m p rogers & associates pl ABN 14 062 681 252
creating better coasts and ports

R810 Rev 0

January 2017

City of Greater Geraldton

Cape Burney to Greys Beach Inundation & Coastal Processes Study

marinas
boat harbours
canals
breakwaters
jetties
seawalls
dredging
reclamation
climate change
waves
currents
tides
flood levels
water quality
siltation
erosion
rivers
beaches
estuaries

www.coastsandports.com.au

m p rogers & associates pl
creating better coasts and ports

Suite 1, 128 Main Street, Osborne Park, WA 6017
p: +618 9254 6600
e: admin@coastsandports.com.au
w: www.coastsandports.com.au

K1357, Report R810 Rev 0
Record of Document Revisions

Rev	Purpose of Document	Prepared	Reviewed	Approved	Date
A	Draft for MRA and Client review	J Chen	C Doak	C Doak	15/12/16
0	Issued for Client use	J Chen	C Doak	C Doak	31/01/17

Form 035 18/06/2013

Limitations of this Document

Executive Summary

In order to progress future coastal planning, the City of Greater Geraldton (CGG) require information regarding the potential coastal vulnerability of the shoreline between Cape Burney and Greys Beach. Having recently completed similar investigations for the Point Moore area and the northern Geraldton shoreline from Town Beach to Drummond Cove, specialist coastal and port engineering consultancy M P Rogers & Associates Pty Ltd (MRA) was engaged by CGG to complete an inundation and coastal processes study for the southern Geraldton shoreline. This assessment was completed in accordance with the requirements of the 2013 version of State Planning Policy No. 2.6: State Coastal Planning Policy (SPP2.6).

SPP2.6 provides guidance for decision-making within the coastal zone, including managing development and land use change; establishment of foreshore reserves; and protection, conservation and enhancement of coastal values. Specifically, SPP2.6 provides guidance for calculating the components of a coastal foreshore reserve that are required for a new development to overcome the risks posed by the two main types of coastal hazards (inundation and erosion). Consideration of the potential impacts of coastal hazards needs to allow for landform stability, natural variability and climate change.

It should be noted that whilst SPP2.6 provides a methodology for assessing the foreshore reserve requirements for new development, it is not retrospective. As a result, the outcomes and requirements of an assessment completed in accordance with SPP2.6 are not directly applicable to existing development. Rather, for existing development, the allowances provided through the implementation of a SPP2.6 assessment methodology should be considered to represent an analysis of potential coastal vulnerability. In such cases there is generally a requirement to implement management actions where the coastal hazard risk is considered to be too high, in order to reduce the risk to acceptable levels.

This inundation and coastal processes study has determined potential coastal vulnerability lines and inundation extents for the present day, as well as planning horizons to 2030, 2070 and 2110. Coastal hazard mapping has been completed to present these results and is included within this report.

Coastal Inundation

SPP2.6 requires that an allowance for storm surge inundation be adopted on all coasts. The Policy states that the allowance for inundation should be the maximum extent of inundation calculated as the sum of the storm surge inundation allowance plus the predicted extent of sea level rise. Given the location of Geraldton, the following types of inundation events were considered.

- Cyclonic storm surge inundation.
- Non-cyclonic storm surge inundation.
- Tsunami induced inundation.

For the general case of freehold development, SPP2.6 requires consideration of a 100 year planning horizon. In particular, it requires new and existing development to plan for and manage risk associated with a 500 year average recurrence interval (ARI) storm surge event, which statistically has an 18% chance of occurrence over the 100 year planning horizon. The potential additional impacts of sea level rise over the 100 year planning horizon must also be considered.

The challenge associated with this requirement of the Policy is that accurate and statistically relevant predictions of the 500 year ARI event cannot be made solely using the available historical water level measurements along the West Australian coastline due to the relatively short durations of the records. This is due to the fact that a continual water level record of about a third (167 years) the recurrence interval in question (500 years) is required to ensure statistical relevance of the prediction. Even the longest reliable water level record within Western Australia (Fremantle) is limited to a little over 60 years (records extend further to before 1900 but are not reliable). Therefore, in the absence of sufficient water level data other methodologies must be considered in order to provide meaningful predictions of the 500 year ARI event.

The most widely accepted methodology for the estimation of the 500 year ARI water level event is to use available information on the frequency and characteristics of key meteorological events and, through modelling, generate a long term synthetic database of events and corresponding water levels. Though this process is still only based on a limited period of available data, the modelling seeks to capture the apparent randomness of the critical components of the meteorological effects through simulation of these events over extended periods of time. This methodology is particularly relevant in cyclone regions, where extremely localised effects on water levels can be observed. Modelling an extended time period therefore helps to ensure that the apparent randomness in cyclone track and severity is accounted for in any estimation of events with long recurrence intervals. This process was used for this study, with a 2,000 year synthetic cyclone record being generated and used to determine potential inundation levels associated with cyclone events at Geraldton.

This assessment has adopted the same methodology as that presented in the *Point Moore: Inundation & Coastal Processes Study* (MRA, 2015). The storm surge induced by non-cyclonic storms was assessed using data from the tide gauge at Geraldton Port. The tide gauge data was interrogated to filter out measurements that corresponded with the passage of tropical cyclones. An extreme analysis was then carried out on the filtered water level data to estimate the non-cyclonic inundation levels.

Finally, the potential impact of tsunami events was considered. From review of available information, the 2004 Indian Ocean Tsunami had an ARI of between 700 and 3,000 years yet only resulted in a maximum inundation level of around 1.75 mAHD. This level is well below the present day 500 year ARI storm induced inundation level. Therefore, it is reasonable to provide no additional allowance to absorb the current risk of tsunami induced inundation.

Results of the assessment of potential inundation levels associated with cyclonic, non-cyclonic and tsunami induced events have been combined in the following table. These levels represent the potential inundation levels associated with the given event severity and sea level rise.

Table E1 Recommended Coastal Inundation Allowance – Cape Burney to Greys Beach

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	2.0	2.2	3.0
2030	0.07	2.1	2.3	3.1
2070	0.39	2.4	2.6	3.4
2110	0.90	2.9	3.1	3.9

Coastal Processes Allowances Assessment

SPP2.6 requires that, on a sandy coast, an allowance for future shoreline movement be investigated. This allowance should be measured from the Horizontal Shoreline Datum (HSD) and should include the following individual allowances plus a 0.2 metre per year allowance for uncertainty.

- Allowance for the current risk of storm erosion.
- Allowance for historical shoreline movement trends.
- Allowance for erosion caused by future sea level rise.

Storm erosion modelling was completed using the SBEACH profile change model to simulate the effect of a storm with a 1 in 100 year average recurrence interval (ARI). As a sensitivity analysis the modelling was completed for both a 100 year ARI cyclone event, as well as for the passage of a 100 year ARI south coast storm – typically associated with the passage of winter cold fronts. Modelling both events showed that the south coast storm had the potential to cause more severe shoreline erosion, largely due to the extended duration of the event when compared to the potential impact of a cyclone. As a result of the modelling, erosion allowances of between 12 and 29 m have been provided. The values determined by SBEACH were used as the allowances for S1, as stipulated by the SPP2.6.

Shoreline movement analysis was completed. As a result of this investigation, allowances for future shoreline movement have been determined based on the observed changes in shoreline position over the period between 1942 and 2015. Generally speaking, the majority of the shoreline within the study area has experienced accretion during this time.

Sea level rise allowances for the shoreline were determined based on the application of the requirements outlined in SPP2.6. Allowances of 7, 39 and 90 m have therefore been provided to account for the potential shoreline recession as a result of sea level rise in 2030, 2070 and 2110 respectively.

The total coastal processes allowances for the four key timeframes, plus an allowance for uncertainty of 0.2 m per year, are presented below. These coastal processes allowance lines highlight areas that could be vulnerable to the action of coastal processes over the respective timeframes. These physical coastal processes allowances are to be measured from the horizontal shoreline datum (HSD) and are relative to the shoreline chainages (distance measured along shoreline) presented in Figure E1.

Figure E1 Shoreline Chainages

Table E2 Total Recommended Coastal Processes Allowance – Present Day

Chainage/ Location	Allowance for Storm Erosion (m)	Allowance for Shoreline Movement Trend (m)	Allowance for Sea Level Rise (m)	Allowance for Uncertainty (m)	Total Allowance (m)
0	27	0	0	0	27
800	27	0	0	0	27
1000	12	0	0	0	12
2400	12	0	0	0	12
2600	27	0	0	0	27
3600	27	0	0	0	27
3800	23	0	0	0	23
4400	23	0	0	0	23
4800	23	0	0	0	23
5000	29	0	0	0	29
6200	29	0	0	0	29
6400	28	0	0	0	28
8200	28	0	0	0	28
8400	17	0	0	0	17
9600	17	0	0	0	17
9800	25	0	0	0	25
10200	25	0	0	0	25
10400	28	0	0	0	28
11400	28	0	0	0	28

Table E3 Total Recommended Coastal Processes Allowance – 2030

Chainage/ Location	Allowance for Storm Erosion (m)	Allowance for Shoreline Movement Trend (m)	Allowance for Sea Level Rise (m)	Allowance for Uncertainty (m)	Total Allowance (m)
0	27	0	7	3	37
800	27	0	7	3	37
1000	12	0	7	3	22
2400	12	0	7	3	22
2600	27	2	7	3	39
3600	27	11	7	3	48
3800	23	12	7	3	45
4400	23	12	7	3	45
4800	23	0	7	3	33
5000	29	0	7	3	39
6200	29	0	7	3	39
6400	28	0	7	3	38
8200	28	0	7	3	38
8400	17	0	7	3	27
9600	17	0	7	3	27
9800	25	3	7	3	38
10200	25	3	7	3	38
10400	28	3	7	3	41
11400	28	3	7	3	41

Table E4   Total Recommended Coastal Processes Allowance – 2070

Chainage/ Location	Allowance for Storm Erosion (m)	Allowance for Shoreline Movement Trend (m)	Allowance for Sea Level Rise (m)	Allowance for Uncertainty (m)	Total Allowance (m)
0	27	0	39	11	77
800	27	0	39	11	77
1000	12	0	39	11	62
2400	12	0	39	11	62
2600	27	7	39	11	84
3600	27	38	39	11	115
3800	23	44	39	11	117
4400	23	44	39	11	117
4800	23	0	39	11	73
5000	29	0	39	11	79
6200	29	0	39	11	79
6400	28	0	39	11	78
8200	28	0	39	11	78
8400	17	0	39	11	67
9600	17	0	39	11	67
9800	25	11	39	11	86
10200	25	11	39	11	86
10400	28	11	39	11	89
11400	28	11	39	11	89

Table E5 Total Recommended Coastal Processes Allowance – 2110

Chainage/ Location	Allowance for Storm Erosion (m)	Allowance for Shoreline Movement Trend (m)	Allowance for Sea Level Rise (m)	Allowance for Uncertainty (m)	Total Allowance (m)
0	27	0	90	20	137
800	27	0	90	20	137
1000	12	0	90	20	122
2400	12	0	90	20	122
2600	27	12	90	20	149
3600	27	69	90	20	206
3800	23	80	90	20	213
4400	23	80	90	20	213
4800	23	0	90	20	133
5000	29	0	90	20	139
6200	29	0	90	20	139
6400	28	0	90	20	138
8200	28	0	90	20	138
8400	17	0	90	20	127
9600	17	0	90	20	127
9800	25	20	90	20	155
10200	25	20	90	20	155
10400	28	20	90	20	158
11400	28	20	90	20	158

Table of Contents

Table of Figures

Table of Tables

1. Introduction

1.1 General

In order to progress future coastal planning, the City of Greater Geraldton (the City) require information regarding the potential coastal vulnerability of the shoreline between Cape Burney and Greys Beach.

To satisfy this requirement, the City commissioned specialist coastal and port engineers M P Rogers & Associates Pty Ltd (MRA) to conduct an inundation and coastal processes study in accordance with the requirements of State Planning Policy No. 2.6: State Coastal Planning Policy (SPP2.6) (WAPC 2013). This assessment was completed for the entire coastline within the study area, as shown in Figure 1.1.

It should be noted that whilst SPP2.6 provides a methodology for assessing the foreshore reserve requirements for new development, it is not retrospective. As a result, the outcomes and requirements of an assessment completed in accordance with SPP2.6 are not directly applicable to existing development. Rather, for existing development, the allowances provided through the implementation of a SPP2.6 assessment methodology should be considered to represent an analysis of potential coastal vulnerability. In such cases there is generally a requirement to implement management actions where the coastal hazard risk is considered to be too high, in order to reduce the risk to acceptable levels.

MRA have previously completed coastal processes and inundation studies for the City covering Point Moore and the northern Geraldton shoreline from Town Beach to Drummond Cove. As part of these studies, a calibrated and validated inundation model was setup to provide inundation data along the entire Geraldton coastline. Results from the inundation model have also been used for the assessment of inundation between Cape Burney and Greys Beach within this study. Methodologies and results for the Point Moore and Town Beach to Drummond Cove studies were presented in MRA report R656 entitled *Point Moore Inundation & Coastal Processes Study* and MRA report R675 entitled *Town Beach to Drummond Cove Inundation & Coastal Processes Study*.

Figure 1.1 Location Diagram

2. Site Setting

2.1 Site Inspection

A coastal engineer from MRA carried out a site inspection in August 2016. During the site inspection it was noted that the shoreline is generally consistent with that described by Short (2006).

The locations of the relevant features in the study area are shown in Figure 2.1.

Figure 2.1 Beach Locations

The southern extent of the study area consists of the Southgate Dunes, which originates from sand blowing north from the northern Greenough River mouth beach. The active Southgate dunes extend 3.5 km to the north. The southern stretch of this shoreline extends from the Greenough River in the south to the westernmost tip of the dune field. This section of shoreline generally consists of a strip of high tide sand fringed by continuous 50 to 100 m wide intertidal calcarenite platforms. Higher waves break over the Southgate reefs, with low waves at the shore. This stretch of shoreline is generally backed by a vegetated foredune (Short 2006). Photographs of the southern shoreline of the Southgate Dunes are presented in Figure 2.2.

Figure 2.2 Photography showing Southern Shoreline of Southgate Dunes

North of the western tip of the Southgate Dunes is a shoreline that extends 1 km north to a small rocky headland. This section of shoreline is topographically controlled by rock platforms and inshore reefs and backed by unvegetated dunes (Short 2006). Further north of the rocky headland the shoreline generally consists of a flat continuous sandy beach that is backed by unvegetated dunes. The northern shoreline of the Southgate Dunes generally experiences low waves at the shore as a result of the protection provided by the Southgate reef.

Photographs of the northern shoreline of the Southgate Dunes are presented in Figure 2.3.

Figure 2.3 Photography showing Northern Shoreline of Southgate Dunes

The shoreline north of the Southgate Dunes is known as Tarcoola Beach and extends approximately 4 km north-northwest from the Southgate Dunes (Short 2006). This stretch of beach is exposed to southerly ocean swells on the more open northern section, which generally consists of a narrow beach backed by a steep vegetated dune.

The southern section of the Tarcoola Beach is more protected and experiences lower waves due to the presence of the Southgate Reef. This section of the shoreline generally consists of a flat wide beach backed by a low vegetated foredune.

Photographs showing the shoreline of Tarcoola Beach are presented in Figure 2.4.

Figure 2.4 Photography showing Southern Tarcoola (Left) & Northern Tarcoola Beach (Right)

North of Tarcoola Beach a crescent shaped shoreline has formed in the lee of the shore attached reef at Separation Point. A significant change of aspect occurs over this section of beach, with the shoreline adjacent to Separation Point having a southern aspect. This stretch of shoreline is known as the Back Beach (Short 2006). The beach alignment and the presence of nearshore reefs at Separation Point offer some protection to this stretch of shoreline, which is generally backed by a wide low foredune.

Photographs showing this section of the shoreline are presented in Figure 2.5.

Figure 2.5 Photography showing Back Beach

The northern extent of the study area consists of a 1.5 km section of shoreline known as Greys Beach (Short 2006), which is located west of Separation Point. Greys Beach faces squarely into the dominant southerly waves and winds, though these wave heights are reduced by the presence of offshore reef. A wide low tide terrace is usually maintained, but may be cut during periods of higher waves. This stretch of shoreline is generally backed by a high scarped foredune.

Photographs showing the Greys Beach shoreline are presented in Figure 2.6.

Figure 2.6 Photography showing Greys Beach

2.2 Sediment Cells

The amended SPP2.6 (WAPC, 2013) makes reference to coastal sediment cells and notes that coastal process assessments should consider the entire sediment cell.

In 2014, Stul et al completed an assessment of the coastal sediment cells between Moore River and Glenfield Beach. Within this study, Stul et al defined sediment cells as “sections of the coast within which sediment transport processes are strongly related” and proposed that these cells could provide a platform for the review and management of coastal processes over varying time and spatial scales.

It should be noted that a sediment cell is different to an administrative boundary for a beach, as a sediment cell may contain more than one beach, and that a beach can span over multiple sediment cells (refer to Figure 2.7).

A sediment cell hierarchy was established that comprised primary, secondary and tertiary level cells. Characteristics of each cell level, as defined by Stul et al, are described below.

- Primary cells – related to large landforms and considers trends of potential change in large landform assemblages or land systems over longer coastal management timescales.
- Secondary cells – describes contemporary sediment movement on the shoreface and potential inter-decadal landform response.
- Tertiary cells – confined to the reworking and movement of sediment in the nearshore and potential seasonal to inter-annual responses.

The adopted cell hierarchy can therefore be used to provide regional scale context to district and local level assessments.

The extent of the sediment cells defined by Stul et al along the shoreline of the site are shown in Figure 2.7. This figure shows that the study shoreline is located within secondary sediment cell 14. There is one tertiary sediment cell point (Separation Point) in the study area.

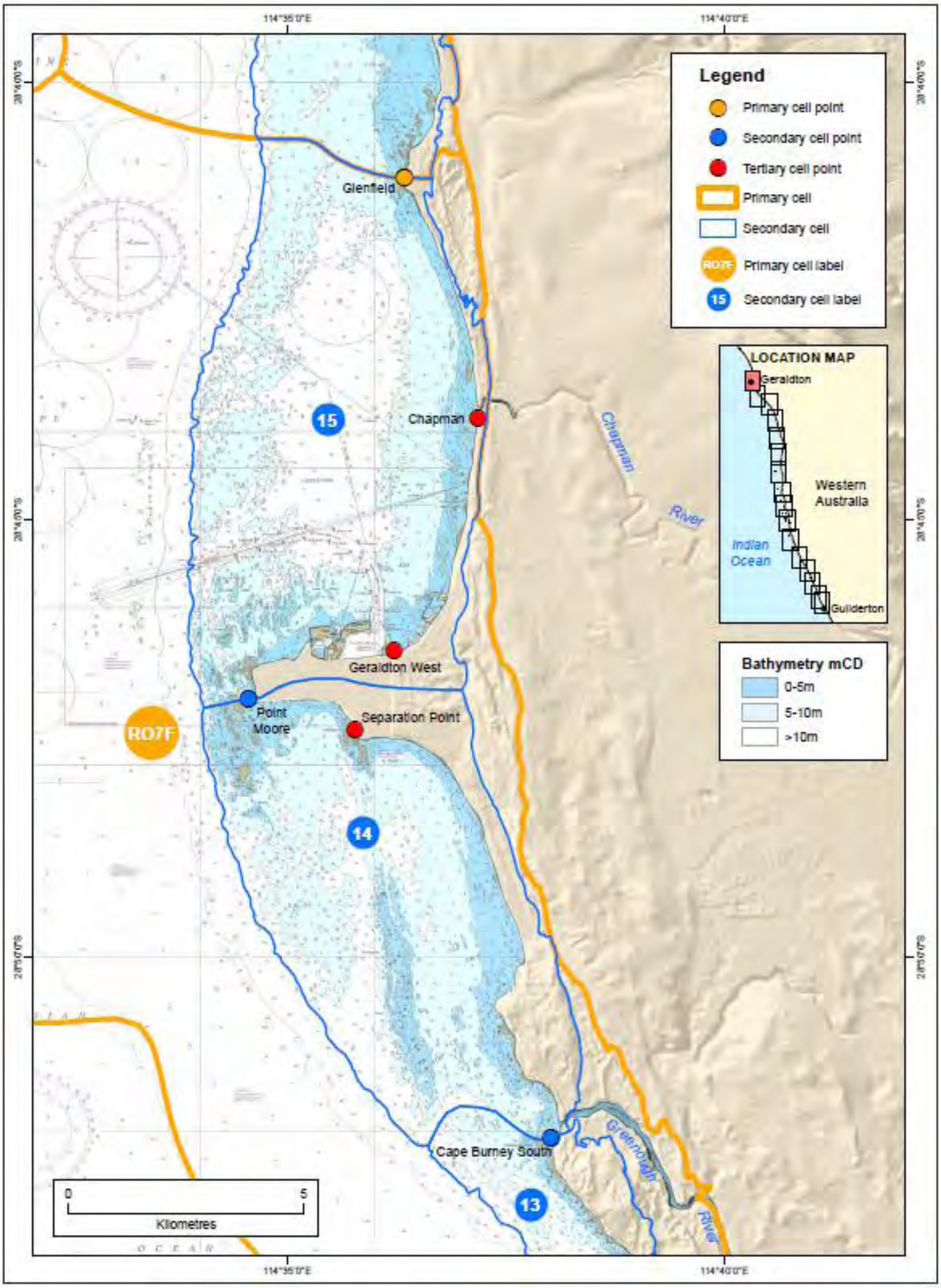


Figure 2.7 Secondary Sediment Cells near Geraldton (source: Stul et al 2014)

3. State Planning Policy 2.6

SPP2.6 provides guidance for decision-making within the coastal zone including managing development and land use change; establishment of foreshore reserves; and protection, conservation and enhancement of coastal values. Specifically, SPP2.6 provides guidance for calculating the components of a coastal foreshore reserve required to overcome the risks posed by the two main types of coastal hazards (inundation and erosion). Consideration of the potential impacts of coastal hazards needs to allow for landform stability, natural variability and climate change. The assessment of these allowances are discussed briefly in the following sections.

3.1 Coastal Inundation Assessment

SPP2.6 requires that an allowance for storm surge inundation be adopted on all coasts when considering proposed new development or risks posed to existing development.. The Policy states that the allowance for inundation should be the maximum extent of inundation calculated as the sum of the storm surge inundation allowance plus the predicted extent of sea level rise. As a result, the following were assessed in order to determine the appropriate allowances for coastal inundation.

- Cyclonic storm surge inundation (Section 4.1).
- Non-cyclonic storm surge inundation (Section 4.2).
- Tsunami induced inundation (Section 4.3).

For this study the potential extent of inundation has been determined for 20, 100 and 500 year average recurrence interval (ARI) events for timeframes to 2030, 2070 and 2110.

3.2 Coastal Processes Allowances Assessment

SPP2.6 requires that an allowance for future shoreline erosion be provided for new development or when considering the risk to existing development. This allowance should be measured from the Horizontal Shoreline Datum (HSD) and should include the following individual allowances plus a 0.2 metre per year allowance for uncertainty.

- Allowance for the current risk of storm erosion (Section 5.1).
- Allowance for historical shoreline movement trends (Section 5.2).
- Allowance for erosion caused by future sea level rise (Section 5.3).

The required allowances have been assessed in the sections referenced above.

3.3 Sea Level Rise

The Intergovernmental Panel on Climate Change (IPCC) has presented various scenarios of possible climate change and the resultant sea level rise in the coming century. The range of these projections is shown in Figure 3.1 (IPCC, 2013).

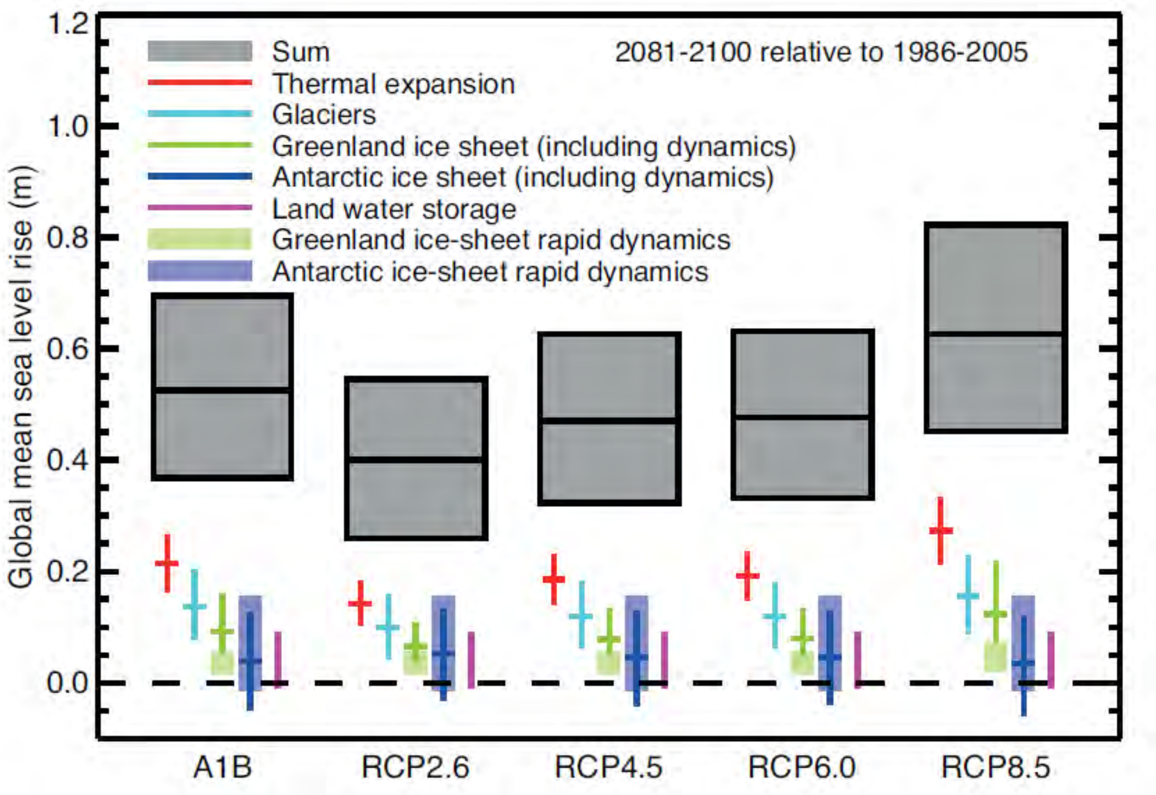


Figure 3.1 IPCC Scenarios for sea level rise (IPCC 2013)

Department of Transport (2010) completed an assessment of the potential increase in sea level that could be experienced on the Western Australian coast in the coming 100 years. This assessment extrapolated work by Hunter (2009) to provide sea level rise values based on the IPCC (2007) A1F1 climate change scenario projections to the year 2110. The derived sea level rise scenario was subsequently adopted by the Western Australian Planning Commission (and SPP2.6) for use in coastal planning along the Western Australian coast. The adopted sea level rise scenario is presented in Figure 3.2.

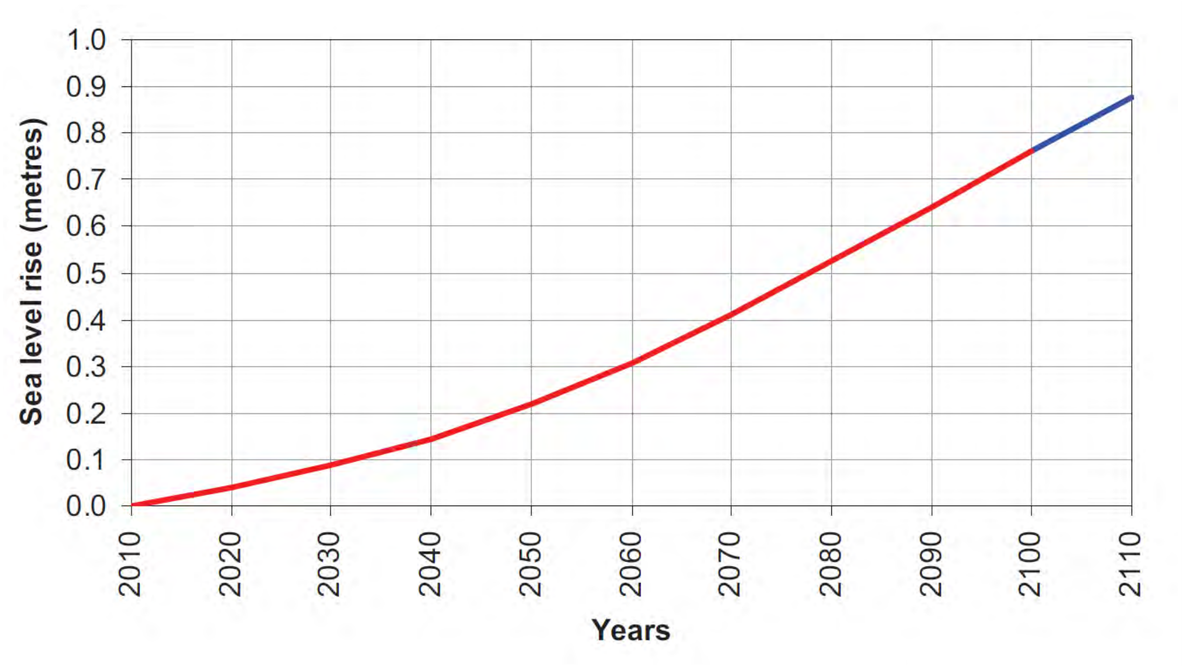


Figure 3.2 Recommended Sea Level Rise Scenario for Coastal Planning in Western Australia (DoT 2010)

Based on Figure 3.2, the required allowances for sea level rise from 2015 to each of the three key time frames, 2030, 2070 and 2110 are presented in Table 3.1.

Table 3.1 Sea Level Rise

Timeframe	Sea Level Rise (m)
2030	0.07
2070	0.39
2110	0.90

4. Coastal Inundation Assessment

4.1 Cyclonic Storm Surge Assessment

Coastal development within Western Australia is guided by the requirements of SPP2.6. This policy outlines the general requirements for the assessment of coastal hazards, which includes coastal erosion hazards and coastal inundation hazards.

For the general case of freehold development, SPP2.6 requires consideration of a 100 year planning horizon. In particular, it requires development to plan for and manage risk associated with a 500 year ARI storm surge event, which statistically has an 18% chance of occurrence over the 100 year planning horizon. The potential additional impacts of sea level rise over the 100 year planning horizon must also be considered.

The challenge associated with this requirement of the Policy is that accurate and statistically relevant predictions of the 500 year ARI event cannot be made solely using the available historical water level measurements along the West Australian coastline due to the relatively short durations of the records. This is due to the fact that a continual water level record of about a third (167 years) the recurrence interval in question (500 years) is required to ensure statistical relevance of the prediction. Even the longest reliable water level record within Western Australia (Fremantle) is limited to a little over 60 years (records extend further to before 1900 but are not reliable). Therefore, in the absence of sufficient water level data other methodologies must be considered in order to provide meaningful predictions of the 500 year ARI event.

The most widely accepted methodology for the estimation of the 500 year water level event is to use available information on the frequency and characteristics of key meteorological events and, through modelling, generate a long term synthetic database of events and corresponding water levels. Though this process is still only based on a limited period of available data, the modelling seeks to capture the apparent randomness of the critical components of the meteorological effects through simulation of these events over extended periods of time. This methodology is particularly relevant in cyclone regions, where extremely localised effects on water levels can be observed. Modelling an extended time period therefore helps to ensure that the apparent randomness in cyclone track and severity is accounted for in any estimation of events with long recurrence intervals.

Based on the above assessment methodology, a cyclone database was synthesised with a Monte Carlo Simulation using key meteorological components from the historical cyclone database. The synthesised cyclone database was subsequently interrogated and ranked based on the impact on the study area. The top synthesised events were then modelled using the Delft3D numerical model to determine the inundation level at the study site. An extreme analysis was carried out on the results of the numerical modelling to determine the cyclonic storm inundation levels. Details of the 20, 100 and 500 year ARI events are presented in Table 4.1. Detailed discussion on the methodology, numerical model setup and validation as well as Monte Carlo Simulation are presented in the following sections.

Table 4.1 Cyclonic Storm Inundation Levels – Cape Burney to Greys Beach

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	1.9	2.2	3.0
2030	0.07	2.0	2.3	3.1
2070	0.39	2.3	2.6	3.4
2110	0.90	2.8	3.1	3.9

Note: 1. Inundation Levels have included sea level rise for each timeframe as presented in Table 3.1.

4.1.1 Assessment Methodology

The approach adopted by MRA to determine the potential cyclonic storm surge inundation levels for the Cape Burney to Greys Beach shoreline is contingent on the use of numerical modelling techniques. This approach is required due to the short availability of water level data within the Geraldton region as compared to the required recurrence interval for prediction. Specifically, water level records at Geraldton Port are only available for a period with a duration totalling around 30 years between 1986 and 2016.

The limited availability of water level data means that an extreme analysis of peak recorded levels would not provide meaningful results when predicting the 500 year ARI event. Consequently, there is the need to use numerical modelling techniques (as outlined in Section 4.1.2) to create a synthetic water level record which can then be used to determine extreme water levels for the study area. The overall modelling approach is summarised below.

- Setup, calibrate and validate the Delft3D cyclone, wave and hydrodynamic model for the region.
- Use the measured water level data at Geraldton Port and historical cyclones that have affected the region and interrogate cyclone tracks and measured water levels to determine a first order storm surge approximation.
- Use a Monte Carlo model to simulate 2,000 years of cyclone tracks and severity.
- Rank the 2,000 years of synthetic cyclones using the first order storm surge approximation combined with the predicted tide to determine the top events.
- Use the Delft3D model to simulate the top events and record the peak water levels at the study area.
- Complete an extreme analysis of peak recorded water levels for the study area.

Further details regarding the adopted approach and the results of the investigations are outlined in the following sections.

MRA have previously used the approach outlined above at a number of locations. In particular, it has been used to determine the 100 year ARI water level at Port Hedland. The results of this assessment provided good agreement with the prediction of the 100 year ARI event determined from analysis of the historical water level record. This result provides confidence that this modelling methodology can provide meaningful outcomes.

4.1.2 Delft Model Setup & Calibration

The Delft3D suite of models provides an integrated model approach that can be used to simulate atmospheric pressure differentials, wind fields, wave climates and water levels associated with the passage of tropical cyclones (Deltares, 2011a). The Delft suite of models has been extensively used around the world and are recognised as high quality models. This integrated modelling approach has been adopted for this study in order to best represent the physical processes that generate storm surge.

The physical processes that lead to the generation of cyclonic storm surge operate on a spatial scale equivalent to that of the cyclone itself. For this reason, to adequately model cyclonic storm surge requires large model domains. However, due to computational limitations it is not efficient to model large areas at high resolutions, therefore a Delft3D domain decomposition model configuration has been used.

Domain decomposition allows a section of the overall grid to be modelled at significantly greater resolution to capture the key features and bathymetry surrounding the area of interest. Figure 4.1 shows the model domain and bathymetry for the coarse and fine grid and Figure 4.2 shows the model domain, topography and bathymetry for the very fine grid used for this study. Bathymetry and topography data was sourced from hydrographic survey information provided by the Department of Transport, local nautical charts, available topographic LIDAR survey provided by the Northern Agricultural Catchment Council (NACC), data from NASA's Shuttle Radar Topography Mission (SRTM) and the Australian Bathymetry and Topography dataset obtained from Geoscience Australia (Whiteway, 2009).

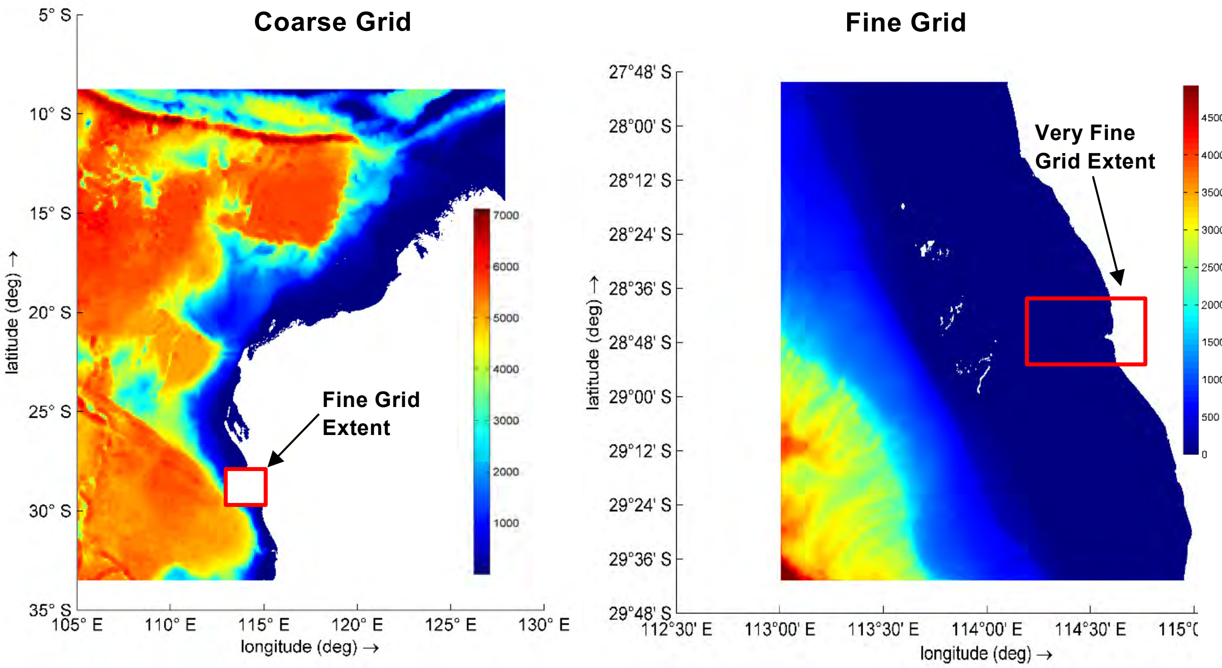


Figure 4.1 Model Domain & Bathymetry for Delft3D Coarse & Fine Grids

Very Fine Grid

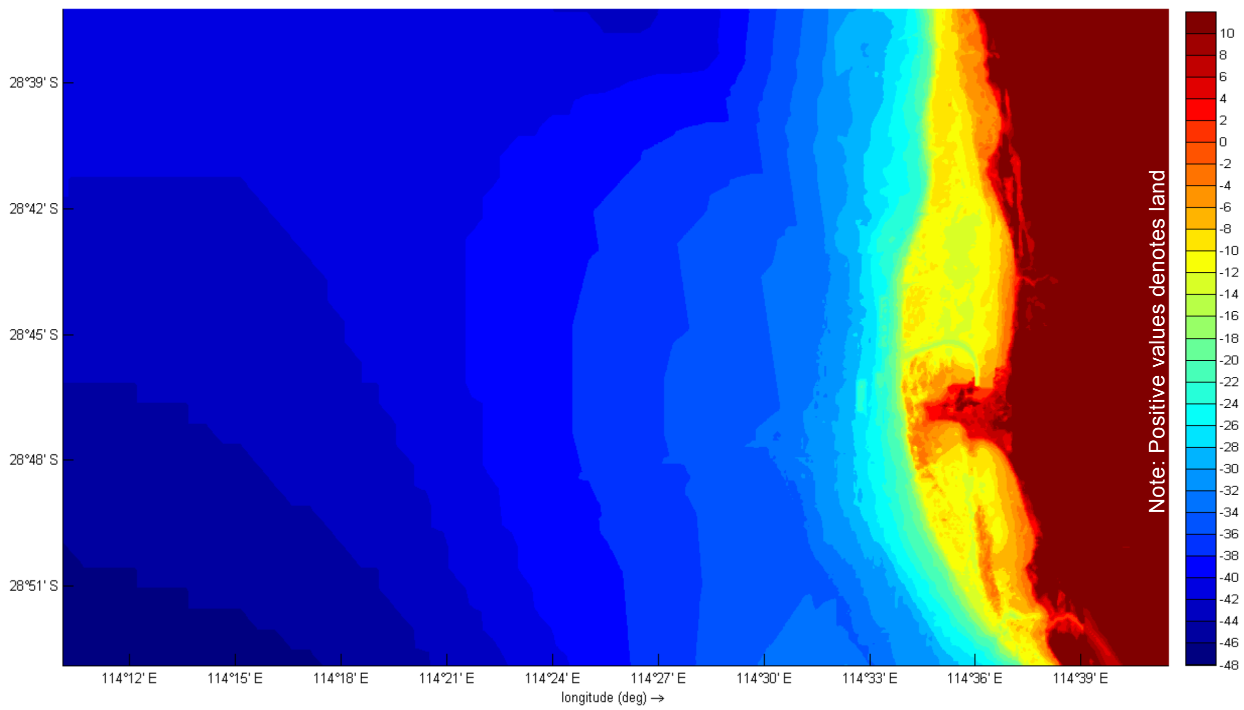


Figure 4.2 Model Domain, Topography & Bathymetry for Delft3D Very Fine Grid

With the model grids established, calibration and validation of the model system is critical in order to ensure that the model predictions adequately reflect the reality. To calibrate and validate the model's ability to accurately determine the wave and storm surge requires historical wave, water level and cyclone track data to be available. Using this information a selection of historical cyclones can be simulated within the model domain to determine if the model predictions match the observation record. To assist with this process historical water level data was obtained from DoT for Geraldton Port. The water level record for Geraldton Port provides a relatively continuous record dating back to 1986. Wave measurements recorded by a Waverider buoy and AWAC device were also available at the entrance of the outer channel of the Geraldton Port, this was provided to MRA by the MWPA. The wave data provides a relatively continuous record dating back to 1999. The location of the tide gauge and AWAC device is shown in the following Figure.

Figure 4.3 Location of Tide Gauge & AWAC (Source: Google Earth)

Water Level Calibration & Validation

To determine suitable model calibration events the periods of water level records were cross referenced against information regarding the passage of tropical cyclones within the region obtained from the Bureau of Meteorology (BoM) cyclone database (BoM, 2015). A summary of the cyclones that came within around 600 km of the study area is provided in Table 4.2. It should be noted that the cyclone record has been clipped to include only data from the 1986 onwards as prior to this period water level records at Geraldton Port are not available for cross reference.

Given the information above, two separate events were chosen for the calibration and validation of the Delft3D model. These events were selected as they generated a reasonably high storm surge, and also had good data coverage within the historical cyclone database and water level record. The chosen events are as follows.

- TC Vincent for calibration and validation with the observed water level record at Geraldton Port.
- TC Bianca for calibration and validation with the observed water level record at Geraldton Port.

Track and intensity plots for TC Vincent and TC Bianca are presented in Figures 4.4 and 4.5 respectively.

MRA has previously adopted this method of calibration at other locations, such as Denham, Cape Lambert and Port Hedland. The result of the validations at these locations indicate a good agreement between the modelled output and the historical measurements.

Table 4.2 Historical Cyclones affecting Geraldton since early 1980s

Name	Date	Name	Date
Billy-Lila	May 1986	Nicholas	February 2008
Herbie	May 1988	Ophelia	March 2008
Ned	March/April 1989	Pancho	March 2008
Vincent	March 1990	Dominic	January 2008
Frank	December 1995	Bianca	January 2011
Rhonda	May 1997	Heidi	January 2012
Vance	March 1999	Iggy	January 2012
Elaine	March 1999	Mitchell	December 2012
Steve	February/March 2000	Narelle	January 2013
Alistair	April 2001	Christine	December 2013
Emma	February/March 2006	Olwyn	March 2015
Glenda	March 2006		

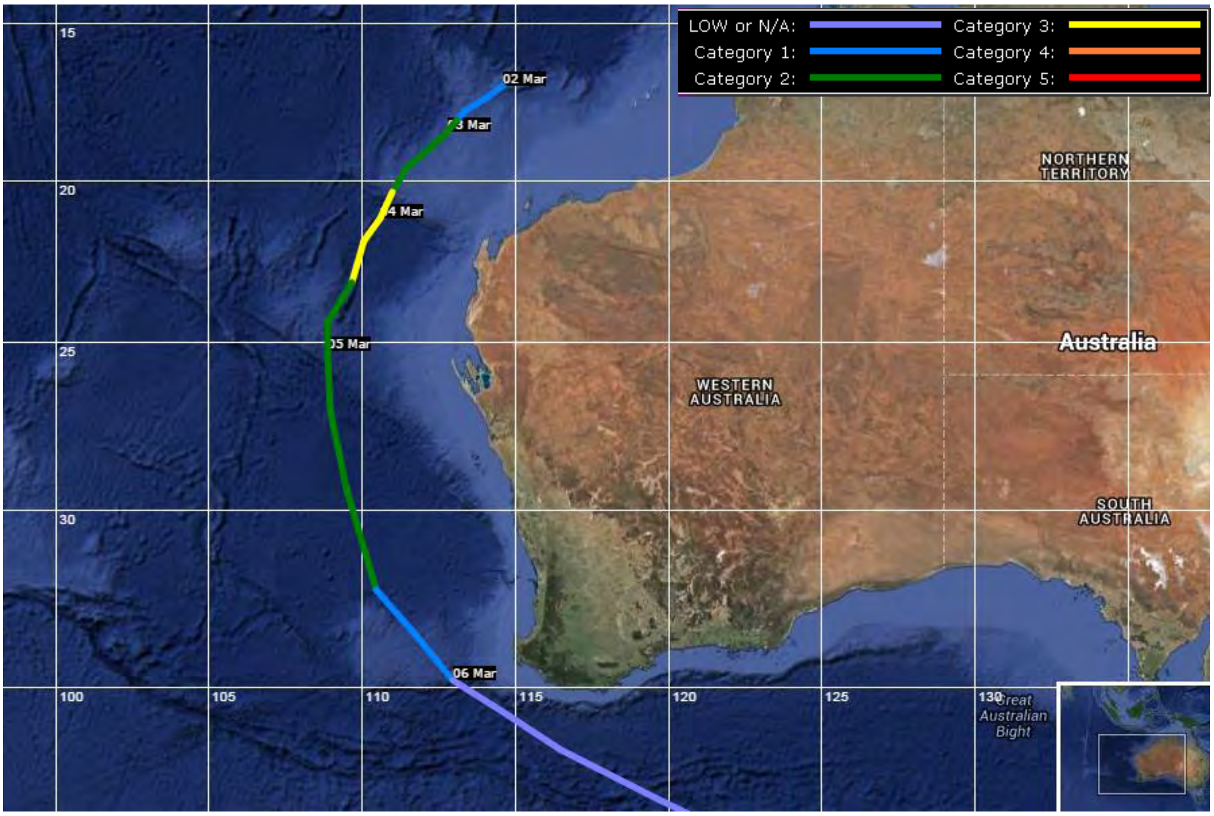


Figure 4.4 Tracks & Severity Plot for TC Vincent

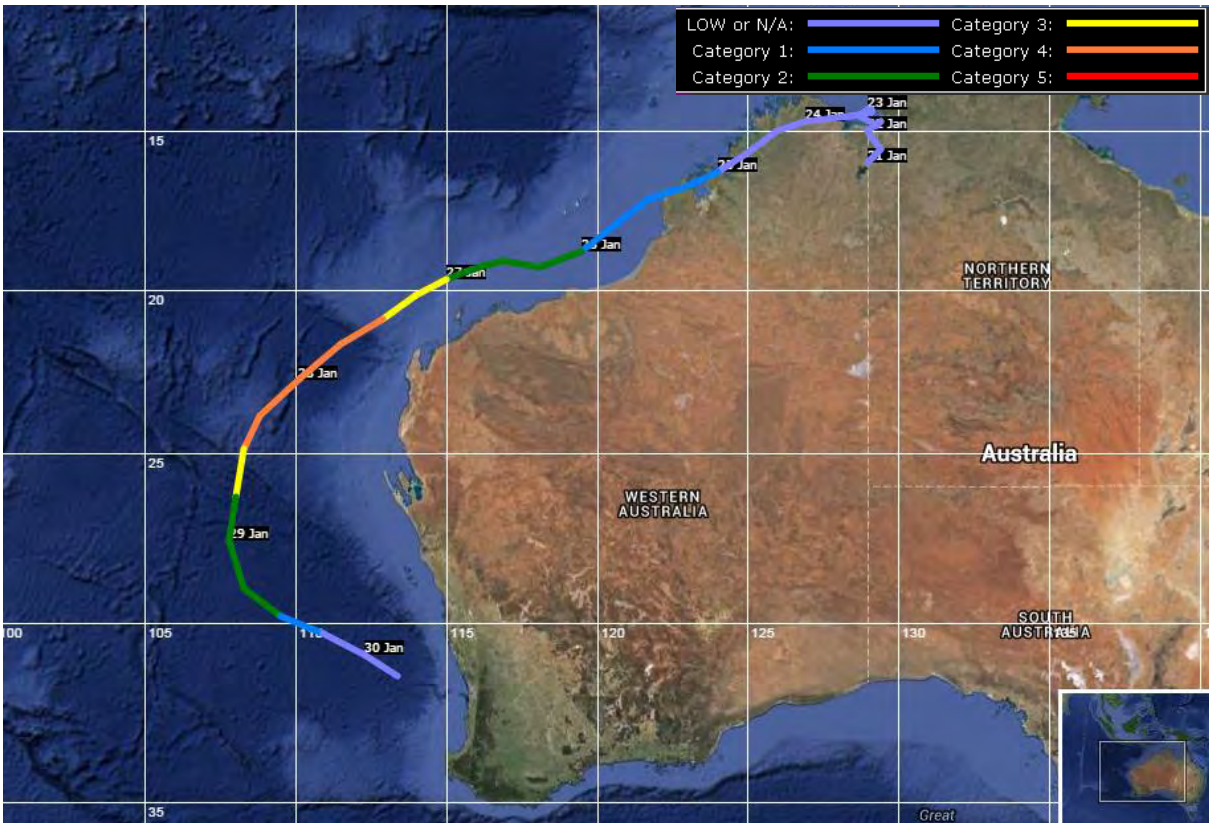


Figure 4.5 Tracks & Severity Plot for TC Bianca

Details of the cyclone tracks and severity were extracted from the BoM cyclone database and were used to generate cyclone wind and pressure fields for input to the Delft3D model. This process was completed using the Delft3D Wind Enhanced Scheme (WES) module (Deltares, 2011b) in combination with a wind field calculated for each event based on the results of Holland (1980).

Each cyclone event was simulated using the Delft3D model, with the modelled water level record extracted at a point within the model that corresponds to the location of the tide gauge. The modelled water level at Geraldton Port for TC Vincent is presented in Figure 4.6 together with the observed water level and the predicted tide. Generally, the measured and modelled water levels show good agreement, as does the measured and modelled surge levels, with the model replicating the measured peak water level and surge within 0.1 of a metre. This is a significant result given the short term fluctuations in water level that are evident in the measured data and reasonably matched by the model. It does appear from the plot of water levels that the timing of the peak surge differs slightly to the observed water level record. The reason for the difference is expected to be attributable to slight differences in the cyclone position from that noted within the cyclone data base (due to the three hour spacing between data points), as well as slight differences between the cyclone characteristics in reality compared to within the model.

Willmott et.al (2011) presented a method to determine a refined index of agreement (d_r) to estimate model accuracy. The value of d_r ranges from -1.0 (poor estimation of observed data) to 1.0 (Perfect estimate of observed data). For the modelling of water level during the passage of TC Vincent, the refined index of agreement between the observed and predicted water level is approximately 0.91. This represents a very close agreement between the measured and modelled data and therefore provides confidence in the model as a reliable predictive tool.

It should be noted that the modelling was completed for the duration of the cyclone record within the database (i.e. as the cyclone reduced in strength it was not tracked within the BoM database and therefore the full tail of the event was not modelled).

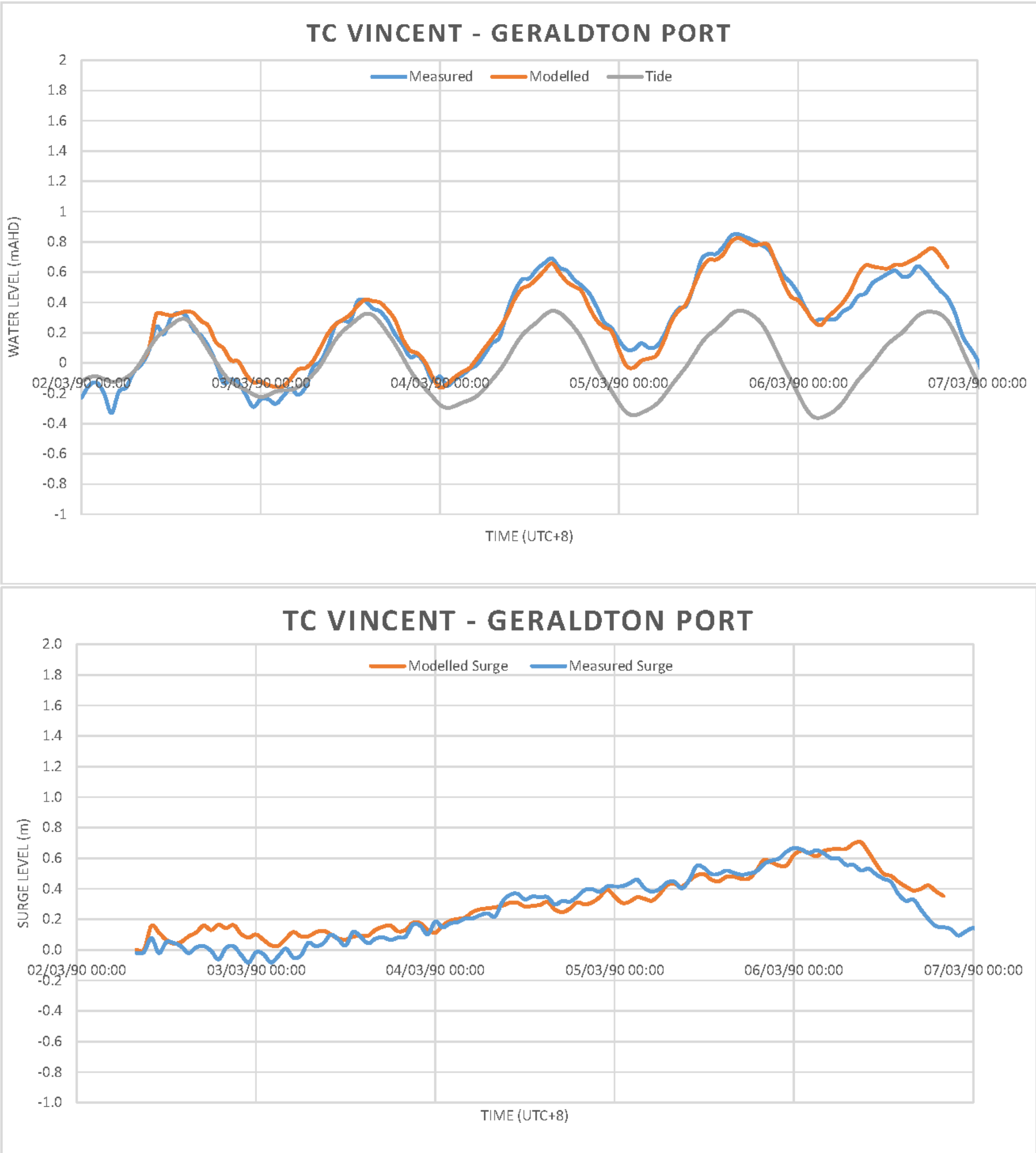
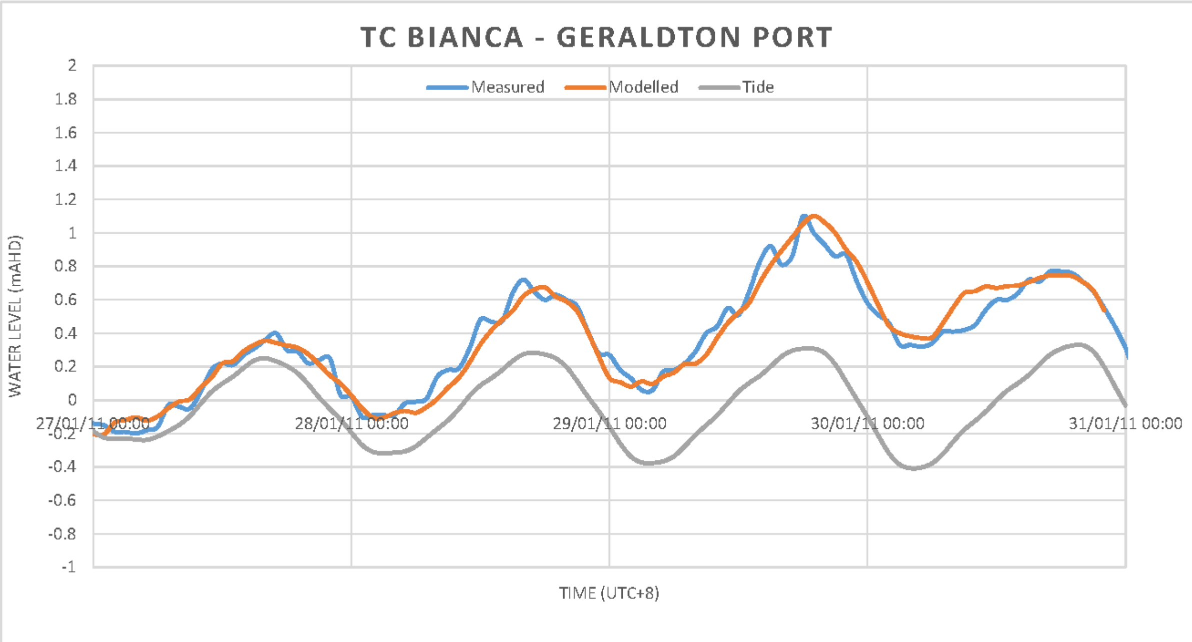


Figure 4.6 Tracks & Severity Plot for TC Vincent

The result of the modelling of TC Bianca are presented in Figure 4.7. As discussed in the previous paragraph, the modelling was completed for the duration of the cyclone record within the database. Nevertheless, the measured and modelled water levels show good agreement, as does the measured and modelled surge levels, with the model replicating the measured peak water level and surge to an accuracy of better than 0.1 of a metre. This high level of accuracy for the modelling of water level during the passage of TC Bianca is further confirmed by the refined index of agreement between the observed and predicted water level, which is approximately 0.85. This again represents a very close agreement between the measured and modelled data and therefore provides confidence in the model as a reliable predictive tool.



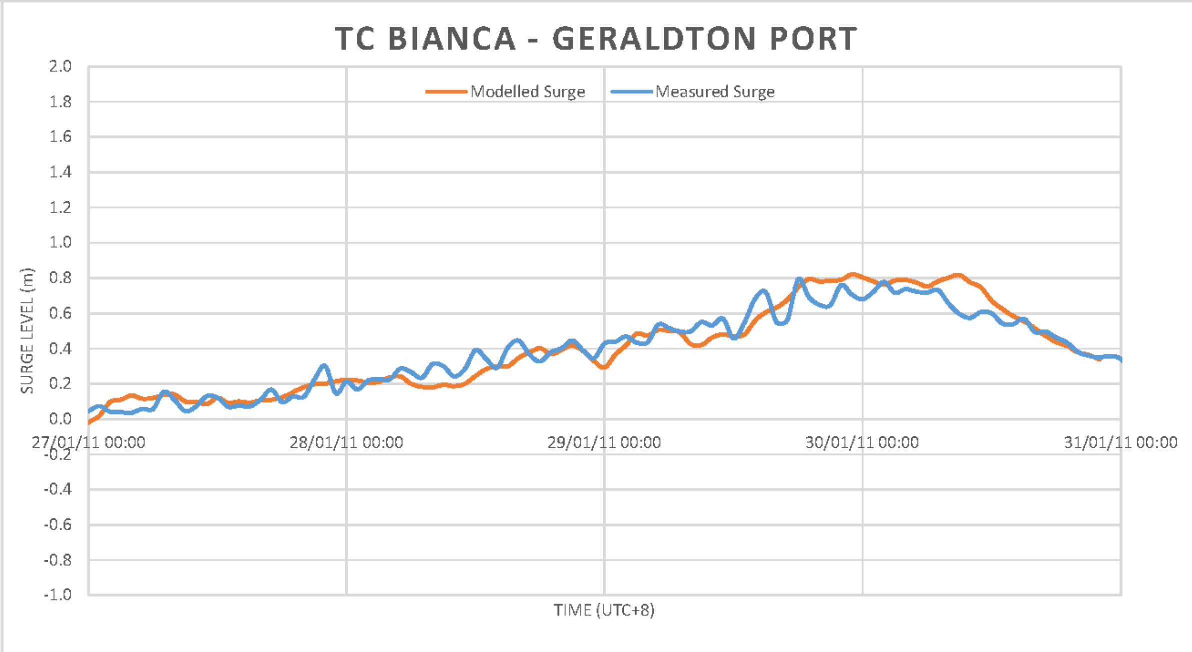


Figure 4.7 Tracks & Severity Plot for TC Bianca

Wave Calibration & Validation

In order to provide an assessment of the suitability of the Delft3D Wave model to simulate the wave conditions, a model simulation would ideally be completed for a period during the passage of one of the tropical cyclone events adopted for water level validation. However, the wave measurements at the outer channel of the Geraldton Port are only available from 1999, and an interrogation of the wave measurements indicated a gap in the data for the period during the passage of TC Bianca. As a result, a model simulation was completed for a period between 1st of August 2005 and 1st of October 2005. This period coincided with the passage of a number of low pressure systems, and is therefore considered to be suitable for wave validation.

The offshore wave input for this simulation was completed using wave hindcast data from WAVEWATCH III, which is a global hindcast model operated by the National Oceanic and Atmospheric Administration (NOAA). The wave hindcast data was generated using the NCEP Climate Forecast System Reanalysis Reforecast (CFSRR) homogeneous data set of hourly 1/2° spatial resolution winds.

Results of the comparison between measured and modelled wave conditions are presented in Figure 4.8. The modelled wave results show good agreement with the measurements taken at the outer channel. The refined index of agreement between the measured and modelled wave height is approximately 0.7, indicating a good estimate of the measured data. The Delft3D Wave Module is therefore considered to be an appropriate tool for the modelling of nearshore wave conditions.

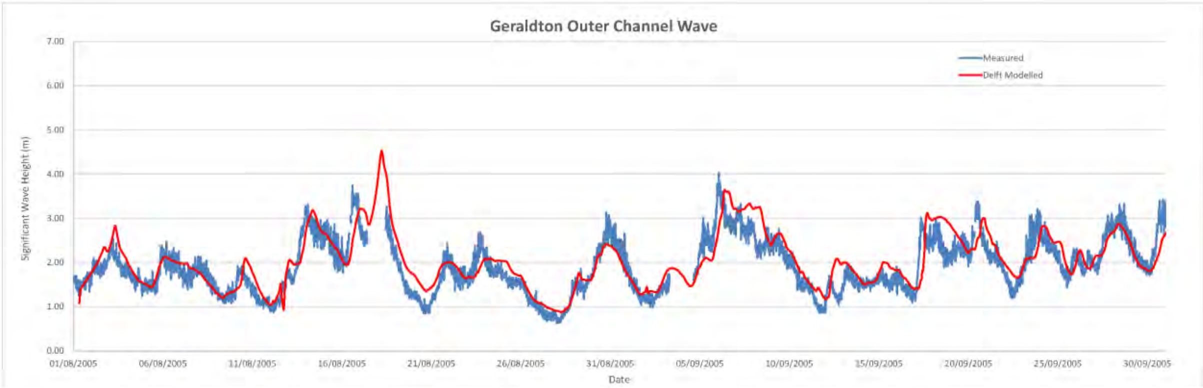


Figure 4.8 Measured versus Modelled Wave Height at the Outer Channel

4.1.3 Cyclone Track Synthesis

To develop a meaningful estimate of events with long average recurrence intervals requires a long duration of reliable data record. Statistically, the length of the record should be around a third the duration of the ARI that is being predicted. However, generally speaking, the longer the available record the greater the accuracy of the prediction. A long cyclone record is therefore required. However, reliable cyclone records only extend back to the early 1960s when satellite imagery became available to track cyclones off the coastline. Therefore the available cyclone track data only spans a period of around 50 years, which is insufficient to reliably predict the 500 year ARI event.

As a result, synthetic data needs to be generated to populate the data space. The extreme conditions can then be determined using extreme value analysis on the outputs from the synthetic events.

A Markov Chain Monte Carlo (MCMC) model was developed for this study based on the methodology described in Risi (2004) and Emanuel et al (2006). A schematic diagram of the MCMC model is provided in Figure 4.9. Further details of the key steps in the process are provided in the following sections.

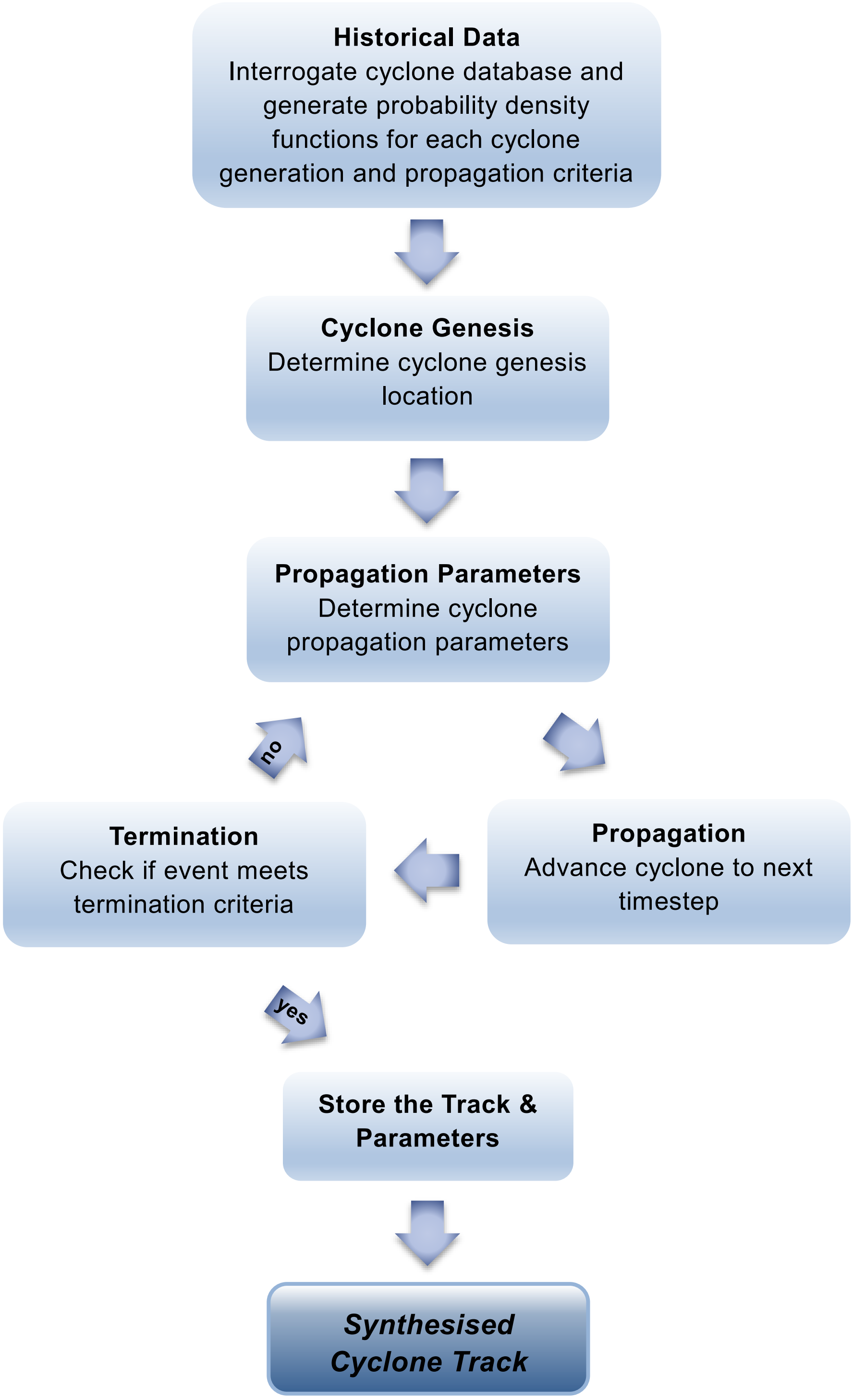


Figure 4.9 Monte Carlo Simulation Scheme

Historical Data Analysis

BoM maintains a cyclone database that contains information regarding tropical cyclones experienced between 1906 and 2015 for the Australian region (BoM, 2015). This database includes information such as cyclone location, central pressure, maximum wind speed and other relevant cyclone track parameters. However, as previously discussed, to ensure data accuracy, the raw cyclone database was filtered to include only data after 1960.

Analysis of the historical cyclone database was completed in order to ascertain spatial and temporal changes in the key parameters required for cyclone generation and propagation. These key parameters include the following.

- Location of origin (refer to as the cyclone genesis location).
- Forward speed of the cyclone.
- Cyclone direction / heading.
- Central pressure.

Statistical distributions for each of the key parameters were then developed on a 2° latitude by 2° longitude grid covering the whole of the Australian region. A separate distribution was developed for each grid in order to ensure that spatial variations in cyclone track and intensity characteristics were captured within the model.

Cyclone Genesis Location

Within the MCMC model, cyclone genesis positions are obtained by sampling from a 3D parametric probability distribution. In order to create the parametric probability distribution, the historical cyclone database was filtered to include only the first recorded location for each cyclone. The filtered genesis information was then smoothed using a Gaussian smoothing kernel in order to ensure a continual coverage over the entire region. The smoothed probability distribution for cyclone genesis is shown in Figures 4.10 and 4.11. It should be noted that this data relates only to cyclone genesis within the Australian region. Additionally, the genesis model was confined to ensure that cyclone genesis could not occur over land.

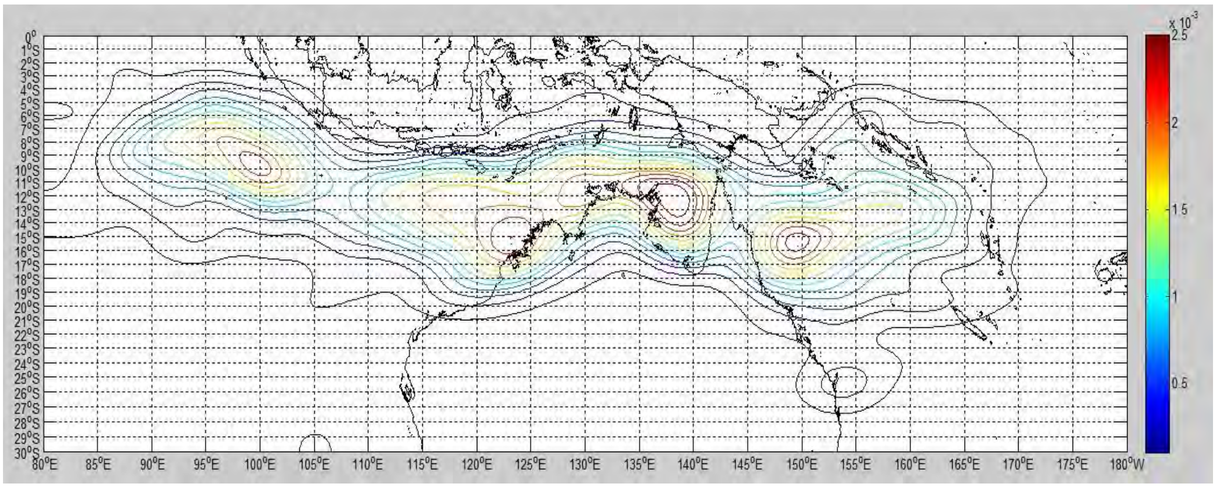


Figure 4.10 Smoothened Genesis Probability Distribution – 2D Plan View

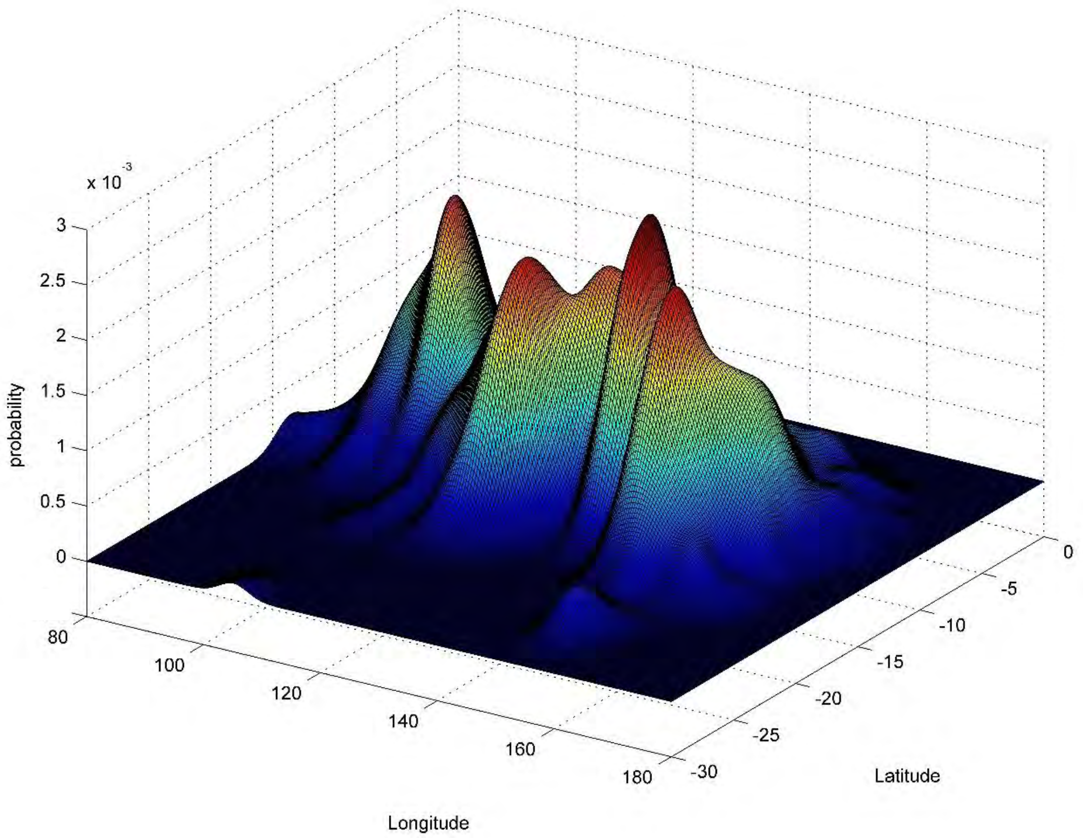


Figure 4.11 Smoothened Genesis Probability Distribution – 3D View

In order to establish a cyclone genesis position for each synthesised cyclone track an initial genesis location was sampled from the genesis probability distribution using a random 3-dimensional (3D) hit and miss algorithm.

Genesis Time

In order to generate a genesis time for each cyclone, the cyclone genesis points within the historical cyclone database were discretised into histograms based on the number of cyclone genesis events per year and the monthly genesis occurrences. These histograms are presented in Figures 4.12 and 4.13 respectively.

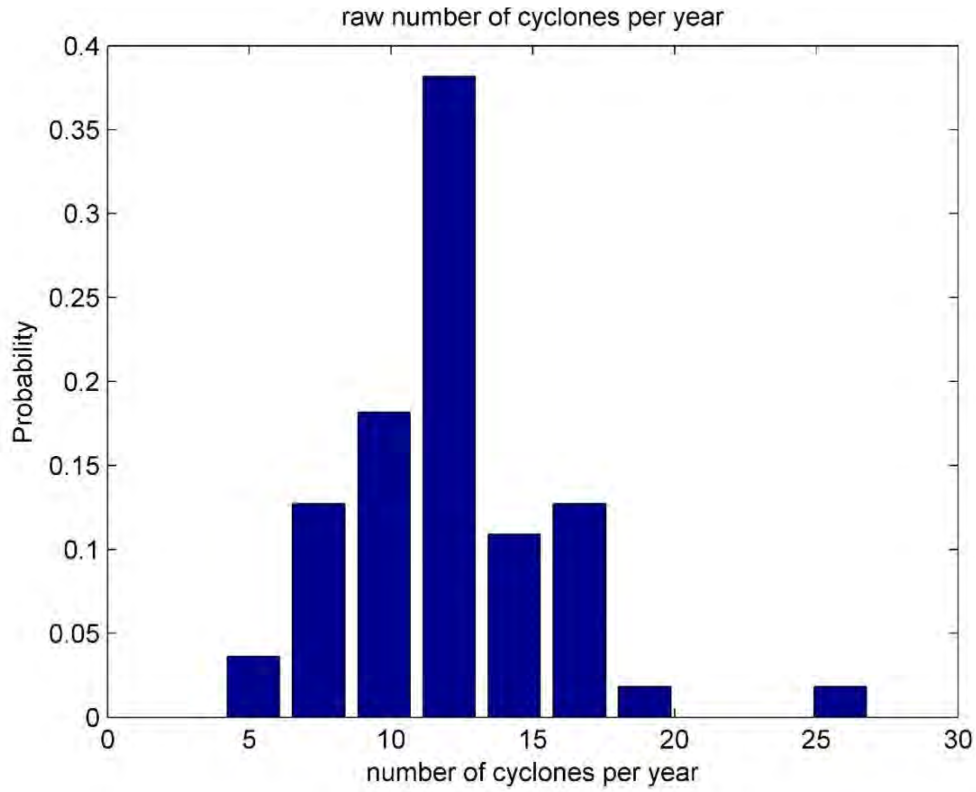


Figure 4.12 Probability of the number of cyclones per year within the Australian Region

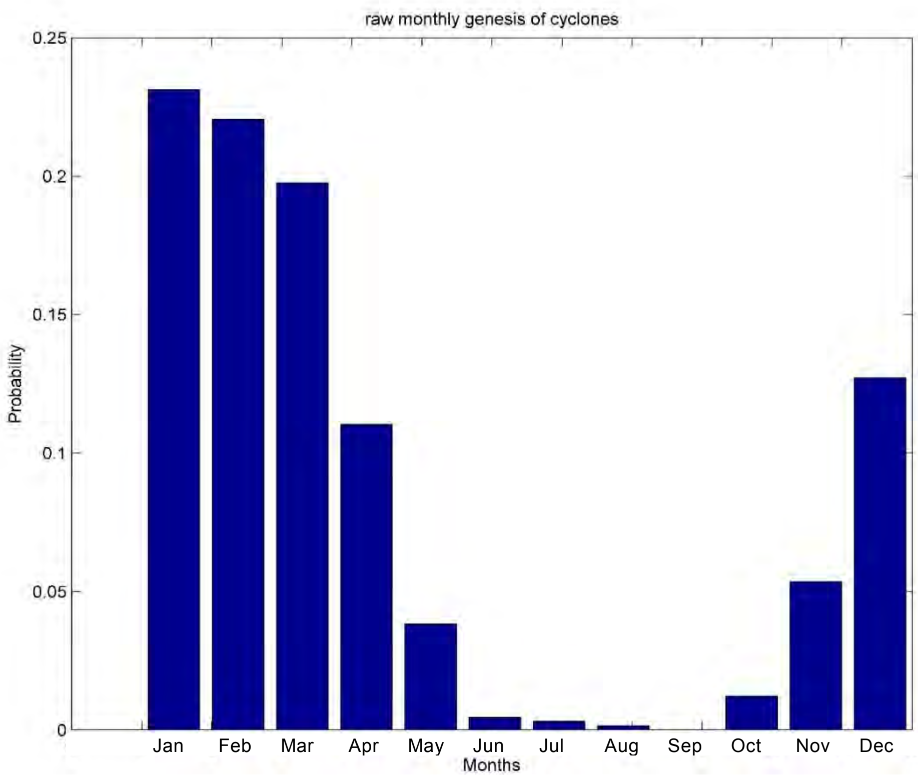


Figure 4.13 Probability of monthly occurrence within the Australian Region

Within the MCMC model the number of cyclones within each year and the times for cyclone generation within that year were randomly sampled from the parametric probability distribution histograms. To sample from the parametric probability distribution (histograms), a one dimensional hit and miss algorithm was adopted.

Genesis Parameters

To initiate a cyclone, initiation parameters were required in addition to the genesis position and time. These parameters included the following.

- Initial forward speed of the cyclone (km/h).
- Initial direction of the cyclone (Cartesian degrees between -180º to 180º)

The initiation parameters were obtained from their corresponding probability distributions. The probability distributions were generated by interrogating the BoM cyclone database.

4.1.4 Propagation

Once the genesis position, time and parameters were determined, the cyclone propagation parameters were required for the cyclone to progress to its next location / timestep.

The main issue with randomly sampling the propagation parameters is that the sampled values must be dependent on the value in the previous state. This is required to prevent random selection of parameters that would otherwise not reflect the physical drivers of cyclone development such as ocean temperature and barometric effects that exist in reality. For example the central pressure at the current location must be dependent on the central pressure at the previous location, otherwise anomalies such as an increase in central pressure during the intensification stage of the cyclone may be observed.

To resolve this issue the concept of predictor and predictands (Risi, 2004) was adopted. A predictor is a variable which is used to predict the predictand. In this case, multiple predictors are required for each predictand. Once the predictors are determined, multiple 3D probability surfaces are subsequently created. The propagation parameters are then sampled from the 3D probability density surface via a 3D hit and miss algorithm.

This is discussed in the following sections.

Choice of Predictor and Predictands

For propagation, the following parameters are required and are therefore chosen as predictands.

- Rate of change of speed.
- Direction.
- Rate of change of central pressure.

To define the new state of the cyclone, the following predictors are adopted.

Geographical Positions (Latitude, Longitude)

A cyclone will have relatively different characteristics depending on its location. For example, cyclones are more likely to intensify at latitudes above 21º S than below due to the sea temperature, and are more likely to dissipate over land.

Previous Rate of Change of Speed

The rate of change of forward speed of a cyclone may not be continuous. In other words, a cyclone could be accelerating at the previous location, but may decelerate at the present location. Therefore, it is essential that the previous rate of change of speed be considered when determining the current rate of change of forward speed.

Rate of Change of Direction

The rate of change of direction is used to predict the propagation direction of the cyclone. It is anticipated that over a long term record there is a very low correlation between the current and previous direction, therefore, it is believed that the rate of change of direction is a more appropriate predictor for direction.

Previous Rate of Change of Central Pressure

To predict the central pressure at a specified location and time, it is again appropriate to adopt the more continuous rate of change of central pressure as a predictor. This enables the cyclone to intensify / dissipate based on a previous rate of change, this eliminates anomalies such as increases in pressure during the intensification of a cyclone.

Propagation Probability Surfaces

Once the predictors were determined, probability surfaces were generated. The probability surfaces generated are as follow

- Rate of change of speed versus previous rate of change of speed.
- Rate of change of direction versus direction.
- Rate of change of central pressure versus previous rate of change of central pressure.

An example of the probability surfaces generated for rate of change of direction versus direction at one grid cell is provided in the following figure.

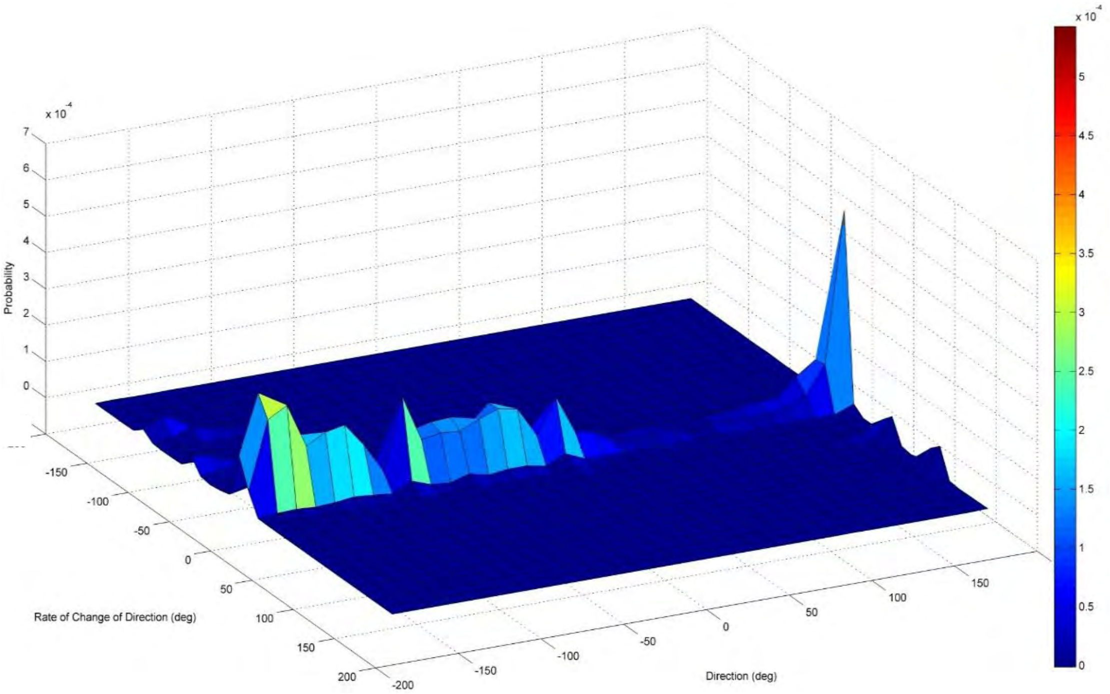


Figure 4.14 Probability Surface for Rate of Change of Direction vs Direction

4.1.5 Track Ranking

In order to rank the synthesised cyclone events based on their influence on the water level at the study site, the cyclonic storm surge combined with the predicted tidal level is considered.

A parametric calculation of likely storm surge has been included within the MCMC model in order to provide predictions of the potential storm surge at the study location. This parametric calculation is based on three cyclone parameters, this includes the bearing (B) of the cyclone, the barometric pressure drop (P_{drop}) caused by the cyclone and the distance (D) from the study site.

To estimate the total water level at the site, the astronomical tide is also calculated and added to the parametric calculation of the storm surge. The tidal level at the study location during the time of the cyclone is calculated using a harmonic analysis (Luick, 2004). The following equation was adopted.

$$h(t) = h_0 + \sum f_n(t) H_n \cos(wt - g_n + V_n(t_0) + u_n(t_0))$$

Where

h_0 – the tidal prediction datum.

f_n – the nodal factor for the equilibrium constituents.

H_n – the amplitude of the specific tidal constituent.

w – the speed (deg/hr) of the tidal constituent.

g_n – the phase lag of the constituent behind $V_n(t_0)+u_n(t_0)$.

$V_n(t_0)$ – the phase of the equilibrium constituent of speed w, evaluated at time t_0.

The use of the above equation generally provides a reasonable prediction of the tidal level.

Each of the synthesised cyclones is then ranked in order of peak water levels, with the top events extracted for further investigation using the Delft3D numerical storm surge model. An additional check is also completed to ensure that any cyclones that track within 400 km of the study site are also extracted for further modelling given limitations in the parametric storm surge estimation. This methodology helps to ensure that all of the top events within the synthesised record are investigated further.

4.1.6 Model Validation

To ensure that the cyclone track model was generating sensible cyclone tracks and parameters, the track model was validated against the historical cyclone database. For this purpose, the model was used to synthesise a 50 year period, equivalent to the period of reliable historical record. By design the model should not exactly reproduce the details of individual historical events, however on average, the characteristics of the entire record should be similar.

Plots of the recorded and modelled cyclone tracks are provided in Figure 4.15. The tracks show general agreement with regard to the densities of events in different areas. To enable a better comparison the data has been further interrogated to show a comparison of the tracks affecting the Geraldton region (Figure 4.16) as well as the key predictands (Figures 4.17 to 4.20).

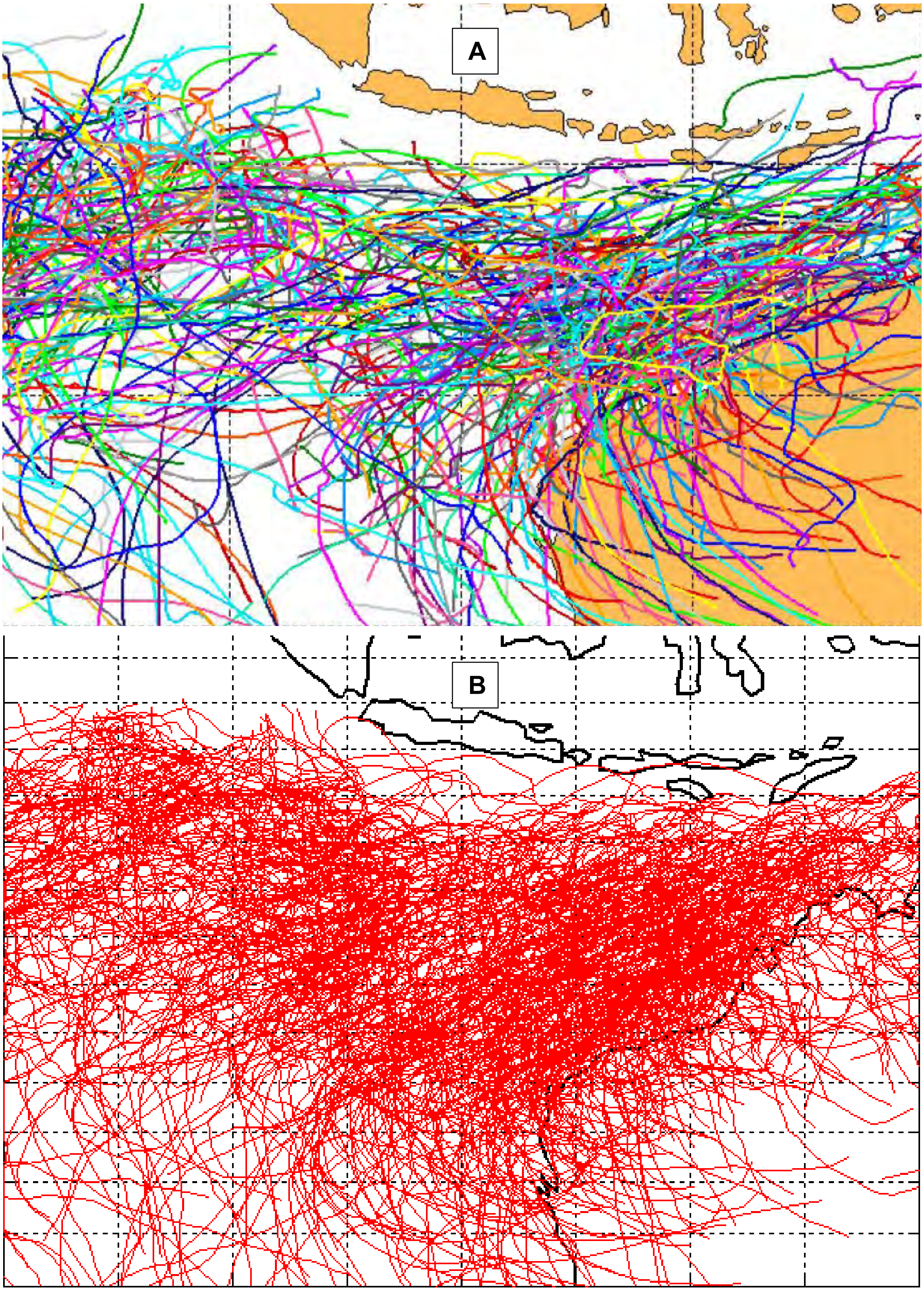


Figure 4.15 (A) Historical cyclone tracks since 1960; & (B) Modelled cyclone tracks for the same period

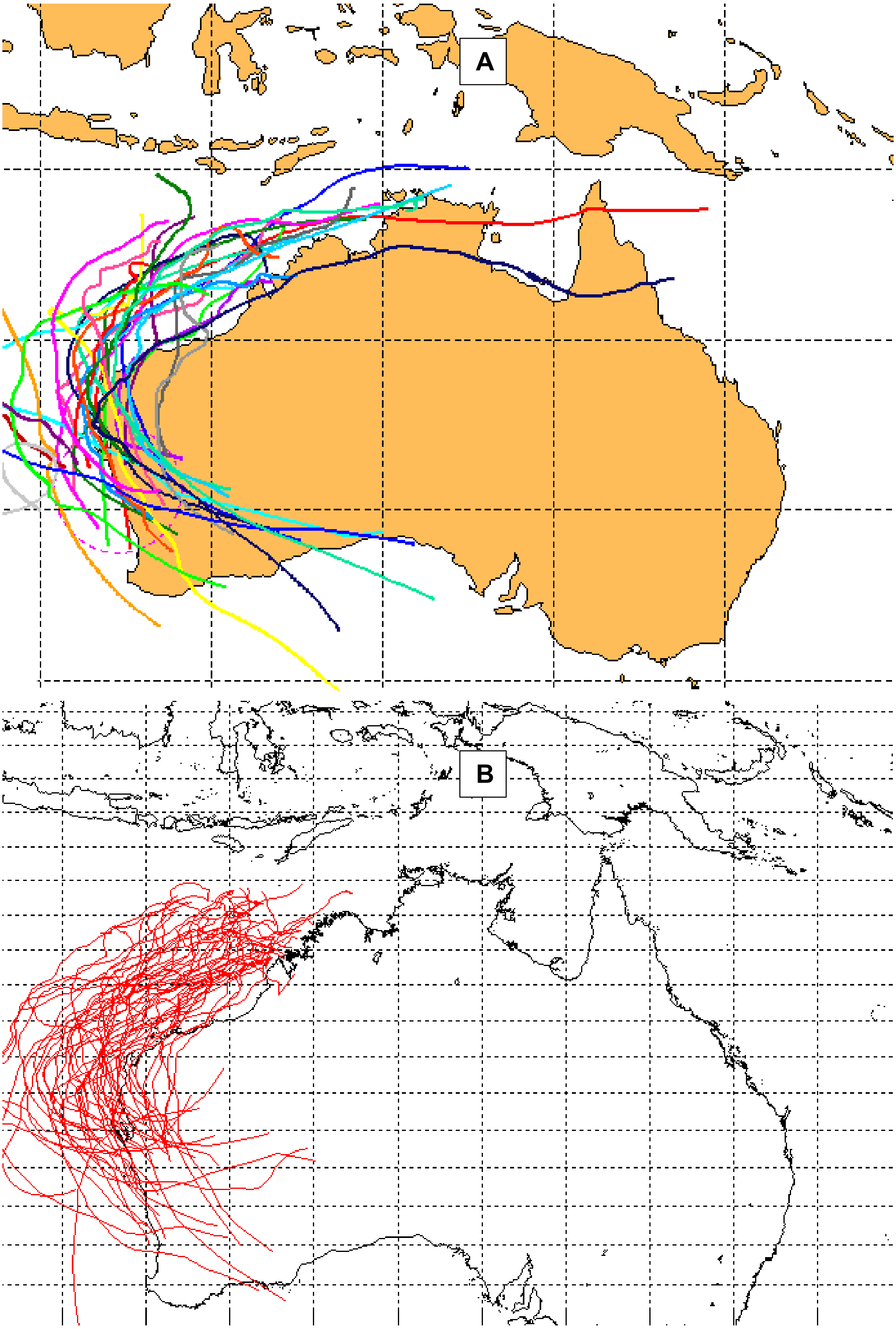


Figure 4.16 (A) Historical cyclone tracks affecting Geraldton since 1960; & (B) Modelled cyclone tracks affecting Geraldton for the same period

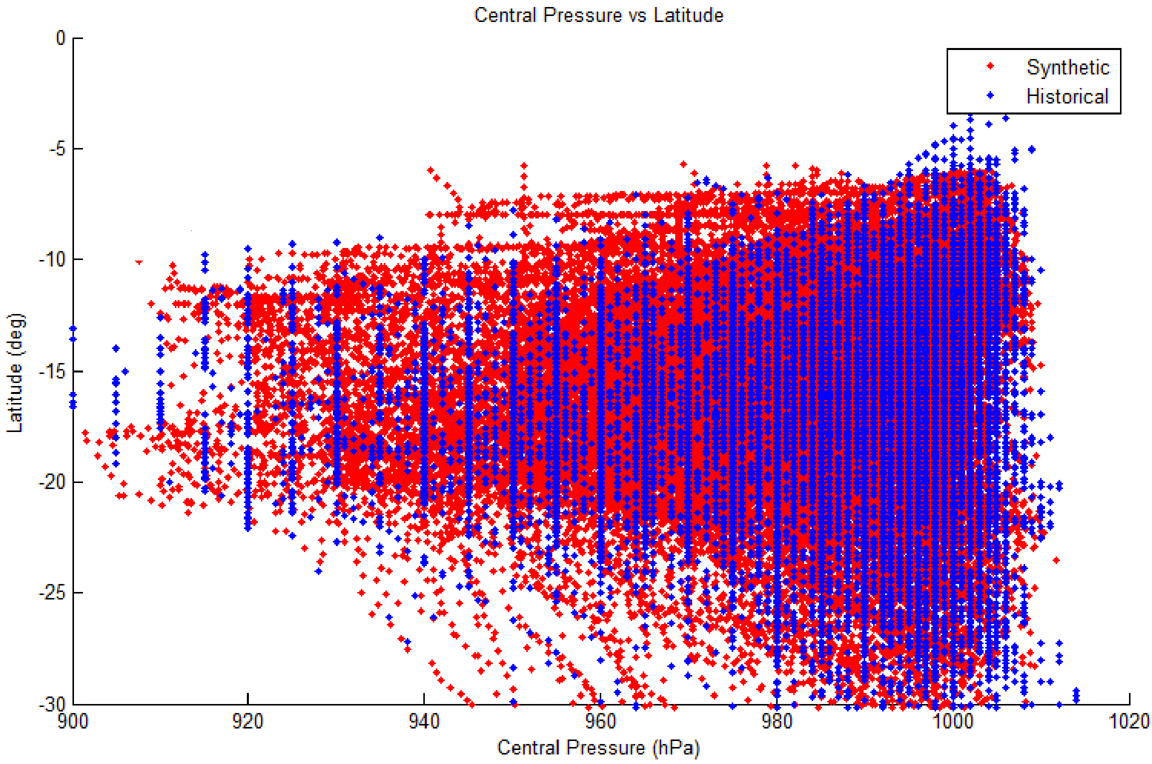


Figure 4.17 Scatter plot of central pressure versus latitude; measured and modelled

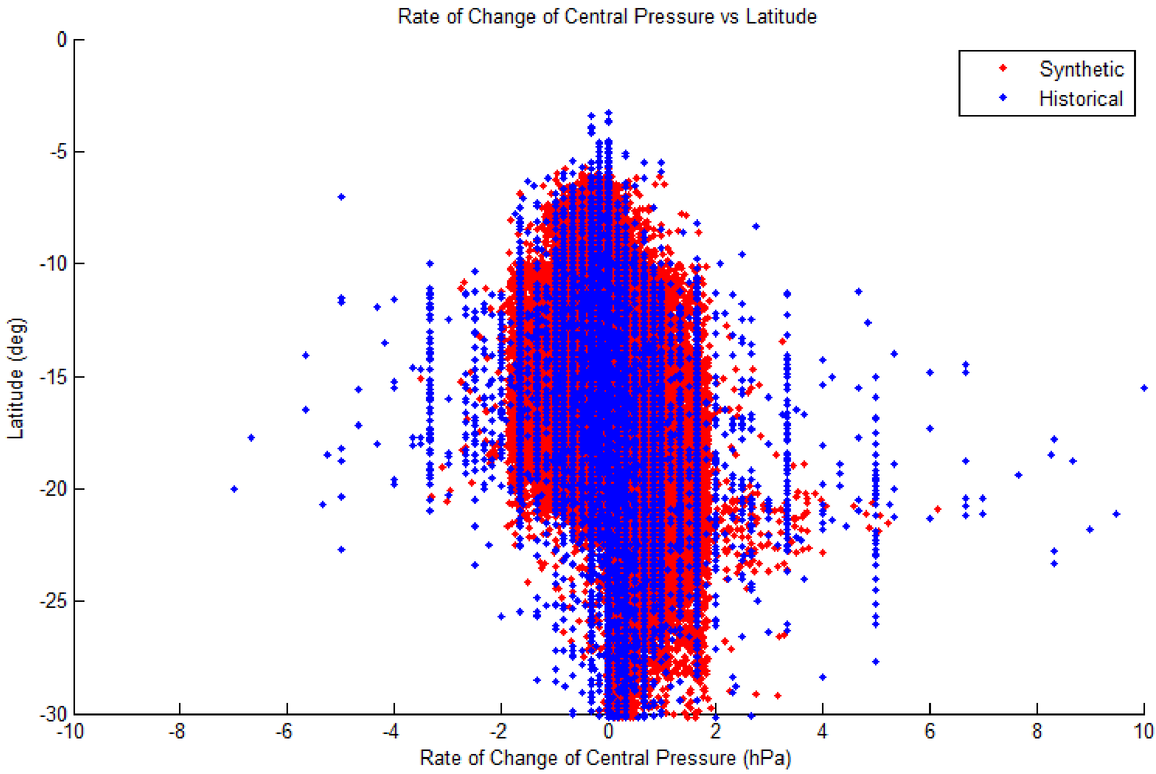


Figure 4.18 Scatter plot of rate of change of central pressure versus latitude; measured and modelled

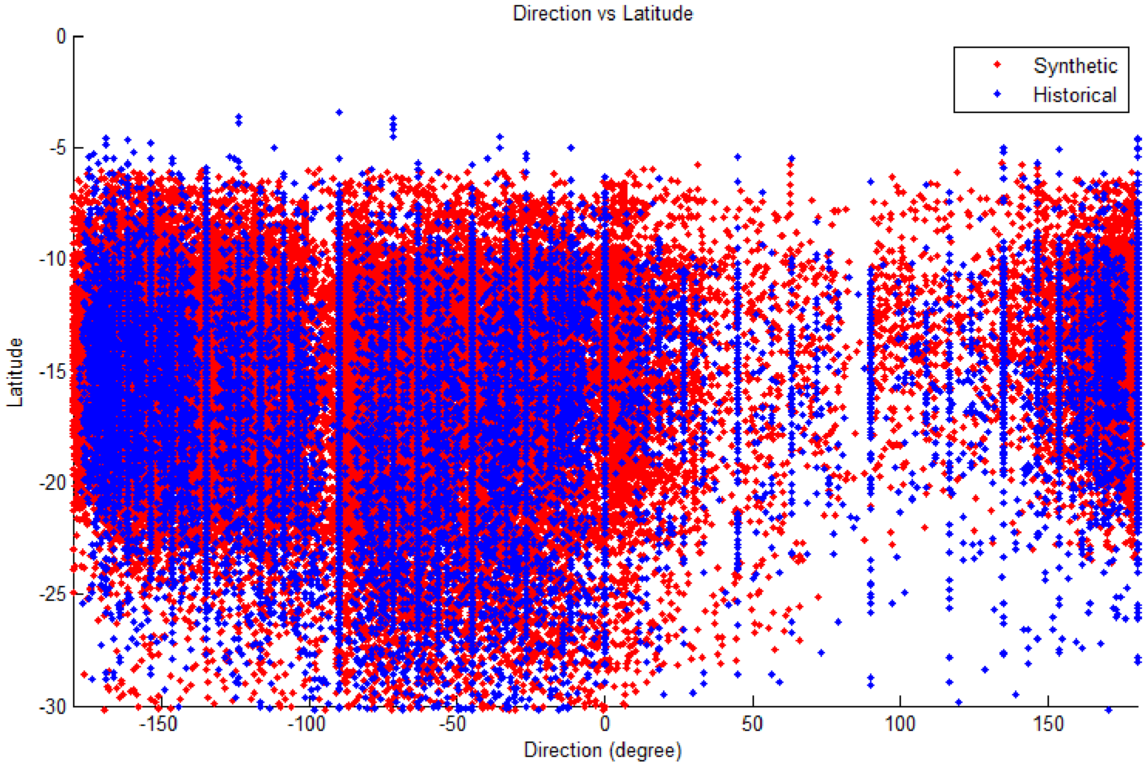


Figure 4.19 Scatter plot of cyclone travel direction versus latitude; measured and modelled

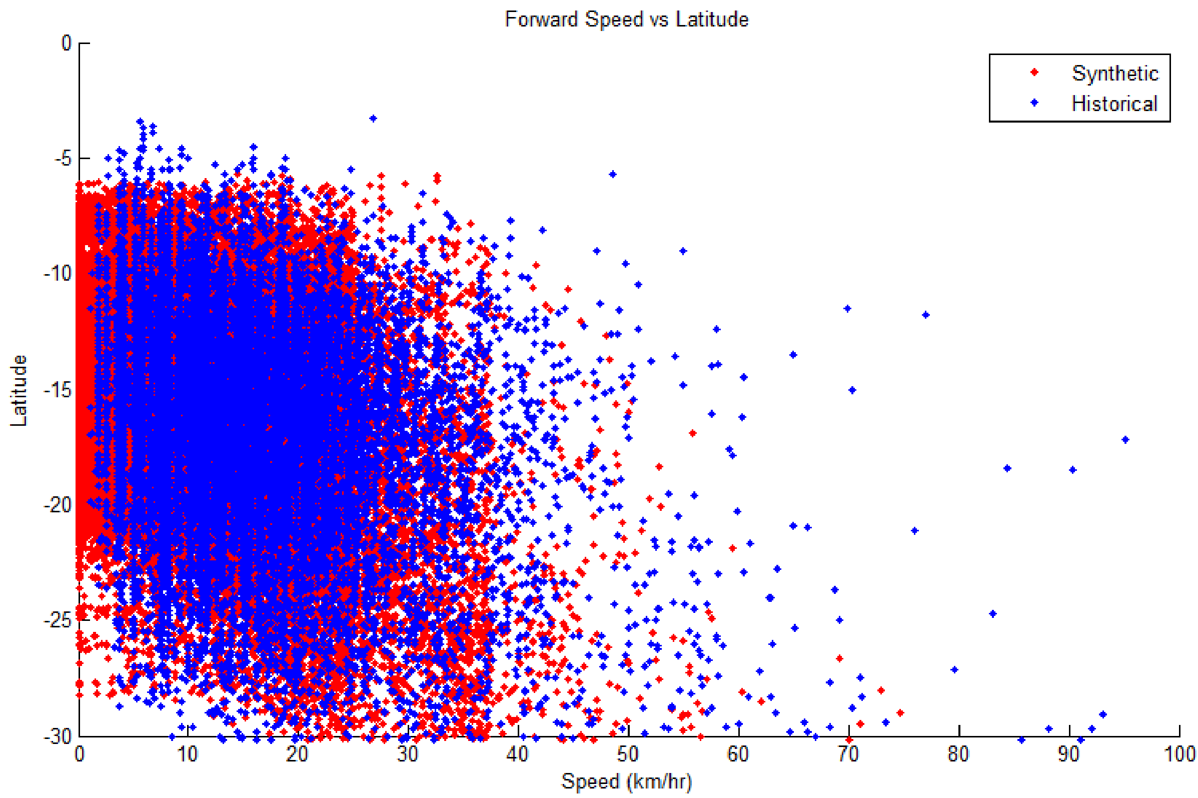


Figure 4.20 Scatter plot of cyclone forward speed versus latitude; measured and modelled

For the purpose of this study, the model continues to track cyclones as they degrade into ex-tropical cyclones. As a result, the model will track cyclones up to a central pressure of 1005 hPa. This is slightly higher than the tracking criteria of BoM, which generally stop tracking a cyclone above 1000 hPa.

Nevertheless, review of the figures shows a high level of agreement between the recorded and modelled data. This high level of agreement confirms that the model provides a suitable tool for the synthesis of a long term cyclone record.

4.1.7 MCMC Model Results

A 2,000 year cyclone record was simulated using the validated MCMC cyclone track model. The synthesised cyclone database was then interrogated based on the proximity of each event to Geraldton and the results of the first order parametric approximation of the water level. Figure 4.21 shows the main events within the synthesised record that would have effected Geraldton.

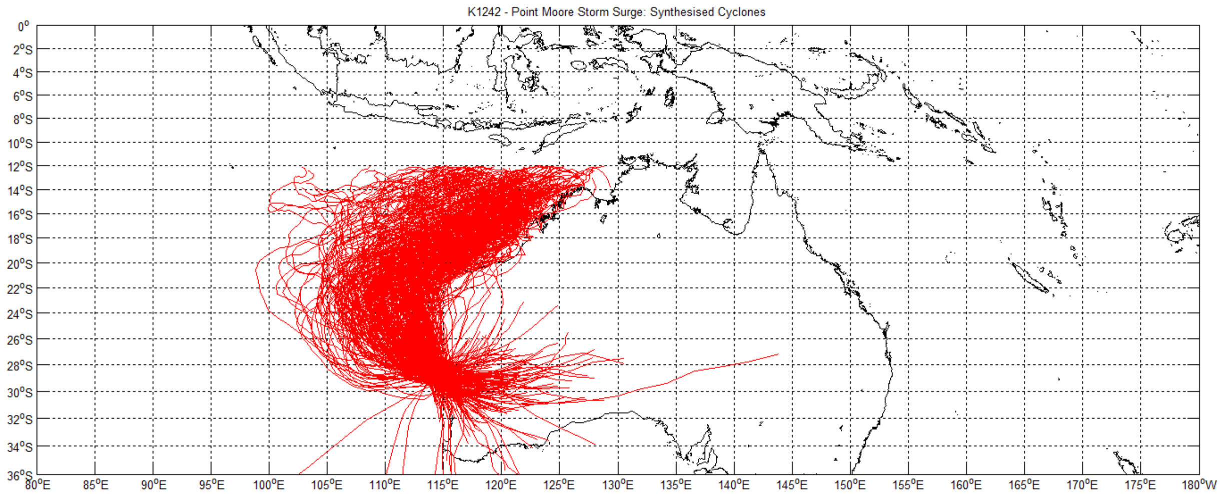


Figure 4.21 Plot of synthesised cyclone tracks within 100 km of Geraldton

Using the track ranking algorithm, a total of 106 events were extracted for further simulation within the Delft3D model.

4.1.8 Wave Setup

The shoreline in the study area is exposed to the open coast. Therefore, the influence of water setup induced by wave actions is assessed and included in this study.

Dean and Walton (2008) provide a comprehensive review of investigations into the extent of wave setup on beaches. The review includes work by Hansen (1978); Guza and Thorton (1981); Holman and Sallenger (1985); Nielsen (1988); Davis and Neilsen (1988); King et al (1990); Yanagishima and Katoh (1990); Greenwood and Osborne (1990); Hanslow and Nielsen (1993); Lentz and Raubenheimer (1999); Raubenheimer, Guza and Elgar (2001) and Stockdon et al (2006). These investigations were completed on a variety of different beach types throughout the world, including in the North Sea, Japan, USA and Australia.

Results from each of the different investigations show varying levels of wave setup for a variety of reasons, including measurement difficulties. However, each of the studies indicated that wave setup does occur in the nearshore area. In particular, findings from many of the studies show that the majority of this setup occurs on the beach face.

Dean and Walton (2008) determined that, as an average over all of the studies, the amount of wave setup was approximately 0.19 times the significant wave height (standard deviation of 0.09). Furthermore, many of the studies found that maximum wave setup values (as opposed to average) were often in the order of half the breaking wave height.

In order to determine the extent of nearshore setup that would occur, a cross-shore profile model, SBEACH, was used to complete a simulation of the potential wave setup. The result suggests that the nearshore water level setup could be in the order of 0.7 to 1.0 m. This is consistent with the findings of the investigations into wave setup, which suggest that maximum setup levels could be in the order of around 0.2 to 0.5 times the breaking wave height, which is estimated to be around 2.5 m inshore at the study area.

4.1.9 Cyclonic Storm Surge Inundation Modelling Results

The top 106 events generated by the MCMC model were simulated using the calibrated Delft cyclone model. The results of the model simulations were then interrogated in order to extract the peak water level for each event at Geraldton. Resulting water levels were ranked according to inundation level and an extreme analysis was completed in accordance with the method outlined in Petrauskas & Aagaard (1971).

The extent of coastal inundation is sensitive to the local coastal geology/geomorphology, with shoreline orientation and prominent shoreline features potentially having a significant impact on the extent of inundation. This is particularly relevant for the Geraldton region, as at this latitude cyclones are generally decaying, meaning that the exposure to severe wind conditions generally reduces with increased distance further south. The prominence of Point Moore as a shoreline feature therefore results in a barrier to wind induced water movement along the coastline.

Typical cyclone wind conditions, which are most severe from the north through west, therefore lead to increased storm surge within Champion Bay on the northern side of Point Moore, while on the southern side of Point Moore the storm surge level is reduced. An example of this is shown in Figure 4.22, which is a spatial plot from the model of a severe cyclone event. The figure shows that under a north-westerly wind regime, as typically encountered with an approaching cyclone, the storm surge is higher on the northern side of Point Moore than it is on the south. This is further illustrated in Figures 4.23a and 4.23b, which present spatial plots showing the passage of the synthesised 500 year ARI event.

Given the above, the storm surge inundation levels over the study area are less than those encountered to the north of Point Moore. The results of the extreme analysis for cyclonic storm surge inundation along the study area were presented in Table 4.1 for the 20, 100 and 500 year ARI events in four key planning timeframes: present day, 2030, 2070 and 2110.

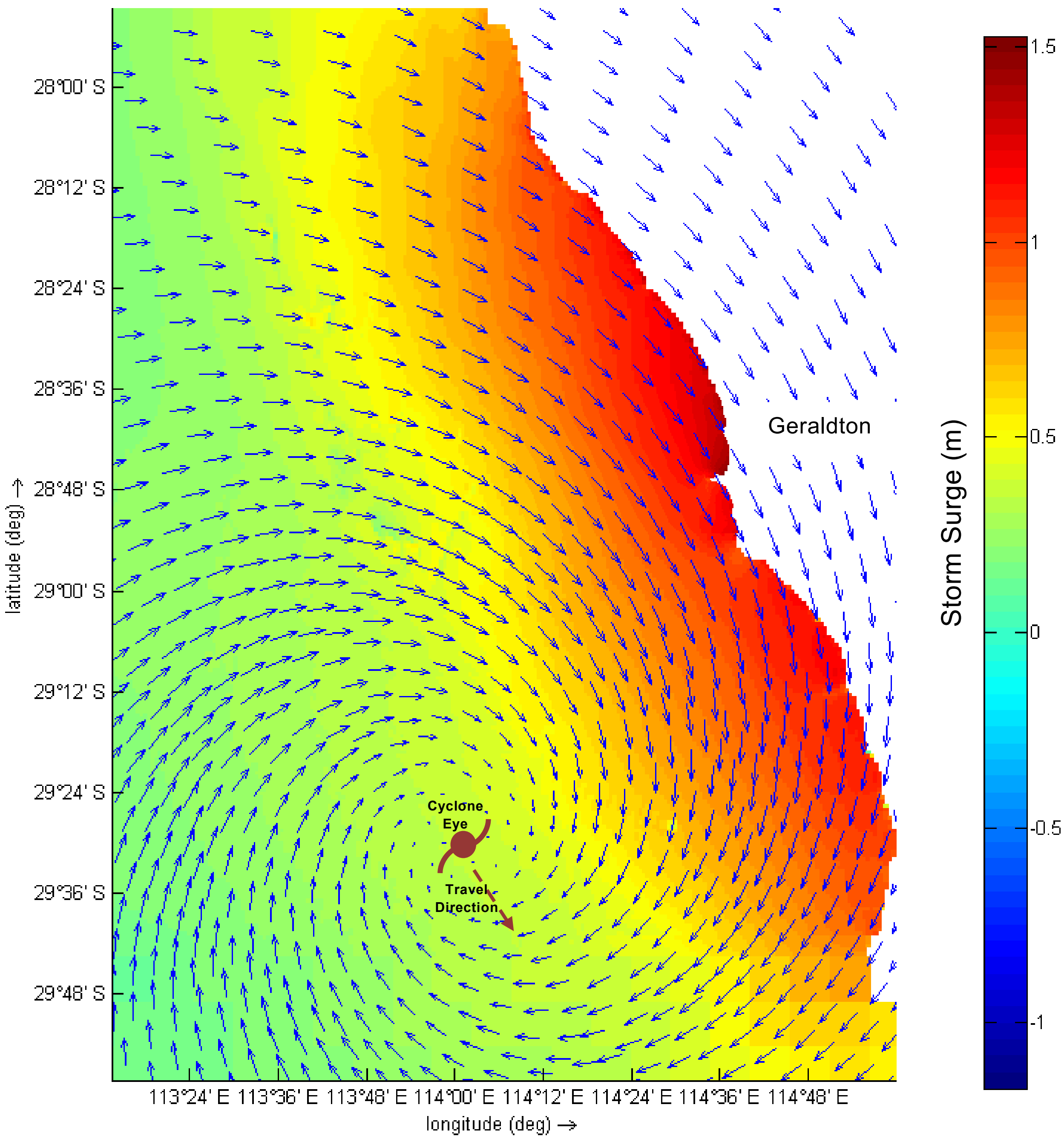


Figure 4.22 Delft3D output plot showing a typical cyclone wind field while tracking to the west of Geraldton

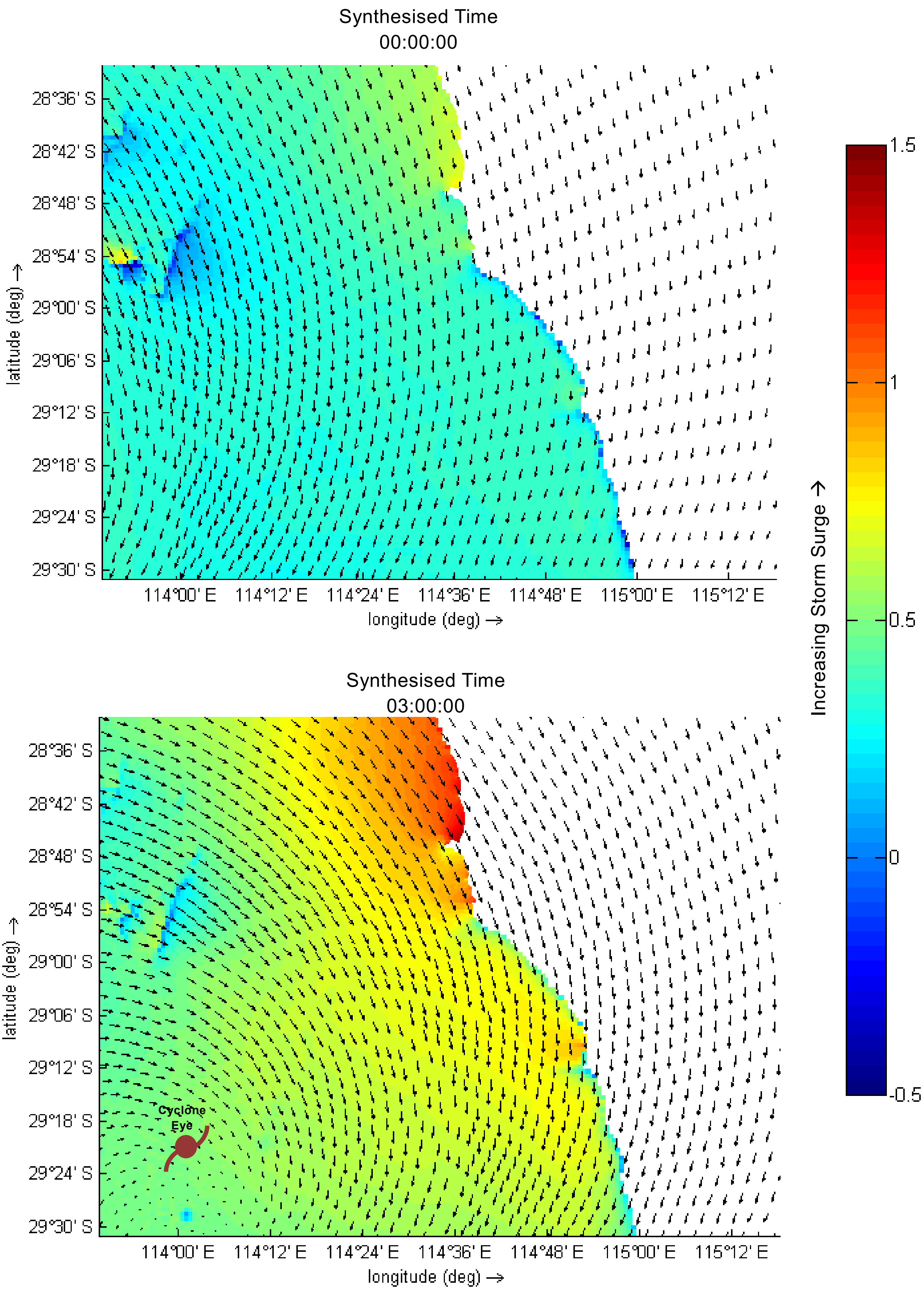


Figure 4.23a Delft3D Output Plots showing the Passage of the Synthesised 500 year ARI Event for the Study Coastline

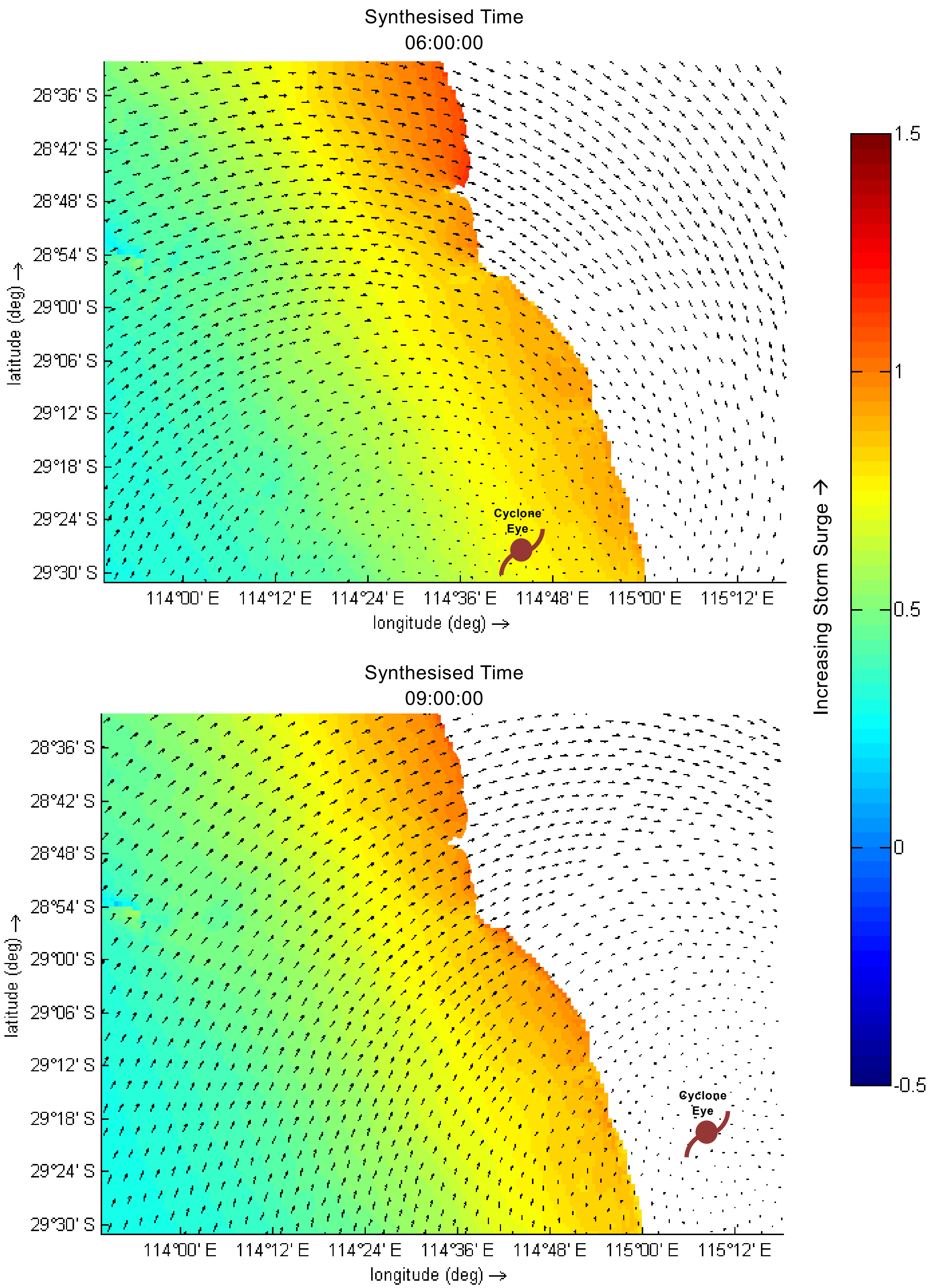


Figure 4.23b Delft3D Output Plots showing the Passage of the Synthesised 500 year ARI Event for the Study Coastline

4.2 Non Cyclonic Storm Surge Assessment

The non-cyclonic storm surge was assessed using data from the tide gauge at Geraldton Port. The tidal gauge data was interrogated to filter out measurements that correspond to the passage of tropical cyclones. An extreme analysis was then carried out on the filtered water level data to estimate the non-cyclonic inundation levels. The result of this assessment is presented in Table 4.3. Details of this assessment are discussed in the following sections.

Table 4.3 Estimated Non-Cyclonic Inundation Levels

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)
Present Day	0	2.0	2.1
2030	0.07	2.1	2.2
2070	0.39	2.4	2.5
2110	0.90	2.9	3.0

Note: 1. Inundation Levels have included sea level rise for each timeframe as presented in Table 3.1.

4.2.1 Supplied Water Level Data

DoT measures the water level at Geraldton Port through a tide gauge. The measured water level data was provided to MRA.

Details of the supplied water level data are provided in Table 4.4.

Table 4.4 DoT supplied Water Level Data Details

Start	Finish	Frequency
January 1966	December 1986	1 hour[1]
April 1986	December 1999	15 minutes
January 2000	July 2016	5 minutes

Notes: 1. DoT note that the accuracy cannot be guaranteed (refer Section 4.2.5).

4.2.2 Astronomical Tides

DoT have prepared a submergence curve from measurements at Geraldton Port, including description of the relevant astronomical tidal levels in Geraldton. The key tidal levels are summarised in Table 4.5.

Table 4.5 Geraldton Tidal Levels (from DoT Submergence Curve)

Tidal Plane	Prefix	Chart Datum (mCD)	Australian Height Datum (mAHD)
Highest Astronomical Tide	HAT	1.20	0.65
Mean Higher High Water	MHHW	0.82	0.27
Mean Sea Level	MSL	0.57	0.02
Mean Lower Low Water	MLLW	0.33	-0.22
Lowest Astronomical Tide	LAT	0	-0.55

The values in Table 4.5 describe the general changes in water level at the site due to astronomical tides. The general astronomical tidal range is described by HAT and LAT, between approximately -0.55 and 0.65 mAHD.

4.2.3 Wave Setup

As discussed in Section 4.1.8, the findings of the investigations show that the majority of wave setup occurs on the beach face, this wave setup is not expected to be included in the water levels that have been recorded at the Geraldton Tide gauge, which is located within the Geraldton Port. This is due to the fact that the water level records within the Geraldton Port have been recorded within waters that are sheltered from wave breaking effects, particularly those on a beach face. As a result, these recorded water levels would not include the nearshore wave effects. The effects of nearshore wave setup should therefore be added to the extreme water level determined from the Geraldton tide gauge records to provide a reasonable estimate of the peak steady water levels at the site. The result of the SBEACH simulations from Section 4.1.8 was also adopted in this assessment.

4.2.4 Identification of Extreme Events

To ensure that only the non-cyclonic water levels were used for the extreme analysis, the raw data was interrogated to remove periods of measurements that correspond to the passage of tropical cyclones or extra-tropical cyclones in the Geraldton area.

In addition, to ensure that only individual events were identified, a 48 hour separation was also used between high water level events. This is demonstrated in Figure 4.24.

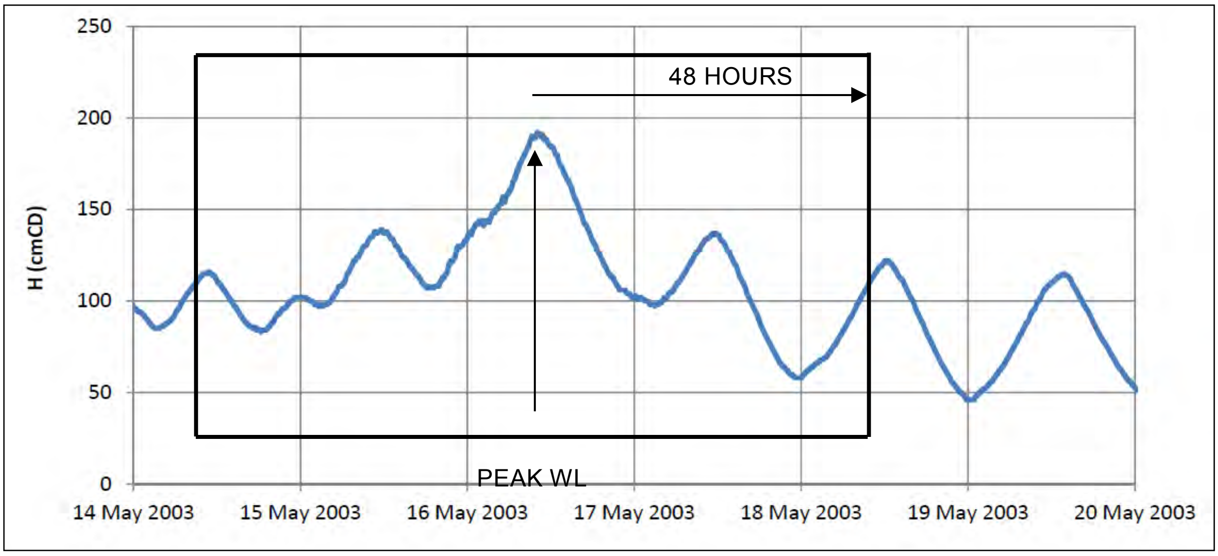


Figure 4.24 Separation of Individual High Water Level Events

As illustrated in Figure 4.24, water levels within the box are considered part of the same event and were not included in the analysis. The next water level that could be considered is 48 hours later (or earlier), which is located outside the box in the figure.

4.2.5 Extreme Analysis

An extreme analysis was subsequently completed on the filtered water level data. However, as part of this process it was noted that events from the 1966 to 1986 period were poorly represented within the list of highest observed water levels, with only 10 out of the top 40 events coming from this period. Upon further review of the data it appears that the 1-hourly data recording is too coarse to adequately capture the peak of the water level events and its use in the extreme analysis would therefore result in erroneously low extreme values. As a result, the 1-hourly data from 1966 to 1986 was excluded from the assessment and only the period between 1986 and July 2016 was considered. The highest 30 individual high water levels were extracted from this period. This correlates to an average of approximately 1 high water level event each year.

The results of the extreme water level analysis were presented previously in Table 4.3, for four key planning time frames: present day, 2030, 2070 and 2110. It should be note that the 500 year ARI event is not presented as it is not reliable given the short duration (30 years) of the reliable record compared to the recurrence interval (500 years) of the prediction.

4.3 Tsunami Induced Inundation Assessment

The Western Australian coastline experiences a relatively high frequency of tsunami occurrence, primarily due to its proximity to the zone of tectonic activity known as Sunda Arc, which skirts the southern edge of the Indonesian archipelago (Burbidge et al, 2008). Geoscience Australia has prepared a paper outlining the results of a probabilistic tsunami hazard assessment for Western Australia completed in 2008. The result of this assessment showed that the level of hazard is highest along the coast from Carnarvon to Dampier, while the hazard is much lower further south of Shark Bay.

Nevertheless, SPP 2.6 requires that an allowance for absorbing the current risk of inundation be adopted based on maximum inundation heights evidenced in tsunami prone areas. From interrogation of the water level records, the maximum tsunami induced water level in the Geraldton region reached 1.75 mAHD, during the Indian Ocean Tsunami on 26 December 2004.

Horspool et al (2010) investigated the impact of the 2004 Indian Ocean Tsunami on Geraldton, the tsunami signal (after the removal of the tidal fluctuation) at the tide gauge in Geraldton Port is presented in the following Figure. This study estimated a nearshore maximum wave height of around 1.2 m.

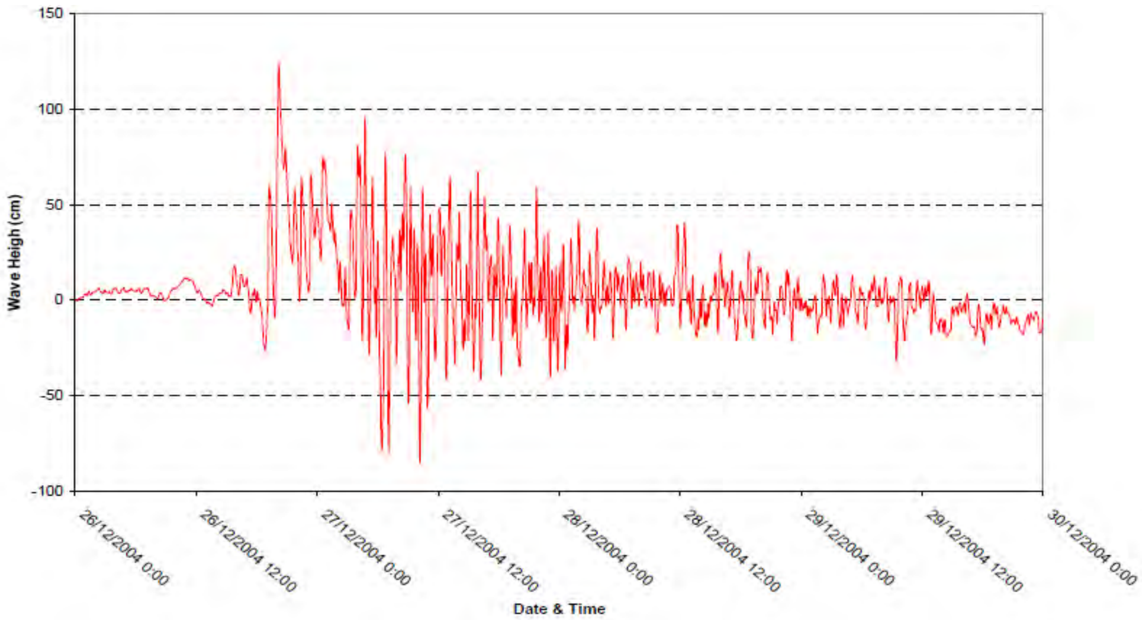


Figure 4.25 December 2004 Tsunami Signal (Source: Geoscience Australia, 2010)

To enable estimation of the recurrence interval of tsunami events, Burbidge et al (2008) completed tsunami hazard curves that correlate the offshore tsunami wave height (at 50 m water depth) to the tsunami recurrence interval. A number of hazard curves were completed along the Western Australia coast for a number of maximum earthquake magnitudes. The corresponding tsunami hazard curves for Geraldton is presented in Figure 4.26, where the different colour lines denote results from different models and earthquake magnitudes. The purple line shows the results from the preferred model for Western Australia.

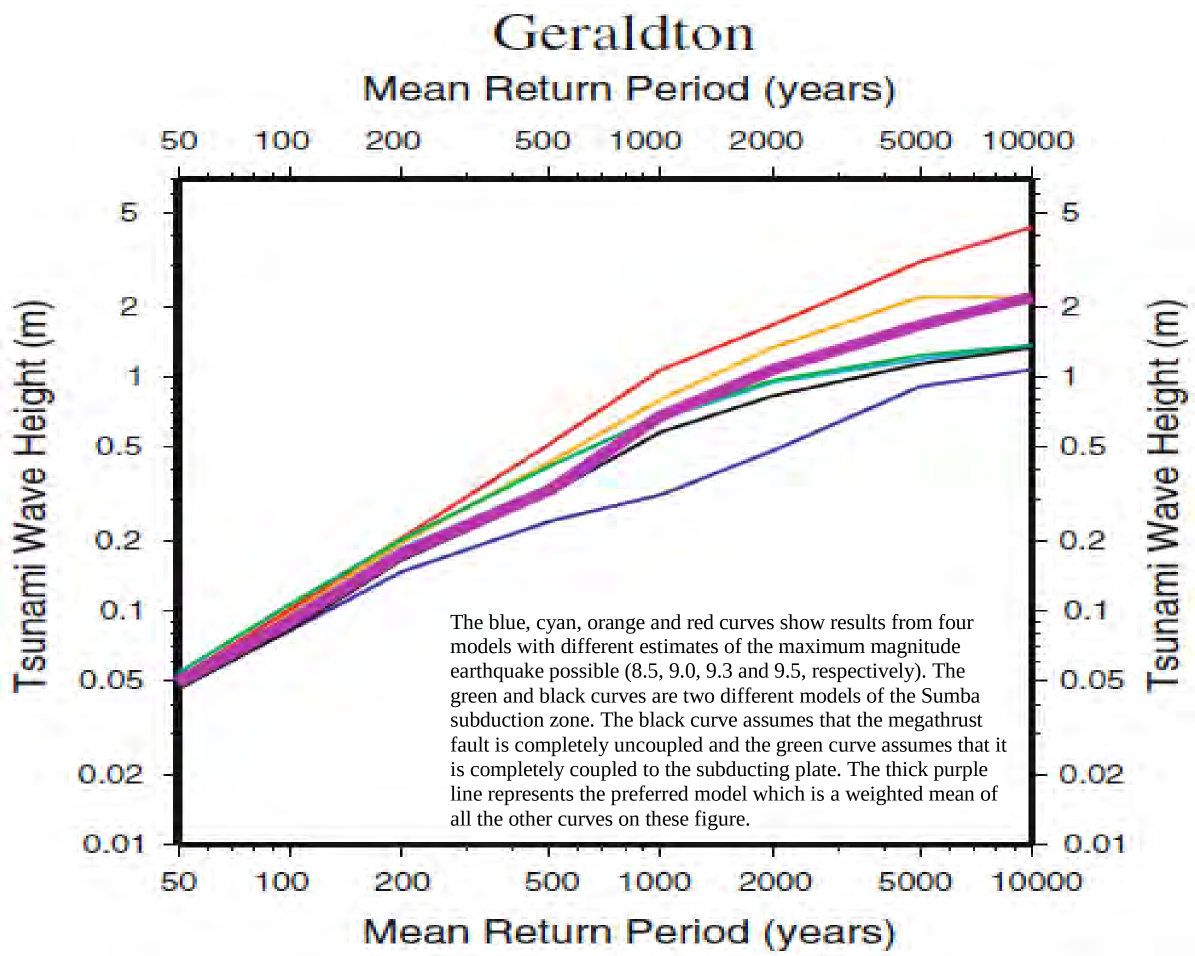


Figure 4.26 Tsunami Hazard Curves for Geraldton (Source: Geoscience Australia, 2008)

Based on the estimated nearshore wave height of the 2004 tsunami and the water depth at the Geraldton tide gauge, the offshore wave height (at 50 m water depth) of the 2004 tsunami can be estimated using an equation provided by the engineering manual 1110-2-1414 of the U.S. Army Corporations of Engineers (USACE), which estimate the increase of the tsunami's wave height as it enters shallow water.

$$\frac{H_s}{H_d} = \left(\frac{d_1}{d_2}\right)^{1/4}$$ (Source: USACE, 1989)

Where H_d and H_s are tsunami wave heights in deep and shallow water and d_1 and d_2 are the water depth at deep and shallow water respectively.

From the above equation, the offshore tsunami wave height during the 2004 Indian Ocean Tsunami was estimated to be in the order of around 0.7 m. Using Figure 4.26, this corresponds to a recurrence interval between 700 and 3000 years depending on the earthquake magnitude.

The above indicates that at Geraldton, the 2004 Indian Ocean Tsunami had an ARI of between 700 to 3000 years yet only resulted in a maximum inundation level of around 1.75 mAHD. This level is well below the predicted present day 500 year ARI storm induced inundation level.

Therefore, it is reasonable to provide no additional allowance to absorb the current risk of tsunami induced inundation.

4.4 Recommended Coastal Inundation Allowance

To determine the coastal inundation allowance for the study area, the cyclonic, non-cyclonic storm surge and potential for tsunami induced inundation have been assessed in the previous sections. Based on the results of the assessments the most critical inundation event for each recurrence interval was adopted. The recommended coastal inundation allowance for the area between Cape Burney and Greys Beach is presented in Tables 4.6.

The potential inundation levels outlined in Tables 4.6 should be considered as part of any coastal hazard risk management and adaptation planning in order to comply with the requirements of SPP2.6. Coastal hazard mapping showing the extent of inundation during the 20 year, 100 year and 500 year ARI event for present day, 2030, 2070 and 2110 are provided in Appendix A.

In addition to the assessment of the inundation levels, SPP2.6 also requires that where a continuous barrier dune is present, the capacity of the dune to provide protection from inundation should be assessed based on the cross sectional area of the dune. It is outlined that if the cross sectional area of a barrier dune above the peak steady water level is less than 100 cubic metres, it should be assumed that the dune will be removed during the storm activity and the maximum extent of storm inundation should be calculated without the dune. This assessment is based on recommendations outlined within the United States Federal Emergency Management Agency (FEMA) Guidelines for Coastal Flooding Analysis and Mapping (FEMA 2003).

To determine the capacity of the dunes within the study area, cross sectional areas of the dune reserve were assessed, with the extent of inundation shown on the hazard mapping reflecting the outcomes of this volumetric assessment.

Table 4.6 Recommended Coastal Inundation Allowance – Cape Burney to Greys Beach

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	2.0	2.2	3.0
2030	0.07	2.1	2.3	3.1
2070	0.39	2.4	2.6	3.4
2110	0.90	2.9	3.1	3.9

Note: 1. Inundation Levels have included sea level rise for each timeframe as presented in Table 3.1.

This concludes the coastal inundation assessment for the study area. Further spatial detail regarding the extent of inundation during the 20 year, 100 year and 500 year ARI events for present day, 2030, 2070 and 2110 are provided in Appendix A. The combined potential impact of coastal inundation and coastal processes is discussed in Section 7.

5. Coastal Processes Allowances

5.1 Acute Storm Erosion Allowance (S1)

Severe storm events have the potential to cause increased erosion to a shoreline, through the combination of higher, steeper waves generated by sustained strong winds, and increased water levels. These two factors acting in concert allow waves to erode the upper parts of the beach not normally vulnerable to wave attack.

If the initial width of the surf zone is insufficient to dissipate the increased wave energy, this energy is often spent eroding the beach face, beach berm and sometimes the dunes. The eroded sand is transported offshore with the return water flow to form offshore bars. As these bars grow, they can cause incoming waves to break further offshore, decreasing the wave energy available to attack the beach. This is shown diagrammatically in Figure 5.1.

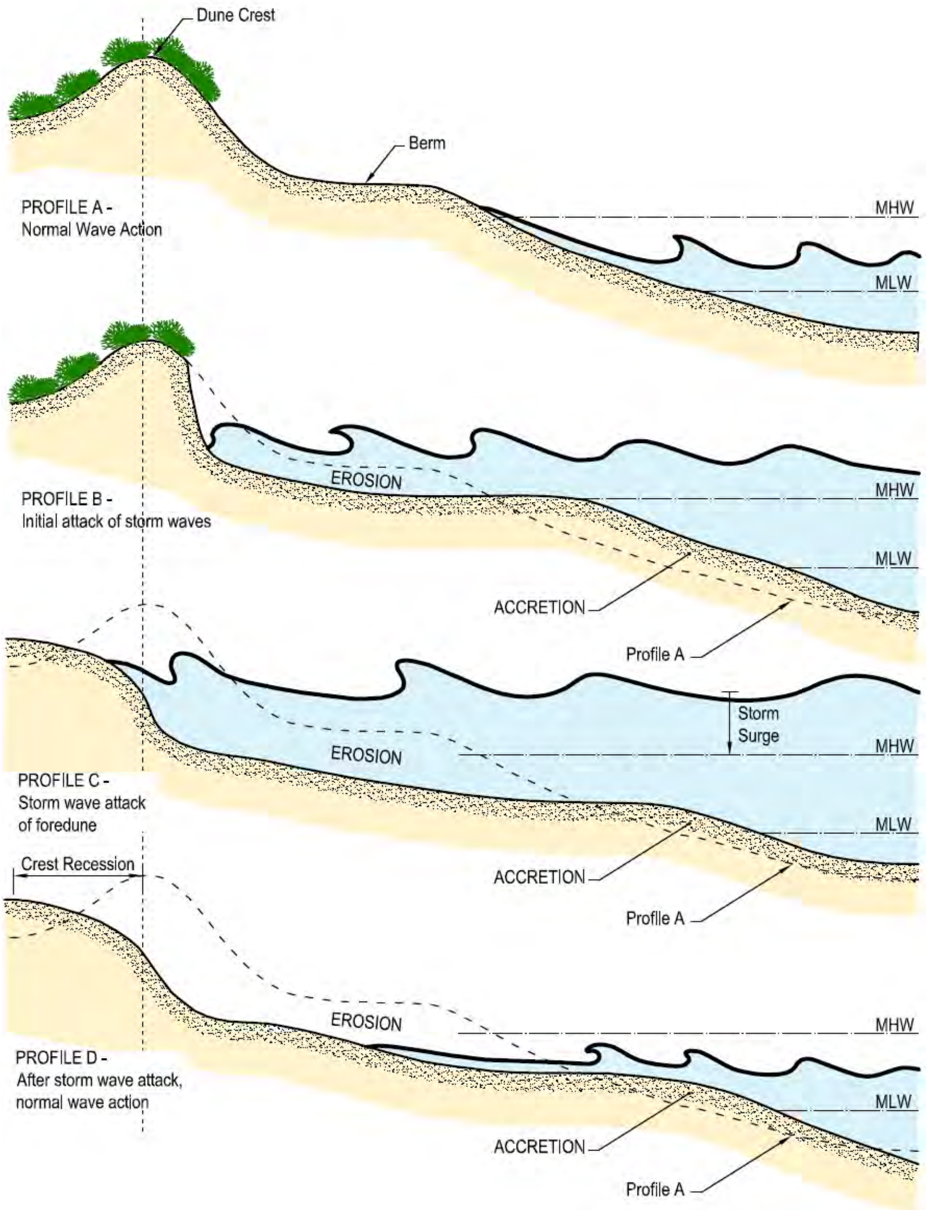


Figure 5.1 Storm Erosion Process (source: CERC 1984)

SPP2.6 recommends that the allowance for absorbing acute erosion be determined using a credible sediment transport model (WAPC 2013). The model should be used to determine the potential shoreline erosion resulting from a storm event with a 1 in 100 annual encounter probability (AEP). This is equivalent to a 100 year Average Recurrence Interval (ARI) event. It is generally accepted that simulation of three repeats of a severe storm sequence experienced along the south west of Western Australia in July 1996 provides a reasonable approximation of the 100 year ARI event for beach erosion. This storm had elevated water levels for a period of approximately 110 hours and caused coastal erosion at a number of locations in Western Australia. Details of the storm conditions modelled to represent three repeats of this severe storm event are provided in Figure 5.2.

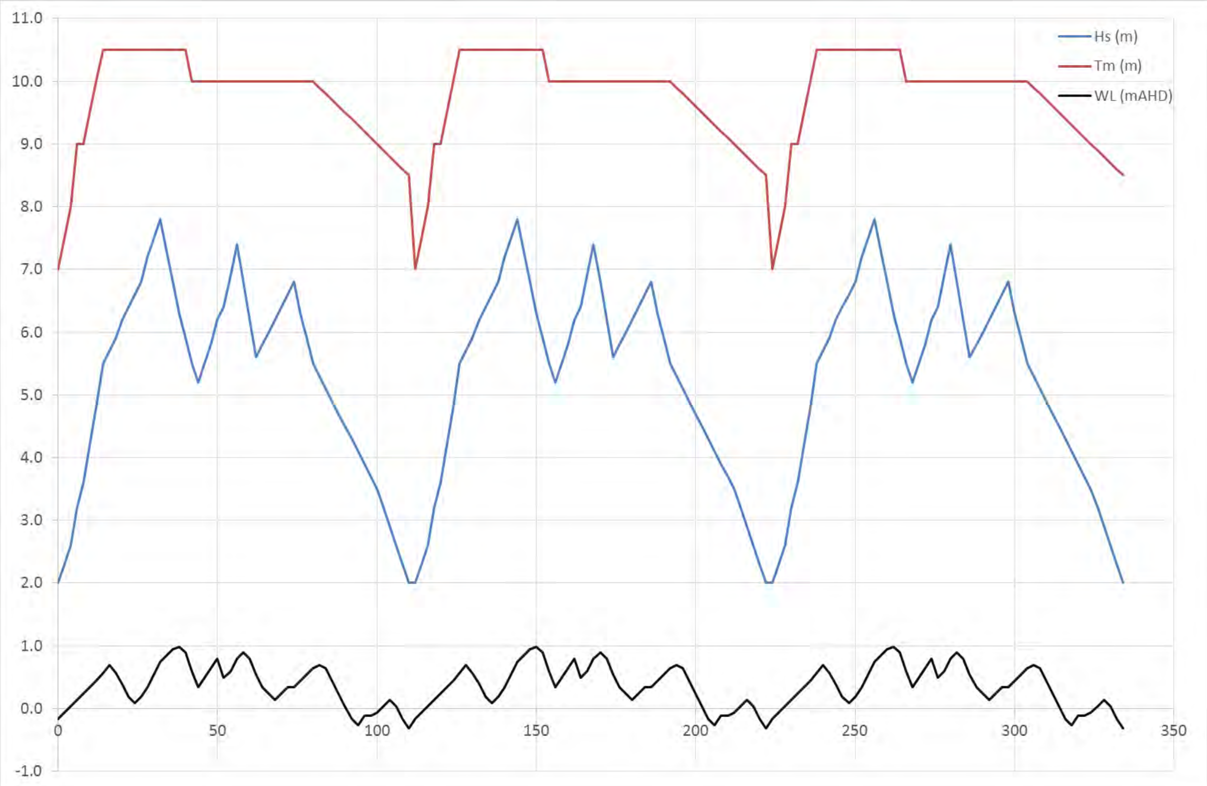


Figure 5.2 July 1996 Storm Conditions for use in Storm Erosion Modelling

To provide a more robust assessment, the synthesised cyclone event that resulted in the 100 year ARI inundation was also simulated. This event had elevated water levels for a period of approximately 64 hours.

The SBEACH computer model was developed by the Coastal Engineering Research Centre (CERC) to simulate beach profile evolution in response to storm events. The SBEACH model has been extensively used for storm erosion modelling within Western Australia, and has been proven to be a credible model for this purpose. It is described in detail by Larson & Kraus (1989). Since this time the model has been further developed, updated and verified based on field measurements (Wise et al 1996, Larson & Kraus 1998, Larson et al 2004).

SBEACH has also been validated in Western Australia by MRA, with results outlined in Rogers et al (2005). This local validation showed that SBEACH can provide useful and relevant predictions of the storm induced erosion provided the inputs to SBEACH, which include time histories of wave height, period and water elevation, as well as pre-storm beach profile and median sediment grain size, are correctly applied and care is taken to ensure that the model is accurately reproducing the recorded wave heights and water levels.

A limitation of the SBEACH model is that it is a single profile model and therefore cannot account for spatial changes of waves and water levels over complex bathymetry. As a result, the complex nearshore reef systems (Point Moore Reef and Southgate reef) around the study area necessitates the use of a more robust wave modelling system that adequately resolves the wave transformation into the nearshore area. Results from the detailed wave modelling have been used as inputs to the SBEACH modelling.

5.1.1 Detailed Wave Modelling

To determine the nearshore wave conditions within the study area to be adopted for SBEACH modelling, detailed wave modelling has been completed. For this analysis the calibrated Delft3D wave model was used.

The offshore wave conditions during the July 1996 storm were recorded by the Waverider buoy, located in 48 metres of water south-west of Rottnest. These offshore wave conditions were then input into the Delft3D model.

To determine the critical nearshore wave conditions, several wave cases were modelled. This included simulation of waves propagating from south, south west, west and northerly directions. The most critical wave conditions experienced at different sections of shoreline were extracted from the Delft3D model and used for SBEACH modelling. A spatial plot of a typical south west wave condition at the study site is provided in Figure 5.3.

The nearshore wave conditions for the 100 year ARI cyclonic storm surge inundation event were extracted from the modelling results of the inundation assessment.

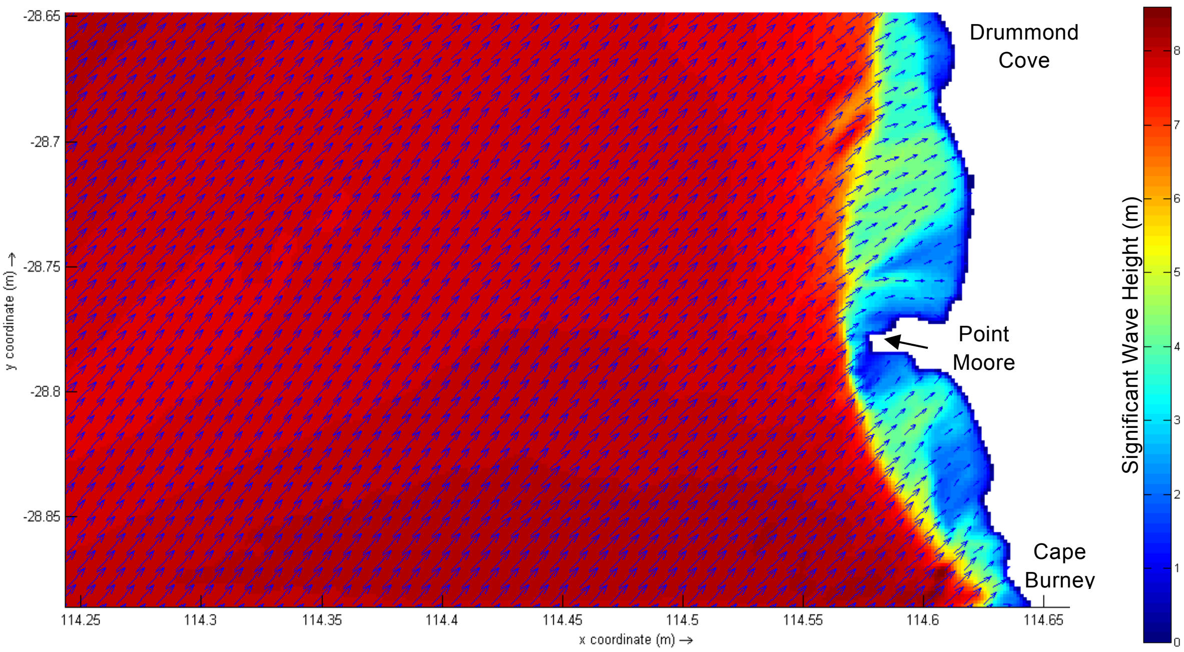


Figure 5.3 Modelled South West Wave Conditions

5.1.2 SBEACH Modelling

To simulate the shoreline response to the storm described above, nine profiles were developed. These profiles were used to investigate the response of different sections of beach to the design storm. The representative beach profiles were taken based on the exposure, aspect and characteristics of the beaches along the study areas (as shown in Figure 5.4). The profiles were compiled from hydrographical survey plans and available topographic LIDAR survey (completed in 2009) of the area. It should be noted a new bathymetric Lidar survey was completed in 2015 and was made available after the completion of this work. It is recommended that this data be incorporated into future studies.

The water levels for the July 1996 storm were recorded in the south west of Western Australia and are representative of the water levels experienced in nearshore waters of 5 m depth. These water levels peak at around 1.0 mAHD, which is equivalent to a return period of around 1 to 2 years for Fremantle. The water level record from the Geraldton region indicates that the water level for a similar return period is around 1.1 mAHD. Therefore, the water levels recorded during the July 1996 storm were increased by 0.1 m before being used in the SBEACH model.

The nearshore wave conditions from the July 1996 storm were extracted from the Delft3D model. Three repeats of the extracted wave conditions were run for a combined total of approximately 330 hours for each profile.

For the 100 year ARI cyclonic inundation event, the water levels and nearshore wave conditions were extracted from the inundation model and inputted into the SBEACH model.

From the results of the SBEACH modelling, the July 1996 storm resulted in more erosion compared to the 100 year ARI cyclonic inundation event. This is mainly attributed to the extended duration simulated for the 1996 storm (330 hours) as compared to 64 hours for the 100 year ARI cyclonic event. Therefore, the July 1996 storm was adopted in the assessment of the storm erosion allowance.

Figure 5.4 Beach Profiles Location for Study Areas

SPP2.6 requires that the allowance for severe storm erosion be calculated by determining the extent of erosion predicted behind the HSD. The HSD is defined as the landward contour corresponding to the peak water level elevation that is experienced during severe storm activity at the site. It should be noted that the HSD is defined based on present day water levels in order to provide a baseline to which the potential erosion extent can be determined over the planning horizon.

SPP2.6 also recommends that for steeply sloping sandy coasts, the distance for absorbing the risk of erosion should extend to the crest of the stable post storm shoreline slope. It is recommended that a 30 degrees slope from the horizontal be adopted for a typical sandy shoreline. This has been adopted in this assessment for the calculation of the S1 allowance for slope steeper than 30 degrees.

Table 5.1 summarises the extent of erosion, landward of the HSD, that could occur at each location during the prescribed severe storm sequence. Model outputs are shown in Figures 5.5 to Figure 5.13. These figures show the initial (pre-storm) profile, final profile and the maximum wave heights and water levels predicted during the storm.

Table 5.1 Acute Storm Erosion Allowance (S1)

Location	S1 Allowance (m)
Northern Greenough River	27
Southern Southgate Dunes	12
Western Southgate Dunes	27
Northern Southgate Dunes	23
Southern Tarcoola Beach	29
Northern Tarcoola Beach	28
Back Beach	17
Separation Point	25
Greys Beach	28

Profile 1

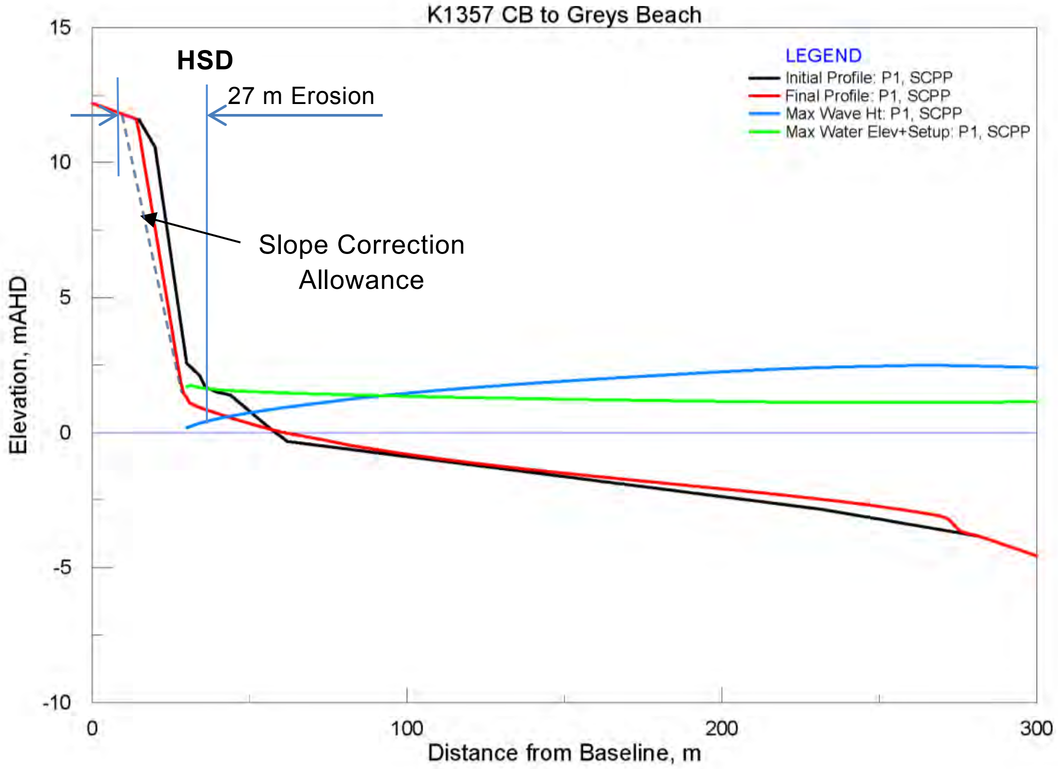


Figure 5.5 SBEACH Simulation Results Profile 1 (Northern Greenough River)

Profile 2

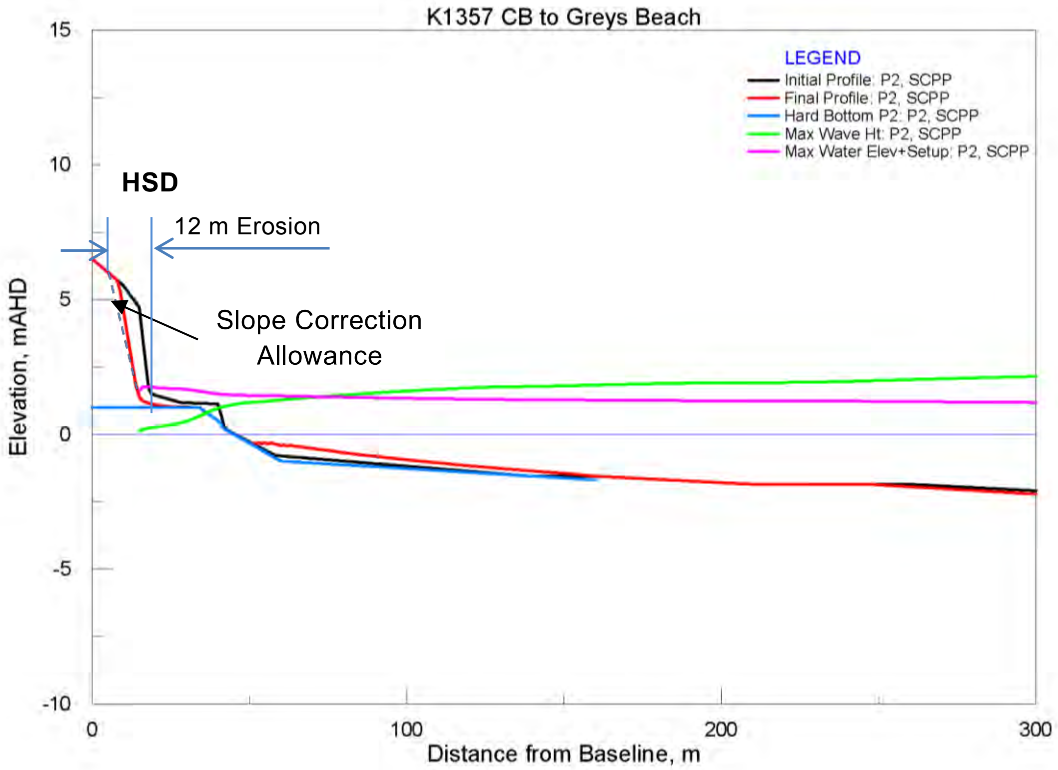


Figure 5.6 SBEACH Simulation Results Profile 2 (Southern Southgate Dunes)

Profile 3

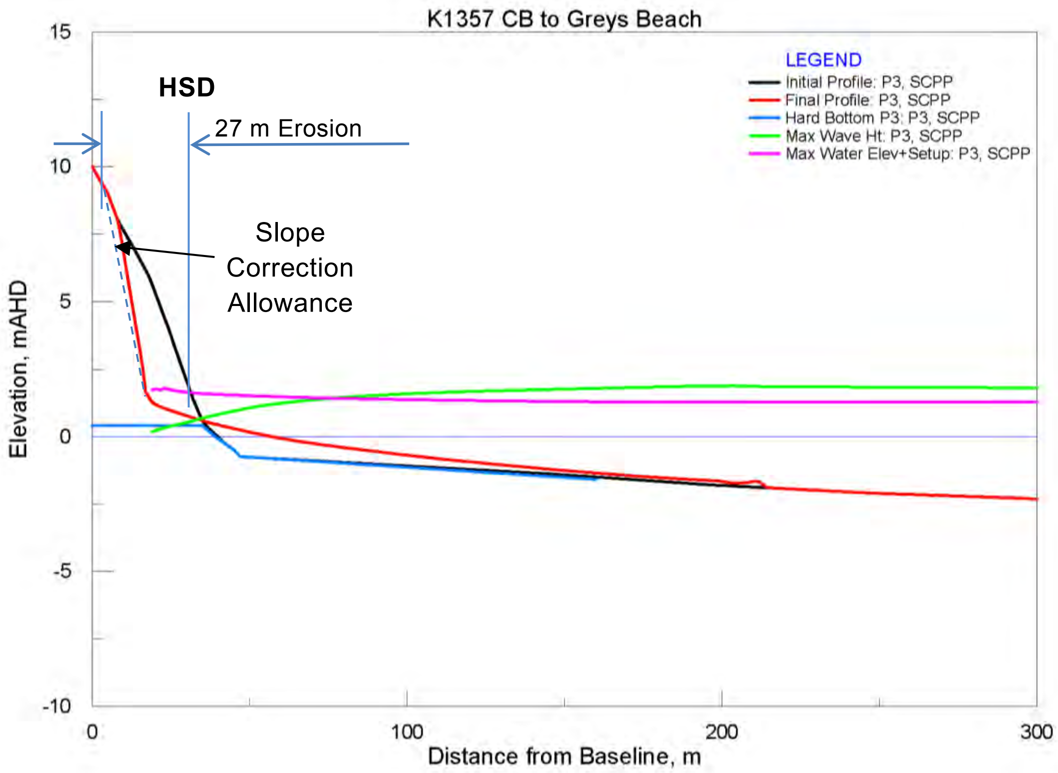


Figure 5.7 SBEACH Simulation Results Profile 3 (Western Southgate Dunes)

Profile 4

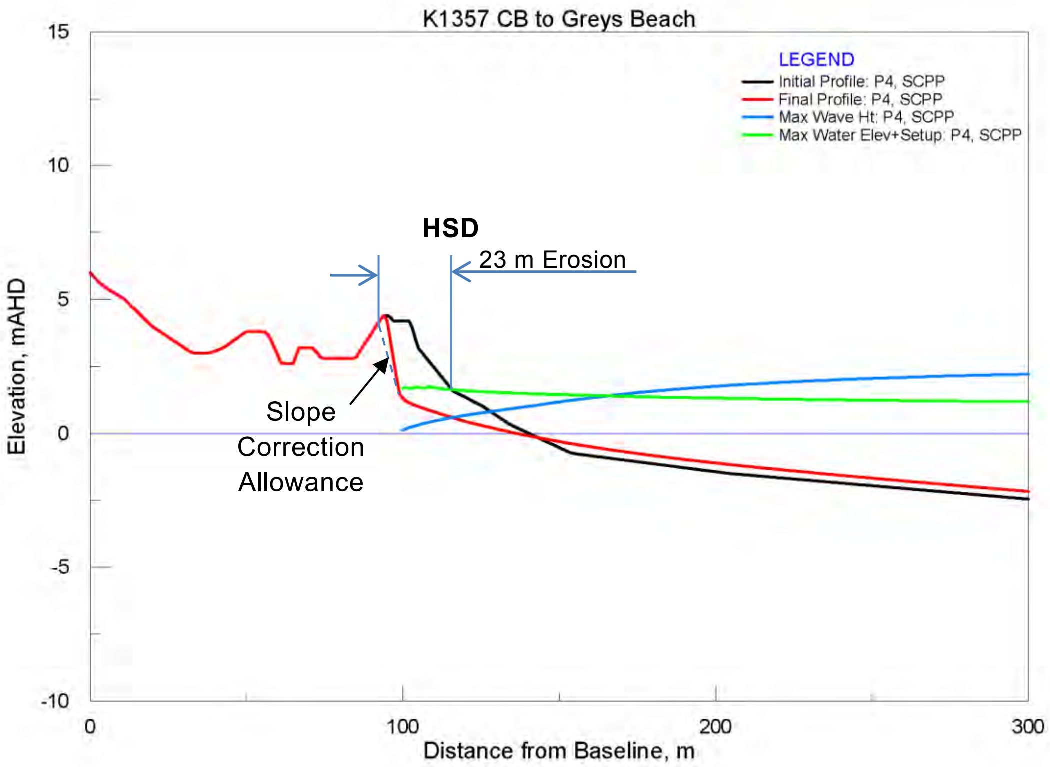


Figure 5.8 SBEACH Simulation Results Profile 4 (Northern Southgate Dunes)

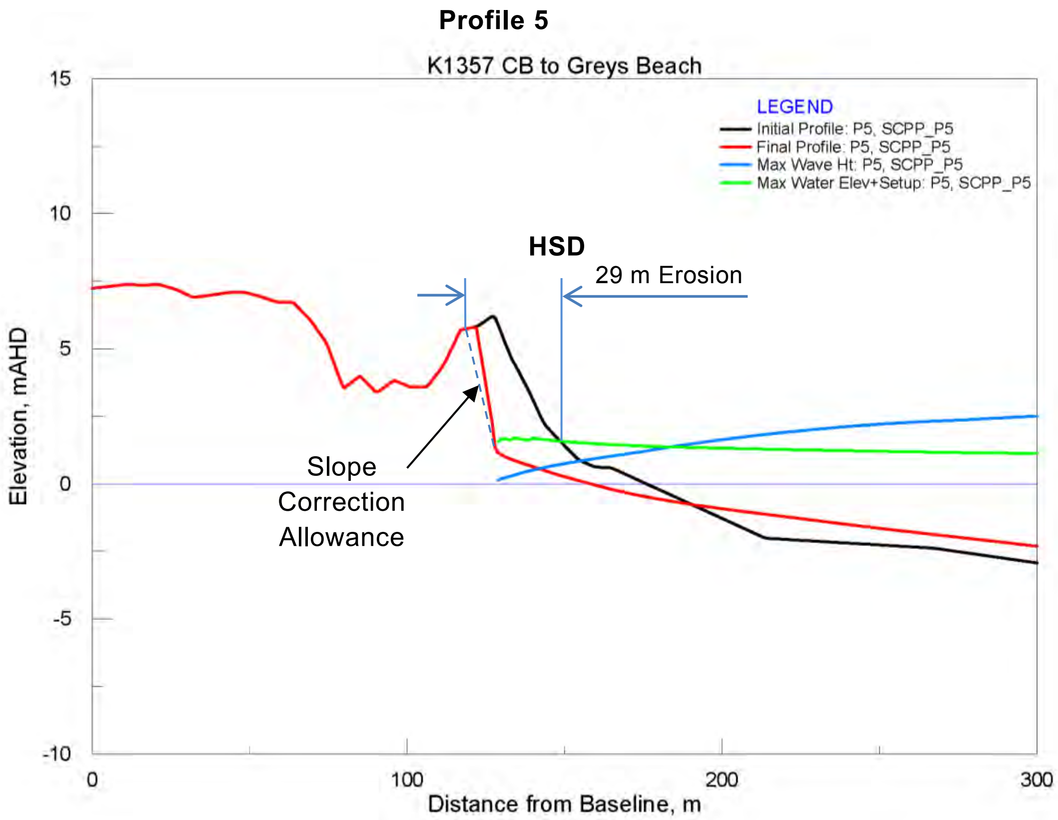


Figure 5.9 SBEACH Simulation Results Profile 5 (Southern Tarcoola Beach)

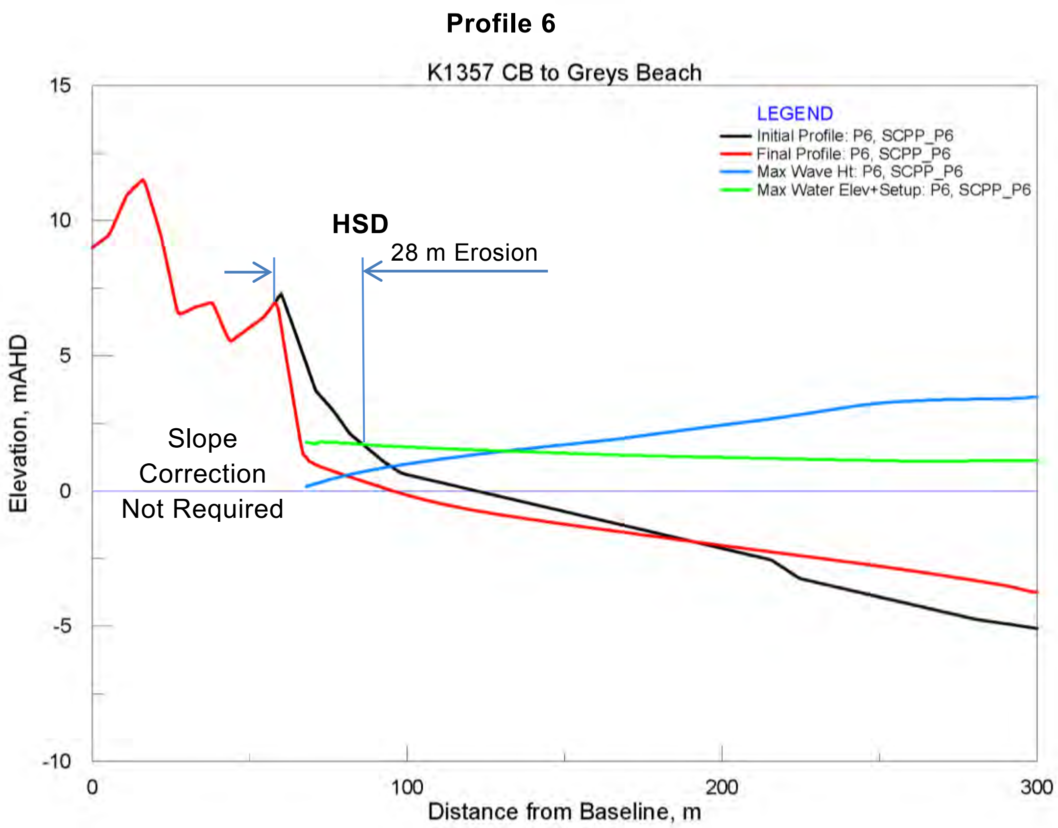


Figure 5.10 SBEACH Simulation Results Profile 6 (Northern Tarcoola Beach)

Profile 7

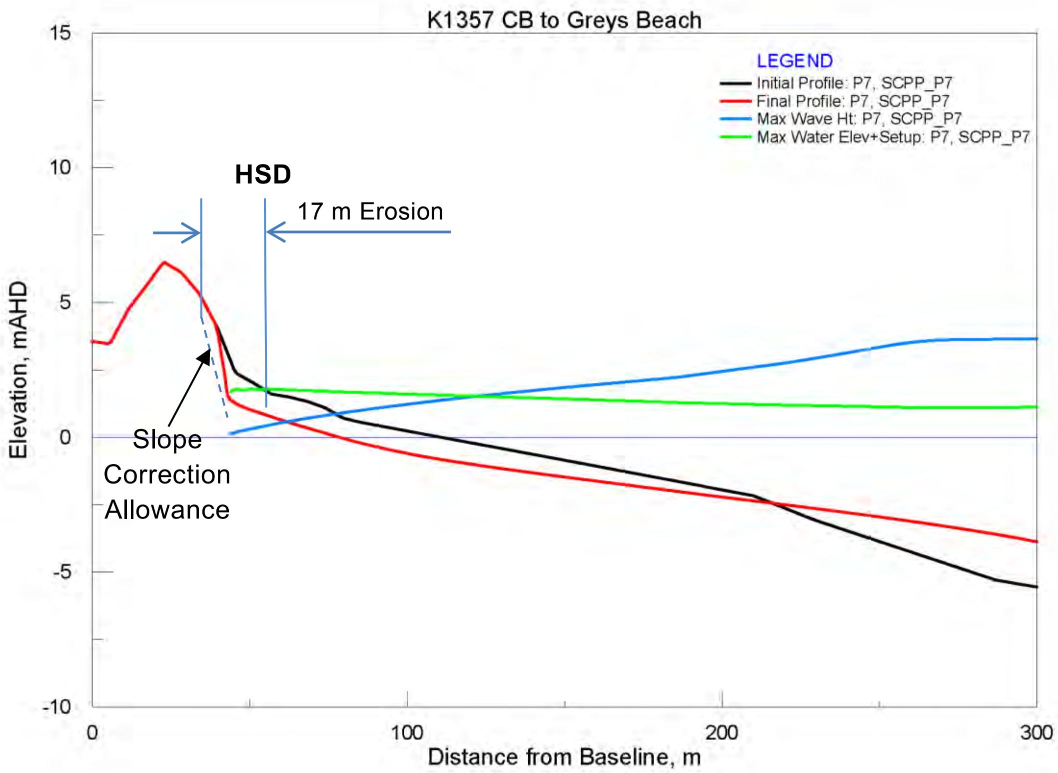


Figure 5.11 SBEACH Simulation Results Profile 7 (Back Beach)

Profile 8

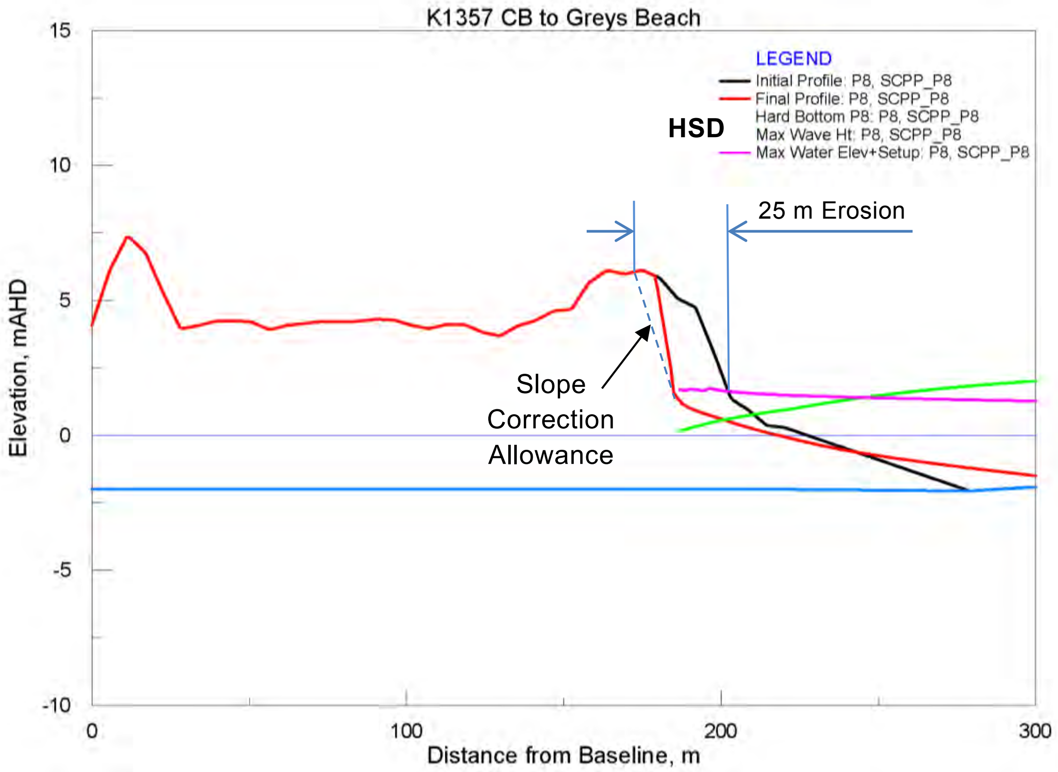


Figure 5.12 SBEACH Simulation Results Profile 8 (Separation Point)

Profile 9

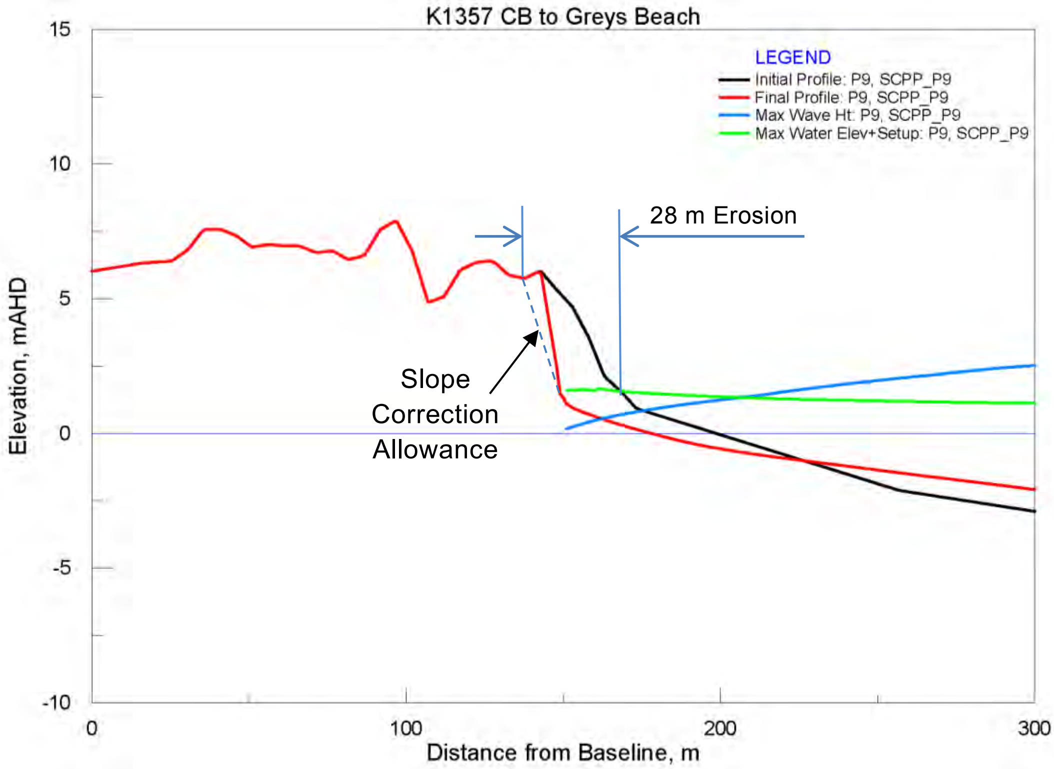


Figure 5.13 SBEACH Simulation Results Profile 9 (Greys Beach)

5.2 Allowance for Shoreline Movement Trend (S2)

Physical coastal processes act on wide ranging time scales, from storm to post storm, seasonal and longer term. The continual action of these processes helps to shape the shoreline.

By monitoring changes in the shoreline over time information can be obtained regarding the net dynamics of an area. Historical aerial photography is therefore used to plot the movement of the shoreline through recent history. In order to examine the long term shoreline movement at the study area, aerial imagery was obtained dating back to 1942, with the most recent aerial image from 2015.

Shoreline positions were mapped from the rectified photography using the methodology outlined in DoT (2009). The relative movement of the shoreline was therefore determined over a 73 year period for the shoreline from Cape Burney to Greys Beach. In general, the position of the vegetation line was used to determine the shoreline movement that has occurred through time; however, as shown in Figure 5.15, the presence of the Southgate Dunes system has eliminated the coastal vegetation in some locations. In these instances, the position of the shoreline was used to estimate shoreline movements. The accuracy of the coastal vegetation lines is believed to be approximately $\pm$ 5 metres in the horizontal plane, while the use of the shoreline introduces the possibilities of errors associated with shoreline position being highly dependent on the tide level. Results determined using the shoreline should be used with appropriate caution.

From the shoreline movement plans, the relative movements of the coastal vegetation line were estimated at 200 m intervals along the study coast. These chainages are presented in Figure 5.15.

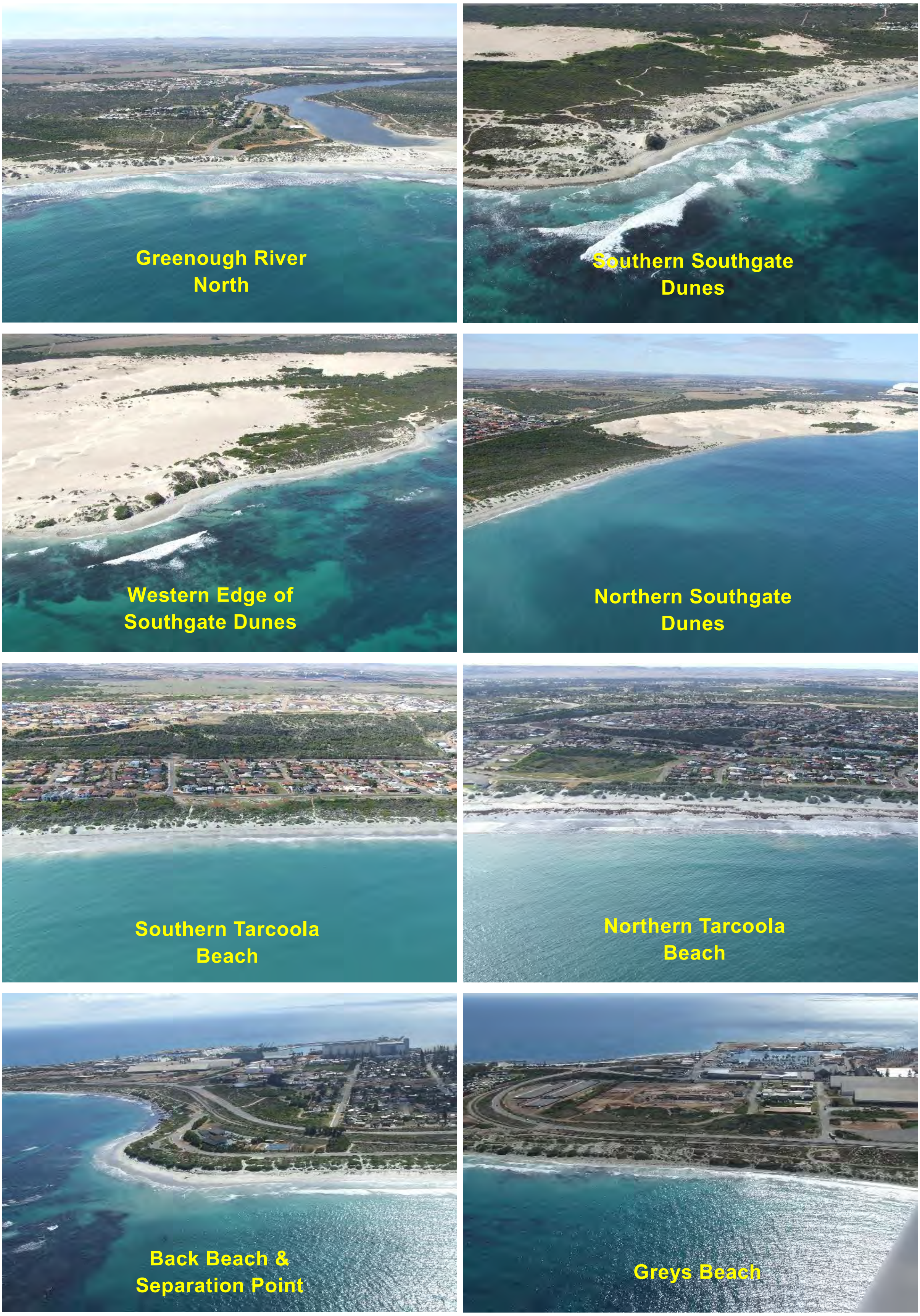


Figure 5.14 Oblique Aerial Photographs of Cape Burney to Greys Beach Shoreline

Figure 5.15 Chainages for Study Area

The movement plot of the shoreline from Cape Burney to Greys Beach relative to its 1942 alignment is presented in Figure 5.16.

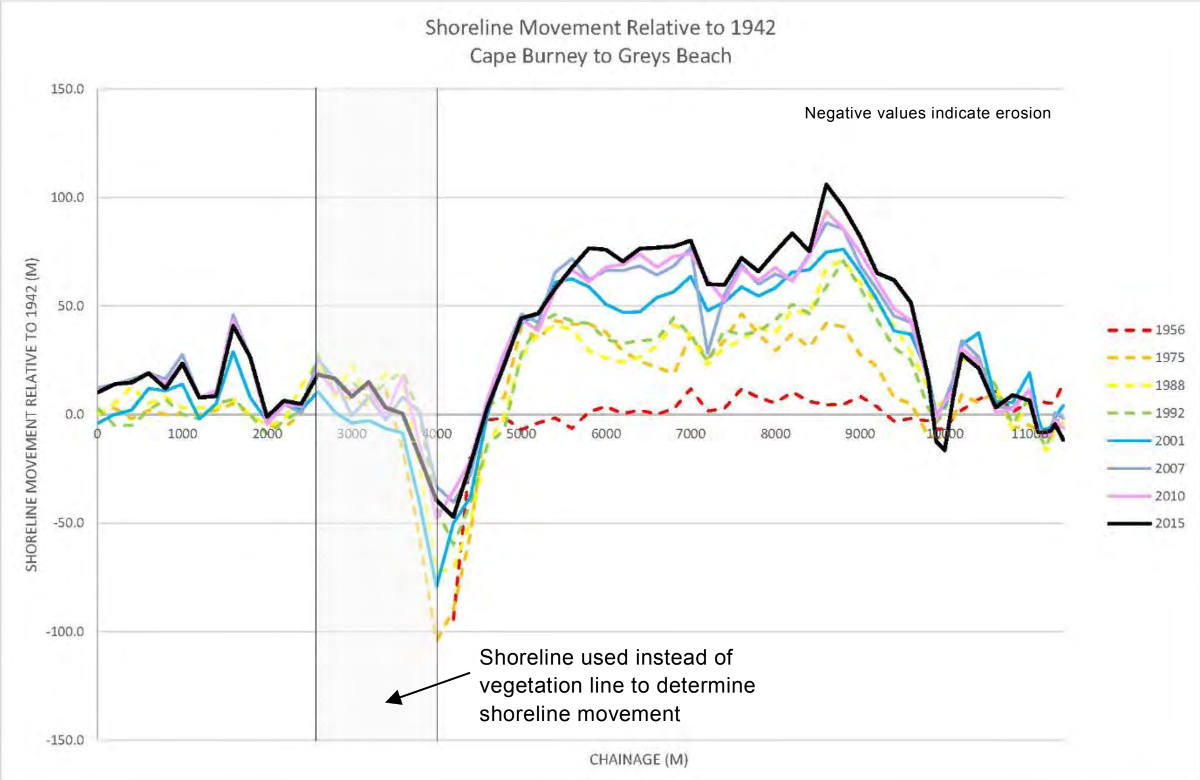


Figure 5.16 Relative Shoreline Movement of Study Shoreline Since 1942

The movement of the shoreline outlined in Figure 5.16 should be considered with regards to the local shoreline features and controls. The Southgate and Point Moore reef system provide protection to the majority of the study shoreline. This is supported by the shoreline movement plan which indicates that the shoreline from Cape Burney to Greys Beach has generally been accreting in the longer term, though there have been periods of minor fluctuation in shoreline position. Net accretion was observed between 1942 and 2015 for the majority of the study shoreline, except for the shoreline located at the northern Southgate Dunes (chainages 3,000 to 4,600 m), Separation Point (chainages 10,000 to 10,100 m) and western Greys Beach (chainages 11,100 to 11,400 m). These sections of shoreline have experienced net erosion, though the majority of this was observed prior to 1975, with accretion thereafter.

An indicative sediment budget presented in MRA (1998) provides some insight into this difference in shoreline behaviour for the study shoreline. This indicative sediment budget was prepared in response to observations of shoreline behaviour over the 1980's and 90's, and is presented in Figure 5.17.

This sediment budget suggests that a net sediment transport from south to north occurs along this section of coastline. This is consistent with the findings of Curtin University (2012) in their study of the Geraldton Embayments Coastal Sediment Budget.

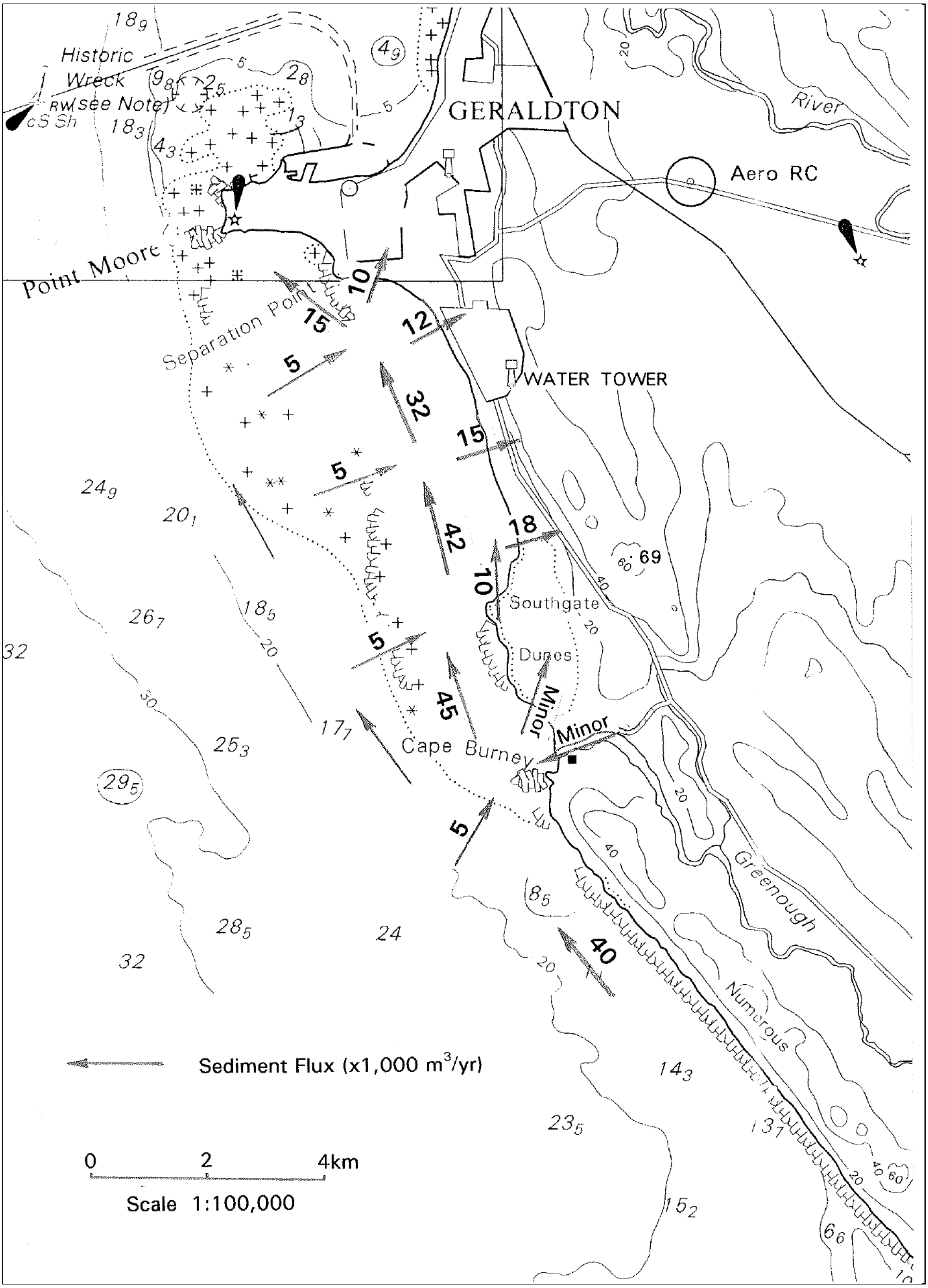


Figure 5.17 Indicative sediment budget from the 1980's and 90's (MRA, 1998)

The sediment budget indicates that this northerly sediment transport in combination with the offshore sand bars and the western Southgate Dunes area provide sediment feed into the study shoreline. This is consistent with the long term accretion trend shown in the shoreline movement plan.

The sediment budget also suggests that in the 1980's and 90's the shoreline in northern Southgate Dunes accreted at an average rate of around 10,000 m^3 per year. Nevertheless, the shoreline movement plan demonstrates that prior to this time the shoreline experienced significant erosion between 1942 and 1975. The reason for the change in shoreline behaviour may be attributed to the northerly migration of the Southgate Dunes and the feed of the sediment from these dunes into the littoral system.

MRA (2013) has previously completed a sediment feed analysis on the active Southgate Dunes using photogrammetric mapping techniques combined with assessment of wind records. The outcome of this assessment suggests that the Southgate Dunes have not always supplied sediment to the littoral system of the Tarcoola Embayment. It is likely that this sediment supply commenced around 50 to 60 years ago, and therefore the dunes in the "natural state" cannot be relied upon to provide sediment feed to the littoral system beyond the next 50 years. This assessment also estimated that the western dune area provides a sediment feed in the order of 31,000 to 37,800 m^3 per year into the littoral system, while the seaward edge of the northern dune area only contributes about 3,000 to 5,000 m^3 per year (around 10% of the total estimated sediment feed into the system) to the littoral system.

To provide a more detailed depiction of the shoreline movement over time, time history plots of shoreline positions relative to 1942 were produced for selected locations and are presented in Figure 5.18.

The time history plots generally show similar trends in shoreline movement. Time history plots of shoreline movement in this area indicate that long-term accretion has occurred, except for chainage 4,000 m and chainage 11,200 m, which showed a net erosion trend for the northern Southgate Dunes and Greys Beach shoreline up until 2015.

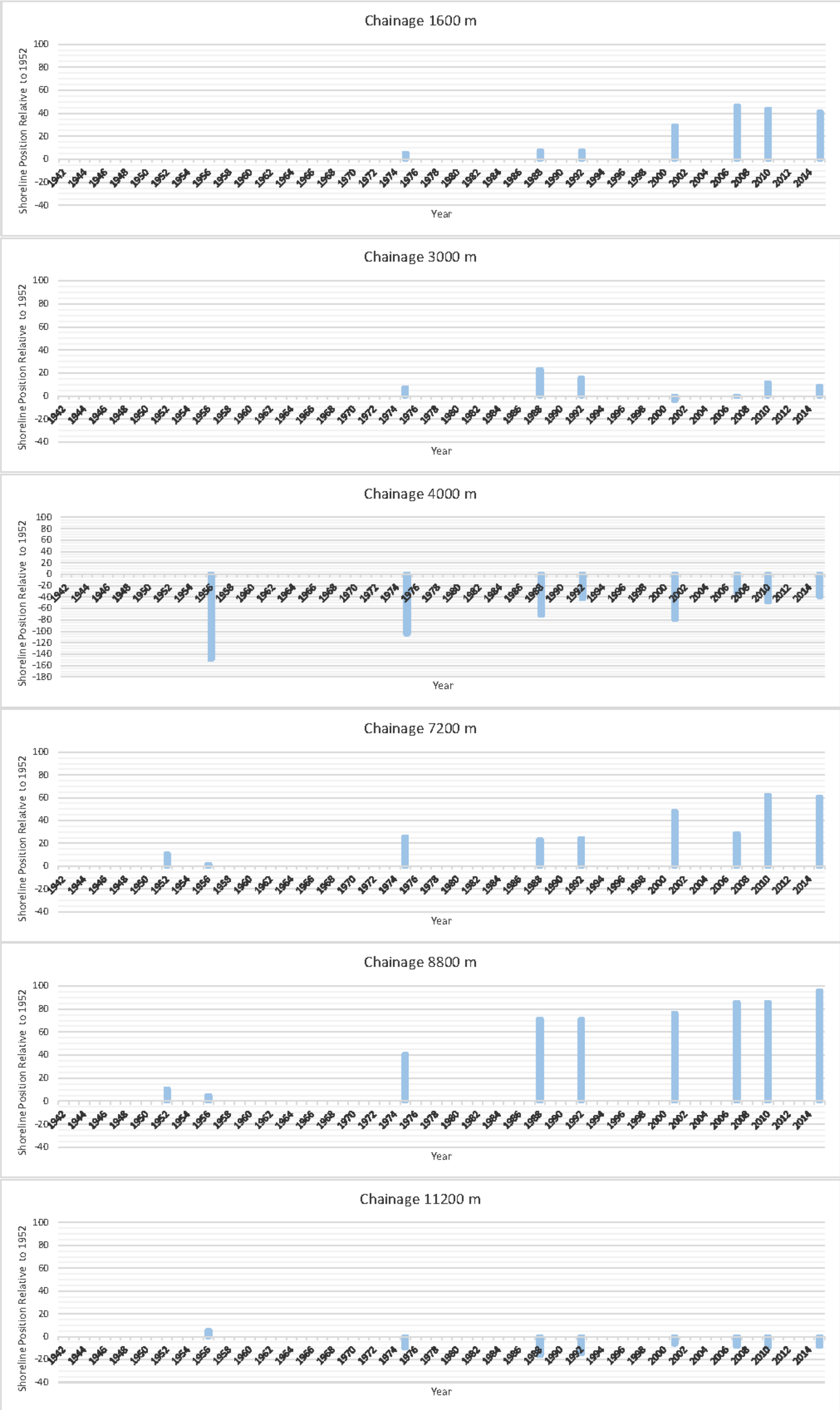


Figure 5.18 Time History Plots – Cape Burney to Greys Beach

The shoreline movement and time history plots for the study shoreline show that generally the trends in shoreline movement have been consistent over the period of record, a shoreline movement rate plot is provided in Figure 5.19. This figure shows the average shoreline movement observed over the periods from 1942 to 2015, 1956 to 2015 and 1975 to 2015.

The shoreline movement rate plot shows that generally the majority of the shoreline from chainage 0 to 2,400 m (Greenough River to southern Southgate Dunes) has experienced accretion between 0 and 0.2 m per year, with some shoreline positions accreting up to 0.56 m per year. SPP2.6 requires that an allowance of 0 m be adopted where historic annual rate of shoreline movement is accretion less than 0.2 metre per year. Therefore, a S2 allowance of zero should be taken for this section of shoreline.

The shoreline movement trend between chainage 2,400 m and 4,800 m has generally been consistent, with some slight erosion up to chainage 3,000 m. Beyond chainage 3,000 m, the shoreline has experienced a long term erosion trend, with an observed erosion rate of up to 0.72 m per year. It is noted that rock platform and inshore reefs are present in this area. However, since these reef and rock platforms are not continuous and a long term erosion trend was observed, it is prudent to allow for the erosion trend to continue into the future.

A long term accretion trend was observed for the shoreline at the Tarcoola embayment (chainage 4,800 m to 9,600 m), which has been partially protected by the Southgate and Point Moore Reef system, with sediment feed from the western Southgate Dunes area and offshore sand bars. Between 1942 and 2015, the accretion rate observed along this stretch of shoreline varied between 0.87 to 1.45 m per year. This is estimated to equate to a volumetric accretion rate of around 40,000 m^3 per year. As a result, it could potentially be reasonable to provide a negative allowance for S2 in this area given the potential for significant future accretion. However, as the result of MRA (2013) highlighted that, although the western Southgate Dune area provides a significant sediment feed (between 31,000 to 37,800 m^3 per year) into the Tarcoola Embayment, due to its migration rate, this source of sediment could potentially be exhausted in the next 50 years. Therefore, there is no compelling evident that the sediment feed from the western Southgate Dunes area and offshore sand bars will continue at the same rate for at least the next 50 years, hence, in accordance with the SPP2.6, a S2 allowance of zero should be taken for this section of shoreline.

For the shoreline on the eastern section of Greys Beach (northwest of Chainage 10,000 m), the historical shoreline movement rate has fluctuated between accretion and erosion of up to about 0.2 m per year. Therefore, a S2 allowance of 0.2 m per year was adopted.

The proposed S2 allowance for the study shoreline is presented in Figure 5.19 relative to the observed rate of shoreline movement for the periods outlined above.

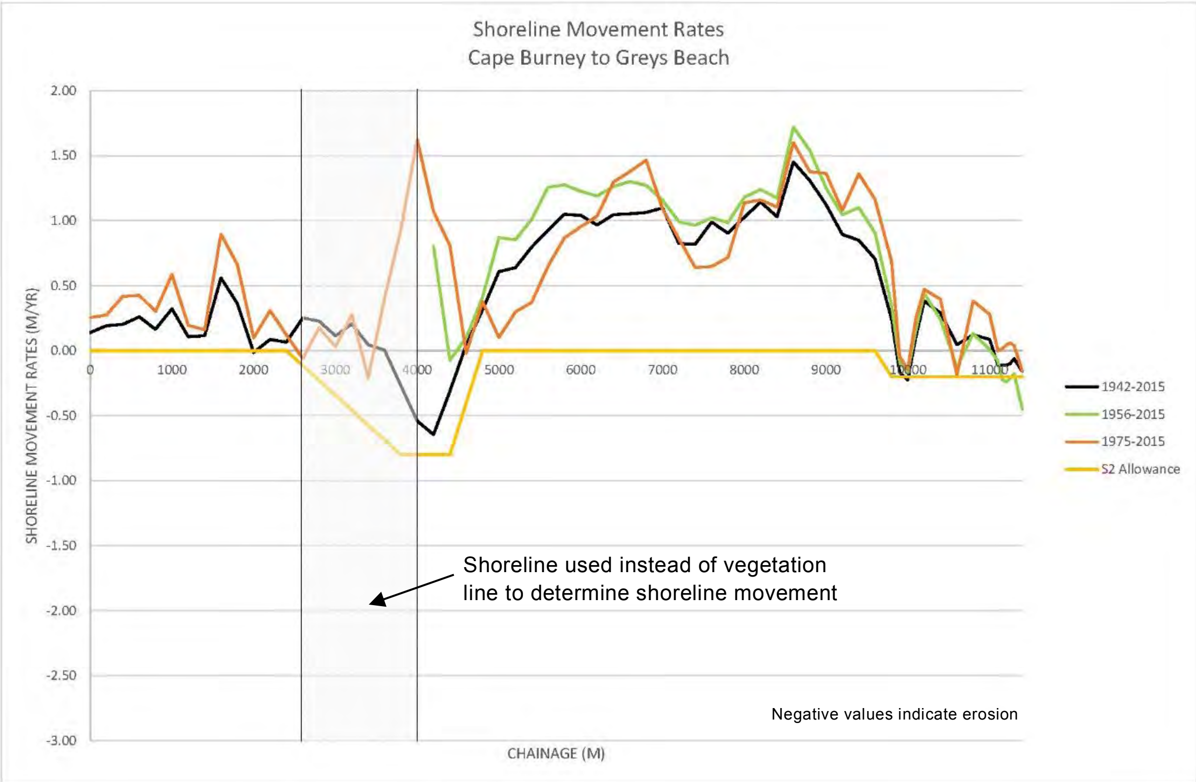


Figure 5.19 Rate of Shoreline Movement of the Study Shoreline

5.3 Sea Level Rise Allowance (S3)

The effect of sea level rise on the coast is difficult to predict. Komar (1998) provides a reasonable treatment for sandy shores, including examination of the Bruun Rule (Bruun 1962). The Bruun Rule relates the recession of the shoreline to the sea level rise and slope of the nearshore sediment bed:

$$R = \frac{1}{\tan(\theta)} S$$

where: *R* = recession of the shore;

θ = average slope of the nearshore sediment bed; and

S = sea level rise.

Komar suggests that the usual range of recession is R = 50S – 100S. However, the "Bruun Rule" does not take into account possible changes in the balance of sediment transported along the shore in response to sea level rise. The SCPP recommends that for sandy shores the potential recession be taken as 100 times the estimated sea level rise.

Based on sea level rise adopted in Table 3.1, the allowance for sea level rise from 2015 to each of the three key time frames, 2030, 2070 and 2110 is presented in Table 5.2.

Table 5.2 Sea Level Rise Allowance (S3)

Timeframe	S3 Allowance (m)
2030	7
2070	39
2110	90

It should be noted that the policy requires that the coastal processes allowances for new development, or for the assessment of risk posed to existing development, be completed based on a 100 year planning horizon. Therefore, for the purpose of this study (to determine the risk posed to existing development and inform decision making for future coastal management), an allowance for sea level rise of 0.90 m has been adopted for 2110. Given the 100S value, the potential recession of the shoreline between Cape Burney and Greys Beach that could occur as a result of the increase in sea level is 90 m in 2110.

6. Total Coastal Processes Allowance

The total recommended allowance for the future action of coastal processes should include the allowances determined in previous sections of this report. Additionally, an allowance for uncertainty of 0.2 m/year should also be included as per the requirements of SPP2.6. The total recommended coastal processes allowances for the four key timeframes: present day, 2030, 2070 and 2110 are presented in Tables 6.1 to 6.4.

Table 6.1 Total Recommended Coastal Processes Allowance – Present Day

Chainage/ Location	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
0	27	0	0	0	27
800	27	0	0	0	27
1000	12	0	0	0	12
2400	12	0	0	0	12
2600	27	0	0	0	27
3600	27	0	0	0	27
3800	23	0	0	0	23
4400	23	0	0	0	23
4800	23	0	0	0	23
5000	29	0	0	0	29
6200	29	0	0	0	29
6400	28	0	0	0	28
8200	28	0	0	0	28
8400	17	0	0	0	17
9600	17	0	0	0	17
9800	25	0	0	0	25
10200	25	0	0	0	25
10400	28	0	0	0	28
11400	28	0	0	0	28

Table 6.2 Total Recommended Coastal Processes Allowance – 2030

Chainage/ Location	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
0	27	0	7	3	37
800	27	0	7	3	37
1000	12	0	7	3	22
2400	12	0	7	3	22
2600	27	2	7	3	39
3600	27	11	7	3	48
3800	23	12	7	3	45
4400	23	12	7	3	45
4800	23	0	7	3	33
5000	29	0	7	3	39
6200	29	0	7	3	39
6400	28	0	7	3	38
8200	28	0	7	3	38
8400	17	0	7	3	27
9600	17	0	7	3	27
9800	25	3	7	3	38
10200	25	3	7	3	38
10400	28	3	7	3	41
11400	28	3	7	3	41

Table 6.3 Total Recommended Coastal Processes Allowance – 2070

Chainage/ Location	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
0	27	0	39	11	77
800	27	0	39	11	77
1000	12	0	39	11	62
2400	12	0	39	11	62
2600	27	7	39	11	84
3600	27	38	39	11	115
3800	23	44	39	11	117
4400	23	44	39	11	117
4800	23	0	39	11	73
5000	29	0	39	11	79
6200	29	0	39	11	79
6400	28	0	39	11	78
8200	28	0	39	11	78
8400	17	0	39	11	67
9600	17	0	39	11	67
9800	25	11	39	11	86
10200	25	11	39	11	86
10400	28	11	39	11	89
11400	28	11	39	11	89

Table 6.4 Total Recommended Coastal Processes Allowance – 2110

Chainage/ Location	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
0	27	0	90	20	137
800	27	0	90	20	137
1000	12	0	90	20	122
2400	12	0	90	20	122
2600	27	12	90	20	149
3600	27	69	90	20	206
3800	23	80	90	20	213
4400	23	80	90	20	213
4800	23	0	90	20	133
5000	29	0	90	20	139
6200	29	0	90	20	139
6400	28	0	90	20	138
8200	28	0	90	20	138
8400	17	0	90	20	127
9600	17	0	90	20	127
9800	25	20	90	20	155
10200	25	20	90	20	155
10400	28	20	90	20	158
11400	28	20	90	20	158

The physical coastal processes allowances are to be measured from the HSD, which was discussed in Section 5.1. The location of the coastal processes allowances for the four timeframes are presented in Appendix B.

7. Combined Inundation & Coastal Processes Mapping

The potential extent of impacts caused by coastal inundation and coastal processes have been discussed in previous sections of this report. These assessments were completed for a number of different timeframes, and in the case of the coastal inundation assessment, also considered the potential impacts of events with different severities. Nevertheless, the general guideline within SPP2.6 is that significant development, such as residential or commercial development, should ideally be located outside of areas that could be impacted by a coastal erosion event with an AEP of 1% (100 year ARI) and an inundation event with an AEP of 0.2% (500 year ARI) for the given timeframe. Where existing development is not located outside of these potential areas of impact, management would likely be required to ensure acceptable risk levels are maintained.

Given the above, it is possible to develop summary plots that highlight areas that would not meet the general guideline of SPP2.6. These plots have been completed for each planning timeframe, including present day, 2030, 2070 and 2110, and are presented in Appendix C.

8. Conclusions

Assessments of the appropriate inundation and coastal processes allowances for the shoreline between Cape Burney and Greys Beach have been made in line with the recommendations and intent of SPP2.6. The following conclusions have been made from this assessment.

- The costal inundation allowances between Cape Burney and Greys Beach have been estimated and presented in Table 4.6. The inundation allowances have been determined based on assessment of cyclonic storm surge inundation, non-cyclonic storm surge inundation and tsunami induced inundation. These allowances provide useful information when considering the potential vulnerability of existing development, or when assessing new development or redevelopment.

- In view of the aspect, exposure and characteristics of the coast along the study area, nine beach profiles were taken to investigate the response of the shoreline to the design storm.

- Storm erosion modelling using the SBEACH profile change model resulted in predicted erosion of between 12 and 29 m for the design storm. The values determined by SBEACH were used as the allowances for S1 for all areas, as stipulated by the SPP2.6.

- Shoreline movement analysis was completed, including review of key changes (such as the migration of the active Southgate Dunes) that may have affected the sediment dynamics in the area. As a result of this investigation, allowances for future shoreline movement have been provided.

- Allowances of 7, 39 and 90 m have been provided to account for the potential shoreline recession as a result of 0.07, 0.39 and 0.9 m of sea level rise in 2030, 2070 and 2110 respectively.

- An allowance for uncertainty of 0.2 m per year has been included.

- The total coastal processes allowances for four key timeframes: present day, 2030, 2070 and 2110 are presented in Table 6.1 to 6.4. These coastal processes allowance lines highlight areas that could be vulnerable to the action of coastal processes over the respective timeframes.

9. References

Bruun, P 1962. “Sea level rise as a cause of shore erosion,” Journal Waterways and Harbours Division, American Society of Civil Engineers. WWI, 88, pp. 117-130.

Burbidge, D, Cummins, RP, Mleczko, R & Thio, HK, 2008. *A Probabilistic Tsunami Hazard Assessment for Western Australia*, Pure and Applied Geophysics.

CERC 1984. *Shore Protection Manual*. Coastal Engineering Research Centre, US Army Corps of Engineers, Vicksburg MS.

Davis, G A & Nielsen, P, 1988. *Field Measurements of Wave Setup*. Chapter 38, ASCE International Conference on Coastal Engineering, Malaga, Spain, pp. 539-552.

Dean, R G & Walton T L, 2008. *Wave Setup – a State of Art Review*. Beaches and Shores Research Centre, Institute of Science and Public Affairs, Florida State University.

Deltares, 2011a. Simulation of Multi-Dimensional Hydrodynamic Flows and Transport Phenomena, including Sediments – User Manual. Version 3.15 revision 18392. Deltares, The Netherlands.

Deltares, 2011b. *Wind Enhanced Scheme for Cyclone Modelling – User Manual.* Version 3.00 revision 15423. Deltares, The Netherlands.

Department of Transport 2009. *Coastal Demarcation Lines for Administrative & Engineering Purposes – Delineation Methodology & Specification.* Published by the Government of Western Australia, Perth.

Department of Transport (DoT) 2010, *Sea Level Change in Western Australia – Application to Coastal Planning*, Coastal Infrastructure, Coastal Engineering Group.

Emanuel, K. A., S. Ravela, E. Vivant, and C. Risi, 2006: A statistical deterministic approach to hurricane risk assessment. Bull. Amer. Meteor. Soc., 87, 299–314.

Federal Emergency Management Agency, 2003. Guidelines and Specifications for Flood Hazard Mapping Partners, Appendix D: Guidance for Coastal Flooding Analysis and Mapping. US Federal Emergency Management Agency, Apr 03.

Guza, R T & Thornton, E B, 1981. *Wave Setup on an Natural Beach*. Journal of Geophysical Research, Vol. 96, No.C2, pp. 4133-4137.

Greenwood, B & Osborne, P D, 1999. *Vertical and Horizontal Structure in Cross Shore Flows: an Example of Underflow and Wave Setup on a Barred Beach*. Coastal engineering, Vol. 14, pp 543-*580. Elsevier Publishing.

Hansen, U A, 1978. *Wave Setup and Design Water Levels*. Journal of Waterway, Port, Coastal And Ocean Division, American Society of Civil Engineers, Vol. 104, No. WW2, pp. 227-240.

Hanslow, D J and Neilson, P, 1992. *Wave Setup on Beaches and in River Entrances*. 23rd International Conference on Coastal Engineering, Venice, Italy, pp.240-252.

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

Holman, R A & Sallenger, A H, 1985. *Setup and Swash on a Natural Beach*. Journal of Geophysical Research, Vol. 90, No. C1, pp. 945-953.

Horspool, N., Griffin, J & Putten, K. V, 2010. Tsunami modelling validation: The impact of the 2004 Indian Ocean Tsunami on Geraldton, Western Australia. Record 2010/01 GeoCat# 68942, Geoscience Australia, Government of Australia.

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, Church, J.A., P.U. Clark, A. Cazenave, J.M. Gregory, S. Jevrejeva, A. Levermann, M.A. Merrifield, G.A. Milne, R.S. Nerem, P.D. Nunn, A.J. Payne, W.T. Pfeffer, D. Stammer and A.S. Unnikrishnan, 2013: Sea Level Change. In: Climate Change 2013: The Physical Science Basis. Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Stocker, T.F., D. Qin, G.-K. Plattner, M. Tignor, S.K. Allen, J. Boschung, A. Nauels, Y. Xia, V. Bex and P.M. Midgley (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA.

King, B A, Blackley, M W L, Carr, A P & Hardcastle, P J, 1990. *Observations of Wave Induced Setup on a Natural Beach*. Journal of Geophysical Research, Vol. 95, C12, pp. 22289-22297.

Komar P D 1998. Beach Processes and Sedimentation (2nd Edition). Prentice Hall Inc, New Jersey, USA.

Larson M & Kraus N.C, 1989. *SBEACH: Numerical Model for Simulating Storm-Induced Beach Change. Report 1: Empirical foundation and model development*. Technical Report CERC-89-9. Coastal Engineering Research Centre, Vicksburg, MS.

Larson M & Kraus N C 1998. SBEACH: Numerical Model for Simulating Storm-Induced Beach Change; Report 5, Representation of Nonerodible (Hard) Bottoms. Technical Report CERC-89-9. US Army Corps of Engineers, Washington, DC.

Larson M, Wise R.A & Kraus N.C, 2004. *Coastal overwash, Part 2: Upgrade to SBEACH.* ERDC/CHL CHETN IV-XX. US Army Engineer Research and Development Centre, Vicksburg, MS.

Lentz, S & Raubenheimer, B, 1999. *Field Observations of Wave Setup*. Journal of Geophysical Research, Vol. 104, No. C11, PP.25867-25875.

Luick, J.L. 2004. *Australian Tidal Handbook*. National Tidal Centre, Australian Government Bureau of Meteorology. Adelaide, South Australia.

Nielsen, P, 1988. *Wave Setup: A Field Study*. Journal of Geophysical Research, Vol.93, No. C12, pp. 15643-15652.

Petrauskas, C. & Aagaard, P. M., 1971. *Extrapolation of Historical Storm Data for Estimating Design Wave Heights.* American Institute of Mining, Metallurgical and Petroleum Engineers Inc.

Raubenheimeer, B, Guza, R T & Elgar, 2001. *Field Observations of Wave Drive Setdown and Setup.* Journal of Geophysical Research, Vol. 106, No. C3, pp. 4629-4638.

Risi, C, 2004. Statistical synthesis of tropical cyclone tracks in a risk evaluation perspective. Massachusetts Institute of Technology, internship report, Cambridge.

MRA, 1998. *Southgate Dunes Coastal Engineering Study.* Report R033 Rev 1. Prepared for Landrow Developments.

MRA, 2015. *Point Moore Inundation & Coastal Processes Study.* Report R656 Rev 1. Prepared for The City of Greater Geraldton.

Rogers, M. P., Saunders, B. S. & Hunt, T. S. 2005. Living on the Coast – But How Close is Safe?. Proceedings of the Coasts and Ports 2005 Conference, Adelaide, Australia.

Short, A. 2006, *Beaches of the Western Australian Coast: Eucla to Roebuck Bay. A guide to their nature, characteristics, surf and safety*, Sydney University Press.

Stockdon, H F, Holman, R A, Howd, P A and Sallenger, A H, 2006. *Empirical Parameterization of Setup, Swash and Runup*. Coastal Engineering 53 573-588, Elsevier Publishing.

Stul, T, Gozzard, JR, Eliot, IG & Eliot, MJ, 2014. *Coastal Sediment Cells for the Mid-West Region between the Moore River and Glenfield Beach*, Western Australia. Report prepared by Seashore Engineering Pty Ltd and Geological Survey of Western Australia for the Western Australian Department of Transport, Fremantle.

Tecchiato, S., Collins, L.B. & Stevens, A.M., 2012. Geraldton Embayments Coastal Sediment Budget Study. Curtin University Department of Applied Geology. Prepared for Department of Transport.

U.S. Army Corps of Engineers (USACE) 1989, *Water Levels and Wave Heights for Coastal Engineering Design*, Engineer Manual 1110-2-1414, U.S. Army Corps of Engineers, Washington, D.C.

WAPC, 2013. Statement of Planning Policy No. 2.6 – State Coastal Planning Policy. Western Australian State Government, Perth.

Whiteway, T.G., 2009. Australian Bathymetry and Topography Grid. Record 2009/21 GeoCat# 67703, Geoscience Australia, Government of Australia.

Willmott, C., Robeson, S.M. & Matsuura, K., 2011. *A refined index of model performance*. International Journal of Climatology, Vol. 32, Issue 13, pp. 2088-2094.

Wise, R. A., Smith, S. J. & Larson, M. 1996. 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.

Yanagishima, S and Katoh, K, 1990. *Field Observations on Wave Setup Near the Shoreline*. Proceedings of 22nd International Conference on Coastal Engineering, Vol.1, Ch. 7, pp. 95-108, ASCE, New York.

10. Glossary

'accretion' refers to shoreline movement where the shoreline shifts seaward increasing the width of a coastal foreshore reserve and or the distance to a fixed feature on the adjoining land.

'annual recurrence interval' means the average or expected value of the periods between exceedances of a given event over a given duration.

'coastal foreshore reserve' is the area of land on the coast set aside in public ownership to allow for likely impacts of coastal hazards and provide protection of public access, recreation and safety, biodiversity and ecosystem integrity, landscape, visual landscape, indigenous and cultural heritage.

'coastal hazard' means the consequence of coastal processes that affect the environment and safety of people. Potential coastal hazards include erosion, accretion and inundation.

'coastal processes' means any action of natural forces on the coastal environment.

'erosion' refers to shoreline movement where the shoreline shifts landward reducing the width of a coastal foreshore reserve and/or the distance to a fixed feature on the adjoining land.

'event' means any occurrence of a particular set of circumstances that can have an adverse impact(s) on the environment. The event can be certain or uncertain, and be a one-off occurrence or a series of occurrences of a particular set of circumstances.

'horizontal shoreline datum (HSD)' defines the active limit of the shoreline under storm activity. It is the line from which a physical processes allowance will be applied from.

'inundation' means the flow of water onto previously dry land. It may either be permanent (for example due to sea level rise) or a temporary occurrence during a storm event.

'likelihood' means the probability that something will occur. Likelihood is generally expressed qualitatively or quantitatively.

'peak steady water level (PSWL)' means the highest average elevation of the sea surface caused by the combined effect of storm surge, tide and wave setup resulting from the storm events.

'risk' is specified in terms of an hazardous event or circumstances and the consequence that may flow from it. Risk is measured in terms of a combination of the likelihood of an event occurring and the consequence of that event occurring.

'sediment cell' means a length of shoreline in which interruptions to the movement of sediment along the beaches or near shore sea bed do not significantly affect beaches in the adjacent lengths of coastline. Within a sediment cell the sediments sources, transport pathways and sinks should be clearly definable.

'storm surge' means the increase in water level at the shoreline due to the forcing of winds (wind-setup) and atmospheric pressure.

11. Appendices

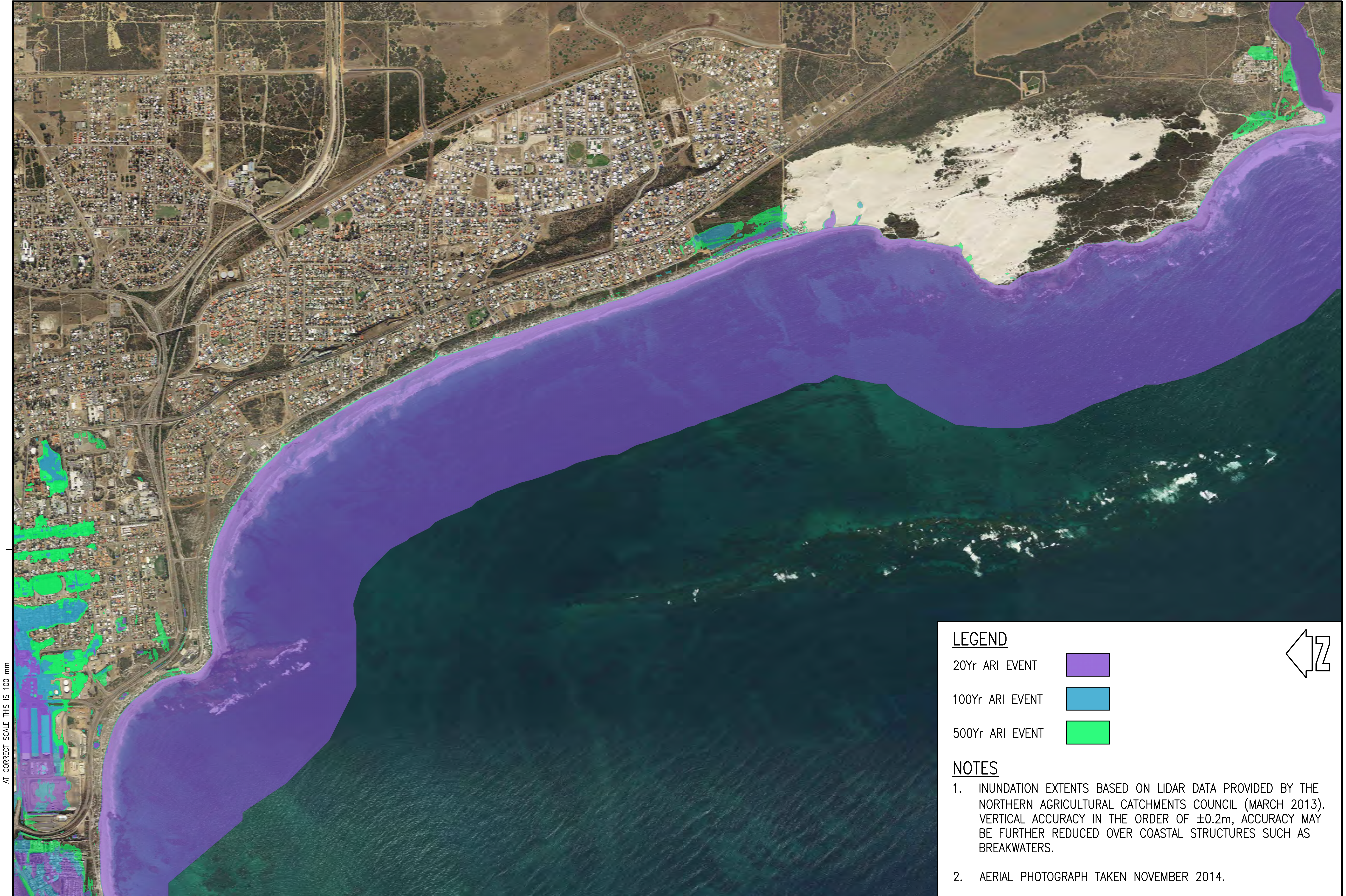
Appendix A Inundation Mapping

Appendix B Coastal Processes Allowance Mapping

Appendix C Combined Coastal Processes & Inundation Mapping

Appendix A Inundation Mapping

AT CORRECT SCALE THIS IS 100 mm



AT CORRECT SCALE THIS IS 100 mm

LEGEND

20Yr ARI EVENT	
100Yr ARI EVENT	
500Yr ARI EVENT	

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF ±0.2m, ACCURACY MAY BE FURTHER REDUCED OVER COASTAL STRUCTURES SUCH AS BREAKWATERS.
2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

m p rogers & associates pl
coastal and port engineers

Suite 1, 128 Main Street
Osborne Park 6017
Western Australia

t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

INUNDATION MAPPING – PRESENT DAY

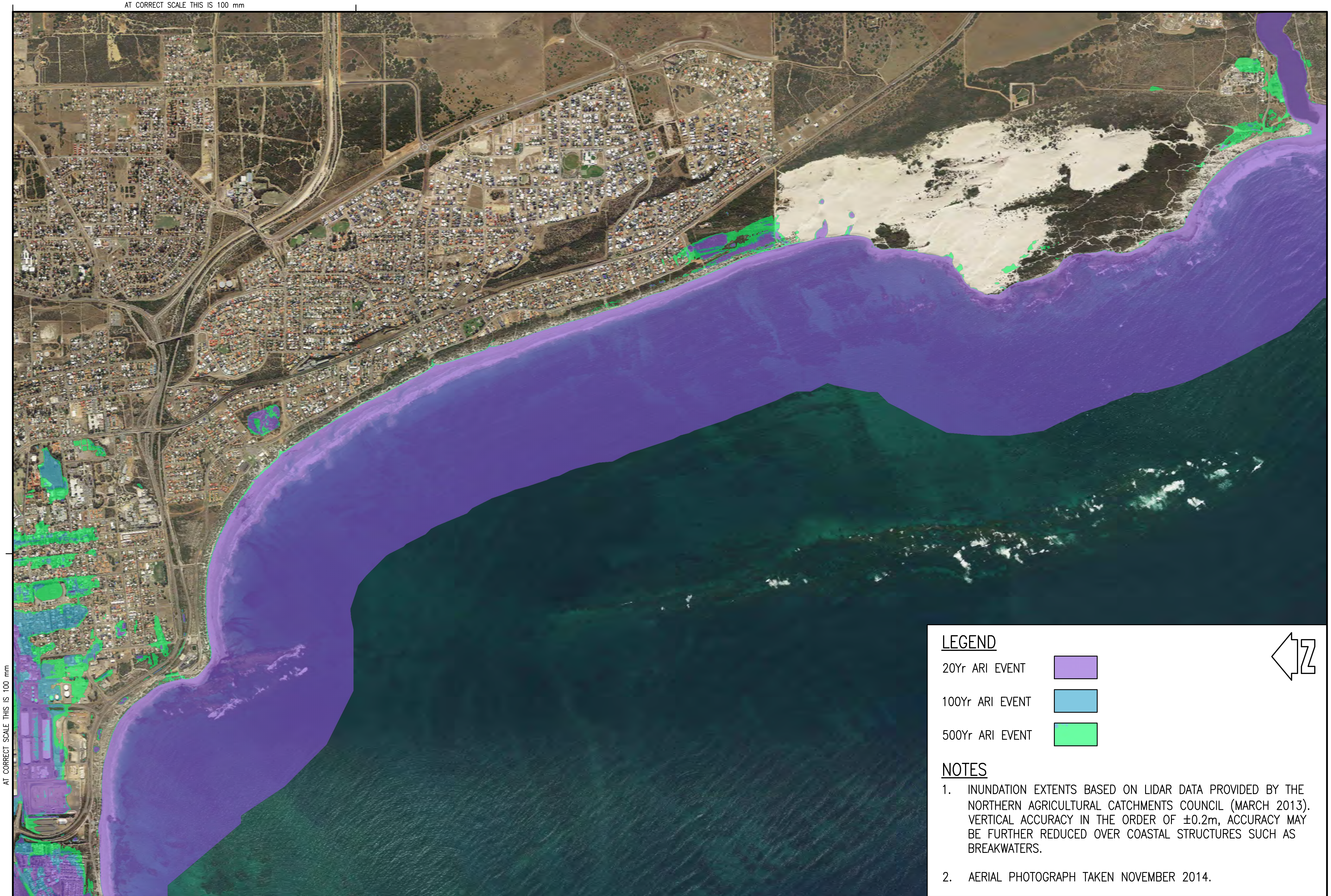
SCALE AT A3 1:25,000

JANUARY 2017

D1357-03-01(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Inundation\K1357-Present Day_CapeBurney

AT CORRECT SCALE THIS IS 100 mm



LEGEND

20Yr ARI EVENT	
100Yr ARI EVENT	
500Yr ARI EVENT	

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF ±0.2m, ACCURACY MAY BE FURTHER REDUCED OVER COASTAL STRUCTURES SUCH AS BREAKWATERS.

2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

INUNDATION MAPPING – 2030

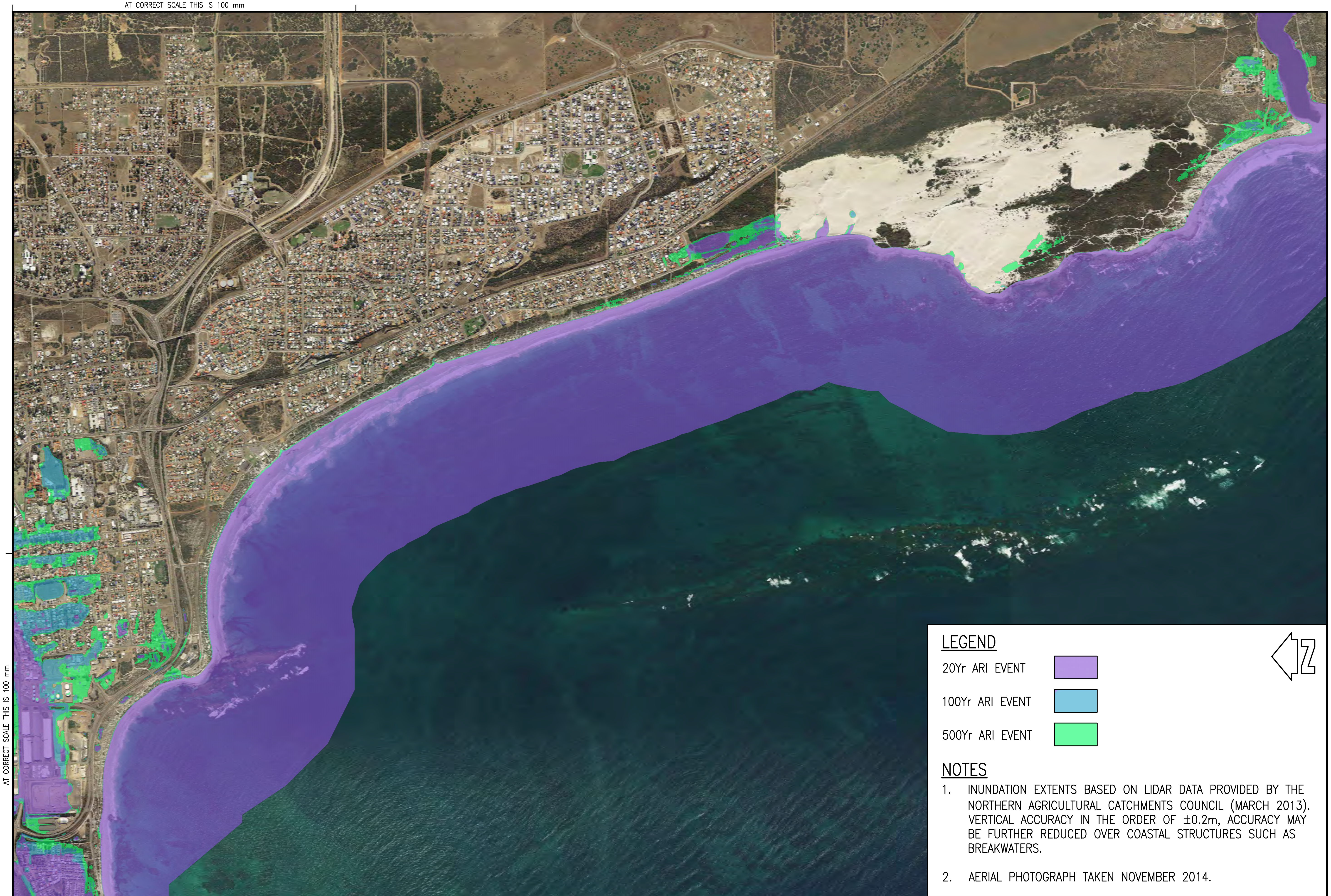
SCALE AT A3 1:25,000

JANUARY 2017

D1357-03-02(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Inundation\K1357-2030_CapeBurney

AT CORRECT SCALE THIS IS 100 mm



LEGEND

20Yr ARI EVENT	(purple)
100Yr ARI EVENT	(blue)
500Yr ARI EVENT	(green)

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF ±0.2m, ACCURACY MAY BE FURTHER REDUCED OVER COASTAL STRUCTURES SUCH AS BREAKWATERS.

2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

INUNDATION MAPPING – 2070

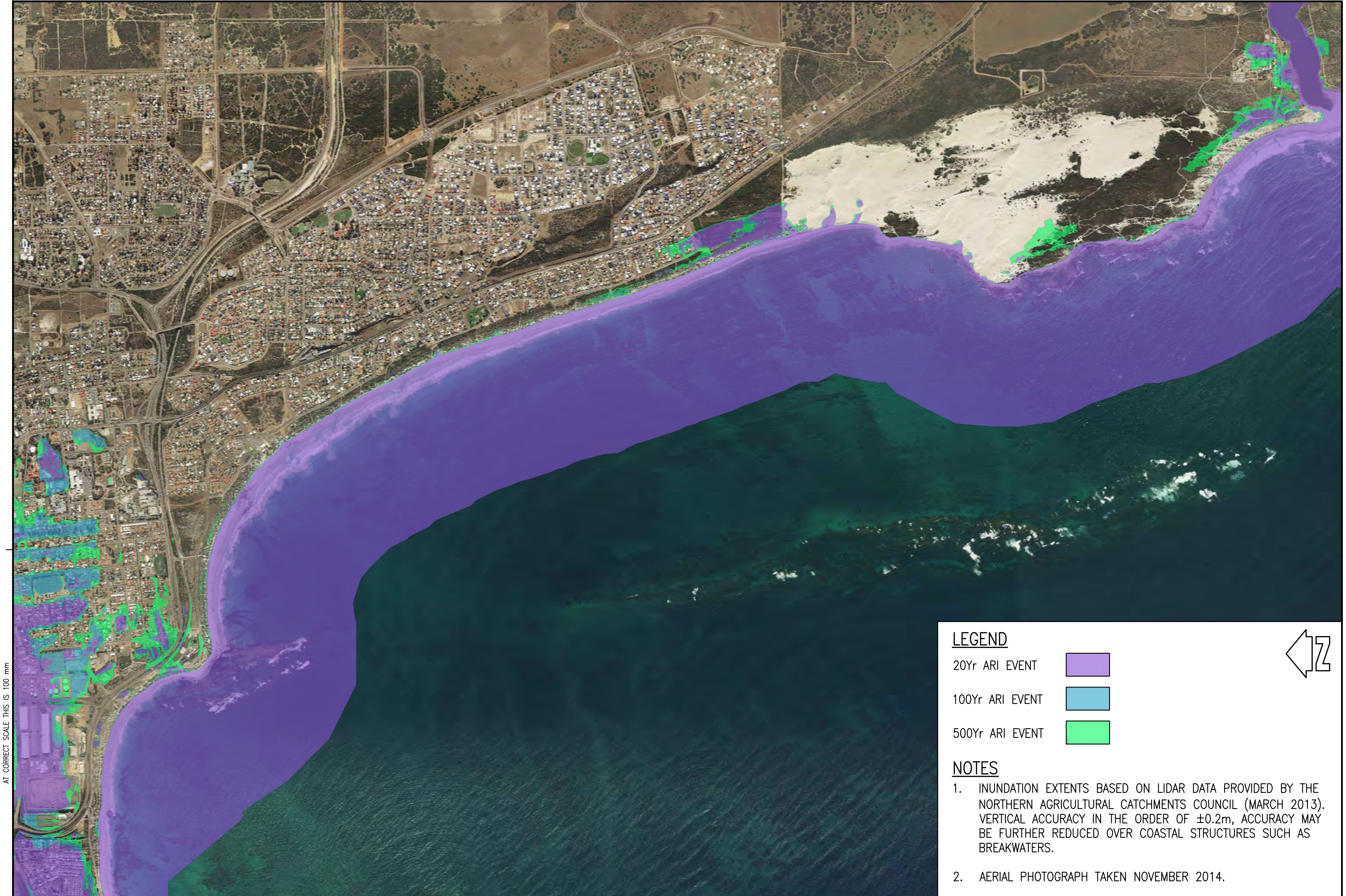
SCALE AT A3 1:25,000

JANUARY 2017

D1357-03-03(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Inundation\K1357-2070_CapeBurney

AT CORRECT SCALE THIS IS 100 mm



LEGEND

20Yr ARI EVENT	
100Yr ARI EVENT	
500Yr ARI EVENT	

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF ±0.2m, ACCURACY MAY BE FURTHER REDUCED OVER COASTAL STRUCTURES SUCH AS BREAKWATERS.

2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

INUNDATION MAPPING – 2110

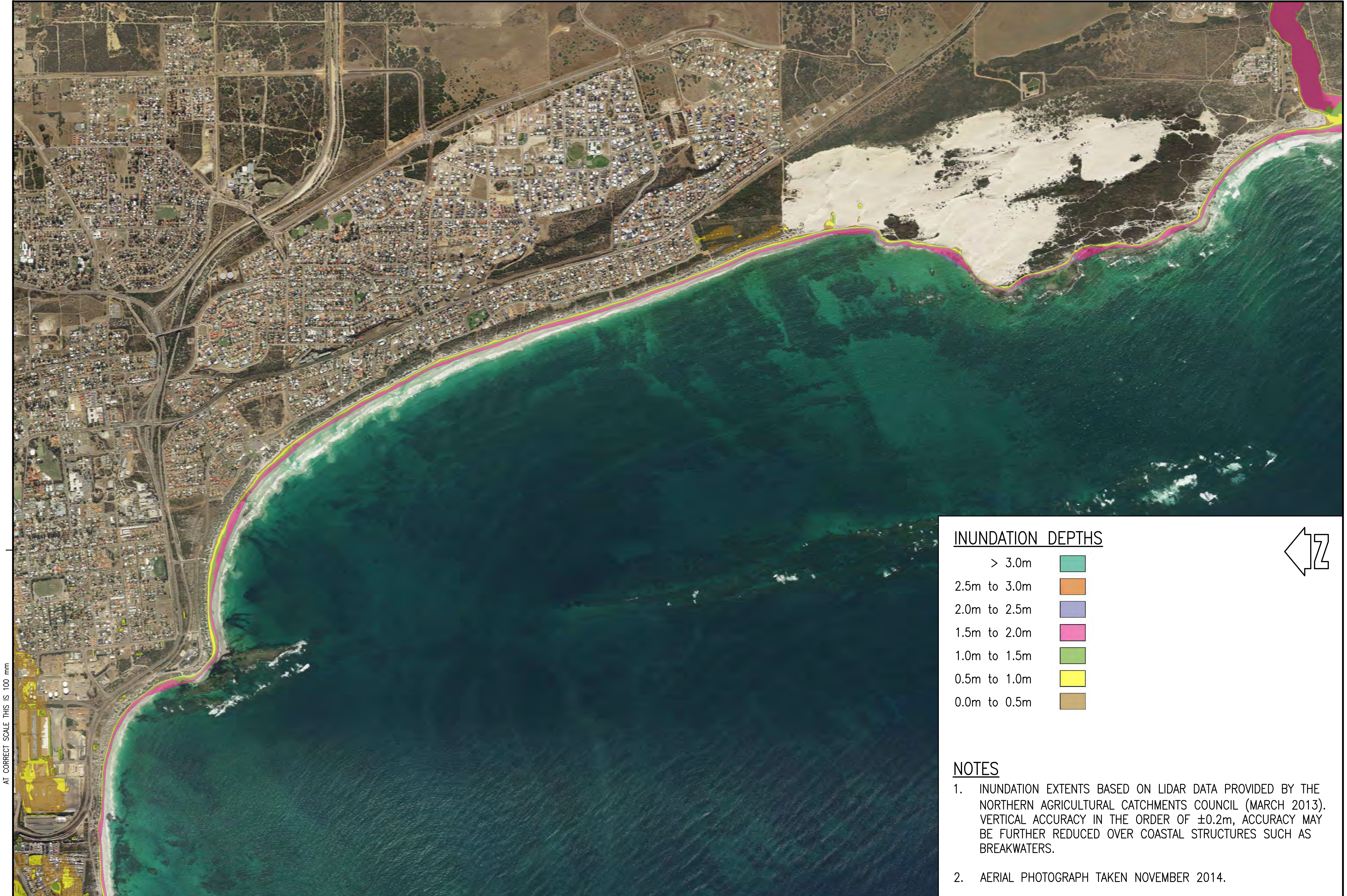
JANUARY 2017

SCALE AT A3 1:25,000

D1357-03-04(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Inundation\K1357-2110

AT CORRECT SCALE THIS IS 100 mm



INUNDATION DEPTHS

> 3.0m	
2.5m to 3.0m	
2.0m to 2.5m	
1.5m to 2.0m	
1.0m to 1.5m	
0.5m to 1.0m	
0.0m to 0.5m	

N

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF ±0.2m, ACCURACY MAY BE FURTHER REDUCED OVER COASTAL STRUCTURES SUCH AS BREAKWATERS.

2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

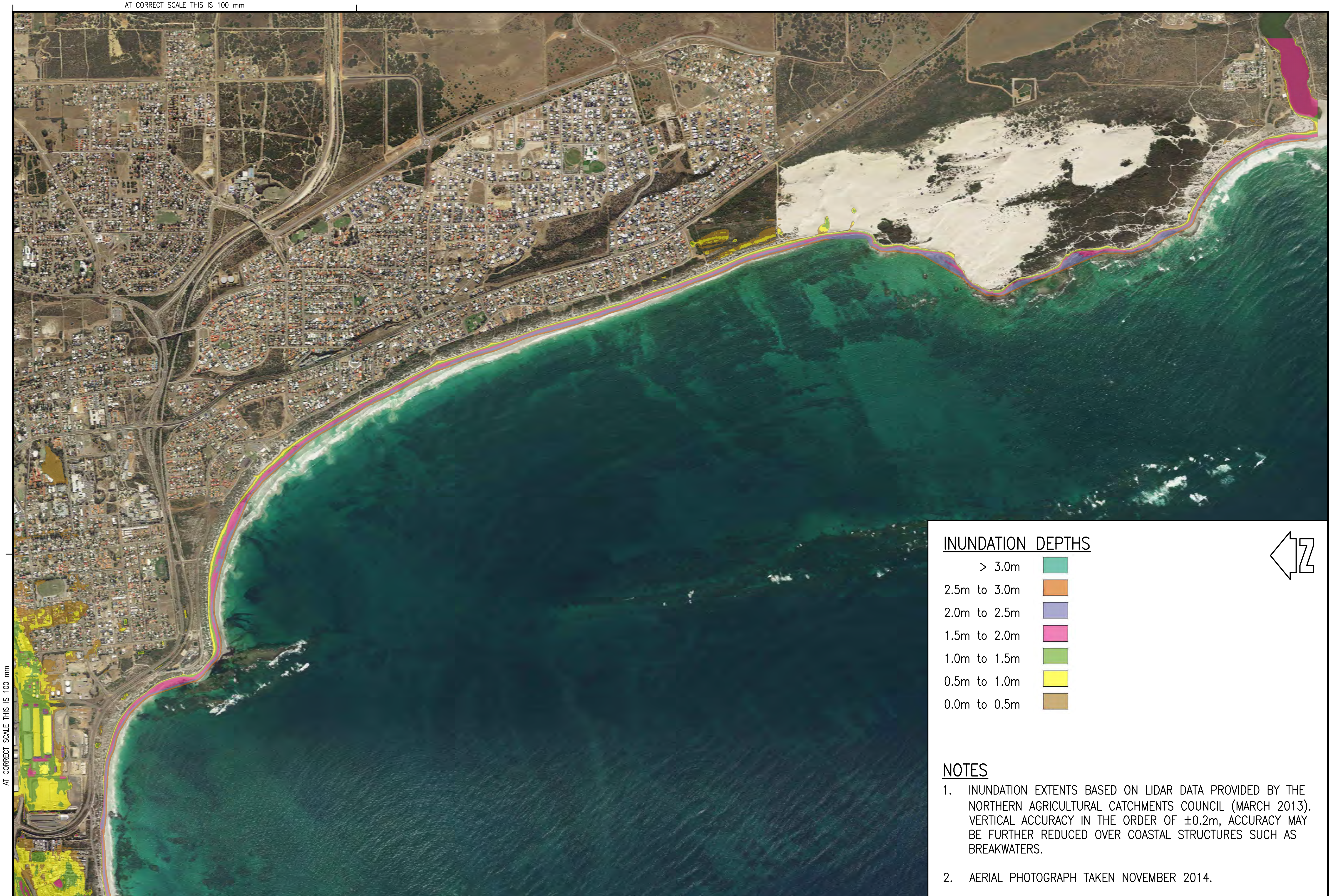
INUNDATION DEPTHS – 2030 20YR ARI EVENT

SCALE AT A3 1:25,000

JANUARY 2017

D1357-03-05(A)

AT CORRECT SCALE THIS IS 100 mm



AT CORRECT SCALE THIS IS 100 mm

INUNDATION DEPTHS

> 3.0m	
2.5m to 3.0m	
2.0m to 2.5m	
1.5m to 2.0m	
1.0m to 1.5m	
0.5m to 1.0m	
0.0m to 0.5m	

N

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF ±0.2m, ACCURACY MAY BE FURTHER REDUCED OVER COASTAL STRUCTURES SUCH AS BREAKWATERS.

2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

m p rogers & associates pl
coastal and port engineers

Suite 1, 128 Main Street
Osborne Park 6017
Western Australia

t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

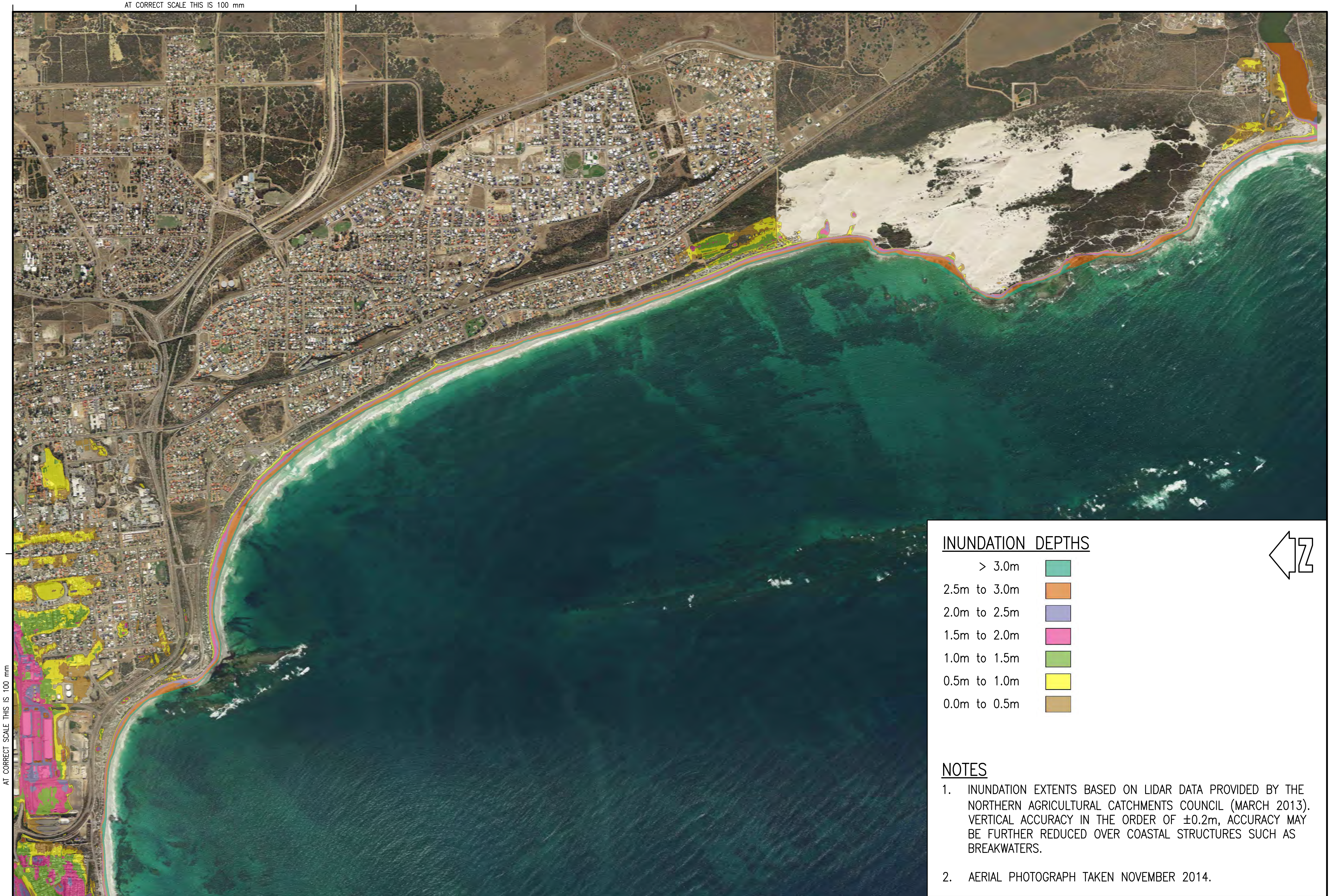
CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

INUNDATION DEPTHS – 2030 100YR ARI EVENT

JANUARY 2017

SCALE AT A3 1:25,000

D1357-03-06(A)



m p rogers & associates pl
coastal and port engineers

Suite 1, 128 Main Street
Osborne Park 6017
Western Australia

t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

INUNDATION DEPTHS – 2030 500YR ARI EVENT

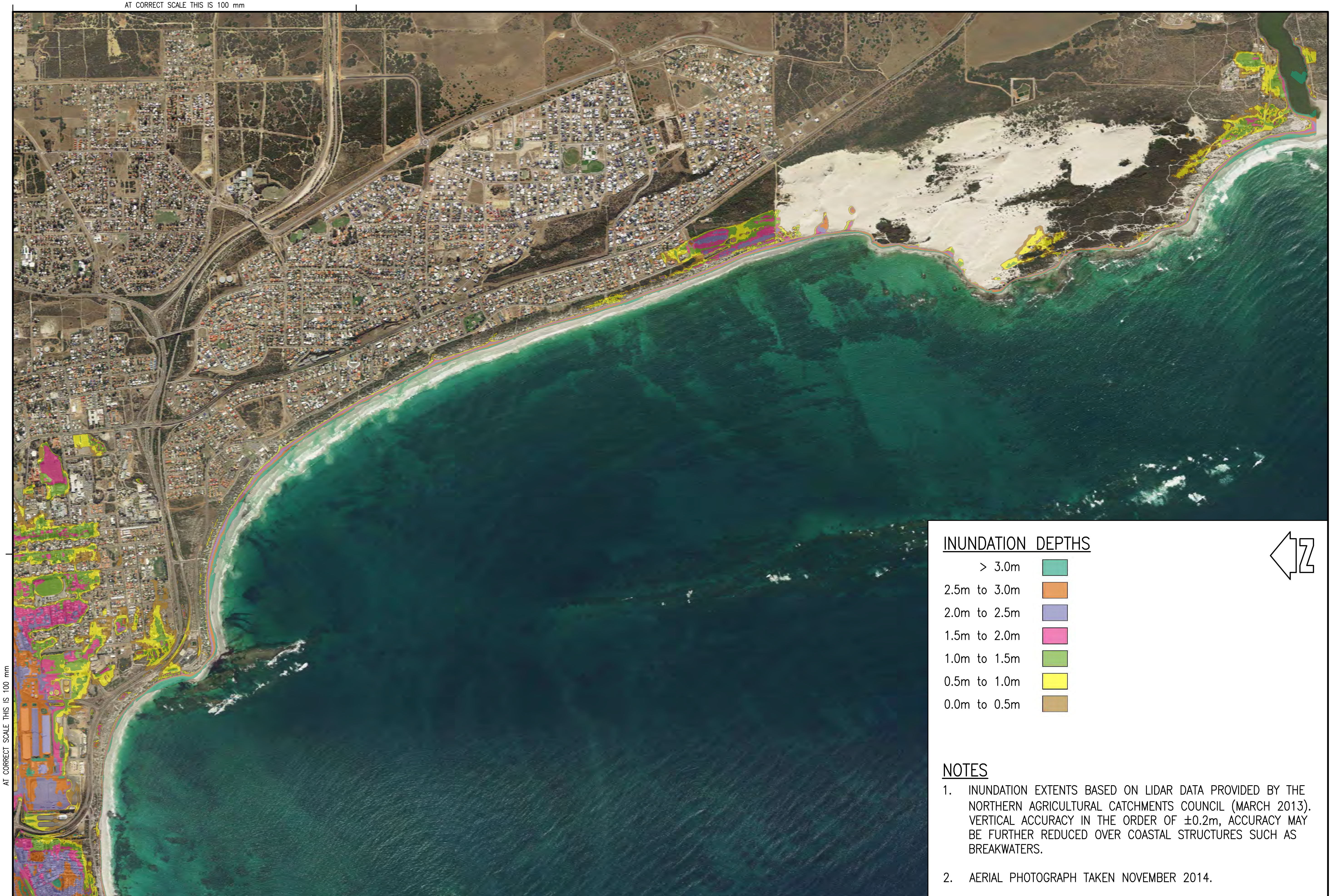
JANUARY 2017

SCALE AT A3 1:25,000

D1357-03-07(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Depths\K1357-ID-2030_500ARI_CapeBurney

AT CORRECT SCALE THIS IS 100 mm



INUNDATION DEPTHS

> 3.0m	
2.5m to 3.0m	
2.0m to 2.5m	
1.5m to 2.0m	
1.0m to 1.5m	
0.5m to 1.0m	
0.0m to 0.5m	

N

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF ±0.2m, ACCURACY MAY BE FURTHER REDUCED OVER COASTAL STRUCTURES SUCH AS BREAKWATERS.

2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

INUNDATION DEPTHS – 2110 500YR ARI EVENT

SCALE AT A3 1:25,000

JANUARY 2017

D1357-03-08(A)

Appendix B Coastal Processes Allowance Mapping

AT CORRECT SCALE THIS IS 100 mm

AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GREYS BEACH COASTAL EROSION AND INUNDATION STUDY

COASTAL PROCESSES ALLOWANCE MAPPING

SCALE AT A3 1:20,000

SEPTEMBER 2016

D1357-04-01(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\5 MRA Dwgs\K1357-04

AT CORRECT SCALE THIS IS 100 mm

AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers

Suite 1, 128 Main Street
Osborne Park 6017
Western Australia

t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GREYS BEACH COASTAL EROSION AND INUNDATION STUDY

COASTAL PROCESSES ALLOWANCE MAPPING

SCALE AT A3 1:20,000

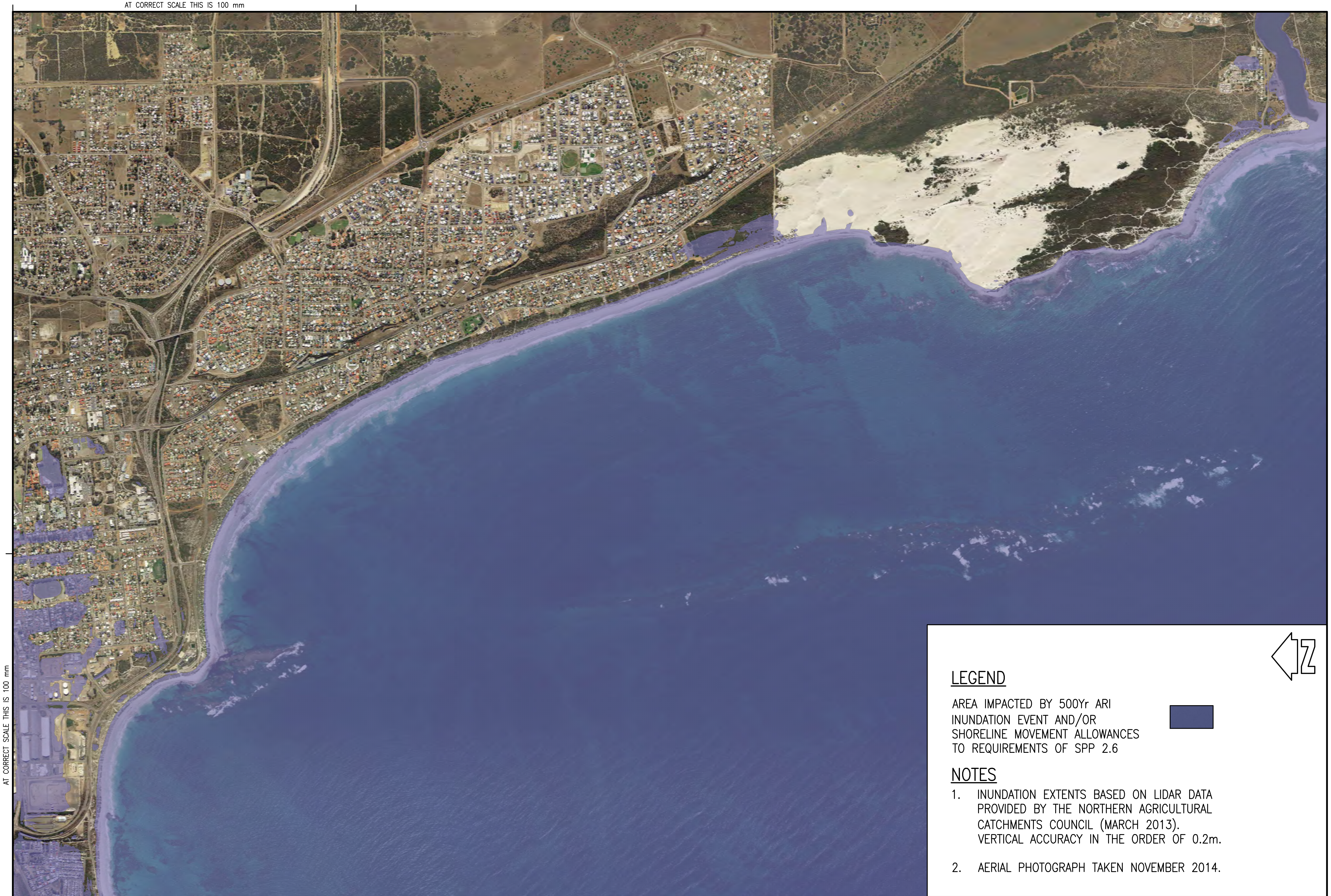
SEPTEMBER 2016

D1357-04-02(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\5 MRA Dwgs\K1357-04

Appendix C Combined Coastal Processes & Inundation Mapping

AT CORRECT SCALE THIS IS 100 mm



AT CORRECT SCALE THIS IS 100 mm

LEGEND

AREA IMPACTED BY 500Yr ARI INUNDATION EVENT AND/OR SHORELINE MOVEMENT ALLOWANCES TO REQUIREMENTS OF SPP 2.6

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF 0.2m.

2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

COMBINED COASTAL VULNERABILITY MAPPING – PRESENT DAY

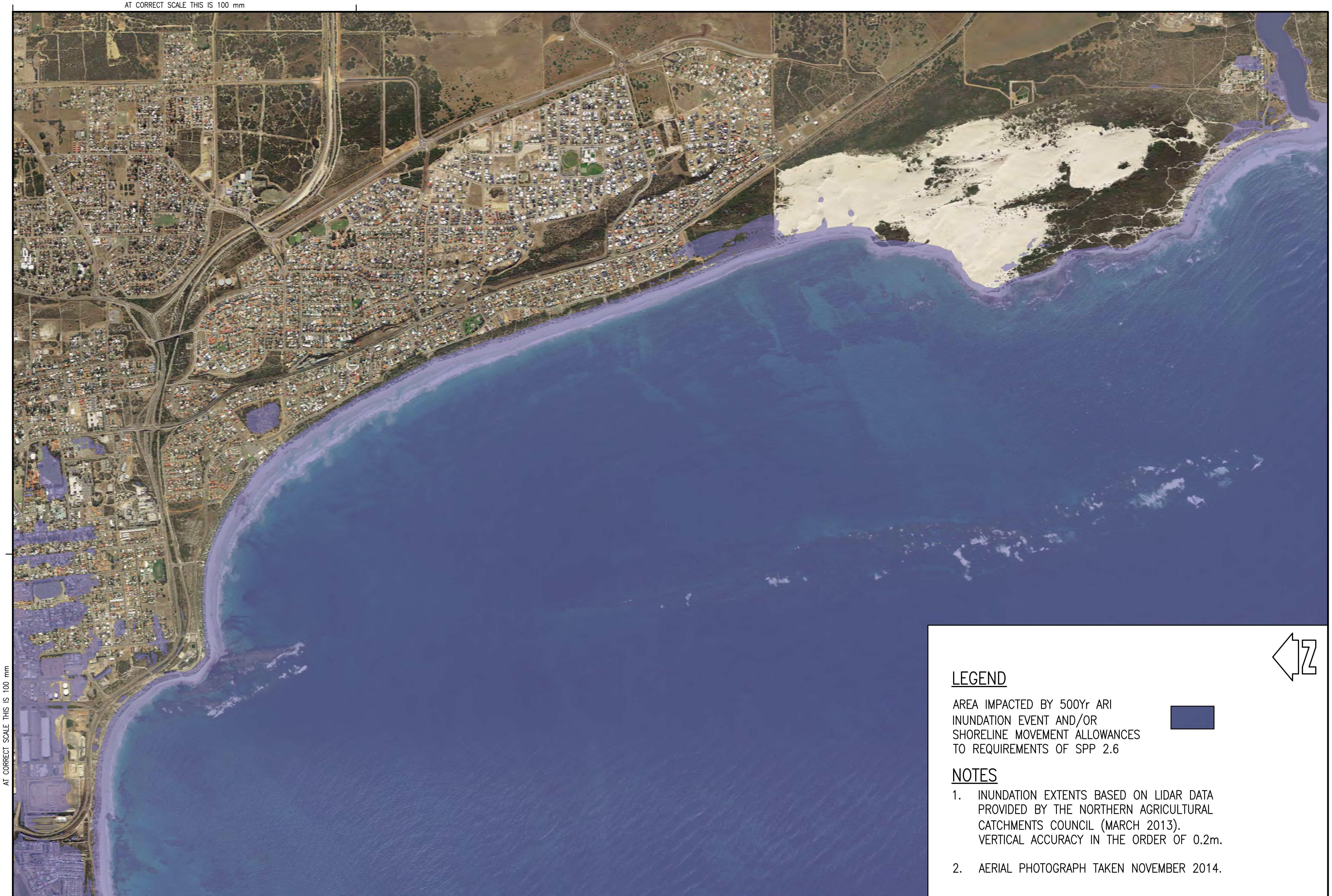
SCALE AT A3 1:25,000

JANUARY 2017

D1357-05-01(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Combined\K1357–Present Day_CapeBurney

AT CORRECT SCALE THIS IS 100 mm



AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

COMBINED COASTAL VULNERABILITY MAPPING – 2030

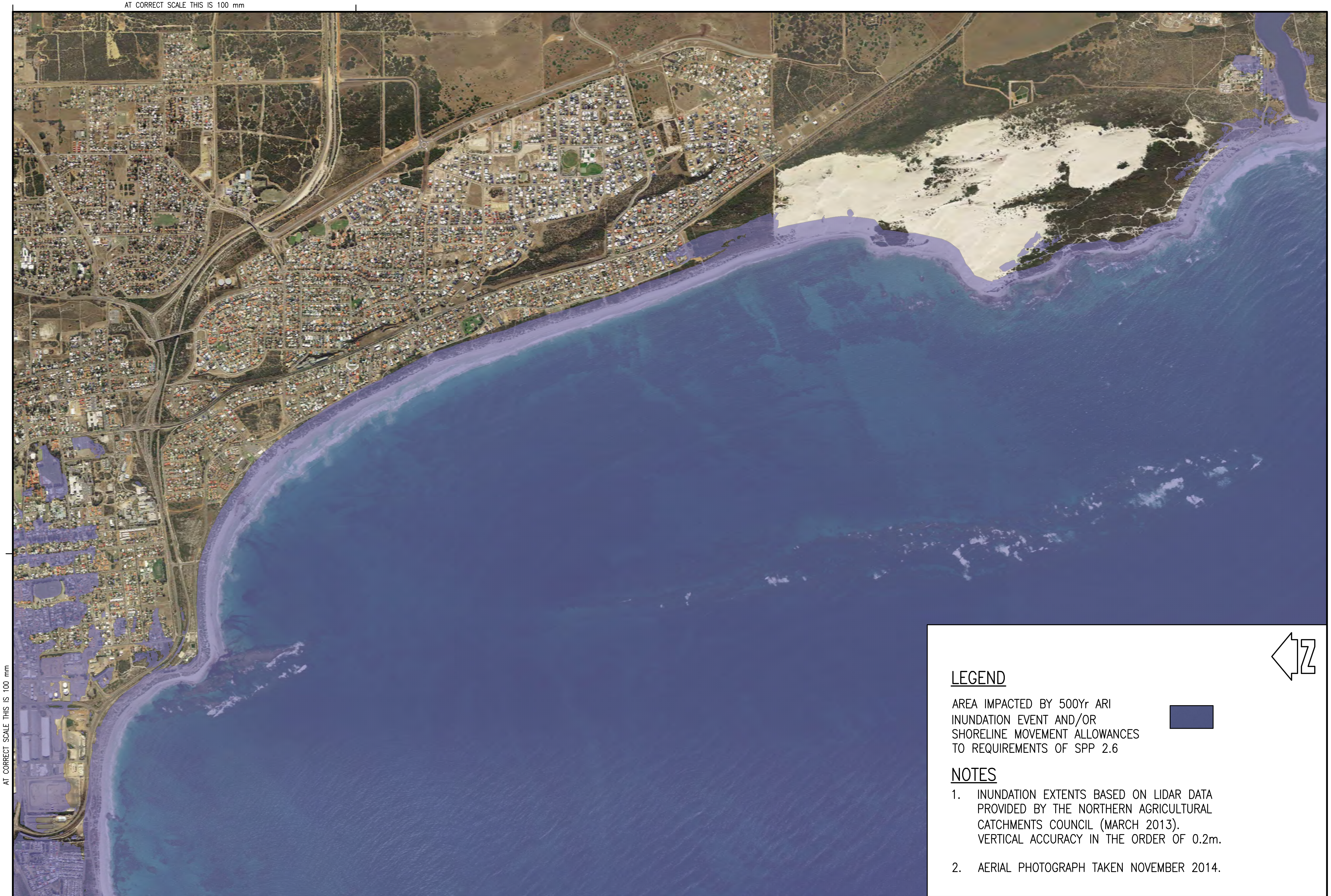
SCALE AT A3 1:25,000

JANUARY 2017

D1357-05-02(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Combined\K1357-2030_CapeBurney

AT CORRECT SCALE THIS IS 100 mm



LEGEND

AREA IMPACTED BY 500Yr ARI INUNDATION EVENT AND/OR SHORELINE MOVEMENT ALLOWANCES TO REQUIREMENTS OF SPP 2.6

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF 0.2m.
2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

AT CORRECT SCALE THIS IS 100 mm

m p rogers & associates pl
coastal and port engineers

Suite 1, 128 Main Street
Osborne Park 6017
Western Australia

t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

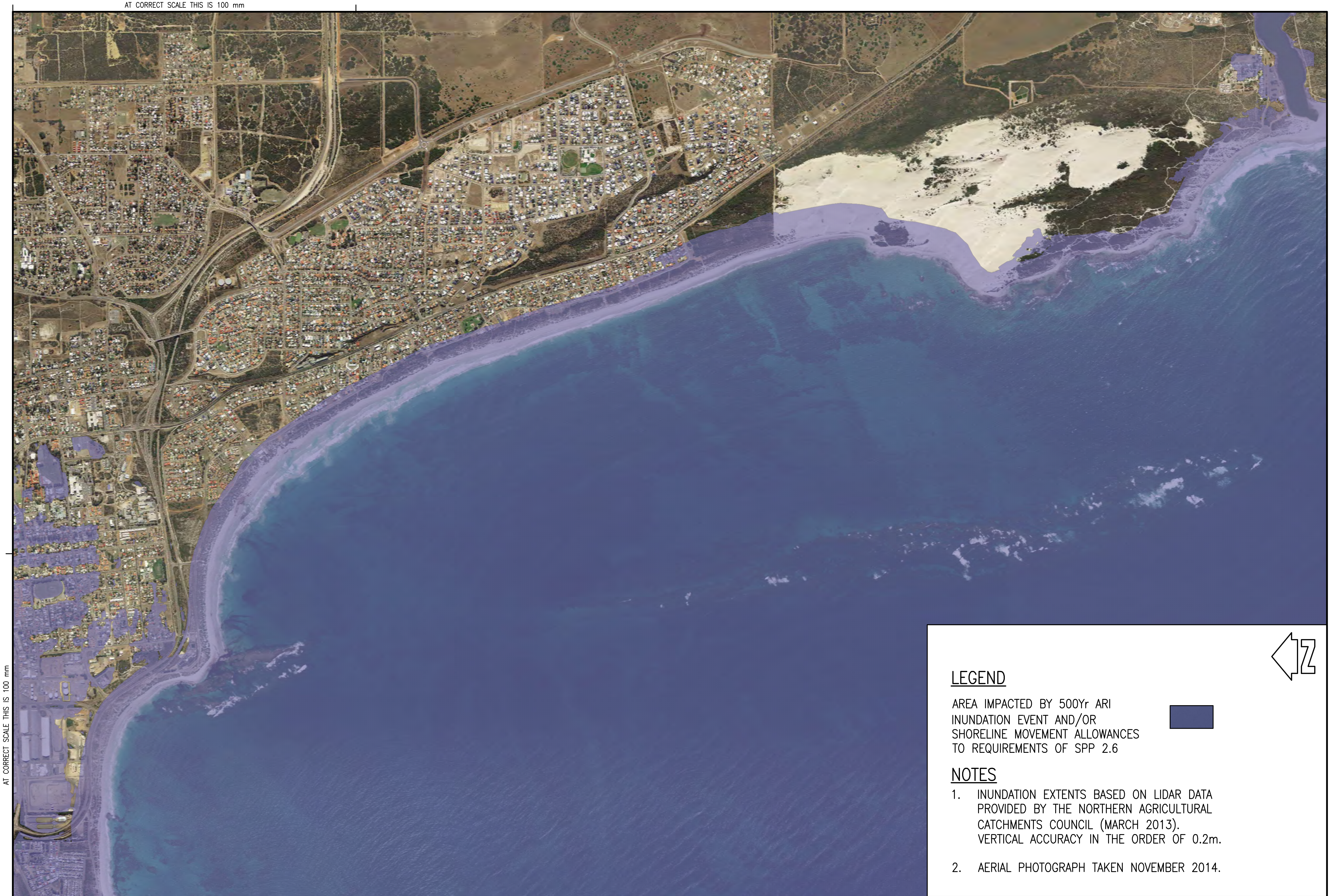
COMBINED COASTAL VULNERABILITY MAPPING – 2070

SCALE AT A3 1:25,000

JANUARY 2017

D1357-05-03(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Combined\K1357-2070_CapeBurney



LEGEND

AREA IMPACTED BY 500Yr ARI INUNDATION EVENT AND/OR SHORELINE MOVEMENT ALLOWANCES TO REQUIREMENTS OF SPP 2.6

NOTES

1. INUNDATION EXTENTS BASED ON LIDAR DATA PROVIDED BY THE NORTHERN AGRICULTURAL CATCHMENTS COUNCIL (MARCH 2013). VERTICAL ACCURACY IN THE ORDER OF 0.2m.
2. AERIAL PHOTOGRAPH TAKEN NOVEMBER 2014.

m p rogers & associates pl
coastal and port engineers
Suite 1, 128 Main Street
Osborne Park 6017
Western Australia
t: +61 8 9254 6600
f: +61 8 9254 6699
admin@coastsandports.com.au

CAPE BURNEY TO GRAYS BEACH EROSION AND INUNDATION STUDY

COMBINED COASTAL VULNERABILITY MAPPING – 2110

SCALE AT A3 1:25,000

JANUARY 2017

D1357-05-04(A)

P:\MRA Paying Jobs\K1357 CoGG – Greenough River to Greys Beach\12 Inundation Mapping\Combined\K1357-2110_CapeBurney

m p rogers & associates pl

www.coastsandports.com.au