United Nations Development Programme

Concept Design Report

Tuvalu Coastal Adaptation Project

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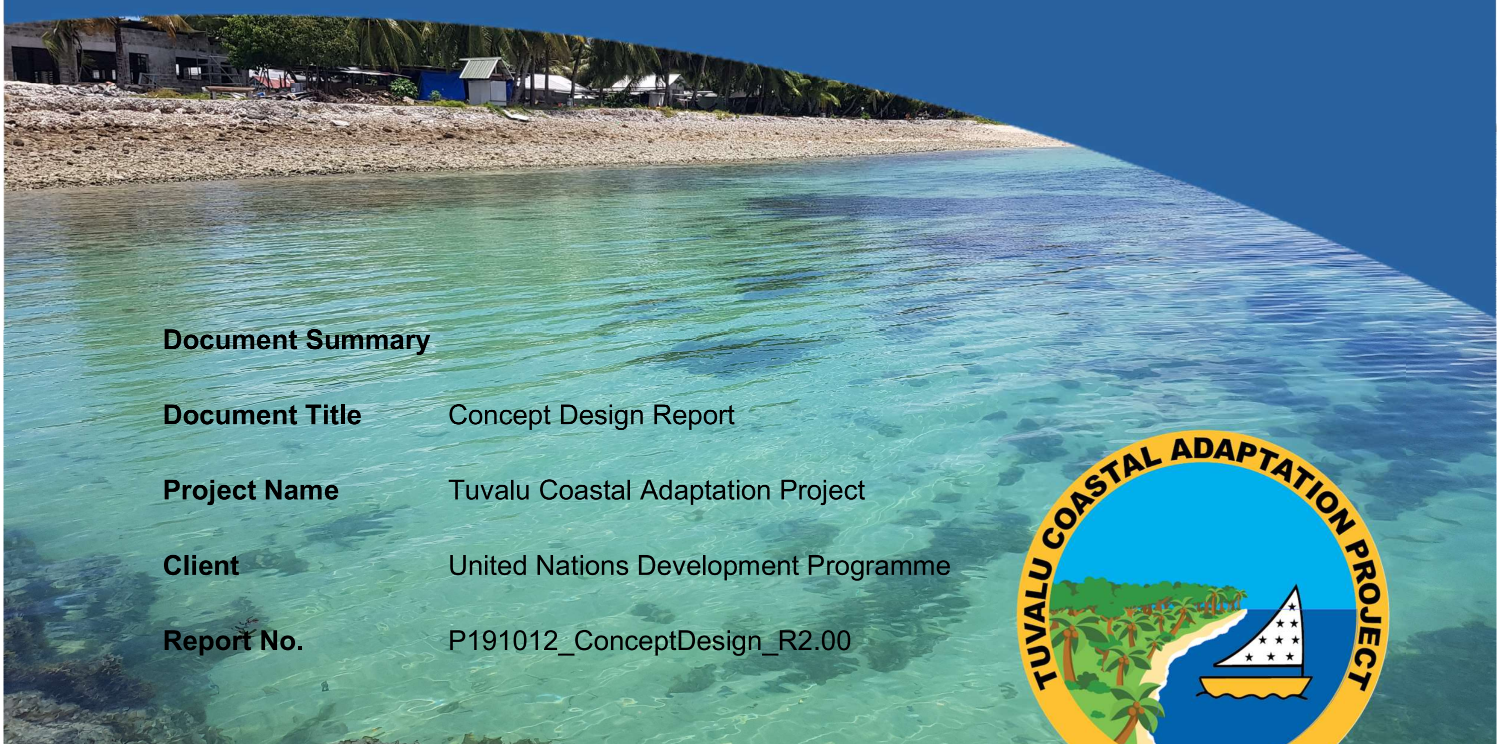

Document Summary

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Table of Contents

List of Figures

List of Tables

1. Introduction

Tuvalu is the fourth smallest nation in the world in land area with a population of just over 10,000. It is one of the most vulnerable countries in the world to the impacts of climate change and particularly sea-level rise and the possibility of intensifying storm events. Recent cyclones have caused population displacement, significant loss and damage of infrastructure as well as destruction of agricultural resources, contamination of ground water and changes in shoreline systems. Such impacts negatively affect the wellbeing of communities and long-term sustainable development aspirations. In response to this increasing challenge, the Government of Tuvalu (GoT) and the Green Climate Fund (GCF) have jointly committed US$38 million for the Tuvalu Coastal Adaptation Project (TCAP).

1.1 Project background

TCAP was approved in June 2016 and the project implementation commenced in September 2017. The project will build coastal resilience which is an urgent national priority. This project is aimed at addressing the financial and technical capacity constraints at all levels and to increase community awareness. The project will run until September 2023 and has 3 main outputs:

Output 1: Strengthening of institutions, human resources, awareness and knowledge for resilient coastal management.

Output 2: Vulnerability of key coastal infrastructure is reduced against wave induced damages in Funafuti, Nanumea and Nanumaga.

Output 3: A sustainable financing mechanism established for long-term adaptation efforts.

The following body of work is to progress Output 2, which has two main task activities:

Activity 2.1.

- Coastal protection design.
- Site-specific assessments undertaken in all islands in a participatory manner.

Activity 2.2.

- Coastal protection measures implemented.

Activity 2.2 cannot proceed until aspects of work associated with Activity 2.1 has been finalised. At the time of writing, the site-specific assessments and ESIA have been completed. The following activities have been undertaken to inform the coastal protection design:

- Site visits to all project sites undertaken in November 2017 and subsequent desktop analysis of available data and testing of different coastal protection solutions undertaken through the first half of 2018.
- A feasibility study was realised recommending desktop-based concept design solutions to progress the GCF funding application
- Detailed analysis of coastal hazards exposure was conducted throughout Funafuti in 2018 in several reconnaissance visits by the Chief Technical Advisor (CTA).
- Initial concepts developed for the funding application were revised by the CTA to be site and project specific.

- At the 2nd TCAP Board Meeting (18th May 2018) TCAP shared the revised concepts for coastal infrastructure design in all three locations. The board approved design revisions.
- Two technical reports (TCAP, 2017 & TCAP, 2018) have been released by the TCAP Project Management Unit (PMU) that detail the development of coastal adaptation concepts on the three project sites of Funafuti, Nanumea and Nanumaga.
- GoT requested changes to the Nanumea designs in August 2018 and GoT / TCAP negotiated the details of those designs through to March 2019.
- A meeting between the funding agency (GCF), GoT, TCAP/UNDP was undertaken to progress and finalise the designs in May 2019. GCF were requested to review the revised designs and grant approval to proceed.
- A subsequent cabinet paper was released in May 2019 (shown below) summarising the status of the design of coastal infrastructure (**Output 2**) and seek governmental approval as well as approval from the GCF as the updated concepts were a significant departure than those proposed in the original feasibility study used for funding application.
- A national Light Detection and Ranging (LiDAR) survey was undertaken throughout Tuvalu's nine islands in August 2019 resulting high resolution land height and shallow (< approx. 40m) sea floor data, providing key baseline data to support wave hazard analysis and detailed infrastructure design.
- A coastal engineer was engaged in October 2019 to finalise and confirm the concept designs and progress to detailed design, procurement and implementation phases.
- The coastal engineer undertook an initial site visit to Funafuti with the CTA and TCAP team. The trip was to coincide with the TCAP board meeting on 28th October 2019
- A preliminary Environmental and Social Impact Assessment (ESIA), geotechnical investigation and Coastal Hazard and Risk Assessment was finalised in August 2020.
- The Kaupule of each island, the ministry of finance and GCF were provided the finalised concept designs and updated on the project and subsequently endorsed the designs and project plan.

1.1.1 Cabinet approved designs

The following is a summary of the existing concept designs as part of Output 2 of TCAP. These designs were approved by the TCAP Board in May 2019 and followed GCF approval.

Funafuti

A large-scale reclamation project is recommended for the Vaiaku foreshore of Funafuti. The proposed TCAP reclamation will start from the northern boundary of Queen Elizabeth Park reclamation (QEP) and extend to the northern Tausoua Beach groyne. It will extend seawards to a similar extent as QEP and its overall dimensions will be approximately 710m in length x 100m wide giving a total area of approximately 7.1Ha (17.5 acres). The proposed layout of the reclamation is presented in Figure 1. Other details of the protection design include:

- The reclamation will be filled to a height at least 2m above the highest measured sea level at Funafuti (HAT highest astronomical tide) and will have areas up to 4m above HAT. The surface will be sloped to ensure good drainage and will have a natural raise berm and buffer at the seaward edge.

- The lagoon foreshore of the reclamation will be hardened with a well-designed and built revetment (seawall).

- The reclamation will incorporate a small boat harbor at the northern end.
- Approximately, 350,000m^3 of fill is required for the proposed reclamation.

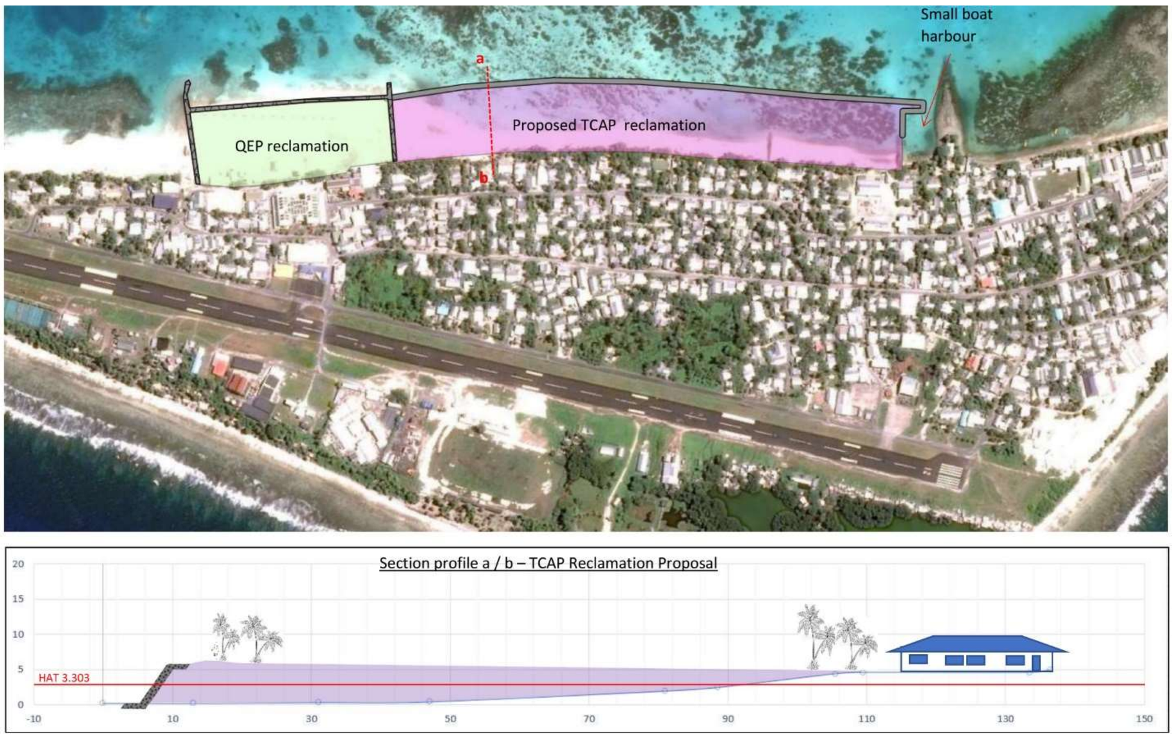


Figure 1: Proposed layout and section profile of the TCAP reclamation in Funafuti (Source: TCAP, 2019)

Nanumea and Nanumaga

A mixture of hard engineering and “softer” engineering “berm top barriers” (BTB) are proposed for both Nanumea and Nanumaga. An example BTB layout is presented in Figure 2. Details of these designs are as follows:

- BTB will be constructed to protect approximately 1.4km (Nanumea) and 700m (Nanumaga) of shoreline adjacent to the main village and important infrastructure.
- A suitable hard revetment will be constructed at the site of the church compound reclamation on Nanumea, to protect the reclamation from further wave damage.
- Hard engineering maybe implemented in the area adjacent to the boat channel / church compound on Nanumaga.
- Approximately 2,500m^3 (Nanumaga) and 4,700m^3 (Nanumea) of material is required for the BTB’s and can be safely sourced from the storm deposits (Cyclone Pam) available on both islands.

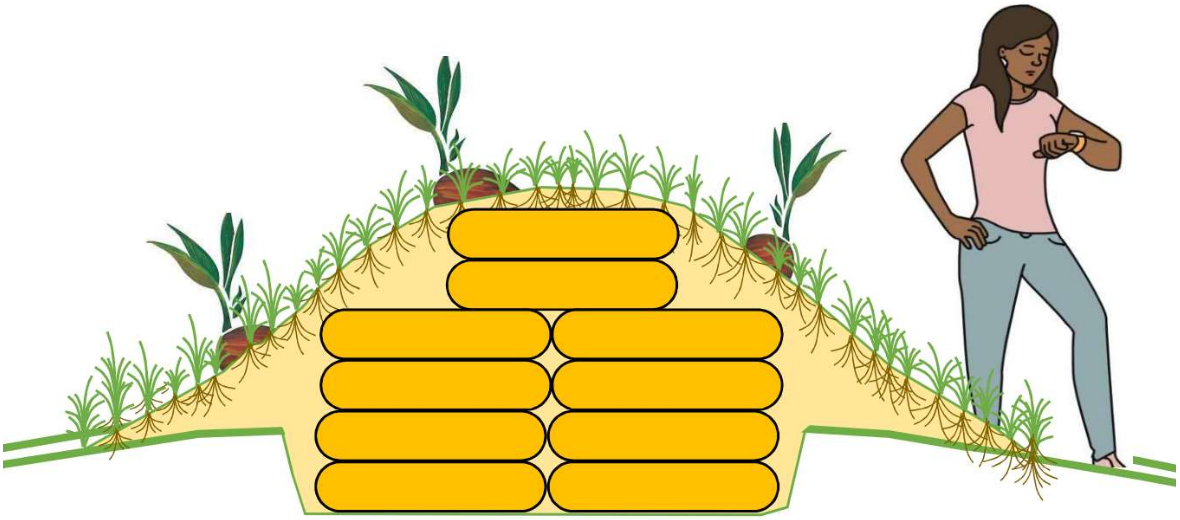


Figure 2: An example of the BTB (berm top barrier) proposed as part of the TCAP coastal protection design for Nanumea and Nanumaga (Source: TCAP, 2019)

1.2 Objectives

The objective of the TCAP project is to enhance coastal resilience in Tuvalu by implementing a robust strategy of coastal measures in three of the country's nine islands. As such, the project is expected to:

1. Implement appropriate coastal engineering approaches adjacent to high value, vulnerable shorelines to ensure they are more resilient to the effects of increased wave hazards. This investment will be undertaken on the islands of Funafuti, Nanumea and Nanumaga, and directed along areas with high concentration of residences and social and economic assets;
2. Strengthen institutional and community capacities for sustaining and replicating project results in other areas of Tuvalu.

The objective of this report is to finalise the concept designs and solidify the rationale behind their selection. This report will produce a preliminary Basis of Design (BoD) for each site to inform the subsequent detailed design of the concepts for tender.

1.3 Scope of this report

The scope of this report is to:

- Collate and centralise the body of work used to determine the updated coastal adaptation infrastructure concepts.
- Define a preliminary Basis of Design (BoD) for each project site.
- Refine concept designs based on updated data collected, financial information and social learnings.
- Refine project costs based on design refinement as well as early contractor engagement.
- Determine approvals pathways needed to complete the construction of the infrastructure

2. Data and literature review

The following datasets, papers, scientific literature and design reports have been provided as part of the project to gain an understanding of the physical and environmental characteristics, past coastal engineering projects and the environmental and social values of each of the project sites. Table 1 shows some of the sourced datasets that have been used to inform the concept design within this report. Scientific papers, journals and technical reports that have been referenced are cited within the report and summarised in the References section appending this report.

There are nine operational meteorological stations in Tuvalu at the time of writing which have been used to inform meteorologic and oceanic (metocean) analysis herein. Multiple observations within a 24-hour period are taken at four stations (Nanumea, Nui, Funafuti and Niulakita) and there are single observation rainfall stations at five locations: Nanumaga, Niutao, Nukufetau, Vaitupu and Nukulaelae. The Funafuti station has the longest record, with rainfall data available from 1927 and air temperature data from 1933 (BoM & CSIRO, 2011).

Table 1 Datasets sourced for the TCAP project

Dataset	Owner, year	Summary
Tuvalu LiDAR	TCAP, 2019	High resolution LiDAR imagery and point cloud of all Tuvalu islands down to -40m (approx.) collected by Fugro.
Funafuti and Vaitupu sea level data	BoM, 2019. SEAFRAME	Hourly sea level data from 1973 to 2019
Funafuti rainfall data	Tuvalu Met Dept, 2019	Hourly rainfall data taken from the Funafuti Met Station from 1974 to 2020
CAWCR wave hindcast	CSIRO, 2020	Hindcast wave and wind data from the CAWCR South Pacific regional model from 1979-2020 for 3 extracted (ocean) points offshore of Nanumea, Nanumaga and Funafuti
SPC wave hindcast	SPC, 2020	Hindcast wave data from SPC's South Pacific model from 1979-2020 for 3 extracted (ocean) points offshore of Nanumea. The model is driven by ERA5 wind data.
Funafuti sediment samples	SPC, 2015	100 jet probes taken in Funafuti Lagoon as part of sand resource/UXO study, includes PSD.
Nanumea and Nanumaga sediment samples	SPC, 2020	Boreholes, core and auger test samples from site visit in Jan 2020 to outer islands

Dataset	Owner, year	Summary
Funafuti Lagoon non-directional wave data	DOWHA, 2017	20day wet season deployment, 2km offshore of Vaiaku shore in 2017 of non-directional wave gauge
Funafuti ocean-side wave data	OCEANOR, 1992	2year deployment off the east coast of Fongafale 1990-1992 of non-directional Wavebuoy
Funafuti Lagoon directional wave data	JICA, 2009	3 separate 2week wet season deployments, 2km offshore of Vaiaku shore from Nov 2009 to Mar 2010 of directional wave ADP

2.1 Recent and concurrent projects

A number of coastal protection, port and infrastructure projects have and continue to occur sporadically across the nine islands of Tuvalu. Generally, these projects are aid-funded and are initiated in response to storm damage and inundation events which threaten the livelihoods and wellbeing of the small communities or critical infrastructure on each island. As a result of these projects, each island and shoreline are at different levels of vulnerability to coastal and climate hazards.

As seen in the previous section there is generally little site-specific data that exists for Tuvalu and without the evidence of remnant structures, there is also little information regarding previous infrastructure projects. Being the administrative capital and having the largest population, Funafuti has the greatest amount of site-specific data and details of previous coastal works. There have been several coastal protection studies and works undertaken for the study site on Funafuti (Vaiaku foreshore). Of most relevance to the proposed TCAP concept is the recent reclamation works undertaken at the Queen Elizabeth Parklands (QEP) in preparation for the Pacific Leaders Forum in 2019. The Government of Tuvalu fast-tracked a design to an approximate 300m length of foreshore (to 90m offshore) running from the Vaiaku Wharf towards the east. The reclamation was opportunistically undertaken utilising a large Cutter-Suction Hopper Dredge (The Amity, Hall Contracting Ltd) which had been undertaking nearby re-nourishment works of the WWII Borrow Pits on the eastern arm of Fongafale Islet. The QEP reclamation made use of an identified large reserve of usable lagoon sediments (up to 24Mm^3) located offshore of the project site. The QEP reclamation abuts the proposed reclamation site and as such has very similar metocean and geotechnical conditions as the project site.

Recent (and ongoing) construction programs in Tuvalu which may have linkages to TCAP are presented in Table 2. Cross-institutional cooperation is generally advocated and where possible, TCAP will aim to work with GoT and other organisations to co-ordinate concurrent projects for efficiencies.

Table 2: Recent and ongoing construction projects in Tuvalu with possible linkages to TCAP

Project	Date	Description and possible linkages to TCAP	Status
Outer Island Marine Infrastructure Project	2017-present	World Bank-funded project to create a boat harbour and port on Nanumaga Island. The project has had several design iterations and location choices. Depending on the final location, there may be cross-over or concurrent works with TCAP proposed options.	Ongoing, siting has now been changed with a possible location to the southwest shore of Nanumaga
Borrow pit remediation project	2018	NZ MFAT-funded project to infill the WWII borrow pits located in the eastern arm of Fogafale which at the were used as garbage dumps. Lessons learnt regarding the sand resources and UXO searched will aid TCAP	Completed 2018
QEP Reclamation	2018	GoT-funded reclamation adjacent to the TCAP project site on Fogafale.	Completed 2018
Nukufetau coastal protection	2016	A Geotextile Synthetic Container (GSC) seawall that was constructed on Savave island on Nukufetau as part of the NAPA project. Construction methodology and building materials will be similar to those proposed for TCAP.	Completed 2017

3. Climatic and geophysical setting

3.1 Tuvalu

Tuvalu is made up of a group of nine islands; three reef islands and six true atolls spread out between the latitude of 5° to 10° south and longitude of 176° to 180°, west of the International Date Line, see Figure 3. The three islands of the TCAP project include Nanumea, Nanumaga and Funafuti. The climate of the Pacific Island region is entirely ocean dependent.

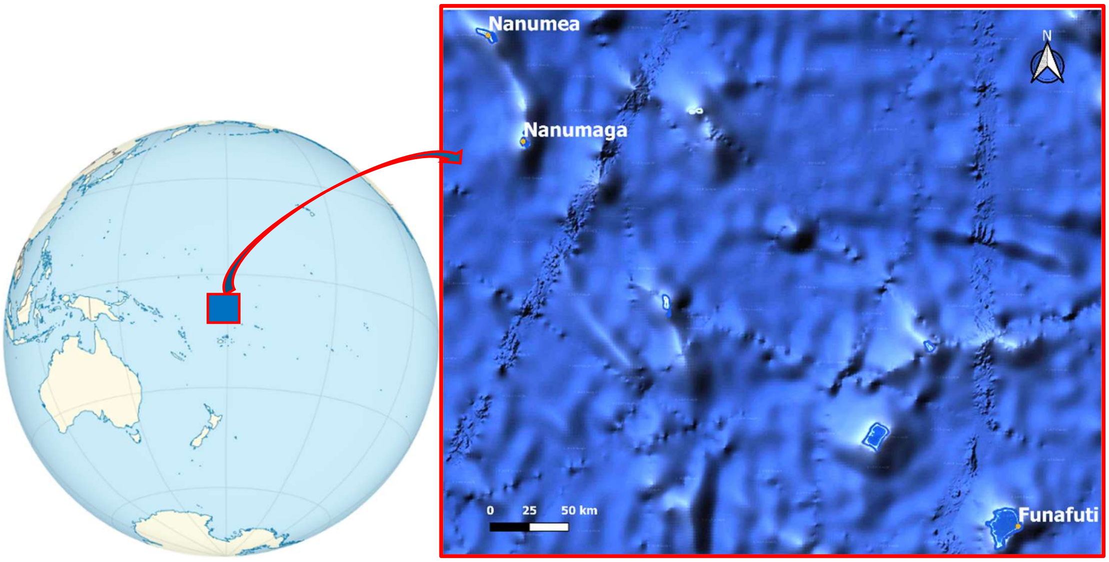


Figure 3Tuvalu geographic setting

3.1.1 Seasonal variation

The northern atolls of Tuvalu lay on the edge of the Inter-Tropical Convergence Zone (ITCZ) or *the doldrums* which is a band of latitude running approximately 5° either side of the equator where there is often little surface wind. The area to the south of the ITCZ where Tuvalu's atolls and islands are found are dominated by the trade winds which converge at this zone, originating from the southeast quadrant and are most frequent between June and August. Between December to March, winds between the west and north usually equal or exceed the south / easterlies in frequency.

The lower latitude islands and atolls of Tuvalu, including the nation's capital Funafuti are situated near the Indo-Pacific Warm Pool (Figure 5).This large body of warm water is characterised by permanent surface temperatures greater than 28 °C . Thunderstorm activity occurs year-round and the area is sometimes referred to as the 'dilution' basin due to the high incidence of tropical rain. The average monthly rainfall in this area is approximately 25% higher in Tuvalu's lower atolls as compared to the northern reaches (Funafuti 200mm/month compared to Nanumea 160mm/month). Tuvalu has two distinct seasons – a wet season from November to April and a dry season from May to October (Figure 4), with rainfall almost doubled between June and December.

The intensity of rainfall as well as the duration of Tuvalu's wet season is affected by the movement, position and strength of the South Pacific Convergence Zone (**Table 4**). This band of heavy rainfall is caused by air rising over warm water where winds converge, resulting in thunderstorm activity. It extends across the South Pacific Ocean from the Solomon Islands to the Cook Islands and is most intense during Tuvalu's wet season (BoM & CSIRO, 2011).

The position of the SPCZ can change depending on seasonal, interannual, and possibly longer (inter/decadal) timescales. The interplay of the SPCZ and the El Niño-Southern Oscillation (ENSO) considerably affect Tuvalu's climate from year to year. ENSO is a natural climate pattern that occurs across the tropical Pacific Ocean and affects weather around the world. There are two extreme phases of the El Niño-Southern Oscillation: El Niño and La Niña, Figure 6. There is also a neutral phase. In the southern islands of Tuvalu (such as Funafuti), El Niño events tend to bring wetter, warmer conditions than normal, while La Niña events usually bring drier, cooler than normal conditions. This is likely due to the warmer ocean temperatures around Tuvalu in El Niño years (BoM & CSIRO, 2011). There is a significant inverse relationship between rainfall and the El Nino/Southern Oscillation Index leading the rainfall response by several months.

In Funafuti, the capital of Tuvalu, there is little variation in temperature throughout the year. The maximum temperature is between 31–32°C and the minimum temperature between 25–26°C all year round. Air temperatures are strongly tied to the ocean temperatures surrounding the islands and atolls of the country, Figure 4. With the high levels of sunshine evapotranspiration rates are also being high, but in most years in the southern atolls, rainfall is sufficient to meet these requirements. Soil moisture deficits are most likely to occur in the northern atolls.

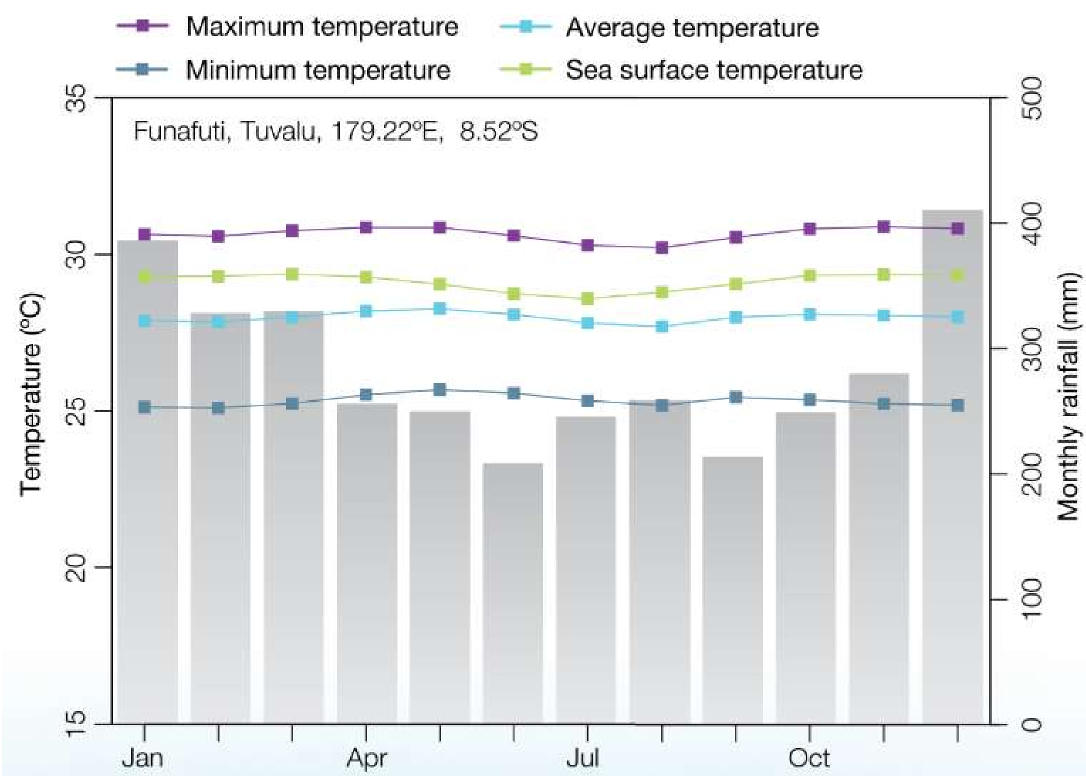


Figure 4 Seasonal rainfall and temperature at Funafuti (source: BoM & CSIRO, 2011).

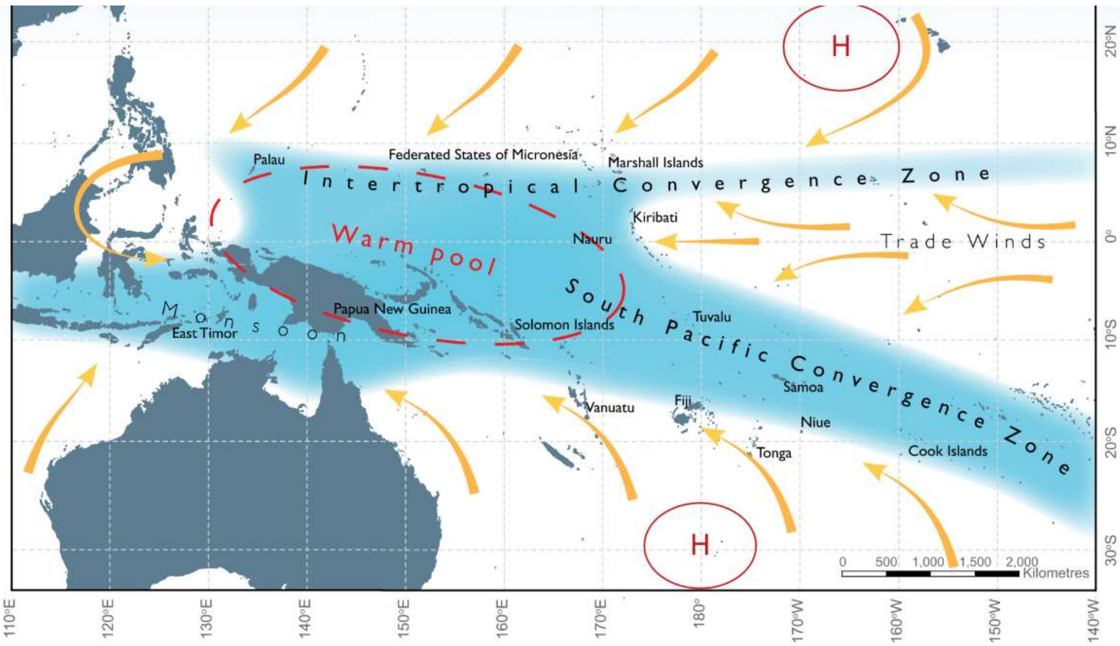


Figure 5 The average positions of the major climate features in November to April. The arrows show near surface winds, the blue shading represents the bands of rainfall convergence zones, the dashed oval shows the West Pacific Warm Pool and H represents typical positions of moving high pressure systems. (source: BoM & CSIRO, 2011).

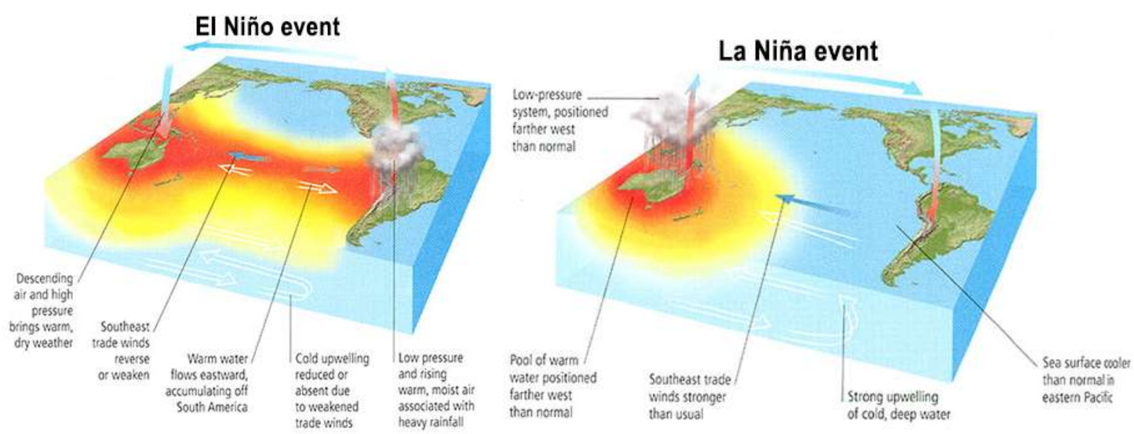


Figure 6: Diagram of El Niño and La Niña Events (source: Mann and Kump, 2016).

3.1.2 Tropical cyclones

Wind speeds over Tuvalu's surrounding ocean average about 10kts year-round and strong winds are not common, wind speeds greater than 20kts only occur on-average around 3% of the time. Tropical Cyclones occasionally develop near to Tuvalu but rarely track through Tuvaluan waters. Tropical cyclone frequency in Tuvalu tend to only occur in the wet season (between November and April) as they are fuelled by heat stored in the upper ocean during the hottest months. The northern atolls and islands are at lower risk of cyclone crossing as cyclones do not occur within 5° of the equator due to the weakness of the "Coriolis Force", a rather subtle effect of the earth's rotation. ENSO affects the frequency of tropical cyclones and location tracks in the South Pacific. During El Niño periods, a northward and eastward shift of tropical cyclone genesis is observed due to a shift of the SPCZ .

In the 43-year period between 1972 and 2015, 26 tropical cyclones passed within a 300 km of Funafuti, an average of just under one cyclone per season. The number of cyclones varies widely from year to year, with none in some seasons but up to three in others. Over this period, cyclones occurred more frequently in El Niño years. Two recent cyclones have had major impacts on the TCAP project islands: TC Pam in 2015 and TC Tino in 2020. Both cyclones formed and passed at significant distance to Tuvalu. Nevertheless, large waves and strong winds affected all three project islands.

Re-analysis of the weather systems at the time of the two tropical cyclones shows that it was the converging flow generated by the synoptic pressure gradient between the cyclone and an abnormal higher pressure belt near the equator that sets up across the ocean fetches to the north and resulted in large seas and swells at Tuvalu (see Figure 7 and Figure 8). This rare combination of events was found to have a strong correlation between extreme El Niño periods that allows cyclone genesis in higher latitudes and causes higher air pressure in the western South Pacific.

A detailed cyclone analysis in respect to wave heights on the ocean shores of the Tuvalu islands has been undertaken in the Nanumea Concept Design Report (Appendix D). The analysis showed that historically extreme wave heights (99.5th percentile) at Nanumea occur exclusively during the passage of nearby tropical cyclones (within 1,000km) and periods of extreme El Niño conditions.

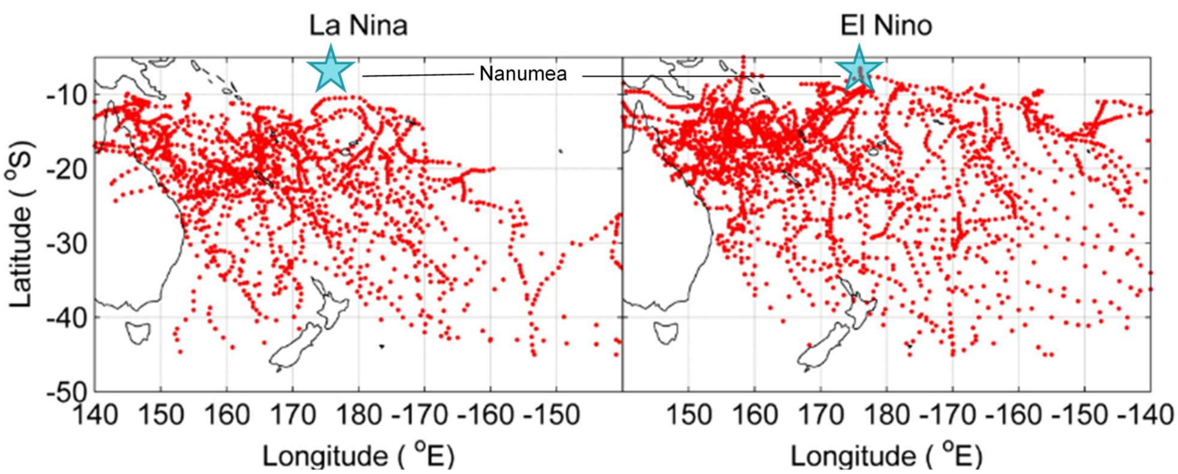


Figure 7: Observed tropical cyclone tracks between 1967 and 2009 for La Nina and El Niño climate conditions (adapted from Stephens, 2014).

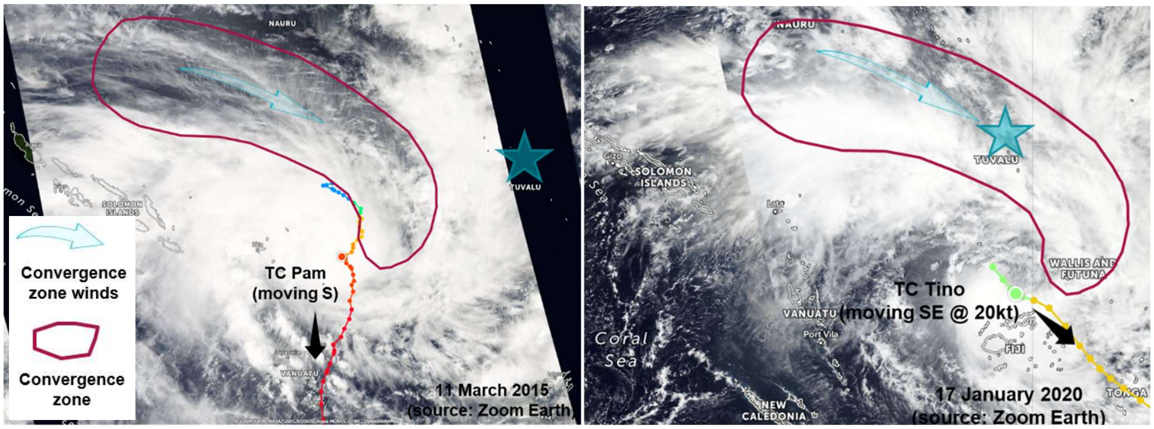


Figure 8: Satellite imagery showing synoptic weather system during formation of TC Pam (left) and TC Tino (right). The cyclone tracks are shown in the coloured dotted lines and the relative location of Nanumea is indicated by the blue star.

3.1.3 Climate change predictions

Climate impacts almost all aspects of life in Tuvalu. Understanding the possible future climate of Tuvalu is important so people and the government can plan for changes (BoM & CSIRO, 2011). The Pacific-Australia Climate Change Science and Adaptation Planning (PACCSAP) Program report (PACCSAP, 2014) describes large-scale climate processes, variability and extremes in the western tropical Pacific. The report also describes projections for the 21st century based on Coupled Model Intercomparison Project (Phase 5) (CMIP5)-based global climate model (GCM) projections for individual countries. The projections are aligned with greenhouse gas and aerosol concentration scenarios and terminology adopted by the Intergovernmental Panel on Climate Change (IPCC) 2013 report; Climate Change 2013: The Physical Science Basis. Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change.

PACCSAP (2014) details observed trends over the past 30-60 years, including air temperature, rainfall and tropical cyclones. The report goes on to make climate projections for atmospheric and oceanic variables. The PACCSAP reports are exported to be updated following the next iteration of the IPCC report in 2022. A summary of the country report for Tuvalu as it relates to the infrastructure developed in the TCAP project is provided below.

Sea Level

Variations in sea level and atmosphere are inextricably linked. The climate of the Pacific Island region is entirely ocean dependent. When the warm waters of the western equatorial Pacific flow east during El Niño, the rainfall, in a sense, goes with them, leaving the islands in the west in drought. El Niño's impact on sea level is mostly felt along the SPCZ, because of changes in the strength and position of the Trade Winds, which have a direct bearing on sea level, and along the equator, due to related changes in ocean currents. Outside these regions, sea levels are influenced by El Niño, but to a far lesser degree. The convergence of the Trade Winds along the SPCZ has the effect of deepening the warm upper layer of the ocean, which affects the seasonal sea level. Tuvalu, which is in the heart of the SPCZ, normally experiences higher-than-average sea levels early each year when this effect is at its peak.

The monthly maximum mean and minimum water level based on hourly values from the Fongafale (Tuvalu Port) tide gauge is shown in Figure 9. Over the period since 1993, there were

38 events (hours) with sea levels greater than 1.25m above mean sea level (MSL), the highest being a 1.39m MSL event which occurred in February 2006. A clear increasing trend in the observed monthly sea level statistics (mean, maximum and minimum) can be seen over the 25year recording period, Figure 9.

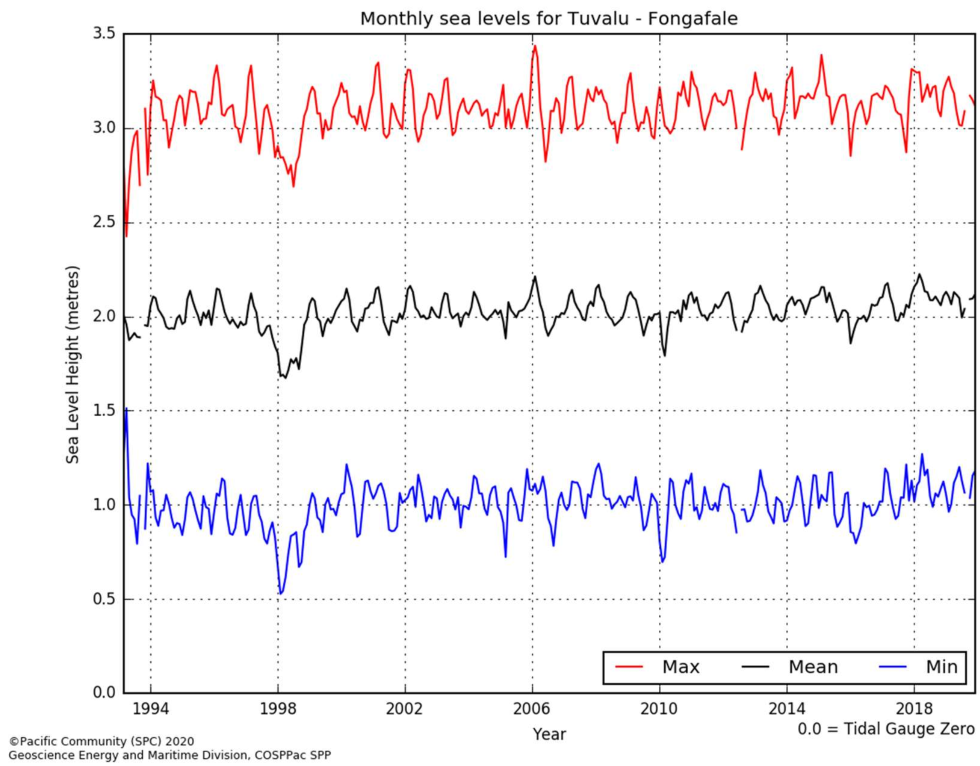


Figure 9 Monthly maximum, mean and minimum water level data from the Funafuti Port tide gauge (source: BoM, 2019).

The datum of the Tuvalu Port tide gauge is regularly checked by surveyors as part of the geodetic monitoring undertaken by the PSLGM project. This work is undertaken by Geoscience Australia. The latest survey report shows that between 2003 and 2017 the tide gauge benchmark has lowered by about 19mm, see (GA, 2018).

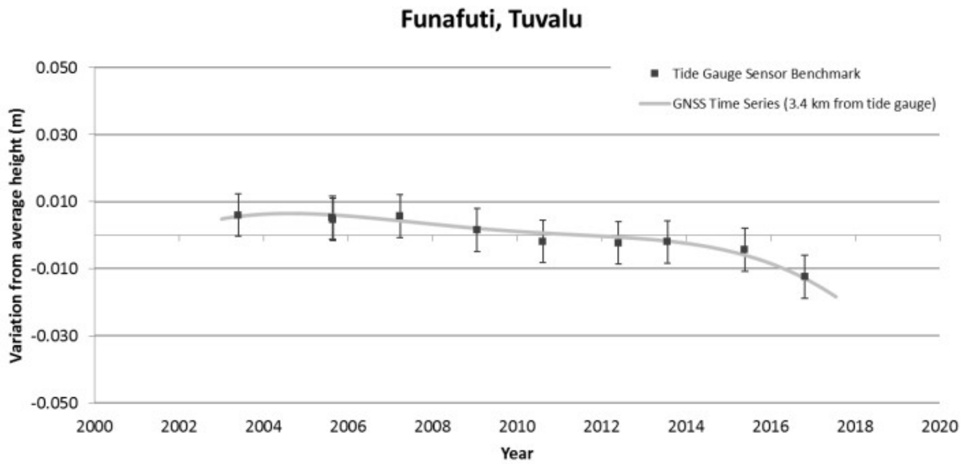


Figure 10 Ellipsoidal height of the tide gauge sensor benchmark (black squares) as determined from GNSS analysis (grey line) and the levelled height difference between the GNSS monument and the tide gauge.

Sea level near Tuvalu has risen and will continue to rise throughout this century (PCCSP, 2011). The latest advice from Intergovernmental Panel on Climate Change, IPCC (2019), on sea level rise (SLR) calls for increases to the allowances in previous documents. The latest global SLR (above 1986 - 2005 baseline) projections for the 'likely' scenario are 0.43m and 0.84m (i.e. 0.1m higher than AR5 projections in IPCC, 2013) by 2100 for RCP2.6 and RCP8.5, respectively (see Table 3 and **Figure 11**). While sea level rise is not globally uniform and varies regionally, these estimates are considered appropriate for planning purposes.

Historical observations suggesting that sea levels are rising at a rate higher than the global average and the land around Funafuti is also subsiding. These suggest that relative sea level rise is a significant potential hazard to TCAP project objectives. Moreover, the design of coastal protection works must consider the possibility of greater levels of sea level rise and/or an increase in the rate of sea level rise.

Table 3: Global sea level rise projections above 1986 - 2005 baseline (IPCC, 2019).

Planning horizon	Global sea level rise projection (above 1986 to 2005 baseline)	
	RCP2.6 (likely range)	RCP8.5 (likely range)
2070	0.37m (0.28-0.45m)	0.51m (0.45 – 0.61m)
2100	0.71m (0.51 – 0.92m)	0.84m (0.64 – 1.10m)
2100 (rate)	4mm/yr (2 – 6mm/yr)	15mm/yr (10 – 20mm/yr)

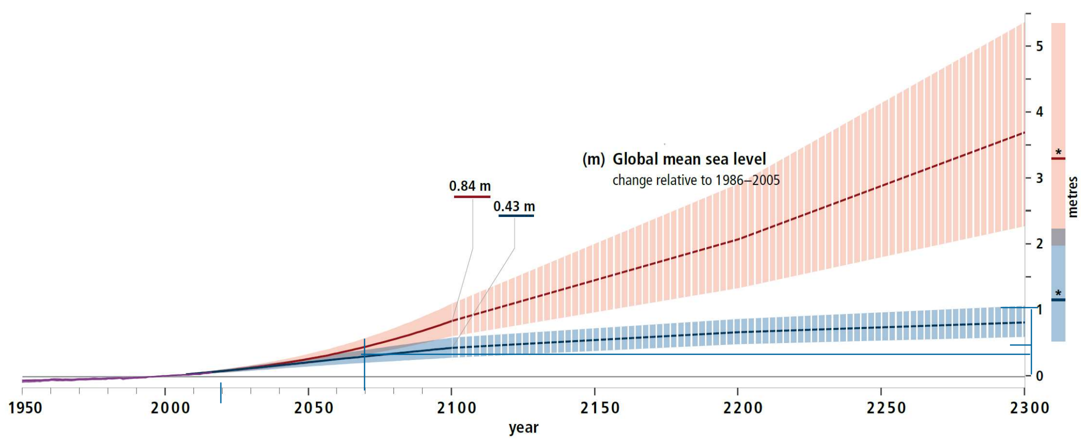


Figure 11: Global sea level rise projections above 1986 to 2005 baseline (IPCC, 2019): (blue) low (RCP2.6) and (red) high (RCP8.5) greenhouse gas emission scenarios

Wind

A review of the historic wind and wave climate was undertaken to identify any trends in their magnitude and direction. Boxplots showing the statistical distribution of annual wind and wave directions is presented in Figure 12. A trend analysis of annual wave directions is shown in Figure 13. The 42-year analysis of wave data shows a very slight change in wave directions over a 40 year hindcast period of -2.94º, equating to a -0.07º/year change. However, this small change is within the standard deviation of the dataset.

Global climate modelling suggests that climate change is expected to affect the frequency and intensity of tropical cyclones. While uncertainty remains, projections in the South Pacific generally indicate an increase in the intensity of the most intense cyclones, along with a reduction in the total number of cyclones (PCCSP, 2011). Importantly for Tuvalu, an increase in the frequency of extreme El Niño events is predicted (Cai et al., 2014). IPCC (2019) suggests that extreme El Niño events are projected to occur about twice as often under both RCP2.6 and RCP8.5 in the 21st century when compared to the 20th century (medium confidence). A higher frequency of extreme El Niño events may also result in a higher number of high-latitude tropical cyclones which as discussed in Section 3.1.2 pose the greatest risk to the TCAP project sites.

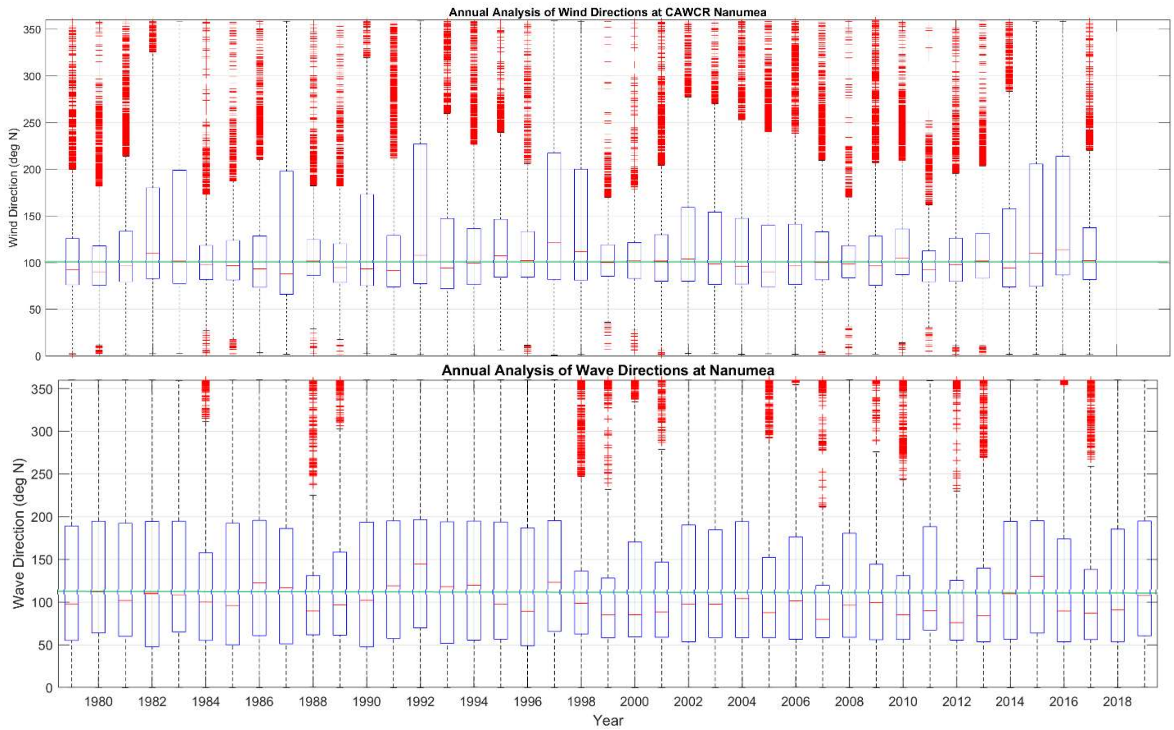


Figure 12: Analysis of (top) wind and (bottom) wave directions derived from global hindcast data. The long-term weighted mean directions are indicated by the green line.

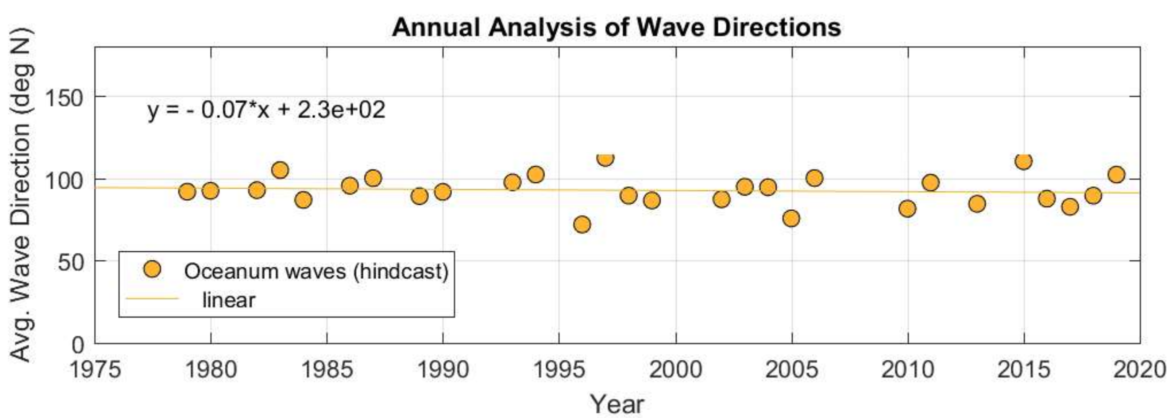


Figure 13: Annual weighted mean wave directions derived from the Oceanum wave hindcast.

3.2 Funafuti

There are at least 29 islets in the atoll of which three inhabited; Fongafale, Funafala in the south, and Amatuku in the north, **Figure 14**. The biggest is Fongafale, which is also the administrative capital of Fongafale and Tuvalu.

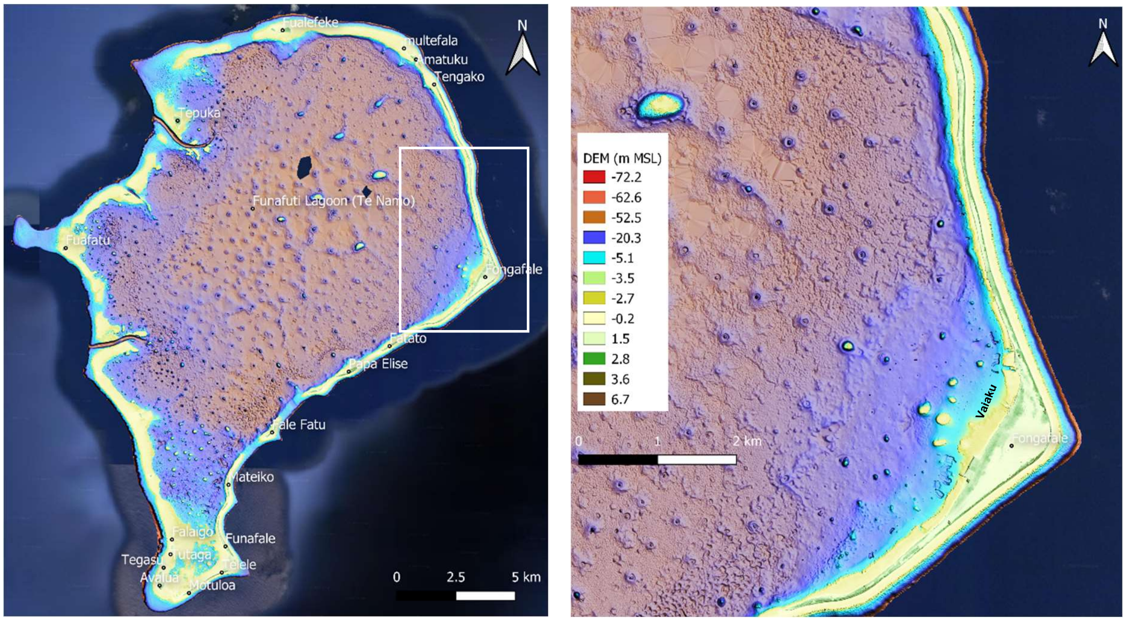


Figure 14: Left: Funafuti digital elevation model. Right: close up of Fongafale Islet. Please note heights have been approximated with respect to Mean Sea Level (MSL). (source: Fugro, 2019)

3.2.1 Geomorphology

The following section is a summary of the work undertaken by SPC in both the Funafuti Geotechnical and ESIA reports as part of their engagement under TCAP (SPC, 2020a & 2020b).

Funafuti Atoll played a key role in Darwin's theory of atoll evolution from volcanic subsidence (Darwin, 1842). Darwin's theory suggested that atolls were the result of mid-ocean volcanos undergoing subsidence coupled with the vertical growth of coral on top of the subsiding volcanos; beginning with fringing reefs, developing into barrier reefs, and eventually into atolls (see Figure 15). In an attempt to test Darwin's theory, the Royal Society Coral Reef Expedition drilled a 340m deep core on Funafuti Atoll in 1896 to 1898. The core failed to reach the underlying volcanic rock and was terminated in limestone (Hinde, 1904). A number of subsequent drilling expeditions in several different locations have encountered underlying volcanics, further proving the theory.

Tuvalu seamount chain, which is a submarine chain of extinct volcanos located on the Mid-Pacific plate, orientated roughly NW-SE between the Gilbert Ridge and the Samoan hotspot. The estimated age of the seamounts in the Tuvalu chain spans the Palaeocene and Eocene epochs, approximately 64 to 47 million years old. The origin of the seamounts is related to the hotspot magmatism theory whereby the source of magma (hotspot) is fixed, while the lithosphere (earth's crust) moves (relative to the hotspot) as a result of plate tectonics, creating a chain of volcanos.

Darwin's subsidence theory of atoll evolution and the hotspot volcano theory account for the long term evolution of the reef structure and volcanic foundation of Funafuti, however the surface reef island morphology is a result of recent processes operating during the Holocene epoch since the end of the last glaciation (Woodroffe & Biribo, 2011). These processes can be broadly characterised as biogenic generation of calcareous sediment, erosion, sediment transportation, and sediment deposition on the reef platform to form the present-day reef island. Basically, the islands making up the atoll are low-lying accumulations of reef-derived sediment. These processes have been operating since approximately 8-9 thousand years ago when post glacial eustatic sea level overtopped the degraded remnants of last inter-glacial reefs exposed to sub-aerial wreathing. In Tuvalu, sea level is estimated to have reached a level approximately 2.3m higher than present day sea levels during the mid-Holocene hydro-isostatic high stand (Dickinson, 2009).

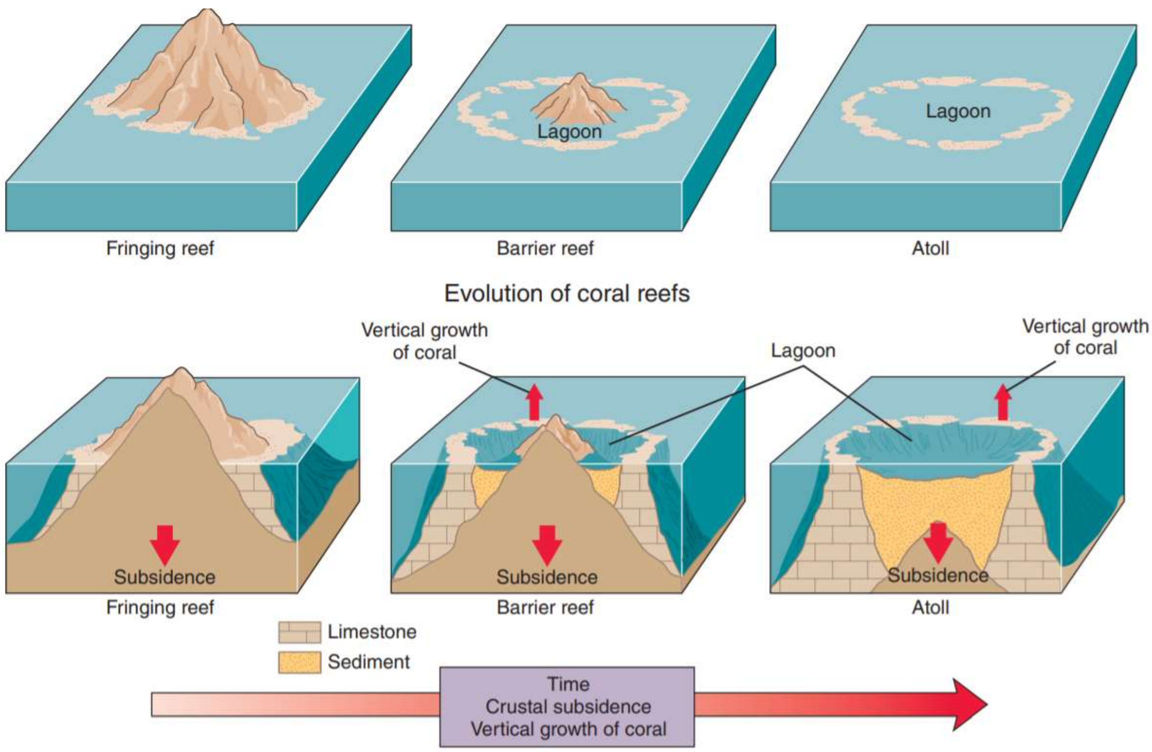


Figure 15: Evolution of an atoll; from a fringing reef, to a barrier, and finally to an atoll (Source: Jones & Bartlett Learning, 2009).

Fongafale is a long narrow (reverse) L-shaped island divided into three geographical areas: the south arm, the central area and the north arm, **Figure 14**. The study site at Funafuti is at the intersection of north and south arms on the Vaiaku shoreline. The island morphology controls the pattern of the longshore currents on the lagoon-side, which transport sand from both the northeast and southwest towards the central area, resulting in well-established beaches in the central area (Vaiaku). Profiles of three transects intersecting the north, central and south arms of the island can be seen in Figure 16.

The transects show representative atoll profiles (after Figure 15). The offshore (outer) reef platform is generally narrow along both arms of the island (less than 100m) dropping very quickly to deep water (>50m) in less than 100m from the reef edge. A discontinuous cemented rubble bank is located on the inner reef flat believed to be the remains of a rubble rampart that was deposited 3m from the reef edge during Tropical Cyclone Bebe in October 1972 (Maragos and Beveridge, 1973). A 5-10m wide conglomerate platform is located between the area of cemented rubble forming a steep beach face with a berm elevation of between 3-4m MSL. The berm is formed by large, unconsolidated pieces of coral rubble and is seen to be the highest land formation of the island (Figure 15, Figure 16).

The lagoon shoreline has changed significantly since intervention from the American army stationed here during WWII. Several boat channels, harbours and coastal structures have been built and subsequently built upon since 1945. The large QEP reclamation (2018) to the south side of the Vaiaku shoreline once again significantly changed the appearance of the shore and local hydrodynamics and coastal processes. The Vaiaku shoreline is now covered with a thin (1-2m) layer of beach sand (a remnant of the 2018 QEP reclamation) overlying beach rock providing an approximate 30m wide beach. The nearshore area is seen to be a thin layer (<1m) of beach sand interspersed over the underlying reef flat and scattered (domal coral) bombies which extends for approximately 800m offshore before the lagoon deepens to greater than -10m.

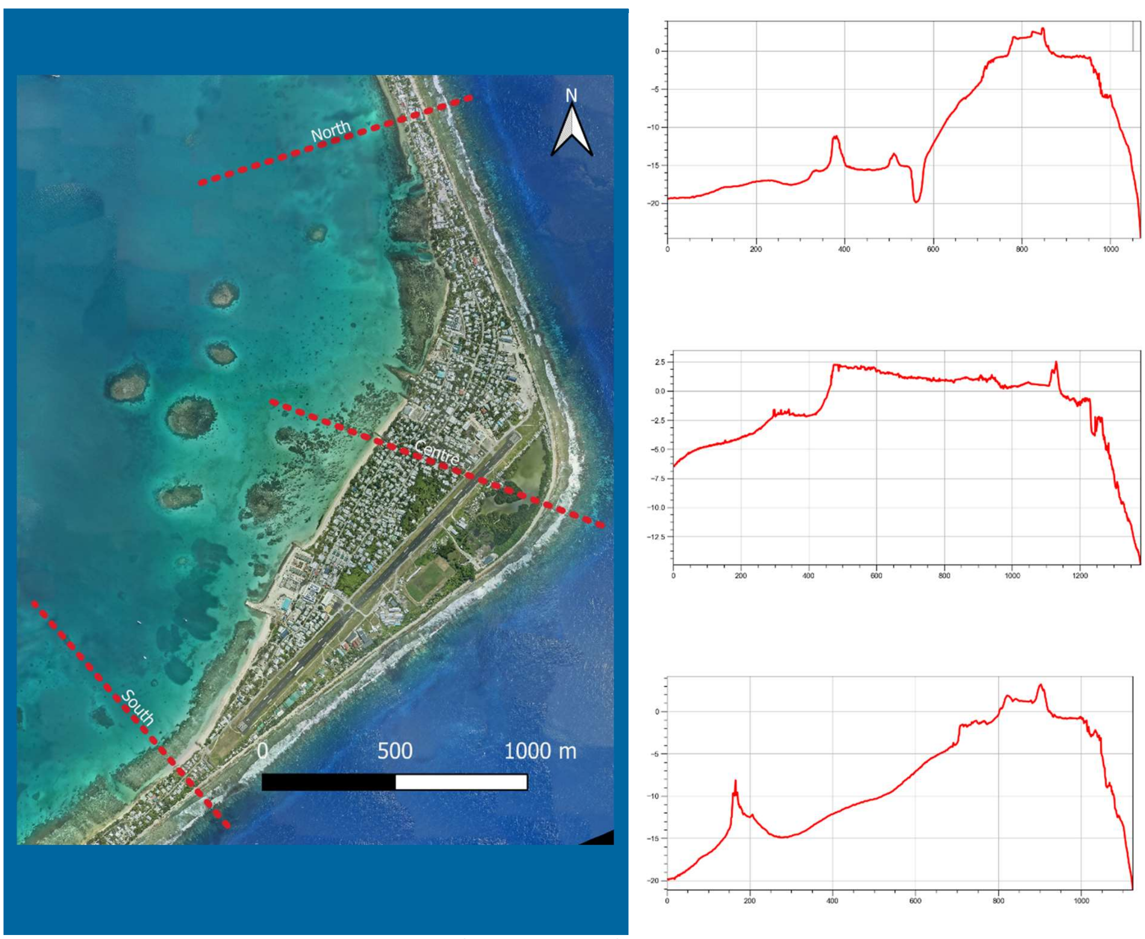


Figure 16: Representative topographic profiles on Fongafales central arm.

3.2.2 Water levels

High quality water level data has been collected on Fongafale since 1993. The tide gauge is located at Tuvalu Port on the eastern side of Fogafale Lagoon. It was established as part of the Pacific Sea Level and Geodetic Monitoring project (PSLGM) (http://www.bom.gov.au/pacific/projects/pslm/). This project is an Australian-funded project with the primary aim to generate an accurate record of variance in long-term sea level for the Pacific region.

A 25-year dataset of hourly water levels recorded at the Funafuti (Tuvalu Port) tide gauge was attained through the BoM online data portal. Based on tidal harmonic analysis undertaken on the entire record, tidal planes and other reference levels are shown in **Table 4** alongside published tidal planes produced by PSLM (2019b).

Table 4: Calculated and published tidal plane data at Funafuti, Vaitupu and Nanumaga.

Tidal Plane	Recorded height Funafuti (m MSL)	Published height Funafuti (PSLM, 2019b) (m TGZ)
Highest Recorded Water Level (HRWL) recorded 28-Feb-2006	1.39	3.44
Highest Astronomical Tide (HAT)	1.30	3.30
Mean High Water Springs (MHWS)	0.82	2.87
Mean High Water Neap (MHWN)	0.33	2.38
Mean Sea Level (MSL)	0.00	2.05
Mean Low Water Neap (MLWN)	-0.33	1.72
Mean Low Water Spring (MLWS)	-0.82	1.23
Lowest Astronomical Tide (LAT)	-1.17	0.89
Lowest Recorded Water Level (LRWL) recorded 27-Feb-1998	-1.49	0.53
Tide Gauge Zero (TGZ)	-2.05	0.00

Figure 17 provides a plot of the recorded water level, the predicted tide and the tidal residual (the difference between the predicted tide and the recorded water level). Generally, large positive residual levels are due to storm conditions (e.g. tropical cyclone) influencing the water level. The Funafuti tide gauge is located within the protected Funafuti Port on the east side of the Funafuti Lagoon (Te Namo). As such, it is anticipated that the biggest influence on the increase of positive residual during storms at Funafuti is most likely attributed to lower mean sea level pressure (MSLP) as well as wind setup across the lagoon. Based on the 25-years of observed sea level data, estimates of return period values for positive residual water levels were calculated, the results can be seen in Figure 18.

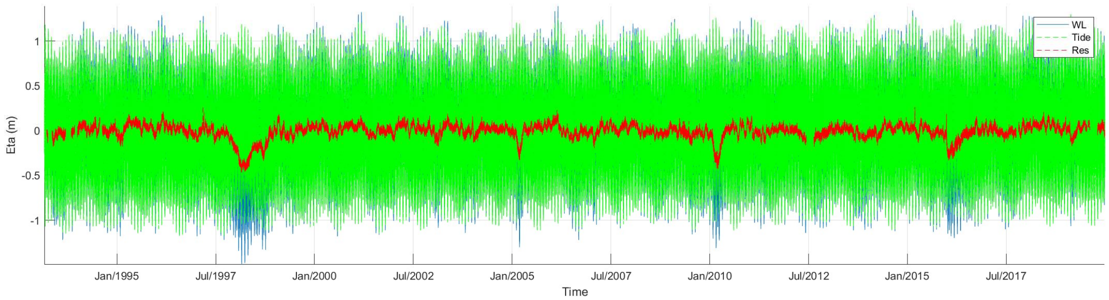
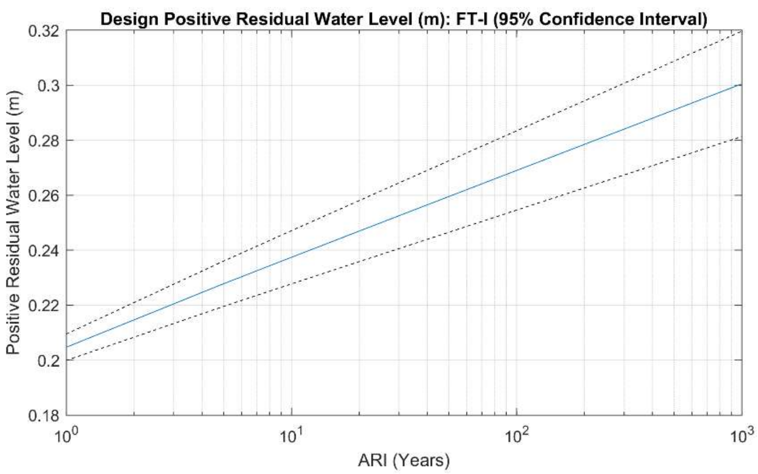


Figure 17: Water level data recorded at the Funafuti Port Tide Gauge (blue), predicted tide (green) and tidal residual (red).



ARI (years)	Residual Level (m)
1	0.20
5	0.23
10	0.24
50	0.26
100	0.27
500	0.29

Figure 18: Annual Return Interval (ARI, years) of positive residual water level (m) recorded at the Tuvalu Port Tide Gauge.

Note: A Gumbel Maximum Likelihood Estimation (MLE) Extreme Value Analysis (EVA) was undertaken on the positive residual component of the recorded water level at the Tuvalu Port tide gauge only (i.e. negative values removed). This EVA neglects the tidal component and is solely concerned with positive residual (surge) of the recorded signal).

In order to calculate a conservative design sea level, the calculated 50year ARI positive residual was added to the highest astronomical tide (HAT) value calculated Table 10. The final calculated 50-year design ARI Water Levels can be seen below.

ARI 50yr WL = HAT + 50year ARI positive residual

= 1.39 + 0.26

= +1.65m MSL (+3.7m TGZ)

3.2.3 Waves

Several wave monitoring exercises have been undertaken within and offshore of the Funafuti Lagoon (Te Namo).The following section summarises the results of these monitoring exercises as well as undertakes an investigation of global hindcast modelling of the region in order to gain an understanding of the wave environments both inside and outside of the atoll to inform coastal protection and reclamation design at Fongafale.

Ocean side

There is very little measured wave data available outside of the Funafuti lagoon. In order to determine the wave climate on the ocean side of Fongafale's eastern shore, a regional wave hindcast model has been used. The wave hindcast evaluated the wave conditions in the region between 1979 and 2019. It was produced by the Centre for Australian Weather and Climate Research (CAWCR) and focussed on the central and south Pacific. The model resolution around the islands of Tuvalu can be seen in Figure 19**.** This resolution, of between 10 to 4 arcminutes (~20 to 8km) is sufficient to determine offshore wave conditions (i.e. it is accurate for waves in greater than about 50m water depth and not in the lee of smaller islands). The model has been verified against wave measurements around the South Pacific, further details of the calibration as well as the model drivers are elaborated in Durrant et al. (2014).

Long term wave information was extracted from the CAWCR hindcast for a deep-water extraction location to the south east of Fongafale Islet and analysed, see Table 5. Long-term average, as well as seasonal wave roses and statistics, can be seen in Figure 19.

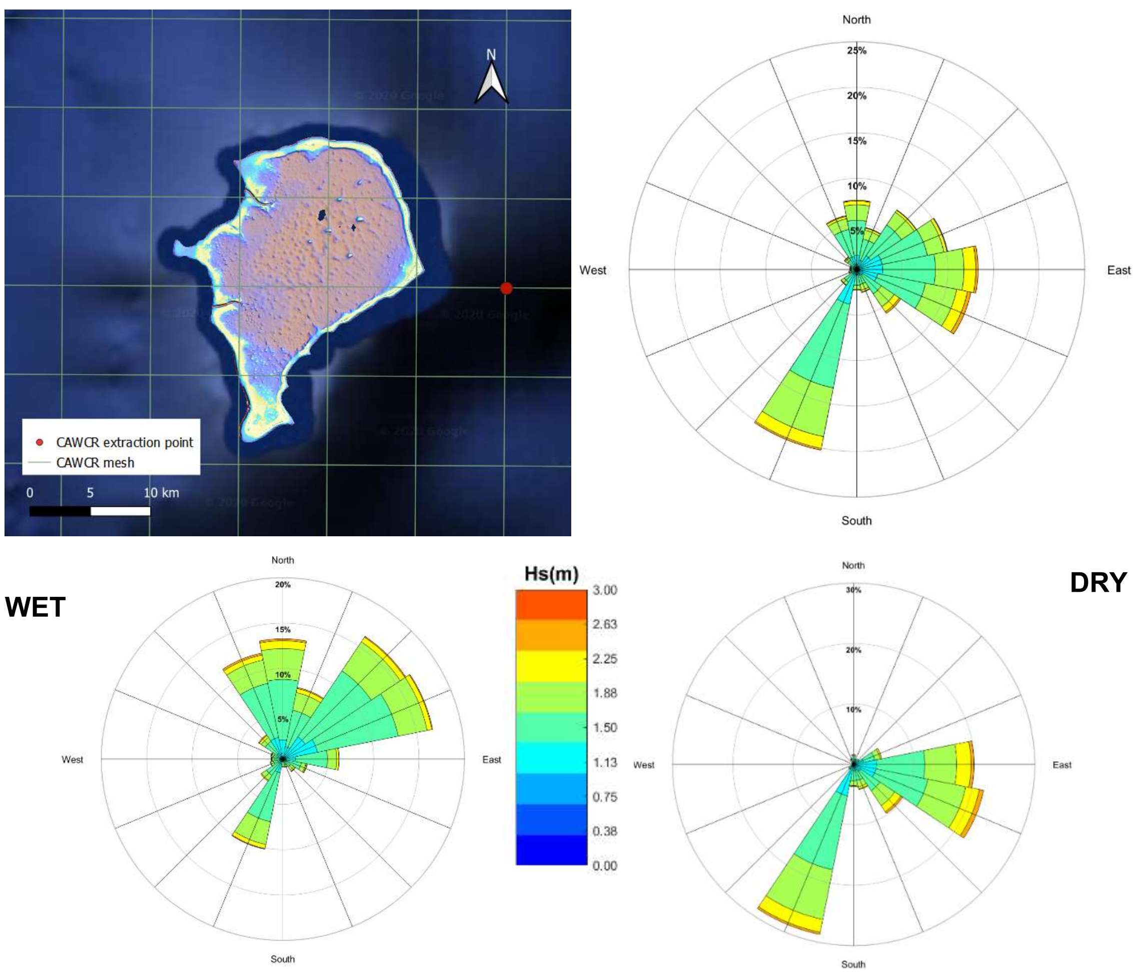


Figure 19: (clockwise from top left): CAWCR grid resolution around Funafuti and model extraction point, long-term and seasonal wave roses for the deep water CAWCR model extraction point offshore for the wave hindcast information 1979-2019

Table 5: Long term statistics calculated for the CAWCR model extraction points in deep water offshore of Fongafale for the wave hindcast information 1979-2019

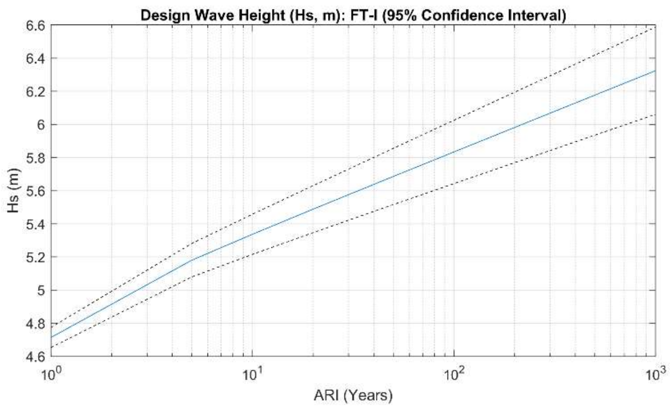
Parameter	Statistic	Long term average (40yrs)	Wet season (Nov- Apr)	Dry season (May – Oct)
Hs (m) Significant Wave Height	Average	1.8	1.7	1.8
	20%ile	1.5	1.5	1.5
	90%ile	2.2	2.1	2.3
	Max	5.4	5.4	3.6
Tp (s) Peak Wave Period	Average	11.8	12.1	11.4
	20%ile	9.1	9.8	8.8
	90%ile	15.9	15.9	15.9
	% of Time Sea (Tp<8s)	3%	1%	5%
	% of Time Swell (Tp>8s)	97%	99%	95%
Dp (°N) Peak Wave Direction	Weighted Average	119	16	151
	Standard Deviation	67	63	51

The statistics provided from the hindcast analysis are for a deep water (>50m) location offshore of Fongafale's eastern reef edge. The location of the extraction point off the south eastern coast of Fongafale means there may be some sheltering by the atoll of waves coming from the west and north and as such is reflected in the wave roses seen in Figure 19.

The extraction location is seen to be dominated by swell waves (wave periods above 8 seconds), meaning wave energy is usually generated some distance from the atoll. Locally generated seas (wave periods below 8 seconds) only occur on average 3% of the time with a higher percentage seen in the dry season (5%). Average Significant Wave Height (Hs) is 1.8m with the largest waves occurring in the dry season attributed to Southern Ocean swells travelling from the south west. The dry season also sees a high frequency of waves generated along the east and south east trade fetch attributed to the strengthening and predominance of the trade winds during those months.

The wet season sees less frequent (and less intense) waves generated from the southerly sector, with a greater frequency in waves arriving from the north (NE to NW), these waves are most likely attributed to large events in the Northern Pacific. The largest waves can be seen to occur from the north and north west and are most likely attributed to the passage of tropical cyclones in the South Pacific Basin during the wet season

In order to gain an understanding of design wave events on the south eastern shore of Fogafale Islet, a joint frequency analysis as well as an extreme value analysis (EVA) was carried out on the deep water CAWCR extracted wave conditions based on a peak over threshold approach (PoT), Figure 20.



ARI (years)	Hs (m) *From EVA*	Tp (sec) *From JFA*
1	4.7	14
5	5.2	14
10	5.3	14
50	5.7	14
100	5.8	14

Joint Frequency Table (%) Showing Hs Against Tp for the Period 01-Jan-1979 to 31-Dec-2018 23:00:00

N=350640	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	Total	Cumul.
0.0-0.375	-	*	-	-	-	-	-	-	-	-	*	*
0.4-0.75	-	*	*	-	-	-	-	-	-	-	*	*
0.8-1.125	-	-	*	0.03	0.21	0.08	0.08	0.04	0.02	0.01	0.48	0.48
1.1-1.5	-	-	*	0.30	8.04	4.80	4.06	2.59	0.98	0.23	21.00	21.49
1.5-1.875	-	-	0.01	1.00	14.79	10.20	9.03	8.26	2.93	0.66	46.89	68.38
1.9-2.25	-	-	*	1.23	7.93	3.56	3.23	4.90	2.11	0.51	23.48	91.86
2.3-2.625	-	-	-	0.46	2.97	0.75	0.52	1.02	0.68	0.17	6.57	98.43
2.6-3	-	-	-	0.11	0.87	0.20	0.09	0.17	0.11	0.03	1.57	100.00
Total	-	*	0.02	3.14	34.81	19.58	17.01	16.98	6.83	1.62		
Cumul.	-	*	0.02	3.16	37.97	57.55	74.56	91.54	98.38	100.00		

Figure 20: Peak Over Threshold (POT) Extreme Value Analysis (EVA) of wave heights of deep water wave heights offshore of Nanumea based on the 40 year hindcast of waves from the CAWCR model and associated wave period based on a joint frequency analysis (JFA) of wave heights and period.

Due to the temporal resolution of the CAWCR model (1hours) it is expected that wind speed peaks associated with the passage of tropical cyclones may not be resolved sufficiently and as such wave heights may be under-predicted during these events. As such the wave analysis of the deep water CAWCR model extraction point undertaken above should be viewed as operational and not extreme, they provide an insight into the day-to-day wave climate at the project site and not the design conditions.

Historical cyclones tracks have been extracted from the International Best Track Archive for Climate Stewardship (IBTrACS) database from 1950 to 2019 in order to understand the exposure of Funafuti to the passage of tropical cyclone. Figure 21 shows that there have been 26 cyclones passing within a 300km radius of Funafuti since 1950. This equates to one cyclone every 2.5 years passing in direct vicinity of the atoll. It is expected however that the ocean side coastlines of Funafuti would still experience the effects of large waves from storms generated in the South Pacific passing outside the 300km search radius due to the remoteness and exposure of the Tuvaluan islands to wind and waves in all directions.

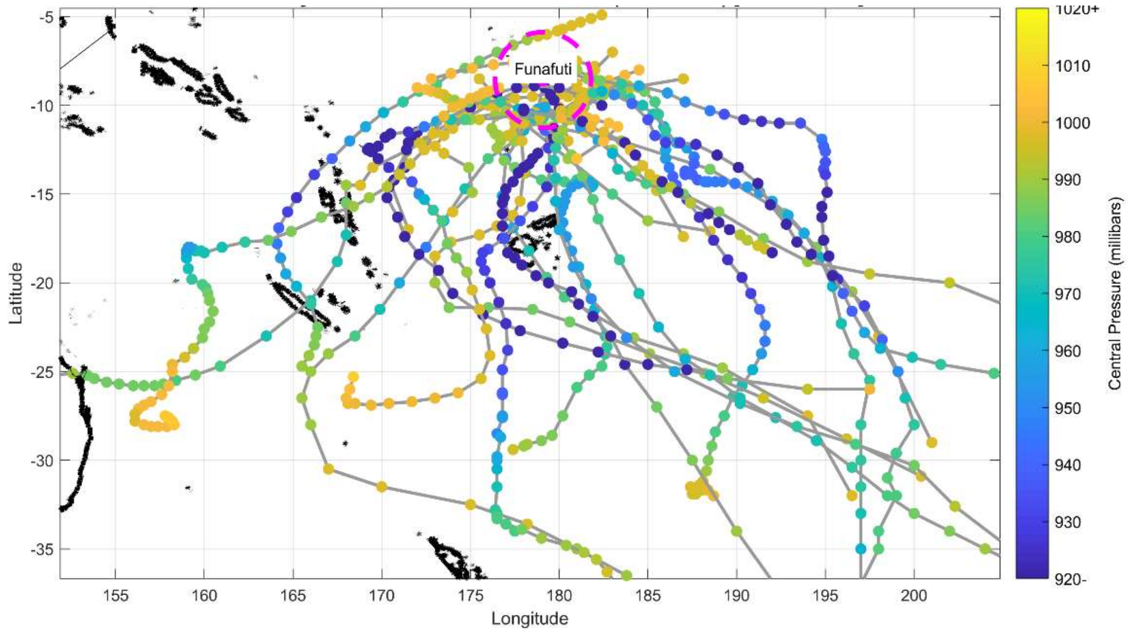


Figure 21: Historical cyclone tracks within a 300km radius of Funafuti between 1950 and 2019 based on the IBTrACS database.

<u>Lagoon side</u>

The lagoon (Te Namo) is protected on all sides from oceanic swell by fringing intertidal reef flats and 29 islets with only five small (navigable) passes. The wave climate within the lagoon is generally limited to that which can be generated across the relatively small 20km north-south and 16km east-west fetches, Visual evidence of generally small (Hs<20cm), long period swells has been noted on the northern and eastern shores of Fongafale Islet during both high and low as well as spring and neap tidal cycles. This suggests that there is some penetration of ocean waves across the reef flats or through the passages of the lagoon.

Design wave heights were calculated for the QEP reclamation along the Funafuti foreshore utilising the fetch-limited empirical method outlined in the Coastal Engineering Manual (USACE , 2008). Incorporating a 50yr ARI design wind speed based on AS1170.2 (40m/s), AECOM (2015) calculated the following design wave parameters at the Funafuti lagoon foreshore as seen in Table 6.

Table 6 Wave hindcasting summary and resulting 50 y ARI design wave height (source: AECOM, 2015).

Input parameter (USACE, 2008)	Value
Fetch Length	16 km (east-west fetch)
Wind Speed	40 m/s
Wind duration	0.2 seconds
Fetch-limited duration	2.7 hour
Fetch-limited speed	23.9 m/s

Input parameter (USACE, 2008)	Value
Design Hs	1.76 m
Design Tp	4.2 seconds

Wave measurements have been undertaken offshore of the project site in 2009/2010 as part of the Japan International Cooperation Agency(JICA) report *Study for Assessment of Ecosystem, Coastal Erosion and Protection / Rehabilitation of Damaged Area in Tuvalu* (JICA, 2011). An Acoustic Doppler Profiler (ADP) was deployed at the frontage of Vaiaku Lagi Hotel offshore of the project site for three separate wet season deployments, statistics for these deployments can be seen in Table 7. The deployments illustrate a relatively benign wave climate dominated by short period (wind) waves from the west.

Table 7: Wave statistic from JICA (2011) wet season deployments offshore of Funafuti foreshore 2009 to 2010.

Wave Parameter	Statistic	Deployment 1: Nov1-21, 2009	Deployment 2: Feb 2-19, 2010	Deployment 3: Feb 20 – Mar19, 2010
Wave height	Max	0.3 m	0.9 m	1.4 m
	Mean	0.2 m	0.4 m	0.4 m
	Min	0.2 m	0.2 m	0.2 m
Wave period	Max	4.0 sec	4.5 sec	4.5 sec
	Mean	2.5 sec	3.6 sec	3.1 sec
	Min	2.0 sec	2.5 sec	2.1 sec
Wave direction	Max	263°N	269°N	279°N
	Mean	251°N	260°N	257°N
	Min	229°N	247°N	231°N

In December 2017, a subsequent 20-day wave monitoring exercise was undertaken 2km offshore of the study site in 23m of water as part of the Ministry of Fisheries study *Master Plan* & Feasibility Study for Coastal Erosion Protection in Tuvalu (MOF, 2018), statistics for these deployments can be seen in Table 8. The deployments also illustrate a relatively benign wave climate with significant wave heights (Hs) over the 20-day deployment period averaging 13cm with an associated period of 7.7sec. A key differentiator of this deployment is the notable presence of long period swell waves (period > 8sec) in the deployment record. It is assumed that these longer period waves(Tp = 10-14sec) would be generated outside the confines of the lagoon. For this to take place either swell is entering the lagoon through the western or northern facing passages or waves are passing over the reef flats on elevated water levels.

Table 8: Wave statistic from MOF (2018) December 2017 deployment 2km offshore of Funafuti foreshore

Parameter		Wave height				Wave period		
statistic	$H_{1/3}$	$H_{1/10}$	H_z	H_{max}	$T_{1/3}$	$T_{1/10}$	T_z	T_{max}
Max	0.25	0.44	0.14	0.91	14.03	10.34	13.83	13.73
Min	0.09	0.11	0.04	0.13	5.25	4.28	5.26	3
Mean	0.13	0.2	0.07	0.3	7.77	5.75	8.37	5.28

Due to the uncertainties in design wave height within the lagoon, a small numerical modelling exercise was undertaken to determine ARI wave heights at the proposed reclamation site, see Design wave height for Funafuti reclamation bund Technical Note (Bluecoast, 2021b). The exercise placed a conservative ARI 100year significant wave height at he proposed reclamation site at just over 2m with an associate wave period of 5 seconds.

3.2.4 Currents

Hydrodynamic simulations of the Funafuti Lagoon have been undertaken by the South Pacific Applied Geoscience Commission (SOPAC) in 2005 to estimate the impact of dredging offshore of the study site (Damlamian, 2005). The study showed that current speeds in the vicinity of the study site (pre-QE2 Reclamation) were seen to be generally low in magnitude (<1m/s) and are dominated by tidal flows and wind. A general description for each tidal phase described in the report has been provided below.

Neap tide

- Current is dominated by wind speed and direction.
- The south-east trade winds generate inflows through passages along the eastern atoll and inter-tidal reef flats.

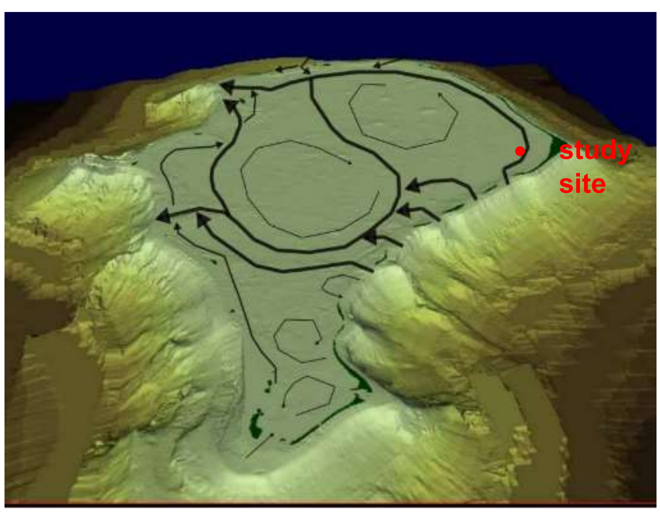
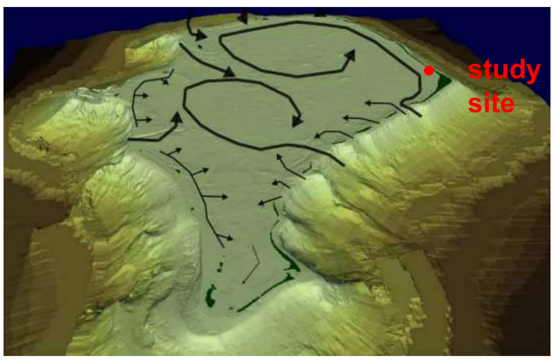
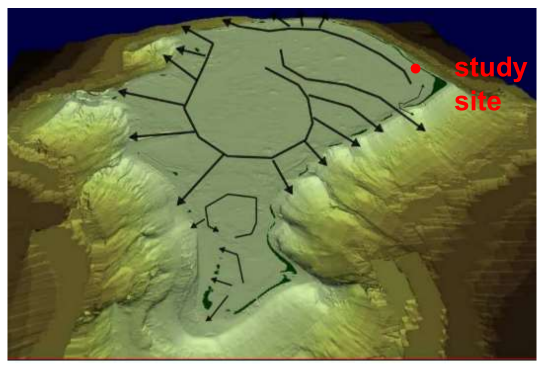


Figure 22: Approximation of neap tide atoll circulation. (source: Damlamian, 2005).

Spring tide

- Current is dominated by tidal flows for both ebb and flood tides.
- The current regime in the vicinity of the study site can be described as generally low speed north to south directed flows for both ebb and flood tides during spring tides.

Flood:	Ebb:
Strongest inflows are located in the north and main west channels	Ebb flows are also through the main west and northern passages
Wind can influence the rate of inflow but not overall circulation.	A strong trade can also affect the outflow in the east, increasing western flows.
A stronger trade wind leads to the creation of two large eddies inside the lagoon	
	

3.2.5 Conceptual coastal process model

Hydrodynamics within the lagoon is constrained by the prevailing wind direction and the atoll's morphology. Chunting and Malologa (1995) proposed the following conceptual coastal process model for Fongafale:

- No lagoon side current occurs while the wind direction is between 55° and 122°.
- A southwest-directed of longshore current develops only along the shore of the south arm when the wind is between 342° and 55°.
- A northwest-directed of longshore current occurs only along the shore of the north arm when the wind is between 122° and 235°.
- The longshore current directions generated by waves are shown in Figure 23 for wind directions in the sectors 235° - 252°, 252° 325° and 325°-342°.

The differences in beach volume between opposite sides of groynes (jetties and wharfs) show that the dominant longshore current is south-eastward along the north arm and north-eastward along the south arm. Recent hydrodynamic modelling (Damlamian, 2005, 2019) has shown that current speeds in the nearshore of Vaiaku beach are generally low speed (<0.5m/s) with propensity for sediment transport of the coarser grained sediments generally only seen in larger wave events.

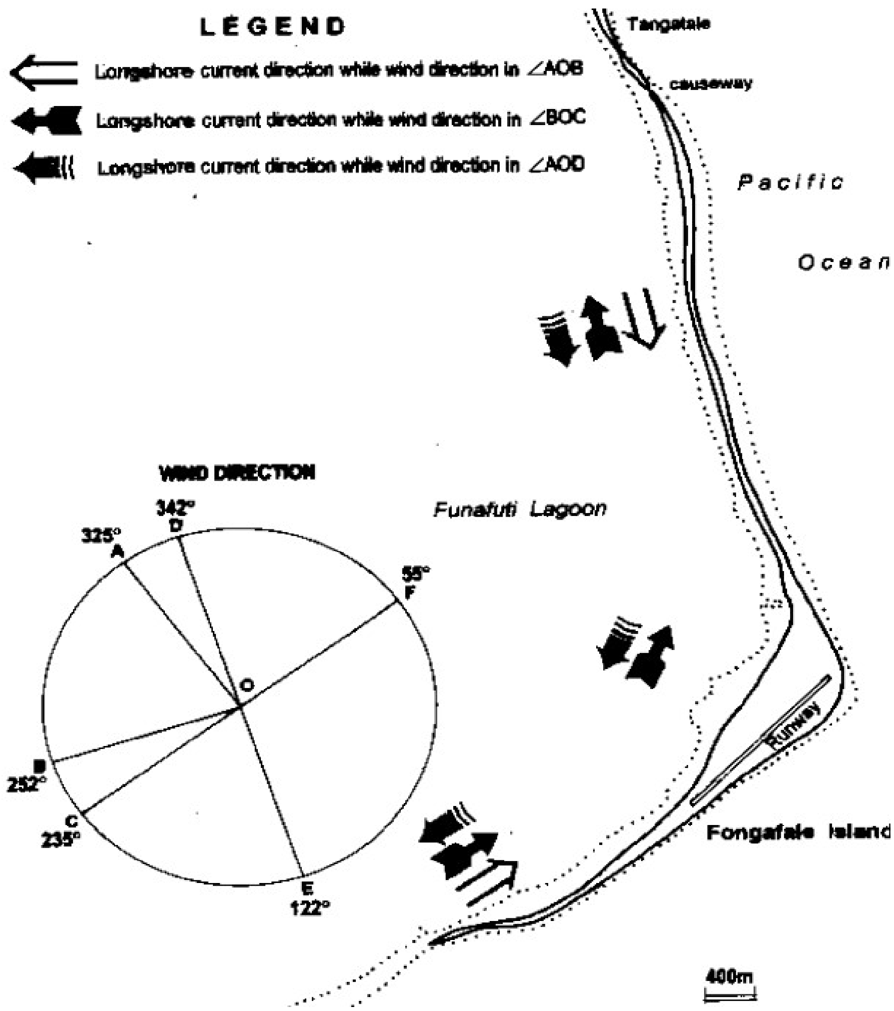


Figure 23: Conceptual coastal process model (Source: Chunting & Malologa, 1995)

3.3 Nanumaga

3.3.1 Geomorphology

Nanumaga is a table reef with a single reef island (Tanyama,1952). The island is oval shaped, replicating the shape of the reef platform. It is approximately 3.1km long in the north-south axis and 1.5km wide in east-West axis. Nanumaga island evolved in the same way as Funafuti atoll, with the lagoon infilling to a maximum state with a remnant low-lying area in its centre (*Figure 24*). A 1964 geotechnical survey showed the island is composed of loose foraminifera rich sands with occasional loose coral gravel layers. The survey logged an exposure in a large pit near the eastern shoreline and found that loose foraminifera rich sand continued to a depth below the level of the present-day fringing reef, suggesting the reef island was formed by calcareous sediment infilling the lagoon of a small atoll.

A 4-6m (above MSL) storm berm is the major geomorphological feature along the western shore of the island and is the highest natural feature, Figure 25. It is difficult to discern its landward margin as it gradually slopes inland, but on average the feature is well over 70m wide (running east or landward of the vegetation line) and its landward edge can be approximated by the main road running roughly parallel to the western shore. The storm berm in its entirety is an "active shoreline system" with differing temporal scales of change. For example, the active sandy beach is subject to daily wave effects and monthly tide cycles, whereas the broader storm berm system described above is subject to change and re-working during annual and interannual seasonal effects such as ENSO and larger wave events. The entire system must be viewed as a dynamic shoreline system and its function protected. Development east of the roadway on Nanumaga should be very carefully considered and as far as possible avoided because this entire shoreline system is capable of rapid change and adjustment and is necessarily subject to wave wash, overtopping as well as sediment deposition and removal.

Eighteen soil pits and 22 soil samples were logged (to a maximum depth of 0.6m) along the storm berm by SPC in 2019. The results of the soil pits and testing were used to generate a soil map of the island, described in detail in SPC (2020a). The storm berm was seen to contain light sandy soils, with unaltered sand on the active beach. The light sandy soils are described as foraminifera rich sand with occasional coral gravel fragments. The light sandy soils are distinguished from the unaltered sands due to the presence of organic content (roots, leaves and branches) in various stages of decomposition, which have stained the calcareous sand to a variety of different shades of grey depending on the organic content. The light sandy soils are mapped on the seaward slope of the western ridge landform unit. Unaltered sands are mapped on the beach (the parent material for the light sandy soils).The unaltered sands are composed of foraminifera rich sand (predominantly calcarina type),with less than one percent fines.

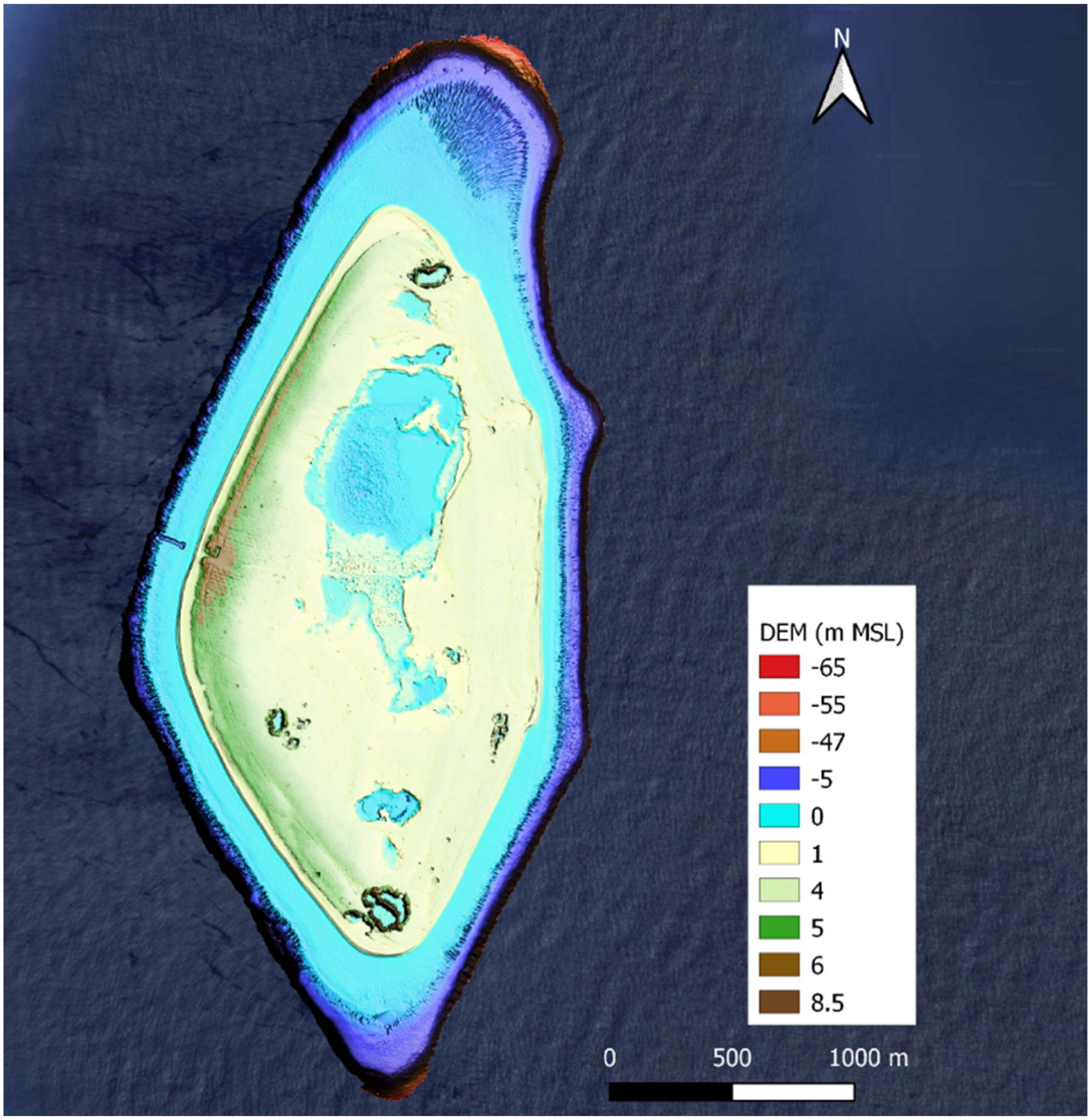


Figure 24: Nanumaga digital elevation model. Please note heights have been approximated with respect to Mean Sea Level (MSL). (source: Fugro, 2019)

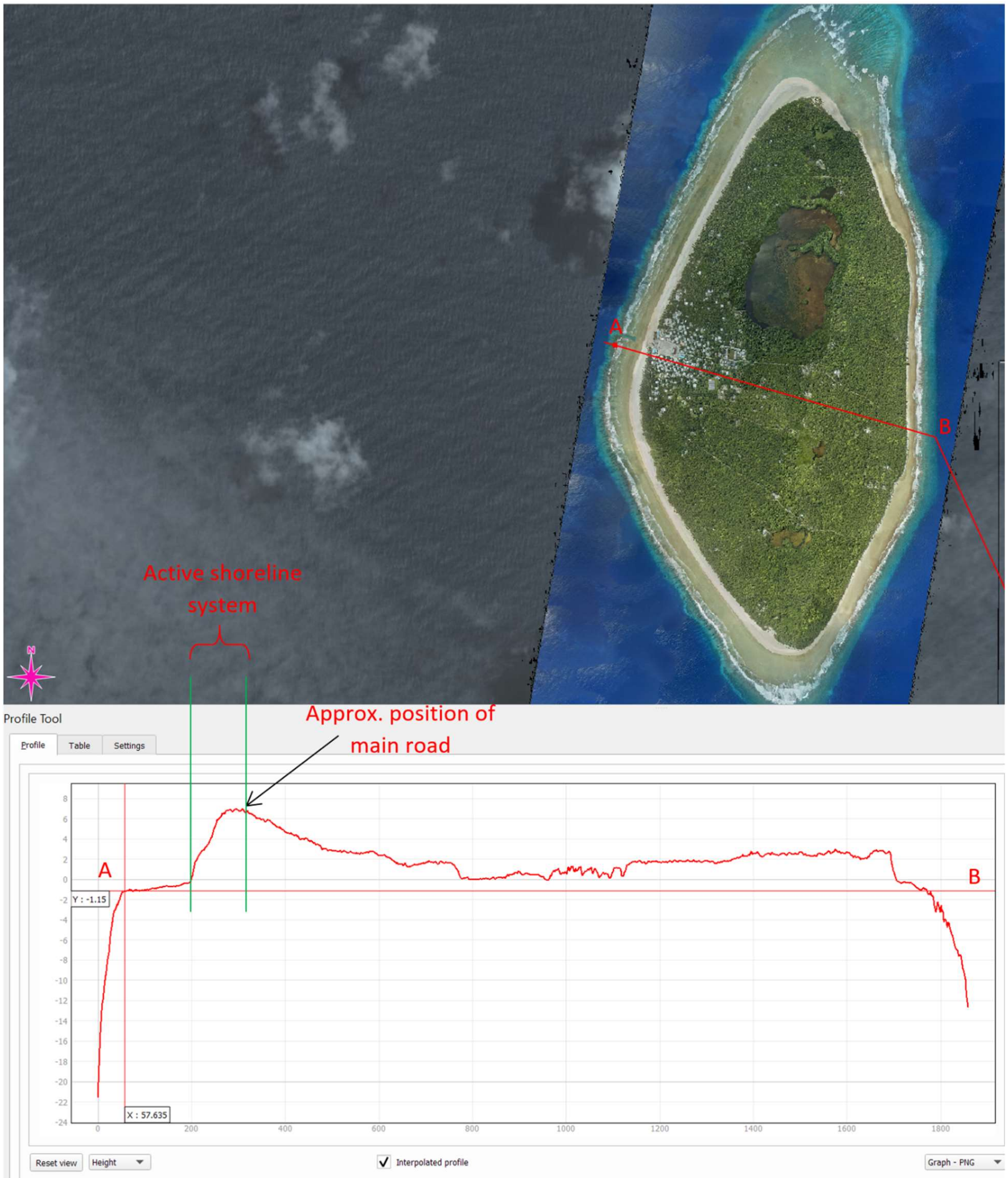


Figure XX Lidar imagery and profile (2019). Note the active shoreline system on the western shore is far larger and higher than the eastern shore because of the prevailing direction of storm waves. Note also that elevation in this profile is accurate, whereas that shown below (McLean et al 1991) is representative only and is included because it shows composition units.

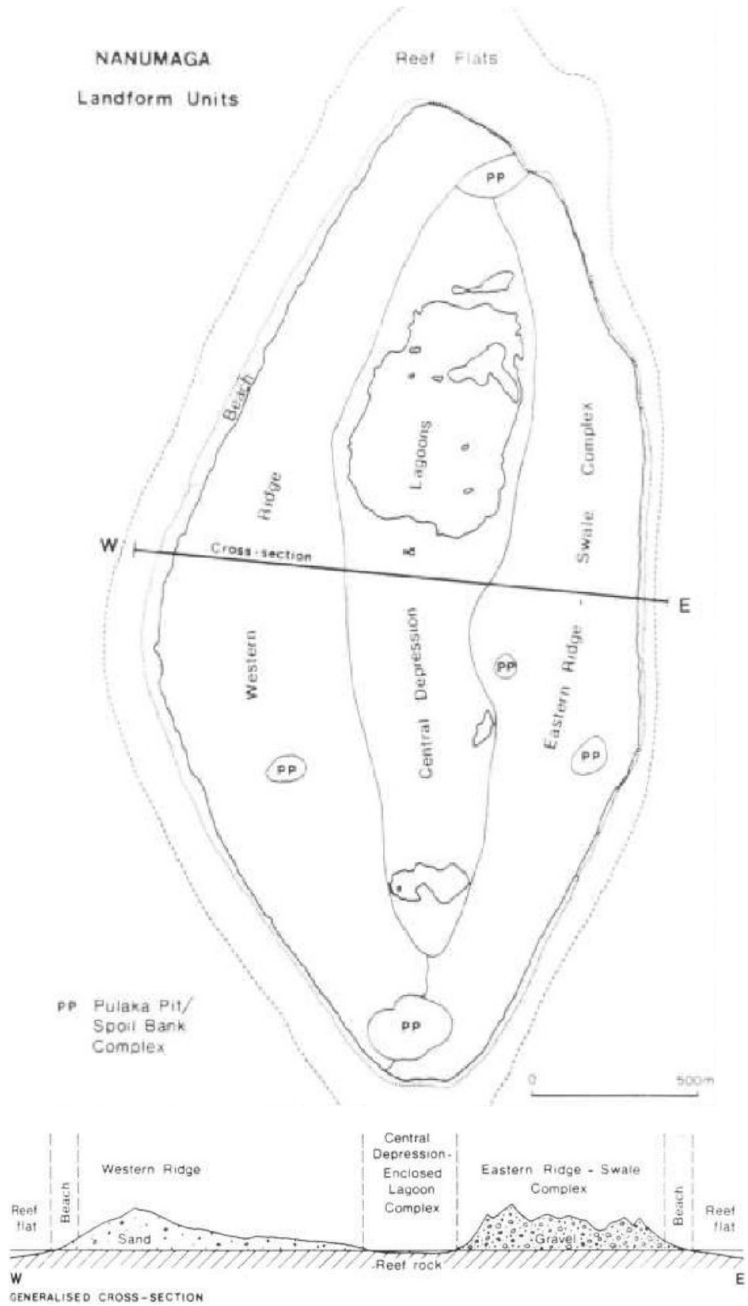


Figure 25: Map of Nanumaga landform units after SPC, 2020a (Source: McLean et al,1991).

Sand Resources

This is an island which is essentially in a "climax state" in terms of its planar size. There is no accommodation space left on the reef flat to enable further growth and the remanent inland lagoon is now closed off from the perimeter shoreline system and tidal flushing which prevents continued natural infilling. If the surrounding reef systems remain productive and protective and the prevailing climactic conditions remain stable over decadal timeframes, the only factors causing significant changes to the shoreline system are major storm events such as TC Pam or TC Tino (Figure 26). Such storms are a natural part of the system’s functioning and are the only mechanism by which the major western shore storm berm system has been built. After such storm events on Nanumaga, the coastal sediment system is disturbed and dramatic sediment redistribution can occur but overall, if reef systems around the island retain their ongoing function the shoreline will set about recovery and will re-establish to absolute capacity. Once sediment capacity is reached (meaning there is no accommodation space left to accumulate additional sediment on the island shore), sediment will start to “leak” from the island to the open ocean over the reef platform edge. This occurs at roughly the equivalent rate to its production from the surrounding reef systems and deposition on the shore. It is this leaking of sediment that predominately occurs to the north and the unusual reef features at the NE reef edge are likely a sign of this.

Otherwise, dramatic movements of shoreline sediment can occur quickly during large storms, during such events sediment can be quickly re-worked and deposited in new areas or lost from the island. This occurred during TC Pam which removed large quantities of sediment from the western foreshore and deposited these at the southern and especially the northern terminal point of the island. These low-lying deposits are ephemeral in nature and will remain highly mobile and dynamic until the shore once again reaches its equilibrium state and position. During both TC Pam and TC Tino large sediment deposits were also lifted and deposited over the western storm berm system, further building this natural defensive barrier. This phenomenon underscores the urgent necessity to leave a substantial no-build zone or buffer over the crest of the main western storm berm and to understand the entire shoreline system (from the toe of the sandy beach to the approximate position of the main road) is dynamic and subject to regular wave impacts.

Figure 26: Nanumaga storm sand deposits (northern point) in September 2019 (left) and again in 2020 after TC Tino. Source SPC and GoT Department of Lands

3.3.2 Water levels

No long-term water level measurements have been undertaken at Nanumaga Island at the time of this report. Tide charts at nearby Vaitupu Island (approximately 285km to the south west of Nanumaga) have been produced to assist with navigation between the outer islands (PSLM, 2019a). The charts have been produced as part of the Pacific Sea-Level Monitoring (PSLM) Project Climate and Oceans Support Program in the Pacific (COSPPac) and the Australian Bureau of Meteorology (BoM). The tidal range for Vaitupu and the corresponding levels calculated at Funafuti Port can be seen in Table 9 with respect to Tide Gauge Zero (TGZ - a historical benchmark at Funafuti Port). The Deltares report (Deltares, 2017) used TOPEX global tide models to extract the six major tidal constituents at Nanumaga. These have been used to reconstruct tidal Planes with respect to mean sea level (MSL) for the island, the results of each investigation can be seen in Table 9.

Table 9: Calculated and published tidal plane data at Funafuti, Vaitupu and Nanumaga.

Tidal Plane	Recorded height Funafuti (m MSL)	Published height Funafuti (PSLM, 2019b) (m TGZ)	Calculated height Nanumaga (TOPEX) (m MSL)	Nanumaga (m TGZ)
Highest Recorded Water Level (HRWL) recorded 28-Feb-2006	1.39	3.44		
Highest Astronomical Tide (HAT)	1.30	3.30	1.33*	3.22
Mean High Water Springs (MHWS)	0.82		0.84	2.73
Mean High Water Neap (MHWN)	0.33		0.32	2.21
Mean Sea Level (MSL)	0.00	2.05	0.00	1.88
Mean Low Water Neap (MLWN)	-0.33		-0.32	1.57
Mean Low Water Spring (MLWS)	-0.82		-0.84	1.05
Lowest Astronomical Tide (LAT)	-1.17	0.89	-1.20*	0.69
Lowest Recorded Water Level (LRWL) recorded 27-Feb-1998	-1.49	0.53		
Tide Gauge Zero (TGZ)		0.00		-0.09

**denotes inferred values for Nanumaga from Funafuti tidal plane*

An extreme value analysis (EVA) was undertaken on the positive residual water levels at Funafuti (Figure 18) in order to determine a design water level and has been described in the previous section. Due to its location within the Funafuti Port on the eastern edge of Te Namo, it is expected that the positive residual water levels recorded here are attributed to a combination of the inverse barometric effect (IBE) and wind setup across the lagoon (note: contributions from wave setup and seiching within the harbour basin are expected to be excluded from the signal due to the layout of the port facility and the water level measurements being averaged over an hour).

Due to the differences in geomorphology, orientation and exposure between Funafuti and Nanumaga, the EVA on positive residual water levels calculated for Funafuti cannot be inferred for Nanumaga. The steep bathymetric drop-off and isolation of Nanumaga means that wind setup at the offshore reef edge is minimal. The primary component of positive residual at this location will be due to the IBE from low pressure weather systems moving over the island, a schematic of IBE is provided in Figure 27 and short description below.

As a general rule, for every 10hPa change in surface air pressure, a change of 10cm is seen at sea level. Lowest Mean Sea Level Pressure (MSLP) readings are usually associated with the passage of tropical storms and cyclones. Hence, in order to determine design water levels at the reef edge of Nanumaga, an understanding of the lowest pressure storm to feasibly pass within the vicinity of the island needs to be determined. The lowest recorded central pressure associated with a tropical cyclone in the South Pacific Basin was associated with TC Winston in 2016 with a central pressure of 884hPa – this would have equated to a sea level rise of approximately 129cm at the centre of the cyclone. Although tracking cyclones is notoriously unpredictable, it is generally accepted that due to the northerly location of Nanumaga ~6° latitude, storms of the intensity of TC Winston would be very unlikely.

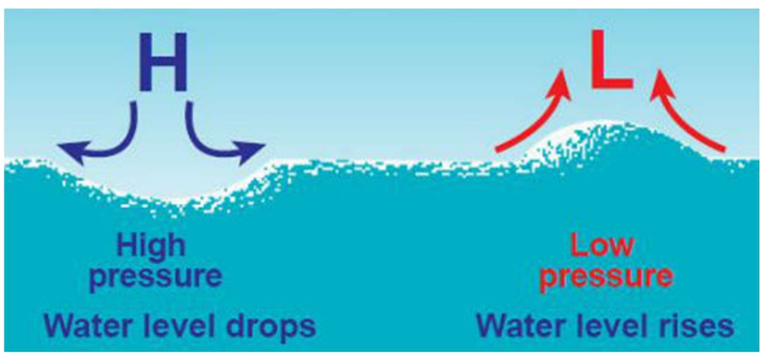


Figure 27 Schematic of the Inverse Barometric Effect, IBE (Source: Swellnet.com.au)

Historical cyclone tracks have been extracted from the International Best Track Archive for Climate Stewardship (IBTrACS) database from 1950 to 2019 (69 years) in order to understand the exposure of Nanumaga to the passage of tropical cyclone. The lowest central pressure recorded to have passed within a 300km radius to Nanumaga was associated with TC Anne (1985) which had a central pressure of 985hPa when passing Nanumaga, this is associated with a 28cm rise in seal level (from 1013hPa).

In order to calculate a conservative design sea level, the highest astronomical tide (HAT) value estimated in Table 9 has been**Table 4** added to the highest IBE value recorded within 300km of Nanumaga. The final calculated 50-year design ARI Water Levels at the offshore reef edge of Nanumaga Island is provided below:

ARI 50yr WL = HAT + max IBTrACS (1950 – 2019)

= 1.33 + 0.28

= **1.61 mMSL**

The largest component of water level at the shoreline will be due to wave setup across the reef flat. This will be caused during large wave events, when sets of wave 'stack-up' on one another against the shoreline forming what is known as an infragravity wave. This temporary increase in water level decreases again as the water is able to drain off the reef. These infragravity waves are described later within the report.

3.3.3 Waves

At the time of writing there had been no measured wave data available for Nanumaga. Long term wave information was once again extracted from the CAWCR hindcast for a deep-water extraction location offshore of Nanumaga and analysed in order to gain an understanding of its operational wave climate. Figure 28 and Table 10 provide details of the CAWCR extraction location, long-term as well as seasonal wave roses and statistics.

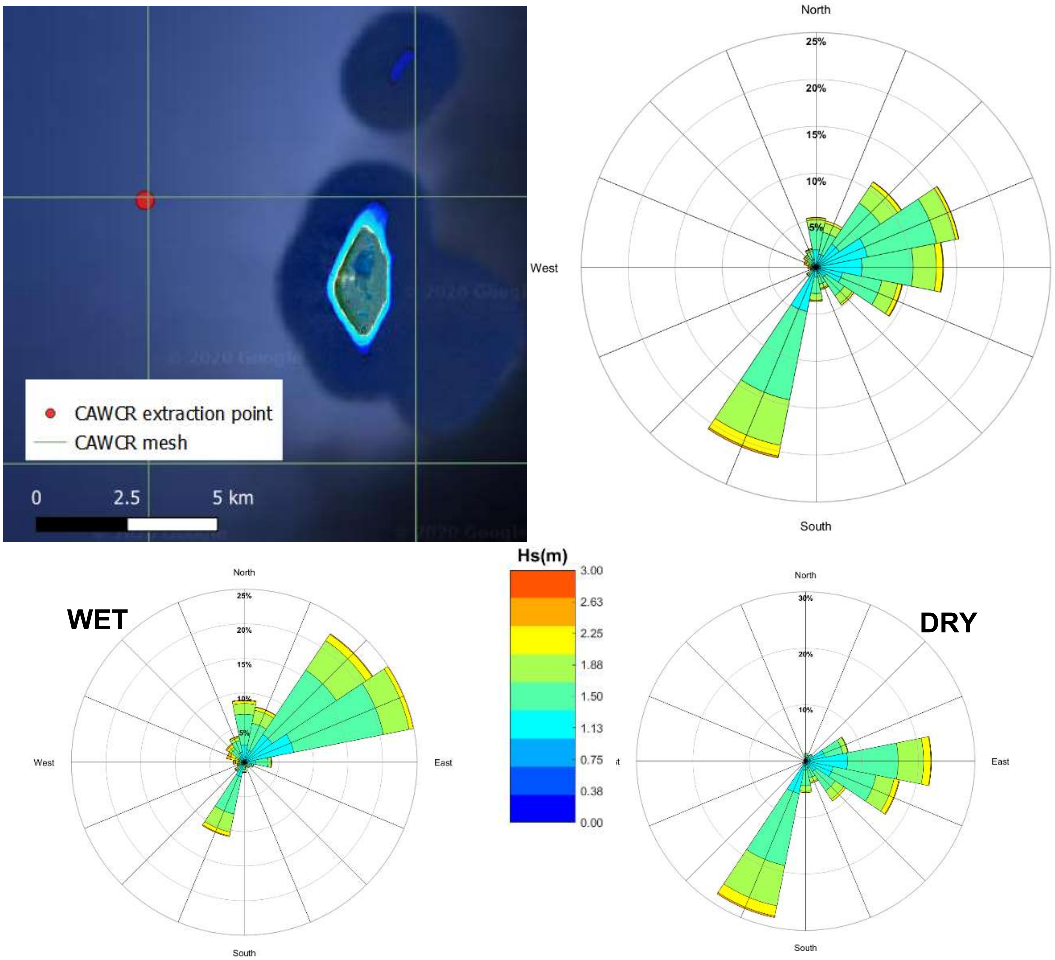


Figure 28: (clockwise from top left): CAWCR grid resolution around Nanumaga and model extraction point, long-term and seasonal wave roses for the deep water CAWCR model extraction point offshore for the wave hindcast information 1979-2019

Table 10: Long term and seasonal statistics calculated for the CAWCR model extraction points in deep water offshore of Nanumaga for the wave hindcast information 1979-2019

Parameter	Statistic	Long term average (40yrs)	Wet season (Nov- Apr)	Dry season (May – Oct)
Hs (m) **Significant Wave Height**	Average	1.7	1.7	1.7
	20%ile	1.4	1.4	1.4
	90%ile	2.1	2.2	2.1
	Max	6.7	6.7	3.5
Tp (s) **Peak Wave Period**	Average	11.4	11.5	11.3
	20%ile	9.1	9.5	8.8
	90%ile	15.6	14.9	15.9

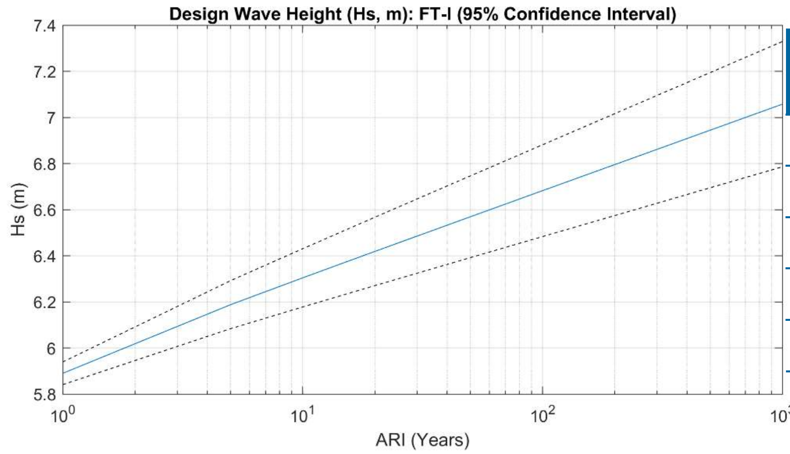
Parameter	Statistic	Long term average (40yrs)	Wet season (Nov- Apr)	Dry season (May – Oct)
	% of Time Sea (Tp<8s)	2%	2%	3%
	% of Time Swell (Tp>8s)	98%	98%	97%
Dp (°N) **Peak Wave Direction**	Weighted Average	129	13	155
	Standard Deviation	64	60	51

The statistics for the deep water (greater than 50m water depth) CAWCR extraction point on the western shore of Nanumaga (Figure 19) may reflect some sheltering by the island to waves from the east, however due to the spatial resolution of the CAWCR model it is expected that this sheltering effect is relatively small and should not affect the analyses at the design site.

The extraction location is seen to be dominated by swell waves (wave periods above 8 seconds), meaning wave energy is usually generated some distance from the island. Locally generated seas (wave periods below 8 seconds) only occur on average 2% of the time. Average Significant Wave Height (Hs) is 1.7m. The dry season is dominated by waves from the eastern and southerly sectors (E to SW) most likely due to waves generated by the trade winds as well as intermittent longer period swells generated in the Southern Ocean. The wet season sees less frequent (and less intense) waves generated from the southerly sector, with a greater frequency in waves generated in the northerly sector (NE to NW), these waves are most likely attributed to long period northern Pacific swells.

The largest waves can be seen to occur from the west and the north west and occur primarily in the wet season and are most likely attributed to the passage of tropical cyclones prevalent between 8°-12° of latitude in the southern hemisphere.

A joint frequency analysis as well as an extreme value analysis (EVA) was carried out on the deep water CAWCR extracted wave conditions based on a peak over threshold approach (PoT), Figure 29.



ARI (years)	Hs (m) *From EVA*	Tp (sec) *From JFA*
1	5.9	12
5	6.2	12
10	6.3	12
50	6.6	12
100	6.7	12

Joint Frequency Table (%) Showing Hs Against Tp for the Period 01-Jan-1979 to 31-Dec-2018 23:00:00

N=350640	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	Total	Cumul.
0.0-0.875	-	*	*	*	*	-	-	-	-	-	0.02	0.02
0.9-1.75	-	-	0.01	0.88	27.26	13.93	9.60	7.30	2.84	0.71	62.52	62.54
1.8-2.625	-	-	*	1.37	13.04	6.80	4.81	6.12	2.71	0.65	35.52	98.06
2.6-3.5	-	-	-	0.20	0.82	0.25	0.05	0.12	0.10	0.03	1.57	99.62
3.5-4.375	-	-	-	*	0.16	0.06	*	-	-	-	0.22	99.85
4.4-5.25	-	-	-	*	0.03	0.07	*	-	-	-	0.10	99.95
5.3-6.125	-	-	-	-	*	0.02	0.01	*	-	-	0.04	100.00
6.1-7	-	-	-	-	-	-	*	*	-	-	*	100.00
Total	-	*	0.02	2.44	41.31	21.13	14.49	13.55	5.65	1.39		
Cumul.	-	*	0.03	2.47	43.78	64.91	79.41	92.96	98.61	100.00		

Figure 29: Peak Over Threshold (POT) Extreme Value Analysis (EVA) of wave heights of deep-water wave heights offshore of Nanumaga based on the 40 year hindcast of waves from the CAWCR model and associated wave period based on a joint frequency analysis (JFA) of wave heights and period.

It is anticipated that the waves generated as a result of the passage of tropical cyclones will be the determining factor on the design of any coastal protection works on the western coast of Nanumaga. Due to the temporal resolution of the CAWCR model (1hours) it is expected that wind speed peaks associated with tropical cyclones may not be resolved and as such wave heights may be under-predicted during these events. As such the design wave conditions determined above should be viewed as operational and not extreme, they provide an insight into the day-to-day wave climate at the project site.

Historical cyclones tracks have been extracted from the International Best Track Archive for Climate Stewardship (IBTrACS) database from 1950 to 2019 in order to understand the exposure of Nanumaga to the passage of tropical cyclone. Figure 30 shows that there have been 6 cyclones passing within a 300km radius of Nanumaga since 1950, it is expected however that Nanumaga would still experience the effects of large waves form cyclones generated in the South Pacific passing outside the 300km search radius due to the remoteness and exposure of the Tuvaluan islands to wind and waves in all directions. A further analysis of the effects of tropical cyclones on the Tuvalu group has been described above and detailed in the Nanumea Concept design report (Bluecoast, 2021).

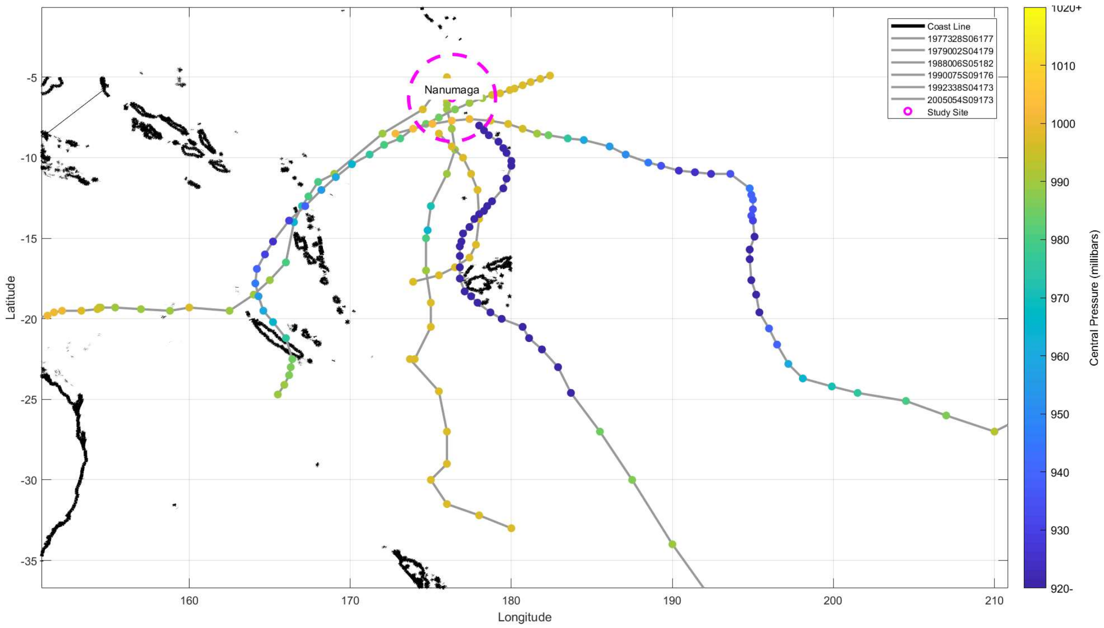


Figure 30: Historical cyclone tracks within a 300km radius of Nanumaga between 1950 and 2019 based on the IBTrACS database.

As a comparison, Deltares (2017) undertook a cyclone modelling exercise to inform the development of ship landing facilities at nearby Niutao Island as well as at Nanumaga. The study defined "design" cyclone conditions based on an EVA of maximum wind speeds around the islands within a radius of 250 km. A worse-case design cyclone track was developed, which could possibly lead to the highest hazards at the two islands, for several return periods. The study estimated a 100year ARI cyclone to have a sustained wind speed of 30 m/s and a mean propagation speed of 3.5 m/s. Deltares then used numerical wave models to transform offshore wave conditions to a 20m depth contour just offshore of Nanumaga, the results of which can be seen in Figure 31. The study has put the100year ARI wave heights along the west coast of Nanumaga in the range of 6.3-6.4m (in 20m water depth). These design wave heights can be seen to be in the same range as that calculated from the EVA of the CAWCR data.

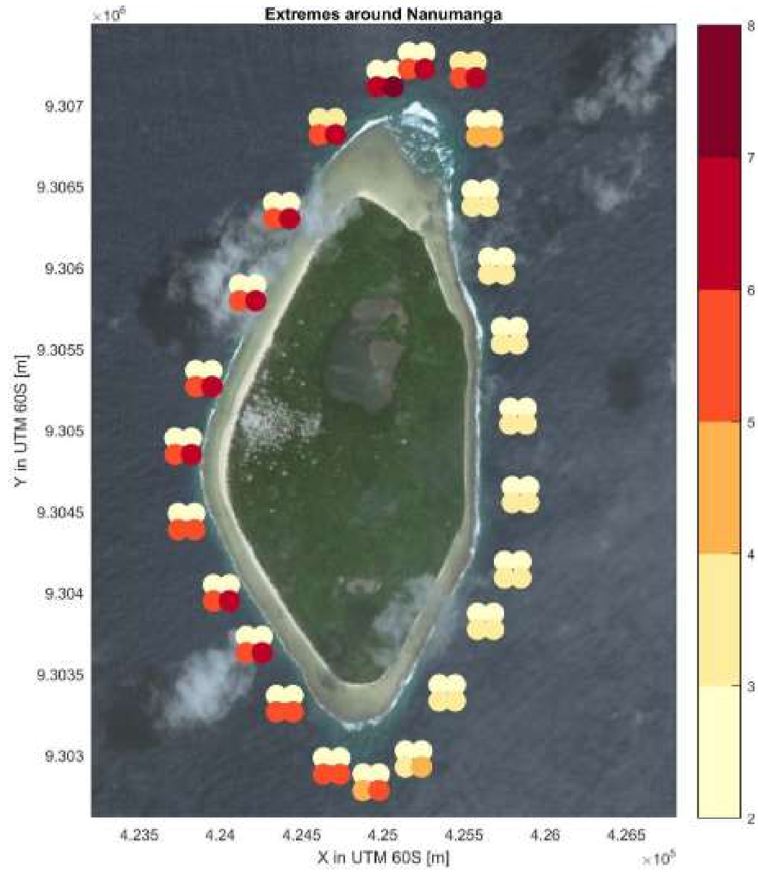


Figure 31 Extreme wave heights around Nanumaga for different return periods year 1 year (upper left) 10 years (upper right) 50 years (lower right) and 100 years (lower left). [source Deltares, 2017}.

3.3.4 Currents

At the time of writing there has been no current measurements in the vicinity of Nanumaga Island. Offshore (deep water) currents are expected to be minimal (<1m/s) and are associated with oceanic circulation and with surface currents being driven by the trade winds. Nearshore currents are expected to be associated with wave transformation processes across the reef flats.

Longshore processes along the west coast of Nanumaga are expected to be associated with refraction of offshore wave energy around each end of the island. Wave energy and setup across the reef will be highest at sections of the coastline exposed to the highest wave energy, having the shortest and shallowest reef flat. During south to east swells, these sections of the reef flat will encounter the highest wave energy. Higher water levels associated with wave setup will drive longshore currents to adjacent (northern) sections of the reef flat where water levels are lower. This is expected to drive a net northerly longshore current along the shoreline during these periods. During the wet season when northerly wave energy is dominant, these longshore processes are expected to reverse with a southerly longshore current expected to dominate.

3.3.5 Conceptual coastal process model

Nanumaga's shoreline processes are dynamic, are linked in a 360-degree whole island process and appear in excellent condition.

Vast quantities of sand and coral debris were added to the upper surface of the western berm complex over much of the length of the shoreline during TC Pam (2015) evidenced by the complete burial of the road in many areas. More work is required to quantify the amount of deposition, but on average the berm appears to have been raised around 30cm over much of its length and in some places, sand was piled nearly a metre deep (SPC, 2020b). The well vegetated upper portion of the berm was not physically reworked during TC Pam other than to have been subject to over wash and deposition. Over the length of the western shore only the lower seaward face of the berm was reworked by storm waves. This natural island-building process has incrementally added to the storm berm height and overall aerial volume of the island. The patterns of deposition and wave reach over the main western berm of Nanumaga during the TC Pam event highlight the importance of set-backs and zoning. A pragmatic approach in Nanumaga would be to avoid any further building west of the main north/south roadway which lay approximately 70m from the present vegetation line. This entire zone is part of the active shoreline system and is not suitable for settlement or building.

Otherwise, the westerly wind and wave energy which attacked the shoreline has pushed a vast deposit of sand northwards (and to a more limited extent southward) to the northern tip of the island. Two large deposits of new sand have accumulated at the northern and southern points of the island. These are being re-worked by daily tides and waves and are being redistributed both along the western and eastern shorelines.

3.4 Nanumea

3.4.1 Geomorphology

The theory of atoll evolution, previously described for Funafuti atoll, can be confidently applied to Nanumea based on the broader scientific body of knowledge, it is also part of the Tuvalu seamount chain. The surface geology of Nanumea was mapped in 1982 as part of a land resources survey of Tuvalu commissioned by the Food and Agriculture Organisation (McLean et al, 1991). The survey identified five major geomorphological landform units on Nanumea islet; the eastern ridge complex, western ridge complex, central depression-lagoon complex, enclosed basin, and lagoon margin (see Figure 32).

Based on the 2019 LiDAR survey and aerial photograph, the atoll structure of Nanumea is shown in Figure 32. The boomerang shaped atoll is 13km in length and 2.5km wide at its widest point, both measurements taken from reef crest to reef crest. The topographic and bathymetric elevation maps show a clear distinction of environments by elevation:

- blues (below -2m) are lagoon and atoll seamount slopes
- greens (between -2m and 0m) are reef flats and sand aprons
- yellows (above 0m) are reef islands comprised predominately of sand.

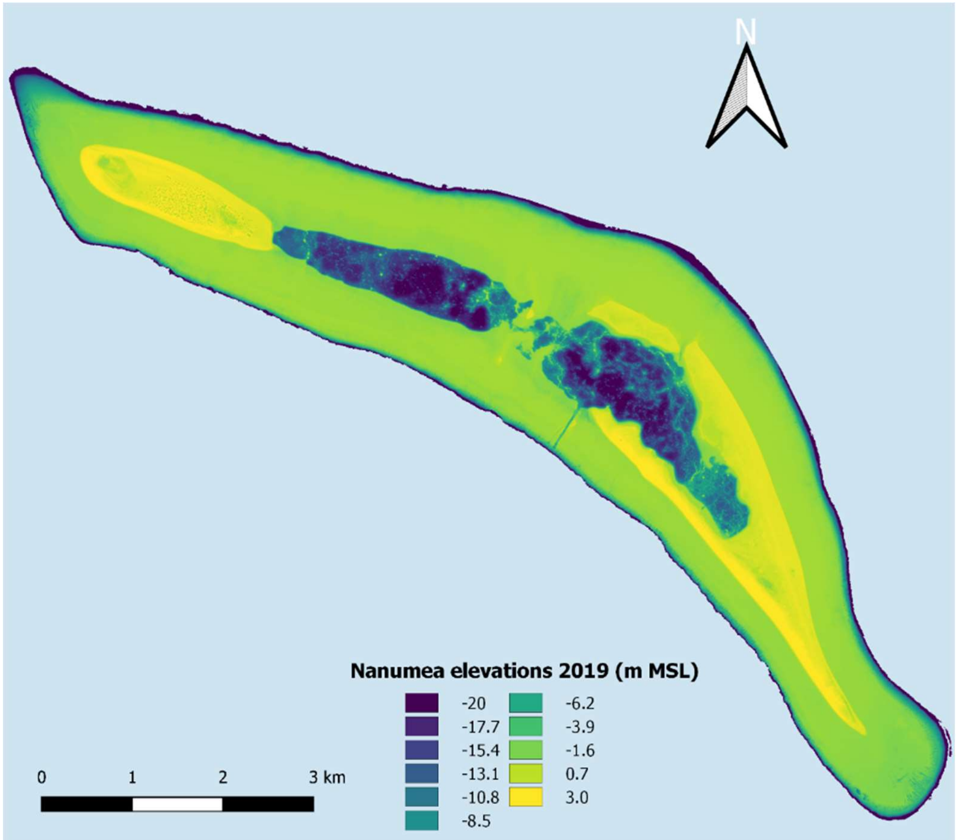
Of the three major reef islands Nanumea is the largest island and covers the south-east portion of the atoll forming a V-shape with two distinct limbs either side of the central lagoon. Each limb of the Nanumea island has a narrow, elongated shape

The reef islands of Nanumea are comprised of unconsolidated biogenic sediments formed by the physical abrasion (under wave action) and biological breakdown of calcium carbonate–secreting organisms that dwell on the adjacent coral reef system. Waves and currents deposit the reef sand and rubble onto the island shores. The location, planform configuration, size, and elevation of islands reflect both the interaction of oceanic swell with reef structures and the availability and grade of sediment for island building (Masselink et al ,2020).

Figure 33 shows a map of the topography (Figure 33a) and cross-sections through Nanumea Island (Figure 33b & c). Figure 34 shows a cross-section of a typical atoll to highlight the structural elements observed on Nanumea (Kench et. al., 2009). Major structural elements of the atoll's morphology include:

- deep lagoon (20–25 m) and reef rim with islands, reef flat, and reef crest
- sand aprons, depositional features created by unidirectional (reef to lagoon) sediment transport, are found on the lagoon-ward edge of the reef flats
- on the islands western coastline reef flats are around 300 to 350m wide with a high ridge or berm (4-5m above MSL) on the ocean side (left) that gently grades down to the lagoon with no ridge present on the lagoon side (right)
- the islands eastern side has a beach and berm system that only rises 2-3m above MSL, the higher western berm (Figure 33a) is due to this coastline having a higher energy wave climate including longer period swells and cyclones
- the islands are comprised of a vegetated core of unconsolidated material mostly loose sand, with some finer organic material from breakdown of vegetation
- active sediment that is mobile under the action of waves and tide in both longshore of cross shore directions, is found on the beaches with the more active sediment transport on the ocean side as opposed to the lagoon shores of the islands.

The island's sediment budget and a conceptual model of sediment movements are discussed further in Section 4.4 and Section 3.4.5, respectively.



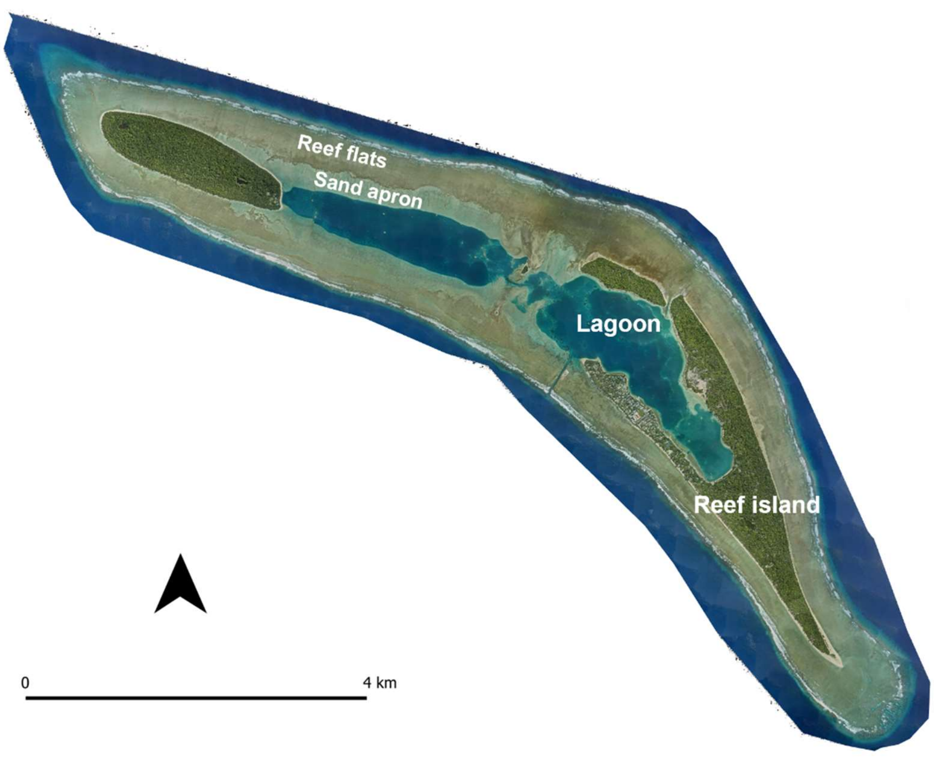


Figure 32: Elevation and aerial maps of Nanumea showing the topography and bathymetry of the entire atoll.

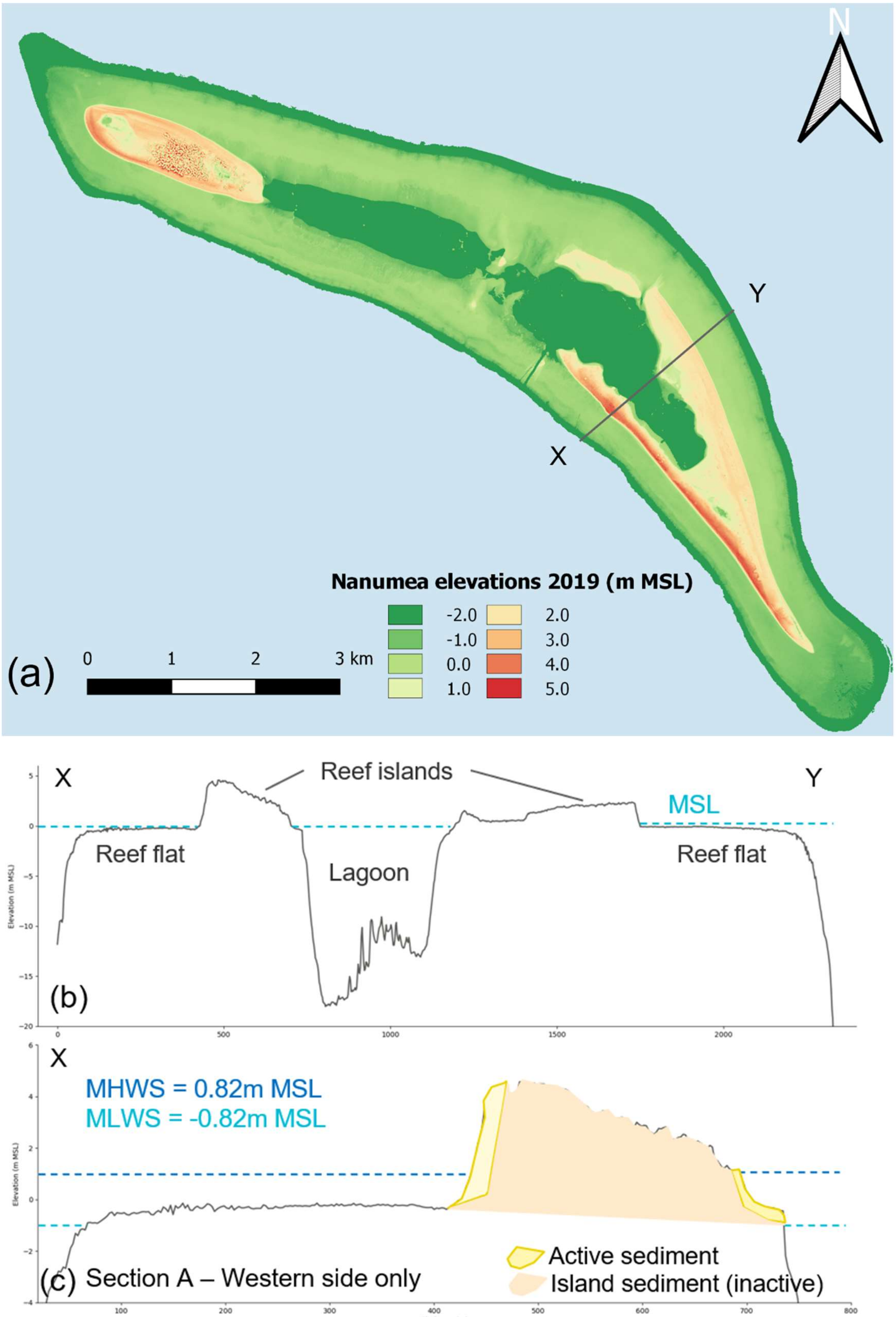


Figure 33: Atoll and reef island morphology and structure. (a) elevation maps of Nanumea showing the topography of the entire atoll. (b) Cross-section of Nanumea atoll showing major structural elements. (c) Western island (village) commonly 50–100m wide with a high ridge on the ocean side.

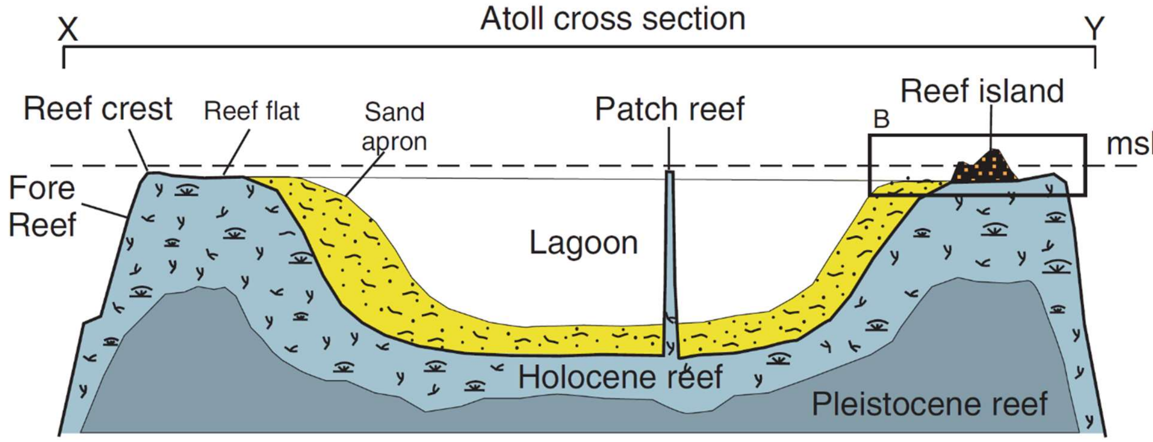


Figure 34: Cross-section of a typical atoll showing major structural elements including deep lagoon (20m) and reef rim with reef crest, reef flat, islands and sand apron identified (source: McLean and Kench, 2015)

3.4.2 Water levels

No long-term water level measurements have been undertaken at Nanumea. The closest tide gauge is located around 460km to the south east at Tuvalu Port, Funafuti (see Figure 35). Due to its relative proximity to Nanumea, similar orientation and geomorphology, the tidal regime at both locations is considered similar. Table 11 presents tidal planes with respect to mean sea level (MSL) at Funafuti based on the measured water levels. Also provided in Table 11 are tidal planes at Nanumea based on the TOPEX global tide model. Good agreement is observed between the comparative tidal planes.

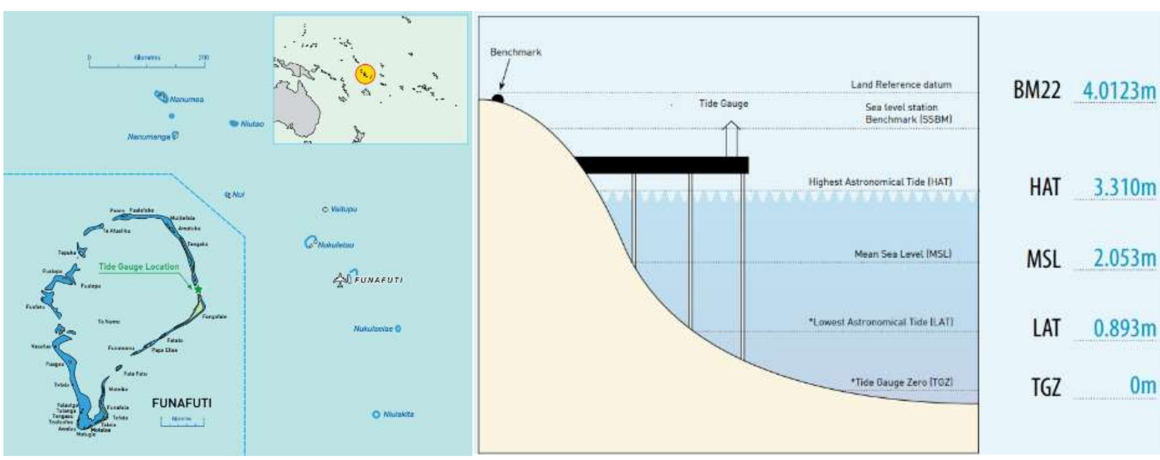


Figure 35: Location of the tide gauge (left) and the predicted heights of key tide components (right) on Funafuti (source: CoSPac, 2020)

Table 11: Tidal planes at Funafuti and Nanumea.

Tidal Plane	Funafuti tide gauge (m MSL)	Nanumea (TOPEX) (m MSL)
Highest recorded water level (28-Feb-2006)	1.39	-

Tidal Plane	Funafuti tide gauge (m MSL)	Nanumea (TOPEX) (m MSL)
Highest Astronomical Tide (HAT)	1.30	1.33*
Mean High Water Springs (MHWS)	0.82	0.86
Mean High Water Neap (MHWN)	0.33	0.33
Mean Sea Level (MSL)	0.00	0.00
Mean Low Water Neap (MLWN)	-0.33	-0.33
Mean Low Water Spring (MLWS)	-0.82	-0.85
Lowest Astronomical Tide (LAT)	-1.17	-1.20*
Lowest recorded water level (27-Feb-1998)	-1.49	-

**denotes inferred values for Nanumea from Funafuti tidal plane*

3.4.3 Waves

The Oceanum wave hindcast (Smit et al, 2020) was selected as the most suitable to characterise the offshore wave climate effecting Nanumea. The Oceanum global wave hindcast is run in WAVEWATCH III v6.07 at a 0.5 deg spatial resolution with 3 hourly outputs This wave hindcast is forced with ERA5 winds as well as ice forcing with data available to the end of 2019. Wave conditions were extracted from a deep-water grid point approximately 60km to the southwest of Nanumea at a location not influenced by any sheltering from. Descriptive statistics are provided in

Table 12. Figure 36 provides the long-term average annual as well as seasonal (dry and wet) wave roses. Wave spectra partitioning for two watershed partitions was also extracted from the third generation hindcast dataset to identify the wind sea and swell waves to further describe the offshore wave climate.

Nanumea's wave climate is dominated by swell waves (wave periods above 8s), meaning wave energy is usually generated some distance from the island. Locally generated seas (wave periods below 8s) only occur on average 5% of the time. The annual average significant wave height (Hs) is 1.67m, with a slight increase in the wet season. The dry season is dominated by easterly wind swells generated by the trade winds as well as intermittent and longer period south south-west swells originating from the South Pacific Ocean and Southern Ocean in the more energetic southern hemisphere winter. In the wet season the southerly swells are less frequent and of lower energy, with a greater frequency of waves generated in the northern sectors. The longer period swells arriving from the north as related to distance northern Pacific storms in the northern hemisphere winter.

The largest waves primarily occur in the wet season and arrive from the west and/or the north-west and are the result of tropical cyclones action in the southern hemisphere (latitudes 8°-12°). The eight largest wave events within the hindcast were identified to cross correlate with tropical cyclone events occurring within the South Pacific.

Table 12: Offshore wave climate statistics for Nanumea from Oceanum dataset.

Parameter	Statistic	Long term averages (42-years)		
		All seasons	Wet (Nov-Apr)	Dry (May-Oct)
Significant wave height (H_s) [m]	Mean	1.67	1.70	1.65
	20%ile	1.39	1.41	1.38
	50%ile	1.63	1.63	1.62
	75%ile	1.85	1.86	1.84
	90%ile	2.10	2.13	2.07
	99%ile	2.81	3.17	2.53
	99.5%ile	3.18	3.54	2.63
	Max	6.77	6.77	3.56
Peak wave period (Tp) [s]	Mean	10.7	10.9	10.5
	20%ile	8.8	9.3	8.5
	50%ile	9.9	10.3	9.3
	75%ile	12.2	12.2	12.2
	90%ile	14.4	13.9	14.8
	99%ile	17.8	16.9	18.1
	% of time sea (Tp < 8s)	5%	3%	7%
	% of time swell (Tp > 8s)	95%	97%	93%
Peak wave direction (Dp) [˚TN]	Weighted mean	109	31	145
	Standard deviation	60	55	49

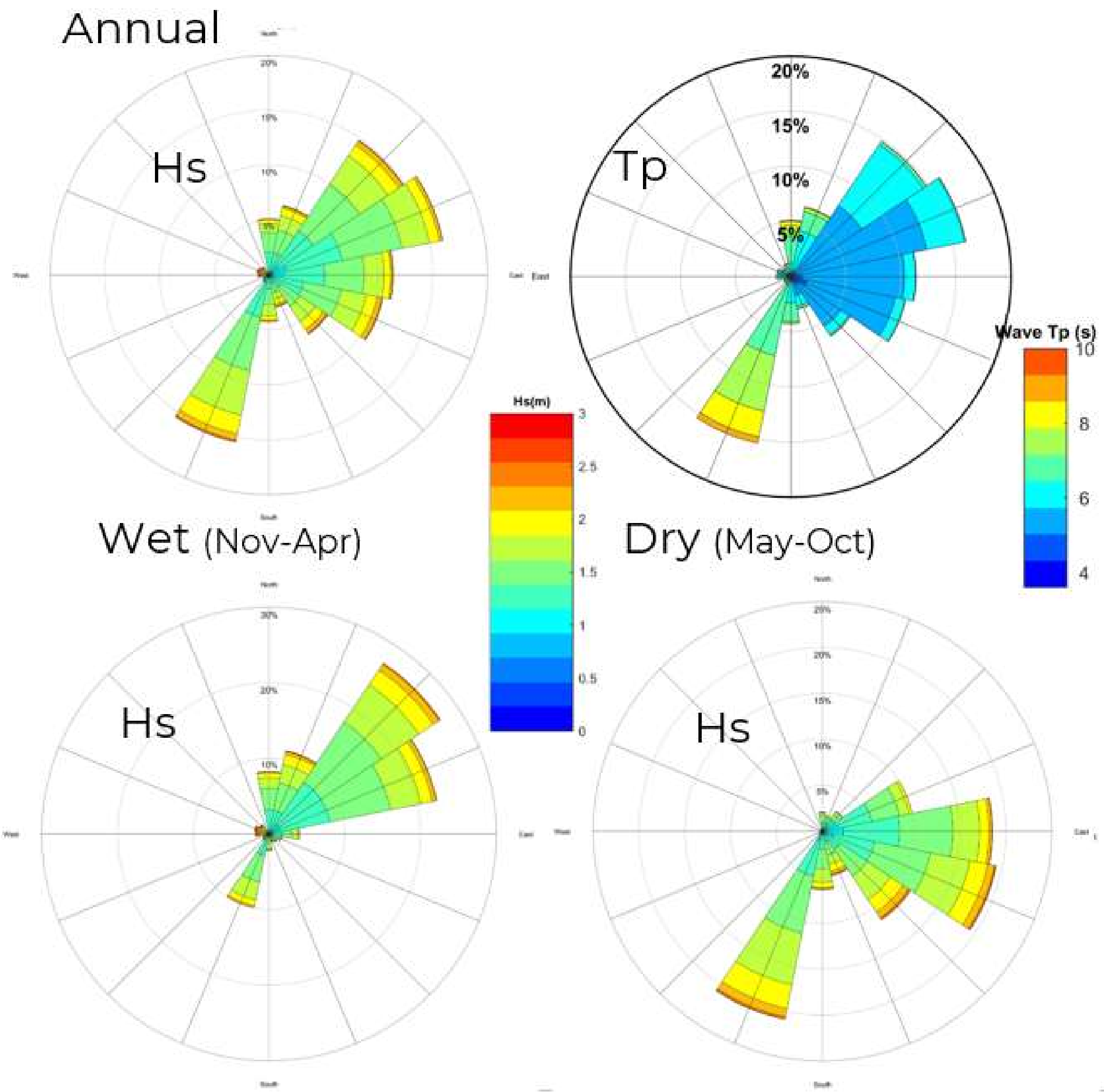


Figure 36: Offshore annual, wet season and dry season wave climate roses from the Oceanum hindcast dataset extracted offshore of Nanumea.

A more detailed analysis pf the wave climate at Nanumea has been undertaken in the Nanumea Concept Design Report (Bluecoast, 2021).

3.4.4 Currents

At the time of writing there has been no water current measurements in the vicinity of Nanumea. Offshore (deep water) currents are expected to be minimal (<1m/s) and are mostly associated with oceanic circulation or with surface currents being driven by the trade winds.

Nearshore currents on the reef tops are expected to be associated with wave processes across the reef flats as well as tidal flows in and out of the small lagoon through the western passage. When waves arrive perpendicular to the reef crest, wave breaking and wave setup over the reef drives currents towards the lagoon. When waves arrive at an oblique angle to the reef crests and island shorelines, longshore currents may along the west coast of Nanumea. During south

south-west swells, net northward longshore currents would be expected, ultimately slowing into the lagoon. This is expected to also drive a net northerly longshore transport of sand along the shoreline when wave heights are large enough. During the wet season when intermittent storms result in westerly wave events, these longshore processes are expected to reverse with currents and sediments flowing toward the islands southern tip.

3.4.5 Conceptual coastal process model

The below sub-sections provide a summary of a quantified coastal process model and sediment budget of Nanumea undertaken as part of the Bluecoast (2021).

Atoll cross-shore processes

The fringing reef that surrounds the atoll are of paramount importance in understanding the morphological response of the sand and gravel islands themselves. Atolls are inherently resilient structures (i.e., their existence relies on the ability to naturally adapt to changes in ocean conditions including sea level rise). Understanding this inherent resilience is critical to understanding the coastal vulnerability of Nanumea island. The key processes of importance in the atoll cross-shore profile are displayed in Figure 37. Working from the ocean to lagoon these are described below.

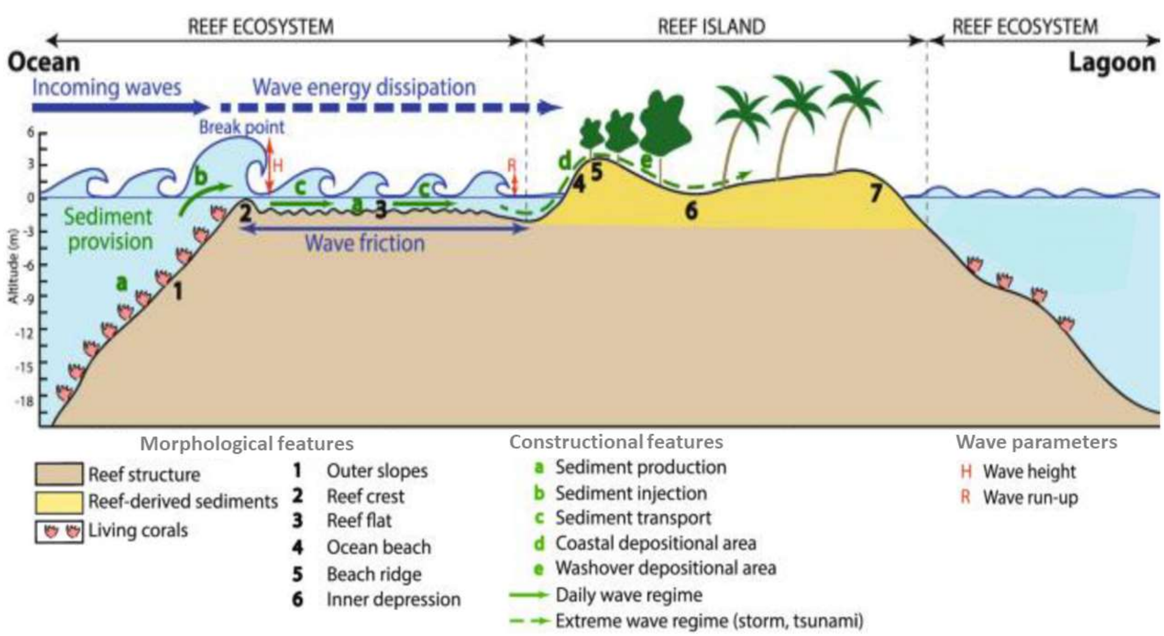


Figure 37: Conceptual model of atoll cross shore processes (Adapted from Duvat et al, 2019)
Note: Nanumea's reeftop is shallower than that depicted above, see Figure 33 for reference.

Sediment production

Sediment production in atoll environments like Nanumea is controlled by two important mechanisms. The first is physical breakdown of the reef structure, that is wave abrasion breaking up reef branches or other reef structures. Physical breakdown by waves tends to produce larger sized material including coral rubble and boulders. While it takes a long time for this larger size material to breakdown to sand-sized sediments the gravel, cobble and boulder sized material itself is important for island building. The production rate of material by mechanical abrasion by waves is influenced by reef health and wave energy, particularly long period swells which tends to have an abrasive effect of the reef. Nanumea has a mildly active long period swell climate and an active cyclone wave climate that assists with the mechanical breakdown of the reef and is an important source of material to support island building and maintenance.

The second mechanism is biological. Foraminifera (single-celled protists with shells – see Figure 38) plays an important role in biological sediment production at Nanumea. The extensive hard reef flats are the key habitat for these organisms. Geotechnical investigations undertaken on Nanumea found the most abundant component of sediment on the nearshore berm deposits were large benthic foraminifera, notably *calcarina* and *baculogypsina sphaerulata*, with occasional *amphistegina* and *marginopora* (SPC, 2020a). Foraminifera dominate the sand fraction of the sediment (>70%). Coral fragments were seen to dominate the gravel fraction of the sediment with occasional bivalves, gastropods and coralline algae intermixed.

Figure 38: Examples of large benthic foraminifera types like those produced on the reef top at Nanumea; *calcarina* (left) and *baculogypsina sphaerulata* (right), (Source: Bernard Remaud)

Another source of biological sand size sediment is mainly by grazing parrot fish but also urchins. By feeding on the reef, these animals break down the reef framework and produce significant quantities of sand-sized sediments. Therefore, the population and spatial distribution of grazers that bio-erode the reef is a key factor in the production rate of sand-sized material. The abundance of reef grazers is a function of reef health. A naturally healthy reef system will tend to support a healthy population of foraminifera and/or reef graziers, maintaining an overall more resilient system.

Carbonate sediment production regimes contribute to island building and maintenance (Perry et. al., 2011). Maintaining the supply rate of sediments, from either physical or biological breakdown, relies on a healthy coral reef system.

Living breakwaters

The living reefs that fringe the atoll act as natural breakwaters in dissipating wave energy with wave breaking on the reef crest. Wave driven currents, water level set-up, wave dissipation and infragravity waves occur on the reef flats. The water levels on the reef control the height of the beach ridges. Interestingly it is also the water level that also controls the height of reef crest – reefs are living structures and can grow vertically upwards with sea level rise.

The fringing reef allow for the sand/coral rubble to accumulate and form islands characterised by oceanward beach ridges built by waves. The relationship of the island elevation to that of the highest water level of the reef top is of particularly significance. Water levels at the oceanward shoreline are also subject to wave set-up and wave runup across the reef platforms, meaning that swash reaches well above high tide levels on the more exposed beaches.

Onshore sediment transport

As with Nanumaga the sediments produced across the fringing reef of Nanumea, both sand-sized and larger, are transported onshore by wave action. This onshore movement of sediment builds and sustains the shoreline systems on the islands. Nanumea is distinct from Nanumaga in that it has significant potential for its inner lagoon to continue to infill with sediments over time and in turn for more island accommodation space to be produced. In essence, Nanumea island is expected to be in a net building phase at this time. This does not mean all shores will expand, it means overall, the net land area should gradually increase over decadal timeframes. This is however entirely contingent on surrounding reef productivity, structure and function, likewise ambient climatic and sea conditions must remain relatively stable. At the time of writing Nanumea lagoon continues to infill with sediments and this is evidenced by widespread lagoon basin infilling apparent around its inner margins from the 1970's to the present. This is because the potential for sediment production and transport rate is far greater than the rates required for shoreline stability alone.

At the island foreshore and like Nanumaga, wave run-up occasionally over washes during periods of cyclonic wave activity depositing sand and coral rubble, contributing to the accretion of the islands. Such island adjustment is driven by wave overtopping processes transferring sediment from the beach face to the island surface as well as large waves and elevated water levels during storms accelerating long shore transport. These process can be disruptive for human settlements but the overall cycle of shoreline response during storms and during normal conditions is part of natural shoreline building and maintenance mechanisms and allowing the "flexing" of shorelines is important for natural adaption and resilience.

The Nanumea islands are characterised by oceanward storm berms built by storm waves; those berm ridges reach heights of 2 to 4.5m above Mean Sea Level (MSL). The relationship of the land elevation to that of the highest water level is of particular significance. Water levels at the oceanward shoreline are also subject to wave set-up and wave runup across the reef platforms, meaning that swash reaches well above high tide levels on the more exposed beaches.
Longshore sediment transport

Wave processes play an important role in the redistribution of sediments along the Nanumea shorelines. Longshore sediment transport is the process of sand or other sediments moving in an alongshore direction and is driven by waves arriving oblique to the shoreline orientation. The role of longshore sediment transport is discussed below for Nanumea in relation to both cyclonic and non-cyclonic wave conditions.

Cyclonic conditions conceptual model

While cyclonic conditions happen very infrequently (less than 0.5% of the time) these conditions are critical to understanding sediment movements and morphological change on the islands. A conceptual model of sediment movements during cyclonic conditions is displayed in Figure 39.

During cyclonic periods, sediment is removed from the beach face by elevated water levels and wave action on the reef flats. The waves overtop and overwash the western berm complex, inundating the island with sediment laden salt water. The sand is deposited as the overwash moves inland, and the saltwater infiltrates into the island.

Due to the westerly to direction of cyclonic waves the longshore movement of sand is predominantly directed southward, with small components moving towards drop overs at the northern tip. The predominate southward gradients drive sediment along the shore towards the southern tip, as evidenced by the deposition observed in the satellite imagery from 2016 and 2020 post cyclones TC Pam and TC Tino, respectively (see Bluecoast, 2021). The sediment deposition at the southern tip, depending on the intensity and duration of the incoming wave energy, can be redistributed around the tip and deposits on the eastern shore and in some instances is lost over the eastern reef edge. The entire southern end of the island may also be re-aligned to the east during these events.

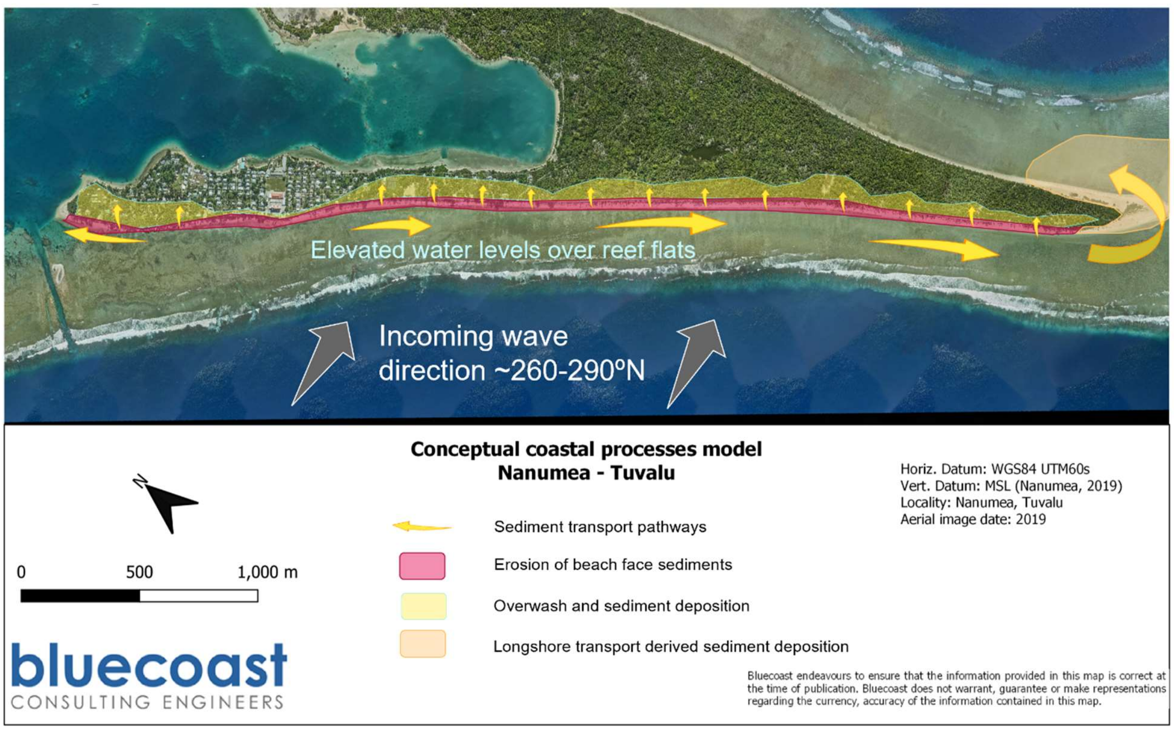


Figure 39: Conceptual coastal processes model for typical cyclonic conditions.

For TC Pam, shoreline data has been combined with the 2019 LiDAR to estimate the amount of sediment transported during the event. Event quantities are:

- A total of 240,000m^3 (±20%) of sand was eroded from the western shoreline of Nanumea.
- Of this it was estimated that 48,000m^3 (±30%) was transported southward by longshore transport and deposited at the southern tip[1]. Around 9,500m^3 (±35%) was transported alongshore in a northward direction and deposited on the drop over at the northern tip.
- By deduction, it is therefore estimated that 182,500m^3 (±25%) was deposited as a layer of sediment over the islands surface (i.e., the developed or vegetated areas). These depositions although a nuisance can be considered island building as it added to the elevation of the island.

[1] It is noted that no estimate was made of any losses of sand over the southern reef crest.

Ambient conditions

The most dominant ambient (non-cyclonic) component of the wave climate effecting Nanumea's village is the southerly swell. These southerly waves are the primary component of the wave spectra around 28% of the year, and more common in the dry season. Moreover, low amplitude southerly swells are a secondary component of the regions wave climate a further 40% of the time. These persistent southerly swells as also critical to understanding sediment movements and morphological change on the islands.

A conceptual model of sediment movements during ambient southerly swell conditions is displayed in Figure 40. This is explained by:

- As discussed above sediment produced across the reef is transported onshore by wave action.
- This loose sand and gravel forms sand islands, the beach systems of which are subject to movement by wave action, particularly under higher-than-normal tides. The planform shape of the islands is influenced by atoll structure and longshore sediment transport. There is a zone of active sediment movement around the island edges
- During ambient conditions, the predominant southerly swell drives net northward sediment transport along Nanumea's western shoreline at approximately 1,000 to 2,000m^3/year. Any sediment that reaches the end of this pathway finds it final deposition on the drop overs (see Figure 41).
- The northward ambient longshore transport direction opposes the southward cyclonic event transport along much of the western shoreline. This provides a mechanism for beach recovery for the ocean facing village shoreline following cyclones. The net sediment transport direction over the long term depends on the frequency and intensity of cyclones events. Due to TC Pam and TC Tino, the recent net transport direction is southward.
- Sediment that reaches the southern tip of Nanumea can be distributed around the southern tip by large waves. Once on the east coast, trade wind swells transport the material northward, ultimately reaching the lagoon and final depositional areas within the lagoon.
- The effect of tidal flows through the American channel of sand movement are unknown but not considered significant, likely having come into an equilibrium sometime after the channel was dredged.

The dynamic nature of the southern tip of the island is reflected in morphological response to both ambient and cyclonic conditions. The lack of human settlement and infrastructure at the southern tip also speaks to its dynamic nature. This area is, therefore, not considered suitable for future development.

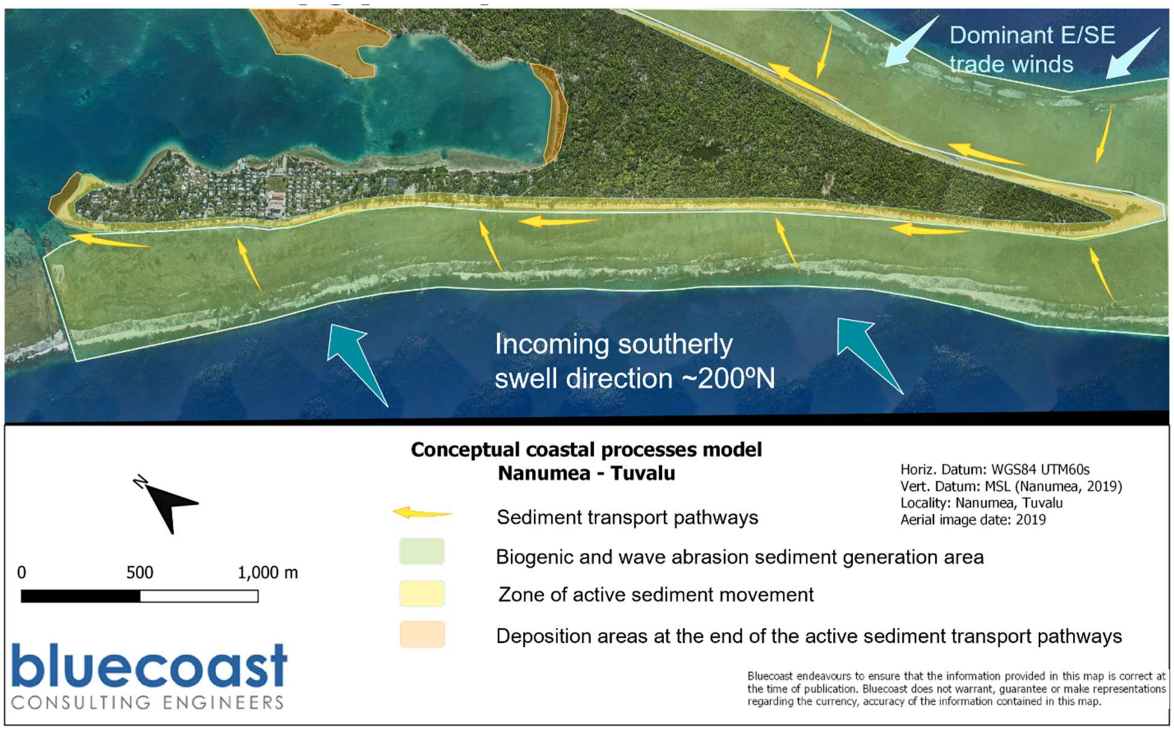


Figure 40: Conceptual coastal processes model for ambient conditions (typical non-cyclonic).

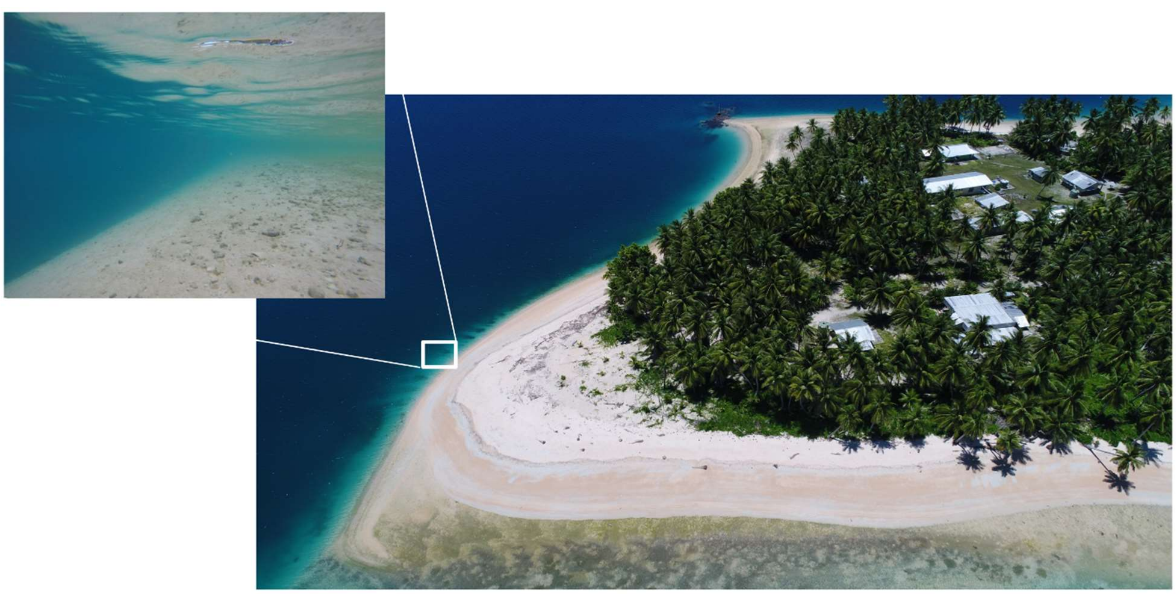


Figure 41: 'Drop overs' when sediment deposits on the north facing slope at the north-west tip of Nanumea (source imagery: SPC)

4. Design development

The first stage in any coastal protection project is identification of the hazard which requires mitigation. A range of measures may then be used to mitigate the hazard, including avoidance of hazardous locations or relocation of assets. Following PRIF (2017), there are three main mechanisms to improve coastal resilience:

1. **Avoidance or Retreat from the hazard,** through either planning restrictions or by relocating assets out of the hazard-affected area, will eliminate the likelihood and therefore the risk
2. **Accommodate** the hazard by reducing the likelihood or magnitude of the hazard or reducing the consequence of the hazard. Use in combination with hazard Avoidance.
3. **Protect** against a particular threat or range of threats through construction of physical works. These works may include “hard” (revetment or seawall) or “soft” (beach nourishment, mangrove planting) or a combination of the two.

PRIF (2017) state that *Protection* options should be used in combination with *Accommodation* options, such as ecosystem-based approaches to widen benefits and minimise adverse effects, and with Avoidance options to ensure long-term resilience. In the setting of the TCAP project, due to the limited availability of land, the options to avoid and or relocate assets are simply not feasible when land availability is limited, or infrastructure is expensive to relocate. In these cases, the land and assets must be protected.

Two initial investigations have been undertaken to determine the extent of shoreline vulnerability on the islands of Nanumea, Nanumaga and Fongafale and to identify possible resiliency measures:

- TCAP Initial island site visit Report: Nanumea and Nanumaga, November 2017 (Webb, 2017a)
- Government of Tuvalu Cabinet Paper: TCAP Output 2 – Design of coastal works on Funafuti, Nanumea and Nanumaga. (Webb, 2018)

The initial rapid site assessment (Webb, 2017a) and follow-up investigations yielded that these islands are highly vulnerable to the impacts of climate change as well as ongoing marine hazards and efforts to improve coastal resilience have many immediate and longer-term benefits. The site visits showed that avoidance or retreat from coastal hazards was not possible due to availability of land (due to both physical restrictions and land ownership and customary technicalities).The subsequent Cabinet Paper (Webb, 2018) introduced both hard and soft conceptual coastal protection measures across each island, specifically as seen in Figure 42 and described below. The cabinet paper was successful in gaining acceptance of the options from the Government of Tuvalu and support from project donors; UNDP and GCF.

- Nanumea and Nanumaga:
  - Berm Top Barrier (or sea dike) along approximately 1315m and 630m length of coast respectively. This is considered a “soft” measure.
  - Seawall (revetment) along approximately 270m and 200m respectively. These are considered “hard” measures.
- Fongafale

- Reclamation. An engineered revetment will be placed along a 710m length of lagoon shoreline in Funafuti to act as a bund for dredged material from the lagoon.

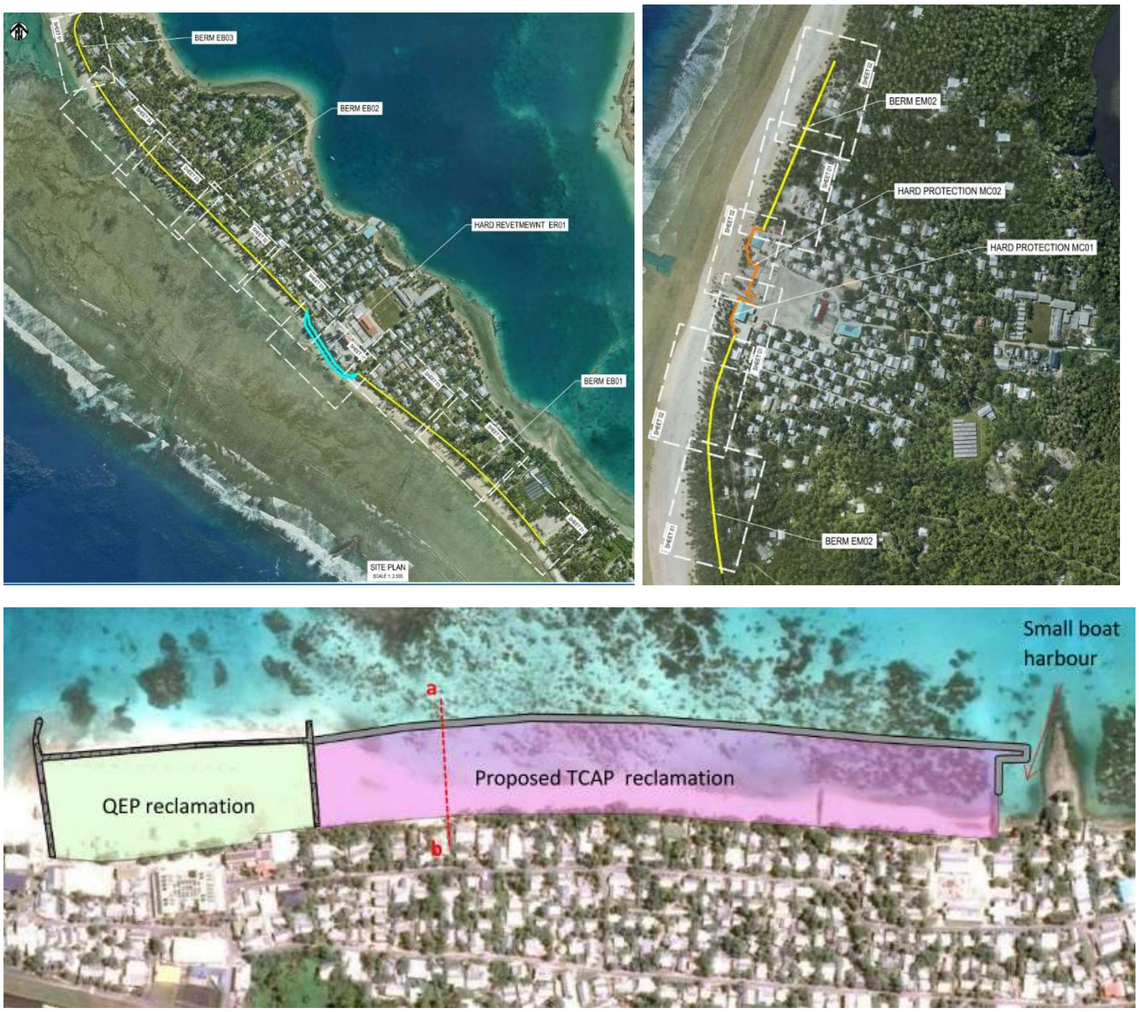


Figure 42: Proposed hard and soft coastal adaptation measures recommended following the TCAP Cabinet Paper to GoT; (clockwise from top left): Nanumea, Nanumaga, Funafuti. (Source: Webb, 2018).

Design and implementation of coastal adaptation measures on remote Small Island Developing States (SIDS) such as Tuvalu is markedly different and more constrained than similar endeavours on more developed and larger Pacific nations such as Australia, New Zealand and even New Caledonia and Fiji to an extent. Similarly, the more remote outer islands and atolls of SIDS present further challenges when it comes to design. In addition to the initial capital expenditure and maintenance costs the following aspects of each site-specific project also need to be addressed;

- wave climate (design wave height),
- water levels at the shoreface
- inundation and overtopping allowances,
- design life,
- availability of suitable materials,

- constructability of design including availability and willingness of suitable contractors, transportation of materials, plant and labour, etc.

Each project site will have site-specific environmental, social, and cultural sensitivities unique to that location. The following sections describe the design development process and reasons for and against each coastal adaptation measure

4.1 Berm Top Barrier

The identified at-risk sites of Nanumea and Nanumaga are described as having reef-mediated shores. The site visit following TC Pam (TCAP, 2017) showed that the shoreline processes are dynamic, include the entire shoreline and that the key morphological feature of both islands is the storm berm on their western shore.

A key impact of sea-level rise and other climate change stress factors on atolls is that living fringing reefs may become less effective at mediating the ocean wave energy before it reaches the islands shores. This is expected to result in more frequent and more destructive erosion and flooding events. If reef edge features are degraded, sea level rise will gradually increase the “wave energy window” over the reef during high tides and potentially increase the propensity of waves to overtop the foreshore berm and cause more marine flooding or over-washing. However as described above, overtopping events like TC Pam also deposit sediment over the berm and incrementally added elevation to the foreshore berm system and inactive sediment zone. This natural building mechanism increases the storm berm height and its resilience to subsequent over topping events.

As an example, recent research by Masselink et al (2020) describes the net effect of sea level rise on the cross-shore profile of atoll islands like Nanumea (see Figure 43). The results present an island-building model whereby island topography can increase in height (adjust vertically) and migrate landward via the rollover process. It follows that storms can be important phenomenon that can either increase or decrease natural resilience to sea-level rise, depending on intensity and frequency. Masselink et al (2020) results found that islands exposed to periodic low-volume overtopping will build vertically at nearly the same rate as sea-level rise. In contrast, episodic high-volume overtopping can flatten islands and increase hazard exposure.

Fatato (Figure 43) is an uninhabited atoll island that is essentially a rubble storm ridge. It has little soil development, no fresh water and very simple vegetation. Masselink’s et al (2020) investigations on Fatato, are thus not directly comparable to more complex island features like Nanumea/Nanumaga, but they do show that atoll shoreline / berm systems may have greater potential resilience than previously thought. This is an extremely important consideration in respect to interventions on such shores. If atoll island shores have a measure of natural resilience (and therefore can offer protection to land further inland) great care must be exercised in interference with for example hard structures which replace these shores or disturb sediment transport.

In addition, Duvat’s (2018) review of over 700 atoll islands globally showed that atoll land area has not shown net reduction in response to sea level rise to date. Over 88% of islands showed stability or an increase in area, while only approximately 11% had contracted. The implications of this research are that atoll islands likely has a degree natural resilience and have some capacity to adjust to higher sea levels. However, the process of vertical island building has only been documented in respect to shoreline systems and an island’s capacity to persist into the future as climate change impacts strengthen and sea level rise accelerate, is in question. Key to the natural capacity for resilience is uninterrupted sediment supply. As climate change threatens sediment productivity (reef decline) as well as transport patterns (changes in the incidence and frequency of storms) any measured capacity for past resilience, cannot be assumed far into the future. Despite the uncertainty, it is clear that sediment is a valuable resource to atoll communities and understanding the sediment budget and island building processes now and into the future will be critical to adapting to and working with the natural resilience of these landforms.

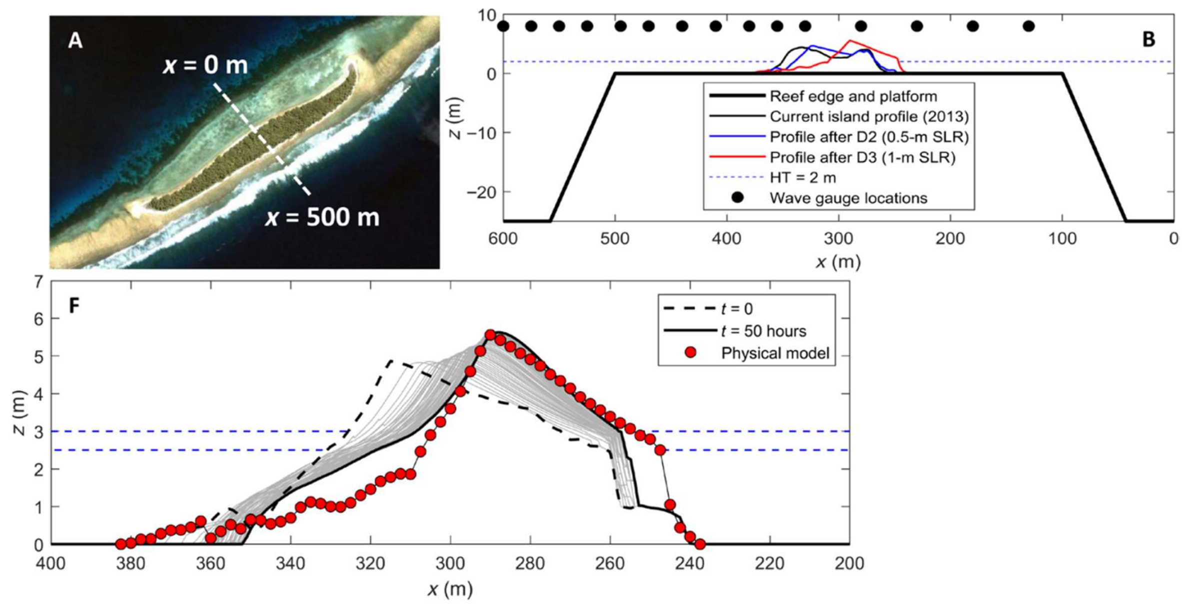


Figure 43: (A) Aerial photograph of Fatato, Funafuti atoll, Tuvalu; white dashed line indicates central profile line. (B) Experimental setup in the physical and numerical model. (F) Measured and modelled reef island morphology after 50 hours with sea level raised from 2.5 m to 3 m. (Source: Masselink et al, 2020)

Without protection, the communities on Nanumaga and Nanumea will be required to manage the risk associated with increased periodic flooding on an adjusting island. The village areas on both island where most buildings are built on concrete slabs on the ground these natural processes are difficult to accommodate. Marine flooding also kills subsistence tree and perennial crops and damages fragile soils and fresh groundwater systems. Thus, measures to raise the berm height to prevent overtopping are urgently required to protect built assets and important natural resources.

In less developed areas of these islands, the communities may have a greater potential to work with nature and integrate hazard management with the way island landforms will naturally evolve. This can be achieved by identifying areas of natural resilience and stability, while adapting infrastructure and agriculture resources to accommodate occasional flooding and sediment movement. For building this means that traditional construction methods whereby floor levels are elevated on piles are preferred. Strong planning controls to build adaptable infrastructure and for smart land use planning around agriculture resources is suggested for all future development. Adequate buffer zones running parallel to the island foreshore are a crucial tool to safeguard development from marine hazards. The incorrect assumption that the dynamic shoreline system stops at the contemporary vegetation line of the island must be supplanted with the more factual definition of the foreshore, which includes the storm berm system which stretches 10's of meters inland from the vegetation line over the crest of the storm berm. The entire storm berm system of these islands is dynamic, it is routinely subject to wave overtopping hazard and sediment deposition. This entire zone is part of the dynamic shoreline system and not appropriate for most human building and development aspirations.

TCAP (2017) made the following recommendations following observations of the impacts TC Pam had on the islands and the subsequent natural island recovery process:

- Over wash was consistently raised as a major concern arising from TC Pam.
- Steepening of the shoreward berm face occurred; however, the overall berm system was seen to not been destroyed but built higher.
- Shoreline processes appeared to be gradually re-establishing the beach face on the lower seaward side of the berm.
- Shoreline processes are dynamic, are linked in a 360-degree whole island process and appear in excellent condition
- By far the most valuable strategy is the prevention of buildings close to the crest of the berm and certainly not on the seaward slope of the berm.
- Land use in the potentially hazardous shoreline zones (anywhere on the berm system) should be avoided and at least closely managed so that property and important infrastructure is kept out of harm's way.
- Hard infrastructure (seawalls) used to replace the natural shoreline would inhibit the natural process of recovery, likely such engineering would result in more damage
- Planned intervention by this Project should not interfere with or prevent the natural and continuing beach recovery process by building hard structures on the shore. Rather it should explore less obtrusive measures to bolster the berm system and reduce over wash risk and associated flood damage.

TCAP (2017) and TCAP (2018) have recommended an innovative, soft engineering approach to coastal protection on these islands, an artificially heightened Berm Top Barrier (BTB). BTB's, although not a new concept in coastal engineering, are a relatively unused methodology throughout atolls and islands. They provide a more sustainable, working-with-nature approach that aligns with the natural island-building processes of these environments by building the height of the western berm complex. A BTB can be viewed as essentially a sloping sea dike.

Historically, sloping dikes have been the most widely used option for sea defences along the coasts of Northern Europe, in particular Scandinavia, the Netherlands, and the UK protecting the land behind from flooding, and sometimes providing additional amenity value. Berms (or sloping dikes) have slopes gentler than 1V:15H.

When designing sloping dikes, the following key parameters need to be understood in order to maximise their efficiency:

Overtopping:

- Overtopping of the dike creates "discharge" on the landward side of the structure and is caused by waves running up (and over) the face of the dike,
- A "green water" case is if run-up levels are high enough to cause a continuous sheet of water over the crest

- Overtopping can also occur when large waves break on the seaward face of the dike, splashing into the air up and over the crest. This volume is less than the “green water” case.
- A form of overtopping carrying less volume again can also occur when strong onshore (towards the land) winds carry the spray from broken waves over the crest of the dike.

Design of dikes uses empirical formulation to inform a design that minimises the amount of overtopping (l/s/m) to within acceptable limits.

Wave Run-up:

- Wave run-up height is defined as the vertical difference between the highest point of wave run-up and the still water level (SWL), as seen in Figure 44.
- $R_{u2\%.}$ is the wave run-up height which is exceeded by 2% of the number of incoming waves at the toe of the structure and is use for the design of sea dikes.

Wave run-up and wave overtopping for coastal dikes and embankment seawalls are mainly determined by empirical formulae derived from experimental investigations due to the stochastic nature of wave processes.

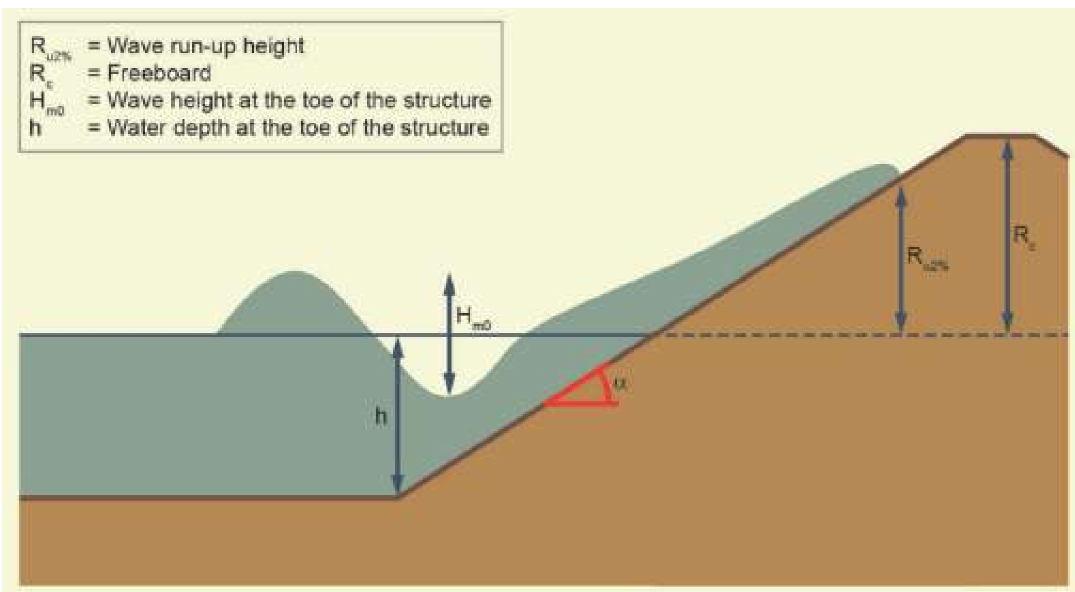


Figure 44: Definition of the wave run-up height Ru2% on a smooth impermeable slope (Source: EurOtop, 2018).

The key factors that control the amount of wave run-up and overtopping of the structure are:

- Wave height at the toe of structure
- Wave period and direction at toe of structure
- Water level
- Wind speed and direction

Design parameters that are able to be altered to minimise overtopping and run-up are:

- Crest height
- Crest width
- Offshore slope
- Permeability of the structure face

Of these, the crest height of the berm will have the largest impact on reducing the amount of overtopping and runup. The following options have been developed for the BTB's on Nanumea and Nanumaga with the aim of increasing the crest height of the storm berm. Each option has been considered with the following key constraints in mind:

- Materials used for the BTB must be sourced on-island or can be readily imported via means of either a beach landing or access through the small boat harbour/channel on each island (generally small vessels with drafts of less than 1m)
- Plant and machinery must also be restricted to semi-manual labour, meaning any equipment required to construct the BTBs should also be sourced on-island or readily imported via means of either a beach landing or access through the small boat harbour/channel on each island.
- Where possible utilise local labour.

4.1.1 Geotextile sand container (layered)

Geotextile Sand Containers (GSC), sometimes referred to as sandbags, geobags or geocontainers, have been used in coastal protection works for the last 30-40years with mixed results. Generally, geotextile fabric has been used as filter material between sub layers in revetments, seawalls, embankments or smaller bags (less than 1m^3) have been used for temporary flooding protection works. Coghlan et al (2010) considered sandbags for use in seawalls as coastal protection as an emergency measure only, and it was concluded that in this context they do not provide the same resistance to erosion as properly engineered coastal structures built from more traditional materials like rock or concrete. WRL (2012) noted that such structures had an approximate design life of 4 weeks to 1 year. These estimations of design life have been made on smaller sandbags (0.75m^3) exposed to UV rays, tidal submersion and wave action and generally used in temporary works.

Recent advancements in geotextiles and customisation of container design have made the use of GSC's much more prevalent, with "standardised" design sizes of GSC for coastal protection becoming larger, 2.5m^3 containers. These containers have nominal dimensions of 2.5m x 1.8m x 0.6m (deep) with units weighing up to 5t. Material advancements mean that GSC units are not only vandalism resistant (Knives, anchors, propellers, etc.) they are also capable of withstanding UV rays and dynamic wave actions for up to 15-20 years when exposed. Should the units be buried, their design life is greatly increased; "Experience and testing of the geotextile material from bags after decades in use indicates that the UV degradation rate is not rapid and that the bags would be expected to last for more than 40 years, especially if buried initially to allow impregnation of material with sand" (Bettington, 2018).

When stacked for use as seawalls or revetments, GSC units can be considered semi-rigid structures allowing some of the incoming wave energy to be dissipated through the structure itself, also allowing an extent of settlement as the seabed or backshore changes. Semi-rigid structures are often better suited to dynamic, higher wave environments and sandy beaches rather than rigid structures, like vertical concrete walls (EurOtop, 2018). This is because failure of semi-rigid structures is not catastrophic but rather segmented.

WRL (2012) details design criteria of ($0.75m^3$) GSC units for use as seawalls on the NSW coast. These design considerations have been used in developing the three options for layered GSC BTBs as seen in Figure 45. The key difference in the presented options is the number of GSC units and the subsequent attainable height. In general, it is recommended that the BTBs augment as much additional crest height to the storm berm for the least cost and resources whilst at the same time retaining structural integrity. As such Option 1C in Figure 45 is recommended as the layered GSC BTB option for Nanumea and Nanumaga.

Option 1C consists of a layered 2 +2 + 1 design with $2.5m^3$ units. There are two layers of GSC units stacked atop of each other side by side, with a single layer overlying the join of the second layer, creating a 1V:1.5H slope. The bottom layer is “keyed-in” to the surface layer of the storm berm by approximately 500mm with a geotextile layer laying atop the exposed section. The whole structure is buried under replaced and locally sourced sand at an angle of naturel repose (30-35°) and is revegetated with native vegetation and larger (palm or coconut) palms on the horizontal extremities of the works.

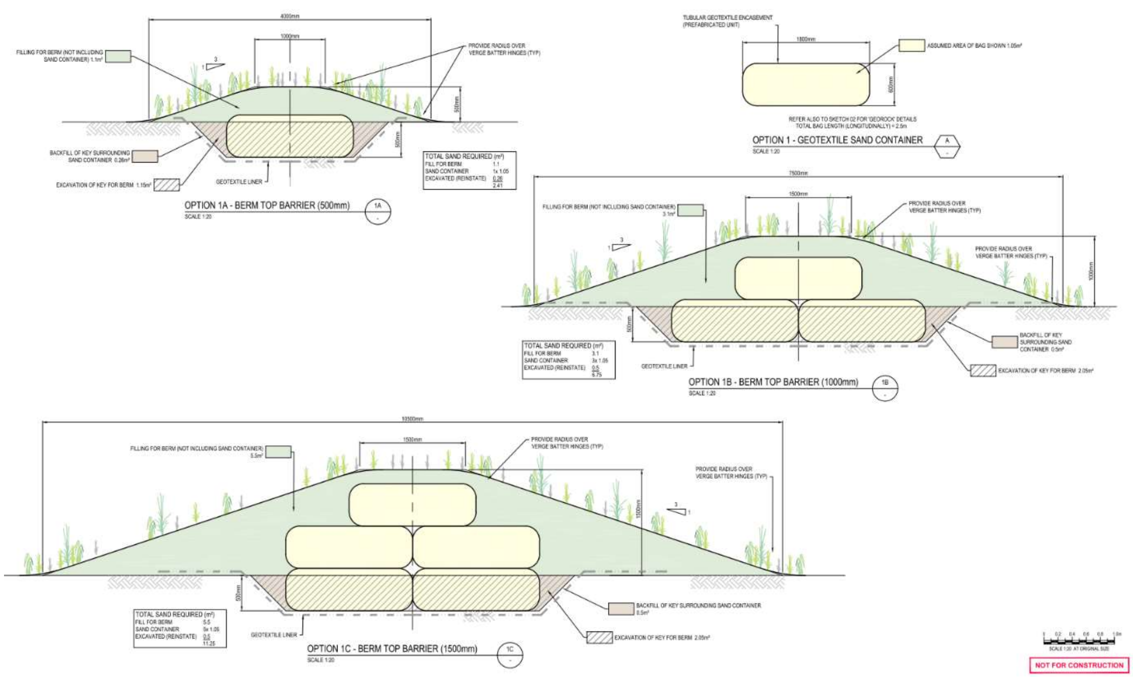


Figure 45 Layered geotextile sand container (GSC) Berm top Barrier (BTB) options. Clockwise from left: 500mm. 1000mm, 1500mm height.

<u>Construction technique:</u>

Although individual contractors may approach the construction of the BTB’s differently, the methodology provided below is anticipated to be relevant to all construction approaches.

Equipment

For the layered GSC BTB option the key equipment and plant required may be as follows:

- 2.5m^3 GSC bags (up to 4,000 across two islands)
- Geotextile fabric (approximately 2,000x5m across two islands)
- Excavator (20t—35t)
- Small excavator (10t)
- J-Bins for filling GSC bags
- Cleaned beach sand fill (approximately 14,000m^3 across two islands)
- Sediment graders and sieves
- Approximately 200m of flexible pipes and pumps for dewatering purposes and to 'wash down sand for GSC filling
- Generators
- Site facilities

Methodology

1. The supply barge or similar vessel will carry materials and plant to the island. The vessel will need to 'land' on the deep water edge of the reef flat on a suitable day (low wind/waves and small tidal range), excavators and required plant will drive across the reef flat, local tractor/trailer will be used to traffic materials to site.
2. An excavator will source sand from the approved source location on each island and transport it systematically to be stockpiled at the current BTB workface.
3. An excavator will dig the BTB footprint – approximate 20m x 10m x 500mm section along BTB alignment.
4. Geotextile fabric will be laid out in the excavation pit prior to placement of GSC units.
5. Empty GSC units will be secured to the J-Bin frame and filled using the excavator from the stockpile
6. Flexible pipes and pumps running to the ocean will be used to wash down and top up bags in the J-Bins.
7. Once filled, the GSC units will be sewn shut and sealed with silicon to ensure that the openings are secure.
8. An excavator will be used to position the GSC units in place according to the survey set out and detailed design and the site supervisor and surveyor will verify placement location.
9. Sand will be placed over the GSC units using the excavator and the initially removed topsoil replaced.
10. Native seedlings and plants will be place on top of the BTB with larger coconut and palm trees placed along the length of the seaward footing.

11. Repair (patch) kits will be given to the Public Works Department (PWD) and Kaupule to complete repairs when necessary following the construction phase should the GSC units be exposed during storms. This will be a critical aspect of extending the overall lifespan of the product and will support all manufacturing requirements.

These large hopper filled sand containers are used in revetments and groynes, due to their large size and high stability, even under the harshest of conditions.

Filling and placement of these heavy (approx. 4.5 tonne filled) and very large containers is achieved using specialized hydraulic filling apparatus and placement cradles provided by Geofabrics to ensure complete filling and a consistent finish.

Scour flap containers are available in this size.

WIDTH	2,200 mm/1,800 mm			
LENGTH	2,750 mm/2,300 mm			
FILLED HEIGHT	600 mm			
MATERIAL	Marine grade & composite marine grade			
FILLED WEIGHT	4,500 kg			
CLOSING SYSTEM	Laced close on site			
FILL FRAME AVAILABLE	Yes			
DRY SAND FILLED	No - hydraulically filled			
ORDER CODES	ER250	ER250V	ER250VF	ER250F
PALLET QUANTITY	18	12	8	13

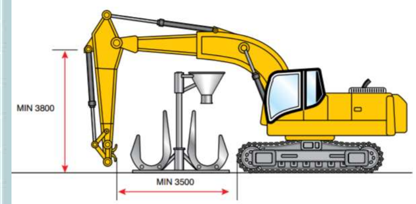


Figure 46: Details of 2.5m³ GSC units, schematic and images of GSC coastal structures during construction (source: Elcorock , 2019 and AECOM)

4.1.2 Geotextile mega-container

Option 2 consists of a single layer geotextile mega-container (GMC) placed end-to-end along the design footprint. The 20m long GMC is to be “keyed-in” to the surface layer of the storm berm by approximately 500mm with a geotextile layer laying atop the exposed section. The whole structure is buried under replaced and locally sourced sand at an angle of naturel repose (30-35°) and is revegetated with native vegetation and larger (palm or coconut) palms on the horizontal extremities of the works.

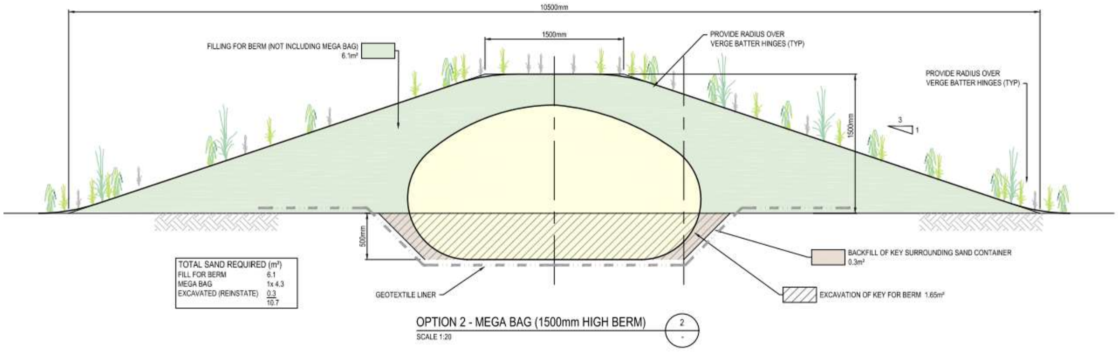


Figure 47 Layered geotextile sand container (GSC) Berm top Barrier (BTB) options. Clockwise from left: 500mm. 1000mm, 1500mm height.

Construction technique:

Although individual contractors may approach the construction of the GMC BTB’s differently, the methodology provided below is anticipated to be relevant to all construction approaches.

Equipment

For the GMC BTB option the key equipment and plant required would be as follows:

- 20m long GMC units (up to 120 units across two islands)
- Geotextile fabric (approximately 2,000m x 5m across two islands)
- Excavator
- Small excavator
- Cleaned beach sand fill (approximately 18,000m^3 across two islands)
- Sediment graders and sieves
- Drag flow pump and approximately 200m of flexible pipes
- Generators
- Site facilities

Methodology

Steps 1-4 same as Option 1C (above).

5. Empty GMC units will be placed in the excavated footprint.
6. Sediment traps or drainage channels will be constructed to ensure excess slurry liquid sieved through the GSC fabric drains to the ocean.
7. The drag flow pumps will be placed over the stockpile and the other end affixed to the GMC valve on the top of the unit. The drag flow pump will be moved through the (cleaned and sieved) stockpile by the excavator
8. The GMC will be left to drain excess water and for the unit to take its natural (filled) shape. The GMC will be sealed and drag flow pump removed.

9. Sand will be placed over the GSC units using the excavator and the initially removed topsoil replaced.

10. Native seedlings and plants will be place on top of the BTB with larger coconut and palm trees placed along the length of the seaward footing.

11. Repair (patch) kits will be given to the PWD and Kaupule to complete repairs when necessary following the construction phase should the GMC units be exposed during storms. This will be a critical aspect of extending the overall lifespan of the product and will support all manufacturing requirements.

CIRCUMFERENCE	4 m to 12 m
LENGTH	Up to 20 m
FILLED HEIGHT	Up to 2 m
MATERIAL	Marine grade & composite marine grade
FILLED WEIGHT	Up to 1,000 t
CLOSING SYSTEM	Lid
FILL FRAME AVAILABLE	No - dredge filled
DRY SAND FILLED	No - hydraulically filled
ORDER CODES	T1, T1V, T2, T2V, T3, T3V, T4, T4V

Figure 48: Details of GMC units and images of GMC coastal structures during construction (ElcoRock, 2019).

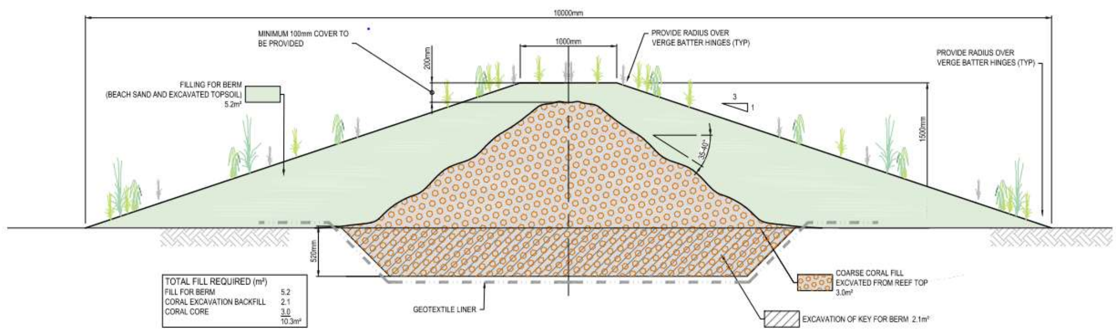
4.1.3 Crushed coral berm core

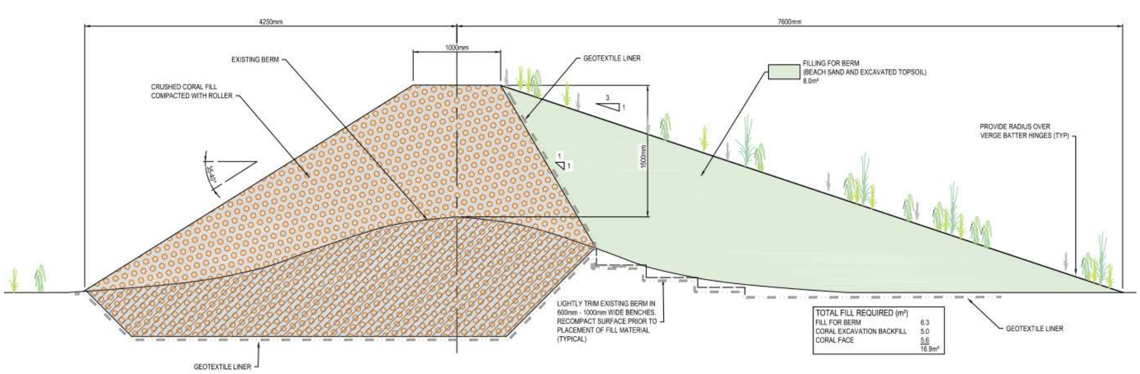
A concurrent, aid-funded project is currently underway to upgrade the boat harbour on Nanumaga. The initial design involves a significant excavation of the reef flat of approximately 15,000m^3. As the material for the proposed BTB on this island will be sourced from on-island resources, discussions are currently underway to reuse the dredged material (spoil) for use as fill for the BTB on this island. Figure 49 presents three BTB concept options that make reuse of the dredge spoil. The dredge spoil is considered to be hardened, calciferous sediment and is not suitable for fill in GSC or GMC units without a significant amount of processing (crushing and grading) which would be costly.

The three dredge-spoil BTB concepts have been designed to make use of the poorly graded, large coral, rubble and sediments that will be spoil from the boat harbour construction process. The buried dredge spoil core option is simply a dumping of the spoil material into an excavated “key” in the top of the existing berm, with a small separation of a geotextile underlayer. The dredge spoil will essentially be a gravity structure with an average angle of repose of between 30°-45°.Excavated topsoil is to be replaced atop the crushed coral core and revegetated as above.

The second option (middle image, Figure 49) is to reuse the dredge spoil as a seaward facing revetment. This would involve keying out a larger excavation of the front face of the existing natural berm to a depth of around 500mm. The spoil is left to fall into a naturally stable position with an average angle of repose of between 30°-45°. The spoil is then to be compacted with a manual roller to harden the surface. The backing (landward side) is replaced with the excavated topsoil and locally sourced clean, beach fill. Vegetation is then to be re-planted on the landward, the seaward face is to be left uncovered.

The third dredge spoil BTB concept (lower image, Figure 49) is a similar concept to the first dredge spoil BTB option. The dredge spoil is placed into an excavated “key” in the top of the existing berm, with a small separation of a geotextile underlayer. The dredge spoil is left to fall under gravity to an average angle of repose of between 30°-45°. The dredge spoil mound is to be manually compacted to create a more homogeneous shape and porosity if possible. The geotextile underlay is then wrapped around the mound of compacted dredge spoil and the overlapping geotextile layer is pinned through the geotextile layers and into the mound with 300-500mm U-pins. A 1000mm wide footpath is to be created along the top of the pinned geotextile layer running along the length of the BTB. The footpath will be retained by either wooden planks or hand-placed coral pates/large shells. The footpath will be 100-150mm thick and will cleaned, finer beach sand. Excavated topsoil is to be replaced atop the sloped faces of the BTB and revegetated and larger coconut/palm trees are to be planted at the foot of the BTB.





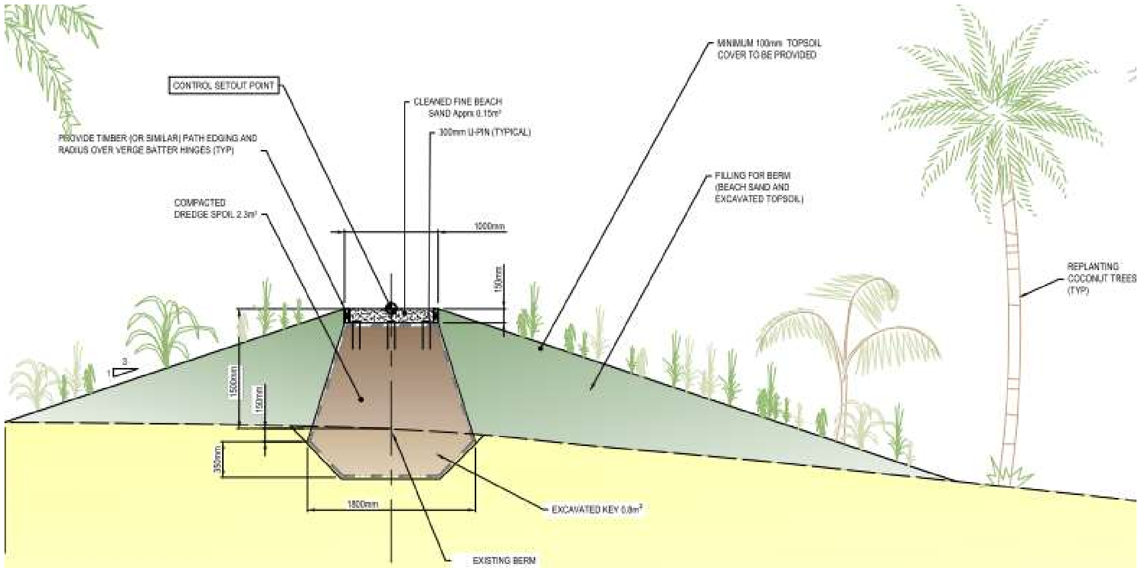


Figure 49 Crushed coral (dredge spoil) BTB core options. Top to bottom: Buried dredge spoil core, dredge soil revetment, enclosed dredge spoil core and path.

Construction technique:

Although individual contractors may approach the construction of the BTB's differently, the methodology provided below is anticipated to be relevant to all construction approaches.

Equipment

For the layered GSC BTB option the key equipment and plant required are as follows:

- Geotextile fabric (approximately 700x8m for Nanumaga) and 200-300mm U-pins
- Excavator (20t—35t)
- Small excavator (10t)
- Dredge spoil (approximately 5,000m^3 for Nanumaga)
- Sediment graders and sieves
- Manual compaction roller
- Large grading beam
- Site facilities

Methodology

1. Landing craft or similar will supply materials and plant on the island.
2. An excavator will systematically stockpile dredge spoil at current workface.
3. An excavator will excavate the workface – approximate 20m x 10m x 500mm section along BTB alignment.

4. Geotextile fabric will be laid out in the excavation pit prior to placement of dredge spoil .
5. The dredge spoil will be dumped and graded into shape atop the rolled out geofabric using the grading beam and manual roller used to compact.
6. Once design crest height of the berm is reached, the geofabric will be laid over the mound with the seaward flap rolled on top facing inland.
7. Three U-pins will be driven into the overlapping geofabric and into the compacted dredge spoil mound to secure in place.
8. Sand will be placed over the geofabric using the excavator and the initially removed topsoil replaced.
9. A 1000mm wide footpath will be cleared atop the structure and wooden planks or large flat coral pieces will be used to retain the replaced topsoil on the outer face of the berm and clean, sieved sand fill will be placed on top of the geofabric to make the path.
10. Native seedlings and plants will be place on the outer faces of the BTB with larger coconut and palm trees placed along the length of the seaward footing.

4.2 Revetment structures

Hard revetment structures, constructed of rock, concrete or steel create a rigid, hard structure which replaces the natural shoreline. TCAP (2017) noted that seawalls which were built to replace the natural shoreline would inhibit the natural process of recovery which is now occurring on Nanumaga' s western shore and that "It would be inappropriate to attempt to fix the shoreline position artificially on this coast and in all likelihood such engineering would result in more damage and greater community exposure, than simply allowing natural processes to proceed." With this in mind, there are several sections of Nanumaga and Nanumea's western shoreline where private and public assets have been built into the dynamic coastal zone, shoreward of the main road. Although in these instances, retreat has been advised as the preferred option, key stakeholders have conveyed the necessity to keep this infrastructure protected and that hard structures are the only remaining option. The following section outlines hard revetment options for these sites.

4.2.1 Rock Revetment

Rock revetments provide a rough surface with a high ratio of voids. Both of these factors work to dissipate wave energy as it travels across the revetment, resulting in reduced wave run-up levels. It is recommended for rock revetments built upon intertidal reef flats that they shall consist of a layered design as shown in the example profile in Figure 50, providing some level of permeability to the structure. An appropriately sized armour layer, a secondary layer of smaller rock, a geotextile layer and compacted fill should be placed on the reef flat (following forming of the embankment slope). The toe of the structure is to be embedded on the hard substrate of the reef top by excavating. The crest height and slope of the structure will be determined following the determination of design wave limits, and acceptable overtopping limits. Should a structure like this be selected for the active coastal zone, it is recommended its effect on coastal processes be minimised. Reducing the verticality of the wall and burying the majority of the structure following construction are methods that may reduce impacts on sediment supply around and in the vicinity of the structure due to scour and wave reflections induced by the structure.

There are no quarries on any of the nine islands of Tuvalu, nor access to rocks of sufficient size for a wave-exposed revetment. All materials for the construction of such a structure would need to be imported, the closest (viable) sources of rock would be Fiji, Samoa or Nauru (although units would need to be much larger due to lower density of rock). On-island transportation and placement of large armour rock during construction will require large construction equipment.

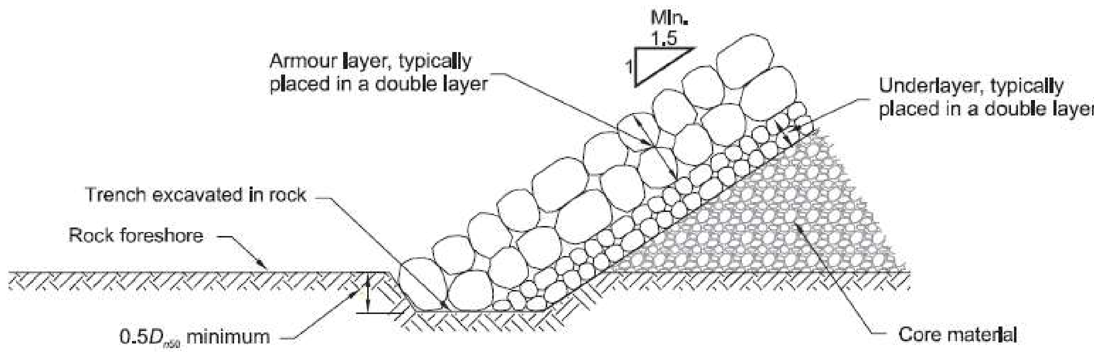


Figure 50 Typical rock revetment profile and toe designs based upon Impermeable layer located near surface level. Trench excavated into impermeable (bedrock) layer. (CIRIA, 2007).

4.2.2 Concrete armour units

Concrete armour units are an option for the replacement of armour rock in revetment design when not available. These units can be cast on-site and made to a mass large enough to be stable. If utilised in Tuvalu, concrete would need to be imported in order to meet strength requirements. Placement of these units on the revetment require large construction equipment that would be expensive and difficult to mobilise to the outer islands of Nanumaga and Nanumea.

Concrete armour units have been previously used in the Pacific but are generally limited to islands with ports or areas of high value, examples of concrete armour units can be seen in Figure 51. Other considerations on the use of concrete in Pacific Islands is provided in Shand et. al (2017). The transportation of large quantities of rock (as well as large individual sizing of armour units) makes the use of rock or concrete armour units on Nanumea and Nanumaga particularly difficult. The lack of suitable port facilities on these islands as well as the logistical problems involved in transportation of this material across the reef flat (on a suitable sized landing craft) makes the use of these materials prohibitive.

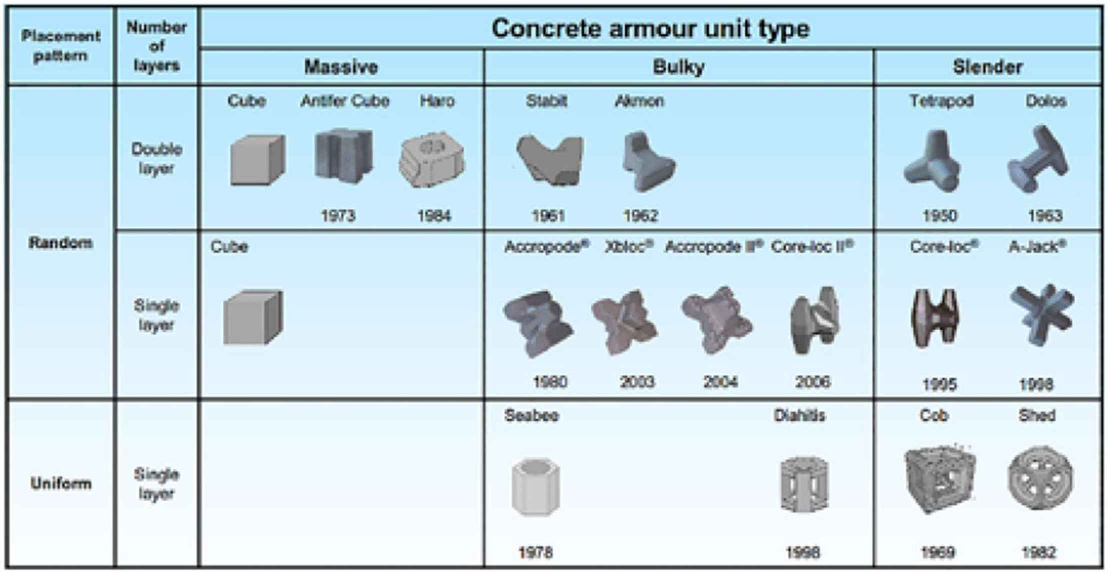


Figure 51: Examples of concrete armour units (source: CIRIA, 2007).

4.2.3 Seabee wall

Due to the lack of availability of suitable armour rock on the islands and relatively small wave heights on the upper shoreline, a pattern placed concrete amour unit structure is considered an appropriate alternative. A suitable armour type would be the Seabee unit, which is a hexagonal concrete block with a hollow core. The amount of wave run-up on a Seabee wall is influenced by the surface roughness of the wall face, the amount of water which penetrates the underlayer and the turbulence generated by the release of trapped air within the voids of each Seabee unit. To maximise energy absorption, a mixture of block heights is proposed to increase surface roughness and decrease wave runup.

A Seabee wall has been proposed for the western shore of Nanumea. A typical cross section through a Seabee wall is shown in Figure 52. The crest height of the seawall will be designed to limit over-topping and the extent of wave run-up during storm events. Further details on the Seabee wall design is provided in the Nanumea Concept Design Report (Bluecoast,2021).

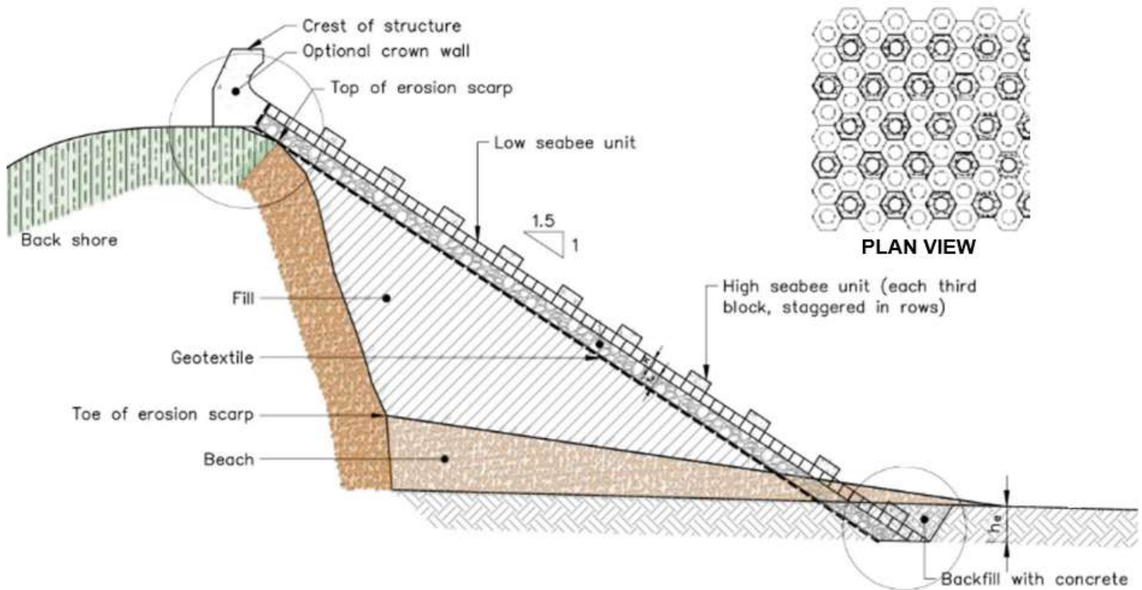


Figure 52: Typical cross-section of a Seabee revetment and plan view of seawall face pattern (inset).

4.2.4 Sheetpiles

Sheetpiles are large, plate steel (plastic or FRC) structures that are generally used in maritime construction environments for quay walls in port design. They are long modular steel units that interlock with one another and are driven into a hard substrate to retain land or water. The use of sheetpiles as seawalls in the active beach zone creates a large impenetrable barrier with a reflective frontage to incoming wave energy. Dean (1986) found that armouring of the beach with a hard, rigid and reflective structure will contribute to frontal effects such as toe scour and depth increases as well as end-of-wall effects, a schematic of beach effects after Dean (1986) can be seen in Figure 53.

Installation of sheetpiles in either Nanumea or Nanumaga would also be very difficult due to the large, specialised pile-driving equipment required for their installation, a schematic of the installation process can be seen in Figure 53. The active corrosion and decay of steel structures in an exposed oceanic environment may also pose a safety hazard to the public when accessing the shoreline past these structures. A design investigation was undertaken for the use of sheetpiles along the central western coastline of Nanumaga (FNC & Bluecoast, 2020).

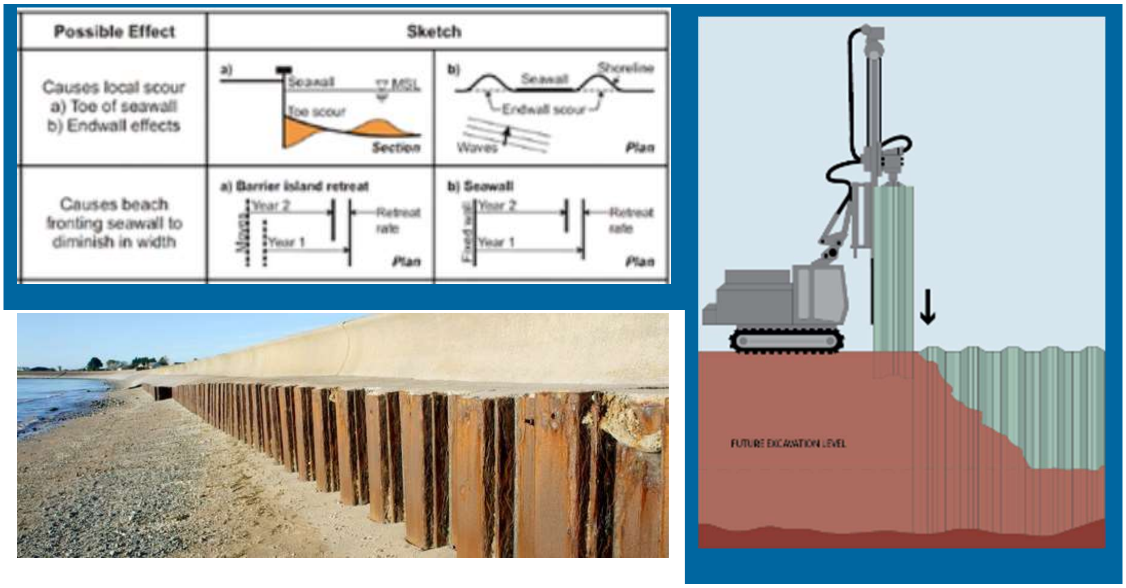


Figure 53: Possible impacts from vertical sheetpile seawall built on sandy beaches (top left, after Dean (1986)). Schematic of sheet pile installation (top right) and example of sheetpile seawall on a sandy beach (source: Google images).

It is strongly recommended that sheetpiling is not used for the hard revetment structures on Nanumea or Nanumaga. Placed within the active zone of a reef-mediated shore, it is expected that the nature of storm events on these shores will cause damage to the sheetpile structures and disrupt the coastal processes. Figure 54 provides an example storm response to FRC sheetpiles placed within the active zone of a reef mediated shore.

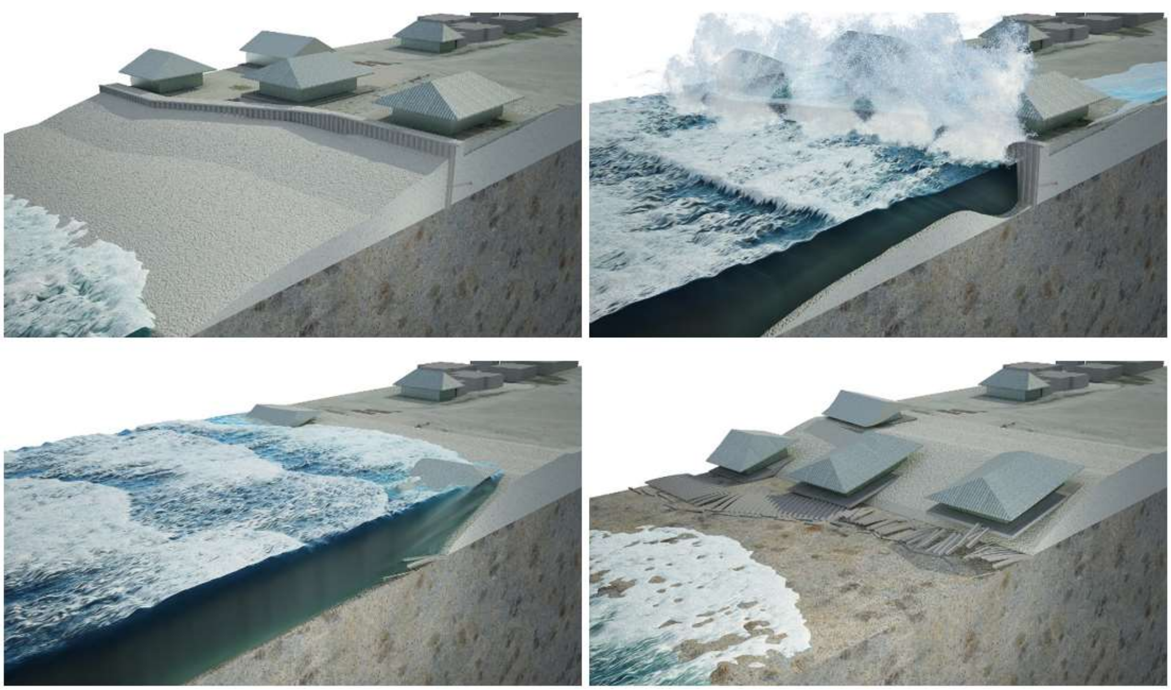


Figure 54: Schematic of expected sheetpile response to storm waves and inundation

4.3 Reclamation bund

The project Feasibility Study (GCF, 2015) built on the previous work undertaken by JICA (2011) and highlighted the need for large-scale reclamation as the most suitable long-term adaptation measure for Funafuti. The original (2011) project aimed to rehabilitate 1,000m of the eroded and degraded foreshore on the lagoon-side of Vaiaku waterfront. The selection of the site was made on the basis that due to the concentration of economic, social, political and institutional assets on Funafuti, should a GCF loan be secured that protecting these resources would provide the greatest benefit to the people of Tuvalu.

JICA (2011) provided a detailed feasibility study for large scale beach re-nourishment which included the current project site as well as the recently completed QEP site. The reclamation footprint was to be filled with gravel mined from the southern islets of Funamanu, Falefatu and Mateika. The Feasibility Study included descriptions and possible impacts to coastal processes, as part of an Environmental and Social Impact Assessment (ESIA), as well as a financial and economic analysis. Preliminary engineering concept designs were developed for the study site, as seen in Figure 55.

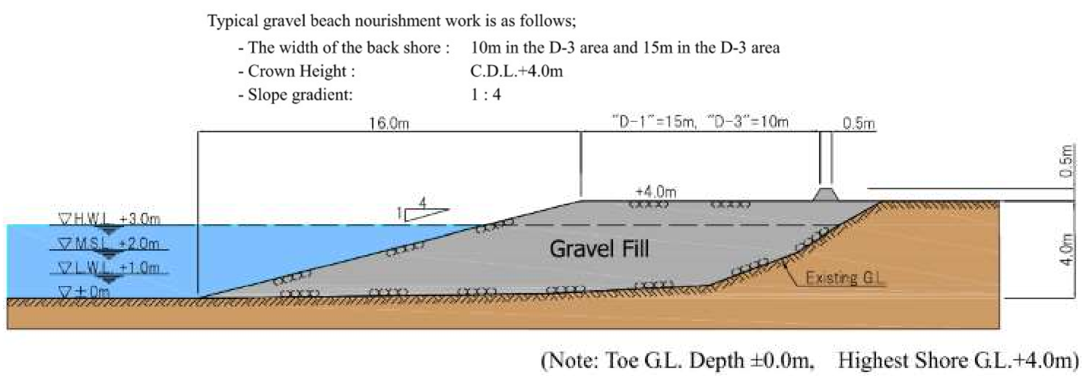


Figure 55: Typical section of proposed gravel beach nourishment work to run from Vaiaku Wharf to Catalina Boat Ramp as part of the study for assessment of ecosystem, coastal erosion and protection / rehabilitation. (source: JICA, 2011).

In preparation for the Pacific Leaders Forum in 2019, the Government of Tuvalu fast-tracked a design to provide nourishment to the western section of the JICA (2011) reclamation area; an approximate 300m length of foreshore (to 90m offshore) running from the Vaiaku Wharf towards the east, as seen in Figure 42. The reclamation was opportunistically undertaken utilising a large Cutter-Suction Hopper Dredge (*The Amity)* which had been undertaking nearby renourishment works of the WWII Borrow Pits on the eastern arm of Fongafale Islet. The QEP reclamation made use of an identified large reserve of usable lagoon sediments (up to 24Mm^3) located offshore of the project site, Figure 57. Following the QEP reclamation, there was rapid loss to the beach face due to coastal processes. A subsequent rock revetment was implemented to "bund" the newly reclaimed land, as seen in the GoT Public Works Department (PWD) designs in Figure 56.

The following sub-sections provide a brief overview of different bund types that may be used for the proposed TCAP reclamation on Funafuti as well as their advantages and disadvantages. Selection of an appropriate bund design will be made based upon the following key criteria:

- Cost; material, construction, whole-of-life

- Design life
- Design wave height, water level and coastal processes
- Drainage requirements
- Public access and amenity
- Future land ownership and land use requirements

Figure 56: Top: Concept design and profile height of rock revetment bund for the Queen Elisabeth II Park Reclamation works (PWD, 2019). Bottom: QEII reclamation prior (left) and during rock revetment construction (right)

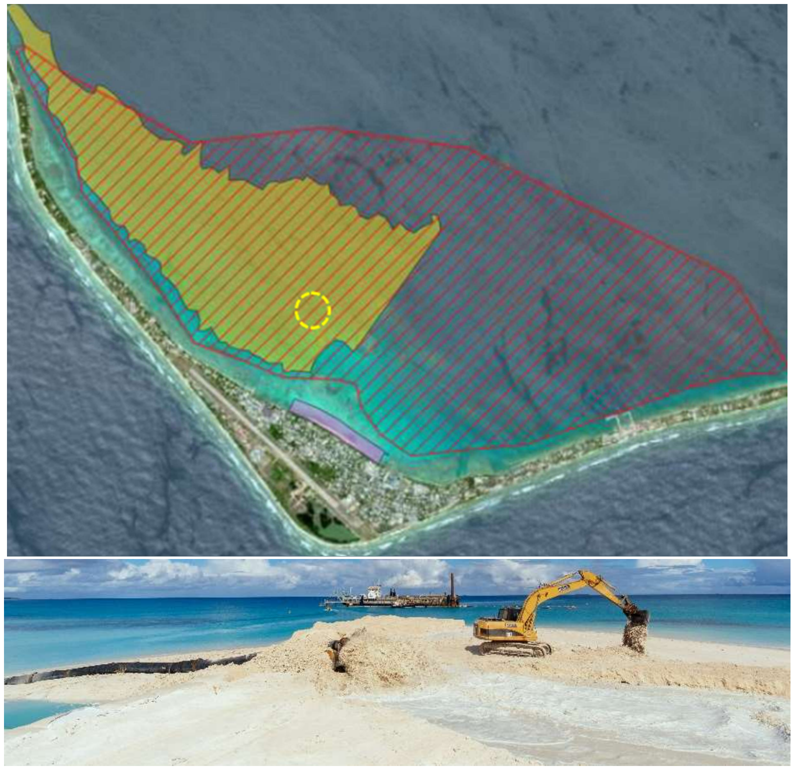


Figure 57: Funafuti sediment resource area. The olive-coloured zone is the established resource area containing an estimated 24,000,000 m^3 of material which can be safely utilised (Smith, 1995). The larger red cross hatched zone is the area identified by Kaly and Peacock-Taylor (2014) as the "the zone of dead coral and eutrophication". The yellow circle approximates the area which would be dredged to a depth of 3.5m to supply 350,000m3. The quality of the dredged sand from this location can be seen in the bottom image taken from the QEII Reclamation (Source: UNDP, 2019 and Hall, 2018).

4.3.1 Rock revetment bund

An appropriate rock revetment bund design would be similar to that described in the Hard Revetment Structures section (above), with a profile like that provided in Figure 50. The location for the revetment bund is planned to be approximately 80-110m offshore of the current shoreline in a relative depth of between -1.2 to -2.7m MSL, see Figure 58. This offshore location is relatively void of sediment (prior to reclamation) making toe scour less of an issue at this depth, this means that the rock revetment could be steeper than that recommended for a revetment located on a sandy shore (1V:3H or less). A steeper revetment face would result in a lower volume of rock required.

As noted in the previous section, there are no suitable sources of rock large enough in Tuvalu which would afford protection from the wave heights expected to be generated within the lagoon (Hs~2m, see Sec 3.2.3). As such, all rock would need to be imported and would add significant cost to the project. Concrete armour units would incur the same significant costs as imported rock and due to the relatively low wave environment within the lagoon, are not considered necessary.

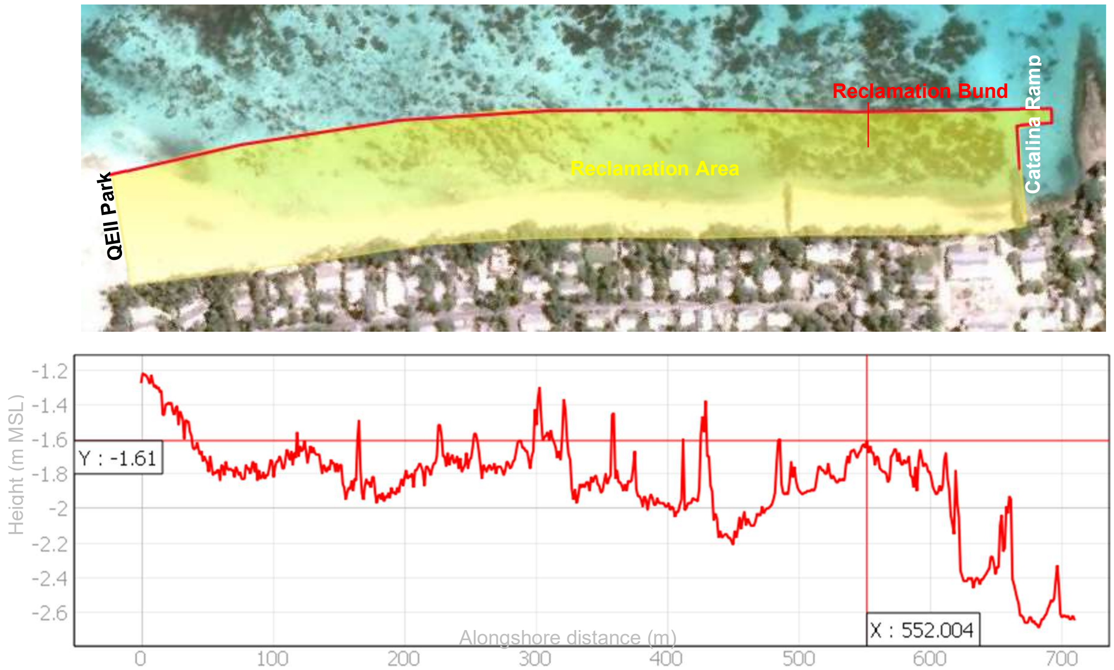


Figure 58: Approximate Funafuti reclamation footprint and bund wall location (red line). Lower image shows approximate bed levels at this location with respect to MSL (2019).

4.3.2 Sheetpile bund

A (steel, plastic or FRC) sheetpile bund for the Funafuti reclamation could be installed similarly to that described in the previous section. It is envisaged that the dredging and reclamation work would be completed all the way to the footprint of the bund shown in Figure 58. The sheetpiles would then be driven into the desired location and any sediment seaward of the sheetpiles would be reclaimed using a long arm excavator and placed within the reclamation area. It is highly recommended that the sheetpiles have some sort of concrete capping running along their top edge. This capping beam would inhibit corrosion and protect the sheetpiles from any damage that may be caused by vessels (inevitably) berthing against the structure.

As the sheetpiles would be in the water for most of their design life, there is a serious risk of Advance Low Water Corrosion (ALWC) on steel sheetpiles which forms just below the waterline of unprotected steel structures in marine environments. The cost for regular application of coatings like silane as well as installation of anodes would also need to be factored into any maintenance plan for the structure.

4.3.3 Geotextile synthetic container bund

A GSC bund could be constructed to retain the reclamation as a cheaper alternative to hard structures such as sheetpiles, concrete units or rock. Several geotextile revetments have been constructed in Tuvalu for coastal protection works in recent years. A single layered seawall was built in Nukufetau in 2017 (Figure 59) as well as a buried GSC berm to repair a breach through the eastern arm of Fongafale atoll as part of the WWII Borrow Pit Remediation Project, Figure 59.

Bettington et al. (2018) has shown that properly maintained GSC structures for coastal protection can have a design life of up to 40 years. Coghlan et al (2009) and Dassanayake et al (2012) have also undertaken detailed modelling and developed empirical formulae relating to the stability and damage levels of GSC coastal structures exposed to varying water levels and wave heights.

Figure 60 shows two possible GSC bund configuration options that may be incorporated in the revetment design; a simple single layer GSC revetment wall using 2.5m^3 units and a revetment utilising a larger GMC core and GSC outer. The benefits of the GMC core come from decreasing construction times as a longer bund length (20m) can be laid in one operation rather than the placement of individual units. The GMC will still require GSC units on its seaward face to protect the unit from piercing or tear due to public access, vessel berthing, anchors, large marine debris or vessels that have come adrift from their mooring during storm events (as recently witnessed during the TC Tino event). A tear or split of a GMC would result in a catastrophic structure failure, whereas failure of an individual GSC unit would require a simple replacement or repair.

Figure 59: Example GSC revetment at Nukufetau (left) and repaired berm using GSC units at the WWII Borrow Pit site on Fongafale (Source: Hall Contracting)

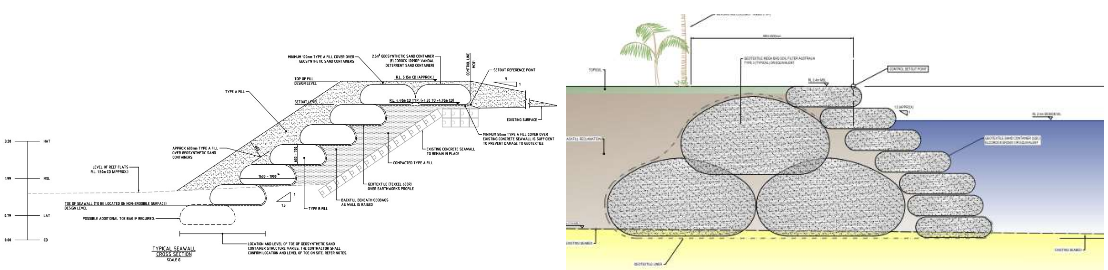


Figure 60 GSC bund options, left: Nukufetau GSC revetment design (Hall, 2015), right: GMC revetment concept

4.4 Design considerations

The following design considerations (or constraints) are to be considered in the development of final concept designs for the coastal protection structures across the three project sites. In addition to these design considerations, a preliminary Basis of Design (BoD) is provided in the subsequent section which will determine the physical parameters of the coastal protection concepts. The BoD will be updated prior to the detailed design phase and as more detailed investigations are undertaken such as the ESIA, metocean monitoring and surveys.

4.4.1 Design Life

Design life is defined as the period for which a structure or a structural element remains fit for use for its intended purpose with appropriate maintenance. The design life will be confirmed during the detailed design and will be cognisant of projected climate change impacts.

The Australian Standard Guidelines for the design of maritime structures (AS 4997-2005) specifically excludes the design of “coastal engineering structures such as rock armoured walls, groynes, etc.” However, a suggested 50-year planning period to estimate total maintenance and construction costs in considering the different spans of design life and frequency of maintenance has been adopted.

4.4.2 Design event

Conventional coastal engineering practice in Australia is to allocate a design Average Recurrence Interval (ARI) storm event ranging from the design life of the project (50-years) up to that suggested in AS 4997-2005. A 100-year ARI value will also be adopted for the stability of any structures examined as part of this concept design development. The recurrence of the return event for overtopping design of the BTB structures is yet to be defined but could, for example, be a one-year average recurrence interval based on safe average overtopping volumes for pedestrian access behind the structure crest.

4.4.3 Physical characteristics

The alignment of the reclamation bund on Funafuti has been defined by the original Feasibility Study (JICA, 2011) and agreed upon in the cabinet paper and GCF funding application. It is understood that there are long-term governmental strategies that will involve significant reclamation into the lagoon offshore of the current project site, however no funding mechanisms have been put in place at the time of writing. It should be considered that in the medium to long term, the footprint of the reclamation area has the potential to be increased.

The length of the BTB protection in both Nanumea and Nanumaga has been agreed upon in the cabinet paper and GCF funding application and was informed from meetings with the islands Kaupule and the Government of Tuvalu to protect prioritised private and public asset areas. The concept of the BTB alignment was to raise the height of the storm berm, so in the first instance the concept footprint will be aligned to follow the ridge or the highest elevation of the storm berm. The next iteration will be to ensure that the design footprint is at a minimum of 10m from infrastructure and roads. If realignment is required, wherever possible this will be done by shifting the footprint inland to increase the distance from the active shore.

The crest height and other physical parameters of the coastal protection structures in Funafuti and the outer islands will be informed in the first instance by the BoD, with iterations being defined by financial constraints and material availability.

The crown height of the seawall on the QEP reclamation is between +2.3-2.8m MSL with the reclamation itself sitting at +2m MSL which equates to approximately 0.6m above HAT (Figure 56). The height of the bund wall and reclamation will be finalised during the Detailed Design phase; however, the design should include considerations for drainage from the reclamation site as well as from the Vaiaku settlement where the current vegetation line sits at approximately +1.2m MSL. Drainage between the proposed reclamation and the QEP reclamation site should also be considered in any drainage designs.

4.4.4 Social

A comprehensive investigation of social considerations and impacts of TCAP on the people of Tuvalu is concurrently being undertaken in the ESIA, the outcomes of which will be used to inform the design process, prior to detailed design and construction. The following social aspects have been considered in the development of the concept deigns:

- The height of the crest of the reclamation will be such that it will now be the highest available land on the island and will be the greatest distance from the ocean side (and subsequent storm waves and inundation).
- As this is newly created land, land ownership will be vested in the Government of Tuvalu and recommendations for land usage will be forwarded in the ESIA to relocate essential public buildings and infrastructure to this location, these include cyclone shelters and disaster management facilities.
- The reclamation will incorporate a small boat harbor at the northern end following feedback from the Funafuti Community to ensure boat access is maintained next to the Catalina Ramp.
- The concept and design philosophy behind the BTBs on the outer islands is to provide a "working-with-nature" approach to coastal protection and to educate and reaffirm with the Kaupule and local communities of these islands the coastal and island-building processes of which the project wishes to supplement. It is hoped that future development or coastal protection works undertaken on the islands could be performed locally and within the ethos and design philosophy of the project.
- Where hard structures have been provided on these islands, it has been deemed that these are a last line of defence for essential public infrastructure and for buildings vital to those communities (churches, maneaba, etc.). In addition, communications with the project team, the community and Kaupule have worked to reiterate the possible consequences of building hard (and ad-hoc) coastal structures within the active coastal zone.

4.4.5 Geotechnical

SPC conducted preliminary geotechnical investigations on Nanumea and Nanumaga to inform the proposed coastal protection works and to also enable appropriate consideration of geotechnical conditions during the ESIA stage of the development process. The information gathered will also be used to inform a subsequent Coastal Vulnerability Assessment (CVA) being undertaken by SPC.

The preliminary geotechnical investigations on the outer islands consisted of the following:

- A desktop study of available information to establish the extent of existing data.
- A geotechnical walkover of the project site.
- Participation in consultations with relevant stakeholders.
- Magnetometer clearance of investigation locations prior to intrusive testing.
- Scala penetrometer tests.
- Hand augers with sample collection in unconsolidated sediments.
- Test pit excavations and sample collection in unconsolidated sediments.

- Boreholes with a portable core drill, and sample collection in rock.
- Particle size distribution tests on selected samples.
- Composition analysis of selected samples.
- Unmanned Aerial Vehicle (UAV) imagery collection of selected areas.

Physical sampling sites were selected based on the concept design alignment. The collected data was analysed, and a preliminary geotechnical investigation report was produced for each site, which have been summarised below.

Nanumaga

The findings of SPC's preliminary geotechnical investigations demonstrate the dynamic nature of the project site and show an ongoing pattern of vertical accretion along the western ridge of the island. The most recent evidence for this vertical accretion is the Cyclone Pam deposits which were encountered in eight of the nine hand augers excavated on the western ridge (the location of the proposed BTB). Underlying the newer Cyclone Pam deposits was generally a layer of organic paleosols ranging between 0.9 – 2m in thickness. Underlaying these organic paleosols are generally layers of biogenic calcareous sediments (foraminifera, coral fragments, bivalves, coralline algae and gastropods) to a depth of between 0.5 to -1m MSL considered to be the limestone reef platform consisting of calcareous sediments (including coral, foraminifera, bivalves and coralline algae) cemented together by calcium carbonate cement. A representative sample taken along the BTB alignment on Nanumaga can be seen in Figure 61.

Figure 61: Example of buried paleosol layers from Nanumaga, Sample NMG 8. (Source: SPC, 2020a)

Nanumea

A similar geotechnical investigation was undertaken on Nanumea. The survey identified five major geomorphological landform units on Nanumea islet; the eastern ridge complex, western ridge complex, central depression-lagoon complex, enclosed basin, and lagoon margin (see Figure 33).

The proposed TCAP coastal protection is located on the western ridge complex unit. The western ridge complex unit is mapped as having an area of 61.18 ha, which accounts for 28% of the total Nanumea islet area. In the location of the proposed TCAP coastal protection on the western ridge complex is characterised as a single, broad asymmetric ridge, with a relatively steep seaward facing slope rising from the reef flat, and more gradual inland facing slope which typically steepens in narrow areas of the islet. The crest of the western ridge complex is typically higher than the eastern ridge complex and reaches elevations approximately 4m to 5m above the reef flat level.

Forty soil pits were logged (to a maximum depth of 1.0m) and thirty-eight soil samples were tested as part of the survey. The results of the soil pits and testing were used to generate a soil map for Nanumea (see Figure 33). A total of seven different soil types were identified on Nanumea islet; light sandy, light gravelly, light gravelly sand, dark sandy, dark gravelly sand, gravelly phosphatic, and old airfield soils. The proposed TCAP coastal protection is located in an area mapped as light sandy soils. The light sandy soils are described as foraminifera rich sand with occasional coral fragments. The light sandy soils are distinguished from the beach deposits due to the presence of organic content (roots, leaves and branches) in various stages of decomposition which have stained the calcareous sand.

Funafuti

Two separate geotechnical investigations have been undertaken by SPC (Smith, 1995 & 2016) to identify suitable sand resources for reclamation. An area offshore of the project site has been identified with an estimated volume of up to 24Mm^3 of usable sediment and has been designated as the sand resource area (SRA). The yellow polygon in Figure 57 shows the location of the SRA and its proximity to the reclamation area. The final location within the boundary of the designated SRA will be determined by the successful contractor and is expected to be driven by the operational limits of the dredging vessel, pumping/haulage distances and quality of the sediments within the selected area. It should be noted that with the high possibility of future reclamation directly offshore of the project site, it is the preference of the GoT that sediment is sourced from deeper areas or those further afield from the project site within the designated SRA to reduce future infilling.

Smith (1995), Kaly and Peacock (2014), as well as recent dredging and sampling activities within the lagoon noted that there is a degree of variability in the quality of sediments across the SRA. Table 13 shows four samples taken on the south west corner of the SRA. The Required sediment characteristics for both the reclamation fill and the GSC and GMC units (as stated in the IFC drawings) have been split into two main categories based on the particle size distribution (PSD) of the sediment, these are provided in Table 14.

Recent dredging activities have revealed inconsistences in the penetration depths during original investigations stated in the SPC reports and that the presence of coral *bombies* are prevalent throughout the SRA.

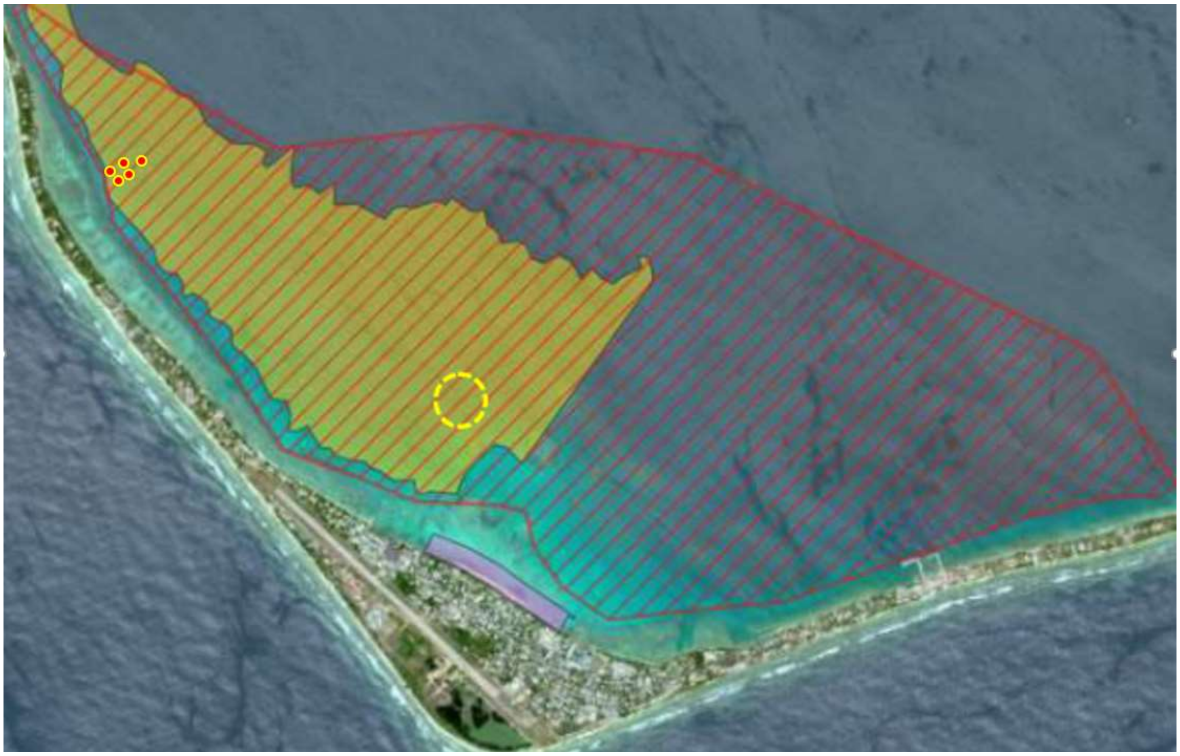


Figure 62: The olive-coloured zone is the established sediment resource area containing an estimated 24,000,000 m^3 of material which can be safely utilised (Smith, 1995). The larger red cross hatched zone is the area identified by Kaly and Peacock-Taylor (2014) as the "the zone of dead coral and eutrophication". The yellow circle approximates the area which would be dredged to a depth of 3.5m to supply 350,000m^3. The red/yellow dots are the (JP#) sample points detailed in Table 13 (Source: UNDP, 2019 and Hall, 2018).

Table 13: Sediment sampling undertaken within the resource area. (Source: Smith, 2015)

JP#	Easting m	Northing m	% Clay	% Silt (0.02-0.006 mm)	% Fine sand (0.06-0.2 mm)	%Medium Sand (0.2-0.6 mm)	% Coarse sand (0.6-2.0 mm	% Fine Gravel (2-6 mm)	%Coarse Gravel
12	739871.6	9055833	0	1.2	6	3.4	43.1	21	0
13	739843.3	9055793	0	1.2	28.7	37.1	29.2	3.4	0
14	739868	9055898	0	0.7	9.3	27.1	34.1	26.4	0
15	739913.7	9055866	0	2.2	13.7	26.4	31.4	21.8	0
16	739939.7	9055835	0	0.7	5	26.1	47.1	19.7	0

Table 14: Sediment size requirements for Funafuti reclamation.

UXO

In 2015 Golden West Humanitarian Foundation under grant from United States Department of State conducted a joint mission with the Tuvalu Police Service to locate, recover and dispose of Explosive Remnants of War (ERW) throughout Tuvalu. The team located and destroyed two-thousand-five-hundred Smalls Arms Ammunition and one 155mm High Explosive Projectile on Nanumea. The team also identified one 500-pound General Purpose Bomb in the Nanumea Lagoon, this ordnance was not destroyed and the survey team recommended all persons avoid the immediate area until such time the ordnance is removed or destroyed[2]. The 500-pound General Purpose Bomb is in the lagoon and therefore not within the vicinity of the TCAP project site, it is not known how many (if any) of the destroyed ERW were collected from within the TCAP project site. The presence of ERW on Nanumea is a potential risk for the TCAP project which we recommend is considered and managed accordingly.

As a standard SPC safety precaution prior to conducting intrusive testing, each investigation location was screened with a JW Fishers Pulse 8X metal detector which has a 6ft maximum detection range. The team was particularly concerned about the risk or ERW on Nanumea given the WWII history and previous identification of ERW on the island.

The team did not encounter any magnetic anomalies while screening at the investigation sites other than small pieces of rusted metal inferred to be eroded from the USS-LST 203 shipwreck on the reef flat (which grounded on the reef flat on 2nd October 1943) and other minor metal garbage. However, the team did observe four empty artillery shells on the beach adjacent to the TCAP project site (see SPC, 2020b for further detail). A detailed magnetometer survey to identify ERW was outside the scope of the preliminary geotechnical assessment, the SPC survey team purely screened sites prior to intrusive testing as a safety precaution.

4.4.6 Government approvals

The Ministry of Natural Resources (MNR) advise that the Tuvalu Government has jurisdiction over the 12-mile zone from the shoreline out to sea which also includes the lagoon area where dredging would occur. MNR will require an application to be made under the Tuvalu Environmental Protection Act (TEPA) 2008 for the dredging. An extensive ESIA is concurrently being undertaken by SPC and will supplement this application. The ESIA process also involves extensive communication with the Funafuti community and Kaupule. The ESIA will also consider the BTBs and hard structures for the outer islands and includes further community and Kaupule consultation at these locations.

4.4.7 Availability of material (sediments)

Following the recommendations made in TCAP (2018) and TCAP (2019), the Green Climate Fund (GCF) sent a Request for Information (RFI) in response to changes to the original concept designs submitted for the application of GCF funding for the TCAP project. In response, a technical memorandum entitled*, Provision of additional guidance to UNDP regarding the information and documentation required by the GCF (*TCAP, 2019) was developed and submitted by Dr Arthur Webb and the TCAP team. The memorandum first introduced the BTB designs and described possible sediment supply sources on each of the islands. The following subsections summarise this investigation and describes any updates or constraints to sediment supply.

<u>Funafuti</u>

In terms of suitability for reclamation, the sediment samples collected within the Funafuti Lagoon SRA (Table 13) were seen to have the following properties:

- absence of clay

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[2] The source of this information is from a letter addressed to the Tuvalu Police Service from the Golden West Humanitarian Foundation; a copy of this letter was obtained by SPC from the Tuvalu Police Service.

- predominance courser grained material (from 29 – 47% in these samples)
- very low percentages of silt size fractions (from 0.7 – 2.2 %).

Courser grained fill is seen to be a better-quality option for reclamation projects in environments exposed to wave/wind action as a reclamation material as its larger size/density is harder to mobilise, TCAP (2018) also noted that larger particles have greater potential for the establishment of vegetation and water infiltration characteristics.

The presence of unexploded ordinance (UXO) have been found within the lagoon sediments as during WWII there were up to 174 vessels moored within the lagoon, with Funafuti being bombed on 9 separate occasions (JICA, 2011). Prior to the dredging work undertaken as part of the Borrow Pit and QEII Park reclamations, CSG Demining Consultants (CSG) conducted a Risk Assessment posed by Explosive Remnants of War (ERW). The key findings of the ERW Risk Assessment were that because there is minimal excavation required for the preparation and filling of the borrow pits, there is minimal potential for encountering ERW. The report found potential to encounter ERW during dredging operations and recommended the following:

- A suitable ERW Awareness Brief be provided to all project staff and visitors.
- Ground-intrusive activities on land be avoided, or if essential, minimised. If significant ground-intrusive activities must be conducted, an ERW clearance of the location is to be conducted.
- The dredge system include a screen on the dredge inlet, a bomb box with hydraulic door on the inlet pipeline and another bomb box on the outlet side of the dredge pipeline and deploy a UXO Technician for, at least, the first month of the dredging operation.

<u>Nanumaga</u>

TCAP (2017) noted a substantial sediment deposition on the north eastern tip of Nanumaga following the passage of TC Pam. TCAP (2019) further explored the possible use of these sediments for the requirements of TCAP on Nanumaga, the following observations and recommendations were made:

- The TC Pam deposit was conservatively estimated at approximately 120,000m^3 of sediment (November 2017). These features are a common occurrence following storms where large volumes of material are rapidly moved and redistributed.
- Nanumaga is in a state of its development that its entire accommodation space has been filled on the reef platform (and its interior, i.e., no lagoon).
- The north eastern end of Nanumaga (as noted above) is deemed as the "terminal point" of the island. Deposits of sediments in these locations are usually not permanent features, they remain unstable and ephemeral and will rapidly reshape in response to ongoing conditions.
- Sediment deposited at this location will eventually be lost over the deep water drop off, meaning it is permanently lost from the island shoreline system.
- The transient deposits on this termination point represent the only sustainable and safe source of aggregate material on island.

- Only about 2% of the estimated volume deposited by TC Pam would be required for the BTB construction.
- Removal of this material is not expected to have any negative impacts on the island (which is essentially being returned to the islands berm system from where it was originally displaced).

The recent passage of TC Tino (February 2020) further reworked sand deposits on Nanumaga and the results of an updated survey will inform the status of sediment sources on the island.

A concurrent World Bank (WB) funded project to upgrade the boat harbour on Nanumaga is underway with construction works preliminary scheduled to possibly overlap with the BTB construction. Discussions are underway to look at project efficiencies in mobilising plant , labour as well as reuse of materials. The boat harbour design involves a significant excavation of reef top material required for the swing basin design with early indications of approximately 15,000m^3 of material to be dredged. Preliminary estimates of sediments required on Nanumaga are in the order of 30% of this material.

Figure 63: left: north eastern tip of Nanumaga showing an approximate120,000m^3 of sediment deposited following TC Pam. right: seaward edge of the deposit looking south 90m from the established vegetation line (Source: TCAP, 2017).

<u>Nanumea</u>

Similar to Nanumaga, TC Pam transported sand both over the natural foreshore berm and deposited a huge volume off the southern terminal point of the island. TCAP(2017) and TCAP (2019) noted the following:

- Cyclone deposit was estimated to contain some 80,000m^3 of material in November 2017, Figure 64.
- The southern tip of Nanumea and the inner lagoon are natural point of departure for sediments as they leave the island's shoreline system and are lost to deeper waters beyond the reef edge.
- The storm deposits at this location are an ever shifting and variable feature.

The recent passage of TC Tino (February 2020) further reworked sand deposits on Nanumea and the results of an updated survey will inform the status of sediment sources on the island.

Figure 64: TC Pam sediment deposits on the shifting southern tip of Nanumea (Source: TCAP, 2019)

An analysis of the sediment budget and coastal processes on Nanumea was undertaken as part of Bluecoast (2021) using the 2019 LiDAR and shoreline datasets. In 2019, the total estimated volume of sediment on Nanumea island above 0m MSL in 2019 was 4.8 million cubic meters (m^3). Based on the limited geotechnical investigation data available as well as our understanding of similar environments, this volume is assumed to be comprised mostly of unconsolidated sand and gravel.

Table 15 provides the estimated volume change around various segments of Nanumea's shoreline. Interesting behaviour is observed between the western and eastern shoreline. During cyclone periods the western shoreline appears to supply sand to the eastern shoreline with this pattern reversed in periods without major cyclones (i.e., recovery periods).

Table 15: Change in island sediment volume for observed periods around Nanumea's shorelines.

Area	Total volume change (m^3): -ive is erosion, +ive is accretion			
	Long term trend (1971 to 2014)	TC Pam (2014 to 2016)	Post-Pam recovery (2016 to 2019)	TC Tino (2014 to 2016)
TCAP project area	-17,400	-68,000	9,500	-18,000
Entire western shoreline	-97,000	-236,000	21,600	-52,000
Eastern shoreline (initial 1,440m from southern tip only)	-75,000	39,000	-13,000	13,000

Based on the volumes presented in Table 15, the total volume in the active oceanward beach system is estimated as being around 450,000m^3. Or 10% of the total islands volume above 0m MSL. The remaining 4.3 million m^3 is interior inactive sediment that is not subject to reworking by wave action. The sediment budget does not extend to other time frames, areas or rates (e.g., rate of lagoon infilling) as there is insufficient data to quantify this element.

4.4.8 Cost

As with any project, costs and available funding is one of the critical factors. The remoteness of the Tuvalu island group means that the significant project costs will be incurred from mobilisation of plant, materials and machinery to site. The Tuvalu Public Works Department (PWD) has resources in Funafuti to undertake small to medium-sized construction projects and general maintenance associated with public infrastructure such as roads, water reticulation and public facility repairs and upgrades. The PWD has a fleet of excavators, tractors, dump trucks as well as a range of smaller plant in Funafuti and some other equipment located on the outer islands undertaking ongoing works, see Figure 65. PWD have some skilled labour and machinery operators and are a local resource that may significantly reduce costs on the delivery of these projects especially on the outer islands.

Figure 65: Tuvalu Public Works Department (PWD) machinery on Nanumea currently undertaking upgrades to the school; Loader, 20T dump truck, 20T excavator, 8T excavator. (source: TCAP, 2020).

For the reclamation works on Funafuti the greatest cost of the project will be associated with mobilisation and demobilisation of a dredge large enough to undertake the volumes of nourishment required to the depths of available sediment at the distance to the site. This is a project cost that cannot be substantially reduced or mitigated using alternate technologies or methods. As such, the major cost savings for this work will be found through efficiencies of the project timeline and workflows. As an example, standby times of the dredge (and associated staff) will be another high-cost item of the reclamation project, reduction of this through selection of the best season to work or selection of a dredge that is able to work in larger wind/wave conditions may result in a larger cost saving than the selection of a dredge with a smaller mobilisation fee. Other efficiencies for the reclamation project could be found from the following:

- As the GMC require a dredge for fill, the removal (or reduction in number) of these as core material from the reclamation bund may incur cost savings as the dredge would be able to complete its filling of the bund and would not be required for further use. In comparison the smaller 2.5m^3 GSC units can be filled manually however take substantially longer to complete the same length of bund.
- The material cost alone of steel sheetpiling the approximate 700m bund wall distance is envisaged to be significantly greater in comparison than rock, concrete or GSC options.
- Due to the significant amount of material required to construct the bund wall, the use of a material that is modular (e.g., can be efficiently shipped/packed within itself) is preferred. It is envisaged that the amount of imported rock or concrete units required would be greater than the capacity of a single barge, whereas the equivalent amount of sheetpiles or GSC/GMC units required would be able to be shipped (through Funafuti Port) or carried upon the dredge/work barge on its initial mobilisation to site.

High (concept) level cost comparison of each concept option are provided in Appendix E. These costs have not been provided by a quantity surveyor, but rather are sourced from literature review and past projects within Tuvalu and the Pacific as well as informal early contractor engagement via emails/phone calls with experienced contractors. An Expression of Interest (EOI) phase will be undertaken following approval of the concept designs and will further inform construction methodologies, contract modalities and subsequently costing of each concept to

4.5 Preliminary basis of design

The following section presents a preliminary basis of design (BoD) for each of the coastal protection designs. The BoD is based on a desktop analysis of available data, global models and first principal investigations. A final BoD will be provided with the detailed design report and designs.

4.5.1 Nanumea

In order to determine design wave heights and water levels at the location of the BTB and Seabee wall on Nanumea it is necessary to transform waves from the nearshore design depth of 20m (calculate in the previous section) across the coral reef platform onto the upper shoreline. To calculate run-up and island inundation, a process-based wave-resolving hydrodynamic model (XBeach Non-Hydrostatic, ‘‘XBNH’’) has been used. Pearson et al. (2017) developed a matrix of these model runs into what they refer as a ‘‘Bayesian Estimator for Wave Attack in Reef Environments’’ (BEWARE) tool which relates incident hydrodynamics and coral reef geomorphology to coastal flooding hazards on reef-lined coasts. The BEWARE tool improves system understanding of reef hydrodynamics by examining the intrinsic reef and extrinsic forcing factors controlling runup and flooding on reef-lined coasts. The Bayesian estimator has high predictive skill for the XBNH model outputs that are flooding indicators and has been validated for several available field cases, for more information see Pearson et al. (2017). It was found that, to accurately predict flooding hazards, water depth over the reef flat, incident wave conditions, and reef flat width are the most essential factors, whereas other factors such as beach slope and bed friction due to the presence or absence of corals are less important.

The BEWARE tool was used at 5 cross-sections along the west coast of Nanumea for a nearshore (reef edge, >20m depth) design wave height and water level to determine transformed wave heights and water levels at the shoreline. The transect locations at Nanumea can be seen in Figure 66**Error! Reference source not found.**. The relationship between offshore wave height and shoreline wave height and water level (due to wave setup) for each of the extracted Nanumea west coast profile locations can be seen in Figure 82 in Appendix A. The preliminary offshore design wave height was calculated as 6.2m. Design water level (offshore) should incorporate the following components: Mean High Water Spring (MHWS) tide level, maximum recorded value of Inverse Barometric Effect (IBE) water level and the current 50-year sea level rise projection, or:

Design Water Level = MHWS + IBE + SLR[3]

= 0.83 + 0.28 + (0.51 – 0.08)

= 1.54m MSL

[3] $SLR = SLR_{RCP8.5,2070} – SLR_{BASE}$
SLR_{BASE} = Sea level at 2020 compared to base level at 2005 = 8mm (see **Error! Reference source not found.**)

As a conservative estimate to inform preliminary concept designs the following design values were used:

- **Offshore (reef edge) wave height: Hs = 7m**
- **Offshore (reef edge) water level: WL = 2m MSL**

It should be noted that the BEWARE tool only provides data interpolation within its matrix of results to a maximum offshore (reef edge) wave height of 5m and water level in intervals of 0.5m up to 2m. As such, design values were linearly extrapolated from the BEWARE matrix to the offshore design wave height (7m) to determine shoreline design parameters. The shoreline wave heights and water levels (due to wave setup) can be seen by the extrapolated (dotted) lines in Figure 82 in Appendix A and tabulated in **Table 16.**

In order to calculate the amount of inundation of different sized BTB structures based on the design wave heights and water levels calculated in **Table 16**, runup and overtopping calculations were made based on empirical formulae provided in EurOtop (2018) for a sea dike as seen in Figure 44. Overtopping calculations were made for the current storm berm dimensions as well as BTB designs with an additional, 500mm, 1000mm, and 1500mm berm crest elevation. Results of the inundation calculations can be seen in Figure 84 in Appendix B.

From these (overly conservative) results, BoD values can be made for wave and water levels on the shoreline at Nanumea based on the highest values attained from the analysis.

- **Shoreline design wave height: Hs = 1.6m**
- **Shoreline design water level: WL = design water level + wave setup**
  = 1.54 + 0.97
  = 2.67m MSL

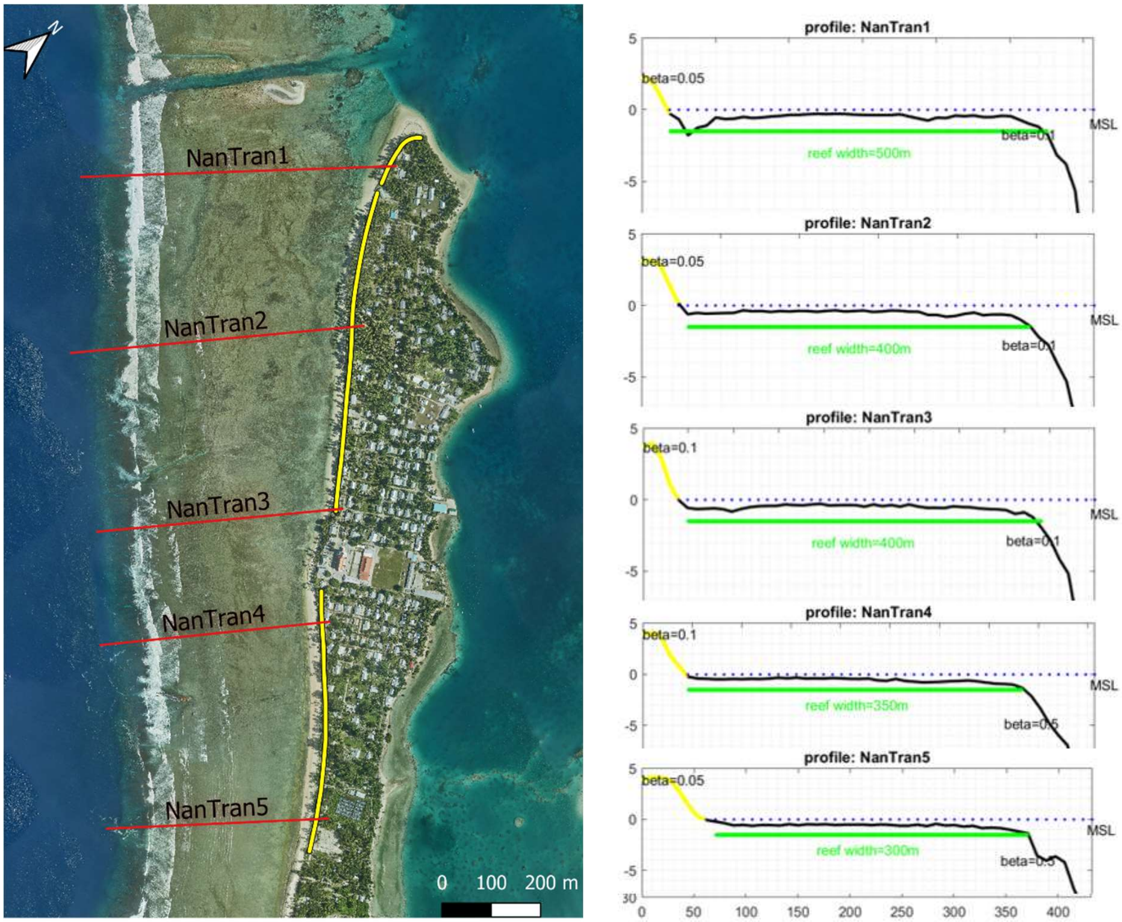


Figure 66: Nanumea profile extraction locations (NanTran1 – NanTran5) along the west coast project site.

Table 16: Inshore wave heights and water levels at the shoreline of Nanumea based on extrapolation of BEWARE outputs

Transect	$H_{OFFSHORE}$	$WL_{OFFSHORE}$	$COT\alpha_{OFFSHORE}$	$WIDTH_{REEF}$ (m)	$COT\alpha_{SHORELINE}$	$H_{SHORELINE}$	$WL_{SHORELINE}$
Nantran1	7	2	0.1	500	0.05	0.92	0.66
Nantran2	7	2	0.1	400	0.05	0.98	0.69
Nantran3	7	2	0.1	400	0.1	1.11	0.66
Nantran4	7	2	0.5	350	0.1	1.64	0.97
Nantran5	7	2	0.5	300	0.05	1.46	0.91

Table 17: Overtopping Rate, Q (ml/s/m) for differing BTB heights at the five reef transects along Nanumea. The colour of the overtopping rate denotes whether the BTB passes EurOtop 2018) safe overtopping rates for the design wave height and water level.

Transect	Overtopping Rate, Q (ml / sec /m)			
	Storm Berm	Storm Berm +500mm	Storm Berm +1000mm	Storm Berm +1500mm
Nantran1	submerged	115,000	5,995	110
Nantran2	31,000	2,189	96	3
Nantran3	9,359	1,708	269	37
Nantran4	31,514	7,211	1,428	251
Nantran5	5,825	669	63	5

Note:

Q = 100ml/s/m is the safe overtopping rate for buildings just back from crest and crest vegetation

Q = 1000ml/s/m is the safe overtopping rate for a person standing just back from the crest

4.5.2 Nanumaga

The same methodology for calculating design water levels and wave heights on Nanumea was undertaken for Nanumaga. The BEWARE tool was used at 5 cross-sections along the west coast of Nanumaga for a nearshore (reef edge, >20m depth) design wave height and water level to determine transformed wave heights and water levels at the shoreline. The transect locations at Nanumaga can be seen in **Figure 67**. The relationship between offshore wave height and shoreline wave height and water level (due to wave setup) for each of the extracted Nanumaga west coast profile locations can be seen in Figure 82 in Appendix A. The preliminary offshore design wave height was calculated as 6.7m. Design water level (offshore) should incorporate the following components: Mean High Water Spring (MHWS) tide level, maximum recorded value of Inverse Barometric Effect (IBE) water level and the current 50-year sea level rise projection, or:

Design Water Level = MHWS + IBE + SLR[4]

= 0.84 + 0.28 + (0.57 – 0.08)

= 1.61m MSL.

As a conservative estimate to inform preliminary concept designs the following design values were used:

- **Offshore (reef edge) wave height: Hs = 7m**
- **Offshore (reef edge) water level: WL = 2m MSL**

[4] $SLR = SLR_{RCP8.5,2070} - SLR_{BASE}$
SLR_{BASE} = Sea level at 2020 compared to base level at 2005 = 8mm (see **Error! Reference source not found.**)

It should be noted that the BEWARE tool only provides data interpolation within its matrix of results to a maximum offshore (reef edge) wave height of 5m and water level in intervals of 0.5m up to 2m. As such, design values were linearly extrapolated from the BEWARE matrix to the offshore design wave height (7m) to determine shoreline design parameters. The shoreline wave heights and water levels (due to wave setup) can be seen by the extrapolated (dotted) lines in Figure 83 in Appendix A and tabulated in **Table 19**.

In order to calculate the amount of inundation of different sized BTB structures based on the design wave heights and water levels calculated in **Table 19Table 16**, runup and overtopping calculations were made based on empirical formulae provided in EurOtop (2018) for a sea dike as seen in Figure 44. Overtopping calculations were made for the current storm berm dimensions as well as BTB designs with an additional, 500mm, 1000mm, and 1500mm berm crest elevation. Results of the inundation calculations can be seen in Figure 85 in Appendix B and are tabulated in **Table 19**.

From these (overly conservative) results, Basis of Design values can be made for wave and water levels on the shoreline at Nanumaga based on the highest values attained from the analysis.

- **Shoreline wave height: Hs = 2.3m**
- **Shoreline water level: WL = design water level + wave setup**
  = 1.61 + 0.95
  = 2.56m MSL

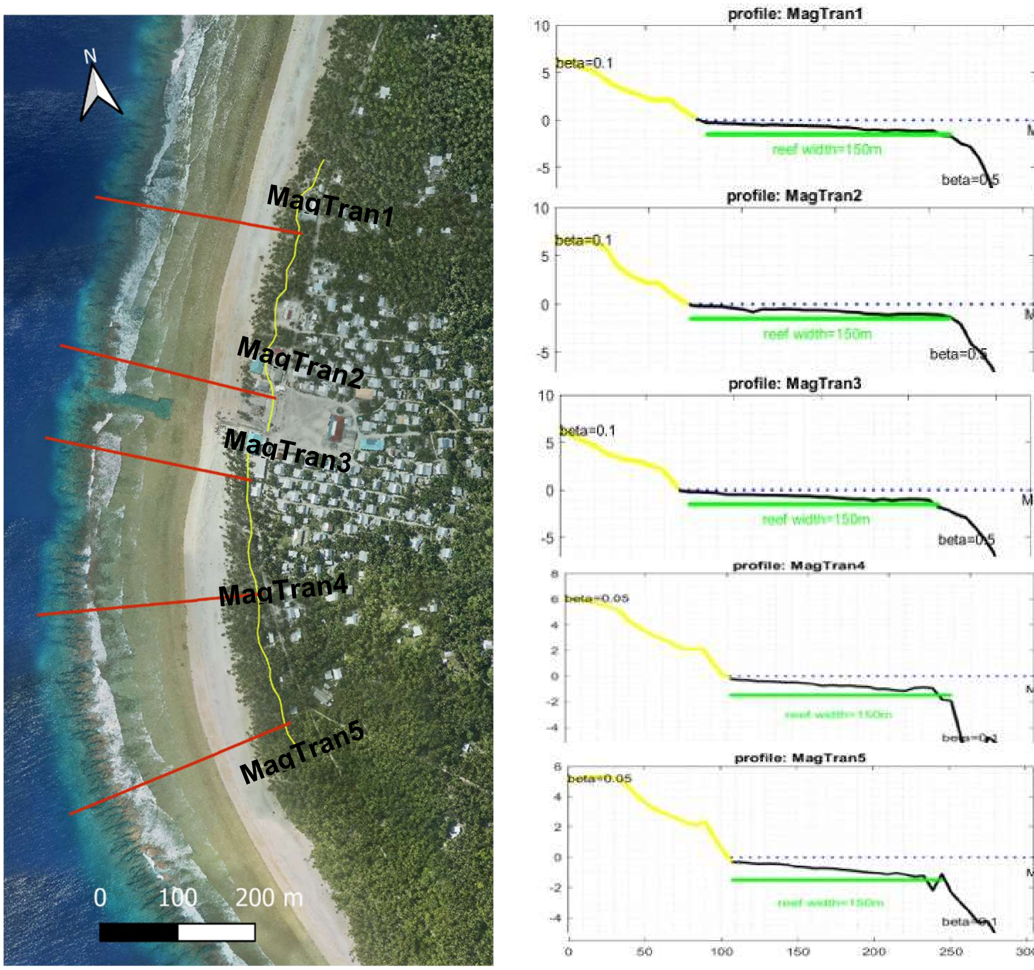


Figure 67 Nanumaga profile extraction locations (MagTran1 – MagTran5) along the west coast project site

Table 18: Inshore wave heights and water levels at the shoreline of Nanumaga based on extrapolation of BEWARE outputs

Transect	$H_{OFFSHORE}$	$WL_{OFFSHORE}$	$COT\alpha_{OFFSHORE}$	$WIDTH_{REEF}$ (m)	$COT\alpha_{SHORELINE}$	$H_{SHORELINE}$	$WL_{SHORELINE}$
Magtran1	7	2	0.5	150	0.1	2.30	0.95
Magtran2	7	2	0.5	150	0.1	2.30	0.95
Magtran3	7	2	0.5	150	0.1	2.30	0.95
Magtran4	7	2	0.1	150	0.05	1.39	0.68
Magtran5	7	2	0.1	150	0.05	1.39	0.68

Table 19: Overtopping Rate, Q (ml/s/m) for differing BTB heights at the five reef transects along Nanumaga. The colour of the overtopping rate denotes whether the BTB passes EurOtop (2018) safe overtopping rates for the design wave height and water level

Transect	Overtopping Rate, Q (ml / sec /m)			
	Storm Berm	Storm Berm +500mm	Storm Berm +1000mm	Storm Berm +1500mm
Magtran1	2.3	0.2	<0.1	<0.1
Magtran2	1.8	0.2	<0.1	<0.1
Magtran3	6.3	0.7	<0.1	<0.1
Magtran4	<0.1	<0.1	<0.1	<0.1
Magtran5	<0.1	<0.1	<0.1	<0.1

Note:

Q = 100ml/s/m is the safe overtopping rate for buildings just back from crest and crest vegetation

Q = 1000ml/s/m is the safe overtopping rate for a person standing just back from the crest

4.5.3 Funafuti

The following conservative estimations have been made to further the design process of the Funafuti reclamation and associated bund. The largest waves are expected to occur during the passage of cyclones past the atoll and will be generated across the relatively small 20km north-south and 16km east-west fetches, with design waves heights being calculated by AECOM (2015) calculated as seen in Table 5. It has been noted that long period (ocean) swells are present within the lagoon and have been witnessed during even neap tides, suggesting that there is swell penetration through the larger reef passes as well as over the intertidal reef flats.

Conservative calculations of wave height at the toe of the reclamation bund have been made in order to inform concept designs. A conservative wave height calculation can be made assuming that the waves at the toe of the reclamation bund will be depth limited. Figure 58 shows the depth at the toe of the reclamation bund, the depth ranges from approximately 1.2m in the west (adjacent to the QEP reclamation) to 2.7m in the east offshore of the Catalina Ramp. The deeper, eastern section appears to be founded on the reef flat whereas the shallower western section has a small deposit of sediment (presumably from the QEII reclamation). Building of the reclamation bund will be founded on the reef flat and as such a value of -2.7m MSL should be used as a conservative estimate of the reef flat depth along the reclamation bund.

In simple wave breaking theory, waves break if the ratio of wave height over water depth exceeds the threshold value of 0.78 (Masselink and Hughes, 2003). This can be a simple method for determining conservative design wave heights for design. As such, depth -limited wave heights at the toe of the structure during means sea level conditions can be calculated as follows:

Reclamation bund (depth-limited) design wave height = 0.78 x depth

= 2.1m

A small numerical modelling exercise was undertaken to determine ARI wave heights at the proposed reclamation site, see Design wave height for Funafuti reclamation bund Technical Note (Bluecoast, 2021b). The exercise confirmed the conservative ARI 100year significant wave height at he proposed reclamation site at just over 2m with an associate wave period of 5 seconds.

The design water level at the reclamation bund has been defined as: ARI 50yr WL + Wave setup + 2100 Sea Level Rise. The U.S. Army Corps of Engineers (USACE) Shore Protection Manual (SPM,1984) gives a value of wave setup being 10% of the wave height for irregular waves on an open coast and for planar beach profiles (uniform slopes) during storm conditions. Therefore,

Reclamation bund design water level = ARI 50yr WL + Wave setup + Sea Level Rise
= 1.65 + (0.1 x 2.1) + 0.75
= +2.61m MSL

5. Concept designs

5.1 Preamble

The following sections detail the concept designs for approval by GCF, GoT and the Kaupule of each island. The level of detail the designs contain is based upon the preliminary basis of design (above) and cost comparison. Basic dimensions, height and materials may change during the detailed design phase and following the consent of the key stakeholders listed above.

5.2 Funafuti

The concept design for the Funafuti reclamation can be seen in Appendix C. The design will have the following key elements.

5.2.1 Alignment

The alignment of the reclamation bund matches that which was proposed in the original GCF application seen in Figure 42 but will omit the already completed QEP reclamation area. The proposed reclamation will carry on eastwards from the boundary of the QEP reclamation for 770m, as shown in Figure 68. The reclamation has the same approximate landward protrusion from the shoreline as the QEP reclamation (between 80m and 110m) as to not disrupt coastal processes along the shoreline. The widest section of the reclamation is in the centre (almost 110m) which curves landward at both extremities, resembling a gently curved artificial headland. This design feature was added to maximise the reclamation area without disrupting the coastal processes or natural aesthetics.

Figure 68: Alignment of the Funafuti reclamation concept design

5.2.2 Reclamation

The maximum height of the reclamation is currently 3.4m MSL, approximately 1m above the preliminary design water level, 2m above HAT and approximately 0.5m above the crest level of the QEP reclamation bund. The highest elevation will be located just landward of the lagoon edge of the reclamation and will slope gently landward at a gradient of approximately. The final drainage design will be informed by a stormwater drainage modelling exercise. A cross-section of the reclamation at chainage 400 (approximately halfway along the 770m reclamation length) can be seen on Figure 69. The total volume of reclamation material is approximately 270,000m^3.

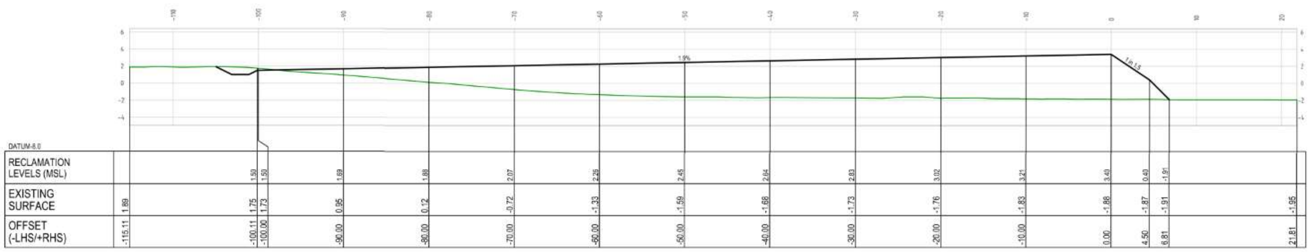


Figure 69: Cross-section through the Funafuti reclamation concept design at Chainage 400 (approximate mid-bund).

5.2.3 Drainage

An open stormwater channel has been designed to drain surface flows from the reclamation. The channel is located at the approximate location of the vegetation line of the Vaiaku waterfront, the extent can be seen in Figure 68 and a profile in Figure 70. The channel is to have at least a 10m width at the base, dropping at a 1V:4H slope from each side. The channel has been placed such that excess stormwater will also drain from the Vaiaku settlement. The stormwater channel has been designed to be open in order to minimise the maintenance requirements culverts and pit/pipe infrastructure will incur.

A drainage assessment will be undertaken at the detailed design phase which may include increasing the slope of the channel or reducing its length by allowing the channel flow to go in both directions from the centre of the bund. This will also incorporate flows from the QEP reclamation.

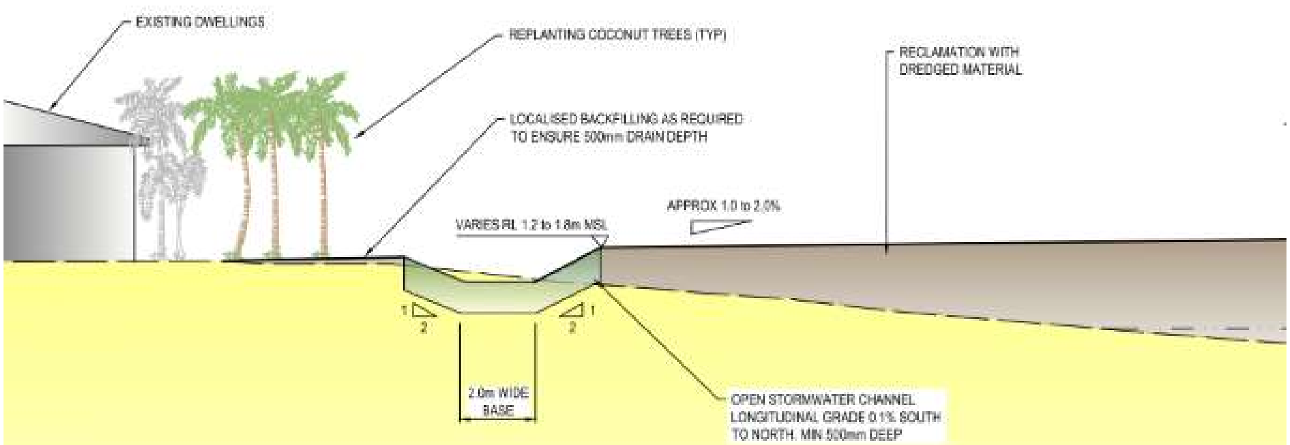


Figure 70: Open stormwater channel at landward edge of the Funafuti reclamation concept design

5.2.4 Bund

It has been determined that the most suitable material for the reclamation bund concept design is Geotextile Sand Containers (GSC). The concept design (Figure 71) shows a combination of Geotextile Mega Containers used as the core of the bund revetment with GSC units used to protect the seaward face of the revetment, advantages of this design have been described in the options analysis provided above. Further detailed design will be undertaken to determine the minimum amount of GSC units require for the bund once design wave heights (and periods) have been confirmed.

The selection of this concept was primarily based on the cost and timeline efficiencies of shipping the GSC and GMC units in comparison to that of rock or concrete armour units. It has also been suggested that further reclamation works (Seaward of both this and the QEP alignment) are part of the GoT's long term strategy. With this is mind, burial of geotextile units will be more cost-efficient, then removing and replacing rock, concrete units or sheetpiles.

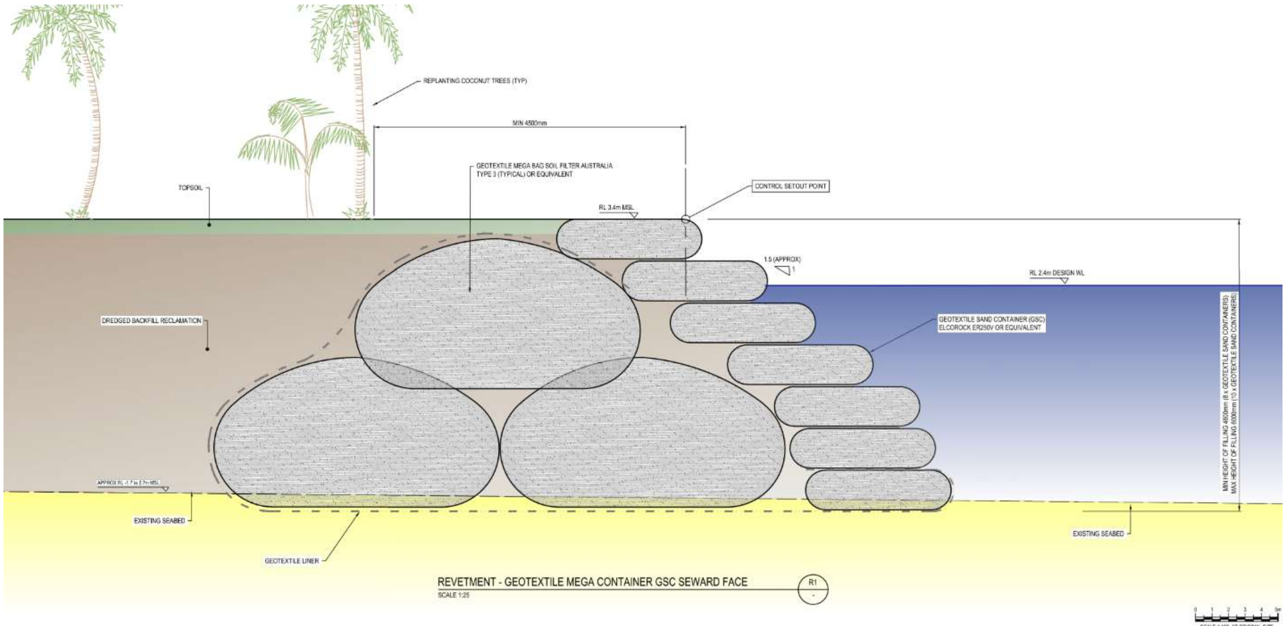


Figure 71: Concept design of GMC core/GSC revetment bund.

5.2.5 Setback

It is recommended that a minimum construction setback be put in place following reclamation. This is due to construction practises threatening the integrity of the buried GMC units and the additional lateral loading on the bund. A planned setback with a seaward slope would also alleviate surface flows over the reclamation. Figure 72 shows the adjacent QEP reclamation area during the construction of facilities associated with the Pacific Climate Summit in 2019. Around the summit site, there can be seen to be a minimum 20m setback from the crest of the rock revetment and 15m setback from the crown of the concrete capping beam. The large structure in the centre of the reclamation is the approximate 50m x 40m concrete slab being poured for the main (now 2 storey) conference building. Although there is currently no adopted national Tuvaluan building code, modern Tuvaluan buildings are generally constructed as floating slabs, with larger housing or community buildings built atop a large, buried water tank with a slab on top and internal concrete walls for internal support.

It is recommended that following construction of the reclamation and bund, that a minimum **20m setback** from the landward crest of the reclamation bund be maintained. The design of the reclamation bund should (as a minimum) incorporate this setback in its stability calculations. It is also assumed (due to the lack of building code or planning acts) that any new buildings will not exceed two storeys.

Figure 72: QEII reclamation during the construction of buildings in reparation for the Pacific Climate Summit 2019.

5.2.6 Boat harbour

The eastern extremity of the reclamation terminates at the location of the Catalina Ramp[5]. The concept design for the eastern bund is designed for the inclusion of a small boat harbour area, sheltered from the lagoon waves by a 30m long GMC core rock breakwater (described below). The purpose of the boat harbour is to provide a protected all-tide access for smaller local fishing vessels to the shore and reclamation area. Having a designated all tide boat harbour will ensue that vessels do not moor on the reclamation bund possibly damaging GSC units with anchors or propellers. The area will also provide a meeting location for fishing vessels and trade.

[5] A remnant WWII dredged harbour and rock revetment where Catalina Seaplanes would berth

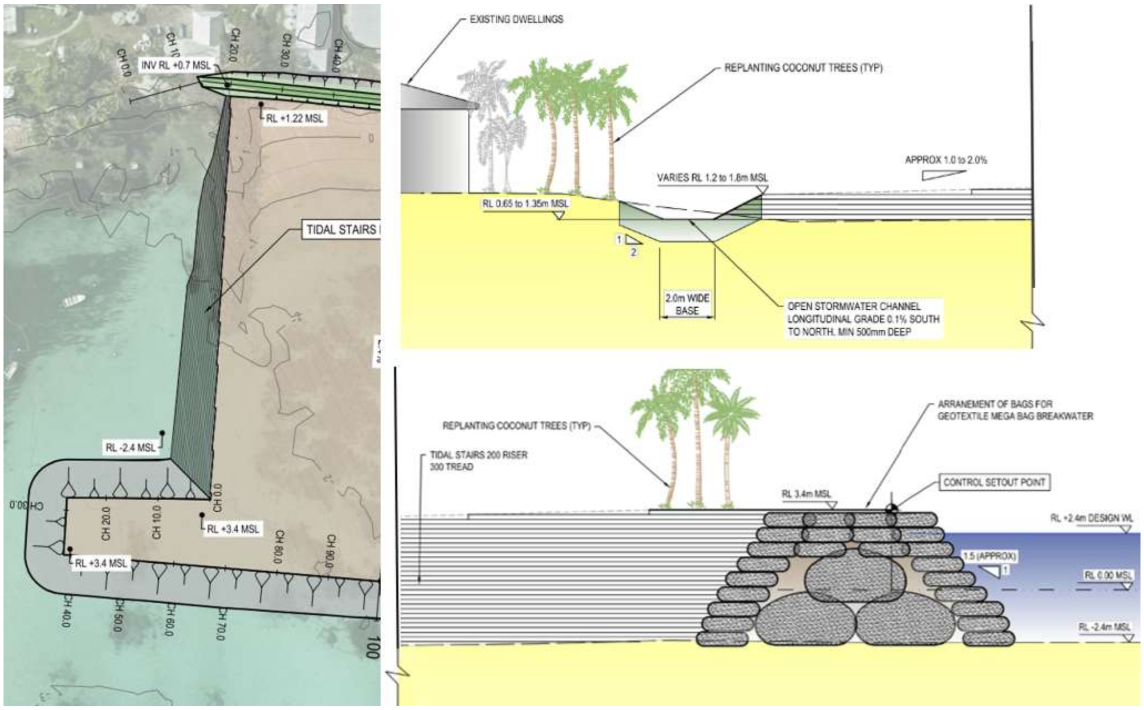


Figure 73: Concept design for the Catalina Ramp boat harbour and breakwater

Groyne reuse

An alternate material for the breakwater construction may be viable with the reuse of the rock on the eastern face of the QEP reclamation as well as the two groynes that lie within the reclamation footprint, seen in Figure 74. The rock appears to be igneous or metamorphic boulders (most probably granite) with a mean diameter (D50) of around 1250mm, with boulders of up to 2000mm also found. The approximate dimensions and estimated available volume of rock of each of the structures is provided in the table below. The rock is considered of high quality and from superficial inspection appears of suitable size (and density) for the expected wave climate within the lagoon to remain stable if used in a breakwater structure at this location.

As the proposed boat harbour is expected to be a long-term feature that is likely to remain undeveloped for a longer period in comparison to the reclamation bund, it is recommended that the armouring of the boat harbour breakwater be constructed of this reused rock material. It is still envisaged that the core of the breakwater consist of the geotextile mega containers.

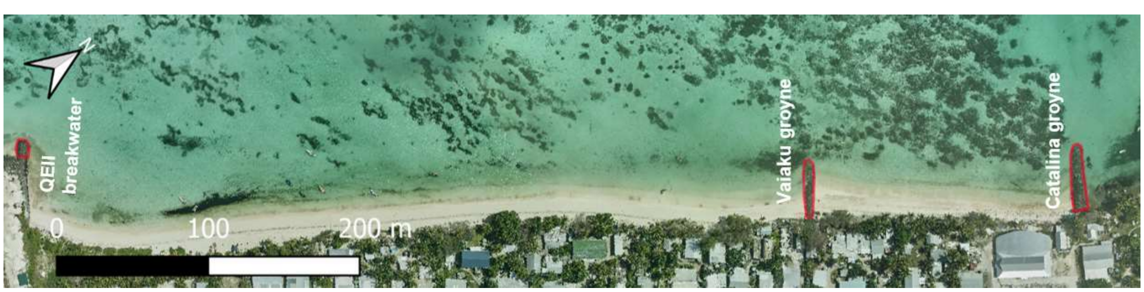


Figure 74: Possible rock sources for re-use, left to right: QEII Park breakwater, Vaiaku and Catalina groynes

Structure	Length (m)	Width (m)	Mean height (mMSL)	Mean bed level (mMSL)	Usable rock (assuming 0.5m thickness)
QEII breakwater	12.5	8.5	1.2	-0.5	45m^3
Vaiaku groyne	37	7	0.85	-0.5	99m^3
Catalina groyne	43	9	0.9	-1.5	117m^3

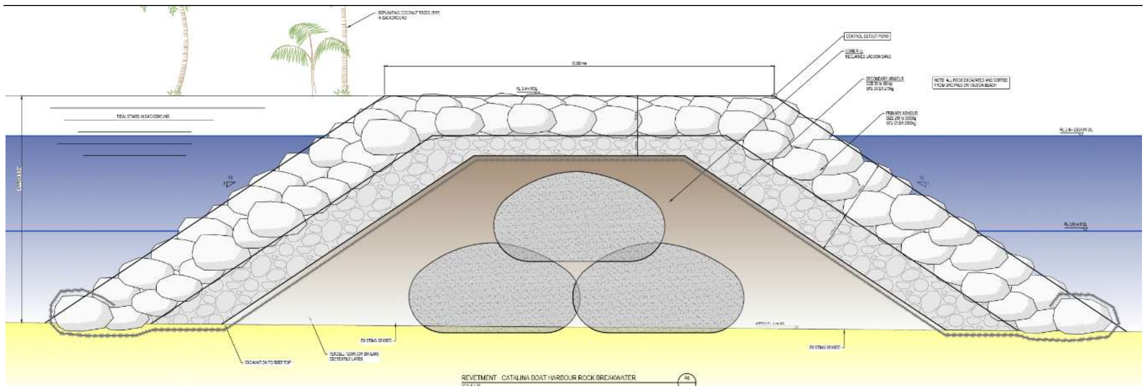


Figure 75: Geotextile mega-container core rock breakwater

5.3 Nanumaga

The concept design for the Nanumea Berm Top Barriers (BTB) can be seen in Appendix C. The design will have the following key elements.

5.3.1 Alignment

The alignment of the 810m BTB on Nanumaga has been designed to follow the ridge of the storm berm on the west coast of the island, with small deviations made where the structure footprint will overlap key public and private infrastructure such as roads, houses or community buildings.

Church

A landward diversion of the alignment can be seen in the centre of the BTB between chainages 400 to 480 inland of the boat harbour (Figure 76). This section of Nanumaga's coastline plays an important role in Nanumagan life; It is the location of the church (washed away during TC Pam), the Pastor's residence, a large store and is the first place arriving boats unload. As the majority of these structures have either been destroyed or badly damaged following the passage of the most recent cyclones (see Figure 77), it is an opportune time to move these structures landward, follow the design intent of the BTB structures and follow the alignment of the ridge of the storm berm along the coastline, which diverts inland at this location. In addition, following good coastal management practise, it is more prudent to avoid rather than accommodate risks wherever possible.

Figure 76; Proposed alignment of the Berm Top Barrier concept design and footprint on Nanumaga.

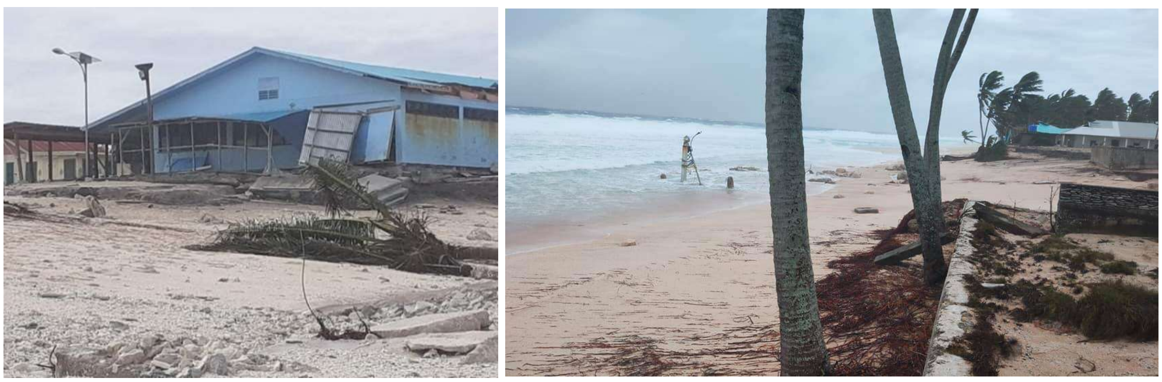


Figure 77: Damages caused by the passage of both TC Pam and Tino on Nanumaga at (left) the Store and (right) the church, where only the concrete slab remains. (Source: Alan Resture)

However due to the importance of these structures to the people of Nanumaga and upon written plea from the Kaupule, design considerations have been made to protect these structures within TCAP. Two options have been described with various levels of (long and short-term) protection and costs.

<u>Option1 : Continuation of BTB</u>

Figure 78 depicts Option 1 for protection of central Nanumaga, a realignment of BTB through old store footprint. This option continues the alignment of the BTB along the western shore of Nanumaga from the south, over the remanence of the store footings. The alignment continues north as far inland as possible to ensure the BTB is positioned as closely as possible to the footings of the old church. As the land height along this alignment dips below 4m MSL, it is expected additional fill will be required to ensure the fished height of the BTB remains constant along its length.

Due to the presence of the boat channel offshore of this section of coastline, it has been seen to be particularly vulnerable to inundation during large wave events, most recently TC Tino. As such, it is expected that during future events there may be some localised scour at the base of the BTB. It is recommended that should this occur, localised fill be replaced immediately following the event to once again cover the BTB and ensure no damage is incurred to the geotextile structure. Introduction of the BTB in this area will create a discontinuity between the beach and the village in this area due to the raising of the land. It is recommended that this area be sufficiently retained with fill atop the GMC to maximise design life of the structure.

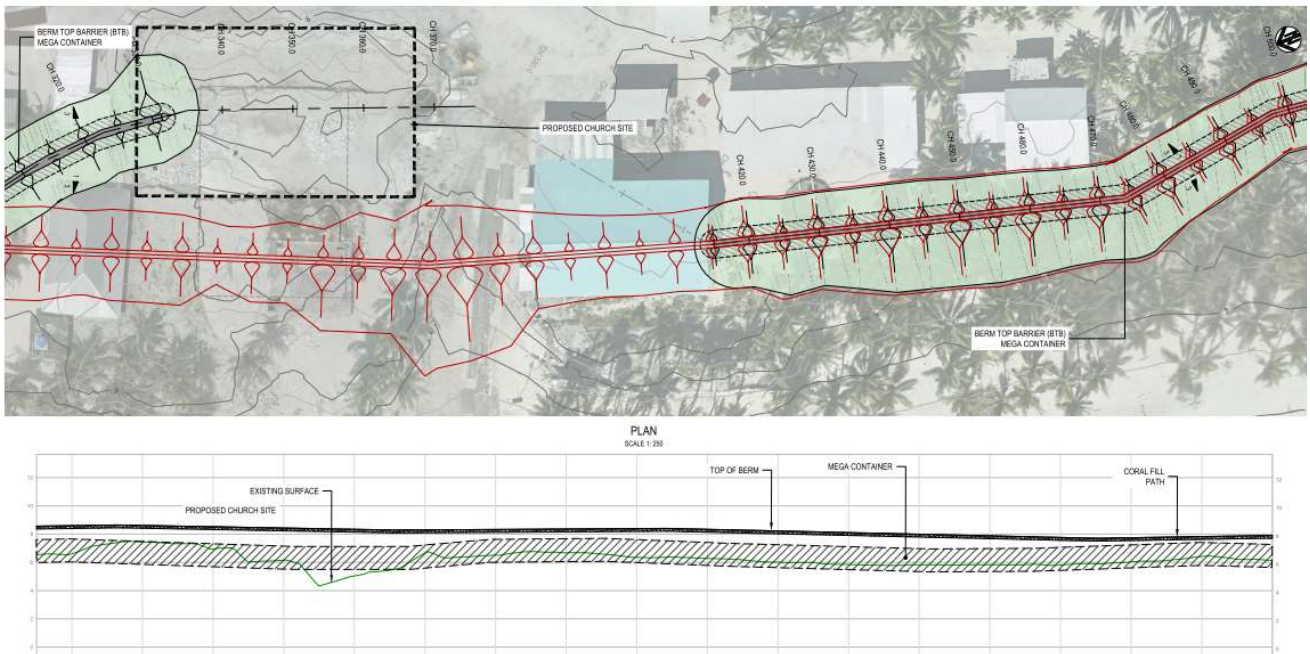


Figure 78: Option1 for protection of central Nanumaga: realignment of BTB through old store footprint.

<u>Option 2: PVC Sheetpiles along structure footings</u>

Figure 79 depicts Option 2 for protection of central Nanumaga, PVC sheetpiles to protect structure footings. Due to the shoreward location of the structures along central Nanumaga; the pastor's house, the church and the store, changing the align of the BTB shoreward of these structures will reduce their efficiency and place the BTB's within the active shoreline. The introduction of PVC sheetpiling along the footing of these structures is a last line of defence to try and limit the damage incurred in the next major storm event. The sheetpiles are to be offset from the footings of each of the buildings by approximately 2-3m. The sheetpiles will extend from the ground surface a maximum of 1m above the footing height and will be capped by either a PVC or timber beam. The sheetpiles will be supported laterally by a bracing beam and a dead anchor (on the landward side), see Figure 79 below.

The PVC sheetpiles will be of the thickest grade commercially available and will be driven until submission, which is expected to be the underlying reef flat (approximately -1m to +1m MSL). The area between the sheetpiles and each of the footings of the structures will be back filled so as to retain a maximum of 0.5m clearance of the sheetpiles from the ground surface. Likewise, on the shoreward face of the sheet piles, sand will be backfilled and placed against the sheetpiles to minimise any vertical faces of the sheetpiles as these will now become a safety hazard.

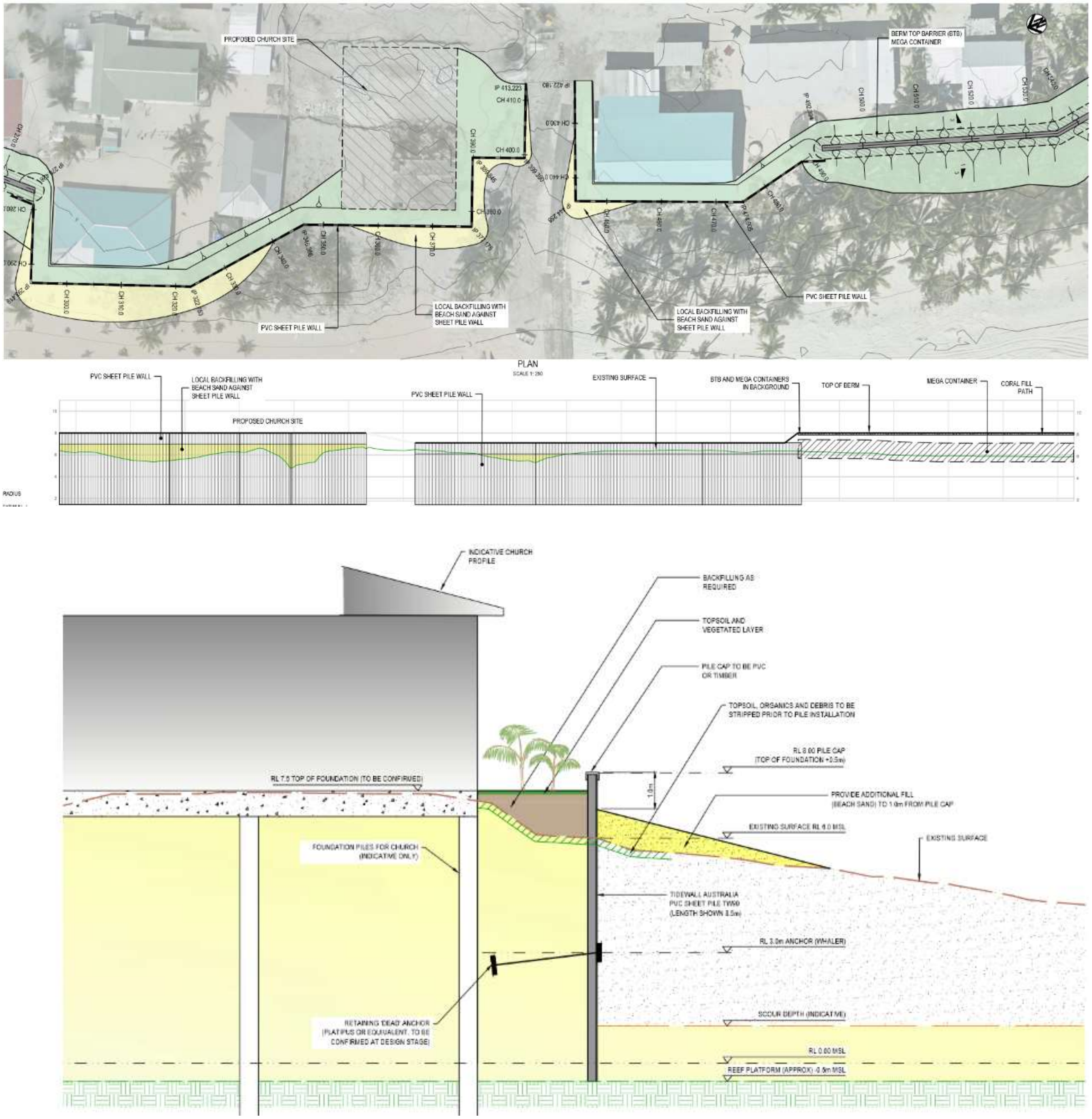


Figure 79: Option2 for protection of central Nanumaga: PVC sheetpiles to protect structure footings

It is strongly recommended that sheetpiling is not used for the hard revetment structures on Nanumea or Nanumaga. Placed within the active zone of a reef-mediated shore, it is expected that the nature of storm events on these shores will cause damage to the sheetpile structures and disrupt the coastal processes. Figure 54 provides an example storm response to FRC sheetpiles placed within the active zone of a reef mediated shore.

5.3.2 BTB material

Nanumaga's BTB will consist of a GMC core berm with a clean sand footpath atop. The design has been described in the sections above and are provided in Figure 47. The height of the BTB will increase the height of the storm ridge berm by a minimum of 1500mm reducing the inundation risk over the berm for the design storm to well below 100ml/s/m, the acceptable overtopping rate for buildings just back from crest and crest vegetation (EurOtop, 2018).

The approximate volume of material required for the BTB's on Nanumaga is provided in the table below. Material for the crushed coral core is planned to be sourced from the extensive sediment deposits on the northern tip of the island that has been deposited during storm periods.

5.4 Nanumea

The concept design for the Nanumea Berm Top Barriers (BTB) and revetment can be seen in Appendix C. The design will have the following key elements. Please note a subsequent Nanumea Concept Design report *Bluecoast, 2021) has been undertaken with updates to the recommended concepts stated below.

5.4.1 Alignment

The alignment of the 1530m BTB on Nanumea has been designed to follow the ridge of the storm berm on the west coast of the island, with small deviations made where the structure footprint will overlap key public and private infrastructure such as roads, houses or community buildings. The BTB is seen to be closer to the active shoreline on Nanumea as compared to Nanumaga due to the distance of the infrastructure to the coastline on this island, with this in mind, the crushed coral footpath has not been included in the Nanumea BTB design.

Figure 80: General alignment of the Berm top Barrier concept design and footprint on Nanumea.

5.4.2 BTB material

Nanumea's BTB will consist of two separate designs; most of the BTB will be a berm consisting of a GMC core as well as a 180m section of hard Seabee revetment from Chainage 800 to 980 of where the church and associated buildings are located inland. This section was planned by the Kaupule to replace the previous hard revetment structure which was destroyed through successive storm events. The designs have been described in the sections above and are provided in Figure 47 and Figure 52. The BTB will increase the height of the storm ridge berm by a minimum of 1500mm reducing the inundation risk over the berm for the design storm to well below 100ml/s/m for most sections and below 1000ml/s/m for all sections, this is the safe overtopping rate for a person standing just back from the crest, see Appendix B for full overtopping analysis

The approximate volume of material required for the BTB's on Nanumea is provided in the table below. As described in the design consideration section, Nanumea is also seen to have extensive sediment deposits on the southern tip of the island that has been deposited during storm periods.

Section	*Approximate volume (m³)*
GMC core BTB: core	5,555
GMC core BTB: cover and fill	5,910

5.4.3 Revetment

The 170m section of coastline on Nanumea to be protected by a hard revetment can be seen in

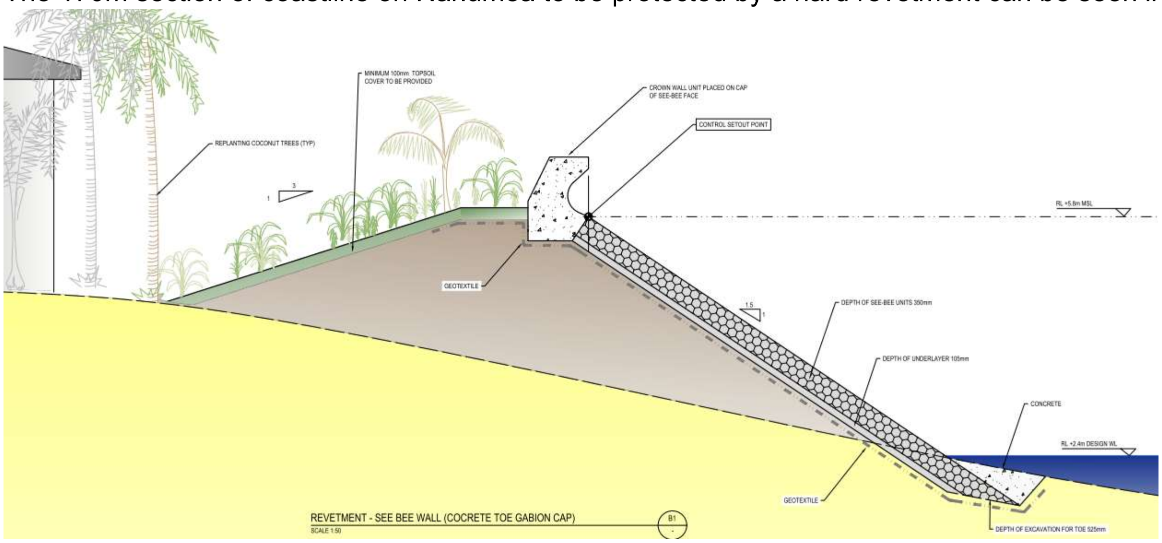
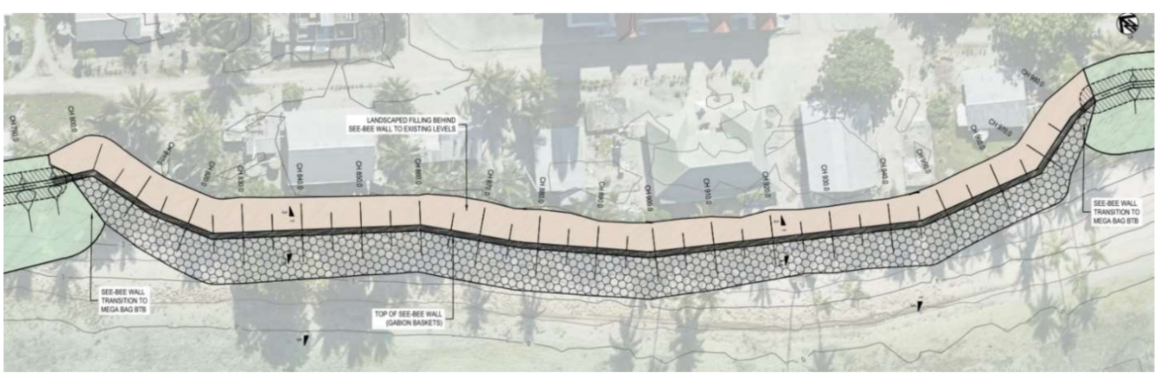


Figure 81. It has been determined that the most practical, cost-efficient and resilient revetment material for use on Nanumea is that of a Seabee wall. The toe of the Seabee revetment will be founded on the reef flat and concreted into the substrate. The Seabee revetment and design details are provided in Figure 52, Figure 81 as well as the specification sheets attached to the drawings in Appendix C.

It is recommended that the individual Seabee units be cast from high strength concrete at the supply location (Fiji, Australia, etc.) and shipped to site. If the units are to be batched on-island in Nanumea with imported materials to reduce wastage/damaged units during freighting, supervision and testing is to ensure that the concrete for the units meet design strength capacity.



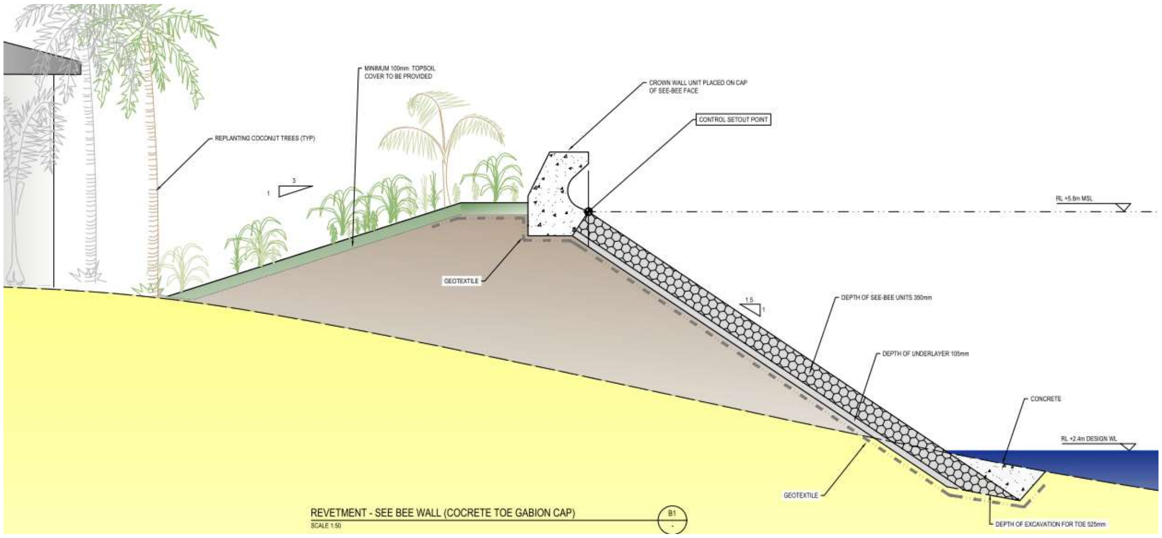


Figure 81: Plan view (top) and profile of Seabee revetment concept design on Nanumea

6. Summary

This Concept Design Report (and the accompanying Nanumea Concept Design Report) has been provided as a summary document detailing the objectives of the Tuvalu Coastal Adaptation Project, definition of the problem and the development of the engineering design. A desktop-based options analysis has also been included for each of the adaptation options to ensure that the chosen designs are the most appropriate for each island. This was also assessed against a high-level investigation into materials, possible costs and construction techniques to verify the feasibility of the proposed options.

This report brings together the key findings of the preliminary ESIA, geotechnical and LiDAR investigations undertaken for the TCAP project. Meteorological and oceanographic (metocean) data as well as global numerical wind and wave models have been used in conjunction with this data to gain an understanding of the environmental conditions and the latest climate change models have been used to inform future climatic scenarios to inform a preliminary basis of design.

The concept designs are provided in Appendix C and a brief summary below:

- Funafuti:
    - Construction of a bunded reclamation located on the Vaiaku Foreshore in Funafuti, consisting of approximately 270,000m^3 of sediment dredged from the Fogafale Lagoon. Approximate dimensions of the reclamation are 700m x 100m raised at its highest point to approximately 2m above the Highest Astronomical Tide . The design of the reclamation bund incorporates 20m long stacked geotextile mega containers (GMC) fronted by a protective revetment consisting of stacked 2.5m^3 Geotextile sand containers (GSC) units. Drainage slopes and channels are to be levelled into the reclamation surface.
    - Construction of a small boat harbour along the eastern extremity of the reclamation catering for small vessels. A 30m long rock breakwater protects the harbour, with

rock sourced from the two rock groynes on Tausoa Beach located within the reclamation footprint.

- Nanumaga:
  - Construction of 665m of Berm Top Barrier (BTB) on the storm berm on the western coast of Nanumaga. The BTB consists of 20m long GMCs laid end to end, covered with locally sourced fill and revegetated.
  - Construction of a pre-cast boat ramp adjoining existing ramp.
- Nanumea: further information on the Nanumea concept designs are provided in the Nanumea Concept Design report (Bluecoast, 2021) and summarised below:
  - Construction of 1,330m of BTB on the storm berm on the western coast of Nanumea, including beach renourishment. The BTB consists of 20m long GMCs laid end to end, covered with locally sourced fill and revegetated.
  - Construction of a 170m long pre-cast concrete Seabee seawall.
  - Construction of seven 25m long Reef Top Barriers (RTB) in the nearshore. The RTB will be pre-cast concrete units keyed into the reeftop.

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8. Appendix A: Inundation analysis

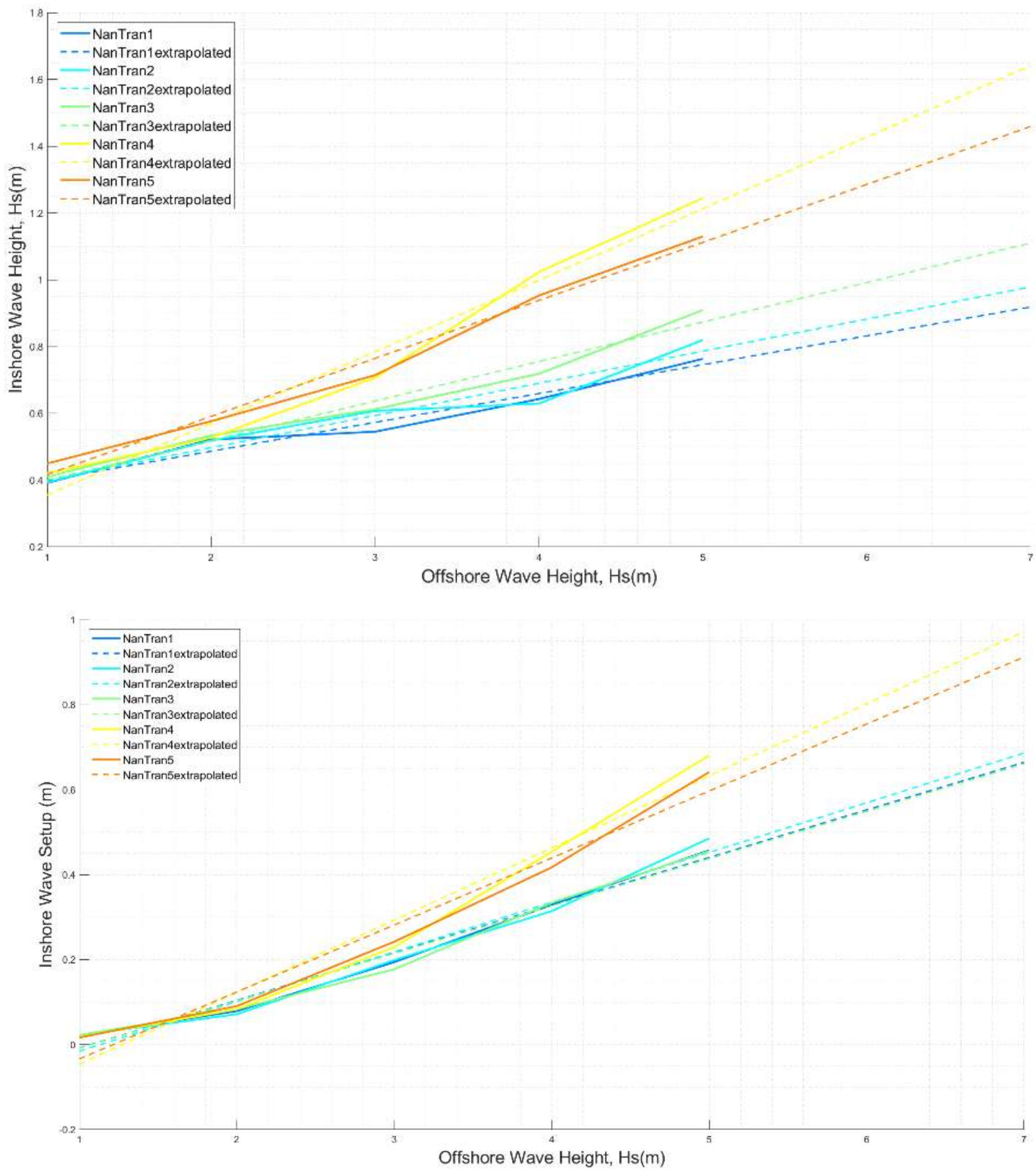
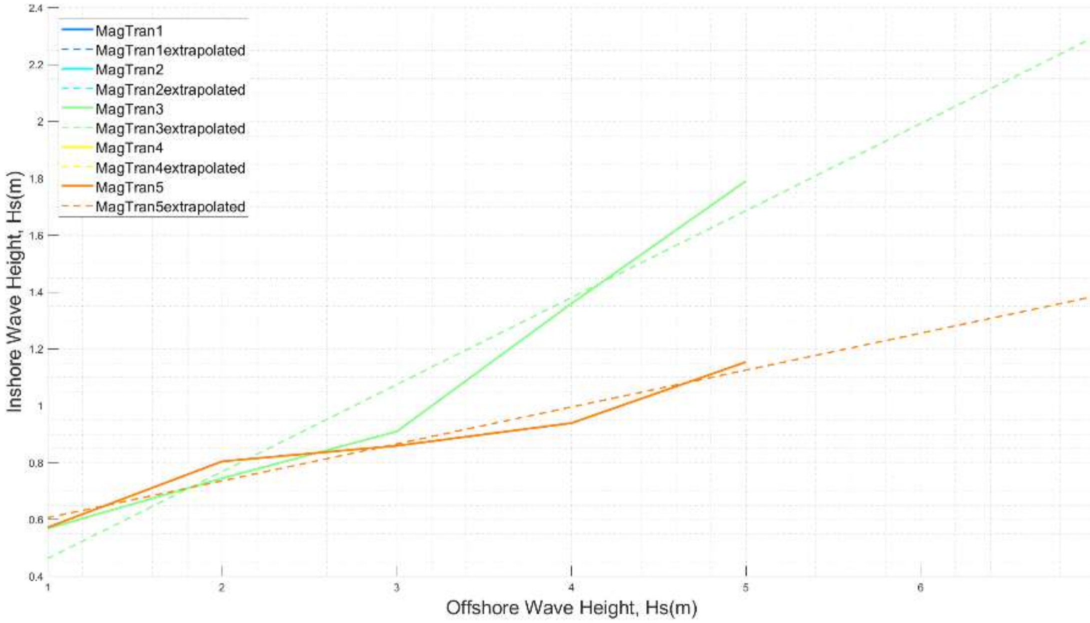


Figure 82 BEWARE reef top wave transformation tool for varying offshore wave heights for Nanumea profile extraction locations (NanTran1 – NanTran5) along the west coast project site.



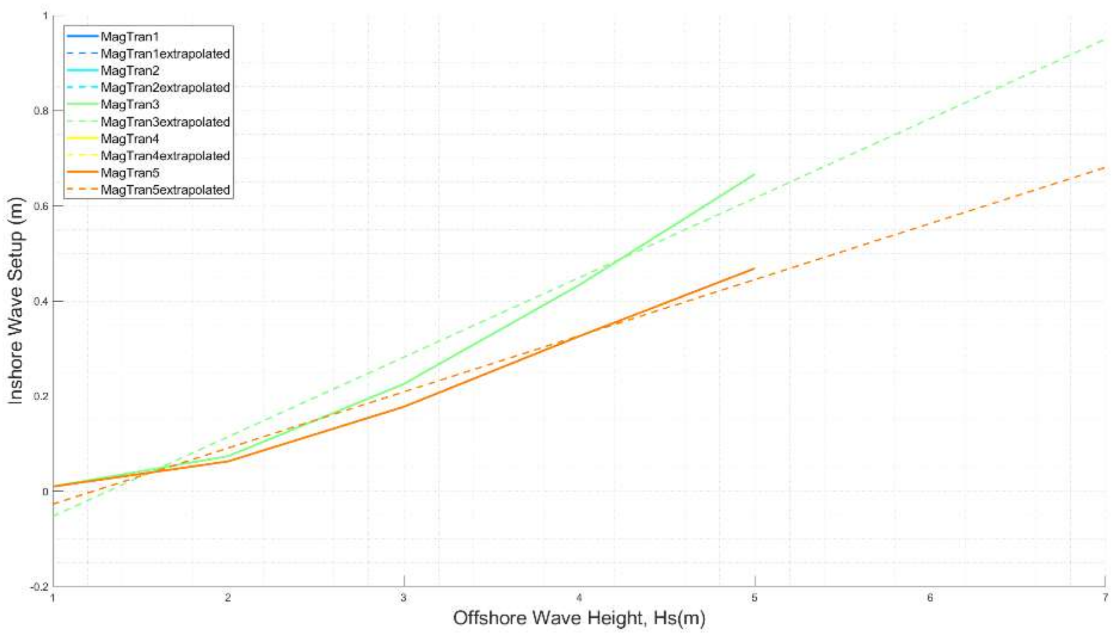
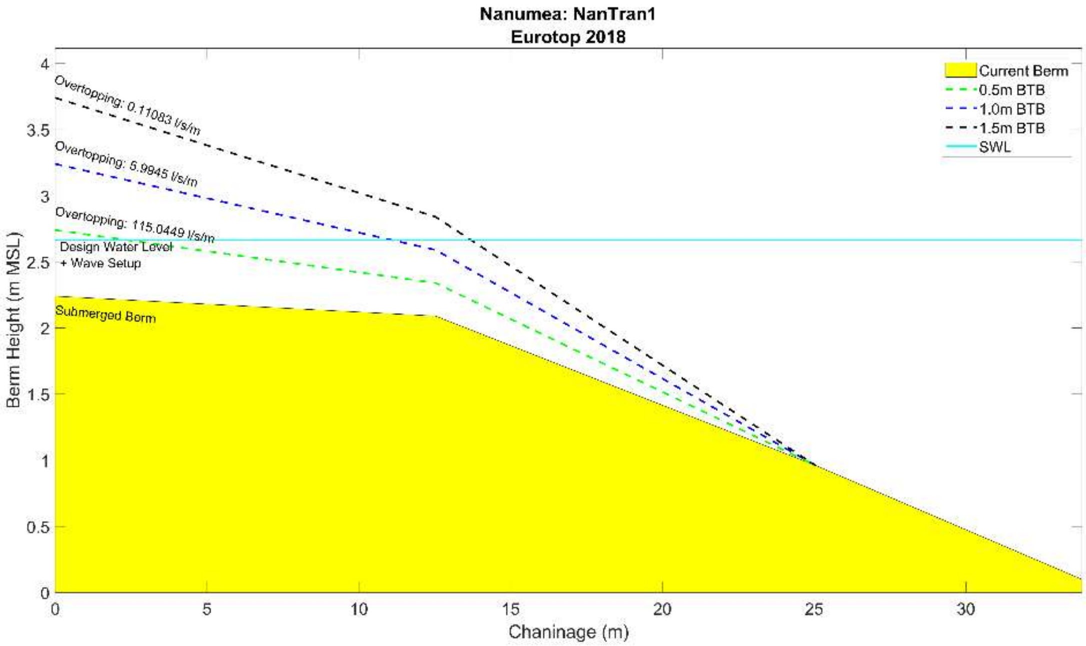
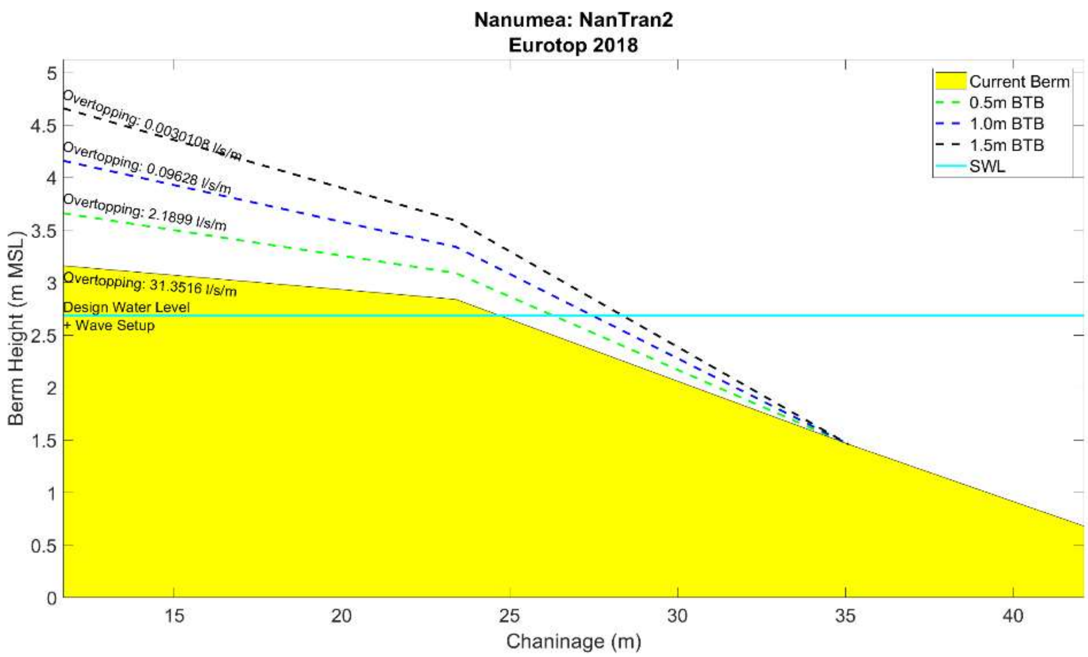
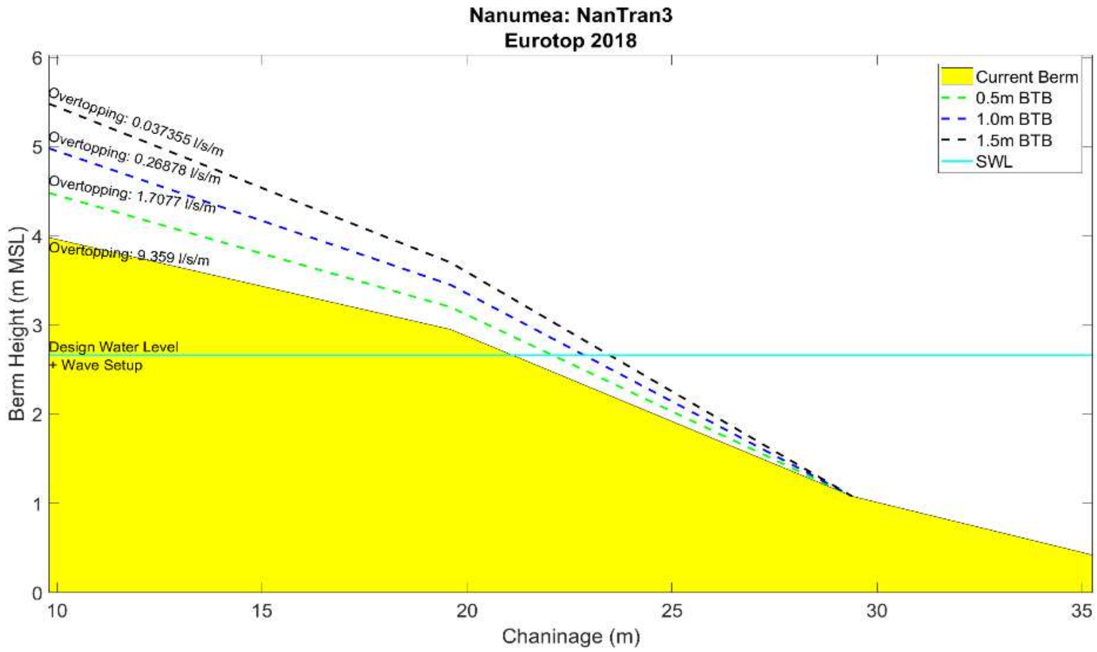


Figure 83 BEWARE reef top wave transformation tool for varying offshore wave heights for Nanumaga profile extraction locations (MagTran1 – MagTran5) along the west coast project site.

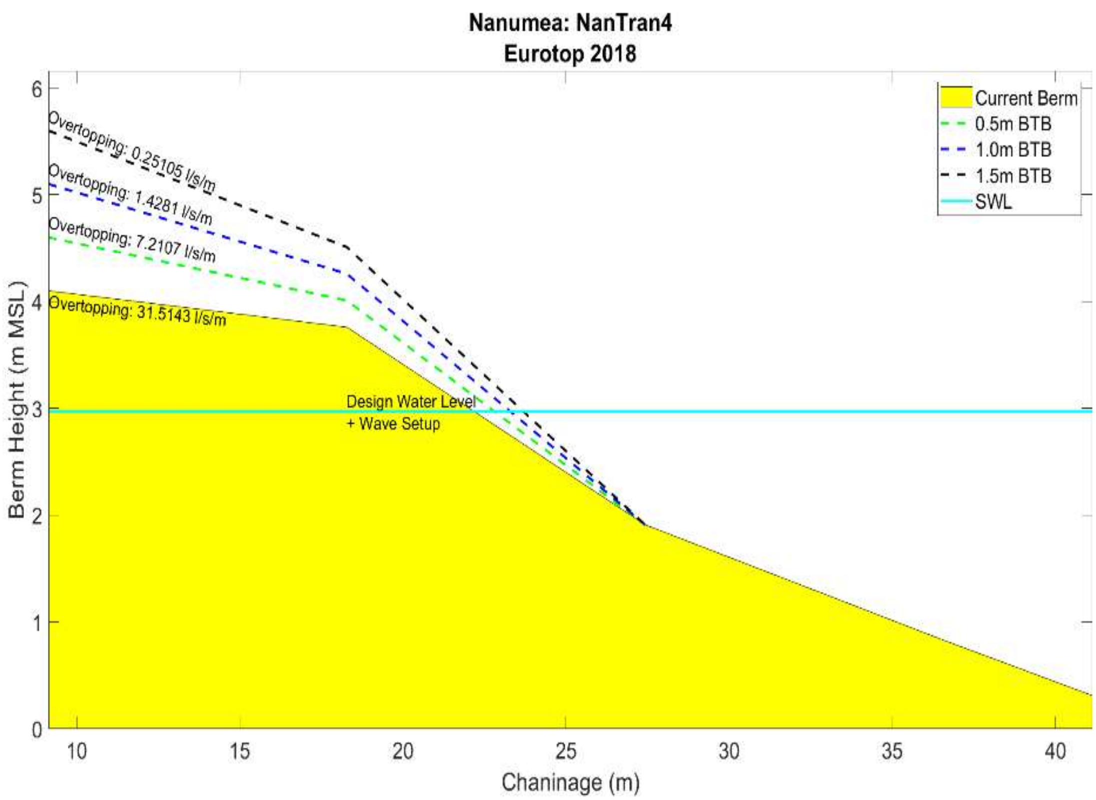
9. Appendix B: Overtopping analysis







Nanumea: NanTran3
Eurotop 2018
Current Berm
0.5m BTB
1.0m BTB
1.5m BTB
SWL
Overtopping: 0.037355 l/s/m
Overtopping: 0.26878 l/s/m
Overtopping: 1.7077 l/s/m
Overtopping: 9.359 l/s/m
Design Water Level
+ Wave Setup
Berm Height (m MSL)
Chaninage (m)



Nanumea: NanTran4
Eurotop 2018
Current Berm
0.5m BTB
1.0m BTB
1.5m BTB
SWL
Overtopping: 0.25105 l/s/m
Overtopping: 1.4281 l/s/m
Overtopping: 7.2107 l/s/m
Overtopping: 31.5143 l/s/m
Design Water Level
+ Wave Setup
Berm Height (m MSL)
Chaninage (m)

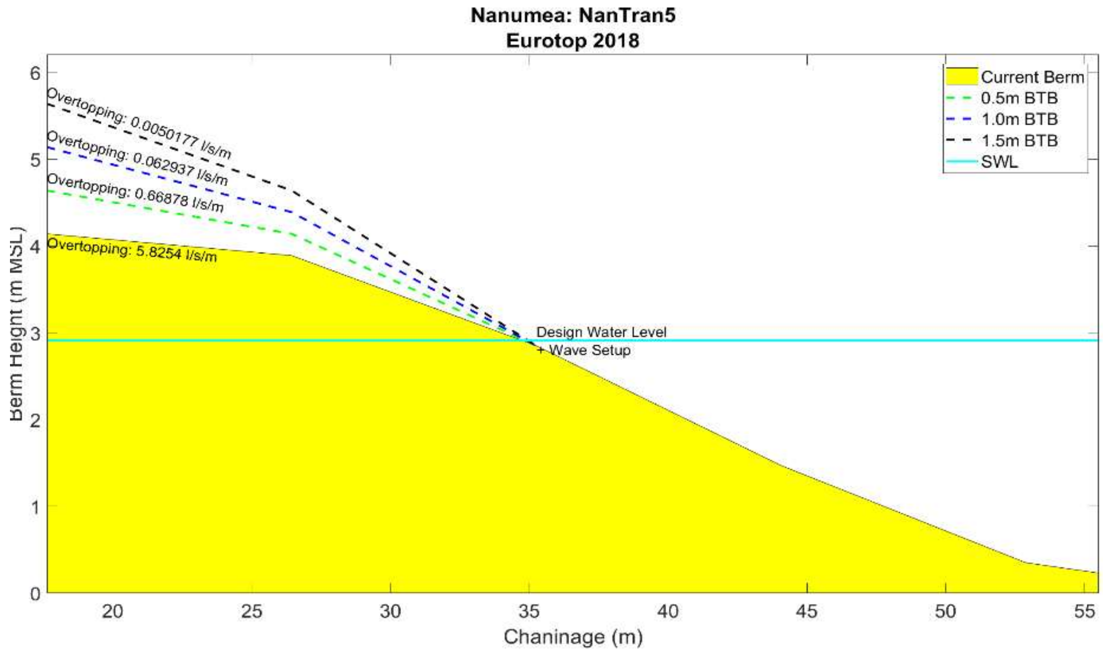
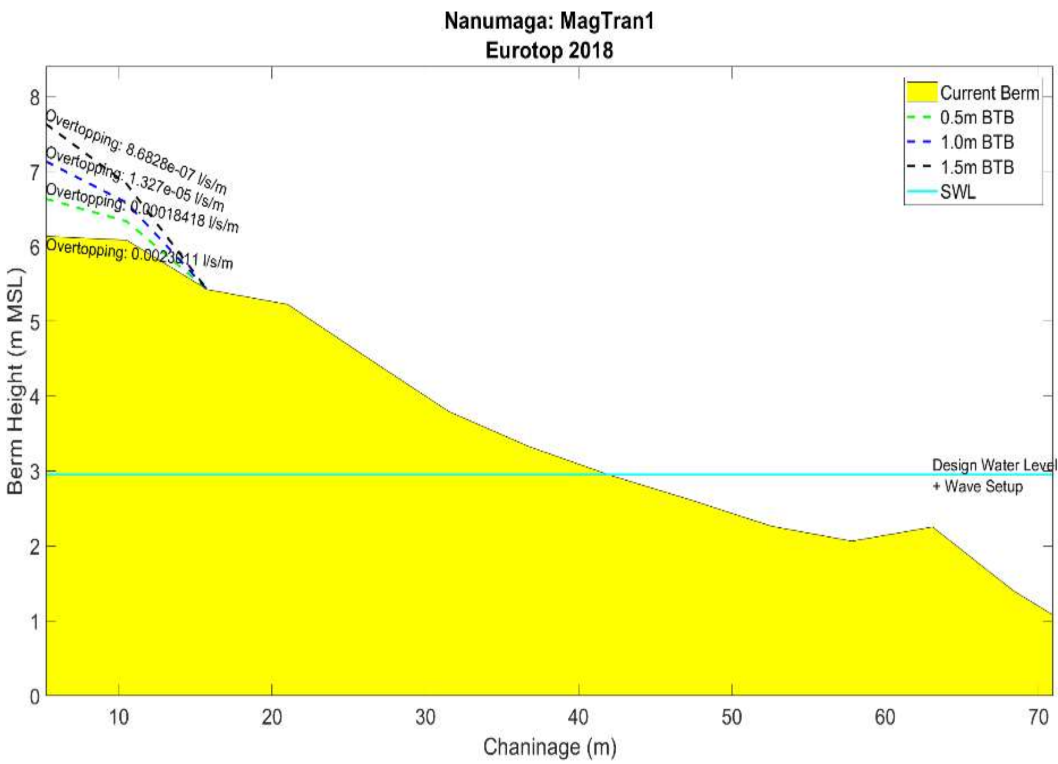
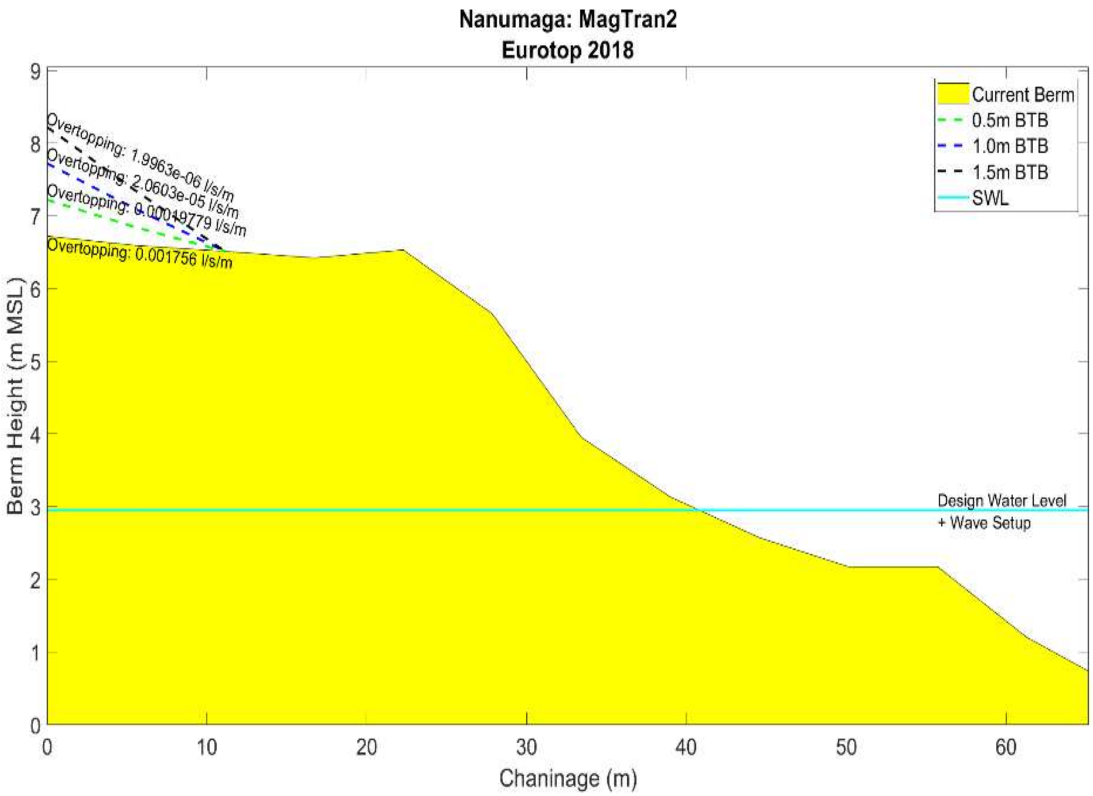
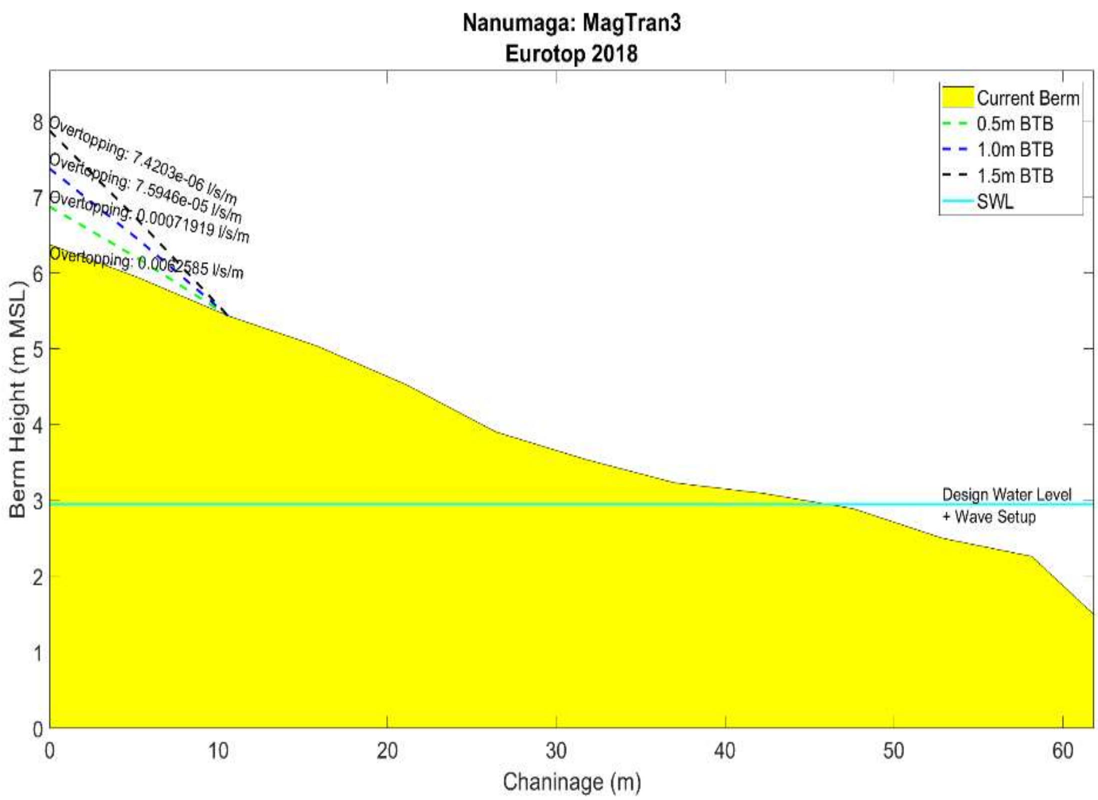
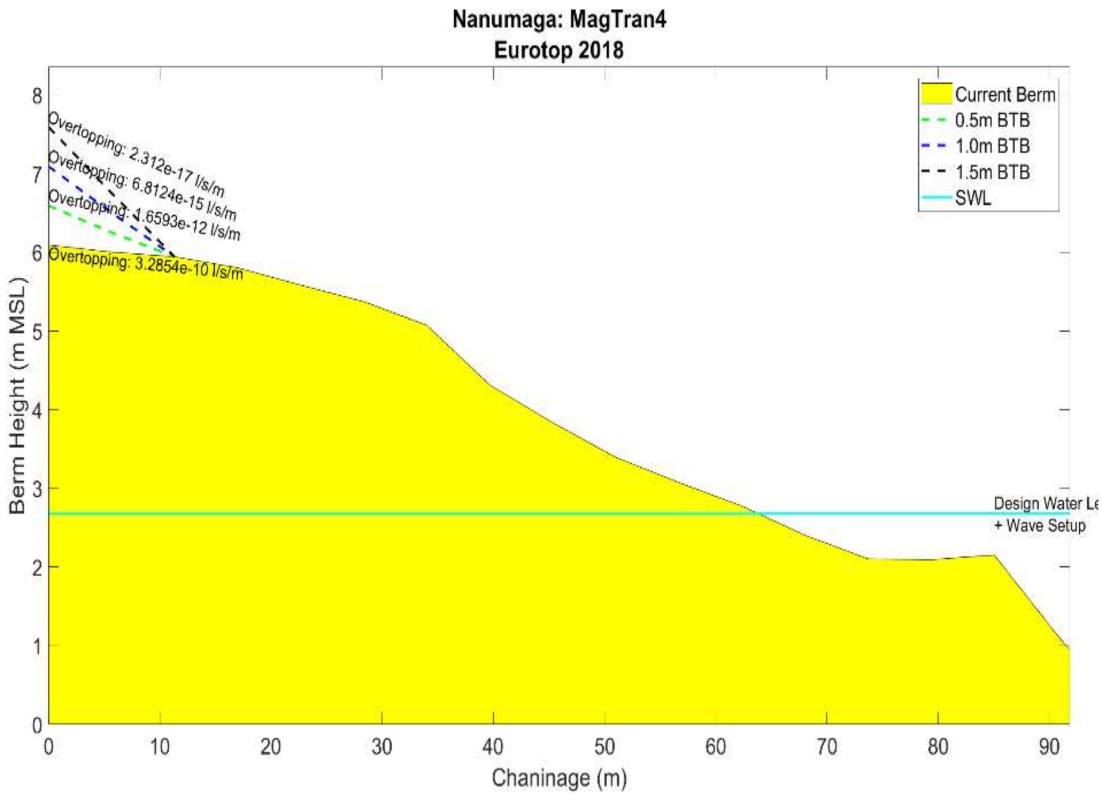


Figure 84: Nanumea wave overtopping analysis of three BTB design heights after EurOtop (2018)




Nanumaga: MagTran2
Eurotop 2018
Current Berm
0.5m BTB
1.0m BTB
1.5m BTB
SWL
Overtopping: 1.9963e-06 l/s/m
Overtopping: 2.0603e-05 l/s/m
Overtopping: 0.00019779 l/s/m
Overtopping: 0.001756 l/s/m
Design Water Level
+ Wave Setup
Berm Height (m MSL)
Chaninage (m)


Nanumaga: MagTran3
Eurotop 2018
Current Berm
0.5m BTB
1.0m BTB
1.5m BTB
SWL
Overtopping: 7.4203e-06 l/s/m
Overtopping: 7.5946e-05 l/s/m
Overtopping: 0.00071919 l/s/m
Overtopping: 0.0062585 l/s/m
Design Water Level
+ Wave Setup
Berm Height (m MSL)
Chaninage (m)



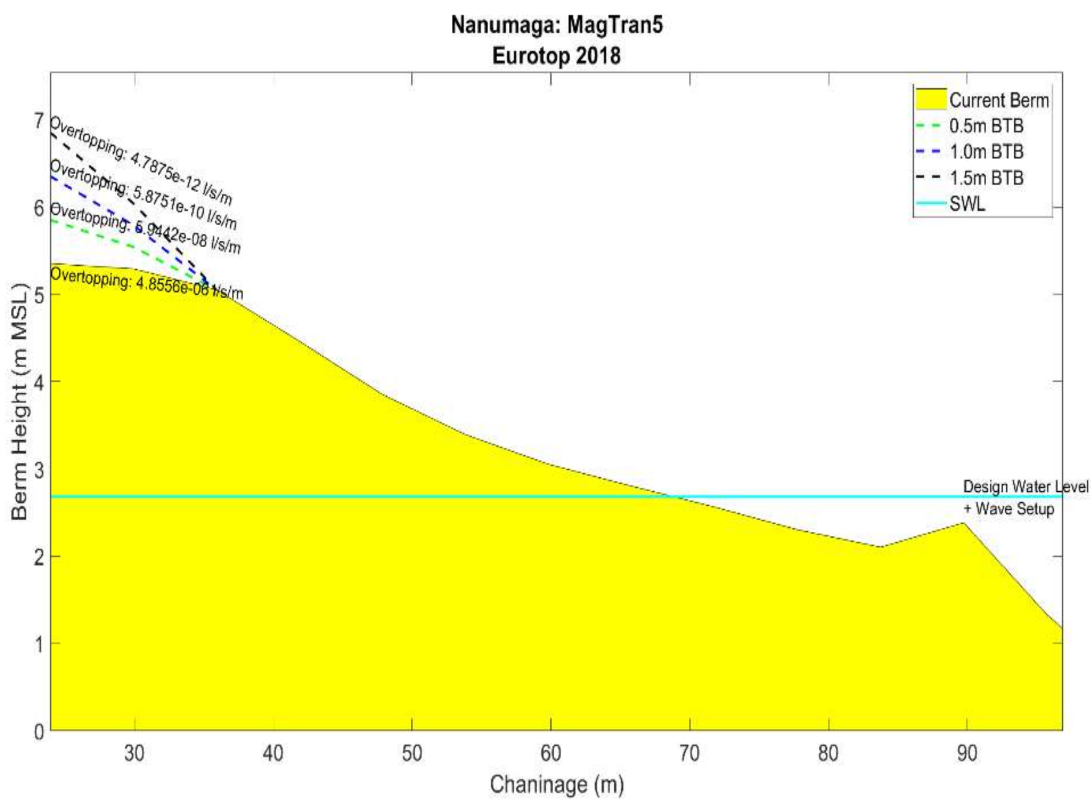


Figure 85: Nanumaga wave overtopping analysis of three BTB design heights after EurOtop (2018)

10. Appendix C: Concept design drawings

11. Appendix D: Nanumea Concept Design Report

12. Appendix E: Preliminary Bill of quantities