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REPORT

Bronte SLSC Redevelopment Seawall and Related Elements Detailed Design

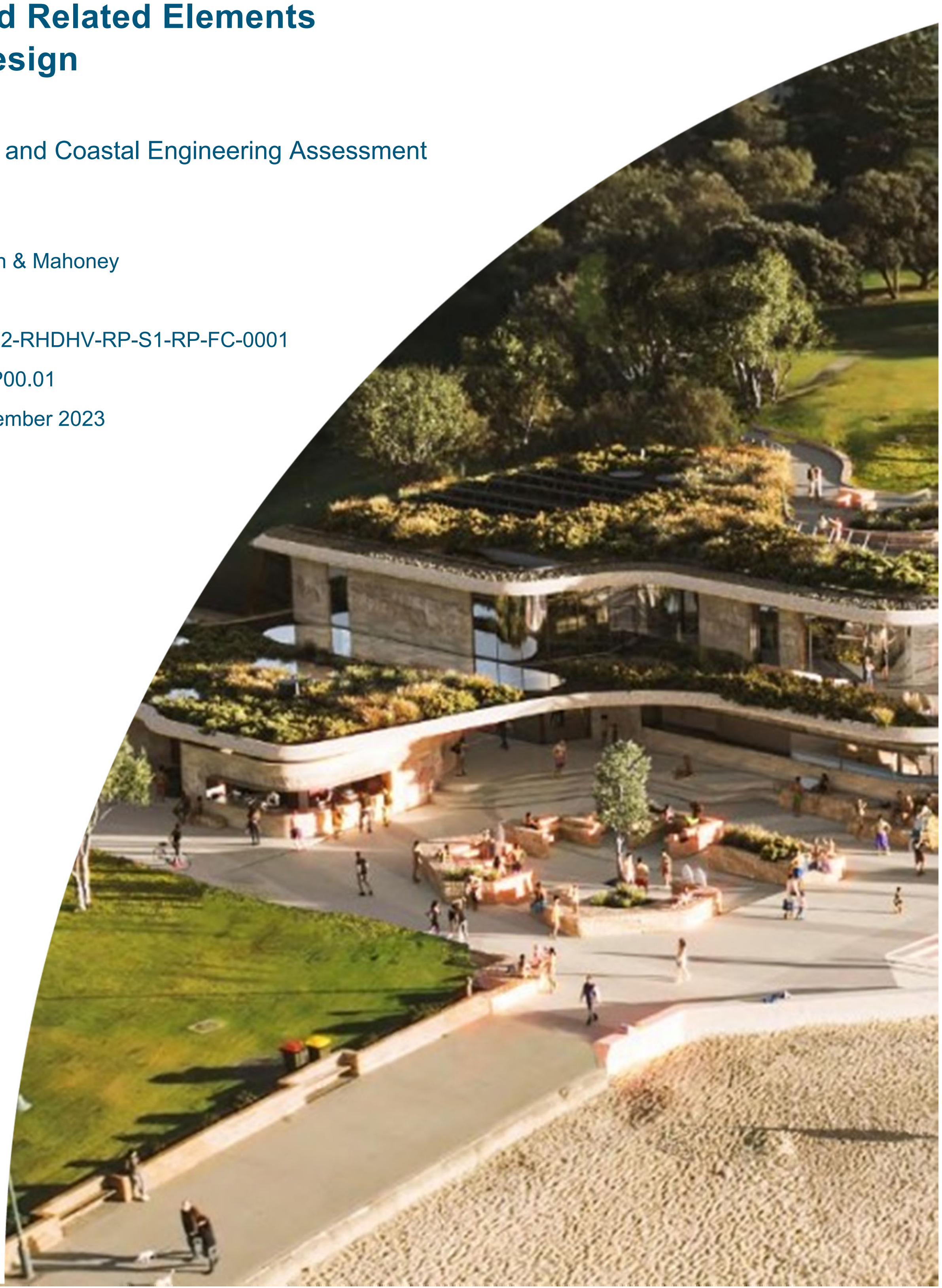
Concept Design and Coastal Engineering Assessment Report

Client: Warren & Mahoney

Reference: PA3572-RHDHV-RP-S1-RP-FC-0001

Status: Draft/P00.01

Date: 5 December 2023



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1 Introduction

1.1 Background

Architects Warren and Mahoney (W&M) have recently resubmitted a revised Development Application (DA) for the redevelopment of the Bronte Surf Lifesaving Club (SLSC), refer to **Figure 1-1** and **Figure 1-2**. The design of the seawall and its related elements were originally to be run internally within Waverly Council (hereafter referred to as the Council), but it was decided that it would be preferable to extract the documentation into W&M's scope so that it could be run concurrently with the SLSC upgrade.

RHDHV developed a return brief, inclusive of subconsultant inputs, to prepare design and tender documentation for the Bronte SLSC Redevelopment Seawall and Related Elements Detailed Design. Given the technical specialisation of the seawall component, at the time of writing W&M are planning to run the seawall design and its related elements as a separate package.

Figure 1-1 Photo montage for revised DA showing seawall and related elements. Subject to design development

Figure 1-2 Architectural visualisations for general arrangement for revised DA showing seawall and related elements. Subject to design development

RHDHV understands that the proposed redevelopment would involve the construction of coastal protection works to protect the SLSC over its design life. There is no certified Coastal Zone Management Plan (CZMP) or certified Coastal Management Program (CMP) in place for Bronte Beach. The Council has completed a Stage 1 scoping study for a CMP (in collaboration with neighbouring councils) and is progressing towards completion of a CMP. Since a CMP is not yet in place, the proposed redevelopment would be a Part 4 matter under the Environmental Planning & Assessment Act 1979, and the consent authority would be the Sydney City Eastern Planning Panel (SCEPP).

At the time of preparing the return brief, it was expected that physical model testing may be a condition of consent, and the approach for the inclusion of physical modelling needed to be addressed. Currently, it is understood that the approved DA, detailed design, construction certificate (CC) and substantial commencement of the project are required by March 2024.

1.2 Study area

Bronte Beach, situated approximately 7km south-east of Sydney's CBD, is characterised by a historical seawall spanning about 250m, owned, and managed by Waverley Council (refer to **Figure 1-3**). The seawall serves the dual purpose of retaining the beach promenade and safeguarding foreshore buildings from inundation (refer to **Figure 1-4**).

Figure 1-3 Aerial photo of the project site (source: Nearmap dated 03 October 2023)

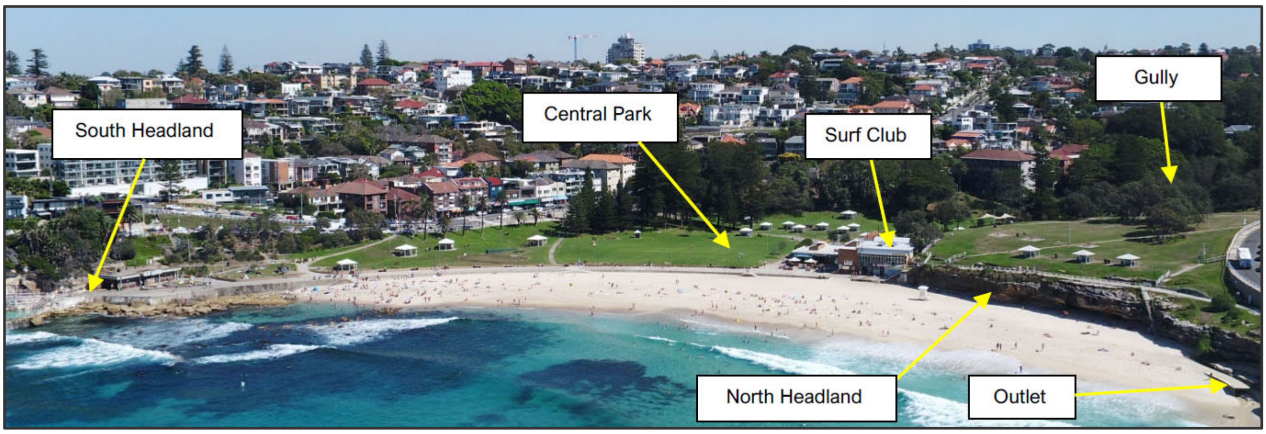


Figure 1-4 Site aerial elevation (AssetGeoEnviro, 2022)

The study area, located in the central portion of Bronte Beach, includes distinct features such as Bronte Gully to the west, a central park immediately west of the beach, the north and south headlands, and a beachfront section with concrete paving and community facilities. Bronte Gully is marked by a narrow-grassed valley floor, steeply vegetated side slopes, and a managed creek system. The central park, hosting site developments like the SLSC and community amenities, contains buried services, including a significant stormwater culvert. The north and south headlands are elevated regions with exposed sandstone cliff lines along the coast.

The ground surface levels in the central park are relatively low and flat, ranging from approximately +4m to +5m AHD. The beachfront section features concrete paving, a cafe, and pergolas, while the remaining project site includes grass, mature trees, pedestrian pathways, and additional community facilities like pergolas and barbecues.

Concerns arise during extreme storm events, both currently and in the future, with projected sea level rise and increased storm frequency. The Bronte Beach seawall is at potential risk of excessive scour due to wave action, potentially undermining the toe and leading to wall toppling. The wall is also relatively low and is therefore exposed to wave overtopping. Addressing these risks is crucial for the resilience of coastal infrastructure.

1.3 Scope of work

RHDHV would assist to develop and confirm the concept design and develop a detailed design for the seawall upgrade fronting the SLSC. Further geotechnical investigation is proposed to “fill the gaps” and confirm geotechnical parameters for foundation design. While desktop assessment would permit a review of wave runup and overtopping, physical modelling was included as a provisional item to optimise the overtopping design and wave loading for structural design.

The scope of work was aimed to undertake investigations leading to preparation of a Coastal Report. Our input would then continue to 100% Final detailed design, inclusive of technical specifications, for the seawall and related elements. The designs would be ready for inclusion in Tender Documents prepared by others. At the time of preparing our reverse brief, the assumed spatial extent for the seawall and related elements for detailed design by RHDHV was as shown in **Figure 1-5**.

Our scope of work comprised the following tasks:

Stage 1 – Review and consolidation of masterplan

(i) Collation and review of background information
(ii) Masterplan update
(iii) Approval process support
(iv) Meetings and project management

Stage 2 – Coastal engineering assessment and concept design

(v) Gap analysis
(vi) Coastal and maritime engineering site inspection
(vii) Additional survey (provisional item)
(viii) Assess coastal erosion and wave runup and overtopping hazard
(ix) Additional geotechnical investigation (provisional item)
(x) Confirm wave overtopping mitigation concept design
(xi) Confirm seawall arrangement and develop a conceptual structural design
(xii) Coastal Report: Investigations, concept design and coastal assessment
(xiii) Peer review liaison
(xiv) Coordination workshops with different disciplines

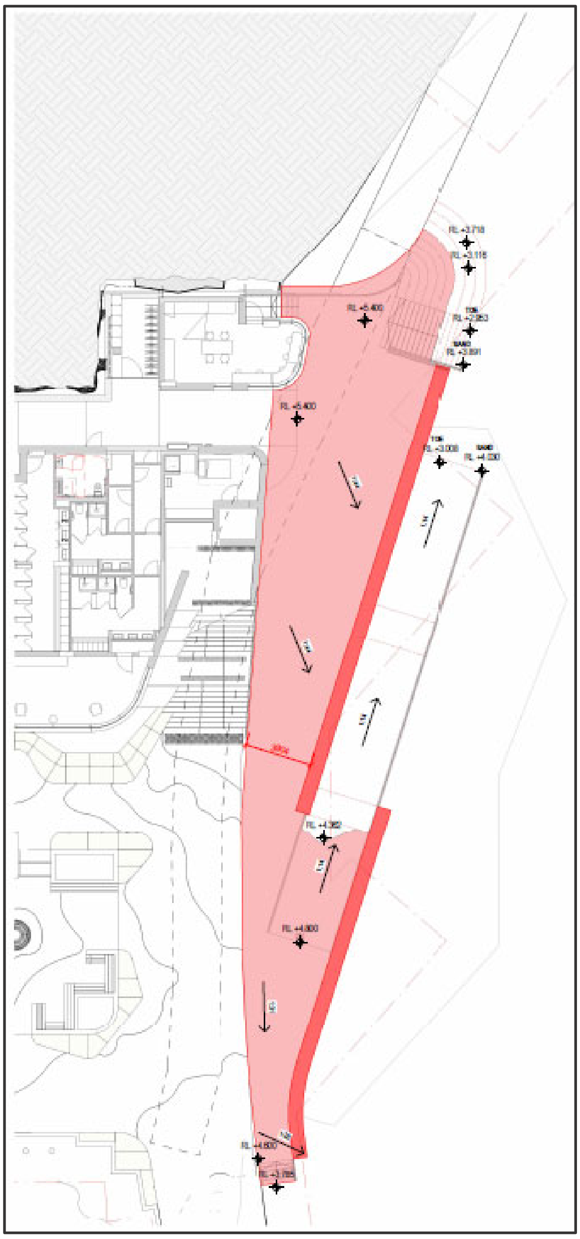


Figure 1-5 Extent of seawall and related element works for detailed design by RHDHV. Subject to design development

(xv) Meetings and project management

Stage 3 – 50 and 100% DD

(xvi) Physical modelling (provisional item)
(xvii) Basis of Design (BOD) and detailed design development
(xviii) Detailed design and drawings
(xix) Technical specification and Method Statement Schedule
(xx) Quantities and Schedule of Rates and Lump Sum Items
(xxi) Detailed design report
(xxii) Meetings and project management

The extent and scope of site investigations and subsequent coastal engineering reporting align with the anticipated level of detail required for the DA. As we move into Stage 3 and progress into the detailed design phase, additional reports will be generated to provide further insights and specifics in accordance with the evolving project requirements.

1.4 Liaison and project management

Early in the investigation, several meetings were held with W&M and its subconsultant design team to provide clarity on the project scope, tasks, timeline, communication channels, and to identify any potential challenges or constraints.

RHDHV also participated in meetings involving the SCEPP and Council to support W&M in working through the approval process for the seawall.

1.5 Abbreviations

Abbreviation	Full description
AHD	Australian Height Datum
BGL	Below ground level
BH	Borehole
HA	Hand auger
IPCC	Intergovernmental Panel on Climate Change
SLSC	Surf Life Saving Club
SECPP	Sydney Eastern City Planning Panel
TP	Test pit
W&M	Warren & Mahoney

2 Review and consolidation of masterplan

2.1 Appreciation

The seawall and promenade were built as part of a Bronte Beach rehabilitation plan between 1914 and 1917 (WorleyParsons, 2011) and (BMT, 2020) . By draining, filling in, and establishing grass in the space behind the promenade, which is now a part of Bronte Park, this effectively divided the beach in half (refer to **Figure 2-1**). The storm water drain that emerges at the northern end of the beach now receives the creek that flows through Bronte Gully. Bronte Beach draws sunbathers, swimmers, and surfers thanks to its big park, picnic area, easy access, and ample parking. The beach is affected by rips, and sand from Bronte Beach occasionally blows onto the promenade, like at Bondi Beach (WorleyParsons, 2011).

In the Eastern Beaches CMP Stage 1 Scoping Study (BMT, 2020), BMT states that Bronte seawall has generally withstood overtopping and severe storm damage, except for repairs needed in 2016 for fencing, railing, and the seawall. An initial condition report conducted afterwards indicated potential defects in the current seawall and associated structures that could propagate and eventually jeopardize the wall's functionality if left unaddressed. The report also highlighted vulnerability to foundation failure due to wave-induced scour, particularly during a 100-year ARI design storm event in the present day. Moreover, the risk of failure increases when considering future sea level rise scenarios in 2050 and 2100.

According to the WorleyParsons report (WorleyParsons, 2011), the crest level of the Bronte seawall varies from around 3.9m AHD in the south to 4.8m AHD in the centre of the beach in front of the amenity block. These levels are considerably below potential runup levels, and overtopping would be anticipated during extreme events. As per this report, this aligns with comments recorded in news reports during previous storm events in 1948 and 1959. However, there is no mention of damage to the seawall, and it is not known what (if any) repair works have been undertaken to the seawall since its construction. Overtopping was also documented in photographs taken during the 1974 storms (refer to **Figure 2-2**).

There was relatively little information on the existing seawall fronting the SLSC. As it is over 100 years old, Horton Coastal Engineering concluded that the structure was well beyond its design life (Horton Coastal Engineering , 2023) and that repairs as proposed in Seawall Technical Study by ARUP in 2016 were “band-aid” solutions.

ARUP undertook a beach wide technical study in 2016 to better understand the structural condition and the stability of the seawall against current and future coastal processes. As part of this study ARUP excavated several shallow test pits on the beach, including near the SLSC. AGE Geotechnical Engineers then followed some years later with two boreholes drilled between the seawall and the SLSC buildings, which indicated bedrock levels between -0.15m and 0.2m AHD (AGE, 2020). As these boreholes are relatively close to the seawall in question, it is likely that bedrock levels affecting the design would be similar. Subsequent AGE boreholes, north of the club, drilled at a much higher level in the reserve, add little information for the SLSC seawall design. Limited core drilling along the seawall assessed concrete strength, and sulphate and chloride ion concentrations.

Figure 2-1 Top: Bronte Beach June 1935 (Source: State Library of NSW) // Bottom: Bronte Beach 1959 (Source: Waverley Library Fact Sheets)

Figure 2-2 Overtopping at Bronte, 1974 (source: Waverley Council)

ARUP engaged Baird Australia to undertake an assessment of coastal processes to inform a technical study of the Bronte Beach seawall. SBEACH modelling undertaken by Baird Australia for ARUP established design scour levels reducing from 2.9m AHD today, to 0.35m AHD at 2100. While in RHDHV's experience these scour levels appear to be somewhat elevated for an open coast beach seawall, we note that bedrock is also likely to be elevated. The seawall structure has been there for over 100 years.

The seawall in the northern portion of the beach is shown by ARUP to be a mass concrete structure ranging between approximately 0.4m thick at the crest to more than 0.8m thick at the base (ARUP, 2016). ARUP reports that the wall is supported on brick columns and founded in sand at between 1.6 and 1.8m AHD (refer to **Figure 2-3**). However, this is quite different from that reported by WorleyParsons (WorleyParsons, 2011) (refer to **Figure 2-4**). No other information on the construction and geometry of the wall is known. As described in **Section 4.4**, additional geotechnical investigations including test pits, boreholes and seismic profiling would be undertaken to assess the footing details and foundation materials below the existing seawall.

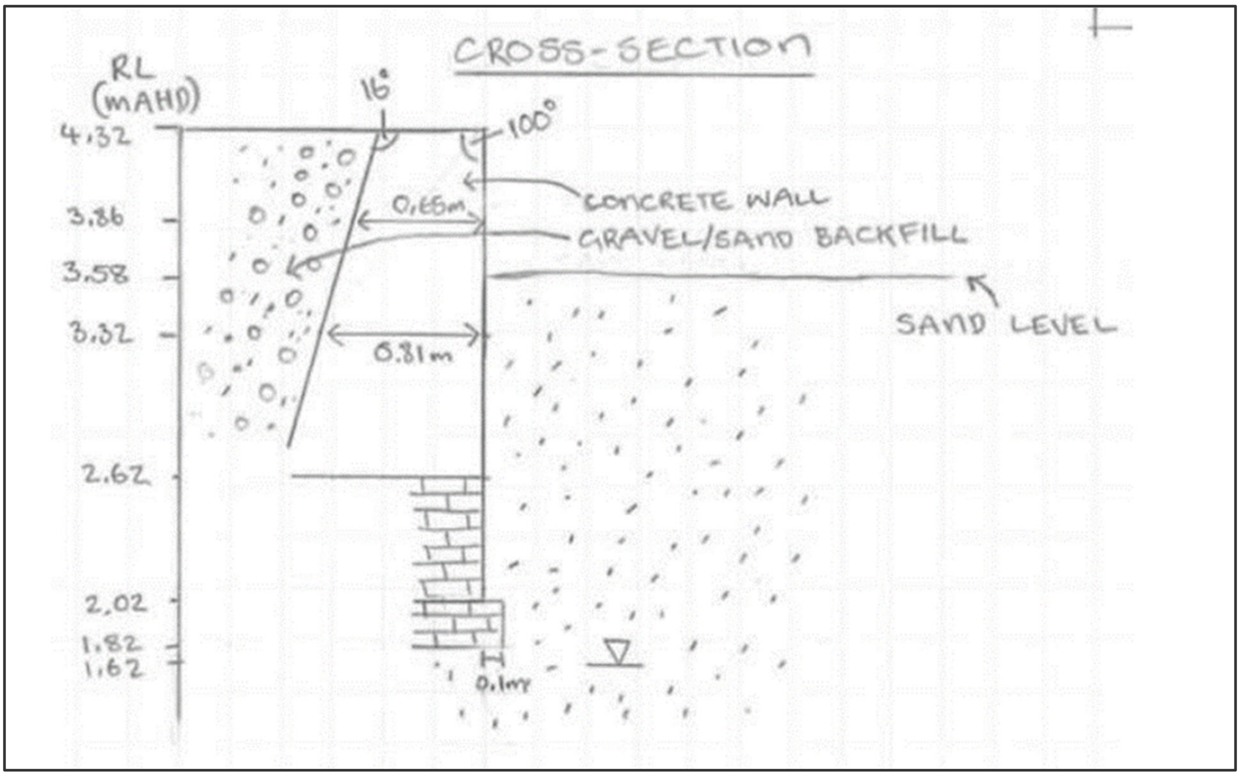


Figure 2-3 Sketch of interpreted representative seawall cross section for the northern-most 100 m of the Bronte Seawall which includes that fronting the SLSC (source: (ARUP, 2016))

ARUP have run stability assessments for the wall, including gross stability, overturning, sliding, and bearing. While acceptable gross stability is reported for all test cases, this is found to be unacceptable for extreme coastal loads, and strengthening is required. ARUP present several options to strengthen the existing seawall including rock or grout bag mattress, cut-off sheetpile, underpinning, modified geometry (including widening landwards), and replenishment of backfill. Bullnose and/ or parapet at the top of the wall to reduce overtopping is recommended. The mattress was ARUP's preferred option. RHDHV note that the seawall at Bondi Beach which is of similar vintage and design to the Bronte seawall, was protected by a reno-mattress apron 36 years ago, probably for similar reasons (refer to **Figure 2-5**).

The promenade and club, as they are currently situated, are exposed to wave runup, and overtopping in storms. Impacts from the June 2016 event (refer to **Figure 2-6**), nominally regarded as about a 30-year ARI event for east facing shorelines in and around Sydney, aptly demonstrate this exposure which could be expected to worsen significantly under sea level rise over the life of the development.

An earlier architectural concept proposed by W&M involved several new spur walls angled onto the beach, extending out from the seawall, and separating/ protecting ramps and steps. The existing SLSC promenade and seawall, and all seawall upgrade works including the new seawall, ramps, and steps, would all be located on Crown Land. The potential impact of the spur walls on the beach is an important consideration for Council, addressed through the concept design process, coastal assessment, and interaction with the Peer Review.

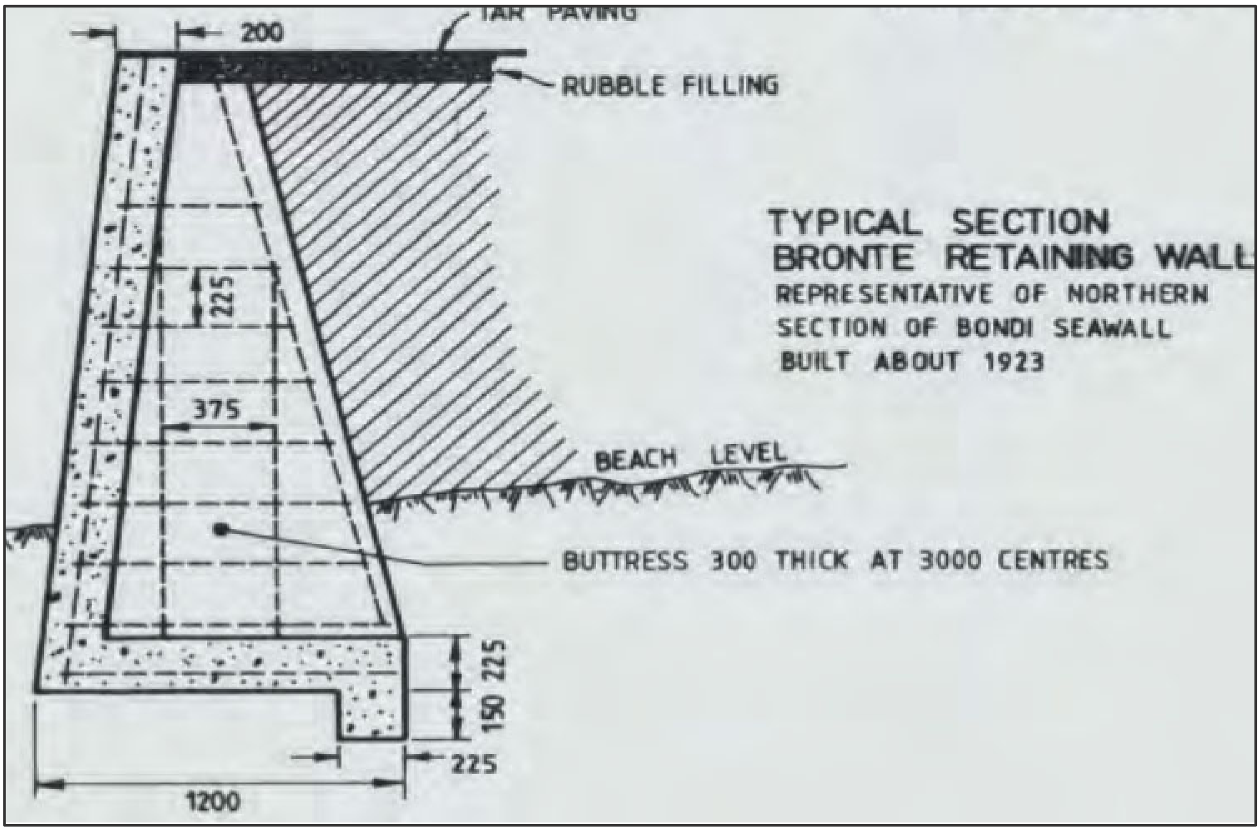


Figure 2-4 Reproduction of Historical Design Drawings of Bronte Seawalls (Source: (WorleyParsons, 2011))

A major stormwater culvert passes through the seawall in the immediate vicinity of the SLSC. The stormwater outlet discharges against the headland at the northern end of beach and has no influence on coastal hydraulic and scour risk for the project. Accommodating the structure of the culvert in the seawall design, and the potential impact of the seawall on scour around the culvert interface with the seawall, are separate matters which require careful consideration.

2.2 Coastal management

Coastal protection works are defined in the Coastal Management Act 2016 at Section 4(1) to be beach nourishment activities or works and activities to reduce the impacts of coastal hazards on land adjacent to tidal waters, including, but not limited to, seawalls, revetments, and groynes. The relevant part of the State Environmental Planning Policy (Resilience and Hazards) 2021 is Part 2.2 Development controls for coastal management areas. Both the Coastal Management Act 2016 and SEPP 2021 must be considered for the seawall upgrade. Since a certified Coastal Management Program is not currently in place covering Bronte Beach, the DA must be determined by the SECPP.

The coastal engineering assessment undertaken for the project addresses these matters.

Figure 2-5 Construction of Bondi seawall reno-mattress toe protection 1987 ((WorleyParsons, 2011)*)*

Figure 2-6 Damage to roller doors at Bronte SLSC in June 2016

2.3 Masterplan update

W&M developed the proposed concepts for the Bronte SLSC redevelopment in close collaboration with the Bronte SLSC, Waverley Council, and the local community, as depicted in **Figure 2-7** to **Figure 2-10**. For architectural details, the reader is directed to the full architectural drawings separately packaged by W&M. The design process involved consultations with the Approvals Authority, the SECPP, and the Design Excellence Advisory Panel.

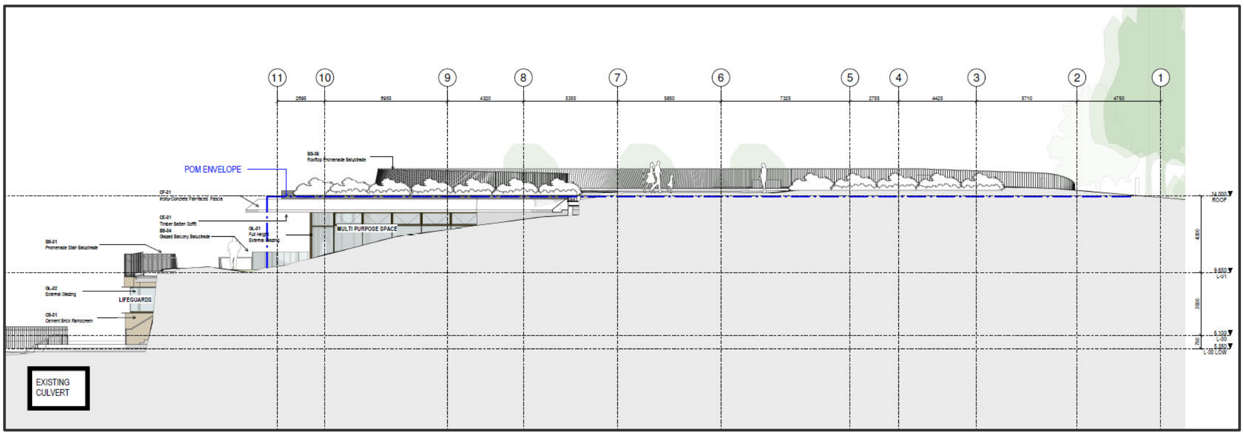


Figure 2-7 Proposed overall north elevation

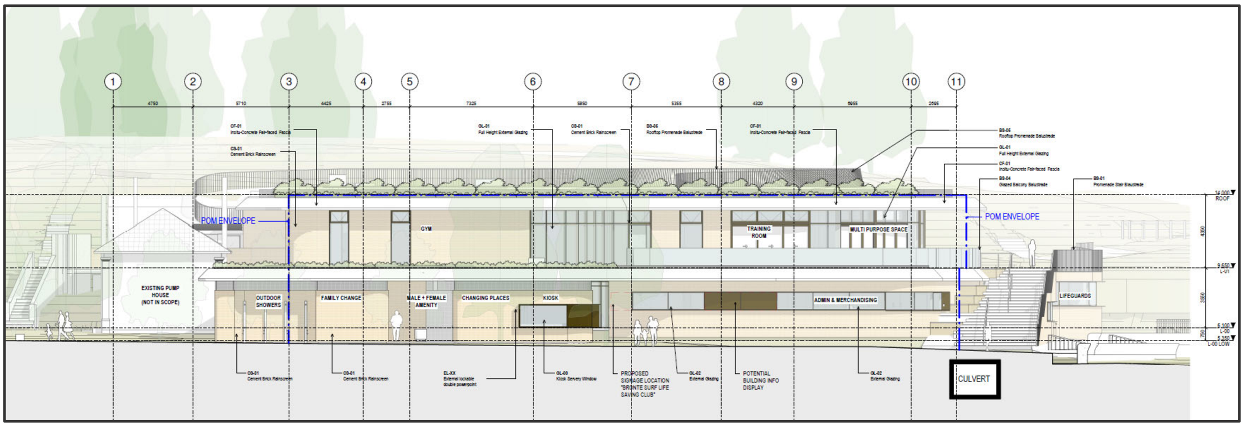


Figure 2-8 Proposed overall south elevation

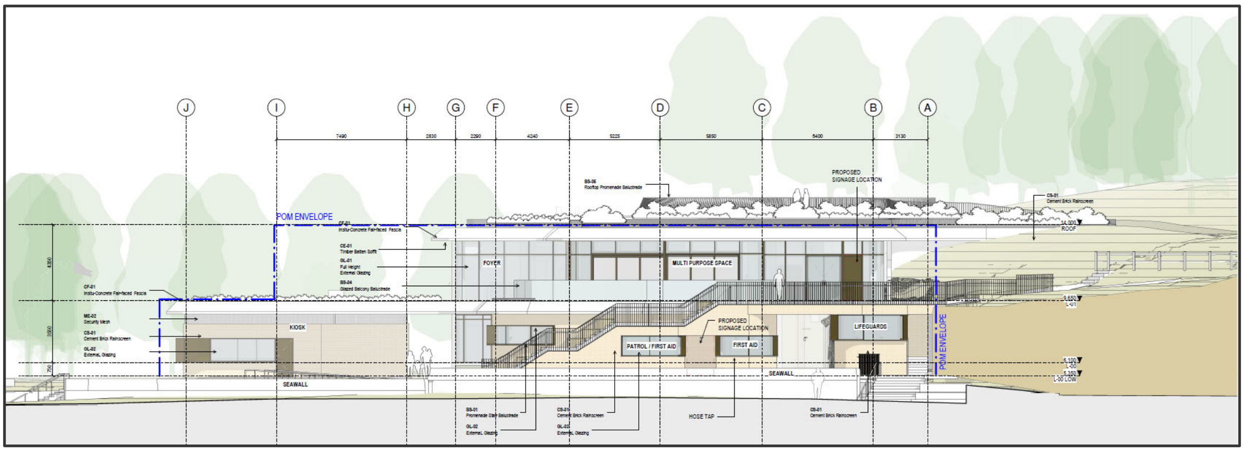


Figure 2-9 Proposed overall east elevation

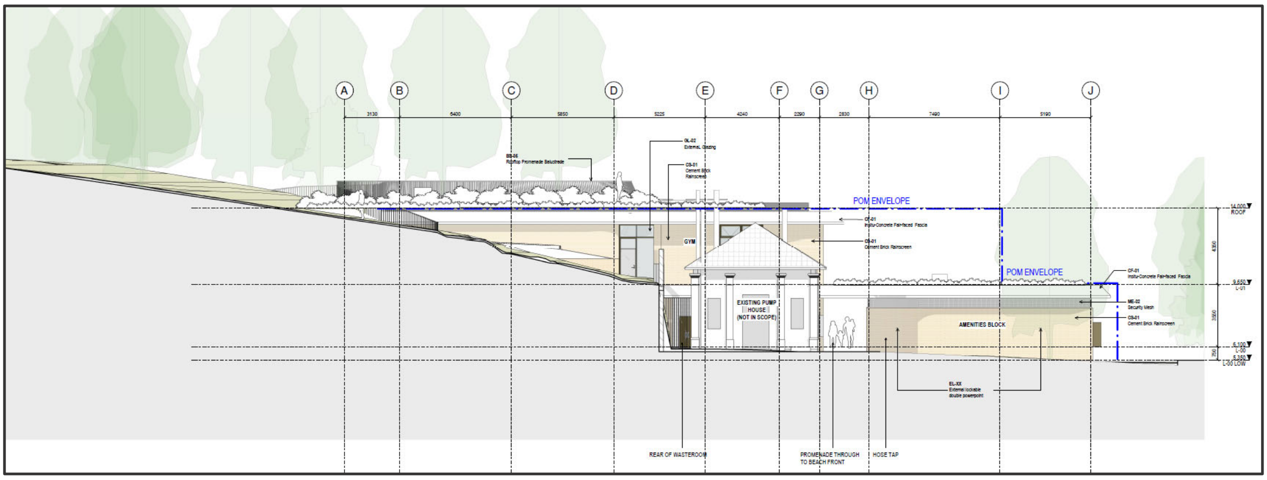


Figure 2-10 Proposed overall west elevation

The proposed plan involves the demolition of the current SLSC building and the construction of a new clubhouse on a comparable footprint. Simultaneously, modifications to the existing seawall, located seaward of the clubhouse, are planned to offer increased protection against erosion, recession, and oceanic inundation (particularly wave runup and overtopping). The objective is to fortify the clubhouse while also providing additional promenade space. This additional space aims to improve public circulation around the clubhouse and enhance accessibility, including provisions for disabled access to the beach.

To support W&M in this endeavour, RHDHV provided expert advice to minimise upfront risks and maximise potential solutions. RHDHV guidance included:

- Planning approval pathway for the SLSC redevelopment.
- Consideration of toe levels for beach access ramps, accounting for beach level fluctuations, and access needs post-erosion events.
- Optimisation of the number, orientation, and grade of ramps for general public access.
- Development of a structural concept for new structures over the beach, independent of the existing seawall.
- Evaluation of the potential impacts of proposed beach structures on the existing seawall and box culvert, addressing issues such as scour and undermining.
- Discussion on protecting vulnerable sections of the SLSC building and minimising encroachment onto the beach.
- Participate in meetings involving W&M, Council, the SCEPP, peer reviewer, and external planners to discuss the revised design concept for works on the beach.

Concerning the masterplan, refer to submitted W&M drawings for architectural information.

2.4 Information provided

RHDHV was provided the information provided in **Table 2-1**.

Table 2-1 Data provided

Item	Document Title	Document Reference	Document Date	Comments
1	Bronte Seawall Technical Study	249632-REP-01	27/09/2016	Technical study to better understand the seawall's structural condition and its stability against current and future coastal processes.
2	Proposed Bronte Surf Life Saving Club Facilities Upgrade Bronte Beach, Bronte NSW Geotechnical Investigation	5613-3-G1	26/03/2020	
3	Proposed Bronte Surf Life Saving Club Facilities Upgrade Bronte Beach, Bronte NSW Additional Geotechnical Investigation	5613-3-G1	19/01/2022	The objective of an additional investigation is to provide information on the surface and subsurface conditions to provide preliminary geotechnical recommendations in foundation requirements and excavation support.
4	Coastal Risk Assessment and Coastal Engineering Advice on Bronte Surf Lifesaving Club and Community Facility Redevelopment	rpJ0573-Bronte SLSC amended DA-v2	31/07/2023	Report prepared by Horton Coastal Engineering Pty Ltd for Warren and Mahoney
5	Overall General Arrangement - GROUND FLOOR PLAN	A10.001	01/12/2023	
6	Ground Floor Plan SLSC ZONE A - GENERAL ARRANGEMENT GROUND FLOOR PLAN	A10.010	01/12/2023	
7	Ground Floor Plan SLSC ZONE B - GENERAL ARRANGEMENT GROUND FLOOR PLAN	A10.011	01/12/2023	
8	Beach Access – GENERAL ARRANGEMENT PLAN	A10.014	01/12/2023	
9	Beach Access – GENERAL ARRANGEMENT PLAN	A10.015	01/12/2023	
10	SLSC ZONE A - GROUND FLOOR WALL SETOUT PLAN	A11.001	01/12/2023	
11	SLSC ZONE A - GROUND FLOOR WALL SETOUT PLAN	A11.002	01/12/2023	
12	Proposed Overall - NORTH ELEVATION	A20.001	01/12/2023	
13	Proposed Overall - SOUTH ELEVATION	A20.002	01/12/2023	
14	Proposed Overall - EAST ELEVATION	A20.003	01/12/2023	
15	Proposed Overall - WEST ELEVATION	A20.004	01/12/2023	
16	Overall Section 1	A30.001	01/12/2023	
17	Overall Section 2	A30.002	01/12/2023	
18	Overall Section 3	A30.003	01/12/2023	
19	Overall Section 4	A30.004	01/12/2023	
20	Overall Section A	A30.005	01/12/2023	
21	Overall Section B	A30.006	01/12/2023	
22	Overall Section C	A30.007	01/12/2023	
23	Overall Section D	A30.008	01/12/2023	

2.5 Literature review

A substantial body of literature in the form of consultant and council technical and management reports exists for the Bronte Beach project. All available literature addressing coastal processes, coastal protection works and coastal management within the Bronte foreshore was considered, with key investigations listed in the following discourse.

2.5.1 Coastal Risks and Hazards Vulnerability Study (2011)

Waverley Council has assessed coastal hazards and climate change vulnerabilities for its beaches and cliffs, determining generally low risks to coastal assets. While ongoing monitoring and periodic geotechnical assessments are appropriate for near-term risk management, anticipated sea level rises pose concerns, including beach width reduction and increased seawall instability. Recommendations for hazard management include incorporating hazard information into planning instruments, notifying affected lots, considering beach nourishment, installing warning signs for cliff face instability, advising property owners, and conducting regular monitoring and maintenance. Additionally, specific actions are proposed for Ben Buckler, Bondi, Bronte, and Tamarama seawalls. These measures aim to address potential risks and enhance the resilience of Waverley's coastal areas.

2.5.2 Bronte Park and Beach. Plan of Management (2017)

This document is a strategic document that guides the sustainable use and management of public land, incorporating research and community input to shape future directions and actions. It aims to balance the interests of diverse user groups, consolidating information about the site and its users. When paired with a masterplan—a comprehensive, long-term design strategy—the combined framework creates a vision and offers strategic and operational guidance for the site's design and management over an extended period. This integrated approach ensures effective and sustainable use of public land while considering the evolving needs of the community.

2.5.3 Eastern Beaches CMP Stage 1 Scoping Study (2020)

The Woollahra Municipal Council, Waverley Council, and Randwick City Council, in collaboration with the NSW Department of Planning, Industry and Environment (DPIE), are developing a Coastal Management Program (CMP) for Sydney's Eastern Beaches. The CMP, aligned with the Coastal Management Act 2016, aims to provide a long-term strategy for coordinated coastal zone management. The first stage, a Scoping Study, has been completed, outlining the strategic context, vision, objectives, geographic areas, priority issues, knowledge gaps, governance considerations, a preliminary business case, community engagement strategy, and a forward plan for the CMP. This study serves as the initial step in a five-stage process defined by the NSW Coastal Management Framework, setting the groundwork for subsequent stages in the comprehensive preparation of the Eastern Beaches CMP.

2.5.4 Eastern Beaches: Regional Sea Level Rise Hazard Assessment (2021)

BMT has been commissioned to conduct a regional sea-level rise vulnerability assessment in collaboration with Randwick City Council, Waverley Council, and Woollahra Municipal Council. The project aims to provide a consistent hazard assessment, identifying key assets and areas at risk. This would enable the councils to strategically allocate resources for managing coastal areas in the face of sea-level rise. The assessment includes detailed studies such as hydraulic modelling and asset management, integrating relevant policies and response activities. Part of Stage 2 of the Eastern Beaches Coastal Management Plan, the study focuses on evaluating coastal hazards, specifically storm effects and sea-level rise through tidal inundation. The findings would inform the councils' long-term management strategies for coastal resilience.

3 Coastal and maritime engineering site inspection

A site inspection would be undertaken by an experienced team of coastal and maritime engineers from RHDHV. The inspection would cover the seawall and adjoining structures to review the assessments presented by Horton Engineering (Horton Coastal Engineering , 2023) and ARUP (ARUP, 2016) reports, to inform our understanding of the condition of the site and opinion as the designer for the upgraded seawall and related elements. At the time of writing this report, the site inspection had yet to be completed however this task is schedule to be undertaken prior to detailed design (Stage 3). Nevertheless, upon thorough analysis, RHDHV can now provide a general overview of the site and a summary of reduced levels.

The seaward edge of the concrete promenade, positioned beyond Bronte SLSC and atop the seawall, maintains a level of around +4.9m AHD near the steps leading north up the headland. This elevation decreases to +4.7m AHD at the southern edge of the steps leading to the beach, about 1.4m south, remaining consistent with the concrete ramp. The level reduces to +4.6m AHD at the southern end of the ramp. Continuing towards the southern end of the SLSC, it further reduces to +4.5m AHD at the double set of steps approximately 5m south of the SLSC.

Proceeding south along Bronte Beach, the top of the seawall gradually decreases: 4.4m AHD about 5m south of the double steps, +4.2m AHD at the double ramp, and +3.7m AHD at the double steps positioned roughly 30m north of the South Bronte Amenity and Community Centre.

The promenade level at the base of the steps leading to the northern section of the SLSC varies from +5.2m AHD to +5.0m AHD. The pathway at the top of these steps is at +5.65m AHD.

A culvert beneath the promenade, turning onto the beach near the northern end of the ramp, discharges approximately 130m to the northeast. The top surface of this culvert is at +4.1m AHD, adjacent to the ramp.

The finished ground floor level of the existing SLSC clubhouse varies between +5.62m and +5.80m AHD over the northern portion and +5.55m and +5.64m AHD over the southern portion.

4 Basis of design process elements

Basis of Design (BoD) process elements for the seawall are outlined below. A risk-based assessment of the design life, design storm events and the acceptable level of damage during storm events has been undertaken to develop a design philosophy to suit the objectives for the seawall structure, taking into consideration the likelihood and consequence of failure. Incipient failure of the seawall in this case is related to an acceptably low level of damage, that which would require some further maintenance/ remediation following the design event.

Discussions with Council would be conducted as required, to work through and gain acceptance of the design philosophy and related design parameters.

4.1 Coordinate system and vertical datum

The horizontal coordinate reference system adopted in the project would be GDA2020 / MGA zone 56. All levels are reported to Australian Height Datum (AHD). Zero metres AHD is approximately equal to Mean Sea Level at the Australian coastline. Directions are in degrees, referenced to the true north and measured clockwise according to the nautical convention.

4.2 Topographical survey

As per the ARUP report (ARUP, 2016) a topographical survey of the Bronte Beach Seawall, beach profile, and its surroundings was conducted by LTS Lockley on May 31, 2016. Following this survey, a significant East Coast Low storm event affected the NSW coast, including Bronte Beach, on June 5-6, 2016. Based on recommendations from ARUP and Baird, the Council decided to commission a post-storm survey of the beach to compare it with the pre-storm condition. LTS Lockley carried out this post-storm survey on June 12, 2016. An extract of the results of this survey focused on the project site is presented in **Figure 4-1**. For topographic survey details, the reader is directed to the full topographic survey included in the ARUP seawall investigation report (ARUP, 2016).

RHDHV understands that sufficient information is available to start the technical studies and at the time of preparing this document, no additional survey was being planned.

4.3 Bathymetric survey

A bathymetric survey is important for understanding the extent of reef protection in the vicinity of the beach and natural slopes across the beach profile. Such information permits a comprehension of the movements of sand, the orientations, and alignments of coastal features, and determining the predominant directions for sediment drift and accumulation.

The NSW Office of Environment and Heritage, in collaboration with The Central Resource for Sharing and Enabling Environmental Data in NSW, provides topographic and bathymetric data based on Airborne LiDAR Bathymetry (ALB) technology conducted by Fugro Pty Ltd from July to December 2018 (refer to **Figure 4-2**). Analysis of the data indicates that the nearshore seabed slope ranges from 1 in 50 (v:h) between the -40m and -10m AHD, and 1 in 40 between -10m and 0m AHD.

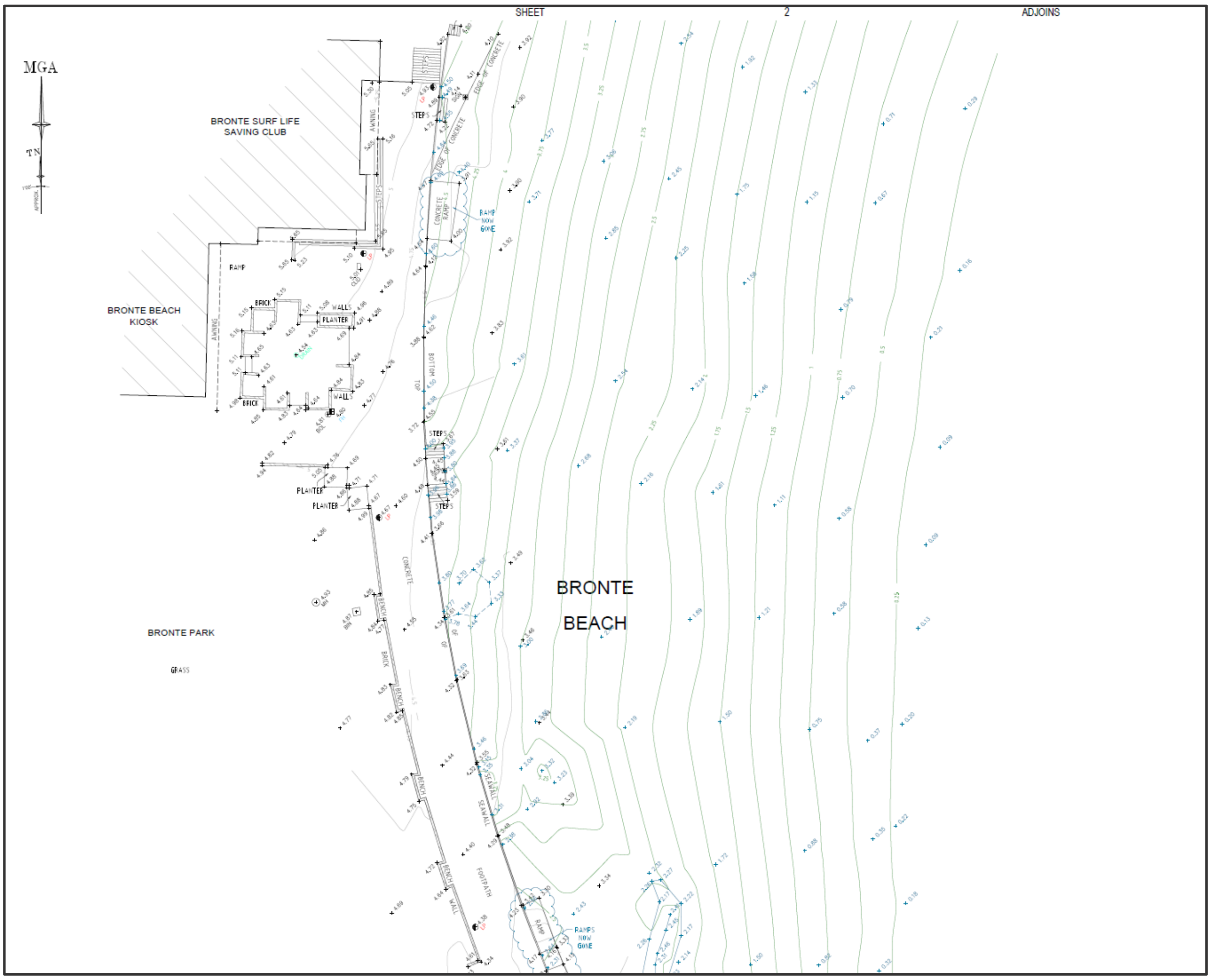


Figure 4-1 Extract from Topographical Survey Drawings. Black text represents survey levels as of May 31, 2016. New levels in blue and new contours in green represent survey results updated on June 16, 2016 (Source: LTS Lockley)

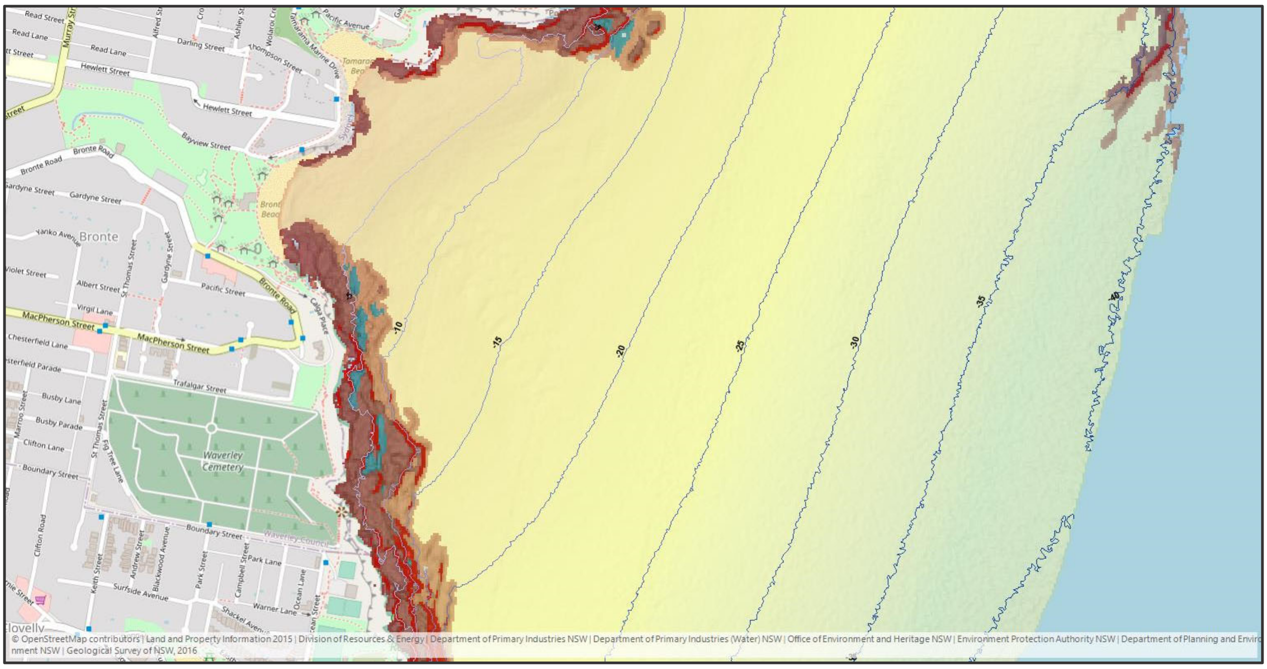


Figure 4-2 NSW Marine Lidar Bathymetry Data 2018 (SEED, The Central Resource for Sharing and Enabling Environmental Data in NSW, https://geo.seed.nsw.gov.au/)

4.4 Geotechnical data

As per the ARUP report (ARUP, 2016), ground investigations took place on June 9, 2016, and included the following scope:

- Five test pits (TP) to confirm the toe level of the wall and the foundation material;
- Two additional test pits (TP104 & 107) to confirm the depth of bedrock in the vicinity of the proposed new lifeguard tower (a separate Council project);
- Ten concrete cores (CC) drilled horizontally into the seawall; and
- Laboratory testing of samples including particle size distribution, chloride content and concrete strength testing.

The selection of test pit locations aimed to cover the extremities of the seawall, providing a comprehensive understanding of the ground conditions and the seawall itself. Test pit and concrete core locations can be seen in **Figure 4-3**.

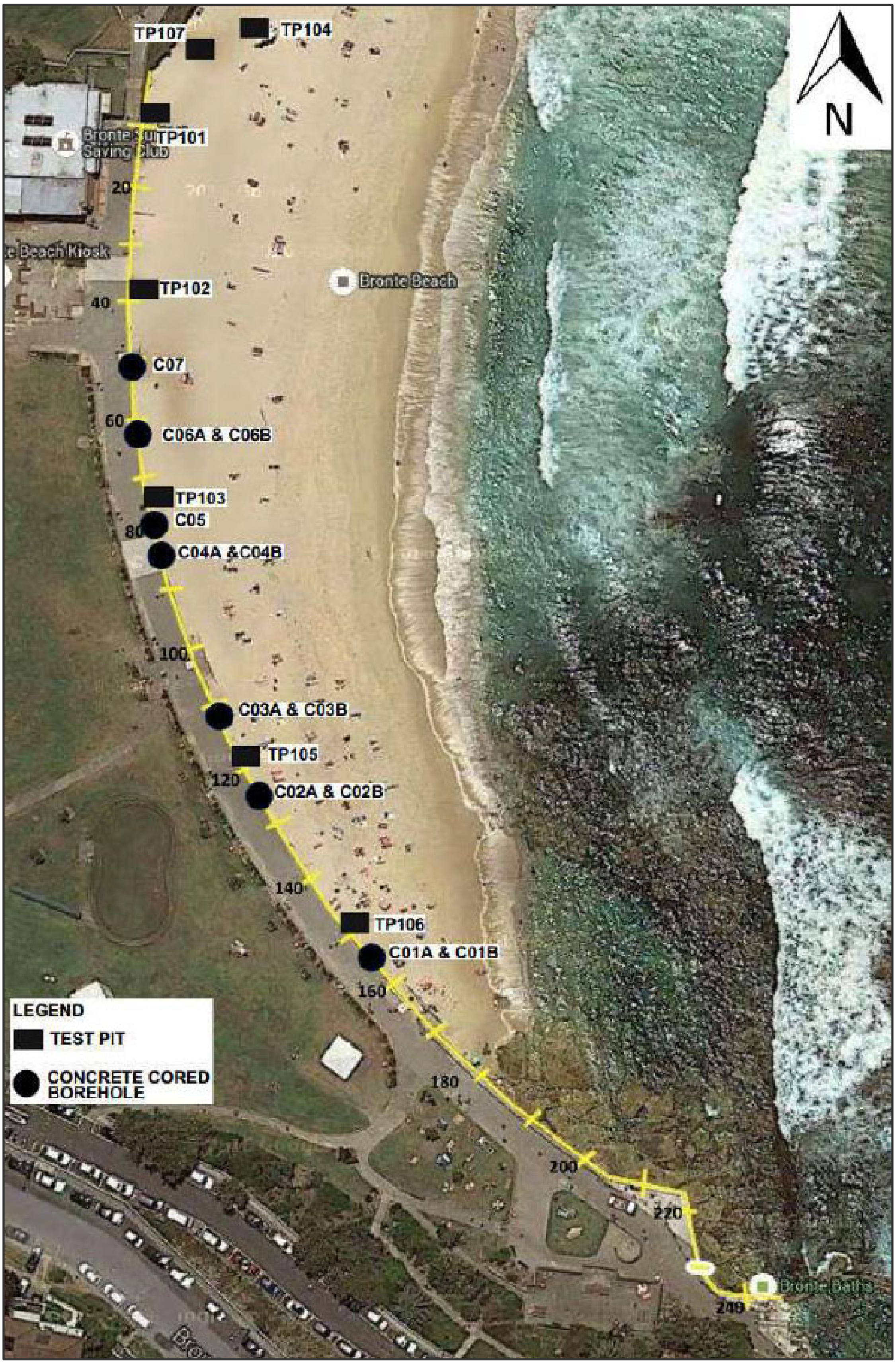


Figure 4-3 Seawall chainages and test pit and seawall coring locations (ARUP, 2016)

A summary of the approximate reduced levels of the test pit locations and subsurface conditions encountered at the three sites is provided in **Table 4-1** and **Table 4-2** based on ARUP's report (ARUP, 2016). The test pits successfully revealed the type and depth of the seawall foundation. In the southern region, the wall appears to be founded on Hawkesbury Sandstone. TP103 exposed the wall sitting on a brick pier. The geotechnical investigation uncovered a subsurface profile characterised by medium- to coarse-grained beach sands overlaying medium- to coarse-grained sandstones. Observations indicated the presence of sandstone outcrops at both the north and southern ends of the seawall.

Table 4-1 Approximate levels of test locations

Test Location	Reduced Level	Test Pit Termination Depth	Test Pit Termination Depth	Depth from top of wall	Seawall toe exposed ?
[-]	[m AHD]	[m BGL]	[m AHD]	[m]	
TP101	4.60	2.30	2.30	2.30	Yes
TP102	3.90	1.90	2.00	1.90	Yes
TP103	4.30	2.70	1.60	4.20*	Yes
TP104	3.90	3.00	0.90	N/A	No
TP105	4.10	3.00	1.10	3.00	Yes
TP106	3.90	2.70	1.20	2.70	Yes
TP107	3.90	4.20	-0.30	4.20**	No

Notes:
* Wall was sitting on brick piers. Depth to top of pier was 1.70m
** Depth to base of culvert

Table 4-2 Summary of subsurface conditions

Strata	Depth to base		Thickness
[-]	[m BGL]	[m AHD]	[m]
Beach sands	Not proven to 4.20	Not proven to -0.30	4.20 to not proven
Hawkesbury Sandstone	Not proven	Not proven	Not proven

Geotechnical investigations at the subject site have been carried out by AssetGeoEnviro (AGE) in 2020 (AssetGeoEnviro, 2020) and 2022 (AssetGeoEnviro, 2022). The 2020 study involved drilling three boreholes (BH1, BH2, and BH3) at the landward, centre, and seaward edges of the development area, respectively (refer to **Figure 4-4**).

Figure 4-4 Borehole location (AssetGeoEnviro, 2022)

A generalised geotechnical model for the site has been developed as shown in **Table 4-3** where the subsurface conditions were generally identified as sand overlying sandstone bedrock. **Table 4-4** provides specific details of these boreholes.

Table 4-3 Generalised site geotechnical model (AssetGeoEnviro, 2020)

Unit	Origin	Description	Depth to Top of Unit	Unit of thickness
[-]	[-]	[-]	[m]	[m]
1	Topsoil	TOPSOIL, Silty SAND/SAND with some silt, brown grey/ dark brown/ light grey, fine to medium grained, trace of grass roots.	Ground surface	0.15 to 0.2
2	Fill	FILL, Sandy CLAY with some silt, trace of glass fragments and some subangular gravels, fine to medium grained gravels, dark brown/ Red orange/ Yellow brown. (Only in BH4). Appeared to be moderately compacted.	0.2	0.4
3	Aeolian	SAND/ SAND with some silt, fine to medium grained, yellow brown with traces of grey to pale brown, grading to medium grained sand with depth below 1.5m depth. Medium dense, becoming dense to very dense with depth.	0.2 to 0.6	0.2 to 1.4
4	Residual	Sandy CLAY/ Clayey SAND, fine to medium grained sand, low 5 to medium plasticity, pale brown with traces of orange, brown. Dense/ Very Stiff.	1.5	0.1
5	Bedrock	Inferred SANDSTONE, low strength, moderately weathered, assessed Class 4 Sandstone.	0.8 to 2.45	Not proven beyond a depth of 2.45 by DCp

Notes:

1. The depths and unit thicknesses are based on the information from the test locations only and do not necessarily represent the maximum and minimum values across the Site.
2. Rock classification to Pells, P.J.N., Mostyn, G. & Walker, B.F., Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

Table 4-4 Boreholes detail (ARUP, 2016)

Borehole	Location	Stated surface level	Surface level from survey	Depth to sandstone bedrock	Level of sandstone bedrock
[-]	[-]	[m AHD]	[m AHD]	[m]	[m AHD]
BH1	About 30m landward of clubhouse	5	5.7	8.2	-2.5
BH2	NW corner of sunken courtyard	4	5.3	3.8	1.5
BH3	Seaward of clubhouse on promenade	4	5.0	4.2	0.8

The back beach in **Figure 4-4** falls in the active coastal zone, where erosion would typically be expected down to -1m AHD on a sandy beach. It is likely that the bedrock surface along the face of the seawall seaward of the SLSC is higher than typical back beach scour levels.

An additional geotechnical investigation is planned to be undertaken as part of the Bronte seawall redevelopment. The geotechnical field work would include the drilling of two boreholes from beach level and extend a minimum 3m into Class III sandstone bedrock inform the seawall pile design (in accordance with (P. J. N. Pells, 2019)). Five test pit excavations would also be undertaken to further assess and confirm the footing details and foundation materials below the existing seawall and culvert, one at each end of the new seawall where end effects would need special attention, a third at the intersection between the existing seawall and culvert, and two at intermediate points along the base of the existing seawall to confirm toe continuity.

Furthermore, a seismic refraction survey would also be requested to map the bedrock levels along the general alignment of the proposed seawall upgrade and cross shore under the beach to help characterise the bathymetry for physical modelling. This survey would be conducted by a specialist with expertise in seismic testing techniques.

4.5 Groundwater

As per the ARUP report (ARUP, 2016), groundwater was encountered in several test locations, as summarised in **Table 4-5**.

Table 4-5 Summary of encountered groundwater

Strata	Depth to groundwater	
[-]	[m BGL]	[m AHD]
TP101	Groundwater not observed	
TP102	Groundwater not observed	
TP103	2.70	1.60
TP104	3.00	0.90
TP105	3.00	1.10
TP106	2.70	1.20
TP107	4.20	-0.30

During the geotechnical investigations carried out by AssetGeoEnviro in 2020 (AssetGeoEnviro, 2020) and 2022 (AssetGeoEnviro, 2022) groundwater was not observed in the boreholes during auger drilling to depths of 0.8m to 1.6m BGL. However, moist to wet sandy soils were observed at around 0.5m depth at BH1, BH2, and BH4 locations. Groundwater was also not observed during the DCP probing. Groundwater detection via DCP test is indicated by wet soil materials attached on the DCP rods and conical tip after rod extraction. No long-term groundwater monitoring was carried out.

4.6 Design life of sea defence

In determining an appropriate design life for a sea defence, three components need to be considered; permissible risk of failure, design event and design life of the asset to be protected. The balance of capital expenditure versus risk and maintenance costs must be considered. Adopting a lengthy design life with a low permissible risk of failure and a rare design event may seem prudent, but it would likely be cost prohibitive. It is therefore necessary to rationalise these design parameters to ensure the remedial works are realistically fundable.

Determining the "appetite for risk" in coastal assets involves understanding the social and economic impacts of potential damage (Gordon, Carley, & Nielsen, 2019). The acceptable consequences of damage must be identified, and then the likelihood criteria for designing protective structures can be established. The specific situation dictates the appetite for risk, with projects like seawalls protecting parkland allowing higher risks based on "tolerable" rather than "acceptable" criteria. Although it cannot be overlooked, risk to life is generally rare and not a primary focus in designing protective structures for most coastal assets.

Establishing the design working life for the seawall is critical to enable estimation of its design parameters. The design life of a structure is related to the typical design components, such as concrete and steel. The design life used in various Australian Standards is as follows:

- AS 1170 (structural design): 50 years
- AS 2870 (residential slabs and footings): 50 years
- AS 3600 (concrete): 40 to 60 years
- AS 4678 (earth-retaining structures): 60 years
- AS 4997 (maritime structures): 50 years for a normal maritime structure and 100 years for a structure protecting residential developments.

In 2007, the Australian Geomechanics Society (AGS) noted in their National Landslide Risk Management Framework for Australia that a design life of at least 50 years would be reasonable for permanent structures used by people and that there is a community expectation that a residential dwelling frequently, with appropriate maintenance, would have a functional life well in excess than 50 to 60 years (Australian Geomechanics Society, 2007). AGS state that a design should include details of required inspections and maintenance to enable risk mitigation measures to remain effective for at least the design life of the structure.

Coastal Engineering Manual EM 1110-2-1100 (Part V) states that it is usual for an economic life of 50 years to be selected for analysis of a coastal structure. This does not imply that the structure would only last 50 years, but that the analysis of benefits and costs is limited to that period.

The proposed seawall for this project aims to provide essential coastal protection to Bronte SLSC and the public users situated behind it. Horton Coastal Engineering initially recommended a 50-year structural engineering design life for the proposed SLSC redevelopment (Horton Coastal Engineering , 2023). However, a 70-year coastal engineering design was ultimately adopted as per the Council's request.

The chosen 70-year design life aligns with standard industry practices, meeting minimum requirements necessary for ensuring the seawall's effectiveness and longevity. The adopted design life dictates the seawall's capability to withstand coastal erosion and wave overtopping events, ensuring an acceptably low risk of damage over its operational lifespan.

4.7 Design event

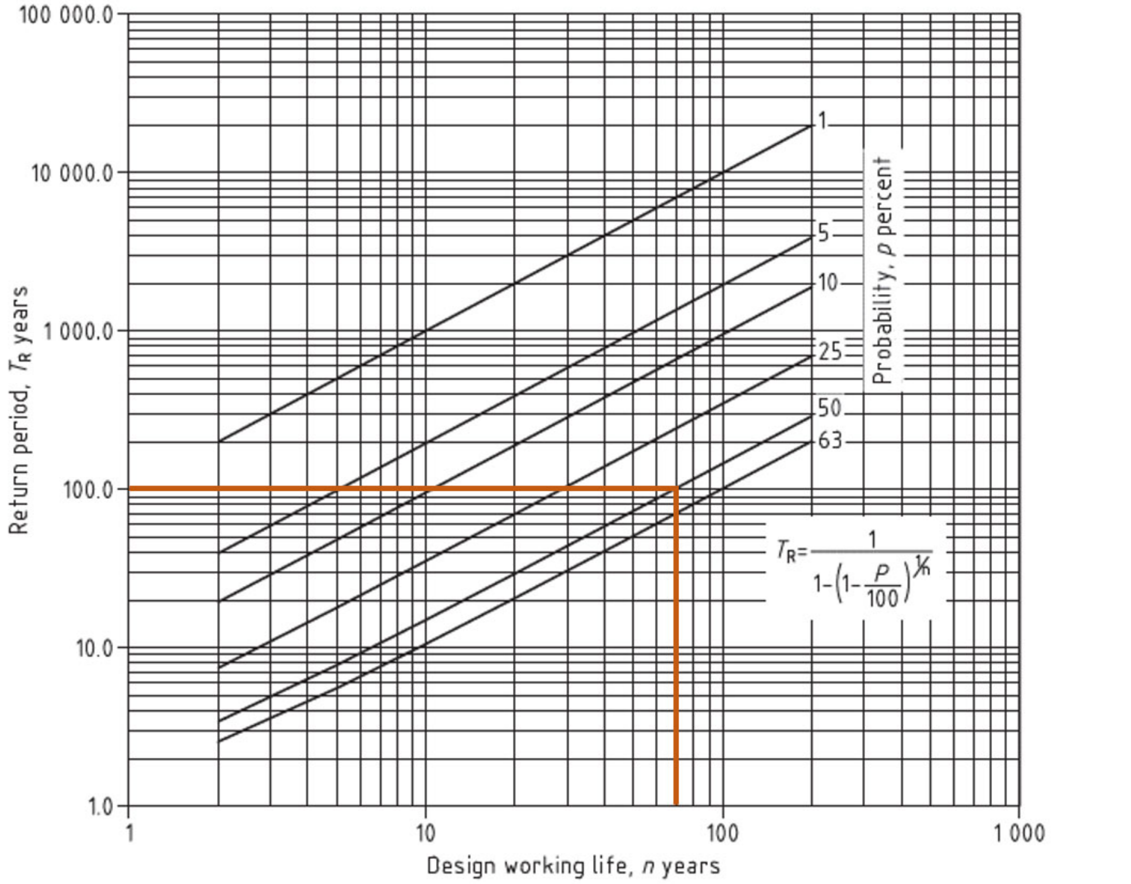
There is a lack of explicit formal guidance available for determining the appropriate design event for open-coast protective coastal structures (Gordon, Carley, & Nielsen, 2019).

In accordance with AS 4997, the recommendation is to establish significant wave heights for marine structures, considering the function and design life of the structure (refer to **Table 4-6**). According to this guideline, opting for the 50-year, 200-year, 500-year, and 1000-year Average Recurrence Interval (ARI) events is suitable for seawalls, which fall under the category of 'normal' maritime structures. It is noteworthy that AS 4997 specifically addresses rigid maritime structures like wharves and concrete seawalls, excluding the design of flexible "coastal engineering structures." Furthermore, the seawalls under evaluation are generally smaller structures, often integral components of broader foreshore management solutions. The prevailing best practices in coastal hazard assessments for local government areas commonly lean towards adopting the 100-year ARI as the design criterion. Consequently, there is a rationale for considering some reduction in the design conditions. Importantly, AS 4997 does not offer specific guidance on the recommended design water level.

Table 4-6 Annual probability of exceedance of design wave events (Standards Australia, 2005)

Functional Category	Design Working Life (Years)			
	5 or less (temporary works)	25 (small craft facilities)	50 (normal maritime structures)	100 or more (special structures / Residential developments)
Structures presenting a low degree of hazard to life or property	1/20	1/50	1/200	1/500
Normal structures	1/50	1/200	1/500	1/1000
High property value of high risk to people	1/100	1/500	1/1000	1/2000

Proposing the adoption of the 100-year ARI design storm event aligns with good practice manuals like British Standards (BS EN 1990:2002+A1) and to our experience in this field a 100-year ARI design event selected for the structures are appropriate. However, given the Council's request for a 70-year design life, the relationship between design working life and return period is expressed in terms of risk of non-performance or exceedance of specified conditions and shows that there is a 50% probability that a 100-year ARI storm event occur in the 70-year design life of the structure (refer to **Figure 4-5**). British Standards. This probability may be unacceptably high for the design.



NOTE T_R is the return period of a particular extreme condition in years. p is the probability of a particular extreme condition occurring during design working life n years.

Figure 4-5 Relationship between design working life, return period and probability of an event exceeding the normal average (BRITISH STANDARD, 2016)

Given the depth-limited conditions, it is important to note that the design wave heights at the seawalls could be generated by an event where the recurrence interval of the deep-water wave height was lower than the 100-year ARI. Similarly, a water level that is higher than 100-year ARI associated with a relatively low deep water wave height could result in the same design wave height at the seawall. This highlights the complexity of assessing wave conditions and emphasizes the need to consider both wave and water level parameters in evaluating the design wave condition.

Horton Coastal Engineering report selects the 100-year ARI event for water level conditions (Horton Coastal Engineering , 2023), while ARUP the 5-year ARI, 20-year ARI, and 100-year ARI for both wave and water level conditions (ARUP, 2016).

RHDHV recommends specific measures for wave overtopping, particularly when generated by depth-limited waves. The following guidelines are suggested:

- Water Level Adoption: Adopt a 100-year ARI water level.
- Wave Parameter Adopt 100-year ARI wave parameters.
- Design scour level: Adopt 100-year ARI scour level set at 2.90m AHD (present day), reducing to between 1.70 and 2.00m AHD (under a 2050 climate change-induced sea level rise scenario),

and to between 0.35 and 0.75m AHD (under a 2100 climate change-induced sea level rise scenario).

- Design wave loads for structural assessment: Adopt 100-year ARI water level combined with 100-year ARI wave parameters.

This approach is considered reasonable as the scour level governs water depths and, consequently, the depth-limited wave heights impacting the proposed seawall. The combination recommended for structural assessment would represent a rarer than 100-year ARI event. Considering a lower ARI event for overtopping is considered reasonable, given its lesser potential damages compared to structural failure. A minimum no-development setback of 10m landward from the crest of the wall is advised at the SLSC development to facilitate some dissipation of wave overtopping.

4.8 Water levels

Water levels at the project site are primarily driven by astronomical tide. Super elevated water levels which are important for structural loading and wave overtopping are primarily influenced by storm surge (barometric setup and wind setup) and wave setup (caused by breaking waves). Individual waves also cause temporary water level increases above the still water level due to the process of wave runup or uprush. Postulated sea level rise over the long term would directly contribute to future water levels.

4.8.1 Tides

Tides in NSW are microtidal-semidiurnal with a diurnal inequality. This implies that the tidal range is less than 2 metres, featuring two high tides and two low tides each day and exhibiting a once-daily inequality in the tidal range. In Sydney, the mean tidal range is approximately one metre, and the tidal period spans around 12.5 hours.

Spring tides coincide with the new or full moon. On average, the spring tidal range is 1.3 metres, with the maximum range extending to 2 meters. Neap tides, occurring around the first and third quarters of the moon, have an average range of approximately 0.8 metres.

The predicted tidal planes for Port Jackson at Sydney (to the north of the project site) derived by Manly Hydraulic Laboratory are provided in **Table 4-7**.

Table 4-7: Predicted Tidal Planes for Port Jackson (Manly Hydraulics Laboratory, 2023)

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

4.8.2 Storm surge and wave setup

The combined effect of barometric pressure setup and wind stress setup is referred to as storm surge. Barometric pressure setup refers to the increase in mean sea level caused by a drop in atmospheric pressure, such as when a low-pressure system is centred over an area. Wind stress setup is the increase in mean sea level caused by the 'piling up' of water on a shoreline by wind action acting on the water surface.

Wave setup is the increase in water level within the surf zone, measured above the still water level, caused by the breaking action of waves. This is due to the kinetic energy in the breaking waves being converted into an elevated inshore water level.

In NSW, storm surge and wave setup can significantly elevate open coast water levels during storms. For a 100 year ARI, it is common practice to allow for a storm surge of 0.6m (comprising barometric setup of up to 0.3m to 0.4m and wind setup of up to 0.2m to 0.3m) and a wave setup of up to 1.5m (typically around 10-15% of the significant wave height in deepwater) (WorleyParsons, 2011).

As per WorleyParsons report (WorleyParsons, 2011), historical water level records at Fort Denison in Sydney Harbour, representative of open coast water levels near Sydney, indicate that the 100-year ARI water level (including astronomical tide and storm surge) is predicted to be 1.5m AHD. This prediction is based on a joint probability analysis of tide and storm surge events.

When factoring in wave setup, typically calculated as 15% of the unrefracted deepwater significant wave height, the 100-year ARI wave setup is estimated to be 1.2m. In less exposed areas, like the northern end of Bronte Beach, equivalent elevated water levels would be diminished due to a lower wave setup. Given the empirical nature of wave setup estimation, it is recommended that the detailed design of coastal protection improvement works incorporate sensitivity analysis. This analysis should be based on wave setup variations ranging from 10% to 20% of the design offshore significant wave height (Guza & Thornton, 1981) (Holman, 1986). Alternatively, site-specific modelling could be undertaken. Such modelling should account for the presence of a seawall which by definition truncates the surf zone. Consequently, the full extent of wave setup on a dissipative beach may not be realised when a seawall is present. A good understanding of these factors is helpful for an accurate assessment of coastal protection measures.

4.8.3 Sea level rise

The Intergovernmental Panel on Climate Change (IPCC) 2021 report (Intergovernmental Panel on Climate Change [IPCC], 2021) provides global mean sea level rise projections for five Shared Socioeconomic Pathways (SSPs). Each SSP comprises a narrative of future socioeconomic development used to develop scenarios of energy use, air pollution control, land use, and greenhouse gas emissions to which Representative Concentration Pathways (RCPs) are applied to achieve an approximate radiative forcing level at the end of the 21st century. The SSPs considered in the IPCC 2021 report include:

- SSP1–2.6 - Low emissions scenario
- SSP2–4.5 - Intermediate emissions scenario
- SSP3–7.0 - High emissions scenario, and
- SSP5–8.5 - Very High emissions scenario.

For each SSP scenario, the IPCC 2021 report provides sea level rise (SLR) projections for future years up to 2150 comprising median values along with a likely range (medium confidence)[1].

The latest IPCC Assessment Report (2021) SLR projections for a range of Shared Socio-Economic Pathway (SSP) scenarios are outlined in **Table 4-8**. These values have been extracted from the NASA Sea Level Projection Tool (https://sealevel.nasa.gov/ipcc-ar6-sea-level-projection-tool) and correspond to the 'Sydney, Fort Denison' location. The predicted values in 2093 are highlighted in bold, based on adoption of a 70-year design life period for the structure.

Table 4-8: SLR projections from IPC 2021 report for 'Sydney, Fort Denison' (noting 2093 values are interpolated) (Source: NASA Sea Level Projection Tool)

Year	SSP1-2.6 (low)	SSP3-7.0 (median)	SSP5-8.5 (median)
[-]	[m]	[m]	[m]
2023	0.00	0.00	0.00
2030	0.03	0.04	0.04
2040	0.06	0.08	0.10
2050	0.09	0.16	0.17
2060	0.11	0.22	0.25
2070	0.15	0.30	0.35
2080	0.17	0.40	0.46
2090	0.20	0.50	0.59
2093	**0.21**	**0.56**	**0.66**
2100	0.22	0.62	0.72
2150	0.33	1.14	1.29

If a 70-year planning period is applied from 2023, the estimated sea level rise in 2093 relative to the present time would be 0.66m if the very high emissions scenario SSP5-8.5 was adopted.

4.8.4 Design still water level

The Coastal Risk Management Guide (DECCW) (Department of Environment, Climate Change and Water, 2010) recommends design elevated water levels for a range of average recurrence intervals, which are presented in **Table 4-9**. This is like be the corresponding value reported by Manly Hydraulics Laboratory (MHL) (Manly Hydraulics Laboratory, 2018)[2]. Applying these values to the present (2023) using a rate of sea level rise of 3mm/year from 2010 to 2023, as recommended in DECCW (Department of Environment, Climate Change and Water, 2010) it was possible to estimate the present day ocean water level (in the absence of wave action) for each ARI.

The adopted design (still) high water levels, based on the analysis presented in the preceding sections are shown in **Table 4-10**.

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[1] The 'likely' range is associated with the 17th to 83rd percentile range for each SSP. The IPCC 2021 report also provides low confidence projections for the SSP5-8.5 scenario, which includes a 'very likely' upper bound projection, i.e., 17th to 95th percentile range.

[2] (Manly Hydraulics Laboratory, 2018) determined a corresponding level of 1.42m AHD (along with lower and upper 95% confidence limits of 1.38m AHD and 1.53m AHD respectively).

Table 4-9: Design still water levels at Fort Denison (NSW Goverment, 2010)[(1)]

Average Recurrence Interval	2010 Design Still Water Level Excluding Wave Setup and Runup [(2)]	2023 Design Still Water Level Excluding Wave Setup and Runup[(3)]
[Years]	[m AHD]	[m AHD]
1	1.24	1.28
10	1.35	1.39
50	1.41	1.45
100	1.44	1.48

Notes:

(1) The design still water levels are only relevant where full ocean tide conditions prevail.

(2) Design still water levels for 2010 were derived from extreme value analysis of Fort Denison tide gauge data from June 1914 to December 2009 (Watson & Lord, 2008). There are negligible tidal friction losses between the ocean and Fort Denison within Sydney Harbour; therefore, Fort Denison data provides an indicative representation of oceanic still-water levels. The design still-water levels inherently incorporate allowance for all components of elevated ocean water levels experienced over this timeframe (including tides, meteorological influences, and other water level anomalies); however, they exclude wave setup and wave runup influences.

(3) Design still-water levels for 2023 incorporate planning benchmark allowances for sea level rise with a reduction of 60 millimetres to accommodate the estimated amount of global average sea level rise that has occurred between 1990 and present. From satellite altimetry, this is estimated to be 3 millimetres/year (CSIRO, 2009).

Table 4-10: Design still high-water levels

Design Life	Average Recurrence Interval	Tide Level	Wave setup	Sea Level Rise	Design High Water Level
	[Years]	[m AHD]	[m]	[m]	[m AHD]
2050	1	1.28	0.8	0.17	2.25
	10	1.39	1.0	0.17	2.56
	50	1.45	1.2	0.17	2.82
	100	1.48	1.2	0.17	2.85
2093	1	1.28	0.8	0.66	2.74
	10	1.39	1.0	0.66	3.05
	50	1.45	1.2	0.66	3.31
	100	1.48	1.2	0.66	3.34
2100	1	1.28	0.8	0.72	2.80
	10	1.39	1.0	0.72	3.11
	50	1.45	1.2	0.72	3.37
	100	1.48	1.2	0.72	3.40

4.9 Wave Climate

4.9.1 Offshore wave climate

Sydney, situated in the south-west Pacific at 34°S, receives waves from the southern Coral and Tasman Seas generated by five meteorological systems: tropical cyclones, east-coast cyclones, mid-latitude cyclones, zonal anticyclonic highs, and local summer seabreezes (WorleyParsons, 2011). Over 20 years of Sydney wave data analysis reveal distinct seasonality, with February, March, and June experiencing the largest average monthly wave heights (refer to **Figure 4-6**). The NSW coast, subject to a moderate wave climate, faces periodic large coastal storm events that can result in coastal inundation, beach erosion, property and marine structure damage, and public safety risks.

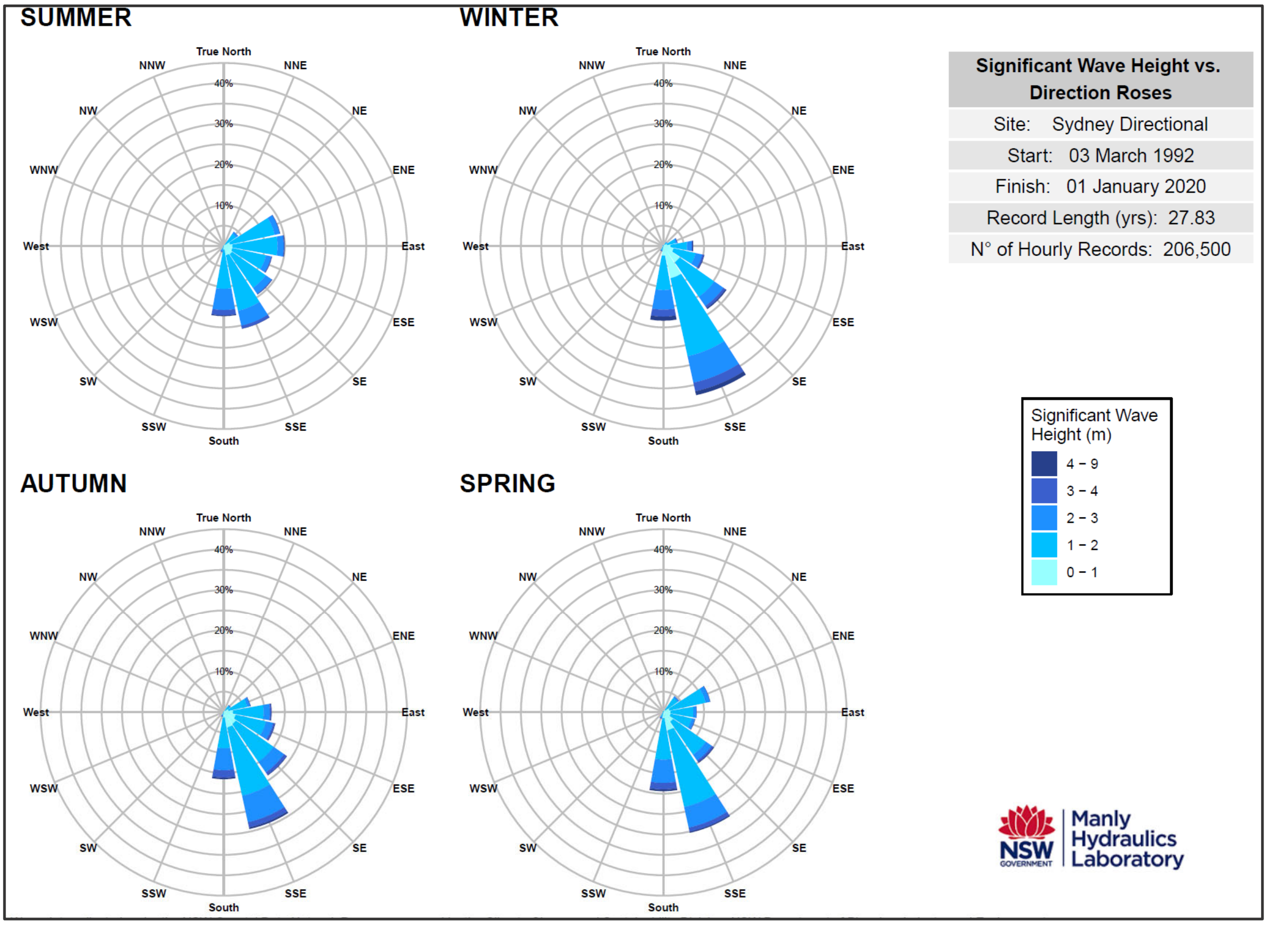


Figure 4-6 Sydney Waverider Buoy. Seasonal wave height and direction roses (Manly Hydraulic Laboratory , 2022)

MHL collects offshore wave data at seven sites off the NSW coast using Waverider buoys. The buoys are strategically located to provide comprehensive deepwater wave data. The Sydney Waverider Buoy, approximately 11km ESE of Long Reef (refer to **Figure 4-7**), is representative of offshore wave conditions influencing the project site.

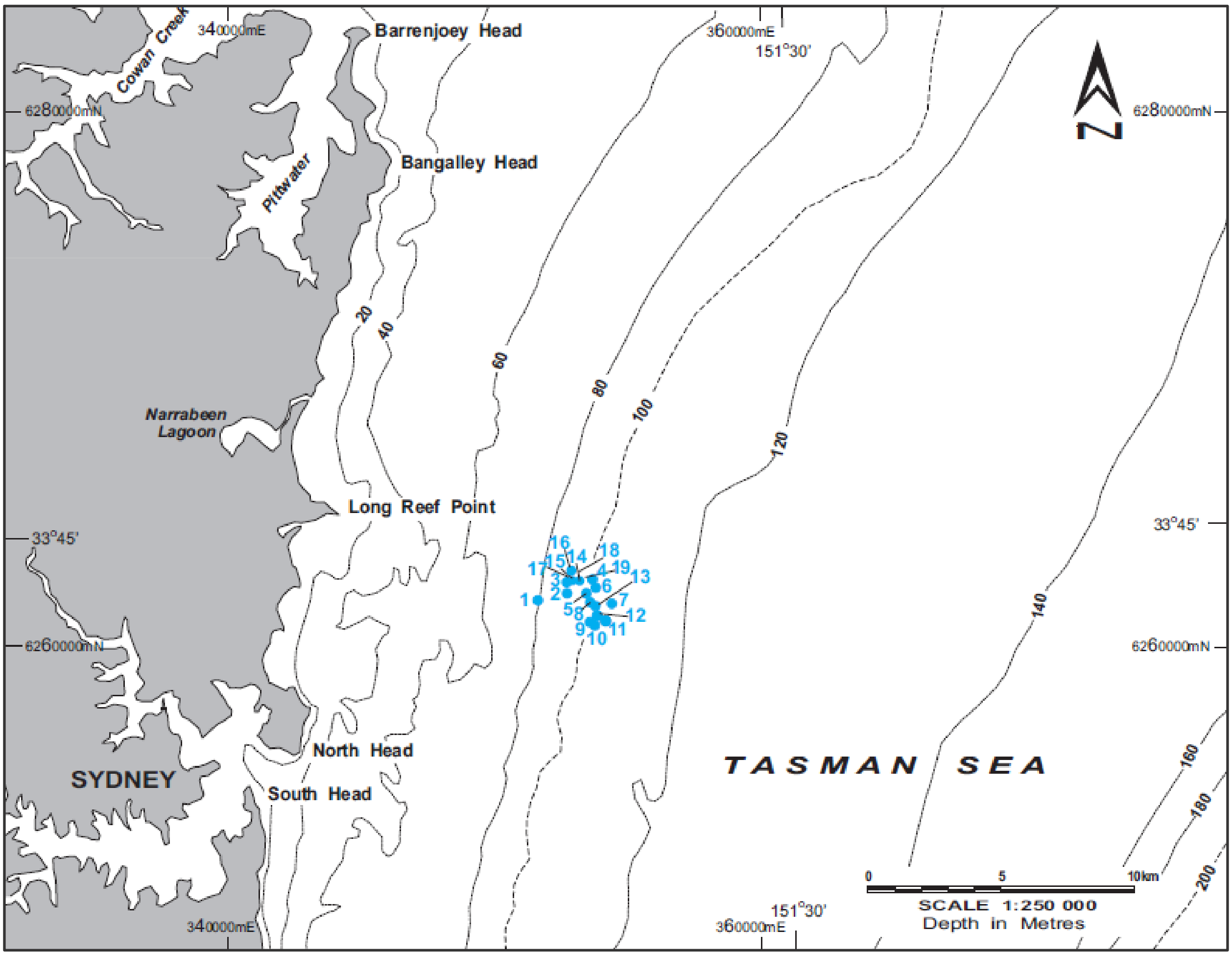


Figure 4-7 Sydney Waverider Buoy and location history (Manly Hydraulic Laboratory , 2022)

Directional analysis from the Sydney Waverider Buoy indicates that approximately 65% of offshore waves propagate from the S-SE sector, originating in the Tasman Sea and Southern Ocean (refer to **Figure 4-8**). Easterly waves make up around 30% of total offshore wave energy, while N-NE waves constitute about 3%[3]. Storm wave analysis reveals that dominant storm wave directions are from the S (38%), SSE (31%), and SE (13%), with waves from E through N accounting for about 9%[3] of storm waves.

[3] *Wave data collected under the NSW Coastal Data Network Program managed by the Climate Change and Sustainability Division, NSW Department of Planning, Industry and Environnment*

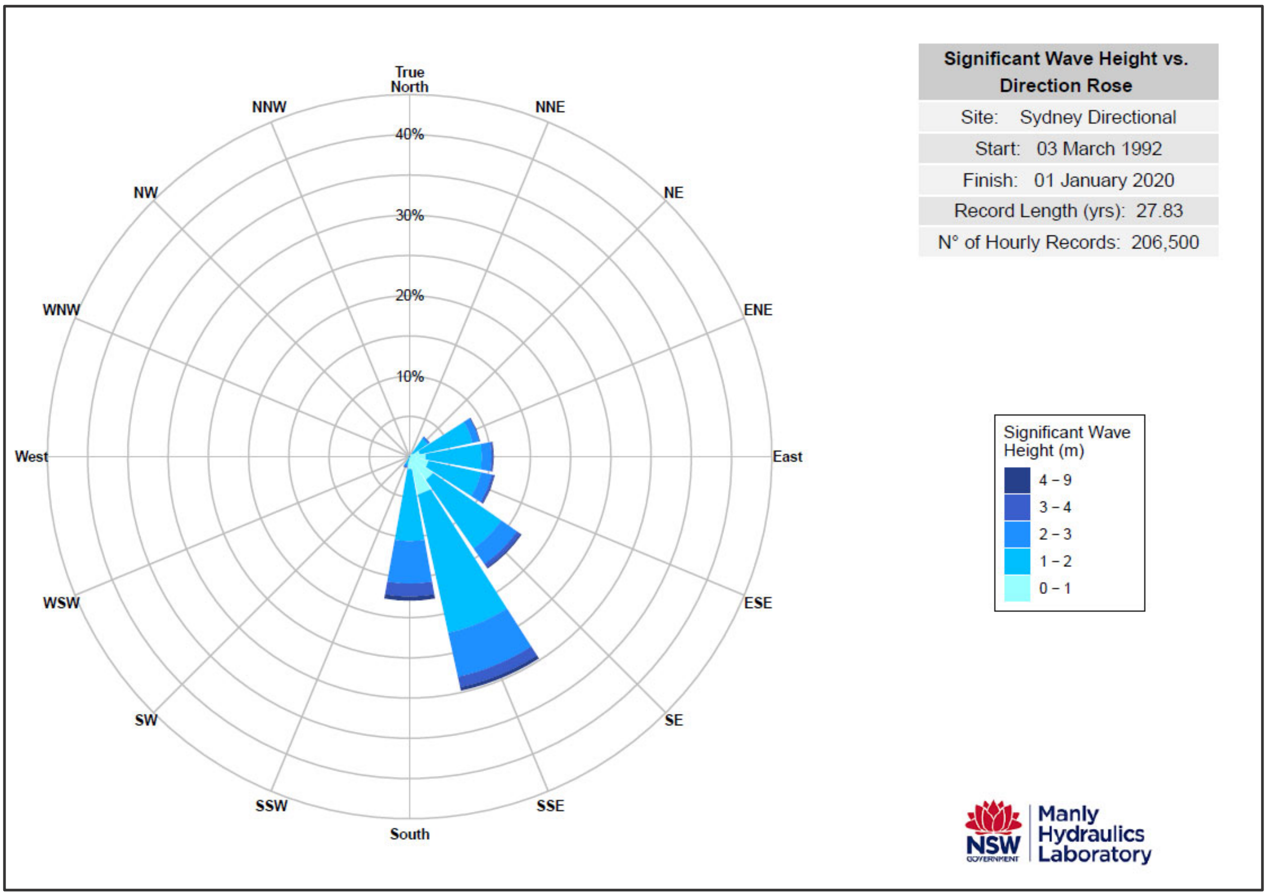


Figure 4-8 Sydney offshore wave rose (Manly Hydraulic Laboratory , 2022)

Directional extreme waves for the 1, 50, and 100-year return periods in the Sydney region were estimated primarily based on analysis of directional data from the Sydney Waverider Buoy (WorleyParsons, 2011). The average wave height expected to occur or be exceeded approximately every 100 years was calculated to be 9.3m. This value aligns well with previously reported estimates for the 100-year return period significant wave height in the Sydney region.

Table 4-1 provides a summary of the directional extreme waves calculated for the offshore region of the study area (WorleyParsons, 2011), utilising data from the Sydney Waverider Buoy.

Offshore wave extreme values reported by WorleyParsons are aligned with recent extreme value offshore wave conditions (since the June 2016 storm) re-evaluated for Sydney by Manly Hydraulic Laboratory (MHL) (Glatz, Fitzhenry, & Kulmar, 2017), based on offshore Waverider buoy records. For Sydney, MHL determined 100-year ARI offshore significant wave heights (H_s) of 9.4m and 8.2m for 1 hour and 6-hour durations, respectively.

Table 4-11 Offshore directional wave extremes for the study region

Average Recurrence Interval	Direction [°N]							
	NE	ENE	E	ESE	SE	SSE	S	SSW
1-year								
Significant Wave Height (Hs) (m)	3.0	4.2	4.8	5.0	5.8	6.4	6.1	3.8
Peak energy period (Tp) (s)	7.6	8.9	9.6	9.8	10.5	11.1	10.8	8.5
50-year								
Significant Wave Height (Hs) (m)	4.1	5.7	6.6	6.9	8.0	8.8	8.4	5.2
Peak energy period (Tp) (s)	8.9	10.5	11.2	11.4	12.4	13.0	12.6	10.0
100-year								
Significant Wave Height (Hs) (m)	4.4	6.0	7.0	7.3	8.5	9.3	8.8	5.5
Peak energy period (Tp) (s)	9.2	10.7	11.6	11.8	12.7	13.3	13.0	10.2

Notes:
Location: 33° 46' 54"S 151° 25' 29"E
Water Depth: 85m
The above are the extremes likely to be reached, or exceeded, once on average every 1-year, every 50-years and every 100-years, respectively for the directional sector indicated at the above location.

Beach erosion and relatively large wave run-up is strongly linked to the occurrence of high wave conditions with elevated ocean water levels, so erosion and run-up are more likely to be significant when large waves coincide with a high tide. Consistent with MHL report (Manly Hydraulics Laboratory , 2016), a 6-hour duration is appropriate for design, as storms with a duration of 6 hours are likely (50% probability) to coincide with high tide on the NSW coast (which is a prerequisite for elevated water levels to occur). A 1-hour duration has less than 10% probability of coinciding with high tide. Therefore, an offshore H_s (or H_{so}) of 8.2m was combined with nearshore wave transformation results, to determine nearshore wave runup levels at the SLSC.

In adopting 100-year ARI design wave conditions at the seawall, it was assumed that the 100-year ARI water level and 100-year ARI offshore wave height occur at the same time, which is conservative. (Shand T. D., et al., 2012) found that considering the joint probability of waves and tidal residuals for Sydney, the wave height for the joint 100-year ARI event reduced by about 10% as the tidal residual increased from 0.05m to 0.4m (with the latter necessary to achieve the design water level). That stated, adopting coincident 100-year ARI water level and 100-year ARI wave conditions is not unreasonable (although conservative), as elevated waves and water levels can be generated by the same weather systems.

A design peak spectral wave period (T_p) of 13s was adopted, based on (Shand, Cox, Mole, Carley, & Peirson, 2011), who determined the associated wave period for the 100-year ARI Hs event on the NSW coast as 13.0s (± 0.7s considering 90% confidence intervals).

The variability observed in the offshore wave climate in the Sydney region may be influenced by climate oscillations such as El Niño/ Southern Oscillation, and climate change could impact future trends in the offshore wave climate.

4.9.2 Nearshore wave climate

Bronte Beach experiences waves originating from offshore storms (swells) or generated locally (wind waves) within the nearshore coastal zone. Swell waves reaching the existing seawall undergo modifications through processes such as refraction, diffraction, wave-wave interaction, dissipation by bed friction, wave breaking, and wind. Similarly, locally generated waves are modified by propagation and dissipation processes.

According to the WorleyParsons report (WorleyParsons, 2011), a previous assessment of nearshore wave conditions at Bondi Beach involved a wave refraction/ diffraction analysis. The study indicated that nearshore wave coefficients (ratios of nearshore to offshore wave heights) in a nearshore water depth of approximately 5m decreased from around 1.0 at the southern end of the beach to approximately 0.6 at the northern end. This variation in nearshore wave conditions along the beach could result in significant differences in beach erosion volumes and wave runup levels. Although no nearshore wave modelling has been conducted for Bronte Beach, observations from available reports and videos of the June 2016 storm event suggest a similar behaviour at Bronte Beach.

Extreme nearshore wave conditions at Bronte Beach for 5, 20, and 100-year ARIs, as estimated by ARUP and Baird (ARUP, 2016), are presented in **Table 4-12**. The wave conditions affecting the seawall would depend on the beach levels eroded during the storm event.

Table 4-12 Nearshore design wave conditions at Bronte Beach (10m water depth contour) (ARUP, 2016) and (Baird, 2016)

Average Recurrence Interval	Offshore Direction	Significant Wave Height	Peak Wave Period	Wave Direction
ARI		H_s	T_p	β
[years]	[-]	[m]	[sec]	[°TN]
5	NE	2.6	10.3	90
	ESE	5.1	12.3	111
	SSE	5.7	13.6	132
20	NE	3.0	11.3	88
	ESE	5.8	12.4	111
	SSE	6.5	13.6	129
100	NE	3.4	11.3	93
	ESE	6.8	13.6	111
	SSE	7.7	14.9	129

To estimate the design waves at the structure, the Goda method for incipient breaking of significant waves was utilised (Goda, Y, 2010). The parameters considered for different ARIs are detailed in **Table 4-12** and include water depth, deep water wave length L_0 based on offshore wave peak periods, and a beach slope of 1:50 (v:h) from the WorleyParsons report (WorleyParsons, 2011).

This approach provided significant wave heights (Hs) for incipient breaking at the toe of the future seawall, considering variable breaker indices, which were then adopted as the design wave height at the structure.

For depth-limited conditions at the structure toe, the EurOtop Manual (EurOtop, 2018) in Section 2.3.2 offers a recommended approach to determining significant wave heights. This method provides the wave height that directly impacts the structure.

Additionally, utilising the methodology presented by Battjes and Groenendijk (Battjes, J, & Groenendijk, 2000) for wave height distributions in the shoaling and breaking zones, values for $H_{10\%}$, $H_{2\%}$, and $H_{1\%}$ were derived. These respective values serve as the design wave heights at the structure for various ARI events.

5 Relevant coastal hazards

5.1 General

The Coastal Management Act 2016 identifies seven coastal hazards:

- beach erosion;
- shoreline recession;
- coastal lake or watercourse entrance instability;
- coastal inundation;
- coastal cliff or slope instability;
- tidal inundation; and
- erosion and inundation of foreshores caused by tidal waters and the action of waves, including
- the interaction of those waters with catchment floodwaters.

Of the above seven coastal hazards, the relevant hazards for consideration of Bronte SLSC are beach erosion, shoreline recession, and coastal inundation.

5.2 Beach erosion and shoreline recession

5.2.1 Beach erosion

Beach erosion refers to the loss of sand from the subaerial beach (that is above the waterline, taken to be approximately Mean Sea Level or AHD) during a coastal storm or closely linked series of coastal storms. The erosion process involves sand being swept seaward off the beach during a storm and then being deposited on the bar near the seaward edge of the surf zone.

5.2.1.1 Wedge Failure Plane Model

The Wedge Failure Plane Model is adopted by NSW Department of Planning and Environment (DPE) as the current understanding for beach and dune erosion and instability on an erodible (sandy) coastline ((Nielsen & Lord, 1992). The progressive description of the erosion and instability process adopted in the model is summarised below (refer **Figure 5-1**):

- Storm waves attack a beach;
- Sand is eroded, the erosion limited by the **Zone of Wave Impact ZWI** (red tone). ZWI taken to be a vertical face. For the purposes of the model, wave scour at the face of ZWI assumed at -1.0m AHD.
- The eroded escarpment dries out after the storm, the factor of safety against gross instability in the **Zone of Slope Adjustment ZSA** (orange tone) reduces to less than 1.0, and it slumps. Back of ZSA sloped at angle of repose of the sand, nominally 34 degrees. The slumped ZSA sand forms a wedge at the base of the eroded face;
- **Zone of Reduced Foundation Capacity ZRFC** (yellow tone) establishes landward of the ZSA. The factor of safety against gross instability in the ZRFC ranges between 1.0 (failure) and 1.5 (stable).
- **Stable Foundation Zone SFZ** (blue tone) is maintained landward of the ZRFC. Here factor of safety against gross instability is greater than 1.5 (stable).

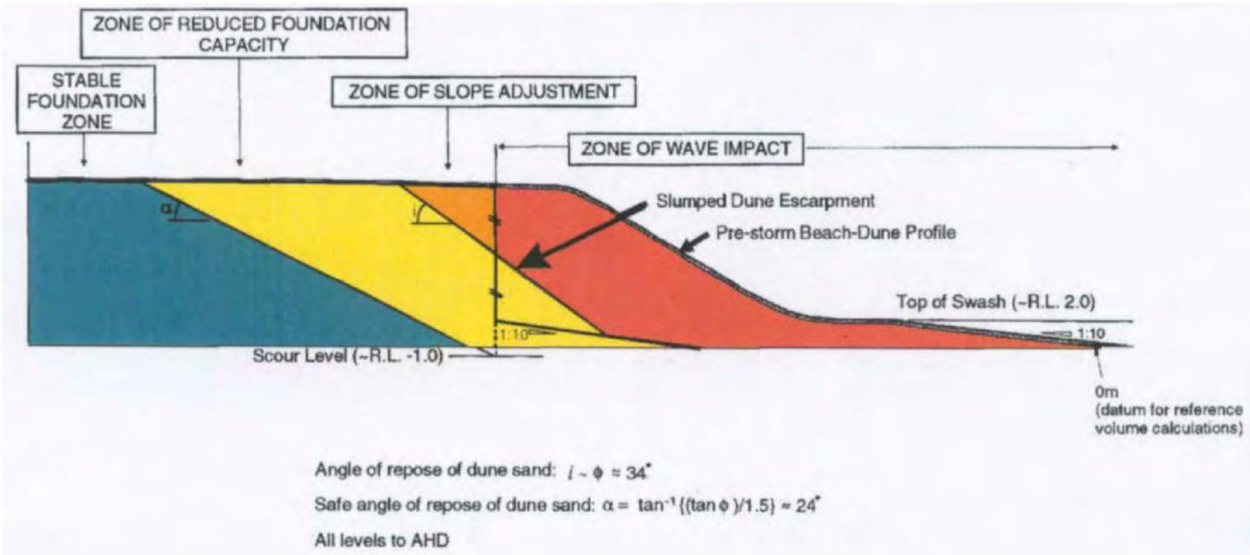


Figure 5-1 Wedge Failure Plane Model after Nielsen et al (1992)

The Wedge Failure Plane would be mapped landward of the erosion envelope, excluding the influence of a seawall.

5.2.1.2 Predicted and measured storm erosion

The design storm erosion demand for mid Bronte Beach for a 100-year ARI storm event is reported as 250m^3/m above AHD in the Waverly Council Coastal Risks and Hazard Vulnerability Study (WorleyParsons, 2011). As no field data existed at the time of this investigation, this value is adopted as a maximum from Nielsen & Lord (1992).

As Bronte Beach was not captured by the NSW Coastal Profile Database, Baird (2016) completed a photogrammetric assessment of available aerial photography for the beach. Ten dates were analysed from 1970 to 2006 with five profiles extracted along the beach for each date as shown in **Figure 5-2**. Profiles L00 and L01 are located approximately in the middle of the beach, fronting the SLSC revedelopment. The terrestrial survey undertaken on June 12, 2016, within one week of the June 2016 storm event, is added to the profiles. The average profile is aslo shown, adopted by Baird as the prestorm profile for their beach erosion modelling. The extracted profiles are shown in **Figure 5-3** and **Figure 5-4**.

Representing a typical beach condition fronting the site, RHDHV have measured the beach volumes above AHD (subaerial) for L01 at selected dates as shown in **Table 5-1**. Profiles have been extrapolated to AHD as required, based on slopes at the lower side of the beach shown by other beach full profiles mapped at L01.

The information in the table tells us that approximately 180m^3/m of sand represents the most beach-full condition in the vicinity of the SLSC. This is noteably less than the 250m^3/m adopted by WorleyParsons in the hazard study. The major erosion event captured in June 2016 probably eroded around 60m^3/m (179 minus 118), or potentially up to a maximum of 90m^3/m if we assume that the portion of the profile below say 2mAHD could reasonably have mimiced the 1976 profile, and accreted in the week post-storm prior to the beach survey. The severity of the June 2016 event for Bronte is discussed by Baird, estimated to range from 10 year ARI to up to 100 year ARI depending on what wave direction is critical for the beach (Baird, 2016). We know that Bronte faces ESE while the June 2016 event was associated with an E to NE offshore wave direction.

Table 5-1 Subaerial beach volumes at L01

Date	Volume above AHD	Comments
[-]	[m^3/m]	[-]
Average	158	Average profile assessed by Baird (2016)
1976	92	Most eroded survey
2006	179	Latest profile in the dataset prior to 2016, and most accreted profile
2016	118	Surveyed within one week of June 2016 storm

Gordon (1987) shows that a 100 year ARI event typically involves beach erosion which is 1.7 to 2 times that of a 10 year ARI event, depending on beach exposure. It follows that a 100 year ARI storm erosion at the site would not be expected to exceed approximately 180m^3/m of subaerial erosion. Thus the seawall is potentially threatened for a 100 year ARI storm, nominally selected as the design event, occurring today. Any long term recession of the beach would increase the exposure of the site (**Section 5.2.2**).

Figure 5-2 Location of photogrammetric profiles

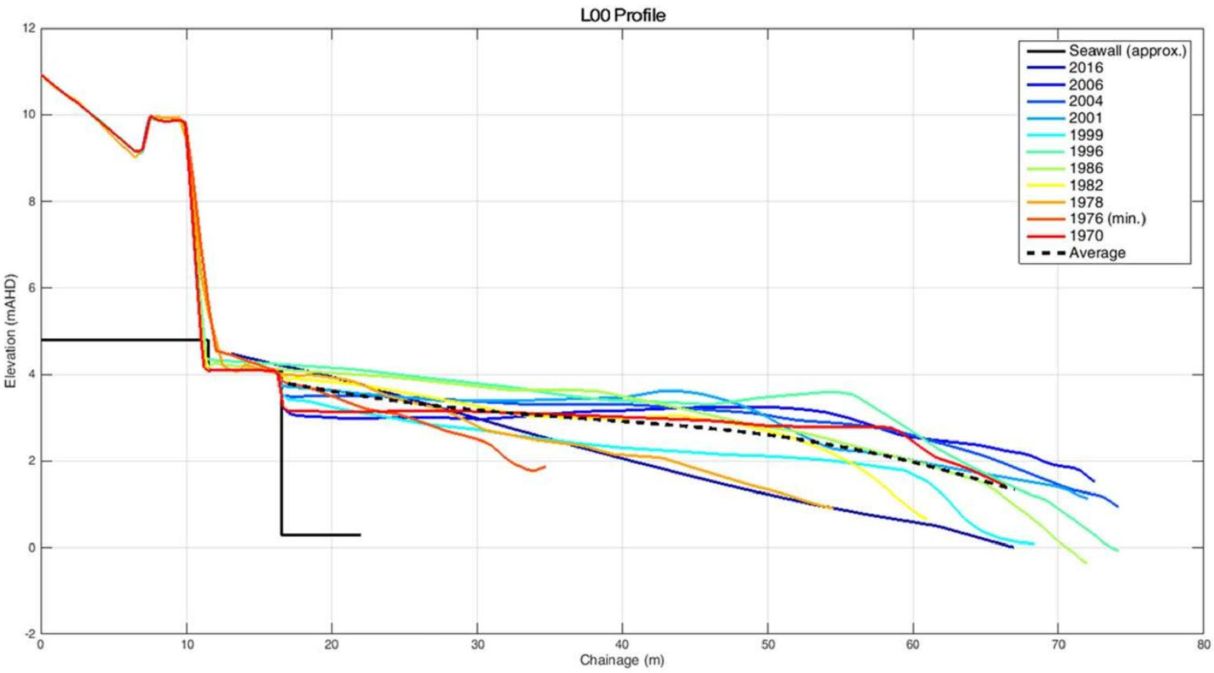


Figure 5-3 Extracted profiles at L00

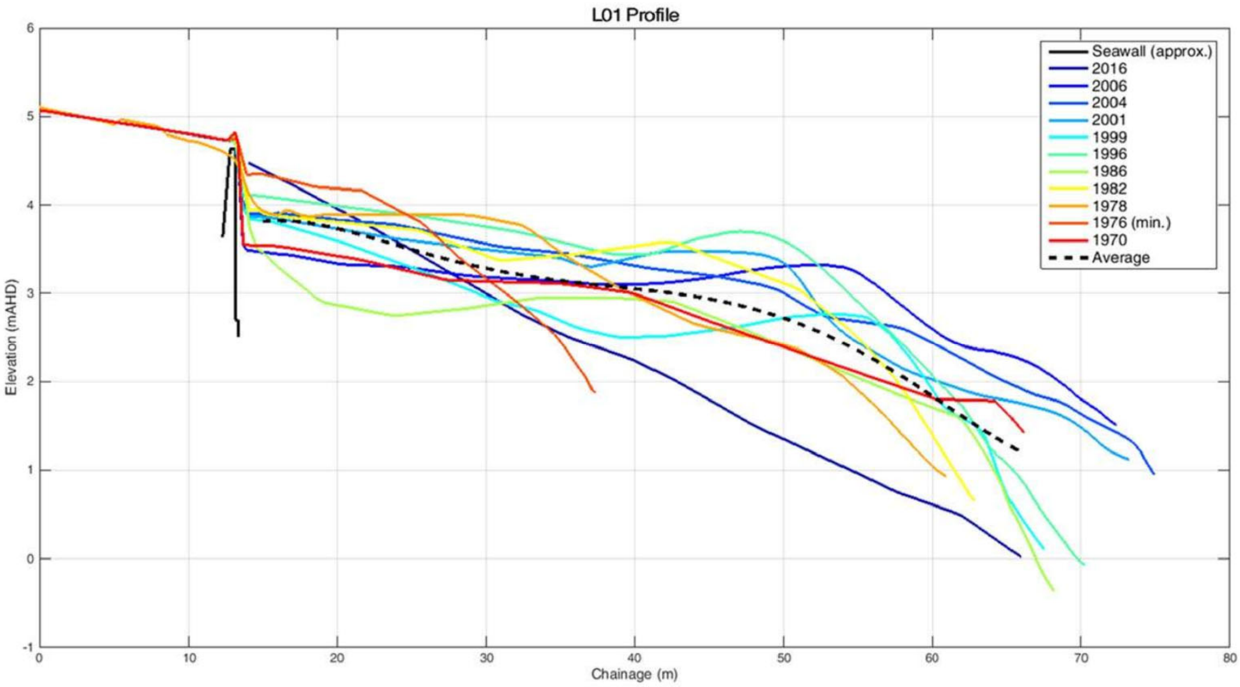


Figure 5-4 Extracted profiles at L01

Baird carried out SBEACH beach erosion modelling for a nominal design event, represented by a 100-year extreme wave event followed by a 20-year ARI storm (Baird, 2016). Back-to-back storms are often modelled in SBEACH recognising that closely linked storms are associated with more severe erosion, and that single event simulations tend to yield erosion results that are lower than expected. The May-June 1974 storms, typically regarded as the design event for the Sydney coastline, were associated with closely linked storms.

SBEACH model results for Profiles L00 and L01 for present day conditions, with and without the design storm from various directions are presented in **Figure 5-5**. These show that waves from the SSE lead to greater erosion than from the ESE and NE, and the enhanced erosion due to a back-to-back 100- and 20-year ARI storm sequence is also demonstrated.

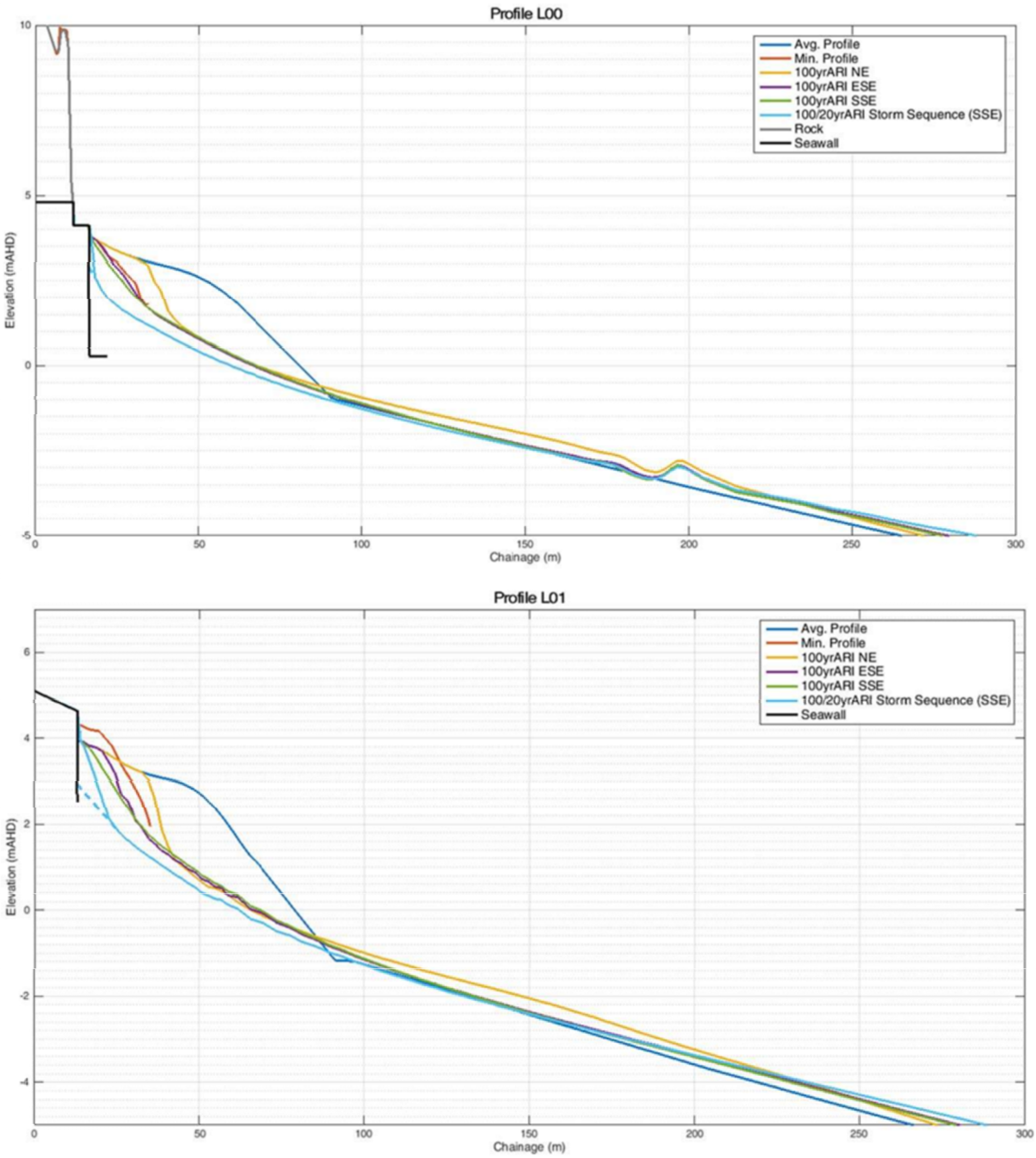


Figure 5-5 SBEACH model results for Profiles L00 and L01 for present day conditions, with and without the design storm from various directions (Baird, 2016)

Design scour levels under 2050 and 2100 climate change scenarios are considered in **Section 5.2.2**.

5.2.2 Shoreline recession

After a storm, the eroded sand deposited on the bar is reworked back to the beach by wave and tidal processes during fairweather. Shoreline recession is the long-term retreat of the shoreline, attributed to incomplete recovery of the beach following beach erosion, combined with any windblown sand transported outside of the beach and dune system.

Based on their beach analysis, Baird suggested that Bronte undergoes large episodic erosional events due to coastal storms, but then recovers and remains relatively stable in intervening periods (Baird, 2016). They observed no recessional trends in the data. However, recession in the future is predicted to a occur as a consequence of climate change. As is normal coastal engineering practice, Baird applied the Bruun Rule to describe beach recession due to sea level rise, reporting existing and future average shoreline positions as shown in **Figure 5-6**. The average beach width in front of the SLSC is predicted to reduce from approximately 70m present day (2016), to 50m in 2050 and slightly more than 20m in 2100. The methodology applied here is appropriate and RHDHV concurs with the beach recession description developed by Baird.

To investigate erosion hazard into the future, Baird ran the 100-year ARI design storm event through their verified SBEACH model for their two sea level rise scenarios. Sequencing with the 20-year ARI storm was omitted in these runs due to what Baird reports is the “highly predictive and uncertain nature of future beach condition estimation.” Erosion from 100-year ARI storms, incident from the SSE, and occurring in 2050 and 2100, are predicted to impinge directly on the seawall at L00 and L01 as shown in **Figure 5-7**.

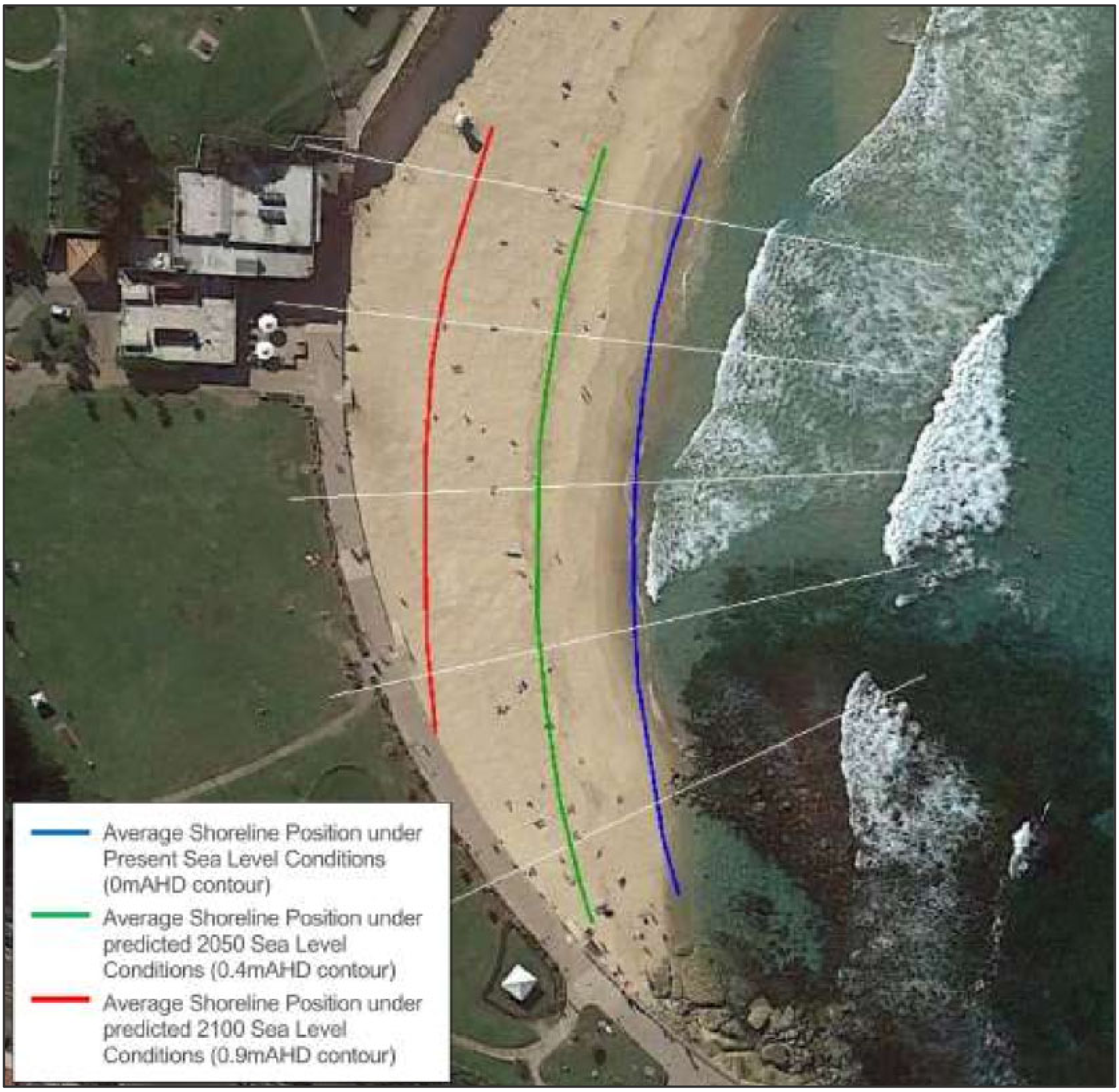
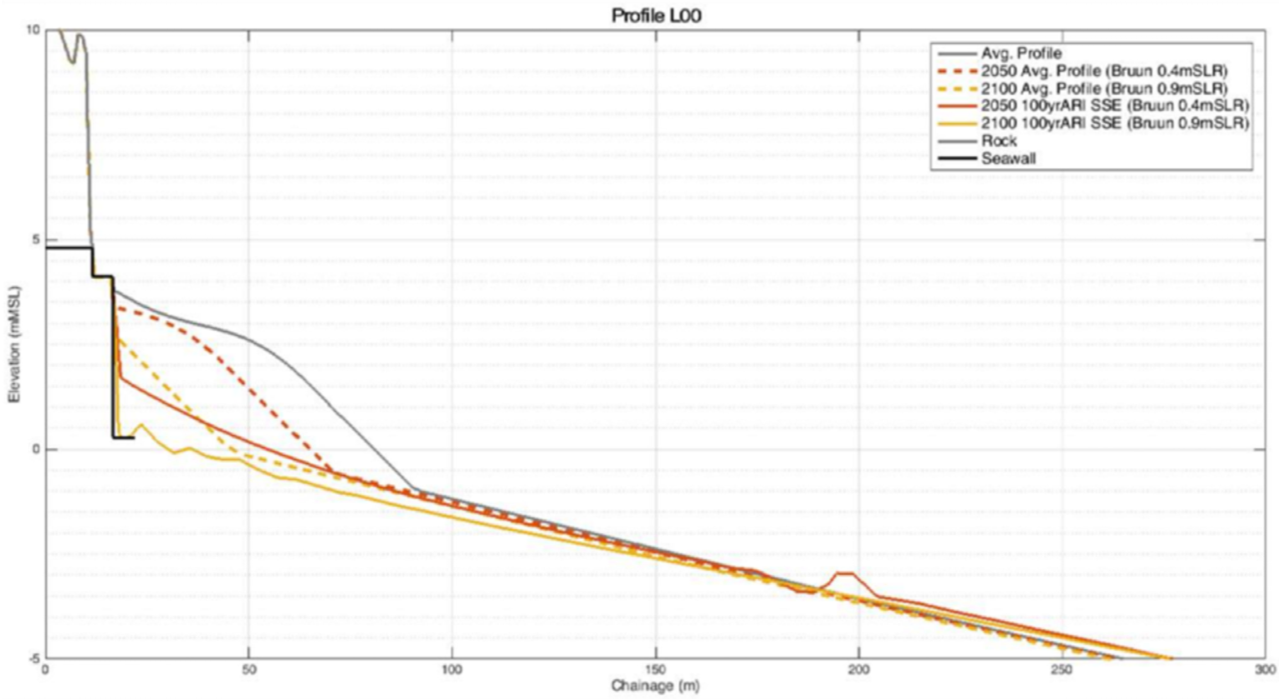


Figure 5-6 Average present day and predicted 2050 and 2100 shoreline position (Baird, 2016)

To investigate erosion hazard into the future, Baird ran the 100-year ARI design storm event through their verified SBEACH model for their two sea level rise scenarios. Sequencing with the 20-year ARI storm was omitted in these runs due to what Baird reports is the "highly predictive and uncertain nature of future beach condition estimation." Erosion from 100-year ARI storms, incident from the SSE, and occurring at 2050 and 2100, are predicted to impinge directly on the seawall at L00 and L01 as shown in **Figure 5-7**.

RHDHV is satisfied that the reduction in predicted sea level reported herein at **Section 4.8.3** compared to that reported by Baird, would not materially change the outcome in respect of future design erosion occurring in 50 to 100 years and impinging directly on the seawall in the vicinity of the SLSC.



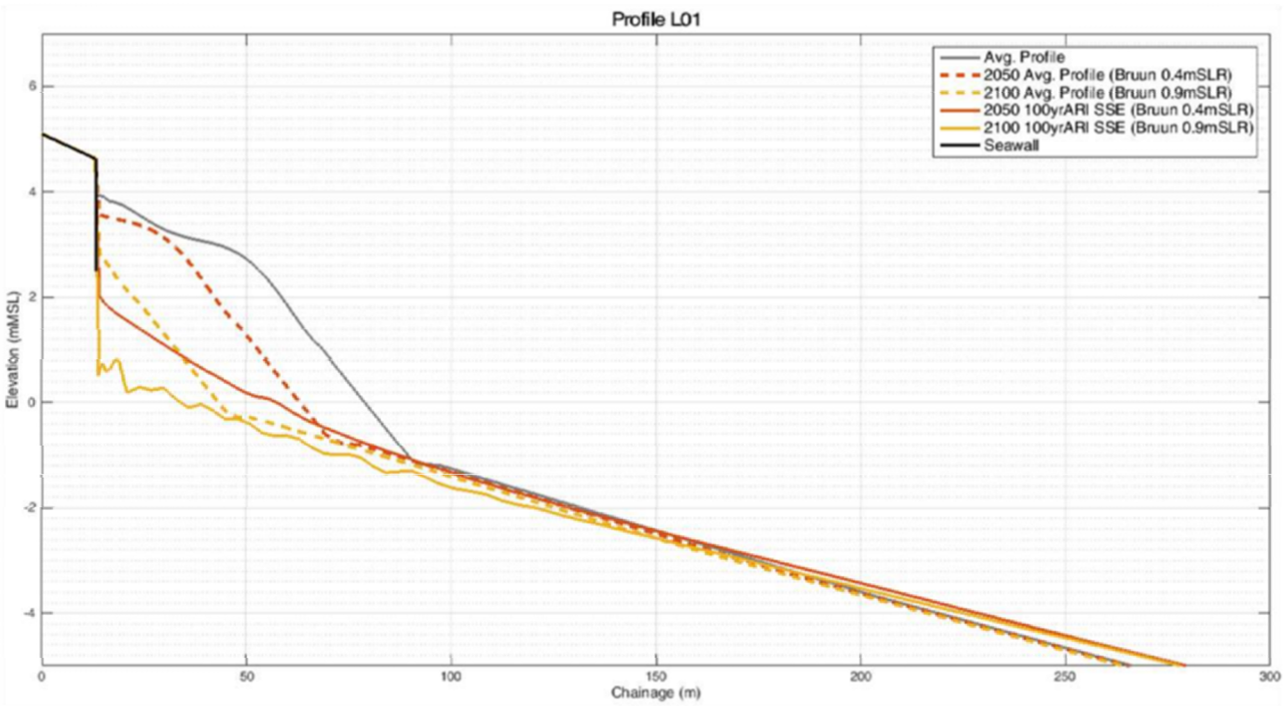


Figure 5-7 SBEACH model results for Profiles L00 and L01 for future 2050 and 2100 climate change scenarios showing average profiles with and without a 100-year ARI design storm directed from the SSE (Baird, 2016)

5.2.3 Beach scour

Baird applies SBEACH to predict design scour levels under present and future climate conditions as summrised for profiles L00 and L01 in **Table 5-2**. Where the base of their modelled erosion does not impinge diectly on the seawall, Baird extrapolates the subaerial slope of the eroded profile to the seawall position to derive the design scour level. This approach is prudent for design as it acknowledges a potential for fluidisation of the back-beach in front of the seawall at the peak of the storm.

Table 5-2 Recommended design scour levels under present climate conditions

Profile	Present day (mAHD)	2050	2100
L00	+2.9	+1.7	+0.35
L01	+2.9	+2.0	+0.75

The analysis above makes no allowance for bedrock which may well be present under the sand above the predicted eroded beach levels. Borehole drilling behind the existing seawall in the immediate vicinity of the SLSC have encountered weathered sandstone at +1.0m AHD approximately 10m behind the seawall in the centre of the site, and at -0.2m AHD approximately 2m behind the seawall slightly further north but still in front of the SLSC building (**Section 4.4**).

The rule of thumb for beach scour at a seawall on the NSW open coast ranges between approximately -1m and -2m AHD. The scour levels predicted at Bronte are considerably higher. The seawall design project would involve additional geotechnical investigations including boreholes, test pits, DCPS and seismic profiling to confirm the bedrock levels and the strength of the rock with depth sufficient to inform the intended piling component for the new seawall design (**Section 4.4**).

5.3 Coastal inundation

The ground floor of the SLSC clubhouse faces potential damage from oceanic water inundation due to wave runup and overtopping, projectile debris, and sand infill during such events (refer to **Section 5.3.2.1**). Projected sea level rise is expected to increase the frequency and depth of these inundation events over time. The risk of damage is significantly mitigated by measures outlined in **Section 5.3.2.4**, aimed to achieve an acceptably low risk of coastal inundation damage throughout the design life (refer to **Section 4.6**).

Physical modelling is a most helpful tool to understand complex coastal processes in the nearshore zone including wave runup and overtopping at seawalls, steps, ramps, and other barriers. This tool can aid in the development of reliable and cost-effective engineering design solutions. Particularly relevant for studying the runup process over coastal structures, 2D physical modelling is proposed by RHDHV during detailed design. This approach aims to enhance the quantification of wave overtopping flows, assess hydraulic loads and potential damage to refine structural designs, and refine features like seawall crest levels and temporary barrier heights to effectively reduce wave overtopping and its risk of damage.

5.3.1 Historical wave overtopping

As per Horton Engineering report (Horton Coastal Engineering , 2023), the Bronte SLSC clubhouse, built in 1974, has withstood multiple oceanic inundation events, including severe storms in 1974 and 2016, without experiencing significant structural damage. Despite causing harm to surrounding areas and impacting landscaping, outdoor furniture and door entries, the clubhouse structure itself has largely remained unscathed. Horton notes a recurring trend of wave overtopping at the southern end of Bronte Beach, resulting in a high-velocity wave runup along the promenade towards the north. In the 2016 storm, this pattern was intensified by northerly flow reaching the clubhouse, in addition to direct wave action.

The East Coast Low storm that occurred between the 5th and the 6th of June 2016 resulted in extreme nearshore waves and severe beach erosion, reaching levels not seen since the coastal storms of the mid-

1970s. The event was unique in that it featured some of the largest wave heights ever recorded off Sydney from north-east to east. Despite the offshore wave height during the event being equivalent to less than an omnidirectional 10-year ARI, the consideration of specific wave direction revealed a much higher ARI for the north-east sector. The storm provided a rare opportunity to observe how the beach responds to large storm wave conditions. ARUP study (ARUP, 2016) incorporates valuable pre- and post-storm beach survey data, along with post-storm observations. **Figure 5-8** illustrates the southern portion of Bronte Beach, showing a notable change in the beach profile before and immediately after the storm.

Figure 5-8 View of southern length of Bronte Beach and seawall showing differences in beach profile before (top image, 1 April 2016) and immediately after (bottom image, 6 June 2016) the East Coast Low storm event (source: *(ARUP, 2016)* (top), *(Baird, 2016)* (bottom)).

Horton Coastal Engineering (Horton Coastal Engineering , 2023) concludes by highlighting that, considering historical events, the existing seawall and promenade lack the necessary crest level to effectively prevent significant wave overtopping during severe storms. Furthermore, the projected rise in sea levels is expected to exacerbate this issue. Consequently, the current promenade is declared unsafe for pedestrians during severe coastal storms, based on Horton's assessment.

5.3.2 Estimation of wave runup and overtopping

The results presented below are based on best-practice desktop calculations. In almost all instances, the use of any of these methods would involve some degree of simplification of the true situation. The further that the structure or design (analysis) conditions depart from the idealised configurations tested to generate the desk top methods and tools discussed, the wider would be the uncertainties. Where the importance is high of the assets being defended, and/ or the uncertainties in using these methods are large, then the design solution may require use of site-specific physical model tests (EurOtop, 2018). Physical modelling is a provisional item within Stage 3 of RHDHV's design investigation (Section 7).

The EurOtop Manual (EurOtop, 2018) provides equations for runup and overtopping calculations on structures such as the those being considered for the Bronte redevelopment project. This method was used to estimate theoretical runup levels and average overtopping rates for a range of design conditions (i.e., 5, 20 and 100-year ARI) and for different eroded states of the beach.

5.3.2.1 Wave runup

The beach crest at Bronte is situated at approximately +4.0m AHD, with variable width along the frontage, reaching a maximum of 45m in front of the SLSC and reducing towards the south to an average of 20m[4]. The nearshore beach slope is approximately 1V:50H, as confirmed by the WorleyParsons and Baird reports [(WorleyParsons, 2011) and (Baird, 2016)].

The calibration case available for wave runup at Bronte is based on videos taken by SLSC during the NSW East Coast Low Event (3 to 7 June 2016) and a recent study done by Bureau of Meteorology, NSW Regional Office, New South Wales Government, Manly Hydraulics Laboratory, and New South Wales Office of Environment and Heritage (Louis, et al., 2016). This storm had the following peak characteristics:

- Storm peak H_{max}=12.0m
- Storm peak H_s= 6.53m
- Storm average T_p=13.5s
- Storm average direction=103° (ESE)
- Maximum water level (excluding wave setup) 1.5 m AHD
- Observed debris lines typically measured between 4 and 6.5 m AHD on most beaches
- The maximum measured run-up levels surveyed for this event reached an elevation of 7.5m AHD at Maroubra

Comparing measured runup and calculated runup using Mase's method (Mase , 1989), the observed debris line aligns with a calculated R_{max} of 6.0m AHD and a calculated $R_{2\%}$ of 5.8m AHD, indicating the appropriateness of Mase's method for estimating wave runup at Bronte Beach.

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[4] The northern half of Bronte Beach is fully protected by high bedrock walls, there is no seawall and wave runup is not an issue because of the elevated ground levels. Also, the available evidence does not highlight a significant erosion problem in the area.

Calculated wave runup values ($R_{2\%}$) for a range of conditions with an accreted beach are shown in **Table 5-3**. $R_{2\%}$ levels are typically used to describe wave runup in coastal engineering and represent the wave runup water level that is exceeded by 2% of incident waves.

These values of wave runup provide estimates of water levels that can be expected to reach the top of the upgraded seawall which is currently proposed to have a maximum crest level of +5.34m AHD near the north-facing steps (steps and bleachers) and gradually decreases to +5.05m AHD and +4.65m AHD at the south limit of the seawall redevelopment.

These calculated wave runup levels exceed the proposed crest during some 20-year ARI events and all cases of 100-year ARI and larger, indicating potential for wave overtopping on the promenade during storm events with these characteristics.

Table 5-3 Wave runup levels and overtopping discharges for accreted beach

Average Recurrence Interval	Planning period	Design High Water Level	Peak significant wave height (1)	Associated peak wave period (2)	Runup 2% (3)
ARI		DHWL	H_s	T_p	$R_{2\%}$
[-]	[-]	[m AHD]	[m]	[s]	[m AHD]
5	Present Day	1.33	5.9	12.0	3.8
	2050	1.50	5.9	12.0	4.0
	2093	1.99	5.9	12.0	4.5
	2100	2.05	5.9	12.0	4.5
20	Present Day	1.41	7.5	12.4	4.4
	2050	1.58	7.5	12.4	4.6
	2093	2.07	7.5	12.4	5.0
	2100	2.13	7.5	12.4	5.1
100	Present Day	1.48	8.2	13.0	4.7
	2050	1.65	8.2	13.0	4.9
	2093	2.14	8.2	13.0	5.4
	2100	2.20	8.2	13.0	5.55

Notes:
1. Peak significant wave heights derived from (Shand T. , et al., 2011)
2. Associated peak wave period inferred from nearshore wave periods (refer to Section 4.9.2)
3. After Mase (1989)

5.3.2.2 Relevant Wave Overtopping Thresholds

The EurOtop Manual (EurOtop, 2018) provides thresholds for wave overtopping for vertical walls (shore protection) including limits for property and people located at the crest or behind the shore protection. Overtopping thresholds of relevance to the proposed structure are outlined in **Table 5-4**.

Table 5-4 Limits for overtopping relevant to the proposed structure (EurOtop, 2018)

Item	Limit State	Return Period	Hazard type and reason	Mean discharge q [l/s per m]	Max volume V_{max} [l per m]	Comment
1	Operational conditions	1-year ARI	Aware pedestrians, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway.	0.1	20-50 at high level or velocity.	Not all these conditions are required, nor should failure of one condition on its own require the use of a more severe limit.
2	Operational conditions	1-year ARI	Damage to equipment set back 5–10m.	0.4	-	These limits relate to overtopping defined at the defence.
3	Ultimate limit state	100-year ARI	Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway.	1-10	500 at low level.	
4	Ultimate limit state	100-year ARI	Building structure elements	1	-	This limit relates to the effective overtopping flow defined at the building.
5	Ultimate limit state	100-year ARI	Damage to grassed or lightly protected promenade or reclamation cover.	50	-	
6	Ultimate limit state	100-year ARI	Damage to paved or armoured promenade behind seawall.	50200	-	

The two main parameters used for wave overtopping thresholds are mean overtopping discharge, q (l/s per m), and maximum overtopping volume V_{max} (l per m). Mean overtopping discharge provides an indication of average conditions over a period. Overtopping discharge is never constant but a dynamic and irregular process, such that the severity of an individual overtopping event is also associated with the wave height and group effect that causes the overtopping. For this reason, maximum overtopping volume provides an additional parameter that is also useful and important in assessing overtopping.

As outlined in Section 3 of EurOtop Manual (EurOtop, 2018) regarding tolerable wave overtopping, most shore protection structures are constructed primarily to limit overtopping volumes and provide adequate design drainage that might otherwise cause flood hazards. Overtopping volumes that can be tolerated would be site specific as the volume of water that can be permitted would depend on the size and use of the receiving area, extent and magnitude of drainage ditches, damage versus inundation curves, return period, wind effects on overtopping processes and rainfall-runoff flows that may coincide with a high wave event.

Other parameters can be relevant to assessing the safety and tolerance of overtopping events and improve design drainage. These may include overtopping velocities and flow depth to categorise flood hazards at return flow paths and inform adequate structure drainage to mitigate these impacts.

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5.3.2.3 Wave overtopping calculations

Overtopping of seawalls is a consequence of the direct impact of waves on the structure, posing a potential threat to freestanding parapets and concrete cappings. Beyond structural concerns, water discharge over the seawall crest presents a hazard to individuals and properties behind it.

Although this phenomenon is often sporadic, perhaps only happening every few waves within a storm (unlike a gently rising still water level caused by tide surge), it still can potentially contribute to localised flooding, but also structural damage and safety issues, if not managed appropriately.

While advancements in empirical estimates of overtopping for coastal structures have been notable over the past decade, the current methods remain primarily suitable for providing order-of-magnitude approximations or for relative comparisons. The EurOtop Manual (EurOtop, 2018) is a state-of-the-art empirical technique, but for precise estimates, site-specific physical modelling is recommended. The Water Research laboratory of the UNSW (WRL) has conducted comparisons between overtopping predictions from the manual and physical models of various coastal structures in wave flumes. Generally, the EurOtop Manual yields reasonable predictions (Mariani, Blacka, Cox, Coghlan, & Carley, 2009).

Quantification of overtopping is articulated in terms of the volume of water discharged over the seawall crest, expressed as L/s per meter length of crest. The estimation of wave overtopping for each structure considers the following factors:

- Structural characteristics of the seawalls, encompassing construction type, crest level, slope, etc
- Design scour levels for the seawalls
- Wave conditions at the structure, specifically wave height and period
- Elevated water conditions, incorporating tides, storm surge, and wave setup.

The current seawall crest level fronting the SLSC is around +4.0m AHD, and inundation due to wave run-up occurs in significant storm events. Recommendations from Horton Coastal Engineering report (Horton Coastal Engineering , 2023) proposed raising the seawall seaward of the northern and southern ends of the clubhouse to 5.8m AHD, including a wave return. However, runup and overtopping calculations to support the proposed seawall crest level are not reported.

The likely coastal flooding at the site has been calculated using the equations of overtopping for vertical walls give in the EurOtop Manual (EurOtop, 2018). Factors considered include structural characteristics, design scour levels, wave conditions, and elevated water conditions. Wave conditions used to carry out the calculation are, as described in **Table 5-5**. The maximum allowable overtopping discharge over the seawall is described in **Table 5-6**.

Table 5-5 Wave conditions used in assessment of overtopping

Case	Average Recurrence Interval	Planning Period	Water Level	Spectral Wave Height	Peak Period	Angle of attack relative to normal[2]
	ARI		DSWL	H_{m0}	T_p	β
[-]	[years]	[-]	[m AHD]	[m]	[s]	[°TN]
1	5	Present day	2.26	N/A	N/A	N/A
2	5	2093	2.92	1.23	13.60	132
3	5	2100	2.98	2.05	13.60	132
4	100	Present day	2.85	N/A	N/A	N/A
5	100	2093	3.34	1.54	14.90	129
6	100[1]	2093	3.34	3.06	14.90	129
7	100	2100	3.40	2.35	14.90	129

Note:

1. *This additional condition considered a highly eroded seabed (-1 m AHD)*
2. *Under oblique wave attack, significant spatial variability of overtopping discharge along a seawall would be observed in the field and measured in physical model studies. At this stage we would consider shore-normal wave attack (obliquity β = 0°) for estimating overtopping rates.*

Table 5-6 Overtopping rates

Case	Average Recurrence Interval	Planning Period	Water Level	Spectral Wave Height at toe	Spectral Wave Period	Overtopping back of crest
	ARI		DSWL	H_{m0}	$T_{m-1,0}$	q
[-]	[years]	[-]	[m AHD]	[m]	[s]	[l/s/m]
1	5	Present day	2.26	N/A	N/A	0.0
2	5	2093	2.92	1.23	11.33	0.1
3	5	2100	2.98	2.05	11.33	8.9
4	100	Present day	2.85	N/A	N/A	0.0
5	100	2093	3.34	1.54	12.42	2.4
6	100[1]	2093	3.34	3.06	12.42	122.5
7	100	2100	3.40	2.35	12.42	39.7

Notes:

- *The spectral mean wave period was derived using the methodology of (Hofland, Chen, & Altomare, (2017) for long-crested waves. In offshore conditions, the spectral mean wave period is approximately equal to the spectral peak wave period (13 s for the 100-year ARI event, refer to Section 0) divided by 1.1 (equal to 11.8s). However, the spectral mean wave period may change considerably if waves are breaking on a very shallow foreshore, caused by the presence of low-frequency or infra-gravity waves.*
- *the structure is considered smooth / impermeable structure is*

The calculation methods employed have inherent limitations in accuracy, providing only 'order of magnitude' estimates. The key findings based on these estimates are as follows:

- For the planning period in 2093 under a 5-year ARI event, there is a high likelihood of wave overtopping being a hazard to pedestrians at the seawall crest
- Wave overtopping during Case 3 events for the future seawall could pose a hazard for people in proximity to the seawall crests
- There is a relatively low likelihood of wave overtopping causing structural damage behind the seawall under 5-year ARI events, but a higher theoretical risk for 20-year ARI or more extreme events
- Overtopping in Cases 5 and 6 could potentially cause structural damage, especially for the SLSC building.

Additionally:

- Wave overtopping discharge is likely to increase from the northern end to the southern end during extreme storm events, presenting greater hazards at the southern end due to lower seawall crest and seabed levels.
- Overtopping would be more pronounced at the southern end of the seawall during the same extreme storm event, originating from the NE direction.
- Wave overtopping could increase in frequency and magnitude under projected future sea level rise scenarios.

The spatial distribution of overtopped discharge may be of interest in determining zones affected by direct wave overtopping hazard (to people, vehicles, buildings close behind the structure crest, or to elements of the structure itself). Under green water (non-impulsive) conditions, the distribution of overtopped water would depend principally on the form of the area immediately landward of the structures crest (slopes, drainage, obstructions etc). Under violent (impulsive) overtopping conditions, consideration would be given to where the airborne overtopping jet comes back to the level of the pavement behind the crest. While this is dependent strongly on local wind conditions, the EuroTop Manual provides guidance on the likely landward distribution of overtopped flows as a proportion of wavelength (EurOtop, 2018).

In further developing the Stage 2 seawall design, further attention would be given to the design of wave return walls on top of the raised seawall seaward of the clubhouse up to the previously proposed elevation of +5.8m AHD. It can be noted that the effectiveness of wave return walls depends on the incident wave and back-beach/ bed conditions.

5.3.2.4 Wave overtopping mitigation measures

When facing the risk of wave overtopping or excessive wave forces during severe storms, the design of the SLSC and seawall can incorporate a range of coastal engineering features to mitigate potential damage. Effective mitigation methods could include a combination of the following:

- Reduce the design life period for the seawall, i.e., reduce the period for which a structure or a structural member is intended to remain fit for use for its designed purpose with maintenance.
- Raising the ground floor and seawall:
    - elevating the ground floor of the clubhouse to 6.1m AHD.
    - Raising the seawall seaward of the northern and southern ends to 5.8m AHD

- Wave return wall:
  - installation of a wider wave return wall.
- Elevated wave return wall:
  - Installing the wave return wall at an elevated position
  - Raising the seawall and adding a wave return would also reduce the risk of windblown and wave-transported sand and debris reaching the promenade and entering the clubhouse.
- Parapet or additional wave return wall:
  - introducing a parapet or an extra wave return wall tailored to future sea level rise thresholds or specific areas, such as the frontage of the old SLSC building.
- Structural elements:
  - creating ramps and steps facing alongshore.
  - designing elevated specific rooms like the Lifeguards Room and First Aid Room.
- Wave barriers and circulation area:
  - employing wave barriers manually deployed during storms in circulation areas.
  - Creating a secure circulation area with a permanent gate to control wave action
- Courtyard and kiosk design:
  - designing wave-resistant courtyard walls to reduce wave overtopping.
  - ensuring wave barriers for the kiosk and its store during storm events.
- Layout planning:
  - The proposed layout directs overtopping away from the clubhouse, enhancing coastal resilience.
  - The clubhouse, ramps, and steps collectively contribute to its structural integrity.
- Sand level maintenance:
  - It is recommended to keep the beach sand level below a specified reference point (e.g., 4m AHD) to prevent sand build-up, acting as a ramp for overtopping during storms.
  - The Horton Coastal Engineering report (Horton Coastal Engineering , 2023) suggests that it may be prudent for the Council to maintain the sand level on the beach below a certain reference level marked on the seawall and steps, such as 4m AHD in the future. This precaution is recommended because higher wave return walls would be more effective at reducing wave overtopping if they are located a greater distance above the beach. If sand were to reach the seawall crest, it could potentially provide a ramp for waves to overtop the seawall during storms if erosion does not lower sand levels.
- Flood-resistant materials:
  - Utilising flood-resistant materials like concrete and tiles to increase resilience
- Glazing impact considerations:
  - Addressing the impact of waves on glazing, possibly incorporating toughened or laminated glass

- Elevated fixtures and emergency preparedness:
    - elevating vulnerable electrical fittings and outlets.
    - Storing items susceptible to inundation at suitable heights
    - Developing and adopting an emergency action plan, including the installation of temporary barriers during severe storms
- Relocation strategies:
    - Incorporating provisions for relocating items before forecasted storms as part of the club's Emergency Action Plan.

In the short term, swift management measures can be implemented, including:

- Temporary flood barriers:
- Swift installation in response to forecasted severe events
- Interior management of the SLSC Building
- Design considerations for the electrical system and immediate response plans for forecasted events.

Further calculations or physical modelling would be required to precisely quantify the effectiveness of each proposed option.

5.3.3 Wave loads due to overtopping

The estimation of wave loads on the Bronte SLSC building, resulting from direct wave impact during events where the seawall is partially submerged due to highly elevated water levels, employed the method proposed by Goda and Tanimoto. This method is recommended by the US Army Corps of Engineers Coastal Engineering Manual (U.S. Army Corps of Engineers, 2002) for impulsive wave loading.

The assessment of wave loads on the Bronte SLSC considered the simplification that the SLSC front wall aligns with the crest of the proposed concrete seawall. This assumption was made due to the unavailability of desktop techniques that allow for the consideration of the building's offset from the edge of the coastal protection structure. RHDHV acknowledges that this methodology is conservative, and wave loads estimation will be refined in next stages of the project.

It's important to note that existing desktop techniques do not encompass the potential reduction associated with the wave return wall on the impact of waves hitting the Bronte SLSC building.

The calculated loads on the Bronte SLSC due to direct wave impact are presented in **Table 5-7**.

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Table 5-7 Loads on Bronte SLSC front wall caused by direct impact wave

Case	Average Recurrence Interval	Planning Period	Water Level	Spectral Wave Height at toe	Spectral Wave Period	Induced Horizontal Load	Hydrostatic Load
	ARI		DSWL	H_{m0}	$T_{m-1,0}$		
[-]	[years]	[-]	[m AHD]	[m]	[s]	[kN/m]	[kN/m]
1	5	Present day	2.26	N/A	N/A	N/A	N/A
2	5	2093	2.92	1.23	11.33	28.9	9.3
3	5	2100	2.98	2.05	11.33	69.1	14.8
4	100	Present day	2.85	N/A	N/A	N/A	N/A
5	100	2093	3.34	1.54	12.42	45.8	11.5
6	100(1)	2093	3.34	3.06	12.42	130.0	21.8
7	100	2100	3.40	2.35	12.42	90.5	16.9

6 Confirmation of seawall arrangement and structural intent

The design details of the current concrete seawall structure are unknown, as no existing design or construction drawings are available. Constructed between 1914 and 1917, the seawall has significantly exceeded its intended design life. Notably deteriorating, it poses a risk of failure during severe coastal storms, inadequately mitigating wave overtopping volumes and threatening infrastructure and pedestrians landward of the seawall, highlighted by Horton Engineering (Horton Coastal Engineering , 2023) and ARUP (ARUP, 2016).

Given the proposed upgrades for the SLSC building and beach access, constructing a new seawall clearly presents as the most viable solution. The constrained space at Bronte rules out the feasibility of rock revetments, making a new concrete seawall the only practical choice. RHDV's design proposal involves constructing a new seawall structure around the outer perimeter of planned access elements, including the promenade, ramps, bleachers, and steps.

The recommended replacement seawall incorporates a secant pile wall design. This involves alternating small diameter reinforced and larger diameter unreinforced concrete piles, overlapped in their plan position, acting as a barrier to coastal erosion and soil migration. This design would acknowledge the future seawall's dual function as a coastal protection and foreshore retaining structure. The envisaged components include a secant perimeter wall and landward freestanding piles, reinforced concrete capping beam(s) and concrete slab. The seawall would present externally as a vertical sand-coloured concrete wall. The proposed seawall arrangement and structural intent is shown in **Figure 6-1**.

While approximately twice the price of a rock wall, the secant pile wall offers the advantage of occupying a substantially narrower footprint, i.e., in the order of 1m compared to 5 to 8m depending on rock wall height. However, potential challenges include the sensitivity of piled structures to toe-level conditions and their reflective nature, leading to increased scour in front of the wall. The impact of scour on the seawall structure becomes academic if the bedrock is relatively elevated.

The proposed seawall aligns with the reconstruction plan for the SLSC clubhouse and is expected to fully encapsulate the filling behind it. TTW expect to need to remove a segment of the existing seawall in front of the SLSC to facilitate structural design work to support the promenade and front of the SLSC. The timing of the various works would need to consider duration of exposure and risk, however the expectation at this time would be that the new seawall would be constructed before the part-removal of the existing one, ensuring continuous protection during the construction process.

The detailed design of the future seawall would involve a comprehensive integration of coastal, structural, and geotechnical engineering considerations. Factors such as subsurface conditions, beach dynamics, and structural stability would be closely considered. The inclusion of a wave return shape at the crest, achieved through angling the seaward face, aims to mitigate wave overtopping during severe storms, would be considered in the engineering design stages.

Future considerations and recommendations:

- Acknowledging the future seawall's dual function as a coastal protection and retaining structure
- Coastal protection works should accommodate potential beach fluctuations and shoreline recession over the projected 70-year lifespan
- Particular attention to the toes of structures is crucial (seawall, ramp, steps), considering potential impacts on beach fluctuations and climate change-induced shoreline recession

The envisaged seawall is expected to include the following components:

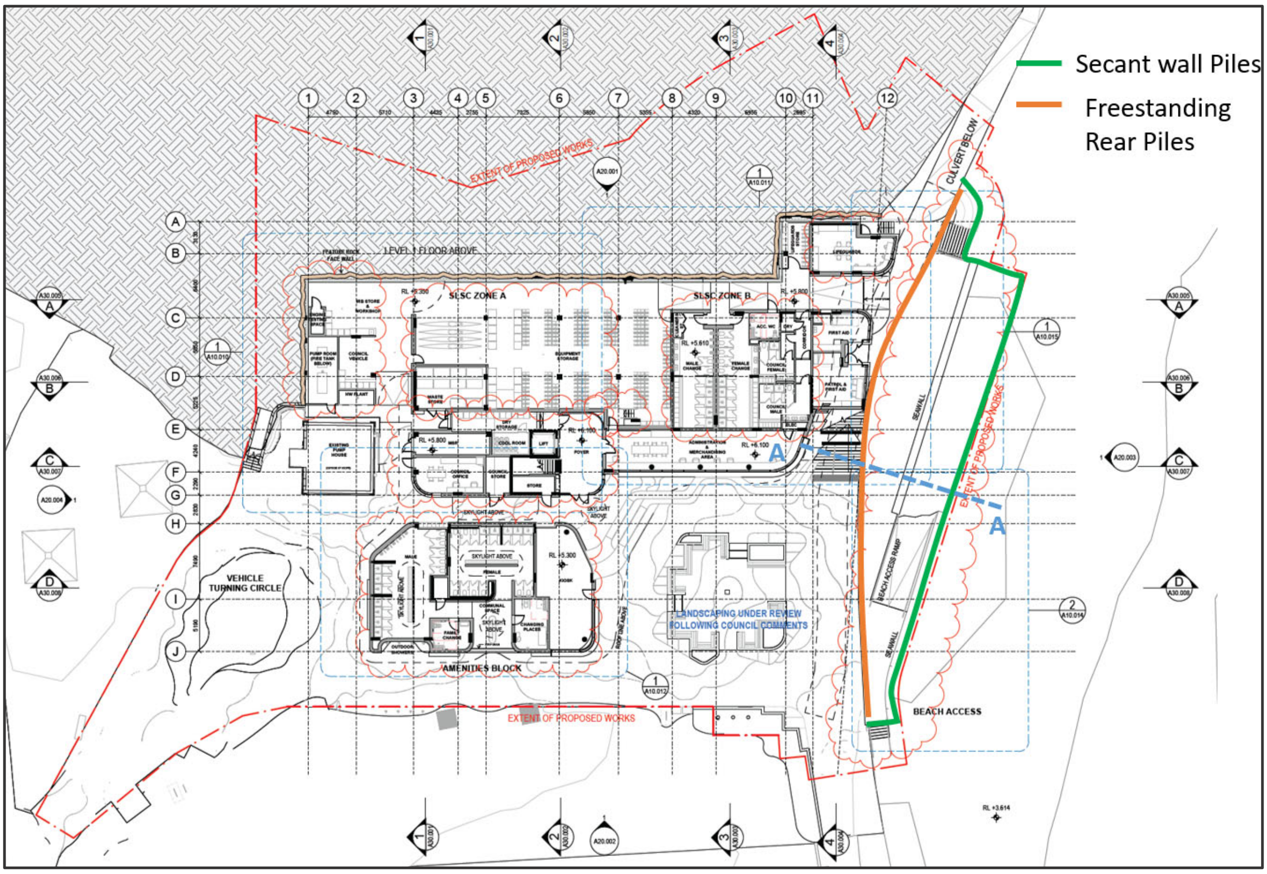
- **External perimeter secant piled wall and landward freestanding piles**
  - The seawall's external perimeter is envisioned as a secant wall, utilizing a combination of small-diameter reinforced ("hard") and larger-diameter unreinforced ("soft") concrete piles.
  - The "hard" piles are bored to overlap into the unreinforced "soft" pile sections, forming a cohesive barrier against soil migration through the wall.
  - Proposed ramps and steps are planned to comprise reinforced concrete slabs/ beams/ upstand walls, supported at their outer edge using the secant pile walls, supported internally on discrete piles if deemed necessary.
- **Reinforced concrete capping beam**
  - A reinforced concrete capping beam would be designed to connect to the tops of the secant piles, providing structural integrity to the seawall.
- **Vertical sand-coloured concrete wall**
  - The seawall includes a vertical sand-coloured concrete wall extending above (and integral with) the capping beam, offering enhanced coastal protection.
- **Weepholes for groundwater management**
  - If necessary, weepholes are incorporated through the seawall to mitigate the risk of groundwater build-up on the landward side.
  - These weepholes would be designed with geotextile socks to prevent soil migration through them.
- **Stability considerations**
  - Anchors tying back the seawall to aid in stability may be necessary, but rather than buried soil anchors or anchors and deadmen, these could utilise the slab on ground and freestanding piles as the anchoring system.
  - Geotechnical constraints, including variable foundation conditions, would be considered in the development planning.
- **Foundation design and bedrock considerations**
  - Depending on the elevation of the bedrock, secant piles may not be required in some areas where bedrock is close to the surface. If this is the case a deep beam founded directly on rock could suffice as the seawall. Ramps and steps would be integrated into this.
- **Connection with existing stormwater and existing seawall**
  - The new seawall would connect into the old seawall at the southern and northern ends of the structure. Also at the northern end the new seawall would connect into the existing stormwater box culvert. The interaction of the new and existing structures would be considered in the design. The structures would need to be tied together to seal between the two to ensure when the beach is in a scoured condition, no loss of retained sand behind the new seawall structure occurs.

A preliminary assessment of wave loading indicates that for a 100-year ARI storm occurring at 2073, at the end of the design life of the seawall, the maximum horizontal wave load at the face of the seawall, would be approximately 130kN/m[5], with the line of action just below the waterline. The consequence of this loading for the design of the seawall is minor in that the space under the promenade, immediately

[5] *Based on Goda formula for irregular waves as set out in CEM. Assumes scour to -1m AHD and breaker coefficient of 0.78.*

behind the seawall, would be filled with suitable material (e.g., imported sand) and compacted. Wave runup at the face of the seawall and overtopping flows would load any wave deflector located at the crest of the seawall. Wave loads at the deflector would be estimated as part of the further Stage 2 design work (completion of this report) and refined through measurements as part of the Stage 3 physical modelling task.

RHDHV recommends a proactive approach to coastal protection, ensuring safety and resilience without relying on the aging existing seawall, which is beyond its design life, has a founding depth too high and of unknown structural design. The proposed new seawall, aligned with SLSC upgrades, is envisioned as an environmentally responsible and cost-effective solution for long-term coastal resilience at the site.



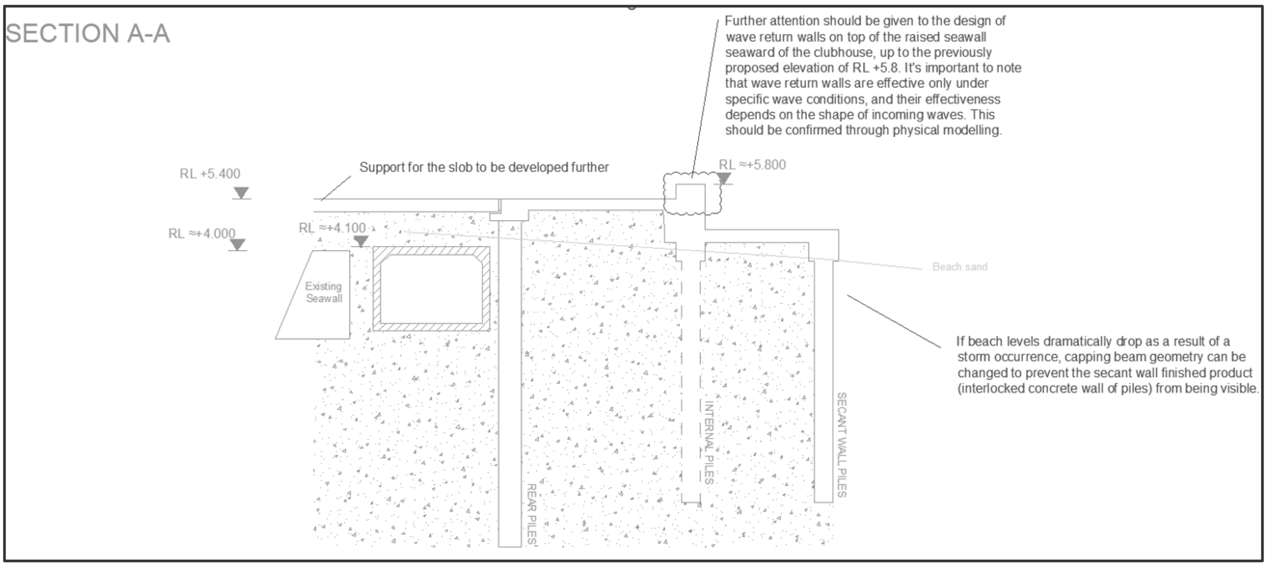


Figure 6-1 Proposed seawall and arrangement and structural intent

7 Physical modelling

Physical modelling plays a valuable role in comprehending intricate coastal processes within the nearshore zone, offering dependable and cost-effective solutions for engineering design. Its significance is particularly notable in refining and optimising coastal structure designs, especially when dealing with complex configurations and site conditions. It is common for physical models to yield optimisations on desk-top evaluations, such as reduced wall crest levels and reconfigured deflectors to achieve overtopping thresholds, and reduced reinforced concrete member sizes in accordance with measured wave loads.

Coastal hydraulic physical modelling is proposed to enhance the quantification of wave overtopping flows, assess hydraulic loads, potential damage, and user safety. The work would involve 2D modelling, incorporating coastal profiles and boundary conditions developed for Bronte. Measurements would focus on overtopping flows (L/s per m), maximum overtopping volumes (L/m), horizontal wave forces on the walls (kN/m), and uplift forces for deflector units (kN/m). Two available flumes, WRL's 1m and 3m wide, are suitable for the study, with the latter potentially offering flexibility for accommodating alongshore changes in structure profile (referred to as "quasi-3D" detail) if this considered helpful.

The chosen model scale would range between 15 and 25, ensuring accuracy in quantifying wave overtopping to 1L/s/m precision, sufficient for the study's objectives. Measurements would likely be conducted up to 50L/s/m, depending on the applied sea level rise and storm conditions. With the load modelling component, the study would aim to resolve the loads for ultimate and serviceability-limit state design.

It is expected physical modelling would be a condition of development consent, to be undertaken to inform the detailed design. Physical modelling is proposed as a Stage 3 investigation task for the seawall design.

8 Coastal assessment

This section sets out a review of the proposal in relation to the following:

- Coastal Management Act 2016;
- State Environmental Planning Policy (Resilience and Hazards) 2021;
- Waverly Local Environmental Plan 2012; and
- Waverly Development Control Plan 2022.

8.1 Coastal Management Act 2016

The relevant section of the Coastal Management Act 2016 is Section 27 within Part 5 Miscellaneous. This Section is reproduced below followed by comments and assessment in Table 8-1.

27 Granting of development consent relating to coastal protection works

(1) Development consent must not be granted under the Environmental Planning and Assessment Act 1979 to development for the purpose of coastal protection works, unless the consent authority is satisfied that—

(a) the works would not, over the life of the works—

(i) unreasonably limit or be likely to unreasonably limit public access to or the use of a beach or headland, or
(ii) pose or be likely to pose a threat to public safety, and

(b) satisfactory arrangements have been made (by conditions imposed on the consent) for the following for the life of the works—

(i) the restoration of a beach, or land adjacent to the beach, if any increased erosion of the beach or adjacent land is caused by the presence of the works,
(ii) the maintenance of the works.

(2) The arrangements referred to in subsection (1) (b) are to secure adequate funding for the carrying out of any such restoration and maintenance, including by either or both of the following—

(a) by legally binding obligations (including by way of financial assurance or bond) of all or any of the following—

(i) the owner or owners from time to time of the land protected by the works,
(ii) if the coastal protection works are constructed by or on behalf of landowners or by landowners jointly with a council or public authority—the council or public authority,

Note. *The Environmental Planning and Assessment Act 1979, section 4.17(6) provides that a development consent may be granted subject to a condition, or a consent authority may enter into an agreement with an applicant, that the applicant must provide security for the payment of the cost of making good any damage caused to any property of the consent authority as a consequence of the doing of anything to which the consent relates.*

(b) *by payment to the relevant council of an annual charge for coastal protection services (within the meaning of the Local Government Act 1993).*

(3) *The funding obligations referred to in subsection (2) (a) are to include the percentage share of the total funding of each landowner, council or public authority concerned.*

Comments and assessment in relation to the Coastal Management Act 2016 would be made following the completion of the Stage 2 seawall design including wave return walls.

8.2 State Environmental Planning Policy (Resilience and Hazards) 2021

8.2.1 General

The relevant part of the State Environmental Planning Policy (Resilience and Hazards) 2021 is Part 2.2 Development controls for coastal management areas. Within this Part there are four relevant Divisions as follows:

- Division 2 Coastal vulnerability area
- Division 3 Coastal environment area
- Division 4 Coastal use area
- Division 5 General

The following sections consider each of these Divisions in turn.

8.2.2 Division 2 Coastal vulnerability area

As yet no Coastal Vulnerability Area Map has been prepared and therefore no coastal vulnerability area has been identified. On the one hand it could be considered that due to the absence of a Map the matter of development within a coastal vulnerability area does not apply. However, it is clear that the proposed works would be located within a coastal vulnerability area once mapped, hence consideration is given to this matter below. The relevant Clause 2.9 is reproduced followed by comments and assessment in Table 6-2.

2.9 Development on land within the coastal vulnerability area

Development consent must not be granted to development on land that is within the area identified as “coastal vulnerability area” on the Coastal Vulnerability Area Map unless the consent authority is satisfied that—

(a) if the proposed development comprises the erection of a building or works—the building or works are engineered to withstand current and projected coastal hazards for the design life of the building or works, and

(b) the proposed development—

(i) is not likely to alter coastal processes to the detriment of the natural environment or other land, and

(ii) is not likely to reduce the public amenity, access to and use of any beach, foreshore, rock platform or headland adjacent to the proposed development, and

(iii) incorporates appropriate measures to manage risk to life and public safety from coastal hazards, and

(c) measures are in place to ensure that there are appropriate responses to, and management of, anticipated coastal processes and current and future coastal hazards.

Comments and assessment in relation to the Coastal Vulnerability Area of SEPP (Resilience and Hazards) 2021 would be made following the completion of the Stage 2 seawall design including wave return walls.

8.2.3 Division 3 Coastal environment area

2.10 Development on land within the coastal environment area

(1) Development consent must not be granted to development on land that is within the coastal environment area unless the consent authority has considered whether the proposed development is likely to cause an adverse impact on the following—

(a) the integrity and resilience of the biophysical, hydrological (surface and groundwater) and ecological environment,
(b) coastal environmental values and natural coastal processes,
(c) the water quality of the marine estate (within the meaning of the Marine Estate Management Act 2014), in particular, the cumulative impacts of the proposed development on any of the sensitive coastal lakes identified in Schedule 1,
(d) marine vegetation, native vegetation and fauna and their habitats, undeveloped headlands and rock platforms,
(e) existing public open space and safe access to and along the foreshore, beach, headland or rock platform for members of the public, including persons with a disability,
(f) Aboriginal cultural heritage, practices and places,
(g) the use of the surf zone.

(2) Development consent must not be granted to development on land to which this section applies unless the consent authority is satisfied that—

(a) the development is designed, sited and would be managed to avoid an adverse impact referred to in subsection (1), or
(b) if that impact cannot be reasonably avoided—the development is designed, sited and would be managed to minimise that impact, or
(c) if that impact cannot be minimised—the development would be managed to mitigate that impact.

(3) This section does not apply to land within the Foreshores and Waterways Area within the meaning of Sydney Regional Environmental Plan (Sydney Harbour Catchment) 2005.

Comments and assessment in relation to the Coastal Environment Area of SEPP (Resilience and Hazards) 2021 would be made following the completion of the Stage 2 seawall design including wave return walls.

8.2.4 Division 4 Coastal use area

The relevant clause is reproduced below followed by comments and assessment in Table 6-4.

2.10 Development on land within the coastal environment area

(1) Development consent must not be granted to development on land that is within the coastal environment area unless the consent authority has considered whether the proposed development is likely to cause an adverse impact on the following—

(a) the integrity and resilience of the biophysical, hydrological (surface and groundwater) and ecological environment,
(b) coastal environmental values and natural coastal processes,
(c) the water quality of the marine estate (within the meaning of the Marine Estate Management Act 2014), in particular, the cumulative impacts of the proposed development on any of the sensitive coastal lakes identified in Schedule 1,
(d) marine vegetation, native vegetation and fauna and their habitats, undeveloped headlands

2.11 Development on land within the coastal use area

(1) Development consent must not be granted to development on land that is within the coastal use area unless the consent authority—

(a) has considered whether the proposed development is likely to cause an adverse impact on the following—

(i) existing, safe access to and along the foreshore, beach, headland or rock platform for members of the public, including persons with a disability,
(ii) overshadowing, wind funnelling and the loss of views from public places to foreshores,
(iii) the visual amenity and scenic qualities of the coast, including coastal headlands,
(iv) Aboriginal cultural heritage, practices and places,
(v) cultural and built environment heritage, and

(b) is satisfied that—

(i) the development is designed, sited and would be managed to avoid an adverse impact referred to in paragraph (a), or
(ii) if that impact cannot be reasonably avoided—the development is designed, sited and would be managed to minimise that impact, or
(iii) if that impact cannot be minimised—the development would be managed to mitigate that impact, and

(c) has taken into account the surrounding coastal and built environment, and the bulk, scale and size of the proposed development.

(2) This section does not apply to land within the Foreshores and Waterways Area within the meaning of Sydney Regional Environmental Plan (Sydney Harbour Catchment) 2005.

Comments and assessment in relation to the Coastal Use Area of SEPP (Resilience and Hazards) 2021 would be made following the completion of the Stage 2 seawall design including wave return walls.

8.2.5 Division 5 General

The relevant clause is reproduced below followed by comments and assessment in .

2.12 Development in coastal zone generally—development not to increase risk of coastal hazards

Development consent must not be granted to development on land within the coastal zone unless the consent authority is satisfied that the proposed development is not likely to cause increased risk of coastal hazards on that land or other land.

Comments and assessment in relation to Division 5 General of SEPP (Resilience and Hazards) 2021 would be made following the completion of the Stage 2 seawall design including wave return walls.

8.3 Waverly Local Environmental Plan 2012

To be addressed following Stage 2 seawall design including wave return walls.

8.4 Waverly Development Control Plan 2022

To be addressed following Stage 2 seawall design including wave return walls.

8.5 Waverley Council coastal risk management policy

To be addressed following Stage 2 seawall design including wave return walls.

9 Peer review liaison

The Council has initiated a peer review of the Coastal Report prepared by RHDHV. This review is entrusted to the UNSW Water Research Laboratory (WRL), which would critically assess the report and offer advice to optimise the design or propose modifications as needed. The primary focus is on reviewing the concept design being developed by RHDHV. The ultimate objective is to obtain comments and recommendations from WRL that would facilitate an agreement on the design among all pertinent stakeholders involved in the project.

An initial meeting with the Peer Reviewer James Carley (JC) was undertaken on November 13, 2023. This meeting was also attended by Gary Blumberg (GPB), Greg Britton (GWB) and Joao Gonsalves (JG) from RHDHV, James Morgan (JM) and Sven Ollmann (SO) from W&M, and Robert Sabato (RS) from Waverly Council. Key notes prepared from the meeting are set out below in **Table 9-1**.

A copy of the presentation discussed at the initial peer review meeting is attached in **Appendix A.1**. a photomontage of use from the beach looking back at the seawall and beach access structures are reproduced below in **Figure 9-1**.

Table 9-1 Notes prepared at the initial peer review meeting

Comments/ Notes	Comments by	Actions against

Royal HaskoningDHV

WARREN AND MAHONEY®

The existing promenade levels range from 5.2 to 5.8m AHD, with proposed promenade levels to range between 4.9 to 5.4m AHD. Proposed glazing at 7.1m AHD.	JC.	Subject to review by all.
RHDHV indicated that new geotech to be undertaken involving 4 pits and 4 boreholes, to confirm rock levels. Existing geotech behind the wall shows rock between -0.2 and +1.0m AHD but there is no information on the beach. The investigation would now also include geophysics, but this may not have been established at the time of the discussion with JC.	GPB	RHDHV to complete
It was noted that if sand remains on the back beach at right angle, runup can be worse (than scoured case with potentially larger waves reaching the wall). The reason for this is that sloping sand provides a ‘ramp’ for the waves to runup and, in addition, the level of the sand against the wall is such that the geometry of the wave return is not as effective.	JC	RHDHV to investigate.
There was general agreement between the Peer Reviewer and RHDHV that inundation hazard may be overstated This comment was in the context of the hazard of inundation vs. the hazard of erosion/recession (undermining), noting also, for example, in respect of inundation that warning time is available (e.g. people can be removed from the risk, additional mitigation can be provided if required [e.g. sand bagging at openings]), for a new structure (as is the case here) the building design can consider the wave loading, and the ground floor level of SLSCs are generally designed to tolerate wave inundation (e.g. concrete floors, used for storage only [mainly], electrical switches elevated).	JC, GWB	
New structure requires sign-off.	JC	W&M
Roller shutters can withstand a splash.	JC	
Linear deflector has been constructed at Collaroy Narrabeen; curved deflectors can be more efficient.	JC	
JC wants to see overtopping and wave force numbers in RHDHV report, express a professional opinion with desk-top calcs, can refine later with physical modelling.	JC	RHDHV to investigate during Stage 2.
Physical modelling prior to detailed design is good, but we can consider delaying this (in the approvals process) depending on the outcome of the desk top assessment. If it is a complex matter (which we cant get a suitable handle on desk-top) then physical modelling could be brought forward.	JC	RHDHV to investigate [Related comment: In discussions with TTW about the loads on the building RHDHV would address the nature/duration of the wave loading, e.g., dynamic (including rise time) and pulsing vs. static, ability for load distribution, and likely 3D nature.
RS referred to JK Geotechnics and Horton reports regarding wave overtopping at Waverly Cemetery, and a proposal for a seawall with wave deflector. Similar to Coll Narr deflector. [RHDHV has now received this from Waverly Council].	RS	RHDHV
Need a development consent in place by Feb 2024.	SO	

Physical model study would take about 3 months	JC	
Investigation may be suited to WRLs 3m flume because it is relatively wide and so offers a partial 3D capability, compared to its normal ~1m wide flumes which are purely for 2D work. Paddle now fitted to 3m flume and commissioned early November.	JC	
Potential for visual impact of the coastal protection works was raised, which would be mitigated by (a) colouring of the concrete, and (b) rendering of the secant wall and/or adoption of a deep capping beam.	GWB	

Figure 9-1 View analysis (Left) Reduced sand levels +3.00m AHD (Right) current sand levels +4.00m AHD. The recommend design scour level under present day climate conditions at the peak of design storm is +2.9m AHD (present day), reducing to +1.70m AHD (2050) and +0.35m AHD (2100) (refer to Section 5.2.3).

10 Coordination workshops with different disciplines

RHDHV would support coordination with other disciplines throughout the project for master planning and concept design related to coastal engineering. The coordination is expected to involve the Access Consultant (with a specific focus on the ramps and stepped structures seaward of the existing seawall) and TTW as the SLSC building Structural Engineer (regarding accommodation of wave loads).

RHDHV understands the existing box culvert would not be subject to any upgrading works by Council. This structure would be incorporated in the subsequent project phases in line with the findings of the "Underground Services Investigation Survey" conducted by RPS in 2022.

11 Summary and conclusions

The preliminary concept design investigations are based on an updated masterplan developed in discussion with W&M and the SCEPP. Basis of design process elements have been described including design life and design event. The coastal engineering investigations cover key coastal processes including water levels and waves, and relevant coastal hazards comprising beach erosion, shoreline recession and coastal inundation. Historical wave loading, estimation of wave runup and overtopping, and estimation of wave loads have been addressed based on accepted desk-top methods.

Discussions within the design team, and involving an initial discussion with the Peer Reviewer, have landed on a structural concept involving a concrete slabs/ shells, fully protected by a row of secant piles. Ramps and steps developed to satisfy the functional requirements for the project are optimally accommodated in the structural concept, which is subject to design development.

The proposed seawall upgrade essentially comprises a vertical piled structure capped with a wave deflector. The deflector profile is likely to be curved, to be confirmed in Stage 3. Physical modelling, to take place early in Stage 3, is proposed to refine the seawall sectional configuration, specifically the crest level and deflector profile.

The coastal assessment task would be completed following completion of the Stage 2 seawall design including wave return walls. This would address the relevant requirements under the Coastal Management Act 2016; State Environmental Planning Policy (Resilience and Hazards) 2021; and Waverly Local Environmental Plan 2012 and Development Control Plan 2022.

The summary and conclusions to this report would be reviewed and developed with the completion of the concept design.

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Coastal Risk Assessment and Coastal Engineering Advice on Bronte Surf Lifesaving Club and Community Facility Redevelopment

AMENDED

prepared by Horton Coastal Engineering Pty Ltd

for Warren and Mahoney

Issue 2

31 July 2023

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TABLE OF CONTENTS

1. INTRODUCTION AND BACKGROUND

It is proposed to demolish the existing Bronte Surf Lifesaving Club (SLSC) and Community Facility clubhouse and to rebuild a new clubhouse over a similar footprint. It is also proposed to rebuild the existing seawall seaward of the clubhouse to provide greater protection to the clubhouse from erosion/recession and oceanic inundation (wave runup). These seawall modifications also provide additional promenade space and structures to enhance public circulation around the clubhouse and access (including disabled access) to the beach.

Waverley Council requires that a coastal engineering assessment is prepared as part of a Development Application (DA) for the clubhouse and seawall works. Horton Coastal Engineering Pty Ltd was engaged by Warren and Mahoney (architects for the clubhouse redevelopment) to complete this assessment, as set out herein.

The DA that is being submitted is an amended application in response to Design Excellence Advisory Panel reviews and planning deferral letters.

The report author, Peter Horton [BE (Hons 1) MEngSc MIEAust CPEng NER], is a professional engineer with 31 years of coastal engineering experience. He has postgraduate qualifications in coastal engineering, and is a Member of Engineers Australia and Chartered Professional Engineer (CPEng) registered on the National Engineering Register. He is also a member of the National Committee on Coastal and Ocean Engineering (NCCOE) and NSW Coastal, Ocean and Port Engineering Panel (COPEP) of Engineers Australia.

Peter has inspected the area in the vicinity of the SLSC on numerous occasions in the last two decades and beyond, including specific recent inspections on 4 February 2020, 21 November 2020, 4 July 2022, 1 December 2022 and 30 March 2023.

All levels given herein are to Australian Height Datum (AHD). Zero metres AHD is approximately equal to mean sea level at present in the ocean immediately adjacent to the NSW mainland.

2. INFORMATION PROVIDED

Horton Coastal Engineering was provided with five drawings prepared by Warren and Mahoney (Drawings DA.100, 101, 110, 200 and 201), all dated 26 July 2023 and Revision C or Revision D.

The following RPS Australia East Pty Ltd site surveys were also provided:

- "Plan showing topographic detail of Bronte SLSC, Bronte Beach, being Lot 102 in DP 1058385", 1 June 2014, Job No. PR122202; and
- "Contour and Detail Survey, Bronte Surf Life Saving Club" Revision C, 25 August 2022, Job No. PR152327.

Other survey information was also referenced, including "Plan of Detail and Levels over Bronte Beach Sea Wall, Bronte Beach, Bronte", Revision B, 24 June 2016, prepared by LTS Lockley.

Information on existing ground floor levels of the SLSC clubhouse was derived from an unattributed Drawing 110A, amendment 1, dated 4 September 2014 and entitled "POM Footprint-Ground Floor".

3. EXISTING SITE DESCRIPTION

3.1 General Description

Views of Bronte SLSC from Bronte Beach are provided in Figure 1 and Figure 2, with vertical and oblique aerial views provided in Figure 3 and Figure 4 respectively.

Figure 1: View of Bronte SLSC from Bronte Beach on 4 July 2022, facing WNW

Figure 2: View of Bronte SLSC from Bronte Beach on 4 February 2020, facing SW (note seaward edge of culvert at arrow)

Figure 3: Vertical aerial view of Bronte SLSC on 1 May 2023

Figure 4: Oblique aerial view of Bronte SLSC on 1 March 2021, facing NW, with culvert discharge location evident on right of image

The seaward edge of the concrete pathway (promenade) seaward of Bronte SLSC (top of the seawall) has a level of about 4.9m AHD adjacent to the base of the steps heading north up the headland ("A" in Figure 3), reducing to 4.7m AHD at the southern edge of the steps to the beach about 1.4m south ("B" in Figure 3) and continuing at that level to the concrete ramp, then reducing to 4.6m AHD at the southern end of the ramp ("C" in Figure 3). This 4.6m AHD level continues to adjacent to the southern end of the SLSC ("D" in Figure 3), then reduces to 4.5m AHD at the double set of stairs located about 5m south of the SLSC ("E" in Figure 3).

The top of the seawall continues to reduce in level moving further south along Bronte Beach, to 4.4m AHD about 5m south of the double stairs, 4.2m AHD at the double ramp, and 3.7m AHD at the double stairs located about 30m north of the South Bronte Amenity and Community Centre.

The promenade level at the base of the stairs leading to the northern section of the SLSC varies from 5.2m AHD (northern end, "F" in Figure 3) to 5.0m AHD (southern end, "G" in Figure 3). The pathway at the top of these stairs is at 5.65m AHD ("H" in Figure 3).

A culvert is located under the promenade, that turns on to the beach adjacent to the northern end of the ramp (see Figure 2), and discharges on the beach about 130m to the NE (see Figure 4). The top surface of this culvert is at 4.1m AHD adjacent to the ramp ("I" in Figure 3).

The sunken courtyard at the SE corner of the clubhouse ("J" in Figure 3) has a minimum level of 4.5m AHD, with internal surrounds at about 4.6 to 4.7m AHD. The external surrounds are at about 4.8m AHD (southern and eastern side) and 5.0m to 5.2m AHD (northern and western side).

The finished ground floor level of the existing SLSC clubhouse varies between 5.62m and 5.80m AHD over the northern portion, and 5.55m to 5.64m AHD over the southern portion.

3.2 Evolutionary Morphology of Bronte Beach

As described by Short (2007), the NSW coast and hinterland has existed in its present form for around 60 million years. The original, mostly plateau rock surfaces have been weathered and eroded to form the slopes, coastal hills and valleys, and coastal plains. The eroded material has been transported by rivers and streams to the coast and reworked by waves and tides into deltas, estuaries and beaches. Sea level has also been moving up and down and the continent has moved northwards over this period.

As further described by Short (2007), during the past 2 million years when most of the present beaches of NSW began forming, mean sea level has varied as much as 150m, with frequencies of oscillation of around 20,000, 40,000 and 120,000 years. During ice ages (glacial maxima) sea level has been at its lowest, including most recently at around 18,000 years ago when mean sea level was 120m below its present level. At this time, the continental shelf around Australia was exposed (above the ocean) and vegetated, and the NSW coastline was about 20 to 60km east of its present location.

After this, the earth began warming and sea level rose for about 11,500 years (at an average rate of around 12mm/year) to reach its present level around 6,500 years ago. Therefore, our present coastline is about 6,500 years old (with headlands composed of older rocks), but also contains remnants of previous shorelines when mean sea level was the same at about 120,000 and 240,000 years ago (Short, 2007).

3.3 Historical Setting

As described by WorleyParsons (2011), no Aboriginal middens or carvings have been found within the Bronte Park area. However, Aboriginal people were well established throughout the present Waverley Council area before European arrival and would have used the area.

Around 1836, European arrivals started claiming land in the area. Mortimer Lewis was granted the land at Bronte Park, with the area of the present SLSC being sandy beach at that time. The property was subdivided for sale in 1882, and in 1886 the NSW government resumed 14 acres of land for the creation of Bronte Park. At that time a creek ran across Bronte Park and discharged at the southern end of the beach (WorleyParsons, 2011; Coast History & Heritage, 2022). A photograph of Bronte Beach around 1910, sourced from the State Library of NSW, is provided in Figure 5.

Figure 5: Bronte Beach in around 1910

As described by Mayne-Wilson & Associates (2003), the first Bronte SLSC clubhouse was constructed in 1910, as visible in Figure 5. The building was extended in 1913-1914.

The seawall was constructed in 1914-1916, with the area landward filled, levelled and turfed. In the mid to late 1920s, the sewage pumping station was constructed landward of the SLSC. In 1930-1931, the first clubhouse was demolished and replaced with a new building, which was extended in the 1950's. This building was destroyed by a fire in 1972. A new building was constructed in February 1974, which is the building evident today (Mayne-Wilson & Associates, 2003).

3.4 Historical Wave Overtopping Events and Damage

Although the current clubhouse was constructed in 1974, and although exposed to oceanic inundation (wave runup and wave overtopping) events, there are no records of significant structural damage to the clubhouse from wave overtopping over the 49 year period to the present.

The most significant coastal storm event in Sydney's recorded history was in May-June 1974, only a few months after the current SLSC building was opened. WorleyParsons (2011) noted that 23m^2 of the seawall, 180m^2 of promenade paving and the culvert were all damaged in this event. At the SLSC, it was noted that doors, shutters and fittings were damaged. It was also noted that sand was deposited and rockeries and garden seats were damaged (presumably in the surrounding park) and there was damage to the building (walls, substructure and interior fixtures) and pool (walls, seating, handrails and chains, with rocks and debris entering the pool) at the southern end of the beach.

The most significant storm since 1974 was in June 2016. This caused damage to three of the SLSC roller doors (Figure 6), and water, sand and debris entered the building (Figure 7 and Figure 8) and the sunken courtyard (Figure 9). More widely in Bronte Park, brick walls were knocked over (Figure 10) and there was damage to the concrete ramp, South Bronte Amenity and Community Centre and pool at the southern end of the beach[1].

Besides the 1974 and 2016 events, there is no known evidence (after extensive liaison with SLSC members and Council staff) of any other overtopping events that have caused notable damage to the clubhouse, and it is reiterated that the 1974 and 2016 events did not cause structural damage to the building.

WorleyParsons (2011) also noted significant wave overtopping events at Bronte Beach prior to 1974, namely in 1942, 1948, 1955 and 1959. A 1948 newspaper article reported that "huge seas swept over [the] promenade at Bronte, flooding Bronte Park to within a few feet of the roadway". A 1959 newspaper article reported that a "30 foot [9.1m] wave overtopped [the] promenade at Bronte and swept 20 yards [18.3m] through the surf club". There may be some journalistic sensationalism in these quotes.

Figure 6: Damage to roller doors at clubhouse in June 2016

[1] Note that the photographs in Figure 6 to Figure 10 (except Figure 8) were supplied by David Finnimore, Director of Sponsorship and Communications at Bronte SLSC.

Figure 7: Sand and debris transported by wave action into clubhouse in June 2016

Figure 8: Sand and debris swept out of clubhouse in June 2016

Figure 9: Sand and debris in sunken courtyard in June 2016

Figure 10: Bricks propelled landward and fences damaged by wave action south of SLSC in June 2016

From discussion with Club members and review of historical videos and photographs, it is evident that wave overtopping on to the promenade tends to preferentially occur at the southern end of the beach, which makes sense given that the seawall crest level and beach width reduces moving south along the beach.

This overtopping at the southern end of the beach tends to cause a flow of high-velocity and shallow-depth wave runup along the promenade to the north. Wave action that reaches but does not overtop the seawall also tends to flow north along the seaward face of the seawall. In the 2016 storm, this northerly flow reached the clubhouse, in combination with wave action directly propagating up the ramp seaward of the clubhouse.

BMT (2021) completed a regional sea-level rise vulnerability assessment for the Randwick, Waverley Council and Woollahra Council areas, thus including Bronte Beach. Using conservative sea level rise projections, they found that wave runup may begin to reach the SE corner of the existing clubhouse, landward of the sunken courtyard and ignoring any attenuation of wave action provided by obstructions, in around 2100 (it is obvious from historical behaviour that wave runup can reach the northern end of the clubhouse at present).

The seawall/promenade seaward of Bronte SLSC has an inadequate crest level to prevent significant wave overtopping in severe storm events, with projected sea level rise exacerbating this issue into the future. The existing promenade is unsafe for pedestrians in severe coastal storms in a 5% annual exceedance probability event at present (Arup, 2016).

3.5 Seawall

As noted in Section 3.3, the seawall at Bronte Beach was constructed in around 1914-1916. This means that the seawall is well beyond its design life. According to Arup (2016), the northern 100m of the seawall (including adjacent to Bronte SLSC) is a mass gravity concrete wall with brick columns founded on sand, and thus vulnerable to damage due to undermining. It appears to have a trapezoidal cross section down to about 2.6m AHD, with the brick columns then extending down to about 1.6m AHD.

Despite being well beyond its design life, Arup (2016) did not consider replacement of the seawall as an option to deal with this risk (or the risk that its crest level is too low to prevent significant wave overtopping in severe storms), instead offering partial remedial options such as the preferred option of installation of engineered scour protection along the seawall toe (a rock or grout bags/mattress system), installation of a sheet pile cut-off wall seaward of the seawall down to rock, underpinning of the seawall, modifications to seawall geometry including widening of the seawall mass landwards, and replacement of seawall backfill. Locally raising the seawall crest with a parapet and/or introducing a wave return/'bull nose' were the recommended mitigation options of Arup (2016) to deal with wave overtopping.

However, the above options may be considered as 'band-aid' solutions to a seawall well beyond its design life. The rebuilt seawall at the SLSC clubhouse proposed for the subject DA is considered to be a more far more effective option.

3.6 Historical Beach Profiles

Arup (2016) derived 11 beach profiles seaward of the northern end of Bronte SLSC, for dates in 1970, 1976, 1978, 1982, 1986, 1996, 1999, 2001, 2004, 2006 and 2016 respectively. This indicated that immediately adjacent to the seawall there had not been significant variability in sand levels, with 6 dates having sand levels approximately at the seawall crest, and the other 5 dates having sand levels within 1m of the crest. Variability in beach sand levels was evident further seaward along the profile, eg about 3m in vertical variation about 40m seaward of the seawall, in response to beach erosion and subsequent beach recovery.

Arup (2016) also derived 11 beach profiles for the same dates seaward of the southern end of Bronte SLSC. Sand levels were within 1m of the crest of the seawall on all dates, with 9 profiles within 0.8m of the crest.

This profile data, and data for three other profiles along the beach, gave no indication of a recession trend at Bronte Beach over this record. This result is also supported from review of the DEA Coastlines (Geoscience Australia Landsat Coastlines Collection 3) data set, which has the median annual position of the shoreline at 0m AHD from 1988 to 2020, with no recession trend evident at Bronte Beach in this data (this data is discussed further in Section 6.3).

This lack of a recession trend is also supported by review of historical aerial and site photography, which indicates no obvious long term change to Bronte Beach over the last 100 years.

3.7 Subsurface Conditions

Geotechnical investigations at the subject site have been undertaken by AssetGeoEnviro (2020, 2022). Based on three boreholes (BH1, BH2 and BH3) drilled for the 2020 study at the landward, centre and seaward edge of the development area respectively, the subsurface can generally be described as sand overlying sandstone bedrock. Specific details of these boreholes are provided in Table 1.

Arup (2016) also excavated test pits along the seawall in an attempt to determine the foundation conditions and levels of the seawall.

Table 1: Borehole details from AssetGeoEnviro (2020)

Borehole	Location	Stated surface level (m AHD)	Actual surface level from survey (m AHD)	Depth to sandstone bedrock (m)	Level of sandstone bedrock (m AHD)
BH1	About 30m landward of clubhouse	5	5.7	8.2	-2.5
BH2	NW corner of sunken courtyard	4	5.3	3.8	1.5
BH3	Seaward of clubhouse on promenade	4	5.0	4.2	0.8

It is evident from Table 1 that in the active coastal zone (where erosion occurs above about -1m AHD on a sandy beach), the natural subsurface at and seaward of the SLSC is likely to have relatively inerodible bedrock in the lower profile.

AssetGeoEnviro (2020, 2022) recommended that the new SLSC clubhouse was founded on bedrock (that is, had footings extending down to bedrock). It has been assumed herein that this would be carried out.

With an appropriately designed and constructed seawall in place seaward of the clubhouse, the foundations of the clubhouse may be designed based on conventional structural and geotechnical considerations, and would not require any coastal engineering input to consider wave and sand slumping loads and the like.

3.8 Extreme Water Levels

Based on Department of Environment, Climate Change and Water [DECCW] (2010), the 100-year Average Recurrence Interval (ARI) ocean water level (in the absence of wave action) as of 2010 in Sydney is 1.44m AHD. This is similar to be the corresponding value reported by Manly Hydraulics Laboratory [MHL] (2018)[2]. Extrapolating the water levels (linear-log) provided in DECCW (2010) for various ARI's, the corresponding 2,000 year ARI value is 1.57m AHD.

Applying these values to the present (2023) using a rate of sea level rise of 3mm/year from 2010 to 2023, as recommended in DECCW (2010), the 100 year ARI and 2,000 year ARI present day ocean water levels (in the absence of wave action) are 1.48m and 1.61m AHD respectively.

Wave setup, caused by breaking waves adjacent to a shoreline, can also increase still water levels. For a 100 year ARI event, this increase may be in the order of 1.5m. Therefore, a 100 year ARI water level of 3.0m AHD applies at Bronte SLSC at present. This is below the level of the promenade, but wave runup can cause wave overtopping of the promenade at times of large waves and elevated ocean water levels.

[2] MHL (2018) determined a corresponding level of 1.42m AHD (along with lower and upper 95% confidence limits of 1.38m AHD and 1.53m AHD respectively).

4. PROPOSED DEVELOPMENT

4.1 Overall Description of Development

It is proposed to demolish the existing SLSC clubhouse and to rebuild a new clubhouse over a similar footprint, and to undertake modifications to the existing seawall seaward of the clubhouse to provide greater protection to the clubhouse from erosion/recession and oceanic inundation (wave runup), and to provide additional promenade space to enhance public circulation around the clubhouse and access (including disabled access) to the beach.

4.2 Design Life

A structural engineering (durability) design life of 50 years has been adopted for the proposed development (that is, at the year 2073), as agreed with Council. This design life is considered to be appropriate as:

- it is consistent with Australian Standards:
    - in *AS 3600-2018 (Concrete structures)*, a 50 years ± 20% design life[3] (that is, 40 years to 60 years) is used in devising durability requirements for concrete structures;
    - in *AS 2870-2011 (Residential slabs and footings)*, for design purposes the life of a structure is taken to be 50 years for residential slabs and footings construction (it is recognised that the SLSC clubhouse is not a residential structure though);
    - in *AS 1170.0-2002* (*Structural Design Actions – General Principles)*, the design life for normal structures (Importance Level 2, as would be expected to apply to the proposed clubhouse) is generally taken as 50 years; and
    - in *AS 4678-2002 (Earth-retaining structures),* the design life for earth-retaining structures (structures required to retain soil, rock and other materials) is noted as 60 years for river and marine structures and residential dwellings; and
- a design life of at least 50 years would be considered to be reasonable for permanent structures used by people (AGS, 2007a, b).

Although a 50 year structural engineering design life has been adopted, a 70 year coastal engineering design was adopted for the proposed development (that is, at the year 2103), as requested by Council. This means that the proposed clubhouse shall be designed to withstand coastal erosion and wave overtopping events with an acceptably low risk of damage over a 70 year life.

4.3 Features to Reduce the Risk of Wave Runup Causing Damage to the Clubhouse

The design of the clubhouse and seawall included iterative coastal engineering input to include features to reduce the risk of wave runup causing damage to the clubhouse in severe storms, as depicted in Figure 11. These features (numbered in Figure 11) include:

1. raising the ground floor over much of the clubhouse to 6.1m AHD;
2. raising the seawall seaward of the northern end of the clubhouse to 5.8m AHD, and including a wave return[4];

[3] Period for which a structure or a structural member is intended to remain fit for use for its designed purpose with maintenance.

[4] Raising the seawall and adding a wave return would also reduce the risk of windblown and wave-transported sand and debris reaching the promenade and entering the clubhouse.

3. raising the seawall seaward of the southern end of the clubhouse to 5.8m AHD, and including a wave return[4];
4. removing the existing ramp to the beach (that faces seaward) located seaward of the clubhouse, and forming two new ramps, and steps, facing alongshore;
5. Lifeguards Room landward of the raised seawall and elevated with a floor level of 6.45m AHD and base of windows at 7.45m AHD;
6. First Aid Room and Patrol Room landward of the raised seawall and with door entries facing alongshore (with wave barriers manually deployed on each door at times of storms), and base of windows at 6.8m and 6.4m AHD respectively (it may also be possible to angle the walls seaward of these rooms to act as a wave return);
7. circulation area landward of elevated seawall, with wave barrier manually deployed at times of storms in the circulation area, and permanent security gate that is open and robust to allow wave action through without getting damaged;
8. pump out pit in equipment storage area to be used in the unlikely event that there was overtopping of the wave barrier in the circulation area (not depicted in Figure 11);
9. courtyard with walls designed to resist wave impact, to act as barriers to reduce the landward extent and depth of wave overtopping reaching the kiosk and kiosk store;
10. kiosk base of windows at 6.4m AHD;
11. the kiosk store may require a wave barrier manually deployed at times of storms (to be determined as part of detailed design);
12. angling the top of the step risers in a seaward direction at the main steps, to act as 'mini' wave returns; and
13. seawall protecting beach groomer ramp with a crest level of 5.1m AHD.

The layout of the clubhouse and ramps and steps (as proposed) reduces the risk that wave overtopping along the promenade and wave action along the beach moving from south to north would impact on the proposed development. This is because the ramps and steps direct the overtopping away from the clubhouse, and direct the wave action along the beach back out to sea.

A feasible wave barrier for the circulation area (to be assessed as part of detailed design) would comprise a central permanent bollard cast-in sleeve (hidden under a cap such as a hinged plate), which would have a bollard inserted and infill panels attached and manually installed when required, with suitable mechanical connections for the panels to the wall at each end. For the individual door wave barriers, the central bollard is unlikely to be required.

Note that relocation of the clubhouse further landward, and raising the finished ground floor level of the clubhouse further, were considered as options. However, based on the *Bronte Park and Beach Plan of Management*, it is understood that restrictions on moving westward (due to the Sydney Water pumphouse west of the clubhouse) and upward (due to a building height limit of 14m AHD) made these options unfeasible.

That stated, it is considered that the SLSC as proposed is feasible from a coastal engineering perspective, and does not require relocation or raising to be at an acceptably low risk of wave overtopping over an acceptably long life, with the measures outlined above being assessed as part of detailed design.

Figure 11: Features to reduce the risk of wave runup damage to SLSC clubhouse

It may be a prudent for Council to maintain the sand level on the beach below a certain level marked on the seawall and steps for reference, eg 4m AHD, in the future. This is because the wave returns would be more effective at reducing wave overtopping if they are located higher above the beach. If sand was at the seawall crest, this would provide a ramp for waves to overtop the seawall if erosion did not lower sand levels during a storm. This could again be assessed as part of detailed design.

4.4 Seawall

It is intended that the seawall is rebuilt seaward of the SLSC clubhouse, comprising ramps and steps as depicted in Figure 11 (Items 2, 3, 4, 12 and 13). It is expected that the seawall would comprise:

- continuous flight auger (alternating reinforced concrete and unreinforced concrete) secant piles founded into bedrock, designed as a barrier to soil migration through the wall;
- a reinforced concrete capping beam connected to the piles;
- a vertical sand-coloured reinforced concrete wall extending above the capping beam; and
- anchors attached to the capping beam (and permanently buried landward of the wall), designed to provide support for the seawall at times of beach erosion when sand levels lower on the seaward side of the wall, if found to be required from stability calculations undertaken as part of detailed design.

The ramps and stairs that are proposed would be expected to be reinforced concrete, and supported on discrete piles as required where seaward of the secant piles.

The alignment of the proposed seawall, ramps and stairs is depicted in Figure 12. This has the seaward edge of the promenade extending 2m seaward of its current position, with the steps and ramps extending further seaward. This proposed layout provides separation from the coastal walkway and surf lifesaving beach access activities (with access to and from the main storage not crossing the coastal walkway path), enhances access to the beach for lifesaving equipment (with an enhanced access ramp compared to the existing ramp), enhances public access to the beach (with enhanced stair and ramp access compared to the existing stairs and ramp), provides disabled access, and provides enhanced protection to the clubhouse from wave runup.

The detailed design of the seawall would be prepared as an integrated coastal, structural and geotechnical engineering investigation. The design solution would have a demonstrated factor of safety exceeding 1.5 for both global stability and structural stability (with consideration of disturbing and balancing forces and moments) taking account of the particular subsurface conditions at the site. As part of detailed design, there would be consideration of coastal engineering issues (beach scour, long term recession due to sea level rise, elevated water levels, and wave and hydrostatic forces), geotechnical engineering issues (subsurface conditions, global stability, analysis to determine pile embedment and anchor capacity) and structural engineering issues (bending moments, shear forces, deflections, strength, serviceability and durability) leading to concrete member and anchor design.

Including a wave return shape at the crest of the seawall (angling the top of the seaward face of the seawall in a seaward direction, either curved or tapered) would assist in reducing the volume of wave overtopping in severe storms.

If necessary, weepholes would be included through the seawall to reduce the risk of buildup of groundwater pressures on the landward side of the seawall. These weepholes would include a geotextile sock to reduce the risk of soil migration through the weepholes.

Figure 12: Alignment of proposed seawall, ramps and stairs at Bronte SLSC (aerial photograph taken 1 May 2023)

5. EROSION/RECESSION COASTLINE HAZARDS

5.1 Generic Explanation of Hazard Zones

Nielsen et al (1992) has delineated various coastline hazard zones as discussed below and depicted in Figure 13, assuming an entirely sandy (erodible) subsurface above -1m AHD. This is likely to be conservative at Bronte Beach based on bedrock in the lower profile, as discussed in Section 3.7

The *Zone of Wave Impact* (ZWI) delineates an area where any structure or its foundations would suffer direct wave attack during a severe coastal storm. It is that part of the beach which is seaward of the beach erosion escarpment.

A *Zone of Slope Adjustment* (ZSA) is delineated to encompass that portion of the seaward face of the beach that would slump to the natural angle of repose of the beach sand following removal by wave erosion of the design storm demand. It represents the steepest stable beach profile under the conditions specified.

A *Zone of Reduced Foundation Capacity* (ZRFC) for building foundations is delineated to take account of the reduced bearing capacity of the sand adjacent to the storm erosion escarpment. Nielsen et al (1992) recommended that structural loads should only be transmitted to soil foundations outside of this zone (ie landward or below), as the factor of safety within the zone is less than 1.5 during extreme scour conditions at the face of the escarpment. In general (without the protection of a terminal structure such as a seawall), dwellings/structures not piled and located within the ZRFC would be considered to have an inadequate factor of safety.

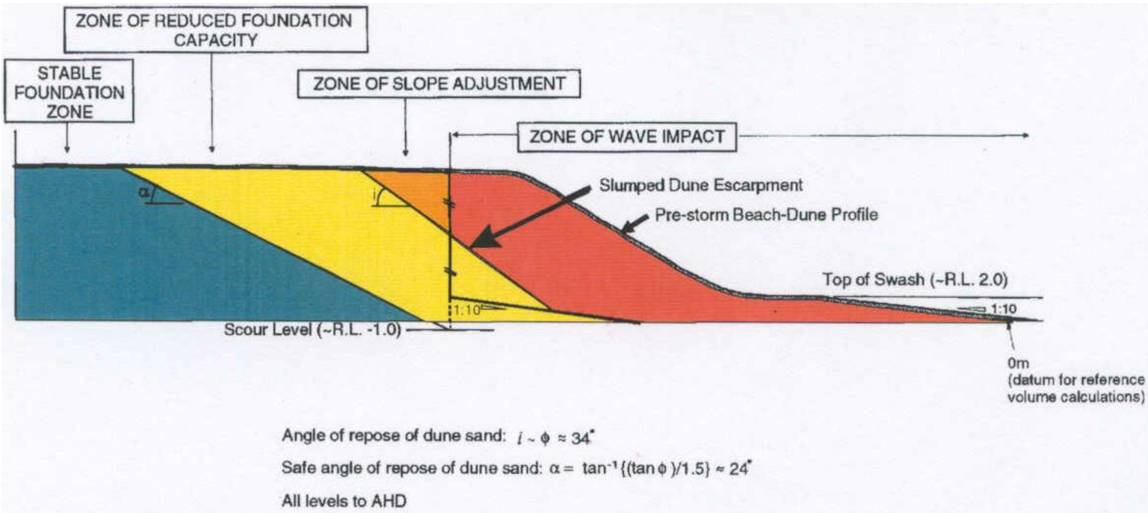

Figure 13: Schematic representation of coastline hazard zones (after Nielsen et al, 1992)

5.2 Current Council Hazard Lines

In the *Waverley Coastal Risks and Hazards Vulnerability Study* (WorleyParsons, 2011), a single hazard line was mapped at Bronte Beach under the assumption that the seawall will be retained into the future, therefore the erosion hazard limit (ZWI and ZSI) would be the seawall, with no change over time. The ZRFC was identified as the distance landward of the seawall, that distance being at a slope of 1:2 (vertical:horizontal) of the retained height of the backfill sand above an extreme scour level of 0 m AHD. That slope provides for a factor of safety for global slope stability of 1.5 for medium dense sand. The ZRFC line at Bronte thus extends about 8m landward of the seawall, within the footprint of the proposed SLSC clubhouse. However, with an appropriately designed and constructed seawall seaward of the clubhouse, in reality there is no ZRFC landward of the seawall.

Without the seawall, the proposed SLSC clubhouse would be expected to be undermined in a severe coastal storm over its design life.

6. FUTURE BEACH BEHAVIOUR

6.1 Preamble

Council requested that the following was addressed as part of the investigation reported herein:

- a detailed assessment of what the beach and adjacent area may be like following a severe storm now and in 2050 and 2080 and in 2103 (location of the cliff face, wave inundation, loss of trees, amenity or infrastructure once the works are completed); and
- an assessment of the anticipated life of the development/use on the current site, given the sea level will continue to rise.

A response to these matters is set out below.

It is important to understand that any recession of the cliff face to the north of the development, or beach recession due to sea level rise, or general wave overtopping of the seawall at the landward edge of the sandy beach, would occur whether the development was constructed or not. That is, the proposed development has no effect on these outcomes whatsoever, except that wave overtopping would be expected to be reduced at the location of the rebuilt seawall compared to the status quo.

Reiterating that long term beach recession due to sea level rise is expected to occur whether the works are constructed or not, note that this recession is expected to be caused by projected greenhouse gas emissions, land use changes and air pollutant controls in the future at a global scale. The proposed works will not cause beach recession, but rather these global processes. Stated another way, beach recession will occur in the same manner if the seawall is not undertaken.

With regard to cliff face recession, it is reiterated that the proposed development would have no effect on any recession of the cliff face to its north. Consideration of recession of this cliff face is a geotechnical rather than coastal engineering matter, and is not discussed further herein, except to state that WorleyParsons (2011) included a geotechnical assessment that considered this headland but did not raise any particular risk concerns. If Council has concerns with the risk of cliff instability to the north of the subject development, this could be addressed as a separate study.

The assessment below focusses on long term recession due to sea level rise, and how this may affect beach width in the future. As discussed in Section 3.6, there does not appear to be a long term trend of recession due to net sediment loss at Bronte Beach, so long term recession of the beach in the future would be expected to be related to sea level rise effects only.

6.2 Sea Level Rise

It is considered to be most appropriate to derive sea level rise values from Intergovernmental Panel on Climate Change [IPCC] (2021), which is widely accepted by competent scientific opinion. Sea level rise values are determined herein for the five illustrative scenarios (shared socioeconomic pathways, SSP's[5]) considered in IPCC (2021)[6], at 2050 (see Table 2), 2080 (see Table 3), and 2103 (see Table 4) and relative to the present (2023).

These sea level rise projections include regional sea level rise variations at Sydney as reported by the Physical Oceanography Distributed Active Archive Center (PO.DAAC), a NASA Earth Observing System Data and Information System data centre operated by the Jet Propulsion Laboratory in Pasadena, California.

Table 2: Sea level rise (m) at Sydney from 2023 to 2050, from IPCC (2021) and PO.DAAC

Emissions Scenario (Shared Socioeconomic Pathway)	Exceedance Probability		
	95% exceedance	Median	5% exceedance
SSP1-1.9	0.09	0.15	0.27
SSP1-2.6	0.07	0.16	0.30
SSP2-4.5	0.09	0.18	0.32
SSP3-7.0	0.11	0.20	0.33
SSP5-8.5	0.13	0.22	0.35
Average	0.10	0.18	0.31

Table 3: Sea level rise (m) at Sydney from 2023 to 2080, from IPCC (2021) and PO.DAAC

Emissions Scenario (Shared Socioeconomic Pathway)	Exceedance Probability		
	95% exceedance	Median	5% exceedance
SSP1-1.9	0.11	0.24	0.48
SSP1-2.6	0.14	0.29	0.55
SSP2-4.5	0.21	0.38	0.65
SSP3-7.0	0.27	0.44	0.74
SSP5-8.5	0.32	0.50	0.83
Average	0.21	0.37	0.65

Table 4: Sea level rise (m) at Sydney from 2023 to 2103, from IPCC (2021) and PO.DAAC

Emissions Scenario (Shared Socioeconomic Pathway)	Exceedance Probability		
	95% exceedance	Median	5% exceedance
SSP1-1.9	0.12	0.34	0.68
SSP1-2.6	0.17	0.39	0.76
SSP2-4.5	0.30	0.54	0.97
SSP3-7.0	0.42	0.69	1.16
SSP5-8.5	0.49	0.79	1.35
Average	0.30	0.55	0.98

Taking the median exceedance probability and average of the 5 SSP's[7], sea level rise values of 0.18m at 2050, 0.37m at 2080 and 0.55m at 2103 (relative to 2023) have been considered herein for the purpose of illustrating the most likely future behaviour.

[5] Known as representative concentration pathways in the previous IPCC (2013) assessment.
[6] The five illustrative scenarios represent varying projected greenhouse gas emissions, land use changes and air pollutant controls in the future.
[7] Note that the SSP5 8.5 scenario has been dismissed as implausible by competent scientific opinion, eg see Pielke and Ritchie (2021).

6.3 Effect of Long Term Recession Due to Sea Level Rise on Beach Profiles

Bruun (1962) proposed a methodology to estimate long term recession due to sea level rise, the so-called Bruun Rule. It can be described by the equation (Morang and Parson, 2002):

$$R = \frac{S \times B}{h + d_c} \tag{1}$$

where R is the recession (m), S is the long-term sea level rise (m), h is the dune height above the initial mean sea level (m), d_c is the depth of closure of the profile relative to the initial mean sea level (m), and B is the cross-shore width of the active beach profile, that is the cross-shore distance from the initial dune height to the depth of closure (m). Equation 1 is a mathematical expression that the recession due to sea level rise is equal to the sea level rise multiplied by the average inverse slope of the active beach profile, with the variables as illustrated in Figure 14.

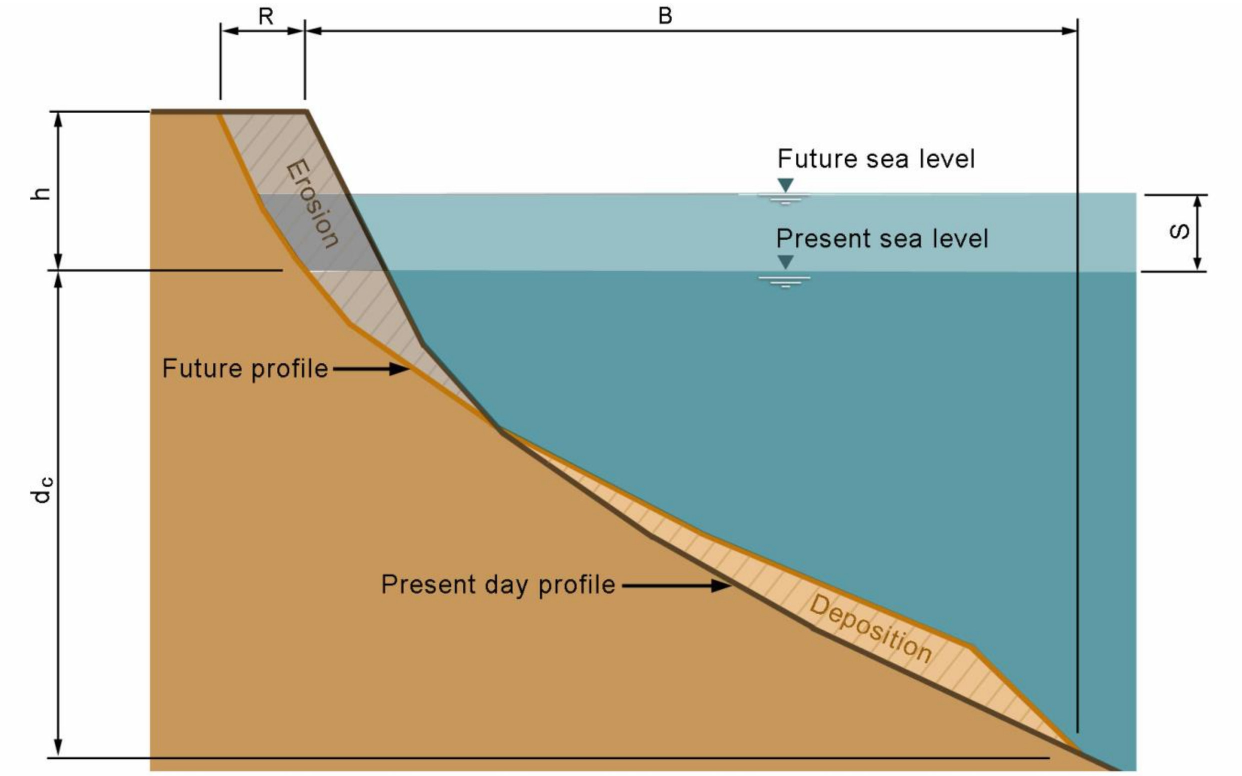

Figure 14: Illustration of variables in the Bruun Rule

There are a number of methods available to estimate the depth of closure, including techniques based on wave (and sediment) characteristics, sedimentological data, and field measurements. Hallermeier (1981, 1983) defined two closure depths, namely "inner" (closer to shore) and "outer" (further from shore) closure depths. The "inner" closure depth is considered to be appropriate to use herein. From Hallermeier (1981), the "inner" closure depth is approximately 12m relative to AHD at Bronte Beach, with the average inverse slope of the active beach profile corresponding to this depth equal to 50[8].

Therefore, for sea level rise values of 0.18m at 2050, 0.37m at 2080 and 0.55m at 2100 (see Section 6.2), long term recession can be estimated as 9m, 18.5m and 27.5m respectively. The subaerial beach can be assumed to narrow by these magnitudes, as sea level rise causes the beach to move upward (by the magnitude of sea level rise) and landward. Note that the

[8] This invoice slope was used in WorleyParsons (2011).

average beach slope at the northern end of Bronte Beach is 1:11.8 (vertical:horizontal) from the beach-face slope dataset for Australia (Vos et al, 2022).

In Arup (2016), the average beach width seaward of the existing seawall to 0m AHD is about 60m for 11 profiles from 1970 to 2016 (see Section 3.6 for discussion on these profiles). This is consistent with the median of 33 years of the median annual position of the shoreline at 0m AHD from 1988 to 2020 of 50m from DEA Coastlines (again see Section 3.6), see Figure 15. That is, the median beach width to the shoreline at 0m AHD seaward of the seawall at Bronte SLSC has been 50m over the last 33 years. This is projected to reduce to about 39m at 2050, 30m at 2080 and 21m at 2103, accounting for long term recession due to sea level rise and the reduction in beach width of 2m due to the new promenade.

Figure 15: Median annual position of the shoreline at 0m AHD from 1988 to 2020 (blue lines) from DEA Coastlines, with median of these 33 years in red, receded 33-year median at 2050, 2080 and 2103 in yellow, brown and cyan respectively, and proposed seawall (plus ramp and stairs) shown in green

It is evident from Figure 15 that there would (on average) be the expectation of a useable beach width seaward of Bronte SLSC at 2103. If sea level continues to rise beyond the design life, and beach recession occurs as projected based on Bruun (1962), and there are no beach nourishment intervention works undertaken, there may be a point mid next century when there is insufficient beach width to maintain sandy beach activities, on average.

It is reiterated that long term recession due to sea level rise would occur whether the proposed works are constructed or not, and that over the 70 year coastal engineering design life of the clubhouse there is the expectation of a useable sandy beach on average. The public and lifesaving benefits of the new seawall 2m further seaward (plus ramps and steps projecting further seaward and providing enhanced beach access[9]), may be considered to outweigh its minor impacts on beach width beyond its design life and over a relatively short length of 50m within the 280m length of beach.

There would be ample opportunity well into the future and beyond the design life of the proposed works, as part of any future development application, to assess the feasibility of continuing to maintain the clubhouse at its proposed location. These decisions would have to be made in the context of the wider beach amenity of Bronte Beach, that are unrelated to the proposed works.

The long term feasibility of the proposed clubhouse in terms wave overtopping impacts is considered in Section 7.

[9] Although note that the lower portions of the ramps and steps would be covered by sand for most of the time.

7. COASTAL INUNDATION COASTAL HAZARDS

The ground floor of the SLSC clubhouse is exposed to potential damage from oceanic water inundation (wave runup), projectile debris at that time, and sand infill carried with the inundation. With projected sea level rise, the frequency and depth of inundation events impacting the clubhouse would be expected to increase over time.

The measures outlined in Section 4.3 significantly reduce the risk of inundation damage to the clubhouse, and are expected to achieve an acceptably low risk of damage to the clubhouse from coastal inundation over the design life.

It will be necessary to design the walls of the SLSC clubhouse to resist wave and hydrostatic forces, as advised by a coastal engineer as part of detailed design. With use of reinforced concrete, in the experience of the author, this could feasibly be achieved.

Other measures that could be considered (where practical) to reduce the risk of inundation damage on the ground floor include:

- using floor finishes and wall materials that would withstand inundation, such as concrete and tiles;
- allowing for wave forces on glazing, or constructing glazing that faces seawards from toughened/laminated glass with appropriate fracture characteristics that present a low hazard when fractured, or such that it holds together when shattered;
- placing electrical fittings and outlets that could be damaged by inundation a suitable distance above the finished floor level;
- storing items that could be damaged by inundation or become polluting due to inundation a suitable distance above the finished floor level;
- developing and adopting an emergency action plan to include installation of a temporary barriers (described in Section 4.3) when severe coastal storms are forecast to impact on the building; and
- allowing for relocation of items prior to a forecast storm, if required, as part of an adopted emergency action plan.

The runup process over seawalls (with wave returns), steps, ramps, barriers and perimeter walls is too complex to define analytically. Although it is reiterated that the measures outlined in Section 4.3, and above, are considered to feasibly achieve an acceptably low risk of damage to the clubhouse from coastal inundation over the design life, to demonstrate this then physical modelling (in a wave flume or basin) could be considered as part of detailed design.

Physical modelling would also give the opportunity to refine the features to reduce the risk of wave runup damage to the clubhouse, such as the seawall crest level and heights of temporary barriers.

Council should also consider, as a separate project (but integrated to the proposed seawall for the SLSC), the necessity to upgrade the entire seawall along Bronte Beach as it is beyond its design life, at risk of failure in a severe coastal storm, and ineffective in sufficiently reducing wave overtopping volumes in severe storms (thus placing infrastructure landward of the seawall at risk of damage, and any pedestrians in the vicinity of the seawall at risk of injury, noting that exclusion of the public from the seawall promenade in storm events is practically difficult to achieve).

8. CATCHMENT AND OVERLAND FLOW FLOODING

Based on TTW (2023), the proposed finished floor levels for the clubhouse are satisfactory from a flood risk perspective. Therefore, the focus of the investigation reported herein is on inundation due to wave runup.

9. MERIT ASSESSMENT

9.1 *State Environmental Planning Policy (Resilience and Hazards) 2021*

9.1.1 Preamble

Based on *State Environmental Planning Policy (Resilience and Hazards) 2021* (SEPP Resilience) and its associated mapping, the subject property is within a "coastal environment area" (see Section 9.1.2) and a "coastal use area" (see Section 9.1.3).

Based on Clause 2.16(2)(b) of SEPP Resilience, the proposed seawall (coastal protection works) is permissible with consent, given that the proponent is a public authority and the study area does not have a gazetted Coastal Zone Management Plan (CZMP) or Coastal Management Program.

The study area is zoned as RE1 (Public Recreation) in *Waverley Local Environmental Plan 2012* (LEP 2012). Coastal protection works are not specifically permitted in this zone. However, SEPP Resilience, as per Clause 2.5(1), prevails over LEP 2012. Furthermore, non-inclusion of protection works as being permitted in this zone is considered to be related more to the restrictive nature of the *Standard Instrument -Principal Local Environmental Plan* rather than any deliberate intention of Council to exclude these works[10].

Community facilities are permissible with consent in the RE1 zone.

9.1.2 Clause 2.10

Based on Clause 2.10(1) of SEPP Resilience, "development consent must not be granted to development on land that is within the coastal environment area unless the consent authority has considered whether the proposed development is likely to cause an adverse impact on the following:

(a) the integrity and resilience of the biophysical, hydrological (surface and groundwater) and ecological environment,
(b) coastal environmental values and natural coastal processes,
(c) the water quality of the marine estate (within the meaning of the *Marine Estate Management Act 2014*), in particular, the cumulative impacts of the proposed development on any of the sensitive coastal lakes identified in Schedule 1,
(d) marine vegetation, native vegetation and fauna and their habitats, undeveloped headlands and rock platforms,
(e) existing public open space and safe access to and along the foreshore, beach, headland or rock platform for members of the public, including persons with a disability,
(f) Aboriginal cultural heritage, practices and places,
(g) the use of the surf zone".

With regard to (a), the proposed works are in an already developed area, with the footprint of the proposed clubhouse similar to the existing clubhouse. Given this, and the fact that existing

[10] This anomaly is common to many Local Government Areas where coastal protection works are considered to be appropriate through the CZMP process, including the *Central Coast Local Environmental Plan 2022* applying to Wamberal Beach, the *Pittwater Local Environmental Plan 2014* applying to Bilgola Beach and Basin Beach, and the *Warringah Local Environmental Plan 2011* applying to Collaroy-Narrabeen Beach.

stormwater drainage arrangements are not to be significantly altered[11], the works would not be expected to adversely affect the biophysical, hydrological (surface and groundwater) and ecological environments. The more seaward alignment of the proposed seawall has been justified in Section 4.4, and would not be expected to significantly affect these matters. The proposed works would not be a source of pollution as long as appropriate construction environmental controls are applied.

With regard to (b), the proposed works would not be expected to affect the natural coastal processes of wave propagation and the like in the surf zone offshore of Bronte Beach, nor uprush on to the sandy beach. The proposed raised seawall would reduce the volume of wave overtopping in severe storms compared to the existing seawall, but reduce the width of sandy beach at this location. This reduced beach width may be considered acceptable given the public and lifesaving benefits this provides (see Section 4.4), and the fact that the overall area of sandy beach at Bronte Beach would only be slightly reduced as a result of the proposal. Therefore, looking at the overall benefits of the more seaward seawall, it can be accepted that coastal environmental values (if beach width is considered to be a coastal environmental value) would not be adversely impacted by the proposed development.

With regard to (c), the proposed works would not adversely impact on water quality as long as appropriate construction environmental controls are applied.

With regard to (d), this is not a coastal engineering matter so is not definitively considered herein. That stated, there are no undeveloped headlands or rock platforms in proximity to the proposed development, and no marine vegetation in the area to be developed. If there is no native vegetation and fauna and their habitats of significance at the site, this clause has been satisfied.

With regard to (e), the proposed works would not impact on public open space and access to and along the foreshore. The proposed development maintains and enhances public access along the promenade to the east of the building, and from the promenade to the beach, and provides a new disabled access ramp.

With regard to (f), an Aboriginal Heritage Due Diligence Assessment has been carried out by Coast History & Heritage, dated 21 September 2022, and this is not a coastal engineering issue.

With regard to (g), the proposed works would not be expected to significantly alter wave and water level processes seaward of the property, and the works enhance public access to the surf zone. Therefore, use of the surf zone is enhanced as a result of the proposed works.

Based on Clause 2.10(2) of SEPP Resilience, "development consent must not be granted to development on land to which this clause applies unless the consent authority is satisfied that:

(a) the development is designed, sited and will be managed to avoid an adverse impact referred to in subclause (1), or
(b) if that impact cannot be reasonably avoided—the development is designed, sited and will be managed to minimise that impact, or
(c) if that impact cannot be minimised—the development will be managed to mitigate that impact".

[11] Based on the Stormwater Management Plan prepared by TTW (Job No 231446, Drawing DA140, Issue P1, 14 July), stormwater is proposed to be captured by pits and pipes to enter the existing culvert, as occurs at present.

The proposed development has been designed and sited to avoid the adverse impacts referred to in Clause 2.10(1), as long as the recommendations in the Aboriginal Heritage Due Diligence Assessment are considered.

9.1.3 Clause 2.11

Based on Clause 2.11(1) of SEPP Resilience, "development consent must not be granted to development on land that is within the coastal use area unless the consent authority:

(a) has considered whether the proposed development is likely to cause an adverse impact on the following:
  (i) existing, safe access to and along the foreshore, beach, headland or rock platform for members of the public, including persons with a disability,
  (ii) overshadowing, wind funnelling and the loss of views from public places to foreshores,
  (iii) the visual amenity and scenic qualities of the coast, including coastal headlands,
  (iv) Aboriginal cultural heritage, practices and places,
  (v) cultural and built environment heritage, and
(b) is satisfied that:
  (i) the development is designed, sited and will be managed to avoid an adverse impact referred to in paragraph (a), or
  (ii) if that impact cannot be reasonably avoided—the development is designed, sited and will be managed to minimise that impact, or
  (iii) if that impact cannot be minimised—the development will be managed to mitigate that impact, and
(c) has taken into account the surrounding coastal and built environment, and the bulk, scale and size of the proposed development".

With regard to (a)(i), the proposed works would enhance beach access, as discussed previously.

With regard to (a)(ii), (a)(iii), and (c), these are not coastal engineering matters so are not considered herein.

With regard to (a)(iv), see Section 9.1.2.

With regard to (a)(v), a Heritage Impact Statement has been prepared by Zoltan Kovacs Architect, dated August 2022. They found "that Council should consent to the proposed development in recognition of its lack of adverse heritage conservation impacts and high architectural merit, and that the existing building to be demolished should be subject to standard archival recording in accordance with NSW Heritage guidelines".

With regard to (b), the proposed development has been designed and sited to avoid any potential adverse impacts referred to in Clause 2.11(1).

9.1.4 Clause 2.12

Based on Clause 2.12 of SEPP Resilience, "development consent must not be granted to development on land within the coastal zone unless the consent authority is satisfied that the proposed development is not likely to cause increased risk of coastal hazards on that land or other land".

The proposed development significantly reduces the risk of coastal hazards (in particular from wave runup) on that land, and is unlikely to cause any increased risk of coastal hazards on any other land, with adjacent areas already having seawalls.

9.1.5 Clause 2.13

Based on Clause 2.13 of SEPP Resilience, "development consent must not be granted to development on land within the coastal zone unless the consent authority has taken into consideration the relevant provisions of any certified coastal management program that applies to the land".

No certified coastal management program applies at the subject property.

9.2 *Coastal Management Act 2016*

Based on Section 27 of the *Coastal Management Act 2016*, "development consent must not be granted under the *Environmental Planning and Assessment Act 1979* to development for the purpose of coastal protection works, unless the consent authority is satisfied that:

(a) the works will not over the life of the works
  - (i) unreasonably limit or be likely to unreasonably limit public access to or the use of a beach or headland, or
  - (ii) pose or be likely to pose a threat to public safety; and,

(b) satisfactory arrangements have been made (by conditions imposed on the consent) for the following for the life of the works:
  - (i) the restoration of a beach, or land adjacent to the beach, if any increased erosion of the beach or adjacent land is caused by the presence of the works,
  - (ii) the maintenance of the works".

With regard to (a)(i), the proposed works enhance public and lifesaving access to and from the beach compared to the existing situation, by providing an enhanced ramp and enhanced steps, and separating walkers from beach users. The small reduction in sandy beach width as a result of these works is considered to be negligible in terms of the overall sandy beach area at Bronte Beach, and given the public and lifesaving benefits that these works provide.

With regard to (a)(ii), the proposed works would pose no significant threat to public safety, as they would be designed to withstand an acceptably rare storm over a 70 year design life, and are less of a threat to public safety than the do-nothing scenario. The proposed works also substantially reduce public safety risks due to wave overtopping of the seawall compared to the existing situation.

With regard to (b)(i), the beach would be expected to naturally accrete and be restored seaward of the proposed works after storm events, and no differently to the existing situation. Any increased erosion (if any) on the beach would be only short term and not be measurable or significant. If any mechanical intervention is desired to accelerate beach recovery, Council has the means to undertake beach scraping.

Further with regard to (b)(i), there are no significant end effects (increased erosion on adjacent land) expected as a result of the proposed works, as the proposed seawall is located adjacent to an existing seawall or rocky headland, and replacing an existing seawall.

With regard to (b)(ii), Council would be responsible for maintaining the proposed works. To maintain the proposed works, it would be necessary for a suitably qualified and experienced coastal engineer to undertake an inspection after severe storms that expose the works, and advise on any required remedial action. Potential maintenance activities would include:

- Inspection of the wall after significant coastal storms. This would comprise inspection of the seaward side for any damage to the concrete structure, gap formation in the piling (where visible), and integrity of weepholes. This would also comprise inspection of the landward side for evidence of the formation of any sinkholes (indicating migration of soil though the wall), wall displacement, and assessment of any wave overtopping damage at the surface.
- Should a significant impact event cause localised damage to the concrete structure exposing reinforcement, the concrete should be locally scabbled and patched with an approved repair mortar. Significant concrete damage is unlikely, with high strength concrete and appropriate cover to reinforcement for a 50 year life proposed to be used.
- Dealing with any gap formation in the piling through either shotcreting from the seaward side (after excavation of sand for access to the gaps as required), on from the landward side (with sand in this case left in place against the gap on the seaward side to act as a “formwork” for the grouting). That stated, the construction procedure would involve hold points to inspect the piling for gaps, to minimise the possibility of gaps occurring in the first place. The construction contract terms would be such that there is an incentive for the contractor to take care with the piling to minimise the potential for gaps, as these defects would be their responsibility to correct and would be inspected by the project engineers.
- If any weepholes were found to be leaking soil they could be filled with concrete. Weepholes would not be necessary for structural integrity of the wall (the wall would be designed assuming limited drainage, with elevated landward groundwater levels, so can be sacrificed if the geotextile sock on the weephole failed).
- Any formation of sink holes on the landward side would be an indication of gap formation in the piling, which could be addressed as described above.
- If significant displacement of the wall occurred, which is not expected, this may be indicative of an anchor failure. To address this issue, it may be necessary to re-drill an anchor. That stated, field testing of anchor performance would be a hold point in the construction procedure, requiring signoff of the project engineers, thus minimising the possibility of sub-standard anchor performance.

As a public authority, Council has a statutory responsibility to maintain both the asset and adjoining land, including the beach. These requirements may be specified in the conditions of consent, with the arrangements outlined in relevant asset management plans.

9.3 *Waverley Local Environmental Plan 2012*

There are no specific coastal engineering issues to address in relation to *Waverley Local Environmental Plan 2012* (LEP 2012). The proposed works are in an RE1 (Public Recreation) zone, for which an objective is to “facilitate and manage public access to and along the coastline for all”. The proposed development maintains and enhances public access along the promenade to the east of the building, and from the promenade to the beach.

9.4 *Waverley Development Control Plan 2012*

The proposed building is located in a “coastal inundation area” in the Waverley Online Mapping Tool. Therefore, based on Chapter B4 of *Waverley Development Control Plan* 2012 (DCP 2012) for “any application for new buildings, significant alterations and/or additions to existing buildings and/or new swimming pools” it is required to submit a Coastal Risk Assessment with the DA, as set out herein.

9.5 Waverley Council *Coastal Risk Management Policy*

The proposed building is located in a “coastal inundation risk area” in the *Coastal Risk Management Policy* (adopted October 2012). Therefore, similar to Chapter B4 of DCP 2012, a “coastal assessment prepared by a suitably qualified expert” is required as part of the DA, as set out herein.

10. CONCLUSIONS

It is proposed to demolish the existing Bronte SLSC clubhouse and to rebuild a new clubhouse over a similar footprint. It is also proposed to rebuild the existing seawall seaward of the clubhouse to provide greater protection to the clubhouse from erosion/recession and oceanic inundation (wave runup). These seawall modifications also provide additional promenade space and structures to enhance public circulation around the clubhouse and access (including disabled access) to the beach.

The current promenade seaward of Bronte SLSC has an inadequate crest level to prevent significant wave overtopping in severe storm events, with projected sea level rise exacerbating this issue into the future. The seawall at Bronte Beach was constructed around 1914-1916, so is well beyond its design life.

It has been assumed herein that the proposed clubhouse would be founded on bedrock (that is, have footings extending down to bedrock). With an appropriately designed and constructed seawall in place seaward of the clubhouse, the foundations of the clubhouse may be designed based on conventional structural and geotechnical considerations, and would not require any coastal engineering input.

Features that have been adopted to reduce the risk of wave runup causing damage to the clubhouse have been listed in Section 4.3.

It is expected that the seawall would comprise secant piles founded into bedrock, with a reinforced concrete wall above, plus discrete-piled reinforced concrete ramps and steps extending seaward of the secant piled wall. The alignment of the proposed seawall is about 2m seaward of the existing seawall. This provides separation from the coastal walkway and lifesaving beach access, enhances access to the beach for lifesaving equipment (with an enhanced access ramp compared to the existing ramp), enhances public access to the beach (with enhanced stair and ramp access compared to the existing stairs and ramp), provides disabled access, and provides enhanced protection to the clubhouse from wave runup.

The detailed design of the seawall would be prepared as an integrated coastal, structural and geotechnical engineering investigation. Including a wave return shape at the crest of the seawall would assist in reducing the volume of wave overtopping in severe storms. It is recommended that Council maintains sand levels seaward of the clubhouse at 4m AHD, as the seawall and steps would be more effective in reducing wave overtopping in a severe storm if that was the case, to be confirmed as part of detailed design.

A coastal engineering design life of 70 years has been adopted for the proposed development. Without the seawall, the proposed SLSC clubhouse would be expected to be undermined in a severe coastal storm over its design life.

There does not appear to be a long term trend of recession due to net sediment loss at Bronte Beach, so long term recession of the beach in the future would be expected to be related to sea level rise effects only. This recession would occur whether the proposed seawall is carried out or not. Sea level rise values of 0.18m at 2050, 0.37m at 2080 and 0.55m at 2103 (relative to 2023) have been considered herein for the purpose of illustrating the most likely future behaviour. Based on Bruun (1962), this would lead to 9m, 18.5m and 27.5m of beach recession respectively.

The median beach width (averaged over the long term) seaward of the seawall at Bronte SLSC is about 50m at present. This is projected to reduce to about 39m at 2050, 30m at 2080 and 21m at 2103, accounting for long term recession due to sea level rise and the reduction in beach width of 2m due to the new seawall. There is thus the expectation of a useable beach width seaward of Bronte SLSC at 2103, on average.

If sea level continues to rise beyond the design life, and beach recession occurs as projected based on Bruun (1962), and there are no beach nourishment intervention works undertaken, there may be a point mid next century when there is insufficient beach width to maintain sandy beach activities, on average. The public and lifesaving benefits of the new seawall 2m further seaward, may be considered to outweigh its minor impacts on beach width beyond its design life and over a relatively short length of 50m within the 280m length of beach. There would be ample opportunity well into the future and beyond the design life of the proposed works, as part of any future development application, to assess the feasibility of maintaining the clubhouse at its proposed location. These decisions would have to be made in the context of the wider beach amenity of Bronte Beach, that are unrelated to the proposed works.

It will be necessary to design the walls of the SLSC clubhouse to resist wave and hydrostatic forces, as advised by a coastal engineer as part of detailed design. With use of reinforced concrete, this could feasibly be achieved. Other measures that could be considered (where practical) to reduce the risk of inundation damage on the ground floor were listed in Section 7.

The runup process over seawalls (with wave returns), steps, ramps, barriers and perimeter walls is too complex to define analytically. Although it is reiterated that the measures outlined in Section 4.3 and Section 7 are likely to achieve an acceptably low risk of damage to the clubhouse from coastal inundation over the design life, to demonstrate this then physical modelling (in a wave flume or basin) could be considered as part of detailed design. Physical modelling would also give the opportunity to refine these features.

The proposed development satisfies the coastal engineering matters in *State Environmental Planning Policy (Resilience and Hazards) 2021*, Section 27 of the *Coastal Management Act* 2016, *Waverley Local Environmental Plan 2012*, Chapter B4 of *Waverley Development Control Plan* 2012, and the Waverley Council *Coastal Risk Management Policy*, as has been outlined.

In particular, the proposed development significantly reduces the risk of coastal hazards (in particular from wave runup) on that land, and is unlikely to cause any increased risk of coastal hazards on any other land, with adjacent areas already having seawalls. The proposed works would pose no significant threat to public safety, as they would be designed to withstand an acceptably rare storm over a 70 year design life, and are less of a threat to public safety than the do-nothing scenario. The proposed works also substantially reduce public safety risks due to wave overtopping of the seawall compared to the existing situation.

The beach would be expected to naturally accrete and be restored seaward of the proposed works after storm events, and no differently to the existing situation. Any increased erosion (if any) on the beach would be only short term and not be measurable or significant. If any mechanical intervention is desired to accelerate beach recovery, Council has the means to undertake beach scraping.

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