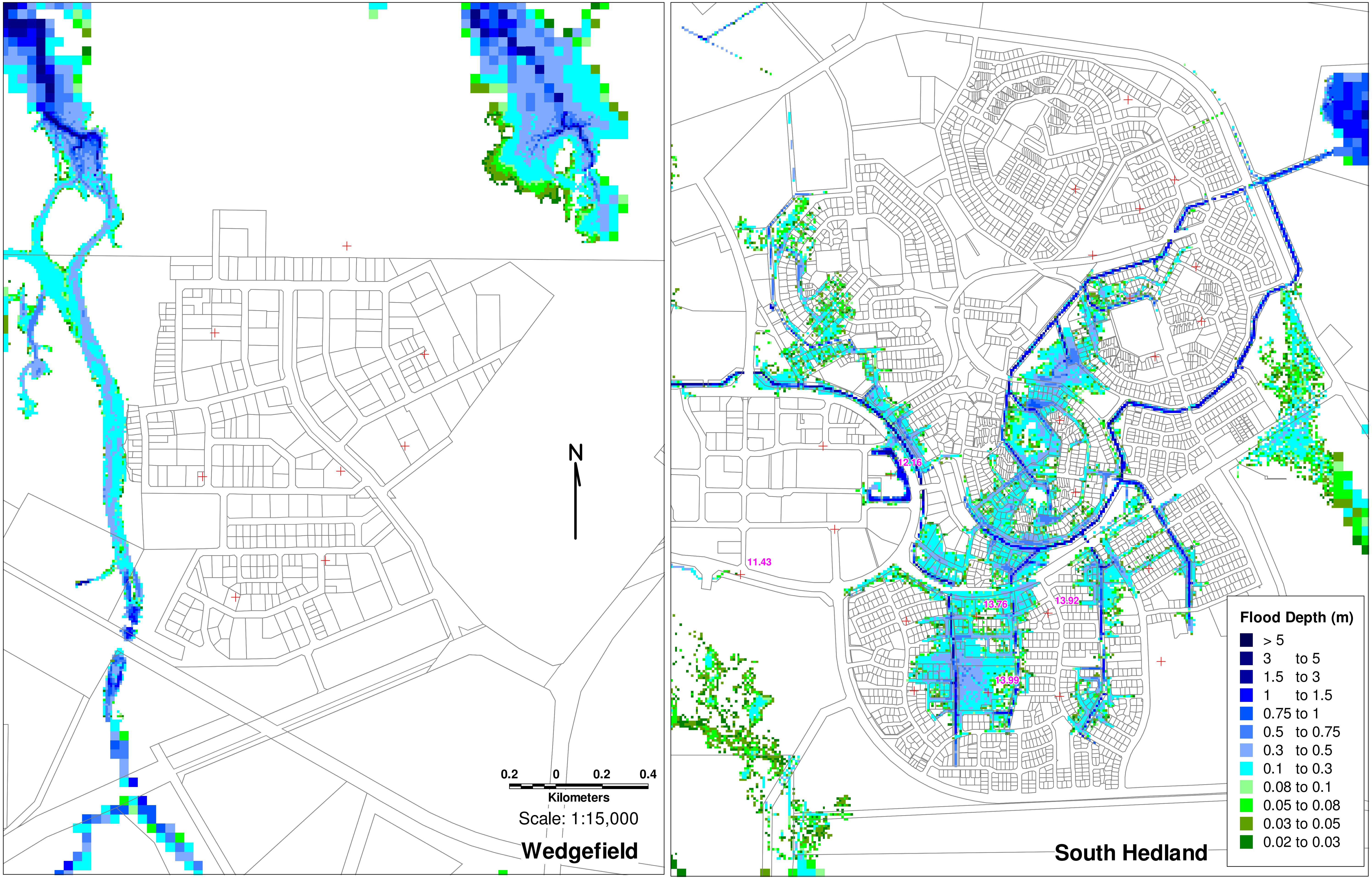
MAP P07 - 10-year Flood Depth - Existing Conditions - 10-year Catchment Flow & Coastal Mean High Water Springs - Port Hedland



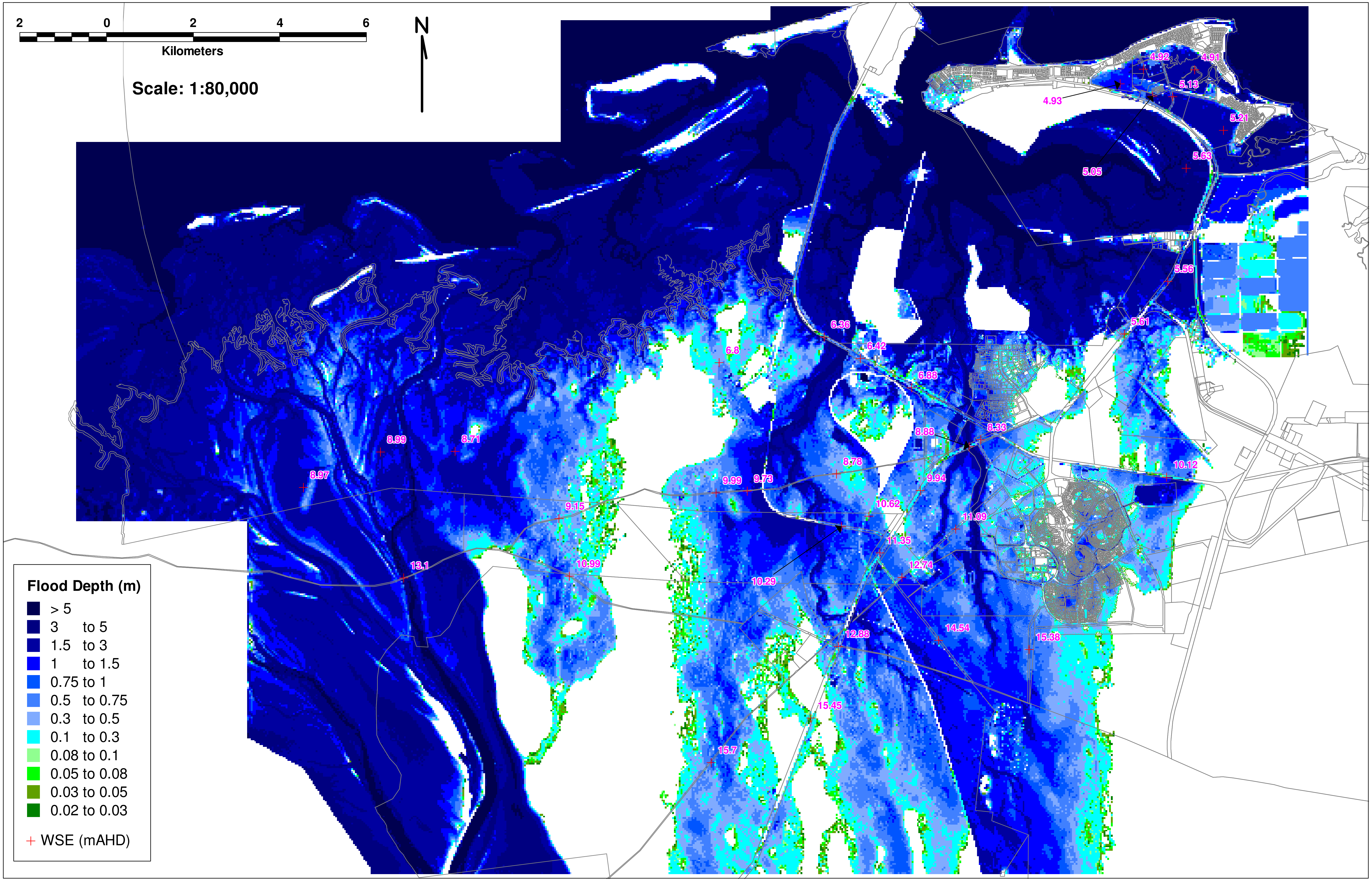
MAP P08 - 10-year Flood Depth - Existing Conditions - 10-year Catchment Flow & Coastal Mean High Water Springs - Wedgefield & South Hedland



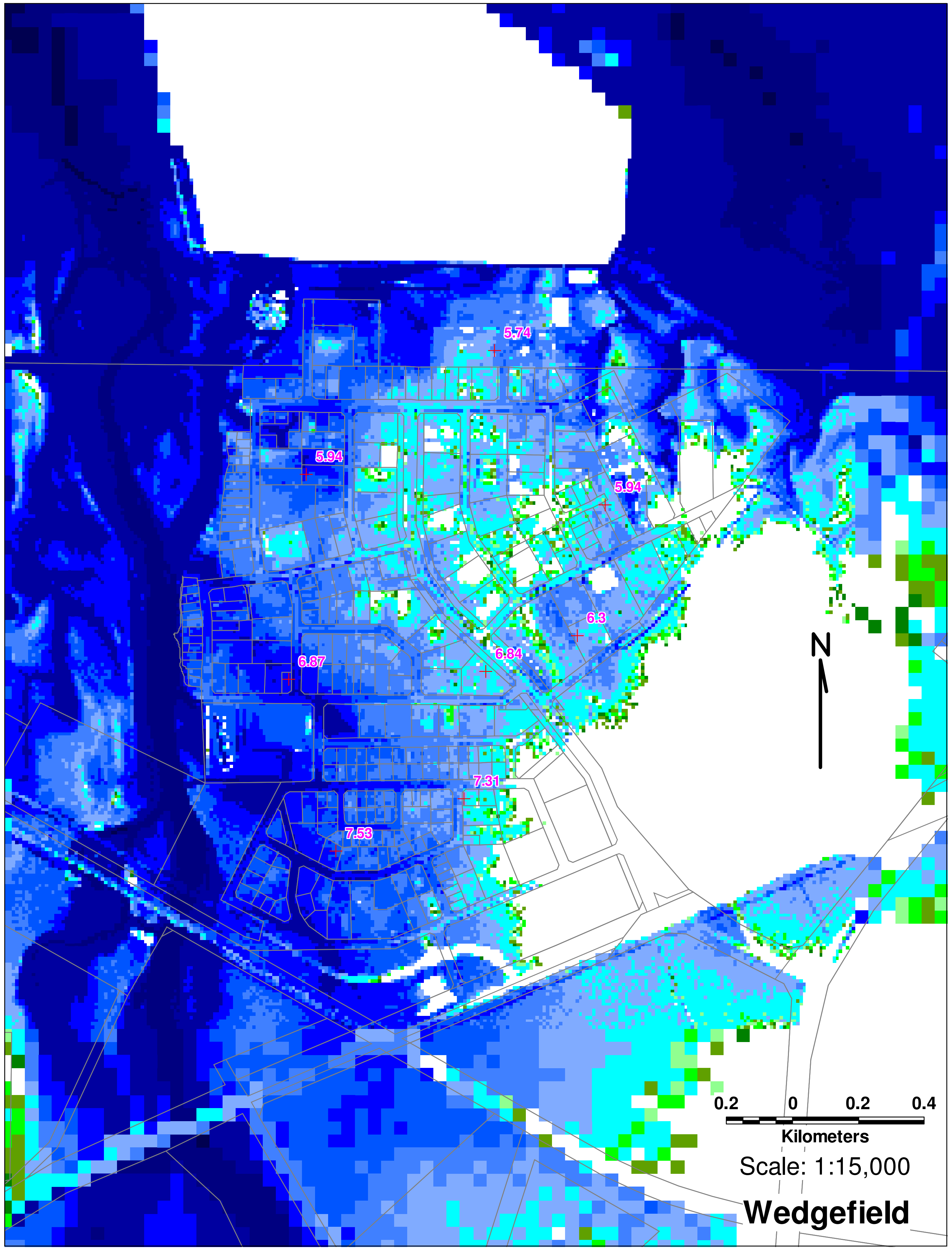
MAP P09 - 2-year Flood Depth - Existing Conditions - 2-year Catchment Flow & Coastal Mean High Water Springs - Port Hedland

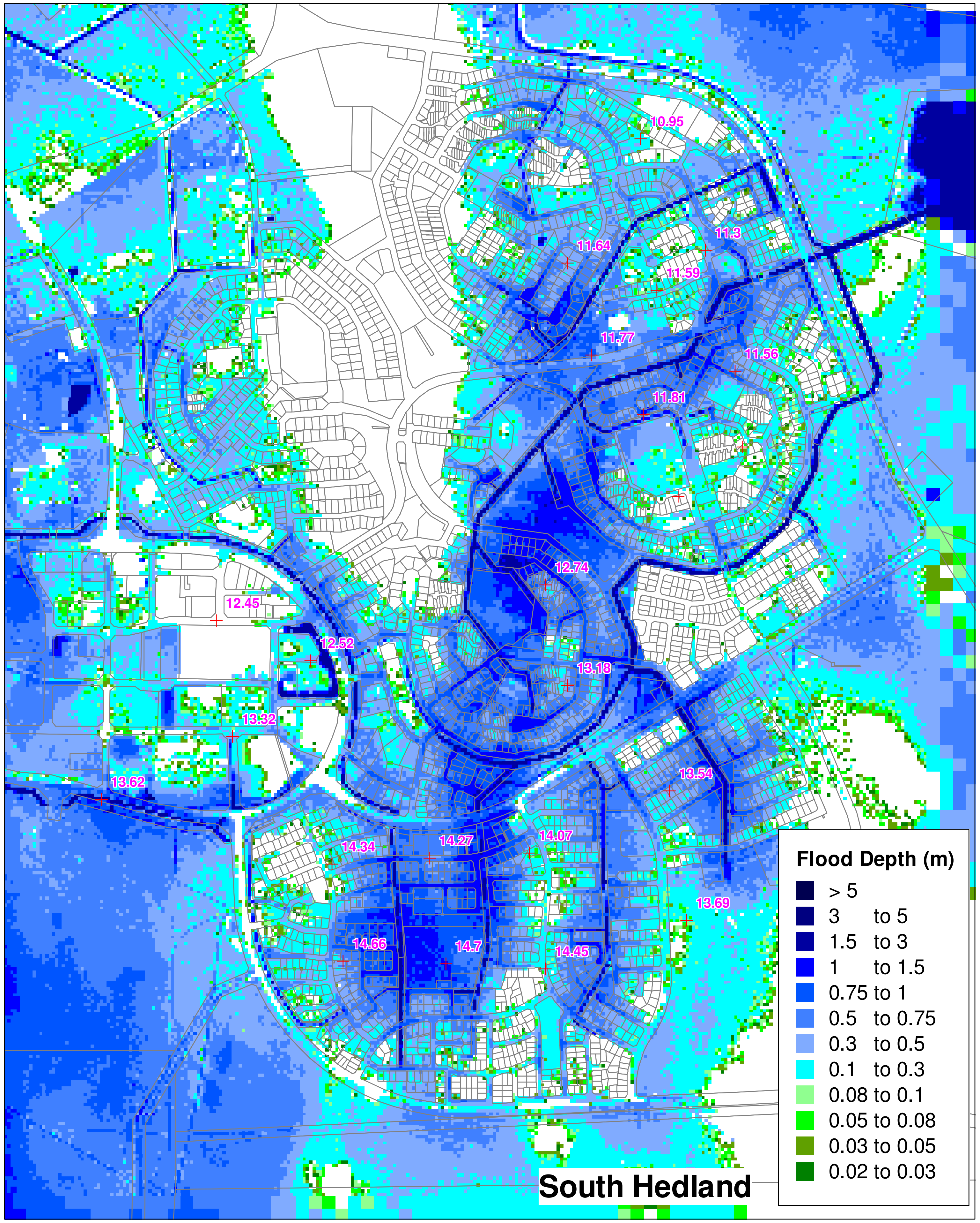


MAP P10 - 2-year Flood Depth - Existing Conditions - 2-year Catchment Flow & Coastal Mean High Water Springs - Wedgefield & South Hedland

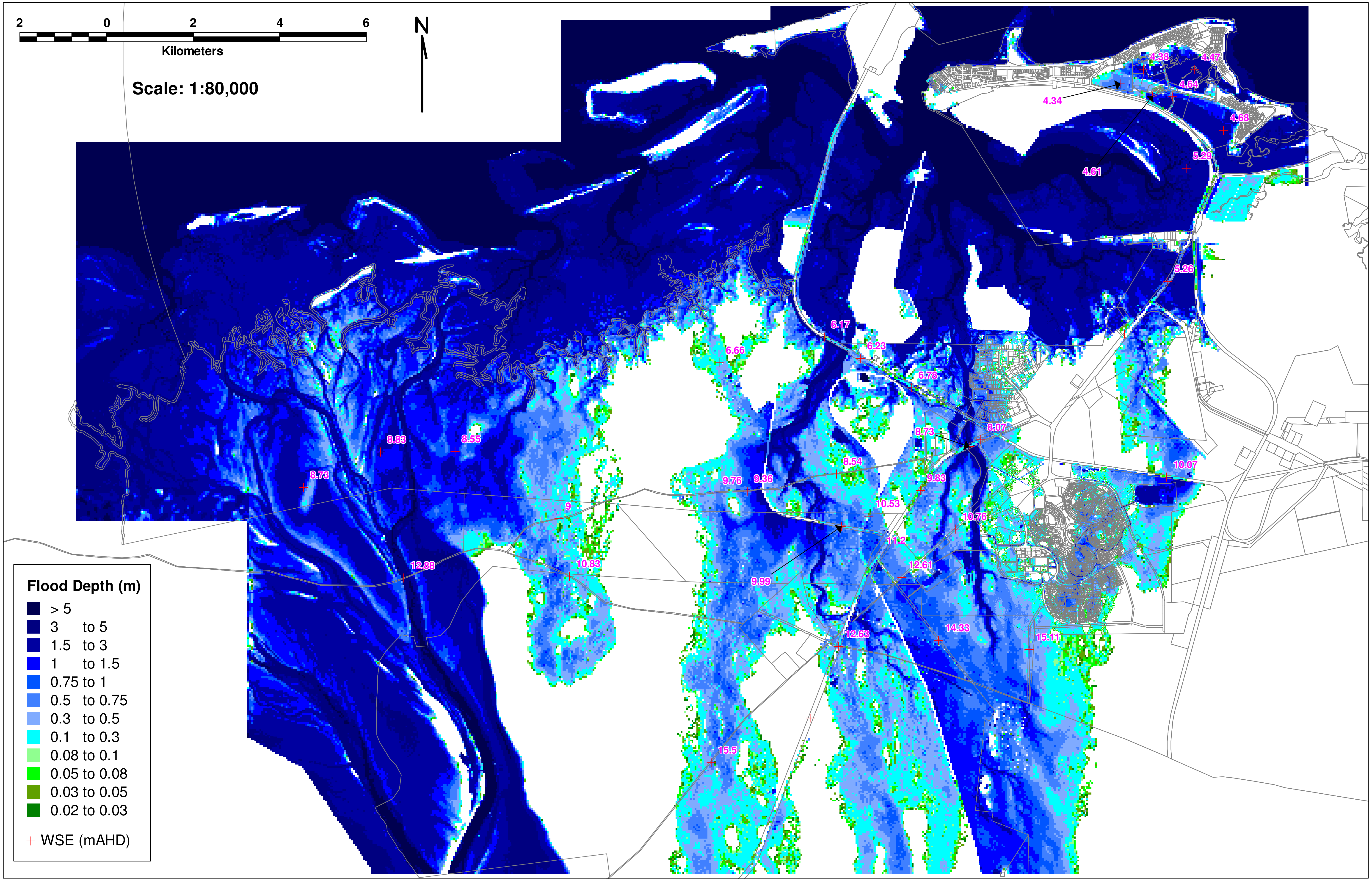


MAP P11 - 500-year Flood Depth - Climate Change 2060 Conditions - Catchment Flow ARI 500-year & Coastal Water Level ARI 50-year - Port Hedland

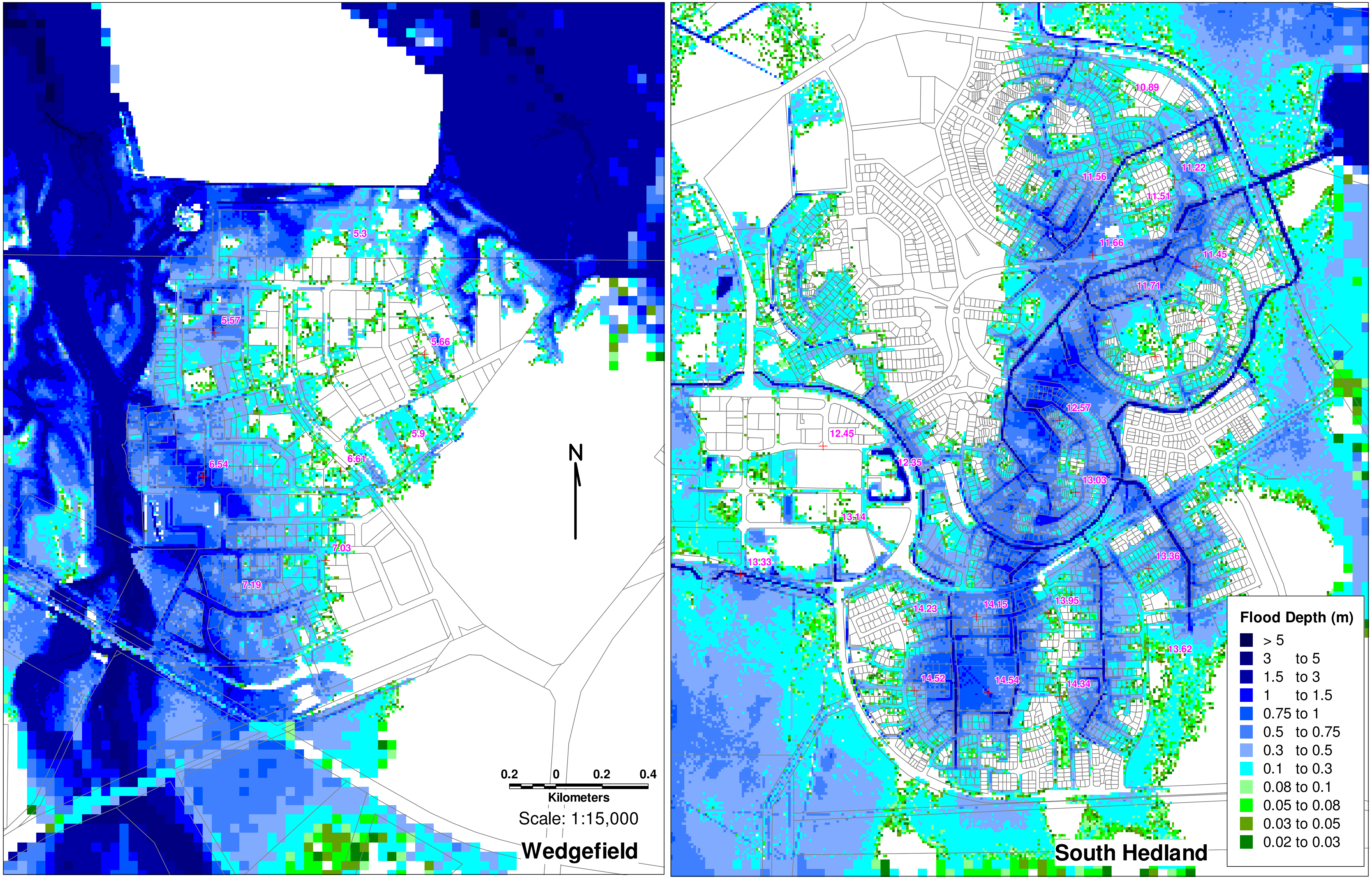




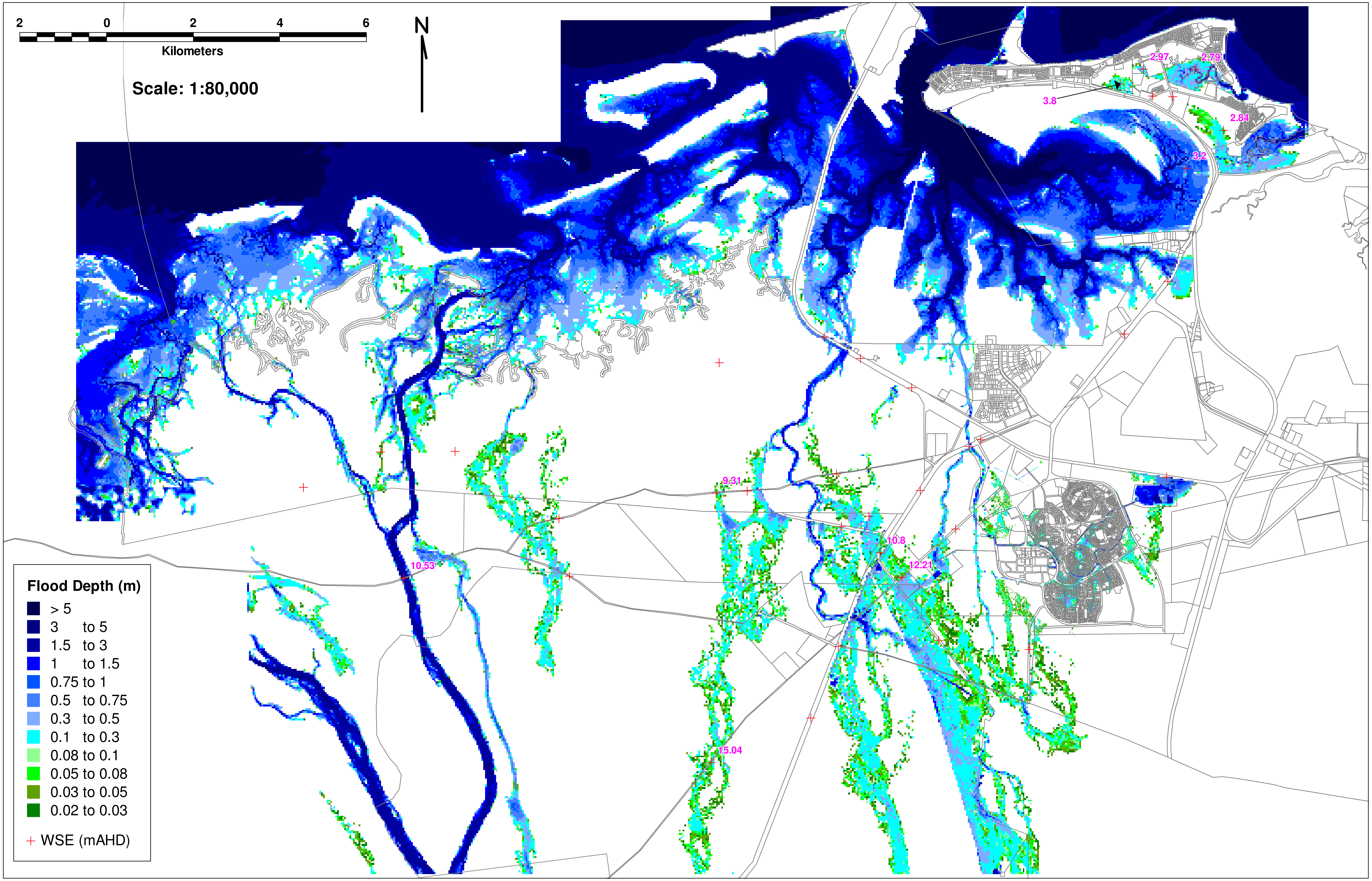
MAP P12 - 500-year Flood Depth - Climate Change 2060 Conditions - Catchment Flow ARI 500-year & Coastal Water Level ARI 50-year - Wedgefield & South Hedland



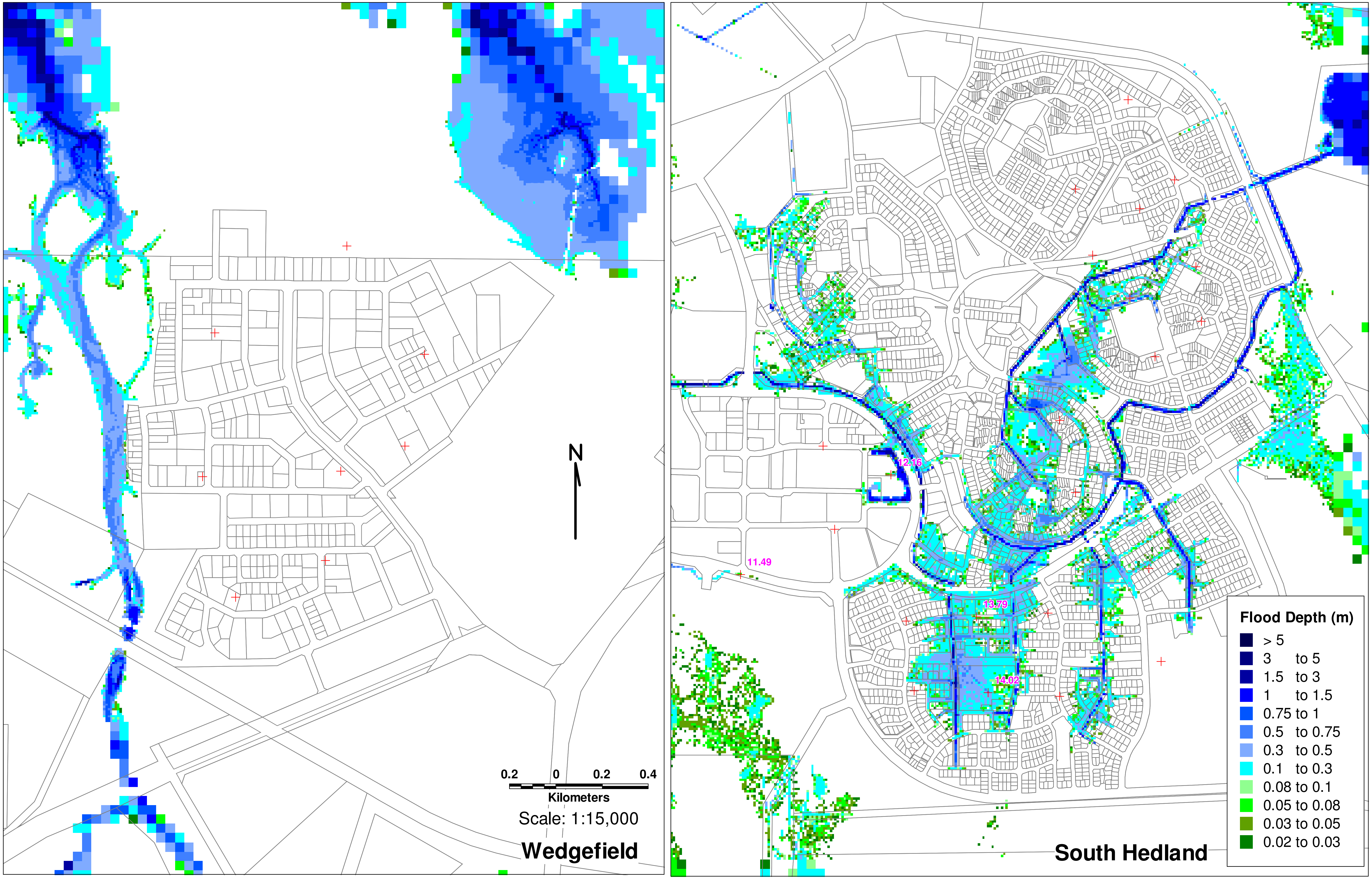
MAP P13 - 100-year Flood Depth - Climate Change 2060 Conditions - 100-year Catchment Flow & 20-year Ocean Water Level - Port Hedland



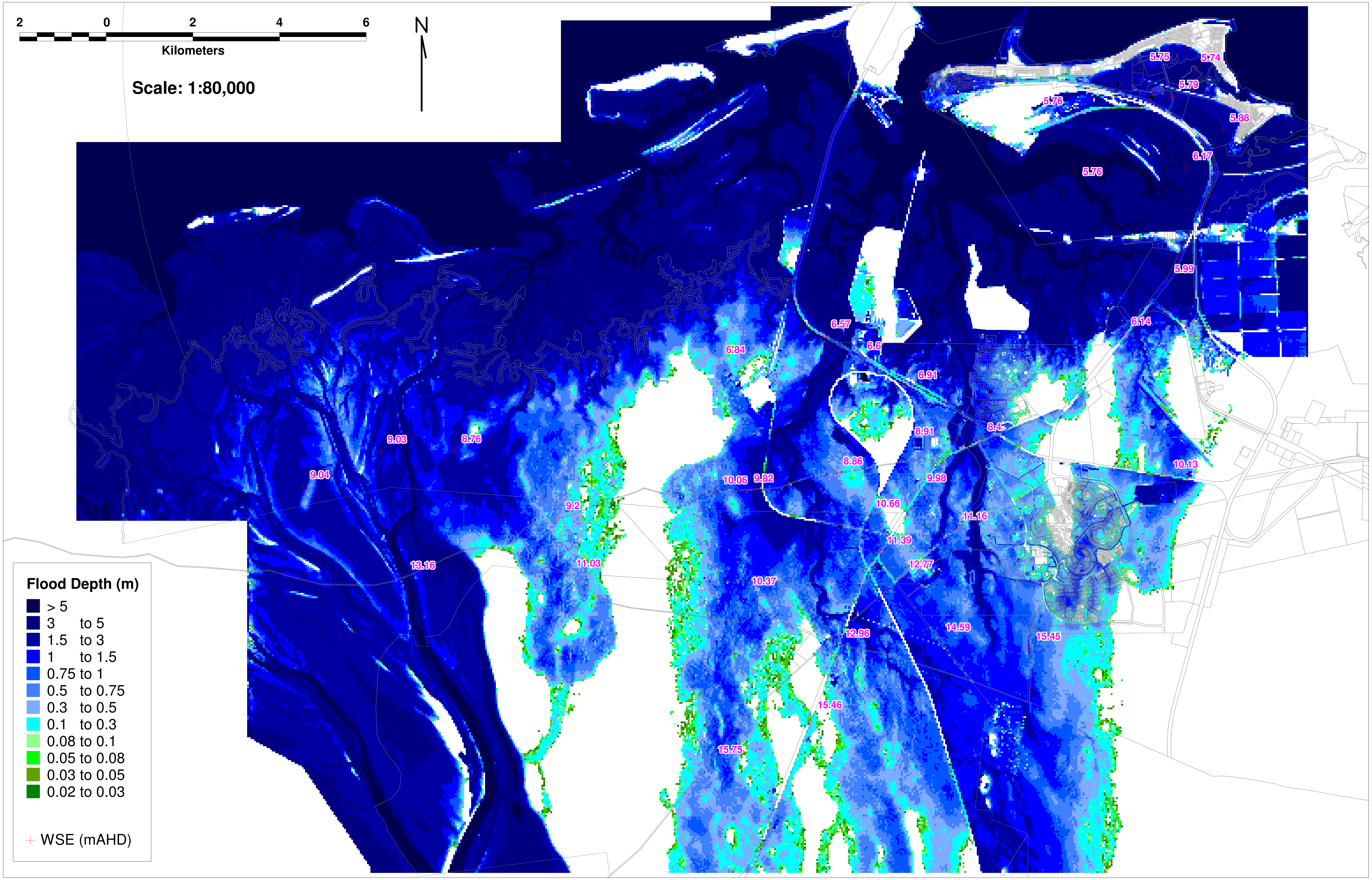
MAP P14 - 100-year Flood Depth - Climate Change 2060 Conditions - 100-year Catchment Flow & 20-year Ocean Water Level - Wedgefield & South Hedland



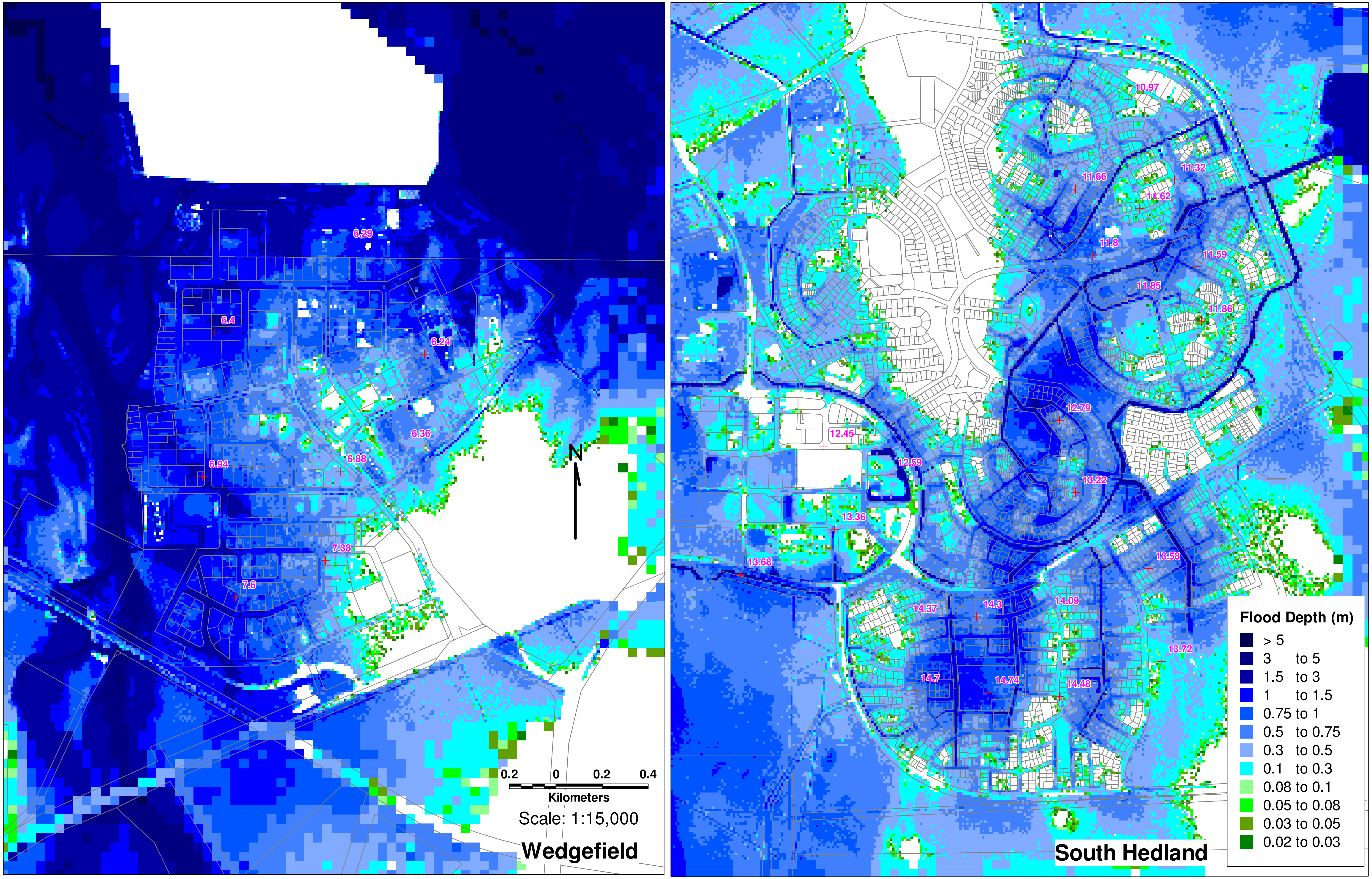
MAP P15 - 2-year Flood Depth - Climate Change 2060 Conditions - 2-year Catchment Flow & Coastal Mean High Water Springs - Port Hedland



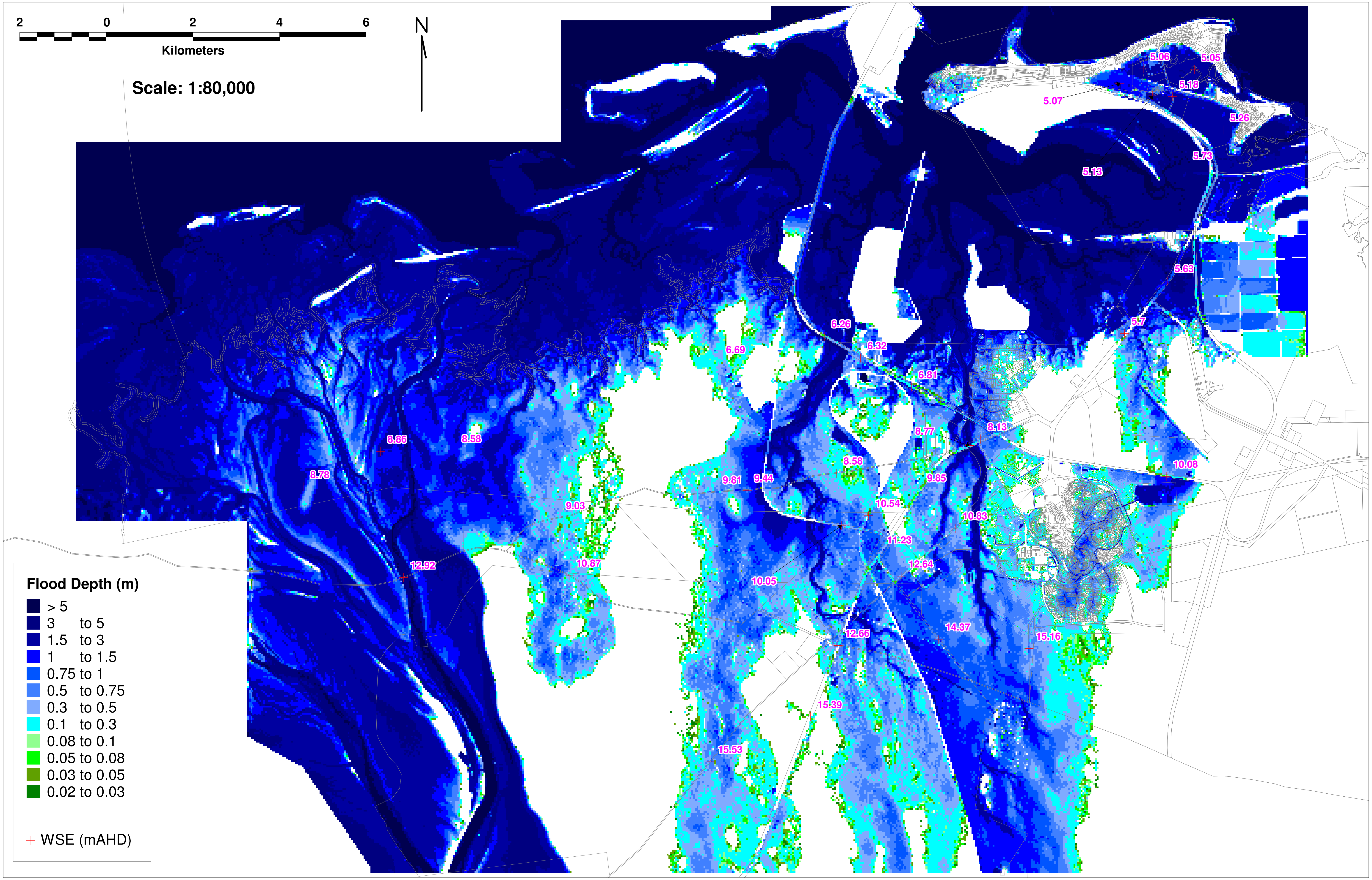
MAP P16 - 2-year Flood Depth - Climate Change 2060 Conditions - 2-year Catchment Flow & Coastal Mean High Water Springs - Wedgefield & South Hedland



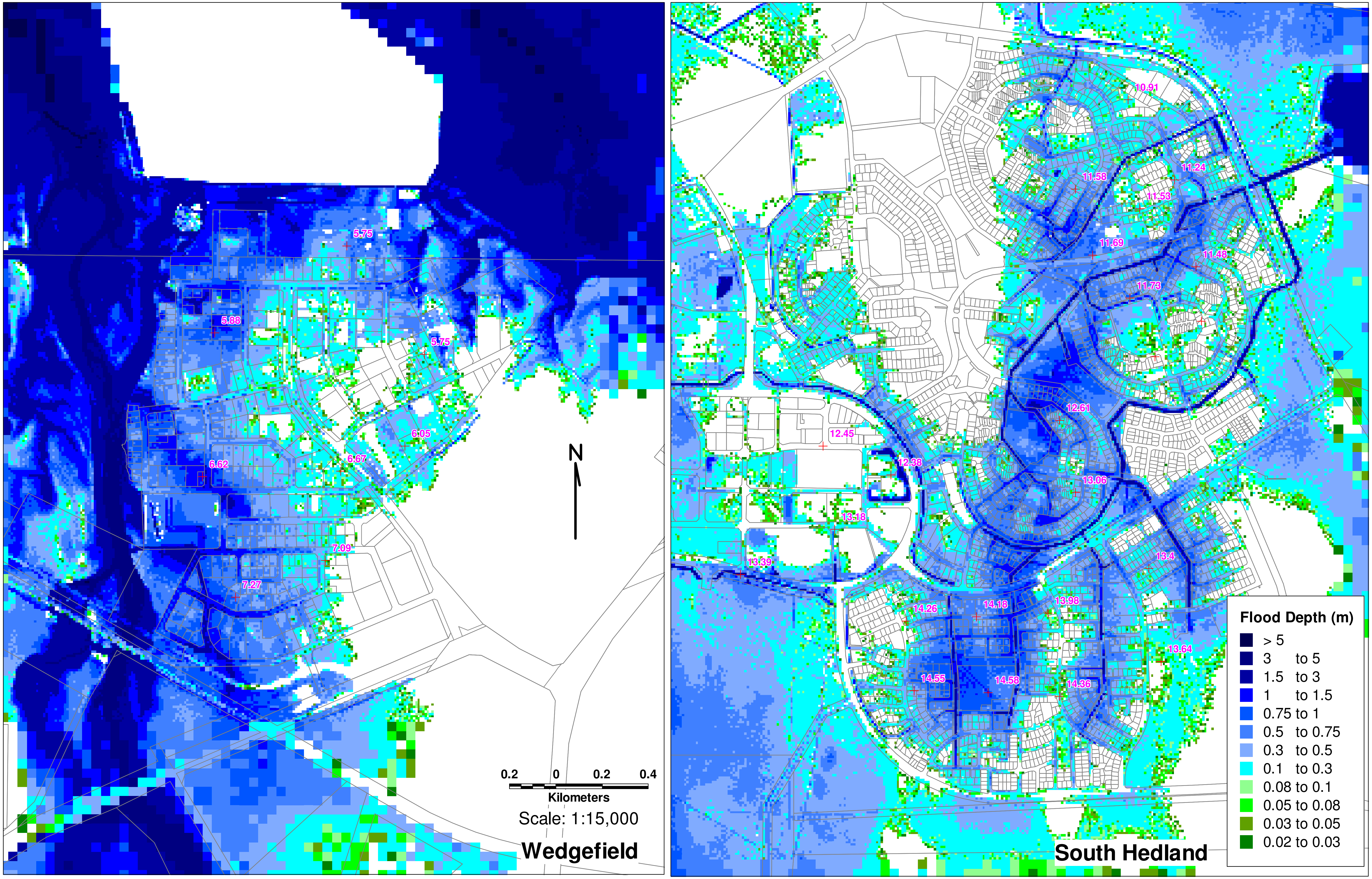
MAP P17 - 500-year Flood Depth - Climate Change 2110 Conditions - Catchment Flow ARI 500-year & Coastal Water Level ARI 50-year - Port Hedland



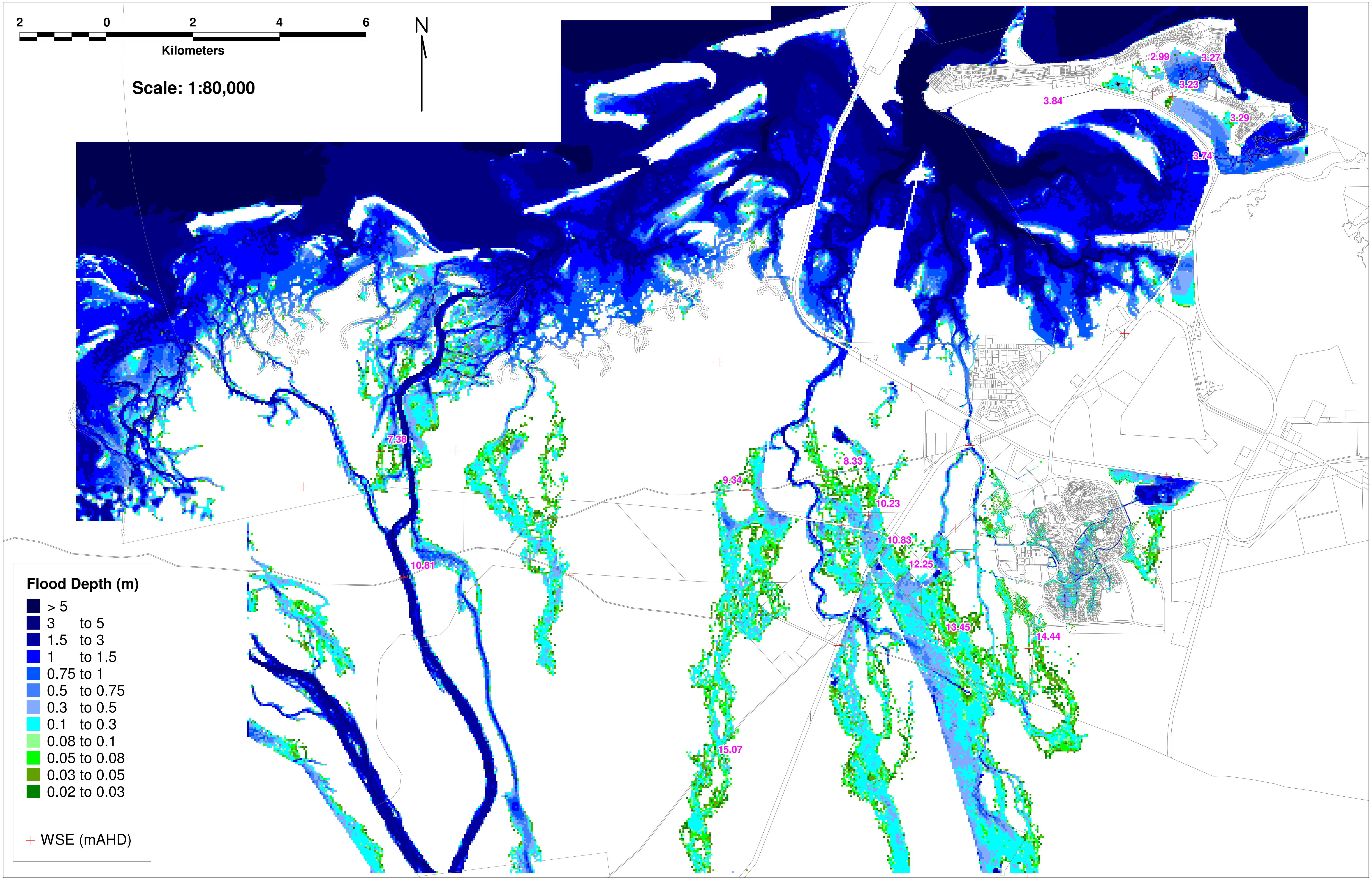
MAP P18 - 500-year Flood Depth - Climate Change 2110 Conditions - Catchment Flow ARI 500-year & Coastal Water Level ARI 50-year - Wedgefield & South Hedland



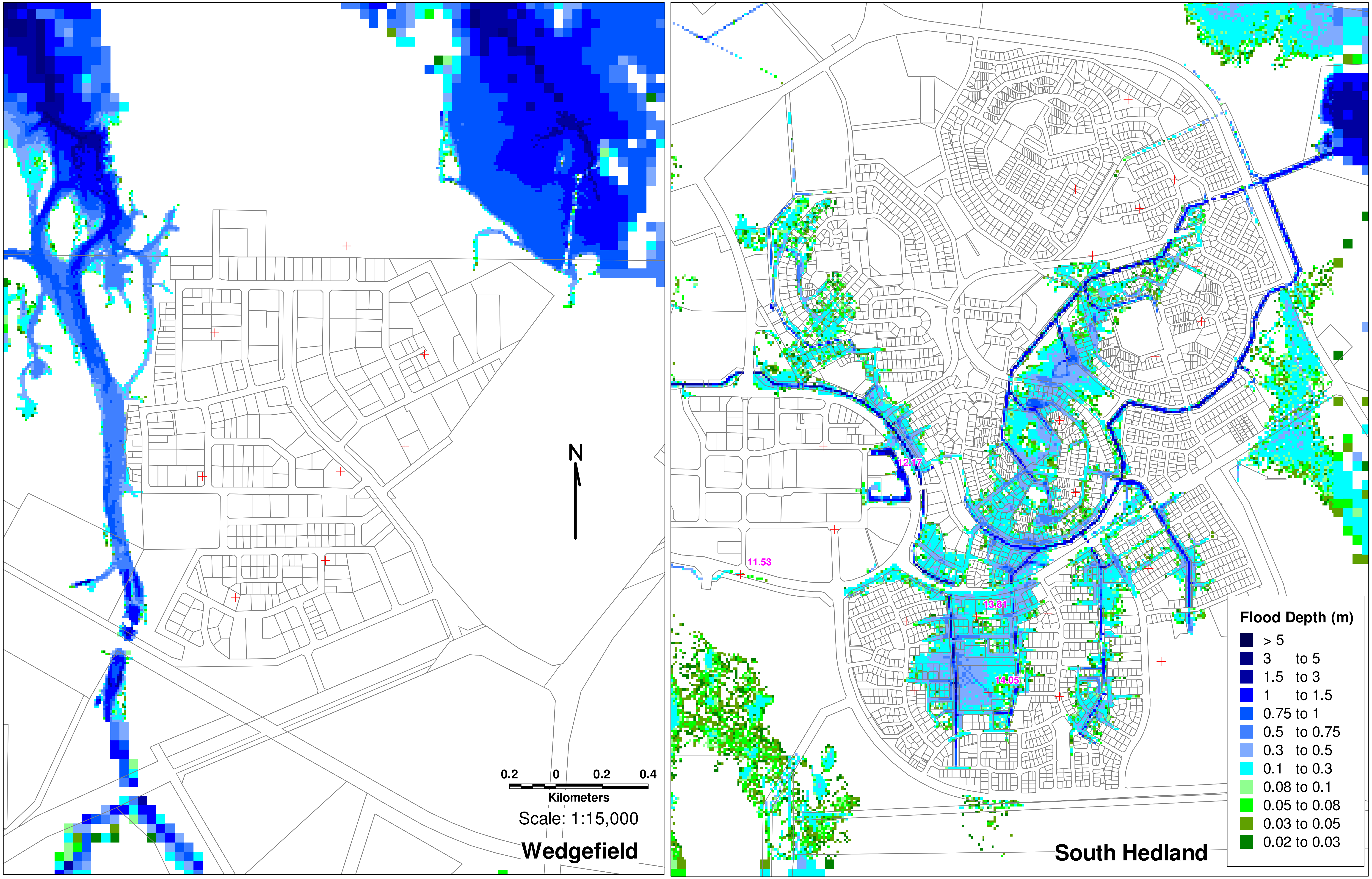
MAP P19 - 100-year Flood Depth - Climate Change 2110 Conditions - 100-year Catchment Flow & 20-year Ocean Water Level - Port Hedland



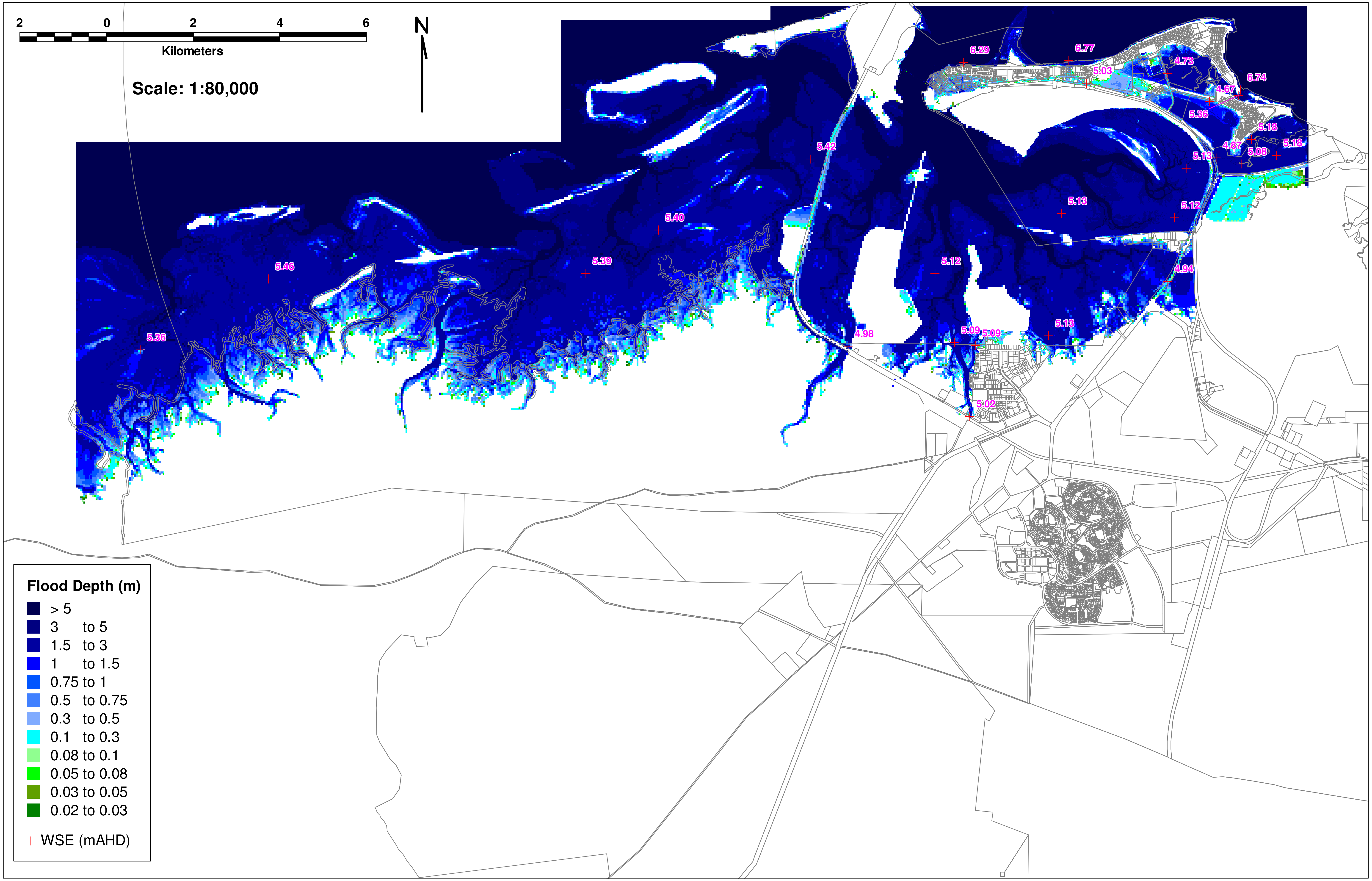
MAP P20 - 100-year Flood Depth - Climate Change 2110 Conditions - 100-year Catchment Flow & 20-year Ocean Water Level - Wedgefield & South Hedland



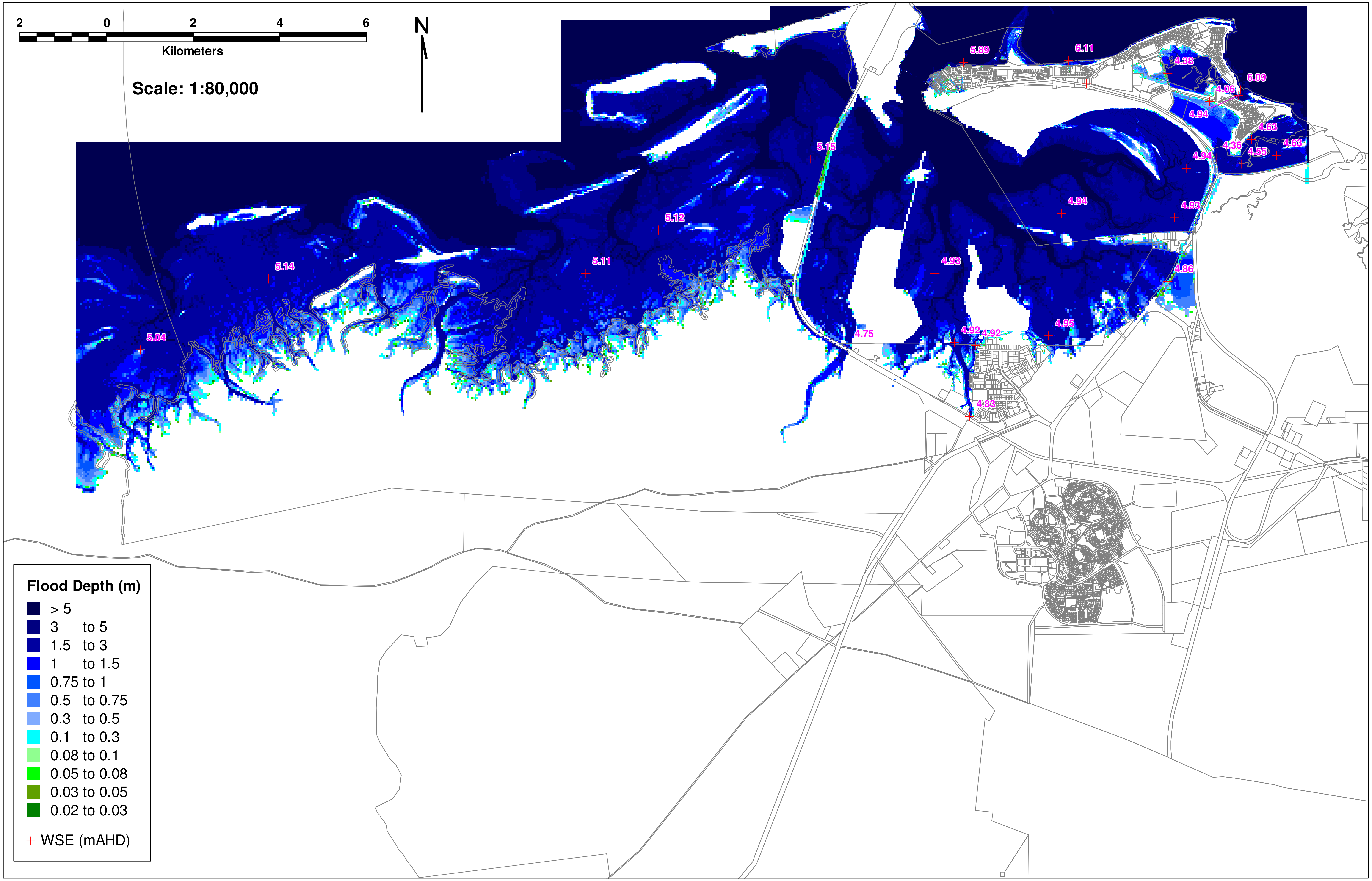
MAP P21 - 2-year Flood Depth - Climate Change 2110 Conditions - 2-year Catchment Flow & Coastal Mean High Water Springs - Port Hedland



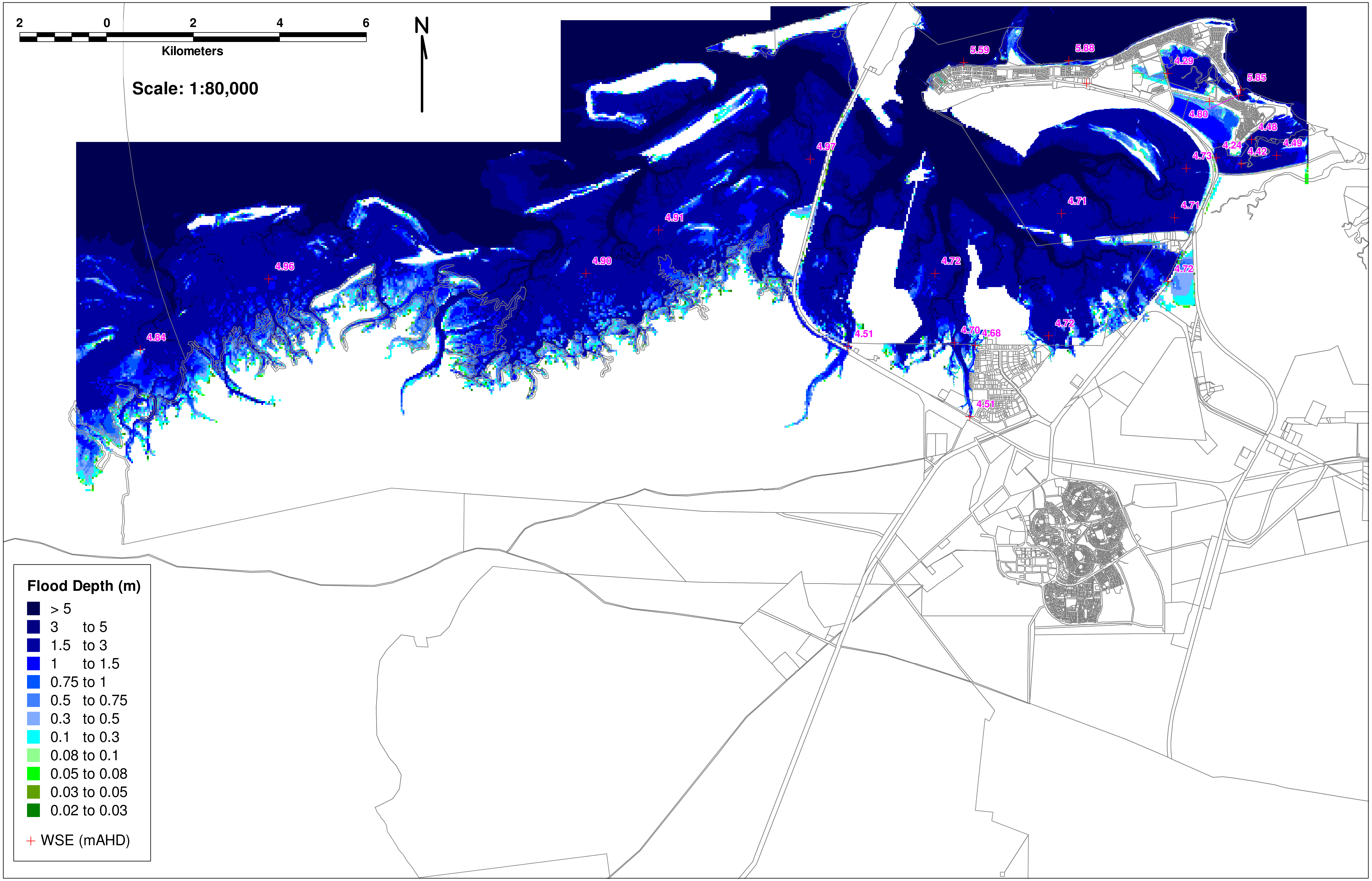
MAP P22 - 2-year Flood Depth - Climate Change 2110 Conditions - 2-year Catchment Flow & Coastal Mean High Water Springs - Wedgefield & South Hedland



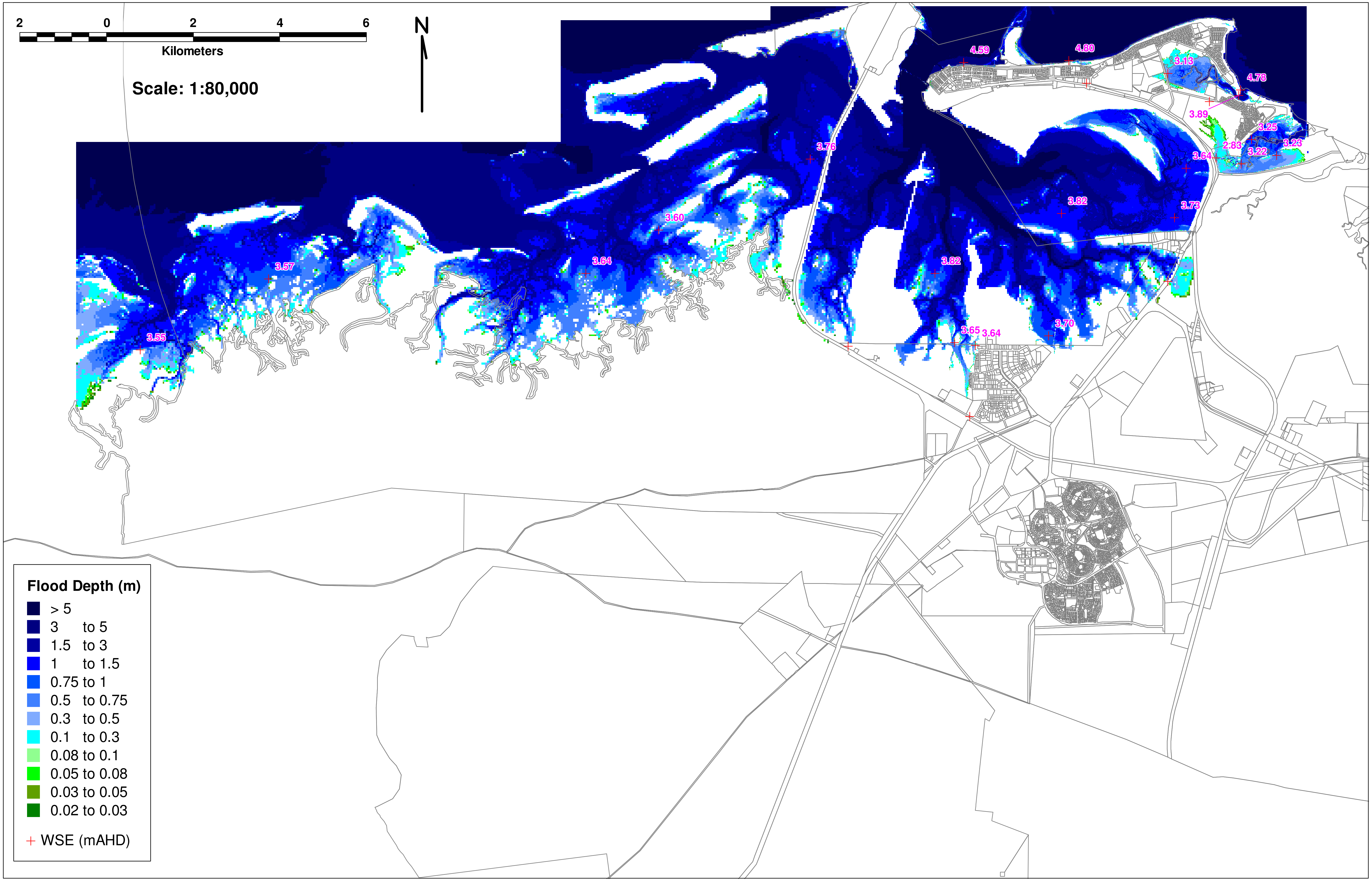
MAP P23 - 500-Year Combined Ocean Inundation Flood Depth - Existing Conditions



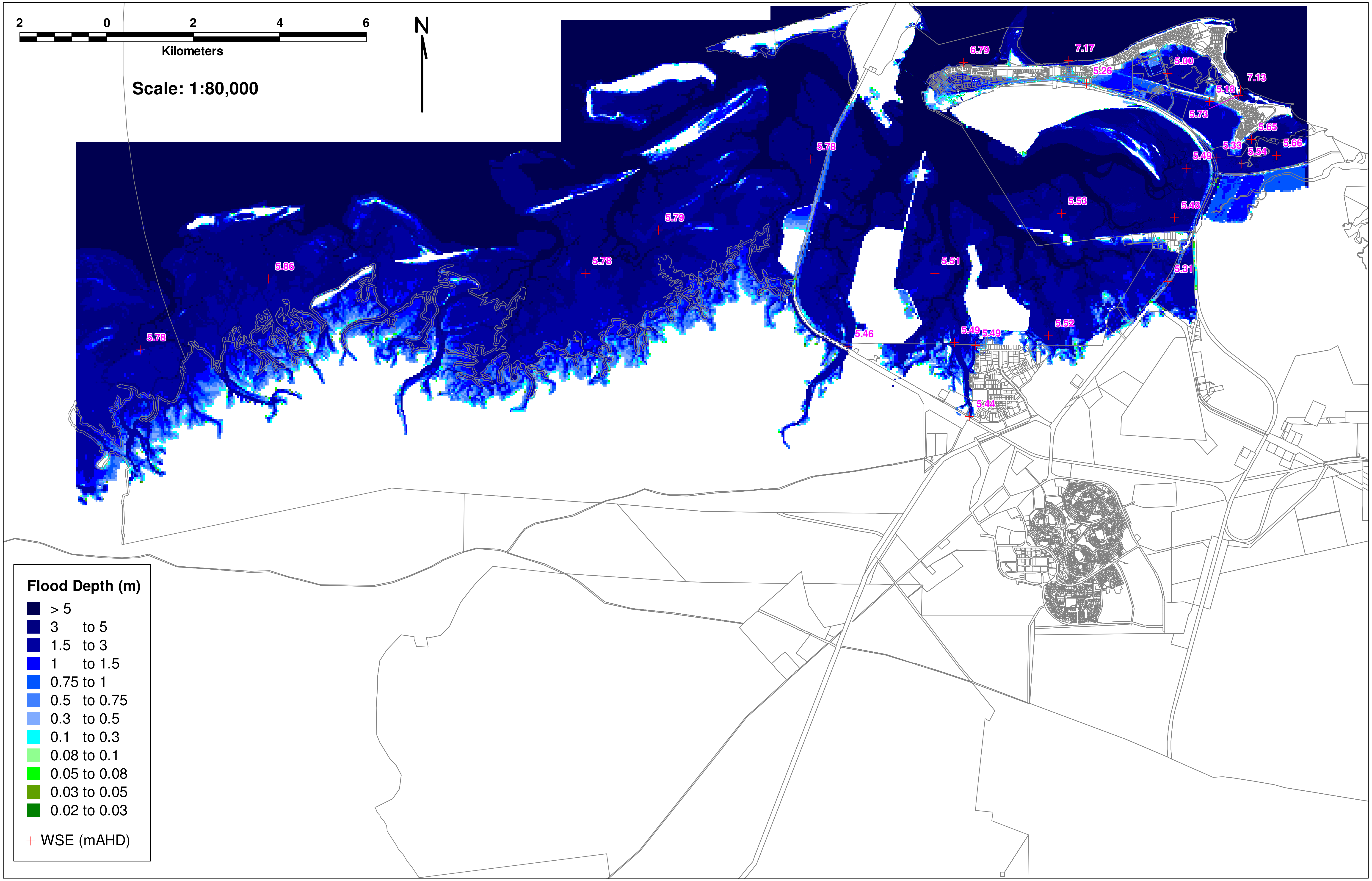
MAP P24 - 200-Year Combined Ocean Inundation Flood Depth - Existing Conditions



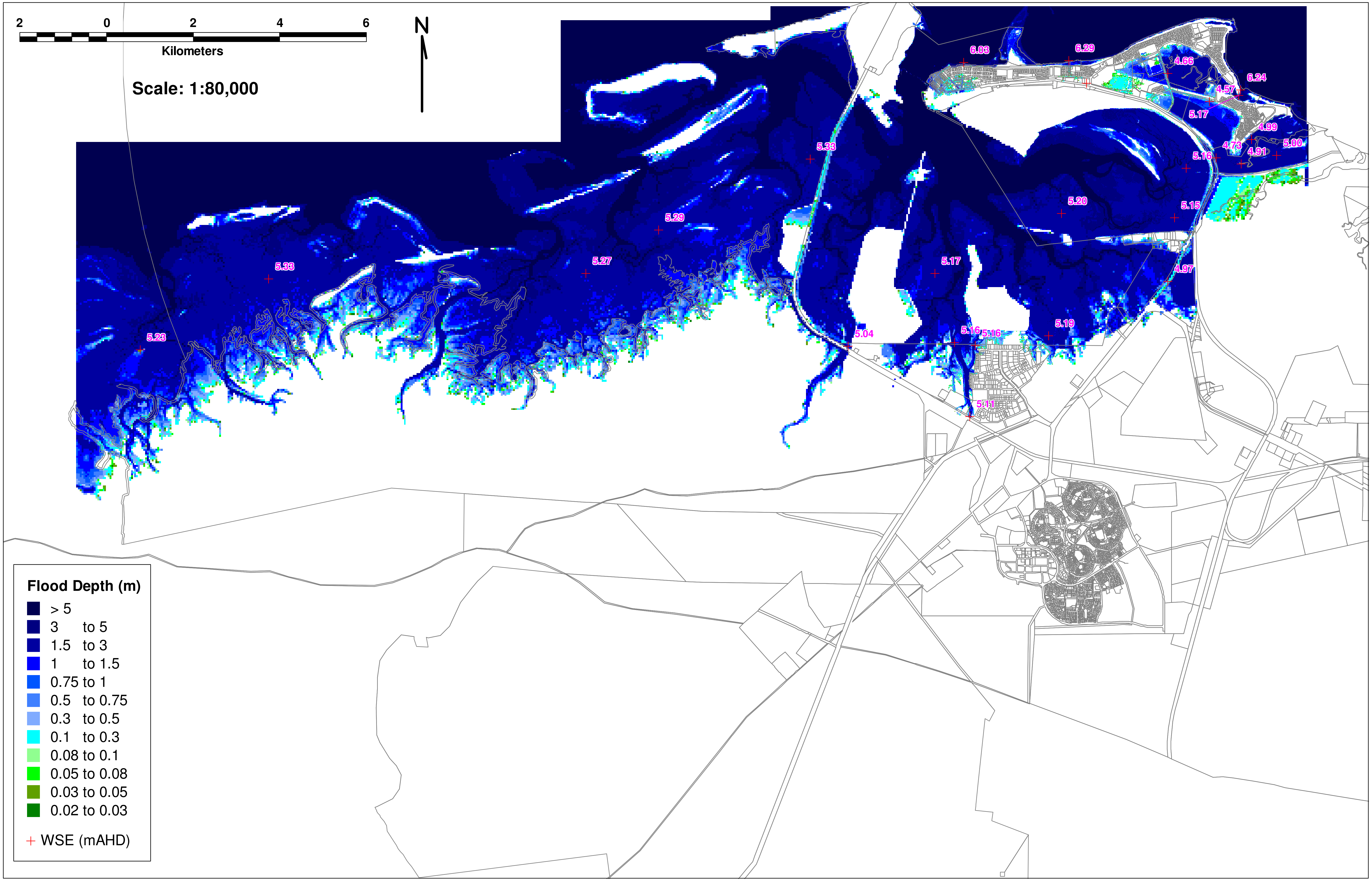
MAP P25 - 100-Year Combined Ocean Inundation Flood Depth - Existing Conditions



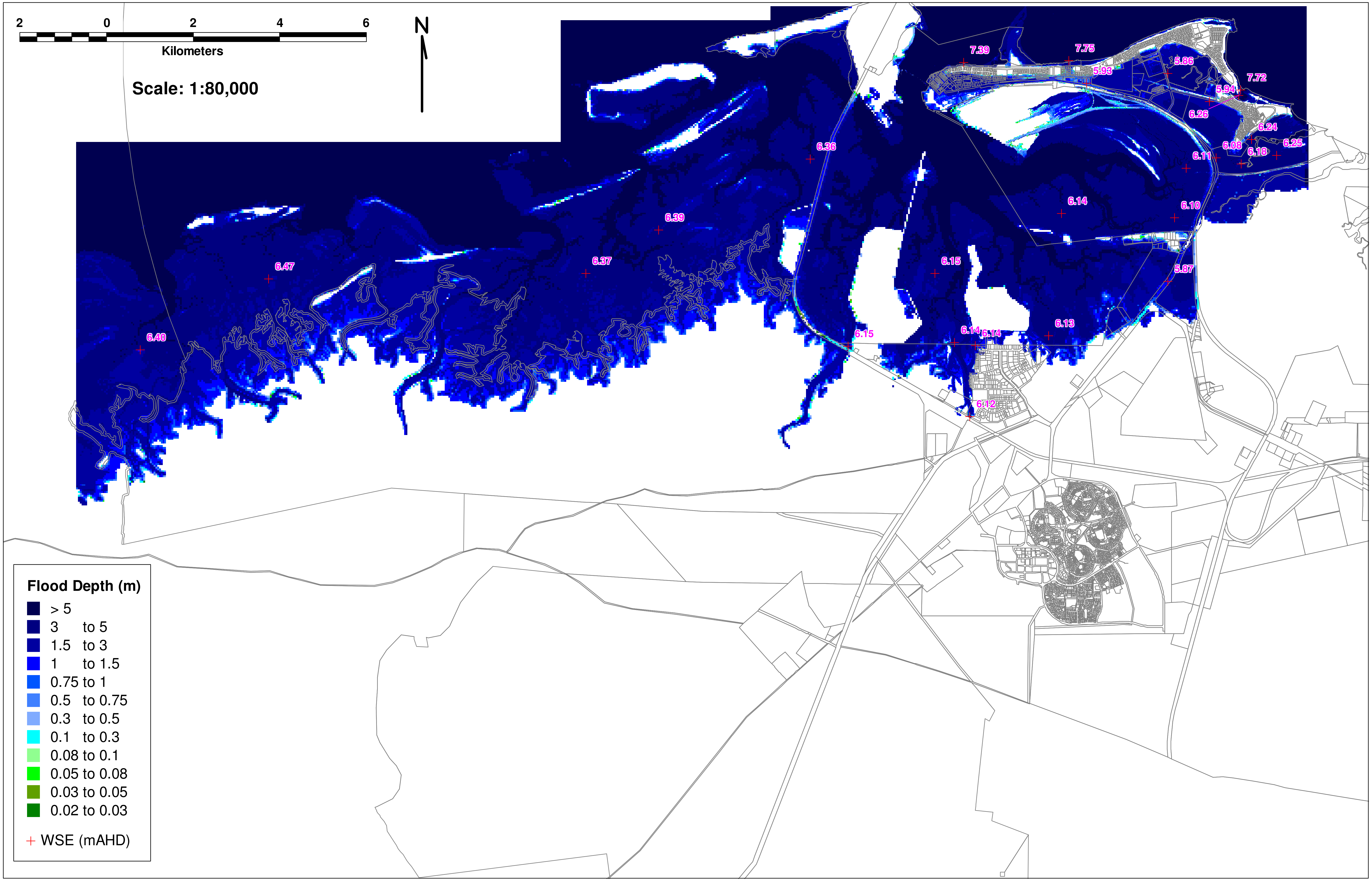
MAP P26 - 10-Year Combined Ocean Inundation Flood Depth - Existing Conditions



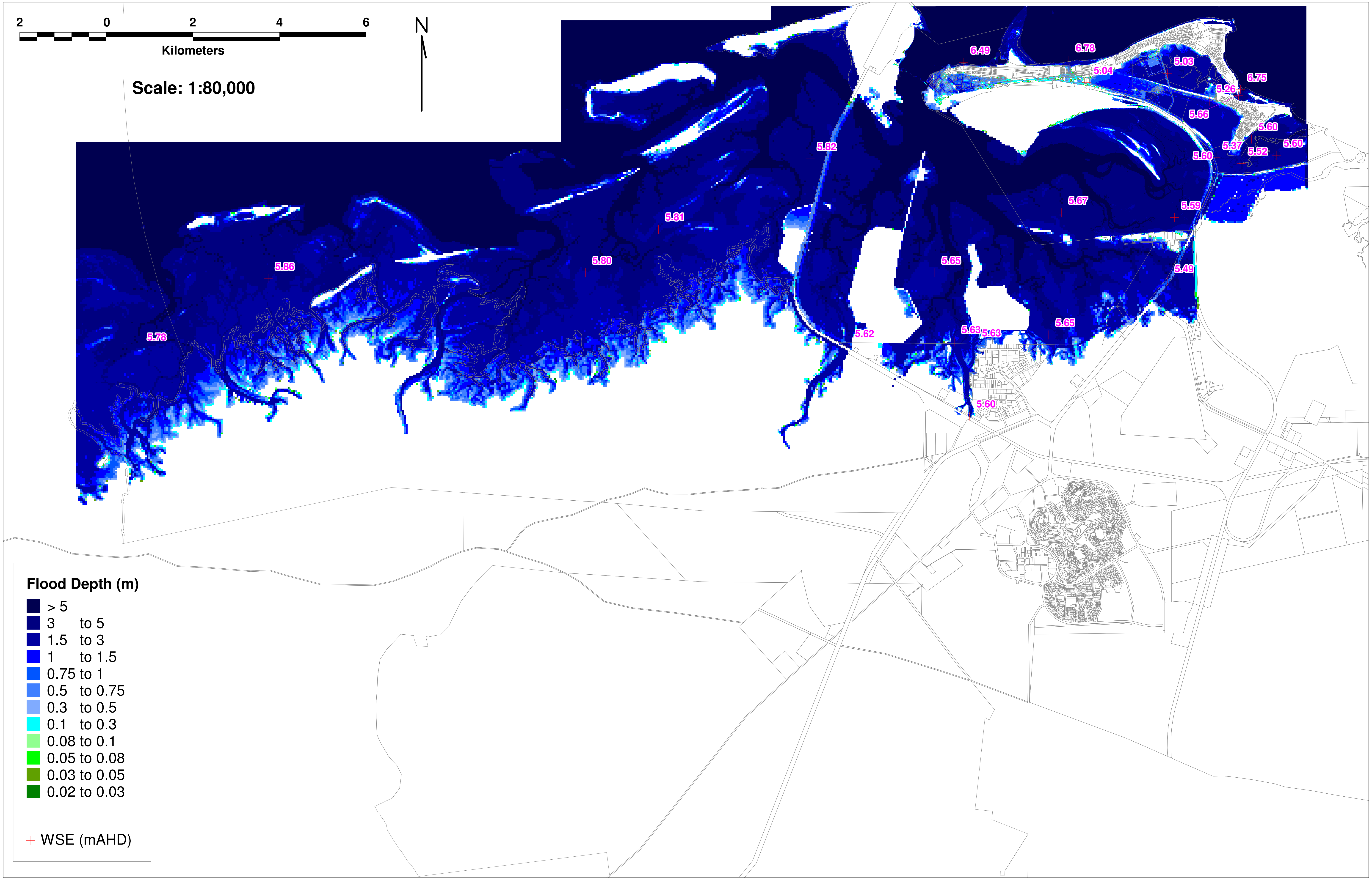
MAP P27 - 500-Year Combined Ocean Inundation Flood Depth - Climate Change 2060 Conditions



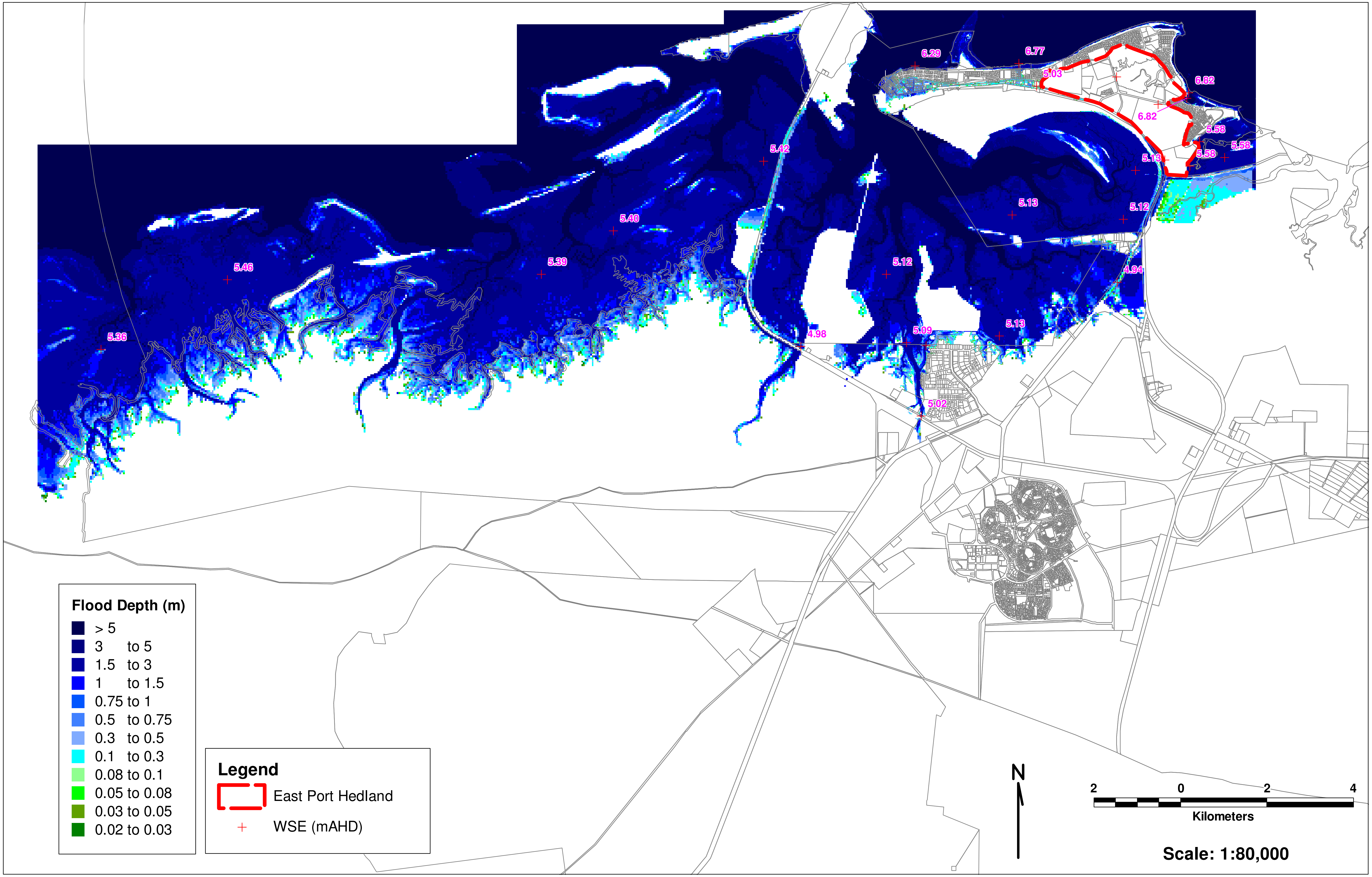
MAP P28 - 100-Year Combined Ocean Inundation Flood Depth - Climate Change 2060 Conditions



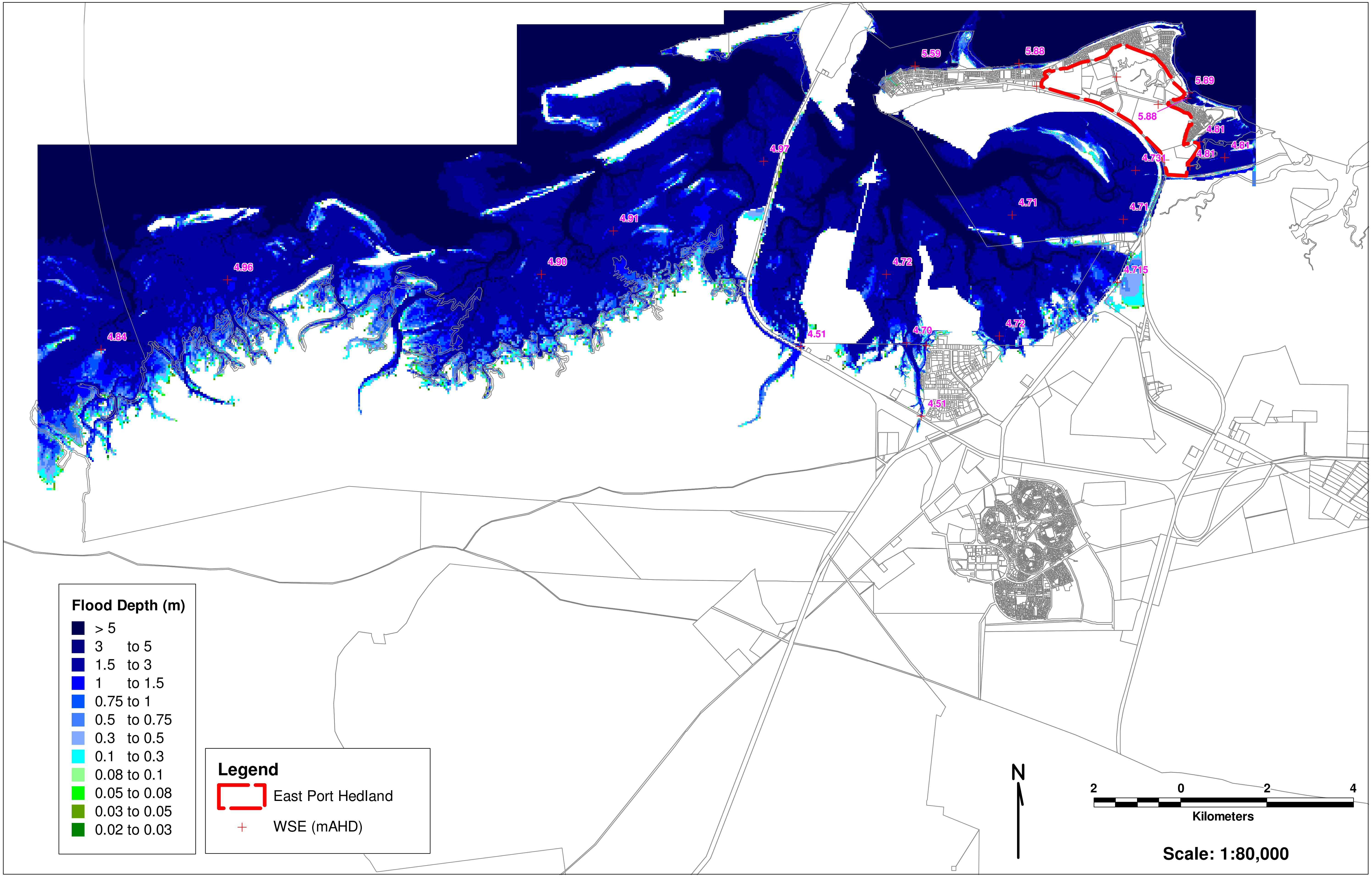
MAP P29 - 500-Year Combined Ocean Inundation Flood Depth - Climate Change 2110 Conditions



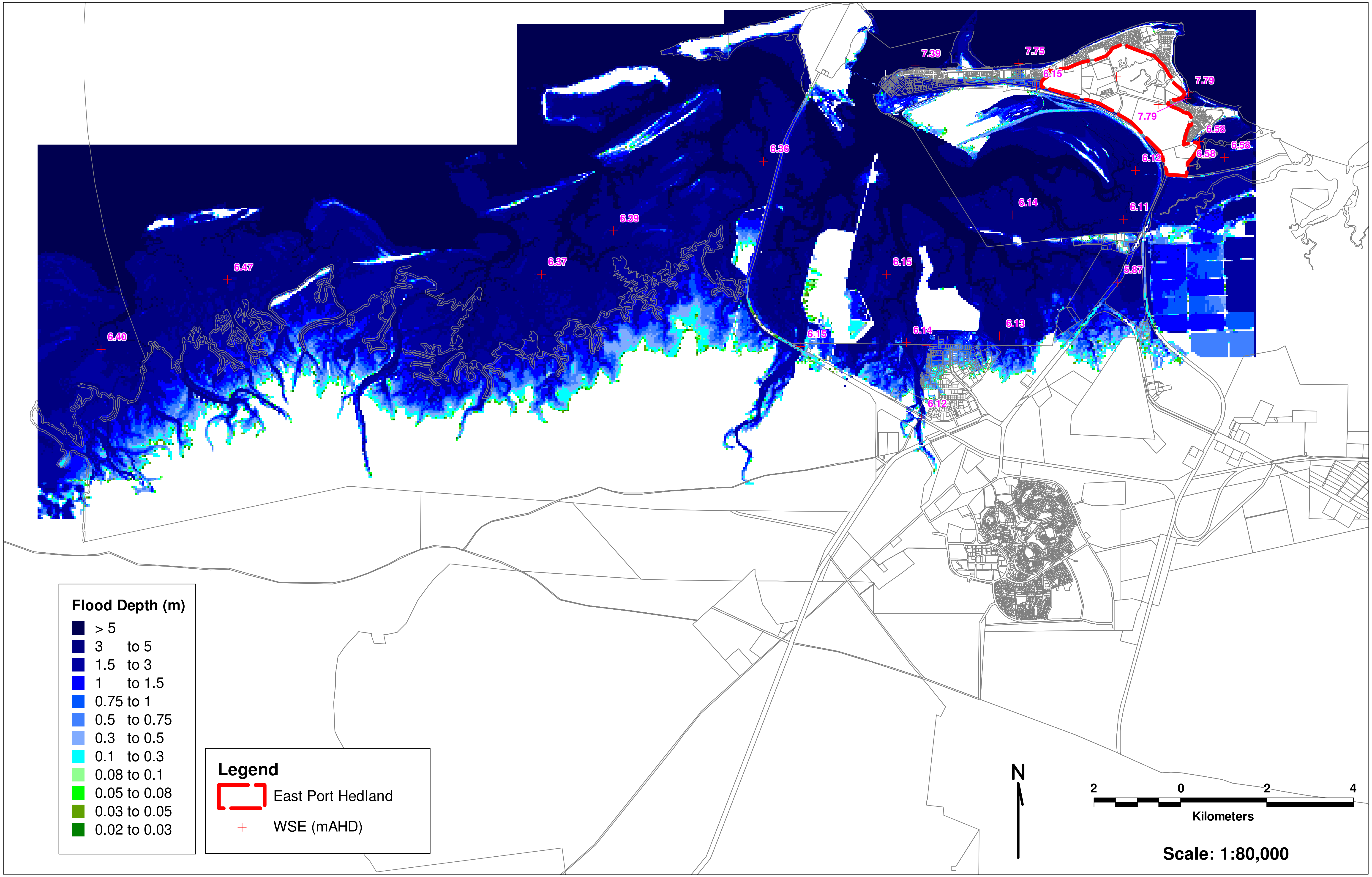
MAP P30 - 100-Year Combined Ocean Inundation Flood Depth - Climate Change 2110 Conditions



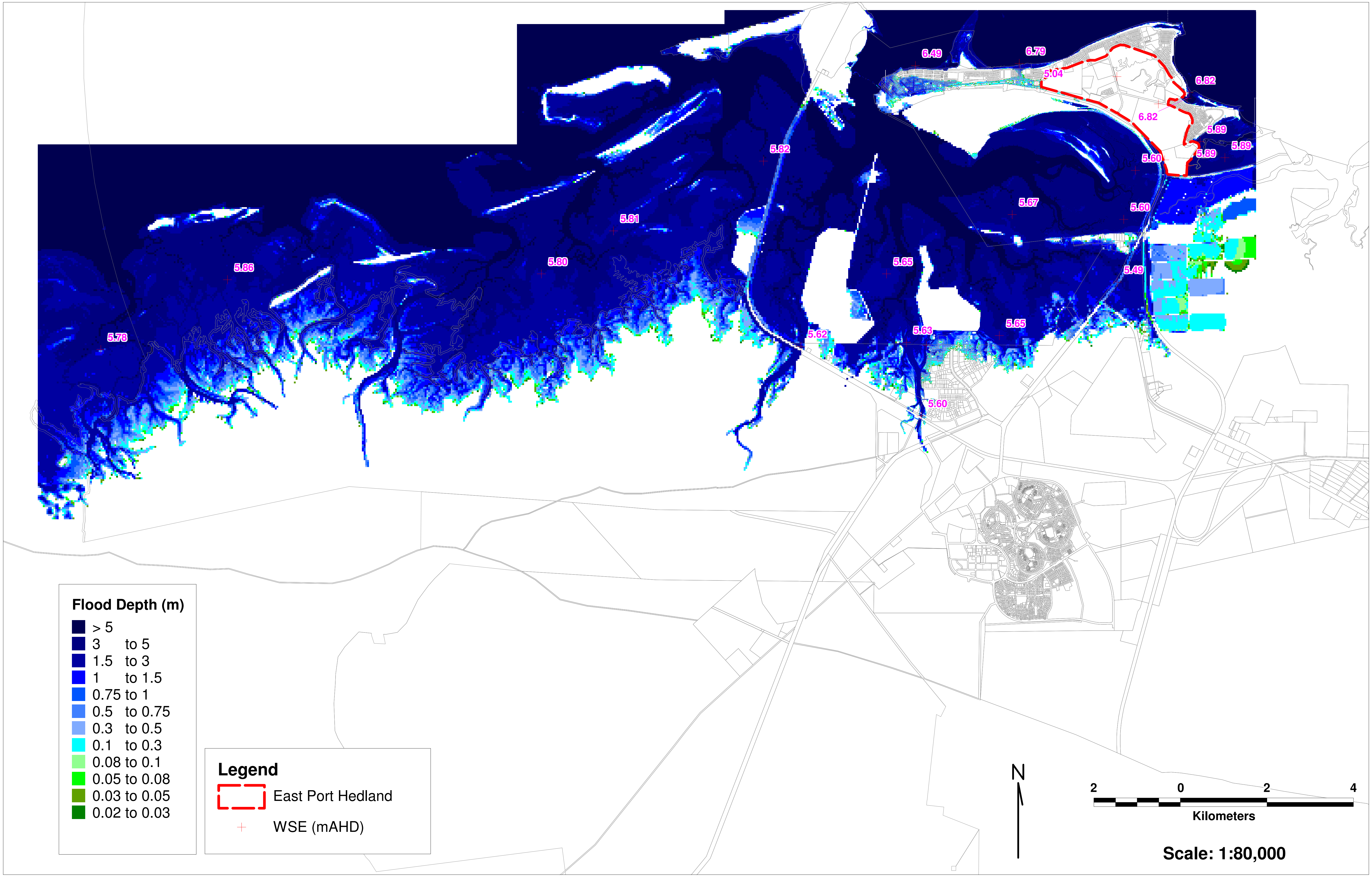
MAP P31 - 500-Year Combined Ocean Inundation Flood Depth - Existing Conditions - Developed East Port Hedland



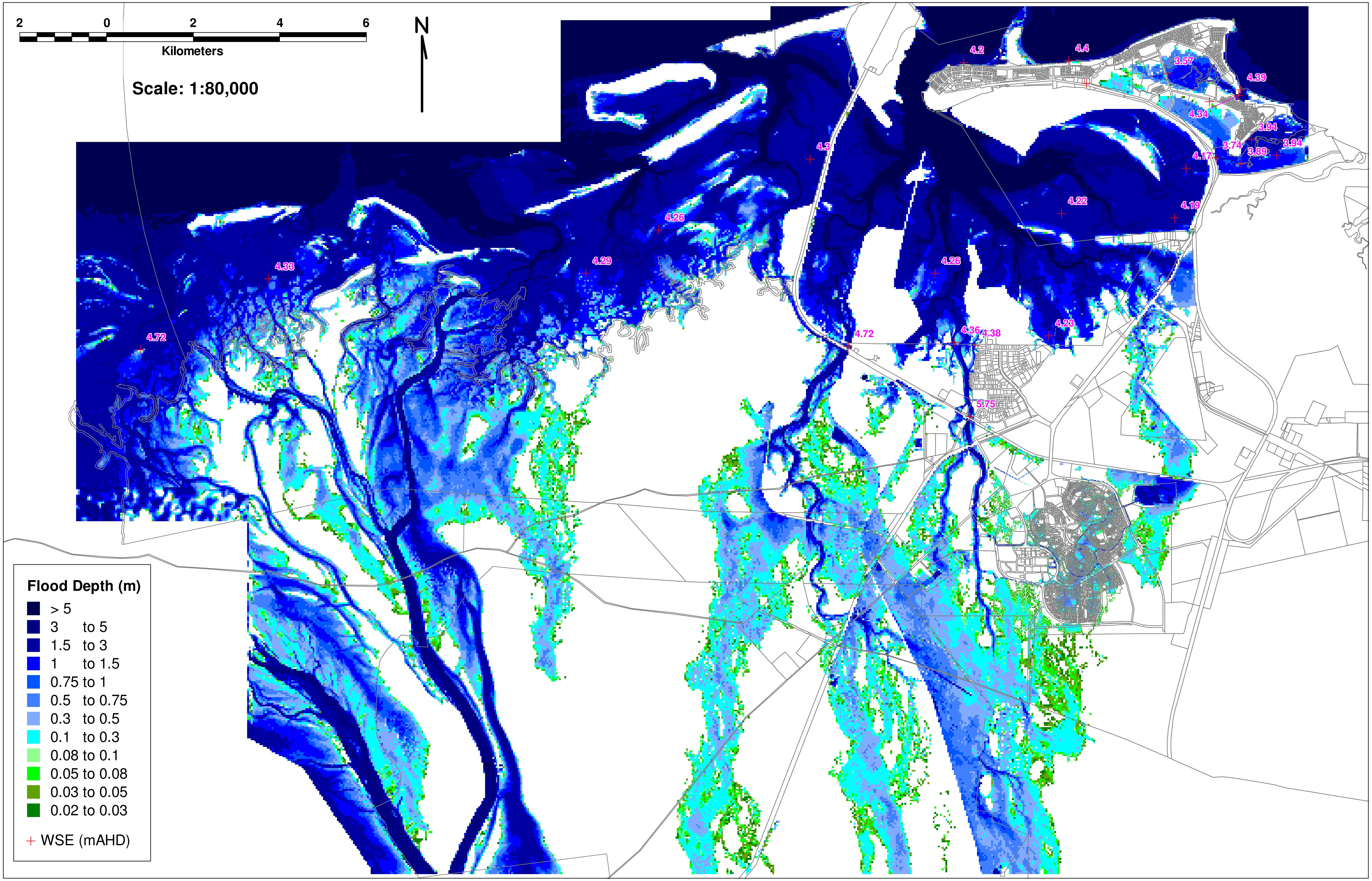
MAP P32 - 100-Year Combined Ocean Inundation Flood Depth - Existing Conditions - Developed East Port Hedland



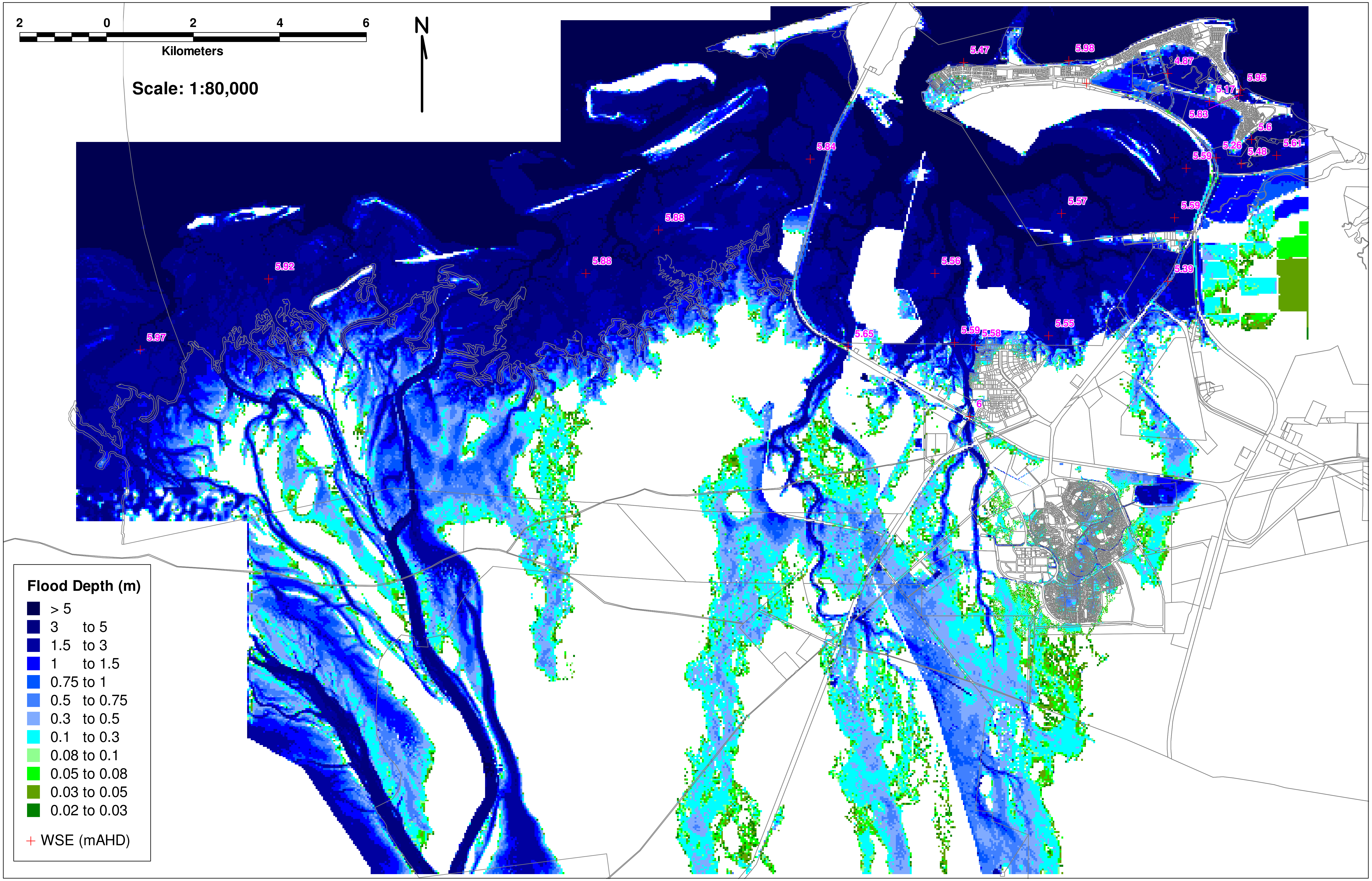
MAP P33 - 500-Year Combined Ocean Inundation Flood Depth - Climate Change 2110 Conditions - Developed East Port Hedland



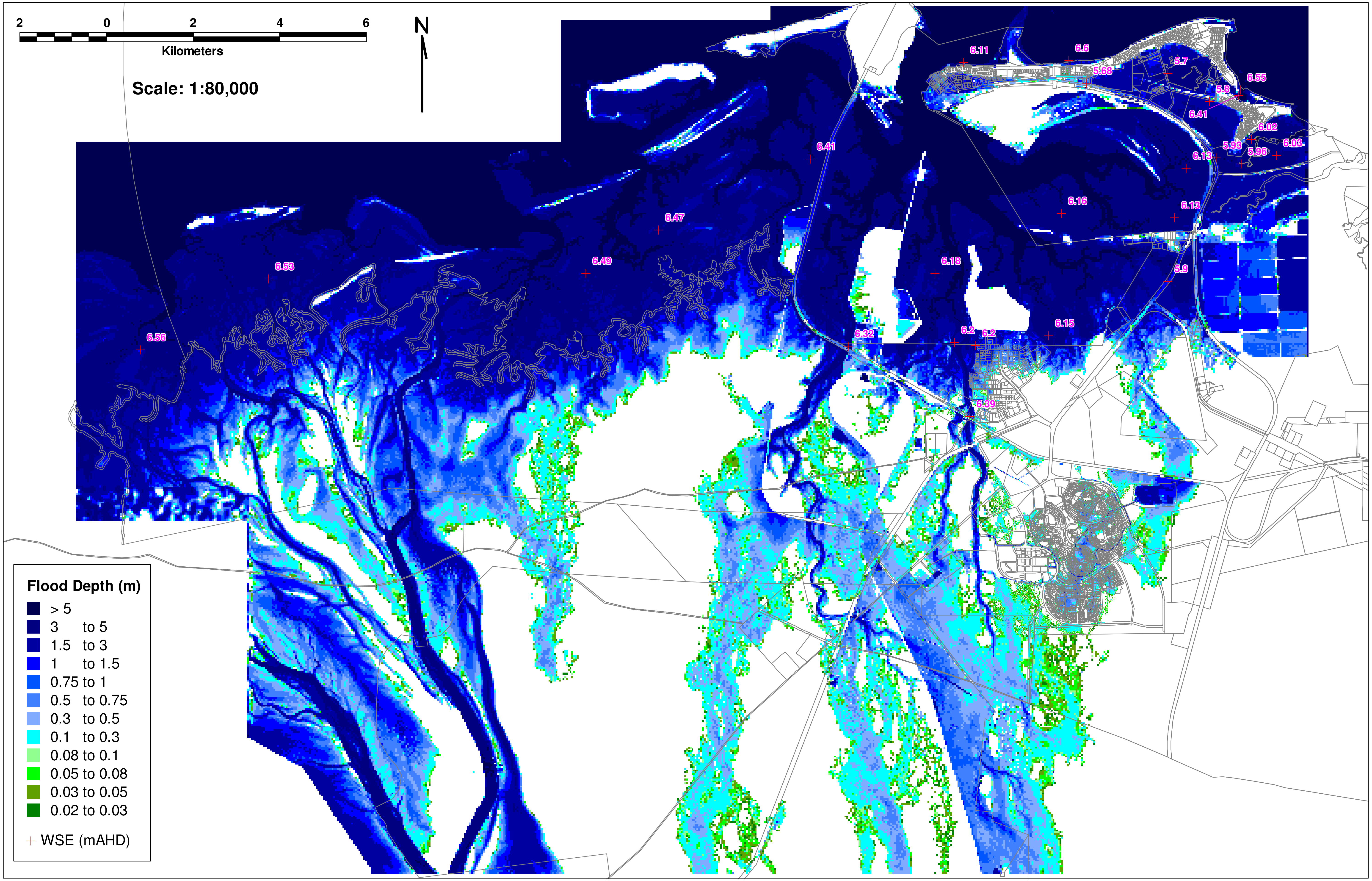
MAP P34 - 100-Year Combined Ocean Inundation Flood Depth - Climate Change 2110 Conditions - Developed East Port Hedland



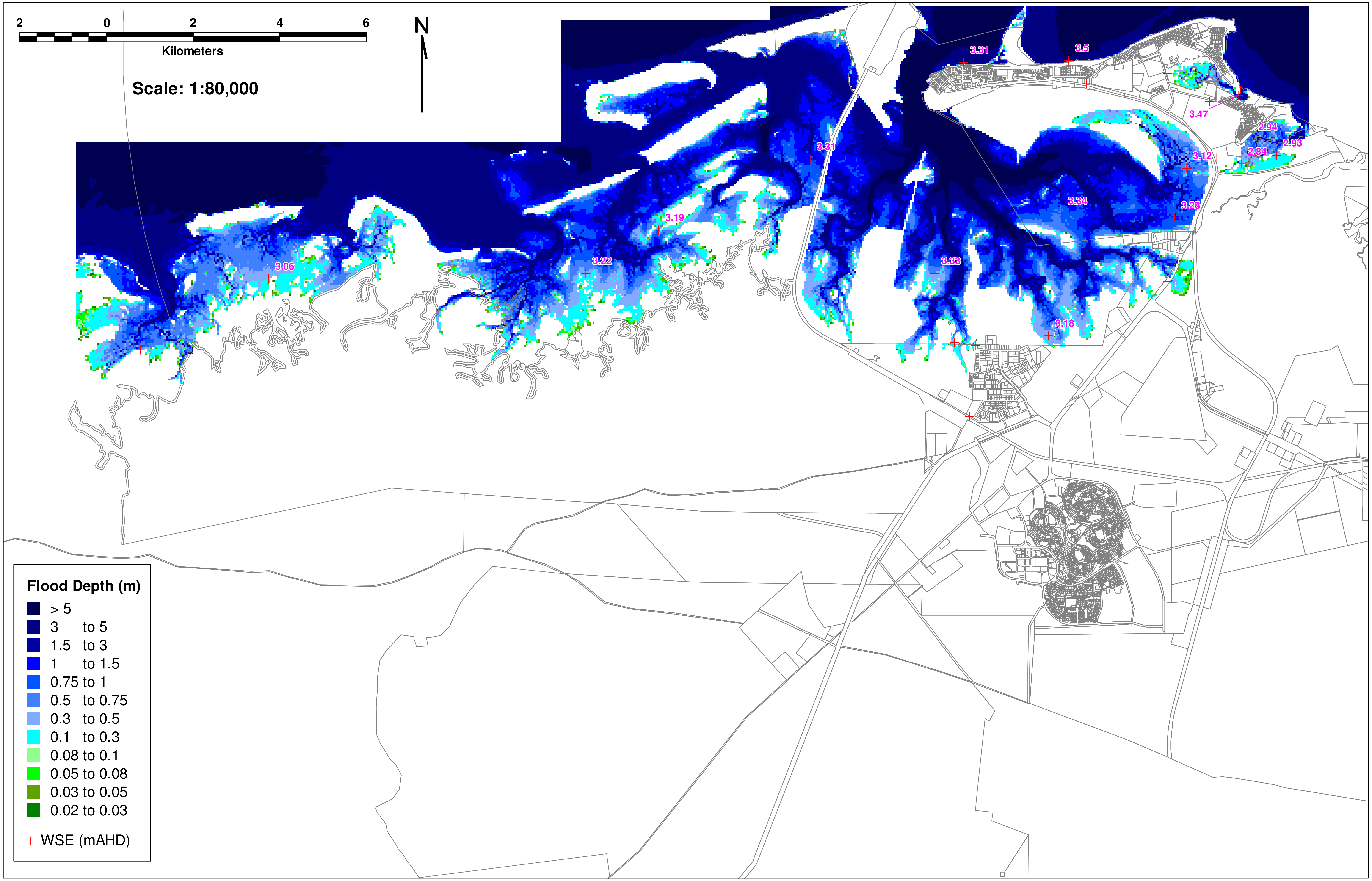
MAP P35 - 50-Year Ocean Inundation & 10-Year Catchment Flow Flood Depth - Existing Conditions



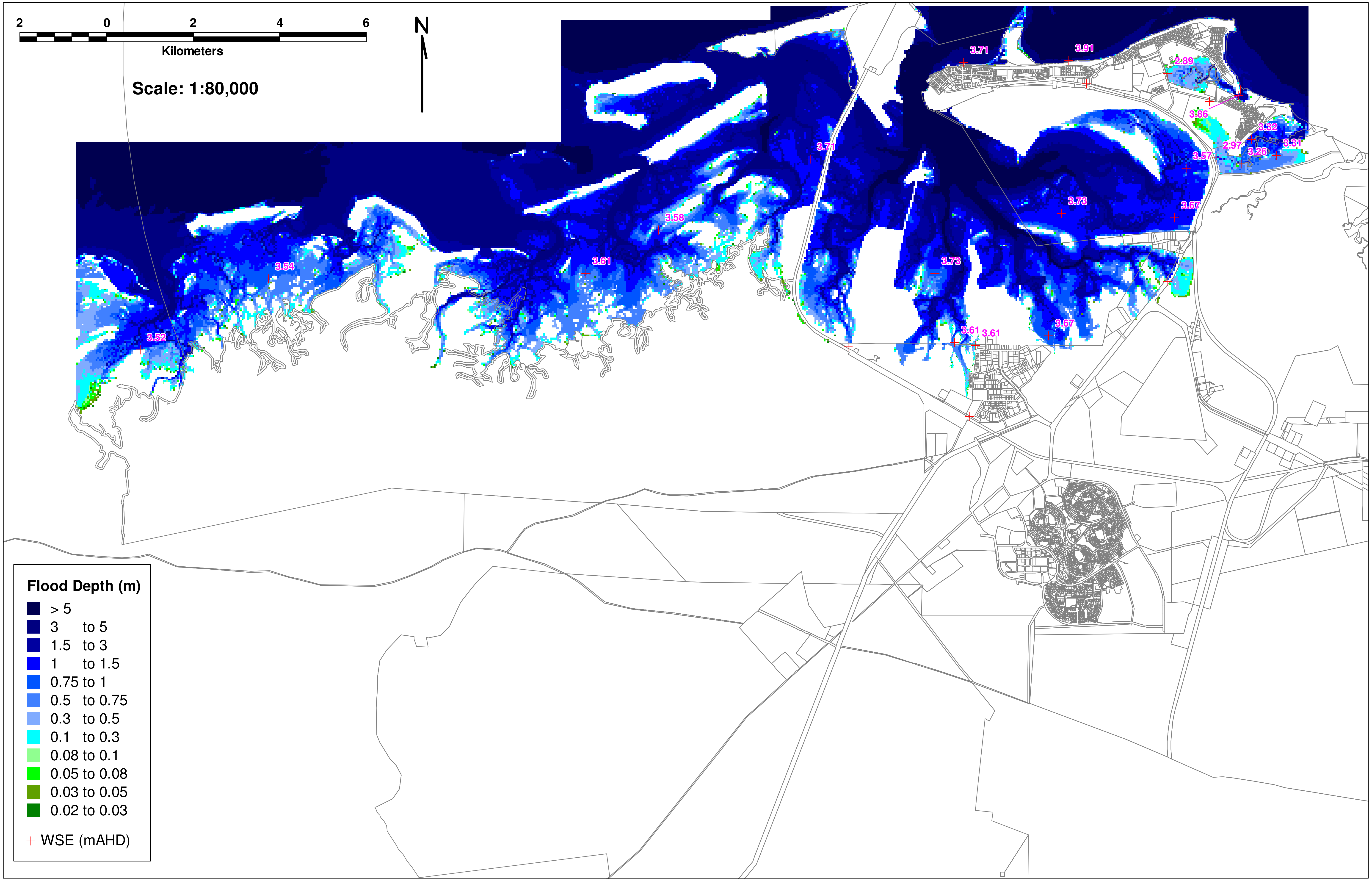
MAP P36 - 50-Year Ocean Inundation & 10-Year Catchment Flow Flood Depth - Climate Change 2060



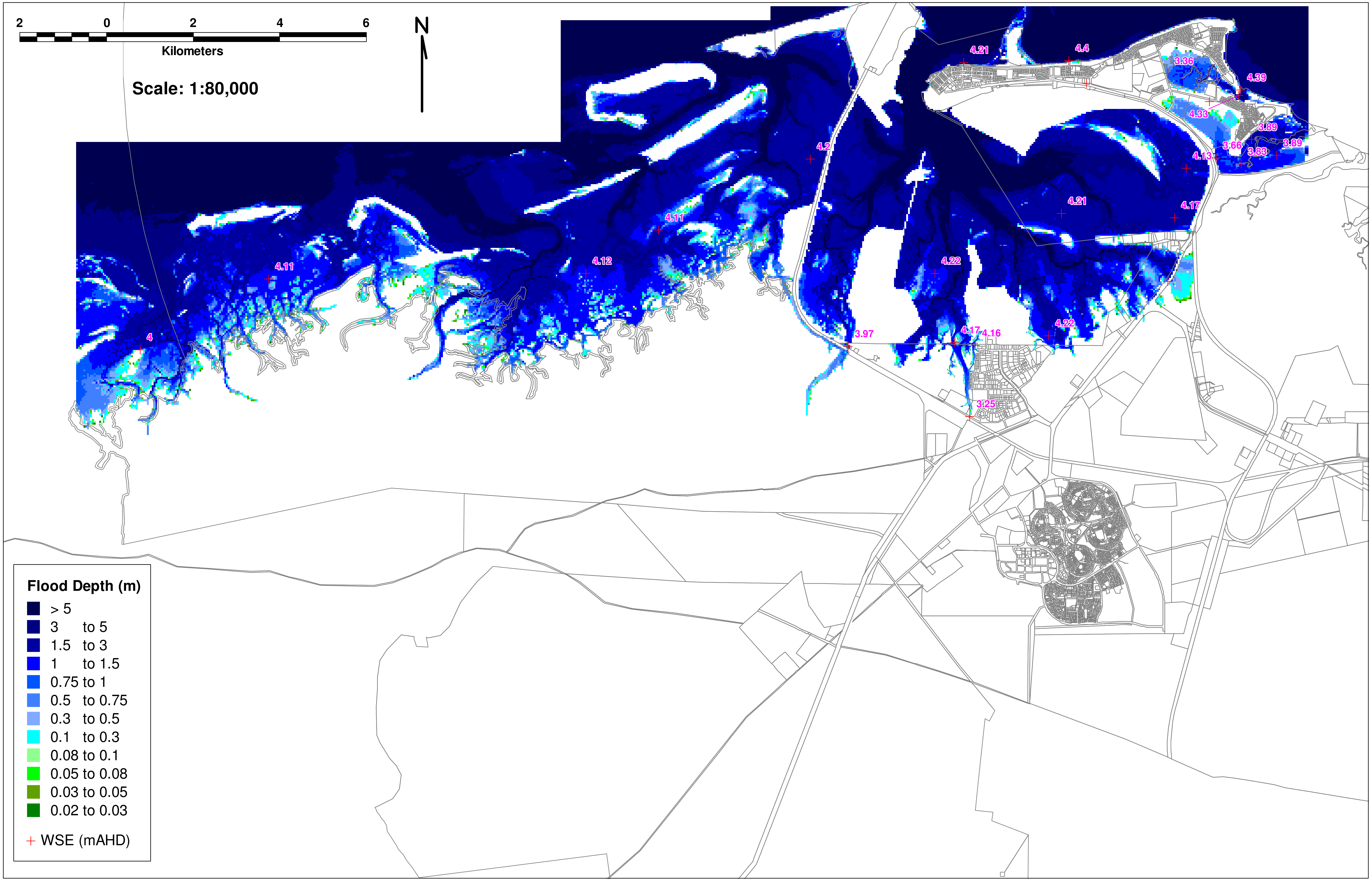
MAP P37 - 50-Year Ocean Inundation & 10-Year Catchment Flow Flood Depth - Climate Change 2110



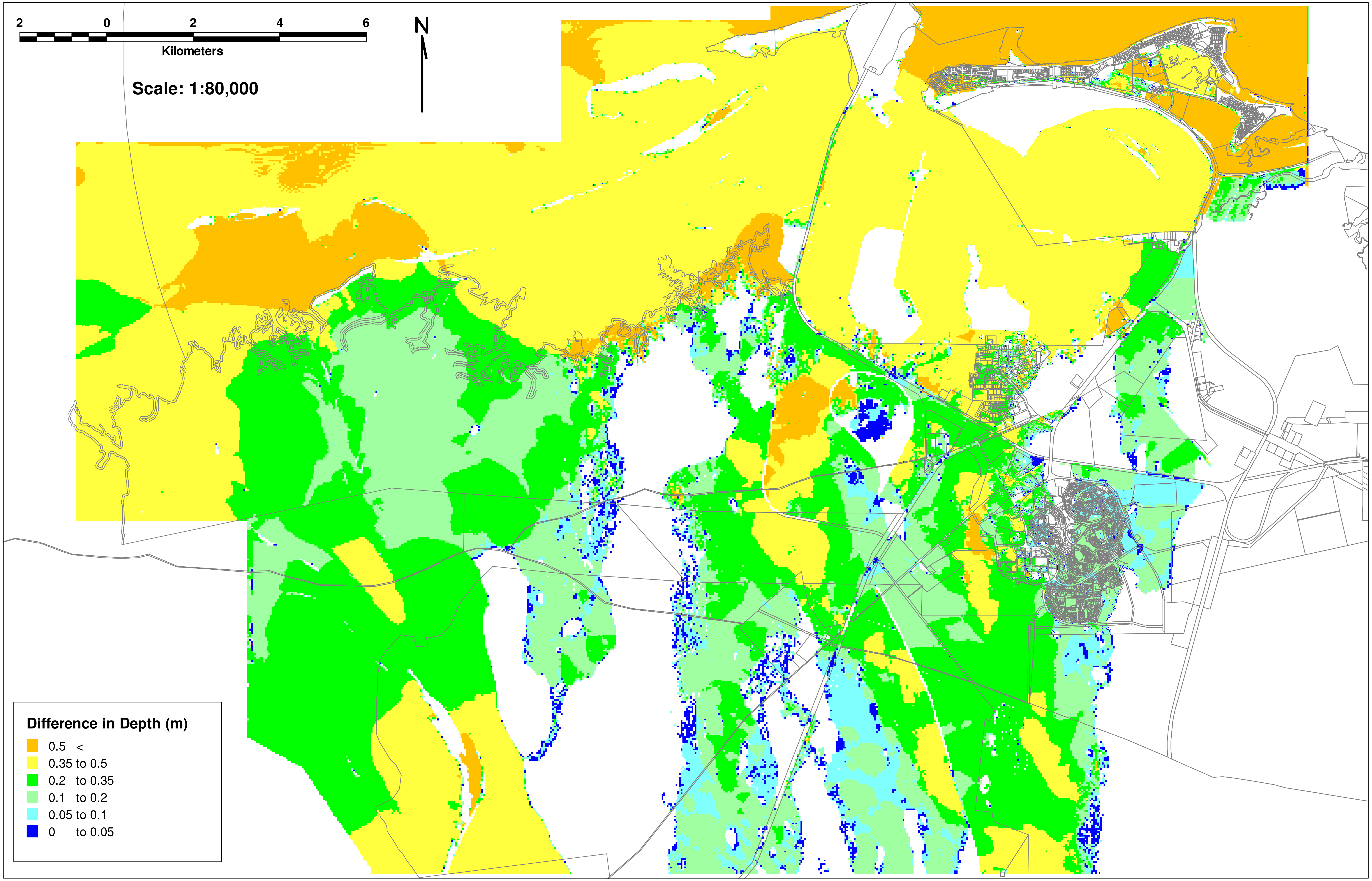
MAP P38 - 2-Year Combined Ocean Inundation Flood Depth - Existing Conditions



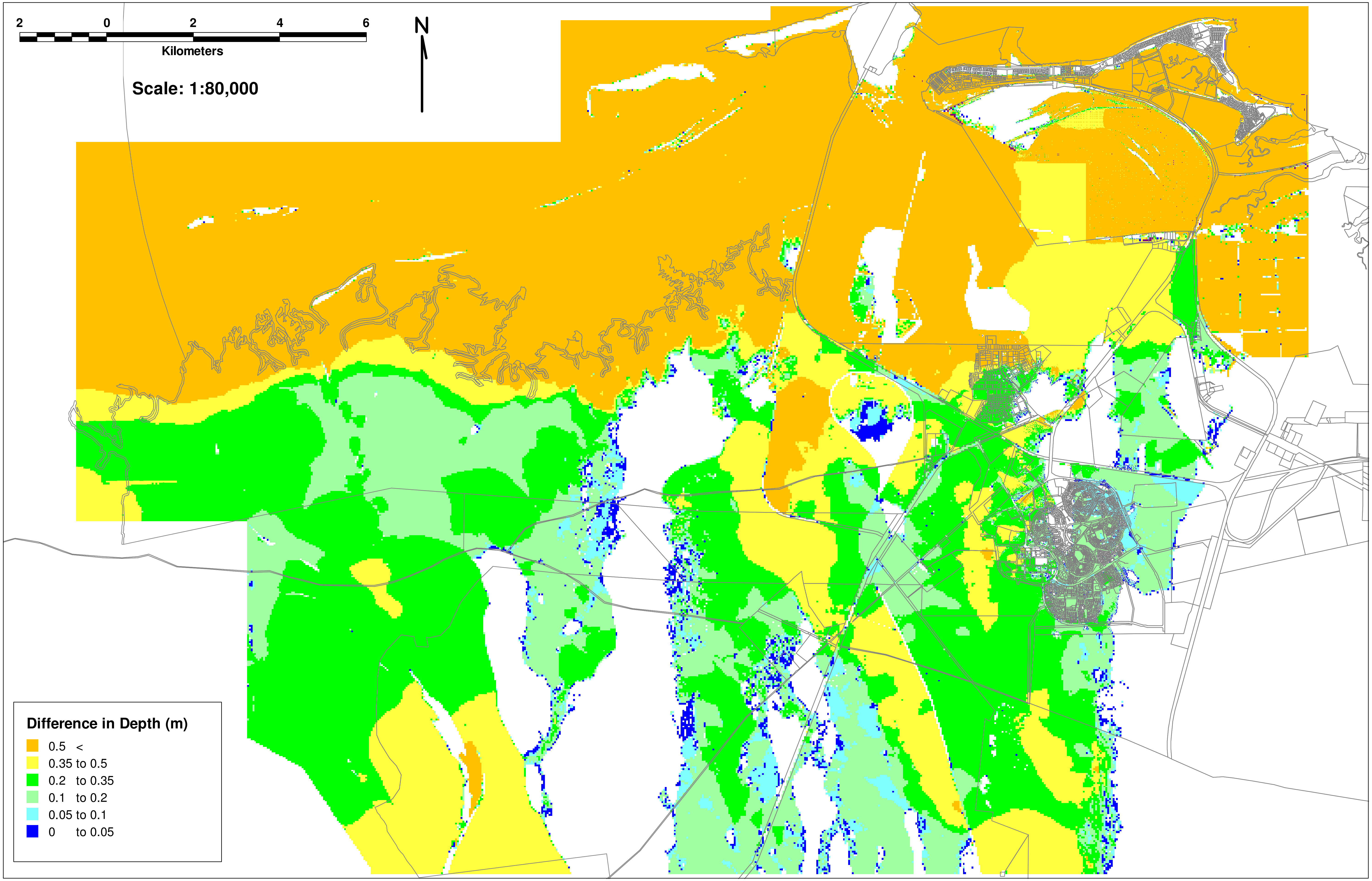
MAP P39 - 2-Year Combined Ocean Inundation Flood Depth - Climate Change 2060



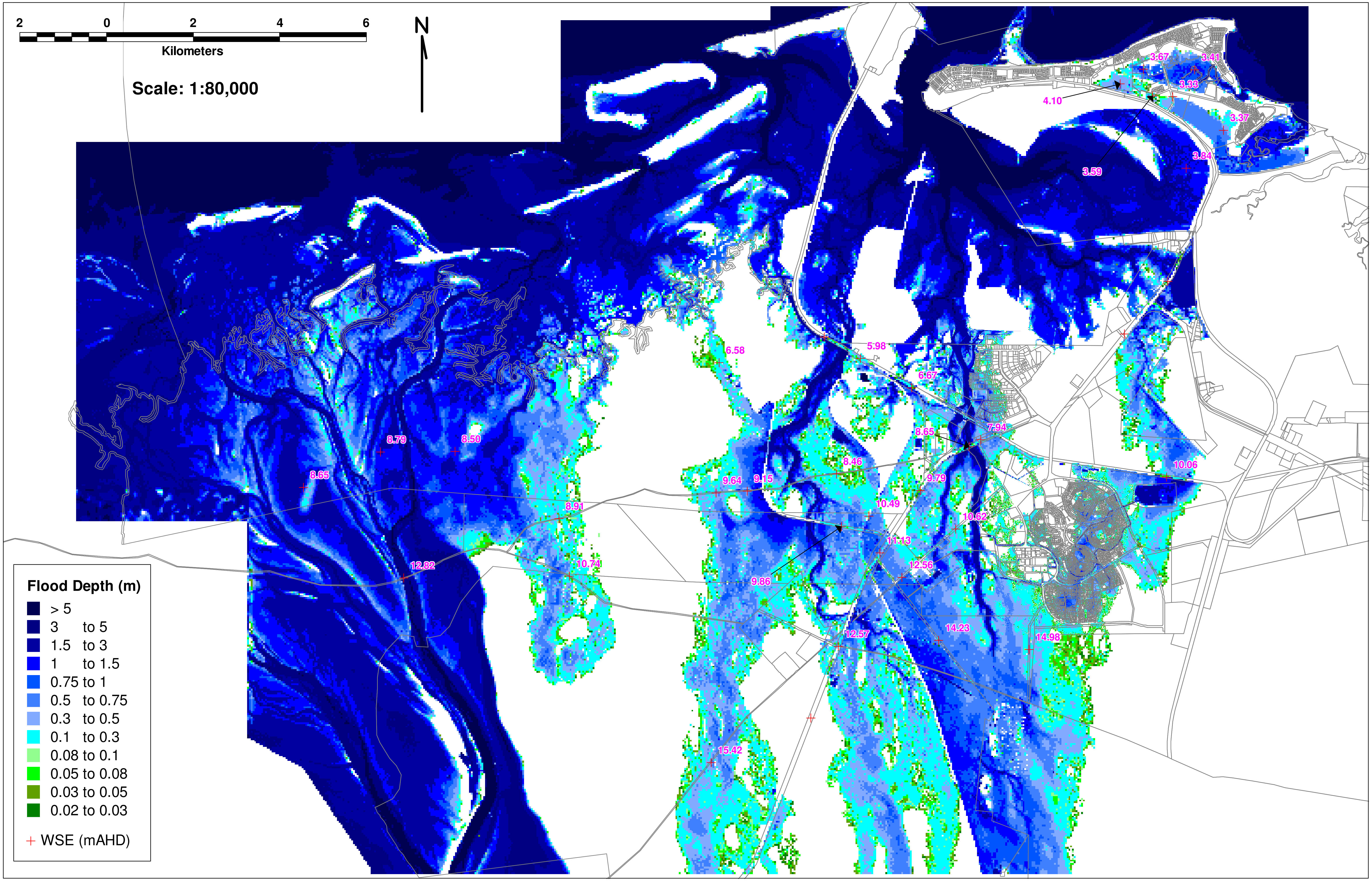
MAP P40 - 2-Year Combined Ocean Inundation Flood Depth - Climate Change 2110



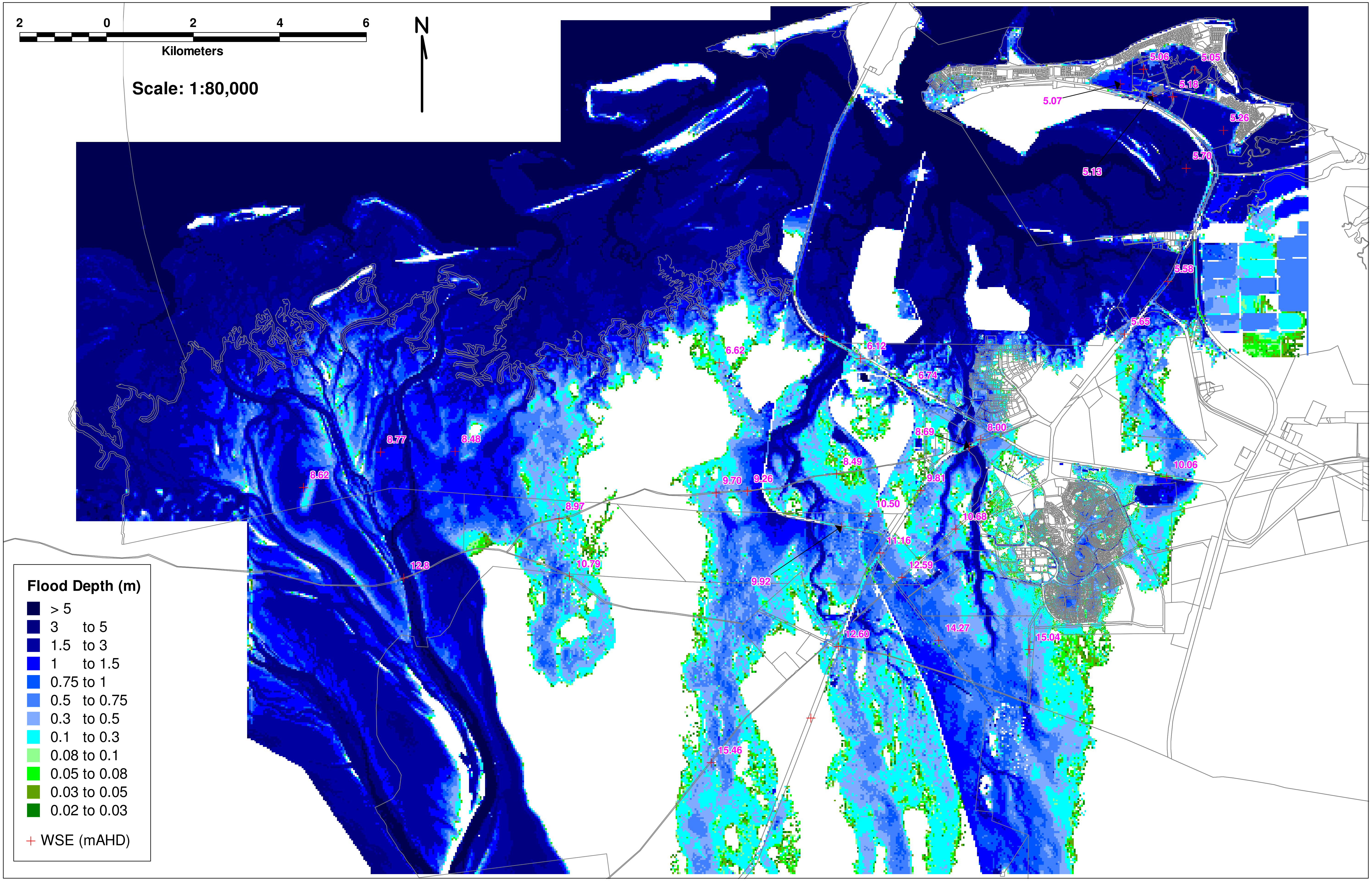
MAP P41 - Depth Difference Plot - 500-year Existing Conditions Minus 100-year Existing Conditions - Port Hedland



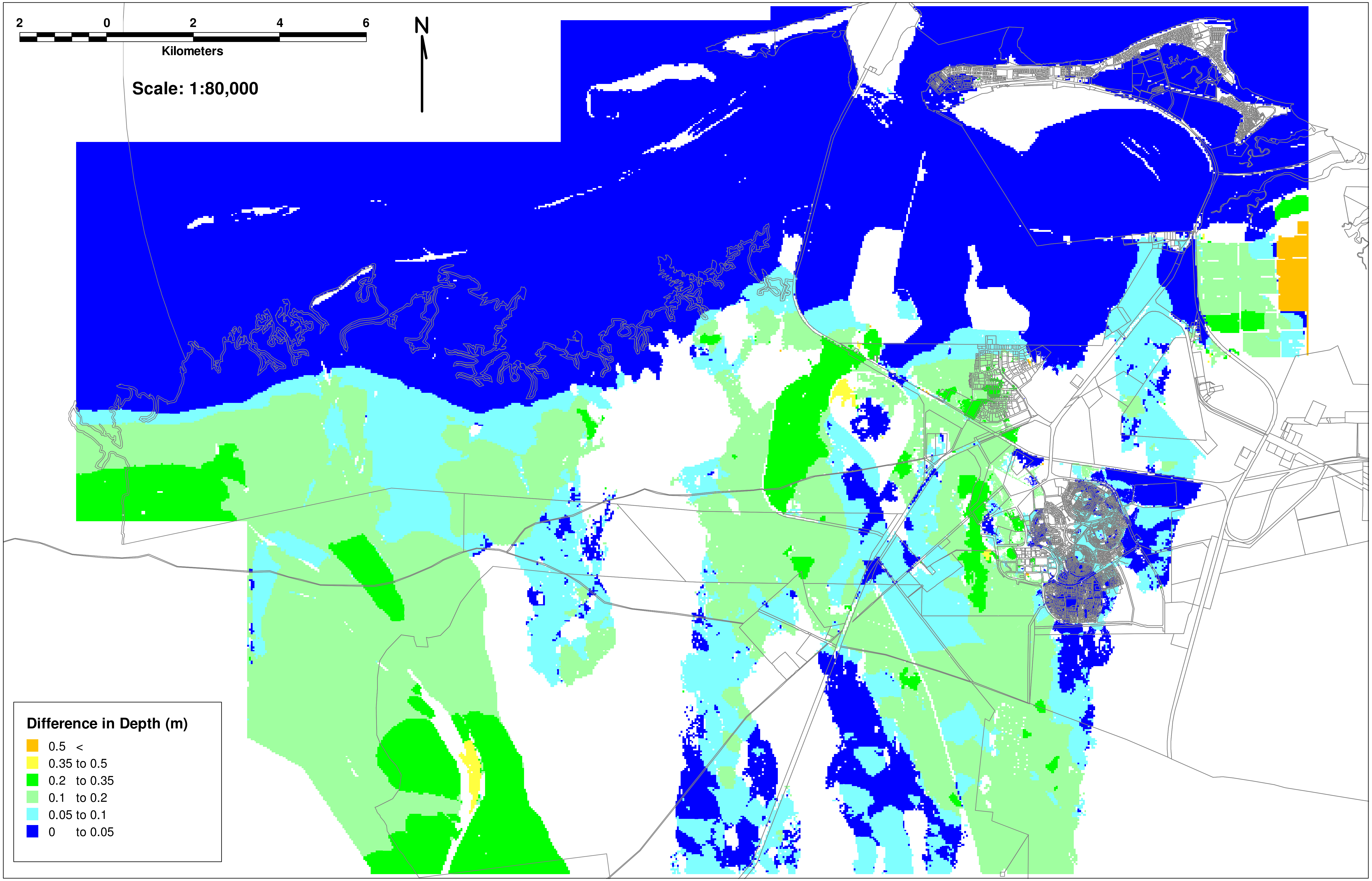
MAP P42 - Depth Difference Plot - 500-year Climate Change 2110 Condition Minus 100-year Climate Change 2110 Conditions - Port Hedland



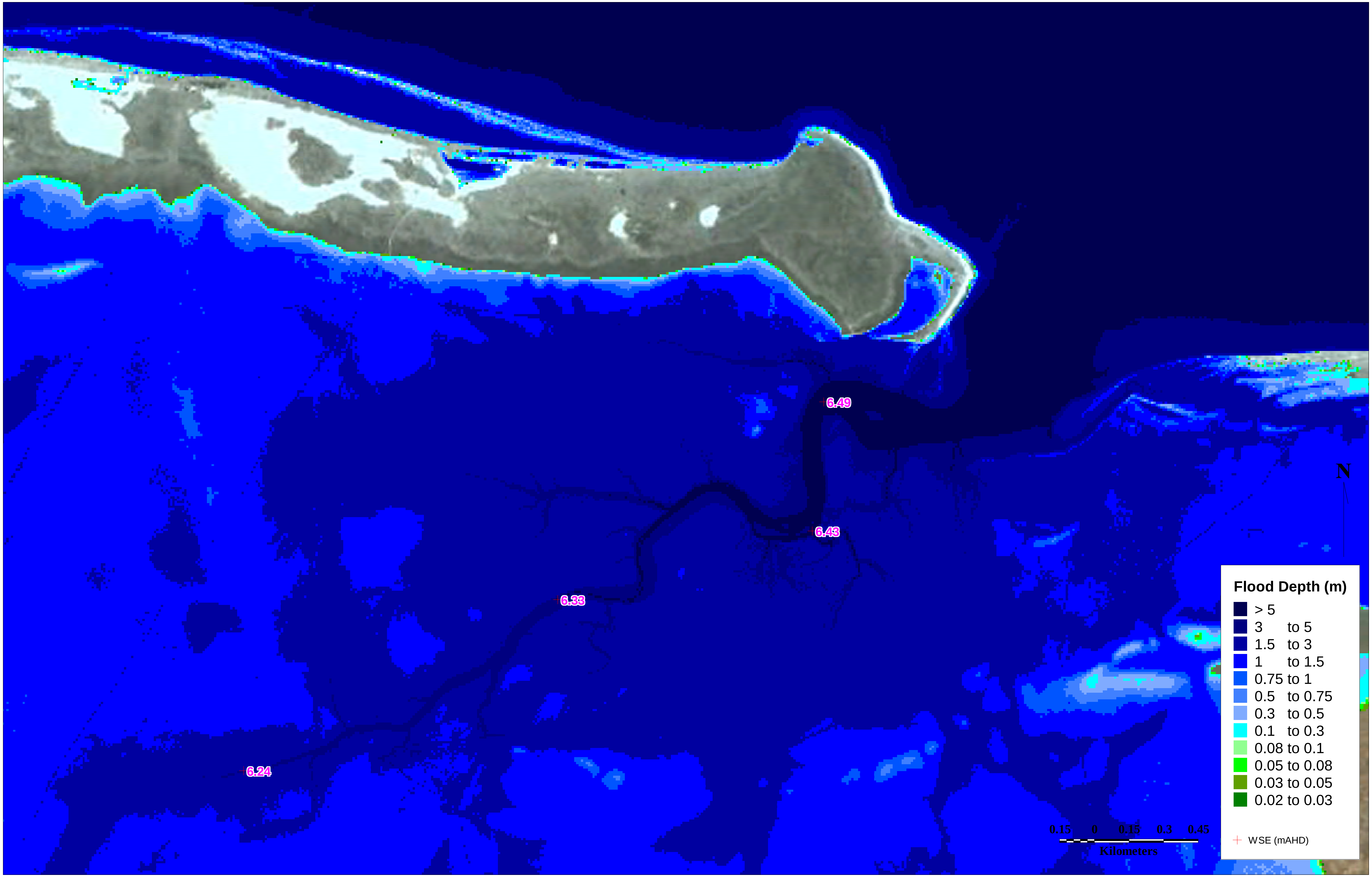
MAP P43 - 100-year Flood Depth - Existing Conditions - Sensitivity Analysis - Port Hedland



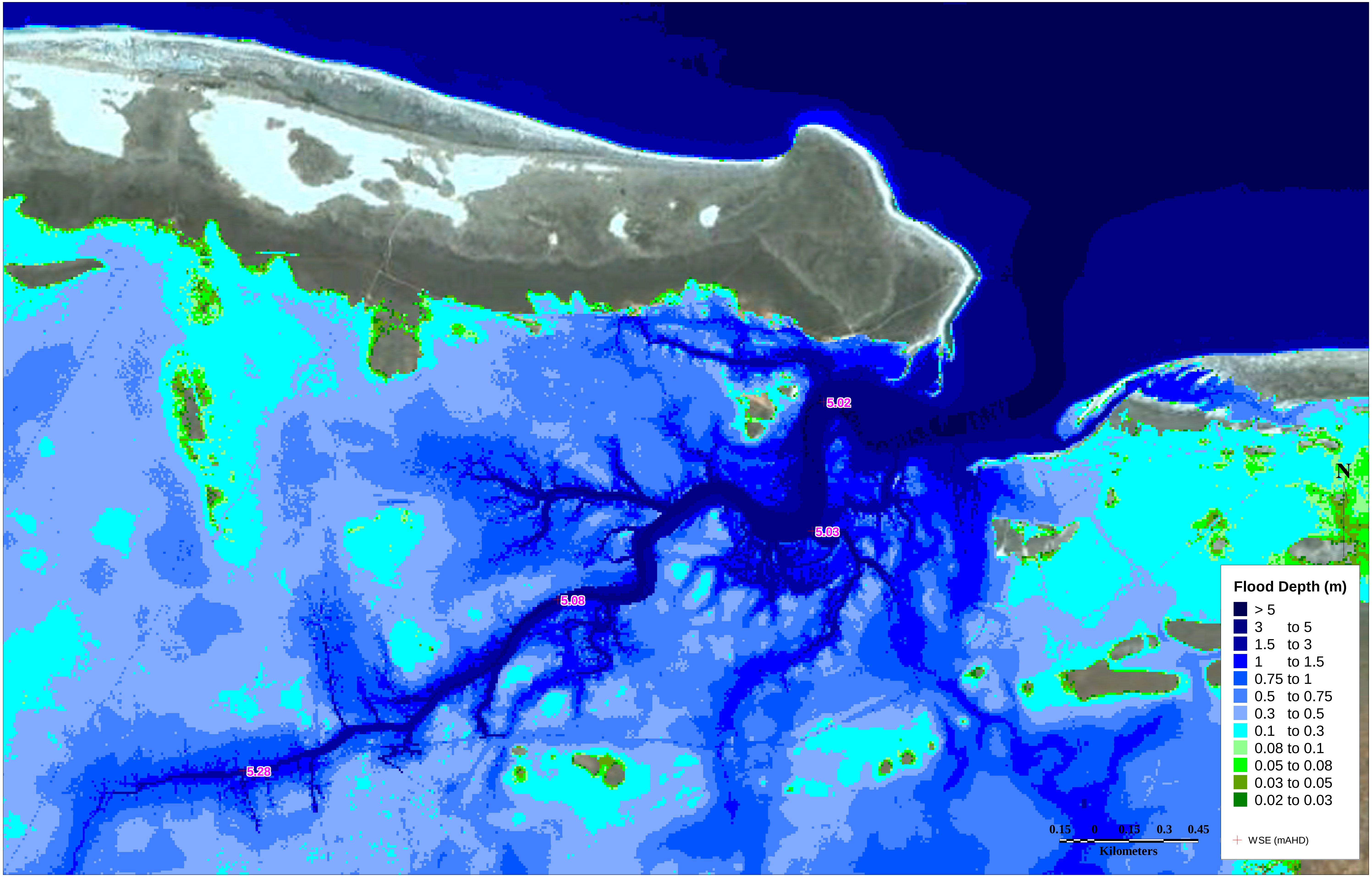
MAP P44 - Flood Depth - 100-year Existing Catchment Inundation with Climate Change 2110 Downstream Conditions- Port Hedland



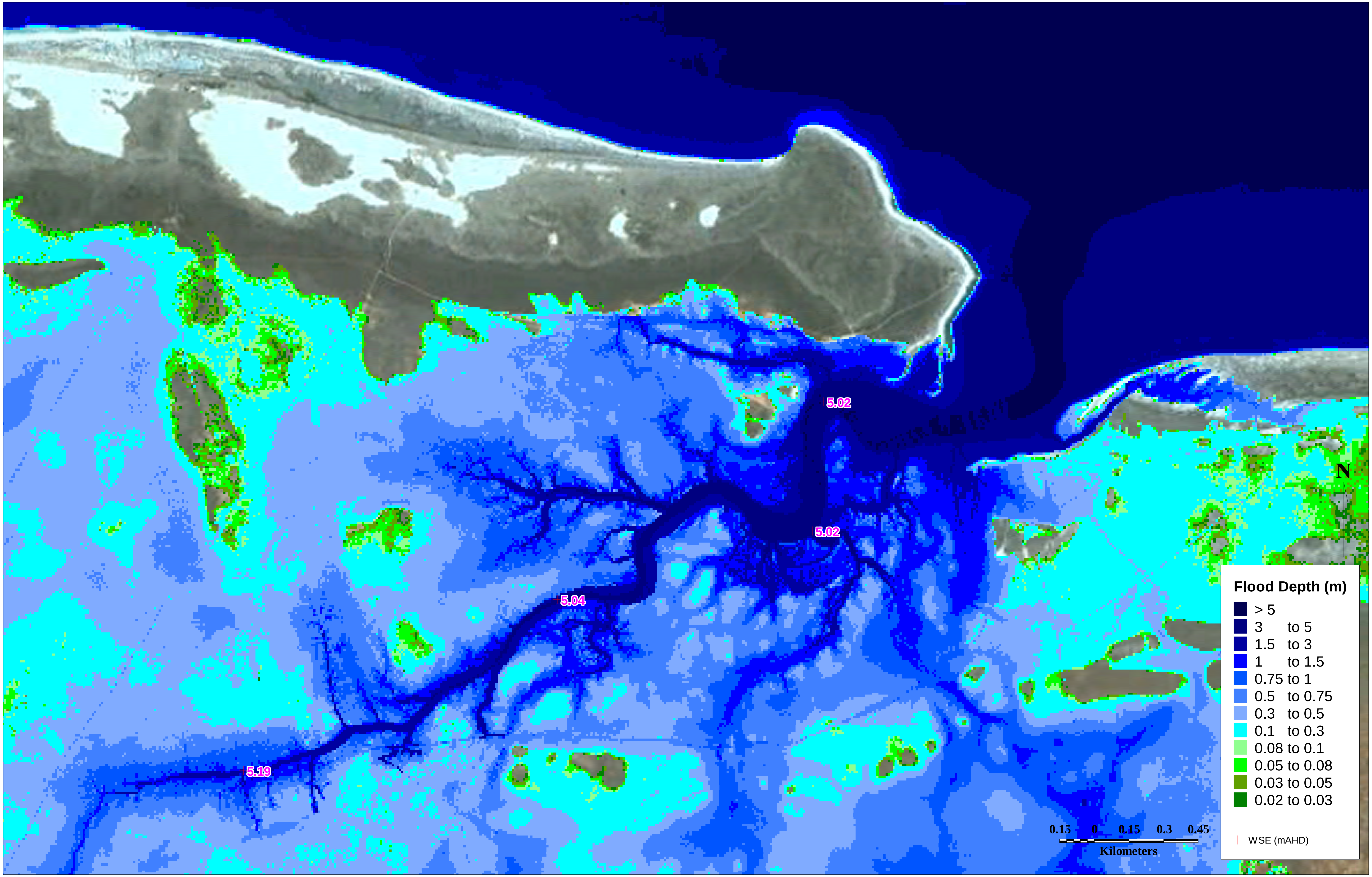
MAP P45 - Depth Difference Plot - 100-year Existing Catchment Inundation with Climate Change 2110 Downstream Condition Minus 100-year Climate Change 2110 Catchment Inundation - Port Hedland



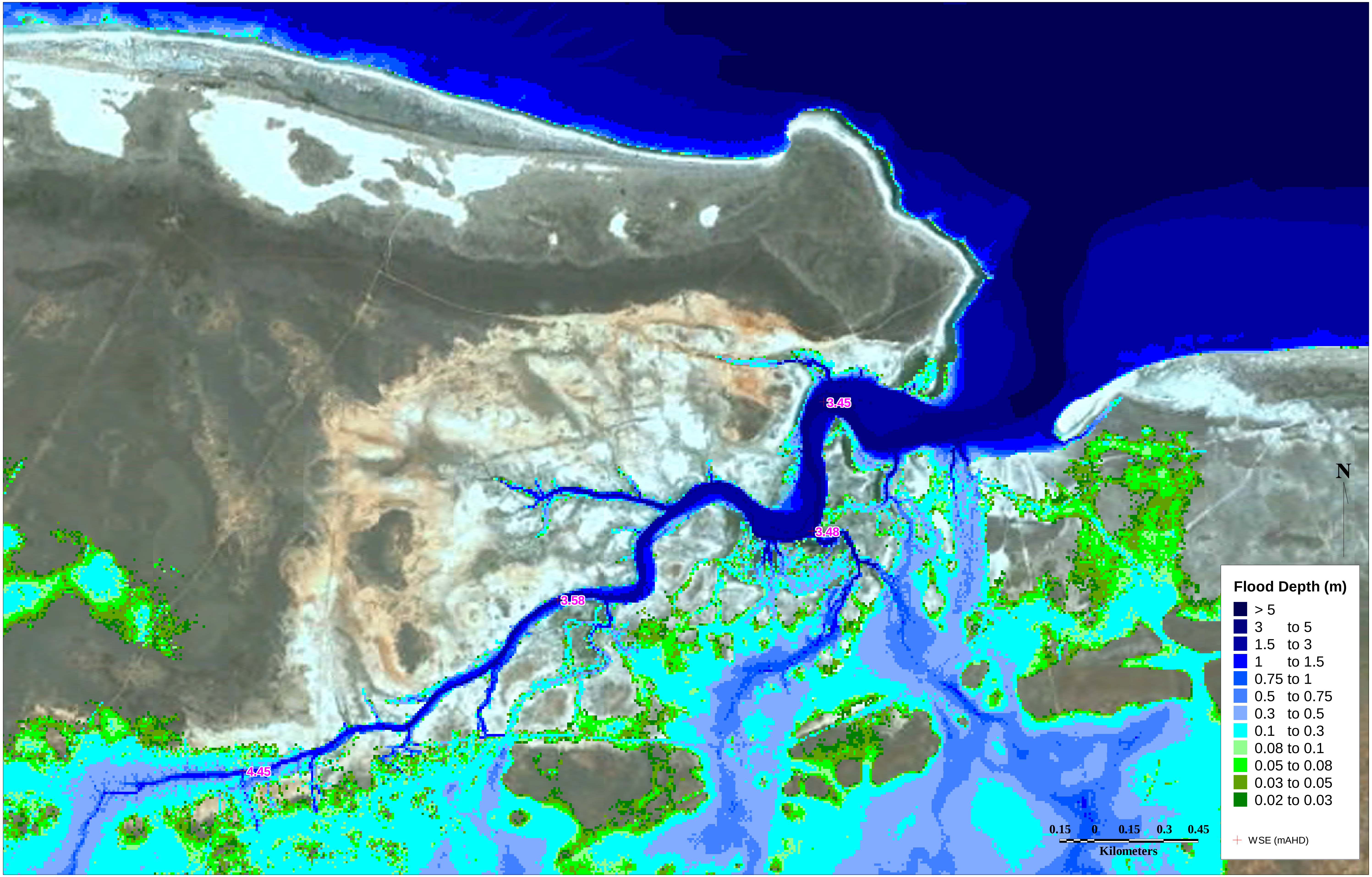
Map S01 - 500 Year ARI Event Flood Depth - Existing Conditions



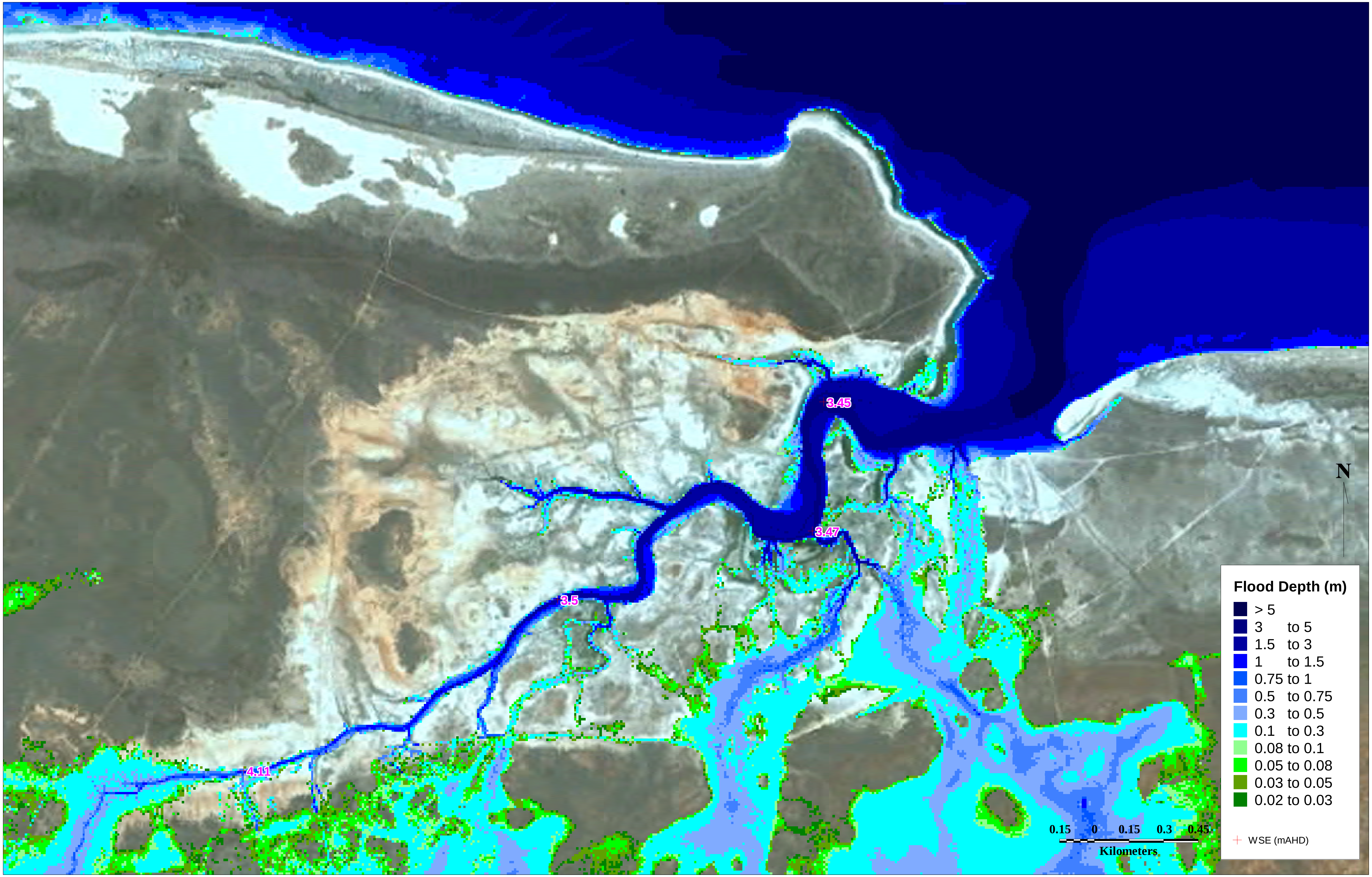
Map S02 - 200 Year ARI Event Flood Depth - Existing Conditions



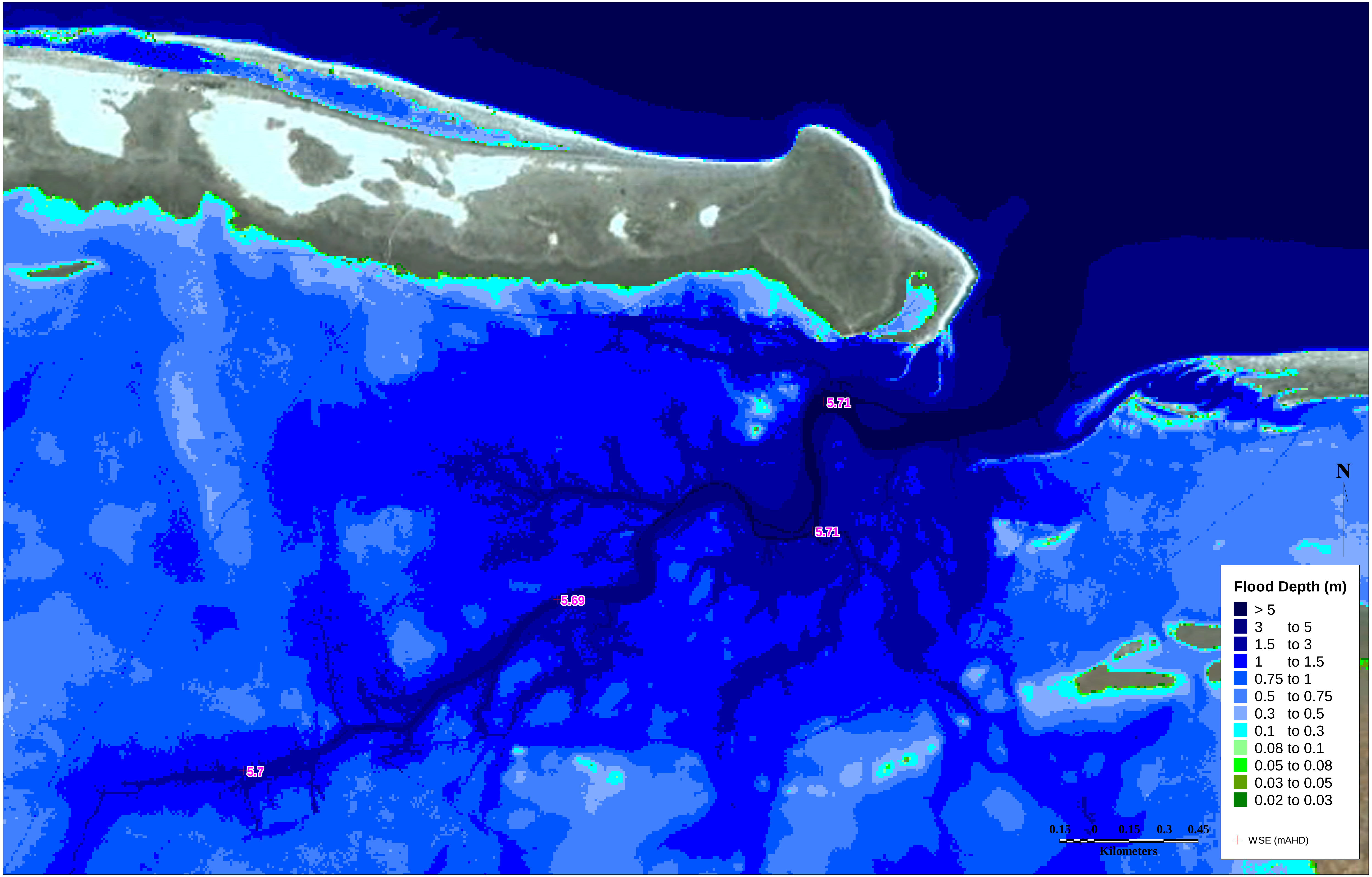
Map S03 - 100 Year ARI Event Flood Depth - Existing Conditions



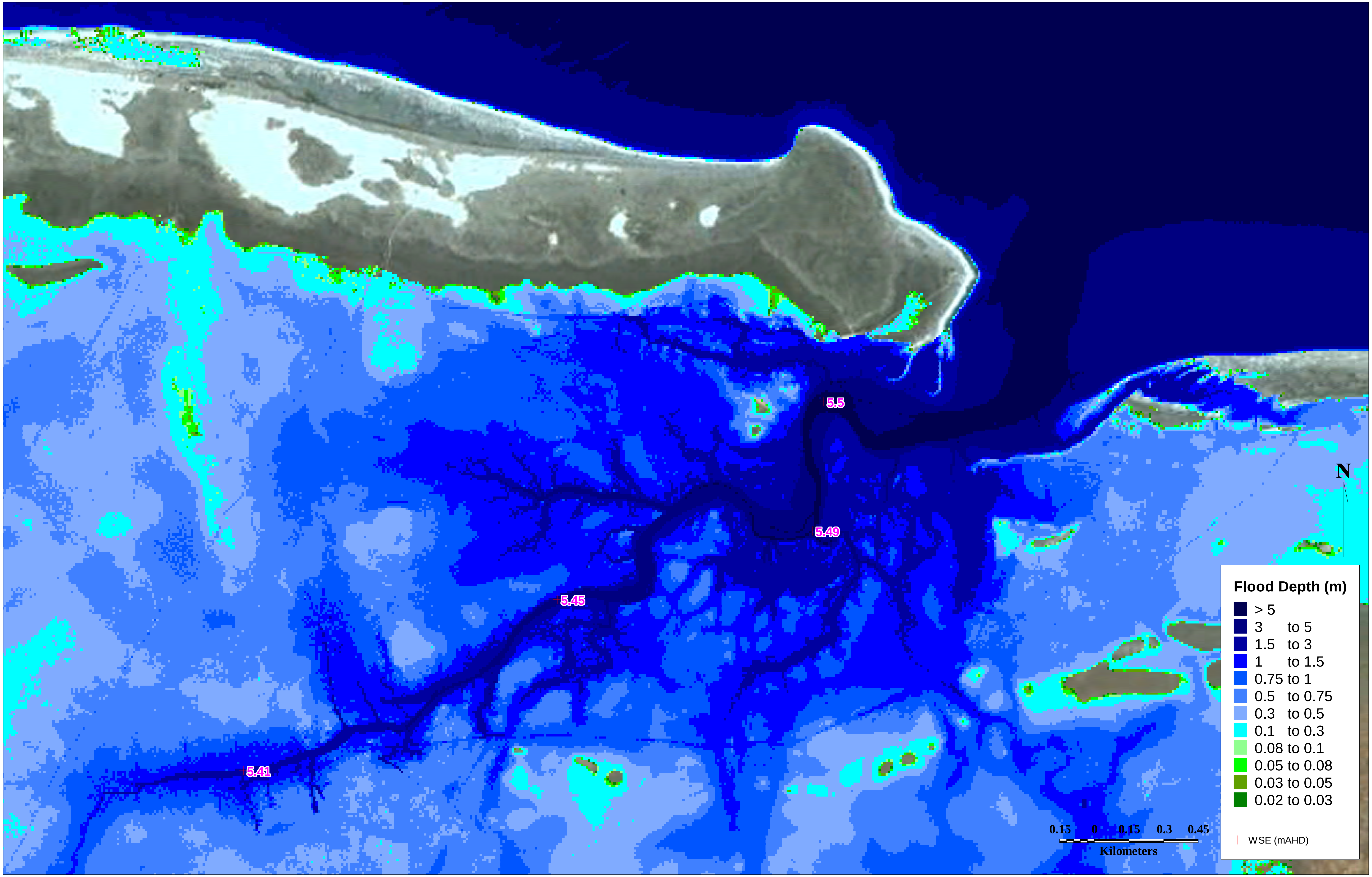
Map S04 - 10 Year ARI Event Flood Depth - Existing Conditions



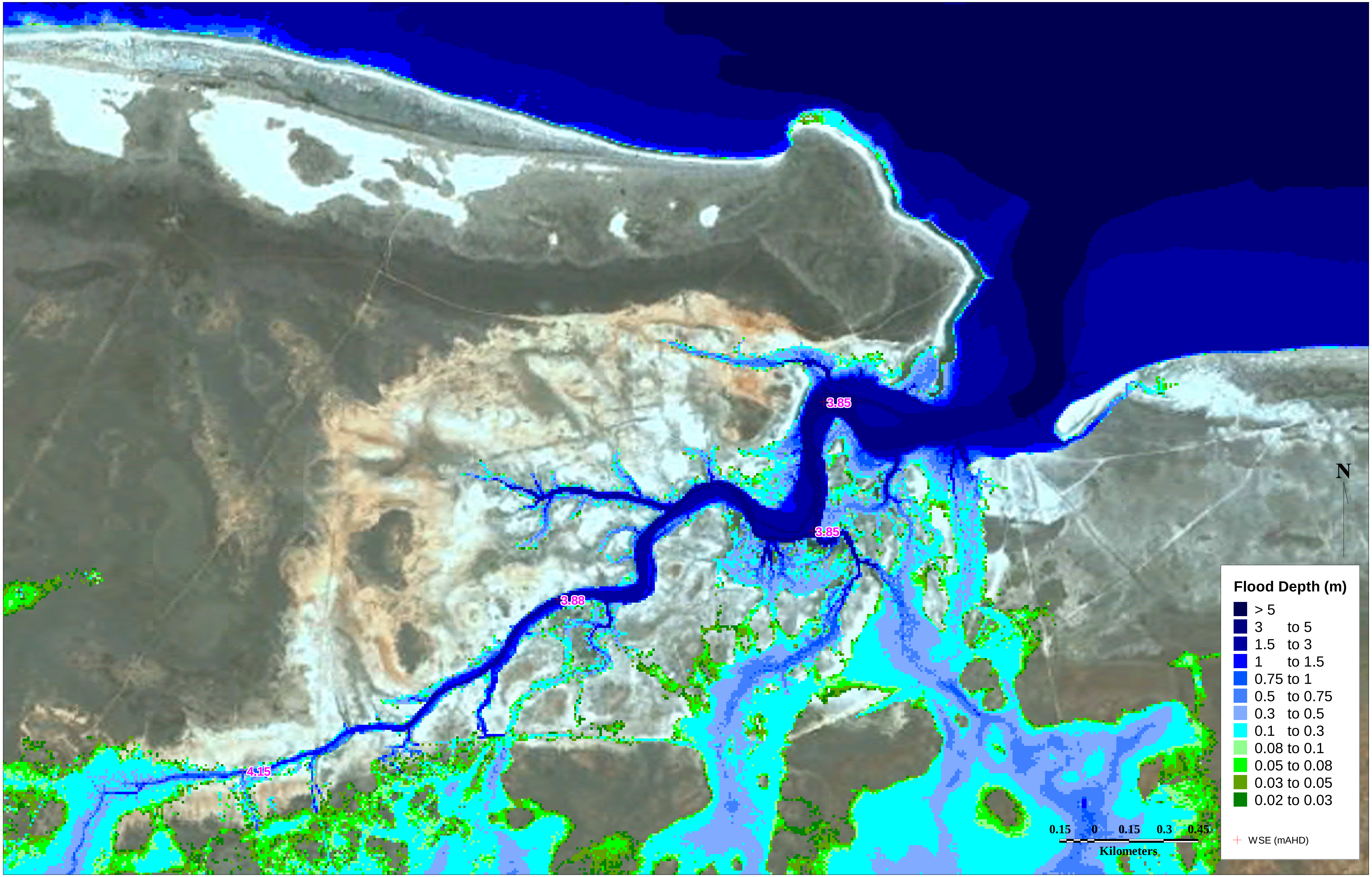
Map S05 - 2 Year ARI Event Flood Depth - Existing Conditions



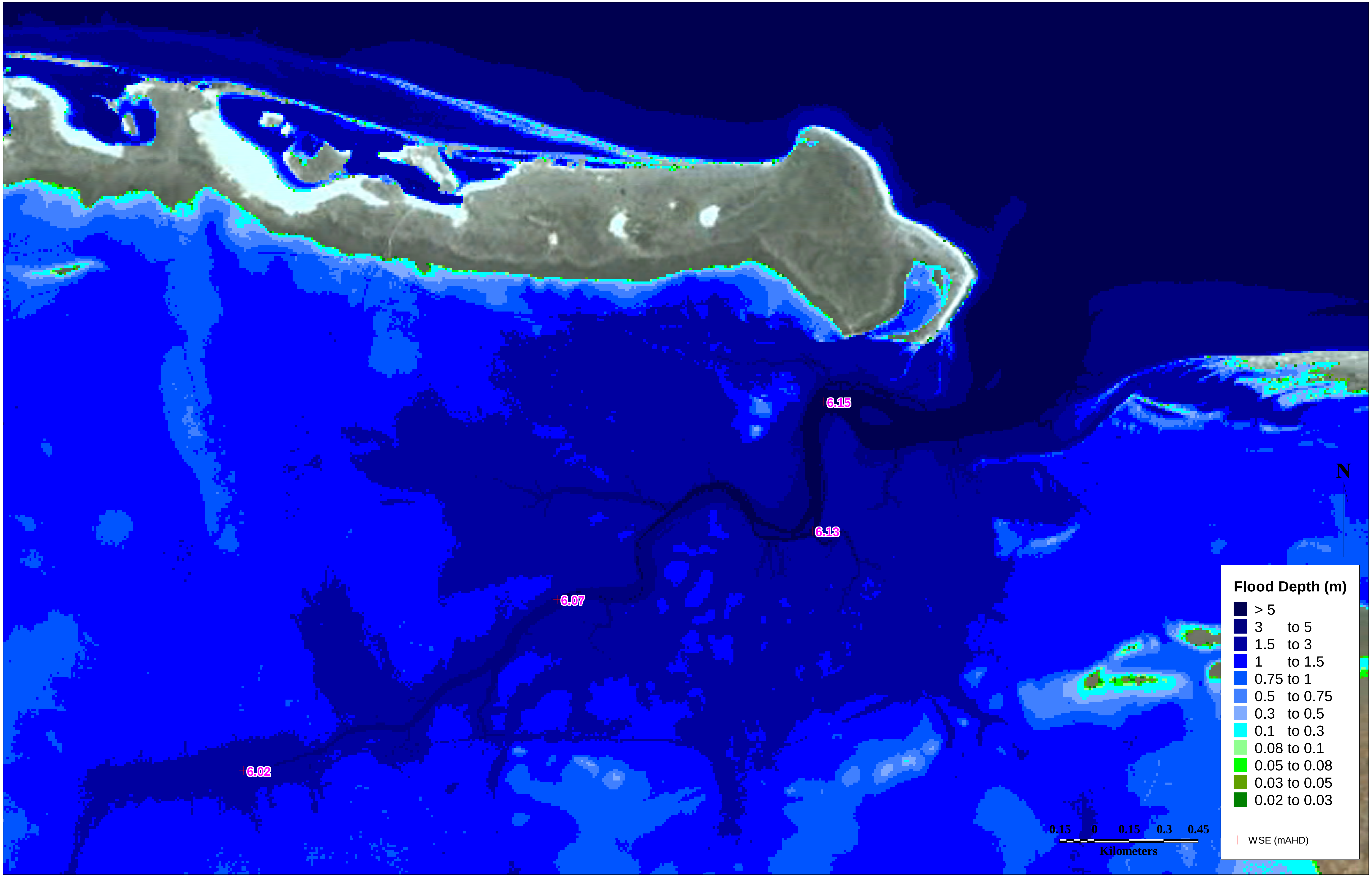
Map S06 - 500 Year ARI Event Flood Depth - 2060 Climate Conditions



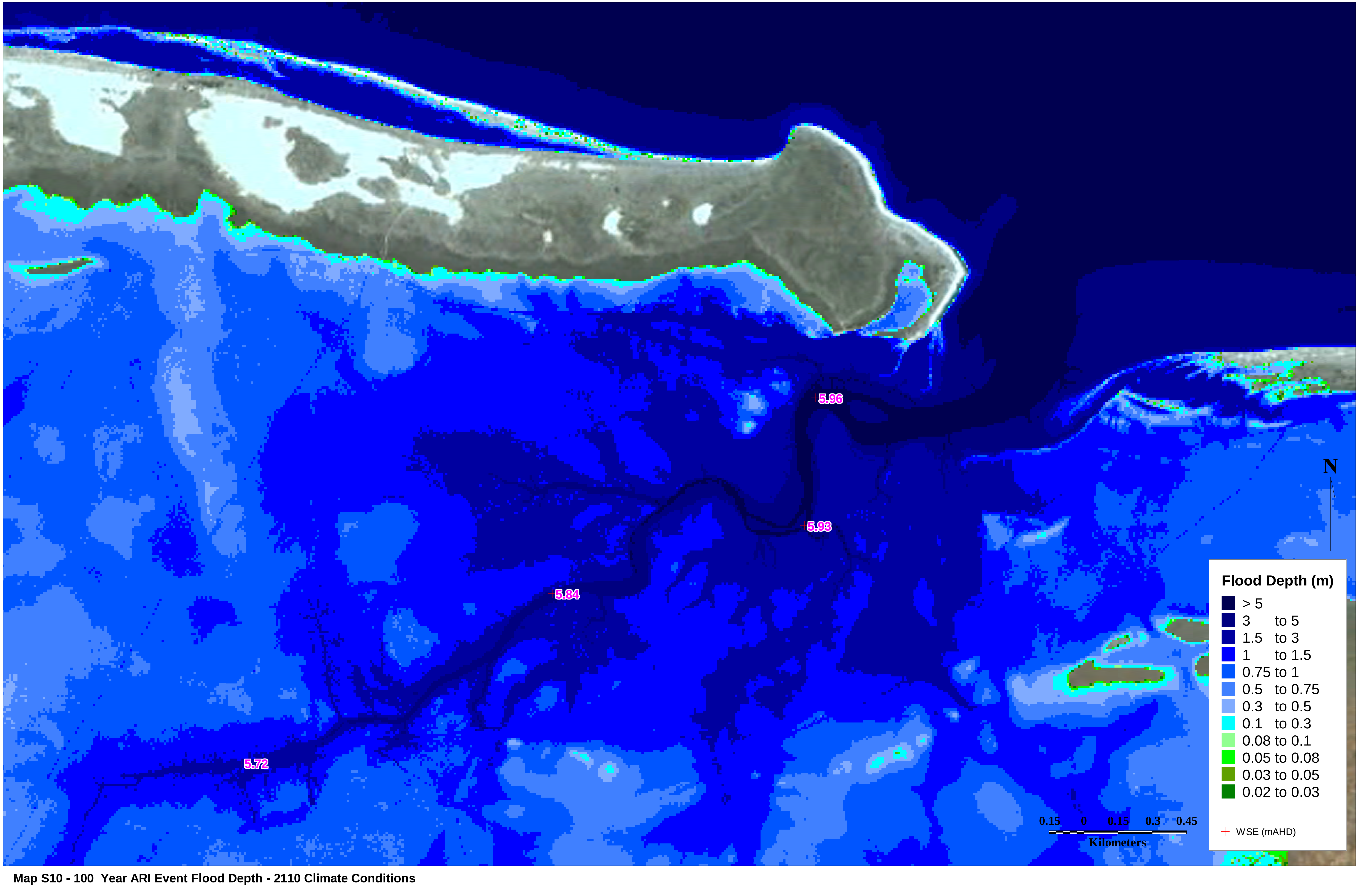
Map S07 - 100 Year ARI Event Flood Depth - 2060 Climate Conditions



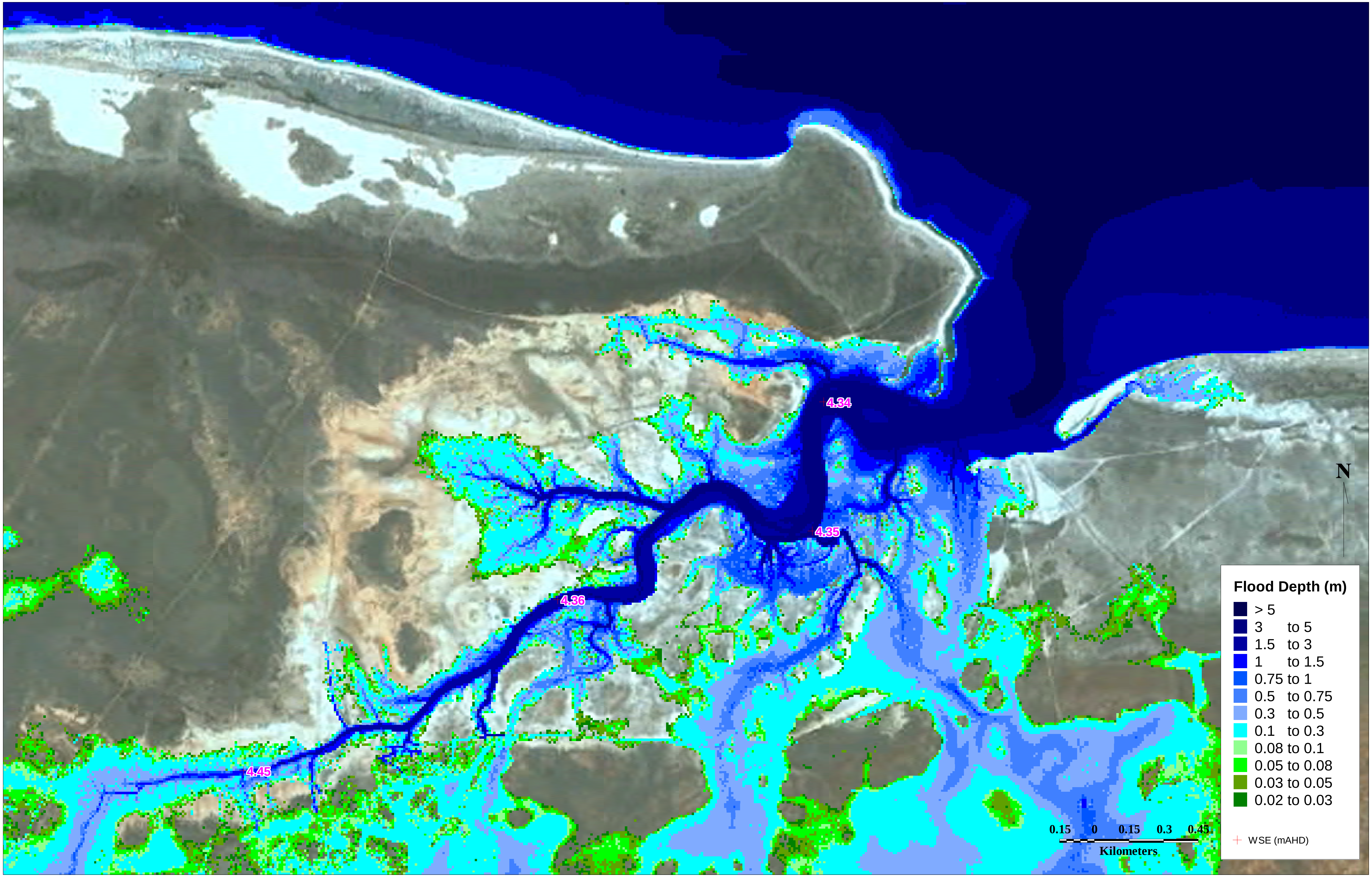
Map S08 - 2 Year ARI Event Flood Depth - 2060 Climate Conditions



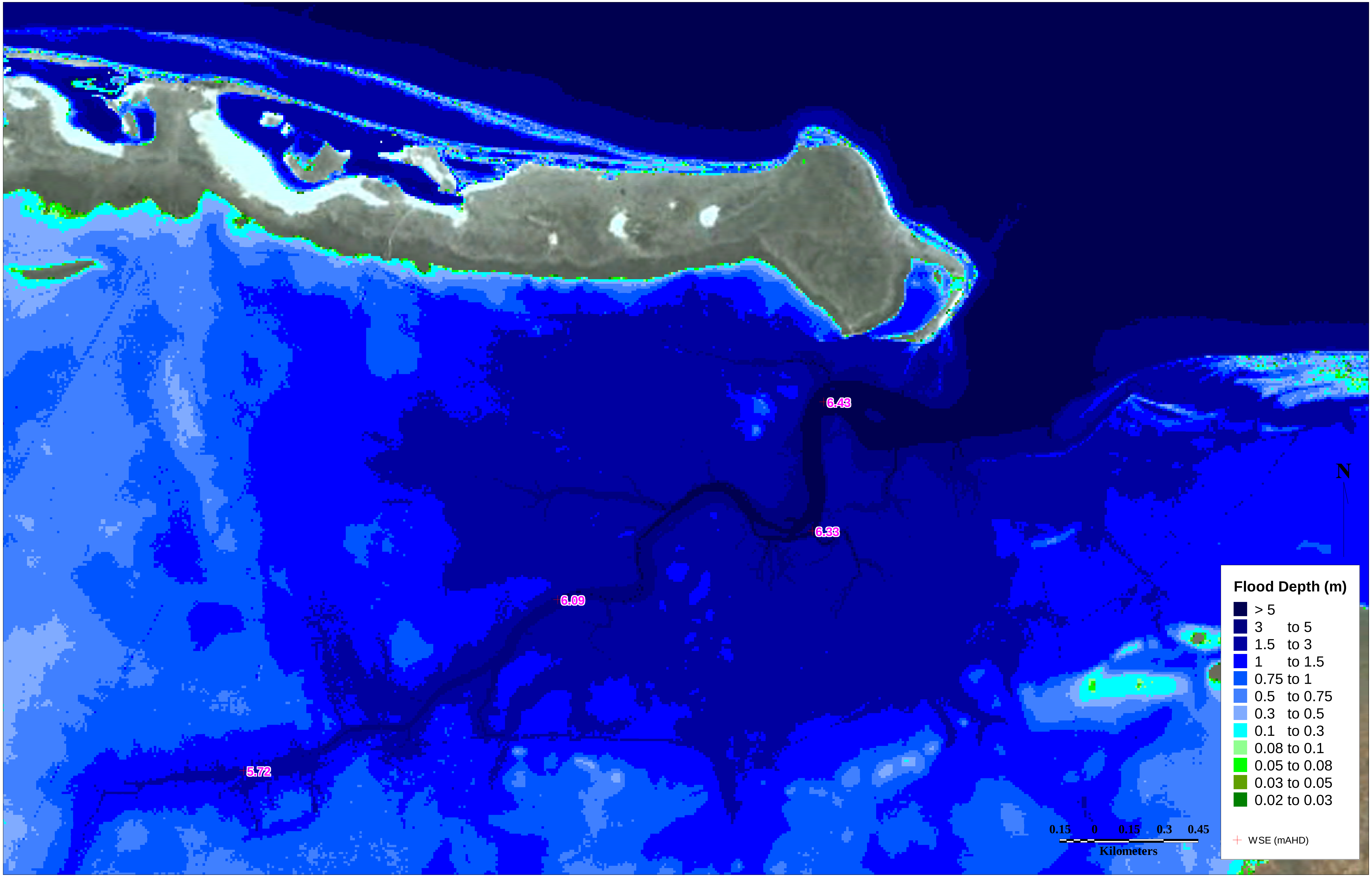
Map S09 - 500 Year ARI Event Flood Depth - 2110 Climate Conditions



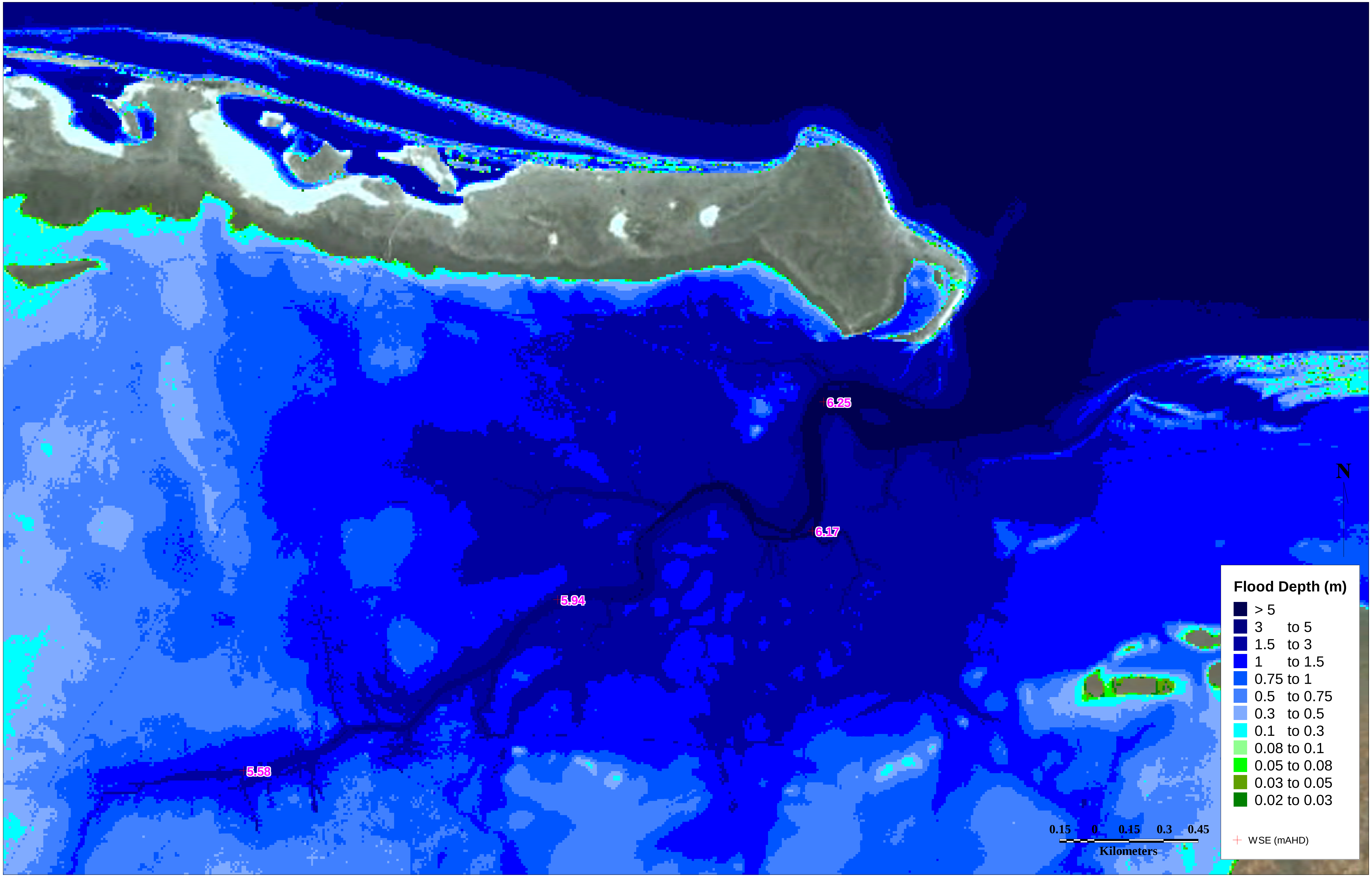
Map S10 - 100 Year ARI Event Flood Depth - 2110 Climate Conditions



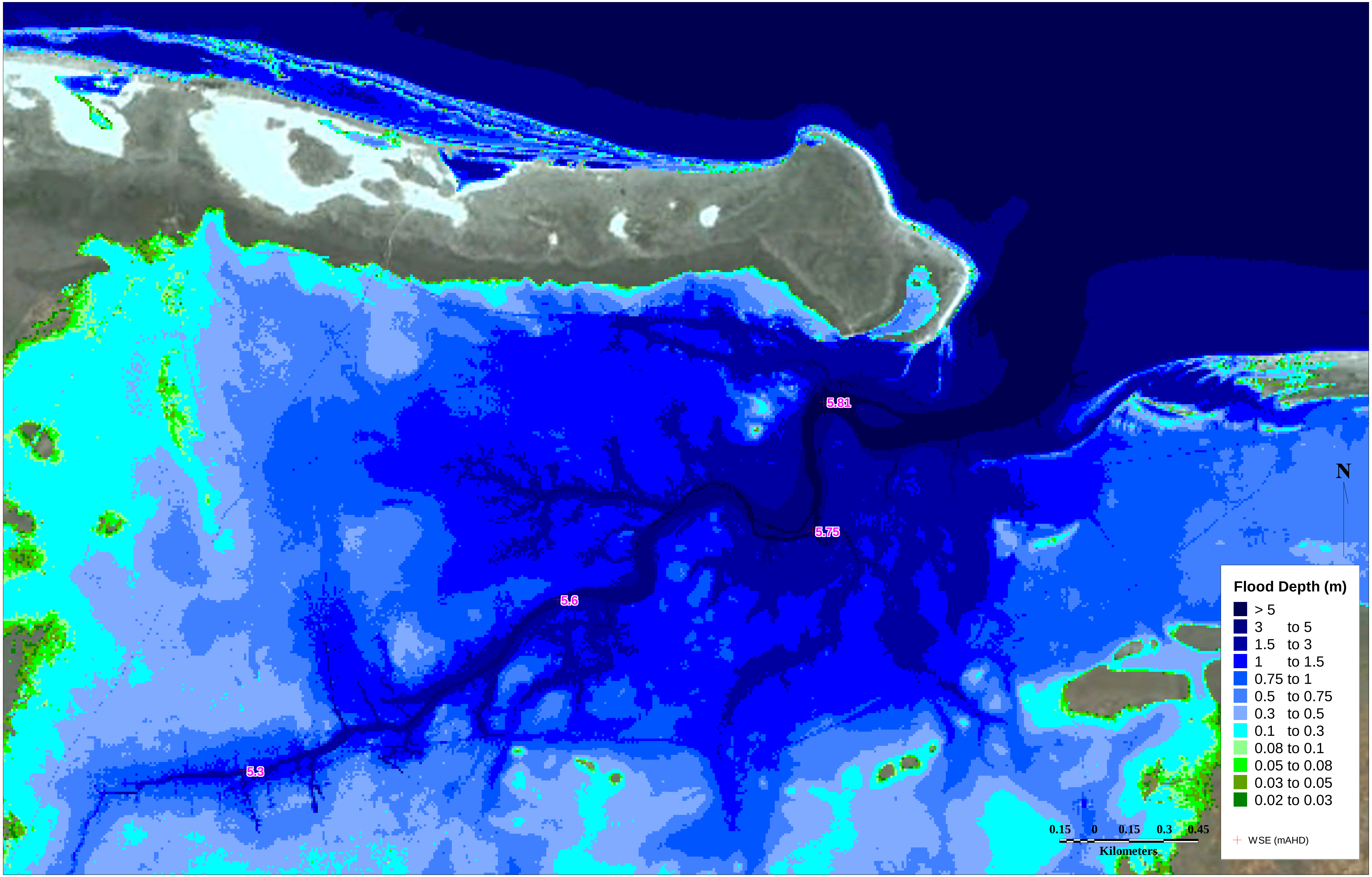
Map S11 - 2 Year ARI Event Flood Depth - 2110 Climate Conditions



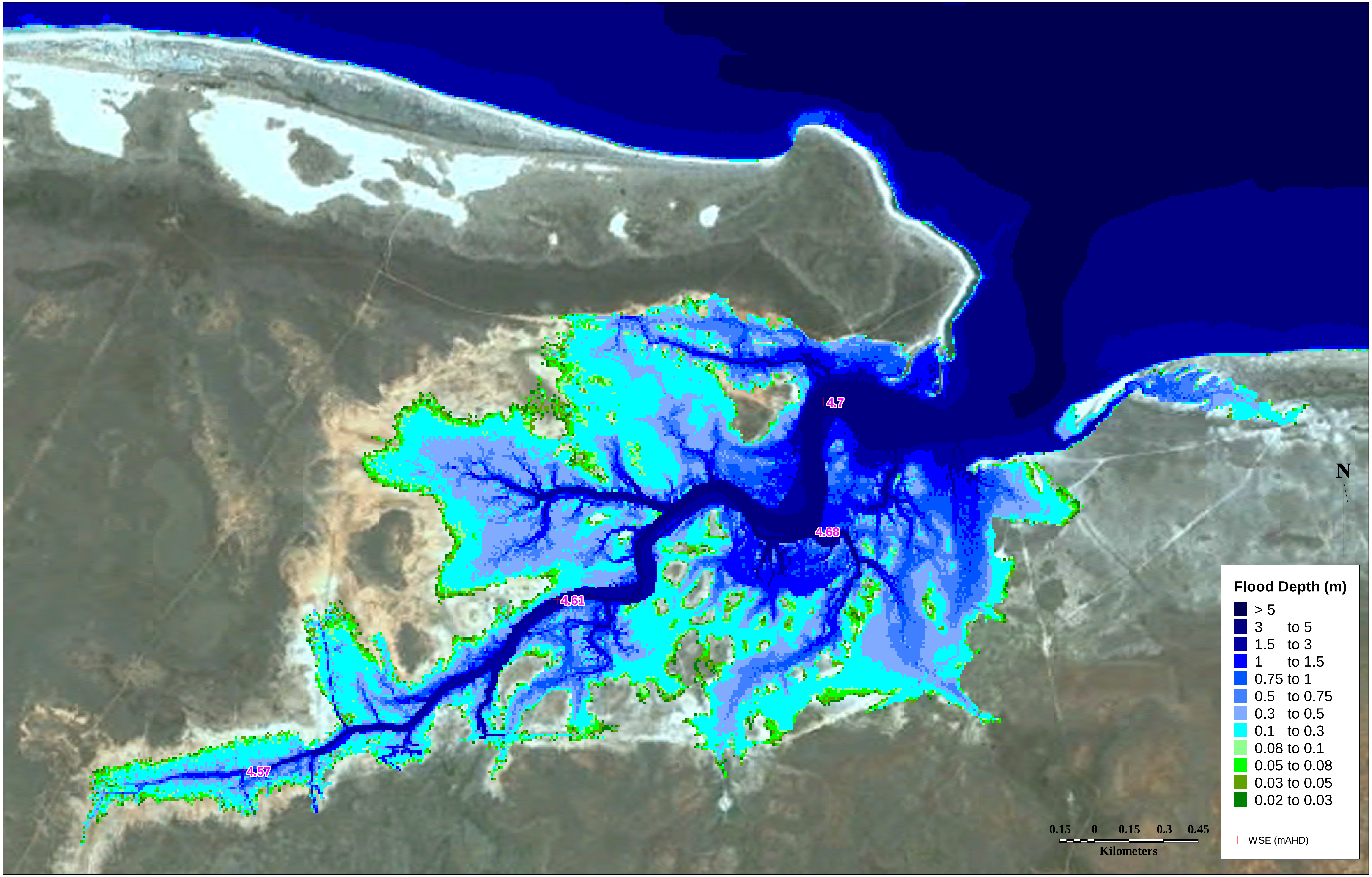
Map S12 - 500 Year Ocean Inundation Flood Depth - Existing Conditions



Map S13 - 200 Year Ocean Inundation Flood Depth - Existing Conditions

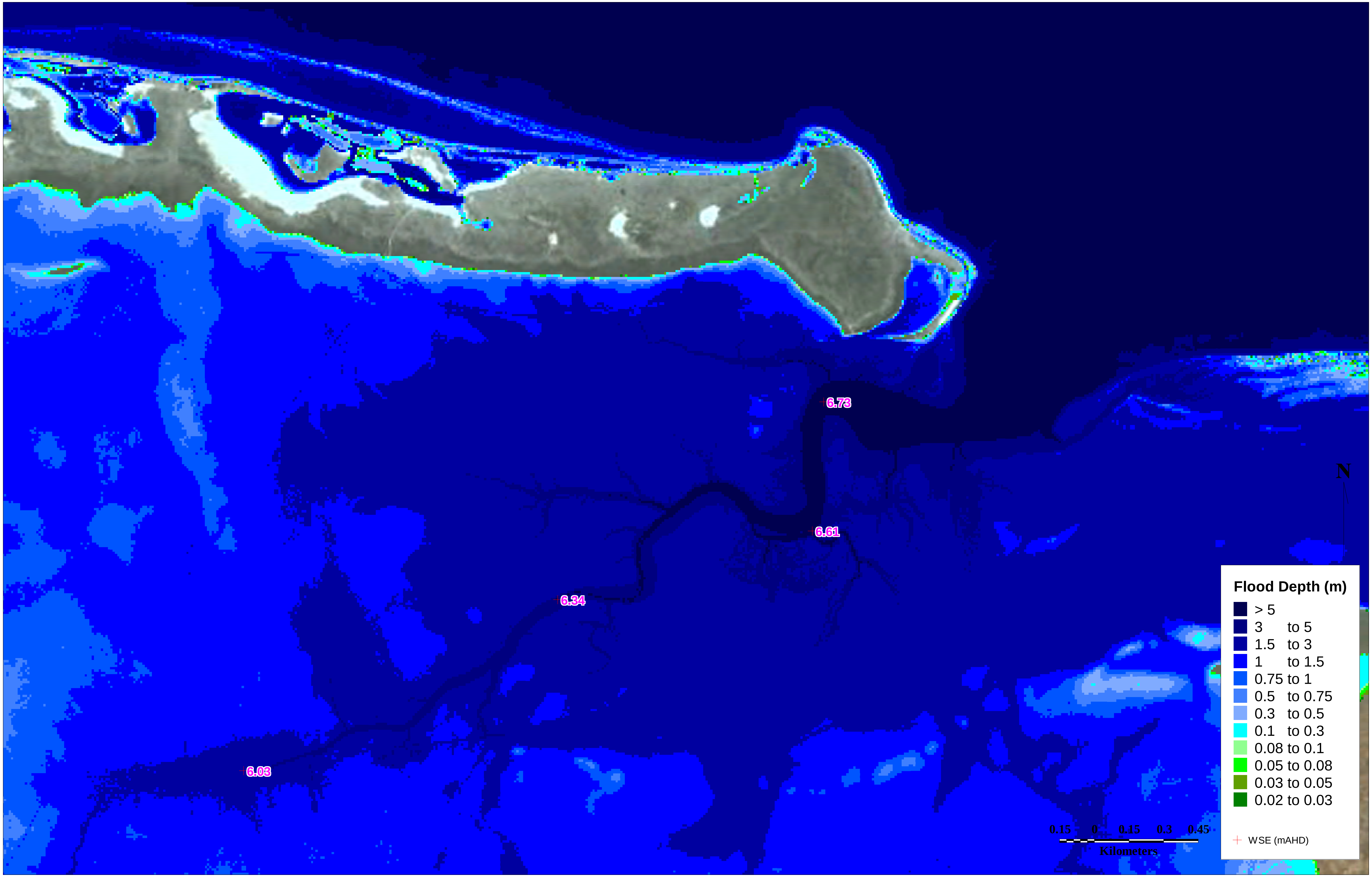


Map S14 - 100 Year Ocean Inundation Flood Depth - Existing Conditions

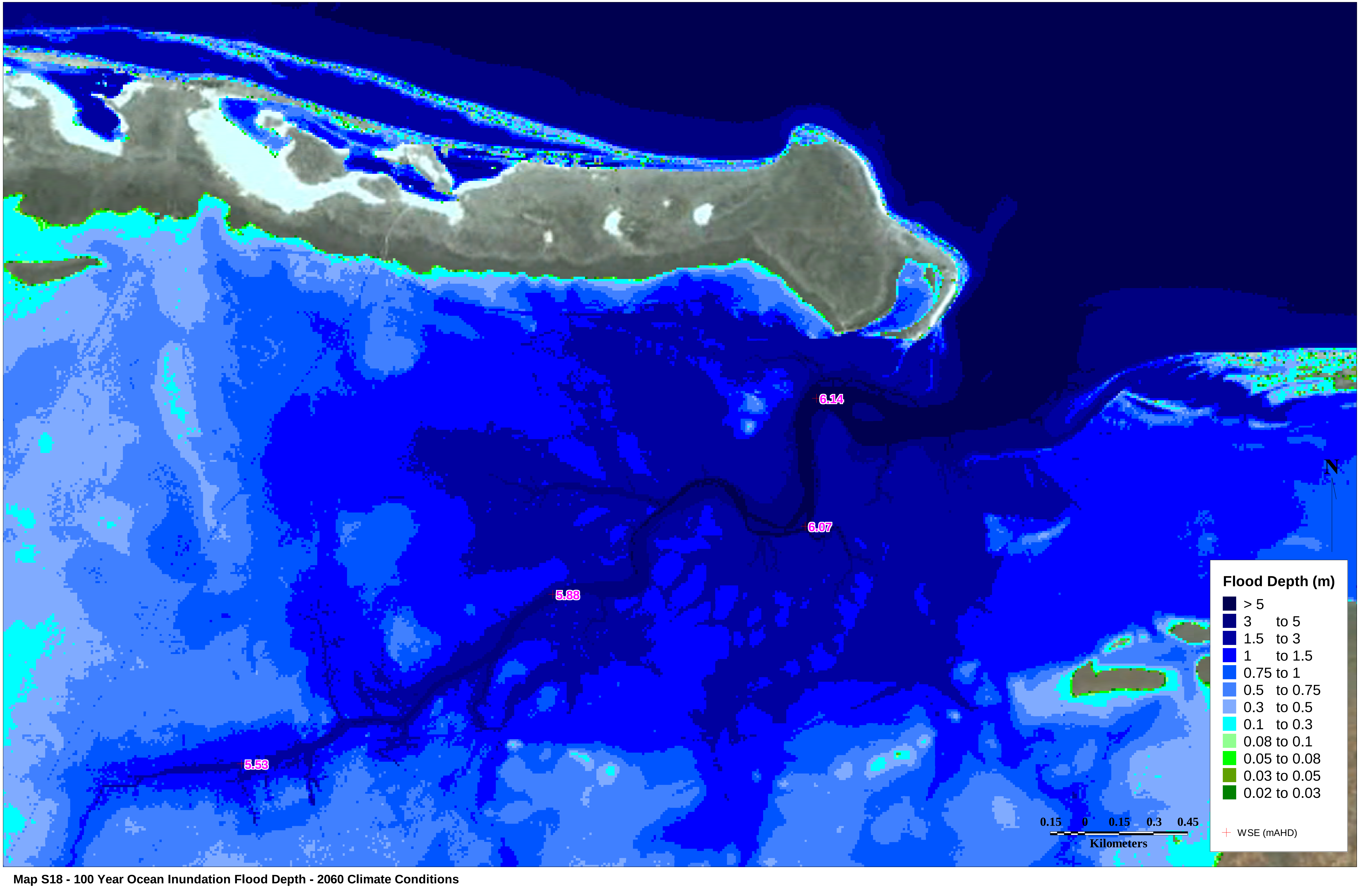


Map S15 - 10 Year Ocean Inundation Flood Depth - Existing Conditions

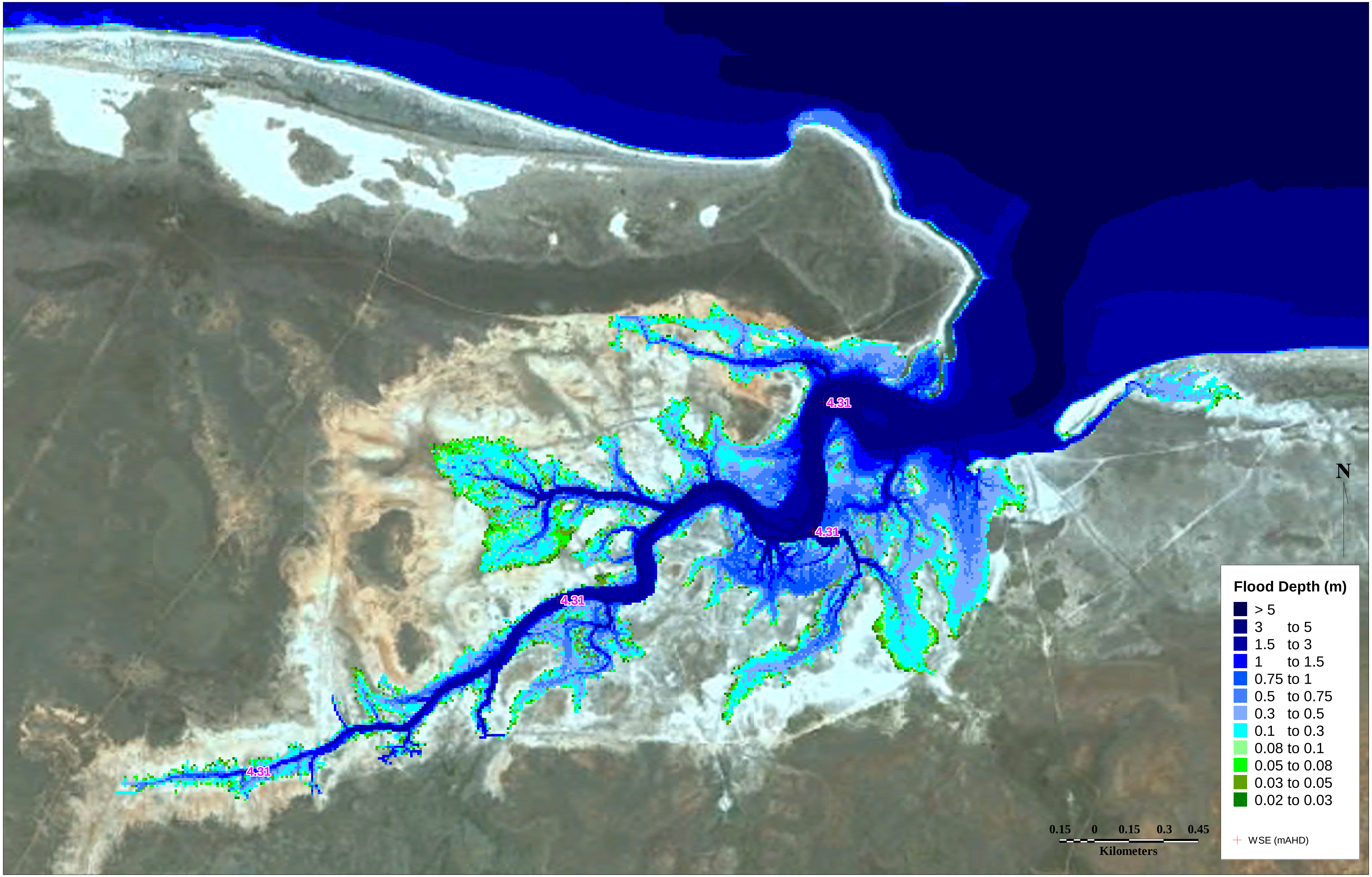
Map S16 - 2 Year Ocean Inundation Flood Depth - Existing Conditions



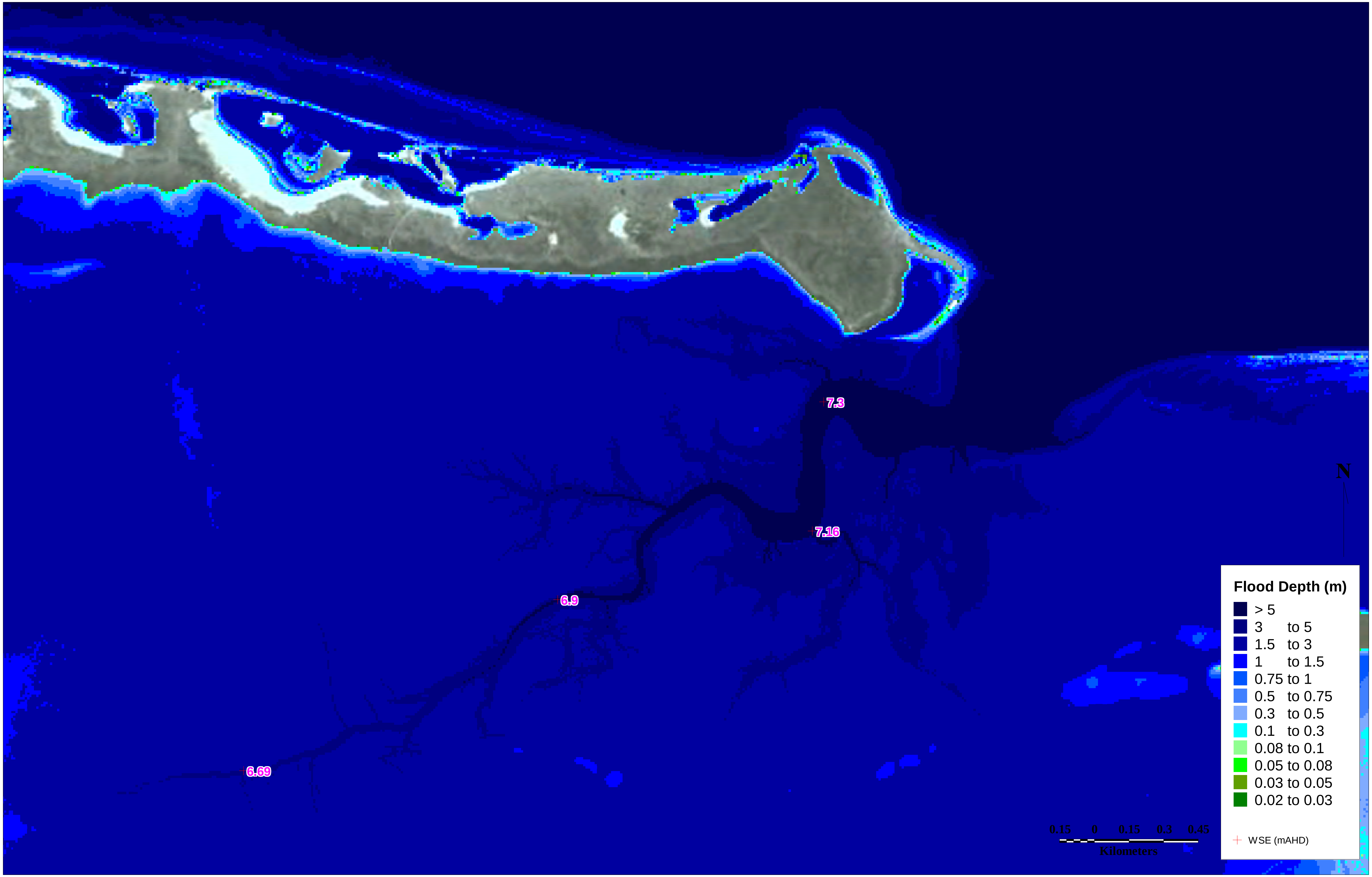
Map S17 - 500 Year Ocean Inundation Flood Depth - 2060 Climate Conditions



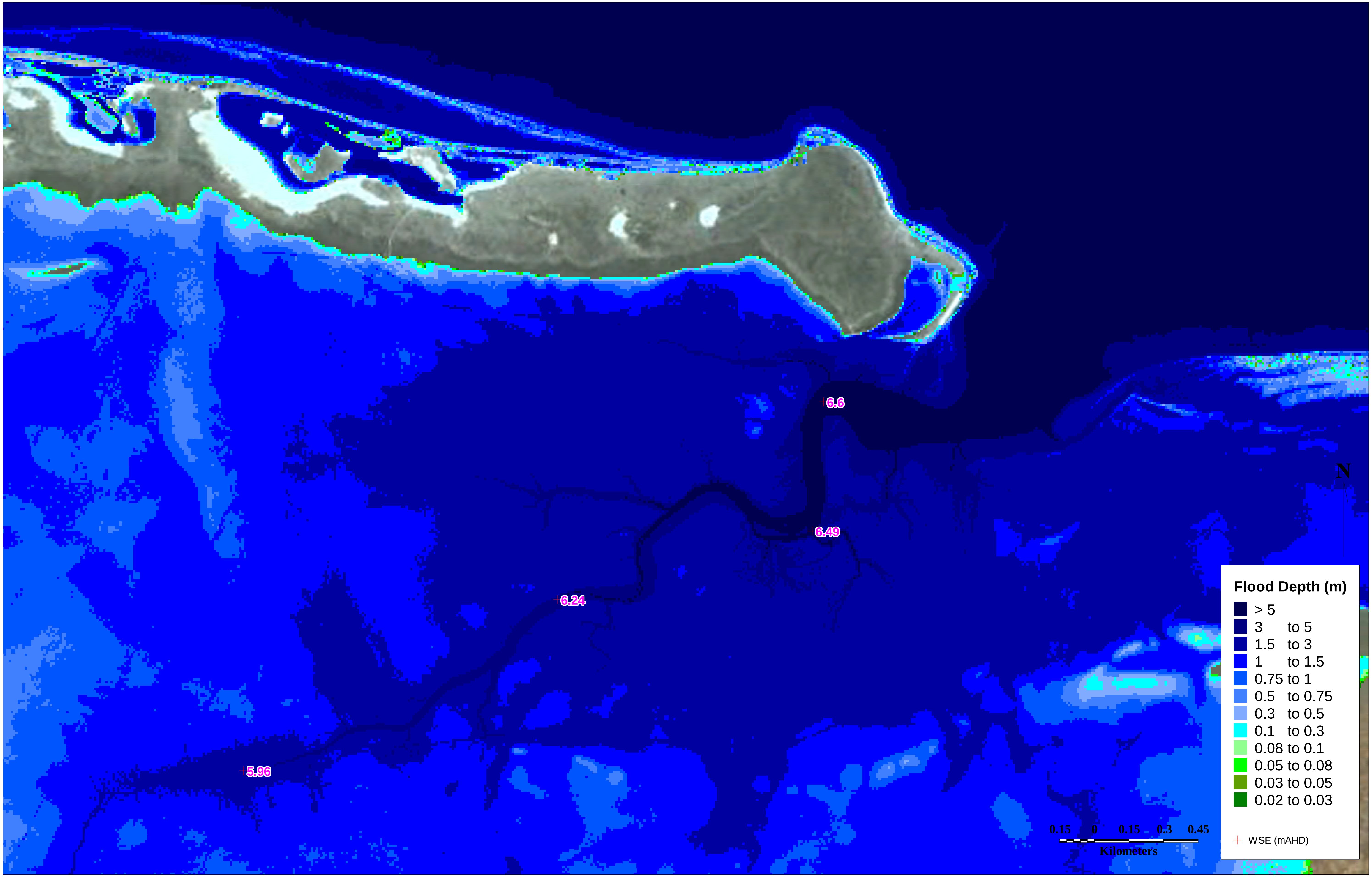
Map S18 - 100 Year Ocean Inundation Flood Depth - 2060 Climate Conditions



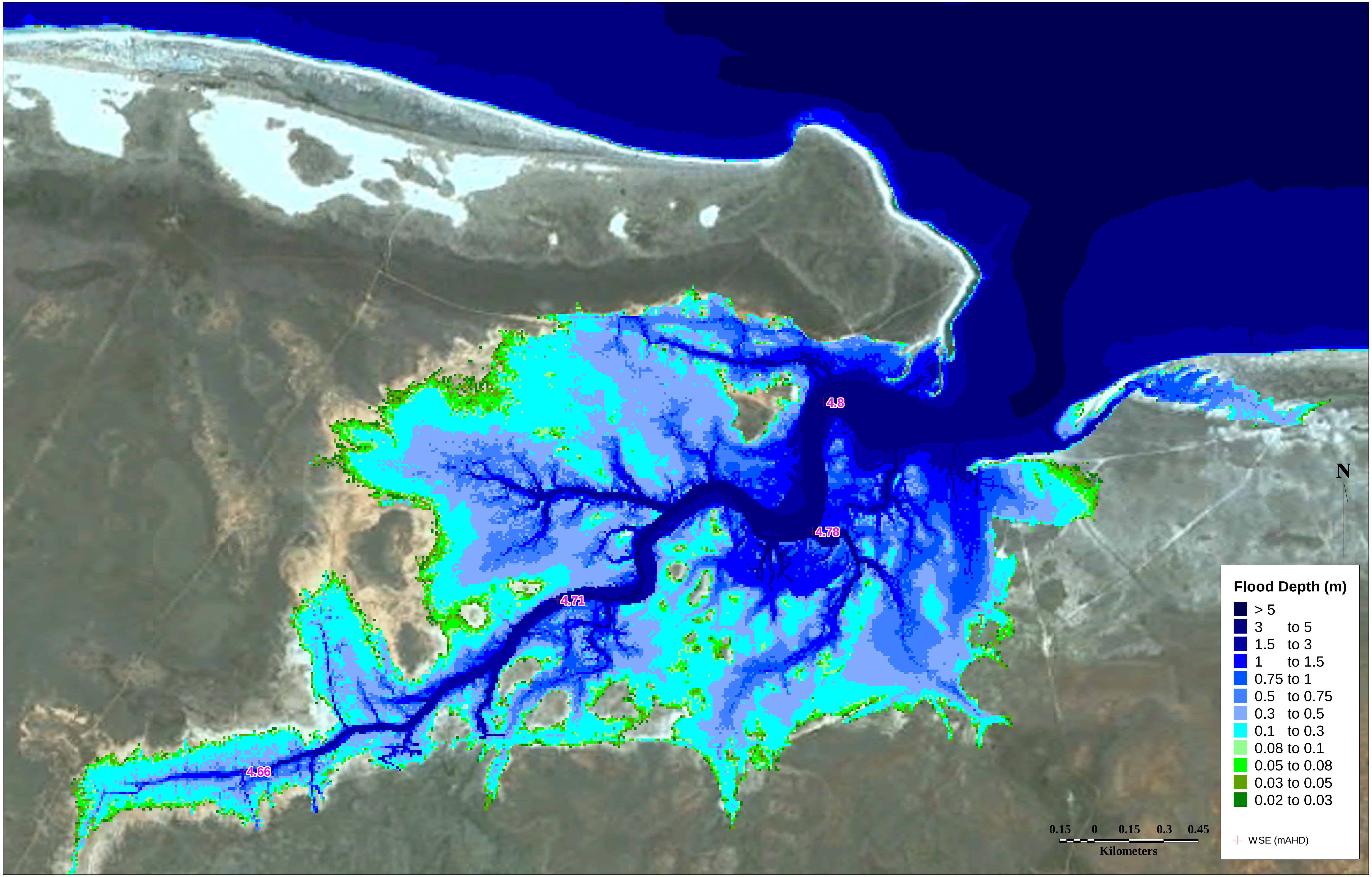
Map S19 - 2 Year Ocean Inundation Flood Depth - 2060 Climate Conditions



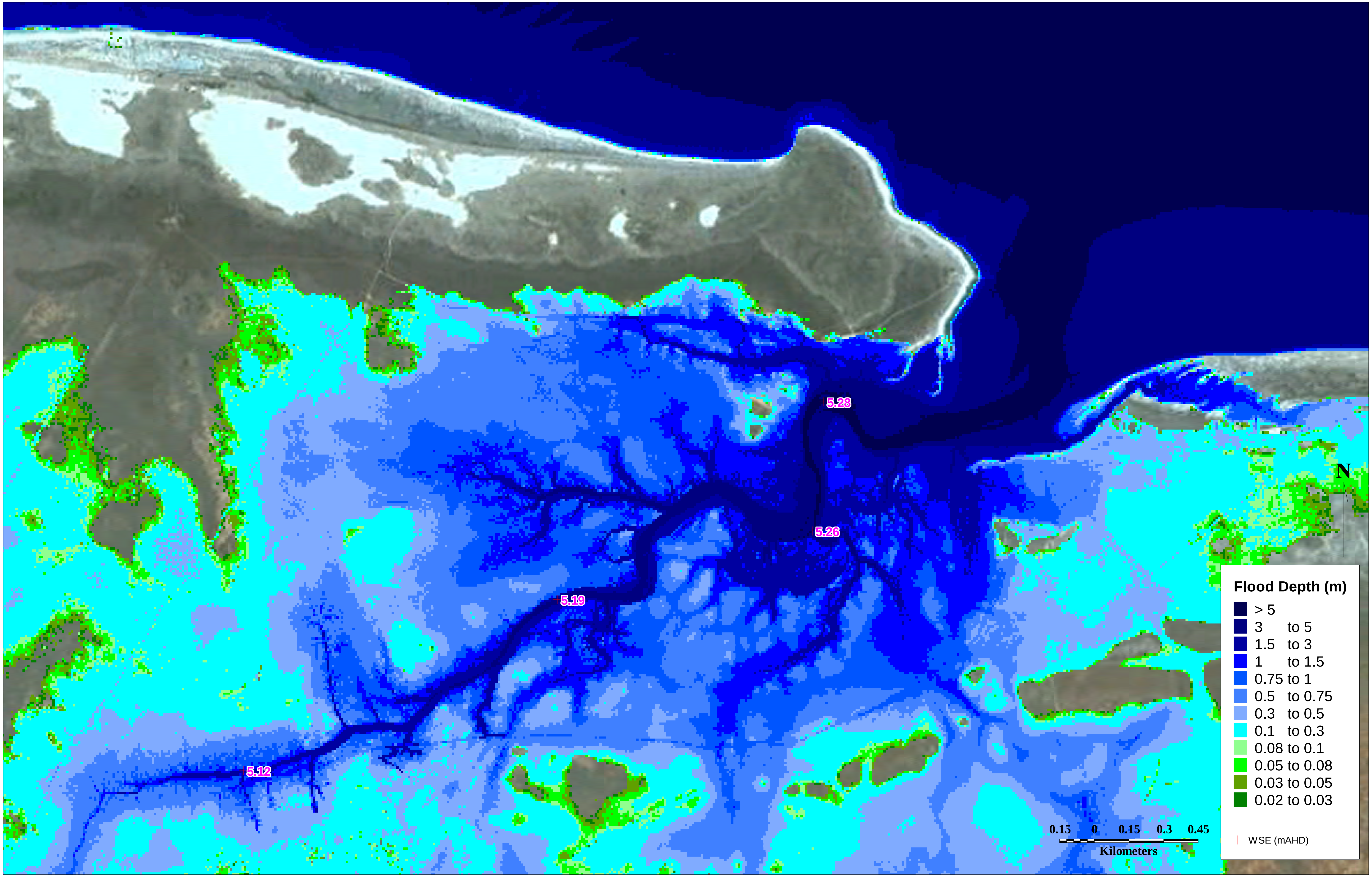
Map S20 - 500 Year Ocean Inundation Flood Depth - 2110 Climate Conditions



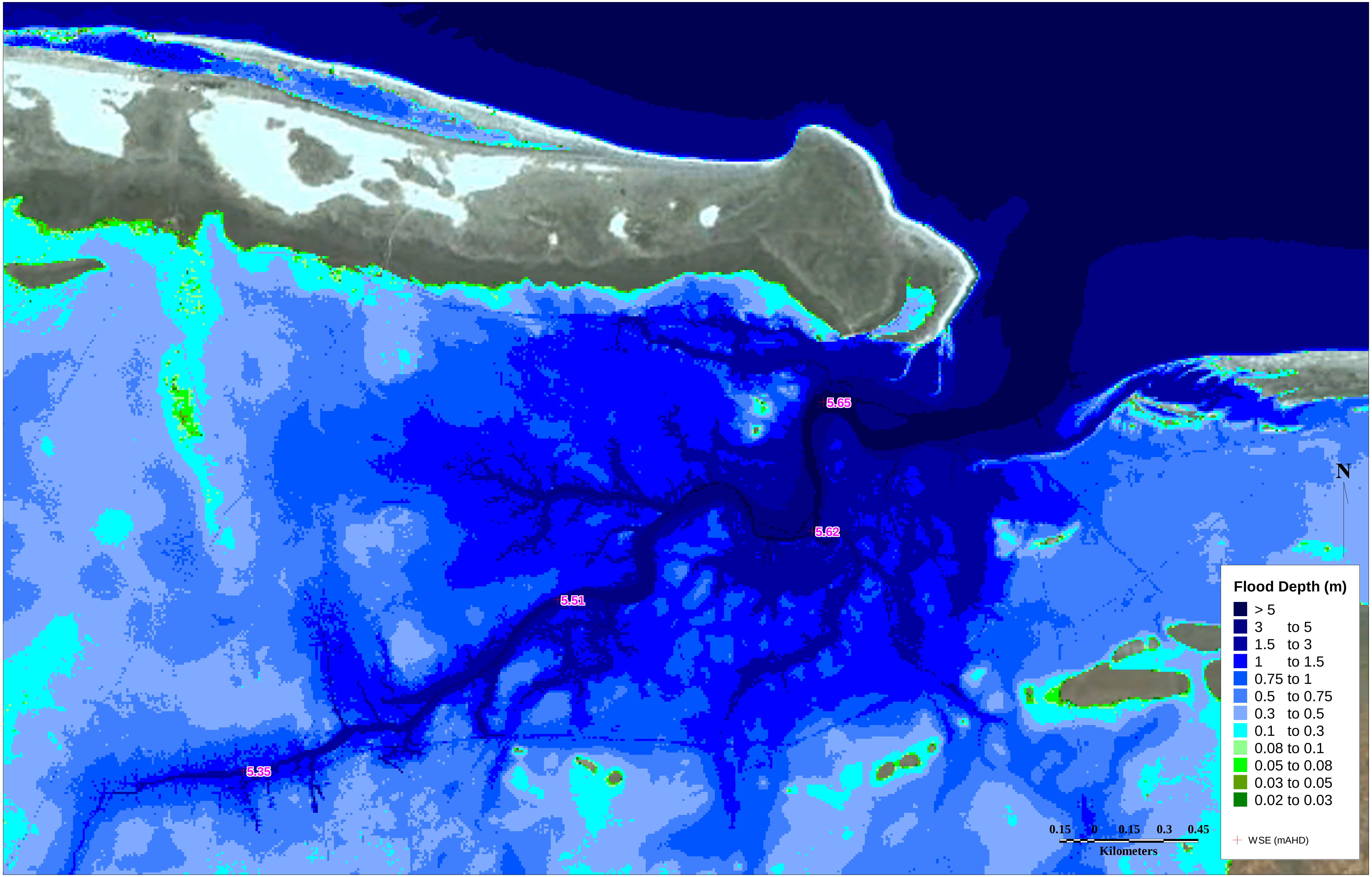
Map S21 - 100 Year Ocean Inundation Flood Depth - 2110 Climate Conditions



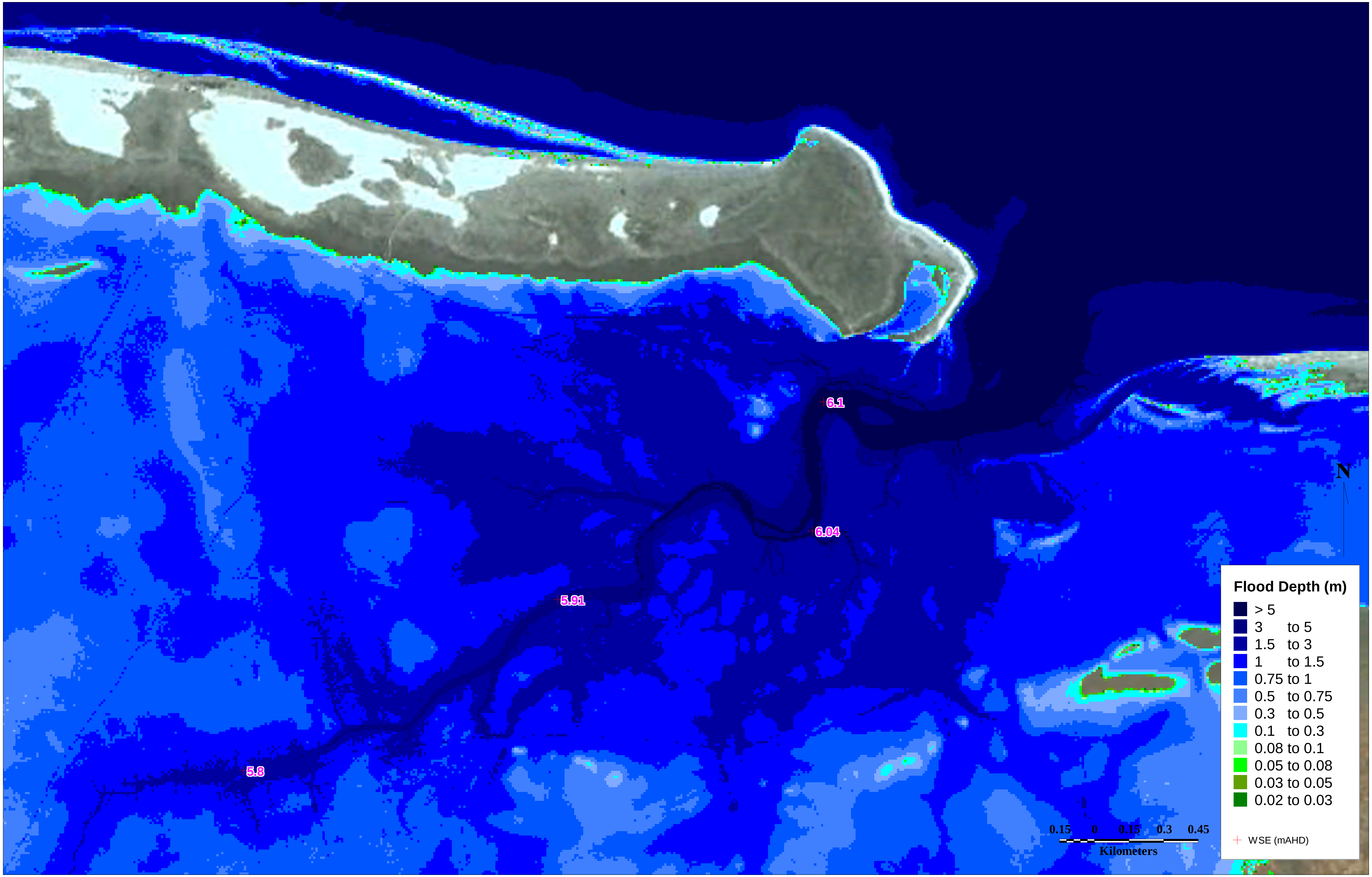
Map S22 - 2 Year Ocean Inundation Flood Depth - 2110 Climate Conditions



Map S23 - 10 Year ARI Event Flood Depth - Existing Conditions - 50 Year Ocean Inundation



Map S24 - 10 Year ARI Event Flood Depth - 2060 Climate Conditions - 50 Year Ocean Inundation



Map S25 - 10 Year ARI Event Flood Depth - 2110 Climate Conditions - 50 Year Ocean Inundation