Horton Coastal Engineering
Coastal & Water Consulting

Coastal Engineering and Flooding Advice for Newport SLSC Clubhouse Redevelopment

Prepared by Horton Coastal Engineering Pty Ltd

for Adriano Pupilli Architects

Issue 3

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1. INTRODUCTION AND BACKGROUND

It is proposed to undertake alterations and additions at Newport SLSC. Northern Beaches Council requires that a coastal engineering assessment is prepared as part of a Development Application (DA) for the works. Horton Coastal Engineering Pty Ltd was engaged by Adriano Pupilli Architects (architects for the clubhouse redevelopment) to complete this assessment, as set out herein.

Horton Coastal Engineering was also engaged to consider rainfall-runoff related flooding risks at the SLSC, but the low risk to the proposed development from flooding meant that Council did not require a merit assessment on this matter.

The report author, Peter Horton [BE (Hons 1) MEngSc MIEAust CPEng NER], is a professional coastal and water engineer with 29 years of experience. He has postgraduate qualifications in coastal and water engineering, and is a Member of Engineers Australia and Chartered Professional Engineer (CPEng) registered on the National Engineering Register. He is also a member of the National Committee on Coastal and Ocean Engineering (NCCOE) and NSW Coastal, Ocean and Port Engineering Panel (COPEP) of Engineers Australia. Peter has inspected the area in the vicinity of the SLSC on numerous occasions in the last two decades, including specific recent inspections on 11 July 2019, 17 August 2019, 26 February 2020, 18 May 2020, 31 May 2020, 17 August 2020, and 31 May 2021.

To provide protection such that the redeveloped SLSC would be at an acceptably low risk from undermining due to coastal erosion/recession, buried coastal protection works (a buried seawall) have also been proposed as part of the DA. Details of the buried seawall design, and a merit assessment of the seawall from a coastal engineering perspective, are provided in separate reports by Horton Coastal Engineering and others. For the purpose of the clubhouse assessment herein, it has been assumed that a seawall is in place, with a minimum design life of 60 years.

Note that all levels given herein are to Australian Height Datum (AHD). Zero metres AHD is approximately equal to mean sea level at present.

The report is structured as follows:

- information provided is listed in Section 2;
- a description of the existing subject site is provided in Section 3;
- the proposed development is described in Section 4;
- erosion/recession hazards and coastal inundation hazards at the subject site are described in Section 5 and Section 6 respectively;
- catchment and overland flow flooding issues are considered in Section 7;
- a merit assessment of the proposed clubhouse in relation to relevant coastal engineering considerations is provided in Section 8;
- conclusions and references are provided in Section 9 and Section 10 respectively;
- a UNSW Water Research Laboratory peer review and desktop assessment are provided in Appendix A and Appendix B respectively;
- structural engineering assessments of a temporary barrier to reduce wave forces on the SLSC building, and the structural feasibility of the building redevelopment, are provided in Appendix C and Appendix D respectively; and
- completed Forms 1 and 1(a) as given in the *Coastline Risk Management Policy for Development in Pittwater* are provided in Appendix E.

2. INFORMATION PROVIDED

Horton Coastal Engineering was provided with 20 drawings (namely Dwg # 000 to 019) prepared by Adriano Pupilli Architects, all Revision A and dated 25 August 2021.

A site survey completed by CMS Surveyors, dated 7 May 2018 (with the survey carried out on 13 April 2018) and reference “17692detail” was also provided.

3. EXISTING SITE DESCRIPTION

3.1 General Description

Views of the SLSC building from Newport Beach on 20 August 2020 are provided in Figure 1 and Figure 2, with an oblique aerial view on 25 January 2021 provided in Figure 3. The concrete pathway (promenade) seaward of the Newport SLSC building has a level of about 5.4m AHD at its seaward edge, increasing slightly to 5.5m AHD at the face of the SLSC. The finished ground floor level of most of the building (the two-storey portion) is 5.7m AHD, with a lower elevation ground floor level of 5.5m AHD over the northern single storey portion. On the ground floor, a total of 6 roller doors, 1 toilet entrance, 1 double timber door, 2 low level windows, and 5 mid-high level windows face seaward. The protruding seaward section with the double timber door has a timber door on both the northern and southern side, four park seats on the seaward side, and one park seat on its southern side under stairs.

Figure 1: View of Newport SLSC building from Newport Beach on 17 August 2020, facing SSW

Figure 2: View of Newport SLSC building from Newport Beach on 17 August 2020, facing north

Figure 3: Oblique aerial view of Newport SLSC on 25 January 2021, facing NW

On the landward side of the SLSC, the top of kerb at the car park varies in level from 6.0m AHD in the north to 5.1m AHD in the south. Levels continue to reduce moving south of the SLSC, to about 3.5m AHD at about 90m south, at the location of a beach accessway swale landward of an ocean stormwater outlet.

Norfolk Island pine trees are located about 8.5m south and 8.1m north of the clubhouse respectively. Water and electricity lines run along the northern end of the accessway north of the clubhouse, that is, about 3.4m north of the clubhouse. A fenced dunal vegetation area is located to the north of this accessway, extending about 20m to the next northern beach accessway.

3.2 Rock Revetment

As described in *Newport Surf Life Saving Club, Season 1973-74, Sixty Third Annual Report*, in a severe coastal storm in May 1974, a concrete ramp and promenade seaward of the SLSC was undermined and collapsed, and wave action and debris (including a concrete beach seat) entered the building (Figure 4). This wave action and associated projectiles caused internal damage to the gear room, power boat shed, and board and ski shed, although review of photographs does not indicate any structural damage to the building. Tonnes of sand also filled the SLSC. There was a 3m to 4m drop to the sand from the promenade due to the erosion that occurred.

Figure 4: Damage to Newport SLSC from coastal storm in May 1974

Rock revetment coastal protection works (Figure 5, Figure 6 and Figure 7) were placed seaward of the SLSC by the then Warringah Shire Council, as an emergency response to the May 1974 storm erosion, which successfully protected the building from being undermined at that time. It would appear that these works were placed without filter layers or underlayers under the primary sandstone armour and with an overly high toe level (Figure 5), with significantly undersized rock between the larger boulders on the outer primary armour layer (Figure 6), and with a typical primary armour dimension of up to about 1m, which is undersized to be hydraulically stable in a severe coastal storm. Based on the *Annual Report* noted above, the works are meant to extend over the entire length of the SLSC.

Figure 5: Rock revetment being placed seaward of Newport SLSC on 28 May 1974

Figure 6: Rock revetment still visible seaward of Newport SLSC in December 1974

Figure 7: Rock revetment still visible seaward of Newport SLSC in February 1975

JK Geotechnics completed excavation of test pits (TP) and Dynamic Cone Penetrometer (DCP) testing near Newport SLSC on 7 August 2019, to assist with understanding the nature of the existing rock revetment. Four boreholes were also drilled. The 4 TP locations were positioned seaward of the concrete promenade seaward of the SLSC, with TP5 in the south and TP8 in the north. Multiple DCP tests were carried out in a cross-shore direction at each TP, to assist in determining the level and extent of the revetment boulders. Refer to a separate JK Geotechnics (2020) report provided as part of the DA documentation for further details on their investigation.

The TP and DCP tests revealed that the existing rock revetment may only comprise a single layer of boulders, has many undersized boulders, is poorly interlocked, and has an inadequately high toe level. The potential for only a single layer of boulders being present was evident with various DCP tests penetrating without obstruction between the upper layer of boulders (namely DCP5-A, 6-A, 6-B, 6-C, 7-C, 7-D, 7-E, 8-B, and 8-F). This is consistent with Figure 5, where only a single layer of boulders appears to be present. Undersized boulders (including cobbles) were evident in the test pits, and this is consistent with the undersized material visible in Figure 6 to the left and right of the people. Poor interlocking of the boulders was evident with numerous gaps found between the boulders, as per the penetrating DCP tests noted above. The toe of the revetment appeared to be at about 1.8m AHD at TP5, 2m AHD at TP6, 1.8m AHD at TP7, and 1.8m AHD at TP8. An outline of the top surface of the revetment from the test pits, relative to historical beach profiles, is provided in Figure 9 and Figure 10.

The existing revetment works may provide some protection in a severe coastal storm, but do not satisfy current design standards, and cannot be certified by a qualified coastal engineer (nor relied upon) as providing an acceptable level of protection. Therefore, future effectiveness of these existing protection works in acceptably reducing the risk of undermining Newport SLSC cannot be guaranteed.

3.3 Historical Beach Profiles

The NSW Government has recorded historical beach profiles at Newport Beach, derived from photogrammetric analysis of aerial photography (or directly from LiDAR[1] data collection in recent years) for 15 dates from 1941 to 2020 inclusive. From review of the NSW Beach Profile Database, there is a photogrammetric profile at Newport SLSC, with other profiles covering the length of Newport Beach at a 50m alongshore spacing (see Figure 8).

Plots of the historical beach profiles at Newport SLSC (that is, at the red profile in Figure 8), along with the location of the top surface of the existing revetment from the JK Geotechnics (2020) test pits, are provided in Figure 9 (broad view) and Figure 10 (zoomed view).

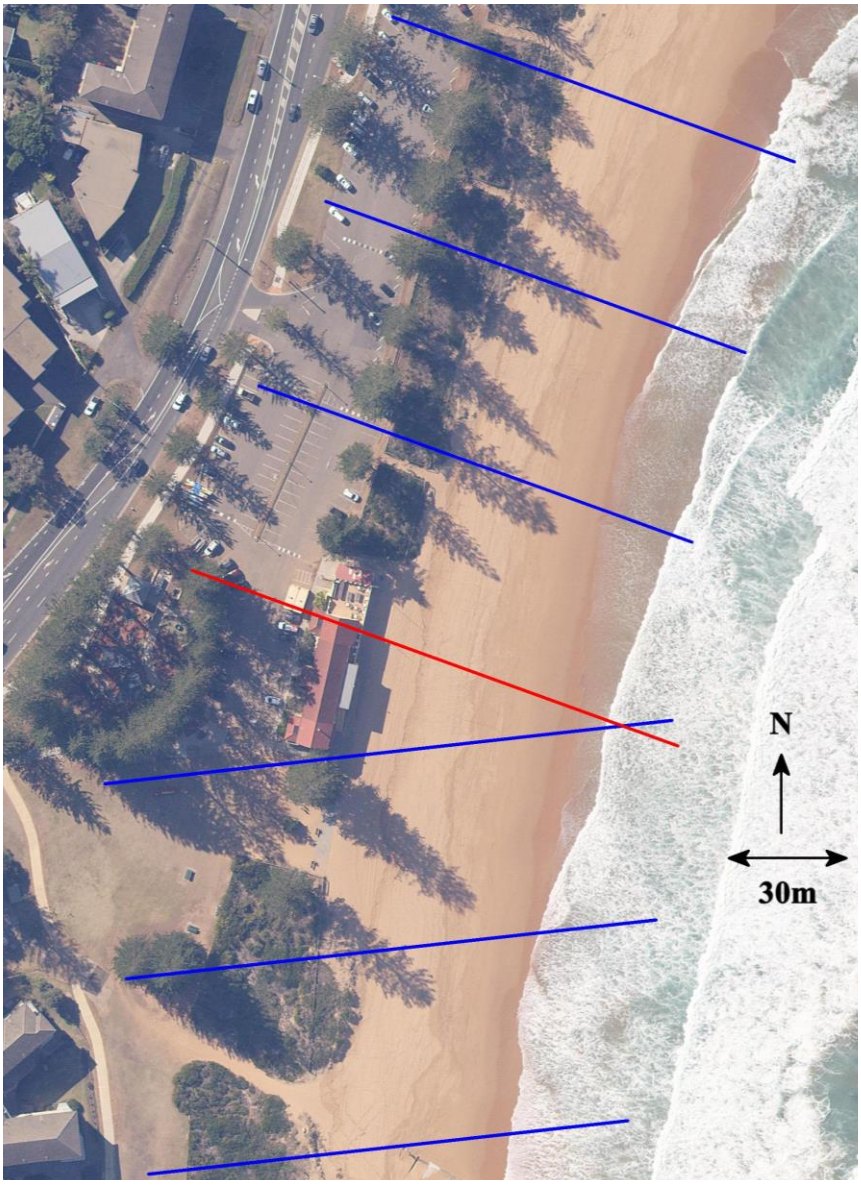

Figure 8: Location of photogrammetric profile at Newport SLSC (red) and other photogrammetric profiles (blue) at Newport Beach (only a selection depicted near the SLSC), with aerial photograph taken on 30 August 2018

[1] LiDAR, which stands for Light Detection and Ranging, uses light in the form of a pulsed laser (typically supported on a flying object such as a plane or drone) to measure distances to the Earth.

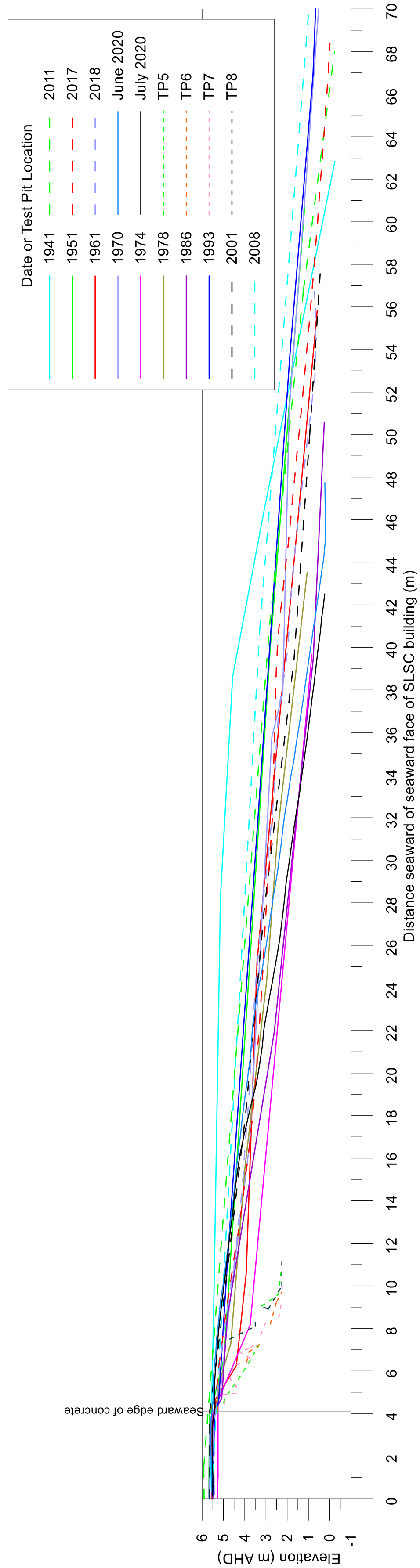

Figure 9: Top surface of revetment from JK Geotechnics (2020) test pits, relative to historical beach profiles derived from NSW Beach Profile Database at Newport SLSC from 1941 to 2020 (broad view)

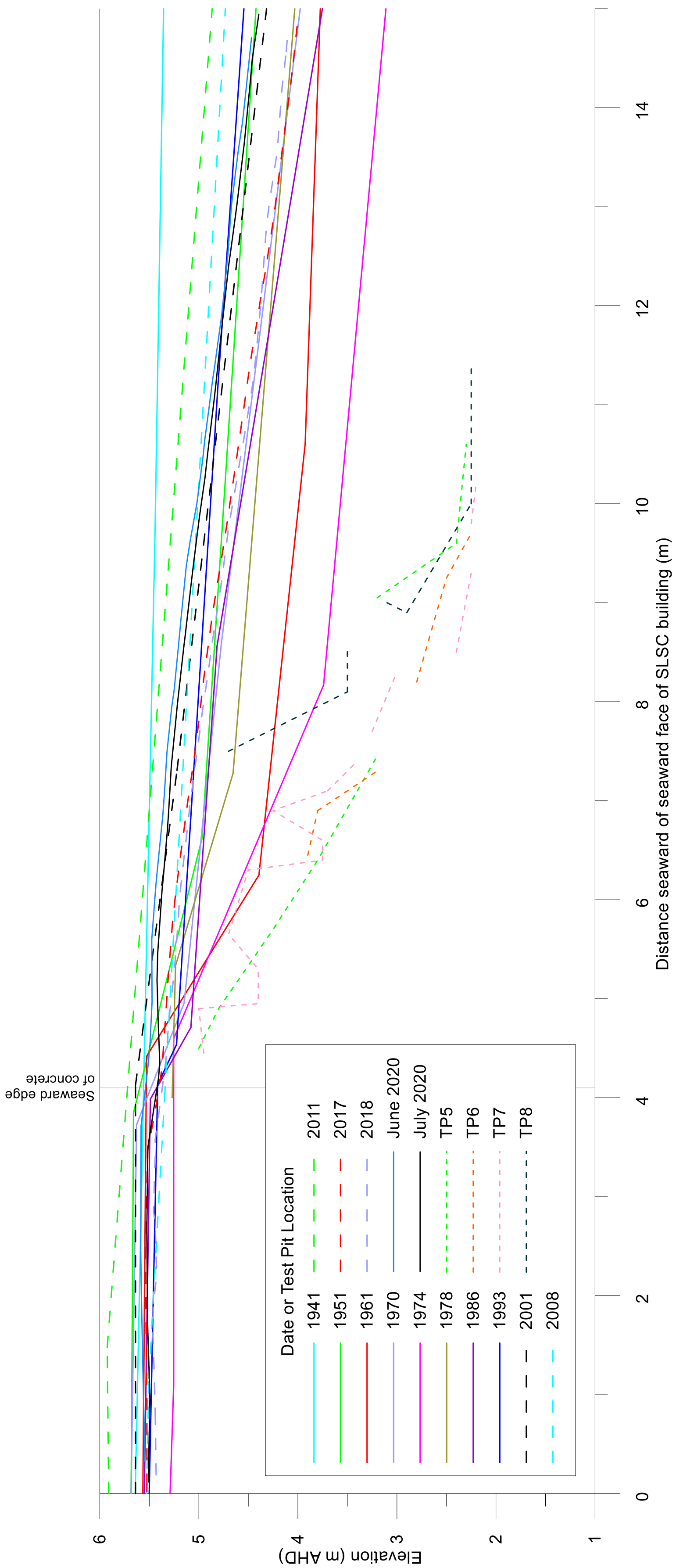

Figure 10: Top surface of revetment from JK Geotechnics (2020) test pits, relative to historical beach profiles derived from NSW Beach Profile Database at Newport SLSC from 1941 to 2020 (close view)

It is evident in Figure 9 and Figure 10 that the 1974 profile is the most eroded profile recorded at Newport SLSC in the NSW Beach Profile Database. However, this profile was captured about 3 weeks after the 29 May 1974 storm, on 19 June 1974, at which time some beach recovery would have occurred, as well as placement of rock boulders and likely some mechanical beach scraping to cover the boulders with sand. Other relatively eroded profiles were in 1986 and 1978. The most accreted profiles were in 1941, 2008 and 1993, although note that the 1941 profile is less accurate than the other profile dates and must be used with caution.

Within a few metres seaward of the concrete promenade seaward of the clubhouse, there have been relatively consistent measured beach profiles over the period of record, with all profiles within about 0.5m in relative level, except in 1961, 1974 and 1978.

In the photogrammetric profiles, the average distance from Newport SLSC to the shoreline at mean sea level is 67m (extrapolating profiles ending above 0m AHD at the same slope as the last two points in the profile).

It is evident in Figure 9 and Figure 10 that the top surface of the revetment generally sits below the 1974 profile (as expected due to beach recovery, discussed above), except at the western edge of TP8. For the profile dates depicted, the last time the revetment would have been significantly exposed was in 1978.

3.4 Variation in Beach Volume and Beach Contour Levels

The analysis in this section was completed for all dates up to 2018.

Plots of the variation in beach volume above 0m AHD seaward of Newport SLSC are provided in Figure 11 (for all photogrammetric dates) and Figure 12 (excluding 1941), along with line of best fit trend lines (dashed). There was a weak recessionary[2] trend including 1941 (of -0.13m^3/m/year), and a stronger accretionary[3] trend excluding 1941 (of +0.39m^3/m/year).

Plots of the variation in various contour positions (these chainages are relative to the landward edge of the red profile in Figure 8) seaward of Newport SLSC are provided as follows, along with line of best fit trend lines (dashed):

- 2m AHD in Figure 13 (for all photogrammetric dates) and Figure 14 (excluding 1941);
- 3m AHD in Figure 15 (for all photogrammetric dates) and Figure 16 (excluding 1941); and
- 4m AHD in Figure 17 (for all photogrammetric dates) and Figure 18 (excluding 1941).

Including 1941, there was a weak recessionary trend or stability for all contour levels (0.0m/year for 2m AHD, -0.04m/year for 3m AHD and -0.06m/year for 4m AHD), and a stronger accretionary trend for all contour levels when excluding 1941 (+0.07m/year for 2m AHD, +0.08m/year for 3m AHD and +0.11m/year for 4m AHD).

[2] A landward movement of the shoreline and the visible beach losing sand volume over the long term.
[3] A seaward movement of the shoreline and the visible beach gaining sand volume over the long term.

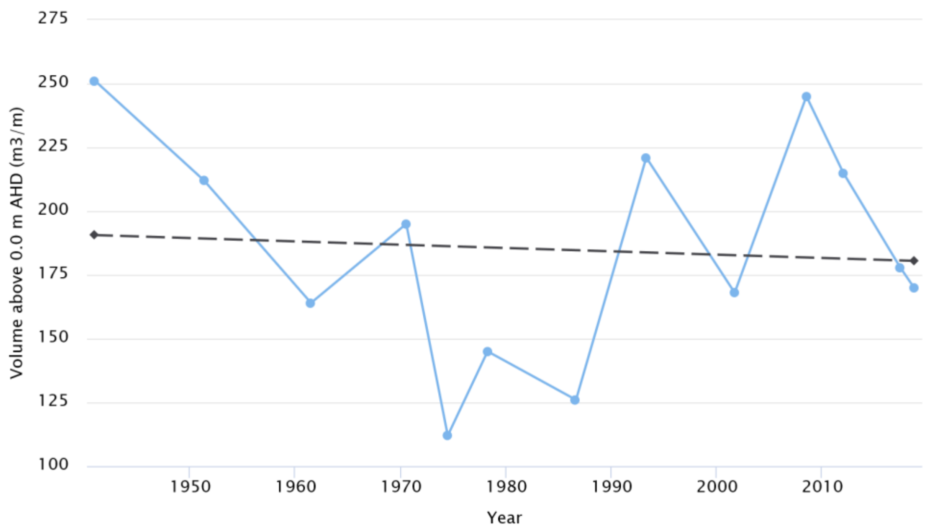

Figure 11: Variation in beach volume above 0m AHD seaward of Newport SLSC for all photogrammetric dates

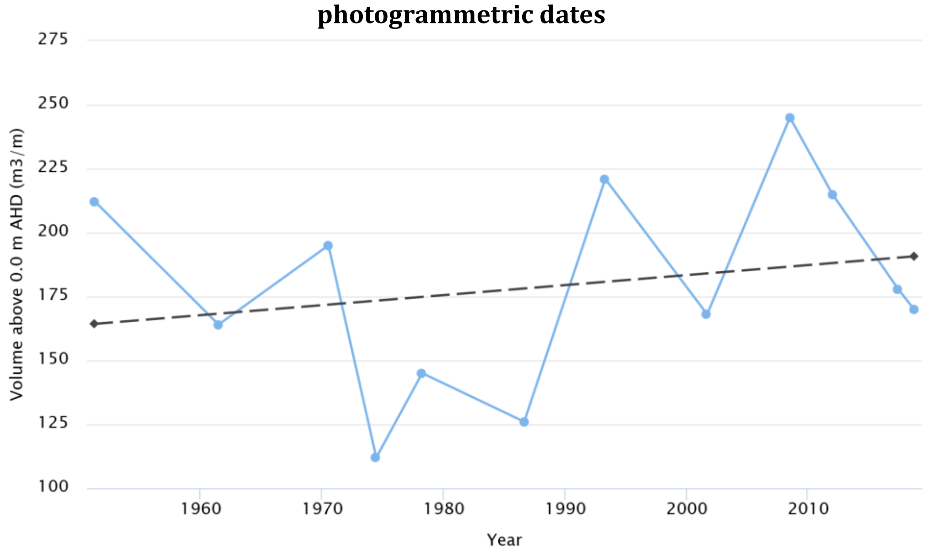

Figure 12: Variation in beach volume above 0m AHD seaward of Newport SLSC for all photogrammetric dates except 1941

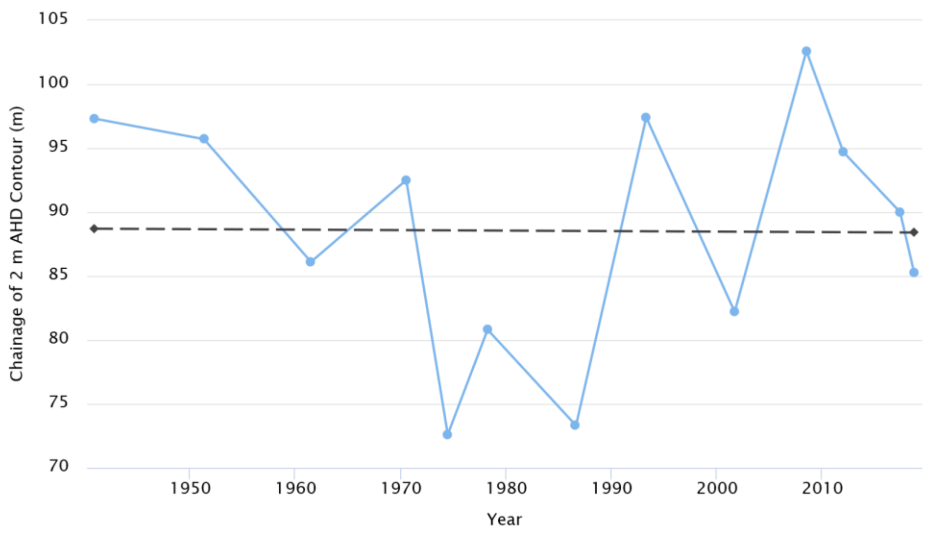

Figure 13: Variation in 2m AHD contour position seaward of Newport SLSC for all photogrammetric dates

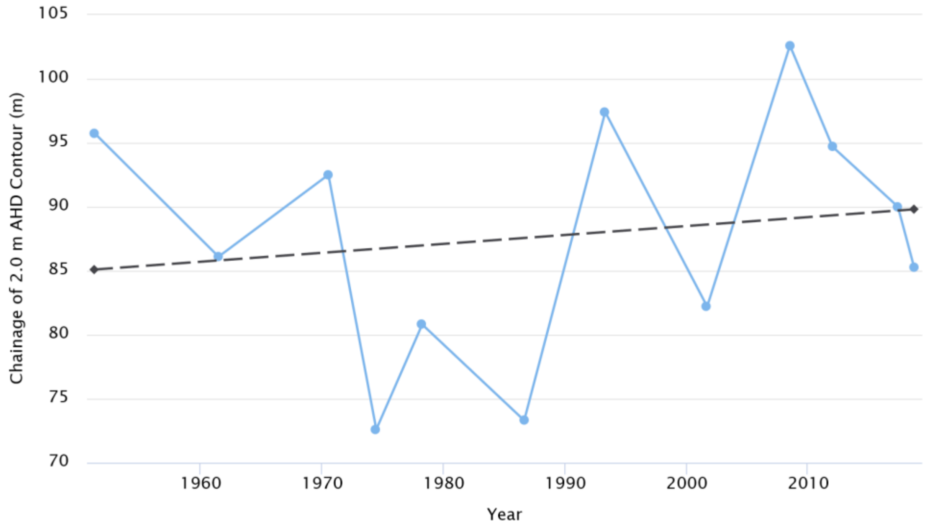

Figure 14: Variation in 2m AHD contour position seaward of Newport SLSC for all photogrammetric dates except 1941

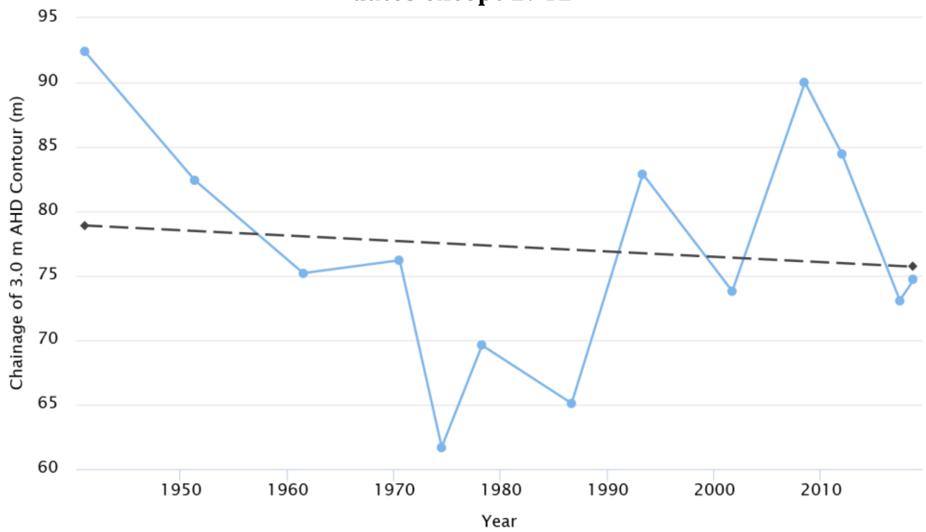

Figure 15: Variation in 3m AHD contour position seaward of Newport SLSC for all photogrammetric dates

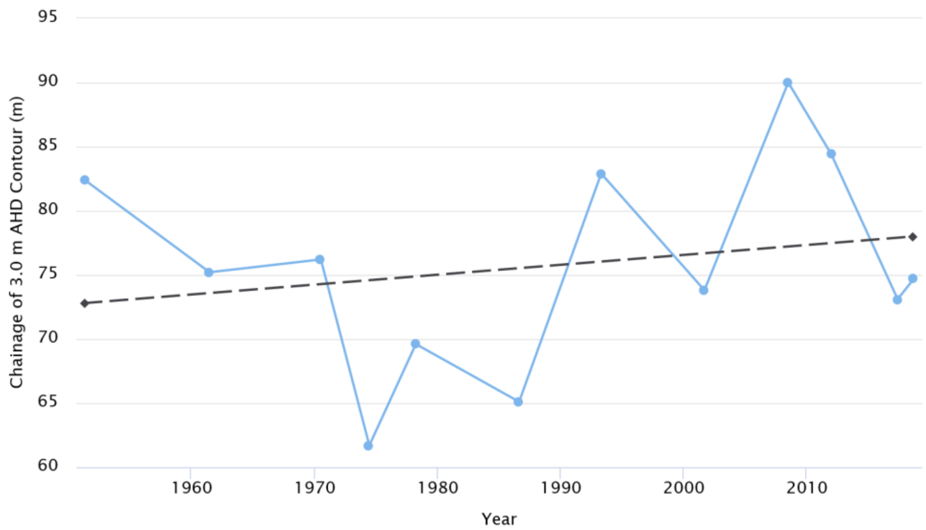

Figure 16: Variation in 3m AHD contour position seaward of Newport SLSC for all photogrammetric dates except 1941

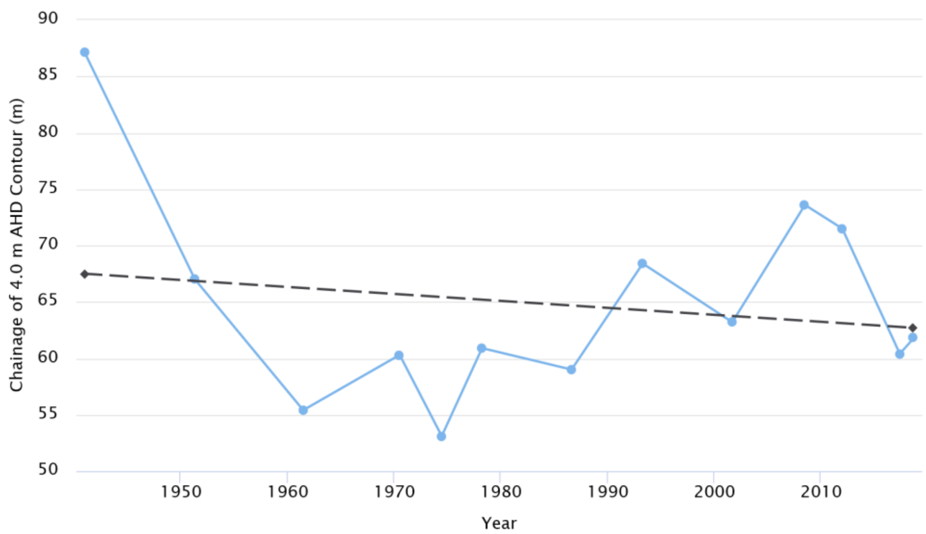

Figure 17: Variation in 4m AHD contour position seaward of Newport SLSC for all photogrammetric dates

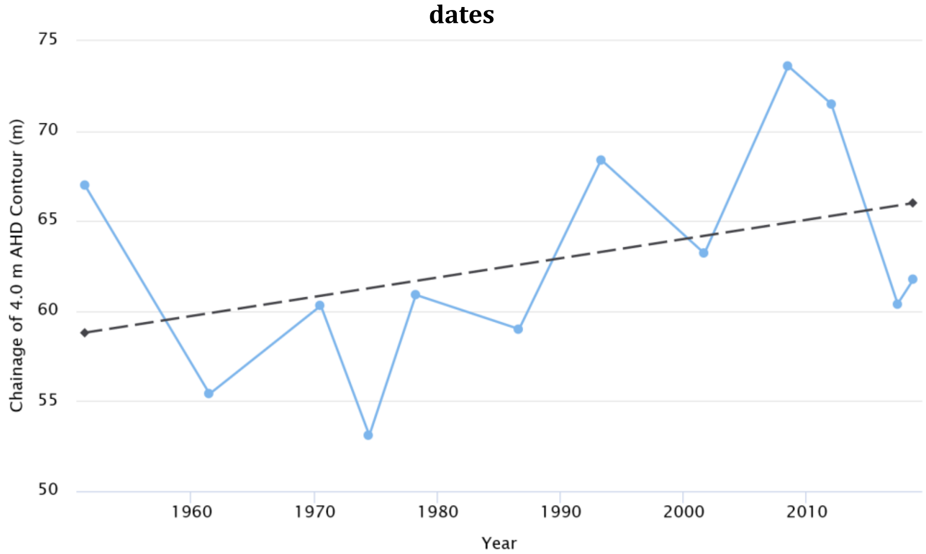

Figure 18: Variation in 4m AHD contour position seaward of Newport SLSC for all photogrammetric dates except 1941

Overall, the plots of the variation in beach volume and contour position in Figure 11 to Figure 18 show the relative long term stability of Newport Beach, without an obvious recessionary or accretionary trend (it is recognised that only one profile has been depicted herein, but the same lack of trend is evident by analysing all profiles).

3.5 Subsurface Conditions

JK Geotechnics completed borehole drilling near Newport SLSC on 7 August 2019, as reported in JK Geotechnics (2020). Boreholes were drilled immediately south and north of the SLSC near its western edge, and on the beach about 15m to 20m seaward of the SLSC. The subsurface was generally found to be sandy down to about:

- -2.5m to -5.1m AHD on the seaward side of the SLSC at BH3 and BH4 (loose to medium dense sand down to -1.2m to -1.7m AHD, then clayey sand or silty sand down to -2.5m to -2.7m AHD, and then medium dense sand at the northern borehole [BH4] down to -5.1m AHD); and

- -0.2m to -2.7m AHD on the landward side of the SLSC at BH1 and BH2 (loose to medium dense sand down to -0.2m to -1.2m AHD, then clayey sand at the northern borehole [BH2] down to -2.7m AHD).

Below the sand:

- stiff to very stiff silty sandy clay was found at BH1 from -0.2m to -3.2m AHD, with clayey sand below this down to -4.7m AHD, then silty sandy clay and clayey sand down to -6.2m AHD, and stiff to very stiff silty clay down to -6.8m AHD;
- very stiff silty clay was found at BH2 from -2.7m to -4.0m AHD, with clayey sand below this down to -6.4m AHD; and
- very stiff silty sandy clay was found at BH3 from -2.5m to -3.9m AHD, with loose sand below this down to -4.9m AHD.

It can thus be concluded that in the active coastal zone (where erosion occurs above about -1m AHD), the natural subsurface seaward of the SLSC would be expected to be fully erodible and sandy, with no constraint on erosion due to stiff clays or bedrock (ignoring any reduction in erosion caused by the existing rock revetment).

4. PROPOSED DEVELOPMENT

The proposed alterations and additions to Newport SLSC comprise internal modifications to the existing building, as well as a two-storey extension of the building at the NW corner.

The initial concept design for the redevelopment of Newport SLSC was completed in June 2018, which retained key heritage aspects of the existing building, while providing a new portion at the northern end within the existing north-south footprint, and with an extension to the west. At that time, it was proposed that the retained and new portions would be placed on conventional foundations (that is, not designed with deep piled foundations to provide support to the building if undermined by coastal erosion/recession), and there was no consideration of constructing coastal protection works (a seawall or revetment) to prevent undermining of the building by coastal erosion/recession.

Since the initial concept, Horton Coastal Engineering (2018, 2020) has prepared reports on the Newport SLSC redevelopment, and there has been considerable coastal engineering analysis and consultation (with Council staff and Club members) on coastal engineering issues in developing the design concept presented in the subject DA. The DA concept has a buried seawall constructed to provide protection to the SLSC building from erosion/recession for an acceptably rare storm over an acceptably long life. This seawall is discussed in a separate report by Horton Coastal Engineering.

Concepts for redevelopment of Newport SLSC were released for public comment in November 2020, with community engagement conducted until January 2021. A *Community and Stakeholder Engagement Report, Newport Surf Life Saving Club building extensions (Stage 2 of 3)* has been prepared by Northern Beaches Council, dated 4 May 2021, and this is provided as part of the DA documentation. In this Engagement Report, it was noted that only 16% of people who responded did not support the proposed extension concept plan, and only 14% of people who responded did not feel that the proposal will improve the existing facility.

Previous versions of the report herein, and the seawall report by Horton Coastal Engineering noted above, were included in the community engagement documentation. The main coastal engineering issue raised by the community was in relation to coastal inundation (wave runup) coastal hazards, as discussed further in Section 6.

The seawall was adopted for design given the:

- risk to the existing and proposed development from coastal erosion/recession;
- necessity to retain the building in its current location, as it has heritage status and surf lifesaving functions; and
- the invasiveness and cost in attempting to retrofit deep foundation piles to the existing SLSC building (which is on shallow footings) as an alternative means of reducing the risk of damage to the existing and proposed development from coastal erosion/recession.

With the buried seawall in place, it is not necessary to alter the shallow footings of the existing clubhouse, nor necessary to found the new portion on deep piles.

An outline of the proposed development is provided in Figure 20, with the new and existing portions depicted (note that the existing portion is to generally have the same footprint as the existing building). The proposed ground finished floor level is 5.7m AHD over the retained portion, and 5.5 m AHD over the new portion, which is the same as existing.

Along the northern half of the ground floor, the seaward face is proposed to comprise four bifold-up doors, which form the entries to a gear storage compound. The seaward face of the central portion is not proposed to change in general form, with the double timber door and 2 low level windows retained, with a life guard room and first aid room immediately adjacent to the seaward entry, and a plant/communications room and lift further landward. At the southern end, the existing 5 high level windows are to be replaced with terracotta tile privacy screens at a higher elevation. The stairs are also proposed to be removed, and the roller door and mid-level window are to be replaced with two compressed fibre cement fold up doors at a low-mid level.

5. EROSION/RECESSION COASTLINE HAZARDS

5.1 Generic Explanation of Hazard Zones

Nielsen et al (1992) has delineated various coastline hazard zones as discussed below and depicted in Figure 19, assuming an entirely sandy (erodible) subsurface above -1m AHD.

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

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

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

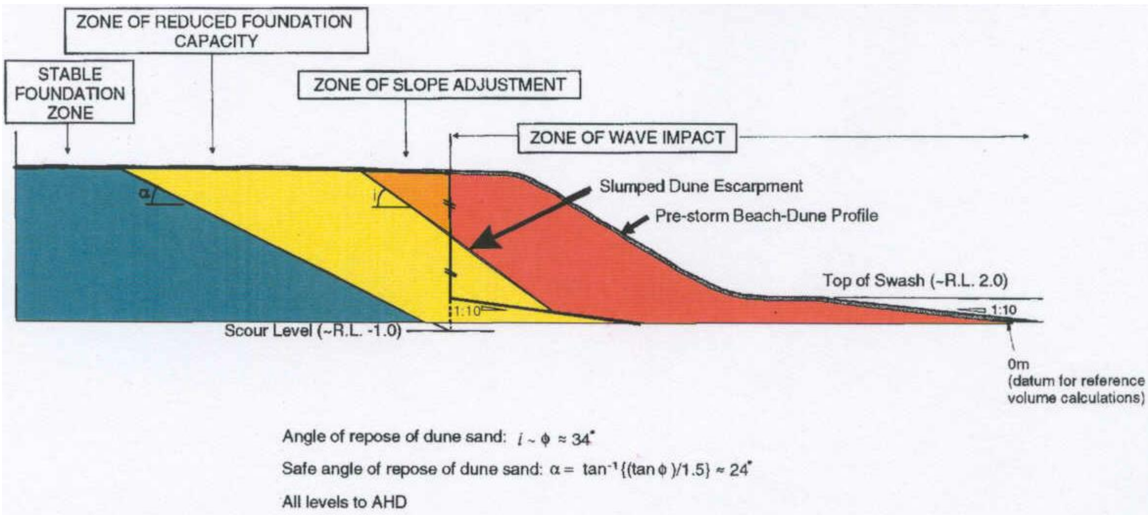

Figure 19: Schematic representation of coastline hazard zones (after Nielsen et al, 1992)

5.2 Current Council Hazard Lines

Lines representing the landward edge of the ZSA and ZRFC for a severe coastal storm (100 year Average Recurrence Interval, or 1 in 100 Annual Exceedance Probability) for immediate, 2050 and 2100 planning periods are depicted in Figure 20, as derived from the *Pittwater Coastline Hazard Definition and Climate Change Vulnerability Study* prepared in 2012. These lines ignore any effectiveness of the rock revetment protection works at Newport SLSC in limiting erosion and recession. An Immediate Wave Runup Line is also depicted in Figure 20.

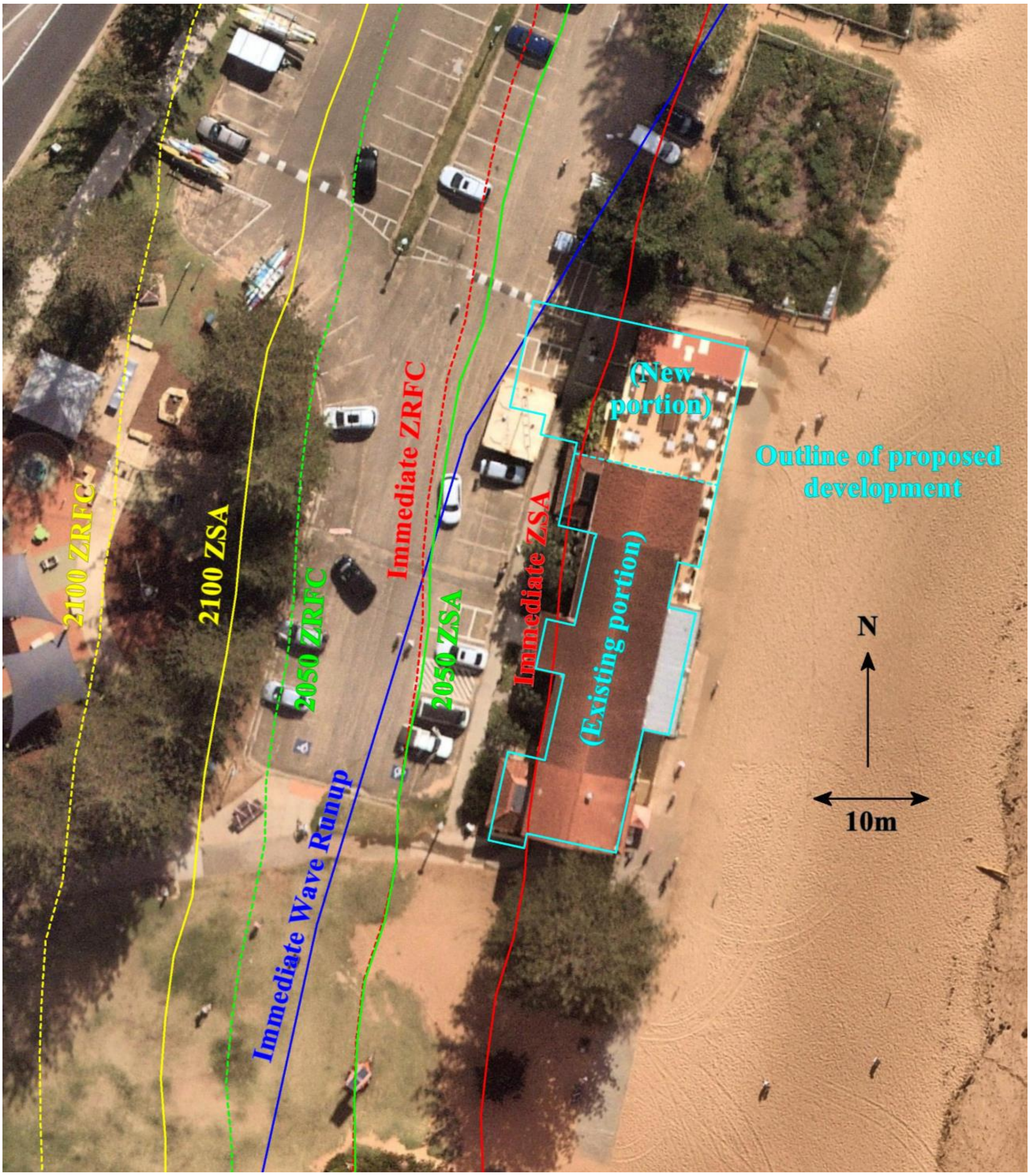

Figure 20: Erosion/recession coastal hazard lines, and Immediate Wave Runup Line, at Newport SLSC

It is evident that the existing/proposed SLSC building is expected to be almost completely undermined in a severe coastal storm at present, ignoring any reduction in erosion from the existing rock boulder protection works. Given that the existing protection works cannot be relied upon to prevent undermining of the SLSC, particularly over the long term (decades), and given that it was considered unacceptable that the SLSC may be substantially damaged in a severe coastal storm (to the extent of having to be completely rebuilt) over its design life, the risk of undermining of the building is proposed to be minimised by construction of a buried seawall seaward of the building.

6. COASTAL INUNDATION COASTLINE HAZARDS

6.1 Potential for Building Damage from Wave Runup

The Immediate Wave Runup Line from the *Pittwater Coastline Hazard Definition and Climate Change Vulnerability Study* is depicted in Figure 20. It is evident that wave runup is expected to extend landward of the SLSC in a severe coastal storm at present, as it did in the 1974 storm. The ground floor of the SLSC building is thus exposed to potential damage from oceanic water inundation, projectile debris at that time, and sand infill carried with the inundation. With projected sea level rise, the frequency and depth of inundation events impacting the SLSC would be expected to increase over time.

6.2 Response to Community Engagement Comments

6.2.1 Nature of Comments

As noted in Section 4, concepts for redevelopment of Newport SLSC were released for public comment in November 2020, and the documentation included previous versions of the report herein, and the separate seawall report by Horton Coastal Engineering. The main coastal engineering issue of relevance raised by the community was in relation to coastal inundation (wave runup) coastal hazards, with assertions that the seawall was not designed to protect the building from direct wave attack and could exacerbate wave attack.

In response, it can be noted that the Horton Coastal Engineering reports released for community engagement in November 2020 did not downplay the issue of wave attack (having included discussion on wave runup damage from the 1974 storms), and outlined measures to reduce the risk of inundation damage and direct wave attack, including the potential use of temporary barriers to reduce wave forces on the building. The proposed seawall would not exacerbate wave attack on the building, as it would reduce wave overtopping when sand levels are below the promenade (due to the effect of the stairs acting as a wave return), and would not alter wave overtopping when sand levels are at the promenade level as at present (ignoring other measures to reduce wave forces on the building than have been included in the concept design, as discussed further below)[4].

6.2.2 WRL Peer Review

Nonetheless, Council commissioned a peer review of the two Horton Coastal Engineering coastal engineering reports that had been prepared for public comment (denoted as the "Horton reports" below). This peer review was undertaken by UNSW Water Research Laboratory (WRL), and is provided as Appendix A.

WRL found that the Horton reports were generally of a high professional standard. WRL noted that some parameters had not been quantified in the Horton reports, and had been deferred until detailed design. WRL also noted that this may be normal practice, but in the case of Newport SLSC the quantification may affect the overall viability or geometry of the project, so additional quantification was recommended.

[4] There was a statement in the community comments that an overtopped vertical wall in the 2016 storm at Collaroy (referring to 1150 Pittwater Road) had a crest level 2m higher than the one proposed at Newport, which is incorrect. The Collaroy wall had a crest level of 5.0m AHD at the time of the 2016 storm, which is lower than the promenade level at Newport of 5.4m AHD.

6.2.3 WRL Desktop Assessment

Accordingly, Council engaged WRL to undertake a desktop assessment of:

- the likely range of sand levels (scour) at the toe of the proposed seawall;
- wave runup and overtopping at the SLSC and associated wave forces on the SLSC building;
- available methods to reduce the wave overtopping hazard; and
- potential end effects as a result of the seawall construction.

The WRL assessment is provided as Appendix B. Note that the end effects issues are considered in the separate seawall report prepared by Horton Coastal Engineering.

WRL found that the scour level adopted in the Horton reports of -2m AHD was conservative. For a 2000 year ARI event at 2080, the most severe event considered, WRL calculated a scour level of -0.7m AHD, some 1.3m higher than adopted in the Horton report. For Council's adopted design scenario of a 500 year ARI event at 2080, the scour level at the seawall calculated by WRL was -0.1m AHD, some 1.9m higher than adopted in the Horton reports.

WRL calculated wave runup and overtopping at the SLSC and associated wave forces on the SLSC building for two scenarios, namely:

1. without any erosion of the beach; and
2. at an eroded (scoured) beach.

For Scenario 1, for the design 500 year ARI event at 2080, overtopping rates were in the order of 20 to 50L/s/m. For Scenario 2, the corresponding rate was about 40L/s/m. These rates would be unsafe for pedestrians, and are no surprise. In the Horton reports, it was noted that significant coastal inundation hazards were expected at the SLSC, and various mitigation measures were proposed to deal with this issue. That stated, the WRL methodology for calculating wave runup levels in Scenario 1 was conservative, as it assumed an infinite height foreshore (ie, that the foreshore extends up to the runup level, even when the foreshore is below that level). In reality, in the opinion of Horton Coastal Engineering, waves would fold over the foreshore crest and travel as a shallow-depth sheet flow, and three-dimensional effects would assist in maintaining these shallow depths (as waves would not overtop the entire length of foreshore seaward of the SLSC at any instant in time).

For Scenario 1, for the design 500 year ARI event at 2080, the total load from wave overtopping was determined to be 103kN/m for the maximum wave (with a depth of 1.3m), and 12KN/m for a force exceeded by 2% of waves. For Scenario 2, the corresponding forces were 30kN/m and 4kN/m. It is considered to be far more likely that Scenario 2 (an eroded beach) would occur in a severe storm, which gave lower wave forces on the SLSC building compared to Scenario 1 in the WRL modelling. It can also be noted that the force calculation of WRL for Scenario 2 was conservative, as it applied at the seaward face of the promenade, whereas forces further landward at the SLSC would be lower.

The proposed seawall would not exacerbate wave overtopping, as it would make no difference to WRL Scenario 1, and would reduce overtopping for WRL Scenario 2 due to its wave return effect.

WRL noted that various measures could be employed to reduce wave overtopping or wave forces namely:

1. installing a wider wave return wall (ie, wider stairs extending further east);
2. installing the wave return wall at a higher elevation (ie, the underside of the stairs at a higher elevation);
3. installing a parapet or wave return wall, noting that this could be in response to a future sea level rise threshold, or may only be needed for the frontage of the old SLSC building;
4. installation of temporary flood barriers in response to a forecast event; and
5. management of the interior of the SLSC building, such as design of the electrical system, and short term response to a forecast event.

Items 3 to 5 were also specifically put forward in the Horton reports, and all of these items have been adopted herein as potential measures to reduce the risk of inundation and waves forces damage to the SLSC building (see Section 6.2.7).

6.2.4 Structural Feasibility of Temporary Barrier to Reduce Wave Forces on Building

James Taylor & Associates, structural engineers for the Newport SLSC seawall design, has undertaken an assessment of the structural feasibility of installation of a barrier for the design 500 year ARI event at 2080, see Appendix C. This was based on the most conservative force estimate of WRL (ie for the maximum wave in Scenario 1, giving 103kN/m). They found that a feasible bollard, infill panel and mechanical connection design could be developed to reduce the likelihood of significant wave forces on the SLSC building for the design event.

6.2.5 Impact of Wave Forces on Design of SLSC Building

Partridge Structural, structural engineers for the Newport SLSC building design, has undertaken an assessment (see Appendix D) of the impact of the most conservative force estimate of WRL (ie, for the maximum wave in Scenario 1, giving 103kN/m) on the existing and proposed new portions of the clubhouse (of course, this assessment assumed that no measures were installed to reduce the magnitude of this force). They found that the existing building would not be able to withstand this impact without remedial measures.

However, feasible remedial measures that were identified comprised introducing a secondary structure to the inside seaward face of the existing building to support the brickwork (either steel stiffening plates or a concrete wall) or introducing a concrete wall on the outside seaward face. These remedial measures would only need to extend up to the level of the wave runup, ie 1.3m in the WRL design event (with this level, and the requirement for any remedial measures, to be refined as part of detailed design). For the new portion, Partridge Structural confirmed that approximately 200mm thick reinforced concrete walls could be used to maintain its structural integrity in the design event (with storage room doors considered to be sacrificial).

The WRL modelling took no account of measures that could be employed to reduce the wave forces on the building, such as solid seating at the seaward edge of the promenade, solid seating at the landward edge of the promenade, and installation of temporary barriers on the promenade, with all of these measures depicted on the architectural drawings and discussed in Section 6.2.4. It is considered that as part of detailed design, eg through a physical modelling assessment, a suitable mix of practical measures would be able to be formulated to reduce the wave forces on the SLSC building to acceptable levels, supplemented by remedial measures to the building(if required) as discussed above. It is not appropriate to detail these options until the forces on the SLSC building are determined with more certainty as part of detailed design, to avoid unnecessary over-design and unnecessary remedial measures based on conservative analytical procedures.

6.2.6 Coastal Planning Level

Wave runup that overtopped the proposed buried seawall at Newport SLSC in severe storms would be expected to be a high-velocity shallow-depth flow, if it occurs. This runup could reach higher elevations when impacting against walls and the like.

For planning purposes consistent with the *Pittwater 21 Development Control Plan*, it is considered to be reasonable to adopt a Coastal Planning Level of 7.2m AHD (which is 1.5m above the highest proposed ground floor level) for areas with openings facing seaward (gear storage compound, lifeguard room, first aid room, and adjacent entry area). For other areas (plant/communications room, storage rooms surrounding the lift, entry area west of the lift, amenities areas), a Coastal Planning Level of 6.7m AHD is considered to be reasonable. A Coastal Planning Level of 6.0m AHD is considered to be reasonable for the plant room on the landward side of the building to the north of the entry.

6.2.7 Adopted Measures to Reduce the Risk of Inundation and Wave Forces Damage to the SLSC Building

Measures to reduce the risk of inundation and wave forces damage (where practical) on the ground floor comprise construction and operational measures. Construction measures that will be considered further as part of detailed design comprise:

- installation of staggered solid seating at the seaward and landward edges of the promenade, seaward of the retained portion of the SLSC building (see the architectural drawings for an indicative layout), to reduce wave forces and inundation depths at the building;
- installing a wider wave return wall (ie, wider stairs or wider promenade extending further east)[5] or installing the underside of the stairs at a higher elevation;
- allowance for bollard cast-in sleeves and the like (if required) for installation of a temporary bollard and infill panel barrier on the promenade prior to coastal storms, seaward of the retained portion of the SLSC building (this could also potentially be extended to seaward of the new portion of the SLSC building), to again reduce wave forces and inundation depths at the building;
- installing remedial measures (if required) on the seaward face of the retained portion of the SLSC building, such as a secondary structure to the inside face to support the brickwork (either steel stiffening plates or a concrete wall) or a concrete wall on the outside face, which would assist in allowing the existing building to resist any applied wave forces;
- using sufficiently thick reinforced concrete walls to maintain the structural integrity of the new portion in the design event (with storage room doors considered to be sacrificial unless the temporary barrier could be extended seaward of the new portion);
- using floor finishes and wall materials (up to the relevant Coastal Planning Level) that would withstand inundation, such as concrete and tiles;

[5] A limit of a 2m extension has been adopted for this option so as not to impact on coastal processes or beach amenity.

- allowing for wave forces on glazing, or constructing glazing that faces seawards from toughened/laminated glass with appropriate fracture characteristics that present a low hazard when fractured, or such that it holds together when shattered;
- placing electrical fittings and outlets that could be damaged by inundation above the relevant Coastal Planning Level, or waterproofing them below this;
- ensuring that the lift and lift shaft includes no items that could be damaged by inundation below 6.7m AHD (noting that the lift car could be sent to the upper level prior to a storm);
- constructing the privacy screen at the southern end of the clubhouse from solid materials resistant to wave forces for at least 0.9m above natural ground, to reduce the potential for inundation to enter the shop/BBQ room down the southern side of the building; and
- designing cross-falls on the concrete promenade seaward of the building and within the building to ensure that inundation would drain away from the building.

Operational measures to reduce the risk of inundation damage (where practical) that could be considered comprise:

- storing items that could be damaged by inundation or become polluting due to inundation above the relevant Coastal Planning Level;
- allowing for relocation of items located below the relevant Coastal Planning Level prior to a forecast storm, as part of an adopted emergency action plan;
- developing and adopting an emergency action plan to include sandbagging of door openings (including the raised opening in the shop/BBQ room), particularly on the seaward side of the building, when severe coastal storms are forecast to impact on the building; and
- developing and adopting an emergency action plan to include installation of a temporary barrier (described above) when severe coastal storms are forecast to impact on the building.

Even with implementation of these construction and operational measures, some non-structural inundation damage to the SLSC is likely to have to be accepted in severe coastal storms, unless the temporary barrier was to be extended along the entire seaward face of the building.

6.2.8 Synthesis

Although additional tasks may be undertaken as part of detailed design, this does not call the feasibility of the proposed development into question. It is considered that the work of WRL, James Taylor & Associates and Partridge Structural described above shows that a suitable mix of practical measures would be able to be formulated to reduce the wave forces on the SLSC building to acceptable levels, and to provide remedial measures to support the seaward face of the existing building against wave forces (if required). The project as proposed is feasible.

For the purpose of the merit assessment in Section 8, it has been assumed that a suitable mix of construction and operational measures would be adopted as part of detailed design, in consultation with a coastal and structural engineer, to reduce the risk of inundation damage to the building to acceptable levels.

7. CATCHMENT AND OVERLAND FLOW FLOODING

As advised by Council, Newport SLSC is not subject to catchment and overland flow flooding controls. There is a mainstream flooding path that flows from north to south landward of the SLSC, with the 1% Annual Exceedance Probability (AEP), or 1 in 100 AEP, flood level being 4.7m AHD, and Probable Maximum Flood (PMF) level being 5.9m AHD.

This is well below the ground floor level for the 1% AEP event, and 0.2m to 0.4m above the floor level for the PMF, and less severe than the effects of oceanic inundation as described in Section 6. That is, oceanic inundation is a more significant risk and extends higher than catchment and overland flow flooding for a given probability.

Therefore, the merit assessment in Section 8 does not consider flooding matters.

8. MERIT ASSESSMENT

8.1 Chapter B3.3 of *Pittwater 21 Development Control Plan*

Chapter B3.3 of the *Pittwater 21 Development Control Plan* (DCP) does not actually apply at the subject site, as this document only applies to private property, but has been considered in general terms for consistency with coastal planning for private development in the area[6]. Based on Chapter B3.3 of the DCP (numbered for convenience herein):

1. all development on land to which this control applies must comply with the requirements of the *Coastline Risk Management Policy for Development in Pittwater* (Part B, Appendix 6 of the DCP);
2. development must be designed and constructed to ensure that every reasonable and practical means available is used to remove risk to an acceptable level for the life of the development;
3. the development must not adversely affect or be adversely affected by coastal processes nor must it increase the level of risk for any people, assets and infrastructure in the vicinity due to coastal processes;
4. the Statement of Environmental Effects [is to include] a statement in relation to the proposed development outlining how it has been designed and will be constructed to address the Coastal (Beach) Hazard;
5. the application is to be accompanied by a report prepared by a NPER Engineer with coastal engineering as a core competency and having an appropriate level of professional indemnity insurance;
6. the report is to provide an assessment of the risk and should demonstrate that the proposal is designed and has been located to achieve the control requirements; and
7. the report should also provide management procedures to be carried out during construction and over the life of the development to achieve an acceptable level of Risk Management.

With regard to Item 1, see Section 8.2.

For Item 2, with a buried seawall constructed as described in a separate report by Horton Coastal Engineering, the proposed development would be at an acceptably low risk of damage from coastal erosion/recession over its design life. Furthermore, if a suitable mix of construction and operational measures as listed in Section 6 are adopted, in consultation with a coastal engineer, the proposed development would be at an acceptably low risk of damage from oceanic inundation over its design life.

For Item 3, the proposed clubhouse development is unlikely to have a significant impact on coastal hazards nor increase the risk of coastal hazards in relation to any other land over its design life, as it is built over the same north-south and seaward extent as the existing building, and constructed landward of a seawall[7]. That is, it would not be expected to adversely affect coastal processes nor increase the level of risk for any people, assets and infrastructure in the vicinity due to coastal processes over its design life.

For Item 4, the proposed building has been designed and would be constructed with a buried seawall in place seaward of the building, and with construction and operational measures to

[6] The DCP version up to Amendment 27 (effective from 18 January 2021) was considered herein.
[7] Potential impacts of the seawall are considered in a separate report by Horton Coastal Engineering.

reduce the risk of coastal inundation, to satisfactorily address the Coastal (Beach) Hazard over the design life.

For Item 5, the report herein, and its author, meet these requirements.

For Item 6, erosion/recession risks and oceanic inundation risks have been considered in Section 5 and Section 6 respectively. The proposed development has been designed to meet the control requirements (Items 1-7) as described in this Section.

For Item 7, erosion/recession risks are to be managed through construction of a buried seawall, and measures to reduce the risk of coastal inundation have been outlined in Section 6.

The proposed development thus satisfies Chapter B3.3 of the DCP.

8.2 *Coastline Risk Management Policy for Development in Pittwater*

Based on Section 8.2(i) of the *Coastline Risk Management Policy for Development in Pittwater*:

a) all structures below the Coastline Planning Level shall be constructed from flood compatible materials;
b) all development must be designed and constructed so that it will have a low risk of damage and instability due to wave action and/or oceanic inundation hazards;
c) all development and/or activities must be designed and constructed so that they will not adversely impact on surrounding properties, coastal processes or the amenity of public foreshore lands;
d) all uncontaminated dune sand excavated during construction operations shall be returned to the active beach zone as approved and as directed by Council;
e) wherever present, remnant foredune systems shall be appropriately rehabilitated and maintained for the life of the development to stabilise an adequate supply of sand (as determined by a coastal engineer) that is available to buffer erosion processes and/or minimise the likelihood of oceanic inundation;
f) all vegetated dunes, whether existing or created as part of coastal protection measures shall be managed and maintained so as to protect the dune system from damage both during construction of the development and as a result of subsequent use during the life of the development;
g) all electrical equipment, wiring, fuel lines or any other service pipes and connections must be waterproofed to the Coastline Planning Level;
h) the storage of toxic or potentially polluting goods, materials or other products, which may be hazardous or pollute waters during property inundation, will not be permitted below the Coastline Planning Level;
i) for existing structures, a tolerance of up to minus 100mm may be applied to the Coastline Planning Level in respect of compliance with these controls;
j) building heights must not exceed 8.0 metres above the Coastline Planning Level or 8.5 metres above existing ground level, whichever is higher; and,
k) where land is also subject to the provisions of the Flood Risk Management Policy for Development around Pittwater, the higher of the Coastline Planning Level and Flood Planning Level shall apply.

For Item (a), it was recommended in Section 6 that floor finishes and wall materials that would withstand inundation be used up to at least the Coastline Planning Level.

For Item (b), the clubhouse development is at an acceptably low risk of damage and inundation from coastal erosion/recession and inundation over a reasonable design life, as discussed above.

For Item (c), it has been noted previously that the proposed development would not be expected to adversely impact on surrounding properties or coastal processes.

Item (d) would be achievable and appropriate during construction, although significant excavation would not be expected.

For Items (e) and (f), existing vegetated dune areas north of the building are to be maintained.

For Item (g), a recommendation was provided in Section 6 that electrical fittings and outlets that could be damaged by inundation were placed above the Coastline Planning Level, or waterproofed below this, where practical.

For Item (h), a recommendation was provided in Section 6 that items that could be damaged by inundation, or become polluting due to inundation, be stored above the Coastline Planning Level or relocated prior to a storm.

Item (j) is not a coastal engineering matter and hence is not addressed herein.

For Item (k), the subject site is not mapped as being significantly affected by catchment flooding, and the more severe coastal inundation controls have been considered herein as discussed in Section 7.

In the *Coastline Risk Management Policy for Development in Pittwater*, it is noted that a Coastline Management Line must be defined. With construction of a buried seawall, the Coastline Management Line would be coincident with the seawall face.

Based on Section 8.2(iii) of the Policy, "new development and major additions to existing development must be sited on the landward side of the 100 year Coastline Management Line". The proposed development is to be landward of a Coastline Management Line (seawall) that has been designed for a minimum 60 year design life, which is considered to be a reasonable design life to adopt (adoption of a 100 year life is not mandatory).

Completed Forms 1 and 1(a) as given in the *Coastline Risk Management Policy for Development in Pittwater* are provided in Appendix E.

The proposed development thus satisfies the *Coastline Risk Management Policy for Development in Pittwater*.

8.3 *State Environmental Planning Policy (Coastal Management) 2018*

8.3.1 Preamble

Based on *State Environmental Planning Policy (Coastal Management) 2018* (SEPP Coastal) and its associated mapping, the subject site is within the "coastal environment area" (see Section 8.3.2) and "coastal use area" (see Section 8.3.3).

8.3.2 Clause 13

Based on Clause 13(1) of SEPP Coastal, "development consent must not be granted to development on land that is within the coastal environment area unless the consent authority has considered whether the proposed development is likely to cause an adverse impact on the following:

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

With regard to (a), the proposed development would not be expected to adversely affect the biophysical, hydrological (surface and groundwater) and ecological environments, as it is renovating a building in an already developed area. The stormwater system will generally remain the same as existing, including the retention of existing rainwater tanks and their connection to roof water. Significant impacts on the hydrological environment are not expected.

With regard to (b), the proposed development would not be expected to adversely affect coastal environmental values or natural coastal processes over a reasonable design life, as it is built over the same north-south and seaward extent as the existing development at an acceptably low risk of being damaged by coastal erosion/recession over a reasonable life.

With regard to (c), the proposed development would not be expected to adversely impact on water quality, as long as appropriate construction environmental controls are applied. No sensitive coastal lakes are located in the vicinity of the proposed development.

With regard to (d), the proposed development would not impact marine vegetation, native vegetation and fauna and their habitats of significance (which are assumed not to exist at the site), and undeveloped headlands and rock platforms, with none of these items in proximity. No significant impacts on marine fauna and flora would be expected as a result of the proposed development, as the development would not be expected to interact with subaqueous areas over its design life.

With regard to (e), it can be noted that the proposed development will not affect public access to Newport Beach, with existing beach accessways to the north and south of the building being maintained. With inclusion of beach access stairs as part of the proposed seawall works, public beach access will be enhanced.

With regard to (f), this is not a coastal engineering matter so has not been considered herein.

With regard to (g), the proposed development would not interact with the surf zone for an acceptably rare storm over a reasonable life, so would not significantly impact on the use of the surf zone.

Based on Clause 13(2) of SEPP Coastal, "development consent must not be granted to development on land to which this clause applies unless the consent authority is satisfied that:

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

The proposed development has been designed and sited to avoid any potential adverse impacts referred to in Clause 13(1).

8.3.3 Clause 14

Based on Clause 14(1) of SEPP Coastal, "development consent must not be granted to development on land that is within the coastal use area unless the consent authority:

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

With regard to Clause (a)(i), the proposed development will not affect public beach access.

Clauses (a)(ii), a(iii), a(iv) and a(v) are not coastal engineering matters so are not considered herein.

With regard to (b), the proposed development has been designed and sited to avoid any potential adverse impacts referred to in Clause 14(1) for the matters considered herein.

Clause (c) is not a coastal engineering matter so is not considered herein.

8.3.4 Clause 15

Based on Clause 15 of SEPP Coastal, "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 building is unlikely to have a significant impact on coastal hazards or increase the risk of coastal hazards in relation to any other land, as it is built over the same north-south and seaward extent as the existing development, and at an acceptably low risk of being damaged by coastal erosion/recession over a reasonable life.

8.3.5 Clause 16

Based on Clause 16 of SEPP Coastal, "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 site.

8.3.6 Synthesis

The proposed development satisfies Clause 13, 14, 15 and 16 of *State Environmental Planning Policy (Coastal Management) 2018* for the matters considered herein.

8.4 *Pittwater Local Environmental Plan 2014*

Clause 7.5 of *Pittwater Local Environmental Plan 2014* (LEP 2014) does not strictly apply at the subject site, as it is not identified as a "Coastal erosion / wave inundation" area on the Coastal Risk Planning Map (Sheet CHZ_018). However, for consistency with coastal planning for adjacent private development, Clause 7.5 of LEP 2014 has been considered herein.

Based on Clause 7.5(3) of LEP 2014, "development consent must not be granted to development on land to which this clause applies unless the consent authority is satisfied that the development:

(a) is not likely to cause detrimental increases in coastal risks to other development or properties, and
(b) is not likely to alter coastal processes and the impacts of coastal hazards to the detriment of the environment, and
(c) incorporates appropriate measures to manage risk to life from coastal risks, and
(d) is likely to avoid or minimise adverse effects from the impact of coastal processes and the exposure to coastal hazards, particularly if the development is located seaward of the immediate hazard line, and
(e) provides for the relocation, modification or removal of the development to adapt to the impact of coastal processes and coastal hazards, and
(f) has regard to the impacts of sea level rise, and
(g) will have an acceptable level of risk to both property and life, in relation to all identifiable coastline hazards".

With regard to (a) and (b), the proposed development would not increase coastal risks nor alter coastal processes and the impacts of coastal hazards over its design life, as it is built over the same north-south and seaward extent as the existing development, and at an acceptably low risk of being damaged by coastal erosion/recession over a reasonable life.

With regard to (c) and (g), with a buried seawall constructed as described in a separate report of Horton Coastal Engineering, the proposed development would be at an acceptably low risk of damage from coastal erosion/recession over its design life. Furthermore, if a suitable mix of construction and operational measures as listed in Section 6 are adopted, in consultation with a coastal engineer, the proposed development would be at an acceptably low risk of damage from oceanic inundation over its design life.

Risk to life related to redevelopment of Newport SLSC is considered to be acceptably low as:

- the proposed clubhouse is to be located landward of a buried seawall, meaning that the risk of undermining of the building is acceptably low over the design life;
- coastal storms (large waves and elevated water levels) are generally foreseeable at least 24 hours in advance, with warnings issued by the Bureau of Meteorology and various forecast websites available for use;
- a large component of elevated water levels is astronomical tide, which can be accurately predicted into the future;
- inundation would generally be expected to be greatest for a few hours near the peak of the tide;
- the progress of erosion on a beach, leading to wave runup propagating further landward, is visible and perceptible, and would not generally be expected to proceed undetected to damage development;
- it is highly unlikely that someone would be occupying the SLSC and would be unaware (or would not have been made aware) that the clubhouse was at imminent threat of damage from inundation;
- the State Emergency Service (SES), if mobilised, has powers to warn and evacuate occupants if required (as does NSW Police); and
- Council could request the SES taking on a Combat Agency role if an actual emergency was occurring and it had not already been mobilised.

These factors mean that the clubhouse would have a low probability of occupancy and/or loss of life during an actual storm event that could threaten the development, and hence have a low risk to life.

With regard to (d), locating the proposed development landward of a buried seawall, and incorporating consideration of the measures listed in Section 6 to reduce the risk of damage from oceanic inundation, would minimise the adverse effects from the impact of coastal processes and the exposure to coastal hazards for the proposed development (which is located seaward of the Immediate Hazard Line with no seawall in place). Given that the proposed development is at an acceptably low risk of damage for a reasonable life, (e) is not necessary.

With regard to (f), sea level rise has been considered herein with regard to the recommendations on coastal inundation in Section 6.

The proposed development thus satisfies Clause 7.5 of LEP 2014.

8.5 Coastal Management Strategy, Warringah Shire

In 1981, a working party was established comprising Warringah Council and Public Works Department (PWD) staff at that time, with the aim of integrating Council's management and planning with coastal engineering advice to produce an overall strategy for coordination of beach reserves management and identification of areas of the coastal zone that required specific development controls (PWD, 1985).

This resulted in the completion of an investigation by PWD (1985), entitled "Coastal Management Strategy, Warringah Shire" in which coastline management strategies were developed for the beaches and headland areas of the entire Warringah Shire Council Local Government Area (LGA), which extended from Freshwater to Palm Beach at that time (thus covering the former Pittwater and Warringah LGA's).

For the subject site, PWD (1985) noted that there should be consideration of relocating the clubhouse further landward when it is to be replaced, renovated or extended. However, given the necessity to retain the building in its current location (as it has heritage status and surf lifesaving functions), and given the invasiveness and cost in attempting to retrofit deep foundation piles to the existing SLSC building, relocation was considered to be unacceptable to Council and could not therefore be adopted.

9. CONCLUSIONS

It is proposed to undertake alterations and additions at Newport SLSC. To provide protection such that the redeveloped SLSC would be at an acceptably low risk from undermining due to coastal erosion/recession, a buried seawall has been proposed, as considered in a separate report by Horton Coastal Engineering. For the purpose of the clubhouse assessment herein, it has been assumed that a seawall is in place, with a minimum design life of 60 years.

With a buried seawall constructed as described in the separate report, the proposed development would be at an acceptably low risk of damage from coastal erosion/recession over its design life. Furthermore, if a suitable mix of construction and operational measures as listed in Section 6 are adopted, in consultation with a coastal engineer, the proposed development would be at an acceptably low risk of damage from oceanic inundation over its design life.

Although additional tasks may be undertaken as part of detailed design, this does not call the feasibility of the proposed development into question. It is considered that the work of WRL, James Taylor & Associates and Partridge Structural described herein shows that a suitable mix of practical measures would be able to be formulated to reduce the wave forces on the SLSC building to acceptable levels, and to provide remedial measures to support the seaward face of the existing building against wave forces (if required). The project as proposed is feasible.

The proposed development satisfies the coastal engineering matters in Chapter B3.3 of *Pittwater 21 Development Control Plan*, the *Coastline Risk Management Policy for Development in Pittwater*, *State Environmental Planning Policy (Coastal Management) 2018*, Clause 7.5 of *Pittwater Local Environmental Plan 2014*, and the "Coastal Management Strategy, Warringah Shire" prepared in 1985, as has been outlined.

As advised by Council, Newport SLSC is not subject to catchment and overland flow flooding controls. Oceanic inundation is a more significant risk and extends higher than catchment and overland flow flooding for a given probability, and this oceanic inundation has been addressed herein.

10. REFERENCES

Horton Coastal Engineering (2018), *Initial Coastal Engineering Advice on Newport SLSC Redevelopment*, 14 August

Horton Coastal Engineering (2020), *Assessment of Options for Redevelopment of Newport SLSC, with Updated Consideration of Risk from Coastal Erosion/Recession*, Issue A, 17 February

JK Geotechnics (2020), *Report to Horton Coastal Engineering Pty Ltd on Geotechnical Investigation for Proposed Alterations and Additions at Newport Surf Life Saving Club, 394 Barrenjoey Road, Newport, NSW*, 9 November, Ref: 32537RErpt Rev1

Nielsen, AF; Lord, DB and HG Poulos (1992), "Dune Stability Considerations for Building Foundations", *Australian Civil Engineering Transactions*, Institution of Engineers Australia, Volume CE34, No. 2, June, pp. 167-173

Public Works Department [PWD] (1985), "Coastal Management Strategy, Warringah Shire, Report to Working Party", *PWD Report 85016*, June, prepared by AD Gordon, JG Hoffman and MT Kelly, for Warringah Shire Council

APPENDIX A: PEER REVIEW OF HORTON COASTAL ENGINEERING REPORTS BY UNSW WATER RESEARCH LABORATORY

14 May 2021

WRL Ref: WRL2021004 JTC LR20210514

Water Research Laboratory
School of Civil and Environmental Engineering

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Dear Bernard,

DRAFT Newport SLSC coastal hazard peer review

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney is pleased to provide an expert peer review of the following document:

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

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

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

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

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

The review has been undertaken by WRL's Principal Coastal Engineer James Carley. James has over 28 years' experience in coastal engineering, serves on Engineers Australia's NSW Coasts, Ocean and Port Engineering Panel (COPEP), and is chair of Engineers Australia's National Committee on Coastal and Ocean Engineering (NCCOE). James is familiar with the site and is a long term surfer and surf life saver.

2. Peer review summary

The Horton coastal engineering reports are generally of a high professional standard. Due to the uniqueness and non-standard nature of such studies, there will always be differences between the work of different practitioners.

A summary of the key design parameters adopted by Horton and WRL's concurrence or otherwise is provided in Table 1. More detailed review comments are provided in Section 3 and 4.

For the quantitative parameters derived in Horton, some values are accepted by WRL, some are more conservative while others are less conservative than would be adopted by WRL. The net effect is that the differences may balance out.

However, some parameters have not been quantified in Horton and have been deferred until detailed design. This may be normal practice, but in the case of Newport SLSC, the quantification may affect the overall viability or geometry of the project, so additional quantification is recommended.

While the decision to retain the existing clubhouse and add a new portion on the ocean front appears to have been made within the project planning process, the philosophy adopted at Freshwater Beach was to construct the new building landward of the old. If the present Newport clubhouse is to be protected to an engineering degree of certainty over 60 years, a seawall will be required.

There are numerous examples where seawalls have survived but infrastructure behind them has been damaged through wave overtopping. Examples of buildings which were damaged/destroyed behind undamaged seawalls occurred in the June 2016 storm include Dee Why (café), Fairy Bower (toilet block and cafe) and Coogee (SLSC clubhouse).

Illustrations of the application of recommendations in this letter (for other sites) are shown in Appendix A.

Table 1: Summary of WRL concurrence

Parameter	Value adopted by Horton	Concurrence or suggested alternative value from WRL	Comment and/or recommendation
Structure design life	60 years	Agree	
Design ARI	500 to 2000 year ARI suggested, but 100 year ARI used	500 ARI indicated	500 to 2000 year ARI suggested, but 100 year ARI used
Extreme water levels	100 year ARI of 1.44 m AHD	Agree but other ARIs needed	Derive other ARI water levels
Extreme offshore wave heights	Hs = 9.5 m for 1 hour Hs = 8.7 m for 6 hour	Agree with values, but these are for S to SE direction. Additional directions and transformation could be considered	Derive other ARI waves and directions, and consider wave transformation
Wave transformation to shore	South to south-east direction considered	Wave transformation modelling recommended and consideration of other directions	Wave transformation modelling is likely to reduce nearshore wave heights and wave setup
Sea level rise	0.44 m by 2080	Acceptable	The sea level rise adopted would be at the end of the design life, so provided a reasonable value is adopted, it is not critical
Recession due to sea level rise	Bruun Factor of 31 13.6 m recession by 2080	Bruun Factor accepted Recession by 2080 can be reduced by 1.9 to 3.8 m	Future recession can be discounted by sea level rise over historic monitoring period
Design scour level	-1 to -2 m AHD	Acceptable as initial estimate, but additional techniques are recommended	Additional techniques as outlined in Carley et al (2015) are recommended to be applied
Local wave height at structure	Plunge distance = 10 m No local wave height or wave setup stated	Agree with plunge distance. Local wave height and wave setup calculations required	Local wave height and wave setup calculations required
Wave forces	Addressed qualitatively only	Initial desktop assessment recommended, with physical model at some point in the design process	This would often be deferred until detailed design, but in this case it may affect the viability of the project
Wave runup and overtopping	Addressed predominantly qualitatively	Additional quantitative techniques should be applied – initially desktop, and later physical modelling	The present design geometry may be too low to act as a wave return wall – additional calculations are recommended
Seawall end effect	No long term impacts, but addressed qualitatively only	Agree with no long term impacts, but short term impacts need to be assessed	Apply a quantitative technique for short term impacts

3. Detailed review of Horton (2020a), "*Coastal Engineering Report and Statement of Environmental Effects for Buried Coastal Protection Works at Newport SLSC*"

Horton (2020a), Section 1, Page 1, Paragraph 4
Given the age of the present clubhouse, advice from a structural engineer and/or piling expert should be sought regarding the feasibility and risk of piling near and installing ground anchors beneath the building.

Horton (2020a), Section 2, Page 3, Paragraph 3
WRL concurs that the existing rock revetment would provide a degree of protection to the existing clubhouse, but this would not be to a certifiable level of engineering certainty over a 60 year design life.

Horton (2020a), Section 2, Page 3, Paragraph 4
The analysis of long term change, which found that there is not a detectable trend is accepted by WRL. We note that during the analysis period of long term change (1941 or 1951 to 2018), mean sea level for Sydney has increased by 1 to 2 mm per year (Watson, 2020). Figure 5 of Watson (2020) is reproduced as Figure 1 in this WRL letter.

That is, Newport Beach has been broadly stable with sea level rise of 1 to 2 mm per year, therefore the predicted future response (recession) to sea level rise could be discounted by the quantum of sea level rise which occurred but produced no recession. Neither the Horton reports nor this WRL review are a detailed processes study, but an onshore or alongshore feed of sand has been postulated at other locations, noting that sea level rise may outpace this feed in the future.

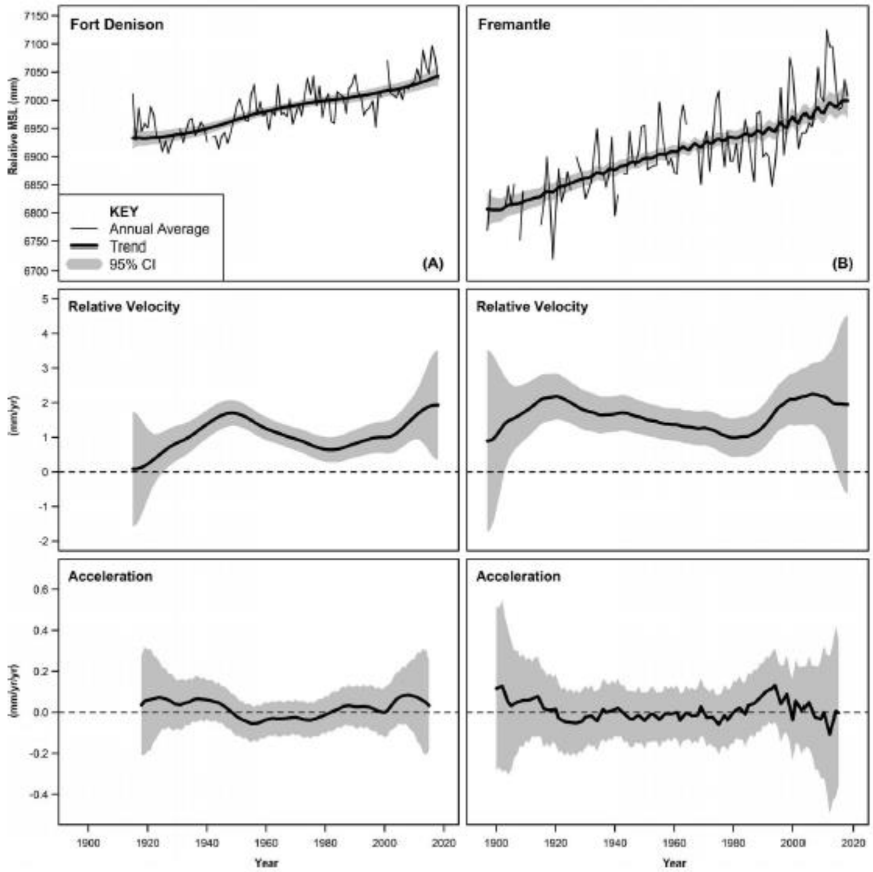


Figure 5. "Relative" mean sea level (trend), velocity, and acceleration for Fort Denison and Fremantle. The scales associated with each of the three panel charts for the respective tide gauge records are equivalent for direct comparison between records. "Relative" mean sea level (top panels) is based on revised local reference datum (PSMSL, 2019) with +200 mm offset applied to Fremantle to align graphic with the Fort Denison panel. See Figure 1 and Table 1 for station details.

Figure 1: Observed sea level rise at Fort Denison (Watson, 2020)

Horton (2020a), Section 3, Page 4, Paragraph 7 and Figure 1; and Section 3, Page 7, Paragraphs 1 and 2

Whether the proposed protection works extend to protect the Norfolk Island pine trees at each end of the clubhouse is a decision beyond coastal engineering. While the cost of this is somewhat addressed, the potential additional seawall end effect of this should be presented. The installation of ground anchors in the vicinity of the trees also needs consideration.

Horton (2020a), Section 3, Page 7, Paragraph 6

The impracticality of disabled beach access at this location is accepted by WRL, but additional reconciliation is required between the stair gradient and a safe gradient for a ramp.

Horton (2020a), Section 3, Page 8, Paragraph 1

This paragraph refers to a separate Horton report regarding the risk of inundation damage. This component is reviewed separately below in this WRL letter.

Horton (2020a), Section 4, Page 9, Paragraph 6

WRL concurs that if the present clubhouse is to be retained for approximately 60 years, some form of seawall is required to provide an engineering level of certainty to the clubhouse. It should be noted that the seawall would protect the existing clubhouse from erosion and undermining, however, may not protect the clubhouse from wave overtopping damage. It should also be realised that this protection afforded may only be for up to a certain quantum of sea level rise over the design life, beyond which protection of the club house may no longer be feasible.

Horton (2020a), Section 5.1 and 5.2, Page 11
The design life stated is accepted by WRL, noting that the standards quoted are more within the expertise of structural engineering.

Horton (2020a), Section 5.3, Page 12, Paragraph 2
The lowest profile recorded in the photogrammetry from 1941 to 2020 was 1974. Horton notes that this elevation "would have been limited by the emergency placement of rock boulders at that time".

From the NSW beach profile database (http://www.nswbpd.wrl.unsw.edu.au/photogrammetry/nsw/) and Foster et al (1975), the following information is added by WRL:

- A level of 3.74 m AHD was measured just seaward of the clubhouse on 19 June 1974
- There is ample evidence of the construction of an ad hoc rock revetment and beach scraping following the 1974 storms
- From the test pit data, the rock revetment is indicated to be founded at 2.5 m AHD or lower
- The May 1974 storm extended from about 25 to 28 May 1974, with peaks on 25 and 26 May 1974
- The June 1974 storm extended from about 3 to 15 June 1974, with a peak on 13 June 1974

Given the above dates, revetment construction and beach scraping, it is almost certain that the sand level fronting the clubhouse at some time in May or June 1974 was lower than 3.74 m AHD, and possibly lower than 2.5 m AHD, but the actual minimum level is unknown.

Foster et al also documented the following damage at Newport:

> *"Severe erosion along northern end of beach. Waves lapped foundation of club house, boatshed 50 per cent destroyed, pavement washed away. Homes at southern end threatened and pines lost. Dunes overtopped causing back flooding. Some rock protection placed after storms offshore sand bed completely removed exposing extensive areas of old lagoon deposits and erosion of clay beds is still occurring. Swimming club house at rock pool completely demolished."*

Horton (2020a), Section 5.4, Page 13
The adopted "Bruun Factor" of 31 and its derivation is accepted.

The sea level rise of 0.44 m by 2080 is within the plausible range. As this is at the end of the initial design life, excessive rumination regarding the actual design sea level rise is unwarranted.

The estimated sea level rise recession of 13.6 m by 2080 may be an overestimate. This is because the lack of underlying recession over the monitoring period, despite 1 to 2 mm per year of sea level rise, has not been considered. Discounting for the sea level rise that has already occurred (and resulted in no recession) would reduce the recession by 1.9 to 3.8 m.

Horton (2020a), Section 5.7, Page 16
The probability (500 to 2000 year ARI) values canvassed and adopted are accepted by WRL, but there is little evidence of the adopted value being used in design, apart from a design scour level

being described as “barely credible” and equated to 2000 year ARI. However, we note that the adopted value is reasonably plausible. Most design parameters within Horton (2020a) have been considered at 100 year ARI.

Horton (2020a), Section 5.8.3, Page 16 and 17

While there are many opinions and scenarios for sea level rise, the value of 0.44 m by 2080 within the plausible range.

Horton (2020a), Section 5.8.4 and 5.8.5, Page 17

The calculations regarding design water level and plunge distance are accepted.

The water levels adopted by Horton are 100 year ARI, versus longer ARIs required by some design standards.

The adoption of an eroded bed level of -1 m AHD for calculating the plunge length (with a deeper scour hole fronting the wall) matches some field observations of WRL engineers, but has limited precedent and has not attached a probability to the level.

Horton (2020a), Section 5.8.6, Page 17

There are no explicit design standards relating to the use of either 1 hour or 6 hour duration wave heights. Different practitioners favour either option. The difference in wave height between 1 hour and 6 hour durations may be important for offshore structures, but for structures well inside the surf zone, the offshore height only influences the nearshore wave setup (see below).

The wave heights adopted by Horton are 100 year ARI, versus longer ARIs required by some design standards.

The wave heights quoted by Horton are offshore deep water waves from a south to south-east direction. They are derived from credible studies. No attempt has been made by Horton to consider refraction of these waves into Newport which will reduce the height of south to south-east waves, or alternatively consider smaller design wavers from the east. However, as above, for structures well inside the surf zone, the offshore height only influences the nearshore wave setup.

Horton (2020a), Section 5.8.7, Page 18

Horton lists credible methods for estimating the design wave height at the structure and highlights the potential for large forces. However, the application of these has been deferred until detailed design.

It is accepted by WRL that some parameters are best calculated after approval and within detailed design, however, we recommend that some initial desktop estimates and opinions be developed, as these could affect the feasibility of the project.

Furthermore, we recommend that estimates of wave overtopping and wave forces on the clubhouse be undertaken, as a scenario could arise such that the seawall prevents the clubhouse being undermined, but the building is damaged or destroyed through wave overtopping. Examples of buildings which were damaged/destroyed behind undamaged seawalls occurred in the June 2016 storm include Dee Why (café), Fairy Bower (toilet block and cafe) and Coogee (SLSC clubhouse).

These overtopping calculations may also result in design changes to the wall crest and/or steps.

Horton (2020a), Section 6.1, 6.2, 6.3, Page 19
These are planning matters outside of WRL's expertise.

Horton (2020a), Section 6.4, Page 19, 20
It is noted in Horton that the PWD (1985) "Coastal Management Strategy, Warringah Shire" was to consider relocating the clubhouse further landward. WRL notes that this strategy was adopted at Freshwater Beach.

As stated elsewhere in this WRL letter, we concur that if the existing clubhouse is to be retained for up to 60 years with an engineering degree of certainty, a seawall engineered to contemporary standards is required.

Horton (2020a), Section 6.6.2 Item (a), Page 21
WRL accepts that the works are likely to have minor end effects due to the substantial sand buffer fronting them. Based on the Carley et al (2013) work cited by Horton, comparable sites such as Curl Curl and Cronulla indicate that there will be no long term end effect, but this does not preclude short term end effect erosion. Nevertheless, an attempt should be made to convey this and identify any unprotected assets affected. Methods to estimate end effects are provided in Horton but not applied.

Horton (2020a), Section 6.7 to 6.10, Page 22 to 29
These sections are primarily policy and/or interpretation of legislation, so have not been reviewed by WRL.

4. Horton (2020c), *"Coastal Engineering and Flooding Advice for Newport SLSC Clubhouse Redevelopment"*

Most coastal engineering components of Horton (2020c) are reproduced in Horton (2020a).

Horton (2020c), Section 6
This provides a good discussion of measures to reduce the risk of inundation damage, but is predominantly qualitative. A generic wave runup level (7.2 m AHD) is cited, but there is little other quantification. Additional techniques, ranging from initial desktop estimates to physical modelling are recommended to be utilised, as this may affect the viability of the project.

5. Other comments relating to coastal engineering matters noted in public submissions

Detailed comments on all public submissions are beyond the scope of this WRL review. Brief responses to themes in the submissions relating to coastal engineering matters which were not addressed in the Horton reports are provided below.

Theme: The seawall will cause erosion
One of the world's most eminent coastal engineers, Professor Bob Dean, noted that: "seawalls don't cause erosion, erosion causes seawalls".

For beaches experiencing high rates of recession, a common response was to construct a seawall. This could lead to no beach being present seaward of the wall. However, there is no identified recession trend at Newport, with the seawall only being required to resist erosion in extreme storm events, with the beach recovering after these events.

A rock rubble seawall has existed on the Newport site since 1974. Seawalls coexist with many iconic beaches, including Noosa, most Gold Coast beaches, Manly and Bondi.

Theme: The seawall works will adversely impact the surrounding surf breaks
Large breakwater or groyne structures may alter surfing conditions, however, the proposal at Newport is for a seawall at the back of the beach. The seawall works will only be impacted by waves on rare occasions with the coincidence of an eroded beach, high tides and large waves. There are scores of examples surf breaks (ranging from world class to locally significant) that coexist with seawalls. Some of these are deemed suitable for international surf contests such as the Gold Coast, Merewether and Manly.

Theme: Newport Reef protects the clubhouse from large waves
Commentary is made by WRL above that more comprehensive wave modelling could be undertaken to account for protection from the south. However, this would have only a minor impact on nearshore wave heights due to its impact on wave setup.

6. Conclusions

A summary is provided at the start of this letter. Please contact James Carley on +61414385053 should you require further information.

Yours sincerely,

Grantley Smith
Director, Industry Research

7. References

Foster, D., Gordon, A.D. and Lawson, N.V. (1975), "The Storms of May-June 1974", Proceedings of the 2nd Australian Conference on Coastal and Ocean Engineering, The Institution of Engineers Australia.

Watson, P.J. (2020), Updated Mean Sea-Level Analysis: Australia. *Journal of Coastal Research*, 36(5), pp.915-931.

8. APPENDIX A: Examples of studies recommended

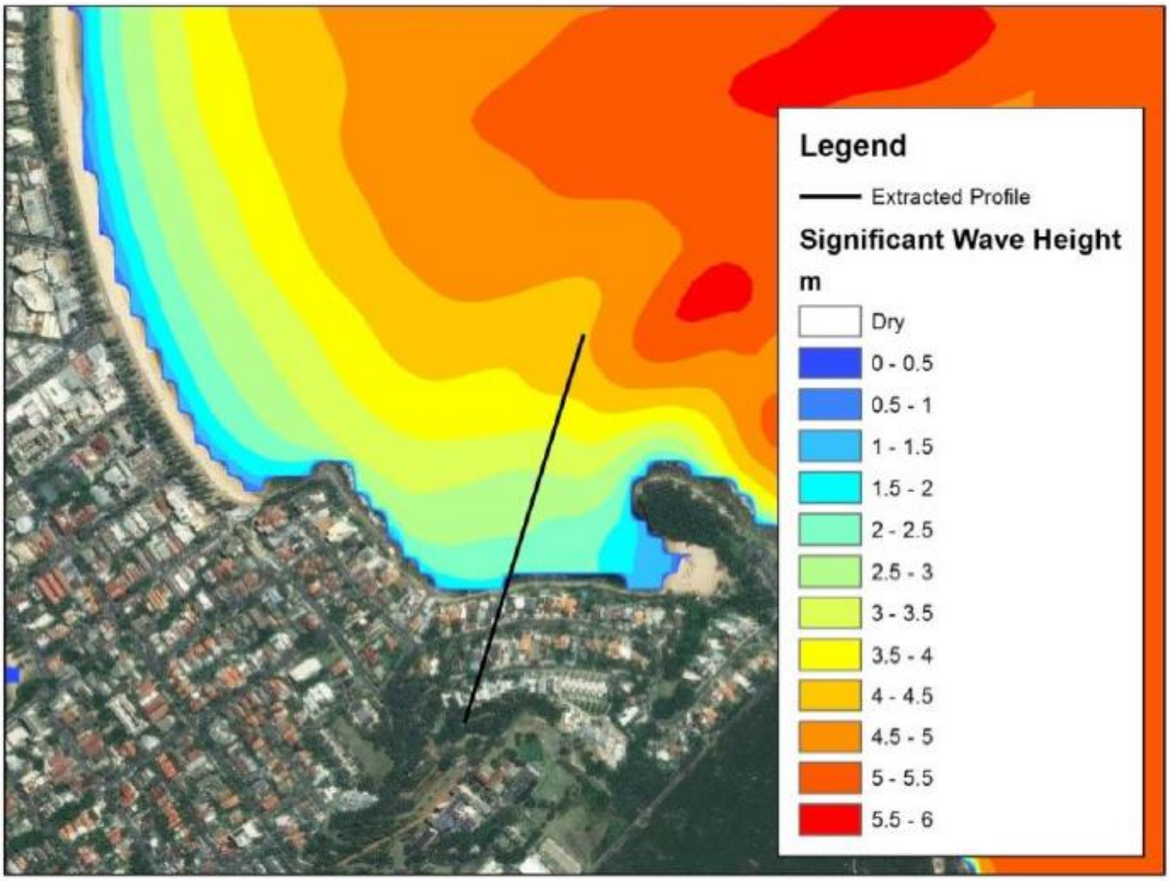


Figure 2: Example of wave transformation modelling

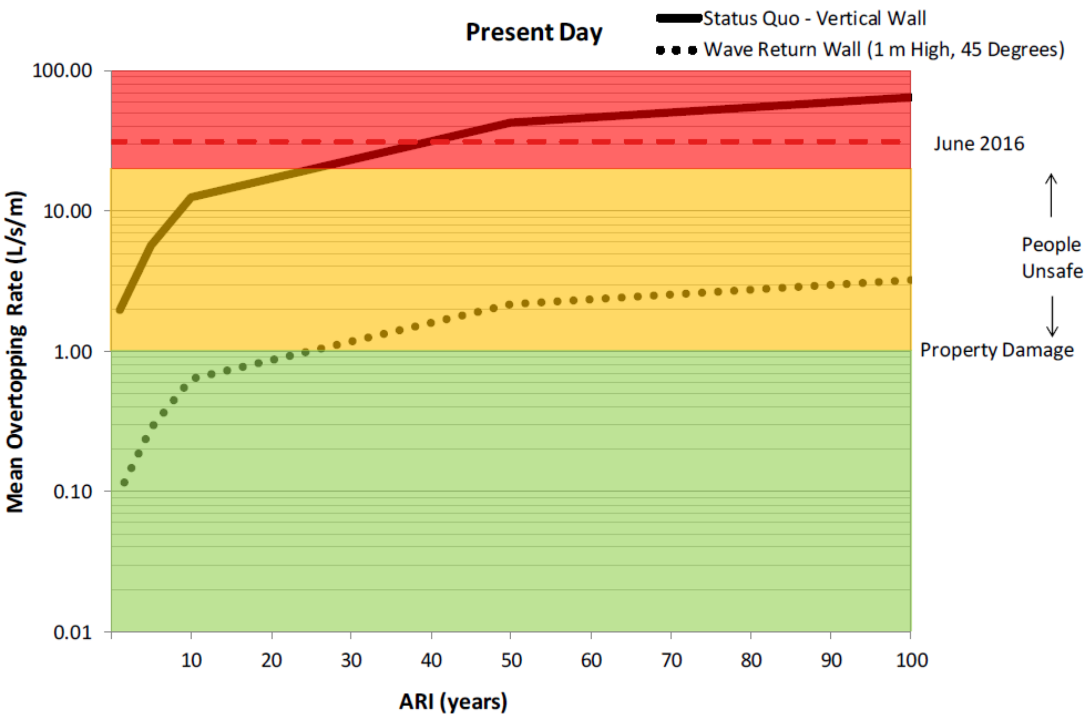


Figure 3: Example of wave overtopping and reduction due to wave return wall

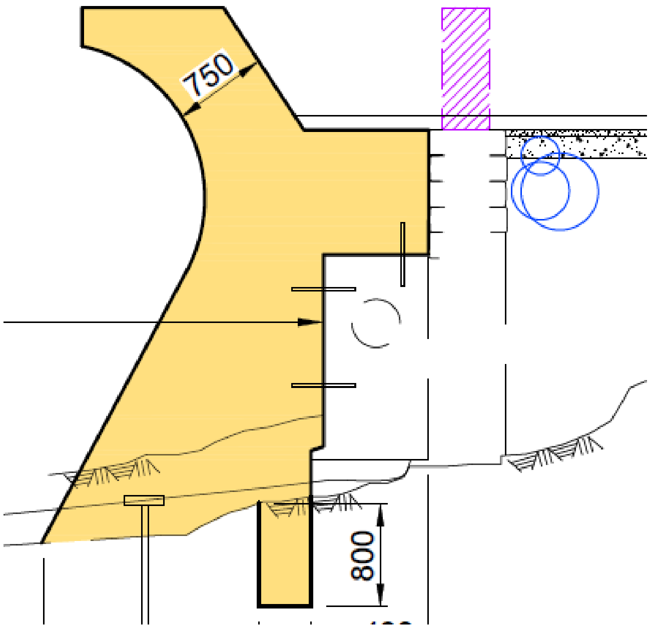


Figure 4: Example of wave return wall required to reduce wave overtopping to acceptable levels

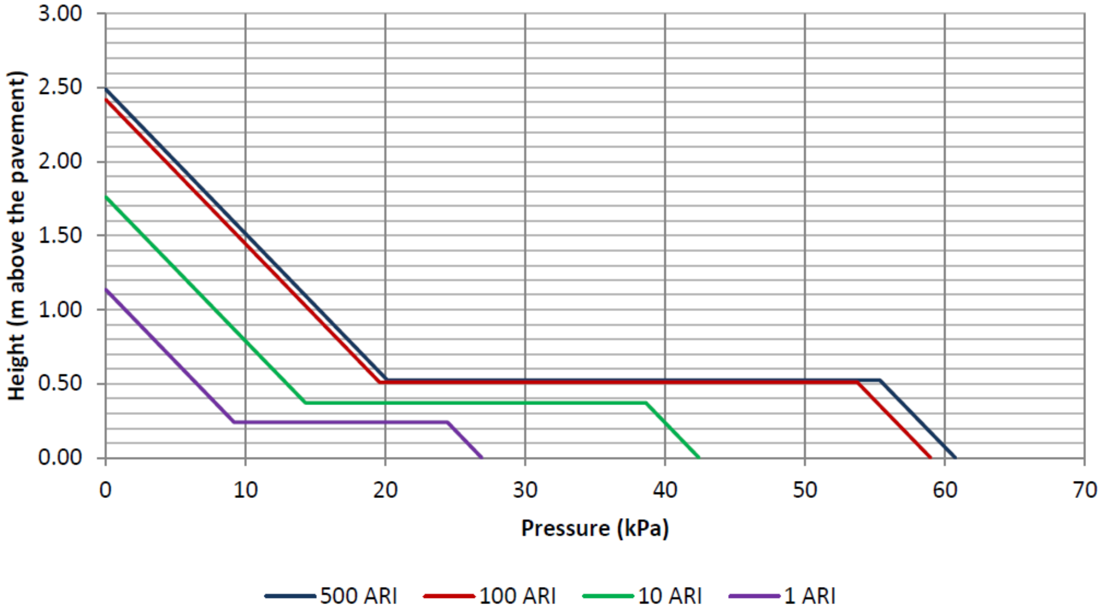
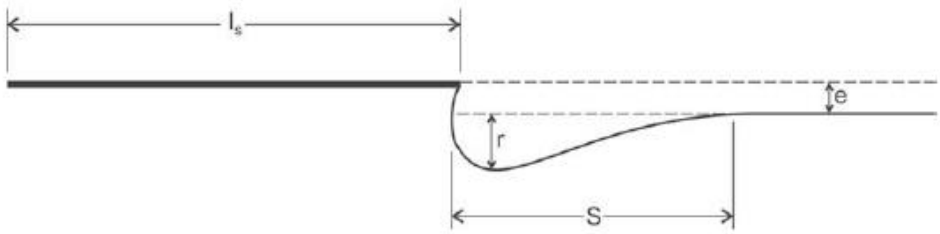


Figure 5: Example of calculated wave forces on building behind seawall



(Source: McDougal et al, 1987)

Gold Coast, 1967 (Source: Delft, 1970)

Figure 6: Example of seawall end effect and calculation method

Figure 7: Example of calculated seawall end effect (red line is erosion extent without seawall)

Figure 8: Example of physical model examining wave forces on a building

APPENDIX B: DESKTOP ASSESSMENT OF DESIGN PARAMETERS (SCOUR, WAVE RUNUP AND OVERTOPPING, WAVE LOADS) AND END EFFECTS BY UNSW WATER RESEARCH LABORATORY

8 July 2021

WRL Ref: WRL2021004 JTC FF LR20210708

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Water Research Laboratory

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Dear Bernard,

Newport SLSC coastal engineering advice

1. Introduction

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

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

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

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

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

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

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

2. Design Conditions

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

Table 1: Design offshore wave conditions

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

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

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

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

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

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

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

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

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

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

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

Table 3: Design offshore wave conditions

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

Notes

(1) SLR was set as 0.26 m for 2050

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

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

3.1 Measured data

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

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

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

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

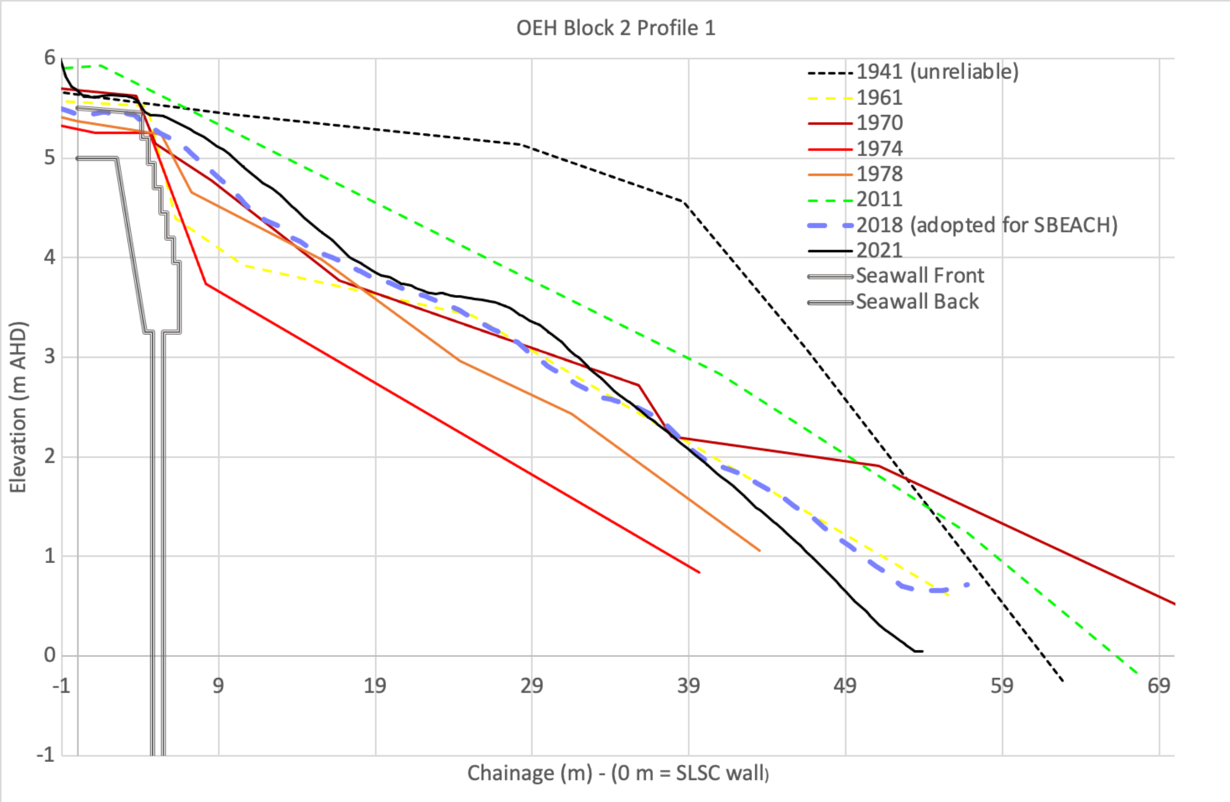


Figure 1: Measured profile data with proposed seawall superimposed

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

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

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

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

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

3.2 Modelling of erosion

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

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

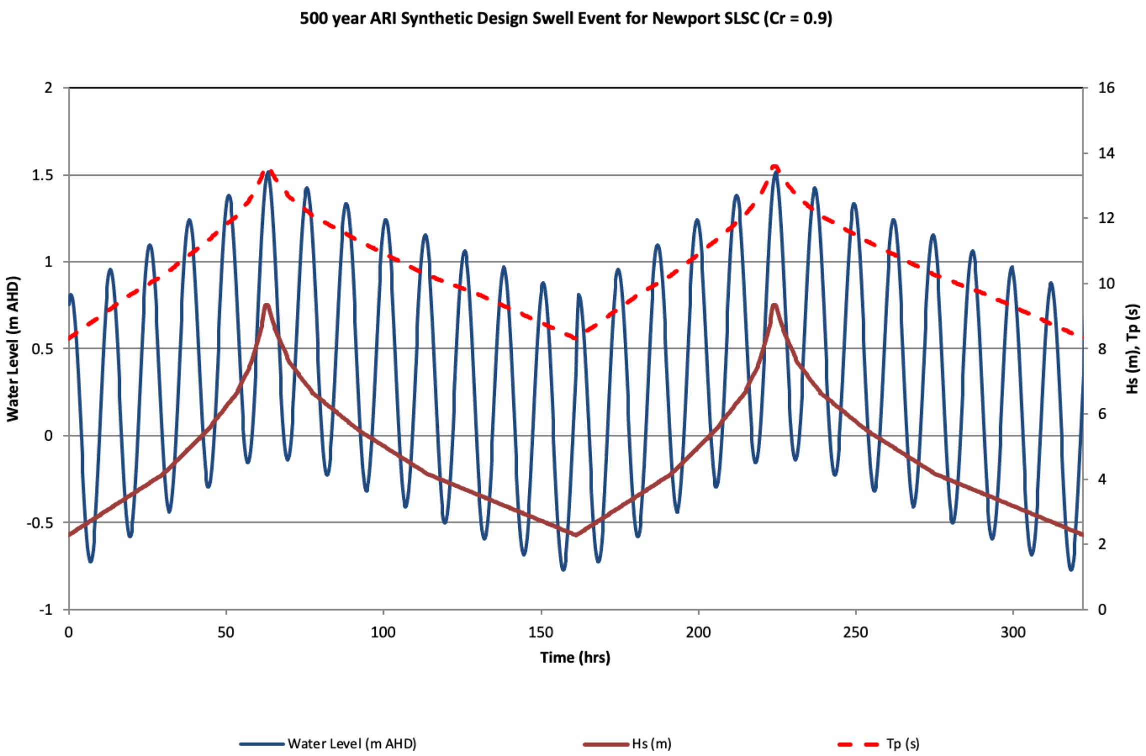


Figure 2: 500 year ARI synthetic design swell time series for Newport Beach (Note that only 2 consecutive storms were used for the study – i.e. erosion volumes derived after 322 hours)

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

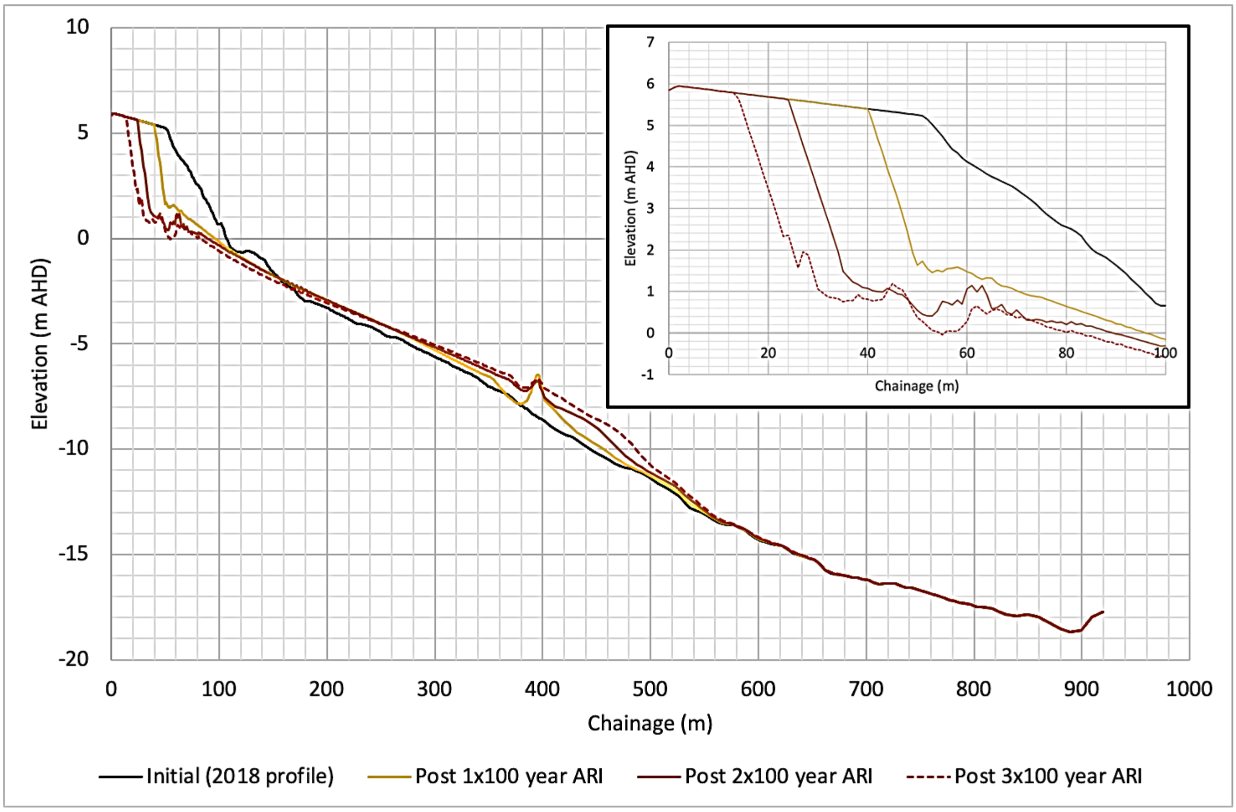


Figure 3: Evolution of beach profiles for consecutive storms in SBEACH with no seawall in place

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

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

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

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

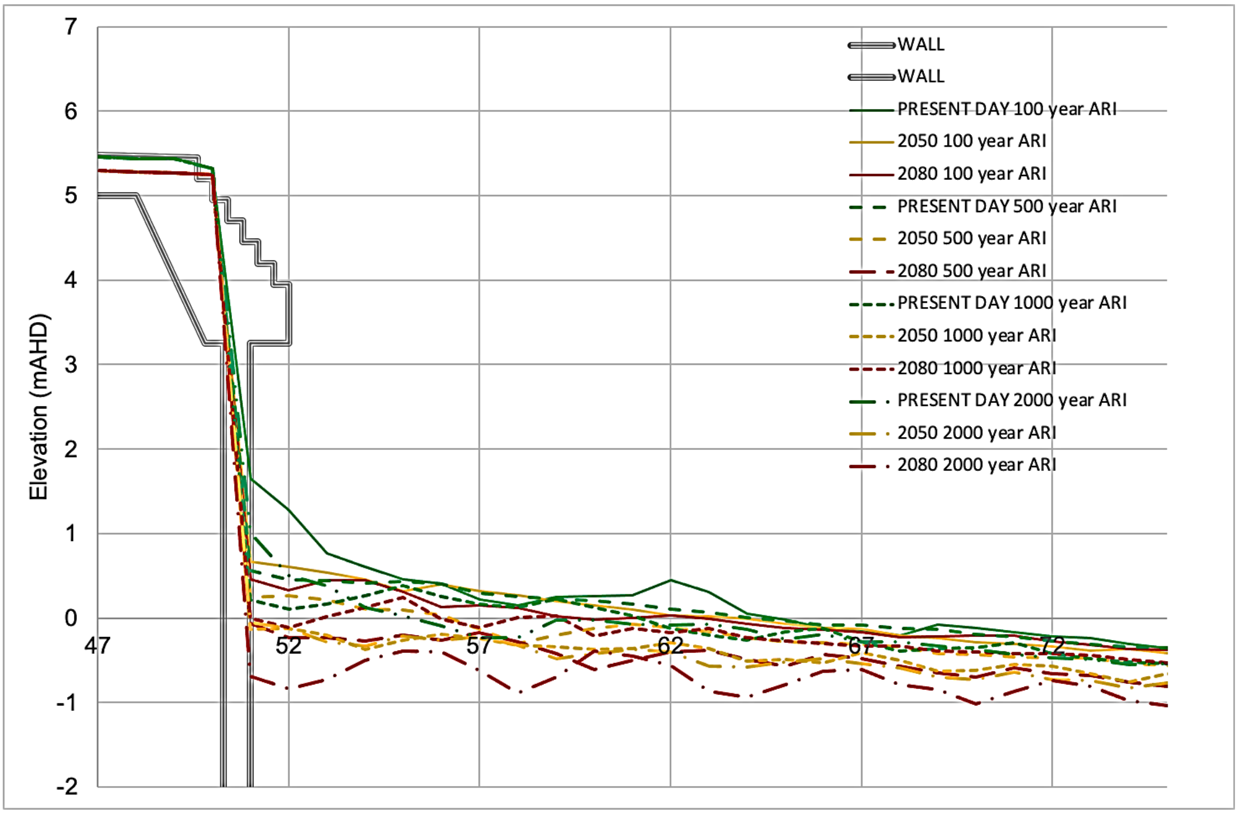


Figure 4: Evolution of beach profiles for consecutive storms in SBEACH with no seawall in place

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

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

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

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

4. Estimation of wave runup and overtopping

4.1 Overview

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

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

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

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

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

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

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

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

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

4.2 Accreted or average beach runup

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

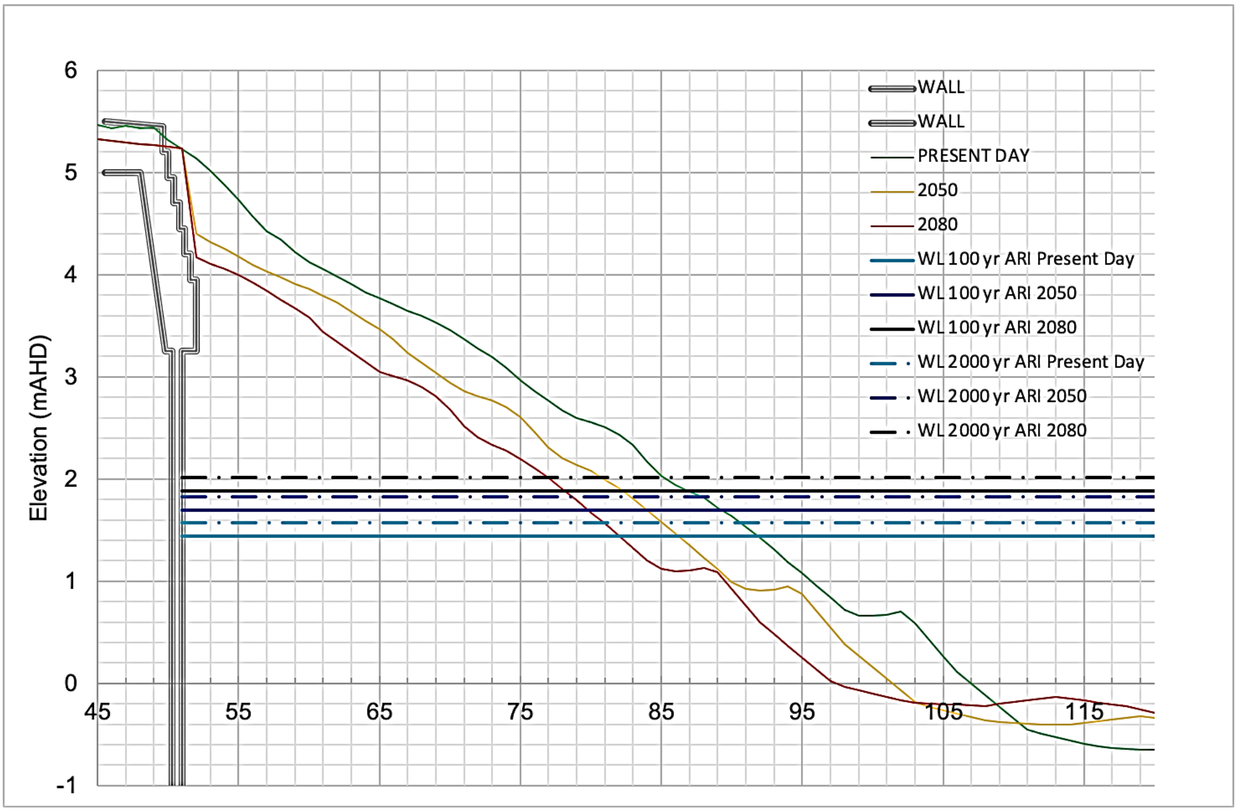


Figure 5: Water levels (no wave setup) for 100 and 2000 year ARI events for present day, 2050 and 2080 planning period

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

This storm had the following peak characteristics:

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

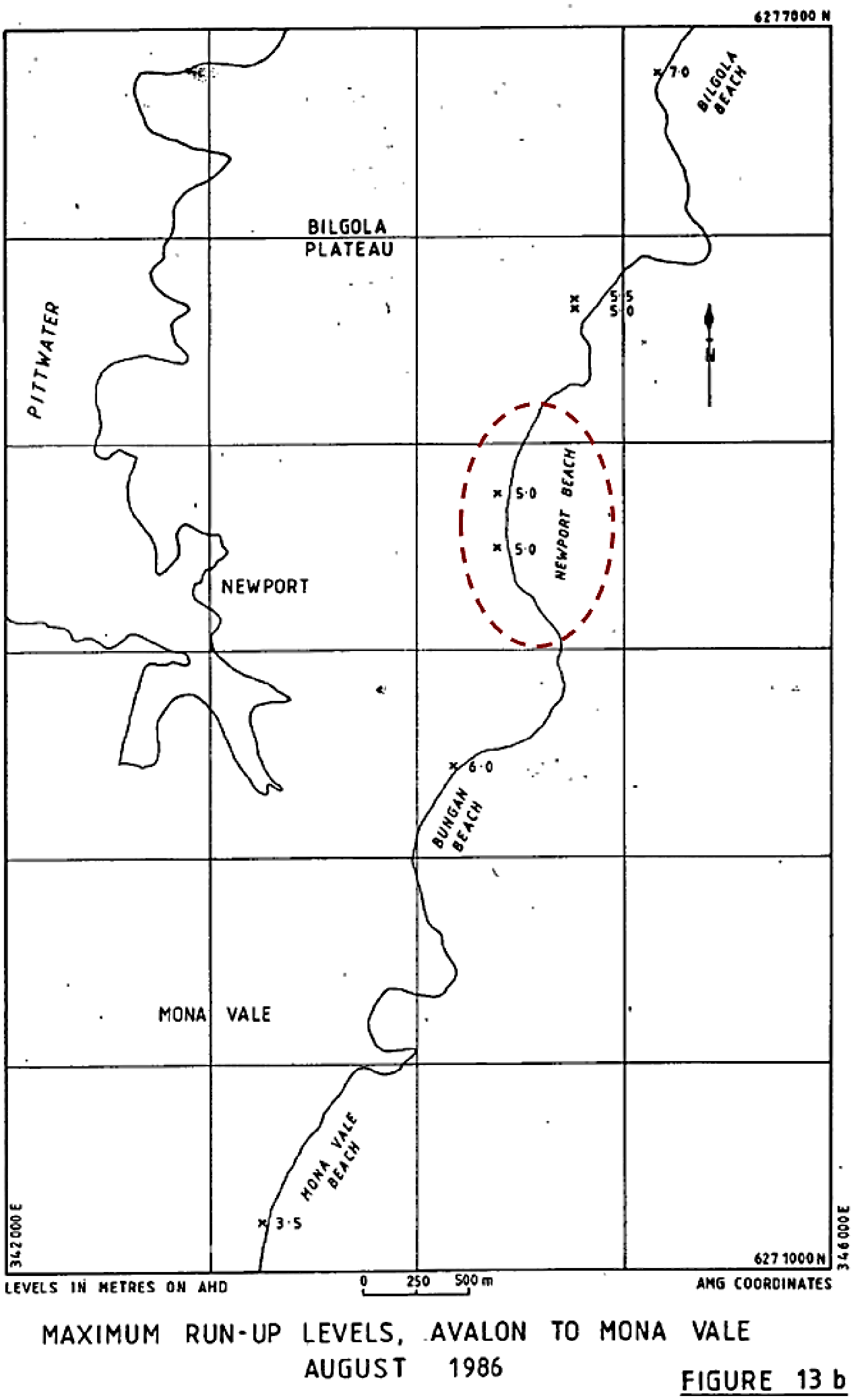


Figure 6: Observed wave runup levels after August 1986 storm based on debris lines [Source: Higgs and Nittim, 1988]

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

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

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

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

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

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

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

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

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

4.3 Wave runup and overtopping for eroded beach

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

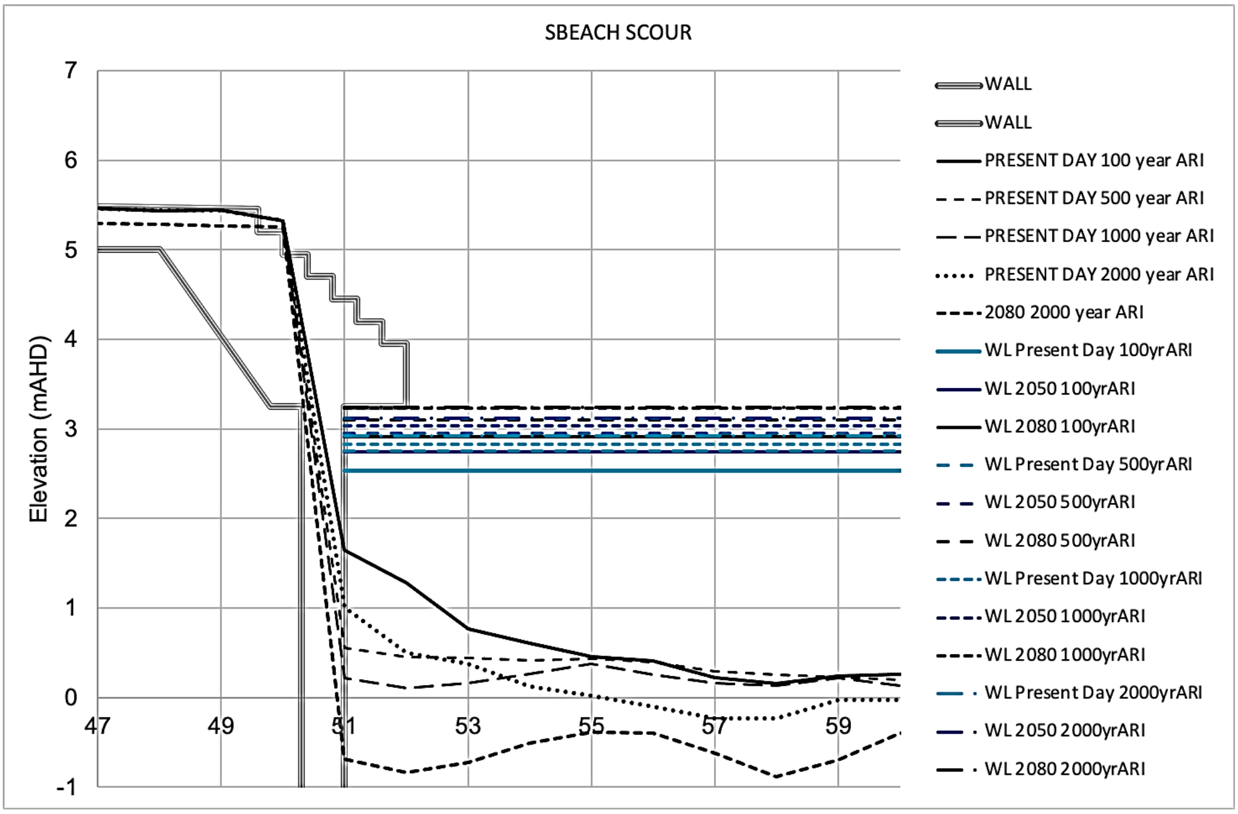


Figure 7: Calculated nearshore water levels (including local wave setup) and scoured levels in front of proposed seawall

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

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

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

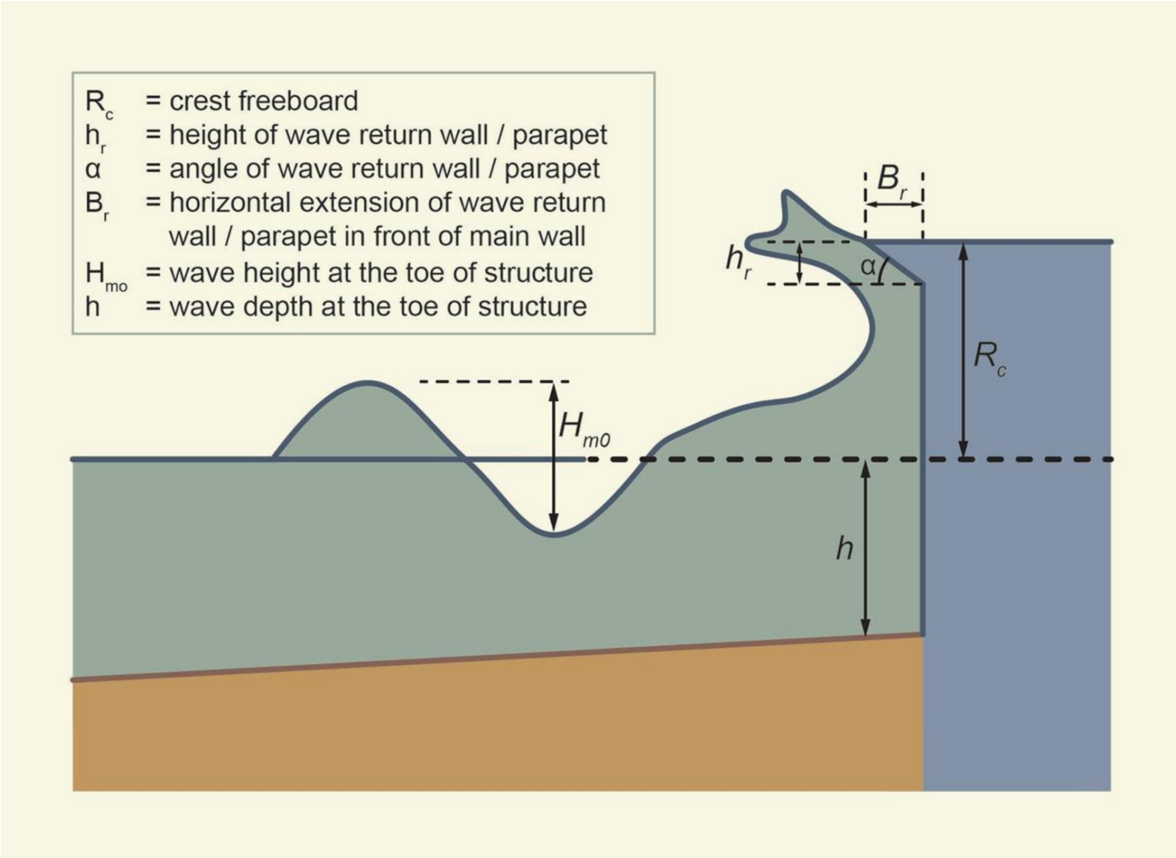


Figure 8: Parameters definitions for vertical seawall with return wall [Source: EurOtop, 2018]

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

Table 8: Overtopping discharges for proposed seawall with return wall

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

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

5. Wave loads due to overtopping

5.1 Overview

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

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

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

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

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

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

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

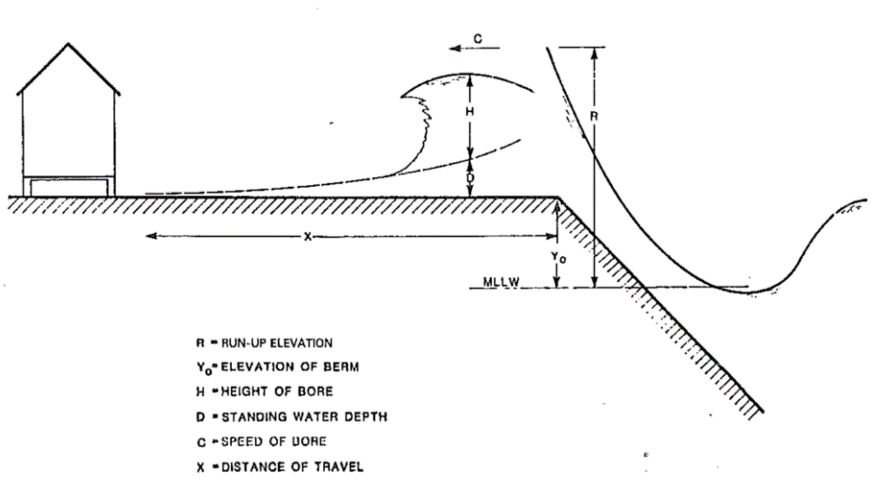


Figure 9: Definition of overtopping parameters [Source: Cox and Machemehl, 1986]

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

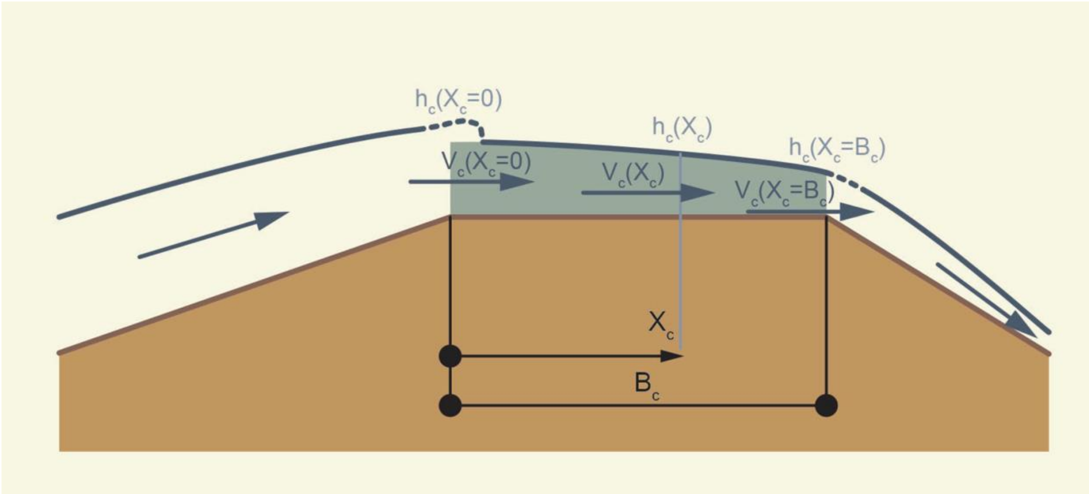


Figure 10: Sketch of overtopping flow parameters [Source: EurOtop, 2008]

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

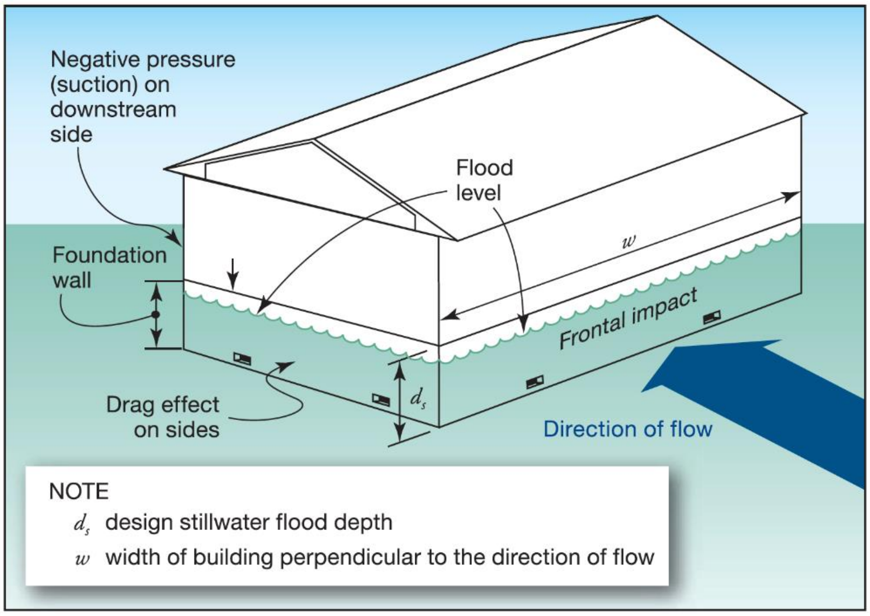


Figure 11: Hydrodynamic loads on a building [Source: FEMA, 2011]

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

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

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

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

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

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

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

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

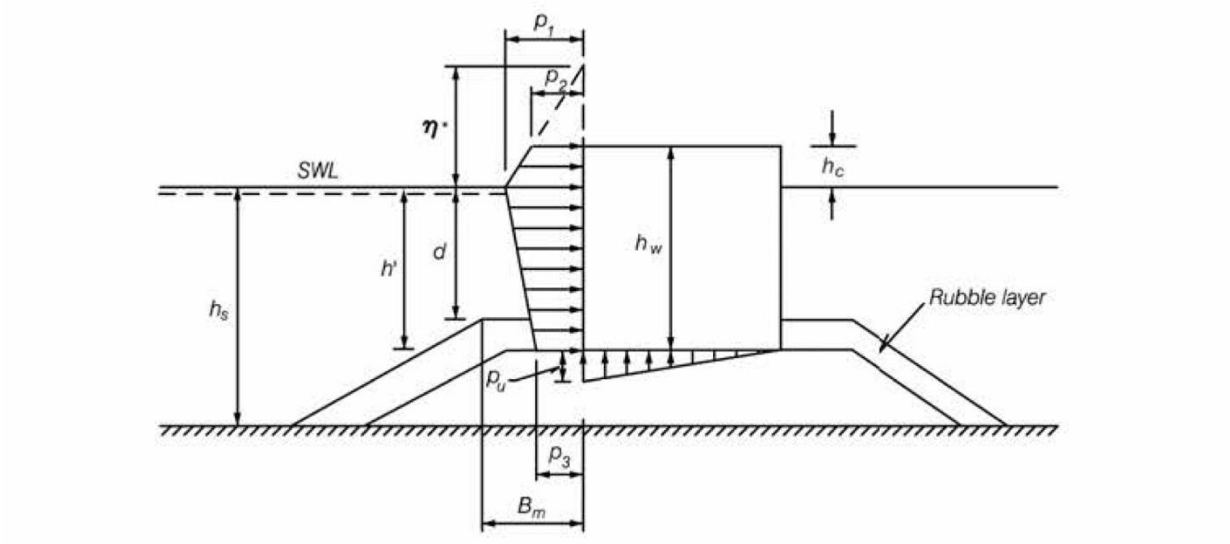


Figure 12: Hydrodynamic loads due to wave impact on a coastal structure [Source: CEM, 2011]

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

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

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

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

6. Review of available methods to reduce overtopping hazard

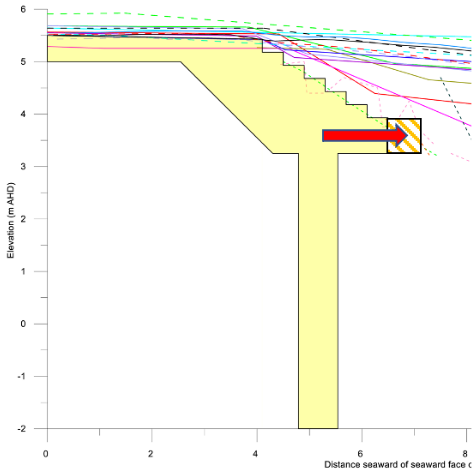
Should the wave overtopping or wave forces be deemed to be excessive, the following methods are available to reduce overtopping (Figure 13):

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

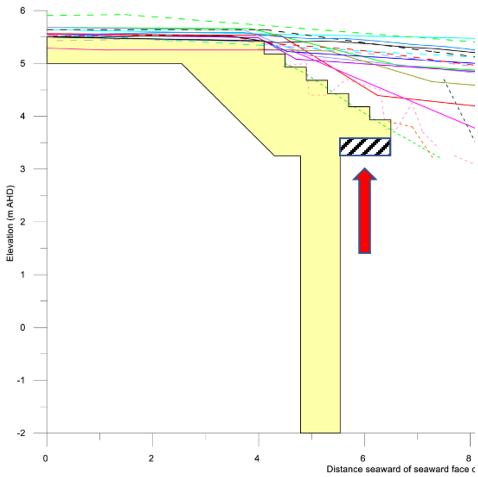
Additionally, the following short term management measures could be undertaken:

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

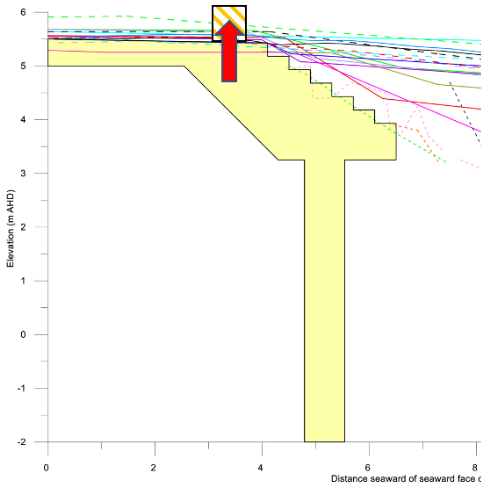
Additional calculations and/or later physical modelling may be required to quantify the benefit of each option.



(a) Extend projection of return wall



(b) Raise level of return wall



(b) Add return wall on crest

Figure 13: Options for reducing wave overtopping

7. Assessment of seawall end effects

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

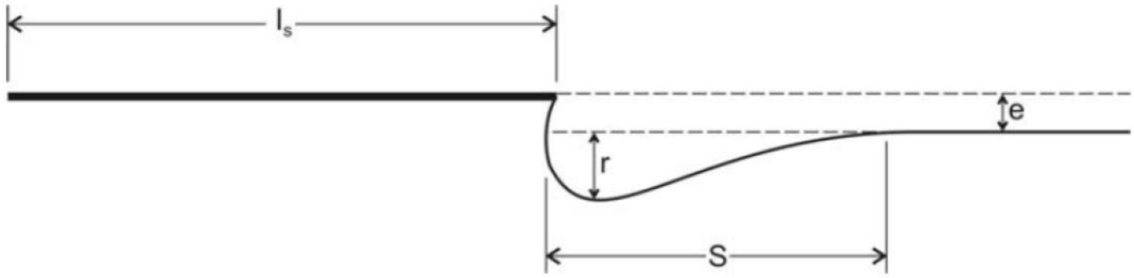


Figure 14: Seawall end effect variables

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

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

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

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

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

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

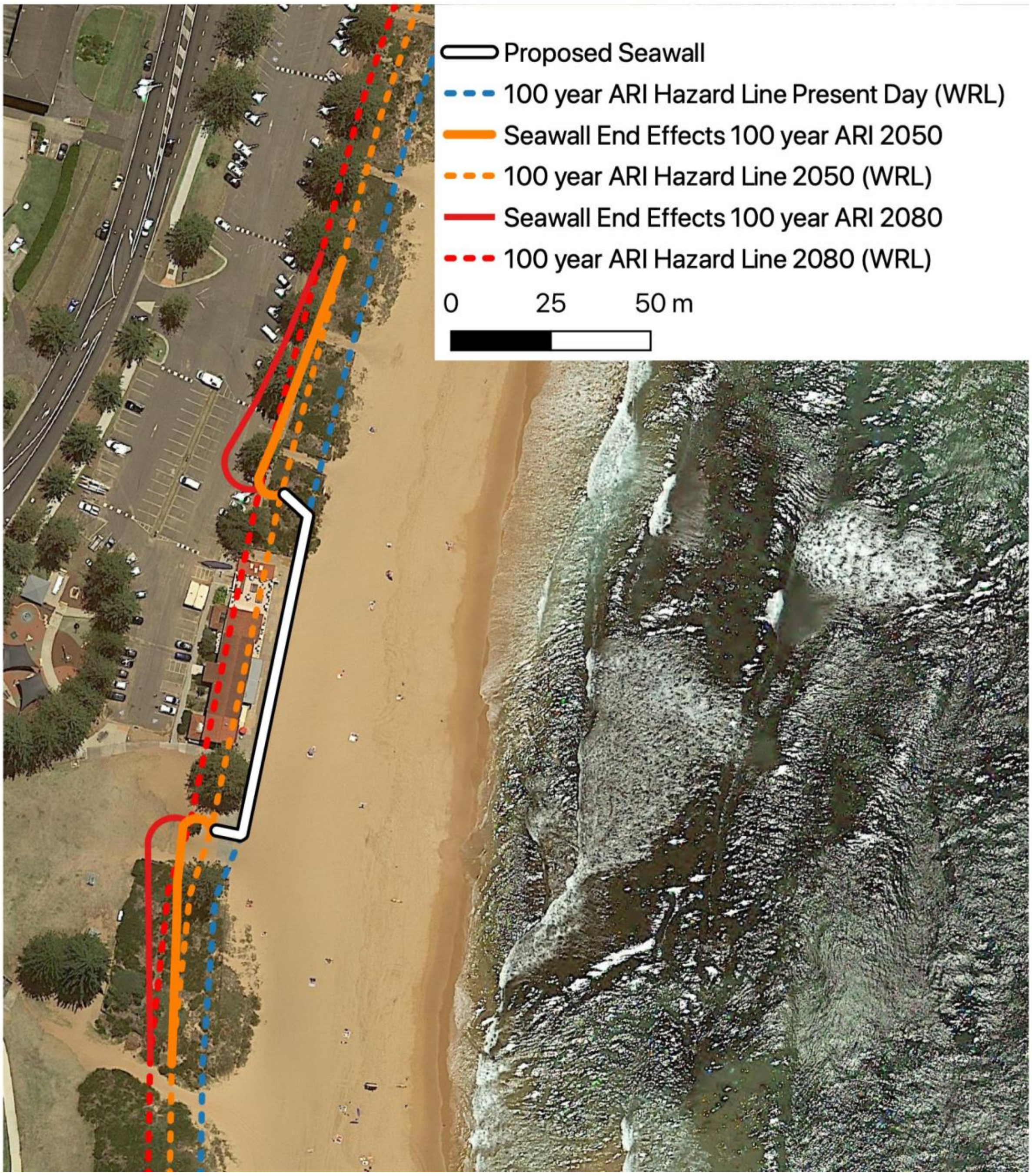


Figure 15: Theoretical seawall end effect for 100 year ARI conditions

8. Summary

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

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

The above parameters were calculated for:

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

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

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

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

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

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

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

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

Yours sincerely,

Grantley Smith
Director, Industry Research

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10. Appendix A Historic photos

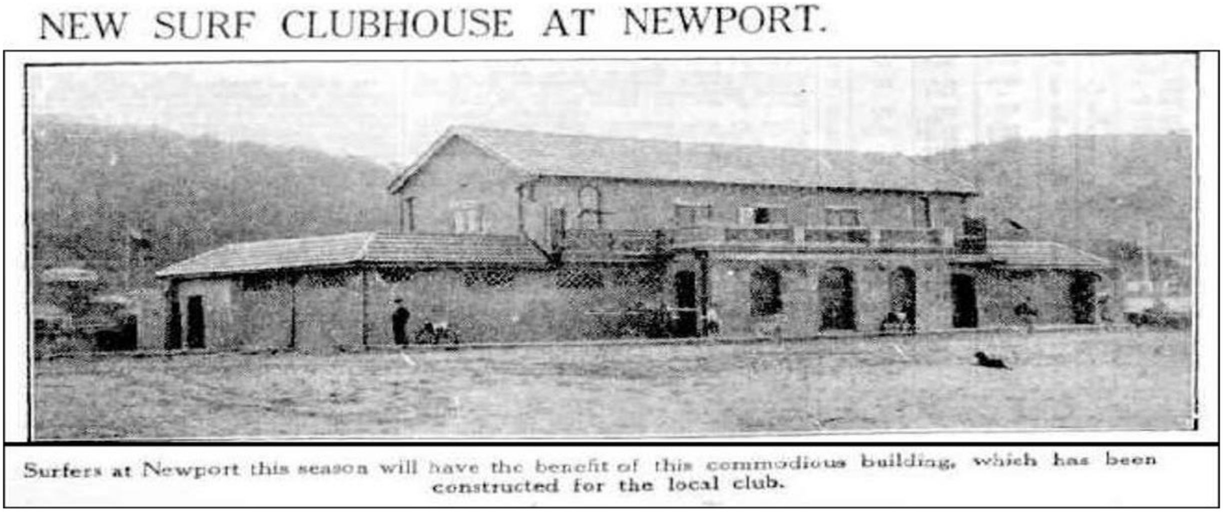

Figure 16: Newport SLSC 1933

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

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

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

APPENDIX C: ASSESSMENT OF TEMPORARY BARRIER TO REDUCE WAVE FORCES ON SLSC BUILDING BY JAMES TAYLOR & ASSOCIATES (STRUCTURAL ENGINEERS FOR SEAWALL)

James Taylor & Associates
Civil & Structural Consulting Engineers
ABN 33 102 603 558
Suite 301, 115 Military Road Neutral Bay NSW 2089
Postal Address: PO Box 742 Cremorne Junction NSW 2090
Tel: +61(0)2 9969 1999
Email: mail@jamestaylorassociates.com.au
Website: www.jamestaylorassociates.com.au

CONSULT AUSTRALIA
Member Firm

Ref:6268:RY:rp

24 August 2021

Newport SLSC
C/- Horton Coastal Engineering Pty Ltd
18 Reynolds Crescent
Beacon Hill NSW 2100

Dear Sir/Madam,

NEWPORT SLSC
BUILDING PROTECTION FROM WAVE FORCES

As requested, we have reviewed the report prepared by UNSW WRL dated 8 July 2021 titled Newport SLSC coastal engineering advice.

This report provides, among other advice, estimates of the wave forces that would act on the Newport SLSC building in the event of a severe coastal storm. The wave forces provided have been derived with the inclusion of a piled concrete seawall as documented in James Taylor & Associates drawings 6268 Revision D. Numerous different cases have been modelled representing differing ARI events.

You have advised that the building owner has stipulated the design parameters as the 500year ARI (2080). This load case is detailed in tables 9 and 10 of the WRL report (section 5). Table 9 describes wave forces based on wave runup for a partially eroded beach scenario. Table 10 describes forces based on an eroded beach with wave impact and overtopping the wall.

The tables are reproduced in total below:

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

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

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

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

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

We have considered the loads for Rmax in table 9 as the governing design load. Given the different erosion scenarios represented by tables 9 and 10, the loads are not considered to act simultaneously.

We have considered various options of constructing a barrier atop the seawall to resist the loads nominated above or to sufficiently reduce wave overtopping forces that would impact on the seaward face of the SLSC building.

These options include a continuous:
a) insitu concrete upstand wall
b) permanent lightweight barrier (steel framed), or
c) removable lightweight frame barrier

Options a) and b) are not considered feasible from an amenity point of view as they would, where present, prevent access from the promenade to the beach and vice versa.

Horton Coastal Engineering advised that the lateral force of 103kN/m should be applied halfway up the inundation depth, ie at 0.65m above the promenade (this force is comprised of a hydrostatic component that acts at one third of the depth above the promenade, and a wave velocity component that acts halfway above the promenade, however the wave velocity component is the significantly dominant component).

The lateral force of 103kN/m applied at a height* of 0.65m above the promenade is a significant load. For comparison, heavy duty road bridge barriers are designed for a lateral force of 250kN/m at a height of 0.5m.

We have considered the 103kN/m to be the ultimate limit state design load and have not applied a further factor to it. Based on this the steel sections with adequate bending strength are listed below for centres of 1m and 1.5m. We have not considered spacings in excess of 1.5m as the horizontal barriers required to span such a distance may become too heavy for easy handling.

* Note that this height is subject to confirmation as it applies at the seaward face of the building, not the seaward face of the promenade, in the WRL report. The barrier could be theoretically located anywhere on the promenade seaward of the SLSC building.

Forces on the barrier could be reduced if located landward of fixed, robust solid seating elements. Alternatively, robust solid seating located landwards of the barrier may assist in providing lateral support to it. The requirement for a barrier, its form and location would be assessed as part of detailed design.

Spacing Bollard	Section Type	Size (Grade 350MPa)	Weight per metre	Weight of Single
1m	SHS	200mmx6mm	36kg/m	65kg/approx
1m	SHS	150mmx9mm	38kg/m	70kg/approx
1m	RHS	200mmx150mmx6mm	31kg/m	55kg approx
1m	CHS	273mmx5mm	32kg/m	58kg/approx
1.5m	SHS	250mmx6mm	45kg/m	80kg/approx
1.5m	CHS	273mmx6mm	42kg/m	75kg/approx

SHS - Square Hollow Section
RHS - Rectangular Hollow Section
CHS - Circular Hollow Section

Removable bollards such as these are typically housed in a cast in sleeve with a locking bolt mechanism. The sleeve is capped with a hinged plate.

Infill panels between the bollards can take several forms including timber sleepers, steel framed panels and composite panels. Panel design would be guided by ease of handling (lightweight and durable), corrosion resistance for long term storage, ease of erection and cost.

Panel pieces will require mechanical connection to the bollards to prevent dislodging during a storm event. Such connections are required to be extremely robust. That stated, it is considered that a feasible bollard, infill panel and mechanical connection design could be developed to reduce the likelihood of significant wave forces on the SLSC building for the design event.

The design of the proposed seawall can accommodate the nominated wave impact forces.

However, the success of such an approach will rely on the removable barrier having the following characteristics:

- Easily found prior to a storm event (where is it stored?)
- Durable for long term storage (not rusted or in poor condition when needed in 20+ years, wall thickness is a consideration here)
- Easily installed appropriately by untrained personnel in a priority situation, possibly in a windy, wet and dark environment.
- Have interchangeable parts (no unique bollards or barrier positioning).
- Have easily prepared fixing points (bollard sleeves free from sand and corrosion, not covered by furniture, debris etc).

The embedment of the bollard spigot will affect the overall width and depth of the piled wall capping beam to ensure adequate concrete embedment is achieved. The significant lateral overturning moment will produce another load case for checking of global stability. The capping beam to pile connection is unlikely to be affected, however the pile embedment may increase marginally.

The proposal to support a removable wave/water barrier on the proposed new seawall at Newport SLSC is feasible with some minor changes to the current seawall design. The barrier design itself would be subject to development, as part of detailed design to ensure an appropriate solution is achieved.

We trust that this information is sufficient for your current requirements. Should you require any further information, please contact the undersigned.

Yours faithfully
JAMES TAYLOR & ASSOCIATES

RICHARD YATES B.E.(Hons) MIEAust CPEng NER 620330
Director

APPENDIX D: ASSESSMENT OF STRUCTURAL FEASIBILITY OF SLSC BUILDING REDEVELOPMENT BY PARTRIDGE STRUCTURAL (STRUCTURAL ENGINEERS FOR SLSC BUILDING)

20th August 2021

Mr Adriano Pupilli
Adriano Pupilli Architects
PO Box 770
MANLY NSW 2095

Dear Mr Pupilli,

RE: STRUCTURAL FEASIBILITY REPORT on PROPOSED ALTERATIONS and ADDITIONS to NEWPORT SLSC

1.0 INTRODUCTION

Further to the request of Mr Adriano Pupilli, of Adriano Pupilli Architects, Mr Peter Standen, Managing Director of Partridge Structural Pty Ltd, consulting structural engineers, carried out an inspection on Thursday 23rd May 2018 of the existing structure at Newport Surf Life Saving Club (SLSC). Also present at the time of the inspection was Mr Adriano Pupilli.

At the time of the inspection the weather conditions were sunny and dry.

Access was provided to the internal spaces in the building as well as the external areas.

The purpose of the inspection and this report was limited to investigate and advise on the feasibility of the proposed alterations and additions and to visually assess the structural adequacy of the existing building to support the proposed ground floor and first floor alterations and additions.

Subsequent to the inspection, Partridge Structural has liaised with Adriano Pupilli Architects and Horton Coastal Engineering with regard to suitable foundations for the proposed clubhouse, and feasible measures to resist wave forces on the seaward face of the clubhouse.

This report lists our observations made during the inspection, and our comments based on our review of the proposed architectural design and discussions with the coastal engineer, Horton Coastal Engineering.

Reference documents:

- Horton Coastal Engineering report “Assessment of Options for Redevelopment of Newport SLSC, with Updated Consideration of Risk from Coastal Erosion/Recession” issue A dated 17 February 2020
- Heritage21 Conservation Management Plan, job number 8133, dated July 2020
- Architectural drawings, NSC 000 – 013 prepared by Adriano Pupilli Architects, revision D.
- UNSW WRL Report dated 8th July 2021
- Horton Coastal Engineering reports “Coastal Engineering and Flooding Advice for Newport SLSC Clubhouse Redevelopment” and “Coastal Engineering Report and

Statement of Environmental Effects for Buried Coastal Protection Works at Newport SLSC" dated August 2021.
- James Taylor and Associates report "Newport SLSC, Building Protection from Wave Forces" dated August 2021.

2.0 DESCRIPTION

The property is situated on the eastern side of Barrenjoey Road and for the purposes of this report the front of the building facing Barrenjoey Road is deemed to be facing west.

The site is relatively flat and located at the western edge of Newport Beach. The existing property comprises a two-storey building currently being used as a Surf Life Saving Club and was originally constructed of timber framed roof and floors supported on load bearing masonry walls and strip footings. The original construction was completed in 1933. Subsequent additions to the building have been constructed with suspended reinforced concrete floor slabs supported on load bearing masonry and founded on a concrete raft slab to the north and the original building strip footings to the south.

3.0 INSPECTION AND DISCUSSION

The inspection consisted of visual observations of the existing structure.

Our summary of observations is listed as follows with our recommendations provided below each summary.

3.1 Existing structure

The building presents in generally good condition considering its age and proximity to the ocean. There is evidence of previous and current concrete spalling, where corroding reinforcement is causing the concrete to crack, which is to be expected given the marine environment and age of the structure.

Based on our visual inspection the load bearing structural elements generally are performing as intended and it is our opinion that the existing walls and foundations will be capable of safely supporting the proposed alterations and additions, without allowing for any wave forces on the building (remedial measures to deal with the wave forces are outlined below). The original walls appear to be of solid 230mm thick masonry construction and the subsequent additions have been constructed with perimeter cavity walls.

We have also undertaken a cursory review of the proposed alterations and additions with Adriano Pupilli Architects and confirm that, in our opinion, the existing structure will be capable of safely supporting the proposed alterations and additions when designed by a suitably qualified and experienced structural engineer. As part of the building works the cavity ties, concrete spalling and steel beams should be checked for evidence of corrosion and repaired as deemed necessary.

The existing structure as originally constructed in 1933 is likely founded on shallow brick (or possibly concrete) strip footings. The existing and original foundations appear to be performing adequately since construction in 1933. According to the

Horton Coastal Engineering report, the existing structure does not comply with current coastal erosion requirements for foundation depths, and is expected to be undermined by coastal erosion and severely damaged in the design event.

The existing structure, in its current condition, will not resist the design wave loading as noted in the WRL or Horton reports.

It would be highly invasive and not cost effective to retrofit deep foundations to the existing structure to allow it to remain supported in the design erosion event. The proposed seawall is therefore necessary for the existing structure to not be undermined and damaged by coastal erosion in the design event, as discussed in the Horton Coastal Engineering reports. The seawall is designed to acceptably reduce the risk of scour occurring below the existing foundations of the clubhouse.

For the existing structure to resist the design wave runup loading, it will be necessary to install measures to reduce wave forces on the building and/or install strengthening elements on the seaward face of the clubhouse. Measures to reduce wave forces on the building are considered in the Horton Coastal Engineering reports, and include permanent seating barriers on the seaward and landward edges of the promenade, adjustments to the seawall stairs, and installation of temporary barriers on the promenade. Horton Coastal Engineering considered that as part of detailed design, a suitable mix of practical measures would be able to be formulated to reduce the wave forces on the existing structure to acceptable levels, in conjunction with strengthening measures on the seaward face of the building (if required) as discussed below.

Feasible remedial measures from a structural engineering perspective to increase the resistance of the seaward face of the existing building to wave forces would include introducing a secondary structure to the inside seaward face of the building to support the brickwork (either steel stiffening plates or a reinforced concrete wall) or introducing a reinforced concrete wall on the outside seaward face (which would need to be considered in conjunction with the heritage preservation objectives). In both cases (ie inside or outside), the secondary structure would not need to extend the full height of the ground floor, with forces acting below a design depth of 1.3m (which may be refined as part of detailed design).

3.2 Proposed structure and footings

We have not been engaged to undertake any calculations or detailed design at this stage of the project, however, following our cursory review of the Architect's design intent we consider that the proposed alterations and additions will be structurally feasible.

The proposed additions to the building should be founded on similar material to the original structure, or there should be consideration of the potential for differential settlement in design of the additions to the building.

Referring to the Horton Coastal Engineering report (2020), we note several options have been considered to address the coastal erosion risks to the structure. We have assessed each of these options together with Horton Coastal Engineering, and we recommend the proposed approach of maintaining the existing building's shallow footings and strengthen as required, and to construct the new portion of the building on shallow foundations, in conjunction with a piled sea wall to protect both original and new portions of the structure (Option 6 in the Horton Coastal Engineering report 2020) as the preferred option.

The proposed new portion of the building structure can be designed to resist the wave loading from the WRL report without wave loading mitigation measures, if required. This can be achieved by having sufficiently thick reinforced concrete walls and/or columns, say 200mm thick (to be confirmed as part of detailed design). The storage room doors would be considered as sacrificial unless measures were installed to reduce wave forces on the new portion of the building.

Consideration could be given to pile foundations for the new portion of the building to reduce the extent of seawall required to the north of the building, if found to be cost effective.

4.0 CONCLUSION

We visually inspected the existing Newport Surf Life Saving Club structure and have undertaken a cursory review of the proposed alterations and additions prepared by Adriano Pupilli Architects, the Conservation Management Plan and the Coastal Engineering reports.

It is our opinion that the proposed ground floor and first floor alterations and additions are structurally feasible with the appropriate structural engineering strengthening and detailing. We recommend adopting a shallow foundation design to match the founding material of the existing portion of the building, combined with the coastal protection measures of a seawall to the east of the building as outlined by the Coastal Engineering Reports. With the construction of the proposed seawall we consider it feasible to design the new structure to resist the WRL wave loading, and feasible to strengthen the existing structure to resist the overtopping forces. We recommend initiatives be pursued to minimise the wave loading by analysing and installing seaward mitigation measures, as discussed in the Horton Coastal Engineering Reports.

Should you have any further queries please do not hesitate to contact the undersigned.

Yours faithfully,

Partridge Structural Pty Ltd

Peter Standen
BE (Hons1) BSc MIEAust CPEng NER (Structural & Civil) GAICD
Managing Director

APPENDIX E: FORMS 1 AND 1(A) FROM *COASTLINE RISK MANAGEMENT POLICY FOR DEVELOPMENT IN PITTWATER*

COASTLINE RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for Adriano Pupilli Architects Name of Applicant **Address of site** 394 Barrenjoey Road Newport (Newport SLSC)

Declaration made by a Coastal Engineer as part of a Coastal Risk Management Report

I, Peter Horton on behalf of Horton Coastal Engineering Pty Ltd
(Insert Name) (Trading or Company Name)

on this the 26 August 2021
(date)

certify that I am a Coastal Engineer as defined by the Coastline Risk Management Policy for Pittwater and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $2 million.

I have:

Please mark appropriate box

- ☑ Prepared the detailed Coastal Risk Management Report referenced below in accordance with the Pittwater Council Coastline Risk Management Policy
- ☐ Am willing to technically verify that the detailed Coastal Risk Management Report referenced below has been prepared in accordance with the Pittwater Council Coastline Risk Management Policy
- ☐ Have examined the site and the proposed development/alteration in detail and, as detailed in my report, am of the opinion that the Development Application only involves Minor Development/Alterations or is sited such that a detailed coastal hazard analysis or risk assessment is not required.
- ☐ Provided the coastal hazard analysis for inclusion in the Coastal Risk Management Report

Coastal Risk Management Report Details:

Report Title:	Coastal Engineering and Flooding Advice for Newport SLSC Redevelopment
Report Date:	26 August 2021
Author:	Horton Coastal Engineering Pty Ltd

Documentation which relate to or are relied upon in report preparation:

See Section 2 and Section 10 of report

I am aware that the above Coastal Risk Management Report, prepared for the above mentioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the coastal risk management aspects of the proposed development have been adequately addressed to achieve an acceptable risk management level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ……………………………………………………………………..

Name …………………Peter Horton……………………………………………..

Chartered Professional Status……MIEAust CPEng NER

Membership No. ……452980……………………………………………

COASTLINE RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1(a) - Checklist of Requirements for Coastal Risk Management Report for Development Application or Part 5 Assessment

Development Application for Adriano Pupilli Architects

Name of Applicant

Address of site 394 Barrenjoey Road Newport (Newport SLSC)

The following checklist covers the minimum requirements to be addressed in a Coastal Risk Management Report. This checklist is to accompany the Coastal Risk Management Report and its certification (Form No. 1).

Coastal Risk Management Report Details:

Report Title: Coastal Engineering and Flooding Advice for Newport SLSC Redevelopment

Report Date: 26 August 2021

Author: Horton Coastal Engineering Pty Ltd

Please mark appropriate box

☑ Comprehensive site mapping conducted Survey provided as per Section 2
(date)

☐ Mapping details presented on contoured site plan to a minimum scale of 1:200 (as appropriate)
N/A, new coastal protection works are proposed

☑ Subsurface investigation required

☐ No Justification ……………………………………………………..

☑ Yes Date conducted Discussed in Section 3.5

☑ Impact by and upon coastal processes identified

☑ Coastal hazards identified

☑ Coastal hazards described and reported

☑ Risk assessment conducted in accordance with Council's Policy

☐ Adequacy of existing coastal protection measures assessed and certified N/A, new works are proposed

☑ Opinion has been provided that the design can achieve the risk management criteria in accordance with Council's Policy provided that the specified conditions are achieved.

☑ Design Life Adopted:

☐ 100 years

☑ Other ……60 years……

specify

☑ Development Controls as described in the Pittwater Coastline Risk Management Policy have been specified

☑ Additional actions to remove risk where reasonable and practical have been identified and included in the Coastal Risk Management Report.

I am aware that Pittwater Council will rely on the Coastal Risk Management Report, to which this checklist applies, as the basis for ensuring that the coastal risk management aspects of the proposal have been adequately addressed to achieve an acceptable risk management level for the life of the structure, taken as at least 100 years unless otherwise specified, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ……Peter Horton……

Name ……Peter Horton……

Chartered Professional Status……MIEAust CPEng NER……

Membership No. ……452980……