GEO-ENVIRONMENTAL

SOLUTIONS

COASTAL VULNERABILITY ASSESSMENT

Whitemark, Flinders Island

CLIENT

Flinders Island Council

Dock4 Architects

October 2018

Geo-Environmental Solutions 29 Kirksway Place, Battery Pont. Ph 6223 1839 Fax 6223 4539

Tables

Figures

Plates

Executive Summary

Flinders Island Council is seeking to assess community vulnerability for Whitemark with specific reference to six investigation areas which cover inland areas up to 800m from the coast and coastal areas covering approximately 2 km of coastline between Whitemark & Nalinga Creek to the south.

JMG Engineers and Planners have undertaken a hydrology review of Nalinga Creek in a report dated October 2018.

This report comprises a coastal vulnerability assessment of four (4) of the six (6) investigation Sites and provides recommendations for management of the coastline including vulnerable areas subject to changing conditions due to climate change.

Project objectives include:

- Produce a coastal vulnerability assessment report for the study area;
- Assess and define the existing & potential coastal hazards associated with climate change projections for the study area. This includes assessing coastline inundation, wave climate, coastline recession & storm erosion;
- Make general recommendations for the coastline in the study area as well as specific recommendations for development on Sites noted Site 1 to Site 5.

The coastline near the project area is exposed to diffracted and refracted swell wave activity from Bass Strait and wind wave activity within Parry's Bay. Waves directed from the north-west across the shallow water tidal flats have resulted in the formation of a beach plus tidal flat geometry, whereas waves directed from the west are more conducive to the development of a reflective type beach profile.

Fine-grained windblown sediments form the higher relief dune ridges and overly coarse-grained beach sand deposits that form the beach face and extend beneath the dune system. The windblown sands are highly vulnerable to wind and wave erosion and are particularly vulnerable to sea level rise. This erosion is most pronounced in areas where shoreline seagrass deposits are not available to provide wave runup protection. The coarse-grained beach sand deposits and the seagrass offers considerable natural armouring to the overlying fine grained sandy sediments.

Due to sea level rise and the lack of natural shoreline armouring at Nalinbga Creek outlet, the fine-grained dune sand is being eroded and mobilised into the intertidal zone where it is actively forming a delta. Aerial photographs indicate that the delta is expanding at an increasingly rapid rate. As a result, sea grass beds in the tidal zone off Nalinga Creek and other eroding parts of the coastline are being buried. The following processes are becoming increasingly pronounced:

- Greater wave energy closer to coastline;
- Denudation of seagrass beds from increased wave activity;
- Burying of the seagrass with fine sediments eroding from shoreline; and
- Amplification of the above processes (due to reduced overall wave attenuation).

Historical aerial photographs provide a reliable measure of shoreline recession in some parts of Parrys Beach. There are strong erosion correlations with sea level rise acting on the dune system immediately south of Nalinga Creek outflow where historical to present erosion rates of 377m per 1 m rise in sea level are discerned based on a 95% R^2 correlation.

A considerably lower erosion rate of 103m per metre sea level rise is calculated for the dune escarpments to the north of Nalinga Creek discerned based on a 71% R^2 correlation (with storm bite and possibly river scour causing much of the deviation).

An overall recession trend for Whitemark Beach of 130 to 150m per metre sea level rise is estimated based on historical photography and Bruun Rule models based on calculated and observed closure depths. Geo-Environmental Solutions (GES) recommend that consideration is given to a 2080 timeframe for town planning in which 90m recession is estimated for northern limit of the project area (Site 5) and 75m recession for southern limit of the project areas (Site 1). Faster recession rates are apparent on the southern (leeward) side of the Jetty, Boat Ramp and Nalinga Creek where longshore currents are displaced.

Two options are presented by JMG for closing Nalinga Creek catchment to allow for the development of an inland fresh water wetland. the first option to create a sea wall parallel to the shoreline is expected to have a design life lasting till 2050 based on the erosion assessments. The second shoreline perpendicular seawall which is outside of the 2100 erosion limits may be effective up until in developing a freshwater wetland over part of Nalinga Creek catchment behind Whitemark (Site 6), provided the ground is naturally impervious to groundwater ingress.

Of the Sites assessed for coastal erosion and inundation hazard, Site 2 appears to be lower risk as it is located further inland and no recession has been historically observed, unlike other Sites along the coast.

As a trial, the characteristic conditions at Site 2 should be replicated at other Sites where high erosion rates are apparent. Conditions identified at Site 2 include a steep elevated beach profile comprising of coarse-grained sand deposits built well above the high tide mark. The upper shoreline is also heavily armoured with seagrass. It may be that fine-grained sand deposits are apparent above 1.5 m AHD at some locations. A trial may involve removing and replacing fine sand deposits with coarse grained material and elevating the beach face.

A coastal development buffer of 90 m (Site 5) through to 75 m (Site 1) should be considered for any town planning. Building within this erosion zone must require specific engineering design to ensure the structures are firmly seated below wave scour levels. These methods involve costly construction methods and long-term servicing issues. Buildings constructed on the coast outside of this erosion zone need to consider wave run-up levels. Developments are not recommended on the eastern side of Whitemark township until a more conclusive flood study is conducted.

List of Abbreviations

AHD(83)	Australian Height Datum
AEP	Annual Exceedance Probability
ARI	Average Reoccurrence Interval
CEM	Coastal Engineering Manual
CEHC	Coastal Erosion Hazards Code
DCP	Dynamic Cone Penetrometer
DEM	Digital Elevation Model
DPAC	Department of Premier and Cabinet
ERMP	Erosion Risk Management plan
GES	Geo-Environmental Solutions Pty Ltd
GIS	Geographical Information System
IPAC	Inundation Prone Areas Code
IPCC	Intergovernmental Panel on Climate Change
IPS	Interim Planning Scheme
LIDAR	Light Detection And Ranging
LIST	Land and Information System, Tasmania
MRT	Mineral Resources Tasmania
NCCOE	National Committee on Coastal and Ocean Engineering
SB	Soil Bore
SPM	Shoreline Protection Manual
SSP	Surf Similarity Parameter
SWAN	Simulating Waves Nearshore
TAFI	Tasmanian Aquiculture and Fisheries Institute
WRL	Water Research Laboratory (University of New South Wales)

1 Introduction

Flinders Island Council is seeking to assess community vulnerability for Whitemark with specific reference to six (6) investigation areas which cover inland areas up to 800m from the coast and coastal areas covering approximately 2km of coastline between Whitemark & Nalinga Creek to the south.

JMG Engineers and Planners have undertaken a hydrology review of Nalinga Creek in a report dated October 2018.

This report comprises a coastal vulnerability assessment of four (4) of the six (6) investigation Sites and provides recommendations for management of the coastline including vulnerable areas subject to changing conditions due to climate change.

2 Objectives

Project objectives include:

- Produce a coastal vulnerability assessment report for the study area;
- Assess and define the existing & potential costal hazards associated with climate change projections for the study area. This includes assessing coastline inundation, wave climate, coastline recession & storm erosion;
- Make general recommendations for the coast line in the study area as well as specific recommendations for development on Sites noted Site 1 to Site 5

3 Site Details

3.1 Project Area Land Title

The investigation Sites studied in this report (Figure 1) presented in are defined by the following title reference:

- CT 203960/1 (33 Esplanade – Flinders Island Golf Course - Site 1, 2, & 3);
- Crown Land (Site 4); and
- CT 129006/1 (16 Esplanade - Tasmanian Ports Corporation Pty Ltd - Site 5)

The parcels of land are referred to as the ‘Project Area’ in this report.

Figure 1 Investigation sites

3.2 Project Area Regional Coastal Setting

The Project Area is located on the West Coast of Flinders Island within the township of Whitemark (Figure 1).

Whitemark is positioned on beach and sheet sand deposits. Lower relief flood plains are located to the east of Whitemark at the foothills of an elongated NNW to SSE directed range (Cannes Hill & Hays Hill located approximately 2.5 to 4km from the coast). The range has approximately 100m relief and is backed by Darling Range which is 6 to 8km inland with an elevation of 500m.

The coastline near the project area is exposed to diffracted and refracted swell wave activity from Bass Strait and wind wave activity within Parry's Bay. The project area is exposed to the following coastal processes:

- Wind fetch and wind setup directed from the south-west to north-west;
- Swell waves from the west to northwest;
- Short term inundation from storm tide, wind and wave setup & wave runup combined with longer-term sea level rise inundation;
- Coastline recession from sea level rise; and
- Short term coastal erosion from storm wave activity.

Figure 2 Regional location of project area – Google Earth

Figure 3 Regional location of project area - Google Earth

Figure 4 Location of project area investigation Sites - Google Earth

4 Planning

4.1 Australian Building Code Board

This report presents a summary of the overall Site risk to coastal erosion and inundation processes. This assessment has been conducted for the year 2080 which allows for a 'normal' 50-year building design life category based on planning up to 2030 (ABCB 2015). Modelling is also presented for 2100 which will allow for planning up to 2050.

As per the Australian Building Code Board (ABCB 2015), when addressing building minimum design life:

> *'The design life of buildings should be taken as 'Normal" for all building importance categories unless otherwise stated.'*

As per Table 3-1, the building design life is 50 years for a normal building.

Table 3-1 Design life of building and plumbing installations and their components

Building Design Life Category	*Building Design Life (years)*	*Design life for components or sub systems readily accessible and economical to replace or repair (years)*	*Design life for components or sub systems with moderate ease of access but difficult or costly to replace or repair (years)*	*Design life for components or sub systems not accessible or not economical to replace or repair (years)*
Short	1 < dl < 15	5 or dl (if dl<5)	dl	dl
Normal	50	5	15	50
Long	100 or more	10	25	100

Note: Design Life (dl) in years

4.2 State Coastal Policy

On 16 April 2003 the State Coastal Policy Validation Act 2003 came into effect. This Act replaces the former definition of the Coastal Zone in the State Coastal Policy 1996. The Act also validates all previous decisions made under the Policy. The following clauses are pertinent to the scope of this report:

1.1. NATURAL RESOURCES AND ECOSYSTEMS

1.1.2. The coastal zone will be managed to protect ecological, geomorphological and geological coastal features and aquatic environments of conservation value.

1.4. COASTAL HAZARDS

1.4.1. Areas subject to significant risk from natural coastal processes and hazards such as flooding, storms, erosion, landslip, littoral drift, dune mobility and sea-level rise will be identified and managed to minimise the need for engineering or remediation works to protect land, property and human life.

1.4.2. Development on actively mobile landforms such as frontal dunes will not be permitted except for works consistent with Outcome 1.4.1.

1.4.3. Policies will be developed to respond to the potential effects of climate change (including sea-level rise) on use and development in the coastal zone.

4.3 Building Act 2000

The Building Act 2000 and Building Regulations 2014 incorporate coastal inundation in provisions relating to land subject to flooding. Under the Regulations, the floor height of habitable rooms must be 300mm above the designated flood level.

Under Regulation 15 of the Building Regulations 2014, the following is defined as the designated flood level:

(a) 600mm above ground level or the highest known flood level, whichever is the highest, for land known to be subject to flooding other than as provided in paragraph (b), (c) or (d);

(b) the level which has a one per cent probability of being exceeded in any year for 10 stipulated floodplains;

(c) 600mm above the ordinary high-water mark of the spring tide for land on which flooding is affected by the rise and fall of the tide; and

(d) in respect of a watercourse floodplain not mentioned in paragraph (b), a level that, according to a report adopted by the relevant council, has a one per cent probability of being exceeded in any year.

4.4 Interim Planning Scheme Overlays

Flinders Island Council does not fall within Interim Planning Scheme (IPS) - the proposed State-wide Planning Scheme. The IPS which has been established in the south of the state (and east coast) and includes regulations for development in coastal vulnerable areas including areas which may be at risk of inundation and coastal erosion.

4.5 Flinders Island Council Planning Scheme

Flinders Island Council Planning Scheme 1994 is the legal document regulating development in the Municipality. The Flinders Island Council Planning Scheme provides for areas affected by coastal waters defined within 100m of the coast. The controls seek to restrict subdivision and require a setback from the coast of 100m from the High-Water Mark (HWM) unless the council are satisfied that it is reasonable to do so.

4.6 Coastal Hazards Technical Report

The following technical report was released by the Department of Premier and Cabinet in December 2016 (DPAC 2016):

'Mitigating Natural Hazards through Land Use Planning and Building Control. Coastal Hazards Technical Report. '

Although the report is not a regulatory document, it contains revised inundation and erosion hazard bands which include areas outside of the Southern Region which are not included in the IPS. Modelling used to develop the inundation hazard bands has been incorporated into this coastal vulnerability assessment.

Modelling includes:

- 2050 and 2100 sea level rise projections and planning allowances for each coastal municipality in the State based on based on the sea level rise projections provided in the Intergovernmental Panel on Climate Change Fifth Assessment Report (IPCC AR5) and the high emissions scenario RCP8.5
- Revised annual exceedance probability data based of tide gauge information used to assess storm surge (from barometric low) and extreme tide variability. The model was refined in 2016 (version 4) to assess extremes across the state.

4.7 Proposed Development

Specific development plans have not been put in place on Flinders Island. This report comprises a scoping study of six (6) individual Sites within the Whitemark area. At least four of these Sites may be vulnerable to coastline recession and storm erosion (Site 1, 2, 4 & 5), and other Sites are located inland (Site 3 & 6) are within the flood plain area and may be vulnerable to coastal inundation as well as fluvial/pluvial inundation.

JMG's task was to assess the change in inland hydrology for Nalinga Creek due to climate change and to interpret the resulting impacts on inland water levels due to any changes in boundary conditions at the coastline. This may include increases in sea level and changes in the bathymetry that would alter river levels. Specific Sites being addressed in this assessment are Sites 3 and Site 6.

5 Physical Site Assessment

5.1 *Geology*

Mapping of surface geology is available from Mineral Resources Tasmania (Figure 5). 1:250,000 Scale Mineral Resources Tasmania geological mapping for North-East Tasmania indicates that the predominant lithology interpreted to underlie the Site comprises undifferentiated Quaternary sediments (sand, gravel and mud) of alluvial of lacustrine and littoral origin. Undifferentiated sediments underlie the floodplain.

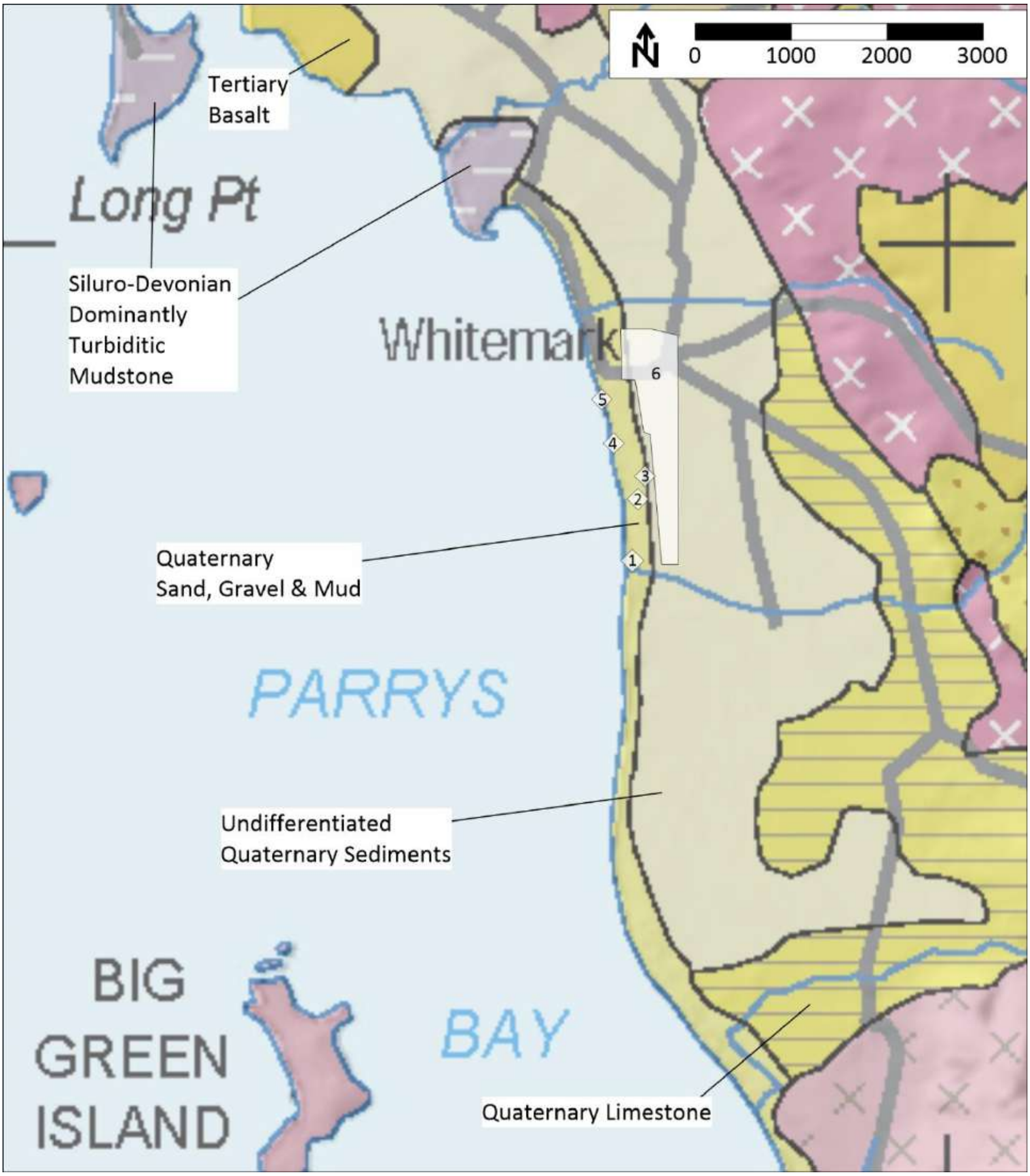


Figure 5 Local Geology (MRT 1:250,000 – North-East Tasmania)

5.2 *Project Area Geomorphology*

Whitemark is located on a sand dune ridge deposit which extends up to 0.5km inland from Whitemark Beach (Figure 6). Remnants of small blowout dunes including deflation hollows and dune mounding is present within the dune ridges along the coastline. The dune ridges range in height from 2m Australian Height Datum (AHD) to 6m AHD with the deflation hollows at the lower end of the elevation range which is like the backing flood plain deposits (Figure 7). There is very little evidence of dune sand mobility except within 50m of the shoreline. The dunes appear to be largely stabilised by introduced vegetation.

Figure 6 Shaded aerial imagery showing individual Site locations

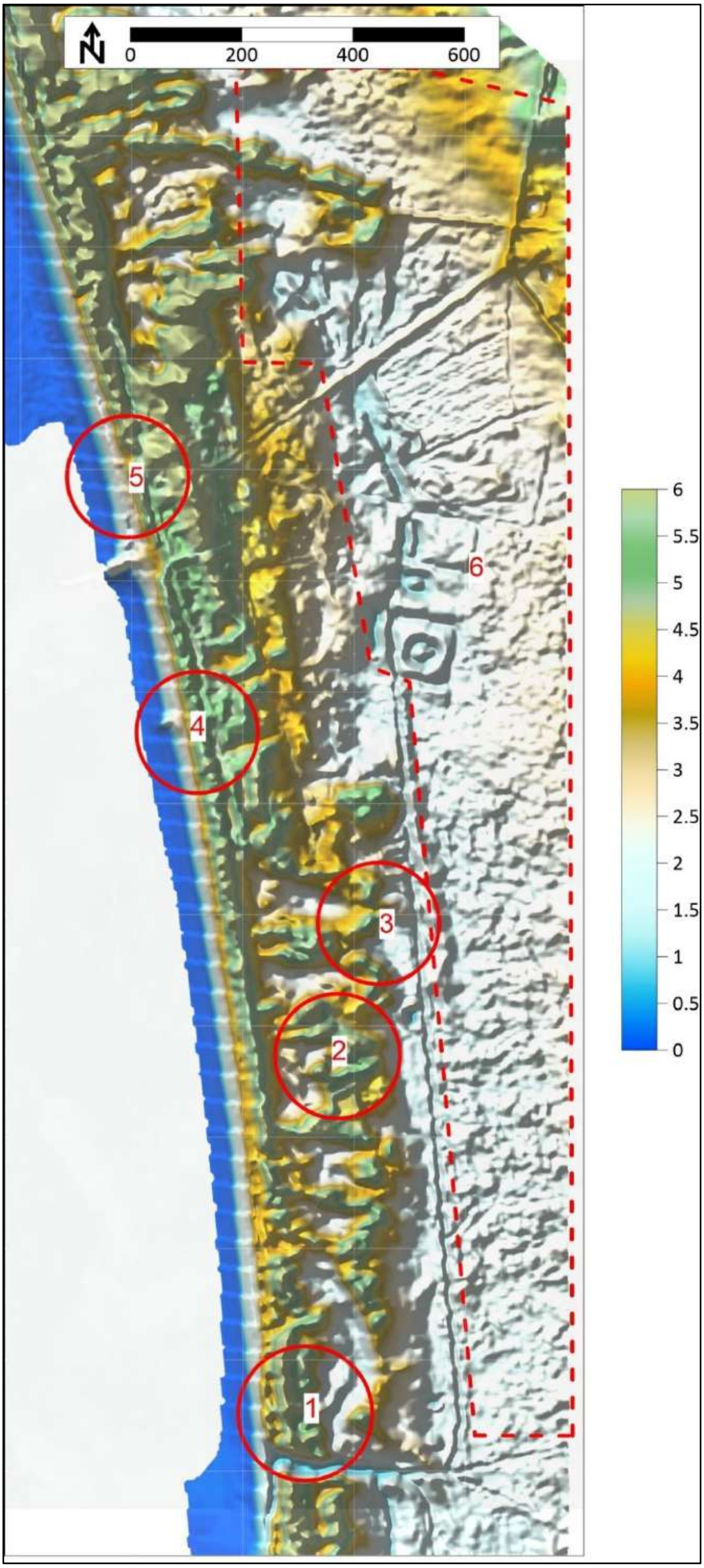


Figure 7 Shaded relief DEM showing individual Site locations

It is apparent that there has historically been abundant sand supply within the nearshore environment which has allowed for coastline progradation to occur. The progradation would have primarily occurred from more recent stable sea level conditions over the last ~5000 years. Radiocarbon dating of shell fragments in beach gravel deposits collected from Nalinga Creek at Site 1 (Plate 1) from a depth of 1.8m indicate an age of 3990 years based on a standard deviation of 100 years (Gill 1968).

Plate 1 Photograph taken at Nalinga Creek (Site 1) outflow directed from east to south. Erosion discernible within escarpment.

Anecdotally, there were extensive lagoon systems backing Whitemark Beach before European settlement. Nalinga Creek exists as a series of culverts which appear to have been excavated to allow the floodplains to drain to the coast (Plate 2 & Figure 8). The dune system in this area comprises of windblown sheet sand deposits which are actively eroding forming steep escarpments (Plate 3) and a delta which is exposed at low tide (Plate 4). Nalinga Creek is brackish and is expected to become increasingly saline, deeper and broader as sea levels continue to rise (Plate 5 & Plate 6).

Plate 2 Photograph taken of Nalinga Creek southern culvert which runs perpendicular to the coastline.

Figure 8 Nalinga Creek Alignment

Historically, extreme inland flooding is expected to have discharged to the ocean through a combination of groundwater flow and potentially sheet-flow of the frontal dune system. There may have been historical flood-water breaches through the dune system during extreme inland flooding events.

Plate 3 Eroding fine-grained sheet sand deposits forming a shoreline escarpment on the southern flanks of Nalinga Creek outflow

Plate 4 Eroded fine grained sediments are forming a small delta deposit at the Nalinga Creek outflow. Photograph directed east towards Whitemark Beach & Nalinga Creek at low tde approximately 180 m from the dunes. The delta is a relatively modern occurrence and is projected to continue to expand rapidly as sea levels rise.

Plate 5 Photo of Nalinga Creek outflow channel incised through the dune system. Seagrass and other marine Debris line the channel up to 800 m inland

Plate 6 Photo of Nalinga Creek which Is actively scouring the fine grained sand dune deposits

Along a large majority of the beach, fine-grained windblown sand deposits overlie coarse-grained marine sand deposits above the average high-water mark (~1.7m AHD) at an estimated elevation of 2.2 to 2.5m AHD (Plate 7). The exception being near Nalinga Creek where the coarse-grained deposits were not visible in the creek bed but were historically identified along the banks (Gill 1968).

Dynamic Cone Penetrometer (DCP) profiling indicates layered clay, silt, sand, gravel and shell or cobble deposits below 2.5m AHD. DCP refusal was typically encountered at an elevation of -1m AHD along the shoreline, frontal dune and blowout dune areas. The more common material discernible below 2.5m AHD was coarse-grained sand which is exposed at low water mark (~1.5m AHD). Based on DCP profiling, frontal dunes between Site 2 and Site 5 comprise of deep coarse-grained sand deposits with occasional fine-grained banding.

Plate 7 Damaged storm water pipe sections which drain the northern (Hayes Hill) catchment were displaced in the recent storm. Regular and combined influence of high tide, wind setup and wave runup is actively scouring fine grained sand on Whitemark Beach ~1km to the north of Whitemark.

Coarse-grained sand deposits were rapidly discharging groundwater along the shoreline at a flow rate consistent with the 2 to 5mm grain size observed (Plate 9). It is expected the gravel deposits continue inland beneath the dunes and is expected to be the primary aquifer in the local area, discharging groundwater from the wetlands and deeper fractured rock aquifers. Between Sites 2 and Site 5, the beach face is typically coarse grained (up to 5mm) and has a relatively steep shoreline profile. The upper beach profile is typically lined with seagrass which offers wave runup scour protection (Plate 8). The seagrass is presumed to predominantly comprise of *Posidonia australis* which is most common in the area (Rees, 1993).

It is apparent that seagrass has formed thick dense matting in the upper shore face. The seagrass is overgrown and stabilised with exotic vegetation in the backshore berm. The seagrass matting is expected to play a critical role in protecting the shoreline from erosion. Likewise, the coarse-grained sand offers resilience to wave runup erosion.

The seagrass is harvested, dried and exported overseas to be generated into commerial products.

Plate 8 Photo Taken Near Site 5. Seagrass lining the shoreline profile offers wave runup protection. Sand deposits are primarily coarse grained and form a moderately steep face.

Plate 9 Groundwater discharging through coarse grained beach sediments near Site 5 during low tide

5.3 Bathymetry

A chain of offshore islands 5 to 10km from the coastline (part of the Furneaux Group) provide partial protection from wind and swell wave activity. On the islands leeward side, the bathymetry has a low gradient and shallows radially within Parry's Bay. As a result, waves are attenuated and largely refracted within the nearshore zone. This is particularly apparent to the north-west of Whitemark. According to available navigation charts (Figure 9), towards Whitemark, the bathymetry shallows from:

- -2.5 m AHD to -1.0 m AHD over 2.5 km (0.14% gradient) to the north-west;
- -5.5 m AHD to -1.0 m AHD over 3.8 km (0.17% gradient) west; and
- -17.5 AHD to -1.0 m AHD over 8.2 km (0.22% gradient) south-west.

The attenuated hydrodynamic regime has allowed for the development of seagrass and seaweed beds within the shallow nearshore zone. Seagrass and seaweed further attenuate wave activity through seabed friction within the nearshore zone as well as through erosion scour reduction on the beach face.

Due to the shallow water conditions, wind setup contributes significantly to coastal inundation, within the project area.

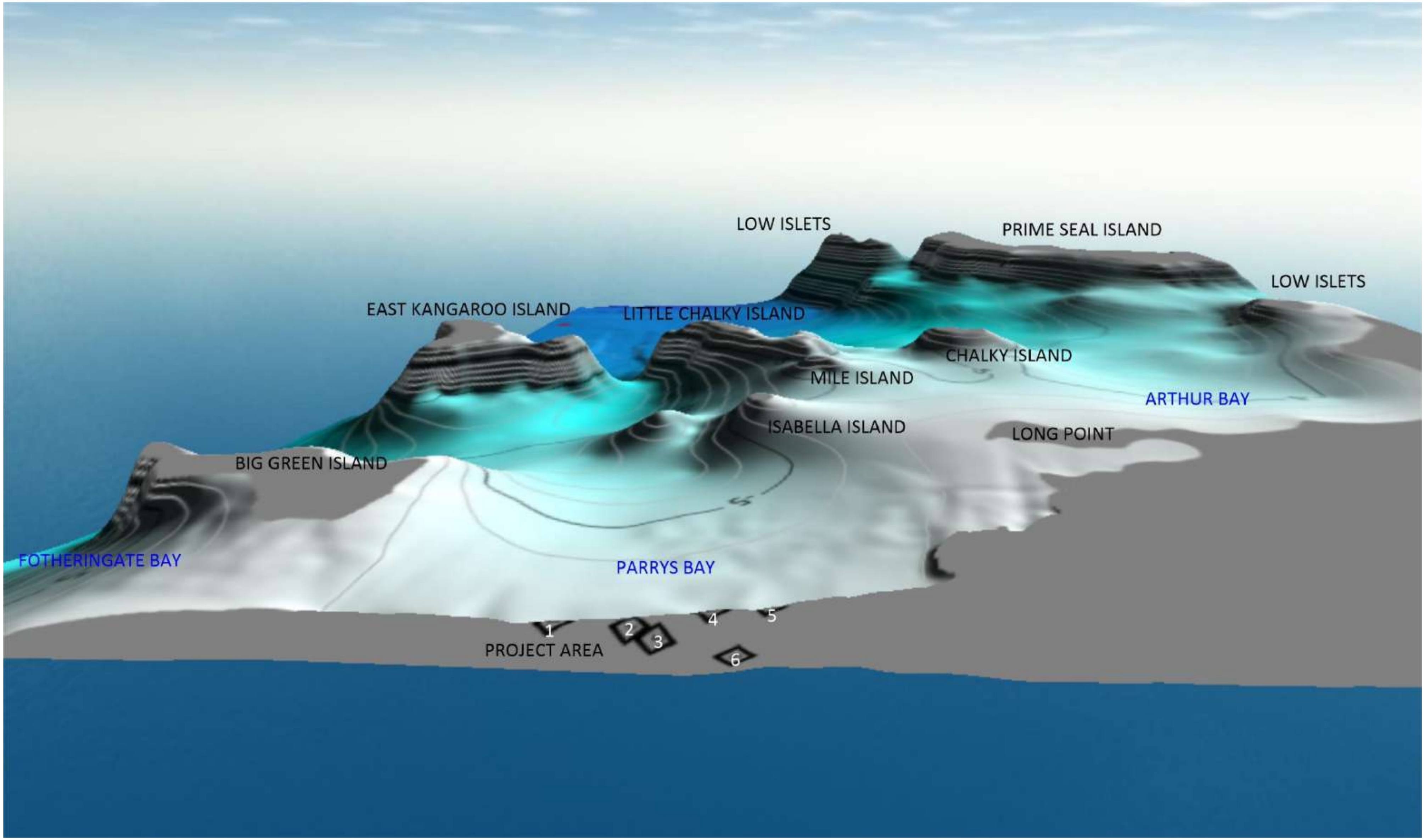


Figure 9 Bathymetry generated from LIDAR (Light Detection And Ranging) imagery (collected during low tide) and admiralty mapping (Navionics 2018)

5.4 Shoreline Geometry

The beach face and nearshore profile is shaped by tidal processes, tidal currents, and attenuated nearshore swell and locally derived wave energy directed from the south-west to north-west.

Photographs of the shoreline taken at low tide illustrate extensive tidal flats extending to Isabella Island and Long Point to the west and Big Green Island to the south-west.

Plate 10 Photograph of Whitemark Beach Tidal flats taken from Whitemark Jetty. Photo Directed to the NNW towards Long Point

Plate 11 Photograph taken from Whitemark Beach illustrating to low tide terrace. Photo directed to the NNW towards Long Point

5.5 Beach Classification

5.5.1 Reflective

Ozcoasts (Short 2015) have identified Whitemark Beach as being of the *reflective* type. *reflective* beaches are typically coarse-grained, have a relatively low wave energy (waves typically ~0.5m high) and have no bar or surf zone as waves move unbroken to the shore where they collapse or surge up the beach face.

5.5.2 Beach Plus Tidal Sand Flat

Beach plus tidal sand flat morphology is also used by Ozcoasts to describe Whitemark Beach. *Beach plus tidal sand flat* type beaches are typically lower energy (waves typically ~0.16m high) compared with *reflective* beach types due to the typical presence of extensive tidal flats.

5.5.3 Low Tide Terrace

Merani *et. al.*, 2012, describe beaches on the North coast of Tasmania as falling into one of two general beach types (Short 2015):

- *Low tide terrace*; and
- *Transverse bar and rip*.

Given the lower wave energy, Whitemark Beach between Sites 2 and 5 fit best into the *low tide terrace* type classification with the following characteristics:

- Moderately steep beach face which is joined at the low tide level to an attached bar or terrace;
- Waves averaging about 1m (higher wave energy than interpreted for Whitemark by Ozcosts);
- A bar usually extends between 20-50m seaward and continues alongshore;
- At high tide when waves are less than 1m, they may pass right over the bar and not break until the beach face (typically reflective at high tide);
- The hydrodynamic regime is not extreme enough to generate rip currents; and
- Cusps may be apparent (not apparent at Whitemark possibly due to the presence of the natural seagrass armouring along the shoreline).

5.5.4 Summary

Under present wave conditions, waves directed from the north-west across the shallow water tidal flats have resulted in the formation of a beach plus tidal flat geometry, whereas waves directed from the west are more conducive to the development of a *reflective* type beach profile.

The low tide terrace is expected to have been more pronounced historically before the extensive reef/sand deposition and seagrass growth which has largely attenuated wave activity. As sea levels rise, the *low tide terrace* (and *reflective types*) may become more pronounced as sea levels rise and wave activity becomes more active within the shoreline due to reduced wave attenuation.

All coastal project area Sites show some evidence of all three beach profile types.

5.6 Coastal Erosion Overview

It has been established that coarse grain marine sediments sourcing from the eroding granite hills to the east form the current shoreline and extend inland beneath the main dune ridges. The sediments are known to have continuity inland based on DCP profiling and knowledge of the groundwater flow rates discharging from the beach sediments. The coarse sediments have contributed to a reflective type beach profile. Silt and clay lagoon deposits are apparent beneath the coarse sediments near Nalinga Creek.

Fine grained windblown sediments overlying the coarse-grained sand and gravel sediments form the higher relief dune ridges. These sands are highly vulnerable to wind and wave erosion. Although introduced vegetation (including grasses) may have largely stabilised the sand from aeolian erosion, wind erosion is prevalent where wave runup scour is washing out the easily erodible fine-grained sand deposits. This erosion is most pronounced in areas where the seagrass matting is not available to provide wave runup protection. Wind erosion from prevailing onshore winds is noted to be active in these areas.

The base of these fine windblown sand deposits is highly variable in elevation and is estimated at a depth of between 1.5 and 2.0m AHD inland and at 1.5 to 5.0m on the beach face. The coarse-grained beach deposits and the seagrass offers considerable natural armouring to the overlying fine-grained sandy sediments. It is apparent that this armouring has been removed around Nalinga Creek when the drainage channels were cut to open new farming land. Due to sea level rise and the lack of natural shoreline armouring at Nalinbga Creek outlet, the fine-grained dune sand is being eroded and mobilised to the intertidal zone where it is forming a delta. Aerial photographs indicate that the delta is expanding at an increasingly rapid rate (Figure 10).

Similarly, both the fine- and coarse-grained sand has been excavated around the large stormwater drainage pipe exiting to the coast to the north of Whitemark. It is apparent the excavated sand was haphazardly depoSited back around and over the top of the pipe and is currently eroding out given the principle material used in the backfill comprised of fine sands which are more vulnerable to erosion compared with the surrounding coarse-grained sediments. Moreover, deepening water around the pipe outlet is causing concentrated wave energy.

Seagrass and sea weed line the upper shoreline along parts of the beach where fine grained sediments are apparent. This is pronounced at Site 5 and most apparently at Site 2.

Figure 10 Nalinga Creek delta continues to expand as sea levels rise

Of concern are the following processes:

- Sea levels rise causing erosion of the fine sandy sediments in the upper beach profile, particularly areas where the coarse-grained sediments are of a slightly lower elevation or where the sediments have been historically etched out to allow for natural drainage (Nalinga Creek and the stormwater pipe to the north of Whitemark);
- Less wave attenuation from seagrass within the nearshore zone due to sea level rise, resulting in
    - Greater wave energy closer to coastline;
    - Denudation of seagrass beds from increased wave activity;
    - Burying of the seagrass with fine sediments eroding from shoreline; and
    - Amplification of the above processes (due to reduced overall wave attenuation).

It is apparent that natural shoreline armouring of coarse-grained sediments and seagrass/seaweed are offering natural protection from these processes.

5.7 Previous Studies

DPAC has developed GIS mapping of coastal inundation and erosion hazards throughout the state (DPAC 2016). The maps are publicly available on The Land and Information System, Tasmania (LIST) Tasmanian Government mapping portal. Erosion of up to 80 m has been inferred for Parry's Beach, and coastal inundation hazards behind the dune systems.

As with this project, LIDAR data is limited to a thin stretch of coastline to the north and south of Whitemark. Beyond these points, inundation hazards can not be discerned.

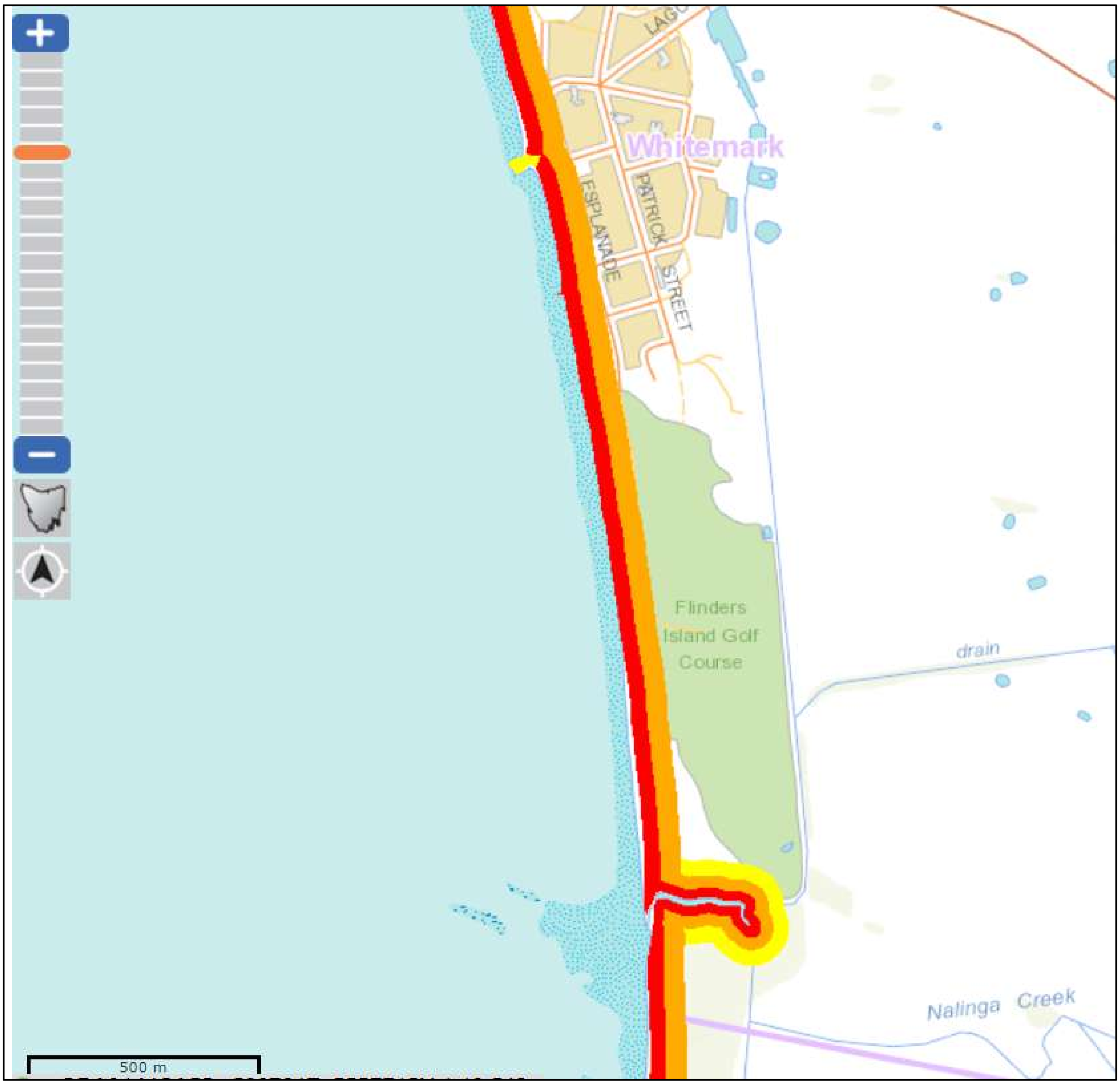


Figure 11 DPAC coastal erosion hazard bands inferring a 35 m high hazard band (Red) and a 45 m medium hazard band (Orange)

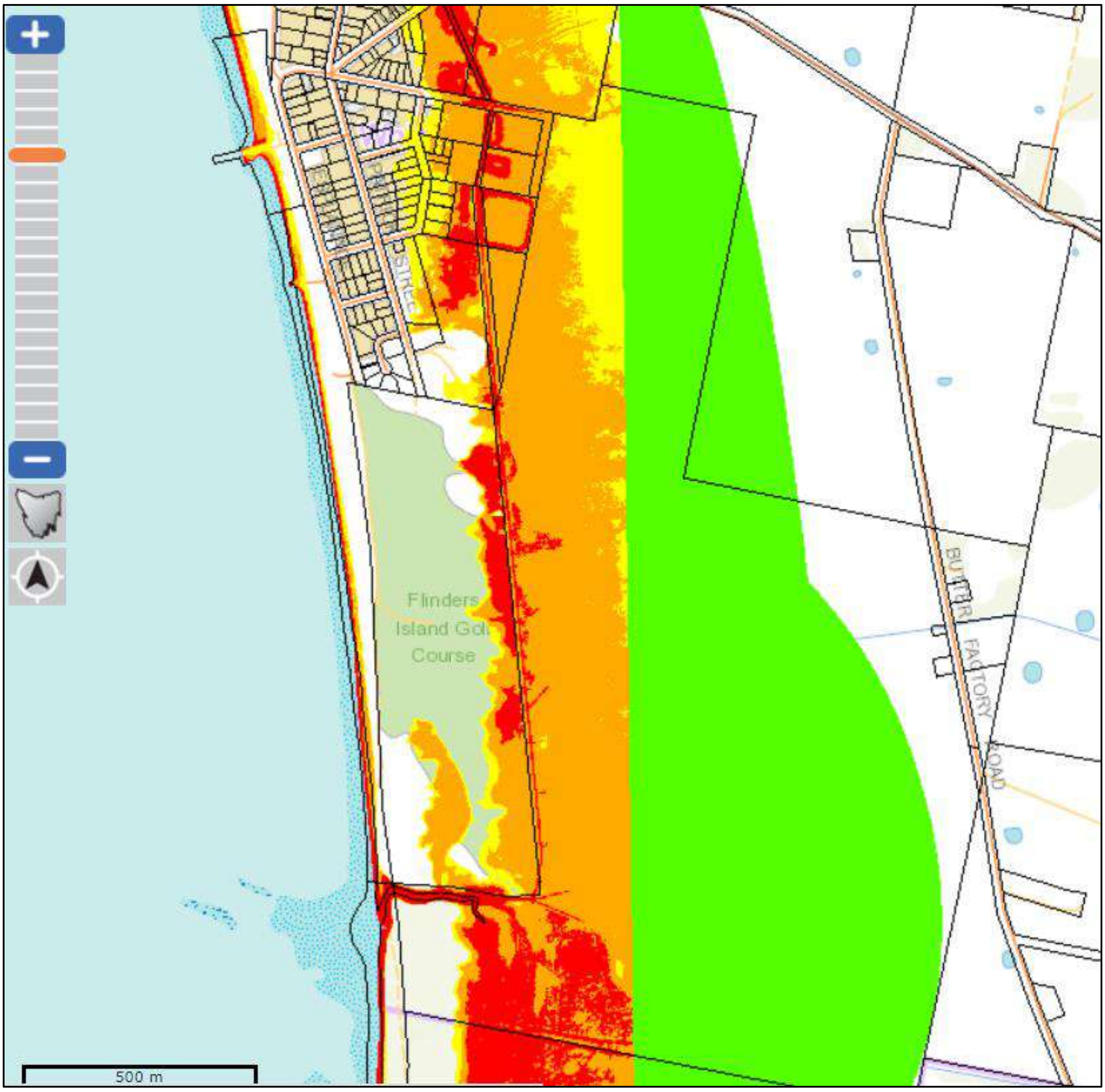


Figure 12 DPAC coastal inundation hazard bands inferring a 1.8 m AHD high hazard band (Red), a 2.4 m AHD medium hazard band (orange), a 3.0 m AHD low hazard band (yellow) and a NO LIDAR zone where inundation hazard bands could not be discerned due to lack of LIDAR data (green).

6 Stillwater Inundation Assessment

6.1 *Previous Studies*

Other than DPAC's production of erosion and inundation hazards bands, GES are not aware of any previous studies within the project area.

6.2 *Site Baseline Seawater Levels*

6.2.1 Storm Tide

Storm tide events may be defined in terms of the culmination of astronomical tide and storm surge events. Maximum storm tide inundation levels have been adopted for the Site based on a 1% Annual Exceedance Probability (AEP) that an inundation event will occur. Storm tide levels are obtained from the IPS (2015) inundation hazard tables.

The storm tide level for the Site based on 0 m sea level for 2010 is 1.85 m (1.93 m based on m AHD)

6.2.2 Sea Level Rise

The IPS (2015) has adopted the following sea level rise estimates based on projections for Whitemark (DPAC 2016) with reference to a 2010 baseline:

- 0.31 m AHD levels by 2050; and
- 1.00 m AHD levels by 2100.

Based on these figures, sea level elevations presented in Table 1 are applied to the Site. The 2080 projections for planning and development provision, used as a basis for this report are presented in Table 1. Other dates presented with the table are used in interpreting inundation levels for aerial photographs.

Table 1 Inundation level based on historical data (Church and White 2011) & DPAC (2012) projections

Year	Sea Level Projections*	
	2010 Baseline (m) m	(1972 Baseline) m AHD83
Historical Sea Level Charts (Church and White 2011)		
1951		-0.038
1973		0.002
1982		0.020
1986		0.029
1989		0.035
1992		0.042
1998		0.055
2003		0.066
2006		0.072
2009		0.079
2012	0.005	0.085
2013	0.007	0.088
2015	0.013	0.094
Sea Levels Based on RCP8.5 sea level rise scenario modelled for the Flinders Council (DPAC 2016)		
2008	0.077	-0.004
2010	0.081	0.000
2013	0.088	0.007
2018	0.104	0.023
2050	0.311	0.230
2068	0.507	0.427
2080	0.671	0.590
2080	0.671	0.590
2100	1.001	0.920

6.2.3 Wind Setup

Wind setup has been calculated based on south-westerly, westerly and north-westerly wind incidents. Procedures and formulations are presented in Kamphuis (2006) and are based on an open ocean scenario with a ~4.0 m AHD shallow to deep water transition zone. Wind velocities adopted are based on procedures outlined in Carley et. al. (2008). Directional multipliers are applied based on AS 1170.2:2002 and duration multipliers are calibrated to the wind fetch scenario. Iterations were calculated for

- 2018 mean tide;
- 2018 1% AEP Storm Tide;
- 2080 1% AEP Storm Tide; and
- 2100 1% AEP Storm Tide

There were only minor differences in wind fetch under these inundation conditions possibly due to only minor changes in wind fetch distances and the ocean wind setup model adopted (Table 2).

Table 2 Summary of wind fetch

Direction	SW	W	NW
Wind Setup (m)	0.11	0.34	0.55

6.2.4 Stillwater Levels

The effects of storm tide may be combined with sea level projections to provide baseline water levels (reported in m AHD) which are referred to as still water levels.

The still-water levels adopted for the Site is based on 1% AEP storm tides and 2080 inundation levels derived from DPAC (2016) estimates for Whitemark (Table 3).

Table 3 Summary of Site stillwater levels for present day, projected 2080 & 2100 inundation levels based on DPAC (2012) estimates.

Stillwater Elevations	2018 RCP8.5	2080 RCP8.5	2100 RCP8.5
Sea Levels (m AHD)	0.10	0.67	1.00
1% AEP Storm Tide Influence (m)	1.85	1.85	1.85
Wind Setup (m)*	0.55	0.55	0.54
Wind Setup Direction	north-west	north-west	north-west
Summary (m AHD)	2.50	3.07	3.39

7 Coastline Hydrodynamics

Coastal process hydrodynamics were assessed at the Site. Information collected is used to assist in interpreting Site specific:

- Maximum Site inundation levels from wave runup and wave setup;
- Effects of storm inundation levels on Site erosion; and
- Longer term recession trends.

Without consideration of Site hydrodynamic wave models, these potential hazards cannot be addressed. It is recognised however, that a Site-specific coastal processes study is imperative in any coastal vulnerability assessment which seeks to identify the potential hazards and potential risks to assets and life.

All information obtained for the hydrodynamic assessment is obtained from available spatial and non-spatial scientific data as well as computations outlined in various manuals and texts including the Coastal Engineering Manual, the Shoreline Protection Manual and various texts from the Advanced Series on Ocean Engineering.

7.1 Methods

7.1.1 Swell Waves

The Site nearshore swell wave heights are derived from the following procedures:

- Extraction of hourly significant wave height and wave period data from hind-cast WAVEWATCH III model (CAWCR 2013) based on a 31-year data period from 1979 to 2010;
- The wave grid point selected is based on the largest swell wave in 40 m water depth (proven to be most accurate by CAWCR 2013);
- Data is extracted for a specific wave direction towards the Site using a targeted wave direction towards Site 1 and Site 5 incidents and using a +/- 5° threshold data capture radius;
- The dataset is projected onto a probability chart to obtain 1% AEP significant wave heights for each incident wave direction (procedures outlined in Chapter 4 of Kamphuis 2006);
- Shoreline Protection Manual (SPM) and Coastal Engineering Manual (CEM) are used to determine wave diffraction and wave refraction around the islands and within Parry’s Bay and determine resulting 1% AEP significant wave height reductions;
- The Simulating Waves Nearshore (SWAN) model is used to determine nearshore wave attenuation;
- All other nearshore wave and erosion processes are based on attenuated significant wave heights; and
- Swell waves are determined for Site 1 and Site 5 as they represent the limits of the project area.

7.1.2 Wind Waves

Wind waves have been modelled at the Site-based procedures outlined by Carley et. al., (2008). The wind fetch wave model has been developed based on the CEM (2008) and SPM (1984) formulations which interpret Site bathymetry, topography and wind speeds. Similarly, wind waves are determine for Site 1 and Site 5 as they represent the limits of the project area.

7.1.3 Wave Attenuation

A separate wave attenuation (CEM 2008) and shoaling model (Kamphuis 2006) has been applied to the deep water swell and wind waves. There are very basic models which may be used to apply to determining the friction coefficient of the seagrass and seaweed beds. The more reliable models are based on CEM principles which apply attenuation from various sea bed grain sizes which are derived from Reynolds number (Soulsby 1997).

Regardless of the type of vegetation, the resulting wave height decay for submerged vegetation is usually described as an exponential function:

$H2/H1 = e^{-k_i \Delta x}$

Where H1 and H2 are the wave heights at shore normal positions separated by the Δx (distance), and the exponent ki is the decay coefficient (Kobayashi et. al., 1993; Moller et. al., 1999). Depending on the characteristics of the vegetation, the decay coefficient may vary from 0.01 (Moller et. al., 1999) to 0.05 (Kobayashi et. al., 1993). Bradley & Houser (2009) were able to demonstrate that wave heights decrease exponentially when passing through seagrass beds based on their acoustic Doppler velocimeter (ADV) studies.

7.1.4 Wave Inundation

Hydrodynamic risks are measured in terms of 1% AEP events. Site specific processes considered in this section include but are not limited to the following (some of which are detailed in Figure 8):

- Wave runup;
- Wave setup; and
- Wind setup.

A 300mm freeboard value has been adopted by the IPS (2015) to account to for the Tasmanian Building Regulations (2014). Site hydrodynamic factors are included within this 300mm freeboard zone which essentially defines any hydrodynamic inundation processes which are above the adopted still water levels.

The 30 mm value will tend to overestimate inundation levels at some Sites and underestimate inundation levels at other Sites.

As wind setup, wave setup and wave runup normally occur simultaneously during storm surge events, these components are combined with extreme tide and storm surge predictions to provide maximum inundation levels for the Site. Wave models have been generated for the Site to define the Site specific hazards.

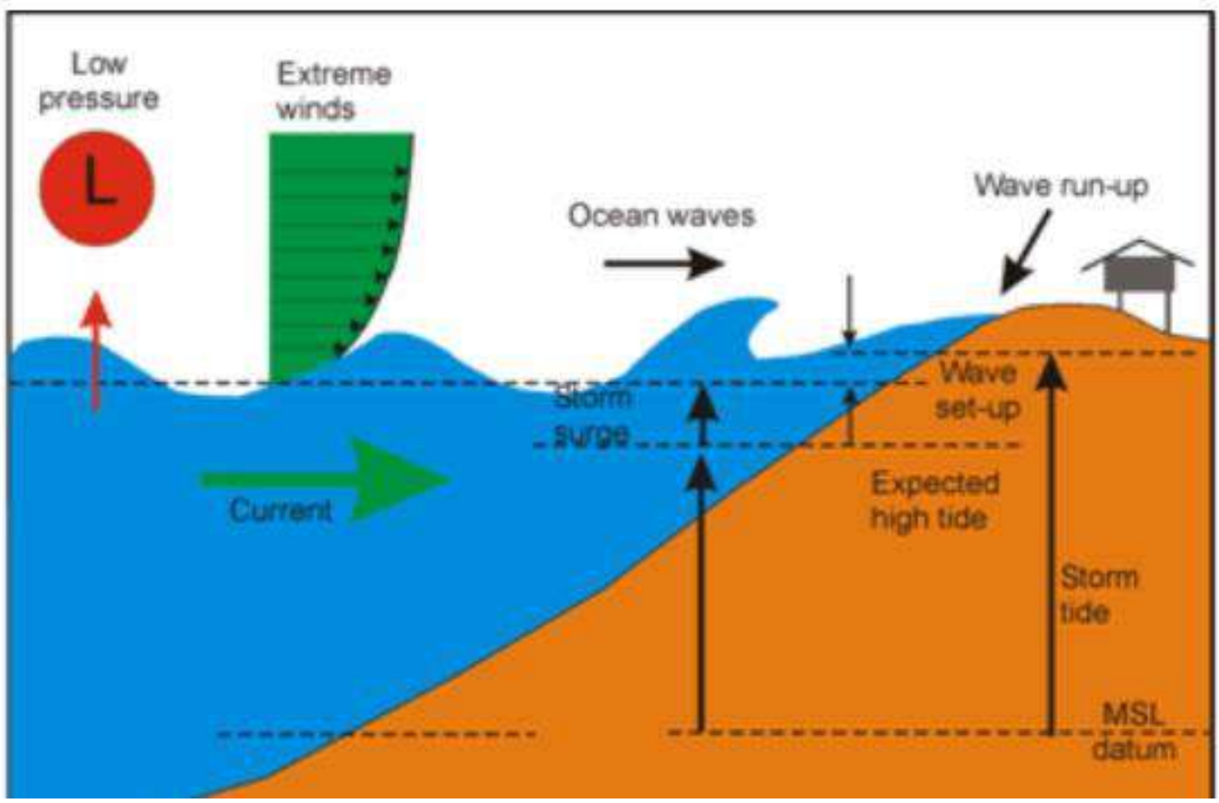


Figure 13 Hydrodynamic parameters associated with storm surge events

7.2 Findings

7.2.1 Significant Swell Waves

The 4' Pacific WAVEWATCH III grid selected for Site has coordinates and corresponding water depth presented in Table 4.

Table 4 Pacific WAVEWATCH III grid point used to interpret offshore swell wave parameters

Coordinate & Depth	Value
Easting (GDA94 Zone 55)	568128
Northing (GDA94 Zone 55)	5549737
Depth (m AHD)	-40.2
Distance offshore (km)	20

Islands and shallow water to the north-west and south-west largely protect the project area from south-westerly and north-westerly swell wave attack. Waves from these directions have therefore not been applied to the Site. Predominate swell waves directed towards the Site diffract between East Kangaroo Island and Little Chalky Island.

Offshore significant wave heights adopted for the Site are summarised in Table 5. Wave incidences from WbS have a 1% AEP wave height of 5.7 m which are slightly larger than waves directed from the WSW at 4.7 m AHD.

Table 5 Annual exceedance probability of various significant waves modelled for the project area

Incidence	1% AEP Significant Wave Height	Wave Period (seconds)
247° (WSW) – Site 5 Incidence	4.7 m	8.0
255° (WbS) – Site 1 Incidence	5.7	8.0

Westerly swell waves are diffracted and refracted radially towards the north-east, east to south-east. Due to the broad range of incidents, the larger wave height from 255° was been applied to the project area. SWAN modelling and diffraction and refraction formulations indicate significant wave heights are reduced to 2.0m within 6km of the shoreline.

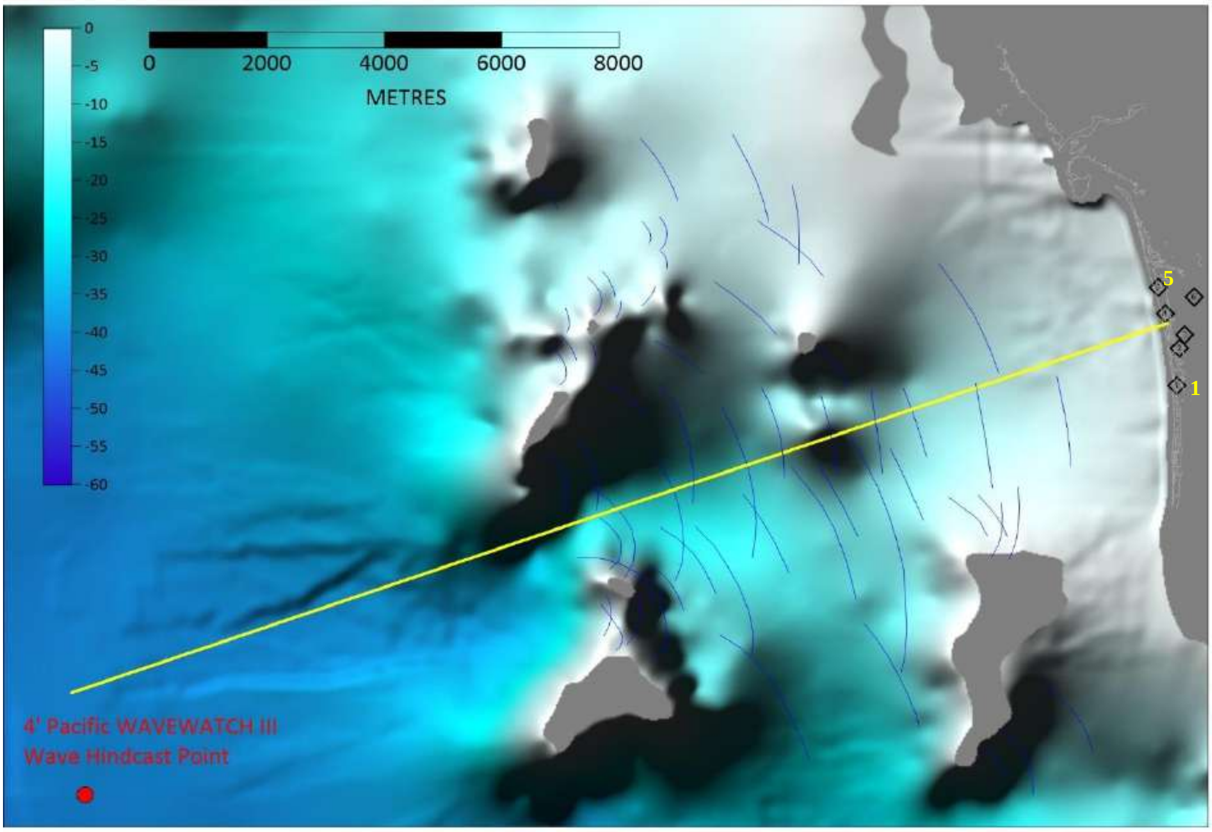


Figure 14 WSW wave diffraction and refraction assessment of wave heights within Parry's Bay

7.2.2 Localised Wind Waves

Localised wind generated significant deep-water wave heights based on a 100-year AEP are summarised in Table 6, Table 7 & Table 8. Parameters used in the analysis are summarised in Appendix 2.

Westerly and south-westerly wind wave heights as well are wave period are larger at Site 1. There is negligible increase in wave heights with sea level rise with the exception for Site 5 where north-westerly wind waves are projected to increase in height.

Table 6 Westerly significant deepwater wave heights calculated for Site 1 and Site 5

	2018	2080	2100
Site 1 (m)	2.20	2.20	2.20
Site 5 (m)	1.76	1.77	1.78

Table 7 North-Westerly significant deepwater wave heights calculated for Site 1 and Site 5

	2018	2080	2100
Site 1 (m)	1.98	2.00	2.01
Site 5 (m)	1.10	1.16	1.20

Table 8 Average wave period calculated for Site 1 and Site 5

	SW	W	NW
Site 1 (m)	-	4.70	4.30
Site 5 (m)	4.15	4.15	3.19

7.2.3 Wave Conditions

Radials and significant swell wave heights used to derive local wave conditions at the Project Area are presented in Appendix 2. Table 9 provides a summary of the dominant waves intercepting Site 1 and Table 10 provides a summary of the dominant waves intercepting Site 5. Summary data presented in Table 9 and Table 10 are based on projected 2080 conditions.

Westerly wind and swell waves are more pronounced at Site 1, whereas wind waves directed towards Site 5 from the south-west have equal significance to westerly waves.

Table 9 Summary of dominant waves intercepting Site 1

Wave Details	Local Wind Fetch	Swell Wave	Local Wind Fetch
Direction	West	West	North-west
Wave Height (m)*	2.2	2.0	2.0
Period (s)*	4.7	8.0	4.3
Approach Angle (degrees)	0	0	30

Table 10 Summary of dominant waves intercepting Site 5

Wave Details	Local Wind Fetch	Local Wind Fetch	Local Wind Fetch
Direction	South-west	West	North-west
Wave Height (m)*	1.8	1.8	1.1
Period (s)*	4.1	4.1	3.2
Approach Angle (degrees)	30	0	30

7.2.4 Dominant Wave Characteristics

The larger 1% AEP wave to intercept Site 1 originates from a local westerly wind wave with a wave height of 2.2 m and a wave period of 4.7 seconds (Table 11). The larger 1% AEP wave intercepting Site 5 originates from south-westerly wind fetch.

Table 11 Details of the dominant wave intercepting Site 1

Wave Position	Parameter	Value
Nearshore	Origin	Local Wind Fetch
	Direction	West
	Approach Angle (degrees)	0
	Deepwater Wave Height (m)	2.2
	Period (s)	4.7
Breaking	Breaker Height (m)	1.2
	Breaking Depth (m)	2.8
	Breaking Angle (degrees)	0
	Nearshore Gradient (%)	8.5

Table 12 Details of the dominant wave intercepting Site 5

Wave Position	Parameter	Value
Nearshore	Origin	Local Wind Fetch
	Direction	Southwest
	Approach Angle (degrees)	30
	Deepwater Wave Height (m)	1.8
	Period (s)	4.2
Breaking	Breaker Height (m)	1.3
	Breaking Depth (m)	2.4
	Breaking Angle (degrees)	20
	Nearshore Gradient (%)	5.7

7.2.5 Wave Attenuation from Sea Bed Friction & Breaker Zone Hydrodynamics

Formulations indicate that the Airy Wave Theory is applicable before the breaker point with an Ursell Number less than 25 in all cases. The Boussinesq equations are subsequently applied to waves in shallow water.

Using a conservative decay coefficient of 0.02 for westerly waves passing across a 350m section of seagrass bed (consistent with waves directed from the west towards Site 5), an attenuation factor of 100% is calculated (Bradley & Houser 2009).

Considering mean tide conditions and Soulsby (1997) formulations, preliminary calculation indicate that 100% westerly wave height reduction can be achieved over 350m with the application of a friction coefficient consistent with a 5 mm grain size. This will result in extreme waves reaching the breaker zone 110m offshore. This same friction coefficient factor has been applied for all wave directions and inundation scenarios modelled for the Site.

The wave attenuation model demonstrates that despite almost complete reduction in wave height during present day mean tide conditions, under extreme inundation (storm tide and sea level rise) conditions, significant wave heights in the nearshore will continue to increase as sea levels rise due to reduced wave attenuation from sea bed bottom friction (Table 13).

Modelling indicates there is a high potential for large south-westerly wind waves to build between East Kangaroo and Big Green Island and impact Site 5 and areas to the north of Site 5 (Table 14).

Table 13 Site 1 - Significant wave heights within 300m of the shoreline

	Present Mean Tide	Present 1% AEP	2080 1% AEP	2100 1% AEP
Westerly Wind (m)	0.0	0.4	0.6	0.8
Westerly Swell (m)	0.0	0.4	0.6	0.8
North-Westerly Wind (m)	0.0	0.4	0.6	0.7

Table 14 Site 5 - Significant wave heights within 300m of the shoreline

	Present Mean Tide	Present 1% AEP	2080 1% AEP	2100 1% AEP
Westerly Wind (m)	0.0	0.5	0.7	0.8
Westerly Swell (m)	0.0	0.4	0.6	0.8
North-Westerly Wind (m)	0.0	0.1	0.2	0.3
South-Westerly Wind (m)	0.2	0.9	1.1	1.3

At Site 1 and Site 5, under extreme inundation conditions, waves will break much closer to the shoreline. Breaking wave heights will increase causing disruption to seagrass beds. Although the effect of seagrass loss/burial from the nearshore zone has not been modelled, wave heights in the nearshore are expected to further increase as seagrass becomes dislodged and/or buried with fine sediment eroding from the upper shoreline profile.

Table 15 Site 1 significant wave heights at breaking point

	Present Mean Tide	Present 1% AEP	2080 1% AEP	2100 1% AEP
Westerly Wind (m)	0.9[1]	1.0[2]	1.15[2]	1.3[2]
Westerly Swell (m)	0.8[1]	0.7[2]	0.8[2]	0.9[2]
North-Westerly Wind (m)	0.1	0.4	0.6	0.7

1 Waves Breaking >1500 m offshore
2 Waves Breaking <300 m offshore

Table 16 Site 5 significant wave heights at breaking point

	Present Mean Tide	Present 1% AEP	2080 1% AEP	2100 1% AEP
Westerly Wind (m)	0.9[1]	1.0[2]	1.15[2]	1.3[2]
Westerly Swell (m)	0.8[1]	0.7[2]	0.8[2]	0.9[2]
North-Westerly Wind (m)	0.1	0.4	0.6	0.7

1 Waves Breaking >1500 m offshore
2 Waves Breaking <300 m offshore

7.2.6 Wave Runup and Wave Setup

Hydrodynamic variables calculated for the Site 1 and Site 5 are presented in Table 17 & Table 18. Inundation levels are calculated from wave runup and wave setup combined with the stillwater levels. Wave runup is applicable to beachfront areas and based on current beach profile, whereas wave setup is based on nearshore gradients and is applicable to backshore areas (Nalinga Creek) where waves are given the opportunity to migrate inland.

Table 17 Site 1 wave hydrodynamics based on 1% AEP present day, 2080 & 2100 scenarios

Coastal Process	2018 RCP8.5	2080 RCP8.5	2100 RCP8.5
Modelled worst case scenario combined wave & wind setup	North-westerly Wind	North-westerly Wind	North-westerly Wind
Wave setup (m)	0.25	0.25	0.21
Wind Setup (m)	0.48	0.48	0.47
Wave Runup Scenario	Westerly Swell	Westerly Swell	Westerly Swell
R2% Wave Runup Based on (Mase 1989)*	2.14	2.61	2.68

*Smooth Beach

Table 18 Site 5 wave hydrodynamics based on 1% AEP present day, 2080 & 2100 scenarios

Coastal Process	2018 RCP8.5	2080 RCP8.5	2100 RCP8.5
Modelled worst case scenario combined wave & wind setup	North-westerly Wind	North-westerly Wind	North-westerly Wind
Wave setup (m)	0.17	0.14	0.16
Wind Setup (m)	0.55	0.55	0.54
Wave Runup Scenario	Westerly Wind	Westerly Wind	Westerly Wind
R2% Wave Runup Based on (Mase 1989)*	1.78	2.05	2.41

*Smooth Beach

8 Coastline Inundation Levels

Table 19 andTable 20 present a summary of the Site inundation levels based on 1% AEP still water, wind setup, wave runup and wave setup inundation levels for present day, 2080 planning and development and 2100 DPAC scenarios. Site 1 and Site 5 have been selected as they represent the bounds of the project area.

Maximum inundation levels within the project area are defined by groundwater levels, coastal inundation levels and inland fluvial and pluvial flooding. This report concerns coastal inundation levels only.

The following broad scenarios will become increasingly apparent within the project area as sea levels rise:

- Groundwater inundation will become increasingly apparent in the form of lakes within the dune deflation hollows and soaks in the lower lying drainage channel areas;
- Inland flooding is expected to be apparent between Whitemark and Trousers Point;
- The existing drainage culvert systems in the backwater areas will play a critical role in channelling stormwater runoff as well as saltwater intrusion and discharge from the expanding lagoon system. These channels are also likely to naturally increase in size through erosion as tidal cycles become pronounced;
- Nalinga Creek entrance is expected to exponentially expand over time, and a permanent deep-water lagoon re-entrance channel is expected to be gouged out by 2100.

8.1 Site 1

At Site 1, although a larger wave setup value of 0.27m has been identified from the west, combined wind and wave setup values from the north-west have been adopted given the higher modelled combined wave and wind setup inundation level of 0.73m. Wind setup levels from the north-west are inferred at 0.48m, and wave setup levels from the north-west are inferred at 0.25m.

Based on a 2080 1% annual exceedance probability (AEP) scenario, stillwater inundation levels are modelled to be at 3 m AHD near Site 1. Wave influenced inundation levels are modelled at 3.25m AHD close to the shoreline (wave setup) and 5.61m AHD within the wave runup zone based on the current beach profile gradient. Steepening of the beach profile over time will induce a higher wave runup limit, and conversely, flattening of the beach profile will result in reduced wave runup.

Table 19 Site 1 coastal inundation levels based on present day, 2080 & 2100 1% AEP scenarios

1% AEP Inundation Levels (m AHD)	2018 RCP8.5	2080 RCP8.5	2100 RCP8.5
Still Water Elevations Including Wind Setup	2.43	3.00	3.32
Wave Setup Inundation	2.68	3.25	3.53
R2% Wave Runup Elevations Based on (Mase 1989)*	4.57	5.61	6.00

*Smooth Beach

For 2050 design purposes at Site 1, any structures located within 100m of and parallel to the coast should be designed based on wave runup levels, with overtopping expected at 3.5m AHD for a structure is elevated to 3.5m AHD (with no wave attenuation armouring).

Structures designed perpendicular (and behind the receding dune front) should be designed based on wave setup inundation, with overtopping expected at 2.96m AHD for 2050 scenario.

8.2 Site 5

At Site 5, although a larger wave setup value of 0.25m has been identified from the west, combined wind and wave setup values from the north-west have been adopted given the higher modelled combined wave and wind setup inundation level of 0.68m. Wind setup levels from the north-west are at 0.55m, and wave setup levels from the northwest are at 0.14m.

Based on a 2080 1% annual exceedance probability (AEP) scenario, stillwater inundation levels (non-wave influenced levels) are modelled to be at 3.07m AHD. Wave influenced inundation levels are modelled at 3.21m AHD close to the shoreline (wave setup) and 5.12m AHD within the wave runup zone based on the present day measured profile.

Table 20 Site 5 coastal inundation levels based on present day, 2080 & 2100 1% AEP scenarios

1% AEP Inundation Levels (m AHD)	2018 RCP8.5	2080 RCP8.5	2100 RCP8.5
Still Water Elevations Including Wind Setup	2.50	3.07	3.39
Wave Setup Inundation	2.68	3.21	3.55
R2% Wave Runup Elevations Based on (Mase 1989)*	4.28	5.12	5.80

*Smooth Beach

9 Sediment Erosion Assessment

9.1 Previous Studies

GES are not aware of any coastal erosion assessments in the Whitemark area.

9.2 Scope of Works

Table 21 presents a summary of the various methods adopted by GES to identify erosion hazards in vulnerable coastal zones.

Table 21 Summary of assessment approaches adopted to identify Site erosion hazards

Investigative Approach	Investigation Details	Typical Application
Invasive Investigation.	Conduct borehole drilling or substrate profiling to make inferences about the susceptibility of the Site to erosion	Where scouring is anticipated, or building foundation can be established on a firm substrate
Site Historical Aerial Imaging	Assess historical long-term shoreline position relative to sea levels at the time and how this may translate to future recession trends	Where the proposed development is in a medium to high risk erosion zone and recession models need confirmation, or may not apply given the coastal setting
	Assess historical short-term shoreline positions relative to known storm events to forward project sediment storm erosion demand.	Used where Tasmarc surveys are not available or there is no previous storm erosion modelling done for the Site.
Shoreline Recession Model	Development of a long-term shoreline recession model based on projected DPAC (2012) sea level rise scenarios and using calculated closure depths and various Bruun Rule formulations (1988)	Where Site is in an inferred to be in an erosion hazard zone and where the proposed development building cannot be founded on a stable foundation.
Storm Erosion Demand	Conduct a detailed assessment of Site storm erosion vulnerability due to coastal processes as well as available geological and geomorphological information	Where Site is in an inferred to be in an erosion hazard zone and where the proposed development building cannot be founded on a stable foundation.
Stable Foundation Zones	Development of a cross section through the Site detailing zone of reduced foundation capacity and the stable foundation zone through Nielsen et. al. (1992) methods	Where Site is in an inferred to be in an erosion hazard zone and where the proposed development building cannot be founded on a stable foundation.
Tidal prism analysis.	Mean low and high tides are assessed to determine likely inland inundation and tidal prism volumes. This can be used to assess erosion scour around Nalinga Creek.	This is applied to situations where a historical lagoon system is projected to reopen as a result of sea level rise.

9.3 Historical Aerial Imagery

A series of historical aerial photographs and Google Earth satellite images were reviewed to assess historical storm surge erosion and shoreline recession (Appendix 2). Imagery has not been orthorectified, there is reasonable confidence in the accuracy of the information. The margin of error of the image georeferencing is expected to be in the order of 2m. A total of eight Sites were assessed for coastline erosion within the project area (Table 22).

Relative changes in the shoreline position are summarised in Table 23 and workings are presented in Appendix 3. Many of the Sites either did not present a reliable recession trend (an R2 of greater than 0.75) or it was apparent that the erosion trend was unreliable as it is affected by structures/features on the coastline such are the beach ramp and jetty which are acting as a groyne or the south of Nalinga Creek outflow which is eroding at considerably different rate to the reminder of the coastline. The outcome is an apparent coastline recession relationship with sea level rise based on pre-2010 historical sea level rise charting (Church & White 2011) and Flinders Island Council projected sea level rise trends (DPAC 2016). A Bruun Rule recession rate of 90 to 100m horizontal per metre rise in sea level has been calculated for the more reliable of the recession trends. A high level of correlation between sea level rise and recession is achieved possibly due to the lack of extreme storm bite from the heavily attenuated swell and wind waves. Storm bite is expected to be more apparent to the north of Whitemark which is impacted by south-westerly wind waves.

9.3.1 Site 1

The most pronounced erosion is apparent immediately to the south of Nalinga Creek, with an estimated Bruun Rule correlation of 377m of coastline recession per metre rise in sea level with a R2 probability of 95%. The northern side of Nalinga Creek entrance has a lower recession rate at 103m per metre sea level rise although the correlation has a lower probability with an R2 of 75%. It is readily apparent that a spit is beginning to form across Nalinga Creek outflow and sand is eroding along the length of the channel as well as on the southern flanks of Nalinga Creek outflow. A delta comprising of fine-grained sediments is forming at the Nalinga Creek entrance. By 2080, Site 1 N is projected to have receded by ~60m and Site 1 S by ~200m.

9.3.2 Site 2

There is no recession trend apparent at Site 2 North and South. It is apparent that Site 2 is a mid-beach section, there is neither loss or gain. Over the 63-year period, only 6 to 7m of erosion variation has been documented, with part of this being accounted for in georeferencing error. The beach profile in this area is more elevated compared with other Sites, has a steeper face, and has larger volumes of seagrass accumulation near the storm wave runup limit.

9.3.3 Site 4

A reasonable trend is apparent at Site 4 North and South with an R2 of 78% and 76% respectively. The shoreline to the south of the boat ramp appears to be receding at approximately double the rate compared with the north. Recession rates are inferred to range from 116m (north) to 206m (south) per metre sea level rise. Similarly, high recession rates are expected on the southern side of the main jetty due to longshore current development from the north and subsequent loss of sand supply and scour on the leeward side. By 2080, Site 4 N is projected to have receded by ~65m and Site 1 S by ~120m.

9.3.4 Site 5

No recession is apparent at Site 5 S, immediately to the north of the main jetty. This is to be expected due to the accumulation of sand on the north side of the structure. By 2080, Site 5 N is projected to have receded by ~50m.

Table 22 Summary of historical shoreline erosion areas

Site Name	Easting	Northing	Comments
Site 1 South	586666	5556556	Located 60m south of Nalinga Creek where a steady recession is apparent within the widening beach area
Site 1 North	586635	5556701	80m north of Nalinga Creek, away from the more dynamic creek outflow areas
Site 2 South	586587	5557186	Directly coastward of the golf clubhouse, there are no particular coastal features apparent other than the steep coarse-grained beach profile and 20 to 30m of vegetation stabilised seagrass
Site 2 North	586558	5557420	200m north of the clubhouse. As above.
Site 4 South	586495	5557890	50m south of Martin Street boat ramp. Sand is observed to have eroded from the southern side of the boat ramp.
Site 4 North	586451	5558094	150m north of Martin Street boat ramp. Sand is observed to have accumulated on the northern side of the boat ramp.
Site 5 South	586389	5558311	70m north of Bowman Street jetty. Sand is observed to have accumulated at this location.
Site 5 North	586326	5558547	300m north of Bowman Street jetty. Appears to be north of the sand accumulation zone.

An assumption is made that any erosion apparent outside (deviating from) the main recession trend may have occurred due to storm events (see Appendix 3). Similarly, a progradation (sand accretion) trend may be apparent in the opposite Site direction. This deviation has been broadly used to characterise and quantify storm bite events. If the deviation is multiplied by the profile thickness a basic storm erosion demand value can be determined. In cases there is strong evidence of a recession trend, theoretically a long historical time series (where data is available) may be used to project storm erosion demand on micro (days) to macro (years) scales. Where this principle is applied to the project area, in the order of 20 to 60 m of storm erosion demand may be assumed.

Table 23 Summary of aerial photograph shoreline erosion analysis

Recession Analysis	Site 1 South	Site 1 North	Site 2 South	Site 2 North	Site 4 South	Site 4 North	Site 5 South	Site 5 North
Dune Height Within Erosion Zone (m)	6	6	5	5	5	5	6	6
Artefact? (Y/N?)~	Y	N	N	N	Y	Y	Y	N
Artefact Type~	Creek				Groyne	Groyne	Groyne	
Date Dange (years)	64	64	64	64	64	64	64	64
Erosion Range (m)	43	17	7	6	25	14	10	11
X	-376.7	-102.7	-7.5	11.1	-206.5	-115.6	50.4	-92.2
Y	-7.51	-4.21	2.04	-2.53	1.80	-0.75	-1.14	0.41
R2	0.95	0.71	0.02	0.06	0.76	0.78	0.44	0.75
Bruun Rule	377	103	*	*	206	116	*	92
Erosion Outside Recession Trend (m)	6.1	5.6	*	*	8.0	4.2	*	3.2
Projected 2050 Recession	78	58	*	*	117	66	*	52
Projected 2080 Recession	214	92	*	*	185	104	*	83
Projected 2100 Recession	338	0	0	0	0	0	0	0
Erosion Applied horizontal (m)	6.1	5.6	6.5	6.2	8.0	4.2	9.5	3.2
Storm Erosion (m3/m)	37	34	33	31	40	21	57	19
Bruun Rule Applicable	^	103	*	*	^	^	*	92

* No Trend Apparent

^ Where there is no recession trend apparent, the horizontal erosion range is applied

~ Artefacts include SALIENTS, RIVER MOUTHS, GROYNS, where there is an anomalie which may create deviation from the local beach tren

9.4 *Tidal Prism Analysis*

A tidal prism analysis has been conducted on Nalinga Creek and backing lagoon system to determine the likely effects of changes in sea level on the inflow and outflow of water within the evolving lagoon system. Over time it is projected that Nalinga Creek will become largely tidal, and to compensate, it is expected the channel will widen is response to ebb and flood flow.

Formulations presented in Neilsen (2012) have been used to describe the changes in the channel cross sectional area in response to changes in tidal prism with sea level rise.

It is discerned that initially changes in tidal prism will be minor with a 14% increase in tidal prism volume by 2080. Beyond 2080, tidal prism is expected to expand considerably, with a 150% increase, and a 10-fold expansion in the channel cross sectional area.

Table 24 Summary of tidal prism analysis*

Sea Level Scenario	2018 RCP8.5	2080 RCP8.5	2100 RCP8.5
Sea level (m AHD)	0.10	0.67	1.00
Mean Low Tide (m AHD)	-1.34	-0.77	-0.44
Mean High Tide (m AHD)	1.55	2.11	2.44
Minimum Volume* (m3)	2965587	4048424	4048424
Maximum Volume* (m3)	8381151	10195250	101952500
Tidal Prism (Difference In Volume) (m3)	5.42E+06	6.15E+06	9.79E+07
Change In Tidal Prism From Present		14%	1505%
Calculated Inlet Cross Section Area (m2)	450	550	5577

*May Underestimate True Volumes & Cross Section Areas due To Limited Extent of LIDAR Coverage

9.5 *Nalinga Creek Erosion*

As is apparent at Nalinga Creek, large volumes of sand are already being lost from the system. This may not necessarily be reflected in a deepening of the channel but in erosion of the dune ridges either side of the channel due to a combination of wave runup scour as well as tidal outflow, particularly during a flood event. Wave scour would be most apparent during storm tide events (particularly under extreme wind setup conditions) where waves are able to penetrate inland beyond the coastal margin.

There is evidence that a spit is beginning to develop on the northern side of Nalinga Creek. It is expected that over time this spit will expand; however, it is not clear if the evolving delta will obstruct or contribute to the formation of the spit.

Given the current trend the coastline to the south of Nalinga Creek will continue to recede inland at an increasing rate. By 2080 it is expected that the bulk of the dune system immediately to the south of Nalinga Creek will have eroded and contributed considerable volume of sand to the delta. It is probable that the dunes system will have eroded flat around the Nalinga Creek and for a considerable distance south. The dune immediately to the north of Nalinga Creek will have similarly retreated but to a lesser extent. A cross section has been inferred through Nalinga Creek defining projected changes in channel cross sectional area and morphology of the surrounding dune systems.

9.6 Bruun Recession Analysis

The Bruun Rule has been applied to Sites 1 and 5 to estimate the response of the shoreline profile to sea-level rise.

The Bruun Rule is widely used by government and non-government bodies to determine recession rates on sandy shores which are at risk of inundation. The Bruun Rule states that a typical concave-upward beach profile erodes sand from the beach face and deposits it offshore to maintain constant water depth. There are a few cases where the Bruun rule cannot be applied, which include where longshore drift is predominant, where there is dominant influence of surrounding headlands and in environments where wave activity is minimal.

9.6.1 Closure Depths

The most contentious variable for the Bruun rule is the closure depth for which various formulations and methods exist. The closure depth may be defined as the depth offshore of a beach where depths do not change with time. The closure depth is calculated based on methods derived by Dean and Darymple (2002). The parameters used in the assessment are presented in Table 25 & Table 26.

Table 25 Parameters used to calculate closure depth at Site 1

Variable	Value
Closure Depth (Vellinga 1983)	1.17
Wave Period (s)	5
Average Sand Grain Size (mm)	1.5
Closure depth (m)	1.50

Table 26 Parameters used to calculate closure depth at Site 5

Variable	Value
Closure Depth (Hallermeier)	1.30
Wave Period (s)	4
Average Sand Grain Size (mm)	1.5
Closure depth (m)	2.04

9.6.2 Bruun Rule Beach Recession Model

The standard Bruun Rule has been applied to the project area to determine sea level rise induced recession from the dominant waves active.

The Standard Bruun Rule is typically expressed as $R = s(L/(D + h))$ and is illustrated in Table 21

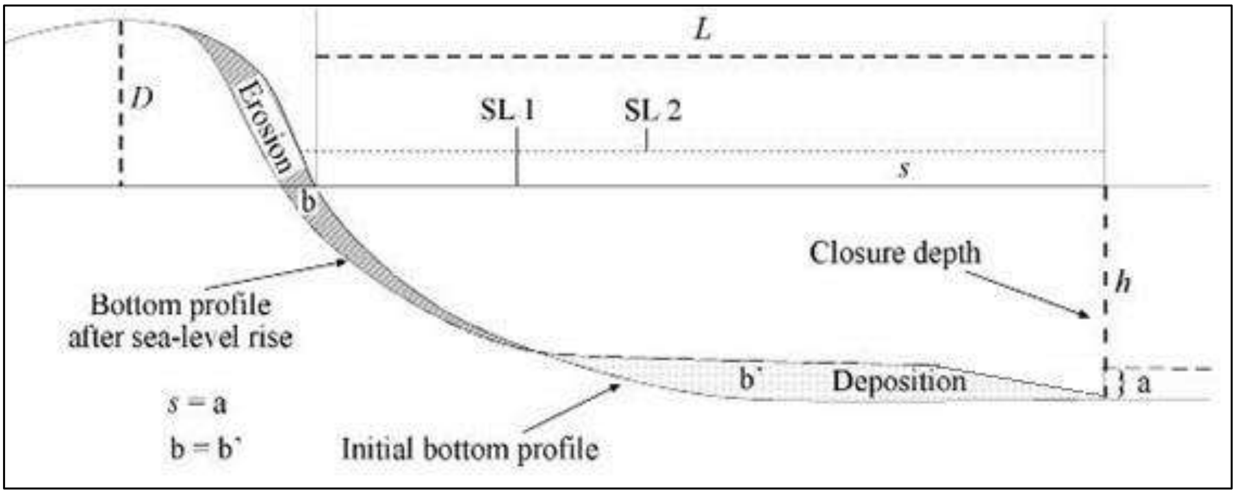


Figure 15 Summary of standard Bruun Rule for calculating beach recession

Table 27 and Table 28 presents a summary of the Bruun Rule variables utilised in the Site recession model which have been obtained from the digital elevation models for the Site.

Table 27 Summary Bruun Rule variables used in Site 1 recession model

Variable	Symbol	Value
Length of Active Erosion Zone (m)	L	0
Profile Closure Depth (m)	h	1.50
Active Dune/Berm Height (m)	D	6.00

Table 28 Summary Bruun rule variables used in Site 5 recession model

Variable	Symbol	Value
Length of Active Erosion Zone (m)	L	0
Profile Closure Depth (m)	h	2.04
Active Dune/Berm Height (m)	D	6.00

The recession rate given the various sea level rise scenarios are presented in Table 29 Table 30.

Table 29 Calculated Bruun Rule recession rate at Site 1

Variable	Symbol	2080 RCP8.5	2100 RCP8.5
Sea Level Rise above 2013 DPAC LiDAR baseline (m)	s	0.58	0.91
Horizontal Recession (m)	R	84	131

Table 30 Calculated Bruun Rule recession rate at Site 5

Variable	Symbol	2080 RCP8.5	2100 RCP8.5
Sea Level Rise above 2013 DPAC LiDAR baseline (m)	s	0.58	0.91
Horizontal Recession (m)	R	69	108

A horizontal recession value of ~84 m is inferred for Site 1 and ~69 m for Site 5 given projections for 2080 based on DPAC trends

From the historical aerial photographs 92 m horizontal recession has been interpreted for Site 1 North and 83m recession for Site 5 North based on 2080 projections. Mid-point estimations are at 88m for Site 1 and 76m for Site 5 for 2080 which may be rounded up to 90m for Site 1 and 80m for Site 5.

These estimations are generalisations which may be applied to only a limited part of the overall beach profile and a more complicated picture is apparent in many parts of the beach due to the presence of obstructions along the shoreline. The most significant variation in this trend is apparent to the south of Nalinga Creek.

9.7 Storm Erosion

Aside from longer term recession attributed to sea level rise, storm erosion events have the potential to cause beach erosion (storm bite) which is followed by a period of beach rebuilding. The erosion and nourishment cycle has historically been in equilibrium (with no net loss or gain over time) unless longer term recession or progradation is occurring.

GES considers a storm erosion demand of 20 m^3/m is applicable for the Site which is consistent with photographic interpretations and beach profile Type 5 as described by Mariani et.l al. (2012) for the north coast of Tasmania and best fits the beach description.

9.8 Stable Foundation Zone

A stable foundation zone assessment has been conducted for the Site. The basis behind this particular assessment involves the use of Nielsen et. al. (1992) methods for assessing stable foundation zones in sand.

Cross sections have been constructed through the Sites to indicate the worst case scenario 2080 sea level rise scenario based on recession modelling (Figure 16 & Figure 17). The storm erosion demand has been constructed based on Nielsen et. al. (1992) equations which use a 1:10 post storm gradient. A storm erosion demand of 50 m^3/m has been applied to the Site to account for a 1% AEP storm event.

9.9 Erosion Summary

A series of Figures have been compiled (Figure 19 to Figure 24) to illustrate likely erosion and inundation extent across Sites 1, 2, 4 and 5.

Figure 16 Site cross sections

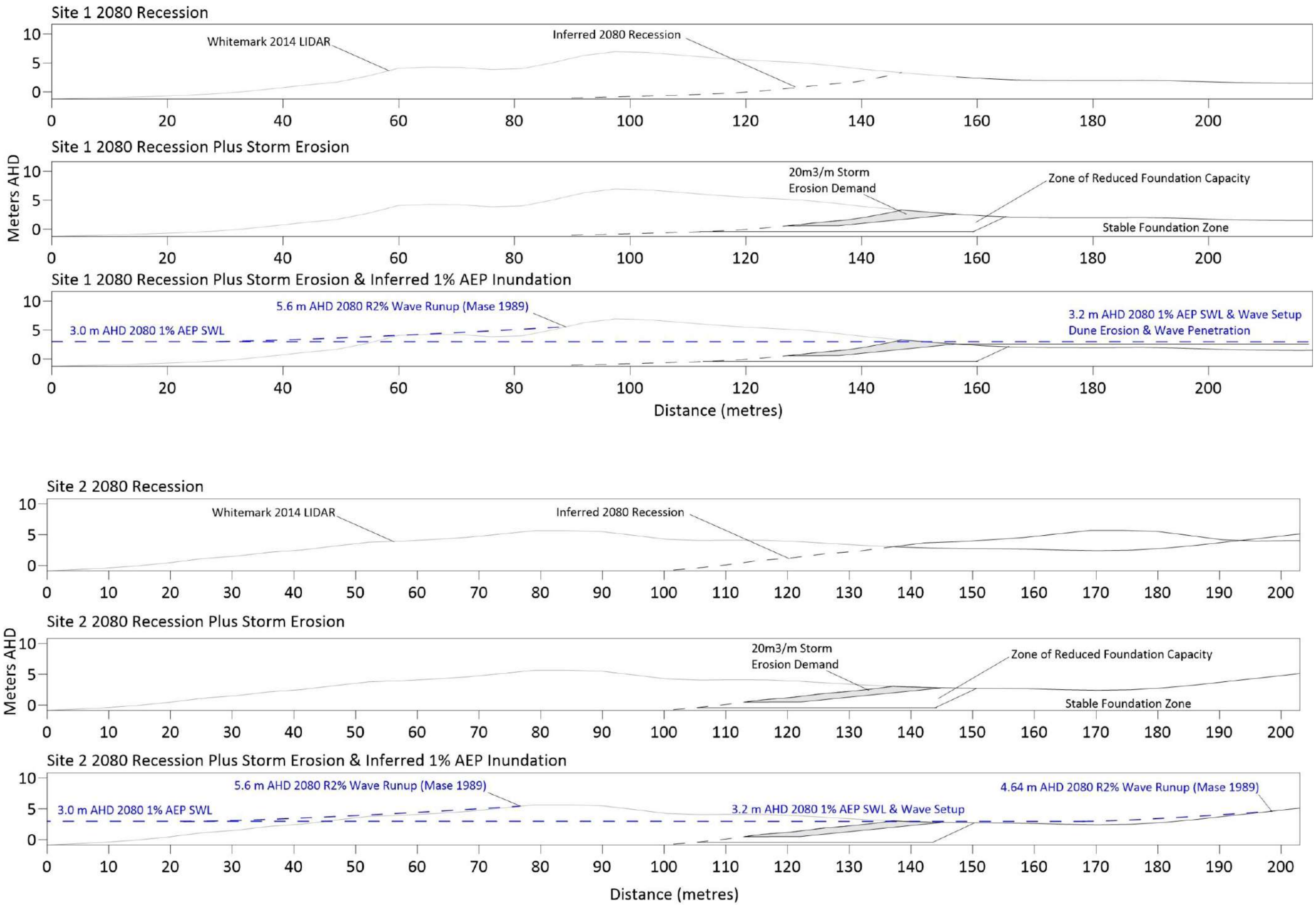
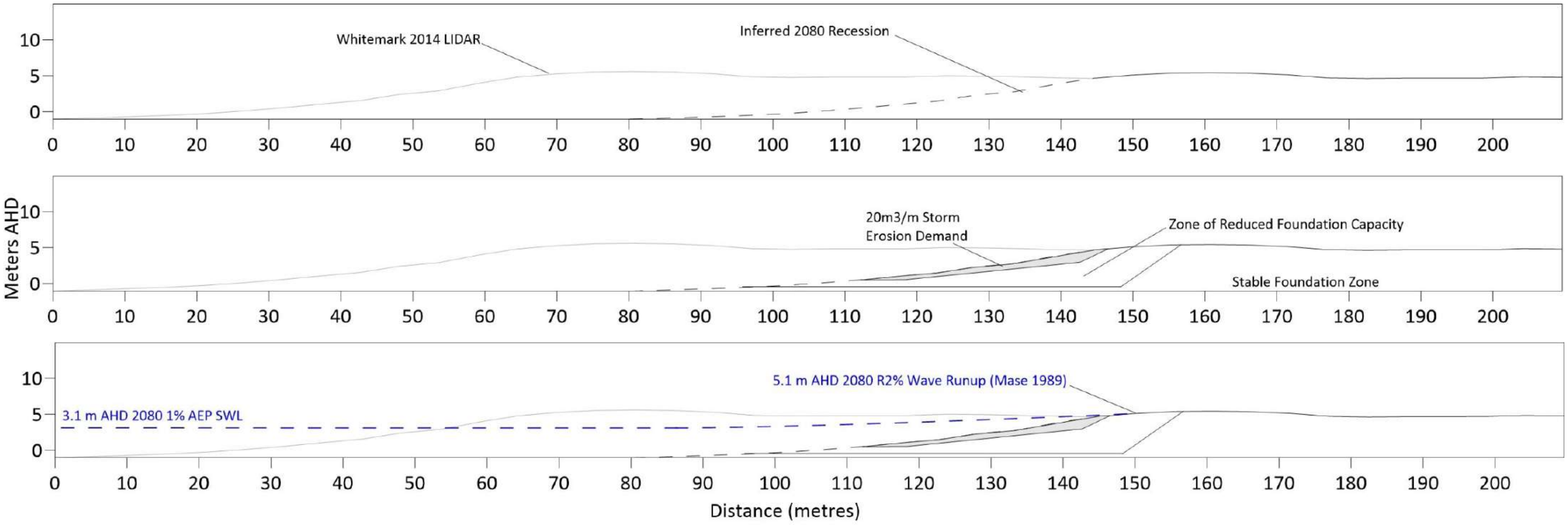


Figure 17 Site cross sections demonstrating 2080 recession, 20 m^3/m storm erosion demand, inferred inundation levels and wave runup extent at Site 1 & Site 2



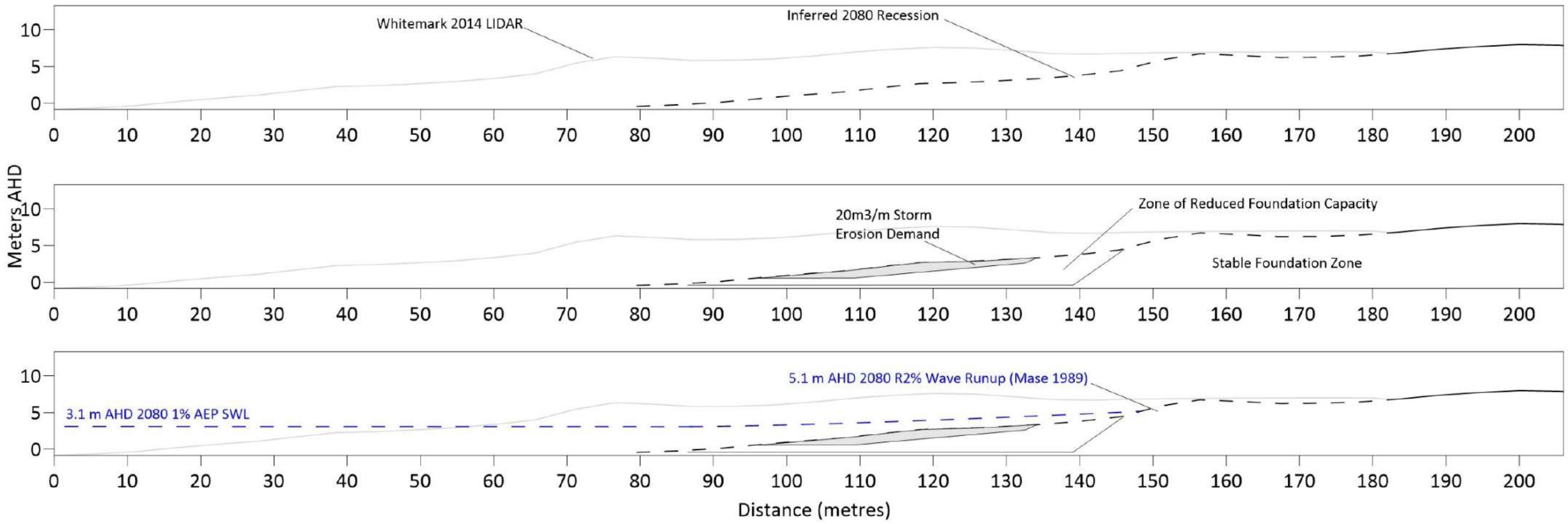


Figure 18 Site cross sections demonstrating 2080 recession, 20 m^3/m storm erosion demand, inferred inundation levels and wave runup extent at Site 4 & Site 5

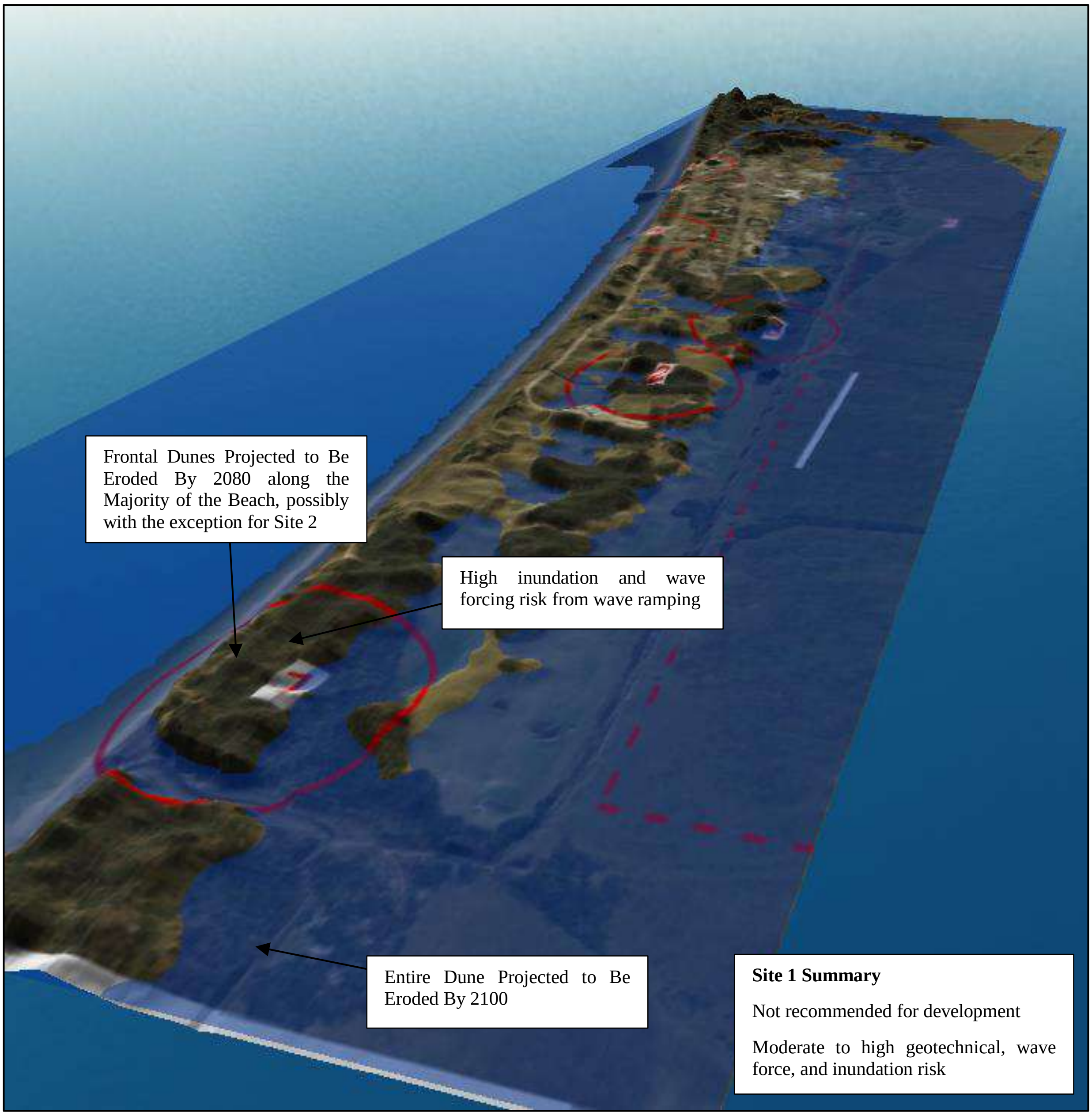


Figure 19 Site 1 Coastal Inundation & Erosion Projections For 2080

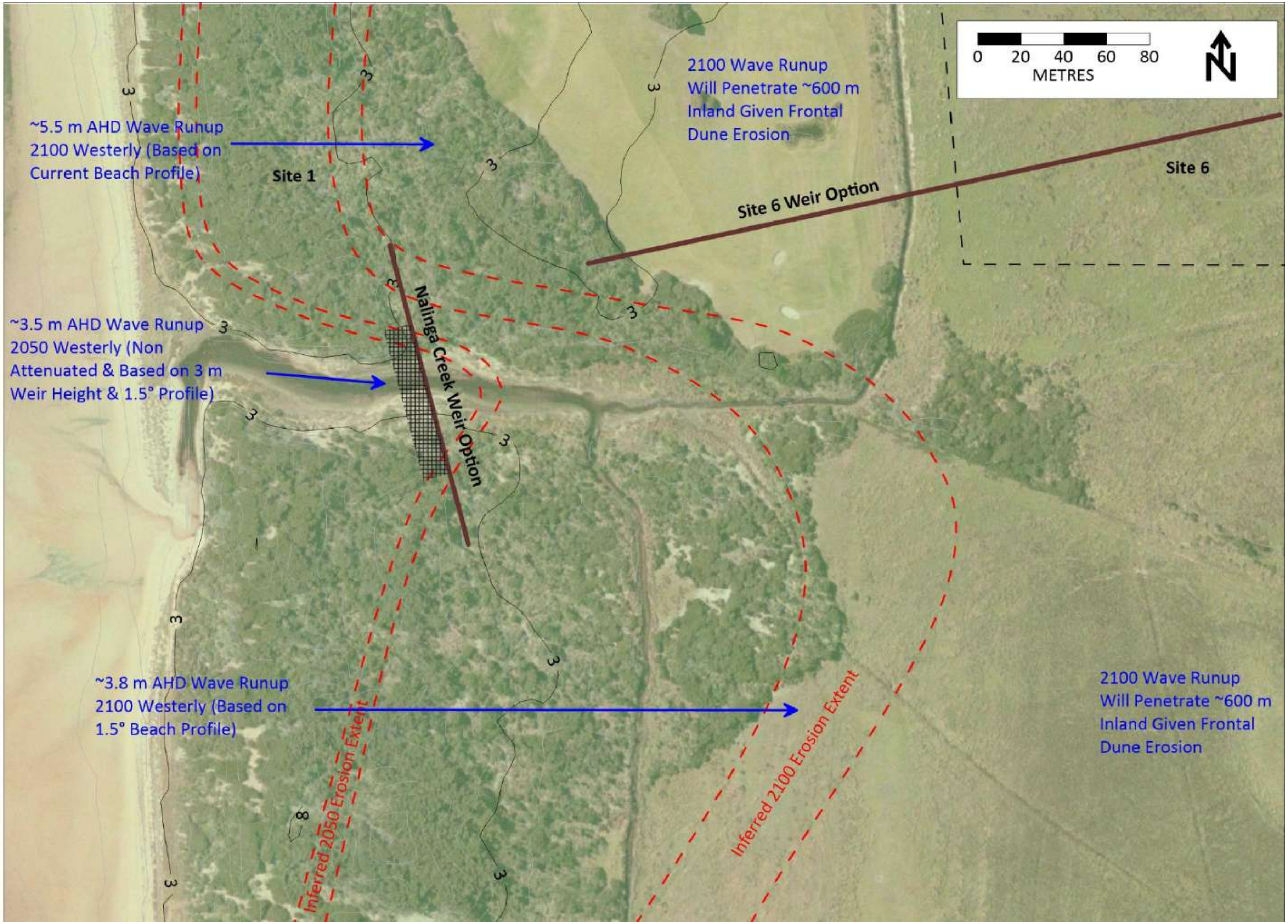


Figure 20 Projected 2050 and 2100 erosion extent, wave runup limits and Nalinga Creek closure scenarios recommended By JMG

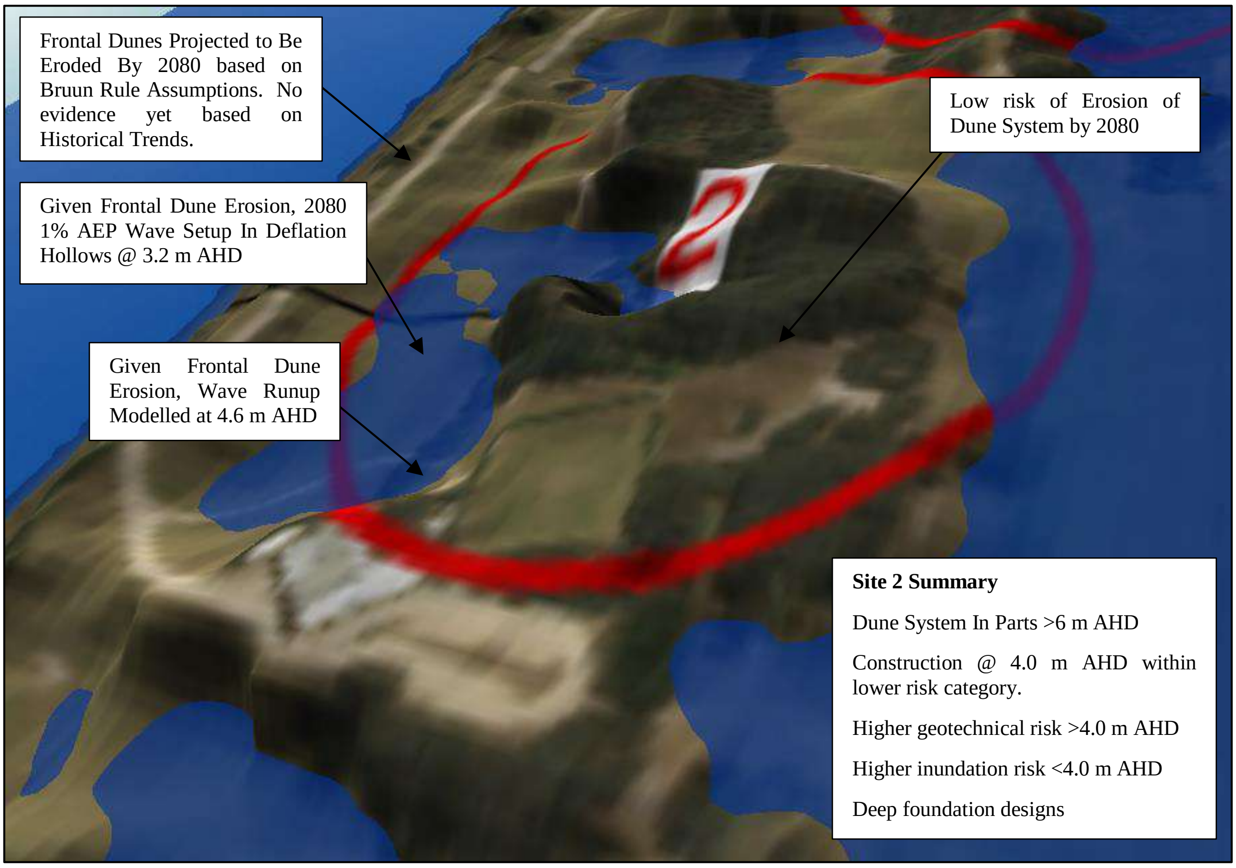


Figure 21 Site 2 inundation & erosion projections for 2080

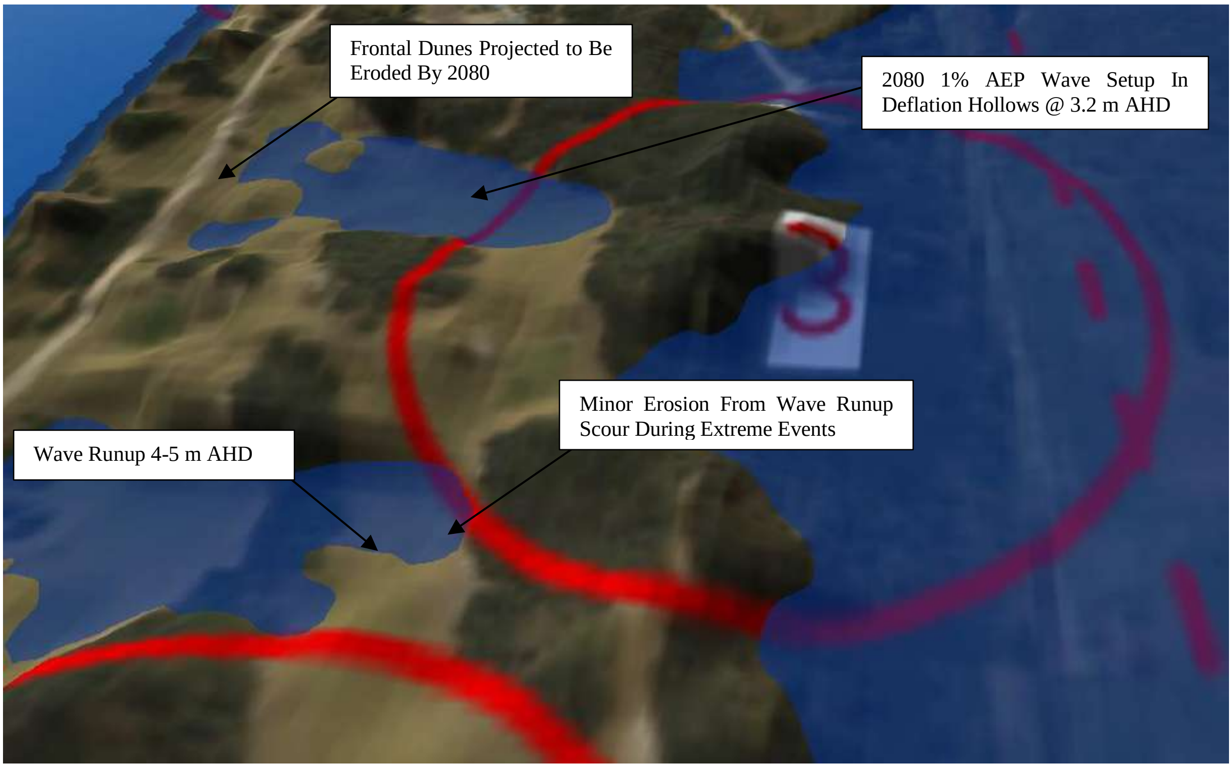


Figure 22 Site 3 inundation & erosion projections for 2080

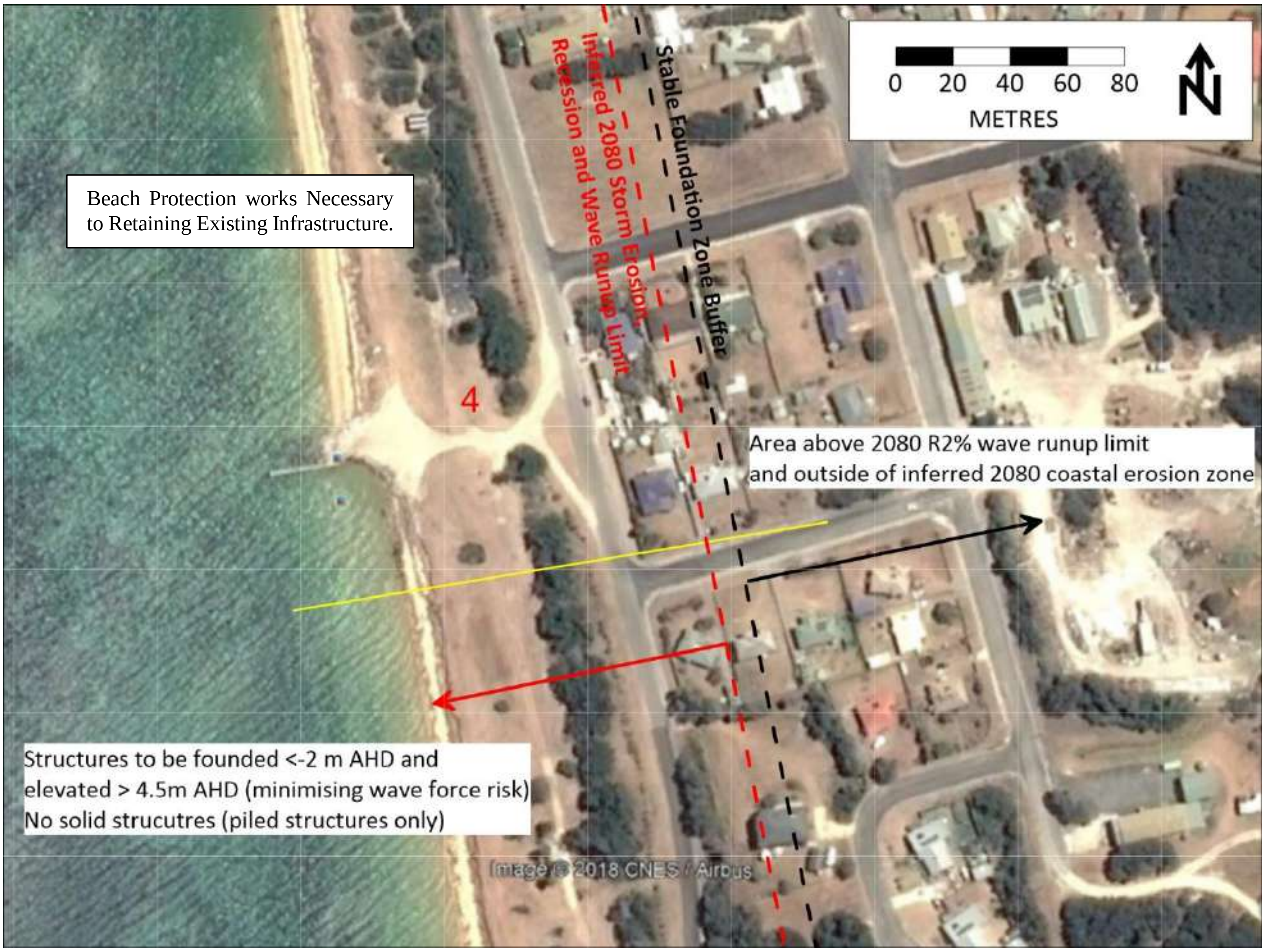


Figure 23 Site 4 inundation & erosion projections for 2080

Figure 24 Site 5 inundation & erosion projections for 2080

10 Conclusions

The following is concluded:

- Swell and wind significant wave heights have been assessed for Site 1 to Site 5, with Site 1 near Nalinga Creek being influenced by westerly swell and westerly to north-westerly wind waves. Site 5 is influenced by predominantly south-westerly, westerly and north-westerly wind waves;

- The dominant wave at Site 1 is from westerly wind waves and as Site 5 is from south-westerly wind waves. The most significant wave discerned to have longer term impact is a south-westerly wind wave acting towards Site 5. With sea level rise, waves from all modelled directions are projected to increase in height and wave energy directed towards the shoreline due to reduced wave attenuation;

- Without sediment accumulation in the nearshore zone, wave runup heights are projected to increase due to the steepening wave runup gradient (breaker depth to ultimate runup limit). Combined with an increase in sea levels over time, higher energy waves will increasingly scour the fine-grained windblown sand deposits located above present-day high-water mark (typically at 1.9m AHD). Fine grained sand deposits are noted below the high-water mark outside of the project area to the north of Whitemark and around Nalinga Creek where wave scour erosion is visibly distributing sand over sea grass beds within the near shore zone;

- The wave modelling conducted takes into consideration for the effect of the seagrass (*Posidonia australis*) communities on wave attenuation. It is noted that seagrass is discerned to be either buried or stripped from the nearshore zone to the north of Whitemark and around Nalinga Creek. Reduced wave attenuation from loss of seagrass will result in increased wave heights impacting the shoreline and more pronounced wave scour in the nearshore zone. Seagrass beds have been identified as major contributors to the marine ecosystem providing sediment stability and habitats for a range of species. The loss of seagrass is a major concern for coastal and marine managers with strong evidence that there is little if any return of *Posidonia australis* beds if destroyed beyond a certain level;

- Methods for stabilising the fine-grained sand dune deposits at and above the high-water mark need to be considered as sea levels rise and the coastline continues to recede at an increasing rate;

- Given the broad flat tidal and subtidal bathymetry and the presence of offshore islands, reef and seagrass beds, storm waves are limited in height and have a less pronounced erosive affect compared with the observed longer term coastline recession from sea level rise. Historical aerial photographs provide a reliable measure of shoreline recession in some parts of Parry's Beach. There are strong erosion correlations with sea level rise acting on the dune system immediately south of Nalinga Creek outflow where historical to present erosion rates of 377 m per metre rise in sea level are discerned based on a 95% R^2 correlation;

- A considerably lower erosion rate of 103 m per metre sea level rise is calculated for the dune escarpments to the north of Nalinga Creek discerned based on a 71% R^2 correlation due to fluctuating erosion trends;

- An overall recession trend for Whitemark Beach of 130 to 150 m per metre sea level rise is estimated based on historical photography and Bruun Rule models based on calculated and observed closure depths. GES recommend that consideration is given to a 2080 timeframe for town planning in which 90 m recession is estimated for northern limit of the project area (Site 5) and 75m recession for southern limit of the project areas (Site 1). Faster recession rates are apparent on the southern (leeward) side of the Jetty, Boat Ramp and Nalinga Creek where longshore currents are displaced;

- Apart from around the river mouth, there are no significant storm erosion events visible in historical aerial photographs, and storm erosion trends are consistent with Type 5 beach setting discerned for the North Coast of Tasmania with a storm erosion demand in the order of 20 m^3/m;

- There are no apparent erosion trends in the central beach section around Site 2 which may be due to the relatively high relief and steep beach profile comprising of coarse-grained sand well above the high-water mark and thick heavy seagrass deposits within the upper wave runup limit (where fine grained sand deposits are present at other Sites).

- Conversely, dunes around Nalinga Creek river-mouth comprise of predominately fine-grained windblown sand deposits which are particularly vulnerable to both wave runup and fluvial scour. Heavy inland flooding events combined with elevated seas (storm tide & wind setup) will allow waves to penetrate inland and scour the sides of the channel. An outflowing flood tide will cause further scour. Over time as sea levels rise, the channel will deepen from an increasing tidal prism, erosion rates will increase, and larger volumes of seawater will begin to move inland. The chance of a higher peak rainfall intensity will also increase with climates change, compounding the erosion issue;

- Based on historical trends, the dune system immediately to the south of Nalinga Creek will have largely eroded and fine sands will have been redistributed across the delta immediately offshore.

11 Recommendations

The following are recommendations for Nalinga Creek:

- Care should be taken with future infrastructure planning around Nalinga Creek. It is a particularly dynamic environment and extreme erosion rates are projected. On the other hand, management measures may be put in place to minimise erosion susceptibility;

- It is clear that erosion of fine-grained sediments around the river mouth will continue to occur at a rapid rate. The flat shoreline profile is essential to Nalinga Creek, and wave scour will continue to act at the toe of the fine-grained dune deposits. Any structures built parallel with the dune system should not be constructed any closer than 100m from the shoreline based a 2050 design life. Beyond 100m there may be insufficient dune height to levee water in; therefore, an alternative option beyond 2050 will need to be considered;

- For design purposes, any structures located within 100m of and parallel to the coast should be designed based on wave runup levels, with overtopping expected at 3.5m AHD for 2050 scenario where a structure is elevated to 3.5m AHD with no wave attenuation armouring;

- Fresh water wetlands may be established by creating a Sea Barrier across the Site 6 tributary that join to Nalinga Creek. A 600m long barrier could be constructed behind the eroding fine grained sand dune deposits. Structures designed perpendicular to the shoreline (and behind the receding dune front) should be designed based on wave setup inundation levels, with overtopping expected at 2.96m AHD for a 1% AEP 2050 scenario. Based on historical trends, without any protection works, the receding front may intercept the seaward edge of the sea barrier by 2100. Wave setup levels are projected at 3.53m AHD based on a 1% AEP 2100 scenario;

- An assessment of a sea barrier may also require an assessment of groundwater hydrology water budgeting. It may be that a wetland will not form as suggested given the particularly high hydraulic conductivity of the coarse-grained sediments discharging into the ocean. A preliminary study may require either:
    - A preliminary hydrogeology study to determine likely 'leaky-ness' of the floodplain; and/or
    - A trial earthen sea barrier study with accompanying water budgeting analysis ie. groundwater, weir, and rain gauge monitoring.

  The latter is likely to prove more economic given modelling real scenarios is more reliable than inferring based on a hypothetical model where multiple inputs and variables are required which do not provide full guarantee. Absolute conditions can be assessed on a preliminary basis from which to make decisions on future risks to the overall system.
  ie. septic tank vulnerability, salinity levels in the lagoon, likely inundation levels in Whitemark from stormflow events, appearance of lagoons within the dune swales, impact of inundation on flora/fauna etc;

- It has been observed that the dune system in the middle of the project area (Site 2) has not shown any historical recession. As a trial, the characteristic conditions at Site 2 should be replicated at an alternative Site where high erosion rates are apparent. Conditions identified at Site 2 include a steep elevated beach profile comprising of coarse-grained sand deposits well above the high tide mark. The upper shoreline is also heavily armoured with seagrass. It may be that fine-grained sand deposits are apparent above 1.5 m AHD at some locations. A trial may involve removing and replacing fine sand deposits with coarse grained material and elevating the beach face;

- Fine grained sand was backfilled around the pipe to the north of the township. This needs to be excavated out up to 5 m inland and replaced with coarse grained sand to prevent further erosion;

- A coastal development buffer of 90 m (Site 5) through to 75 m (Site 1) should be considered for any town planning. Building within this erosion zone must require specific engineering design to ensure the structures are firmly seated below wave scour levels. These methods involve costly construction methods and long-term servicing issues. Buildings constructed on the coast outside of this erosion zone need to consider wave run-up levels. Developments are not recommended on the eastern side of Whitemark township until a more conclusive flood study is conducted.

Kris J Taylor BSc (Hons)

Environmental & Engineering Geologist

References

ABCB 2014. Resilience of Buildings to Extreme Weather Events. Final Paper. Australian Building Code Board 2015.

ABCB 2015. Durability in Buildings Including Plumbing Installations. Second Edition. Handbook. Non-Mandatory Document. Australian Building Code Board 2015.

AMS 2007. American Meteorological Society Glossary of Meteorology. Retrieved 2007-06-30. Antarctic Climate & Ecosystems Cooperative Research Centre (ACE CRC), 2010.

AS 1170.2:2011. Australian and New Zealand Standard. Structural Design Actions. Part 2: Wind Actions.

Australian Bureau of Meteorology (2007). (BOM) Meteorological Averages. Weather Station Data; http://www.bom.gov.au/climate/data/weather-data.shtml, accessed September 2010

Bradley, K., and Houser, C., 2009. Relative velocity of seagrass blades: Implications for wave attenuation in low-energy environments. Fournal of Geophysical Research. Vol 114. F01004 2009.

Bruun, P., 1988, "The Bruun Rule of Erosion by Sea Level Rise: A Discussion on Large Scale Two- and Three-Dimensional Usages", Journal of Coastal Research, 4(4), 627-648.

CARLEY, J.T., BLACKA, M.J., TIMMS, W.A., ANDERSEN, M.S., MARIANI, A., RAYNER, D.S., McARTHUR, J. & COX, R.J., 2008: Coastal Processes, Coastal Hazards, Climate Change and Adaptive Responses for Preparation of a Coastal Management Strategy for Clarence City, Tasmania; Technical Report 2008/04, Water Research Laboratory, University of New South Wales, November 2008.

CAWCR 2013 (Durrant, Thomas; Hemer, Mark; Trenham, Claire; Greenslade, Diana). The Centre for Australian Weather and Climate Research (CAWCR) Wave Hindcast 1979-2010. (2013).

CEM. United States (Coastal Engineering Model) 2008, EM 1110-2-1100, 2008.

Church, J. A. and White, N. J., 2011, 'Sea-level rise from the late 19th to the early 21st century',Surveys in Geophysics 32(4–5), 585–602

Cowell, P.J., Thom, B.G., Jones, R.A., Everts C.H., Simanovic, D., 2006. Management of Uncertainty in Predicting Climate Change Impact on Beaches. Journal of Coastal Research, 22(1), 232-245. West Palm Beach (Florida), ISSN 0749-0208

CSIRO (Commonwealth Scientific and Industrial Organisation) 2012, Sea level rise: understanding the past, improving projections for the future.

Davies, J.L., 1959: Sea Level Change and Shoreline Development in South-Eastern Tasmania; Papers and Proceedings of the Royal Society of Tasmania, Vol. 93, p. 89 – 95.

Davies, J.L., 1961: Tasmanian Beach Ridge Systems in Relation to Sea Level Change; Papers and Proceedings of the Royal Society of Tasmania, Vol. 95, p. 35 – 40.

Davies, J.L., 1978: Beach Sand and Wave Energy in Tasmania; in: J.L. Davies & M.A.J. Williams (Eds), *Landform Evolution in Australasia*, ANU Press, Canberra, p. 158-167.

DCC (Department of Climate Change) 2009, Climate Change Risks to Australia's Coasts, A First Pass National Assessment.

DPAC 2016. Department of Premier and Cabinet. Mitigating Natural Hazards through Land Use Planning and Building Control.

Dean, R.G. & Darymple, R.A. 1991. WATER WAVE MECHANICS FOR ENGINEERS AND SCIENTISTS. Advanced Series on Ocean Engineering — Volume 2. Published by World Scientific Publishing Co. Pte. Ltd. 5 Toh Tuck Link, Singapore 596224

Dean, R.G. & Darymple, R.A. 2002: *Coastal Processes with Engineering Applications*; Cambridge University Press, UK.

Dickson, M.E., Walkden, M.J.A. and Hall, J.W., 2007. Systematic impacts of climate change on an eroding coastal region over the twenty-first century. Climatic Change, in press.

DPIPWE, 2008. Sea-Level Extremes in Tasmania, Summary and Practical Guide for Planners and Managers.

DPIWE, 2008, Coastal Hazards. In Tasmania General Information Paper, DPIWE Tasmania Page

Estimating Sea Level Rise in an Uncertain Future. Sea Level rise extremes assessment Web Tool. web tool www.slr.sealevelrise.info accessed on September 2010.

Gill, E.D., 1968. Aboriginal bone implement from fossil bone bed, Tasmania. Rec Q. Vict. Mus. Launceston.

http://www.climatechange.gov.au/publications/coastline/climate-change-risks-to-australias-coasts.aspx. Accessed September 2010.

Hunter, J. 2008, Historical and Projected Sea-Levels Extremes for Hobart and Burnie, Tasmania, Technical Report prepared by the Antarctic and Climate and Ecosystems Cooperative Research Centre – December 2007. Published by the Department of Primary Industries and Water, Tasmania.

Hunter, J., 2010. Estimating Sea-Level Extremes Under Conditions of Uncertain Sea-Level Rise, Climatic Change, 99:331-350, DOI:10.1007/s10584-009-9671-6.

IPCC (Intergovernmental Panel on Climate Change) 2007, Climate Change – The Physical Science Basis. Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change, (ISBN 978 0521 88009-1 Hardback; 978 0521 70596-7 Paperback), [Solomon, S., D. Qin, M. Manning, Z. Chen, M. Marquis, K.B. Averyt, M. Tignor and H.L. Miller (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA, 996 pp. 2007

IPCC (Intergovernmental Panel on Climate Change) 2013, Climate Change 2013: The physical Science Basis. Contribution of Working Group I to the Fifth Assesssment Report of the Intergovernmental Panel on Climate Change (Stocker, T.F., D. Qin, G.K. Plattner, M. Tignor, S.K. Allen, J. Boschung, A. Nauels, Y. Xia, V. Bex and P.M. Midgley (eds). Cambridge University Press, Cambridge, United Kingdom and New York, USA.

Kamphuis, J.W., 2006. Introduction to Cawstal Engineering and Management. Advanced Series on Ocean Engineering – Volume 16. World Scientific. Queens University Canada.

Kobayashi, N., Raichle A.W., and Asano, T., 1993. Journal of Waterway, Port, Coastal and Ocean Engineering, Vol 119, No 1.

Komar, P.D., 1998. Beach Processes and Sedimentation. Second Edition. Colege of Oceanic and Atmospheric Sciences Oregon State University. Prentice Hall. Upper Saddle River, New Jersey 07458.

Kulmar, M., D.Lord & B.Sanderson, 2005. “Future Directions For Wave Data Collection In New South Wales”, Proceedings of Australasian Coasts and Ports conference, Adelaide, The Institute of Engineers Australia.

Lord, D.B. and M. Kulmar, 2000. “The 1974 storms reviSited: 25 years’ experience in Ocean Wave Measurement along the South East Australian Coast”, Proceedings International Conference of Coastal Engineering, pp 559-572, American Society of Civil Engineers, USA.

McInnes, K L, O’Grady, J G, Hemer, M, Macadam, I, Abbs, D J, White, C J, Corney, S P, Grose, M R, Holz, G K, Gaynor, S M and Bindoff, N L, 2011 Climate Futures for Tasmania: extreme tide and sea-level events technical report, Antarctic Climate and Ecosystems Cooperative Research Centre, Hobart.

McInnes Kl, Monselesan D, O’Grady JG, Church JA and Xhang, X, 2016: sea-Level Rise and Allowances for Tasmania based on the IPCC AR5, CSIRO Report 33pp.

McInnes and O’Grady, 2016: Tasmanian Extreme Sea Level Modelling Assessment, CSIRO Report 20 pp

Mase, H. (1989), ‘Random Wave Runup Height on Gentle Slopes’, Journal of the Waterway, Port, Coastal and Ocean Engineering Division, American Society of Civil Engineers, pp 593-609

Merani,A., Shand, T.D., Carley J.T., Goodwin, I.D., Splinter, K., Davey, E.K., Flocard, and Turner I.L., 2012. Technical Report: Generic Design Coastal Erosion Volumes and Setbacks for Australia. Water Research Laboratories, University of New South Wales. School of Civil and Environmental Engineering. Antarctic Climate & Ecosystems Cooperative Research Centre, Hobart Tasmania.

Moller I., Spencer T., French J.R., Leggett D.J., and Dixon M.. 1999. Wave transformation over salt marshes: A field and numerical modelling study from north Norfolk, England. Estuarine, Coastal and Shelf Science 49, 411-426.

NCCOE, (National Committee on Coastal and Ocean Engineering, Engineers Australia) 2004, Guidelines for responding to the effects of Climate Change in coastal and Ocean Engineering, The Institution of Engineers Australia.

NSW Coastal Risk Management Guide, Incorporating sea level rise benchmarks in coastal risk assessments, August 2010, Department of Environment, Climate Change and Water, New South Wales, Australia.

Nielsen, A.F., D.B.Lord & H.G.Poulos, 1992. Dune Stability Considerations for Building Foundations. Engineers Australia, Vol CE34, No 2, June.

Nielsen, P., 2012. Costal and Estuarine Processes. Advanced Series on Ocean Engineering Volume 29. World Scientific.

Pilkey,O.H and J.A.G. Cooper, 204. “Society and Sea Level Rise”, Science, 303, pp1781-1782.

Pugh, D.T. (1987), Tides, Surges and MeanSea-Level, John Wiley and Sons, Chichester,UK.

Ranasinghe, Roshanka, Phil Watson, Doug Lord, David Hanslow and Peter Cowell, 2007. “Sea Level Rise, Coastal Recession and the Bruun Rule”, Proceedings of Australasian Coasts and Ports Conference, Melbourne, The Institute of Engineers Australia.

Rees, C.G., 1993. Tasmanian Seagrass Communities. Submitted in partial fulfilment of the requirement for the degree of Master of Environmental Studies (By Coursework). Centre for Environmental Studies University of Tasmania

Sharples, C, Walford, H and Roberts, L, 2013, Coastal erosion susceptibility zone mapping for hazard band definition in Tasmania, Department of Premier and Cabinet, Tasmania.

Sharples & Woodward 2013. Geomorphology background to Coastal Erosion Hazard Zoning for Tasmania. Tasmanian Government, Smartline, Bluewren Group, University of Tasmania.

Sharples, C. 2006. Indicative Mapping of Tasmanian Coastal Vulnerability to Climate Change and Sea Level Rise: Explanatory Report; 2nd Edition. Consultant Report to Department of Primary Industries & Water, Tasmania. http:/www.dpiw.tas.gov.au/climatechange.

Sharples, C., Mount, R., Pedersen, T., 2009. THE AUSTRALIAN COASTAL SMARTLINE GEOMORPHIC AND STABILITY MAP VERSION 1: MANUAL AND DATA DICTIONARY. School of Geography & Environmental Studies, University of Tasmania . Manual version 1.1

Shore Protection Manual. 1984. 4th ed., 2 Vol., U.S. Army Engineer Waterways Experiment Station, U.S. Government Printing Office, Washington, D.C., 1,088 p.

Soulsby, R., Dynamics of Marin Sands. Thomas Telford Books. Distributed by the US American Society of Civil Engineers

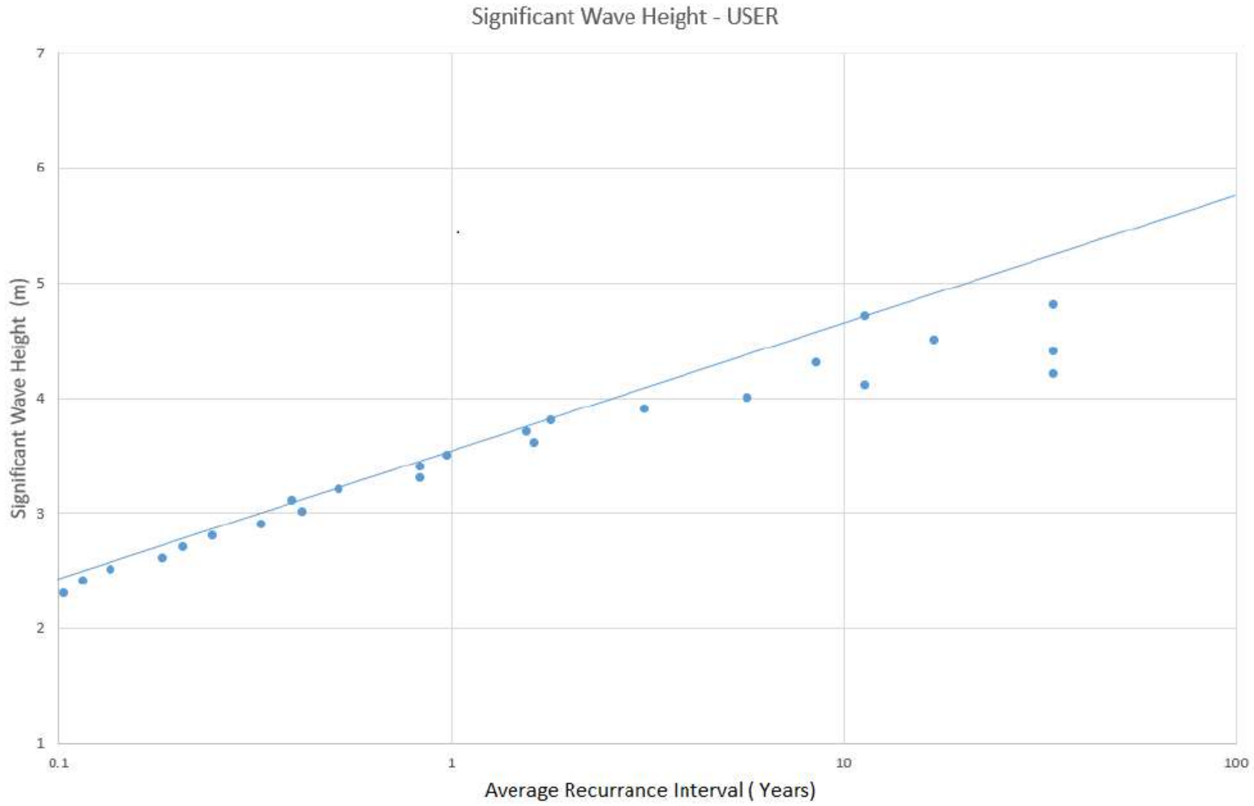
SPM (Shore Protection Manual) 1984, 4th ed., 2 Vol., U.S. Army Engineer Waterways Experiment Station, U.S. Government Printing Office, Washington, D.C., 1,088 p.

TCCO (Tasmanian Climate Change Office) 2012, Derivation of the Tasmanian Sea Level Rise Planning Allowances. Technical Paper

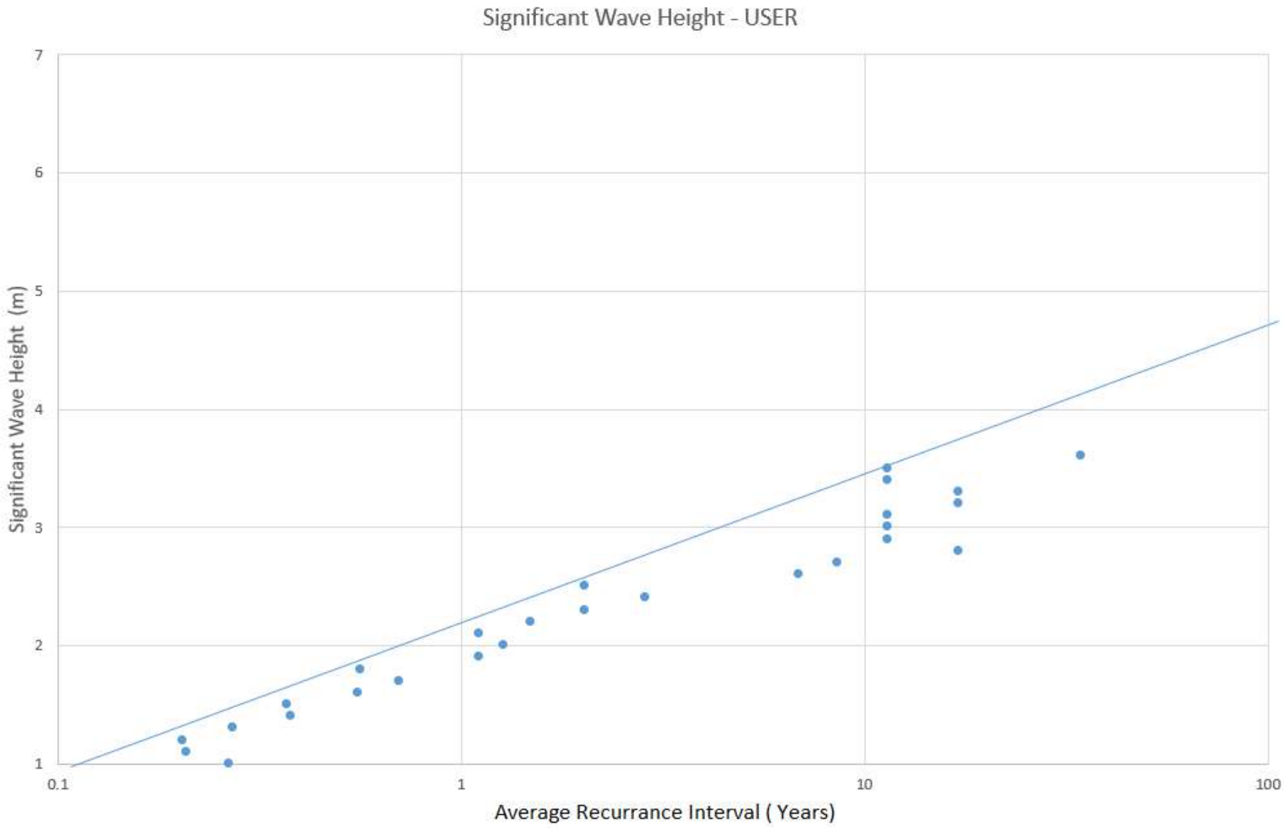
USGS (United States Geological Survey) 2003. Glossary of Coastal Terminology, US Coastal and Marine Geology. Washington Department of Ecology.

Appendix 1 Wave Model Data

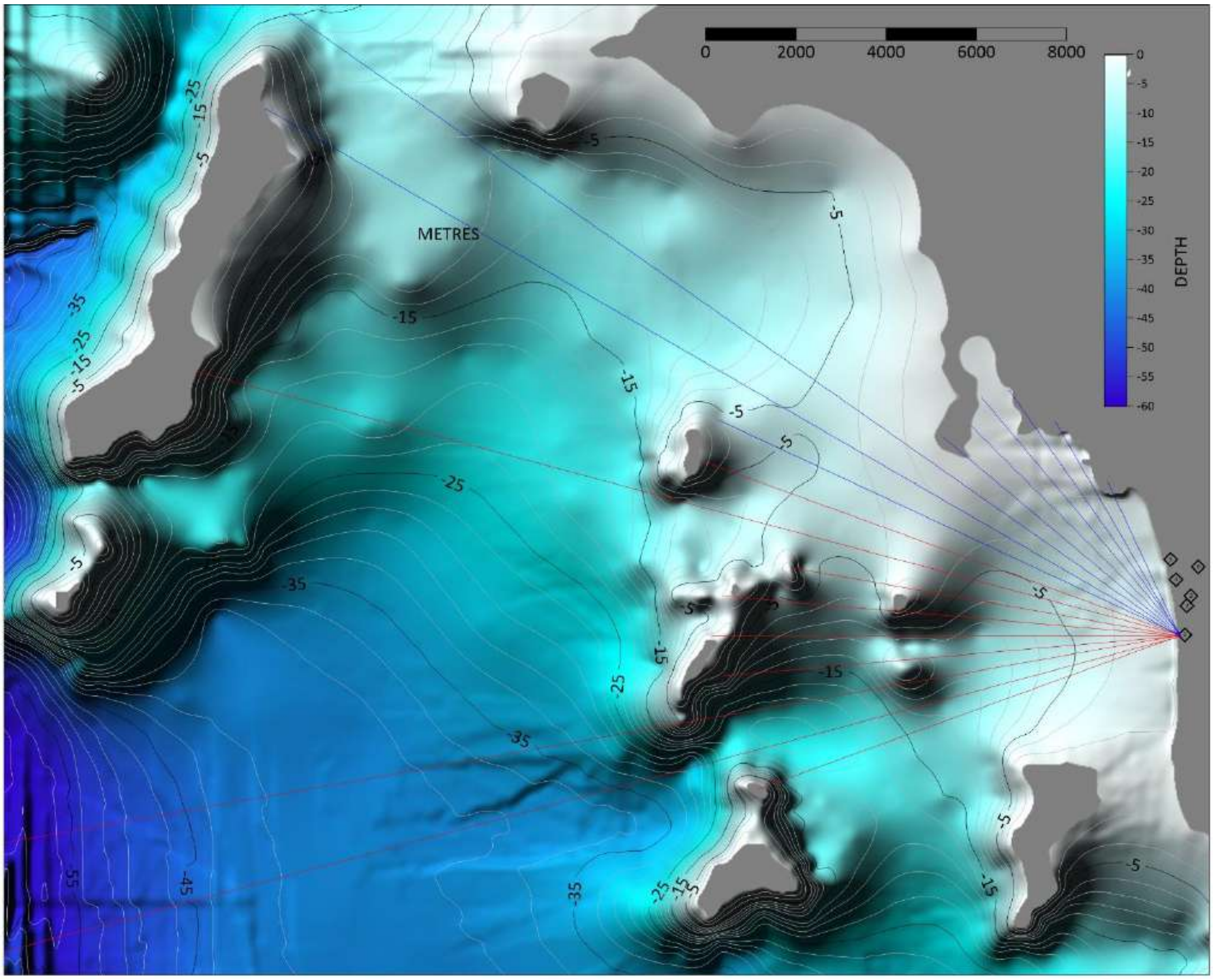
Site 1 Significant Wave Heights



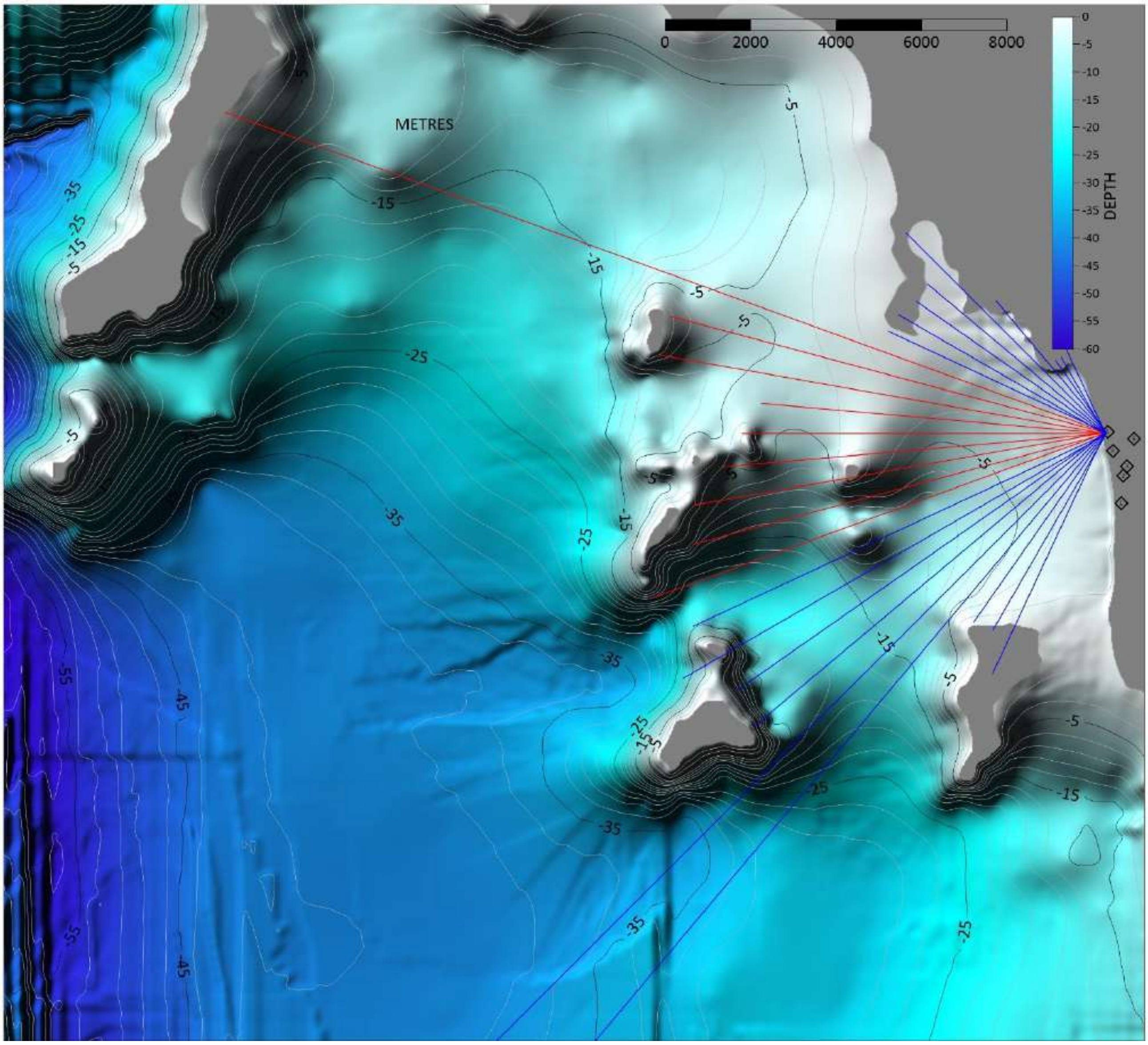
Site 5 Significant Wave Heights



Site 1 Radials For Calculating Wind Waves



Site 5 Radials For Calculating Wind Waves



2018 Localised Wind Wave Model for Site 1	SW	W	NW
Duration Multiplier Duration Relative to 1 hr		1.05	1.07
Time (s)		535	334
Wind Direction Multiplier		0.90	1.00
Design Wind Velocity Result (m/s)		25.6	29.1
Fetch Average for 2018 (km)		13.7	9.7
Bathymetry Average for 2018 (m)		27.7	11.2
Significant Wave Height (m)		2.20	1.98
Wave Period (s)		4.70	4.31

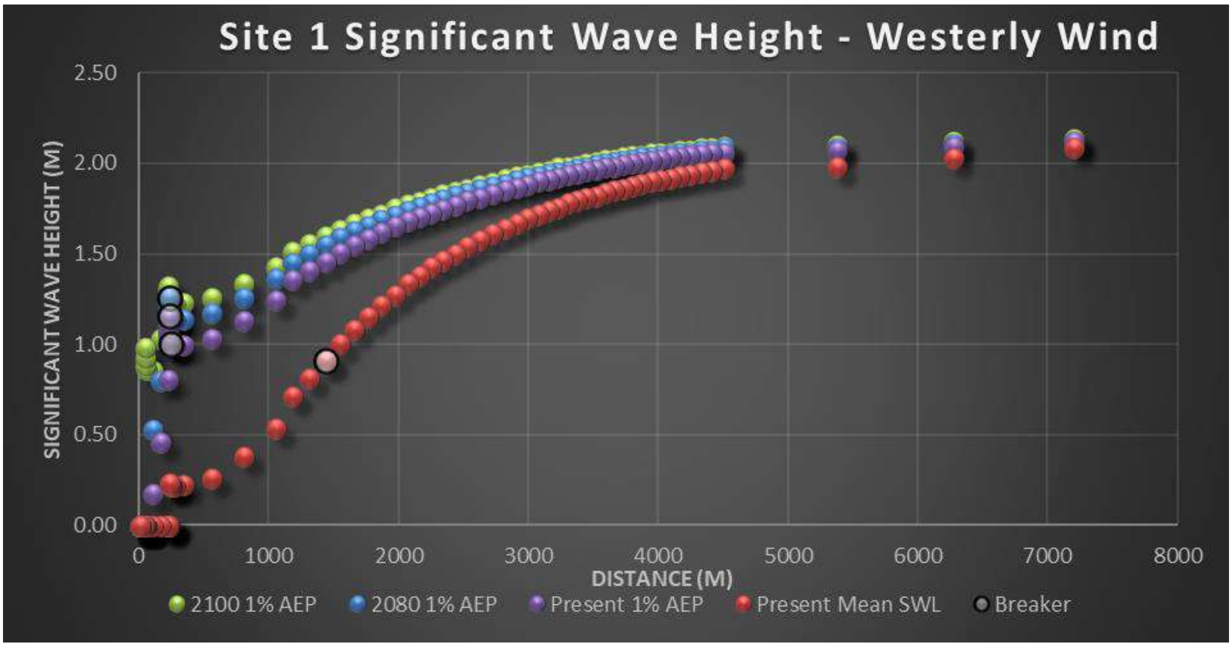
2080 Localised Wind Wave Model for Site 1	SW	W	NW
Duration Multiplier Duration Relative to 1 hr		1.05	1.07
Time (s)		535	334
Wind Direction Multiplier		0.90	1.00
Design Wind Velocity Result (m/s)		25.6	29.1
Fetch Average for 2018 (km)		13.7	9.7
Bathymetry Average for 2018 (m)		28.3	11.7
Significant Wave Height (m)		2.20	2.00
Wave Period (s)		4.71	4.33

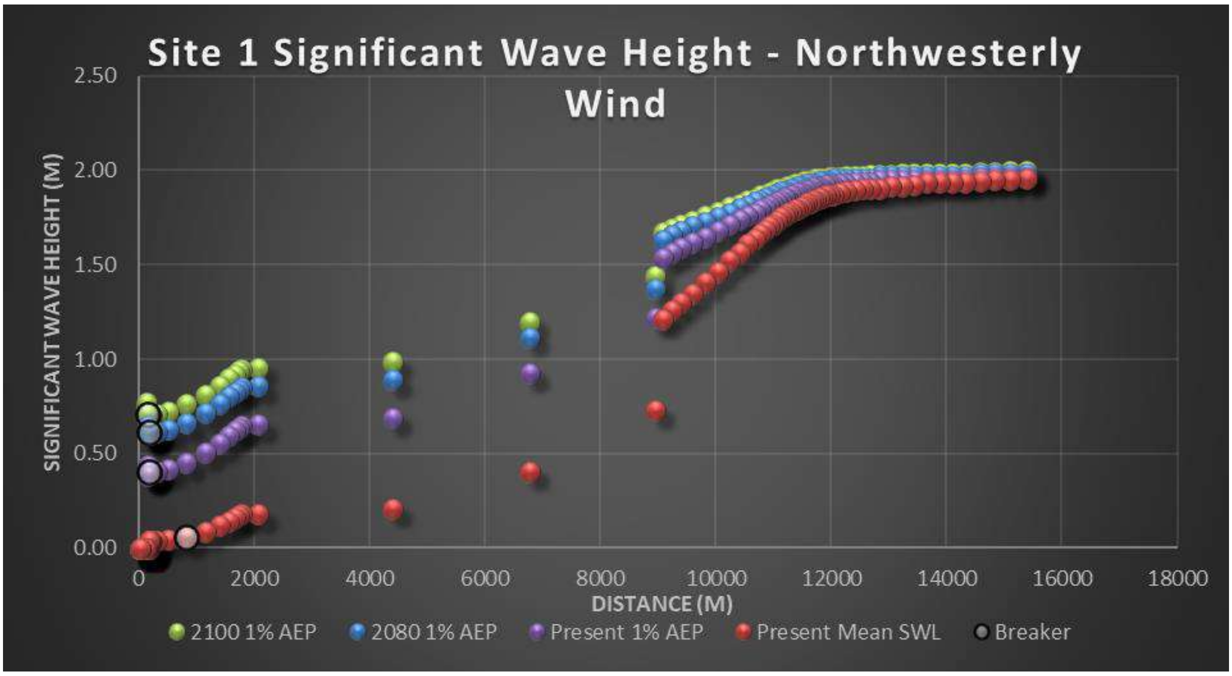
2100 Localised Wind Wave Model for Site 1	SW	W	NW
Duration Multiplier Duration Relative to 1 hr		1.05	1.07
Time (s)		535	334
Wind Direction Multiplier		0.90	1.00
Design Wind Velocity Result (m/s)		25.6	29.1
Fetch Average for 2018 (km)		13.7	9.7
Bathymetry Average for 2018 (m)		28.6	12.1
Significant Wave Height (m)		2.20	2.01
Wave Period (s)		4.71	4.33

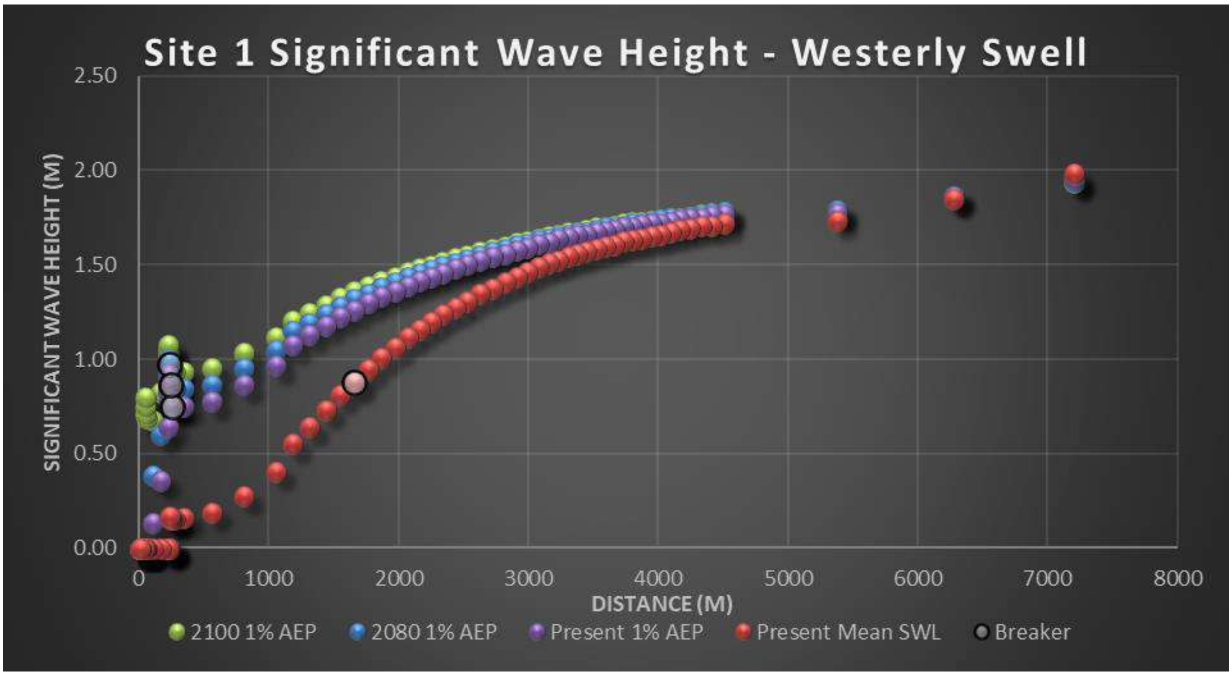
2018 Localised Wind Wave Model for Site 5	SW	W	NW
Duration Multiplier Duration Relative to 1 hr	1.05	1.07	1.12
Time (s)	410	381	127
Wind Direction Multiplier	0.85	0.90	1.00
Design Wind Velocity Result (m/s)	24.2	26.1	30.4
Fetch Average for 2018 (km)	9.9	10.0	3.9
Bathymetry Average for 2018 (m)	23.7	10.2	3.1
Significant Wave Height (m)	1.75	1.76	1.10
Wave Period (s)	4.15	4.14	3.16

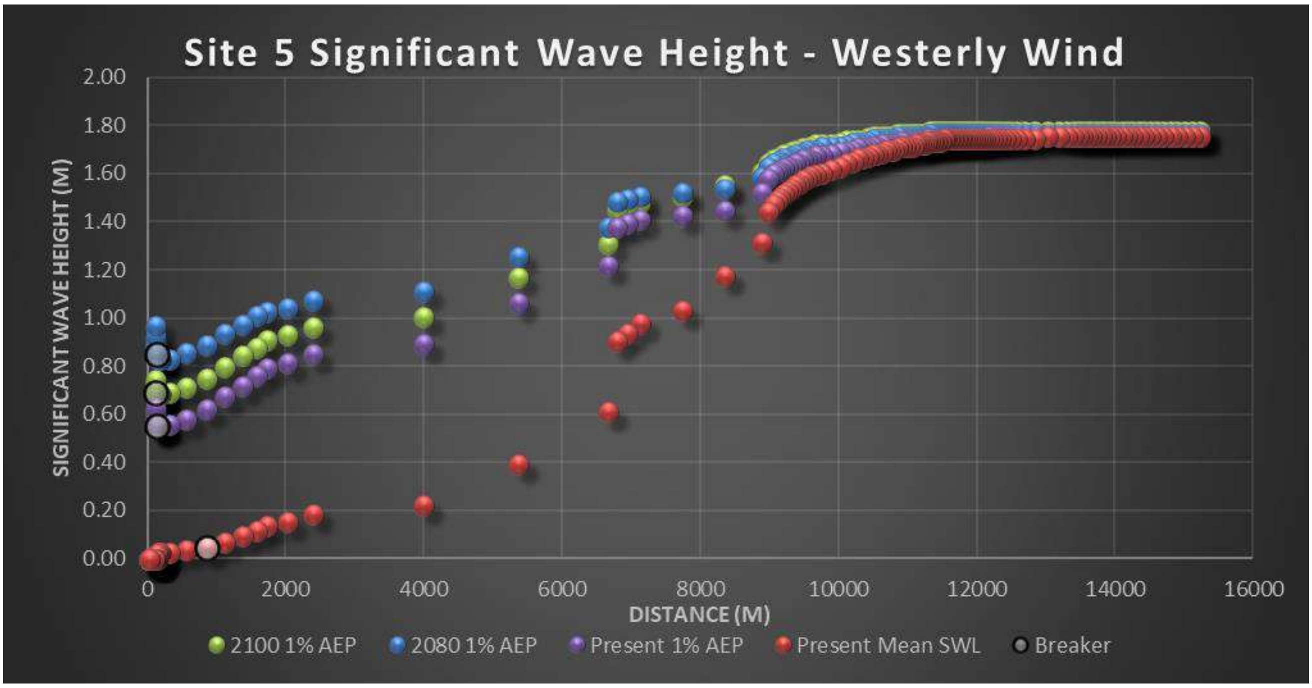
2080 Localised Wind Wave Model for Site 5	SW	W	NW
Duration Multiplier Duration Relative to 1 hr	1.05	1.07	1.12
Time (s)	410	381	127
Wind Direction Multiplier	0.85	0.90	1.00
Design Wind Velocity Result (m/s)	24.2	26.1	30.4
Fetch Average for 2018 (km)	9.9	10.0	3.9
Bathymetry Average for 2018 (m)	24.2	10.8	3.7
Significant Wave Height (m)	1.75	1.77	1.16
Wave Period (s)	4.15	4.15	3.19

2100 Localised Wind Wave Model for Site 5	SW	W	NW
Duration Multiplier Duration Relative to 1 hr	1.05	1.07	1.12
Time (s)	410	381	127
Wind Direction Multiplier	0.85	0.90	1.00
Design Wind Velocity Result (m/s)	24.2	26.1	30.4
Fetch Average for 2018 (km)	9.9	10.0	3.9
Bathymetry Average for 2018 (m)	24.6	11.2	4.1
Significant Wave Height (m)	1.75	1.78	1.20
Wave Period (s)	4.15	4.16	3.21

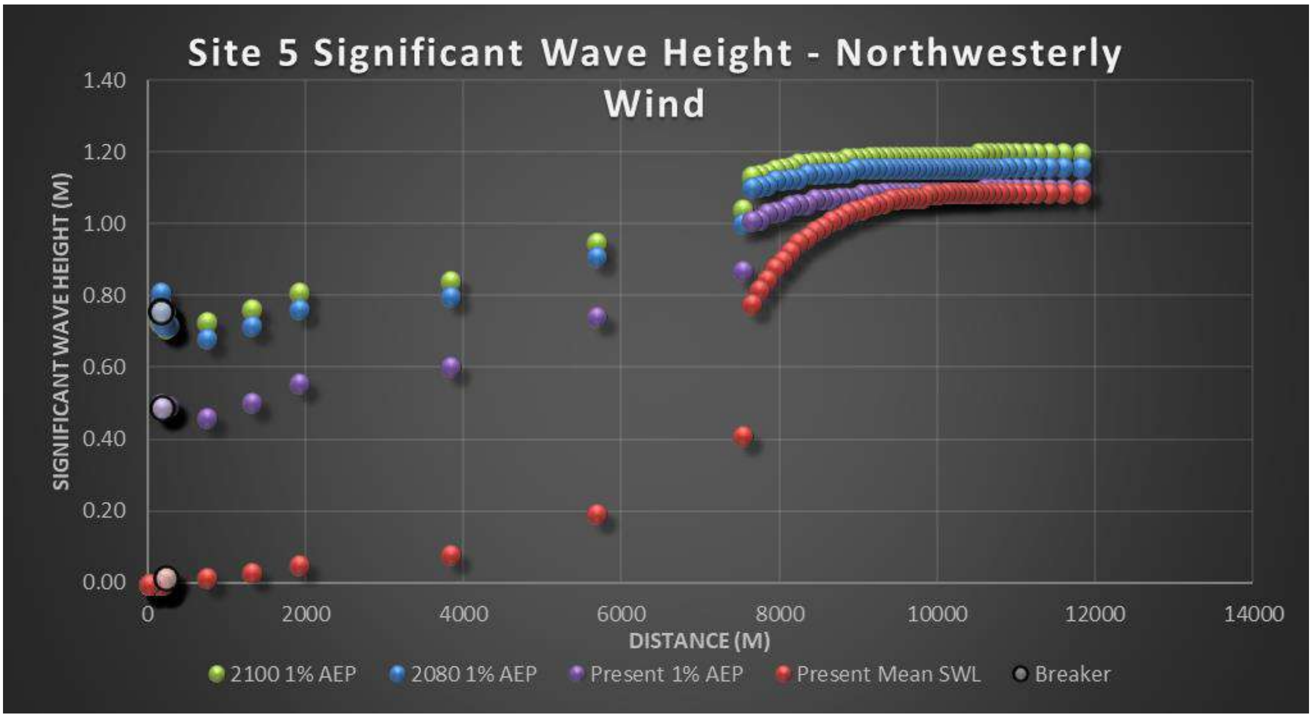

Site 1 Significant Wave Height - Westerly Wind
SIGNIFICANT WAVE HEIGHT (M)
2.50
2.00
1.50
1.00
0.50
0.00
0
1000
2000
3000
4000
5000
6000
7000
8000
DISTANCE (M)
2100 1% AEP
2080 1% AEP
Present 1% AEP
Present Mean SWL
Breaker


Site 1 Significant Wave Height - Northwesterly Wind
SIGNIFICANT WAVE HEIGHT (M)
2.50
2.00
1.50
1.00
0.50
0.00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
DISTANCE (M)
2100 1% AEP
2080 1% AEP
Present 1% AEP
Present Mean SWL
Breaker

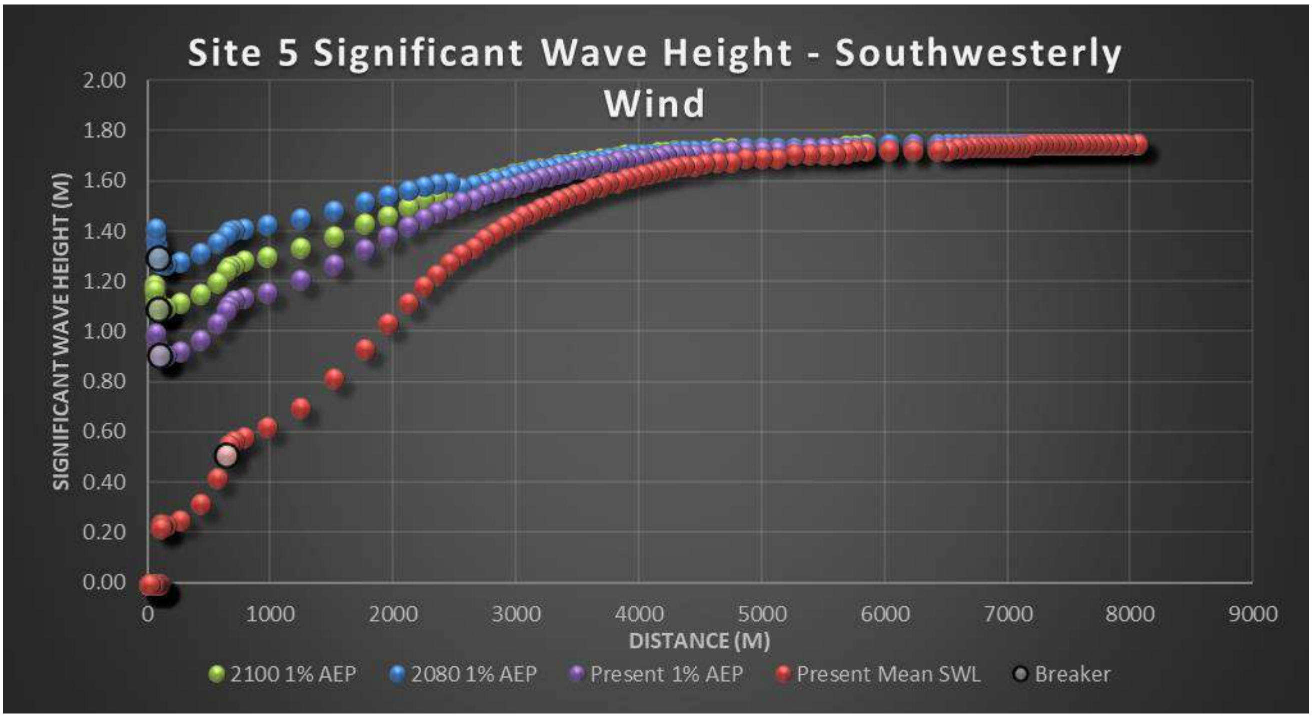

Site 1 Significant Wave Height - Westerly Swell
SIGNIFICANT WAVE HEIGHT (M)
2.50
2.00
1.50
1.00
0.50
0.00
0
1000
2000
3000
4000
5000
6000
7000
8000
DISTANCE (M)
2100 1% AEP
2080 1% AEP
Present 1% AEP
Present Mean SWL
Breaker

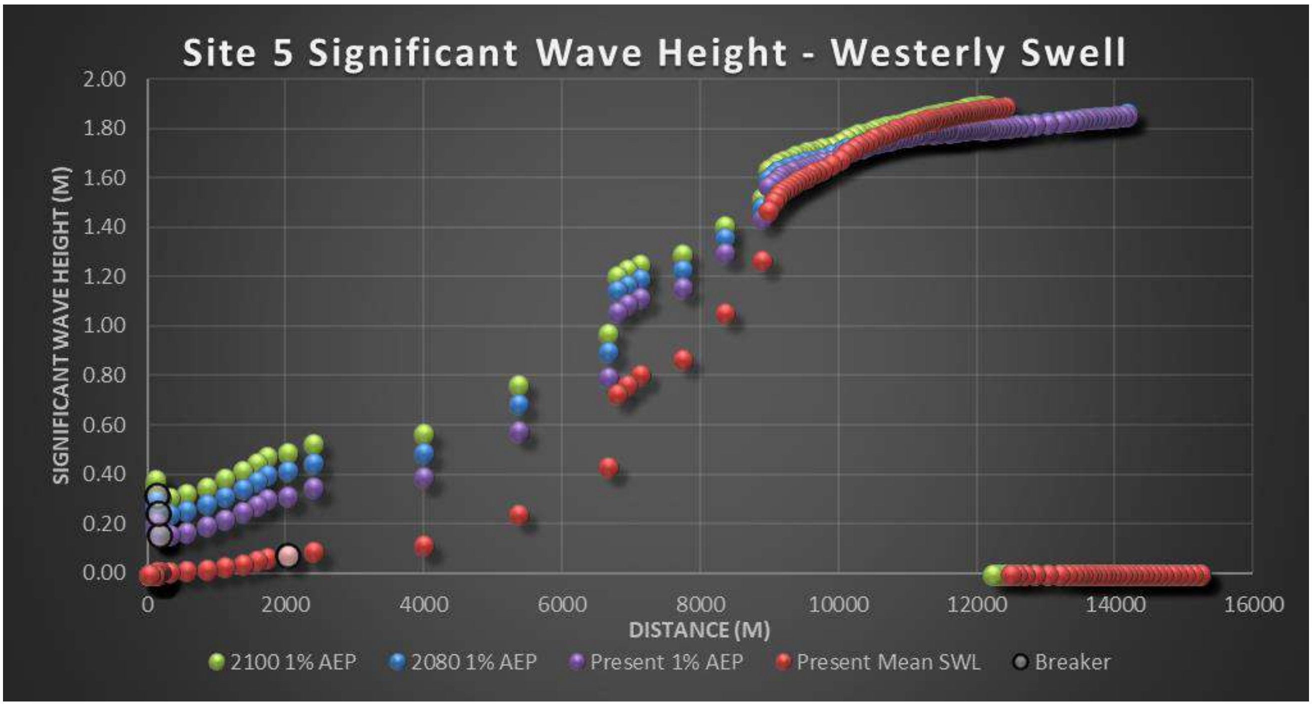


Site 5 Significant Wave Height - Westerly Wind
SIGNIFICANT WAVE HEIGHT (M)
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0
2000
4000
6000
8000
10000
12000
14000
16000
DISTANCE (M)
2100 1% AEP
2080 1% AEP
Present 1% AEP
Present Mean SWL
Breaker

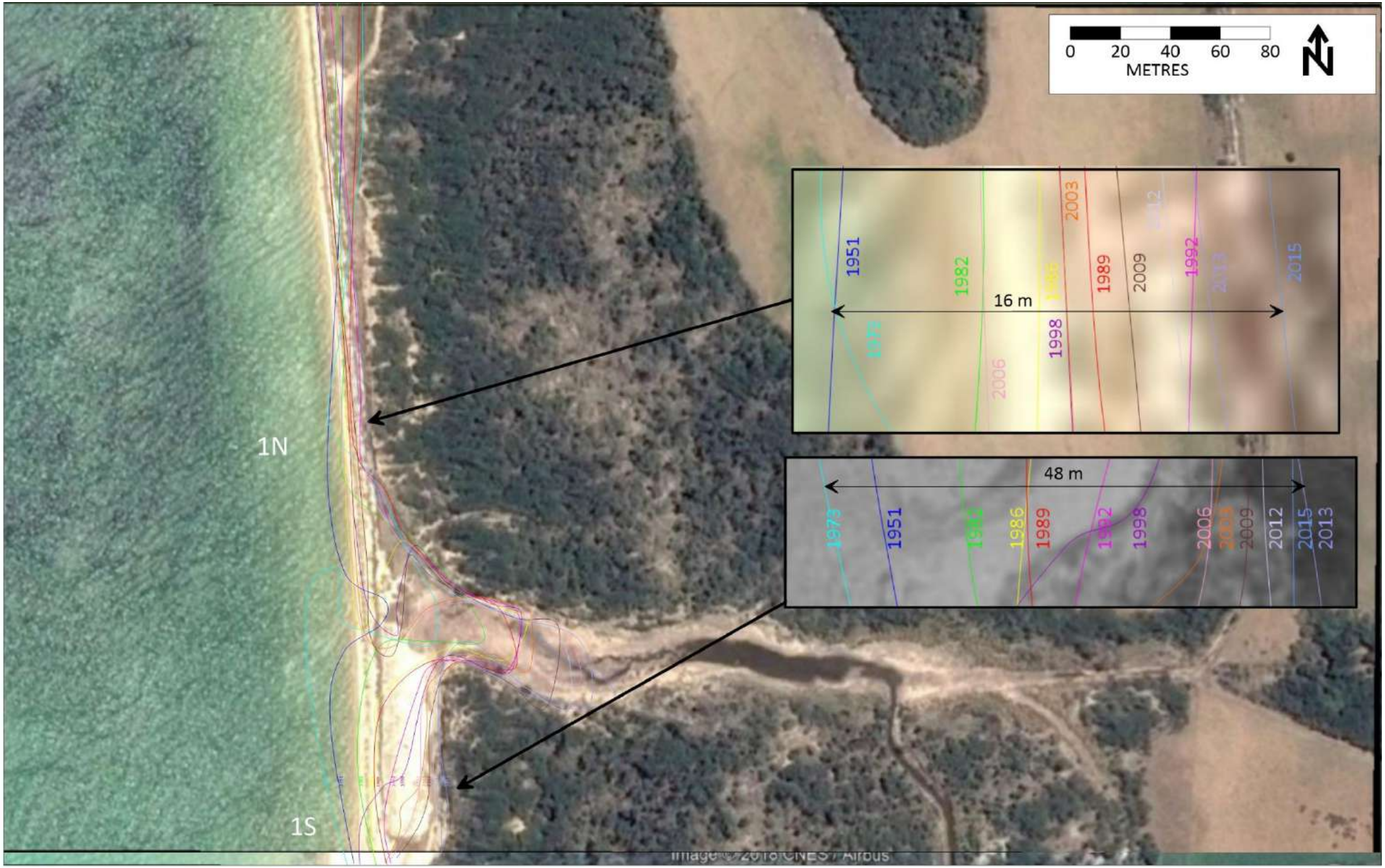


Site 5 Significant Wave Height - Northwesterly Wind
SIGNIFICANT WAVE HEIGHT (M)
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0
2000
4000
6000
8000
10000
12000
14000
DISTANCE (M)
2100 1% AEP
2080 1% AEP
Present 1% AEP
Present Mean SWL
Breaker

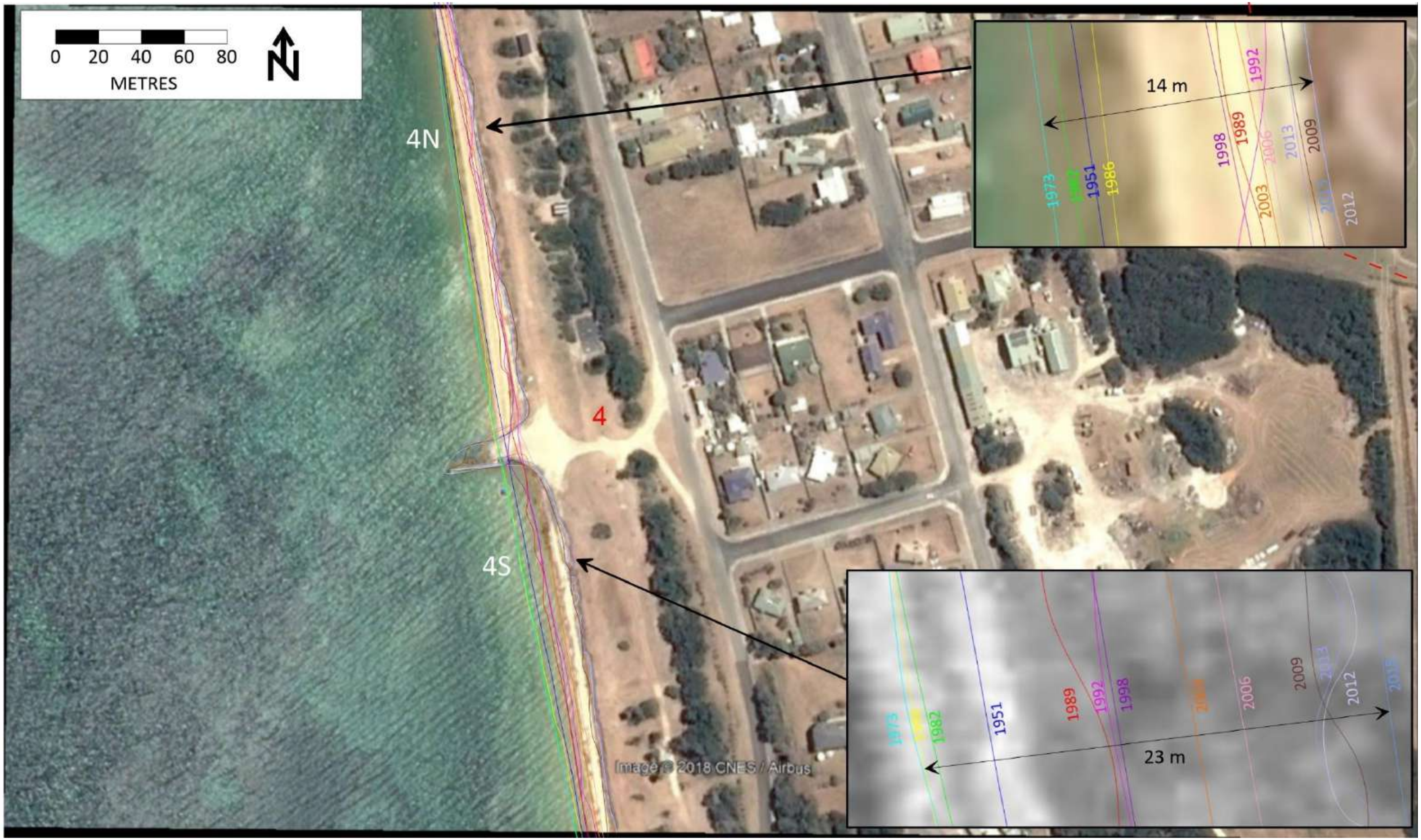

Site 5 Significant Wave Height - Southwesterly Wind
SIGNIFICANT WAVE HEIGHT (M)
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
DISTANCE (M)
2100 1% AEP
2080 1% AEP
Present 1% AEP
Present Mean SWL
Breaker


Site 5 Significant Wave Height - Westerly Swell
SIGNIFICANT WAVE HEIGHT (M)
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0
2000
4000
6000
8000
10000
12000
14000
16000
DISTANCE (M)
2100 1% AEP
2080 1% AEP
Present 1% AEP
Present Mean SWL
Breaker

Appendix 2 Historical Aerial Photographs

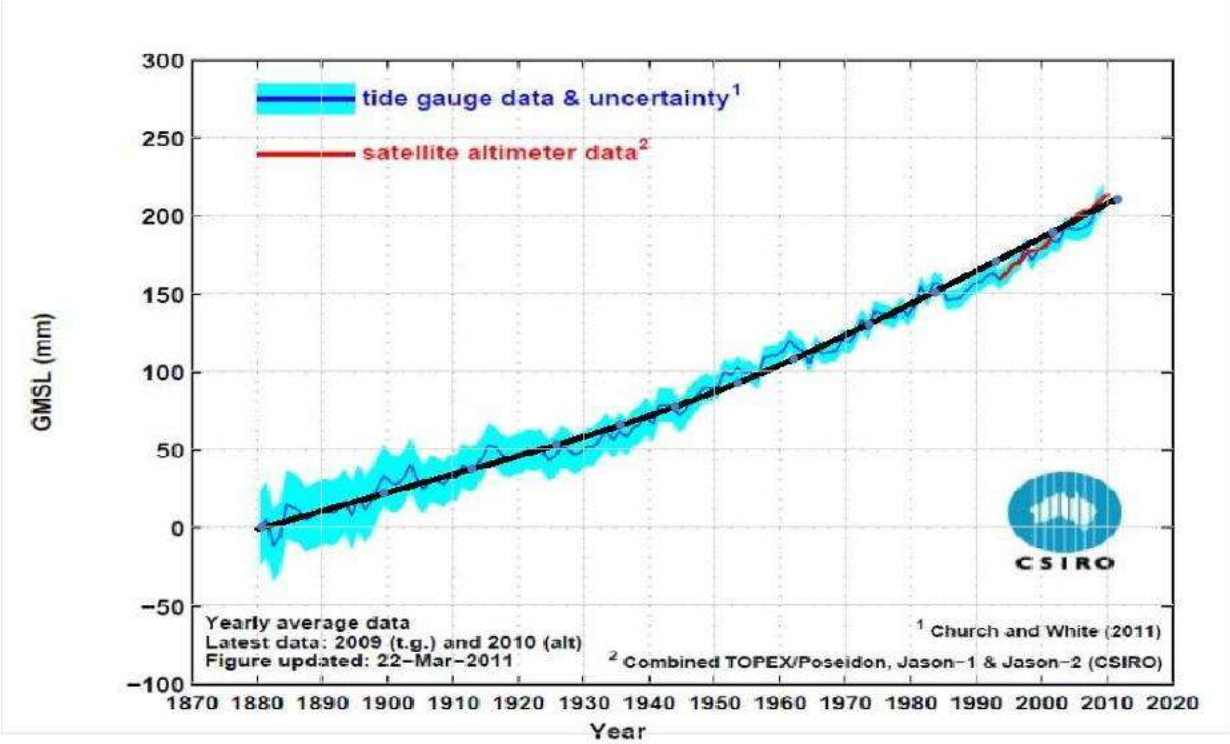



0
20
40
60
80
METRES
N
2N
2S
1973
2003
1986
1998
1992
1989
2006
2009
1982
2013
2012
1951
2015
5 m
1998
2013
2003
1951
2009
1973
2006
2012
1989
2015
1986
1992
1982
6.5 m
Image © 2018 CNES / Airbus


0 20 40 60 80
METRES
N
4N
4
4S
14 m
1973
1951
1986
1998
1989
1992
2003
2009
2012
23 m
1951
1989
1992
1998
2006
2009
2012
Image © 2018 CNES / Airbus


0 100 200 300 400
METRES
N
5N
5
5S
5.087
11.4 m
9.6 m
1951
1982
1973
1989
2003
2009
1992
1998
2012
2015
2006
2013
1986
Image © 2018 CNES / Airbus

Appendix 3 Shoreline Recession Analysis



Correlation Data

Year	Years Since 1880	SLR (mm)
1880.89	0.00	0.00
1899.53	18.64	21.77
1912.94	32.05	37.11
1925.84	44.95	52.96
1935.59	54.70	65.50
1943.95	63.06	77.18
1953.54	72.65	92.51
1962.07	81.18	107.67
1973.57	92.68	129.79
1983.68	102.79	150.52
1992.91	112.02	170.21
2001.62	120.73	189.20
2011.55	130.66	210.28

Polynomical Trend Analysis - Historical Sea Level Rise (1880 baseline)

X5	X4	X3	X^2	X
-2.79511E-09	2.9134E-07	7.04316E-05	-0.005162952	1.246008027

Polynomial Trend Analysis - Flinders Council (2010 baseline) based on RCP8.5 (DPAC 2016)

X^2	X
8.94444E-05	0.002172222

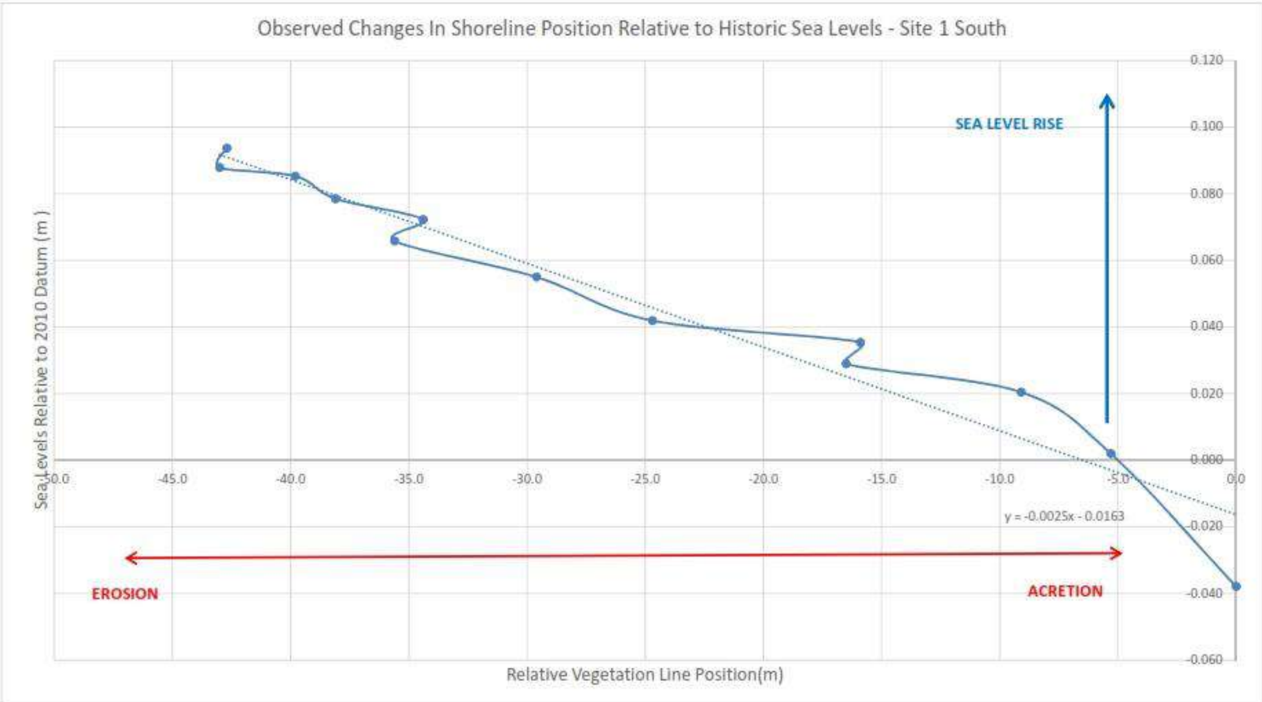
Year	Relative to 1880	Sea Level 1880 Baseline	Relative to 2010	Sea Level 2010 Baseline	Sea Level (m AHD)	
	Years Since	(mm)	Years Since	(m)	(m)	
1972	91.11	126.46		-0.08	0.00	m AHD Baseline
2010	129	207		0.000	0.081	DPAC Baseline

Aerial Photograph Analysis

1951	70	89			-0.038
1973	92	128			0.002
1982	101	147			0.020
1986	105	155			0.029
1989	108	162			0.035
1992	111	168			0.042
1998	117	181			0.055
2003	122	192			0.066
2006	125	199			0.072
2009	128	205			0.079
2012			2	0.005	0.085
2013			3	0.007	0.088
2015			5	0.013	0.094

Page 1

Temporal Beach Profile Changes: Profile Site 1 South



Shoreline Features Which May Influences Erosion Trends: Rivermouth - Bruun Value Can Not Be Applied To Whole Beach

Sea Level Rise (See Page 1) Vs Coastline Recession Correlations:

X	Y	R^2
-376.6899314	-7.508766024	0.95

Bruun Rule Corelation:
377 **metres horizontal recession per metre sea level rise**

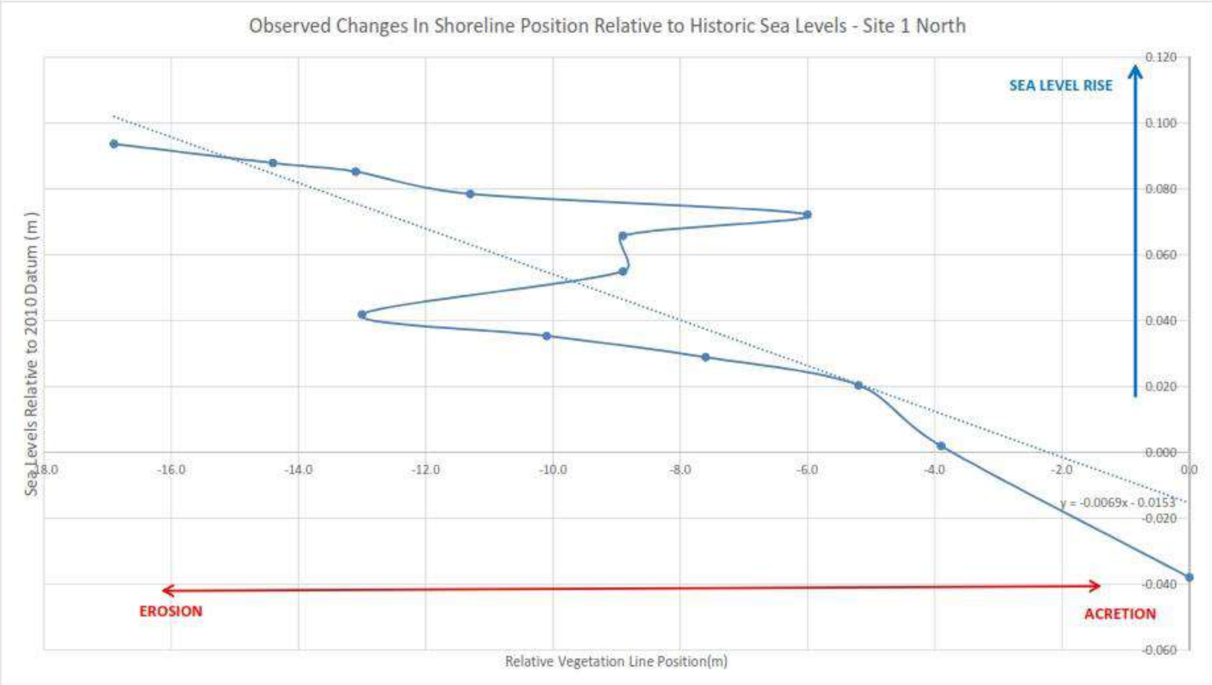
Projected Shoreline Position Relative to 2018 Based On Historical Recession Trends and Modelled RCP8.5 Sea Level Rise Trends For Flinders Council

Year	Sea Levels[A]	Recession (m)[B]
2050	0.207	78
2080	0.567	214
2100	0.897	338

Overal Erosion Range[C]	43
Erosion Deviation From Trendline[C]	6
Dune Height Within Erosion Zone (m)	6
Horizontal Storm Erosion Applied (m)[C,E]	6
Storm Erosion Over Period (m^3/m)[F]	37
Period Range (years)	64

A 2018 Baseline
B Note: To be Used As a Guideline Only. Shoreline Positions May Not Always Show Trends Consistent With Historical Observations
C Based on Horizontal Aeril Interpretation (m)
D Based on erosion deviation from trend line where R^2 >0.7 otherwise the horizontal erosion range is applied
F - Obtained By Multiplying Distance By Dune Height

Temporal Beach Profile Changes: Profile Site 1 North



Shoreline Features Which May Influences Erosion Trends: -

Sea Level Rise (See Page 1) Vs Coastline Recession Correlations:

X	Y	R^2	Bruun Rule Corelation:	
-102.6627747	-4.206519884	0.71	103	metres horizontal recession per metre sea level rise

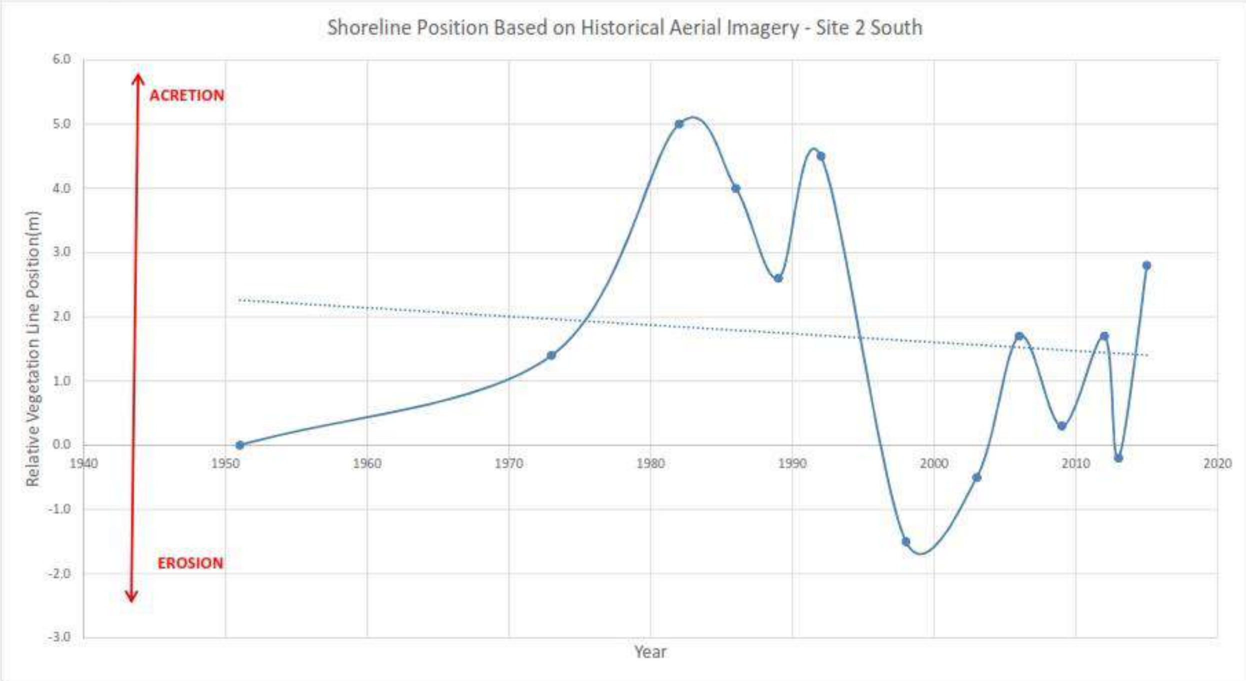
Projected Shoreline Position Relative to 2018 Based On Historical Recession Trends and Modelled RCP8.5 Sea Level Rise Trends For Flinders Council

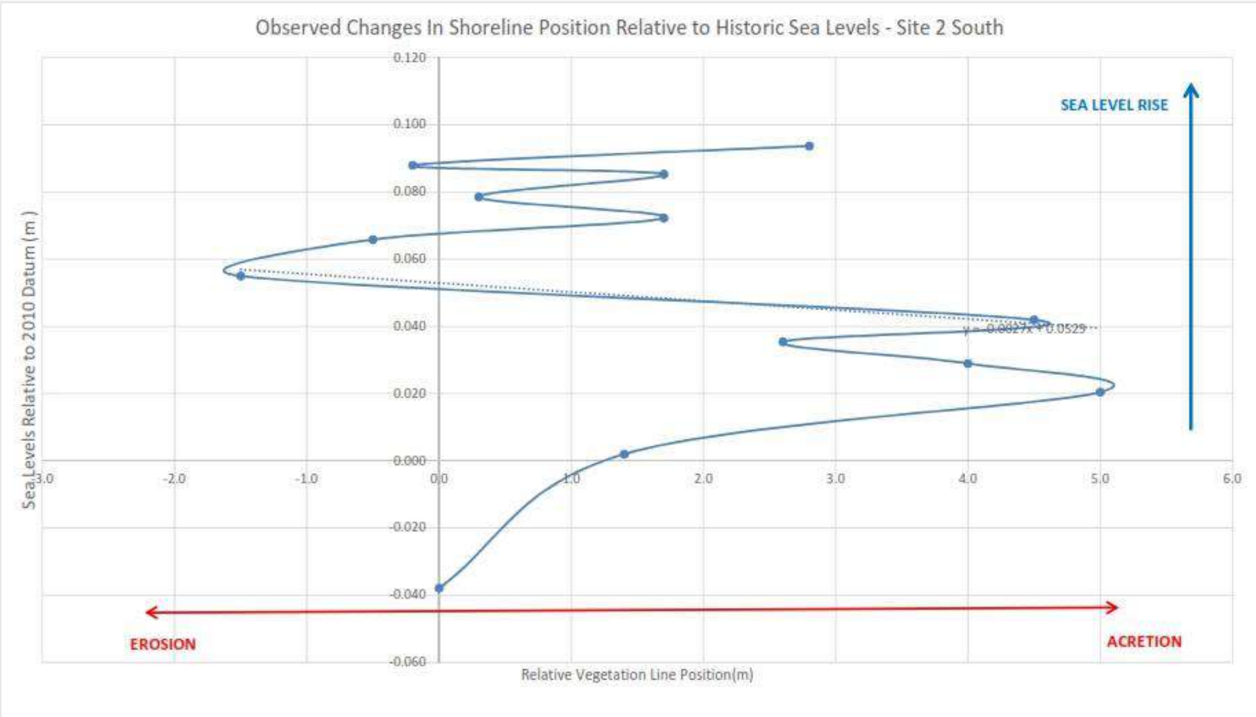
Year	Sea Levels[A]	Recession (m)[B]
2050	0.207	21
2080	0.567	58
2100	0.897	92

Overal Erosion Range[C]	17
Erosion Deviation From Trendline[C]	6
Dune Height Within Erosion Zone (m)	6
Horizontal Storm Erosion Applied (m)[C,E]	6
Storm Erosion Over Period (m^3/m)[F]	34
Period Range (years)	64

A 2018 Baseline
B Note: To be Used As a Guideline Only. Shoreline Positions May Not Always Show Trends Consistent With Historical Observations
C Based on Horizontal Aeril Interpretation (m)
D Based on erosion deviation from trend line where R^2 >0.7 otherwise the horizontal erosion range is applied
F - Obtained By Multiplying Distance By Dune Height

Temporal Beach Profile Changes: Profile Site 2 South





Shoreline Features Which May Influences Erosion Trends: -

Sea Level Rise (See Page 1) Vs Coastline Recession Correlations:

X	Y	R^2
-7.485738783	2.039344023	0.02

Bruun Rule Corelation:
* **metres horizontal recession per metre sea level rise**

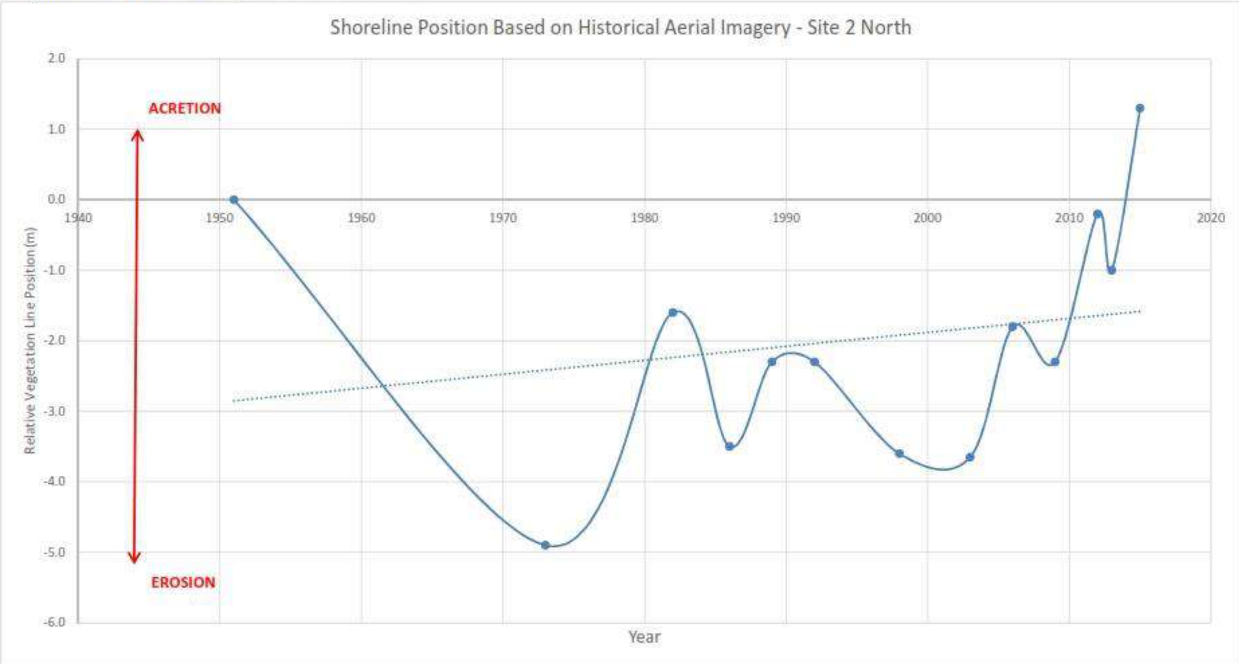
Projected Shoreline Position Relative to 2018 Based On Historical Recession Trends and Modelled RCP8.5 Sea Level Rise Trends For Flinders Council

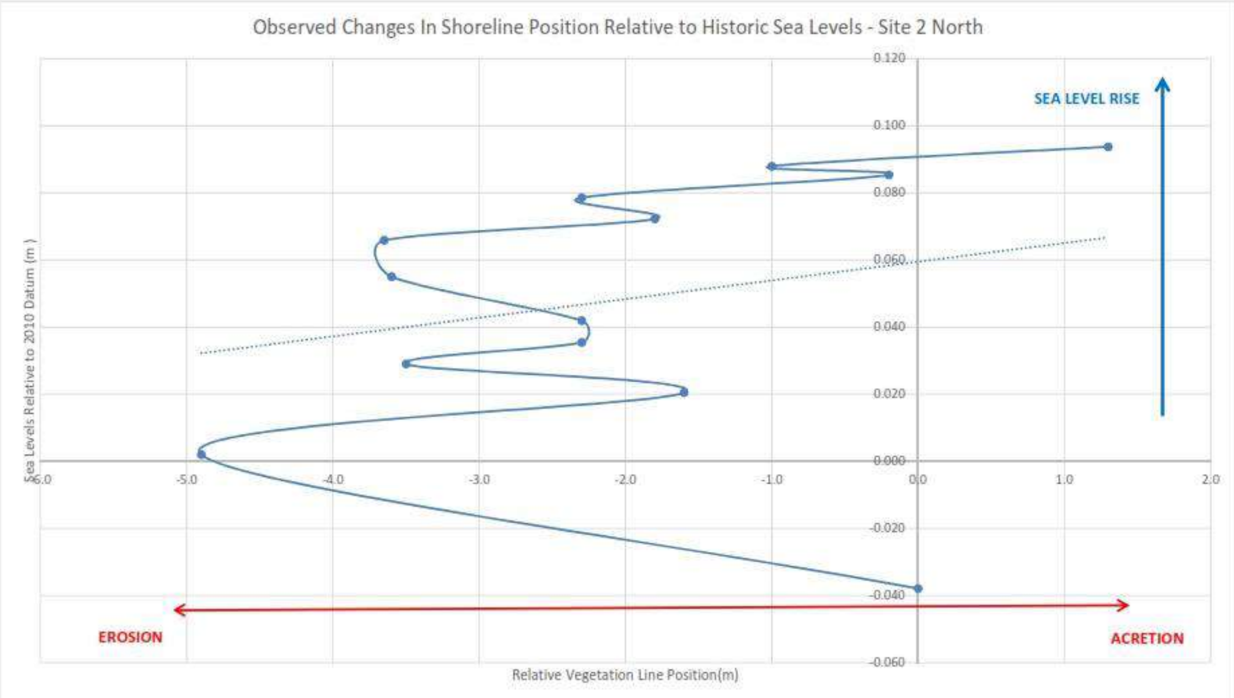
Year	Sea Levels[A]	Recession (m)[B]
2050	0.207	*
2080	0.567	*
2100	0.897	*

Overal Erosion Range[C]	7
Erosion Deviation From Trendline[C]	*
Dune Height Within Erosion Zone (m)	5
Horizontal Storm Erosion Applied (m)[C,E]	7
Storm Erosion Over Period (m^3/m)[F]	33
Period Range (years)	64

A 2018 Baseline
B Note: To be Used As a Guideline Only. Shoreline Positions May Not Always Show Trends Consistent With Historical Observations
C Based on Horizontal Aeril Interpretation (m)
D Based on erosion deviation from trend line where R^2 >0.7 otherwise the horizontal erosion range is applied
F - Obtained By Multiplying Distance By Dune Height
* No Recession Trend Apparent Based On R2 Value

Temporal Beach Profile Changes: Profile Site 2 North





Shoreline Features Which May Influences Erosion Trends: -

Sea Level Rise (See Page 1) Vs Coastline Recession Correlations:

X	Y	R^2
11.1221176	-2.526937232	0.06

Bruun Rule Corelation:
* **metres horizontal recession per metre sea level rise**

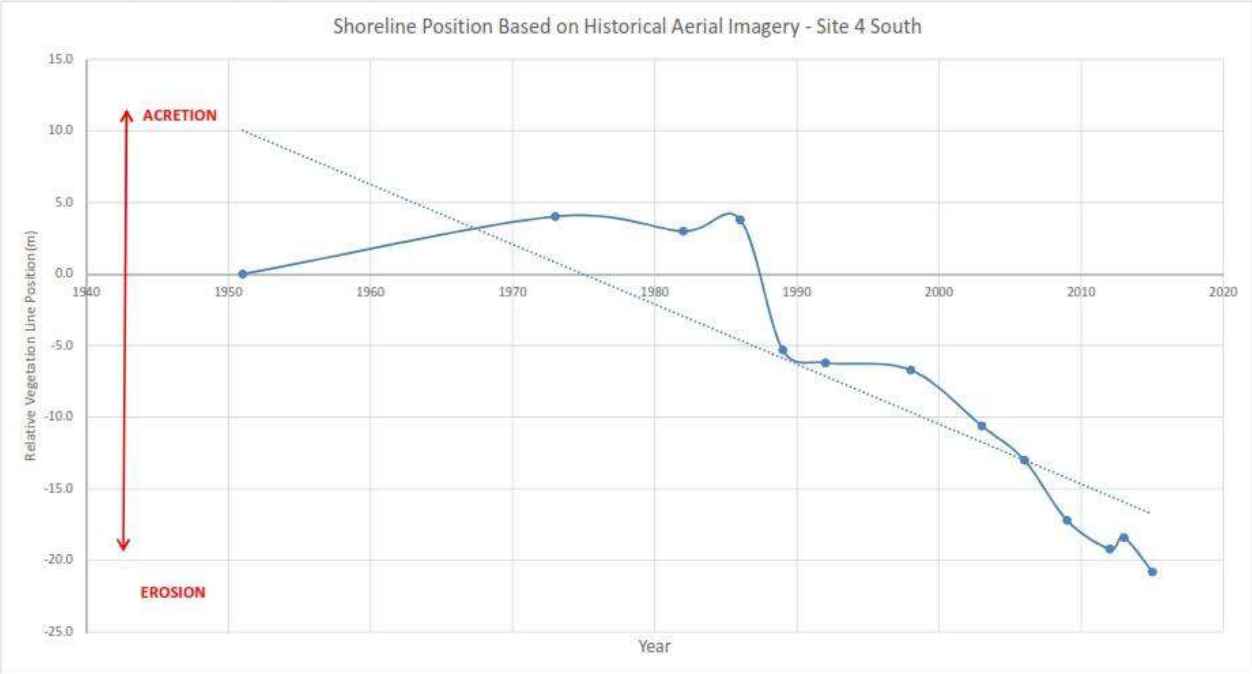
Projected Shoreline Position Relative to 2018 Based On Historical Recession Trends and Modelled RCP8.5 Sea Level Rise Trends For Flinders Council

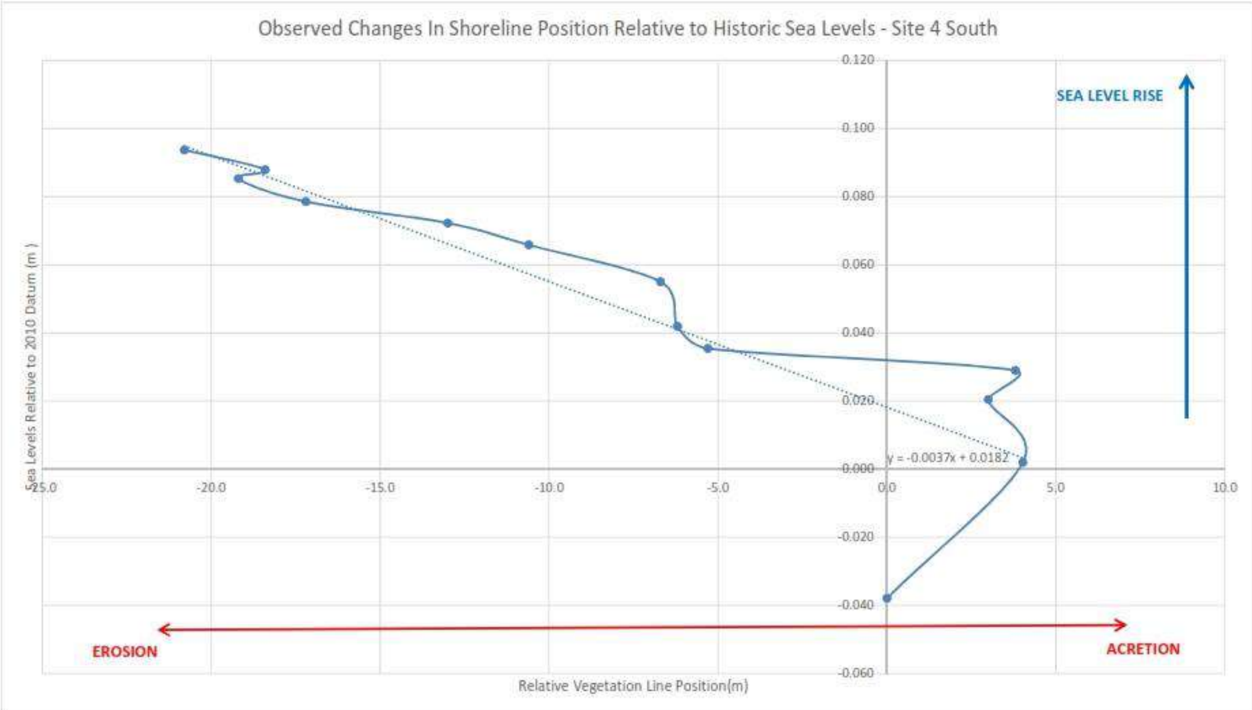
Year	Sea Levels[A]	Recession (m)[B]
2050	0.207	*
2080	0.567	*
2100	0.897	*

Overal Erosion Range[C]	6
Erosion Deviation From Trendline[C]	*
Dune Height Within Erosion Zone (m)	5
Horizontal Storm Erosion Applied (m)[C,E]	6
Storm Erosion Over Period (m^3/m)[F]	31
Period Range (years)	64

A 2018 Baseline
B Note: To be Used As a Guideline Only. Shoreline Positions May Not Always Show Trends Consistent With Historical Observations
C Based on Horizontal Aeril Interpretation (m)
D Based on erosion deviation from trend line where R^2 >0.7 otherwise the horizontal erosion range is applied
F - Obtained By Multiplying Distance By Dune Height
No Recession Trend Apparent Based On R2 Value

Temporal Beach Profile Changes: Profile Site 4 South





Shoreline Features Which May Influences Erosion Trends: Groyne - Bruun Value Can Not Be Applied To Whole Beach

Sea Level Rise (See Page 1) Vs Coastline Recession Correlations:

X	Y	R^2
-206.4503174	1.796800321	0.76

Bruun Rule Corelation:
206 **metres horizontal recession per metre sea level rise**

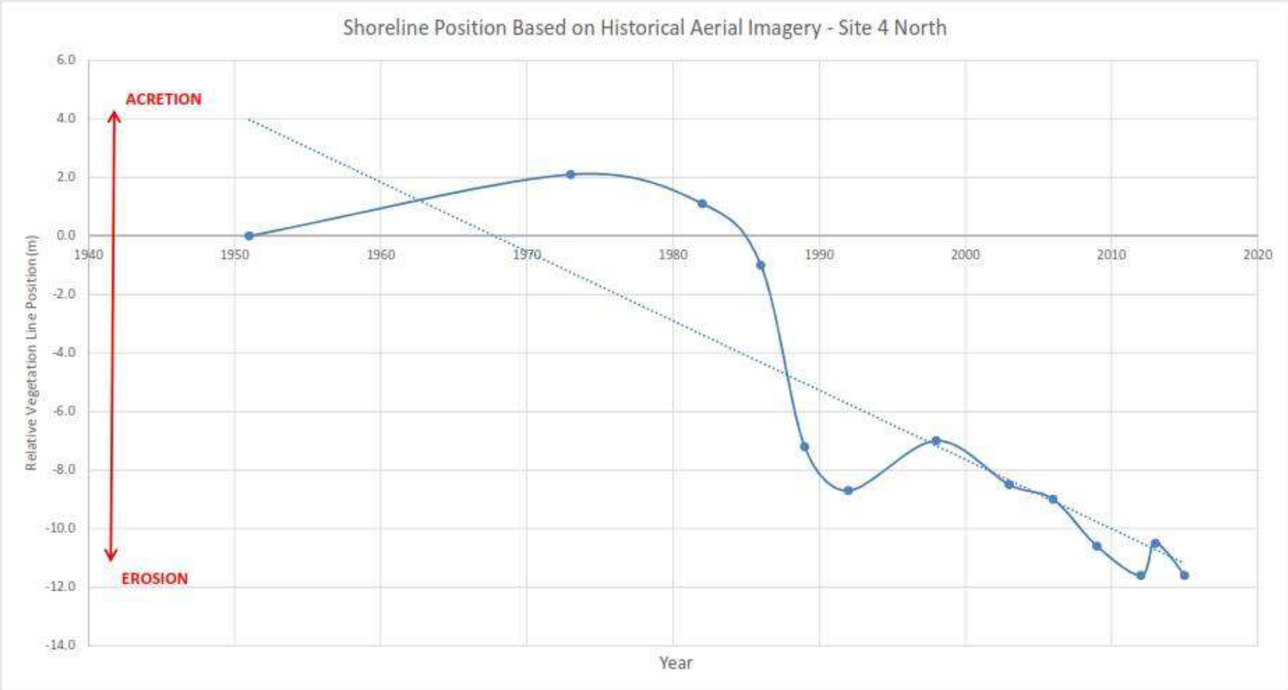
Projected Shoreline Position Relative to 2018 Based On Historical Recession Trends and Modelled RCP8.5 Sea Level Rise Trends For Flinders Council

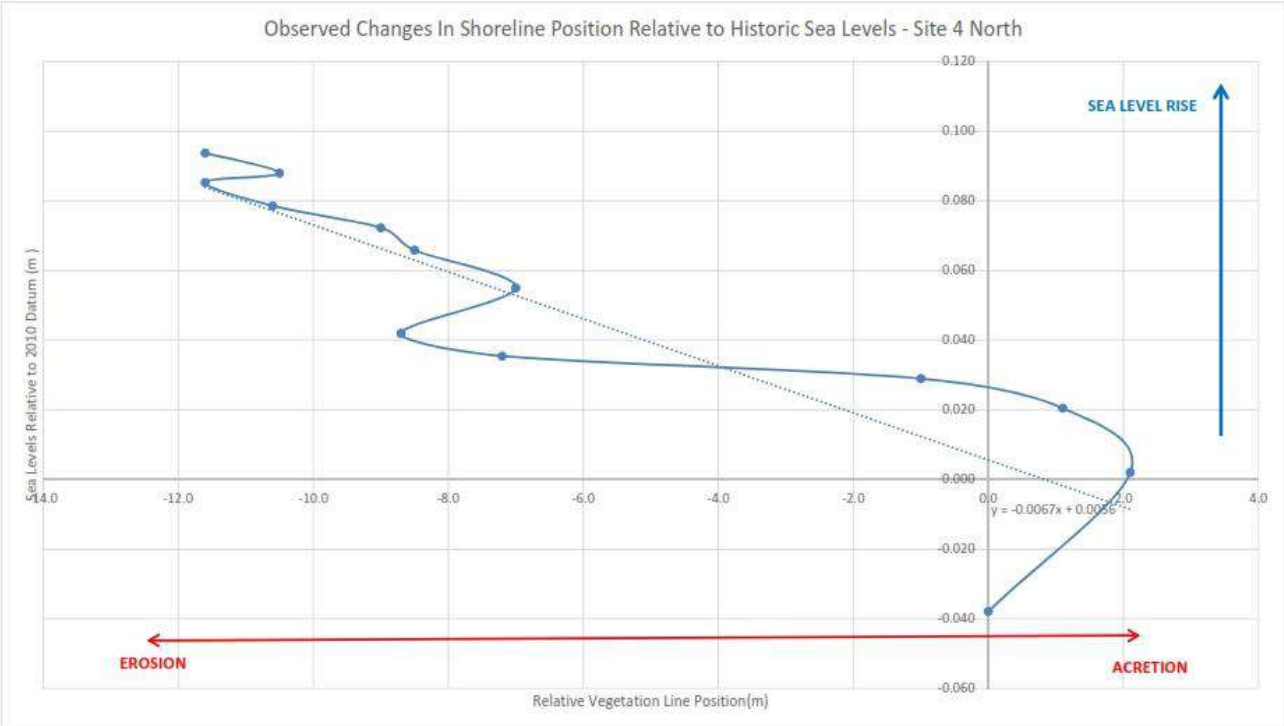
Year	Sea Levels[A]	Recession (m)[B]
2050	0.207	43
2080	0.567	117
2100	0.897	185

Overal Erosion Range[C]	25
Erosion Deviation From Trendline[C]	8
Dune Height Within Erosion Zone (m)	5
Horizontal Storm Erosion Applied (m)[C,E]	8
Storm Erosion Over Period (m^3/m)[F]	40
Period Range (years)	64

A 2018 Baseline
B Note: To be Used As a Guideline Only. Shoreline Positions May Not Always Show Trends Consistent With Historical Observations
C Based on Horizontal Aeril Interpretation (m)
D Based on erosion deviation from trend line where R^2 >0.7 otherwise the horizontal erosion range is applied
F - Obtained By Multiplying Distance By Dune Height

Temporal Beach Profile Changes: Profile Site 4 North





Shoreline Features Which May Influences Erosion Trends: Groyne - Bruun Value Can Not Be Applied To Whole Beach

Sea Level Rise (See Page 1) Vs Coastline Recession Correlations:

X	Y	R^2
-115.612952	-0.748769717	0.78

Bruun Rule Corelation:
116 **metres horizontal recession per metre sea level rise**

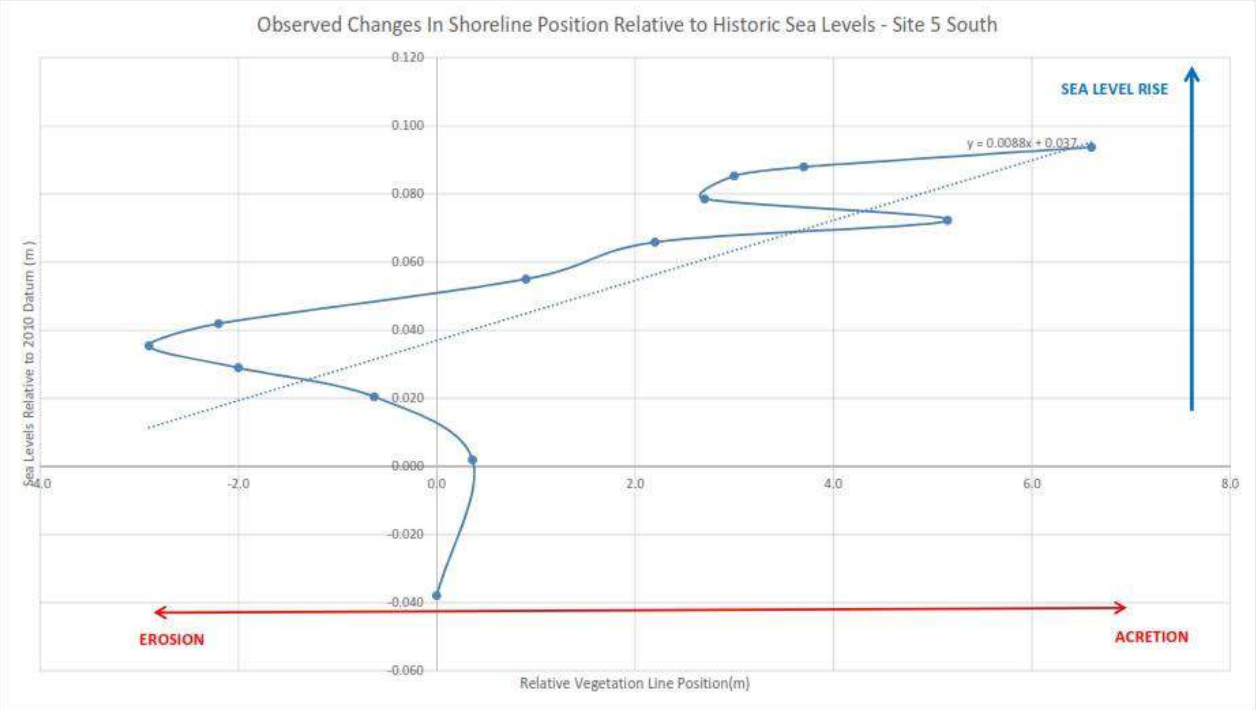
Projected Shoreline Position Relative to 2018 Based On Historical Recession Trends and Modelled RCP8.5 Sea Level Rise Trends For Flinders Council

Year	Sea Levels[A]	Recession (m)[B]
2050	0.207	24
2080	0.567	66
2100	0.897	104

Overal Erosion Range[C]	14
Erosion Deviation From Trendline[C]	4
Dune Height Within Erosion Zone (m)	5
Horizontal Storm Erosion Applied (m)[C,E]	4
Storm Erosion Over Period (m^3/m)[F]	21
Period Range (years)	64

A 2018 Baseline
B Note: To be Used As a Guideline Only. Shoreline Positions May Not Always Show Trends Consistent With Historical Observations
C Based on Horizontal Aeril Interpretation (m)
D Based on erosion deviation from trend line where R^2 >0.7 otherwise the horizontal erosion range is applied
F - Obtained By Multiplying Distance By Dune Height

Temporal Beach Profile Changes: Profile Site 5 South



Shoreline Features Which May Influences Erosion Trends: Groyne - Bruun Value Can Not Be Applied To Whole Beach

Sea Level Rise (See Page 1) Vs Coastline Recession Correlations:

X	Y	R^2
50.40402549	-1.141841969	0.44

Bruun Rule Corelation: * **metres horizontal recession per metre sea level rise**

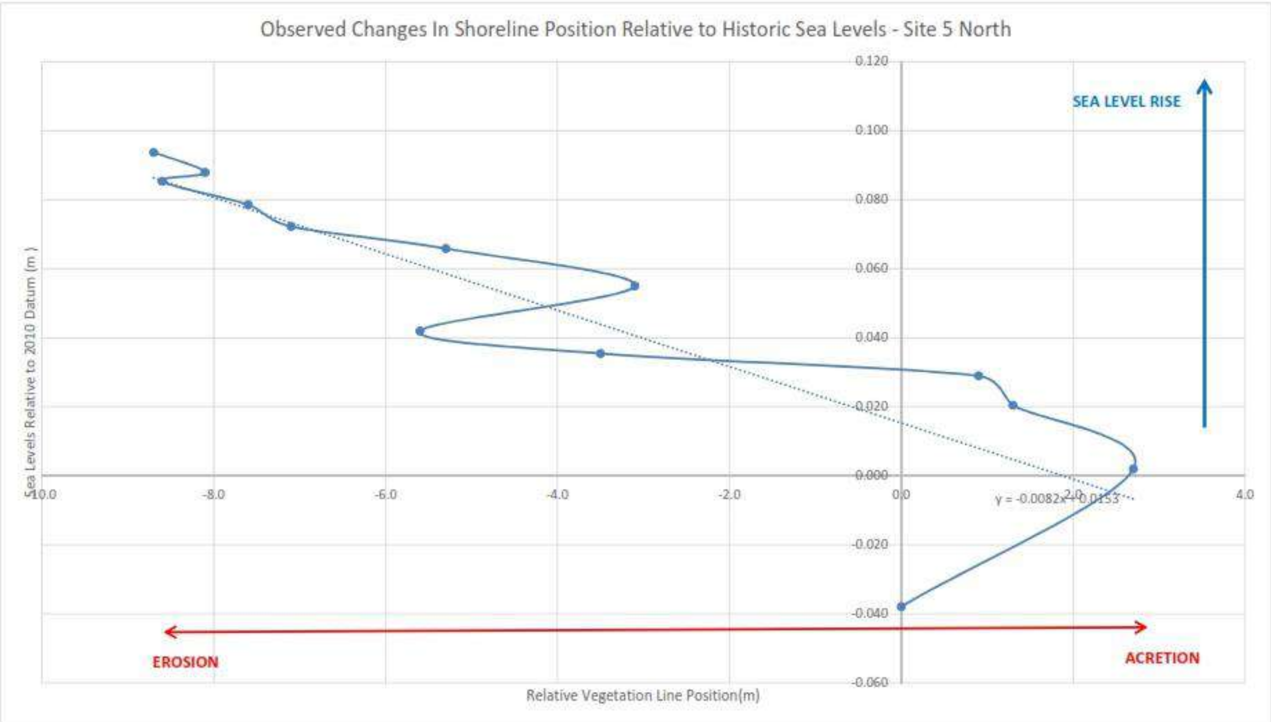
Projected Shoreline Position Relative to 2018 Based On Historical Recession Trends and Modelled RCP8.5 Sea Level Rise Trends For Flinders Council

Year	Sea Levels[A]	Recession (m)[B]
2050	0.207	*
2080	0.567	*
2100	0.897	*

Overal Erosion Range[C]	10
Erosion Deviation From Trendline[C]	*
Dune Height Within Erosion Zone (m)	6
Horizontal Storm Erosion Applied (m)[C,E]	10
Storm Erosion Over Period (m^3/m)[F]	57
Period Range (years)	64

A 2018 Baseline
B Note: To be Used As a Guideline Only. Shoreline Positions May Not Always Show Trends Consistent With Historical Observations
C Based on Horizontal Aeril Interpretation (m)
D Based on erosion deviation from trend line where R^2 >0.7 otherwise the horizontal erosion range is applied
F - Obtained By Multiplying Distance By Dune Height
No Recession Trend Apparent Based On R2 Value

Temporal Beach Profile Changes: Profile Site 5 North



Shoreline Features Which May Influences Erosion Trends: -

Sea Level Rise (See Page 1) Vs Coastline Recession Correlations:

X	Y	R^2
-92.24760434	0.412308091	0.75

Bruun Rule Corelation:
92 **metres horizontal recession per metre sea level rise**

Projected Shoreline Position Relative to 2018 Based On Historical Recession Trends and Modelled RCP8.5 Sea Level Rise Trends For Flinders Council

Year	Sea Levels[A]	Recession (m)[B]
2050	0.207	19
2080	0.567	52
2100	0.897	83

Overal Erosion Range[C]	11
Erosion Deviation From Trendline[C]	3
Dune Height Within Erosion Zone (m)	6
Horizontal Storm Erosion Applied (m)[C,E]	3
Storm Erosion Over Period (m^3/m)[F]	19
Period Range (years)	64

A 2018 Baseline
B Note: To be Used As a Guideline Only. Shoreline Positions May Not Always Show Trends Consistent With Historical Observations
C Based on Horizontal Aeril Interpretation (m)
D Based on erosion deviation from trend line where R^2 >0.7 otherwise the horizontal erosion range is applied
F - Obtained By Multiplying Distance By Dune Height

