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UNSW 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.

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

- Horton (2018) "Initial Coastal Engineering Advice on Newport SLSC Development", prepared by Horton Coastal Engineering Pty Ltd for Adriano Pupilli Architects, dated 14 August 2018.
- Horton (2020b) "Assessment of Options for Redevelopment of Newport SLSC, with Updated Consideration of Risk from Coastal Erosion/Recession", prepared by Horton Coastal Engineering Pty Ltd (Horton) for Adriano Pupilli Architects, Issue A, dated 17 February 2020.
- Horton (2020c), "Coastal Engineering and Flooding Advice for Newport SLSC Clubhouse Redevelopment", prepared by Horton Coastal Engineering Pty Ltd (Horton) for Adriano Pupilli Architects, Issue 2, dated 9 November 2020.

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

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

2. Design Conditions

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

Table 1: Design offshore wave conditions

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

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

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

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

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

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

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

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

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

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

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

Table 3: Design offshore wave conditions

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

Notes

(1) SLR was set as 0.26 m for 2050

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

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

3.1 Measured data

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

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

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

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

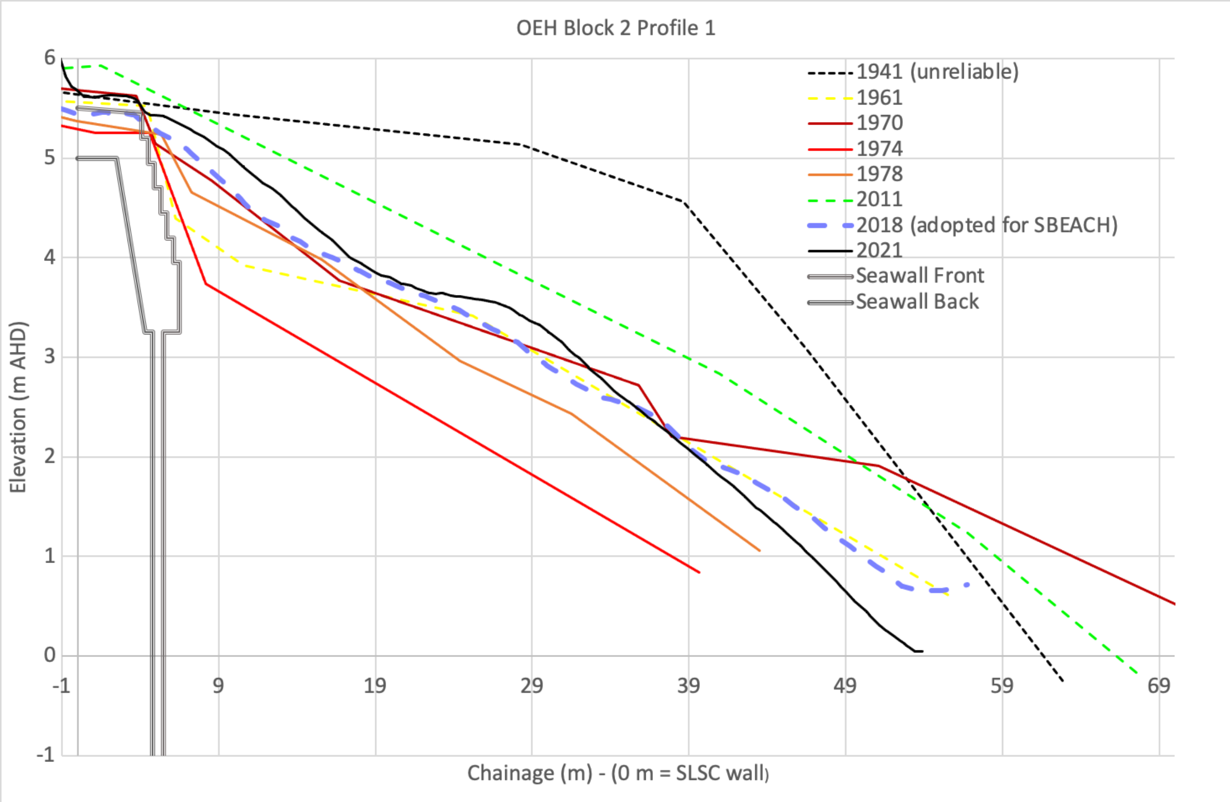


Figure 1: Measured profile data with proposed seawall superimposed

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

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

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

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

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

3.2 *Modelling of erosion*

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

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

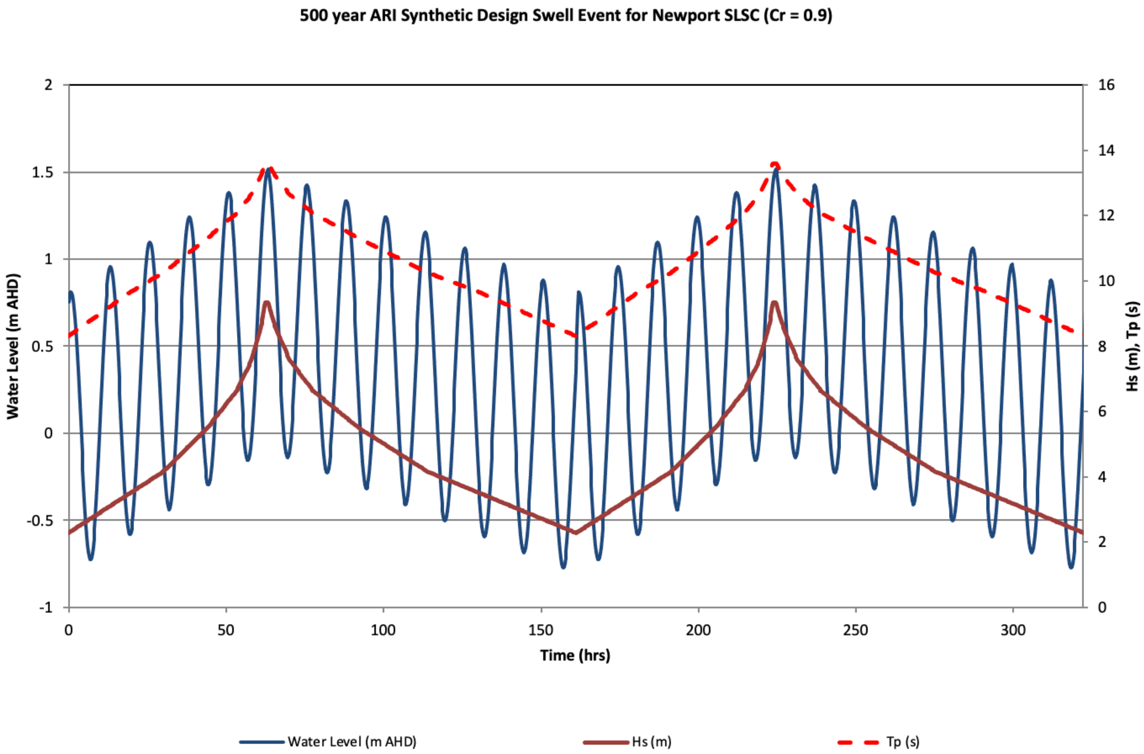


Figure 2: 500 year ARI synthetic design swell time series for Newport Beach (Note that only 2 consecutive storms were used for the study – i.e. erosion volumes derived after 322 hours)

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

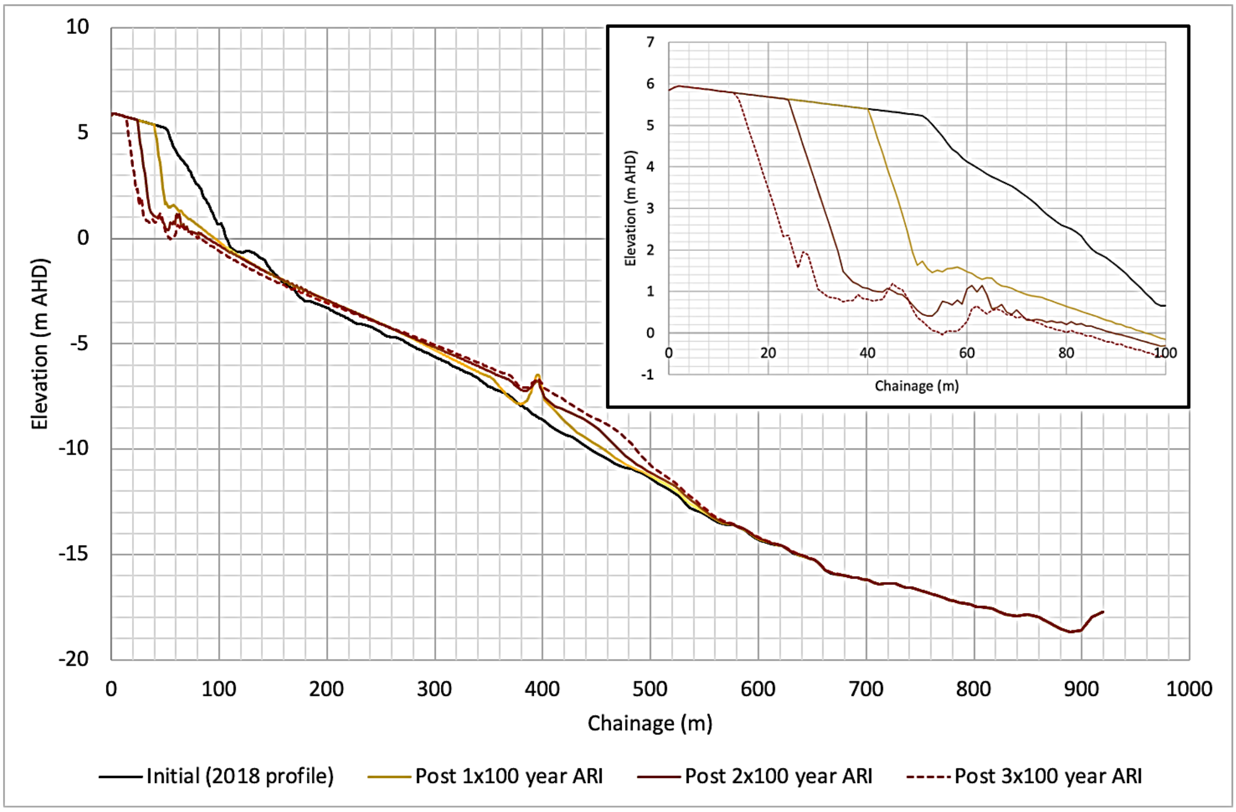


Figure 3: Evolution of beach profiles for consecutive storms in SBEACH with no seawall in place

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

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

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

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

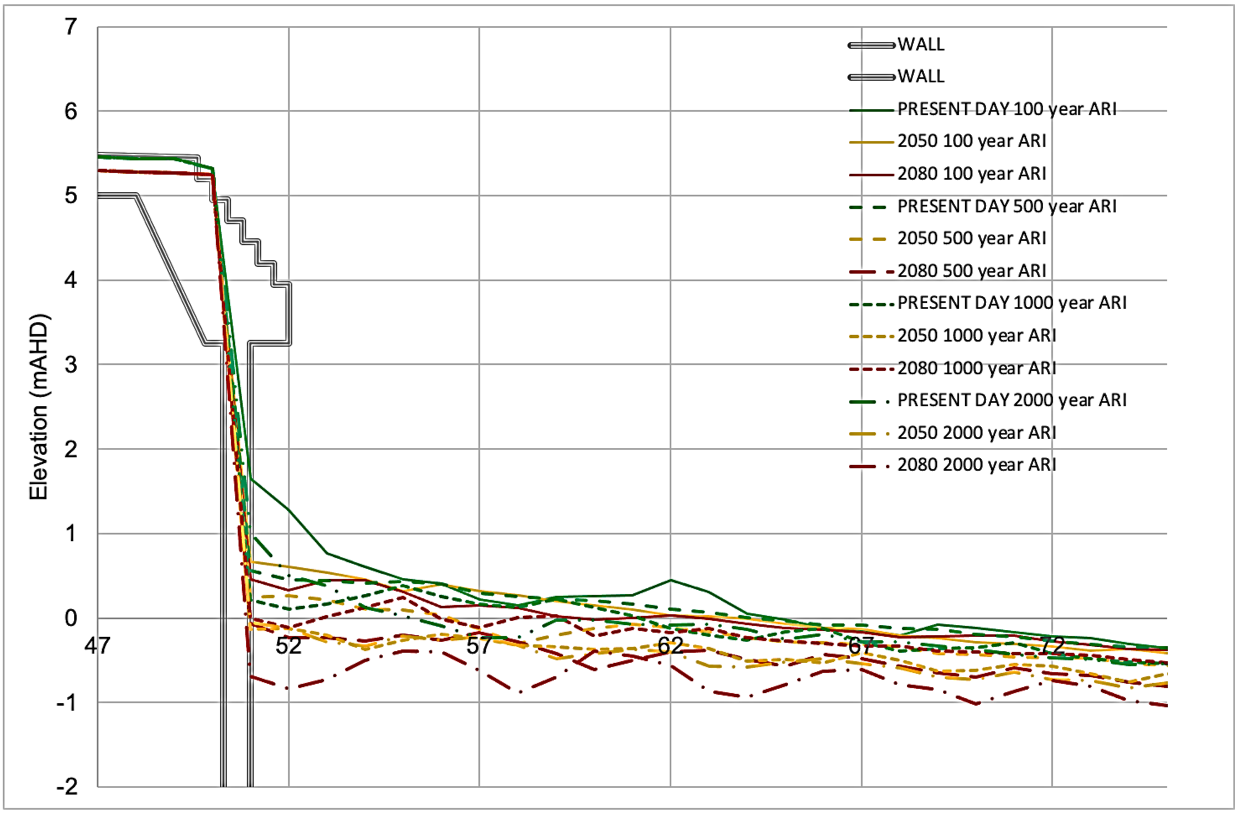


Figure 4: Evolution of beach profiles for consecutive storms in SBEACH with no seawall in place

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

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

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

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

4. Estimation of wave runup and overtopping

4.1 Overview

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

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

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

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

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

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

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

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

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

4.2 Accreted or average beach runup

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

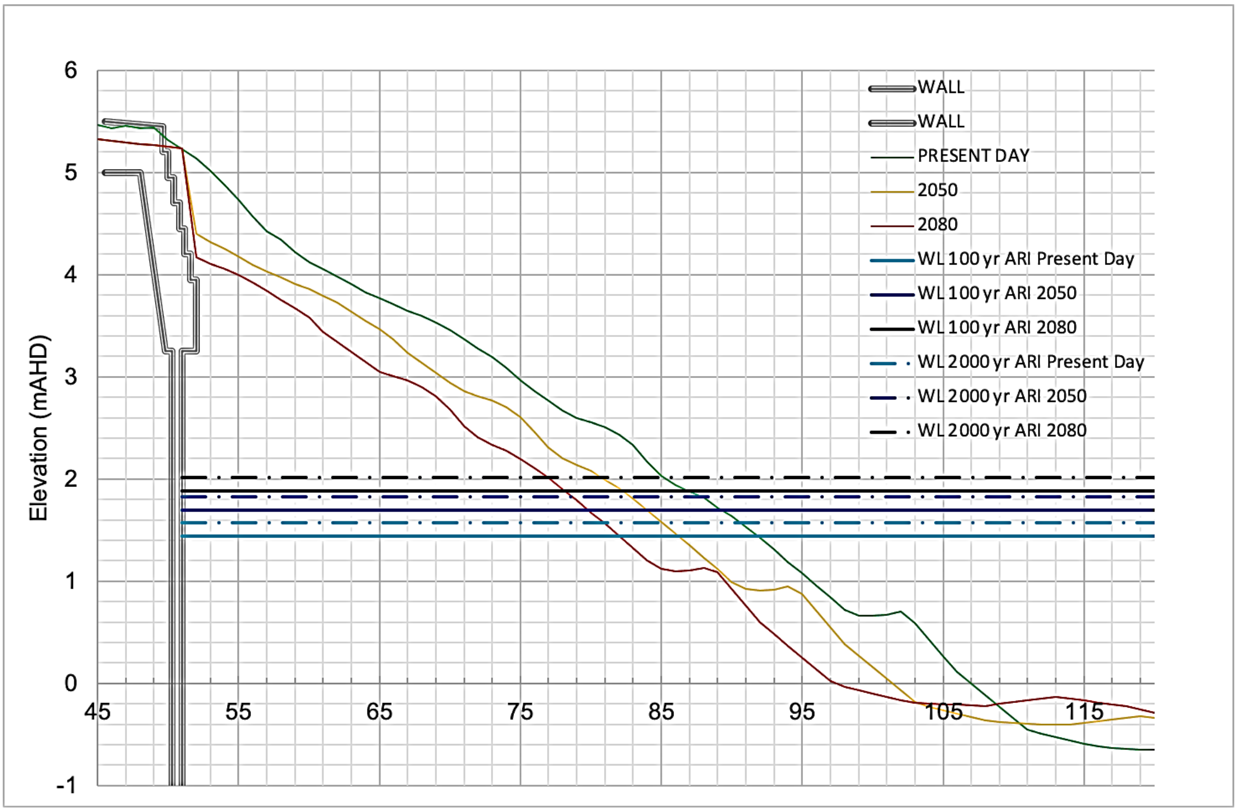


Figure 5: Water levels (no wave setup) for 100 and 2000 year ARI events for present day, 2050 and 2080 planning period

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

This storm had the following peak characteristics:

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

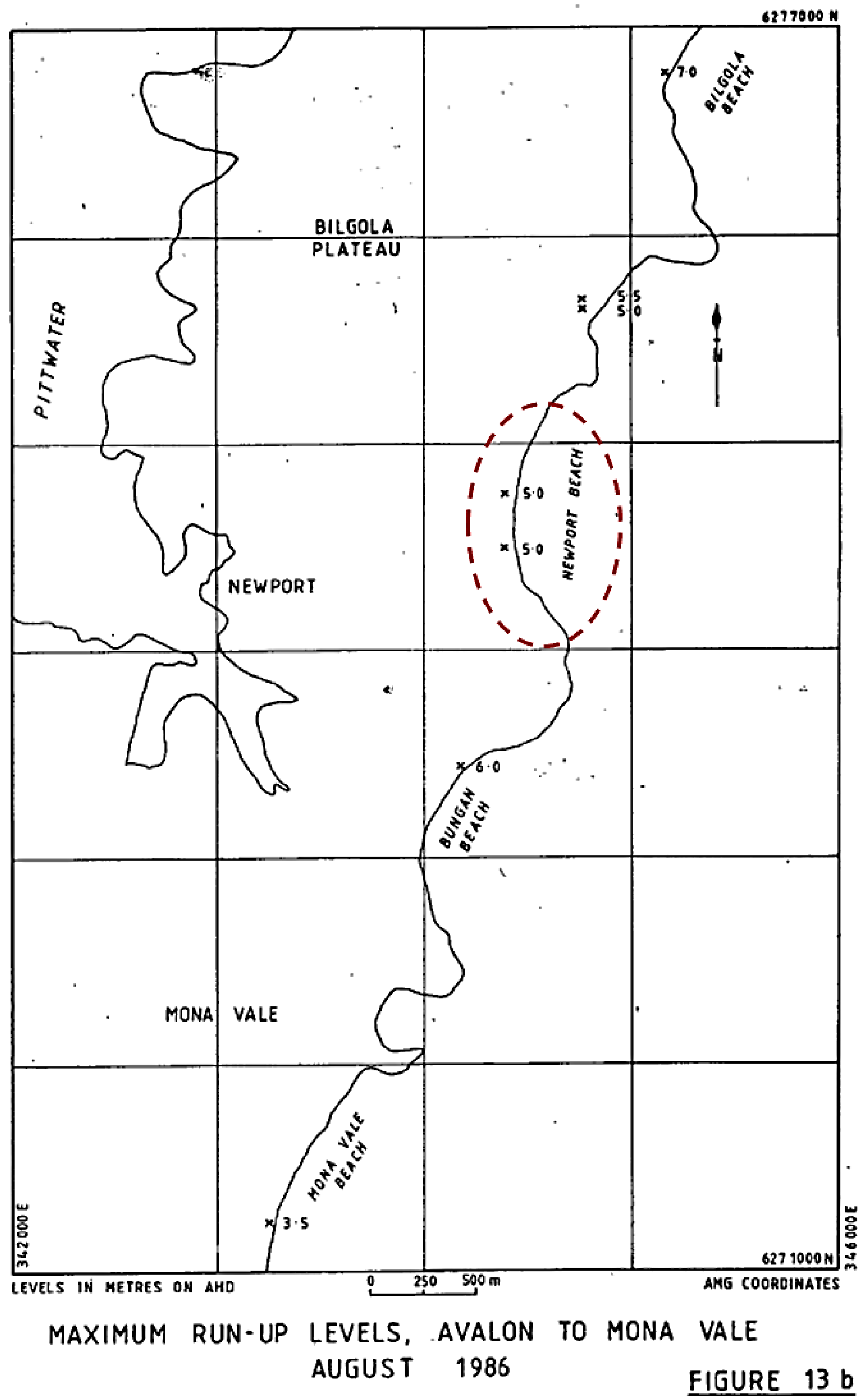


Figure 6: Observed wave runup levels after August 1986 storm based on debris lines [Source: Higgs and Nittim, 1988]

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

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

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

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

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

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

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

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

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

4.3 *Wave runup and overtopping for eroded beach*

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

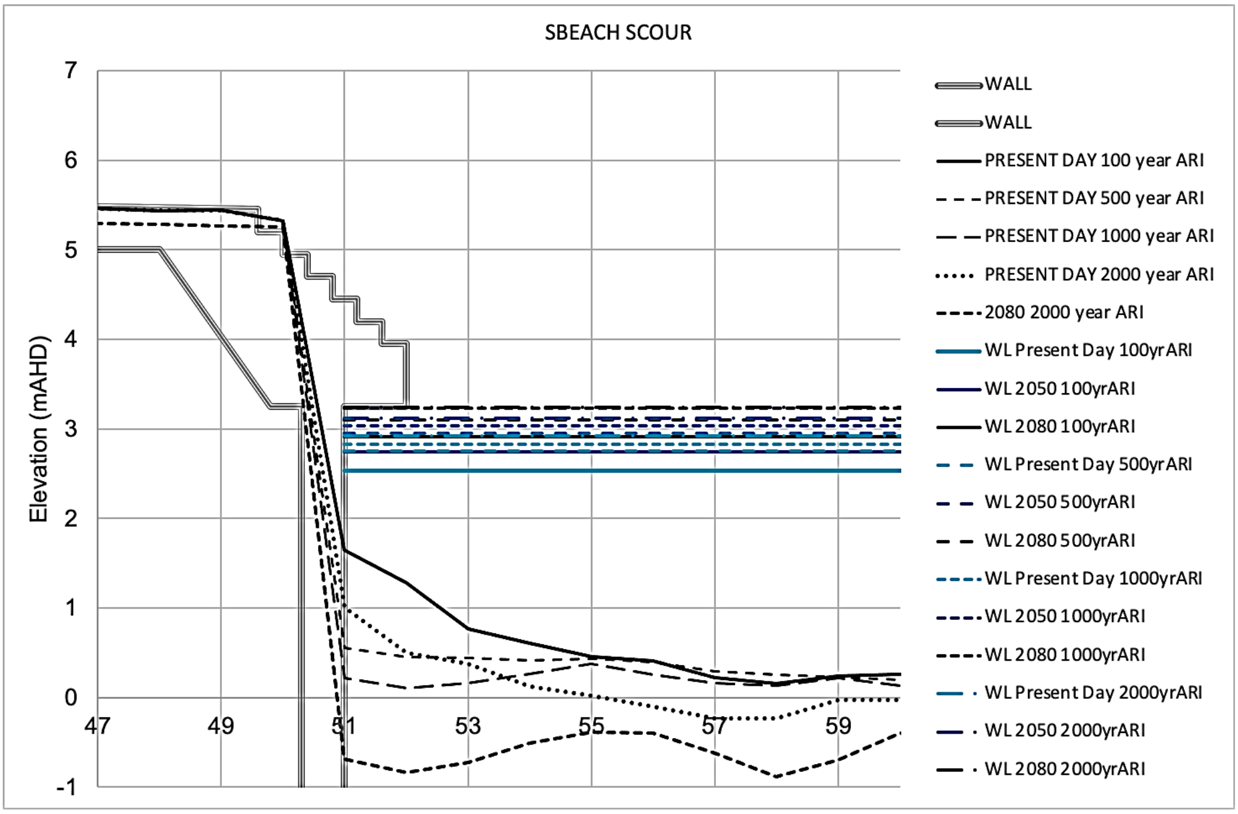


Figure 7: Calculated nearshore water levels (including local wave setup) and scoured levels in front of proposed seawall

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

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

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

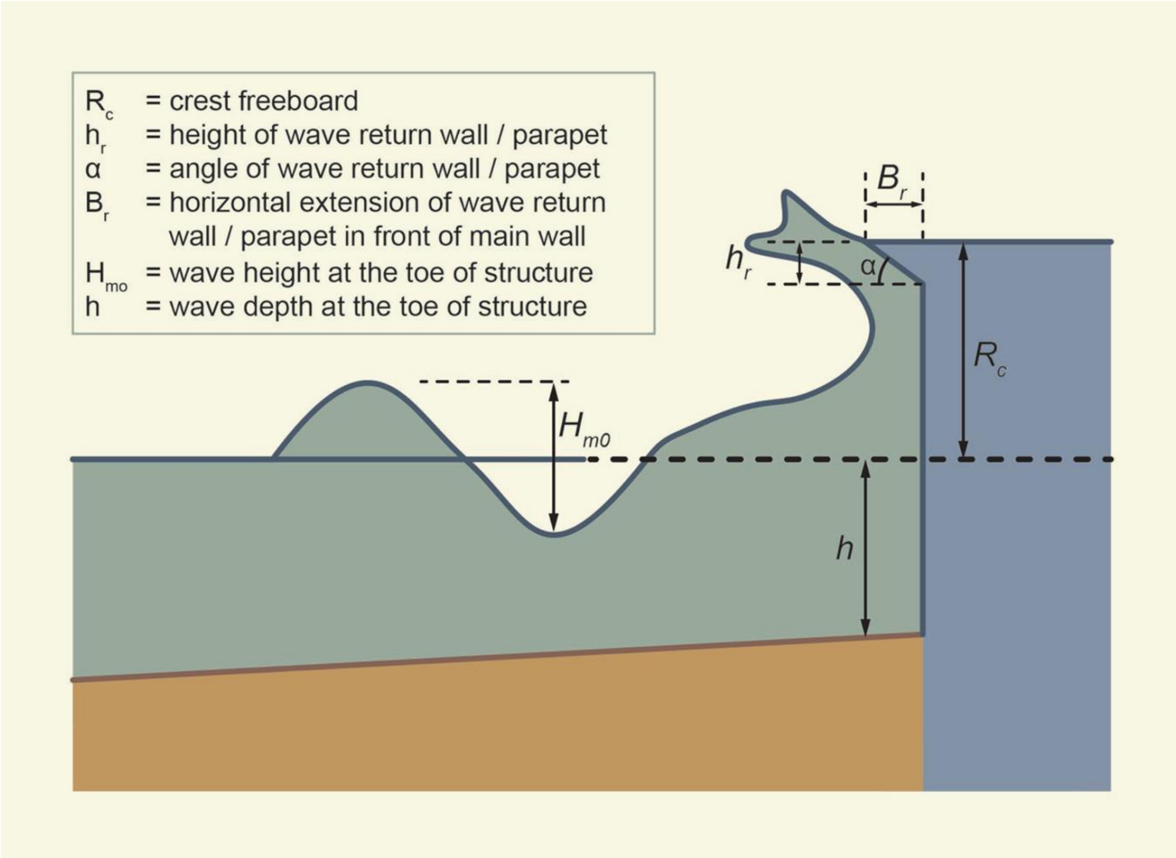


Figure 8: Parameters definitions for vertical seawall with return wall [Source: EurOtop, 2018]

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

Table 8: Overtopping discharges for proposed seawall with return wall

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

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

5. Wave loads due to overtopping

5.1 Overview

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

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

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

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

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

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

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

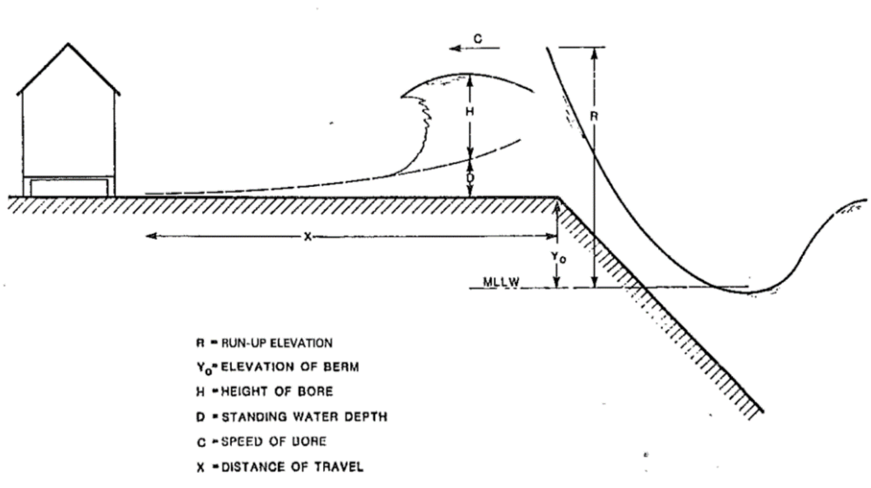


Figure 9: Definition of overtopping parameters [Source: Cox and Machemehl, 1986]

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

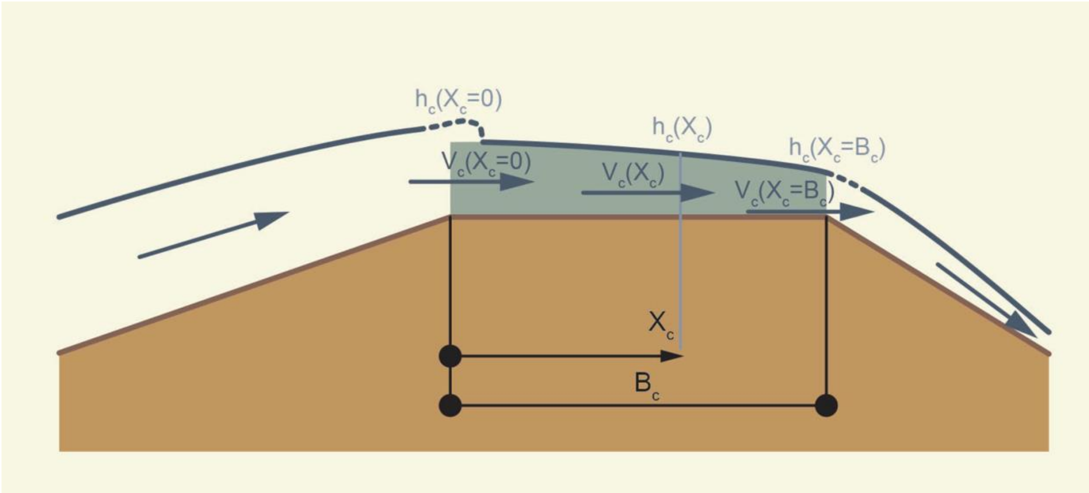


Figure 10: Sketch of overtopping flow parameters [Source: EurOtop, 2008]

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

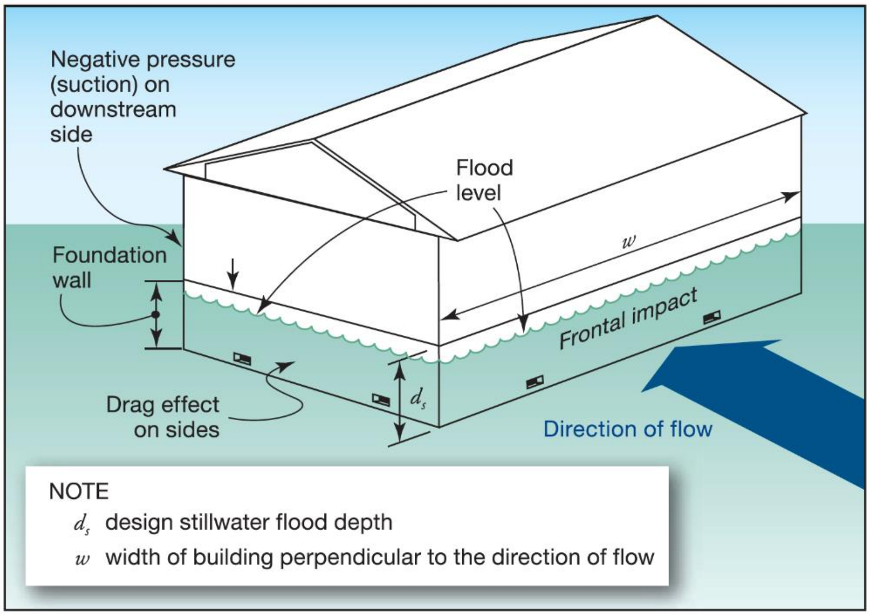


Figure 11: Hydrodynamic loads on a building [Source: FEMA, 2011]

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

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

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

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

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

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

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

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

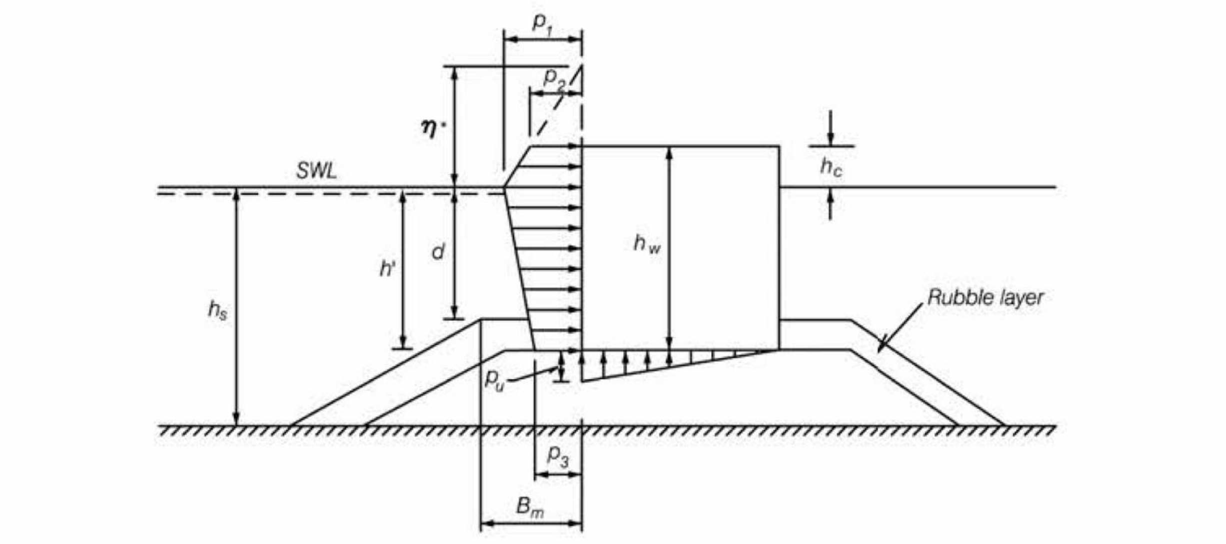


Figure 12: Hydrodynamic loads due to wave impact on a coastal structure [Source: CEM, 2011]

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

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

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

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

6. Review of available methods to reduce overtopping hazard

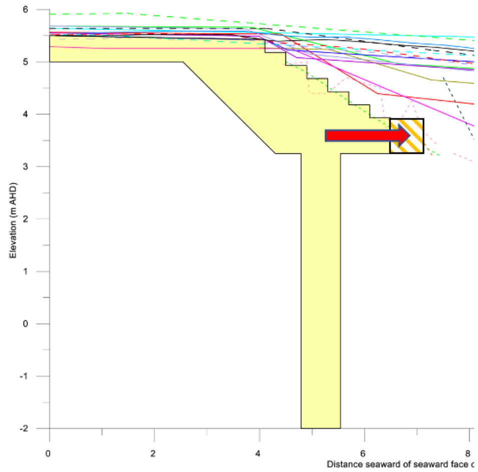
Should the wave overtopping or wave forces be deemed to be excessive, the following methods are available to reduce overtopping (Figure 13):

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

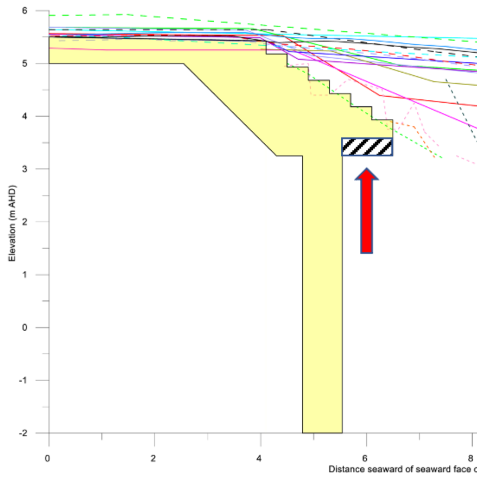
Additionally, the following short term management measures could be undertaken:

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

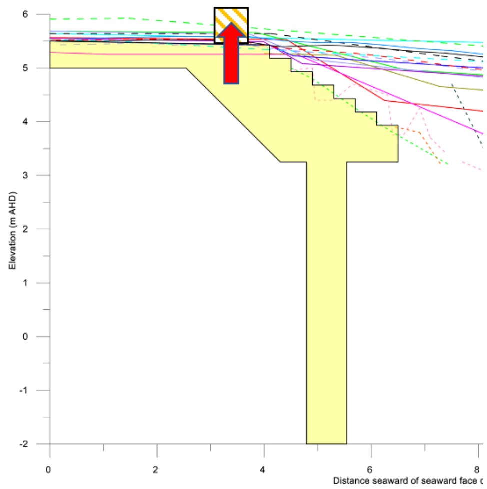
Additional calculations and/or later physical modelling may be required to quantify the benefit of each option.



(a) Extend projection of return wall



(b) Raise level of return wall



(b) Add return wall on crest

Figure 13: Options for reducing wave overtopping

7. Assessment of seawall end effects

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

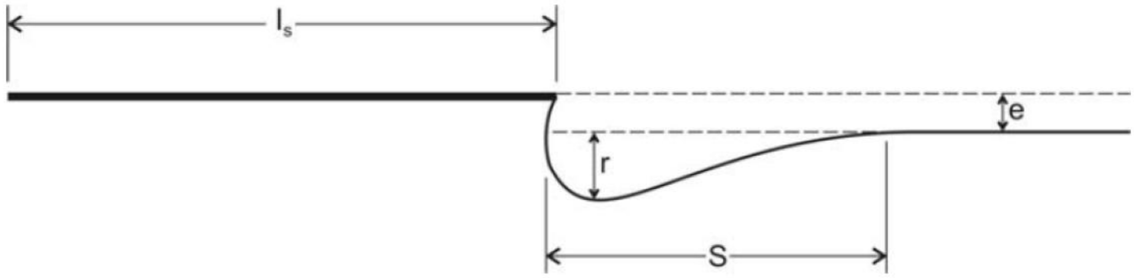


Figure 14: Seawall end effect variables

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

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

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

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

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

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

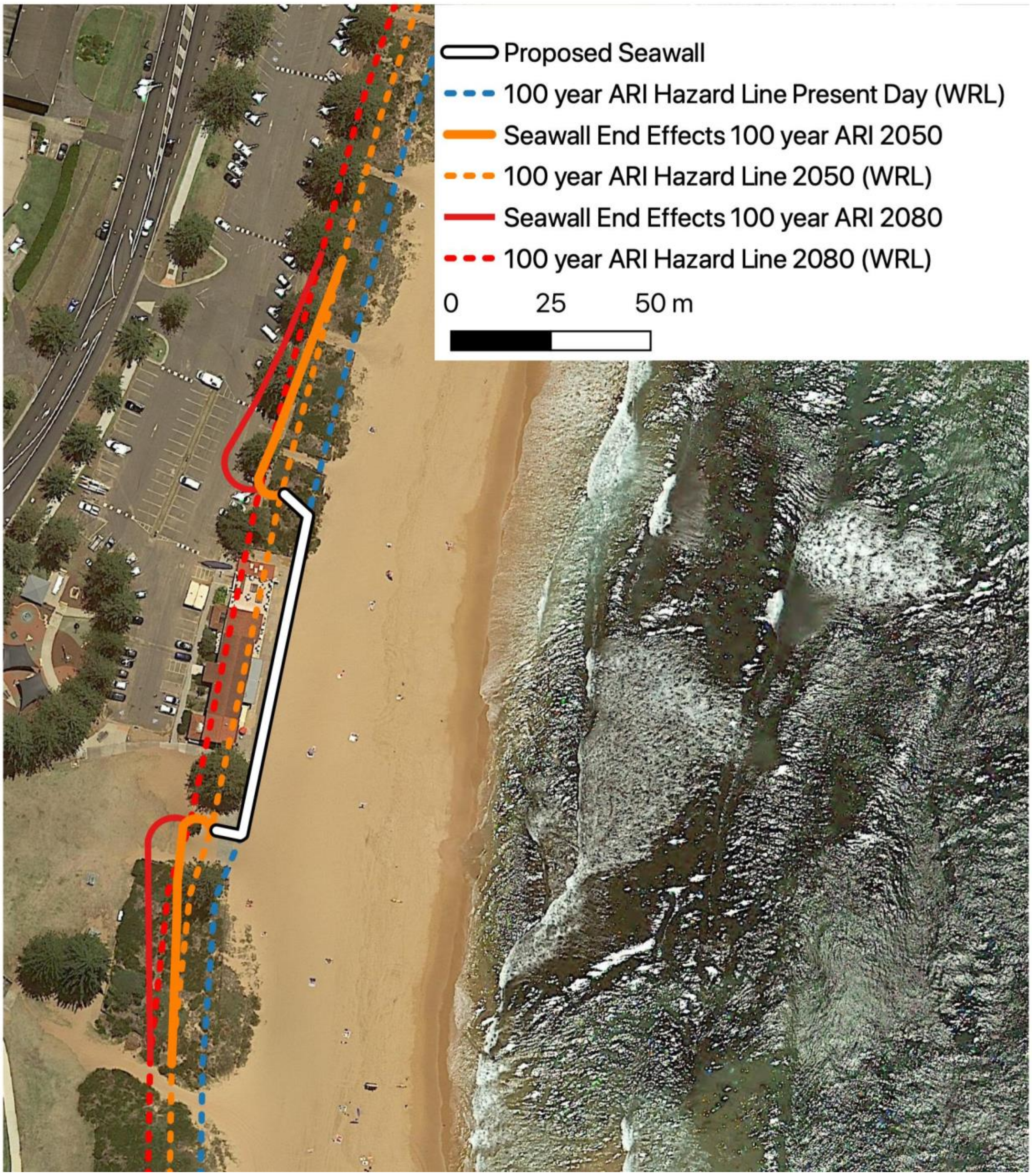


Figure 15: Theoretical seawall end effect for 100 year ARI conditions

8. Summary

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

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

The above parameters were calculated for:

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

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

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

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

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

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

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

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

Yours sincerely,

Grantley Smith
Director, Industry Research

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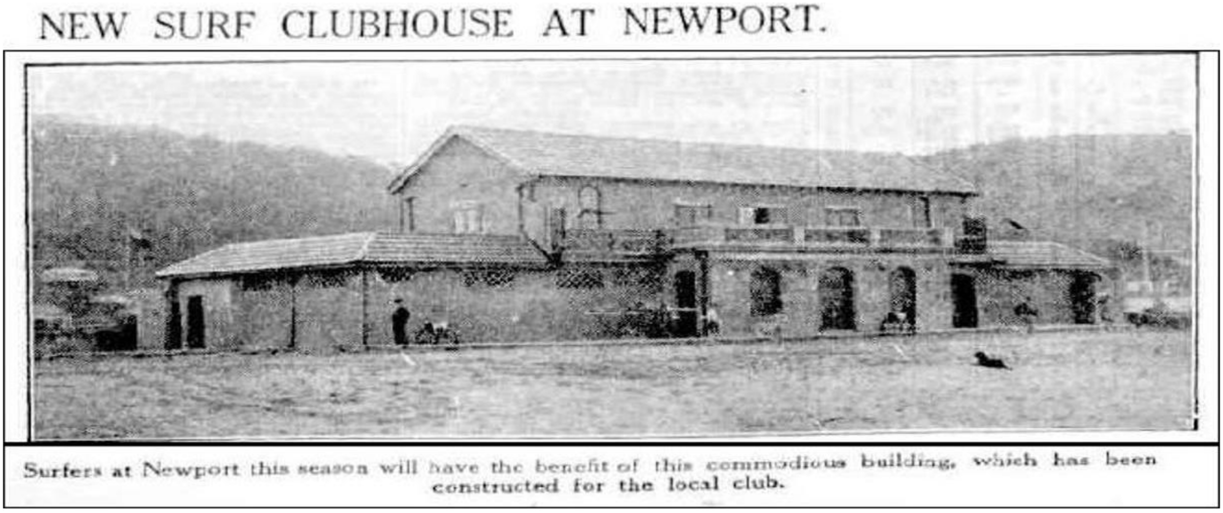
10. Appendix A Historic photos


NEW SURF CLUBHOUSE AT NEWPORT.

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

Figure 16: Newport SLSC 1933

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

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

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