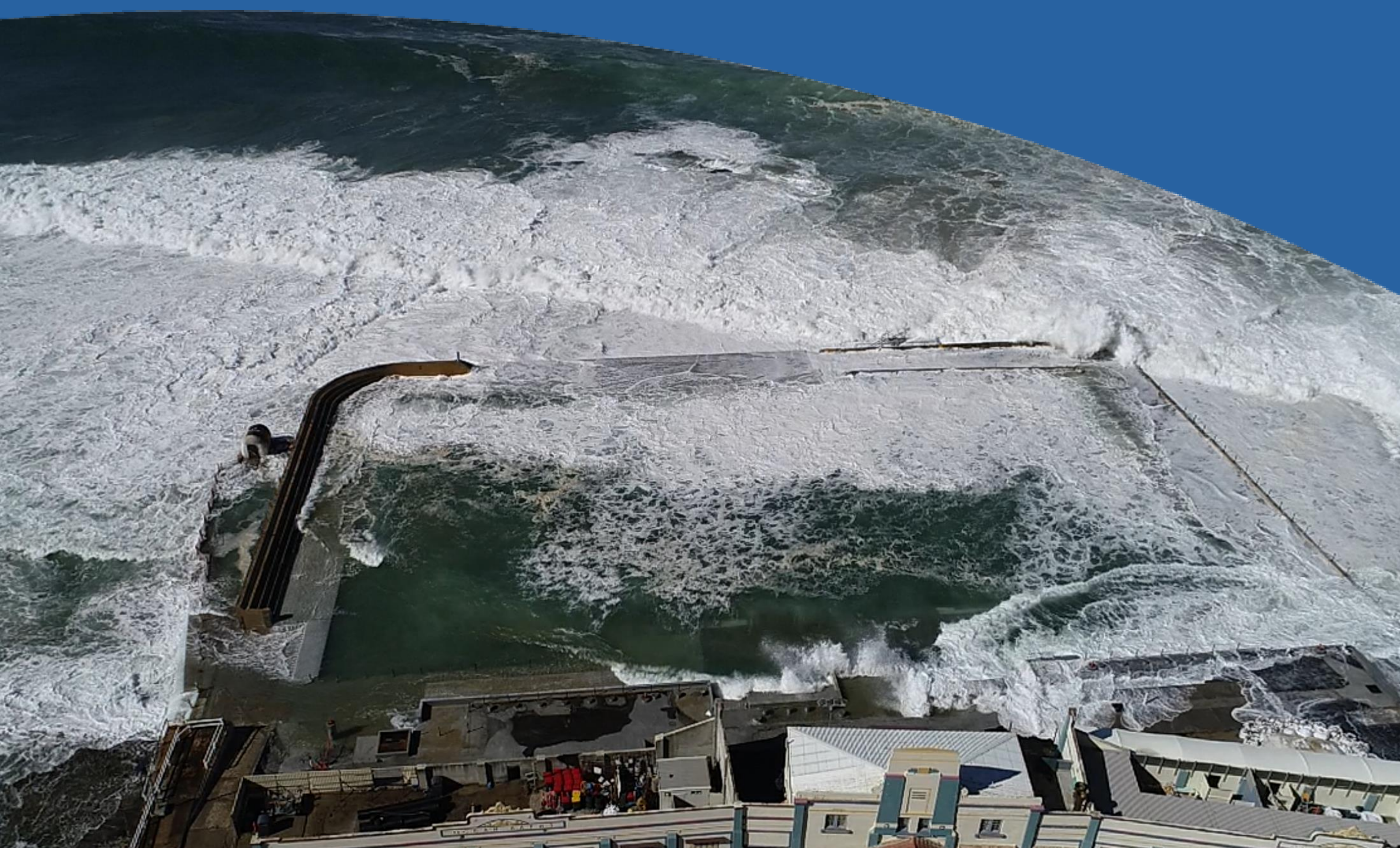
City of Newcastle

Inundation Report

Newcastle Ocean Baths coastal inundation assessment

7 September 2023

Report No: P22294_NewcastleBathsInundation_R2.00



Document Summary

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Executive summary

The City of Newcastle (CN) are currently renewing the Newcastle Ocean Baths. This report provides the outcomes of a coastal inundation assessment that is intended to inform the renewal project. The renewal project splits across two stages; Stage 1 covers the pools and promenade and is currently under construction; Stage 2 focuses on the pavilion buildings and adjacent public domain and is currently in the tendering and design stage.

The objective of this assessment is to provide coastal inundation advice, data and recommendations to inform Stage 2 of the Baths renewal project. The results this are present in the form of inundation extents, levels as well as likelihoods for key locations.

To ensure the objectives of this assessment were met, the following tasks were undertaken:

- Review of data to assess the environmental conditions the site is exposed to for present day and sea level rise (SLR) conditions. This data was used in a joint probability analysis to generate the extreme conditions used in the modelling and assessment.
- Numerical modelling of the inundation at the site based on input wave conditions derived from the data review. Modelling was validated using field measurements captured during a large swell event. The figure below shows a comparison of imagery captured during the monitoring event and from the numerical model.
- Interpretation of the modelling results to present the inundation extents and likelihood of inundation for exposed areas of the facility. These are presented as levels against the finished Stage 1 ground levels and existing building location (see below figure). Mapping of the inundation extents as well as wave loading on key areas are also presented.
- Presentation of options to reduce the impact of inundation on the facility through physical and monitoring measures. These options included:
    - Raised floor levels.
    - Temporary inundation barriers.
    - Moving bleachers.
    - Complementary measures including monitoring and response and building resilience.

The inundation assessment found that the facility in its current configuration will be exposed to increasing and frequent inundation that will affect the design of the Stage 2 facilities and ongoing operation. Extreme conditions at both present day and sea level rise scenarios showed significant inundation in front of the pavilion buildings with water entering the carpark in all cases. Results showed that the central and northern building were most exposed to inundation at present conditions, with inundation depths distributed in front of the buildings evenly under SLR conditions.

An inundation likelihood analysis was conducted using the results of a matrix of modelled conditions to determine the annual number of times certain inundation thresholds are reached for key areas in the facility. This is set up as one of the key inputs for a Trigger Action Response Plan (TARP) framework which has been laid out as a complementary mitigation measure. This plan would be completed with the owners and operators to determine appropriate triggers and inform the number of times that response actions would occur annually. This framework is targeted at being applied to this facility but provides flexibility to be applied to similar facilities subject to inundation.

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Tonkin+Taylor

Table of Contents

List of Figures

List of Tables

1. Introduction

1.1 About this report

The City of Newcastle (CN) are currently renewing the Newcastle Ocean Baths. This report provides the outcomes of a coastal inundation assessment that is intended to inform the renewal project (referred to henceforth as ***the Project***). The renewal project splits across two stages; ***Stage 1*** covers the pools and promenade and is currently under construction; ***Stage 2*** focuses on the pavilion buildings and adjacent public domain and is currently in the tendering and design stage. The Newcastle Ocean Baths (baths) have been constructed on the rock platforms at the northern end of Newcastle Beach (see Figure 1). The pool, promenades, pavilion buildings and associated public domain are all subject to varying degrees of coastal inundation.

The purpose of this report is to provide:

- an improved understanding coastal inundation at the Newcastle Ocean Baths site
- advice on the risk of coastal inundation and make recommendations to inform the design and delivery of Stage 2 of the project.

This technical study forms a part of the design investigations being completed for Stage 2 of the bath's renewal project. It has been prepared in line with the *Coastal Management Act 2016* (CM Act), the NSW Coastal Management Manual (CM Manual) and associated Toolkit. It fulfills the requirements set out in CN's study brief and accords with Bluecoast's proposal (Q22294 dated 21 January 2023).

This report has been prepared by Bluecoast and Tonkin + Taylor and was peer-reviewed by James Carley from UNSW's Water Research Laboratory (WRL).

1.2 Study context

The Newcastle Ocean Baths were constructed in the early 20th century. The baths are iconic to Newcastle's coastline and remain a popular coastal destination and free public swimming spot. However, due to the harsh nature of the coastal environment, the pavilion, promenade and pools are rapidly aging and require upgrades to continue to provide the community with a safe and suitable facility. CN are upgrade these facilities so they can continue to operate as an important community facility.

Due to the compressed schedule of the Stage 1 renewal works as well as the non-critical inundation of the pool, a preliminary coastal inundation assessment was completed for this stage (GHD, 2022). Stage 2 involves retrofitting or reconstruction of the southern and middle pavilions and the rebuilding of the northern pavilion. The intended uses of these renewed buildings (e.g., café's, community spaces) are more sensitive to inundation hazards therefore a more detailed coastal inundation assessment was required for Stage 2.

1.3 Project site

The Newcastle Ocean Baths occupying a prime site on the ocean side of Shortland Esplanade between Newcastle Beach and Nobby's Beach. Figure 1 provides a site map with features of the bath's facility labelled.

The baths are directly influenced by the ocean's wave climate and tidal water levels. The relationship between the baths' infrastructure and ocean is integral to the facility's functionality but also exposes the site to hazards. The Newcastle Ocean Baths present a complex interaction of built and natural elements within a coastal context, warranting careful engineering assessment for the renewal, safe operations and maintenance.

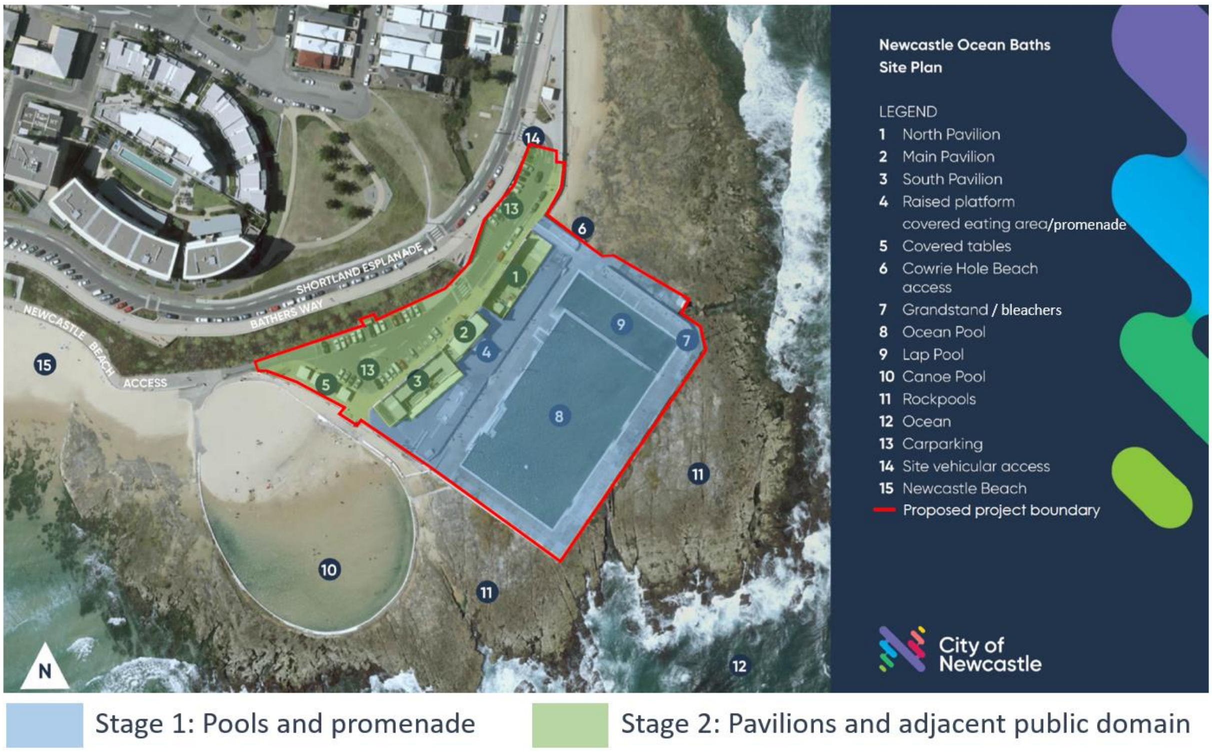


Figure 1. Newcastle Ocean Baths site plan and project stages (source: adapted from CN, 2023)

1.4 Study objectives

The principal objectives of the coastal inundation assessment are to assist in assessing:

- environmental (i.e., wave and water level) triggers for management of the baths area due to unsafe conditions
- wave loading on the pavilion buildings/ façade to inform structural design
- design or operational measures for provide resilience against coastal inundation.

1.5 Scope and structure of this report

This report sets out the findings of the coastal inundation assessment. The study uses a data-driven approach combined with detailed numerical wave modelling to understand and quantify the facilities exposure to coastal inundation, now and in the future. This information is then used to identify, develop and assess potential options to improve resilience. The report provides assumptions and limitations associated with the methods adopted. It provides justification for identified resilience options as well as the evaluation of these options.

The report is set out as follows:

- Section 2 provides background information.
- Section 3 documents the guiding principles used in the coastal inundation assessment for the bath's renewal project.
- Section 4 outlines the data and collected for this assessment including targeted analysis.
- Section 5 details the establishment, calibration and application of a detailed and site-specific numerical model used to simulate wave generated inundation at Newcastle Baths.

- Section 6 describes the coastal inundation assessment results.
- Section 7 contains a summary of study outcomes and recommendations.

2. Background information

2.1 What is coastal inundation?

Coastal inundation is the flooding of coastal areas due to actions of the sea. It typically occurs when a combination of marine and atmospheric processes raises ocean water levels above normal elevations and inundate low-lying areas or waves overtop dunes, structures, and barriers. It is often associated with coastal storms which result in elevated water levels (storm surge) and waves. Sea level rise, which is already occurring because of climate change and is projected to accelerate will lead to more frequent and hazardous coastal inundation, particularly for coastal facilities on fixed rock platforms like the Newcastle Ocean Baths.

Processes contributing to coastal inundation can be broadly classified into two main components: static (or quasi-static) and dynamic processes. These components are depicted in Figure 2 and described as:

1. **Quasi-static components:** These include processes that are occur slower than individual waves or a group of waves, such as sea-level rise, tides, storm surges, and wave setup. These factors are responsible for changes in the average or 'still' water level.

- Sea-level rise: This is a slow but continuous process mainly driven by climate change and anthropogenic activities. It raises the mean water level, making coastal areas more prone to flooding.
- Tides: Tidal fluctuations are caused by the gravitational interactions between the Earth, Moon, and Sun. They are predictable and occur over a span of roughly 12 hours for semi-diurnal tides.
- Storm surge: Storm surges are the rise above the normal water level driven by storm-induced changes in atmospheric pressure and wind. While they happen faster than tides or sea-level rise, they are still relatively slow compared to wave dynamics.
- Coastal trapped waves: Long period waves with periods of days to weeks, generated by strong wind events on the southern Australian coastline and Bass Strait effect the NSW coastline.
- Ocean circulation: The East Australian Current (EAC) can raise the coastal water levels along the NSW coast for extended periods by transporting large quantities of water onshore (e.g., migration of eddy currents along a coastline).
- Wave setup: This is an increase in water level due to the action of waves breaking near the coastline, resulting in a super-elevation of the mean water level inshore of wave breaking.

2. **Dynamic components:** These include processes that occur rapidly and are associated with wave actions, such as wave run-up and overtopping.

- Surf beat: Is caused by infragravity (IG) waves and the variation in wave energy within a wave group (i.e., observed as a ‘set’ of large waves followed by a ‘lull’). Free propagating IG waves results in significant water level fluctuations inshore of wave breaking (e.g., along a shore or within the Newcastle Ocean Baths site) over periods ranging from 30 seconds to around five (5) minutes.
- Wave runup: This is the rush of water up a beach or structure following the breaking of a sea or swell waves (i.e., SS waves with periods less than 25 seconds). The extent of runup is a dynamic process related to the wave's energy and the beach or structure's slope and roughness. Surf beat is typically included in wave runup.

- Wave overtopping: This happens when waves exceed the height of a coastal structure or landform (like a seawall or dune), leading to water flowing over the top. It's a highly dynamic process, very sensitive to both wave and structural characteristics. Overtopping occurs at the SS and IG frequency and can cause impulsive aerial (injecting) and overland flows with potentially damaging force.

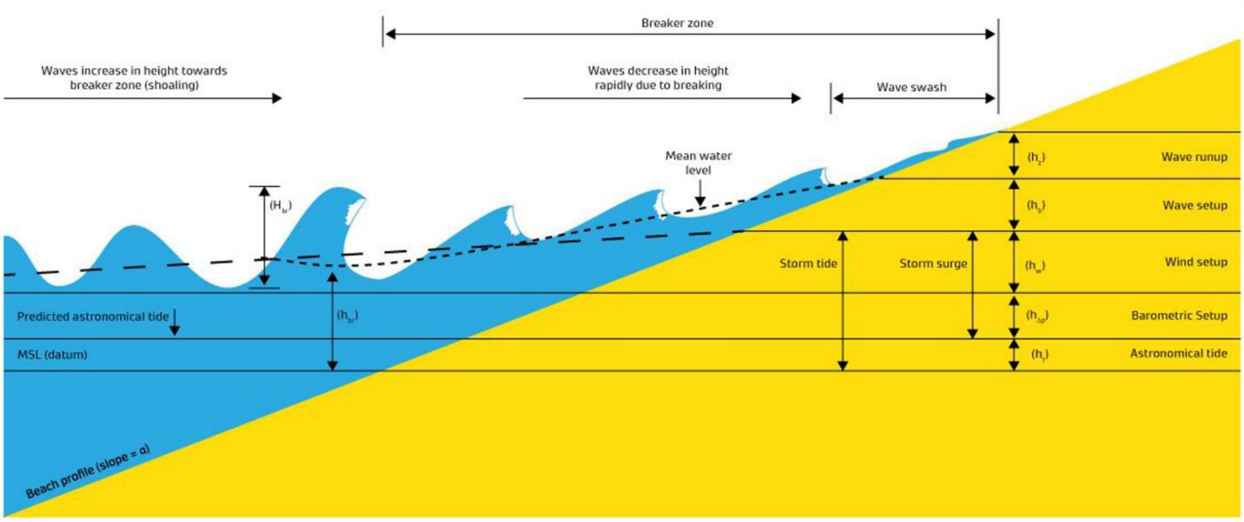


Figure 2. Processes contributing to coastal inundation.

2.2 Previous coastal inundation assessments and management at Newcastle Ocean Baths

The most recent coastal inundation assessment that incorporated with Newcastle Ocean Baths site was completed in 2022 as part of Newcastle Southern Beaches Coastal Management Program (Royal HaskoningDHV, 2022). This assessment was regional in nature, adopting a simple 'bathtub' approach to map calculated quasi-steady components as well as wave run-up levels ($R_{2\%}$) onto a digital elevation model based on DPE's 2018 Coastal LiDAR. Figure 3 provides a summary of the adopted quasi-steady and wave run-up levels and mapping for the baths site. The Newcastle Ocean Baths were listed as an asset that is at immediate risk of coastal inundation (i.e., 2022 planning period). The risk level to the public and asset increases in subsequent planning periods in line with sea level rise projections. These findings were consistent with the previous coastal hazard assessments which also found an existing and worsening coastal inundation and wave overtopping risk at the project site (BMT WBM, 2014).

As reflected in the 2014 and 2022 coastal hazard studies, coastal inundation at the Newcastle Ocean Baths facility has been a known and understood issue for significant period. These issues are managed by the City of Newcastle through the Newcastle Coastal Zone Management Plan (CZMP) 2018 (NCC, 2018). GHD 2022 list 10 actions in the 2018 CZMP that can be related to the management of the Newcastle Ocean Baths action ID's . These 10 actions can be distilled as:

- Consideration of coastal inundation hazard in the design when renewing or constructing coastal infrastructure.
- As required, undertake additional coastal hazard investigations.
- Consider management options to minimise coastal inundation impacts along coastal promenades/seawalls.
- Undertake post storm inspections of coastal hazards, to identify potential works that may be required.
- Develop an emergency procedure for the closure of coastal roads impacted by coastal inundation.

- Educate the community about the coastal hazards impacting on the Newcastle community.

The 2018 Newcastle CZMP will be superseded by the Southern Beaches Coastal Management Program (CMP) once it is completed. The CMP may result in changes to or new management actions that related to coastal inundation at the Newcastle Ocean Baths.

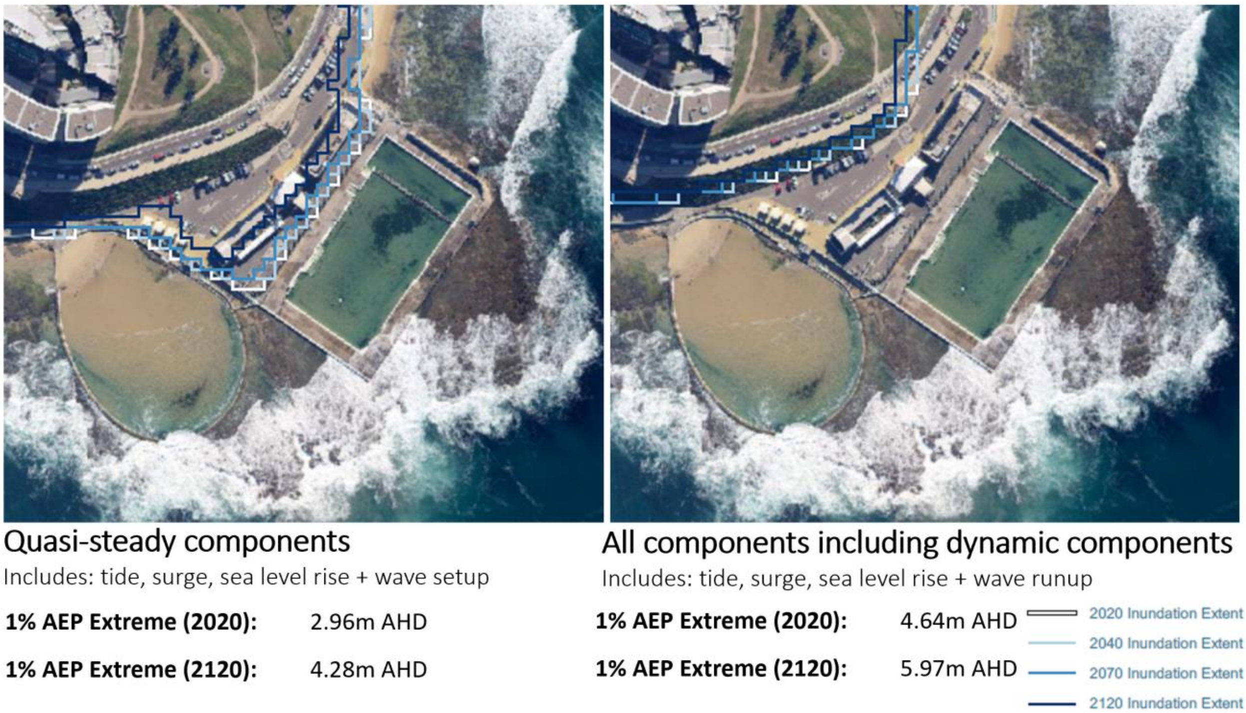


Figure 3. Coastal inundation levels and extents from the most recent Newcastle-wide assessment (adapted after Royal HaskoningDHV, 2022).

Note: Royal HaskoningDHV's 2022 report does not specifically state the inundation levels stated above, these have been summed from the components presented in the report. The quasi-steady and wave runup levels and extents shown are based on upper bound extreme 1% AEP for present and future sea levels.

2.3 Existing (pre-project) site features

The Newcastle Ocean Baths have been constructed on the rock platform between Newcastle Beach and Nobbys Beach. Figure 1 and Section 1.3 provide a basic overview of the project site. Features which are most relevant to this assessment are:

- Within the footprint of Stage 1:
    - Ocean pool which is configured as a main pool and adjacent lap pool separated by a timber footbridge on concrete piers. The pool floor is uneven bedrock, often overlain with sand. The pool is flushed by tidal and wave action as well as pumping.
    - Concrete promenade surrounding the pool.
    - Art Deco-style concrete bleachers along the northern side of the baths.
    - A raised platform on the western side of the pool with a rendered brick retaining wall separating the raised area from the pool's promenade.
- Within the footprint of Stage 2:

  - Art Deco-style pavilion consisting of three separate buildings. In the pre-renewal configuration, the southern pavilion was used for amenities (change rooms etc), the central pavilion for a kiosk, workshop and first aid while much of the northern pavilion was unused.
- To the west of the pavilions is a relatively flat carpark. Further landward from the carpark the terrain rises sharply up to Shortland Esplanade.
- The Canoe Pool lies adjacent to the southern boundary of the baths. Adjacent to the northern boundary of the site is the Crowie Hole.

2.4 Heritage value of the baths

The 100-year-old baths have significant cultural, social and heritage value to the local and regional community. The baths are located within the Newcastle East Heritage Conservation Area and are themselves a listed heritage item of local significance (see item I489 in Schedule 5 of the Newcastle Local Environmental Plan 2012). The renewal is to consider the heritage value of this historic facility alongside its recreational, amenity and community-use value.

The western façade of the facility is a culturally significant element of the site. The façade is currently propped within the footprint of the northern pavilion but has shown deterioration in its structural condition due the wind loading on the ocean side. The facades condition is heavily degraded with non-uniform reinforcing across the structure. An extensive cathodic protection system was installed on the façade in 2007-2008 which has had mixed success in extending the design life of the structure.

2.5 Renewal project

Stage 1 aims to improve safety, water quality and accessibility to the pool and is currently under construction. Figure 4 presents the extent of work for Stage 1. Works includes:

- New raised the pool deck with a finished level of 2.475m AHD along the outer edge and a 1% fall to the pool edge at 2.380m AHD.
- New bleachers and reconstruction of circular art-dec stairs along retaining wall between the lower pool deck and the upper pavilion deck. The bleachers will be covered by flexible (removable) shade sails. The upper pavilion deck is flat with a design elevation of 4.000m AHD.
- New lifeguard observation space with glass frontage to improve sightlines.
- Improved accessibility by augmentation of the ramp entry to the facility (car park to pool) and new access ramp into pool.
- Replacement of the pool walls, concreting a flat bottom floor for the pool and refurbishment of the existing timber boardwalk.
- Replacement of the pump system and addition of a skimmer system.
- New showers and seats on the lower pool deck.

We have reviewed the Stage 1 plans to ensure the site-specific features and elevation of the Stage 1 works were considered in the coastal inundation assessment. These are discussed further in Section 4.2 and Section 5.3.

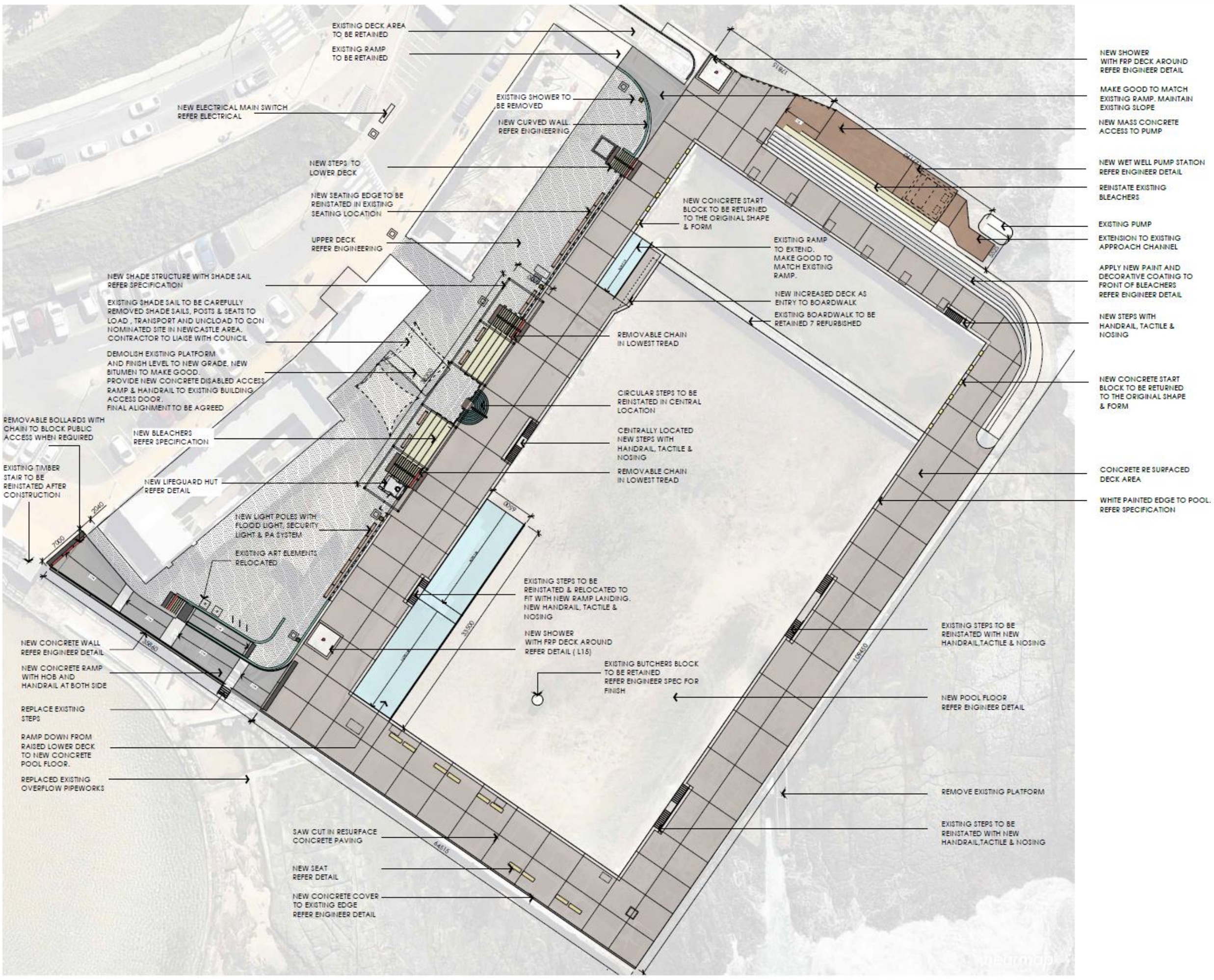


Figure 4. Overview and general layout of Stage 1 of Newcastle Ocean Baths renewal (source: Terras 2021).

Stage 2 of the upgrade of Newcastle Ocean Baths will focus on the pavilion and surrounding public areas. Stage 2 is subject to ongoing community consultation with the design recently tendered. A User Needs Analysis for Stage 2 was recently completed (GHD, 2023). Based on several assessments (i.e., guiding principles, benchmarking, needs assessment, CPTED statement and a preliminary business case) four (4) footprint options were presented that explore potential location and the functional relationships of the Stage 2 renewal area. As an example, Figure 5 presents one of these options. The follow key ideas and concepts employed by GHD when developing the Stage 2 options are relevant to the coastal inundation assessment:

- Amenities (incl. change room facilities) to be located together and sited on the lower level of the southern pavilion, as the amenities are best placed to withstand incidences of coastal inundation. Amenities are to be enclosed with sky lighting over.
- Café positioned to take advantage of the northern aspect afforded at that end of the northern pavilion. This would also allow for a positive interface with users of the Bathers Way Coastal Walk, and at the same time offering the flexibility to use external seating throughout the summer months to increase capacity and cater for demand.
- Any development of second storey spaces should implement the centralisation of amenities (toilet facilities) designated to the function of those spaces.

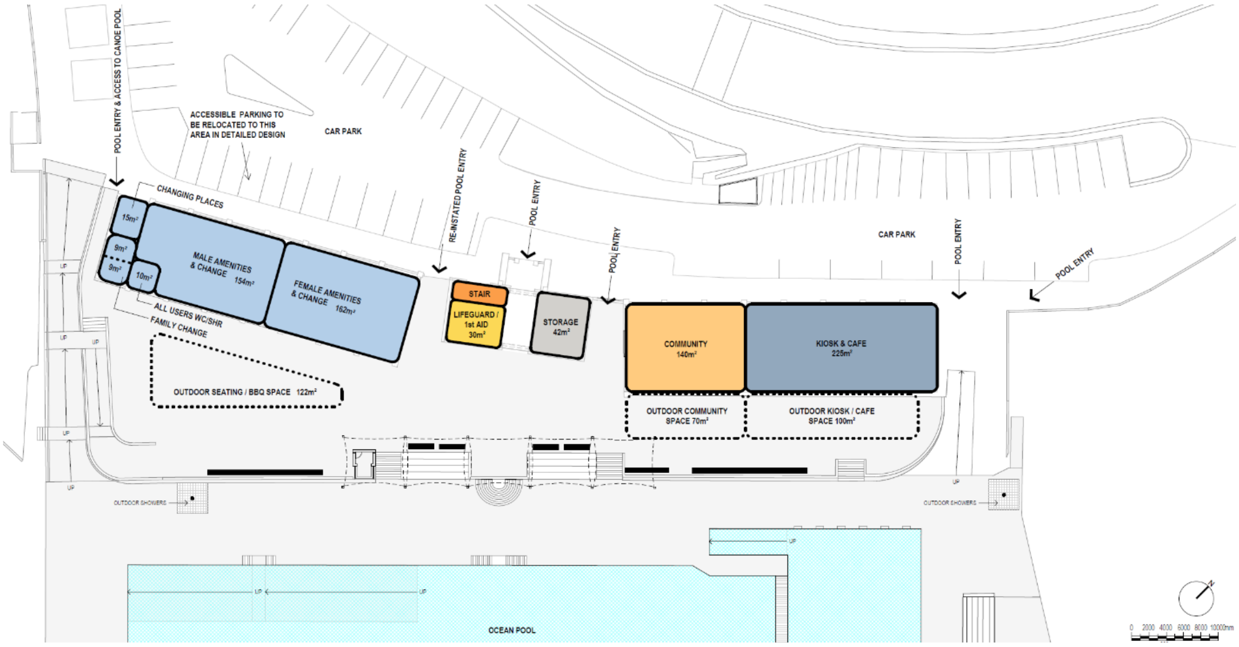


Figure 5. Stage 2 ground floor sketch (Option A) (source: GHD, 2023).

3. Guiding principles and legislation

3.1 Guiding principals

Assessing and designing resilience measures to address coastal inundation at coastal facilities such as ocean baths is complex. There is a myriad of factors that need to be considered. Based on our review, these guiding principles are proposed to inform the coastal inundation assessments of Stage 2 of the bath's renewal project:

1. Climate projections: the project will be developed to incorporate the latest climate change projections for sea-level rise, wave action and storm surge. These changes can significantly impact coastal the baths site and are at the core of this inundation assessment.

2. Site-specific factors: site-specific factors such as bathymetry, local topography, geology and existing and planned structures are to be considered.

3. Data-driven: use of high-quality data and observations of coastal inundation at the baths site or the factors that influencing it will underpin a greater understanding of the hazards and support evidence-based decision making. Measurements of waves and water levels, both in the nearshore and within the bath's site, is a critical to providing valuable insight and quantification and are discussed in Section 4.3 to Appendix A.

4. Appropriate assessment methods: this assessment adopts an approach centred around the use of a phase resolving wave model (XBeach-NH) which is supported by:

   – data analysis of high-quality local forcing data

   – targeted field measurements of coastal inundation at the baths site

   – probabilistic analysis by combining input data and model results to determine the exposure to inundation hazard (now and in the future) different events (like storm surges or sea-level rise)

This provides the best balance between resolving the required processes within an achievable scale and timeframe. **Appendix B:** provides a summary of methods for assessing coastal inundation and further justification for the approach adopted herein.

5. Avoidance of vulnerability: assess the vulnerability of the pavilion buildings and associated public domain (as renewed) when planning the layout of the various usages for the Stage 2 facilities.
6. Resilience measures: Evaluate potential measures for increasing the site's resilience. These could include raising floor levels, using alternative building materials.
7. Adaptive management: Develop an adaptive management plan that incorporates designing/planning with uncertainty and emphasizes monitoring and continual improvement in understanding, thus ensuring the flexibility necessary to respond effectively to the unpredictable shifts brought about by climate change.
8. Stakeholder engagement: consult with the various stakeholders as part of the assessment process. This includes CN, local communities, bath users, government agencies, designers who may be affected by or have an interest in the outcome of the assessment.
9. Legal and regulatory compliance: Ensure that all assessments, plans, and actions are compliant with local, state, and federal laws and regulations. This includes environmental regulations, building codes, and any specific regulations related to ocean baths or similar facilities.

3.2 Relevant legislation and guidelines

The legislation and guidelines relevant to coastal inundation and the renewal of the Newcastle Ocean Baths are contained in Table 1.

Table 1. Summary of relevant legislation and guidelines.

Reference	Relevance
Legislation and other planning instruments	
Coastal Management Act 2016	
State Environment Planning Policy (Resilience and Hazards) 2021	
State Environment Planning Policy (Transport and Infrastructure) 2021	
Southern Beaches Coastal Management Program (once certified)	
Newcastle Coastal Zone Management Plan 2018	
Guidelines	
NSW Coastal Management Manual	
EurOtop (2018). Manual on wave overtopping of sea defences and related structures	
USACE (2011). Coastal Engineering Manual – Part VI.	

4. Data review

4.1 Data used in this report

The data reviewed in this section is used to inform he long term environmental condition at the ocean baths facility. It is used to derive the wave and water level conditions that will be the modelling input parameters for the inundation scenarios. These are presented in Section 5.5.1.A summary of these datasets used in this assessment are presented in Table 2, with a map of associated monitoring sites (where applicable) provided in Figure 6.

Table 2: Data used in this report.

ID	Description	Source	Dates
Water Levels	Water levels from: • Stockton Bridge (15-minutes) • Shoal Bay (15-minutes) • Patonga (15-minutes)	MHL	Dec 1984 – May 2023 Sep 1985 – Mar 2023 June 1992 – May 2023
Waves	Measured wave heights, directions, and periods at Sydney WRB in around 60-80m water depth	MHL	Mar 1992 – Mar 2023 (direction since 1999)
	Measured wave heights, directions, and periods at Crowdy Head WRB in around 80m water depth	MHL	Oct 1985 – May 2022
	Measured wave heights, directions, and periods at Port of Newcastle Inner WRB in around 20-25m water depth	Port Authority of NSW	Nov 2009 - May 2023
	NSW nearshore wave transformation hindcast (both 10 and 30m contours)	MHL	Nov 1999 – Jan 2023
Wind	Measured wind at Nobbys AWS (30 minutes)	BOM	Oct 2001 – Mar 2023
Survey	Topo-bathy DEM derived from 2018 NSW Coastal LiDAR Project (5m resolution)	DPE	2018
	Newcastle topographic LiDAR (0.5m resolution)	CN	-
	Ocean Baths topographic survey (0.5m resolution)	CN	2020
Inundation event monitoring	Metocean instruments measuring wave, water level and currents (ADCP and pressure sensor)	Bluecoast	8 to 10 May 2023
	Video recording of wave dynamics, overtopping and inundation	Bluecoast	8 to 10 May 2023
Media	Community sourced media capturing inundation extents of past extreme wave events	CN	-

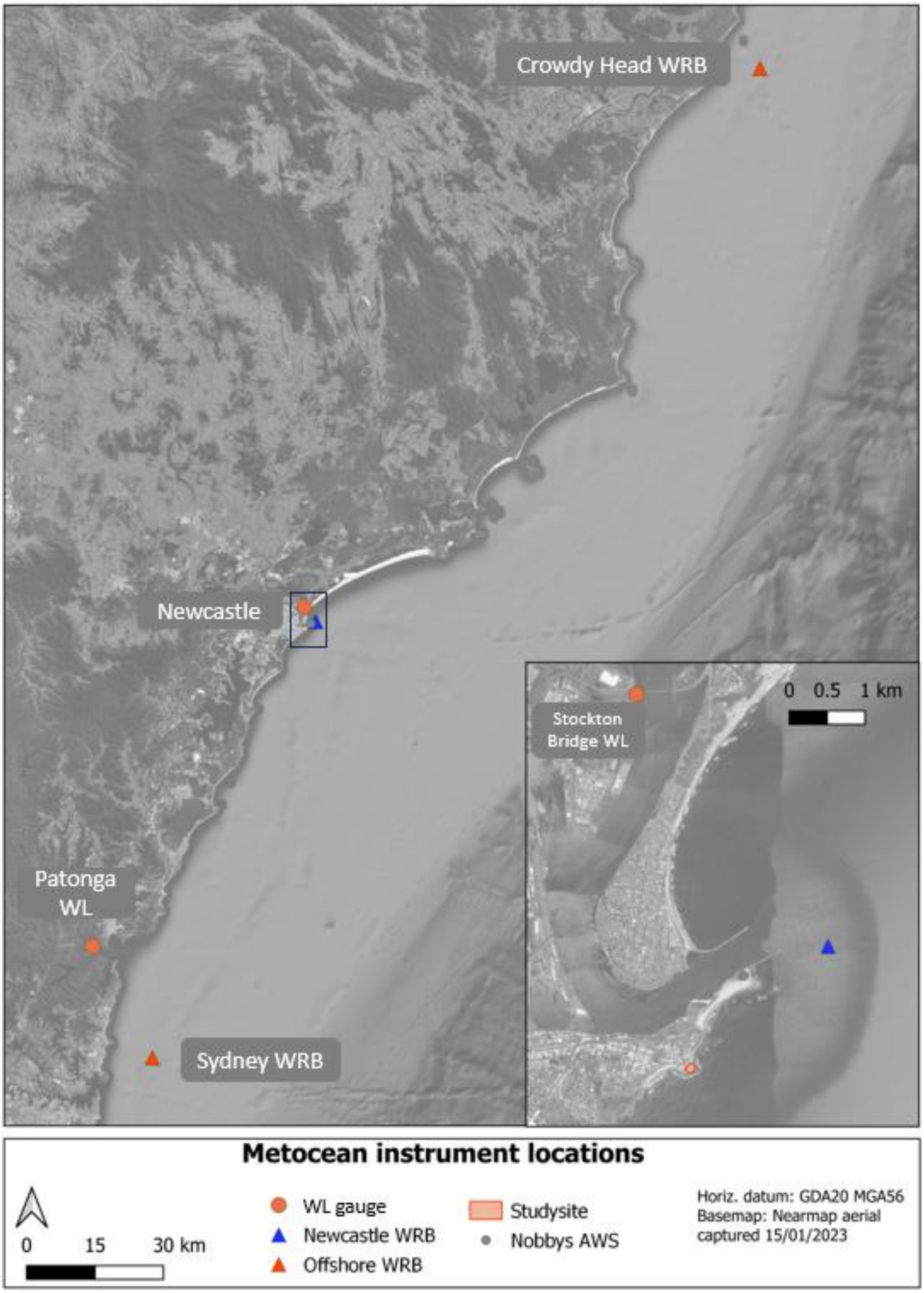


Figure 6: Map of metocean monitoring sites used in this report.

4.2 Coastal morphology

The coastal region where the Newcastle Ocean Baths are located is characterized by rocky headlands, cliffs, and sandy beaches. The baths themselves are constructed within a natural rock shelf that has been modified to create a safe swimming area. Figure 7 shows the Ocean Baths site in their geological setting with annotations of the geological units derived from the NSW Seamless Geology dataset.

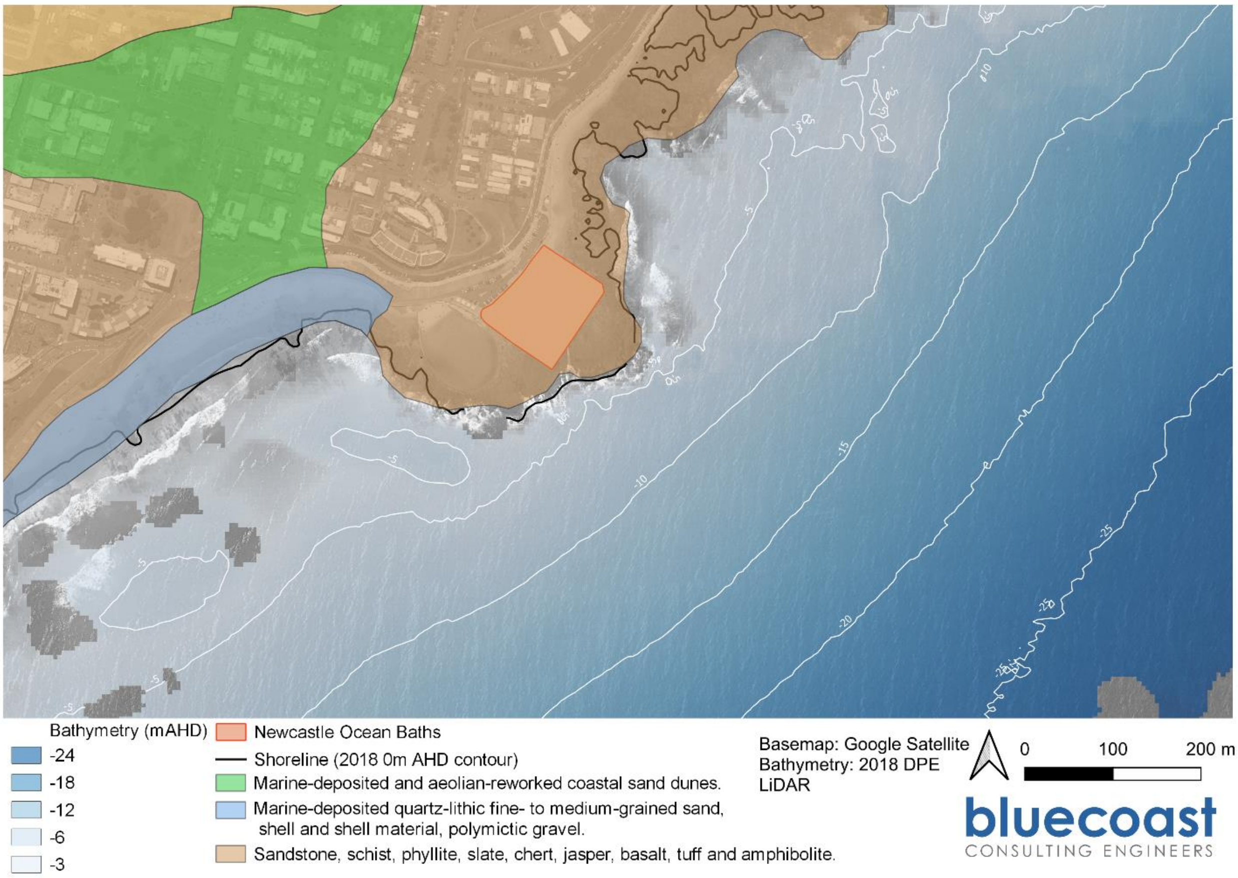


Figure 7: Geological setting of the Newcastle Ocean Baths.

Figure 8 maps the elevations across the Ocean Baths sites and immediate surrounds. The elevations have been mapped by combining several existing surveys along with the Stage 1 development design levels. A profile is shown that shows the relative level and slope of the rock shelf on the ocean side of the baths as well as the design levels across the baths site.

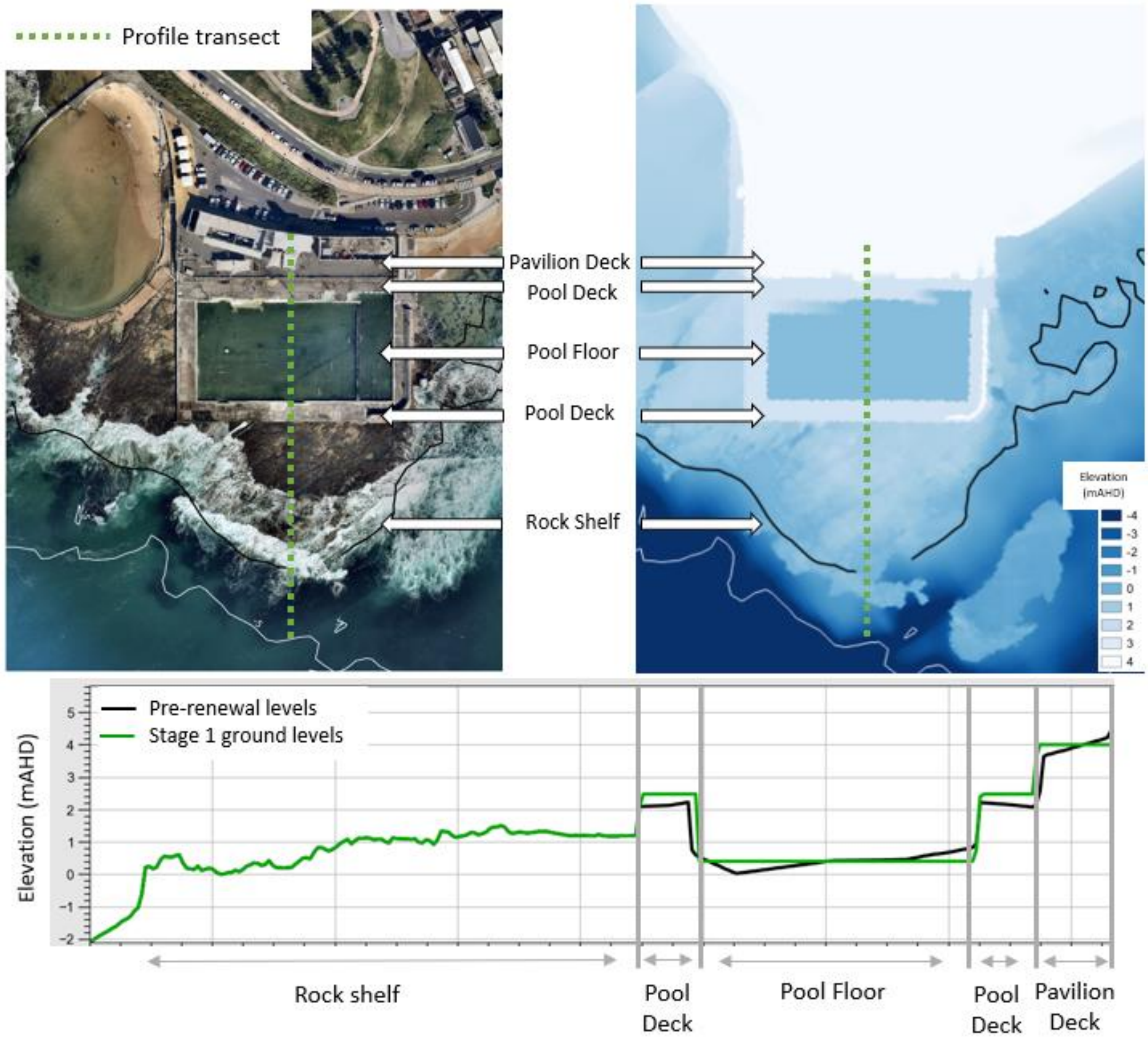


Figure 8: Coastal profile of the Ocean Baths at Stage 1 design level.

4.3 Wave climate

4.3.1 Offshore wave climate

The study area's open coast is subject to a moderate wave climate predominantly from the south to south-east. It is exposed to waves generated from three primary sources: Tasman Sea swells, locally generated wind-waves and waves from east coast lows (ECL) systems.

A review of observed wave data from the deepwater Sydney WRB was undertaken (see Figure 6 for WRB locations). The Crowdy Head waverider buoy (WRB) was also reviewed and used to compare various wave events against the Sydney and Newcastle nearshore WRB but is not presented herein. The Sydney WRB data was supplied as an hourly timeseries of standard wave parameters. Sydney offshore directional wave climate for total wave energy as well as swell (swell waves, Tp >8s) and sea (local sea, Tp <8s) is provided in Figure 9. Average and seasonal wave statistics are given in Table 3.

The deepwater wave sites are seen to be dominated by moderate energy, south to southeasterly swell waves, with mean significant wave heights 1.63 m. The largest waves with largest period typically occur from the south-east to south sector in the winter months. 63% of offshore waves propagated from the south-east to south sector, originating in the Tasman Sea and Southern Ocean. Easterly waves (from east north-east to east south-east) make up the 30% of the total offshore wave energy.

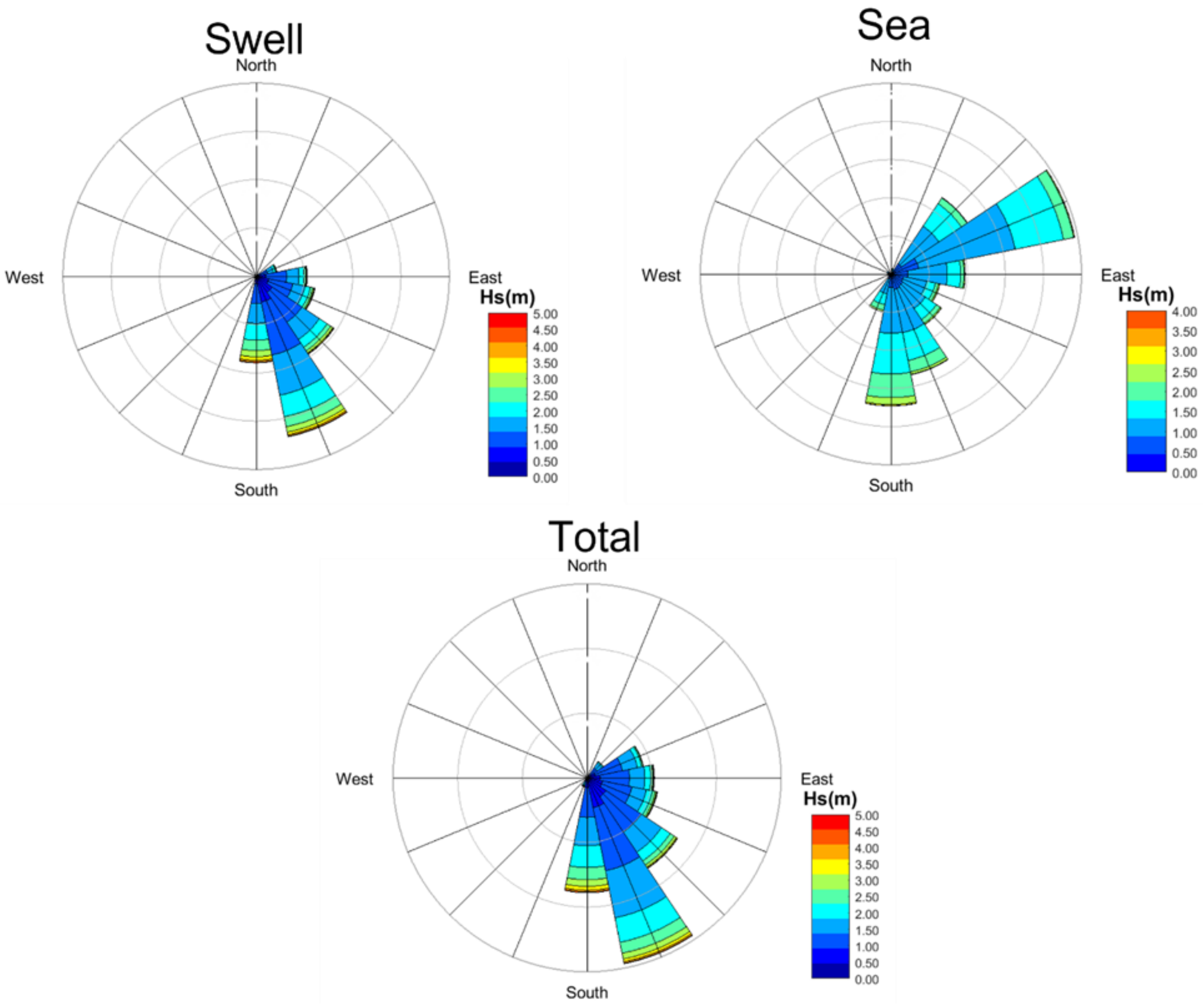


Figure 9: Wave roses at Sydney WRB for swell conditions (Tp > 8s), sea conditions (Tp < 8s) and total.

Table 3: Wave measurement statistics derived from Sydney WRB.

Parameters	Statistics	LTA (31 years)	Winter	Autumn	Summer	Spring
Significant wave height (H_s) [m]	Mean	1.63	1.65	1.67	1.59	1.59
	20%ile	1.05	0.97	1.06	1.12	1.07
	50%ile	1.46	1.43	1.49	1.46	1.44
	75%ile	1.94	2.01	2.03	1.86	1.88
	90%ile	2.56	2.78	2.64	2.37	2.45

Parameters	Statistics	LTA (31 years)	Winter	Autumn	Summer	Spring
	99%ile	4.22	4.69	4.22	3.60	3.95
	99.5%ile	4.70	5.24	4.79	3.94	4.42
	Max	8.43	7.76	8.43	6.53	6.22
Peak wave period (T_p) [s]	Mean	9.8	10.5	10.3	9.1	9.4
	20%ile	7.7	8.8	8.3	7.0	7.3
	50%ile	9.8	10.5	10.3	8.9	9.3
	75%ile	11.5	12.1	11.8	10.5	10.8
	90%ile	12.9	13.5	13.3	12.1	12.5
	99%ile	15.4	16.0	16.0	14.9	15.4
	% of time sea (Tp < 8s)	22%	12%	15%	34%	29%
	% of time swell (Tp > 8s)	78%	88%	85%	66%	71%
Peak wave direction (D_p) [°N]	Weighted average	150	154	148	141	154
	Mean	137	145	136	126	136
	STD	37	32	35	40	40

An extreme value analysis (EVA) of the Sydney WRB data was undertaken to find the 1-hour duration wave heights for a given return probability (e.g., 1% Annual Exceedance Probability (AEP)). A peak over threshold analysis of the reanalysis wave heights identified the extreme events and a Weibull distribution was fitted to the extreme wave heights to provide the annual exceedance probability (AEP) wave heights. The resulting design ARI wave conditions are presented in Table 4. Figure 10 shows the extreme value distribution of significant wave heights. The 50-year and 100-year ARI significant wave heights are 8.03m and 8.35m.

Table 4: Annual exceedance probability (AEP) wave heights for Sydney WRB.

AEP	ARI (years)	H_s (m)	98% confidence limit (m)
63%	1	5.94	5.85 – 6.03
20%	5	6.87	6.51 – 7.23
10%	10	7.23	6.72 – 7.74

AEP	ARI (years)	H_s (m)	98% confidence limit (m)
5%	20	7.69	6.96 – 8.42
2%	50	8.03	7.12 – 8.93
1%	100	8.35	7.27 – 9.44

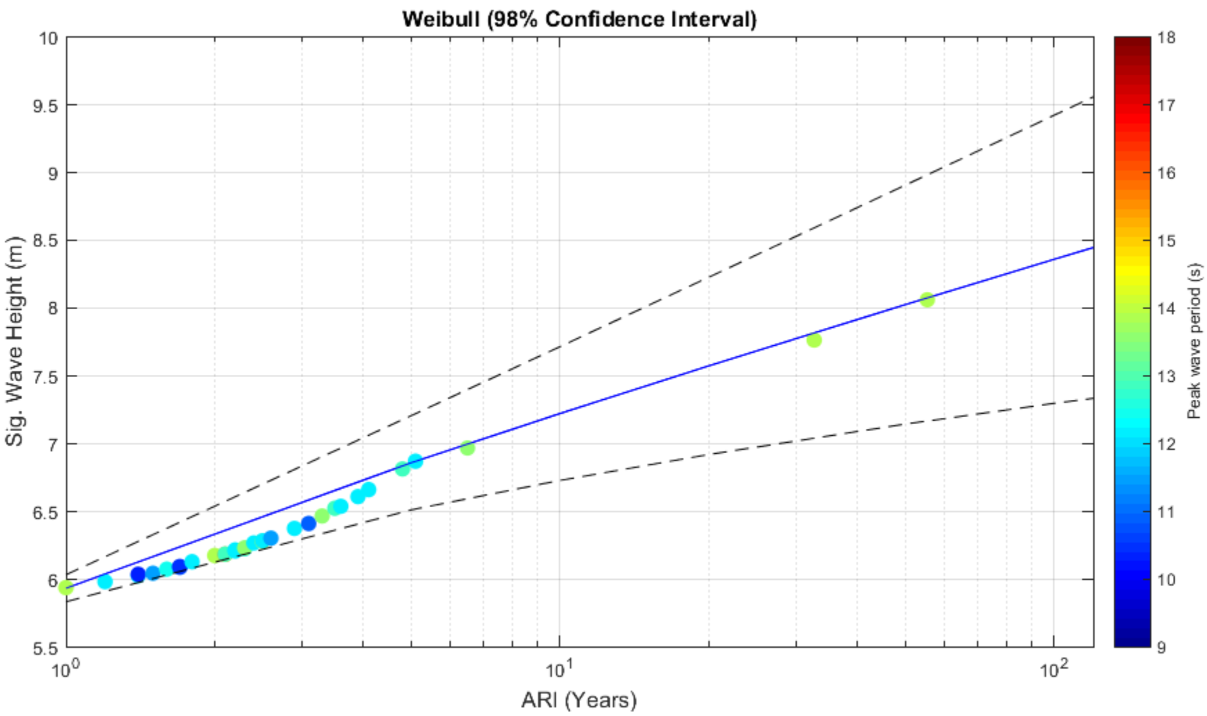


Figure 10: Results of extreme value analysis at Sydney WRB.

4.3.2 Nearshore wave climate

A 13-year record of nearshore wave measurements at the Port of Newcastle WRB (inner buoy), located 2.5 km from the baths site, were purchased from NSW Port Authority for use on the project. Wave measurements were supplied as a timeseries of standard wave parameters at 10-minute intervals. A 10-minute sampling interval is not common[1] for open coast wave rider buoys and significant noise was noted in wave parameters particularly during larger wave conditions. Averaging over each hour was used to remove some of the noise and resample the timeseries to be hourly[2].

The nearshore offshore directional wave climate for total wave energy as well as swell (swell waves, Tp >8s) and sea (local sea, Tp <8s) is provided in Figure 11. The average wave climate statistics for the Port of Newcastle WRB are provided in Table 5.

At the nearshore Port of Newcastle WRB, the mean significant wave height is 1.43m. Seasonal variation like that observed at the Sydney WRB are seen. For example, over summer waves tender to be shorter of lower wave heights and more from the east. Due to wave refraction, the wave directions are more

[1] Open coast wave rider buoys are typically set up to collect a wave trace for just over 17 minutes and use this to determine standard wave parameters (Hs, Tp, Dp etc).
[2] It is recommended that this data source is reviewed in line with standard coastal engineering practices if used for ongoing monitoring activities.

easterly when compared to the deeper Sydney WRB. For example, the weighted average peak wave direction at Sydney WRB is 150°N, while at the Port of Newcastle WRB this is 140°N. Over the measured period at Newcastle, the 99.5th percentile significant wave height was 4.58 m, while the maximum was 8.39 m.

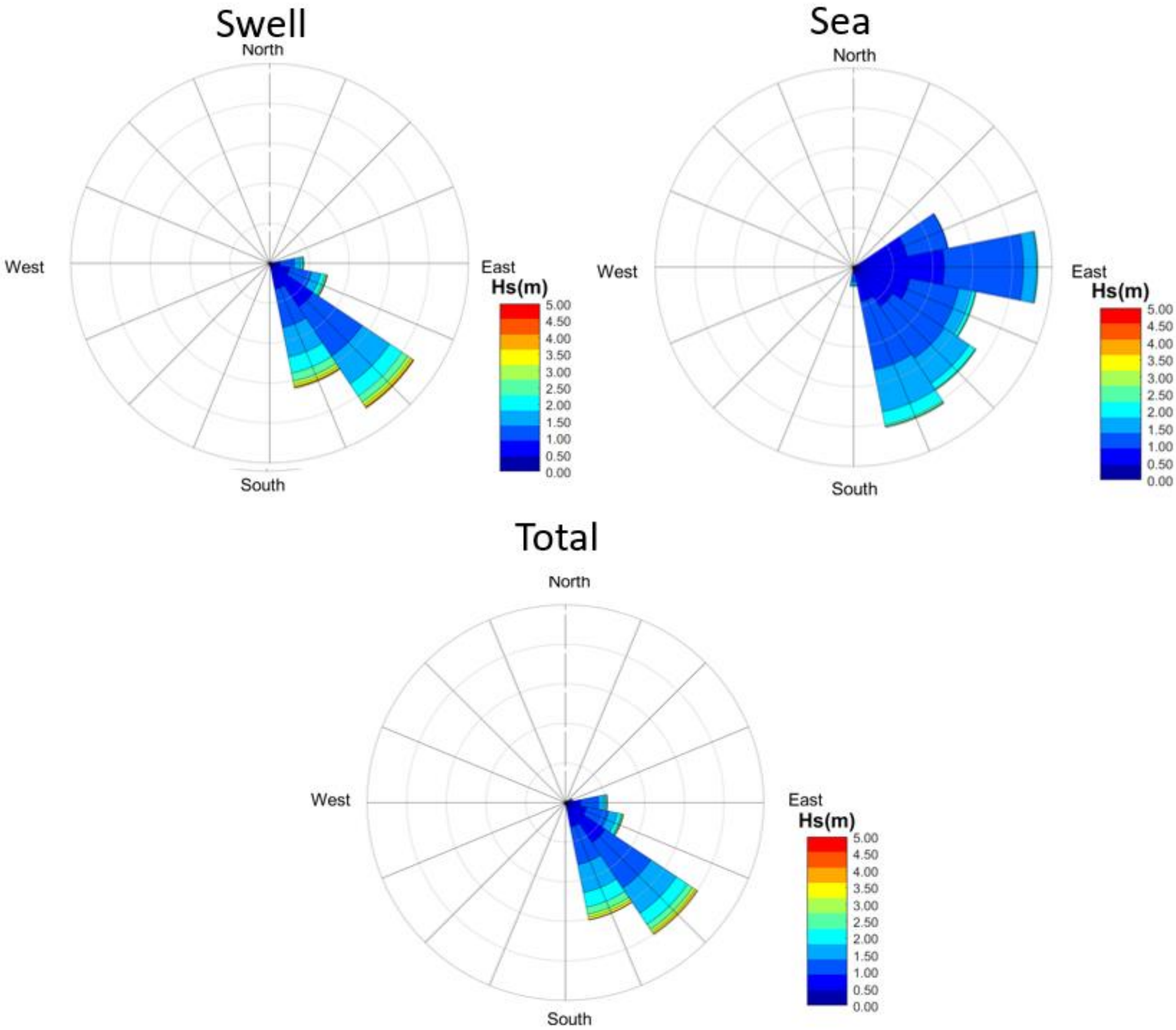


Figure 11: Wave roses at Newcastle nearshore WRB for swell conditions (Tp > 8s), sea conditions (Tp < 8s) and total.

Table 5: Wave measurement statistics derived from the Newcastle nearshore WRB.

Parameter	Statistic	LTA (14 years)	Winter	Autumn	Summer	Spring
Significant wave height (H_s) [m]	Mean	1.43	1.52	1.47	1.35	1.40
	20%ile	0.89	0.84	0.90	0.91	0.88
	50%ile	1.26	1.33	1.29	1.20	1.26
	75%ile	1.73	1.90	1.78	1.58	1.70
	90%ile	2.33	2.62	2.39	2.08	2.23
	99%ile	4.03	4.52	4.23	3.43	3.64
	99.5%ile	4.58	5.09	4.74	3.85	4.09
	Max	8.39	7.59	8.39	5.96	6.52
Peak wave period (Tp) [s]	Mean	10.8	11.5	11.2	9.8	10.6
	20%ile	8.8	9.8	9.2	7.8	8.5
	50%ile	10.8	11.4	11.1	9.8	10.6
	75%ile	12.4	12.9	12.7	11.3	12.4
	90%ile	13.8	14.2	14.1	12.8	13.7
	99%ile	16.3	16.8	17.0	15.3	16.1
	% of time sea (T_p < 8s)	13%	4%	8%	22%	16%
	% of time swell (T_p > 8s)	87%	96%	92%	78%	84%
Peak Wave Direction (Dp) [˚TN]	Weighted average	140	139	142	135	144
	Mean	133	137	134	126	137
	Standard deviation	22	18	21	26	22

An extreme value analysis (EVA) of the Newcastle nearshore WRB data was undertaken using the same analysis technique as used for the Sydney WRB. The 1-hour duration extreme wave heights at this nearshore location are contained in Table 6. Figure 12 shows the extreme value distribution of significant wave heights at Newcastle WRB.

Table 6: Annual exceedance probability (AEP) wave heights for Newcastle WRB.

AEP	ARI (years)	Hs (m)	98% Confidence Limits (m)
63%	1	5.93	5.71 - 6.14
20%	5	7.15	6.61 – 7.68
10%	10	7.63	6.94 – 8.32
5%	20	8.09	7.22 – 8.96
2%	50	8.69	7.56 - 9.82
1%	100	9.13	7.79 - 10.47

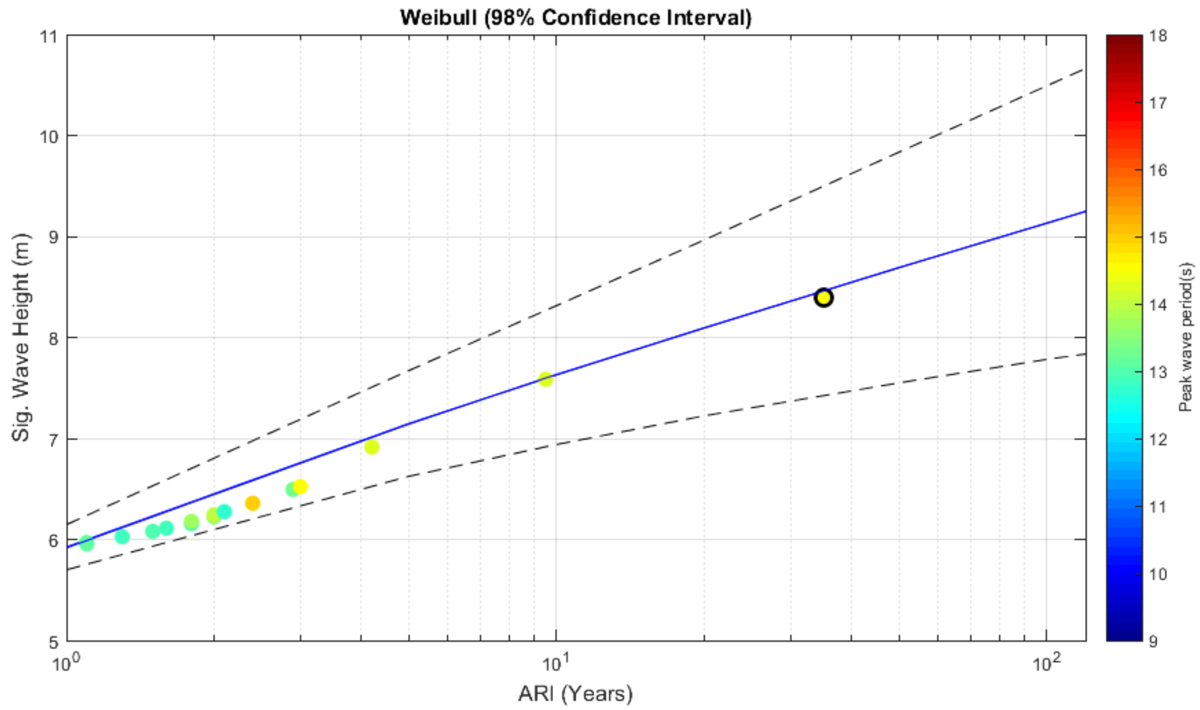


Figure 12: Results of EVA at the Port of Newcastle WRB.

It is noted that the estimated extreme wave heights at the Newcastle nearshore WRB are slightly larger than at the Sydney WRB (e.g., 100-year ARI wave height of 9.13m at the Newcastle nearshore WRB compared to 8.35m at Sydney offshore). This is the case despite the average wave climate being lower at Newcastle. Reasons for this believed to be:

- The observed period at the Newcastle nearshore WRB is only 14 years, while at Sydney it is 31-years. The last 2-years were a stormy period with more extreme wave events when compared to the average. Given the short-observed period, the recent storm period has a larger influence on the EVA results at Newcastle WRB. If an EVA is run on the Sydney WRB from the start of the Newcastle nearshore WRB to the latest available (March 2023) then higher extreme waves are estimated (e.g., 100-year ARI wave height of 8.64m). Moreover, there was a large wave event in May 2023 that was included in the analysis of the Newcastle WRB but not of the Sydney WRB.
- Not all the noise in the supplied 10-minute wave parameter timeseries was removed by the averaging applied, as described above.

When comparing the significant wave height and peak wave period for the extreme events, Figure 13 shows that most of the extreme events have peak wave periods between 10 and 15 seconds. This is typically higher that those observed for extreme events for other NSW WRBs (typically 12-13 seconds), it is believed that this is due to the same 10-minute sampling noise as for the significant wave height.

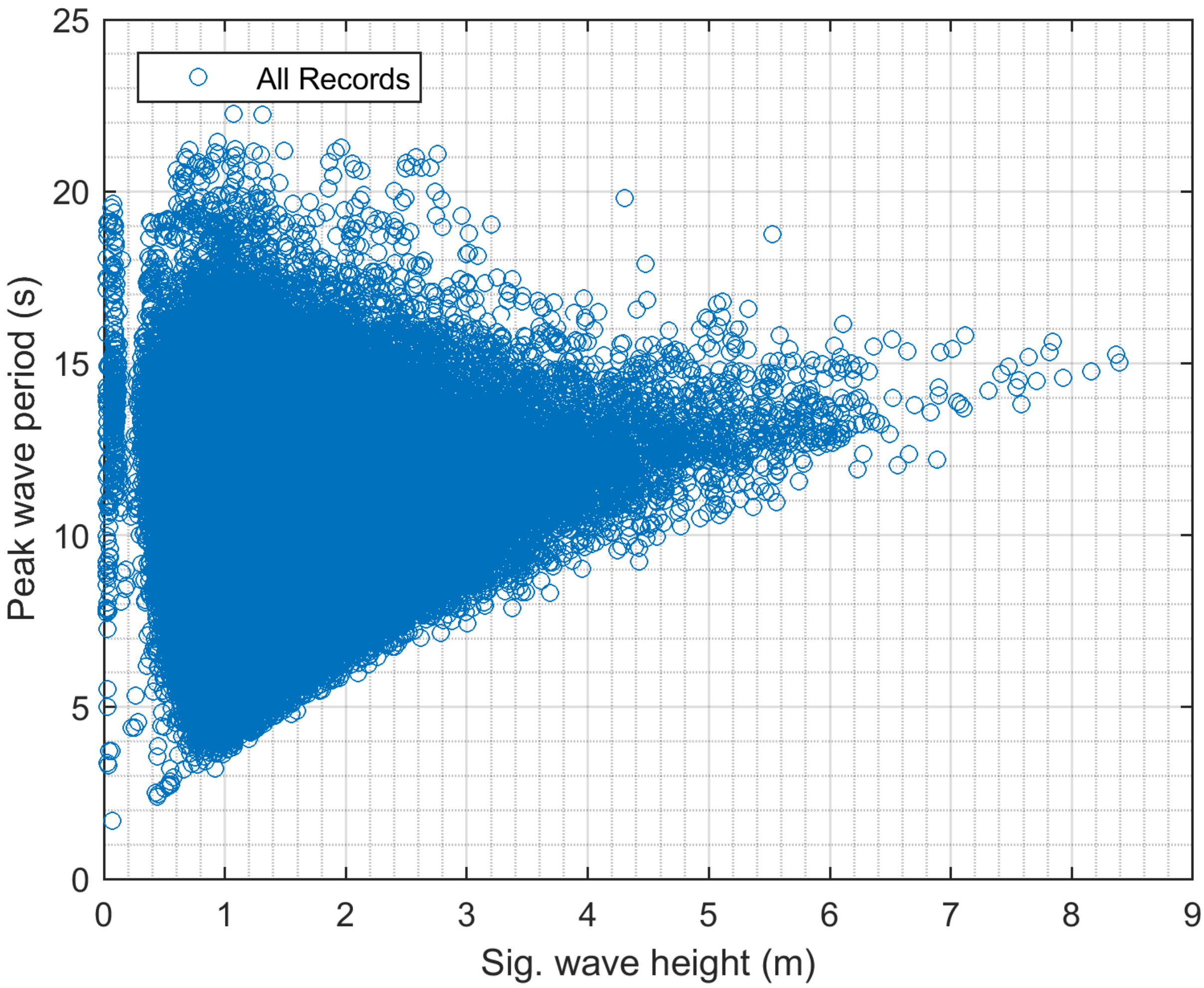


Figure 13: Peak wave period vs. significant wave height for the Newcastle nearshore WRB

4.4 Tides and other water level variations

Newcastle experiences semi-diurnal tides (two high and two tides each day) with significant diurnal inequality. Following a comparison of nearby tide gauges most representative of ocean water levels (i.e., Stockton Bridge, Shoal Bay and Patonga), the Patonga tide gauge was selected as being most representative of ocean water levels offshore of the Newcastle Ocean Baths. While the Stockton gauge is geographically closer, it may be influenced by river effects making Patonga a more appropriate selection. Tidal planes at the Patonga tide gauge are provided in Table 7. A mean spring tidal range of around 1.3 m are observed at Patonga.

Table 7: Tidal planes for Patonga tide gauge (source: MHL, 2023).

Tidal plane	Height (metres relative to AHD)
High High Water Solstice Springs (HHWSS)	1.032
Mean High Water Springs (MHWS)	0.691
Mean Sea Level (MSL)	0.059
Mean Low Water Springs (MLWS)	-0.574
Indian Spring Low Water (ISLW)	-0.817

Along the NSW coast, ocean water levels[3] can also be influenced by other non-tidal variations such as storm surge, coastal trapped waves and ocean circulation (see Section 2.1 for descriptions of these). The water level exceedance curve shown in Figure 14, shows the total range of water level variation measured at the Patonga gauge over the 30-year period observed at this site. When positive non-tidal water level variations combine with spring high tides extreme ocean water levels can result. Extreme value analyses of water levels measured at the Patonga tide gauge data was completed. The estimated ARI water levels are shown in Table 8. The highest recorded water level of 1.40m AHD occurred on 25 December 2016.

Table 8: Annual exceedance probability (AEP) water level from Patonga.

AEP	Water Level (m)	98% Confidence Limits (m)
63%	1.25	1.24 – 1.26
20%	1.33	1.31 - 1.36
10%	1.36	1.33 - 1.39
5%	1.39	1.36 - 1.42
2%	1.42	1.38 - 1.46
1%	1.44	1.40 - 1.49

[3] The term 'ocean water levels' is used to refer to water levels offshore of wave breaking. Inshore of wave breaking additional non-astronomical processes can also influence water levels including wave setup and wave runup.

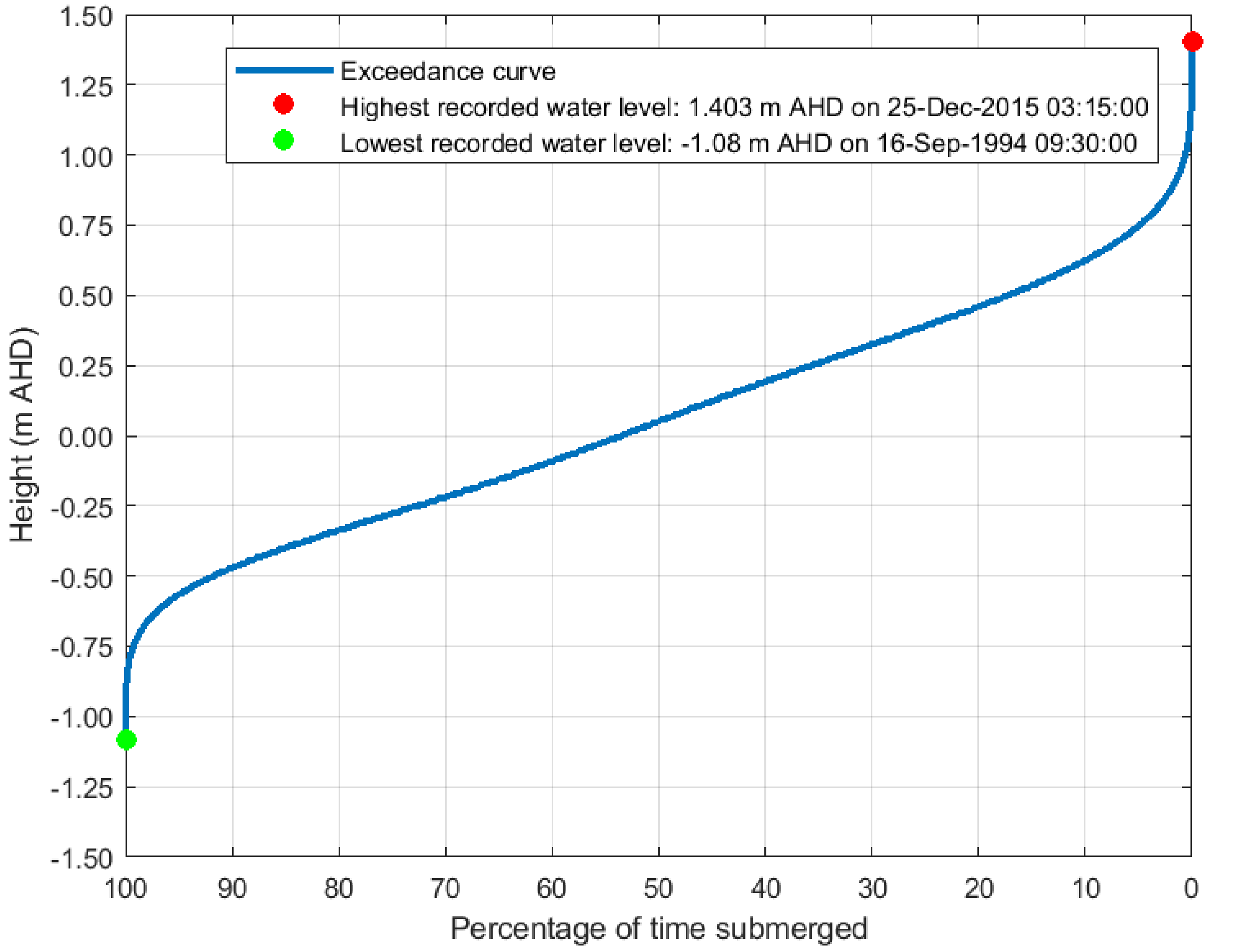


Figure 14: Water level exceedance curve for Patonga tide gauge.

4.5 Joint probability of waves and water levels

Analysis of the joint occurrence of observed significant wave heights from the Newcastle nearshore WRB and observed water levels at Patonga was undertaken to inform the selection of appropriate scenarios for the inundation assessment. The observed water level includes the still water level components of wind setup and inverted barometric setup but exclude any wave-driven contributions. The observed joint occurrences of the two parameters are shown in Figure 15 and suggests that there is a slight positive bias between the observed wave heights and water levels, i.e., larger wave heights often coincide with higher water levels.

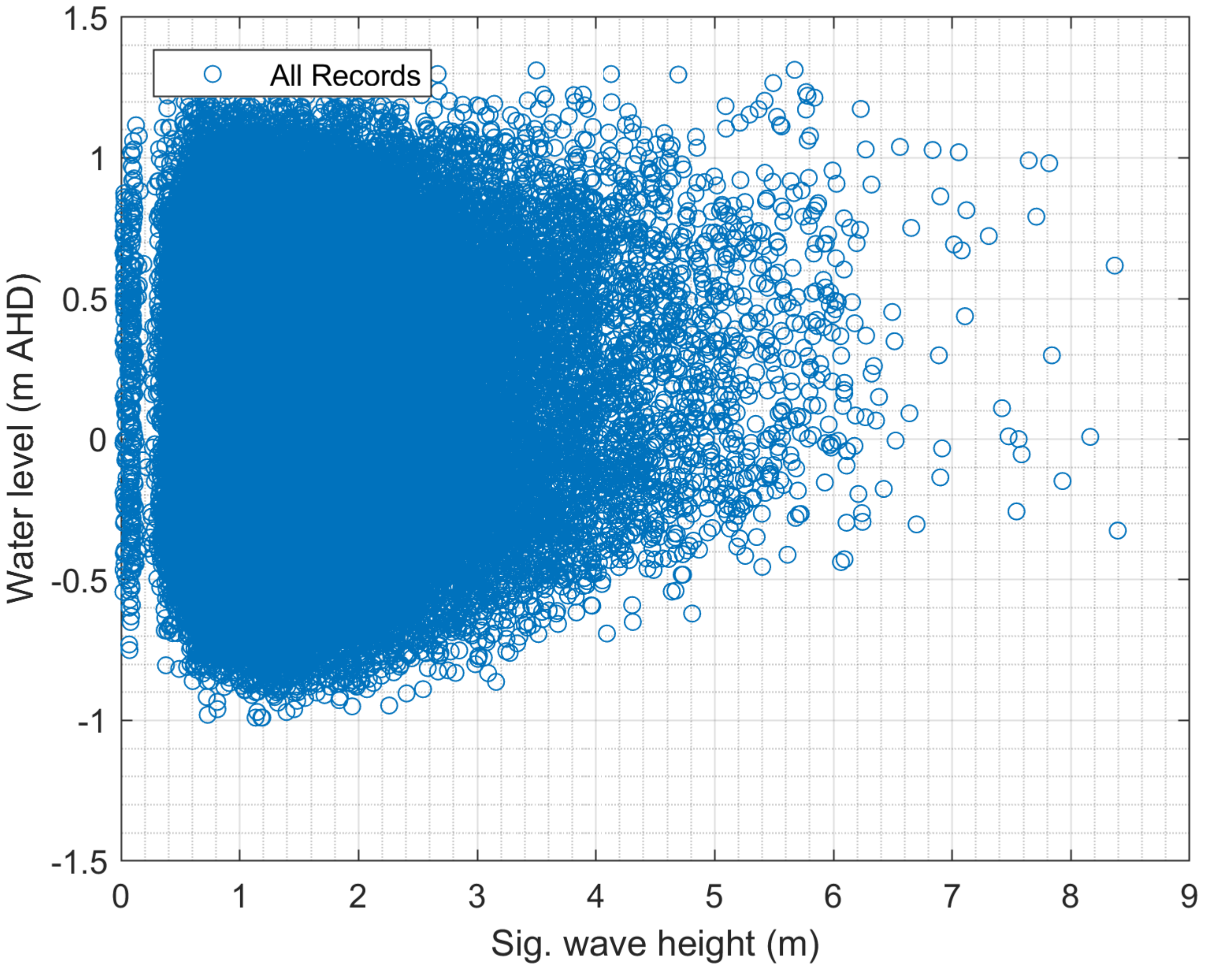


Figure 15: Joint occurrence of significant wave heights (Newcastle nearshore WRB) and water levels observed at Patonga (left) over 13.1-year WRB record.

The joint probability of extreme still water level and significant wave heights (or coincidence of the two) was calculated using a multivariate copula analysis. Copulas are mathematical functions that characterise the correlation structure among multiple time-independent random variables. The joint probability was calculated using independent extreme wave events (peak significant wave heights) as the primary variable and corresponding maximum water levels within a three-hour period before or after the peak in the storm wave conditions. The joint probability results are shown in Figure 16. The estimated joint AEP values are given in Table 9.

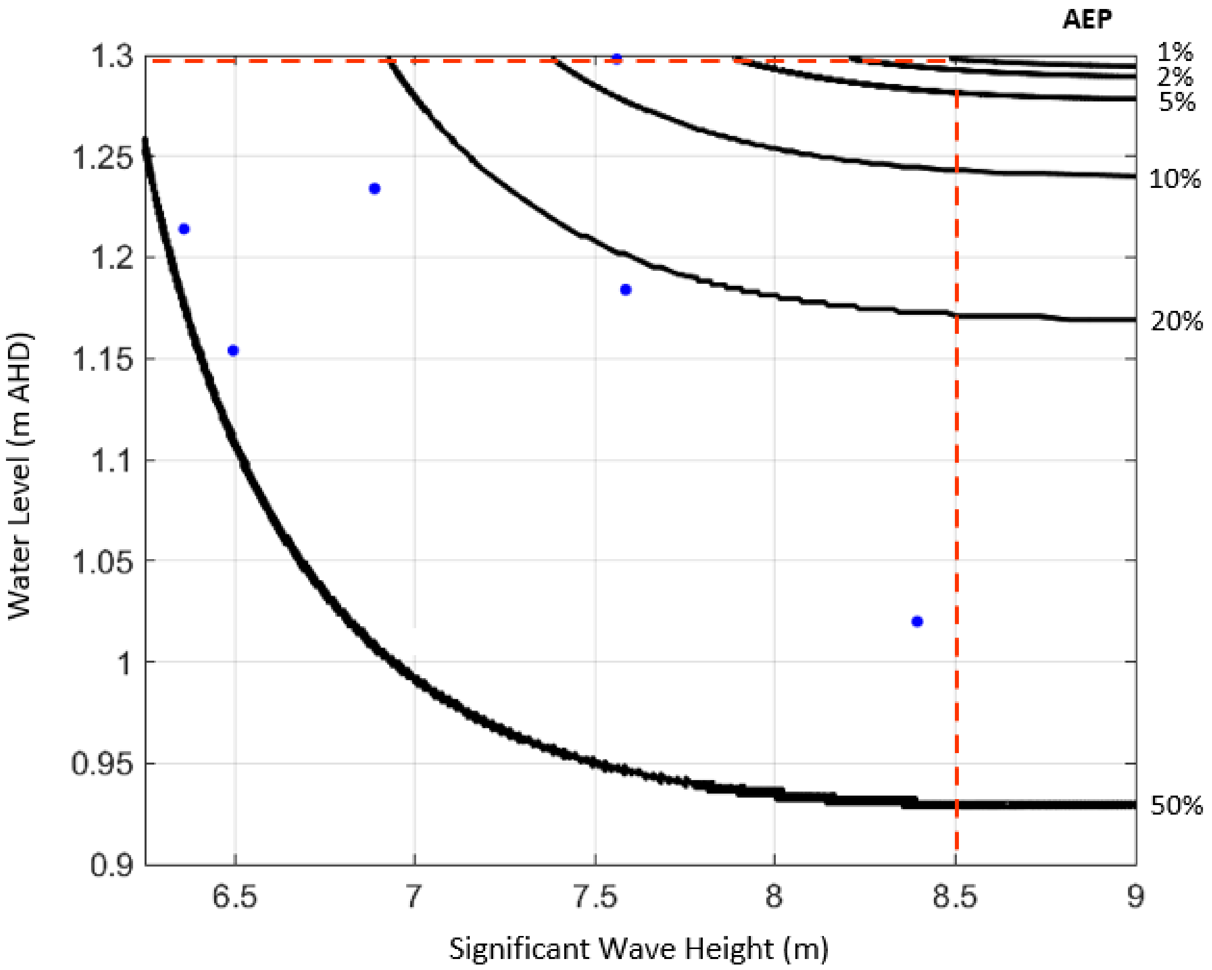


Figure 16. Joint occurrence of significant wave heights (Newcastle nearshore WRB) and water levels (Patonga) observed between 2013 and 2023.

Note: Multivariate return period isolines, as obtained from joint probability associated with a Galambos copula are shown in black. 1% AEP conditions are shown by the dashed red line. Blue dots shown represent some of the input conditions used for the analysis, axes of the figure have been limited to show isolines more clearly.

Table 9: AEP values for the joint probability analysis.

AEP	Hs (m)	WL (m)
50%	6.59	1.08
20%	7.29	1.23
10%	7.48	1.29
5%	7.77	1.29
2%	8.23	1.30
1%	8.50	1.30

4.6 Climate change and climate drivers

4.6.1 Introduction

Climate change related to anthropogenic global warming has become the main driver of climate change over the last century. This is due to the additional heat retained from increased amounts of carbon dioxide and other greenhouse gases from the global industrial way-of-life. Climate cycles are impacted by climate change. Sea level rise, which is already occurring because of climate change, is the main factor to influence future overwash and overtopping hazard at Newcastle Ocean Baths site. It's future contribution to the mean sea level at the site, based on the latest climate change projections, is described below.

The southeast Australian coastline is impacted by natural climate variability. Fluctuations in climate variability are natural and driven by oscillations in sea surface temperature and occur on seasonal, interannual and decadal periods. Over the yearly to decadal timescale, climate variability is largely due to changes in circulation patterns associated with Pacific Ocean climate drivers; El Niño Southern Oscillation (ENSO) and the Inter-decadal Pacific Oscillation (IPO). As ENSO is the largest contributor to climate variability it is described in section 4.6.3.

4.6.2 Sea level rise

Climate change results in sea level rise (SLR) around Australia as the sea level changes in response to fluctuations in ocean mass and the expansion of water as it warms. The contribution of melting of ice sheets is also predicted to increase into the future. Coastal inundation can occur because of SLR as the highest astronomical tide (highest tide of the year, approximately) will also rise relative to AHD and begin to exceed current inundation thresholds.

The latest advice from IPCC (AR6) on sea level rise describes significant sea level rise likely by 2100. The latest global SLR (above 1995 to 2014 baseline) projections for the 1.5°C, 2°C, 3°C, 4°C and 5°C warming scenarios are 0.44 m, 0.51 m, 0.61 m, 0.70 m and 0.81 m, respectively. The historical and projected global sea level rise trends are shown in Figure 17.

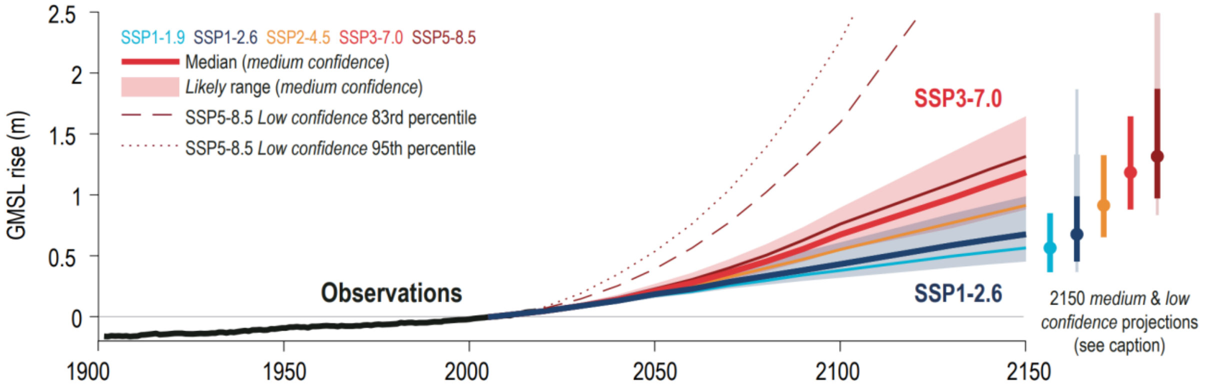


Figure 17: Global mean sea level rise (m) projections above the 1995 to 2014 baseline (IPCC, AR6) for various greenhouse gas emissions scenarios.

IPCC (AR6) assesses the climate response to five illustrative scenarios that cover the range of possible future development of anthropogenic drivers of climate. The report concludes that in the longer term, sea level is committed to rise for centuries to millennia due to continuing deep ocean warming and ice sheet melt and will remain elevated for thousands of years. In the shorter term, it is certain that global mean sea level will continue to rise over the 21st century.

The latest SLR (above 1995 - 2014 baseline) projections for Newcastle[4] for the 'likely' mean SLR ranges (17th to 83rd percentiles) by 2100 are given below (refer to Figure 18):

- 0.38m (0.25-0.57m) under the very low greenhouse gas (GHG) emissions scenario (SSP1-1.9[5])
- 0.43m (0.30-0.62m) under the low GHG emissions scenario (SSP1-2.6)
- 0.57m (0.43-0.79m) under the intermediate GHG emissions scenario (SSP2-4.5)
- 0.72m (0.55-0.96m) under the high GHG emissions scenario (SSP3-7.0)
- 0.82m (0.63-1.11m) under the very high GHG emissions scenario (SSP5-8.5).

The adopted sea level rise values for the coastal inundation assessment are presented in Table 10 (extracted from IPCC AR6 Sea Level Projection Tool).

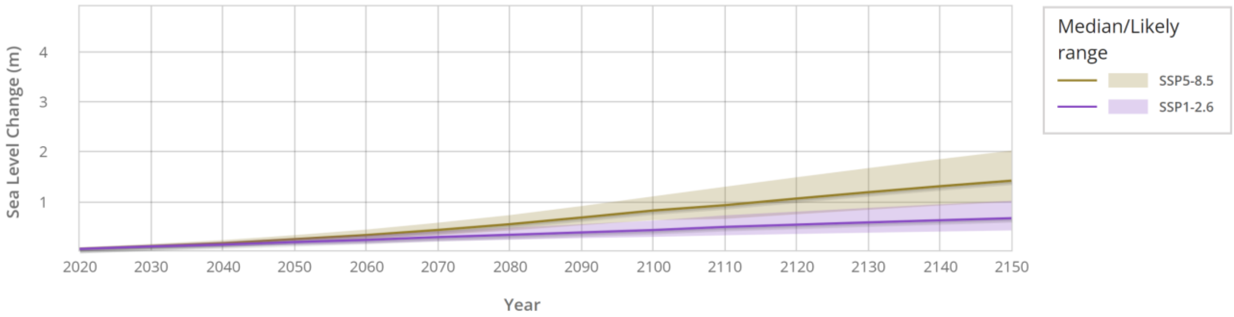


Figure 18: IPCC AR6 sea level rise projections (for Newcastle, NSW) relative to 1995 - 2014 baseline for the low and very high future greenhouse gas emission scenarios (Garner et al., 2021).

Table 10: Sea level rise projections in metres relative to 1995 - 2014 baseline (for Newcastle, NSW).

Quantile	2050	2100	2120	Comment
5th percentile	0.10	0.22	0.27	SSP2.6
17th percentile	0.14	0.30	0.36	SSP2.6
50th percentile	0.23	0.71	0.93	Mid-value between above (17th %ile) and below (83rd %ile) values
83rd percentile	0.33	1.11	1.49	SSP8.5
95th percentile	0.40	1.35	1.82	SSP8.5

The 50th percentile SLR projection has been adopted for used in the inundation scenarios presented in Section 5.5.1. This scenario is considered as it provides a pragmatic approach to adopting realistic SLR projections.

[4] Regional SLR projections are available for Sydney and Newcastle. Conservatively, Newcastle was adopted in this study due slightly higher values compared to Sydney.
[5] Shared Socioeconomic Pathways (SSPs) are scenarios of projected socioeconomic global changes up to 2100. They are used to derive greenhouse gas emissions scenarios with different climate policies.

4.6.3 El Niño Southern Oscillation

The ENSO is the oscillation in sea surface temperature in the central and eastern Pacific that leads to seasonal and interannual shifts (typically 18 months) in weather patterns and circulation in across the Pacific. The state and intensity of the ENSO impacts the intensity of trade winds, mean sea level, frequency and intensity of storms. The Southern Oscillation Index (SOI) is often used to determine if the climate is in a state of El Niño or La Niña and is calculated from monthly and seasonal fluctuations in air pressure. El Niño in Australia, indicated by a sustained negative SOI, is associated with drier conditions, a reduction in rainfall, and a decrease in the strength of the easterly trade winds. La Niña, indicated by a sustained positive SOI, is linked to stronger easterly trade winds, increased cloudiness and increased risk of tropical cyclones across northern Australia and ECLs in southeast Australia. Along the east Australian coast, storm events have been found to show strong correlation with the state of ENSO, with storm wave direction, long term mean sea level and the rate of storms being impacted (Davies et al, 2017). Typically, during El Niño events waves are bi-directional with southeast and easterly waves conditions. La Niña events are associated with a unidirectional south easterly wave climate (Mortlock and Goodwin, 2016) and are linked to higher rates of upper beach erosion for the East Australian Coast (Silva et al, 2021). Recent erosion along Newcastle's southern beaches has been associated with a La Niña period.

The variability in ENSO (SOI) in Australia over the last 10 years is displayed in Figure 19.

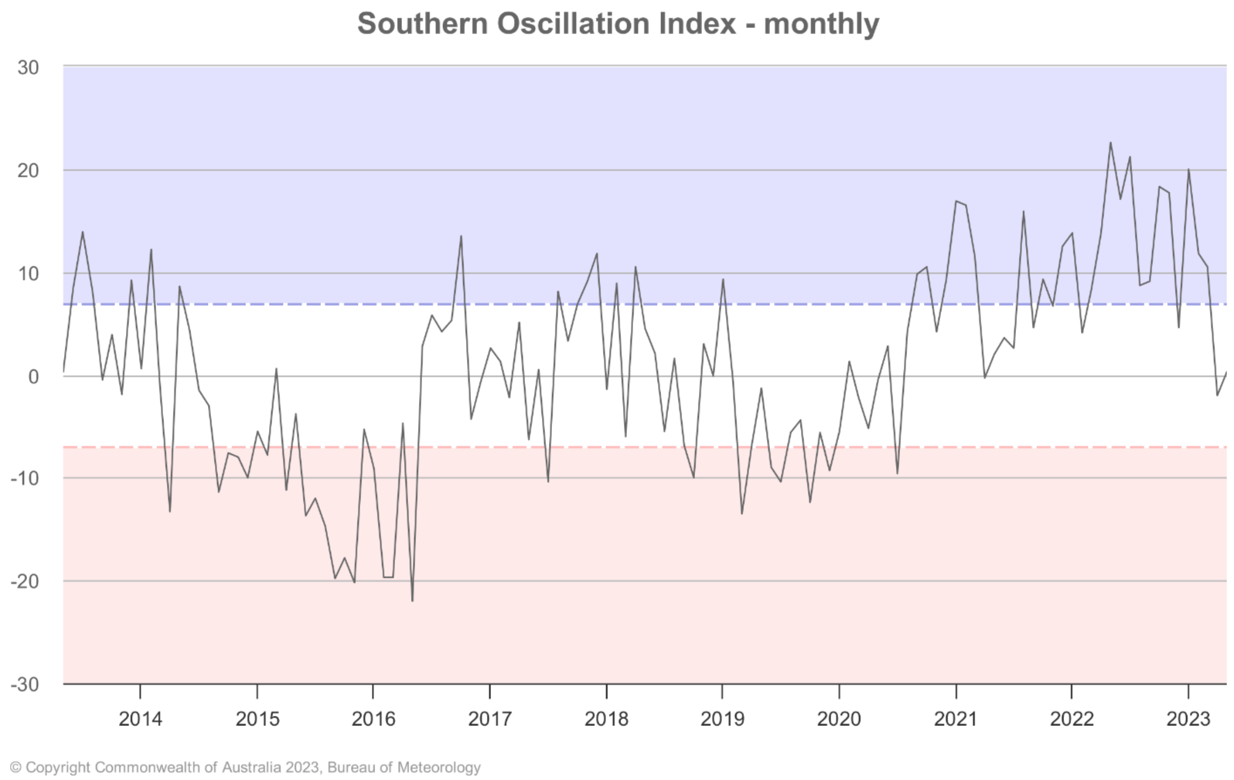


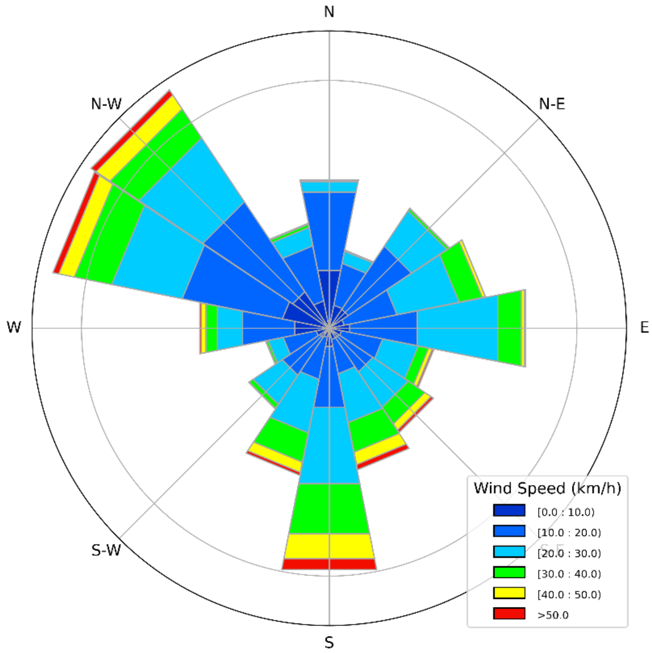
Figure 19: ENSO (Southern Oscillation Index) between 2013 and 2023 (source: Bureau of Meteorology). La Niña threshold is shown in blue, and the El Niño threshold in red.

The variability of the wave climate because of the shift between El Nino and La Nina given above shows the variability both annually and seasonally. As for long term projection of wave climate trends, there is no definitive resource that provides quantified projections that could be incorporated into future scenarios for this study. As such, the wave climate adopted in this study are those at present day based on available measurement records. The WRB data available from Sydney and Newcastle are only 31 and 13 years respectively which is insufficient to make any predictions of how the wave climate may vary in the future.

4.7 Wind climate

An analysis of measured wind records from 2001 until 2023 at Bureau of Meteorology (BOM) Nobbys Head weather station was carried out. Overall and seasonal wind roses for Nobbys are presented in Figure 20.

Over the summer period, winds predominantly arrive from the north-east to the south with the highest wind speeds coming from the east to northeast associated with East Coast Low (ECL) systems. Whereas over the winter and autumn synoptic periods, winds are predominantly from a north-westerly direction and associated with anticyclones (high pressure systems) across southern Australia as well as light morning land breezes.



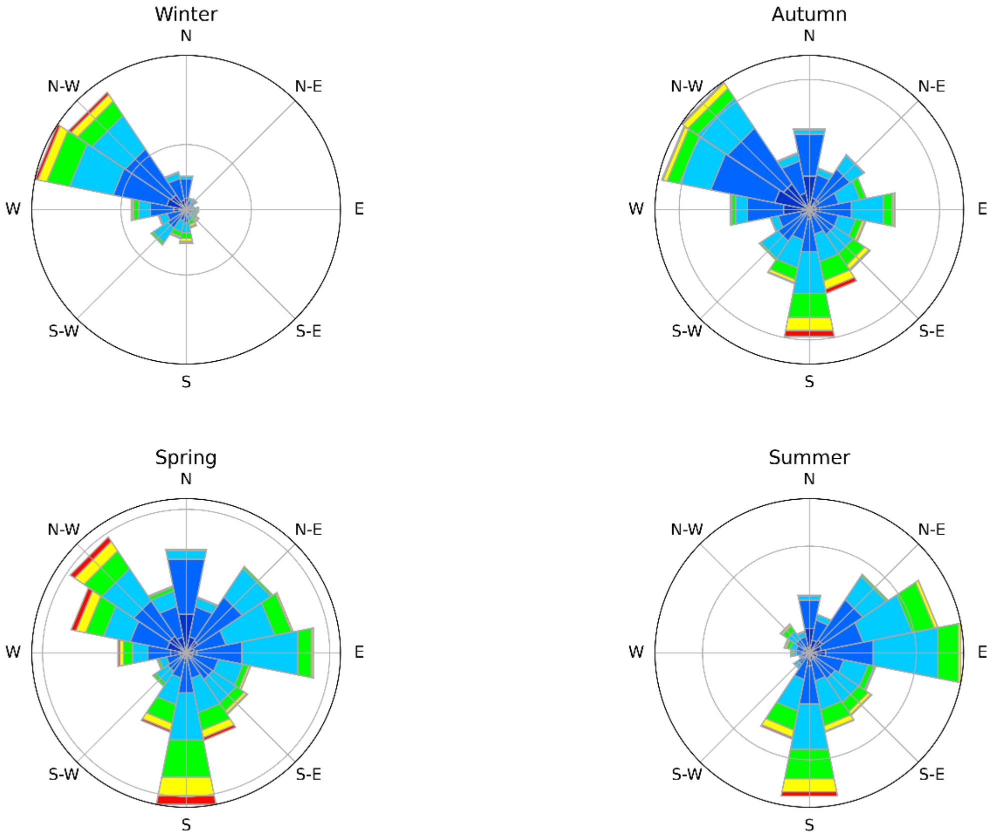


Figure 20: Annual (top) and seasonal wind roses at Nobbys Newcastle BOM weather station.

Note: The grey circle represents the 10% probability contour, i.e., wind roses that pass the 10% line occur more often than 10% of the time for that period.

4.8 Inundation data

4.8.1 Community imagery and contractor's log

To assist in initial calibration of the numerical model, CN sourced photos and videos of inundation events from the community. An analysis was completed of the wave and water level conditions at the time the photos and videos were taken to (i) provide an insight into the wave overwash and overtopping process occurring under a range of events and (ii) as a qualitative measure by which to calibrate and validate the model simulations. Photos or video were provided for seven (7) separate inundation events ranging in intensity from minor (pool closing) to major overtopping events.

Construction logs provided by Stage 1 contractor Daracon provided an additional point of reference for these events. A summary example of community and contractor data from one event on the 24 August 2022 is shown in Figure 21.

Event 6: 24 August 2022

- Downtime from 'large swells' logged on 24-25 August
- 4 photos all on the south-side of complex

Source: M. Ware 24/08/2022

Construction log

AUGUST 2022 – 4 DAYS LOST FOR THE MONTH			
9th August 2022	Rain All Day		1.0
10th August 2022	Rain in the morning		0.2
11 August 2022	Large Swells – Pool flooded in the morning		0.3
23 August 2022	Prepare for Large Swells – Low Pressure System		1.0
24th August 2022	Large Swells – Low Pressure System – pool flooded		1.0
25th August 2022	Large Swells – Low Pressure System – pool flooded in the morning		0.5

Maximum wave conditions (any tide)

Hs (m)	Tp (s)	Dp (deg)	Tide level (m)
3.04	10.7	151	0.21

Wave conditions at highest tide

Hs (m)	Tp (s)	Dp (deg)	Tide level (m)
2.08	9.63	147	0.94

Figure 21: Example community imagery, hindcast waves and construction logs.

Comparison of the inundation extents produced by the model were compared to the provided media hindcast wave conditions from the MHL nearshore wave transformation hindcast.

4.8.2 Project specific inundation monitoring

While the above data was useful in the early stages of calibration, no high-quality measurements of wave, water level, flow or overtopping during a coastal inundation event at Newcastle Ocean Baths was available. To address this gap and improve the project outcomes, a coastal inundation monitoring exercise was undertaken during a large swell event that occurred in early May 2023. The objectives of the inundation monitoring were to:

- Improve the understanding of wave, flow and overtopping dynamics during large wave events with high water levels.
- Improve the calibration and/or validation of the XBeach numerical model being developed and thereby improve the confidence in the inundation assessment.

Using lessons learned from the qualitative calibration, key locations and parameters were identified for a data collection activity to further assist in model calibration and validation. These included:

- Measured offshore wave height and water level to be used as model inputs.
- Wave height and water level in the centre of the pool.
- Wave height and water level on the pool deck (in front of pavilion deck wall).
- Discharge rate at the northwest corner of the pool where water drains into Cowrie Hole.
- Timestamped video from several locations to compare the readings from the instruments to examine impacts on the existing buildings.

The scope, approach and results of this targeted data collection exercise is detailed in Appendix A:. Examples of the conditions captured are provided in Figure 22.

Figure 22. Example images captured at the morning high tide (left, bottom right) and evening high tide (bottom right).

5. Numerical modelling

5.1 Modelling approach

The numerical wave model XBeach-NH was used to simulate wave generated inundation at the Newcastle Ocean Baths. XBeach-NH is a non-hydrostatic model that resolves the water level and velocity of individual waves (phase-resolving) and wave induced surf zone processes that contribute to coastal inundation, such as wave setup and infragravity waves. The model solves the dynamic interaction between these different processes and the underlying bathymetry. For a full description of the model, refer to the manual and key reference papers (Zijlema et al., 2021; McCall et al., 2014). An example of the model application to the Newcastle Ocean Baths site is in Figure 23.

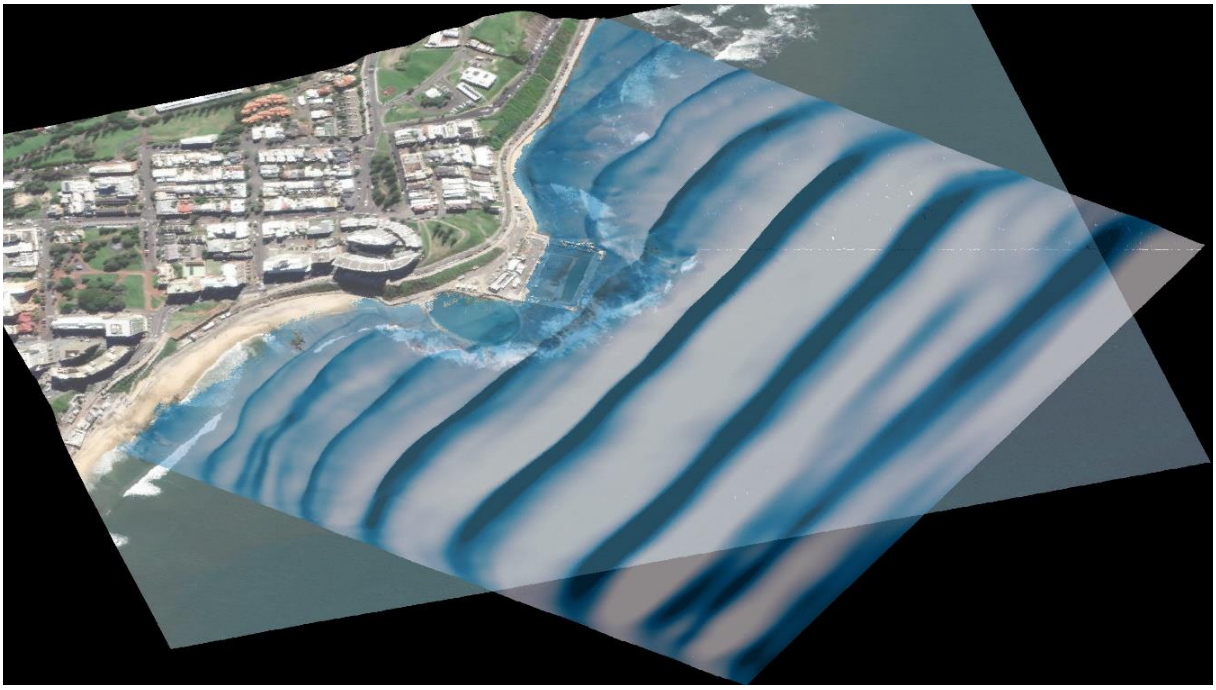


Figure 23: Example of the XBeach-NH model applied to Newcastle Baths, showing waves approaching the coast and breaking on the rocks before interacting with the bath's facility.

The modelling approach adopted for the Newcastle Ocean Baths involved the following steps:

- **Establish model**
  - design topography and bathymetry
  - wave and water level conditions at seaward boundary
  - evaluate model sensitivity and stability.
- **Calibration / verification**
  - 1D calibration on overtopping data set on idealised rock platform (reported internally)
  - 2D validation to recent events for initial model development, which was superseded when site specific measurements were obtained (see Section 4.8.2)
  - 2D validation of model using project data collection during the event on 8th to 10th of May 2023 (see Appendix A:).
- **Application to inundation hazard assessment**
  - extreme conditions based on select annual exceedance probabilities and sea level rise scenarios.
  - inundation scenarios across a range of present day and future scenarios for wave conditions and water level.
  - application to assess preferred options for reducing inundation effects.

5.2 Model workflow

The modelling workflow used for all simulations is presented in Figure 24. Key model parameters, inputs, and outputs are shown. The model schematisation used a quasi-3D implementation by resolving two (2) vertical layers and two (2) horizontal dimensions. The model bathymetry is a 2m x 2m resolution elevation ground model (i.e., buildings and non-terrain objects are excluded) that extends 850 m by 900m. More details of the model bathymetry are presented in Figure 24 below.

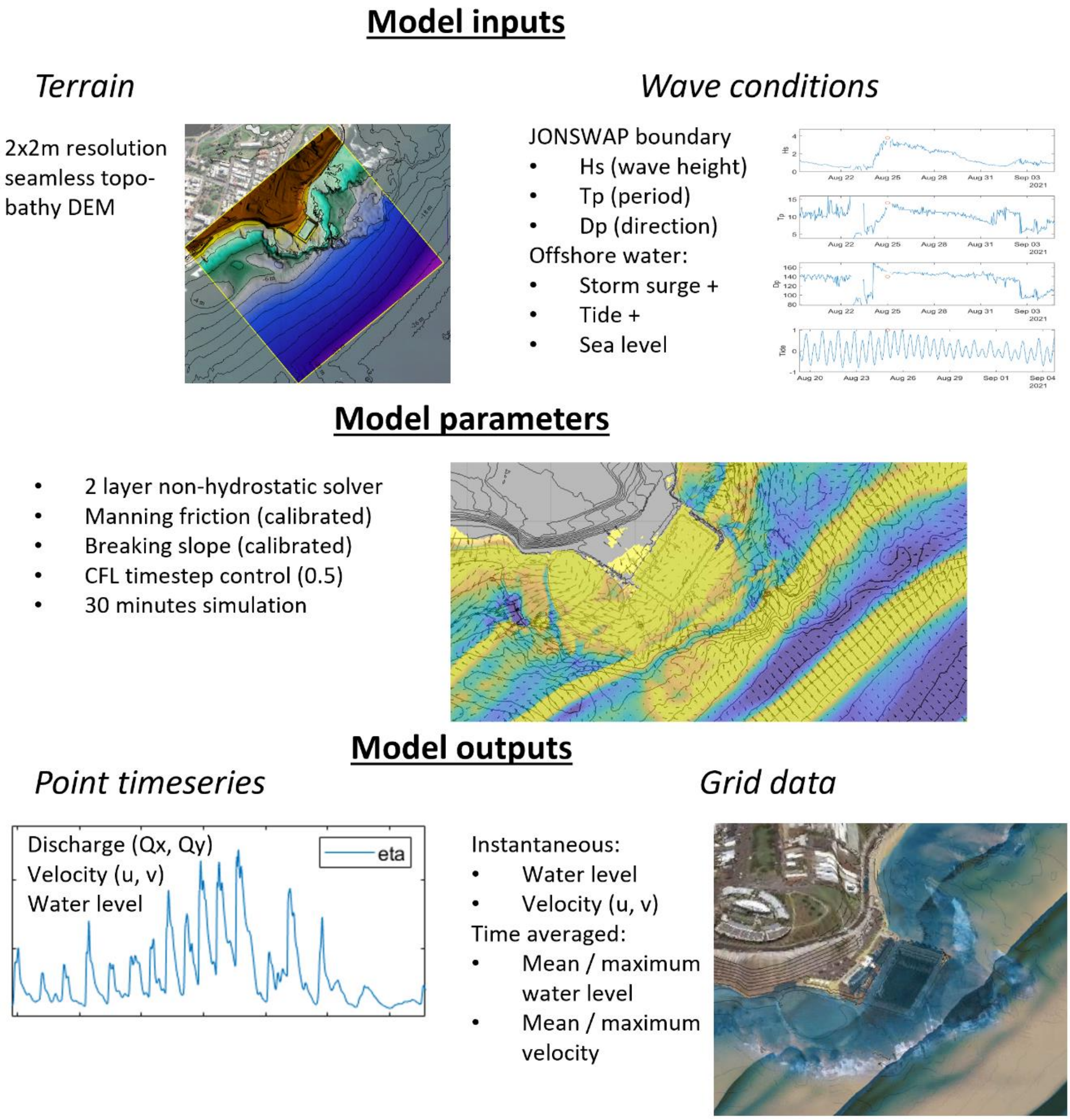


Figure 24: Example of the XBeach-NH model workflow.

5.3 Bathymetry development

Three different topographic and bathymetric survey sources were used to establish the baseline model for pre-renewal project conditions of the baths site and surrounds. All vertical references are in Australia Height Datum (AHD) referred to as reduced level (RL). The sources were used in the following hierarchy from highest level to lowest level:

1. Ocean Baths topographic survey (0.5m gridded resolution)
2. Newcastle topographic LiDAR (0.5m resolution)

3. Topo-bathy DEM derived from 2018 NSW Coastal LiDAR Project (5m resolution)

All terrain layers were processed using EPSG code of 28356 (GDA 94 MGA zone 56). The process of combining the different layers, and smoothing transitions is presented in Figure 25.

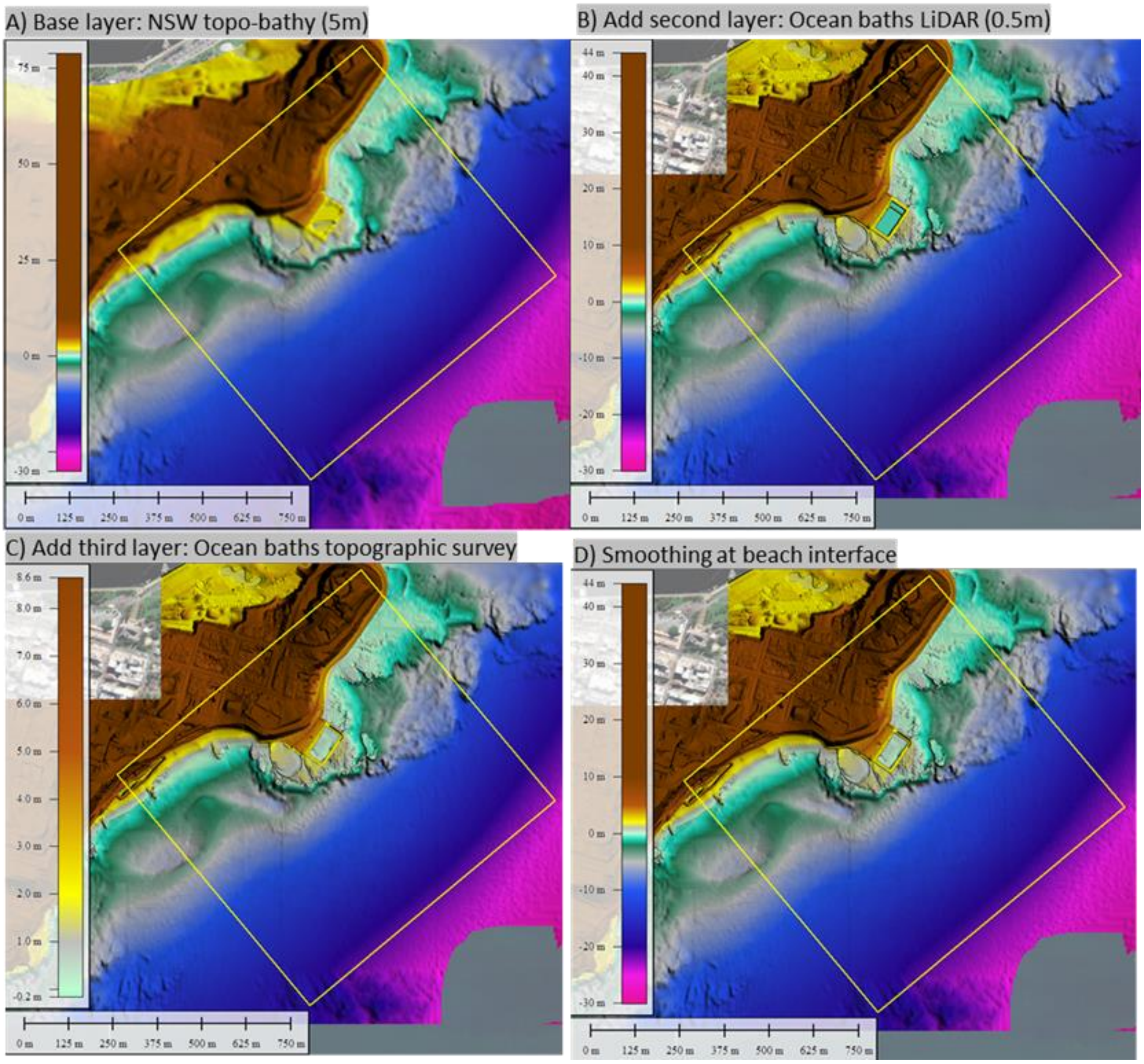


Figure 25: Construction of the XBeach-NH bathymetry using the three primary data sources.

The XBeach-NH model used for calibration was implemented at a resolution of 2m with a rotated bathymetry that aligns with the orientation of the -20m bathymetry and prevailing wave direction. To create this, the 0.5m resolution regular ASCII grid was rotated and interpolated to 2m (Figure 26). The interpolation to 2m resolution results in minor smoothing of steep sections of bathymetry, such as vertical walls at the pool complex (Figure 26). This is considered appropriate for the phase-resolving non-hydrostatic model being used.

The final elevations used for XBeach model description does not include buildings. This is the recommended approach for assessing flow depth and velocity at the building platform area as it removes interference from reflection that could mask the incident flow. That lateral boundary of the XBeach-NH model is a 50m thick sponge layer. Near the sponge layer boundary, the model's elevations are flat to optimise dissipation and avoid instabilities (e.g., the across-shore profile 20m landward of the boundary is repeated to the boundary).

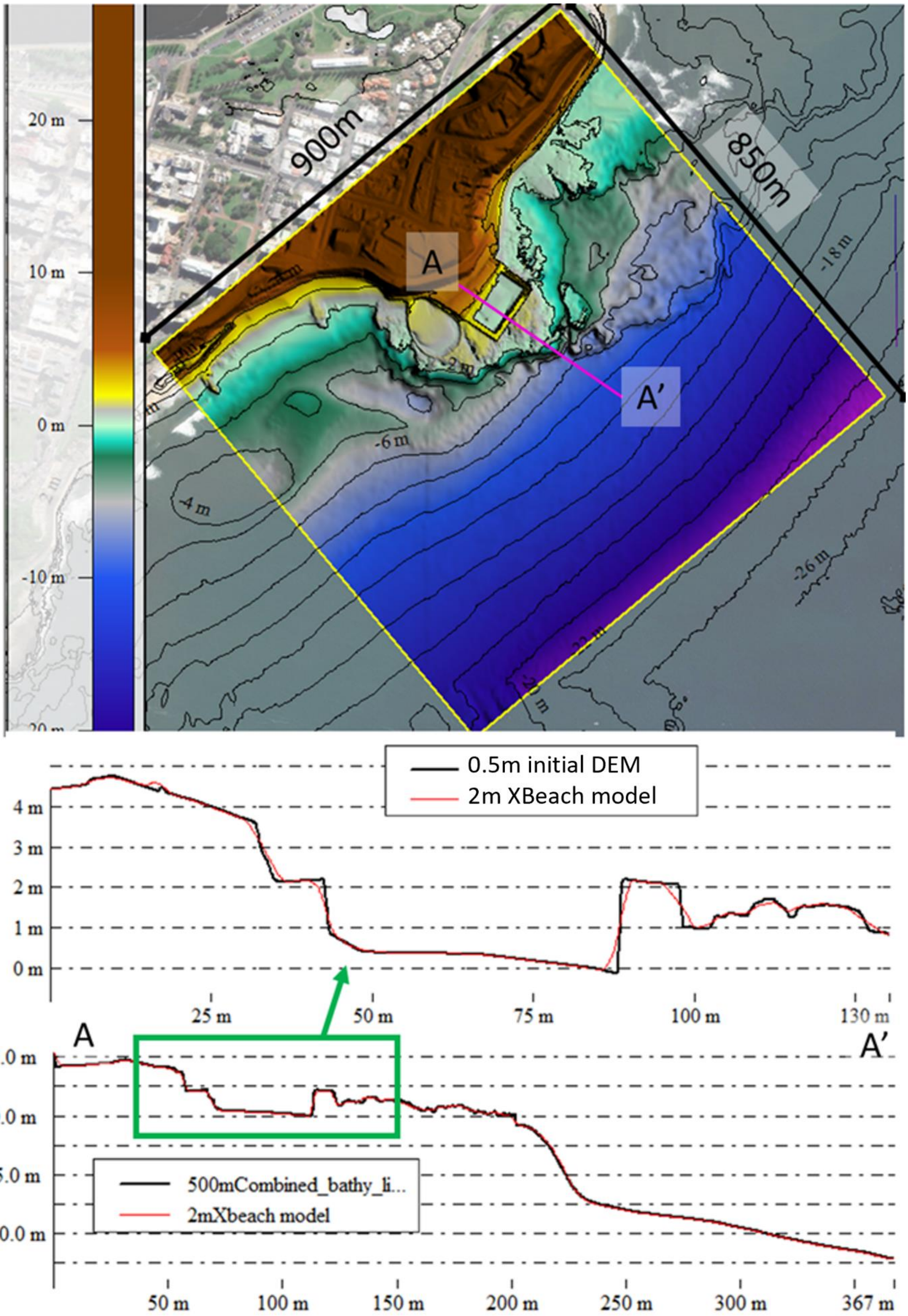


Figure 26: Comparison of the supplied 0.5m DEM and the 2m DEM used in XBeach-NH.

5.4 Model validation

5.4.1 Overview

The observations collected during the early May 2023 swell event provided a good dataset for verifying that the numerical model can resolve the processes contributing inundation and wave effects across the baths site including at the pavilions. The inundation monitoring collected data over two days, which is summarised by 56 bursts representing a 30 min timestep. The event collected offshore wave height, wave period, and direction at a nearby wave buoy to provide boundary conditions for simulating conditions in the numerical model. For more detail on this targeted inundation monitoring see Appendix A:.

For the early May 2023 inundation event simulations, the models' elevations were based on the topographic survey of the baths (Figure 26) which did not have the Stage 1 construction features for the pool deck and raised platform where the pavilions are located. A close-up of the pool complex bathymetry and measurement locations is presented in Figure 27 below.

Model behaviour was assessed by comparing model outputs for water level at the RBR deployed on the pool deck, and at ADCP location in the pool centre. No suitable data was available for comparing flow velocities. The model's ability to simulate the occurrence of wave overtopping on the pavilion deck was tested using the camera installed during the event. The landward extent of inundation was verified against drone imagery captured during the event.

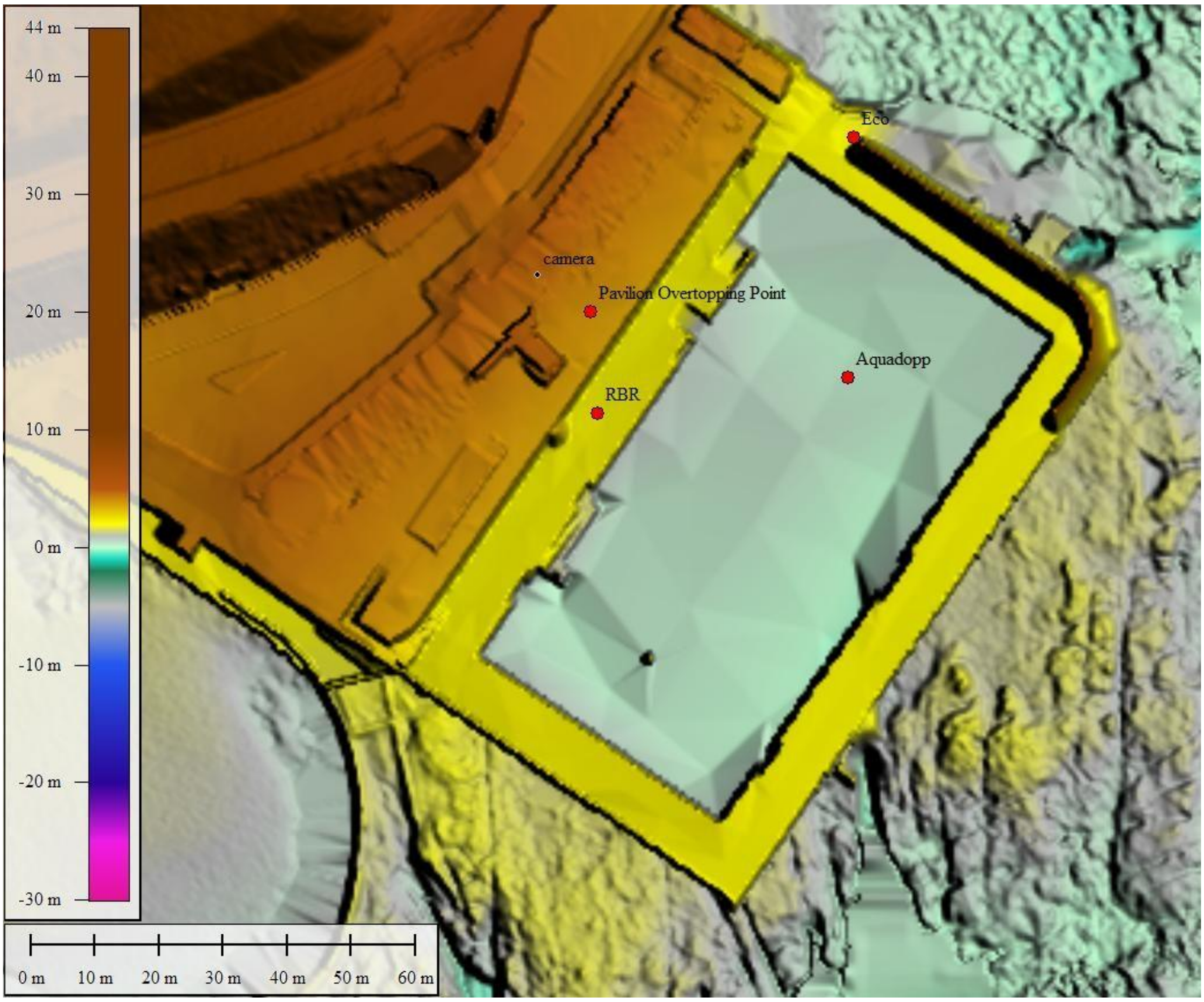


Figure 27: Model bathymetry and key wave output locations. Note that the bathymetry is based on the topographic survey.

5.4.2 Summary of observations

Measured water levels at the RBR and ADCP (Aquadopp) were processed to understand the mean water level, 1% and 2% exceeded water level for comparison to the model outputs. Wave overtopping onto the pavilion deck level was identified by analysing the recorded camera video into 'time stacks'. The time stacks can then be used to count the individual wave overtopping events in each half hour burst. An example time-stack used to detect overtopping is presented below in Figure 28. Note that the time stack is based on slicing one vertical frame from each timestep in the hour-long video and stacking these in time (plotted on the x-axis) to identify when white water was present on the pavilion deck level. For more details on the analysis method refer to this open-source repository Surf Zone Fun.

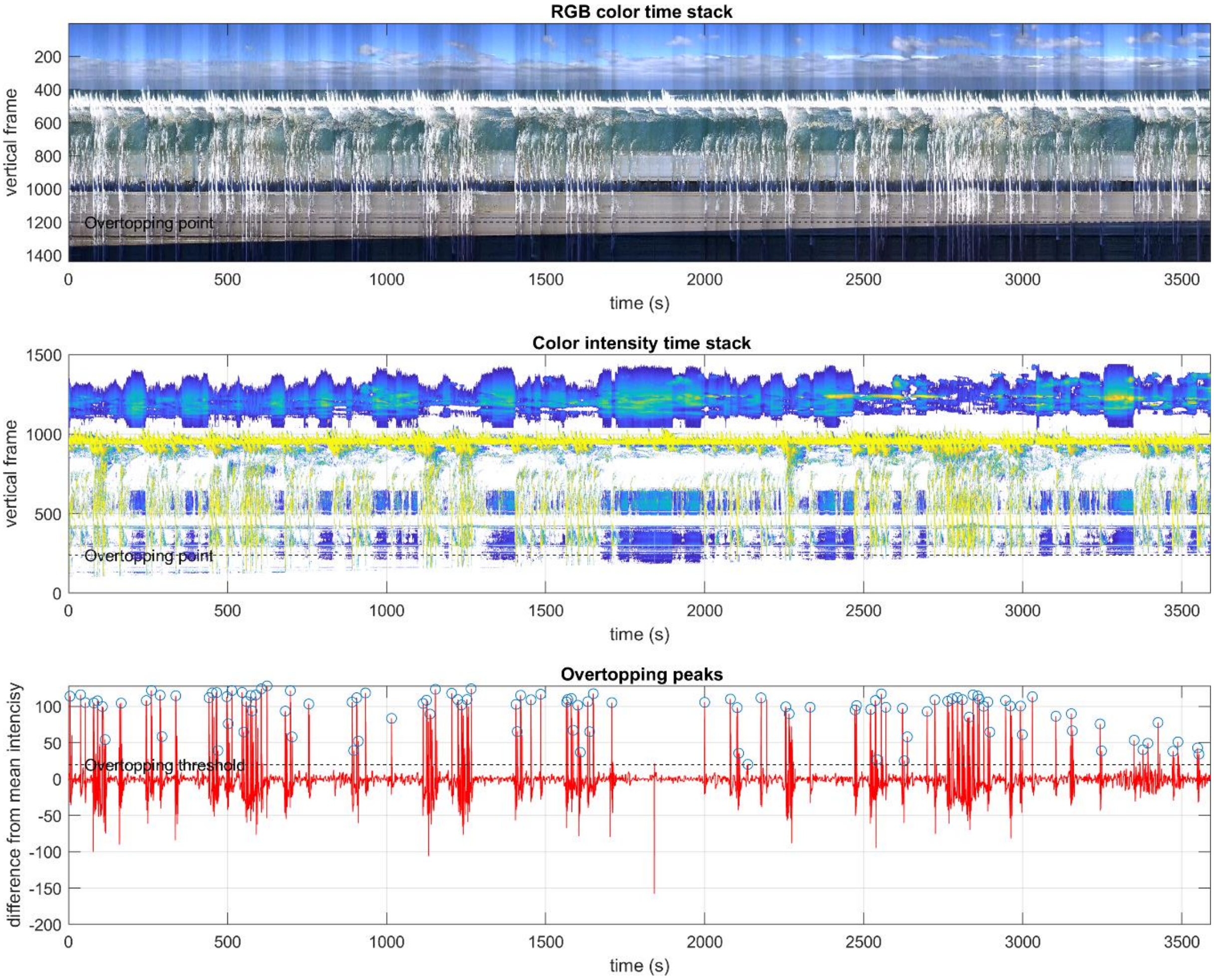


Figure 28: Example time-stack analysis of pavilion deck camera used to detect wave overtopping events (from 11am on 8th May 2023).

The measured conditions during the event are summarised in the plots in Figure 29. The plots show the two high tides over the wave event, with significant wave heights at the nearby Newcastle nearshore WRB of around 6m with long wave periods. Elevated water levels were recorded in the pool and on the lower pool deck level, as indicated by the mean, 1% and 2% exceed water level plots. Note that these levels are based on high-frequency sampling, so the metrics capture the wave crest level above the surge height, which is a good indicator of hazardous flow conditions. Wave overtopping was detected on the pavilion deck level during both high tides with some splashing occurring at the super elevated low tide.

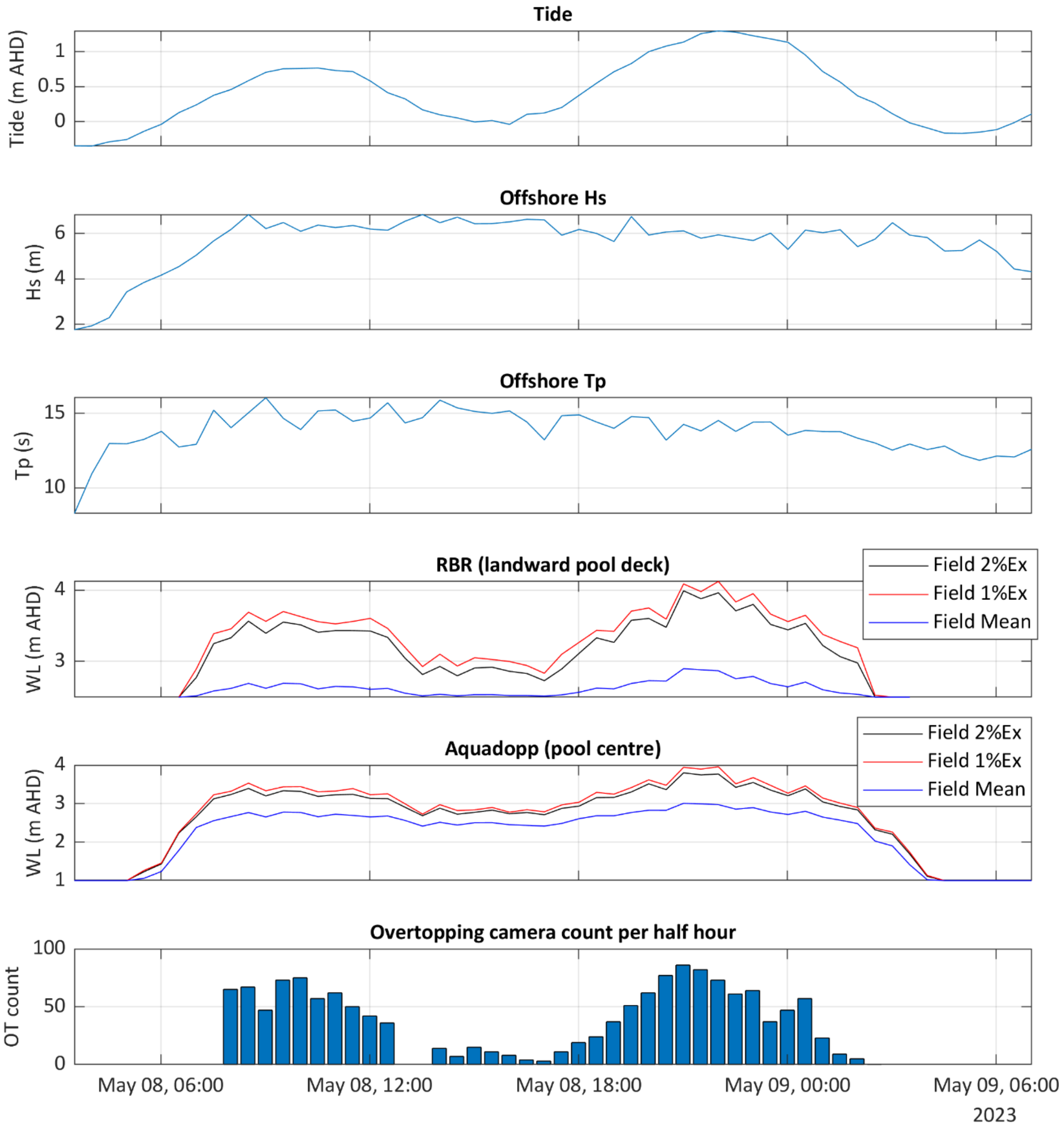


Figure 29: Field measurements used for model validation.

5.4.3 Replicating the event in XBeach-NH

All 56 half hourly bursts measured on 8th -9th May 2023 were replicated in the XBeach-NH model. Wave output points at the RBR location, ADCP location and on the pavilion deck level to detect overtopping events. Each simulation was run for 20 minutes to capture the offshore water level and wave conditions.

The model performance was assessed by comparing water level observations form the model and field observations, as summarised as a timeseries in Figure 30 and scatter plots Figure 31 below. This shows a slight over-prediction of extreme water levels at the ADCP but a good comparison to the extreme water level at the RBR location. The comparison between observed and modelled was closer for the 1% and 2% exceeded water level, while there was a wider scatter for comparing maximum water level. This is expected, as maximum water level is sensitive to multiple co-incident processes including interacting wave frequencies and reflections. The 2% and 1% exceeded water level also provide a more robust indicator of wave and inundation hazards, which are similar to how wave runup is generally characterised as a 2% exceeded value. The presence of overtopping on the pavilion deck level is also closely matched to overtopping events captured by the camera. Initial simulations identified that a default Manning coefficient of 0.02, and a breaking slope threshold of 0.8 are suitable for the site. Outputs are not sensitive to these parameters.

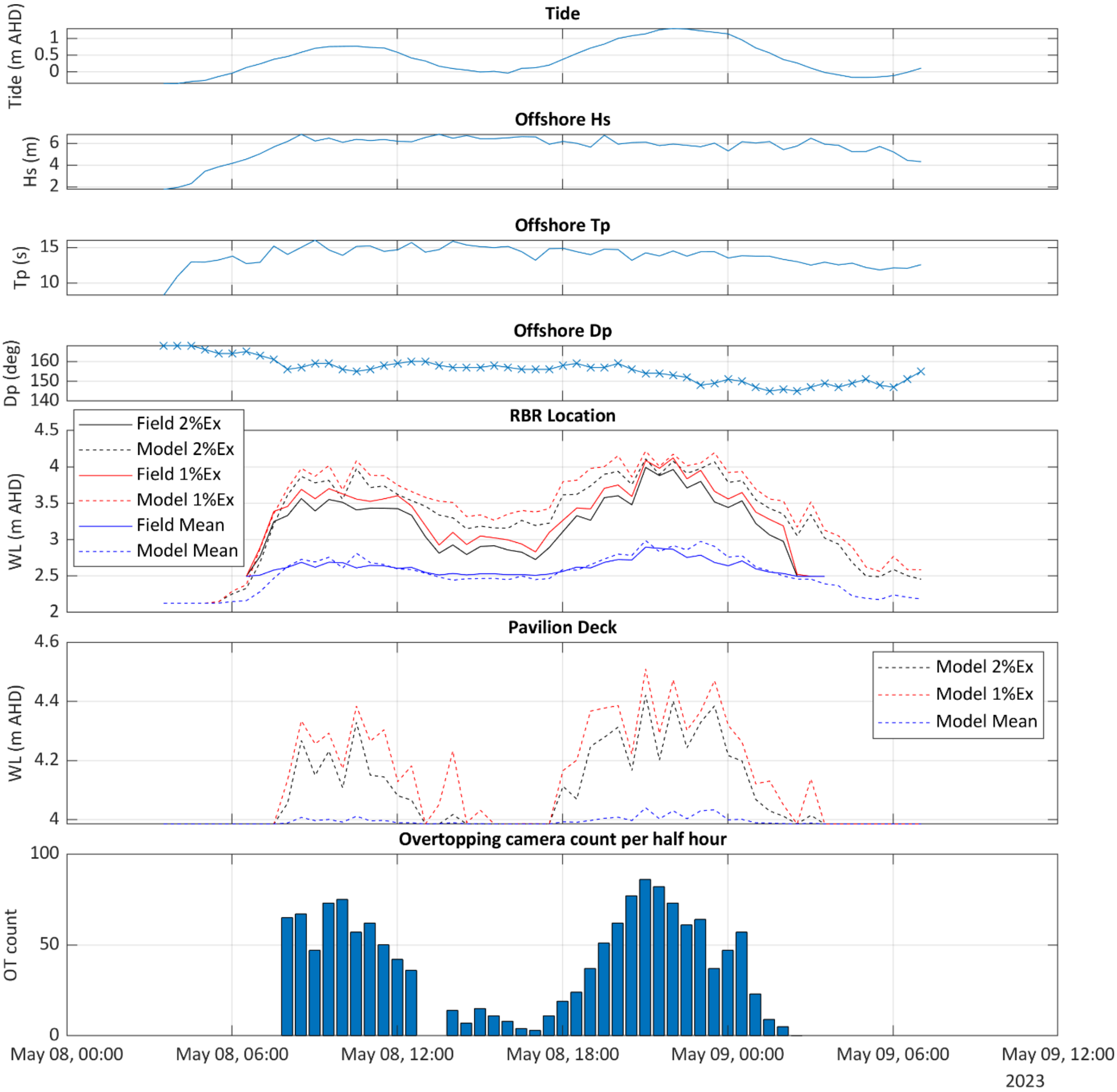


Figure 30: Timeseries comparison of measured and modelled conditions during the 8-9 May 2023 event.

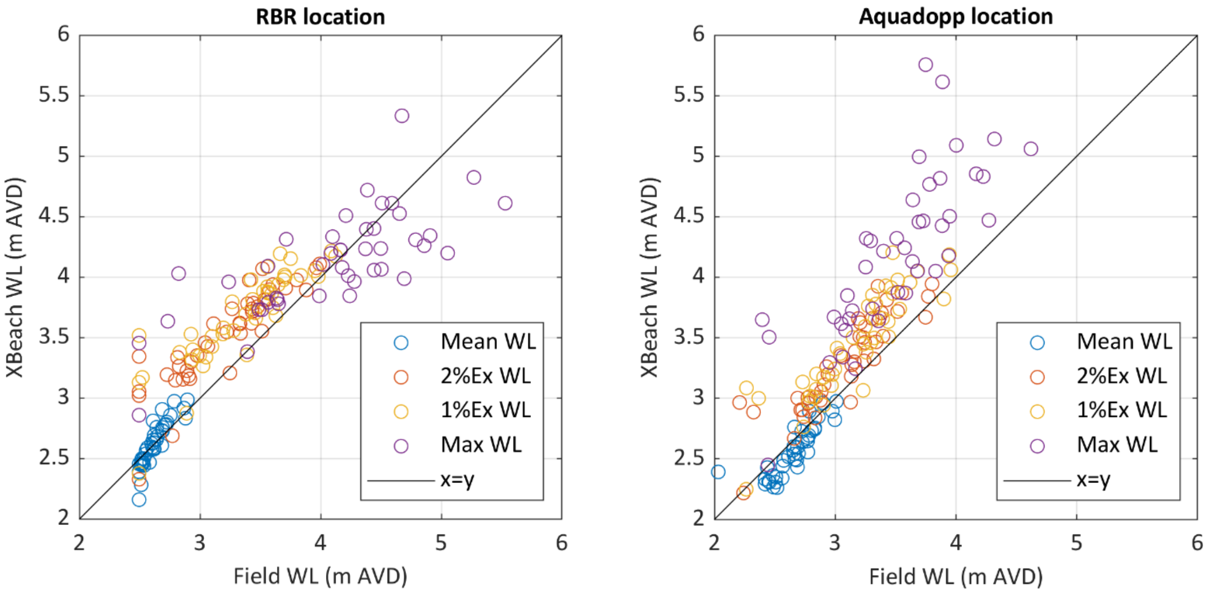


Figure 31: Scatterplot comparison of measured and modelled water level at the RBR and ADCP location during the 8-9 May 2023 event.

A final comparison is presented in Figure 32 to show the extent of inundation modelled for conditions at 10pm on the 8 May 2023 (Hs = 6m; Tide = 1.3m; Tp = 14.5s). This extent is generally consistent with the inundation extents from drone imagery collected at this time (see Appendix A for additional detail).

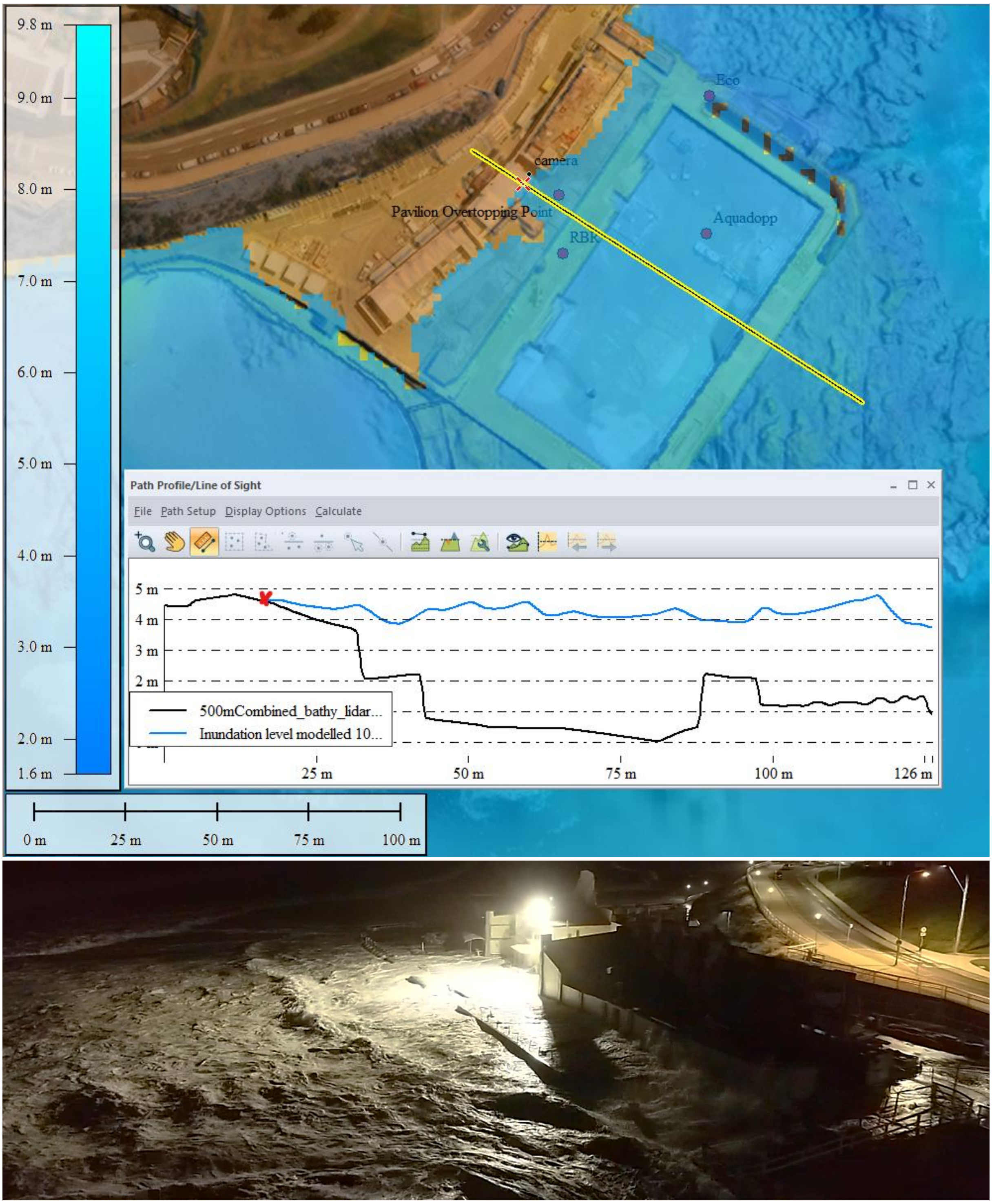


Figure 32: Modelled coastal inundation extents for input wave and water level conditions measured at 10pm on 8 May 2023 (top) and drone image captured from the same period (bottom).

5.4.4 Summary

The model calibration and validation process is summarised as:

- The XBeach-NH model, implemented in 2 horizontal layers at a 2x2m resolution was capable of simulating wave transformation processes from the offshore, across the rock platform, into the pool and onto the pavilion deck level.

- The model is stable, with no instances of crashing due to instability with the complex bathymetry.
- Model validation to the observed conditions show good agreement for the mean water level and 2% exceeded water level at the RBR and ADCP locations.
- Some scatter was observed when comparing the maximum water level and 1% exceeded water level. This was largely expected due to the 'randomness' of coincident and reflected wave crests.
- The occurrence and extent of overtopping on the pavilion deck level was captured well in the model based on a comparison of model outputs and camera and drone observations.

Based on these outcomes, the model is deemed suitable for assessing extreme water levels, caused by waves and overtopping on the pool complex and pavilion deck level.

Limitations in the model calibration that could influence the application of results:

- Measurements of flow velocity or rate (discharge) were not available to calibrate / assess model performance for overtopping dynamics that influence force impacts on the building edge.
- Construction was in progress during the event, with the terrain in transition towards the Stage 1 design, including stairs that were not included in the model bathymetry used of calibration simulations.

5.5 Inundation assessment scenarios

The field verified model was used to assess coastal inundation hazards for the Newcastle Ocean Baths site using two approaches:

1. Specific extreme combinations of wave height and water level, including a total of 27 scenarios that represent annual exceedance probabilities of 50% to 1% at present sea level, with additional SLR magnitudes.
2. An iterative range of wave heights, water levels and wave periods that represent the range of conditions expected during the complex design life.

The model bathymetry used in these simulations was adjusted to account for the Stage 1 complex features, including ramps, steps, and leveling or raising deck levels at the pool and pavilion. These features are presented in Figure 33, along with an example timeseries output locations that are used for analysis in Section 6.

Note that the model bathymetry used in the inundation assessment does not include the Pavilion buildings as structures in the bathymetry. This is a recommended approach because the buildings cause complex reflection patterns that are likely not fully represented in the model bathymetry and will be sensitive to site specific features and drainage beyond the model resolution. This is also consistent with the model approach taken when verifying the model to field observations. By excluding buildings from the model bathymetry, the baseline hazard inundation is understood without being complicated by reflections, and inundation forces directed to the building can be obtained. However, this may also result in some over-prediction of inundation hazards leeward of buildings at the carpark, and a potential under-prediction of inundation hazards on the Pavilion deck, as waves reflect off the building and converge with incoming waves to form a temporary amplification. For this reason, additional simulations were undertaken with buildings included in the bathymetry to understand the sensitivity of different approaches. These are discussed further in Section 6.3

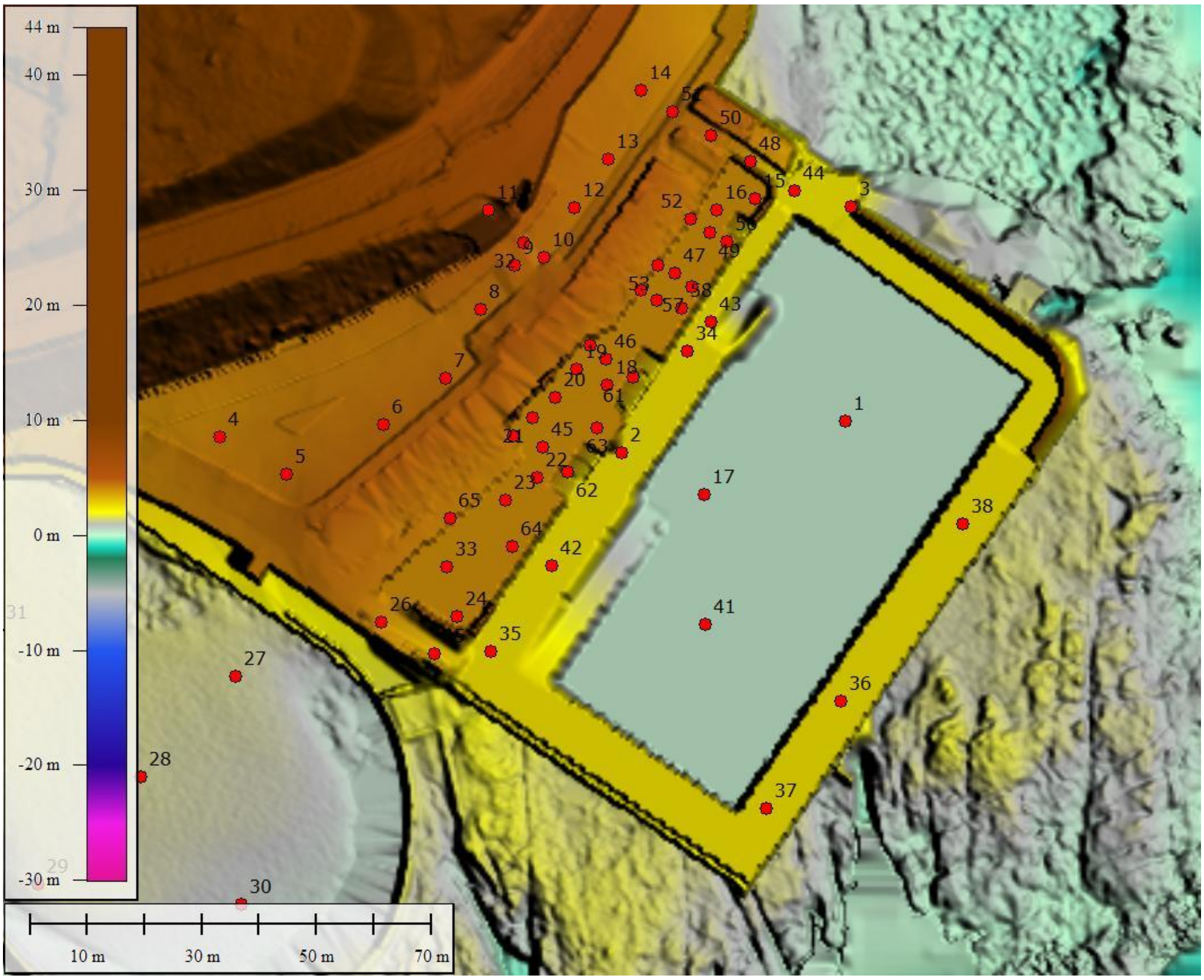


Figure 33: Model bathymetry (not full extent), including Stage 1 completion features, and key wave output locations for the coastal inundation assessment.

5.5.1 Adopted extreme scenarios

The scenarios adopted for the modelling and inundation analysis represent conditions based on the analysis conducted in the previous section and making reasonable assumptions about future conditions. As such, the wave and water level conditions from the joint probability analysis along with 50th percentile SLR conditions have been adopted for modelling the inundation of the facility. These are shown in Table 11 below.

Table 11: Adopted scenarios for inundation modelling.

Scenario	Hs (m)	Water level (m)			Tp (s)	Dp (deg)
		2023	2050 + 50th %ile SLR (+0.23m)	2120 + 50th %ile SLR (+0.93m)		
50% AEP	6.59	1.08	1.31	2.01	12.5	140
20% AEP	7.29	1.23	1.46	2.16	12.5	140

Scenario	Hs (m)	Water level (m)			Tp (s)	Dp (deg)
		2023	2050 + 50th %ile SLR (+0.23m)	2120 + 50th %ile SLR (+0.93m)		
10% AEP	7.48	1.29	1.52	2.22	12.5	140
5% AEP	7.77	1.29	1.52	2.22	12.5	140
2% AEP	8.23	1.30	1.53	2.23	12.5	140
1% AEP	8.50	1.30	1.53	2.23	12.5	140

5.5.2 Inundation scenarios

A total of 180 additional inundation scenarios were simulated, using a range of ten wave heights, two wave periods, and nine water level combinations. A design wave direction aligned with the model boundary was used for all simulations. These conditions are shown in Table 12.

Table 12: Inundation scenarios used for XBeach-NH modelling.

Hs (m)	WL (m)	Tp (s)	Dp (°N)
2.5	-0.5	10	140
3.5	0	15	
4.5	0.5		
5.5	1		
6.5	1.5		
7.5	2		
8.5	2.5		
9.5	3		
10.5	3.25		
11.5			

5.5.3 Model outputs

Model outputs for all extreme and inundation scenarios include: gridded maps, water level included time averaged, time maximum and instantaneous values for:

- water level (m AHD)
- wave height (m)
- velocity (m/s).

Point data at each of the probe locations from Figure 33 were also output for water level, depth, velocity, and discharge to assess inundation thresholds.

6. Coastal inundation assessment

6.1 Introduction

Building on the data review and numerical modelling completed in the previous sections, this assessment sets out the extents, levels and likelihoods of coastal inundation at the Ocean Baths facility.

Inundation mapping as well as maximum water depths and wave loads are reported for the adopted AEP input scenarios including SLR for the selected planning periods.

A probabilistic assessment of the coastal inundation levels is used to determine the likelihood and percentage of time that certain parts of the facility will experience inundation/overwash. It is also used to provide a likelihood of conditions affecting ongoing operations for both the pool area and the pavilions. The probabilistic assessment is based on combining the recorded time histories of wave and water level conditions and the XBeach model results.

6.2 Maximum and mean water levels

At the Ocean Baths, inundation is wave driven. Due to the dynamic processes discussed in Section 2.1, water level changes associated with wave driven inundation occur rapidly (e.g., wave run-up and overtopping cause inundating levels to an area during a single wave or multiple waves). Statistics, such as the mean, maximum and one percentile exceedance value, are commonly used to describe inundation levels in these circumstances and this has been adopted herein.

Figure 34 shows a water level timeseries over a 30-minute period for a location on the pavilion deck in front of the central building. This example come from a model simulation of the 2023 1% AEP scenario. With reference to this figure, the inundation levels used herein are:

- Maximum water level occurs at the crest of the highest wave in the observed period. This parameter is available as a 2D model output and adopted herein for mapping of results for each design (AEP + SLR) scenarios.
- One percentile exceedance water level (or 1% exceeded water level) is the water level that is exceeded one percent of the time during the period. This water level needs to be calculated from the model timeseries results at each output location and is therefore only available at these discrete locations. The 1% exceeded water levels are used to represent the highest levels for each scenario, offering a more reliable depiction of wave-driven water levels than the maximum (see Section 5.4.2 for justification).
- The mean water level is the average over the period (not shown in the figure). The mean water level for each scenario is used to indicate the still water level for each scenario. This is also available as a 2D output and has been shown on the inundation maps.

Figure 35 shows an example of the mean and 1% exceeded water levels observed during the inundation monitoring event.

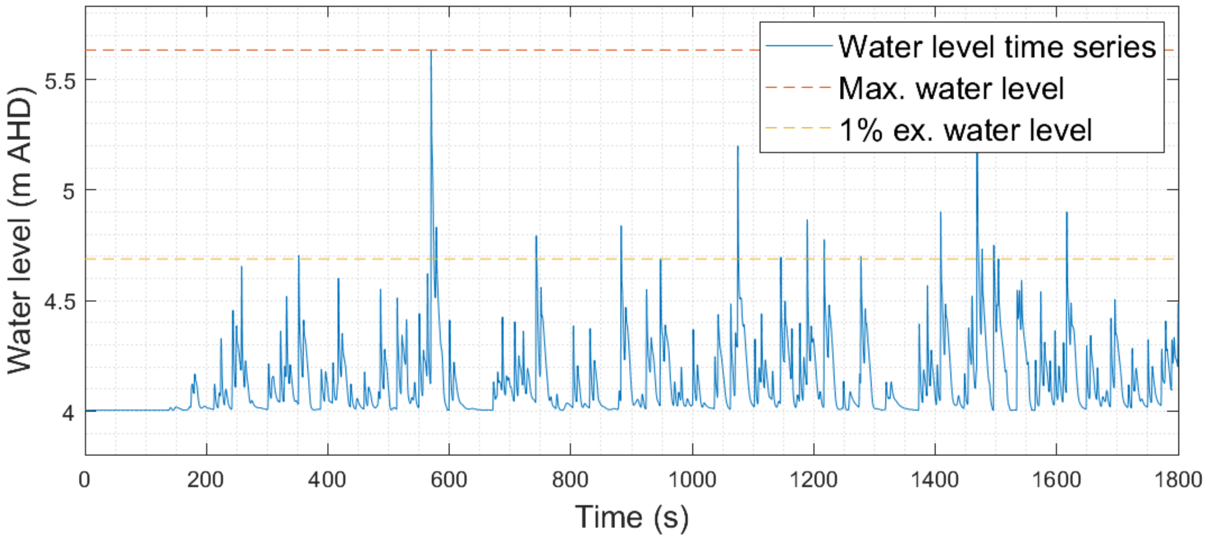


Figure 34: High frequency timeseries of the water levels (m AHD), inclusive of wave action, for an output point on the pavilion deck.

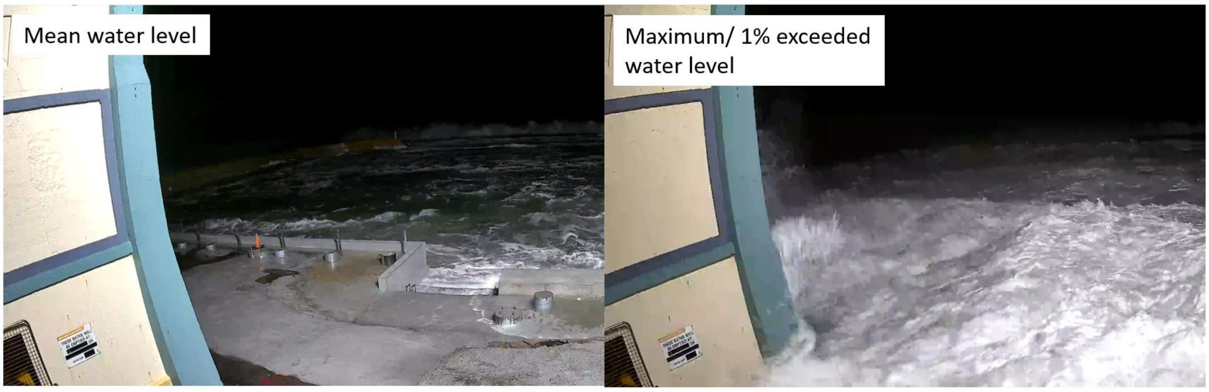


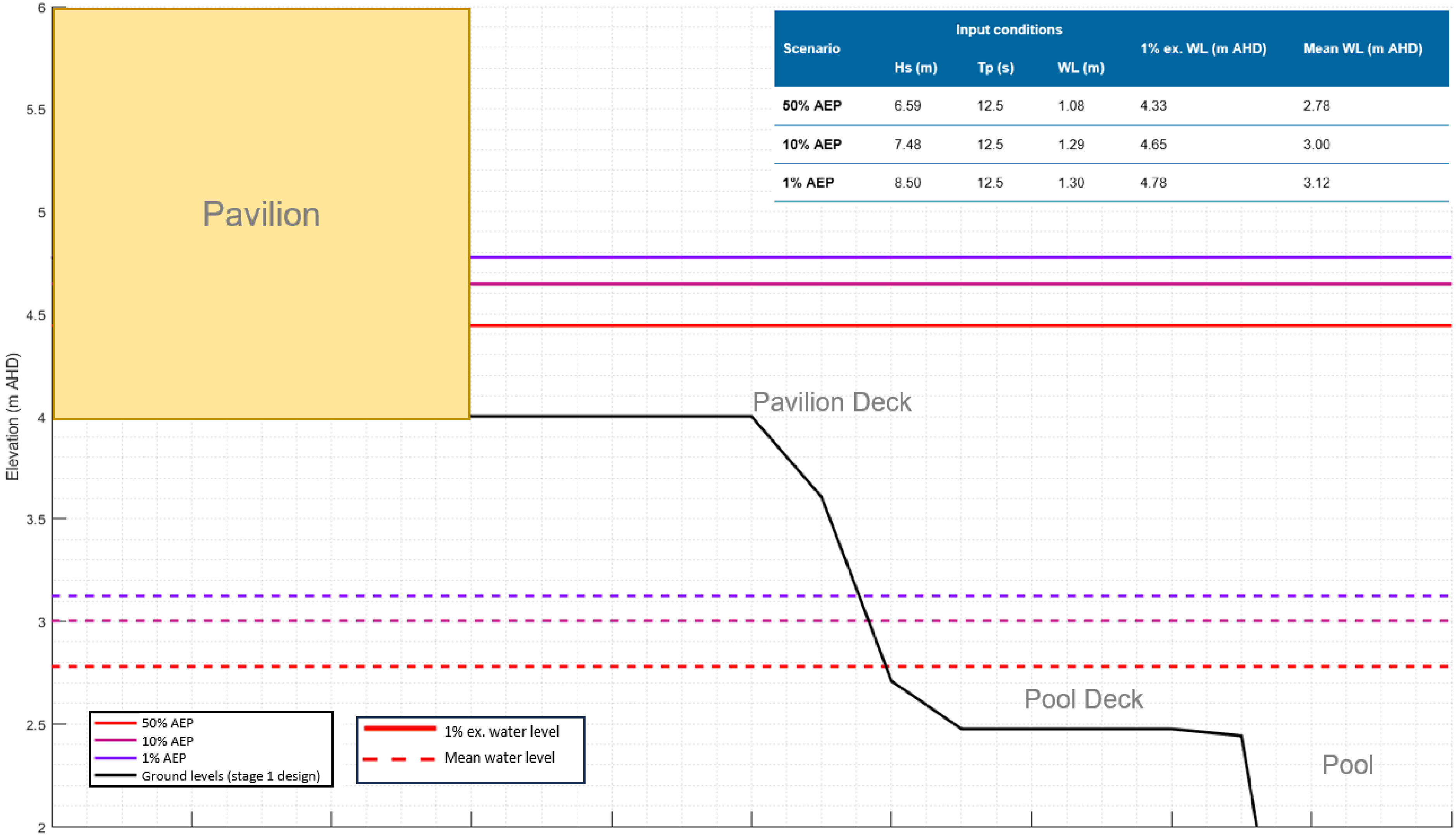
Figure 35: Examples of the various water level definitions during an inundation event.

Figure 36 and Figure 37 present the mean and 1% exceeded water levels in comparison to the Stage 1 design ground levels for a cross section in the central area on the pavilion level. Figure 36 provides a comparison of the present-day scenarios. The findings include:

- 1% exceeded water level of 4.33m for the 2023 50% AEP scenario.
- 1% exceeded water level of 4.65m for the 2023 10% AEP scenario.
- 1% exceeded water level of 4.78m for the 2023 1% AEP scenario.

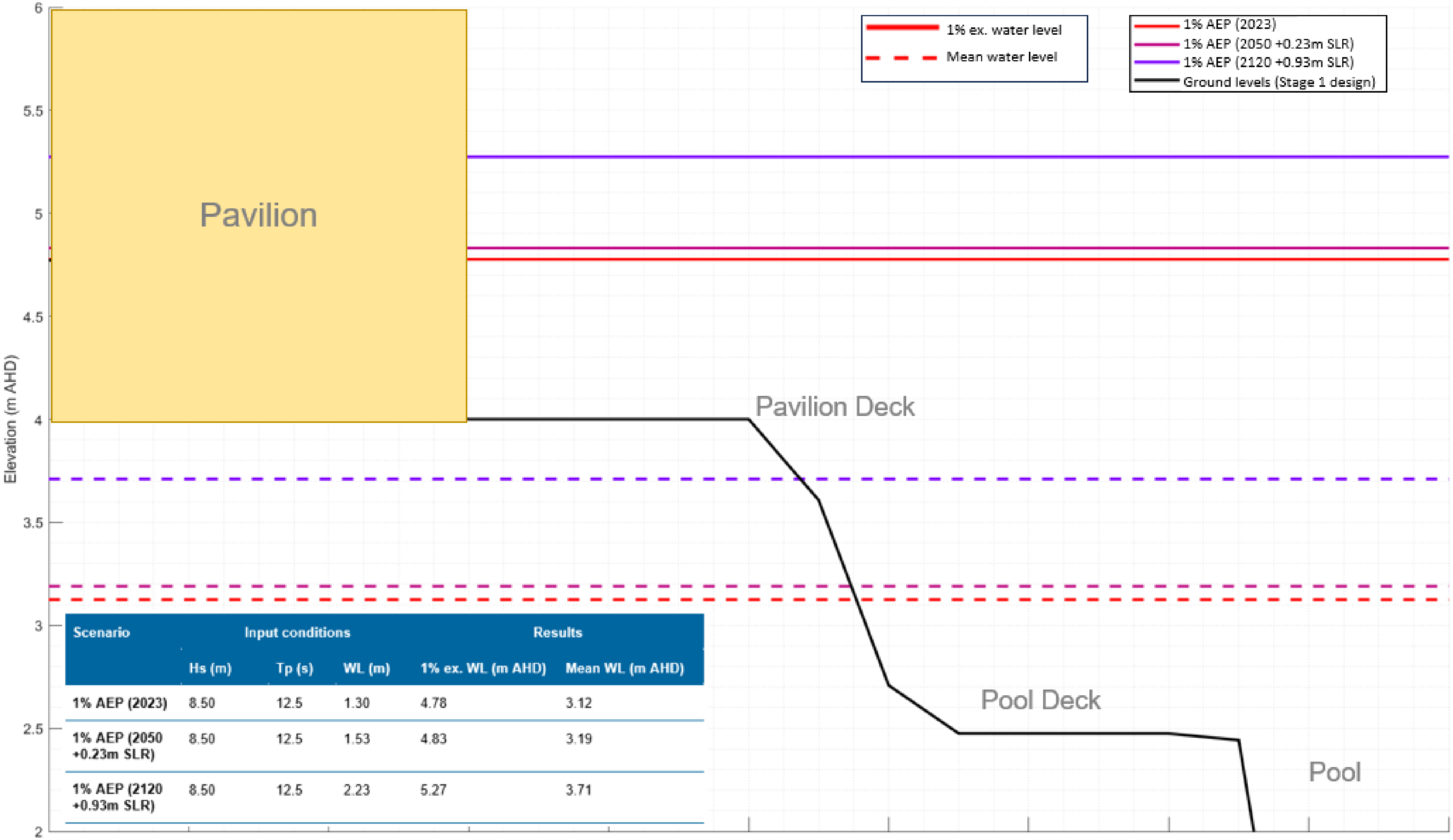
Figure 37 illustrates the 1% AEP scenarios which also factoring in sea level rise (SLR). The 2120 scenario shows a significantly higher mean and 1% exceeded water levels compared to present or 2050 scenarios. An increase in the initial water level disproportionately impacts the resulting water levels at the pavilion. As the sea levels rise the pool's ability to drain after a set of waves decreases, resulting in a higher wave setup inside the pool for higher sea levels.

Figure 38 presents the 1% exceeded inundation depth at each building for all design (AEP + SLR) scenarios. Across all SLR scenarios, the central building experiences the highest water depth. For the 2023 and 2050 scenarios, the water depth at the northern building surpasses that of the southern building. However, this trend is reversed in the 2120 SLR scenario, with the southern building witnessing a more significant inundation depth. This increased inundation for the 2120 SLR scenario near the southern building can also be observed in the inundation map in Figure 37.



Scenario	Input conditions			1% ex. WL (m AHD)	Mean WL (m AHD)
	Hs (m)	Tp (s)	WL (m)		
50% AEP	6.59	12.5	1.08	4.33	2.78
10% AEP	7.48	12.5	1.29	4.65	3.00
1% AEP	8.50	12.5	1.30	4.78	3.12

Figure 36: 1% exceeded and mean water levels for the design scenarios in 2023.



Scenario	Input conditions			Results	
	Hs (m)	Tp (s)	WL (m)	1% ex. WL (m AHD)	Mean WL (m AHD)
1% AEP (2023)	8.50	12.5	1.30	4.78	3.12
1% AEP (2050 +0.23m SLR)	8.50	12.5	1.53	4.83	3.19
1% AEP (2120 +0.93m SLR)	8.50	12.5	2.23	5.27	3.71

Figure 37: 1% exceeded and mean water levels for 1% AEP design scenario including allowances for SLR in 2050 and 2120.

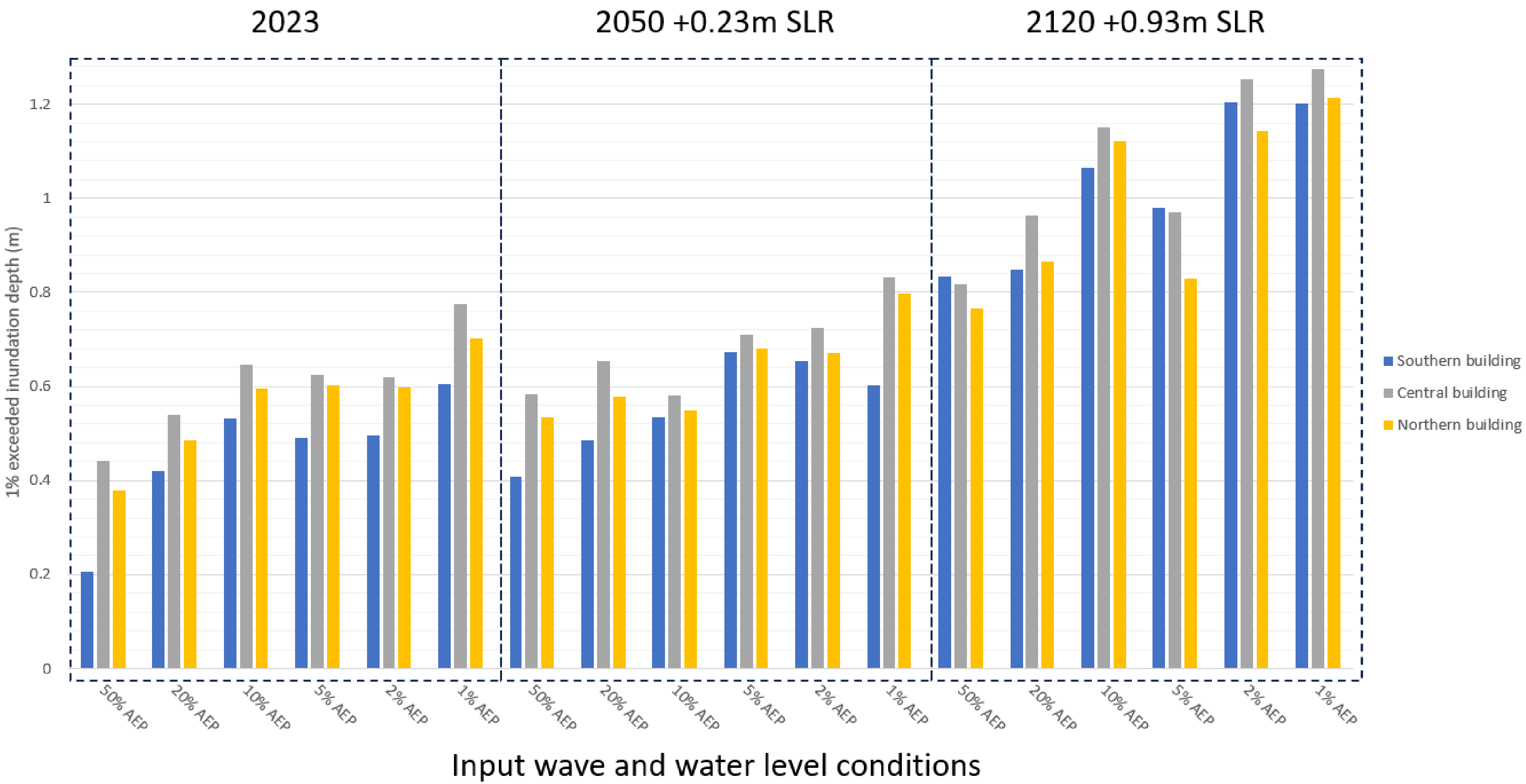


Figure 38: 1% exceeded water depth for all adopted AEP input scenarios at the pavilion buildings.

6.3 Inundation maps

Coastal inundation maps showing the maximum inundation extent and depth for the adopted design scenarios are provided in **Appendix C**. These maps show the gridded output of the maximum instantaneous depth during each respective simulation (rather than the 1% exceeded depth explained above). An example map is shown in Figure 39 to illustrate the inundation extents.

The present day 50% AEP scenario maximum water depth is shown in Figure 66 (Appendix C). The inundation at the pavilion deck level is at a depth between 0.5 and 1.0m. This is consistent with the levels observed during the inundation monitoring activity that was used to validate the model, undertaken during similar conditions.

The present-day 1% AEP scenario shows significant inundation at the pavilion deck (1.0 to 1.5m depth). Inundation is worse in front of the central and pavilion buildings where bleachers and stairs allow the waves to travel up the slope, dissipating less energy than when encountering a vertical wall. The southeast corner of the northern building (adjacent to the central entrance) is the most affected due to the incident wave direction and the stairs/bleachers concentrating the waves to this area. At this location, waves traverse the largest uninterrupted section of the pool. This allows offshore wave energy to transfer with minimal interference, leading to larger wave heights impacting this section of the wall. The southern building is less effected, in part due to the extended wall on the southern corner of this deck. Figure 39 shows the inundation extents for the present day 1% AEP conditions with the buildings modelled.

The 1% AEP scenario with 2120 SLR (Figure 68) shows maximum water depths in front of all pavilion buildings to be 1.5 to 2.0m. The pavilion deck is most affected centrally with the stairs and bleachers in these areas being points leading to increased inundation.

The extents from the present day and 2120 1% AEP + SLR scenarios are comparable to the previous estimates presented in Royal HaskoningDHV (2022), refer to Figure 3.

6.4 With buildings sensitivity tests

As discussed in Section 5.5, the model geometry followed the recommended approach of not including buildings for assessing the inundation and loads in front of the buildings. However, this causes an overestimation of inundation in the carpark and an underestimated on the seaward side of the pavilions (because of additional wave reflections). Additional simulations with the buildings modelled as rectangular bocks over the existing footprints were completed as a sensitivity test to give an indication of the extents into the carpark for extreme scenarios. A difference map is shown in Figure 70 (Appendix C) comparing the inundation depths with and without the modelled buildings confirming the increase in front and reduction behind the buildings. A quantification of the effect of the modelled buildings on the 1% exceeded water depth in front of the buildings is shown in Table 13.

Table 13: Comparison of the 1% exceeded water level with and without modelled buildings.

Inputs			Pavilion deck level (1% ex. water depth)		
Scenario	**Hs (m)**	**WL (m AHD)**	**Baseline**	**Buildings**	**Reduction factor**
50% AEP 2023	6.59	1.08	0.18	0.27	1.51
1% AEP 2023	8.5	1.3	0.43	0.86	1.98
50% AEP 2050	6.59	1.31	0.23	0.39	1.66
1% AEP 2050	8.5	1.53	0.46	0.81	1.77
50% AEP 2120	6.59	2.01	0.57	1.14	1.99
1% AEP 2120	8.5	2.23	0.91	1.70	1.86

Note: The reduction factor is *modelled building depth/no building depth*. Values less than one are a reduction, values greater than one are an increase.

Figure 39: Inundation map of the 1% AEP 2023 condition with modelled buildings.

6.5 Probabilistic inundation assessment

To provide a more complete understanding of the likelihood of coastal inundation throughout the facility a probabilistic inundation assessment was completed.

This involved combining a 13-year timeseries of observed ocean wave and water level (i.e., input) conditions to synthesise inundation levels across the baths facility by interpolating the input conditions across the results from the inundation matrix described in Section 5.5 (Table 12). An example of the output matrix for a single output point at the building platform edge is presented in Figure 40.

Figure 40 shows that inundation levels are sensitive to wave period, as significantly higher inundation levels on the pavilion deck were identified for a 15 second period (swell) compared to a 10 second period (closer proximity storm). The threshold for waves to start interacting with the pavilion deck is a combination of wave height, period, and water level, which can be tracked using the contour lines in this figure. For example, the 4.1 m contour line tracks the combination of wave height and water level associated with a water depth of 0.1 m on the pavilion deck.

The step-by-step approach used to determine exceedance probabilities at each model output location was:

- A timeseries of observed wave height, period and water level from the available Newcastle WRB data and Patonga tide gauge was created. This timeseries includes hourly results over a 13-year period from November 2009 to May 2023.
- Interpolate each hourly input condition to obtain the inundation levels at each output location using the results from the inundation scenarios matrix. The interpolation method used was inverse distance weighting which estimates values at unmeasured points based on the weighted averages of nearby known points, with closer points having a greater influence. The method was validated by applying it to the extreme AEP scenarios (Table 11).
- Generate probability of exceedance for each output point. The results of this give a probability that the inundation level will exceed a given depth.
- Probabilities are presented as percentages as well as hours per year, to give a more accessible perspective of the likelihood.

This approach allowed direct comparison of the depths and loads at output points for input conditions that fall between matrix values. Out of the 30 model runs tested, a coefficient of determination (R^2) of 0.91 was obtained. This value indicates that the interpolation method consistently produced dependable results for the given output points.

The probability of exceedance plots for selected output points on the pool deck and pavilion deck are shown in Figure 41. Both the maximum and 1% exceeded water level are shown in these plots. For consistency with previous analysis, the 1% exceeded water level is adopted as a pragmatic inundation level in the following results. The inundation mapping in Section 6.3 shows variability in inundation depth across the pavilion deck. A comparison of the depth exceedance probability was conducted to determine how to consider the inundation risk to this area. Figure 42 shows that there is variability in the probability in front of the south, central and northern buildings, the probability of inundation for each of these locations should therefore be considered separately.

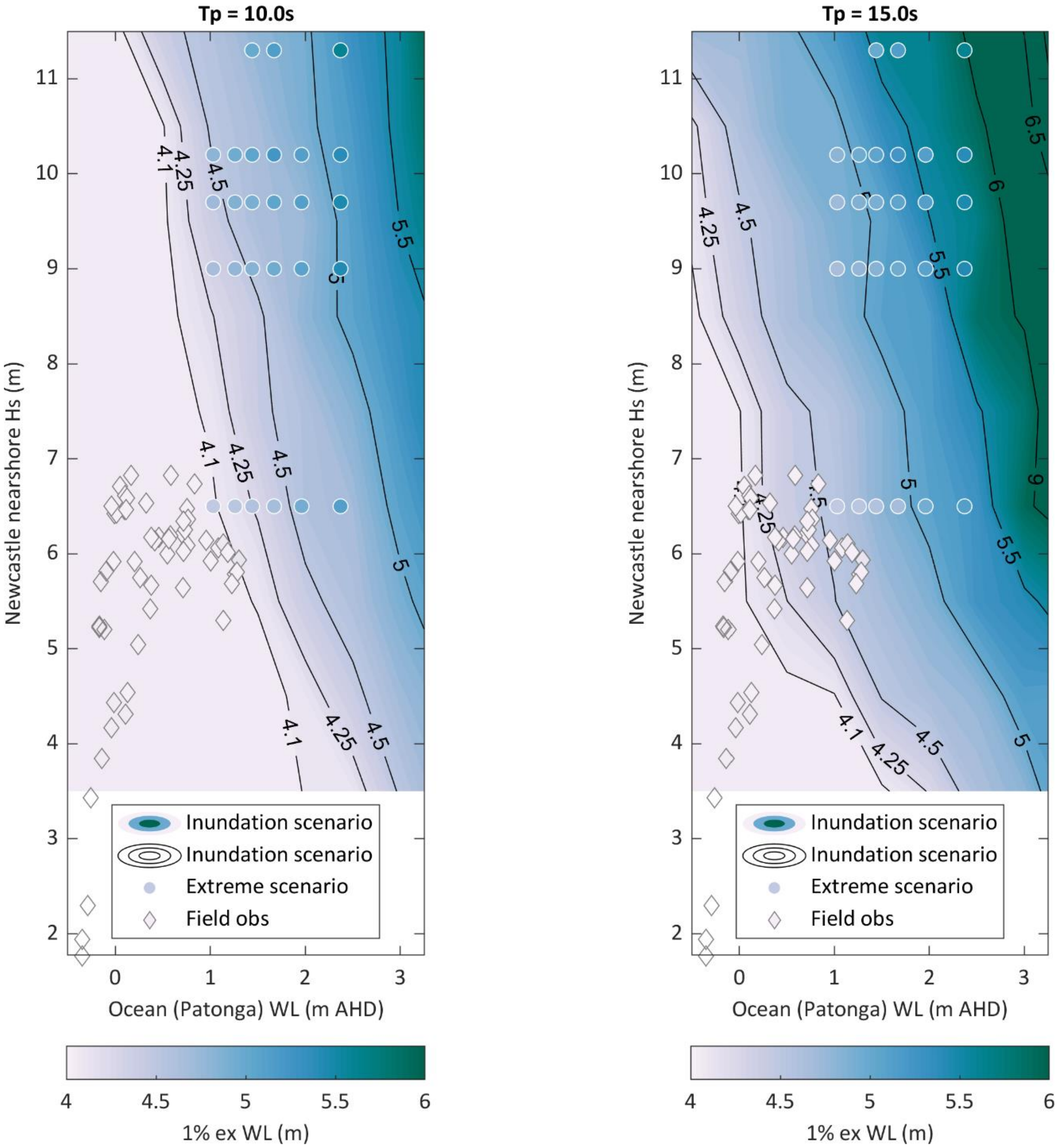


Figure 40: Inundation model outputs for 1% exceeded water level at point 52 on the pavilion deck level (edge of the northern building).

Note: Ground level is 4.0m RL so the 4.1m RL contour indicates a depth of 0.1m. Coloured circles are from the modelled extreme inundation scenarios with a 12.5 s wave period. Diamonds indicate the range conditions observed in the field for reference. Diamond colour is white and does not represent a modelled or measured inundation level.

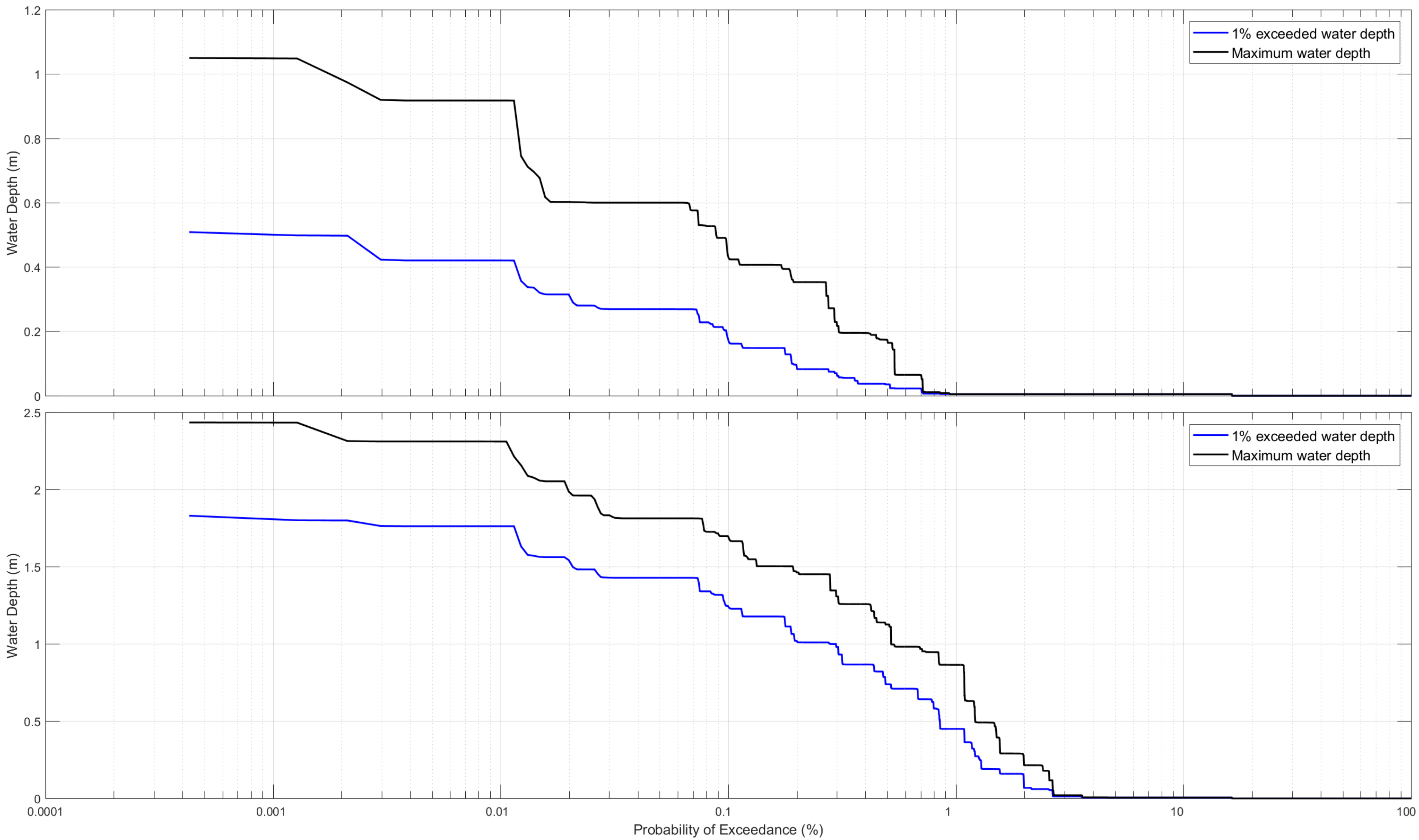


Figure 41: Probability of exceedance plots for water depths at the pavilion deck (top) and the pool deck (bottom).

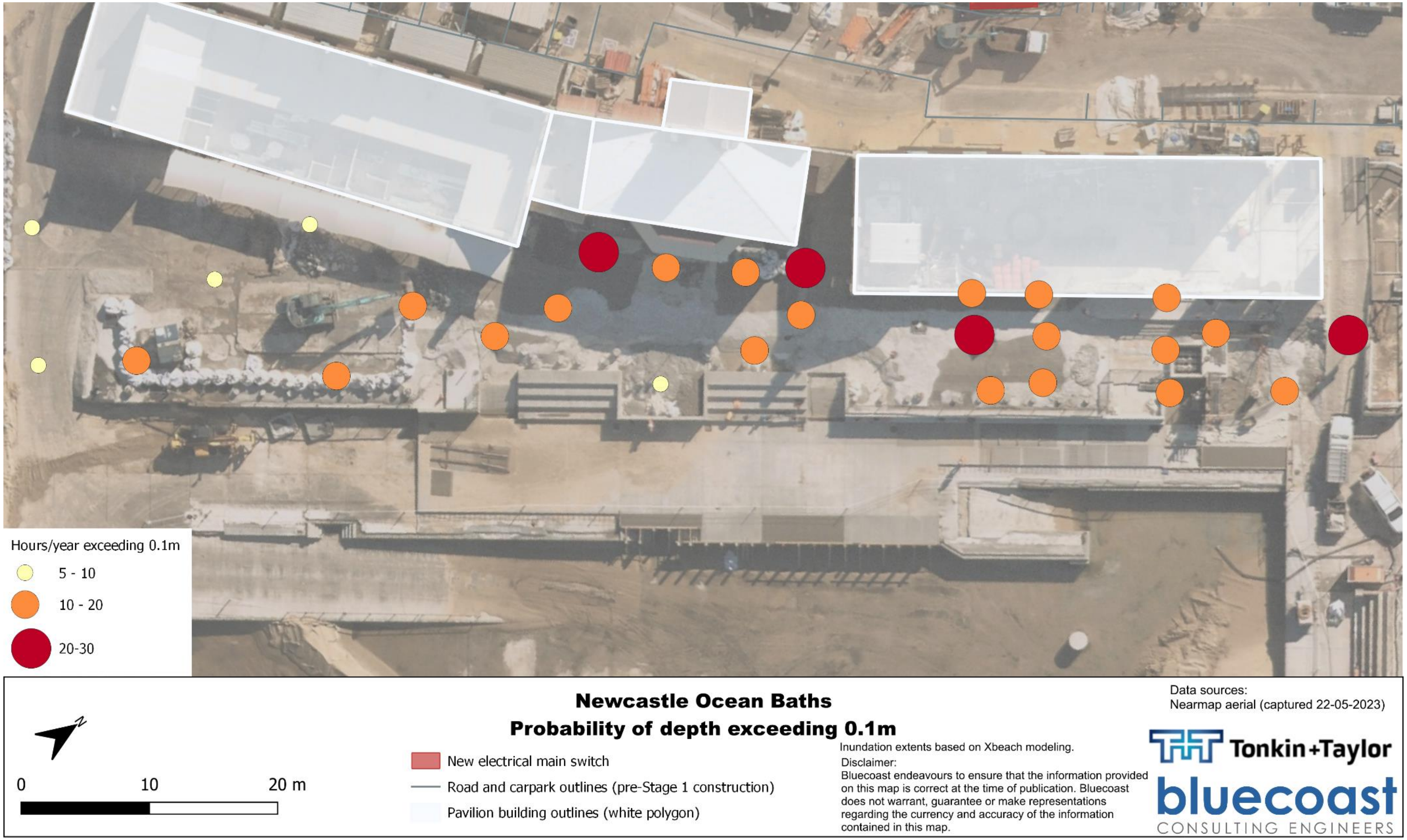


Figure 42: Probability of water depth due to wave run up at present sea level exceeding 0.1m across the pavilion deck.

For the inundation probability analysis, the facility is broken down into 5 ‘zones’, as shown in Figure 43. This helps guide the assessment for design and operational impact on the facility in Section 7.5. The probabilities of the 1% exceeded water level for threshold depths up to 1m are shown in Table 14.

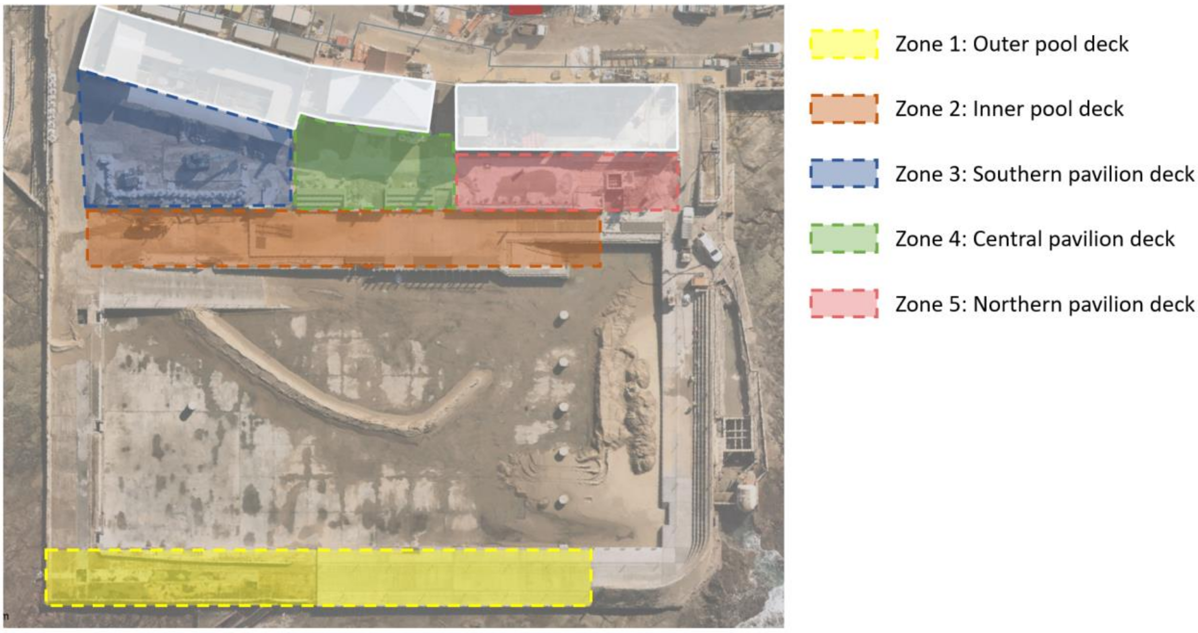


Figure 43: Zones used for the probabilistic risk assessment.

Table 14: Threshold depth exceedance probabilities (present sea level) for the zones shown in Figure 43.

Threshold depth (m)	Zone 1		Zone 2		Zone 3		Zone 4		Zone 5	
	%	Hrs/yr	%	Hrs/yr	%	Hrs/yr	%	Hrs/yr	%	Hrs/yr
0.1	7.56	662	1.98	174	0.19	16	0.36	31	0.29	26
0.2	4.25	372	1.56	136	0.08	7	0.19	17	0.16	14
0.3	3.44	301	1.21	106	0.03	2	0.12	10	0.07	6
0.4	1.97	173	1.09	95	0.01	1	0.09	8	0.02	1
0.5	1.49	131	0.85	74	0.00	0	0.03	2	0.01	1
0.6	1.09	95	0.82	72	0.00	0	0.02	2	0.00	0
0.7	0.82	72	0.68	59	0.00	0	0.01	1	0.00	0
0.8	0.52	45	0.49	43	0.00	0	0.00	0	0.00	0
0.9	0.49	43	0.44	38	0.00	0	0.00	0	0.00	0
1.0	0.29	26	0.30	27	0.00	0	0.00	0	0.00	0

6.6 Inundation event likelihood

Utilising the analysis from Section 6.5, the likelihood of inundation exceeding certain thresholds and the number of events per year that could affect operation are determined. The events and thresholds given in Table 15 have been used based on observation made on the inundation data provided in Section 5.5. Engagement with facility owners and operators would provide more informative guidance on what are appropriate thresholds for certain events (this is discussed further in Section 7.5).

Table 15: Inundation likelihoods for certain events effecting future operations.

Event	Zone affected	Threshold	Likelihood		
			%	Hours/year	# events per year
Overwash into the pool	1	Depth exceeding 0.3m	7.56	301	121
Small waves reaching inner pool deck	2	Depth exceeding 0.2m	1.29	105	11
Large waves reaching inner pool deck	2	Depth exceeding 0.7m	0.68	59	5
Waves reaching pavilion deck	3, 4, 5	Depth exceeding 0.1m	0.35	25	2.5
Waves impacting southern pavilion building	3	Depth exceeding 0.4m	0.01	1	0.5
Waves impacting central pavilion building	4	Depth exceeding 0.4m	0.09	8	1.25
Waves impacting northern pavilion building	5	Depth exceeding 0.4m	0.01	1.5	0.5

6.7 Wave loading

Figure 46 provides a summary of the wave loads for the adopted AEP input scenarios, specifically for areas in front of each of the pavilion buildings. To represent the extremities of potential loads, the figures presented are based on the maximum observed values from each designated output location pertaining to each building.

The computation of these loads is based on the equation proposed by Streicher et al. in 2018 to determine the peak load associated with the dynamic impact force of the overtopping bore on the vertical wall (F1), see Figure 44. This equation estimates the dynamic impact load as a function of two crucial parameters: the instantaneous water depth and velocity. A detailed discussion of this equation, along with its underlying assumptions and limitations, can be found in Appendix B:.

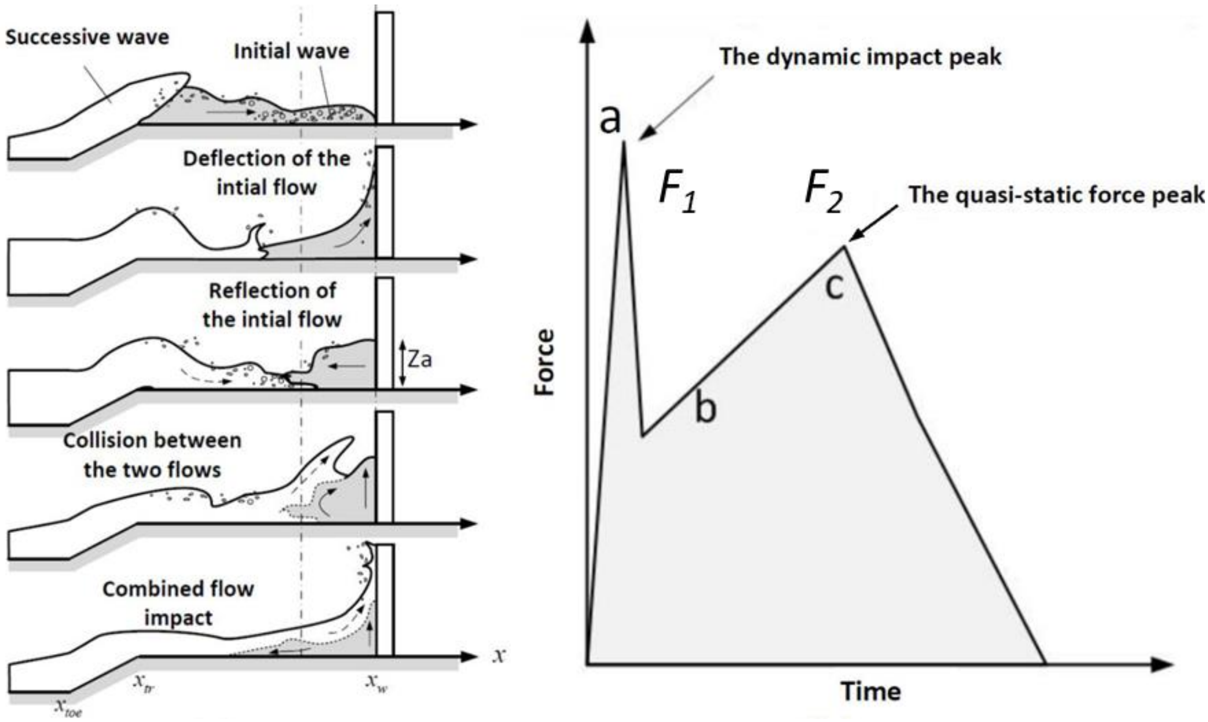


Figure 44: Schematic diagram of double peaked force impact from wave overtopping flow (adapted from: Chen et al. 2017).

It's important to highlight that wave loads, by their nature, are influenced by a multitude of dynamic factors including the type and location of wave breaking, wave reflection, 3D effects and phasing with infragravity waves. The occurrence and combination of these various processes could result in higher peak loads under certain conditions, i.e., if waves were to reform riding on an infragravity surge and break directly on the structure. While it is likely that waves are arriving at the buildings as broken bore rather than breaking directly on the structures (see Figure 45), the potential for these interactions introduces an inherent uncertainty in the load calculations, an effect that becomes more pronounced as wave and water level conditions intensify.

Figure 45: Wave impact on building during monitoring event.

Given these complexities and uncertainties, caution is advised when interpreting and applying these load estimates. Specifically, when these wave load estimations are used as a basis for future building designs, it is strongly recommended to incorporate appropriate factors of safety or undertaken further scale (physical) model testing that would be better able to resolve these wave loads.

In addition to wave loading on the building, the wave loads acting on the lifeguard tower to be installed as part of Stage 1 works are included in **Appendix D:.**

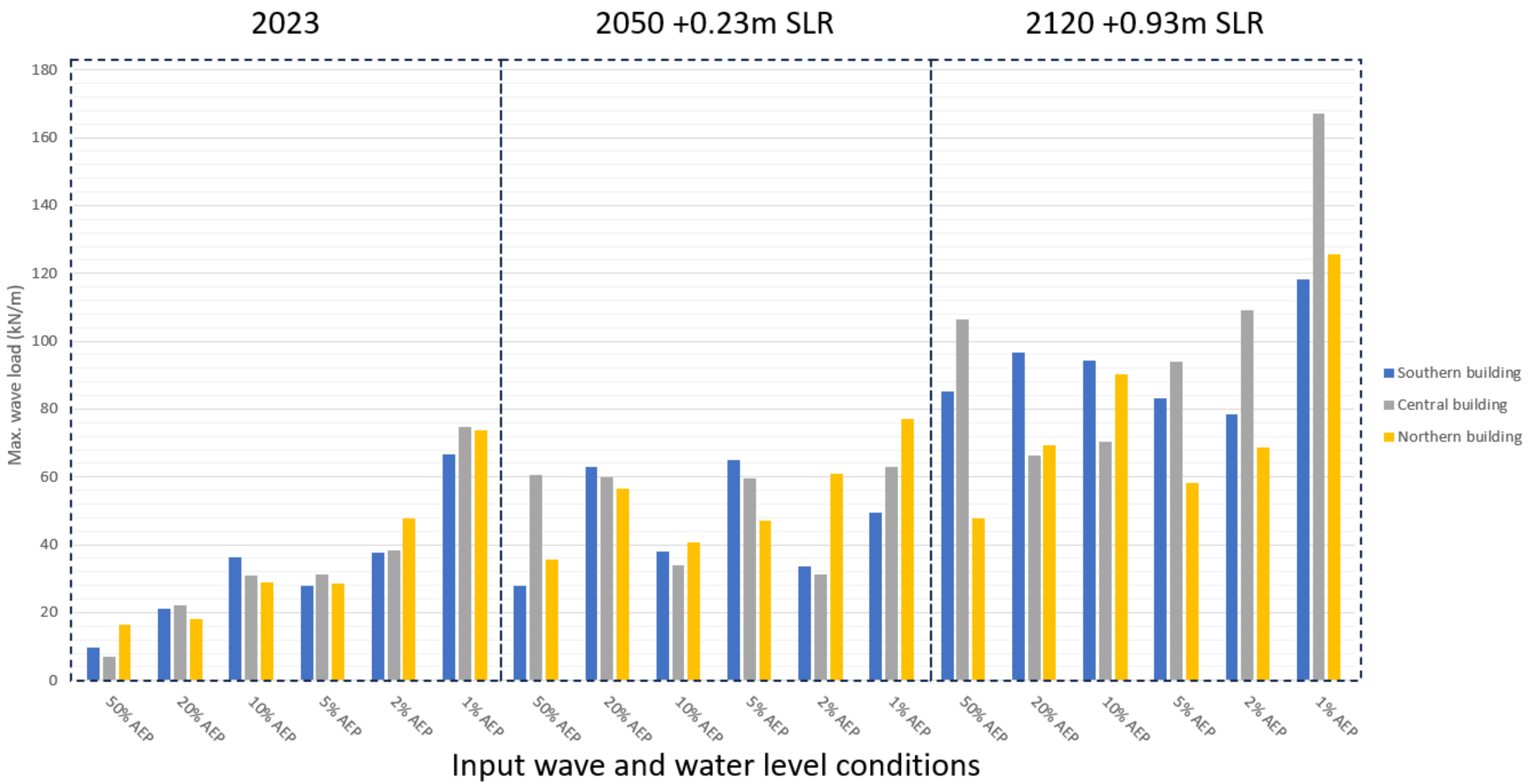


Figure 46: Maximum wave loading (kN per linear metre) for all adopted AEP input scenarios at the pavilion buildings.

7. Inundation mitigation measures

7.1 Introduction

Following on from the outcomes of the inundation assessment this section presents several management alternatives to mitigate the coastal inundation at the baths facility. The options consider the feasibility of implementation given the Stage 1 design and how they could be implemented during the Stage 2 design phase. The options considered are as follows:

- **Option 1** - Raised floor levels of the pavilion buildings.
- **Option 2** - Temporary inundation barrier at pavilion deck edge
- **Option 3** - Moving the bleachers
- **Complementary** - Monitoring and response measures
- **Complementary** - Resilient building design

Evaluation of each of the options has been conducted using the results from the numerical modelling (or additional modelling) as well as the ongoing operational and social impact these would have on the facility.

7.2 Raised floor levels (Option 1)

7.2.1 Description and rationale

Raised floor levels are a simple way to increase the resilience of the Stage 2 pavilion buildings to inundation.

Raising internal floor levels would reduce the likelihood of the building's internals being affected by inundation while maintaining the original façade. Consideration would have to be given to the height and the effect of vertical run up and splashing on the seaward face of the building and the effect these would have on building openings where ingress could occur.

Another way raised floor levels could be achieved would be through elevating the buildings on piles to allow waves to pass underneath the structure. As this would lead to significantly increased inundation in the carpark and demolishing of the existing façade, this has been assumed to be an unsuitable option.

This option did not require any re-modelling as the facility layout and levels were essentially unchanged. Utilising the results from the inundation assessment in Section 6, the desired 1% exceeded water levels and maximum wave loads can be used to determine an appropriate level to raise the finished floor level to create a building resilient to inundation. This option should be considered along with permanent or temporary methods of protecting any openings from splashing or rogue run up.

7.3 Temporary inundation barrier (Option 2)

7.3.1 Description and rationale

A temporary (storm) barrier could be installed at the seaward edge of the pavilion deck. The barrier would be raised in anticipation of a coastal inundation event. Once raised it would need to be continuous along the length of the pavilion deck and curve around the lateral sides. The barrier would likely not extend to the ramps on either side of the pavilion deck.

Several design conditions would need to be considered, including:

- Crest height would need refining to achieve required reduction or prevention in wave overtopping volume. This may require empirical calculations or physical modelling in additional to numerical modelling.
- Installation around existing features such as the lifeguard tower and stairs could be challenging.
- Installation time with regards to operational closure and trigger threshold in wave forecast may influence feasibility.
- Barrier needs to be stored somewhere when not in use.
- If waves overtop the structure, there will be limited capacity for drainage which could result in ponding and potentially deeper inundation levels (but reduced wave loads) than if no barrier were installed. Crest level would need to be refined at the next stage of design if this option is taken forward.
- A temporary structure that is feasible to install may not withstand the wave forces at the site. This would likely require physical modelling to inform the feasibility and design.

7.3.2 Model simulations of this option

The performance of a temporary storm barrier was assessed by adding a wall along the pavilion deck and simulating a set of repeat wave conditions in the XBeach model. The wall crest level was approximately 1m above the deck level at 5m AHD (Figure 47). An initial set of simulations was undertaken without the barrier, and the exact same wave series was used for the comparison simulations with the structure to allow a wave-by-wave comparison without influence of stochastic wave generation. Note that the model resolution of 2m x 2m does not replicate a thin vertical structure, and some of the physical processes contributing to wave overtopping (e.g., splashing) are not resolved in the phase-resolving model. More refinement of the performance could be established using empirical overtopping equations for a vertical wall, using wave height and water level at the toe.

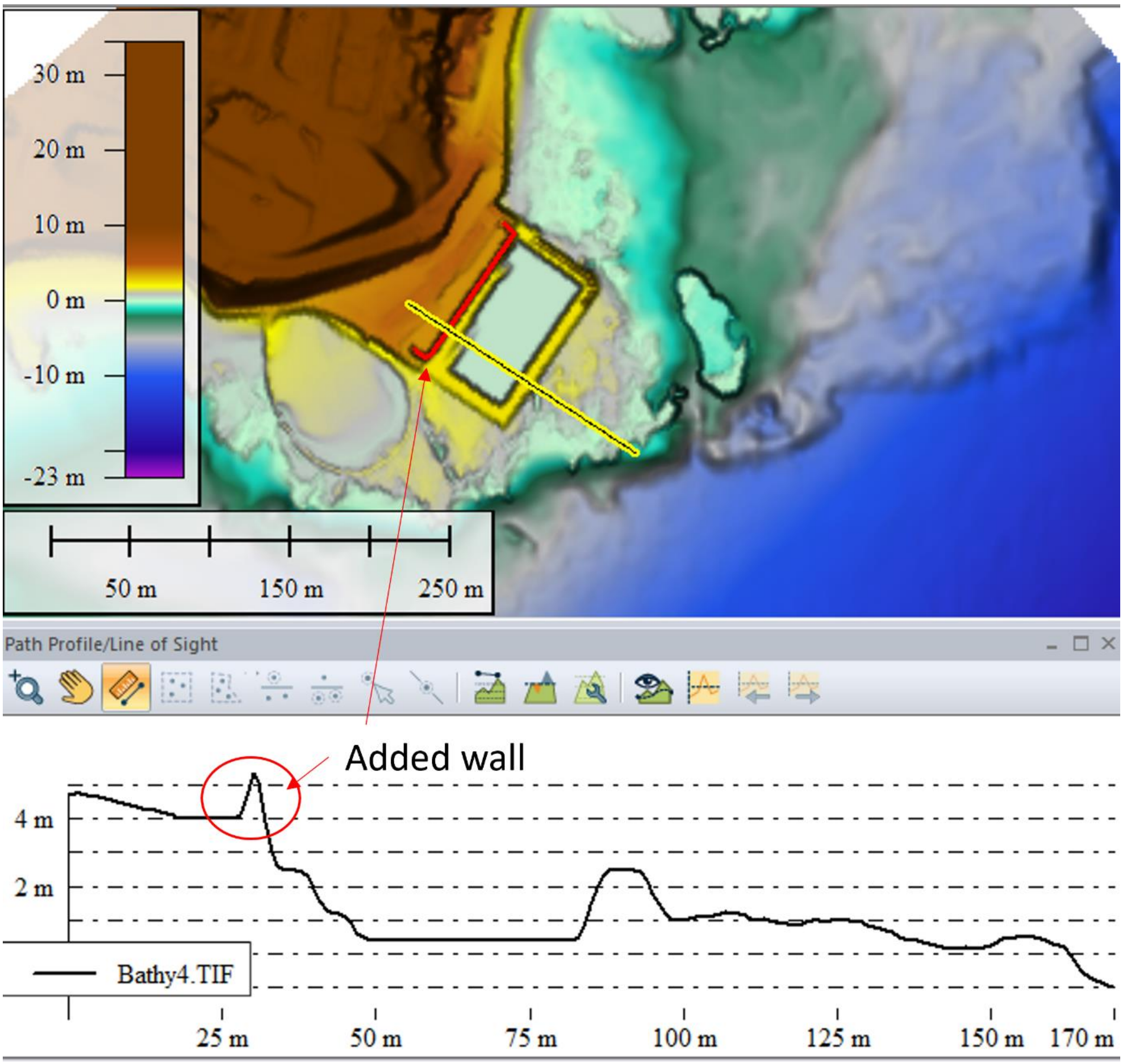


Figure 47: XBeach bathymetry with the added wave barrier along the pavilion deck.

Results presented in Figure 48, Figure 49 and Table 16 generally show that a wave barrier could be effective at reducing some wave inundation at the pavilion deck level and at the building and carpark areas. The 1% exceeded water depth on the pavilion deck level was 40-90% of the comparative baseline simulation. The most benefit was modelled for the 50% AEP event at 2023 and 2050. Minimal benefit was noted for the more extreme events and at 2120. The wave barrier as modelled in XBeach reduced but did not prevent inundation for the lower energy events. Some ponding was evident in these scenarios, where wave overtopping volume is ponded between the barrier and buildings which could be problematic and difficult to manage with drainage. Another consideration is that there is a notable increase in the 1% exceeded water depth at the pool deck level because of wave reflection.

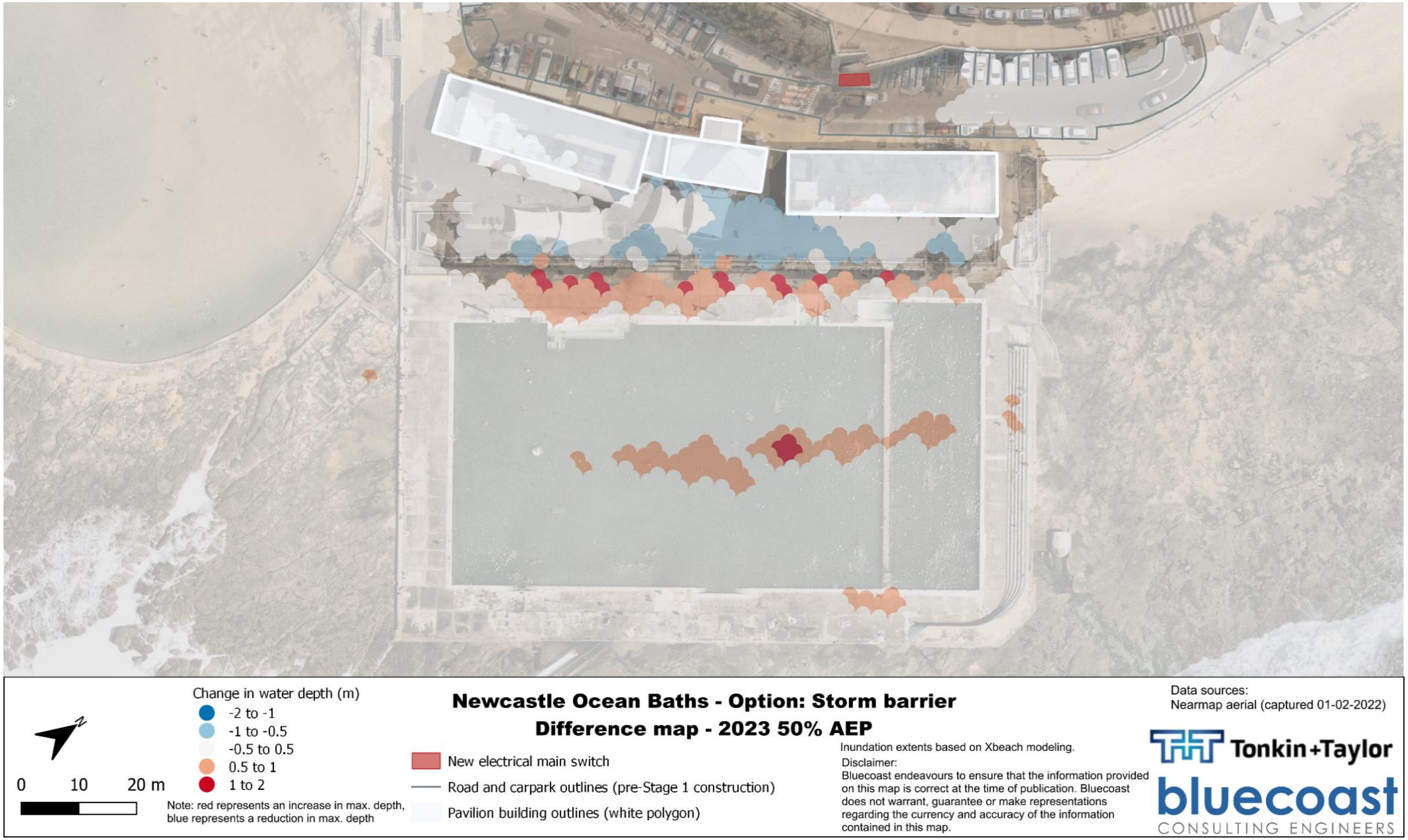


Figure 48: Difference map showing the change in maximum water depth between the baseline and storm barrier configurations.

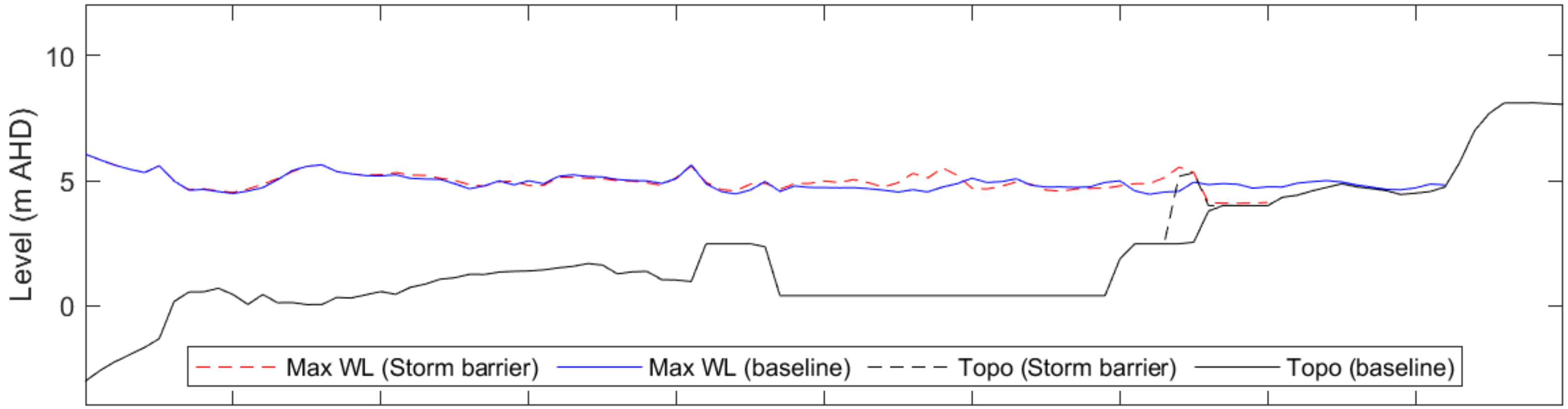


Figure 49: Cross section showing max. WL for baseline and storm barrier configurations (50% AEP 2023 input conditions).

Table 16: Results for 1% exceeded water depth with the storm barrier compared to the baseline configuration.

Scenario	Inputs		Pavilion deck level (1% ex. water depth)			Pool deck (1% ex. water depth)		
	Hs (m)	WL (m AHD)	Baseline	Storm barrier	Reduction factor	Baseline	Storm barrier	Reduction factor
50% AEP 2023	6.59	1.08	0.18	0.07	0.41	1.43	1.73	1.21
1% AEP 2023	8.50	1.30	0.43	0.35	0.81	2.00	2.55	1.27
50% AEP 2050	6.59	1.31	0.23	0.09	0.40	1.53	1.90	1.24
1% AEP 2050	8.50	1.53	0.46	0.37	0.80	1.99	2.50	1.26
50% AEP 2120	6.59	2.01	0.57	0.52	0.90	2.29	2.91	1.27
1% AEP 2120	8.5	2.23	0.91	0.83	0.91	2.77	3.55	1.28

Note: Note: reduction factor is *storm barrier depth/baseline depth*. Values less than 1 are a reduction, values greater than 1 are an increase.

7.4 Moving the bleachers (Option 3)

7.4.1 Description and rationale

In the existing location, the bleachers tend to block northward directed waves and flowing water from draining from the baths site while providing minimal benefit in terms of preventing waves entering the complex. This option involves relocating the bleachers to the southeastern corner of the complex (see Figure 50). In this position the bleachers would act as a barrier preventing some wave energy entering the complex. A secondary benefit would be that removal of the existing bleachers could improve drainage and reduce setup of water level in the pool.

Several considerations would be required, including:

- Heritage and visual impacts, which may render such an option unfeasible.
- Structure design to withstand wave forces over the design life (which are likely to be significant).
- Length and location could be optimised for reducing inundation exposure.
- Relocation of the bleachers may have negative effect on other operational and social elements of the baths including landscape visual and aesthetics.
- Effects on the local area need to be considered, including reflection that could influence coastal processes or nearby surf-breaks.

7.4.2 Model simulations of this option

The performance of a relocated bleachers option was assessed by adding the approximate topographic profile to the model bathymetry and simulating a set of repeat wave conditions in the XBeach model. The bleachers crest level in the model was approximately 5m AHD and the total length and ‘L’ shape were consistent with the existing structure, but in an alternative alignment (Figure 50). The same set of baseline simulations as used to compare the storm barrier were reused, with the exact same wave series for wave-by-wave comparison. Note that the model resolution of 2m x 2m does not replicate a perfectly vertical structure and some of the physical processes contributing to wave overtopping (e.g., implosive splashing) are not resolved in the phase-resolving XBeach model.

Results presented in Figure 51, Figure 52 and Table 17 generally show that an alternative bleachers location could be effective at reducing wave inundation to the full complex, including the pool, the pavilion deck level, the buildings, and carpark area. The 1% exceeded water depth on the pavilion deck level was 46% of the comparative baseline simulation for the 1% AEP event for 2023 and 2050, resulting in depths below 0.2 m, which may be manageable compared to the baseline scenario. Some benefit was also noted for the 2120 scenarios (Table 17).

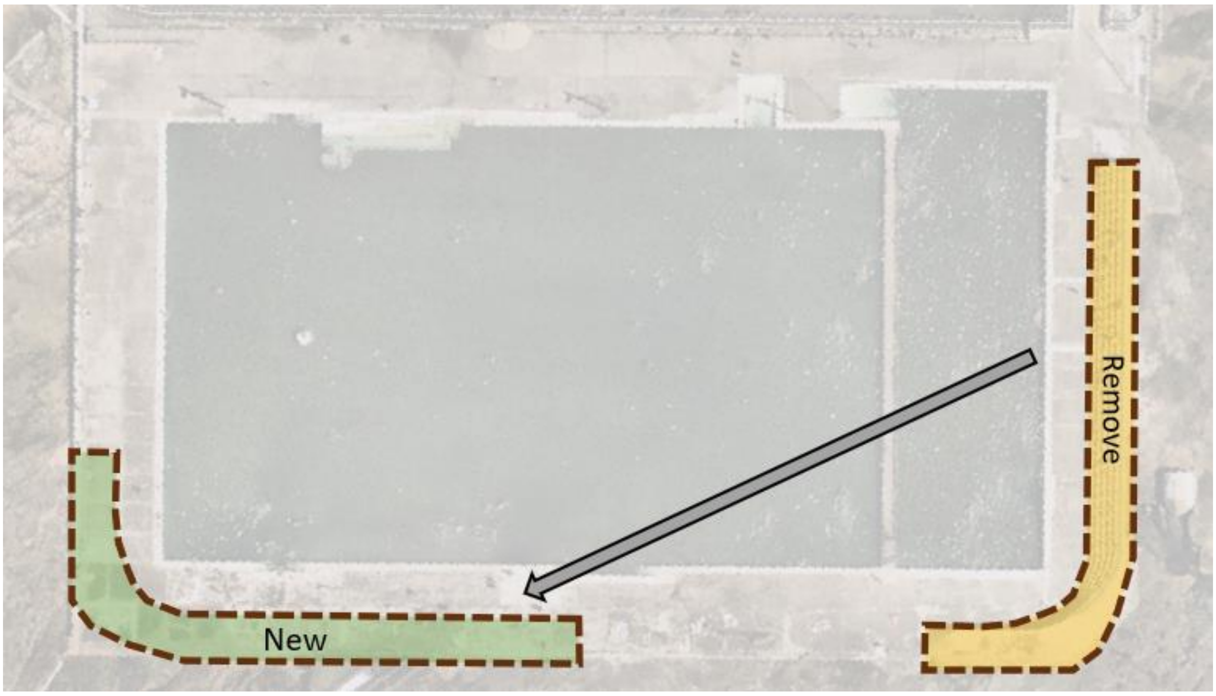


Figure 50: XBeach bathymetry showing baseline and moved bleachers configuration.

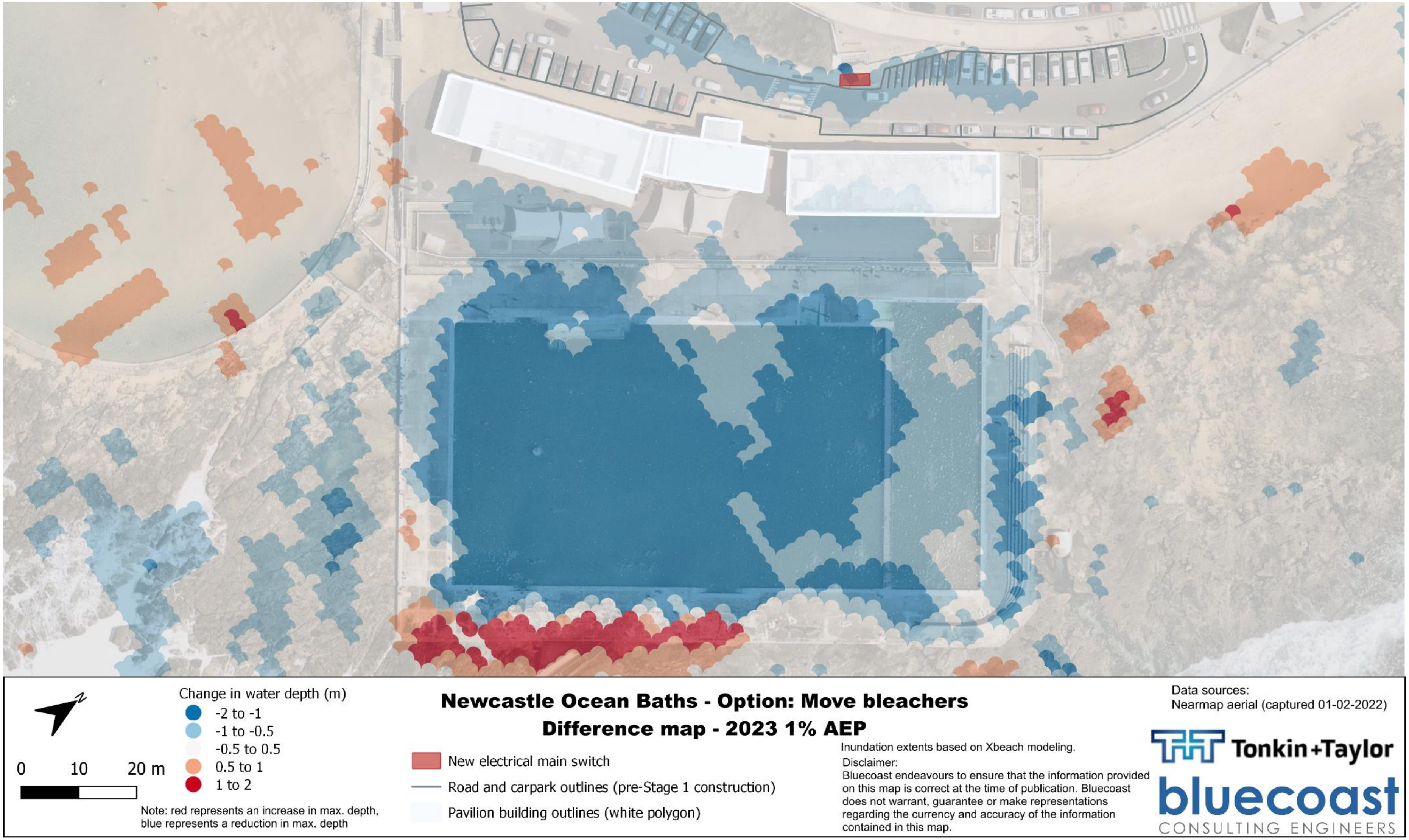


Figure 51: Difference map showing the change in max. water depth between the baseline and moved bleacher configurations.

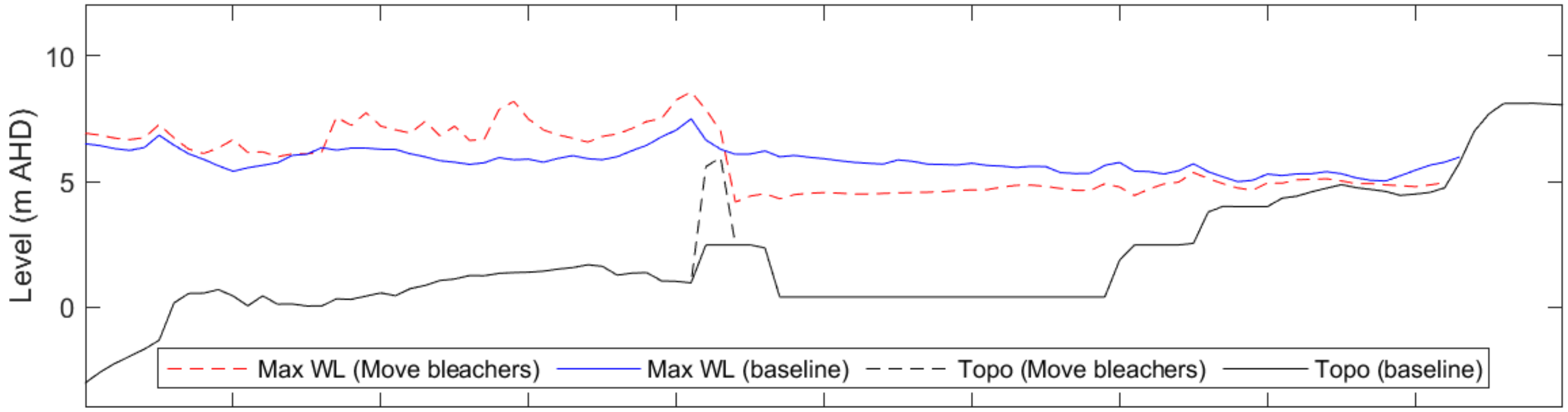


Figure 52: Cross section showing max. WL for baseline and moved bleacher configurations (1% AEP 2023 input conditions).

Table 17: Results for 1% exceeded water depth with the moved bleachers compared to the baseline configuration.

	Inputs		Pavilion deck level (1% ex. water depth)			Pool deck (1% ex. water depth)		
Scenario	**Hs (m)**	**WL (m AHD)**	**Baseline**	**Move bleachers**	**Reduction factor**	**Baseline**	**Move bleachers**	**Reduction factor**
50% AEP 2023	6.59	1.08	0.18	0.03	0.17	1.43	0.76	0.53
1% AEP 2023	8.5	1.3	0.43	0.20	0.46	2.00	1.26	0.63
50% AEP 2050	6.59	1.31	0.23	0.07	0.29	1.53	0.88	0.57
1% AEP 2050	8.5	1.53	0.46	0.21	0.46	1.99	1.39	0.70
50% AEP 2120	6.59	2.01	0.57	0.30	0.52	2.29	1.65	0.72
1% AEP 2120	8.5	2.23	0.91	0.63	0.69%	2.77	2.16	0.78

Note: reduction factor is *moved bleachers depth/baseline depth*. Values less than 1 are a reduction, values greater than 1 are an increase.

7.5 Monitoring and response measures

Building on the event likelihoods outlined in Section 6.6, this complementary measure intends to set out a framework for the development of a Trigger Action Response Plan (TARP) that could be utilised at this or similar facilities. This framework takes the modelling and assessment results from this study and extends them by incorporating operation experience and desired outcomes from the facility owner and operator to deliver a usable tool for managing inundation hazard, these steps are shown in Table 18.

Table 18: Framework for developing TARP.

Item	Description	Required inputs
Draft response protocols and triggers	Based on the experience of previous operators defined protocols for actions should be developed based on known thresholds and consequences. The final configuration of the Stage 2 building should be considered when developing the triggers as this will affect the available responses whether they be restriction to access or supplementary protection.	Facility owner Facility operators Community users
Early warning systems	Following the developed triggers, the metocean condition that are likely to activate them can be identified and fed into an early warning system to alert operators when there are likely to be inundation events and their corresponding severity.	Technical personnel
Ongoing monitoring and refining of triggers and response	Monitoring of conditions at the facility will allow the validation and improvement of the early warning system and revision of the triggers. This could be in the form of logs taken by staff on site as well as instrumentation, such as that used in the May2023 monitoring event described in Appendix A:. Long term data gathering is the most useful way to refine the triggers and responses based on the observed conditions. Monitoring is likely to take the form of a permanent high frequency pressure sensor installed in the pool and a camera or more than one camera located to observe and record overtopping and inundation.	Facility operator Facility owner Technical personnel
Public awareness and education	For this, and similar, public facilities, awareness and education of inundation risks ensure that the facility can remain open as much as possible, without compromising safety.	Facility owner Community users

The above points are intended to provide a framework for the development of an ongoing trigger and response plan that could be implemented at this and other similar facilities. It is suggested that this program could be initiated and if successful implemented across a wider range of assets through an action in Southern Beaches CMP process as these facilities are local government assets and the process already includes detailed analysis of the wave and water level conditions, including inundation assessment.

7.6 Resilient building measures

Complementary to the structural options given above, designing resiliency into the layout and internals can decrease the likelihood of damage to the facility. Table 19 presents options and examples of measures that can be considered during the design phase of Stage 2.

Table 19: Measures to improve resilience of facilities.

Measure	Example
Elevated Electrical Connections: Elevate all electrical installations to mitigate potential damages from ingress.	
Example: Following the 2010-11 Queensland floods new measures were put in place to ensure that all main switch boards and electrical sockets were installed above recorded flood levels.	 Source: ABC
Barriers at Openings: Install flood barriers at entry points to deter ingress.	
Example: Low lying countries such as the Netherlands and Belgium deploy movable storm wall to reduce impacts and ingress by overtopping waves.	 Source: EurOtop 2018
Weathertight Doors: Use specialized doors to prevent water intrusion.	
Example: The Oceanographic Institute in Monaco has integrated ship-like weathertight doors. These doors ensure that labs and exhibits, housing sensitive marine life and equipment, remain safeguarded from potential sea surges	 Source: Shutterstock
Reinforced Windows and Shutters: Install storm-resistant windows and shutters to shield from impact or splashing.	

Measure	Example
Example: Buildings along the Florida coastline, especially in hurricane-prone areas like Miami, often feature reinforced windows or protective shutters. This design consideration ensures that the interiors remain shielded during storms, reducing potential damage from wind-borne debris.	 Source: Guardian Hurricane Shutters

8. Summary

A comprehensive coastal inundation assessment of the Newcastle Ocean Baths facility has been conducted based on the Stage 1 design in preparation for Stage 2 renewal. A review of historic metocean data as well as project specific monitoring during a significant inundation event was conducted to inform XBeach modelling scenarios to understand the inundation likelihoods and extents.

A summary of the key outcomes of the inundation assessment is given below:

- At present day sea levels, the facility will be inundated at a depth of 0.33m for the 2% AEP scenario and 0.78m for the 1% AEP scenario (depth above the Stage 1 ground level).
- For the 1% AEP scenario the 2050 and 2120 SLR projections will increase this to 0.83 and 1.27m respectively.
- Inundation mapping showed that higher water depths will occur in front of the central and northern pavilion buildings.
- The carpark area landward of the pavilion buildings will be subject to water depths up to 0.5m for the 1% AEP scenario at present sea levels and up to 1m for 1% AEP scenario with 2120 projected SLR.
- The probabilistic inundation assessment concluded that these water levels would reach the pavilion deck up to 3 events per year.

Based on the inundation assessment, inundation mitigation measures were investigated using modelling and considered the feasibility of implementation These included:

- Raised floor levels: increased internal floor levels to reduce impact of inundation on internal fit out. The Stage 2 designers can utilise results from the inundation assessment to make an informed decision.
- Temporary inundation barrier: to be installed at the pavilion deck level to decrease the impact of inundation on the pavilion buildings. Model results showed a decrease in water levels as a result for lower energy inundation events.
- Moving bleachers: removal of existing bleachers and building new ones in the southeast corner of the pool. Modelling found that this would lead to a decrease of water levels at the pavilion deck of between 0.5m and 1m for the present day 1% AEP scenario.

- Monitoring and response measures and resilient building measures were also considered as complementary options to the above which could be implement alongside or independent of the above options.

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Appendix A: Coastal inundation monitoring

Introduction

The processes contributing to coastal inundation at the Newcastle Ocean Baths site are complex. Sufficient nearshore metocean and survey data existed. However, while the imagery provided by community was useful in qualitative sense, there was no measurements of wave, water level, flow or overtopping during a coastal inundation event to describe the nearshore wave and water level climate to create a seamless description of the site's elevations. To remove this gap and improve the project outcomes, a coastal inundation monitoring exercise was undertaken during a large swell event that occurred in early May 2023. The objectives of the inundation monitoring were to:

- Improve the understanding of wave, flow and overtopping dynamics during large wave events with high water levels.
- Improve the calibration and/or validation of the XBeach numerical model being developed and thereby improve the confidence in the inundation assessment.
- These objectives were achieved by simultaneously measuring wave heights and water levels across the pool site using metocean instrumentation as well as imagery collected from fixed cameras and a drone. These targeted and high-quality data lead to an improved understanding of the inundation processes and an improved reliability and accuracy of numerical modelling.

Scope of monitoring

Inundation monitoring at the Newcastle Ocean Baths occurred over three (3) days in early May 2023 when a storm with large waves and elevated water levels which impacted the central and southern NSW coast. The instruments used to capture waves, currents and overtopping are outlined in Table 20. Figure 53 provides a map of the instruments deployed around the baths site. The instruments were deployed on the morning of the 8 May and recovered on the afternoon of the 10 May.

The location of each of the monitoring sites were selected for spatial coverage of key areas of interest and were informed by the previously captured videos and images of historic large swell events. The final position of the instruments was levelled using a Trimble R8 RTK (survey grade GPS). The aerial shown in for Figure 53 was captured by drone on 5 May 2023 and reflects the configuration of the site during the monitoring event as works had commenced on Stage 1 of the renewal project. This information assisted with the modelling calibration process (see section 5.4.3) as the geometry previously being used for calibration did not include several of the features such as: new concrete pool deck, higher pavilion deck level and stairs/bleachers in front of the pavilion.

Table 20: Instruments used for inundation monitoring.

Instrument	Purpose	Location	Coordinates (GDA94Z56)	Elevation (m AHD)
ADCP (Aquadopp)	Measurement of waves, current, water levels	Pool floor	386,976 6,355,888	1.005
ADCP (Eco)	Measurement of currents	Northern pool deck, above drainage culvert	386,976 6,355,925	2.083

Instrument	Purpose	Location	Coordinates (GDA94Z56)	Elevation (m AHD)
Pressure transducer (RBR)	Measurement of water level/wave height	Pool deck at base of retaining wall	386,934 6,355,884	2.495
Monitoring camera (x2)	Capture wave dynamics and overtopping	Bleachers looking south. Building looking east	386,986; 6,355,918 386,928; 6,355,899	-
Drone footage (x2)	Capture wave dynamics and overtopping	Above site during the two high tides on the 8 May		-

Description of monitoring instruments

The following instruments were used for the inundation monitoring:

- 2 x ADCP type instruments (Nortek Aquadopp Profiler (2MHz) and Nortek Eco Current Profiler)
- High frequency pressure transducer (RBR Virtuoso^3)
- Cameras (Wyze Cam v3 Pro, DJI Phantom 4 Pro)

ADCP type instruments

Acoustic Doppler Current Profiler (ADCP) type instruments were used to measure current, waves and water depth. These instruments measure the flow velocity of water by transmitting short sound pulses and measuring the Doppler shift of the reflected signal. The acoustic signal is reflected by 'scatters' (small particles) assumed to be passively flowing in suspension. By measuring wave orbital velocities and sea surface elevation at a high frequency in a similar fashion to the above, the surface displacement can also be calculated using the principle of linear wave theory. More details of the specific instruments are provided below.

- The Nortek Aquadopp Profiler 2 MHz is a highly versatile ADCP with an operational frequency of 2,000kHz, most suited for very shallow water deployments (< 10 metre water depth). The right-angle head Aquadopp adopted in this study has a low-profile design making it ideal for shallow depths. This instrument is simple in design, having three angled beams used to measure 3D water velocities.
- The Nortek Eco is a small ADCP unit ideally suited to shallow water deployments where only current measurements are required. It features built-in GNSS, temperature, pressure and tilt sensors and utilises the same acoustic doppler technology as other Nortek units. It records the temperature and pressure along with current velocity and direction in three bands of the water depth.

Configuration of the ADCP units for the deployment are provided in Table 21. Figure 54 shows photographs of the instruments in their deployment location. The Aquadopp was mounted on top of 360mm blocks to raise it off the pool bottom. This was to ensure it would not be buried in the sand which was present in the pool prior to the monitoring event. The Eco was mounted in a low-profile tripod frame and bolted into the concrete in the deployment location shown in Figure 53.

Figure 53: Deployment locations for monitoring instruments and equipment.

Table 21: Configuration used for ADCPs.

Parameter	Aquadopp	Eco
Instrument(s)	Nortek Aquadopp 2MHz	Nortek Eco
Mounting	Upward looking	Upward looking
Vertical resolution (bin size)	0.5m	Dynamic
Blanking distance	0.2m	0.1m
Current measurement interval	1,800s	120s
Current averaging interval	60s	Dynamic
Wave measurement interval	1,800s	-
Wave sample duration	1,024s	-

Figure 54: Mounting arrangement of Aquadopp (left) and Eco (right).

High frequency pressure transducer

The RBR Virtuoso^3 high frequency pressure transducer can measure water depth at high frequency, from which water level and wave height can then be determined. The Virtuoso^3 was mounted to an aluminium channel which was bolted to the concrete. Configuration information is provided in Table 22 while Figure 55 shows the mounting location.

Table 22: Configuration for the RBR pressure transducer

Parameter	Value
Instrument	RBR Virtuoso
Mounting	Bolted onto pool deck
Measurement interval	Continuous
Raw sampling rate	16Hz

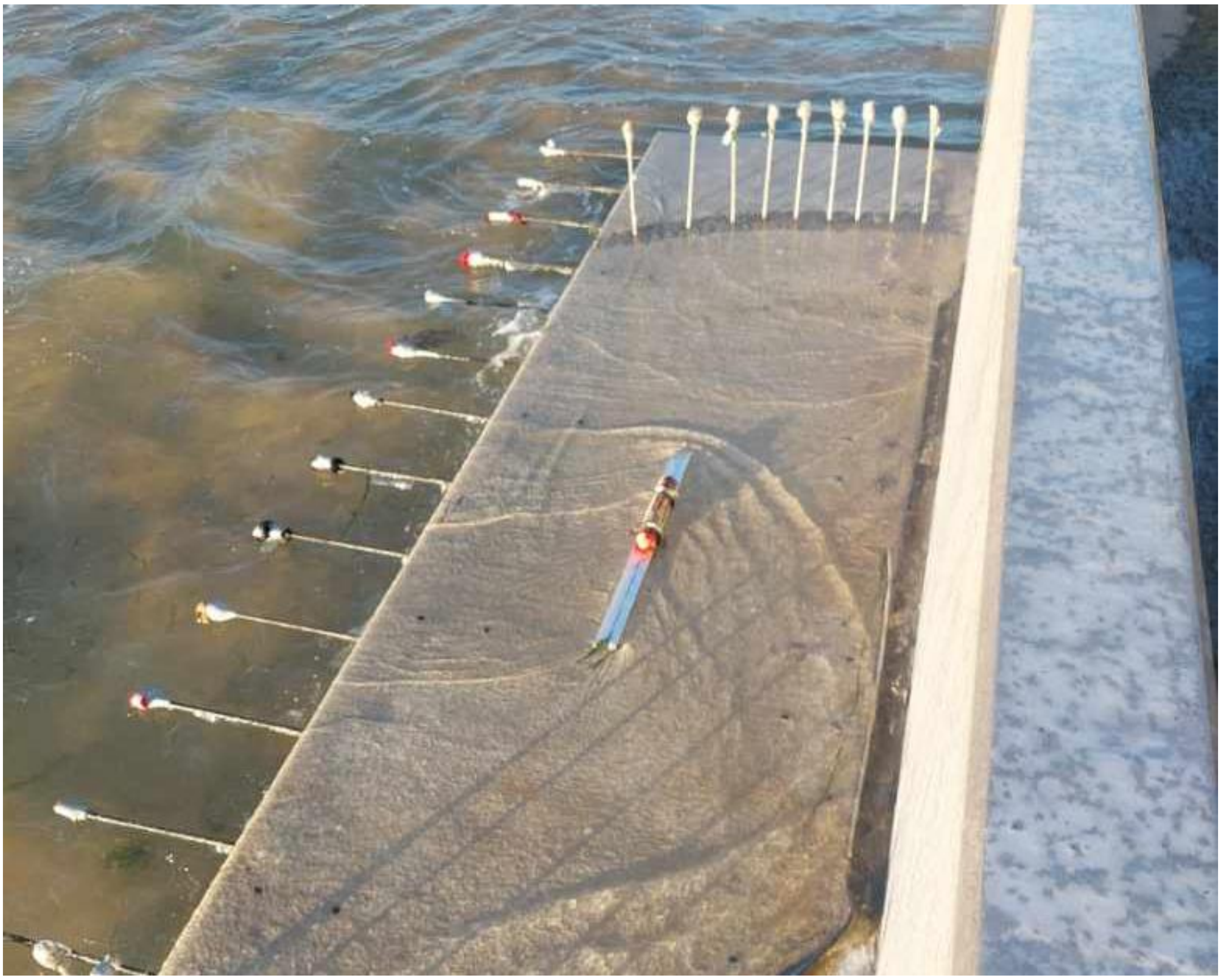

Figure 55: RBR pressure transducer mounted prior to event.

Cameras

Two types of cameras were used to record the wave dynamics and instances of overtopping and inundation:

- Wyze cam v3 Pro cameras are capable of recording 2K video resolution directly to a microSD card. The camera automatically splits the recordings into 1-minute videos. These cameras are IP65 weather resistant and powered by a battery or power supply.

- The DJI Phantom 4 Pro v2 is a UAV (or drone) capable of recording 4K video at 60 frames per second. 30-minute flight times with intelligent flight control allows it to be flown safely in moderate to high winds and variable conditions.

Two Wyze Cam Pro cameras were used to continuously capture video of waves moving across the pool and overtopping of the wall in front of the pavilion. The building camera was connected to power supply available from the pavilion. The bleachers camera was battery powered. Figure 56 shows the field of view of each camera.

Figure 56: Field of view for camera mounted to the pavilion building (left) and on the bleachers (right).

Drone footage was captured using the DJI Phantom Pro 4 at both high tides on 8 May 2023 (approximately 10:12am and 10:15pm). Example images are shown in Figure 57.

Figure 57: Example images captured from the drone video at the morning high tide (left) and evening high tide (right).

Monitoring results

QA processing

Raw data from the metocean instruments has been processed into quality assured (QA) data using the below workflow.

- The instrument manufactures' software was used to process the raw measurements and to produce measurements for the various parameters.
  - For the Aquadopp ADCP, Nortek Storm was used to derive wave estimates in both time and frequency domains using the PUV methods. The data was separated into bursts and output to a Matlab data structure.

- The Nortek Eco data is processed through a dedicated app which undertakes QA and filters out any unsuitable data.
- For the RBR pressure transducer, Ruskin software was used to undertake initial screening.

- Further post-processing of outputs from the instrument manufacturer software was then undertaken to ensure spurious data not detected are removed. For waves, this processing included, applying a band-pass frequency filter to the pre-processed data to ensure wave periods outside of the detectable frequency band are excluded. Spurious data was also removed using a manual data selection tool and despiking tool. This further post-processing was undertaken in Matlab using Bluecoast's metocean data analysis toolbox.
- All water depth (measured above the sensor head) were reduced to water level above AHD using spot heights of the sensor head collected using the RTK.

Results – Waves and water levels

Timeseries plots of wave and water level from the inundation monitoring instruments (Aquadopp and RBR) as well as the ocean conditions from the Newcastle nearshore WRB and Patonga tide gauge are shown in Figure 58. The following observations are noted:

- As would be expected, wave heights within the bath's sites are tidal modulated, with larger wave heights measured during high tide. While the significant wave heights at the WRB remained relatively consistent throughout the monitoring period (middle plot, Figure 58). At both instrumented sites the peak wave heights occur at the two high tides on 8 May, with the larger wave heights occurred at the higher high tide (during the evening).
- Water levels above the instrument height do not register until approximately 0600 on the 8 May when the significant wave height reached 4m and the tide rises above about -0.2m below AHD. From this time the pool began to fill due to wave overwash, with water entering the pool faster than it drains. The pool is full by approximately 0700, the pool remains full until low tide on 9 May when the low water level and declining wave allowed the pool to drain out. The drainage gates in the northwest corner of the pool were open during the monitored event. This outlet, a short culvert that discharges to the Cowrie Hole rock platforms, is at approximately 0.5m AHD.
- The pool deck level is approximately 2m AHD and the water level at the Aquadopp represents the pool being up to 0.5m above, or super elevated above the pool deck. The super elevation of the water levels in the pool area results from broken waves pushing water across the rock platform into the site. Swell waves travel in groups or sets. When a large set breaks across the site the water levels set-up (i.e., this process is referred to as surf beat). The hydraulics of the site, including the curved bleachers on the northern side act to (i) capture this north-north-west flow during large wave sets and (ii) hinder the drainage of water from the site in between waves or in between sets of waves. Surf beat leads to the second or third wave of the set having greater penetration from offshore into inside the pool thereby generating greater overtopping of the pavilion retaining wall with potential to inundate the buildings.
- The peak wave periods recorded at the two baths sites were consistent with the peak wave periods recorded at the Newcastle nearshore WRB.

The results from the Eco are pre-processed through Nortek's Eco software. The velocity data did not meet the instruments internal standards required to output reliable data, so no current readings were available for this location. The instruments pressure sensor recorded water depth and the timeseries is shown in Figure 59. These results should be approached with caution as (i) the flow in this area chaotic and prone to short surges (ii) depth recordings are averages and may not necessarily reflect the depth of a specific wave train.

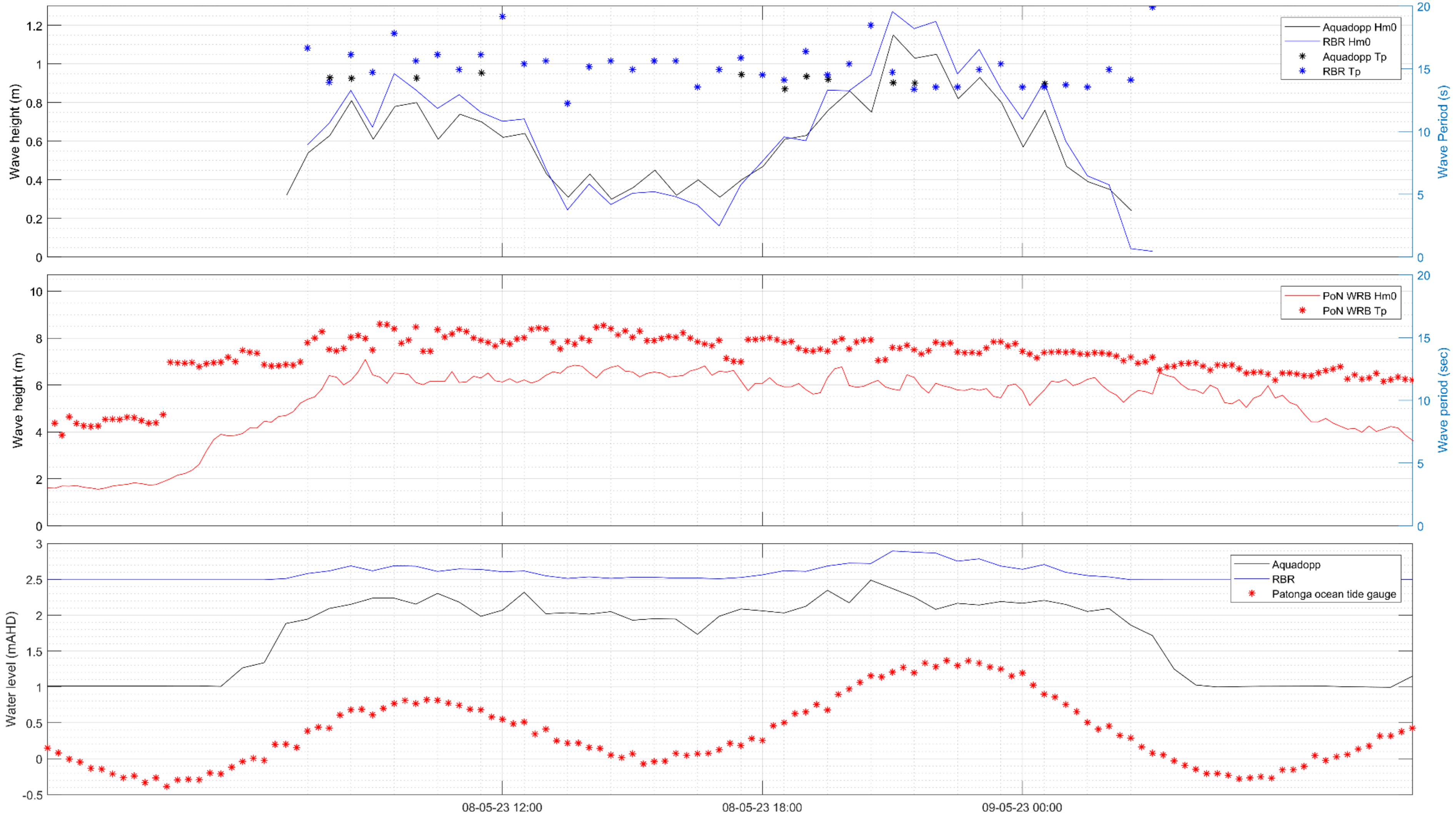

Figure 58: Timeseries showing Hm0 and Tp at the baths sites (top), Hm0 and Tp at the Newcastle nearshore WRB (middle), water level at the bath's sites and Patonga tide gauge.

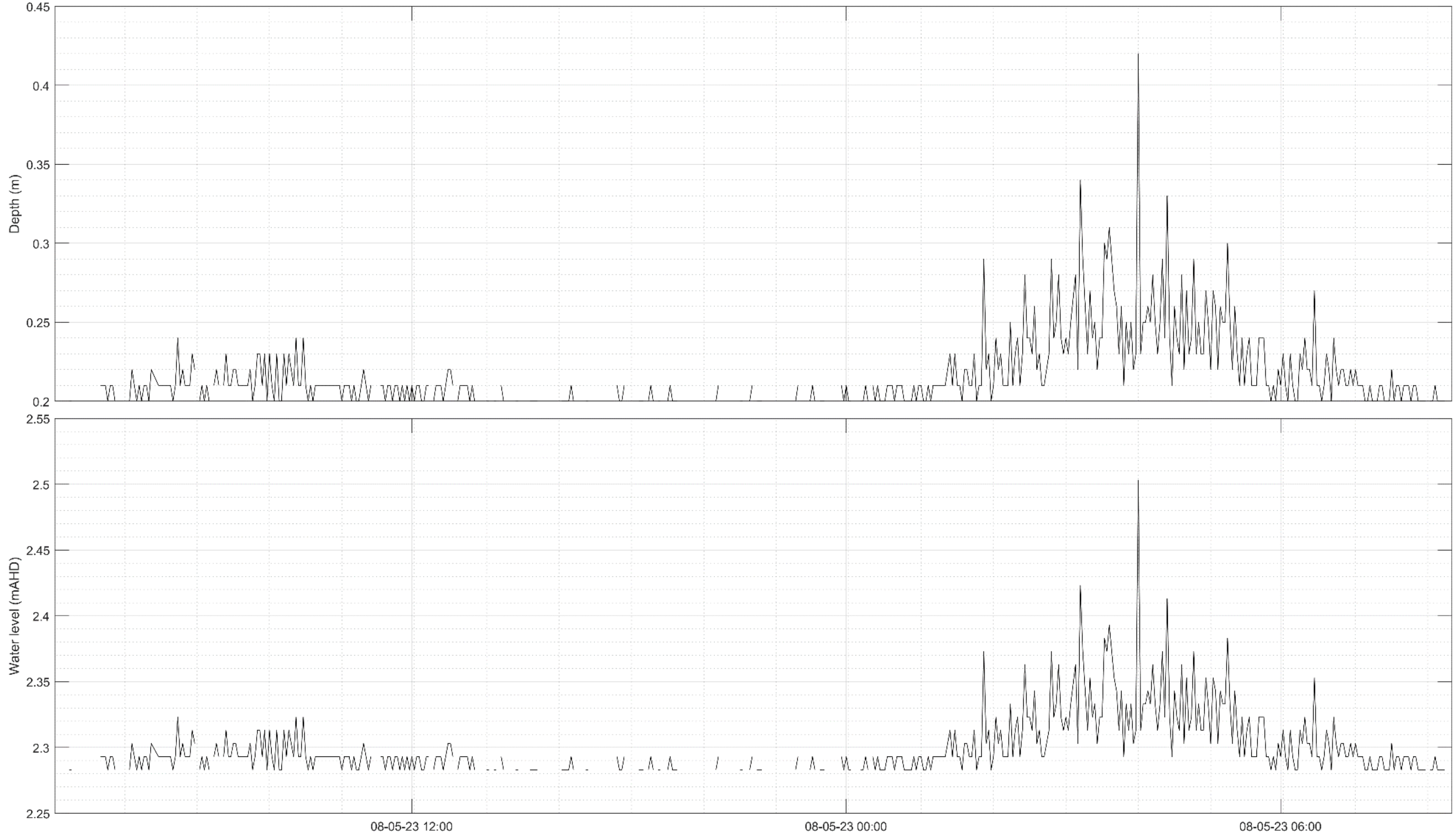

Figure 59: Timeseries of water level recorded at Eco.

Results – Cameras

Imagery from the cameras supplemented the more quantitative measurements from the instruments. Specifically, the cameras allow measurements of the degree of overtopping (incidents and intensity) and inundation at the pavilions. During the monitoring period both the building and bleachers cameras captured overtopping and inundation of the pavilion as well as the overwash wave dynamics moving across the pool.

Figure 60 shows an overtopping event captured by the building camera. Also shown is the corresponding water level captured by the RBR which was located directly in front of the wall in the immediate field of view. The sequence is:

- the lowest water level as the wave approaches
- a spike as the wave hits the retaining wall
- a reduction and short sustained period as the wave moves past the sensor.
- This is highlighted in image 3 where the peak has past the sensor and the wave is impacting the building, but the level is still sustained over the instrument's location. The slow reduction in water level following image 3 is due to the water draining back from the pavilion deck into the pool.

As shown in Figure 61, the crest of the pool entrance (i.e., between the two buildings) is approximately 5m AHD. The landward carpark is at a slightly lower level. While the extent of the water ingress from the overtopping event shown in Figure 60 was not captured past what is shown, the flow volume shown in the video and the height of impact shown on the corner of the northern building indicate that this flowed through the entrance into the carpark. This is confirmed by the event captured by the drone vantage in Figure 61 where water can be seen flowing into the carpark. The building camera captures the height of impact on the corner of the building.

The angle of the wave approach to the building and the number of consecutive large was seen to influence where the largest impacts were on each of the buildings. Figure 62 shows successive waves impacting the pavilion buildings as captured by the drone at approximately 10.27pm on 8 May. The first wave was seen to impact the southern building significantly, while the overtopping from the following wave barely reached the building. Conversely, the northern building experienced a much larger impact on the second than the first wave. It was observed in the video that the water level in the northwest corner increased during consecutive waves which then increased the severity of the impact of overtopping on the building.

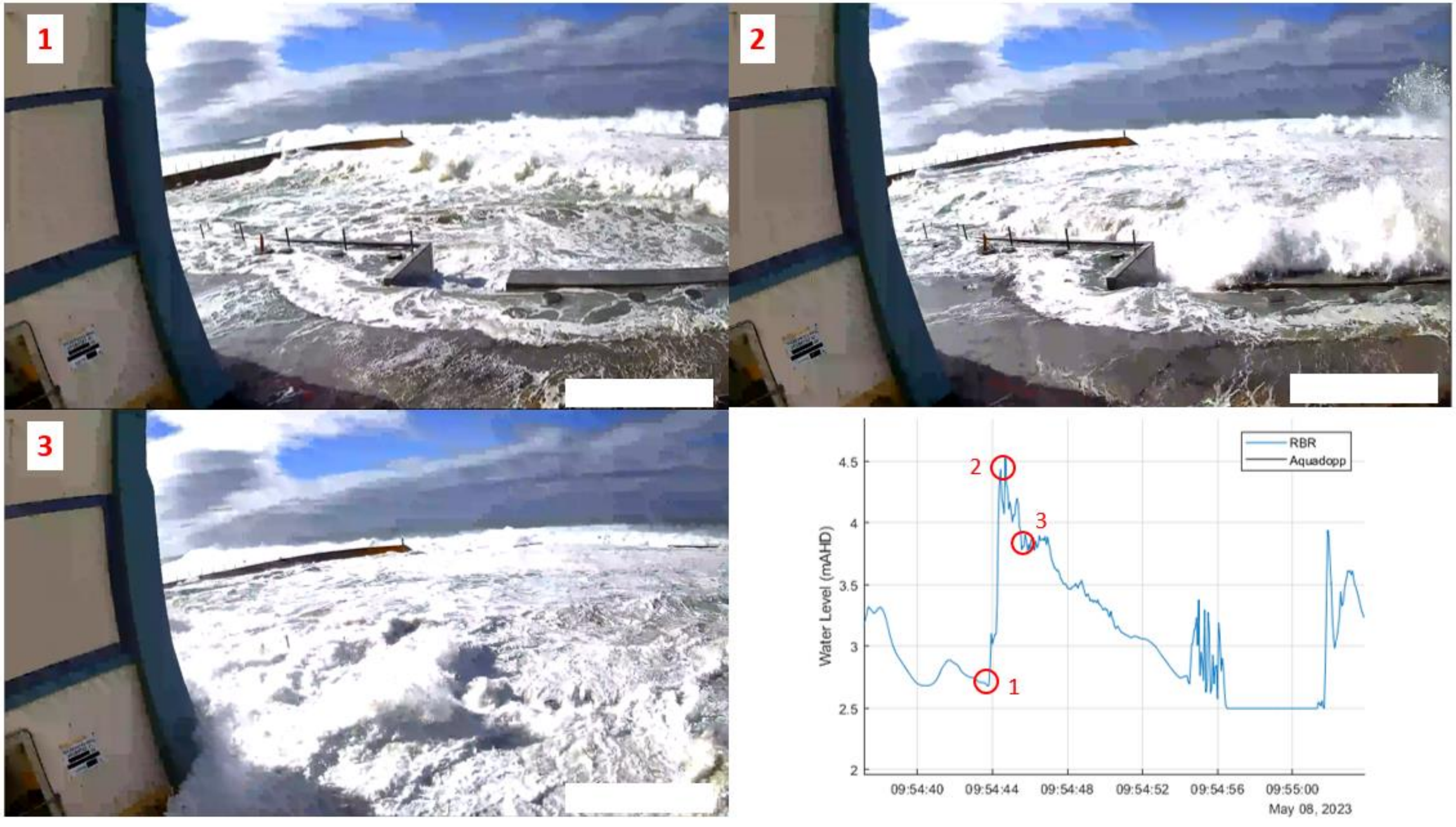

Figure 60: Overtopping captured by the building camera and corresponding water level recorded by the RBR.

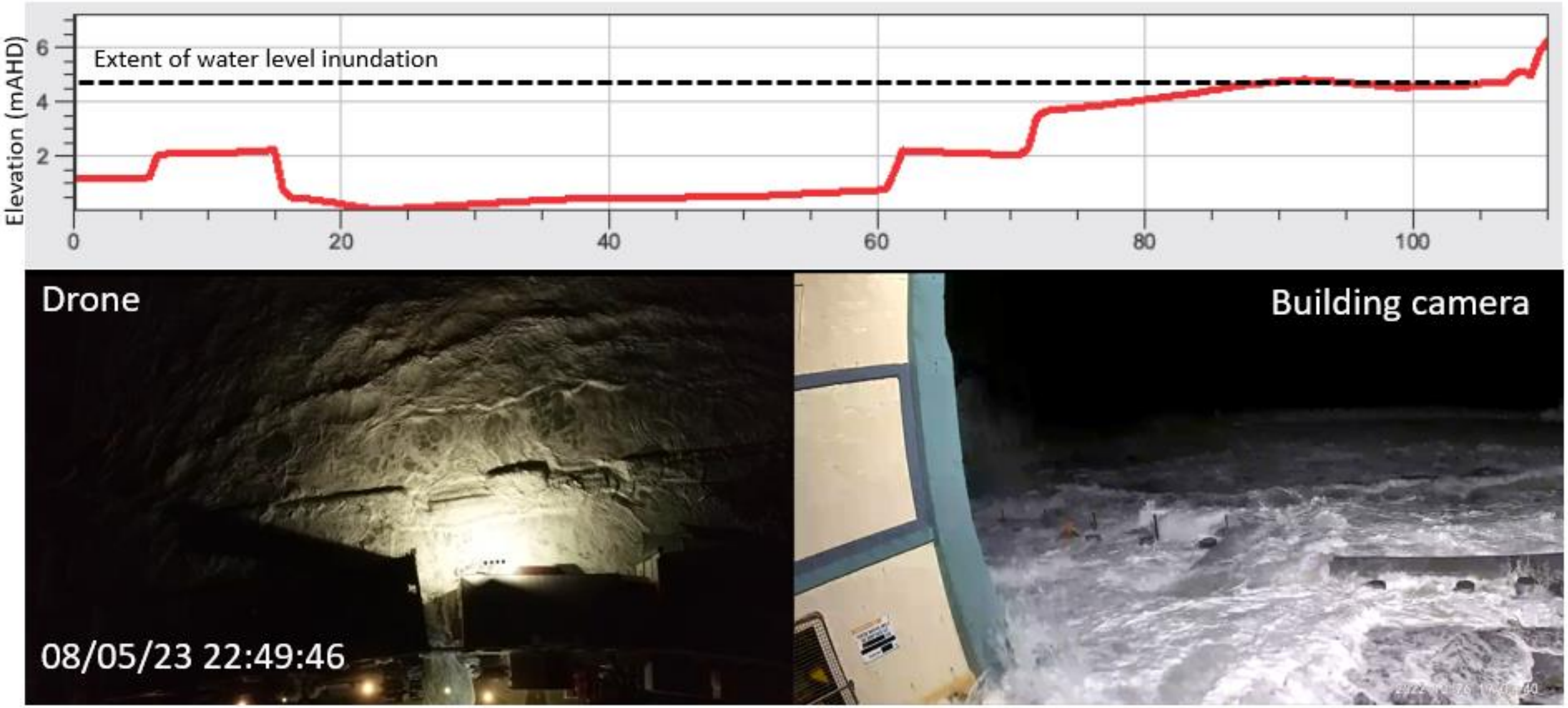

Figure 61: Observed inundation level derived from the drone marked against the digital elevation model for the pre-works facility.

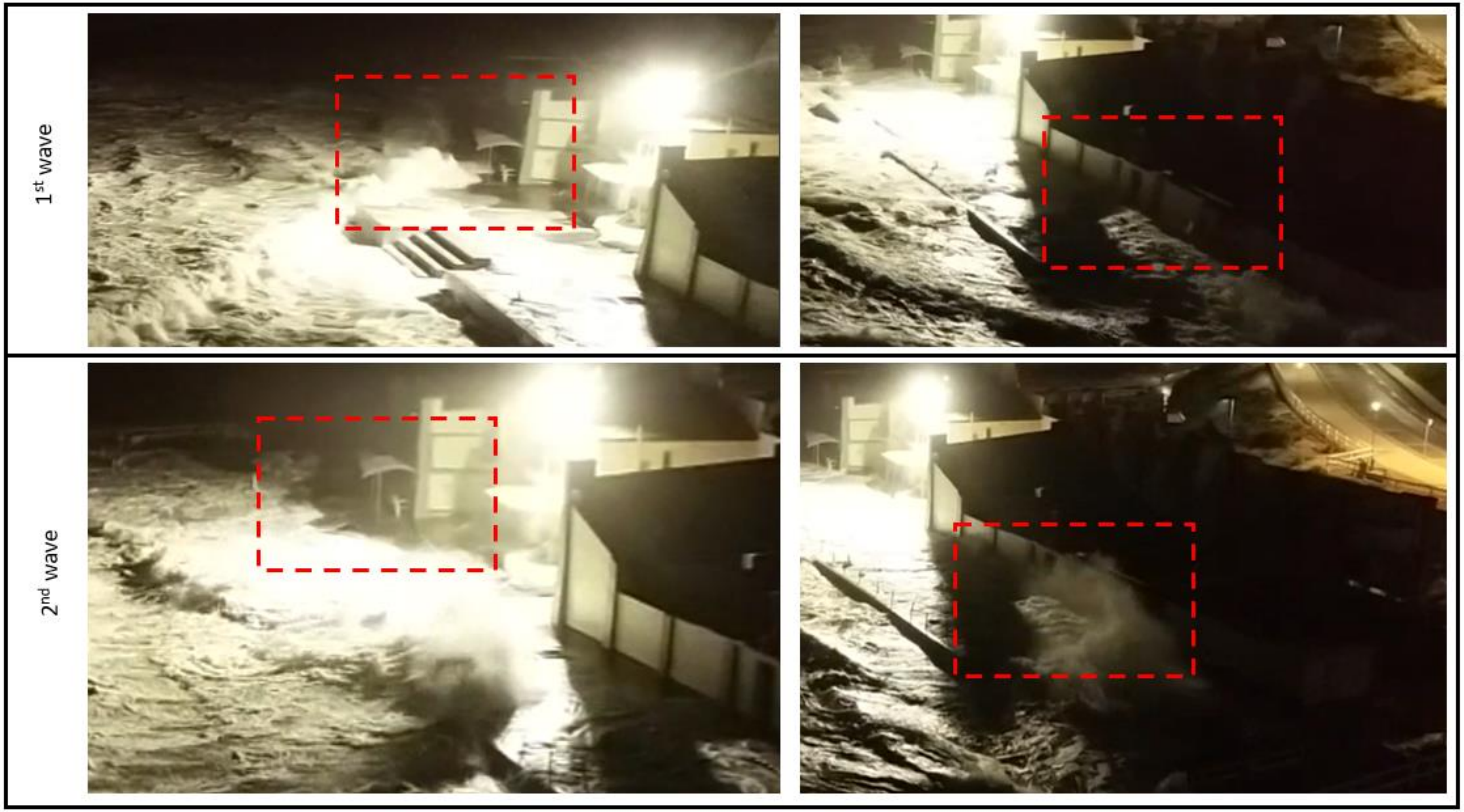

Figure 62: Successive wave impacts on the pavilion buildings.

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Appendix B: Technical input to guiding principals

Methods for assessing coastal inundation

Several methods can be employed to assess coastal inundation. A summary of methods is presented in Table 23. These methods range from generalised empirical approaches such as bathtub mapping, to computationally advanced numerical modelling. Choosing the best method for assessing coastal inundation depends on:

- the characteristics of the site being assessed (e.g., its size and complexity)
- the available data
- the purposed of the assessment (e.g., the processes that need to be resolved and the required outputs (e.g., levels, extents, depths, flow dynamics and/or wave forces). Often, a combination of these methods is used to gain a comprehensive understanding of coastal inundation risks.
- the level of detail, accuracy and certainty required
- the available effort and time that can be expended.

For this assessment the selected approach is centred around the use of a phase resolving wave model (XBeach) supported by:

- data analysis of high quality local forcing data
- targeted field measurements of coastal inundation at the baths site
- probabilistic analysis by combining input data and model results to determine the exposure to inundation hazard (now and in the future) different events (like storm surges or sea-level rise)

This provides the best balance between resolving the required processes within an achievable scale and timeframe. The use or more details methods are not easily justified. Representing the entire baths site and nearshore bathymetry in a 3D basin type physical model would be difficult and far exceed the assessment's budget. A CFD model would provide additional resolution (both processes and spatially) but simulation times for be such that only a very limited number of scenarios could be simulated.

Table 23. Summary of methods for assessing coastal inundation focused on numerical modelling.

Method	Processes resolved	Limitations	Benefits
Bathtub	Empirical calculations of quasi-static and/or dynamics water level components that are mapped as contour levels on the terrain to identify inundation areas.	• Does not account for spatially or temporally variable processes that influence surge or wave runup levels. • Does not represent site specific factors for dynamic processes of runup and overtopping.	• Does not require detailed information on bathymetry (required for hydrodynamic modelling). • Can be applied at regional or national spatial scales to represent many scenarios (e.g., 0.1m sea level rise increments).
Hydrodynamic model	Quasi-static water level components.	• Does not resolve waves. Setup can be added as a tail water but that is not realistic.	• Works well in areas with minimal wave effects like estuaries that have significant tidal flows and tidal geometry behaviour.

Method	Processes resolved	Limitations	Benefits
		• Requires detailed bathymetric inputs and calibration.	• Not generally limited by temporal or spatial scales of interest to events.
Phase-average wave model	Quasi-static components with dynamic inundation from infragravity wave runup and overtopping flows.	• Requires detailed topographic and bathymetric terrain data. • Requires site specific calibration. • Temporal and spatial resolution: • Temporal (days) • Spatial (~<100 km^2)	• Resolves velocity and free surface flow and the wave group. • Wave setup is resolved spatially in a spatially and temporally dynamic way. • Resolving the surf-beat flow is a computationally efficient way to map inundation that is suitable for embayment wide inundation assessments. • Resolves fundamental processes attributed to significant inundation from waves and quasi-static processes.
Phase resolving wave model	All quasi-static and dynamic inundation processes	• Requires detailed topographic and bathymetric terrain data. • Requires site specific calibration. • Does not resolve injected water flow in the air during splashing and overtopping (limited to surface flow). • Temporal and spatial resolution: • Temporal (hours) • Spatial (~<10 km2)	• Resolves velocity the free-surface flow from all water surface processes contributing to coastal inundation, including SS and IG waves. • All processes are dynamically coupled and influence each other.
Computational fluid dynamics (CFD) model	Focused on wave effects (setup, runup, overtopping) above the storm tide level.	• Requires detailed topographic and bathymetric terrain data. • Requires site specific calibration. • Temporal and spatial resolution: • Temporal (minuets) • Spatial (<1 km2) • Computationally slow which limits scenarios.	• Resolves three-dimensional processes including water injected into the air. • Resolves 3D terrain features such as recurved walls.

Method	Processes resolved	Limitations	Benefits
Reduced scale physical model	Focused on wave effects (setup, runup, overtopping) above the storm tide level.	• Temporal and spatial resolution is scale and basin/flume dependent. • Can be difficult to measure desired outputs (e.g., limited wave probes)	• Resolve complex 3D terrain and associated flow dynamics. • Scaling laws are well understood and if guidance is followed physical models don't generally need calibration. • Are commonly used in engineering design to represent future extreme scenarios.

Methods for assessing hazards

Assessing forces imposed on buildings by waves is difficult due to the complex and variable flows associated with wave overtopping and the highly site-specific nature of the environment. This means that empirical methods developed for simplified cases are not generally applicable to specific sites and more detailed methods are generally required. This section reviews available literature on the subject and provides a recommendation for assessment of forces on buildings at the Newcastle Baths.

Tolerable overtopping

In coastal engineering design, the effect of wave overtopping flows on buildings is typically considered through tolerable limits on the mean overtopping discharge and maximum volumes. These tolerable limits are set out in the EurOtop (2018) and USACE (2006) design guidelines. While these values can be assessed by empirical (for simplified cases) or numerical methods, the resultant values do not provide guidance on the actual loads that should be designed for, but rather if a hazard exists.

Table 24. Tolerable overtopping flow limits for buildings, from Table 3.2 in EurOtop (2018) and Table VI-5-6 in USACE (2011).

Source	Hazard description	Mean discharge q (l/s/m)	Max Volume V_{max} (l per m)
EurOtop (2018)	Damage to building structure elements for H_{m0}=1-3m	1	1,000
USACE (2011)	Minor damage to building fittings, signposts, etc.	0.001	Not specified
USACE (2011)	Structural damage to buildings	0.03	Not specified

Flood flow limits

Some literature also considers building stability under flood flows (e.g., Ball et al., 2019). Further to these equations relying on flow depths and velocities when determining impact forces, building stability under both flood flows and tsunami flows is more broadly considered in terms of flow depth and velocity relationships (Figure 63). These stability relationships are based on the collation of numerous empirical studies which have explored damage to structures under flood and tsunami impacts.

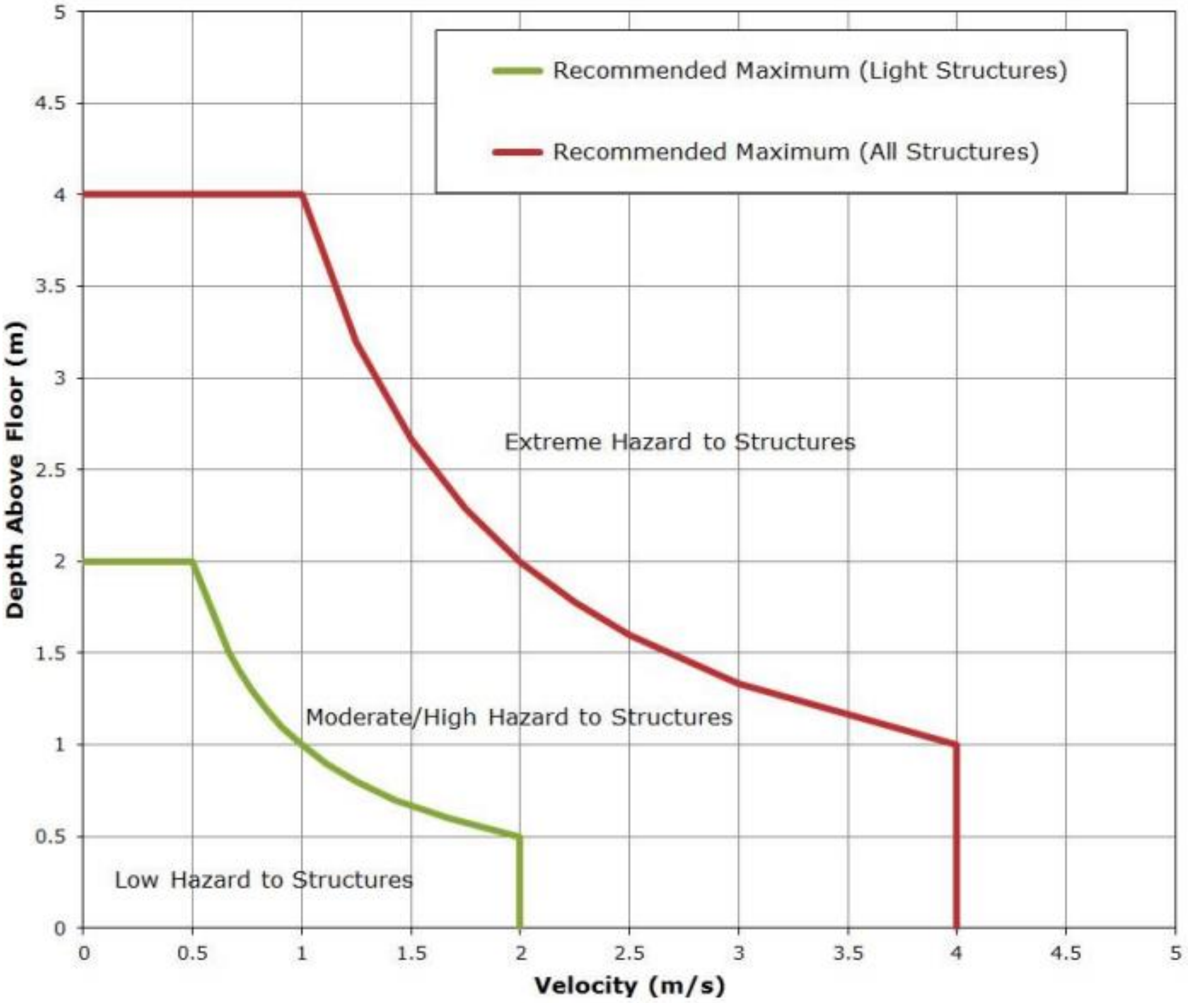

Figure 63. Proposed thresholds for building stability in floods (source: WRL, 2014, which was also reproduced in Ball et al. (2019).

Jansen et al. (2020) have been provided reference scale for indicative damage expected for a given flow velocity and depth. This study investigated the damage to buildings under flood flows, but it is anticipated the relationship to damage will be like wave overtopping flows. However, periodic impulsive flow from waves may be differences that are not well understood.

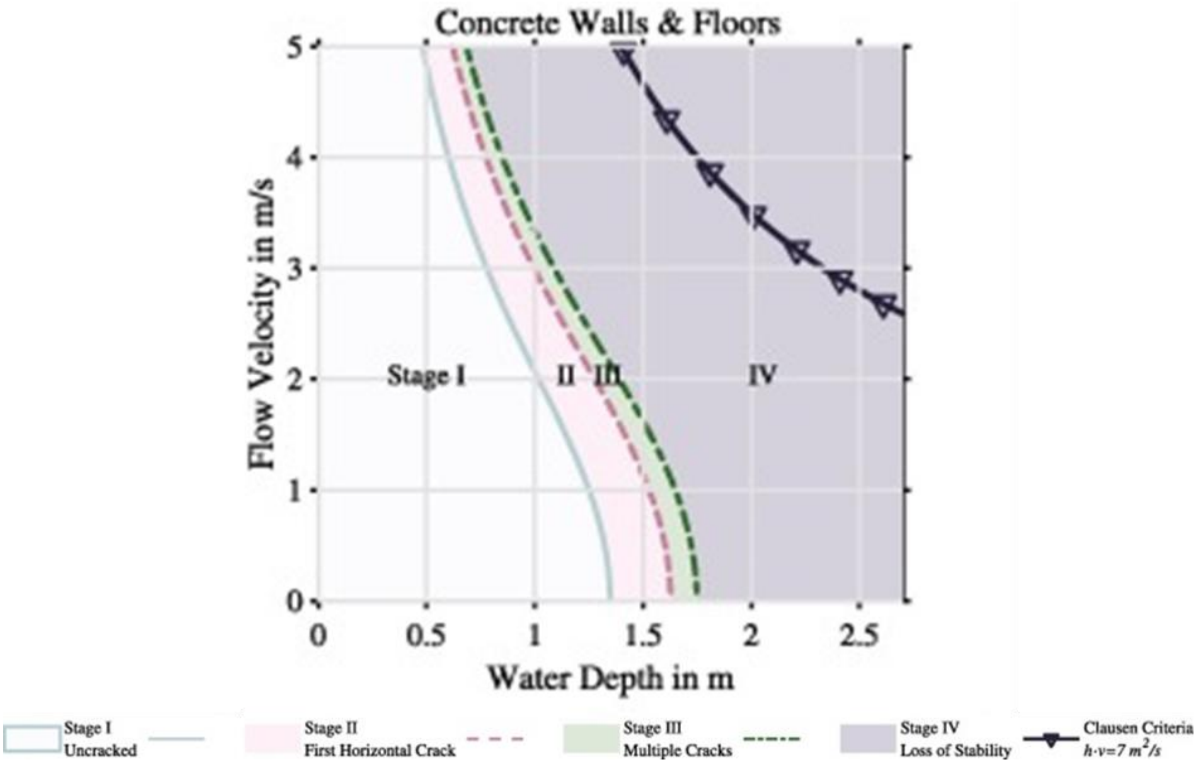

Figure 64. Damage onset relative to flood flow velocity and depths for a case study example with concrete walls. Source: Jansen et al. (2020).

Methods to calculate force

Analytical methods

Formal design guidance was not identified for assessing impact loads on buildings specific to wave overtopping flows. However, guidance is available tsunami bore propagation, and similar principles could be relevant to individual waves or surf-beat surges. Tsunami bore loading on buildings (and other vertical structures) is well documented in existing literature and design guidance (e.g., MBIE, 2020 and FEMA, 2019). These guidance documents break the impact force from tsunami bore propagation into three key forces:

3. *Hydrostatic force*

   $F_h = ½\gamma_s \, bh^2$ (MBIE, 2020) (1)

4. *Hydrodynamic (or drag) force*

   $\mathrm{F}_{dx} = 1/2\, \rho_s \, \mathrm{B(hu^2)} C_d \mathrm{C}_{cx} \times 1.25$ (MBIE, 2020) (2)

5. *Impulsive (or impact) force*

   $\mathrm{F}_w = 3/4\, \rho_s C_d \, \mathrm{B(hu^2)}_{bore} \times 1.25$ (3)

   (essentially 1.5 x hydrodynamic load) (MBIE, 2020).

Note: where γ_s= seawater specific weight density, b=width of vertical element (building wall), h = bore depth, ρ_s= seawater mass density, u= bore velocity, C_d=drag coefficient, and C_{cx}= proportion of closure coefficient.

Based on the above equations, the key forces for tsunami bore propagation are a function of the bore flow depth and velocity, and from this it is anticipated that the same will apply to wave overtopping flows.

Empirical methods

While no formal guidance was found on the determination of forces on buildings from wave overtopping flows, the estimation of force on vertical walls and buildings exposed to overtopping flood flows has been the topic of recent research (e.g., Streicher et al. 2018, De Rouck et al. 2012 & Chen et al. 2017). This research expands on the flood loads typically proposed for tsunami bores and provides the results of physical modelling experiments using wave overtopping flows impacting vertical storm walls that are set back from a coastal dike's crest.

As with the impact load from tsunami flows, the force impact produced by wave overtopping flows impacting vertical structures is double peaked (Streicher et al. 2018 & Chen et al. 2017) (Figure 65).

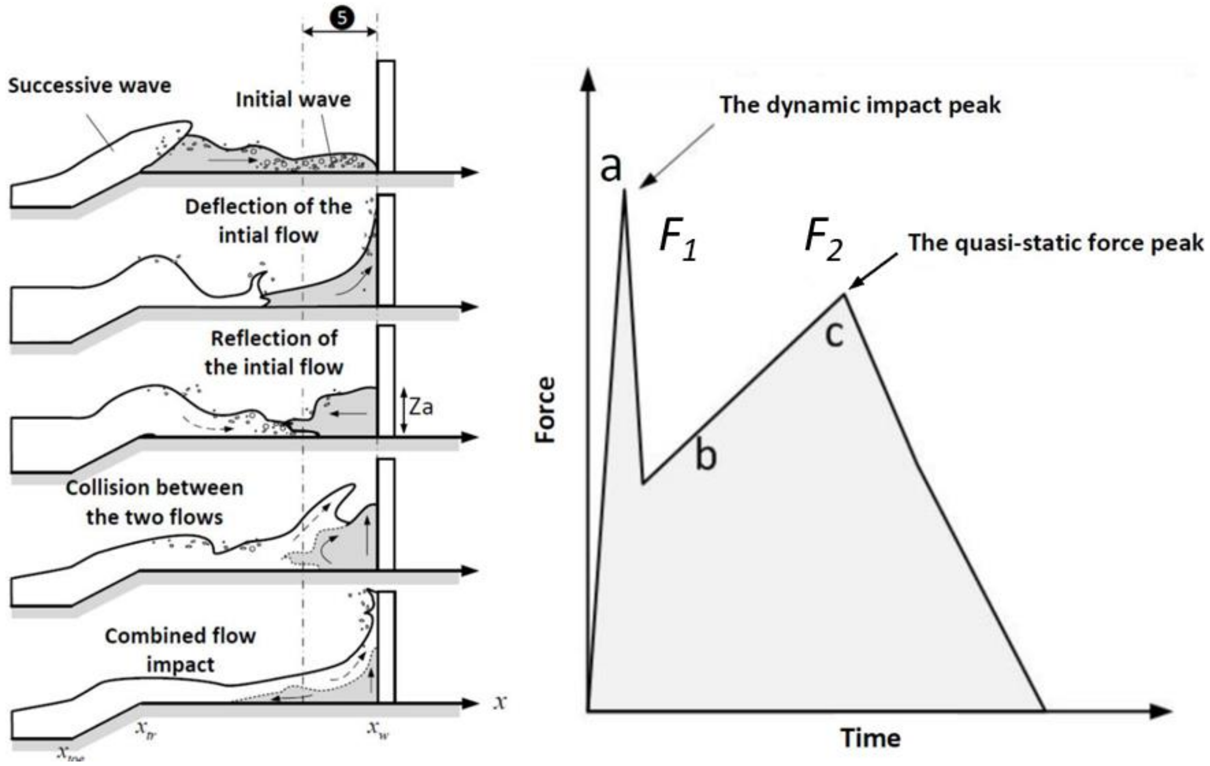

Figure 65. Schematic diagram of double peaked force impact from wave overtopping flow (adapted from: Chen et al. 2017).

A study conducted by Streicher et al. 2018 determined this first peak to be associated with the dynamic impact force of the overtopping bore on the vertical wall (F_1) and the second peak (sometimes referred to as the quasi-static peak) was associated with the force from maximum runup and down-rush of the vertical wall (F_2). Further to this, Streicher et al. 2018 concluded that the magnitude of the dynamic and quasi-static force peaks did not differ substantially. As a result, of this, it is considered reasonable to estimate the peak force by either the dynamic impact force or the quasi-static force.

A summary of empirical equations developed from wave overtopping studies for these two force peaks is given in Table 25. The dynamic peak is determined using the overtopping flow velocities and depths whereas the quasi-static or runup force is determined using the runup elevation on the vertical wall.

Table 25. Summary of existing empirical predictions for maximum impact load

Source	Force	Equation	R^2 error	MAPE error
Streicher et al. 2018	F_1	$F_1 = 2\rho_w h_{max} u_{max}^2$ (N/m) [Eq 1]	0.62	0.84
De Rouck et al. 2012	F_1	$F_1 = 1.09 u_{max} + 52.1 h_{max} - 9.5$ (N/m) [Eq 2]	0.64	0.93
Streicher et al. 2018	F_2	$F_2 = 0.32 \rho_w g R_h^2$ (N/m) [Eq 3]	0.29	1.0

Where ρ_w = water density, h_{max} = maximum overtopping depth, u_{max}= maximum overtopping velocity, and R_h= maximum runup against the wall.

Appendix C: Inundation maps

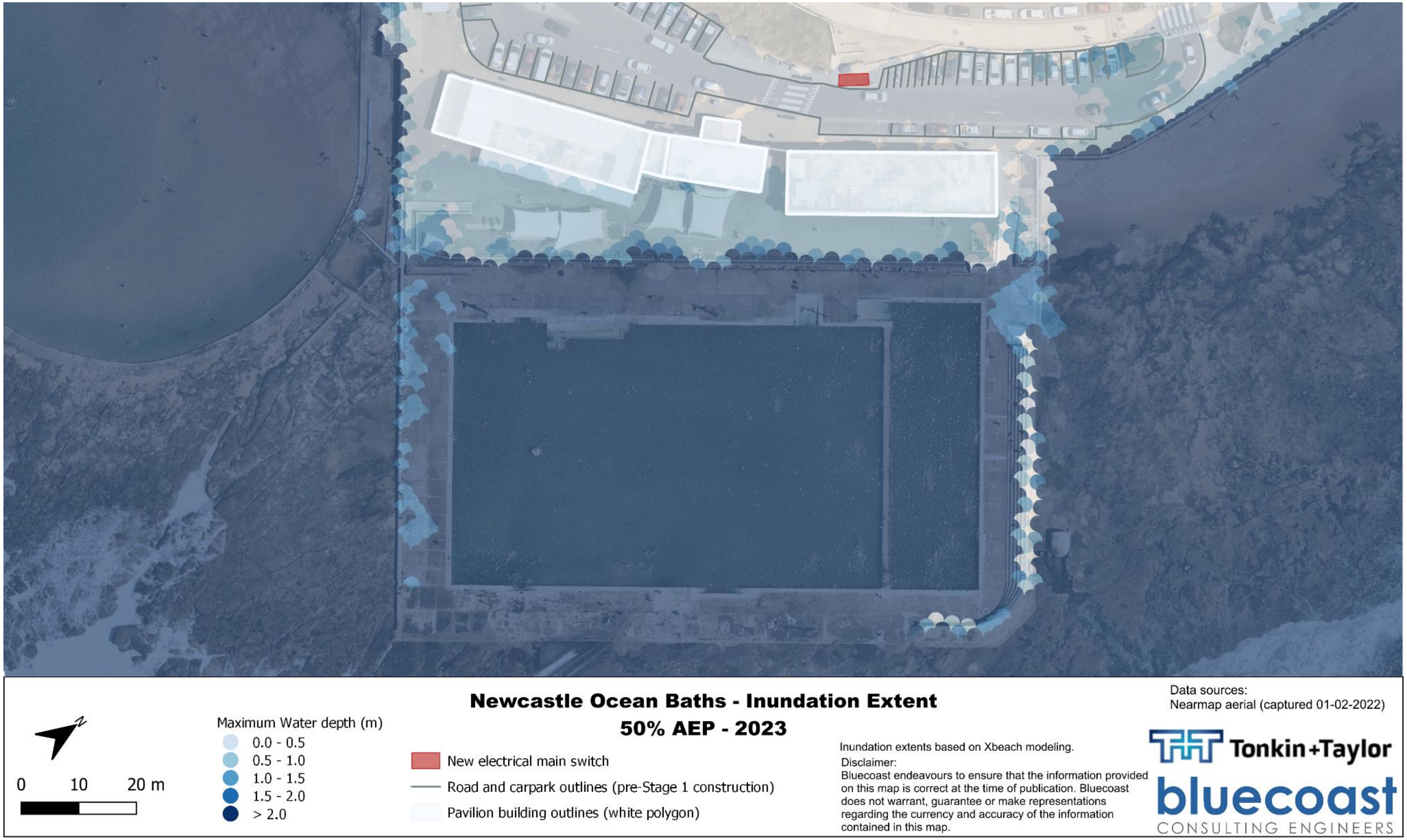

Figure 66: Inundation map of the 50% AEP 2023 condition.

Figure 67: Inundation map of the 1% AEP 2023 condition.

Figure 68: Inundation map of the 1% AEP 2120 condition.

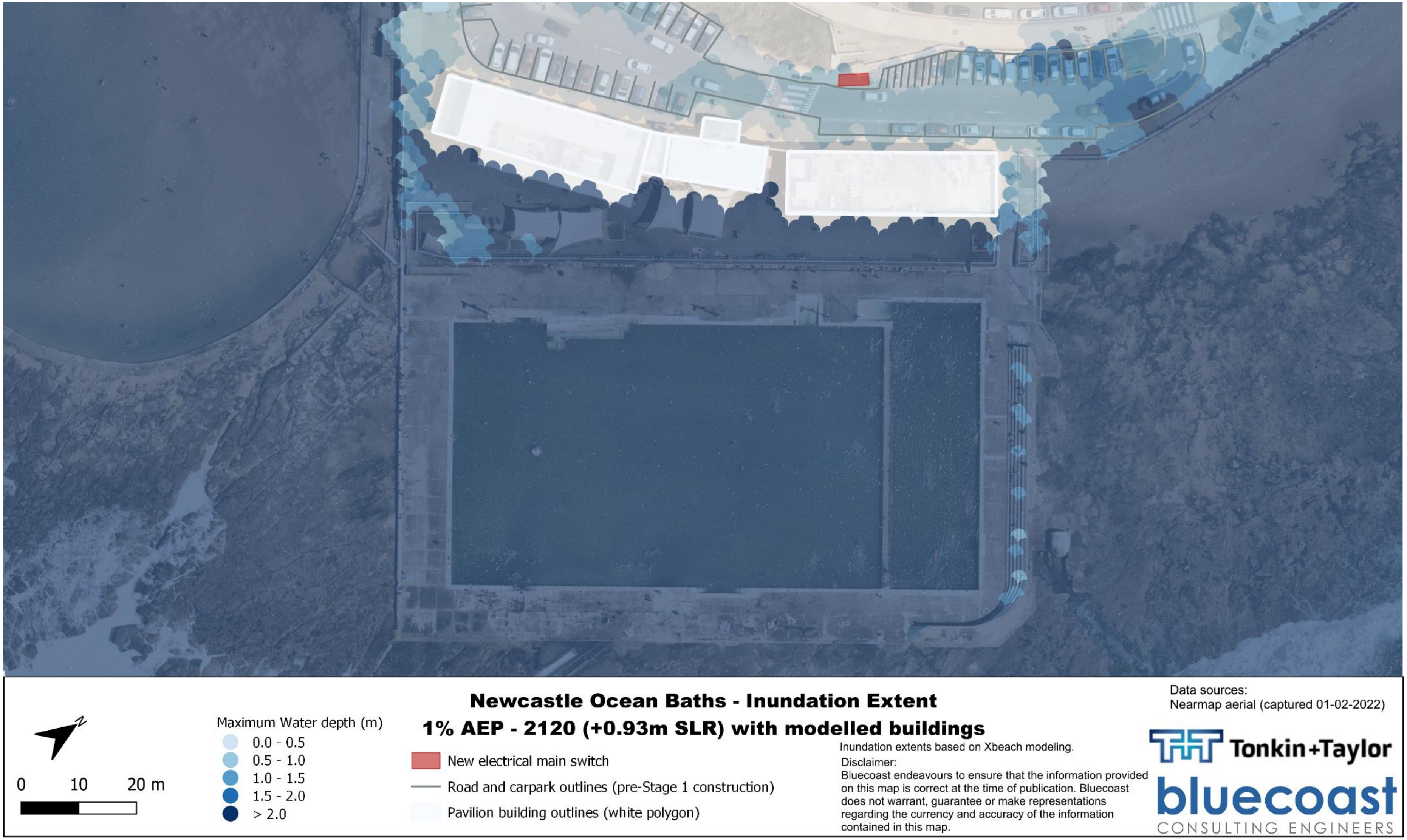

Figure 69: Inundation map of the 1% AEP 2120 condition with modelled buildings.

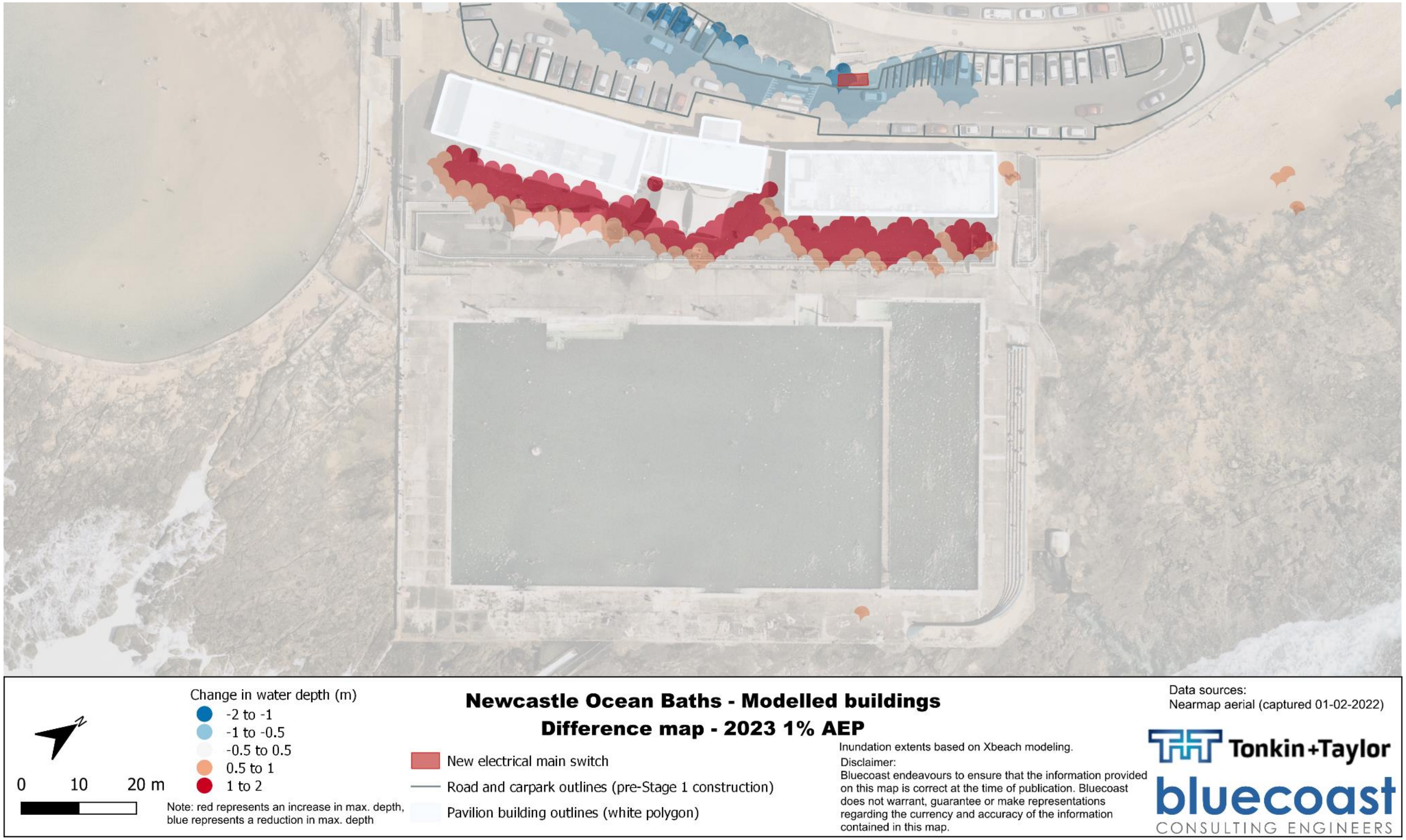

Figure 70: Difference map of 2023 1% AEP scenario demonstrating effect of modelled buildings.

Appendix D: Loads on Stage 1 lifeguard tower

Wave loads on the glass of the lifeguard hut were calculated based on results from Xbeach model for design (or extreme) scenarios. The output point used (WG42) is on the pool deck as this is considered most representative of flow conditions impacting the tower, see Figure 71. The wave loads were calculated using the same methodology as in Section 6.7, applied only to the seaward facing window area then divided by window width to give pressure. The results for several input scenarios and SLR cases are given in Table 26

These results should be used with caution noting that the method used doesn't resolve high, short-duration impact loads if we get waves breaking directly on the structure. While this is less of a concern here based on how the wave bores reach this area (see Section 6.7), there is a chance a wave could reform across the pool and slam into windows causing instantaneous loads which will be higher than the loads calculated here. Significant FOS should therefore be considered by the designer. To resolve short-duration loads would require physical model testing (or detailed CFD) which was beyond the scope of this assessment.

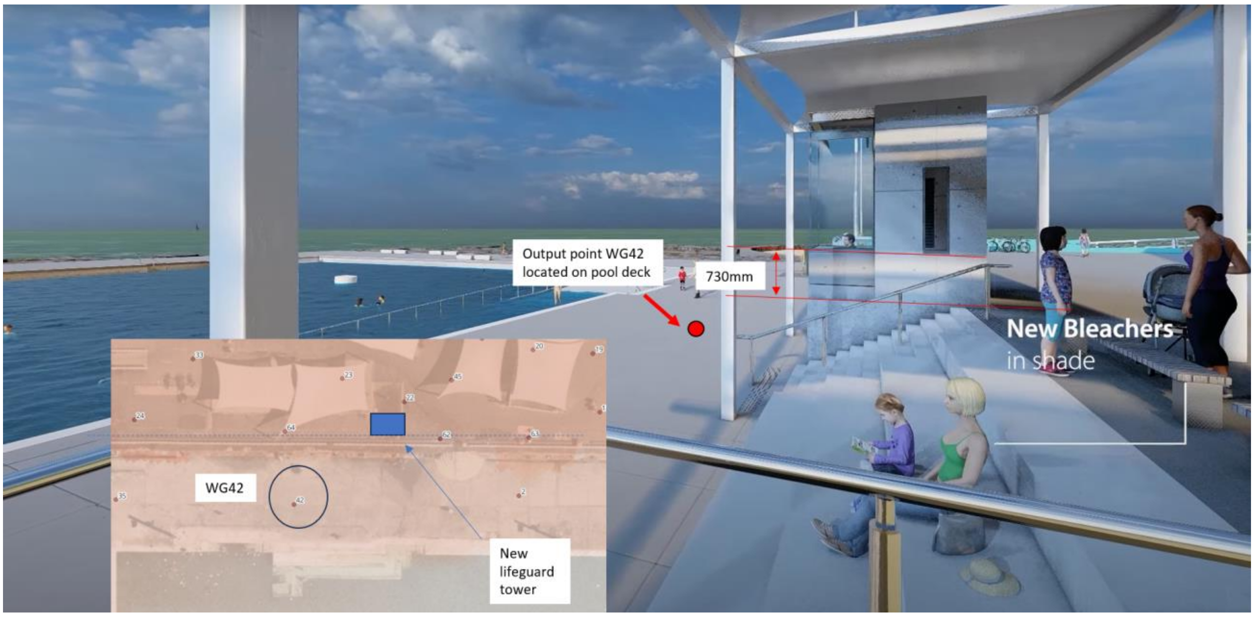

Figure 71: Location of new lifeguard hut (source: Terras) and location of Xbeach output point used for load calculation.

Table 26: Loads on lifeguard hut seaward window.

Scenario	Pressure on window (kPa)		
	2023 (+0m SLR)	2050 (+0.23m SLR)	2120 (+0.93m SLR)
50% AEP	0.0	0.0	9.6
10% AEP	1.6	2.2	24.0
1% AEP	8.3	**9.8**	26.4