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R656 Rev 1

December 2015

City of Greater Geraldton

Point Moore
Inundation & Coastal Processes Study

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Executive Summary

Point Moore is located in the City of Greater Geraldton on the Western Australian coastline. It is a prominent sandy foreland formed in lee of the Point Moore Reefs. Point Moore is bounded to the east by John Willcock link, with the remaining portion bounded by the Indian Ocean.

A coastal processes study has previously been completed for Point Moore by Worley Parson in 2010 for the then City of Geraldton – Greenough (now City of Greater Geraldton), Geraldton Port Authority (now Mid-West Port Authority) and the Department of Planning and Infrastructure (now Department of Planning, and Department of Transport (DoT)). Subsequent to the initial coastal processes study by Worley Parson, State Planning Policy No. 2.6: State Coastal Planning Policy (SPP2.6) was revised in 2013. Therefore, the City of Greater Geraldton (CGG) commissioned specialist coastal and port engineers M P Rogers & Associates Pty Ltd (MRA) to conduct an inundation and coastal processes study in line with the contemporary version of 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 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.

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 Point Moore, the following different inundation methodologies were assessed.

- 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 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. Therefore, to provide a relatively low risk of development being adversely impacted by coastal inundation over this planning horizon the development levels required by the Policy are based on the 500 year average recurrence interval (ARI) event, plus an allowance for sea level rise over the planning horizon.

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. 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 Point Moore.

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 3000 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.

Combining the potential inundation levels associated with each of the different inundation events considered above, the following tables.

Table E1 Recommended Coastal Inundation Allowance - Point Moore North

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	2.0	2.6	3.3
2030	0.07	2.1	2.7	3.4
2070	0.39	2.4	3.0	3.7
2110	0.90	2.9	3.5	4.2

Table E2 Recommended Coastal Inundation Allowance - Point Moore South

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	2.0	2.2	2.9
2030	0.07	2.1	2.3	2.9
2070	0.39	2.4	2.6	3.2
2110	0.90	2.9	3.1	3.8

Coastal Processes Allowances Assessment

SPP2.6 requires that on a sandy coast, an allowance for future shoreline movement should 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.

In view of the aspect, exposure and characteristics of the coast along the study area the shoreline was divided into three sectors to investigate the response of each sector to potential future changes associated with each of the above allowances. Furthermore, given the complex bathymetry surrounding the site, detailed wave modelling was completed in order to provide improved accuracy for the storm erosion modelling.

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 5 and 26 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, including review of sand extraction data provide by Mid-West Ports Authority. 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 2014. Generally speaking, there has been some erosion of the southern beach (Greys Beach), but the western and northern shorelines have accreted over 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 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).

Table E3 Total Recommended Coastal Processes Allowance – Present Day

Sector	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
Southern Shoreline	23	0	0	0	23
Western Shoreline	5	0	0	0	5
Northern Shoreline	26	0	0	0	26

Table E4 Total Recommended Coastal Processes Allowance – 2030

Sector	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
Southern Shoreline	23	9	7	3	42
Western Shoreline	5	-4	7	3	11
Northern Shoreline	26	-4	7	3	32

Table E5 Total Recommended Coastal Processes Allowance – 2070

Sector	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
Southern Shoreline	23	33	39	11	106
Western Shoreline	5	-16	39	11	39
Northern Shoreline	26	-16	39	11	60

Table E6 Total Recommended Coastal Processes Allowance – 2110

Sector	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
Southern Shoreline	23	60	90	20	193
Western Shoreline	5	-30	90	20	85
Northern Shoreline	26	-30	90	20	106

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1. Introduction

1.1 General

Point Moore is located in the City of Greater Geraldton on the Western Australian coastline. It is a prominent sandy foreland formed in lee of the Point Moore Reefs. Point Moore is bounded to the east by John Willcock link, with the remaining portion bounded by the Indian Ocean. The location of Point Moore is shown in Figure 1.1.

A coastal processes study has previously been completed for Point Moore by Worley Parson in 2010 for the then City of Geraldton – Greenough (now City of Greater Geraldton), Geraldton Port Authority (now Mid-West Port Authority) and the Department of Planning and Infrastructure (now Department of Planning, and Department of Transport (DoT)). Subsequent to the initial coastal processes study by Worley Parson, State Planning Policy No. 2.6: State Coastal Planning Policy (SPP2.6) (WAPC,2013) was revised in 2013. Therefore, it is required that the results of the physical coastal processes assessment be revised to reflect the requirements of the contemporary version of the Policy.

To satisfy this requirement, the City of Greater Geraldton (CGG) commissioned specialist coastal and port engineers M P Rogers & Associates Pty Ltd (MRA) to conduct an inundation and coastal processes study in line with the contemporary version of SPP2.6. This assessment will be completed for the entire coastline within the study area.

Figure 1.1 Location Diagram

2. Site Setting

2.1 The Site

The shoreline within the study area (refer Figure 1.1) is protected by the Point Moore reef system and extensive limestone platforms that exists along the shoreline in most areas (as shown in Figure 2.1). This stretch of the shoreline is disrupted by outcrops of limestone reef close to or adjacent to the shore (Damara, 2012).

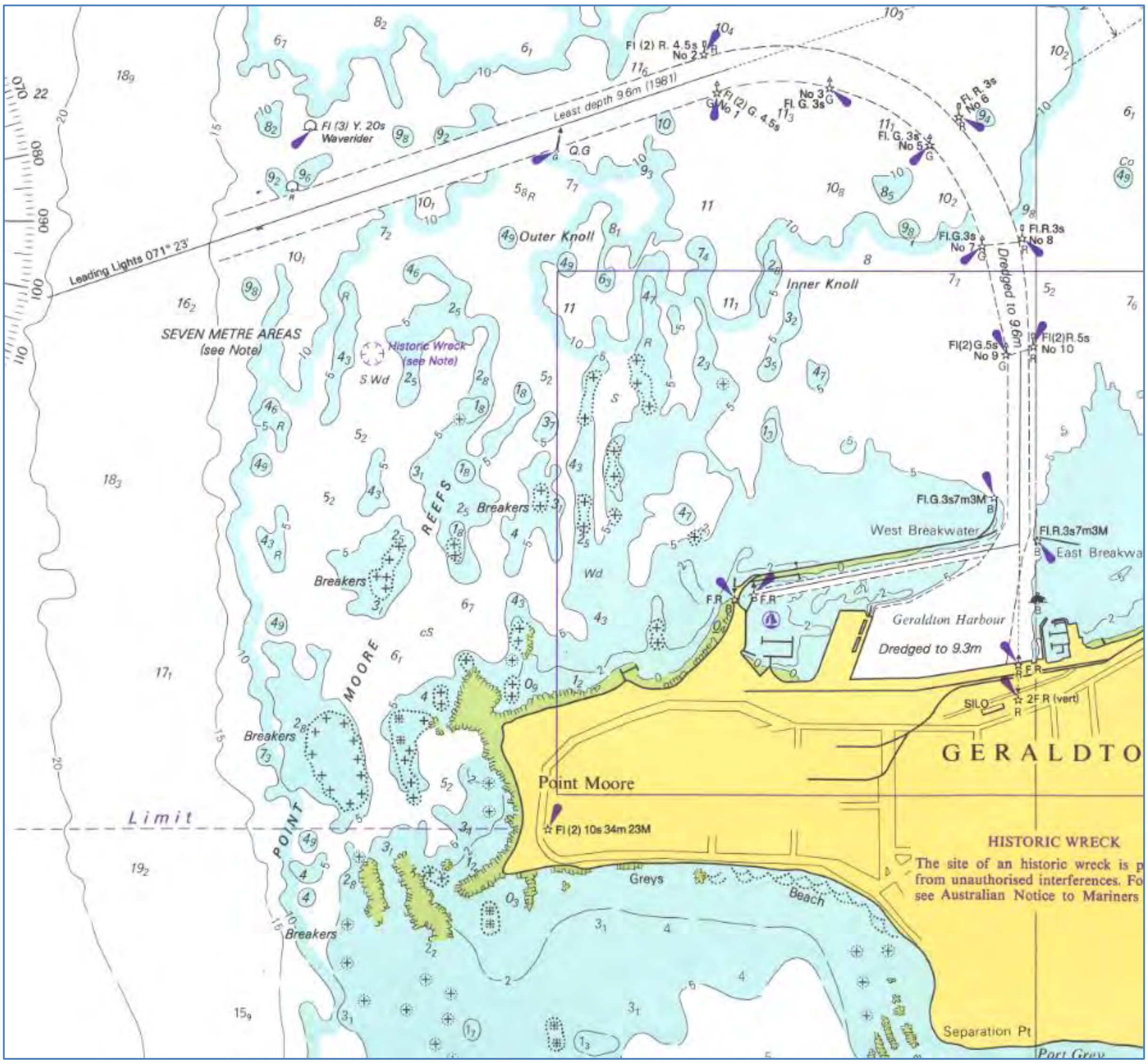


Figure 2.1 Extract from Local Nautical Chart (Source: AHS AUS81)

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.

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 2.2. This figure indicates that the Point Moore shoreline is contained within two secondary sediment cell (cell 14 & cell 15). The potential impacts caused by coastal processes in the relevant sediment cells have been accounted for as part of this investigation.

In particular, the sediment movement in the study area consists of sediment transport across the boundary of cells 14 and 15, which accumulates on the beach (Pages Beach) west of the Geraldton Port, where it is managed by the Mid-West Ports Authority (MWPA) through sand extraction. The sand extraction carried out by the MWPA has been extracting sand from Pages Beach since 2000 to carry out sand nourishment on beaches north of Point Moore. Therefore, the effect of the sand extraction from Pages Beach on the shoreline has been analysed and incorporated into this study to provide a more robust coastal processes assessment.

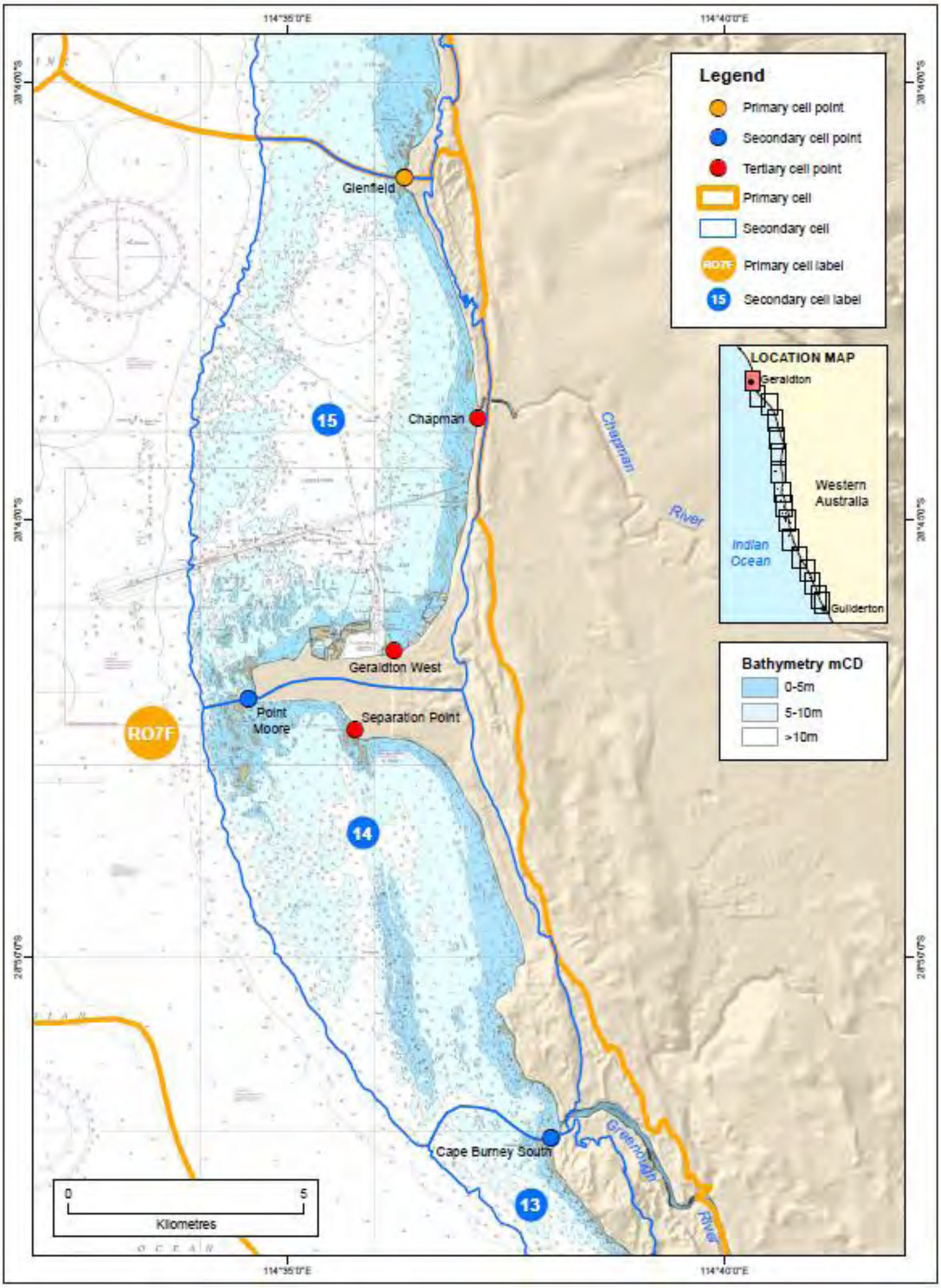


Figure 2.2 Secondary Sediment Cells near Geraldton (source: Stul et al 2014)

2.3 Site Inspection

A site inspection was carried out by coastal engineers from MRA in autumn 2015. During the site inspection it was noted that the shoreline is generally consistent with that described by Short (2006).

Based on the aspect, exposure and observed local characteristics, the shoreline was categorised into three sectors. These beach sectors are shown in Figure 2.3.

Figure 2.3 Beach Sectors

Sector 1 of the shoreline covers the southern side of Point Moore, which includes a large part of Greys Beach. The western extent of this section is a low sandy foreland formed in lee of attached intertidal calcarenite reefs, with additional reefs extending up to 1 km east of Point Moore (Short, 2006). This section is generally orientated towards the dominant waves and winds (South). However, the waves are reduced by the reefs and result in a low tide terrace being maintained, which may be eroded during periods of higher waves. The shoreline is generally backed by a high, scarped, and vegetated fore dune.

Photos showing this section of the shoreline are presented in Figure 2.4.

Figure 2.4 Photography showing Sector 1 of the shoreline

Sector 2 of the shoreline extends from the western side of the sandy foreland (western extent of Sector 1) and curves north for about 800 m (Short, 2006). The shoreline is topographically controlled by reef platforms and inshore reefs. A reef platform fronts the northern half of the beach, this combines with the outer reefs to provide protection to the shoreline. This results in a lower energy, and consequently a low tide terrace is maintained along this section of beach. As a result, this stretch of shoreline is generally backed by a wide, though relatively low elevation dune reserve. Photos showing this section of the shoreline are presented in Figure 2.5.

Figure 2.5 Photography showing Sector 2 of the shoreline

Sector 3 extends east from the prominent sandy foreland (northern extent of Sector 2) to the western breakwater at the Geraldton Harbour. This sector consists of two beaches, the first beach commences at the sandy foreland and extends west up to a shore parallel groyne, and is known as Explosive Beach. The second beach is known as the Pages Beach, which is located between the groyne and the western side of a 1.5 km long breakwater that formes the main protection for Geraldton Harbour.

This section of the shoreline generally consists of a flat, continuous, and sandy beach. At Pages Beach the shoreline is backed by a narrow dune reserve, then a large recreational reserve. West of the groyne, the shoreline is backed by a wide, vegetated dune reserve (Short, 2006). This stretch of beach is sheltered from most waves by the western boundary reef and its north to north westerly orientation, with generally low waves to calm conditions at the shore.

During the site inspection, beach rock was not observed along the shoreline. Photos showing this section of the shoreline are present in Figure 2.6.

Figure 2.6 Photography showing Sector 3 of the shoreline

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. 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 on a sandy coast, an allowance for future shoreline erosion should be provided. 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).

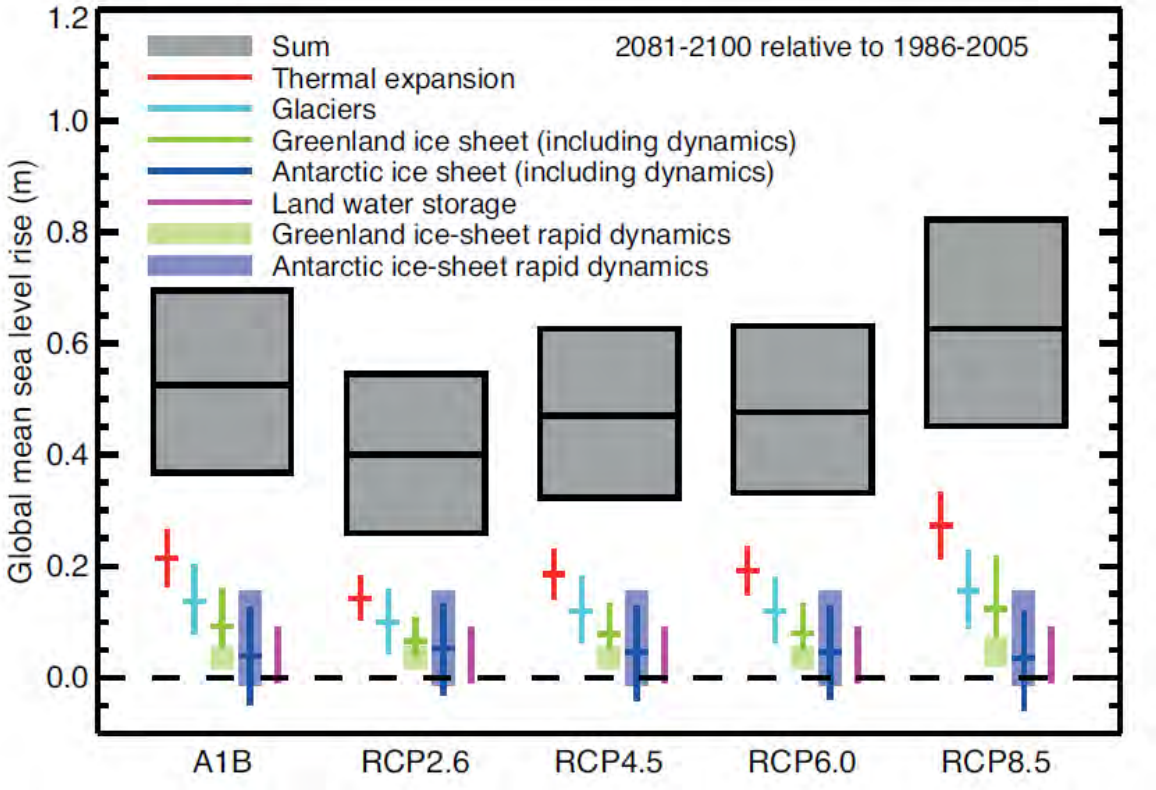


Figure 3.1 IPCC Scenarios for sea level rise (IPCC 2013)

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.

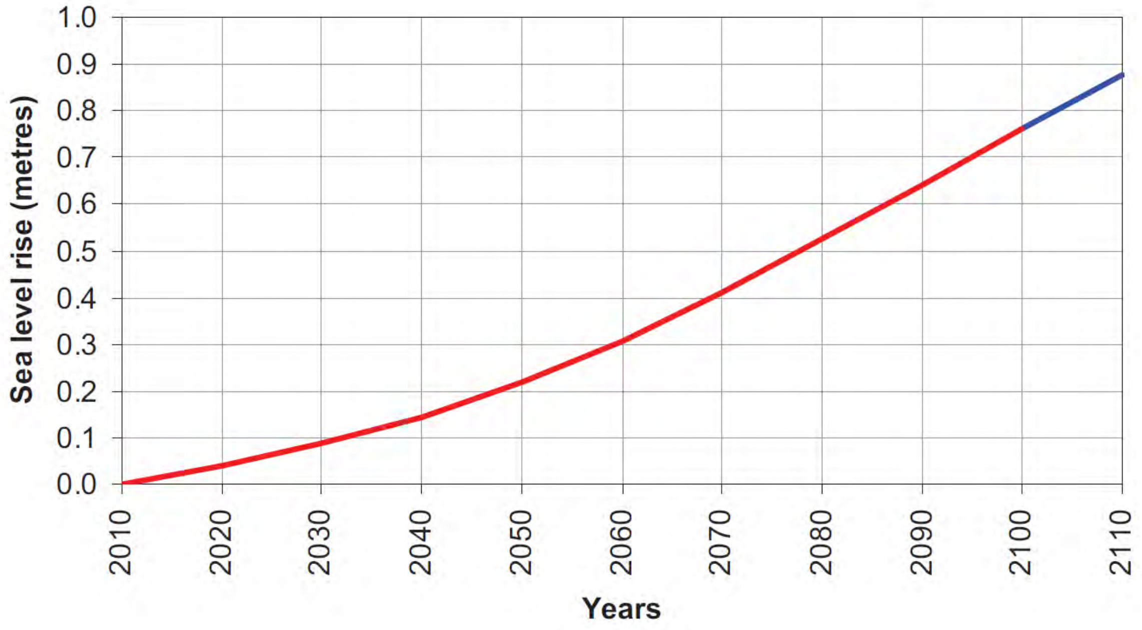


Figure 3.2 Recommended Sea Level Rise Scenario for Coastal Planning in Western Australia (DoT 2010)

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 coastal development, which includes coastal setback distances (required to minimise the potential for erosion of development areas and foreshore infrastructure) and development levels (required to minimise the potential for coastal inundation).

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. Therefore, to provide a relatively low risk of development being adversely impacted by coastal inundation over this planning horizon the development levels required by the Policy are based on the 500 year average recurrence interval (ARI) event, plus an allowance for sea level rise over the 100 year planning horizon.

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. The 20, 100 and 500 year ARI events are presented in Table 4.1 and 4.2. 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 – Point Moore North

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	1.9	2.6	3.3
2030	0.07	2.0	2.7	3.4
2070	0.39	2.3	3.0	3.7
2110	0.90	2.8	3.5	4.2

Table 4.2 Cyclonic Storm Inundation Levels – Point Moore South

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	1.9	2.2	2.9
2030	0.07	2.0	2.3	2.9
2070	0.39	2.3	2.6	3.2
2110	0.90	2.8	3.1	3.8

4.1.1 Assessment Methodology

The approach adopted by MRA to determine the potential cyclonic storm surge inundation levels at Point Moore 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 (the closest location to Point Moore) are only available for a period with a duration totalling around 30 years between 1986 and 2015.

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 Point Moore. 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 Point Moore.
- Complete an extreme analysis of peak recorded water levels for Point Moore.

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).

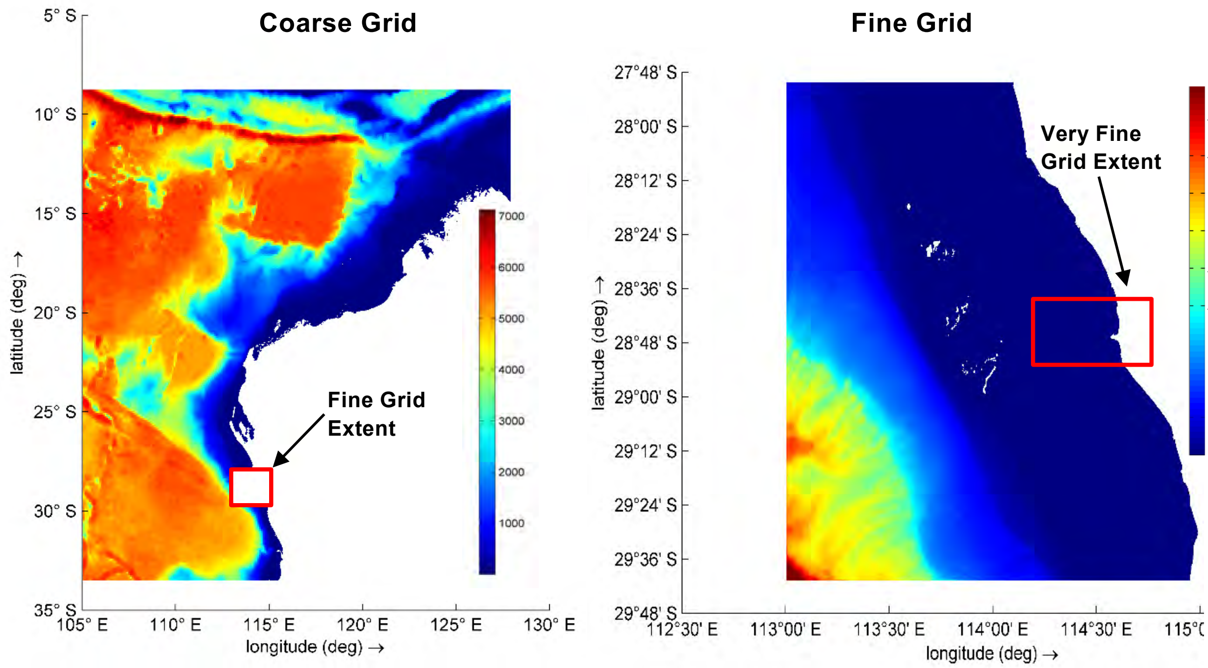


Figure 4.1 Model Domain & Bathymetry for Delft3D Coarse & Fine Grids

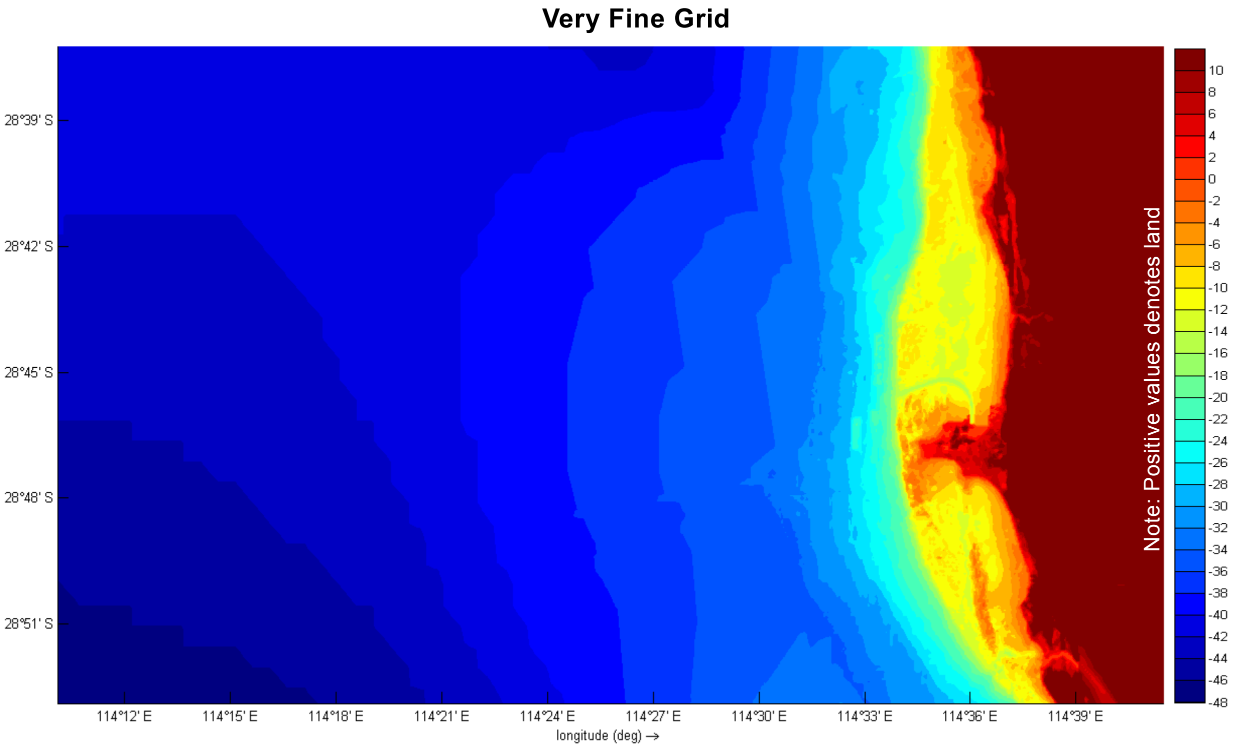


Figure 4.2 Model Domain, Topography & Bathymetry for Delft3D Very Fine Grid

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 Point Moore is provided in Table 4.3. 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 4.3 Historical Cyclones affecting Point Moore Region 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		

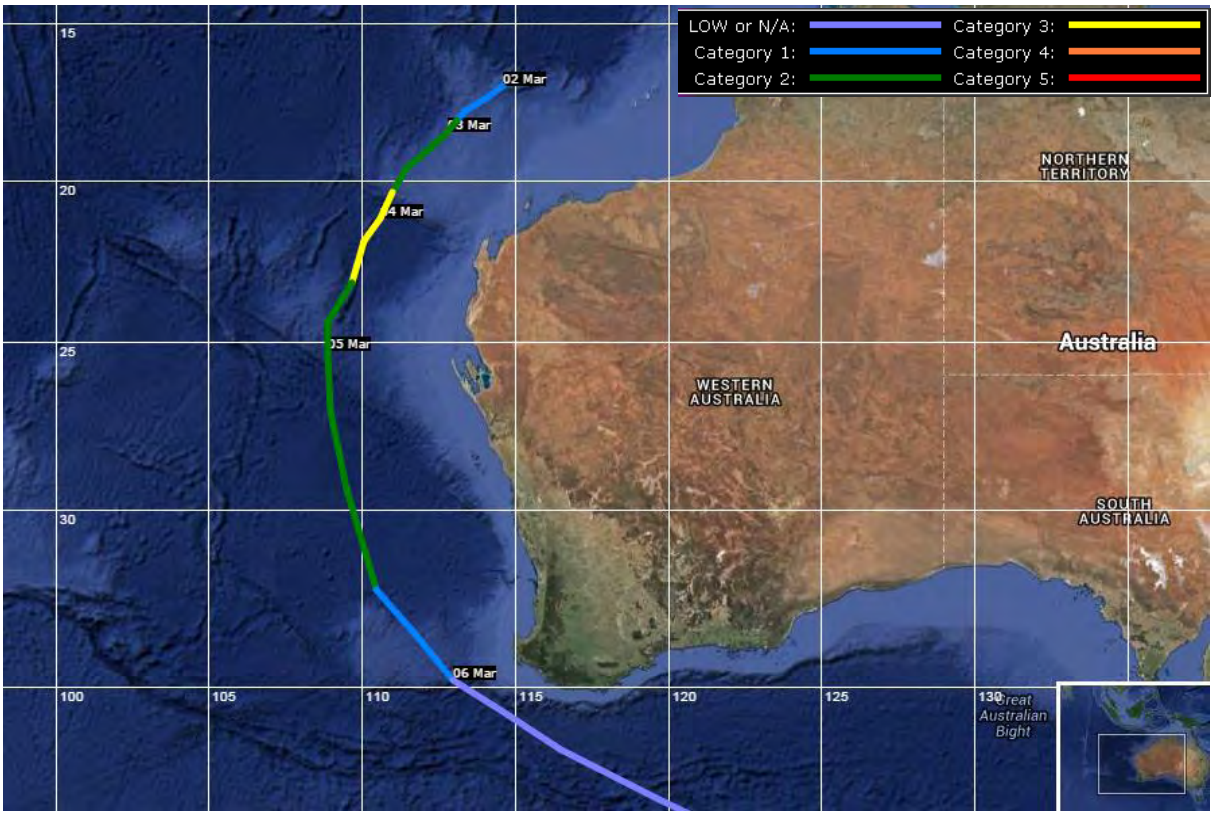


Figure 4.4 Tracks & Severity Plot for TC Vincent

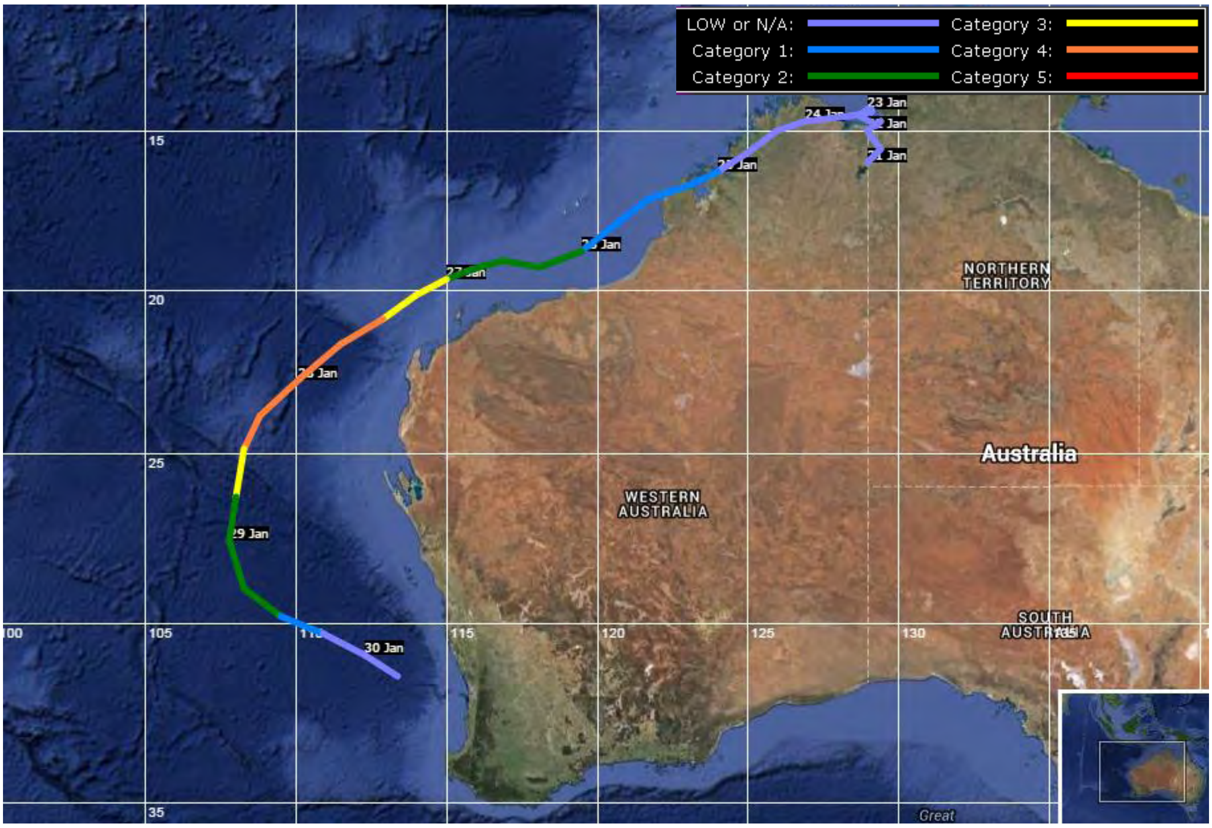


Figure 4.5 Tracks & Severity Plot for TC Bianca

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 trail of the event was not modelled).

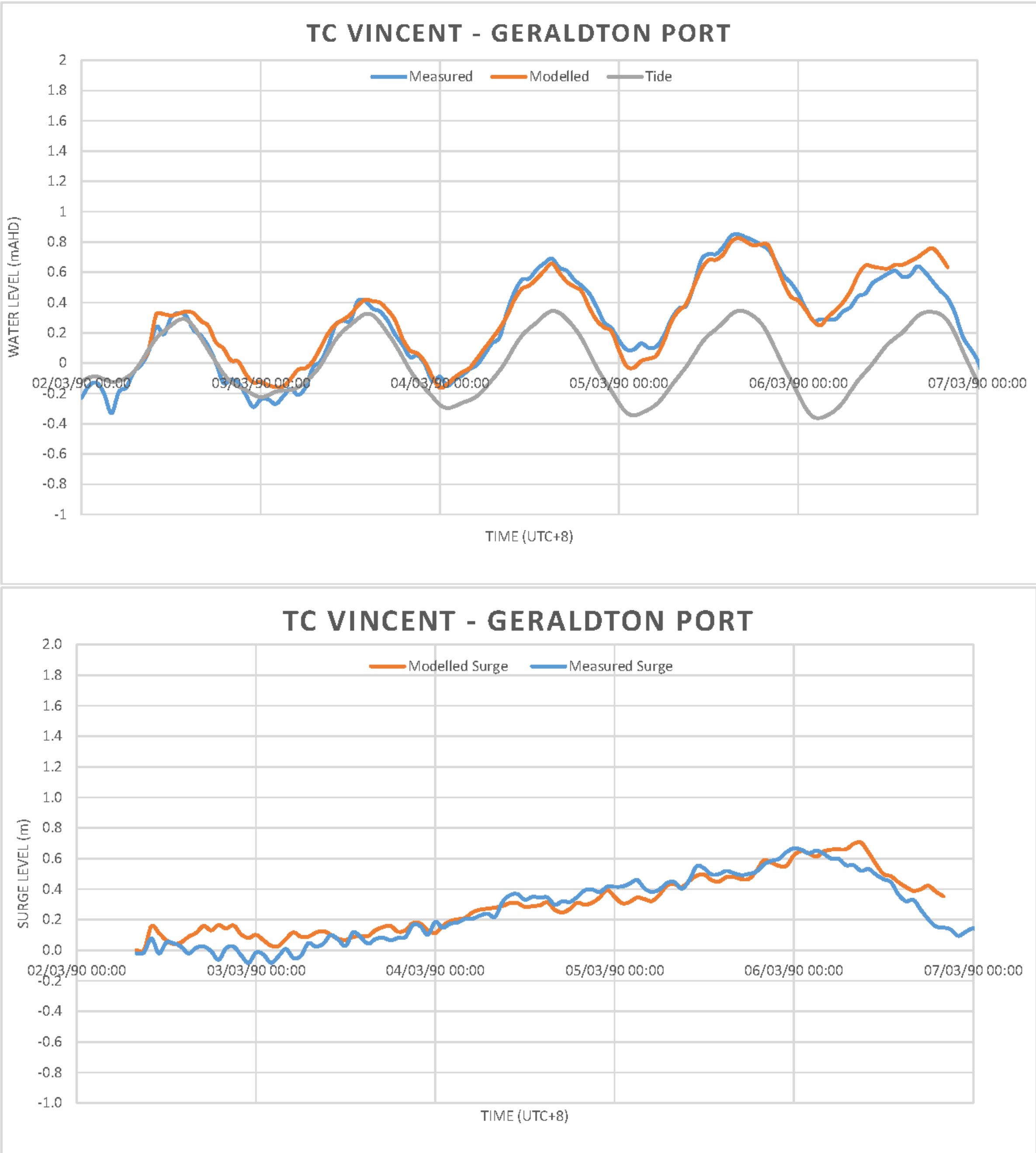


Figure 4.6 Tracks & Severity Plot for TC Vincent

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.

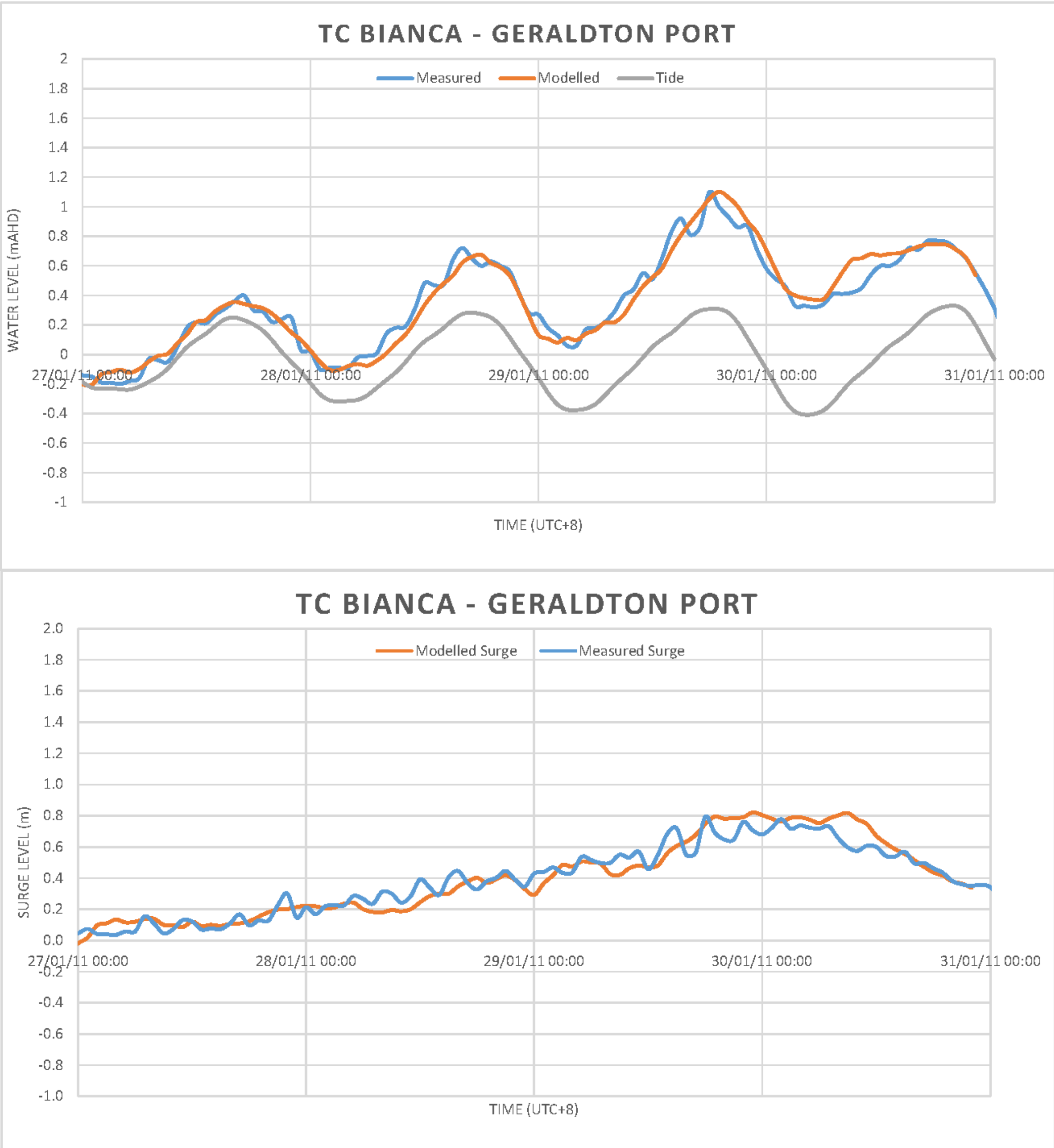


Figure 4.7 Tracks & Severity Plot for TC Bianca

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 measurement taken at the outer channel. The refined index of agreement between the measured and modelled wave height is approximately 0.7, indicate 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.

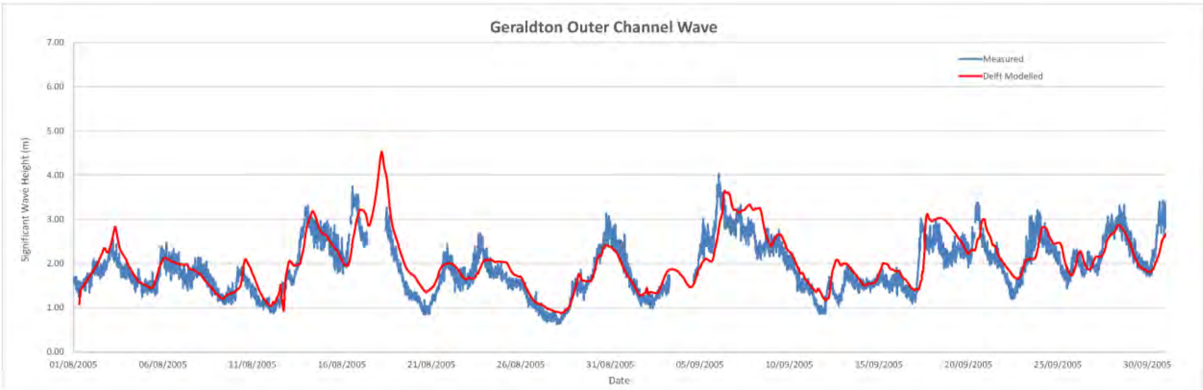


Figure 4.8 Measured versus Modelled Wave Height at the Outer Channel

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.

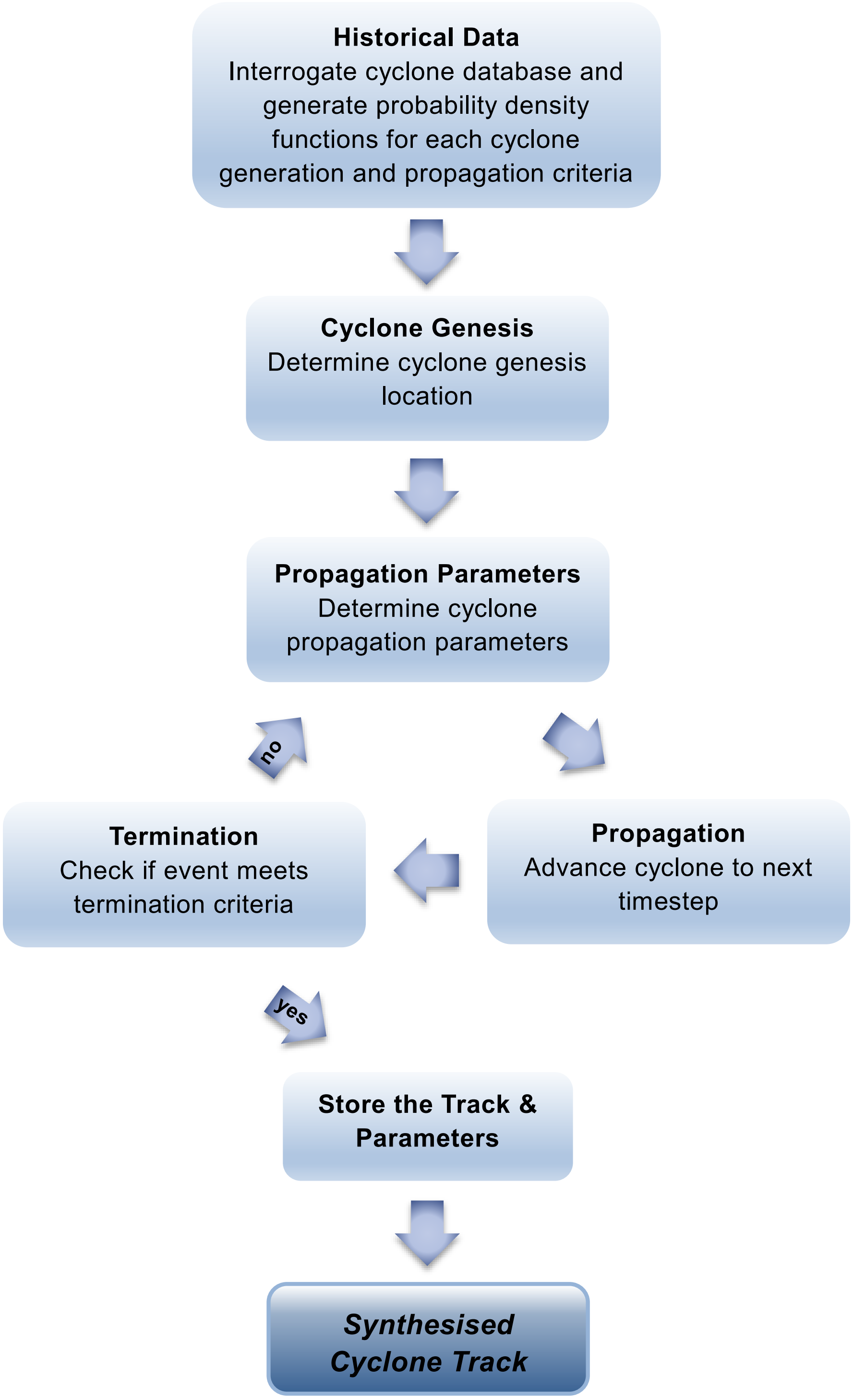


Figure 4.9 Monte Carlo Simulation Scheme

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.

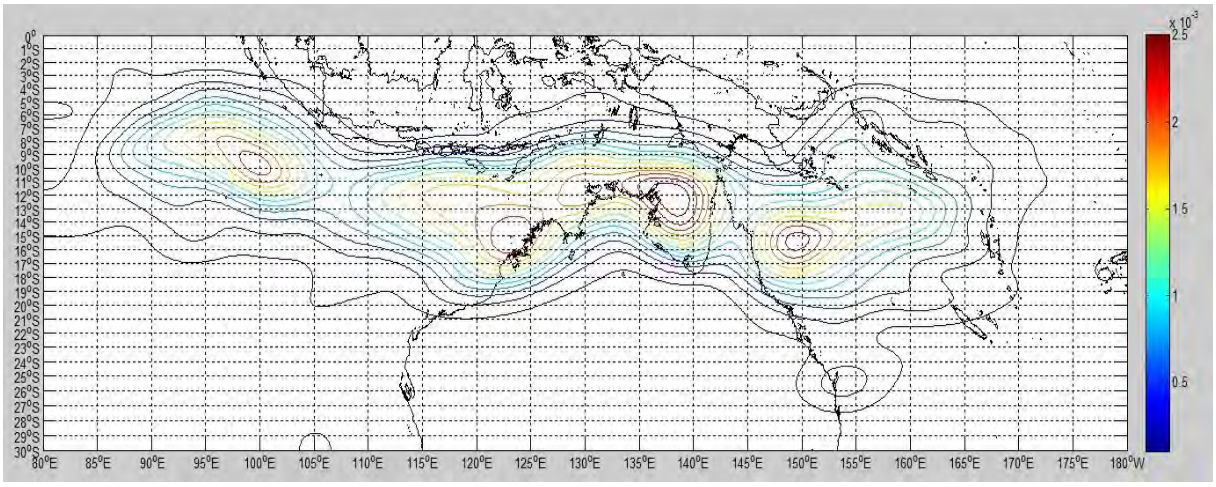


Figure 4.10 Smoothened Genesis Probability Distribution – 2D Plan View

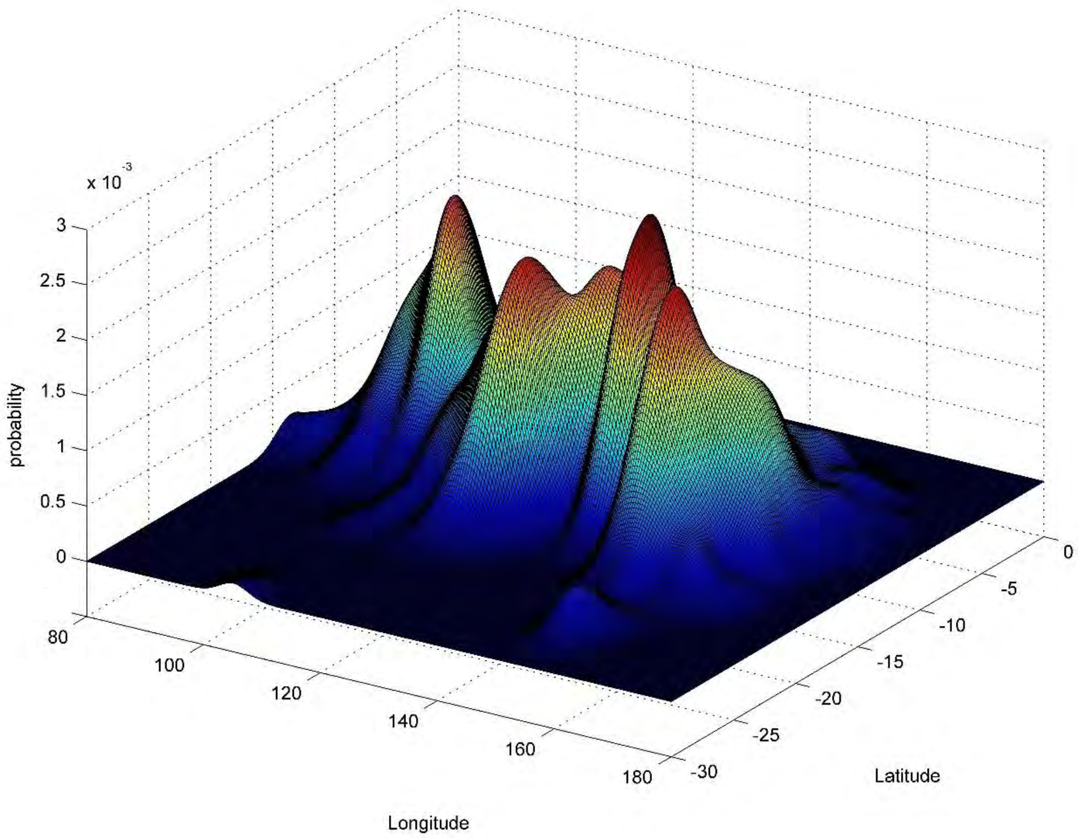


Figure 4.11 Smoothened Genesis Probability Distribution – 3D View

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.

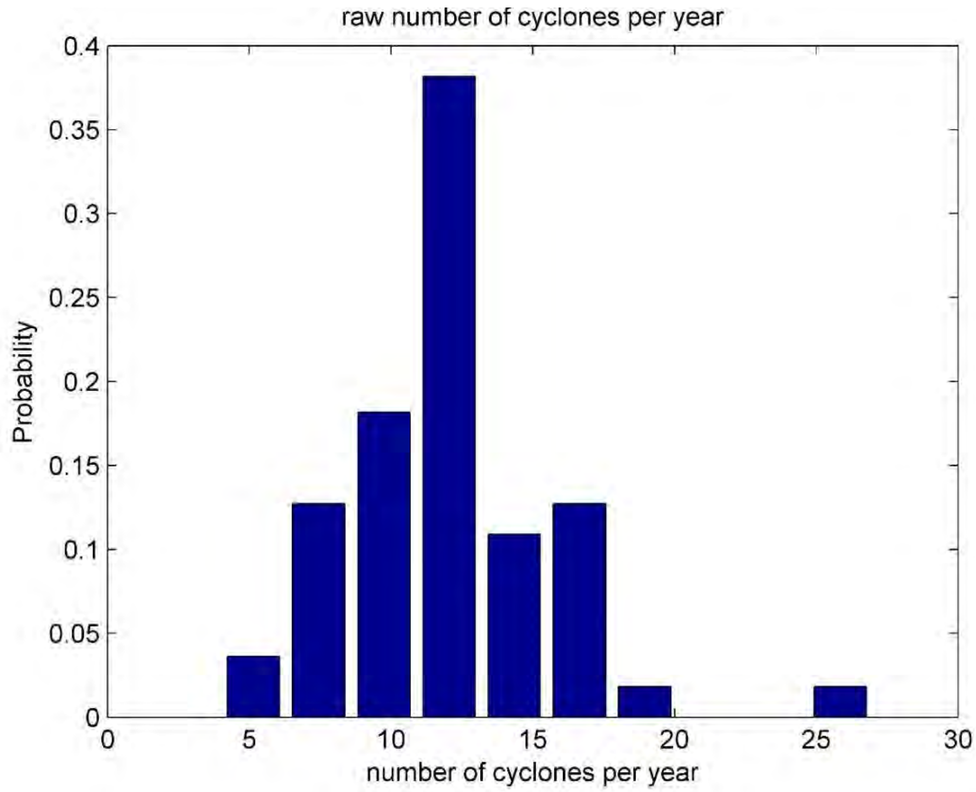


Figure 4.12 Probability of the number of cyclones per year within the Australian Region

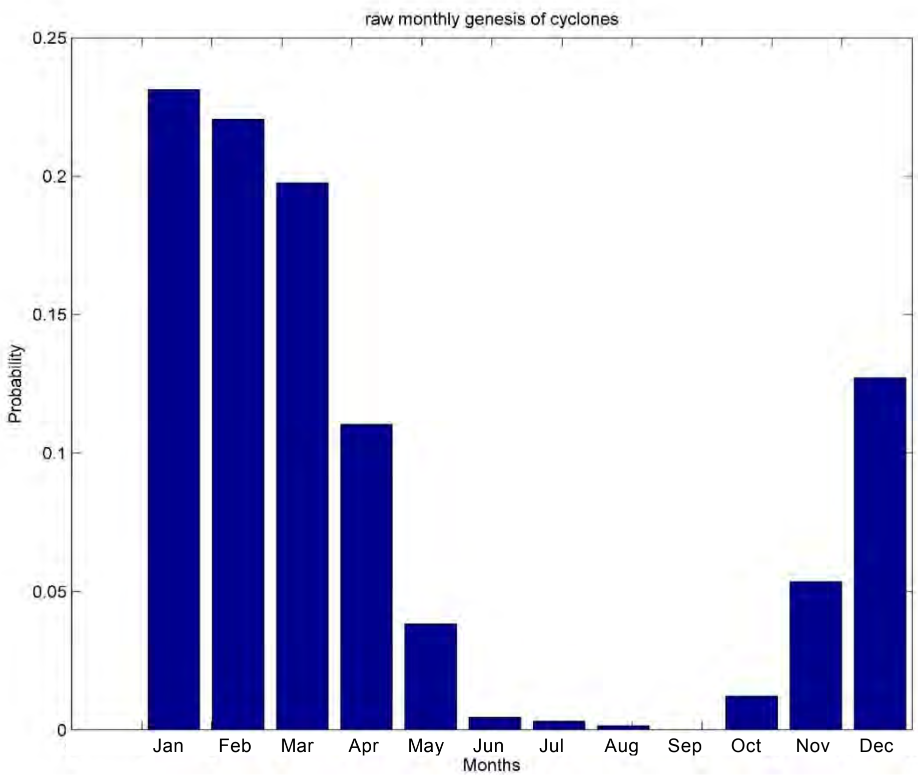


Figure 4.13 Probability of monthly occurrence within the Australian Region

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.

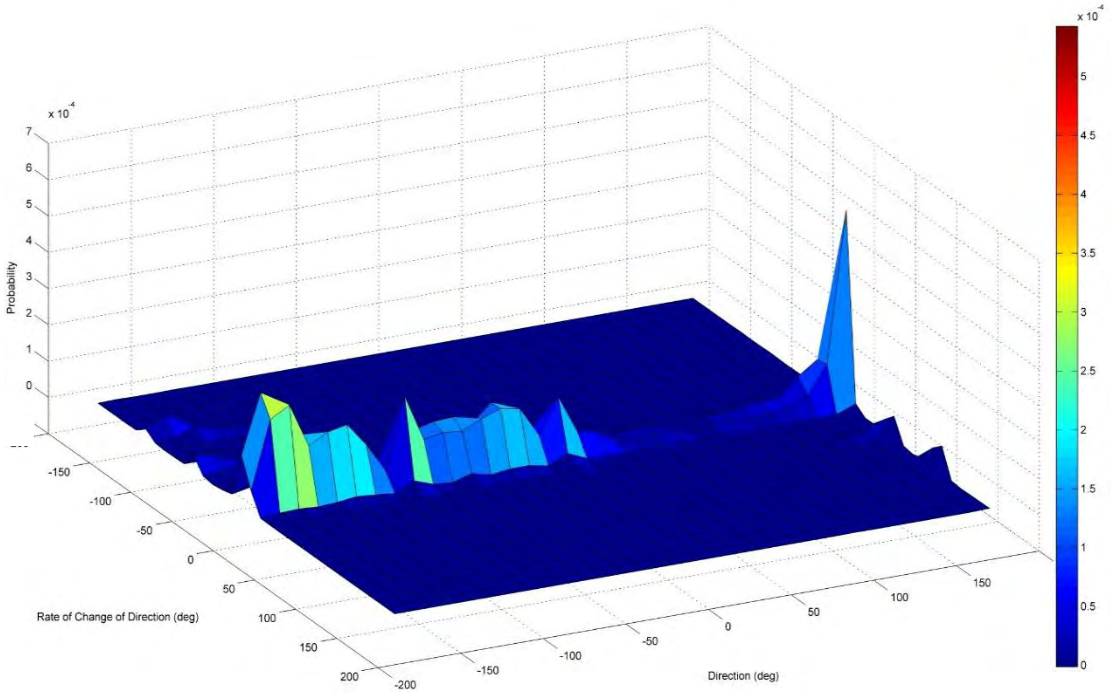


Figure 4.14 Probability Surface for Rate of Change of Direction vs Direction

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).

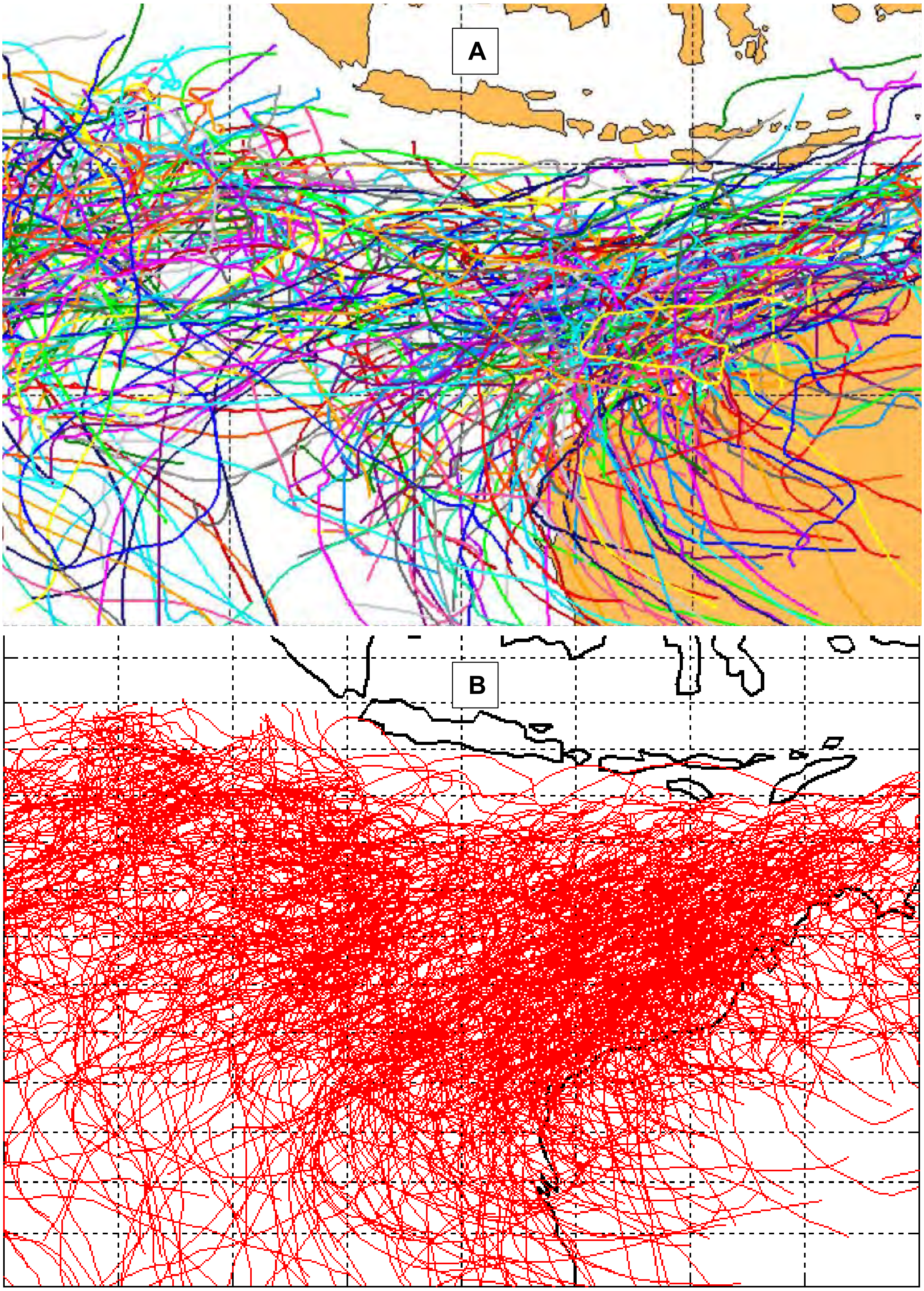


Figure 4.15 (A) Historical cyclone tracks since 1960; &
(B) Modelled cyclone tracks for the same period

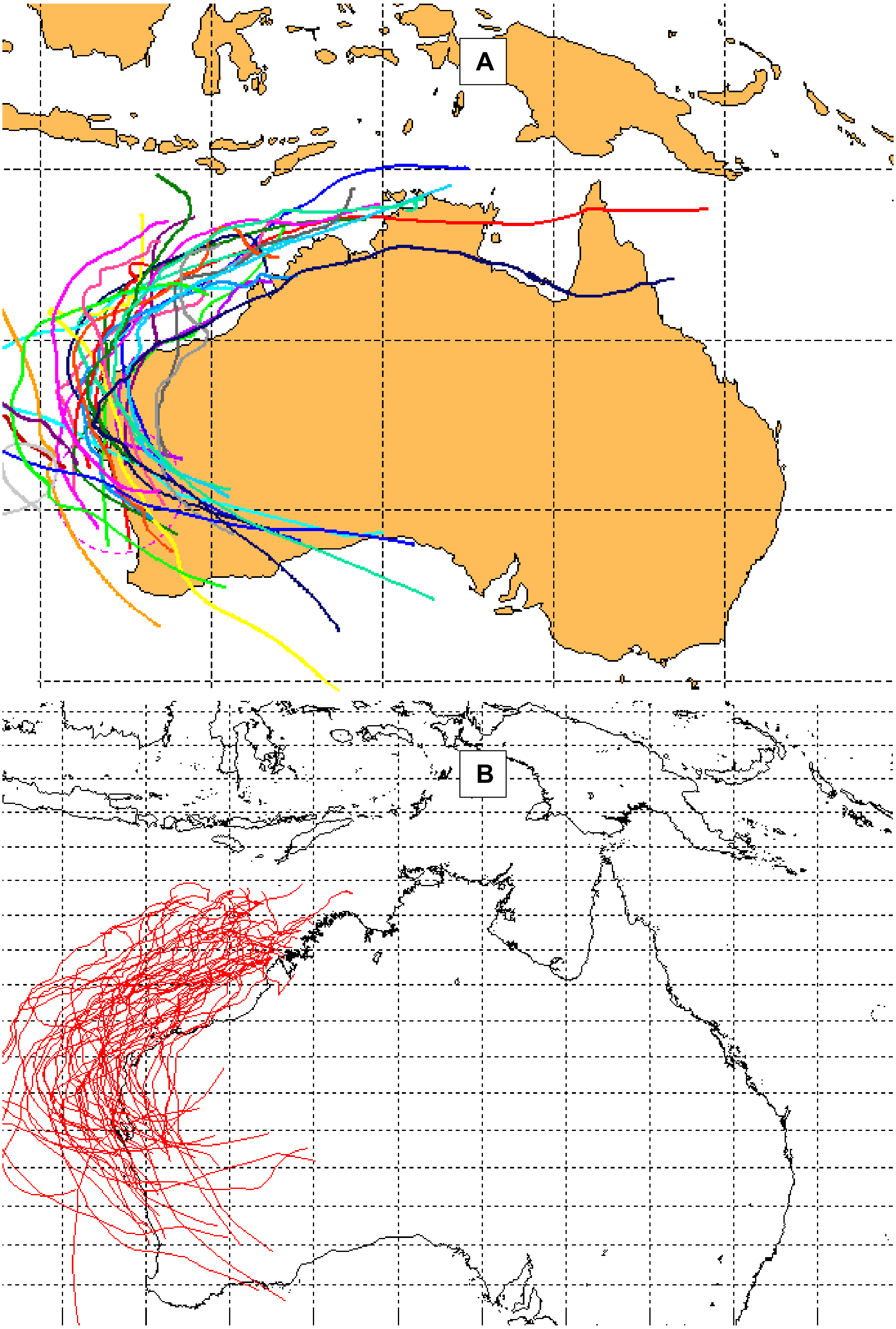


Figure 4.16 (A) Historical cyclone tracks affecting Geraldton since 1960; &
(B) Modelled cyclone tracks affecting Geraldton for the same period

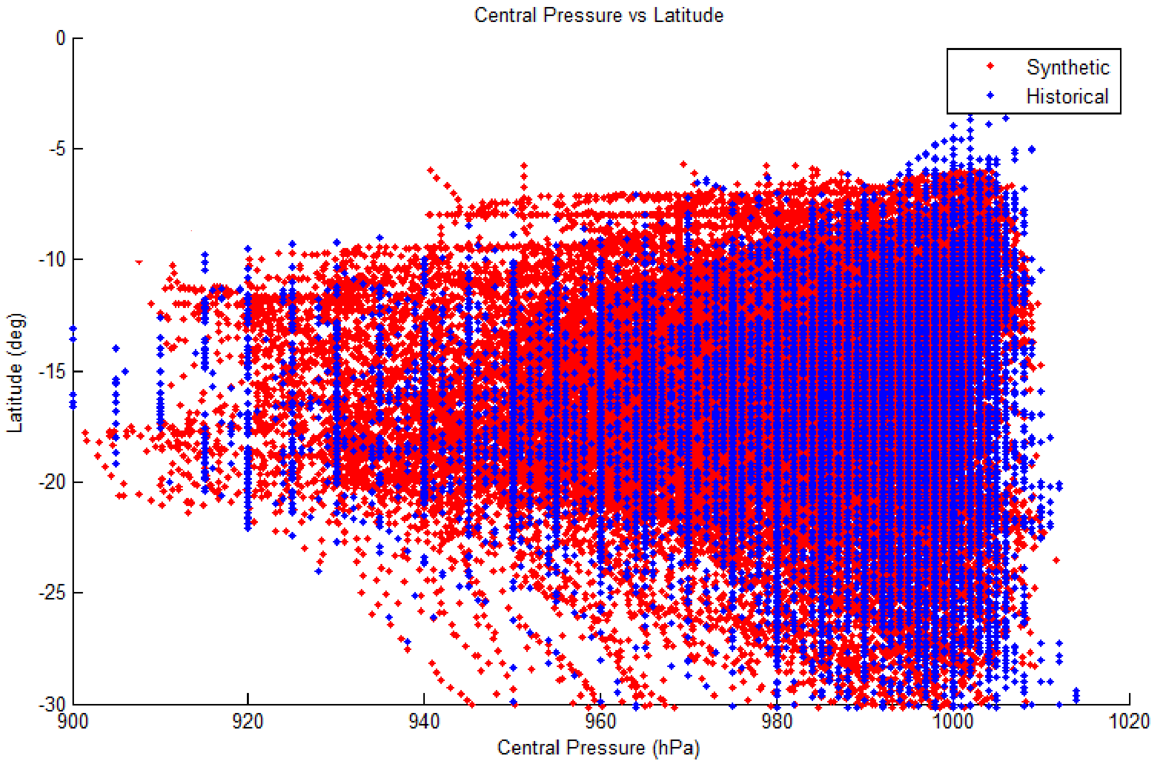


Figure 4.17 Scatter plot of central pressure versus latitude; measured and modelled

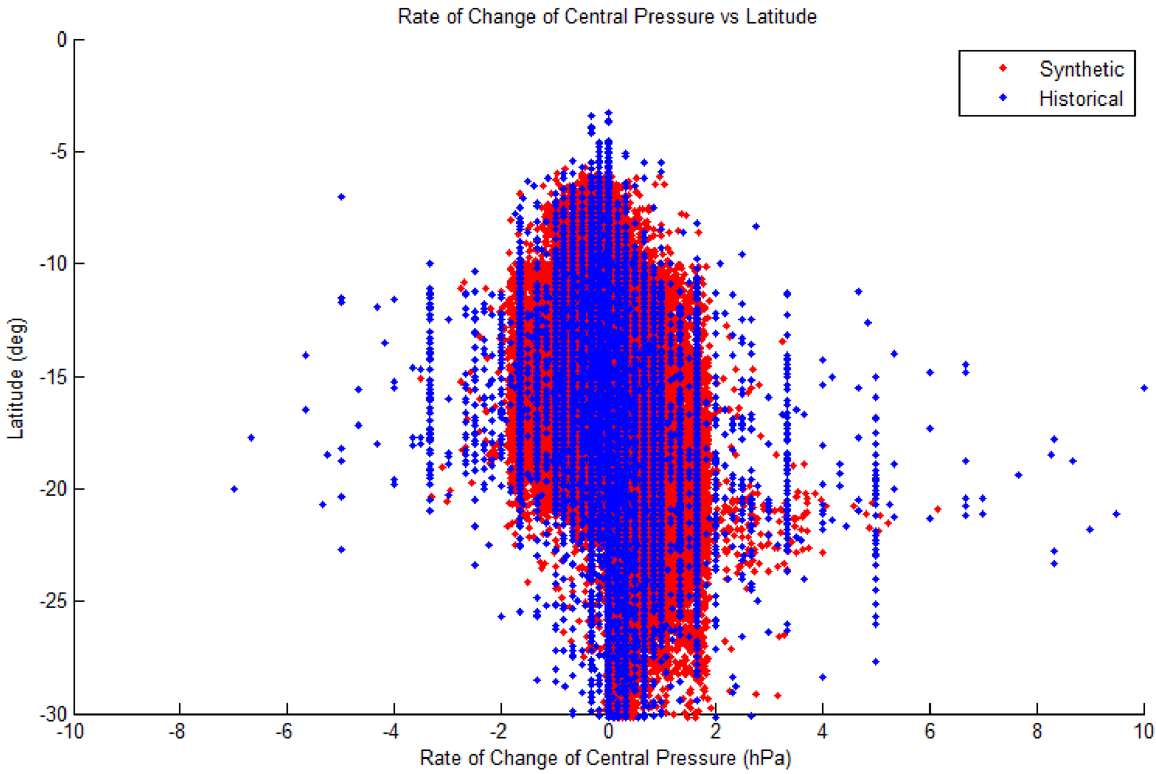


Figure 4.18 Scatter plot of rate of change of central pressure versus latitude; measured and modelled

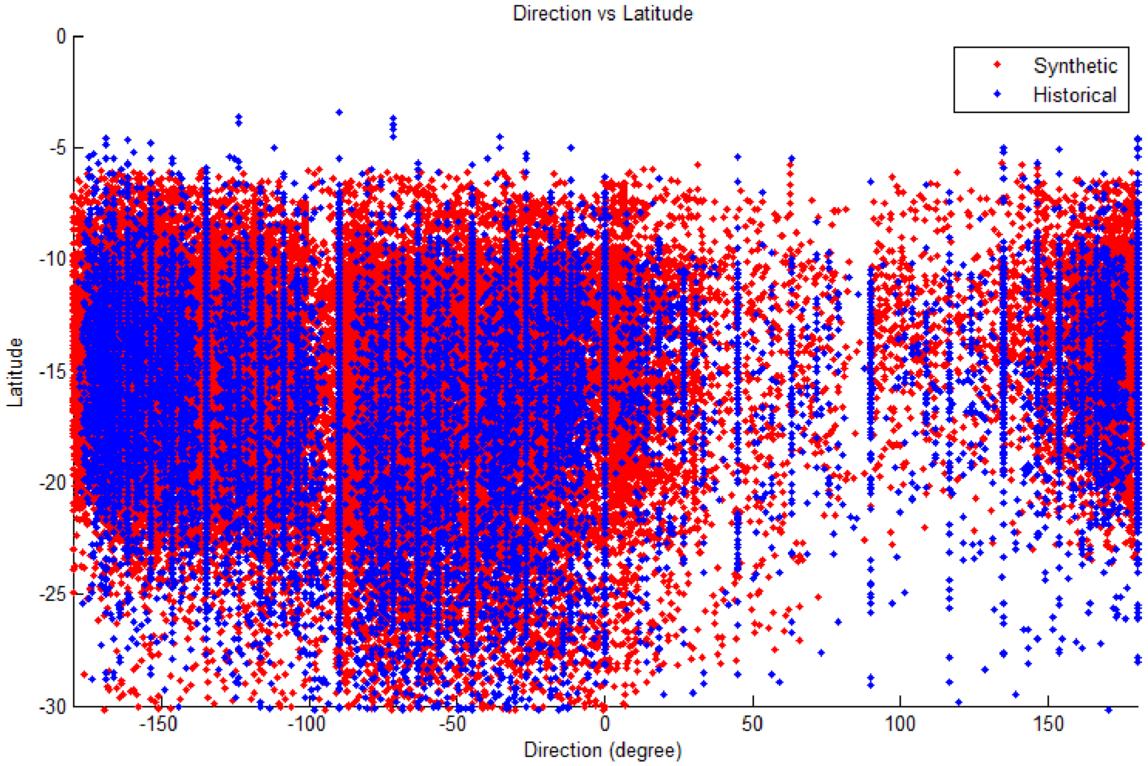


Figure 4.19 Scatter plot of cyclone travel direction versus latitude; measured and modelled

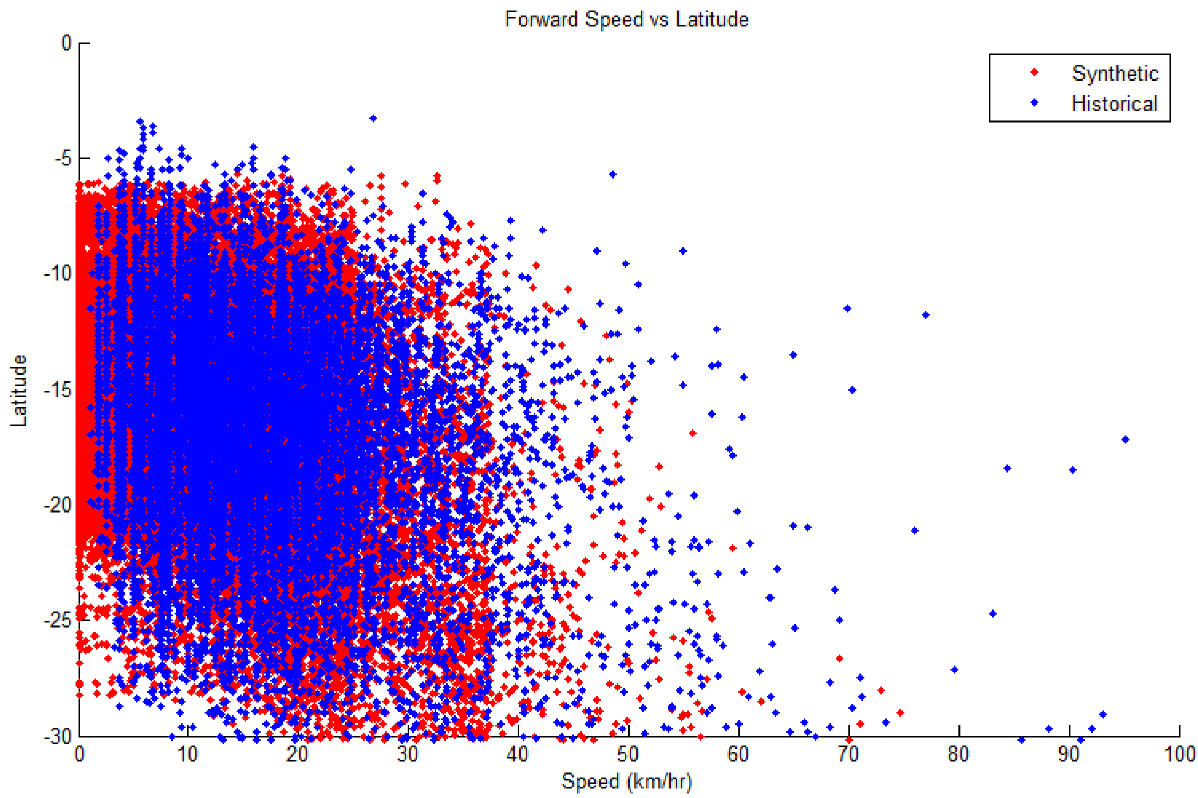


Figure 4.20 Scatter plot of cyclone forward speed versus latitude; measured and modelled

For the purpose of this study, the model continues to track cyclones as they degrade into extra-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 Point Moore 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 Point Moore.

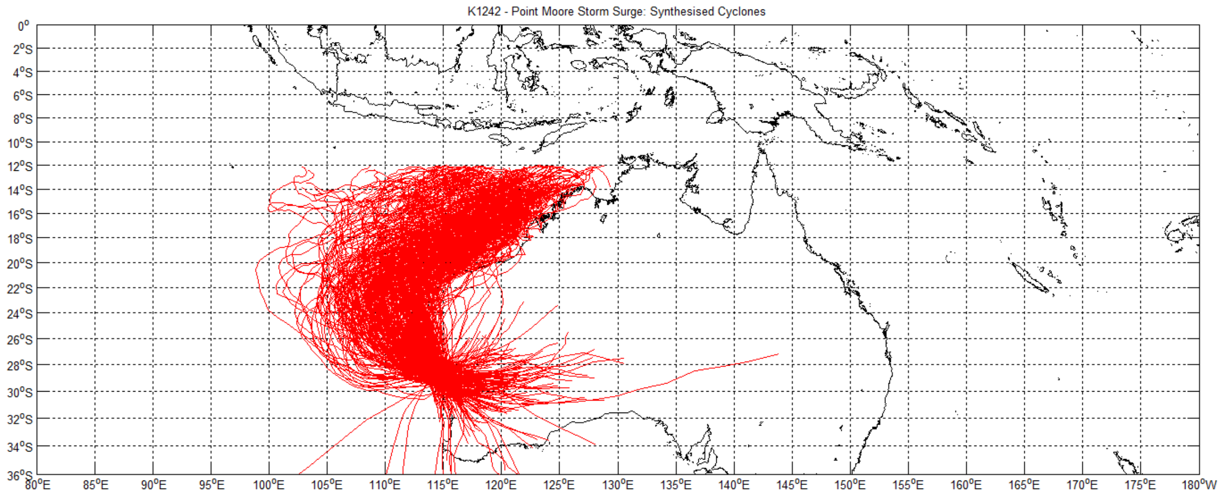


Figure 4.21 Plot of synthesised cyclone tracks within 100 km of Point Moore

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 Point Moore shoreline 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 0.8 to 1.1 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 Point Moore.

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 Point Moore. 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).

SPP2.6 requires that the extent for calculation of coastal processes be defined based on the coastal geology/geomorphology. In this regard, it is noted that the cyclonic storm induced inundation will be different for the shorelines on the northern and southern sides of Point Moore. This is mainly due to the fact that to maximise the associated water levels at the shoreline requires onshore winds.

The ability to generate onshore winds on the northern and southern shorelines of Point Moore is fundamentally reliant on the location of the cyclone or ex-cyclone centre. For example, onshore winds will be experienced on the northern shoreline when the cyclone or ex-cyclone is generally located to the west of Point Moore, while onshore winds on the southern shoreline will generally be associated with a cyclone or ex-cyclone located to the east of Point Moore. By virtue of this fact, onshore winds on the northern shoreline will typically be stronger than those on the southern shoreline due to the fact that the cyclone or ex-cyclone is more likely to be located over the ocean (west of Point Moore) and therefore be stronger than a cyclone located to the east of Point Moore and over land. In addition, the effect of the forward speed of the cyclone, which is most likely to be travelling in a southerly direction, will also cause an increase in wind speeds from the north (due to the additive effects of the cyclone forward speed and wind speed), compared to the wind speeds from the south as the system travels away from Point Moore. The effect of these factors, acting in concert, results in differences in storm surge and inundation levels north and south of Point Moore. This is illustrated in Figures 4.22 and 4.23, which show spatial plots of the model output from a severe cyclone event east and west of Point Moore.

Given the above, the data interrogation, extraction and ranking processes were carried out on the northern and southern side of Point Moore, as defined by the sediment cells shown in Figure 2.2. The results of the extreme analysis for cyclonic storm surge inundation (north and south of Point Moore) are presented in Table 4.1 and 4.2 for the 20, 100 and 500 year ARI events in four key planning timeframes: present day, 2030, 2070 and 2110.

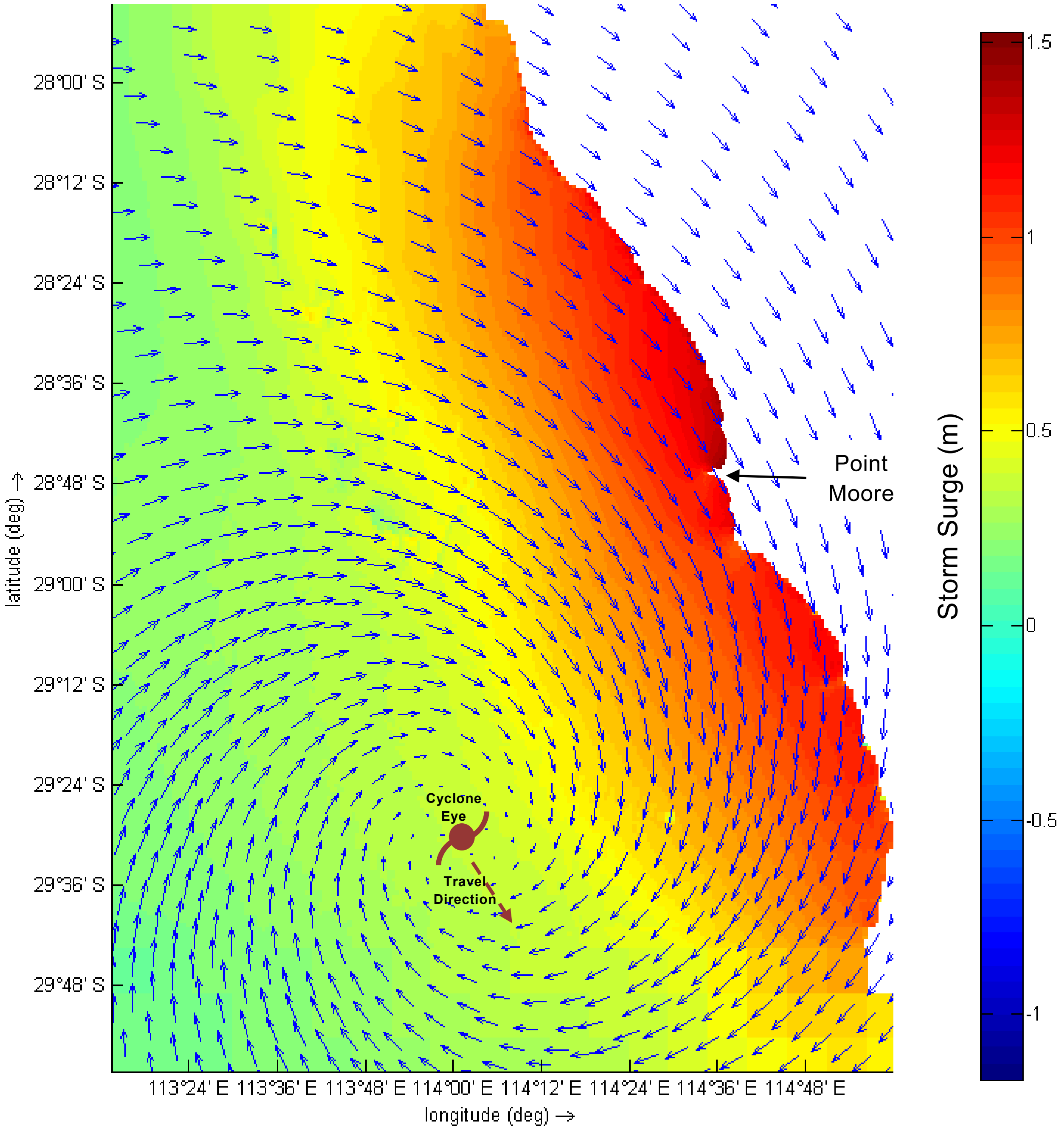


Figure 4.22 Delft3D output plot showing a typical cyclone wind field while tracking to the west of Point Moore

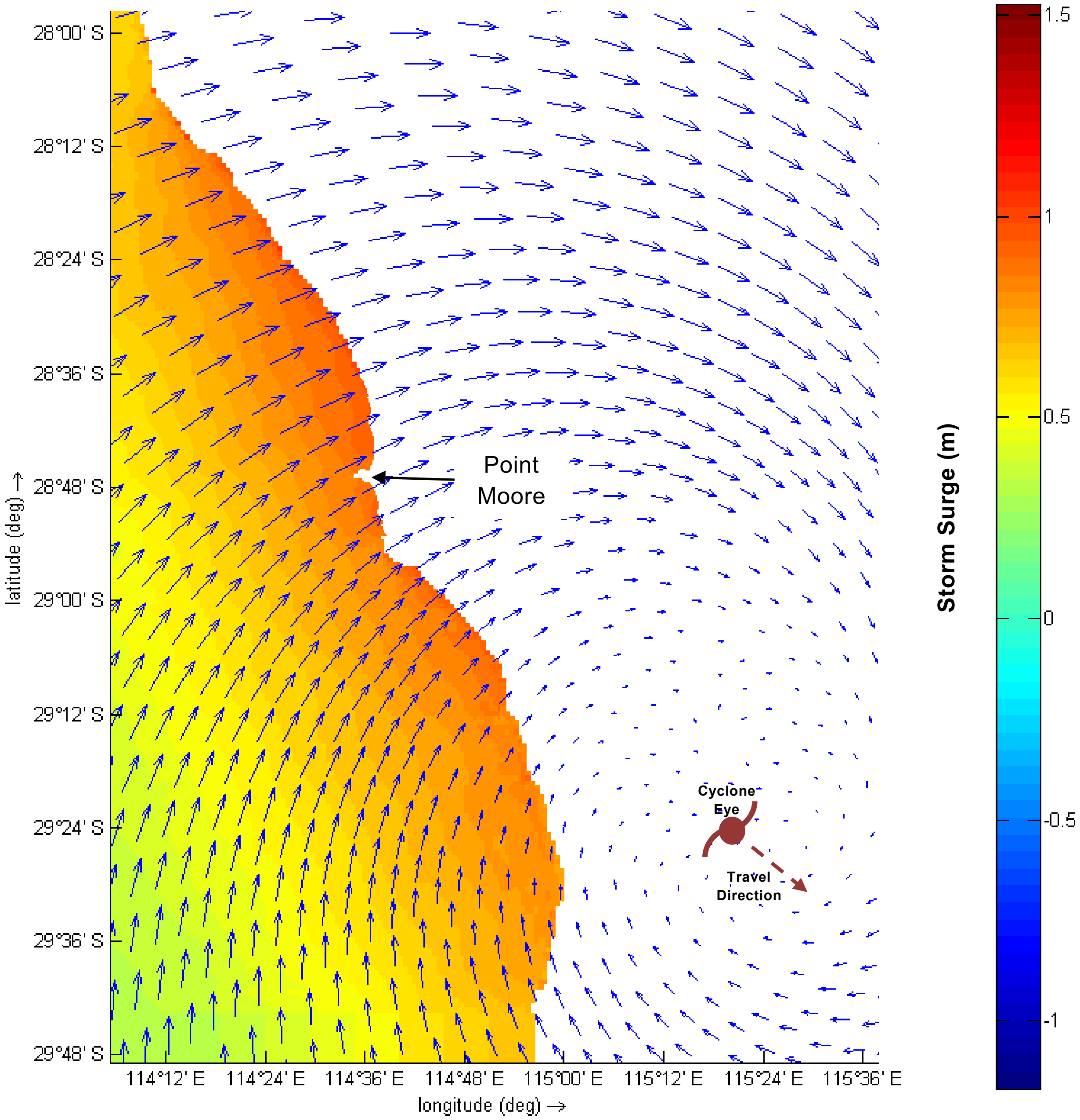


Figure 4.23 Delft3D Output plot showing a typical cyclone wind field while tracking to the east of Point Moore

4.2 Non Cyclonic Storm Surge Assessment

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. The result of this assessment is presented in Table 4.4. Details of this assessment are discussed in the following sections.

Table 4.4 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

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 4.5.

Table 4.5 DoT supplied Water Level Data Details

Start	Finish	Frequency
January 1966	December 1986	1 hour[1]
April 1986	December 1999	15 minutes
January 2000	March 2015	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 4.6.

Table 4.6 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 4.6 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.

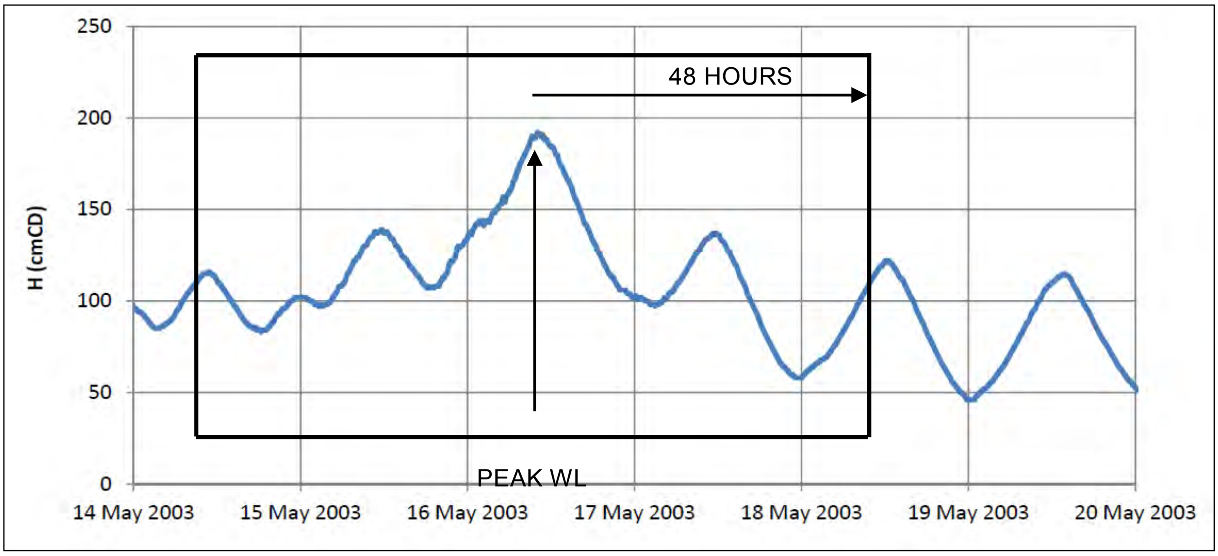


Figure 4.24 Separation of Individual High Water Level Events

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 2015 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 levels were presented previously in Table 4.4, 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.

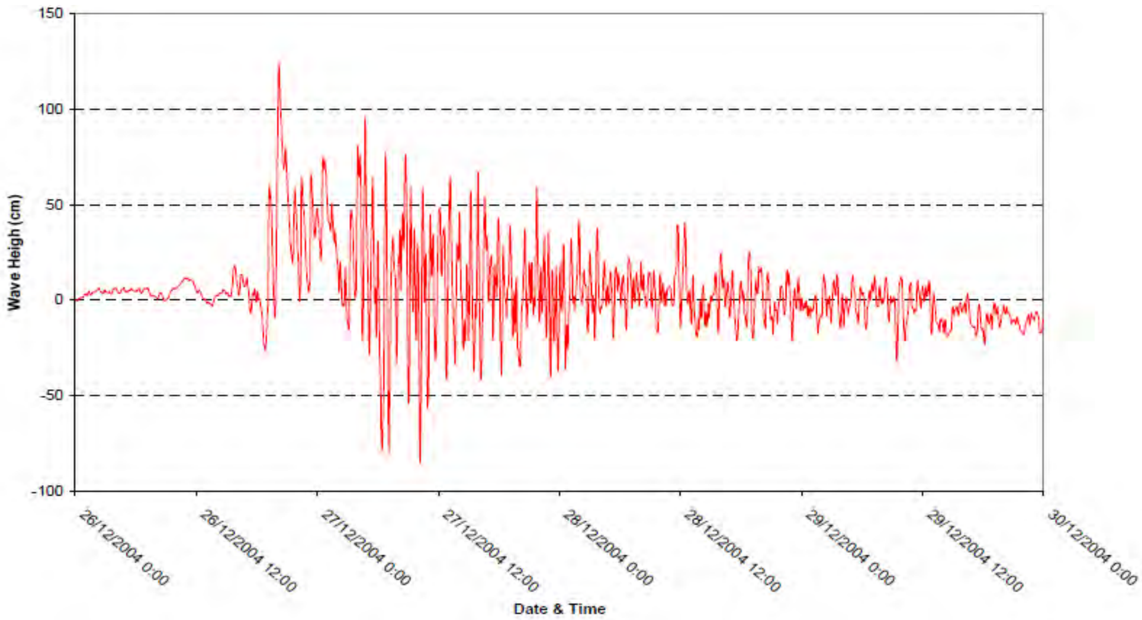


Figure 4.25 December 2004 Tsunami Signal (Source: Geoscience Australia, 2010)

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 4.25, 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.

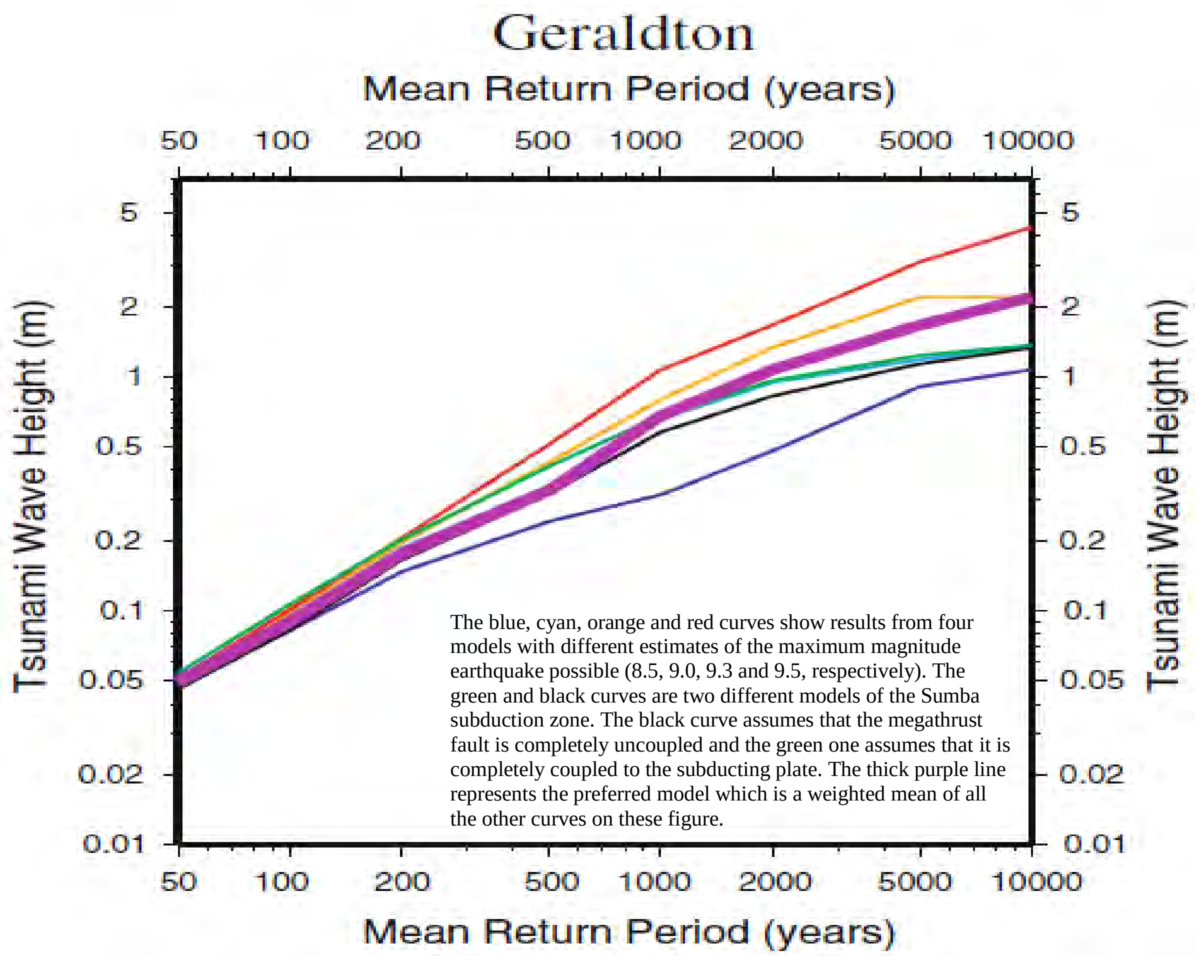


Figure 4.26 Tsunami Hazard Curves for Geraldton (Source: Geoscience Australia, 2008)

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.

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

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 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 Point Moore, 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 Point Moore is presented in Tables 4.7 and 4.8.

The potential inundation levels outlined in Tables 4.7 and 4.8 should be considered as part of the coastal hazard risk management and adaptation planning in order to comply with the requirements of SPP2.6.

Table 4.7 Recommended Coastal Inundation Allowance - Point Moore North

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	2.0	2.6	3.3
2030	0.07	2.1	2.7	3.4
2070	0.39	2.4	3.0	3.7
2110	0.90	2.9	3.5	4.2

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

Table 4.8 Recommended Coastal Inundation Allowance - Point Moore South

Timeframe	Sea Level Rise (m)	20 year ARI (mAHD)	100 year ARI (mAHD)	500 year ARI (mAHD)
Present Day	0	2.0	2.2	2.9
2030	0.07	2.1	2.3	2.9
2070	0.39	2.4	2.6	3.2
2110	0.90	2.9	3.1	3.8

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

In addition to the assessment of the inundation levels, SPP2.6 also requires that where a continuous barrier dune is present, as in the case for Point Moore, the capacity of the dune to provide protection from inundation should be assessed based on the cross sectional area of the dune. It is also noted that on the western and northern side of Point Moore, there are tracks at low elevation that run through the barrier dunes and provide access to the shoreline. There is concern within the Point Moore community that these tracks may become a pathway for inundation during severe inundation events. Therefore, consideration has been given on this aspect in the assessment of the barrier dunes.

SPP2.6 states 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 surrounding the Point Moore development, cross sectional areas of the dune reserve were assessed at three locations against the present day inundation levels in Table 4.7 and 4.8. This assessment is presented in Appendix A.

The result of the cross sectional area assessment show that the barrier dune on the northern and western shorelines of Point Moore have insufficient cross sectional area above even the 20 year ARI inundation level to be considered as an effective barrier. Therefore, this means that during the 20 year ARI inundation event, both the barrier dunes and the low elevation tracks along the western and northern side of Point Moore could be inundated. The barrier dune on the southern shoreline of Point Moore has over 100 cubic metres of cross sectional area above the 500 year ARI inundation level and is therefore considered to be stable for the purposes of this assessment.

Given the above, the maximum extent of inundation was determined without the barrier dune along the northern and western shorelines of Point Moore. This means that even though there may currently be no direct connection between the ocean and low lying areas behind the dunes, for the purposes of this assessment it is assumed that these low lying areas will be inundated. It is acknowledged that this may be different to observations during previous high water level events. For instance, anecdotally a severe event in July 2010 pushed water to the road's edge at Pages Beach, which topographic information suggests is higher than some of the developed areas of Point Moore, yet these low lying areas were not inundated due to the presence of these topographic barriers.

This concludes the coastal inundation assessment for Point Moore. 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 B. 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.

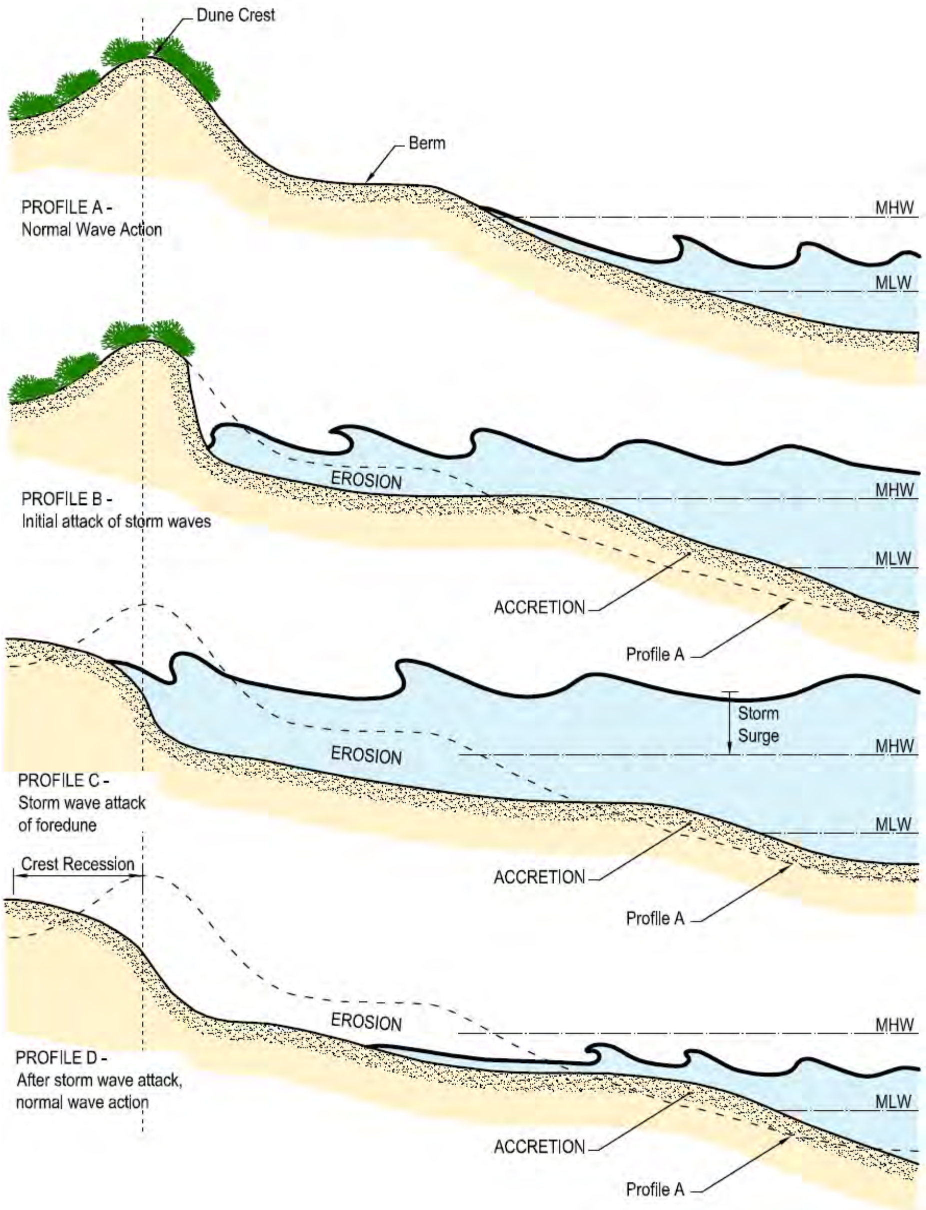


Figure 5.1 Storm Erosion Process (source: CERC 1984)

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.

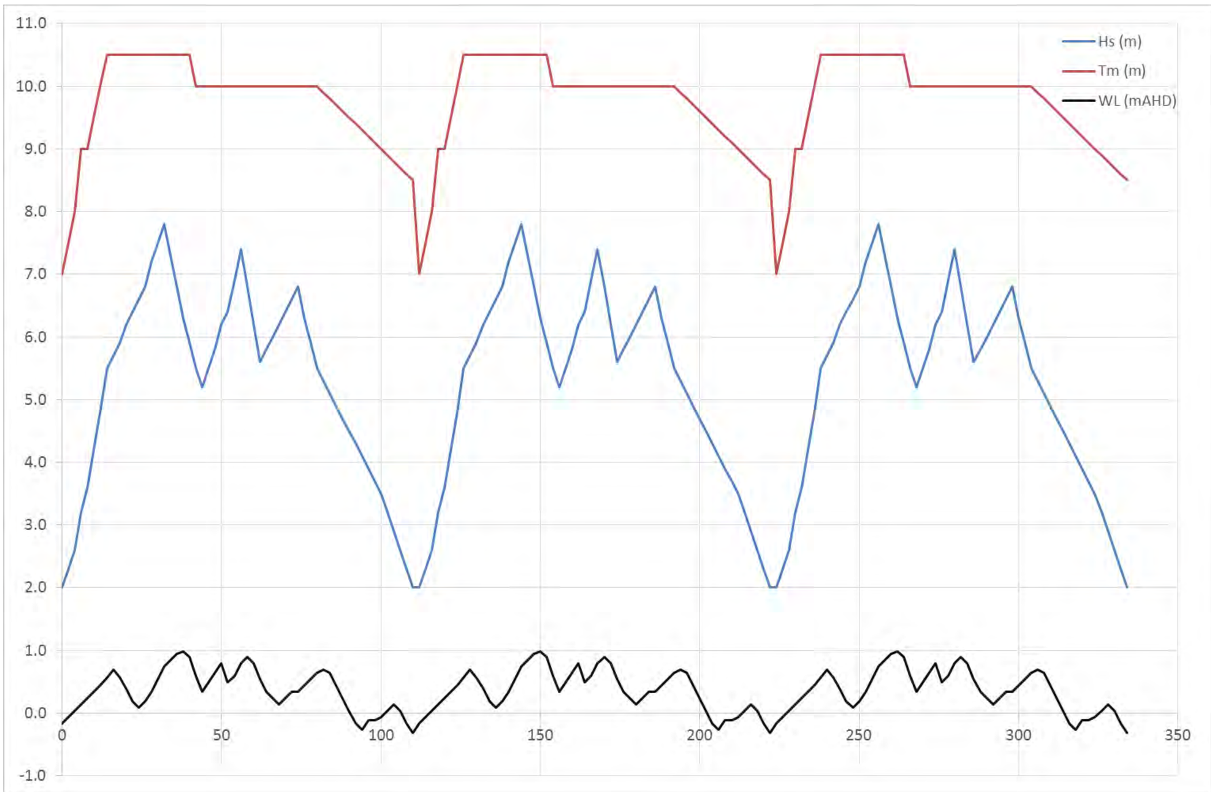


Figure 5.2 July 1996 Storm Conditions for use in Storm Erosion Modelling

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 system around Point Moore 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 will be used as inputs to the SBEACH modelling.

5.1.1 Detailed Wave Modelling

The Point Moore reef system protects the Point Moore shoreline from severe offshore wave conditions. Therefore, to determine the nearshore wave conditions to be adopted for SBEACH modelling, detailed wave modelling has been completed. For this analysis the calibrated Delft3D wave model (refer Section 4.1.2) was used.

The offshore wave conditions during the July 1996 storm were recorded by the DPI's 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 each of the different shoreline sectors 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.

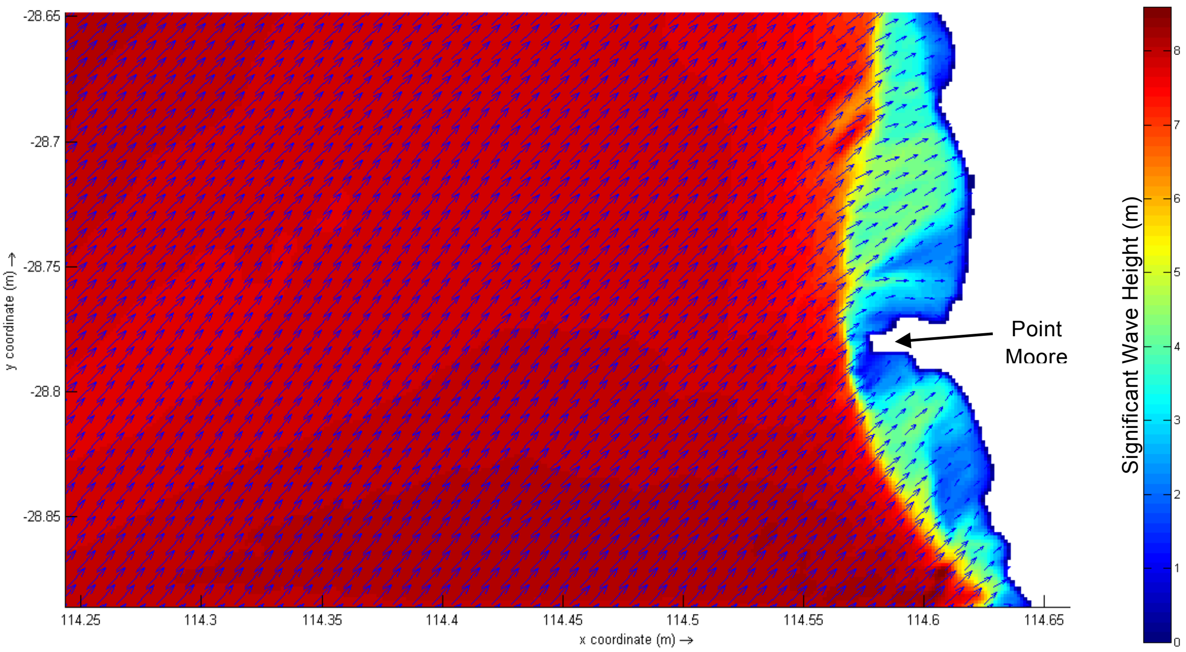


Figure 5.3 Modelled South West Wave Conditions

5.1.2 SBEACH Modelling

To simulate the shoreline response to the storm described above, three profiles were developed. These profiles were used to investigate the response of different sectors of beach to the design storm. The beach was divided into these sectors based on the exposure, aspect and characteristics of the beaches along the study area (as shown in Figure 2.3). The profiles were compiled from hydrographical survey plans and available topographic LIDAR survey of the area.

The water levels for the July 1996 storm were recorded by DPI 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.

The results of the SBEACH modelling for the July 1996 storm are presented in Figure 5.4 to Figure 5.6. These figures shows the initial (pre-storm) profile, final profile and the maximum wave heights and water levels predicted during the storm.

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 HSD is defined in present day condition to provide a baseline to which the erosion extent over the planning horizon can be determined.

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.

Table 5.1 summarises the extent of erosion, landward of the HSD that would be expected at each location during the prescribed severe storm sequence. These values are taken from results of the modelling, as presented in Figures 5.3 to Figure 5.5. These values should be used as the S1 allowance in assessing the appropriate coastal processes allowances for each of the locations. It should be noted that whilst it may appear that the amount of erosion modelled for the Sector 1 profile is abnormally low, this results from the fact that a very wide, flat beach exists along this shoreline. A large amount of energy in the modelled storm is therefore spent eroding the beach before there can be any effect on the dune system. The subsequent erosion of the dune system, behind the HSD, is therefore quite small.

Table 5.1 Acute Storm Erosion Allowance (S1)

Sector	S1 Allowance (m)
1	23
2	5
3	26

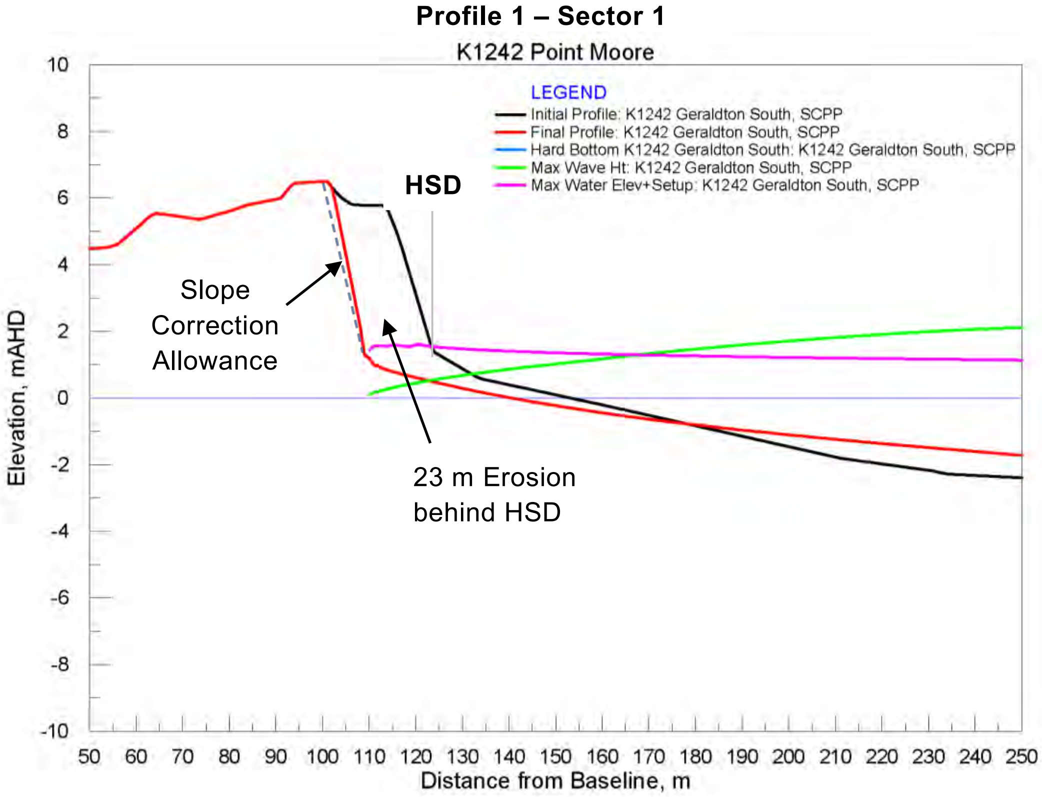


Figure 5.4 SBEACH Simulation Results Profile 1

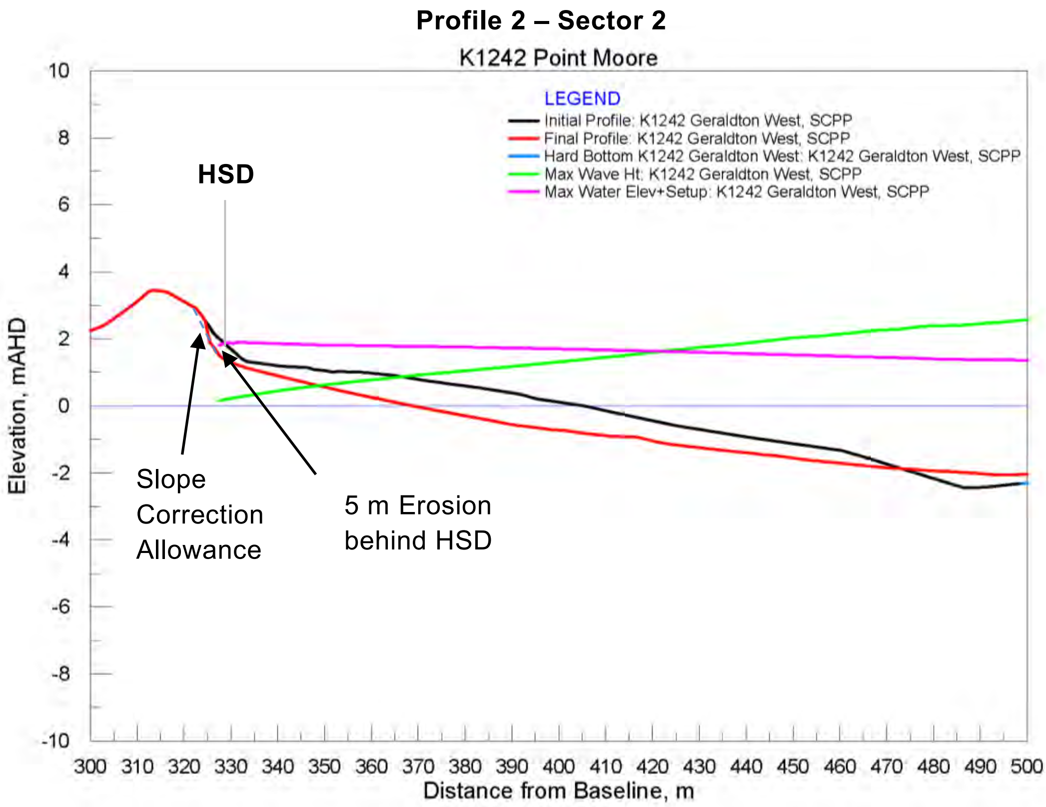


Figure 5.5 SBEACH Simulation Results Profile 2

Profile 3 – Sector 3

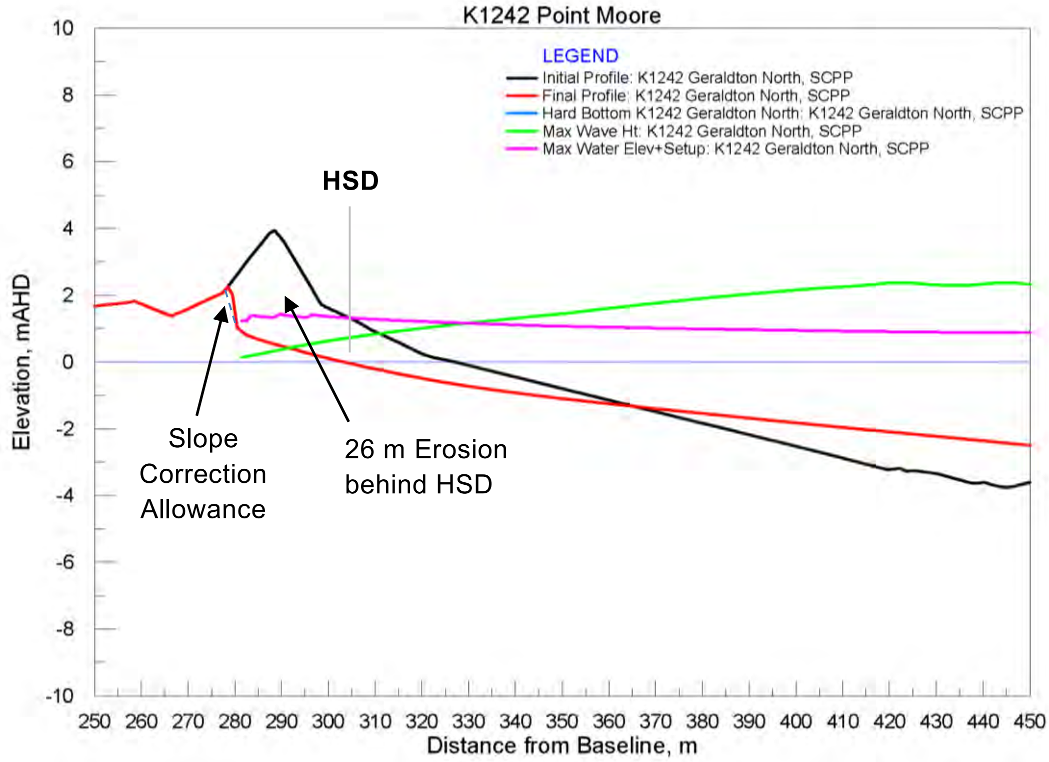


Figure 5.6 SBEACH Simulation Results Profile 3

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. Aerial imagery was obtained dating back to 1942, with the most recent aerial image from 2014.

The shoreline movement trend of the Point Moore shoreline has been analysed in three sectors, as defined in Figure 2.3. 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. In general the position of the vegetation line was used to determine the shoreline movement.

From the shoreline movement plans, the relative movements of the coastal vegetation line were estimated at 100 m intervals along each sector of the coast. These chainages are presented in Figure 5.7. The movement of the shoreline for each sector relative to its 1942 alignment is presented in Figure 5.8 to 5.10.

It is noted that a rough dumped rock wall has been constructed on the southern shoreline of Sector 1 between chainage 100 and 200 m. Whilst these rocks may have some impact on the shoreline behaviour in the short term, the short length of the structure, as well as the lack of landward returns and low crest elevation mean that, in its current form, this structure is unlikely to have any significant impact on the behaviour of the shoreline over the longer term. As a result, this structure has been excluded from further consideration within this assessment. This is consistent with SPP2.6, which suggests that if a structure is in poor condition and requires significant upgrade work within the planning horizon, then it should be excluded from further consideration within the assessment.

Figure 5.7 Oblique Aerial Photographs of Point Moore

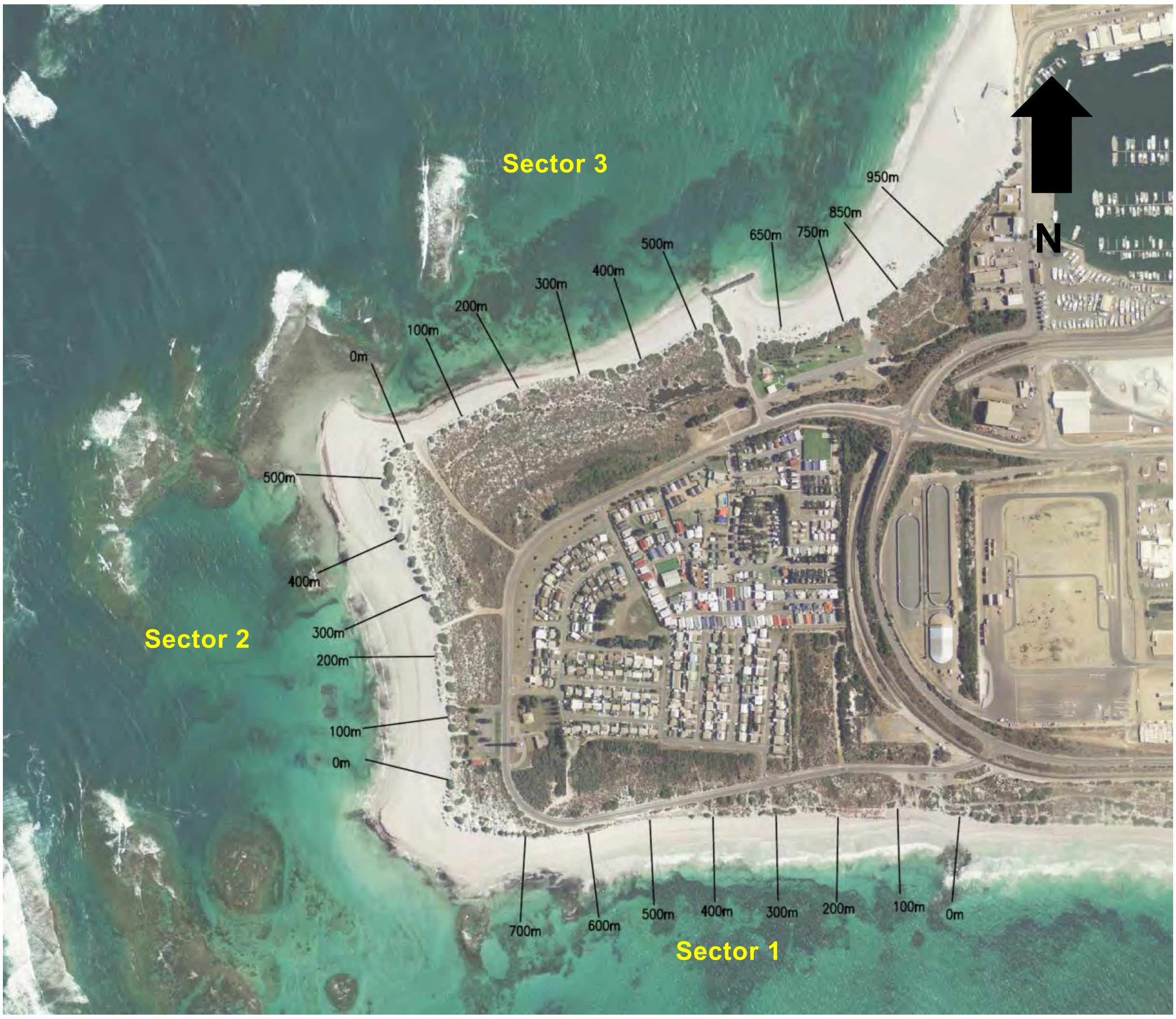
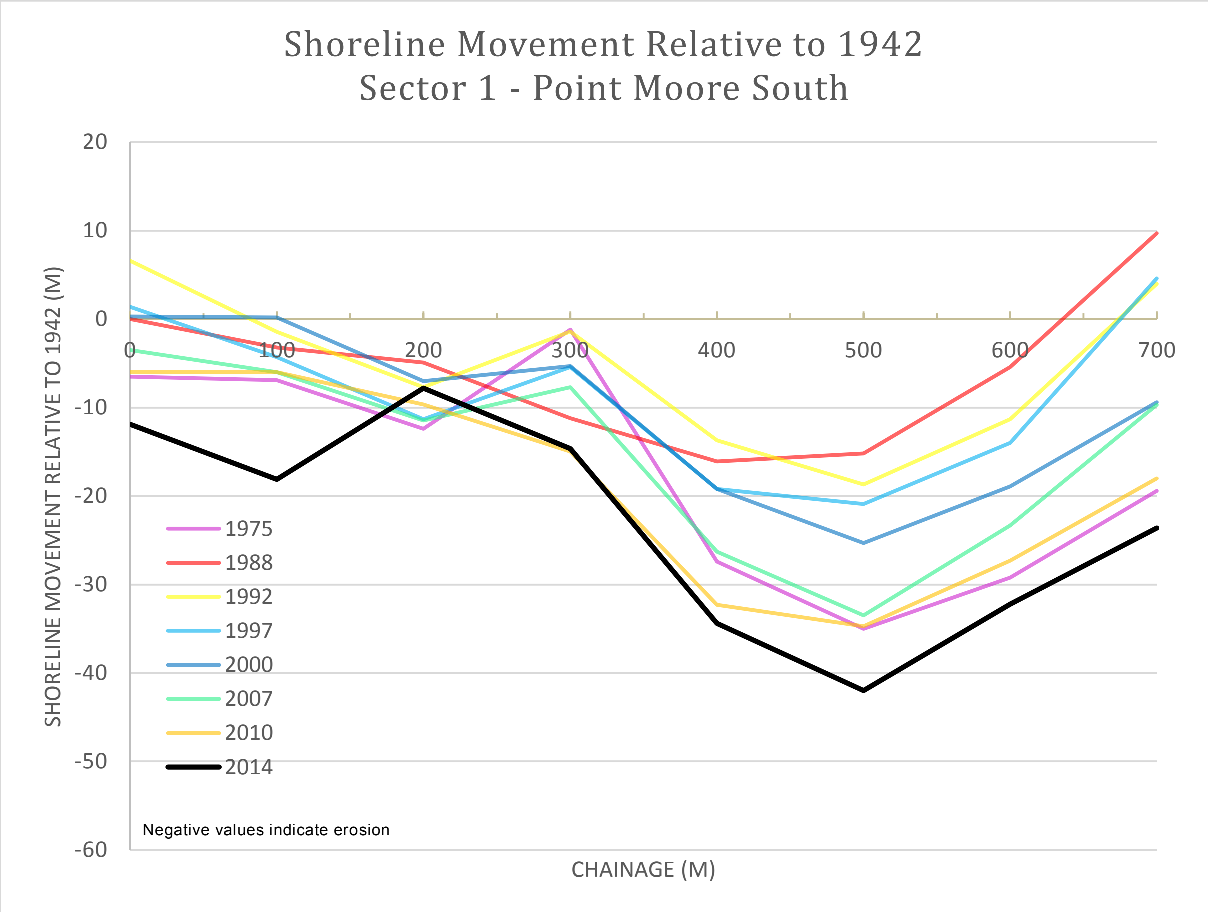
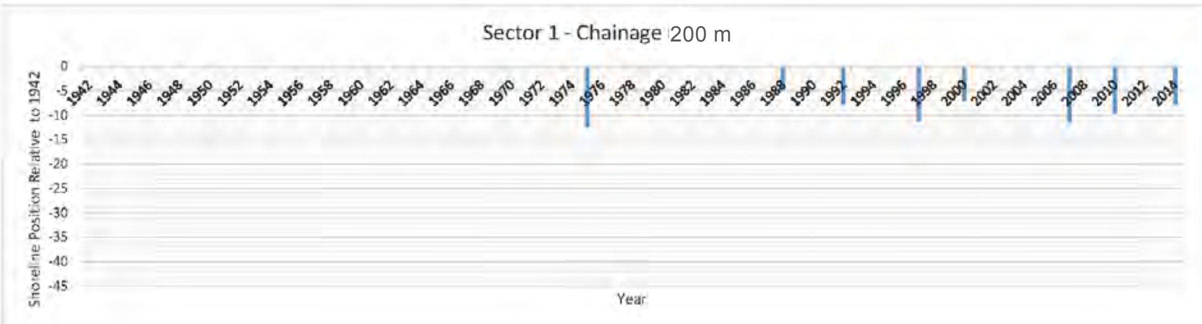


Figure 5.8 Chainages for Study Area





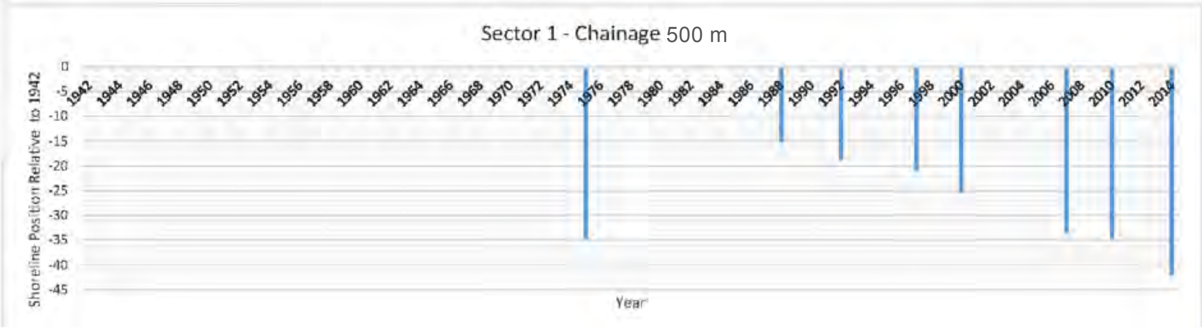
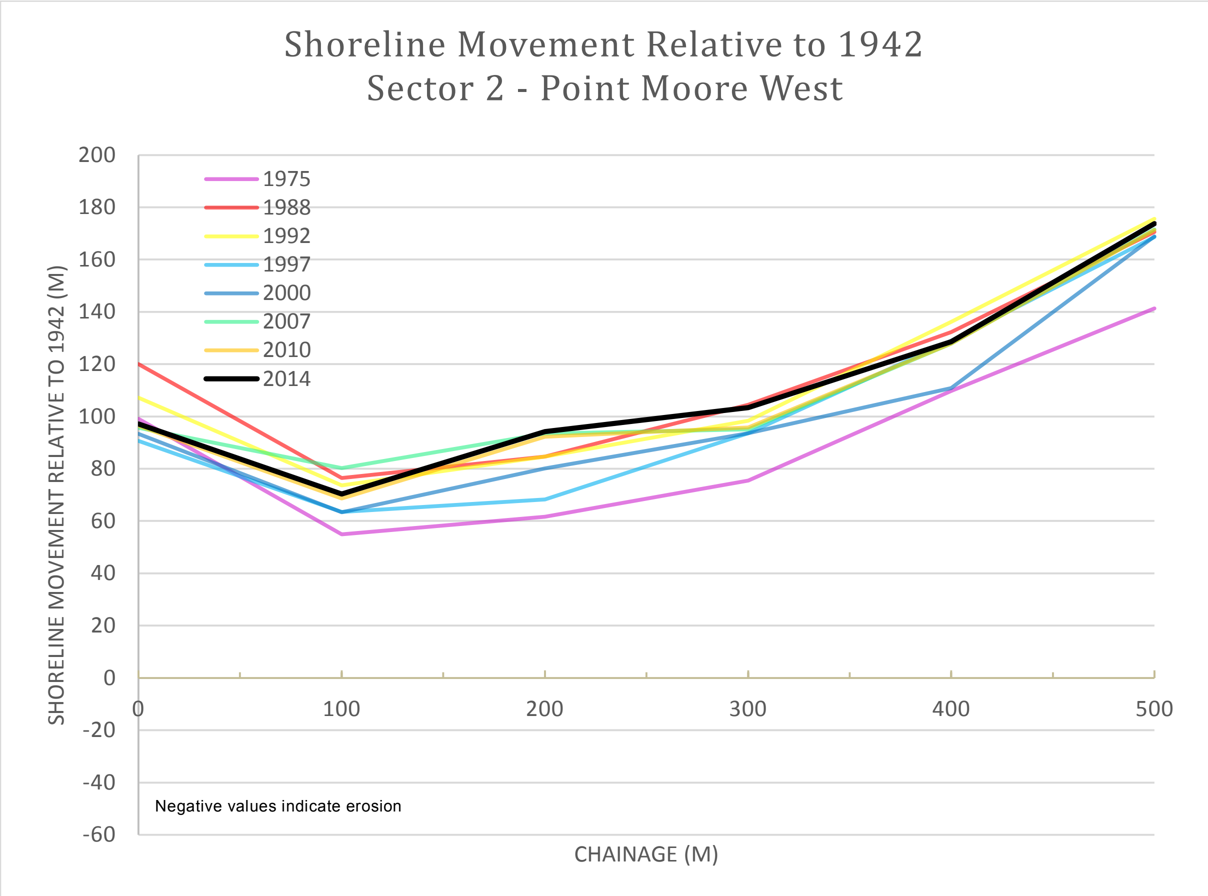


Figure 5.9 Shoreline Movement Plots for Sector 1 – Point Moore South



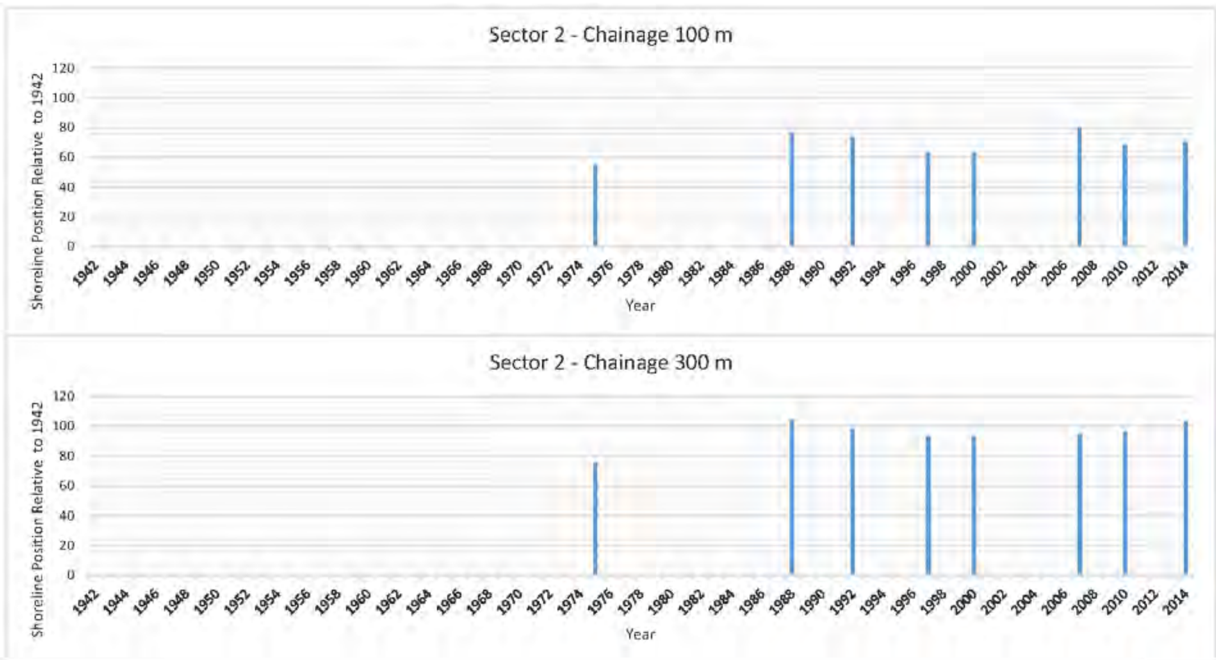
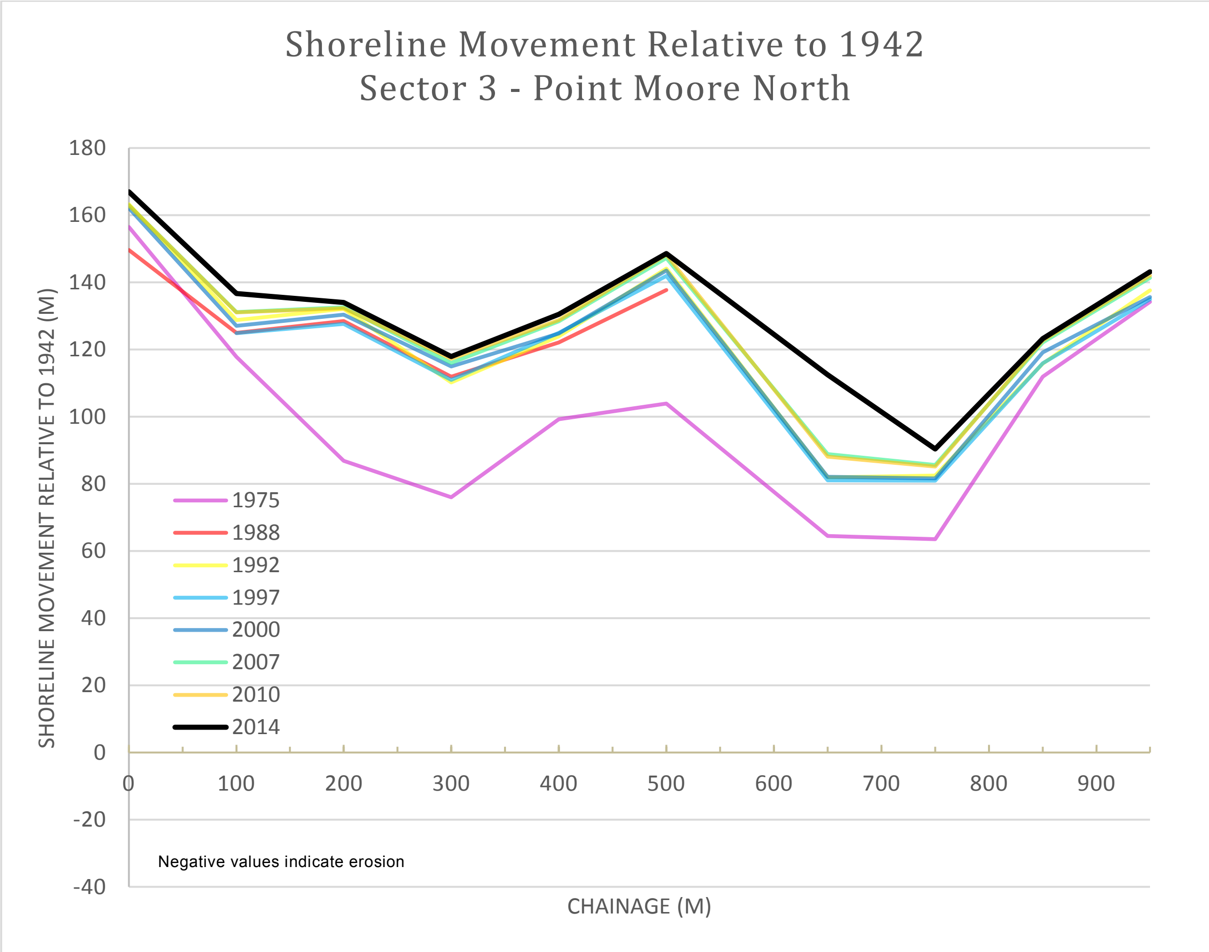


Figure 5.10 Shoreline Movement Plots for Sector 2 – Point Moore West



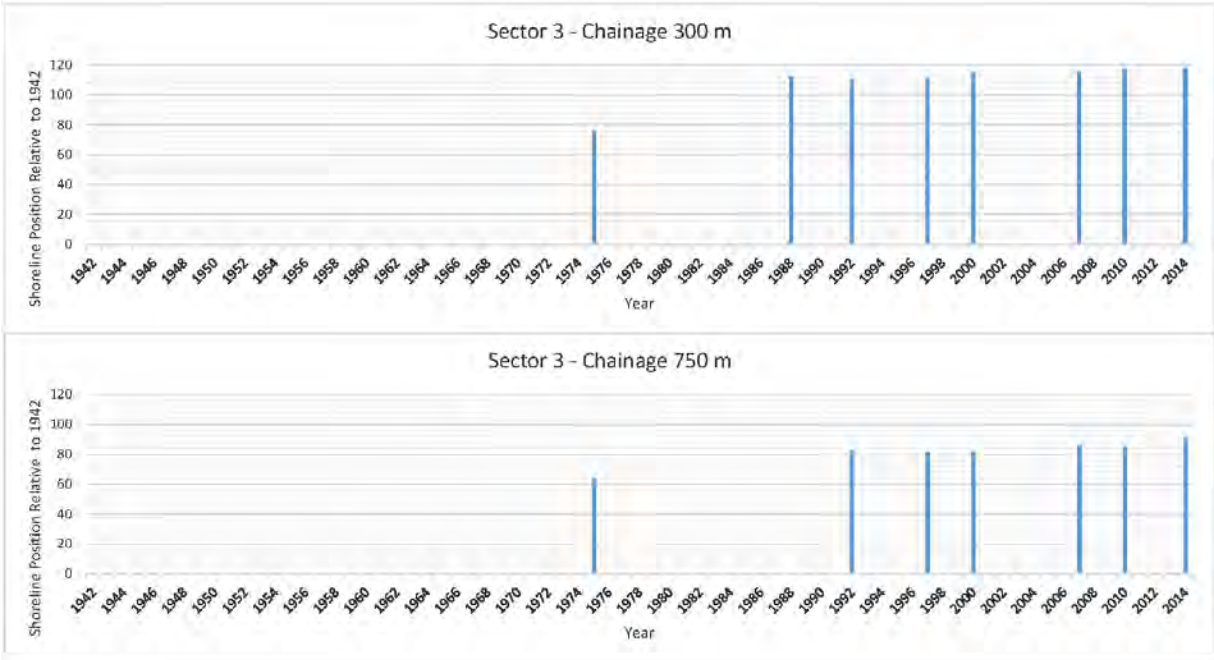


Figure 5.11 Shoreline Movement Plots for Sector 3 – Point Moore North

Figure 5.8 shows that the sandy shoreline that lies to the south of Point Moore (Sector 1) has generally experienced net erosion since 1942. This sector of the shoreline experienced erosion prior to 1975, with slight accretion between 1975 and 1988. After 1988, the shoreline has generally experienced erosion.

The movement of the shoreline in Sectors 2 and 3, as outlined in Figures 5.9 and 5.10 respectively, should be considered with regard to the local shoreline features and controls. Short (2006) outlines that the offshore Point Moore reef system provides protection to wave action and the inshore rock platforms (that correspond with northern and southern extent of Sector 2 in the current study) provide topographic control to the shoreline in this region. In addition, the western breakwater of the Geraldton Harbour enables the accumulation of sediments on Pages Beach and thus also act as a topographic control point. This is supported by the shoreline movement plans which indicate that significant accretion of the shoreline has occurred in Sector 2 and Sector 3 over the 73 year observation period, though most of the accretion occurred prior to 1988, and the shoreline has remain relatively stable with gradual accretion after this time.

The findings of Techiato et.al (2012) in their study of the Geraldton Embayments Coastal Sediment Budget (refer Figure 5.12) provides some insight into the shoreline behaviour of Sector 2 and Sector 3. This study suggests that a net sediment transport from south to north occurs along the coastline of the study area.

The trapping of this northerly sediment transport on the southern side of the reef platforms and at the western breakwater of the Geraldton Harbour is expected to have contributed to the accretion that has been observed along the shoreline of Sector 2 and 3. The presence of these shoreline controls suggests that change to the movement trend of the shoreline in the future should be minimal along these stretches of coast, particularly since no known changes are proposed to the shoreline within the southern sediment cell (Cell 14, Figure 2.2) that would affect the sediment dynamics in this area.

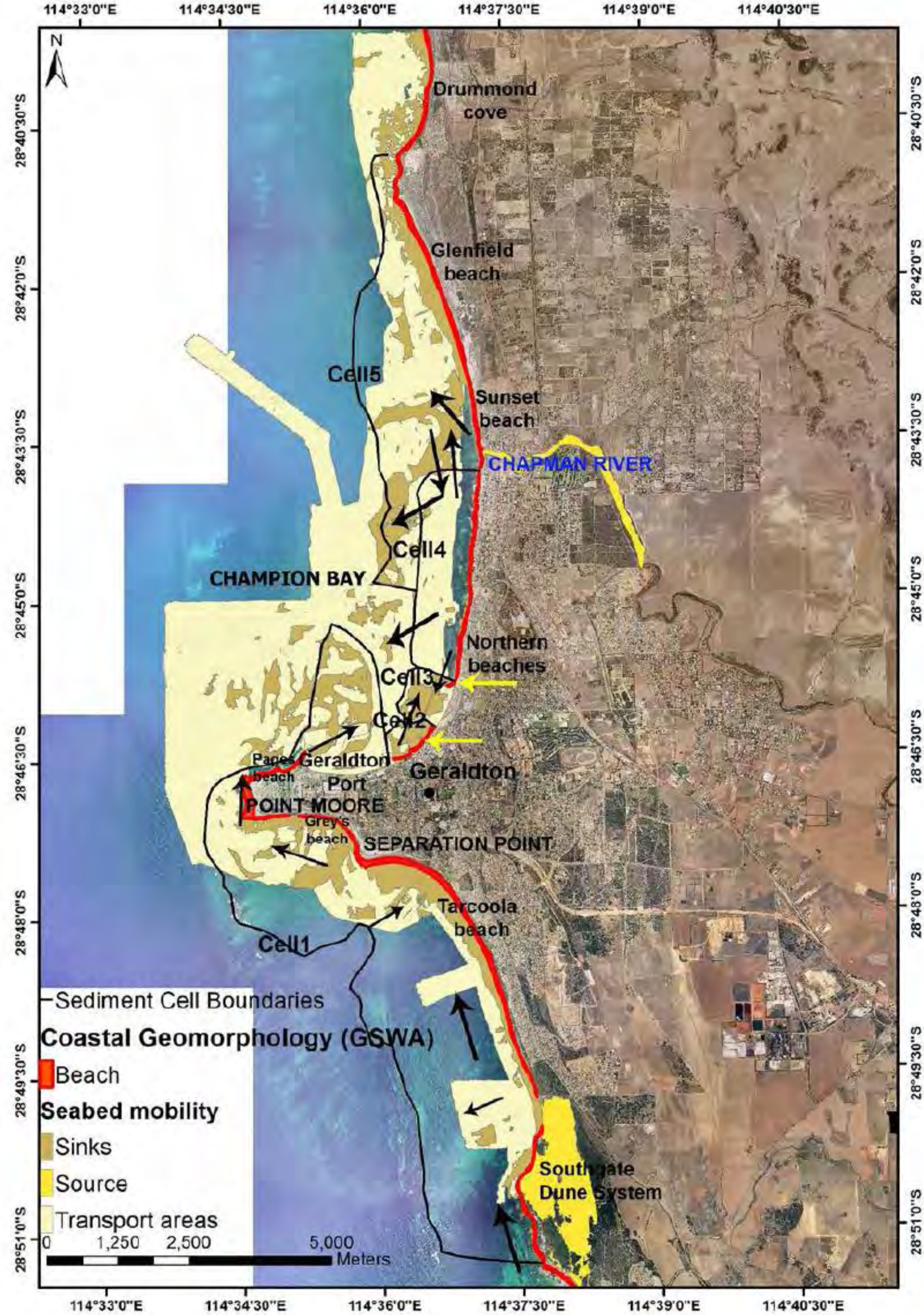


Figure 5.12 Sediment Pathways in the Geraldton Coastal System (Curtin University, 2012)

The shoreline movement in Sectors 2 and 3 should also consider the effect of sand extraction from Pages Beach by the Mid-West Ports Authority (MWPA), which reduces the available volume of sediment on the beach, but is essential to maintain the status quo of sediment movement within sediment cell 15 (Figure 2.2). To determine the effect of the sand extraction on the shoreline stability, the total sediment feed onto Pages Beaches was estimated and assessed against the total volume of sand extraction. Shoreline monitoring surveys (from 2004 to 2014) of Pages Beach and sand extraction data (from 2000 to 2014) are available and were provided to MRA by the MWPA. To provide a consistent assessment, the total volume of sediment accumulation shown on the surveys and the total sand extraction volumes have been estimated for the period from 2004 to 2014. Table 5.2 summarises the sand extraction data provided by MWPA.

Table 5.2 Volumes of Sand Extracted from Pages Beach

Year	Volume Removed (m^3)
2004	17,382
2005	N/A
2006	14,328
2007	13,070
2008	16,882
2009	8,351
2010	20,025
2011	N/A
2012	18,230
2013	6,238
2014	8,334
Total	**122,840**

Note: 1. Volumes estimated based on data provided by MWPA.

2. Sand extraction was not carried out in 2005.

3. Sand extraction volumes were not specified in 2011 and the last quarter of 2013.

From Table 5.2, the total volume of sand removed from Pages Beaches between 2004 and 2014 is approximately 122,840 m^3, which is estimated to equate to an average rate of around 11,000 m^3 per year.

To determine the total volume of sediment accumulation shown on the survey, the shoreline monitoring surveys were analysed. The shoreline monitoring surveys consist of cross section surveys of the beach profile at six locations (separated at approximately equal distance) over the extent of Pages Beach. To estimate the total change in sediment volume from 2004 to 2014, the change in cross sectional area of the beach profile from 2004 to 2014 was estimated at each location, with the total change in volume estimated by integration of the cross sectional areas over the extent of the shoreline. The total change in volume from 2004 to 2014 is estimated to be an increase of approximately 85,000 m^3 of beach volume.

Based on the result of the assessment, the total change in sediment volume of the shoreline (from 2004 to 2014) was added to the total volume of sand removed between 2004 to 2014, which gives a total sediment feed into Pages Beach of approximately 207,840 m^3 between 2004 to 2014. This is equivalent to an accretion rate of around 19,000 m^3 per year.

The results of the sediment assessment indicate that at the current rate of sand extraction (11,000 m^3/year), the accretion at Pages Beach is likely to continue (net accretion of around 8,000 m^3/year). It should be noted that this assessment has assumed that the current sand extraction rate is maintained in the future, however, should the MWPA increase the sand extraction rate in order to address long term erosion on the shoreline north of the Batavia Coast Marina, the long term rate of shoreline movement may reduce and would need to be reassessed.

To determine the S2 allowance for each sector, the long term shoreline movement rates for each sector have assessed. The proposed allowances for S2 for each shoreline Sector are presented in Figure 5.12 to 5.14 together with the shoreline movement rates observed between 1942 and a number of years (1975, 2000 and 2014).

For Sector 1, the shoreline movement rate (refer Figure 5.13) shows that this sector of shoreline has experienced continuous erosion. The average annual rate of erosion from 1942 to 2014 is estimated to be approximately 0.6 m/year at the worst location. SPP2.6 requires that the S2 allowance for shorelines experiencing continuous erosion be calculated based on the historical annual rate of erosion. Therefore a S2 allowance of 0.6 m/year has been adopted for Sector 1.

Figure 4.9 shows that the Sector 2 shoreline experienced continuous accretion over the last 73 years, though from the trend of the long term movement rate (refer Figure 5.14) the accretion rate is reducing gradually over time. This means that in some locations of Sector 2 (e.g. at chainage 100) the accretion rate is likely to reduce to around 0.6 m/year in the next 50 years. Therefore, it is necessary for the S2 allowance of Sector 3 to provide consideration for the reducing accretion rate over time, despite the likely continuous accretion in the future. SPP2.6 states that where the historical annual rate of shoreline movement is continuously in excess of 0.2 m/year, and there is compelling evidence that accretion is likely to continue at the same rate for at least the next 50 years, the S2 allowance can be taken as half the historical long-term annual rate of accretion. Therefore, it is reasonable to take a S2 allowance of -0.3 m/year (half of the expected long term annual rate of accretion) for the Sector 2 shoreline.

The shoreline at Sector 3 also showed continued accretion in the last 73 years, even with the extraction of sand by the MWPA for bypassing. The trend of the shoreline movement rate (refer Figure 5.15) indicates that the accretion rate has been gradually decreasing, most likely as a result of the sand bypassing. However, on the basis of the review of the sand bypassing rates and resultant shoreline behaviour, it is expected that the shoreline will continue to accrete, albeit at a reduced rate compared to previously. This means that in some locations of Sector 3 (e.g. at chainage 750) the longer term accretion rate is likely to reduce to around 0.7 m/year in the next 50 years. Therefore, a S2 allowances of -0.3 m/year (about half of the long term annual rate of accretion) has been adopted for the Sector 3 shoreline.

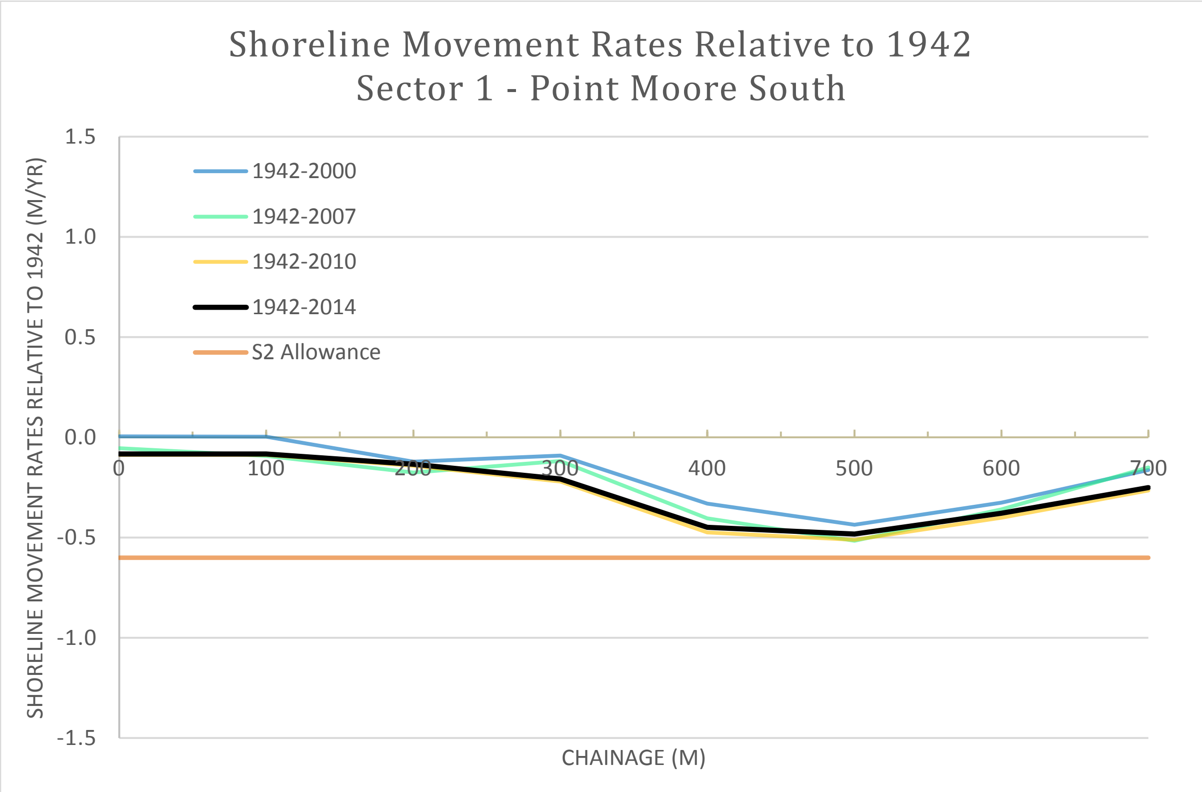


Figure 5.13 Long Term Shoreline Movement Rates & the S2 allowance – Sector 1

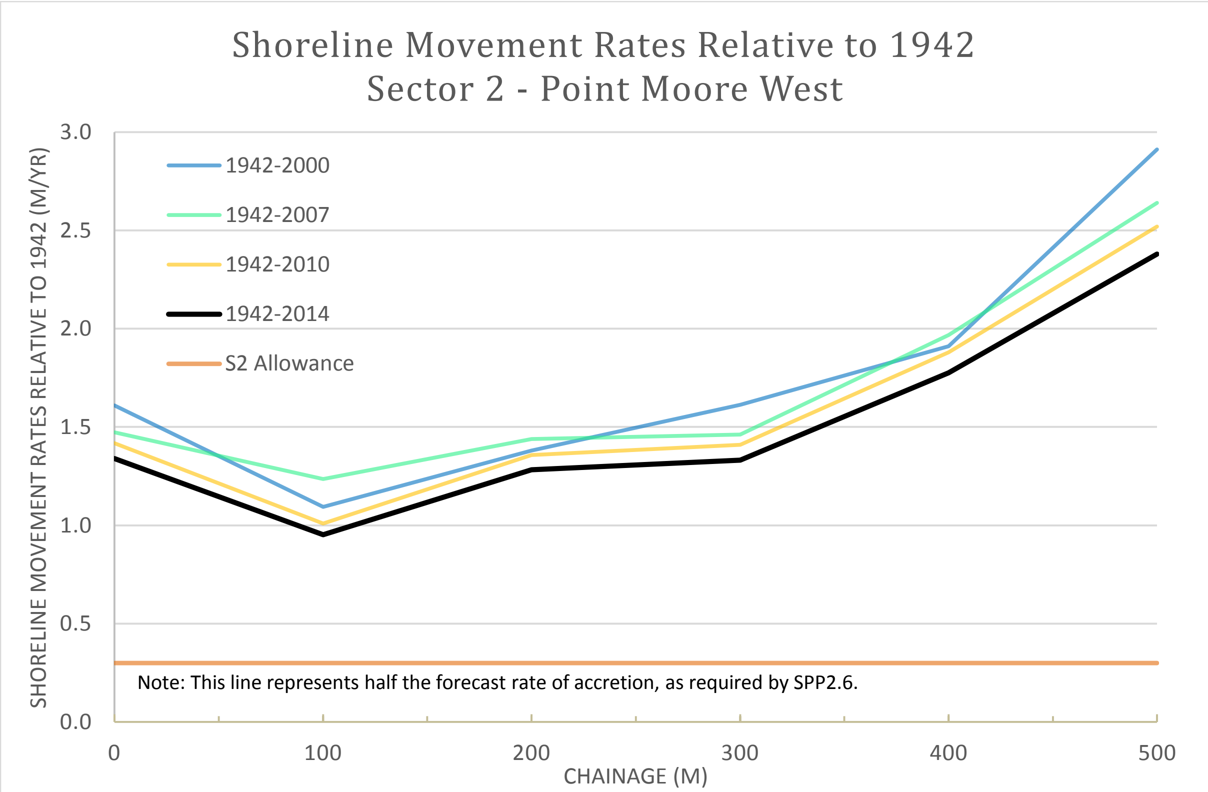


Figure 5.14 Long Term Shoreline Movement Rates & the S2 Allowance – Sector 2

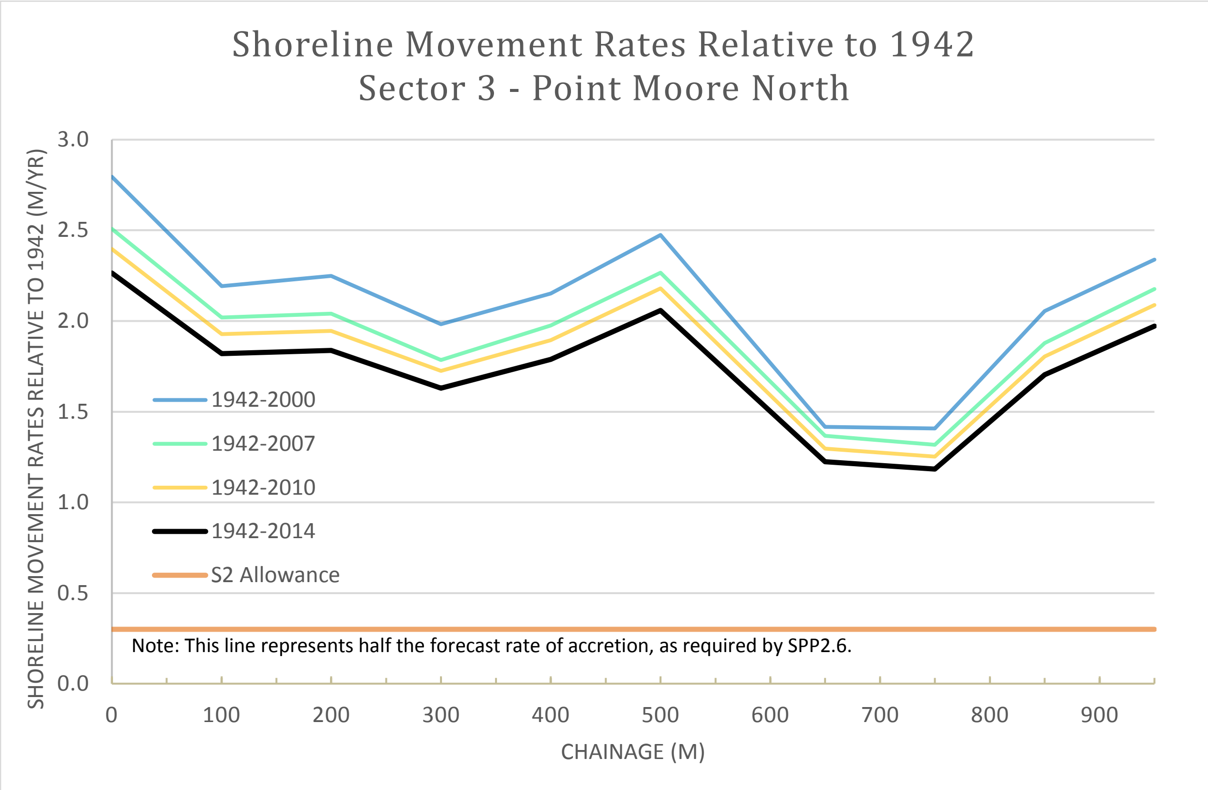


Figure 5.15 Long Term Shoreline Movement Rates & the S2 Allowance – Sector 3

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. SPP2.6 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 5.3.

Table 5.3 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 development be completed based on a 100 year planning horizon. Therefore an allowance for sea level rise of 0.90 m has been adopted for 2110. Given the 100S value, the potential recession of the Point Moore shoreline that could occur as a result of the increases in sea level is 90 m in 2110. This is likely to be conservative for Sector 2 and Sector 3 which is stabilised by the inshore reef.

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

Sector	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
1	23	0	0	0	23
2	5	0	0	0	5
3	26	0	0	0	26

Table 6.2 Total Recommended Coastal Processes Allowance – 2030

Sector	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
1	23	9	7	3	42
2	5	-4	7	3	11
3	26	-4	7	3	32

Table 6.3 Total Recommended Coastal Processes Allowance – 2070

Sector	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
1	23	33	39	11	106
2	5	-16	39	11	39
3	26	-16	39	11	60

Table 6.4 Total Recommended Coastal Processes Allowance – 2110

Sector	S1 (m)	S2 (m)	S3 (m)	Allowance for Uncertainty (m)	Total Allowance (m)
1	23	60	90	20	193
2	5	-30	90	20	85
3	26	-30	90	20	106

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 C. Further details regarding the combined potential impact of the physical processes allowances and the potential coastal inundation are provided in Section 7.

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 development is not located outside of these potential areas of impact, management would likely be required.

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 D.

8. Conclusions

Assessments of the appropriate inundation and coastal processes allowances for Point Moore 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 for Point Moore have been estimated and presented in Table 3.8 and 3.9. 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 the shoreline was divided into three sectors to investigate the response of each sector to the design storm.

- Storm erosion modelling using the SBEACH profile change model resulted in a predicted erosion of between 5 and 26 m for the design storm. The values determined by SBEACH were used as the allowances for S1, as stipulated by the SPP2.6.

- Shoreline movement analysis was completed, including review of sand extraction data provide by MWPA. As a result of this investigation allowances for future shoreline movement have been provided.

- Allowances for shoreline recession as a result of sea level rise of 7, 39 and 90 m have been provided to account for the potential shoreline recession as a result 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 5.1 to 5.4. These coastal processes allowance lines highlight areas that could be vulnerable to the action of coastal processes over the respective timeframes.

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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 **Dune Inundation Assessment Plan**

Appendix B **Inundation Mapping**

Appendix C **Coastal Processes Allowance Mapping**

Appendix D **Combined Coastal Processes & Inundation Mapping**

Appendix A Dune Inundation Assessment Plan

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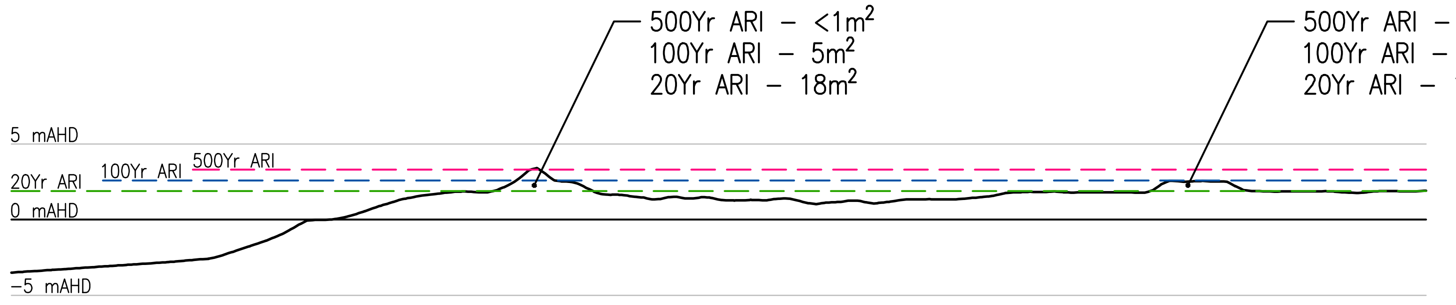

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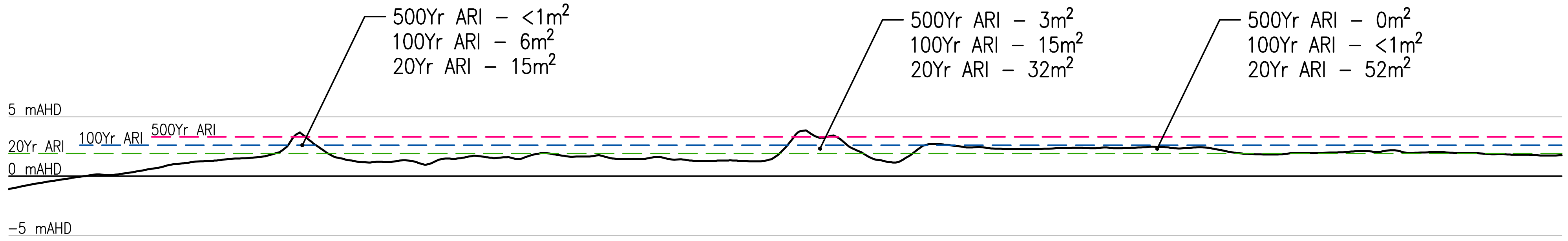
1:500 V



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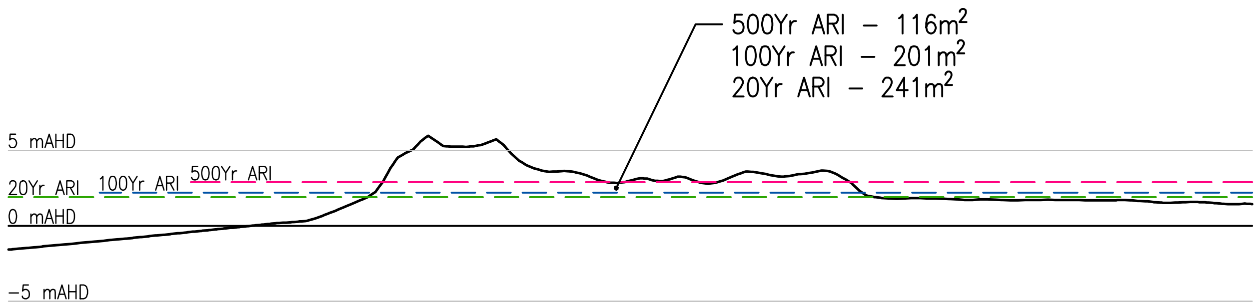
1:500 V



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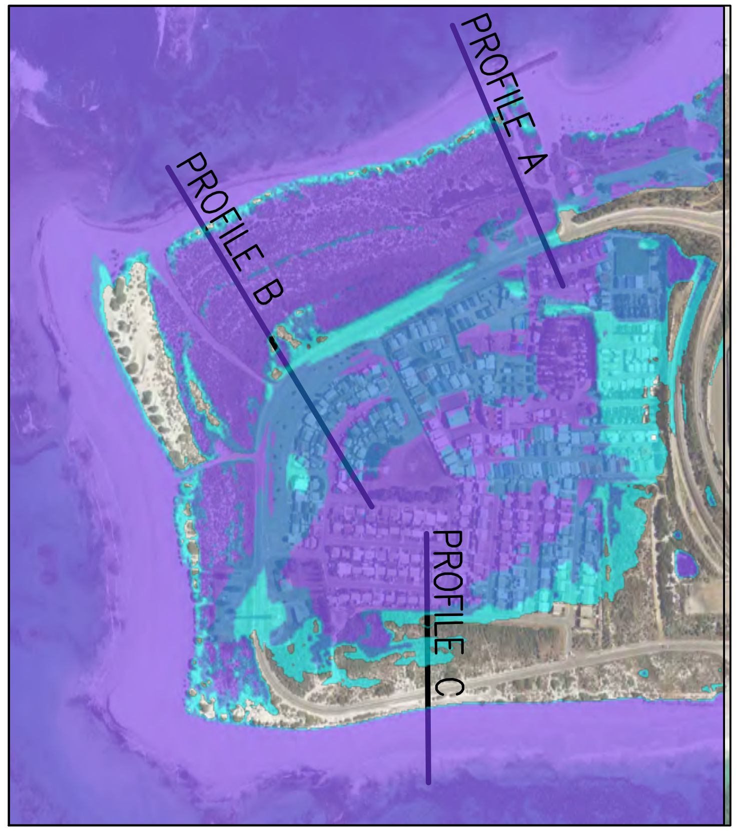

LOCATION PLAN

LEGEND

20Yr ARI EVENT

100Yr ARI EVENT

500Yr ARI EVENT

AT CORRECT SCALE THIS IS 100 mm

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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

INUNDATION – DUNE RESERVE (PRESENT DAY)

SCALE AT A3 AS SHOWN

JULY 2015

D1242-02-05

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Appendix B Inundation Mapping

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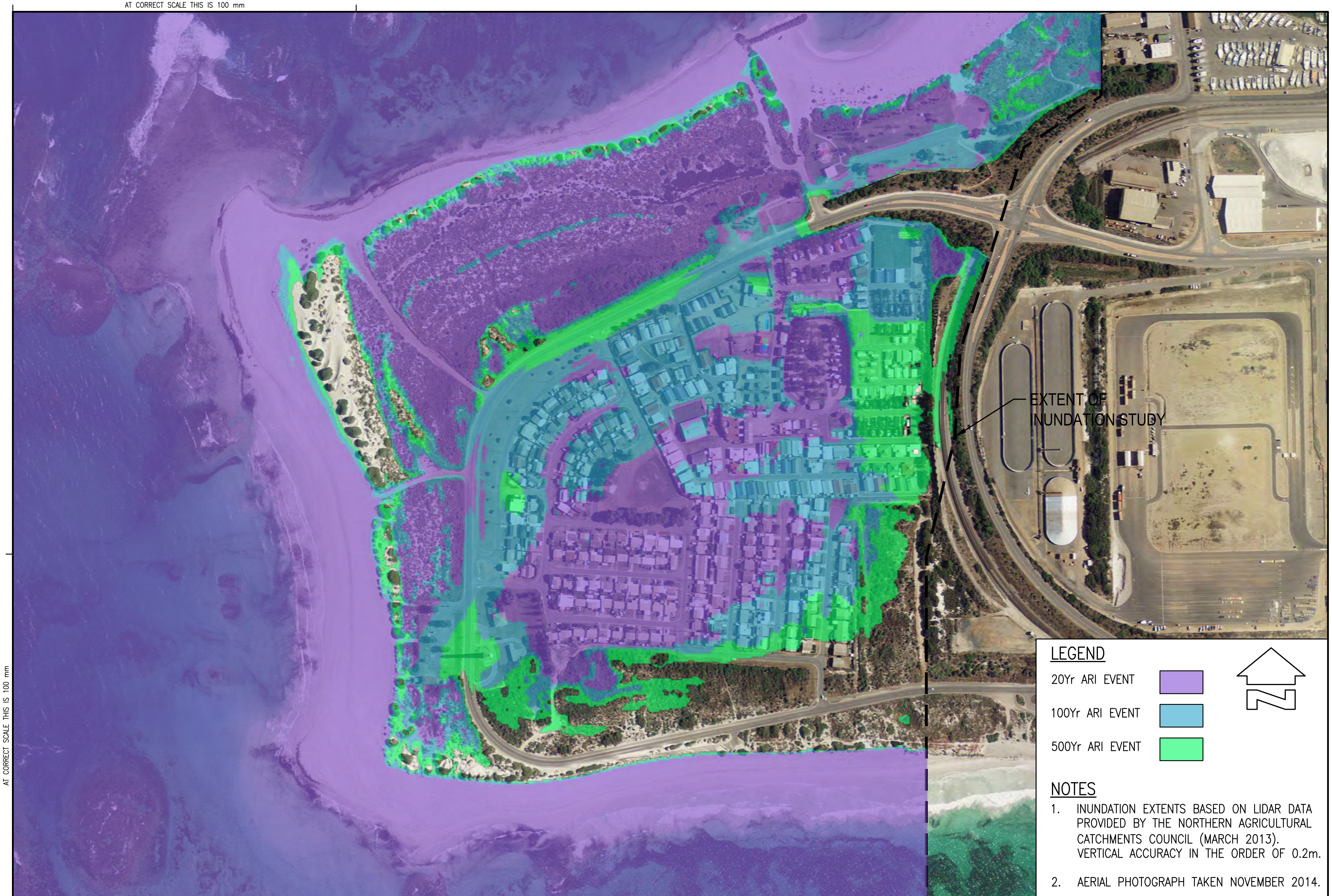

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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

INUNDATION MAPPING – PRESENT DAY

JULY 2015

SCALE AT A3 1:4,000

D1242-02-01(B)

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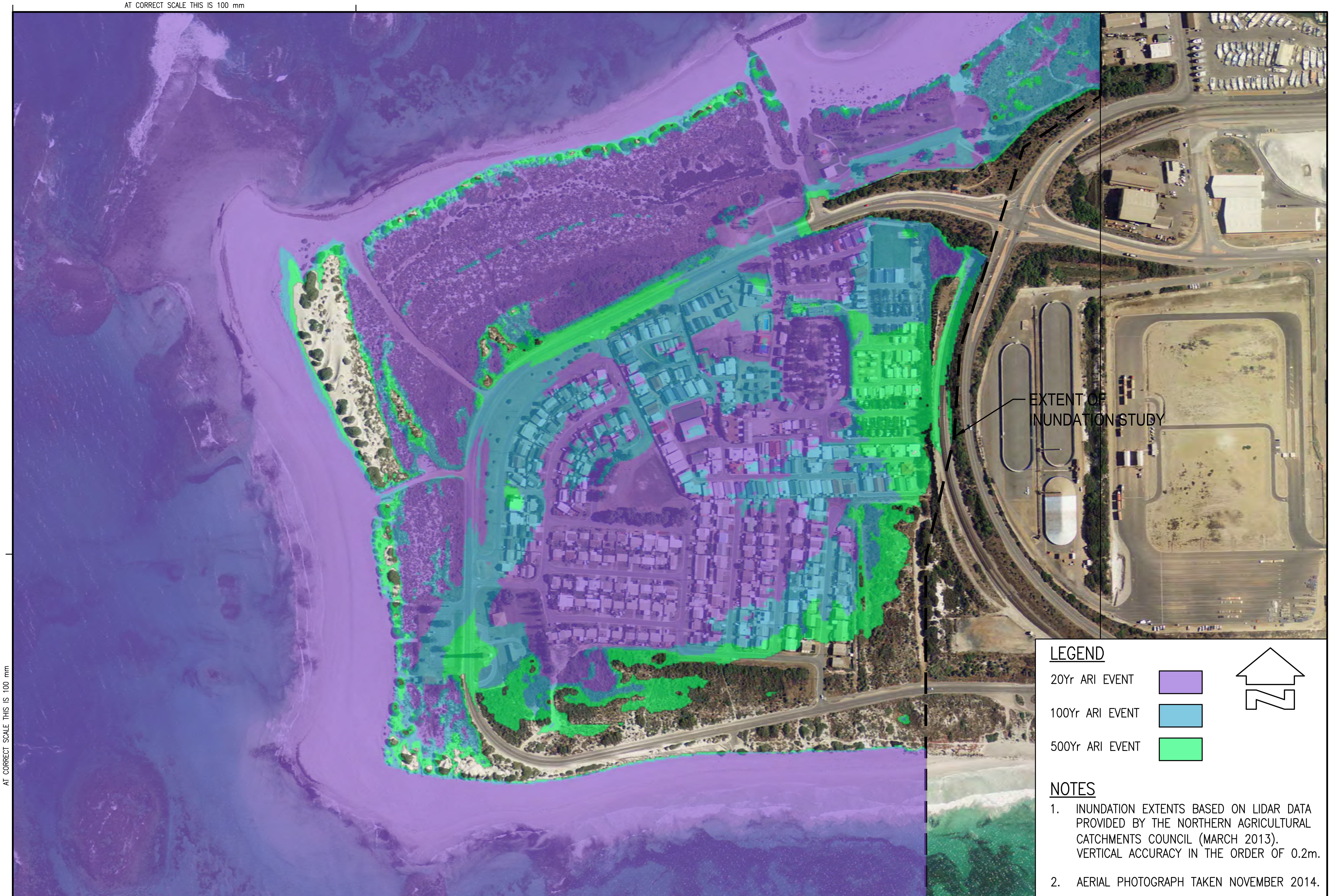

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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

INUNDATION MAPPING – 2030 (+0.07 SLR)

SCALE AT A3 1:4,000

JULY 2015

D1242-02-02(B)

P:\MRA Paying Jobs\K1242 CGG – Point Moore Inundation Study\MRA DWGs\Inundation\2030\K1242-2030

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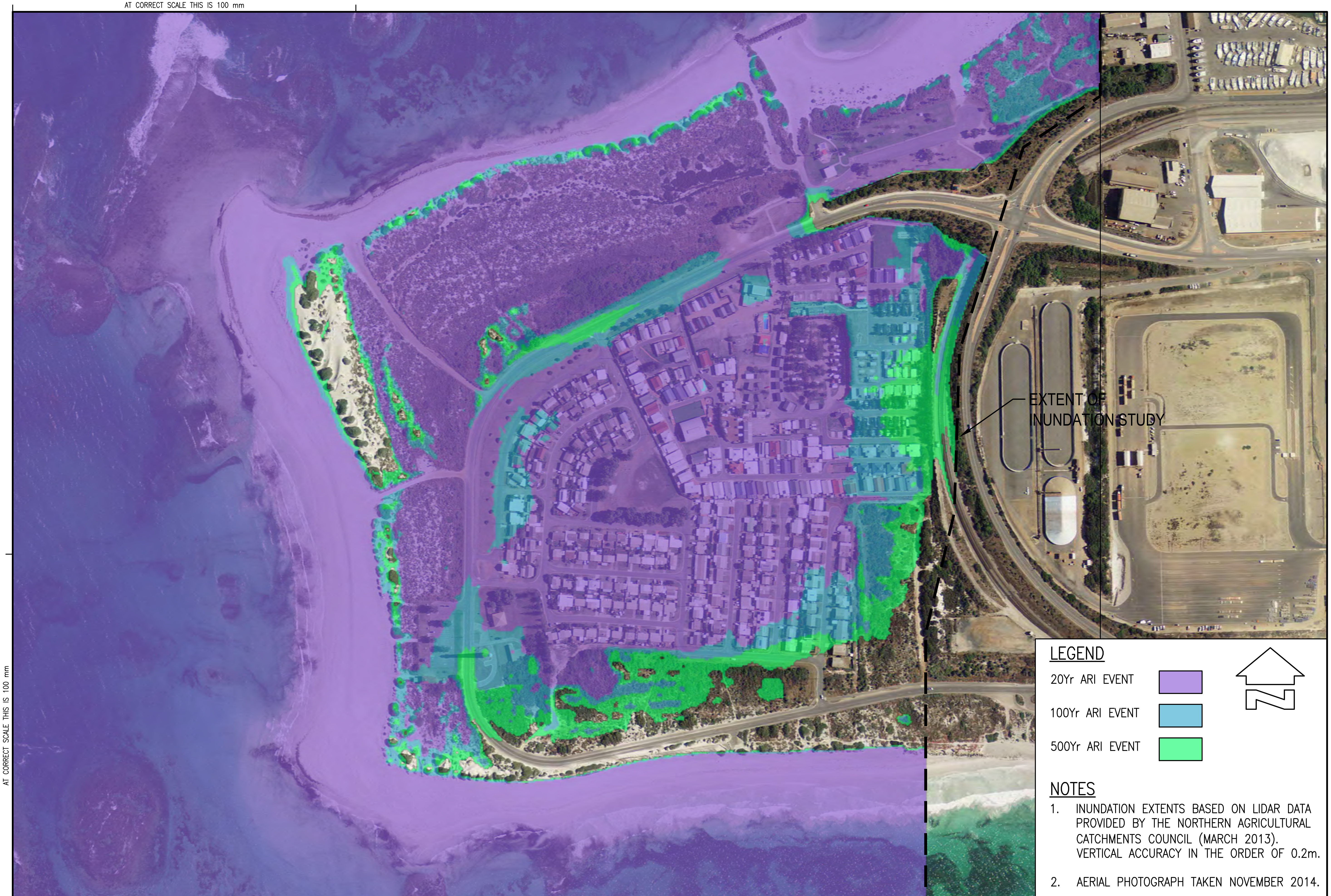

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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

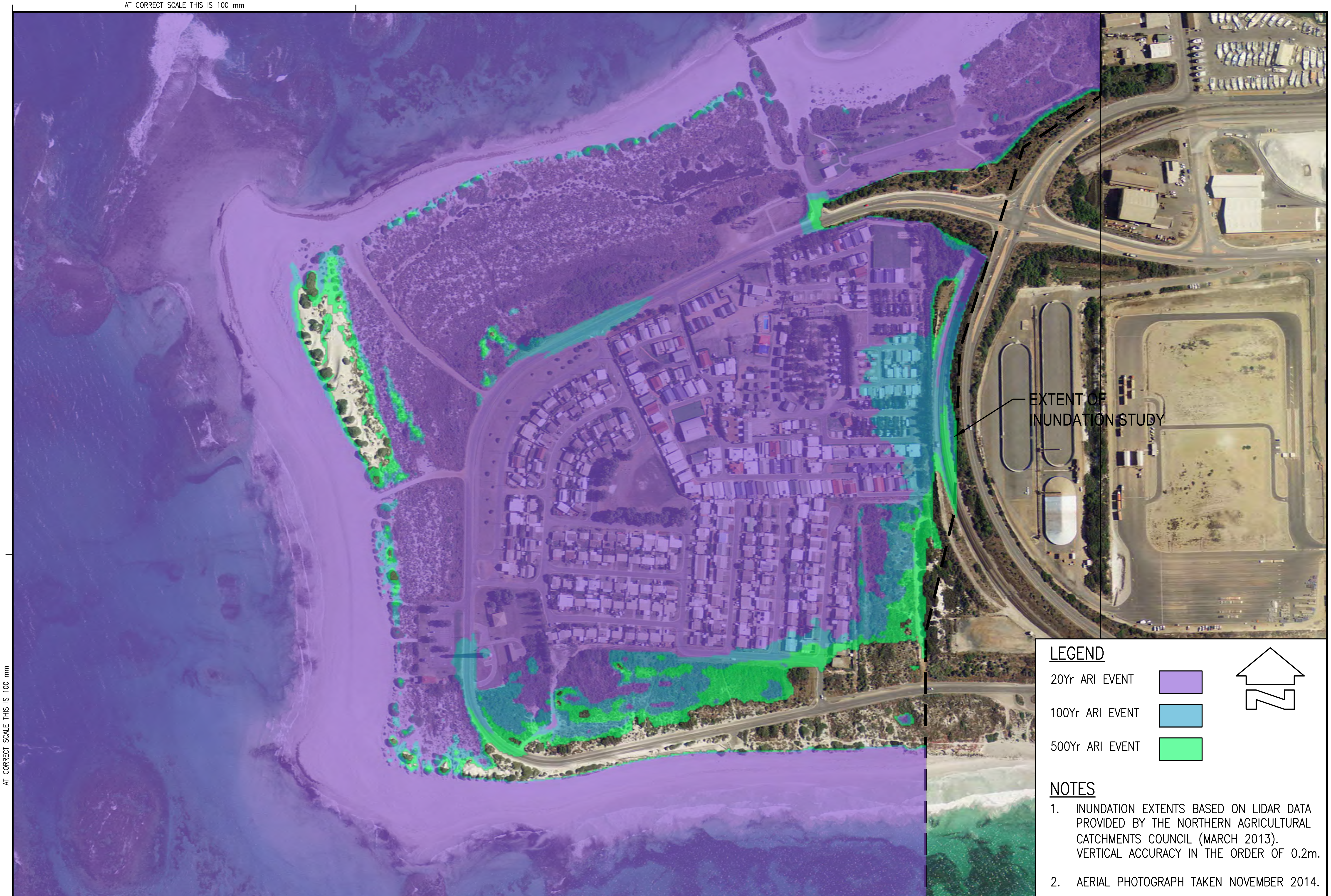
INUNDATION MAPPING – 2070 (+0.39 SLR)

SCALE AT A3 1:4,000

JULY 2015

D1242-02-03(B)

AT CORRECT SCALE THIS IS 100 mm



AT CORRECT SCALE THIS IS 100 mm

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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

INUNDATION MAPPING – 2110 (+0.9 SLR)

SCALE AT A3 1:4,000

JULY 2015

D1242-02-04(B)

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Appendix C Coastal Processes Allowance Mapping

AT CORRECT SCALE THIS IS 100 mm

AT CORRECT SCALE THIS IS 100 mm

REV	DATE	APPROVED	AMENDMENT	REV	DATE	APPROVED	AMENDMENT
A	03.07.15	CRD	PRELIMINARY ISSUE				

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CLIENT: CITY OF GREATER GERALDTON

DESIGNED:
CHECKED: C. DOAK
APPROVED:
DRAWN: T. VAN BEEM
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PROJECT: POINT MOORE COASTAL EROSION AND INUNDATION STUDY

TITLE: SETBACK PLAN (SPP 2.6 METHODOLOGY) PRESENT DAY TO 2110

SCALE AT A1: 1:2,500

DRAWING NUMBER: D1242-01-02

REV: A

Appendix D Combined Coastal Processes & Inundation Mapping

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

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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

COMBINED COASTAL VULNERABILITY MAPPING – PRESENT DAY

SCALE AT A3 1:4,000

DECEMBER 2015

D1242-06-01(B)

P:\MRA Paying Jobs\K1242 CGG – Point Moore Inundation Study\MRA DWGs\Innundation\Combined Mapping\D1242-06-01

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

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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

COMBINED COASTAL VULNERABILITY MAPPING – 2030 (+0.07 SLR)

SCALE AT A3 1:4,000

DECEMBER 2015

D1242-06-02(B)

P:\MRA Paying Jobs\K1242 CGG – Point Moore Inundation Study\MRA DWGs\Innundation\Combined Mapping\D1242-06-02

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.
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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

COMBINED COASTAL VULNERABILITY MAPPING – 2070 (+0.39 SLR)

SCALE AT A3 1:4,000

DECEMBER 2015

D1242-06-03(B)

AT CORRECT SCALE THIS IS 100 mm

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POINT MOORE COASTAL EROSION AND INUNDATION STUDY

COMBINED COASTAL VULNERABILITY MAPPING – 2110 (+0.9 SLR)

SCALE AT A3 1:4,000

DECEMBER 2015

D1242-06-04(B)

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