

Coastal Processes

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8 Coastal Processes

8.1 Introduction

This chapter assesses the coastal processes relevant to the existing and proposed Lindeman Great Barrier Reef Resort. Wave conditions at the site are affected by occasional tropical cyclones and south-easterly trade winds. During the summer, persistent winds from the north-northeast sector can occur, but those conditions will be offshore and not of concern in terms of wave conditions at the site. Despite the limited fetch from Shaw Island, about 5 kilometres, wave conditions at the current resort jetty are unsuitable for berthed vessels and passenger safety. An assessment of available data indicated that there was very little site specific met-ocean data available for this site, apart from tidal planes (Queensland Tides, 2016) and a single data point for storm tide (SEA/GHD, 2003). Wind data is available from nearby Hamilton Island airport and historical cyclone track data from the Bureau of Meteorology. Hence it has been necessary to undertake a range of data collection tasks and numerical modelling in order to develop the data necessary to assist the design of shoreline facilities as part of the EIS process. These investigations have been undertaken being cognisant of projected climate change parameters, mainly in terms of sea level rise and intensification of cyclones. The findings of these models has optimised the project design by ensuring that potential deleterious effects are removed or ameliorated early in the design process.

This chapter is supported by a technical appendix prepared by Cardno included in **Appendix H – Coastal Processes**.

Addendum: This EIS was initially prepared assuming that the safe harbour was to be part of the Lindeman Great Barrier Reef Resort Project. With the commencement of the Great Barrier Reef Marine Park Authority's (GBRMPA) Dredging Coral Reef Habitat Policy (2016), further impacts on Great Barrier Reef coral reef habitats from yet more bleaching, and the recent impacts from Tropical Cyclone Debbie, the proponent no longer seeks assessment and approval to construct a safe harbour at Lindeman Island. Instead the proponent seeks assessment and approval for upgrades to the existing jetty and additional moorings in sheltered locations around the island to enable the resort's marine craft to obtain safe shelter under a range of wind and wave conditions. Accordingly, remaining references to, and images of, a safe harbour on various figures and maps in the EIS are no longer current.

8.2 Existing Site Conditions

The present beach is generally only usable at high tide and is often affected by strong south-easterly winds. Hence White Horse Australia Lindeman Pt Ltd propose that the redeveloped resort will have a pool and beach area on the leeward side of the existing natural beach. The present beach area at the resort is held between two low crested rock groynes that are submerged at high tide. Cardno undertook a site visit on 16 and 17 April 2015 and investigated the coastal features of the site at low and high tides. Selected site photographs are presented in this section.

Figures 8-1 and **8.2** describe the existing boat navigation channel at low and high tides. Coral patches either side of the channel can be seen, together with the exposure of the reef at low tide – see other photographs as well.

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Figures 8-3 and **8-4** provide views of the resort beach at mid-tide and low tide, showing the eastern and western groynes and the small extent of sandy beach. Both views are from the existing jetty.

Figures 8-5 to **8-12** present a range of views of the western groyne and shoreline protection works, as well as the beach at low tide. One can see that rock armour has been placed beneath the existing pool surround area and administration building. The caretaker who guided Cardno during this site visit advised that he had seen wave run-up and overtopping of the pool surround in a past storm(s).

Figure 8-13 presents a view of the beach and pool deck area from the east at high spring tide.

Figure 8-14 provides a closer view of the existing pool area at high tide showing the flat beach slope.

Figures 8-15 to **8-17** describe the boat ramp and eastern shoreline areas, showing the rocky form of this shoreline with scattered sand patches and revetted back beach area near the eastern groyne, jetty and boat ramp. **Figure 8-18** presents an aerial view of the western coastline of the island showing high rocky bluffs and enclosed small beaches. Some redevelopment buildings are planned for this region.

Figure 8-1. Existing navigation channel – low tide.



Figure 8-2. Existing navigation channel – high tide.



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Figure 8-3. Resort beach – looking west – mid tide.

Figure 8-4. Resort beach – looking west – low tide.



Figure 8-5. Existing groyne western end of beach.



Figure 8-6. Rock revetment – western end of existing resort.







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Figure 8-7. Resort beach – looking east – mid tide.

Figure 8-8. Storm water drain and revetment west of existing beach.



Figure 8-9. Existing rock revetment and beach near existing administration buildings.



Figure 8-10. Existing central facilities building.





Figure 8-11. Resort beach and existing pool area.



Figure 8-13. Resort beach – looking west – high tide.

Figure 8-14. Resort beach – high tide.





Figure 8-15. Shoreline looking east from jetty.



Figure 8-16. Shoreline and boat ramp looking east.



Figure 8-17. Boat ramp area.



Figure 8-18. Aerial view of western coastline.





8.3 Bathymetry

The bathymetric data applied in this study have been derived from several sources. In the immediate region of the resort, project specific bathymetric survey of the site was available in digital form – see **Appendix H** – **Part C**. Navigation charts AUS 252, 253, 254 and 370 and 371 provided bathymetric data beyond the immediate study area. Much of this data had been prepared for previous investigations for sites at Shute Harbour and Bowen. In the more distant deep ocean areas, up to 1000km from the Whitsunday Islands, regional bathymetric data was obtained from ETOPO1, which is a 1 arc-minute global relief model (Amante et al 2009). That model grid is largely generated by combining quality-controlled ship depth soundings with interpolation between sounding points guided by satellite-derived gravity data. The ETOPO1 bathymetric data set is the industry standard for global bathymetric data sets and provides a sufficiently accurate depiction of the land and ocean areas of the Earth in deeper waters. This data was combined to form a comprehensive Digital Elevation Model (DEM) of the study site and model area. All data was converted to a datum of AHD.

8.4 Cyclone Track Data

Recorded historical cyclone track data was obtained from two separate databases. Cyclone track data from 1955-2009 was obtained from the Australian Bureau of Meteorology (BoM, 2011) Pacific Tropical Cyclone Data Portal, which contains historical information about tropical cyclones in the Pacific and Indian Ocean regions of the Southern Hemisphere. This tropical cyclone archive has been prepared by the National Climate Centre (Australian Bureau of Meteorology) using best track data obtained from the Regional Specialised Meteorological Centres in La Reunion (France) and Nadi (Fiji), along with the Tropical Warning Centres of Australia (Brisbane, Darwin and Perth) and New Zealand (Wellington). More recent cyclones were investigated on the Bureau of Meteorology web-site.

Table 8-1 lists the 20 most severe tropical cyclones and other tropical low pressure systems that have affected the region prior to Cylone Debbie.

Year	Cyclone Name	Year	Cyclone Name
1955	Agnes	1978	Kerry
1957 (1)	Unnamed	1979	Paul
1957 (2)	Unnamed	1984	Pierre
1958	Connie	1987	Charlie
1969	Ada	1989	lvor
1970	Gertie	1995	Celeste
1973	Vera	2002	Erica
1975	Dawn	2009	Ului
1975	Watorea	2010	Anthony
1978	Gordon	2014	Marcia

Table 8-1. Historical Tropical Cyclones Selected for Numerical Modelling (prior to Cyclone Debbie).

In recent times three tropical cyclones were by far the most severe tropical cyclones to have affected this region. They are:-

- <u>Severe Tropical Cyclone Ului:</u> TC Ului in 2009 crossed the Queensland coast north of Lindeman Island;
- <u>Severe Tropical Cyclone Anthony</u>: TC Anthony in 2010 crossed the coast near Bowen and caused gale force winds near Mackay; and
- <u>Severe Tropical Cyclone Debbie:</u> TC Debbie in 2017 crossed the coast near Airlie Beach and caused damage to Whitsunday Islands, including Hamilton Island, where wind gusts up to 263 km/h were recorded.

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Other equally or more severe tropical cyclones may have occurred before reliable records were kept.

In these analyses a peripheral pressure of 1010hPa was adopted when calculating the wind and pressure fields for cyclone modelling. Although TC Yasi was a category 5 cyclone, it crossed the north Queensland coast in 2011 between Innisfail and Cardwell and did not cause as much damage at Lindeman Island as other closer cyclones; in terms of storm tide and wave-caused erosion at the resort site.

SEA/GHD (2003) present storm tide data for Lindeman Island which is appropriate to this project, but it does not include nearshore wave data or describe the wave set-up component; nor wave conditions. That study undertook an equivalent 50,000 years of cyclone event modelling. At Lindeman Island Jetty, the results for present day and 2100 (including a then projected sea level rise of 0.48m), are described in **Table 8-2**.

Epoch	50-years	100-years	500-years	1,000-years	10,000-years
Present Day	3.1	3.3	3.7	3.9	4.4
2100 with 0.48m SLR	3.7	3.9	4.5	4.7	5.4

Table 8-2. Storm Tide Levels (m AHD) – SEA/GHD (2003) – Including Wave Set-up.

The 2100 storm tide levels also include a 20% increase in the maximum probable cyclone intensity.

8.5 Tide Data

The tides of the Whitsunday region will affect the currents developed during a tropical cyclone and in normal conditions. For navigation and maritime works, a low tide level datum is used, nominally Chart Datum (CD), which is used as the datum of navigation charts and tide predictions. Chart Datum nowadays is commonly specified in terms of lowest astronomical tide (LAT). This is the lowest predicted tide over a period of about 19 years, where the predictions are best made using tidal constants determined from recordings of water levels made over a period of more than one year.

Daily observations of sea level are affected by meteorological and large scale oceanic processes. These variations are commonly up to 0.2m, but are potentially much greater during a tropical cyclone, for example.

Indicative tidal planes near this site are presented in **Table 8-3** – based upon Queensland Tide Tables for 2016 and site survey information (refer to **Appendix H – Part C**).

Tidal Plane	m LAT
Highest Astronomical Tide (HAT)	4.94
Mean High Water Springs (MHWS)	3.78
Mean High Water Neaps (MHWN)	2.95
Mean Sea Level (MSL)	2.32
Mean Low Water Neaps (MLWN)	1.49
Mean Low Water Springs (MLWS)	0.66
Lowest Astronomical Tide (LAT)	0.0



The tidal planes at this site, described in terms of MHWS, for example, indicate that tides in this area are mixed, semi-diurnal. In other words, there are typically two high tides and two low tides every day. AHD is 2.28m LAT. Tidal water levels used to drive the regional oceanographic models were predicted using ten offshore deep water tidal constants. Offshore tidal constants for the hydrodynamic and wave model boundaries were extracted from the Oregon State University global model of ocean tides (TPXOv7.2) which uses track-averaged data from the satellites TOPEX/Poseidon and Jason (on TOPEX/POSEIDON tracks since 2002). The methodology of the global tide models is described in Egbert and Erofeeva (2002). TPXOv7.2 provides up to 10 tidal constants on a 1/4 degree resolution full global grid. The tidal constants are provided as complex amplitudes of earth-relative sea-surface elevation for eight primary (M₂, S₂, N₂, K₁, O₁, P₁, Q₁) and two higher order (M₄, MS₄) harmonic constituents. Additionally, 22 tidal constants for Lindeman Island were provided by Maritime Safety, Queensland. That data was used to undertake tidal predictions at Lindeman Island as part of tidal model calibration and also used in the storm tide investigation.

8.6 Waves

There is no recorded wave data for this site. Hence wind wave modelling was required for this investigation in order to develop the required design wave parameters used in foreshore erosion modelling and shoreline revetment wave run-up investigations.

Hindcast modelled wave data was obtained from the WaveWatch III[™] (WW-III) global wind and wave model for a location offshore and southeast of the site. WW-III is a full-spectral, third-generation wind-wave model that has been developed by the Marine Modelling and Analysis Branch of the Environmental Modelling Center of the National Center for Environmental Prediction (NCEP) in the United States of America (Tolman 1997, 1999, 2009). WW-III is based on WW-I developed at Delft University of Technology, and WW-II developed at the NASA Goddard Space Flight Center. WW-III differs from its predecessors in most aspects, including governing equations, program structure, numerical and physical approaches (Tolman, 2009).

Recorded wave data time-series from the Queensland Government Waverider buoy records for a site north of Lindeman Island and Bowen were obtained from historical (BPA) data reports published on their web site. That data was used for wave model calibration. Only the Lindeman Island site proved to be suitable for wave model verification.

8.7 Wind Data

In addition to cyclonic wind speeds, SE trade winds were also assessed, because they are the predominant winds that affect the resort site. Recorded hourly data was available from the Bureau of Meteorology for Hamilton Island Airport for a period covering the years 2002 to 2007. This weather station is located on the coast at an approximate elevation of 59m above sea level. As such, the wind speed data was reduced assuming a power-law wind profile, and the following formula:-

$$\frac{U_{10m}}{U_{zm}} = \left(\frac{10}{z}\right)^{\frac{1}{7}}$$

A wind rose prepared from the Hamilton Island wind data is represented in **Figure 8-19**. It shows that the majority of winds come from the east to south quadrant, and from this it can be deduced that the southern shore of the study site is more commonly affected by local wind waves than the western shore; noting that Shaw Island limits the fetch of these waves. These south-easterly waves are generally small; in the order of 0.3m, but this is too high for comfortable and safe embarkation and disembarkation. The wind data was

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discretised into 22.5 degree directional sectors and then the data for the, E, ESE, SE and SSE directional sectors were analysed. The peaks of independent wind events were analysed and design winds were determined for each direction. The results are given in **Table 8-4** below. They are lower than the cyclonic winds.

ARI (years)	Wind Speed for each Directional Sector (m/s)				
	E	ESE	SE	SSE	
10	15.9	16.9	16.6	15.7	
25	16.4	17.6	17.1	16.4	
50	16.8	18.0	17.5	16.9	
100	17.3	18.5	17.8	17.4	
200	17.7	19.0	18.2	17.9	

Table 8-4 Ha	milton Island	Directional [Desian Wind	Sneeds	(Modified)
	inniton islana.	Directionaria	Jesign mina	opecus	(mounica).

A spatially variable wind data set for the region was available from the BoM in the form of the ACCESS weather model (Australian Community Climate and Earth-System Simulator). The ACCESS model has been developed and tested by research staff from the Centre for Australian Weather and Climate Research (CAWCR) and is based on the on the UK Meteorological Office's Unified Model/Variational Assimilation system. The ACCESS wind data set is available from March 2011 to the present over a region bound by 55° S to 4°44' N and from 95° E to 169°41' E. The spatial resolution is 0.11° – 12km, with a 1 hour time-step. However, it was generally found that currents near the site were strongly tidally dependent, as well being affected by reliable bathymetric description; provided by the project specific bathymetric data.

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8.8 Sediment Particle Size Data

Sediment sampling undertaken in October 2015 for locations identified in **Figure 8-20** has found scattered, thin patches of poorly sorted sediments (refer to **Appendix H - Part D**), which is typically of D_{50} of 0.5mm or greater. Hence, sediment transport impacts of the formerly proposed safe harbour and proposed jetty are likely be minor and adsorbed contaminant concentrations will be low. Further inspection of this data shows that the nearshore sediments are poorly sorted, with a large range of particle sizes. **Appendix H – Part D** also shows that sediments are coarser near the shoreline and that the percent of fines increases as one moves offshore.

Comparing the particle size data with particle sorting figures extracted from a teaching document prepared by Maggie Williams, University of Liverpool, one can infer that the nearshore sediments found at this reef site are not deposited in an energetic marine environment. Natural sediment systems affected by currents and waves are normally well sorted over long periods of time, where waves and currents are sufficiently energetic. Conditions become more energetic further offshore.



Figure 8-20. Sediment Sampling Locations.

8.9 Sediment Quality Data

The sediment samples described in **Section 8.8** were also analysed to determine potential contaminants. The survey findings identified that there were no sites that had Tributyltin (TBT) or heavy metal concentrations above the ANZECC guidelines (refer to **Appendix H – Part D**).

8.10 Current Data

No site specific current data was available for this project. Hence it was necessary to undertake two campaigns of current data collection, one in October 2015 (1.5 months of data) and the other in the period from mid-January to mid-May in 2016 (4.5 months of data). In both cases ADCP transects were undertaken along three lines near the resort site at times of spring tides.

The fixed ADCP was deployed in a depth of about 12m at LAT between Lindeman and Seaforth Islands (refer to **Figure 8-21**). The figure also indicates the ADCP transect lines, but note that Line 2 was extended much further towards the south-east into Kennedy Sound during the second campaign. This longer transect was undertaken to provide a greater areal description to assist with model calibration. The fixed instrument was deployed at 1m above the seabed and bin sizes of 1m were adopted. This data is shown in time-series plot form and current rose form in **Appendix H – Part E**. Current speed and direction are shown to have significant spatial and temporal variations, but with persistent westerly directed flow at the fixed ADCP location. There is an observable variation in horizontal current speed at different water column heights.







8.11 Bases for Assessments

8.11.1 AS3962

Australian Standard AS3962 provides guidelines for marina design. **Table 8-5**, extracted from that document, provides guidelines for suitable wave conditions for vessels moored at marina facilities. Reference to this table will be made later in this chapter. Note that there are three levels of acceptable wave conditions specified. Of those, this author's experience with marina/small craft harbour design is that users 'expect' excellent wave conditions within a marina – see blue below. Such marinas/safe harbours commonly have a range of users, including occasional recreational users who are less experienced with wave conditions. The 'excellent' criteria in AS3962 provide the safest conditions for inexperienced users to board safely.

Direction and Book Bariad of	Significant Wave Height		
Design Harbour Wave	Wave Event Exceeded Once in 50 Years	Wave Event Exceeded Once Every Year	
Head Seas Less than 2s	Conditions not likely to occur during this event	Less than 0.3m wave height	
Head Seas Greater than 2s	Less than 0.6m wave height	Less than 0.3m wave height	
Oblique Seas Greater than 2s	Less than <mark>0.4m</mark> wave height	Less than 0.3m wave height	
Beam Seas Less than 2s	Conditions not likely to occur during this event	Less than 0.3m wave height	
Beam Seas Greater than 2s	Less than <mark>0.25m</mark> wave height	Less than <mark>0.15m</mark> wave height	

Table 8-5. Criteria for a 'Good' Wave Climate in a Small Craft Harbour.

Note: For criteria for an **'excellent'** wave climate multiply wave height by 0.75, and for a **'moderate'** wave climate multiply wave height by 1.25. For vessels of less than 20m length, the most severe wave climate should satisfy moderate conditions. For vessels larger than 20m in length, the wave climate may be more severe and an Hs=0.2m has been adopted at 1-year ARI for the inter-island ferry that will bring visitors to the resort.

Source: Adapted from MERCER, A.G., ISAACSON, M. and MULCHAHY, M.W. Design wave climate in small craft harbours. 18th Conference on Coastal Engineering. Capetown, 1982, as presented in Australian Standard AS3962.

Tables 8-6 and **8-7** set down the criteria for 'excellent' and 'moderate' wave conditions for berthed small craft.



Table 8-6. Criteria for a 'Excellent' Wave Climate in a Small Craft Harbour.

Table 8-7. Criteria for a 'Moderate' Wave Climate in a Small Craft Harbour.

	Significant Wave Height			
Direction and Peak Period of Design Harbour Wave	Wave Event Exceeded Once in 50 Years	Wave Event Exceeded Once Every Year		
Head Seas Less than 2s	Conditions not likely to occur during this event	Less than 0.38m wave height		
Head Seas Greater than 2s	Less than 0.75m wave height	Less than 0.38m wave height		
Oblique Seas Greater than 2s	Less than 0.5m wave height	Less than 0.38m wave height		
Beam Seas Less than 2s	Conditions not likely to occur during this event	Less than 0.38m wave height		
Beam Seas Greater than 2s	Less than 0.31m wave height	Less than 0.19m wave height		

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8.12 Short Term Storm Erosion

In order to address the coastal erosion hazards at the study site, it was necessary to assess the potential for periodic, short-term erosion caused by extreme waves and water levels associated with severe storms such as tropical cyclones. Lindeman Island is located on the Australian east coast at a latitude of \approx 19.5° south, and hence is periodically affected by tropical cyclones, which have the potential to cause significant short term erosion. This erosion will not affect the proposed new resort, but will reduce the amenity for some months without intervention.

The short-term storm erosion at the site was assessed using the SBEACH modelling package, which simulates the impact of storm waves and water levels on specified beach profiles. Note, however, that much of the current resort site is protected by existing revetments and the re-developed site will include new revetments, see **Appendix H – Part I**, that will prevent storm and long-term erosion – and which have been raised to a crest level of 5.6m AHD in order to function adequately in the projected sea level rise environment. Moreover, the proposed dune/steps beach-access feature that would protect the upgraded internal pool area would be protected by the ocean-side steps, which would need to be designed as a revetment; see **Appendix H – Part I**.

8.12.1 Design Wave Criteria

There is no recorded wave data for this site. As a result, wave modelling was required for this investigation in order to develop the required design wave parameters used in the storm induced erosion modelling. Because the most extreme waves at the site are likely to be cyclonic-wind driven waves, design wave heights at the site were determined using extremal analysis of the hindcast cyclone events to determine these design wave parameters.

Because Lindeman Island is neighboured to the east and south by Shaw Island, the propagation of waves from this sector will be limited by this fetch. Similarly, waves from the south-west will be limited by Seaforth Island.

Wave modelling also included the 2050 and 2100 projected sea level rises. This is common practice in coastal and estuarine modelling. The resultant 50 and 100-years-ARI wave heights at the site are shown in **Tables 8**-8 and **8-9**. There is little difference between the 50 and 100-years ARI design wave and water level parameters.

Seenario	Water Level (m AHD)	Southern Shoreline Area	
Scenario		Hs (m)	Tp (s)
Present Day	3.1	2.3	4.6
2050	3.4	2.4	4.7
2100	3.8	2.5	4.8

Table 8-8. Study Area Design Wave Criteria – 50-years ARI.

Table 8-9. Study Area Design Wave Criteria – 100-years ARI.

Scenario	Water Level (m AHD)	Southern Shoreline Area		
		Hs (m)	Tp (s)	
Present Day	3.3	2.6	4.8	
2050	3.6	2.7	4.9	
2100	4.0	2.8	5.0	



8.12.2 Shoreline Erosion Modelling

Storm induced erosion modelling was undertaken on a selected profile located near the centre of the beach. This profile included the back-beach revetment structure. The location of this profile is shown in **Figure 9.1** (Appendix H), with the cross-section elevations and modelling outcomes depicted in **Figure 9.2** (Appendix H). The beach profile was based on the project survey and a characteristic sediment particle size was adopted to be D₅₀ of 0.5mm, based on the site specific sediment data. The beach at this site is a veneer of sand overlying base reef-rock. A maximum beach sand thickness of 3m was adopted at the revetment.

Storm erosion of the shoreline at the site was investigated using the SBEACH modelling system. SBEACH was developed by the U.S. Army Corps of Engineers (USACE) to investigate storm induced profile response on fine to medium grain sand beaches. It is an empirically based model that includes wave shoaling, refraction, breaking, set-up and run-up. The model can simulate a temporally varying wave breaking-point, which produces offshore bar migration. The model has been widely applied at sites all over the world and has demonstrated reasonable levels of calibration.

An SBEACH analysis was undertaken for the selected profile at the study site using the 100-years ARI cases described in **Table 8-10** as part of a 12 hours storm, peaking at 6 hours.

Planning Period	Hs (m)	Tz (s)	Peak Water Level (m AHD)
Present Day	2.6	4.8	3.3
2050	2.7	4.9	3.6
2100	2.8	5.1	4.0

Table 8-10. SBEACH Wave and Water level Inputs

The results of the SBEACH erosion modelling indicated that the foreshore erosion width (loss of beach amenity) is about 9m for all three scenarios at the 2m AHD contour, but is slightly more severe in terms of beach level loss for future cases – about 0.5m loss in beach level for present day and 1m loss in crest level, respectively. Note though that projected sea level rises will cause long term shoreline recession and that this process will affect the future amenity as outlined in the following section.



8.13 Long Term Shoreline Erosion

In order to addresses the coastal hazards at the study site, it was additionally necessary to assess the potential for long-term shoreline recession (erosion). Shoreline recession is the progressive landward shift of the average long-term position of the coastline. Recession is a different phenomenon to short-term beach erosion, although they both may be caused by the storm processes; but recession is affected also by sediment loss and sea level rise.

Recession of a sandy beach is the result of a long term and continuing net loss of sand from the beach system. This occurs when more sand is leaving than entering a particular beach compartment. Shoreline recession tends to occur when:

- The outgoing longshore transport from a beach compartment is greater than the incoming longshore transport;
- Offshore transport processes move sand to offshore "sinks" from which it does not return, for example, seaward of the reef shelf; and
- There is a landward loss of sediment by windborne transport (i.e. from onshore winds).

Because the present resort beach is held between headlands and groynes, loss by longshore transport may be negligible. From time-to-time, sand may be lost by transport seaward beyond the reef edge, but that loss is not quantifiable. Some windblown sand loss may occur, but the new revetments at a crest level of 5.6m AHD (see later) will generally prevent this process and resort management procedures will likely return any windblown sand to the beach.

8.13.1 Change in Shoreline Wave Conditions

The resort beach is held within 'headlands' formed by the existing groynes.

The most likely loss at this site is shoreline recession caused by projected sea level rise -0.8m by 2100. This would cause a landward movement of the shoreline of about 25m. This erosion will affect the amenity of the beach, but not the Erosion Prone Area Widths, as described below.

As the size and period of vessel wash is related to the speed at which vessels travel, slow vessel speed is a mitigation control that would be incorporated into the Resort Operational Management Plan. There will be a designed 'no wash zone' within 500 m of the jetty that includes signage controlling vessel speeds to 4 kts and keeping boat wash at negligible levels.



8.14 Assessment of Erosion Prone Area Widths

For the purposes of informing permissible development locations in the coastal zone of Lindeman Island Resort, the Queensland Coastal Hazard Guidelines provided by the Department of Environment and Heritage Protection were used to determine the erosion prone areas widths (EPAW). **Appendix H** provides the relevant definitions for this coastal management region.

The Queensland Coastal Hazard Guidelines (2015) outline a method for assessing erosion prone area widths (EPAWs). These widths are intended to accommodate both short-term and long-term erosion over specified planning periods. This erosion prone area width is a specified horizontal distance back from the HAT water line, and is calculated by the following equation provided in the guidelines.

 $E = [(N \times R) + C + S] \times (1 + F) + D$

Where:-

- E = erosion prone area width (metres).
- N = planning period (years).
- R = rate of long term erosion (metres per year).
- C = short term erosion from the design storm (metres).
- S = erosion due to sea level rise (metres).
- F = factor of safety.
- D = dune scarp component to allow for slumping of the erosion scarp (metres)

Due to the presence of revetments or shoreline rock formations this analysis has not been undertaken/required, but is discussed in general terms for completeness.

Planning period (N)

The planning period for this erosion prone area width assessment was to 2100. This planning period was deemed to be the most practical when taking into consideration the design life of the study site structures and appropriate level of risk. This affects the sea level rise applied to present-day highest astronomical tide (HAT).

Rate of Long Term Erosion (R)

This process was not quantified because the existing rocky shorelines and proposed modified revetments will provide a fixed erosion line.

Short Term Storm Erosion (C)

The values for short term erosion from design storms (C) are described by the outcomes of the SBEACH modelling described above.

Erosion Due to Sea Level Rise (S)

The estimated sea level rise is based on the best information currently available, and the current value adopted for calculating the erosion prone area is 0.8m by 2100 (relative to 1990 MSL). The horizontal foreshore erosion due to sea level rise (S) was determined for the 100 years planning period and adopting a nearshore seabed slope of 1V:30H – possibly too steep, but restricted by the revetments.

The factor of safety (F) of 0.4 has been adopted by DEHP for use in this equation. This means that design long term, storm and sea level rise erosion distances were scaled up by 40%.

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Dune Scarping Component (D)

Dune slump (D) was accounted for in the SBEACH modelling, but there is no dune at the resort beach, which is not proposed as the main resort beach amenity, which will be an internal beach around the main pool.

8.14.1 Erosion Prone Area Width

EPAW conditions have been determined for twelve cross-section lines at which resort buildings are closest to the shoreline – see Location Plan and Site Sections in **Appendix H**. Many of these figures include perimeter revetments or equivalent back-beach structures.

The Site Sections Location Plan shows the proposed development super-imposed over the site. Site Sections 1 to 12 include present-day HAT (2.66m AHD) and HAT+0.8m for 2100 contours and 10m and 40m setback lines.

Note, however, that at all nearshore resort facilities sites there are existing or proposed revetment works. Hence shoreline erosion will not occur landward of these structures. In this case **Appendix H** advises that the EPAW is 10m landward from the seaward side of the crest level (to be raised to 5.6m AHD from about 3m to 4m AHD, based on HAT=2.66m AHD for Cross Sections 2 to 12 – outside of the formerly proposed harbour. The HAT+0.8m point and 10m setback lines are shown on the development sections – blue lines.

This 10m setback is required for post-storm maintenance access. At this site, all revetments would be accessible from the ocean side at low tide but that site is well protected from cyclone waves. Hence building construction may be allowable closer than the default 10m from HAT + 0.8m.

SEA/GHD (2003) provides a present-day 100-years average recurrence interval (ARI) storm tide level of 3.3m AHD (excluding wave set-up) at Lindeman Island. They also advise a 0.7m increase in this level by 2100 – based on 0.48m sea level rise (SLR) and a 20% increase in cyclone intensity. Queensland Coastal Protection code requirements stipulate a 0.8m SLR by 2100, noting that DEHP stipulate that erosion prone area width (EPAW) criteria are to be based on SLR to 2100. This storm tide level is similar to that determined by Cardno as part of this study, but SEA/GHD (2003) do not include wave data in their reporting. That information was determined by Cardno for the Lindeman Island resort site. The estimated 100-years ARI peak storm Hs is 2.6m for present day climate conditions, increasing to an estimated 2.8m for 2100. These wave heights are likely to break before impinging upon shoreline revetment structures.

Incorporating these changes, the 2100 design storm tide is 4.2m AHD (including wave processes) – leaving aside the difference between 10 and 20% increases in cyclogenesis - 20% adopted by SEA/GHD. This level includes wave set-up and this component is to be removed for wave run-up calculations. Typical wave set-up may be estimated to be 10% of the incident Hs = 0.28m, leading to a 'still water level' of 3.9m AHD. Wave run-up was estimated in terms of R2 and following PIANC (1992) and adopting a roughness reduction factor of 0.6 for the revetment. R2 is the run-up height exceeded by only 2% of incident waves – hence some run-up exceeds this height. This is a commonly adopted wave run-up parameter.

Near the base of the revetments scour to 1.5m AHD may occur. This is a local area scour that affects wave propagation to the revetment toe, rather than scour at the toe directly, which may be deeper. Hence, water depth there may be 2.4m (3.9m - 1.5m). Based on Goda (2000), a beach slope of 1V:30H and a wave period



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of Tz=4 seconds, a design breaking wave height of 1.6m was estimated. The equivalent shallow water significant wave height (Hs) is then also 1.6m (Goda, 2000) in this water depth. Run-up (R2) may then occur to 6.3m AHD; some overtopping during a rare severe cyclone will occur; about 3L/m/s on average, which will not affect pool levels or the resort buildings materially. Short periods of higher overtopping rate would occur. Some sand ingress can be expected in such a storm event.

Note that rock revetments are at concept design stage only.

Figure 8-22. Cross Section Locations.





8.14.2 Jetty Precinct - Cross Section 1

Cross Section 1 describes the jetty precinct and arrival pavilion. Current floor level is shown at 4m AHD and may suffer some inundation in the 100-years ARI ocean storm at 2100; as well as some wave uplift force. At 50 and 100-years ARI, design water levels (including wave set-up) are 3.9 and 4.2m AHD, respectively. The structure is not set back more than 10m from the revetment but has not been designed for habitable use and is a refurbishment of the existing building.

8.14.3 Internal Pool Area – Cross Section 2

Appendix H describes the proposed shoreline section through the new pool area, including the Beach Hire facility and Beach Club building. Floor levels are 5.6m AHD or higher. The pool has been elevated relative to the existing 'pool' and is set back from the revetment by \approx 4m. On the beach side there is a revetment having a slope of 1V:2H and crest/top level of 5.6m AHD – higher than the existing back beach, which is about 4.5m AHD, but varies, leading down to the ocean-side beach. There is a paved walkway of width \approx 4m and a wider but lower walkway within the Beach Hut facility, which is likely to suffer damage in a future severe storm. The main issue is whether or not the elevated back-beach pool area would be overtopped in a severe future cyclone. The analysis presented above shows that some overtopping would occur in a future very severe storm following projected sea level rise. This process may transport sand and debris into the pool area.

Note that proposed structures are not set back 10m from the seaward edge of the proposed revetment.

8.14.4 Internal Pool Area – Cross Sections 3 & 4

Appendix H describes these proposed shoreline sections through the new pool area, including proposed new resort buildings. The pool has been elevated relative to the existing 'pool' and is set back from the revetment by 4m. On the beach side there is a revetment having a slope of 1V:2H and crest/top level of 5.6m AHD – higher than the existing shoreline which is about 3m AHD, but varies, leading down to the ocean-side beach. There is a paved/suspended walkway of width 5m with no detail about the 'handrail' shown on these sections. Note that the walkway projects seaward from the revetment crest about 2m and may be subject to some wave uplift forcing in the design wave and water level conditions.

The analysis presented above shows that some overtopping would occur in a future very severe storm following projected sea level rise. This process may transport sand and debris into the pool area.

Note that the main buildings are set sufficiently back (>10m) from the revetment crest, but the pool and walkway are not. This matter will need to be discussed with DEHP.

8.14.5 Internal Pool Area – Cross Section 5

Appendix H describes the proposed shoreline section through the new pool area, including a 'look-out' and beach access stairs which are part of the revetment with crest/top level of 5.6m AHD – higher than the existing shoreline which is about 3m AHD, but varies, leading down to the ocean-side beach. These steps will need to be founded on piles that extend below LAT (\approx -1.8m AHD) to provide scour resistance, or to the underlying rock, should it be encountered above LAT. The alongshore extent will also need to be considered in the context of other structures and facilities and how the steps intersect/connect/merge with them. The main issue is whether or not the elevated back-beach pool area would be overtopped in a severe future cyclone. The analysis presented above shows that some overtopping would occur in a future very severe storm following projected sea level rise. This process may transport sand and debris into the internal pool area.



8.14.6 Internal Pool Area – Cross Section 6

Cross Section 6 describes an area near the main administrative building. It shows a walkway 4m wide at a level of 5.6m AHD. It is protected by a rock revetment. Both of these structures are new. Immediately landward of the walkway there is to be a pool of \approx 16m. The proposed new residential buildings are set back more than 10m from the seaward edge of the walkway. The analysis presented above shows that some overtopping would occur in a future very severe storm following projected sea level rise. This process may transport sand and debris into the internal pool area.

8.14.7 Internal Pool Area – Cross Section 7

Cross Section 7 describes the western apartment area of the main resort apartments. It shows a walkway 4m wide at a level of 5.6m AHD. It is protected by a rock revetment. Crest level of the new seawall is 5.6m AHD. An 8m-wide pool and a 4m wide path separate the revetment from the apartments. Finished floor levels will be at 5.6m AHD and are well above the 2100 storm tide level of 4.2m AHD. The toe level and toe protection of the revetment will require detailed design at a later stage.

Note also that the new apartments are about 12m landward of the HAT+0.8m contour on the new revetment, but the pool is not, whereas 10m is required by current EPAW (from 3 February 2016) DEHP requirements. The pool area is within the EPAW. However, dispensation may be allowed because the new structures are to be constructed on/replacing existing works (pool where there is currently a revetment and pathway, apartments where there are currently accommodation buildings) foundations.

The analysis presented above shows that some overtopping would occur in a future very severe storm following projected sea level rise. This process may transport sand and debris into the pool area.

8.14.8 Internal Pool Area – Cross Sections 8 and 9

Cross Sections 8 and 9 describe the western-most of the main resort apartments. It shows a walkway 3m wide at a level of 5.6m AHD. It is protected by a rock revetment. Crest level of the new seawall is 5.6m AHD. A 3m-wide plunge pool and the 3m wide path separate the revetment from the apartments. Finished floor levels will be at 5.6m AHD and are well above the 2100 storm tide level of 4.2m AHD. The toe level and toe protection will require detailed design at a later stage. Note also that the new apartments are about 8m landward of the HAT+0.8m contour, and less than the 10m required by current EPAW (from 3 February 2016) DEHP requirements. However, dispensation may be allowed because the new buildings are to be constructed on existing foundations and there is access for maintenance from the seaward side at low water.

Note that the current plans do not appear to show any turn-back of the new revetment at the western end of the resort buildings.

8.14.9 Cross Section 10

This Cross Section describes the 'open air' bar proposed on the rocky south-western promontory. The underside of the deck will be at about 12.8m AHD and so will not be affected by wave run-up and ocean inundation. However, the structure is located less than 10m from the HAT+0.8m contour. Nevertheless, maintenance access will be available from the landward side.

Access to the 'open air' bar is understood to be from the golf course, but a shoreline board-walk has been mooted. Should that structure be along the rocky back-beach area between the bar and the main resort area it will need tidal works approval and be built at a level of about 5m AHD to avoid ocean inundation at 2100 of 4.2m AHD (including wave set-up). Wave uplift forces would need to be considered in design.

8.14.10 Cross Section 11



Cross Section 11 is on the south-western side of Lindeman Island and describes a number of isolated buildings that include individual pools on their seaward sides. They are to be constructed on rock (pers. com. Williams – Treloar, Cardno). The underside of the most seaward pool is to be at about 18m AHD and will not be affected by storm tide and wave run-up.

The most seaward edge of the pool is located about 30m landward of the HAT+0.8m contour and hence is further than the EPAW width of 10m required by DEHP for erosion resistant shorelines.

8.14.11 Cross Section 12

Cross Section 12 illustrates the 'Glamping Area' proposed in the western precinct of the resort is substantially rocky above the beach level – pers. com. Williams-Treloar of Cardno. The lowest structure is described as a Beach Node with an underside deck level of 5.4m AHD and the seaward deck area being supported on piles. Should these piles be constructed in a sandy beach section, the pile design will need to include scour, probably to \approx -2m AHD – to be defined in final design. Horizontal wave load will need to be assessed. It is understood that this structure is not proposed for accommodation.

Adopting the 100-years, 2100 storm tide level of 3.9m AHD and 100-years ARI Hs of 2.8m, the wave run-up level is 5.5m AHD. This is based on Holman (1986) and the 1:10 beach slope shown on the cross section. This relationship has been developed for sandy shorelines. Hence only minor wave run-up is likely to reach the underside of the deck in the 2100 design scenario. Nevertheless, some confinement of wave run-up under the deck will occur and a 5kPa uniform uplift design load is advised.

On the basis that much of the Beach Node structure is on the sandy back beach and would fall within the 40m default EPAW width (no revetment), this condition will need to be discussed with DEHP – it is not likely to be a 'permanent' structure and offers no accommodation – hence may be acceptable and partly sacrificial.

The results of this analysis are included in the following figures.



Figure 8-23. Erosion Prone Area Mapping – Beach Resort.



Point derived from site section

HAT + 0.8 metre (derived from site sections)

HAT +0.8m 10 metre setback

HAT +0.8m 40 metre setback

- HAT (derived from DBI Design survey plan)

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8.15 Floor Planning Levels

Additionally, wave-overtopping rates were calculated based upon EurOtop 2007 guidelines. These waveovertopping rates were determined based upon back beach levels at the site, revetment levels and associated dynamic freeboard. Potential wave run-up levels (R2) have been calculated to be up to 6.5m AHD in a future very severe cyclone (2100, 100-years ARI), and will be high enough in the 50-years planning period to have a \approx 39% probability of overtopping the perimeter revetments. The results of these analyses indicate that waveovertopping rates are relatively consistent around the main resort area at about 3L/m/s.

Therefore an uncertainty freeboard of 0.3m is incorporated into the determination of preliminary design floor levels and Cardno advise that the finished floor levels (preliminary design) need to be 5.6m AHD or higher to account for some potential plunge pool water level rise – provided that these floors are set back from the revetments.

Note that during a severe storm the water particle speeds within the overtopping waves would be from 2 to 4m/s and this constitutes another hazard – to human safety; where there is no plunge pool; and all people will need to assemble in a designated safe area or evacuated before cyclone arrival. In these circumstances, all occupants of the island may need to be assembled into one building that includes all services and food.

As part of the resort proposal, a Resort and Marina Management Plan and Warning System is proposed to be developed. This Management Plan will need to monitor Bureau of Meteorology warnings and seek advice as to when evacuation of people and boats would be required; noting that not all cyclones within the Whitsunday region will cause the need for evacuation.



8.16 Siltation

8.16.1 Marine Sediment Transportation Data

As part of the EIS investigations Cardno has undertaken a siltation analysis for the existing boat channel, which was constructed circa 1965. Survey data from 1992 (sourced from the Qld Government) and 2015 undertaken as part of this project have been analysed. The analysis of changes in depths (bed levels) over this time shows essentially no siltation over the 23 years period. If there were significant sediment transport along the reef shelf, this would have been trapped by the boat channel because it is approximately 3m deeper than the natural reef shelf. The 2015 survey data has identified that this accretion has not occurred and hence it is likely that transport rates are very low. This is confirmed by the presence of coral (refer to **Figure 8-25**), which do not live in sandy, mobile, seabed areas and the sparse reef shelf sediment.

The following provides a summary of the key findings:-

- In order to develop an estimate of the long shore sediment transport rate in the reef shelf region of the proposed safe harbor at Lindeman Island, the estimated siltation rate of the navigation channel, or rate of infilling of the channel has been prepared by comparing the average bed level of a number of cross-sections for two points in time (1992 and 2015). Surveys to datums LAT and AHD were available, the earlier one provided by Maritime Safety Queensland.
- > The surveys used for this calculation are:-
 - Drawing G700-007: Lindeman Island Jetty, Barge Ramp, Channel and Approaches Hydrographic Level and Detail Survey. Queensland Transport Maritime and Ports Division. May 1992;
 - Drawing HRP15078-004-SK002: Lindeman Island Marina Proposed Safe Harbour Marina Layout. Cardno, August 2015.

Seven cross-sections along the navigation channel were adopted (see **Figure 8-26**), whereby their position in space on both drawings was matched as well as possible. Subsequently, an average depth was determined for each of the seven cross-sections, for both 1992 and 2015. The results have been tabulated in **Table 8-11**.

Figure 8-25. Coral in the existing boat channel.





Cross-Section Number	Avg Level 1992 (mLAT)	Avg Level 2015 (mLAT)	Level Difference (m) (>0 = Siltation)
1	-2.4	-2.2	+0.2
2	-2.6	-2.4	+0.2
3	-2.3	-2.6	-0.3
4	-2.4	-2.3	+0.1
5	-2.5	-2.4	+0.1
6	-3.0	-3.0	+0.0
7	-3.0	-2.8	+0.2

Table 8-11. Changes in Bed Level in the Existing Harbour Entrance Channel.





The level differences between the 1992 and 2015 situation presented in **Table 8-11** fall well within the margins of survey and data extraction error (considering the method applied). It is reasonable to assume that negligible infilling (if any) of the navigation channel has occurred over the past 23 years between the two surveys and most likely the 50 years since construction in circa 1965. No maintenance dredging has been required.

This condition was confirmed by on-site inspection of the bed of the navigation channel that showed that there was about 0.1m of fine sediment on the bed. Over a period of about 50 years since construction, this amounts to an average deposition rate of about 2mm/year – consistent with the outcome of the analysis of survey data.

Hence longshore sediment transport on the reef shelf is likely to be very small, or negligible.

Generally, peak reef shelf current speeds are too high for sediments to settle on the reef shelf and this characteristic may contribute to the coral abundance at this site.



8.17 Sediment Plumes

8.17.1 Land Based Pollution

As the freshwater streams on Lindeman Island are ephemeral, limited opportunity exists to undertake baseflow monitoring. Despite this, two rounds of event based testing were able to be conducted at the site within the waterways. For the March 2016 event, there were two rounds of sampling were conducted for the same event due to issues with transporting samples from the island. Given the recommended holding times, only the later results have been referenced. Based on the event based Water Quality Objectives, the current water quality meets the specified objectives. In June, at site LIND06 there was an elevated level of TSS.

Further additional water quality testing is proposed at the terrestrial sites prior to the commencement of construction. The *Water Quality Guidelines for the Great Barrier Reef Marine Park* (2010) describe the concentrations and trigger values for sediment, nutrients and pesticides that have been established as necessary for the protection and maintenance of marine species and ecosystem health of the Great Barrier Reef. Parameters that are not listed default to the Queensland Water Quality Guidelines, which in turn default to the Australian and New Zealand Guidelines for Fresh and Marine Water Quality. Gap Creek and several other small ephemeral freshwater streams traverse the site and discharge to the ocean. Due to the steepness of the island these freshwater streams have negligible interaction with tidal waters, because the tidal zone on the island is limited to the shoreline area.

The stormwater and water management strategy for the Lindeman Island Great Barrier Reef Resort aims to reduce the pollutant load being discharged to streams that drain to the Great Barrier Reef Marine Park. Specifically, stormwater and water management strategies will be adopted that:

- Re-use rainwater, reducing potable and irrigation water demand and stormwater pollutant loads;
- Treat and re-use wastewater for non-potable uses on site;
- Minimise the potential sources of stormwater pollutants;
- Treat storm water runoff to remove sediment and nutrient load;
- Replicate existing flow patterns;
- Reduce potential for scour and erosion; and
- Integrate open space with stormwater drainage corridors and treatment areas to maximise public access and recreation and preserve waterway habitats and wildlife corridors

8.17.2 Potential Acid Sulfate Soils

Due to the nature of the marine environment it is not expected that any acid sulfphate soils will be present. Prior to the undertaking of any dredging it is proposed that core samples be undertaken to confirm this. If acid sulphate soils are identified then they will be managed in accordance with an Acid Sulphate Soil management plan.



8.18 Potential Impacts and Mitigation Measures

The following table provides an assessment of the key potential impacts and mitigating measures associated with the project.

Table 8-12.	. Risk assessmen	t matrix –	coastal	processes.

Potential Impact	Significance of Impact: Unmitigated	Mitigation Measure			Significance of Impact: Mitigated
		Design	Construction	Operation	
Erosion and Inun	dation				
Shoreline Erosion	Medium (5)	 New and extended revetments prevent shoreline erosion and the site allows 10m+ access for maintenance. All buildings have been sufficiently set back behind the revetment structures and there is adequate post storm access for maintenance 	-	 Marina operation is to include annual hydrographic surveys, until it has been confirmed that siltation is low. 	Low (1)
Ocean Inundation and consequential damage to resort infrastructure	Medium (5)	 New and extended revetments have a crest level at 5.6m AHD which accommodates predicted wave overtopping to 2100. 	 Adoption of appropriate floor levels that address storm tide inundation and sea level rise. Revetments constructed in accordance with Australian Engineering Standards. 	 Revetment condition to be monitored to ensure satisfactory performance. 	Low (2)
Changes to Resort Beach profile	Low (3)	 Storm erosion is not major, but Sea Level Rise will have a long term effect. 	•-	 Beach condition to be monitored to ensure satisifcatory performance 	Low (2)

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8.19 Summary

A range of data collection tasks (e.g. current data) and numerical modelling has been undertaken in order to develop the data necessary to assist the functional design of shoreline facilities as part of the EIS process. These investigations have been undertaken being cognisant of projected climate change parameters, mainly in terms of sea level rise and intensification of cyclones, and has been used to optimise the project design so that potential deleterious effects were ameliorated early in the design process.

Coastal Processes

- There was no recorded wave data for Lindemand Island. Hence, wind-wave modelling was required for this investigation in order to develop the required design wave parameters used in foreshore erosion modelling, and shoreline-revetment wave run-up investigations;
- A wind rose prepared for Hamilton Island (closest weather station) shows that the majority of winds come from the east to south quadrant; and from this it can be deduced that the southern shore of the study site is more commonly affected by local wind waves than the western shore; noting that Shaw Island limits the fetch of these waves. These south-easterly waves are generally small; in the order of 0.3m, but this is too high for comfortable and safe embarkation and disembarkation;
- No site-specific current data was available for this project. Hence two campaigns of current data collection, one in October 2015 (1.5 months of data) and the other in the period from mid-January to mid-May in 2016 (≈4.5 months of data). In both cases ADCP transects were undertaken along three lines near the proposed resort site at times of spring tides;
- Wave, tide and storm surge/current investigations undertaken throughout the study area required application of high level model systems (e.g. Delft3D and SWAN) capable of simulating a range of processes. Median wave height (Hs) is about 0.25m and the 1-year wave height (Hs) is about 0.9m;
- There are typically two high tides and two low tides every day at the site with AHD is 2.28m Lowest Astronomical Tide;
- Investigations confirm that this is a very low siltation site with sediment sampling undertaken in October 2015 finding scattered thin patches of poorly sorted sediments, which is typically of D₅₀ of 0.5mm or greater; and
- The sediment sampling findings identified that there were no sites that had Tributyltin (TBT) or heavy metal concentrations above the ANZECC guidelines;

Coastal Erosion and Sea Level Rise

- Erosion Prone Area Widths have been determined for twelve cross-section lines at which resort buildings are closest to the shoreline;
- New and extended revetments prevent shoreline erosion and the site allows 10m+ access for maintenance. New and extended revetments have a crest level at 5.6m AHD which limits wave overtopping to 2100;
- The results of the SBEACH erosion modelling indicated that the foreshore erosion width (loss of beach amenity) is about 9m at the 2m AHD contour, but is slightly more severe in terms of beach level loss

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for future cases – about 0.5m loss in beach level for present day and 1m loss in crest level, respectively. projected sea level rises will cause long term shoreline recession;

- Investigations have shown that changes in wave heights are small on the resort beach, except at its eastern end where they will be reduced. During 100-years ARI storm, Hs may increase by 0.1m seaward of breakwaters. The wave caused seabed shear stresses will not exceed 18N/m² anywhere for existing and post-construction cases at 1-year ARI, but does exceed this selected threshold for existing and developed cases for the 100-years ARI (equivalent) cyclone wave case, with some increase caused by the safe harbour breakwater; and
- An uncertainty freeboard of 0.3m is incorporated into the determination of preliminary design floor levels and Cardno advise that the finished floor levels (preliminary design) need to be 5.6m AHD or higher to account for some potential plunge pool water level rise – provided that these floors are set back from the revetments.