Naval Point Wave Study

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1.Introduction

1.1 Background

Christchurch City Council (CCC) are interested in developing Naval Point (NP) within Lyttelton Harbour in order to improve safety and public facilities. The location for consideration is shown in Figure 1.1. Several proposed development options are being considered by CCC, including the removal of part of the existing rock breakwater (see Figure 1.1) and two different design layouts (Figure 1.2 and Figure 1.3) that can offer some protection to the environments.

As part of that development, CCC need to initially understand the likely wave climate at the site, both for the existing configuration and considering any potential development. Further, depending on the development option chosen, potential impacts of modifications on the broader environments may need to be considered, with respect to wave propagation, sediment morphological response and hydrodynamics (e.g. current velocities).

CCC has commissioned MetOcean Solutions (MOS) - a division of Meteorological Service of New Zealand Ltd to provide a summary of metocean conditions off Naval Point, Lyttleton Harbour (see Figure 1.1) and assess wave propagation into Naval Point harbour for the existing layouts and the two proposed wave attenuation design layouts (Figure 1.2 and Figure 1.3)..

Numerical hindcasting techniques are the primary source of oceanographic and meteorological data used in preparing this report.

The wave climate including summary of the data sources, analytical methods and extreme statistics are presented in Section 2. The wave penetration assessment is presented in Section 3 and include a description of the wave resolving CGWAVE model setup, model results for both existing and proposed design layouts. Section 4 provides a summary of this report and the references cited are listed in the final Section 5.

Note that the standard oceanographic directional conventions are applied in this report, with waves and winds reported in the 'coming from' directional reference.

Figure 1.1 Location of Lyttelton Port and Naval Point development (red box). A view shows the existing rock breakwater areas where improvements to the harbour structures are considered.

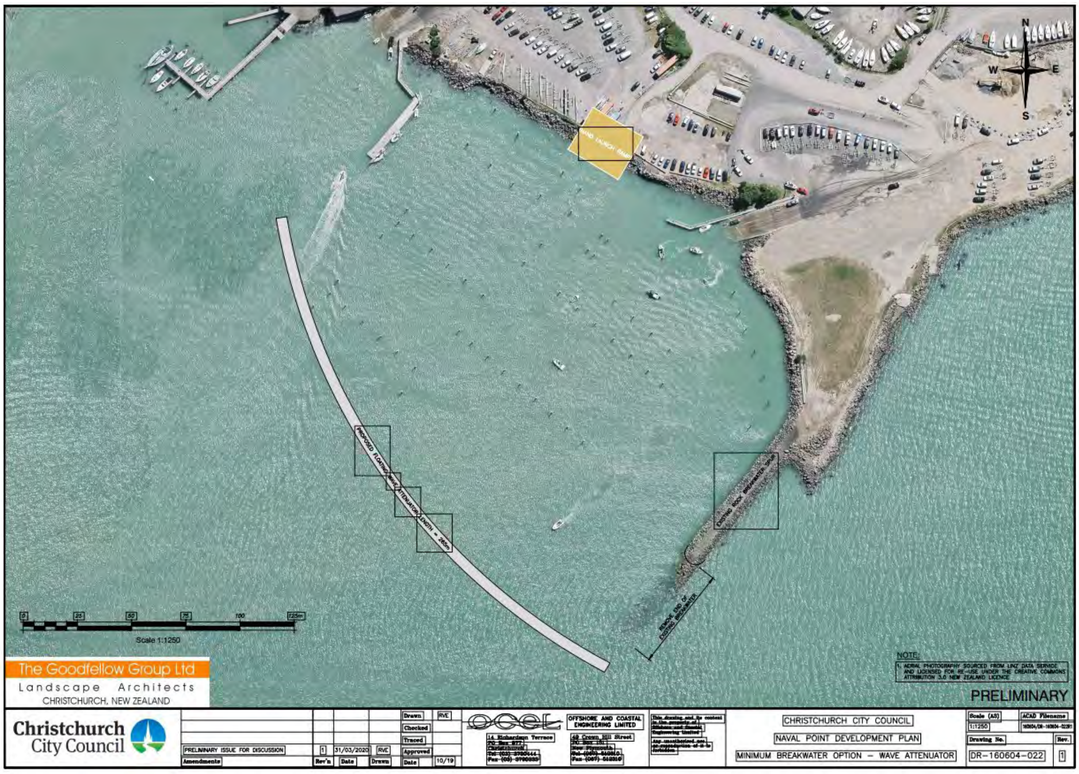

Figure 1.2 *Design layout 1. A floating wave antenuator is proposed as the minimum breakwater option. The length and the width of the floating attenuator are 265m and 5m respectively. This structure has 2m deep with 0.5m freebard (or 1.5m draft) with a transmission coefficient of 0.3.*

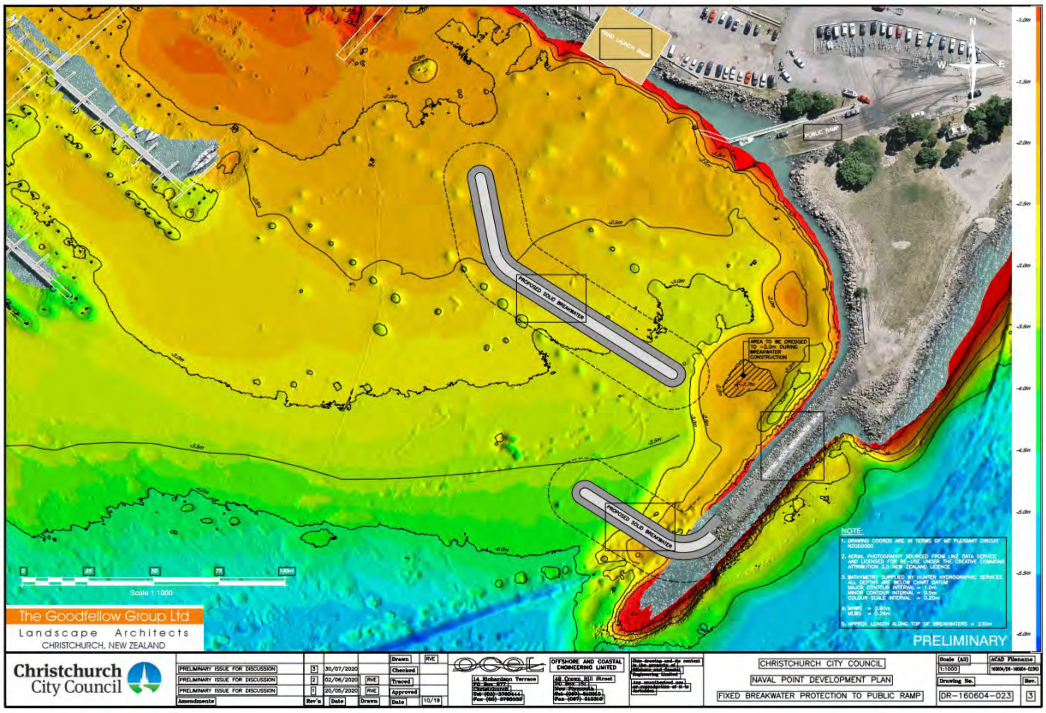

Figure 1.3 *Design layout 2. Two fixed rock break waters are proposed to protect the public ramp from the waves and water environments.*

1.2 Wave Criteria

The AS3962-2001 Guidelines for Design of Marinas standard considers the 1 year and 50-year ARI event for the assessment of 'good' wave climate in small craft harbour, however as this design project aims at providing good condition for the safe launching and retrieval of vessel at the Naval Point boat ramp, a specific design wave criteria was adopted and is described below:

For safe launching and retrieval of vessels, the Naval Point ramps need to be sufficiently protected from waves and swell. Two cases should be considered:

- anticipated normal conditions, and
- extreme events.

For anticipated normal conditions, the wave climate should be well controlled, provide a high level of amenity and be very safe in launch and retrieve conditions (i.e. a low wave height and limited surge conditions). For extreme events, it should be anticipated that very few vessels would use the ramp, mainly those caught out in adverse weather (i.e. an unexpected southerly front). In these conditions, the amenity expectation would be lower, with the focus being on the ability to safely launch/retrieve a vessel, possibly on a limited section of the ramp. This extreme case would be used for design loads on structures. Noting that the extreme event and associated loads on the marine structures will likely be a controlling case for the engineering design.

In order to establish the design wave criteria (wave height and period) for both cases, a return period event and acceptable wave height has been selected.

For the typical case, there is no specific guidance on the appropriate return period. Ultimately what is appropriate will be reliant on user's expectation on how often wave conditions are above a level which detracts from their enjoyment of using the ramp. It was proposed that the risk of this event occurring a single time in one year should be no more than 20%. This equates to a 5-year return period event.

In terms of wave height, the Australian Standard (AS3962-2001) for marina design recommends boat ramps are "aligned to the dominant waves from swell. Sea and boat wash" and sheltered from waves greater than 0.2m. Although Permanent International Association of Navigational Congresses (PIANC) recommends launching and retrieving areas are subject to no more than 0.15m high waves.

For the extreme case, PIANC suggests that a design event of 1 in 50 years is appropriate for marina type structures with a design life of 30 years. This results in a 45% chance of the design storm occurring in the projects life. There is no guidance for an acceptable wave height in these types of conditions, although PIANC notes for marinas that 'moderate conditions' can be up to 1.67 times 'excellent conditions. This would equate to a design wave height of 0.25m in the 1 in 50-year event. Noting that this recommendation is for boats moored in a marina, not for vessels launching at a ramp so should be used with care.

There is no readily available guidance for wave period and surge conditions, however the wave model should be interrogated to evaluate how longer period waves may impact on the ramp use and amenity.

A summary of the potential design criteria is included in Table 4.

Table 1-1: Summary of design wave criteria

Design case	**Return period event**	**Max wave height (m) within protected area.**	**Considerations**
Normal conditions	5 year	0.15	Period and swell angle.
Extreme event	50 year	0.25	

2.Wave Climate

2.1 Data

2.1.1 Wind data

The 10-min averaged wind data at 10-m elevation were prescribed by a regional atmospheric hindcast carried out by MOS from 2009-2018 (inclusive). The WRF (Weather Research and Forecasting) model was established over all New Zealand at hourly intervals and approximately 12 km resolution with a nested domain over central regions at 4 km resolution. The hindcast was specifically tuned to provide highly accurate marine wind fields for metocean studies around New Zealand.

The WRF model boundaries were sourced from the CFSR (Climate Forecast System Reanalysis) dataset distributed by NOAA (Saha et al., 2010).

Validation of the WRF reanalysis has been undertaken at various locations around New Zealand.

2.1.2 Wave data

The wave modelling was performed using a modified version of SWAN1. This section describes details of the wave model and the technique employed in the simulations.

Model description

SWAN is a third generation ocean wave propagation model which solves the spectral action density balance equation (Booij et al., 1999). The model simulates the growth, refraction and decay of each frequency-direction component of the complete sea state, providing a realistic description of the wave field as it changes in time and space. Physical processes that can be modelled include the generation of waves by surface wind, dissipation by white-capping, resonant nonlinear interaction between the wave components, bottom friction and depth-induced wave breaking dissipation. A detailed description of the model equations, parametrisations and numerical schemes can be found in Holthuijsen et al. (2007) and in the SWAN documentation[2].

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[1] Modified from SWAN version of the 40.91 release
[2] http://swanmodel.sourceforge.net/

Model setup

The wave hindcast was set and run for a 10-year period, from 2009 to 2018. The model was configured in non-stationary mode including all third-generation physics. The source term parameterisations of Van der Westhuysen et al. (2007) and the bottom friction scheme of Collins (1972) with coefficient of 0.015 were applied. Depth-induced wave breaking dissipation was modelled according to Battjes and Janssen (1978). The wave spectra were discretised with 36 directional bins (10 deg directional resolution) and up to 44 frequencies logarithmically spaced between 0.0412 and 3.002 Hz at 10% increments.

A dynamical downscaling nesting approach was applied to resolve the nearshore region around the site of interest (see Figure 1.1and Table 2-1). To fully capture the details of the coastal line and bathymetry in the area, 4 regular SWAN nests were defined with resolutions of ~4 km, ~ 400 m, ~ 50 m and a fine grid of ~ 10 m to resolve the small-scale bathymetric features.

Table 2-1 Coordinate and approximate water depth at the representative data reporting site.

Site	World Geodetic System 1984 (WGS84)		Water depth (m, MSL)
	Longitude	Latitude	
P0	172.705572° E	-43.614748° N	5.0

Full spectral boundaries for the parent SWAN hindcast domain were prescribed from a global implementation of the WAVEWATCHIII (WW3) spectral wave model (Tolman, 1991), run at 0.5 deg resolution with the source terms of Ardhuin et al. (2010). Bathymetry to setup the SWAN domains was derived by processing and combining data from the General Bathymetric Charts of the Oceans (GEBCO) global database Weatherall et al. (2015), Electronic Nautical Charts (ENCs) and survey data. The model was forced with surface winds from a configuration of the Weather Research and Forecasting (WRF) as described in the previous subsection. The wave model also included tidal level derived from the constituents obtained from the POM (Princeton Ocean Model) model.

Examples of significant wave height (Hs) wave fields for the three last SWAN nests are given in Figure 2.1 and Figure 2.2 for the cases of waves entering the Canterbury Basin form the NE and S sectors, respectively.

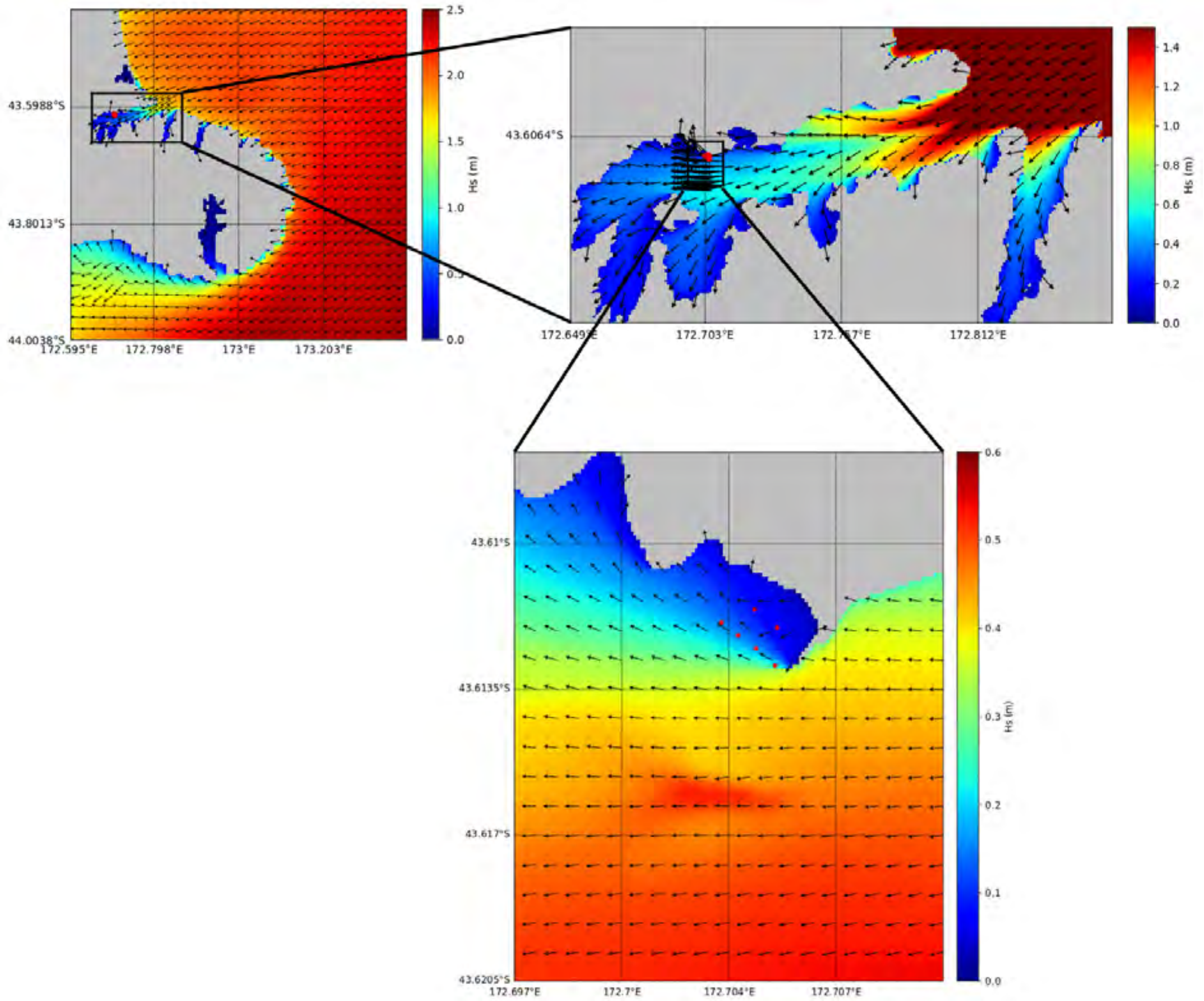

Figure 2.1 *Example of modelled Hs field output for waves incoming from the NE sector for the 3 last SWAN nests. Vectors represent modelled wave direction.*

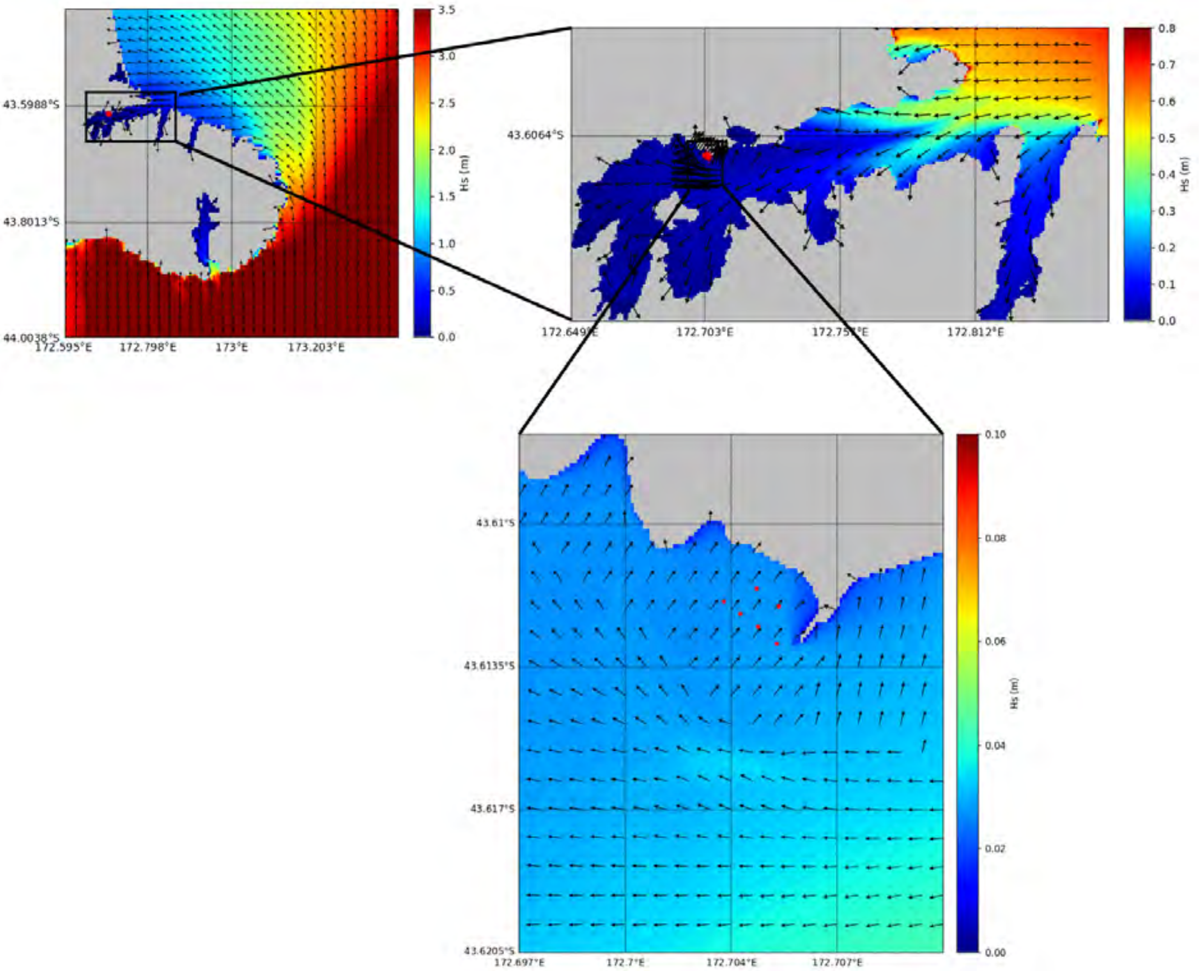

Figure 2.2 Example of modelled Hs field output for waves incoming from the S sector for the 3 last SWAN nests. Vectors represent modelled wave direction. Red dots show sites for output results.

2.2 Analytical methods

2.2.1 Wave

The wave spectra were post-processed to calculate wave statistics for the total wave field, as well as for sea and swell components. The spectral partitioning method consists of a split at the frequency corresponding to 8 s period, with sea and swell assigned to the high- and low-frequency parts, respectively. For the total spectra and each partition, one-dimensional frequency spectra were defined by integrating over all directions:

$$E(f) = \int_{-\pi}^{\pi} E(f,\theta)d\theta. \qquad (5.1)$$

Spectral moments were calculated as

$$m_x = \iint f^x E(f,\theta) df\, d\theta, \tag{5.2}$$

The significant wave height, Hs, mean direction at peak energy, θp, and peak wave period, Tp , are defined as:

$$H_s = 4\sqrt{m_0}, \tag{5.3}$$

$$Dpm = \tan^{-1}\frac{\int_{-\pi}^{\pi} E(f_p,\theta)\sin\theta\, d\theta}{\int_{-\pi}^{\pi} E(f_p,\theta)\cos\theta\, d\theta}, \tag{5.4}$$

$$T_p = 1/f_p, \tag{5.5}$$

where *fp* is the peak wave frequency of the one-dimensional spectra and En(fp,θ) is the energy contained in the peak wave frequency band. Note that *Tp* and *Dpm* require spectral peaks within a given partition and are not defined when peaks are not identified for that partition.

2.2.2 Extreme

Directional return period values have been calculated from the hindcast time series of wave parameters.

A *Peaks over Threshold* (POT) sampling method is used for event selection, applying the 95th percentile exceedance level as the threshold with a 24-hour window. For wave EVA, the selected events were fitted to a Pareto distribution, with the location parameter fixed by the threshold and the Maximum Likelihood Method (MLM) used to obtain the scale and shape parameters.

Bivariate return period values were calculated for significant wave height and peak period. The method of Repko et al. (2005) was employed, which considers the distribution of H_s and wave steepness, *s*. A joint probability distribution function (PDF) is calculated by multiplying marginal distributions of H_s and *s* (thus assuming they are independent), after which the PDF is transformed back into H_s/T_p space. In addition, a minimum wave steepness threshold of 0.005 is applied to exclude events with very long wave periods, which are not believed to be representative of extreme conditions.

The marginal distributions for H_s and *s* are estimated by fitting the POT values to a Weibull distribution using the maximum likelihood method (as implemented in the WAFO toolbox). Contours of the return period values were constructed from the joint PDF using the Inverse FORM method (Winterstein et al., 1993) at the return year levels.

The methods used to estimate extreme maximum individual wave height (H_{max}) and maximum wave crest (C_{max}) account for the long-term uncertainty in the severity of the environment and the short-term uncertainty in the severity of the maximum wave of a given sea state, as suggested by Tromans and Vanderschuren (1995) and recommended by ISO (2015). The most probable value of the extreme individual wave height (H_{mp}) of each storm is obtained from the product of the Forristall distributions of individual wave height in each hindcast interval within the storm duration (Forristall, 1978; ISO, 2015). The same technique is used for the most probable value of the extreme individual wave crest (C_{mp}) but using the Weibull distribution with scale and shape parameters dependent on the wave steepness and the Ursell number (ISO, 2015; Forristall, 2000). Note that the resulting short-term distributions for each storm are dependent on the number of intervals with H_s values near the region of maximum peak H_s. The uncertainty in the height and crest of the maximum wave of any storm is represented as a short-term probability distribution conditional on H_{mp} and C_{mp}, respectively (Tromans and Vanderschuren, 1995). The long-term distributions of H_{mp} and C_{mp} are then fitted to Pareto distributions. Finally, the convolutions of the short- and long-term distributions give the complete long-term distributions of H_{max} and C_{max} (Tromans and Vanderschuren, 1995; ISO 2015).

Note an arbitrary minimum number of 10 storm peaks has been was chosen for reliable distribution fitting. This results in specific directional return period values being omitted.

2.3 Ambient Waves Statistics

A summary of the total significant wave height statistics (Hs) at P0 is provided in Table 2-2.

The monthly and annual significant wave height exceedance probabilities are presented in Table 2-3.

The annual joint probability distribution of the total significant wave height and peak period is presented in Table 2-4. The annual joint probability distribution of the total significant wave height and mean wave direction at peak energy is presented in Table 2-5.

Wave roses for the monthly and annual total significant wave height are presented in Figure 2.3, showing the predominance of waves incoming from the E sector.

Table 2-2 Annual and monthly total significant wave height statistics at P0.

Period (01 Jan 2009 – 31 Dec 2018)	Total significant wave height statistics (1)													
	Total significant wave height (m)				Exceedance percentile for total significant wave height (m)									Main (2) Direction(s)
	min	max	mean	std	p1	p5	p10	p50	p80	p90	p95	p98	p99	
January	0.01	0.88	0.22	0.12	0.04	0.06	0.08	0.20	0.33	0.39	0.44	0.51	0.54	E SW
February	0.02	0.76	0.21	0.12	0.03	0.06	0.07	0.20	0.32	0.37	0.41	0.48	0.52	E SW
March	0.01	0.81	0.20	0.12	0.03	0.05	0.07	0.18	0.30	0.37	0.42	0.48	0.54	E SW
April	0.01	0.96	0.19	0.13	0.03	0.05	0.06	0.16	0.28	0.35	0.43	0.53	0.61	E SW
May	0.01	0.76	0.19	0.13	0.02	0.04	0.06	0.16	0.28	0.38	0.46	0.55	0.60	E SW
June	0.01	0.71	0.19	0.13	0.01	0.03	0.05	0.16	0.29	0.37	0.44	0.53	0.58	E SW
July	0.01	0.79	0.18	0.12	0.02	0.04	0.05	0.15	0.28	0.36	0.43	0.50	0.54	E SW W
August	0.01	0.66	0.19	0.12	0.02	0.04	0.06	0.17	0.29	0.37	0.43	0.50	0.54	E SW
September	0.01	0.86	0.19	0.12	0.03	0.05	0.07	0.17	0.29	0.36	0.43	0.51	0.56	E SW
October	0.01	0.73	0.19	0.11	0.03	0.05	0.07	0.17	0.28	0.35	0.40	0.47	0.51	E SW
November	0.01	0.58	0.21	0.11	0.03	0.06	0.07	0.19	0.31	0.37	0.42	0.46	0.50	E SW W
December	0.01	0.63	0.22	0.12	0.03	0.06	0.08	0.20	0.33	0.40	0.45	0.51	0.53	E SW
Winter	0.01	0.79	0.19	0.12	0.02	0.04	0.05	0.16	0.29	0.37	0.44	0.51	0.55	E SW
Spring	0.01	0.86	0.20	0.12	0.03	0.05	0.07	0.17	0.29	0.36	0.42	0.48	0.52	E SW
Summer	0.01	0.88	0.22	0.12	0.04	0.06	0.08	0.20	0.33	0.39	0.44	0.50	0.53	E SW
Autumn	0.01	0.96	0.19	0.13	0.03	0.04	0.06	0.16	0.29	0.36	0.44	0.52	0.59	E SW
All	0.01	0.96	0.20	0.12	0.03	0.05	0.06	0.17	0.30	0.37	0.43	0.50	0.55	E SW

Notes: (1) All statistics derived from hindcast wave data for the period 01 January 2009 to 31 December 2018.

(2) Main directions are those with greater than 15% occurrence and represent directions from which the waves approach.

Table 2-3 Monthly and annual total significant wave height exceedance probabilities (%) at P0.

H_S (m)	Exceedance (%)												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	annual
>0	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
>0.1	83.44	81.31	76.49	71.96	72.65	72.11	70.30	71.99	75.29	78.10	80.74	82.94	76.42
>0.2	49.63	49.04	44.68	36.49	36.76	39.58	36.75	39.29	40.10	40.47	47.01	49.36	42.40
>0.3	25.08	24.39	20.69	16.25	17.42	19.26	17.41	18.27	17.33	16.75	21.08	26.37	20.01
>0.4	8.80	6.66	6.71	6.63	8.32	7.43	6.94	7.10	6.79	5.32	6.85	10.00	7.30
>0.5	2.27	1.37	1.59	2.79	3.60	2.74	2.08	2.14	2.18	1.32	1.10	2.62	2.16
>0.6	0.36	0.24	0.67	1.07	1.02	0.72	0.47	0.22	0.64	0.16	0.00	0.09	0.47
>0.7	0.13	0.07	0.40	0.47	0.17	0.01	0.13	0.00	0.36	0.05	0.00	0.00	0.15
>0.8	0.12	0.00	0.07	0.29	0.00	0.00	0.00	0.00	0.08	0.00	0.00	0.00	0.05
>0.9	0.00	0.00	0.00	0.08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01

Table 2-4 Annual joint probability distribution (in %) of the total significant wave height and peak period at P0.

Hs (m)	Peak period (s)																			
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	Total	Exceed%
>0<=0.1	12.73	1.20	4.85	2.82	0.05	0.02	0.02	0.05	0.11	0.13	0.15	0.10	0.11	0.01	-	-	*	*	22.35	100.00
>0.1<=0.2	29.71	3.83	0.13	0.17	-	-	-	-	-	-	-	-	-	-	-	-	-	-	33.84	76.42
>0.2<=0.3	11.18	10.95	0.02	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	22.15	42.40
>0.3<=0.4	0.48	12.27	0.05	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	12.80	20.01
>0.4<=0.5	-	5.02	0.07	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	5.09	7.30
>0.5<=0.6	-	1.37	0.28	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1.65	2.16
>0.6<=0.7	-	0.15	0.16	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.31	0.47
>0.7<=0.8	-	0.03	0.08	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.11	0.15
>0.8<=0.9	-	*	0.03	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.03	0.05
>0.9<=1	-	-	0.01	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.01	0.01
Total	54.10	34.82	5.68	2.99	0.05	0.02	0.02	0.05	0.11	0.13	0.15	0.10	0.11	0.01	-	-	-	-	100.00	
>Exceed%	98.36	44.33	9.43	3.76	0.76	0.71	0.69	0.67	0.61	0.51	0.38	0.23	0.13	0.02	0.01	0.01	0.01	*		

Notes: * represents less than 0.005%.

Table 2-5 Annual joint probability distribution (in %) of the total significant wave height and mean wave direction at peak energy at P0.

Hs (m)	Mean wave direction at peak energy (degT)									
	337.5-22.5	**22.5-67.5**	**67.5-112.5**	**112.5-157.5**	**157.5-202.5**	**202.5-247.5**	**247.5-292.5**	**292.5-337.5**	**Total**	**Exceed%**
>0<=0.1	0.09	0.11	14.49	0.43	1.95	3.99	2.51	0.43	24.00	100.00
>0.1<=0.2	0.03	0.03	15.10	0.23	2.18	10.22	5.59	0.49	33.87	76.42
>0.2<=0.3	-	*	12.62	0.02	0.51	6.39	2.60	*	22.14	42.40
>0.3<=0.4	-	-	8.78	*	0.16	2.84	1.01	-	12.79	20.01
>0.4<=0.5	-	-	3.34	*	0.08	1.44	0.23	-	5.09	7.30
>0.5<=0.6	-	-	0.96	*	0.01	0.64	0.04	-	1.65	2.16
>0.6<=0.7	-	-	0.17	-	0.01	0.13	0.01	-	0.32	0.47
>0.7<=0.8	-	-	0.08	-	*	0.03	-	-	0.11	0.15
>0.8<=0.9	-	-	0.03	-	-	*	-	-	0.03	0.05
>0.9<=1	-	-	0.01	-	-	-	-	-	0.01	0.01
Total	0.12	0.14	55.58	0.68	4.90	25.68	11.99	0.92	100.00	

Notes: * represents less than 0.005%.

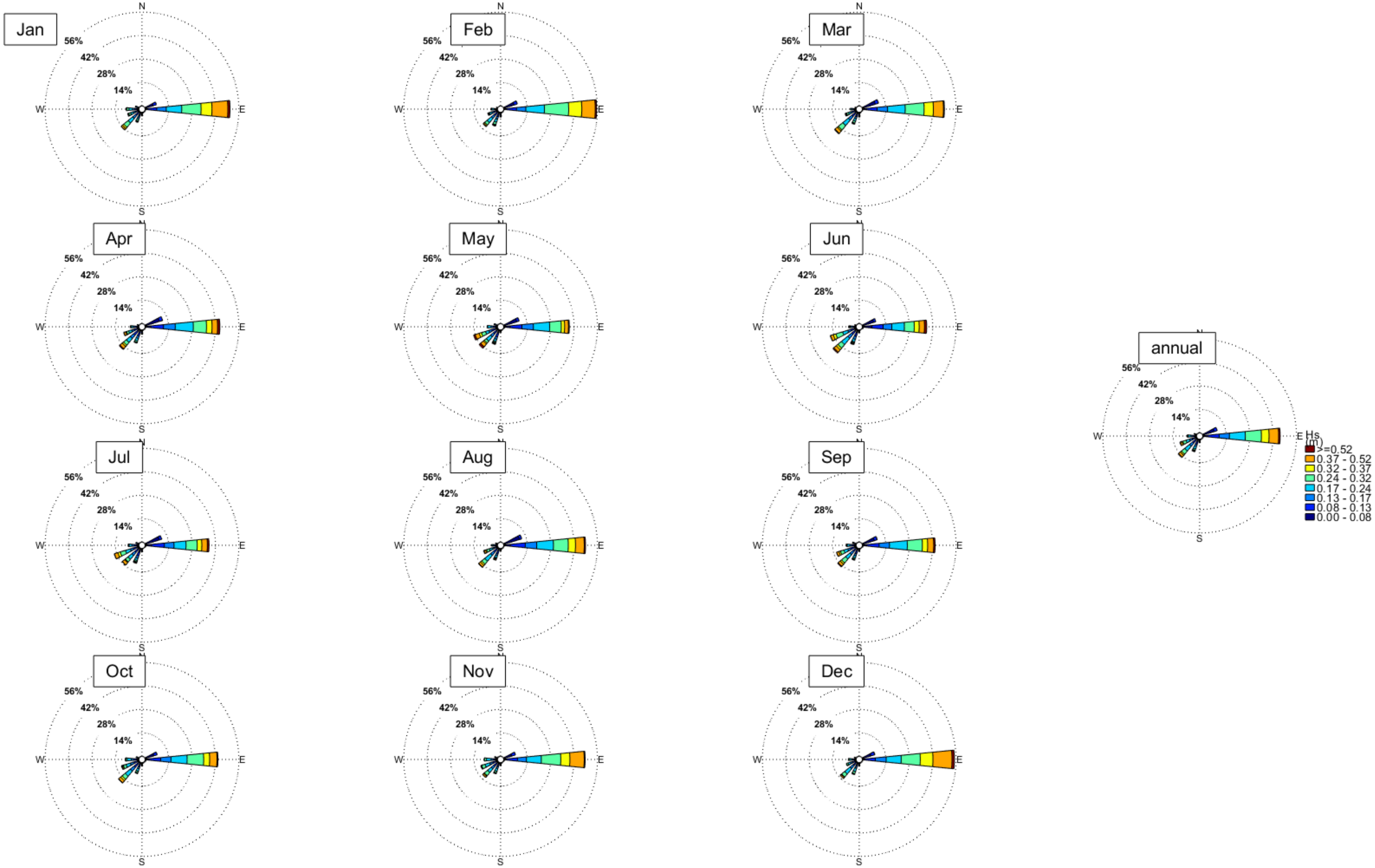

Figure 2.3 Monthly and annual wave rose plot for the total significant wave height at P0. Sectors indicate the direction from which waves approach.

2.4 Extreme Waves Statistics

The directional return period values for wave extremes are given in Table 2-6 to Table 2-10 for 1, 5, 10, 20 and 50-year return periods.

Contour plot of omni-directional bi-variate return period values for significant wave height and peak wave period are presented in Figure 2.4.

Table 2-6 Annual independent omni-directional extreme criteria for wind, wave and current at P0.

Parameter	Symbol	Units	Return period (year)				
			1	5	10	20	50
Significant wave height	H_s	*m*	0.74	0.84	0.88	0.91	0.95
Peak wave period	T_p	*s*	3.33	3.58	3.67	3.76	3.87
Maximum individual wave height	H_{max}	*m*	1.40	1.58	1.63	1.69	1.77
Maximum individual wave crest	C_{max}	*m*	0.81	0.92	0.94	0.98	1.02

Table 2-7 Annual independent East extreme criteria for wind, wave and current at P0.

Parameter	Symbol	Units	Return period (year)				
			1	5	10	20	50
Significant wave height	H_s	*m*	0.72	0.83	0.88	0.93	0.98
Peak wave period	T_p	*s*	3.34	3.59	3.68	3.77	3.89
Maximum individual wave height	H_{max}	*m*	1.31	1.50	1.58	1.65	1.73
Maximum individual wave crest	C_{max}	*m*	0.74	0.88	0.92	0.96	1.02

Table 2-8 Annual independent South extreme criteria for wind, wave and current at P0

Parameter	Symbol	Units	Return period (year)				
			1	5	10	20	50
Significant wave height	H_s	*m*	0.39	0.53	0.59	0.65	0.72
Peak wave period	T_p	*s*	2.23	2.55	2.66	2.75	2.87
Maximum individual wave height	H_{max}	*m*	0.74	0.99	1.08	1.15	1.23
Maximum individual wave crest	C_{max}	*m*	0.41	0.57	0.63	0.68	0.72

Table 2-9 Annual independent South-West extreme criteria for wind, wave and current at P0.

Parameter	Symbol	Units	Return period (year)				
			1	5	10	20	50
Significant wave height	H_s	m	0.65	0.73	0.76	0.78	0.80
Peak wave period	T_p	s	2.70	2.82	2.85	2.88	2.92
Maximum individual wave height	H_{max}	m	1.23	1.40	1.46	1.50	1.58
Maximum individual wave crest	C_{max}	m	0.70	0.81	0.85	0.89	0.93

Table 2-10 Annual independent West extreme criteria for wind, wave and current at P0.

Parameter	Symbol	Units	Return period (year)				
			1	5	10	20	50
Significant wave height	H_s	m	0.46	0.54	0.57	0.60	0.65
Peak wave period	T_p	s	2.38	2.54	2.60	2.65	2.72
Maximum individual wave height	H_{max}	m	0.89	1.04	1.08	1.13	1.19
Maximum individual wave crest	C_{max}	m	0.51	0.59	0.64	0.66	0.69

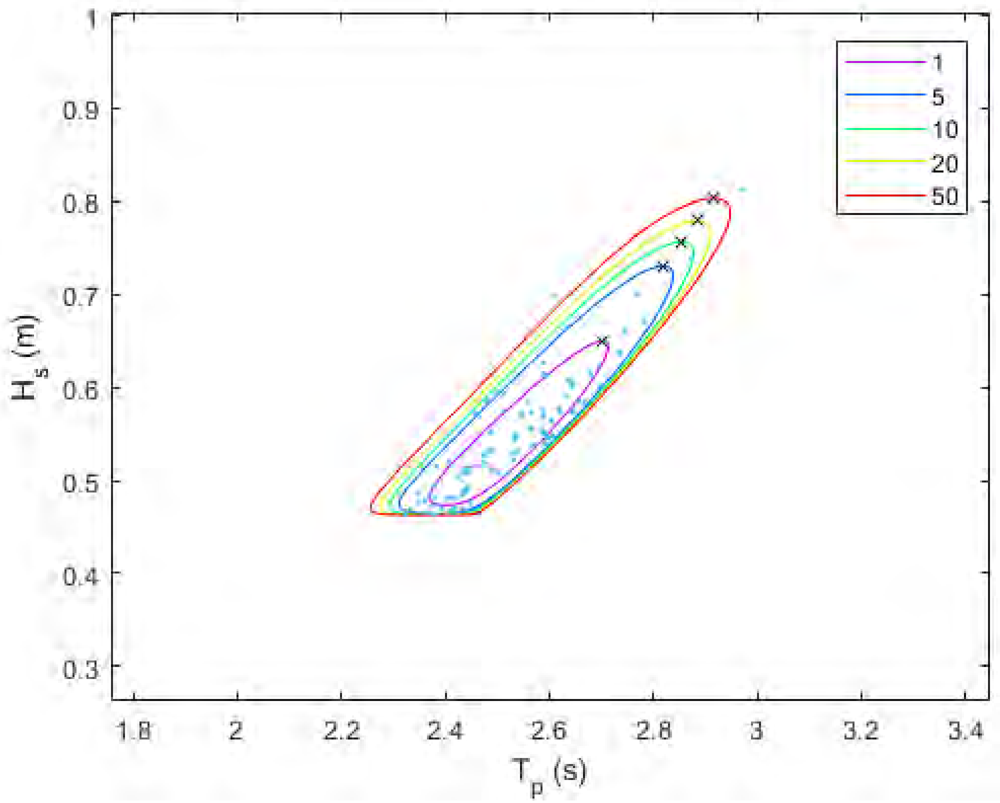

Figure 2.4 Contour plot of omni-directional bi-variate (Hs-Tp) return period values for 1, 5, 10, 20 and 50-year ARIs. The dark crosses correspond to the estimated deterministic Hs and associated Tp return period values for each ARI indicated in the legend at P0.

3.Wave Penetration Assessment

3.1 Phase Resolving Wave Model

The CGWAVE model is an industry-standard wave modelling tool used in harbours and coastal regions with complex bathymetry. This wave model simulates the combined effects of wave refraction-diffraction within the mild-slope equation, and includes the effects of reflection, wave dissipation by friction, breaking, nonlinear amplitudes dispersion, diffraction and wave refraction from rocky shorelines and other structures (Demirbilek and Panchang, 1998).

CGWAVE uses a finite-element (triangular) mesh (FEM), able to be structured such that key bathymetric features are suitably resolved (e.g. channels, shoals, port structures, rocky reef outcrops etc.). CGWAVE is particularly suited for the present study given the site complexity and the multiple fine-scale wave transformation processes occurring over this region (e.g. refraction, diffraction) as well as the effect of the proposed wave screen.

3.2 Computational domain

CGWAVE requires a triangle unstructured mesh and bathymetry information at every grid point. In this study, the same computational domain extent is applied to the existing and design layouts in order to allow direct comparison of the results. The grid was generated using SMS and the same scale paving density function so that there is no change in the position of each grid point. The bathymetry survey provided by CCC was interpolated into the triangle mesh using linear interpolation method with a minimum water depth of 1m. The open boundary is semi-circle with a radius of about 360m; The grid size ranges from 4 to 10m. All grid points along the wave attenuator and around the breakwaters are aligned to ensure the computational mesh correctly resolves the design layout features.

The final grid used for existing and the design layout with a wave attenuator (design layout 1) has 13431 grid points and 26294 elements (see Figure 3.1 and Figure 3.2). To represent the floating wave attenuator in the design layout 1, all mesh elements within the attenuator position were selected for a different material type (e.g. floating dock) during the model setup with a draft of 1.5m and a coefficient of 0.3.

For the design layout with two breakwaters (design layout 2), the number of grid points and elements are 13335 and 26002 respectively, which are slightly smaller than other layouts as this mesh does include dry points on the breakwaters (see Figure 3.3).

Wave parameters model results are extracted at 6 locations within Naval Point as presented in Table 3-1.

Table 3-1 Extract point locations

Easting	Northing
1576084.0	5171100.2
1576144.8	5171123.6
1576218.3	5171126.8
1576266.0	5171138.0
1576228.9	5171188.5
1576245.2	5171227.8

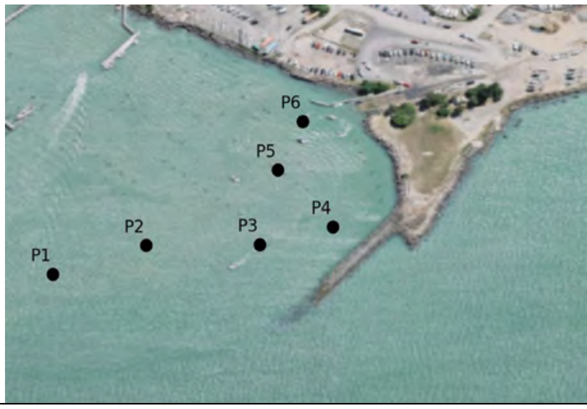

Figure 3.1 The computational mesh and bathymetry for the existing layout

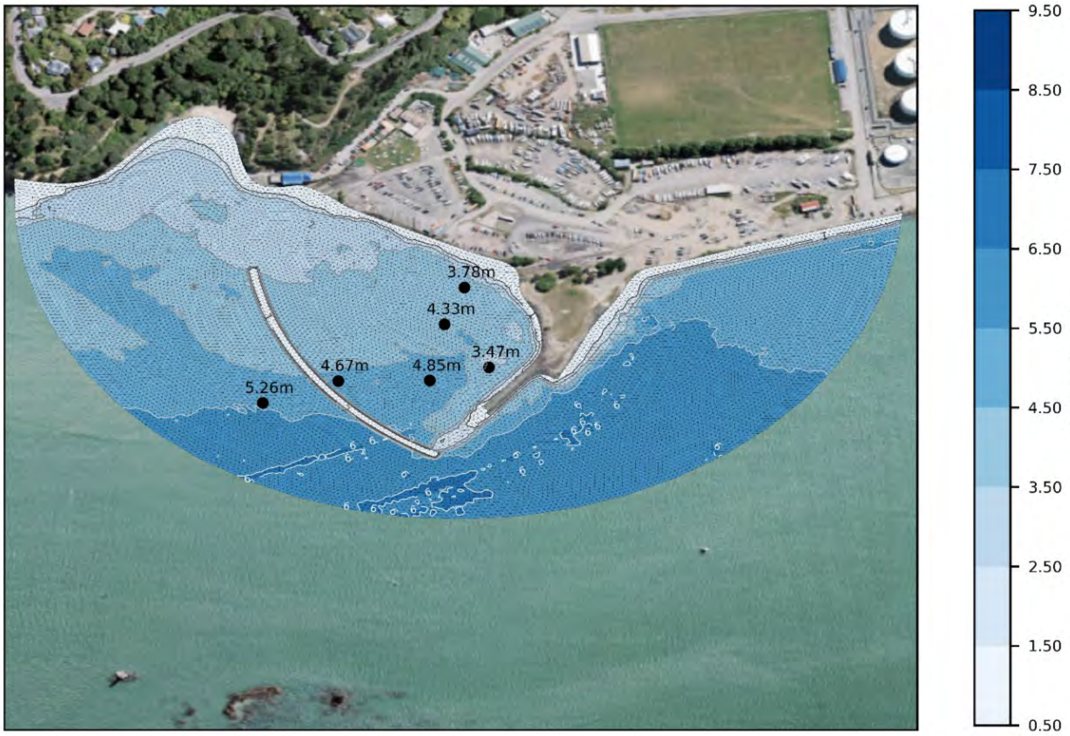

Figure 3.2 *The computational domain and bathymetry used for the design layout 1 with a wave attenuator (length =265m, width= 5m, draft =1.5m; transmission coefficient = 0.3)*

Figure 3.3 *The computational domain and bathymetry used for design layout 2 (with two rock breakwaters*

3.3 Model input

Incident wave conditions are specified at the open boundary of the computational mesh. These wave data were taken from the results of the 10-year wave hindcast study presented in Section 2 together with the 5-year and 50-year Design Wave Criteria adopted and discussed in Section 1.2 .

Extreme statistics are given in Table 2.1. These extreme values were then used for incident wave conditions required by CGWAVE

Table 3-2 Extreme 5-year and 50-year Return Period Wave Parameters off Naval Point (Location P0)

Direction	5-year return period		50-year return period	
	Hs (m)	*Tp (s)*	*Hs (m)*	*Tp (s)*
E	0.83	3.59	0.98	3.89
S	0.53	2.55	0.72	2.87
SW	0.73	2.83	0.8	2.92
W	0.54	2.54	0.65	2.72

3.4 Results

The following sections present modelled results from the existing layout and two proposed layouts based on 5 and 50 year-return period for different wave directions. In each layout option, a table is given to summarise modelled results of water elevations and wave heights. In model detail, plots of these parameters over computational domains are also presented.

3.4.1 Existing layout

Results for the easterly, southerly, south-westerly and westerly 5year and 50-year Return period are shown on Figure 3.4 to Figure 3.11 and Table 3-3.

Based on the design wave criteria (Section 1.2), the existing layout only gives a good wave climate for Easterly wave conditions as the existing breakwater protects the harbour from wave penetration. The modelled wave height in the harbour is expected to be very small under both 5- and 50-return period (see Figure 3.5). For other wave directions, modelled results indicate that the existing layout does not satisfy the design wave criteria. The most severe scenario is with waves propagating into the harbour from south-west with wave is expected to be about 0.8m and 0.9m based on 5- and 50-year return period extremes (see Figure 3.9).

Table 3-3:Significant wave height for the 5-year and 50-year Return Period at extracted points based on existing layout. Green shading shows conditions below the design wave criteria of 0.15m (for the 5-year RP) and of 0.25m (50-year RP).

	5 -year return period						50-year return period					
Wave Direction	1	2	3	4	5	6	1	2	3	4	5	6
E	0.57	0.08	0.15	0.03	0.01	0.06	0.61	0.07	0.15	0.02	0.01	0.06
S	0.59	0.55	0.31	0.29	0.60	0.48	0.82	0.65	0.45	0.36	0.73	0.60
SW	0.68	0.61	0.84	0.75	0.78	0.64	0.74	0.69	0.91	0.81	0.85	0.73
W	0.69	0.62	0.74	0.34	0.59	0.46	0.83	0.72	0.85	0.46	0.68	0.56

3.4.1.1 Easterly Waves

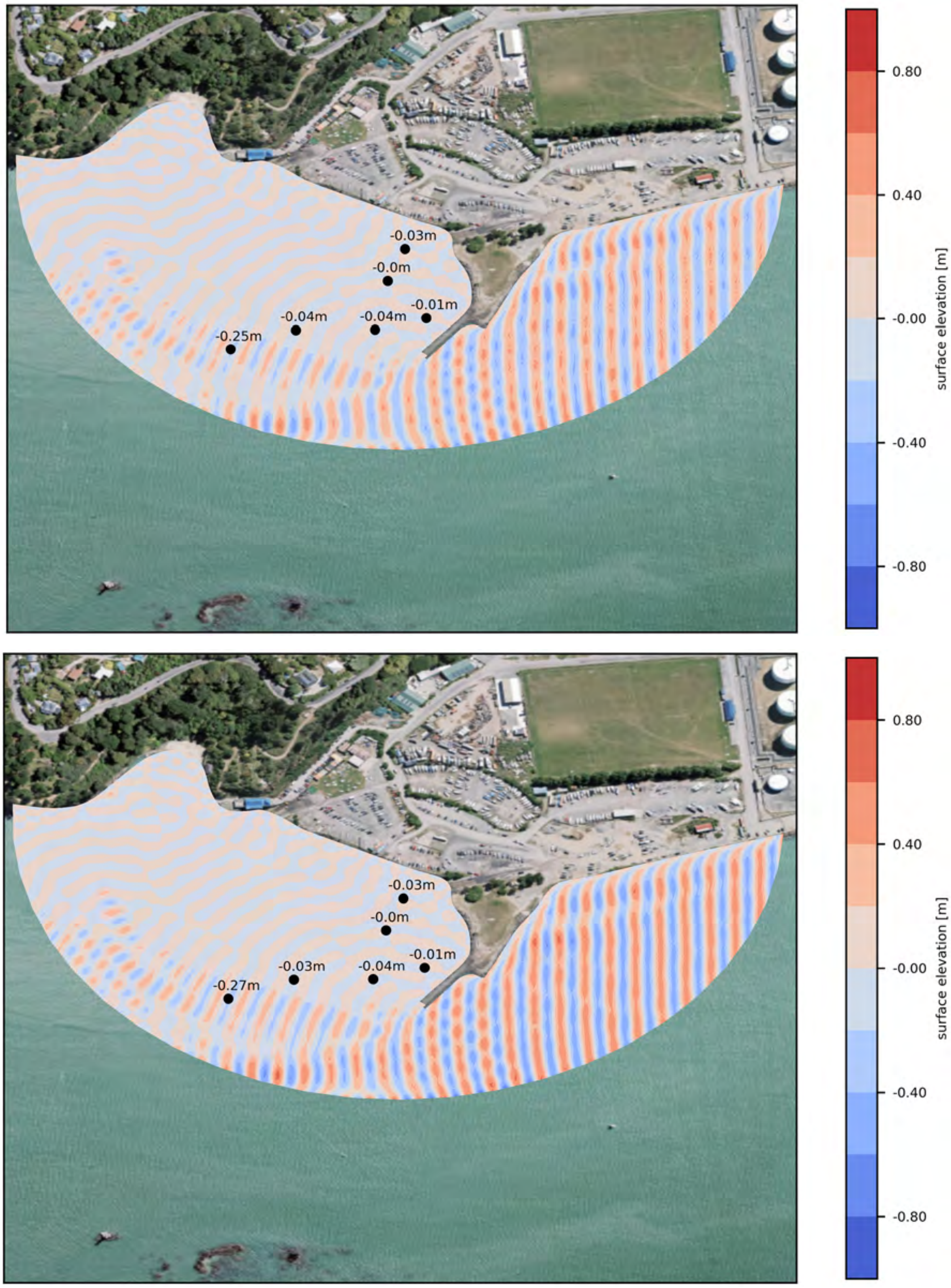

Figure 3.4 The surface elevation modelled for existing layout with Easterly waves: Hs=0.83m; Tp=4s (5-year return period, top panel) and Hs=0.98m, Tp=4s (50 year return period, bottom panel).

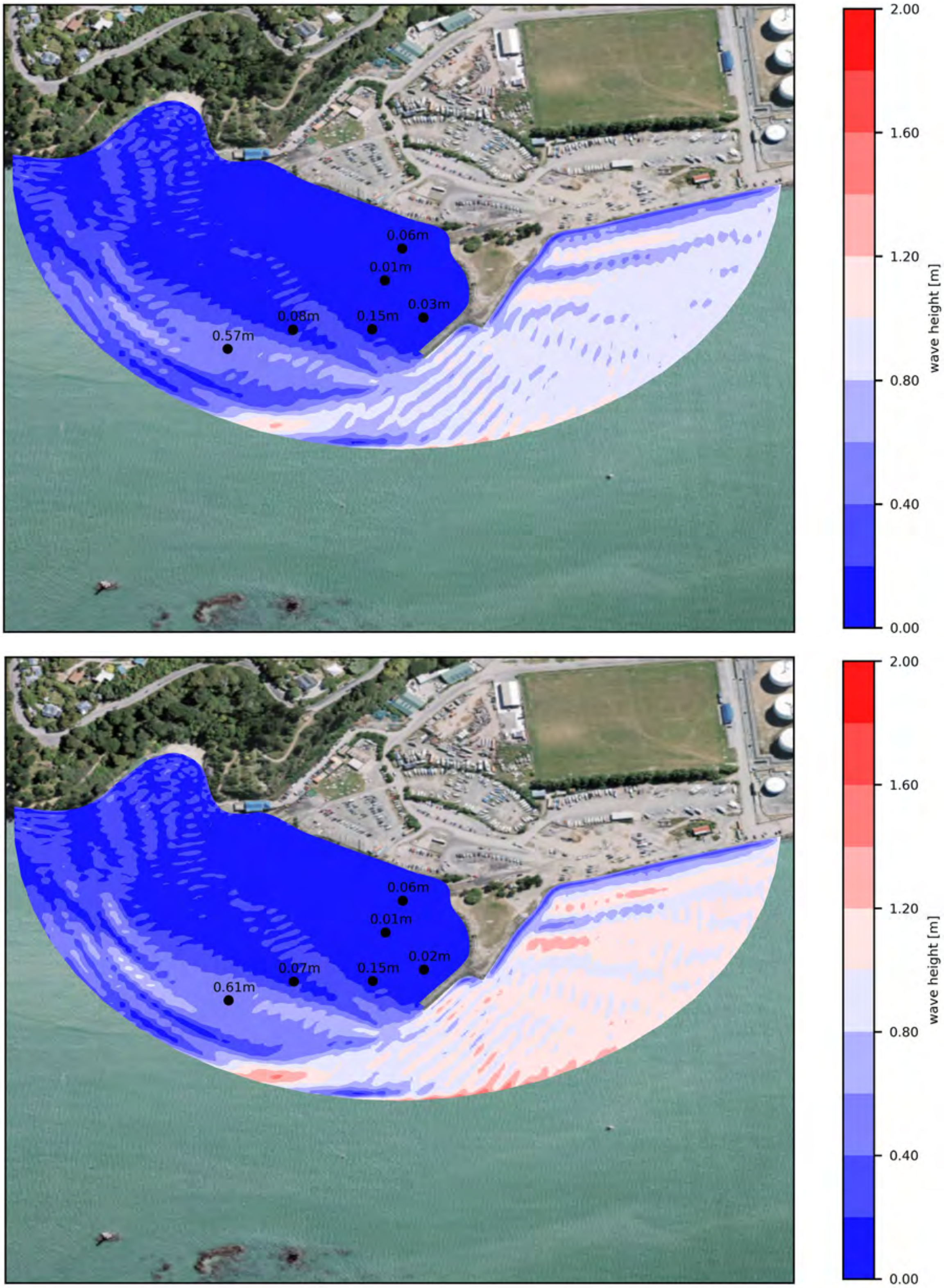

Figure 3.5 Wave height modelled for existing layout with Easterly waves: Hs=0.83m; Tp=4s (5-year return period, top panel) and Hs=0.98m, Tp=4s (50 year return period, bottom panel).

3.4.1.2 South Extremes

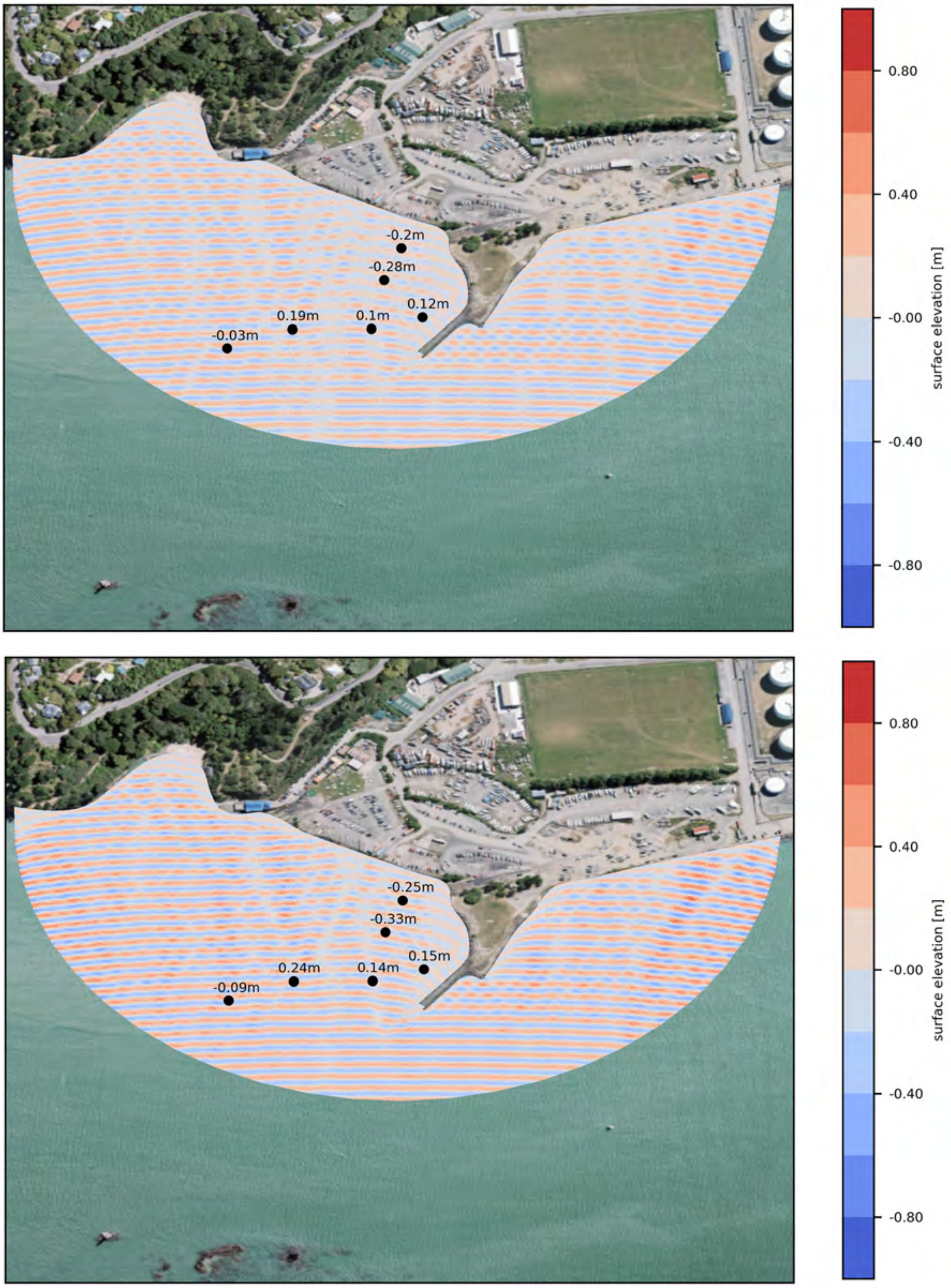

Figure 3.6 Surface elevation modelled for existing layout with Southerly waves: Hs=0.53m, Tp=3s (5-year return period, top panel) and Hs=0.72m, Tp=3s (50-year return period, bottom panel).

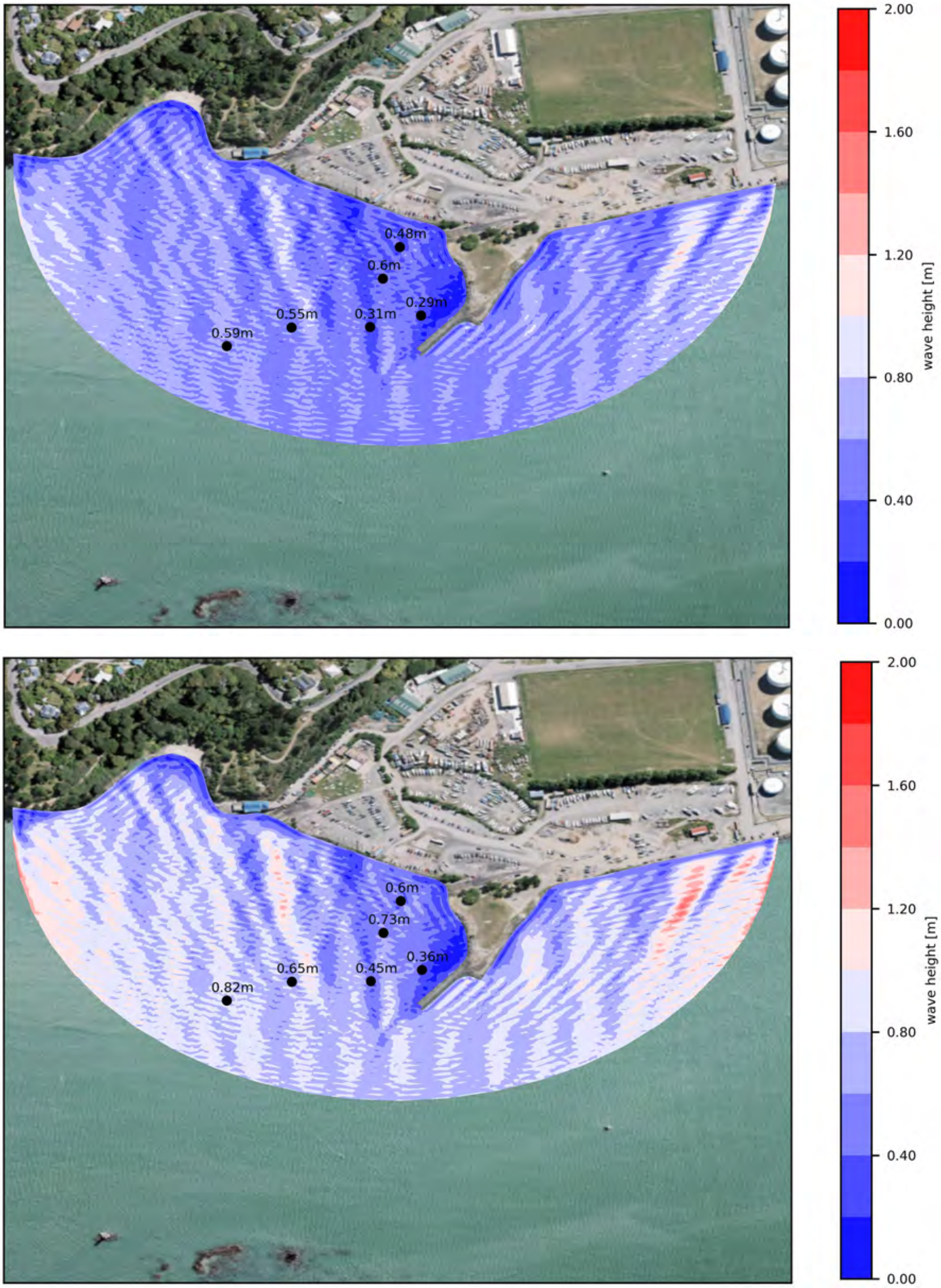

Figure 3.7 Wave height modelled for existing layout with Southerly waves: Hs=0.53m; Tp=3s (5-year return period, top panel) and Hs=0.72m, Tp=3s (50-year return period, bottom panel).

3.4.1.3 South-West extremes

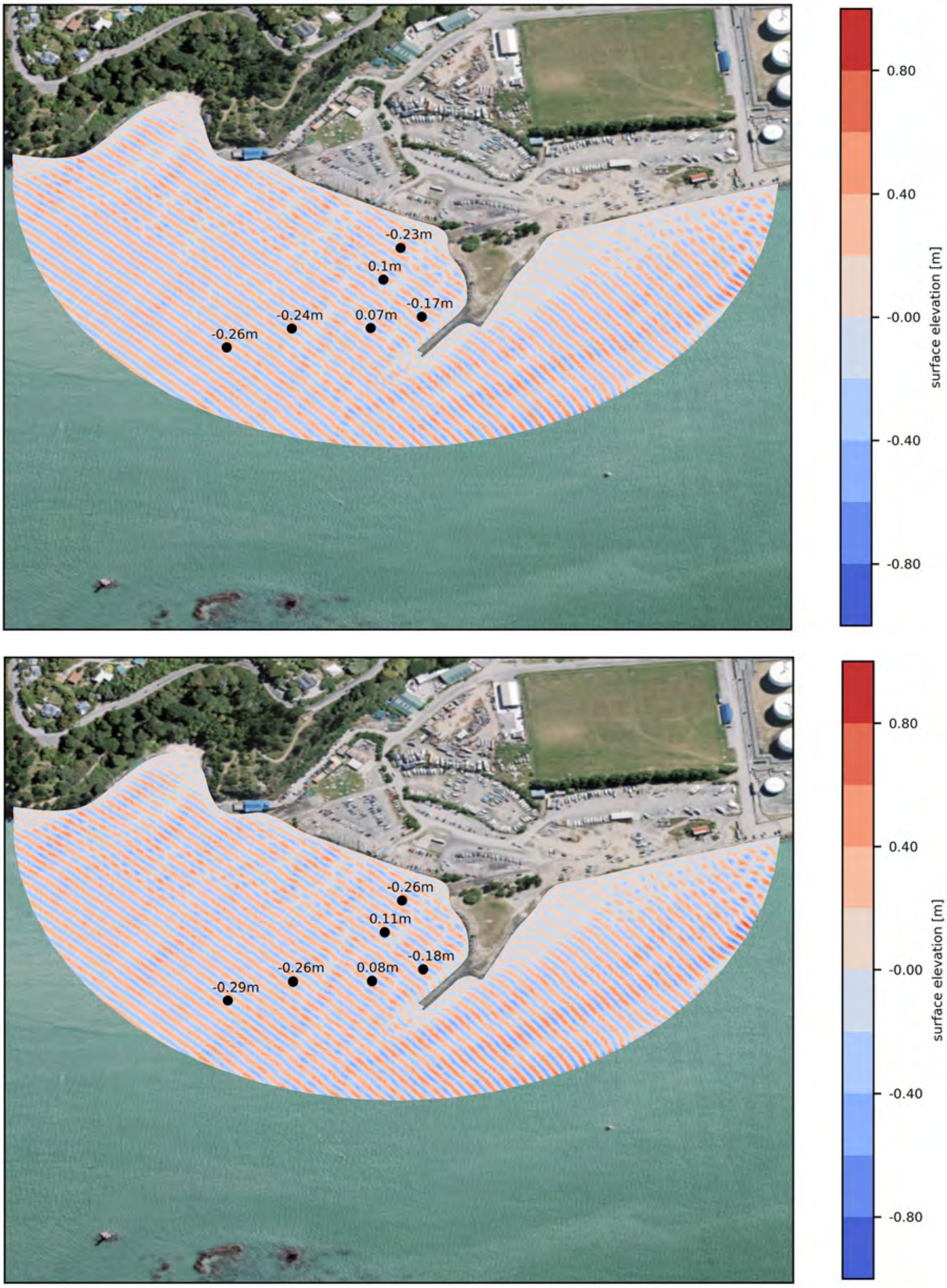

Figure 3.8 Surface elevation modelled for existing layout with South-westerly waves: Hs=0.73m; Tp=3s (5-year return period, top panel) and Hs=0.8m, Tp=3s (50-year return period, bottom panel).

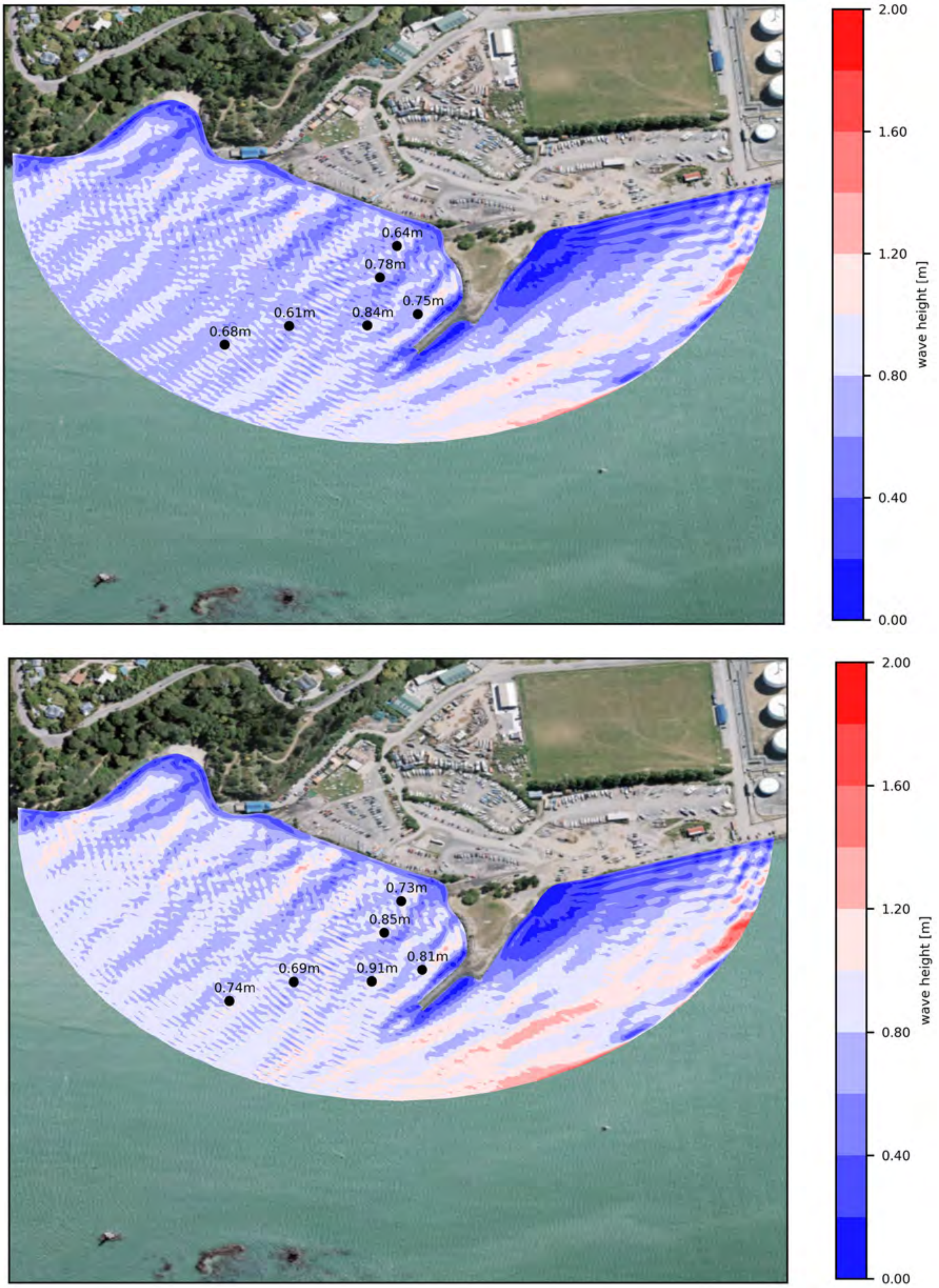

Figure 3.9 Surface elevation modelled for existing layout with South-westerly waves: Hs=0.73m; Tp=3s, (5-year return period, top panel) and Hs=0.8m, Tp=3s (50 year return period, bottom panel).

3.4.1.4 West extremes

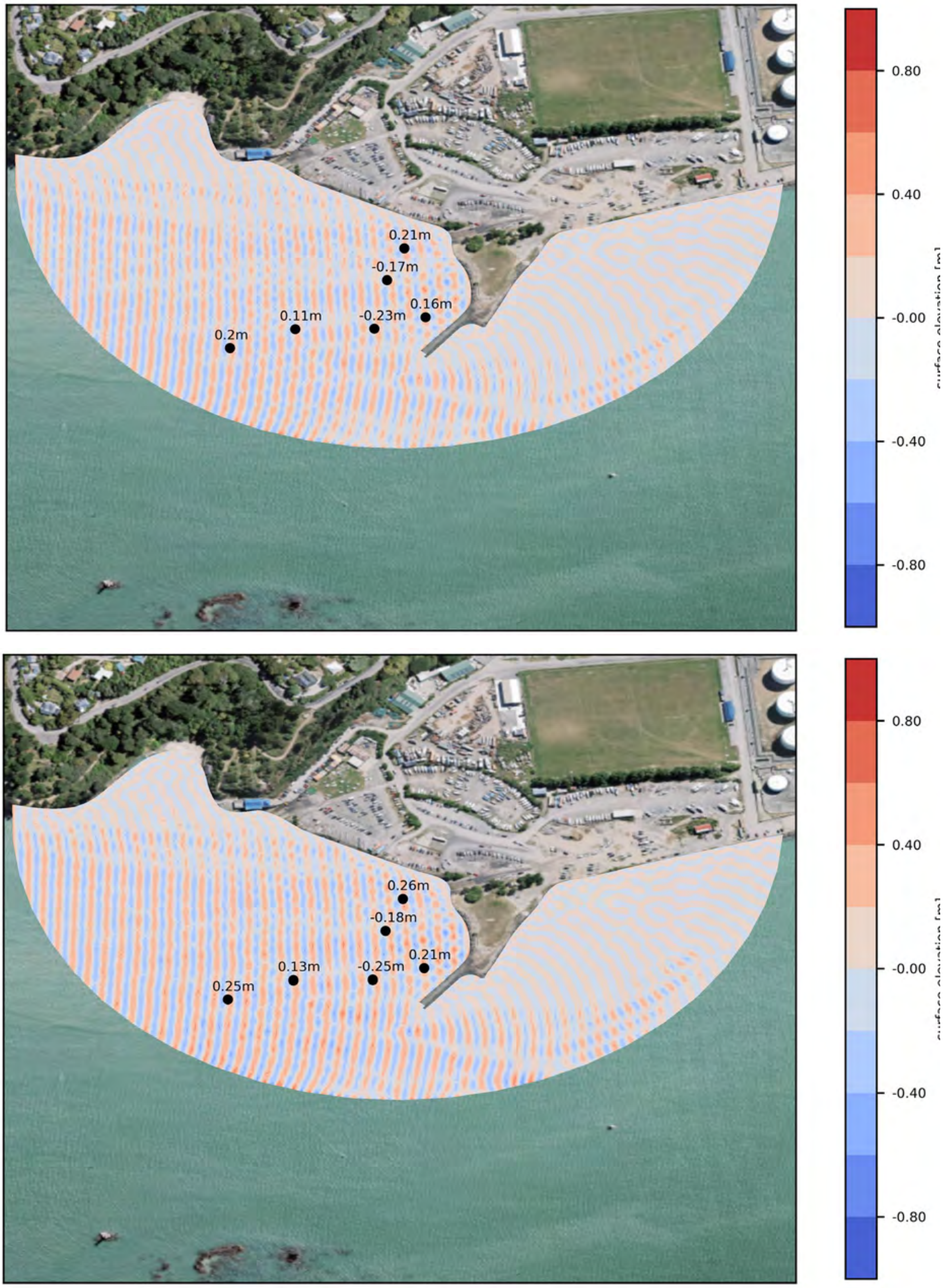

Figure 3.10 Surface elevation modelled for existing layout with Westerly waves: Hs=0.54m; Tp=3s (5 year return period, top panel) and Hs=0.65m, Tp=3s (50 year return period, bottom panel).

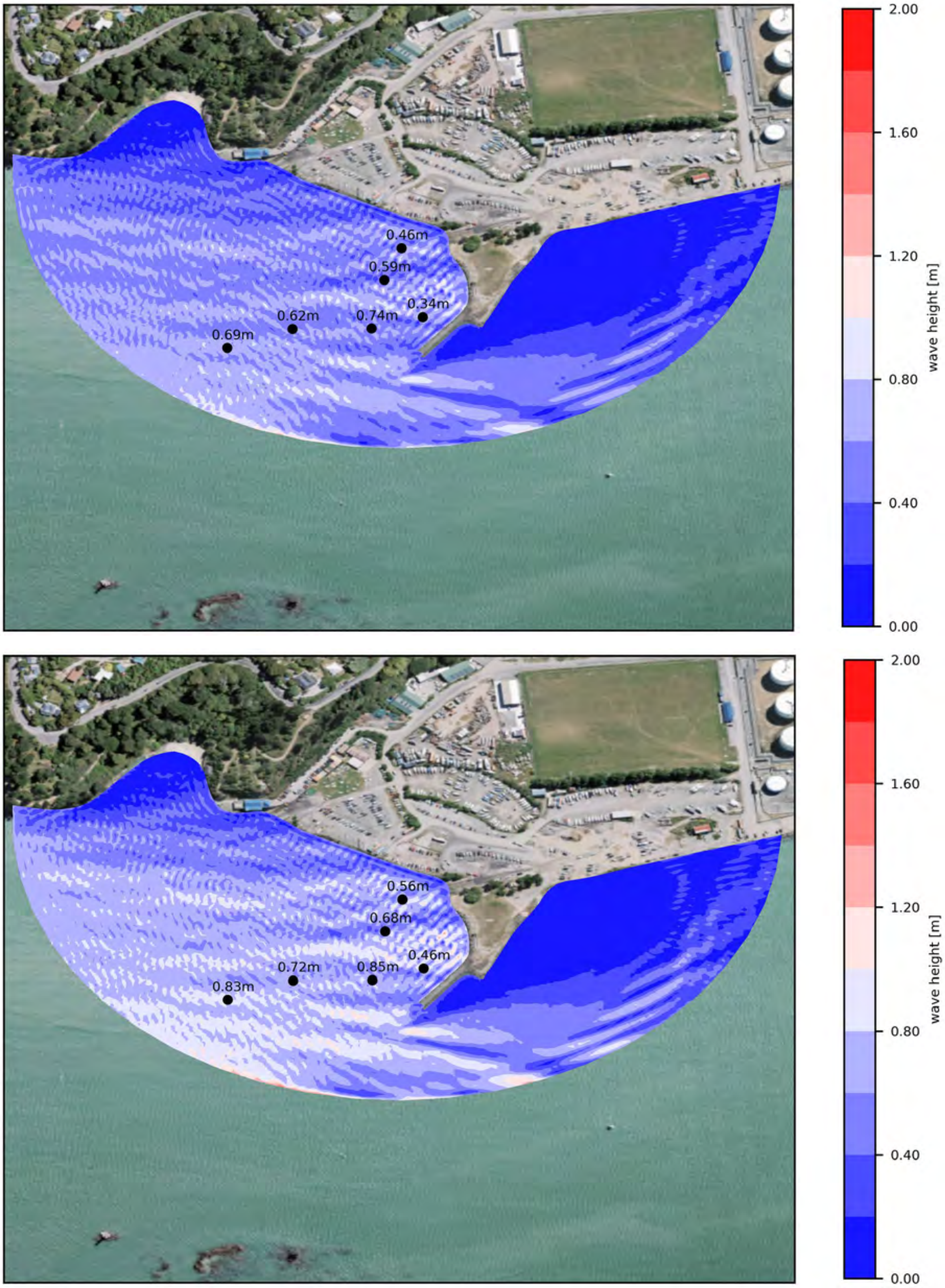

Figure 3.11 Wave height modelled for existing layout with Westerly waves: Hs=0.54m; Tp=3s (5 year return period, top panel) and Hs=0.65m, Tp=3s (50 year return period, bottom panel).

3.4.2 Design layout 1 - a wave attenuator

Results for the easterly, southerly, south-westerly and westerly 5year and 50-year Return period are shown on Figure 3.12 to Figure 3.19 and Table 3-4.

Based on the design wave criteria (Section 1.2), the existing layout only gives a good wave climate for Easterly wave conditions as the existing breakwater protects the harbour from wave penetration.

For this design layout, the proposed floating wave attenuator mainly attenuate waves from a south-westerly and westerly direction. For Southerly wave, modelling results show that wave height inside the harbour can be up to 0.6m based on 50-year return period. As shown on Figure 3.16 and Figure 3.17, the gap between the heads of the existing break water and the floating wave attenuator allow southerly waves to propagate into the harbour.

Overall the proposed floating attenuator reduce wave height below the design wave criteria for the 50-year RP at locations 6 except for the southerly waves. For the 5-year RP event the improvement in wave conditions within the harbour is minimal and wave height remains above the design criteria (0.15m).

Table 3-4:Significant wave height for the 5-year and 50-year Return Period at extracted points based on design layout 1. Green shading shows conditions below the design wave criteria of 0.15m (for the 5-year RP) and of 0.25m (50-year RP).

	5 -year return period						**50-year return period**					
Wave Direction	1	2	3	4	5	6	1	2	3	4	5	6
E	0.42	0.24	0.10	0.05	0.03	0.07	0.45	0.24	0.12	0.06	0.03	0.07
S	0.70	0.18	0.15	0.24	0.53	0.45	0.93	0.18	0.16	0.26	0.60	0.52
SW	0.61	0.31	0.35	0.39	0.42	0.17	0.68	0.32	0.35	0.40	0.41	0.18
W	0.60	0.29	0.31	0.20	0.30	0.16	0.73	0.29	0.35	0.21	0.34	0.20

3.4.2.1 Easterly Waves

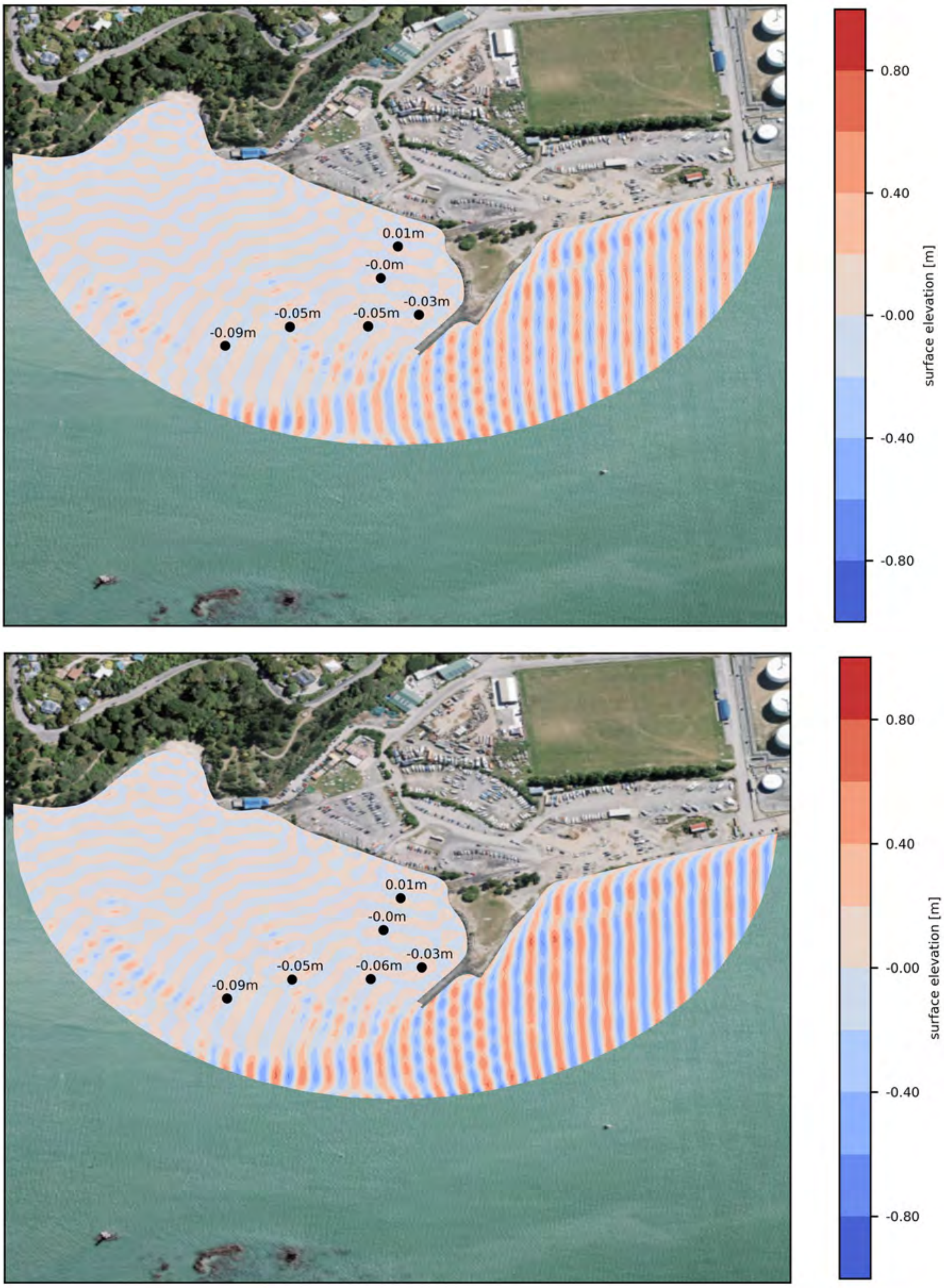

Figure 3.12 Surface elevation modelled for design layout 1 with Easterly waves: Hs=0.83m; Tp=4s (5 year return period, top panel) and Hs=0.98m, Tp=4s (50 year return period, bottom panel).

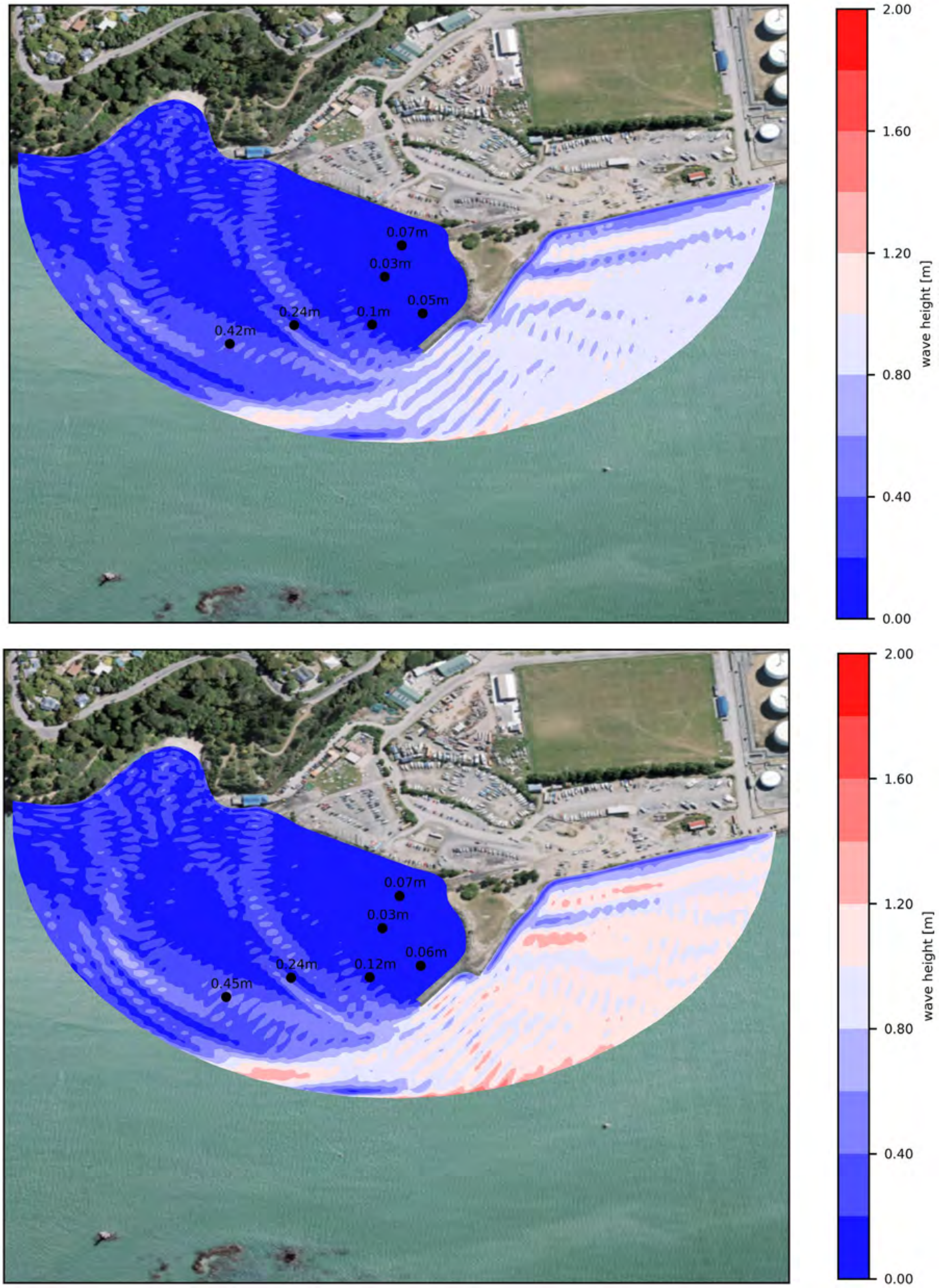

Figure 3.13 Wave height modelled for design layout 1 with Easterly waves: Hs=0.83m; Tp=4s (5 year return period, top panel) and Hs=0.98m, Tp=4s (50 year return period, bottom panel).

3.4.2.2 Southerly extremes

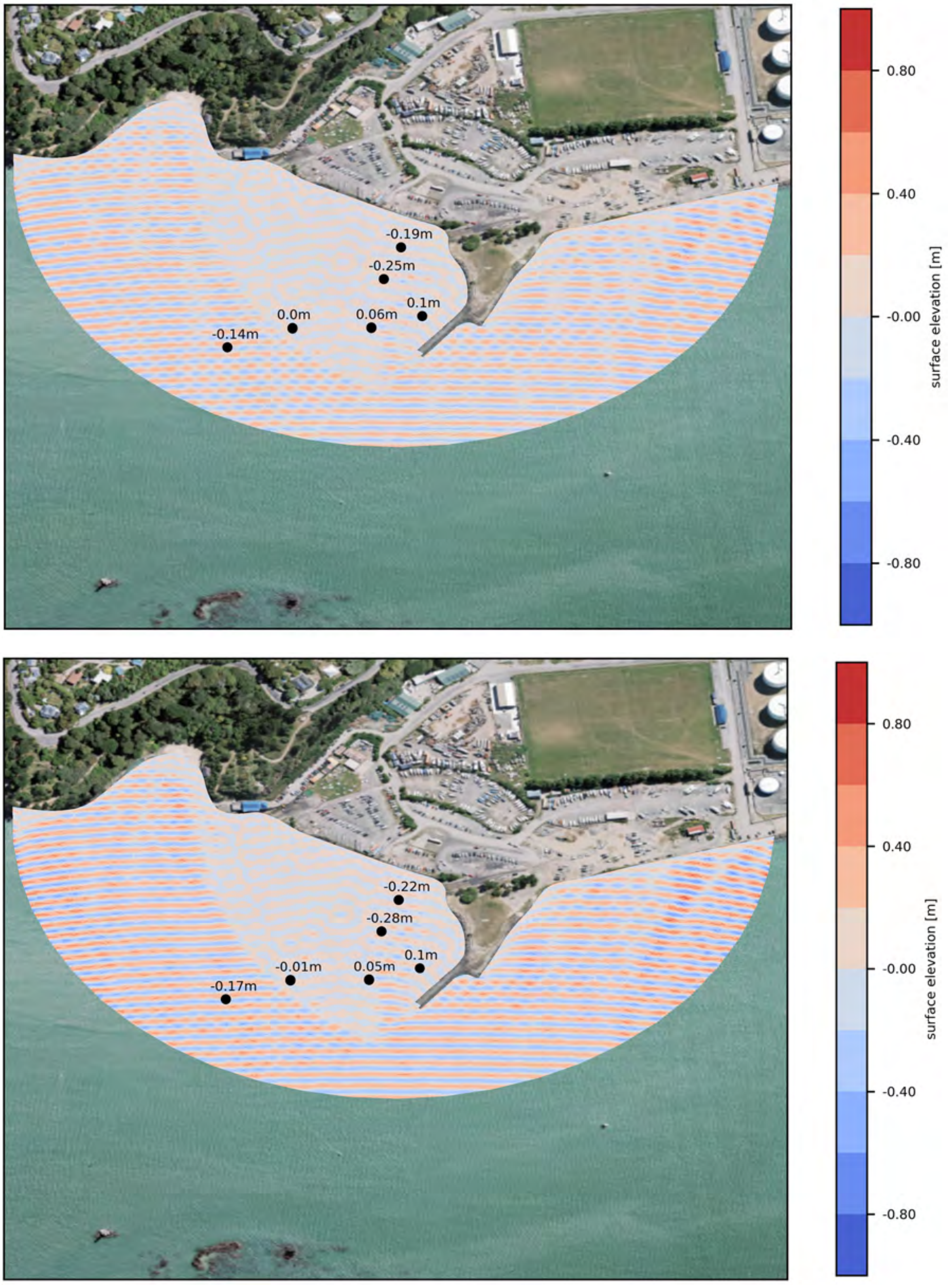

Figure 3.14 Surface elevation modelled for design layout 1 with Southerly waves: Hs=0.53m; Tp=3s (5 year return period, top panel) and Hs=0.72m, Tp=3s (50 year return period, bottom panel).

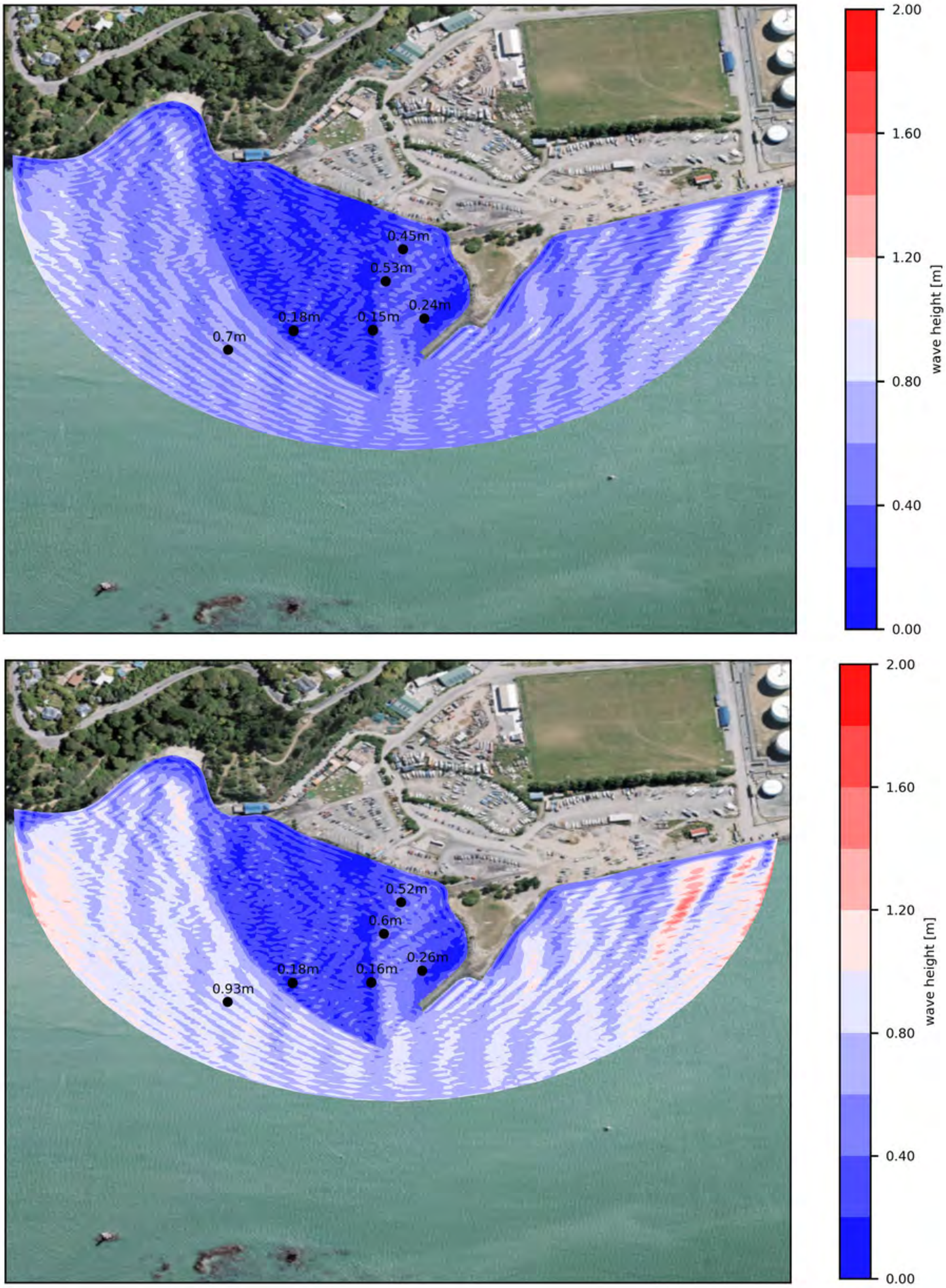

Figure 3.15 Surface elevation modelled for design layout 1 with Southerly waves: Hs=0.53m; Tp=3s (5 year return period, top panel) and Hs=0.72m, Tp=3s (50 year return period, bottom panel).

3.4.2.3 South-Westerly Waves

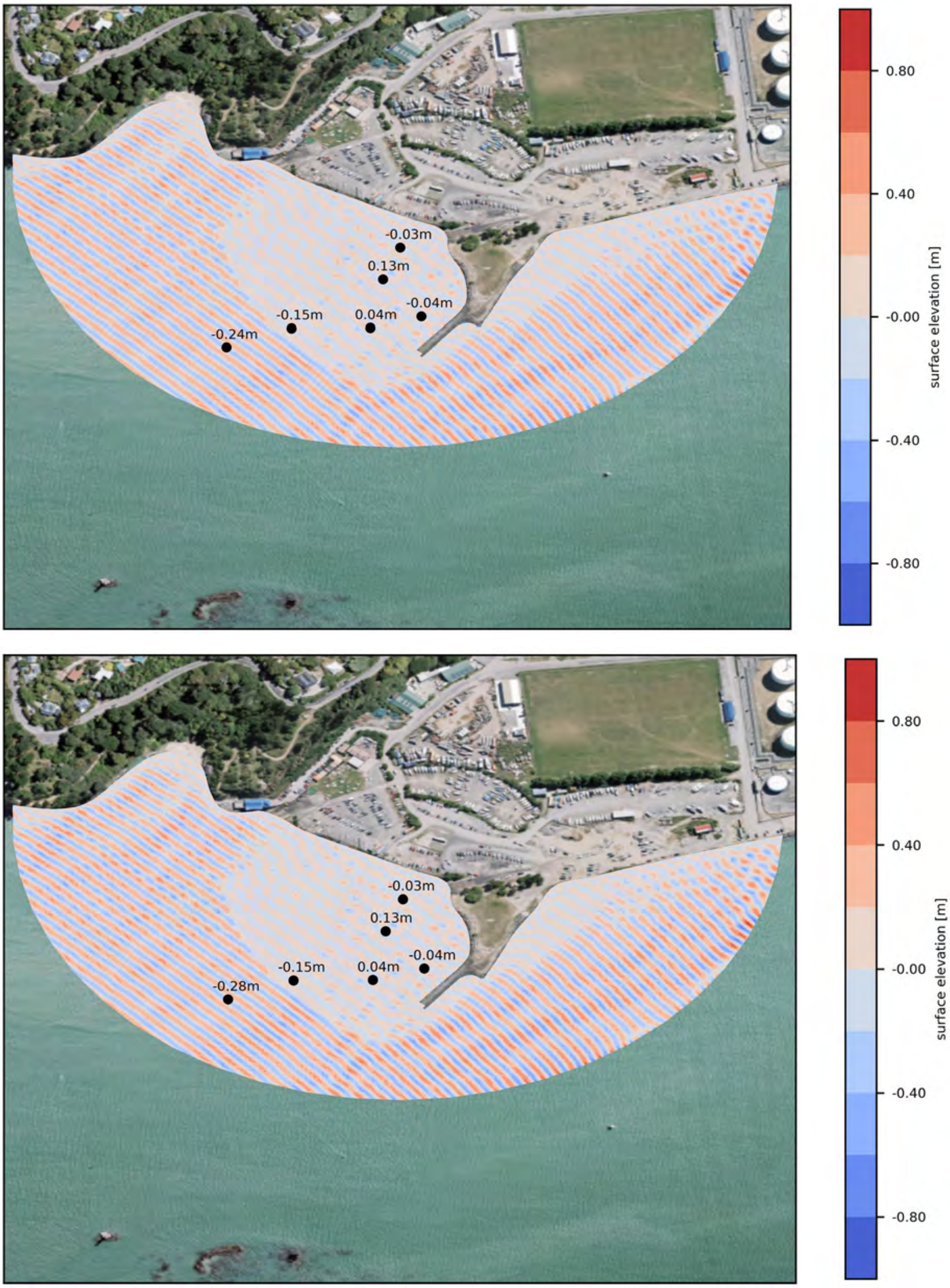

Figure 3.16 Surface elevation modelled for design layout 1 with South-westerly waves: Hs=0.73m; Tp=3s (5 year return period, top panel) and Hs=0.80m, Tp=3s (50 year return period, bottom panel).

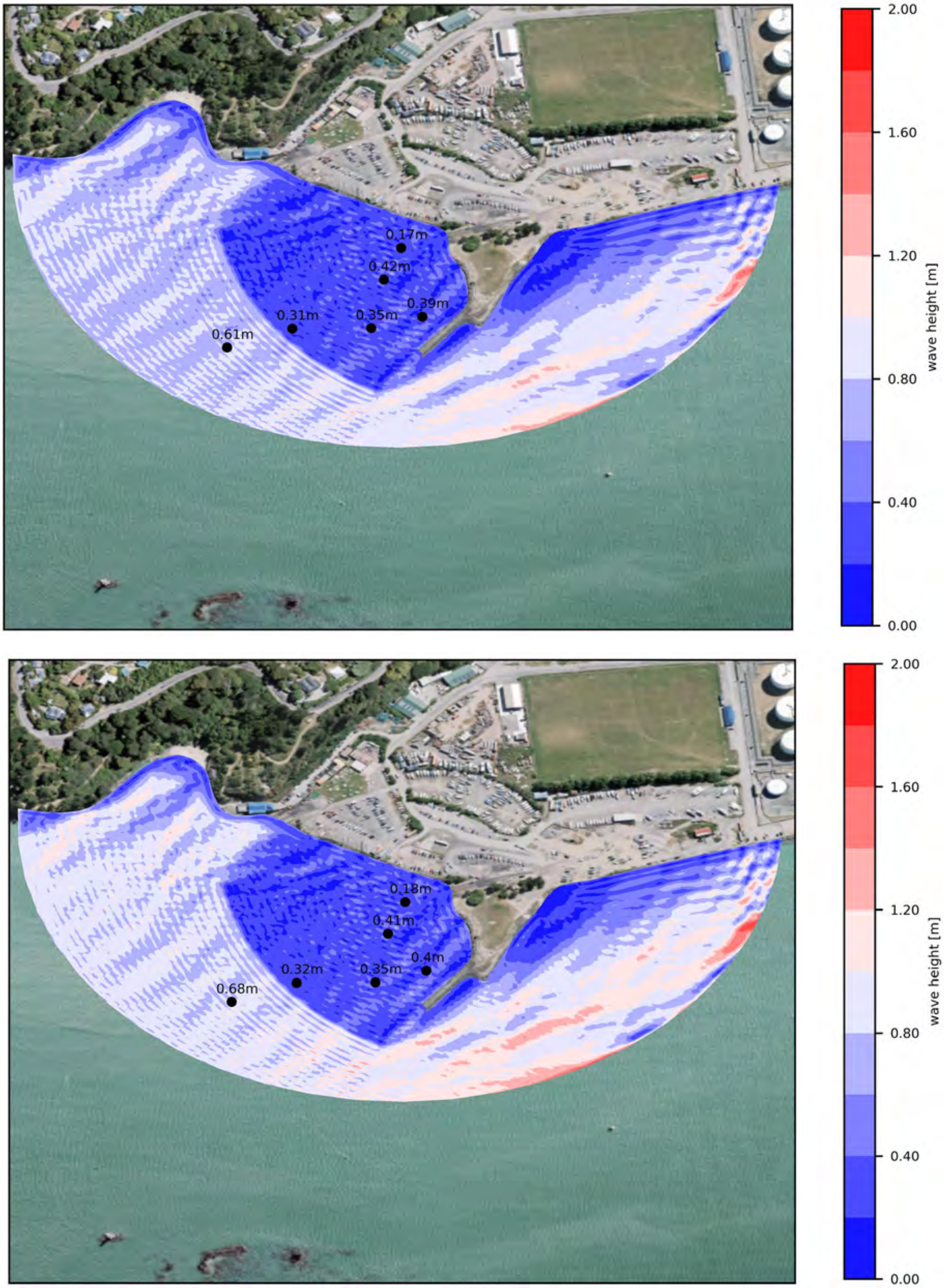

Figure 3.17 Wave height modelled for design layout 1 with South-westerly waves: Hs=0.73m; Tp=3s (5 year return period, top panel) and Hs=0.80m, Tp=3s (50 year return period, bottom panel).

3.4.2.4 Westerly Waves

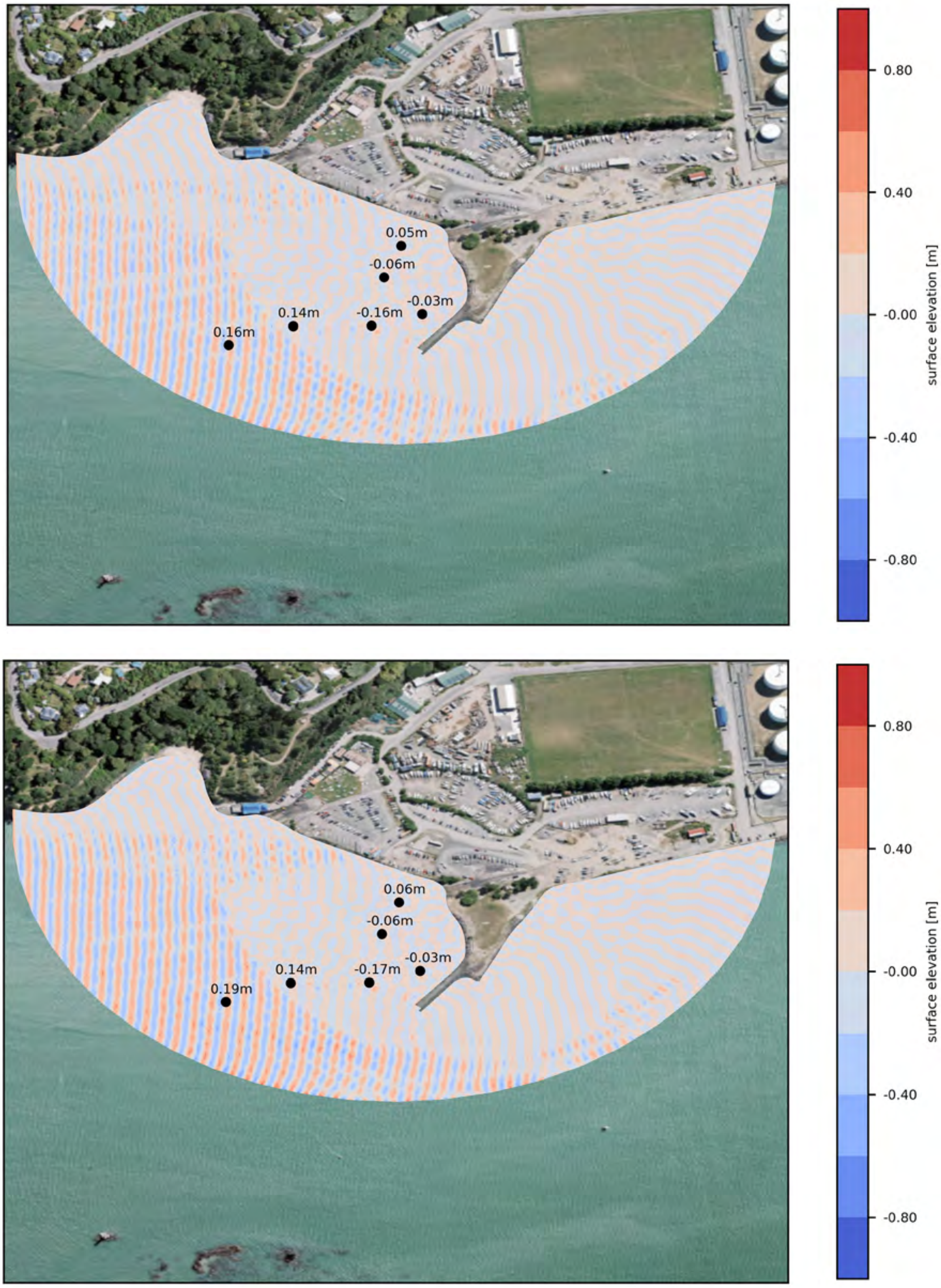

Figure 3.18 Surface elevation modelled for design layout 1 with Westerly waves: Hs=0.54m; Tp=3s (5 year return period, top panel) and Hs=0.65m, Tp=3s (50 year return period, bottom panel).

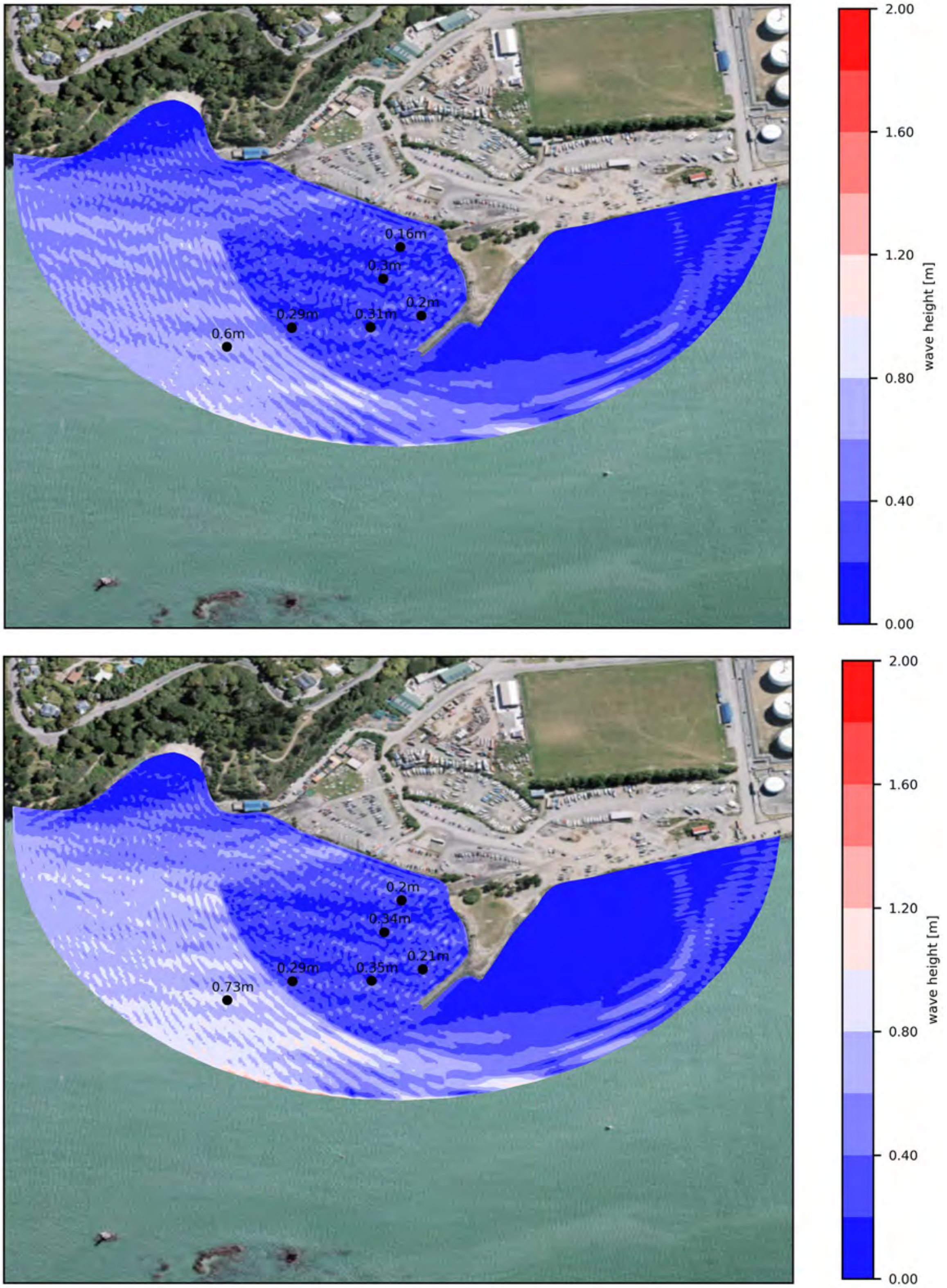

Figure 3.19 Surface elevation modelled for design layout 1 with Westerly waves: Hs=0.54m; Tp=3s (5 year return period, top panel) and Hs=0.65m, Tp=3s (50 year return period, bottom panel).

3.4.3 Design layout 2 - Breakwaters

Results for the easterly, southerly, south-westerly and westerly 5-year and 50-year Return period are shown on Figure 3.20 to Figure 3.26 and Table 3-5.

Overall, the two breakwaters configuration provide good wave protection for the inner harbour areas . However, it is noted that some wave penetration is occurring between two breakwaters' heads with waves from the south and south-west and most significantly from west direction (Figure 3.26).

Based on the design wave criteria (Section 1.2), the Breakwater Design Layout 2 layout provide a good wave climate for most wave conditions except Westerly waves.

Overall the proposed floating attenuator reduce wave height below the design wave criteria for the 50-year RP at locations 5 and 6 except for the westerly waves. For the 5-year RP event the wave conditions at locations 5 and 6 are below or very slightly above the criteria except for westerly waves.

Table 3-5:Significant wave height for the 5-year and 50-year Return Period at extracted points based on design layout 2. Green shading shows conditions below the design wave criteria of 0.15m (for the 5-year RP) and of 0.25m (50-year RP).

	5 -year return period						50-year return period					
Wave Direction	1	2	3	4	5	6	1	2	3	4	5	6
E	0.36	0.23	0.02	0.04	0.04	0.04	0.39	0.26	0.02	0.04	0.04	0.03
S	0.68	0.58	0.30	0.13	0.13	0.17	0.93	0.73	0.36	0.13	0.13	0.16
SW	0.68	0.61	0.67	0.46	0.20	0.06	0.73	0.68	0.74	0.47	0.20	0.06
W	0.75	0.58	0.44	0.18	0.26	0.26	0.91	0.69	0.53	0.26	0.26	0.29

3.4.3.1 Easterly Waves

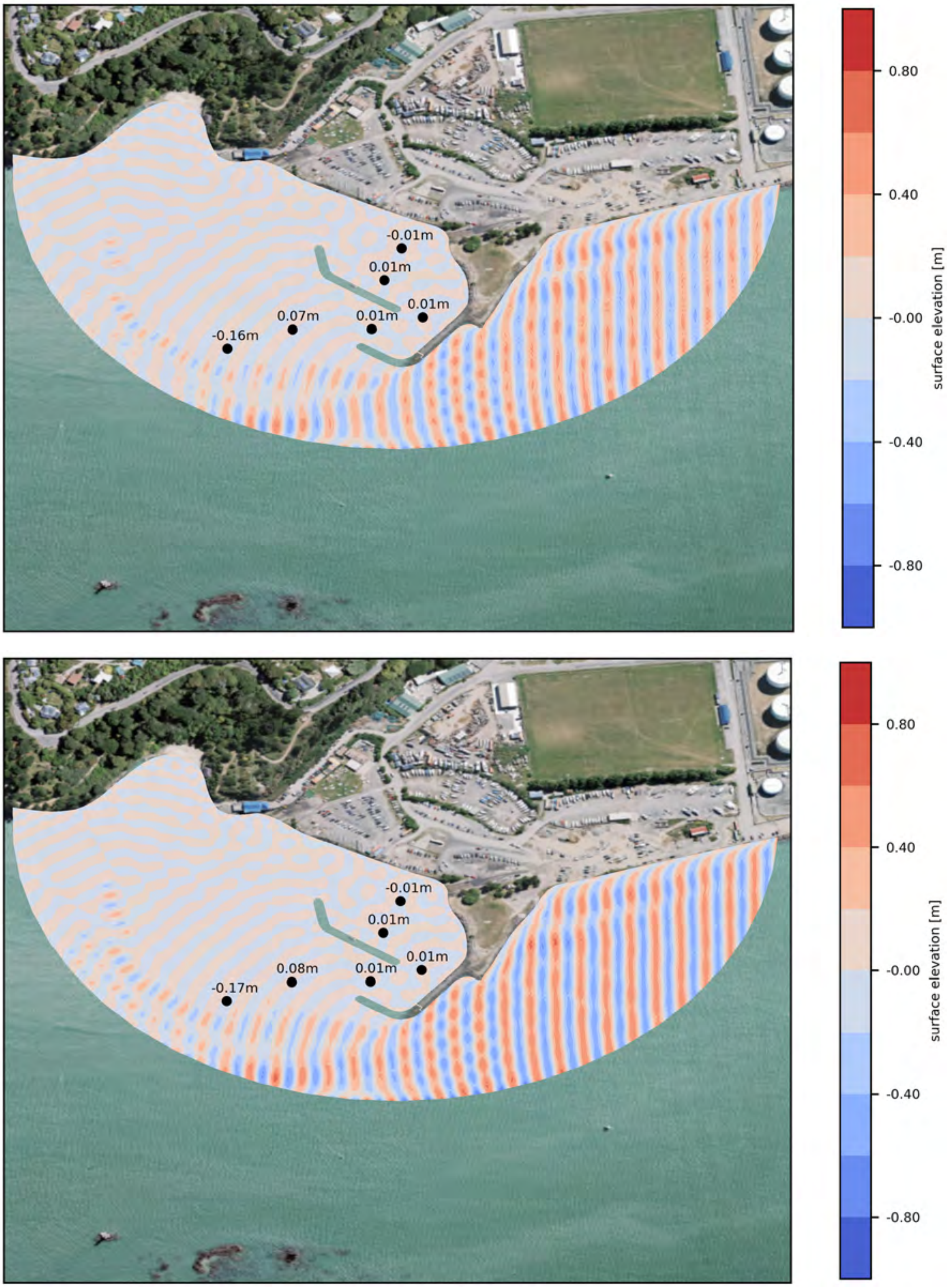

Figure 7.17 Surface elevation modelled for design layout 2 with Easterly waves: Hs=0.83m; Tp=4s (5-year return period, top panel) and Hs=0.98m, Tp=4s (50-year return period, bottom panel).

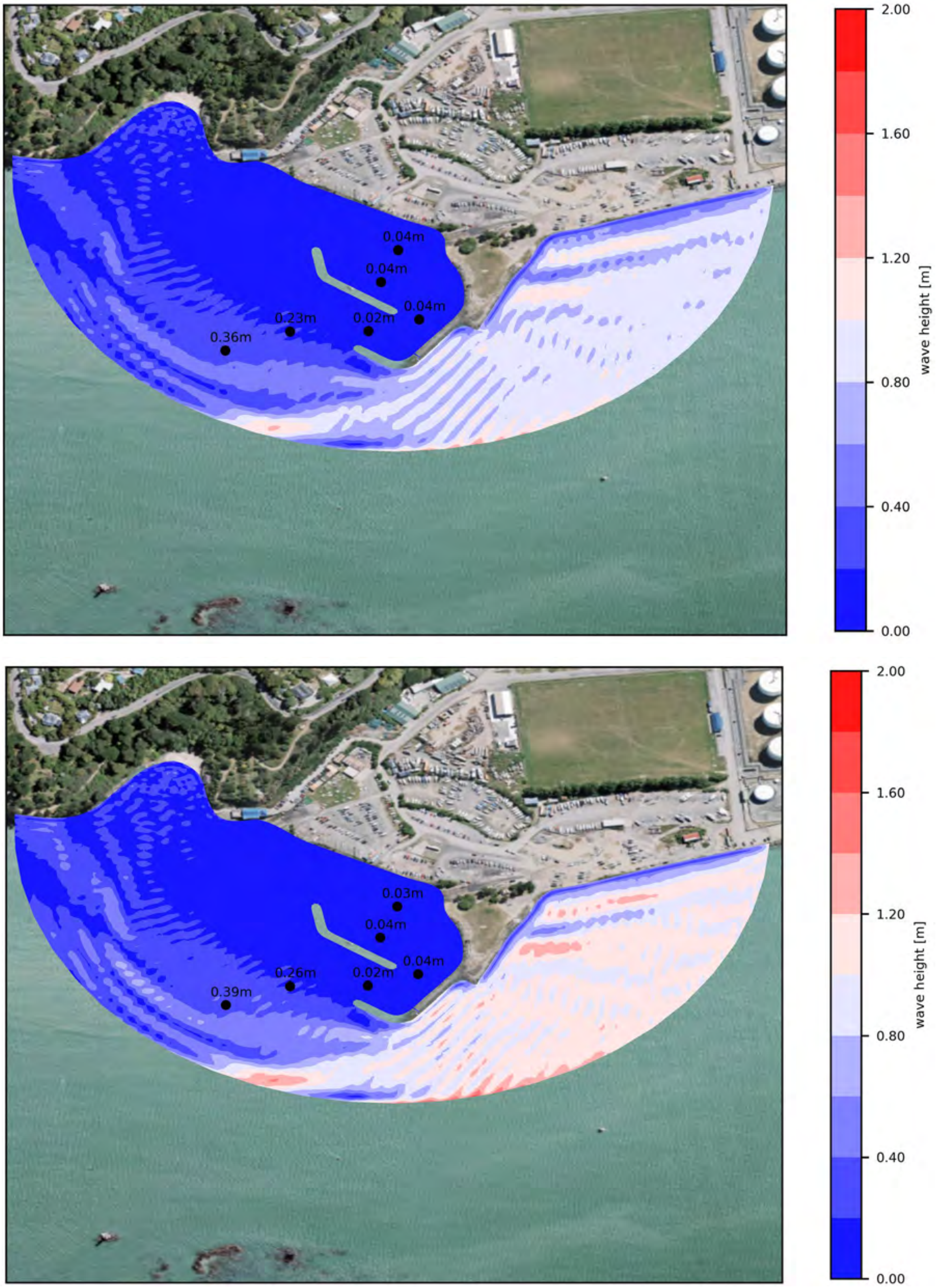

Figure 3.20 Wave height modelled for design layout 2 with Easterly waves: Hs=0.83m; Tp=4s (5 year return period, top panel) and Hs=0.98m, Tp=4s (50 year return period, bottom panel).

3.4.3.2 Southerly Waves

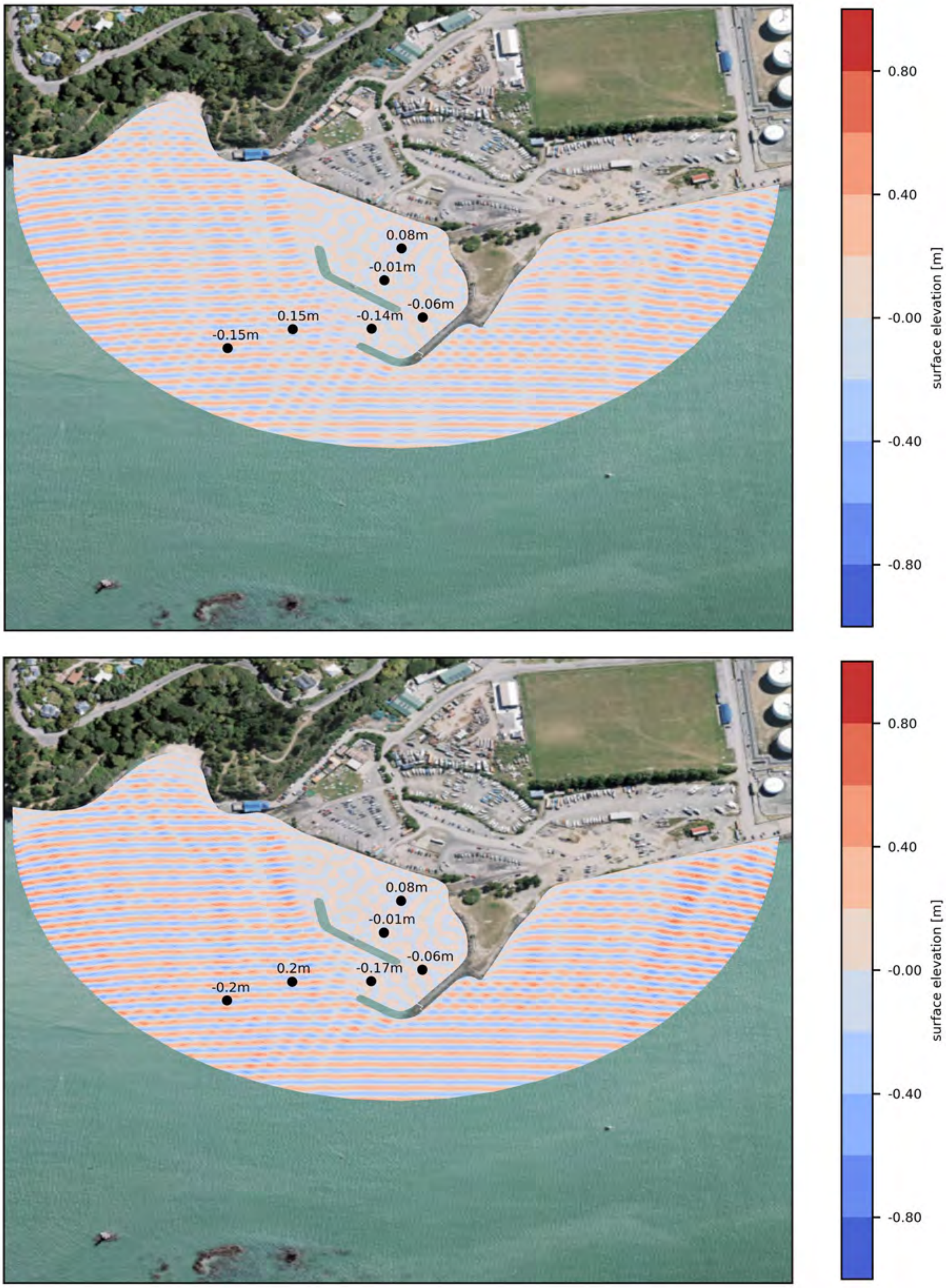

Figure 3.21 Surface elevation modelled for design layout 2 with Southerly waves: Hs=0.53m; Tp=3s (5 year return period, top panel) and Hs=0.72m, Tp=3s (50 year return period, bottom panel).

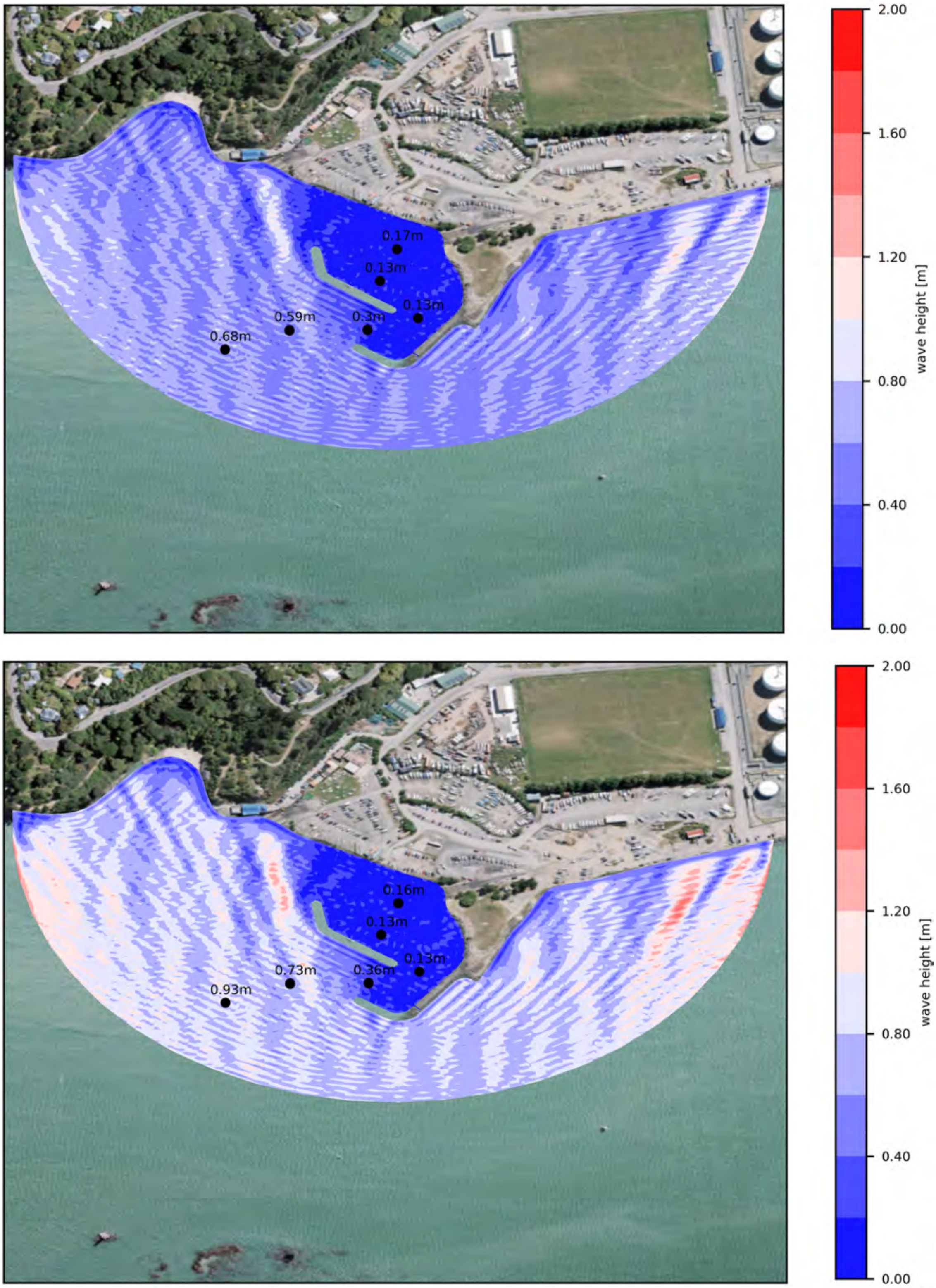

Figure 3.22 Wave height modelled for design layout 2 with Southerly waves: Hs=0.53m; Tp=3s (5 year return period, top panel) and Hs=0.72m, Tp=3s (50 year return period, bottom panel).

3.4.3.3 South-Westerly Waves

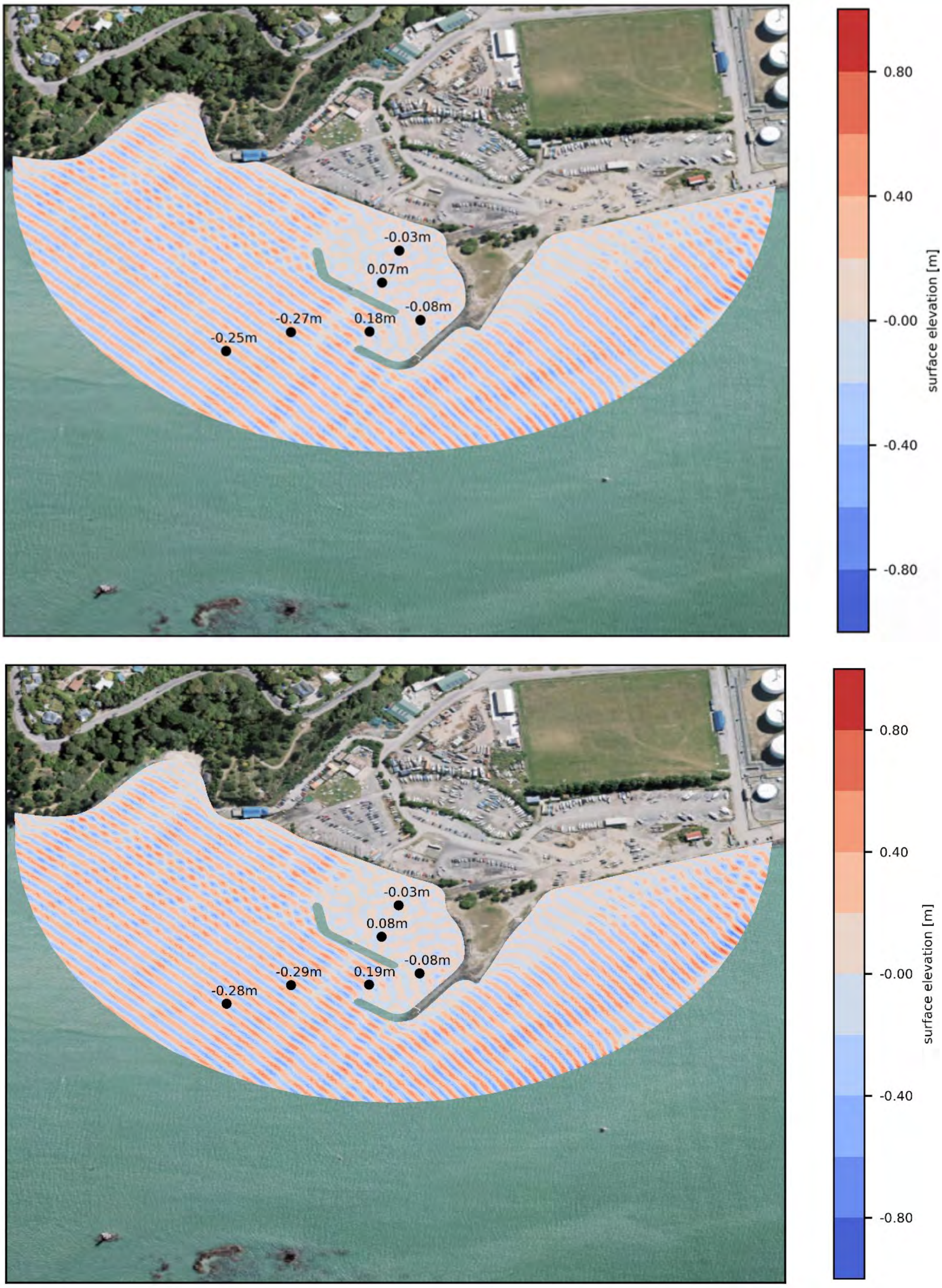

Figure 3.23 Surface elevation modelled for design layout 2 with South-westerly waves: Hs=0.73m; Tp=3s (5 year return period, top panel) and Hs=0.80m, Tp=3s (50 year return period, bottom panel).

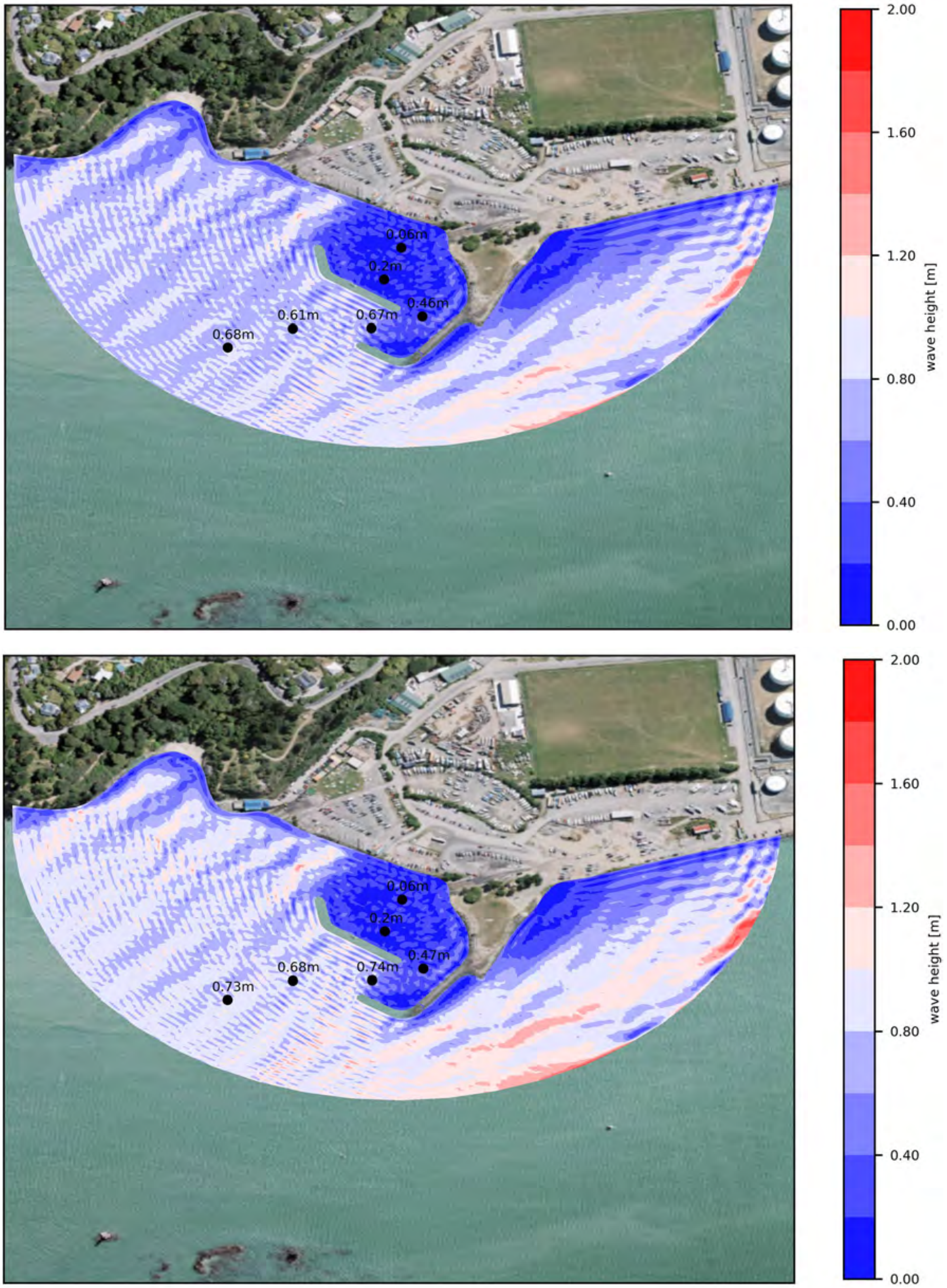

Figure 3.24 Wave height modelled for design layout 2 with South-westerly waves: Hs=0.73m; Tp=3s (5 year return period, top panel) and Hs=0.80m, Tp=3s (50 year return period, bottom panel),

3.4.3.4 Westerly Waves

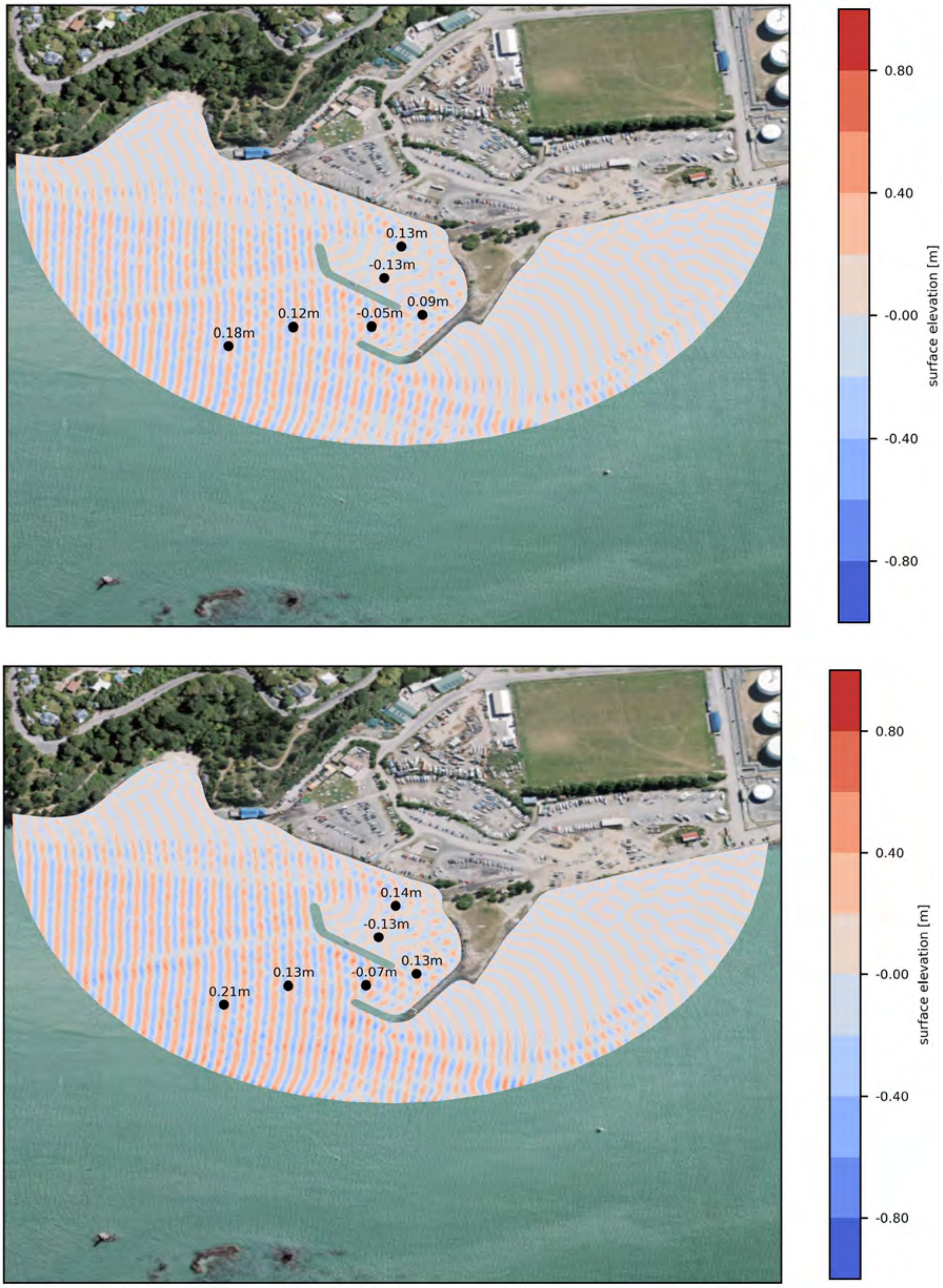

Figure 3.25 Surface elevation modelled for design layout 2 with Westerly waves: Hs=0.54m; Tp=3s (5 year return period, top panel) and Hs=0.65m, Tp=3s (50 year return period, bottom panel).

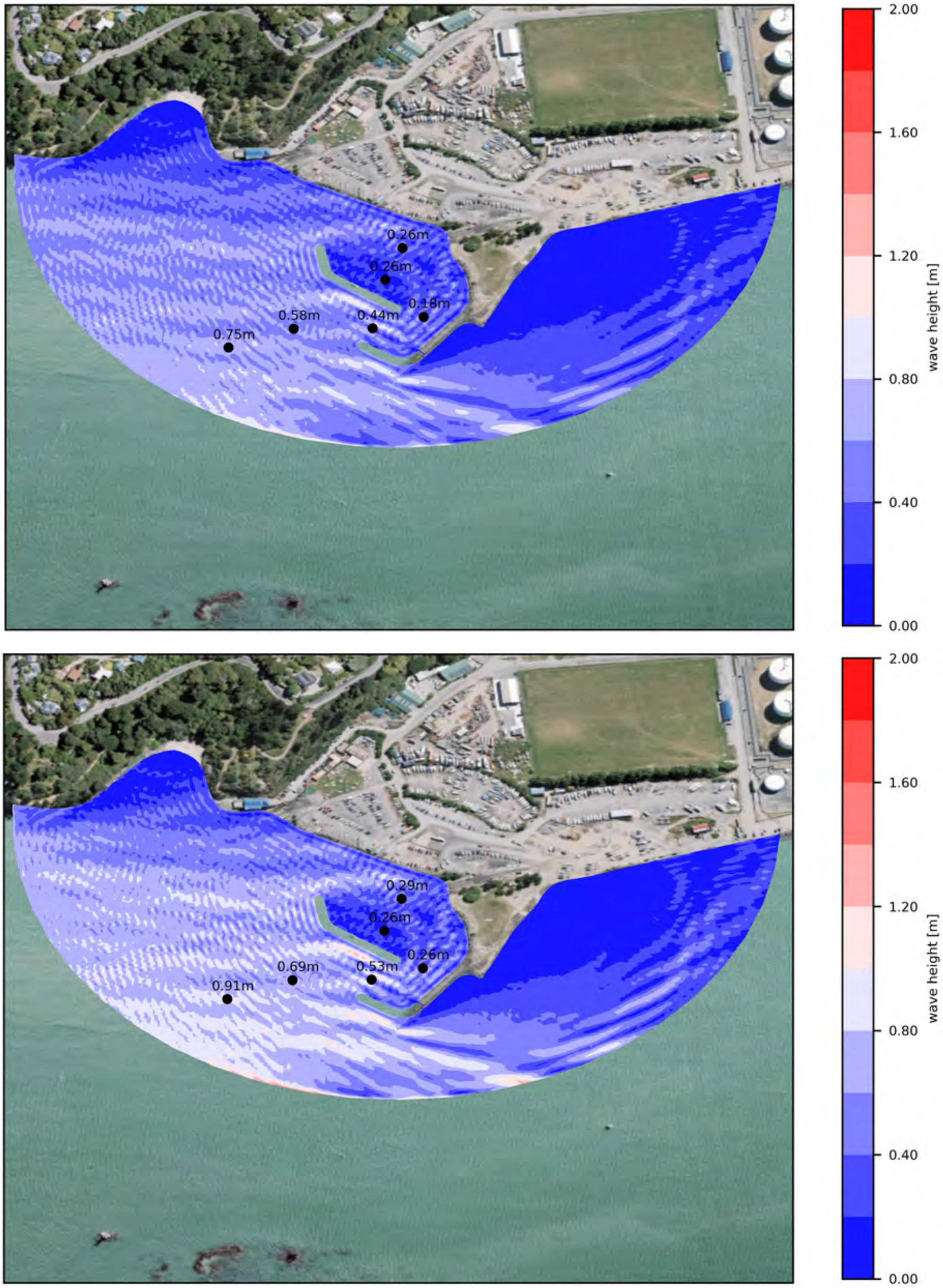

Figure 3.26 Wave height modelled for design layout 2 with Westerly wave s: Hs=0.54m; Tp=3s (50 year return period, top panel) and Hs=0.65m, Tp=3s (50 year return period, bottom panel).

4. Summary

This report presents an assessment of the wave climate near Naval Point harbour and investigation of the wave penetration with two proposed design layouts.

The proposed design wave criteria is based on wave conditions related to the vessel launching criteria at the Naval Point boat ramp, and is as follow:

- Wave height less than 0.15m for the 5-year Return Period
- Wave height less than 0.25m for the 50-year Return Period

A 10-year wave hindcast was prepared using a SWAN model nested approach and extreme values analysis was undertaken to determine the return period wave parameters at a location directly south of the existing breakwater. Extreme significant wave height for the 50-year Return Period are in the order 0.7m to 1.0m. Wave period are representative of locally generated wind wave with peak period of about 2 to 4 seconds. Longer period swell wave propagating from east are reaching the site however with small wave height typically less than 0.1m.

A wave penetration assessment using the CGWAVE phase-resolving wave model was undertaken for the existing layout and two proposed design layouts, i.e. a floating attenuator and a configuration with two breakwaters. Modelling scenarios were based on 5-year and 50-year wave return period from the east, south, south-west and west wave direction.

Results showed that :

- Wave for the existing layout are above the proposed design wave criteria except for waves coming from an easterly direction with existing breakwater providing sheltering.
- The proposed floating wave attenuator can significantly reduce wave height within the harbour, up to 50% at some locations. Near the boat ramp the wave design criteria is almost satisfied or satisfied for the 5-year and 50-year Return Period, respectively, except for southerly waves (with wave propagating between the head of the existing breakwater and the floating wave attenuator).
- The two-breakwater configuration is expected to provide the best wave protection. Near the boat ramp the wave design criteria is almost satisfied or satisfied for the 5-year and 50-year Return Period, respectively, except for westerly waves (with wave propagating between the coast and the proposed head breakwater)

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