Bronte SLSC 2D physical modelling

WRL TR 2024/16, July 2024

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UNSW Water Research Laboratory

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1 Introduction

Bronte Surf Life Saving Club (SLSC) in Sydney (NSW) is proposed for redevelopment. As part of this redevelopment, coastal protection structures will be constructed to protect the SLSC over its design life. The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney was engaged by Haskoning Australia (RHDHV) to undertake two-dimensional (2D) physical modelling of two seawall cross-section designs proposed for sections of the foreshore fronting Bronte SLSC.

An aerial photo of the project site is provided in Figure 1.1 and an overview of the seawalls to be modelled and tested is shown in Figure 1.2. Note that only two of the seawall cross-sections in Figure 1.2 were included in the physical model: Vertical Seawall with Wave Deflector (“Seawall 2”) and Stepped Seawall (“Seawall 1”). That is, “Seawall 3” was not included in the test program.

Physical modelling was used to confirm design characteristics such as overtopping and wave loading for the coastal structures associated with the proposed works.

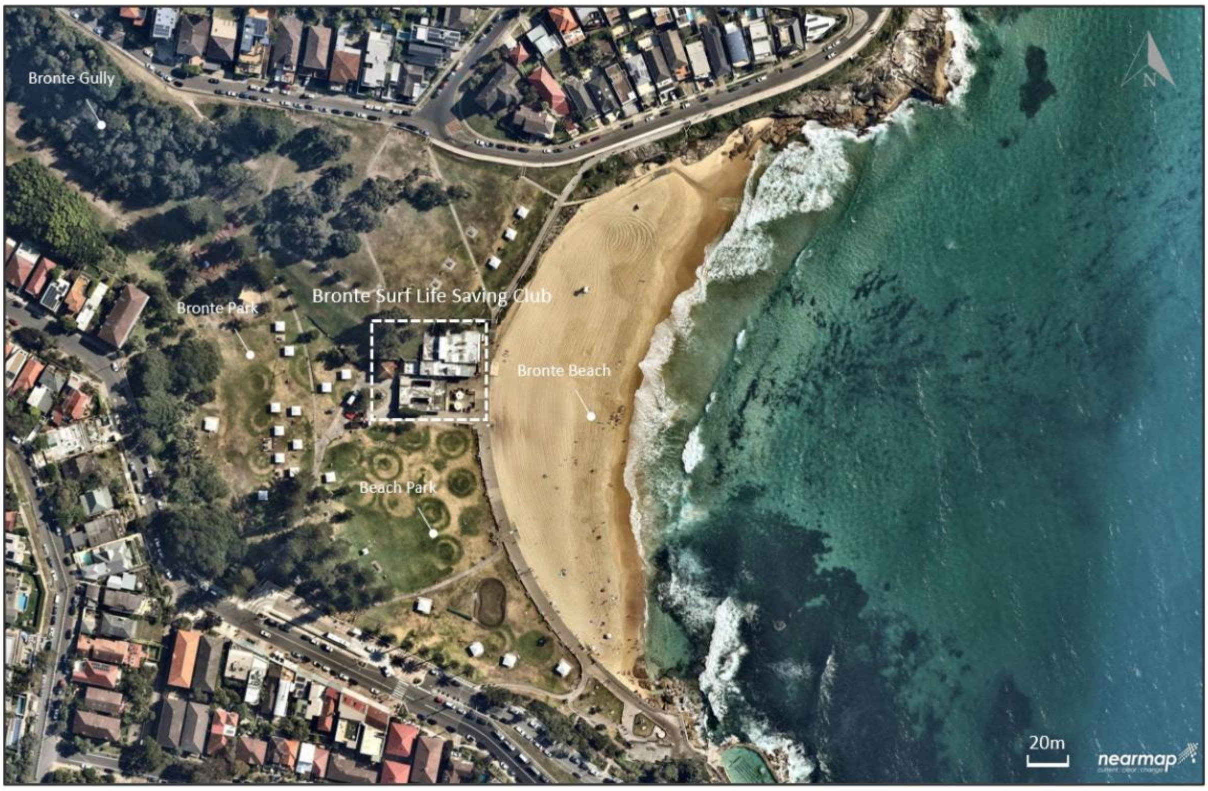


Figure 1.1 Aerial photo of the project site (Nearmap 03 October 2023) [Source: RHDHV, 2023]

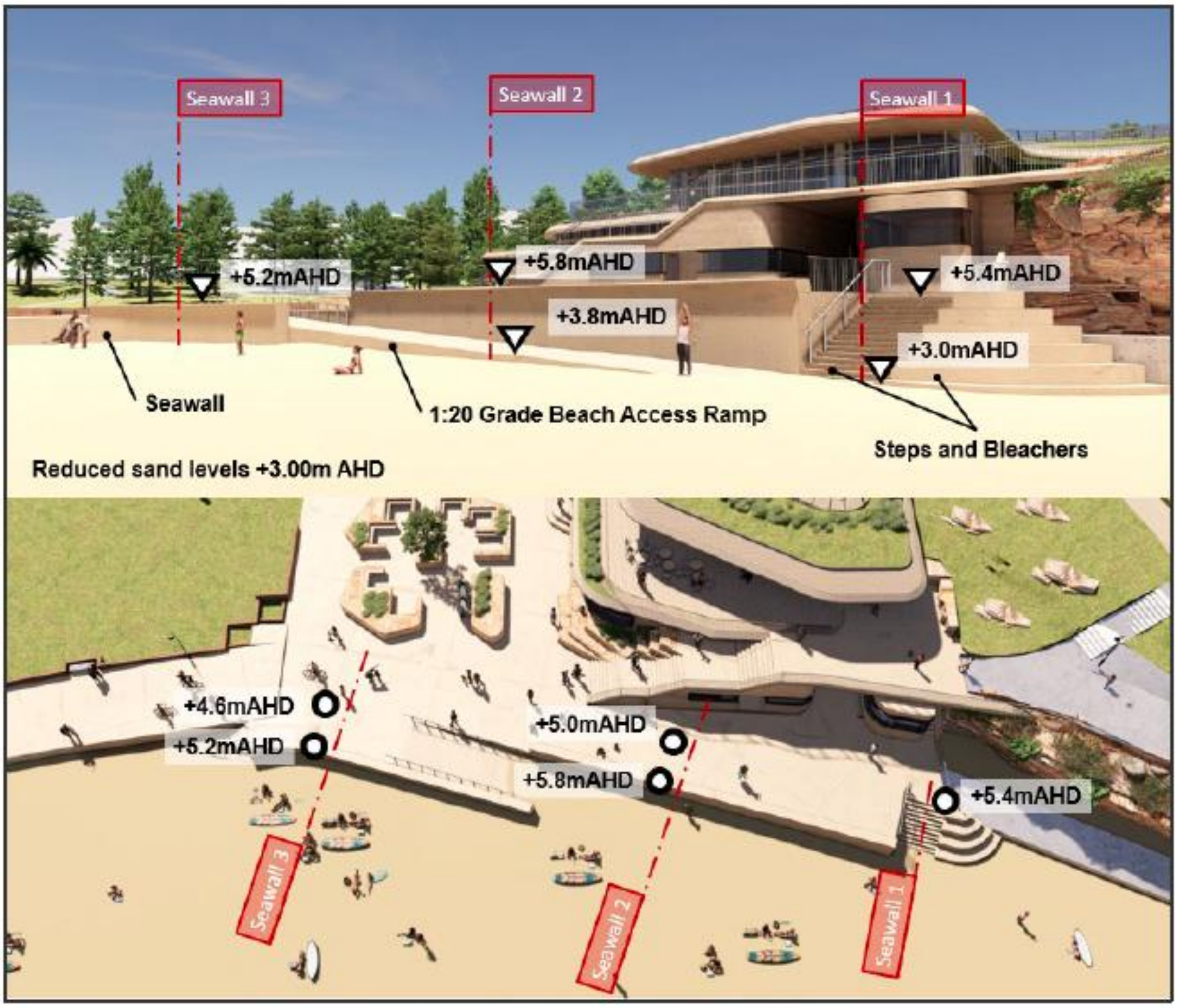


Figure 1.2 Overview of two seawalls to be modelled and tested: Vertical Seawall with Wave Deflector ("Seawall 2") and Stepped Seawall ("Seawall 1") [Source: RHDHV, 2023]

2 Study objectives

WRL and RHDHV developed a physical modelling program to assess wave overtopping and design wave loading behaviour for the site, previously described using desktop methods and reported in RHDHV's Concept Design and Coastal Engineering Assessment Report (RHDHV, 2024a).

The adopted physical modelling approach focused on wave overtopping flow impacts on the SLSC precinct and, while ultimately not required during the program, this approach allowed potential modifications to the seawall wave deflector to be introduced and tested. Ultimate limit state wave impact loads were to be measured on the proposed wave deflector on top of "Seawall 2" and on a vertical wall offset landward, to inform the structural design for the wave deflector and the seaward ground floor walls of the proposed SLSC building.

The 2D physical modelling was undertaken by WRL in accordance with best practice international guidelines. The scope of the program was developed collaboratively between WRL and RHDHV to assist RHDHV to design an optimised seawall for the site. The primary objective of the physical modelling was to measure overtopping rates for two seawall cross-sections fronting the Bronte SLSC, including:

- Vertical Seawall with Wave Deflector (denoted "Seawall 2" in Figure 1.2) – A vertical seawall with a wave deflector (deflector crest level 5.8 m AHD) and a 3.6 m wide access ramp located directly in front of the SLSC building (promenade level 5.20 m AHD).
- Stepped Seawall (denoted "Seawall 1" in Figure 1.2) – A stepped concrete seawall providing access to the concrete promenade (promenade level 5.31 m AHD) fronting the SLSC building.

Overtopping testing of the two structures was conducted with representative accreted and eroded nearshore profiles (refer to Section 3.3.1).

For the Vertical Seawall with Wave Deflector, WRL also conducted wave load testing on the wave deflector and SLSC building wall (with an eroded nearshore profile).

Unless otherwise stated, all dimensions in this report are stated in prototype (real-world) units.

3 Model setup and operation

3.1 Testing facility

The physical modelling program was carried out in WRL's 0.9 m wide wave flume. The flume's dimensions are 36 m (length) by 0.9 m (width) by 1.6 m (height). The flume walls are primarily constructed of rendered and painted blockwork, with the exception of a glass panelled section through which visual observations can be made. The permanent floor of the flume is constructed of concrete. A false floor constructed from plywood was used to represent the model bathymetry (see Section 3.3.1). A figure of the complete flume setup with dimensions is provided in Appendix A.

The flume has a piston type wave generator powered by an electric wave making system. This system is capable of generating both monochromatic and irregular wave spectra and custom user-defined wave time series or specific historical storms.

3.2 Model design and scaling

3.2.1 Overview

Model scaling was based on geometric similarity between the prototype (real world) and model with an undistorted length scale of 1:27. Selection of the length ratio was primarily based on the upper limit wave height able to be generated in the 0.9 m wave flume.

The scaling relationship between length and time was determined by Froudian similitude, with the relevant scale ratios (prototype divided by model) being adopted for the model, as shown in Table 3.1. Force had an additional scaling factor ($N_{\gamma w}$) to adjust for the ratio between the fluid densities in the prototype (salt water; 1024 kg/m^3) and the model (fresh water; 998 kg/m^3).

Table 3.1 Model scale ratio

Quantity	Unit	Froude relation	Scaling factor
Length	m	N_L	27
Time	s	$N_L^{0.5}$	5.20
Overtopping volume per unit length	L/m	N_L^2	729.0
Overtopping rate per unit length	L/s/m	$N_L^{1.5}$	140.3
Water Density	kg/m^3	$N_{\gamma w}$	1.026
Force per unit length	kN/m	$N_L^2 N_{\gamma w}$	748.0

3.2.2 Commentary on alternative scaling laws for force

Wave loads on vertical seawalls and their associated infrastructure can be divided between:

- Slowly acting loads, having durations of approximately 0.2 to 0.5 times a wave period, which are referred to as “pulsating” or “quasi-static” loads and are generally associated with non-breaking waves; and
- Short duration (often closer to 0.01 times the mean wave period or less), high intensity loads, which are referred to as “impulsive” or “impact” loads and are generally associated with waves breaking directly on the structure which may entrap and compress an air pocket (Cuomo et al., 2010).

It is well accepted that “pulsating” or “quasi-static” loads can be scaled by the simple Froude relationships for force described in Table 3.1 with negligible scale effects (Cuomo et al., 2010). However, use of Froude scaling for “impulsive” loads may lead to over-estimation of force at prototype (real-world) scale and, unfortunately, a simple and reliable scaling relationship for short duration “impact” loads remains an unresolved problem which requires further research (HYDRALAB III, 2007).

Loading due to breaking waves is difficult to predict and the underlying processes are poorly understood, in part, because the shape of individual waves at impact determines the way in which air between the structure and the approaching wave is expelled, entrapped and/or entrained, which then influences the force generated (Bullock et al., 2004; 2007). If a wave overturns as it strikes a seawall, it can trap an air pocket, or if the wave has already broken, large quantities of air can be entrained so that a turbulent air-water mixture strikes a seawall. In both cases, the compressibility of the trapped or entrained air will affect the dynamics.

In a scale model, the compressibility of air is far less significant than in the prototype (real-world) since the increases in pressure above atmospheric are so much lower. Bullock et al. (2001) also found that model tests using fresh water waves entrained less air than salt water waves with similar geometry, resulting in comparatively higher peak impact pressures and shorter pressure rise times with fresh water. Since a two-phase fluid with greater air content is more compressible, it has been argued that impact pressures generated by salt water ocean waves will be lower than those predicted by Froude scaling of fresh water, scale laboratory experiments (Bullock et al., 2005). While entrained air content is less in physical models, the size of air bubbles is greater due to surface tension effects, making the extent of conservatism difficult to quantify (Hughes, 1993).

During the design storm events modelled for the Bronte seawall, individual waves generated both “pulsating” and “impulsive” vertical loads on the wave deflector and SLSC building wall. In the design of this model, WRL adopted the recommendations of key physical modelling guidelines (Hughes, 1993 and HYDRALAB III, 2007) for minimising scale effects on vertical seawall structures by maximising the model scale and the data acquisition sampling rates for force. While it is acknowledged that alternative scaling laws which provide less conservatism exist, WRL has universally adopted Froude scaling for wave-generated forces as it will provide conservative results for RHDHV’s subsequent structural design. For a process known to contain unresolved scientific uncertainties, we consider that this a reasonable application of the precautionary principle.

3.3 Model construction and setup

3.3.1 Bathymetry

A false floor was constructed in the wave flume from water-resistant plywood (orange line in Figure 3.2 and Figure 3.3) with the following characteristics representing an eroded profile (informed by survey data provided in RHDHV, 2024b):

- Flat bathymetry extending 17.4 m from the toe of the structure (excluding access ramp) at an elevation of 0.0 m AHD
- 1V:45H slope from 0.0 m AHD to -0.7 m AHD
- 1V:42H slope from -0.7 m AHD to -9.1 m AHD
- Seaward of -9.1 m AHD, the false floor had a transition slope of 1V:10H until intersecting the permanent flume floor at -23.0 m AHD, consistent with HYDRALAB III (2007) modelling guidelines
- 140.6 m of flat bathymetry at -23.0 m AHD in front of the wave paddle

The location of the bathymetry transect provided by RHDHV and modelled by WRL, and the location of the wave paddle relative to the structure, is shown below in Figure 3.1.

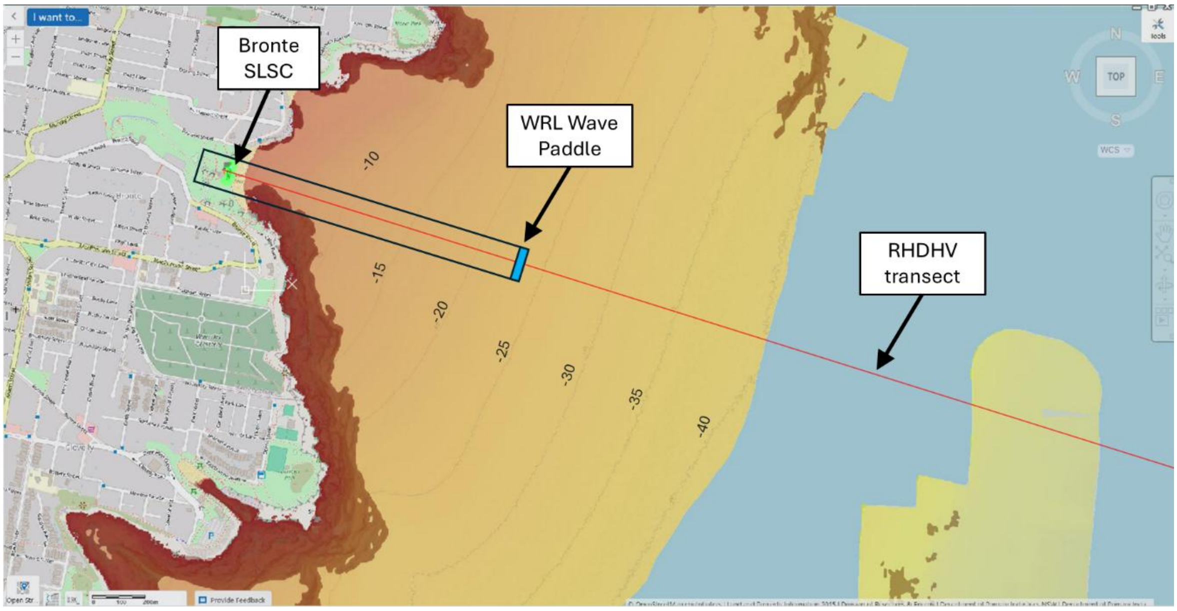


Figure 3.1 Location of the bathymetry transect (red line) provided by RHDHV and modelled by WRL, and the location of the wave paddle (blue rectangle) relative to the structure

This model bathymetry was representative of the site bathymetry for a distance of at least 4.6 wavelengths (approximately 400 m) seaward of the model seawall structure to -9.1 m AHD, which is in accordance with the minimum recommended value of 3 wavelengths by HYDRALAB III (2007).

The modelled bathymetry layout within the wave flume is presented in Figure 3.2 and a detailed view at the structure with the Vertical Seawall with Wave Deflector is provided in Figure 3.3. A complete view of the entire wave flume and bathymetry layout is provided in Appendix A.

To test the sensitivity of wave overtopping to the state of the beach at the peak of a storm, a representative accreted profile was also constructed seaward of the structures (as a modification of the base, eroded profile; green line in Figure 3.2 and Figure 3.3). This comprised a 1V:10H ramp profile extending from the eroded profile at -0.7 m AHD until intersecting with the structure at 4.2 m AHD for the Vertical Seawall with Wave Deflector and at 4.4 m AHD for the Stepped Seawall.

A 3 m long dissipative beach made out of reticulated foam was fitted across the back wall (landward end) of the flume to minimise reflections during wave climate calibration and testing.

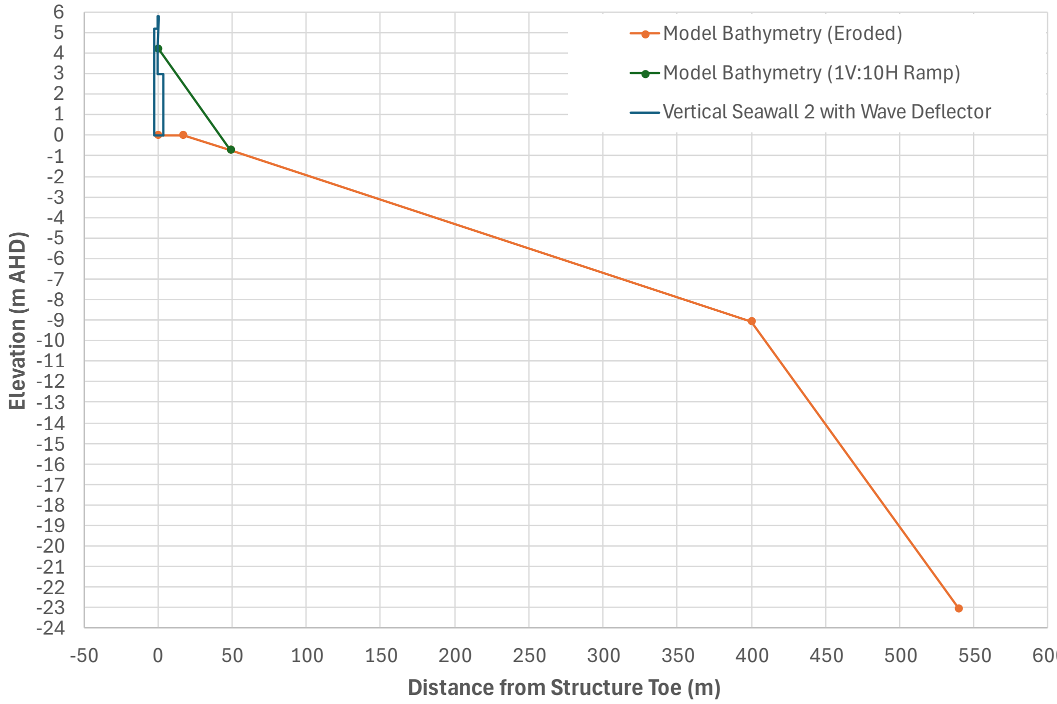


Figure 3.2 Modelled bathymetries: eroded and 1V:10H ramp (waves travelling in a direction from right to left)

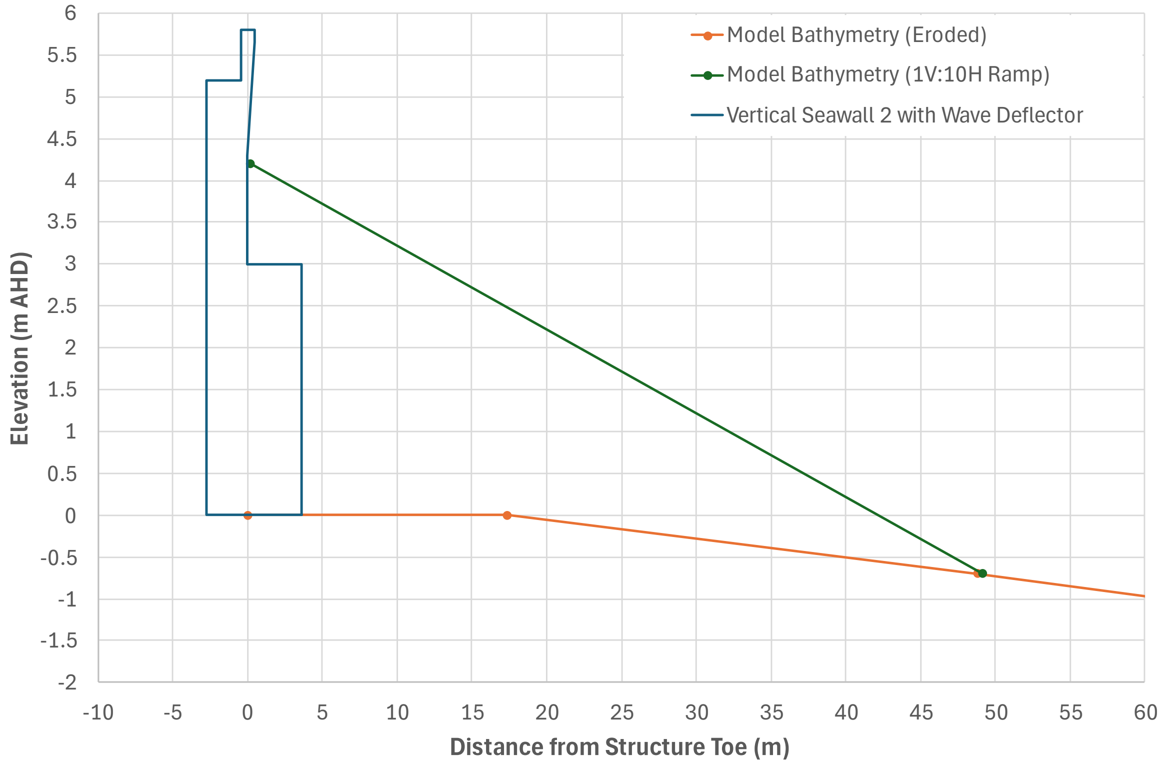


Figure 3.3 Modelled bathymetries: eroded and 1V:10H ramp (detailed view at structure)

3.3.2 Model structures

The two structures were built from water-resistant plywood and water-resistant timber. The model structures were installed on the bathymetric profile with a toe level of 0 m AHD. The 1V:10H representative accreted profile was positioned on top of the eroded profile as indicated in Figure 3.4.

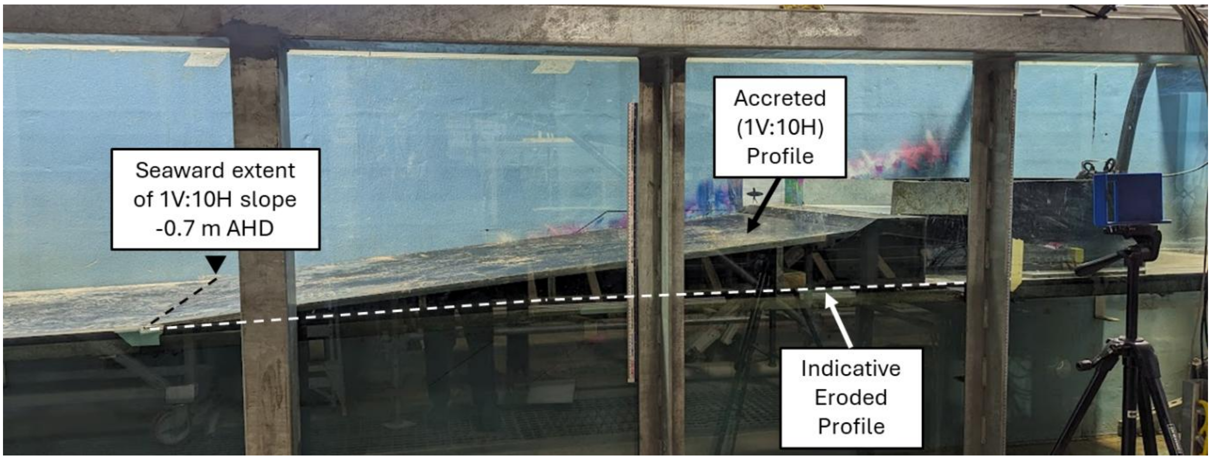


Figure 3.4 Accreted and indicative eroded profiles

Annotated photos of the constructed models in the flume are provided in Figure 3.5 to Figure 3.8.

In the prototype (real-world), wave runup and overtopping processes would act in three dimensions due to the influence of the inclined 3 m wide beach access ramp fronting the Vertical Seawall with Wave Deflector (as shown in Figure 1.2). However, it was still important to include the access ramp as a feature of this seawall which would influence wave overtopping and wave loads. Following confirmation with RHDHV, the access ramp was modelled as a uniform (flat) 3 m wide step at its lowest elevation of 2.67 m AHD (i.e. the elevation of the bottom of the ramp; see Figure 3.5).

With an eroded nearshore profile, the access ramp would partially impede incident waves 3 m seaward of the seawall. Selecting the minimum elevation for this uniform step was considered conservative for assessing overtopping on the seawall in conjunction with this eroded nearshore profile.

The steps of the Stepped Seawall were conservatively modelled as a smooth profile sloped at 1V:1.82H (see Figure 3.7).

The width of the footpath landward of the seawall crests was 5.6 m (for the Vertical Seawall with Wave Deflector) and 4.2 m (for the Stepped Seawall) which corresponded to the minimum respective distances to the SLSC building wall.

All dimensions were consistent with RHDHV design drawings (refer to Appendix C for RHDHV design drawings with WRL markup).

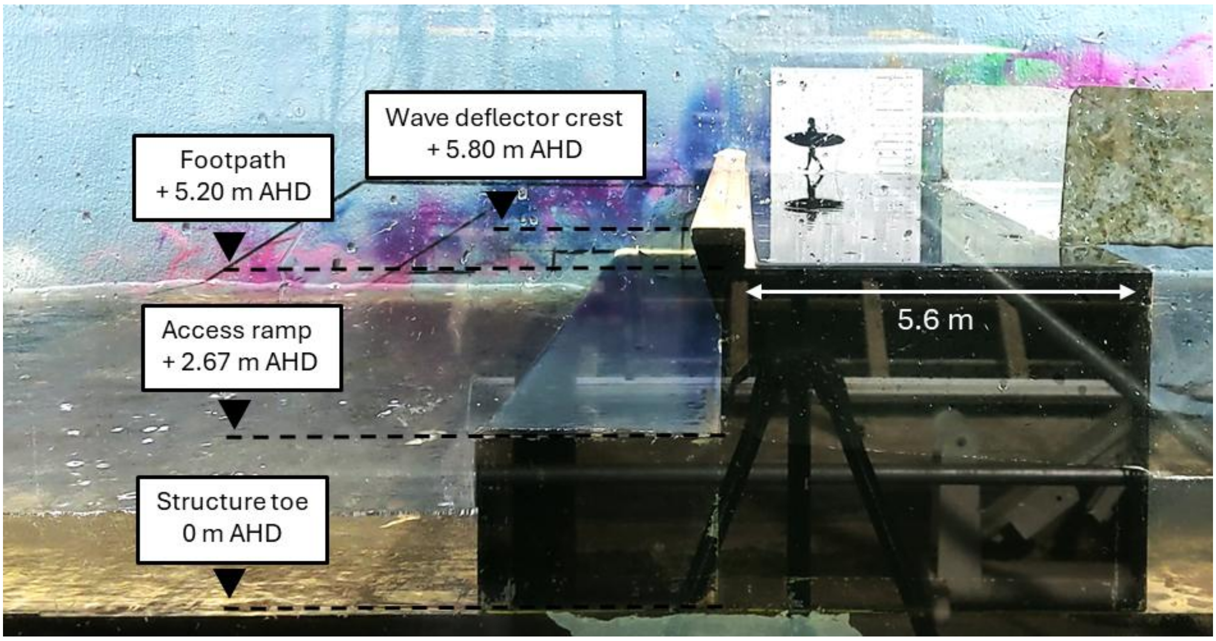


Figure 3.5 Vertical Seawall with Wave Deflector (eroded profile)

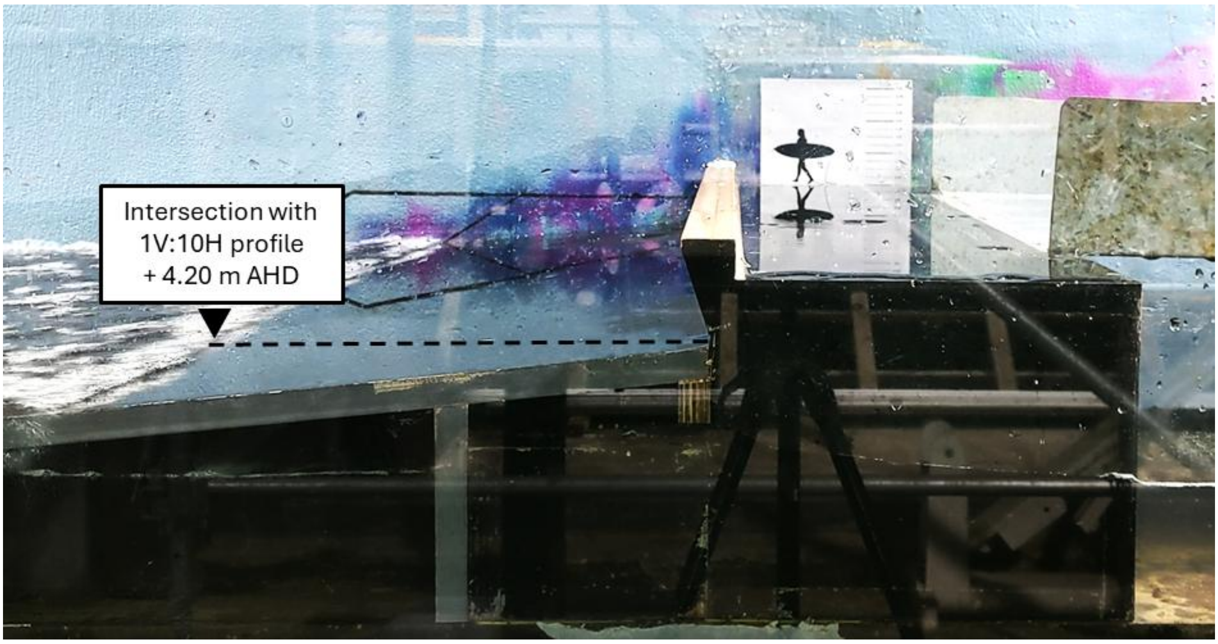


Figure 3.6 Vertical Seawall with Wave Deflector (accreted profile)

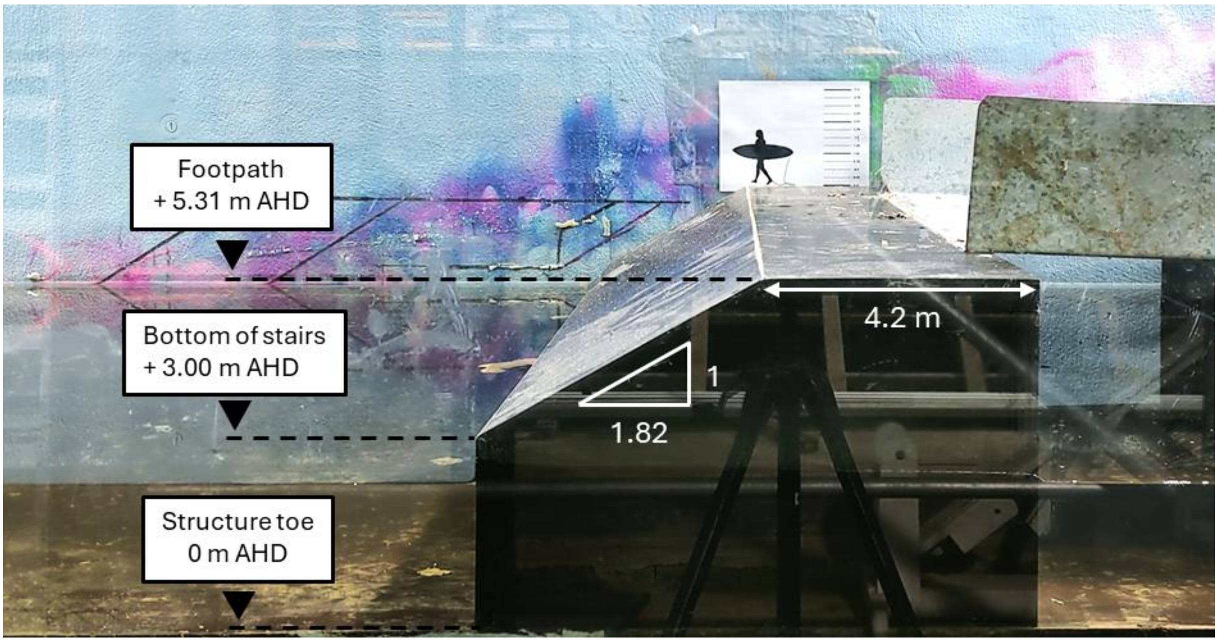


Figure 3.7 Stepped seawall (eroded profile)

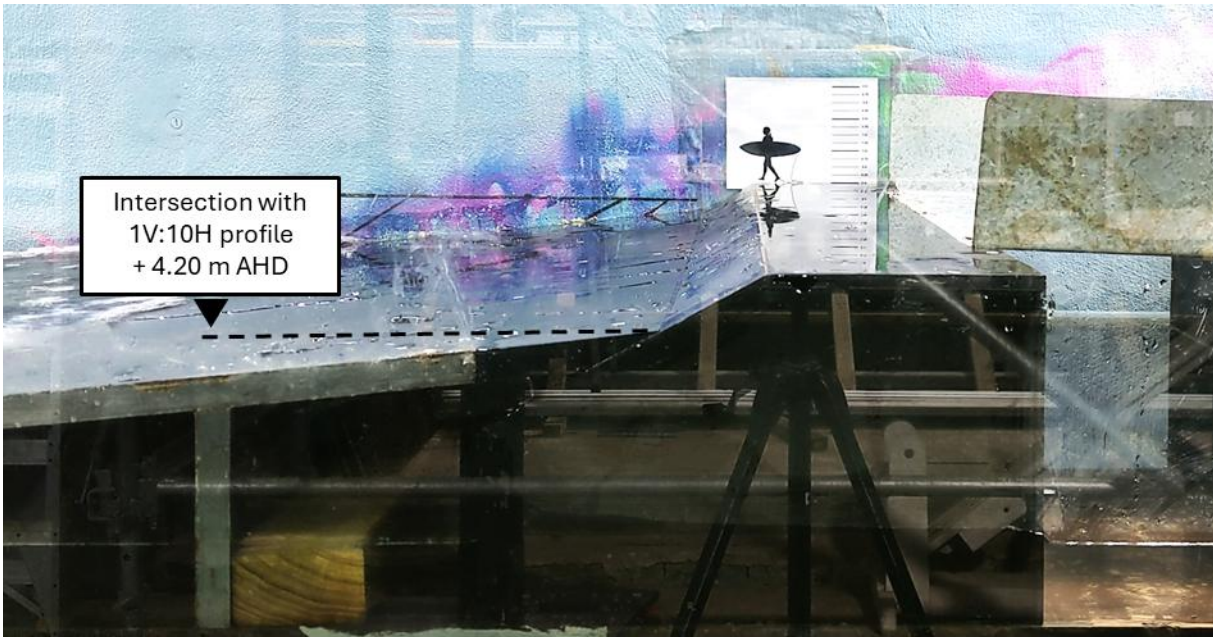


Figure 3.8 Stepped seawall (accreted profile)

3.4 Data collection and analysis

3.4.1 Wave data

Wave conditions and water levels were measured continuously throughout all tests at several locations within the flume. Measurements were collected using high-accuracy capacitance wave probes sampled at a frequency of 10.5 Hz (prototype scale).

For wave climate calibrations, three-probe arrays (3PA) were used to measure wave conditions offshore at -23.0 m AHD, nearshore at -5.0 m AHD and the structure toe at 0 m AHD. These arrays enabled separation of the incident and reflected wave time series using the least-squares method of Mansard and Funke (1980). The 0 m AHD structure 3PA was removed during model testing and a wave probe was positioned in the overtopping catch tray for overtopping testing. Details of the wave probe locations for the different test types are summarised in Table 3.2.

Table 3.2 Wave measurement locations

Wave probe name	Bed elevation (m AHD)	Test type
Offshore 3PA	-23.0	All tests
Nearshore 3PA	-5.0	All tests
Structure 3PA	0.0	Wave climate calibration
Overtopping WP	n/a (overtopping catch tray)	Overtopping testing

Zero up-crossing and zero down-crossing analyses were completed for each wave probe record after each test. The zero crossing analyses, supplemented with spectral analysis, were used to determine wave statistics such as:

- T_z: Mean wave period (s)
- T_p: Peak wave period (s)
- $T_{m-1,0}$: Spectral wave period (s)
- $H_{1/3}$: Significant wave height defined as the average height of the highest 1/3rd of waves (m)
- H_{m0}: Significant wave height using the zero moment of the spectrum (m)
- H_{max}: Maximum wave height (m)

3.4.2 Overtopping

During overtopping tests, the volume of water overtopping a 16.2 m long section of the crest was collected using a catch tray placed on the leeside of the model structure. Overtopping water was collected 5.6 m landward of the seawall crest (for the Vertical Seawall with Wave Deflector) and 4.2 m landward of the seawall crest (for the Stepped Seawall), respectively. The overtopping water was channelled to the catch tray through a folded sheet steel channel (Figure 3.9).

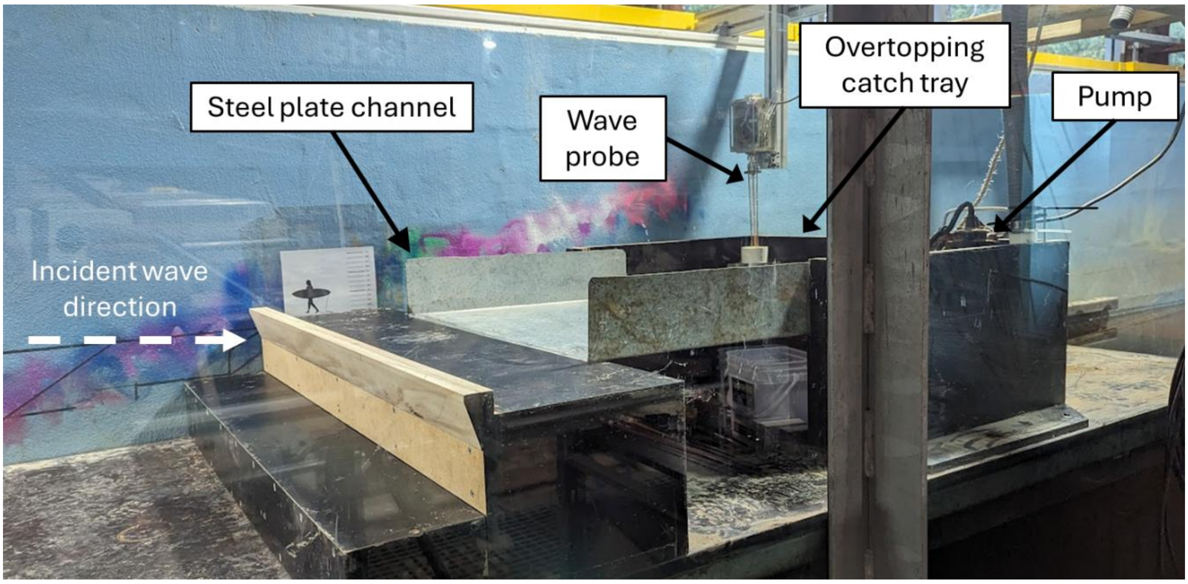


Figure 3.9 Arrangement for measurement of overtopping

If the volume of overtopping approached the capacity of the catch tray, the water in the catch tray was pumped out, volumetrically measured and tallied to give a cumulative overtopping volume for the test duration. This setup allowed the measurement of mean overtopping discharge, *q* (L/s per m of crest length). *q* was calculated by dividing the total volume of water that overtopped the structure, by the duration of the test and normalised by the tested length of crest (16.2 m).

Individual overtopping events were also estimated by measuring the volume of water to overtop the crest during large individual wave overtopping occurrences (i.e. group of waves). A wave probe recorded a timeseries of the water level in the catch tray, which was then converted to volume and normalised by the overtopping crest width to obtain a volumetric timeseries (L per m of crest length) of individual waves. A low-pass filter was applied to the time series to remove high frequency waves within the catch tray. This approach to the collection and analysis of wave overtopping data is in accordance with the procedure for measuring individual overtopping events in HYDRALAB III (2007).

The process of extracting individual overtopping events and specifically V_{max} from the cumulative overtopping timeseries for Test 1 (Vertical Seawall with Wave Deflector, 20 year ARI waves and 2093 planning period) is illustrated in Figure 3.10.

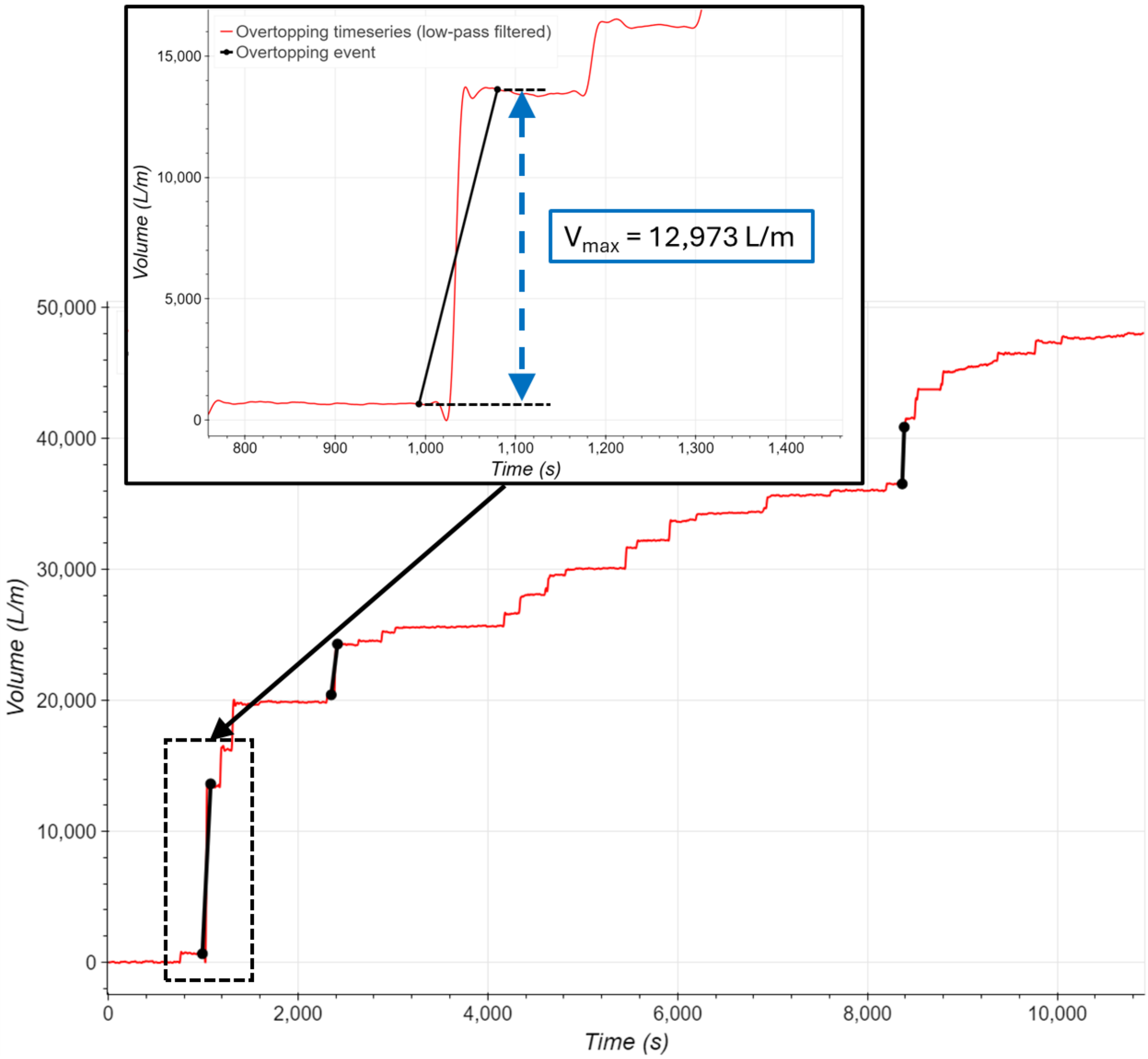


Figure 3.10 Cumulative overtopping timeseries and the largest overtopping volume (V_{max}) extracted from the filtered overtopping timeseries (top) for Test 1 – Vertical Seawall with Wave Deflector, 20 year ARI waves and 2093 planning period

The measured overtopping rates do not allow for the effects of wind due to the complexities that this would introduce into the model setup, however, wind has been shown to have an impact on actual overtopping rates that occur. Adjustments for wind effects can be undertaken using techniques from USACE (2006).

3.4.3 Wave loads

Wave load testing was conducted on the wave deflector and a 1.2 m high section of the SLSC building wall for the Vertical Seawall with Wave Deflector structure. The wave deflector and SLSC building wall load test sections were both 8.1 m wide and were offset in the alongshore direction, to minimise any influence from the presence of the three-dimensional (3D) load cell (attached to the wave deflector) on direct overtopping impacting the SLSC building wall. To prevent overtopping water from remaining pooled between the deflector and the SLSC building wall (as it is expected to drain laterally in the real-world), the model SLSC building wall did not occupy the full flume width to allow drainage pathways either side of it, as indicated in Figure 3.12.

3D load cells were mounted to the leeward side of the wave deflector and SLSC building wall sections to measure loads in both horizontal and vertical axes, although only the horizontal forces were reported for the SLSC building wall. The 3D load cells were capable of measuring forces up to 748 kN/m in the horizontal direction and 449 kN/m in the vertical direction. The total force on the wave deflector was calculated using Pythagoras theorem from the horizontal and vertical force components, as shown in Equation 3.1.

$$Total\ Force = \sqrt[2]{(Horizontal\ Force)^2 + (Vertical\ Force)^2} \qquad (3.1)$$

Force measurements were collected with a sampling rate of 192 Hz. A limitation with recording at such a high sampling rate (1,000 Hz in the model) was the file size limits with the 3D load cell software, requiring the full test force timeseries to be recorded in 11 “blocks”. For the Wave Deflector and SLSC building wall timeseries, there was a total of 10 measurement gaps, each lasting approximately 48 seconds prototype (9 seconds model). The measurement gaps were strategically timed with smaller waves to ensure that substantial wave force impacts in the timeseries were not missed.

Figure 3.11 shows a drawing of the flume arrangement for load testing. Photos of the wave loading testing setup are provided in Figure 3.12 and Figure 3.13.

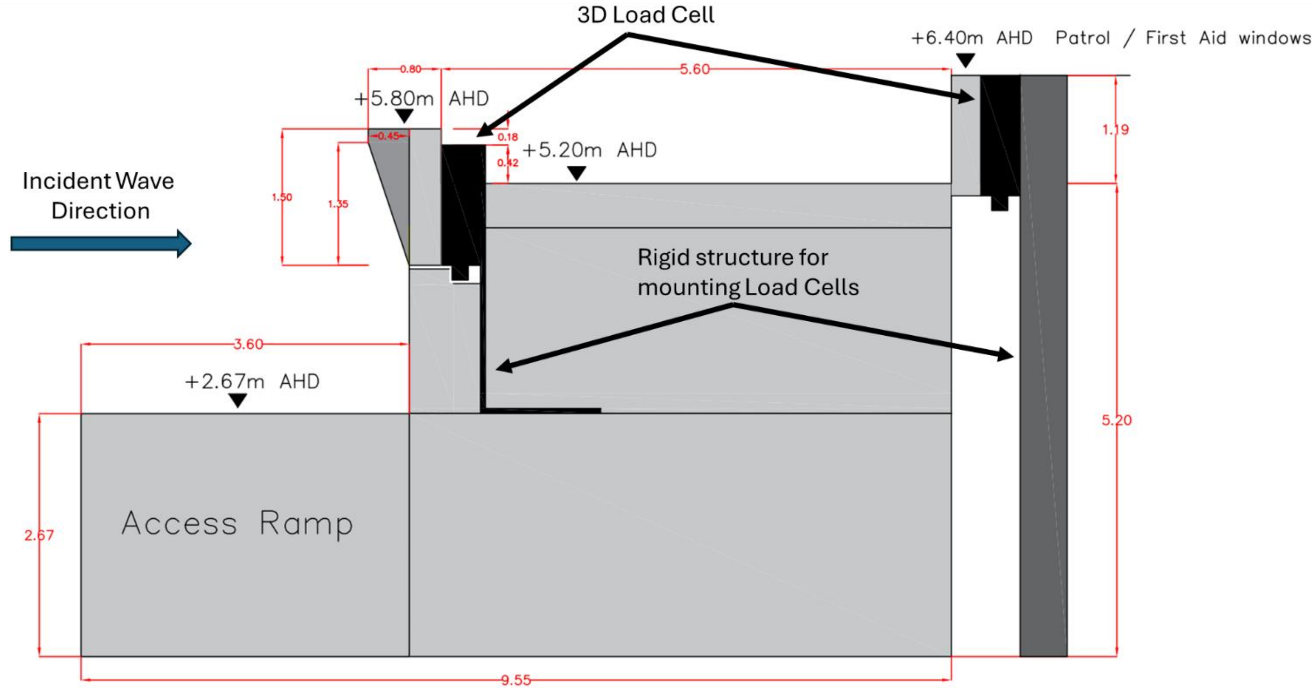


Figure 3.11 Load testing arrangement for the Vertical Seawall with Wave Deflector (all dimensions in metres)

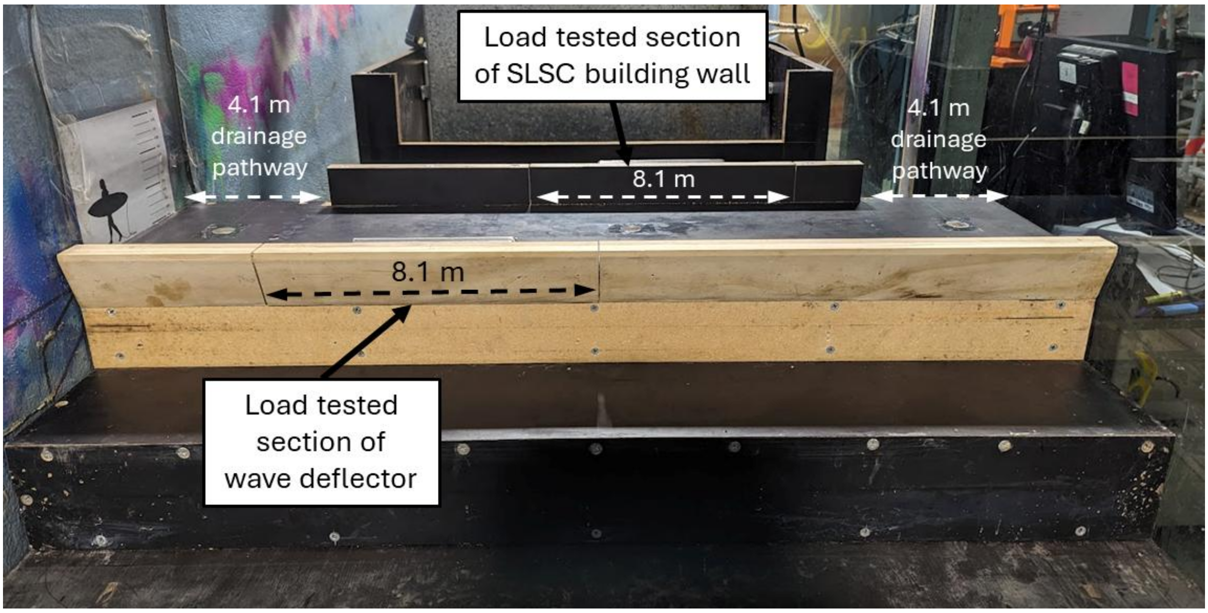


Figure 3.12 Landward view of flume arrangement for wave load testing

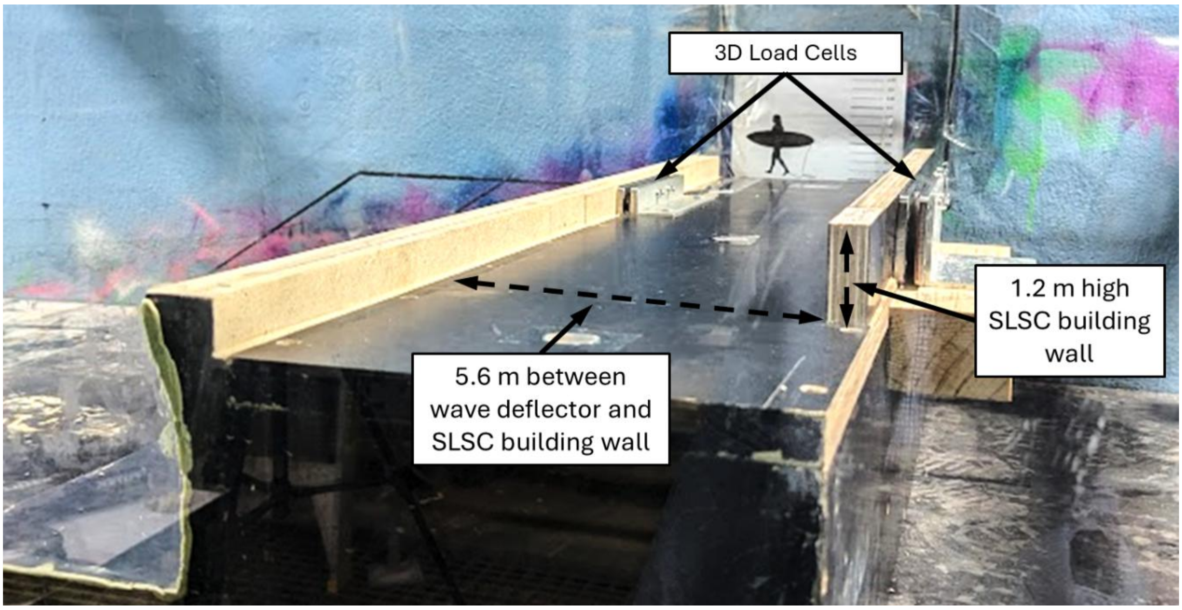


Figure 3.13 Side view of flume arrangement for wave load testing

A dynamic in-situ “push test” was completed using a separate uni-directional load cell, to quantify mechanical losses in the load-sensing section of the structure, and to verify that all forces were being correctly distributed through the instrument rig. The extent of instrumentation noise relative to typical loads measured in the wave flume was also assessed during the “push test”.

On the basis of these sensor-setup verification tests, a 10% uncertainty factor was applied to all provided load measurements in this report to allow for accuracy limitations in the model setup (i.e. all measured forces have been multiplied by 1.1).

3.4.4 Media and data files sharing

Recorded data, including overtopping timeseries, wave load time series and videos for all conducted flume tests were provided to RHDHV in a secured OneDrive folder.

Individual media folders were created for each test and typically included:

- Two side view (close and far) videos of the full test duration
- 10 second videos of the three largest overtopping or wave load events
- Overtopping or wave load timeseries

4 Wave climate calibration

4.1 Design wave conditions

RHDHV provided WRL with six target wave climate and water level (WL) conditions to be calibrated for two different planning periods (present day and 2093). Initial wave climate data provided was for the -10 m AHD contour (RHDHV, 2024c), however given that wave breaking would occur (contributing to wave setup) seaward of this location for the larger events, WRL proposed to use the deepwater significant wave height ($H_{1/3}$) values from MHL's Sydney wave buoy (Table 4.1) to inform the methodology for the physical model.

Table 4.1 Design wave conditions

Design cond. #	ARI (years)	Planning period	Design still water level, excluding wave setup (m AHD)#	Peak spectral wave period (s)#	Deepwater $H_{1/3}$ at MHL Sydney wave buoy (m)*
1	1	Present day	0.66	13.4	5.8
2	1	2093	1.323	13.4	5.8
3	20	2093	2.07	13.6	8.2
4	100	Present day	1.48	14.9	9.4
5	100	2093	2.14	14.9	9.4
6	500	2093	2.14	15.1	10.7

Design still water level (excluding wave setup) and peak spectral wave period values provided in RHDHV, 2024c

* Deepwater $H_{1/3}$ values are 1 hour duration (all directions) derived from Table 8 of Glatz et al., 2017 except 500 year ARI inferred by WRL using log-linear extrapolation.

HR Wallingford wave generation software (HR Merlin v2.50; HR Wallingford, 2021) was used to generate JONSWAP spectra (Hasselmann et al., 1973) and corresponding synthetic wave time series. Inputs into the software included a random seed, a peak enhancement factor of 3.3 (i.e. the default value), and the design peak spectral wave periods (T_P) and the deepwater significant wave height ($H_{1/3}$) shown in Table 4.1.

By changing the random seed while keeping other parameters constant (peak enhancement factor, T_P, $H_{1/3}$), different time series can be generated. While only one time series was generated for each test condition for the Bronte SLSC site, if a range of different random seeds were used to generate multiple time series, WRL considers that the resulting mean overtopping discharge (q) would be similar but that the largest overtopping event (V_{max}) may vary between the time series. If the peak enhancement factor was also changed, or non-JONSWAP spectra were used, greater variability in the resulting mean overtopping discharge would be expected.

All design conditions were generated and calibrated with a minimum of 1,000 waves to be statistically relevant (as recommended in HYDRALAB III, 2007). These time series corresponded to prototype storm durations between 2.9 and 3.4 hours (based on 1,000 waves × mean wave period; T_Z).

At the adopted scale of 1:27, the largest offshore $H_{1/3}$ that could be produced in the physical model at the wave maker was approximately 6.0 m. As this maximum achievable offshore Hs condition was less than the target offshore design conditions for the proposed 20, 100 and 500 year ARI events, WRL raised the test still water level to account for the reduced nearshore wave setup generated in the wave flume. This was necessary due to the fact that nearshore wave conditions (i.e. close to the proposed seawall toe) were depth limited and, as such, the wave height at the seawall was strongly dependent on the total water depth including wave setup.

The process of adjusting the still water level to account for the difference in wave setup between design and achievable deepwater wave heights is summarised in the following two steps:

1. At the adopted scale of 1:27, the full 5.8 m significant wave height for the 1 year ARI event was reproduced in the deep section of the 0.9 m flume. The resulting wave setup measured at 0 m AHD for the 1 year ARI event was then divided by the measured deepwater $H_{1/3}$ (Equation 4.1) to establish the ratio between wave setup and deepwater significant wave height for the 20, 100 and 500 year ARI events.

$$Ratio = \ Measured\ wave\ setup\ \div Measured\ deepwater\ H_{1/3} \tag{4.1}$$

2. This ratio was multiplied by the difference between the target deepwater $H_{1/3}$ and the paddle limited $H_{1/3}$ (approximately 6.0 m) for the 20, 100 and 500 year ARI events to determine the water level adjustment (Equation 4.2) to account for the reduced wave setup generated in the flume.

$$Water\ level\ adjustment = \ Ratio\ \times \left(target\ deepwater\ H_{1/3} - paddle\ limited\ deepwater\ H_{1/3}\right) \tag{4.2}$$

The wave setup achieved at the toe of the proposed structure (i.e. 0 m AHD) during the wave condition calibration was then measured to ensure that the total water level (TWL) including wave setup matched the target TWL derived from the deepwater $H_{1/3}$ ratio method.

4.2 Results

During the wave calibration, waves were measured using three different three-probe arrays referred to as the Offshore 3PA (-23 m AHD), Nearshore 3PA (-5 m AHD) and Structure 3PA (0 m AHD). Incident and reflected irregular wave trains were separated using the Mansard and Funke (1980) method during post-processing analysis. To minimise wave reflections, wave climate calibration was conducted without the seawall structure and absorptive foam against the end of the flume (see Appendix B).

Calibration of the wave conditions was based on wave statistics on the incident waves observed at the Offshore 3PA location (-23 m AHD). For the two 1 year ARI conditions, the target $H_{1/3}$ at -23 m AHD was within 0.1 m and all offshore T_P values were within 0.2 s of the target.

For the 1 year ARI tests, the average wave setup measured at 0 m AHD for the present day and 2093 was 3.9% of the deepwater significant wave height (5.8 m). This ratio was used to determine the water level adjustment and the target TWL at the 0 m AHD for the 20, 100 and 500 year ARI events. For example, the water level adjustment and target TWL for Design Condition 4 - *100 year ARI and present day planning period,* were calculated in Equations 4.3 and 4.4.

$$Water\ level\ adjustment = 3.9\% \times (9.4\ m - 5.8\ m) = 0.14\ m \quad (4.3)$$

$$Target\ TWL = 1.48\ m\ AHD + (3.9\% \times 9.4\ m) = 1.85\ m \quad (4.4)$$

Following this approach, WRL matched the TWL at 0 m AHD to within 0.02 m of the target TWL for the 20, 100 and 500 year ARI conditions. Wave climate statistics at the Offshore, Nearshore and Structure locations are presented in Table 4.2.

Table 4.2 Measured wave climate conditions at the Offshore, Nearshore and Structure locations and total water level at the Structure

WRL design cond. #	Target Offshore (-23 m AHD)		WRL Measured Offshore (-23 m AHD)		WRL Measured Nearshore (-5 m AHD)		WRL Measured Structure (0 m AHD)		WRL Measured TWL at Structure (0 m AHD)	
	$H_{1/3}$ (m)	T_P (s)	$H_{1/3}$ (m)	T_{P1}* (s)	$H_{1/3}$ (m)	T_{P1}* (s)	$H_{1/3}$ (m)	T_Z** (s)	Target (m AHD)	Measured (m AHD)
1	5.8	13.4	5.8	13.3	3.1	13.3	0.6	7.2	0.89	0.89
2	5.8	13.4	5.7	13.3	3.4	13.2	0.9	7.3	1.55	1.54
3	8.2	13.6	5.8	13.5	3.7	13.6	1.2	7	2.39	2.39
4	9.4	14.9	5.8	14.8	3.4	14.8	0.9	7.1	1.85	1.86
5	9.4	14.9	5.9	14.9	3.8	14.9	1.3	8.3	2.51	2.52
6	10.7	15.1	6.3	15.3	3.9	15.1	1.3	8	2.56	2.58

* T_{P1} is calculated according to "Method 2 (so called Read method)" using a value of 4 for the exponent n as outlined in Table 4.11 of *The Rock Manual* (CIRIA; CUR; CETMEF, 2007)

** T_Z provided instead of T_{P1} at the Structure (0 m AHD) as a significant portion of broken waves resulted in long wave generation.

A complete summary of wave climate calibration statistics for the Offshore, Nearshore and Structure 3PA is provided in Appendix D.

5 Results

5.1 Overtopping results

5.1.1 Preamble

A total of 10 tests were performed using a selected combination of two seawall configurations, two nearshore profiles and five wave conditions. Overtopping volumes were measured along a 16.2 m long section of the seawall. Mean overtopping rates were obtained by averaging the total overtopping volumes over the duration of the test, and overtopping volumes from individual events were extracted from the overtopping timeseries.

5.1.2 Mean overtopping

A summary of the mean overtopping test results is provided in Table 5.1. A comparison between mean overtopping rates for the two seawall structures and the two nearshore profiles is provided in Figure 5.1 and Figure 5.2, respectively.

Table 5.1 Mean overtopping rates for seawall configurations

RHDHV test ref. #	Structure	Bathymetry	Design cond. #	Waves ARI (years)	Planning period*	Mean overtopping rate, q (L/s/m)
1	Vertical seawall 2 with wave deflector	Fully eroded	3	20	2093	4.1
2		1V:10H ramp	3	20	2093	6.7
3		1V:10H ramp	5	100	2093	8.3
4		1V:10H ramp	2	1	2093	1.4
5		Fully eroded	4	100	PD	0.8
6		Fully eroded	5	100	2093	5.2
7		Fully eroded	1	1	PD	0.1
8	Stepped seawall	Fully eroded	5	100	2093	44
9		1V:10H ramp	2	1	2093	5.6
11	Vertical seawall 2 with wave deflector	1V:10H ramp	1	1	PD	0.2

* PD: Present day

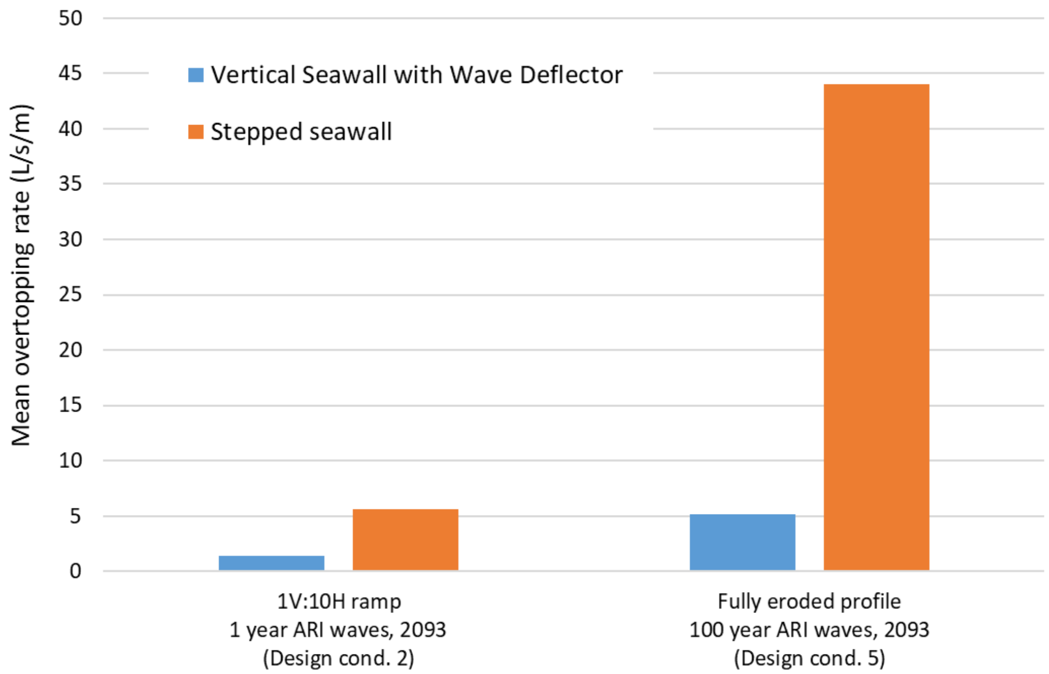


Figure 5.1 Comparison of mean overtopping rates for the Vertical Seawall with Wave Deflector and Stepped Seawall structures

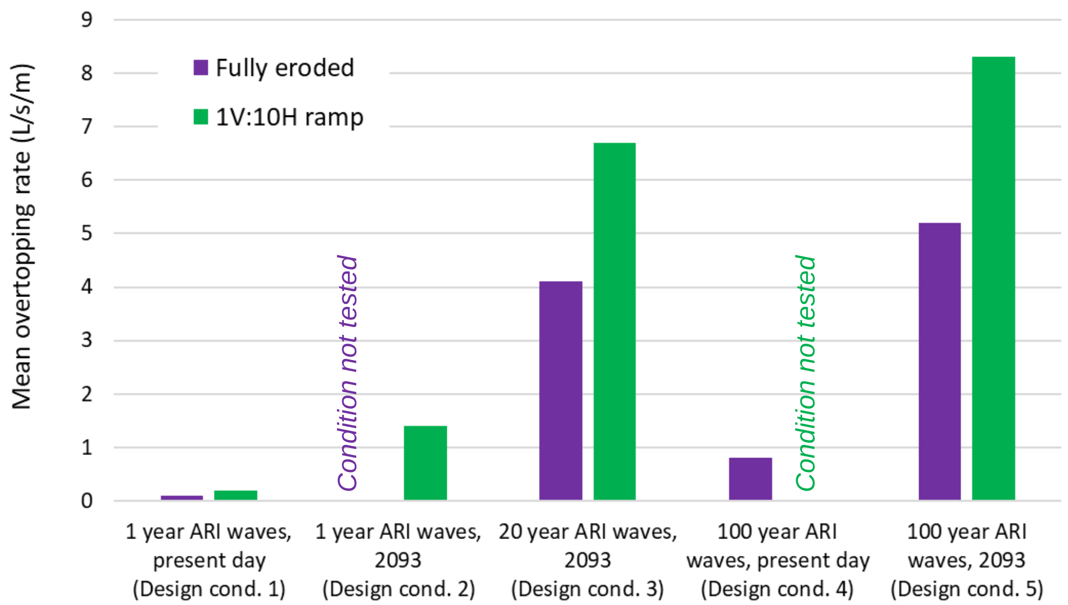


Figure 5.2 Comparison of mean overtopping rates for the Vertical Seawall with Wave Deflector with a fully eroded profile and an accreted (1V:10H ramp) profile

The following insights were derived from comparing mean overtopping rates between the different seawall structures, nearshore profiles and wave climates.

- Mean overtopping rates for the Vertical Seawall with Wave Deflector were 75-90% less than for the Stepped Seawall structure for two different wave conditions (design cond. 2 and 5) tested.
- Mean overtopping rates for the Vertical Seawall with Wave Deflector with the fully eroded profile were 40-50% less compared with the 1V:10H ramp for three different wave conditions (design cond. 1, 3 and 5) tested.

5.1.3 Individual overtopping events

For each test, individual overtopping volumes were reported as the largest overtopping event (V_{max}), the average of the five largest overtopping events (V_{avg5}), the average of the ten largest overtopping events (V_{avg10}). WRL have also reported the ratio of the largest overtopping event to the mean overtopping volumes (V_{max}/q) and the number of individual overtopping events that exceeded EurOtop's 600 L/m guideline for pedestrian access (EurOtop, 2018).

A summary of the individual overtopping events is provided in Table 5.2. A comparison of the overtopping timeseries for the two seawall structures and the two nearshore profiles is provided in Figure 5.3 and Figure 5.4, respectively. An image sequence comparison of the V_{max} event for the two seawall structures (Test 4 and 9) and the two nearshore profiles (Tests 1 and 2) are provided in Figure 5.5 and Figure 5.6, respectively.

The following insights were derived from comparing the largest individual overtopping events between the different seawall structures, nearshore profiles and wave climates:

- V_{max} was 37-47% less and V_{avg10} was 63% less for the Vertical Seawall with Wave Deflector compared to the Stepped Seawall structure for two different wave conditions (design cond. 2 and 5) tested.
- For the Vertical Seawall with Wave Deflector structure, V_{max} was 14-36% less, and V_{avg10} was 33-35% less, with the fully eroded profile compared with the 1V:10H ramp for two different wave conditions (design cond. 3 and 5) tested.

5.1.4 Discussion

The test results demonstrated that the mean overtopping rate and maximum individual overtopping volumes were higher with the 1V:10H ramp profile compared with the eroded profile. While the 1V:10H ramp was fixed in the physical model, it represented an accreted sandy beach profile in the real-world which would be subject to erosion during a storm. The volume of erosion would increase with storm rarity which would influence the wave runup and overtopping processes at the seawall. Since the wave load test was to be conducted with an extreme 500 year ARI storm event and erosion processes (i.e. a moveable bed) were not included in the physical model, RHDHV directed WRL to undertake the subsequent wave loading test using the eroded profile.

Nine out of ten overtopping tests included at least one wave in the 1,000 wave timeseries that resulted in an overtopping event exceeding EurOtop's 600 L/m tolerable limit for pedestrian access (EurOtop, 2018). The frequency of these large individual overtopping events was strongly dependent on the planning period. For example, the number of overtopping events exceeding 600 L/m for the 2093 planning period was 5-8 times that of the present day planning period (i.e. Test 5 vs 6; Test 4 vs 11). Subsequently, managing pedestrian access at Bronte SLSC will become increasingly more important with projected sea level rise.

While the measured V_{max}/q ratios (Table 5.2) for the two seawall configurations proposed for Bronte SLSC are high, similar values may be found in literature. For example, experiments by Franco et al. (1994) measured V_{max}/q ratios of up to 10,000 s for small q values (in the order of 1 L/s/m) on vertical structures. It was also observed that almost all large individual overtopping events involved a large group of waves breaking offshore, coincident with substantial dynamic wave setup at the structure toe, then superposition at the point of overtopping (informally referred to as "doubling up").

Table 5.2 Overtopping volumes for individual overtopping events

RHDHV test ref. #	Structure	Bathymetry	Design cond. #	Waves ARI (years)	Planning period*	V_{max} (L/m x10^3) **	V_{avg5} (L/m x10^3) **	V_{avg10} (L/m x10^3) **	# OT events > 600 L/m	V_{max}/q (s)
1	Vertical seawall 2 with wave deflector	Fully eroded	3	20	2093	13.0	5.5	3.5	21	3,171
2	Vertical seawall 2 with wave deflector	1V:10H ramp	3	20	2093	20.3	8.3	5.4	31	3,030
3	Vertical seawall 2 with wave deflector	1V:10H ramp	5	100	2093	13.5	7.3	5.4	44	1,627
4	Vertical seawall 2 with wave deflector	1V:10H ramp	2	1	2093	10.0	3.1	1.7	5	7,143
5	Vertical seawall 2 with wave deflector	Fully eroded	4	100	PD	3.0	1.2	-	3	3,750
6	Vertical seawall 2 with wave deflector	Fully eroded	5	100	2093	11.6	5.2	3.6	26	2,231
7	Vertical seawall 2 with wave deflector	Fully eroded	1	1	PD	-	-	-	0	-
8	Stepped seawall	Fully eroded	5	100	2093	18.4	12.3	9.6	159	418
9	Stepped seawall	1V:10H ramp	2	1	2093	18.7	7.3	4.6	19	3,339
11	Vertical seawall 2 with wave deflector	1V:10H ramp	1	1	PD	1.6	-	-	1	8,000

* PD: Present day

** Note, "-" denotes tests where individual overtopping events were too small to be extracted from the overtopping timeseries for Vmax, and for Vavg5 and Vavg10, "-" denotes tests that had less than 5 or 10 individual overtopping events, respectively.

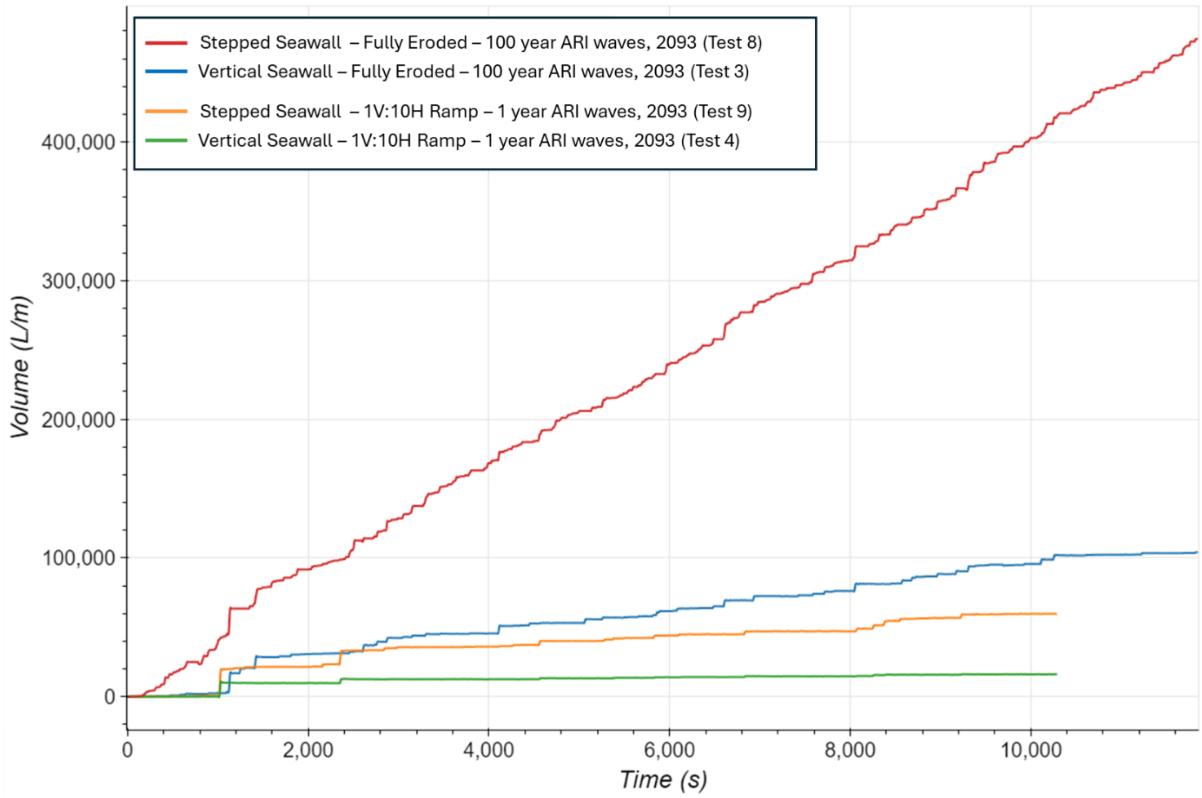


Figure 5.3 Comparison of overtopping timeseries for the Vertical Seawall with Wave Deflector and Stepped Seawall structures

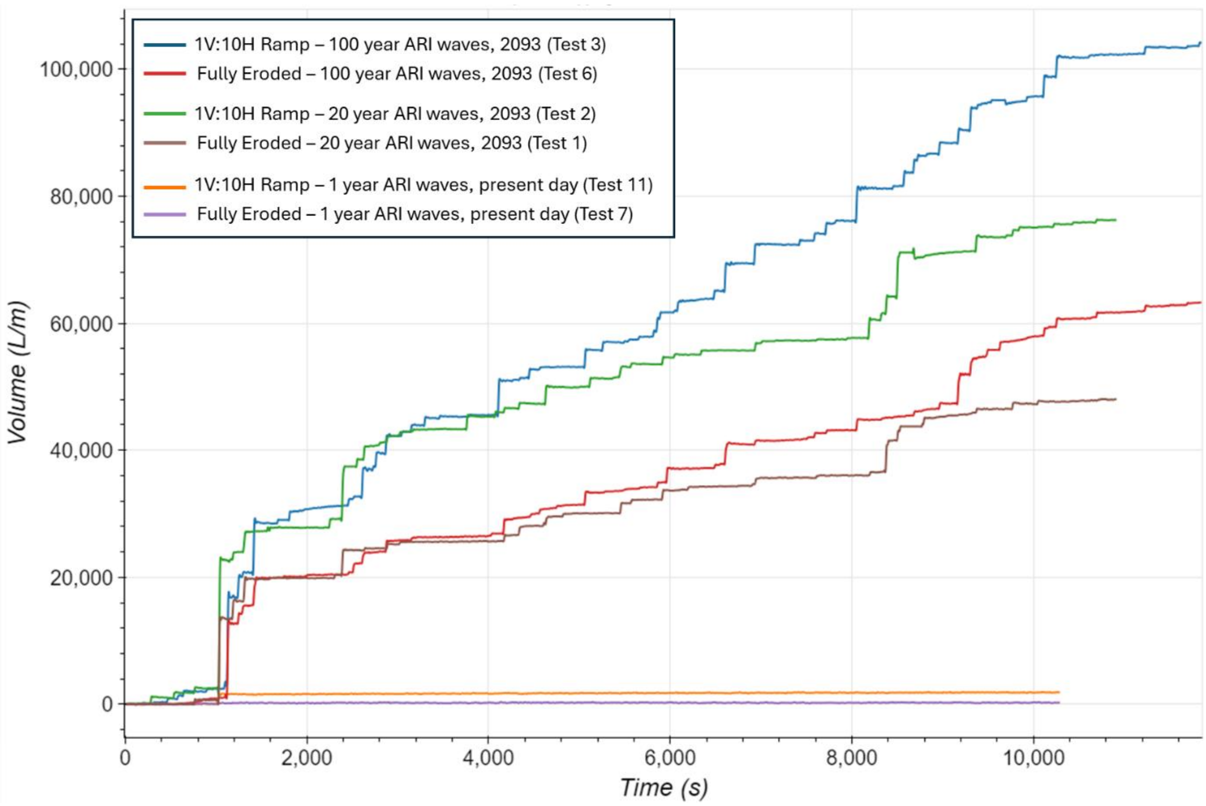


Figure 5.4 Comparison of overtopping timeseries for the Vertical Seawall with Wave Deflector structure with a fully eroded profile and an accreted (1V:10H ramp) profile

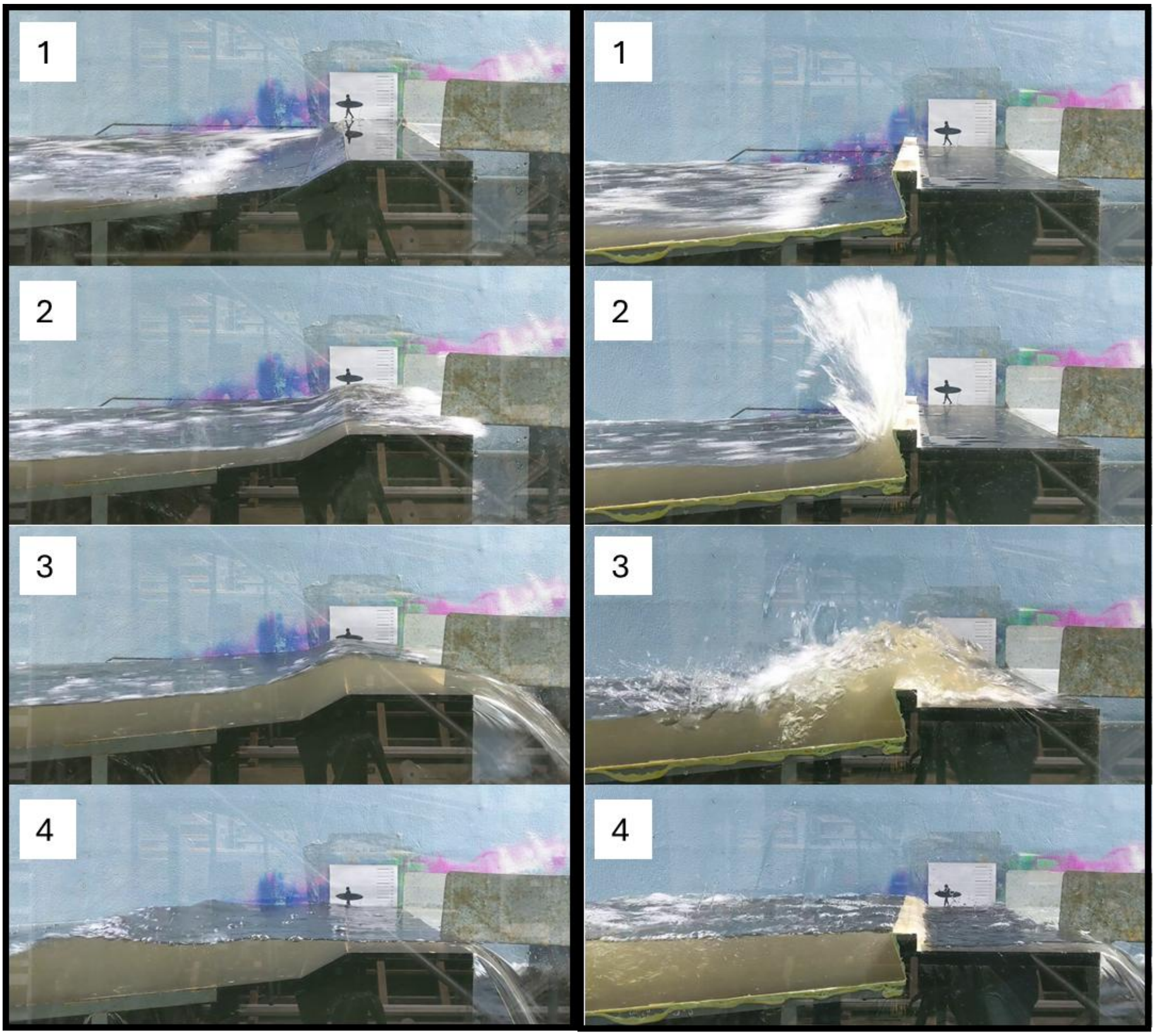


Figure 5.5 Image sequence comparison of V_{max} event (18,700 L/m) for the Stepped Seawall (left) [Test 9] and the V_{max} event (10,000 L/m) for the Vertical Seawall with Wave Deflector (right) [Test 4] subject to Design Condition 1

Figure 5.6 Image sequence comparison of V_{max} event (13,000 L/m) for the accreted profile (left) [Test 2] and the V_{max} event (20,300 L/m) for the eroded profile (right) [Test 1] with the Vertical Seawall with Wave Deflector structure subject to Design Condition 3

5.2 Wave loading results

A single load test (Test 10) was conducted on 8.1 m wide sections of the wave deflector and the SLSC building wall for the Vertical Seawall with Wave Deflector Structure, subject to Design Condition 6 - *500 year ARI waves and 2093 planning period*. A summary of load test results including the largest force measured (F_{max}) on both the wave deflector and the 1.2 m high section of SLSC building wall, is provided in Table 5.3.

The total force time series (combined horizontal and vertical) for the wave deflector is provided in Figure 5.7. A timeseries excerpt and an image sequence of the wave deflector F_{max} event are provided in Figure 5.8 and Figure 5.9, respectively. The horizontal force values are positive in the landward (i.e. incident wave) direction. The horizontal force time series for the SLSC building wall is provided in Figure 5.10. A timeseries excerpt and an image sequence of the SLSC building wall F_{max} event are provided in Figure 5.11 and Figure 5.12, respectively.

Following RHDHV's request, WRL conducted a complimentary repeat of Test 10 with the SLSC building wall extended to the wave flume glass wall (i.e., no drainage pathway) to provide a clearer view of the overtopping dynamics against the SLSC building wall during the largest wave loading events. Image sequences provided in Figure 5.9 and Figure 5.12 were captured during this repeat test.

Table 5.3 Summary of wave loading results

RHDHV test ref. #	Structure	Bathymetry	WRL Design cond. #	Waves ARI (years)	Planning period	F_{max} on wave deflector (kN/m)	F_{max} on SLSC building wall (kN/m)
10	Vertical seawall 2 with wave deflector	Fully eroded	6	500	2093	195	53

Review of the largest wave forces during Test 10 indicated the typical total duration (rise and fall) of the force impacts on the wave deflector to be approximately 0.2 to 0.6 s and on the SLSC building wall to be 0.8 to 1.4 s.

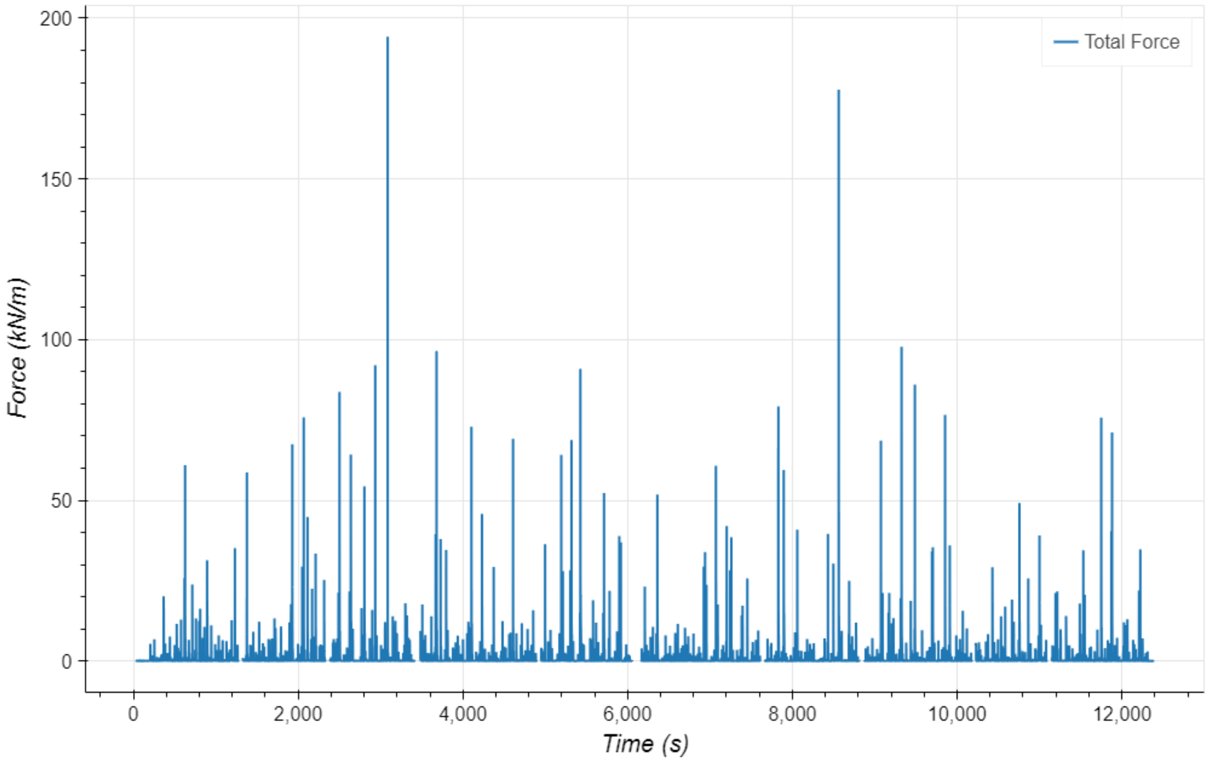


Figure 5.7 Total force timeseries for the wave deflector

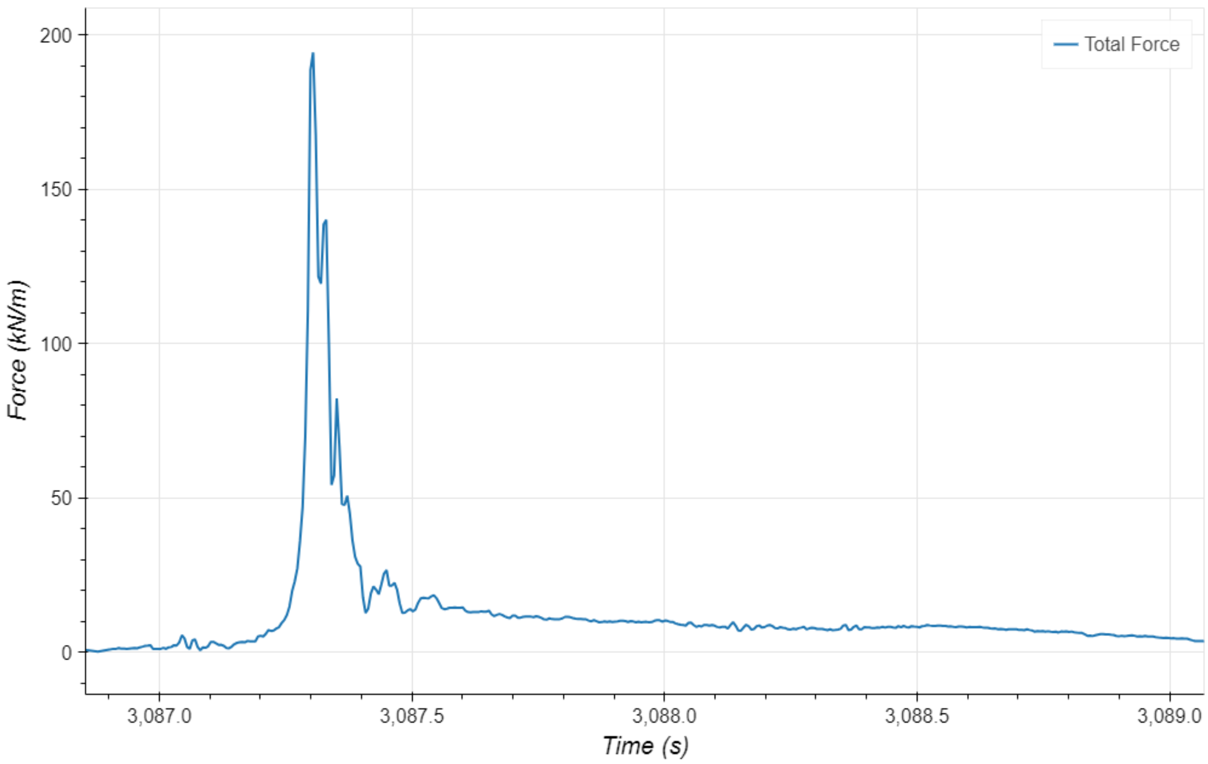


Figure 5.8 F_{max} event (195 kN/m) for the wave deflector

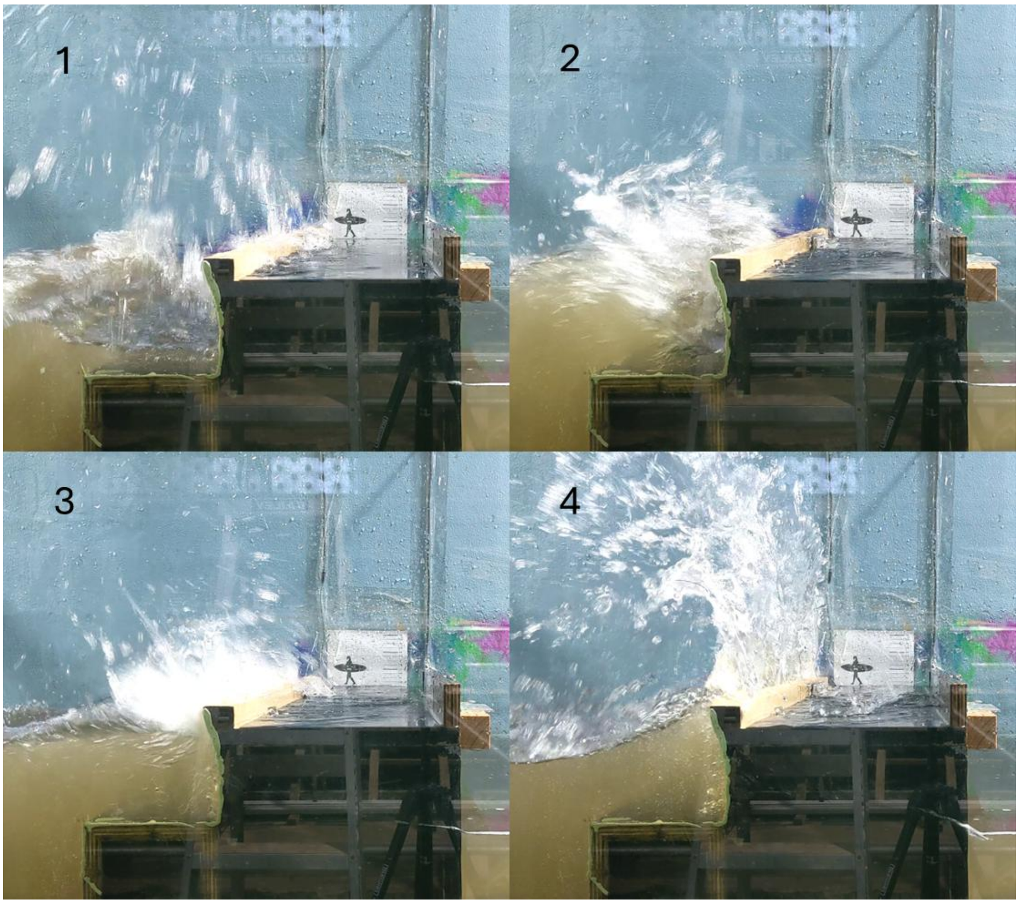


Figure 5.9 Image sequence of the F_{max} event (195 kN/m) at 3,087 s on the wave deflector during repeat of Test 10

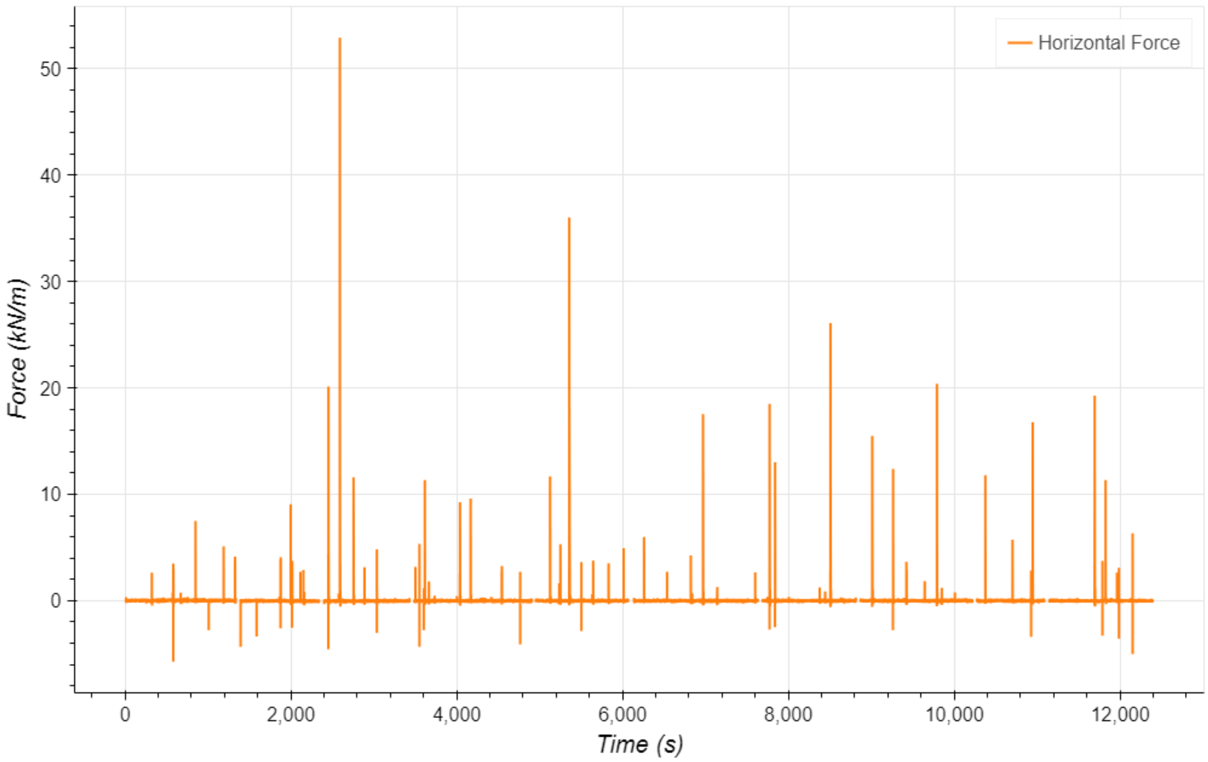


Figure 5.10 Total force timeseries for the 1.2 m high section of SLSC building wall

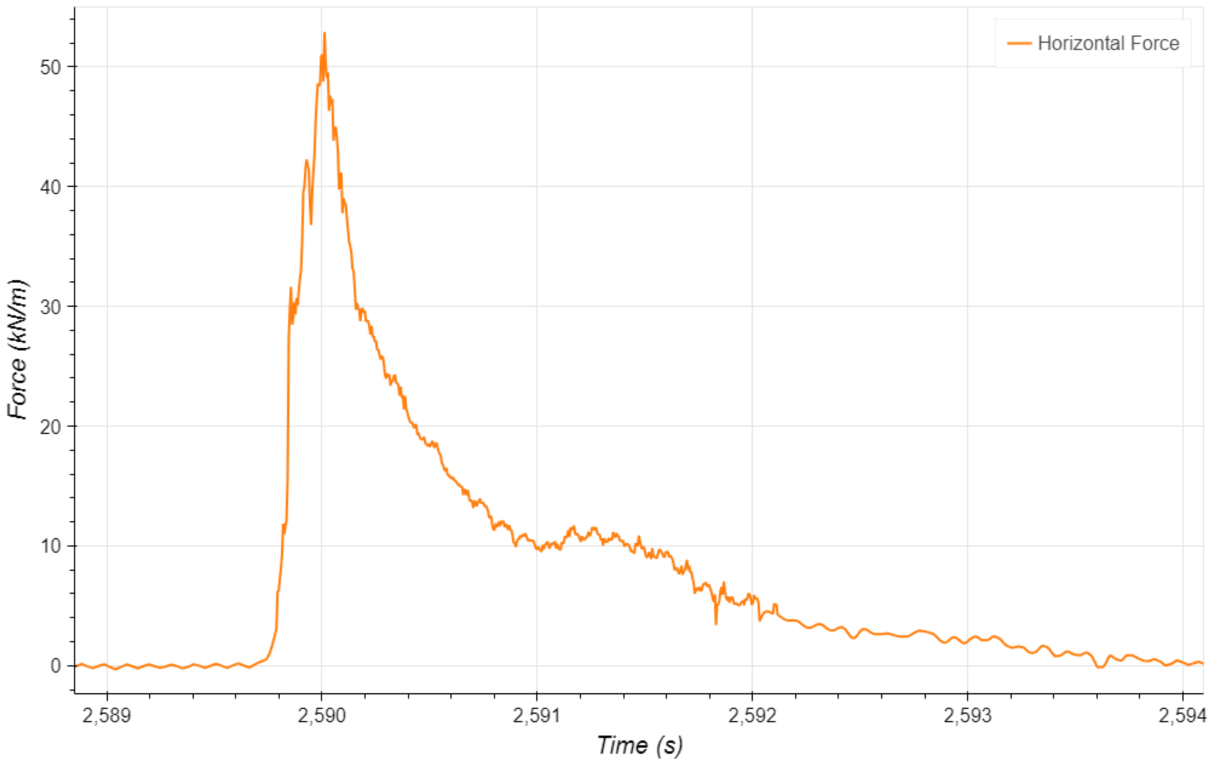


Figure 5.11 F_{max} event (53 kN/m) for the 1.2 m high section of SLSC building wall

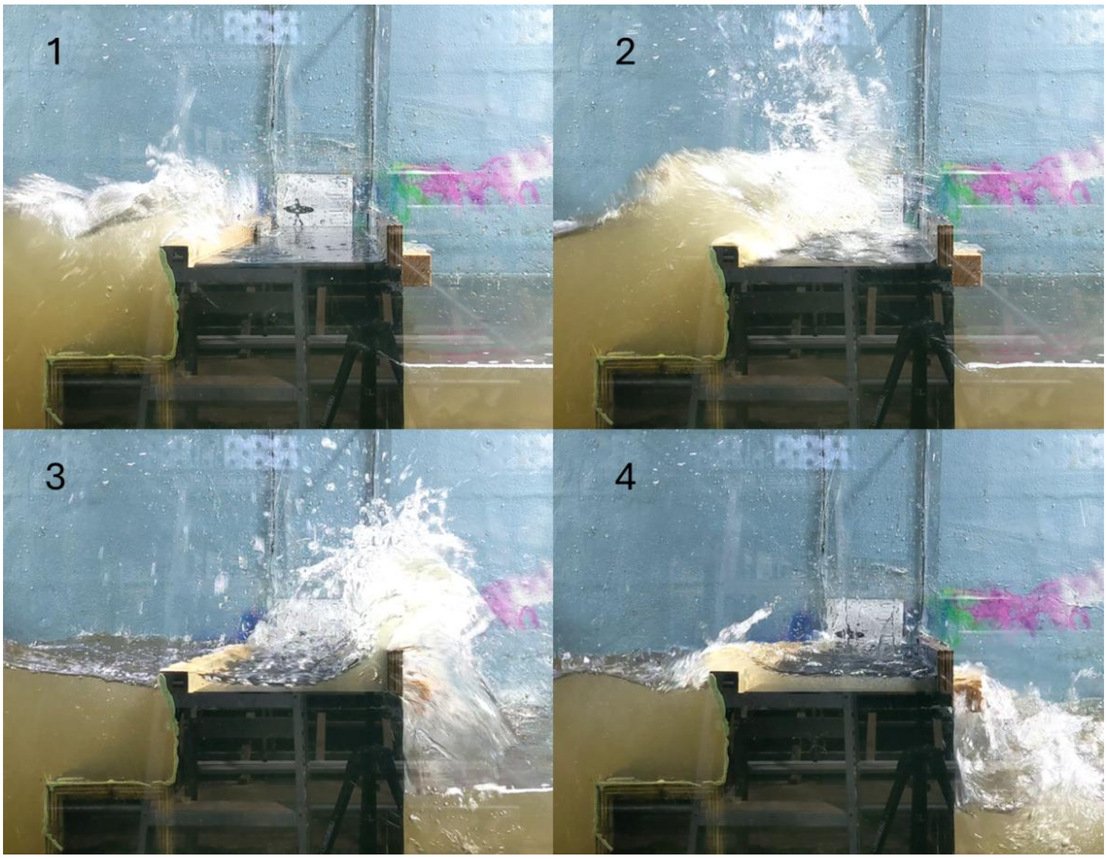


Figure 5.12 Image sequence of the Fmax event (53 kN/m) at 2,590 s on the SLSC building wall for the repeat test

6 Summary

6.1 Overview

WRL completed physical modelling for two seawall cross-sections fronting the proposed redevelopment of the Bronte SLSC, including:

- Vertical Seawall with Wave Deflector – A vertical seawall with a wave deflector (deflector crest level 5.8 m AHD) and a 3.6 m wide access ramp located directly in front of the SLSC building (promenade level 5.20 m AHD).
- Stepped Seawall – A stepped concrete seawall providing access to the concrete promenade (promenade level 5.31 m AHD) fronting the SLSC building.

The physical modelling program was carried out in WRL's 0.9 m wide wave flume. Model scaling was based on a geometric similarity (Froude scaling) between the prototype (real world) and the model, with an undistorted length scale of 1:27.

A range of wave climates comprising of four wave conditions (1 year ARI, 20 year ARI, 100 year ARI and 500 year ARI) and two planning periods (present day and 2093) were examined in the physical modelling. Overtopping testing of the two structures was conducted with representative accreted and eroded nearshore profiles. For the Vertical Seawall with Wave Deflector, WRL also conducted wave load testing on the wave deflector and SLSC building wall (with an eroded nearshore profile).

6.2 Overtopping

A total of 10 tests were performed using a selected combination of the two seawall configurations, two nearshore profiles and five wave climates. Mean overtopping rates and individual overtopping volumes were recorded for all tests.

A comparison of overtopping volumes for the Vertical Seawall with Wave Deflector and the Stepped Seawall structure, indicated:

- Mean overtopping rates for the Vertical Seawall with Wave Deflector were 75-90% less than for the Stepped Seawall structure for two different wave conditions (design cond. 2 and 5) tested.
- V_{max} was 37-47% less and V_{avg10} was 63% less for the Vertical Seawall with Wave Deflector compared to the Stepped Seawall structure for two different wave conditions (design cond. 2 and 5) tested.

A comparison of overtopping volumes for the Vertical Seawall with Wave Deflector with the eroded profile and with the accreted (1V:10H) profile, indicated:

- Mean overtopping rates with the fully eroded profile were 40-50% less compared with the 1V:10H ramp for three different wave conditions (design cond. 1, 3 and 5) tested.
- V_{max} was 14-36% less, and V_{avg10} was 33-35% less, with the fully eroded profile compared with the 1V:10H ramp for two different wave conditions (design cond. 3 and 5) tested.

6.3 Wave loads

A single load test (Test 10) was conducted on an 8.1 m wide section of the wave deflector and an 8.1 m wide x 1.2 m high section of the SLSC building wall for the Vertical Seawall with Wave Deflector Structure and subject to Design Condition 6 - *500 year ARI waves and 2093 planning period*.

3D load cells were used to measure the combined horizontal and vertical forces on the angled wave deflector and horizontal forces on the SLSC building wall. The largest forces (F_{max}) measured on the wave deflector and SLSC building wall throughout the 3.4 hour test, were 195 kN/m and 53 kN/m, respectively.

Review of the largest wave forces indicated the typical total duration (rise and fall) of the force impacts on the wave deflector to be approximately ranging between 0.3 to 0.6 s and on the SLSC building wall between 0.8 to 1.4 s.

6.4 Conclusion

WRL and RHDHV developed a physical modelling program to assess wave overtopping and design wave loading behaviour for the Bronte SLSC site. The 2D physical modelling was undertaken by WRL in accordance with best practice international guidelines. The scope of the program was developed collaboratively between WRL and RHDHV to assist RHDHV to design an optimised seawall for the site.

WRL understands that additional tests in the physical model were considered by RHDHV to provide further detail on wave loading at the SLSC building, however these were not progressed due to program and budgetary constraints.

7 References

Bullock, GN, Crawford, AR, Hewson, PJ, Walkden, MJA & Bird, PAD 2001, ‘The influence of air and scale on wave impact pressures’, *Coastal Engineering*, 42(4), pp.291-312.

Bullock, GN, Obhrai, C, Müller, G, Wolters, G, Peregrine, DH & Bredmose, H 2004, ‘Characteristics and design implications of breaking wave impacts’, Proc. Coastal Engineering 2004: (In 4 Volumes) (pp. 3966-3978).

Bullock, GN, Obhrai, C, Müller, G, Wolters, G, Peregrine, DH & Bredmose, H 2005, ‘Advances in the understanding of wave-impact forces’. *Proc. Coastlines, Structures and Breakwaters*, London: Thomas Telford, pp.111-120.

Bullock, GN, Obhrai, C, Peregrine, DH & Bredmose, H 2007, ‘Violent breaking wave impacts. Part 1: Results from large-scale regular wave tests on vertical and sloping walls’, *Coastal Engineering*, 54(8), pp.602-617.

CIRIA; CUR; CETMEF 2007, The Rock Manual, The Use of Rock in Hydraulic Engineering (2nd edition), C683, CIRIA, London, accessed 10 July 2024, <https://kennisbank-waterbouw.nl/DesignCodes/rockmanual/>.

Cuomo, G, Allsop, W & Takahashi, S 2010, ‘Scaling wave impact pressures on vertical walls’, *Coastal Engineering*, 57(6), pp.604-609.

EurOtop 2018, Manual on wave overtopping of sea defences and related structures: An overtopping manual largely based on European research, but for worldwide application, Van der Meer, JW, Allsop, NWH, Bruce, T, De Rouck, J, Kortenhaus, A, Pullen, T, Schüttrumpf, H, Troch, P & Zanuttigh, B, accessed 20 June 2024, <http://www.overtopping-manual.com/>.

Franco, L, de Gerloni, M, Van der Meer, JW 1994, 'Wave overtopping on vertical and composite breakwaters', Proceedings of Coastal Engineering 1994 - the 24th International Conference, Kobe, Japan, 23-28 October 1994.

Glatz, M, Fitzhenry, M & Kulmar, M 2017, ‘It’s time for an update – extreme waves and directional distributions along the New South Wales Coastline’, 26th NSW Coastal Conference 8-10 November 2017, Port Stephens, accessed 9 April 2024, https://www.coastalconference.com/2017/papers2017/Matthieu%20Glatz%20Full%20paper.pdf.

Hasselmann, K, Barnett, TP, Bouws, E, Carlson, H, Cartwright, DE, Enke, K, Ewing, JA, Gienapp, A, Hasselmann, DE, Kruseman, P, Meerburg, A, Muller, P, Olbers, D, Richter, K, Sell, W and Walden, H, 1973. Measurements of wind-wave growth and swell decay during the Joint North Sea Wave Project (JONSWAP), Deutches Hydrographisches Institut, Hamburg.

HR Wallingford 2021, HR Merlin wave generation program, Software manual for v2.50, AAK2150-RT007-R06-00, June 2021

Hughes, SA 1993, *Physical models and laboratory techniques in coastal engineering*, Advanced Series on Ocean Engineering, World Scientific, USA.

HYDRALAB III 2007, Wolters, G (lead author), Guidelines for Physical Model Testing of Breakwaters: Rubble Mound Breakwaters, Deliverable NA3.1-2, Final Report (Version 1.3), European Commission.

Mansard, EPD & Funke, ER 1980, ‘The Measurement of Incident and Reflected Spectra Using a Least Squares Method’, Proceedings of the 17th International Conference on Coastal Engineering, Sydney, Australia, pp. 154-172.

RHDHV 2023, “2D Physical Model Testing Method Statement”, PA3572-RHDHV-MS-S2-ME-PY-01, Royal HaskoningDHV (Pty) Ltd Water & Maritime.

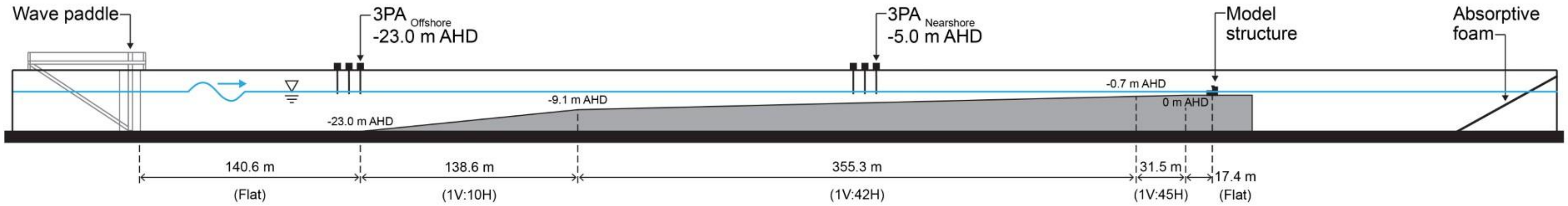
RHDHV 2024a, “Concept Design and Coastal Engineering Assessment Report”, PA3572-RHDHV-RP-S1-RP-FC-0001, Royal HaskoningDHV (Pty) Ltd Water & Maritime.

RHDHV 2024b, RE: 10155 - Bronte SLSC -physical modelling timeline (attachment: “Physical Modelling.dwg”), [E-mail] Message from J Goncalves (joao.goncalves@rhdhv.com) to F Flocard (f.flocard@wrl.unsw.edu.au) and I R Coghlan (i.coghlan@wrl.unsw.edu.au), Sent 20/03/2024 5:03 PM.

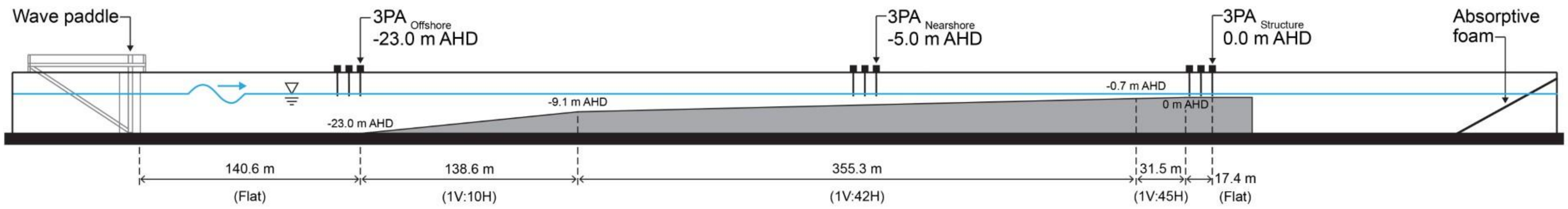
RHDHV 2024c, RE: 10155 - Bronte SLSC -physical modelling timeline, [E-mail] Message from J Goncalves (joao.goncalves@rhdhv.com) to I R Coghlan (i.coghlan@wrl.unsw.edu.au), Sent 04/04/2024 6:37 PM.

US Army Corps of Engineer [USACE] 2006, Coastal Engineering Manual, Engineer Manual 1110-2-1100, Washington D.C., Volumes 1-6, accessed 20 June 2024, <https://www.publications.usace.army.mil/USACE-Publications/Engineer-Manuals/>.

Appendix A Flume setup – Testing



Appendix B Flume setup – Wave climate calibration



Appendix C Design drawings (provided by RHDHV, WRL markup)

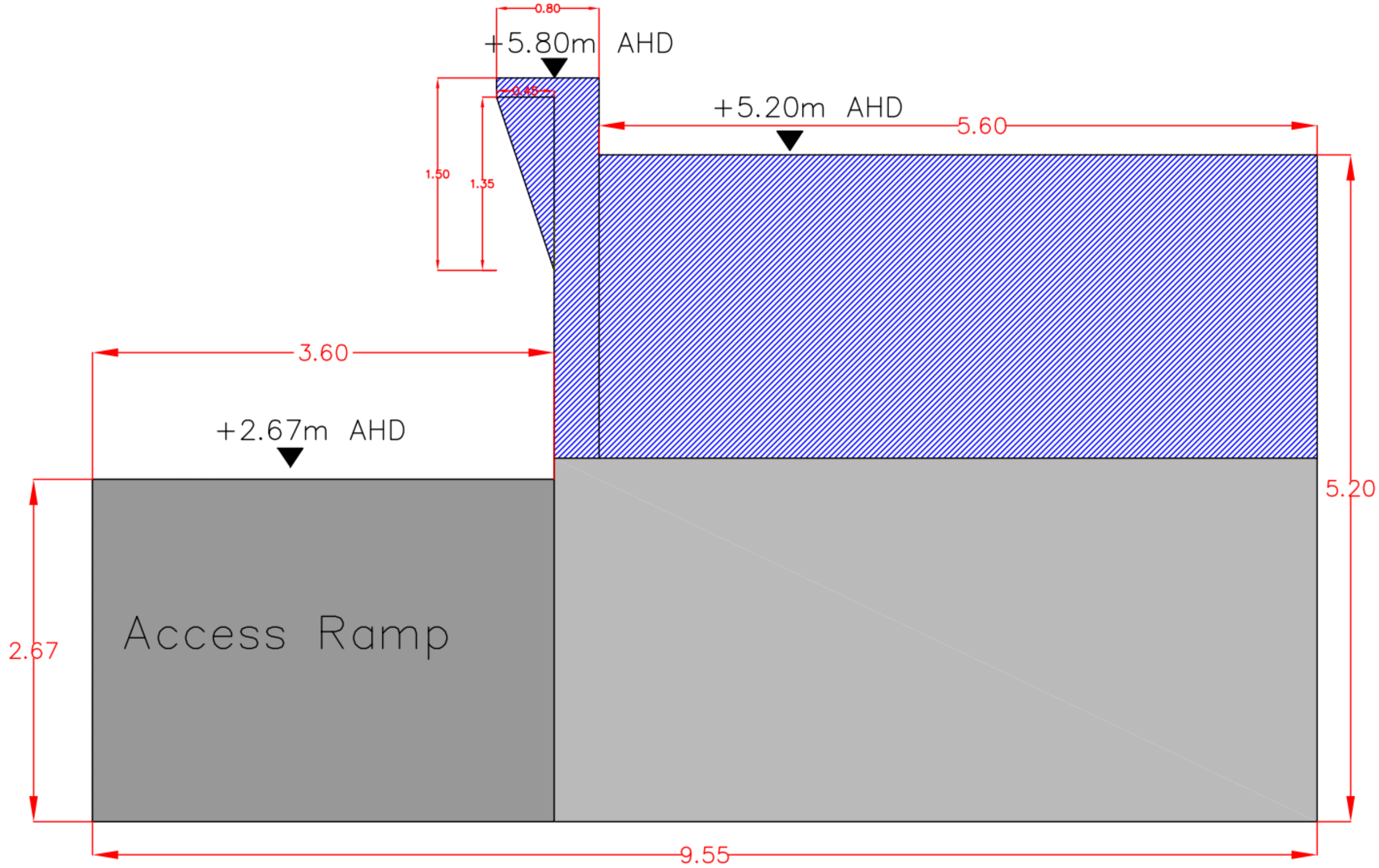


Figure C-1 Vertical Seawall with Wave Deflector (all dimensions in metres)

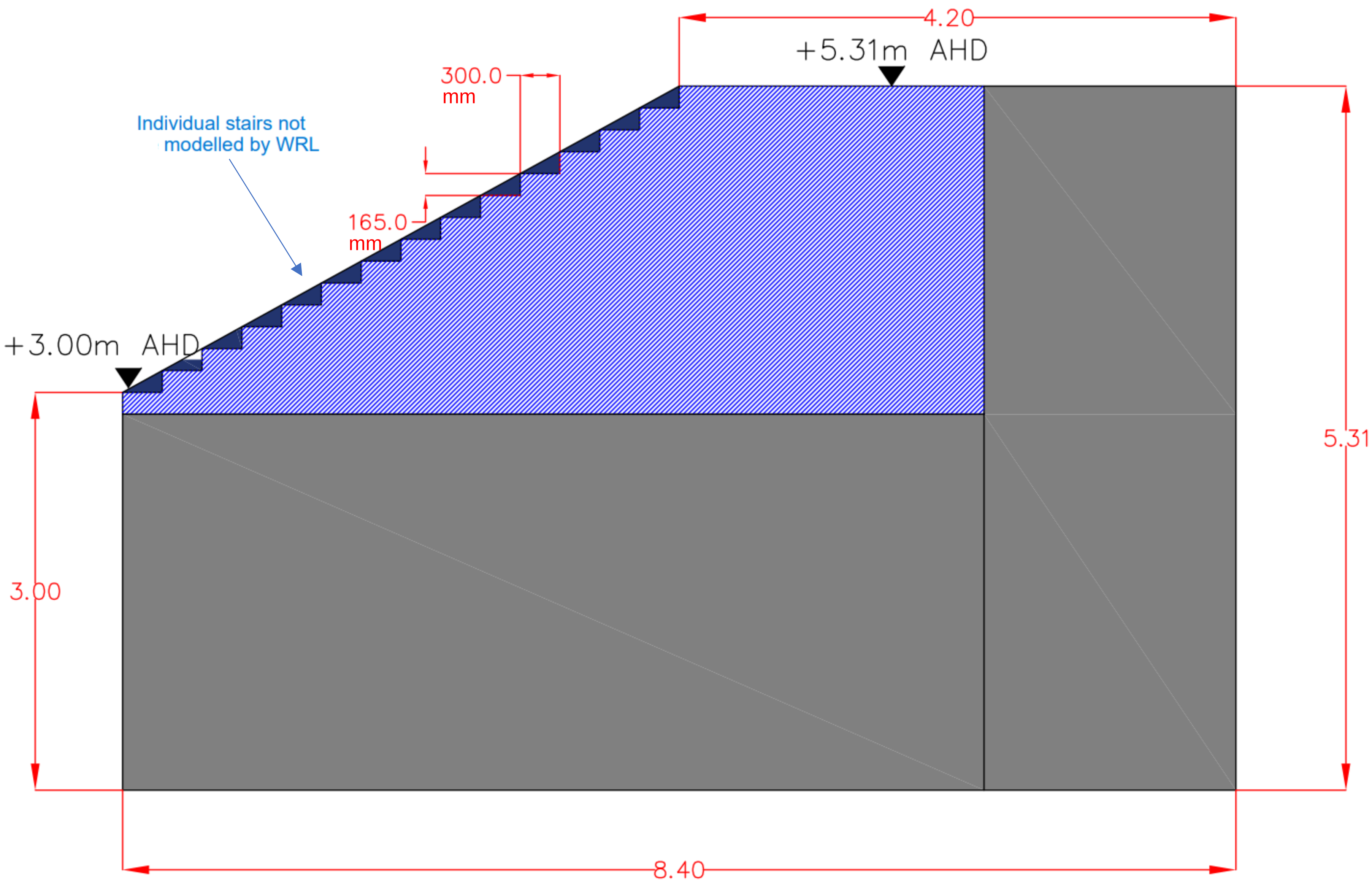


Figure C-2 Stepped Seawall (all dimensions in metres unless otherwise stated)

Appendix D Wave climate calibration

Design Condition 1 - 1 year ARI waves, present day planning level

Name	Wave period (s)			Wave height (m)							Wave reflections removed
	T_{p1}	$T_{m\_1,0}$	T_z	$H_{1/3}$	H_{m0}	H_{avg}	$H_{10\%}$	$H_{5\%}$	$H_{1\%}$	H_{MAX}	
$X1_{Offshore}$	13.2	14.2	10.6	5.98	5.99	3.71	7.76	8.72	10.44	12.28	
$X2_{Offshore}$	13.2	14.3	10.8	5.93	5.94	3.71	7.66	8.54	10.16	12.20	
$X3_{Offshore}$	13.3	18.2	10.9	5.93	5.92	3.72	7.66	8.62	10.59	12.73	
$3PA_{Offshore}$	**13.3**	**11.9**	**10.4**	**5.80**	**5.89**	**3.61**	**7.40**	**8.16**	**9.80**	**11.74**	**x**
$X1_{Nearshore}$	13.5	20.0	10.1	3.95	3.82	2.79	4.55	4.82	5.39	5.98	
$X2_{Nearshore}$	13.5	21.1	10.1	3.74	3.64	2.64	4.26	4.50	4.97	5.64	
$X3_{Nearshore}$	13.6	22.6	10.4	3.61	3.50	2.60	4.11	4.36	4.80	5.48	
$3PA_{Nearshore}$	**13.3**	**11.2**	**8.0**	**3.08**	**3.32**	**2.04**	**3.53**	**3.74**	**4.16**	**4.68**	**x**
$X1_{Structure}$	-	-	14.4	0.93	1.17	0.61	1.21	1.39	1.74	2.08	
$X2_{Structure}$	-	-	13.7	0.95	1.22	0.62	1.22	1.37	1.70	1.98	
$X3_{Structure}$	-	-	14.1	0.83	1.06	0.51	1.12	1.28	1.59	1.72	
$3PA_{Structure}$	**-**	**-**	**7.2**	**0.60**	**0.69**	**0.34**	**0.78**	**0.88**	**1.06**	**1.44**	**x**

* T_{P1} and $T_{m-1,0}$ at the structure (0 m AHD) not included as a significant portion of broken waves resulted in long wave generation.

Wave Condition 2 - 1 year ARI waves, 2093 planning level

Name	Wave period (s)			Wave height (m)							Wave reflections removed
	T_{p1}	$T_{m\_1,0}$	T_z	$H_{1/3}$	H_{m0}	H_{avg}	$H_{10\%}$	$H_{5\%}$	$H_{1\%}$	H_{MAX}	
$X1_{Offshore}$	13.2	14.0	10.7	5.92	5.93	3.70	7.62	8.51	10.17	11.86	
$X2_{Offshore}$	13.2	14.0	10.8	5.86	5.88	3.68	7.55	8.42	10.03	12.13	
$X3_{Offshore}$	13.3	14.1	10.9	5.87	5.87	3.70	7.56	8.47	10.37	12.79	
$3PA_{Offshore}$	**13.3**	**11.9**	**10.4**	**5.74**	**5.84**	**3.56**	**7.34**	**8.09**	**9.67**	**11.52**	**x**
$X1_{Nearshore}$	13.4	17.6	10.0	4.31	4.13	2.96	4.97	5.26	5.80	6.81	
$X2_{Nearshore}$	13.4	17.9	9.9	4.10	3.94	2.82	4.70	4.96	5.38	5.95	
$X3_{Nearshore}$	13.3	19.0	10.3	3.98	3.84	2.82	4.53	4.81	5.22	5.53	
$3PA_{Nearshore}$	**13.2**	**10.9**	**7.9**	**3.44**	**3.67**	**2.26**	**3.95**	**4.16**	**4.58**	**5.03**	**x**
$X1_{Structure}$	-	-	10.6	1.29	1.46	0.84	1.60	1.77	2.08	2.53	
$X2_{Structure}$	-	-	10.7	1.31	1.54	0.86	1.64	1.81	2.12	2.54	
$X3_{Structure}$	-	-	10.7	1.17	1.37	0.74	1.48	1.64	1.99	2.42	
$3PA_{Structure}$	**-**	**-**	**7.3**	**0.86**	**0.98**	**0.53**	**1.07**	**1.16**	**1.37**	**1.73**	**x**

* T_{P1} and $T_{m-1,0}$ at the structure (0 m AHD) not included as a significant portion of broken waves resulted in long wave generation.

Design Condition 3 - 20 year ARI waves, 2093 planning level

Name	Wave period (s)			Wave height (m)							Wave reflections removed
	T_{p1}	$T_{m\_1,0}$	T_z	$H_{1/3}$	H_{m0}	H_{avg}	$H_{10\%}$	$H_{5\%}$	$H_{1\%}$	H_{MAX}	
$X1_{Offshore}$	13.5	13.9	10.9	5.98	6.01	3.75	7.73	8.65	10.35	11.83	
$X2_{Offshore}$	13.5	14.0	10.9	5.91	5.96	3.72	7.63	8.49	10.08	12.05	
$X3_{Offshore}$	13.5	14.0	11.1	5.99	6.00	3.79	7.71	8.63	10.42	12.57	
$3PA_{Offshore}$	**13.5**	**12.2**	**10.7**	**5.83**	**5.94**	**3.65**	**7.43**	**8.22**	**9.86**	**11.64**	**x**
$X1_{Nearshore}$	13.7	15.6	10.1	4.72	4.45	3.16	5.42	5.75	6.34	6.69	
$X2_{Nearshore}$	13.7	16.5	10.3	4.56	4.33	3.11	5.24	5.57	6.11	6.71	
$X3_{Nearshore}$	13.6	17.2	10.4	4.34	4.16	3.03	4.89	5.13	5.67	6.29	
$3PA_{Nearshore}$	**13.6**	**11.0**	**7.8**	**3.71**	**4.01**	**2.43**	**4.23**	**4.47**	**4.92**	**5.74**	**x**
$X1_{Structure}$	-	-	9.8	1.95	2.08	1.28	2.34	2.53	2.91	3.24	
$X2_{Structure}$	-	-	9.9	1.81	1.93	1.17	2.22	2.42	2.86	3.59	
$X3_{Structure}$	-	-	10.1	1.71	1.83	1.08	2.13	2.33	2.75	3.16	
$3PA_{Structure}$	**-**	**-**	**7.0**	**1.20**	**1.39**	**0.76**	**1.45**	**1.57**	**1.80**	**2.31**	**x**

* T_{P1} and $T_{m-1,0}$ at the structure (0 m AHD) not included as a significant portion of broken waves resulted in long wave generation.

Design Condition 4 - 100 year ARI waves, present day planning level

Name	Wave period (s)			Wave height (m)							Wave reflections removed
	T_{p1}	$T_{m\_1,0}$	T_z	$H_{1/3}$	H_{m0}	H_{avg}	$H_{10\%}$	$H_{5\%}$	$H_{1\%}$	H_{MAX}	
$X1_{Offshore}$	14.8	15.9	11.7	5.88	5.87	3.65	7.66	8.65	10.64	12.36	
$X2_{Offshore}$	14.8	15.9	11.7	5.91	5.86	3.65	7.70	8.69	10.54	12.08	
$X3_{Offshore}$	14.9	15.9	11.9	5.96	5.92	3.70	7.70	8.62	10.36	12.16	
$3PA_{Offshore}$	**14.8**	**13.2**	**11.7**	**5.80**	**5.84**	**3.61**	**7.45**	**8.28**	**9.85**	**11.32**	**x**
$X1_{Nearshore}$	15.1	18.9	10.1	4.52	4.21	2.92	5.25	5.59	6.22	6.78	
$X2_{Nearshore}$	15.0	19.8	10.4	4.35	4.03	2.88	5.04	5.37	5.96	6.50	
$X3_{Nearshore}$	15.0	22.4	10.6	4.18	3.89	2.81	4.81	5.08	5.48	6.00	
$3PA_{Nearshore}$	**14.8**	**11.8**	**8.1**	**3.44**	**3.69**	**2.21**	**3.94**	**4.15**	**4.56**	**5.01**	**x**
$X1_{Structure}$	-	-	9.9	1.55	1.75	0.95	1.93	2.11	2.43	3.00	
$X2_{Structure}$	-	-	10.4	1.39	1.59	0.87	1.75	1.92	2.26	2.84	
$X3_{Structure}$	-	-	10.5	1.37	1.58	0.84	1.77	1.98	2.42	2.94	
$3PA_{Structure}$	**-**	**-**	**7.1**	**0.95**	**1.11**	**0.58**	**1.18**	**1.29**	**1.52**	**1.78**	**x**

* T_{P1} and $T_{m-1,0}$ at the structure (0 m AHD) not included as a significant portion of broken waves resulted in long wave generation.

Design Condition 5 - 100 year ARI waves, 2093 planning level

Name	T_{p1}	$T_{m\_1,0}$	T_z	$H_{1/3}$	H_{m0}	H_{avg}	$H_{10\%}$	$H_{5\%}$	$H_{1\%}$	H_{MAX}	Wave reflections removed
	Wave period (s)			Wave height (m)							
$X1_{Offshore}$	14.8	15.8	11.8	5.97	5.99	3.73	7.71	8.67	10.56	12.42	
$X2_{Offshore}$	14.9	15.8	11.7	5.97	5.95	3.69	7.75	8.70	10.50	12.08	
$X3_{Offshore}$	14.9	15.7	11.9	6.00	6.01	3.75	7.72	8.61	10.29	11.83	
$3PA_{Offshore}$	**14.9**	**13.2**	**11.8**	**5.90**	**5.95**	**3.70**	**7.55**	**8.40**	**9.98**	**11.49**	**x**
$X1_{Nearshore}$	15.1	17.5	10.7	4.77	4.49	3.15	5.49	5.80	6.48	7.54	
$X2_{Nearshore}$	15.0	18.2	10.7	4.61	4.36	3.07	5.28	5.62	6.30	7.59	
$X3_{Nearshore}$	15.0	19.2	10.9	4.44	4.19	3.02	5.06	5.33	5.83	6.37	
$3PA_{Nearshore}$	**14.9**	**11.7**	**8.3**	**3.79**	**4.01**	**2.43**	**4.33**	**4.56**	**5.00**	**6.15**	**x**
$X1_{Structure}$	-	-	10.4	2.06	2.20	1.34	2.49	2.71	3.12	3.86	
$X2_{Structure}$	-	-	11.2	1.96	2.07	1.28	2.40	2.60	3.07	3.56	
$X3_{Structure}$	-	-	11.0	1.86	1.98	1.19	2.32	2.57	3.05	3.51	
$3PA_{Structure}$	**-**	**-**	**8.3**	**1.31**	**1.46**	**0.83**	**1.58**	**1.70**	**1.95**	**2.23**	**x**

* T_{P1} and $T_{m-1,0}$ at the structure (0 m AHD) not included as a significant portion of broken waves resulted in long wave generation.

Design Condition 6 - 500 year ARI waves, 2093 planning level

Name	T_{p1}	$T_{m\_1,0}$	T_z	$H_{1/3}$	H_{m0}	H_{avg}	$H_{10\%}$	$H_{5\%}$	$H_{1\%}$	H_{MAX}	Wave reflections removed
	Wave period (s)			Wave height (m)							
$X1_{Offshore}$	15.3	17.2	12.1	6.44	6.36	3.96	8.32	9.12	10.83	12.58	
$X2_{Offshore}$	15.3	17.2	12.0	6.41	6.32	3.93	8.32	9.18	10.97	12.80	
$X3_{Offshore}$	15.3	16.9	12.0	6.46	6.37	3.95	8.36	9.23	11.05	12.65	
$3PA_{Offshore}$	**15.3**	**13.4**	**11.8**	**6.30**	**6.30**	**3.86**	**8.10**	**8.89**	**10.46**	**12.40**	**x**
$X1_{Nearshore}$	15.2	20.3	10.9	4.88	4.62	3.25	5.61	5.94	6.53	7.20	
$X2_{Nearshore}$	15.1	21.8	11.0	4.71	4.48	3.17	5.43	5.77	6.35	7.39	
$X3_{Nearshore}$	15.1	23.7	11.1	4.50	4.29	3.09	5.18	5.51	5.96	6.42	
$3PA_{Nearshore}$	**15.1**	**11.9**	**8.5**	**3.90**	**4.09**	**2.48**	**4.50**	**4.76**	**5.27**	**6.07**	**x**
$X1_{Structure}$	-	-	10.8	2.12	2.28	1.38	2.59	2.82	3.24	3.61	
$X2_{Structure}$	-	-	11.2	2.01	2.14	1.29	2.45	2.68	3.09	3.82	
$X3_{Structure}$	-	-	10.9	1.89	2.03	1.19	2.36	2.59	3.07	3.54	
$3PA_{Structure}$	**-**	**-**	**8.0**	**1.32**	**1.48**	**0.83**	**1.60**	**1.74**	**2.02**	**2.50**	**x**

* T_{P1} and $T_{m-1,0}$ at the structure (0 m AHD) not included as a significant portion of broken waves resulted in long wave generation.