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Waverley Council

Application No: 28/02/2024

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REPORT

Bronte SLSC Redevelopment Seawall and Related Elements Detailed Design

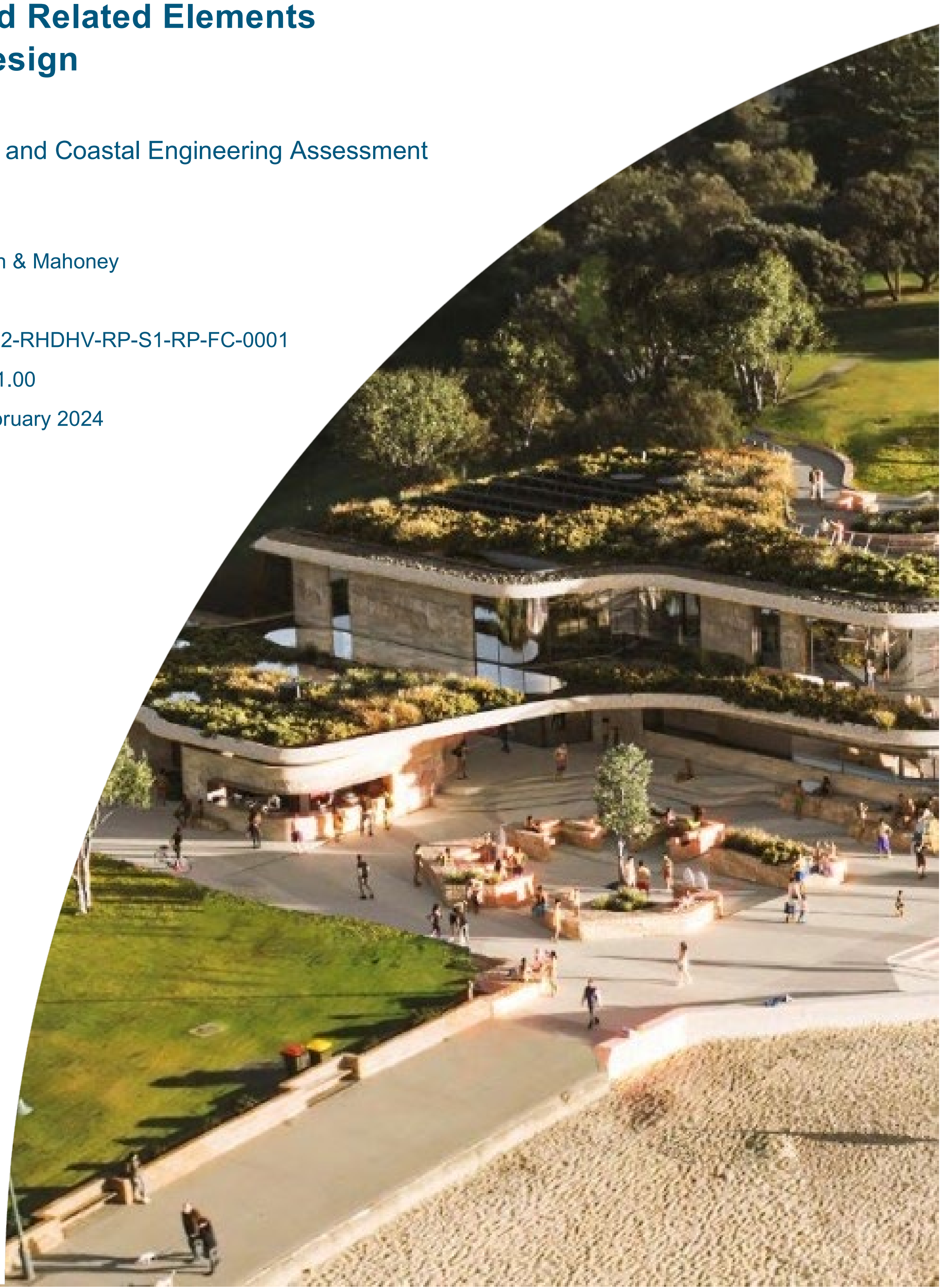
Concept Design and Coastal Engineering Assessment Report

Client: Warren & Mahoney

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

Status: S1/P01.00

Date: 28 February 2024



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Project name:	Bronte SLSC Seawall Upgrade
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Classification
Project related

Revision	Date	Description	Prepared	Checked	Approved
01	05/12/2023	Preliminary Draft	JG	GPB, SH	GPB
02	20/12/2023	Draft	JG, GPB	GPB	GPB
03	16/02/2024	Final Draft	JG, GPB	GPB, GWB	GPB
04	28/02/2024	Final	JG, GPB	JG, GPB	GPB

Executive Summary

Background

Architects Warren and Mahoney (W&M) have developed a design for the upgrade of the Bronte SLSC buildings. The redevelopment involves the construction of coastal protection works to protect the SLSC over its 70-year design life[1]. There is no certified Coastal Zone Management Plan (CZMP) or certified Coastal Management Program (CMP) in place for Bronte Beach. Since a CMP is not yet in place, the consent authority is the Sydney City Eastern Planning Panel (SCEPP). It is noted that the Bronte SLSC Building Operational Management Plan (BOMP) is a source document and a tool for managing the coastal hazards. The BOMP would be updated to reflect the findings of the Concept Design and Coastal Engineering Assessment report.

Royal HaskoningDHV (RHDHV) is assisting to develop a Concept and Detailed Design for the seawall upgrade fronting the SLSC. Further geotechnical investigation is underway at the time of writing, to advise on parameters for foundation design[2]. While desktop assessment would permit a review of wave runup and overtopping, and wave loading, sufficient for Concept Design and Development Assessment, physical modelling is proposed to optimise these parameters for detailed structural design.

The seawall and promenade were built as part of a Bronte Beach Rehabilitation Plan between 1914 and 1917. There is little information on the existing seawall fronting the SLSC. It is over 100 years old and therefore well beyond its design life[3]. In balancing the requirements for the location of SLSC, promenade and beach access enhancements, erosion protection, and reduction of wave overtopping over a 70-year life, RHDHV's recommendation is to construct a new 60m long seawall segment fronting the SLSC.

Risk and coastal hazards

All available literature addressing coastal processes, coastal protection works, and coastal management within the Bronte foreshore has been considered. A site visit was conducted, and Basis of Design (BoD)[4] process elements were outlined. A risk-based assessment accounting for design life, design storm events, and acceptable damage has been used to develop a design philosophy for the seawall structure.

Having regard to the suite of coastal hazards covered in the Coastal Management Act 2016, relevant for consideration at the Bronte SLSC site are beach erosion (including the effects of sand slope instability), shoreline recession, and coastal inundation[5].

The design storm erosion demand for Bronte Beach for a 100-year ARI storm event is adopted as 250m^3/m above AHD. As this exceeds the beach-full sand volumes, it follows that the seawall is

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[1] A 70-year coastal engineering design was adopted for the proposed development as requested by Waverley Council. The means that the proposed clubhouse would be designed to withstand coastal erosion and wave overtopping events with an acceptably low risk of damage over a 70 year life (Horton Coastal Engineering , 2023).
[2] At the time of finalising the Concept Design report the additional geotechnical site investigation work by JK Geotechnics had been completed. A preliminary statement on the additional work and findings was issued on 20 February 2024 (JK Geotechnics, 2024). The full report would follow in mid-March, to be documented as part of the Detailed Design report for the proposed new seawall. Refer Section 4.4.
[3] Horton Coastal Engineering reports that a design life for concrete structures, consistent with Australian Standards, ranges between 40 years and 60 years (Horton Coastal Engineering , 2023)
[4] A BoD sets the key criteria for developing a design, in this case covering matters such as design life and event, available survey, geotechnical information, and coastal hydraulic parameters. The BoD in the Concept Report would be further developed in the Detailed Design report.
[5] The remaining hazards listed in the Coastal Management Act 2016 as (i) coastal lake or watercourse entrance instability, (ii) coastal cliff or slope instability, (iii) tidal inundation; and (iv) erosion and inundation of foreshores caused by tidal waters and the action of waves, including the interaction of those waters with catchment floodwaters, are not relevant to the site.

necessary to limit erosion into Bronte Park. It is noteworthy that the June 2016 event (3-7/6/16), nominally denoted as 20–30-year ARI event in the Sydney region, eroded around 60m^3/m or potentially up to a maximum of 90m^3/m. Baird report (Baird, 2016) suggested that Bronte Beach undergoes large episodic erosional events due to coastal storms, but then recovers and remains relatively stable in intervening periods. Initial beach recovery is rapid, occurring over days to weeks, but full recovery, not unlike most other NSW beaches, could take months to years[6]. Available photos of the beach taken immediately after the storm on 6/6/19 and then 12 days later on 18/6/19 are reproduced below (courtesy of David Finnimore, Bronte SLSC). While these do capture different areas of the beach and it is well understood the southern area was more severely affected than the northern, the principle of early rapid recovery is indicated.

Bronte Beach 6/6/19

[6] *By way of example for the site, Nearmap imagery indicates substantial recovery of Bronte Beach within 1 month of the June 2016 storm event (Section 5.2.1.3).*

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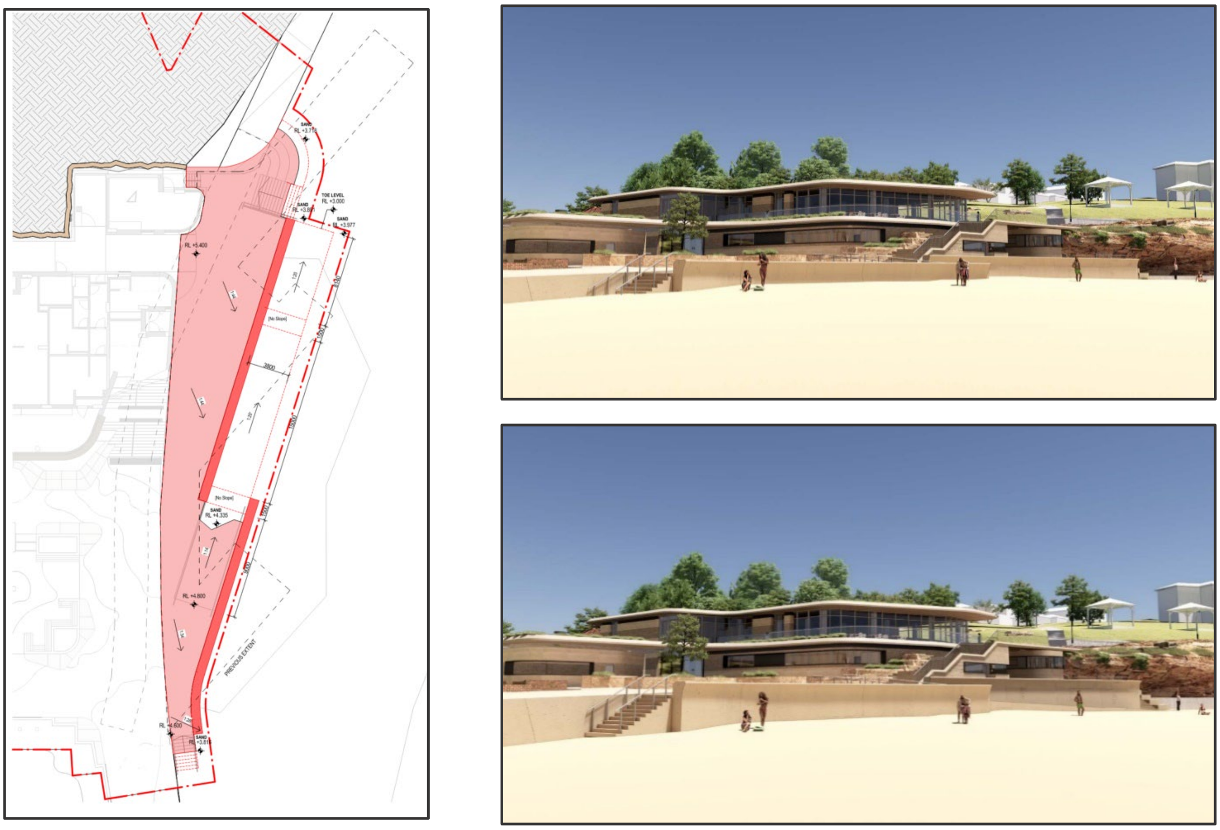
Bronte Beach 18/6/19

No recessional trends have been observed in the data. However, recession in the future is predicted to occur as a consequence of sea level rise due to climate change. To investigate erosion hazard into the future, Baird ran the 100-year ARI design storm event through their verified SBEACH model for various sea level rise scenarios. 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.

Baird also applied SBEACH to predict design scour levels under present and future climate conditions. Applying prudent techniques, Baird recommended design scour reducing from RL2.9m AHD today (2016), to a minimum of RL0.35m in 2100. The analysis above makes no allowance for bedrock which, based on available borehole drilling behind the existing seawall, may well be present above the predicted eroded beach levels. A preliminary statement regarding additional geotechnical site investigations, underway at the time of writing, indicates weathered sandstone bedrock between RL0.0m AHD and RL-0.9m AHD in the vicinity of the proposed new seawall.

Choice of new concrete seawall

The constrained space at Bronte Beach rules out the feasibility of rock revetments, making a new concrete seawall the only practical choice. RHDV's design proposal, shown below, involves constructing a new seawall structure around the outer perimeter of planned access elements, including the promenade, ramps, bleachers, and steps.



Proposed new seawall concept. Plan outline showing all-ability access ramp, steps/bleachers and seawall with deflector (dashed black outline depicts an earlier layout, since moved closer to the SLSC). Design montage showing proposed new seawall as viewed from the beach for typical accreted beach levels (top, beach RL4m AHD), and for a heavily depleted beach (bottom, beach RL3m AHD. The proposed crest level of the seawall deflector is RL5.4m AHD for the southern segment above the beach, and RL5.8m AHD for the northern segment above the ramp.

The recommended seawall structure incorporates a secant pile design which involves alternating small diameter reinforced and larger diameter unreinforced concrete piles, overlapped in their plan position, acting as a barrier to coastal erosion and retaining the promenade and SLSC[7]. Over the past 15 years secant pile seawalls have been successfully used in NSW for stabilising sandy beach shorelines including at North Steyne (Manly), South Curl Curl, Trial Bay[8] (Arakoon) and Kingscliff[9] (Tweed Shire).

Wave overtopping and loads

Due to its low crest level, the existing seawall is exposed to overtopping in storms. Based on observations and various assessments[10], the current promenade is unsafe for pedestrians during severe coastal storms. Best-practice desk top calculations of wave overtopping are presented based on procedures set out in EuroTop (2018). For the case of no deflector:

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[7] Overlapping secant pile design avoids sand leakage problems associated with contiguous (closely spaced but not water tight) circular concrete piles, and problems with corrosion and toe penetration associated with conventional steel sheet piles.
[8] Under construction.
[9] Two installations here, the first protecting the Cudgen Headland SLSC constructed 2010, and the second adjoining the SLSC immediately to the north constructed 2017.
[10] Assessments made in (ARUP, 2016), (Horton Coastal Engineering , 2023) and by RHDHV for this report (Section 5.3.3).

- for a 1-year ARI event at planning date 2093, there is a high likelihood of wave overtopping (estimated up to 0.2L/s per m) being a hazard to pedestrians at the new seawall crest; and

- predicted overtopping at 2093 under a 100-year ARI event (estimated 0.5-41L/s per m, depending on back-beach sand levels) could potentially cause structural damage to the promenade and the SLSC building [11].

Overtopping quantities are estimated to reduce by approximately 80% with the inclusion of a typical wave deflector (32-degree deflection from vertical, deflector length 0.9m). This indicates that for planning date 2093, with the inclusion of a wave deflector, overtopping for the 1-year ARI event would not be hazardous to pedestrians, and under a 100-year ARI event would not cause structural damage to the promenade but potentially still lead to damage to the SLSC building. Mitigation for the SLSC building damage would involve strengthened building construction methods such as the use of reinforced concrete external walls at promenade level.

Total induced hydrodynamic wave loads are estimated to range between 50 and 100kN/m for storm events up to 500yr ARI occurring at planning date 2093. These loads would be accounted for in the Detailed Design of the new seawall.

Physical modelling

Physical modelling is proposed in the next phase of the seawall design, to provide further information for seawall design development. The following would be achieved from physical modelling:

(i) Review and refinement of the incident wave and water level conditions at the seawall.
(ii) Review and refinement of the wave deflector configuration.
(iii) Review and refinement of wave runup and overtopping rates and volumes, to consider the safety of overtopping flows, and drainage requirements for overtopped flows.
(iv) Review and refinement of wave loads to achieve an optimised structural design to the seawall and its deflector elements. Uplift loads on the deflector would be of particular interest.
(v) By inspection and video in the flume, to gauge wave overtopping trajectories and water bore behaviour with respect to potential loading of the walls and windows/door openings at the new SLSC building.

Recommendations in the Coastal Assessment involving quantification of wave overtopping would be reviewed and updated if necessary, following the completion of physical modelling. 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.

At the time of writing, it is expected that physical modelling would commence in early March 2024 and be completed by late April 2024. [12]

[11] *Available guidance on what levels of overtopping cause damage to structures at or immediately behind the seawall crest are discussed in Section 5.3.3. Mitigation would be achieved with the aid of a wave deflector at the seawall crest. The desk-top calculations of overtopping rates and quantities would be reviewed in the proposed physical modelling.*
[12] *The proposed scope for the physical modelling is outlined in Section 7. The findings of the physical modelling would be documented as part of the Detailed Design report.*

Detailed Design

The Detailed Design would be completed following the additional geotechnical investigation, physical modelling investigation, and dedicated maritime structural design development for the coastal protection works.

Coastal Assessment

The Coastal Assessment sets out a review of the proposal in relation to the Coastal Management Act 2016, State Environmental Planning Policy (Resilience and Hazards) 2021, Waverley Local Environmental Plan 2012; and Waverley Development Control Plan 2022. Coastal assessment responses are provided to inform the planning process for Development Consent. Key findings made in the Coastal Assessment are summarised below. Note that the BOMP, which forms part of the DA package, would be updated to reflect the Coastal Assessment.

Access and amenity

- It is considered that the works would not, over the life of the works, unreasonably limit public access to or the use of the beach.
- The proposal facilitates enhanced public and lifesaving access between the beach and the SLSC area and promenade, by providing a new ramp and steps, and bleachers.
- The proposal improves the public amenity of the Coastal Walk and Bronte Park in the immediate vicinity of the upgraded SLSC building. The promenade spaces catering for longshore pedestrian access are slightly widened, assisting with through traffic. With the north down ramp alignment and new steps and bleachers at the northern end, beach users are directed to the north so improving access to the only and safest area on the beach for the lifeguards to put the flags up.
- The proposed works are located as far landward as possible and comprise structural elements common for coastal protection works, e.g., secant pile wall and drop-down beam. To minimise the impact on visual amenity, the proposed seawall is to include of a deflector to reduce its crest level and therefore seawall height, a deep drop-down beam would be provided to limit the visible upper portion of the secant pile wall at times of low beach levels, and the concrete would be coloured to match the beach sand. Managing the visual impact this does not affect the design integrity of the seawall performance.
- The BOMP would oversee the management of Access and Amenity issues.

Protection from coastal hazards and associated safety risks

- The existing seawall, which is beyond its design life, could not be relied upon to protect the SLSC building. The proposed coastal protection works comprising a secant pile wall, drop-down beam, slabs, and discreet CFA piles, when fully detailed, would be capable of preventing undermining of the SLSC building.
- The proposed coastal protection works, over the life of the works, would not be expected to pose or be likely to pose a threat to public safety, in respect of the beach erosion/ shoreline recession hazard.

- The consent authority can be satisfied that a design solution, in combination with operational measures, could be found to ensure that the proposed works would not, over the life of the works, pose or be likely to pose a threat to public safety due to the coastal inundation hazard[13], but the design solution requires further development as part of the Detailed Design which would include physical modelling.

- 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 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[14].

Interaction with coastal processes

- The beach is expected to naturally accrete and be restored seaward of the proposed works after storm events, and for all intents and purposes no differently to the existing situation. Increased erosion on the beach (if any) would be only short term and not be measurable or significant. No end-effects are expected as the works would merge with the existing seawall or bedrock cliff. No sand from the beach would be impounded behind the seawall, thus the structures would be of no consequence to shoreline recession.

  In spite of the findings above, if any mechanical intervention is desired to accelerate beach recovery, Council has the means to undertake beach scraping. Council owns a posi-track and beach rake which regularly scrapes sand at its beaches to the levels required for beach cleaning, safety, access and after storm events. In large storm events and sand washouts, Council hires excavators for moving sand and cleaning up debris.

- The proposed works are not expected to alter coastal processes into the future.

Engineering and maintenance

- The consent authority can be satisfied that the proposed works would be engineered to withstand the current and projected beach erosion/ shoreline recession for the 70 year design life of the works, having regard to the Basis of Design (developed to concept level herein), the peer review process (commenced), and the coastal and maritime engineering advice based on Baird (2016), and further developed by Horton (2023) and RHDHV for this report.

- 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 and maintenance plans. At the time of writing the maintenance guide in the BOMP had been updated.

The proposed development significantly reduces the risk of coastal hazards, in particular from potential failure of the existing seawall fronting the SLSC and 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 or protected by bedrock features.

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[13] *Refer to comparison of mean overtopping rate measurements with accepted thresholds set out in Section 5.3.3.*
[14] *Refer to BOMP for the management of public safety and beach closures.*

Waverley Council Local Planning and Policies

- Requirements in Waverley Local Environmental Plan 2012, Waverley Development Control Plan 2022, and Waverley Council Coastal Risk Management Policy are satisfied.

Peer Review

Waverley Council has initiated a peer review of the Coastal Report prepared by RHDHV. This review is being undertaken by the UNSW Water Research Laboratory (WRL). WRL 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, and any operating guidelines would be amended in the BOMP.

Co-ordination with other disciplines

Co-ordination has involved TTW as the SLSC building Structural Engineer, regarding management of wave loads and interface of the landside promenade (TTW) with the promenade extension into the new seawall structure (RHDHV). This report would be provided to the façade specialist (Prism Facades) regarding management of wave loads at windows openings at promenade level.

Synthesis and Conclusion

The Concept Design investigations are based on an updated masterplan developed in discussion with W&M and the SCEPP. The location of the SLSC is set having regard to the Planning and Regulatory Framework (as the building cannot be located anywhere else due to the restriction in the Bronte Park Plan of Management), but also based on the life-saving requirements with adjacent beach access. The seawall and ramp structure needs to be located on the seaward side of the culvert to allow accessible access to the beach. The existing promenade has been taken into account, with the new seawall arrangement, separating the life-saving activity from the public walkway to increase public safety.

Basis of Design [15] 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.

Deliberations within the design team, including discussions with the Peer Reviewer, have determined the structural concept involving a concrete slabs/ shells, fully protected by a row of secant piles. A single ramps and steps, developed to satisfy the functional requirements for the project, are optimally accommodated in the structural concept, which is subject to Detailed Design development.

The proposed seawall upgrade essentially comprises a vertical piled structure capped with a wave deflector. The deflector profile would be confirmed in the subsequent design stage. Physical modelling, to take place as part of the Detailed Design, would refine the seawall sectional configuration, specifically the crest level and deflector profile. At the time of writing, it is expected that physical modelling would commence in early March 2024 and be completed by late April 2024.

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[15] *The Basis of Design (BoD) documents the principles, assumptions, rationale, criteria, and considerations used for calculations and decisions required during design.*

The Coastal Assessment sets out a review of the proposal in relation to the Coastal Management Act 2016, State Environmental Planning Policy (Resilience and Hazards) 2021, Waverley Local Environmental Plan 2012; and Waverley Development Control Plan 2022. Coastal assessment responses are provided to inform the planning process.

The CMP vision for Sydney's Eastern Beaches including Bronte calls for resilience through integrated and co-ordinated planning and management that protects and improves its unique cultural, biodiverse, and economic values now and for the communities, development, and climate changes of the future. With the completion of Stage 1 of the CMP, it is the submission of Royal HakoningDHV that the coastal engineering design and Coastal Assessment reported herein are consistent with this vision pending finalisation of the CMP.

Based on the investigations undertaken for the Concept Design and Coastal Engineering Assessment for the Bronte SLSC Redevelopment, Seawall and Related Elements project, the consent authority can be satisfied that the requirements of the relevant legislation are suitably addressed with regard to coastal engineering matters, consistent with a permission for Development Consent.

Table of contents

Table of Tables

Table of Figures

Appendices

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 Waverley 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

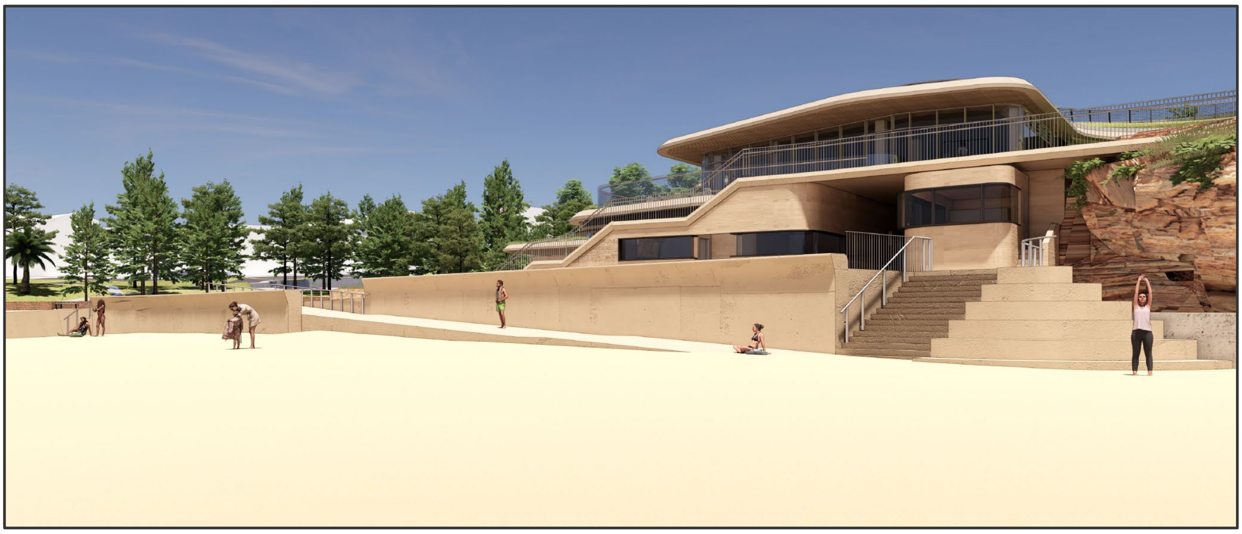

Figure 1-2 Architectural visualisations for general arrangement for revised DA showing seawall and related elements. Subject to design development

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**). The proposed seawall protects the promenade and buildings from beach erosion and coastal inundation due to wave runup and overtopping.

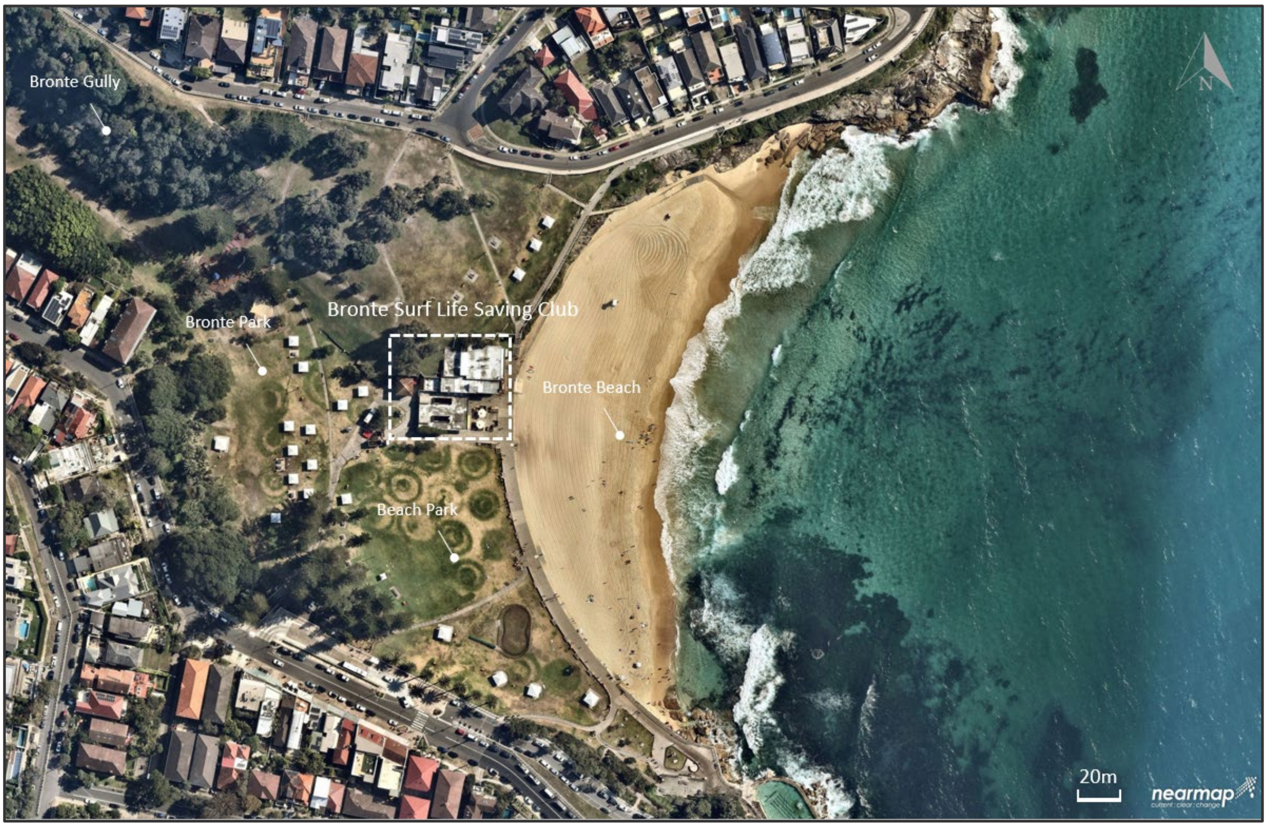

Figure 1-3 Aerial photo of the project site (source: Nearmap dated 03 October 2023)

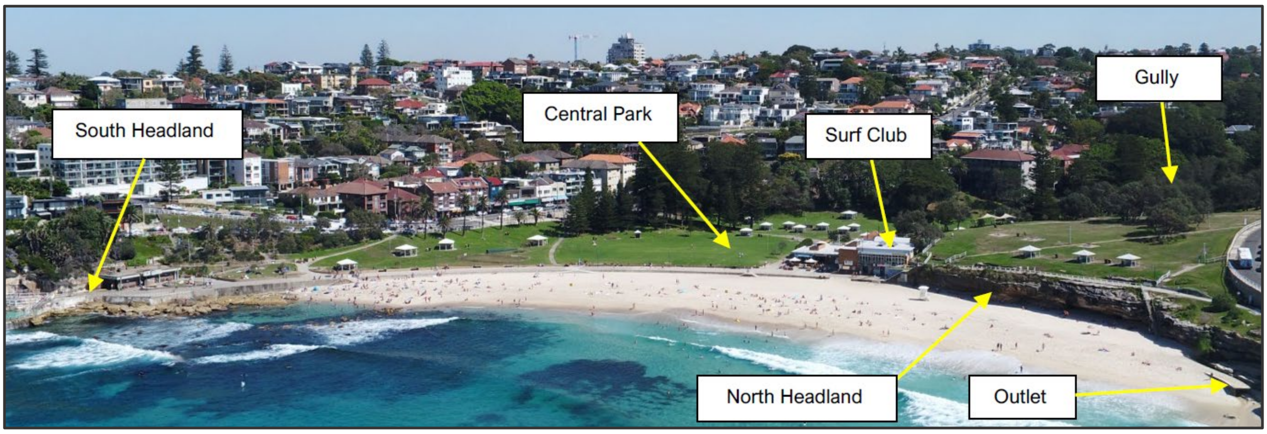

Figure 1-4 Site aerial elevation (AssetGeoEnviro, 2022)

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 natural rock 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 potentially 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, has no parapet or wave deflector/ return wall, 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. RHDHV 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 the 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
(xv) Meetings and project management

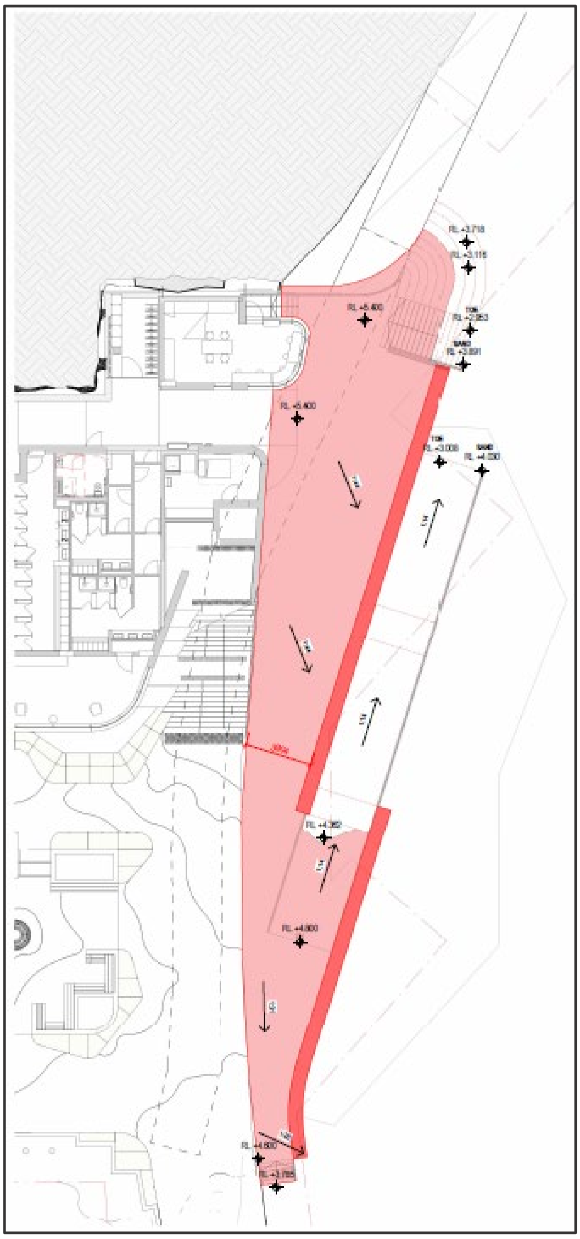

Figure 1-5 Extent of seawall and related element works for Detailed Design by RHDHV. Subject to design development

Stage 3 – 50 and 100% DD

(xvi) Physical modelling
(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 the project moves into Stage 3 and progresses into the Detailed Design phase, additional reports would 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
SCEPP	Sydney City Eastern 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 Average Recurrence Interval (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 wave runup levels, and wave 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 is 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 (ARUP, 2016). 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 Bronte Park, 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 in the present day, to 0.35m AHD at 2100 (details of the SBEACH modelling study are presented in **Section 5.2.3**). While in RHDHV's experience these scour levels appear to be elevated for an open coast beach seawall, it is noted that bedrock is also likely to be elevated and providing a limit on the scour level that can occur.

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.

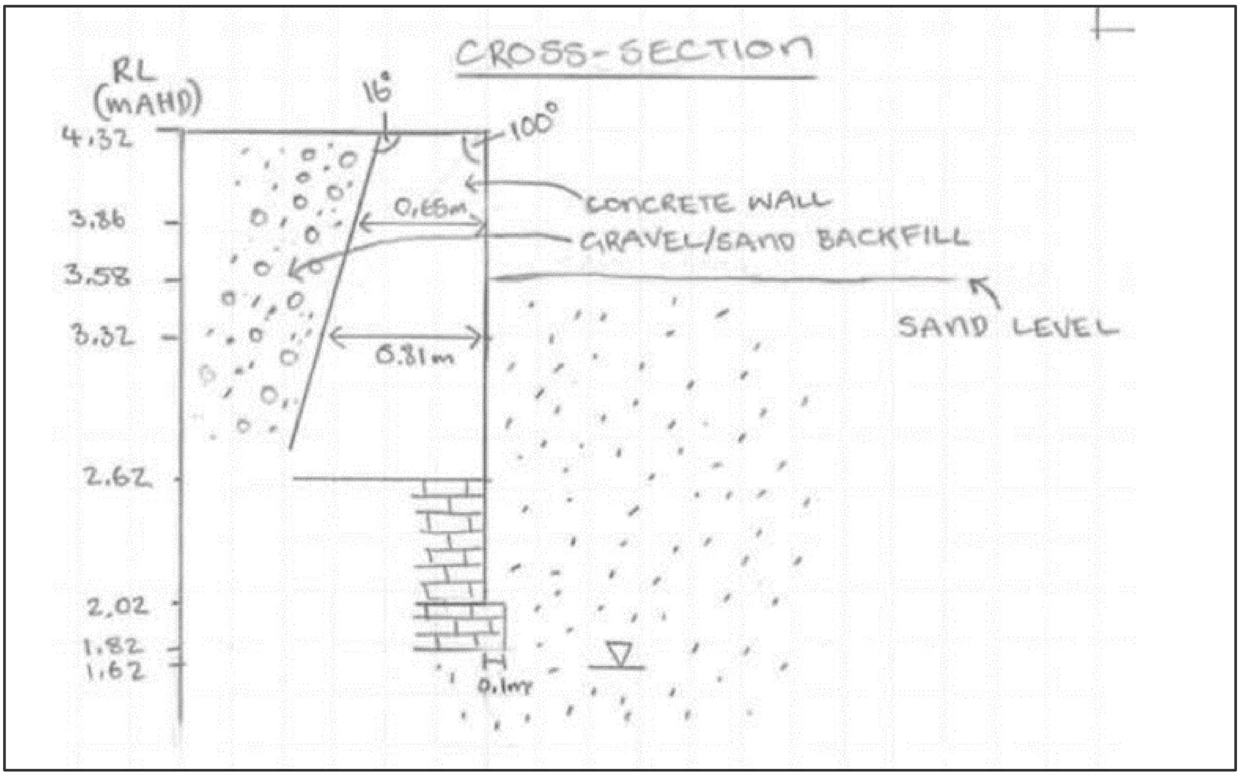

Figure 2-3 Sketch of interpreted representative seawall cross section for the northern-most 100 m of the Bronte Seawall which includes that fronting the SLSC (source: (ARUP, 2016))

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 was found to be unacceptable for extreme coastal loads, and strengthening was considered necessary. ARUP present several options to strengthen the existing seawall including rock or grout bag mattress, cut-off sheetpiles, underpinning, modified geometry (including widening landwards), and replenishment of backfill. Bullnose and/ or a parapet wall at the top of the wall to reduce overtopping was recommended. The mattress was ARUP's preferred option. RHDHV note that the seawall at Bondi Beach which is of similar age 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 a 20 to 40-year ARI event for east facing shorelines in and around Sydney based on discussions with WRL, 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 Reviewer.

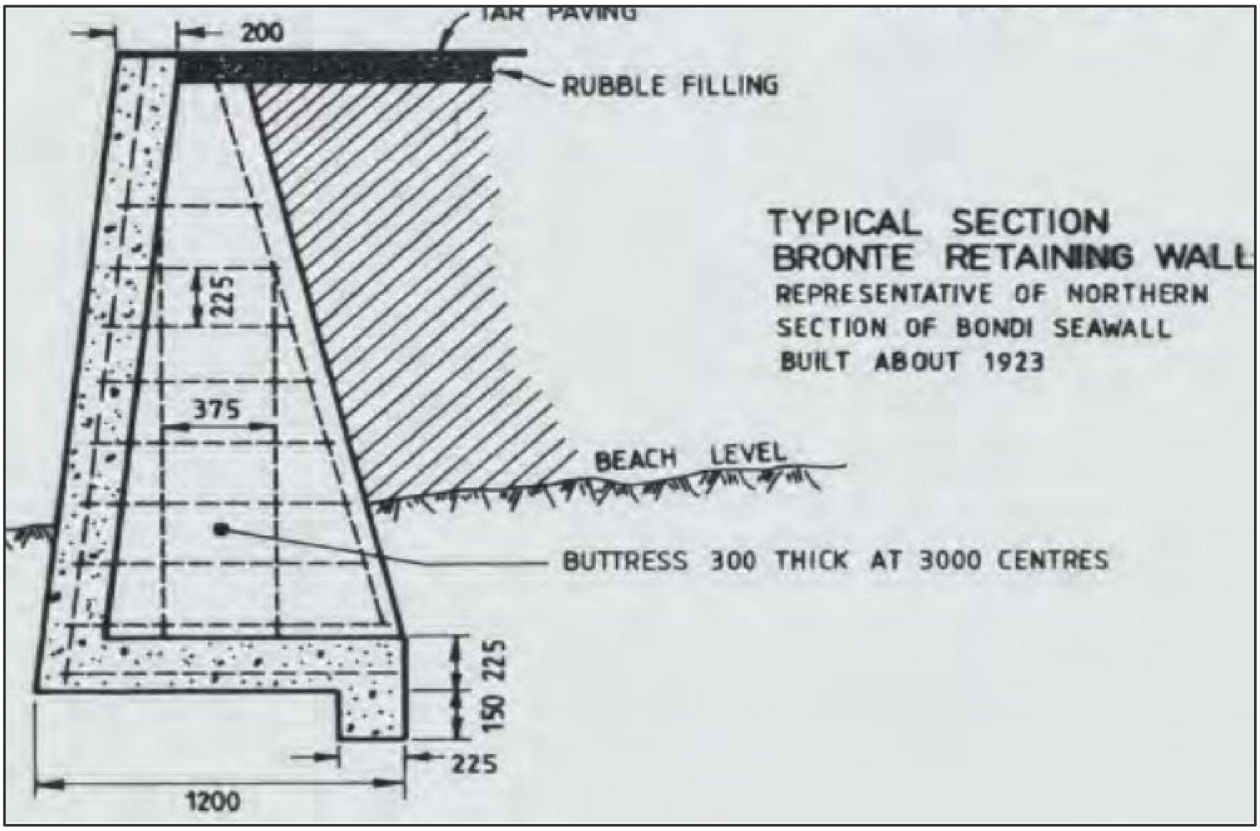

Figure 2-4 Reproduction of Historical Design Drawings of Bronte Seawalls (Source: (WorleyParsons, 2011))

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 SCEPP.

The Bronte SLSC Building Operational Management Plan (BOMP) is a source document and a tool for managing the coastal hazards (Waverley Council, 2024). The BOMP would respond to the operational conditions and risk profile developed for the site. The BOMP would be updated to reflect the findings of the Concept Design and Coastal Engineering Assessment report.

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. It is understood that masterplan design process involved consultations with the Approvals Authority, the SCEPP, and the Design Excellence Advisory Panel.

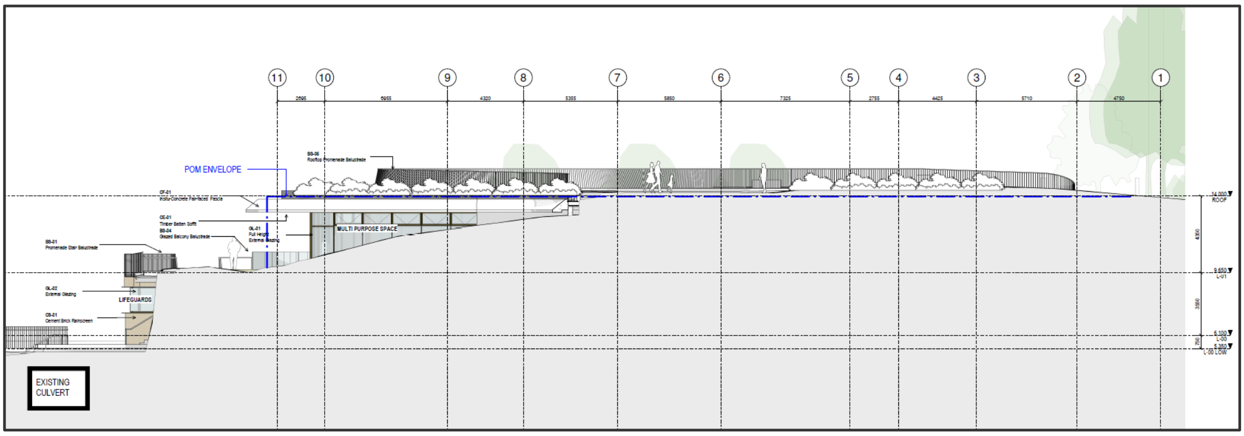

Figure 2-7 Proposed overall north elevation

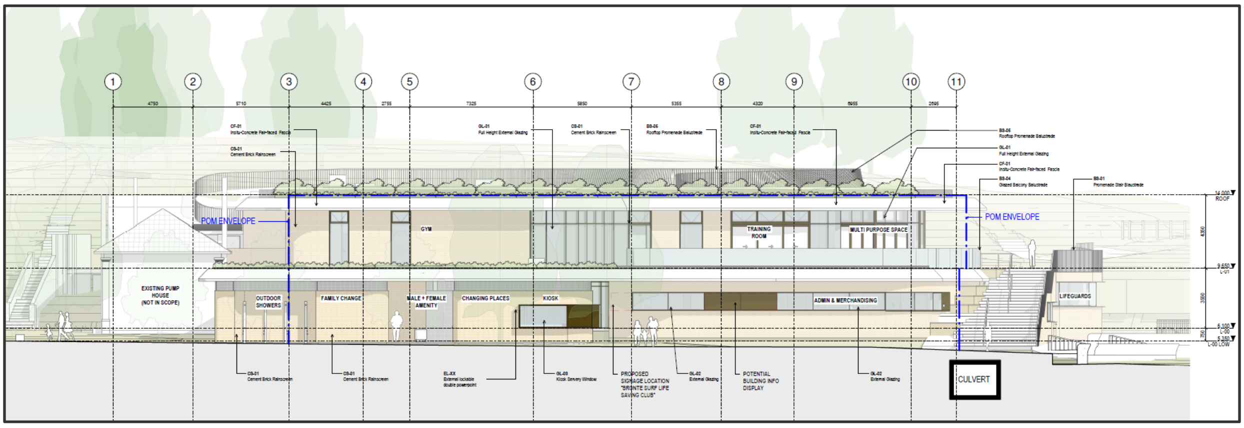

Figure 2-8 Proposed overall south elevation

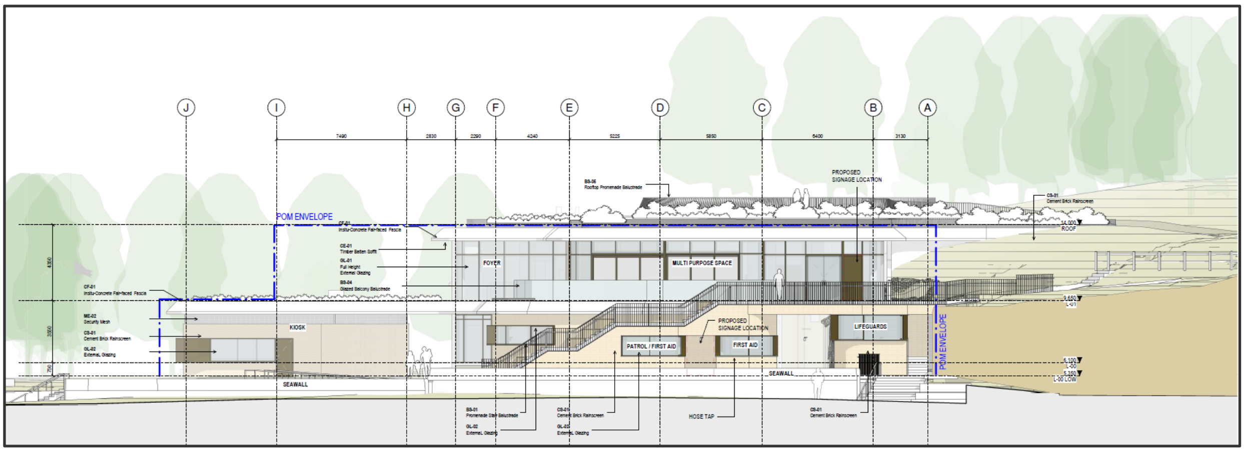

Figure 2-9 Proposed overall east elevation

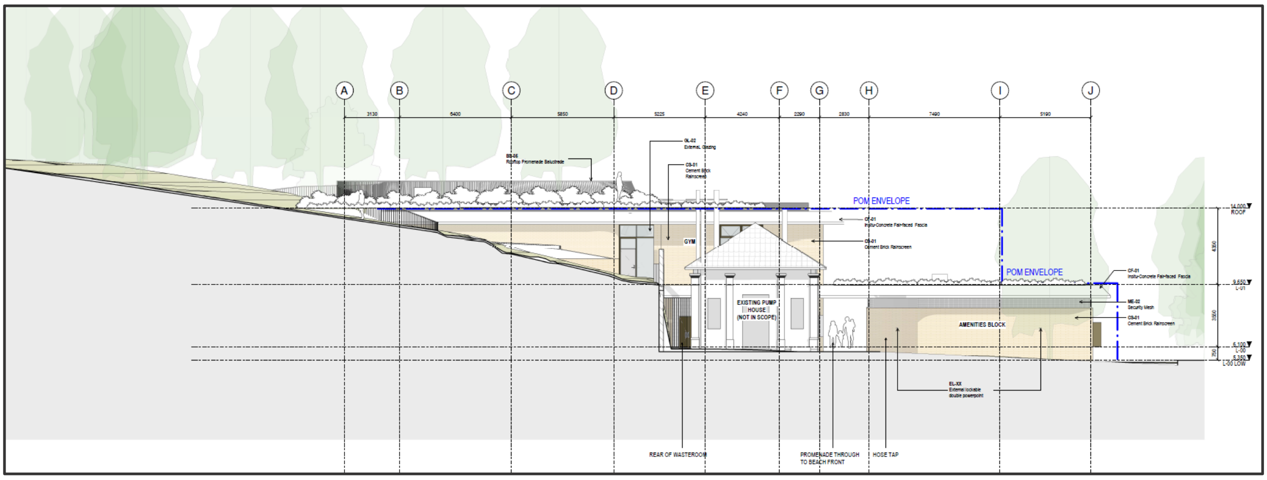

Figure 2-10 Proposed overall west elevation

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 with the information included in **Table 2-1**.

Table 2-1 Information provided to RHDHV

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 Site visit observations

A site visit was conducted on Thursday 07th December 2023. The visit was focused on the seawall and adjoining structures to review the assessments presented by the Horton Coastal Engineering (Horton Coastal Engineering , 2023) and ARUP (ARUP, 2016) reports, to inform an understanding of the condition of the site and an opinion as the designer for the upgraded seawall and related elements.

The areas and the coastal components visually inspected during the site visit are shown in **Figure 3-1**. A photographic commentary record of the site visit is presented in **Appendix A1**.

Figure 3-1 Photographic key plan for Bronte Beach

From a general understanding of the site, 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 between +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 at present approximately equal to Mean Sea Level at the NSW coastline. Directions are in degrees, referenced to 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 contours, and 1 in 40 between the -10m and 0m AHD contours.

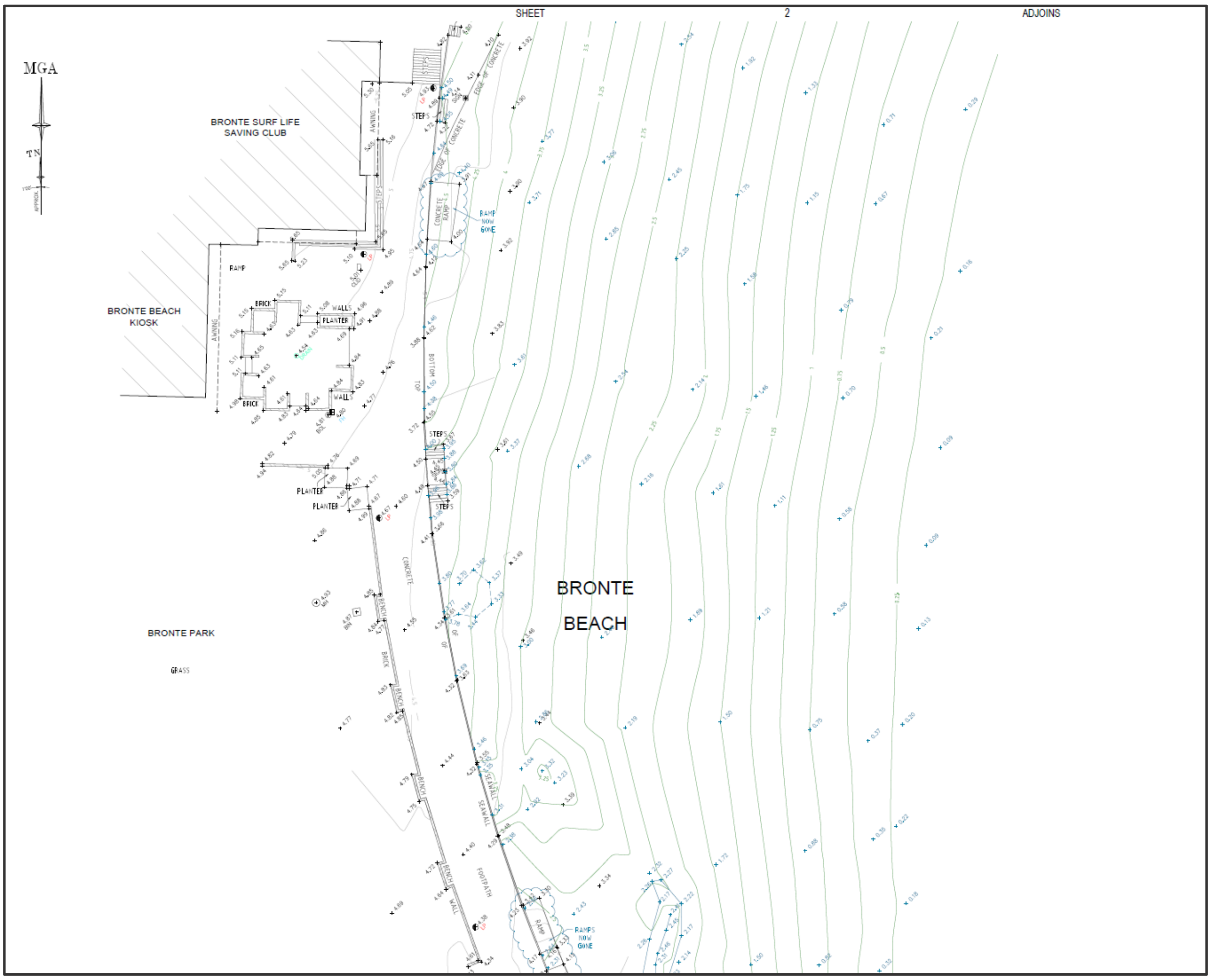

Figure 4-1 Extract from Topographical Survey Drawings. Black text represents survey levels as of May 31, 2016. New levels in blue and new contours in green represent survey results updated on June 16, 2016 (Source: LTS Lockley)

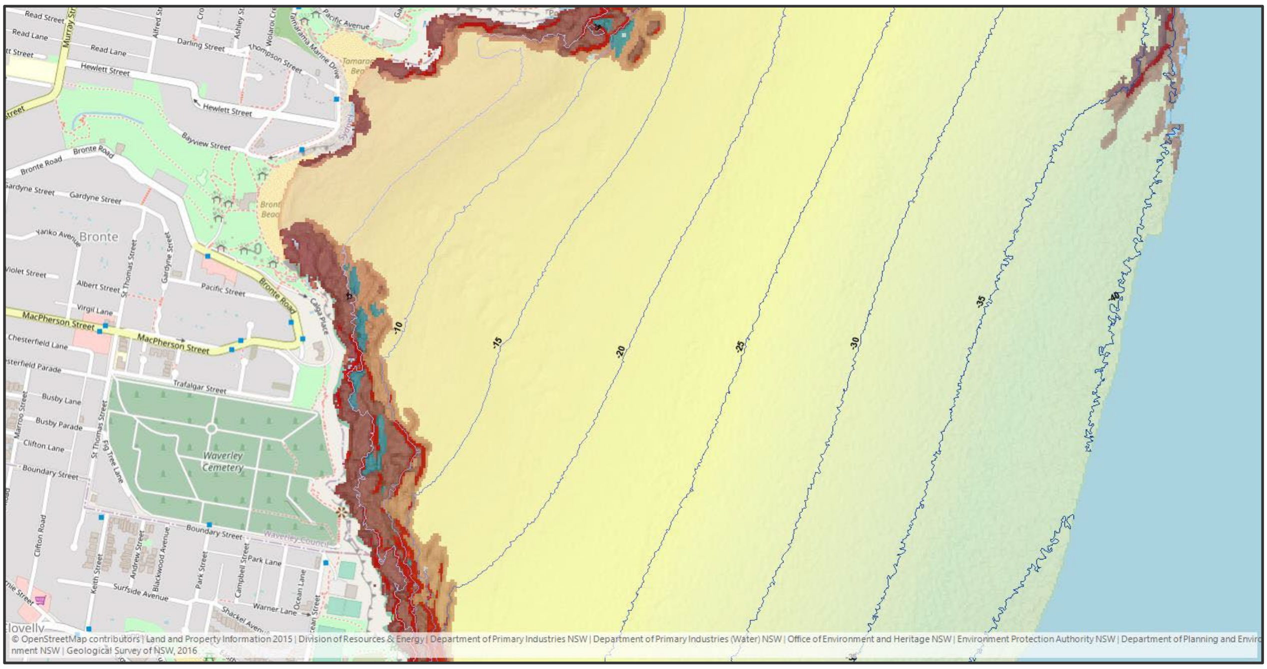

Figure 4-2 NSW Marine Lidar Bathymetry Data 2018 (SEED, The Central Resource for Sharing and Enabling Environmental Data in NSW, https://geo.seed.nsw.gov.au/)

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 seawall appears to be founded on Hawkesbury Sandstone. TP103 exposed the seawall 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 northern 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**).

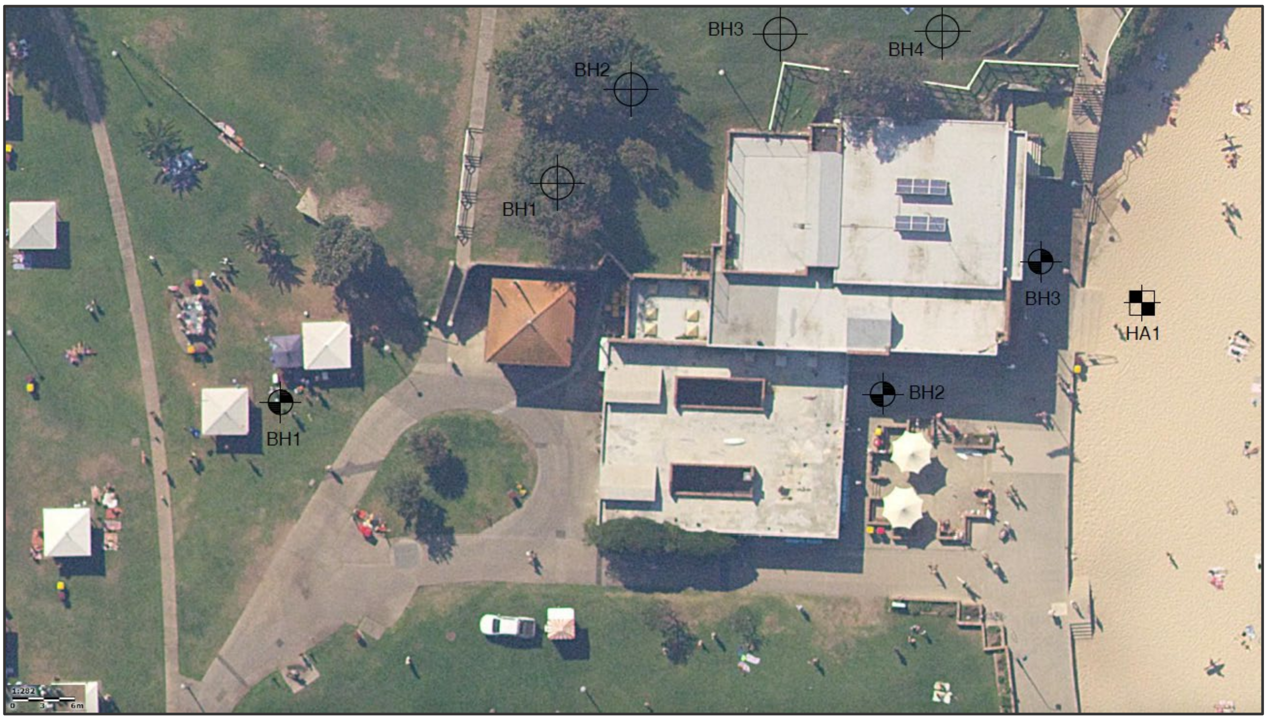

Figure 4-4 Borehole location (AssetGeoEnviro, 2022)

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 when sand only is present.

An additional geotechnical investigation by JK Geotechnics was underway at the time of writing. The geotechnical field work, completed by 20 February 2024, included drilling of 3 (three) boreholes from beach level, extended a minimum 3m into Class III sandstone bedrock to inform the seawall pile design (in accordance with (P. J. N. Pells, 2019)). Five test pit excavations, including 5 (five) adjacent Dynamic Cone Penetrometer (DCP) tests, were also undertaken to further assess and confirm the footing details and foundation materials below the existing seawall and culvert. The location of the boreholes and test pits are shown in **Figure 4-5**. A preliminary statement on the work and findings was issued on 20 February 2024 (JK Geotechnics, 2024), a copy of which is attached at **Appendix A2**. Based on JK Geotechnics' initial review of the field results, weathered sandstone bedrock was encountered in the boreholes at the approximate depths and levels listed in **Table 4-5**. It is noted that inferred bedrock levels based on the DCP test results would be provided in due course, after further review of the field results. A seismic refraction survey has also been carried out to map the bedrock levels along the general alignment of the proposed seawall upgrade and in a cross-shore direction under the beach to help characterise the bathymetry (erodible profile) for physical modelling, however at the time of writing the results of this survey were not available.

The full additional geotechnical investigation report would follow in mid-March, to be documented as part of the Detailed Design report for the proposed new seawall.

Figure 4-5 Testing locations for additional geotechnical investigation. BH03 is jointly located with TP03 (JK Geotechnics, 2024)

Table 4-5 Initial review of weathered sandstone bedrock levels from additional geotechnical investigation (JK Geotechnics, 2024)

Borehole	Approximate surface (i.e., beach) level	Approximate depth to weathered sandstone bedrock	Approximate Reduced Level of weathered sandstone bedrock
[-]	[m AHD]	[m]	[m AHD]
BH101	3.7	4.6	-0.9
BH102	3.6	3.6	0.0
BH103	3.8	4.3	-0.5

4.5 Groundwater

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

Table 4-6 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.

The BOMP requires updates to ensure the ongoing maintenance and inspections necessary for preserving the integrity of the proposed seawall during its design life. Considerations for additional procedures in the BOMP include:

- Waverley Council should consider inspecting the seawall to check its structural integrity, assess if there are any off-site adverse impacts (e.g., beach erosion), identify any risks to public safety, and check that reasonable public access to the beach is being maintained, and

- A criterion for major repair or enhancement works. Special attention should be given to post-significant coastal storms to address any potential damages.

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 4-7 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**). This probability may be unacceptably high for the design.

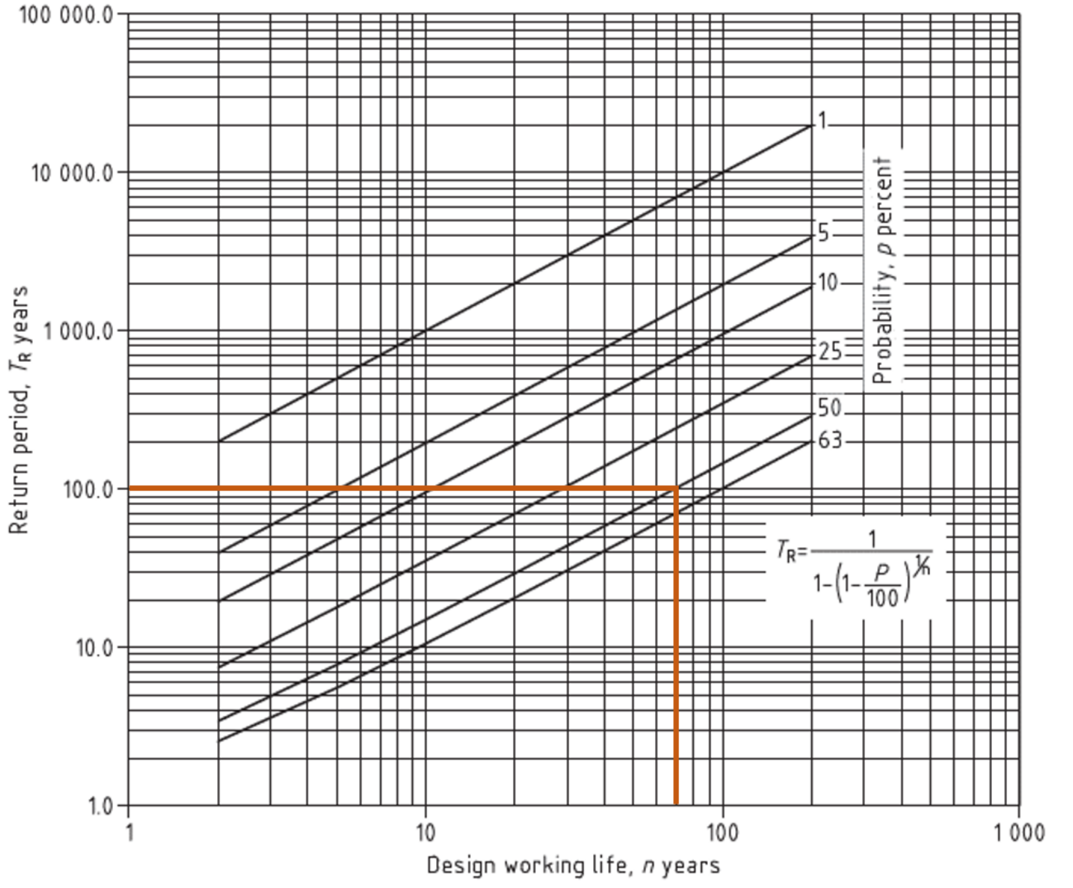

Figure 4-6 Relationship between design working life, return period and probability of an event exceeding the normal average (BRITISH STANDARD, 2016)

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 500-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 metres. 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 4-8: 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. The 100-year ARI water level of 1.5m AHD in the WorleyParsons report (WorleyParsons, 2011) is slightly higher than the 1.44m AHD (NSW Goverment, 2010) value presented in **Section 4.8.4**, however, this 0.06m difference is minor.

When factoring in wave setup, typically calculated as 15% of the unrefracted deepwater significant wave height (Coastal Engineering Research Center, 1984), 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 at Bronte during large wave events. 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) [xvi].

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 4-9: 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 the corresponding value reported by Manly Hydraulics Laboratory (MHL) (Manly Hydraulics Laboratory, 2018) [xvii]. 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**.

[xvi] 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.
[xvii] (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 4-10: 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 4-11: 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

The wave run-up and overtopping calculations presented in **Section 5.3** have used the WaveWatch III spectral wave model to describe the inshore coastal wave and water level conditions, which have been used in turn to develop breaking and broken wave conditions incident at the coastal protection structures. The water levels output from the WaveWatch III spectral wave model includes wave setup. However, it is understood that the water levels adopted in the EurOtop Manual analyses exclude wave setup, which is developed naturally through the process of wave transformation to the shoreline or shoreline structure being considered. It follows that the assessment of wave run-up, wave overtopping, and wave loading presented in this draft report is expected to be conservative with potential double-counting of wave setup. Therefore, the design still high-water level used in **Section 5.3** excludes wave setup and is presented in **Table 4-11**.

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Table 4-12: Design still high-water levels

Design Life	Average Recurrence Interval	Tide Level	Sea Level Rise	Design High Water Level
	[Years]	[m AHD]	[m]	[m AHD]
2050	1	1.28	0.17	1.28
	10	1.39	0.17	1.56
	50	1.45	0.17	1.62
	100	1.48	0.17	1.65
2093	1	1.28	0.66	1.65
	10	1.39	0.66	1.99
	50	1.45	0.66	2.05
	100	1.48	0.66	2.11
2100	1	1.28	0.72	2.00
	10	1.39	0.72	2.11
	50	1.45	0.72	2.17
	100	1.48	0.72	2.20

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.

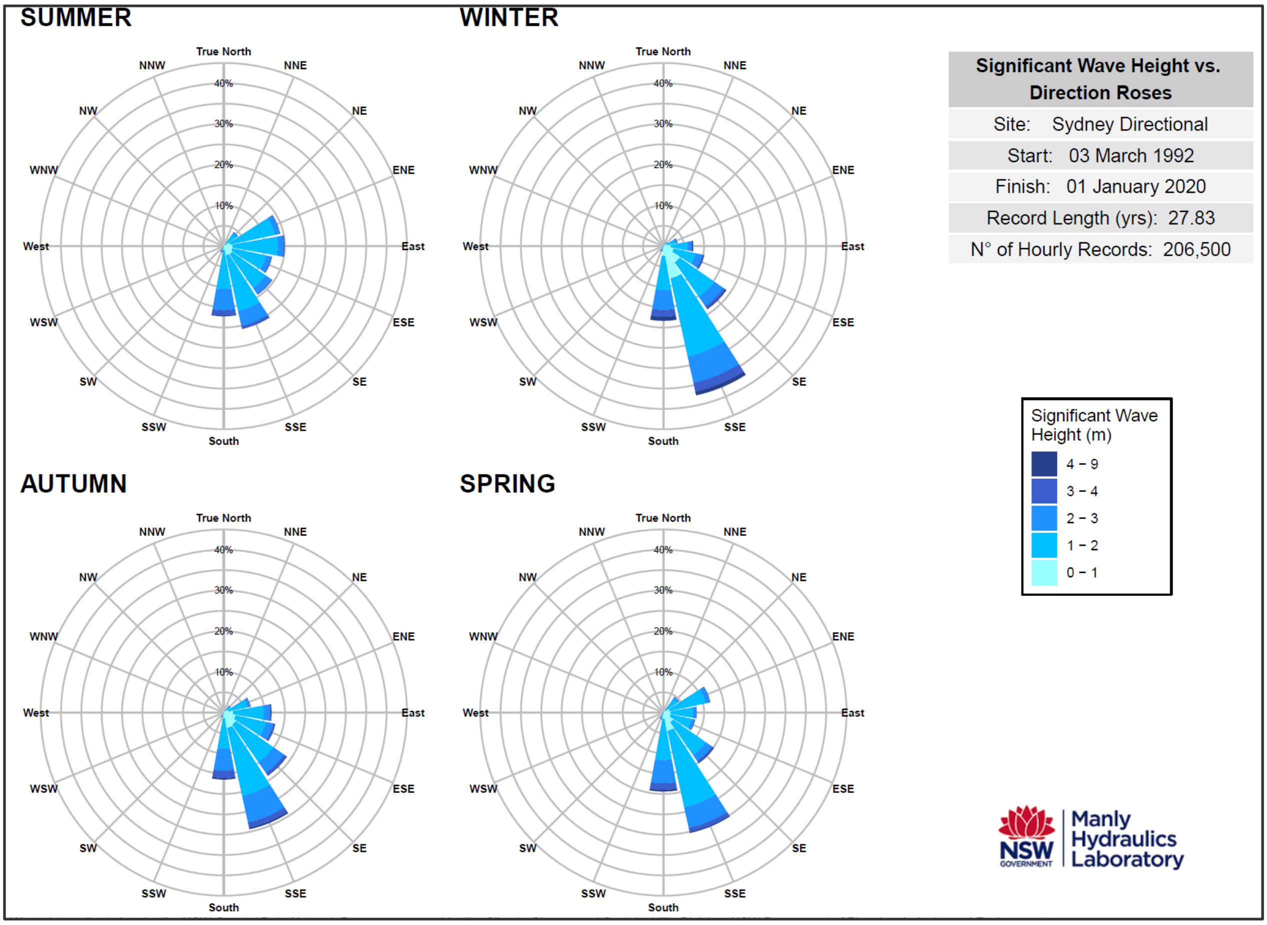


Figure 4-7 Sydney Waverider Buoy. Seasonal wave height and direction roses (Manly Hydraulic Laboratory , 2022)

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.

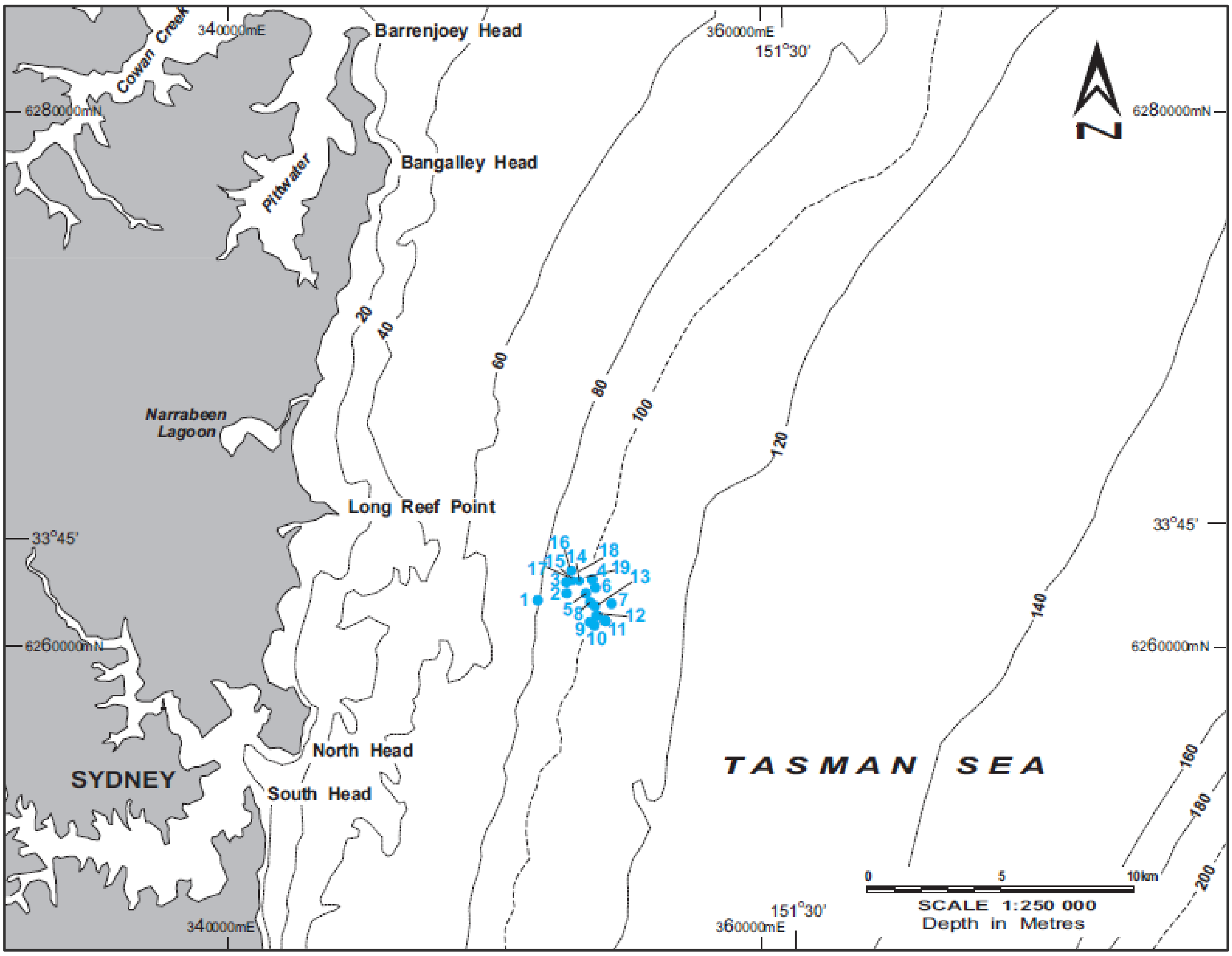


Figure 4-8 Sydney Waverider Buoy and location history (Manly Hydraulic Laboratory , 2022)

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% [xviii]. 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%[xviii] of storm waves.

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

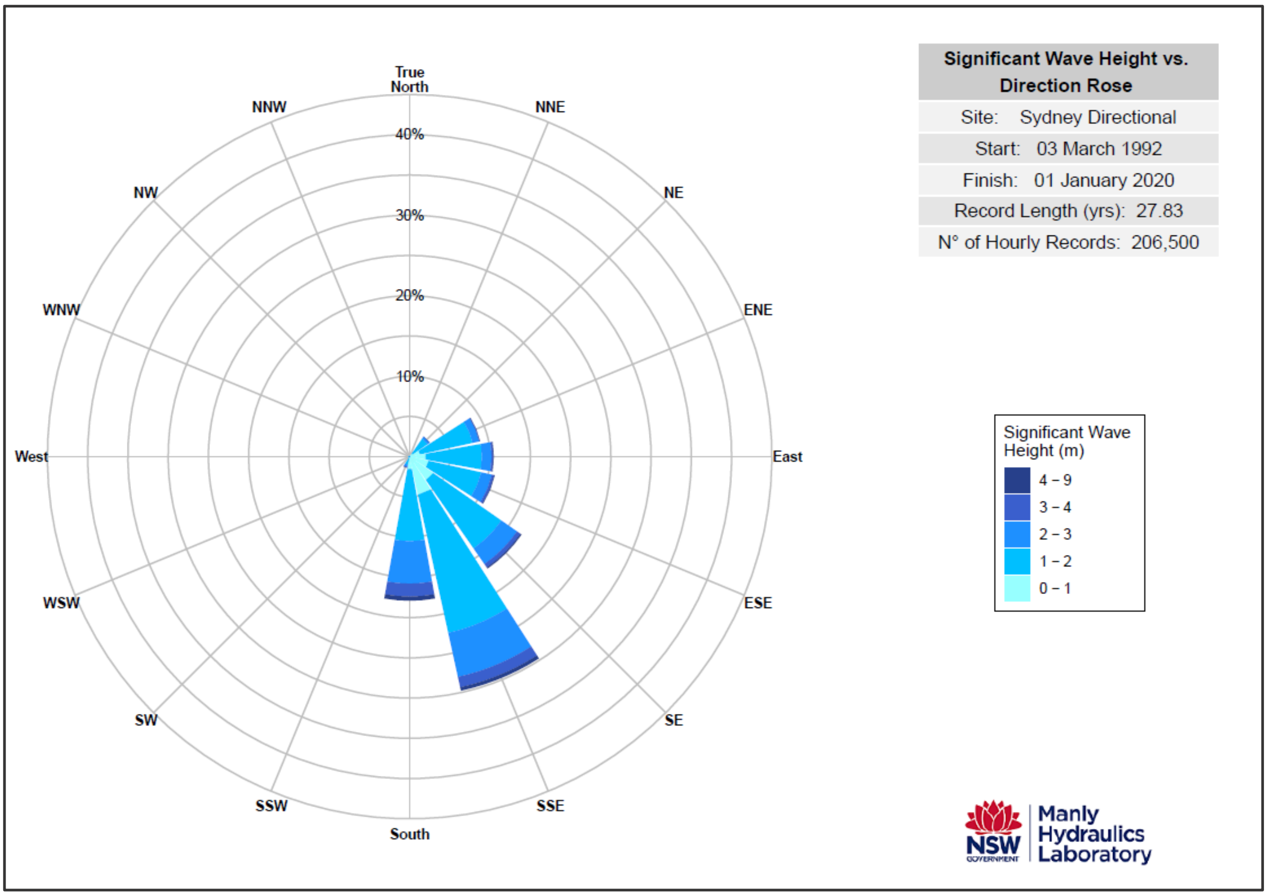


Figure 4-9 Sydney offshore wave rose (Manly Hydraulic Laboratory , 2022)

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 wave height likely to occur or be exceeded, on average, every 100 years was estimated 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 4-13 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. For this assessment an offshore H_s (or H_{so}) of 9.0m (1-hour duration) (Shand T. , et al., 2011) 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 trend at Bronte Beach even if orientation and lengths of the two beaches play an important role in this regard.

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 4-13**. The wave conditions affecting the seawall would depend on the beach levels eroded during the storm event.

In **Figure 4-9**, there is a comparison between reported significant wave heights from ARUP (ARUP, 2016) and Baird (Baird, 2016). The black marks on the graph show the extrapolated significant wave heights for 1-year and 500-year scenarios.

Table 4-14 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]
1(1)	NE	2.7	10.7	90
	ESE	5.2	12.2	111
	SSE	5.8	13.4	132
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 [xix]	7.7	14.9	129
500(1)	NE	6.3	14.9	93
	ESE	13.2	14.9	111
	SSE	15.4	14.9	129

Notes:

1. *Values of H_s and T_p have been extrapolated from ARUP (ARUP, 2016) and Baird (Baird, 2016) reports. Wave directions and peak wave periods for 1 and 500-year ARI have been assumed to be the same as 5 and 100-year ARI respectively.*

[xix] *Discussions with WRL have raised concerns about the specific wave parameters from the SSE sector, which will be reviewed as part of the physical modelling process.*

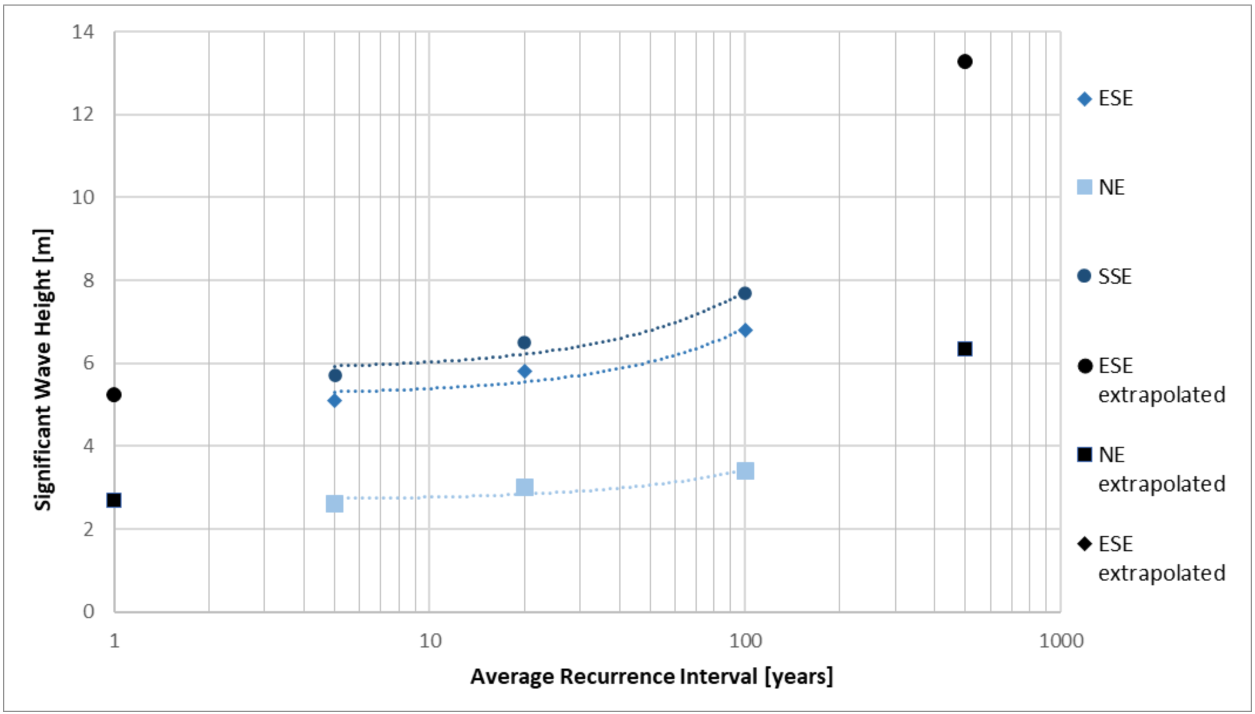


Figure 4-10 Relationship between significant wave height and ARI

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 4-13** and include the following:

- water depth.
- deep water wavelength L_0 based on offshore wave peak periods.
- nearshore slope of 1:50 (v:h) from the WorleyParsons report (WorleyParsons, 2011), aligning with the analysis conducted **Section 4.3**. This beach slope was assumed to be representative of the natural beach at Bronte Beach until it reaches the level of the sandstone bedrock, as reported in **Section 4.4**.
- As described in **Figure 4-10** the slope for the cross-shore gradient of the subaqueous (below MSL) profile is generally much lower than the beach-face slope, which is assumed to be 1:10 [xx] (v:h) as a slope adjustment following an erosion event, as described in **Section 5.2.1**.

[xx] This value aligns with the dataset of beach-face slopes for the Australian coastline derived from a novel remote sensing technique in (Vos, Deng, Dean Harley, Turner, & Splinter, 2022). The dataset covers 13 200 km of sandy coast and provides an estimate of the beach-face slope every 100 m alongshore accompanied by an easy-to-apply measure of the confidence of each slope estimate. The dataset offers a unique view of large-scale spatial variability in the beach-face slope. The beach-face slope dataset relevant to Bronte beach suggested an average beach slope of 1:11 (v:h), tan β equal to 0.086.

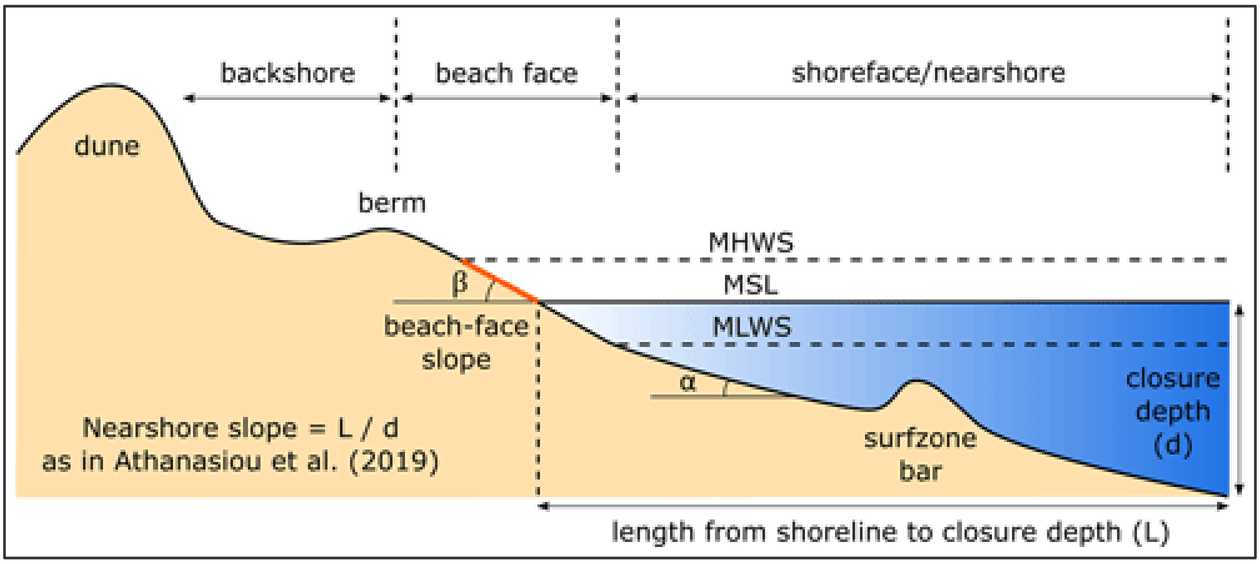


Figure 4-11 Schematic of a beach profile from the dune to the depth of closure, adapted from Shore Protection Manual (Coastal Engineering Research Center, 1984). The beach-face slope (tan β) that is mapped in this work is a proxy for the slope of the portion of the profile that is highlighted in orange, extending from mean sea level (MSL) up to mean high water springs (MHWS). The beach-face slope complements the global dataset of nearshore slopes presented in (Athanasiou, et al., 2019) that represents the slope extending from the depth of closure up to MSL.

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.

This approach provided significant wave heights (H_s) 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. Wave conditions used to carry out the overtopping and wave load calculations are as described in **Table 4-14**.

Table 4-15 Wave conditions used in assessment

Case	Average Recurrence Interval	Planning Period	Water Level	Scour level [xxi]	Spectral Wave Height at toe	Peak Period	Angle of attack relative to normal
	ARI		DSWL		H_{m0}	T_p	β
[-]	[years]	[-]	[m AHD]	[m AHD]	[m]	[s]	[°TN]
1	1	Present day	1.28	2.90	0.7 [xxii]	13.4	132
2	1	2093	1.94	1.70	0.5	13.4	132
3	1	2100	2.00	0.35	1.5	13.4	132
4	100	Present day	1.48	2.90	0.9[xxii]	14.9	132
5	100	2093	2.14	1.70	0.7	14.9	129
6	100	2093	2.14	-1.00 [xxiii]	2.1	14.9	129
7	100	2100	2.20	0.35	1.7	14.9	129
8	500	2093	2.14	-1.00[xxiii]	2.2	14.9	129

[xxi] Baird carried out beach erosion modelling for a nominal 100-year extreme wave event to estimate design scour levels at the seawall for each defined profile location along the structure (Baird, 2016). Recommended design scour levels at seawall toe under climate change-induced sea level rise scenarios for a 100-year ARI storm have been used for Case 1 to 3 which is a conservative approach.

[xxii] The seawall at Bronte would be constructed at the back of a beach such that breaking waves never reach the seawall, at least not during frequent events where overtopping in relation to public safety is of primary importance. For these conditions, particularly for typical shallow foreshore slopes like that considered for Bronte Beach, design wave conditions may be given by waves which start breaking (possibly quite some distance) seaward of the wall. These broken waves arrive at the wall as a highly aerated mass of water, giving rise to loadings which exhibit relatively short-duration peak values under impulsive conditions (as the leading edge of the mass of water arrives at the wall), but smaller in magnitude due to the high level of aeration. For case 1 and 4 conditions where the toe of the wall is emergent ($h \leq 0$ m), an alternative method suggested by EurOtop Manual (EurOtop, 2018) was used to estimate mean overtopping discharge with emergent toe ($h < 0$ m) of vertical wall on a foreshore slope 1:10.

[xxiii] A storm scour level of -1m AHD is typically adopted at NSW beaches. This is based on stratigraphic evidence of historical scour levels and observed scour levels occurring during major storms (Carley, Coghlan, & Flocard, 2015). This condition accounts for a highly eroded seabed (-1 m AHD). At this stage, -1.00m AHD is deemed a conservative approach to identify the bedrock level as the limiting factor for a potential scour.

5 Relevant coastal hazards

5.1 General

The Coastal Management Act 2016 identifies seven coastal hazards:

(1) beach erosion;

(2) shoreline recession;

(3) coastal lake or watercourse entrance instability;

(4) coastal inundation;

(5) coastal cliff or slope instability;

(6) tidal inundation; and

(7) 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 (1) beach erosion, (2) shoreline recession, and (4) coastal inundation. These three hazards are discussed below.

5.2 Beach erosion and shoreline recession [Hazards 1 and 2]

5.2.1 Beach erosion[xxiv]

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 to **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).

[xxiv] *Including the effects of sand slope instability*

- **Stable Foundation Zone SFZ** (blue tone) is maintained landward of the ZRFC. Here the factor of safety against gross instability is greater than 1.5 (stable).

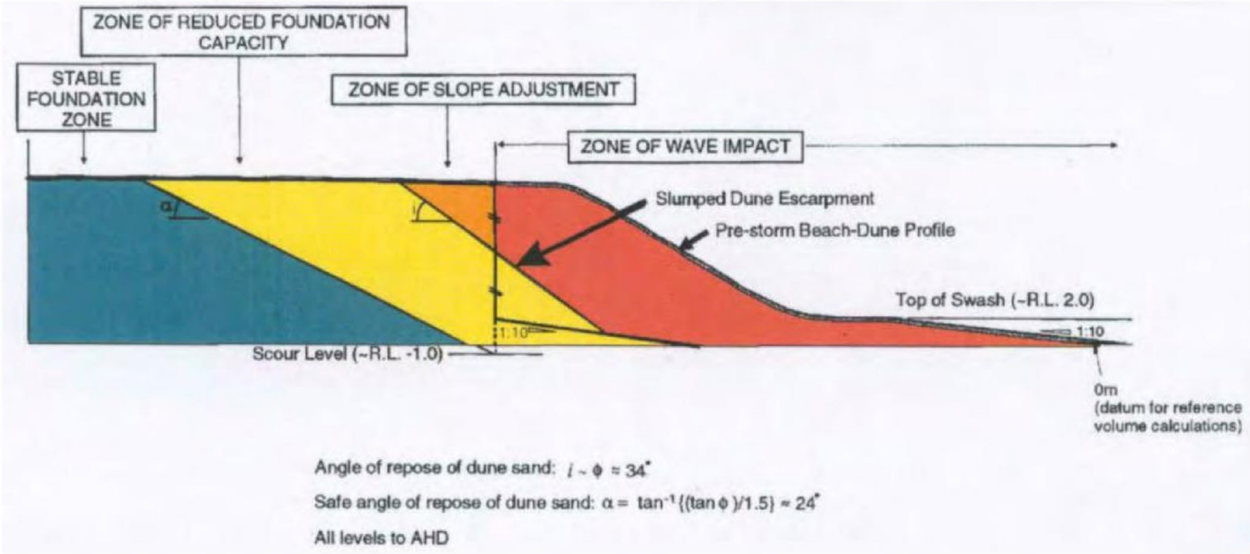


Figure 5-1 Wedge Failure Plane Model after Nielsen et al (1992)

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 Waverley 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 also 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 indciates 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 it is assumed that the portion of the profile below say 2mAHD could reasonably have mimicked 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). It is known 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

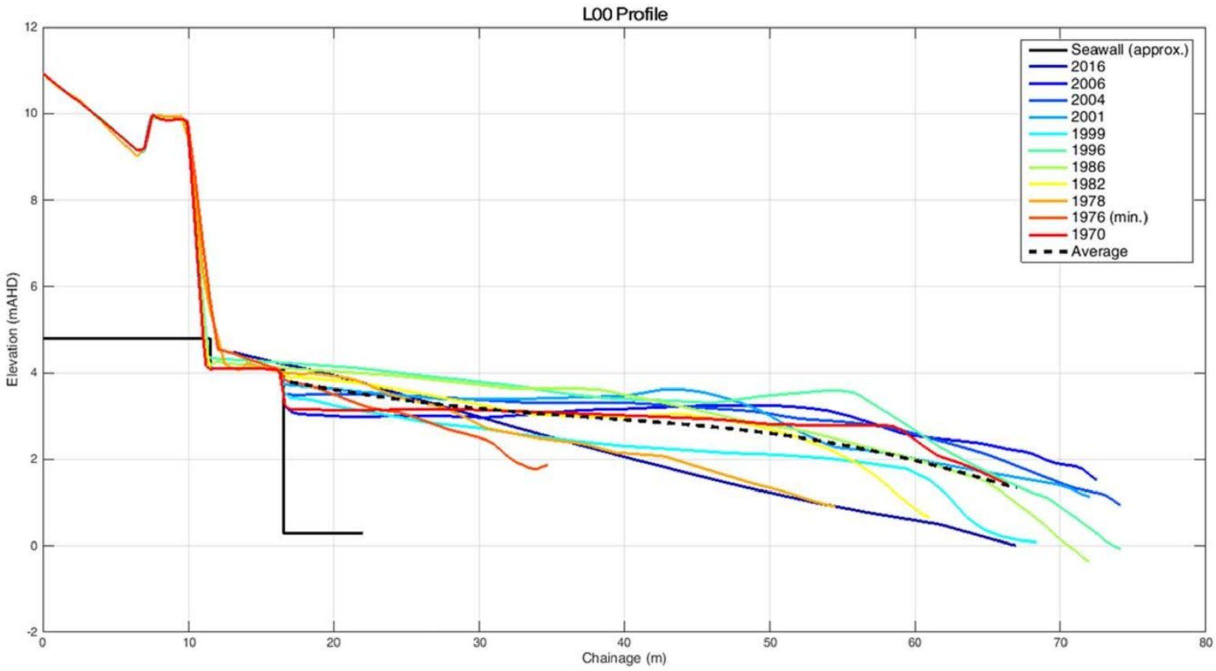


Figure 5-3 Extracted profiles at L00

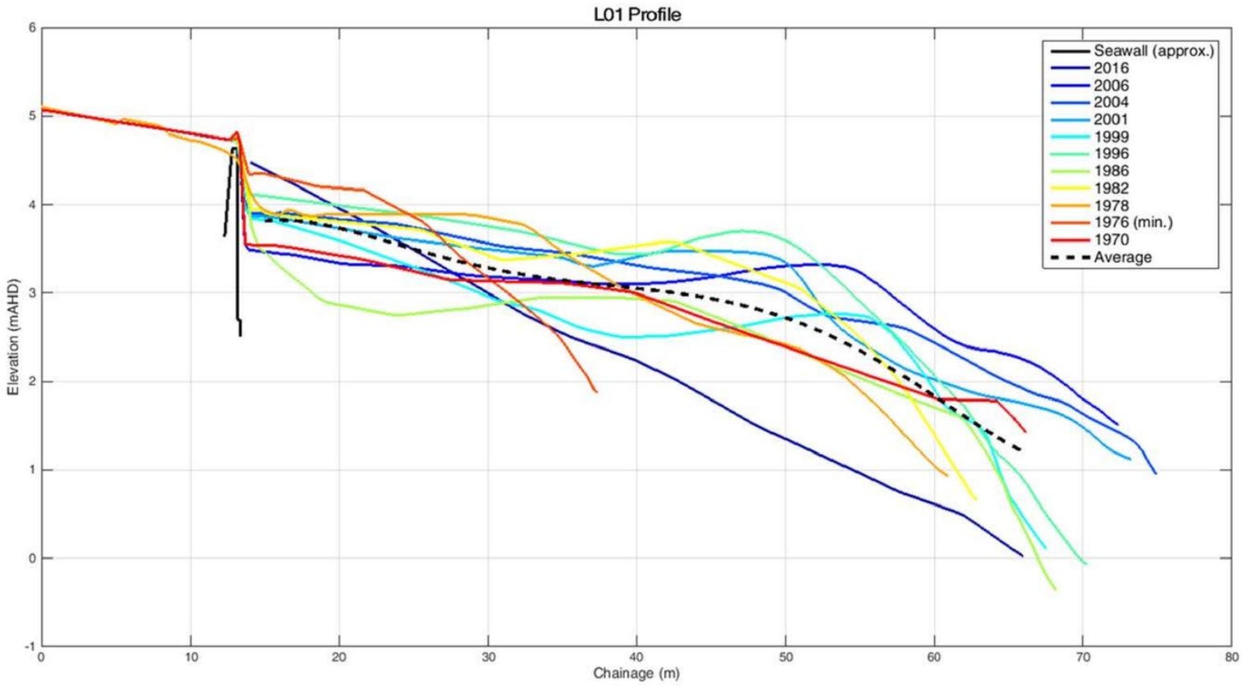


Figure 5-4 Extracted profiles at L01

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.

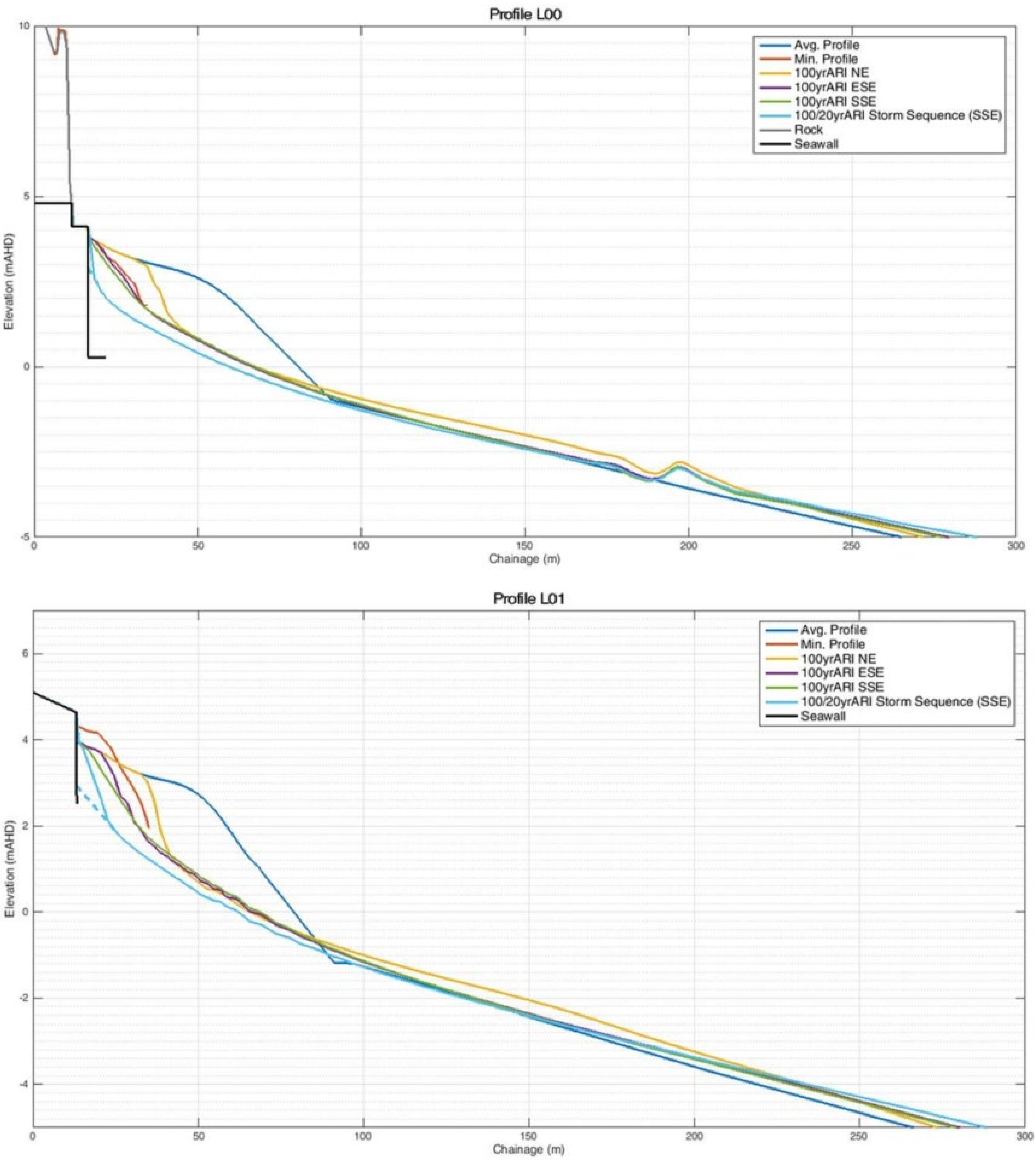


Figure 5-5 SBEACH model results for Profiles L00 and L01 for present day conditions, with and without the design storm from various directions (Baird, 2016)

5.2.1.3 Beach recovery following erosion

Available evidence suggests that Bronte Beach fully recovers after storms and that this recovery is initially relatively rapid occurring over days to weeks. Not unlike most other NSW beaches, full recovery could take months to years. The Waverley Coastal Risks and Vulnerability Study (Worley Parsons, 2011) identifes major storms impacting the beach in 1942, 1948, 1952 and 1959, resulting in heavily overtopping of the seawall, on two of these occaions resulting in water washing over the promenade and into the park. Video of the 2016 storm also shows heavy overtopping entering the park. The earliest (1943) and most recent (2023) airhotos shows the beach planform little changed over the last 80 years (refer to **Figure 5-6**).

Figure 5-6 Bronte Beach 1943 left (SIX maps) and 2023 right (Nearmap)

Air and land photos of Bronte Beach before and after the recent June 2016 storm event, point to substantial recovery within a month of this storm nominally regarded as a 20 to 40-year ARI event for east facing shorelines in and around Sydney based on discussions with WRL (refer to **Figure 5-7)**.

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

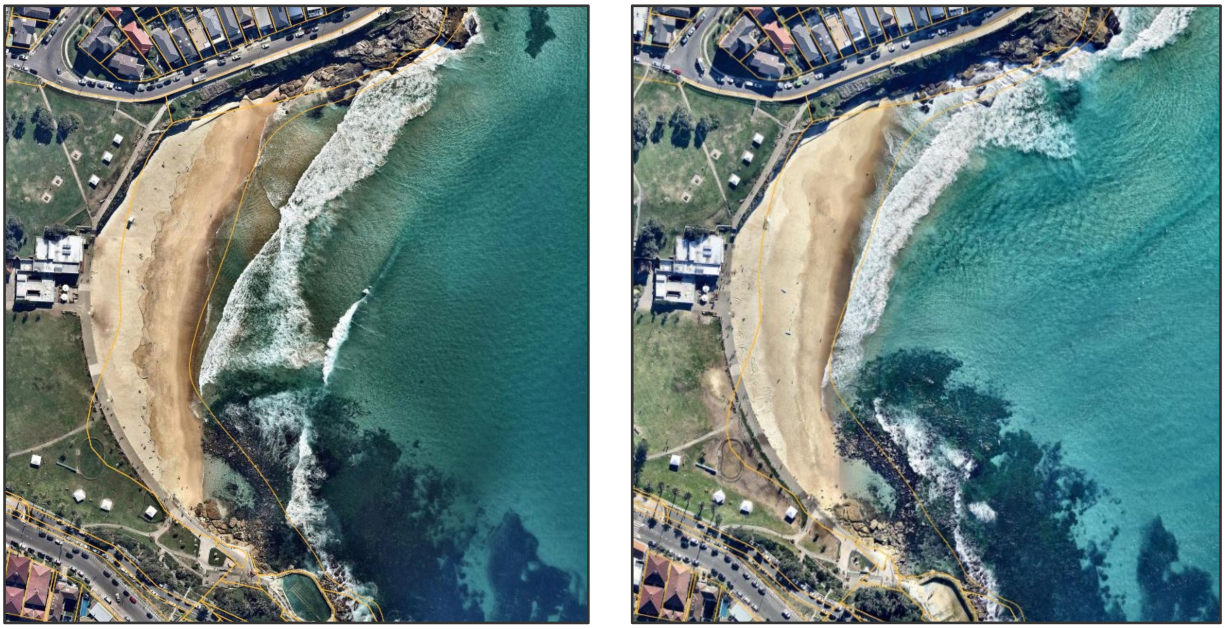


Figure 5-7 Bronte Beach before (left 5/5/16) and after (right 2/7/16) the June 2016 storm event

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 fair weather. 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). Like Worley Parsons (2011) found for the Bondi, Tamarama and Bronte Beach compartments which they examined as a group, Baird (2016) observed no recessional trends in the data.

However, recession in the future is predicted to a occur as a consequence of sea level rise due to 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 5-8**. The average beach width in front of the SLSC is predicted to reduce from approximately 70m at the 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 the 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 5-9**.

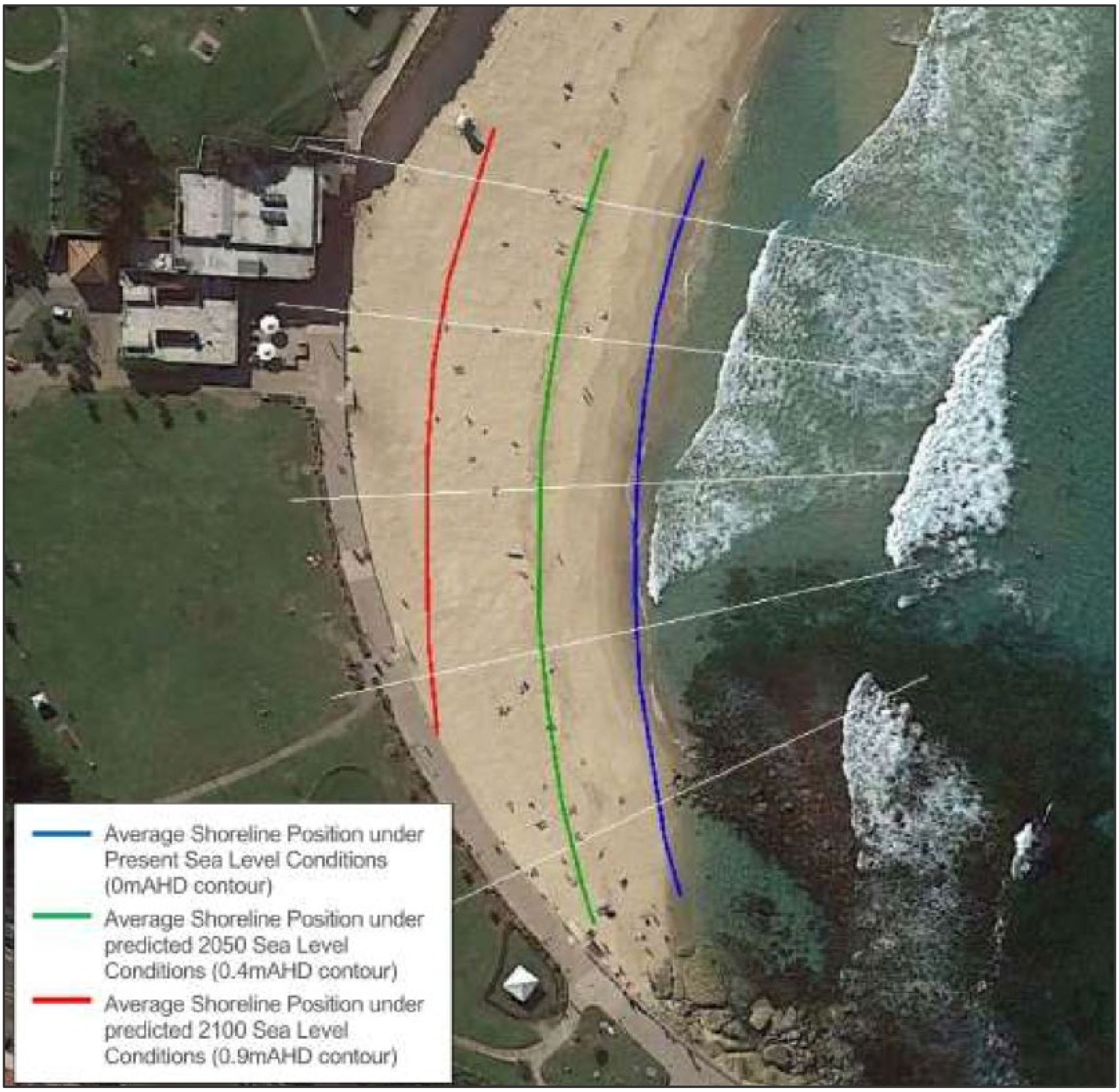


Figure 5-8 Average present day and predicted 2050 and 2100 shoreline position (Baird, 2016)

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.

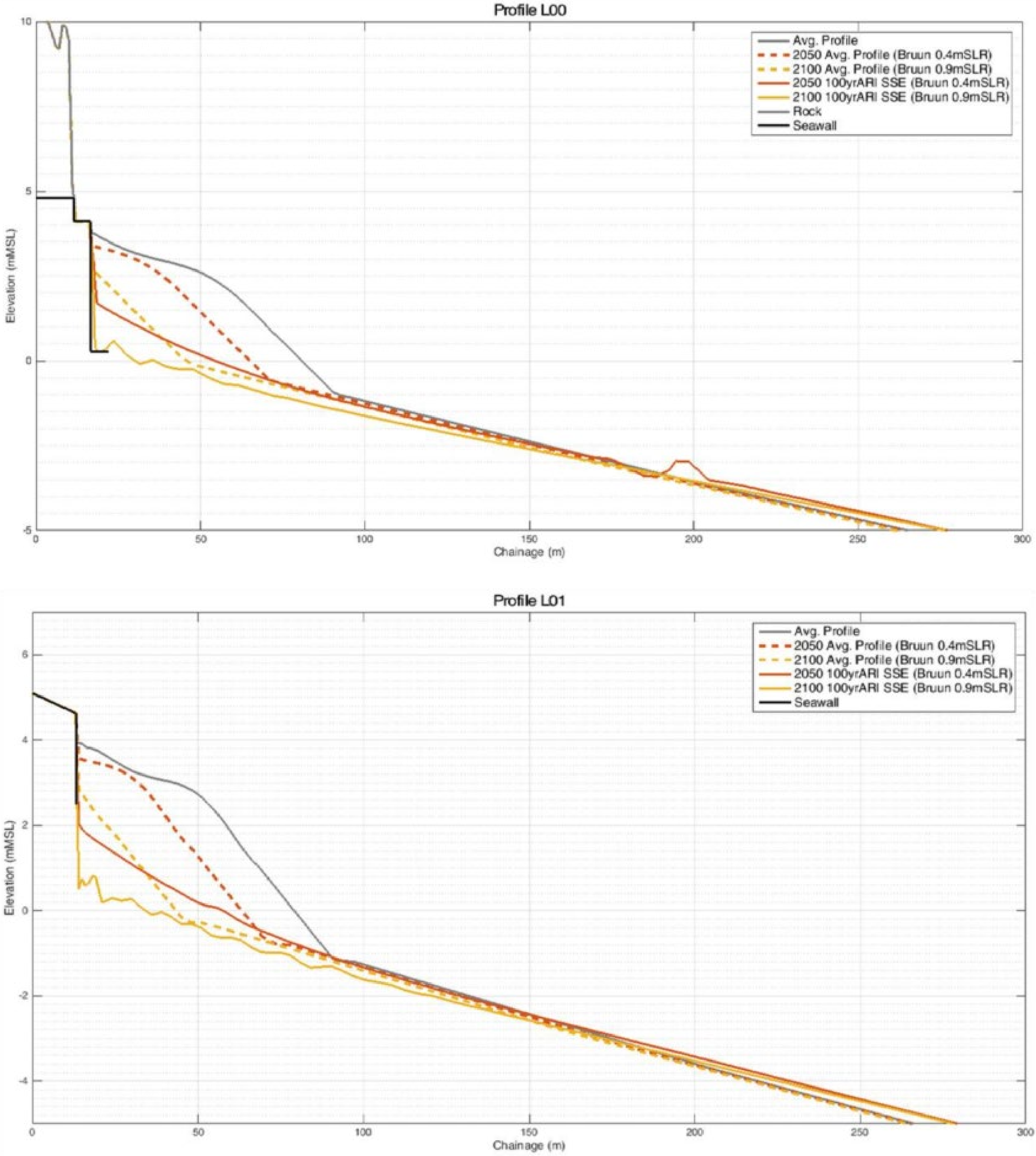


Figure 5-9 SBEACH model results for Profiles L00 and L01 for future 2050 and 2100 climate change scenarios showing average profiles with and without a 100-year ARI design storm directed from the SSE (Baird, 2016)

5.2.3 Beach scour

Baird applied SBEACH to predict design scour levels under present and future climate conditions as summarised 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 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, but possibly accounted for due to the likely presence of relatively elevated bedrock. 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 [Hazard 4]

The ground floor of the existing 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 5.3.3.1**). 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 5.3.3.8**, 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. The ARUP study (ARUP, 2016) incorporates valuable pre- and post-storm beach survey data, along with post-storm observations. **Figure 5-10** illustrates the southern portion of Bronte Beach, showing a notable change in the beach profile before and immediately after the storm.

Figure 5-10 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.

RHDHV acknowledges that there was no significant structural damage during the storms of 1974 and 2016. However, it is noted that there was damage to landscaping and roller shutters. It is widely accepted that the current promenade protected by the existing seawall is unsafe for pedestrians during extreme storms, a situation commonly managed through access restrictions.

For the renewed SLSC building and the new seawall, the primary management concern is to avoid structural damage to the building and seawall. Additionally, there is a focus on restricting inundation to levels deemed acceptable or tolerable. This approach emphasises the importance of balancing safety considerations with the need to mitigate potential damage during extreme storm events.

5.3.2 Scenarios

As outlined in **Section 2.3**, the seawall layout and geometry are diverse, incorporating various elements such as standard vertical walls, vertical walls with ramps, and sections with varying levels, stairs, and bleachers. For the purposes of this study, the following three sections, progressing from north to south (refer to **Figure 5-10**), were evaluated:

- Seawall 1: This refers to the stairs of the seawall situated at the northern access point of the seawall.
- Seawall 2: This refers to a composite vertical wall with the ramp structure positioned in front of the wall.
- Seawall 3: This involves the continuous section of the seawall extending from the southern end to the beach ramp access.

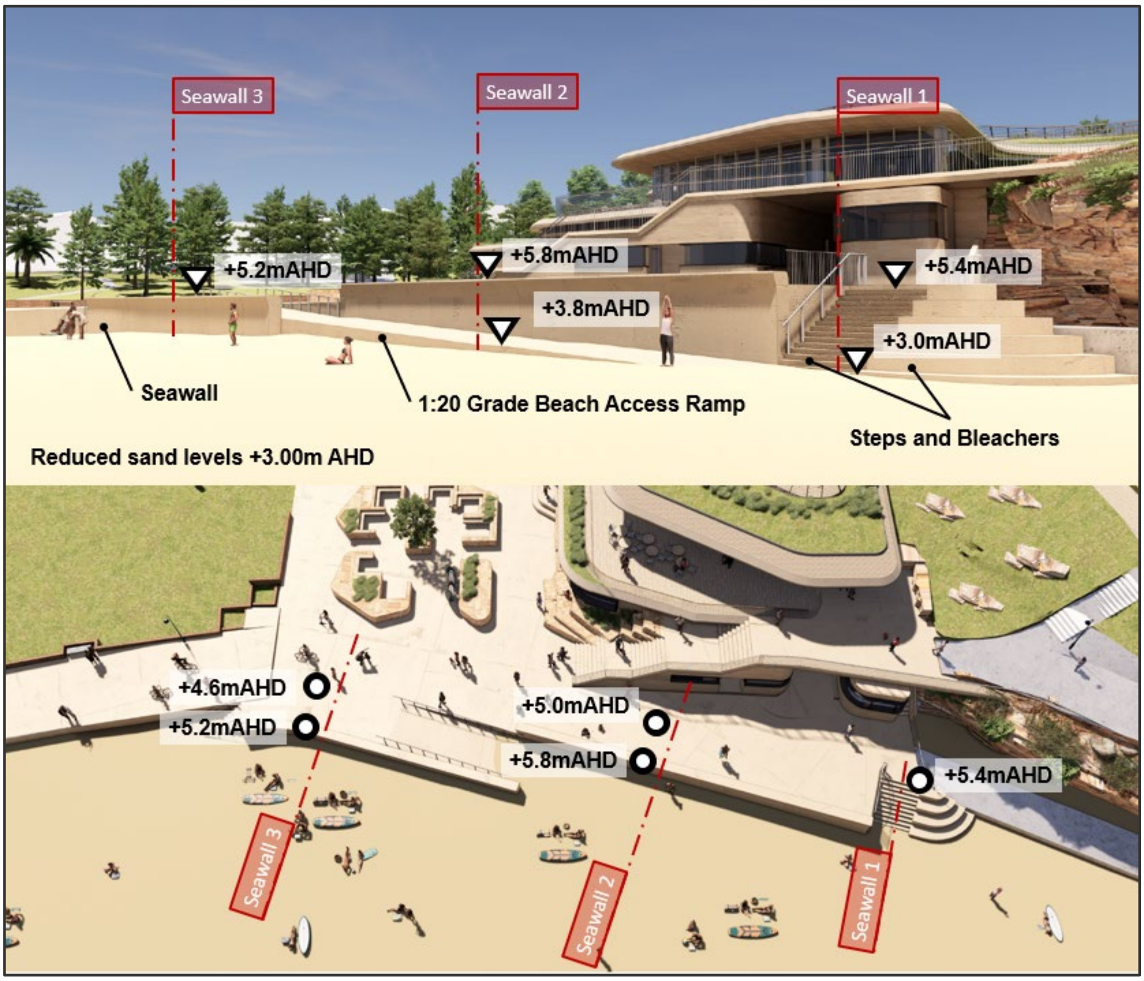


Figure 5-11 Seawall sections

5.3.3 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 desktop 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 to be included 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.

The estimation of wave runup and overtopping involves the following considerations:

- A 100-year ARI event is deemed acceptable for inundation but may not adequately represent wave forces on the building. RHDHV has opted for a more conservative approach by utilising a 500-year ARI event.

- It is recognised that the derivation of the 100-year ARI event may be conservative, as detailed in **Section 4.7.**
- The addition of a full wave setup at the seawall is acknowledged as potentially conservative, given the likelihood of truncation of wave setup at the seawall. This conservatism would be addressed through more detailed numerical modelling and/ or subsequent physical modelling.

5.3.3.1 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 [xxv]. 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. RHDHV acknowledges that Mase's method is based on laboratory data obtained from extensive tests of random waves on gentle, smooth, and impermeable slopes. It is recognised that this method is more suitable for natural beaches and may not easily incorporate a seawall. In **Section 5.3.3.3**, dedicated wave overtopping calculations involving a seawall are conducted, and Mase's method calculations can be regarded as an initial approximation.

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. Maximum runup is a crucial parameter for identifying areas susceptible to coastal inundation. However, measuring the most extreme runup is often challenging due to conditions where wave runup impacts or overtops dunes or dune scarps. As a result, most of the calculation methods focus on determining the maximum runup on beaches where the runup is not truncated.

[xxv] 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.

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 method (Mase, 1989)

The potential effectiveness of a wave return wall [xxvi] in reducing wave overtopping to acceptable or tolerable levels over the design life is contingent on the completion of a physical model during the Detailed Design phase. The physical model would play a useful role in assessing the performance and functionality of the wave return wall under various conditions, providing essential data to validate its effectiveness in mitigating wave overtopping.

5.3.3.2 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**.

[xxvi] *Also referred to elsewhere and herein as a deflector, bull-nose, and parapet return wall.*

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

Item	Limit State	Average Recurrence Interval	Hazard type and reason	Mean discharge	Maximum volume	Comment
		ARI		q	V_{max}	
[-]	[-]	[-]	[-]	[l/s per m]	[l per m]	[-]
1	Operational conditions	1-year	Damage to equipment set back 5–10m.	1	1,000	These limits relate to overtopping defined at the defence.
2	Operational conditions	1-year	People at seawall/dike crest. Clear view of the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	 0.3 1 10-20 No limit	 600 600 600 No limit	
3	Ultimate limit state	1-year	Building structure elements, H_{m0}=1-3m	1	1,000	This limit relates to the effective overtopping flow defined at the building.
4	Ultimate limit state	100-year	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.	
5	Ultimate limit state	100-year	Damage to grassed or lightly protected promenade or reclamation cover.	50	Not provided	
6	Ultimate limit state	100-year	Damage to paved or armoured promenade behind seawall.	200	Not provided	

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 structures, 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 drainage structures to mitigate these impacts.

5.3.3.3 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 metre 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 +4.8m 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 given 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 4-14**. The mean and maximum allowable overtopping discharge and estimated maximum overtopping rates and volumes over the seawall are described in the following tables.

- Existing seawall: refer to **Table 5-5**
- Seawall 1: refer to **Table 5-6**
- Seawall 2: refer to **Table 5-7**
- Seawall 3: refer to **Table 5-8**

Table 5-5 Wave conditions used in assessment of overtopping for the existing seawall (crest level +4.80m AHD)

Case	Average Recurrence Interval	Planning Period	Water Level	Scour level[xxi]	Spectral Wave Height at the toe	Spectral Wave Period	Angle of attack relative to True North [xxvii]	Method steps			Overtopping	
								Is there an influence of foreshore or not?	Is there a significant mound present?	Is there a likelihood of impulsive overtopping conditions?	Mean overtopping discharge back of crest [xxviii]	Maximum Overtopping Volume
	ARI		DSWL		H_{m0}	T_{m-1}	β				q	V_{max}
[-]	[years]	[-]	[m AHD]	[m AHD]	[m]	[s]	[°TN]	[-]	[-]	[-]	[l/s/m]	[l/m]
1	1	Present day	1.28	2.90	0.7[xxii]	12.2	0	Yes	No	Yes	0.0 [0.0]	-
2	1	2093	1.94	1.70	0.5	12.2	0	Yes	No	Yes	0.3 [0.4]	6
3	1	2100	2.00	0.35	1.5	12.2	0	Yes	No	Yes	15.0 [20.0]	>1000
4	100	Present day	1.48	2.90	0.9[xxii]	13.6	0	Yes	No	Yes	0.0 [0.0]	-
5	100	2093	2.14	1.70	0.7	13.6	0	Yes	No	Yes	1.2 [1.7]	40
6	100	2093	2.14	-1.00[xxiii]	2.1	13.6	0	Yes	No	Yes	61.7 [87.0]	>1000
7	100	2100	2.20	0.35	1.7	13.6	0	Yes	No	Yes	33.8 [48.3]	>1000

[xxvii] *Under oblique wave attack, significant spatial variability of overtopping discharge along a seawall would be observed in the field and qualified in physical model studies. At this stage we would consider shore-normal wave attack (obliquity β = 0°) for estimating overtopping rates.*
[xxviii] *The two numbers represent values calculated with the mean value approach, and the value within brackets is calculated with the design or assessment approach.*

Table 5-6 Wave conditions used in assessment of overtopping Seawall 1

Case	Average Recurrence Interval	Planning Period	Water Level	Scour level[xxi]	Spectral Wave Height at the toe	Spectral Wave Period	Angle of attack relative to True North[xxvii]	Method steps			Overtopping	
								Is there an influence of foreshore or not?	Is there a significant mound present?	Is there a likelihood of impulsive overtopping conditions?	Mean overtopping discharge back of crest[xxviii]	Maximum Overtopping Volume
	ARI		DSWL		H_{m0}	T_{m-1}	β				q	V_{max}
[-]	[years]	[-]	[m AHD]	[m AHD]	[m]	[s]	[°TN]	[-]	[-]	[-]	[l/s/m]	[l/m]
1	1	Present day	1.28	2.90	0.7[xxii]	12.2	0	Yes	No	Yes	0.0 [0.0]	-
2	1	2093	1.94	1.70	0.5	12.2	0	Yes	No	Yes	0.1 [0.2]	3.8
3	1	2100	2.00	0.35	1.5	12.2	0	Yes	No	Yes	7.8 [11.0]]	>500
4	100	Present day	1.48	2.90	0.9[xxii]	13.6	0	Yes	No	Yes	0.0 [0.0]	-
5	100	2093	2.14	1.70	0.7	13.6	0	Yes	No	Yes	0.6 [0.9]	24
6	100	2093	2.14	-1.00[xxiii]	2.1	13.6	0	Yes	No	Yes	32.4 [46.2]	>1,000
7	100	2100	2.20	0.35	1.7	13.6	0	Yes	No	Yes	17.2 [24.6]	>1,000

Note:

1. *Typical roughness coefficient for stepped seawall is 0.95*

Table 5-7 Wave conditions used in assessment of overtopping Seawall 2

Case	Average Recurrence Interval	Planning Period	Water Level	Scour level[xxi]	Spectral Wave Height at the toe	Spectral Wave Period	Angle of attack relative to True North[xxvii]	Method steps: Is there an influence of foreshore or not?	Method steps: Is there a significant mound present?	Method steps: Is there a likelihood of impulsive overtopping conditions?	Overtopping: Mean overtopping discharge back of crest[xxviii]	Overtopping: Maximum Overtopping Volume
	ARI		DSWL		H_{m0}	T_{m-1}	β				q	V_{max}
[-]	[years]	[-]	[m AHD]	[m AHD]	[m]	[s]	[°TN]	[-]	[-]	[-]	[l/s/m]	[l/m]
1	1	Present day	1.28	2.90	0.7[xxii]	12.2	0	Yes	No	Yes	0.0 [0.0]	-
2	1	2093	1.94	1.70	0.5	12.2	0	Yes	No	Yes	0.0 [0.2]	5
3	1	2100	2.00	0.35	1.5	12.2	0	Yes	No	Yes	5.9 [8.4]	>500
4	100	Present day	1.48	2.90	0.9[xxii]	13.6	0	Yes	No	Yes	0.0 [0.0]	-
5	100	2093	2.14	1.70	0.7	13.6	0	Yes	No	Yes	0.5 [0.7]	20
6	100	2093	2.14	-1.00[xxiii]	2.1	13.6	0	Yes	No	Yes	28.9 [41.22]	>1,000
7	100	2100	2.20	0.35	1.7	13.6	0	Yes	No	Yes	12.7 [18.19]	>1,000

Table 5-8 Wave conditions used in assessment of overtopping Seawall 3

Case	Average Recurrence Interval	Planning Period	Water Level	Scour level[xxi]	Spectral Wave Height at the toe	Spectral Wave Period	Angle of attack relative to True North[xxvii]	Method steps			Overtopping	
								Is there an influence of foreshore or not?	Is there a significant mound present?	Is there a likelihood of impulsive overtopping conditions?	Mean overtopping discharge back of crest[xxviii]	Maximum Overtopping Volume
	ARI		DSWL		H_{m0}	T_{m-1}	β				q	V_{max}
[-]	[years]	[-]	[m AHD]	[m AHD]	[m]	[s]	[°TN]	[-]	[-]	[-]	[l/s/m]	[l/m]
1	1	Present day	1.28	2.90	0.7[xxii]	12.2	0	Yes	No	Yes	0.0 [0.0]	-
2	1	2093	1.94	1.70	0.5	12.2	0	Yes	No	Yes	0.2 [0.3]	5
3	1	2100	2.00	0.35	1.5	12.2	0	Yes	No	Yes	10.0 [14.0]]	>1,000
4	100	Present day	1.48	2.90	0.9[xxii]	13.6	0	Yes	No	Yes	0.0 [0.0]	-
5	100	2093	2.14	1.70	0.7	13.6	0	Yes	No	Yes	0.8 [1.1]	30
6	100	2093	2.14	-1.00[xxiii]	2.1	13.6	0	Yes	No	Yes	41.2 [58.8]	>1,000
7	100	2100	2.20	0.35	1.7	13.6	0	Yes	No	Yes	22.0 [31.4]	>1,000

Mean overtopping discharge at the back of the seawall crest are provided in Table 5-9 for different seawall sections and ARIs. The overtopping rates have been correlated to guideline thresholds for pedestrian safety, equipment and building damage provided in EurOtop Manual (EurOtop, 2018).

Table 5-9 Comparison of Mean Overtopping Rate Measurements with other EurOtop Thresholds

Seawall section	Planning Period	Storm Average Recurrence Interval	Spectral Wave Height at the toe	Mean overtopping discharge back of crest	Hazard Code						
		ARI	H_{m0}	q	Aware Pedestrian	Equipment	People at seawall	Building Elements	Trained Staff \	Grassed promenad	Paved promenad
[-]	[-]	[years]	[m]	[l/s/m]							
Existing seawall	Present day	1	0.7[xxii]	0.0							
	2093	1	0.5	0.3							
	2100	1	1.5	15.0							
	Present day	100	0.9[xxii]	0.0							
	2093	100	0.7	1.2							
	2093	100	2.1	61.7							
	2100	100	1.7	33.8							
Seawall 1	Present day	1	0.7[xxii]	0							
	2093	1	0.5	0.1							
	2100	1	1.5	7.8							
	Present day	100	0.9[xxii]	0.0							
	2093	100	0.7	0.6							
	2093	100	2.1	32.4							
	2100	100	1.7	17.2							
Seawall 2	Present day	1	0.7[xxii]	0.0							
	2093	1	0.5	0.0							
	2100	1	1.5	5.9							
	Present day	100	0.9[xxii]	0.0							
	2093	100	0.7	0.5							
	2093	100	2.1	28.9							
	2100	100	1.7	12.7							
Seawall 3	Present day	1	0.7[xxii]	0.0							
	2093	1	0.5	0.2							
	2100	1	1.5	10.0							
	Present day	100	0.9[xxii]	0.0							
	2093	100	0.7	0.8							

	2093	100	2.1	41.2							
	2100	100	1.7	22.0							

Not applicable

Safe/ No Damage

Marginal

Unsafe/ Damage

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 1-year ARI event, there is a high likelihood of wave overtopping being a hazard to pedestrians at the seawall crest.
- Wave overtopping for the planning period in 2100 under a 1-year ARI event for the future seawall could pose a hazard for people in proximity to the seawall crests.
- There is relatively low likelihood of wave overtopping causing structural damage behind the seawall under 1-year ARI events, but higher theoretical risk for 100-year ARI or more extreme events.
- Predicted overtopping at 2093 under a 100-year ARI event 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.

5.3.3.4 Spatial distribution of overtopping

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 structure 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).

The resulting landward distributions for various laboratory wind speeds give the proportion of total overtopping discharge which has landed within a particular distance shoreward of the seaward crest. The lower (conservative) envelope of the data gives the approximate guidance that:

- 50% of the violently overtopped discharge would land within a distance of 0.06 × Lm-1,0
- 90% of the violently overtopped discharge would land within a distance of 0.20 × Lm-1,0

- 95% of the violently overtopped discharge would land within a distance of 0.25 × Lm-1,0

Where $L_{m-1,0}$ is the spectral wavelength in deep water (m).

Indicative (conservative) spatial distributions of overtopping associated with the above calculations are shown in **Table 5-10**. It should be noted that these results are approximate and do not consider structure geometry, wave return effects and site-specifics.

Table 5-10 Indicative (conservative) spatial distributions of wave overtopping based on EurOTop (2018)

Average Recurrence Interval ARI [years]	a) 50% of the violently overtopped discharge would land with a distance of	b) 90% of the violently overtopped discharge would land with a distance of	c) 95% of the violently overtopped discharge would land with a distance of
Present day and 2093	14m	46m	58m

A setback of at least 10m landward of the landward edge of the vertical has been adopted for the SLSC development. This setback is intended to facilitate some dissipation of wave overtopping and has been established as a maintenance standard for the proposed structure. While this setback may provide some dissipation of wave overtopping, it is crucial for the design to showcase effective drainage of overtopping volumes and flows. This is particularly important due to the relative proximity of building footprints to the seawall crest.

5.3.3.5 Effect of oblique waves

Seawalls are often not perfectly aligned with incoming waves. For the proposed new seawall, it is estimated that incident wave obliquity could potentially range up to -15/ +25 degrees from the proposed alignment. EurOtop indicates that the effects of such incident wave obliquity would not change wave overtopping by more than 10% (EurOtop, 2018). As such variability would be considered small in relation to the accuracy of overtopping predictions generally, the effects of wave obliquity would be neglected in the final design for wave overtopping.

5.3.3.6 Inclusion of wave return wall

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 elevations (refer to **Section 5.3.2**).

The design of the vertical seawall can include some form of seaward overhang (recurve/ parapet/ wave return wall/ bullnose) as part of the structure with the intent of reducing wave overtopping by deflecting back seaward up-rushing water. In general, these designs are often relatively small structures at the top of the wall, and they work best if overtopping is not too large. RHDHV notes that the effectiveness of wave return walls depends on the incident wave and back-beach/ seabed conditions.

As per the EurOtop Manual (EurOtop, 2018) the mechanisms determining the effectiveness of a bullnose/ wave return wall are complex and not yet fully described. The guidance presented in the EurOtop Manual (EurOtop, 2018) is based upon existing guidance and physical model studies. The parameters for the assessment of overtopping at structures with bullnose/ recurve walls are shown in **Table 5-11** and **Table5-12**.

In this section the key structure geometries assumed for the bullnose are summarised below:

- Height of wave return wall/ parapet/ bullnose, hr=0.80m
- Horizontal extension of wave return wall/ parapet/ bullnose in front of main wall, Br=0.5m

Given the considerable scatter in the original data and the recognition that the methodology lacks a secure foundation in detailed physical mechanisms and processes, it is suggested that designing for k_{bn} < 0.05, i.e., predicting reductions in mean discharges by factors greater than 20, may be impractical due to a lack of confidence in predictions. It is appropriate that physical modelling, which is to commence shortly (**Section 7**), be carried out to verify estimated reductions in wave overtopping attributed to wave bull nose/ deflector configurations, particularly where loads are high and could have a material consequence for the design of structures behind the seawall such as the proposed Bronte SLSC buildings.

5.3.3.7 Sensitivity to input parameters

Sensitivity to wave period

The spectral mean wave period, $T_{m-1,0}$ (s), serves as an input parameter in wave overtopping calculations, particularly for locations with very shallow foreshores (h/Ho < 1), applicable to the design location, where low-frequency (or so-called infragravity) wave regimes dominate wave overtopping.

Regarding infragravity dominated overtopping conditions, particular limitations of the EurOtop (EurOtop, 2018) methods are noted in Section 1.4.1 reproduced below:

> *Recent studies have shown that low frequency waves caused by wave breaking may become very important for wave overtopping prediction. This is certainly the case if the foreshore is relatively steep, say steeper than 1:50, and the water depth at the structure in reality reduces to a few decimetres (prototype). In such a case the short wave spectrum may completely disappear and transform to a spectrum with mainly infragravity waves of half a minute or more. These kind of circumstances are not yet fully understood, not by numerical modelling, nor by wave flume experiments. The manual gives guidance for very shallow water with long waves developing in Sections 1.4.7 (definition of shallow foreshore areas), 2.3.2 (wave heights at depth-limited situations) and 2.3.3 (wave periods at depth-limited situations), but one should not rely completely on the given formulae in this manual and consider physical model tests.*

Nearshore slopes at Bronte Beach are steeper than 1:50, which in fact is the case at most beaches in NSW. While it common to assume back beach scour to RL0m AHD and lower in severe storms, the Peer Reviewer has questioned whether wave runup and overtopping could be controlled early in the storm when the eroded beach profile is just reaching the seawall in which case the shallow foreshore criteria discussed above may apply, and infragravity response dominates the wave overtopping. Again, this would not be peculiar to Bronte Beach.

Due to the identified limitations in the (EurOtop, 2018) methods for handling infragravity-dominated overtopping regimes, the accuracy of overtopping values in this scenario is considered to be low and is challenging to verify empirically. EurOtop accordingly recommends physical model validation unless the design can demonstrate an adequate factor of safety. Physical modelling of the proposed seawall is to commence shortly (**Section 7**).

Table 5-11 Wave conditions used in assessment of overtopping Seawall 2

Case	Average Recurrence Interval	Planning Period	Water Level	Scour level[xxi]	Spectral Wave Height at the toe	Spectral Wave Period	Angle of attack relative to True North[xxvii]	Method steps			Overtopping	
								Is there an influence of foreshore or not?	Is there a significant mound present?	Is there a likelihood of impulsive overtopping conditions?	Mean overtopping discharge back of crest without bull nose[xxviii]	Mean overtopping discharge back of crest with bull nose
	ARI		DSWL		H_{m0}	T_{m-1}	β				$Q_{without\ bullnose}$	$Q_{with\ bullnose}$
[-]	[years]	[-]	[m AHD]	[m AHD]	[m]	[s]	[°TN]	[-]	[-]	[-]	[l/s/m]	[l/s/m]
1	1	Present day	1.28	2.90	0.7[xxii]	12.2	0	Yes	No	Yes	0.0 [0.0]	N/A) [xxix]
2	1	2093	1.94	1.70	0.5	12.2	0	Yes	No	Yes	0.0 [0.2]	N/A[xxix]
3	1	2100	2.00	0.35	1.5	12.2	0	Yes	No	Yes	5.9 [8.4]	N/A[xxix]
4	100	Present day	1.48	2.90	0.9[xxii]	13.6	0	Yes	No	Yes	0.0 [0.0]	N/A[xxix]
5	100	2093	2.14	1.70	0.7	13.6	0	Yes	No	Yes	0.5 [0.7]	N/A[xxix]
6	100	2093	2.14	-1.00[xxiii]	2.1	13.6	0	Yes	No	Yes	28.9 [41.22]	5.8) [xxx]
7	100	2100	2.20	0.35	1.7	13.6	0	Yes	No	Yes	12.7 [18.19]	N/A[xxix]

[xxix] *In a high-relative freeboard regime, the bullnose/wave return wall demonstrates optimal performance by effectively deflecting up-rushing water seaward. However, EurOtop reduction methods propose substantial reductions in overtopping, exceeding 95%. To validate these significant reductions, a thorough physical model study is recommended.*
[xxx] *This reduction needs to be confirmed trough physical modelling*

Table 5-12 Wave conditions used in assessment of overtopping Seawall 3

Case	Average Recurrence Interval	Planning Period	Water Level	Scour level[xxi]	Spectral Wave Height at the toe	Spectral Wave Period	Angle of attack relative to True North[xxvii]	Method steps			Overtopping	
								Is there an influence of foreshore or not?	Is there a significant mound present?	Is there a likelihood of impulsive overtopping conditions?	Mean overtopping discharge back of crest without bull nose[xxviii]	Mean overtopping discharge back of crest with bull nose
	ARI		DSWL		H_{m0}	T_{m-1}	β				$Q_{without\ bullnose}$	$Q_{with\ bullnose}$
[-]	[years]	[-]	[m AHD]	[m AHD]	[m]	[s]	[°TN]	[-]	[-]	[-]	[l/s/m]	[l/s/m]
1	1	Present day	1.28	2.90	0.7[xxii]	12.2	0	Yes	No	Yes	0.0 [0.0]	N/A[xxix]
2	1	2093	1.94	1.70	0.5	12.2	0	Yes	No	Yes	0.2 [0.3]	N/A[xxix]
3	1	2100	2.00	0.35	1.5	12.2	0	Yes	No	Yes	10.0 [14.0]]	N/A[xxix]
4	100	Present day	1.48	2.90	0.9[xxii]	13.6	0	Yes	No	Yes	0.0 [0.0]	N/A[xxix]
5	100	2093	2.14	1.70	0.7	13.6	0	Yes	No	Yes	0.8 [1.1]	N/A[xxix]
6	100	2093	2.14	-1.00[xxiii]	2.1	13.6	0	Yes	No	Yes	41.2 [58.8]	N/A[xxix]
7	100	2100	2.20	0.35	1.7	13.6	0	Yes	No	Yes	22.0 [31.4]	N/A[xxix]

Sensitivity to breaker index

Determining the design significant wave height at the structure toe is crucial for calculating overtopping discharge. The Goda methodology (Goda, Y, 2010), which links the incipient breaker index of significant waves to breaker depth, offshore wavelength, and beach slope, was used for this purpose.

Section 2.3.2 of EurOtop Manual (EurOtop, 2018) outlines the recommended approach for determining significant wave heights in depth-limited conditions at the structure toe. This method is based on the well-established methods of Goda (Goda, 2000), as illustrated in Figure 2.4 of EurOtop Manual (EurOtop, 2018).

It's important to acknowledge that the laboratory datasets supporting Goda's methods have inherent variability. Wave breaker indices on foreshore slopes of 1:30, as depicted in Figure 1c of Goda (Goda, Y, 2010), vary between 0.65 and 0.97 for relative water depths [xxxi] of 0.01-0.02 (similar to those of the proposed coastal protection structure), indicating a range of approximately 0.3.

The calculation of wave overtopping discharge is highly sensitive to the breaker index used to determine the design significant wave height at the structure toe. In the absence of specific justification and assurance, using a breaker index value based on methods presented in Section 2.3.2 of EurOtop Manual (EurOtop, 2018) is considered more suitable and conservative for design purposes without resorting to physical modelling. Physical modelling of the proposed seawall, which is to commence shortly [xxxii], would fully account for the complexity around breaker index, properly simulating the condition which would apply at the site.

Additionally, it's worth noting that bi-modal wave directions during storm events can impact wave heights at the structure. Observations, such as the June 2016 storm, indicate the occurrence of bi-modal wave spectra at the study location (Mortlock, Goodwin, McAneney, & Roche, 2017). Such conditions can lead to a superposition of wave heights, potentially resulting in increased wave overtopping discharge and volume. As bi-modal wave simulation is not possible in a 2D flume such as that proposed for the project, it appropriate that some analytical adjustment be considered to account for this process.

5.3.3.8 Wave overtopping mitigation measures

When facing the risk of excessive wave overtopping or 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 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:

[xxxi] Relative water depth is the ratio of water depth to wave length
[xxxii] At the time of writing, it is expected that physical modelling would commence in early March 2024 and be completed by late April 2024.

- 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 (refer to Section 15 of the BOMP (Waverley Council, 2024)).
- Trigger action response plans:
  - Include in the BOMP a robust framework for mitigating overtopping risks by defining response actions based on escalating trigger levels.
  - These actions and their expected frequency of occurrence can be implemented into construction or operational programs and adjusted as new data becomes avialabe or engineering modifications are made.

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.

These short-term measures are captured by the monitoring and response protocols for wave overtopping and coastal erosion Section of the BOMP (Waverley Council, 2024).

This Concept Design report examines wave overtopping based on desk-top techniques and develops strategies to manage the impacts over the life of the SLSC facility including the incorporation of a wave deflector and the expected reinforced concrete construction of external walls of the building at promenade level (refer **Section 10**). The need for and refinement of these and/ or other measures listed above would be considered at the next design stage. Further calculations or physical modelling would be required to precisely quantify the effectiveness of each proposed option[xxxiii].

5.3.4 Wave loads due to overtopping

Considering the wave runup calculations (refer to **Section 5.3.3.1**), the estimation of loads on the Bronte SLSC building has been conducted. Wave forces acting on the seaward face of the surf club comprise a hydrostatic component resulting from still water pressure and a dynamic component related to wave induced water movements. Various empirical techniques were employed based on the conditions generating the loading, including:

- Impact from wave runup (**Section 5.3.4.1**): This occurs when wave runup occurs on a partially eroded beach reaching the crest of the buried seawall and generating a bore-like discharge over the top of the wall.
- Direct wave impact (**Section 5.3.4.2**): This is applicable during events where the seawall is completely submerged due to elevated water levels and (reformed) breaking waves directly impinge on the building.

[xxxiii] *At the time of writing, it is expected that physical modelling would commence in early March 2024 and be completed by late April 2024*

The risk of overdesign can be effectively mitigated by using a physical model, as recommended in **Section 7**. Physical modelling is widely acknowledged for its capacity to decrease the risk of unforeseen failures and to offer a more precise representation, thereby minimising the likelihood of unexpected outcomes in design scenarios. Physical modelling often results in refined (reduced) wave overtopping and loading predictions, however in specific situations, results from physical modelling may exceed those obtained through desktop methods.

Physical modelling is proposed to inform the Detailed Design for the seawall and façade elements for the SLSC building. At the time of writing, it is expected that physical modelling would commence in early March 2024 and be completed by late April 2024.

5.3.4.1 Wave loads cause by wave runup (partially eroded beach)

For Concept Design and pending refinement using physical modelling[xxxiv] for Detailed Design, the following techniques were used to estimate the wave loads on the Bronte SLSC that would arise from wave runup reaching the crest of the proposed seawall that is buried or partially exposed and producing bore-like discharges:

1 Use the wave runup values calculated at the crest of the proposed seawall. Estimate the associated depth of water at the Bronte SLSC front wall (i.e., 10m from the seawall crest edge). Utilise the Federal Emergency Management Agency (FEMA) guidelines (FEMA, 2018) recommended method of Cox and Machemehl (1986) to calculate the inland limit of overtopping bore was adapted to compute the bore height and velocity profile overland.

2 Calculate velocities for the overtopping flow reaching the Bronte SLSC front wall by applying a decay of flow velocity along the crest and promenade using EurOtop (EurOtop, 2018).

3 Calculate wave loads on the Bronte SLSC front wall, comprising a hydrostatic component from water pressure and a hydrodynamic component due to horizontal bore velocity. The primary method for calculating wave forces is derived from the FEMA (FEMA, 2011) (refer to **Figure 5-12**).

[xxxiv] *At the time of writing, it is expected that physical modelling would commence in early March 2024 and be completed by late April 2024*

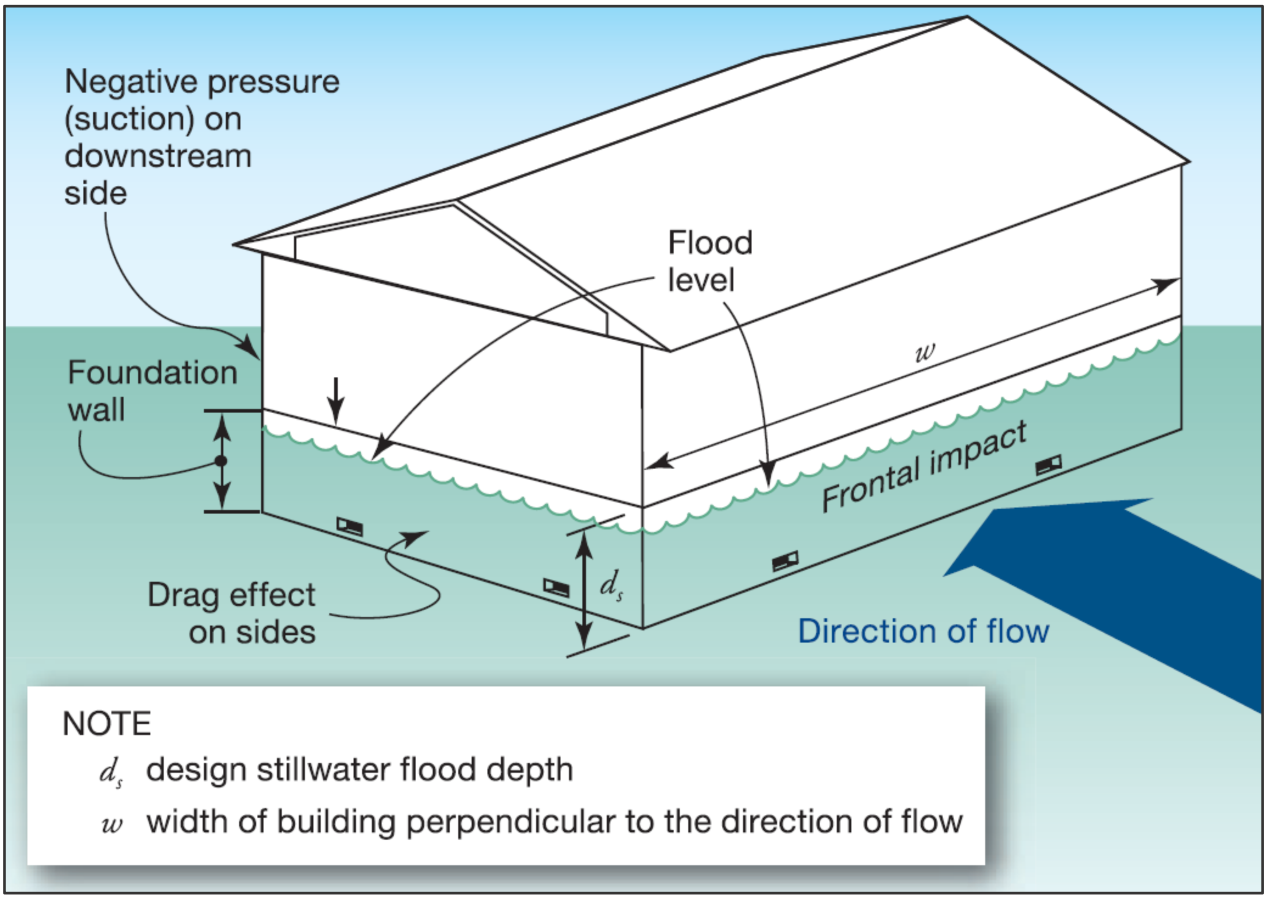


Figure 5-12 Hydrodynamic loads on a building *(FEMA, 2011)*

The estimated forces on the Bronte SLSC building due to wave runup are provided for both $R_{2\%}$ and R_{max} water levels, offering a range of potential impact loads. $R_{2\%}$ runup loads may be experienced a few times during the storm, while R_{max} runup loads represent the maximum expected during the considered design event. It is important to note that the hydrodynamic component's duration typically lasts around one wave period (10 to 15 seconds) before diminishing as overtopping dissipates between waves.

The calculated loads on SLSC building caused by wave runup reaching the crest of the buried seawall and creating a bore-like discharge over the top of the seawall are presented in **Table 5-13**.

Table 5-13 Loads on Bronte SLSC front wall caused by wave runup

Case	Average Recurrence Interval	Planning Period	Water Level	Run-up level exceeded by 2% of incident waves	Depth of $R_{2\%}$ at SLSC	Maximum run-up of all waves in a sea state	Depth of R_{max} at SLSC	Total Load $R_{2\%}$	Total Load R_{max}
	ARI		DSWL	$R_{u2\%}$		R_{umax}		F_{dyn}	F_{dyn}
[-]	[years]	[-]	[m AHD]	[m AHD]	[m]	[m AHD]	[m]	[kN/m]	[kN/m]
1	1	Present day	1.28	3.9	N/A	4.3	N/A	N/A	N/A
2	1	2093	1.94	4.6	N/A	4.9	N/A	N/A	N/A
3	1	2100	2.00	4.7	N/A	5.0	N/A	N/A	N/A
4	100	Present day	1.48	5.3	N/A	5.7	N/A	N/A	N/A
5	100	2093	2.14	5.9	≈0.0	6.3	0.3	≈0.0	1.0
6	100	2093	2.14	5.9	2.6	6.3	3.0	45.5	52.0
7	100	2100	2.20	6.0	1.4	6.4	1.7	15.6	19.0
8	500	2093	2.14	7.5	3.0	8.0	2.9	53.0	51.0

5.3.4.2 Wave loads cause by wave impact on exposed vertical seawall (scoured beach levels)

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, 2006) for impulsive wave loading.

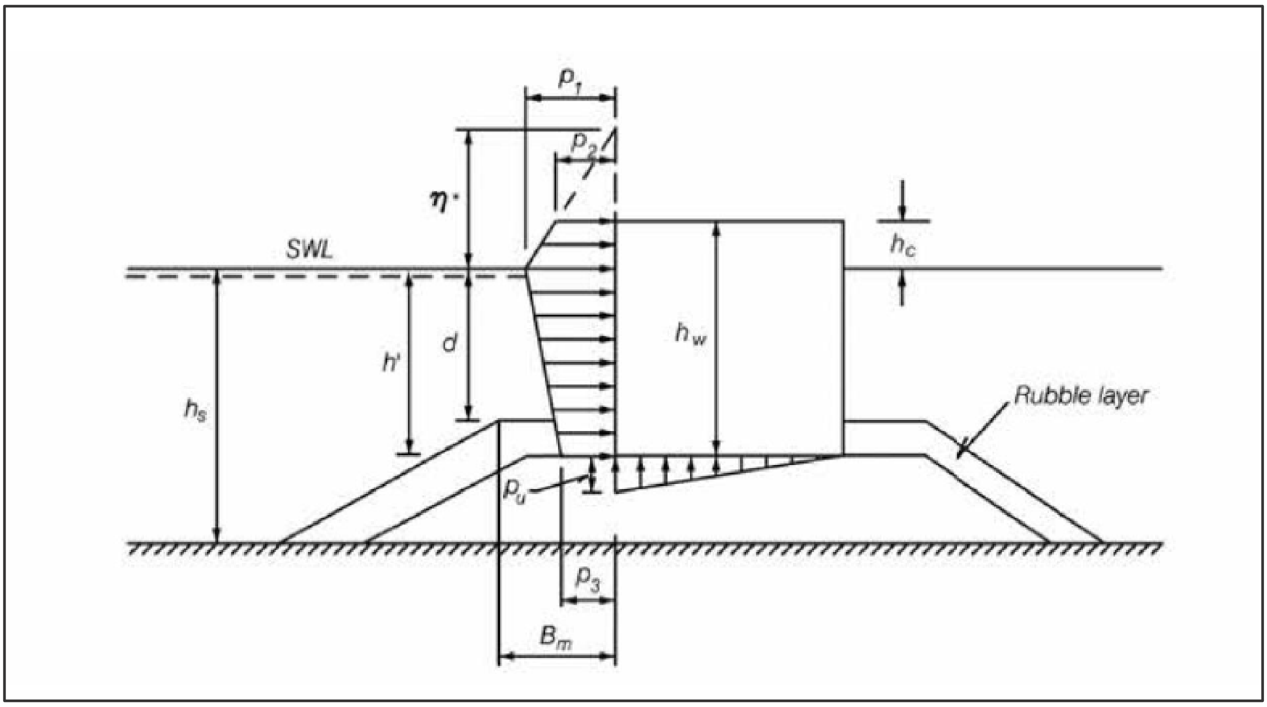


Figure 5-13 Hydrodynamic loads due to wave impact on a coastal structure *(U.S. Army Corps of Engineers, 2006)*

For Seawall 1, the available literature on wave impacts for stepped seawalls is limited. According to SPM (Coastal Engineering Research Center, 1984), the forces acting on stepped structures should be calculated for design purposes using the same method as for vertical walls, as the dynamic pressures are within a similar range. Therefore, for Concept Design purposes, the wave loads on Seawall 1 are of the same order of magnitude to the ones reported for Seawall 3 (refer to **Table 5-15**).

For Seawall 2, the pressure distribution towards the vertical elements was estimated, considering the different wall geometry and the effect of the concrete ramp in front of the purpose seawall (refer to **Table5-14**). 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 would be refined in next stages of the project.

For Seawall 3, the pressure distribution on the vertical wall considered the specific wall geometry at this location, and it was assumed that the depth in front of the seawall is the same as the design scour level (refer to **Table 5-15**).

It's important to note that existing desktop techniques do not include the potential reduction of the impact of waves hitting the Bronte SLSC building associated with the wave return wall. This refinement is expected to be provided from the physical modelling[xxxv] to take place as part of the subsequent design stage for the project.

The calculated loads on the Bronte seawall and SLSC building due to direct wave impact are presented in **Table 5-14** and **Table 5-15** for Seawall 2 and Seawall 3, respectively.

[xxxv] *At the time of writing, it is expected that physical modelling would commence in early March 2024 and be completed by late April 2024*

Table 5-14 Loads on Bronte SLSC front wall caused by direct impact wave Seawall 2

						Seawall			Bronte SLSC
Case	Average Recurrence Interval	Planning Period	Water Level	Spectral Wave Height at toe	Spectral Wave Period	Induced Horizontal Load	Hydrostatic Load	Total Load	Horizontal Load
	ARI		DSWL	H_{m0}	$T_{m-1,0}$	F_H	F_u	F	F_H
[-]	[years]	[-]	[m AHD]	[m]	[s]	[kN/m]	[kN/m]	[kN/m]	[kN/m]
1	1	Present day	1.28	0.7[xxii]	12.2	N/A	N/A	N/A	N/A
2	1	2093	1.94	0.5	12.2	N/A	N/A	N/A	N/A
3	1	2100	2.00	1.5	12.2	13.0	12.0	25.0	0.0
4	100	Present day	1.48	0.9[xxii]	13.6	N/A	N/A	N/A	N/A
5	100	2093	2.14	0.7	13.6	N/A	N/A	N/A	N/A
6	100)	2093	2.14	2.1	13.6	41.4	17.3	58.7	4.0
7	100	2100	2.20	1.7	13.6	25.4	13.2	38.6	0.0
8	500	2093	2.14	2.2	13.6	43.7	17.9	61.6	5.0

Table 5-15 Loads on Bronte SLSC front wall caused by direct impact wave Seawall 3

						Seawall		
Case	**Average Recurrence Interval**	**Planning Period**	**Water Level**	**Spectral Wave Height at toe**	**Spectral Wave Period**	**Induced Horizontal Load**	**Hydrostatic Load**	**Total Load**
	ARI		DSWL	H_{m0}	$T_{m-1,0}$	F_H	F_u	F
[-]	[years]	[-]	[m AHD]	[m]	[s]	[kN/m]	[kN/m]	[kN/m]
1	1	Present day	1.28	0.7[xxii]	12.2	N/A	N/A	N/A
2	1	2093	1.94	0.5	12.2	N/A	N/A	N/A
3	1	2100	2.00	1.5	12.2	73.0	11.0	84.0
4	100	Present day	1.48	0.9[xxii]	13.6	N/A	N/A	N/A
5	100	2093	2.14	0.7	13.6	15.4	6.1	21.5
6	100[)]	2093	2.14	2.1	13.6	180.0	16.7	196.7
7	100	2100	2.20	1.7	13.6	95.0	13.0	108.0
8	500	2093	2.14	2.2	13.6	190.0	17.2	207.2

6 Confirmation of seawall arrangement and structural intent

The design details of the existing 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 Beach rules out the feasibility of rock revetments, making a new concrete seawall the only practical choice. RHDHV'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 drop-down 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 cost of a sloping rock revetment, the secant pile wall offers the advantage of occupying a substantially narrower footprint, i.e., in the order of 1m compared 10m at the site depending on the sloping rock revetment 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 during storms. 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 building 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 seawall, 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 are to be closely considered. The inclusion of a wave return shape at the crest, achieved through angling the seaward face, and aiming 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 design life

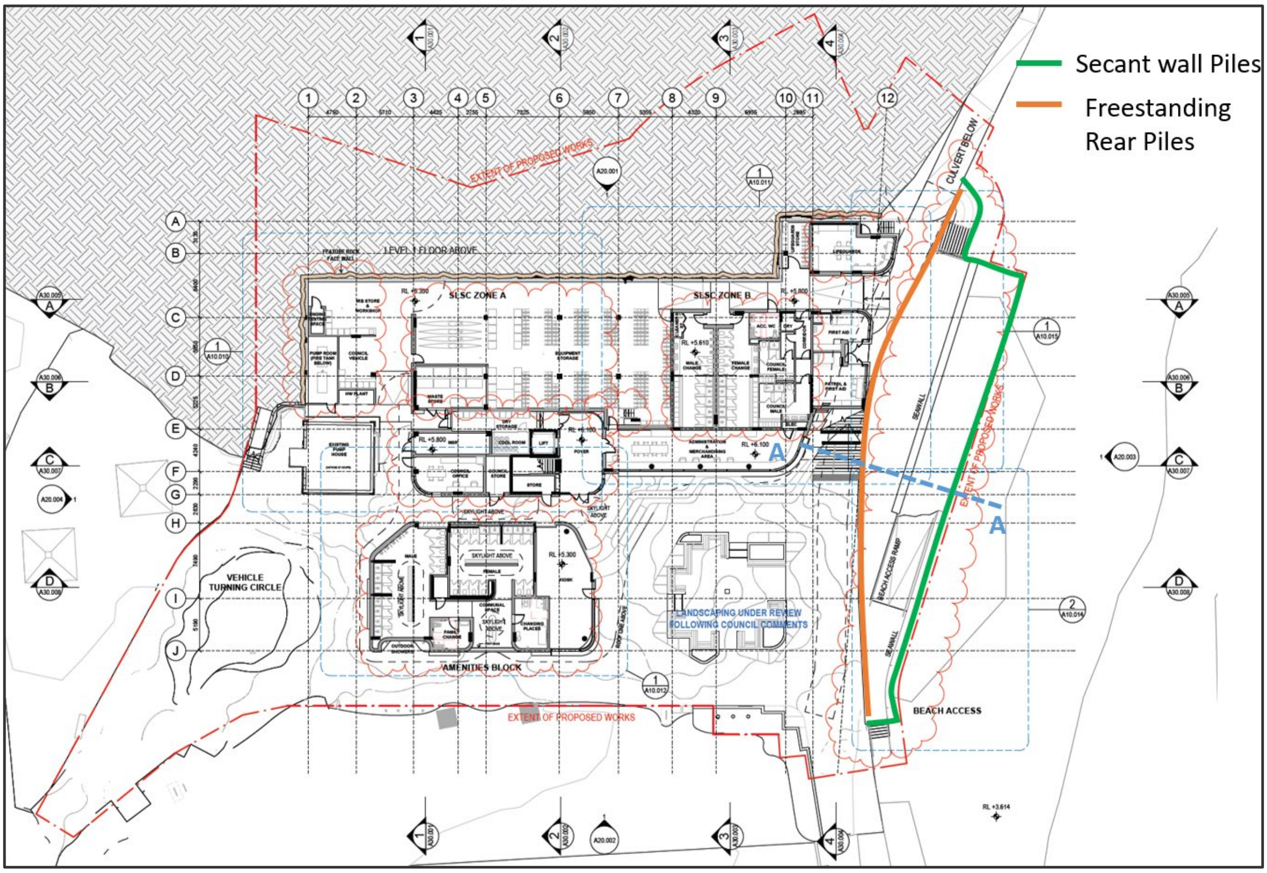
- Particular attention to the toes of structures is crucial (seawall, ramp, steps), considering potential impacts on beach fluctuations and climate change-induced shoreline recession

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 as deemed necessary.
- **Reinforced concrete drop-down beam**
  - A reinforced concrete drop-down 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 drop-down 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 adjoin the existing seawall at the southern and northern ends of the structure. Also, at the northern end the new seawall would adjoin the existing stormwater box culvert. The interaction of the new and existing structures would be considered in the design. The design of the junction of the structures would need to ensure that 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 in the order of approximately 50-100kN/m [xxxvi], with the line of action just below the waterline. The consequence of this loading for the design of the seawall is considered minor in that the space under the promenade, immediately behind the seawall, would be filled with suitable material (e.g., imported suitable soil) 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 is 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.



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

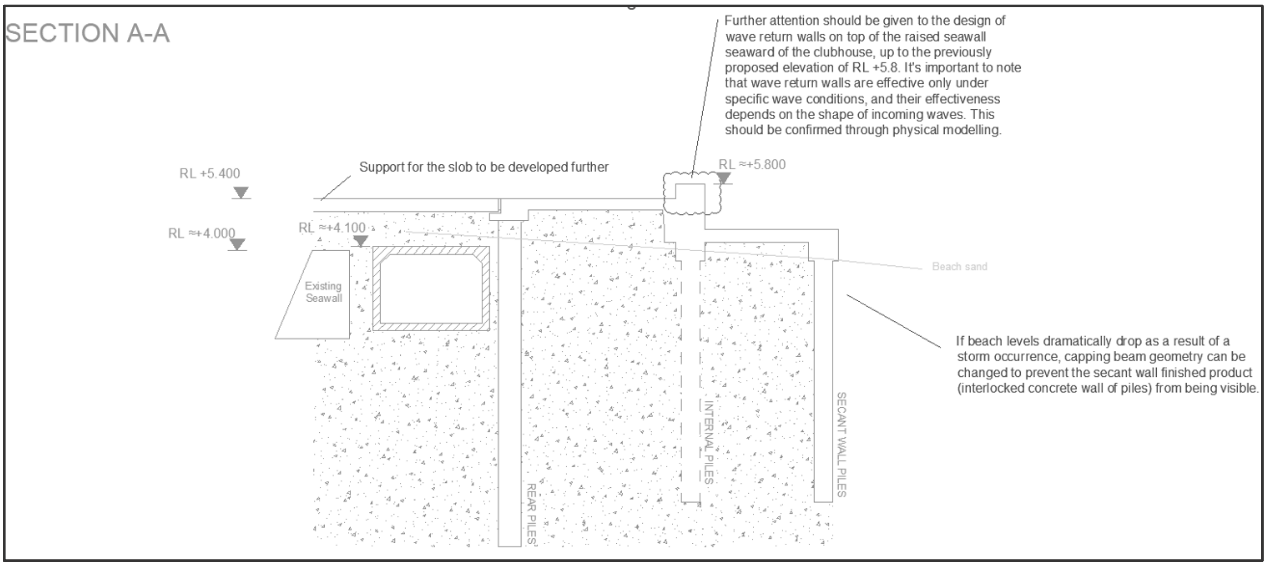


Figure 6-1 Proposed seawall and arrangement and structural intent

7 Physical modelling

7.1 Background

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.

7.2 Proposed physical modelling program

Coastal hydraulic physical modelling is proposed to take please at Water Research Laboratory (WRL) of UNSW, to enhance the quantification of wave overtopping flows, assess hydraulic loads, potential damage, and user safety. At the time of writing, it is expected that physical modelling would commence in early March 2024 and be completed by late April 2024.

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). It is proposed to use the 0.9m wide flume at WRL.

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.

At the time of writing, WRL and RHDHV have developed a testing programme that focuses in on simulations that would most effectively confirm the wave overtopping and design wave loading behaviour for the site, previously described using desk-top methods and reported herein. The model results would permit the overtopping flow impact in the vicinity of the SLSC to be scrutinised and modifications to the seawall wave deflector introduced if required. The results would also provide refined wave loads in order to finalise the structural design for the wave deflector and the seaward ground floor (promenade level) walls of the proposed SLSC building.

It is proposed to identify upfront a worst-case coastal profile for the centre of Bronte Beach which maximises wave runup and overtopping for a typical severe storm condition occurring today (20 yr ARI selected), chosen from either a fully eroded coastal profile potentially taken down to RL-1 in front of the seawall (unless limited by bedrock), and a critical ramped sand profile expected to develop during a severe storm event. The ramped sand profile would be sloped at 1:10, from an accreted beach level at the face of the new seawall (nom RL3.5m AHD, to be confirmed), down to Mean Sea Level (unless limited by bedrock). All subsequent testing in the physical modelling program would be conducted for the chosen critical coastal profile (i.e., nearshore model bathymetry). All substrates would be modelled in smooth plywood.

It is proposed to calibrate the flume, with the selected critical coastal profile, for conditions representing 1yr, 20yr and 100yr events occurring today and at the end of structure life in 2093 (6 calibration tests), and for a 500yr ARI storm event at the end of structure life (1 calibration test). In consultation with WRL, RHDHV would identify boundary water level and wave conditions applicable at RL-5m AHD for simulation in the flume. Inshore wave setup would develop naturally through the modelling process.

Two seawall sections are to be tested. Seawall section 1 at the steps (promenade level RL5.4m AHD) and seawall section 2 in the midportion of the ramp (deflector crest level RL5.8m AHD fronting the promenade at RL5.0m AHD). The seawall section 1 steps would be conservatively represented by a smooth profile sloped at 1:1.5, taken down to the nearshore model bathymetry. Seawall section 2 would be represented by a vertical seawall with deflector. A nominal configuration for the deflector, selected based on experience but to be confirmed, incorporates a 35 degrees rotation from the vertical and a 600mm long deflector blade. The 3m wide ramp section would be incorporated into seawall section 2 profile. Overtopping tests would be conducted at both seawall sections 1 and 2, with load tests limited to seawall section 2. A total of nine (9) overtopping tests and one (1) load test are proposed within the scoped testing program. Additional tests can be undertaken as may be needed to assess modifications to the seawall crest level and deflector configuration.

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;
- Waverley Local Environmental Plan 2012; and
- Waverley 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.*

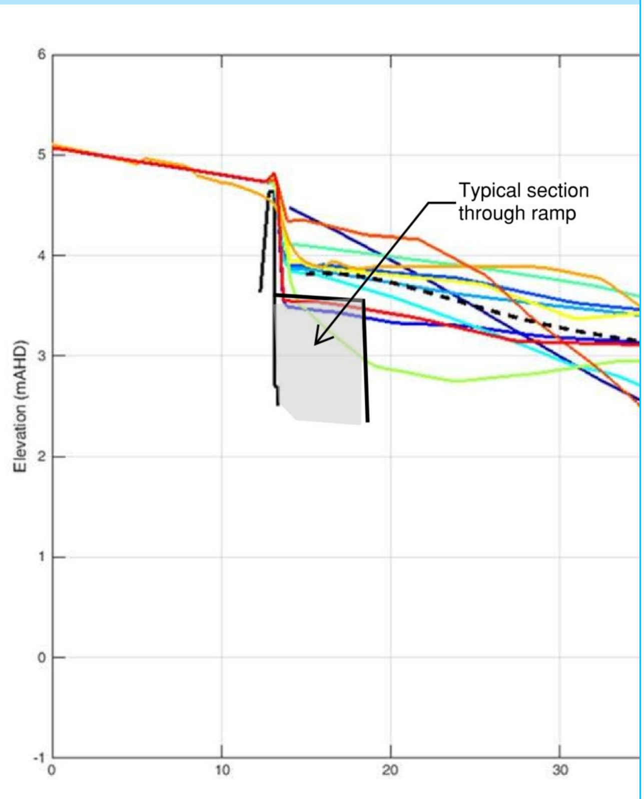
(b) *by payment to the relevant council of an annual charge for coastal protection services (within the meaning of the Local Government Act 1993).*

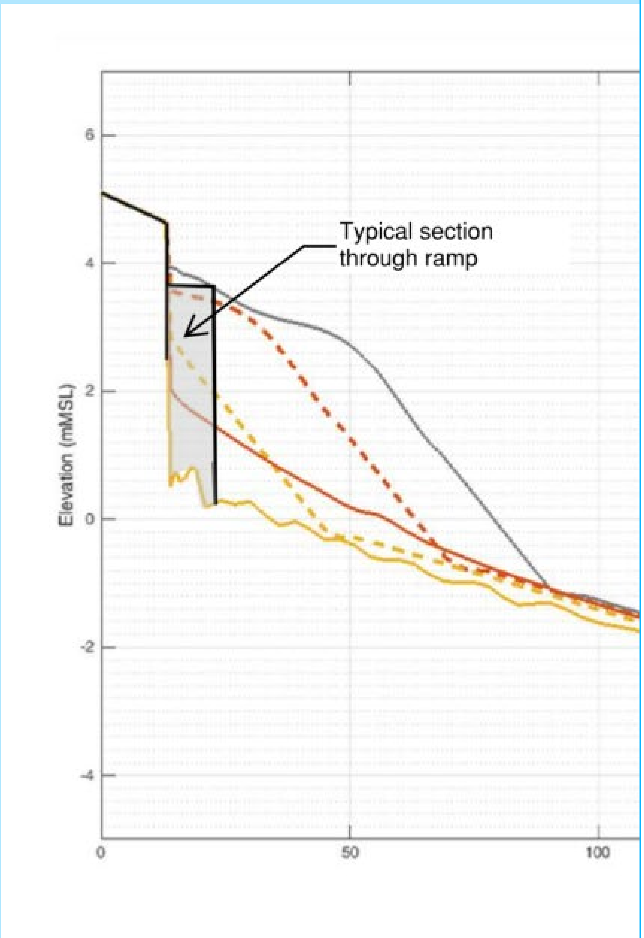
(3) *The funding obligations referred to in subsection (2) (a) are to include the percentage share of the total funding of each landowner, council or public authority concerned.*

Preliminary Coastal Assessment responses in relation to the Coastal Management Act 2016 are set out below. These would be reviewed and updated as required following the completion of the Stage 2 seawall design including wave return walls.

Table 8-1 Coastal Management Act 2016 – Comments and Assessment

Coastal Management Act 2016 Section 27	Comments/Assessment
(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	The proposal would facilitate and enhance public and lifesaving access between the beach and the SLSC area and promenade, by providing a new ramp and steps, and bleachers. The new seawall and beach access facilities protrude up to 10.4m onto the beach from the face of the existing seawall, extending over a shoreline distance of 59m (average protrusion assessed to be 5.2m for a typical present-day accreted back-beach level of RL3.7m AHD). The reduction in sandy beach width as a result of these works can be assessed with reference to the SBEACH modelling undertaken by Baird, discussed in Section 5.2 and reproduced in the figures below. These capture the nominal beach state today (2016), and in 2050 and 2100, before and following a 100 year ARI storm. The local reduction in sandy beach width as a result of the seawall and associated beach access structures is up to 10% today (2016), increasing to 14% in 2050 and 33% in 2100. For a normal beach state not affected by storms, high tide beach width in 2050 would reduce from 45m to 38m, and in 2100 from 24m to 17m. These changes, which occur locally in front of the proposed SLSC seawall, the overall length of which is approximately 25% of the length of the beach, are considered acceptable given the public and lifesaving benefits that the works provide. While the width at other areas along Bronte Beach would

Coastal Management Act 2016 Section 27	Comments/Assessment
	remain unchanged, it is acknowledged that not all the beach is used to the same degree with user density greatest where the flags are typically placed. With respect to the suitability of the design for beach access over the life of the works, beach recovery following severe storms would be initially relatively rapid, expected to mostly occur over a period of days to weeks. Immediately following these storms, Council may need to assist to reinstate the eroded beach at the base of the ramp and steps, scraping sand up to the proposed design toe level of 3m AHD. The proposed access to and from the beach is a substantial improvement over the existing situation. It is considered that the works would not, over the life of the works, unreasonably limit public access to or the use of the beach. Sectional profiles developed from photogrammetry and SBEACH modelling, overlaid on a typical section through the new ramp, are shown below:  Typical section through the ramp shown to be mostly covered with sand, based on beach profiles between 1970 and 2016. (Section L01 from Baird 2016, 20m

Coastal Management Act 2016 Section 27	Comments/Assessment
	south of centre of SLSC buildings. Coloured profiles show photogrammetrics surveys of the beach between 1970 and 2016 with black dash representing an average profile). Note that the ramp section here is simply overlaid onto the surveyed profiles. The influence of the ramp would have been minor on the profiles as surveyed.  Typical section through the middle portion of the ramp over the life of the works, showing the nominal existing (2016) average beach profile (grey), and model predicted profiles in 2050 and 2100 before (brown dash and yellow dash respectively) and following a 100-year ARI storm (brown full and yellow full respectively). (Section L00 from Baird 2016, 15m north of centre of SLSC buildings). Note that the ramp section here is simply overlaid onto the modelled profiles and is not included in the modelling. The influence of the ramp would result in the modelled profiles being slightly lower than those shown, particularly for the 2050 post-storm profile (brown full), and the 2100 pre-storm profile (yellow

Coastal Management Act 2016 Section 27	Comments/Assessment
	dash). Assuming a mass balance across the section, the adjustment would be expected to entail a lowering of up to 0.5m across the subaerial profile, from the face of the ramp out to the mid-tide waterline.
(ii) pose or be likely to pose a threat to public safety.	The proposed coastal protection works, over the life of the works, would not be expected to pose or be likely to pose a threat to public safety, in respect of the beach erosion/ shoreline recession hazard. The existing seawall, which is beyond its design life, could not be relied upon to protect the SLSC building. The proposed coastal protection works comprising a secant pile wall, drop-down beam, slabs and discreet CFA piles, when fully detailed, would be capable of preventing undermining of the SLSC building. The are rigorous operational methods outlined in the BOMP that would be activated should there be a coastal inundation hazard. The consent authority can be satisfied that a design solution, in combination with operational measures, could be found to ensure that the proposed works would not, over the life of the works, pose or be likely to pose a threat to public safety due to the coastal inundation hazard, but the design solution requires further development as part of the Detailed Design. 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. By implementing the proposed works it would not be necessary to carry out emergency erosion protection works during and after storms, at which times staff of emergency agencies and volunteers would otherwise place themselves at some safety risk.
(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	To make an assessment in this regard it is first necessary to consider whether any increased erosion of the beach or adjacent land would be

Coastal Management Act 2016 Section 27	Comments/Assessment
erosion of the beach or adjacent land is caused by the presence of the works,	caused by the presence of the works. This can be considered under three main topics: (i) additional scour/ erosion immediately seaward of the works; (ii) end effects on immediately adjacent land; (iii) consequences due to 'locking up' of sand behind the coastal protection works. <u>Additional scour/ erosion immediately seaward of the works.</u> Research has shown that concerns that seawalls cause additional scour/ erosion immediately seaward and greatly delay post-storm beach recovery are probably false, as there are no known data or physical arguments to support these concerns (U.S. Army Corps of Engineers, 2006). Furthermore, and more importantly in relation to Bronte Beach, there is an existing seawall that merges with an adjacent bedrock cliff to the north that together, effectively protect the full beach compartment. As such, the proposed works which have the effect of shifting seawards by an average of approximately 5m [xxxvii] a 60m sub-length of the 250m back beach shoreline seawall/ bedrock cliff would not be expected to cause any significant increase in scour/ erosion immediately seaward of the works compared to the existing situation. At the time of finalising the Concept Design report an additional geotechnical site investigation had recently been completed. The field work comprised boreholes, test pits, DCP tests and sub-ground seismic refraction. The investigation has yet to be fully reported on, however preliminary findings have confirmed bedrock levels in the vicinity of the proposed new seawall between 0.0 and -0.9m AHD, limiting potential scour at the seawall toe. <u>End-effects on immediately adjacent land</u> Increased erosion of immediately adjacent land could potentially occur due to end-effects, caused by localised wave reflections and diffraction, due to the presence of seawall works.

[xxxvii] *Average 5.2m at sand level of RL3.7m AHD*

Coastal Management Act 2016 Section 27	Comments/Assessment
	However, no erosion of immediately adjacent land is expected as a result of the proposed coastal protection works, as the proposed works are located adjacent to, and at their ends merge with, the existing Bronte Beach seawall (to the south) and the stormwater culvert, bedrock cliff and headland (to the north). However, design consideration would need to be given to the potential additional localised scour adjacent to the works at times of storms that impact the works, subject to the location and elevation of bedrock. <u>Consequences due to 'locking up' of sand</u> There are two potential consequences of the 'locking up' of sand behind the coastal protection works: (i) additional localised erosion to meet the storm erosion demand; and (ii) impact on long-term shoreline recession. The volume of sand, potentially 'locked up' behind the coastal protection works is found to be small and immaterial to this particular risk, as demonstrated below. The estimated volume of sand potentially 'locked up' behind the coastal protection works as far landward as the 2070 coastal hazard line (refer Figure 5-9), measured above 0m AHD, is approximately 200m^3. Distributing this volume over the depth of the active profile and the length of Bronte Beach would give an equivalent shoreline recession of less than 0.1m to 2070. The above underlying shoreline recession estimate due to a net sediment loss to 2093 may be compared to the expected shoreline recession to 2093 due to sea level rise, equal to 46m (Baird, 2016) [xxxviii], and is therefore less than 0.5% of the total estimated shoreline recession. Notwithstanding the prediction for potential impoundment of beach sand behind the works to be of minimal impact to long-term recession, it is proposed that the existing sand that is potentially 'locked up' by the new works is removed and placed

[xxxviii] *Refer to loss of beach widths in Section 5.2.2.*

Coastal Management Act 2016 Section 27	Comments/Assessment
	on the beach, and that suitable imported filling is placed and compacted to replace the native sand. It follows that for the Bronte seawall project no sand would in fact be lost from the beach compartment immediately following the completion of the works. <u>Synthesis</u> 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, due to the closed system and availability of sand. Increased erosion on the beach (if any) would only be expected to be short term and not be significant. There are no end-effects expected as a result of the proposed works, as the proposed works merge with the existing seawall or bedrock cliff/ rocky headland, and there are no unprotected erodible materials behind the flanks of the works. Impoundment of sand behind the new coastal protection works are assessed to be minimal, or non-existent if sand is removed and replaced with imported fill (which is proposed), and therefore of no consequence to shoreline recession. Notwithstanding the findings above, if any mechanical intervention is desired to accelerate beach recovery, Council has the means to undertake beach scraping. Council owns a posi-track and beach rake which regularly scrapes sand at its beaches to the levels required for beach cleaning, safety, access and after storm events. In large storm events and sand washouts, Council hires excavators for moving sand and cleaning up debris.
(ii) the maintenance of the works.	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 and maritime engineer to undertake an inspection after severe storms that expose the works and advise on any required remedial action. Due to the basis of design, and the checking and governance processes employed throughout the design and construction of the works, the need for significant maintenance over the life of the works would not be expected. In the event significant maintenance was necessary, potential maintenance activities could include (adapted from Horton Coastal Engineering, 2023):

Coastal Management Act 2016 Section 27	Comments/Assessment
	• Inspection of the seawall after significant coastal storms. This would include inspection of the seaward side of the wall for any damage to the concrete structure, gap formation in the secant piling (where visible), and integrity of weepholes. This would also include inspection of the landward side for evidence of the formation of any significant cracking of concrete slabs indicating possible migration of fill though the seawall and loss of fill compaction, and/or wall movement, 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, given that high strength concrete and appropriate cover to reinforcement would be specified for the proposed 70 year life of the structures. • 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), or 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 any gaps, as these defects would be their responsibility to correct and would be inspected during the course of the works by the project engineers. • If any weepholes were found to be leaking soil they could be filled with concrete. All weepholes would not be necessary for structural integrity of the wall since the wall would be designed assuming limited drainage. 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 BOMP,

Coastal Management Act 2016 Section 27	Comments/Assessment
	and relevant asset management and maintenance plans. It is proposed that a draft condition would be prepared to ensure compliance by the Applicant with Section 27 (1)(b)(ii), hence the matter of maintenance of the works over the life of the works would be addressed.
(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:	It is understood a draft condition would be prepared to satisfactorily address Section 27(2). Funding arrangements are not strictly a coastal engineering matter, although it is noted that calculation of the dollar amount to ensure adequate funding may require coastal engineering input (Horton Coastal Engineering , 2023).
(a) by legally binding obligations (including by way of financial assurance or bond) of all or any of the following—	Refer above
(i) the owner or owners from time to time of the land protected by the works,	Refer above
(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.	Refer above
(b) by payment to the relevant council of an annual charge for coastal protection services (within the meaning of the *Local Government Act 1993*).	Refer above
(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.	Not applicable

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.

Preliminary Coastal Assessment responses in relation to the Coastal Vulnerability Area of SEPP (Resilience and Hazards) 2021 are set out below. These would be reviewed and updated as required following the completion of the Stage 2 seawall design including wave return walls.

Table 8-2 Coastal Vulnerability Area - Comments and Assessment

SEPP Clause 2.9	Comments/Assessment
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	The consent authority can be satisfied that the proposed works would be engineered to withstand the current and projected beach erosion/ shoreline recession for the design life of the works (70 years), having regard to the basis

SEPP Clause 2.9	Comments/Assessment
hazards for the design life of the building or works	of design set out in Section 4, the peer review (commenced but to be completed), and the coastal engineering advice based on Baird (2016), and further developed by Horton (2023) and RHDHV for this report. The Detailed Design would be completed in due course, having regard to the full results of the additional geotechnical investigation (expected in March 2024), physical modelling investigation (expected to commence in early March 2024 and be completed by late April 2024), and dedicated maritime structural design development for the coastal protection works.
(b) the proposed development: **(i) is not likely to alter coastal processes to the detriment of the natural environment or other land**	The proposed works are not expected to alter coastal processes into the future to the detriment of the natural environment or other land given the beach morphological responses described above. The condition of consent referred to above in relation to Section 27 (1)(b)(i) of the Coastal Management Act 2016 would be triggered to restore the land as a result of any increased erosion caused by the presence of the works. It is noted here that the wording of sub-clause 2.9 (b)(i) in State Environmental Planning Policy (Resilience and Hazards) 2021 is somewhat at odds with sub-clause 27 (1)(b)(i) in the Coastal Management Act 2016 which specifically anticipates that coastal protection works may increase erosion but that this is only acceptable if conditions can be imposed to restore it. It is understood that if there is any inconsistency between the Policy and the Act, the Act would override the Policy.
(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	The proposal improves the public amenity of the Coastal Walk and Bronte Park in the immediate vicinity of the upgraded SLSC building. The promenade spaces to cater for longshore pedestrian access are slightly widened, assisting with through traffic. Importantly with respect to access, this is enhanced and direct between the beach and the SLSC area. With the north down ramp alignment and new steps and bleachers at the northern end, beach users are directed to the north so improving access to the only and safest

SEPP Clause 2.9	Comments/Assessment
	area on the beach for the lifeguards to put the flags up. The proposed seawall and related access structures would protrude an average of 5.2m and up to 10.4m seaward from the face of the existing seawall. For a typical back beach level of RL3.7m AHD, the proposed seawall and related access structures would reduce the high tide drying minimum back-beach widths in this area from 53m today (no new wall), to 19m in 70 years time (no new wall), or 10m [xxxix] in 70 years time with the proposed new seawall. The average high tide drying minimum back-beach width opposite the proposed new seawall in 70 years is assessed to be 11m [xl]. While there is a significant reduction in the available high tide drying minimum back-beach width which would apply at the end of the 70 year life, it is assessed that the sandy beach fronting the SLSC would remain fully accessible to longshore pedestrian movements over the life of the upgraded facility. Headlands and rock platforms are well removed from the proposed structures so access to these features would not be affected. The net impact on amenity and access is considered to be modest and acceptable in relation to the overall outcome of the seawall upgrade for the SLSC redevelopment.
(iii) incorporates appropriate measures to manage risk to life and public safety from coastal hazards	
(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	The proposed seawall upgrade addresses the unacceptable condition of the existing seawall, restoring stability to the shoreline and protecting the new SLSC from coastal erosion over the design working life of the seawall (70 years). The crest level of the existing seawall would be raised by between 0.5 and 1.1m (average 0.8m), predicted to significantly reduce the threat to public safety from the effects of wave overtopping.

[xxxix] Critical location along the proposed new seawall results in a 9m reduction here, compared to a 10.4m reduction which would apply in if considered in average terms.
[xl] It is assessed that changes to future fairweather back-beach levels as indicated from Baird's modelling would make no material difference to this assessment. While post storm beach recovery is expected to be rapid (mostly occurring in days to weeks), it is possible that Council may need to intervene with local scraping to provide immediate improvements to long-shore pedestrian access.

SEPP Clause 2.9	Comments/Assessment
	To mitigate the risk to life and public safety from the effects of wave overtopping, it is provisionally recommended that pedestrians be excluded from the promenade area between the SLSC and the upgraded seawall as follows[xli],. These recommendations, to be actioned in the BOMP, would be reviewed and updated if necessary following the completion of physical modelling. • For present day sea level conditions, during storm events, no threat is predicted for storms up to 100 year ARI hence no exclusions need apply. • For sea level conditions predicted at 2093, at the end of the design working life of the new seawall, during storm events exceeding approximately 1 and 100yr ARI for some incident water depth conditions.

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—

[xli] *Storm recurrences to be reviewed following completion of Detailed Design and having regard to the results of the physical modelling.*

Royal HaskoningDHV

WARREN AND MAHONEY

(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.

Preliminary Coastal Assessment responses in relation to the Coastal Environment Area of SEPP (Resilience and Hazards) 2021 are set out below. These would be reviewed and updated as required following the completion of the Stage 2 seawall design including wave return walls.

Table 8-3 Coastal Environment Area - Comments and Assessment

SEPP Clause 2.10	Comments/Assessment
(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	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 stormwater drainage arrangements are not to be significantly altered, 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 addressed in this report 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 (adapted from Horton Coastal Engineering, 2016).
(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	The proposed works would not adversely impact on water quality as long as appropriate construction environmental controls are applied.
(d) marine vegetation, native vegetation and fauna and their habitats, undeveloped headlands and rock platforms	Not a coastal engineering consideration.

SEPP Clause 2.10	Comments/Assessment
(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	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 all ability access ramp (adapted from Horton Coastal Engineering, 2016). The average 5.2m excursion of the works onto the beach, assessed for a typical back beach level of RL3.7m AHD and over the total 59m length of the seawall works, is considered to be acceptable in relation to its impact on access along the beach as discussed above. Refer also to discussion on public access in earlier responses, eg. in relation to Section 27 (1)(a)(i) of the CM Act 2016 and Clause 2.9 (b)(ii) of the SEPP (Resilience and Hazards) 2021.
(f) Aboriginal cultural heritage, practices and places	Not a coastal engineering consideration.
(g) the use of the surf zone	The proposed works would not be likely to cause an adverse impact on use of the surf zone as the works are located at the back of the beach and would only be expected to interact with the surf in severe storms. Use of the surf by beachgoers would not be expected at such times. The works enhance public access to the beach and 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	It is considered that the proposed works have been generally designed, sited and managed to avoid, minimise and mitigate the impacts referred to in subsection (1). <u>Note</u>: this statement is made on the basis that siting of the SLSC building is constrained to its existing location as previously demonstrated by Warren & Mahoney. In particular it is noted that: • the proposed coastal protection works are sited as far landward as practicable;

SEPP Clause 2.10	Comments/Assessment
	• a maintenance plan would be prepared as a condition of consent; • a condition of consent would be imposed to ensure satisfactory arrangements are in place, for the life of the works, for restoration of the beach and land adjacent to the beach, if increased erosion of the beach or adjacent land is caused by the presence of the works. Management of the SLSC facility to avoid adverse impacts are outlined in the BOMP.
(b) if that impact cannot be reasonably avoided—the development is designed, sited and would be managed to minimise that impact, or	Refer above
(c) if that impact cannot be minimised—the development would be managed to mitigate that impact.	Refer above

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.*

Preliminary Coastal Assessment responses in relation to the Coastal Use Area of SEPP (Resilience and Hazards) 2021 are set out below. These would be reviewed and updated as required following the completion of the Stage 2 seawall design including wave return walls.

Table 6-4 Coastal Use Area - Comments and Assessment

SEPP Clause 2.11	Comments/Assessment
(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	The proposed works would enhance beach access, as discussed previously. Refer to discussion on public access in earlier responses, eg. in relation to Section 27 (1)(a)(i) of the CM Act 2016 and Clause 2.9 (b)(ii) of the SEPP (Resilience and Hazards) 2021.
(ii) overshadowing, wind funnelling and the loss of views from public places to foreshores,	Not coastal engineering considerations.
(iii) the visual amenity and scenic qualities of the coast, including coastal headlands	The writer acknowledges that he is not an expert in visual impact assessment, however a number of coastal engineering considerations influence the potential for coastal protection works to impact on the visual amenity and scenic qualities of the coast hence it is considered reasonable to provide some commentary below on the visual amenity aspects.

SEPP Clause 2.11	Comments/Assessment
	In order to satisfy the State Environmental Planning Policy (Resilience and Hazards) 2021 the proposed coastal protection works must be engineered to withstand current and projected coastal hazards over the design life of the works, and must incorporate appropriate measures to manage risk to life and public safety from coastal hazards, eg. refer clauses 2.9(a) and 2.9(b)(iii) of the Policy. The requirement to satisfy the Policy dictates that the proposed works must have a certain structural robustness, eg. be able to accommodate without failure the design wave conditions, beach scour level, geotechnical conditions, etc. and must have a minimum crest level eg. to manage the wave overtopping and inundation risk to life and risk to property to an acceptable level. The outcome is necessarily a substantial structure. The cross-shore position of coastal protection works also influences the potential for the works to impact on visual amenity. Generally, the works should be located as far landward as possible, which also benefits other factors such as potential impacts of the works on coastal processes. The proposed works are located as far landward as possible while meeting the objectives of providing improved public access and improved access for life saving, and comprise structural elements common for coastal protection works, eg. secant pile wall and drop-down beam. It is proposed to incorporate the following to further mitigate potential visual amenity impacts by measures such as: • inclusion of a deflector to reduce otherwise the seawall crest level (while still achieve acceptable wave overtopping), mitigating impacts on views from the promenade to the beach, and reducing the height the wall when viewed landward from the beach particularly at times of low beach levels following storms; • Detailed Design to reduce the visible upper portion of the secant pile wall at times of low beach levels, e.g., by adoption of a deep drop beam; • colouring the concrete to match the colour of the beach sand.

SEPP Clause 2.11	Comments/Assessment
(iv) Aboriginal cultural heritage, practices and places	Not a coastal engineering consideration.
(v) cultural and built environment heritage	Not a coastal engineering consideration.
(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	The proposed development has been designed and sited to avoid any potential adverse impacts referred to in Clause 2.11(1).
(ii) if that impact cannot be reasonably avoided—the development is designed, sited and would be managed to minimise that impact, or	Refer above
(iii) if that impact cannot be minimised—the development would be managed to mitigate that impact	Refer above
(c) has taken into account the surrounding coastal and built environment, and the bulk, scale and size of the proposed development	Not coastal engineering considerations.

8.2.5 Division 5 General

The relevant clause is reproduced below followed by comments and assessment in Table 8-4.

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.

Table 8-4 General – Comments and Assessment

	Comments/Assessment
SEPP Clause 2.12	
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 potential failure of the existing seawall fronting the SLSC and 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 or protected by natural bedrock features. The potential for increased localised scour adjacent to the works would be addressed by design, subject to the level of bedrock which would provide natural scour protection.

	Comments/Assessment
SEPP Clause 2.13	
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.

8.3 Waverley Local Environmental Plan 2012

As noted in Horton Coastal Engineering (2016), 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.

The proposed Waverley LEP 2022 provides minor updates to the 2012 LEP. Changes proposed under the draft WLEP 2022 were finalised as Amendment 24 to the WLEP 2012 in September 2022. Coastal engineering issues are also absent from Amendment 24.

8.4 Waverley Development Control Plan 2022

As noted in Horton Coastal Engineering (2016), 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.

8.5 Waverley Council Coastal Risk Management Policy

As noted in Horton Coastal Engineering (2016), 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.

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 will 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 Goncalves (JG) from RHDHV, James Morgan (JM) and Sven Ollmann (SO) from W&M, and Robert Sabato (RS) from Waverley 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 A3**. 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
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 the '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 cannot get a suitable handle on via desk-top assessment) 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 Waverley Cemetery, and a proposal for a seawall with wave deflector. Similar to Coll Narr deflector. [RHDHV has now received this from Waverley 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 drop-down beam.	GWB	

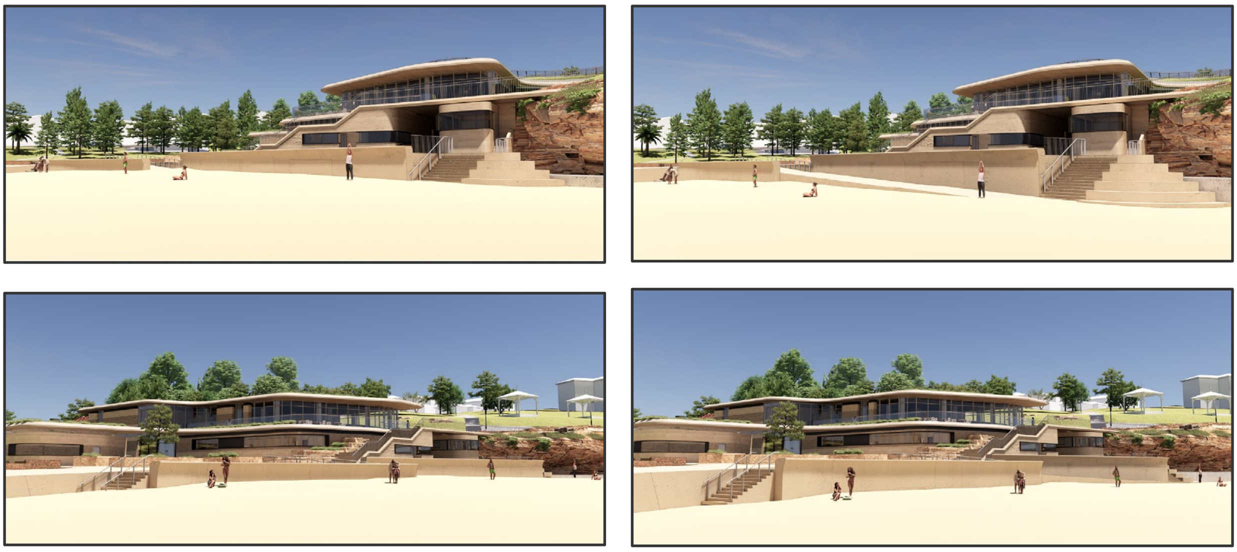


Figure 9-1 View analysis (Left) Reduced sand levels +3.00m AHD (Right) current sand levels +4.00m AHD. The recommend design scour level under present day climate conditions at the peak of design storm is +2.9m AHD (present day), reducing to +1.70m AHD (2050) and +0.35m AHD (2100) (refer to Section 5.2.3).

At the time of updating this report the Peer Reviewer had reviewed draft Version 1 of the report (5/12/23) and comments from the peer review had been attended to in preparing the current Version 3 report. Version 2 (20/12/23) was distributed internally for discussion with Warren & Mahoney. A copy of the Peer Review of the Version 1 report is attached at **Appendix A4**.

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. At the time of updating this report the coordination has involved TTW as the SLSC building Structural Engineer, regarding management of wave loads and interface of the landside promenade (TTW) with the promenade extension into the new seawall structure (RHDHV). Based on the desk-top assessment of wave loads developed for Concept Design it is TTW's expectation that reinforced concrete construction would be required for the external walls of the SLSC building at promenade level (Structural Workshop No 2, Padraig Clery TTW 6/2/24, pers comm).

This Concept Design report should be provided to the façade specialist (Prism Facades) regarding the management of wave loads at windows openings at promenade level.

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.

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11 References

AGE. (2020, March 20). Proposed Bronte Surf Club & Community Facility Upgrade, Bronte Beach, Bronte NSW. Asset Geo Enviro, Our ref: 5613-G1.

ARUP. (2016, Septemebr 27). Bronte Seawall Technical Study. *Report 249632-REP-01*. Sydney, New South Wales, Australia: Waverley Council.

ARUP. (2016, September 27). Bronte Seawall Technical Study: Ground Investigation Factual Report. *249632-REP-02*. Sydney, New South Wales, Australia: Waverley Council.

AssetGeoEnviro. (2020, March 26). Geotechnical Investigation. *Proposed Bronte Surf Life Saving Club Facilities Upgrade Bronte Beach, Bronte NSW*.

AssetGeoEnviro. (2022, January 19). Additional Geotechnical Investigation. *Proposed Bronte Surf Life Saving Club Facilities Upgrade Bronte Beach, Bronte NSW*.

Athanasiou, P., van Dongeren, A., Giardino, A., Vousdoukas, M., Gaytan-Aguilar, A., & Ranasinghe, R. (2019). Global distribution of nearshore slopes with implications for coastal retreat. *Earth Syst Sci. Data, 11, 1515–1529*.

Australian Geomechanics Society. (2007, March). Landslide Risk Management. *Australian Geomechanics, Volume 42(No 1)*. Australian Geomechanics Society .

Baird. (2016). *BRONTE SEAWALL TECHNCIAL STUDY – COASTAL PROCESS ASSESSMENT.* 14 Septemer 2016, Reference # 12569.101.L1.Rev0, prepared for Arup.

Battjes, J, & Groenendijk, H. (2000). Wave height distributions on shallow foreshores. *Coastal Engineering 40 (2000) 161–182*. ELSEVIER.

BMT. (2020, June). Stage 1 Scoping Study. *Eastern Beaches CMP* . Woollahra, Waverley and Randwick Councils.

BRITISH STANDARD. (2016). Maritime works – Part 1-2: General – Code of practice for assessment of actions. *BS 6349-1-2:2016*. The British Standards Institution.

Carley, J. T., Coghlan, I. R., & Flocard, F. (2015, September 15-18). Establishing the Design Scour Level for Seawalls. *Australasian Coasts & Ports Conference*. Auckland, New Zealand.

Coastal Engineering Research Center. (1984). Shore Protection Manual. *Coastal Engineering Research Center: Shore protection manual,*. Washington , Washington, D.C., USA: Department of the Army US Army Corps of Engineers.

Coastal Risks and Hazards Vulnerability Study. (2011, December). *301015-02526-CS-REP-0001*. Waverley Council.

CSIRO. (2009). Marine and atmospheric research sea level rise. *website www.cmar.csiro.au/sealevel/, accessed 26 October 2009*.

Department of Environment, Climate Change and Water. (2010). Coastal Risk Management Guide. *Incorporating sea level rise benchmarks in coastal risk assessments*. NSW Government.

EurOtop. (2018). Manual on wave overtopping of sea defences and related structures. *An overtopping manual largely based on European research, but for worldwide application*. Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B. Retrieved from www.overtopping-manual.com

FEMA. (2011, August). Principles and Practices of Planning, Siting, Designing, Constructing, and Maintaining Residential Buildings in Coastal Areas. *Coastal Construction Manual, Volume II*. Federal Emergency Management Agency .

FEMA. (2018, Feburary). Coastal Wave Runup and Overtopping. *Guidance for Flood Risk Analysis and Mapping*. Federal Emergency Management Agency.

Glatz, M., Fitzhenry, M., & Kulmar, M. (2017, November). It's Time for an Update - Extreme Waves and Directional Distributions Along the New South Wales Coastline. Port Stephens: Paper presented at the 26th New South Wales Coastal Conference.

Goda, Y. (2010, January 5). Reanalysis of Regular and Random Breaking Wave Statistics. *Coastal Engineering Journal, Vol. 52, No. 1 (2010) 71–106*. World Scientific Publishing Company and Japan Society of Civil Engineers. doi:10.1142/S0578563410002129

Goda, Y. (2000). Random Seas and Design of Maritime Structure. *Advanced Series, 2nd edition*. Singapore: World Scientific. doi:ISBN 981-02-3256-X

Gordon, A., Carley, J., & Nielsen, A. (2019, September). Design Life and Design for Life. Hobart: Australasian Coasts & Ports 2019 Conference.

Guza, R., & Thornton, E. (1981, May 20). Wave Set-Up on a Natural Beach. *JOURNAL OF GEOPHYSICAL RESEARCH, VOL. 86(NO. C5)*, 4133-4137.

Holman, R. A. (1986, March). Extreme value statistics for wave run-up on a natural beach. *Coastal Engineering, 9(6)*, 527-544. Elsevier B.V.

Horton Coastal Engineering . (2023, July 31). Coastal Risk Assessment and Coastal Engineering Advice on Bronte Surf Lifesaving Club and Community Facility Redevelopment. *issue 2*.

Intergovernmental Panel on Climate Change [IPCC]. (2021). Climate Change 2021: The Physical Science Basis. Contribution of Working Group I to the Sixth Assessment Report of the Intergovernmental Panel on Climate Change. (V. P. Masson-Delmotte, Ed.) Cambridge and New York, United Kingdom and USA: Cambridge University Press. doi:10.1017/9781009157896

JK Geotechnics. (2024, February 20). Geotechnical Investigation. *PROPOSED SEAWALL UPGRADE - BRONTE SURF LIFE SAVING CLUB, BRONTE ROAD, BRONTE, NSW*. JK Geotechnics.

JK Geotechnics. (2024). *Geotechnical Investigation - Proposed Seawall Upgrade, Bronte SLSC, Bronte Road, Bronte, NSW .*

Louis, S., Couriel, E., Lewis, G., Glatz, M., Kulmar, M., Golding, J., & Hanslow, D. (2016). NSW East Coast Low Event - 3 to 7 June 2016. *Weather, Wave and Water Level Matters*.

Manly Hydraulic Laboratory . (2022, February). NSW WAVE CLIMATE ANNUAL SUMMARY 2020–2021. *Report MHL2858*. NSW Department of Planning, Industry and Environment (Biodiversity and Conservation Division).

Manly Hydraulics Laboratory . (2016, December). Collaroy-Narrabeen Beach. Coastal Protection Assessment. *Report MHL2491*. Northern Beaches Council.

Manly Hydraulics Laboratory. (2018, December). NSW EXTREME OCEAN WATER LEVELS.

Manly Hydraulics Laboratory. (2023, April). NSW TIDAL PLANES ANALYSIS 2001-2020 HARMONIC ANALYSIS. *Report MHL2786*. NSW Department of Planning and Environment Biodiversity and Conservation Division.

Mariani, A., Blacka, M., Cox, R., Coghlan, I., & Carley, J. (2009, January). Wave Overtopping of Coastal Structures. *Physical Model Versus Desktop Predictions, SI 56* , p.534-538. (J. o. Research, Ed.)

Mase, H. (1989, September). Random Wave Runup Height on Gentle Slope. *Journal of Waterway, Port, Coastal, and Ocean Engineering, Volume 115(Issue 5)*. ASCE.

Mortlock TR, G. I. (2017). The June 2016 Australian East Coast Low: Importance of Wave Direction for Coastal Erosion Assessment. *Water*, 9(2), 121.

Nielsen, A., & Lord, D. a. (1992, June). Dune Stability Considerations for Building Foundations”. *Australian Civil Engineering Transactions, Volume CE34, No. 2*, pp. 167-17. Institution of Engineers Australia.

NSW Goverment. (2010, August). Coastal Risk Management Guide. *Incorporating sea level rise benchmarks in coastal risk assessments*. Sydney, New South Wales, Australia: Department of Environment, Climate Change and Water NSW.

P. J. N. Pells, G. M. (2019, June). *Classification of sandstones and shales in the Sydney region: A forty year review, Colume 54, Number 2*. Australian Geomechanics Journal.

Shand, T. D., Wasko, C. D., Westra, S., Smith, G. P., Carley, J. T., & Peirson, W. L. (2012, March). Joint Probability Assessment of NSW Extreme Waves and Water Levels. *Technical Report 2011/29*. (U. W. University, Ed.) Manly Vale, Australia: NSW Office of Environment and Heritage.

Shand, T., Cox, R., Mole, M., Carley, J., & Peirson, W. (2011). Coastal Storm Data Analysis: Provision of Extreme Wave Data for Adaptation Planning. *WRL Research Report 242*. Perth, Australia: Proceedings of the 20th Australasian Coasts and Ports Conference.

Shand, T., Wasko, C., Goowin, I., Carley, J., You, Z., Kulmar, M., & Cox, R. (2011). Long Term Trends in NSW Coastal Wave Climate and Derivation of Extreme Design Storms. *NSW coastal conference , Volume 2*, 8-11.

Standards Australia. (2005, September 28). AS 4997—2005. *Guidelines for the design of maritime*. Australian Standard.

U.S. Army Corps of Engineers. (2006). Coastal Engineering Manual (CEM). *Engineer Manual 1110-2-1100, 6 volumes*. Washington, D.C.: U.S. Army Corps of Engineers.

Vos, K., Deng, W., Dean Harley, M., Turner, I. L., & Splinter, K. D. (2022). Beach-face slope dataset for Australia. *Earth Syst. Sci. Data, 14, 1345–1357, Volume 14(issue 3)*. doi:https://doi.org/10.5194/essd-14-1345-2022

Watson, P. J., & Lord, D. B. (2008, October). Fort Denison Sea Level Rise Vulnerability Study. *Report prepared by the Coastal Unit*. Sydney: DECC.

Waverley Council. (2024, 02 27). Waverley Council and Bronte Surf Lifesaving BSLSC Facilities. *Building Operation Management Plan(Version E)*. Waverley Council.

WorleyParsons. (2011). Coastal Risks and Hazards Vulnerability Study. *Ref: 301015-02526-CS-REP-0001*. Waverley Council.

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Appendices

A1 Site visit

Area	Photo	Comments
A	 	Bronte Gully is characterised by a narrow, grassy valley floor with steep side slopes densely covered with trees. The management of stormwater in this area involves an engineered natural creek running through the centre of the valley, culminating in a waterfall at the head of the Bronte gully.

B		The central park is a gently sloping, low-lying area situated immediately west of the beach. It is adorned with grass and encompasses existing site developments, including the SLSC and community amenities, as well as various pathways. The area also has several buried services, including a sizable stormwater culvert. This culvert runs to the south of the SLSC building, then changes direction to the north-east, ultimately leading to an ocean outlet beneath the northern headland.
C.1		The Bronte Beach Seawall and promenade were built from 1914 to 1917, likely using reinforced concrete counterfort wall construction. This seawall extends the full length of the beach between the headlands.

C.2		View of the beach looking north.
D.1		The existing Bronte SLSC building is a concrete-masonry structure. The beachfront area of the club features concrete paving, covering community facilities like a cafe and pergolas. The remainder of the project site includes grassy areas, mature trees, pedestrian pathways, and additional community facilities such as pergolas and barbecues located in the central park. Notably, the SLSC appears to be in good condition, with no apparent signs of ground settlement.

D.2		Topographic data indicates variations in the crest level of the Bronte seawall. The crest level is approximately +3.9 m AHD in the south, rising to +4.8 m AHD in the centre of the beach, particularly in front of the amenity block. Just north of the amenity block, the seawall returns to the cliff face and concludes. In the northern section of the beach, where the cliff face is present, there is no seawall.
E.1		The northern concrete ramp is in good condition.

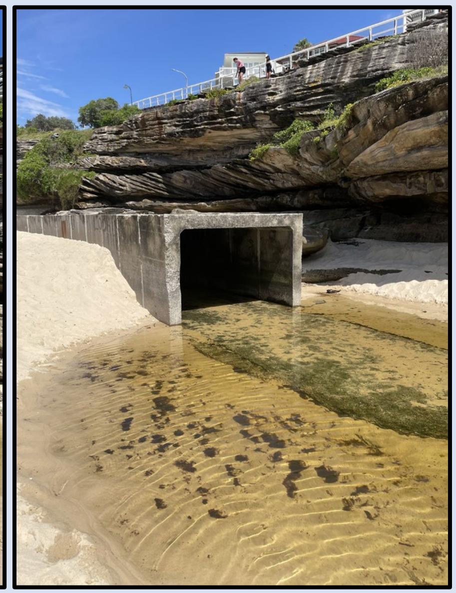
E.2	 	A void has opened, and the concrete wall has displaced approximately 60mm from the pavement.
E.3		Large vertical crack (>0.5 mm) in the concrete seawall. It is also possible to see repair methods delaminating.

E.4	 	Northern concrete stairs are in good condition.
E.5		Slab in good condition, no evidence of cracking, joints at 3.1m.

E.6		Brick wall in good condition.
E.7		Localised corrosion spots, delamination due to weathering, and horizontal cracking (>0.5mm) in the concrete seawall. Landwards brick retaining wall between the pavement and grassed area seems to be in good condition.

		Weepholes along the seawall are likely to relieve the water (hydrostatic pressure) that allows drainage from behind the seawall.
E.8		South concrete stairs in good condition.

		Bronte Beach signage.
E.9		The south headland is an elevated area with an exposed sandstone cliff near the ocean edge. The stainless-steel handrail along the southern portion of the seawall and access stairs is in good overall condition. However, there are minor instances of surface corrosion in certain localised areas.

F.1		The north headland is an elevated region characterised by an exposed sandstone cliff line along the ocean edge. To the west and south, it features a shoulder that consists of an open grassed hill.
F.2	 	The large storm water culvert is present under the concrete surface in the beach frontage area at the eastern side of the SLSC building and continues to the north of the site within the beach area at the base of the cliff. The channel appeared to be about 3.2m wide and 3m deep.

F.3	 	View of the stormwater outfall.
F.4	 	Localised rust staining and cracks at the exposed part of the storm outfall.

F.5	 	Sandstone outcrops are noticeable at several points on the northern side of the Bonte breach, and these include weathered sandstones. Erosion and cavity features are observable on the sandstone cliff. The surface soils consist of loose sands.
F.6	 	Different details of the transition between the existing seawall and stormwater outfall. It is visible that there are some horizontal and vertical surface cracking/voids in the concrete elements. Delamination/spalling of the concrete surface generally appears to be due to weathering.


MACQUARIE BANK
BONDI to BRONTE Ocean Swim
Sunday 3 December 2023
LIFEGUARD

A2 Preliminary statement from additional geotechnical investigation (JK Geotechnics, 2024)

Date: 20 February 2024

Ref: 36494PElet

Waverley Council
Level 1, 87 Oxford Street
Bondi Junction NSW 2022

Attention: Matthew Henderson
Email: matthew.henderson@waverley.nsw.gov.au

GEOTECHNICAL INVESTIGATION
PROPOSED SEAWALL UPGRADE
BRONTE SURF LIFE SAVING CLUB, BRONTE ROAD, BRONTE, NSW

JK Geotechnics recently carried out a geotechnical investigation to assist with the design of the proposed seawall upgrade at the Bronte Surf Life Saving Club, Bronte Road, Bronte, NSW. The investigation was commissioned by Joao Goncalves of Royal HaskoningDHV by email dated 6 December 2023. The commission was on the basis of our fee proposal, Ref. 'P59363PE Rev1' dated 9 November 2023.

The fieldwork was carried out between 13 & 15 February 2024 and comprised the following:

- Three boreholes (BH101 to BH103) drilled to depths of 3.7m (BH102) and 4.7m (BH101 & BH103) below existing beach levels using spiral augering techniques. All three boreholes were subsequently extended to their final depths of 10.9m (BH101), 9.8m (BH102) and 7.8m (BH103) using HQ diamond coring techniques with water flush. The boreholes were positioned along the proposed seawall alignment at the northern end of Bronte Beach.
- Five test pits (TP104 to TP108) excavated to depths of 1.0m (TP104), 0.9m (TP105), 2.9m (TP106), 2.3m (TP107) and 0.9m (TP108) below existing beach levels using a six tonne excavator (TP104 to TP107) or hand tools (TP108). The test pits were located beside the existing seawall and concrete culvert at the northern end of Bronte Beach.
- Five Dynamic Cone Penetration (DCP) tests (DCP104 to DCP108) completed adjacent to the corresponding test pits which extended to termination/refusal depths of approximately 3m (DCP104 & DCP105), 1.6m (DCP106), 1.1m (DCP107) and 2.8m (DCP108) below existing beach levels. The refusal depth of the DCP tests can provide an indicative depth to bedrock, though we note that refusal can also occur on obstruction in fill, 'floaters' and other hard layers, and not necessarily on bedrock.

In addition to the above, a seismic refraction survey will be carried out (by others) on 21 February 2024 across the northern end of the beach. The purpose of the survey is to assess the bedrock levels within the subject site using seismic energy generated on the surface using a sledgehammer.

The preparation of the factual information (e.g. borehole logs, test pit cross sectional sketches, DCP test results sheets, figures etc.) and laboratory analysis is currently underway. Based on our initial review of the field results, weathered sandstone bedrock was encountered in the boreholes at the depths tabulated below. We note that the inferred bedrock levels based on the DCP test results will be provided in due course after further review of the field results.

Borehole	Approximate Surface (i.e. Beach) Level (mAHD)	Approximate Depth to Weathered Sandstone Bedrock (m)	Approximate Reduced Level of Weathered Sandstone Bedrock (mAHD)
BH101	3.7	4.6	-0.9
BH102	3.6	3.6	0.0
BH103	3.8	4.3	-0.5

Following completion of the seismic refraction survey outlined above, our geotechnical report will be issued with the factual results of the investigation and our comments and recommendations on site preparation, pile design parameters and other pertinent geotechnical issues relevant to the proposed seawall upgrade.

Should you require any further information regarding the above, please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JK GEOTECHNICS

Michael Egan
Associate Geotechnical Engineer

A3 Coastal protection peer review presentation

Bronte Surf Life Saving Club

Coastal protection peer review presentation

PRELIMINARY

1.0 Agenda

- Process so far
- Overview
  - Site Considerations
  - Design Strategies
- Response to key issues
  - Facilities siting
  - Building movement
  - Promenade and Seawall
  - Accessibility
- Revised design
  - Plan and section
  - View analysis
  - Physical modelling

Process so far

DA Submission - October 2022

Amended DA - August 2023

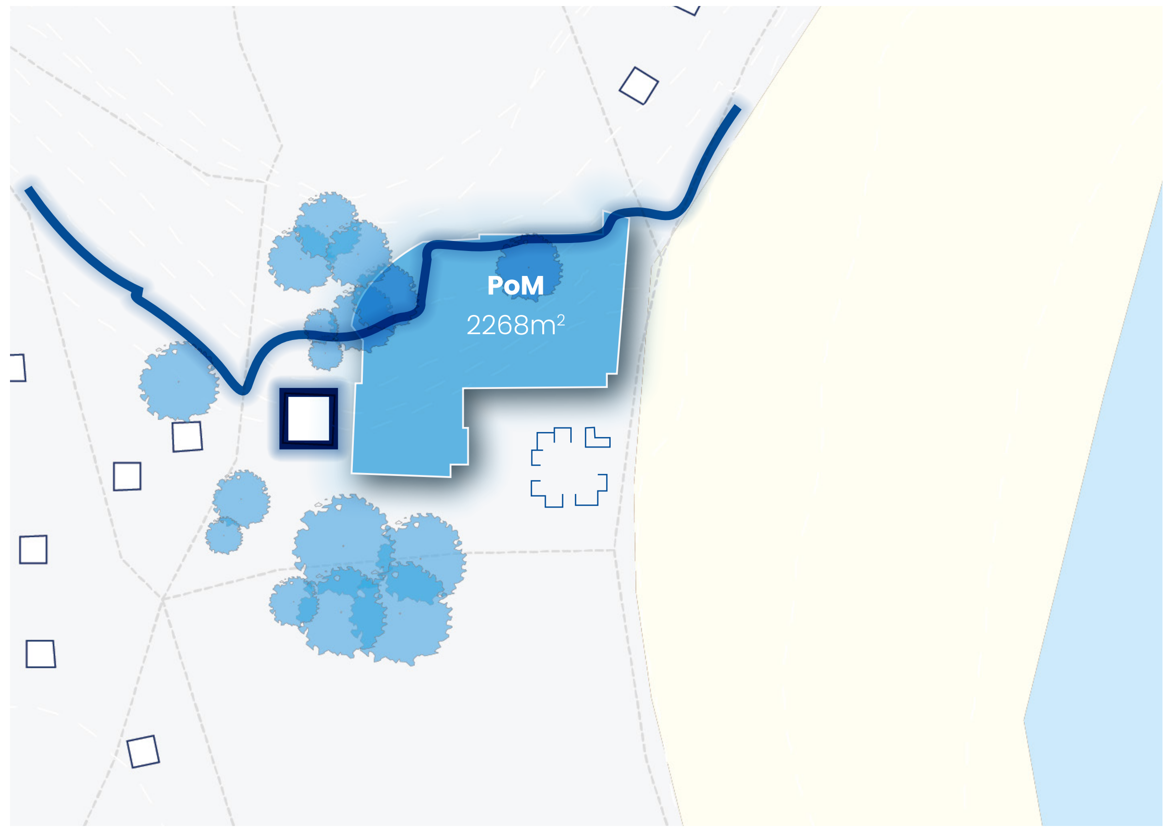
Sea wall update - October 2023

Key issues:

- Site location and sustainability (inundation & design life)
- Operation and noise
- Accessibility
- Excavation and heritage
- View impacts from residential areas

Overview

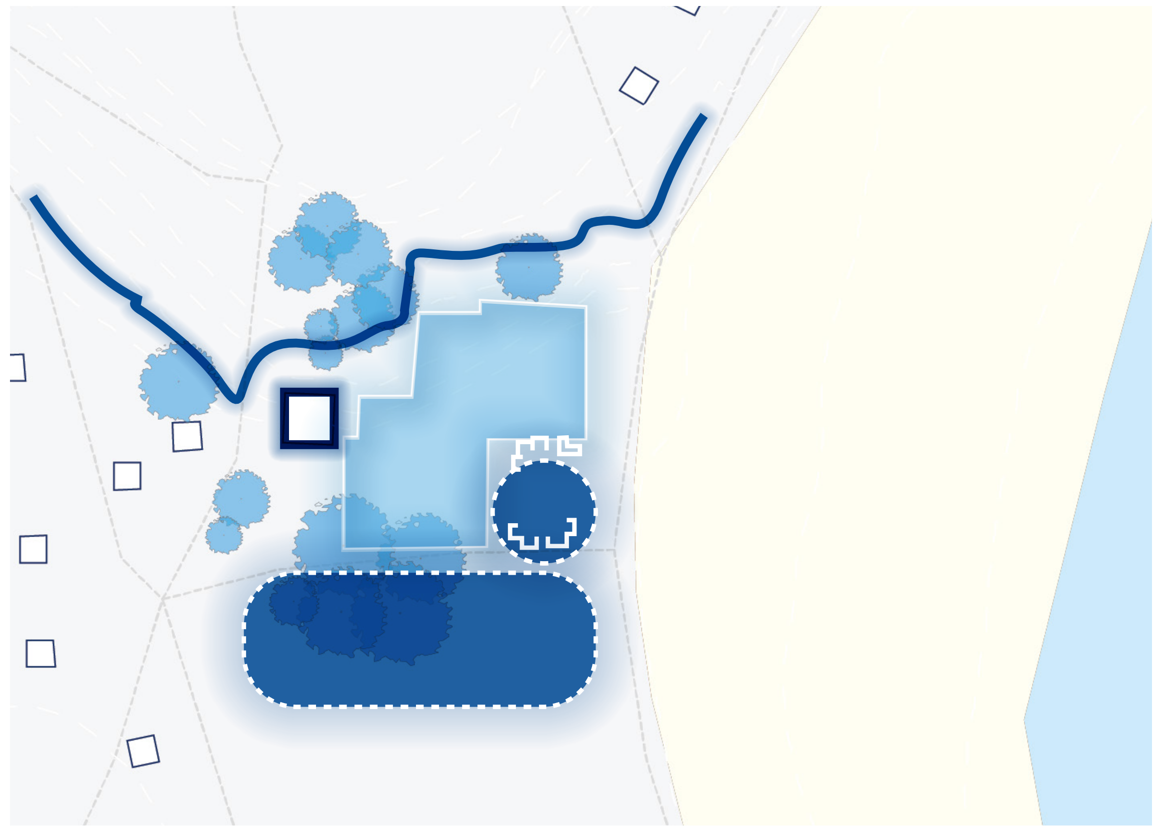
Site Considerations



The Plan of Management envelope is inconsistent with current operational needs and community expectations:

- Increases bulk on the water.
- Increases overshadowing on the park.
- Removes open space at beach level.
- Doesn't resolve the conflict of beach monitoring and SLSC operations with pedestrian movement along the promenade.

Reflection
- POM footprint has accommodated an appropriate area growth.
- Lifeguard, park and requirements have changed.
- Height and setback expectations have been clarified through specific community engagement.



Protect quantity and amenity of open space:

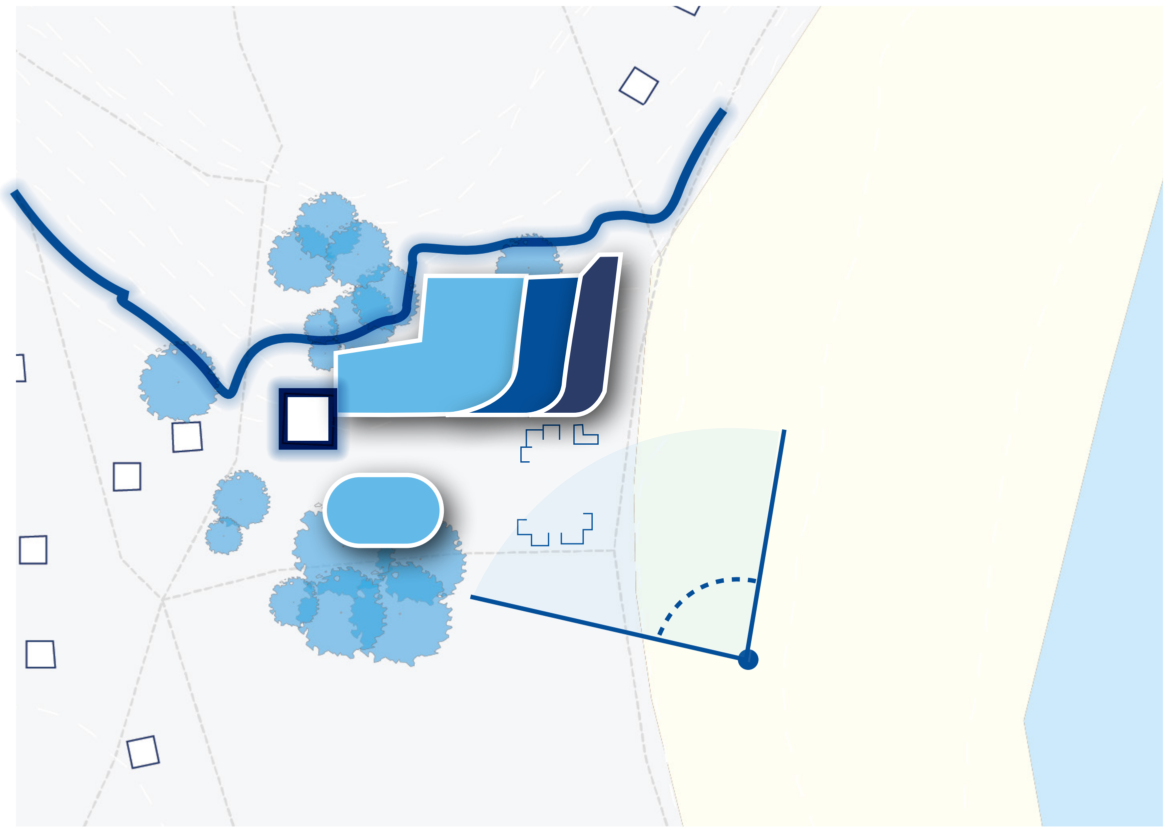
- The plan of management enables expansion into public open space that is of high value to the community.

Reflection
- The community has clearly indicated through the design development and engagement process that a high priority is placed on:
  - no loss of usable green space
  - no more overshadowing of green space to the south

Protect sight lines and views between park and beach:

- The plan of management envelope enables reduced visibility between the two most used areas of the beach highlighted by community and technical reviews.

Reflection
- Level 1 to stay behind the heritage pump house alignment.
- Ground floor to go no further than the existing alignment at the south.
- Integrate the heritage pump-house through building alignments and creating a layered collection of forms.



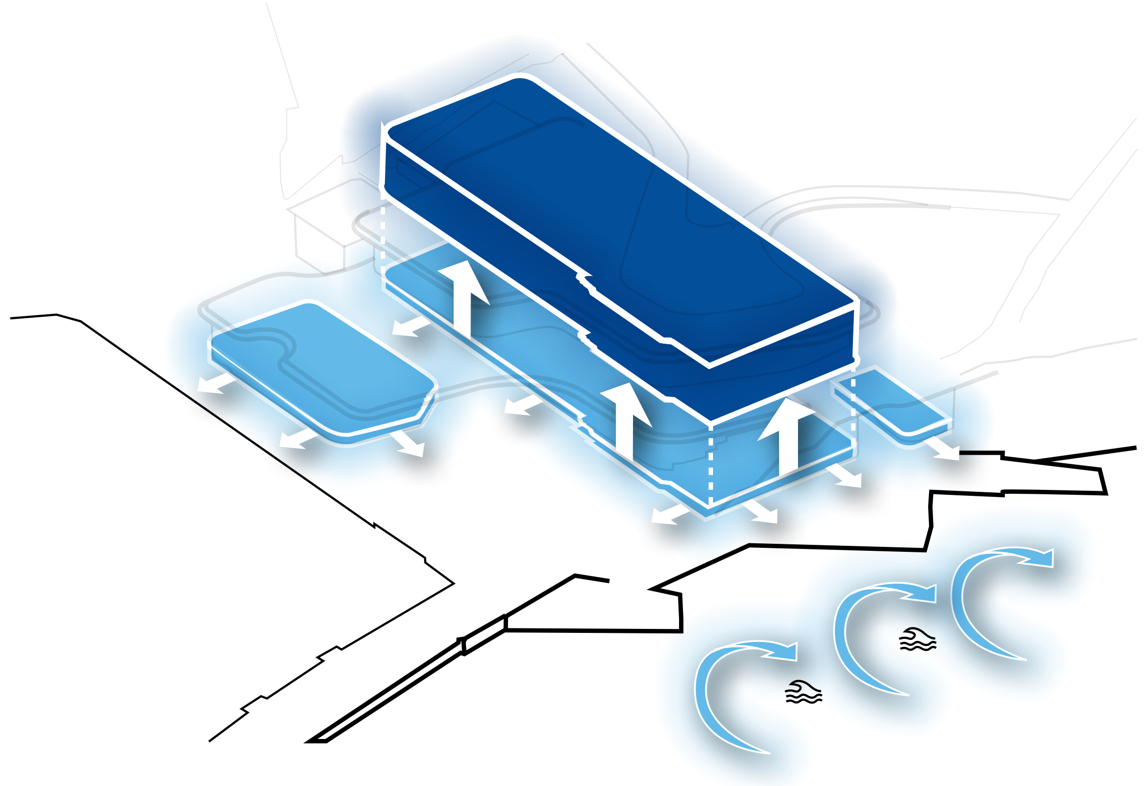
Minimise bulk and scale

- The plan enables a building that closer to the water at upper levels increasing bulk and scale.

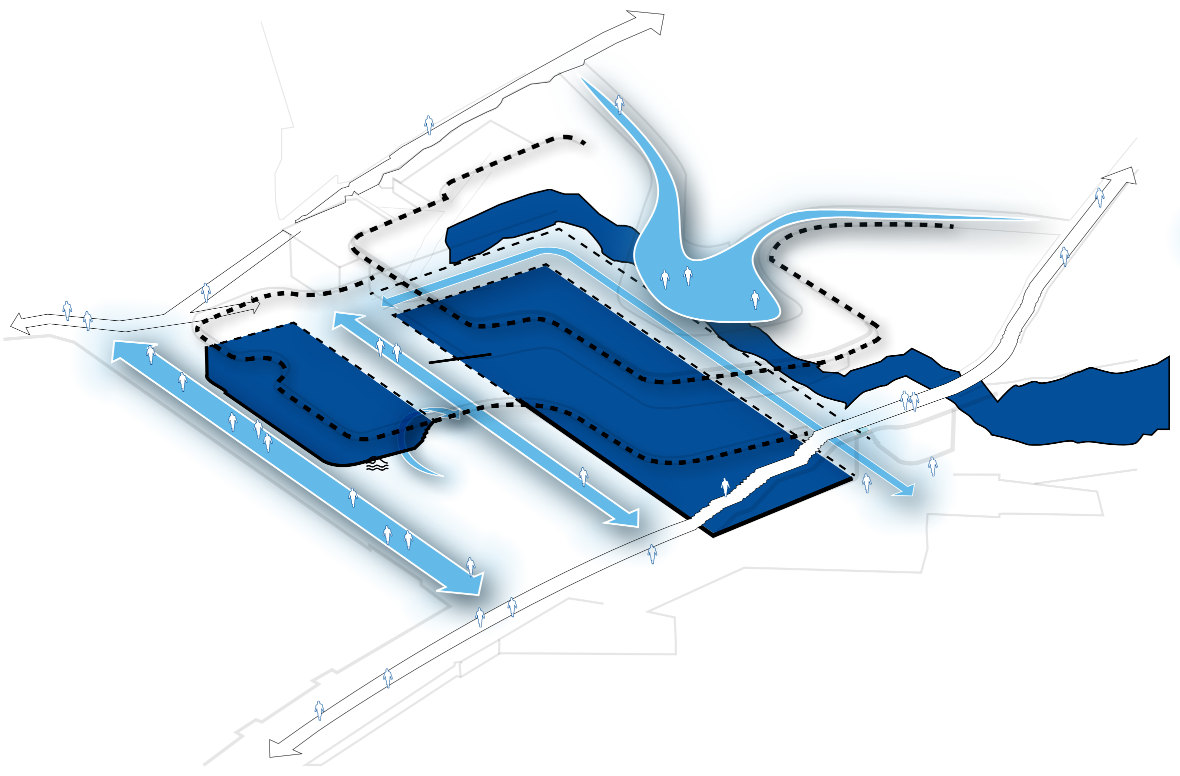
Reflection
- Layered increase in height away from the beach and southern park to reduce the dominance of built form from the south.
- Nestle the building into the headland.

Architecture Strategies

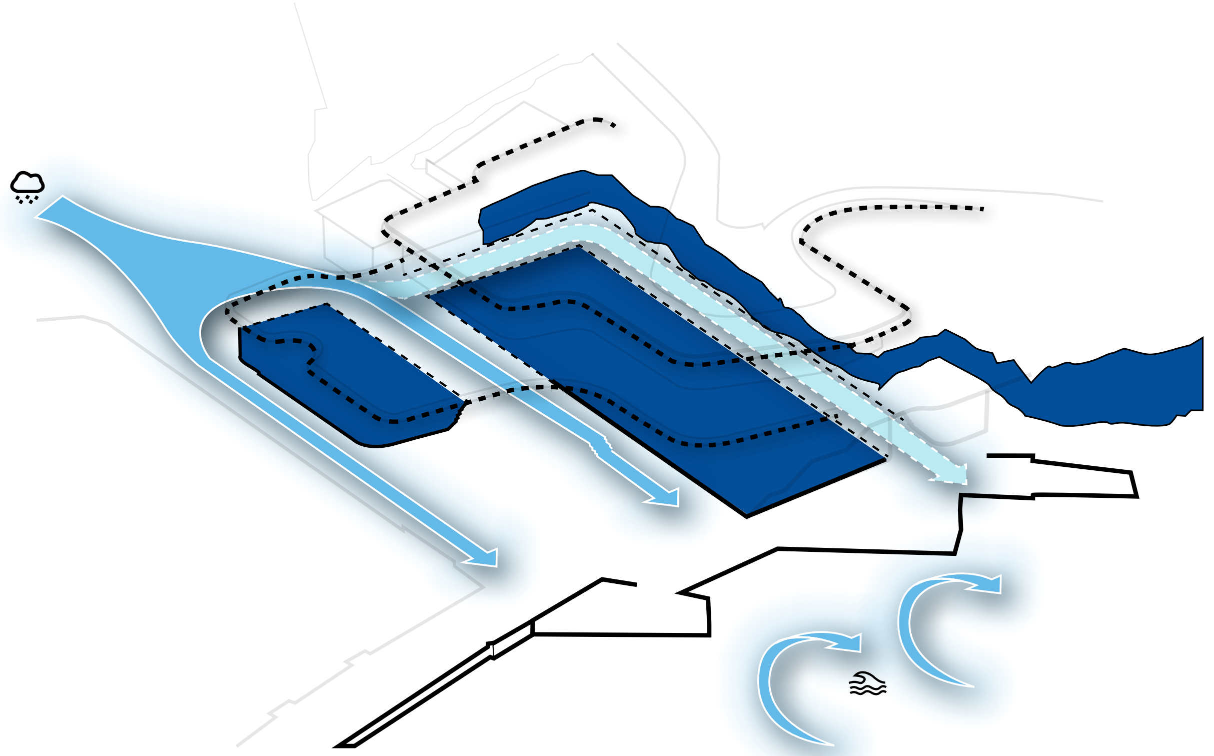
An Appropriate Scale



Operational Resiliance



Prioritse Circulation and Experience



Working with Water

Response to key issues

Key Issues

SCEPP Meeting minutes

Key issues discussed

- Applicant has engaged new coastal engineer with peer reviewer to be engaged to overview coastal works and include a detailed hazard assessment and performance of works (noting coastal inundation and erosion)
- Encroachment of sea walls and other walls onto the beach and adequacy of protection works
- Proposal to include works immediated in front of the building, further approval required for other works along the beach
- Clarification of proposed sand level in relation to wave action and impact on proposed wall

Key Considerations

- Promenade and sea wall to enable a separation between operations and public and wave action protection
- Existing sea walls noted as not able to match the design life of the proposed structure and need for proposed works for protect new building
- Access consolidation to be considered and alignment of ramps

Panel Comments

- Need to address coastal risk management
- Discussion required between the applicant and Council to address coastal risk management issues
- Consideration of total seawall importance to the proposed structure
- Agree that building location is appropriate

PRELIMINARY

Operational Needs and Facilities Siting

	PARK MGMT	PUBLIC FACILITIES	LIFEGUARD	SLSC
PARK	Maintenance of Picnic Area Playground Waste Water Infrastructure Natural Habitats	Change Rooms Toilets Food & Beverages/Kiosk BBQ's		
BEACH	Beach Grooming Community Events Waste	Toilets Food & Beverages/Kiosk	Patrol Patrol First Aid Area	Patrol Water Safety Nippers & Dippers Community Water Events First Aid

Operational Needs and Facilities Siting

LIFEGUARD

Bronte is recognised as one of the two most treacherous patrolled beaches in NSW.

The Coroner's Inquest into the death of Matthew Thomas Ritchell in 2014 documents the conditions of Bronte quickly turning from benign to treacherous.

As a result of this tragic event, Waverley Council provide a Lifeguard presence 365 days a year.

The sand and water conditions restrict the ability for the lifeguards to use buggies or other vehicles in aiding people in need.

This drives a need to provide facilities central to the beach to enable rapid response by foot to any area of the beach and a reliance on boards as the primary water-craft that can be safely used at Bronte.

PRELIMINARY

Operational Needs and Facilities Siting

SLSC

The Bronte Surf Life Saving Club supports the work of Council and the lifeguard patrol through a combination of educational programs as well as beach surveillance.

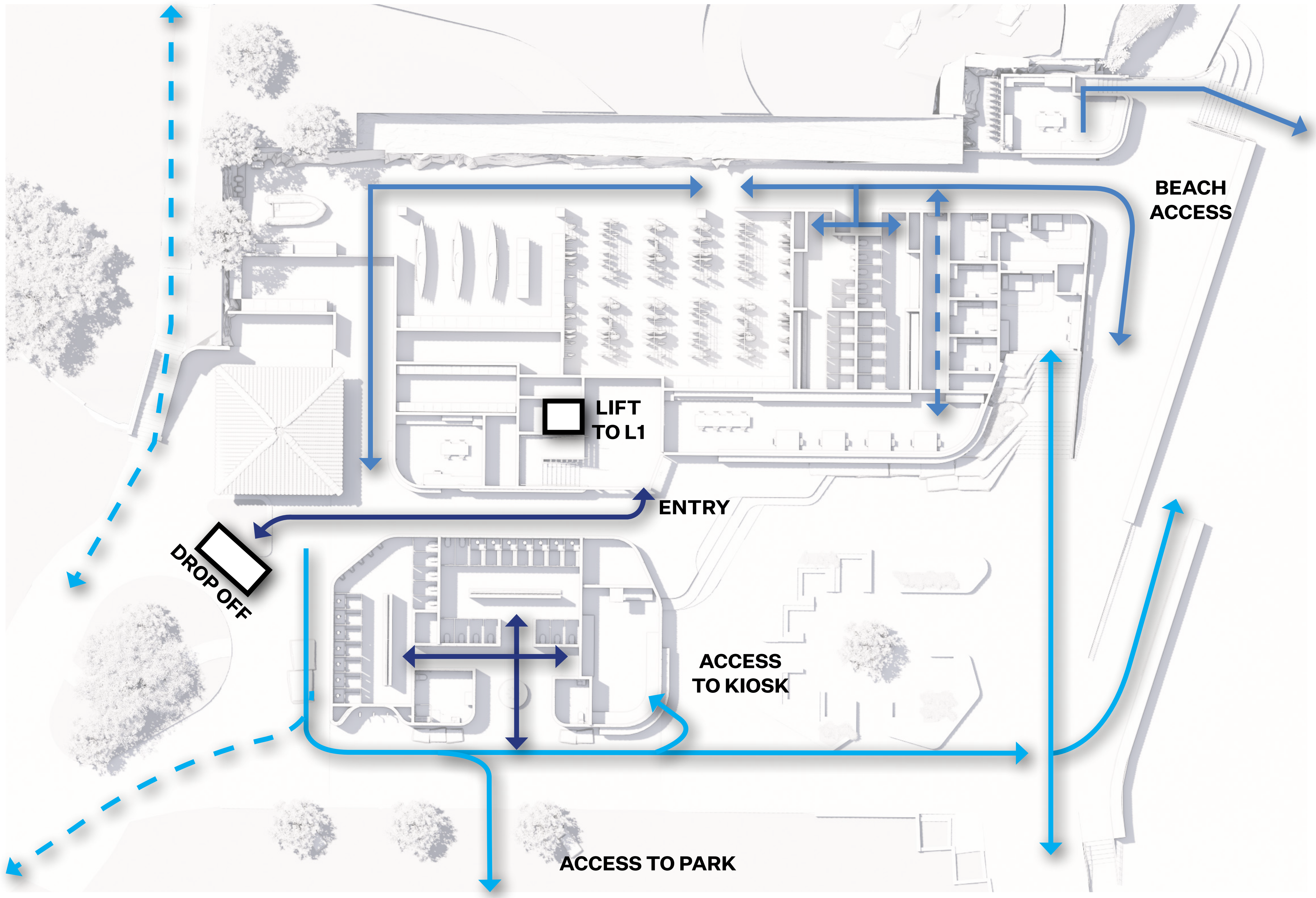
As a club it has the the most non-competition water training (boards and swimming) in comparison to other local clubs in direct response to the treacherous conditions providing a significant reliance on boards and board storage in adequately training volunteer surf life savers and providing surf life saving services to people in need.

It provides beach surveilliance every weekend and public holiday from September to April, 9am to 6pm.

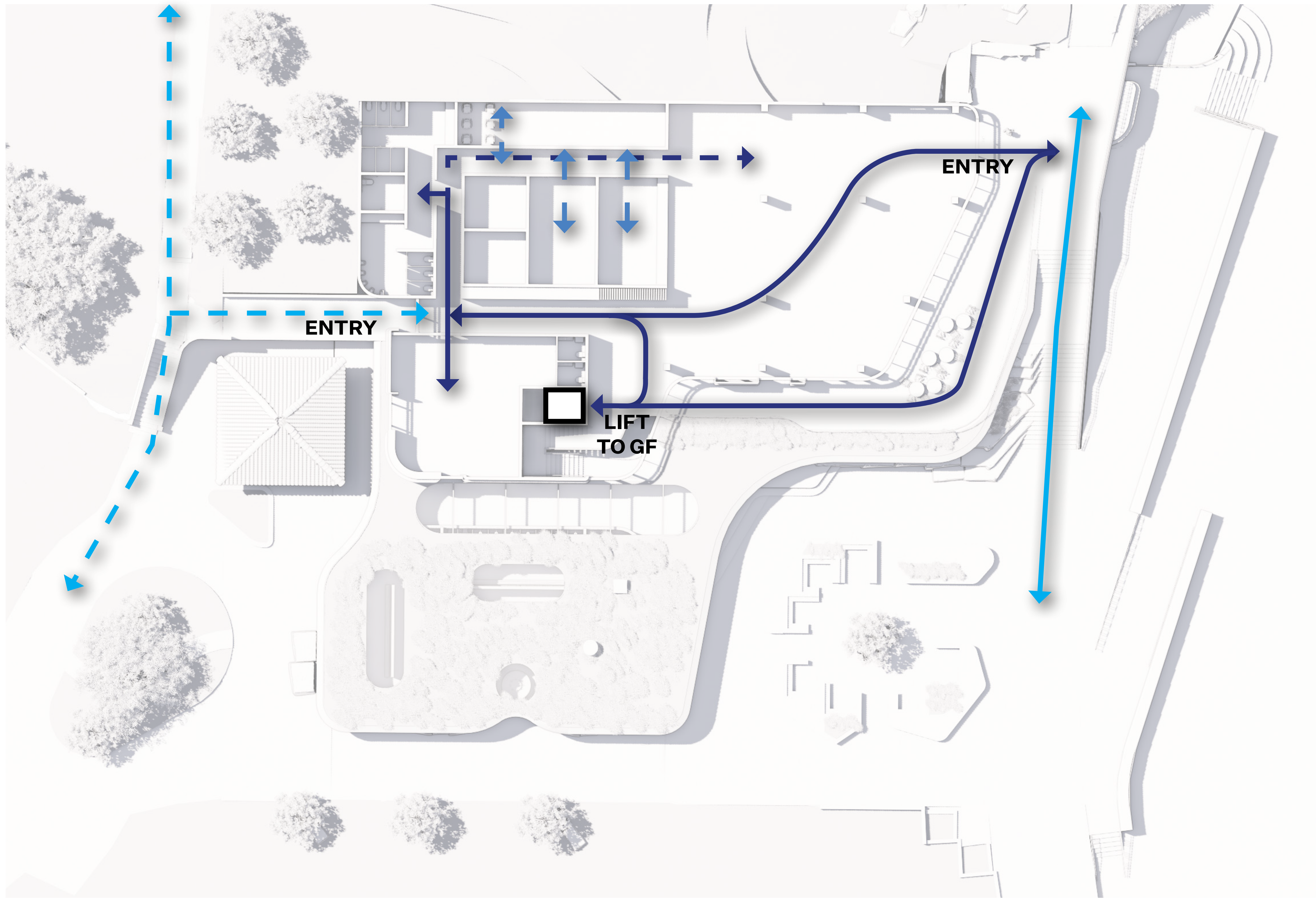
Building Movement

Key

- Public Movement
- Operation Movement
- User Movement
- Primary
- Secondary

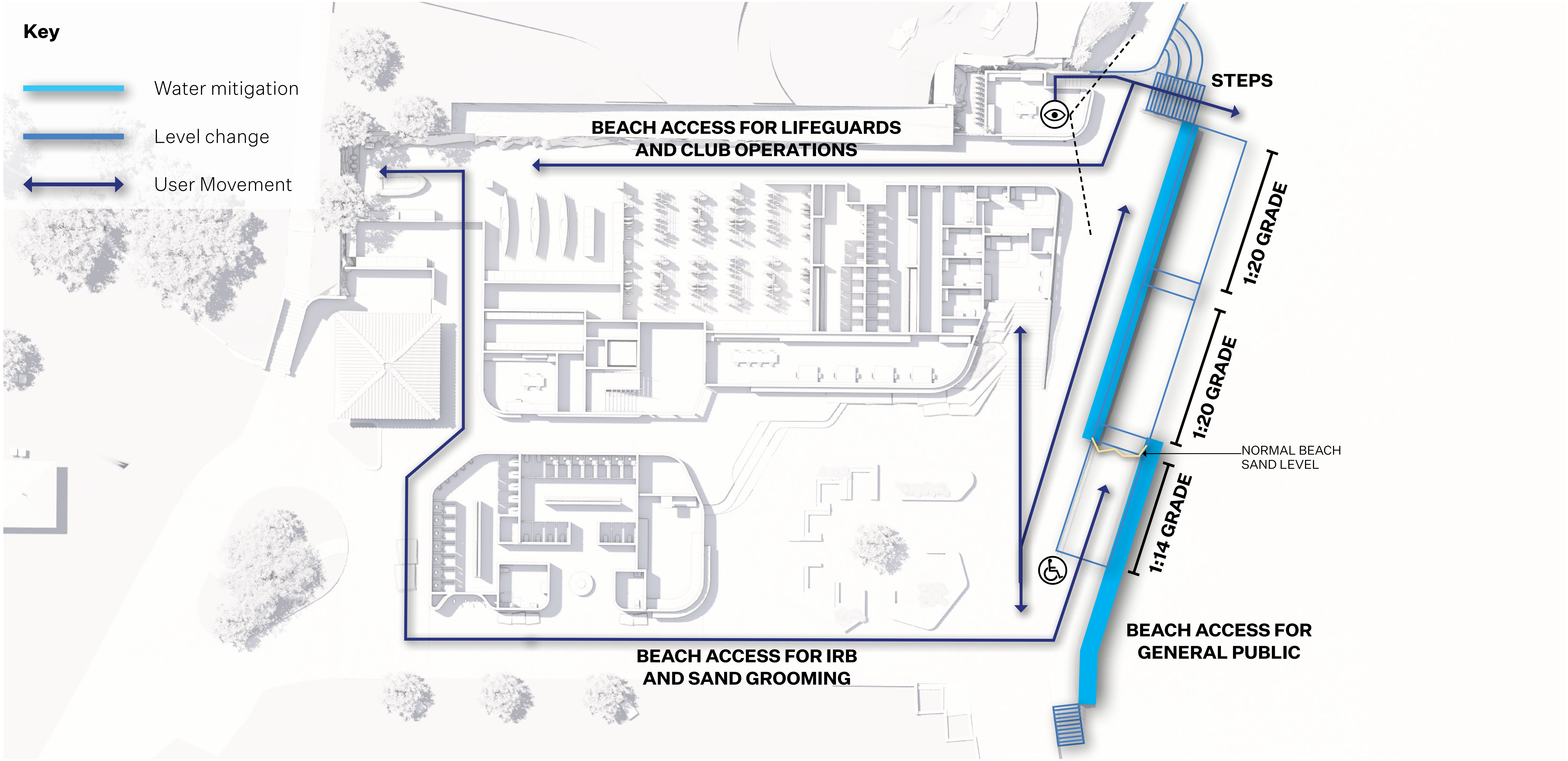


Ground



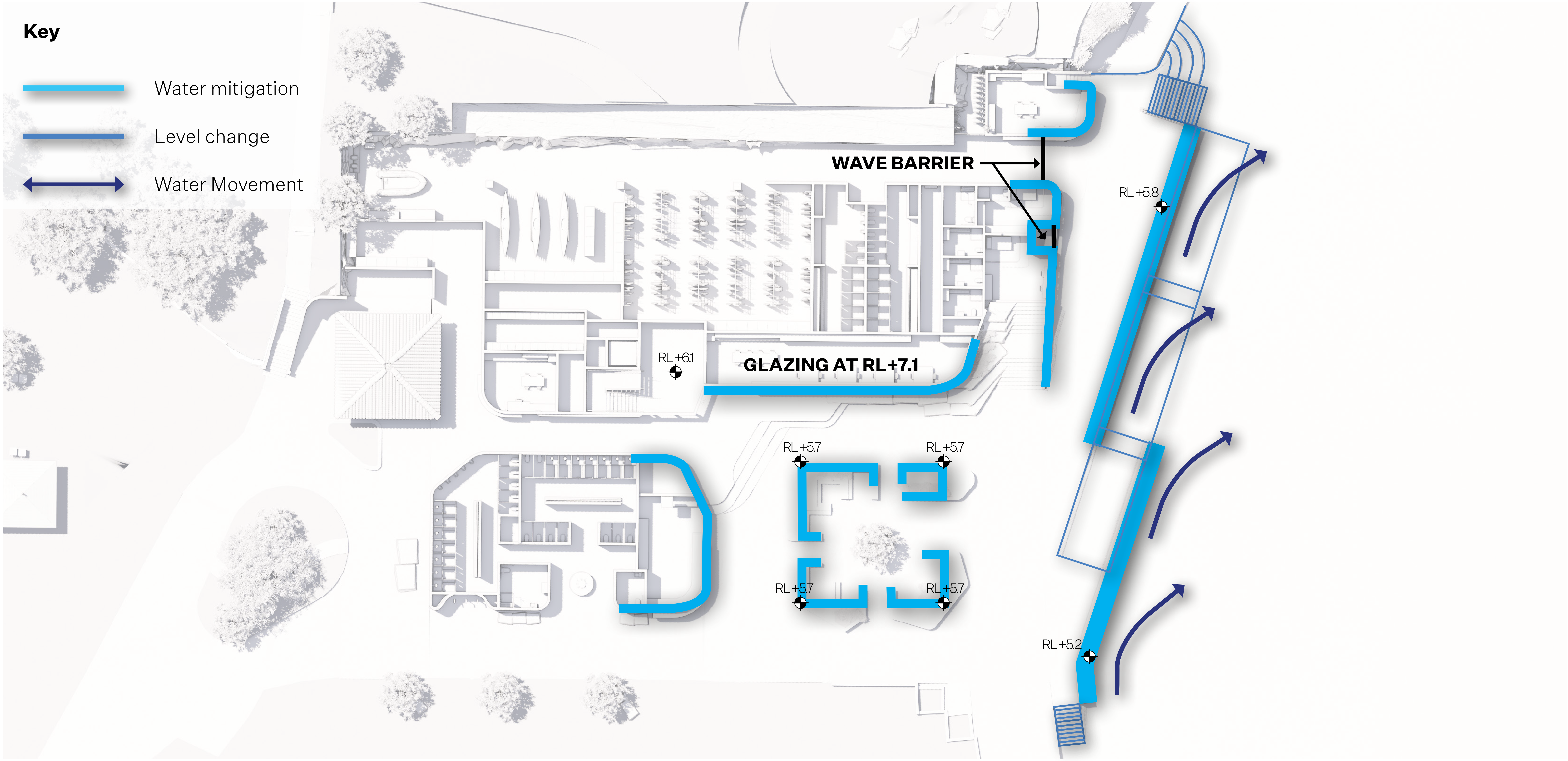
Level 01

Promenade and Seawall



PRELIMINARY

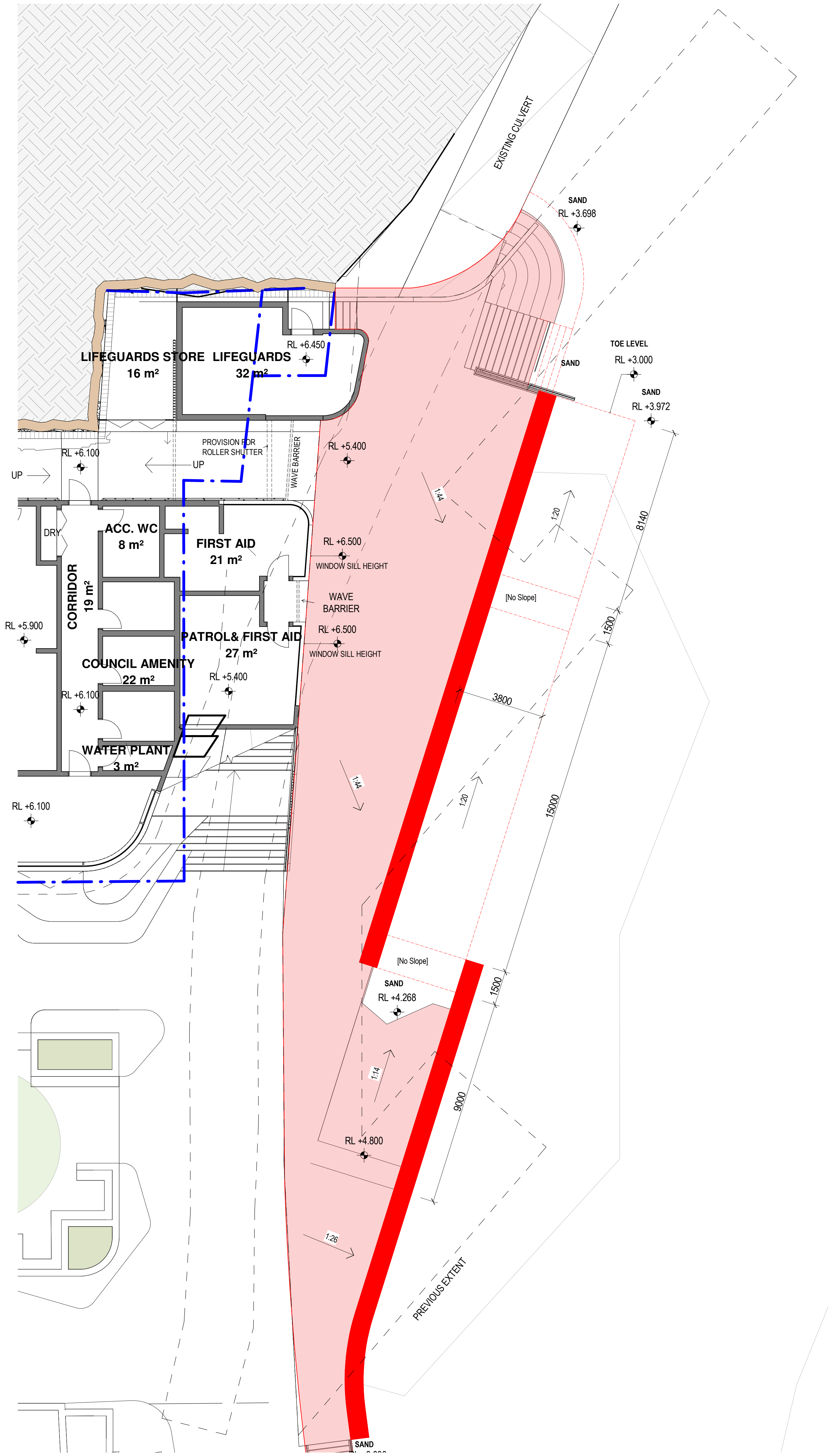
Promenade and Seawall




PRELIMINARY

Revised Design

Ground Plan

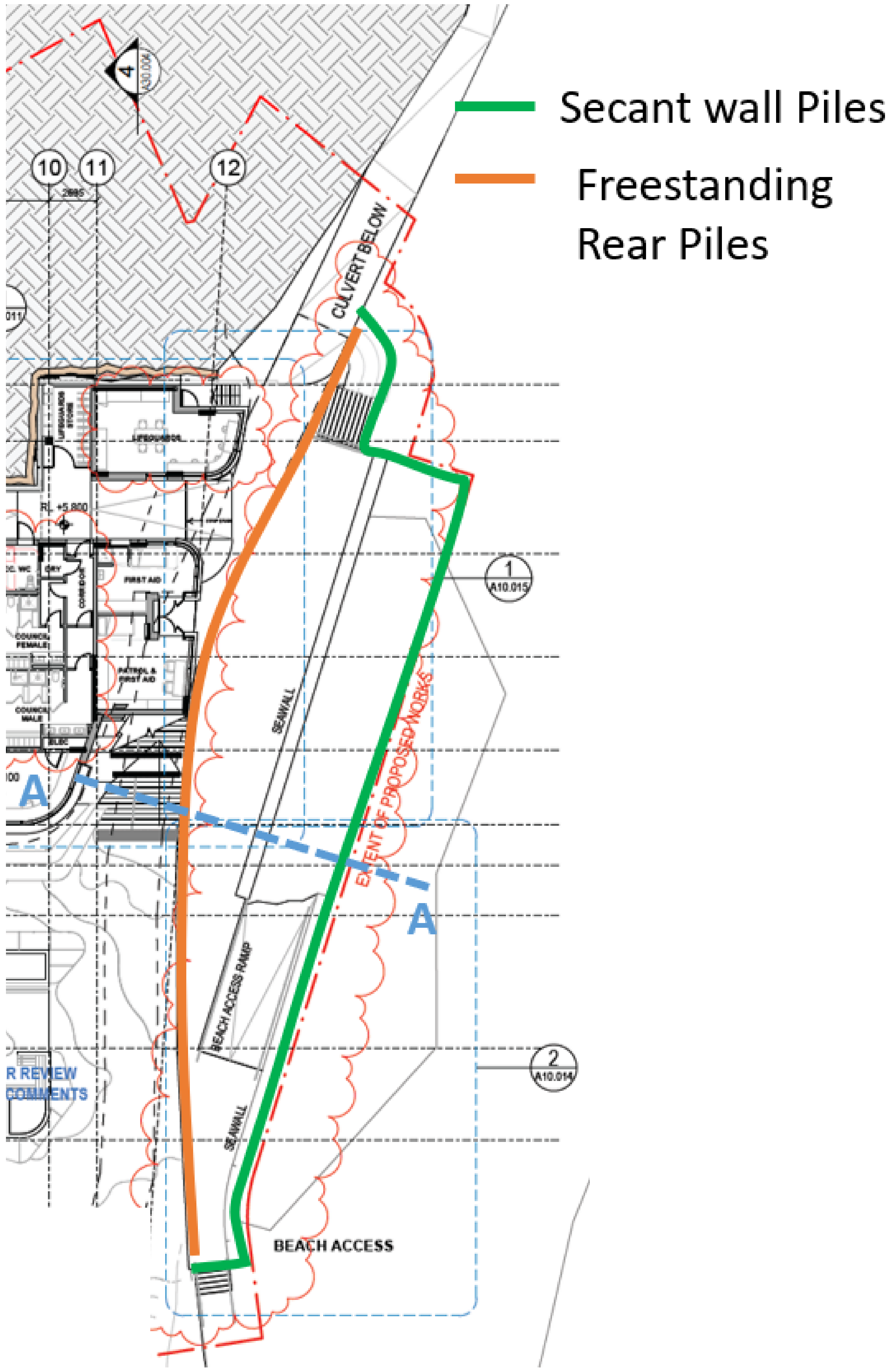

LIFEGUARDS STORE
16 m²
LIFEGUARDS
32 m²
RL +6.450
PROVISION FOR ROLLER SHUTTER
WAVE BARRIER
RL +6.100
UP
ACC. WC
8 m²
FIRST AID
21 m²
CORRIDOR
19 m²
PATROL& FIRST AID
27 m²
COUNCIL AMENITY
22 m²
WATER PLANT
3 m²
RL +5.900
RL +5.400
RL +6.500
WINDOW SILL HEIGHT
WAVE
BARRIER
EXISTING CULVERT
SAND
RL +3.698
TOE LEVEL
RL +3.000
RL +3.972
[No Slope]
RL +4.268
RL +4.800
3800
15000
8140
1500
9000
PREVIOUS EXTENT

Structural System (pre-concept)

RHDHV would consider feasible the option of combining secant pile wall elements and freestanding piles supporting the seawall spurs, ramps and steps and fully protecting the landward shoreline. This avoids the need to rely on the existing seawall to protect the SLSC.

This option therefore does two things – it supports the on-beach structural elements and acts as a seawall. The secant pile wall is formed by the installation of overlapping reinforced concrete (hard) and unreinforced concrete (soft) piles to form a continuous vertical wall along the shoreline topped with a reinforced concrete capping beam.

If the bedrock is elevated, then the piles may not be needed and a beam on rock solution should suffice. A suspended slab would infill the space between the capping beam and the seawall, and the ramps and steps would be integrated into the slab.



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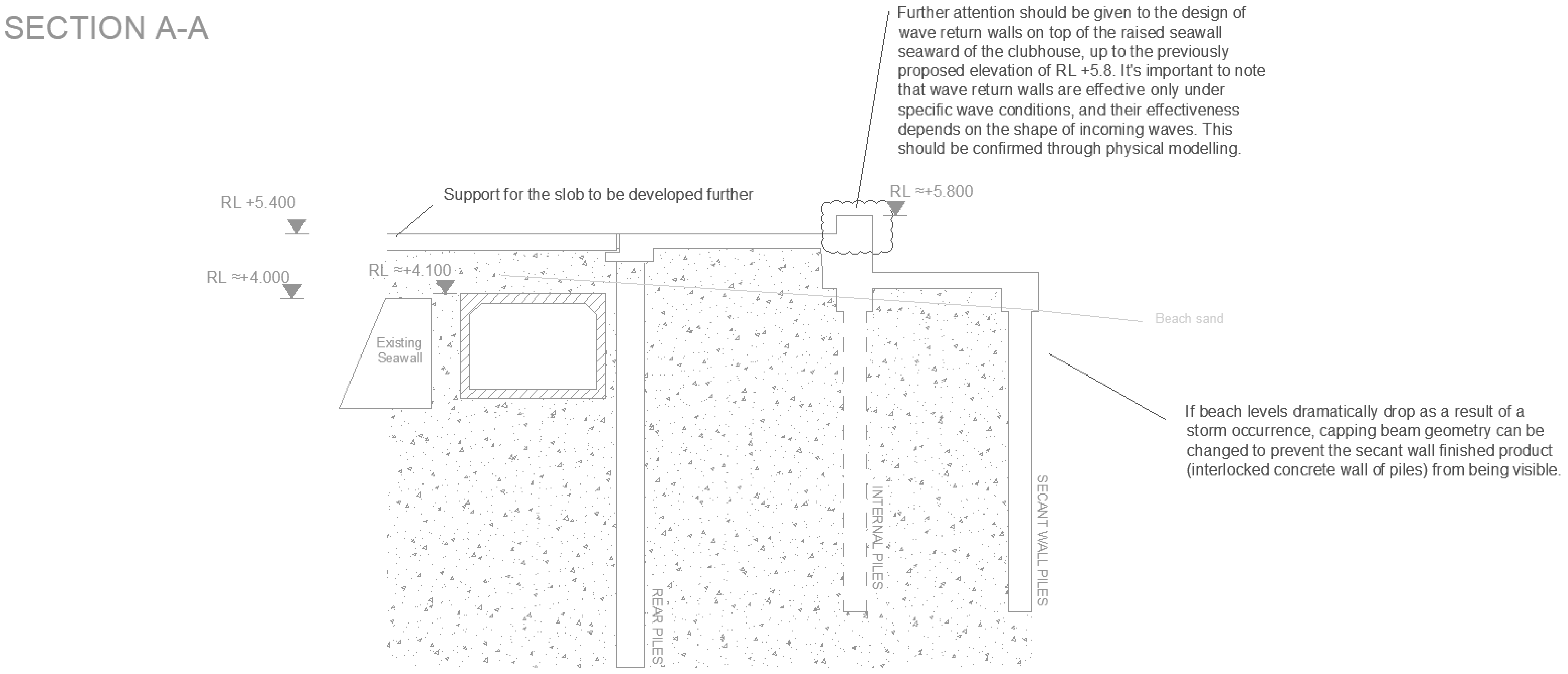


View Analysis

SE aerial view - Previous design

View Analysis

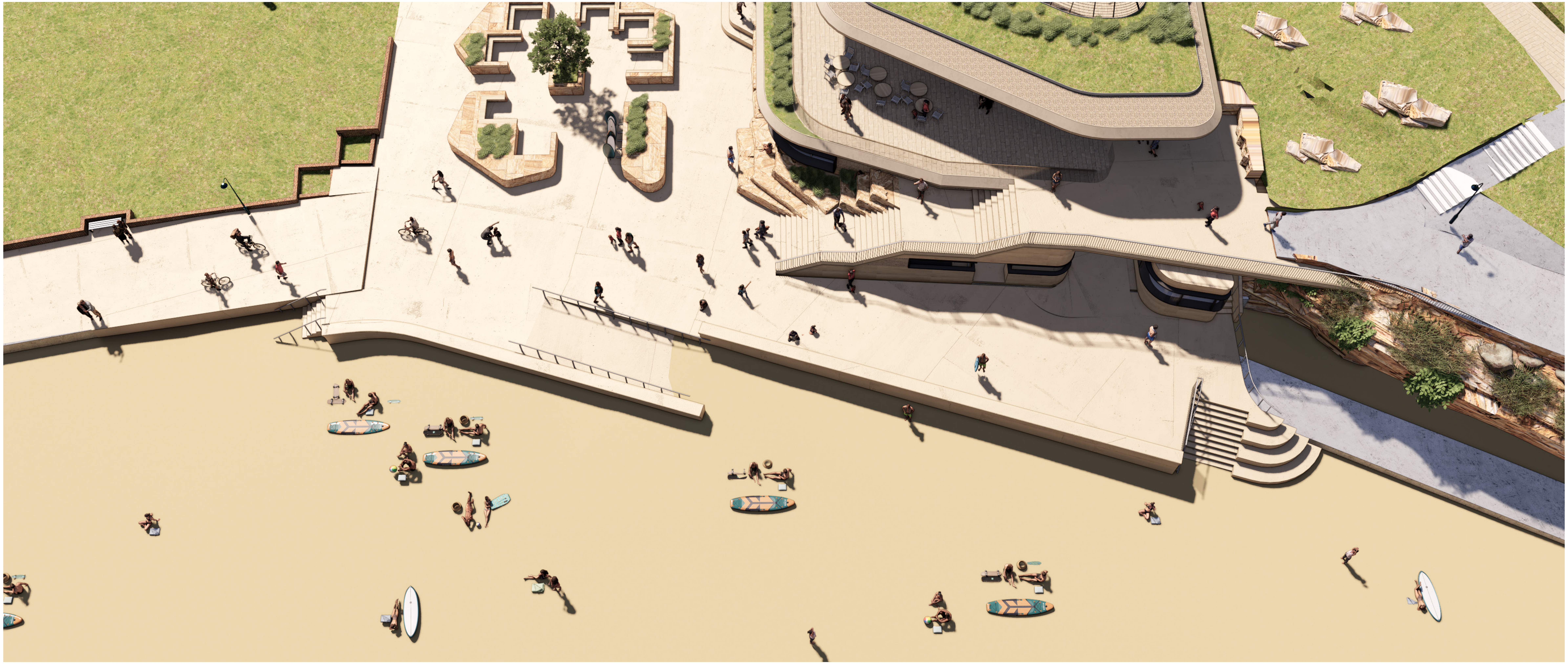
SE aerial view - New design

View Analysis

Eastern aerial view - Previous design

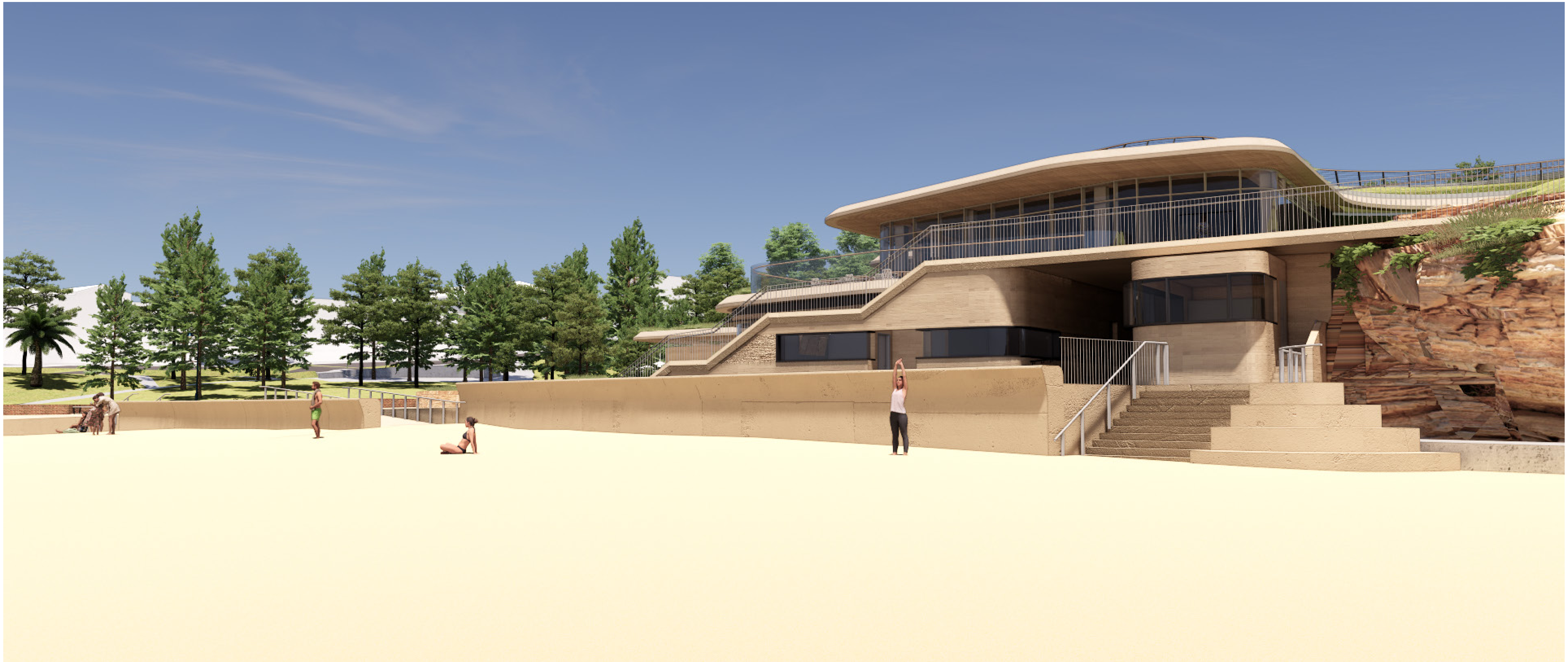
View Analysis

Eastern aerial view - New design



View Analysis

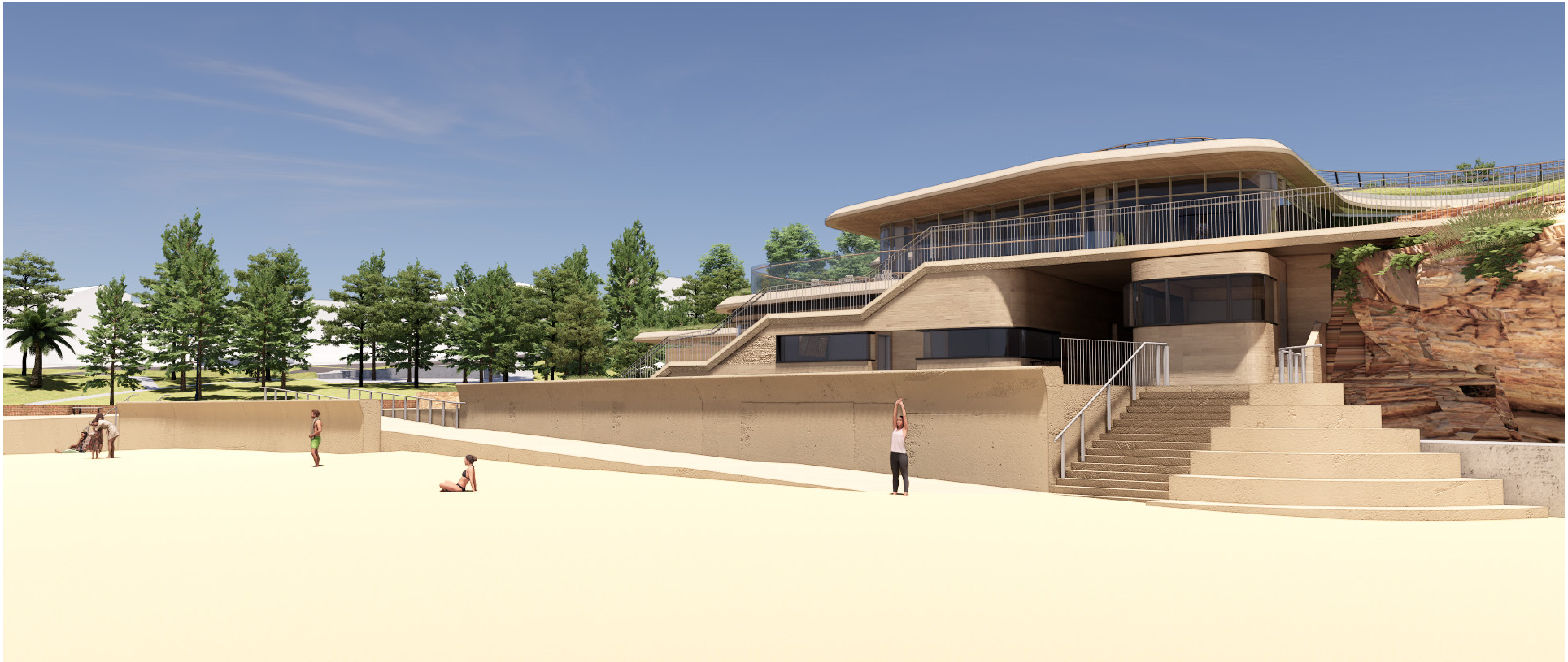
Current sand levels



PRELIMINARY

View Analysis

Reduced sand levels



View Analysis

Current sand levels

View Analysis

Reduced sand levels

Concept design development

Concept design development and linkage to physical modelling (if required)

- Functional Arrangement: The layout for the redeveloped SLSC, promenade, and beach access has been carefully planned in consultation with users and the Council.
- Coastal Hazards: The SLSC is exposed to coastal hazards, primarily erosion and coastal inundation. A seawall is required to protect the facility.
- Seawall Replacement: The existing seawall at Bronte is no longer reliable and must be replaced to safeguard the new SLSC.
- Seawall Structural Concept: The structural concept for the new seawall includes secant piles, concrete slabs, and deflector elements. These are designed to meet wave runup and overtopping requirements.
- Design Concept and Peer Review: The concept design has been the outcome of lengthy consultation and would be subject to peer review and agreement in principle with the peer reviewer, including consideration of matters under the CM Act and SEPP. As such, there would not be expected to be any fundamental changes to the design concept as a consequence of the physical modelling but rather only refinement of the engineering details, eg wave return wall geometry and wave loading.
- Wave Loading for New SLSC: The SLSC building is a new build hence wave loading determined from the physical modelling can be readily taken into account in the structural design, as opposed to any uncertainty whether an existing structure could be feasibly retrofitted/strengthened.

PRELIMINARY

Thank you

WARREN AND MAHONEY®

A4 Bronte SLSC redevelopment – Peer Review

18 December 2023

WRL Ref: WRL2023082 LR20231218 JTC

Jo Zancanaro
Acting Manager, Development Assessment
Waverley Council
Corner Paul Street and Bondi Road
Bondi Junction NSW 2022

By email: jo.zancanaro@waverley.nsw.gov.au

UNSW
Water Research
Laboratory

Dear Jo,

Re: Bronte SLSC redevelopment – peer review

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney is pleased to provide this letter report to Waverley Council (hereafter "Council") for Bronte SLSC – peer review.

The review was undertaken by WRL's Principal Coastal Engineer, James Carley.

The following document was reviewed:

- Royal HaskoningDHV and Warren and Mahoney (RHDHV, 2023), "Bronte SLSC Redevelopment: Seawall and Related Elements Detailed Design: Concept Design and Coastal Engineering Assessment Report", Reference: PA3572-RHDV-RP-S1-RP-FC-0001, Draft/P00.01, 5 December 2023

The RHDV (2023) report is in response to a WRL review dated 23 October 2023 (ref: WRL2023082 LR20231023 JTC) of three previous documents associated with this project, and a meeting between WRL, RHDV, Warren and Mahoney, and Council on Monday, 13 November 2023.

2. Summary of peer review

The document, "Bronte SLSC Redevelopment: Seawall and Related Elements Detailed Design: Concept Design and Coastal Engineering Assessment Report" (RHDHV, 2023) was reviewed by WRL's Principal Coastal Engineer, James Carley. The document is of a good professional standard, particularly in light of the short time available to prepare it.

The proposed upgraded seawall and likely reinforced concrete construction of the proposed new SLSC building is likely to better serve the function of surf life saving at Bronte.

The works proposed are likely to be able to manage coastal hazards for appropriate and foreseeable design events and sea level rise over the next 50 to 70 years subject to additional engineering design. The predominant hazard to be managed will be coastal inundation and wave forces through wave overtopping. For the existing and proposed new SLSC building, the hazards of erosion and recession are/will be managed through the presence of a seawall, provided that the seawall does not fail.

RHDHV (2023) canvassed available existing studies regarding numerous inputs for the project (e.g. extreme water levels), however, many sections of the report would benefit from a short summary of what was actually adopted, especially when there are differences between existing studies and/or interpretation required.

It is noted that substantial calculations regarding overtopping have been undertaken in RHDHV (2023). These calculations appear to be predominantly sound, with the following caveats.

The most extreme ARI calculated was 100 year ARI. Some valid discussion was provided that indicated that the assumptions behind this were “conservative”. While this is somewhat accepted, an additional ARI in the range 500 to 2500 years is recommended to comply with a range of standards. Furthermore, while no comparisons are known to WRL, the use of 6 hour wave height rather than 1 hour is potentially non-conservative, however, it is a common, but not universal practice.

The assumption that the seaward face of the SLSC structure is in the same cross shore position as the seawall is very conservative. While difficult to locate, some techniques are available to account for the setback – such as in the FEMA (USA) Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States (2005) and Coastal Construction Manual (2011).

No calculations were undertaken in RHDV (2023) for a wave return wall. This is highly likely to be able to substantially reduce wave overtopping to tolerable or acceptable levels. Desktop design methods for wave return walls are available, however, physical modelling would be required for detailed design of this feature.

As noted in RHDHV (2023), physical modelling will be required for detailed design to progress, with the calculations presented informing preliminary and concept design. Physical modelling reduces the risk of both underdesign (unanticipated failure) and overdesign (excessive capital cost).

Based on the reviewer’s experience in comparable locations, it is likely that an appropriate certifiable detailed design can be developed within the presented concept design if additional design work is undertaken.

For high sea level rise scenarios, the future of a sandy beach at Bronte may require active management, noting that the present SLSC proposal does not significantly change the status quo, except for extending the life of the present situation/seawall alignment.

3. Detailed peer review of RHDHV (2023)

RHDHV (2023) is of a good professional standard.

In many places “Waverly” should be spelt as Waverley. WRL has not reviewed for any other spelling or grammar issues.

Section 1.2
The proposed seawall protects the promenade and buildings from erosion and inundation, where inundation is due to wave runup and overtopping.

Add “potentially” before “increased storm frequency”.

Add that the existing seawall has no parapet or wave return wall.

Section 2.1
Minimal details of the SBEACH modelling are presented, however, it is noted that it was undertaken by Baird for ARUP (2016). This has not been sighted by WRL. Therefore, while WRL agrees with the statement that the ARUP (2016) design scour levels appear to be high (elevated), no details are known.

WRL agrees that the June 2016 storm was probably a 20 to 40 year ARI, however, no attribution is given to RHDHV’s comment that it was a 30 year ARI.

Section 2.3
Figures 2-7, 2-8 and 2-9 could be portrayed in landscape mode for better legibility.

Section 2.5.4
Page is blank.

Section 4.1
Add “at present” before “approximately equal to Mean Sea Level”. Suggest replacing “Australian” by “NSW” in “at the Australian coastline”.

Section 4.7
The most extreme ARI calculated was 100 year ARI. Some valid discussion was provided that indicated that the assumptions behind this were “conservative”. While this is somewhat accepted, and likely acceptable for the inundation hazard, an additional ARI in the range 500 to 2500 years for structural design is recommended to comply with a range of standards.

Last paragraph on page 27, check sentence and punctuation in “(refer to Figure 4-5). British Standards. This probability may be unacceptably high …”

RHDHV later noted that the probability of a 100 year ARI storm event over the 70- year design life “… may be unacceptably high …”

The relatively elevated design scour levels are likely acceptable due to the presence of bedrock.

While WRL concurs that the design waves at the site will be depth limited, larger offshore waves cause larger nearshore wave setup and therefore the potential for larger depth limited waves.

Section 4.8.2
The 100 year ARI water level of 1.5 m AHD (WorleyParson, 2011) is slightly higher than the 1.44 m AHD (Watson and Lord, 2008) value presented in Section 4.8.4, however, this 0.06 m difference is minor.

As later stated in RHDHV (2023), the full quantum of wave setup may not be realised at the seawall. This is because typical wave setup calculations are at the shoreline of a beach, the seawall truncates the surf zone at Bronte during large wave events.

Section 4.8.4

Reword sentence "This is like be the corresponding value reported by Manly Hydraulics Laboratory (MHL) (Manly Hydraulics Laboratory, 2018)[2]".

Section 4.9

While no known comparisons are known to WRL, the use of 6 hour wave height rather than 1 hour is potentially non-conservative. However, it is a common, but not universal practice.

Section 5.2

A plot of the coastal erosion hazard lines in the event of seawall failure would assist in understanding the erosion hazard and the dependence on the seawall as management of the hazard.

Section 5.2.1.2

This section confuses the volume of sand available on the beach (180 m^3/m) with the design erosion volume (250 m^3/m), but indicates that there is insufficient sand to meet the design erosion volume, and hence the need for a seawall.

Section 5.2.2

The "average beach width" is presumably mid tide with small waves.

The third paragraph is repeated (in the fourth paragraph).

Section 5.3.1

WRL agrees that there was no significant structural damage in the storms of 1974 and 2016, but notes that there was damage to landscaping and roller shutters.

It is well accepted that the current promenade is unsafe to pedestrians during extreme storms. This applies to many beachfront promenades and is usually managed through access restrictions. For the renewed SLSC building and associated new seawall, the management issue is the avoidance of structural damage to the building and seawall, and restricting inundation to an acceptable or tolerable level.

Section 5.3.2.1

WRL has found that the wave runup method of Mase (1989) has performed well, however, it is only strictly applicable to natural beaches, and cannot readily incorporate a seawall. A note should be made to this effect, but given that wave overtopping calculations are undertaken elsewhere in RHDHV (2023), the Mase (1989) calculations can be retained as a first approximation.

Words to the effect of the following should be added: A wave return wall is likely to be able to reduce wave overtopping to acceptable or tolerable levels over the design life, provided that a physical model is undertaken within detailed design.

Section 5.3.2.3

The following notes are made regarding the wave overtopping calculations:

- A 100 year ARI event is likely acceptable for inundation, but not wave forces on the building
- It is acknowledged that the derivation of the 100 year ARI event may be conservative
- The addition of full wave setup at the seawall may be conservative, as it is likely to be truncated at the seawall – more detailed numerical modelling and/or later physical modelling can resolve this

Case 6 and 7 in Table 5-5 and Table 5-6 may be reversed, or this may be an artefact of the calculation procedure changing equations due to different inputs.

No calculations were undertaken in RHDV (2023) for a wave return wall. Some initial calculations should be undertaken. This is highly likely to be able to substantially reduce wave overtopping to tolerable or acceptable levels. Desktop design methods for wave return walls are available, however, physical modelling would be required for detailed design of this feature.

The assumption that the seaward face of the SLSC structure is in the same cross shore position as the seawall is very conservative. While difficult to locate, some techniques are available to account for the setback – such as in the FEMA (USA) Coastal Construction Manual (2011).

As noted in RHDHV (2023), physical modelling will be required for detailed design to progress, with the calculations presented informing preliminary and concept design.

WRL agrees that "… Cases 5 and 6 could potentially cause structural damage …" , however, the building could be designed to withstand these forces; the forces may be conservative due to the calculations being undertaken without a setback for the building; a wave return wall could reduce the wave forces at the building.

Section 5.3.2.4
WRL concurs with the management options presented. With regard to glazing, toughened or laminated glass will be required for a public building regardless of wave forces, but it may be required to be stronger than that required for wind loads.

Section 5.3.3
WRL accepts that the desktop methods are likely conservative. This risk of overdesign can be reduced with a physical model as suggested. WRL notes that there are desktop methods available to allow for a building to be set back behind a seawall (FEMA, 2011). However, it is also noted that in some circumstances, physical modelling results exceed desktop methods, with the physical modelling able to reduce the risk of unanticipated failure.

Case 6 and 7 in Table 5-7 may be reversed, or this may be an artefact of the calculation procedure changing equations due to different inputs.

Section 6
Change "rock wall" to sloping rock rubble revetment. This is to avoid confusion with vertical sandstone walls such as at Manly.

Change the format of the footnote (5) in "130kN/m[5] ", as it connotes a power on the metres unit.

Figure 6-1: The high portions of the ramp may also need a wave return wall, and/or the wave return wall could be on the ramp, not the main seawall.

Section 7
As stated previously, physical modelling reduces the risk of both underdesign (unanticipated failure) and overdesign (excessive capital cost).

Section 8
All opinions on planning acts and policies within RHDHV (2023) have been deferred until later drafts.

4. Summary

Thank you for the opportunity to provide this peer review. Please contact James Carley on 0414385053 should you require further information.

Yours sincerely,

Francois Flocard
Acting Director, Industry Research

23 October 2023

WRL Ref: WRL2023082 LR20231023 JTC

COMMERCIAL IN CONFIDENCE

UNSW
Water Research
Laboratory

Jo Zancanaro
Acting Manager, Development Assessment
Waverley Council
Cnr Paul St and Bondi Road
Bondi Junction NSW 2022

By email: jo.zancanaro@waverley.nsw.gov.au

RECEIVED
Waverley Council

Application No: DA-455/2022

Date Received: 23/11/2023

Dear Jo,

RE: Bronte SLSC – peer review

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney is pleased to provide this letter report to Waverley Council (hereafter "Council") for Bronte SLSC – peer review.

The review was undertaken by WRL's Principal Coastal Engineer, James Carley.

The following documents were reviewed:

- (Horton, 2023), *Coastal Risk Assessment and Coastal Engineering Advice on Bronte Surf Lifesaving Club and Community Facility Redevelopment report prepared by Horton Coastal Engineering Pty Ltd dated 31 July 2023*
- Warren and Mahoney (2023a), *Project: Bronte Surf Life Saving Club; Drawing Title: Sea wall layout; Drawing No SK.123 Revision A*
- Warren and Mahoney (2023b) Coastal protection peer review, November 2023 Revision A <20231108_Coastal Peer Review Presentation.pdf>

The following items were specifically requested by Council to be reviewed by WRL:

a. Overview and assessment of recommended coastal works proposed
b. Detailed risk assessment (including physical modelling, where possible or available)
c. Performance of works proposed in terms of coastal inundation and management of risk

2. Summary of peer review

Three documents (reports/drawings/presentations) were reviewed by WRL's Principal Coastal Engineer, James Carley. The documents are of a good professional standard.

The proposed upgraded seawall and likely reinforced concrete construction of the proposed new SLSC building is likely to better serve the function of surf life saving at Bronte.

The works proposed are likely to be able to manage coastal hazards for appropriate and foreseeable design events and sea level rise over the next 50 to 70 years subject to additional engineering design. The predominant hazard to be managed will be coastal inundation and wave forces through wave overtopping. For the existing and proposed new SLSC building, the hazards of erosion and recession are/will be managed through the presence of a seawall.

However, there are no calculations presented regarding wave runup and overtopping. Such calculations are complex – they could be initially undertaken using desktop methods such as EurOtop (2018) supplemented with physical modelling, or undertaken solely in a physical model.

Nevertheless, based on the reviewer's experience in comparable locations, it is likely that an appropriate certifiable detailed design can be developed within the presented concept design if additional design work is undertaken.

For high sea level rise scenarios, the future of a sandy beach at Bronte may require active management, noting that the present SLSC proposal does not significantly change the status quo, except for extending the life of the present situation/seawall alignment.

3. Detailed peer review

3.1 Horton (2023)

Horton (2023) is of a good professional standard.

The works proposed are likely to be able to manage coastal hazards for appropriate and foreseeable design events and sea level rise over the next 50 to 70 years subject to additional engineering design. The predominant hazard to be managed will be coastal inundation and wave forces through wave overtopping. For the existing and proposed new SLSC building, the hazards of erosion and recession are/will be managed through the presence of a seawall.

Horton (2023) documented two major coastal inundation events and reported other events over the past 100 years.

Horton listed a range of suitable techniques to manage wave forces and inundation.

However, there are no calculations presented regarding wave runup and overtopping. Such calculations are complex – they could be initially undertaken using desktop methods such as EurOtop (REF) supplemented with physical modelling, or undertaken solely in a physical model.

Nevertheless, based on the reviewer's experience in comparable locations, it is likely that an appropriate certifiable detailed design can be developed within the concept design presented in Horton (2023).

Furthermore, no adaptive pathways in response to future climate change are presented. An example could be the future construction of a wave return wall when a threshold of future sea level rise is reached – with such a structure possibly not required in the present day.

It is suggested that a range of design event Average Recurrence Intervals be developed and presented. The structural elements of the proposed new SLSC building are largely covered within the Building Code of Australia, Australian Standard 1170.0 and AS 4997.

An appropriate design event for other components of the project is not specifically covered by published standards, but the project would benefit from stated design conditions.

Examples of suggested components (subject to further discussion) could be:

- Structural elements of building and seawall (Building Code of Australia, AS1170.0 Structural design actions: General principles and AS 4997-2005, Guidelines for the design of maritime structures):
    - 500 to 2500 year ARI
- Replaceable building elements (e.g. Shutters, doors and windows):
    - 10 to 100 year ARI
- Inundation of building:
    - 10 to 100 year ARI
- Wave overtopping of promenade:
    - 1 month to 10 year ARI

A portion of the proposed works occupy a portion of sandy beach. The revised layout by Warren and Mahoney (2023a) reduces this portion and simplifies the layout.

The estimated recession due to sea level rise may be overstated by Horton (2023). WRL accepts the use of the Bruun Rule with a Bruun Factor of 50. It is recommended that recession be tabulated in a similar manner to Table 2 of Horton for each sea level rise scenario.

Horton (2023) and studies referenced within it indicate that Bronte Beach has remained stable or accreted over past decades while the sea has risen an average of 1.9 mm per year. This value was calculated by (Watson, 2020) based on data from 1914 to 2018 at Fort Denison, and accounts for changes in sea surface level and vertical land motion to determine the velocity of SLR relative to land. That is, the future recession due to sea level rise could be discounted by the sea level rise that has been occurring, making the recession calculated by Horton an overestimate. Although speculative, sources of sand supply could be offshore shelf supply and cliff erosion.

Therefore, calculated recession for 2103 (0.55 m of sea level rise) could be reduced by approximately 8 m from the values presented by Horton (2023).

It should also be noted that the beach will be narrower than its MSL width more than 50% of the time due to tides, wave setup and wave runup. Desktop, high level calculations for other locations by WRL have typically assessed the beach width at approximately 2 m AHD, while more comprehensive studies have undertaken an hourly assessment of ocean conditions, wave runup and the estimated hourly actual dry beach width.

Thus, for high sea level rise scenarios, the future of a sandy beach at Bronte may require active management, noting that the present SLSC proposal does not significantly change the status quo, except for extending the life of the present situation/seawall alignment.

Horton (2023) also suggested beach scraping as a means of accelerating beach recovery following storm events – this is supported by the reviewer, noting that there may be limits to this under future high sea level rise scenarios.

3.2 Warren and Mahoney (2023a)

Warren and Mahoney (2023a) is of a good professional standard. Warren and Mahoney (2023a) provides a simplified layout for the seawall and beach access over the Horton (2023) design and occupies less present sandy beach space.

Similar comments apply as to Horton (2023) regarding calculation of wave overtopping and wave forces. That is, there are no calculations presented regarding wave runup and overtopping. Such calculations are complex – they could be initially undertaken using desktop methods such as EurOtop supplemented with physical modelling, or undertaken solely in a physical model.

Nevertheless, based on the reviewer's experience in comparable locations, it is likely that an appropriate certifiable detailed design can be developed within the concept design presented in Warren and Mahoney.

3.3 Warren and Mahoney (2023b)

Warren and Mahoney (2023b) is of a good professional standard. It is a presentation providing comparison between the original design of Horton (2023) and the revised design in Warren and Mahoney (2023a).

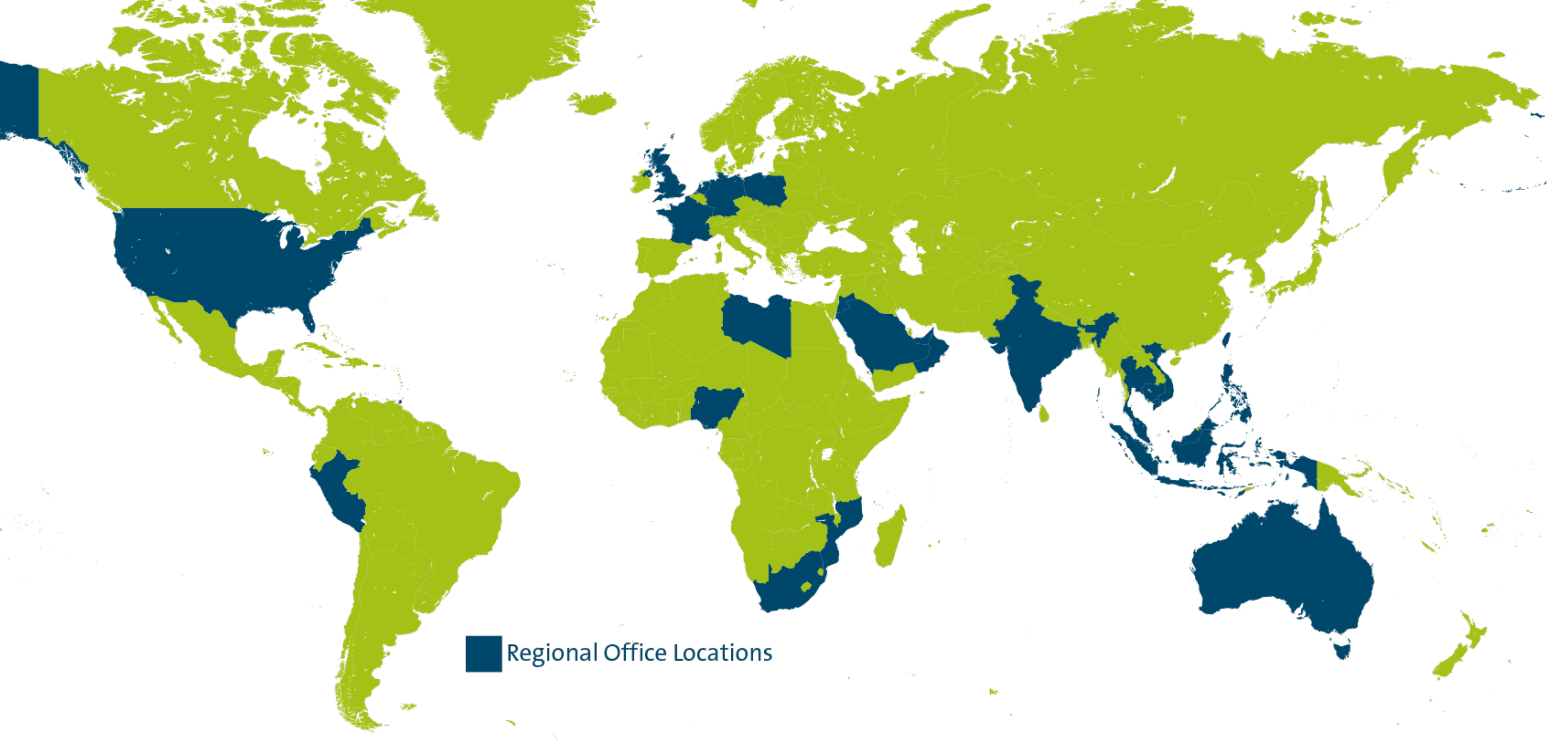
The only comments specific to Warren and Mahoney (2023b) are that a raised seawall crest combined with a wave return wall are indicated in front of the proposed new SLSC building. As discussed previously, there are no calculations presented regarding wave runup and overtopping. Such calculations are complex – they could be initially undertaken using desktop methods such as EurOtop supplemented with physical modelling, or undertaken solely in a physical model.

4. Summary

Thank you for the opportunity to provide this peer review. Please contact James Carley on 0414385053 should you require further information.

Yours sincerely,

Brett Miller
Director, Industry Research



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