28 April 2025

WRL Ref: WRL2025012 LR20250428ab JTC

PRIVILEGED AND CONFIDENTIAL

Ms Ballanda Sack and Mr Andrew Beatty
Beatty Hughes and Associates
Level 21, 8 Chifley Square
Sydney NSW 2000

By email: ballanda@beattyhughes.com.au
andrew@beattyhughes.com.au

Dear Ballanda and Andrew,

Cliff stabilisation works at 217A Beach Road, Denhams Beach

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney is pleased to provide this letter report to Canplay Pty Ltd via Beatty Hughes and Associates.

This letter provides advice regarding the coastal engineering aspects of the cliff stabilisation works at 217A Beach Road, Denhams Beach (in Batemans Bay, NSW) – “the subject property” (Figure 1-1 and Figure 1-2).

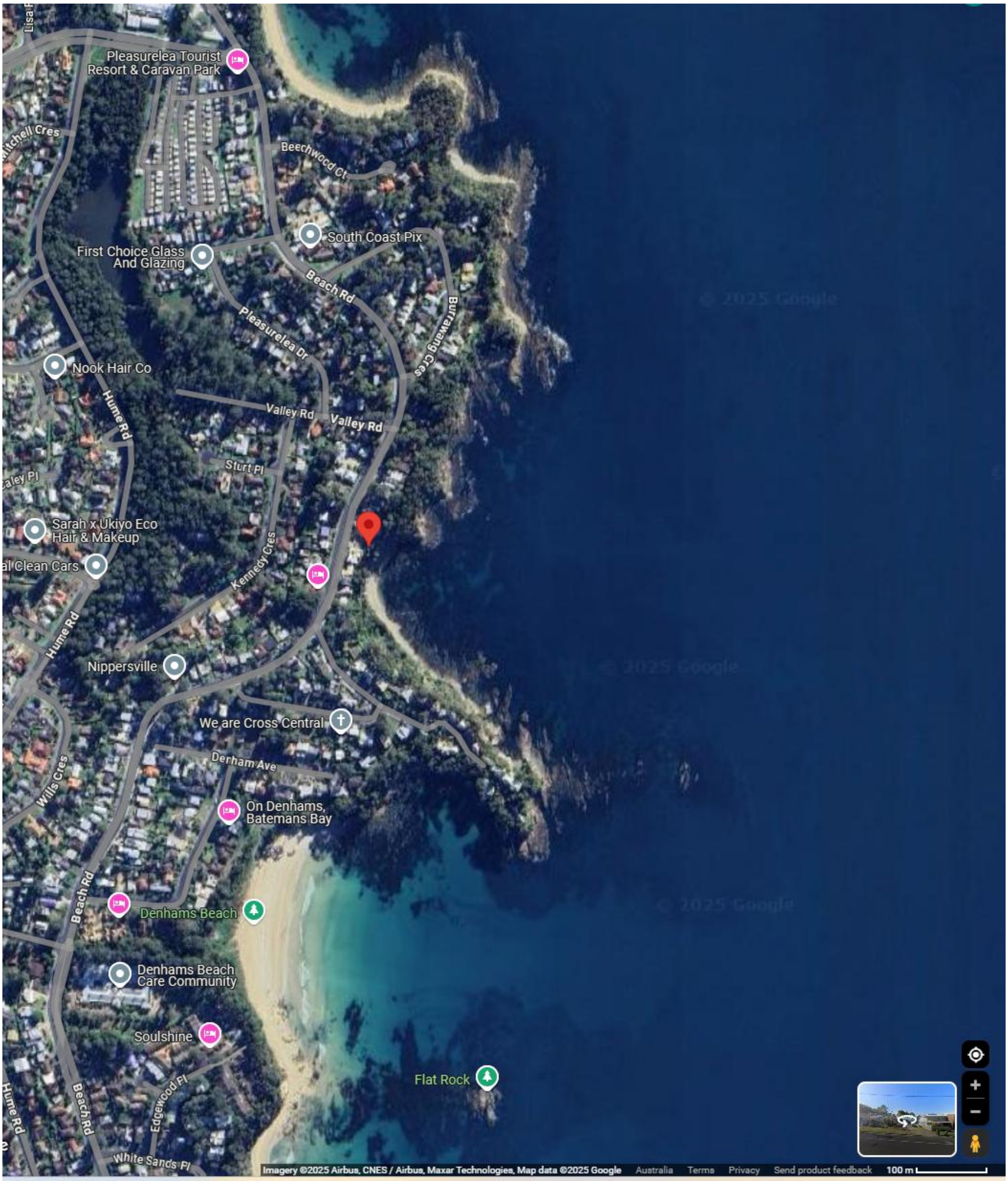


Figure 1-1 Site location indicated with red marker (Google maps)

Figure 1-2 Subject property indicated with blue arrow (Nearmap)

2. Executive summary

The site was visited by WRL's Principal Coastal Engineer James Carley on Monday, 10 March 2025.

The beach fronting the subject property has not been listed in "Beaches of the NSW Coast" by Professor Andrew Short (Short, 2007), and has been named North Denhams Beach in this report.

The works are more than 5 m landward of the seaward private property boundary. There is no public access to the beach via land, with access only through eight private properties or via water through the small keyhole gap in the surrounding rocky reefs.

The works will not unreasonably limit or be likely to unreasonably limit public access to or the use of the beach or headland.

The works are unlikely to cause any increased erosion of the beach or adjacent land.

Conditions for maintenance of the works are detailed in the body of this report, and consist primarily of annual or post-storm event inspections by a qualified engineer, and the development of an emergency action plan should the works be damaged or fail.

Wave runup is under extreme storms is estimated to be above the crest of the outer wall, but below the crest of the inner wall.

3. Instructions to WRL

The following instructions were received from Beatty Hughes and Associates dated 12 February 2025, with the relevant sections of the report where this is addressed indicated in square brackets:

> (*1) Review the documents and provide a preliminary view (verbal) as to whether the retaining walls are likely to have a significant impact on coastal processes or adjoining land;*
> *(2) View the site; and*
> *(3) Prepare a report suitable for provision to the Southern Regional Planning Panel that addresses:*
> *a. Whether the works will unreasonably limit or be likely to unreasonably limit public access to or the use of a beach or headland;* [addressed Section 6 and 6.6 of this letter report]
> *b. If the works are likely to cause any increased erosion of the beach or adjacent land* [addressed in Section 6 of this letter report]*;*
> *and*
> *c. Appropriate conditions for maintenance of the works* [addressed in Section 7 of this letter report]*; and*
> *d. If appropriate comment on the design suitability/wave overtopping risk* [addressed in Section 8, 10, 11 and 12 of this letter report].

4. Site visit and observations

The site was visited by WRL's Principal Coastal Engineer James Carley on Monday 10 March 2025. Photos from the site visit are shown in Figure 4-1 to Figure 4-4.

The beach fronting the subject property has not been listed in "Beaches of the NSW Coast" by Professor Andrew Short (Short, 2007). It lies between Sunshine Bay and Denhams Beach. In the absence of a geographic name, WRL has named it North Denhams Beach. There was no assessment or management recommendations for North Denhams Beach within the Coastal Hazard Study (Coghlan et al., 2017) nor the Coastal Management Program (CMP, Rhelm, 2022).

The beach is predominantly a rocky cove with steep cliffy headlands at each end, and is backed by a steep cliff fronting private properties. The beach consists of an apparently thin veneer of sand, pebbles and shells, over predominantly exposed bedrock and reef. The sand/pebble/shell beach is suitable for walking on and for entering the water through a narrow keyhole gap between the rocky outcrops (Figure 1-2).

The subject property has terraced retaining walls fronting the beach at the base of its cliff. The walls are constructed from Baines Masonry "Verti-Block" precast concrete interlocking blocks. These blocks have hollow cores for gravel and/or reinforced concrete filling. The standard unfilled blocks have a course height of 610 mm and a mass of 790 kg, with various component blocks having a mass ranging from 440 to 1590 kg (Figure 4-5). Neighbouring properties also have cliff stabilisation works, including terraced gabions, timber, (possibly) geogrids and shotcrete.

Based on the drawings supplied and observations from the site visit, the sand/pebble level against the subject works is estimated to be 1.5 to 2 m AHD. This is above the highest astronomical tide (1.2 m AHD), but below the wave runup level in major storms.

The bedrock is emergent (sandy beach ends) approximately 5 to 10 m from the subject property boundary (10 to 15 m from the cliff stabilisation works). Based on the structural drawings and the beach gradient, it is estimated that the sand veneer thickness is between 0.2 and 1 m over the bedrock at the seaward extent of the subject property's seaward retaining wall.

Based on direct site observations and advice from Eurobodalla Council (via Beatty Hughes and Associates), there is no public access to this beach from land, either via public pathways or around the surrounding headlands. The only access is via the water or through eight private properties which front the beach and have stair access down the cliff. Seven of these eight private properties have some form of coastal protection works at the base of the cliff (predominantly gabions, Figure 4-3) and five of these properties have a boatshed type structure fronting the beach (Figure 4-3).

There are numerous additional private properties to the south on the surrounding cliffs, but these do not appear to have stair access to North Denhams Beach. While no repeat beach profile data is available, the following commentary is provided by WRL:

- The thin veneer of sand/pebbles/shell over the bedrock is likely to vary over time, but erosion is limited by its thin veneer extent over the bedrock.
- Erosion of the surrounding cliffs and reefs, and shell production may continue to supply limited sand/pebbles/shell, but the beach is predominantly a rocky cove.

The quantum of sea level rise adopted for the CMP is:

- 2050: 0.22 m
- 2065: 0.33 m
- 2075: 0.43 m
- 2085: 0.54 m
- 2100: 0.71 m

Figure 4-1 View from above to the south

Figure 4-2 View from beach to the south

Figure 4-3 Boatsheds and gabion walls to south

Figure 4-4 Property to north of subject property bounded by headland to north

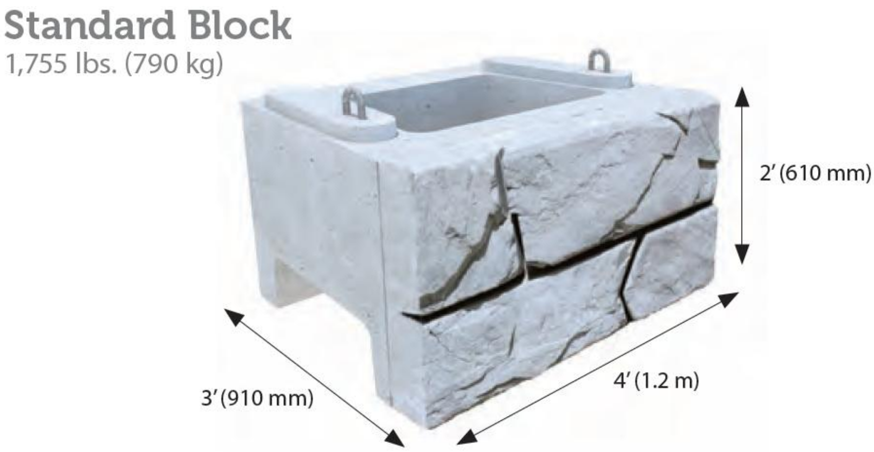


Figure 4-5 Baines Masonry Verti-Block standard block dimensions

5. Geotechnical report

A Geotechnical report has been prepared by ACT Geotechnical Engineers on 30 June 2023 (ref: OB/C14369). Figure 2 of the Geotechnical report indicates that landslides have occurred on the properties on either side of the subject property (Figure 5-1 of this WRL letter). The bedrock found at three boreholes on the upper portion of the site was described as:

> "SHALE: Extremely weathered and Highly weathered, excavated as Sandy CLAY, extremely weak, pale grey, some gravel, fine grained."

Figure 5-1 Extract from geotechnical report

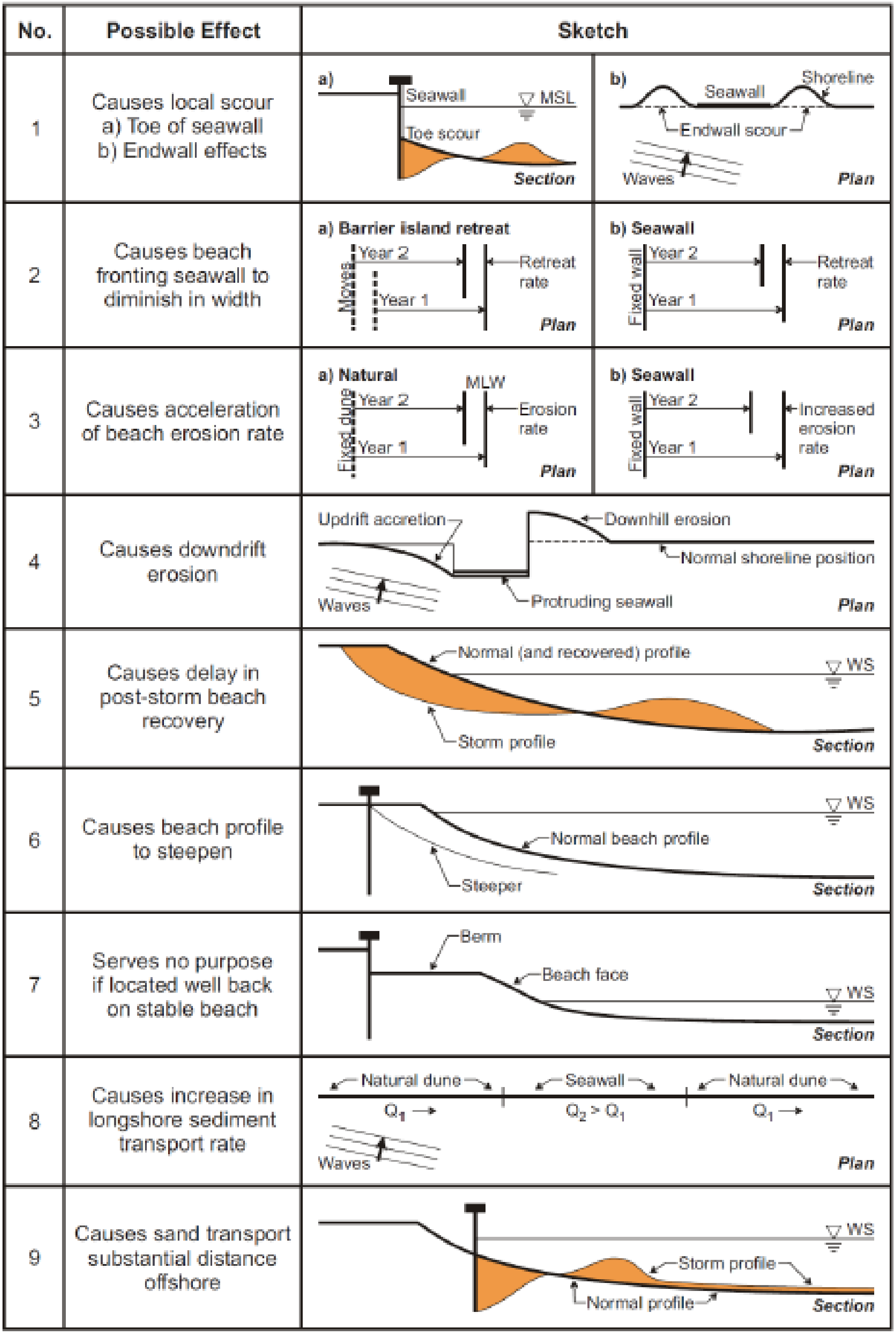
Page 8 of the (ACT, 2023) Geotechnical Report noted the following:

> "Further recommendations for the slope stabilisation include the following measures:
> …
> To limit future scour at the toe of the slope due to wave action, it is recommended that some form of wave dissipator be installed along the toe of the cliff. This could include large boulders (preferably strong, durable, volcanic rock), or some form of retaining wall. It is advised that the boulders or retaining wall are founded on bedrock to prevent undermining."

6. Impact of the works

6.1 Postulated seawall impacts

Basco (2004) and Dean (1986) listed nine postulated impacts of seawalls on beaches, as shown in Figure 6-1. A compendium of the findings of Basco (2004), Dean (1986) and CEM (EM 1110-2-1100 (Part V) 2003 p V-3-31) are shown in Table 6-1.



(Basco, 2004 based on Dean, 1986)

Figure 6-1 Postulated impacts of seawalls

Table 6-1 Postulated generic seawall impacts

Postulation	Description	True/false*	Further discussion
1a	Toe scour/erosion	Possibly	Section 6.2
1b	End effects/flanking	True	Section 6.3
2	Reduced beach width	True	Section 6.4
3	Increase in long-term recession	False	-
4	Downdrift erosion	True	Section 6.5
5	Delayed beach recovery	False	-
6	Profile steepening	False	-
7	Serves no purpose if landward	Possibly true**	Required for geotechnical stability
8	Increased longshore transport	False	-
9	Transports sand far offshore	False	-

* As defined in CEM (2003).
** Listed as False in CEM (2003), however, it notes that a seawall must interact with waves/water to act as a seawall, hence changed by WRL to "Possibly true".

6.2 Toe scour

The Coastal Engineering Manual (CEM, EM 1110-2-1100 (Part V) 2003 p V-3-32)) quoting numerous references noted the importance of a seawall's position on the active beach. It noted:

> *"Frontal impacts. Beach profile change, toe scour during storms and nearshore bar differences have been attributed to seawalls. Conventional wisdom has been that these impacts were due to wave reflection."*

Quoting numerous further references and physical model tests, CEM concluded:

> *"…that reflection is not a significant factor in profile change or toe scour. In the field, toe scour is more dependent on local, sediment transport gradients and the return of overtopping water (through permeable revetments or beneath walls) than a result of direct, cross-section wave action… conclusions also negate the common perception that sloping and permeable surfaces produce less effects than vertical, impermeable walls".*

After seawall construction, sand trapped behind the wall is not available for mobilisation and transport offshore and to adjacent beaches during and after storm events (Basco et al., 1997). This results in excess erosional stress along the front of the structure and on unprotected adjacent sections of the beach (CEM, 2006). Dean (1986) proposed the "approximate principle" relating the volume of toe scour at a wall to the volume that might be potentially scoured in the absence of that wall (Figure 6-2). This principle was verified in small and mid-scale physical model testing (Barnett et al., 1988; Hughes and

Fowler, 1990; Miselis, 1994) some of which found that the additional eroded volume was only 60% of the theoretical amount should the foreshore be erodible (i.e. no seawall present). However, it was not observed in field studies by Griggs (1990). Scaling limitations of the sediment means that most physical model tests should be primarily considered qualitative Kraus and McDougal (1996).

Irrespective of the uncertainty regarding generic toe scour for the North Denhams Beach site, the limited veneer of sand and generally cliffed nature of the back beach would limit any toe scour potential due to the presence of the cliff stabilisation works.

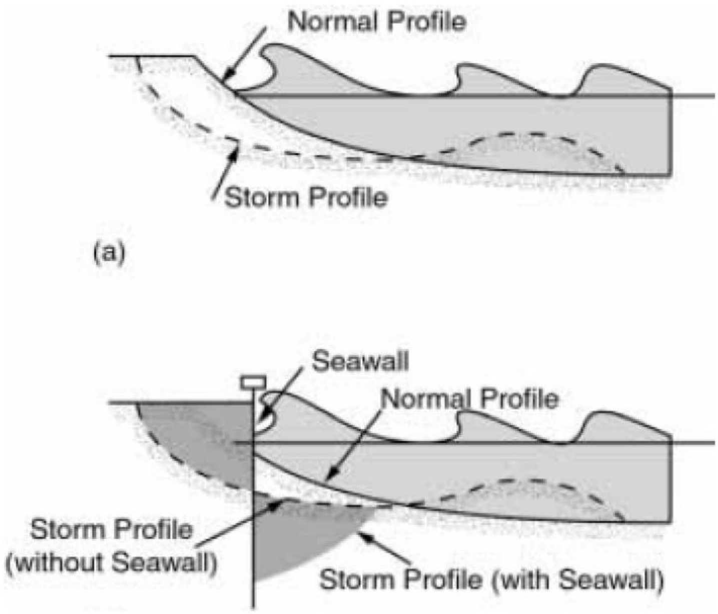


Figure 6-2 Dean postulation for toe scour

6.3 End effects

The drawing by Adhami Pender architecture (Job J000167; Drawing C00010; Issue DA-C; Date 11.10.24) indicates that the seaward extent of the works is a minimum of 5.385 m inside the seaward property boundary. A perusal of the latest (18 February 2025) aerial photo from NearMap indicates a setback from the seaward boundary consistent with the design value.

While the bedrock and slope are weak and weathered, they still do not erode in the manner of beach sand in response to waves and elevated water levels, and maintain a slope of 40 to 70 degrees from horizonal (ACT Geotechnical Engineers, 2023). The five properties to the south all have some form of gabion works protecting the seaward portion of their property from wave forces (and undermining of the cliff), and are thus not impacted by any perceived end effects from the subject property’s works.

The property to the north (217 Beach Road) has extensive geotechnical stabilisation works on its upper slope, and vegetation and fabric protection on its lower slope, but is somewhat unprotected from undercutting on the lower slope for an alongshore distance of up to 10 m, until the seaward boundary reaches the natural rock headland (Figure 4-4).

The expected quantum of erosion from an individual storm event is small, due to the rocky nature of the slope, however, the high consequence of cliff collapse through undercutting is noted. Without cliff stabilisation works, a landslide risk is present on the subject property (ACT Geotechnical Engineers, 2024). With previous landslides identified for 217 Beach Road, the risk of undercutting and further landslide collapse is likely with or without the cliff stabilisation works on the subject property. With the quantum of short term erosion being so small, the works on the subject property do not exacerbate the potential erosion at 217 Beach Road, but rather buttress its southern boundary, while its northern boundary is buttressed by the natural headland.

6.4 Beach width

The limited beach fronting the subject property is backed by a natural cliff. The cliff stabilisation works are more than 5 m landward of the private property boundary. As stated above, there is approximately 5 to 10 m of sand/pebble beach seaward of the private property boundary (10 to 15 m seaward of the cliff stabilisation works), and the works serve to stabilise the base of this cliff to prevent landslides.

6.5 Downdrift erosion

In a similar manner to the end effect discussion above, there is likely no net longshore littoral drift transport at this beach. There is only a small gap at 217 Beach Road in coastal protection works fronting the subject property, between the subject property's retaining wall and the natural headland to its north. The coastal protection works on the subject property and natural headland to the north are likely to buttress the slope at 217 Beach Road from both sides.

6.6 Impact of the works on use of the beach or headland - Instruction 3 (a)

The works do not materially limit the beach width beyond the status quo of the backing cliff, and protect the beach from landslide risk, noting that there is no public access to this beach via land. That is, the works do not limit use of the beach.

The works are remote from the surrounding headlands, so do not limit use of the headlands.

7. Design life and maintenance of the works - Instruction 3 (c)

The works are of semi-rigid construction and are above the intertidal zone

7.1 Design life

Further discussion is provided in Section 8. Australian Standard AS 4997-2005 guidelines for the design of maritime structures lists the following indicative design life, with 50 years adopted for the subject works:

- Temporary structure: 5 years
- Normal structure: 50 years
- Special structure: 100 years

7.2 Maintenance

Recent collection of information by WRL has yielded the following information, noting that most of this data is personal communication from professional engineers and asset managers, but is not comprehensively documented.

Common practice has been to assign an annual maintenance cost of 1% of the capital cost of the structure. This can be traced back to Gordon (1989). Reinforced concrete structures at the back of a beach which are only subject to infrequent wave forces are reported to have almost zero maintenance (apart from cleaning). Structures (especially boulders/rubble) in the intertidal zone exposed to frequent wave impacts are likely to have higher maintenance requirements and costs. Geotextile structures generally have higher maintenance (repair) requirements.

There is little documented information, however, the following maintenance costs (as a proportion of capital cost) are suggested:

- Rigid (concrete) structures at the back of the beach: 0% per annum
- Predominantly buried structures: 0% per annum
- Flexible (rock and concrete armour) structures exposed to occasional wave forces: 1% per annum
- Flexible (rock and concrete armour) structures exposed to frequent wave forces (intertidal zone): 2% per annum
- Geotextile structures exposed to occasional wave forces: 2% per annum
- Geotextile structures exposed to frequent wave forces (intertidal zone): 5% per annum

For the subject property, with a semi-rigid concrete structure above the intertidal level, the likely maintenance for a 50 year design life is zero.

7.3 Monitoring

Nevertheless, the following monitoring is recommended, which is broadly based on Oliver et al. (1988):

- Inspection of the works by a qualified engineer (coastal, structural or geotechnical) at least once per year.
- Additional inspection of the surrounding land by a qualified geotechnical engineer at least once per year, or as recommended by a qualified geotechnical engineer.
- An additional inspection of the works within 2 weeks of the significant wave height exceeding 5.5 m (slightly above a 1 year ARI event) on either the Port Kembla, Batemans Bay or Eden wave buoys (to allow for a buoy being out of operation).
- Development of an emergency action plan should the works be damaged or fail. This plan would require input from coastal, structural and geotechnical engineers.

8. Design standard and design event

8.1 Australian Standard AS 4997-2005

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 8-1. 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. AS 4997 recommends that the design water levels accompanying these waves should not be below Mean High Water Springs (MHWS).

Table 8-1: Annual Probability of Exceedance of Design Wave Events (source AS 4997-2005)

			Design Working Life (Years)			
Function Category	**Structure Description**	**Encounter Probability (a, b)**	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)	20 year ARI	50 year ARI	200 year ARI	500 year ARI
2	Normal structures	10%	50 year ARI	200 year ARI	500 year ARI	1,000 year ARI
3	High property value or high risk to people	5%	100 year ARI	500 year ARI	1,000 year ARI	2,000 year ARI

(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 Equation 3.1

(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%.

Based on WRL's interpretation of AS 4997-2005, the subject structure should be designed for a 50 year life and a 500 year ARI design event. Consideration of wave runup and overtopping is usually undertaken for 100 year ARI conditions for consistency with flood policies and the CMP.

8.2 Other standards

WRL has assumed that a range of design standards have been used by the designers of the works covering geotechnical and structural aspects of the works.

9. Input data

9.1 Bathymetry and topography

The NSW Department of Planning, Industry and Environment (NSW DPIE), provides topographic and bathymetric data based on Airborne LiDAR Bathymetry (ALB) technology conducted by Fugro Pty Ltd from July to December 2018. The bathymetric data was accessed through the ELVIS portal (https://elevation.fsdf.org.au/) and downloaded at a resolution of 5 m.

A transect location through the subject property is shown in Figure 9-1 with the transect shown in Figure 9-2.

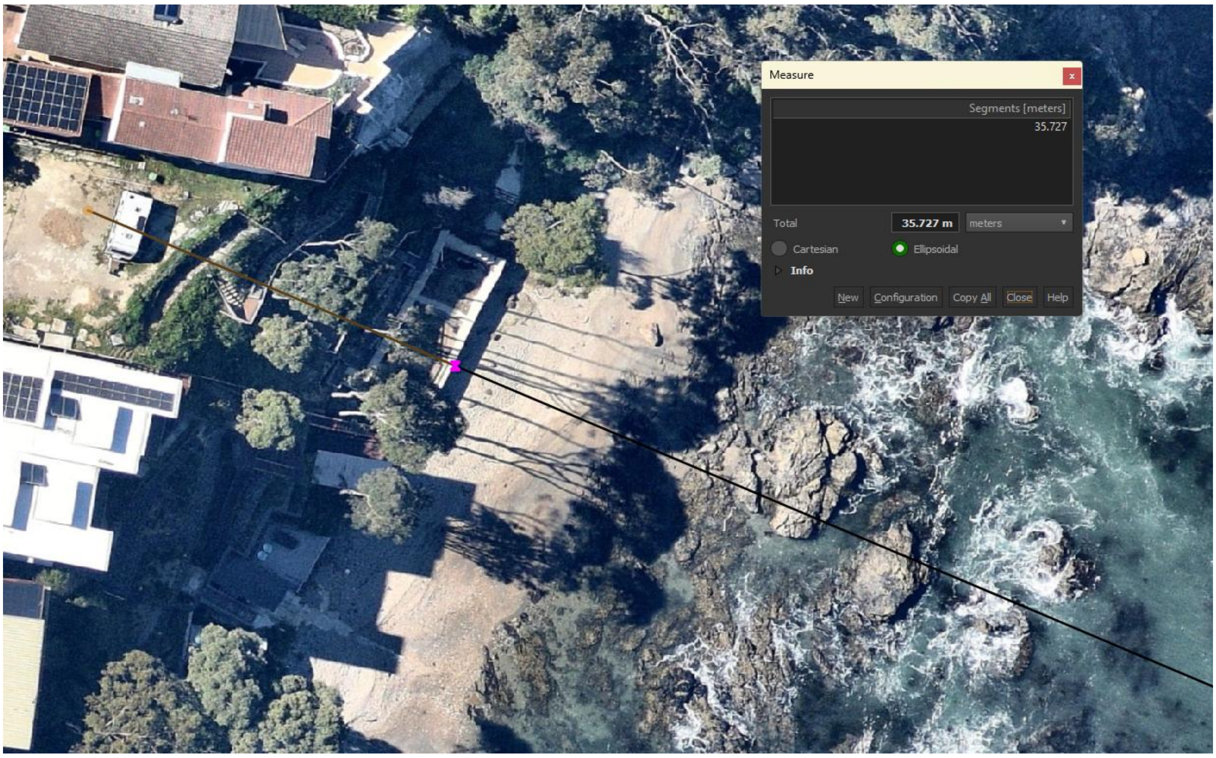


Figure 9-1 NSW marine LiDAR 2018 bathymetry and topography transect location

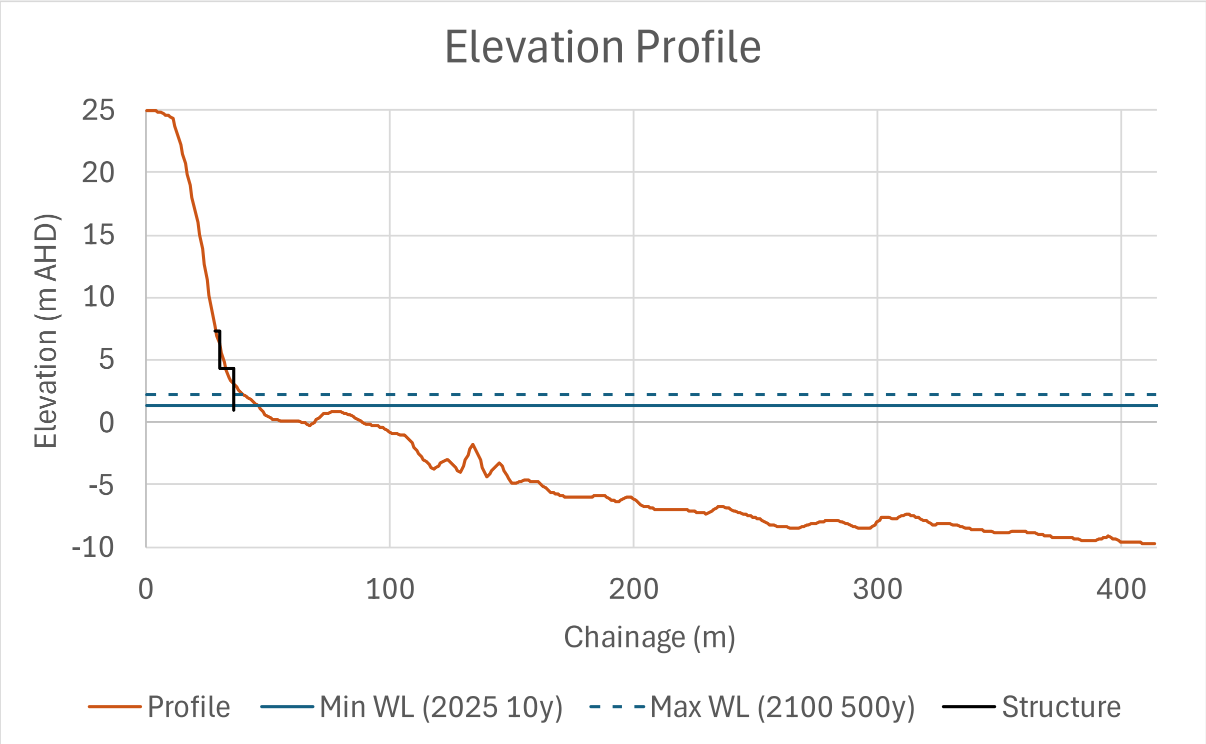


Figure 9-2 Bathymetry and topography transect

9.2 Water levels and waves

9.2.1 Water levels

The water levels used are shown in Table 9-1 based on the CMP and WRL (2017). Note that these values exclude wave setup and runup effects, which are calculated separately.

Table 9-1 Design water levels (from CMP) excluding wave setup and wave runup

ARI (years) or tidal plane	2025 design still water level (m AHD)
High High Water Solstices Springs (HHWSS)	0.92
Mean High Water Springs (MHWS)	0.61
Mean High Water (MHW)	0.51
Mean High Water Neaps (MHWN)	0.41
Mean Sea Level (MSL)	0.05
Mean Low Water Neaps (MLWN)	-0.31
Mean Low Water (MLW)	-0.41
Mean Low Water Springs (MLWS)	-0.51
Indian Spring Low Water (ISLW)	-0.74
10	1.33
100	1.43
500 (1)	1.51

(1) These water level values were extrapolated by WRL using a log-linear fit.

9.3 Cross section of works at site

The cliff stabilisation works are documented in the following drawings from adhami pender architecture (APA):

- Job J000167; Drawing C000 ; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0001; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0002; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0003; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0004; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0005; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0006; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0007; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0008; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0009; Issue DA-C; Date 11.10.24

- Job J000167; Drawing C0010; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0011; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0012; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0013; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0014; Issue DA-C; Date 11.10.24
- Job J000167; Drawing C0015; Issue DA-C; Date 11.10.24

An extract from Drawing C0011 from the above set is shown in Figure 9-3 below. Along with Drawing C0010, this indicates the following key variables:

- Elevation of crest of seaward wall: 4.3 m AHD
- Elevation of crest of landward wall: 7.35 m AHD
- Distance between the rear face of seaward wall and front face of landward wall: 4.8 m

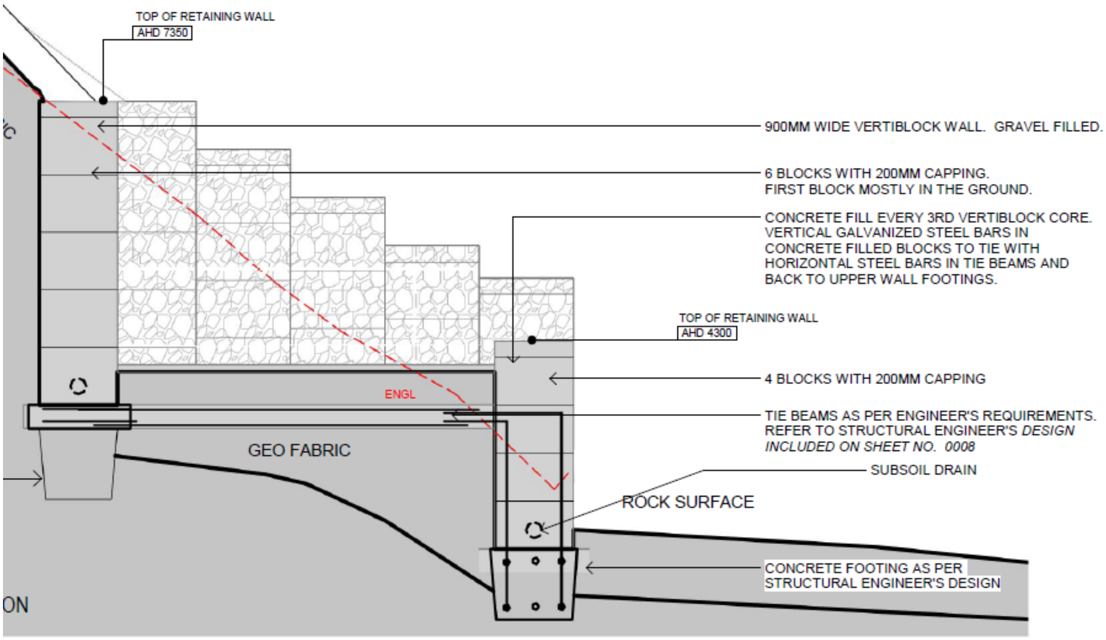


Figure 9-3 Extract from Drawing C0010 showing cross section of structure

9.4 Future sea level rise

This report is for a design working life of 50 years. Based on a nominal construction date of 2025, the design life of the structure extends to 2075.

The IPCC reports provide global mean sea level rise projections for five Shared Socioeconomic Pathways (SSPs), with each SSP capturing different emissions scenarios. The sea level rise values adopted for the CMP (Rhelm, 2023) are shown in Table 9-2, with mid-decade values interpolated by WRL. Although they predate the latest IPCC, they are broadly consistent with the latest IPCC AR6 (2021) median estimate for the SSP5-8.5 (Very High emissions scenario – medium confidence), using the NASA sea level projection tool (NASA, 2025) for Jervis Bay.

Table 9-2 Sea level rise projections (CMP, relative to 2017)

Planning Period (year)	Sea Level Rise (m)[(1)]
2050	0.22
2065	0.33[(2)]
2075	0.43[(2)]
2085	0.54[(2)]
2100	0.71

(1) SLR values were adjusted to 2025 as IPPC (2021) SLR values are relative to 2020.

(2) 2065, 2075 and 2085 SLR values were interpolated by WRL.

9.5 Waves

The offshore wave climate for the Batemans Bay region is characterised by a moderate to high energy, however the offshore reefs and emergent rock shelves fronting the subject property result in a generally mild wave climate at the subject cliff stabilisation works. Estimates for 100 year ARI (Average Recurrence Interval) non-directional offshore waves (Glatz et al., 2017) and directional extreme offshore waves (Shand et al., 2011a) for Batemans Bay are provided in Table 9-3. For this analysis, unrefracted offshore waves from the east to south-east wave direction were used to quantify wave runup.

Table 9-3 Offshore directional extreme wave conditions at Batemans wave buoy

(Source: Glatz et al., 2017 and Shand et al., 2011a)

Offshore direction		**1 hour Hs**		
Label	Degrees	10 year ARI	100 year ARI	500 year ARI
All directions [(1)]		6.3	7.7	8.7
NE[(2)]	45		6.2	
ENE[(2)]	67.5		6.2	
E[(2)]	90		7.3	
ESE[(2)]	112.5	6.3	7.7	8.7
SE[(2)]	135		7.7	
SSE[(2)]	157.5		7.7	
S[(2)]	180		7.3	

(1) These values were reported in Galtz et al. (2017).

(2) These values were reported in Shand et al. (2011a).

Offshore peak wave period for design conditions from the Eden wave buoy (extrapolated for 500 year ARI), (Shand et al., 2011b) are provided in Table 9-4, with these being broadly applicable for Batemans Bay.

Table 9-4 Corresponding peak wave period Tp for ARI

ARI (years)	Offshore Tp (s)
10	12.5
100	13.3
500	13.8

10. Wave modelling

As part of the Eurobodalla wide coastal hazard study (Coghlan et al., 2017), WRL undertook a SWAN wave transformation (predominantly refraction, shoaling and friction) model study (Figure 10-1). An example of output for the Batemans Bay domain for 100 year ARI conditions from the east-south-east is shown in Figure 10-2. The output from this model was utilised for a representative location for the North Denhams Beach site at approximately the -10 m AHD isobath. Offshore conditions (derived from analysis of the Batemans Bay wave buoy) are shown in Table 9-3.

These waves were transformed to a representative offshore location with SWAN, then propagated to the shore using the Dally et al. (1985) surf zone model. Two transects were used as shown in Figure 10-3, with a transect fronting the subject property (greater wave dissipation, greater wave setup, lower wave height), and a transect through the keyhole portion of the beach (lower wave dissipation, lower wave setup, higher wave height) examined for comparison. This letter only presents the modelling for the transect fronting the subject property.

The profile and cross shore wave decay for multiple wave conditions are shown in Figure 10-4.

Local wave heights, wave setup and the setup water level at a typical plunge distance (10 m) from the wall are shown in Table 10-1.

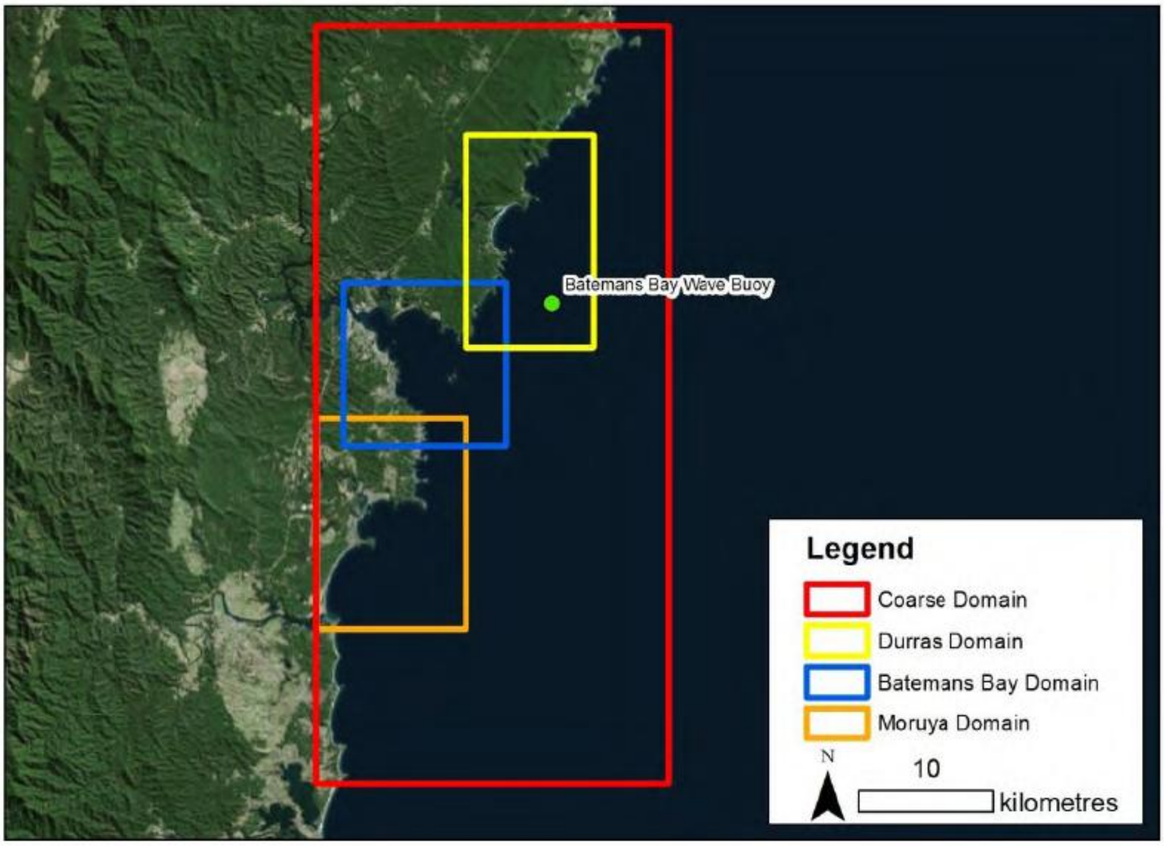


Figure D-1: SWAN model domains

Figure 10-1 Eurobodalla SWAN wave modelling

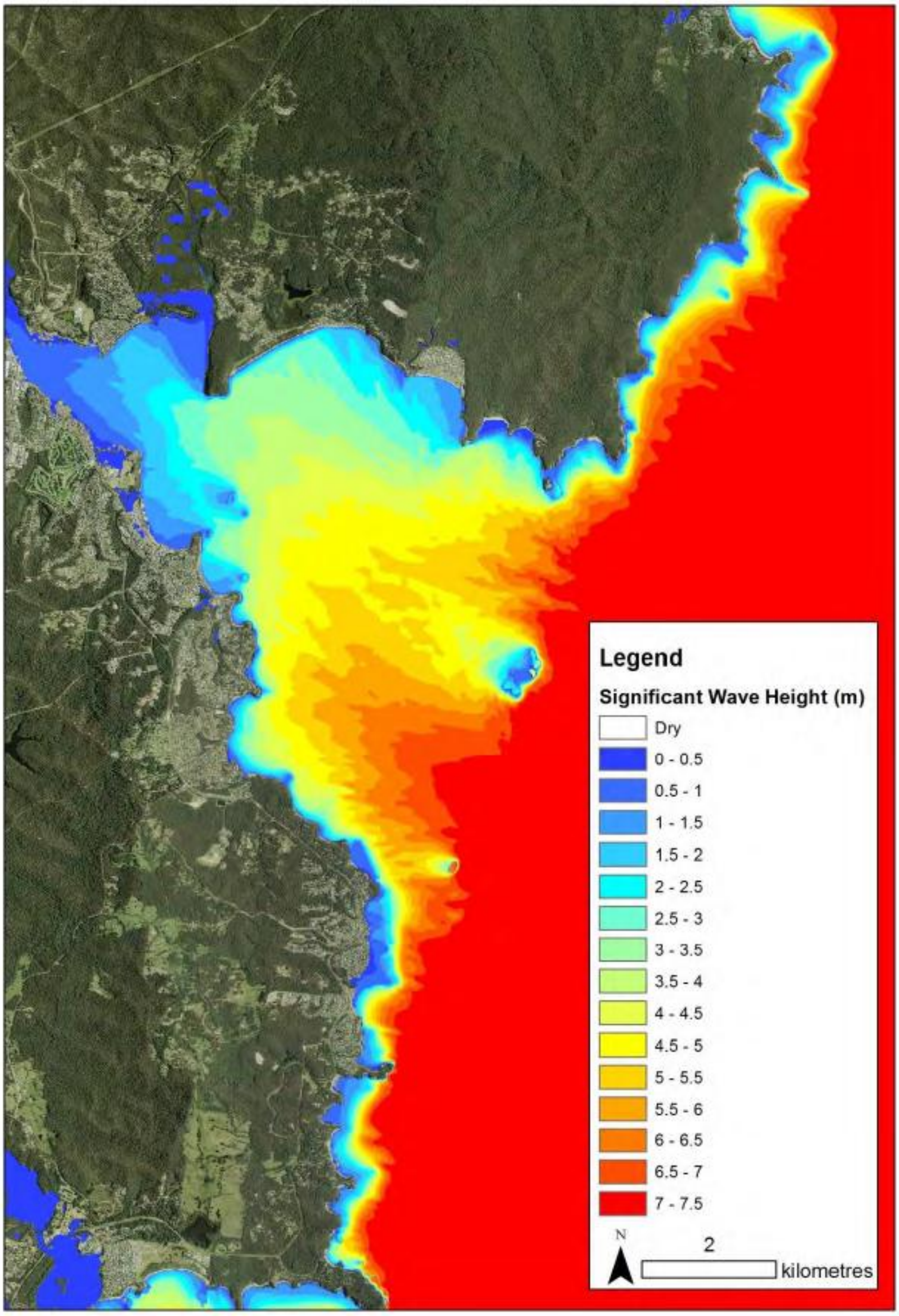
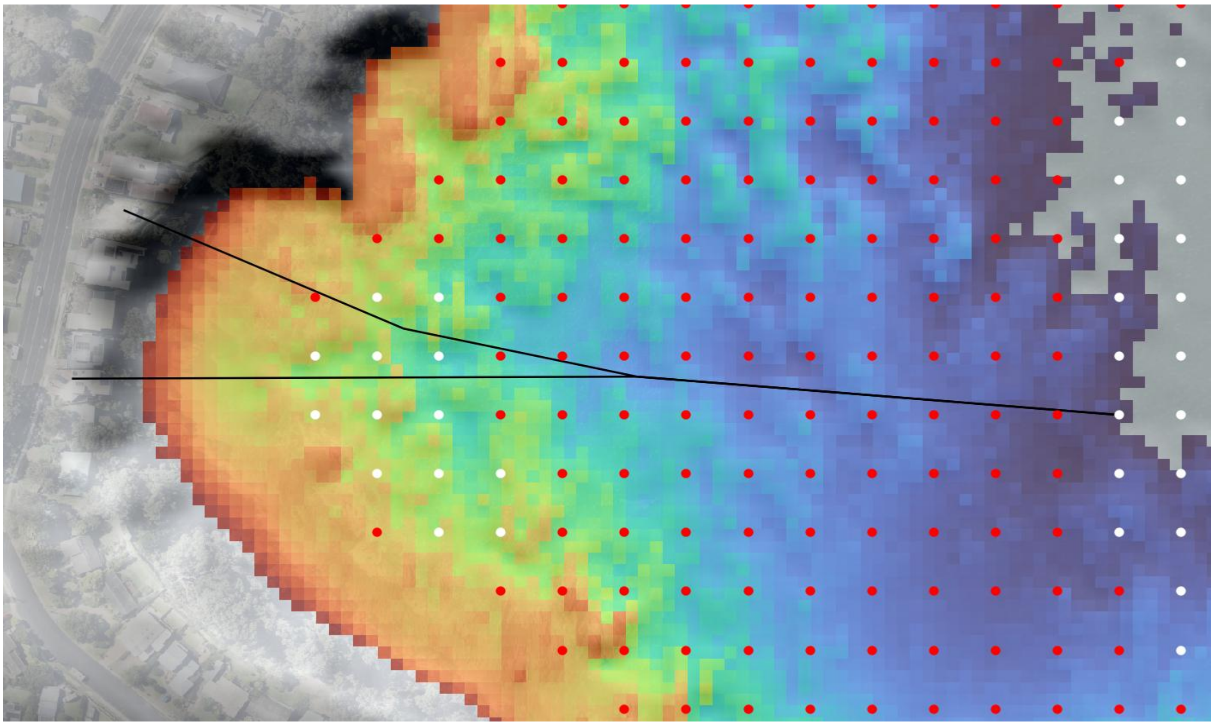
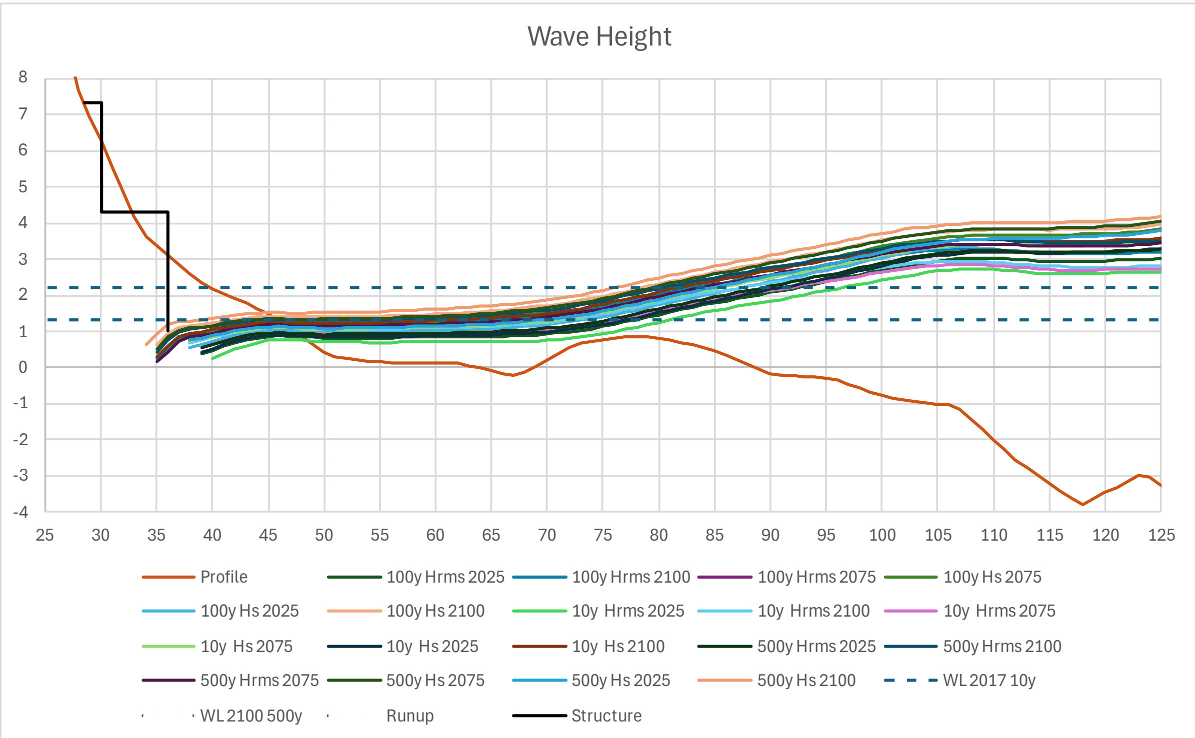


Figure 10-2 Example SWAN model output for Batemans Bay model



Northern transect applies for subject property

Figure 10-3 Nearshore wave transects for subject property



Note: Vertical dotted line is nearshore base of slope used for runup modelling.

Figure 10-4 Transect for subject property and wave decay

Wave runup height was estimated for 10, 100 and 500 year ARI conditions for 2025, 2075 and 2100 using Equation 6.2 in the EurOtop (2018) Overtopping Manual. The results are shown in Table 10-1, with some outputs for 10 and 500 year ARI omitted for clarity. Note that splash, spray and some individual waves may exceed this design runup level – particularly during strong onshore winds.

As noted above, the elevation of the crest of the seaward wall is 4.3 m AHD, and the elevation of the crest of the landward wall is 7.35 m AHD. For 100 year ARI conditions, the seaward wall would not be overtopped with present day conditions, but would be slightly overtopped in 2075 (0.43 m sea level rise). The landward wall would not be overtopped in 100 year ARI conditions in 2100.

Table 10-1 Wave and water level conditions for structure (allowing for 10 m plunge distance)

Year	SLR (m)	ARI (years)	SWL (m AHD)	Wave setup (m)	Setup water level (m AHD)	Local Hs (m)	R2% (m AHD)
2025	-	10	1.33				4.2
2025	-	100	1.43	1.03	2.46	0.97	4.7
2025	-	500	1.51				5.1
2075	0.43	10					5.3
2075	0.43	100	1.86	0.94	2.80	1.24	6.0
2075	0.43	500					6.4

11. Wave forces

Wave forces are provided in separate advice to the Structural Engineer.

12. Comments on stability of wall

12.1 Failure modes

For seawalls generally, and particularly rigid seawall structures, the main failure modes can be classified as:

- Undermining, in which the sand or rubble toe level drops below the footing of the wall, causing the wall to subside and collapse in the hole
- Sliding, in which the wall moves away from the retained profile
- Overturning, in which the wall topples over
- Slip circle failure, in which the entire embankment fails
- Loss of structural integrity, due to wave impact
- Erosion of the backfill, caused by wave overtopping, high water table levels, or leaching through the seawall
- Corrosion, abrasion and impact damage
- Outflanking and end scour

12.2 Undermining

Following work completed in the UK in the late 1980s and early 1990s, it was documented in CIRIA (1991, 2007) that “around 34% of seawall failures arise directly from erosion of beach or foundation material, and that scour is at least partially responsible for a further 14%”. That is, approximately 48% of documented seawall failures in the UK were either partly or solely attributed to toe erosion.

With the subject works founded on bedrock, failure through erosion of the toe over the planning timeframe is not possible.

12.3 Sliding

This is covered within the geotechnical and structural engineering design of the wall.

12.4 Overturning

This is covered within the geotechnical and structural engineering design of the wall.

12.5 Slip circle failure

This is covered within the geotechnical and structural engineering design of the wall.

12.6 Structural integrity

The wall has been designed by a structural and geotechnical engineer. Wave forces were not considered in this design.

12.7 Erosion of backfill

The design crest elevation of the seaward wall is 4.3 m AHD. The landward wall has a design crest elevation of 7.35 m AHD. The area between the two walls is backfilled with gravel and parts of the two walls are joined by reinforced concrete tie beams. The landward wall crest is above the runup levels. The nature of the fill between the two walls makes erosion unlikely and the stability of the two walls is not dependent on this backfill.

12.8 Corrosion, abrasion and impact damage

Corrosion is covered by the structural design of the wall, noting that the vertical steel reinforcement is within the cores of the blocks. Ballistic damage from rocks during storms is rarely considered for coastal engineering design in Australia. All parts of the two walls are backed by earth/backfill, meaning that such impacts are unlikely to cause the wall to fail.

12.9 Outflanking and end scour

There are landward returns on the walls which would prevent outflanking. See also Section 6.3. The property to the south has embankment protection with gabions. The property to the north has a range of embankment protection, with the lower portion relatively unprotected, but with less than 10 m between the works on the subject property (217A Beach Road) and the natural headland to its north.

13. Summary

Thank you for the opportunity to provide this advice. Please contact James Carley or myself should you require further information.

Yours sincerely,

James Carley	**Dr Francois Flocard**
Principal Coastal Engineer	Director, Industry Research

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