Newport SLSC stepped seawall physical modelling

WRL TR 2024/20, August 2024

By J T Carley and Y Doherty

UNSW Water Research Laboratory

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Project details

Report title	Newport SLSC stepped seawall physical modelling
Authors(s)	J T Carley and Y Doherty
Report no.	2024/20
Report status	Final
Date of issue	August 2024
WRL project no.	2024007
Project manager	James Carley
Client	King Wood Mallesons on behalf of Northern Beaches Council
Client address	Level 61, Governor Phillip Tower 1 Farrer Place Sydney NSW 2000
Client contact	Stella Zhao (KWM); Eskil Julliard (NBC)
Client reference	#

Document status

Version	Reviewed by	Approved by	Date issued
Final	FF	FF	06/08/24

This report should be cited as: Carley, J.T. and Doherty, Y., Newport SLSC stepped seawall physical modelling, WRL Technical Report 2024/20, UNSW Water Research Laboratory.

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Contents

1 Introduction **1**
2 Study objectives **5**
3 Model setup and operation **7**
3.1 Testing facility 7
3.2 Model design and scaling 7
3.2.1 Overview *7*
3.2.2 Commentary on alternative scaling laws for force *8*
3.3 Model construction and setup 9
3.3.1 Bathymetry *9*
3.3.2 Model structures *12*
3.4 Data collection and analysis 20
3.4.1 Wave data *20*
3.4.2 Overtopping *20*
3.4.3 Wave loads *22*
3.4.4 Media and data files sharing *25*
4 Wave climate calibration **26**
4.1 Design wave conditions 26
4.2 Results 27
5 Overtopping results **30**
5.1 Preamble 30
5.2 Guidelines for overtopping 30
5.3 Overtopping 30
5.4 Discussion 35
6 Wave forces/pressures **39**
6.1 Overview 39
7 Summary **47**
7.1 Overview 47
7.2 Overtopping 48
7.3 Wave loads 48
7.4 Conclusion 48
8 References **49**
Appendix A Model design inputs **A-1**
Appendix B Previous WRL coastal engineering advice **B-1**

List of tables

Table 3.1 Model scale ratio 7
Table 3.2 Key structure component dimensions 14
Table 3.3 Wave measurement locations 20
Table 4.1 Design wave conditions 26
Table 4.2 Measured wave climate conditions and total water level 29
Table 5.1 Mean overtopping volumes – Design 1, original DA 32
Table 5.2 Mean overtopping volumes – Design 2, BCA stairs and bleachers 33
Table 5.3 Mean overtopping volumes – Design 3, Quasi 3D stairs and bleachers 34
Table 6.1 P_{max} on SLSC wall – Design 4, wave return parapet (old SLSC) 40
Table 6.2 P_{max} on SLSC wall – Design 4, no wave return parapet (new SLSC) 40

List of figures

Figure 1.1 Aerial photo of the project site (Nearmap 21/01/2024) 2
Figure 1.2 Original SLSC building, 1933 2
Figure 1.3 Storm damage and emergency works, 28 May 1974 (Horton, 2020) 3
Figure 1.4 Boulder wall fronting SLSC building, December 1974 (Horton, 2020) 3
Figure 1.5 Overview of initial DA seawall (Horton, 2020) 4
Figure 3.1 Bathymetry transect (yellow line) modelled by WRL 10
Figure 3.2 Idealised bathymetry for modelling 10
Figure 3.3 Modelled bathymetries: eroded and 1V:10H ramp (detailed view at structure) 11
Figure 3.4 Average (red) and eroded (yellow) profiles in flume 12
Figure 3.5 Generic dimensions of components of structures tested 13
Figure 3.6 Design 1, Original DA seawall (eroded profile) 15
Figure 3.7 Design 1, Original DA seawall (eroded profile), preliminary recurved parapet 15
Figure 3.8 Design 2, BCA compliant stair seawall (eroded profile) 16
Figure 3.9 Design 2, BCA compliant 4 bleachers seawall (eroded profile) 16
Figure 3.10 Design 2, BCA compliant 4 bleachers seawall (eroded profile), recurved parapet 17
Figure 3.11 Design 2, BCA compliant 4 bleachers seawall (eroded profile), splayed parapet 17
Figure 3.12 Design 3, Quasi 3D stair, 3 bleachers (eroded profile), recurved parapet 18
Figure 3.13 Design 4, 4 bleachers and SLSC wall (eroded profile), no parapet 18
Figure 3.14 Design 4, 4 bleachers and SLSC wall (eroded profile), recurved parapet 19
Figure 3.15 Arrangement for measurement of overtopping 21
Figure 3.16 Cumulative overtopping timeseries for (V_{max}) 22
Figure 3.17 Load testing arrangement for the SLSC building 23
Figure 3.18 Landward view of flume arrangement for wave load testing 24
Figure 3.19 Side view of flume arrangement for wave load testing 24
Figure 5.1 EurOtop (2018) guidelines for tolerable overtopping 30
Figure 5.2 Overtopping timeseries for quasi 3D seawall with parapet 35
Figure 5.3 Overtopping timeseries for 3 bleacher seawall with and without parapet 36
Figure 5.4 V_{max} event for quasi 3D stairs100 year ARI 2024 36

Figure 5.5 V_{max} event for quasi 3D stairs1,000 year ARI 2024 37
Figure 5.6 V_{max} event for quasi 3D stairs100 year ARI 2084 37
Figure 5.7 V_{max} event for quasi 3D stairs1,000 year ARI 2084 38
Figure 6.1 Pressure lower SLSC wall (1.5 m height) 100 year ARI 2024 with parapet 41
Figure 6.2 P_{max} lower SLSC wall (1.5 m height) 100 year ARI 2024 with parapet 41
Figure 6.3 Pressure lower SLSC wall (1.5 m height) 1,000 year ARI 2024 with parapet 42
Figure 6.4 P_{max} lower SLSC wall (1.5 m height) 1,000 year ARI 2024 with parapet 42
Figure 6.5 Pressure lower SLSC wall (1.5 m height) 100 year ARI 2084 with parapet 43
Figure 6.6 P_{max} lower SLSC wall (1.5 m height) 100 year ARI 2084 with parapet 43
Figure 6.7 Pressure lower SLSC wall (1.5 m height) 1,000 year ARI 2084 with parapet 44
Figure 6.8 P_{max} lower SLSC wall (1.5 m height) 1,000 year ARI 2084 with parapet 44
Figure 6.9 P_{max} lower SLSC wall (1.5 m height) 100 year ARI 2024 with parapet 45
Figure 6.10 P_{max} lower SLSC wall (1.5 m height) 1,000 year ARI 2024 with parapet 45
Figure 6.11 P_{max} lower SLSC wall (1.5 m height) 100 year ARI 2084 with parapet 46
Figure 6.12 P_{max} lower SLSC wall (1.5 m height) 1,000 year ARI 2084 with parapet 46

Glossary

AHD	Australian Height Datum
ARI (year)	Annual Recurrence Interval (year)
H_s (m)	Significant wave height
JONSWAP	Joint North Sea Wave Project spectra
MHWS	Mean High Water Spring tide level
SLR	Sea Level Rise
SWL	Still Water Level
Tp (s)	Peak wave period
Tz (s)	Mean wave period
WRL	Water Research Laboratory

1 Introduction

Alterations and additions to Newport Surf Life Saving Club (SLSC) in the Northern Beaches Council area of Sydney (NSW) are proposed. An aerial photo of the project site is provided in Figure 1.1. The existing clubhouse was built in approximately 1933 (Figure 1.2).

The existing building is vulnerable to coastal hazards (coastal erosion, recession due to sea level rise, and coastal inundation in the form of wave runup and overtopping). An existing rock boulder seawall was placed seaward of the existing clubhouse in the aftermath of the May 1974 *Sygna* storm (Figure 1.3 and Figure 1.4), but these are not to a certifiable engineering standard. Therefore, engineered coastal protection works will be required to provide protection to the existing clubhouse and assist with protecting the extension over a 60 year future design life.

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney was engaged by King Wood Mallesons (KWM) on behalf of Northern Beaches Council (Council) to undertake two-dimensional (2D) and limited quasi three-dimensional (Q3D) physical modelling of seawall and stair cross-sections proposed to front Newport SLSC. Technical advice was also provided to Council by Mr Greg Britton of Haskoning Australia (RHDHV) and Ms Louise Collier of Rhelm Australia (Rhelm).

An overview of the initial DA proposed seawall to be modelled and tested is shown in Figure 1.5.

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

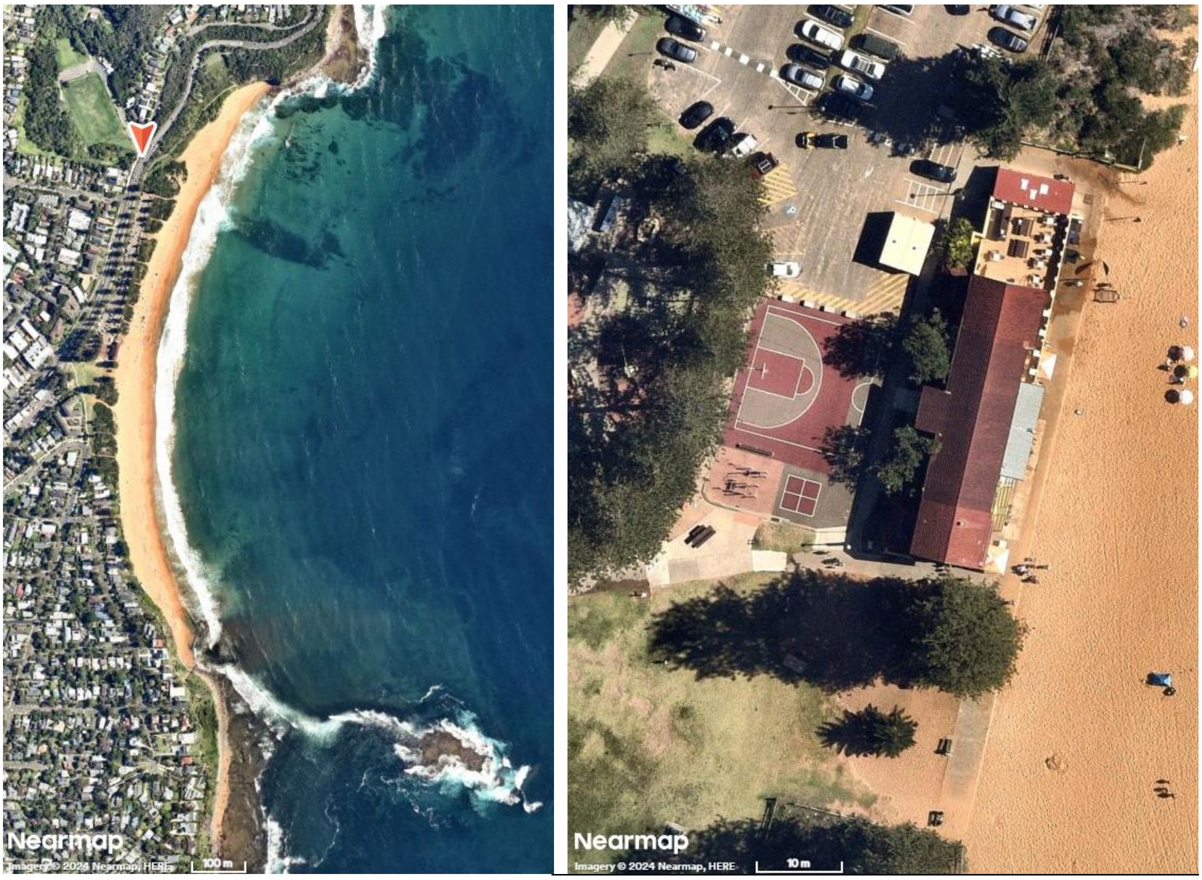


Figure 1.1 Aerial photo of the project site (Nearmap 21/01/2024)

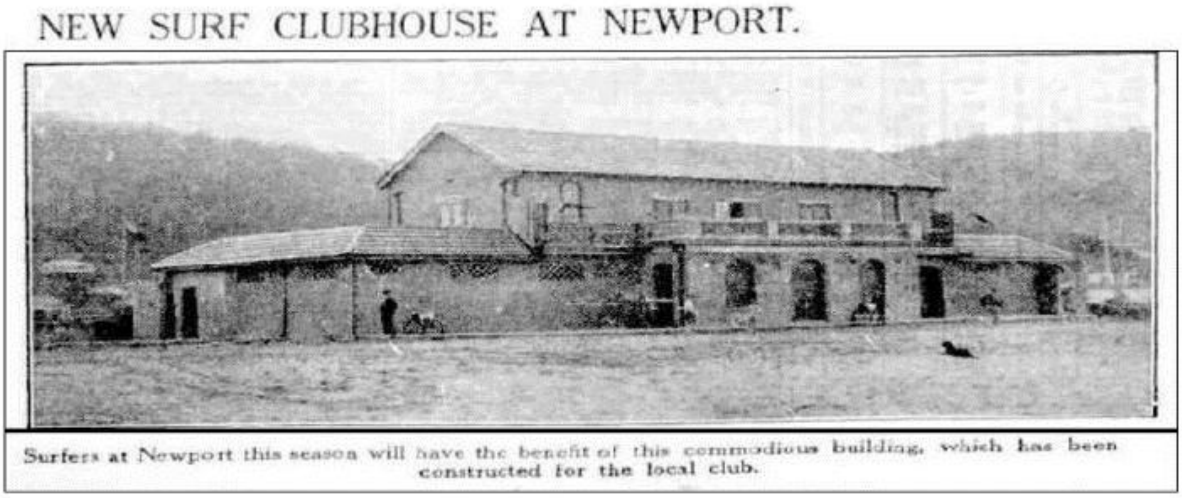


Figure 1.2 Original SLSC building, 1933

Figure 1.3 Storm damage and emergency works, 28 May 1974 (Horton, 2020)

Figure 1.4 Boulder wall fronting SLSC building, December 1974 (Horton, 2020)

(Top: accreted sand levels; Bottom: eroded sand levels)

Figure 1.5 Overview of initial DA seawall (Horton, 2020)

2 Study objectives

WRL, the client and their technical advisors developed a physical modelling program to assess wave overtopping and design wave loading behaviour for the proposed alterations and additions to Newport SLSC. Numerous combinations of design conditions and inputs were tested. The physical modelling focused on wave overtopping flow impacts on the SLSC precinct, including the SLSC building and surrounds.

The following material supplements this report:

- Appendix A: Development of the model inputs
- Appendix B: Desktop coastal engineering advice

The evolution of designs tested in the model is shown in Section 3 (Figure 3.6 to Figure 3.14) and included:

1. The original DA design (Horton, 2020) of a sloping seawall incorporating trafficable stairs
2. A minor modification to the DA design, incorporating stair dimensions complying with the Building Code of Australia (BCA) and alternating areas of trafficable stairs and larger terraced seats (“bleachers”)
3. A quasi three dimensional stair arrangement and three bleachers fronting the main entry of the SLSC building
4. A four bleachers 2D configuration
5. Wave deflector parapet walls of approximately 1 m height and various configurations, including
    a. Straight/vertical and recurved 800 mm high
    b. Recurved 1050 mm high with a 700 mm radius on the seaward face
    c. Splayed at 45° on the seaward face and 1050 mm high

Tests were conducted for the following events (present day) and included consideration of future sea level rise (SLR) of 0.53 m in 2084:

- 1 year ARI
- 10 year ARI
- 100 year ARI
- 1,000 year ARI

Overtopping testing of the structures was conducted with representative average and eroded nearshore profiles (Section 3). Comparative tests between configurations were generally undertaken for 100 year ARI conditions, mostly with an eroded beach state.

Extreme structural wave load tests were undertaken for 100 and 1,000 year ARI conditions to inform the structural design for the wave deflector parapet and the seaward ground floor walls of the proposed SLSC building.

The physical modelling was undertaken by WRL in accordance with best practice international guidelines. The scope of the program was developed collaboratively between WRL and the client to optimise the proposed seawall for the site.

For the vertical seawall with wave deflector, WRL also conducted wave load testing on the wave deflector and the 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 1.2 m wide wave flume. The flume's dimensions are 44 m (length) by 1.2 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.

The flume has a piston type wave generator powered by a hydraulic 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 1:25. Selection of the length ratio was primarily based on the upper limit wave height able to be generated in the 1.2 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	25
Time	s	$N_L^{0.5}$	5.0
Overtopping volume per unit length	L/m	N_L^2	625
Overtopping rate per unit length	L/s/m	$N_L^{1.5}$	125
Water density	kg/m^3	$N_{\gamma w}$	1.026
Force per unit length	kN/m	$N_L^2 N_{\gamma w}$	641.25
Pressure	kN/m^2; kPa	$N_L N_{\gamma w}$	25.65

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.
- 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 not fully 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 full scale salt water ocean waves will be lower than those predicted by Froude scaling of fresh water, as the full scale salt water breaking/broken waves have greater aeration (“foaming”), (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 Newport SLSC seawall, individual waves generated both “pulsating” and “impulsive” vertical loads on the wave deflector and the 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 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

Further details are provided in Appendix A. A false floor was constructed in the wave flume from water-resistant plywood (orange line in Figure 3.3) with the following characteristics representing accreted, average and an eroded profile.

The NSW Department of Planning, Industry and Environment (NSW DPIE), provides topographic and bathymetric data based on Airborne LiDAR Bathymetry (ALB) technology conducted by Fugro Pty Ltd from July to December 2018. The bathymetric data was accessed through the ELVIS portal (https://elevation.fsdf.org.au/) and downloaded at a resolution of 5 m.

Analysis of this bathymetric data indicates that the nearshore seabed slope fronting the proposed seawall is relatively mild and constant across the embayment (Figure 3.1 and Figure 3.2). It can be idealized (see dashed line in Figure 3.2) as:

- 1V:45H between -20 m AHD and -10 m AHD
- Slightly steepening to 1V:35H between -10 m AHD and -2 m AHD
- A relatively flat 50 m wide intertidal terrace and an upper beach face of about 1V:10H

Based on analysis of more than 50 years of beach profile data and modelling (Appendix B), the following sand levels against the seawall were modelled:

- Accreted beach: 5.8 m AHD
- Average beach: 4.0 m AHD
- Eroded beach: -1 m AHD

Note that during eroded conditions, an offshore storm bar is likely to form, causing the beach to become more dissipative, and therefore further reducing nearshore wave heights. As no data was available for this, the single measured offshore profile was used. This would result in slightly larger waves at the seawall in the physical model, making the model somewhat conservative. Similarly, Appendix B canvassed future beach states due to sea level rise, but did not derive any scour level deeper than -1 m AHD. Therefore, -1 m AHD at the structure was adopted in the model for an eroded beach state.

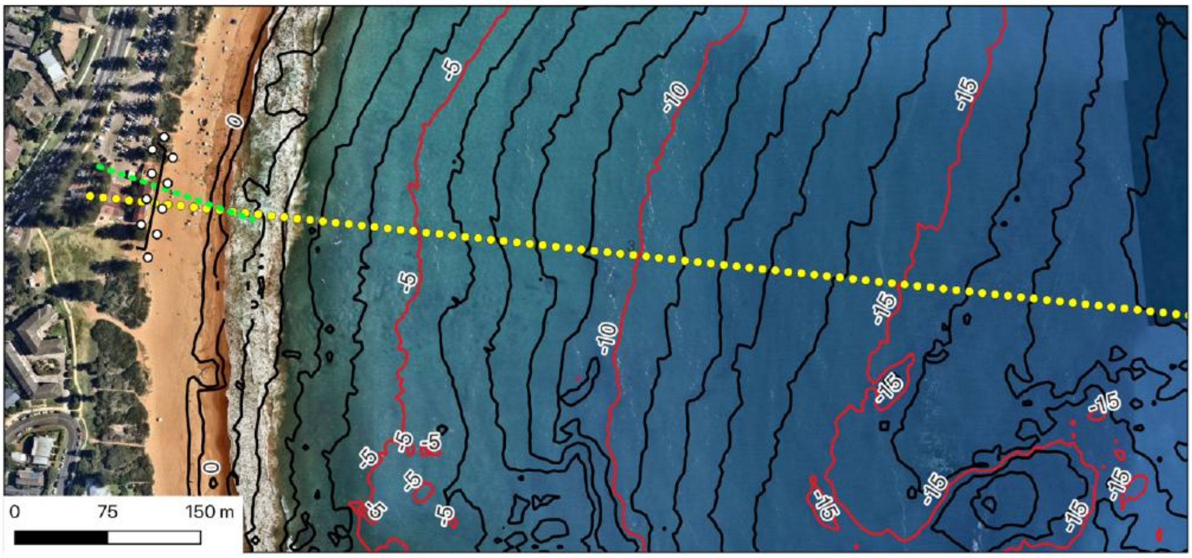


Figure 3.1 Bathymetry transect (yellow line) modelled by WRL

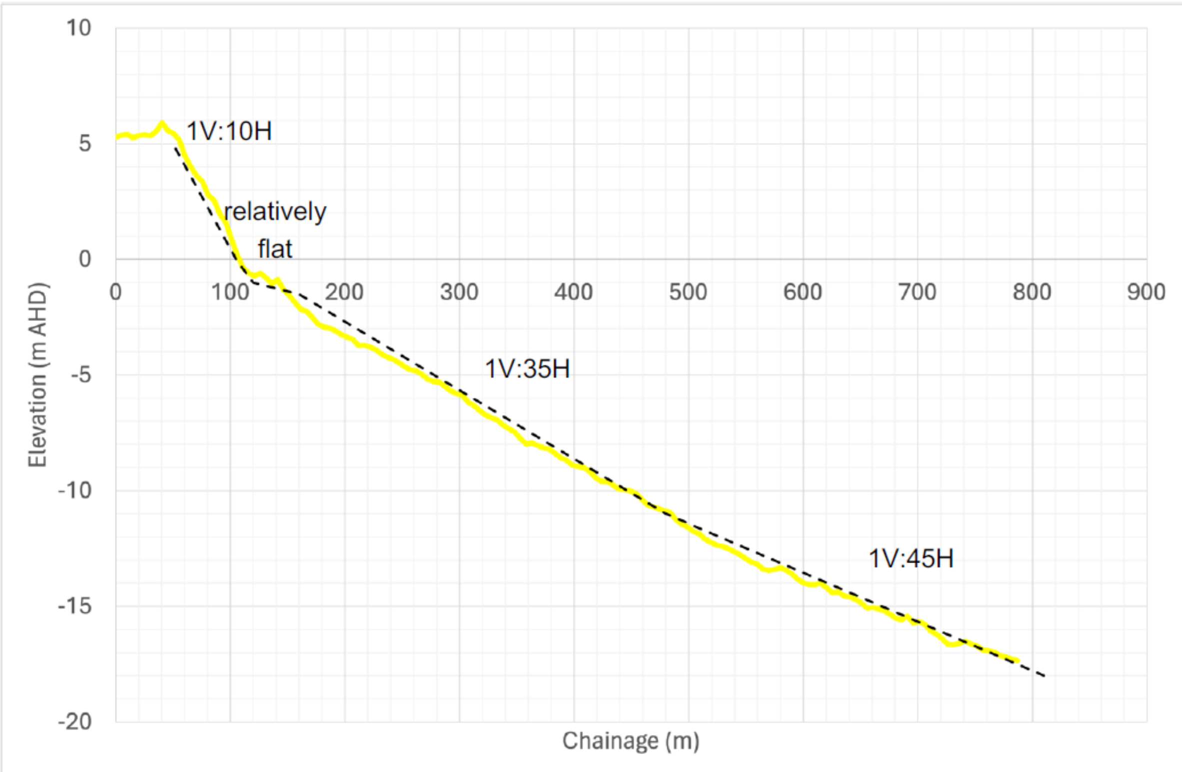


Figure 3.2 Idealised bathymetry for modelling

The modelled bathymetry layout within the wave flume is shown in Figure 3.2 and with the original DA seawall (distorted scale) shown in Figure 3.3.

This model bathymetry was representative of the site bathymetry for a distance of at least 8 wavelengths (approximately 800 m) seaward of the model seawall structure to -18 m AHD, which is in accordance with the minimum recommended value of 3 wavelengths by HYDRALAB III (2007).

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

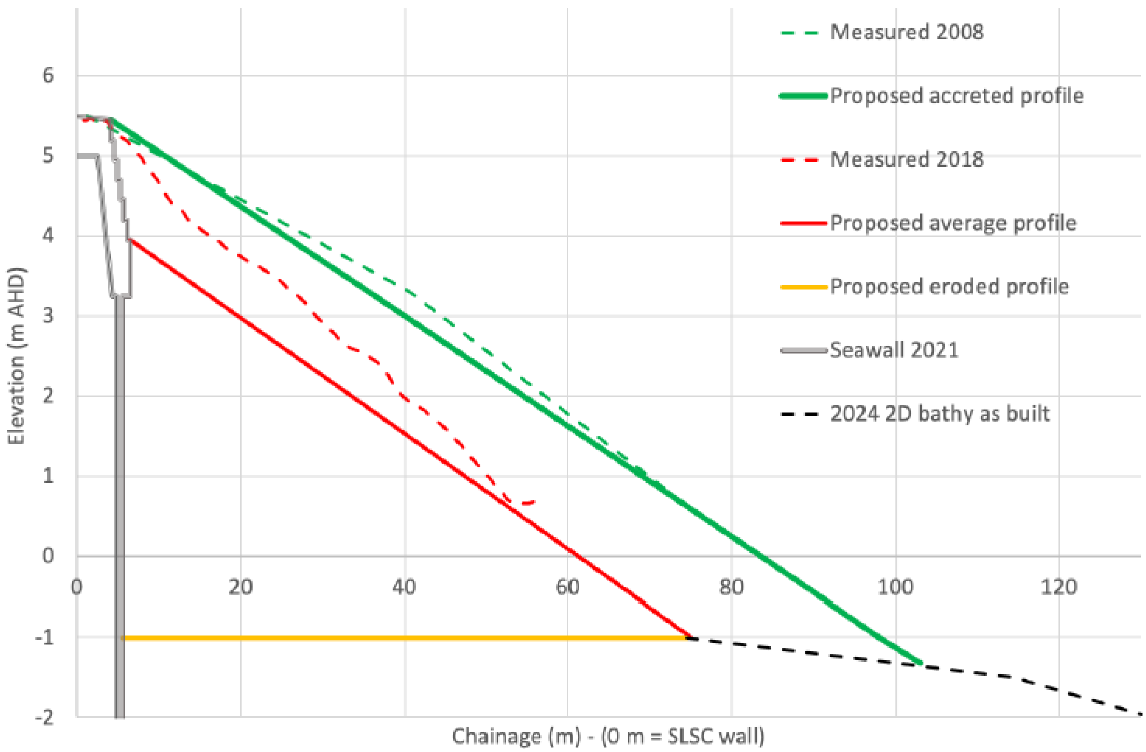


Figure 3.3 Modelled bathymetries: eroded and 1V:10H ramp (detailed view at structure)

3.3.2 Model structures

The 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 -1 m AHD. The 1V:10H representative average profile was positioned on top of the eroded profile as indicated in Figure 3.4.

Figure 3.4 Average (red) and eroded (yellow) profiles in flume

Generic components of the key structures are shown in Figure 3.5 with the key structure dimensions for a shown in Table 3.2. Annotated photos of the constructed models in the flume are provided in Figure 3.6 to Figure 3.14.

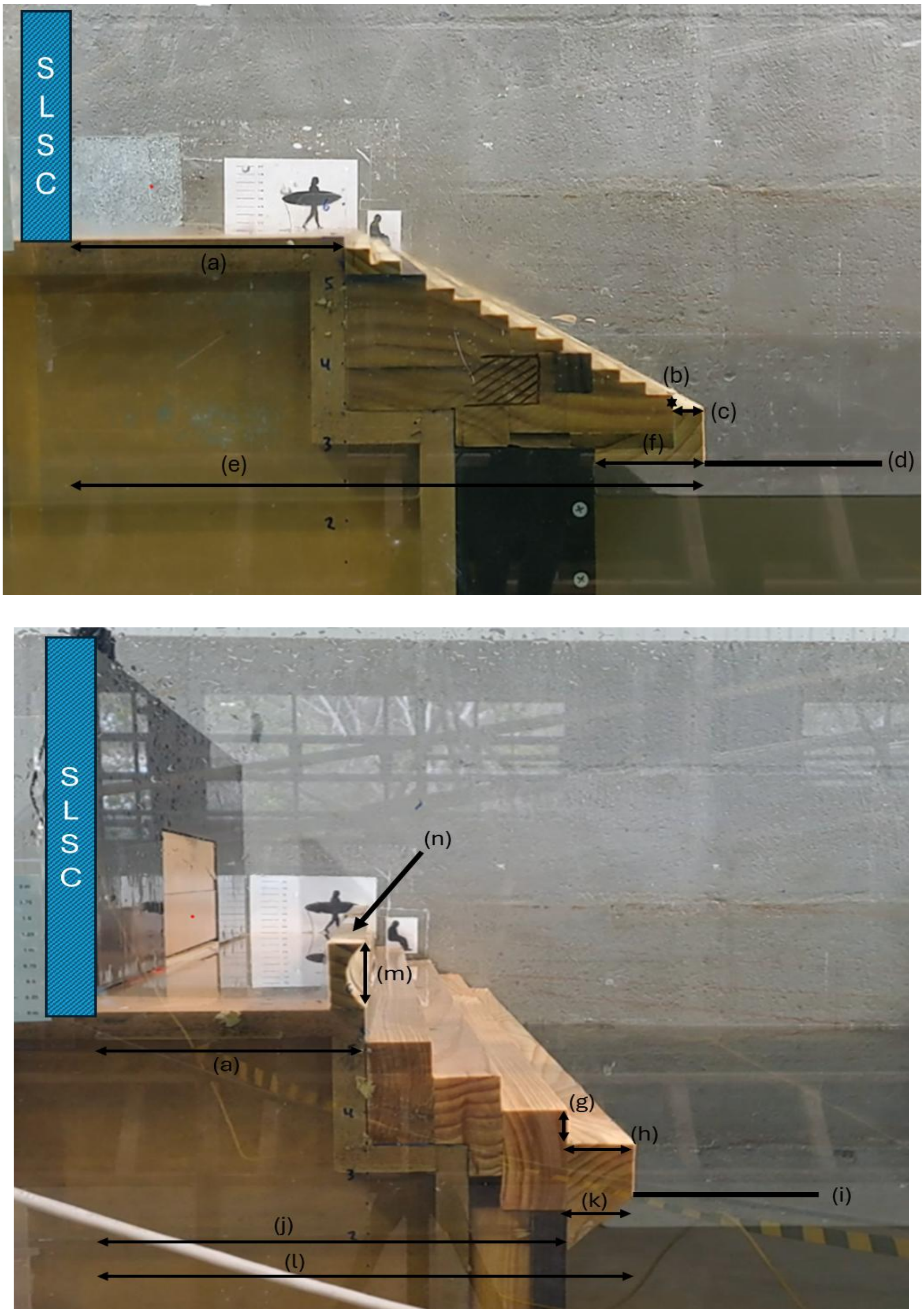


Figure 3.5 Generic dimensions of components of structures tested

Table 3.2 Key structure component dimensions

#			(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)	(m)	(n)
	Design	Test numbers	Min Prom width (m)	Stair riser (mm)	Stair going (mm)	Stair soffit (m AHD)	SLSC to stair limit (m)	Stair overhang (m)	Bleacher riser (mm)	Bleacher going (mm)	Bleacher soffit (m AHD)	SLSC to vertical wall (m)	Bleacher overhang (m)	SLSC to bleacher limit (m)	Parapet height (where fitted) (m)	Parapet shape (where fitted)
1	Horton 2020 DA	24-53	4.1	250	400	3.00	6.50	1.00	-	-	-	5.50	-	-	0.80	Recurved, straight
2	BCA complying, bleachers	54-73	4.1	170	350	2.59	8.65	1.35	510	1050	2.76	7.30	1.00	8.30	1.05	Recurved, splayed
3	Quasi 3D stairs, 3 bleachers	74-84	4.1	170	350	2.99	7.25	1.05	510	1050	2.99	6.20	1.05	7.25	1.05	Recurved
4	4 bleachers	85-94	4.0	190	300	2.92	7.60	0.90	570	900	2.92	6.70	0.90	7.60	1.05	Recurved

Figure 3.6 Design 1, Original DA seawall (eroded profile)

Figure 3.7 Design 1, Original DA seawall (eroded profile), preliminary recurved parapet

Figure 3.8 Design 2, BCA compliant stair seawall (eroded profile)

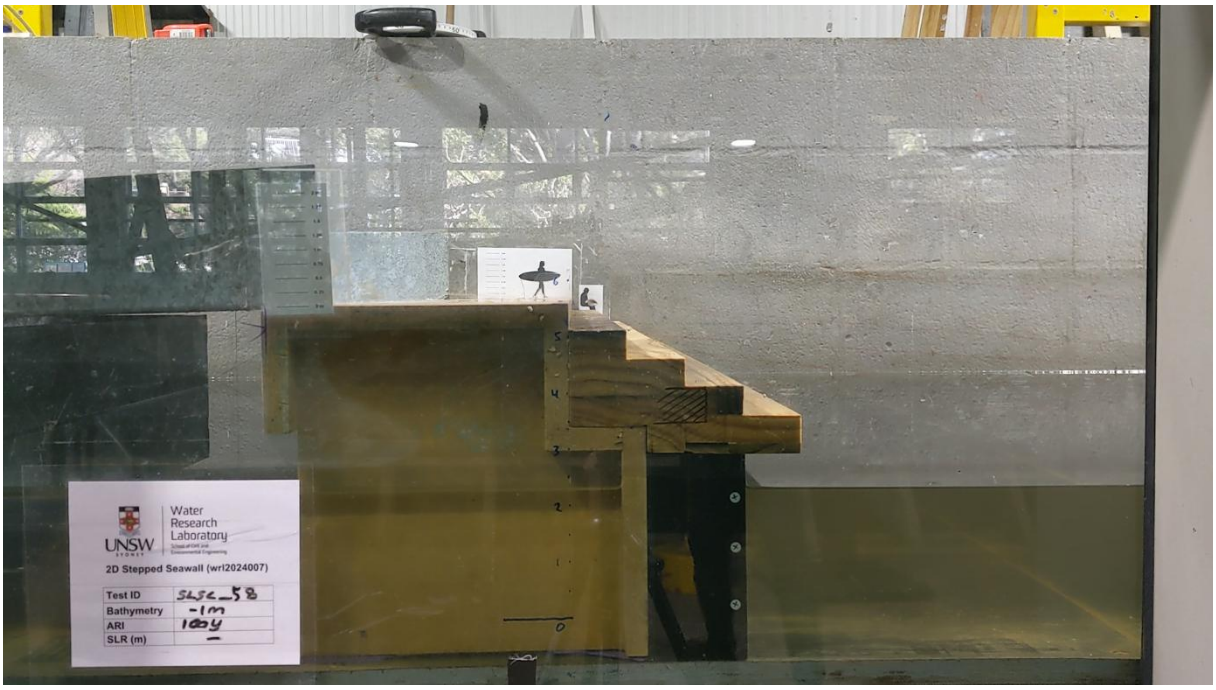


Figure 3.9 Design 2, BCA compliant 4 bleachers seawall (eroded profile)

Figure 3.10 Design 2, BCA compliant 4 bleachers seawall (eroded profile), recurved parapet

Figure 3.11 Design 2, BCA compliant 4 bleachers seawall (eroded profile), splayed parapet

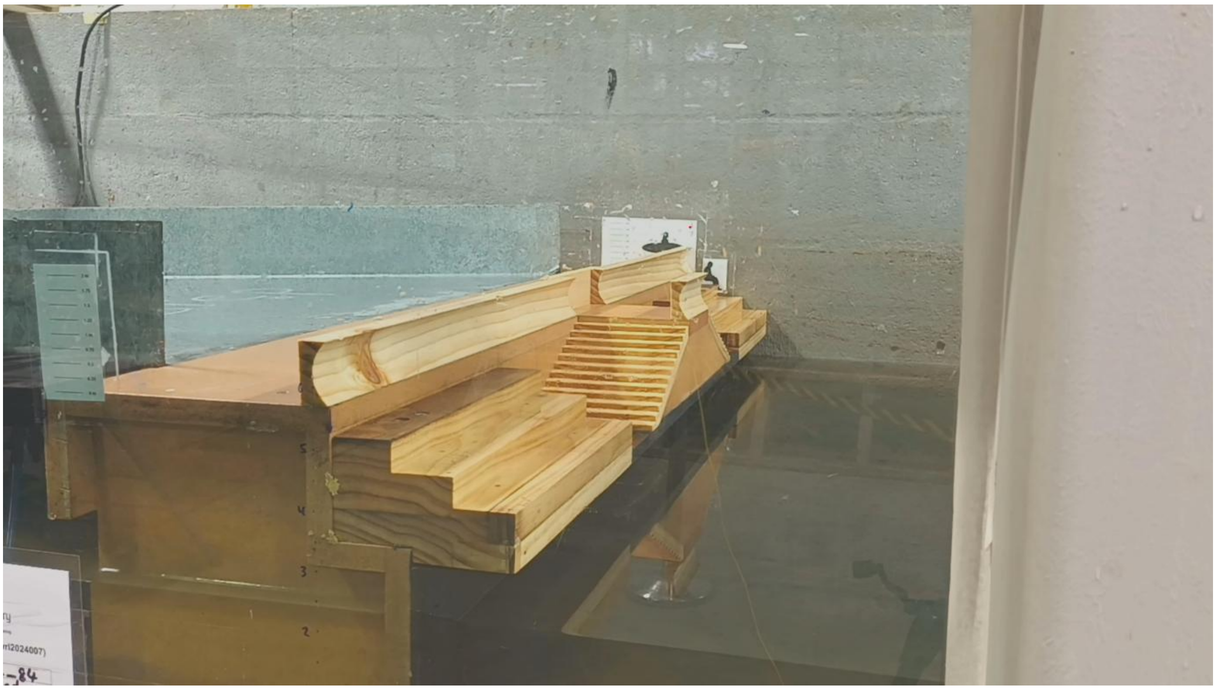


Figure 3.12 Design 3, Quasi 3D stair, 3 bleachers (eroded profile), recurved parapet

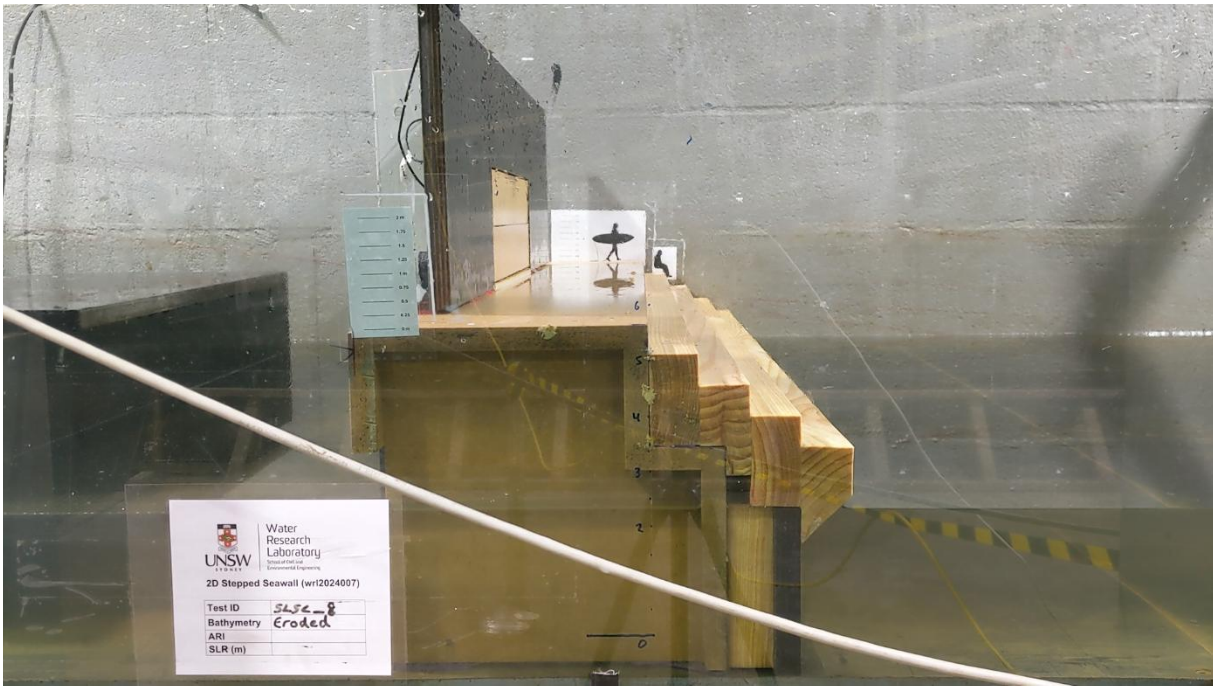


Figure 3.13 Design 4, 4 bleachers and SLSC wall (eroded profile), no parapet

Figure 3.14 Design 4, 4 bleachers and SLSC wall (eroded profile), recurved parapet

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 25 Hz (model scale).

For wave climate calibrations, three-probe arrays (3PA) were used to measure wave conditions offshore at -23.0 m AHD and the structure toe at -1 m AHD. These arrays enabled separation of the incident and reflected wave time series using the least-squares method of Mansard and Funke (1980). During model testing 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 3.3.

Table 3.3 Wave measurement locations

Wave probe name	Bed elevation (m AHD)	Test type
Offshore 3PA	-23.0	All tests
Structure 3PA	-1.0	All tests
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 third 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 25 m long section of the crest was collected using a catch tray placed on the leeside of the model structure. Overtopping water was collected 4.1 m landward of the seaward edge of the promenade (seawall crest), which approximates the location of the most seaward face of the existing SLSC building. The overtopping water was channelled to the catch tray through a folded sheet steel channel (Figure 3.15).

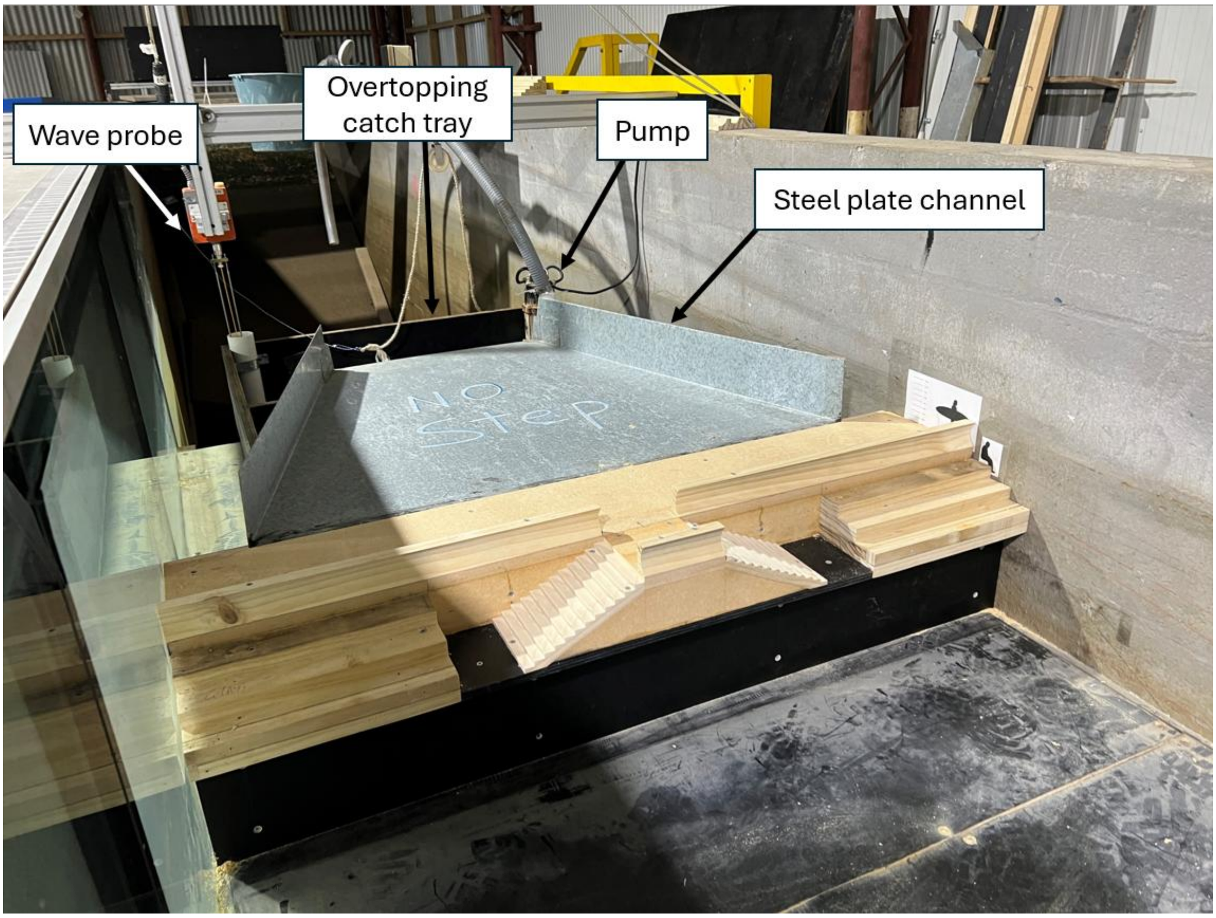


Figure 3.15 Arrangement for measurement of overtopping

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 (25 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 is shown in Figure 3.16.

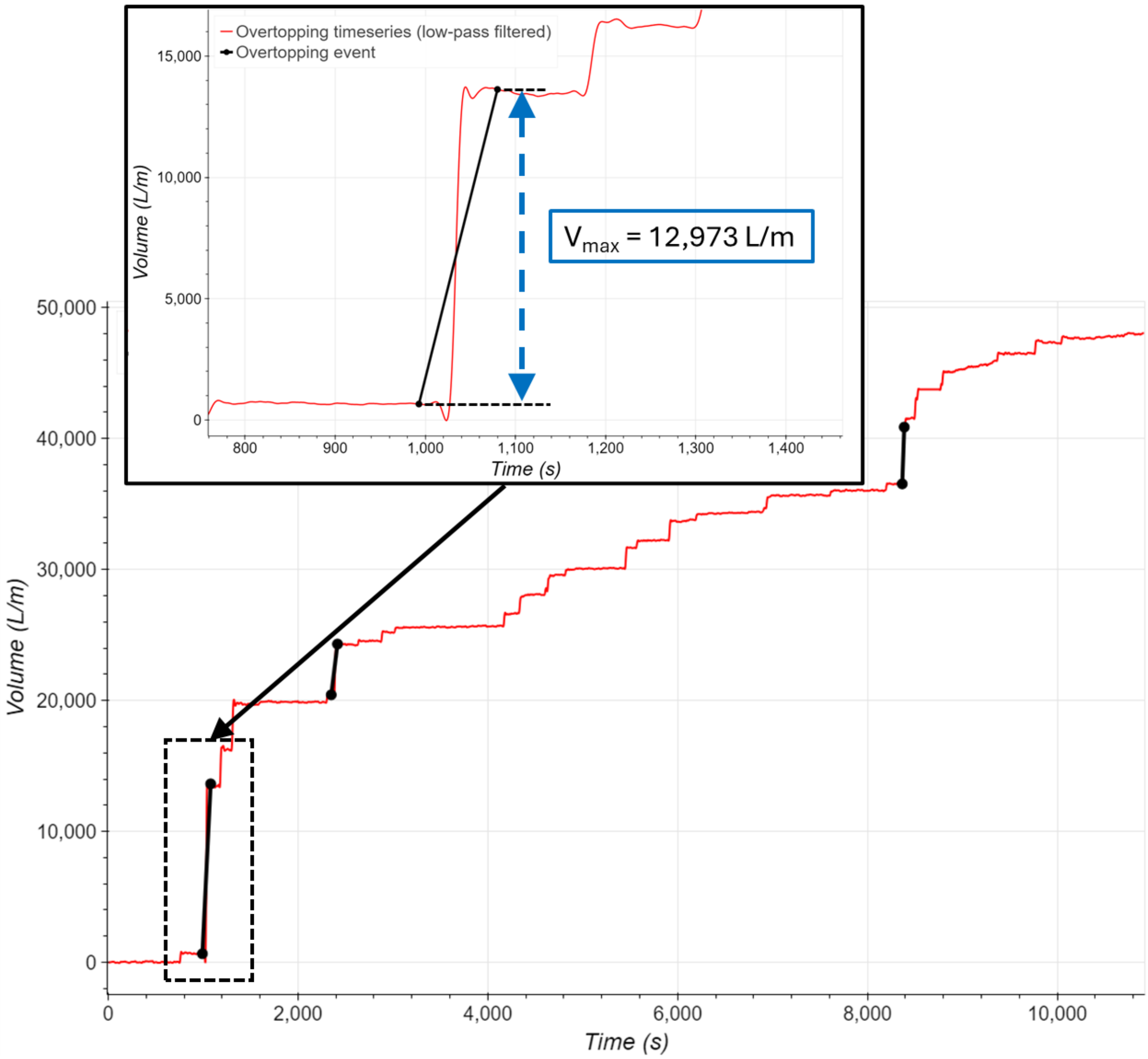


Figure 3.16 Cumulative overtopping timeseries for (V_{max})

The measured overtopping rates do not allow for the effects of wind due to the complexities that this would introduce into the model, noting that 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 two distinct 1.5 m high sections of the SLSC building wall (from the ground floor level to 1.5 m above, and from 1.5 m above the ground floor level to 3 m above) as per Figure 3.17. The wave deflector and the SLSC building wall load test sections were both 10 m wide and were tested separately. 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. This allowed drainage pathways either side of it, as indicated in Figure 3.18.

3D load cells were mounted to the leeward side of the wave deflector and the 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. Force measurements were collected with a sampling rate of 200 Hz (1,000 Hz in the model).

Figure 3.17 shows the flume arrangement for load testing. Photos of the wave loading testing setup are provided in Figure 3.18 and Figure 3.19.

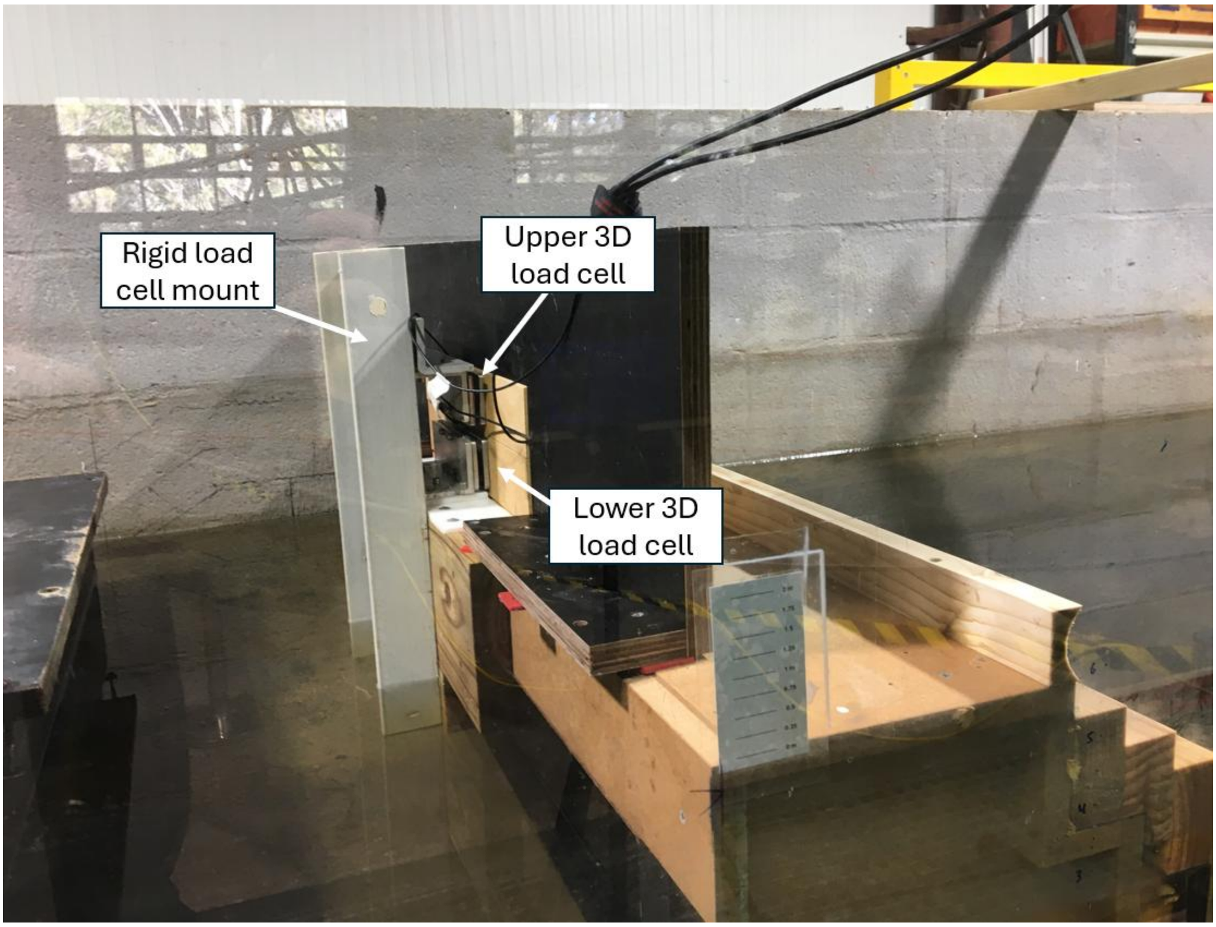


Figure 3.17 Load testing arrangement for the SLSC building

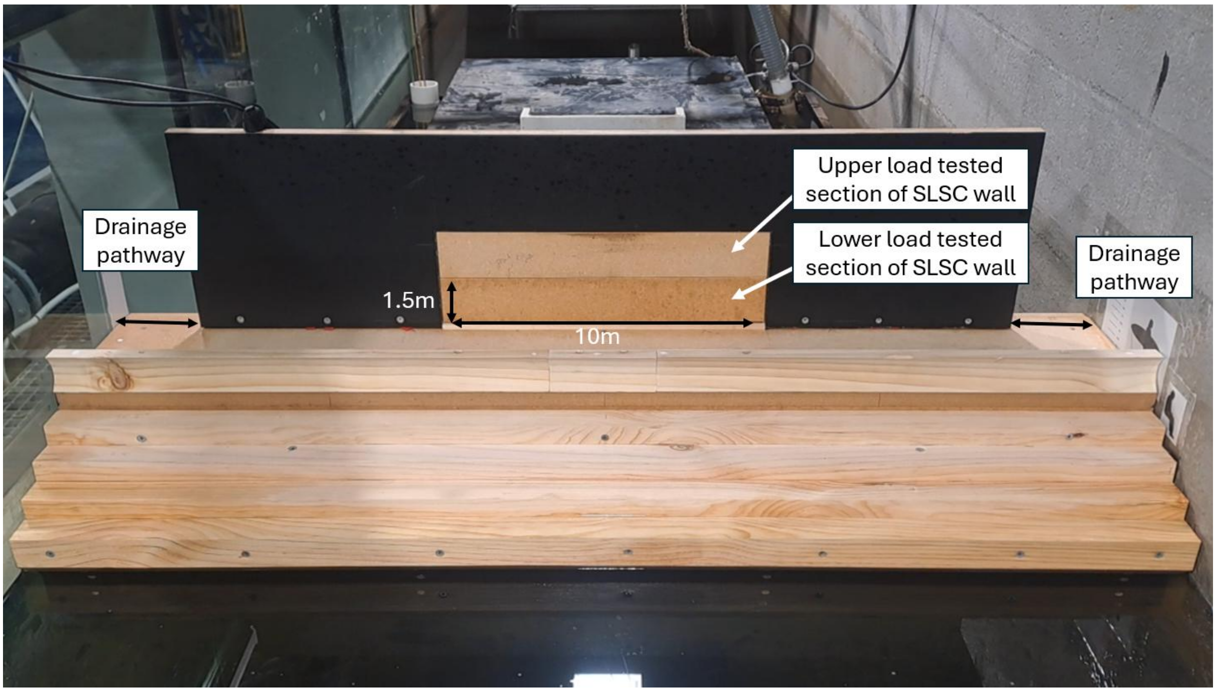


Figure 3.18 Landward view of flume arrangement for wave load testing

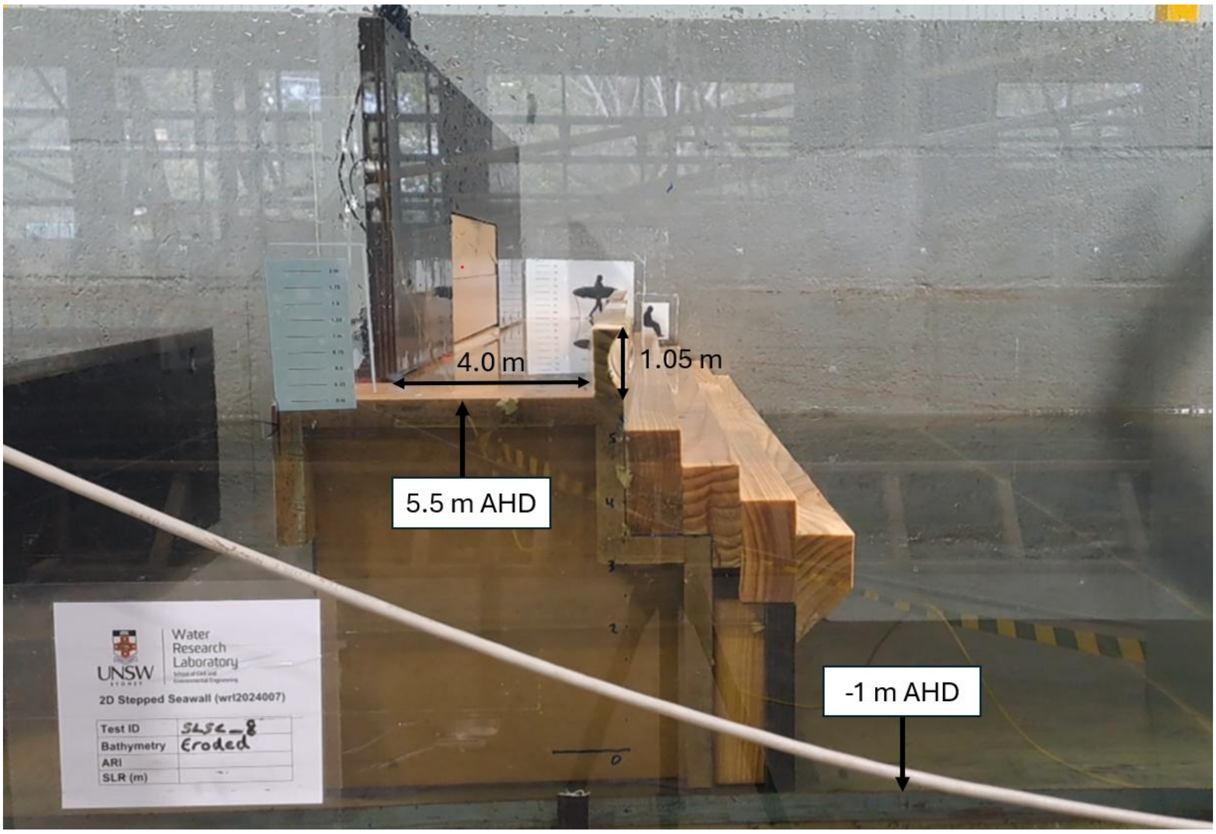


Figure 3.19 Side view of flume arrangement for wave load testing

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”.

3.4.4 Media and data files sharing

Recorded data, including overtopping timeseries, wave load time series and videos for all conducted flume tests will be provided to structural designers and the client.

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

Further details on the wave climate are provided in Appendix A. Eight target wave climate and water level (WL) conditions were calibrated for two different planning periods (present day and 2084). Initial wave climate data was for the -10 m AHD contour, 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_s) 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_s at MHL Sydney wave buoy (m)*
1	1 (waves) + MHWS(3) SWL	2024	0.67	4.4	11.0
2	10	2024	1.34	6.2	12.1
3	100	2024	1.46	7.8	13.0
4	1,000(4)	2024	1.58	9.6	13.8
5	1 (waves) + MWHS(3) + SLR	2084	1.20	4.4	11.0
6	10	2084	1.87	6.2	12.1
7	100	2084	1.99	7.8	13.0
8	1,000(4)	2084	2.11	9.6	13.8

(1) The design water levels do not include wave setup and wave runup and these will be inherently generated within the wave flume through wave processes.

(2) Design offshore wave conditions are provided based on the deep water wave buoy analysis. Design offshore wave heights in the physical modelling will be limited by the maximum achievable wave height by the flume wave maker and will require adjustment to the test still water level to account for the reduced wave setup generated.

(3) The 1 year ARI wave condition was conducted with a MHWS tide water level.

(4) The 1,000 year ARI values were extrapolated by WRL using a log-linear fit.

At the scale of 1:25, the largest offshore H_s which can be produced in the physical model at the wave maker is 6.2 m. As this maximum achievable offshore H_s condition is less than the target offshore design conditions for the proposed 100 year ARI and 1,000 year ARI, 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) are depth limited and, as such, the wave height at the seawall will be strongly dependent on the total water depth including wave setup.

Numerical modelling was first undertaken, before the start of the physical modelling, to determine the extent to which the test still water levels for the 100 year ARI and 1,000 year ARI should be raised above the respective design still water levels to account for the reduced wave setup in the flume.

The design offshore conditions presented in Table 4.1 were applied as a boundary to the Dally, Dean and Dalrymple (1984) 2D surf zone model (SBEACH) for the full bathymetry profile at Newport (see Section 3) and wave setup at the -1 m AHD contour was derived.

The SBEACH model was then re-run with the reduced maximum achievable wave height condition that can be run in the wave flume to obtain the reduced wave setup at the -1 m AHD contour. The calculated difference in wave setup was used to inform the amount by which the test still water levels need to be raised above the respective design still water levels during all tests in the flume.

All wave climates were generated using JONSWAP spectra (Hasselmann et. al., 1973), with a peak enhancement factor of $\gamma = 3.3$. The design peak spectral wave periods (T_P) and the deepwater significant wave height ($H_{1/3}$) are shown in Table 4.1.

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).

4.2 Results

During the wave calibration, waves were measured using two different three-probe arrays referred to as the Offshore 3PA (-23 m AHD) and Structure 3PA (-1 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 all 1 year and 10 year ARI conditions, the target H_s at -23 m AHD was within 0.1 m and all offshore T_P values were within 0.4 s of the target.

For the 1 year and 10 year ARI tests, the average wave setup measured at -1 m AHD for the present day and 2084 was approximately 5.1% of the deepwater significant wave height. This ratio was used to determine the water level adjustment and the target TWL at the -1 m AHD contour for the 100 and 1,000 year ARI events. For example, the water level adjustment and target TWL for Design Condition 3 *(100 year ARI and present day planning period),* were calculated using Equations 4.3 and 4.4. The maximum achievable wave height for Design Condition 3 in the flume is 5.65 m. The design condition H_s is 7.8 m, and the still water level 1.46m.

$$Water\ level\ adjustment = 5.1\% \times (7.8\ m - 5.65\ m) = 0.11\ m \tag{4.3}$$

$$Target\ TWL = 1.46\ m\ AHD + (5.1\% \times 7.8\ m) = 1.86\ m \tag{4.4}$$

Following this approach, WRL matched the TWL at -1 m AHD to within 0.05 m of the target TWL for the 100 and 1,000 year ARI conditions. Wave climate statistics at the offshore and structure locations are presented in Table 4.2.

Table 4.2 Measured wave climate conditions and total water level

						Offshore 3PA		Structure 3PA			
Design cond. #	**ARI (years)**	**Planning period**	**Design SWL, excl. wave setup (m AHD)**	**Hs at wave buoy (m)***	**Tp (s)#**	**Hs (m)**	**Tp (s)**	**H_s (m)**	**T_Z** (s)**	**TWL Target (m AHD)**	**TWL Measured (m AHD)**
1	1	2024	0.67	4.4	11.0	4.4	10.6	1.0	7.5	0.89	0.90
2	10	2024	1.34	6.2	12.1	6.3	11.9	1.4	7.6	1.66	1.69
3	100	2024	1.46	7.8	13.0	<u>5.7</u>	13.1	1.6	8.0	1.86	1.88
4	1,000	2024	1.58	9.6	13.8	<u>6.1</u>	13.8	1.7	8.5	2.07	2.12
5	1	2084	1.20	4.4	11.0	4.3	10.6	1.2	6.7	1.42	1.41
6	10	2084	1.87	6.2	12.1	6.2	12.2	1.7	7.5	2.19	2.19
7	100	2084	1.99	7.8	13.0	<u>5.6</u>	12.9	1.8	7.2	2.39	2.42
8	1,000	2084	2.11	9.6	13.8	<u>6.1</u>	13.8	1.9	7.1	2.60	2.61

* T_P 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_P at the Structure (-1 m AHD) as a significant portion of broken waves resulted in long wave generation.

5 Overtopping results

5.1 Preamble

A total of 94 tests were performed on combinations of three seawall/stair geometries, two nearshore profiles, two planning periods and four wave conditions. Overtopping volumes were measured along a 25 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.

5.2 Guidelines for overtopping

Published guidelines for overtopping from EurOtop (2018) are shown in Figure 5.1. With significant waves at the seawall of approximately 2 m under large events, a range of 1 to 10 to 20 L/s/m could be considered tolerable for people.

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea.		
H_{m0} = 3 m	0.3	600
H_{m0} = 2 m	1	600
H_{m0} = 1 m	10-20	600
H_{m0} < 0.5 m	No limit	No limit
Cars on seawall / dike crest, or railway close behind crest		
H_{m0} = 3 m	<5	2000
H_{m0} = 2 m	10-20	2000
H_{m0} = 1 m	<75	2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Figure 5.1 EurOtop (2018) guidelines for tolerable overtopping

5.3 Overtopping

A summary of the mean overtopping test results is provided in Table 5.1, Table 5.2 and Table 5.3, noting that the configuration for Design 3 (Quasi 3D stairs) is not truly 2 dimensional, but the overtopping has been collected for a 25 m alongshore length of this compound structure.

The progress of overtopping for selected events is shown in Figure 5.2 and Figure 5.3. For each test, individual overtopping volumes were reported as the largest overtopping event (V_{max}), noting that EurOtop's guideline limit for pedestrian access is 600 L/m (EurOtop, 2018). Other overtopping parameters are provided in the digital files associated with this report. Examples include the average of the five largest overtopping events (V_{avg5}), the average of the ten largest overtopping events (V_{avg10}) and the ratio of the largest overtopping event to the mean overtopping volumes (V_{max}/q).

Image sequences V_{max} events are provided in Figure 5.4 to Figure 5.7.

Table 5.1 Mean overtopping volumes – Design 1, original DA

Test #	Structure	Wave return parapet	Parapet position	Beach state	Duration	Planning period	Design ARI (years) *	Mean q (L/s/m)	V_{max} (L/m)
SLSC_0025	Stairs	NIL		Eroded	Full	2024	100	6.6	4,441
SLSC_0026	Stairs	NIL		Average	Full	2024	100	6.7	10,280
SLSC_0027	Stairs	NIL		Average	Full	2024	10	5.0	8,771
SLSC_0028	Recurve flush	Recurve 800 mm	Seaward	Average	Full	2024	10	0.6	2,025
SLSC_0029	Square set back	Straight 800 mm	Setback 1 m	Average	Full	2024	10	1.4	5,520
SLSC_0030	Square set back	Straight 800 mm	Setback 1 m	Average	Full	2024	100	2.0	4,651
SLSC_0031	Square set back	Straight 800 mm	Setback 1 m	Average	Full	2084	100	4.9	9,841
SLSC_0032	Stairs	NIL		Average	Full	2084	100	15.7	14,223
SLSC_0033	Stairs	NIL		Average	Full	2084	10	11.4	9,900
SLSC_0034	Stairs	NIL		Average	Full	2024	1	0.0	Low
SLSC_0035	Stairs	NIL		Average	Full	2084	1	0.0	436
SLSC_0036	Stairs	NIL		Average	Full	2024	1,000	11.5	11,816
SLSC_0037	Stairs	NIL		Average	Full	2084	1,000	41.6	29,867
SLSC_0038	Recurve set back	Recurve 800 mm	Seaward	Average	Subset	2084	100	6.3	9,524
SLSC_0039	Square set back	Straight 800 mm	Landward	Average	Subset	2084	100	6.8	9,693
SLSC_0042	Square set back	Straight 800 mm	Setback 1 m	Average	Subset	2024	100	3.4	4,363
SLSC_0043	Recurve set back	Recurve 800 mm	Setback 1 m	Average	Subset	2024	100	3.1	3,969
SLSC_0044	Stairs	NIL		Average	Subset	2024	100	11.3	8,077
SLSC_0045	Stairs	NIL		Average	Subset	2084	100	19.3	11,547
SLSC_0046	Stairs	NIL		Eroded	Full	2084	100	27.7	8,611
SLSC_0047	Stairs	NIL		Eroded	Subset	2084	100	28.5	7,129
SLSC_0048	Recurve set back	Recurve 800 mm	Setback 1 m	Eroded	Subset	2084	100	11.0	3,364
SLSC_0049	Square set back	Recurve 800 mm	Setback 1 m	Eroded	Subset	2084	100	12.0	4,062
SLSC_0050	Square set back	Recurve 800 mm	Setback 1 m	Eroded	Subset	2024	100	3.4	1,212
SLSC_0051	Recurve set back	Recurve 800 mm	Setback 1 m	Eroded	Subset	2024	100	2.5	1,092
SLSC_0052	Stairs	Recurve 800 mm	Setback 1 m	Eroded	Subset	2024	100	10.0	3,733
SLSC_0053	Stairs	NIL		Eroded	Full	2084	100	27.6	9,707

Note: Low V_{max} was below the detection limit for instantaneous overtopping

Table 5.2 Mean overtopping volumes – Design 2, BCA stairs and bleachers

Test #	Structure	Wave return parapet	Parapet position	Beach state	Duration	Planning period	Design ARI (years) *	Mean q (L/s/m)	V_{max} (L/m)
SLSC_0054	Stairs	NIL		Eroded	Full	2024	100	11.2	5,217
SLSC_0055	Stairs	NIL		Eroded	Full	2084	100	44.3	9,993
SLSC_0056	Stairs	NIL		Eroded	Full	2024	100	10.2	6,238
SLSC_0057	Stairs	NIL		Eroded	Full	2084	100	37.8	9,742
SLSC_0058	Bleachers	NIL		Eroded	Full	2084	100	31.1	10,346
SLSC_0059	Bleachers	NIL		Eroded	Full	2024	100	6.6	3,662
SLSC_0060	Bleachers	NIL		Average	Full	2024	100	6.6	11,772
SLSC_0061	Stairs	NIL		Average	Full	2024	100	8.0	12,055
SLSC_0062	Stairs	NIL		Average	Full	2084	100	19.0	13,935
SLSC_0063	Bleachers	NIL		Average	Full	2084	100	16.5	12,798
SLSC_0064	Bleachers + lip	Recurve 1050 mm	Seaward	Average	Full	2024	100	1.1	3,110
SLSC_0065	Bleachers + lip	Recurve 1050 mm	Seaward	Average	Full	2084	100	2.6	7,291
SLSC_0066	Bleachers + back	Recurve 1050 mm	Landward	Average	Full	2084	100	2.1	6,647
SLSC_0067	Bleachers + back	Recurve 1050 mm	Landward	Average	Full	2024	100	0.9	2,952
SLSC_0068	Bleachers + back	Recurve 1050 mm	Landward	Eroded	Full	2084	100	2.9	3,750
SLSC_0069	Bleachers + lip	Recurve 1050 mm	Seaward	Eroded	Full	2084	100	4.3	3,840
SLSC_0070	Bleachers + lip	Recurve 1050 mm	Seaward	Eroded	Full	2024	100	0.8	Low
SLSC_0071	Bleachers + back	Recurve 1050 mm	Landward	Eroded	Full	2024	100	0.2	Low
SLSC_0072	Bleachers + Lip - angle	Splayed 1050 mm	Seaward	Eroded	Full	2084	100	3.5	Low
SLSC_0073	Bleachers + Lip - angle	Splayed 1050 mm	Seaward	Eroded	Full	2024	100	0.8	Low

Note: Low V_{max} was below the detection limit for instantaneous overtopping

Table 5.3 Mean overtopping volumes – Design 3, Quasi 3D stairs and bleachers

Test #	Structure	Wave return parapet	Parapet position	Beach state	Duration	Planning period	Design ARI (years) *	Mean q (L/s/m)	V_{max} (L/m)
SLSC_0074	Quasi-3D	Recurve 1050 mm	Seaward	Eroded	Full	2024	100	1.0	765
SLSC_0075	Quasi-3D	Recurve 1050 mm	Seaward	Eroded	Full	2084	100	5.5	4,962
SLSC_0076	(Repeat test)								
SLSC_0077	Quasi-3D	Recurve 1050 mm	Seaward	Eroded	Full	2024	1	0.0	Low
SLSC_0078	Quasi-3D	Recurve 1050 mm	Seaward	Eroded	Full	2084	1	0.1	62
SLSC_0079	Quasi-3D	Recurve 1050 mm	Seaward	Eroded	Full	2024	10	0.4	351
SLSC_0080	Quasi-3D	Recurve 1050 mm	Seaward	Eroded	Full	2024	1,000	3.6	3,626
SLSC_0081	Quasi-3D	Recurve 1050 mm	Seaward	Eroded	Full	2084	10	3.1	1,941
SLSC_0082	Quasi-3D	Recurve 1050 mm	Seaward	Eroded	Full	2084	1,000	15.8	11,863
SLSC_0083	(Repeat test)								
SLSC_0084	(Repeat test)								

Note: Low V_{max} was below the detection limit for instantaneous overtopping.

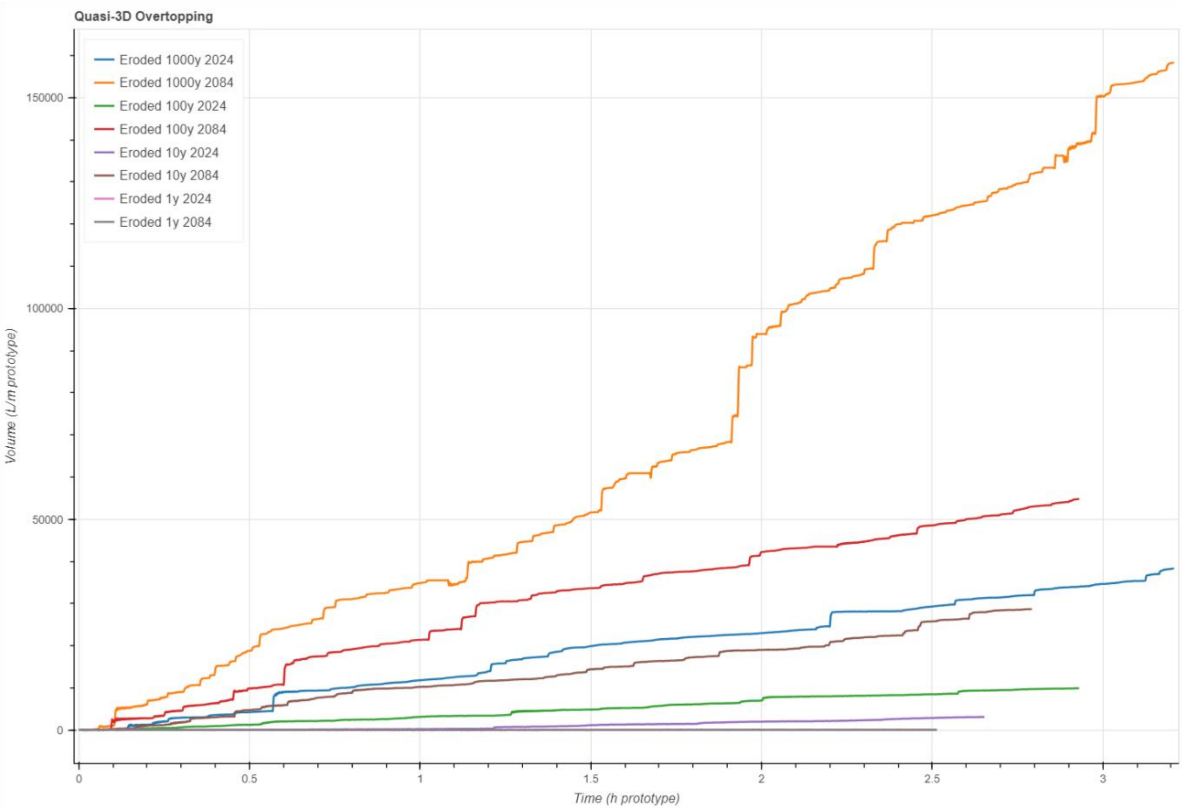
The following insights were derived from comparing mean overtopping rates between the different seawall structures, nearshore profiles and wave climates:

- Mean overtopping rates with the parapet wall were 75-90% less than for without it.
- Mean overtopping rates for the fully eroded profile and average profile were similar.

5.4 Discussion

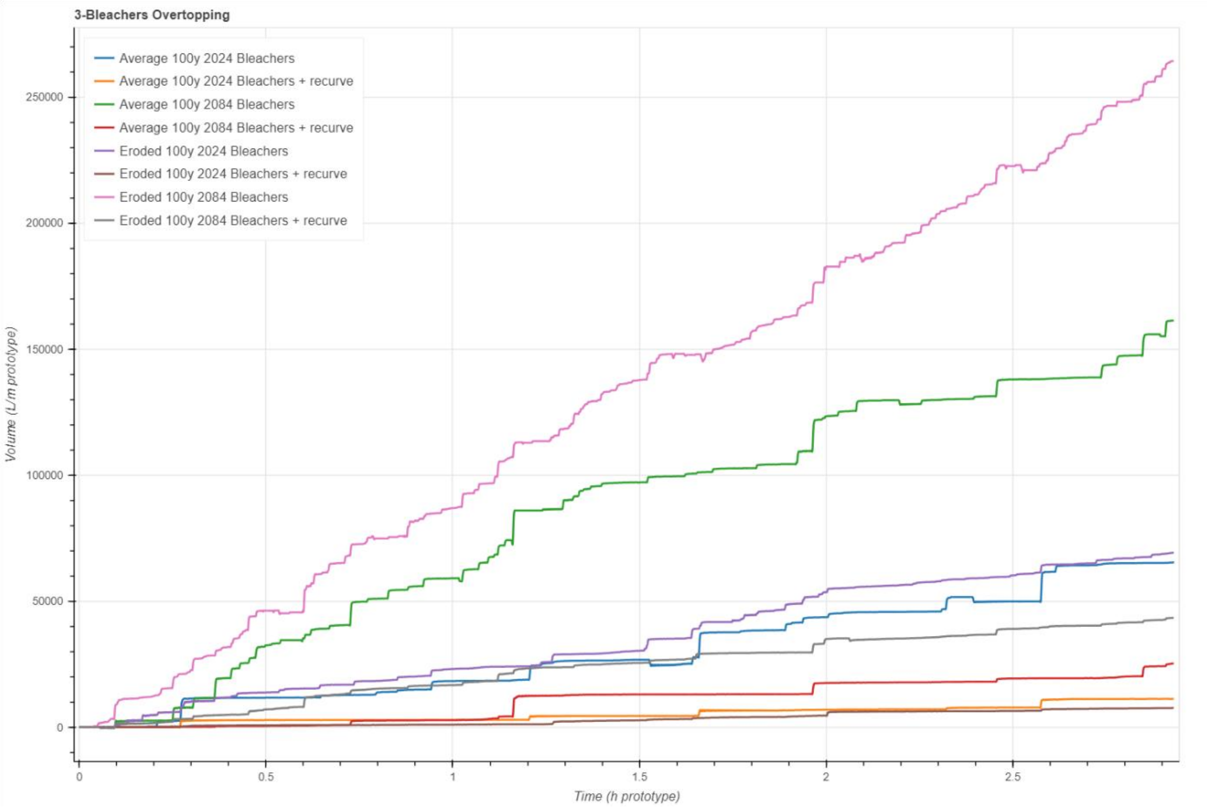
The test results demonstrated that the mean overtopping rate and maximum individual overtopping volumes were similar with the average and the eroded profiles. While the 1V:10H ramp was fixed in the physical model, it represented an average sandy beach profile in the real-world, and would be subject to 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 tests were conducted with extreme 100 and 1,000 year ARI storm events, WRL undertook the subsequent wave loading tests using the eroded profile.

While the measured V_{max}/q ratios 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, which caused and/or coincided with substantial dynamic wave setup at the structure toe, then superposition of multiple waves at the point of overtopping (informally referred to as "doubling up").



Note: Minor decreases in timeseries were due to the pumping rate out of the tray temporarily exceeding overtopping.

Figure 5.2 Overtopping timeseries for quasi 3D seawall with parapet



Note: Minor decreases in timeseries were due to the pumping rate out of the tray temporarily exceeding overtopping.

Figure 5.3 Overtopping timeseries for 3 bleacher seawall with and without parapet

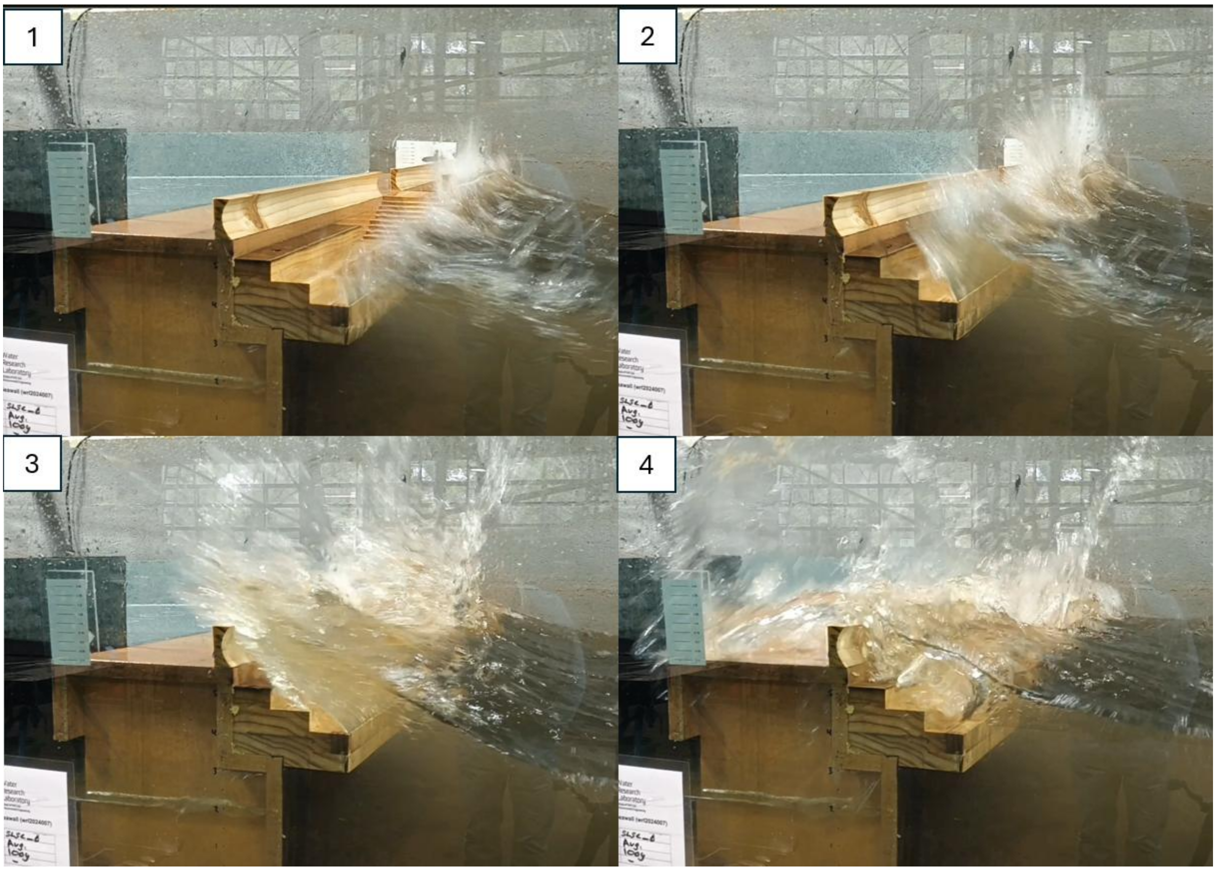


Figure 5.4 V_{max} event for quasi 3D stairs100 year ARI 2024

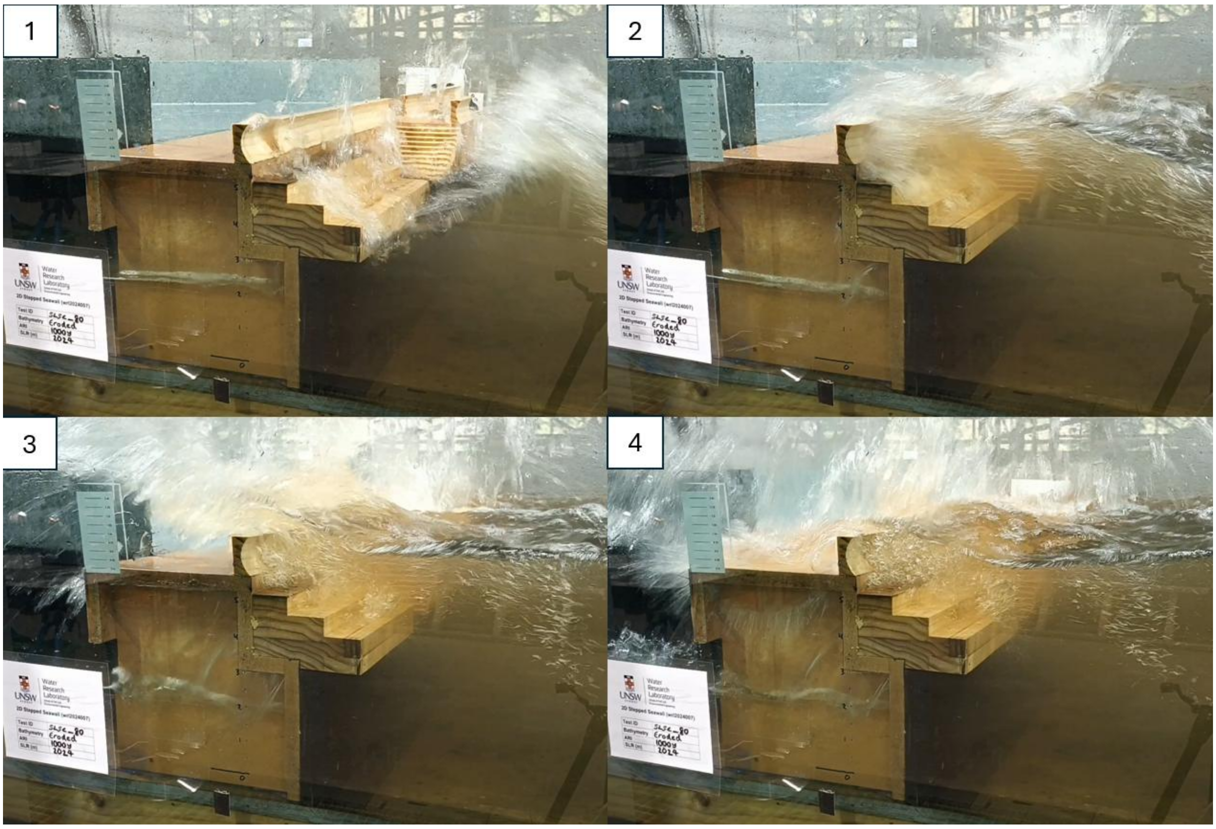


Figure 5.5 V_{max} event for quasi 3D stairs1,000 year ARI 2024

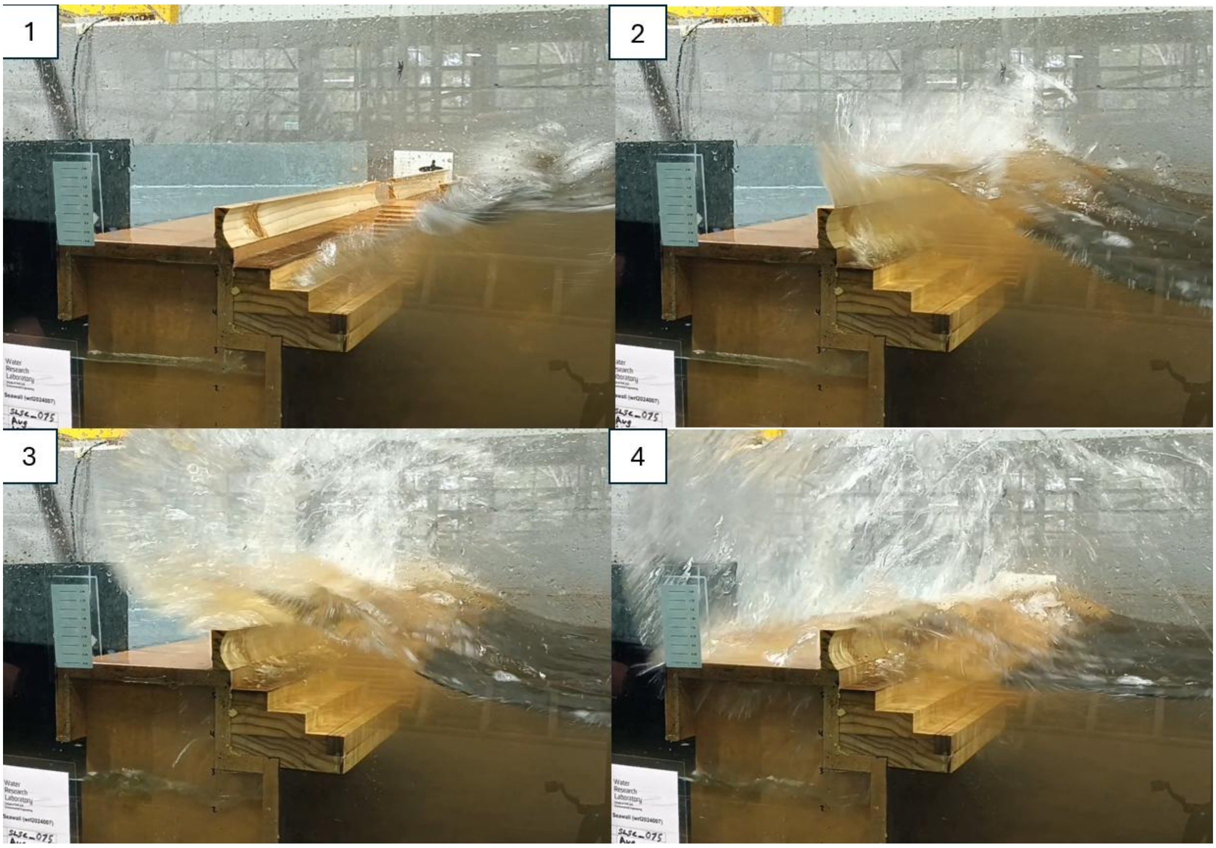


Figure 5.6 V_{max} event for quasi 3D stairs100 year ARI 2084

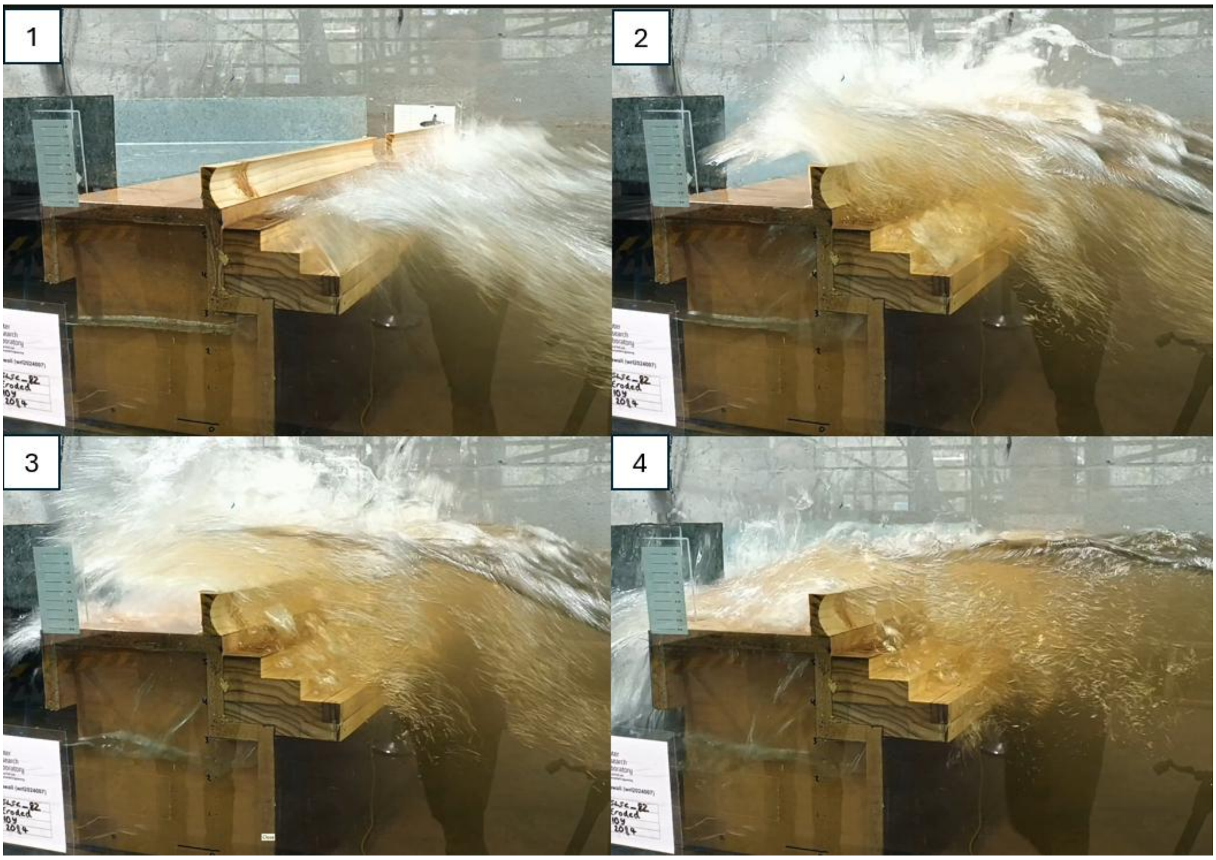


Figure 5.7 V_{max} event for quasi 3D stairs1,000 year ARI 2084

6 Wave forces/pressures

6.1 Overview

Load tests were undertaken for 1,000 waves on 10 m wide sections of the SLSC building wall for 100 and 1,000 year ARI conditions, with and without the parapet wall, with an eroded beach state. All test data was/will be provided to the client and/or their designers for their interpretation. The maximum pressures (P_{max}) reported herein are of very short duration (discussion below), with lower pressures prevailing over longer durations. Interpretation of the pressure data and reductions for longer duration on the differing structural types in the altered SLSC building are the responsibility of a structural engineer.

A summary of load test results including the largest pressure measured (P_{max}) on two 1.5 m high sections of the SLSC building wall is provided in Table 6.1 and Table 6.2.

The following discussion is provided regarding the geometry and testing:

- The promenade level is 5.5 m AHD and width is 4 m
- Ground floor level is 5.66 m AHD = approx. 160 mm above promenade
- First floor level is 9.0 m AHD
- Eaves level is approximately 12.1 m AHD
- The lower load cell covered the vertical range: 5.66 m AHD to 7.16 m AHD (1.5 m), for 10 m width
- The upper load cell covered the vertical range: 7.16 m AHD to 8.66 m AHD (1.5 m), for 10 m width
- The SLSC building was represented as a fixed wall (4.0 m promenade width) up to the eaves (12.1 m AHD), leaving a 2.5 m (100 mm model scale) gap at the flume sides for drainage
- A 1.05 m high recurved parapet is proposed to front the retained portion of the SLSC building, whereas no parapet is proposed for the new portion, as it is likely that a new wall can be designed to withstand the prevailing forces

The horizontal pressure time series for the SLSC building wall are provided in Figure 6.1 to Figure 6.12. The logging frequency for this data was 200 Hz prototype. The largest wave forces on the SLSC building wall had a typical total duration (rise and fall) of 0.1 to 1 s, with the actual peak lasting for as little as 0.005 s. The P_{max} values on the upper portion were generally similar to (but usually smaller – except for 2024 100 year ARI) P_{max} values registered on the lower portion. However, there were significantly less wave pressure events reaching the upper wall. The wave return parapet almost always reduced the wave pressures on the lower wall, while they were mostly similar on the upper wall with or without the wave return parapet.

Table 6.1 P_{max} on SLSC wall – Design 4, wave return parapet (existing SLSC)

Design ARI (years) *	Planning period	Lower P_{max} (kPa)	Upper P_{max} (kPa)
100	2024	7	12
100	2084	13	11
1,000	2024	26	25
1,000	2084	54	21

Table 6.2 P_{max} on SLSC wall – Design 4, no wave return parapet (new SLSC)

Design ARI (years) *	Planning period	Lower P_{max} (kPa)	Upper P_{max} (kPa)
100	2024	24	15
100	2084	54	12
1,000	2024	33	8
1,000	2084	63	25

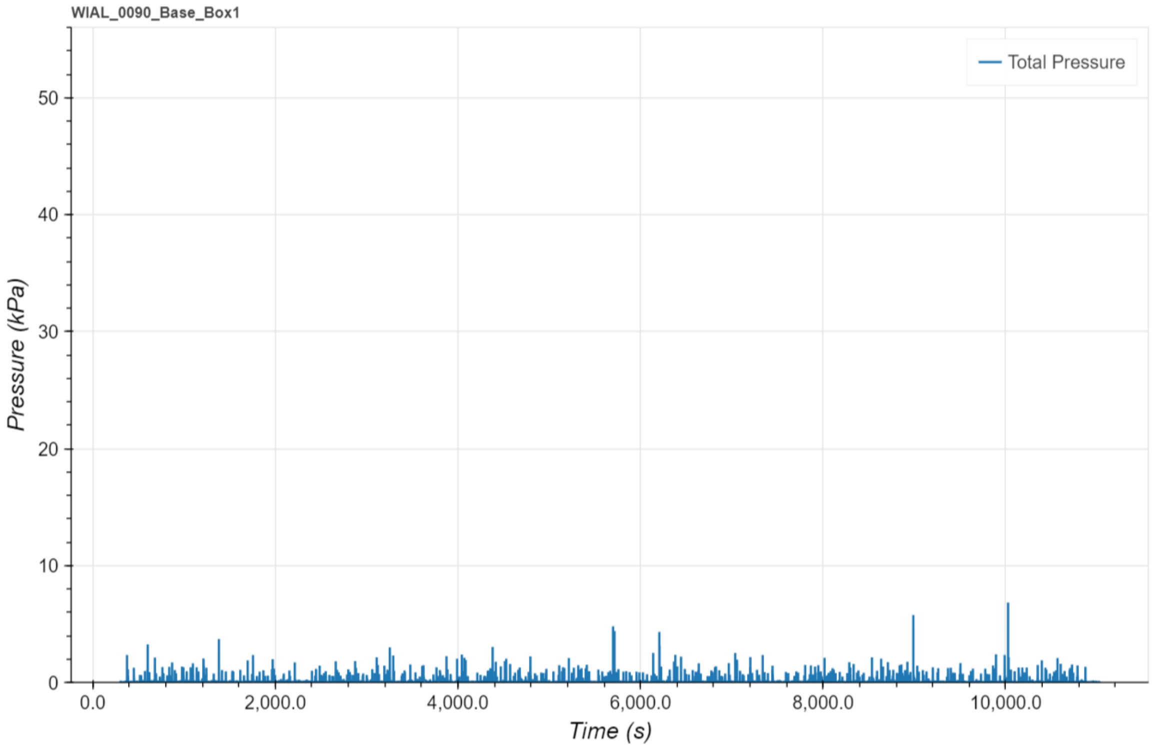


Figure 6.1 Pressure lower SLSC wall (1.5 m height) 100 year ARI 2024 with parapet

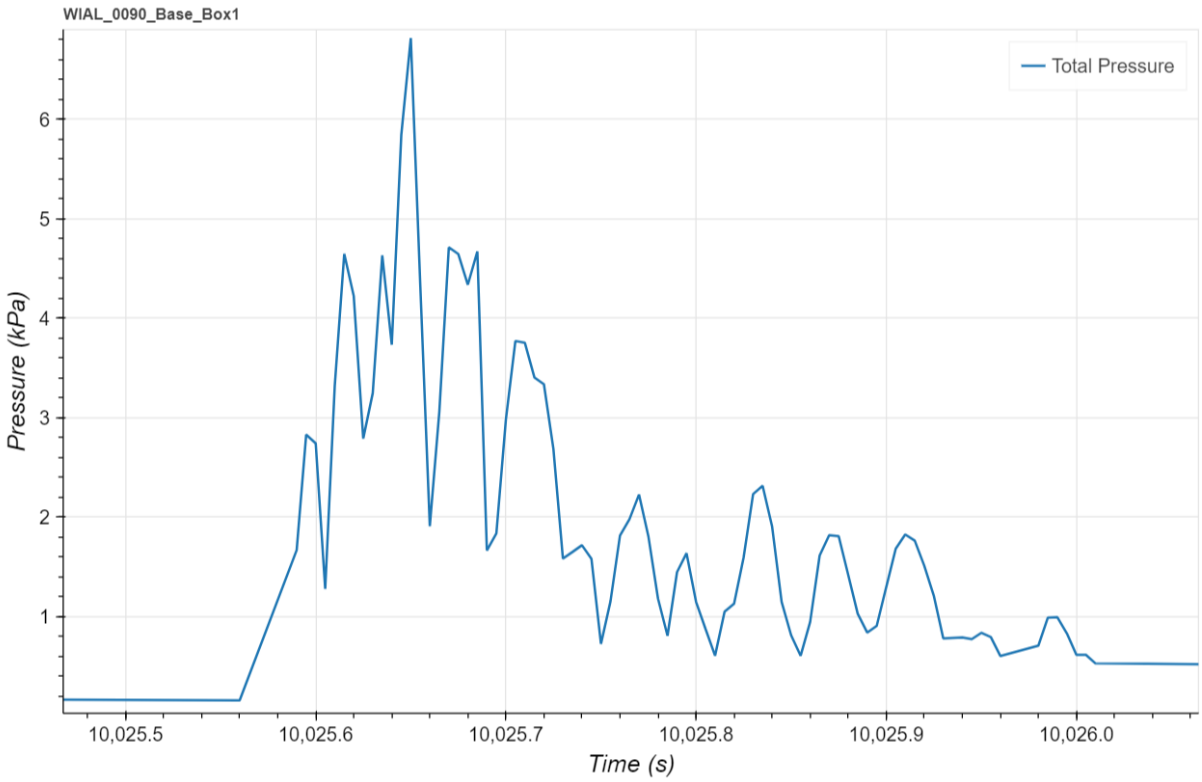


Figure 6.2 P_{max} lower SLSC wall (1.5 m height) 100 year ARI 2024 with parapet

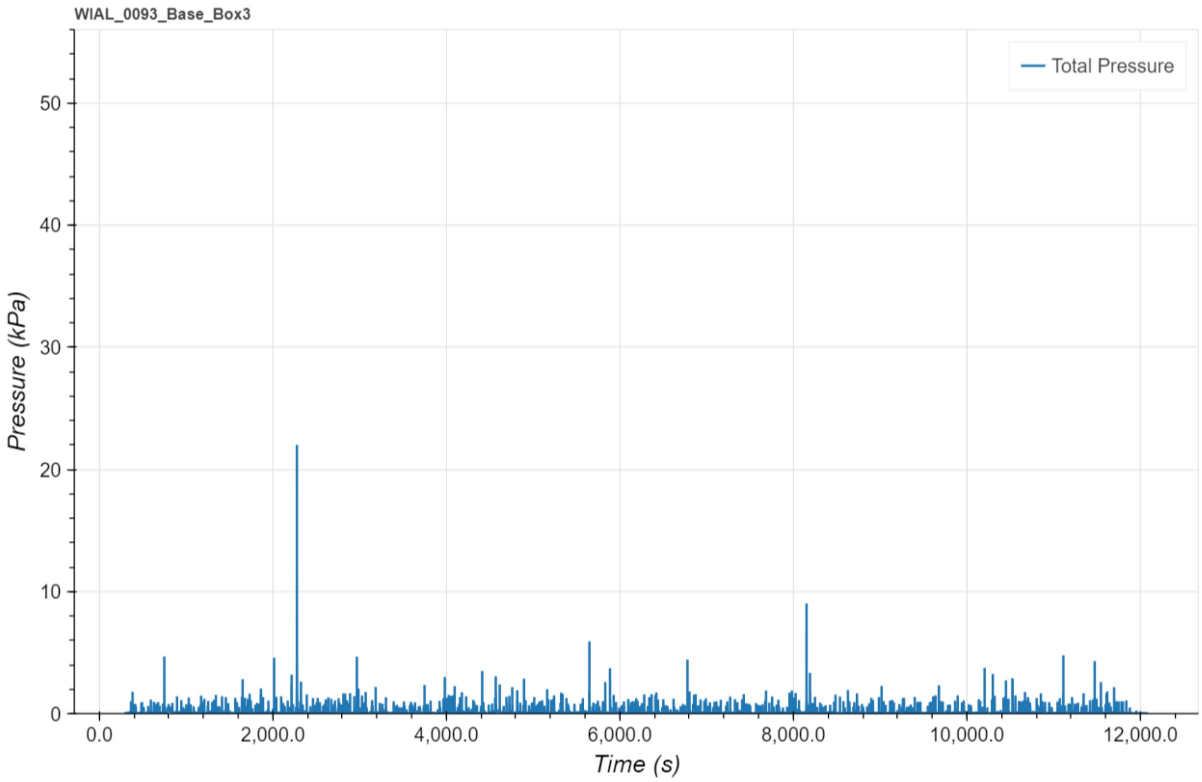


Figure 6.3 Pressure lower SLSC wall (1.5 m height) 1,000 year ARI 2024 with parapet

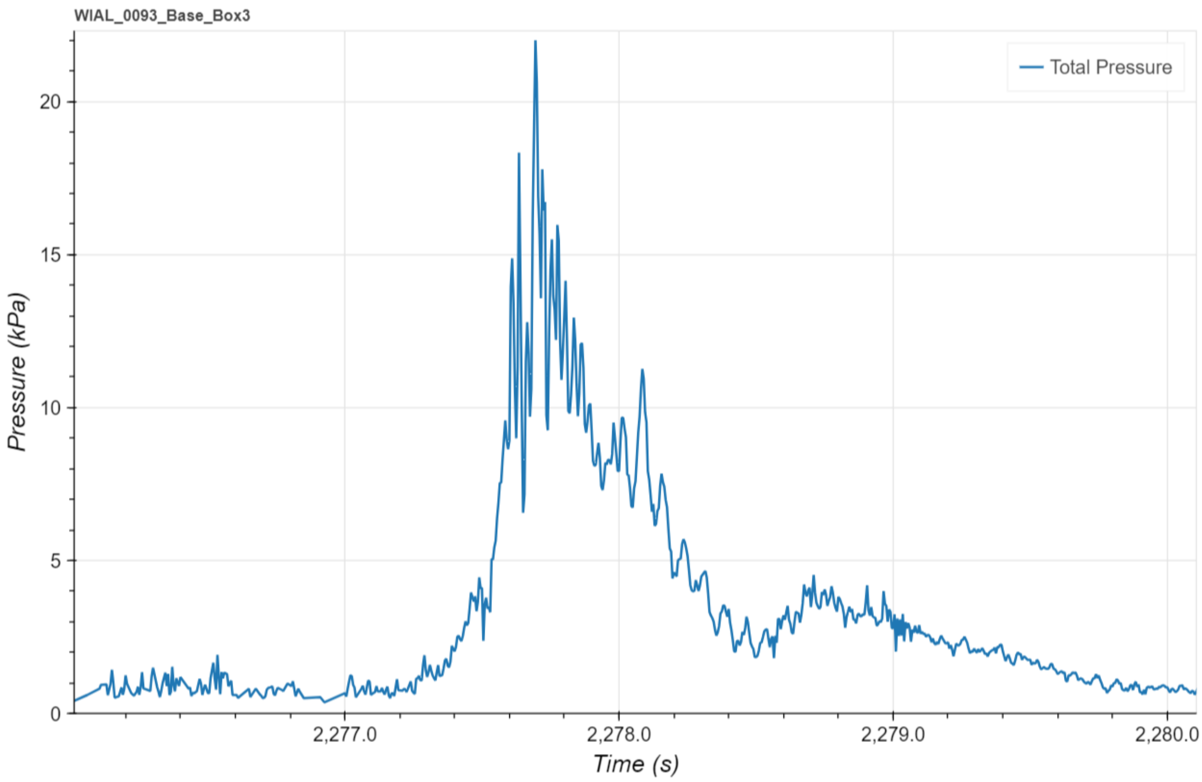


Figure 6.4 P_{max} lower SLSC wall (1.5 m height) 1,000 year ARI 2024 with parapet

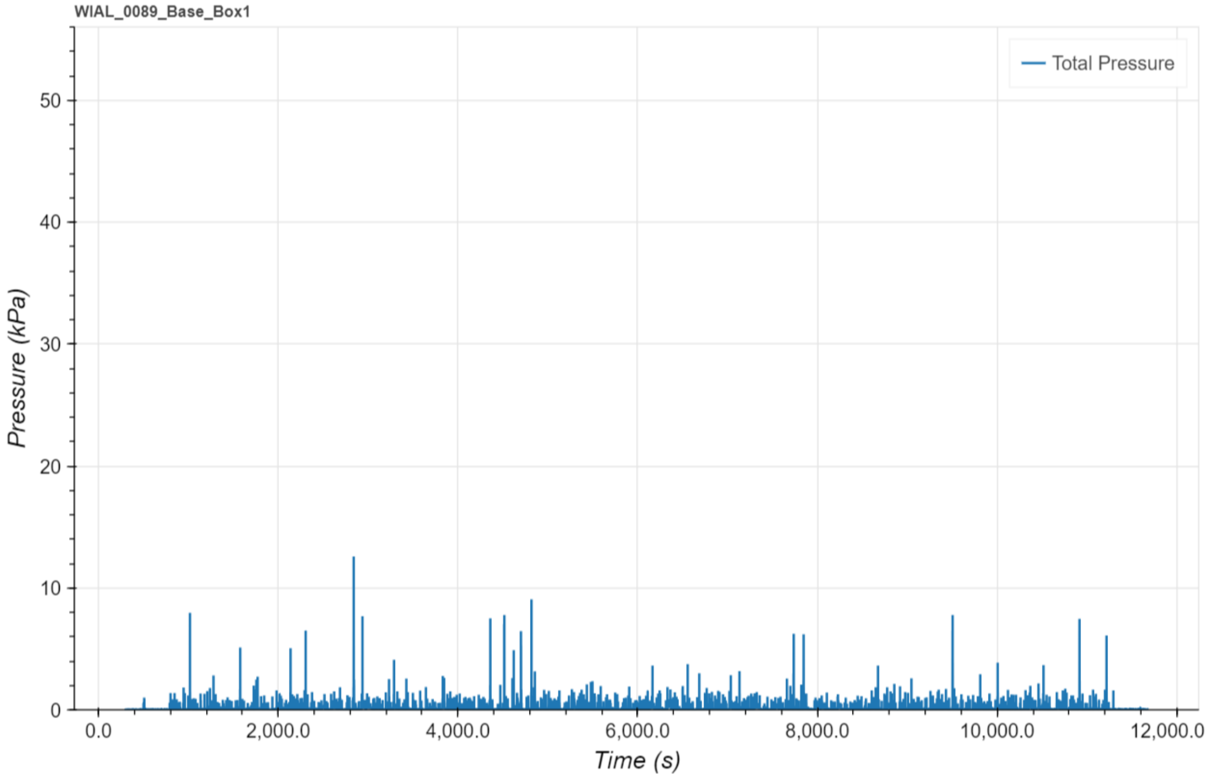


Figure 6.5 Pressure lower SLSC wall (1.5 m height) 100 year ARI 2084 with parapet

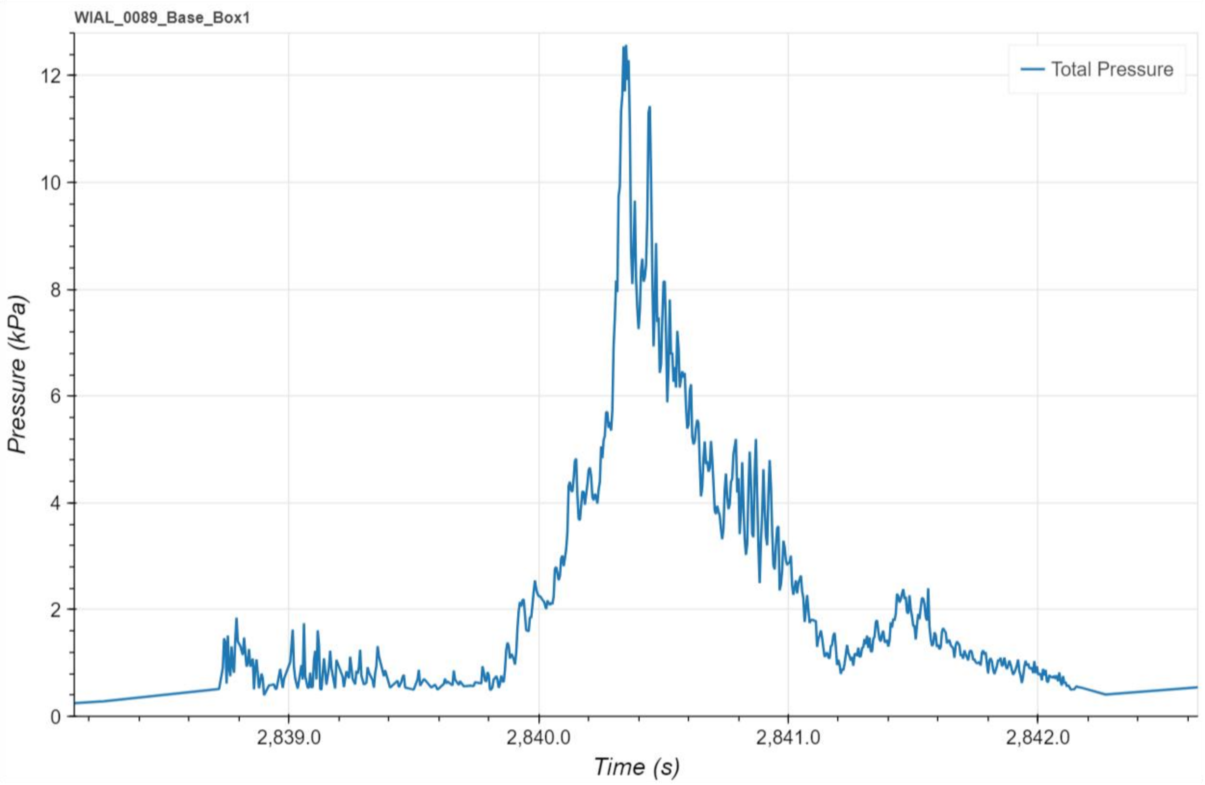


Figure 6.6 P_{max} lower SLSC wall (1.5 m height) 100 year ARI 2084 with parapet

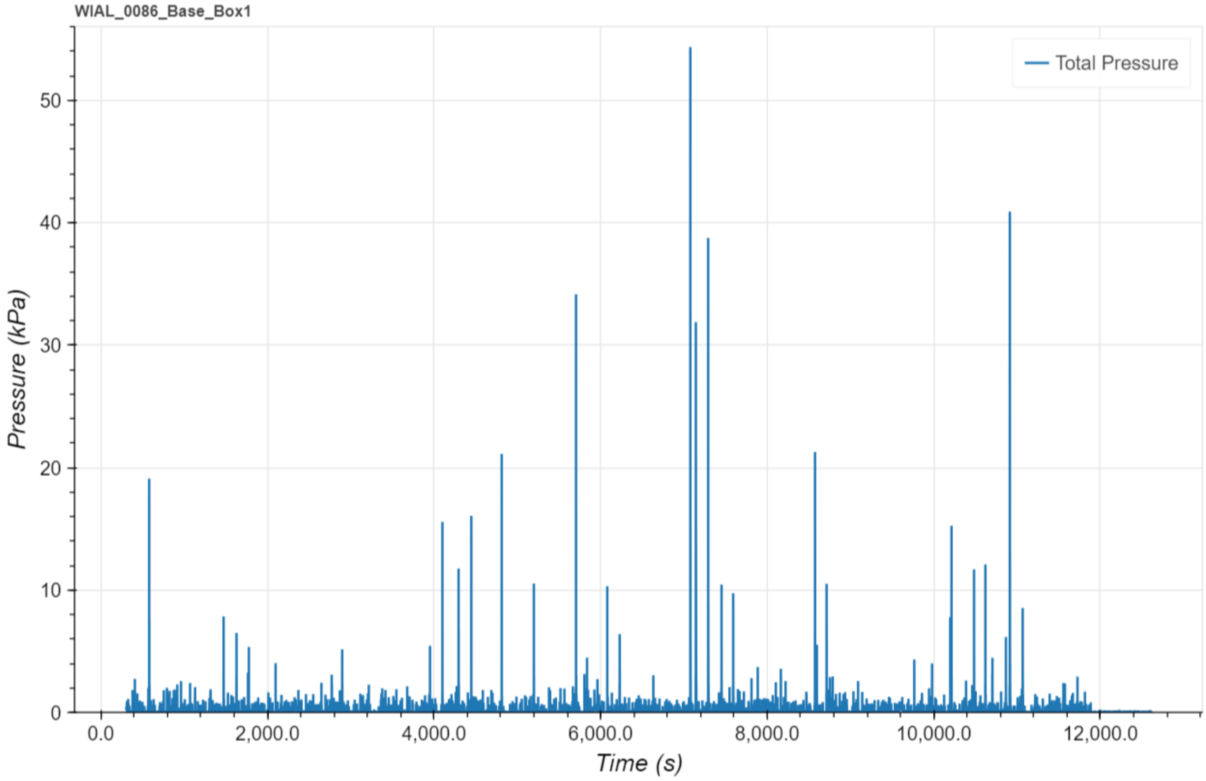


Figure 6.7 Pressure lower SLSC wall (1.5 m height) 1,000 year ARI 2084 with parapet

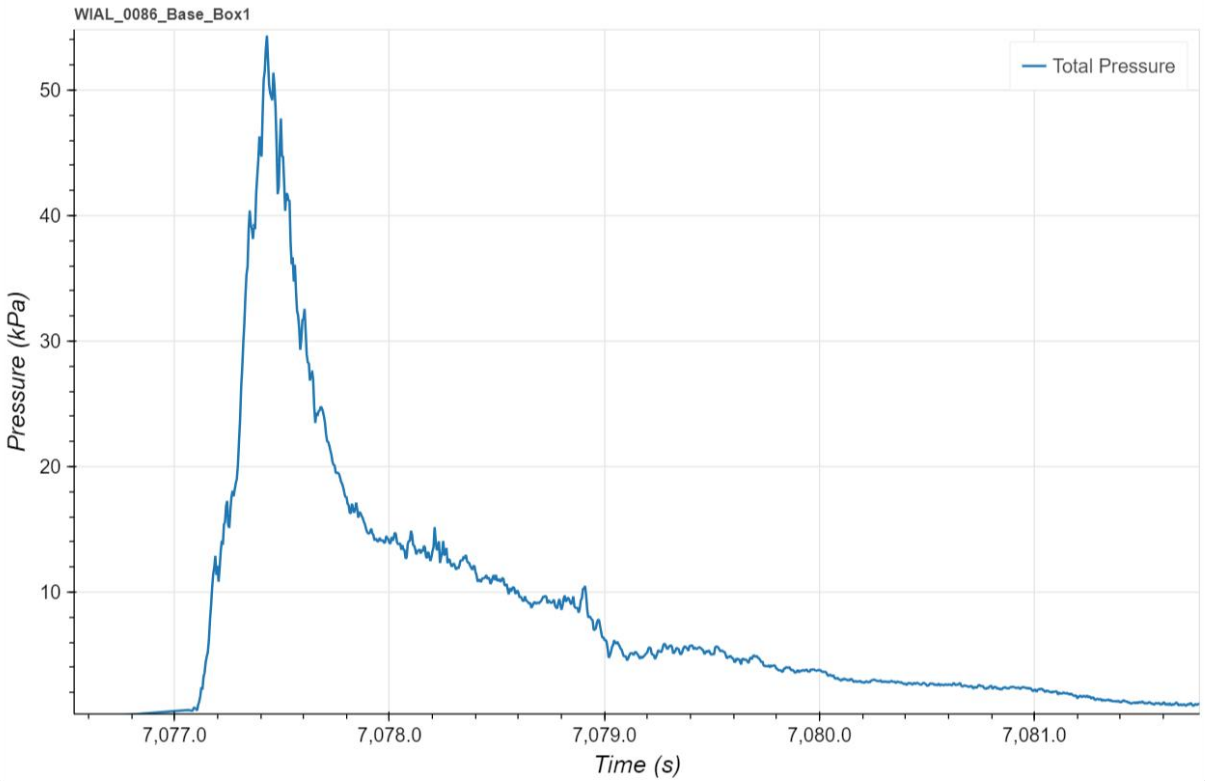


Figure 6.8 P_{max} lower SLSC wall (1.5 m height) 1,000 year ARI 2084 with parapet

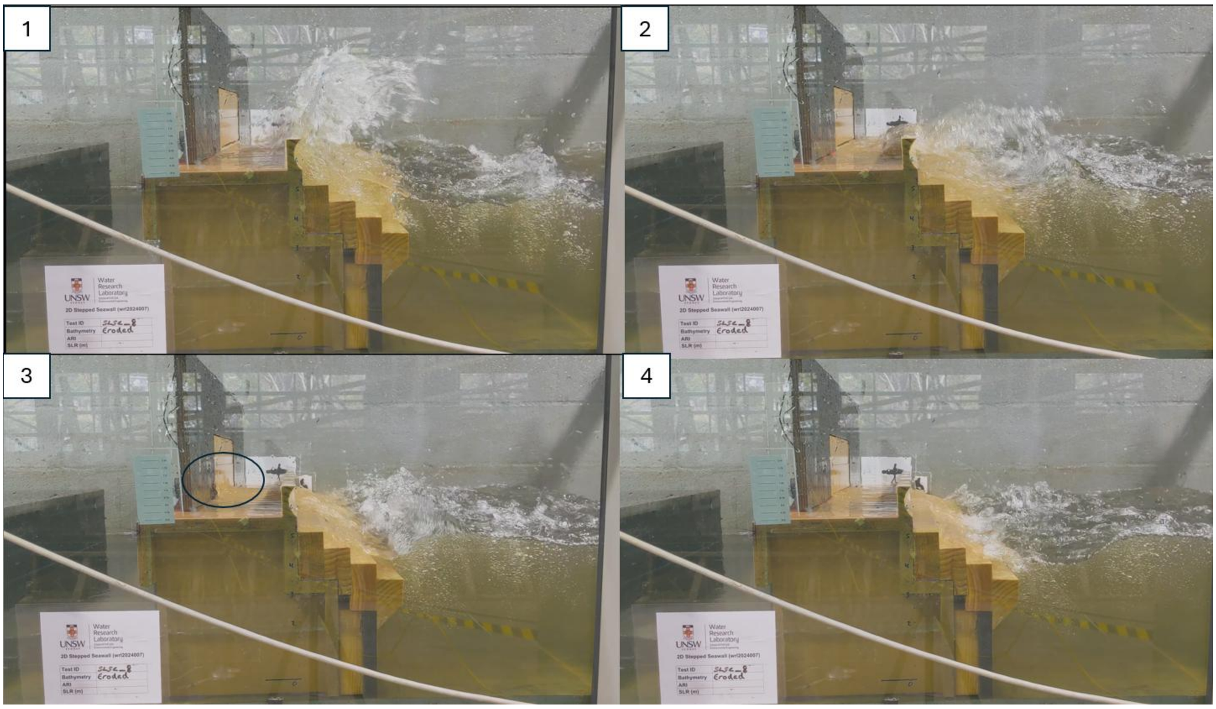


Figure 6.9 P_{max} lower SLSC wall (1.5 m height) 100 year ARI 2024 with parapet

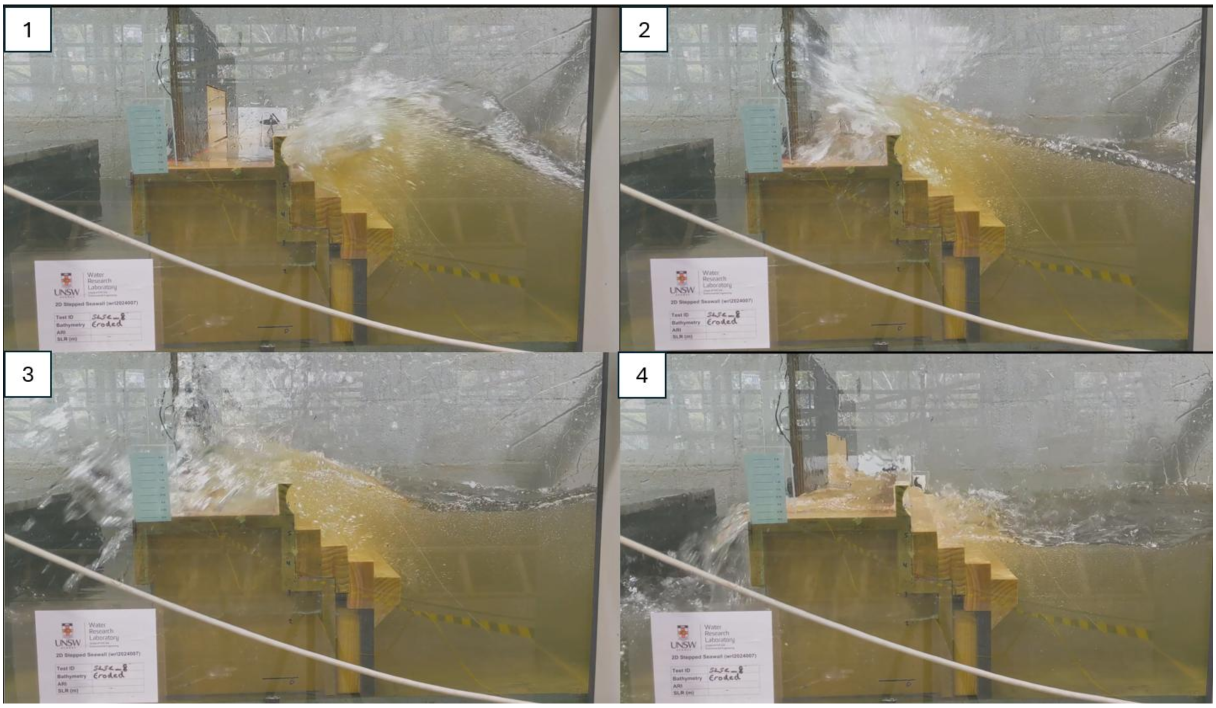


Figure 6.10 P_{max} lower SLSC wall (1.5 m height) 1,000 year ARI 2024 with parapet

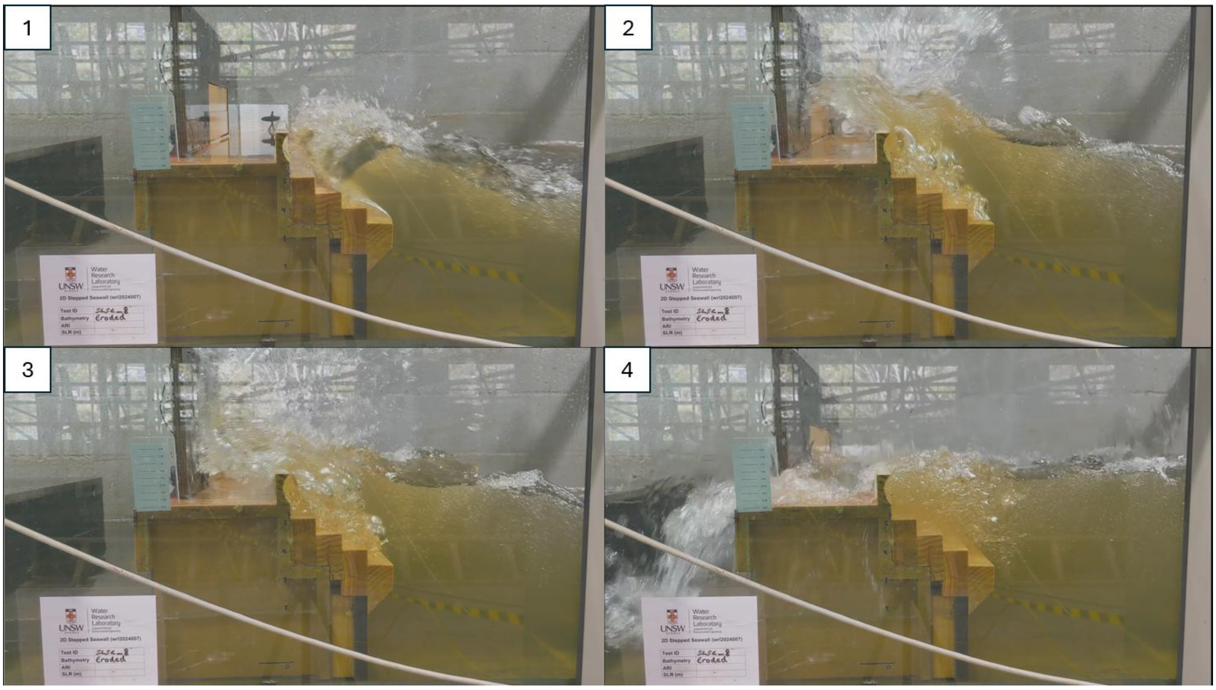


Figure 6.11 P_{max} lower SLSC wall (1.5 m height) 100 year ARI 2084 with parapet

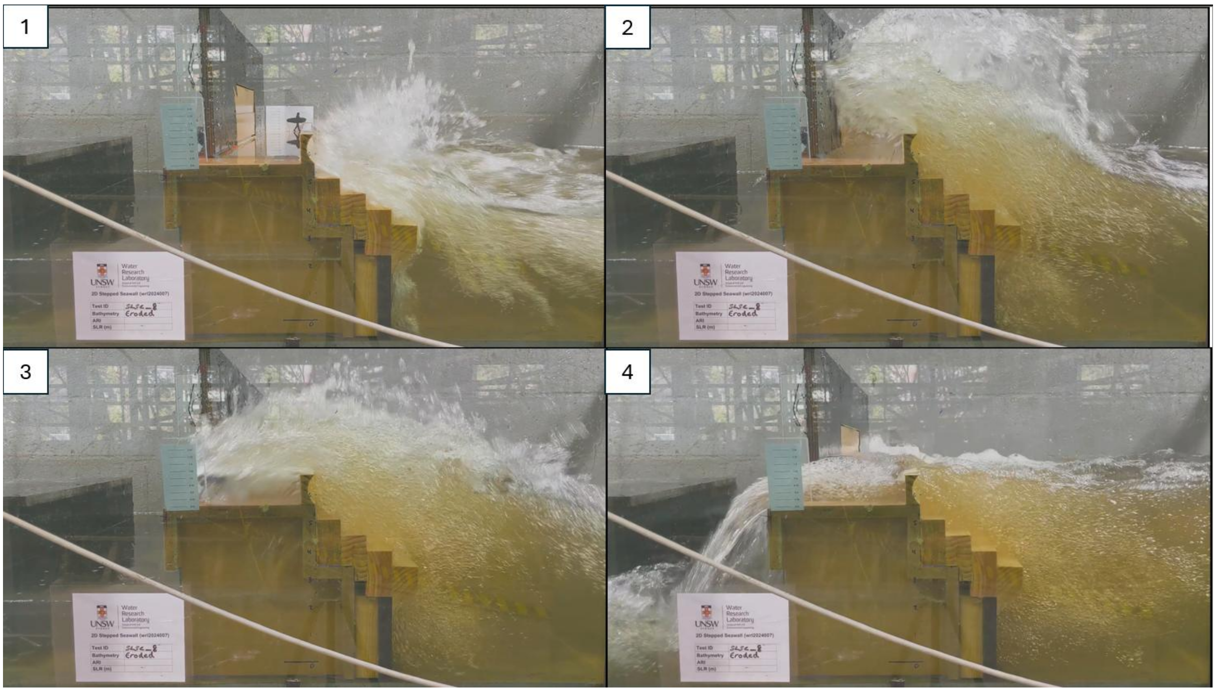


Figure 6.12 P_{max} lower SLSC wall (1.5 m height) 1,000 year ARI 2084 with parapet

7 Summary

7.1 Overview

WRL, the client and their technical advisors developed a physical modelling program to assess wave overtopping and design wave loading behaviour for alterations and additions including a proposed seawall for Newport SLSC.

Numerous combinations of design conditions and modelled structure configurations were tested. The physical modelling focused on wave overtopping flow impacts on the SLSC building and surrounds.

The evolution of designs tested in the model included:

1. The original DA design (Horton, 2020) of a sloping seawall incorporating trafficable stairs
2. A minor modification to the DA design, incorporating stair dimensions complying with the Building Code of Australia (BCA) and alternating areas of trafficable stairs and larger terraced seats (“bleachers”)
3. A quasi three dimensional stair arrangement and three bleachers fronting the main entry of the SLSC building
4. A four bleachers 2D configuration
5. Wave deflector parapet walls of approximately 1 m height and various configurations, including
    a. Straight/vertical
    b. Recurved with a 700 mm radius on the seaward face
    c. Splayed at 45° on the seaward face

Tests were conducted for the following events:

- 1 year ARI
- 10 year ARI
- 100 year ARI
- 1,000 year ARI

Overtopping testing of the structures was conducted with representative average and eroded nearshore profiles.

Extreme structural load tests were undertaken for 100 and 1,000 year ARI conditions to inform the structural design for the wave deflector parapet (in later tests) and the seaward ground floor walls of the existing and proposed SLSC building.

Comparative tests between configurations were generally undertaken for 100 year ARI conditions.

The physical modelling was undertaken by WRL in accordance with best practice international guidelines. The scope of the program was developed collaboratively between WRL and the client to optimise the proposed seawall for the site.

7.2 Overtopping

A total of 94 tests were performed using selected combinations of the seawall, two nearshore profiles, two planning periods and four storm wave conditions. Mean overtopping rates and individual overtopping volumes were recorded for all tests.

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

- Mean overtopping rates with the parapet wall were 75-90% less than without it.
- Mean overtopping rates for the fully eroded profile and average profile were similar.

7.3 Wave loads

Load tests were undertaken for 1,000 waves on 10 m wide sections of the SLSC building wall for 100 and 1,000 year ARI conditions, with and without the parapet wall, with an eroded beach state. The interpretation of the pressure data is the responsibility of a structural engineer.

A summary of load test results including the largest pressure measured (P_{max}) on two 1.5 m high sections of the SLSC building wall is provided in Table 6.1 and Table 6.2.

The lower load cell covered the vertical range: 5.66 m AHD to 7.16 m AHD (1.5 m), for 10 m width. The upper load cell covered the vertical range: 7.16 m AHD to 8.66 m AHD (1.5 m), for 10 m width.

The following maximum wave pressures were observed on the lower panel for very short durations (0.1 to 0.2 s) with a parapet in place:

- 100 year ARI, 2024: 7 kPa
- 100 year ARI, 2084: 13 kPa
- 1,000 year ARI, 2024: 26 kPa
- 1,000 year ARI, 2084: 54 kPa

The largest wave pressures indicated the typical total duration (rise and fall) of the impacts on the SLSC building wall were between 0.1 and 1 s, with the actual peak lasting for as little as 0.005 s. A longer averaging of the duration would reduce their value.

7.4 Conclusion

WRL undertook a physical modelling program to assess wave overtopping and design wave loading behaviour for the Newport SLSC site. The physical modelling was undertaken by WRL in accordance with best practice international guidelines. The scope of the program was developed collaboratively between WRL, the client and the client’s technical advisors to test the DA design and undertake minor modifications to further optimise the seawall for the site.

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Appendix A Model design inputs

The following advice (WRL Ref: WRL2024007 LR20240405 FF) was sent to the Client on 5 April 2024 regarding design model inputs. Concurrence was obtained from the Client's technical advisors.

5 April 2024

WRL Ref: WRL2024007 LR20240405 FF

SUBJECT TO LEGAL PRIVILEGE

Eskil Julliard
Northern Beaches Council
c/o- King & Wood Mallesons (Contact: Steven Adler)
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Sydney NSW 2000

By email: Stella.Zhao@au.kwm.com steven.adler@au.kwm.com; kate.dean@au.kwm.com

Dear Stella and Steven,

Newport SLSC seawall physical modelling advice
Stage 1: Revised physical modelling inputs

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney was engaged by Northern Beaches Council (hereafter "Council") care of King & Wood Mallesons (KWM) to provide coastal engineering advice regarding proposed upgrades and extensions to the building and seawall for Newport SLSC (Surf Life Saving Club). This letter report outlines the proposed physical modelling design conditions which are recommended to be used for the wave flume two-dimensional (2D) physical modelling which will be undertaken at a later stage in this project. This letter report supersedes WRL's previous letter reports (ref: LR20240305 issued 5 March 2024 and ref: LR20240314 issued 14 March 2024) following discussions regarding physical modelling conditions on 8 March 2024 and 27 March 2024 meetings with project stakeholders.

2. Executive summary

A summary of the proposed flume test inputs is shown in Table 2.1, with the details of these shown in subsequent sections.

Table 2.1 Summary of the proposed flume test inputs

		1 year ARI	10 year ARI	100 year ARI	1,000 year ARI
Proposed model scale	1:25				
Design life	60 years				
Design ARIs					
Pedestrian safety	1	✓			
Minor inundation	10		✓		
Major inundation and structural loads on parapet	100			✓	
Structural loads on parapet or building fabric	1000				✓
Still water level (m AHD)		1.22	1.34	1.46	1.58
Design waves					
Offshore Hs (m)		4.4	6.2	7.8	9.6
Offshore Tp (s)		11.0	12.1	13.0	13.8
Critical offshore direction	ESE	✓	✓	✓	✓
Sea level rise	Present Day 2024 (0 m)	✓	✓	✓	
	2084 (+0.53 m)	✓	✓	✓	✓
Design sand level against wall (m AHD)					
5.8	*Initial sensitivity test*			✓	
4.0	*Initial sensitivity test*			✓	
-1.0	*Initial sensitivity test*			✓	
	Later tests to be determined following testing of three initial profiles	✓	✓		✓

3. Design events

3.1 Design risk, standards and design events

3.1.1 Probability terminology

The following definitions, adopted from Pilgrim (1987), are provided in Table 3.1.

Table 3.1 Probability terminology used for design events

Terminology	Definition
Risk	Likelihood (or probability) times consequence.
Average Recurrence Interval (ARI)	The average time between exceedances (e.g. large wave height or high water level) of a given value, also known as Return Period.
Annual Exceedance Probability (AEP)	The probability (expressed as a percentage) of an exceedance (e.g. large wave height or high water level) in a given year.
Project Life (N)	Also known as planning timeframe or planning horizon.
Encounter Probability	The probability (expressed as a percentage) of an exceedance (e.g. large wave height or high water level) over the project/planning duration.

3.1.2 Design events for coastal protection structures

When establishing the suitable design events for a given coastal protection structure, several key factors should be taken into account:

- The design life of the coastal protection structure (i.e. the seawall)
- The acceptable risk of failure of the coastal protection structure (i.e. withstand and resist the forces exerted by waves)
- The acceptable level of serviceability of the coastal protection structure (i.e. provide acceptable levels of protection from wave overtopping)
- The design life of the asset being protected (i.e. the SLSC building)

According to the Coastal Engineering Manual EM 1110-2-1100 (Part V; USACE, 2006), it is common practice to select an economic life of 50 years for the analysis of coastal structures. This choice does not suggest that the structure is designed to last only 50 years, but rather that the analysis of its benefits and costs is focused on that specific period. Substantial work was published in Gordon, Carley and Nielsen (2019) regarding the acceptable probability of failure for a given design life for coastal structures, depending on the type of asset being protected. The suggested design life for coastal protection structures fronting low density public areas ranges between 20 and 40 years and between 60 and 100 years when fronting normal residential assets.

The primary function of the proposed seawall is to provide coastal protection to both Newport SLSC and the public users located behind it. The current proposal by Council is to adopt a 60 year coastal engineering design, which conforms to industry standards and requirements to ensure the seawall's effectiveness and durability.

Australian Standard (AS) 4997-2005 *Guidelines for the Design of Maritime Structures* recommends design wave heights based on the function and design life of the structure as reproduced in Table 3.2. According to the Australian Standard (AS) 4997-2005, a design event for a Function category 2 (normal structure) is from 500 up to 1,000 year Average Recurrence Interval (ARI) for a 50 to 100 year design life. Common practice in Australia for coastal hazard assessments often favours using the 100 year ARI as the design criterion, which aligns with many flood policies.

Table 3.2 Annual probability of exceedance of design wave events (source: AS 4997-2005)

Function Category	Structure Description	Encounter Probability (a, b)	Design Working Life (years) 5 or less (temporary works)	25 (small craft facilities)	50 (normal maritime structures)	100 or more (special structures/ residential developments)
1	Structures presenting a low degree of hazard to life or property	~20%(c)	1/20	1/50	1/200	1/500
2	Normal structures	10%	1/50	1/200	1/500	1/1,000
3	High property value or high risk to people	5%	1/100	1/500	1/1,000	1/2,000

(a) Apart from the column "Encounter Probability (calculated by WRL), the table is a direct quote from AS 4997-2005

(b) Inferred by WRL based on encounter probability equation

(c) The encounter probability for temporary works, normal maritime structures and special structures in Function Category 1 is ~20%

Note that AS 4997 specifically addresses rigid maritime structures such as wharves and concrete seawalls. Seawalls being considered are typically smaller structures that are often part of broader foreshore management solutions

It is recommended that coastal structure performance response be assessed for a range of events, including conditions above and below the nominal design level, and not just for a single design event (CIRIA; CUR; CETMEF, 2007). This approach is particularly crucial when considering serviceability of the proposed structure and its overtopping performance for lower ARI design events. Table 3.3 presents the risk of event occurrence during the 60 year design lifetime of a structure.

Table 3.3 Chance of event occurring during the 60 year life (adapted from CIRIA; CUR; CETMEF, 2007)

Structure Description	Design life (years)	Event probability (%) for various return periods				
		1 year	20 years	100 years	500 years	1000 years
Permanent structure	60	>99	94	45	11	5

Table 3.4 provides the event frequencies for a range of design events.

Table 3.4 Event frequency of design events for temporary and permanent coastal structures (adapted from CIRIA; CUR; CETMEF, 2007)

Structure Description	Design life (years)	Event frequency				
		Frequent	Probable	Occasional	Remote	Improbable
Temporary works	<1	0.01 year ARI	0.1 year ARI	1 year ARI	10 year ARI	100 year ARI
Permanent structure	30 to 100	0.1 year ARI	1 year ARI	10 year ARI	100 year ARI	1,000 year ARI

3.2 Coincidence of extreme waves with extreme water levels

Contemporary engineering practice in Australia is to assume the concurrence of extreme waves and extreme water levels, with generally similar magnitude ARI events. That is, 100 year ARI waves are assumed to coincide with 100 year ARI water levels. While this is somewhat conservative, it also manages the inherent uncertainty of oceanic processes.

As the peaks of both parameters are unlikely to exactly coincide, some practitioners utilise a 6 hour exceedance wave height, rather than a 1 hour exceedance to reduce the conservatism, but there is minimal work demonstrating the mathematical veracity of this. Since long records of measured data and model output are available, it is possible to create long records of multiple variables and sample the combined result, however, such an approach is in its infancy and beyond the scope of this study.

For this project, WRL made the following assumptions regarding coincidence

- 1 year ARI (1 hour exceedance) wave height coincides with MHWS (Mean High Water Springs) water level
- 10, 100 and 1,000 year ARI (1 hour exceedance) wave heights coincide with 10, 100 and 1000 year ARI water levels, respectively

3.3 Proposed ARI of design events for Newport SLSC seawall

Key performance criteria for consideration include:

- **Safety of path users** – provide safe passage (from overtopping) to users during events. Dangerous conditions can be managed through a forecasting system and safety plan. It is suggested that initial assessment of this be **for a 1 year ARI assuming Present Day (2024) or 2084 water levels**, including no violent overtopping at seawall. Taken to be a tolerable limit of 1 L/s/m in this event (EurOtop, 2018).
- **Potential minor damage or minor inundation to SLSC exposed infrastructure** (from overtopping) during events up to a **10 year ARI assuming Present Day (2024) or 2084 water levels**, including no violent overtopping at seawalls. Taken to be a tolerable limit of 5 L/s/m in this event (and/or measured forces with a threshold to be determined.

- **Structural stability of seawall/parapet and/or major inundation** – minimal damage during a **100 year ARI event assuming 2084 water levels** measured through wave forces with a threshold to be determined.
- **Structural stability to seawall/parapet and SLSC building** – minimal damage during a **1,000 year ARI event assuming 2084 water levels** measured through wave forces with a threshold to be determined.

4. Water levels and waves

4.1 Preamble

Design water levels are caused by elevated water levels coupled with extreme waves impacting the coast. The elevated water levels consist of (predictable) tides and what is referred to as a tidal anomaly (or tidal residual). Tidal anomalies primarily result from factors such as wind setup and barometric effects, which are often referred to as "storm surge". Additionally, water levels within the surf zone (i.e. nearshore) are also subject to wave setup and wave runup. Design water levels for the proposed seawall with its adopted design life of 60 years will also require consideration of sea level rise.

4.2 Storm tide (astronomical tide + anomaly)

Astronomical tidal planes for Sydney (Port Jackson), based on the HMAS Penguin tide gauge record, are shown in Table 4.1 from MHL (2023).

Table 4.1 Average annual tidal Planes (2001-2020) Sydney (for Port Jackson at HMAS Penguin, Source: MHL, 2023)

Tidal Planes	Level (m AHD)
Highest Astronomical Tide (HAT)	1.150
Mean High Water Springs (MHWS)	0.663
Mean High Water (MHW)	0.540
Mean High Water Neaps (MHWN)	0.418
Mean Sea Level (MSL)	0.044
Mean Low Water Neaps (MLWN)	-0.330
Mean Low Water (MLW)	-0.452
Mean Low Water Springs (MLWS)	-0.575
Lowest Astronomical Tide (LAT)	-0.860

As stated above, tidal anomalies primarily result from factors such as regional wind setup (or setdown) and barometric effects, which are often combined as “storm surge”. Additional anomalies occur due to “trapped” long waves propagating along the coast. Design storm surge levels (astronomical tide + anomaly) were recommended in the Coastal Risk Management Guide (NSW DECCW, 2010 after Watson and Lord, 2008) based on data from the Fort Denison tide gauge in Sydney.

An updated analysis was conducted in 2018 by NSW Government’s Manly Hydraulic Laboratory (MHL, 2018) with slightly lower design water levels overall. The proposed design levels for this study were derived from these two previous studies for 2024 by accounting for historical sea-level rise (taken as 2 mm/year) at Fort Denison based on the work of Watson (2020). A summary of adopted design water levels is presented in Table 4.2; note that these values exclude wave setup and runup effects which can be significant where waves break on shorelines.

Table 4.2 Design tidal water levels + anomaly (Sydney) excluding wave setup and wave runup

ARI (years)	2008 water level (m AHD) (NSW DECCW, 2010)	2017 water level (m AHD) (MHL, 2018)	2024 design still water level (m AHD)(2)
1	1.24	1.18(1)	1.22
10	1.35	1.31(1)	1.34
20	1.38	1.35	1.38
100	1.44	1.42	1.46
500	1.54(1)	1.51(1)	1.55
1,000	1.58(1)	1.55(1)	1.58

(1) These water level values were extrapolated by WRL using a log-linear fit

(2) The 2024 design water levels were derived from (NSW DECCW, 2010) and (MHL, 2018) adjusted to 2024 using a constant historical SLR rate of 2 mm/year. The proposed 2024 design still water levels are an average of the adjusted NSW DECCW and MHL water levels

4.3 Future sea level rise

Given the adopted design working life of 60 years and assuming the structure is to be built in 2024, its working life will end in 2084.

In the absence of official NSW sea level rise (SLR) benchmarks, the SLR values adopted by WRL were based on the more recent IPCC AR6 (2021) report. The IPCC report provides global mean sea level rise projections for five Shared Socioeconomic Pathways (SSPs), with each SSP capturing different emissions scenarios. WRL adopted SLR values for this study were based on SSP5–8.5 (Very High emissions scenario – medium confidence) using the NASA sea level projection tool (NASA, 2024) for the Sydney location.

Table 4.3 Sea level rise projection (Source IPCC, 2021)

Planning Period (year)	Sea Level Rise (m)[1]
2024	0.00
2050	0.16
2080	0.45
2084	*0.50*[2]
2090	0.57
2100	0.71
2150	1.28

(1) SLR values were adjusted to 2024 as IPPC (2021) SLR values are relative to 2020

(2) 2084 SLR values were interpolated using a 2nd degree polynomial fit

Based on the adopted design working life of 60 years and assuming the structure is to be built in 2024 and further discussion with project stakeholders, a sea level rise of 0.53 m (increase above 2024 MSL in 2084) was adopted for adjusting future design water levels. Note that there are numerous other SSP scenarios – most of which have a lower sea level rise projection than the above. For testing purposes, a single value of 0.53 m has been adopted. This would occur at the end of the planning period.

Present engineering practice is to consider this sea level rise in the design of the structure. There is an argument that a more frequent design condition can be accepted at the end of the structure's life, however, techniques for this are not yet well developed.

4.4 Waves

Newport Beach is characterised by moderate to high energy wave climate (typically offshore generated wave swell) with some protection offered from swell waves from the south by Newport Reef (Little Reef, offshore of Bungan Head). Nearshore wave heights beyond the surf zone are typically 90% of those at a fully exposed open ocean beach and would be further characterised during the physical modelling.

Estimates for the 1, 10, 50, and 100 year return periods of non-directional (Glatz et al., 2017) and directional extreme waves Shand et al., 2011a) in the Sydney region were derived from an analysis of directional data collected by the Sydney wave buoy and are provided in Table 4.4.

The significant wave height from the southeast direction (at the offshore wave buoy) expected to occur or be exceeded for approximately 1 hour every 100 years was calculated to be 7.8 m for the direction of interest (east-southeast) for the project site.

Table 4.4 Offshore directional extreme wave conditions at Sydney wave buoy (source: Glatz et al., 2017 and Shand et al., 2011a)

Offshore Wave Direction		One Hour Exceedance H_S (m)				
		1 year ARI	5 year ARI	10 year ARI	50 year ARI	100 year ARI
All directions(1)	-	5.8	7.1	7.6	8.9	9.4
N to E(2)	0 to 90	3	4.2	4.5	5.4	5.7
E to SE(2)	90 to 135	4.4	5.7	6.2	7.4	7.8
SE to SW(2)	135 to 225	5.9	7	7.5	8.6	9

(1) These values were reported in Galtz et al. (2017)

(2) These values were reported in Shand et al. (2011a)

Offshore peak wave period for design conditions from the Sydney wave buoy (Shand et al., 2011b) are provided in Table 4.5.

Table 4.5 Offshore (Sydney) extreme peak wave conditions

ARI (years)	Offshore Tp (s)
1	11.0
10	12.1
50	12.7
100	13.0

4.5 Proposed design conditions

The proposed test conditions for the physical modelling to be conducted in a 2D wave flume at WRL are provided in Table 4.6.

Table 4.6 Proposed offshore wave and water level test conditions for physical modelling

Case	ARI (years)	Planning Period (year)	Design still water level (m AHD)(1)	Hs (m) (2)	Tp (s)
1	1 (waves) + MHWS(3) SWL	2024	0.67	4.4	11.0
2	10	2024	1.34	6.2	12.1
3	100	2024	1.46	7.8	13.0
4	1,000(4)	2024	1.58	9.6	13.8
5	1 (waves) + MWHS(3) + SLR	2084	1.20	4.4	11.0
6	10	2084	1.87	6.2	12.1
7	100	2084	1.99	7.8	13.0
8	1,000(4)	2084	2.11	9.6	13.8

(1) The design water levels do not include wave setup and wave runup are these will be inherently generated within the wave flume through wave processes.

(2) Design offshore wave conditions are provided based on the deep water wave buoy analysis. Design offshore wave heights in the physical modelling will be limited by the maximum achievable wave height by the flume wave maker and will require adjustment to the test still water level to account for the reduced wave setup generated.

(3) The 1 year ARI wave condition is to be conducted with a MHWS tide water level.

(4) The 1,000 ARI values were extrapolated by WRL using a log-linear fit.

At the proposed scale of 1:25, the largest offshore H_s which can be produced in the physical model at the wave maker will be 6.2 m. As this maximum achievable offshore H_s condition will be less than the target offshore design conditions for the proposed 100 year ARI and 1,000 year ARI, WRL will raise the test still water level to account for the reduced nearshore wave setup generated in the wave flume. This will be necessary due to the fact that nearshore wave conditions (i.e. close to the proposed seawall toe) are depth limited and, as such, the wave height at the seawall will be strongly dependent on the total water depth including wave setup.

Numerical modelling will be first undertaken, before the start of the physical modelling, to determine the extent to which the test still water levels for the 100 year ARI and 1,000 year ARI should be raised above the respective design still water levels to account for the reduced wave setup in the flume.

The design offshore conditions presented in Table 4.6 will be applied as a boundary to the Dally, Dean and Dalrymple (1984) 2D surf zone model (SBEACH) for the full bathymetry profile at Newport (see Section 5.1) and wave setup at the -1 m AHD contour will be estimated.

The SBEACH model will then be re-run with the reduced maximum achievable wave height condition that can be run in the wave flume to obtain the reduced wave setup at the -1 m AHD contour. The calculated difference in wave setup will be used to inform the amount by which the test still water levels need to be raised above the respective design still water levels during all tests in the flume.

5. Bathymetry and beach levels at toe of proposed seawall

5.1 Introduction

The NSW Department of Planning, Industry and Environment (NSW DPIE), provides topographic and bathymetric data based on Airborne LiDAR Bathymetry (ALB) technology conducted by Fugro Pty Ltd from July to December 2018. The bathymetric data was accessed through the ELVIS portal (https://elevation.fsdf.org.au/) and downloaded at a resolution of 5 m.

Analysis of this bathymetric data indicates that the nearshore seabed slope fronting the proposed seawall is relatively mild and constant across the embayment (Figure 5.1). It can be idealized (see dashed line in Figure 5.2) as 1V:45H between -20 m AHD and -10 m AHD, slightly steepening to 1V:35H between -10 m AHD and -2 m AHD, with the presence of relatively flat 50 m wide intertidal terrace and an upper beach face of about 1V:10H.

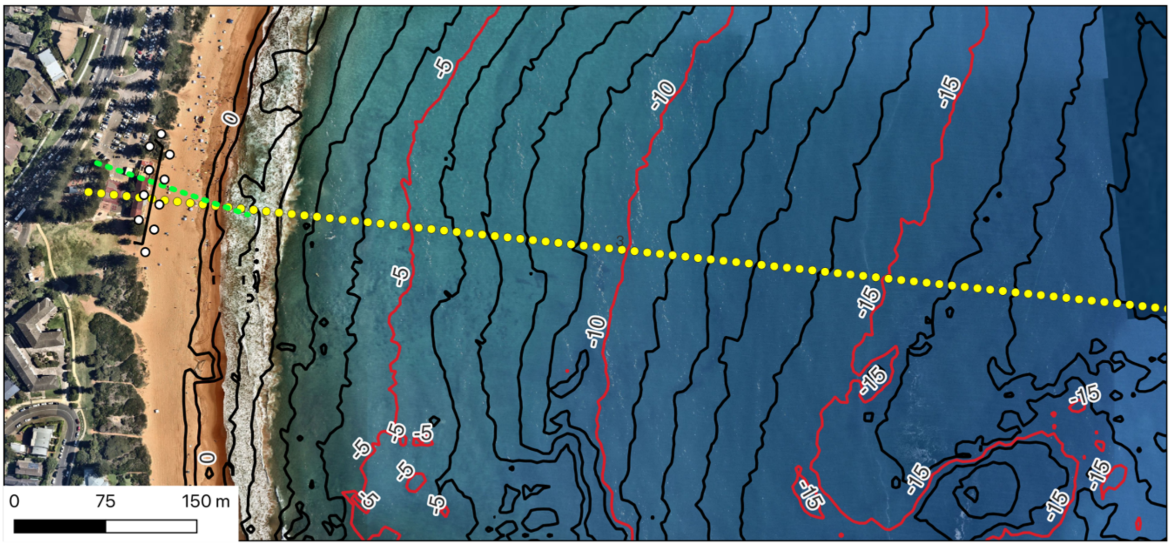

Figure 5.1 NSW Marine Lidar Bathymetry Data 2018 at Newport Beach

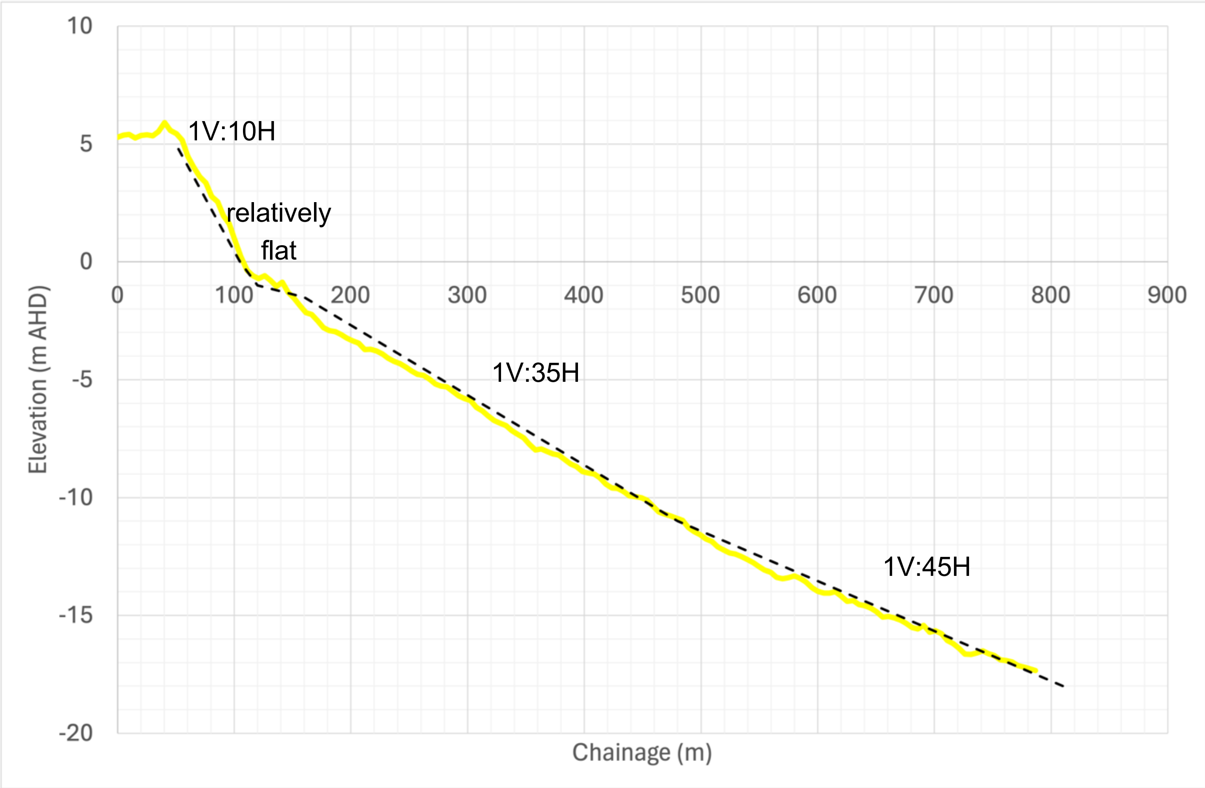

Figure 5.2 Typical beach profile fronting proposed seawall at Newport Beach

5.2 Estimation of likely range of sand level (scour) at toe of wall

5.2.1 Measured data

Available measured profiles from the NSW Beach Profile Database (http://www.nswbpd.wrl.unsw.edu.au/photogrammetry/nsw/) are shown in Figure 5.3 for the profile intersecting the proposed seawall (plotted in green on Figure 5.1). The indicative position of the proposed seawall has been added based on the 2021 design information previously reviewed by WRL (see WRL 2021005 JTC FF LR20210706).

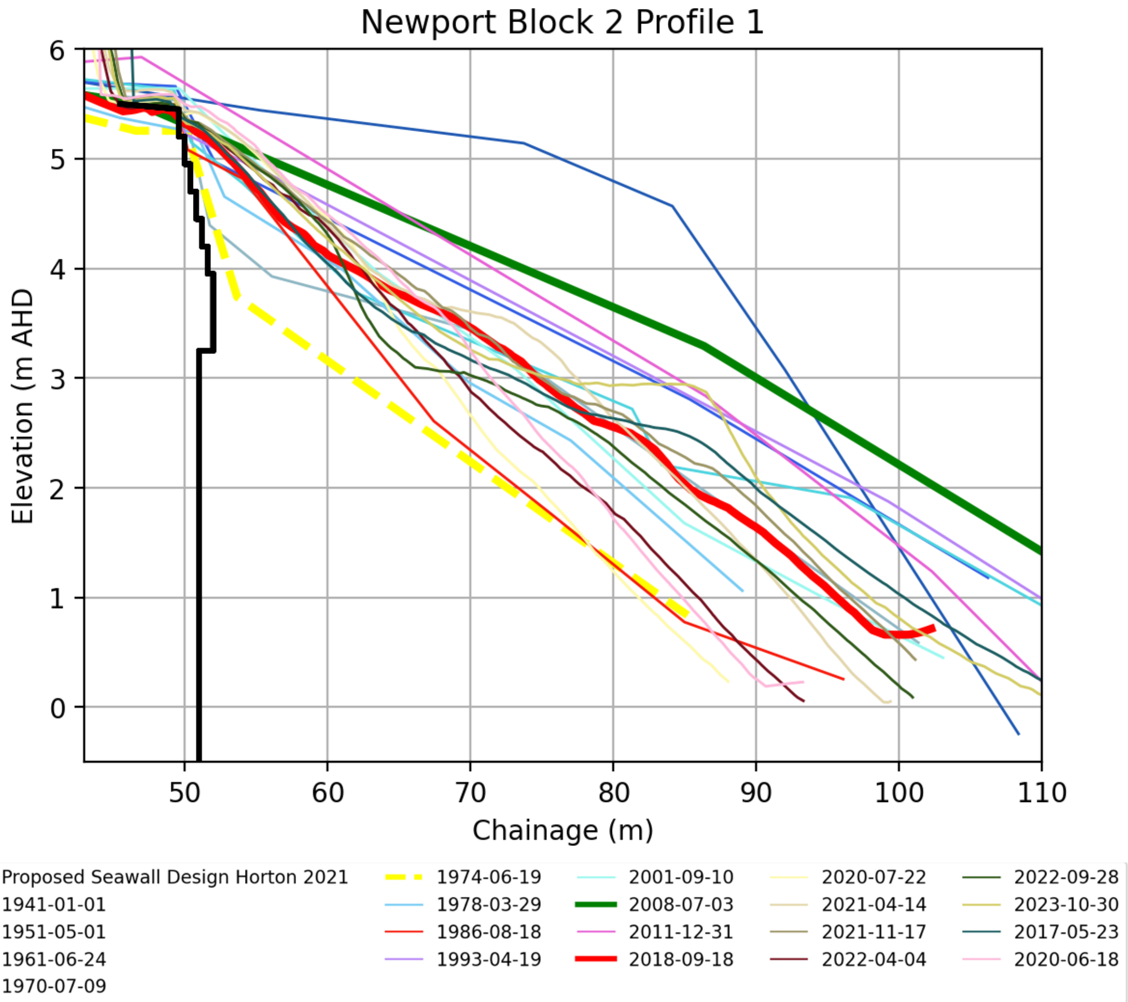

Figure 5.3 Measured profile data with proposed seawall superimposed data

The most eroded profile was 1974, which was collected on 19 June 1974. The renowned 1974 storms were actually a sequence of storms, with the largest being 25 to 29 May 1974 and 3 to 15 June 1974 (an exceptionally long duration), Foster et al, 1975. Rock rubble was placed seaward of the SLSC building in response to these storms, so the profile may have been more eroded at some point during the storm than on 19 June 1974.

The most recent accreted profile was measured in 2008 with an average slope 1V:15H between 0 m AHD and +5 m AHD. The 2018 profile was representative of the average beach state with a slightly steeper beach face of 1V:10H between +1 m AHD and +4 m AHD.

5.2.2 Modelled scour levels

WRL carried out beach erosion numerical modelling for nominal design events with the presence of the proposed 2021 vertical seawall in front of the Newport SLSC (see WRL 2021005 JTC FF LR20210706). WRL set up a two-dimensional numerical beach erosion model using SBEACH (Larson, Kraus and Byrnes, 1990) to predict scour levels for an agreed range of ARI events (e.g. 100, 500, 1000 and 2000 year ARI) at the toe of the proposed buried seawall for Present Day (2021) and future planning horizons using the methodology detailed in Carley et al. (2015).

Figure 5.4 presents estimates of the scour depth at the toe of the 2021 proposed seawall design at Newport Beach for design conditions ranging from 100 year ARI to 2000 year ARI. SBEACH modelling indicated that scour levels between -0.5 m AHD and -1 m AHD could be expected to occur in front of the proposed seawall, which is in agreement with historical scour levels and observed scour levels during major storms in front of existing permeable and non-permeable seawalls along the NSW coast (Foster et al., 1975).

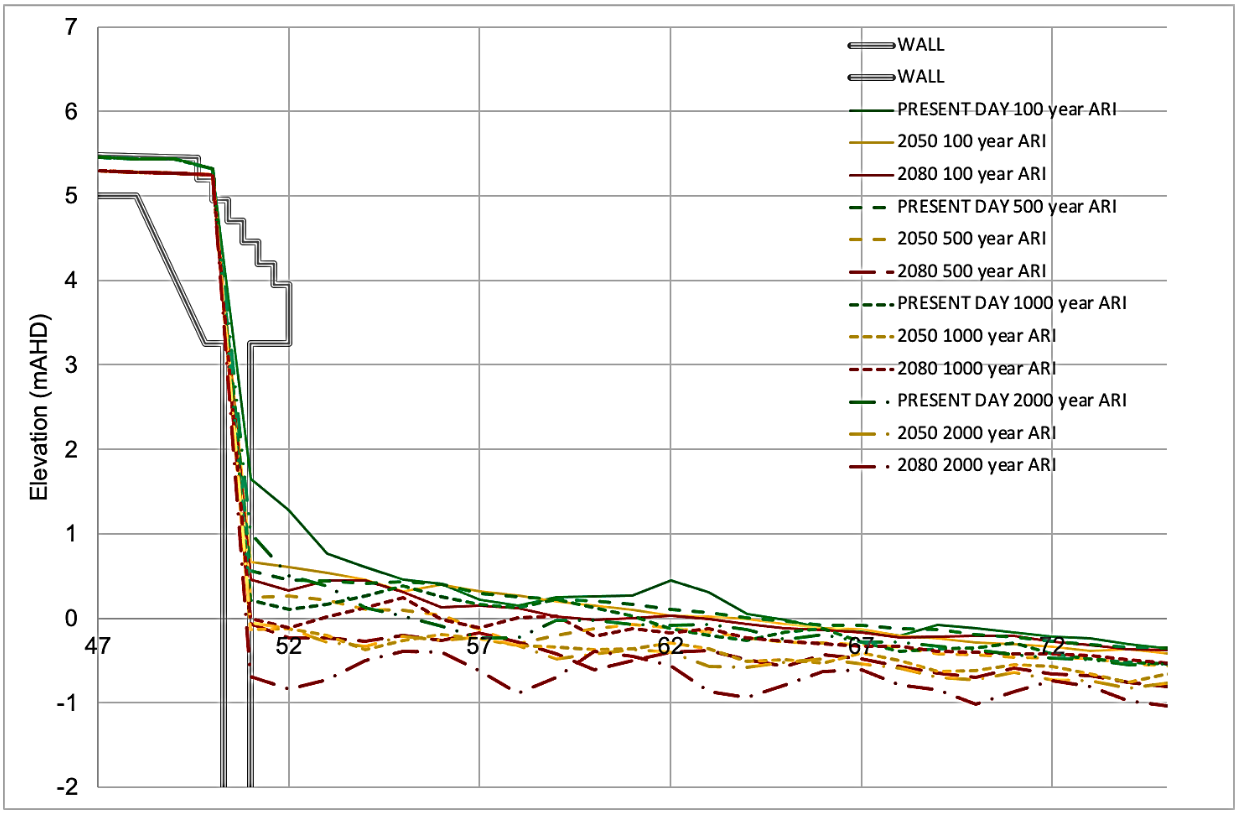

Figure 5.4 Evolution of beach profiles for consecutive storms in SBEACH with seawall in place

5.3 Proposed nearshore bathymetry and toe levels at the wall

It is proposed during the initial stages of the physical modelling to first identify a worst-case nearshore coastal profile for the project area which maximises wave runup and overtopping for a typical severe storm condition (100 year ARI selected) occurring today (Present Day 2024).

This critical profile (Figure 5.5) is proposed to be chosen from either:

- A fully eroded flat coastal profile at -1 m AHD in front of the seawall
- An average profile with a 1V:10H slope from 4 m AHD down to -1 m AHD (note that this would cover any potential wave deflector feature elevated below +4 m AHD)
- An accreted sand profile with a 1V:15H slope from the promenade level (approximately 5.8 m AHD) down to approximately -1 m AHD

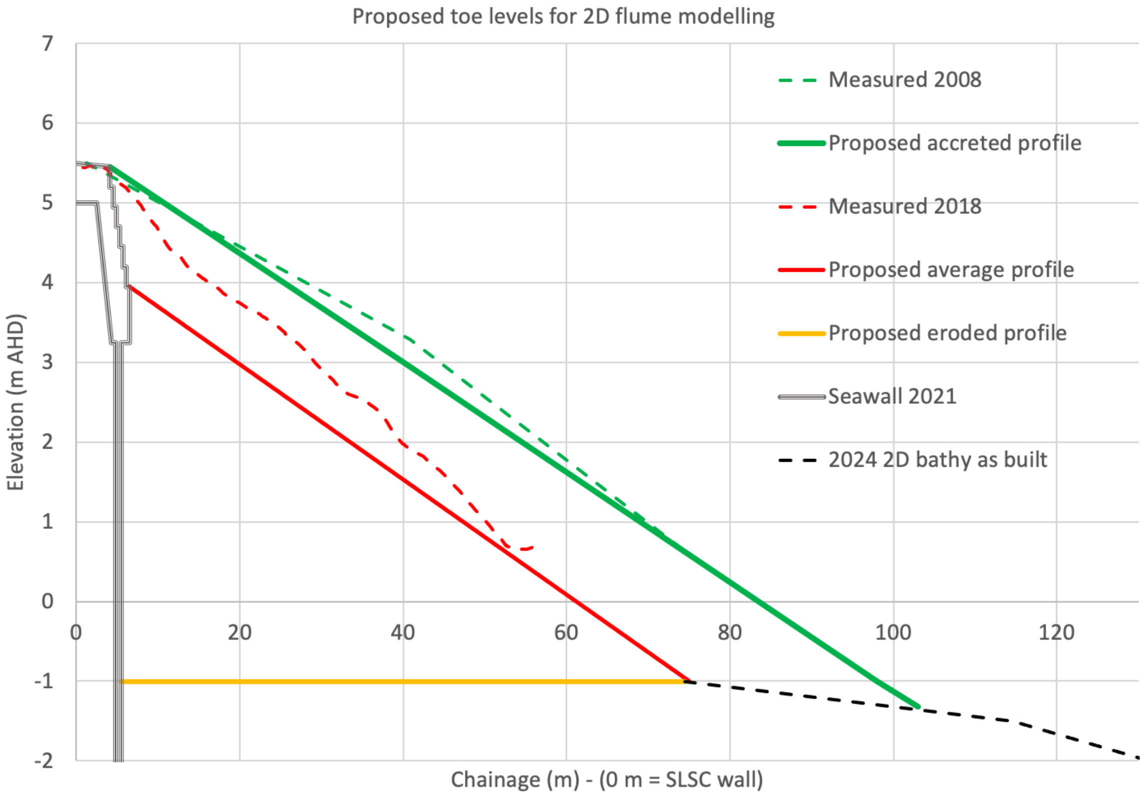

Figure 5.5 Proposed profiles and associated toe/crest levels for critical profile identifications (orange = eroded; red=average, green=accreted)

All subsequent testing in the physical modelling program would be conducted for the chosen critical coastal profile informed by the initial modelling (i.e. nearshore model bathymetry) which would be extended offshore down to an approximate depth of -15 m AHD using the 2018 LiDAR bathymetry data discussed in Section 5.1. All substrates will be modelled using rigid smooth marine plywood.

6. Proposed model scale

Based on consideration of the above variables and our knowledge of the capabilities of WRL's 1.2 m wide wave flume, it is proposed to test the project at a length scale of 1:25 (subject to minor adjustment and/or further detailed calculations).

7. Summary

An executive summary is provided in Section 2 of this letter. Please contact Dr Francois Flocard (0420 423 382) or James Carley (0414 385 053) by phone or email should you require further information.

Yours sincerely

Brett Miller
Director, Industry Research

8. References

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Appendix B Previous WRL coastal engineering advice

The following advice (WRL Ref: WRL2021004 LR20210708 JTC FF) was sent to the Client on 8 July 2021 regarding coastal engineering advice for the site.

8 July 2021

WRL Ref: WRL2021004 JTC FF LR20210708

UNSW
Water Research
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Bernard Koon
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Dear Bernard,

Newport SLSC coastal engineering advice

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney is pleased to provide this coastal engineering advice in relation to proposed coastal protection works at Newport SLSC.

WRL provided a peer review of the following documents on 14 May 2021:

- Horton (2020a), "Coastal Engineering Report and Statement of Environmental Effects for Buried Coastal Protection Works at Newport SLSC", prepared by Horton Coastal Engineering Pty Ltd for Adriano Pupilli Architects, Issue 2 dated 16 November 2020.

As part of this review process, the following feeder documents were sourced and sighted, but not reviewed in detail:

- Horton (2018) "Initial Coastal Engineering Advice on Newport SLSC Development", prepared by Horton Coastal Engineering Pty Ltd for Adriano Pupilli Architects, dated 14 August 2018.

- Horton (2020b) "Assessment of Options for Redevelopment of Newport SLSC, with Updated Consideration of Risk from Coastal Erosion/Recession", prepared by Horton Coastal Engineering Pty Ltd (Horton) for Adriano Pupilli Architects, Issue A, dated 17 February 2020.

- Horton (2020c), "Coastal Engineering and Flooding Advice for Newport SLSC Clubhouse Redevelopment", prepared by Horton Coastal Engineering Pty Ltd (Horton) for Adriano Pupilli Architects, Issue 2, dated 9 November 2020.

Additional work arising from the peer review is presented below, and provides enhanced quantification and detail on a number of design parameters, namely:

- Estimate the likely range of sand level (scour) at toe of proposed seawall
- Estimate wave runup levels and overtopping which could impact Newport SLSC
- Estimate wave loads due to overtopping which could impact Newport SLSC
- Assessment of seawall end effects

2. Design Conditions

Substantial work was published in Gordon, Carley and Nielsen (2019) regarding the acceptable probability of failure for a given design life for coastal structures, including reference to Australian and international standards. Suggested design life and design event are shown in Table 1.

Table 1: Design offshore wave conditions

Type of asset to be protected	Category	Acceptable Encounter Probability (%)	Design Life for Asset (years)	Design ARI for Protective Structure (years)
Temporary works	1	20 to 30	5 to 10	20 to 50
Parkland and low value infrastructure	2	10 to 12	20 to 40	200 to 300
Normal residential	3	4 to 5	60 to 100	1,000 to 2,000
High value assets and intense residential	4	2 to 3	100	3,000 to 5,000
Very high value natural or built assets	5	"No damage"	100+	10,000

Australian Standard (AS) 4997-2005 *Guidelines for the Design of Maritime Structures* recommends design wave heights based on the function and design life of the structure as reproduced in Table 2. Note that while this standard covers rigid maritime structures (e.g. wharves and concrete seawalls), it specifically excludes the design of flexible "coastal engineering structures such as rock armoured walls, groynes, etc." However, in the absence of any other relevant Australian Standard, it is commonly considered in the assessment of probability in contemporary Australian coastal engineering practice.

Table 2: Annual Probability of Exceedance of Design Wave Events (source AS 4997-2005)

Function Category	Structure Description	Encounter Probability (a, b)	Design Working Life (Years)			
			5 or less (temporary works)	**25** (small craft facilities)	**50** (normal maritime structures)	**100 or more** (special structures/ residential developments)
1	Structures presenting a low degree of hazard to life or property	~20%(c)	1/20	1/50	1/200	1/500
2	Normal structures	10%	1/50	1/200	1/500	1/1000
3	High property value or high risk to people	5%	1/100	1/500	1/1000	1/2000

(a) Apart from the column "Encounter Probability (calculated by WRL), the table is a direct quote from AS 4997-2005.

(b) Inferred by WRL based on encounter probability equation.

(c) The encounter probability for temporary works, normal maritime structures and special structures in Function Category 1 is ~20%. However, the encounter probability for small craft facilities in Function Category 1 is 39%.

Design conditions for the potential design life of the seawall fronting the Newport SLSC have been defined for average recurrence intervals (ARIs) of 100, 500, 1000, and 2000 years to better estimate the probability of failure throughout the design life of both the seawall and the asset it designed to protect, that is the Newport SLSC.

The design conditions considered for this study were established using a combination of elevated water levels (including future sea level rise) and nearshore waves to assess the scour levels at the coastal structure, wave overtopping and wave loads under direct wave impact.

Newport Beach is characterised by moderate to high energy wave climate (typically offshore generated wave swell) with some protection offered from swell waves from the south by Newport Reef (Little Reef, offshore of Bungan Head). Nearshore wave heights beyond the surf zone are typically 80 to 90% of those at a fully exposed open ocean beach (Mariani and Coghlan 2012).

Table 3 provides the offshore design conditions used for this study, with extreme water levels derived from MHL (2018) with appropriate SLR for each considered planning period (but not wave setup) and offshore design wave conditions derived from (Shand et al., 2010).

Table 3: Design offshore wave conditions

ARI	Planning Period	WL (m AHD)	Hs (m)	Tp (s)
100	Present Day	1.44	8.23	13.02
100	2050	1.69(1)	8.23	13.02
100	2080	1.88(2)	8.23	13.02
500	Present Day	1.52	9.33	13.60
500	2050	1.77(1)	9.33	13.60
500	2080	1.96(2)	9.33	13.60
1000	Present Day	1.55	9.79	13.84
1000	2050	1.80(1)	9.79	13.84
1000	2080	1.99(2)	9.79	13.84
2000	Present Day	1.58	10.26	14.06
2000	2050	1.83(1)	10.26	14.06
2000	2080	2.02(2)	10.26	14.06

Notes

(1) SLR was set as 0.26 m for 2050

(2) SLR was set as 0.44 m for 2080 as per Horton (2020a)

3. Estimation of likely range of sand level (scour) at toe of wall

3.1 Measured data

Available measured profiles from the NSW Beach Profile Database (http://www.nswbpd.wrl.unsw.edu.au/photogrammetry/nsw/) are shown in Figure 1. The most eroded profile was 1974, which was collected on 19/06/1974. The renowned 1974 storms were actually a sequence of storms, with the largest being 25 to 29 May 1974 and 3 to 15 June 1974 (an exceptionally long duration), Foster et al, (1975). Rock rubble was placed seaward of the SLSC building in response to these storms, so the profile may have been more eroded at some point during the storm than on 19 June 1974.

Analysis of measured data indicates the following maximum change above AHD:

- 1970 to 1974: 100 m^3/m
- 2011 to 1974: 120 m^3/m

Away from the SLSC building, measured erosion volumes from 1970 to 1974 were assessed to be ranging from 100 to 170 m^3/m.

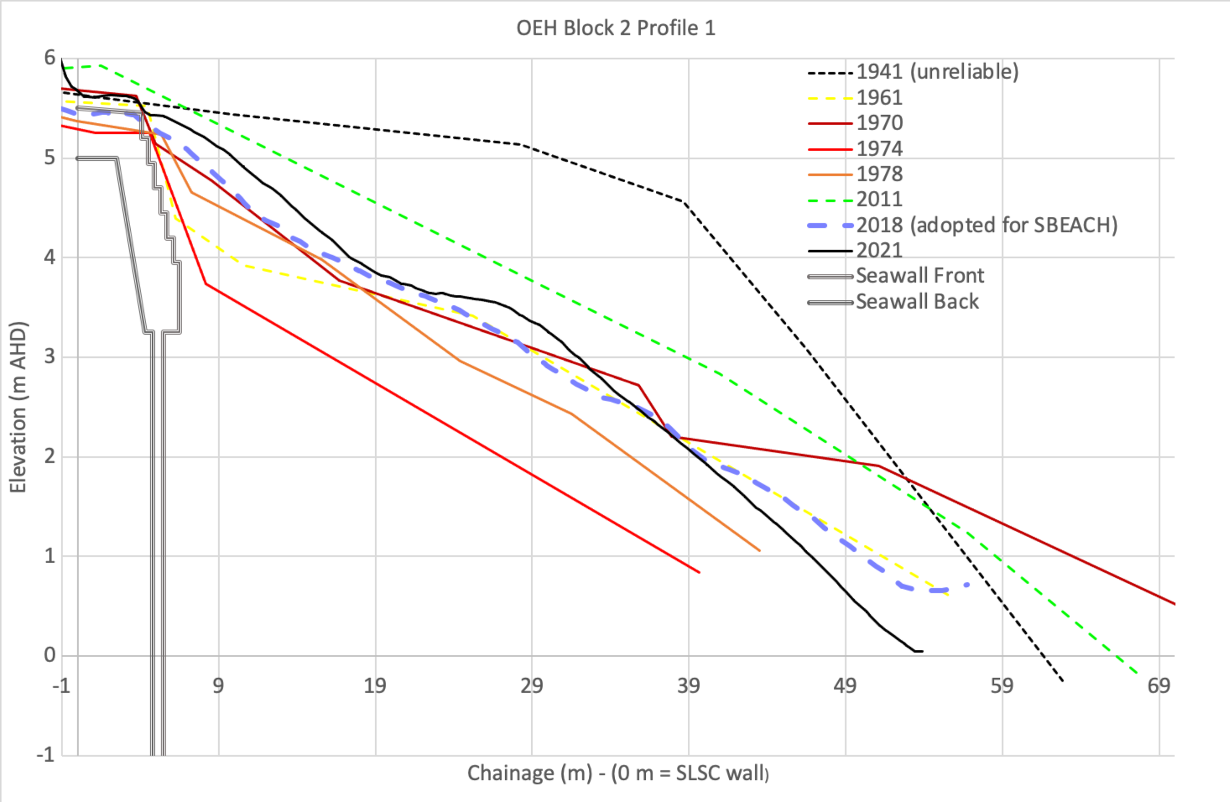


Figure 1: Measured profile data with proposed seawall superimposed

The storm erosion is lower than for highly exposed beaches, but similar to "low demand open beaches" in Gordon (1987). The low demand may be due to:

- Protection by Newport Reef from large southerly waves
- Underlying offshore reefs
- Rock protection fronting the SLSC building

As such, the estimated storm demand for a 100 year ARI design event was assessed to be around 170 m^3/m.

Analysis of photogrammetric and LiDAR data from 1941 to 2021 for long term change indicates that there is no detectable recession trend. That is, Newport Beach has been broadly stable even with sea level rise of 1 to 2 mm per year. Neither the Horton reports nor this WRL advice are a detailed processes study, but an onshore or alongshore feed of sand has been postulated at other locations, noting that sea level rise may outpace this feed in the future. As such, zero long term recession (excluding that caused by future sea level rise) due to net sediment loss was adopted by WRL for this assessment.

Recession due to sea level rise was assumed to be 7 m by 2050 and 13 m by 2080 using a Bruun Factor of 31 (as per Horton, 2020a).

3.2 Modelling of erosion

WRL set up a two-dimensional numerical beach erosion model using SBEACH (Larson, Kraus and Byrnes 1990) to predict scour levels for an agreed range of ARI events (e.g. 100, 500, 1000, 2000 year) at the toe of the proposed buried seawall for present day and future planning horizons using the methodology detailed in Carley et al. (2015). SBEACH considers sand grain size, the pre-storm beach profile and dune height, plus time series of wave height, wave period and water level in calculating a post-storm beach profile.

Time series of consecutive, synthetic storm events (Shand et al. 2011) were applied in SBEACH without a structure in place such that the modelled change in dune volume for a 100 year ARI sequence of storms approximated the observed storm demand in May-June 1974. Example time series for the 500 year ARI event, which was used for assessment of scour levels in more extreme design event, is shown in Figure 2.

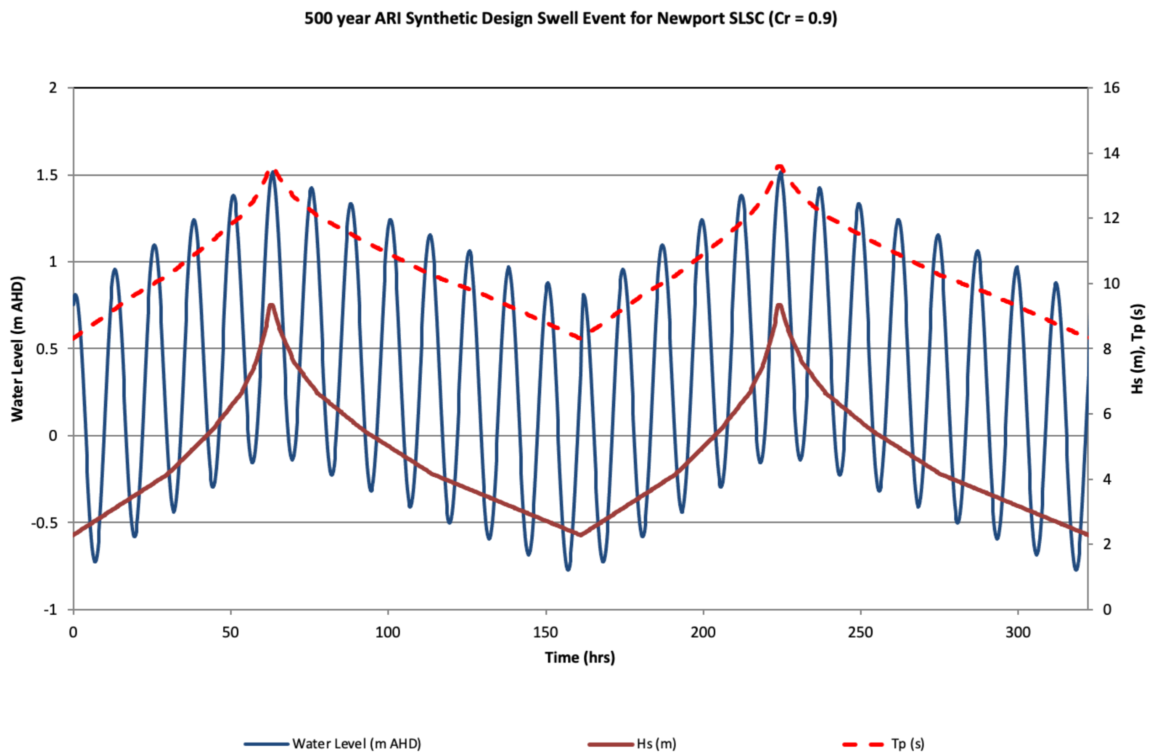


Figure 2: 500 year ARI synthetic design swell time series for Newport Beach (Note that only 2 consecutive storms were used for the study – i.e. erosion volumes derived after 322 hours)

Modelling indicated that the change in dune volume for each storm becomes asymptotic as the profiles approached a dissipative equilibrium (Table 4). Good agreement (within 20 m^3/m) was found between the modelled storm demand for two sequential 100 year ARI storms (190 m^3/m) and that determined from photogrammetric analysis (170 m^3/m). This approach is considered to model similar erosion volumes as those recorded during the most erosive period of the historical storm sequence for which accurate measurements exist; three weeks during May-June 1974. On this basis, the erosion modelled from two sequential storms for each design event (100, 500, 1000 and 2000 year ARI) was adopted to determine the scour level at the proposed seawall.

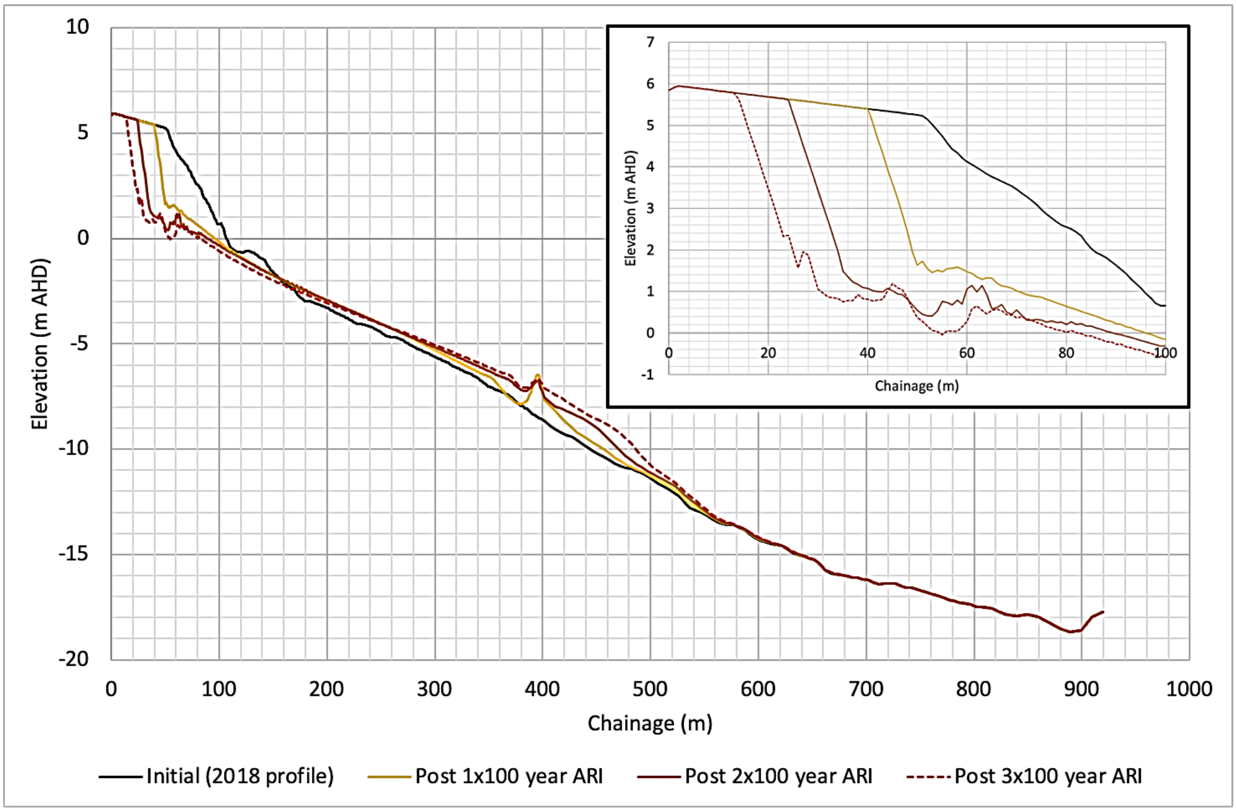


Figure 3: Evolution of beach profiles for consecutive storms in SBEACH with no seawall in place

Table 4: Change in dune volume for three design consecutive storms (no seawall in place)

No. of Storms in Sequence	**(1) Change in Dune Volume** (m3/m above 0 m AHD)	
	Per Storm	Cumulative
Initial	0	0
1x100 year ARI	110	110
2x100 year ARI	80	190
3x100 year ARI	50	240

The proposed structure was then introduced to the model such that erosion of the dune is prevented. The time series of storm events (which resulted in the adopted storm demand without a structure in place) was used in SBEACH with the buried seawall in place to estimate the scour level at the toe. The same methodology was repeated for higher ARI events (500, 1000 and 200 year ARI) to estimate scour levels for future planning horizons incorporating underlying and sea level rise recession rates.

Figure 4 presents estimates of the scour depth at the toe of the proposed seawall at Newport Beach for the range of considered environmental conditions. Based on the SBEACH modelling, scour levels between -0.5 m AHD and -1 m AHD can be expected to occur in front of the proposed seawall, which is in agreement with historical scour levels and observed scour levels during major storms in front of existing permeable and non-permeable seawalls along the NSW coast (Nielsen et al. 1992; Foster et al. 1975).

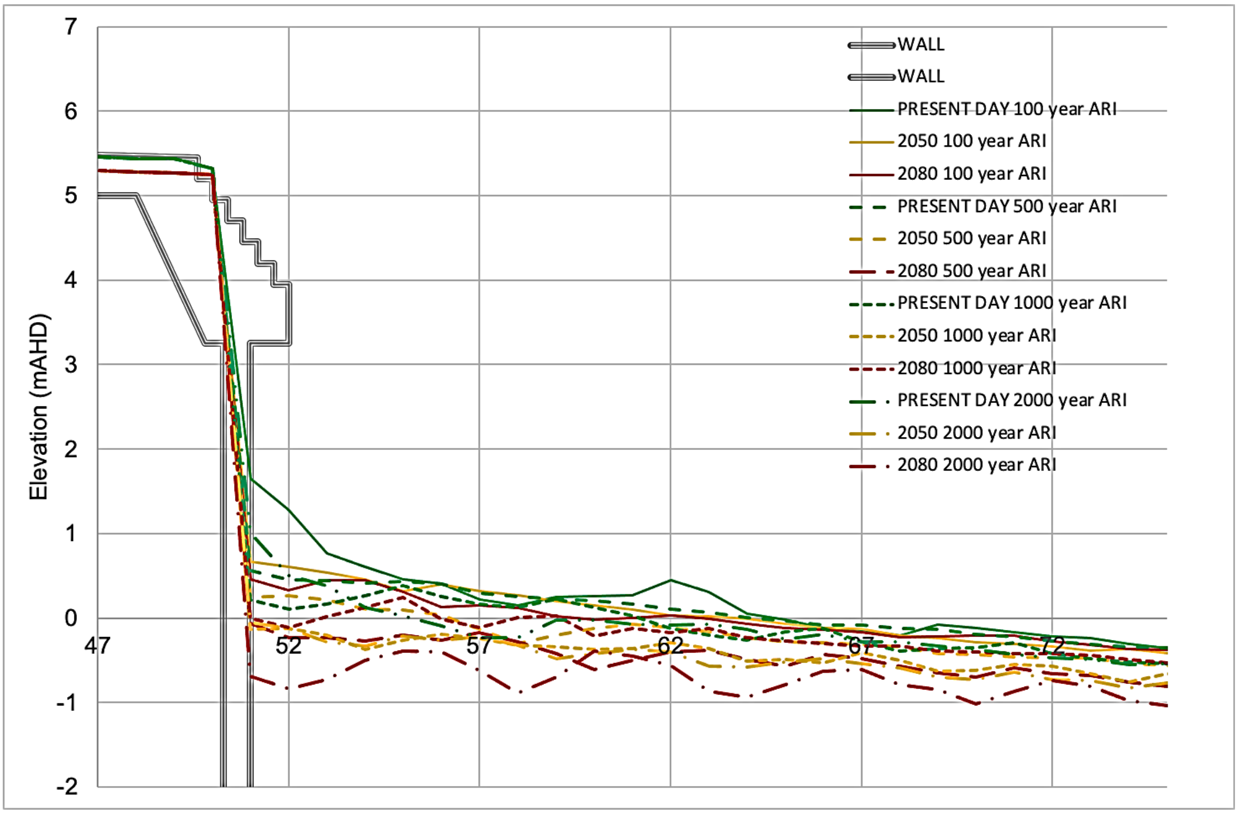


Figure 4: Evolution of beach profiles for consecutive storms in SBEACH with no seawall in place

A summary of indicative scoured seabed levels directly in front of the proposed seawall and one plunge length away from wall (i.e. 10 m distance offshore) is provided in Table 5. Minor adjustments were made in some cases to the calculated scoured seabed level values in SBEACH to remove modelling artefacts (i.e. seabed undulations) when scoured seabed levels at the wall were deeper than further offshore.

Table 5: Calculated seabed scoured levels at wall and one plunge length offshore

ARI	Planning Period	WL (mAHD)	Hs (m)	Tp (s)	Scoured bed levels (m AHD) In front of wall	Scoured bed levels (m AHD) 10 m in front of wall
100	Present Day	1.44	8.23	13.02	1.6	0.3
100	2050	1.69	8.23	13.02	0.7	0.1
100	2080	1.88	8.23	13.02	0.5	0.0
500	Present Day	1.515	9.33	13.60	0.6	0.2
500	2050	1.77	9.33	13.60	0.2	-0.1
500	2080	1.96	9.33	13.60	-0.1	-0.5
1000	Present Day	1.545	9.79	13.84	0.2	0.0
1000	2050	1.80	9.79	13.84	-0.1	-0.4
1000	2080	1.99	9.79	13.84	0.0	-0.1
2000	Present Day	1.575	10.26	14.06	-0.1(1)	-0.1
2000	2050	1.83	10.26	14.06	-0.1	-0.4
2000	2080	2.02	10.26	14.06	-0.7	-0.7(1)

Note: (1) adjusted scoured seabed level to remove modelling artefact

4. Estimation of wave runup and overtopping

4.1 Overview

WRL used a combination of empirical techniques to estimate wave runup and overtopping of the proposed buried seawall. Wave setup was calculated using the one dimensional surf zone model for wave setup developed for erosion modelling above. The state-of-the-art empirical technique for estimating overtopping is the EurOtop (2018) "Overtopping Manual". WRL have compared predictions of overtopping determined using the methods set out in the manual with several coastal structures physically modelled in wave flumes, and found that in general, the Overtopping Manual provides reasonable predictions (Mariani et al., 2009).

The results presented below are best practice desktop calculations, however, if the results are deemed to be critical, EurOtop (2018) recommends site specific physical modelling which could be undertaken at a later stage.

The Overtopping Manual provides equations for runup and overtopping calculations on structures such as the one considered at Newport SLSC. This method was used to estimate theoretical runup levels and average overtopping rates for a range of pre-agreed design conditions (i.e. 100, 500 and 2000 years) and for different eroded states of the beach.

Overtopping was quantified in terms of the volume of water being discharged over the seawall crest and expressed in L/s per metre length of crest. Wave overtopping volume was estimated taking into account the following factors:

- Structural characteristics of the seawall (crest height, return wall)
- Design scour levels for the seawall or the accreted beach
- Wave conditions at the structure i.e. wave height and period one plunge length (i.e. 10 m) from the toe of the considered structure
- Elevated water level incorporating tides, storm surge and wave setup for the different planning periods considered

The calculated overtopping values can be compared to available overtopping guidelines regarding hazard levels to people and infrastructure (EurOtop, 2007; CIRIA, 2007) presented in Table 6.

Table 6: Limits for tolerable mean wave overtopping discharge (EurOtop, 2007)

Hazard Type	Mean Overtopping Discharge Limit (L/s per m)
Aware pedestrian and/or trained staff expecting to get wet	**0.1 (pedestrian) to 1-10 (staff)**
Damage to grassed promenade behind seawall	**50**
Damage to paved promenade behind seawall	**200**
Structural damage to seawall crest	**200**
Structural damage to building	**1[(1)]**

Note: (1) this limit related to effective overtopping defined at the building

4.2 *Accreted or average beach runup*

For the case of an accreted or average beach (Figure 5), the wave return protrusion (Figure 5) may remain buried beneath the sand. In this case wave runup can be estimated using methods such as Mase (1989) and Nielsen (1991).

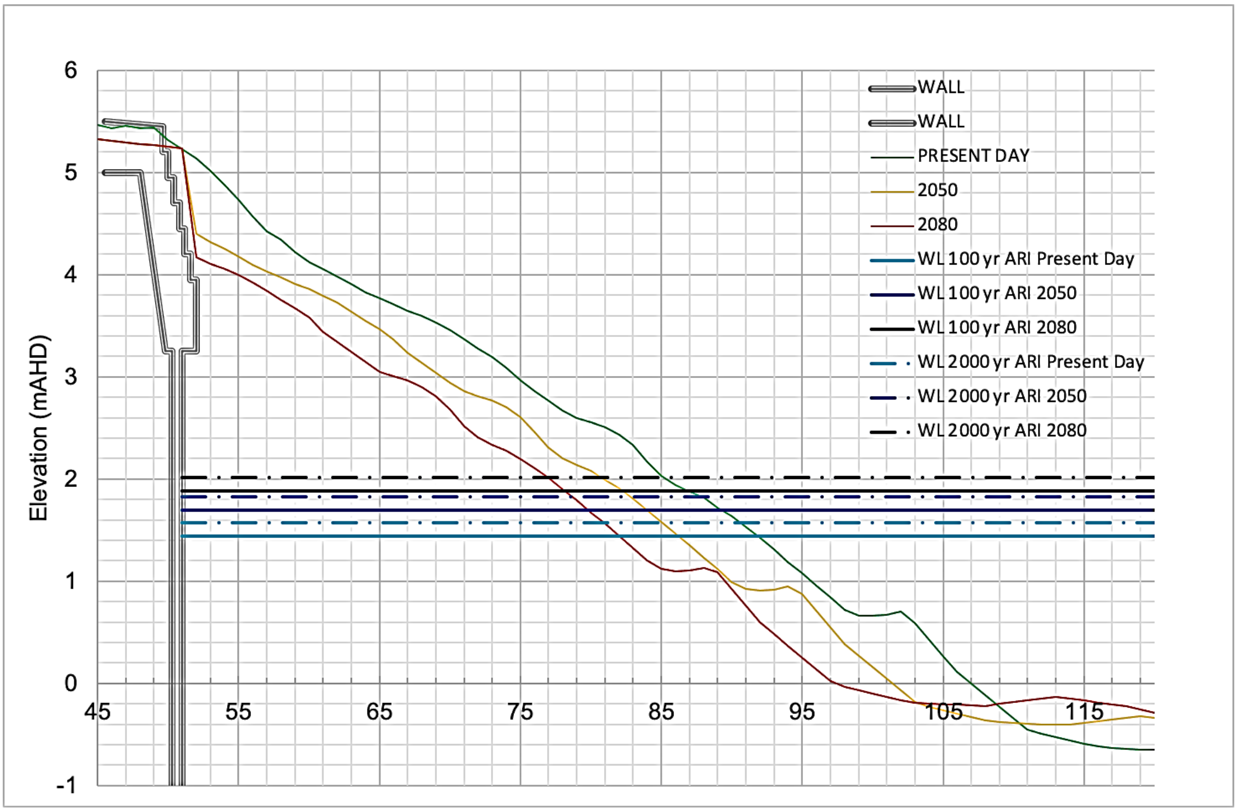


Figure 5: Water levels (no wave setup) for 100 and 2000 year ARI events for present day, 2050 and 2080 planning period

The only calibration case available for wave runup at Newport is based on surveys of debris lines undertaken by WRL (Higgs and Nittim, 1988) at a series of northern beaches following the August 1986 storm (Figure 6).

This storm had the following peak characteristics:

- Peak significant wave height Hs=7.5 m
- Associated peak wave period Tp=13.2 s
- Storm Direction SE
- Maximum water level (excluding wave setup) 1.0 m AHD

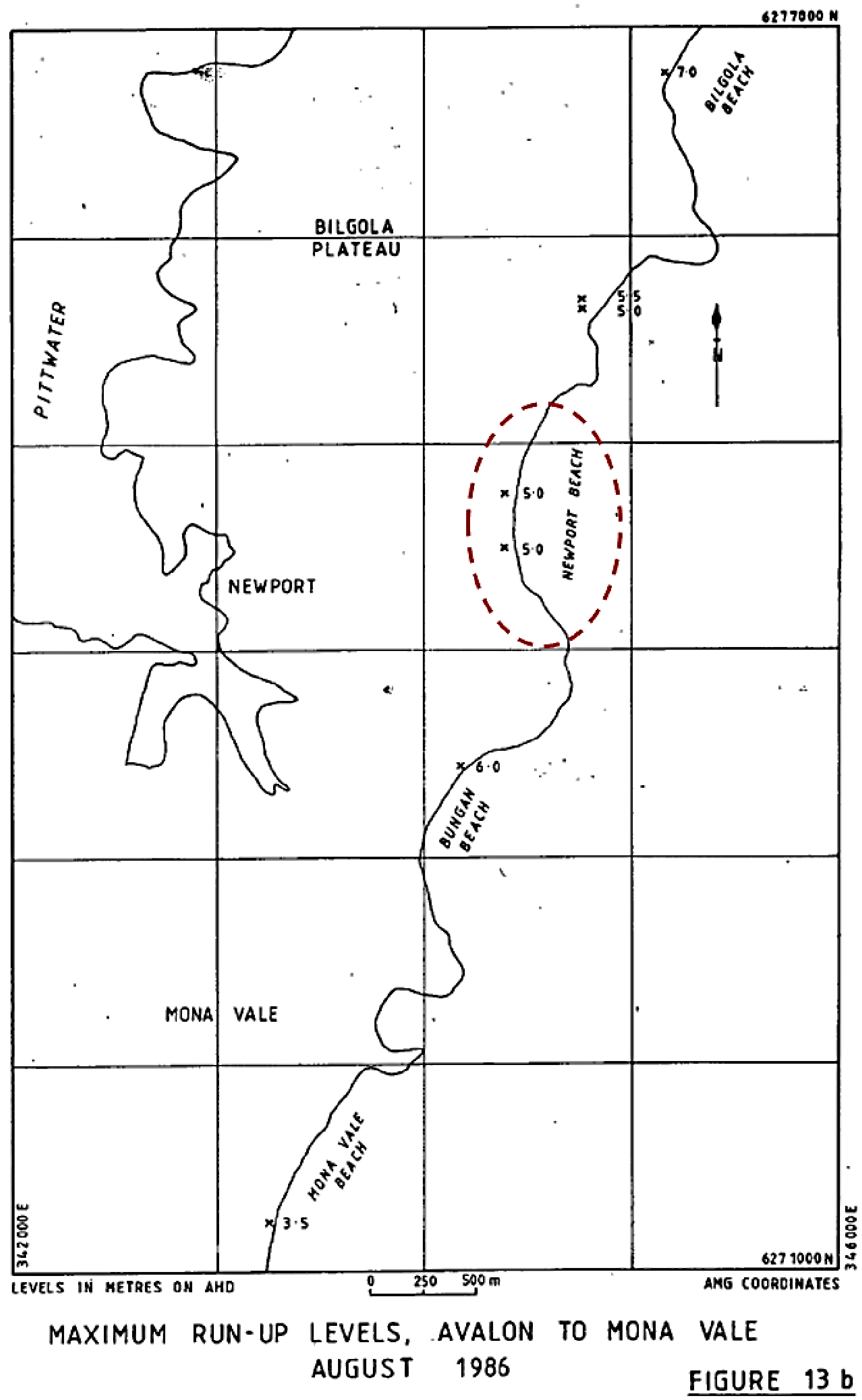


Figure 6: Observed wave runup levels after August 1986 storm based on debris lines [Source: Higgs and Nittim, 1988]

The following comparison is made of measured runup and calculated runup, using the method of Mase (1989), for the August 1986 event:

- Observed debris line by Higgs and Nittim (1988) : 5.0 m AHD
- Calculated R_{max} using the method Mase (1989): 5.3 m AHD
- Calculated $R_{2\%}$ using the method Mase (1989): 4.8 m AHD

The observed debris line approximates maximum wave runup (R_{max}) of the 1986 storm, which shows that the method of Mase (1989) is appropriate to estimate wave runup at Newport Beach.

Calculated wave runup values ($R_{2\%}$) for a range of conditions with an accreted beach are shown in Table 7. $R_{2\%}$ levels are typically used to describe wave runup in coastal engineering and represent the wave runup water level that is exceeded by 2% of incident waves.

These values of wave runup provide estimates of water levels that can be expected to reach the top of the proposed seawall which is currently proposed to have a maximum crest level of +5.5 m AHD (similar to the ground levels of the promenade fronting the Newport SLSC building).

Calculated wave runup levels exceed the proposed crest level of 5.5 m AHD indicating the potential for wave overtopping to occur on the promenade during storm events of 100 year ARI and larger.

Estimates of overtopping discharges over the crest of the proposed seawall and across the promenade were calculated using a range of methods described in EurOtop (2018) given the possibility of the buried seawall to be partially exposed, and wave runup occurring over either a sandy foreshore or concrete steps. Given the complexity of the site, available methods are suitable as order of magnitude estimates or for relative comparison purposes.

Table 7: Wave runup levels and overtopping discharges for accreted beach

					Nielsen, 1991	Mase, 1989	EurOtop (2018)
ARI	**Planning Period**	**WL (m AHD)**	**Hs (m)**	**Tp (m)**	**Runup 2% (m AHD)**	**Runup 2% (m AHD)**	**Overtopping discharge (L/s)**
100	Present Day	1.44	8.23	13.02	6.11	6.71	**[1.4 - 5.1]**
100	2050	1.69	8.23	13.02	6.36	6.93	**[4.1 - 13.3]**
100	2080	1.88	8.23	13.02	6.55	7.15	**[7.3 - 23.4]**
500	Present Day	1.52	9.33	13.60	6.71	7.30	**[4.6 - 15.4]**
500	2050	1.77	9.33	13.60	6.96	7.51	**[10.5 - 34]**
500	2080	1.96	9.33	13.60	7.15	7.70	**[17.2 - 54.7]**
1000	Present Day	1.55	9.79	13.84	6.96	7.43	**[6.1 - 21.8]**
1000	2050	1.80	9.79	13.84	7.21	7.70	**[14.1 - 45.8]**
1000	2080	1.99	9.79	13.84	7.40	7.88	**[22.6 - 71.7]**
2000	Present Day	1.58	10.26	14.06	7.20	7.72	**[7.7 - 30.1]**
2000	2050	1.83	10.26	14.06	7.45	7.93	**[17.4 - 59.2]**
2000	2080	2.02	10.26	14.06	7.64	8.12	**[29.3 - 92.4]**

4.3 Wave runup and overtopping for eroded beach

When the beach is eroded, the cantilever of the proposed stairs on the seawall can act as a wave return wall. A range of scoured seabed levels and nearshore water levels including wave setup are shown in Figure 7.

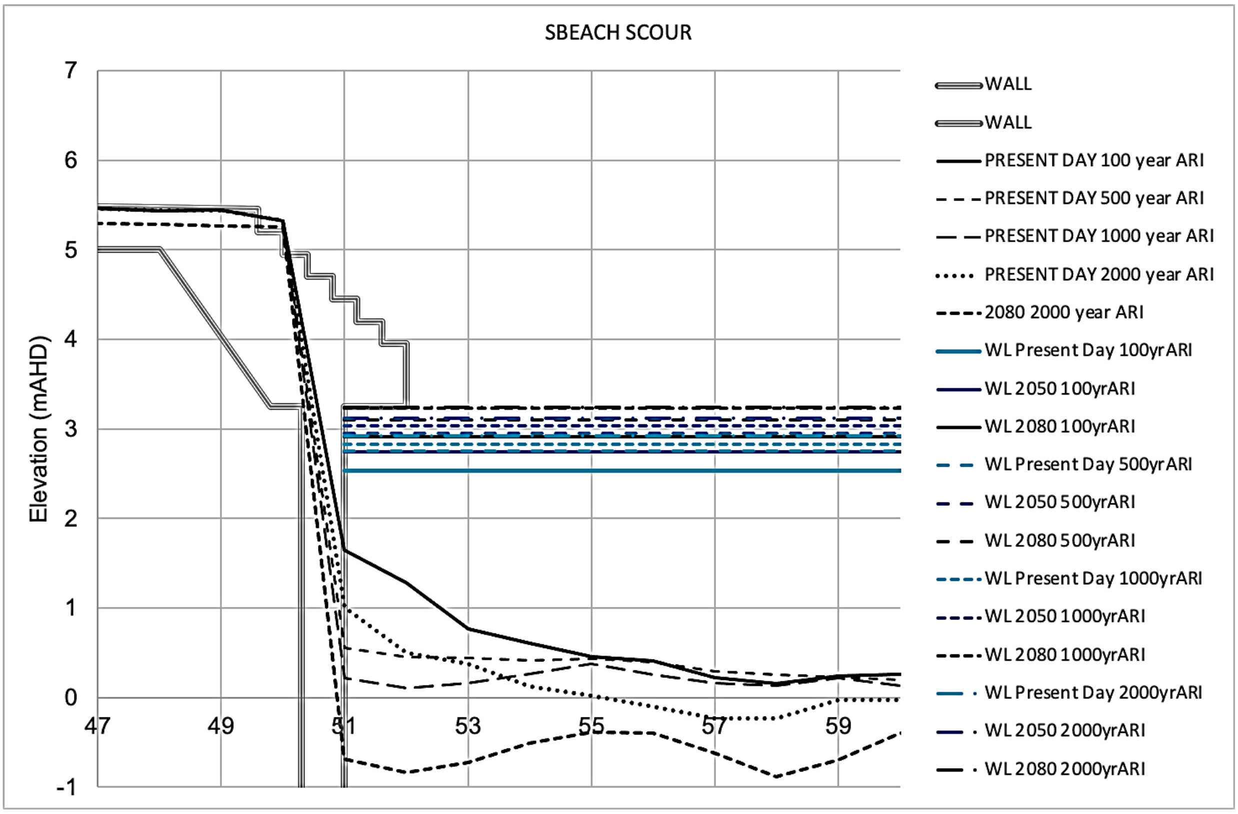


Figure 7: Calculated nearshore water levels (including local wave setup) and scoured levels in front of proposed seawall

Wave overtopping on vertical walls can vary greatly depending on the type of waves reaching the seawall. Based on the range of estimated scoured seabed levels and water levels with local wave setup, it is expected that plunging waves will reach the proposed seawall resulting in impulsive wave conditions. Overtopping discharges under these conditions can typically be characterised by a violent up rushing jet of aerated water.

It is anticipated that the return wall at the bottom of the steps will reduce overtopping uprush for lower water levels. However, based on the estimated design water levels with wave setup, this return wall may be submerged at higher water levels and bigger waves, reducing its effectiveness on limiting wave overtopping.

The geometric parameters for overtopping of seawalls with a wave return wall are shown in Figure 8.

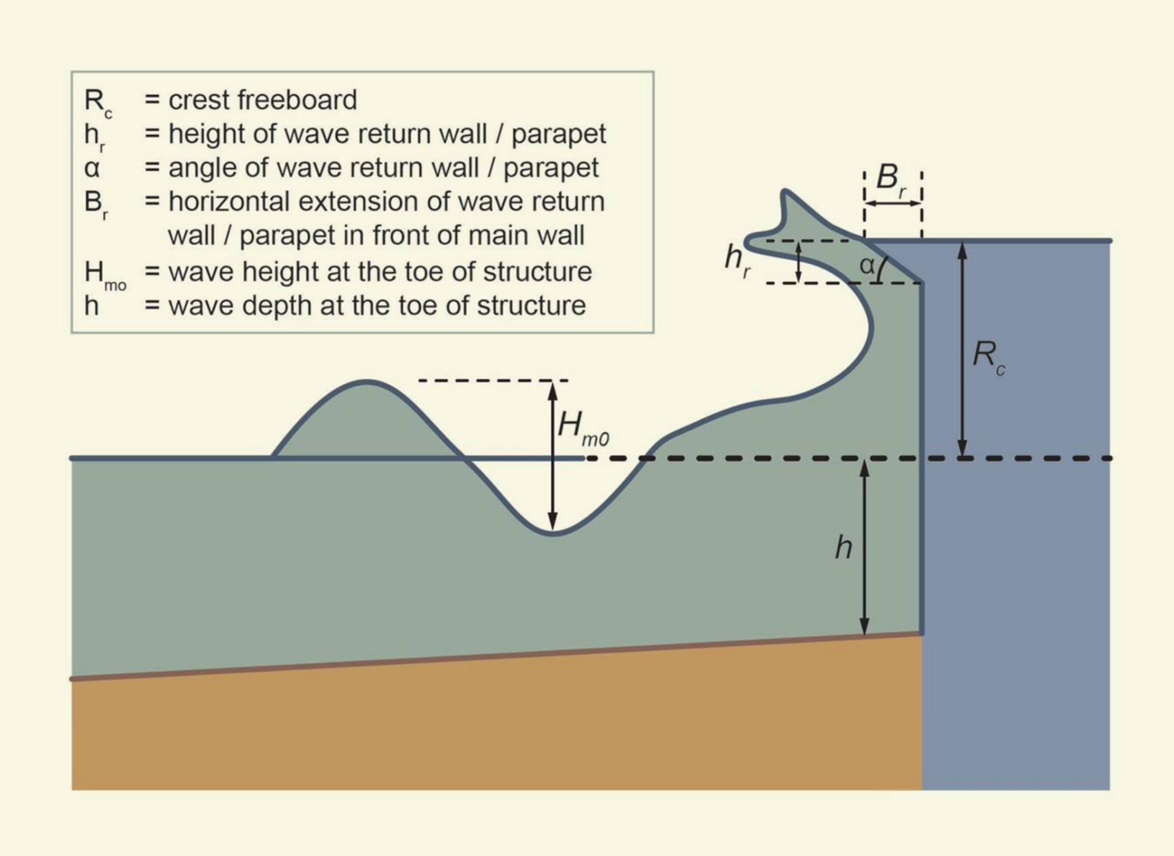


Figure 8: Parameters definitions for vertical seawall with return wall [Source: EurOtop, 2018]

Calculated overtopping discharge rates for a range of conditions for a scoured beach and exposed seawall are shown in Table 8.

Table 8: Overtopping discharges for proposed seawall with return wall

ARI	Planning Period	WL (mAHD)	Hm0 (m)	Tm-1,0 (s)	Design OT for vertical with return wall (L/s/m)
100	Present Day	1.44	1.48	11.83	**0.38**
100	2050	1.69	1.72	11.83	**5.87**
100	2080	1.88	1.89	11.83	**13.31**
500	Present Day	1.515	1.69	12.37	**4.00**
500	2050	1.77	2.05	12.37	**17.94**
500	2080	1.96	2.27	12.37	**37.36**
1000	Present Day	1.545	1.83	12.58	**7.02**
1000	2050	1.80	2.20	12.58	**27.42**
1000	2080	1.99	2.18	12.58	**34.09**
2000	Present Day	1.575	1.95	12.78	**11.12**
2000	2050	1.83	2.26	12.78	**33.27**
2000	2080	2.02	2.39	12.78	**54.07**
2000(1)	2080	2.02	2.66	12.78	**84.31**

Note:(1) This additional condition considered a highly eroded seabed (-1 m AHD)

5. Wave loads due to overtopping

5.1 Overview

Based on the results of the wave runup calculations, loads on the Newport SLSC building were estimated. Wave forces on the seaward face of the surf club would consist of a hydrostatic component from water pressure, and a dynamic component due to horizontal wave velocity.

A combination of empirical techniques were applied depending on the nature of the conditions generating the loading, namely:

- Impact caused by wave runup reaching the crest of the buried seawall and creating a bore-like discharge over the top of the wall
- Direct wave impact on the Newport SLSC for events where the seawall is completely submerged due to elevated water levels

Physical model testing is the most reliable method to calculate wave forces, particularly with the complex ancillary structures present, and is strongly recommended for this project at the detailed design stage if the present geometry is to be used.

5.2 Wave loads caused by wave runup (partially eroded beach)

Wave loads on the Newport SLSC caused by wave runup reaching the crest of the buried (or partially exposed) proposed seawall and creating bore-like discharges were estimated using a combination of the following methods to best estimate the overtopping processes:

1. Use the wave runup values obtained at the crest of the proposed seawall and estimate the associated depth of water at the Newport SLSC front wall (i.e. 5 m from the seawall crest edge) using the FEMA (2005) recommended method of Cox and Machemehl (1986) (Figure 9).

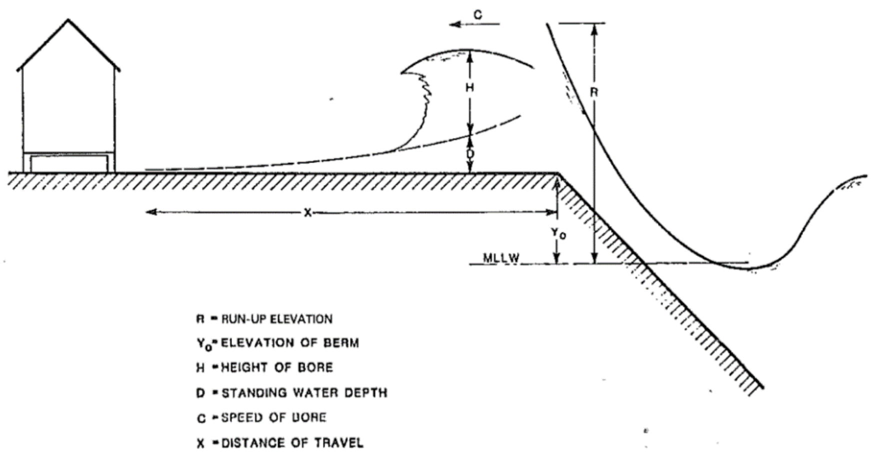


Figure 9: Definition of overtopping parameters [Source: Cox and Machemehl, 1986]

2. Calculate velocities for the overtopping flow reaching the Newport SLSC front wall by applying a decay of flow velocity long the crest and promenade using EurOtop (2018) (Figure 10).

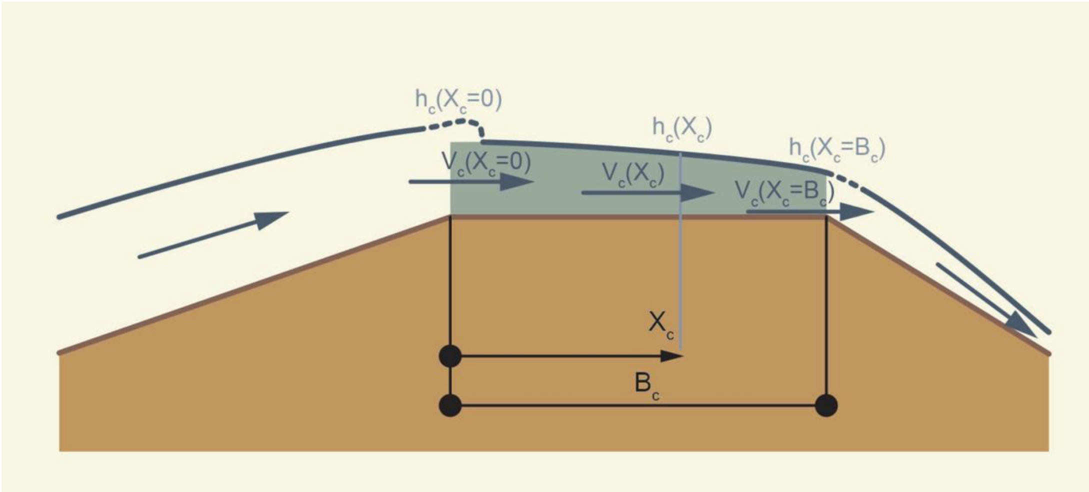


Figure 10: Sketch of overtopping flow parameters [Source: EurOtop, 2008]

3. Calculate wave loads on the Newport SLSC front wall, consisting of a hydrostatic component from water pressure, and a hydrodynamic component due to horizontal bore velocity. The main method used to calculate wave forces was derived from FEMA (2011) "Coastal Construction Manual" (Figure 11).

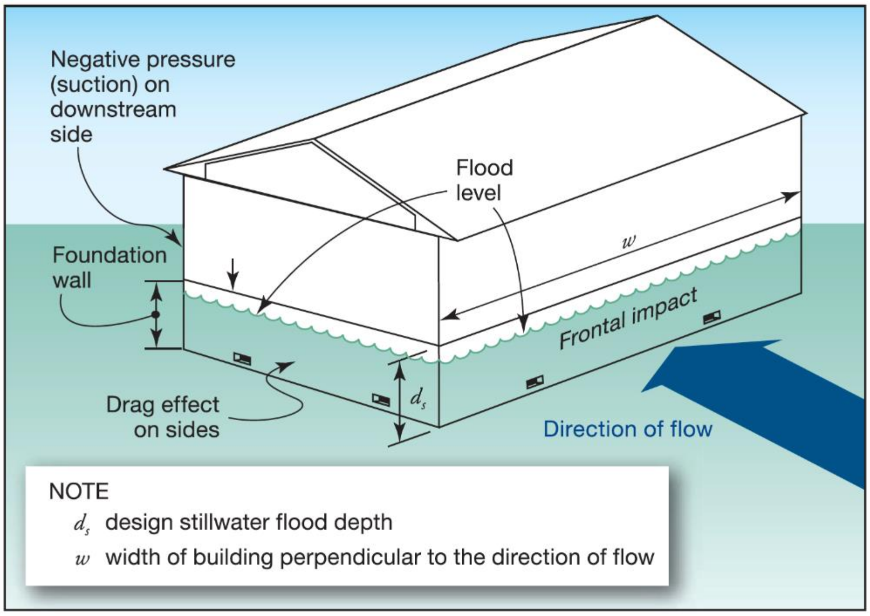


Figure 11: Hydrodynamic loads on a building [Source: FEMA, 2011]

The forces on the Newport SLSC building due to wave runup were estimated for both $R_{2\%}$ and R_{max} water levels, to provide a range of potential impact loads. The loads associated with $R_{2\%}$ runup could be expected to be experienced a small number of times by the building during the storm while the loads associated with Rmax runup represent the maximum that is expected to occur during the considered design event.

It should be noted that the duration for which the hydrodynamic component of the load is typically expected to last is around one wave period (i.e. around 10 to 15 s) before reducing when overtopping would dissipate between waves.

Table 9: Loads on Newport SLSC front wall caused by wave runup

ARI	**Planning Period**	**WL (m AHD)**	**R2% (m AHD)**	**Depth of R2% at SLSC (m)**	**Rmax (m AHD**	**Depth of Rmax at SLSC (m)**	**Total Load R2% (kN/m)**	**Total Load Rmax (kN/m)**
100	Present Day	1.44	6.41	0.08	7.62	0.61	**1.3**	**39**
100	2050	1.69	6.64	0.16	7.88	0.76	**2.6**	**51**
100	2080	1.88	6.85	0.24	8.12	0.89	**4.3**	**63**
500	Present Day	1.52	7.00	0.34	8.32	1.01	**6.3**	**74**
500	2050	1.77	7.23	0.45	8.58	1.17	**9.0**	**90**
500	2080	1.96	7.42	0.55	8.80	1.30	**11.5**	**103**
1000	Present Day	1.55	7.19	0.44	8.55	1.15	**8.8**	**87**
1000	2050	1.80	7.45	0.58	8.84	1.33	**12.3**	**106**
1000	2080	1.99	7.64	0.69	9.05	1.46	**15.3**	**121**
2000	Present Day	1.58	7.46	0.69	8.86	1.34	**15.0**	**108**
2000	2050	1.83	7.69	0.73	9.12	1.51	**16.6**	**126**
2000	2080	2.02	7.88	0.84	9.34	1.65	**20.0**	**142**

5.3 Wave loads caused by wave impact on exposed vertical seawall (scoured beach levels)

Wave loads on the Newport SLSC building caused by direct wave impact for events where the seawall is completely submerged due to highly-elevated water levels were estimated using the method by Goda and Tanimoto as recommended by USACE CEM (2011) for impulsive wave loading.

The wave loads on the Newport SLSC were considered using the simplification that the SLSC front wall was aligned with the crest of the proposed concrete seawall as no available desktop technique allows consideration of the offset of the building from the edge of the coastal protection structure.

It should also be noted that available desktop techniques do not capture the potential reduction associated with the wave return wall on the wave impacting the Newport SLSC building.

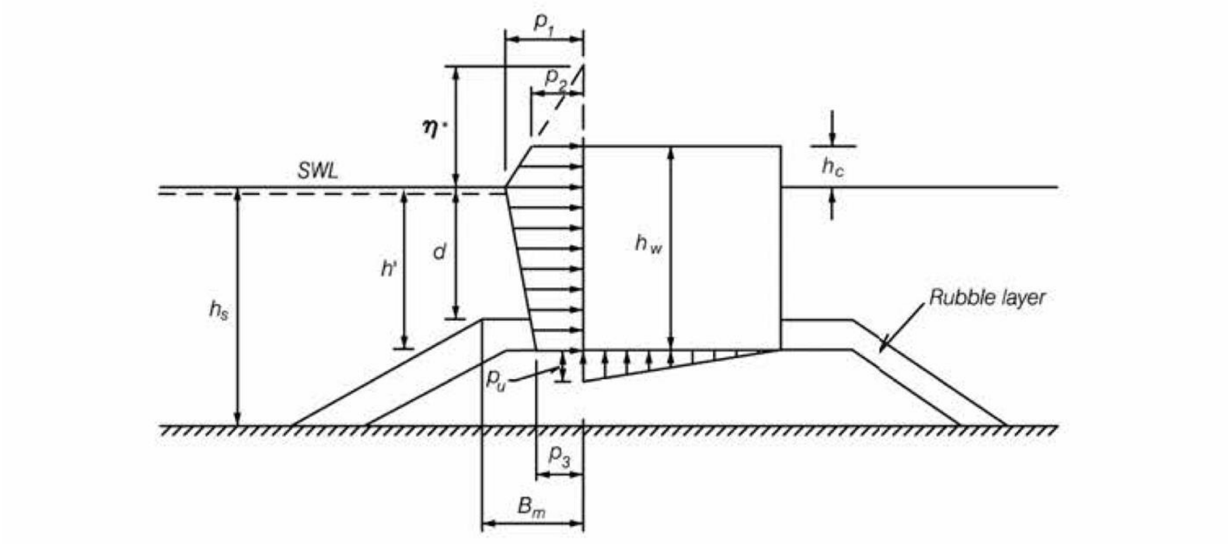


Figure 12: Hydrodynamic loads due to wave impact on a coastal structure [Source: CEM, 2011]

The calculated loads on the Newport SLSC due to direct wave impact are presented in Table 10.

Table 10: Loads on Newport SLSC front wall caused by direct wave impact

ARI	Planning Period	WL (mAHD)	H design at toe (m)	Tm-1,0 (s)	Induced Horizontal Load FH (kN/m)	Hydrostatic Load FH (kN/m)	Total Load (kN/m)
100	Present Day	1.44	1.77	11.83	0.0	0.0	**<1.0**
100	2050	1.69	2.06	11.83	0.6	0.6	**<2.0**
100	2080	1.88	2.27	11.83	3.7	3.4	**7.0**
500	Present Day	1.515	2.02	12.37	0.5	0.4	**<1.0**
500	2050	1.77	2.46	12.37	7.2	6.5	**13.7**
500	2080	1.96	2.73	12.37	15.7	14.4	**30.1**
1000	Present Day	1.545	2.18	12.58	2.0	1.8	**3.8**
1000	2050	1.80	2.65	12.58	12.5	11.4	**23.9**
1000	2080	1.99	2.62	12.58	15.1	13.8	**28.9**
2000	Present Day	1.575	2.33	12.78	4.7	4.3	**9.0**
2000	2050	1.83	2.72	12.78	15.8	14.4	**30.3**
2000	2080	2.02	2.88	12.78	23.1	21.1	**44.3**
2000(1)	2080	2.27	3.21	12.78	35.9	32.9	**68.8(1)**

Note (1): This additional condition considered a highly eroded seabed (-1 m AHD)

6. Review of available methods to reduce overtopping hazard

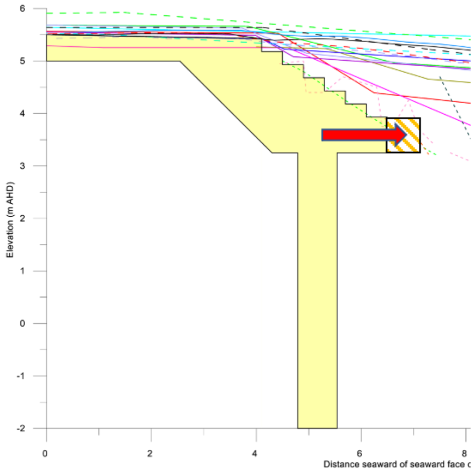
Should the wave overtopping or wave forces be deemed to be excessive, the following methods are available to reduce overtopping (Figure 13):

- Installation of a wider wave return wall
- Installing the wave return wall at a higher elevation
- Install a parapet or wave return wall, noting that:
    - This could be in response to a future sea level rise threshold, or
    - This may only be needed for the frontage of the old SLSC building

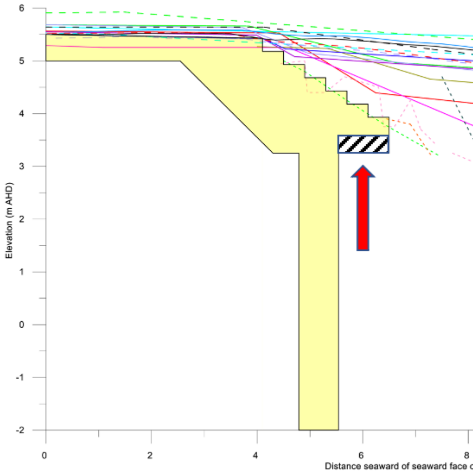
Additionally, the following short term management measures could be undertaken:

- Installation of temporary flood barriers in response to a forecast event
- Management of the interior of the SLSC building, such as design of the electrical system, and short term response to a forecast event

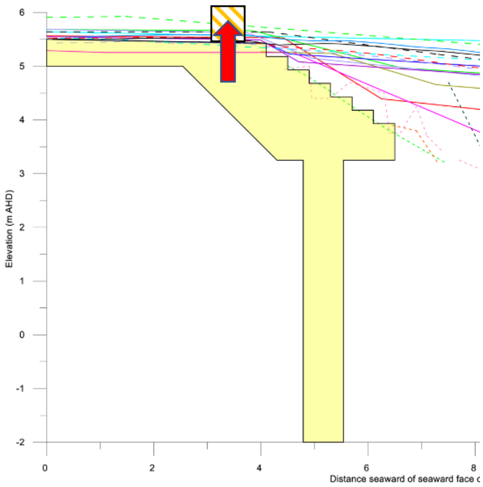
Additional calculations and/or later physical modelling may be required to quantify the benefit of each option.



(a) Extend projection of return wall



(b) Raise level of return wall



(b) Add return wall on crest

Figure 13: Options for reducing wave overtopping

7. Assessment of seawall end effects

The coastal process impact of the proposed works over their design life has been assessed through the impact on a nominal coastal hazard line. An illustration of the theory of seawall end effects is shown in Figure 14.

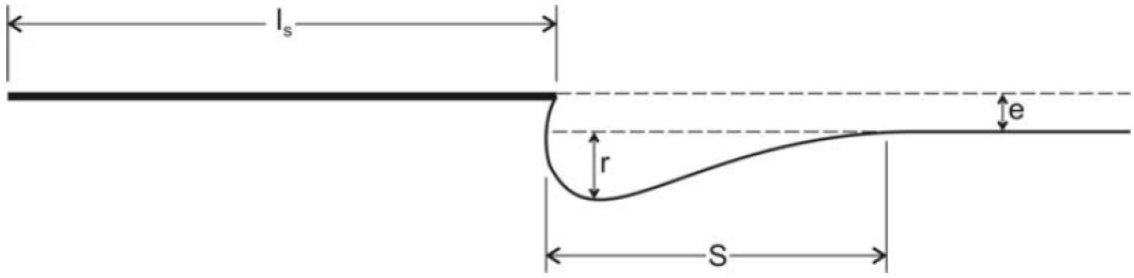


Figure 14: Seawall end effect variables

The assessment for the proposed buried seawall in front of the Newport SLSC has been undertaken using methodologies from McDougal et al (1987), who presented the seawall end effect diagram shown in Figure 14, and Carley et al (2013) based on their review of numerous Australian seawalls.

The classic work presenting seawall end effects is McDougal et al (1987), who presented the seawall end effect diagram shown in Figure 14. No time or storm dependence (i.e. ARI of considered storm event) was provided for the planform depicted, nor any dependence of the end effect on the sand volume seaward of the seawall.

Work by Carley et al (2013) on numerous Australian seawalls found that even for long seawalls, the maximum 'S' was approximately 400 m, while the quantum for 'r' was dependent on whether a seawall was frequently exposed to waves or predominantly buried in sand. They found that within the photogrammetric data, no seawall end effect could be observed for some seawalls not frequently exposed to waves, however, this does not preclude a short term end effect during major erosion events.

For assessment of seawall end effects at Newport, the works of McDougal et al (1987), Carley et al (2013) and Dean (1986) were combined. The generic geometry of McDougal et al (1987) was used, with the excess erosion (r) determined as follows. Using the Dean approximate principle, the volume of sand that is locked up behind the seawall and would otherwise be available to supply storm erosion demand, was offset as a seawall end effect at each end of the seawall.

Management of seawall end effects involves the erosion of parkland and not structural design. Therefore, the seawall end effect assessment was conducted for 100 year ARI conditions (rather than higher ARIs) for the three considered planning periods, with a proposed seawall crest length of 85 m. It was found that no significant seawall end effect will likely be observed under present day conditions up to 100 year ARI, as a sufficient sand buffer will be fronting the seawall. Seawall end effects will be experienced for the 2050 and 2080 planning period when considering the reduction of sand supply fronting the seawall due to recession associated with future SLR.

The results of the seawall end effect assessment are shown for 100 year ARI conditions in Figure 15. It should be noted that overall seawall end effects would be reduced should the overall length of the proposed seawall be reduced, e.g. through protecting the building only, and not extending it to protect surrounding Norfolk Island Pine trees.

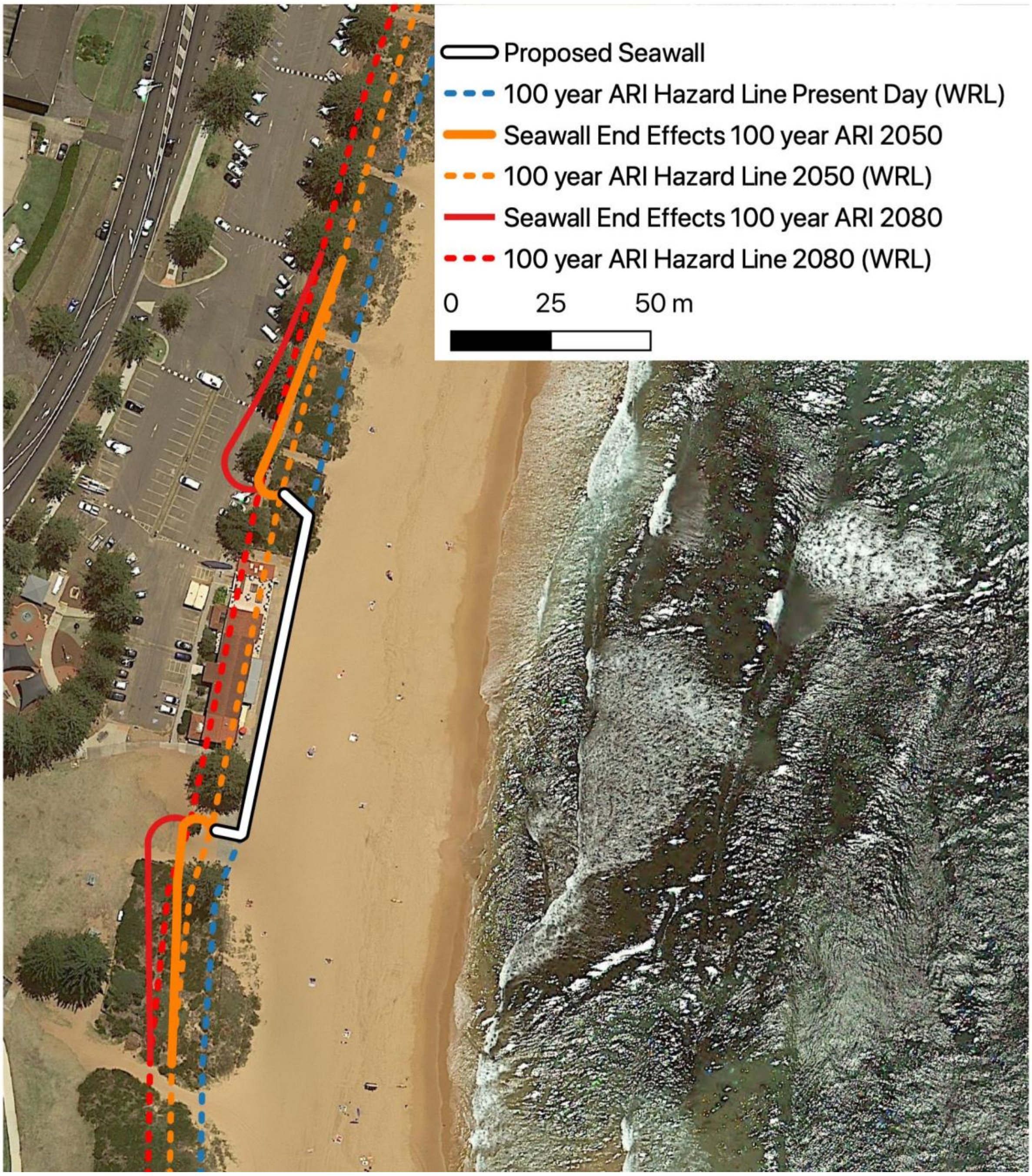


Figure 15: Theoretical seawall end effect for 100 year ARI conditions

8. Summary

As a consequence of WRL's peer review dated 14 May 2021, WRL completed a range of desktop calculations regarding proposed extensions to Newport SLSC. These included:

- Estimating the likely range of sand level (scour) at toe of wall
- Estimating wave runup and overtopping
- Estimating wave loads due to overtopping
- Options to reduce the wave overtopping hazard
- Assessment of seawall end effects
- Liaison with Horton Coastal Engineering

The above parameters were calculated for:

- ARIs of: 100, 500, 1000 and 2000 years
- Planning horizons and sea level rise of: 2021, 2050 (0.3 m SLR), 2080 (0.44 m SLR)

Subject to the input of a structural engineer, the proposed new portion of the SLSC building is likely to be able to withstand the estimated wave forces. Additional input from a structural engineer would be needed to estimate the likely resilience of the existing building.

Additional measures to reduce wave overtopping and wave forces are presented, namely:

- Installation of a wider wave return wall
- Installing the wave return wall at a higher elevation
- Install a parapet or wave return wall, noting that:
    - This could be in response to a future sea level rise threshold, or
    - This may only be needed for the frontage of the old SLSC building

Additionally, the following short term management measures could be undertaken to manage wave overtopping and wave forces:

- Installation of temporary flood barriers in response to a forecast event
- Management of the interior of the SLSC building in response to a forecast event

Best practice coastal engineering desktop techniques appropriate to the scale of the proposal were applied. The reference material relied upon recommends that physical modelling be undertaken for critical decisions. WRL recommends that this be undertaken during the detailed design of the project.

Thank you for the opportunity to provide this information. Please contact James Carley on +61414 385 053 should you require further information.

Yours sincerely,

Grantley Smith
Director, Industry Research

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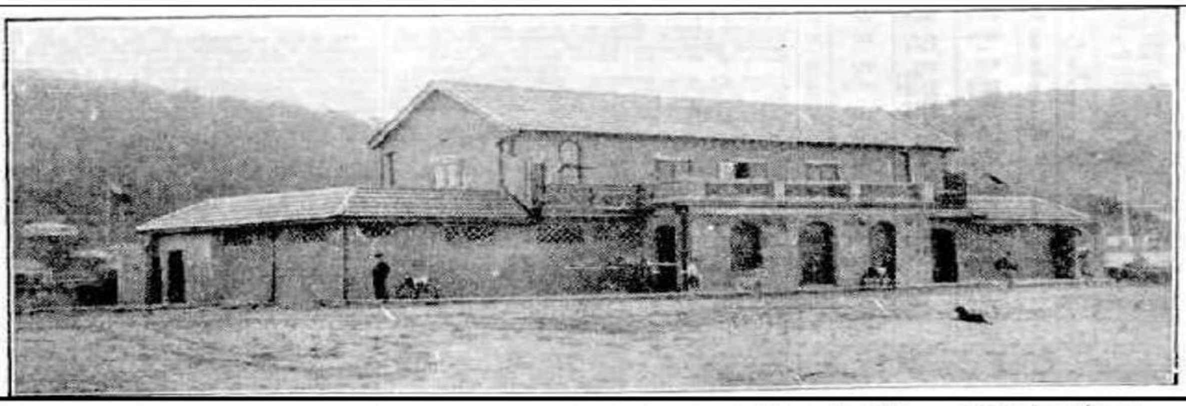
10. Appendix A Historic photos

NEW SURF CLUBHOUSE AT NEWPORT.



Surfers at Newport this season will have the benefit of this commodious building, which has been constructed for the local club.

Figure 16: Newport SLSC 1933

Figure 17: May 1974 (from Horton, 2020a)

Figure 18: 28 May 1974 (from Horton, 2020a)

Figure 19: December 1974 (from Horton, 2020a)