Geobag walls at Clarkes Beach, Byron Bay

WRL TR 2021/12, September 2021

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UNSW Water Research Laboratory

UNSW SYDNEY

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Project details

Report title	Geobag walls at Clarkes Beach, Byron Bay
Authors(s)	J T Carley and F Flocard
Report no.	2021/12
Report status	Final
Date of issue	September 2021
WRL project no.	2020065
Project manager	J T Carley
Client	Crown Lands | Department of Planning, Industry and Environment
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Client contact	Catherine Knight catherine.knight@crownland.nsw.gov.au
Client reference	#

Document status

Version	Reviewed by	Approved by	Date issued
Preliminary Draft			07/06/2021
Draft	GPS	GPS	27/08/2021
Final	GPS	GPS	23/09/2021

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Executive summary

Clarkes Beach and Main Beach Byron Bay have experienced beach erosion in 2020 and 2021 that has been described as “unprecedented”. This has resulted in the loss of vegetation, closure of many beach access ways, exposure of normally buried rocks and reefs, diminished beach amenity, unearthing of indigenous artefacts, and the potential collapse of built assets such as those within the Reflections Holiday Park, Clarkes Beach, Byron Bay (Reflections) and the Beach Byron Bay Cafe building (the Cafe, also commonly referred to by its previous name of Clarkes Beach Cafe). The Cafe was determined by coastal, geotechnical and structural engineers to be at imminent risk of collapse onto the beach. Interim geobag seawalls were designed and constructed so as to prevent this collapse, while longer term management is being developed.

Interim geobag (0.75 m^3) walls were constructed fronting Reflections in July 2019 in two lengths of approximately 70 m each, separated by a short length (22 m) comprising a stormwater pipe, degraded gabions, coffee rock, boulders and cobbles, with a total effective length of approximately 160 m. In October/November 2020, an approximately 90 m long geobag wall was constructed in front of the Beach Byron Bay Cafe. The new wall is contiguous with and westward of the Reflections geobag wall. An additional course of geobags was added to a large section of the crest of the Café geobag wall in December 2020 in response to a large storm wave event that overtopped the geobag wall and eroded some of the backfill. The wall was offset seaward of the base of the erosion escarpment to provide geotechnical stability to the Café building and the sand dune. It was backfilled with compatible sand at a stable angle of repose.

The geobag works fronting the Café were originally proposed for a 90 day design life, so that additional planning regarding the future of the Café could be made. This report examines a pathway to extend the design life by an estimated 5 years, to allow additional time to plan for the future of the café.

This report provides an assessment of the individual and combined impact of the geobag works (that is, both the Reflections and Cafe works) on coastal processes over the estimated 5 year design life, and the monitoring and maintenance requirements that will be associated with the geobag works. The report will inform the development application and environmental impact statement that is being submitted for the works under NSW planning legislation.

Numerous studies have quantified the coastal processes and coastal hazards prevailing at Clarkes Beach and Byron Bay since 1978. Many of these have been more focussed on Belongil, where more than 50 houses are located within the hazard zone. All studies confirm that most of the Byron Bay embayment is undergoing long term recession, however, this involves long periods of erosion and accretion, with sand slugs travelling through the system from east to west. The long term recession is partially due to the southward flowing East Australian Current transporting sand into deeper water to the south-east, thereby removing sand from the active littoral system. Ongoing recession due to sea level rise is also likely to be occurring.

The most recent coastal hazard definition study was undertaken in 2013, and utilised the accreted beach state from 2007 as the baseline for calculations. Therefore, 2021 is now approximately 33% of the way from the then “present” hazard lines (2007) and the 2050 hazard lines. This WRL report has produced revised “present” hazard lines for 2021. The progression towards 2050 on a receding coast, accompanied with sea level rise can account for much of the recent “unprecedented” erosion on Clarkes Beach and Main Beach.

Long term measured wave buoy data (Byron Bay wave buoy), measured tide gauge data (Coffs Harbour and Brunswick Heads) and sophisticated wave transformation modelling estimated extreme waves at the geobag structure. The stability of the geobags was estimated using techniques developed by WRL and accepted in international coastal engineering literature and practice.

The geobag structures are underlain by a reef/rock layer which limits vertical scour in their vicinity. With ongoing beach accretion being evident during 2021, sand fronting the geobags reduces the wave height and force able to impact the geobags. If the beach scours down to the reef/rock layer, WRL estimated that more than 2% of the geobags would be displaced in a 5 year average recurrence interval (ARI) wave event, necessitating repairs. Overtopping may also erode some of the backfill sand. Detailed calculations found that the waves that impacted the geobags during the December 2020 storm event were approximately 1 to 2 year ARI. The geobag wall was undamaged, however, wave overtopping eroded some of the backfill sand, which was subsequently topped up.

The impacts of the geobag works on coastal processes were also assessed. The end effects observed to date are minor. The sand retained by the works and therefore withheld from the littoral system is less than 1% of the quantity of other littoral processes, and is far less than even the incidental/accidental removal of beach sand by people visiting the beach. Potential seawall end effects extend into two beach access points. Collaboration with Byron Shire Council is proposed to ensure preservation of beach access.

Until such time that the interim works can be removed, management of the impacts of the works is best undertaken through the following means:

- Management of public safety risk through regular inspection of the beach and dune (Section 8), removal of vegetation at risk of imminent collapse, grading of the erosion scarp to a maximum gradient of 1V:1.5H (34°)
- Establishment of a rolling easement of vegetation, through additional revegetation to replace that lost due to erosion/recession within the end effect area
- Restoration or consolidation of neighbouring beach accesses in consultation with Byron Shire Council
- Sand management through importation of nourishment sand or beach scraping in conjunction with Byron Shire Council

Principles and options for ongoing monitoring and maintenance are presented, with a detailed program to be developed as a condition of approval. Extending the design life from 90 days to 5 years means that additional maintenance may be required and there is a moderate chance of damage requiring repairs or reconstruction.

Within the report, an engineering opinion is provided regarding how the works comply with Section 27 of the NSW Coastal Management Act 2016.

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1 Introduction

Interim geobag (0.75 m^3) walls were constructed fronting Reflections Holiday Park, Clarkes Beach, Byron Bay (Reflections) in July 2019 in two lengths of approximately 70 m each, separated by a short length (22 m) comprising a stormwater pipe, degraded gabions, coffee rock, boulders and cobbles, with a total effective length of approximately 160 m. Figures and details are shown in subsequent sections of this report.

In October/November 2020, a 90 m geobag wall was constructed in front of the Beach Byron Bay Cafe (the Cafe, also commonly referred to by its previous name of Clarkes Beach Cafe). The new wall is contiguous with and westward of the Reflections geobag wall. An additional course of geobags was added to a large section of the crest of the Cafe geobag wall in December 2020.

The geobag works fronting the Cafe were originally proposed for a 90 day design life, so that additional planning regarding the future of the Cafe could be made. This report examines a pathway to extend the design life by an estimated 5 years, to allow additional time to plan for the future of the Cafe. Due to the contiguous nature of the Cafe and Reflections walls, many coastal planning issues are common to both sites.

This report provides an assessment of the individual and combined impact of the geobag works (that is, both the Reflections and Cafe works) on coastal processes over the estimated 5 year design life, and the monitoring and maintenance requirements that will be associated with the geobag works. The report will inform the development application and environmental impact statement, that is being submitted for the works under NSW planning legislation.

The main areas covered in the report include:

- Coastal processes and hazards at Clarkes Beach and within the Byron Bay embayment
- The stability of the geobags under wave forces
- Impacts of the geobags on coastal processes and management of the impacts over the life of the works
- Monitoring and future removal of the geobags

2 Interim geobag walls

2.1 Reflections geobags

Interim geobag (0.75 m^3) walls were constructed fronting Reflections Holiday Park in July 2019 in two lengths of approximately 70 m each (Figure 2.1), separated by a short length (22 m) comprising a stormwater pipe, degraded gabions, coffee rock, boulders and cobbles, with a total effective length of approximately 160 m.

2.2 Cafe geobags

Erosion continued during 2020 such that coastal engineers (WRL) and structural/geotechnical engineers (Ardill Payne) determined that the Cafe building was at imminent risk of collapsing down the embankment onto the beach. A new 90 m long geobag wall was consequently constructed in front of the Cafe in October-November 2020 (Figure 2.1, Figure 2.2), which abutted the western end of the Reflections geobags. The two geobag structures now comprise a near contiguous structure of about 250 m in length, with the eastern end of the Cafe geobags tied in with the western end of the Reflections geobags. Photos are shown in Figure 2.3 and Figure 2.4.

The cross shore geometry of the geobags was determined through collaboration between coastal engineers (WRL) and structural-geotechnical engineers (Ardill Payne). Plans and cross sections of the Cafe geobags are shown in Figure 2.5, noting that due to wave overtopping during large waves and elevated water levels, a significant portion of the crest was raised by one course of geobags in December 2020 (Figure 2.6).

Due to the urgency of the works and the initial 90 day design life, detailed coastal engineering calculations were not undertaken at the time of initial design/construction, but have subsequently been undertaken (Section 6). The geobag size (0.75 m^3) and crest elevation (2.0 to 2.5 m AHD) were based primarily on engineering experience/judgement, including consideration of nearby comparable functional geobag structures throughout the Byron Bay embayment. The cross shore position of the geobags was set seaward from the base of the dune to provide acceptable geotechnical stability to the Cafe building, as determined by Ardill Payne.

A geotextile was placed under the geobags. Based on the publication of Coghlan et al (2009), a double layer of geobags were used, with the crest bags added in December 2020 comprising only a single layer. Backfilling of the geobags and the steep face of the dune was undertaken through the importation of compatible sand. Initial construction involved the importation and placement of 3,035 tonnes (approximately 1,900 m^3 in situ), followed by 1,289 tonnes (approximately 800 m^3 in situ) as a top up in December 2020, giving a total placed sand volume of 4,324 tonnes (approximately 2,700 m^3 in situ).

Figure 2.1 Foreshore features 4 July 2020 – prior to Café geobags (Nearmap)

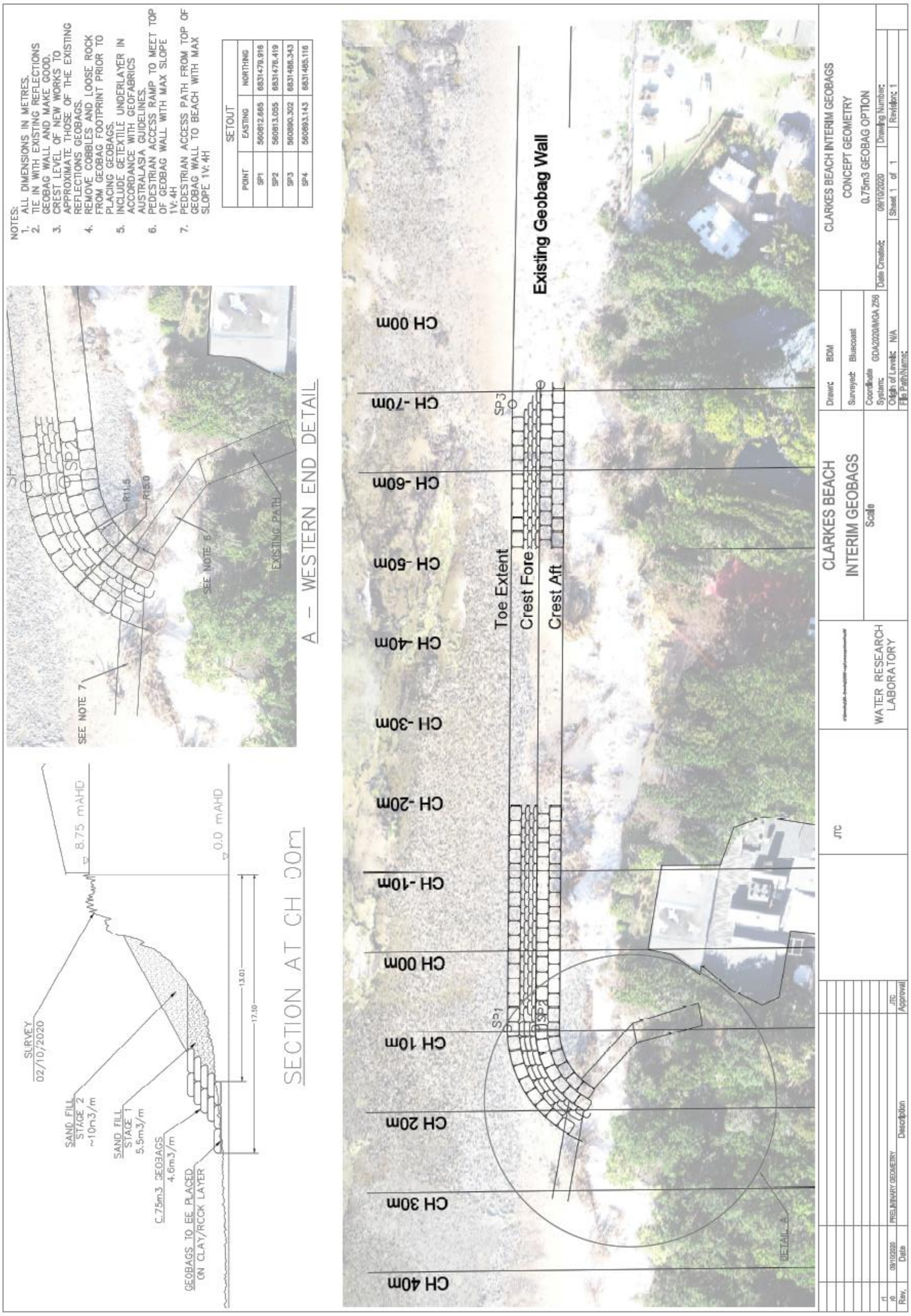
Figure 2.2 Foreshore features 26 November 2020 (Nearmap)

(Photo: Catherine Knight)

Figure 2.3 Geobag walls fronting Reflections Holiday Park and Cafe December 2020

(Photo: Catherine Knight)

Figure 2.4 Geobag walls fronting Cafe December 2020



Note: One single width course was added to a significant portion of the crest in December 2020

Figure 2.5 Plan and cross section of Cafe geobags – initial design

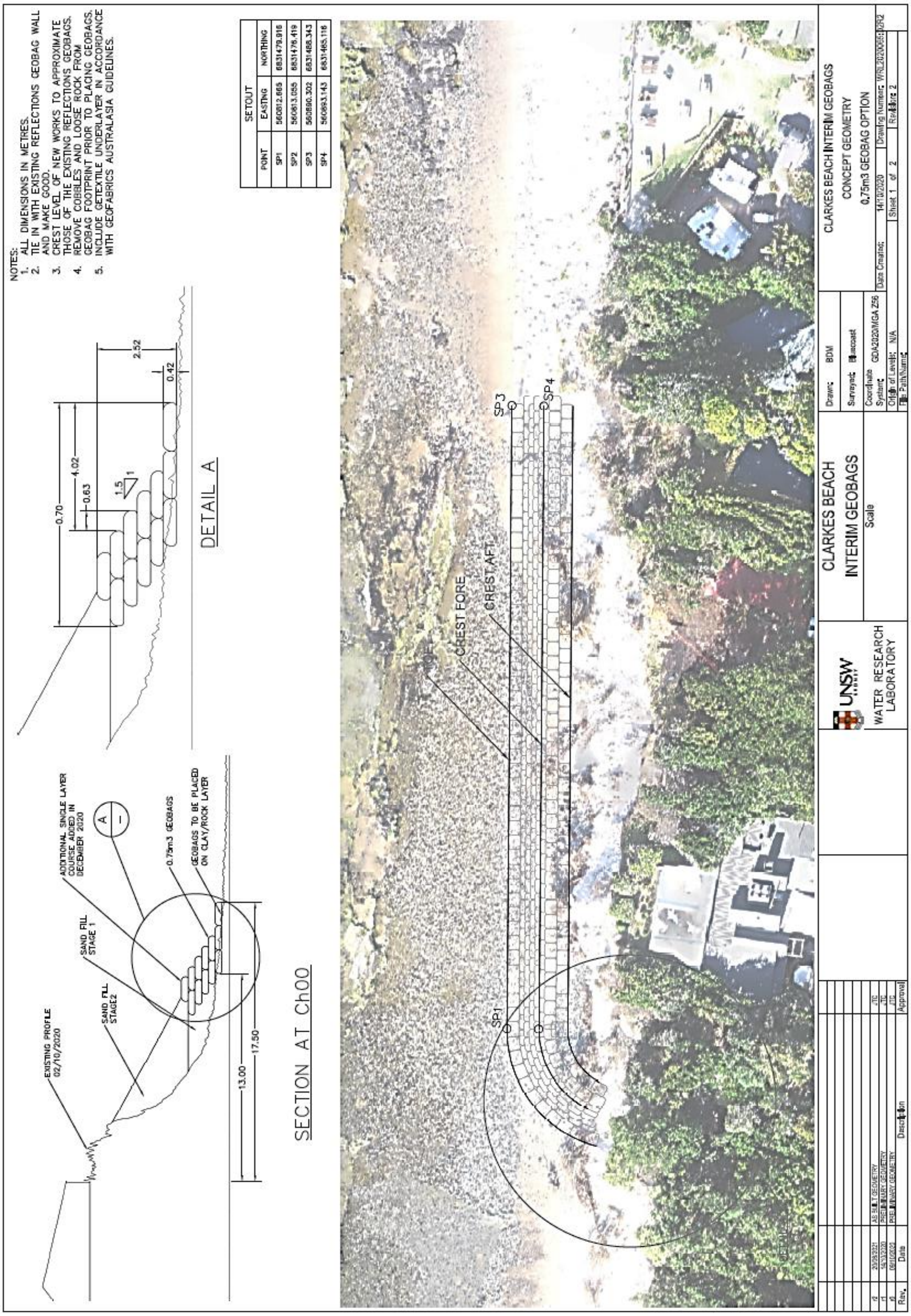


Figure 2.6 Plan and cross section of Cafe geobags – as modified

3 Coastal processes, sand transport and coastal hazards

3.1 Previous studies

There have been numerous studies of the coastal processes, sand transport, coastal hazards and coastal management for Byron Bay. Many of these have focussed on the Belongil area where approximately 50 houses are estimated to be vulnerable to present day coastal hazards.

The major studies have been:

- PWD (Gordon et al., 1978)
- WBM (2002)
- Gordon (2011)
- BMT WBM (2013)
- WRL (Carley et al, 2016)
- Patterson (2010)
- Goodwin et al. (2013)
- Carley et al. (2017)
- Murray/GCCM (2020)

3.2 Sand transport

3.2.1 PWD (1978)

PWD (Gordon et al., 1978, Figure 12.5) is reproduced in Figure 3.1 of this WRL report. The main features of this (littoral drift rates) are:

- 65,000 m^3/year travels northward along Tallow Beach
- 50,000 m^3/year is lost to the south-east of Cape Byron due to the action of the East Australian Current, and is deposited into the Cape Byron sand lobe
- 15,000 m^3/year bypasses Cape Byron and travels westward from Wategos Beach towards Main Beach
- The littoral drift rate increases to 80,000 m^3/year at Belongil Creek
- The littoral drift rate increases to 120,000 m^3/year at Brunswick Heads

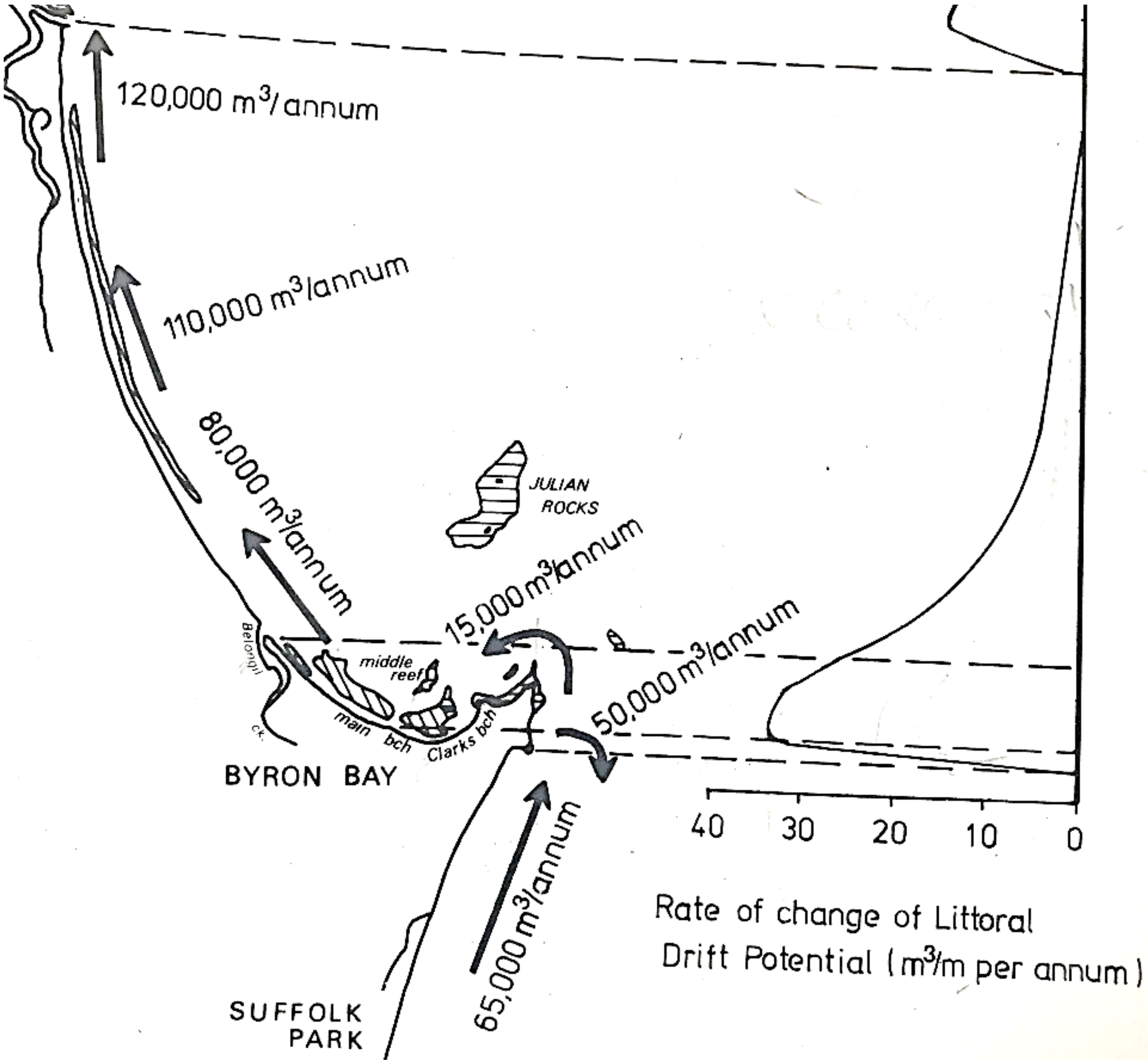
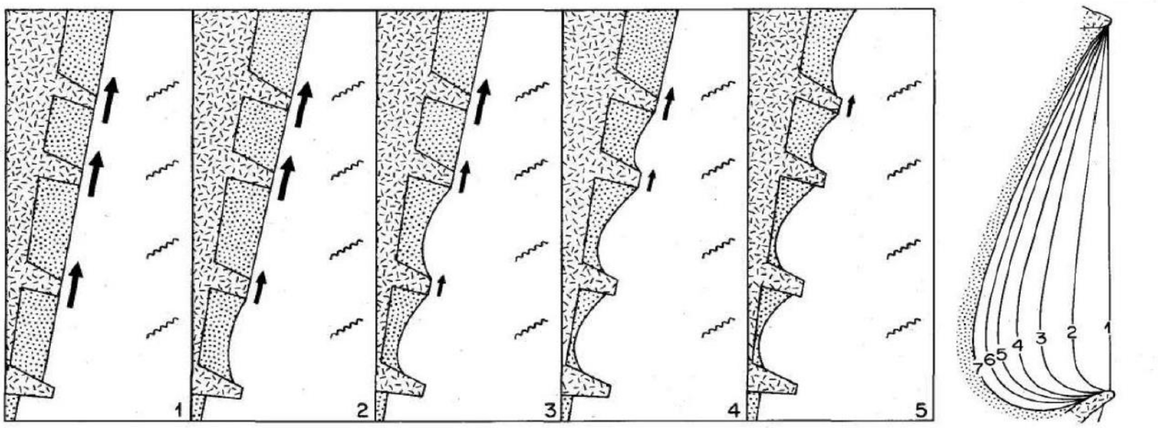


Figure 3.1 Sand transport estimates (PWD, Gordon et al, 1978)

3.2.2 Stephens et al. (1981)

The impact of the differential littoral drift described above is for the planform of the bay to deepen, as shown in Stephens et al. (1981), reproduced in Figure 3.2. With its location in the south-eastern hook of the Byron Bay embayment, the Clarkes Beach area is vulnerable to long term recession in the Stephens illustration.



Source: Stephens et al., (1981)

Figure 3.2 Coastal evolution on a littoral drift coast

3.2.3 BMT WBM (2013)

BMT WBM (2013) presented a conceptual sand transport pattern (Figure 3.3) which considered computer modelling by Patterson (2010). This estimated much higher littoral drift quantities than PWD (1978), including cross embayment transport, but still indicated a deficit of 50,000 m^3/year in the bay.

Figure 3.3 Conceptual sand transport (BMT WBM, 2013)

3.2.4 Goodwin et al. (2013)

Goodwin et al. (2013) presented a graphic of sand transport in the Byron Bay embayment (Figure 3.4). They estimated that approximately half of the longshore transport was nearshore littoral drift and half was cross embayment

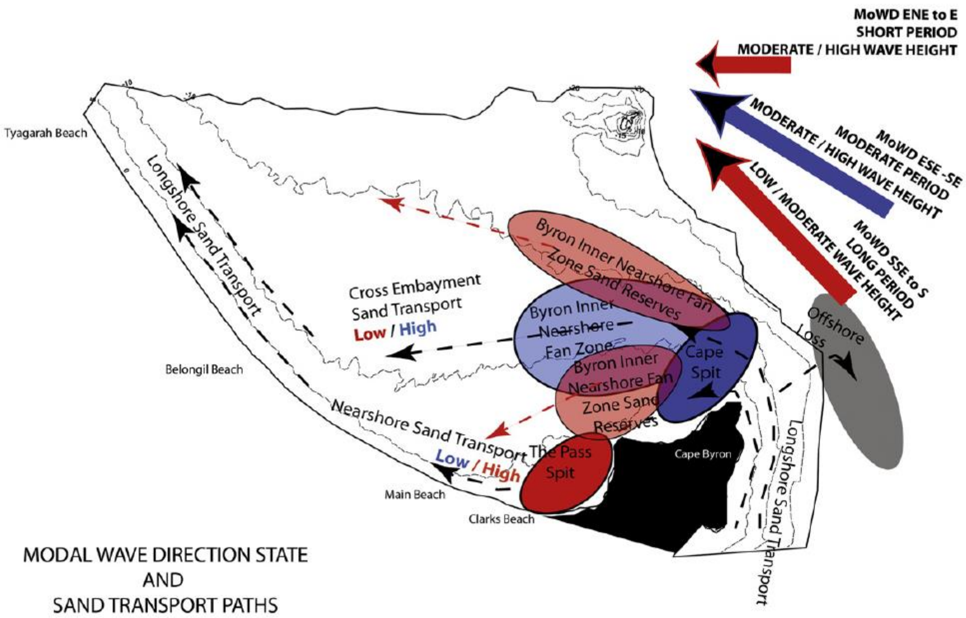


Fig. 9. Conceptual diagram of the relative headland bypass pathways, sand deposition and transport for two wave climates. The blue (red) colours represent the sediment transport patterns for a unimodal south-easterly (bimodal east and south-southeasterly) wave climates. Note that the bypass strand fluctuates between the 10 m and 15 m depth contours for unimodal or bimodal wave climates. The nearshore (cross-embayment) path dominates for east and south–south-easterly (south-easterly) modal wave direction. Higher wave energy results in a stronger outer bar bypassing system and enhanced sand deposition on the Byron Inner Nearshore Lobe.

Figure 3.4 Sand transport pathways (Goodwin et al, 2013)

3.2.5 Gordon (2011)

Gordon (2011) provided the following description of sand transport in the vicinity of Cape Byron and into the Byron Bay embayment.

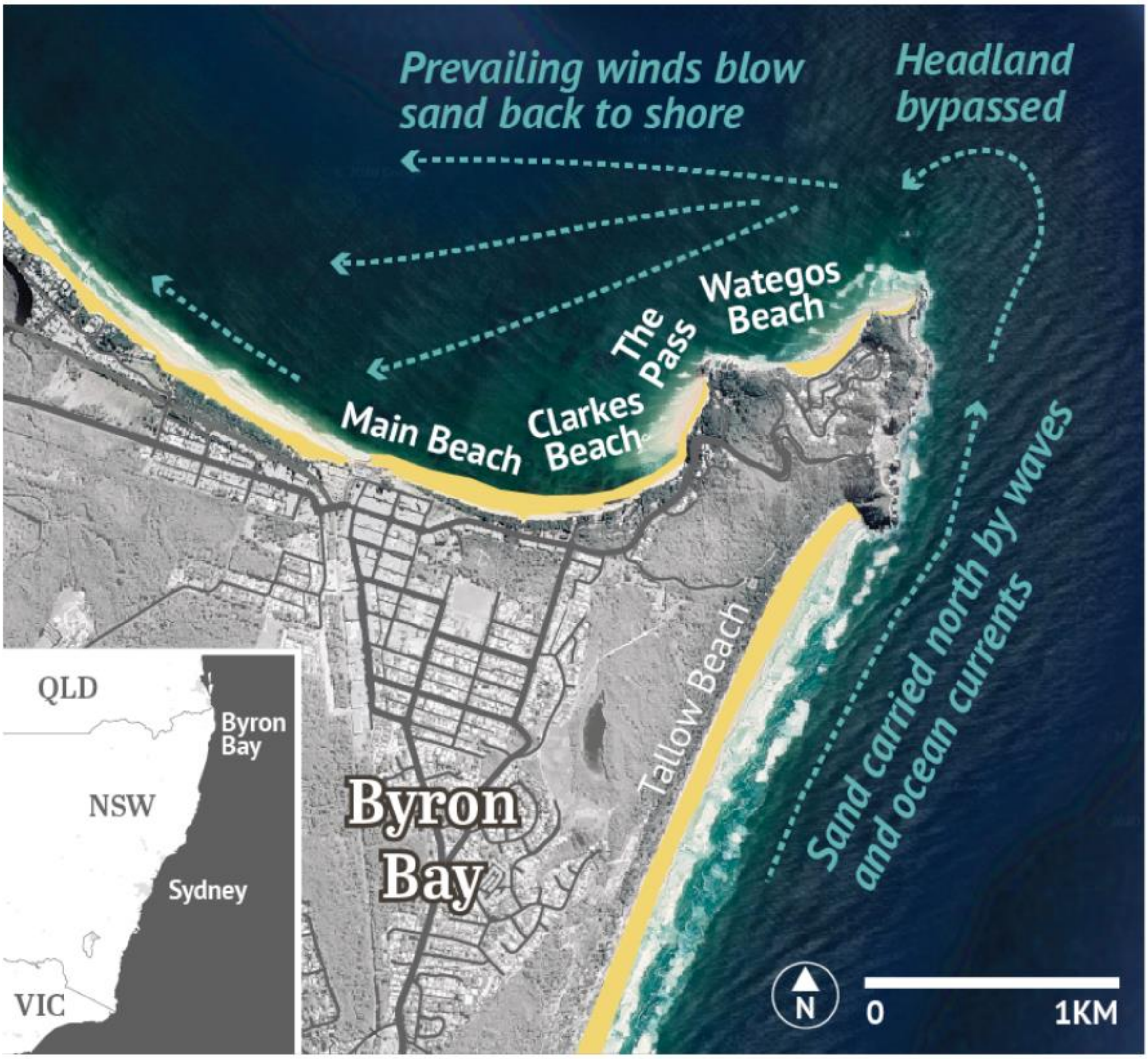
"Cape Byron is a rocky outcrop located more than 2km to the east of its associated rocky hinterland ... The broad coastal plain connecting the Cape to the hinterland, and thereby making it a headland, is the legacy of many thousands of years of accumulation of marine and terrestrial sediments. As the easternmost cape on the Australian coastline it clearly performs a role as an anchor for the shape of the east coast shoreline.

The net [northward] *longshore drift of sand along Tallow Beach delivers sand to Cosy Corner immediately to the south of the Cape Byron promontory. Here sand accumulates in the nearshore and offshore zone awaiting conditions favourable for its transport out, around and along, the 1 km long rocky cliff. Once reaching the north eastern tip of the Cape the sand can then theoretically spill northward around the headland and into the Byron Bay coastal compartment. Being the easternmost point of the coastline however the Cape extends sufficiently seaward to be affected by the on-shelf component of the East Australia Current (EAC), which is variable in strength but nearly always heads in a southerly direction off the Cape. As the north eastern tip is the most easterly point of the Cape, it is where the shelf current collides with the northward moving, wave induced littoral drift. Depending* *on the relative strengths of these two opposing currents at any particular time the sand either rounds the Cape or is stripped offshore in a south easterly direction; often in the large (approximately 200 m dia.) eddy formations shed off the northeast tip of the Cape. These eddies, which can readily be observed when in operation have sufficient angular velocity to maintain the sand in suspension and carry it well offshore, out of range of the coastal littoral drift system. The sand is eventually deposited in a south east trending lobe which, over the Holocene has been progressively growing and now extends more than 5 km south and is up to 40 m thick ... The situation at Cape Byron is further complicated by the water depths directly off the 1 km long, near vertical, face of the Cape. The seabed slopes steeply out so that relatively close to the Cape the water depth is 50m. However the depth to the seabed at face of the Cape is dependent on the immediate prior history of wave action. When extreme events such as the cyclones experienced in the late 1960s and early 1970s occur the reflected wave action strips sand from the underwater profile near the cliff face and deposits it offshore onto the lobe region where it is then redistributed southward by the EAC driven shelf current ... This means that a northward rebuilding of the subaqueous sediment profile needs to occur from the Tallow Beach supply before full bypassing can be re-established. Depending on the severity of the storm(s) this may take months or even years; a moderately heavy storm lowering the seabed by 3m means a delay of the order of 12 months before full by-passing can be again achieved. Hence the sediment budget of the Byron Bay embayment can be subjected to a short to medium term deficit of sand. This in turn can produce apparently difficult to explain episodes of beach erosion and beach fluctuations, even under mild wave conditions.*

In summary, the combination of both the intermittent interference of the southbound shelf current generated by the EAC and the stripping of sand from the subaqueous profile immediately off the face of the Cape, from time to time, impacts on the actual by-passing performance of the headland. This occurs regardless of the fact that the apparent littoral drift, as determined by considerations of wave energy flux and the apparently adequate supply of sand to the south might suggest otherwise."

3.2.6 Murray/GCCM (2020)

Murray/GCCM (2020) also presented work on headland bypassing of Cape Byron, including the work of Goodwin et al (2013). A graphic from GCCM indicating three pathways for nearshore and cross embayment transport is shown in Figure 3.5, noting that this figure does not show the loss to the south-east from Cape Byron to the Cape Byron sand lobe, nor the nearshore sand slugs propagating through Clarkes Beach.



Note: Loss to the south-east from Cape Byron to the Cape Byron sand lobe not shown

Figure 3.5 Sand transport pathways (Murray/GCCM, 2020)

3.3 Summary of coastal processes

Numerous qualitative and quantitative coastal process studies and/or models have been undertaken or developed for Byron Bay. While the quantitative studies differ in their adopted magnitude of cross embayment transport, all studies conclude that the Byron Bay embayment, including Clarkes Beach is receding due to a sand deficit. The quantitative studies concurred that the southward loss of sand due to the East Australian Current was one of the dominant causes of this recession.

Pulses or slugs of sand can enter the embayment from the east, causing substantial widening of the beach for extended periods of time. Conversely, extended periods of erosion can occur, and, due to the complex headland bypassing mechanisms, the erosion may not be directly attributable to large waves and/or elevated water levels. While the passage of pulses or slugs of sand can predominate for extended periods of time, the net long term trend is recession. Therefore, because of this long term recession, the geobag structures at Clarkes Beach are only suitable as an interim measure while longer term management is developed.

4 Coastal hazards

4.1 Definition

The NSW Coastal Management Act (2016) defines seven coastal hazards, namely:

a) beach erosion
b) shoreline recession
c) coastal lake or watercourse entrance instability
d) coastal inundation
e) coastal cliff or slope instability
f) tidal inundation
g) erosion and inundation of foreshores caused by tidal waters and the action of waves, including the interaction of those waters with catchment floodwaters

Without the geobags, the hazards of beach erosion, shoreline recession and slope instability are threats to land and the existing infrastructure at Reflections and the Cafe. A cross section of these hazards is shown in Figure 4.1. Lines representing the landward extent of the Zone of Slope Adjustment (ZSA) and the seaward extent of the Stable Foundation Zone are typically shown as coastal hazard lines when completing coastal hazard mapping.

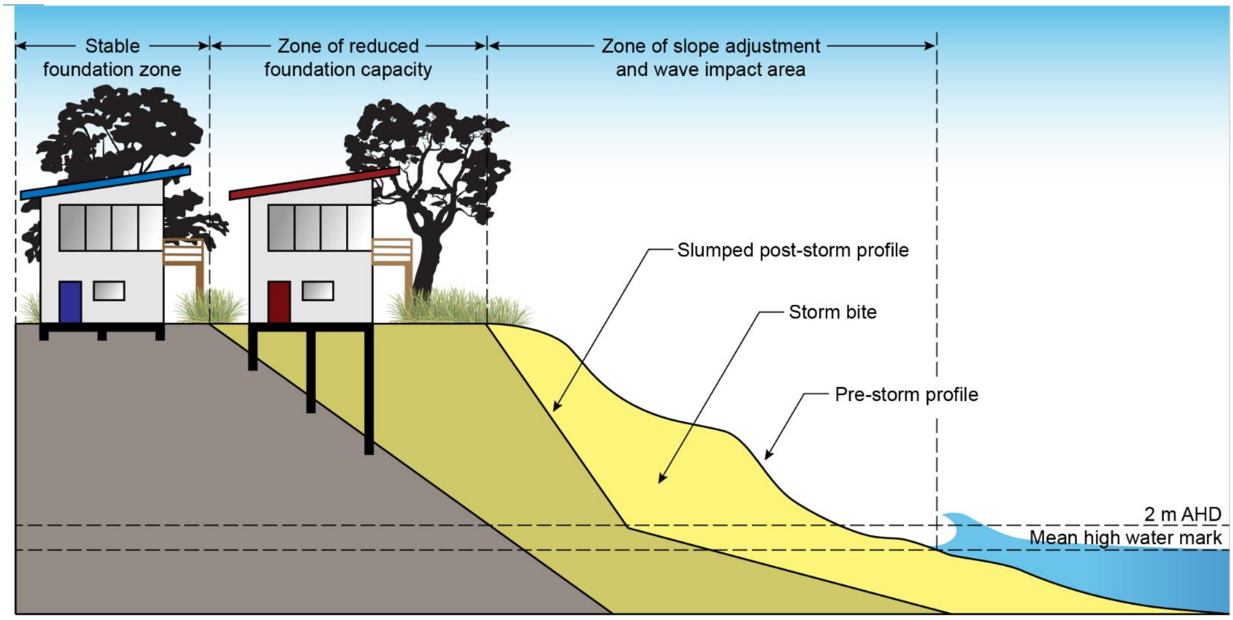


Figure 4.1 Dune stability schema (after Nielsen et al., 1992)

4.2 2007-2013 coastal hazard assessment

The most recent coastal hazard definition study for Clarkes Beach was undertaken by BMT WBM (2013), which was further developed into probabilistic hazards by Carley et al. (2016). For the Clarkes Beach precinct the following hazard values and comments apply:

- “Design” storm erosion (nominally 100 year ARI): 150 m^3/m

- Probabilistic storm demand (Carley et al., 2016; Gordon, 1987):
    - 1 year ARI: 5 m^3/m
    - 2 year ARI: 26 m^3/m
    - 5 year ARI: 53 m^3/m
    - 10 year ARI: 74 m^3/m
    - 20 year ARI: 95 m^3/m
    - 50 year ARI: 122 m^3/m
    - 100 year ARI: 150 m^3/m

- Underlying recession (best estimate): 0.2 m/year
- Recession due to sea level rise: (best estimate) effective Bruun Factor of 88, noting that this apparently high value (typical values are 50, but can range from 10 to 100) was due to the result of shoreline modelling by BMT WBM (2013) of the downdrift effects of structures and headlands, together with the very flat gradient of the bathymetry in this region – that is, it is an effective Bruun Factor, rather than a direct application of the Bruun Rule
- The hazard lines mapped the landward limit of the ZSA (Figure 4.1)
- For a dune elevation of about 7 m AHD, the width of the ZRFC is about 14 m, subject to further specialist geotechnical analysis

The central estimate hazard lines from BMT overlain on a recent aerial photo (12 April 2021) are shown in Figure 4.2 and Figure 4.3. It can be seen that between Reflections and Byron Bay SLSC, the present day "design" or nominally 100 year ARI hazard line is located approximately on or slightly seaward of the vegetation line on 12 April 2021, noting that the hazard line was relative to the 2007 beach state. Given that the BMT work was based on the 2007 profile, the present day (2021) is actually 14 years since the baseline, and therefore approximately 33% of the elapsed time between 2007 and 2050. A hypothetical hazard line using the BMT inputs, but adjusted to 2021 by WRL is also shown in the hazard line figures. The use of linear long term trends are an approximation, with many perturbations, however, these perturbations are covered within the storm erosion component.

Hazard lines from BMT WBM (2013), 2021 line by WRL

Figure 4.2 Erosion-recession hazard lines – embayment view

Hazard lines from BMT WBM (2013), 2021 line by WRL

Figure 4.3 Erosion-recession hazard lines – close up

5 Change since 2007 baseline beach position

As described above, the latest Coastal Hazard Definition Study was undertaken by BMT WBM (2013). Such studies generally use an accreted beach state as the starting point for calculations. For the BMT WBM (2013) study, the 10 March 2007 beach state was used as the baseline condition for calculations.

From the 2007 base date, the following data sources are available:

- Three dimensional data from photogrammetry (aerial photos) and LiDAR
- High resolution satellite photos (<0.8 m) from NearMap, which have been utilised after the last photogrammetry date
- Low resolution satellite photos (3 m) from PlanetLabs and other sources, which are available almost daily

The dates for each data set used in the analysis are shown in Table 5.1.

Photogrammetric or LiDAR data following the construction of the Reflections geobags (July 2019) is limited to 22 June 2020 and 12 April 2021, plus a Bluecoast UAV survey on 7 October 2020, while only the 12 April 2021 data is available after construction of the Cafe geobags. Therefore, aerial/satellite photos of the vegetation line were also utilised to assess recent change. Based on WRL's analysis, the 3 m AHD contour approximates the vegetation line. Due to the steep erosion scarp, the vegetation line will depict erosion well, but accretion of the lower portion of the profile will not be depicted by the vegetation line, since such accretion is primarily driven by the wind and may take many years to eventuate.

Table 5.1 Beach profile and width data sources

Photogrammetry /LiDAR	Hi res photo	Lo res photo*	Used for post 2007 analysis	Used for geobag end effect analysis
10/03/2007			✓	
11/01/2010			✓	
06/09/2012			✓	
04/05/2013			✓	
01/08/2016			✓	
25/05/2017			✓	
04/08/2018			✓	
05/03/2019			✓	
	11/06/2019		✓	✓
29/07/2019			✓	✓
		01/08/2019	✓	✓
	08/09/2019		✓	✓
		01/10/2019	✓	✓
		01/11/2019	✓	✓
	05/12/2019		✓	✓
	19/12/2019		✓	✓
		01/01/2020	✓	✓
		01/02/2020	✓	✓
		01/03/2020	✓	✓
	11/04/2020		✓	✓
		01/05/2020	✓	✓
		01/06/2020	✓	✓
22/06/2020			✓	✓
	04/07/2020		✓	✓
		01/08/2020	✓	✓
		01/09/2020	✓	✓
		01/10/2020	✓	✓
07/10/2020**				
		01/11/2020	✓	✓
	26/11/2020		✓	✓
		04/12/2020	✓	✓
		01/01/2021		
	15/01/2021		✓	✓
		01/02/2021		
		01/03/2021		
12/04/2021	12/04/2021			✓
		01/05/2021		
		01/06/2021		

* Low resolution photos are available almost daily, with monthly photos used in this analysis

** Bluecoast survey not in NSW Beach Profile Database (http://www.nswbpd.wrl.unsw.edu.au/)

The following information is presented in the following figures:

- Figure 5.1: 3 m AHD contour in 2007 and equivalent vegetation line on 4 December 2020 for the coast from The Pass to Jonson Street
- Figure 5.2: 3 m AHD contour in 2007 and equivalent vegetation line on 4 December 2020 in the vicinity of the Cafe

These figures show the following:

- Minimal erosion of the 3 m AHD contour from The Pass to the Captain Cook car park/ Thompsons Rock, just east of Reflections, due to the predominance of rock outcrops (including Thompsons Rock, Figure 2.2) backing the beach
- Horizontal erosion of 13 to 26 m between the Captain Cook / Thompsons Rock car park and Jonson Street

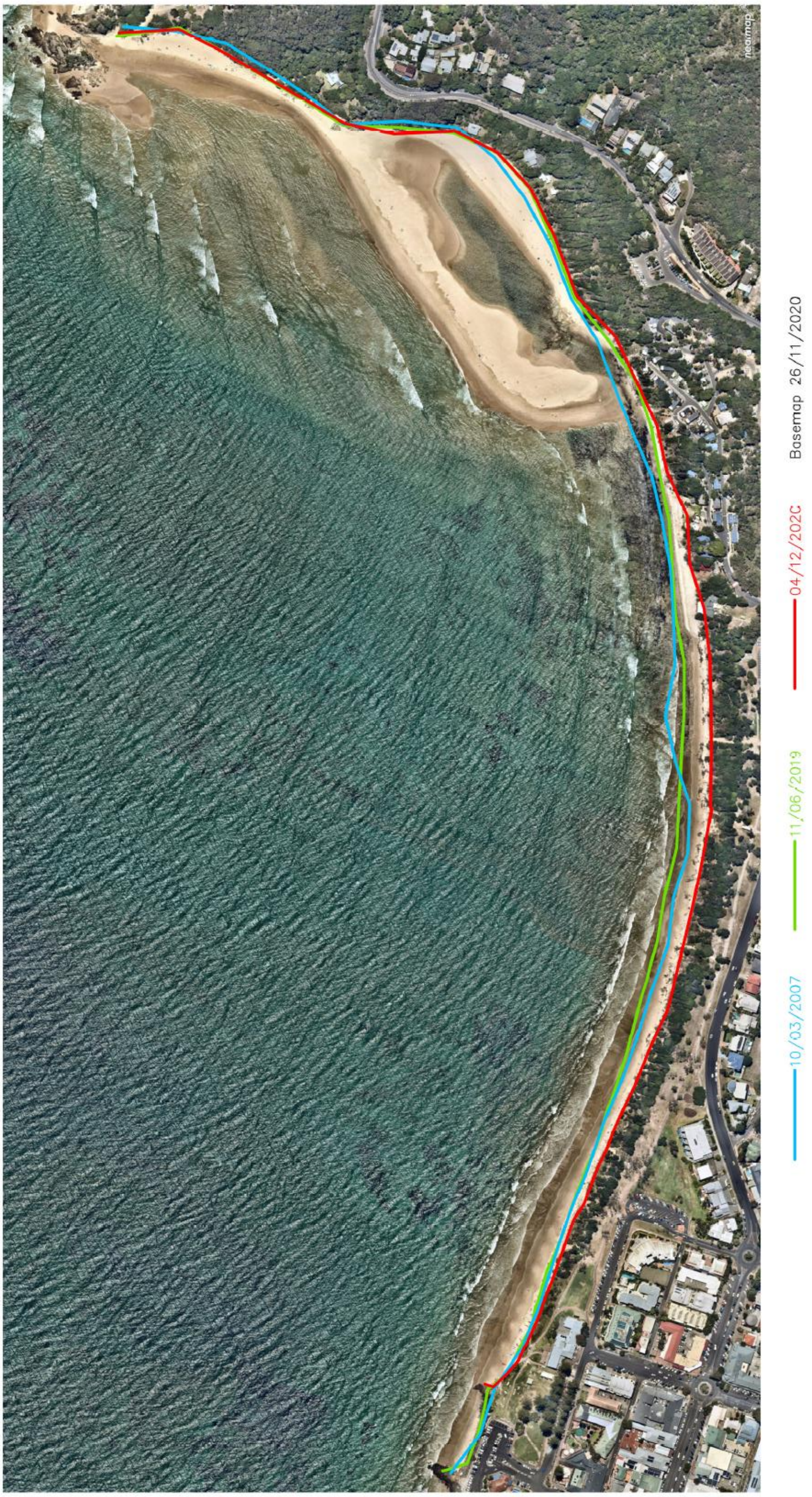


Figure 5.1 3 m AHD contour 2007 and 4 December 2020 – The Pass to Jonson Street

Figure 5.2 3 m AHD contour 2007 and 26 November 2020 – Reflections and Cafe

5.1 Contributions to erosion since 2007

As per Section 3, Clarkes Beach has exhibited a trend of recession since the first photogrammetry in 1947, noting that cycles of erosion and accretion still underly this trend. Erosion of the dune face appears to have accelerated from 2013, however, episodic sand slugs have periodically accreted the intertidal beach, including through 2021. Increased erosion has been evident immediately west of the front of the sand slug.

The recent erosion is likely to be caused by the following factors, with details provided below:

- Several recent large storm wave events (east coast lows and Tropical Cyclones) between 2013 and 2020 causing waves from the east to north-east, such as Tropical Cyclone Uesi, which caused littoral drift transport away from Cape Byron in both directions. That is, southward to Tallow Beach and north-west into Byron Bay
- This likely reduced the available volume of sand close to Cape Byron for headland bypassing, and therefore the available supply to Clarkes and Main Beach (Gordon, 2011)
- A sand slug had filled Wategos and The Pass in late 2020 - it had not yet reached Clarkes Beach in 2020, but propagated to the west of the Cafe geobags during 2021
- Average losses of 50,000 m^3/year to the south of Cape Byron (PWD, 1978) due to the East Australian Current have likely continued, noting that no updates on this estimate have yet been undertaken since PWD (1978) and no readily available studies quantifying the EAC in the vicinity of Cape Byron with regard to sediment transport potential are known
- Sea level rise, sea level rise induced recession and ongoing underlying recession have continued

5.1.1 East Australian Current

Apart from river and lake mouths, sand transport on most open coast beaches is due to action of waves. However, being the easternmost point of the Australian continent, Cape Byron extends into the zone of the East Australian Current (EAC) at times resulting in an average loss of 50,000 m3/year into the Cape Byron sand lobe.

Changes to the EAC could clearly alter these losses. Watson (2020) alluded to this, noting the very high sea surface height trends offshore of the NSW-Victoria border (exceeding 11 mm/year) are the result of a strengthening and southward extending EAC, bringing more warm water from the tropics south. It would be possible to further study changes to the EAC in the vicinity of Cape Byron, however, it would be complex and is beyond the scope of this study.

5.1.2 Recent sea level rise and vertical land movement

Sea level change can be expressed as an absolute quantity, relative to the fixed centre of the earth, or a relative quantity. Much of the surface of the earth also undergoes long term movement (vertical land movement). Relative sea level is of most importance for local coastal processes and hazards, and is defined as the change between the absolute sea level and the local land level. Watson (2020) provided sea surface height trends (both relative to vertical land movement, and absolute) for the following stations close to Byron Bay: 21 Tweed Heads; 22 Brunswick Heads; 23 Yamba. The trends for 1992 to 2019 are shown in Table 5.2, which have been estimated from Figure 3 of Watson (2020). The vertical land motion (VLM) is not known for Byron Bay-Clarkes Beach, so the value for Brunswick Heads has been adopted. That is, a sea surface height trend relative to the land of +3.0 mm/year.

Table 5.2 Land movement and sea surface height trends 1992-2019 (Watson, 2020)

Station	Name	Vertical Land Motion (mm/year)	Altimetry SLR (mm/year)	Relative SLR (mm/year
21	Tweed Heads	-2.0	+2.8	+4.8
22	Brunswick Heads	+0.2	+3.2	+3.0
23	Yamba	-1.0	+3.2	+4.2

5.1.3 Sand volume change from recession since 2007 baseline

With 14 years having elapsed since the 2007 baseline of the BMT WBM (2013) hazard study, ongoing recession and sea level rise recession components based on historic long term averages would be:

- Underlying recession: 14 x 0.2 m/year = 2.8 m
- Sea level rise recession: 14 x 0.003 m/year x 88 = 3.7 m
- Total additional recession: 6.5 m

A first order estimate of the total recession component volumes from 2007 to 2021 are as follows, noting that due to rock outcrops, there has been minimal erosion/recession above AHD from The Pass to Thompsons Rock to the Captain Cook car park. For an active dune height of +5 m AHD and a distance of 1200 m between the Captain Cook car park and Jonson Street, this total recession would be equivalent to the following sand volumes:

- 32.5 m^3/m above AHD from 2007 to 2021
- 39,000 m^3 above AHD from 2007 to 2021
- 2,760 m^3/year above AHD from 2007 to 2021

Based on the BMT WBM (2013) study, the estimated recession due to sea level rise is:

- 18 m^3/m above AHD from 2007 to 2021
- 21,600 m^3 above AHD from 2007 to 2021

The underlying recession component is:

- 14 m^3/m above AHD from 2007 to 2021
- 16,800 m^3 above AHD from 2007 to 2021

5.1.4 Incidental and accidental human removal

All users of the beach can attest to sand accidentally leaving the beach with them – stuck to their bodies, swimming costumes, wetsuits and surf craft. Few quantifications of this are available. Wynne et al. (1984) documented and reviewed such losses for the beaches of Adelaide, which comprise approximately 30 km of sandy shore and were estimated to have 500,000 to 1,000,000 person-visits per year. Wynne et al. (1984) reviewed calculations undertaken in 1972 and estimated 100 m^3 per year of sand was removed inadvertently by beach users. This is a rate of 0.0001 to 0.0002 m^3 per person-visit. Although no direct statistics are available, the higher wave climate and popularity of swimming, bodysurfing and surfing at Byron Bay are likely to make accidental sand removal rates higher than Adelaide.

Using a range of sources, Anning (2015) estimated the following beach use for Byron Bay:

- Tourist beach visits: 1,800,000 per year
- Resident beach visits: 350,000 per year
- Total beach visits: 2,150,000 per year

Adopting the higher end of Wynne's estimate yields 430 m^3/year of sand lost through incidental and accidental human removal.

5.1.5 Stormwater outflows

Stormwater pipes and gullies cause localised erosion due to scour from the discharged water. They sometimes also deliver sand to the littoral system from the catchment. Detailed studies would be needed of the stormwater discharge points in the vicinity of Reflections and the Cafe, but the effect on embayment wide coastal processes is likely to be negligible.

5.2 Summary of sand quantities and contribution to erosion

A summary of indicative sand quantities and their contribution to erosion is shown in Table 5.3 and Figure 5.3. The estimates regarding end effects from the geobags are presented in Section 7. Change is also presented between the dates July 2019 (last photogrammetry prior to Cafe geobags, plus used as a start date for recent erosion) and November 2020 (date just after construction of Cafe geobags). The BMT WBM (2013) hazard study estimated that an uncertainty of $\pm$20% on the recession estimates was plausible, and this was incorporated into the Carley et al (2016) probabilistic hazard modelling.

As stated in the table, some sand quantities occur below the water (e.g. cross embayment transport), while others occur above the water. The primary purpose of the table is to compare the indicative relative magnitudes of various components of coastal change in Byron Bay. The assumed EAC contribution is based on previous estimates (Gordon et al., 1978), and changes to the EAC could easily make it the largest contribution to recent change, however, this is presently unknown and beyond the scope of this study.

As per Section 7, Table 5.3 and Figure 5.3, the direct impact of the Reflections and Cafe geobag works is almost imperceptible compared with embayment-wide sand quantities, and is less than accidental/incidental removal by people. Nevertheless, the Reflections and Cafe geobag works have localised end effects as detailed in Section 7. The largest change component (inferred change due to waves, storms etc.) would generally be expected to reverse in the near future, with beach recovery evident through 2021, though this trajectory is not certain over the next 5 years. However, the two recession components are not presently reversible.

Table 5.3 Sand quantities The Pass to Jonson St

Component (below) and sand volume (right)	Design or measured value m³/m above AHD	m³ above AHD	Average annual rate m³/year	Change 2007-2020 m³	Change July 2019-Nov 2020 m³	Proportion relative to high measured change 2007-2020 %
Coastal process estimates						
Nearshore littoral drift – low estimate	-	-	15,000	195,000	18,750	-
Nearshore littoral drift – high estimate	-	-	200,000	2,600,000	250,000	-
Cross embayment transport– low estimate	-	-	0	0	0	-
Cross embayment transport– high estimate			200,000	2,600,000	250,000	-
EAC losses			-50,000	-650,000	-62,500	-
Measured change and component estimates Captain Cook car park to Jonson St						
"Design" storm erosion volume	-150	-180,000	-	-	-	-
Measured change 2007 to 2020 - low	-65	-	-	-117,000	-	-
Measured change 2007 to 2020 - high	-130	-		-234,000	-	-
Measured change 2007 to 2020	-	-100,000	-	-100,000	-	100
Inferred change due to storms and recent events - high						58
SLR change 2007 to 2020	-17	-20,400	-1,569	-20,400	-1,962	20
Recession change 2007 to 2020	-13	-15,600	-1,200	-15,600	-1,500	16
Accidental human removal	-		-430	-5,590	-538	5.6
End effect Reflections geobags*	-175				-240	0.24
End effect Reflections + Cafe geobags*					-500	0.50
Stormwater outlets	minor	minor	minor	minor	minor	-

* Note that the geobags have not been present since 2007, but the end effect attributable to them (Section 7) has been compared to other long term coastal change components

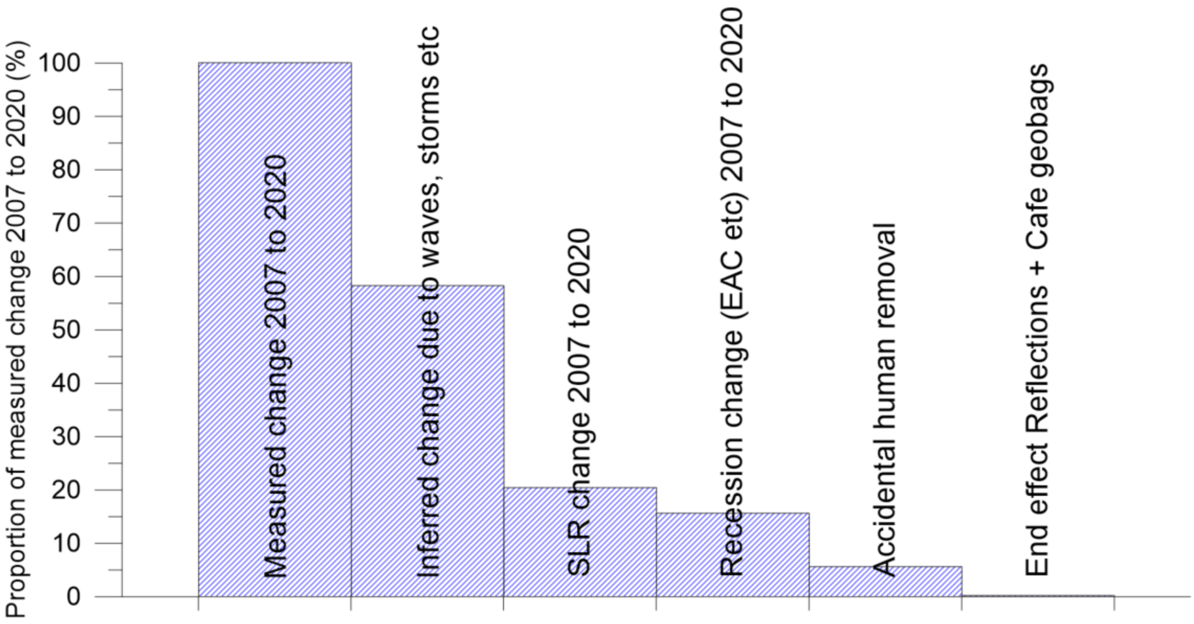


Figure 5.3 Indicative contributions to measured sand change above AHD (2007 to 2020)

6 Geobag stability

6.1 Probability Terminology

The following definitions are provided, adopted from Pilgrim (1987):

Risk: Likelihood (or probability) times consequence.

Average Recurrence Interval (ARI): The average time between exceedances (e.g. large wave height or high water level) of a given value, also known as Return Period.

Annual Exceedance Probability (AEP): The probability (expressed as a percentage) of an exceedance (e.g. large wave height or high water level) in a given year.

Project Life (N): Also known as planning timeframe or planning horizon.

Encounter Probability: The chance of an event being equalled or exceeded over the design life of a project life.

The use of ARI, though superficially simple, has been criticised as misleading some stakeholders, who may believe that the event will recur only at regular intervals. This is particularly the case when it is described as *Return Period*, which connotes some sort of regularity in the event.

AEP has been enshrined in many policies and regulations, in particular a 1% AEP, which is reasonably well understood. However, AEPs less than this are harder to comprehend. For example, 0.02% AEP is generally more difficult to comprehend than the equivalent 5,000 year ARI.

6.2 Design risk

The selection of a level of risk can be expressed by the following equations (British Standard 6349; Borgman, 1963; Kite, 1988), with tabulated values presented in Table 6.1 and Table 6.2. These tabulated values are also shown graphically in Figure 6.1. For “partial series” but not “annual series”, Pilgrim (1987) expressed the probability that a structure will encounter a given ARI event over its design life as:

$$P = 1 - e^{\left(-N/_{ARI}\right)} \qquad \text{(Equation 1)}$$

Conversely, the required design ARI event, can be determined by manipulating Equation 1 to

$$ARI = {}^{-N}\!\Big/\!\left(ln\,(1-P)\right) \qquad \text{(Equation 2)}$$

The equivalent AEP for an ARI is

$$AEP = 1 - e^{\left(\frac{-1}{ARI}\right)} \qquad \text{(Equation 3)}$$

For an ARI of about 10 years or more, Equation 3 can be approximated by

$$AEP \approx \frac{1}{ARI} \qquad \text{(Equation 4)}$$

where: ARI is average recurrence interval in years

P is the accepted probability of exceedance (range 0 to 1, with 0 being “no chance” of exceedance and 1 being 100% probability of exceedance)

N is the expected project life in years

AEP is the annual exceedance probability

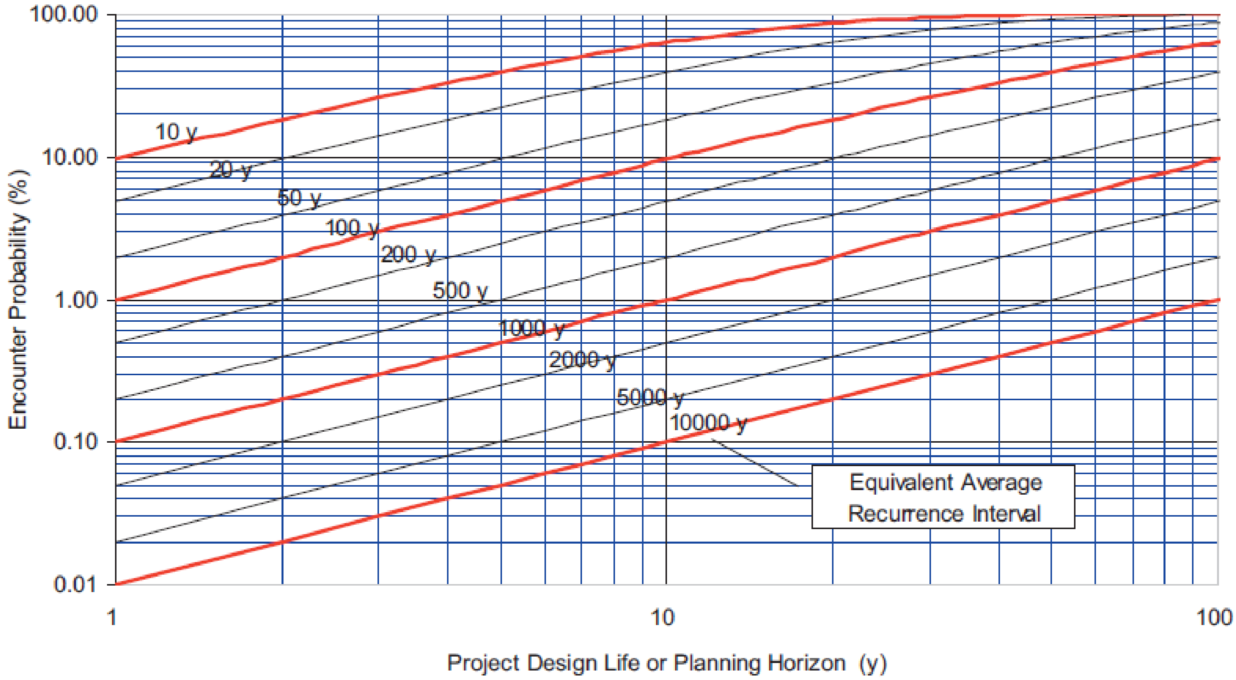
e is the transcendental constant used as the base to natural logarithms (≈ 2.7182818).

Table 6.1 Encounter Probability (Probability of Exceedance) for given ARI and Life

		Probability of Exceedance (%) for Design ARI (years)										
ARI		**1**	**2**	**5**	**10**	**20**	**50**	**100**	**500**	**1000**	**2000**	**10000**
AEP		63.21%	39.35%	18.13%	9.52%	4.88%	1.98%	1.00%	0.20%	0.10%	0.05%	0.01%
Project Life (years)	1	63.21%	39.35%	18.13%	9.52%	4.88%	1.98%	1.00%	0.20%	0.10%	0.05%	0.01%
	2	86.47%	63.21%	32.97%	18.13%	9.52%	3.92%	1.98%	0.40%	0.20%	0.10%	0.02%
	5	99.33%	91.79%	63.21%	39.35%	22.12%	9.52%	4.88%	1.00%	0.50%	0.25%	0.05%
	10	100.00%	99.33%	86.47%	63.21%	39.35%	18.13%	9.52%	1.98%	1.00%	0.50%	0.10%
	20	100.00%	100.00%	98.17%	86.47%	63.21%	32.97%	18.13%	3.92%	1.98%	1.00%	0.20%
	50	100.00%	100.00%	100.00%	99.33%	91.79%	63.21%	39.35%	9.52%	4.88%	2.47%	0.50%
	100	100.00%	100.00%	100.00%	100.00%	99.33%	86.47%	63.21%	18.13%	9.52%	4.88%	1.00%

Table 6.2 Design ARI event for given accepted risk of exceedance and project life

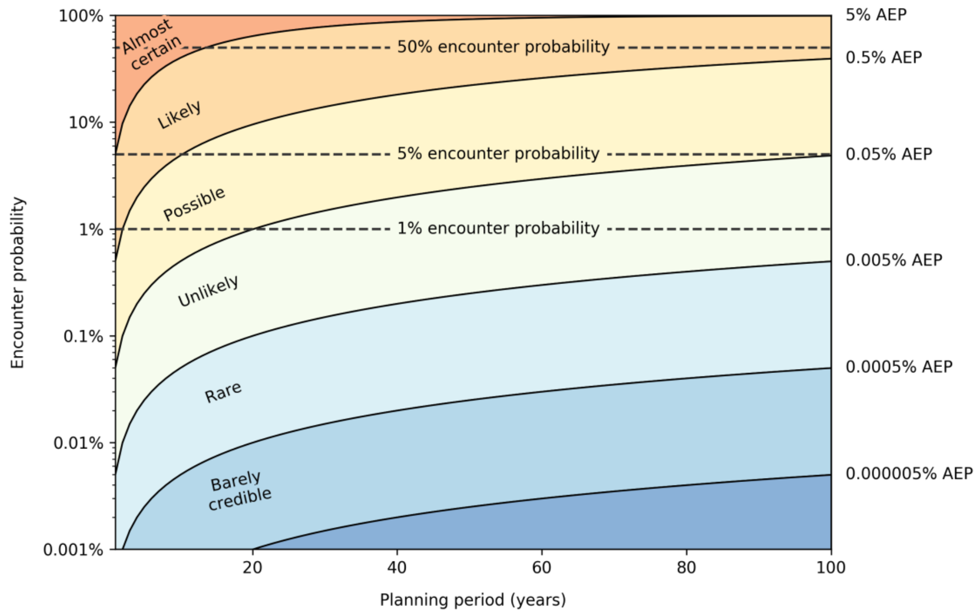
Project Life (years)	Required Design ARI (years) for Accepted Risk of Failure (Encounter Probability)											
	1%	**2%**	**5%**	**10%**	**20%**	**25%**	**33%**	**50%**	**75%**	**90%**	**95%**	**99%**
1	99	49	19	9.5	4.5	3.5	2.5	1.4	0.7	0.4	0.3	0.2
2	199	99	39	19	9.0	7.0	5.0	2.9	1.4	0.9	0.7	0.4
5	497	247	97	47	22	17	12	7.2	3.6	2.2	1.7	1.1
10	995	495	195	95	45	35	25	14	7.2	4.3	3.3	2.2
20	1990	990	390	190	90	70	50	29	14	8.7	6.7	4.3
50	4975	2475	975	475	224	174	125	72	36	22	17	11
100	9950	4950	1950	949	448	348	250	144	72	43	33	22



Note: Figure sourced from NCCOE, 2012

Figure 6.1: Encounter probability, design life and average recurrence interval

Figure 6.2 shows qualitative descriptions of likelihood for a range of encounter probabilities and planning periods (design lives) for a 100 year planning period.



Note: Figure adapted from AGS, 2007

Figure 6.2 Likelihood descriptions of encounter probabilities (100 year planning period)

Pilgrim (1987) noted that encounter probability "can assist in making an essentially subjective decision about an acceptable "risk of failure"". For example (with reference to Table 6.1), a structure built to last for 50 years (i.e. it has a 50 year design life) has a 39% chance of being exposed to a 100 year ARI event and a 10% chance of being exposed to a 500 year ARI event. Designing to resist damage for the latter condition might be more expensive, but it will mean there is a much lower likelihood that the structure will have to be repaired during its operational lifetime.

6.3 Design standards and design event

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 events from Gordon, Carley and Nielsen (2019) are shown in Table 6.3.

Table 6.3 Asset and design probability

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 6.4. 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 6.4 Annual probability of exceedance of design wave events (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%

When applying the encounter probability from the above standards and ISO 21650:2007 to flexible of semi-flexible coastal structures such as geobags, for "normal" structures the following probabilities are typically adopted:

- Initial (2%) damage of the structure at an ARI of 2 times the design life
- Failure of the structure at an ARI of 10 times the design life

For a design life of 5 years this yields:

- Initial (2%) damage at 10 year ARI, with ARI being for the wave height at the geobags
- Failure at 50 year ARI, with ARI being the wave height at the geobags

Structures with a design capacity below this may involve higher than normal maintenance or repair costs (Section 8), and/or a higher probability of failure than best practice (Figure 6.3). The recent December 2020 storm event which impacted the geobags is discussed in Section 6.6.2.

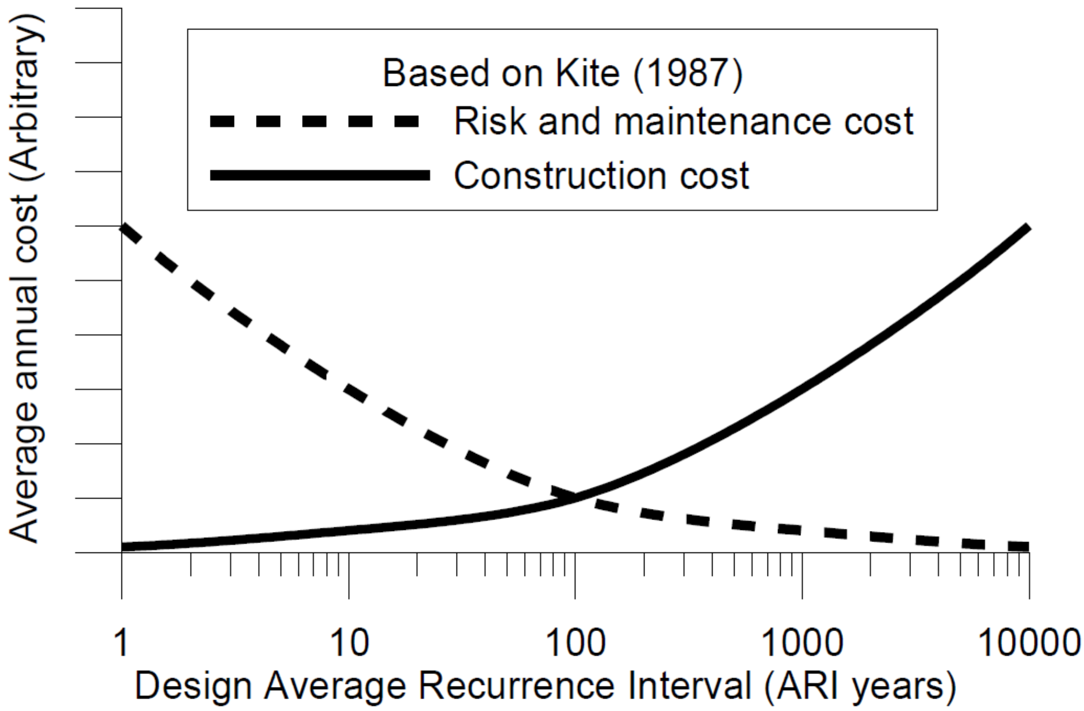


Figure 6.3 Balance between risk, maintenance and capital cost

6.4 Seawall failure and damage

6.4.1 Failure modes of seawalls

The US Corps of Engineers (USACE, 2006), defines the failure of a coastal structure as:

"*Damage that results in structure performance and functionality below the minimum anticipated by design.*"

The most common reasons for the failure of a coastal defence structure are (Oliver *et al.*, 1998; USACE, 2006; CIRIA, 2007):

- Design failure: this occurs when either the structure as a whole, including its foundation, or individual structure components cannot withstand load conditions within the design criteria
- Load exceedance failure: this results from an underestimation of the design conditions
- Construction failure: this can be caused by unsuitable construction techniques or poorly suited construction materials
- Deterioration failure: this failure is the result of structure deterioration and lack of project maintenance

The main two dimensional potential failure/damage modes of seawalls are illustrated in Figure 6.4 and Figure 6.5 for a structure built from (single layer) sandbags. The main failure modes can be detailed 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 – for Clarkes Beach, this is limited by the rock/reef layer

- Sliding, in which the wall moves away from the retained profile – for Clarkes Beach, the global stability of the works was modelled by Ardill Payne
- Overturning, in which the wall topples over – for Clarkes Beach, the global stability of the works was modelled by Ardill Payne
- Slip circle failure, in which the entire embankment fails – for Clarkes Beach, the global stability of the works was modelled by Ardill Payne
- Loss of structural integrity, due to wave impact (Section 6.6)
- Erosion of the backfill, caused by wave overtopping, high water table levels, or leaching through the seawall (Section 6.4.1)
- Corrosion, abrasion and impact damage – for Clarkes Beach, this is managed through selection of a durable geobag fabric (Geofabrics Australasia Texcel 600R) and ongoing monitoring (Section 8)
- Outflanking and end scour (Section 7)

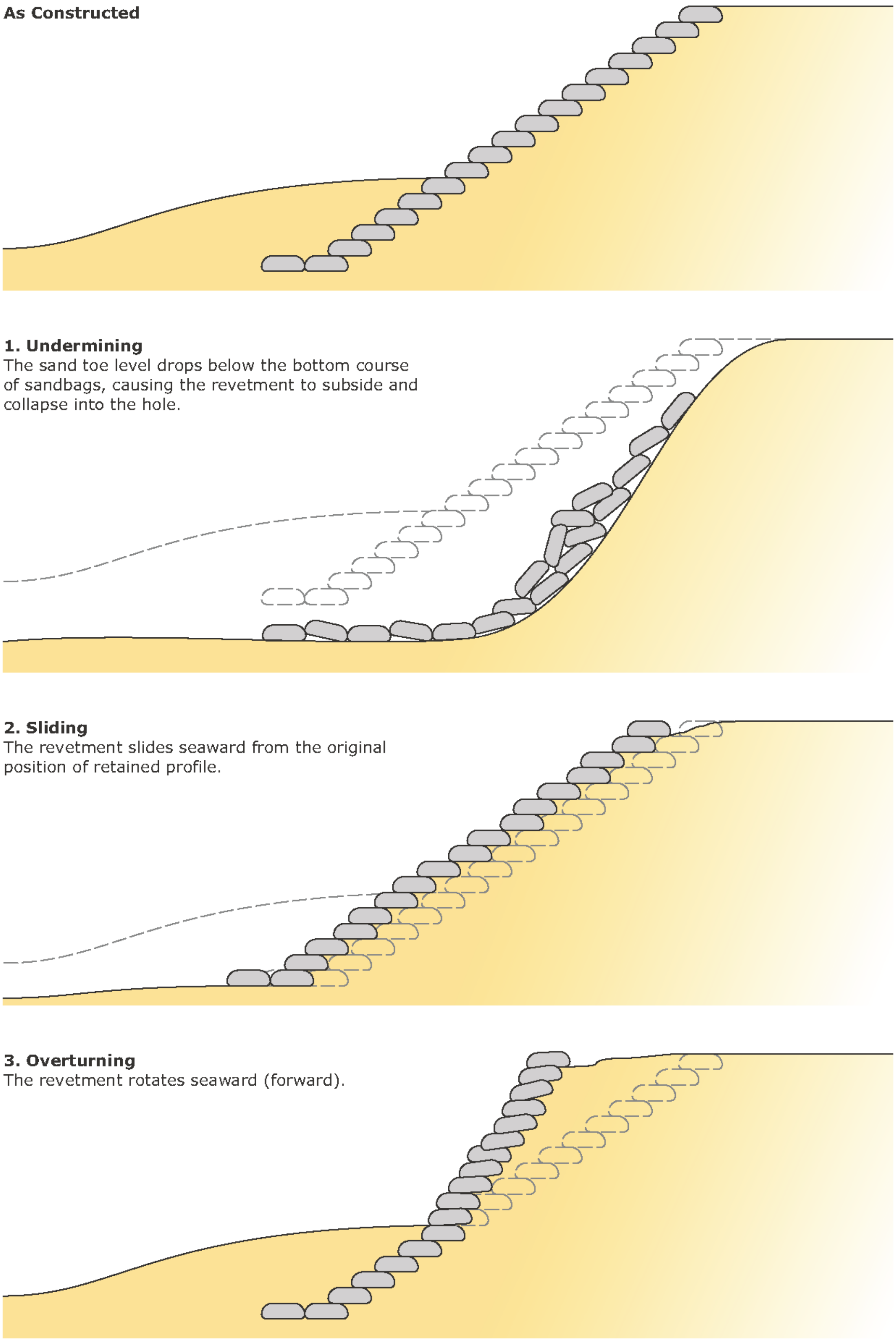


Figure 6.4 Seawall Potential Failure / Damage Modes (1 of 2)

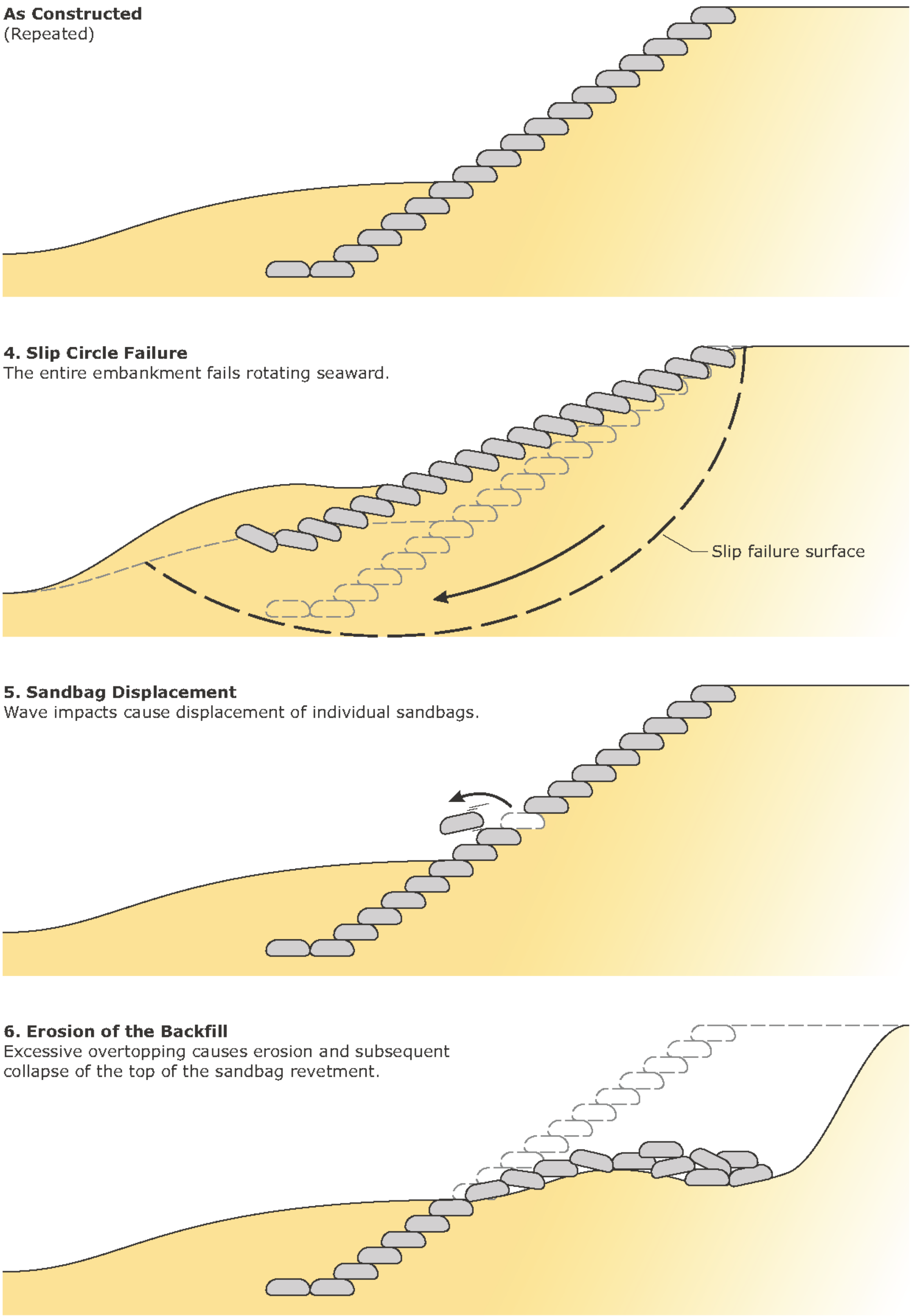


Figure 6.5 Seawall Potential Failure / Damage Modes (2 of 2)

6.5 Design conditions for Clarkes Beach

Design conditions for the potential design life of the geobag wall fronting the Cafe have been defined for average recurrence intervals (ARIs) of 1, 2, 5, 10, 50 and 100 years to better estimate the probability of failure throughout the design life.

The design conditions considered for this study were established using a combination of elevated water levels including wave setup, and depth limited nearshore waves, to assess the risk of damage to geobags under direct wave impact.

6.5.1 Water levels

Design water level conditions for Coffs Harbour based on MHL (2018) are shown in Table 6.5. While tide gauges are available at Brunswick Heads and Ballina, both of these gauges are sometimes elevated above ocean levels by river flood flows, whereas Coffs Harbour is not. Therefore, Coffs Harbour water levels were adopted for Byron Bay.

Table 6.5 Design water levels Coffs Harbour (MHL, 2018)

ARI (years)	No wave setup WL (m AHD)
1	1.27
2	1.31
5	1.34
10	1.39
20	1.43
50	1.46
100	1.49

6.5.2 Nearshore waves

Offshore wave design conditions from the Byron Bay wave buoy were transformed to the nearest -10 m AHD isobath point in the NSW Government wave transformation tool (Figure 6.6 and Figure 6.7). The offshore data was derived by Shand (2009), with the transformation data provided by NSW DPIE. The offshore and nearshore wave conditions are shown in Table 6.6.

As shown in Figure 6.8, the Cafe geobags are fronted/founded on a weathered bedrock/reef layer which has lower erodibility than beach sand.

Waves for the worst case for each ARI from the nearshore node (10 m isobath) were transformed to the geobag location using the Dally et al (1984) surf zone model with the NSW Government 2018 bathymetric LiDAR data. This model also calculates nearshore wave setup. Since waves do not reach the geobags when the profile is accreted, the survey conducted by Bluecoast on 7 October 2020 (in which the beach was eroded) was used for the profile close to shore.

WRL assessed the design wave conditions for the Cafe geobags using the depth limited significant wave height based on Goda (2010). Wave conditions were established for the most scoured scenario, that is, when the beach fronting the geobag wall is eroded down to the reef and/or weathered bedrock layer, resulting in maximum water depth and waves potentially reaching the structure.

Design wave heights were estimated 10 m offshore from the toe of the geobag seawall to account for the typical plunge distance of waves with a seabed depth of +0.15 m AHD, based on the eroded beach survey conducted by Bluecoast on 7 October 2020. In this survey, the beach was observed to be in an eroded state with large areas of exposed clay and eroded bedrock indicating that the measured levels just offshore from the geobags were the lowest possible seabed levels over the estimated 5 year design life.

The adopted design wave heights at the geobags on Clarkes Beach based on the worst case for each ARI at the 10 m isobath are presented in Table 6.7, where Hs is the "significant" wave height and Tp is spectral peak wave period.

Figure 6.6 Byron Bay wave buoy and nearshore wave model output

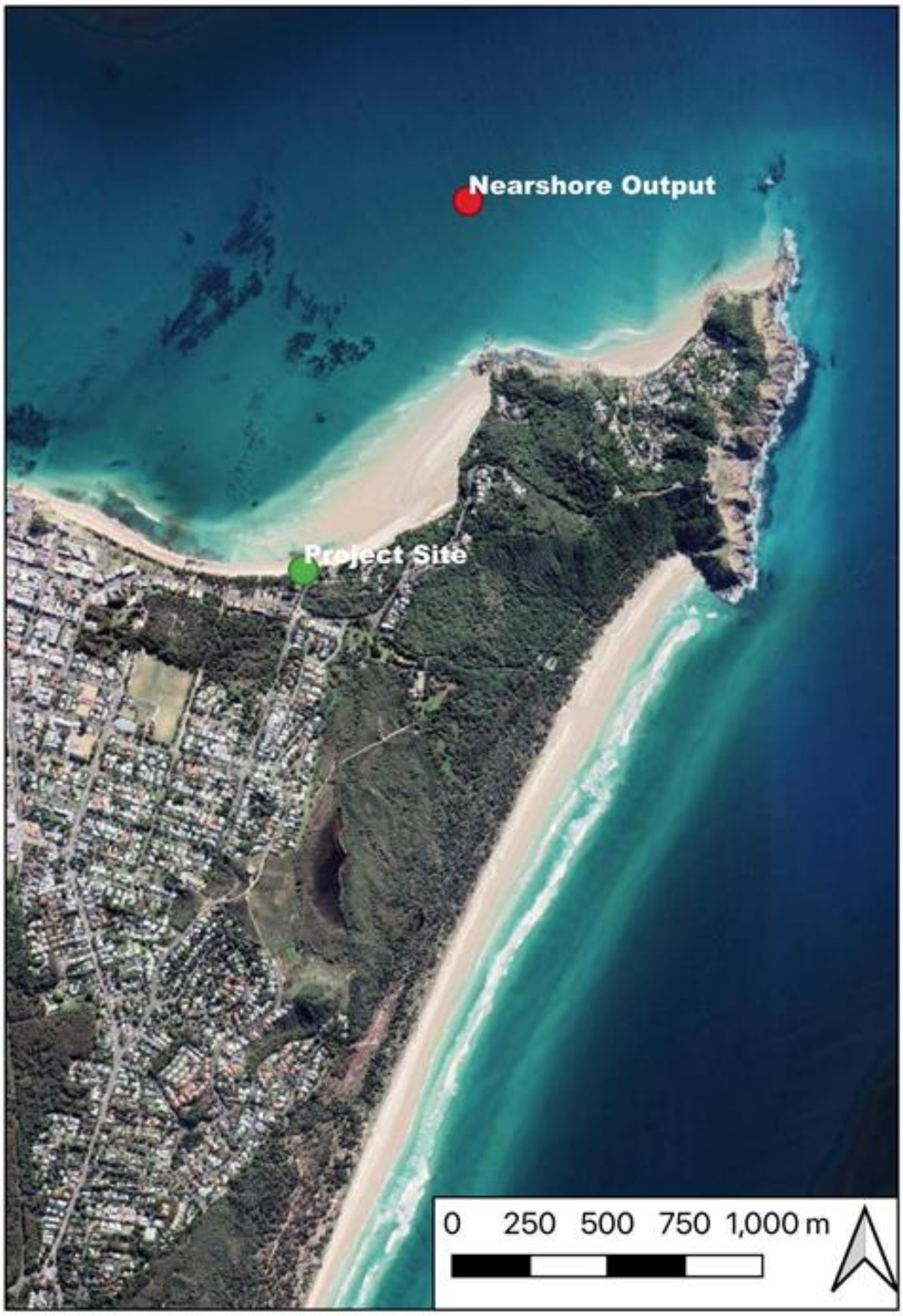


Figure 6.7 Nearshore wave model output close up

Table 6.6 Offshore and 10 m isobath wave conditions (NSW DPIE Node 1000547)

ARI	Hos (m)	Tpo (s)	Dir o (° TN)	Hs 10 (m)	Tp 10 (s)	Dir o (° TN)
1	3.5	10.8	45	2.7	10.7	42.2
1	5.2	11.9	112.5	3.6	11.8	62.6
1	5.0	11.7	157.5	2.3	11.6	67.3
5	4.1	11.2	45	3.2	11.1	42.3
5	6.1	12.4	90	4.7	12.2	55.5
5	5.8	12.2	135	3.6	12.2	66.1
20	4.6	11.5	45	3.6	11.4	42.1
20	6.8	12.8	90	5.2	12.7	53.9
20	6.5	12.6	135	4.1	12.6	65.0
100	5.1	11.8	45	4.0	11.7	41.6
100	7.6	13.3	90	5.8	13.2	51.8
100	7.2	13.1	135	4.6	13.1	66.2

Notes: Hos is offshore deep water wave height, Tpo is offshore deep water spectral peak wave period, Dir o is offshore deep water wave direction, ° TN is True North, Hs 10 is wave height at -10 m isobath, Dir 10 is wave direction at -10 m isobath

Table 6.7 Design wave and water level conditions at structure

ARI (years)	WL excl setup (m AHD)	Hs o (m)	Tp o (s)	Nearshore Hs 10 m (m)	Nearshore Tp 10 m (s)	Clarkes geobags setup WL (m AHD)	Clarkes geobags Hs (m)	Clarkes geobags Tp (s)
1	1.27	4.5	11.0	4.05	11.0	1.87	1.06	11.0
2	1.31	4.8	11.5	4.32	11.5	2.08	1.19	11.5
5	1.34	5.2	12.0	4.68	12.0	2.18	1.26	12.0
10	1.39	5.6	12.5	5.04	12.5	2.32	1.34	12.5
20	1.43	5.9	13.0	5.31	13.0	2.45	1.42	13.0
50	1.46	6.2	13.5	5.58	13.5	2.55	1.49	13.5
100	1.49	6.5	14.0	5.85	14.0	2.67	1.56	14.0

The design setup water level at the geobags for an eroded beach and the design wave height at the geobags shown above are the most accurate coastal engineering estimates available for the works. Previous studies did not provide output for smaller ARI events. Furthermore, wave setup in estuaries is generally lower than for the open coast due to the deeper water depths in the estuary mouth.

Comparative examples from other studies include:

- SMEC (2009) flood study for Belongil Creek:
    - 100 year ARI excl wave setup 1.87 m AHD (1.49 m AHD in present WRL study)
    - 100 year ARI incl wave setup 2.42 m AHD (2.67 m AHD in present WRL study)
- BMT WBM (2014) flood study for Belongil Creek:
    - 5 year ARI incl wave setup 1.9 m AHD (2.18 m AHD in present WRL study)
    - 10 year ARI incl wave setup 2.0 m AHD (2.32 m AHD in present WRL study)
- BMT WBM (2013) presented the following values for Sydney and the Gold Coast, including for estuary inundation in Byron Bay (but not open coast wave setup):
    - 100 year ARI incl wave setup 2.29 m AHD (2.67 m AHD in present WRL study)

6.6 Geobag stability

6.6.1 Design stability

Design guidelines for geobag stability under wave attack was published in Coghlan et al. (2009) and Hornsey et al. (2011). The design curve for 0.75 m^3 geobags is shown in Figure 6.9. This curve is for "initial damage", defined as less than 2% of the geobags moving, noting that this is not "zero damage".

It should be noted that design conditions were established for a beach in an eroded state, resulting in the complete geobag wall structure being exposed to direct wave impact from design wave heights. A review of recent aerial imagery (Section 5) shows that the beach has already started to recover from the erosion experienced in 2020, resulting in higher beach levels in front of the seawall and complete burial of the toe and lower geobag layers. The geobags are almost completely buried as of August 2021. This average/accreted state of the beach and partial burial of the geobag wall will offer additional protection from wave impact and lower the risk of failure of the structure.

A summary of expected geobag performance is as follows:

- The estimates below are somewhat "conservative", as they assume an eroded beach and have utilised wave setup levels from a higher energy beach – detailed local studies for wave setup would likely yield lower values
- Damage will be less than the estimates below (or higher ARI events can be withstood) if the beach fronting the geobags is not scoured to the bedrock
- If the beach fronting the geobags is eroded down to the bedrock, for the most likely 13 s wave period, the geobags will have **less than 2% damage for a 2 year ARI** event
- The geobags will experience **more than 2% damage in a 5 year ARI** event
- Wave overtopping of the geobags may erode the sand above them – ongoing monitoring of this will be required

Based on Table 6.1, the above analysis indicates that for an estimated 5 year design life, if the beach fronting the geobags is eroded down to the reef:

- There is a 92% probability of a 2 year ARI wave event (which would result in less than 2% damage)
- There is a 63% probability of a 5 year ARI wave event (which would result in more than 2% damage)
- These probabilities would reduce if sand is present in front of the geobag wall and reduces the depth limited waves able to reach the geobags

Figure 6.8 Weathered rock and reef offshore from Cafe

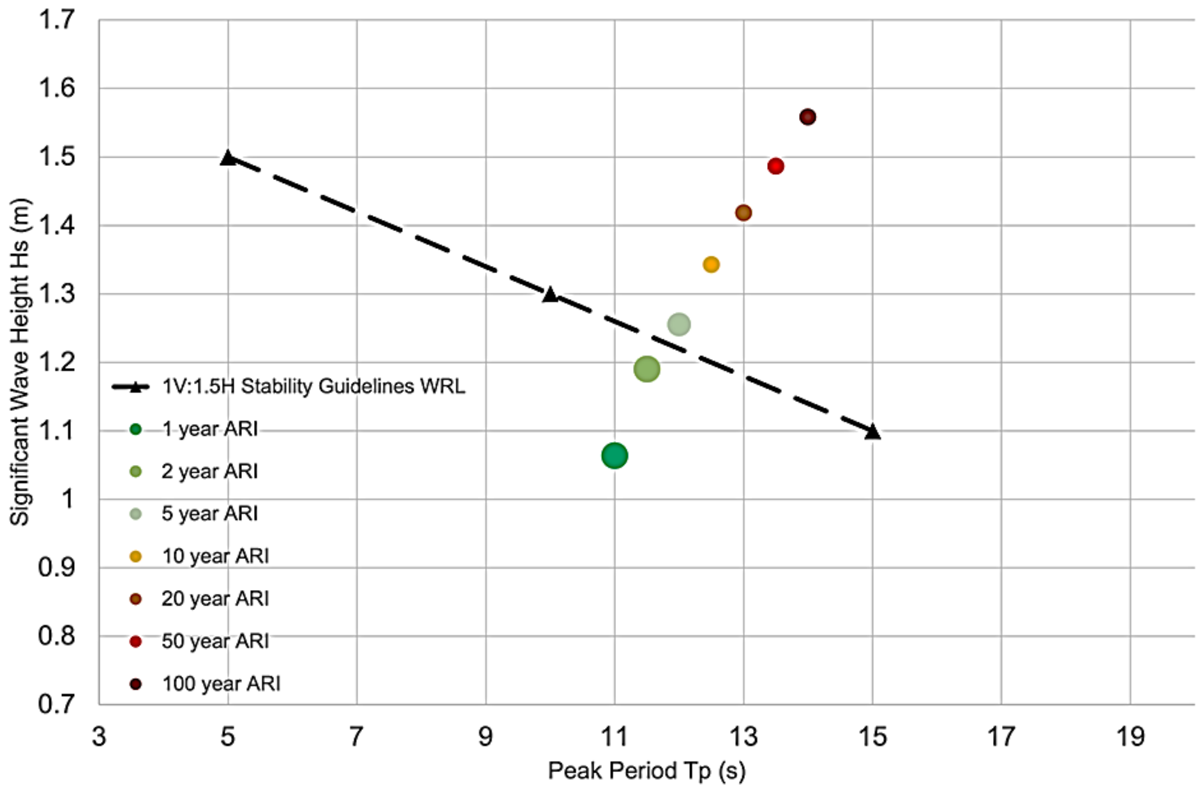


Figure 6.9 Geobag stability estimate

6.6.2 December 2020 event

An easterly trough low weather event impacted the geobag works in December 2020 (Figure 6.10). The beach was eroded at the time, meaning that the geobags were fully exposed and impacted by waves on most high tides.

Measured water levels were obtained for Brunswick Heads and Coffs Harbour. Note that water level data for the Gold Coast sand bypassing jetty was unavailable for the peak of the storm. Higher water levels were measured at Brunswick Heads compared with Coffs Harbour. This could be due to rainfall and flooding elevating the Brunswick Heads level, but greater local intensity of the storm surge cannot be excluded. Other nearby tide gauges such as Tweed Heads and Ballina may also be influenced by river floods.

The following conditions were measured:

- Peak Hs = 4.4 m from E to NE – this is approximately 1 year ARI (Shand et al., 2011), (Figure 6.11)
- Peak still water level 1.1 m AHD at Coffs Harbour – approximately 0.5 year ARI (MHL, 2018)
- Peak still water level 1.4 m AHD at Brunswick Heads – approximately 10 year ARI (MHL, 2018)

Using the same wave transformation and wave setup process described above, hourly wave heights at the geobags for the December 2020 event are shown in Figure 6.12. This indicates a peak nearshore wave height at the geobags of:

- Hs = 1.2 m using Brunswick Heads water level (~2 year ARI)
- Hs = 1.0 m using Coffs Harbour water levels (slightly less than 1 year ARI)

The geobag structure withstood this event with minimal displacement of geobags. However, the structure was overtopped by waves, and sand was eroded from above the crest. An additional single layer course of geobags was added to the crest and additional sand was placed to restore the embankment behind the geobags.

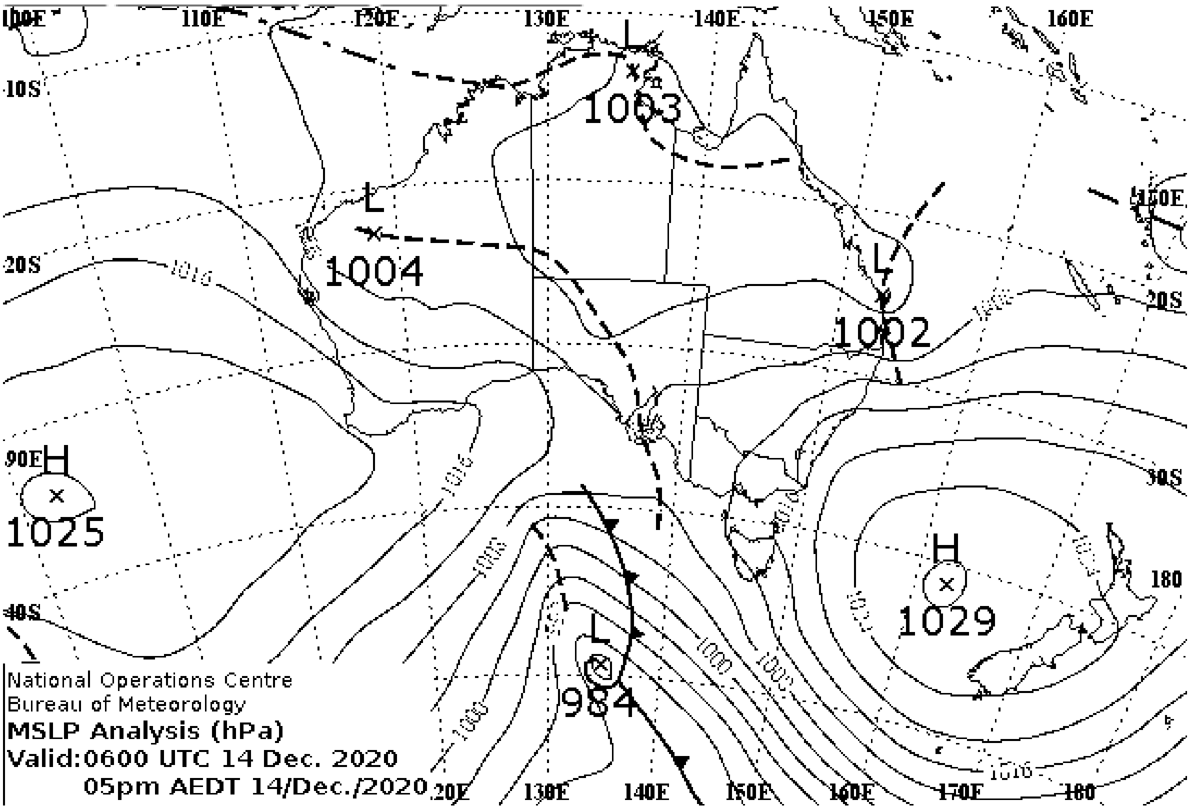


Figure 6.10 Synoptic chart 14 December 2020

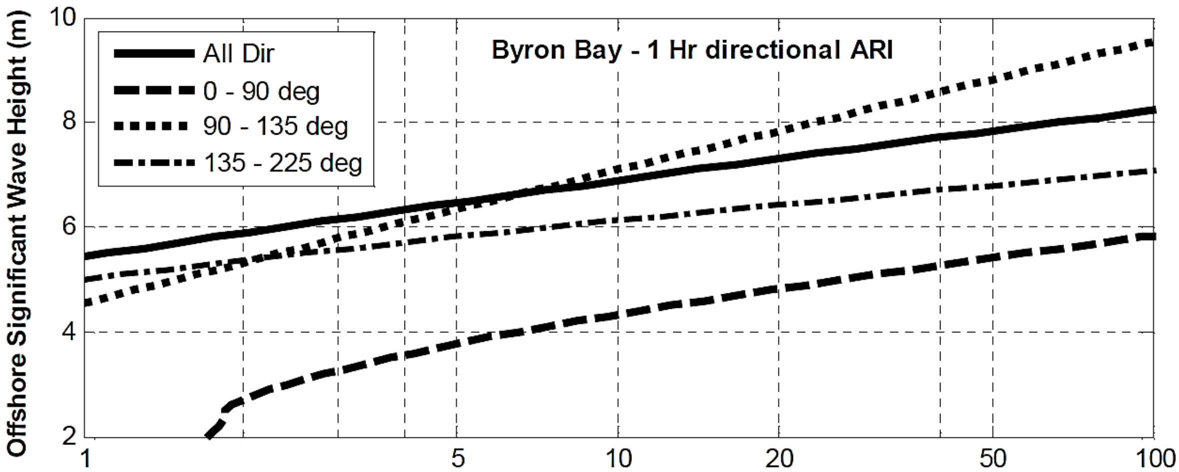


Figure 6.11 Extreme offshore wave statistics for Byron Bay (Shand et al., 2011)

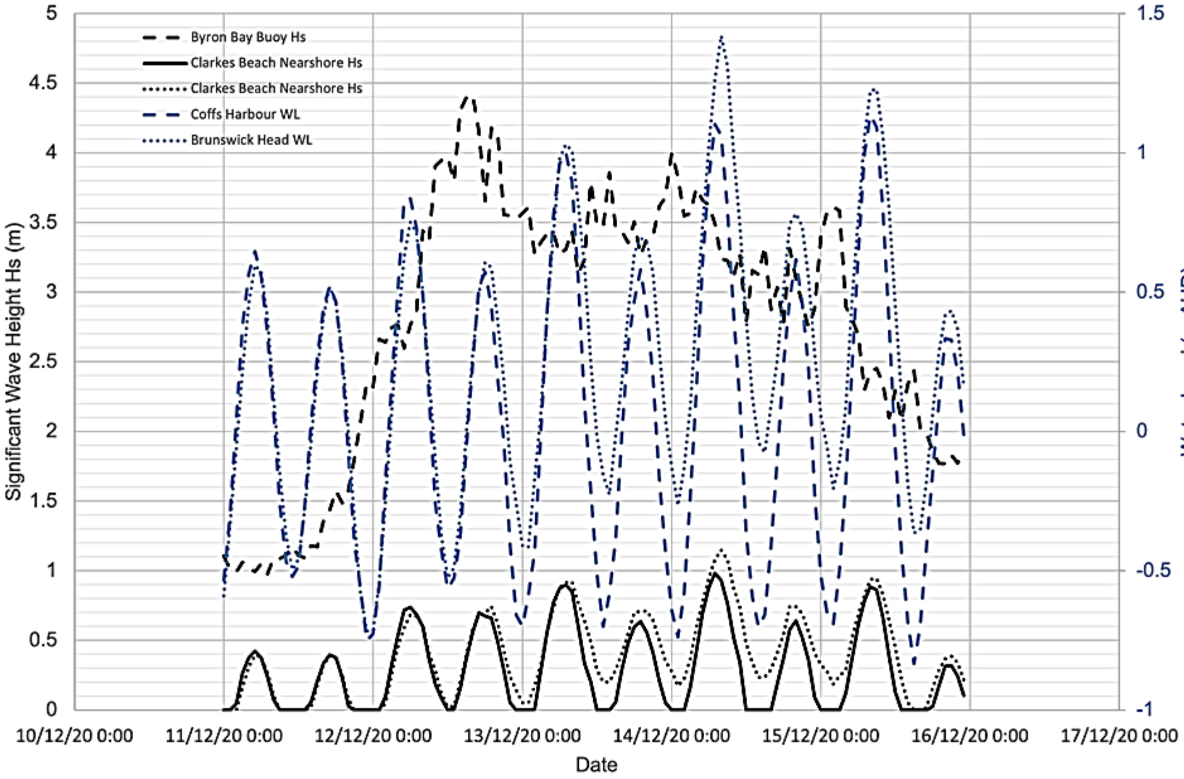


Figure 6.12 Waves and water levels for December 2020 storm event

7 Impact on coastal processes

7.1 Theory on seawall end effects

Substantial work regarding seawall impacts is presented in Kraus and Pilkey (1988) and Carley et al. (2013). Seawall end effects (Figure 7.1 and Figure 7.2) may occur at both ends of seawalls when the waves approach square on (swash aligned beach), whereas they may only occur at the downdrift side of seawalls where the waves almost always approach from one side, which is the case for Clarkes Beach (drift aligned beach).

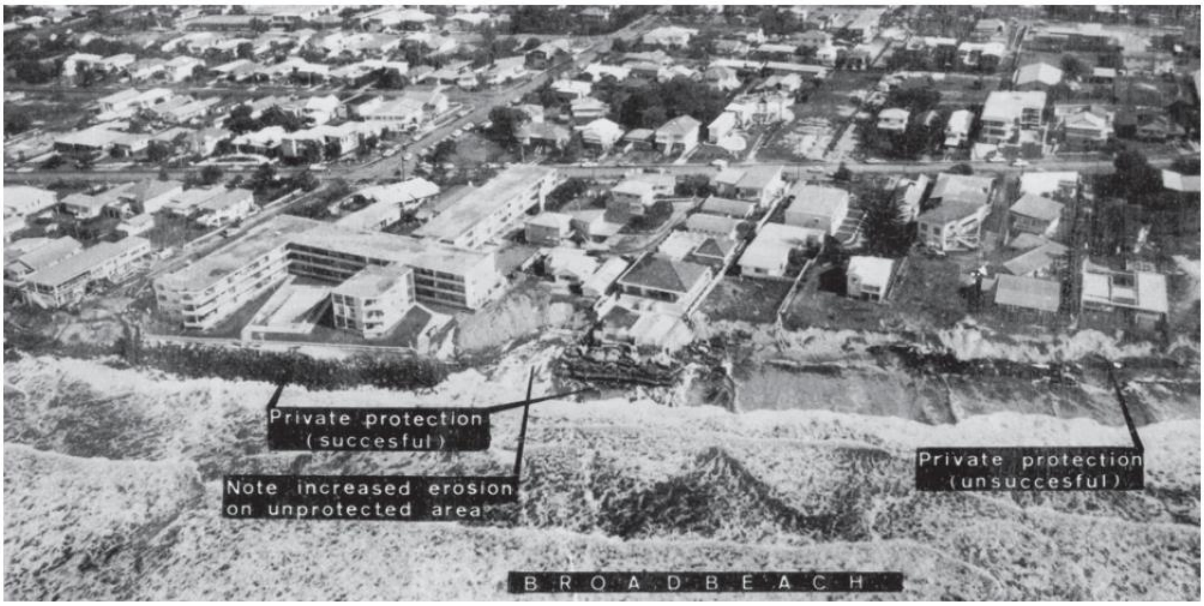
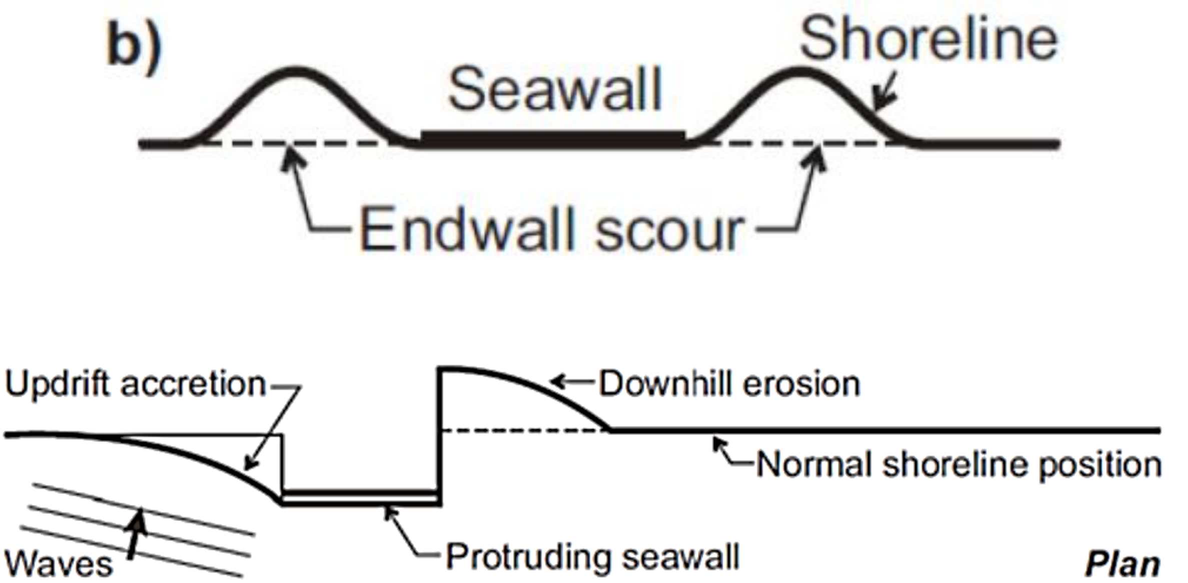


Figure 7.1 Seawall end effect Gold Coast 1967 (Delft, 1970)



Source: Basco (2004) based on Dean (1986)

Figure 7.2 Seawall end effect on swash aligned (top) and drift aligned beaches (bottom)

The classical work presenting seawall end effects is McDougal et al. (1987), who presented the seawall end effect diagram shown in Figure 7.3. No time dependence or erosion event intensity was provided for the planform depicted, nor any dependence of the end effect on the sand volume seaward of the seawall.

Their estimates were:

- r = 10% of l_s, where l_s is the length of the seawall
- S = 70% of l_s
- $r/S = 1/7$

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 as per Figure 7.4. They found that no seawall end effect could be observed for seawalls not frequently exposed to waves.

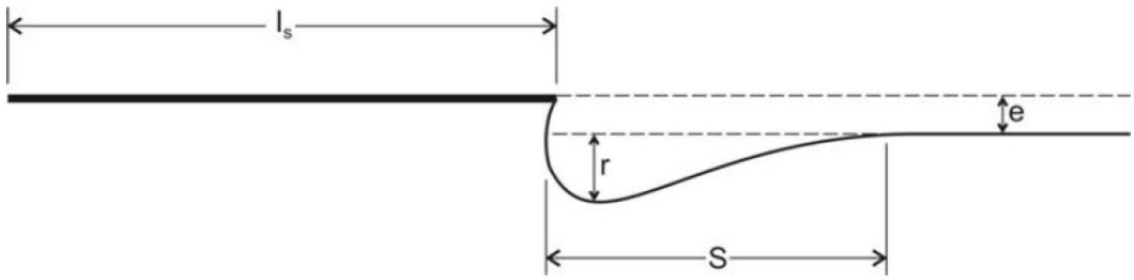


Figure 7.3 Seawall end effect variables

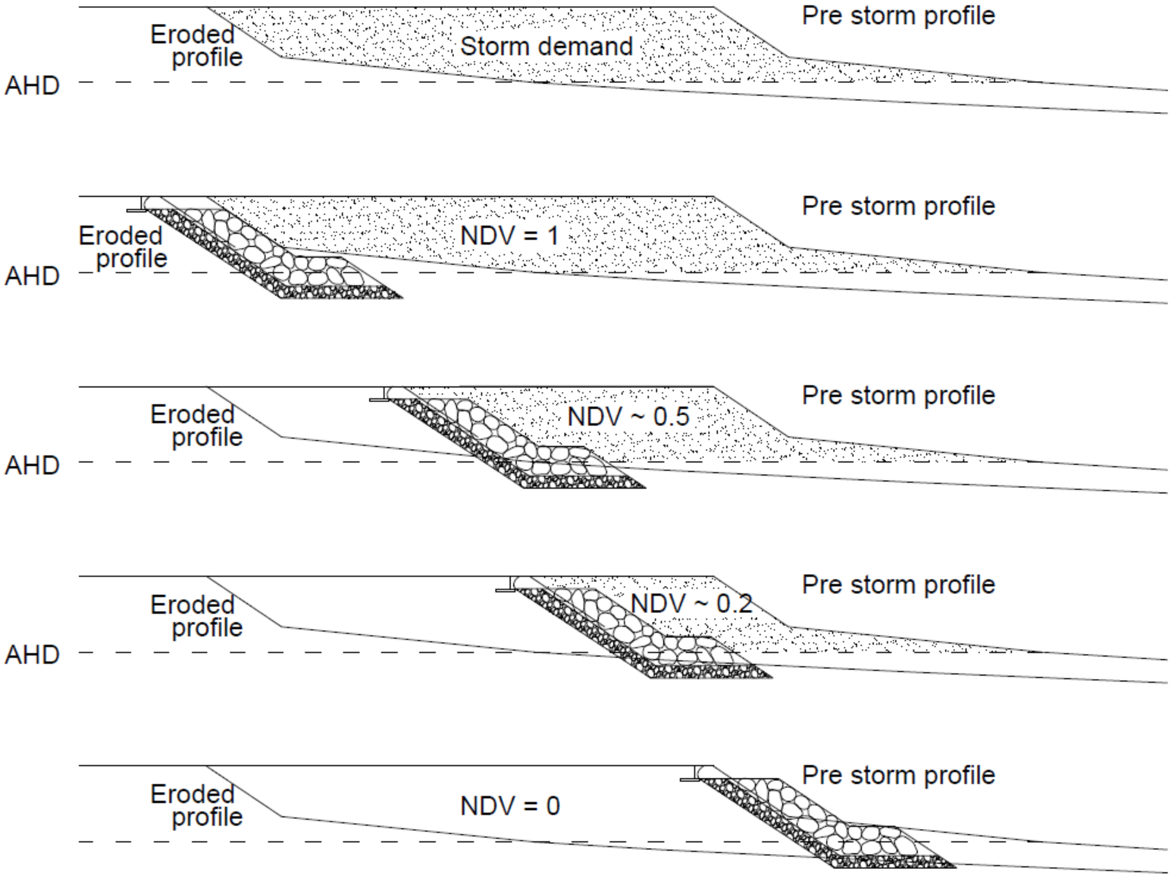


Figure 7.4 Seawall cross shore position

7.2 Observed end effect at Clarkes Beach

Measurements were made of end effects of the Reflections geobags plus the combined Reflections plus Cafe geobags using the data detailed in Table 5.1. The plan position of the vegetation line or 3 m AHD contour was accurate to approximately ±1.5 m for the high resolution photos and ±3 m for the low resolution photos. It should be noted that vegetation can trap wind blown sand, and therefore increase the buffer prior to an erosion event, however, vegetation offers little resistance to open coast erosion caused by the action of waves and tides on sand dunes.

In many photos, an end effect from the Reflections geobags is barely perceptible, and is simply consumed within the beach planform controlled by the rock outcrops further to the east (Captain Cook car park, Thomsons Rock and The Pass).

Observed end effects were estimated in the manner shown in Figure 7.3, that is by mapping the vegetation line, the embayment wide erosion and the local erosion associated with the seawall. End effects observed to date are summarised in Table 7.1. With Clarkes Beach generally appearing to be accreting during 2021, the recent observations may be the maximum extent of end effects, however, this trend cannot be extrapolated for the next 5 years.

The end effects observed to date may not be the totality of end effects over the design life of the works. Scenarios of estimated theoretical end effects during "design" erosion are covered in Section 7.3.

The observed gap between the geobags and the end effect scarp is because:

1. The geobags were returned into the existing embankment, but the embankment eroded further after construction
2. The top of the erosion scarp defined by the vegetation line is higher than the top of the geobags

Table 7.1 Observed end effects

Geobag wall	Embayment Zoom	Close up	r (m)	S (m)
Reflections only	Figure 7.5	Figure 7.6	4	20
Reflections plus Cafe	Figure 7.5	Figure 7.7	5	35

Figure 7.5 Observed end effect – embayment view

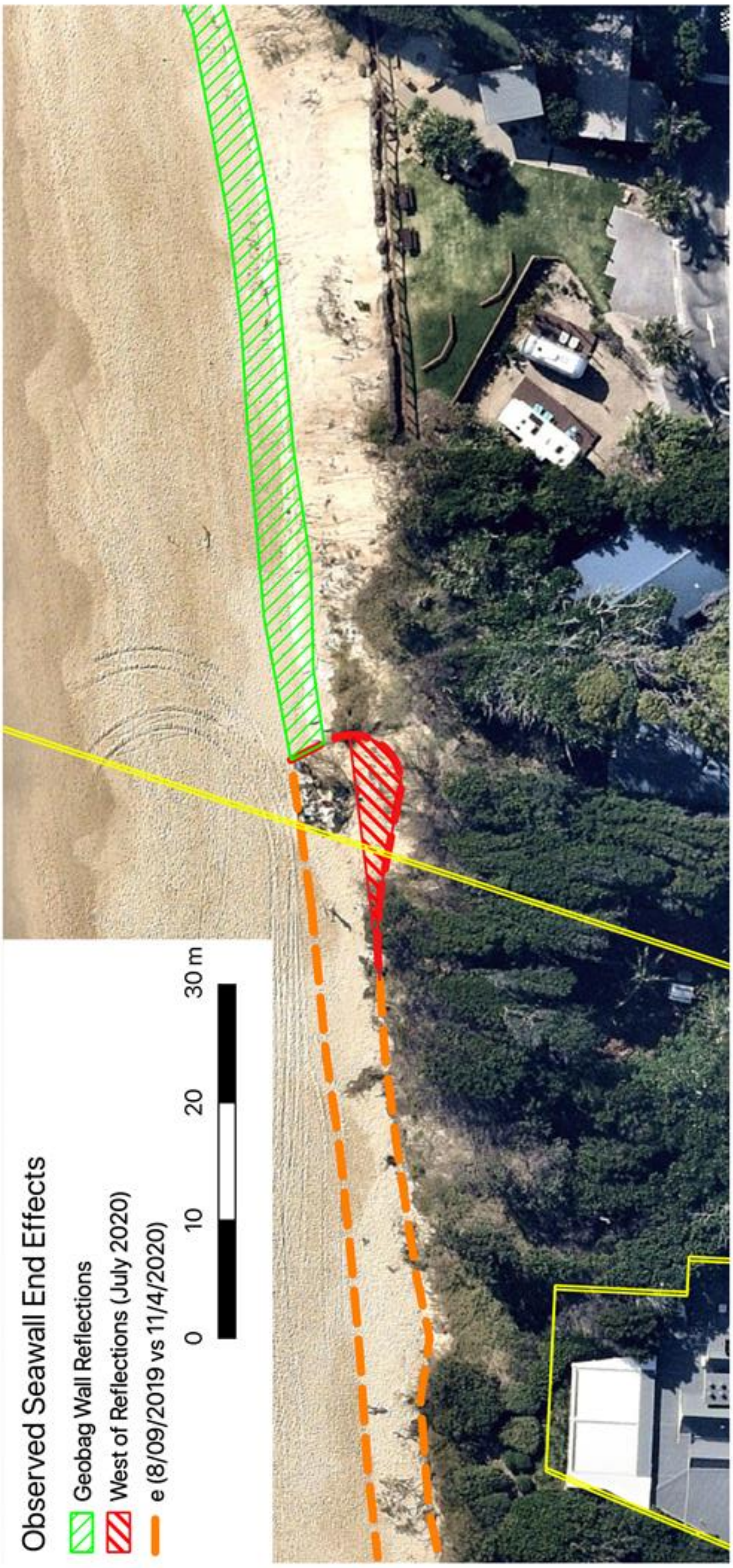


Figure 7.6 Observed end effect to date – Reflections only close up

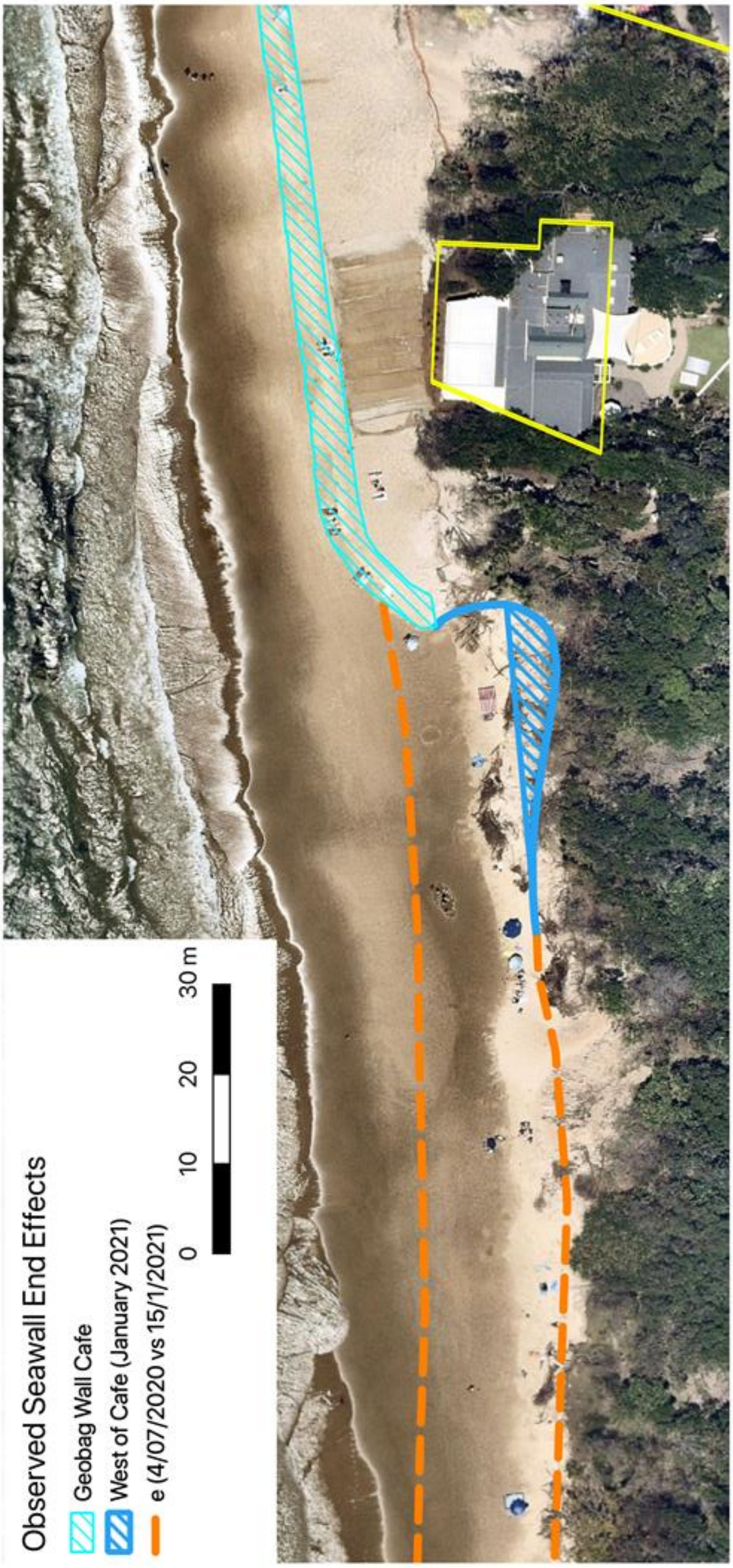


Figure 7.7 Observed end effect to date – Reflections plus Cafe close up

7.3 Future seawall end effect estimates

Two sets of end effect estimates are detailed in Table 7.2, namely the techniques of McDougal et al. (1987) and Carley et al. (2013), in both embayment view and close up. The McDougal et al (1987) technique is the main method which has been published in a peer reviewed journal, however, as stated above, it does not consider:

- The seawall cross shore position on the beach profile
- The magnitude of the end effect has no dependency on the ARI of erosion events, noting that the same end effect can still be added to erosion events of differing magnitude
- The time scale and separation of recession from erosion

The Carley et al. (2013) method attempted to account for these gaps based on measurements from seven Australian seawalls.

Theoretical end effects were applied to the foreshore position from the most recent photogrammetry date of 21 April 2021, so assume that an erosion event (of a designated ARI) occurs on the existing beach state, not on an accreted beach as per BMT WBM (2013).

Probabilistic design erosion volumes were determined in Section 4.2. In the 21 April 2021 photogrammetry, approximately 25 m^3/m of sand above AHD was present seaward of the geobags. This volume was sufficient to withstand a 2 year ARI erosion event, and therefore has no end effect. For application of the Carley et al. (2013) method, the maximum ARI erosion event applied was 20 year ARI. This is because, based on the analysis in this report, the geobags would fail in such an event, and would therefore not withhold sand from the end effect area.

Table 7.2 and the accompanying figures cover the following permutations:

- No geobags
- Reflections geobags only
- Cafe geobags only
- Reflections plus Cafe geobags

As per observations to date, actual end effects may be less that theory because:

- The project life is an estimated 5 years, which may not be sufficient time for the end effect to fully develop
- The beach is presently in a recovery/accretion phase, so there may be a sand buffer fronting the geobags, although this cannot be guaranteed
- Overtopping of the geobags will deliver sand to the end effect area
- Should an extreme event occur, the geobags are likely to be damaged, and may be outflanked on their western side, delivering sand to the end effect area, and therefore limit the end effect

Table 7.2 Future end effect estimates – embayment view and close up

Geobag wall	Method	Erosion ARI (years)	Embayment zoom Figure	Close up Figure	Ls (m)	S (m)	r (m)
No geobags	-		Figure 7.8	Figure 7.9	-	-	-
Reflections	McDougal	5, 10, 20		Figure 7.10	160	110	16
	Carley	5		Figure 7.11		196	3
		10		Figure 7.11		196	6
		20		Figure 7.11		196	10
Cafe	McDougal	5, 10, 20		Figure 7.12	90	62	9
	Carley	5		Figure 7.13		154	3
		10		Figure 7.13		154	6
		20		Figure 7.13		154	10
Reflections + Cafe	McDougal	5, 10, 20		Figure 7.14	250	172	25
	Carley	5		Figure 7.15		250	3
		10		Figure 7.15		250	6
		20		Figure 7.15		250	10

Figure 7.8 Erosion-recession hazard lines – embayment view

Figure 7.9 Erosion-recession hazard lines – close up

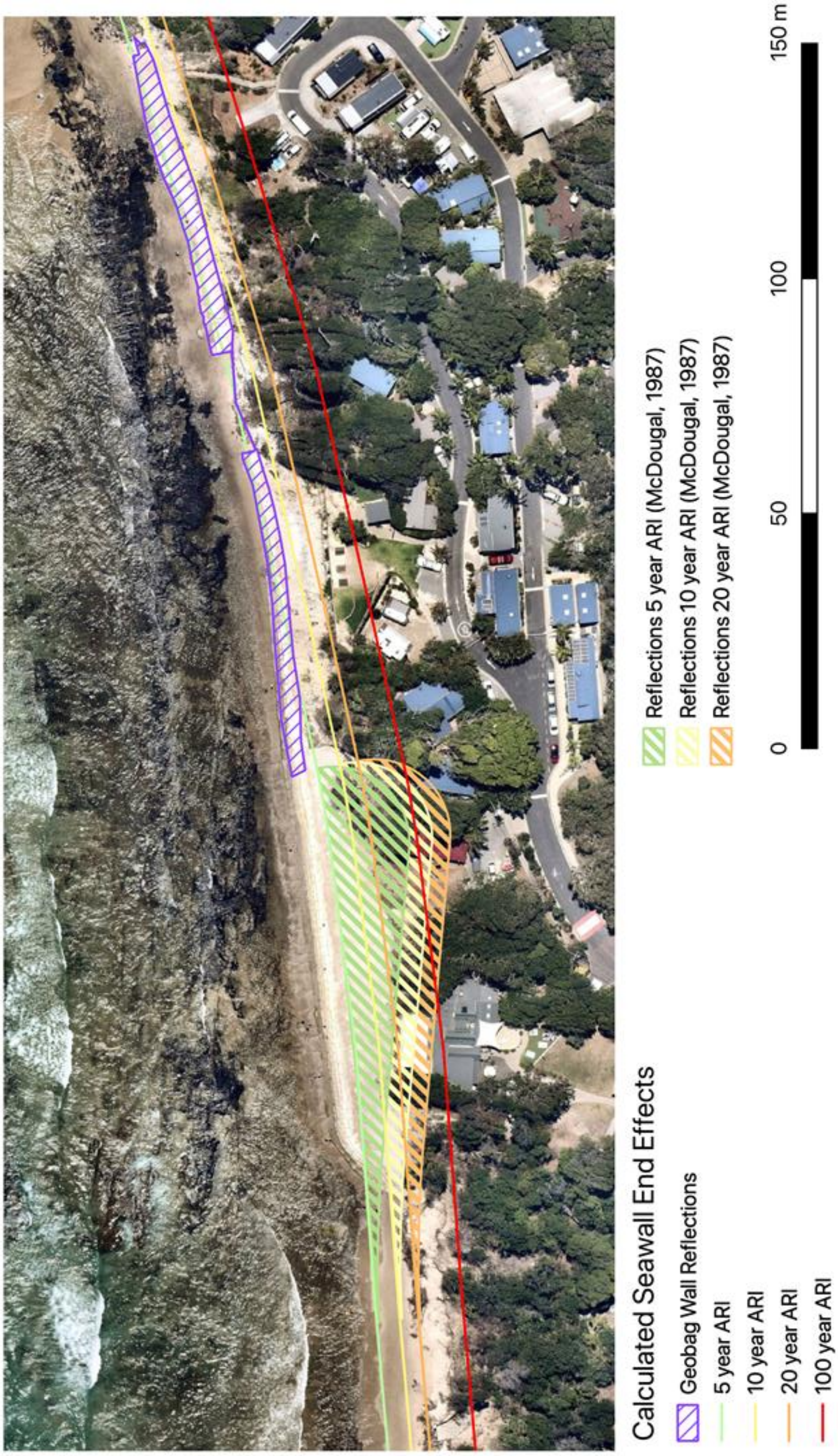


Figure 7.10 Future end effect estimates – Reflections, McDougal, closeup

Figure 7.11 Future end effect estimates – Reflections, Carley, close up

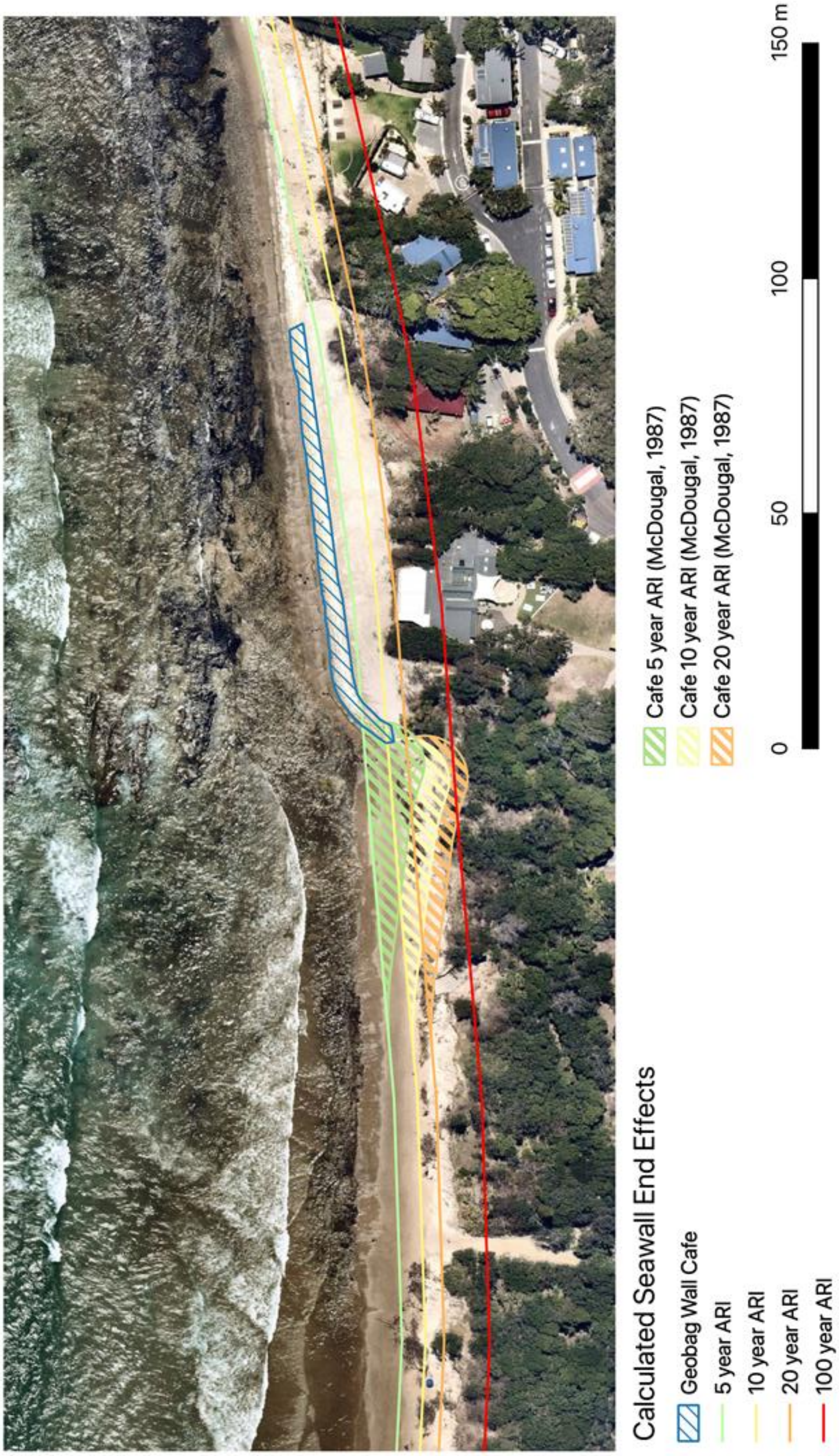


Figure 7.12 Future end effect estimates – Cafe, McDougal, close up

Figure 7.13 Future end effect estimates – Cafe, Carley, close up

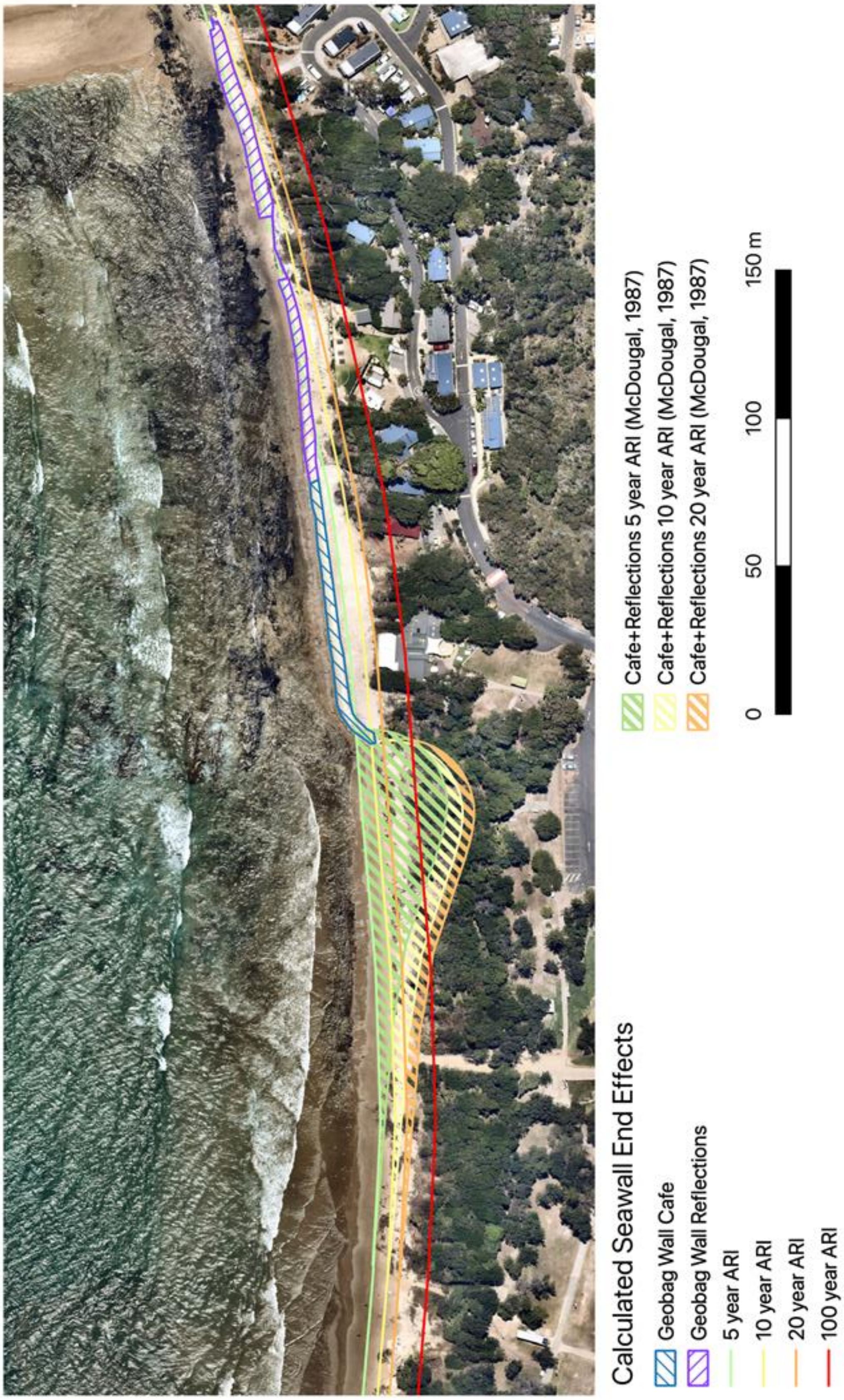


Figure 7.14 Future end effect estimates – Reflection + Cafe, McDougal, close up

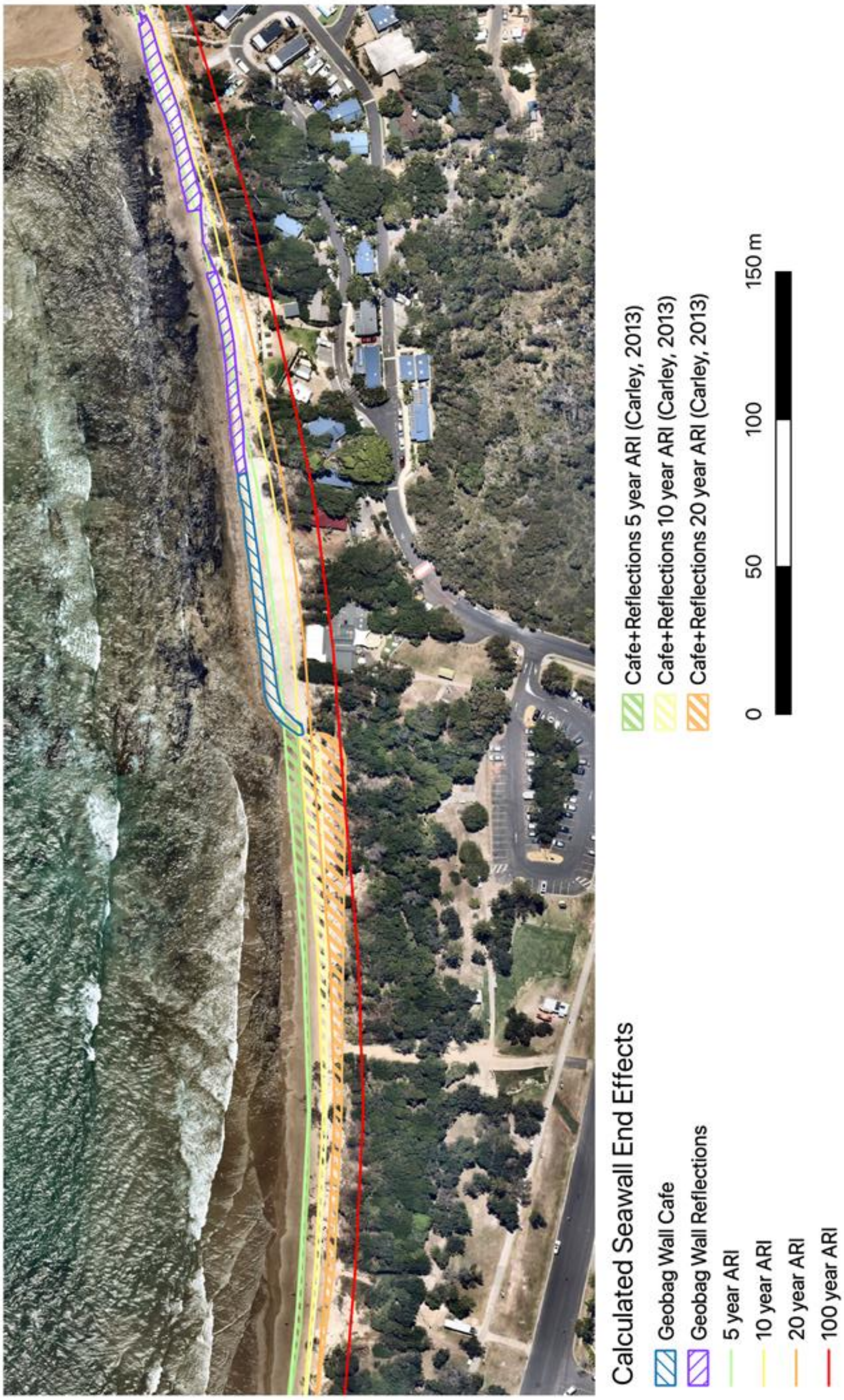


Figure 7.15 Future end effect estimates – Reflection + Cafe, Carley, close up

7.3.1 Locked up sand based on 'approximate principle'

For the long term sediment budget, the Dean (1986) 'approximate principle' applies to coastal recession, not short term erosion. It is used to determine sand contribution requirements to offset seawall impacts in some states of the USA (Lester, Coast to Coast, 2014).

Dean (1986) suggested that excess erosional stress along the front of the structure produces a defined scouring of the level of bed fronting the seawall as shown in Figure 2.2. 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 7.16).

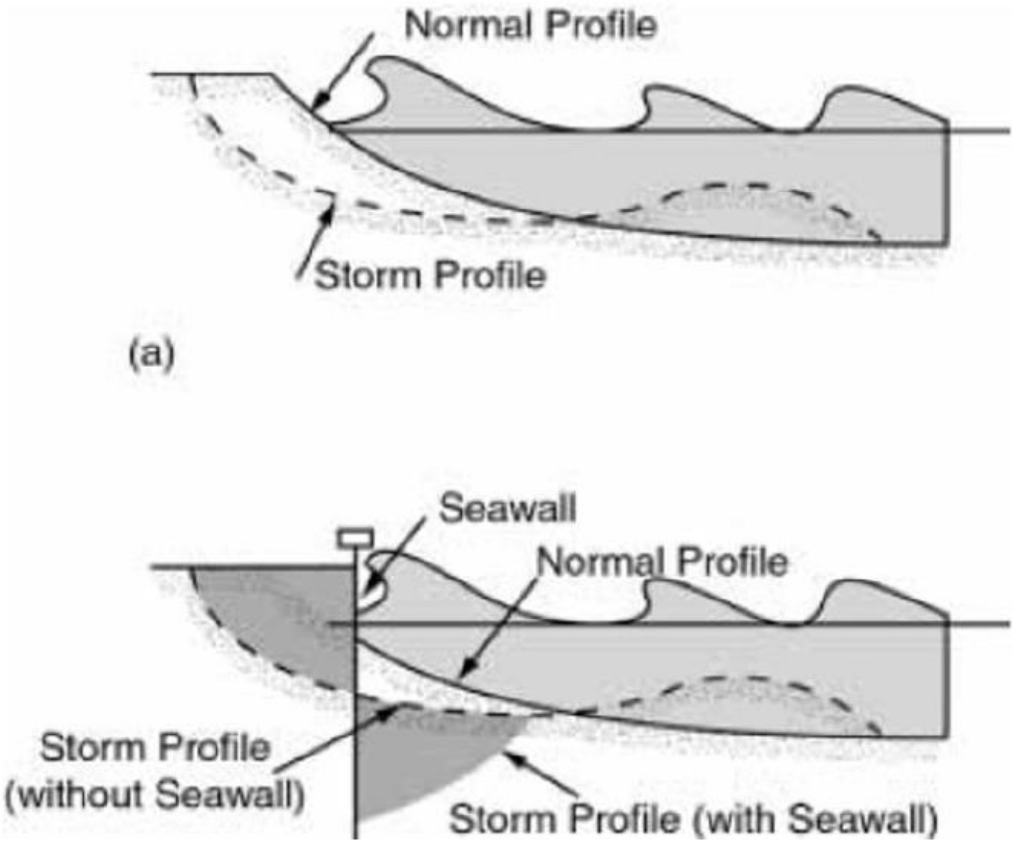


Figure 7.16 Dean (1986) approximate principle of locked up sand

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. However, field evidence of this degree of frontal scour remains unclear, probably because any such effects are quickly re-distributed along and across the beach and such three dimensionalities are not accounted for in the physical models cited. Scaling of sediment dynamics in most small to medium physical model tests is often limited and results should be primarily considered qualitative (Kraus and McDougal, 1996).

Kraus and McDougal (1996) suggested that the approximate principle will not necessarily apply in cases where the profile is in near equilibrium and no demand is made for sand to move out of the profile. For Clarkes Beach, toe scour is limited by the rock/reef, however, as stated above, its application to long term recession quoted in Lester (Coast to Coast, 2014) is a valid means to estimate the locked up sand.

The following inputs were used for application of the Dean (1986) ‘approximate principle’ as shown in Table 7.3:

- Underlying recession: 0.2 m/year
- Sea level rise recession 0.26 m/year
- Active height of protected area: 2.5 m - sand is able to be eroded from above the geobags, however, the volumes would double if it is assumed that the embankment above does not erode, that is, an assumed active height of 5 m

Thus, depending on the assumed active height and wall configuration, the Reflections and Cafe geobag walls could deprive the long term littoral system of 104 to 576 m^3/year when exposed to waves under long term average conditions. This can be compared to other sand quantities in Section 5.2. The BMT WBM (2013) hazard study estimated an uncertainty of $\pm$20% on the recession components. This quantum is a suitable initial allowance for uncertainty.

Table 7.3 Locked up sand based on approximate principle

Variable	Reflections	Cafe	Reflections + Cafe
Underlying recession (m/year)	0.20	0.20	0.20
SLR recession (m/year)	0.26	0.26	0.26
Total recession (m/year)	0.46	0.46	0.46
Wall height (m)	2.5	2.5	2.5
Total active height (m)	5.0	5.0	5.0
Wall length (m)	160	90	250
Volume (m^3/year) for 2.5 m height	**184**	**104**	**288**
Volume (m^3/year) for 5.0 m height	**368**	**208**	**576**

7.4 Management of end effects

It should be noted that the recent major erosion events in 2020 have had an embayment wide scale. A comparison of the magnitude of various factors contributing to coastal change is presented in Section 5.2. The end effect signals from the Reflections and Cafe geobags are difficult to detect in many aerial images and surveys.

The observed geobag end effects are smaller than theory, probably because of the following factors:

- The structure is low and able to be overtopped by waves, thus providing sand from above its crest
- The structure has only been in place for a short duration, and the beach has accreted since the works were constructed, taking the structure out of the active zone
- When the beach is in an eroded state the planform in the vicinity of Reflections and the Cafe is governed by rock outcrops below the Captain Cook car park and Thompsons Rock, further west towards The Pass, together with rock/reefs just offshore from the Cafe which limit erosion depth and control the planform of the beach – this could be further investigated within a revised coastal hazard definition study and/or coastal processes study

Removal of the geobags is discussed in Section 8. Over the estimated 5 year design life, observed and calculated theoretical end effects are not indicated to impact built assets such as roads or car parks, but may impact two pedestrian beach accesses.

Until such time that the interim works can be removed, management of end effects is best undertaken through the following means:

- Management of public safety risk through regular inspection of the beach and dune (Section 8), removal of vegetation at risk of imminent collapse, grading of the erosion scarp to a maximum gradient of 1V:1.5H (34°)
- Establishment of a rolling easement of vegetation, through additional revegetation to replace that lost due to erosion/recession within the end effect area
- Restoration or consolidation of neighbouring beach accesses in consultation with Byron Shire Council
- Sand management through importation of nourishment sand or beach scraping in conjunction with Byron Shire Council

As stated in Section 5.2, the sand volumes locked up by the geobag walls are small relative to other littoral processes. Imported nourishment sand would be viable to maintain the embankment above the geobags and preserve beach access, however, its addition in the quantities attributable to that locked up by the seawall would be imperceptible to embayment wide erosion away from the immediate area of the geobags.

8 Monitoring of geobags and surrounding beach

8.1 Monitoring of geobags

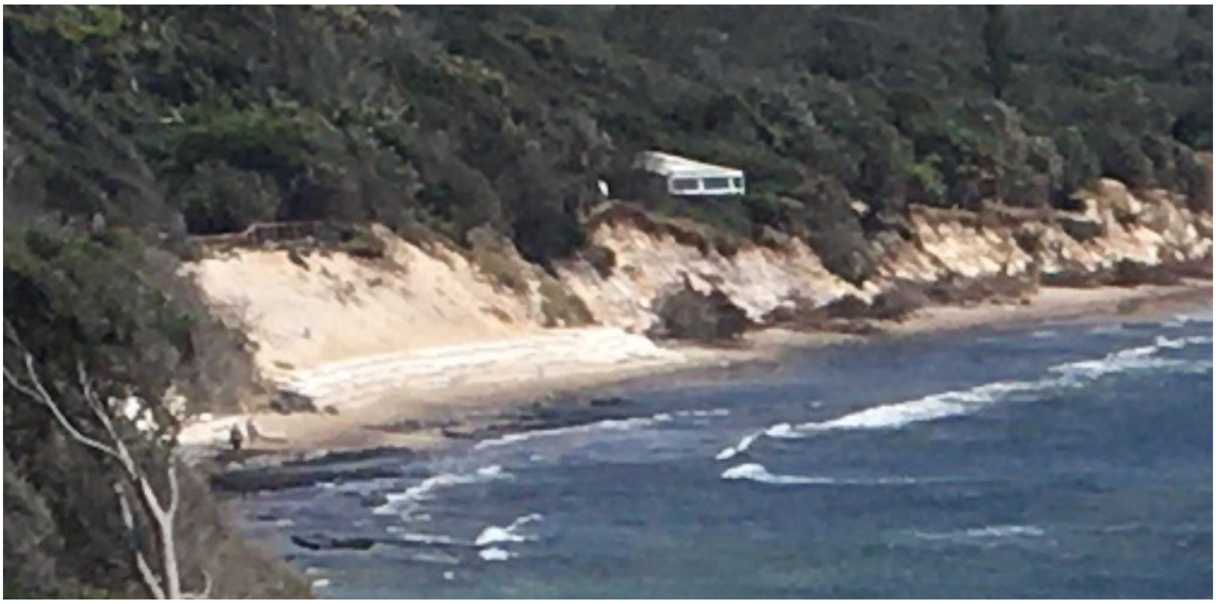
A monitoring and maintenance plan specific to the Cafe and Reflections geobags is proposed to be developed. An example of a sample monitoring plan for a different location is shown in Appendix A. It is recommended that a similar detailed plan be adapted for the site. High level principles for monitoring are described below.

WRL is aware that the installation of a CoastSnap station between The Pass and Clarkes Beach is imminent (Figure 8.1). Photos from this station can detect the Cafe geobags (Figure 8.2, Figure 8.3).

Figure 8.1 Location of proposed CoastSnap station

Source: Chloe Dowsett

Figure 8.2 View from proposed CoastSnap station



Source: Chloe Dowsett

Note: Image was taken prior to construction of Cafe geobags. Reflections geobags are visible. Due to distance, geobags are not highly visible, but exposure can be observed, and can be detected by automated routines

Figure 8.3 Zoomed image from proposed CoastSnap station

The following activities are recommended for monitoring of the geobags:

- A weekly photo from the CoastSnap station or closer reference location by a designated party with basic analysis by Crown Lands. If two or less courses of geobags are visible, and those courses are not substantially displaced, no additional monitoring is required for that week.
- If more than two courses are exposed, or the BoM issues warnings for dangerous surf, damaging surf or abnormally high tides, additional daily high and low tide photos should be taken for Crown Lands by a designated party and forwarded to Crown Lands.
- If the photos reveal substantially exposed geobags or displaced geobags, the works should be inspected as soon as practicable by a coastal engineer or competent person to assess for damage and/or hazardous geobags (e.g. having incipient instability). Subject to an assessment of the beach state, a UAV survey consistent with the initial survey should be initiated to assess sand and geobag change.
- Should the sand behind the geobags become eroded, a geotechnical/structural engineer should be consulted for advice.
- Until the Cafe can be retreated safely and/or the erosion risk otherwise managed, loss of more than 2% of geobags from the wall (which equates to approximately 14 geobags) over the 90 m of Cafe wall, displacement or loss of crest bags such that their ability to resist wave runup is reduced, or loss of the sand slope needs to be rectified as soon as practicable. The sand slope above the geobags is unlikely to accrete naturally over the estimated 5 year project life, so any erosion of this will need to be managed through the placement of imported sand.

8.2 Monitoring of beach and geobag impacts

The following activities are recommended for monitoring of beach and geobag impacts:

- An initial UAV LiDAR/optical land survey undertaken at low tide, extending at least 500 m alongshore beyond the geobags
- Three monthly assessment of the evolution of the geobag end effects using the following hierarchy of data sources, as they become available, and a minimum data frequency of once per month:
    - Measured LiDAR/optical/photogrammetry land survey
    - High resolution aerial or satellite photos
    - Shoreline or vegetation mapping through a CoastSnap station
    - Low resolution aerial or satellite photos

8.3 Future removal of geobags

As stated in Section 2, structural and geotechnical analysis by Ardill Payne indicates that removal of the geobags while the beach is in an eroded state, and consequent erosion of the sand slope fronting the Cafe will result in the collapse of all, or part of, the Cafe onto the beach below. Under most historic beach conditions, the geobags are likely to be predominantly buried. Removal of fully buried geobags is not recommended, as this will require substantial excavation, and contribute to additional dune erosion, disturbance of established dune vegetation and disruption.to beach users.

Removal of the geobags needs to be undertaken in a coordinated manner, with consideration of the geotechnical stability of the Cafe building and Reflections Holiday Park assets. The 5 year timeline of the proposals is to enable strategic planning and accompanying actions to manage the erosion risks at both sites. When these reviews and corresponding actions are completed within the 5 year timeframe, subject to the concurrence of a geotechnical and/or structural engineer, the geobags can be removed opportunistically if/when they are exposed. It should be noted that this may cause additional erosion of the slope above the geobags. Management of the slope requires advice from a geotechnical engineer. Removal needs to consider public safety, coordination with development at Reflections and suitable beach conditions – sufficient sand for safe working, but sufficient erosion for the geobags to be predominantly exposed.

Removal is best affected by cutting one end of the geobag, emptying the sand and reusing it in the area of most benefit (e.g. beach access), and disposing of the empty geobag.

9 Coastal Management Act 2016

9.1 Section 27 of Coastal Management Act 2016

Section 27 of the Coastal Management Act 2016 states:

27 Granting of development consent relating to coastal protection works

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

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

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

9.2 Cafe geobags and Section 27 of Coastal Management Act 2016

The following commentary is provided regarding the interaction of the Cafe geobags and Section 27 of the Coastal Management Act 2016.

Life

The life of the works is an estimated 5 years. This is the estimated time frame required to reconfigure/relocate the Cafe.

Public access

The works will impact alongshore pedestrian access when the beach is in an eroded state, such as it was in November and December 2020 (Figure 9.1), especially at high tide. This intermittent impact on alongshore access will be limited to the estimated 5 year life of the works. When the beach is accreted, especially at low tide, the works do not affect beach access. In consultation with Byron Shire Council, it is proposed to repair and restore the beach access point to the west of the works, so that public access to Clarkes Beach is preserved.

Given that impacts to alongshore public access are limited to times when the beach is in an eroded state, and that the structure is intended to be retained for the estimated 5 year life, the overall impact to public access is not considered to be unreasonable.

Photos: James Carley 18/12/2020

Figure 9.1 Alongshore public access when beach is eroded

Beach use – surfing and swimming

The works have no impact on surfing and swimming when the beach is accreted. During times of an eroded beach state and high tides, the works may impact entry and egress to the water. Geobags are softer than rock or concrete, but may become slippery if they are frequently in the splash zone. It should be noted that the works are distant from the main surfing areas of Byron Bay (The Pass, Main Beach, The Wreck) and the patrolled swimming area of Main Beach. Whatever the beach state, the works will not adversely impact the predominant open water swimming route which runs from The Pass to Byron Bay SLSC. In light of the above, impacts to beach use are not considered to be unreasonable.

Increased erosion

The impacts of the works on coastal processes are discussed in Section 7. The works prevent the transfer of sand from the dune to the lower beach. It is proposed to import sufficient sand to replace the sand locked up by the works.

It should also be noted that parts of Clarkes and Main Beach without protection, this dune sand contribution is insufficient to cover the exposed rock/reefs, and involves the collapse of vegetation and formation of potentially dangerous scarps (Figure 9.2). The works prevent this occurring over their alongshore extent for storm wave conditions up to an approximately 5 year ARI storm wave event.

Photos: James Carley 18/12/2020 top, 23/04/2021 bottom

Figure 9.2 Erosion and collapsing vegetation away from the works

Public safety

The main potential risks to public safety are:

- Geobags having incipient instability
- The collapse of trees or steep erosion scarps
- The collapse of the Cafe building onto the beach and/or Reflections Holiday Parks assets, such as Whites Cottage, onto the beach

Provided that the monitoring and maintenance program is completed and followed, the threat to public safety arising from the above risks is low.

Beach restoration

Section 7.4 of this report provides a method to restore the beach and land adjacent to the works, with the main components being a rolling easement of vegetation and grading of the scarp to a safe angle. Should the beach continue on its present trajectory towards an average/accreted state, there will not be further erosion/recession over the life of the works. If a new trend of erosion/recession establishes at the site, restoration of the beach can be effected by the following means:

- Beach nourishment with small scale imported sand sufficient to restore beach access and replace sand locked up by the works
- Beach scraping
- As per the design life, reconfigure/relocate the Cafe, remove the works and make the surrounding scarps safe
- An embayment wide beach nourishment scheme or other action within a Coastal Management Program (outside the scope of this geobag proposal)

Maintenance

A detailed monitoring and maintenance plan for the structure and surrounding beach will be prepared (Section 8).

Rock structures are generally considered to have maintenance requirements of approximately 1% per annum of the capital cost (Gordon, 1989), however, this may be episodic and reactive.

Maintenance of the works fronting the Cafe may involve the following activities:

- Replacement of displaced geobags
- Replacement of damaged geobags
- Re-establishment of the sand embankment behind the geobags
- Grading of the surrounding escarpments
- Maintenance or restoration of beach access

Based on engineering experience, the geobag structure fronting the Cafe is likely to have maintenance costs of 2 to 5% per annum of the capital cost. This would amount to $20,000 to $50,000 per annum on average, noting that if the beach remains accreted, the maintenance requirements will be minor over the estimated 5 year life.

Sand locked up by works

The ‘approximate principle’ (Dean, 1986; Section 7.3.1) regarding sand withheld by the Cafe works under recession was applied. This is based on the 2.5 m high geobag wall withholding sand above it to an active height of 5 m.

This yields the quantities shown below for the case where the beach remains eroded:

- Sand locked up by recession acting on Cafe geobags: 208 m^3/year
- Sand locked up by recession acting on Reflections geobags: 368 m^3/year
- Sand locked up by recession acting on Reflections + Cafe geobags: 576 m^3/year

The proposed beach nourishment with imported sand associated with the works can offset this locked up sand. The BMT WBM (2013) hazard study estimated an uncertainty of $\pm$20% on the recession components. This quantum is a suitable initial allowance for uncertainty.

10 Summary

This summary is reproduced in the Executive Summary at the start of this report.

Clarkes Beach and Main Beach Byron Bay have experienced beach erosion in 2020 and 2021 that has been described as “unprecedented”. This has resulted in the loss of vegetation, closure of many beach access ways, exposure of normally buried rocks and reefs, diminished beach amenity, unearthing of indigenous artefacts, and the potential collapse of built assets such as those within the Reflections Holiday Park, Clarkes Beach, Byron Bay (Reflections) and the Beach Byron Bay Cafe building (the Cafe, also commonly referred to by its previous name of Clarkes Beach Cafe). The Cafe was determined by coastal, geotechnical and structural engineers to be at imminent risk of collapse onto the beach. Interim geobag seawalls were designed and constructed so as to prevent this collapse, while longer term management is being developed.

Interim geobag (0.75 m^3) walls were constructed fronting Reflections in July 2019 in two lengths of approximately 70 m each, separated by a short length (22 m) comprising a stormwater pipe, degraded gabions, coffee rock, boulders and cobbles, with a total effective length of approximately 160 m. In October/November 2020, an approximately 90 m long geobag wall was constructed in front of the Beach Byron Bay Cafe. The new wall is contiguous with and westward of the Reflections geobag wall. An additional course of geobags was added to a large section of the crest of the Café geobag wall in December 2020 in response to a large storm wave event that overtopped the geobag wall and eroded some of the backfill. The wall was offset seaward of the base of the erosion escarpment to provide geotechnical stability to the Café building and the sand dune. It was backfilled with compatible sand at a stable angle of repose.

The geobag works fronting the Café were originally proposed for a 90 day design life, so that additional planning regarding the future of the Café could be made. This report examines a pathway to extend the design life by an estimated 5 years, to allow additional time to plan for the future of the café.

This report provides an assessment of the individual and combined impact of the geobag works (that is, both the Reflections and Cafe works) on coastal processes over the estimated 5 year design life, and the monitoring and maintenance requirements that will be associated with the geobag works. The report will inform the development application and environmental impact statement that is being submitted for the works under NSW planning legislation.

Numerous studies have quantified the coastal processes and coastal hazards prevailing at Clarkes Beach and Byron Bay since 1978. Many of these have been more focussed on Belongil, where more than 50 houses are located within the hazard zone. All studies confirm that most of the Byron Bay embayment is undergoing long term recession, however, this involves long periods of erosion and accretion, with sand slugs travelling through the system from east to west. The long term recession is partially due to the southward flowing East Australian Current transporting sand into deeper water to the south-east, thereby removing sand from the active littoral system. Ongoing recession due to sea level rise is also likely to be occurring.

The most recent coastal hazard definition study was undertaken in 2013, and utilised the accreted beach state from 2007 as the baseline for calculations. Therefore, 2021 is now approximately 33% of the way from the then “present” hazard lines (2007) and the 2050 hazard lines. This WRL report has produced revised “present” hazard lines for 2021. The progression towards 2050 on a receding coast, accompanied with sea level rise can account for much of the recent “unprecedented” erosion on Clarkes Beach and Main Beach.

Long term measured wave buoy data (Byron Bay wave buoy), measured tide gauge data (Coffs Harbour and Brunswick Heads) and sophisticated wave transformation modelling estimated extreme waves at the geobag structure. The stability of the geobags was estimated using techniques developed by WRL and accepted in international coastal engineering literature and practice.

The geobag structures are underlain by a reef/rock layer which limits vertical scour in their vicinity. With ongoing beach accretion being evident during 2021, sand fronting the geobags reduces the wave height and force able to impact the geobags. If the beach scours down to the reef/rock layer, WRL estimated that more than 2% of the geobags would be displaced in a 5 year average recurrence interval (ARI) wave event, necessitating repairs. Overtopping may also erode some of the backfill sand. Detailed calculations found that the waves that impacted the geobags during the December 2020 storm event were approximately 1 to 2 year ARI. The geobag wall was undamaged, however, wave overtopping eroded some of the backfill sand, which was subsequently topped up.

The impacts of the geobag works on coastal processes were also assessed. The end effects observed to date are minor. The sand retained by the works and therefore withheld from the littoral system is less than 1% of the quantity of other littoral processes, and is far less than even the incidental/accidental removal of beach sand by people visiting the beach. Potential seawall end effects extend into two beach access points. Collaboration with Byron Shire Council is proposed to ensure preservation of beach access.

Until such time that the interim works can be removed, management of the impacts of the works is best undertaken through the following means:

- Management of public safety risk through regular inspection of the beach and dune (Section 8), removal of vegetation at risk of imminent collapse, grading of the erosion scarp to a maximum gradient of 1V:1.5H (34°)
- Establishment of a rolling easement of vegetation, through additional revegetation to replace that lost due to erosion/recession within the end effect area
- Restoration or consolidation of neighbouring beach accesses in consultation with Byron Shire Council
- Sand management through importation of nourishment sand or beach scraping in conjunction with Byron Shire Council

Principles and options for ongoing monitoring and maintenance are presented, with a detailed program to be developed as a condition of approval. Extending the design life from 90 days to 5 years means that additional maintenance may be required and there is a moderate chance of damage requiring repairs or reconstruction.

Within the report, an engineering opinion is provided regarding how the works comply with Section 27 of the NSW Coastal Management Act 2016.

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Appendix A Example geobag monitoring program

5 May 2016

WRL Ref: 2015085 L20160505

UNSW AUSTRALIA

Water Research Laboratory

Mr Warren Brown and Mr Michael Hourn
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Gosford City Council
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By e-mail: Warren.Brown@gosford.nsw.gov.au

Dear Warren,

Monitoring and Inspection Program for Ettalong Geobag Wall

1. Introduction

The Water Research Laboratory (WRL) was engaged by Gosford City Council (GCC) to design emergency coastal protection works at Ocean Beach Ettalong Point, in response to severe erosion threats to The Esplanade and its associated foreshore footpath/cycleway. Sand-filled geotextile containers (or sandbags) were chosen to mitigate further erosion. Ongoing monitoring of the structure was recommended, with repairs, modification and maintenance to be undertaken if required.

This document outlines a monitoring program of the geotextile seawall and surrounding environment for GCC to implement. It has been based on the best practice coastal engineering methods outlined in the Coastal Engineering Manual (CEM, 2006), CIRIA The Rock Manual (2007) and the US Army Corps of Engineers (USACE, 1998).

The document provides background information to support GCC staff during inspections by detailing possible modes of damage and failure as well as including guidance on suitable timeframes for periodic inspections. A single page inspection checklist is attached in Appendix A as a guide for GCC staff to follow during inspections.

2. Background

This section details relevant background information pertaining to the monitoring program and the condition index (Oliver et al. 1998) that is recommended by WRL to be applied to the Ettalong Point geotextile seawall in the inspection checklist.

Monitoring is an integral part of life cycle management and a regular monitoring program enables structures to be evaluated for safety, functionality and structural soundness. The complexity and scope of a monitoring program can vary from simple periodic onsite visual inspections to elaborate long term measurement programs (USACE, 2011). Techniques should be repeatable and provide quantitative information on the condition of the structure and how it relates to potential failure modes (CIRIA, 2007).

Any monitoring activity should aim to document:

- The present state of the structure (including information on its deterioration and the effect of the surrounding environment);
- The future rate of deterioration and likely life, and any maintenance required to maintain the design or amended design life.

Inspections should be undertaken on a regular basis as part of an effective monitoring program. PIANC (2004) describes three basic types of inspections, these being:

- Periodic inspections;
- Extended inspections which are conducted if recommended following a period inspection; and
- Special inspections which occur following damage or a major storm event.

3. Condition Index

The Condition Index System (Oliver et al. 1998; Pirie et al. 2005) is a rational and consistent method for long-term evaluation of a structure's condition and functionality based on periodic inspections. It is a performance based evaluation system, with emphasis placed upon how well the structure is functioning rather than just the current physical condition relative to the original condition.

Evaluation is performed using numerical ratings as follows.

The **Structural Index** indicates physical condition and structural integrity. Appropriate ratings are determined for the six categories listed below. Rating guidance tables are provided by Pirie et al. (2005), and follow the general Structural Index rating table format (Table 1):

- Loss of elevation or alignment;
- Structural damage or defects;
- Material deterioration or defects;
- Loss of fill level;
- Loss of scour and wave protection; and
- Loss of foundation support.

The **Functional Index** assesses how well the structure is performing relative to the design objectives. The general form of the Functional Index scale is shown in Table 2. Potential structure functions are divided into four main areas, detailed below, noting that some of the functions relate to breakwaters rather than seawalls. Detailed guidelines on assigning function ratings for each category as given by Pirie et al. (2005) are:

- Controls waves and currents to permit full use of the harbour area;
- Controls waves and currents to permit full use of the navigation channel and entrance;
- Controls movement, build-up, and loss of sediment within navigation areas and along adjoining shorelines; and
- Protects nearby structures, or portions of itself, from wave attack or erosion damage.

The **Condition Index** is a weighted combination of the Functional and Structural Indices and provides an overall quantitative assessment of performance. The Condition Index scale is shown in Table 1, with detailed calculation methods given by Oliver et al. (1998) and Pirie et al. (2005).

Effective monitoring relies on the availability of quantitative methods of inspection and evaluation of the condition and performance of a structure. When inspections are conducted by multiple observers, as is usually the case over the life of a structure, individual evaluations are largely subjective with experience and prior site visits playing an important role. The Conditional Index System has the advantage of providing uniform criteria for evaluation of condition and performance, resulting in consistency between observations, as well as better tracking over time, through use of numerical ratings.

Table 1: Condition Index scale (Source: Oliver et al, 1998)

Observed Damage Level	Index Range	Condition Level	Description
Minor	85 to 100	EXCELLENT	No noticeable defects. Some aging or wear may be visible.
	70 to 84	GOOD	Only minor deterioration or defects are evident.
Moderate	55 to 69	FAIR	Some deterioration or defects are evident, but function is not significantly affected.
	40 to 54	MARGINAL	Moderate deterioration. Function is still adequate.
Major	25 to 39	POOR	Serious deterioration in at least some portions of the structure. Function is inadequate.
	10 to 24	VERY POOR	Extensive deterioration. Barely functional.
	0 to 9	FAILED	No longer functions. General failure or complete failure of a major structural

PIANC (1992) recommends that inspection programs should also aim to document the following information:

- The present state of the structure including information on its deterioration and the effect of the surrounding environment;
- The future rate of deterioration and likely life; and
- The maintenance required to maintain the design or amended design life.

PIANC (1992) also suggests that the following information should be recorded at each inspection:

- Drawings;
- Photos and video recordings; and
- Site measurements, e.g. to monitor settlement or erosion/accretion trends.

PIANC (1992, Part 6) suggests that periodic inspections of geotextile coastal protection structures should occur several times a year and especially following severe storm events.

4. Identification of damage

It is essential for GCC staff members carrying out the inspection program of the geotextile seawall, to be aware of the appropriate features and issues to be identified during each inspection. It is also useful to understand the different mechanisms by which a geotextile seawall may fail or experience damage, which are detailed below. The identification of seawall stability issues can be made by GCC staff but full quantification of the issue may require additional input from a coastal engineer (CE) and a geotechnical engineer (GE).

A classification system to quantify damage and failure to geotextile seawalls from wave impacts was proposed by Coghlan et al (2009) which is reproduced in Table 2. Geotextile seawall failure/damage was expressed in percentage terms and defined as the number of displaced sandbags divided by the total number of sandbags within a reference region × 100%. This damage classification has been included in the inspection checklist to track damage over time and to distinguish when the seawall has been deemed to have failed.

Table 2: Damage Classification for Sandbag Seawalls (1.0V:1.0H to 1.0V:2.0H)

Damage Classification	**Double Layer** (% Displaced)
No Damage	0%
Initial Damage	0-2%
Intermediate Damage	2-15%
Failure	≥15%

4.1.1 Two Dimensional Failure/Damage Modes

The main two dimensional potential failure/damage modes of sandbag seawalls are listed below, with further input required by either a Coastal Engineer (CE) or a Geotechnical Engineer (GE) to fully quantify the seawall stability. The main two dimensional potential failure/damage modes are:

- Undermining (CE), in which the sand or toe level drop below the footing of the seawall, causing the seawall to subside and collapse into the hole (Figure 1);
- Sliding (GE), in which the seawall slides seaward from the retained profile (Figure 1);
- Overturning (GE), in which the seawall rotates seaward (forward) (Figure 1);
- Deep seated shear failure (GE), in which the entire embankment fails (Figure 1) -note this is often referred to as "slip circle" failure but in reality a circular failure mode is not the critical kinematic mechanism in sands;
- Deep seated shear failure through the sand behind the wall (Figure 1) and through a sandbag wall;
- Sandbag displacement (CE), due to wave impact (Figure 1); or
- Erosion of the backfill (CE), caused by wave overtopping, high water table levels, or leaching through the seawall (Figure 1).

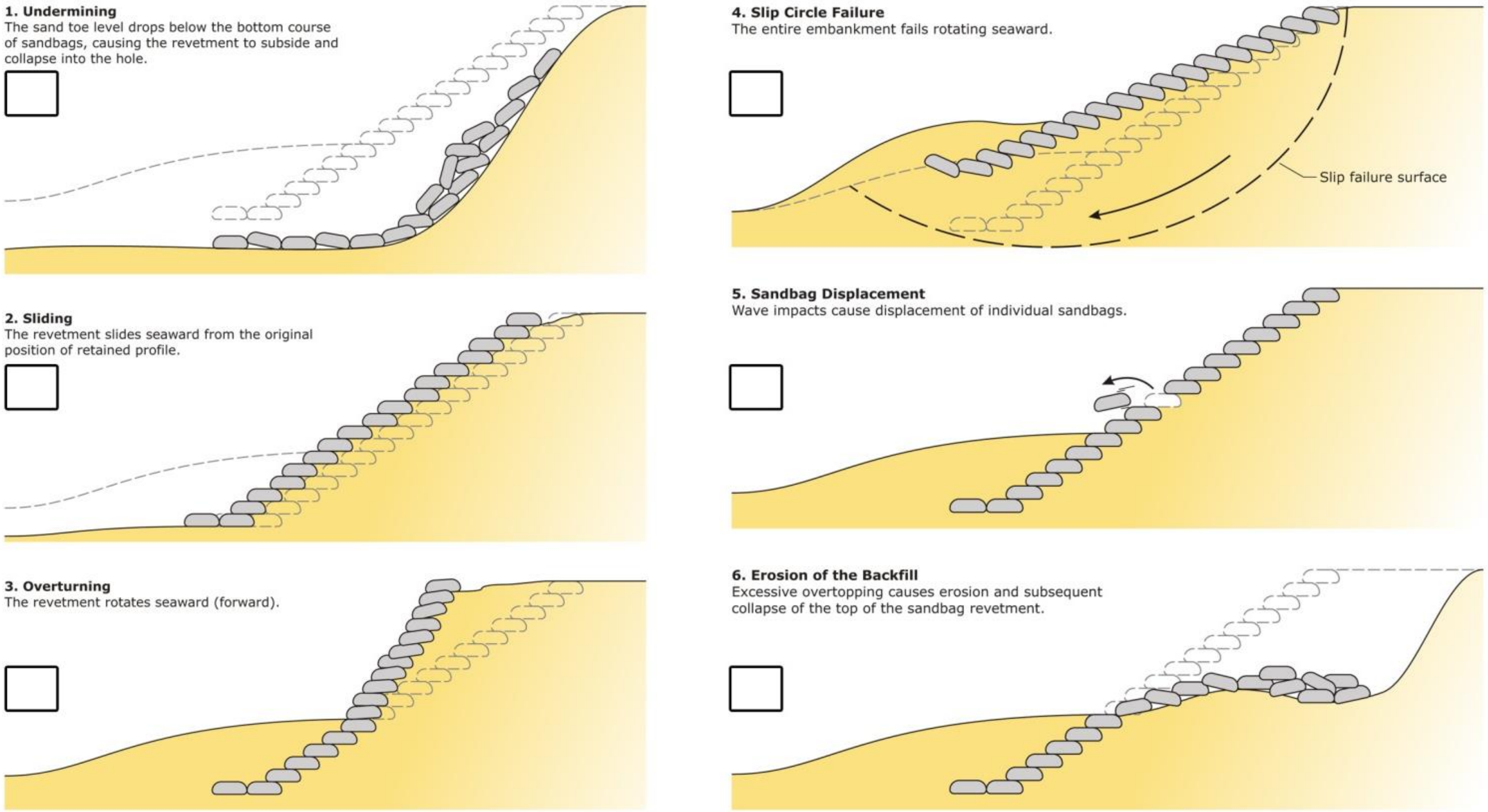

Figure 1: Modes of damage and failure of sandbag seawalls

1. Undermining

5. Sandbag displacement

6. Erosion of crest backfill (from overtopping)

6. Erosion of crest backfill (from overtopping) and 5. Sandbag displacement

Figure 2: Examples of sandbag seawall damage

4.1.2 Three Dimensional Failure/Damage Modes

In addition to the failure/damage modes listed above, seawalls may fail at their ends due to erosion outflanking the seawall – the seawall "end effect" (Figure 3).

In addition to damaging the seawall, the seawall "end effect" may cause erosion or recession on neighbouring properties, or in the case of Ettalong Point, may threaten the road or cycleway beyond the limits of the seawall.

Figure 3: Potential seawall erosion "end effect"

4.1.3 Impact of Potential Failure on Public Safety

Sandbags (as used at Ettalong Point) having a volume of 2.5 m^3 have a mass exceeding 4 to 5 tonnes each. All failure modes have the potential to cause death or injury to people who are positioned above, on or below a sandbag seawall. A similar hazard can exist for steep natural sand dunes, however, beach sand will usually collapse in smaller amounts.

For a person on or below a sandbag seawall, failure due to overturning or sliding presents the greatest hazard, however, injury or death due to other failure modes cannot be excluded.

For a failure due to sandbag displacement (by waves), it could be argued that when the waves are large enough to displace 2.5 m^3 sandbags (mass > 4000 kg), people (typical mass 50 to 100 kg) would be injured by such wave impacts and would/should therefore not be on or below the structure. However, there may be times when dislodged sandbags are shifted to a position or orientation of incipient instability after a storm and may move further during mild conditions.

Failure due to undermining may often be gradual provided no large voids with incipient instability form in the seawall.

Erosion of backfill generally only occurs at times of high waves and water levels. This is generally of lower hazard to the public than the other failure modes, unless people position themselves close to the edge of a steep scarp above or directly under it. The presence of sandbag walls would slightly reduce these risks relative to a natural sand dune, since the lower portion of the slope would be armoured in the sandbag wall case.

5. Inspection Frequency

WRL recommends that inspection of the seawall occurs (Table 3):

- Periodically up until the design life of the structure of 5 years;
- When significant wave heights (Hs) exceed (or are forecast to exceed) a height threshold;
- When tide levels are predicted to exceed a certain threshold;
- Following reports of damage to the structure.

It is also suggested that drone surveys be conducted over the structure periodically for the design life of the structure to provide high resolution data on the performance of the structure and the surrounding sediment movement.

Table 3: Suggested Inspection Frequency

Inspection type	Suggested inspection dates	Who
Periodical	3 month 6 month 12 months 1.5 years 2 years 2.5 years 3 years 4 years 5 years	Drone inspection and GCC staff
High wave threshold (1)	When Hs > 3 m	GCC staff
High water level threshold	When high tides > 2 m LAT 6-8th May 2016 4-6th June 2016 2-5th July 2016 15-17th Nov 2016 14-16 Dec 2016	GCC staff
Special (2)	Following reports of damage to the structure Erosion scarp too close to road/cycleway (3) Suspected GE/CE issue	GE/CE

Notes:

1. As measured on the Sydney wave buoy or as forecast by the Bureau of Meteorology.
2. It is recommended that reporting channels be developed (e.g. with lifeguards or residents).
3. Slope stability to be determined by geotechnical engineer.

6. Inspection Checklist

An inspection checklist has been provided in Appendix A which provides a quantitative ranking of the performance of the geotextile seawall in four categories, namely:

- Structural;
- Serviceability;
- Erosion; and
- Alignment.

A ranking is given to each of these categories out of 100 with a score of 0 equating to completely dysfunctional and 100 being perfect condition. The condition of the wall is ranked for chainages between 0-33 m, 33-66 m and 66-100 m as shown in Figure 4.

The inspection checklist also prompts the inspector to trigger a special inspection by a geotechnical or coastal engineer if specific issues are suspected to be occurring that require specialised advice.

Figure 4: Chainage sections of the structure used in the inspection checklist

7. Summary

A monitoring program has been developed based on best practice coastal engineering for the Ettalong Point geotextile seawall and surrounding environment for GCC to implement. Background information has also been included to support GCC staff during inspections including possible modes of damage and failure as well as including guidance on suitable timeframes for periodic inspections. It is recommended to continue drone inspections of the seawall for the full design life of the structure (5 years) to provide high resolution information of its performance and surrounding sediment movement. A single page inspection checklist has been attached in Appendix A as a guide for GCC staff to follow during inspections.

Thank you for the opportunity to provide this letter report. Please contact James Carley in the first instance should you require further information.

Yours sincerely,

G P Smith
Manager

8. References

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Monitoring Checklist: Ettalong Geotextile Seawall

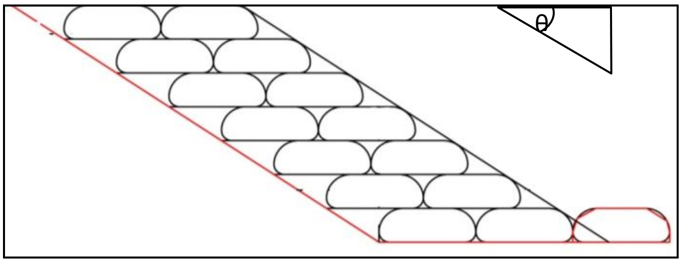
Inspector:			
Inspection Date:		Time:	
Inspection Type:	☐Regular check	☐Post storm check	☐Other_______________________________

Checklist	Chainage		
	0-33 m (west)	33-66 m (centre)	66-100 m (east)
No. courses visible			
Crest level (m AHD)			
Wall slope (θ)			

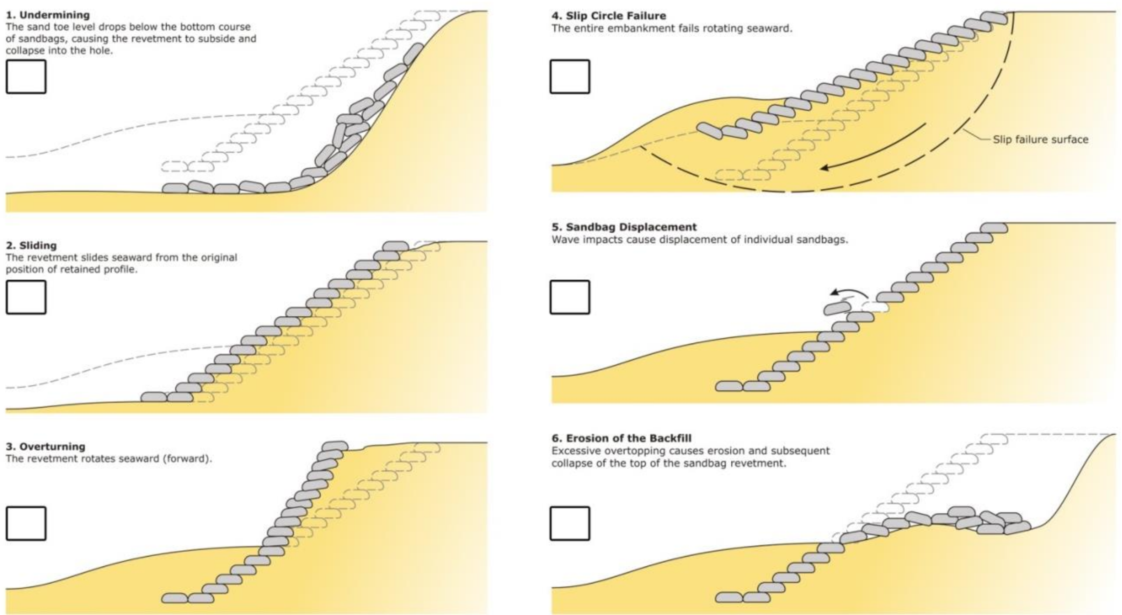
Annotations

☐ Draw debris runup level

☐ Draw sand level



Failure Modes (if identified on site then trigger a geotechnical/coastal engineering special inspection)



Rating Categories	Deficiencies (circle if applicable)	Chainage 0-33 m (west)						33-66 m (centre)						66-100 m (east)					
Structural	No. bags displaced	0	1-5	5-10	10-20	20-30	>30	0	1-5	5-10	10-20	20-30	>30	0	1-5	5-10	10-20	20-30	>30
	Structural rating	100	80	60	40	20	0	100	80	60	40	20	0	100	80	60	40	20	0
Serviceability	No. bags with tearing /vandalism/ deterioration/ deformation/	0	1-5	5-10	10-20	20-30	>30	0	1-5	5-10	10-20	20-30	>30	0	1-5	5-10	10-20	20-30	>30
	Serviceability rating	100	80	60	40	20	0	100	80	60	40	20	0	100	80	60	40	20	0
Erosion	Toe undermining/erosion of crest backfill	Nill		Moderate 1-2 m void		High >2 m void		Nill		Moderate 1-2 m void		High >2 m void		Nill		Moderate 1-2 m void		High >2 m void	
	Erosion rating	100		50		0		100		50		0		100		50		0	
Alignment	End erosion proximity to road	>10m		9-5 m		5-0 m		>10m		9-5 m		5-0 m		>10m		9-5 m		5-0 m	
	Alignment rating	100		50		0		100		50		0		100		50		0	

Comments: