Nanumaga Wave-Driven Inundation Assessment, Tuvalu: Physical Modelling Report

WRL TR 2021/19, August 2021

By M J Blacka and M Deiber

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Acronyms and abbreviations

BTB	Berm Top Barrier
MSLA	Mean Sea Level Anomaly
SPC	The Pacific Community
TCAP	Tuvalu Coastal Adaptation Project
UNDP	United Nations Development Programme
WRL	Water Research Laboratory

1 Introduction

The Pacific Community (SPC) is carrying out an assessment of wave-driven coastal inundation hazards for the islands of Nanumaga, Nanumea and Funafuti in Tuvalu (Figure 1.1). The assessment forms part of the Tuvalu Coastal Adaptation Project (TCAP) currently being implemented by the United Nations Development Programme (UNDP) in partnership with the Government of Tuvalu. SPC engaged the Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney to complete a scale physical modelling assessment of wave processes, runup and overtopping hazards for Nanumaga Island (Figure 1.1, Figure 1.2), to support their investigation. Outcomes from the physical modelling will improve the understanding of wave processes across the reef profile, and ultimately help to better understand the inundation risks to the community. The results will also help SPC to verify their existing probabilistic inundation predictions conducted using the Xbeach numerical wave model, and assist with better aligning modelled inundation predictions with community observations from Tropical Cyclone Pam (2015).

Figure 1.1: Location map of Tuvalu and Nanumaga

Figure 1.2: Nanumaga Island, Tuvalu

The investigation site is located on the western side of the reef island of Nanumaga as shown in Figure 1.2. The site is fronted by a sandy beach that is mediated by a narrow fringing reef and very steep fore-reef slope. Typical of reef-islands, the wave processes at Nanumaga are complex and include a combination of wave refraction, shoaling, breaking and dissipation as waves cross the reef. During large wave conditions, these processes generate a significant amount of reef-top wave setup and low frequency infragravity wave energy, which has previously resulted in inundation along the coastal margins (SPC, 2021). To reduce the frequency and impacts of wave overtopping events into the future, it is proposed to increase the elevation of the storm berm at the crest of the beach through the installation of a series of berm-top-barriers along the western coastline of the island (Bluecoast, 2021).

This report documents the objectives, design, testing, and results from WRL's physical modelling investigation. Unless stated otherwise, all dimensions in this report are presented as real-world scale and in S.I. units.

2 Model arrangement, design and operation

2.1 Modelling objectives and arrangement

The key objectives of the physical modelling study included analysis of the following parameters for a range of extreme wave and water level scenarios:

- Wave characteristics and transformation as waves cross the reef profile
- Reef-top water levels, including sustained wave setup and low frequency components
- Wave runup/inundation levels at the beach
- Reduction in wave overtopping achieved by Berm-Top-Barrier coastal protection structures

In reviewing the coastal processes at the site, it was agreed by WRL and SPC that a two-dimensional (2D) cross-shore physical model would sufficiently capture the majority of the key wave and water level processes, and meet the core objectives of the physical modelling study. The 2D approach was considered a reasonable approximation of actual conditions on the western side of Nanumaga given the nature of the reef, with the omission of longshore processes considered a slightly conservative, but cost-effective approach in this instance. While it is acknowledged that a full 3D physical model would have enabled simulation of additional processes such as wave refraction and longshore reef-top water level gradients and currents, these processes were considered to be of secondary importance with regards to their impact on the modelling objectives.

The physical modelling arrangement comprised the testing of a single cross-shore 2D reef profile extending from the village, seaward across the beach and reef flat, terminating at a sufficient depth on the fore-reef slope.

2.2 Modelling facility

The physical modelling program was conducted in the 1.2 m wave flume at WRL. This flume measures approximately 44 m in length, 1.2 m in width and 1.6 m in depth. The flume walls are primarily constructed of waterproofed concrete masonry blockwork, with the exception of a 12 m long glass panelled section where models are constructed, which allows for visual observations during testing. The permanent floor of the flume is constructed of concrete, and during the testing undertaken for this study a representative bathymetry profile was constructed in the flume as a raised plywood platform supported by timber framing.

The wave maker in this flume is of a piston-type and is powered by a 30 kW hydraulic pump. The system is capable of generating both monochromatic and irregular wave series, with only irregular waves tested in this investigation. The wave signal is generated and supplied to the wave maker using a PC with a custom developed wave generation software package.

2.3 Model design

2.3.1 Model scale

Model scaling was based on geometric similarity with an undistorted length scale of 1:25 used for all tests. The scaling relationship between length and time was determined by Froudian similitude, with the following relevant scale ratios (prototype divided by model) being adopted for the model:

- Length ratio $L_R = 25$
- Time ratio $T_R = L_R^{0.5} = 5$
- Velocity ratio $V_R = L_R^{0.5} = 5$
- Overtopping rate ratio $Q_R = L_R^{1.5} = 125$

The model length scale of 1:25 was selected on the basis of maximising the model wave heights for the target wave periods, while within the capacity limits of the wave generator. This model scale ensured that any scale effects were minimised and within acceptable limits, as outlined in physical modelling guidelines (HYDRALAB III, 2007a; HYDRALAB III, 2007b).

2.3.2 Bathymetric/topographic profile

A representative real-world 2D bathymetric and topographic profile was provided by SPC for the physical modelling. The profile was constructed to scale within the wave flume as a simplification of the real-world profile, with Figure 2.1 showing both the real-world and simplified profiles. The model profile covered a cross-shore distance of approximately 260 m, extending seaward from the storm berm at the top of the beach, across the reef flat and down the fore-reef slope to a nominal level of -25 m MSL. The reef and beach profile were constructed in the wave flume from rigid, water resistant plywood and installed across the full width of the flume.

The natural storm berm level at the back of the beach varies in elevation along the length of the village. Two different natural berm levels were tested in the physical modelling program, a higher level of 7.2 m MSL (broadly representative of the berm crest in the area near the former church site), and a lower level of 6.3 m MSL, which is representative of zones further to the north and south along the beach. In both cases the same reef profile and beach slope was used in the model, with the only change being the terminal crest level at the top of the 1V:8H beach slope.

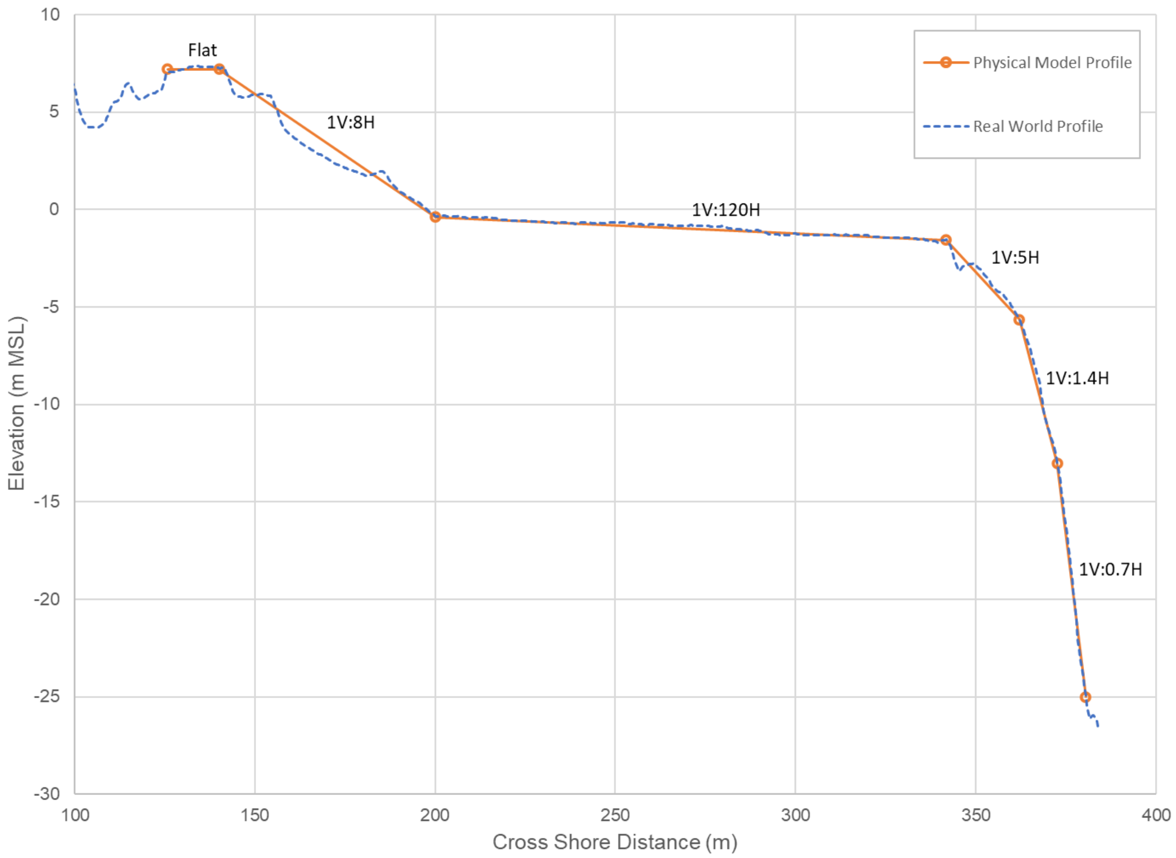

Figure 2.1: Physical model bathy-topo profile

2.3.3 Reef roughness

While it is acknowledged that wave breaking and dissipation processes are dominated by bathymetric profile/shape, bed roughness is also known to influence reef-top wave and water level processes in some cases, albeit to a much smaller magnitude (Buckley et. al 2016; Delgado et al. 2018; Yao et al. 2020). Bed roughness is often omitted from physical models of smooth-bottomed sandy bed sites, however, the reef platform at Nanumaga is somewhat rougher (Figure 2.3) and it was considered pertinent to represent the influence of roughness in the physical model to the best possible degree.

A quantitative analysis of the reef platform bed roughness was undertaken using the high resolution LiDAR elevation data captured in 2019. The analysis considered several representative cross-shore profiles of the reef in the area north of the navigation passage at the village. Figure 2.2 shows a typical transect across the reef where the contrast between the smooth sandy beach and the exposed reef flat is immediately apparent. The physical dimensions of the reef surface roughness were found to range as follows:

- Typical vertical height range of roughness projections: 0.05 – 0.25 m
- Typical horizontal span of roughness projections: 2 – 5 m

The size of the roughness elements is significantly larger than the spatial resolution of the LiDAR point cloud (approximately 13 points per m^2), and was therefore considered to be a reasonable approximation. This was further compared visually with imagery from the reef flat area, and also found to be consistent (Figure 2.3).

Hughes (1993) considered the scaling of boundary layers and surface roughness in short wave (swell/wind wave) physical models, and concluded that for turbulent boundary layer conditions under breaking waves (such as the wave breaking zone on a reef platform), roughness elements should be scaled according to the model length scale ratio (L_R). This scaling approach was adopted for the reef-top bed roughness elements in the Nanumaga physical model. Irregularly shaped and spaced stones were affixed to the base bathymetry across the reef-top area of the model, with the typical vertical height projection of the stones being approximately 8-11 mm model scale (0.20 – 0.28 m real world scale), and the horizontal span of roughness elements being approximately 125 – 210 mm model scale (3.1 – 5.2 m real world scale). The upper surface of the roughness elements was aligned with the target bathymetry profile as shown in Figure 2.1.

On the upper section of the fore-reef slope (1V:5H slope section), the roughness elements were arranged into a “spur-and-groove” formation, which is visible in satellite imagery of the fore-reef at Nanumaga. The size of the spur-and-groove features was measured from the LiDAR data set and approximated in the physical model. Figure 2.4 shows photographs of the roughness on the reef-top and fore-reef slope of the model.

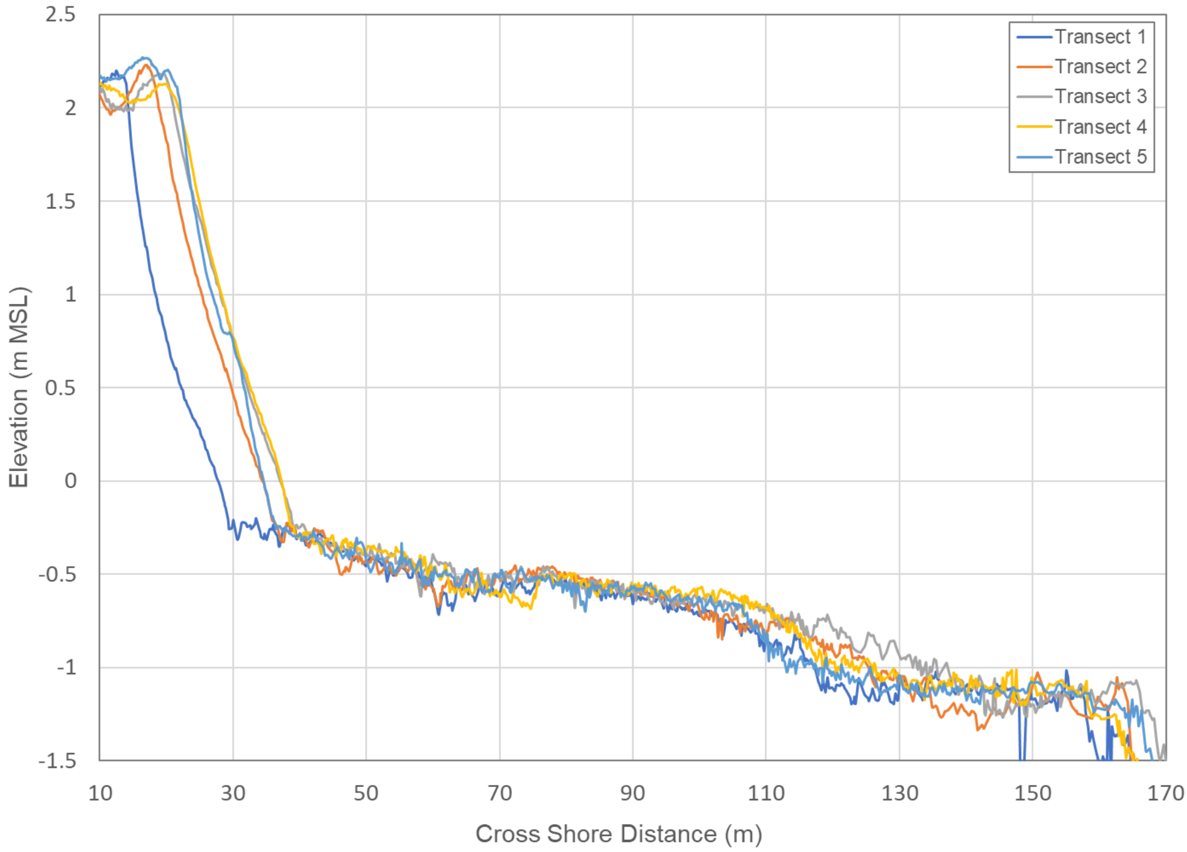

Figure 2.2: Transects of Nanumaga reef-top

Figure 2.3: Nanumaga reef platform at low tide (photo: Arthur Webb)

Figure 2.4: Model roughness elements on reef platform (top) and upper fore-reef

2.4 Measurements and analysis

2.4.1 Wave and water level measurements

Wave conditions and water levels were measured continuously throughout all tests at several locations within the flume, namely just offshore of the reef and at six other individual locations across the reef profile. Measurements were collected using high-accuracy capacitance wave gauges sampled at a frequency of 5 Hz (real world scale). Figure 2.5 shows the layout of wave gauges within the flume. The offshore wave conditions measured at the -25 m MSL location were collected using an array of three carefully spaced wave gauges, which enabled separation of incident and reflected wave time series using the least squares method of Mansard and Funke (1980). This was particularly important during the wave climate calibration process, to ensure that the incident wave climate matched the target wave climate for each test condition, without the interference of wave reflections.

Wave data recorded at each location was analysed using both spectral and zero crossing analysis, to generate a number of wave statistics for each test run. Table 2.1 provides a summary of each wave and water level parameter presented in this report, however, a much larger suite of parameters are available in the full suite of testing results. Due to the generation of low-frequency infragravity waves on the reef-top, the wave analysis considered the infragravity (IG; $f<0.04$ Hz) and sea/swell (SS; $f>0.04$ Hz) energy components separately. Recorded timeseries data for each test has also been provided as an appendix to this report.

Table 2.1 Summary of measured wave and water level parameters

Parameter	Description
$H_{s,25}$ (m)	Significant wave height at -25 m depth
$T_{p,25}$ (s)	Spectral peak wave period measured at -25 m depth
$H_{s,ss}$ (m)	Significant wave height at reef-top locations, including only SS component
$\bar{\eta}$ (m)	Average wave setup height (above tide + MSLA)
h (m MSL)	Total average water level (tide + MSLA + $\bar{\eta}$)
$\eta_{50,IG}$ (m)	Infragravity water level height (above tide + MSLA) exceeded 50% of time
$\eta_{2,IG}$ (m)	Infragravity water level height (above tide + MSLA) exceeded 2% of time
$h_{2,IG}$ (m MSL)	Total water level exceeded by infragravity wave 2% of time (tide + MSLA + $\eta_{2,IG}$)

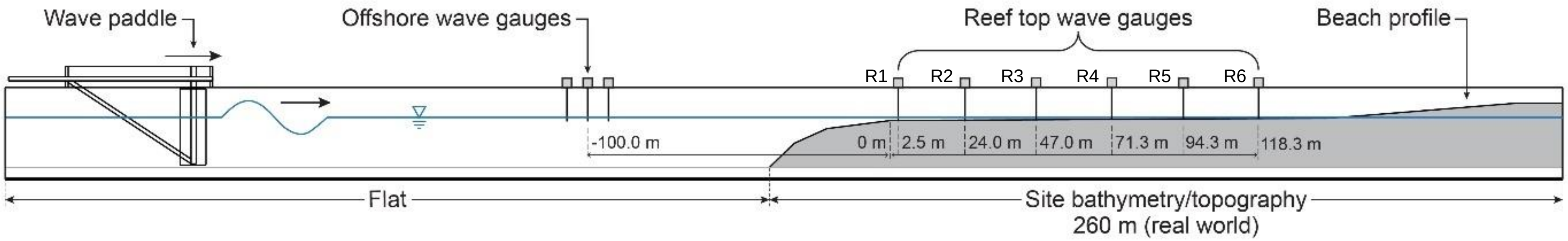

Figure 2.5: Layout of wave/water level gauges within flume

2.4.2 Runup/Overtopping measurements

Runup levels were measured continuously throughout all tests using a 2D LiDAR mounted over the beach, as well as through visual analysis using an overhead camera. While the intention was to process the runup data to determine the 2% runup level, extreme wave runups exceeded the crest of the beach berm in all tests. Wave overtopping of the beach berm crest was also measured for all tests, and this parameter was used instead as the measure of inundation intensity. Figure 2.6 shows the arrangement of instrumentation for measuring runup/overtopping in the wave flume.

Average wave overtopping rates, Q_{ave} (L/s/m) were measured by recording the total volume of water to overtop the beach berm crest during a full test, then dividing this by the test duration and normalising it to the beach berm crest length of the model. Peak wave overtopping volumes, V_{max} (L/m) were also estimated by measuring the volume of water to overtop the beach berm crest during individual large wave overtopping occurrences. Both Q_{ave} and V_{max} are widely used within coastal engineering literature as a measure of wave overtopping intensity and hazard. Measured values from the Nanumaga physical model tests are discussed relative to guideline values from literature in Section 4.5 of the report.

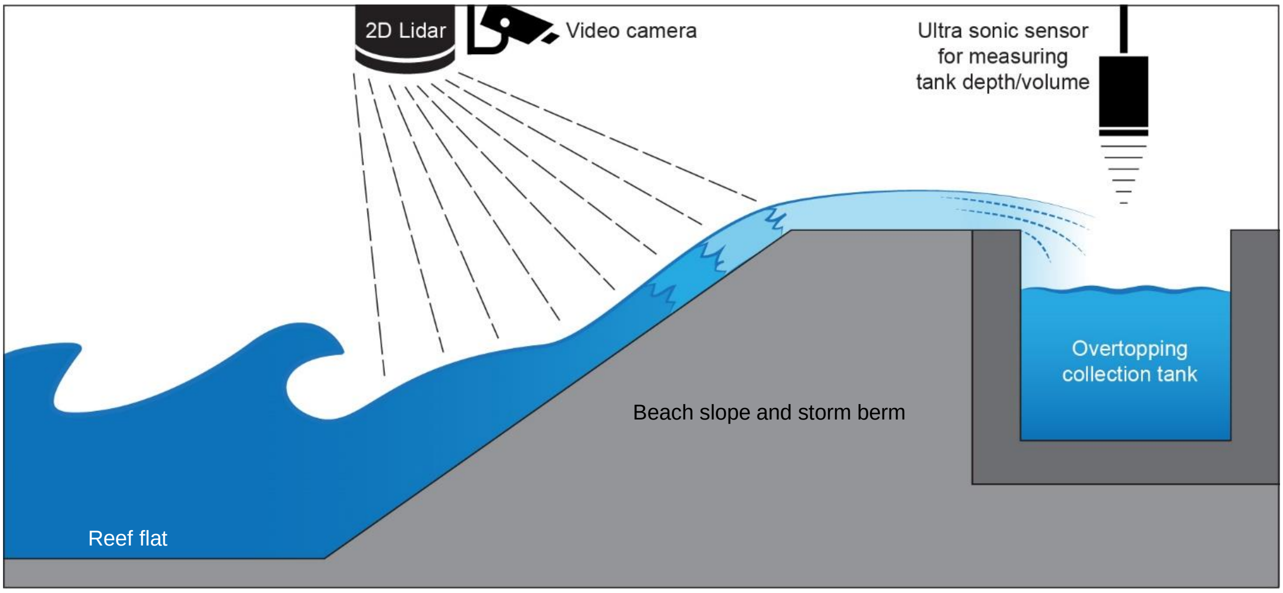

Figure 2.6: Layout of wave runup/overtopping instrumentation

3 Test program summary

The physical modelling program was divided into four stages as summarised in Table 3.1.

Table 3.1 Summary of test program stages

Modelling Stage	Description
Stage 1	Testing with high crest level beach profile (crest = 7.2 m MSL)
Stage 2	Testing with high crest level beach profile to explore sensitivity for: • reef resonance (alternative swell and infragravity wave periods) • wave groupiness and wave spectral shape
Stage 3	Testing with low crest level beach profile (crest = 6.3 m MSL)
Stage 4	Testing with low crest level beach profile and with BTB installed

In total, 20 different tests were completed as part of the formal testing program, with results for each test presented in Section 4.

The testing program primarily investigated five different 'baseline' test scenarios, each with a unique combination of wave and water level conditions. The target conditions were specified by SPC as "offshore" conditions representative of open ocean deep water conditions. As the physical model has a seaward boundary at the -25 m MSL contour (see Section 2.3.2 for further description), it was initially required to transform these "offshore" wave conditions to the -25 m MSL boundary location. This transformation was undertaken using the 1D numerical model SBEACH operated with a fixed/hard bottom. This model is based on the empirical formulation of Dally, Dean and Dalrymple (1984), and has previously been verified by WRL for use in locations with steep reef-type bathymetry profiles. Table 3.2 provides a summary of the target "offshore" test conditions provided by SPC, as well as the target conditions transferred to the -25 m MSL physical model boundary location.

The target test conditions were simulated in the model on the basis of a standard JONSWAP wave spectrum (with the exception of the sensitivity tests in modelling Stage 2). The target wave parameters H_s and T_p were used to generate each test wave sequence, which consisted of an irregular wave time series with a duration of approximately 1,000 waves.

Table 3.2 Target test conditions

Test Condition	H_s (offshore) (m)	H_s (@-25 m MSL)[1] (m)	T_p (s)	Water Level[2] (m MSL)
TC Pam Peak	5.3	5.1	15.6	0.39
100 Year ARI	5.5	5.2	14.3	0.67
250 Year ARI	6.0	5.7	14.8	0.65
250 Year ARI (2050)[3]	6.0	5.7	14.8	0.97
250 Year ARI (2100)[3]	6.0	5.7	14.8	1.49

1. Value determined by wave transformation calculation using SBEACH
2. Water level includes tide plus mean sea level anomaly (tide + MSLA)
3. Values for RCP8.5 adopted from SPC (2021)

All five test scenarios were calibrated in the wave flume prior to beginning the formal test program. The calibration process involved running each test sequence, recording wave data, and checking wave statistics against the target test parameters. Slight adjustments to the wave amplitude were then made and the sequence re-ran and new data recorded. This process is repeated until the measured wave conditions within the flume were an accurate match for the target conditions (typically required running each sequence two to five times). For the wave calibration process, the incident wave climate measured using the three-gauge array located at -25 m MSL (with reflected waves removed), was matched to the target wave conditions as summarised in Table 3.2. To further reduce the influence of reflected waves within the flume, the wave climate calibration process was undertaken prior to installation of the reef profile and with only a mild sloping dissipative beach installed at the far end of the flume.

4 Modelling results

4.1 Stage 1: Testing with high beach crest level

During Stage 1 of the testing program, measurements focussed on defining wave dissipation, wave setup and wave runup/overtopping for each of the five wave and water level conditions. Testing was undertaken with the beach profile extending up to a crest elevation of 7.2 m MSL. Test results for wave measurements are shown in Table 4.1 and Figure 4.1. (see Figure 2.5 for locations of reef-top wave measurement locations R1 to R6). Test results for water levels, infragravity waves and overtopping are shown in Table 4.2, Figure 4.2 and Figure 4.4.

Table 4.1 Measured wave conditions for Stage 1 (7.2 m MSL beach berm crest level)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$H_{s,ss}$ (m)					
				R1	R2	R3	R4	R5	R6
TC Pam	0.39	15.5	5.0	4.6	3.4	2.7	2.3	1.8	1.8
100 Yr ARI	0.67	14.1	5.2	4.9	3.4	2.8	2.3	1.9	1.8
250 Yr ARI	0.65	14.7	5.6	5.1	3.7	3.0	2.5	2.1	2.0
250 Yr ARI (2050)	0.97	14.7	5.7	5.3	3.8	3.1	2.6	2.2	2.1
250 Yr ARI (2100)	1.49	14.7	5.8	5.6	4.1	3.4	2.8	2.4	2.3

It can be seen in Figure 4.1 and Figure 4.3 that waves approaching the reef broke on the reef rim but maintained almost their full height up to the reef edge. Significant wave dissipation then occurred across the outer reef flat, resulting in rapid decay in wave height, with dissipated waves almost reaching an equilibrium height by the time they were approximately 100 m inside of the reef edge. At the most landward measurement location (118 m landward of the reef edge), significant wave heights were measured to be approximately 35% to 40% of the incident wave height outside of the reef, and this is expected to be a reasonable estimate of the significant wave height impacting the beach.

Average wave setup was measured to increase across the outer 100 m of reef flat, reaching a maximum by the most landward wave gauge (Figure 4.2). The measured wave setup at the landward measurement location ranged from 1.2 to 1.35 m, resulting in total reef-top water levels of 1.75 to 1.9 m MSL for the tests with present day sea levels. Wave setup was generally found to decrease or stay similar with greater submergence of the reef flat (during SLR tests), nevertheless the total reef-top water levels were greater due to the sea level rise component. Total reef-top water levels nearest the shoreline for the 250 year ARI 2050 test scenario were measured to be 2.3 m MSL, increasing to 2.7 m MSL for the 2100 scenario. Analysis of the infragravity component on the reef-top showed that low frequency surges in water level will occasionally reach up to approximately 3.7 m MSL with present day sea level, and up to 4.4 m MSL with 2100 sea level.

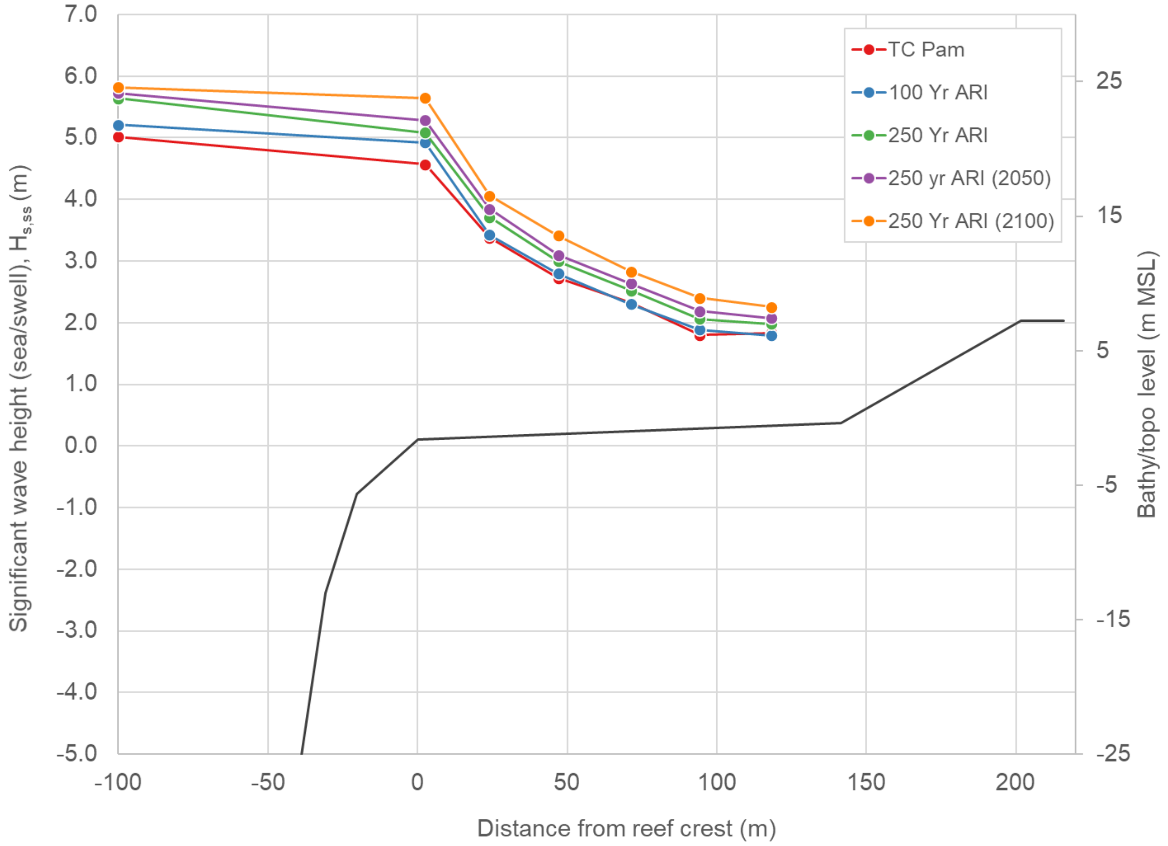

Figure 4.1: Variation in H_s across reef profile for Stage 1 testing

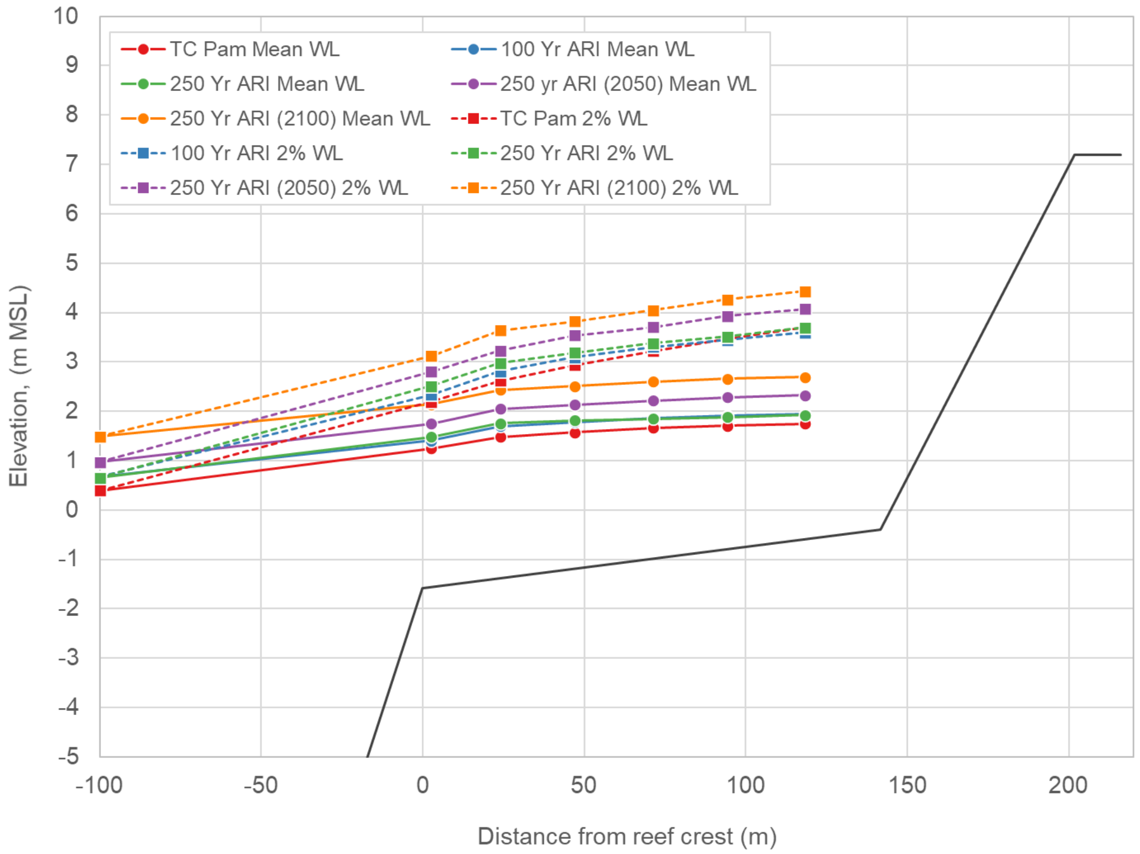

Figure 4.2: Variation in water levels across reef profile for Stage 1 testing

Figure 4.3: Wave breaking on fore-reef slope and dissipating across reef platform

Wave runup during large wave/infragravity occurrences was found to exceed the crest height of the beach berm at 7.2 m MSL for all test conditions, resulting in wave inundation/overtopping. The average wave overtopping rate measured 12.5 m landward of the crest of the beach berm was found to be smallest for the 100 Year ARI test condition with just over 1 L/s/m measured, increasing to just over 4 L/s/m for the 250 year ARI test condition. Sea level rise was found to further increase the measured overtopping rates, with over 9 L/s/m measured for the 250 Year ARI (2100) test scenario. This result clearly demonstrates the impact that sea level rise could have on inundation rates during extreme storm events, with measured wave overtopping for the 2100 scenario more than double the measured rate for present day sea levels.

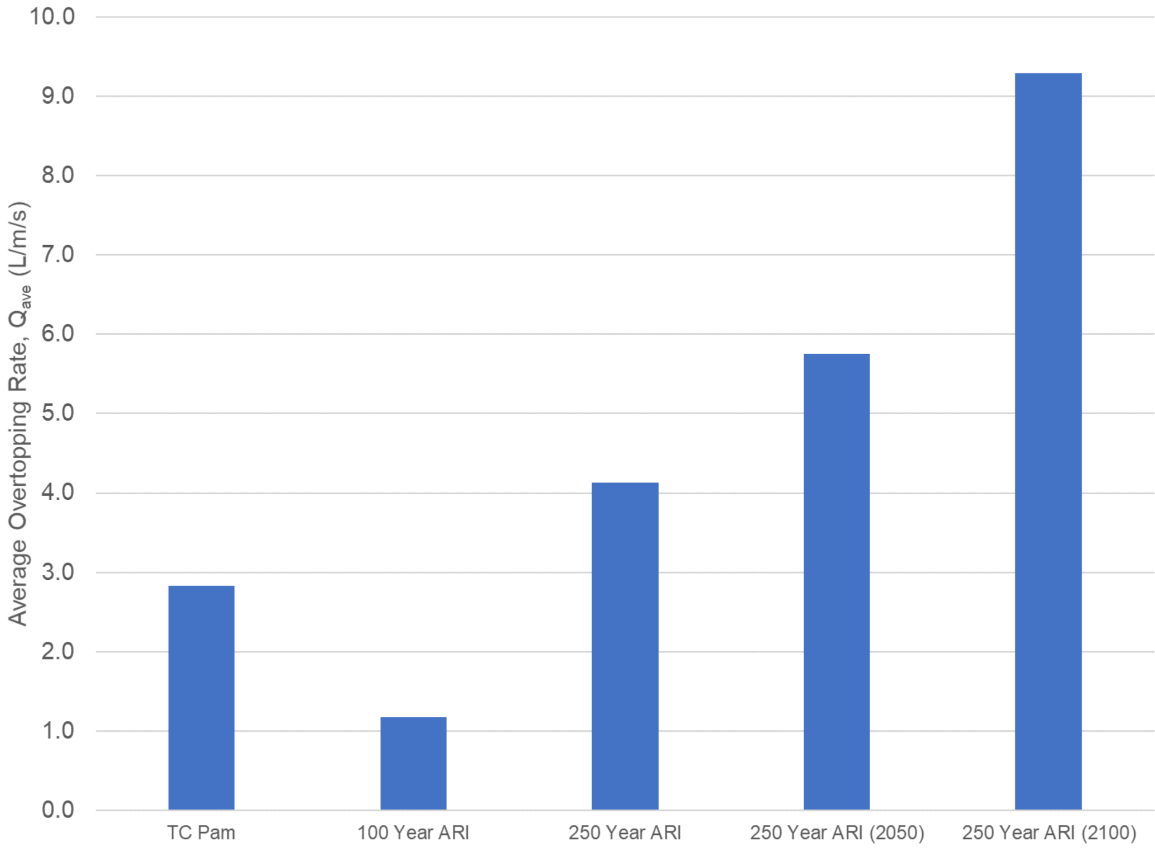

Figure 4.4: Average wave overtopping rates for Stage 1 testing (7.2 m MSL beach berm crest level)

Table 4.2 Measured water levels, Infragravity waves and overtopping for Stage 1 (7.2 m MSL beach berm crest level)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\bar{\eta}$ (m)						h (m MSL)					
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6
TC Pam	0.39	15.5	5.0	0.86	1.09	1.18	1.27	1.31	1.36	1.25	1.48	1.57	1.66	1.70	1.75
100 Yr ARI	0.67	14.1	5.2	0.73	1.02	1.11	1.19	1.24	1.27	1.40	1.69	1.78	1.86	1.91	1.94
250 Yr ARI	0.65	14.7	5.6	0.83	1.10	1.17	1.19	1.23	1.27	1.48	1.75	1.82	1.84	1.88	1.92
250 Yr ARI (2050)	0.97	14.7	5.7	0.78	1.08	1.17	1.25	1.31	1.35	1.75	2.05	2.14	2.22	2.28	2.32
250 Yr ARI (2100)	1.49	14.7	5.8	0.65	0.95	1.02	1.11	1.17	1.21	2.14	2.44	2.51	2.60	2.66	2.70

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\eta_{2,IG}$ (m)						$h_{2,IG}$ (m MSL)						Ave. Overtopping, Q_{ave} (L/s/m)
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6	
TC Pam	0.39	15.5	5.0	1.80	2.23	2.54	2.83	3.07	3.30	2.19	2.62	2.93	3.22	3.46	3.69	2.8
100 Yr ARI	0.67	14.1	5.2	1.67	2.14	2.43	2.63	2.77	2.92	2.34	2.81	3.10	3.30	3.44	3.59	1.2
250 Yr ARI	0.65	14.7	5.6	1.86	2.33	2.53	2.73	2.86	3.05	2.51	2.98	3.18	3.38	3.51	3.70	4.1
250 Yr ARI (2050)	0.97	14.7	5.7	1.83	2.26	2.57	2.74	2.96	3.10	2.80	3.23	3.54	3.71	3.93	4.07	5.8
250 Yr ARI (2100)	1.49	14.7	5.8	1.63	2.15	2.33	2.56	2.77	2.94	3.12	3.64	3.82	4.05	4.26	4.43	9.3

4.2 Stage 2: Sensitivity testing with high beach crest level

Stage 2 of the modelling program explored the sensitivity of reef-top waves, water levels and runup/overtopping to two incident wave climate parameters:

1. Resonant mode excitation or sensitivity of reef-top processes to alternative spectral peak wave periods.
2. Resonant mode excitation or sensitivity of reef-top processes to wave groupiness.

Excitation of resonant modes on fringing coral reef platforms has previously been noted to occur, in particular during large wave events, and to result in significant oscillations in reef-top water levels (Pe´quignet et al., 2009). If excitation of a resonant mode results in an antinode occurring at the shoreline, then additional wave runup/overtopping can be expected. Under typical wave conditions and reef submergence, resonant modes for reef platforms are typically associated with waves having a period of the order of tens of minutes. However, during large wave events the additional storm surge and wave setup experienced across reef platforms results in greater reef submergence, and therefore a reduced resonant mode period. It then becomes possible that waves in the infragravity frequency band, or grouping of waves in the sea-swell frequency band, can generate excitation of a resonant mode for a given reef platform width and submergence.

The potential for resonance on the reef platform at Nanumaga was investigated through a range of tests, firstly with varying spectral peak wave period, and secondly with varying wave groupiness parameter and spectral shape. All sensitivity tests were undertaken using the target wave height and water level for the 100 year ARI test condition.

4.2.1 Sensitivity testing for wave period

Three separate tests were completed for conditions having target spectral peak wave periods (T_p) of 11 s, 13 s and 20 s, in addition to the baseline 100 year ARI test condition that had a target T_p of 14.3 s. Test results for wave measurements are shown in Table 4.3 and Figure 4.5, while test results for water levels, infragravity waves and overtopping are shown in Table 4.4, Figure 4.6 and Figure 4.7.

Table 4.3 Measured wave conditions for Stage 2a (varying spectral peak wave period, 7.2 m MSL beach berm crest level)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$H_{s,ss}$ (m)					
				R1	R2	R3	R4	R5	R6
Sens. Test 1	0.67	10.7	5.2	4.8	2.9	2.3	1.9	1.6	1.5
Sens. Test 2	0.67	13.0	5.4	4.9	3.4	2.7	2.3	1.8	1.8
Sens. Test 3	0.67	14.1	5.2	4.9	3.4	2.8	2.3	1.9	1.8
Sens. Test 4	0.67	19.8	6.4	5.0	3.8	3.2	2.6	2.1	2.2

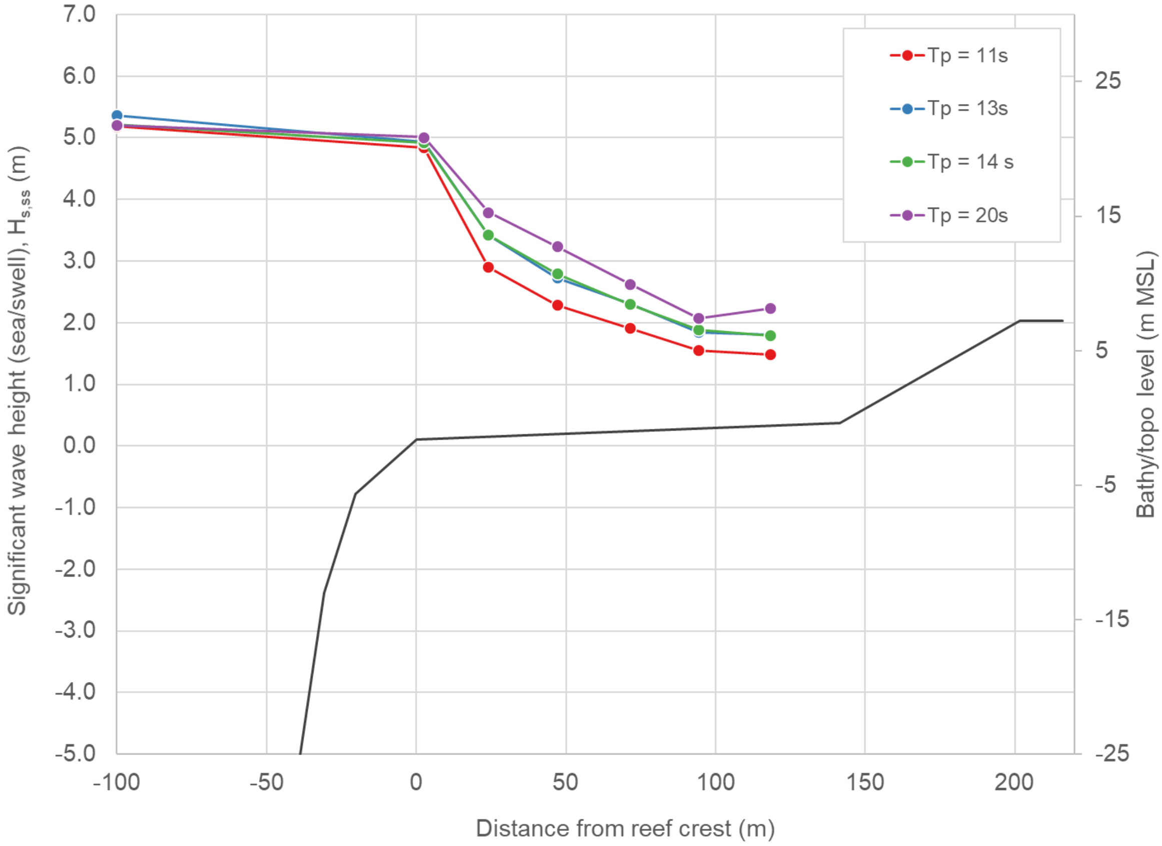

Figure 4.5: Influence of wave period on reef-top wave heights

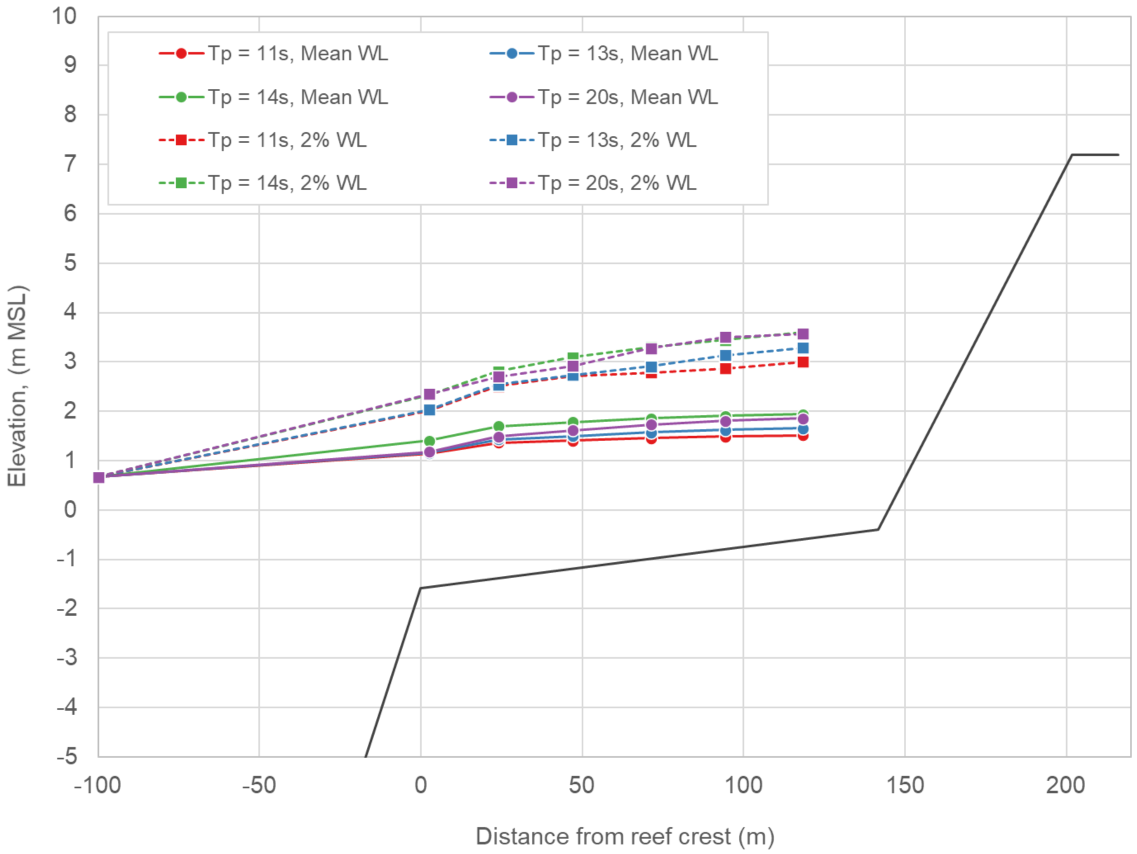

Figure 4.6: Influence of wave period on reef-top water levels

For each of the four wave period sensitivity tests, the wave height measured nearest the shoreline was found to increase with increasing wave period (Figure 4.5). The nearshore wave height varied from ~29% for a wave period of 11 seconds, up to ~43% for a wave period of 20 seconds, when expressed as a percentage of the offshore wave height. This indicates a significant reduction in the amount of wave energy dissipated across the reef flat for longer-period wave conditions.

Reef-top water levels also showed a sensitivity to wave period, with longer wave periods tending to result in more wave setup and infragravity energy (Figure 4.6). Nevertheless, there was very little difference in wave setup and extreme water levels measured between the two tests with longer wave period of ~14 s and ~20 s.

While there was some scatter in overtopping results (Figure 4.7), it was very evident from both observations and recorded data, that the average wave overtopping rate was significantly higher for the test conditions with the longer 20 second spectral peak wave period, compared with wave periods of 14 seconds and shorter. This is expected to be the result of the higher infragravity levels and nearshore wave heights reaching the beach during the longer period test conditions. The order-of-magnitude increase in overtopping rate between wave conditions with 14 second and 20 second wave period, indicates that the occurrence of wave-driven inundation for Nanumaga will be highly sensitive to the period of storm wave conditions, and the island particularly vulnerable during very long swell wave conditions.

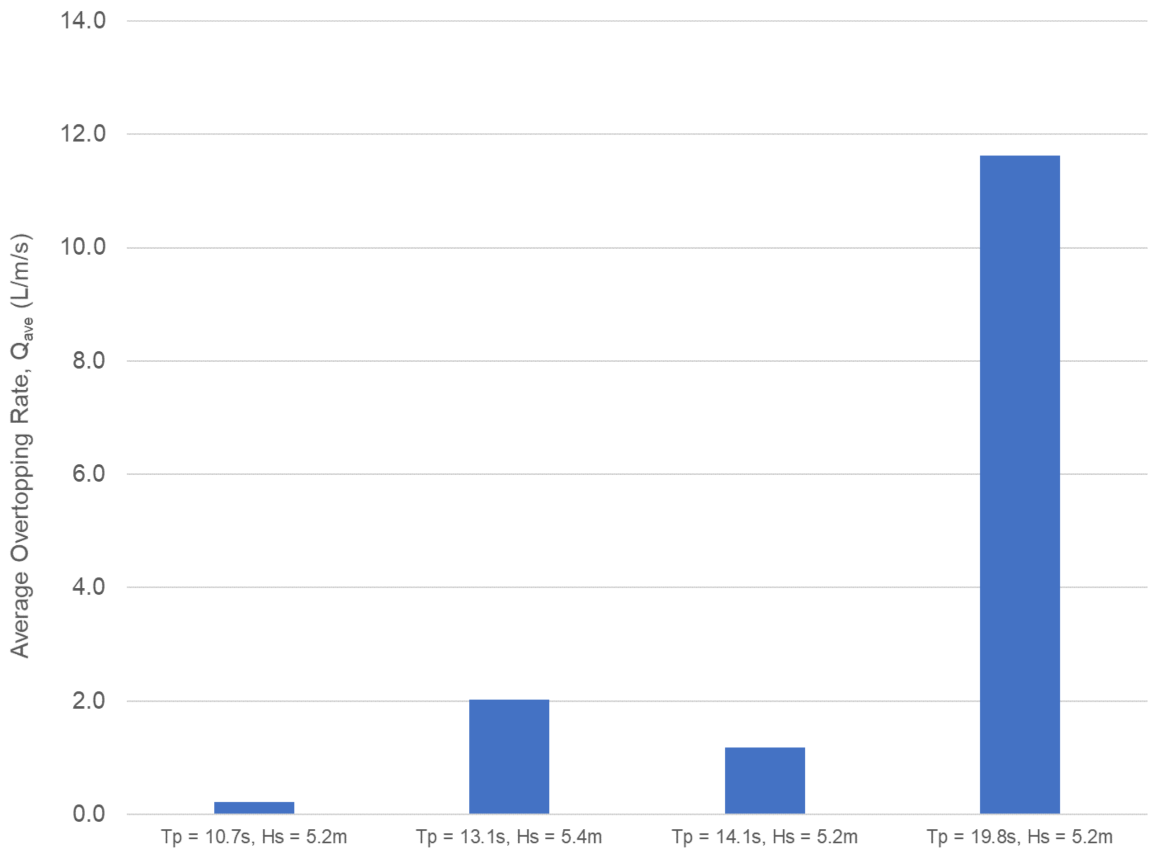

Figure 4.7: Influence of wave period on wave overtopping rates

Table 4.4 Measured water levels, Infragravity waves and overtopping for Stage 2a (Sensitivity testing for wave period, 7.2 m MSL beach berm crest level)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\bar{\eta}$ (m)						h (m MSL)					
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6
Sens. Test 1	0.67	10.7	5.2	0.48	0.70	0.73	0.78	0.82	0.84	1.15	1.37	1.40	1.45	1.49	1.51
Sens. Test 2	0.67	13.0	5.4	0.48	0.76	0.83	0.90	0.96	0.99	1.15	1.43	1.50	1.57	1.63	1.66
Sens. Test 3	0.67	14.1	5.2	0.73	1.02	1.11	1.19	1.24	1.27	1.40	1.69	1.78	1.86	1.91	1.94
Sens. Test 4	0.67	19.8	6.4	0.51	0.82	0.94	1.06	1.14	1.19	1.18	1.49	1.61	1.73	1.81	1.86

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\eta_{2,IG}$ (m)						$h_{2,IG}$ (m MSL)						Ave. Overtopping, Q_{ave} (L/s/m)
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6	
Sens. Test 1	0.67	10.7	5.2	1.34	1.84	2.05	2.12	2.20	2.33	2.01	2.51	2.72	2.79	2.87	3.00	0.2
Sens. Test 2	0.67	13.0	5.4	1.37	1.87	2.06	2.24	2.47	2.61	2.04	2.54	2.73	2.91	3.14	3.28	2.0
Sens. Test 3	0.67	14.1	5.2	1.67	2.14	2.43	2.63	2.77	2.92	2.34	2.81	3.10	3.30	3.44	3.59	1.2
Sens. Test 4	0.67	19.8	6.4	1.68	2.03	2.25	2.61	2.83	2.90	2.35	2.70	2.92	3.28	3.50	3.57	11.6

4.2.2 Sensitivity testing with for wave spectrum and groupiness

Three separate tests were completed for wave conditions having the same spectral peak wave period (T_p) and significant wave height (H_s), but varying wave groupiness and spectral shape. For the purpose of reporting the specific changes to the test wave conditions, the groupiness was adjusted via the Groupiness Parameter (κ) and the spectral shape by the spectral Peakedness Factor (γ). The Groupiness Parameter is a measure of wave grouping defined as follows (Battjes and van Vledder, 1984):

- $\kappa = \frac{1}{m_0}\left| \int_0^{\infty} E(f)\, e^{i2\pi f\tau} df \right.$

Where τ denotes the mean time lag separating two successive waves. The baseline test wave sequence for this modelling program had a $\kappa \sim 0.52$, and for the sensitivity testing two additional wave series with $\kappa \sim 0.44$ and $\kappa \sim 0.61$ were also explored.

The Peakedness Factor, γ, is a descriptor of the spectral shape of a given wave condition, and determines the concentration of the spectrum about the peak frequency. For the baseline test conditions completed in this modelling program, test wave sequences had a spectral Peakedness Factor, $\gamma \sim 3.3$ (standard Jonswap spectral shape). For the sensitivity testing the additional two wave series had $\gamma \sim 1.0$ and $\gamma \sim 6.0$. The measured incident wave spectrum offshore of the reef profile for the three different sensitivity tests are shown in Figure 4.8.

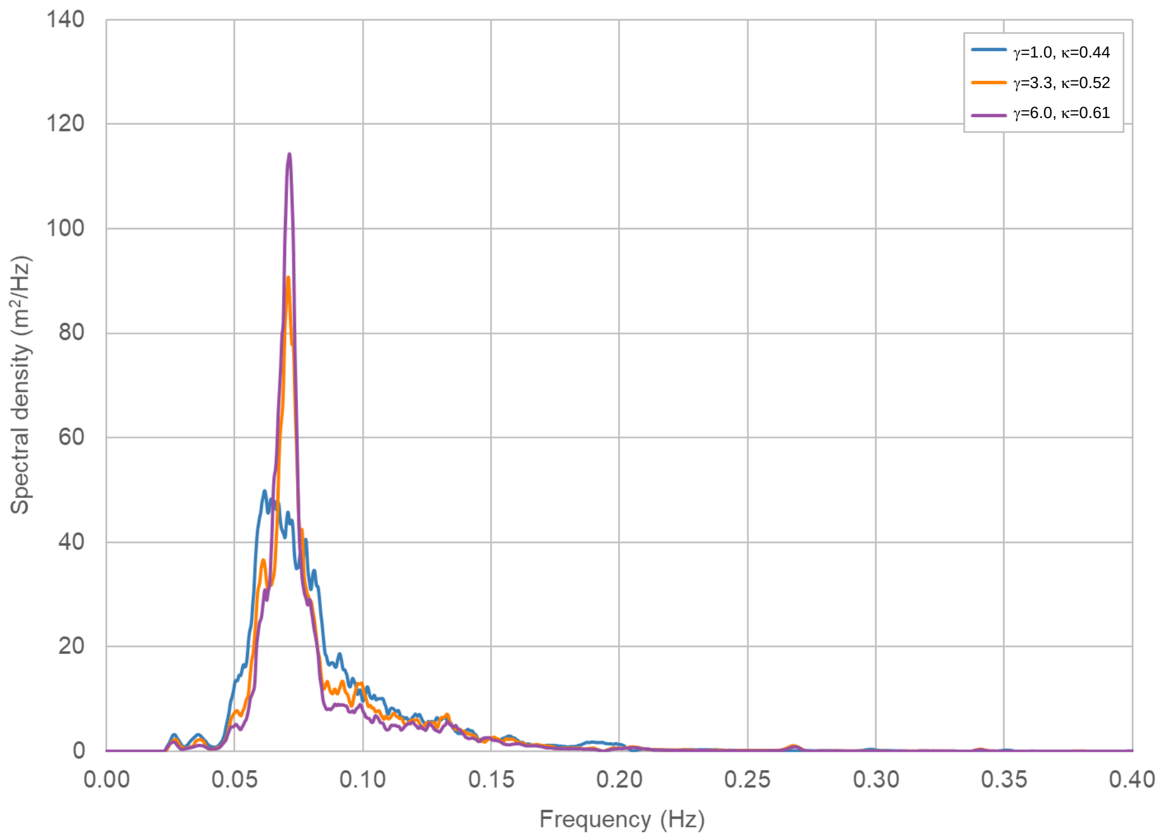

Figure 4.8: Measured wave spectral shape for sensitivity tests

Test results for wave measurements are shown in Table 4.5 and Figure 4.9, while test results for water levels, infragravity waves and overtopping are shown in Table 4.6, Figure 4.10 and Figure 4.11.

For each of the three sensitivity tests, the wave height measured nearest the shoreline expressed as a percentage of the offshore wave height, showed no measurable variation with in response to differing wave groupiness and spectral shape (Figure 4.9). Average reef-top wave setup and water level also showed negligeable sensitivity to these parameters. However, extreme infragravity water levels tended to be higher for waves with higher groupiness (Figure 4.10). Wave overtopping rates were very similar for all three sensitivity tests, with a slight trend towards increasing overtopping rate with increasing wave groupiness, expected to be a result of the additional infragravity wave energy (Figure 4.11).

Table 4.5 Summary of measured wave conditions for Stage 2b (varying wave groupiness and spectrum, 7.2 m MSL beach berm crest level)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$H_{s,ss}$ (m)					
				R1	R2	R3	R4	R5	R6
Sens. Test 5	0.67	14.6	5.4	4.8	3.4	2.7	2.3	1.8	1.8
Sens. Test 6	0.67	14.1	5.2	4.9	3.4	2.8	2.3	1.9	1.8
Sens. Test 7	0.67	14.1	5.1	4.8	3.5	2.7	2.3	1.9	1.8

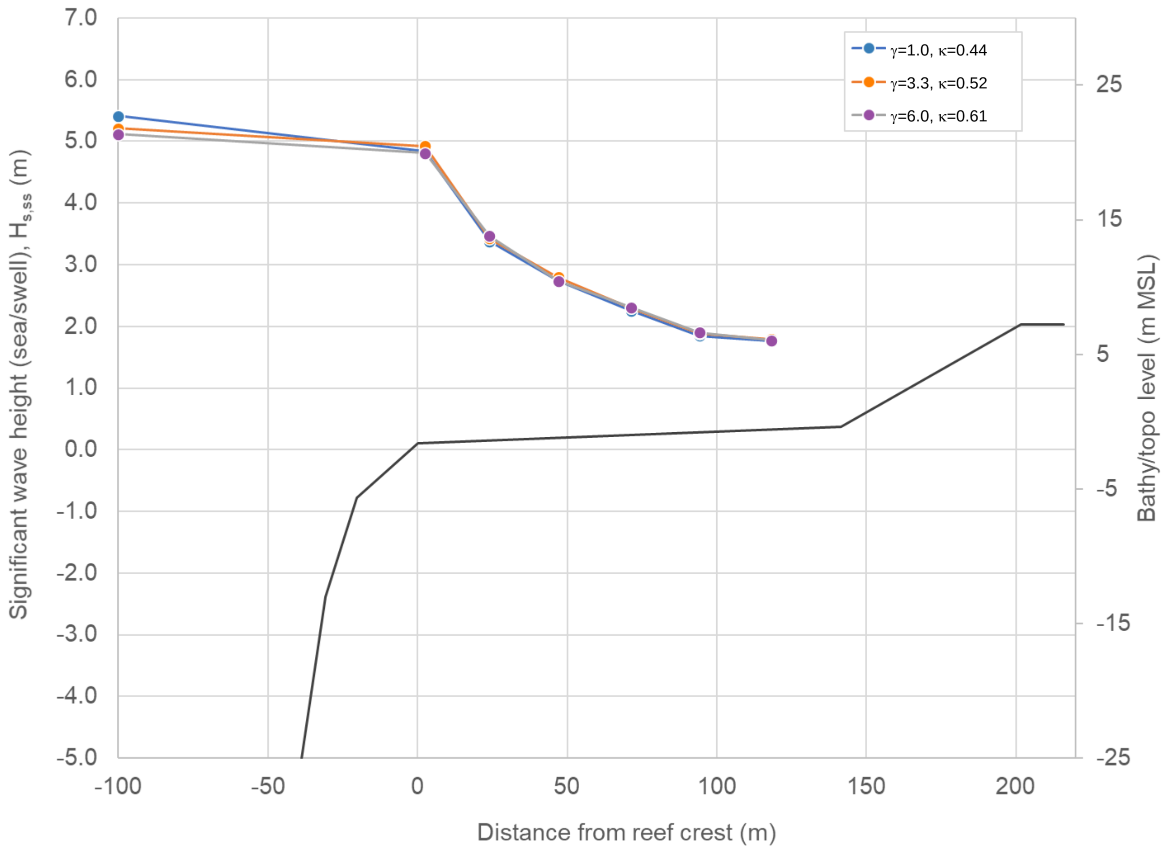

Figure 4.9: Influence of spectral shape and groupiness on reef-top wave heights

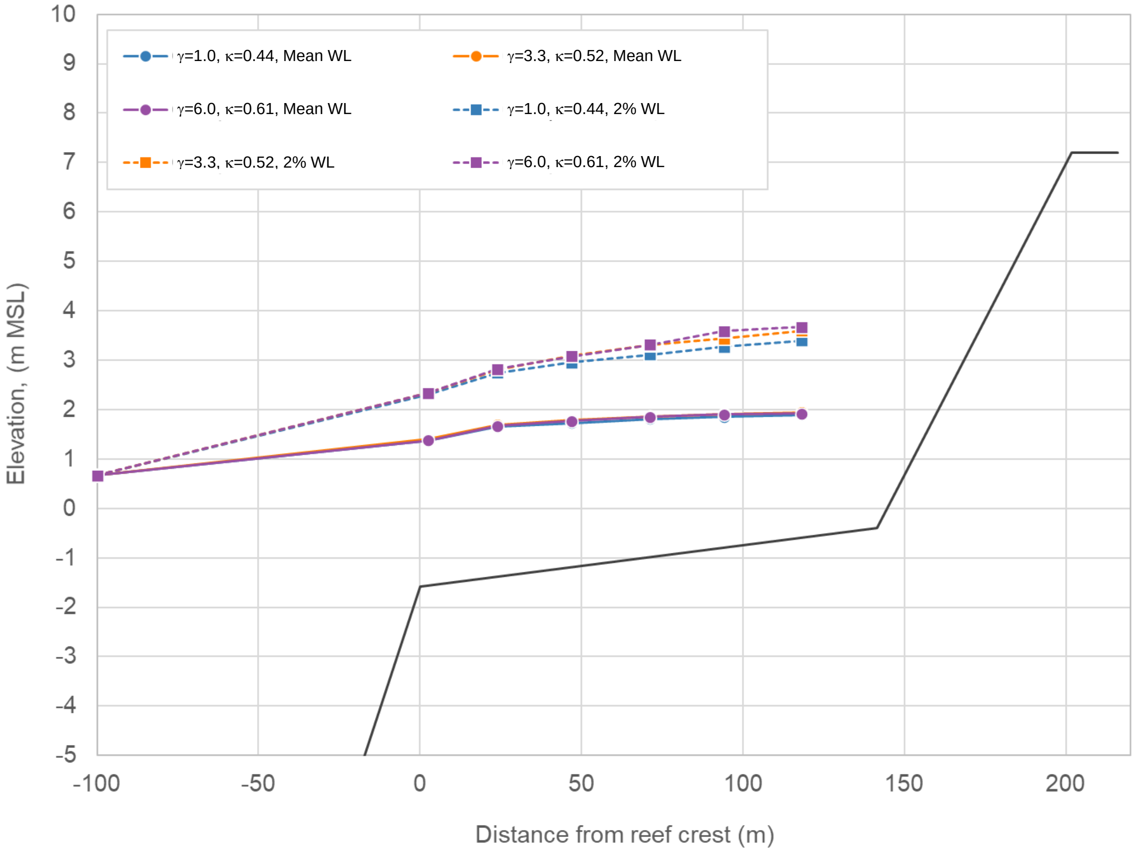

Figure 4.10: Influence of spectral shape and groupiness on reef-top water levels

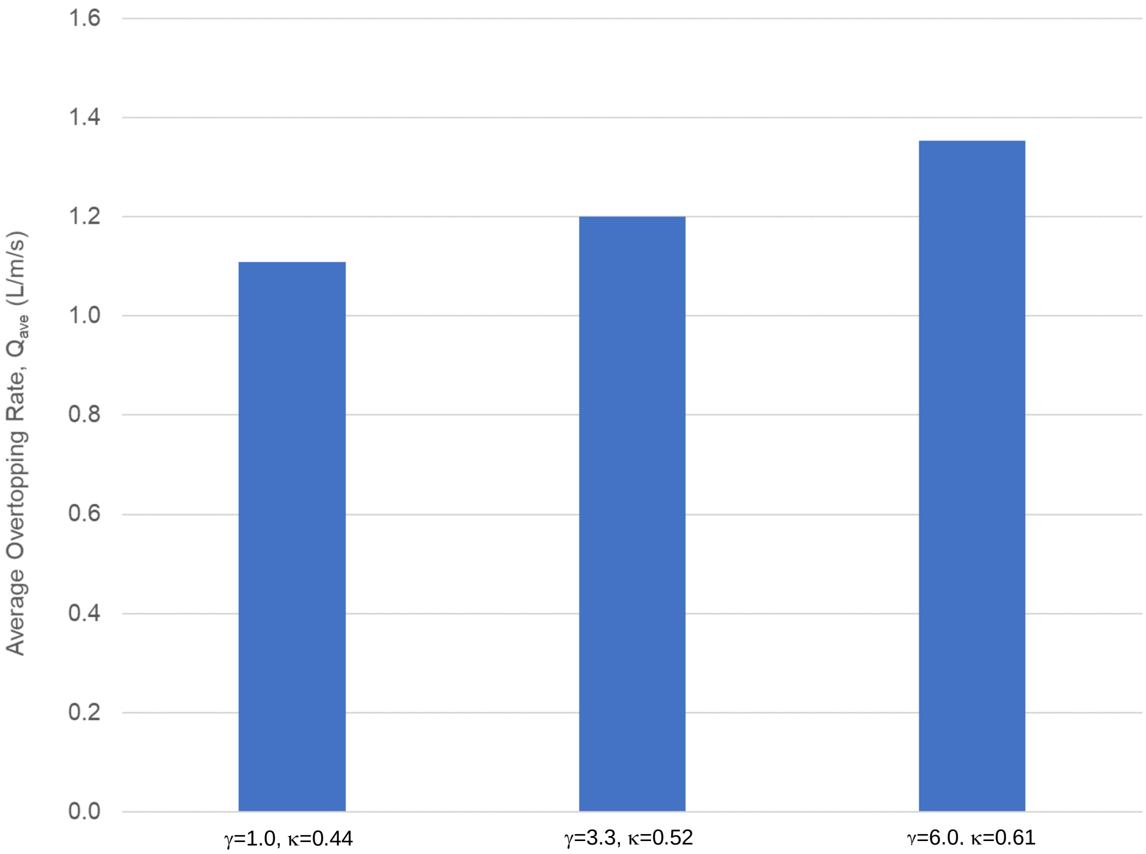

Figure 4.11: Influence of spectral shape and groupiness on wave overtopping rates

Table 4.6 Measured water levels, Infragravity waves and overtopping for Stage 2b (Sensitivity testing for spectral shape and groupiness, 7.2 m MSL beach berm crest level)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\bar{\eta}$ (m)						h (m MSL)					
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6
Sens. Test 5	0.67	14.6	5.4	0.70	0.99	1.06	1.14	1.19	1.22	1.37	1.66	1.73	1.81	1.86	1.89
Sens. Test 6	0.67	14.1	5.2	0.73	1.02	1.11	1.19	1.24	1.27	1.40	1.69	1.78	1.86	1.91	1.94
Sens. Test 7	0.67	14.1	5.1	0.71	1.00	1.10	1.18	1.23	1.25	1.38	1.67	1.77	1.85	1.90	1.92

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\eta_{2,IG}$ (m)						$h_{2,IG}$ (m MSL)						Ave. Overtopping, Q_{ave} (L/s/m)
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6	
Sens. Test 5	0.67	14.6	5.4	1.64	2.07	2.28	2.44	2.60	2.73	2.31	2.74	2.95	3.11	3.27	3.40	1.1
Sens. Test 6	0.67	14.1	5.2	1.67	2.14	2.43	2.63	2.77	2.92	2.34	2.81	3.10	3.30	3.44	3.59	1.2
Sens. Test 7	0.67	14.1	5.1	1.67	2.15	2.41	2.65	2.92	3.00	2.34	2.82	3.08	3.32	3.59	3.67	1.4

4.3 Stage 3: Testing with low beach crest level

Stage 3 of the modelling program investigated the influence of beach berm crest level on rates of wave overtopping/inundation. The beach in the model was adjusted by reducing the upper crest level from 7.2 m MSL down to 6.3 m MSL. This lower beach crest level is representative of several areas north and south of the reef navigation passage at Nanumaga. All five of the baseline test conditions were run in the flume, with Table 4.7 and Figure 4.12 showing the results for wave dissipation, Table 4.8, Figure 4.13 and Figure 4.14 showing the results for water levels and wave overtopping.

Table 4.7 Measured wave conditions for Stage 3 (6.3 m MSL beach berm crest level)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$H_{s,ss}$ (m)					
				R1	R2	R3	R4	R5	R6
TC Pam	0.39	15.5	5.0	4.6	3.5	2.7	2.3	1.8	1.8
100 Yr ARI	0.67	14.1	5.2	4.9	3.5	2.8	2.3	1.9	1.8
250 Yr ARI	0.65	14.7	5.7	5.1	3.7	3.0	2.5	2.1	2.0
250 Yr ARI (2050)	0.97	14.7	5.7	5.3	3.9	3.2	2.6	2.2	2.1
250 Yr ARI (2100)	1.49	14.7	5.8	5.7	4.2	3.4	2.9	2.4	2.3

It can be seen in Figure 4.12 and Figure 4.13, that measured waves and water levels were consistent with the results from the same tests completed in Stage 1 of the testing program (Figure 4.1 and Figure 4.2). This demonstrates a high degree of repeatability in the achieved wave conditions and associated results.

As expected, observations collected during the Stage 3 model test runs showed that waves overtopped the crest of the beach much more regularly with the lower beach crest level of 6.3 m MSL than with the higher beach crest level of 7.2 m MSL investigated in Stage 1 of the modelling program. This resulted in significantly higher measured wave overtopping rates across all test conditions, with overtopping rates 3.5 to 4.5 times greater with the lower beach crest level, as presented in Figure 4.14.

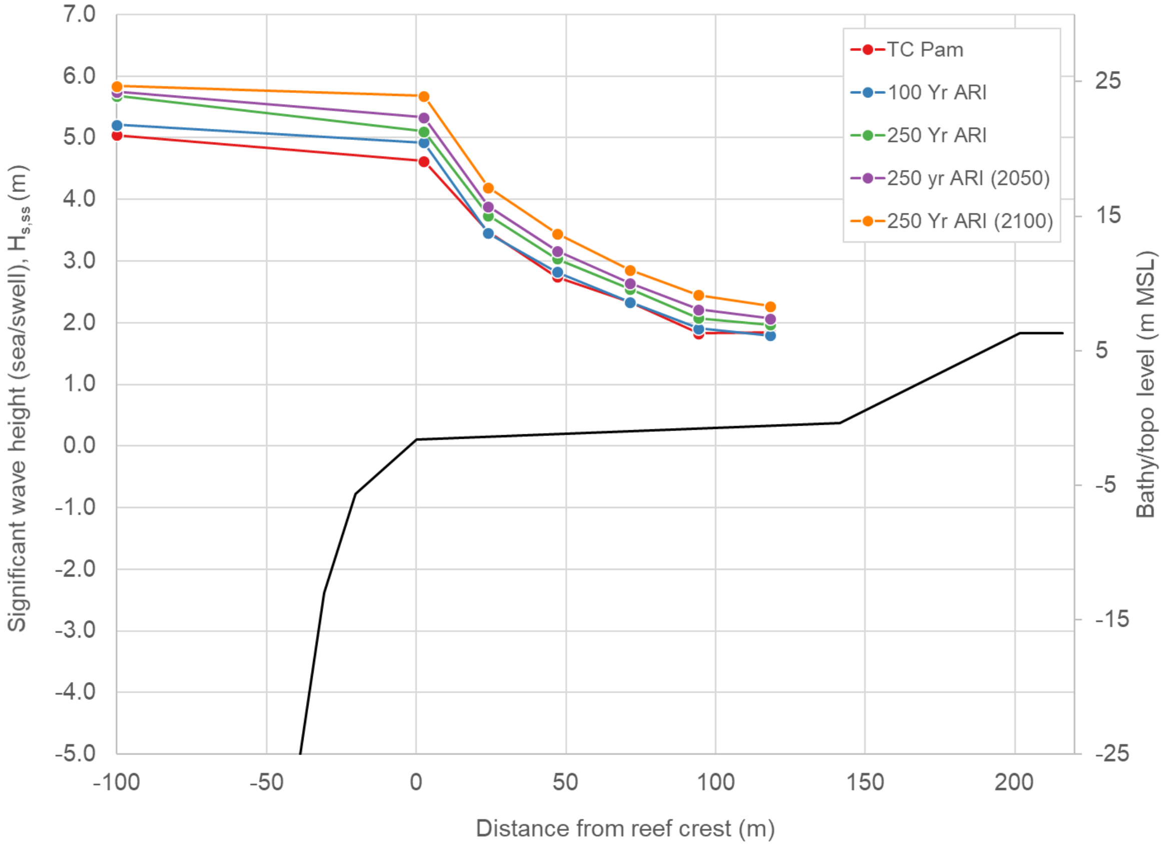

Figure 4.12: Variation in H_s across reef profile for Stage 3 testing

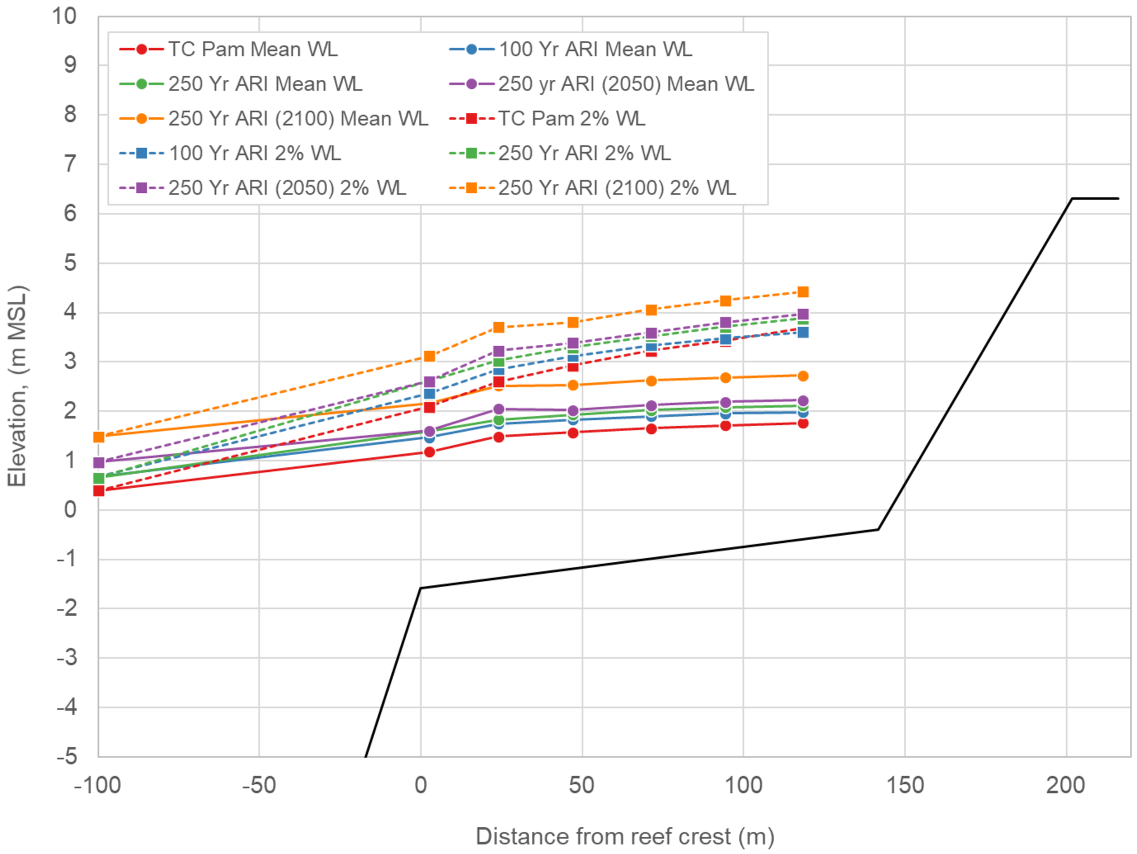

Figure 4.13: Variation in water level across reef profile for Stage 3 testing

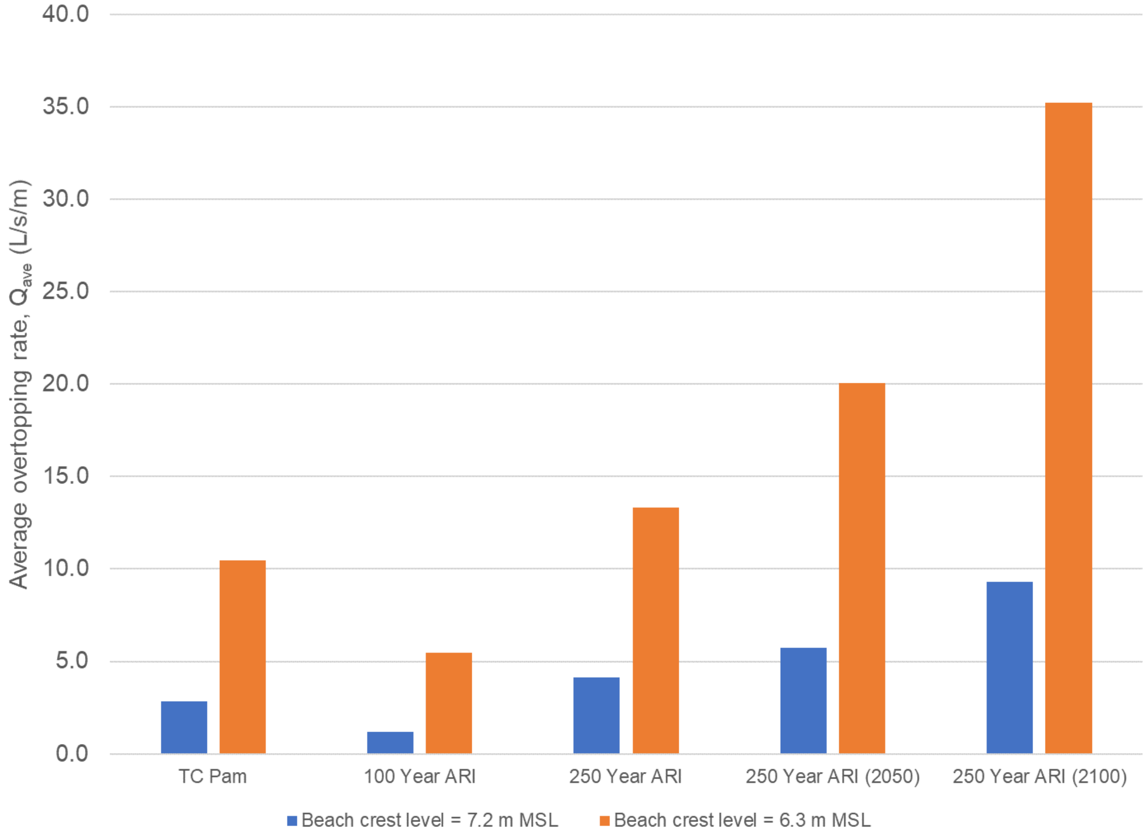

Figure 4.14: Average wave overtopping rates for Stage 3 testing (6.3 m MSL beach berm crest level)

Table 4.8 Measured water levels, Infragravity waves and overtopping for Stage 3 (6.3 m MSL beach berm crest level)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\bar{\eta}$ (m)						h (m MSL)					
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6
TC Pam	0.39	15.5	5.0	0.39	0.79	1.10	1.19	1.27	1.33	1.18	1.49	1.58	1.66	1.72	1.76
100 Yr ARI	0.67	14.1	5.2	0.67	0.80	1.08	1.16	1.23	1.29	1.47	1.75	1.83	1.90	1.96	1.99
250 Yr ARI	0.65	14.7	5.7	0.65	0.95	1.18	1.28	1.37	1.44	1.60	1.83	1.93	2.02	2.09	2.11
250 Yr ARI (2050)	0.97	14.7	5.7	0.97	0.64	1.08	1.06	1.15	1.22	1.61	2.05	2.03	2.12	2.19	2.22
250 Yr ARI (2100)	1.49	14.7	5.8	1.49	0.67	1.03	1.05	1.14	1.19	2.16	2.52	2.54	2.63	2.68	2.73

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\eta_{2,IG}$ (m)						$h_{2,IG}$ (m MSL)						Ave. Overtopping, Q_{ave} (L/s/m)
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6	
TC Pam	0.39	15.5	5.0	1.70	2.21	2.53	2.83	3.04	3.29	2.09	2.60	2.92	3.22	3.43	3.68	10.5
100 Yr ARI	0.67	14.1	5.2	1.69	2.18	2.45	2.67	2.81	2.93	2.36	2.85	3.12	3.34	3.48	3.60	5.5
250 Yr ARI	0.65	14.7	5.7	1.96	2.38	2.64	2.87	3.06	3.23	2.61	3.03	3.29	3.52	3.71	3.88	13.3
250 Yr ARI (2050)	0.97	14.7	5.7	1.63	2.26	2.42	2.63	2.84	3.00	2.60	3.23	3.39	3.60	3.81	3.97	20.1
250 Yr ARI (2100)	1.49	14.7	5.8	1.63	2.21	2.31	2.57	2.76	2.94	3.12	3.70	3.80	4.06	4.25	4.43	35.2

4.4 Stage 4: Testing with BTB installed

Stage 4 of the modelling program assessed the level of protection offered by the installation of a berm-top-barrier (BTB) on top of the natural storm berm at the crest of the beach. The BTB is a “nature based” approach for reducing inundation from wave overtopping that is proposed for Nanumaga as part of the TCAP. The approach increases the natural storm berm level at the crest of the beach with the installation of a series of continuous shore-parallel geotextile mega containers to form a core that is subsequently buried and revegetated to create a more natural barrier/bund. The addition of the BTB effectively increases the crest level of the natural storm berm by approximately 1.5 m. Figure 4.15 shows a cross section of the BTB design (Bluecoast, 2021).

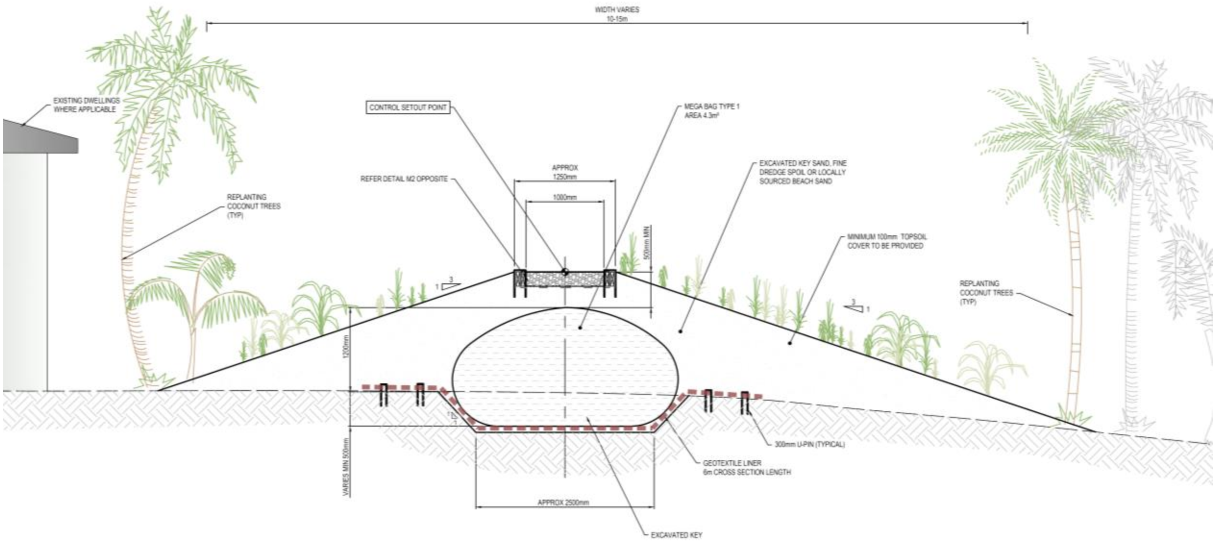

Figure 4.15: Conceptual BTB arrangement for Nanumaga

Within the physical model, the BTB was constructed to represent the northern BTB (GB01) at chainage 220 m, as per the draft design drawings (Bluecoast, 2021). The BTB was installed directly behind the crest of the beach within the model, with the specifications as shown in Figure 4.16. The model BTB was fabricated from impermeable plywood, and formed to the design finished barrier profile (i.e. 1V:3H side slopes and 2 m crest width). This model set up, including the full reef and beach profile was identical to that tested in Stage 3 of the modelling program, only with the addition of the BTB on the top of the beach crest.

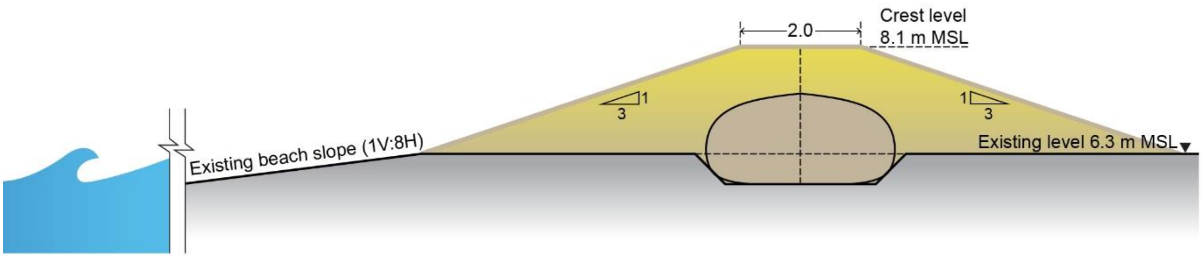

Figure 4.16: BTB as simulated in the physical model
(Design barrier GB01 at chainage 220 m adopted for the model design)

All five of the baseline test conditions were run in the flume with the BTB installed, with Table 4.9 and Figure 4.17 showing the results for wave dissipation across the reef-top, Table 4.10, Figure 4.18 and Figure 4.19 showing the results for water levels and wave overtopping.

Table 4.9 Measured wave conditions for Stage 4
(6.3 m MSL beach berm crest level, with additional BTB installed to crest level of 8.1 m MSL)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$H_{s,ss}$ (m)					
				R1	R2	R3	R4	R5	R6
TC Pam	0.39	15.5	5.1	4.6	3.5	2.8	2.3	1.8	1.8
100 Yr ARI	0.67	14.1	5.2	4.9	3.5	2.8	2.3	1.9	1.8
250 Yr ARI	0.65	14.7	5.7	5.1	3.8	3.0	2.5	2.0	2.0
250 Yr ARI (2050)	0.97	14.7	5.7	5.3	3.9	3.2	2.6	2.2	2.1
250 Yr ARI (2100)	1.49	14.7	5.8	5.7	4.1	3.4	2.9	2.4	2.3

It can be seen in Figure 4.17 and Figure 4.18, that measured waves and water levels were very similar to the results from the same tests completed in Stage 1 and Stage 3 of the testing program, confirming that the tested BTB design would not have a measurable effect on wave processes across the reef platform.

A significant reduction in both the frequency and intensity of wave overtopping was observed with the installation of the BTB. Both the Stage 3 and Stage 4 modelling had a natural beach storm berm crest level of 6.3 m MSL, however, Stage 4 modelling also had the addition of the BTB, effectively increasing the crest of the profile up to 8.1 m MSL. It can be seen in Figure 4.19 that the addition of the BTB resulted in a direct reduction in average wave overtopping rates by a factor of 7 to 14. Importantly, the BTB was shown to reduce future wave overtopping rates for 2100 sea levels, below the values for equivalent storm conditions with present day sea levels (i.e. the BTB not only offsets the additional overtopping expected to be caused by SLR, but further improves on it). It is important to note that these reductions in overtopping will only occur if the crest of the BTB maintains its design profile and is not scoured by wave overtopping.

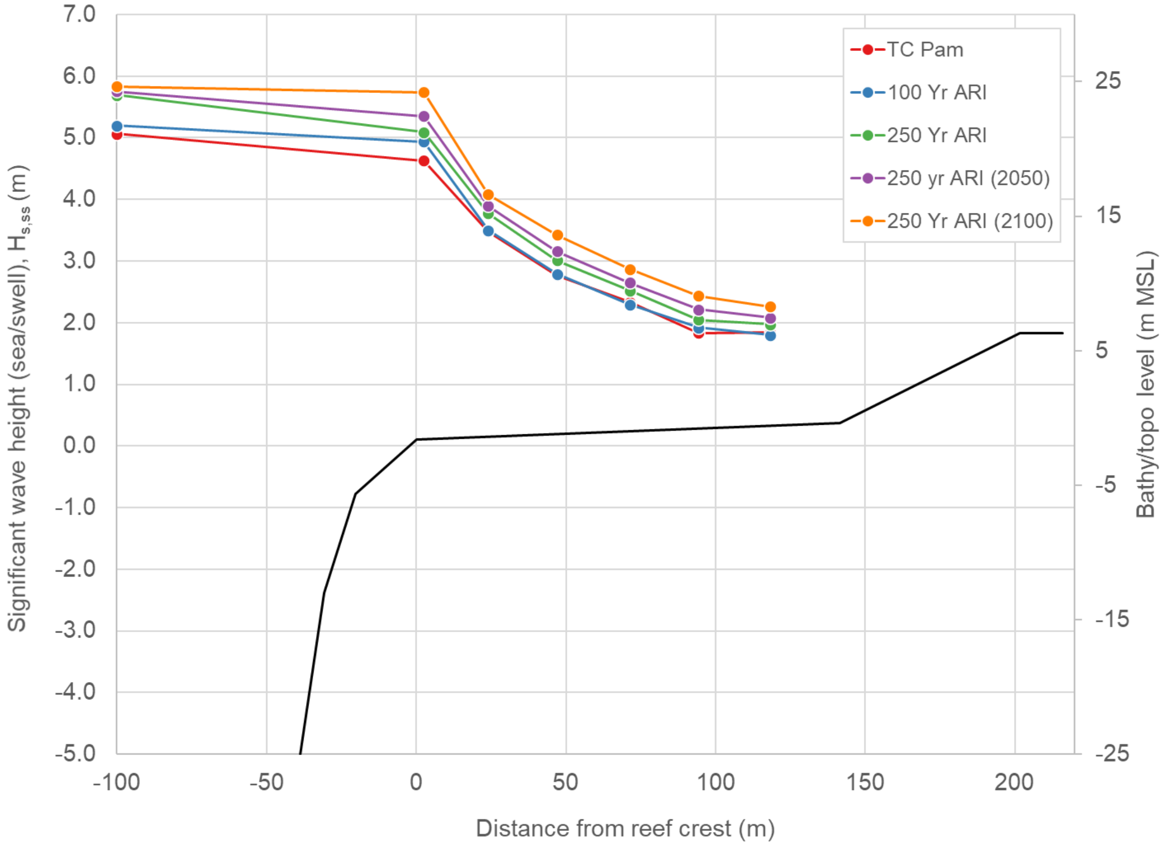

Figure 4.17: Variation in H_s across reef profile for Stage 4 testing

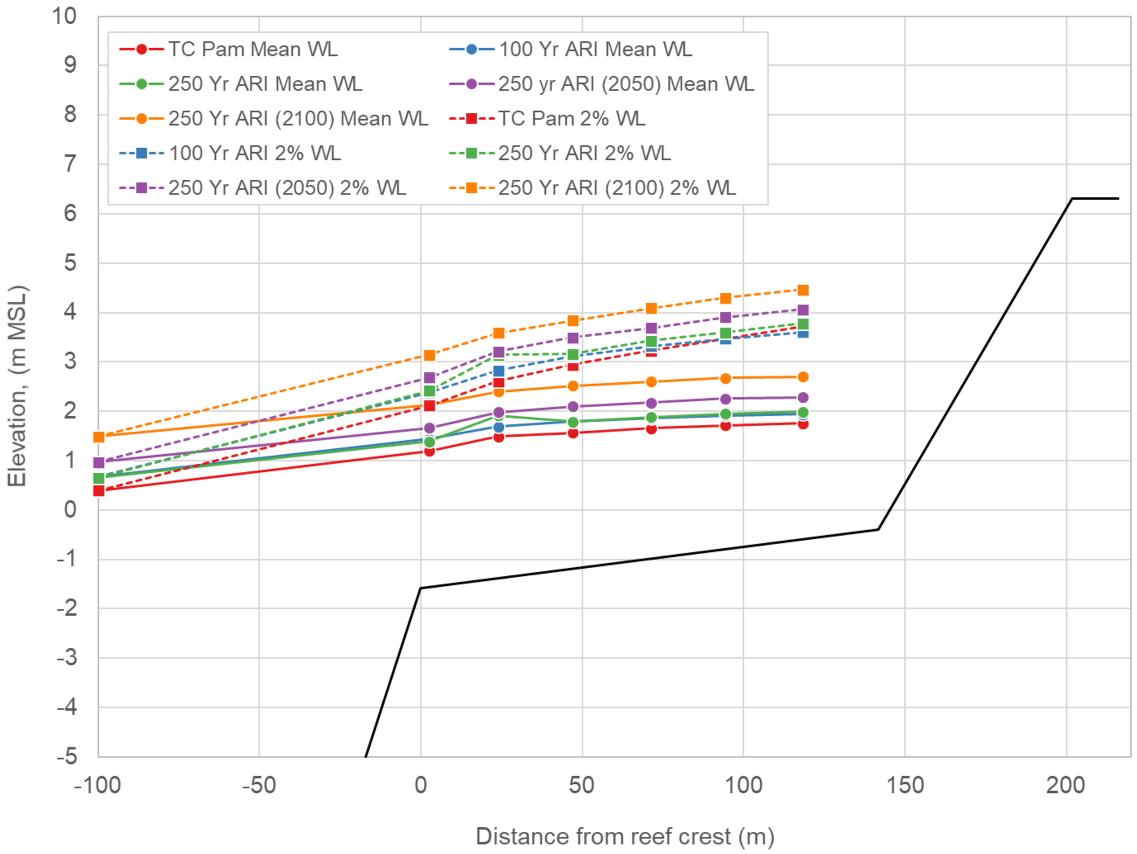

Figure 4.18: Variation in water level across reef profile for Stage 4 testing

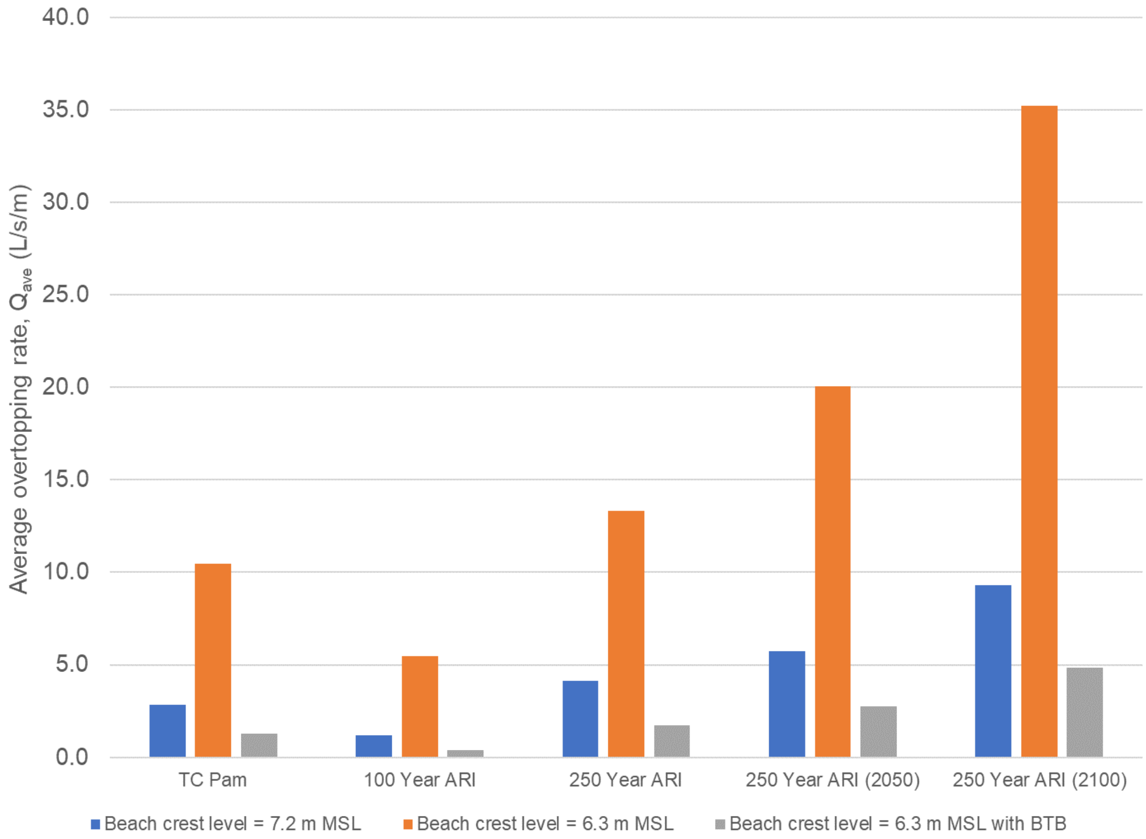

Figure 4.19: Average wave overtopping rates for Stage 4 testing
(6.3 m MSL beach berm crest level, with additional BTB installed to crest level of 8.1 m MSL)

Table 4.10 Measured water levels, Infragravity waves and overtopping for Stage 4 (6.3 m MSL beach berm crest level, with BTB installed to crest level of 8.1 m MSL)

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\bar{\eta}$ (m)						h (m MSL)					
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6
TC Pam	0.39	15.5	5.1	0.80	1.09	1.18	1.27	1.33	1.36	1.19	1.48	1.57	1.66	1.72	1.75
100 Yr ARI	0.67	14.1	5.2	0.77	1.02	1.12	1.19	1.25	1.27	1.44	1.69	1.79	1.86	1.92	1.94
250 Yr ARI	0.65	14.7	5.7	0.74	1.27	1.14	1.24	1.30	1.34	1.39	1.92	1.79	1.89	1.95	1.99
250 Yr ARI (2050)	0.97	14.7	5.7	0.69	1.01	1.13	1.20	1.28	1.31	1.66	1.98	2.10	2.17	2.25	2.28
250 Yr ARI (2100)	1.49	14.7	5.8	0.64	0.91	1.03	1.11	1.18	1.21	2.13	2.40	2.52	2.60	2.67	2.70

Test Condition	Tide + MSLA (m MSL)	$T_{p,25}$ (s)	$H_{s,25}$ (m)	$\eta_{2,IG}$ (m)						$h_{2,IG}$ (m MSL)						Ave. Overtopping, Q_{ave} (L/s/m)
				R1	R2	R3	R4	R5	R6	R1	R2	R3	R4	R5	R6	
TC Pam	0.39	15.5	5.1	1.73	2.22	2.56	2.84	3.08	3.32	1.75	2.12	2.61	2.95	3.23	3.47	1.3
100 Yr ARI	0.67	14.1	5.2	1.71	2.15	2.45	2.65	2.80	2.93	1.94	2.38	2.82	3.12	3.32	3.47	0.4
250 Yr ARI	0.65	14.7	5.7	1.77	2.49	2.51	2.77	2.94	3.13	1.99	2.42	3.14	3.16	3.42	3.59	1.7
250 Yr ARI (2050)	0.97	14.7	5.7	1.72	2.24	2.52	2.71	2.93	3.09	2.28	2.69	3.21	3.49	3.68	3.90	2.8
250 Yr ARI (2100)	1.49	14.7	5.8	1.65	2.09	2.35	2.60	2.80	2.97	2.70	3.14	3.58	3.84	4.09	4.29	4.8

4.5 Discussion of wave overtopping/inundation results

One of the core objectives of the physical modelling program was to better understand the potential for wave-driven inundation to occur at Nanumaga during the test conditions investigated. As identified in the physical model results, all test conditions are predicted to result in wave runup that exceeds the natural storm berm, and to result in at least a degree of inundation of the areas immediately landward of the storm berm. For some of the tested conditions, the wave overtopping was observed to be quite intense.

There are a number of approaches to categorise the intensity of wave-driven inundation flows, most of which relate hydrodynamic parameters to the potential impacts that would result. For stormwater flooding related situations, the hazard/risks of flood flows to people, cars and buildings are typically categorised by the depth and velocity of the flows in guidance such as the Flood Hazard guideline (AIDR, 2017). For wave overtopping inundation, guidance such as EurOtop typically relates the hazard level to the average wave overtopping rate, Q_{ave}, or the peak individual wave overtopping volume, V_{max}. To help provide context and interpretation for the wave overtopping rates presented in this report, relevant tolerable or guideline limits for safe levels of wave overtopping are summarised in Table 4.11.

Table 4.11 General limits for wave overtopping (Based on EurOtop, 2018 and Allsop, 2008)

Hazard type	Average overtopping rate, Q_{ave} (L/s/m)	Max. wave overtopping volume, V_{max} (L/m)
Damage to building structure elements	≤1	<1,000
Damage to equipment set back 5 - 10m	≤1	<1,000
Safety of people in the area near crest of storm berm	0.1-1.0	50 - 600

When compared with the guidance values in Table 4.11, the results of the physical model testing indicate that with the BTB installed, average wave overtopping rates would likely be within safe limits for people just landward of the crest of beach berm, in events up to an ARI of approximately 100 years (present day sea levels, and depending on wave period). Without the BTB, the physical modelling indicates that wave-driven overtopping/inundation would be unsafe for people and damaging for nearby buildings for all conditions/events tested.

By comparing the time series of recorded wave overtopping volume with the time series of waves and water levels measured by the wave gauge nearest to the beach, it is very apparent that wave overtopping occurs as a result of large sea/swell waves coinciding with infragravity wave peaks. Whereas for non-reef coastlines wave overtopping is predominantly related to the height and period of the incident sea/swell waves only, this is not the case for reef-type coastlines where the infragravity oscillations in water level have a much more significant role. As an example, Figure 4.20 shows an extract of data recorded for the TC Pam test condition, where the influence of infragravity wave on the occurrence of overtopping is clearly visible.

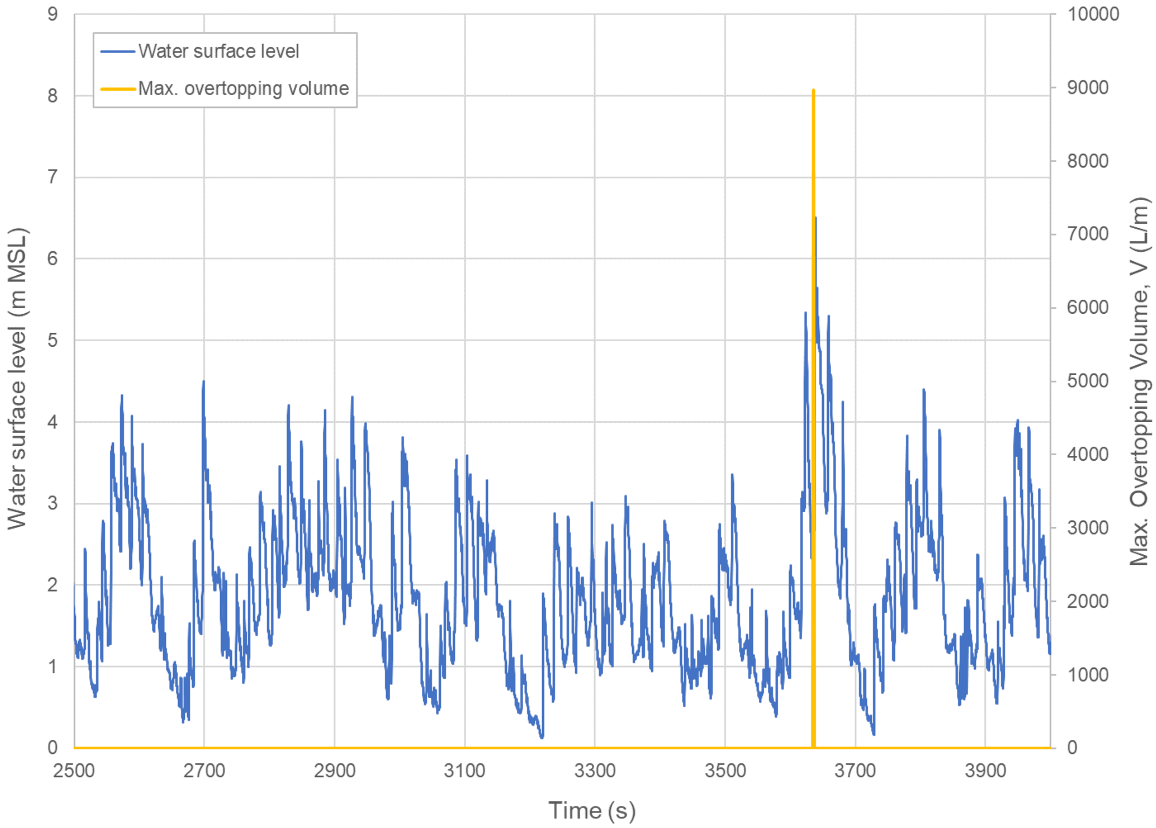

Figure 4.20: Extract of reef-top wave/water surface measurements and overtopping event

Scale physical modelling is generally accepted by the coastal engineering industry as one of the most accurate techniques to estimate wave processes, including wave overtopping. The results presented in this report are likely to be one of the best available estimates for wave overtopping risk that is available, short of a larger 3D physical model or full scale measurements at the site. Nevertheless it is prudent to also acknowledge that the wave overtopping results are likely to be slightly conservative. This is due to a number of reasons:

- The conditions simulated in the modelling represent the peak overtopping risk for each tested event magnitude. Such risk levels would only persist for a short period of time in reality, due to changing tides and moving storm conditions.
- The 2D model is a simplification of the real world situation which has 3D elements that may reduce wave-driven inundation. This predominantly includes wave approach angle relative to the reef (the 2D model simulates perpendicular wave attack which is the most severe case), and the possibility of longshore currents and drainage of the reef platform through the reef passage which reduces reef-top wave setup levels (though the reef passage at Nanumaga is relatively small in scale compared with the magnitude of storm waves tested in this model).
- The beach slope was represented as a smooth impermeable surface in the physical model. In reality the beach has micro and macro scale roughness (it is not planar), possibly absorbs some wave runup through permeability (depending on the degree of beach and storm berm saturation), and the upper beach storm berm is vegetated.

5 Conclusions

SPC is carrying out an assessment of wave-driven coastal inundation hazards for the islands of Nanumaga, Nanumea and Funafuti in Tuvalu. WRL was engaged to conduct a scale physical modelling assessment of wave processes, runup and overtopping hazards for Nanumaga island, to support SPC's investigation. The key objectives of the physical modelling study included analysis of the following parameters for a range of extreme wave and water level scenarios:

- Wave characteristics and transformation as waves cross the reef profile
- Reef-top water levels, including sustained wave setup and low frequency components
- Wave runup/inundation levels at the beach
- Reduction in wave overtopping achieved by Berm-Top-Barrier coastal protection structures

A testing program comprising 20 separate test runs was completed using a 1:25 scale, 2D cross-shore reef profile model for the western coast of Nanumaga. The tests investigated five separate wave/water level scenarios:

1. TC Pam: H_s = 5.3m, T_p = 15.6s, Tide + MSLA = 0.39m MSL
2. 100 Year ARI: H_s = 5.5m, T_p = 14.3s, Tide + MSLA =0.67m MSL
3. 250 Year ARI: H_s = 6.0m, T_p = 14.8s, Tide + MSLA =0.65m MSL
4. 250 Year ARI (2050): H_s = 6.0m, T_p = 14.8s, Tide + MSLA = 0.97m MSL
5. 250 Year ARI (2100): H_s = 6.0m, T_p = 14.8s, Tide + MSLA = 1.49m MSL

During all test conditions, waves were observed to approach the reef and break on the fore-reef slope, nevertheless they maintained almost their full height right up to the reef edge. Significant wave dissipation was then observed to occur across the outer reef flat, resulting in rapid decay in wave height, with dissipated waves almost reaching an equilibrium height by the time they were 100 m inside of the reef edge. At the most landward measurement location (118 m landward of the reef edge), significant wave heights were measured to be approximately 35% to 40% of the incident wave height outside of the reef, under all test conditions.

Wave setup was measured to increase landward across the outer 100 m of reef flat, reaching a maximum value approximately 100 m inside the reef edge. The measured wave setup at the landward measurement location ranged from 1.2 to 1.35 m, resulting in total reef-top water levels of 1.75 to 1.9 m MSL for the three test conditions with present day sea levels. For the sea level rise scenarios, wave setup was generally found to decrease or stay similar due to the greater submergence of the reef flat, nevertheless the total reef-top water levels were greater due to the sea level rise component. Total average reef-top water levels nearest the shoreline for the 250 year ARI 2050 test scenario were measured to be 2.3 m MSL, increasing to 2.7 m MSL for the 2100 scenario. Analysis of the infragravity component of reef-top water levels showed that low frequency surges in water level will occasionally reach peaks of up to approximately 3.7 m MSL with present day sea level, and up to 4.4 m MSL with 2100 sea levels.

Wave runup was found to exceed the crest of the storm berm for all completed tests, with the frequency and intensity of inundation from wave overtopping varying significantly amongst the test conditions and with different beach storm berm levels. Under all test conditions and with the existing storm berm levels, inundation from wave overtopping was recorded at intensities that exceed guideline values for the safety of people and buildings behind the beach crest. The addition of the BTB was found to decrease wave overtopping rates by a factor of 7 to 14, and for present day sea levels, the resulting overtopping rates were reduced to be at the safe limit during the 100 year ARI test condition. For the TC Pam and 250 year ARI conditions with present day sea levels, overtopping rates were measured to be slightly above guideline values.

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