Wamberal Terminal Coastal Protection Assessment

Stage 3 – Seawall Concept Design Options

December 2021

Prepared for:

Central Coast Council

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Foreword

In May 2020 NSW government's professional specialist advisor, Manly Hydraulics Laboratory (MHL) in association with the Water Research Laboratory (WRL) of UNSW Sydney and Balmoral Group Australia (BGA) were commissioned by Central Coast Council to undertake the *Wamberal Terminal Coastal Protection Assessment*. The assessment outcomes are being delivered via a series of reports for the following stages of work:

1. Review of previous studies
2. Coastal protection amenity assessment
3. **Seawall concept design options (this report)**
4. Sand nourishment investigation
5. Provision of coastal monitoring (online webpage)
6. Cost benefit analysis and distributional analysis of options

This report provides the outcomes of Stage 3 of the Wamberal Terminal Coastal Protection Assessment, namely the development of seawall concept design options for Wamberal Beach. The report documents the development of different seawall concept design options for Wamberal Beach including a brief outline of adopted design parameters and engineering standards, concept design cross-sections and descriptions, preliminary seawall alignments and footprints, preliminary cost estimates and an initial comparison of options.

This report is issued as Final and is classified as publicly available.

Executive Summary

Over the past 50 years development along the foredune of Wamberal Beach has had a history of damage and loss due to coastal erosion events. Managing risks to public safety and built assets, pressures on coastal ecosystems and community uses of the coastal zone make up the priority management issues of the certified Gosford Beaches Coastal Zone Management Plan (CZMP, 2017). Undertaking a review of terminal protection design for Wamberal Beach, coupled with the provision of beach nourishment (in accordance with Section 27 of the Coastal Management Act 2016), was a key recommended action of the CZMP (2017).

This report forms part of a broader series of work, the Wamberal Terminal Coastal Protection Assessment, recently undertaken to progress the key recommended management actions for Wamberal Beach from the Gosford Beaches Coastal Zone Management Plan (2017). The Wamberal Terminal Coastal Protection Assessment includes a detailed review of previous studies (Stage 1), amenity assessment of coastal protection options (Stage 2), *development of seawall concept design options (Stage 3 current report),* sand nourishment investigation (Stage 4), implementation of coastal monitoring initiatives (Stage 5) as well as an updated cost-benefit analysis and distributional analysis of management options for Wamberal Beach (Stage 6).

This report provides the outcomes of Stage 3 of the Wamberal Terminal Coastal Protection Assessment, namely the development of seawall concept design options for Wamberal Beach. Design objectives used for the development of seawall concept design options for Wamberal Beach were:

- Develop concept plans for constructing a defensive structure at Wamberal Beach between Wamberal and Terrigal Lagoon (approximately 1360 m in length) that complies with current seawall design practices, engineering standards and scour protection design.
- The concept design options will aim to protect the existing at risk beach front properties and infrastructure from coastal erosion with initial damage (non-rigid structures) at 100 year ARI wave and water level conditions and design failure at a minimum of 500 year ARI conditions (to be confirmed during detailed design that is expected to follow the selection of a preferred option by Council and community). Concept options also account for high end (RCP 8.5) projected sea level rise of up to 0.45 m from 2020 to 2070 with adaptation opportunities to future sea level rise beyond the 50-year initial design period.
- The concept design options will be sensitive to the maintenance and enhancement of amenity at Wamberal Beach foreshore relative to the present state of the beach, seek to minimise impact on coastal processes as well as provide a whole of embayment solution to the protection for public lands and built assets.
- The concept design options and alignment will be sensitive to the broader Wamberal-Terrigal embayment coastal environmental values, including the adjoining Terrigal and Wamberal Lagoons, between which the seawall extends.
- The preliminary alignment for the concept design should be located as far landward as practical to minimise encroachment into the active beach profile and impacts on public beach amenity, while maintaining uniformity of alignment within the constraints of adjacent properties and setback requirements.
- The concept design options include both traditional and promenade-style (composite) approaches.

A review of relevant engineering standards and design parameter selection for the development of concept designs has been provided. A total of five alternative seawall concept designs have been developed with cross-section drawings and footprint mapping for Wamberal Beach comprising:

Option 1: Basalt Rock Revetment
Option 2: Sandstone Rock Revetment
Option 3: Vertical Seawall
Option 4: Vertical Seawall with Rock Toe
Option 5: Tiered Vertical Seawall with Promenade

Preliminary alignment of the concept designs has been proposed following a re-evaluation of the former crest (rear of structure) alignment proposed by Couriel et al. (1998) with regard to:

- present day setback to existing buildings/structures for access
- alignment relative to the foredune erosion scarp present in the beach profile
- alignment relative to the characteristic (natural) shoreline curvature of Wamberal Beach based on measurements of beach width from 1987 to present.

The former alignment was assessed to be suitable for adoption in the present study with only minor changes at regions of stepped crest level changes in the former design.

The adopted crest (rear of structure) alignment has sought to keep the seawall located as far landward as practicable to minimise encroachment into the active beach profile and impacts on amenity, while maintaining uniformity of the previous design within the constraints of adjacent properties and setback requirements. With the adopted alignment (with all options having a common crest alignment and varying seaward toe locations), vertical wall options are considered to have the least impact on beach amenity and coastal processes considering their relatively small footprints compared with the sloped rock revetment designs which encroach further into the active beach profile, impeding more frequently on beach access, public amenity and coastal processes. A more detailed assessment of the impacts of each concept design on amenity and available beach width is provided in the Stage 2 report findings.

Preliminary cost estimates for total capital works and maintenance for each of the five concept design options are listed below:

Option 1: Basalt Rock Revetment $26.5M (equivalent to $19,500 per linear m).
Estimated annual maintenance cost of approximately $265,400 per year. [a]

Option 2: Sandstone Rock Revetment $25.0M (equivalent to $18,400 per linear m). [b]
Estimated annual maintenance cost of approximately $249,900 per year. [a]

Option 3: Vertical Seawall $34.0M (equivalent to $25,000 per linear m).
Estimated annual maintenance cost of approximately $34,000 per year. [a]

Option 4: Vertical Seawall with Rock Toe: $34.7M (equivalent to $25,500 per linear m).
Estimated annual maintenance cost of approximately $34,700 per year. [a]

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[a] Maintenance costs give preliminary estimates of potential repairs to damaged rock armour (non-rigid structures), post-storm condition inspections, and promenade maintenance for option 5 (removal of wind-blown sand and public safety control measures during storms).
[b] Capital and maintenance costs for the sandstone rock revetment may vary depending on the source and durability of rock armour selected during detailed design.

Option 5: Tiered Vertical Seawall with Promenade: $40.1M (equivalent to $29,500 per linear m). Estimated annual maintenance cost of approximately $60,100 per year. [a]

Preliminary costings for capital works include allowances for structural components supply & install, wave return wall construction, removal of existing ad-hoc rock protection on beach (reused where suitable), earthworks, site establishment and preparation, supervision/surveying, further geotechnical investigations as well as contingency covering environmental approvals and removal of potential hazardous materials in existing beach fill. Seawall construction is also to be coupled with the provision of sand nourishment for all options as recommended in the Gosford Beaches CZMP (2017). Costs for sand nourishment to maintain beach width amenity fronting the seawall are addressed separately in *Stage 4: Sand Nourishment Investigation (MHL2795, 2021)*.

A summary of seawall concept options including an overview of design specifications, advantages, disadvantages and cost estimates is provided in Table E.1. Preliminary cost estimates for total capital works range from approximately $25.0M for the more conventional sandstone rock revetment (option 2) to $40.1M for the tiered vertical seawall with foreshore promenade (option 5). Although the most capital cost intensive structure, the tiered vertical seawall with promenade (option 5) is considered to provide the greatest value to the broader community via enhanced access and foreshore amenity.

A preferred option will be determined following consultation with community, stakeholder groups and Council and also consider outcomes of the beach amenity impact assessment (Stage 2), sand nourishment investigation (Stage 4), cost-benefit analysis and distributional analysis (Stage 6) currently being undertaken in parallel with the present study. It is noted that a preferred option may comprise different concept design options along different sections of the beach as may be determined from consultation and subsequent detailed design stages. As an outcome of consultation with community and stakeholder groups, Council may wish to consider the development for a Master Plan for the Wamberal Beach foreshore that details the alongshore values and uses of the foreshore to inform the preferred option design.

Wherever practicable and suitable, existing ad-hoc and emergency rock protection present on the beach will be utilised in the proposed new seawall, with details depending on the final design of the preferred option. For example, existing emergency rock bag protection may form suitable fill material landward of the promenade under option 5. Where not suitable, allowance has been made for any existing ad-hoc material and emergency rock toe protection works seaward of the proposed new seawall to be removed to enhance beach amenity.

Any existing rock protection removed with construction should be replaced by sand nourishment wherever possible to extend the level of the natural beach berm or foredune seaward of the seawall. All sand excavated during the construction of the proposed seawall should be screened (to remove any oversized materials) and placed seaward of the works with any necessary fill landward of the seawall comprised of the separated materials (if suitable) and/or suitable clean fill that would be imported to the site. This will maximise the amount of sand added to the beach area as a result of the works.

As part of the concept design development, a number of issues have been identified that will require further consideration as part of detailed design of a proposed option. These include (but are not limited to):

- Estimates of bearing capacity of any underlying material remaining in the dune substrate.
- Review of the adequacy of available geotechnical knowledge of the study area for detailed design which is dependent on the preferred seawall concept progressed to detailed design.
- Refined design crest (and promenade if adopted) levels with alongshore variability in wave exposure, overtopping and characteristic foredune/berm elevations along the structure. Three-dimensional physical modelling should be considered to refine assumptions and reduce costs as part of detailed design including refining structural dimension, wave overtopping estimates and crest levels along the structure considering safety aspects for people and property. This is particularly important for the promenade design where the promenade level and structural details can be significantly refined to accommodate high wave over topping rates during design storm conditions without damage.
- Land tenure matters and detailed design refinements related to specific concept design options (described in this report) including refinement of adopted thickness and steel reinforcement of concrete panels, length of supporting piles, alongshore details of the seawall toe design, drainage design and promenade elevations.
- Detailed property by property assessment of alignment including buildings, decks, patios and other structures, as well as consultation with beachfront homeowners.
- Associated landscaping and design of beach access points within the structure.
- Environmental approvals including environmental impact assessment, crown licences and further geotechnical assessment.
- Detailed design of termination points at Wamberal and Terrigal Lagoon entrances.

Table E.1: Executive summary – seawall concept design options summary table

Option 1: Basalt Rock Revetment	Option 2: Sandstone Rock Revetment	Option 3: Vertical Seawall	Option 4: Vertical Seawall with Rock Toe	Option 5: Tiered Vertical Seawall with Promenade
Overview				
• Sloping rock revetment with basalt rock armour • Protection infrastructure • E.g., Lennox Head, Stockton, Belongil, Port Kembla	• Sloping rock revetment with sandstone armour • Protection infrastructure • E.g., Collaroy	• Vertical seawall (concrete panel with supporting H columns) with sheet pile toe protection and piled foundations • Protection infrastructure • E.g. vertical seawalls: Flynns Beach, Bondi, Manly, South Cronulla	• Vertical seawall (concrete panel with supporting H columns) with non-rigid rock toe protection and piled foundations. • Protection infrastructure • E.g. vertical seawalls: Flynns Beach, Bondi, Manly, South Cronulla	• Tiered vertical seawall with promenade • Vertical lower and upper walls, 3m wide walkway, sloping backfill. • Protection infrastructure & community asset. • E.g. Newcastle (City), Wollongong (Blue Mile)
Concept Design Specifications				
• +8 m AHD crest[a], berm scour apron, -1 m AHD toe • 2 layers of basalt primary rock armour: M_{50} =6t, D_{50} =1.3 m • Wave return wall at crest • ~23 m wide footprint (slope 1V:1.5H)	• +8 m AHD crest[a], berm scour apron, -1 m AHD toe • 2 layers of sandstone primary rock armour: M_{50} =11t, D_{50} =1.7 m • Wave return wall at crest • ~23 m wide footprint (slope 1V:1.5H)	• +8 m AHD crest[a], -3 m AHD toe, concrete piled • Wave return wall at crest • ~1 m wide footprint (slight sub-vertical incline)	• +8 m AHD crest[a], -1 m AHD rock toe, concrete piled • Wave return wall at crest • ~5 m wide footprint (slight sub-vertical incline)	• +8 m AHD crest[a], +4 m AHD promenade[a], -3 m AHD toe, concrete piled • Wave return wall at lower/upper wall • ~7 m wide footprint
Advantages [b]				
• Lower cost • Adaptable to sea level rise • Conventional design & non-rigid structure	• Lower cost • Adaptable to sea level rise • Conventional design & non-rigid structure • Sandstone aesthetic appeal	• Smallest footprint • Low environmental & social impacts (low beach encroachment) • Low construction impacts • Adaptable to sea level rise	• Small footprint • Low environmental & social impacts (low beach encroachment) • Adaptable to sea level rise	• Enhanced access & amenity (coastal walk) • Broad community and economic benefits • Adaptable to sea level rise • Relatively low environmental & social impacts (relatively low beach encroachment) • Existing rock reuse and enhanced maintenance corridor opportunities
Disadvantages [b]				
• Relatively low aesthetics • Wide footprint • High environmental & social impacts: high beach encroachment, more frequent interaction with coastal processes, reduced available beach width, access constraints, post-storm safety risks • Higher maintenance requirements & increased risk of more frequent periodic sand nourishment	• Relatively low aesthetics • Wide footprint • High environmental & social impacts: high beach encroachment, more frequent interaction with coastal processes, reduced available beach width, access constraints, post-storm safety risks • Rock armour durability • Higher maintenance requirements & increased risk of more frequent periodic sand nourishment	• Moderate to high cost • Vertical drop post-storm (safety concern) • Higher reflected wave energy during storms though with limited temporal and spatial extent [c]	• Moderate to high cost • Vertical drop post-storm (safety concern) • Higher reflected wave energy during storms though with limited temporal and spatial extent [c]	• Highest cost • More complex detailed design • Privacy considerations • Public access management during storms • Relatively higher reflected wave energy during storms though with limited temporal and spatial extent [c]
Cost Estimates				
Capital: $26.5M **Maintenance:** $265K per year **Nourishment:** Refer to Stage 4 Report	**Capital:** $25.0M **Maintenance:** $250K per year **Nourishment:** Refer to Stage 4 Report	**Capital:** $34.0M **Maintenance:** $34K per year **Nourishment:** Refer to Stage 4 Report	**Capital:** $34.7M **Maintenance:** $35K per year **Nourishment:** Refer to Stage 4 Report	**Capital:** $40.1M **Maintenance:** $60K per year **Nourishment:** Refer to Stage 4 Report

[a] Refined design crest (and promenade if adopted) levels considering alongshore variability in wave exposure, wave overtopping, privacy of beachfront residences and characteristic foredune/berm elevations along the structure are to be determined during detailed design.
[b] A preferred option is to be determined in consultation with community, stakeholder groups and Council and also consider outcomes from the beach amenity impact assessment (Stage 2), sand nourishment investigation (Stage 4) and cost-benefit analysis (Stage 6) currently being undertaken in parallel with the present study.
[c] Limited to occurrences when the beach is eroded by major storm waves and sections of the seawall are exposed. This effect is mitigated by a more landward cross-shore position of a vertical seawall within the active beach profile and presence of an accreted beach fronting the seawall for most of the time.

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1.1 Background

Wamberal Beach is within the traditional boundaries of Darkinjung (Darkinyung) land, which extends from the Hawkesbury River in the south, Lake Macquarie in the north, the McDonald River and Wollombi up to Mt Yengo in the west and the Pacific Ocean in the east.

Wamberal Beach is a sandy ocean coast shoreline, situated within the Wamberal-Terrigal embayment on the NSW Central Coast as shown in Figure 1.1. A more detailed description of the study site including regional wave climate is provided in the *Stage 1 Report (MHL2778, 2021)*. Over the past 50 years development along the foredune of Wamberal Beach has had a history of damage and loss due to coastal erosion events. Managing risks to public safety and built assets, pressures on coastal ecosystems and community uses of the coastal zone make up the priority management issues of the certified Gosford Beaches Coastal Zone Management Plan (CZMP, 2017) with the primary objective:

> *"to protect and preserve the beach environments, beach amenity, public access and social fabric of the Open Coast and Broken Bay beaches while managing coastal hazard risks to people and the environment".*

Major actions recommended for Wamberal Beach from the CZMP (2017) were the following:

- *"TW11 Terminal protection - Council to action review, design and funding of terminal protection structure for Wamberal."*
- *"TW14 Investigate sources of sand and feasibility of beach nourishment for Wamberal Beach."*
- *"TW15 Beach nourishment coupled with a terminal revetment to increase buffer against storm erosion."*

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1. Review of previous studies
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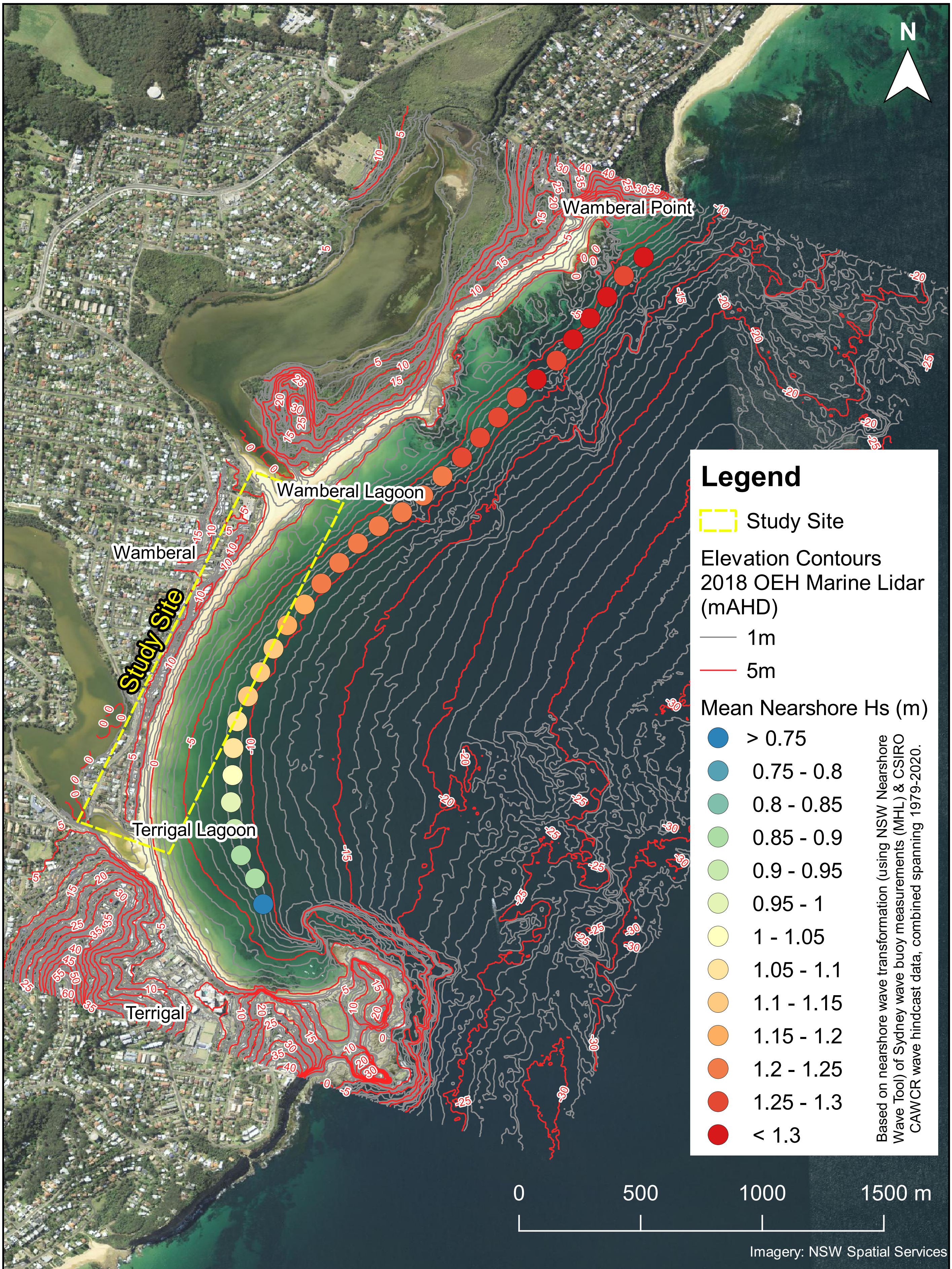
This report provides the outcomes of Stage 3 of the Wamberal Terminal Coastal Protection Assessment, namely the development of seawall concept design options for Wamberal Beach.



NSW GOVERNMENT

LOCATION OF STUDY SITE SHOWING ELEVATION CONTOURS (2018 OEH MARINE LIDAR) AND AVERAGE NEARSHORE WAVE CLIMATE (-10M CONTOUR)

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Report MHL2780

Figure

1.1

Figure1.1.pdf

1.2 Stage 3 objectives

The primary objective of Stage 3 of the *Wamberal Coastal Protection Assessment* is to develop different seawall concept design options for Wamberal Beach. The Stage 3 study seeks to provide a brief outline of adopted design parameters and engineering standards, concept design cross-sections and descriptions, preliminary seawall alignments and footprints, preliminary cost estimates and comparison of the advantages/disadvantages of each option.

1.3 Stage 3 overview

The Stage 3 report includes the following:

- Description of preliminary design parameters used for seawall concept design development for Wamberal Beach (Section 2)
- Review of engineering standards relevant to concept seawall designs for Wamberal Beach (Appendix A)
- Review of available geotechnical data (Appendix B)
- Description and cross-sections of different seawall concept designs for Wamberal Beach (Section 3)
- Preliminary seawall alignment and concept design footprints for Wamberal Beach (Section 4)
- Preliminary cost estimates and comparison of the advantages/disadvantages for each seawall concept design option. (Section 5)

2 Preliminary design parameters

2.1 Introduction

This section provides a description of the design parameters used for development of seawall concept design options for Wamberal Beach. These include:

- Geotechnical conditions - used to assess the adequacy of existing foundation conditions for a particular concept design.
- Scour levels at the toe of the structure – used to determine the toe depth required to prevent undermining and evaluate the depth limited breaking wave height at the structure.
- Wave and water level conditions including sea level rise implications – used to evaluate the hydraulic performance (wave runup and overtopping) and stability of the structure.

As part of the selection of design parameters, a detailed review and summary of engineering design standards for Wamberal terminal coastal protection design was undertaken by Carley (2020) and is provided in Appendix A. The review recommended the following values for seawall designs at Wamberal Beach:

Initial design life:	50 years
Initial damage for rubble structure:	100 to 200 year ARI
Failure for rubble structure or rigid structure:	500 to 2000 year ARI

Where ARI = Annual Recurrence Interval

It is noted that establishing the acceptable risk of failure (encounter probability) is not exclusively an engineering decision and should involve numerous stakeholders. Numerous standards also exist for the detailed design and specification of specific materials (if they are selected) e.g. standards for strength and durability testing of rock, standards for concrete. These would be further considered in the detailed design and contract documentation phase of the project.

In accordance with Appendix A, seawall concept design options for Wamberal Beach have been developed for:

- Initial design life of 50 years
- Initial damage for rubble structure of 100 year ARI

As recommended in Appendix A, rock revetment concept options were designed with initial damage (few units displaced) for the adopted 100 year ARI wave and water level conditions, with design failure (underlayer visible) estimated at 1000-2000 year ARI conditions based on extrapolated design curve estimates. Vertical seawall concept options (rigid structures) have given preliminary consideration to footing type, scour protection and basic wall configuration, with design failure of materials and structural components to be determined in accordance with standards recommended in Appendix A during detailed design stages.

More detailed structural components of the seawalls are to be confirmed during detailed design in accordance with initial damage and failure standards recommended in Appendix A. Design conditions for the final structure would also consider stakeholder feedback to help determine an accepted level of risk.

Selection of subsequent design parameters for the purpose of concept design development are described in the sections below.

2.2 Geotechnical conditions

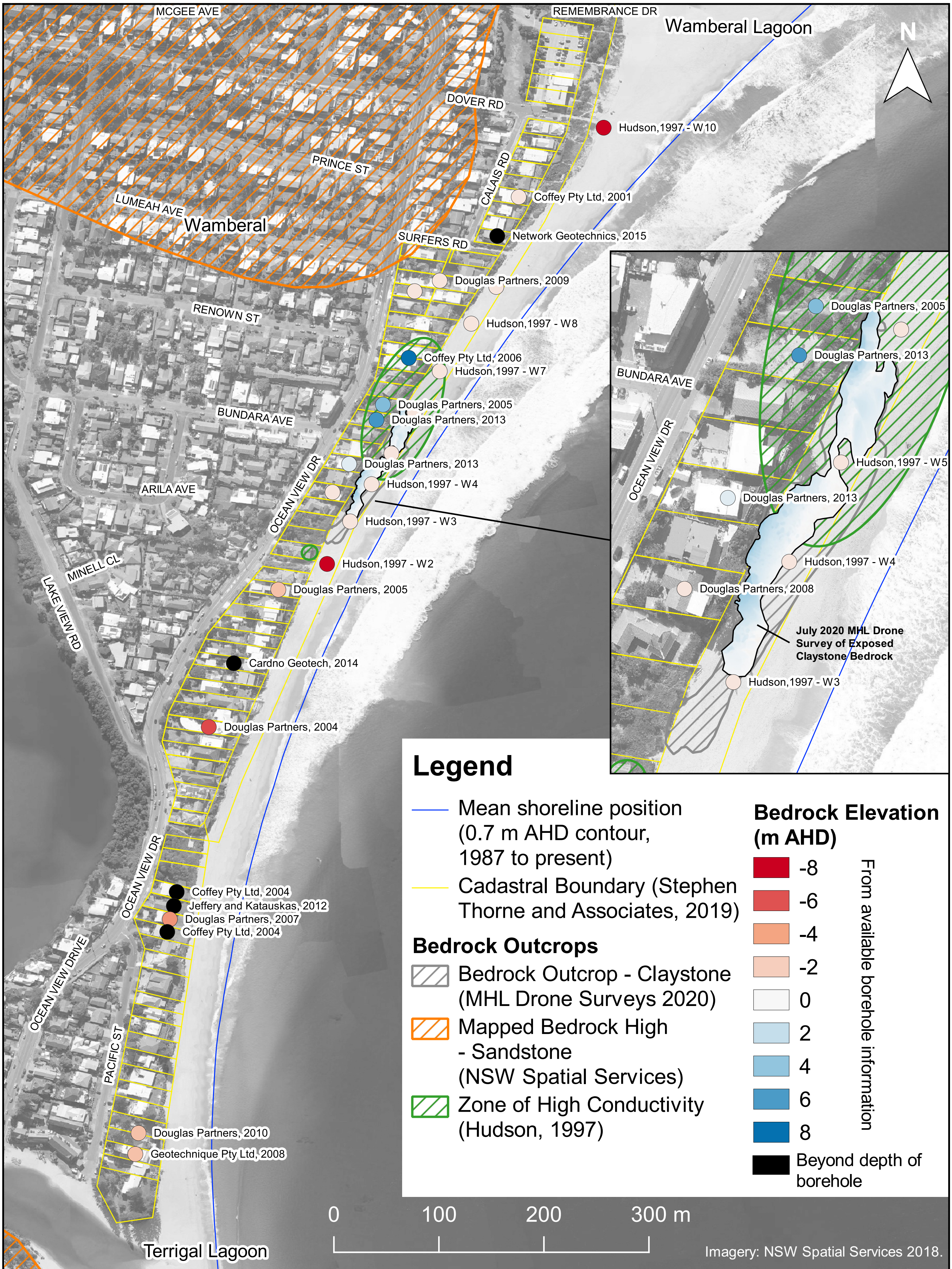
Geotechnical information is used to determine the appropriate type of structure and foundations for design. A review of available geotechnical data for Wamberal Beach is provided in Appendix B including information from a geotechnical investigation by Hudson (1997), more recent studies undertaken as part of property development applications and information of existing ad-hoc material/protection works present in the foredune substrate. A summary of available geotechnical information is provided in Figure 2.1, showing depth to bedrock from borehole drilling and regions of elevated bedrock. More detail of available geotechnical information is provided in Appendix B.

The studies report a typical stratigraphy characterised by fine to medium grained marine sands with some gravelly deposits at depths, overlying weathered (very stiff to hard) siltstone/claystone and weathered sandstone.

Typical depths to the siltstone/claystone unit have been found to vary along the study site as shown in Figure 2.1. In the southern and mid sections of the study site (south of 73 Ocean View Dr), this is situated between -2 to below -10 m AHD. In the mid-north of the site, a 400 m section of elevated siltstone/claystone is situated north of 73 Ocean View Dr with shallower depths of -2 to +1 m AHD and is temporarily exposed during erosion events. The claystone bedrock returns to lower depths in the north of the study site. Other than existing ad-hoc and emergency protection works, the foredune is predominantly unconsolidated quartz sand from the surface to below 0 m AHD other than a small region between Bundara Ave and Renown St where elevated siltstone/claystone of up to +8 m AHD has been identified.

The review also noted various ad-hoc materials and protection works present in the foredune substrate shown in Figure 2.2. It is difficult to assess the stability of ad-hoc materials present in the beach without further geotechnical investigation, however the inconsistent nature of this fill is likely to add complexities during construction and is recommended to be removed or where suitable re-used. Indicative allowances for handling this material has been included as part of preliminary costings in Section 5.2. Estimates of bearing capacity of any underlying material remaining in the dune substrate would be required in detailed design to evaluate settlement of non-rigid structures if proposed. Clean sand won (liberated) during construction of a seawall through excavation of the existing landform should be used to nourish the beach seaward of the structure.

The available geotechnical information for Wamberal Beach is considered sufficient to undertake the development of alternative seawall concept design options for the study site. The adequacy of this information, including any geotechnical knowledge gaps should be reviewed as part of detailed design for the preferred design option. A preliminary allowance for further geotechnical investigations as part of detailed design has been considered in preliminary cost estimates in Section 5.2.



NSW GOVERNMENT

SUMMARY OF AVAILABLE GEOTECHNICAL INFORMATION
WAMBERAL BEACH.

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Figure
2.1

Figure 2.1.pdf

Legend

Cadastral Boundary (Stephen Thorne and Associates, 2019)

Materials present:

- Emergency Concrete Block Armour
- Emergency Kyowa Rock Bags Stage 1A
- Emergency Rock Armour Protection Stage 1A
- Emergency Rock Armour Protection Stage 1B
- Emergency Rock Bags Stage 1B
- Existing Ad-hoc Rock Protection
- Existing Concrete Capping
- Existing Concrete Seawall
- Existing Concrete Tank Protection
- Existing Gabion Rock Mattress
- Existing Terracotta Seabee Protection
- Other materials

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EXISTING MATERIALS PRESENT ON BEACH POST JULY 2020 STORM EROSION

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Figure

2.2

Figure 2.2.pdf

2.3 Climate change (sea level rise) and shoreline recession

In 2013, the Intergovernmental Panel on Climate Change (IPCC) released its Fifth Assessment Report of the state of knowledge of climate change and its environmental implications. As part of the report the IPCC developed a range of future sea level rise projections (relative to 1986-2005) associated with different greenhouse gas emission scenarios, termed representative concentration pathways (RCPs) (Church et al., 2013).

Sea level rise projections for the NSW coast for each of these RCP emissions scenarios are provided in Glamore et al. (2015). Considering initial design planning period from 2020 to 2070, 2070 sea level rise projections (averaged along the NSW coast) for the lowest emissions scenario with strong mitigation (RCP 2.6) range between 0.19-0.42 m (relative to the 1986-2005 mean with range equivalent to 66% confidence limits in projections). For the highest (unmitigated) emissions scenario (RCP 8.5), 2070 projections range between 0.31-0.59 m (relative to the 1986-2005 mean with range equivalent to 66% confidence limits in projections). Sea level rise of 0.45 m was adopted for the present design, equivalent to the 2070 projection to a central value for a high-end (unmitigated) emissions scenario of RCP8.5. More detailed consideration of sea level rise is to be undertaken in detailed design.

Underlying shoreline recession and predicted recession due to sea level rise for Wamberal Beach was analysed as part of the Coastal Processes and Coastal Hazard Definition Study by WorleyParsons (2014) as reviewed in the *Stage 1 works (MHL2778, 2021)*. The study found a long-term underlying recession of 0.2 m/year and a Bruun Factor of 43 (that is that recession due to sea level rise will be 43 times the magnitude of sea level rise).

Based on these findings, adopted shoreline recession values for concept design development in the present study to inform scour level protection include:

- Underlying recession of 10 m by 2070.
- Recession associated with adopted 0.45 m sea level rise of 19.4 m by 2070 estimated using Bruun Rule approximations. An increase in profile elevation of 0.45 m is also estimated using these techniques.

Any sand nourishment undertaken as part of the project should be incorporated to offset sea level rise and beach recession effects. Preliminary nourishment requirements to offset sea level rise and underlying recession impacts over a 50-year design period are provided in the accompanying *Stage 4: Sand Nourishment Investigation (MHL2795, 2021)* report.

2.4 Design scour levels

A review of available methods to estimate scour at seawalls is provided in Carley et al. (2015). For rigid structures located at the back of the active beach profile, a structure toe depth of approximately -1 to -2 m AHD is commonly used for coastal engineering design in NSW, based on observed and stratigraphic evidence of historical scour levels fronting permeable and non-permeable structures during major storms that impacted the coastline in the mid-1970s (Foster et al., 1975; Nielsen et al., 1992). More recently during storms in June 2016 a scour of -0.5 m AHD was observed fronting a rock rubble seawall at Collaroy Beach via a continuously scanning Lidar (Couriel et al., 2020).

Differences in scour fronting vertical and sloping seawall designs are noted in the literature. In the Shore Protection Manual (1984) sloping seawalls are said to experience reduced scour due to better energy dissipation, resulting from such structures. In contrast, the Coastal Engineering Manual (2006) states that scour is less dependent on wave reflection and more so on local sediment transport gradients and wave overtopping. Nielsen et al., (1992) recommends that for reflective seawalls (such as the vertical and stepped structures) scour is likely to be greater than on a natural beach, and that a scour level of -2 m AHD be adopted for design.

It should be noted that when located sufficiently landward at the back of the active beach, scour erosion occurs only temporarily during storm events when the seawall is exposed to high wave energy, and this erosion typically infills in the weeks following as the beach recovers with mild wave conditions (Couriel et al., 2020). Long-term field measurement studies have struggled to identify differences in long-term beach profile changes between vertical and sloped seawall locations (Griggs et al., 1991, 1997; Griggs, 1994).

As a minimum the structure is expected to be stable for a scour depth of -1 m AHD. Given an adopted total design recession of 29.4 m and sea level rise of 0.45 m (Section 2.3), it is considered appropriate that the design be able to accommodate an erosion scour depth of -2 m AHD. For the purpose of structural design, the present study has adopted a design toe level of -1 m AHD for rock revetment structures with a berm toe apron designed to accommodate more severe scour down to -2 m AHD (SPM, 1984). Protection at the toe of vertical structures has likewise been designed to accommodate scour to -2 m AHD.

2.5 Design water level conditions

Understanding the magnitude and recurrence of the tidal climate, tidal anomalies and extreme sea level events is required for effective coastal management, including the assessment of coastal erosion, inundation and structural design. Ocean water level data is collected by Manly Hydraulics Laboratory for the Climate Change and Sustainability Division of Department of Planning, Industry and Environment at 26 stations along the NSW coastline from Tweed Heads in the north to Eden in the south, comprising more than 30 years of continuous records, including five offshore open ocean stations, 11 onshore open ocean or open bay stations and 10 onshore river entrance stations.

More than 100 years of continuous water level records at the Fort Denison (Station number 60370) ocean water level gauge in Sydney Harbour, located approximately 50 km south of Wamberal Beach, were used to determine a 100-yr ARI extreme water level of +1.42 m AHD used for design of initial damage (few units displaced) of non-rigid concept options (in accordance Appendix A). This was compared with approximately 30 years of continuous timeseries data collected at the Patonga ocean water level station, located approximately 17 km south of Wamberal Beach. The Patonga dataset has undergone recent analysis by MHL (MHL2236, 2018) to extrapolate a 100-yr ARI extreme water level using a generalised pareto (GP) model fit based on the nearby Fort Denison record. Comparison between the Patonga and Fort Denison 100-yr ARI extreme water level are provided in Table 2.1 and show close agreement.

Table 2.1: 100-year ARI extreme water levels Patonga and Fort Denison (MHL2236, 2018)

Station	100-year ARI water level using GP (m AHD)		
	Model	Lower limit	Upper limit
Fort Denison	1.42	1.38	1.53
Patonga	1.43	1.39	1.59

An estimated 500-yr ARI extreme water level of +1.49 m AHD was extrapolated from the Fort Denison water level exceedance as a minimum design water level for failure of rigid and non-rigid structures (in accordance with Appendix A). Design failure (under layer visible) of rock revetment concept options was estimated at 1000-2000 year ARI conditions based on extrapolated Fort Denison water level estimates.

Vertical seawall concept options (rigid structures) have given preliminary consideration to footing type, scour protection and basic wall configuration, with design failure of materials and structural components to be determined in accordance with standards recommended in Appendix A during detailed design stages.

2.5.1 Wave setup

Available field measurement results of wave setup exhibit a wide range of wave setup to wave height ratios. Some of the variability between results at different locations is likely due to the effect of profile slope which is not accounted for explicitly in all of the analyses as well as the effects of wave breaking in depths greater than the shallow water limit (Dean and Walton Jr. 2008). It is generally accepted that wave setup at the shoreline on the NSW open coast to be around 15% of the offshore significant wave height (Hs) measured by wave buoys (Mummery, 2016). This is consistent with adopted shoreline wave setup in the Coastal Hazard Definition for Wamberal Beach (WorleyParsons, 2014).

However, with a seawall the surf zone is truncated in the nearshore compared to a natural beach. In the present study design wave conditions are calculated at the wave plunge point at -1 m AHD seaward of the structure toe, where wave setup is expected to be slightly lower compared to that at the shoreline of a natural beach. A surfzone wave breaker decay model by Dally et al. (1985) was run for a representative profile at Wamberal Beach to determine a design wave setup at the wave plunge point, found to be approximately 10% of the offshore Hs.

2.6 Design wave conditions

Although the design wave height for the Wamberal Beach seawall is depth-limited, the offshore wave climate is important to determine wave setup, which affects the design water depth. Further, the design breaking wave height at the structure is affected also by the wave period that is characterised by the offshore wave conditions.

Wave data is collected by Manly Hydraulics Laboratory for the Climate Change and Sustainability Division of Department of Planning, Industry and Environment to provide essential input to design, construction and performance monitoring of coastal zone projects undertaken by the NSW Government.

Since 1974 wave data have been collected at over 40 locations along the NSW coast using a variety of wave motion sensors. Wave data collected by the Waverider buoy network and by other project specific stations has been incorporated into an extensive long-term database maintained at MHL. Waverider buoys are typically moored in water depths between 60 and 100 metres between 6 and 12 kilometres from the shoreline. At the buoy location the water depth is sufficiently deep that wave refraction, diffraction, shoaling and friction attenuation effects are minimal.

Offshore wave data statistics from the Sydney Waverider buoy located approximately 35 kilometres south of Wamberal Beach has been used to determine design conditions and wave setup. The buoy was first deployed in 1987, was upgraded to a directional Waverider buoy in 1992, and has been recording almost continuously since its initial deployment, resulting in an approximately 33-year record. Wave parameters recorded include:

- Significant wave height (Hs)
- Maximum wave height (Hmax)
- Significant wave period (Ts)
- Peak spectral wave period (Tp)
- Zero crossing wave period (Tz)

Figure 2.3 shows the offshore significant wave height/duration return period determined from the Sydney Waverider buoy based on the data recorded from August 1987 to December 2019 as well as the distribution of the peak spectral wave period (MHL2538, in draft). An offshore significant wave height (Hs) of 8.7 m, associated with 100 year ARI design wave conditions for a 3-hour storm duration, was adopted for design of initial damage (few units displaced) of non-rigid concept options (in accordance Appendix A). An associated peak wave period (Tp) of 13 s was adopted based on Hs-Tp joint occurrence analysis of Sydney wave buoy data from March 1992 to December 2019. The joint probability of such wave conditions occurring with the adopted 100 year ARI water levels is likely to be more rare than the 100 year ARI conditions (Shand et al., 2012).

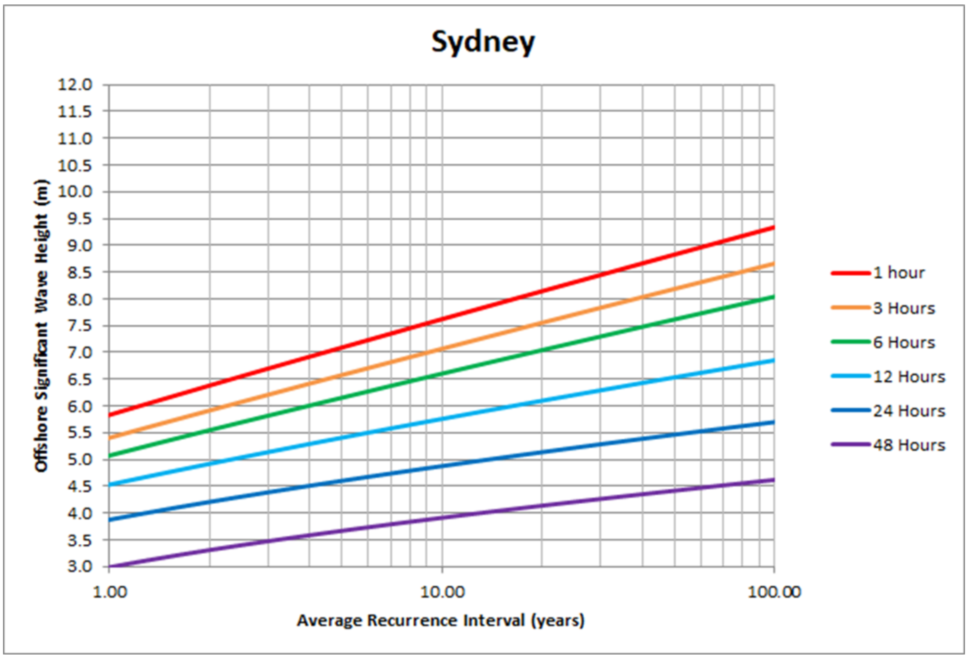


Figure 2.3: Sydney waverider offshore significant wave height extreme value analysis 1987 to 2019. From (MHL2538, in draft).

An estimated 500-yr ARI offshore significant wave height of 9.8 m was extrapolated from the Sydney wave height exceedance curves (3h duration) as a minimum offshore significant wave height for failure of rigid and non-rigid structures (in accordance with Appendix A). Design failure (under layer visible) of rock revetment concept options was estimated at 1000-2000 year ARI conditions based on extrapolated Sydney wave height exceedance curves.

Vertical seawall concept options (rigid structures) have given preliminary consideration to footing type, scour protection and basic wall configuration, with design failure of materials and structural components to be determined in accordance with standards recommended in Appendix A during detailed design stages.

Design wave conditions are primarily concerned with waves that break on, or just seaward, of the structure. Given that the structure is going to be located in very shallow water, the design breaking wave height will be depth limited. Therefore, the design wave height will be a function of the nearshore water level and the bed level at the toe of the structure. This was adopted for the design breaking wave. The procedure to calculate the design breaking wave heights can be outlined as follows:

1. Nearshore wave setup just offshore (plunge distance) of the structure was calculated as 10% of the offshore wave height as described in Section 2.5.1.
2. The still water level (SWL) at the structure was calculated as the sum of the design water level (Section 2.5), calculated wave setup component and sea level rise (Section 2.3).
3. The water depth at the toe of the structure (d_s) was calculated as the sum of the SWL and design scour depth (Section 2.4).
4. Design breaking wave height at the structure was calculated using design curves given in the Figure 7-4 Shore Protection Manual (Coastal Engineering Research Center, 1984) based on the d_s, design peak spectral wave period (T_p) and a nearshore slope of 1:33 (Lawson and Treloar, 1984).

Preliminary design wave breaker wave height (Hb) at the structure was calculated to be 4.0 m with a peak wave period (Tp) of 13.0 s. Sensitivity to design wave conditions is to be undertaken for each part of the structure in detailed design and may benefit from scaled three-dimensional model testing.

2.7 Concept design crest level and wave overtopping

Given the high level of residential encroachment and the design objective to align the seawall as far landward as practical for beach amenity and environmental purposes, minimal wave overtopping is required to protect properties and ensure the stability of the seawall structure or buildings landward. Selected limits for wave overtopping for structural design of seawalls in the presence of people, grass coverings and promenades are shown in Table 2.2 from the 2018 Eurotop Manual (Pullen et al., 2018).

A design mean wave overtopping 4 L/s/m was adopted as part of the former 1998 Seabee seawall design (Couriel et al., 1998) with a minimum 3 m setback to adjacent foreshore development. A nominal design crest level of +8.0 m AHD was adopted in the former design, with final design crest levels refined along the structure based on results of physical modelling as part of detailed design.

Table 2.2: Limits for wave overtopping for structural design of seawalls. From EuroTop (2018).

Hazard type	Mean overtopping q (l/s per m)	Max Volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	 0.3 1 10-20 No limit	 600 600 600 No limit
Building structure elements; H_{m0} = 1-3 m	≤1	<1,000
Damage to equipment set back 5-10m	≤1	<1,000
Grass covered crest and landward slope; maintained and closed grass cover; H_{m0} = 1 – 3 m	5	2,000-3,000
Damage to paved or armoured promenade behind seawall	< 200	Not provided

[a] For promenades pathways not structurally integrated into seawall structure such as a tiered vertical wall.

Preliminary wave overtopping calculations have been undertaken following design and assessment overtopping formulations from the 2018 Eurotop Manual (Pullen et al., 2018). These have been calculated using a design storm duration of three hours, profile slope of 1:33 (Couriel et al., 1998) and scour level of -1.0 m AHD. Sensitivity to the adopted design wave conditions are recommended to be undertaken as part of detailed design and may benefit from scaled three-dimensional model testing.

A mean wave overtopping threshold of ≤1 l/s per m was adopted for concept design development based on potential damage thresholds in Table 2.2 and considering a minimum 3 m setback of residential buildings at Wamberal Beach. A uniform crest height has been used for comparative purposes of the concept design options, with +8.0 m AHD (including wave return) found to best achieve the mean overtopping criteria for majority of options. Further detailed refinement of design crest levels with alongshore variability in wave exposure and overtopping is to be considered as part of detailed design. Three-dimensional physical modelling should be considered as an option to refine overtopping estimates and crest (and promenade) levels along the structure as part of detailed design.

2.8 Summary of design parameters for seawall options

A summary of minimum engineering design parameters used to develop seawall concept design options for Wamberal Beach seawall area shown in Table 2.3. As recommended in Appendix A, rock revetments concept options were designed with initial damage (few units displaced) for the adopted 100 year ARI wave and water level conditions, with design failure (under layer visible) estimated at 1000-2000 year ARI conditions based on extrapolated design curve estimates. Vertical seawall concept options (rigid structures) have given preliminary consideration to footing type, scour protection and basic wall configuration, with design failure of materials and structural components to be determined in accordance with standards recommended in Appendix A during detailed design stages. Previous studies of the area of interest and empirical knowledge from similar design locations have also been used to inform the development of proposed concept designs.

More detailed structural components of the seawalls are to be designed and reviewed during detailed design in accordance with initial damage and failure standards recommended in Appendix A. Design conditions for the final structure would also consider stakeholder feedback to help determine an accepted level of risk.

Table 2.3: Minimum engineering design parameters for Wamberal Beach

Parameter		Parameter value
Initial design life		50 years
Design Sea Level Rise (SLR) (RCP 8.5, 2070)		0.45 m
Design scour level		-2.0 m AHD [a]
Preliminary concept design crest level [b]		+8.0 m AHD
Annual Recurrence Interval (ARI) for initial damage of non-rigid structures and serviceability of rigid structures		**100 year ARI [c]**
Design conditions for initial damage of non-rigid structures	Offshore wave height (Hs)	8.7 m
	Breaker wave height (Hb) at toe of structure	4.0 m [d]
	Peak spectral wave period (Tp)	13 s
	Storm duration	3 h
	Design water level	+1.42 m AHD
	Wave setup	0.87 m
	Design water level + SLR + setup	+2.74 m AHD
Annual Recurrence Interval (ARI) design failure of rigid and non-rigid structures		**500 year ARI [c]**
Design conditions for failure of non-rigid and rigid structures	Offshore wave height for 3h duration (Hs)	9.8 m [e]
	Breaker wave height (Hb) at toe of structure	4.2 m [d]
	Peak spectral wave period (Tp)	13 s
	Design water level	+1.49 m AHD [e]
	Wave setup	0.98 m
	Design water level + SLR + setup	+2.92 m AHD

[a] The present study has adopted a design toe level of -1 m AHD for rock revetment structures with a berm toe apron designed to accommodate more severe scour down to -2 m AHD (SPM, 1984). Protection at the toe of vertical structures has likewise been designed to accommodate scour at -2 m AHD.

[b] Further detailed refinement of design crest levels with alongshore variability in wave exposure and overtopping is to be considered as part of detailed design.

[c] Joint probability of adopted wave and water level conditions is likely to be rarer than 100 or 500 year ARI design conditions.

[d] Design breaker wave height calculated for depth limited conditions as a function of the nearshore water level and the bed level at the toe of the structure.

[e] Extrapolated from Fort Denison water level and Sydney wave height exceedance curves.

3 Seawall concept design options

3.1 Concept design objectives

Design objectives for the development of seawall concept options for Wamberal Beach are listed below:

- Develop concept plans for constructing a defensive structure at Wamberal Beach between Wamberal and Terrigal Lagoon (approximately 1360 m in length) that complies with current seawall design practices, engineering standards and scour protection design.
- The concept design options will aim to protect the existing at risk beach front properties and infrastructure from coastal erosion with initial damage (non-rigid structures) at 100 year ARI wave and water level conditions and design failure at a minimum of 500 year ARI conditions (to be confirmed during detailed design that is expected to follow the selection of a preferred option by Council and community). Concept options also account for high end (RCP 8.5) projected sea level rise of up to 0.45 m from 2020 to 2070 with adaptation opportunities to future sea level rise beyond the 50-year initial design period.
- The concept design options will be sensitive to the maintenance and enhancement of amenity at Wamberal Beach foreshore relative to the present state of the beach, seek to minimise impact on coastal processes as well as provide a whole of embayment solution to the protection for public lands and built assets.
- The concept design options and alignment will be sensitive to the broader Wamberal-Terrigal embayment coastal environmental values, including the adjoining Terrigal and Wamberal Lagoons, between which the seawall extends.
- The preliminary alignment for the concept design should be located as far landward as practical to minimise encroachment into the active beach profile and impacts on public beach amenity, while maintaining uniformity of alignment within the constraints of adjacent properties and setback requirements.
- The concept design options include both traditional and promenade-style (composite) approaches.

As part of the concept design development, a detailed review and summary of engineering design standards for Wamberal terminal coastal protection design was undertaken by Carley (2020) and is provided in Appendix A, and will help inform requirements for detailed design in future stages.

3.2 Concept design options for seawall

A total of five seawall concept design options have been developed for Wamberal Beach, including two rock revetment designs, two vertical seawall designs and a tiered vertical seawall design with promenade. A brief outline of each concept design is provided in Table 3.1.

It should be noted that each seawall option is to be coupled with sand nourishment to maintain beach amenity. Sand nourishment requirements, sources and preliminary costings are provided in the *Stage 4: Sand Nourishment Investigation (MHL2795, 2021)* report.

The following sections provide a more detailed description of each concept design option including key design features, preliminary overtopping estimates, typical design cross-sections and photographs of comparable structures. Discussion of the advantages and disadvantages of each concept design option as well as the former Seabee seawall design for Wamberal Beach (Couriel et al., 1998) is provided in Section 5.3.

Table 3.1: Seawall concept design options for Wamberal Beach

Option	Brief Description
1	**Basalt rock revetment** - conventional rubble mound rock seawall with basalt rock armour, buried by sand.
2	**Sandstone rock revetment** - conventional rubble mound rock seawall with sandstone rock armour, buried by sand.
3	**Vertical seawall** – piled vertical seawall located at the back of the beach comprising precast concrete panels between supporting H-column and sheet pile toe.
4	**Vertical seawall with rock toe -** composite seawall arrangement comprising a piled vertical seawall design with a non-rigid rock rubble toe.
5	**Tiered vertical seawall with promenade –** piled vertical seawall arrangement composed of lower and upper seawall either side of a mid-level promenade along the back of the beach.

3.3 Concept options 1 and 2: Rock revetments

These concept design options provide a conventional rock armoured revetment, also referred to as rubble mound seawall, comprising two layers of graded rock armour overlying a graded rock filter layer. Example rock revetment seawall designs are shown in Figures 3.1 to 3.4 at Stockton Beach (Newcastle), Belongil (Byron Bay), Lennox Head, Port Kembla (Wollongong) and Collaroy Beach (Sydney). Other examples of rock revetment seawalls include the A-line on the Gold Coast.

Concept rock revetment designs have been considered for basalt (option 1 - rock density 2670 kg/m^3) and sandstone (option 2 - rock density 2300 kg/m^3) rock armour. Typical slopes for rubble mound revetments range between 1V:1.5H and 1V:3H. A steeper slope of 1V:1.5H was chosen to reduce the footprint of the structure to minimise encroachment into the active beach profile. Rock sizing was undertaken using Van der Meer methodology (1998) for two-layer armoured non-overtopped structures. A toe level of -1.0 m AHD was adopted with a horizontal berm apron at the toe of the structure designed in accordance with SPM (1984), with sufficient width and thickness to accommodate scour below the toe of the structure to a level of -2.0 m AHD. As a rock revetment is a non-rigid structure it is designed to withstand a certain degree of rock armour movement. An initial damage (few units displaced, S=2) was adopted in the design calculations and is consistent with that recommended in Appendix A.

Preliminary design variables and estimated wave overtopping are shown in Table 3.2. Preliminary concept design cross-sections are shown for a basalt revetment in Figure 3.5 and sandstone revetment in Figure 3.6. The rock revetment would include a wave return wall to significantly reduce wave overtopping to tolerable thresholds during storm events as shown in Table 3.2 and also impacts of sea level rise.

There are a number of sandstone quarries located within the Gosford region. However, testing for other recent projects has found that sandstone of suitable quality and specifications was not able to be sourced from local quarries (Central Coast Council per comms, 2021). Other potential sandstone sources include Bundanoon in the Southern Highlands, where sandstone rock armour was more recently sourced for construction of the Collaroy Carpark seawall on the Northern Beaches of Sydney in 2019 (Figure 3.4). Basalt rock armour could potentially be sourced from quarries further north in the Port Stephens and Forster regions. High density igneous rock is also available at quarries in the Newcastle area, used for construction of the South Entrance Beach rock groyne in 2017. It should be noted that final design and costing of a rock revetment structure is subject to rock availably at the time of tendering, noting that rock sometimes becomes available through construction excavation projects, while at other times there is competition between projects for available rock.

Table 3.2: Preliminary design variables and estimated wave overtopping for basalt and sandstone rock revetment.

Option	M_{50} (t)	D_{50} (m)	Estimated Overtopping (l/s/m)	
			No wave return	With wave return
1. Basalt rock revetment	6	1.3	20	< 1
2. Sandstone rock revetment -	11	1.7	20	< 1

NOTES: Wave overtopping rate estimated using design and assessment approach from 2018 Eurotop Manual for preliminary design crest level of +8.0 m AHD, 100 year ARI wave and water level conditions for a 3h storm duration and scour level of -1 m AHD.

M_{50} = median rock mass D_{50} = median rock diameter

Sand excavated during construction can be used to partially bury the structure and potentially re-vegetate its surface. However, this is likely to be eroded (including loss of vegetation) during storm events when exposed to wave activity and require ongoing nourishment or beach scraping to keep the structure buried with time, particularly for narrower sections of the beach where more frequent exposure to wave activity is likely.

At narrower sections of the beach, access is typically provided by designated walkways (often timber stairs) constructed over the revetment. Where the structure is located further back in the dune to the north of the study site, the revetment can be buried, and the dune reshaped to provide similar beach access walkways as to present without the requirement for stairs.

It is noted that rock revetment options have the largest (widest) footprint compared with other concept designs and will likely impede more frequently on alongshore beach access following storms and encroach further into the active beach profile. A more detailed comparison of the advantages/disadvantages of design options is provided in Section 5.3.

Beyond a 50-year design planning period, rock revetments are adaptable to future sea level rise by gradually topping up the structure with slightly larger (heavier) rock during maintenance works. Crest elevations may be raised by the placing of larger rock also along the crest or raising the height of the wave return wall at the crest with future sea level rise.

Sources: https://storymaps.arcgis.com/stories/75f46591f216484597cdcafc14fd8131 - Newcastle City Council
https://www.realestate.com.au/sold/property-house-nsw-stockton-134503234

Figure 3.1: Stockton Beach basalt rock revetment.

Images provided by Newcastle City Council and Dowling Real Estate

Figure 3.2: Rock Revetments at Belongil Byron Bay (left) and Lennox Head (right).

Images provided by James Carley (WRL).

Figure 3.3: Port Kembla basalt rock revetment constructed 2020.

Images provided by MHL and James Carley (WRL).

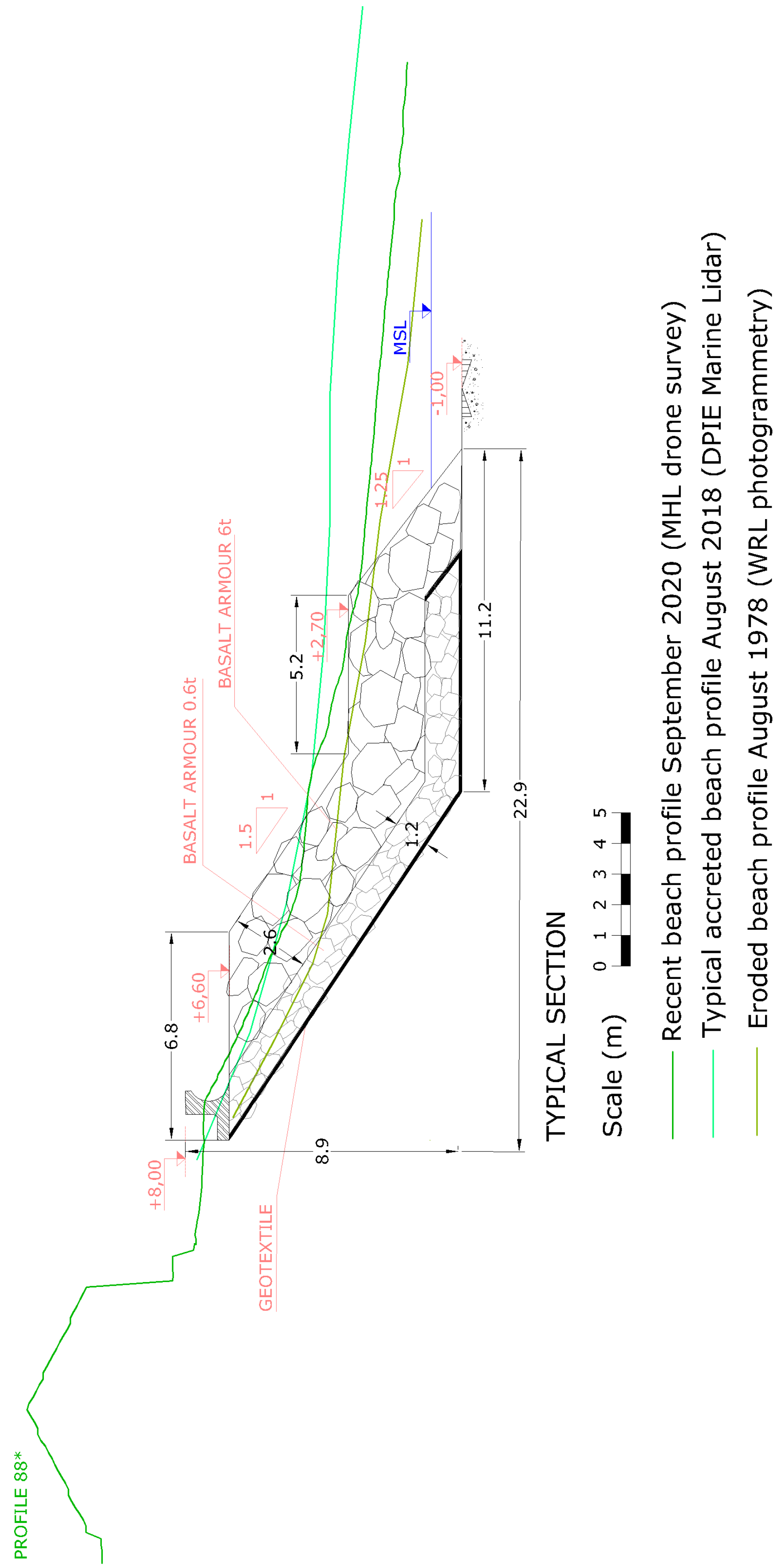
Sources:

https://www.scs.nsw.gov.au/soil-conservation-services-projects/collaroy-seawall-project-protection-from-coastal-erosion

https://www.engineersaustralia.org.au/sites/default/files/events-attachments/COPEP%20Sept19%20SiteVisit.pdf

Figure 3.4: Collaroy carpark sandstone rock revetment seawall during construction 2019 prior to being buried with sand.

Images provided by NSW Soil Conservation Service.



CONCEPT DESIGN OPTION 1:
BASALT ROCK REVETMENT

NSW GOVERNMENT

Manly Hydraulics Laboratory

Report MHL2780

Figure

3.5a

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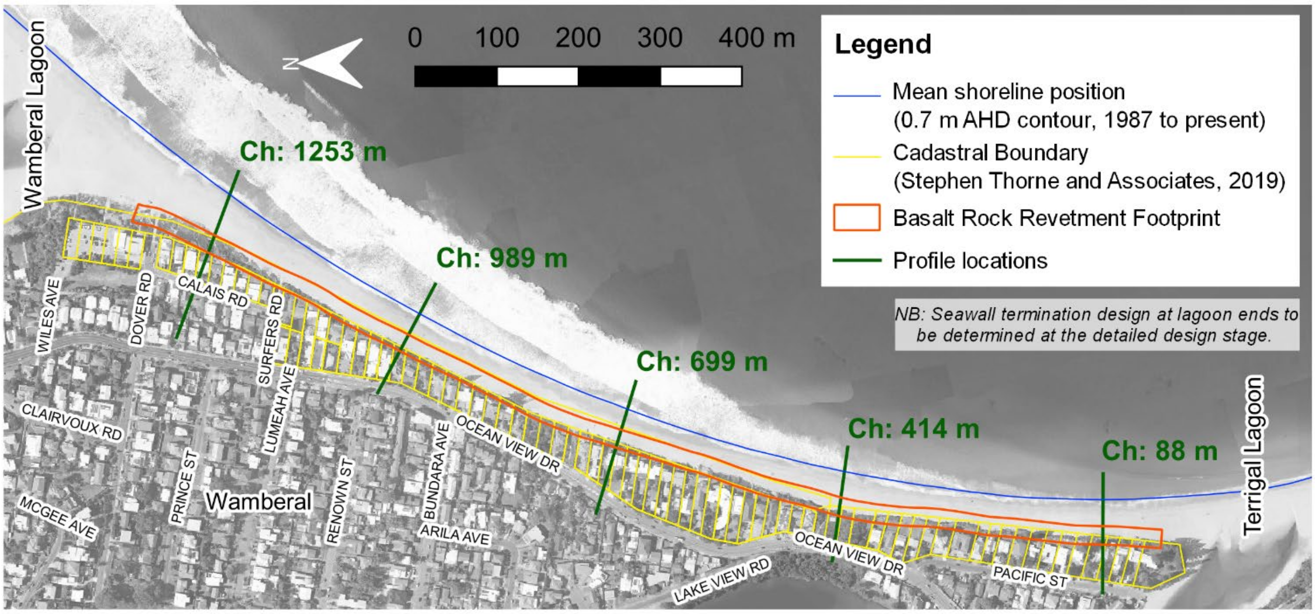

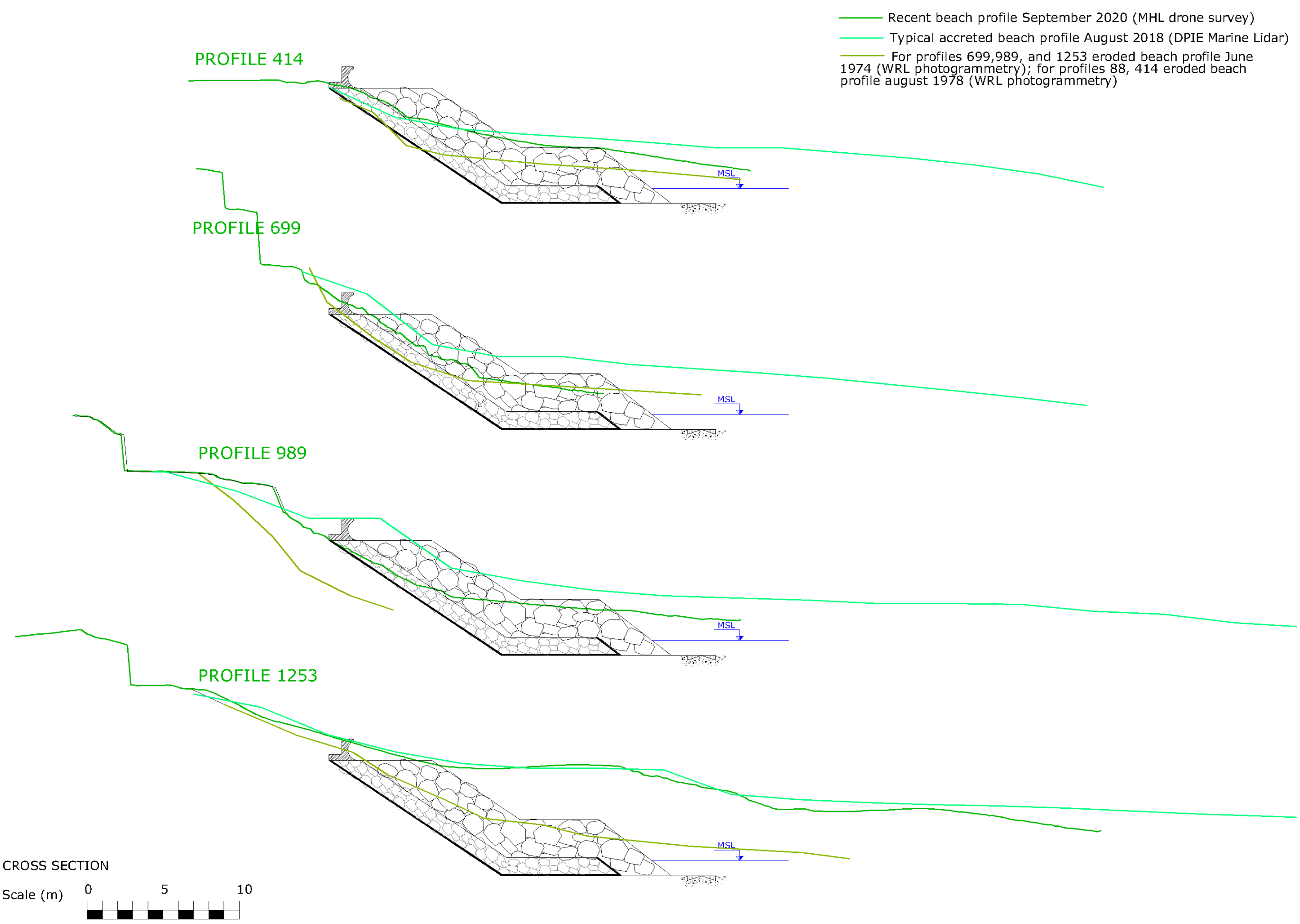

CONCEPT DESIGN OPTION 1:
BASALT ROCK REVETMENT - PROFILES

Manly Hydraulics Laboratory

Report MHL2780
Figure
3.5b

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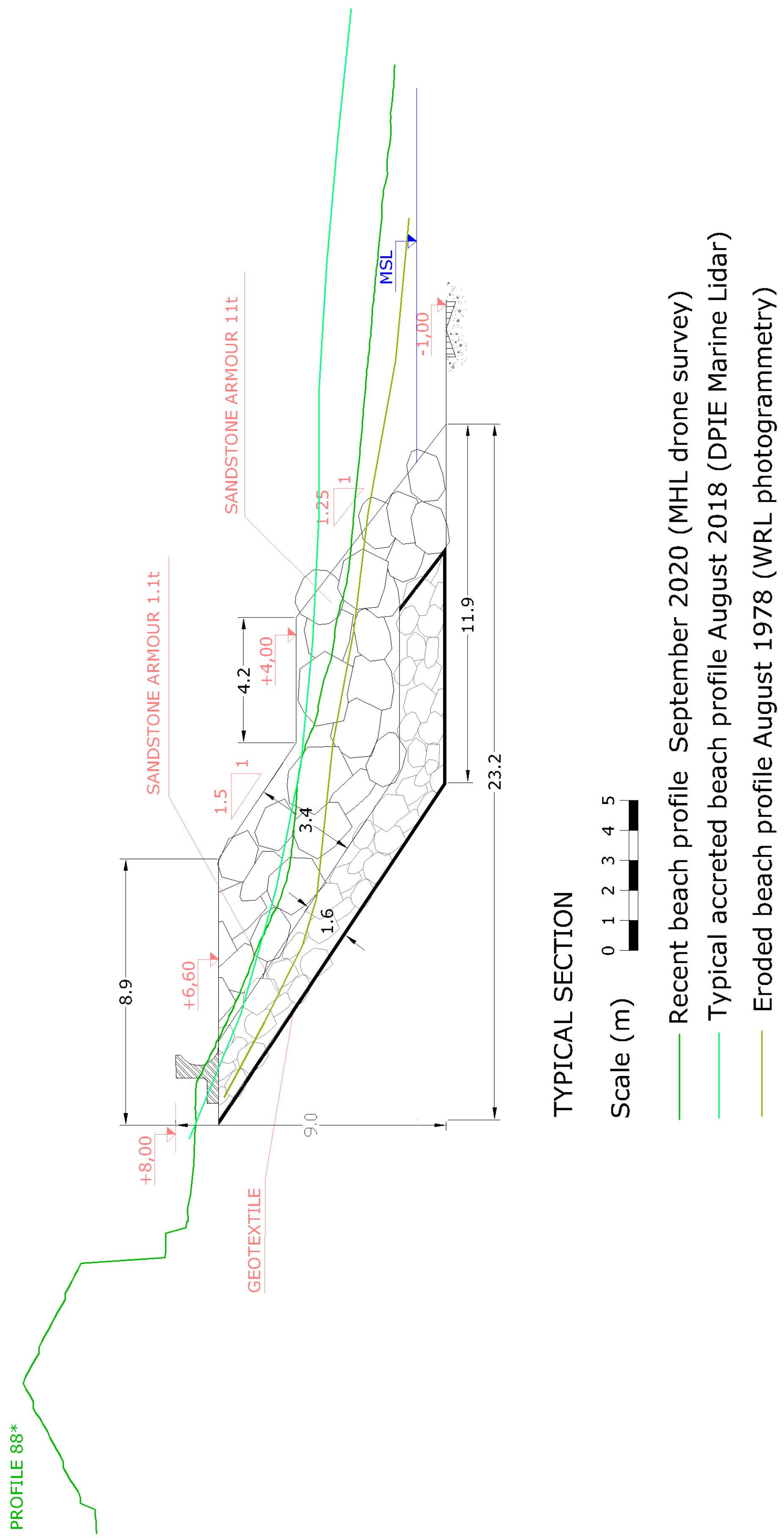

NSW GOVERNMENT

CONCEPT DESIGN OPTION 2:
SANDSTONE ROCK REVETMENT

Manly Hydraulics Laboratory

Report MHL2780
Figure
3.6a

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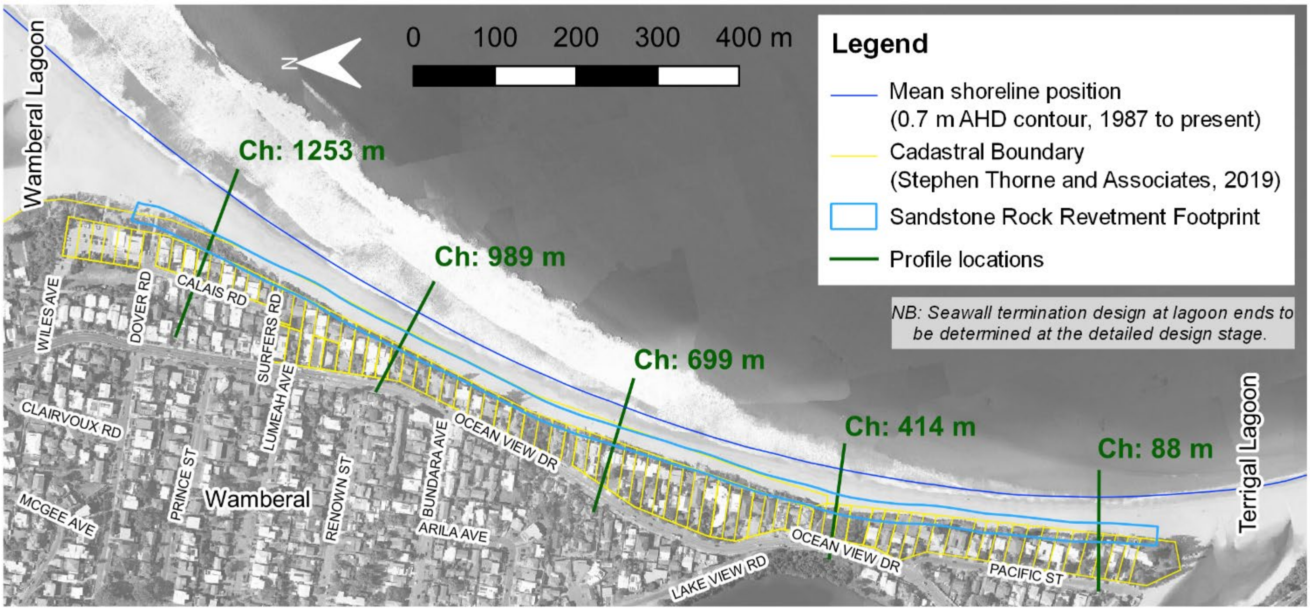
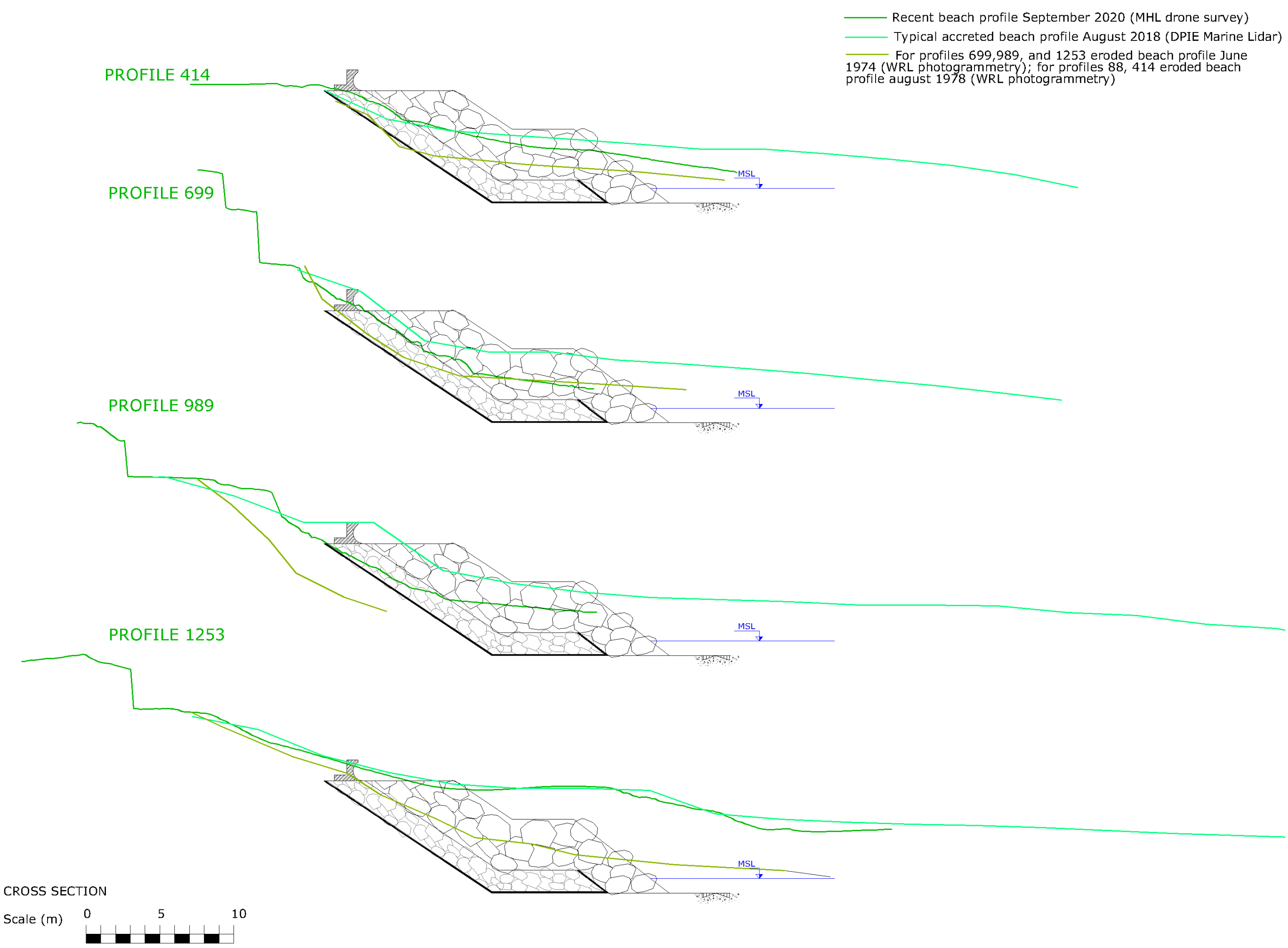




CONCEPT DESIGN OPTION 2:
SANDSTONE ROCK REVETMENT - PROIFLES

Manly Hydraulics Laboratory

Report MHL2780
Figure
3.6b

Sections.pdf

3.4 Concept options 3 and 4: Vertical seawalls

This concept design option is for a piled vertical (with slight sub-vertical tilt) seawall comprising precast concrete (high-strength MPa) panels between supporting H-columns with a sheet pile toe. Examples of vertical/concrete seawalls are shown at Flynns Beach Port Macquarie in Figure 3.7, South Cronulla Beach in Figure 3.8 and Manly (dimensioned sandstone laid in mortar) and Dee Why Beach in Figure 3.9. There are many other examples of vertical walls including Bondi Beach and Newcastle Beach.

Preliminary design variables and estimated wave overtopping are shown in Table 3.3. Preliminary concept design cross-sections are shown in Figure 3.10 and Figure 3.11. The top concrete panel would include a suitably designed wave return wall to minimise the seawall crest overtopping as demonstrated in Table 3.3. Weep holes in the lower face of the wall allow for drainage and release water pressure landward of the seawall. Different finishes to the subaerial seaward face of the vertical seawall could be considered to improve aesthetics including sand coloured concrete or sandstone architectural facing.

The structure would be supported by piles, horizontal ground anchors and a concrete capping beam at 0 m AHD. A sheet pile toe below the capping beam between 0 and -3 m AHD would be used to protect against scour at the toe of the structure. The required depth of piles is to be determined with detailed design. In regions of elevated bedrock, the seawall could be supported by ground anchors and/or tiebacks where required.

As an alternative option to the sheet pile toe (option 3), a vertical wall with a non-rigid rock toe (option 4) is also provided in Table 3.3 and Figure 3.11. This option consists of a rock two-layer apron between -1 and +2 m AHD design to protect the toe of the structure from scour instead of the sheet piling option.

Where the beach is wider in the north, the structure would be buried in the existing dune and revegetated. Beach access for the vertical wall option could be provided via designated structural staircases incorporated into the structure as shown in Figure 3.7.

Table 3.3: Preliminary estimated wave overtopping for vertical seawall options with and without a rock toe.

Option	Estimated Overtopping (l/s/m)	
	No wave return	With wave return
3. Vertical seawall	36	2
4. Vertical seawall with rock toe	21	1

NOTES: Wave overtopping rate estimated using design and assessment approach from 2018 Eurotop Manual for preliminary design crest level of +8.0 m AHD, 100 year ARI wave and water level conditions for a 3h storm duration and scour level of -1 m AHD.

[a] Estimated using mean value approach (no design approach provided) from 2018 Eurotop Manual for composite vertical wall with rock toe.

Importantly, the structure is to be located as far landward as possible to reduce encroachment into the active beach, while maintaining uniformity of alignment within the constraints of adjacent properties and setback requirements. Any existing ad-hoc material and rock protection works seaward of the vertical face of the seawall should be removed to enhance beach amenity, or where suitable used as fill where required landward of the seawall. Existing rock protection removed with construction should be replaced by sand nourishment to extend the level of the natural berm or foredune landward, back to the vertical face of the seawall.

The thickness of concrete panels and length of supporting piles would be determined as part of detailed design based on wave force impact loading and/or hydrostatic forces from wave overtopping. Details of the toe design should also be confirmed as part of detailed design. For example, the design scour level and presence of rock or other non-erodible materials may affect the depth to which concrete panels and/or other toe protection are provided.

Design of the crest of the vertical wall should accommodate the ability to adapt to future sea level rise beyond the 50-year design planning period. The final design could incorporate supporting H columns that extend above the crest level to accommodate the addition of future wave return wall panels to raise the vertical wall crest and reduce wave overtopping during major coastal storms.

Source: https://www.pmhc.nsw.gov.au/About-Us/What-We-Are-Up-To/What-we-have-delivered/Port-Macquarie-Projects/Flynns-Beach-Seawall

Figure 3.7: Flynns Beach, Port Macquarie vertical seawall design with ramp and stepped access. Constructed 2019.
Images provided by Port Macquarie-Hastings City Council.

Figure 3.8: South Cronulla vertical seawall with wave return.
Images provided by MHL.

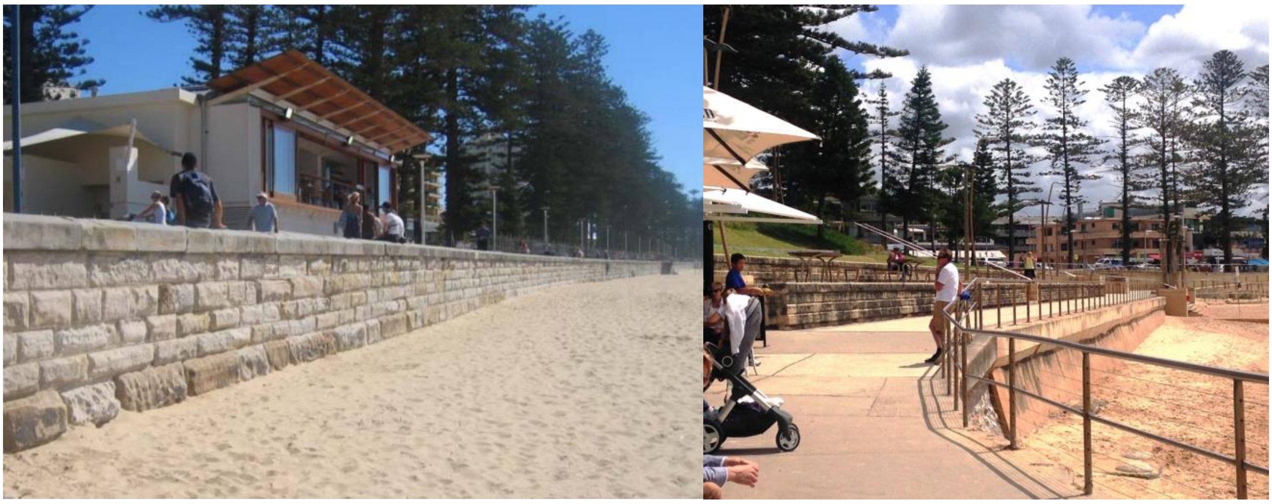
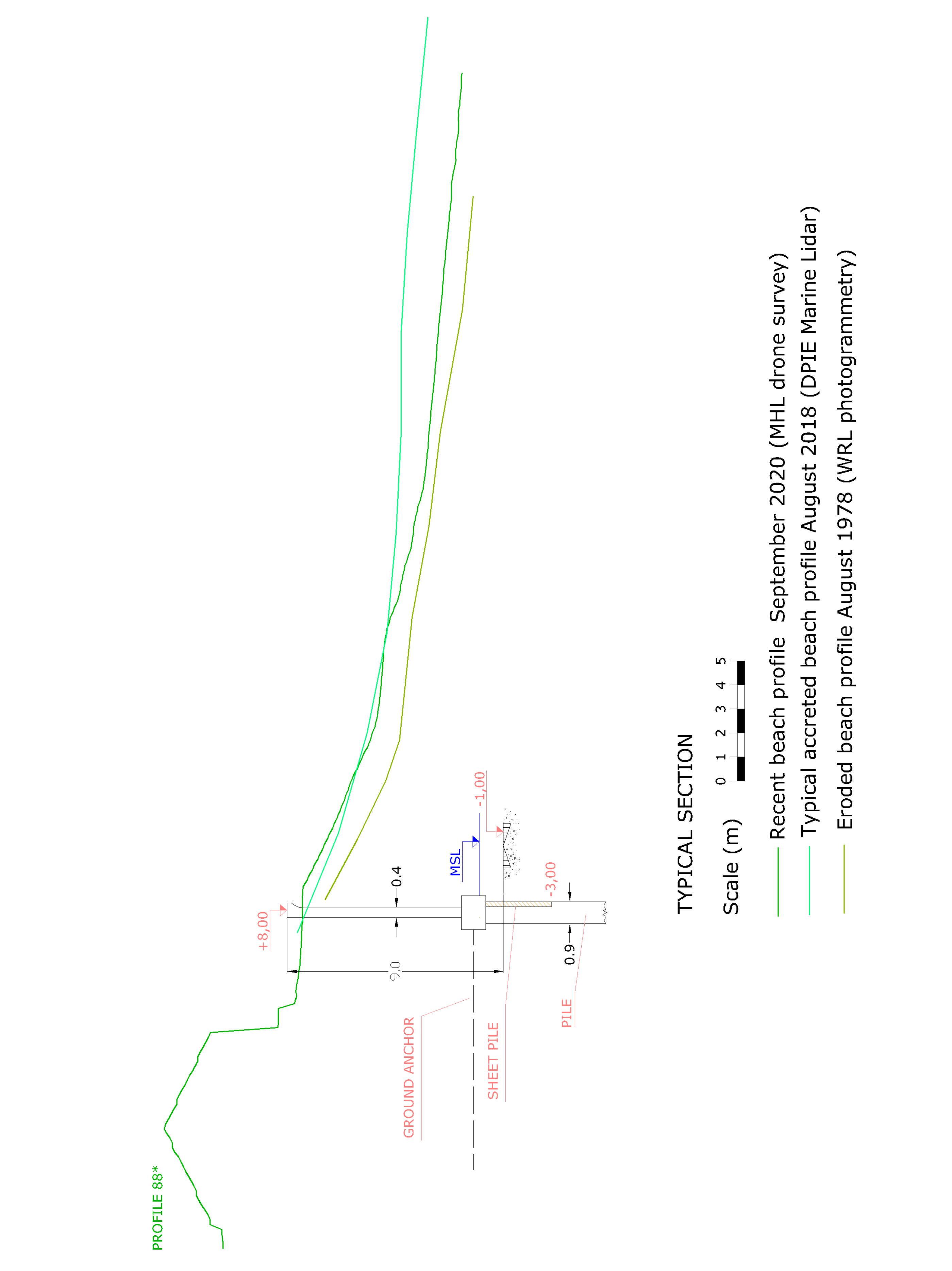


Figure 3.9: Vertical seawall at Manly (left, dimensioned sandstone laid in mortar) and Dee Why Beach (right) on Sydney's Northern Beaches.

Images provided by James Carley (WRL) and Seafarwide. **Source:** https://www.weekendnotes.com/the-beach-shed-cafe/



CONCEPT DESIGN OPTION 3:
VERTICAL SEAWALL

Manly Hydraulics Laboratory

Report MHL2780
Figure
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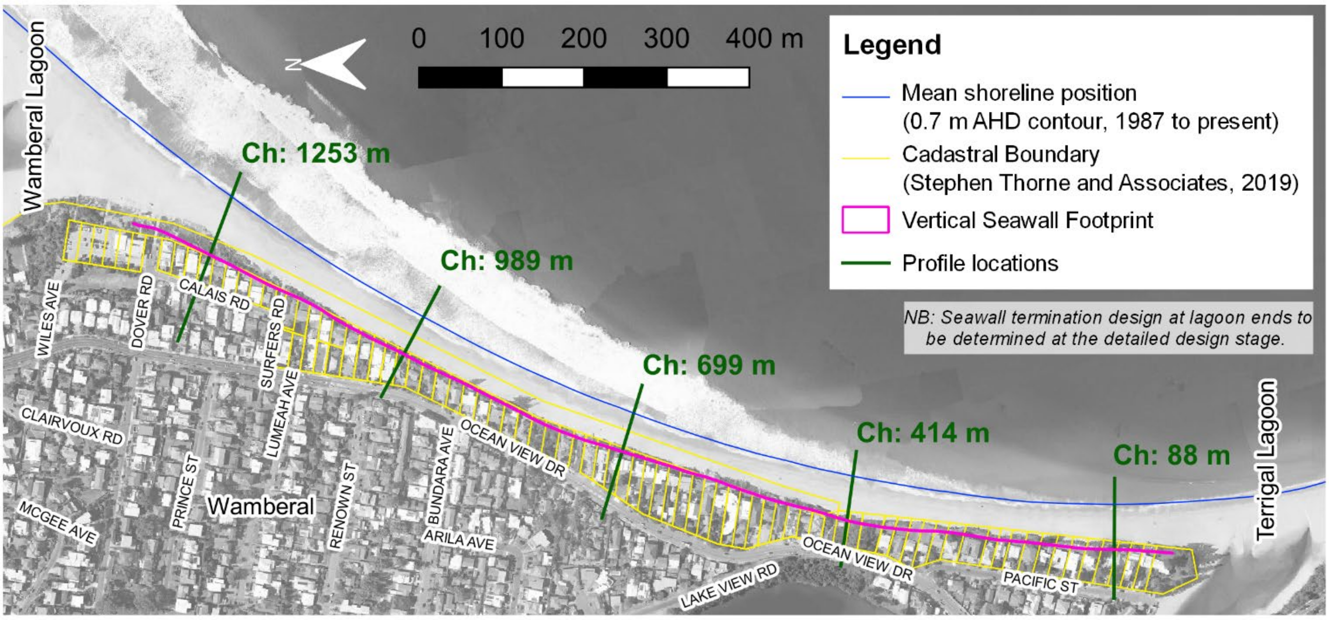

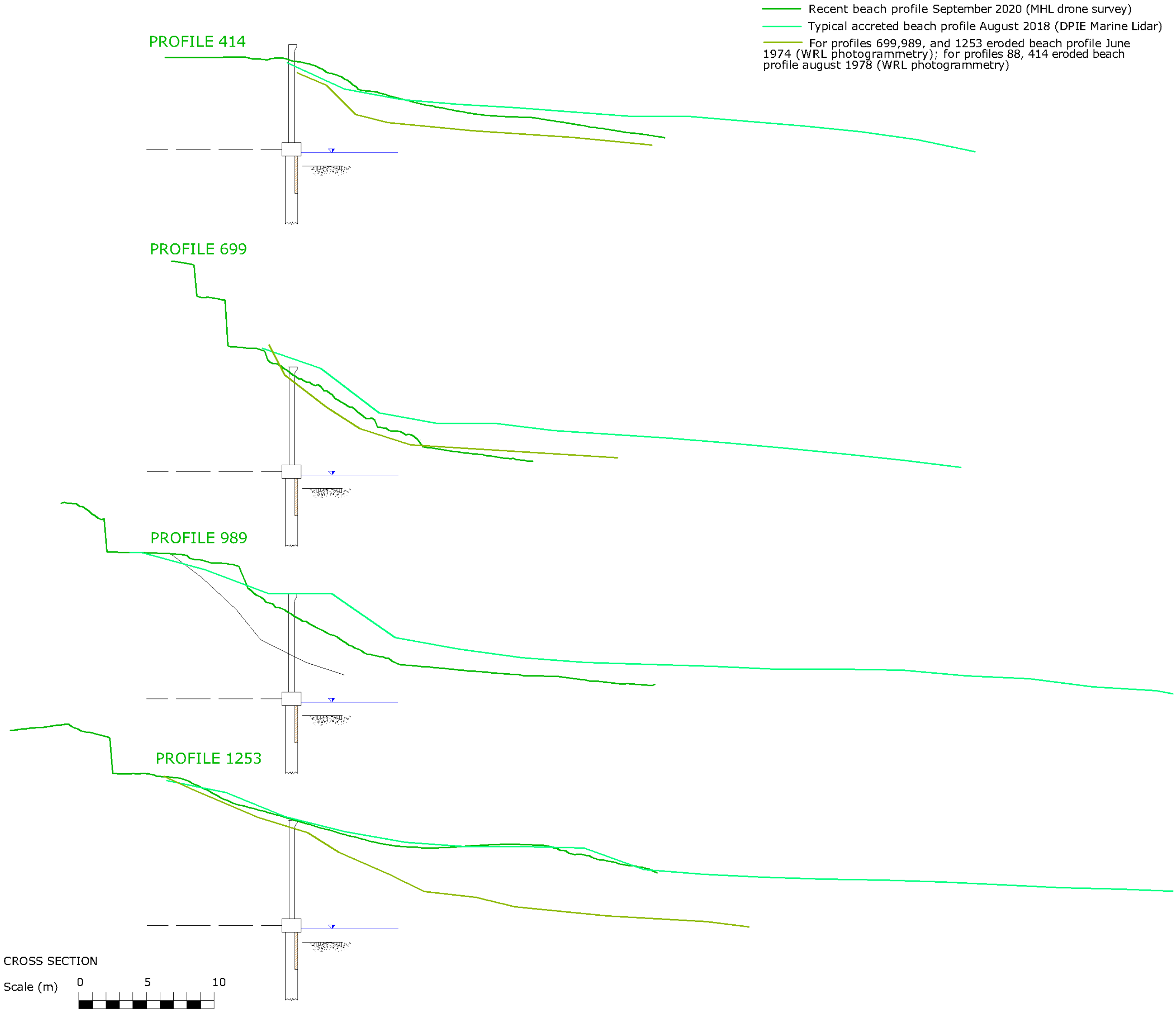

CONCEPT DESIGN OPTION 3:
VERTICAL SEAWALL - PROFILES

Manly Hydraulics Laboratory

Report MHL2780
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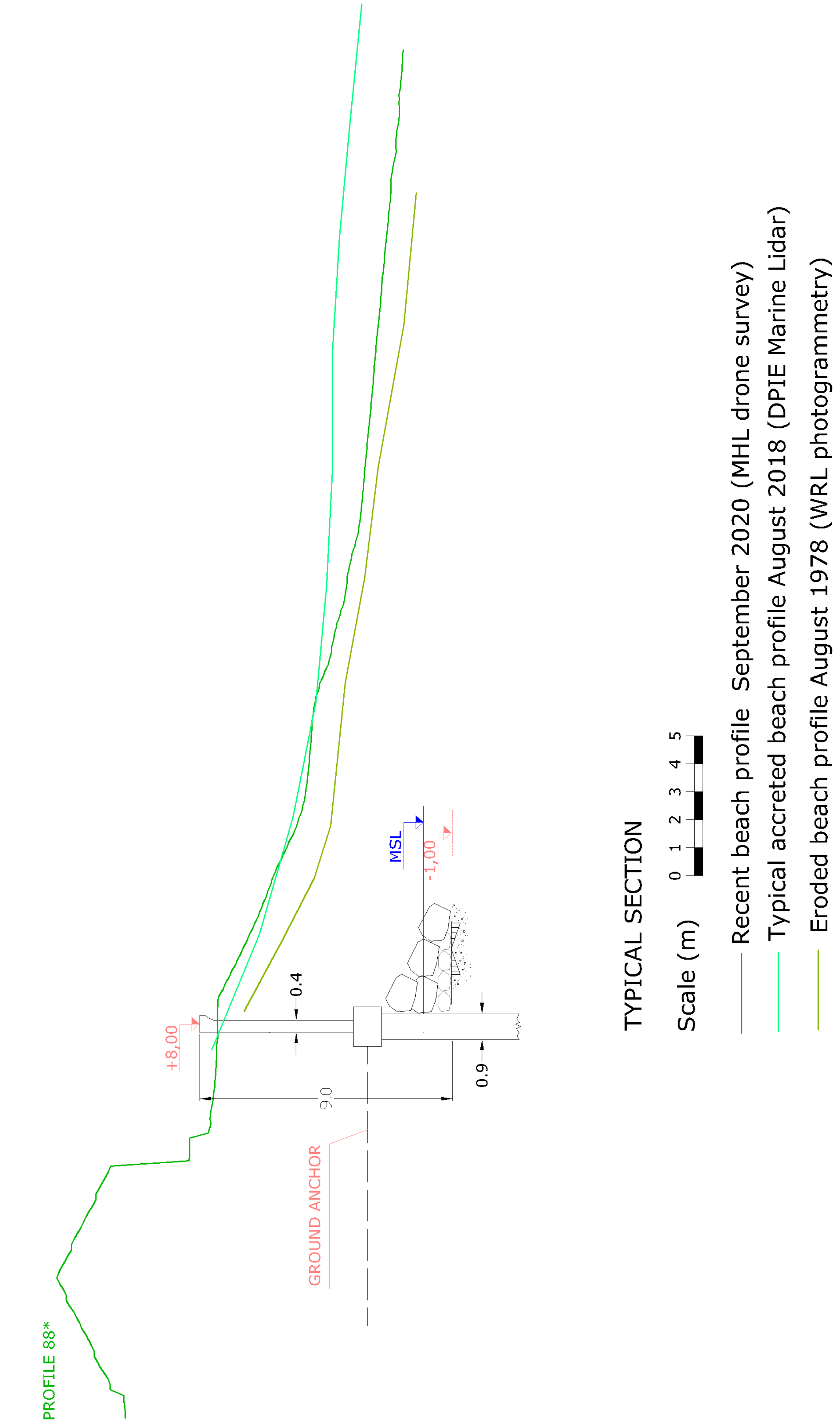

NSW GOVERNMENT

CONCEPT DESIGN OPTION 4:
VERTICAL SEAWALL WITH ROCK TOE

Manly Hydraulics Laboratory

Report MHL2780

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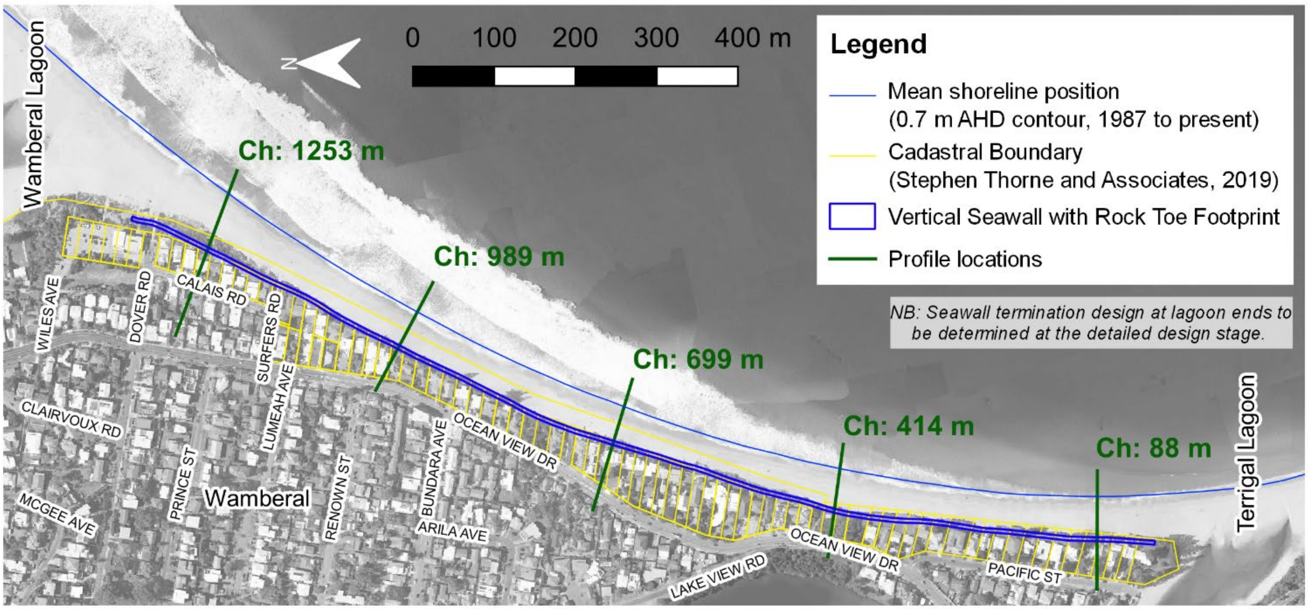

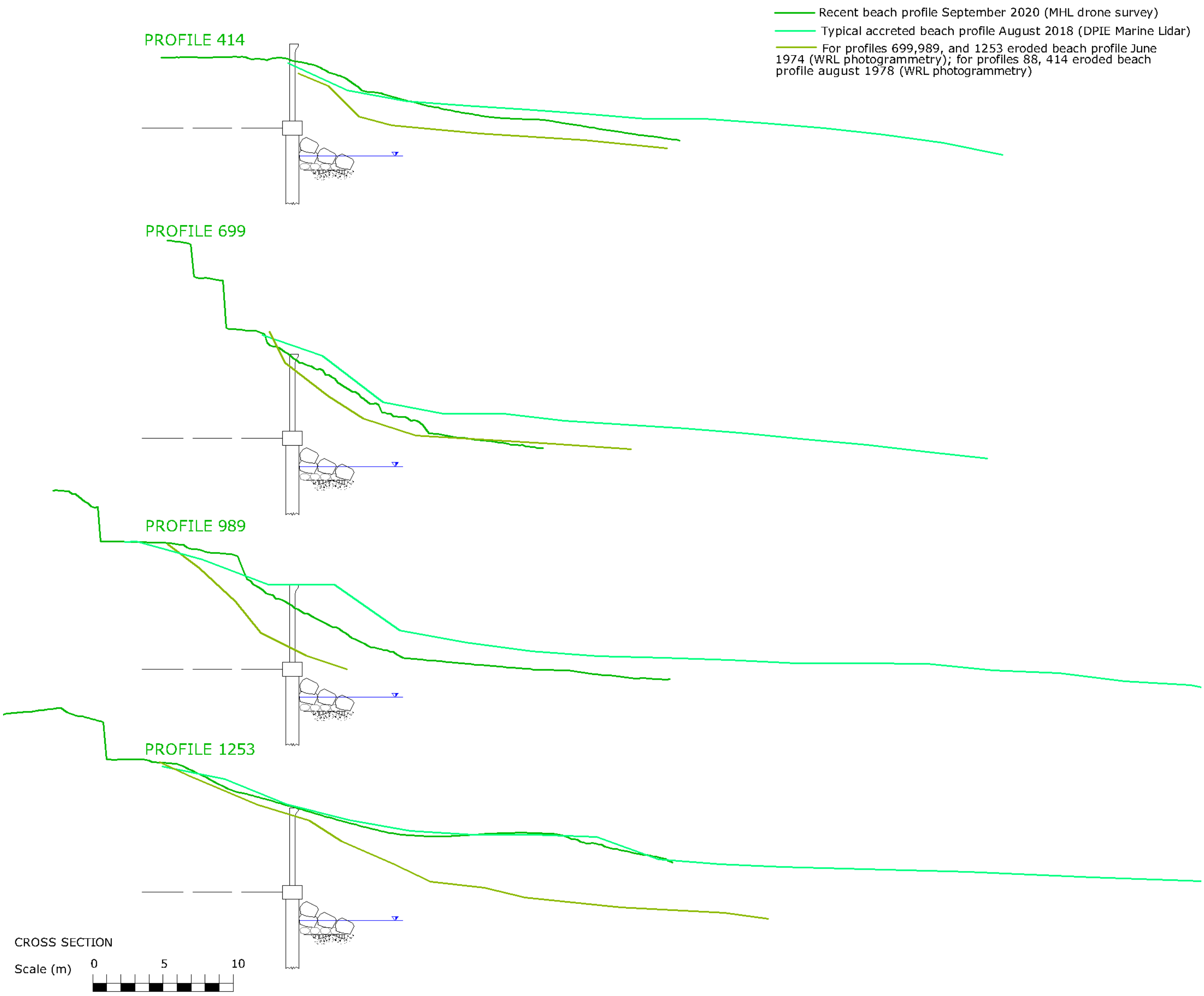

Manly Hydraulics Laboratory

CONCEPT DESIGN OPTION 4:
VERTICAL SEAWALL WITH ROCK TOE - PROFILES

Report MHL2780
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3.5 Concept option 5: Tiered vertical seawall with promenade

Concept design option 5 is a piled vertical (with slight sub-vertical tilt) seawall with a tiered (promenade) configuration, comprising a seaward (lower) and landward (upper) vertical seawall either side of a mid-level pathway to enhance beach access and foreshore amenity. Examples of comparable vertical walls with promenades are shown at the Blue Mile Tramway Wollongong in Figure 3.12, Sydney's Eastern Suburbs in Figure 3.13 and Newcastle Beach in Figure 3.14.

A preliminary concept design cross-section is shown in Figure 3.15 and preliminary wave overtopping estimates are shown in Table 3.4. The vertical components of the design would include those similar to options 3 and 4, including precast concrete (high-strength MPa) panels, H-columns, a piled toe on the seaward vertical wall, horizontal ground anchors as well as a precast concrete promenade (as an integrated structural element to the design) approximately 3 m wide for community amenity and to accommodate maintenance access. Geotextile and existing four tonne rock bags on the beach would be reused to provide fill to the area behind the landward wall to accommodate wave overtopping during major storms and create 1:1 slope from the rear promenade wall to a design crest level.

Careful design of wave overtopping and structure drainage will be a key component of detailed design. During large storms with high wave overtopping, the promenade would require to be temporarily closed to the public for safety reasons aided by automatic signage and data-driven alerts (such as that used at Figure Eight Rock Pools in Sydney's Royal National Park, Manly's Fairy Bower promenade overtopping monitoring system, or similar to those used for automated road closure & warning signage during road flooding). Both vertical walls would be topped with a wave return wall at their crest to reduce wave overtopping. The promenade would have a slight seaward slope with weep holes in the vertical walls to allow drainage through the structure. Three-dimensional physical modelling is recommended as part of detailed design to confirm design wave overtopping along the structure and refine the structural and drainage elements of the seawall design.

Table 3.4: Preliminary estimated wave overtopping for a tiered vertical wall with promenade concept design.

Option	Estimated Overtopping (l/s/m)	
	No wave return	With wave return
Tiered vertical seawall with promenade		
Lower wall crest (+4.0 m AHD)	670	130
Upper wall crest (+6.0 m AHD)	30	13
Backfill crest (+8.0 m AHD)	2	<1

NOTES: Wave overtopping rate estimated using design and assessment approach from 2018 Eurotop Manual for preliminary design crest level of +8.0 m AHD, 100 year ARI wave and water level conditions for a 3h storm duration and scour level of -1 m AHD.

An indicative promenade elevation of 4 m AHD backed by a 2 m vertical wall with a wave return at the crest has been adopted for concept design purposes. The final design promenade elevation would likely vary along the structure and should consider:

- Alongshore variability in foredune and berm elevations to minimise vertical relief from the promenade to sand level below to enhance beach access. After a storm, when the beach is eroded the vertical relief from the promenade to the beach will temporarily increase prior to the beach berm elevation fronting the wall rebuilding with natural recovery processes (typically to +2.5 to +3.5 m AHD) under mild wave conditions within 1-2 months following a major storm (Couriel et al., 2020).
- Alongshore variability in wave exposure and overtopping to satisfy a design frequency of promenade closures to public during larger wave events.
- Reducing privacy impacts on beach front residents with preference to a lower promenade elevation and screening effect of landward wall.
- Accumulation of wind-blown sand on promenade.

To accommodate adaptation to sea level rise beyond the 50 year design planning period, the concept design includes a handrail with either stainless steel or glass reinforced composite bar posts (and timber top railing) that can be used as reinforcement should raising the vertical face of the seaward wall be required in the future.

To reduce impacts on the privacy of beachfront homeowners it is recommended the height of the promenade also consider the landward line of sight of residential dwellings located typically higher on the foredune crest (typically +10 m AHD). A 2 m high vertical wall landward of the promenade has been included in the concept design to reduce privacy impacts and wave overtopping of the structure during large storms. Finishes to the subaerial seaward faces of the vertical components could be considered to improve aesthetics and potentially reducing wave runup and overtopping.

As per the other design options, the structure is to be located as far landward as possible to reduce encroachment into the active beach, while maintaining uniformity of alignment within the constraints of adjacent properties and setback requirements. Any existing ad-hoc material and rock protection works seaward of the vertical face of the seawall should be removed to enhance beach amenity or where suitable be re-used as part of the design. Void spaces underneath the promenade could be filled with undesirable materials presently on the beach.

The thickness of concrete panels and length of supporting piles would be determined as part of detailed design based on wave force impact loading and/or hydrostatic forces from wave overtopping. Details of the toe design should also be determined as part of detailed design.

Detailed design should consider where a promenade walkway is to ideally start and end along the structure, linking local community points of interest while considering constraints arising due to land tenure and other detailed design considerations.

Source: www.skyviewaerial.com.au - https://www.wollongong.nsw.gov.au/council-projects/ongoing-projects/the-blue-mile

Figure 3.12 Vertical wall with promenade at Blue Mile pathway, Wollongong. Completed 2018. Image provided by Wollongong City Council.

Sources: https://www.ozbeaches.com.au/blogs/beaches/bronte-beach-a-favourite-of-locals-and-tourists-in-sydneys-eastern-suburbs
https://www.waverley.nsw.gov.au/recreation/arts_and_culture/bondi_beach_sea_wall

Figure 3.13: Vertical walls with promenades at Bronte and Bondi Beach, Sydney. Images provided by OZ BEEACHES and Waverley Council.

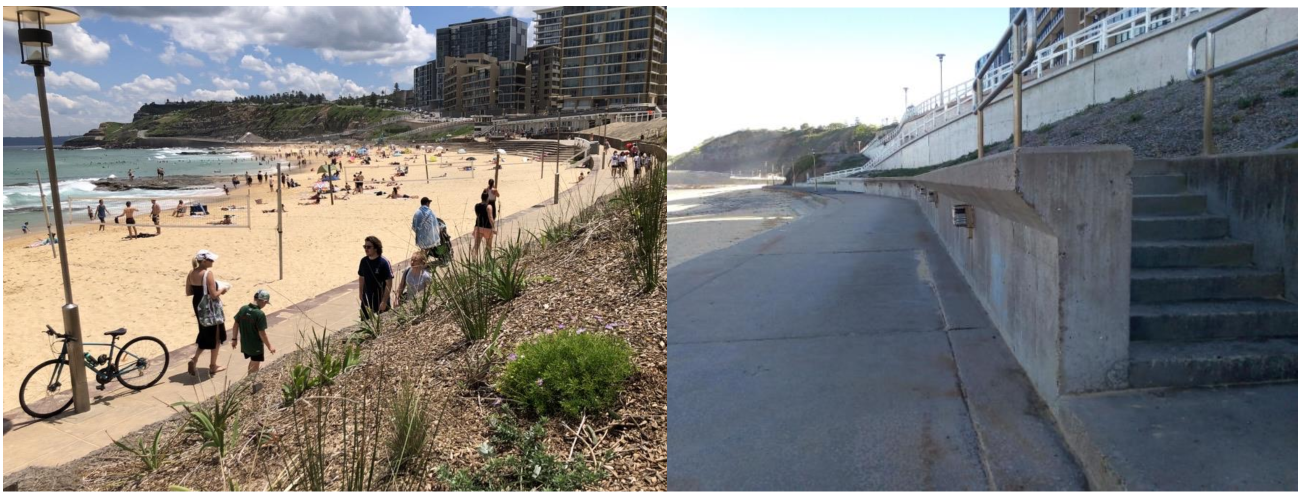
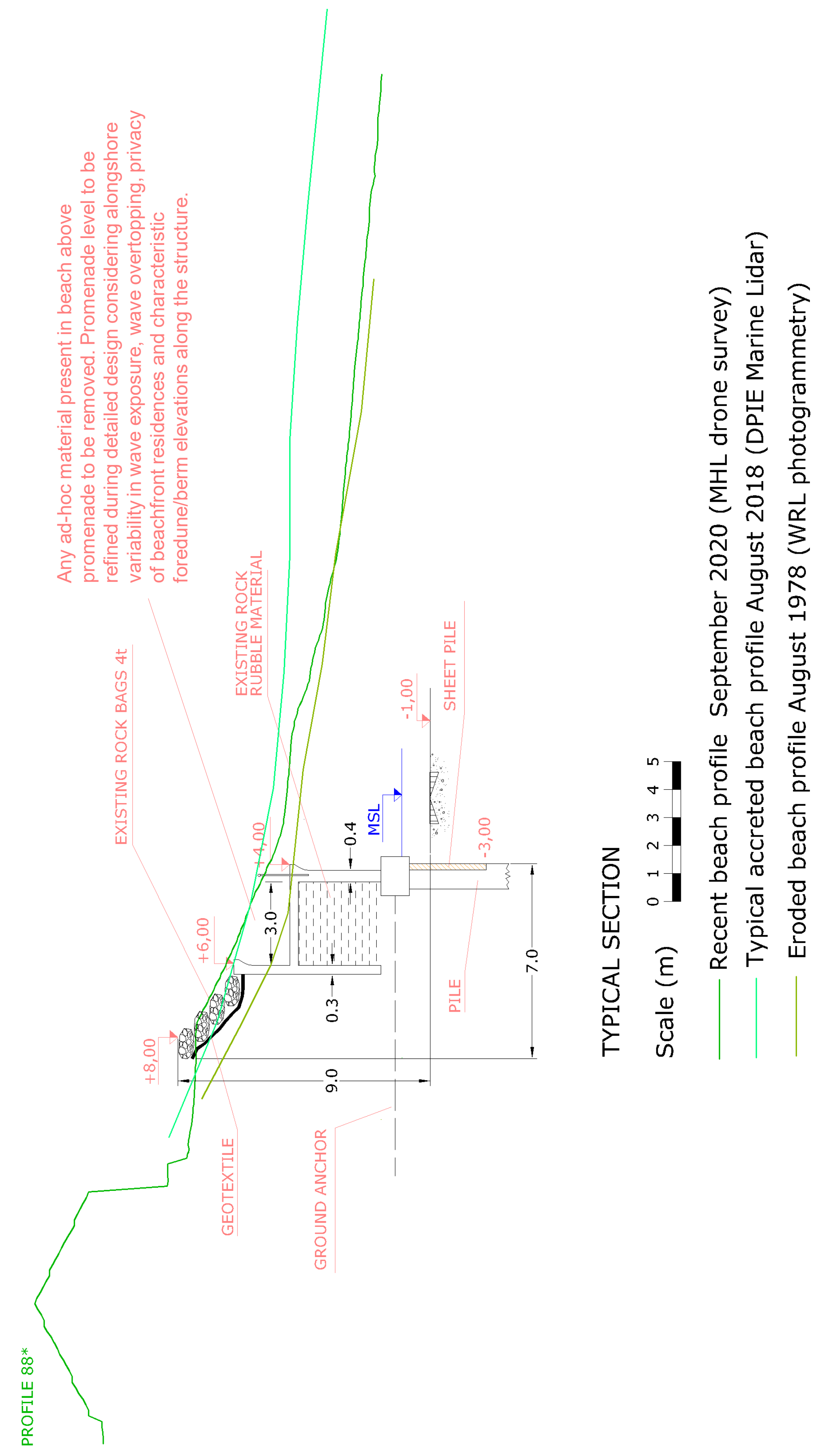


Figure 3.14: Tiered vertical wall (with stepped sections) and promenade at Newcastle Beach showing landward wall wave return on right. Images provided by Paul Donaldson (Central Coast Council) and MHL.



NSW GOVERNMENT

CONCEPT DESIGN OPTION 5:
TIERED VERTICAL SEAWALL WITH
PROMENADE

Manly Hydraulics Laboratory

Report MHL2780
Figure
3.15a

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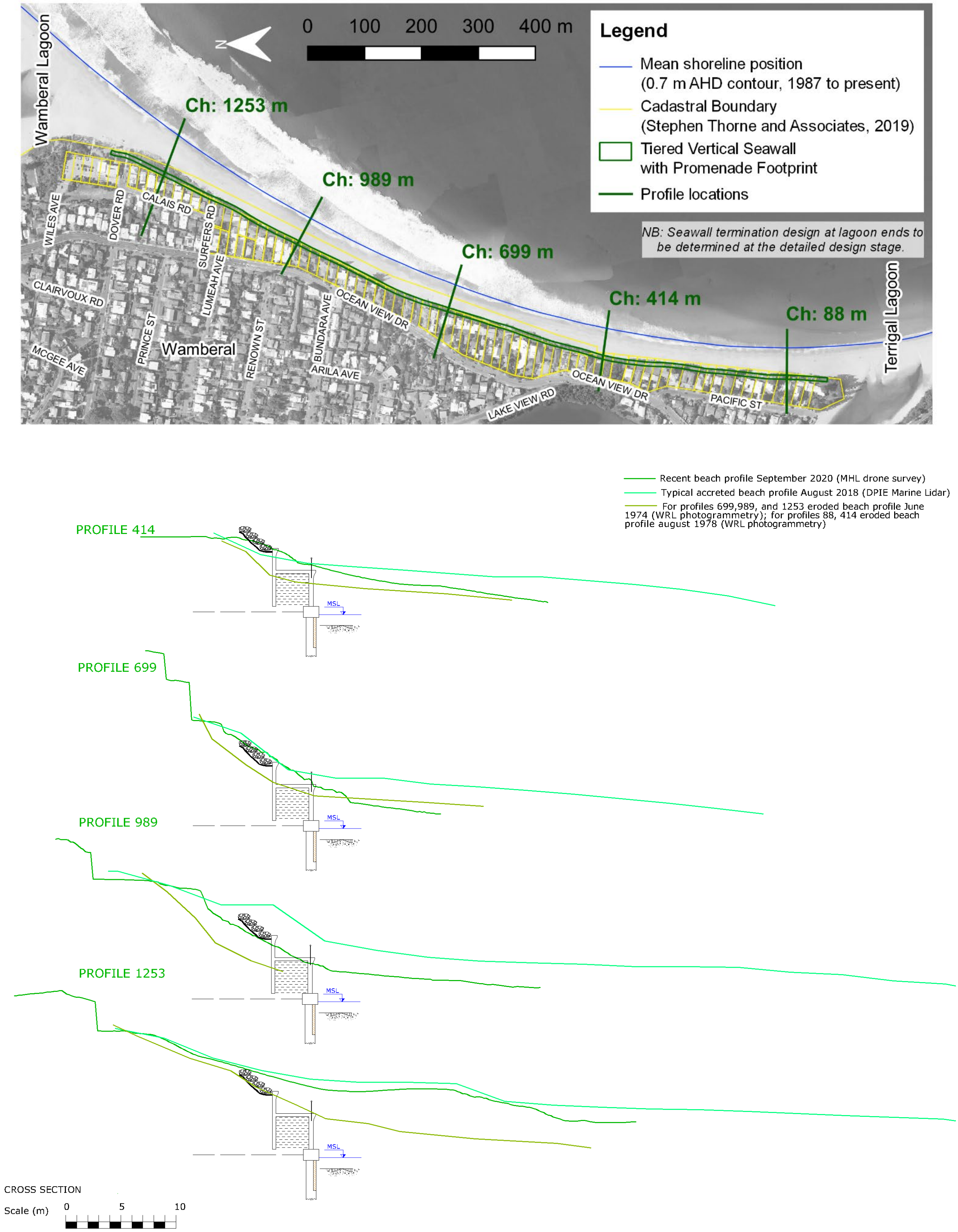

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CONCEPT DESIGN OPTION 5:
TIERED VERTICAL SEAWALL WITH PROMENADE - PROFILES

Report MHL2780
Figure
3.15b

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3.6 Summary of seawall options

A summary of the five seawall concept design options is provided in Table 3.5 including a brief overview description and concept design specifications. It should be noted that each seawall option is to be coupled with sand nourishment to maintain beach amenity. Sand nourishment requirements, sources and preliminary costings are provided in the *Stage 4: Sand Nourishment Investigation (MHL2795, 2021).* Advantages and disadvantages of each option are discussed in Section 5.3. Impacts on beach width amenity of each the concept design options are assessed in the *Stage 2 Coastal Protection Amenity Assessment (MHL2779, 2021)*.

Table 3.5: Summary of seawall concept design options

Option 1: Basalt Rock Revetment	Option 2: Sandstone Rock Revetment	Option 3: Vertical Seawall	Option 4: Vertical Seawall with Rock Toe	Option 5: Tiered Vertical Seawall with Promenade
Overview				
• Sloping rock revetment with basalt rock armour • Protection infrastructure • E.g., Lennox Head, Stockton, Belongil, Port Kembla	• Sloping rock revetment with sandstone armour • Protection infrastructure • E.g., Collaroy	• Vertical seawall (concrete panel with supporting H columns) with sheet pile toe protection and piled foundations • Protection infrastructure • E.g. vertical seawalls: Flynns Beach, Bondi, Manly, South Cronulla	• Vertical seawall (concrete panel with supporting H columns) with non-rigid rock toe protection and piled foundations. • Protection infrastructure • E.g. vertical seawalls: Flynns Beach, Bondi, Manly, South Cronulla	• Tiered vertical seawall with promenade • Vertical lower and upper walls, 3m wide walkway, sloping backfill. • Protection infrastructure & community asset. • E.g. Newcastle (City), Wollongong (Blue Mile)
Concept Design Specifications				
• +8 m AHD crest[a], berm scour apron, -1 m AHD toe • 2 layers of basalt primary rock armour: M_{50} =6t, D_{50} =1.3 m • Wave return wall at crest • ~23 m wide footprint (slope 1V:1.5H)	• +8 m AHD crest[a], berm scour apron, -1 m AHD toe • 2 layers of sandstone primary rock armour: M_{50} =11t, D_{50} =1.7 m • Wave return wall at crest • ~23 m wide footprint (slope 1V:1.5H)	• +8 m AHD crest[a], -3 m AHD toe, concrete piled • Wave return wall at crest • ~1 m wide footprint (slight sub-vertical incline)	• +8 m AHD crest[a], -1 m AHD rock toe, concrete piled • Wave return wall at crest • ~5 m wide footprint (slight sub-vertical incline)	• +8 m AHD crest[a], +4 m AHD promenade[a], -3 m AHD toe, concrete piled • Wave return wall at lower/upper wall • ~7 m wide footprint

[a] *Refined design crest (and promenade if adopted) levels considering alongshore variability in wave exposure, wave overtopping, privacy of beachfront residences and characteristic foredune/berm elevations along the structure are to be determined during detailed design.*

4 Preliminary seawall alignment

4.1 Preliminary alignment objectives

The primary objective for the preliminary alignment of the proposed concept designs is to align the seawall as far landward as practical to minimise encroachment into the active beach profile and impacts on amenity, while maintaining whole of embayment uniformity within the constraints of adjacent properties and setback requirements to provide sufficient maintenance and adaptation.

To achieve this the present study has re-evaluated the previous Wamberal seawall design alignment (rear of structure at crest) initially proposed by Couriel et al. (1998) including a later realignment by MHL (2004) at its northern end. The previous 1998/2004 design was assessed against present day building footprints, post July 2020 erosion scarp lines and long-term (1987 to present) characteristic shoreline curvature to determine its suitability for adoption in the present study.

Impacts on beach amenity and available beach width are to be analysed in detail separately as part of the *Stage 2 Coastal Protection Amenity Assessment (MHL2779, 2021)*. Wave overtopping was not considered as part of the preliminary alignment and would be investigated as part of detailed design.

4.2 Previous 1998/2004 design alignment

In 1998 the Water Research Laboratory UNSW were engaged by the former Gosford City Council to undertake the design of a Terminal Protective Structure (TPS or seawall) for Wamberal Beach (Couriel et al., 1998). Detailed design of a Seabee seawall with gabion/reno toe was provided as a preferred option at the time. A design alignment was proposed as part of the detailed design that comprised of a series of straight, plane sections between designated change points as shown in Figure 4.1. Various alignment constraints were considered as part of the final design alignment and are summarised in Table 4.1.

Table 4.1: Previous 1998/2004 design crest alignment criteria

Crest Alignment Constraint	Criteria
Wave overtopping	Minimum setback level of 3m from foreshore development based on a design mean wave overtopping rate of 4 L/s/m (determined from physical modelling) for a nominal final design crest level of 8m AHD. Where adopted final design crest levels were lower than 8m AHD, minimum setback distance was proportionately greater, ranging from 3 to 38 m, as determined from physical modelling.
Setback for access corridor	Minimum of 3m setback from buildings to allow for access during construction and subsequent maintenance following major storms.
Foredune erosion scarp	Follow foredune erosion scarp as much as practical to achieve a net cut and fill volumes for earthworks and maximise surplus clean sand for burial of TPS.
Crest alignment curvature	Uniform curvilinear alignment following foredune erosion scarp. Maximum angle of 3 degrees at change points to maintain aesthetically uniform alignment.

The study also included community consultation and negotiations with individual property owners that were considered as part of the seawall detailed design. As part of this process the crest and toe alignment was pegged out onsite for the benefit of affected landowners, clearly defining the crest and toe location for the proposed seawall at each property. Design refinement using physical model testing also included Council and community engagement to achieve a final design.

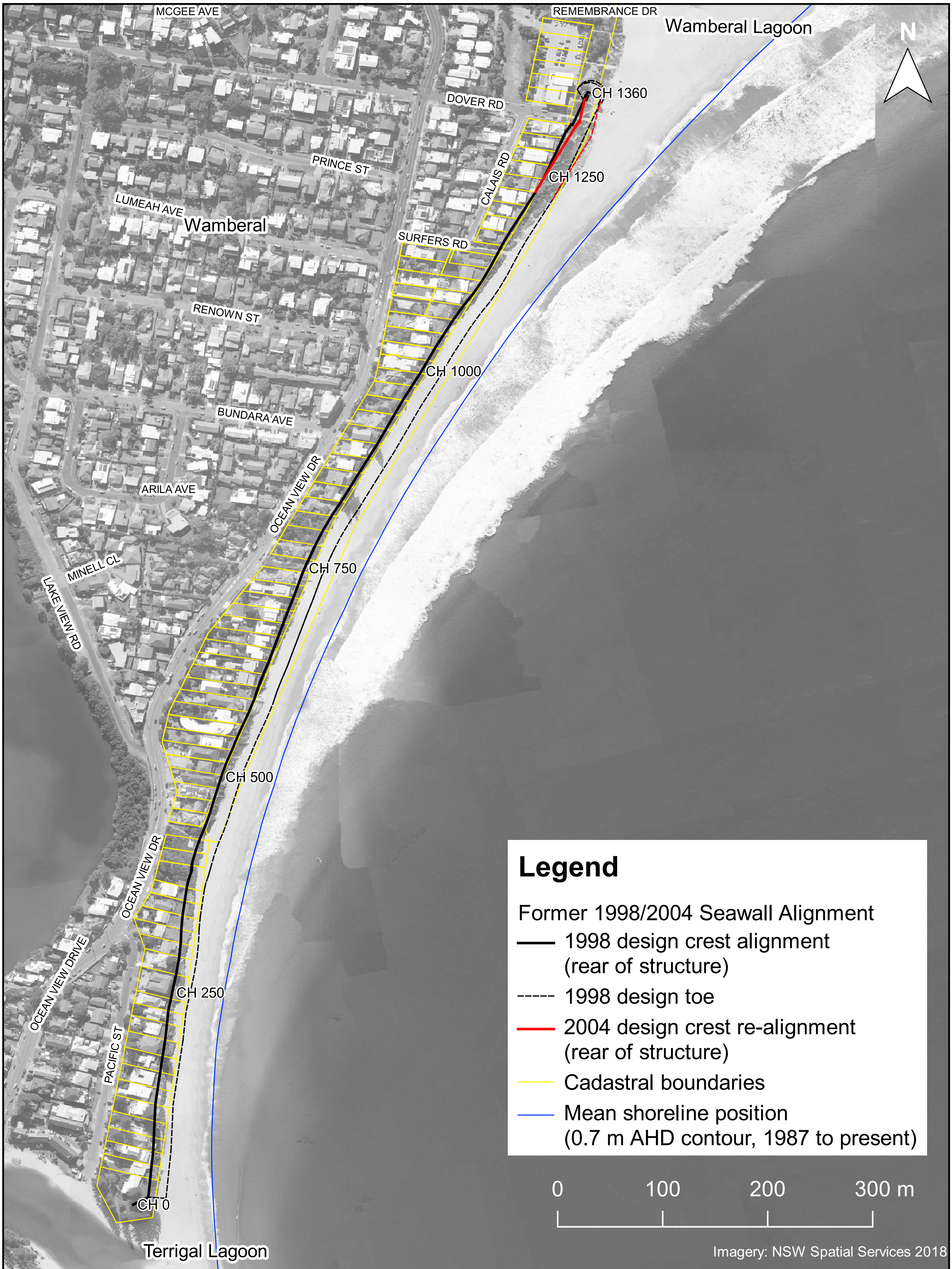
An Environmental Impact Statement (EIS) was prepared for the design in 2003 (MHL, 2003). Relevant to the alignment the study noted:

> *"The alignment selected is intended to maximise the available beach width following the construction whilst maintaining a minimum set back from existing development which is being protected."*

Details of encroachment for each property were considered as part of the detailed design and EIS. The EIS noted that structure will provide a more uniform and ordered appearance than occurs at present given the existing dis-contiguous ad-hoc protection on the beach.

In 2004 Manly Hydraulics Laboratory were commissioned by Gosford City Council to determine the realignment of the 1998 design due to the construction of a new residential development at 17 Calais Rd, Wamberal (MHL, 2004). A 120 m section of the seawall at the northern end, between 9 Calais Rd and Wamberal Surf Club, was realigned by up to approximately 5 m seaward of the previous 1998/2004 design as shown in Figure 4.1, re-joining the 1998 design at either end. It is noted that the typical beach width along this location is significantly wider than to the south of the study site and hence the 5 m seaward realignment is considered have negligible impacts on beach amenity.

Given the extensive work outlined above in determining the 1998/2004 alignment (including detailed design refinements, physical model testing, property by property consultation, environmental impact statement etc.), the present study has adopted the previous 1998/2004 alignment (rear of structure at crest) as a starting point to evaluating a suitable alignment in the current work.



NSW GOVERNMENT

PREVIOUS 1998 TERMINAL PROTECTIVE STRUCTURE DESIGN ALIGNMENT WITH 2004 REALIGNMENT

Manly Hydraulics Laboratory

Report MHL2780

Figure

4.1

Figure 4.1.pdf

4.3 Re-evaluation of previous 1998/2004 design alignment

The previous 1998/2004 design crest alignment (rear of structure - 1998 design with 2004 realignment) was re-evaluated against present day information to determine its suitability for adoption in the present study. Constraints used to assess the previous 1998/2004 design is summarised in Table 4.2 and include available setback corridor for maintenance, alignment relative to July 2020 erosion scarping and alignment relative to the long-term characteristic shoreline curvature. Criteria for each constraint are categorised to identify negative (red), neutral (neutral) and positive (blue) aspects of the previous 1998/2004 alignment in regard to each constraint and implications for the proposed concept designs outlined in Section 3.

Table 4.2: Re-evaluation constraints for previous 1998/2004 design crest alignment.

Crest Alignment Constraint	Datasets	Measure	Criteria
Setback for access corridor	- Digitised building footprints from 29 July 2020 drone survey - NSW Spatial Services 2018 aerial imagery - Classified point data from 2018 Marine Lidar Survey	Perpendicular distance from alignment to existing structure footprints	< 3 m less than minimum 3 – 6 m constricted > 6 m wide
Alignment to foredune erosion scarp	- Elevation data from 29 July 2020 drone survey - Elevation data from 2018 Marine Lidar Survey	Perpendicular distance from alignment to erosion scarp crest	> 5 m seaward of scarp crest 0-5 m seaward of scarp crest Landward of scarp crest
Alignment to characteristic shoreline curvature	- Beach width dataset described in Stage 2 reporting (MHL2779, 2021)	Deviations of alignment to mean shoreline curvature (0.7m AHD contour, 1987-2020 seasonally averaged)	>2 m seaward of mean shoreline curvature Within ± 2 m of mean shoreline curvature >2 m landward of mean shoreline curvature

4.3.1 Setback of existing structures for access corridor

Setback of the crest from existing buildings and structures is required to provide access along the seawall during construction and subsequent maintenance following major storms. A minimum of 3 m setback from the crest line to existing building and structure footprints was adopted following Couriel et al. (1998). Building footprints were digitised from georeferenced drone surveys undertaken by MHL in July 2020 and 2018 NSW Spatial Services aerial imagery. Building footprints were refined using point data classified as buildings/structures from the 2018 Marine Lidar Survey (OEH, 2019). Building footprints include land covered by awnings joined to buildings but exclude other structures such as uncovered decks, stairs, patios, pavement and protection works.

Figure 4.2 shows setback distances of the previous 1998/2004 design crest alignment to existing building and structure footprints. Of the total 1355 m crest length, approximately 1154m (85%) was situated more than 6 m from existing buildings, 184 m (14%) between 3 - 6 m and 17 m (1%) less than the minimum setback distance. Areas less than the minimum setback distance are shown in Figure 4.2, situated at 59-61 and 75 Ocean View Dr where covered awnings extend within 3 m of the crest with building footprints are located beyond 3m of the alignment.

<u>Design implications</u>

Overall, the previous 1998/2004 crest alignment is considered adequately distanced from existing building footprints to provide a 3 m access corridor suitable for each concept design. Potential actions to provide maintenance access in areas less than 3 m could be:

- Update the development controls to trigger the imposition/establishment of a maintenance corridor with future DAs while maintaining access either side of these locations in the interim. As the length of these areas is relatively short, maintenance access could be provided from either side and potentially utilise the public beach access between 65 and 67 to gain access in between.
- Alternatively, the awnings could be removed.
- Alternatively, maintenance access could be designed along the structure such as the option 5 vertical wall with promenade concept design.
- Alternatively, the alignment could be moved seaward at these locations to provide a 3 m access. However this option is considered undesirable compared to the options above due to potential broader impacts on coastal processes in the embayment with increased encroachment in the active beach profile.

Detailed design should further consider property specific structures that may obstruct access such as awnings, decks, stairs (including concrete staircase at 37 Ocean View Dr), patios, paved area and existing protection works.

It is noted for the tiered vertical wall with promenade option, an access corridor along the seawall is provided as part of the structure via the promenade pathway. This option is beneficial for alleviating access issues where existing structures are within 3 m of the crest and may potentially be aligned a further 3 m landward to reduce encroachment on the active beach and impacts on beach amenity.

NSW GOVERNMENT

SETBACK OF BUILDING FOOTPRINTS TO PREVIOUS (1998/2004) DESIGN CREST ALIGNMENT (REAR OF STRUCTURE).

Manly Hydraulics Laboratory

Report MHL2780

Figure

4.2

Figure 4.2.pdf

4.3.2 Alignment relative to foredune erosion scarp

A crest alignment may also aim to follow the most landward storm erosion scarp in the foredune profile in an effort to reduce fill required during construction and situate the structure at or landward of the extremity of historical storm erosion to minimise encroachment into the active beach profile. Since the 1998 crest alignment, further storm erosion at Wamberal has caused recession of the foredune erosion scarp in certain areas of the beach. Post-storm georeferenced drone surveys undertaken by MHL between the 18th and 29th July 2020 were used to map the erosion scarp line in the foredune profile. These surveys extend landward to include slip line erosion cracks in the foredune crest evident in the post-storm surveys. The distance from the erosion scarp crest was calculated to determine the degree to which the previous 1998/2004 design crest was situated seaward of the scarp crest. It should be noted that erosion scarping during the July 2020 storm was heavily concentrated in the central region of the beach. Storm erosion scarping can also concentrate toward the southern region or across the majority of beach including the northern region (e.g., as in 1974 or 2016) depending on the magnitude and direction of storm waves. Relic historical scarp lines are still evident along the beach. Where evident in the 2020 dataset, the most landward erosion scarp in the beach profile was taken as the adopted scarp line for the present analysis and referred to as the erosion scarp line.

Figure 4.3 shows the locations of the previous 1998/2004 design alignment to the erosion scarp crest. In regions where a foredune scarp is evident in the beach profile, approximately 43% of the previous alignment is located landward (in blue) of the erosion scarp crest. This is particularly the case for the southern half of the embayment. Approximately 42% of the previous alignment is located between 0 to 5 m (yellow) of the erosion scarp crest. Regions in red show where the previous alignment is situated more than 5 m seaward of the scarp crest, totalling approximately 185 m (~15%) and typically reflect discontinuities in the scarp line during recent erosion events. All areas of the alignment are located landward of the erosion scarp toe as estimated from July 2020 post-storm surveys.

<u>Design implications</u>

The previous 1998/2004 alignment is considered sufficiently aligned to the erosion scarp line, with the majority of the alignment situated either landward or within 5 m of the observed scarp crest line. Importantly no areas of the previous alignment are located seaward of the scarp toe, minimising encroachment into the active beach profile and reducing impacts on available beach width. Regions located more than 5 m seaward of the crest typically reflect discontinuities in the scarp line during recent erosion events.

It is noted that since the post-storm July 2020 surveys, emergency rock protection and beach scraping works have been undertaken and now cover much of the erosion scarp. Where the seawall is aligned landward of the scarp crest, any existing protection or ad-hoc material seaward of the structure, where suitable should be integrated into the seawall or removed from the beach where necessary. This provides additional width to the active beach profile, reducing the present encroachment of existing emergency and ad-hoc protection. Preliminary allowance for use of and/or management of these materials has been made in the comparative costing of the different concept design options in Section 5.2.

When aligned crest to crest, the smaller footprint of the vertical wall options in comparison to the larger rock revetment footprint further reduces encroachment into the active profile. The impacts of each concept design on available beach width is further described in Section 5.3 and analysed in the *Stage 2 Coastal Protection Amenity Assessment (MHL2779, 2021)*.

NSW GOVERNMENT

PREVIOUS (1998/2004) DESIGN CREST (REAR OF STRUCTURE) ALIGNMENT PROXIMITY EROSION SCARP CREST.

Manly Hydraulics Laboratory

Report MHL2780
Figure
4.3

Figure 4.3.pdf

4.3.3 Alignment relative to characteristic shoreline curvature

A crest alignment may also consider the characteristic curvature of the shoreline where measured data is available. Seasonally averaged shoreline data of Wamberal Beach from 1987 to 2020 collected from a range of sources has been assembled as part of the *Stage 2 Coastal Protection Amenity Assessment (MHL2779, 2021)*. A mean shoreline position (0.7 m AHD contour, approximately MHW) was extracted from the dataset and overlaid on the previous 1998/2004 design alignment (by subtracting the median offset) to identify regions of the alignment that deviate from the characteristic curvature of the mean shoreline.

Figure 4.4 shows the previous 1998/2004 design alignment relative to the characteristic shoreline curvature from measured data between 1987 and 2020. Sections in blue indicate regions of the crest alignment that are situated relatively landward (> 2 m) of the characteristic shoreline curvature and account for approximately 516 m (38%). This occurs at the 300 m northern end where the alignment deviates landward into the foredune area as well as at the ruins and also at the very south of the alignment.

Sections in yellow indicate regions where the crest is relatively close (within ± 2 m) of the characteristic shoreline curve and account for approximately 444 m (33%), occurring at various locations.

Sections in red indicate regions where the previous 1998/2004 design crest alignment is situated relatively seaward (> 2 m) of the characteristic shoreline curve and account for 395 m (29%). Sections in red indicate where the previous alignment tends seaward relative to the average shoreline curve and include areas between 25-31 Pacific St (CH 240-310), 37-61 Ocean View Dr (CH 520-720) and 75-85 Ocean View Drive (840-940). The maximum seaward deviation of the alignment is situated between 43 and 55 Ocean View Drive (CH 570-670), reaching up to 6.7 m seaward of the characteristic shoreline curvature.

In Figure 4.4 the shaded region in blue corresponds to the envelope enclosed by the seasonally averaged minimum and maximum shoreline positions. This region reflects a typical zone of seasonal shoreline variability and does not encompass variability associated with shorter-term shoreline fluctuations (i.e. extreme erosion events such as storms). Importantly all seawall footprints are situated landward of this region of typical zone of seasonal shoreline variability. A comprehensive assessment of the beach width and encroachment impacts of each concept design options including shorter-term shoreline variability is provided in the *Stage 2 Coastal Protection Amenity Assessment (MHL2779, 2021).*

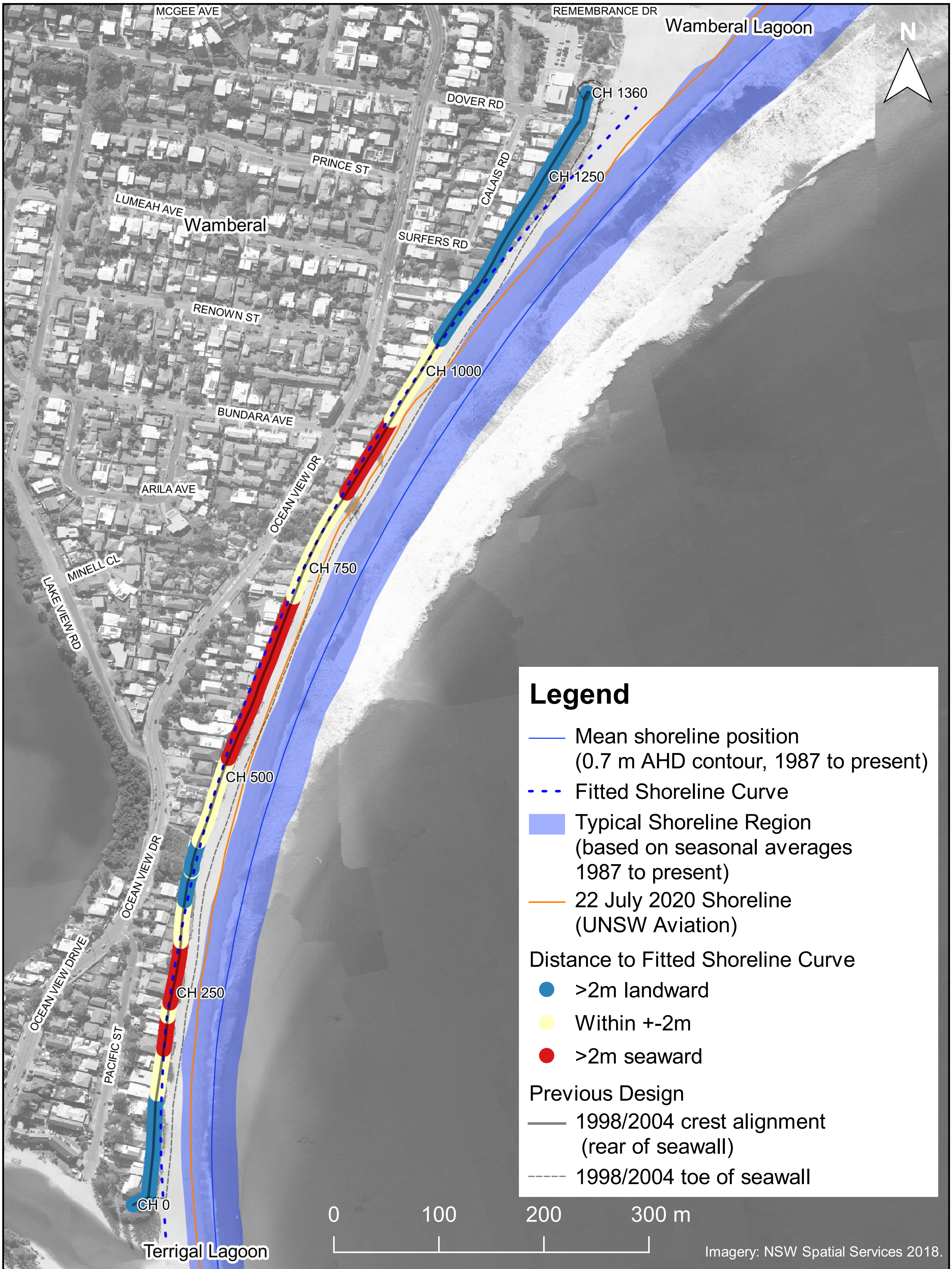
<u>Design implications</u>

In addition to the erosion scarp line, the mean shoreline position provides a reference curvature to be considered as part of the alignment. Ideally an alignment should seek to minimise seaward deviations to this curvature in regions where the beach is typically narrower and preference a smooth curvilinear alignment as far landward as possible to minimise encroachment on coastal processes.

For the purpose of a preliminary design alignment, the previous 1998/2004 design is considered sufficient in this regard. For regions identified in red in Figure 4.4, a more landward positioning may be limited by the high degree of urban encroachment in these areas. The option for the vertical seawall with promenade may alleviate these pinch points by potentially allowing a further 3 m landward alignment via utilising the promenade as also a maintenance corridor.

Without a more detailed assessment of individual properties including buildings, decks, patios and other structures, it is difficult to determine whether a more landward alignment can be achieved while maintaining suitable access along the structure. It is recommended that these considerations be investigated as part of detailed design.

It should be also noted that each seawall option is to be coupled with sand nourishment to maintain beach amenity, with sand nourishment availability also to be considered as part of selecting the final design alignment.



NSW GOVERNMENT

PREVIOUS (1998/2004) DESIGN CREST (REAR OF STRUCTURE) ALIGNMENT RELATIVE TO AVERAGE SHORELINE (0.7m AHD CONTOUR) CURVATURE.

Manly Hydraulics Laboratory

Report MHL2780

Figure

4.4

Figure 4.4.pdf

4.4 Preliminary seawall alignment and footprints

Following the findings from Section 4.3 the previous 1998/2004 design crest alignment (rear of structure - 1998 design with 2004 realignment) was considered a suitable preliminary crest alignment that has been adopted in the present study. Only minor changes to the previous alignment have been made to create smooth transitions at areas with stepped crest level transitions in the former Seabee design, located near The Ruins and Dover Rd.

The adopted preliminary crest (rear of structure) alignment and concept design footprints are shown in Figure 4.5 and Figure 4.6 for the southern and northern ends of Wamberal Beach respectively. It should be noted that each seawall option is to be coupled with sand nourishment to maintain beach amenity, with sand nourishment availability also to be considered as part of selecting the final design alignment. Sand nourishment requirements, sources and preliminary costings are provided in the *Stage 4 Sand Nourishment Investigation (MHL2795, 2021)* report.

The largest seawall footprints are associated with the rock revetment options, approximately 25% larger in footprint than the previous 1998/2004 Seabee seawall design as shown in Table 4.3. The rock revetment options encroach further into the active beach profile and when the beach is in an eroded state, these structures are likely to impede on access along the beach and amenity. The smallest footprint in Table 4.3 is for the vertical seawall option (95% smaller than previous design), increasing for the vertical seawall with rock toe (77% smaller than previous design) and promenade (56% smaller than previous design) respectively. *The smaller footprint of the vertical wall options, combined with an alignment as far landward as possible, reduces encroachment into the active beach profile and associated impacts on coastal processes and beach amenity.* For this reason, the adopted seawall alignment combined with seawall options 3, 4 or 5 are expected to provide the best amenity outcomes for the beach and its users.

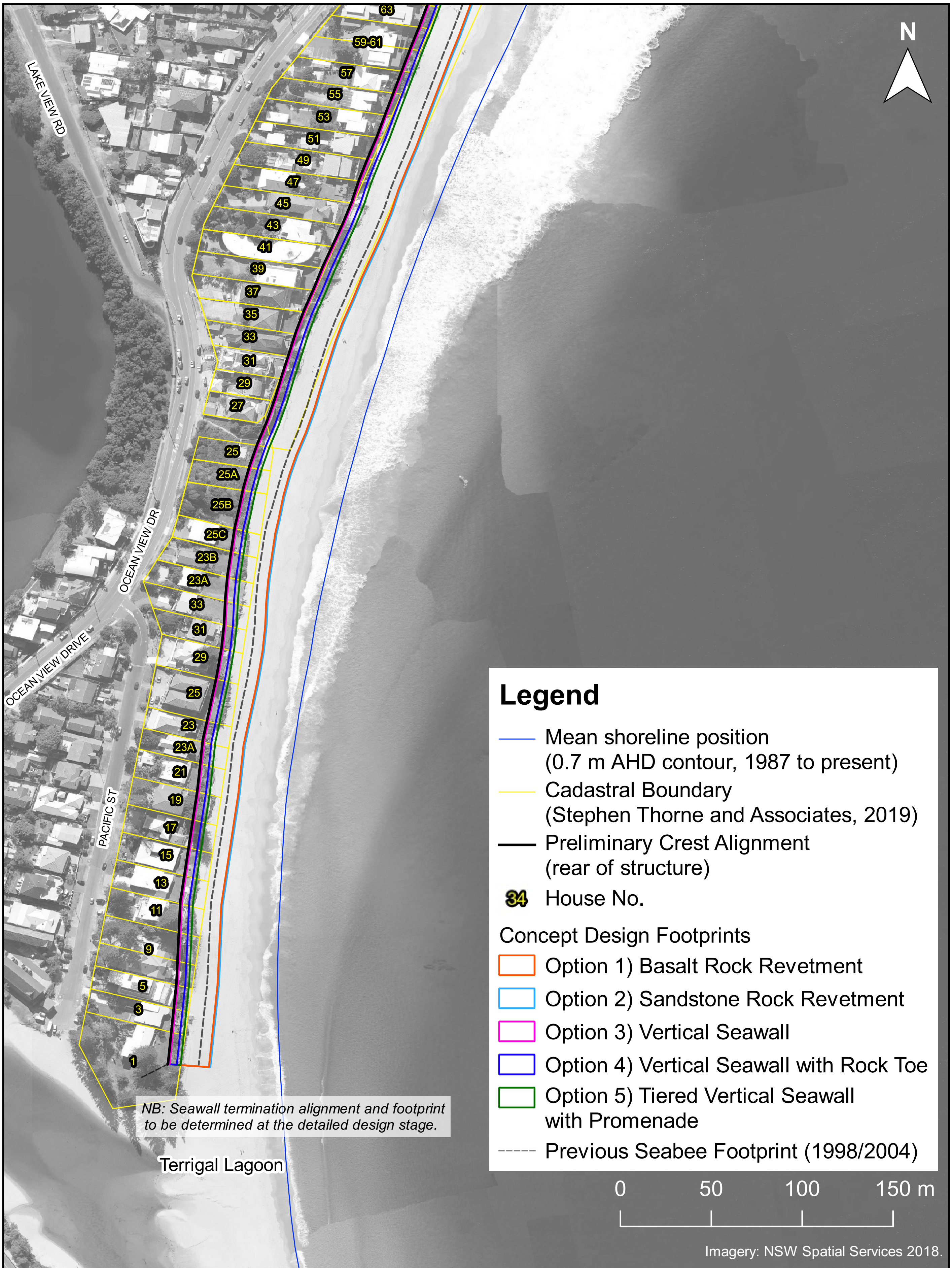
Property boundaries are also shown in Figure 4.5 and Figure 4.6, provided from the Detail and Contour Survey of Terrigal Lagoon to Wamberal Lagoon undertaken on the 13th June 2019 by Stephen Thorne and Associates. The proportion of each concept design footprint area located within private land boundaries is summarised in Table 4.3. All concept designs have a proportion of their footprint in private land with large footprint designs extending further seaward into public land. The concept alignment and footprints encroach furthest into private residential land along the southern 400 m of the structure, south of the vacant land at 25 Ocean View Dr (also known as “The Ruins”). In particular, the construction of a public promenade walkway in this region requires consideration of associated land tenure issues, with an option for the promenade to turn in at “The Ruins” (connecting to a roadside walkway running down Pacific St) and the seawall transitioning to a vertical wall design to the region south of The Ruins. As per the previous 1998/2004 design process, community consultation and consent of private property owners would be required.

It is strongly recommended that the final design alignment (to be determined with detailed design) be located as far landward as possible to reduce encroachment into the active beach, while maintaining uniformity of alignment within the constraints of adjacent properties and setback requirements. Any existing ad-hoc material and rock protection works seaward of the proposed seawall should be removed, or where suitable re-used for design purposes, to enhance beach amenity.

It should be noted that the tiered vertical seawall with promenade may potentially be located a further 3 m landward by providing maintenance access via the promenade. Unsuitable and ad-hoc materials present on the beach for this option are to be encapsulated under the promenade structure with emergency rock bag protection present on the beach re-used in the design landward of the promenade.

Table 4.3: Concept design indicative footprint areas

Concept design option	Footprint width (m)	Approx. Total footprint area (m^2)	Footprint area relative to previous 1998/2004 design (%)	Private Land (%)	Public Land (%)
1. Basalt Armour Rock Revetment	22.6	30200	+24%	23%	77%
2. Sandstone Armour Rock Revetment	23.4	31200	+29%	22%	78%
3. Vertical Seawall	0.9	1200	-95%	77%	23%
4. Vertical Seawall with Rock Toe	4.9	6600	-73%	62%	38%
5. Vertical Seawall with Promenade	7.0	9400	-61%	53%	47%
Previous 1998/2004 Seabee Design	*16.9 (south)* *18.4 (north)*	*24250*	-	*28%*	*72%*



NSW GOVERNMENT

ADOPTED PRELIMINARY CREST (REAR OF STRUCTURE) ALIGNMENT AND INDICATIVE CONCEPT DESIGN FOOTPRINTS SOUTHERN END.

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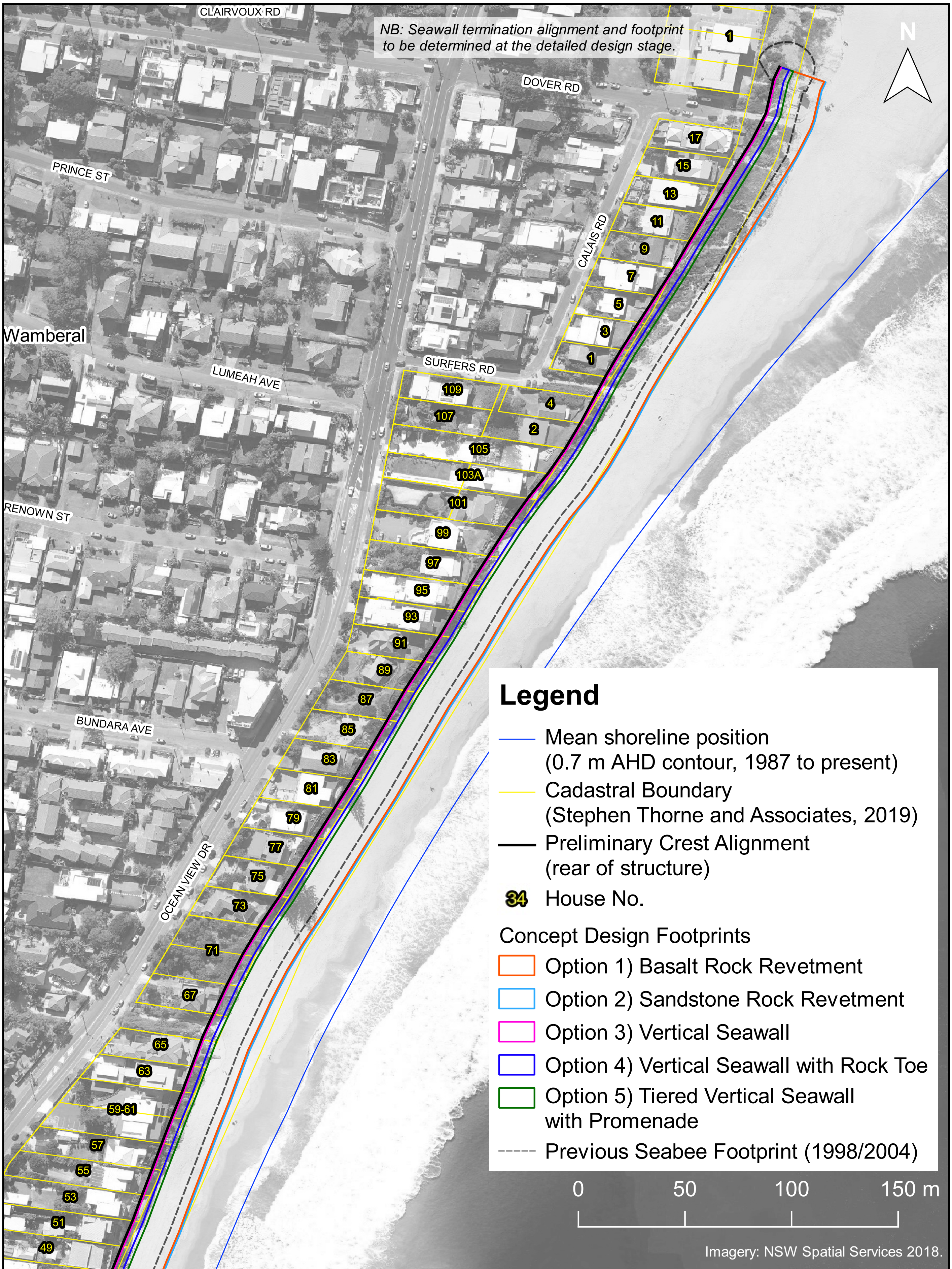
Figure

4.5

Figure 4.5.pdf



NSW GOVERNMENT

ADOPTED PRELIMINARY CREST (REAR OF STRUCTURE) ALIGNMENT AND INDICATIVE CONCEPT DESIGN FOOTPRINTS NORTHERN END.

Manly Hydraulics Laboratory

Report MHL2780

Figure

4.6

Figure 4.6.pdf

4.5 Seawall termination considerations

The proposed terminal protection structure extends between Wamberal and Terrigal Lagoons, approximately 1360 m in length. The design of the termination points at each of the lagoon ends, may differ depending on the final design selected at each of the structure ends, with design specifics of termination points to be undertaken during detailed design.

Potential end erosion impacts of the proposed seawall are assessed in the *Stage 2 Coastal Protection Amenity Assessment (MHL2779, 2021)* and are expected to be limited in extent and duration, and unlikely to affect other developed areas along the beach given the proposed contiguous structure extending from Terrigal Lagoon to Wamberal Lagoon. Termination of the structure at either end will transition landward of the active beach region, with minimal end erosion effects expected for vertical seawall options (Options 3 to 5). Higher encroachment of the rock revetment structures (Options 1 and 2) in the active beach at the southern end may result in slightly higher sand losses during rare storm erosion events that expose the seawall end to wave action. This region is also governed by dynamic lagoon entrance processes, a rocky backshore to the south and the Ocean View Dr Bridge constriction to the west such that traditional end erosion estimates are not applicable. The proposed periodic maintenance nourishment would further limit this effect (*Stage 4 report*). End erosion impacts are discussed in more detail in the *Stage 2 Coastal Protection Amenity Assessment (MHL2779, 2021).*

This section provides a brief review of the former 1998 Seabee termination design (Couriel et al., 1998) and outlines seawall termination design considerations at the Terrigal and Wamberal Lagoon ends to be further considered as part of detailed design.

4.5.1 South end: Terrigal Lagoon

The former 1998 termination design (Couriel et al., 1998) at the south end near Terrigal Lagoon is shown in Figure 4.7. This consisted of a vertical sheet pile structure running along the southern end of the structure and extending 11.2 m landward into the property of 1 Pacific St. The crest of the sheet pile structure transitions from +4.0 m AHD at its most landward extent to +6.0 m AHD at the transition with the Seabee armoured structure. The structure was cantilevered with a toe level between -8.0 to -10.0 m AHD, ground anchors and a concrete capping beam.

Due to the lowering of natural ground levels in the area, the vertical sheet pile structure was designed to be significantly overtopped and/or outflanked by storm waves. The vertical sheet pile design was selected in consultation with the landowner and Council, preferred over a rock rubble structure due to its smaller footprint and lower impact on existing pine trees situated in the area. A site-specific geotechnical investigation was also conducted in July 1998 (PSM, 1998) to develop the design.

The detailed design of the seawall termination should examine structural stability during rare to extreme flood events as well as design coastal events and potential impacts on lagoon entrance processes. A preliminary comparison to recent lagoon flood extent mapping from the Coastal Lagoons Catchment Overland Flood Study (MHL2590, 2020) indicates the former 1998 termination design is likely to have low potential impact on adjacent lagoon entrance processes given its location outside the typical entrance channel region and extreme floodwater extents. However further work is required to assess any potential stability issues in the likelihood of entrance scour occurring to the south-west of structure due to flooding.

In comparison to the former 1998 termination design, any proposals to extend structures further to the west (e.g., adjoining Pacific St and/or Terrigal Lagoon bridge) situated in low lying areas on the northern channel embankment are beyond the scope of the present coastal protection assessment and would require further hydraulic and flood impact assessment as part of detailed design to determine potential impacts on lagoon entrance processes and flooding.

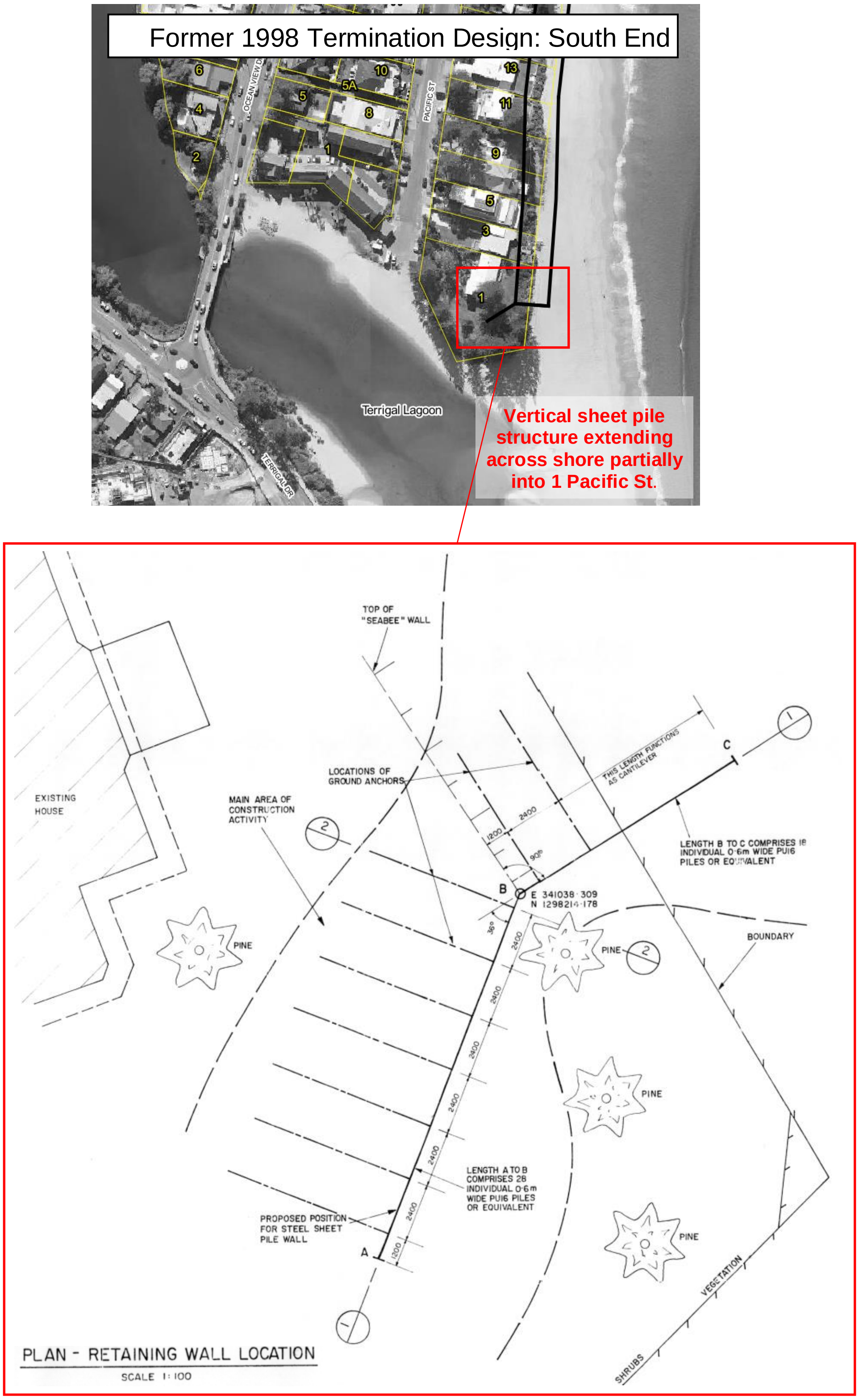


Figure 4.7: Former 1998 termination design south end at Terrigal Lagoon (Couriel et al. 1998)

4.5.2 North end: Wamberal Lagoon

The former 1998 termination design (Couriel et al., 1998) at the north end near Wamberal Lagoon is shown in Figure 4.8. This consisted of a buried and vegetated basalt rock rubble roundhead situated east of the present Wamberal Surf Life Saving Club (SLSC) building. The structure was designed with a crest level +6.0 m AHD, toe level of -1.0 m AHD, slope of 1V:1.5H with berm apron on its seaward side.

The structure was designed to protect the northern end of the proposed Seabee seawall and former Wamberal SLSC building from coastal erosion hazards. It is noted that since the former 1998 seawall design was undertaken, the surf club has been rebuilt with a larger building footprint extending north toward the surf club carpark. The former 1998 termination design now only partially protects the new surf club building from the coastal erosion hazard and should be reviewed to extend slightly north in order to provide more adequate protection to Wamberal SLSC. The design should also be reviewed to comply with present-day design standards including sea level rise considerations outlined in this report.

Should Council wish to extend protection works further north up to the Wamberal Lagoon entrance, this would require further hydraulic and flood impact assessment as part of detailed design to assess potential impacts on lagoon entrance processes and flooding.

Depending on the adopted final seawall design for this northern area, the termination design may also consider a vertical structure with a smaller footprint/encroachment and less construction impacts on vegetated dunes areas compared to a rock rubble roundhead design

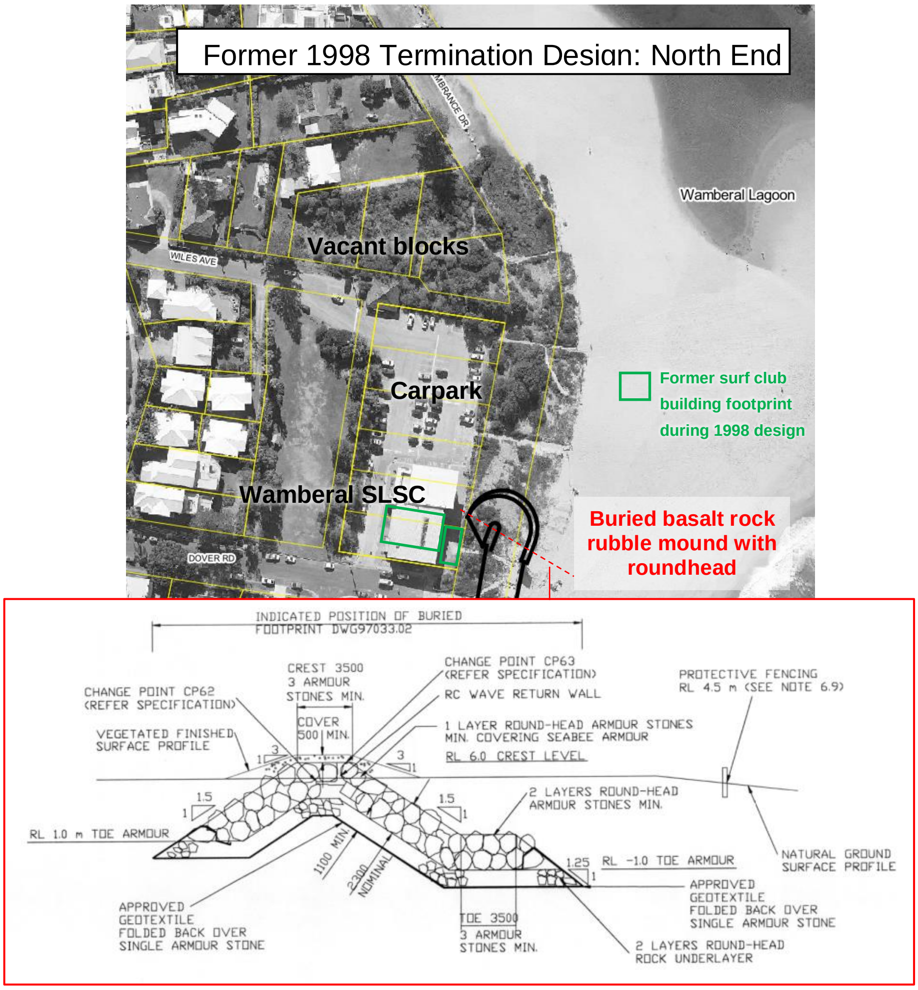


Figure 4.8: Former 1998 termination design north end at Wamberal Lagoon (Couriel et al. 1998)

4.6 Further alignment considerations

Refinement of the final design alignment to be determined as part of detailed design should include:

- Considerations outlined in the current section.
- Beach amenity impact assessment findings undertaken in Stage 2 reporting.
- A proposed future nourishment program to maintain a desired beach width for amenity and public use purposes addressed in Stage 4 reporting.
- Setback requirements for wave overtopping with design crest level refinements. Physical modelling is likely to provide value in determining accurate wave overtopping estimates for design crest level and setback. For example physical modelling undertaken as part of the previous 1998 seawall design was used to refine the design of the wave return to significantly reduce overtopping rates by 86% and provided a cost-effective design tool to review other aspects of seawall performance (Turner and Couriel, 1997). This is particularly important for the promenade design where the promenade level and structural details can be significantly refined to accommodate high wave over topping rates during design storm conditions without damage.
- Detailed property by property assessment including implications of alignment on other structures such as decks, patios, beach access, fences, walls, awnings located in the structure footprint and required setback region for wave overtopping and maintenance.
- Refined cut and fill volumes for the proposed structure including refined estimates of existing rock protection in foredune substrate.
- Potential implications on alignment of localised bedrock outcrops.
- Detailed plans to remove or reuse existing ad-hoc material and protection works on beach.
- Detailed design of termination points at Lagoon ends to minimise encroachment and impact on both coastal and lagoon entrance processes.
- Consultation with private property owners and the broader community to find agreement on a preferred seawall concept design option and alignment.

5 Comparison of options

5.1 General

A total of five seawall concept options were proposed for Wamberal Beach in Section 3, including a basalt rock revetment (option 1), sandstone rock revetment (option 2), vertical seawall (option 3), vertical seawall with rock toe (option 4) and a tiered vertical seawall with promenade (option 5). This section provides preliminary cost estimates of these options including total capital costs and estimated ongoing costings. Preliminary costings are to be treated as indicative and are for the purpose of comparing the different seawall concept design options. Costings will be refined as part of detailed design with refinement of crest levels and other design aspects.

5.2 Preliminary cost estimates

Preliminary cost estimates of seawall conception design options were calculated based on the following sources:

- Tender information and previous design reports from similar projects undertaken and/or available to the project team.
- Cost estimates from Rawlinsons (2015) "Australian Construction Handbook – 2015" indexed to present day values.

The following items were considered as part of preliminary costings:

- Construction periods of 18 months for rock revetments (option 1 and 2), 12 months for the vertical seawall options (option 3 and 4) and 18 months for the tiered vertical seawall with promenade (option 5).
- Supervision and survey including supervising engineer, engineer/surveyor, engineer/surveyor's assistant and foreperson.
- Excavation volume estimates based on topographic elevations of Wamberal Beach from georeferenced drone surveys undertaken on 17 September 2020. Excavation costs are predominantly for clean sand with some rubble with dewatering at lower elevations.
- Concrete wave return wall based on design and costings from Couriel et al. (1998) – including supply and placement of concrete, reinforcement and formwork.
- Site establishment costs.
- Further geotechnical investigation including borehole and seismic investigation.
- Removal and stockpiling of recently placed emergency rock protection accounting for potential replacement/reuse where suitable for design purposes.
- Other site preparation works including removal and waste disposal of existing concrete and timber structures and ad-hoc fill and material present on beach.
- Basalt and sandstone rock armour & underlay supply and install with estimated differences due to haulage. Preliminary cost estimates for sandstone rock armour have assumed supply from local quarry sources. Should local sandstone sources be deemed not suitable, supply from regional quarries is expected to increase total capital costs for Option 2 by approximately 20%. It should be noted that final design and costing of a rock revetment structure is subject to rock availably at the time of tendering.

- Other components as relevant to specific concept designs including concrete panels, H-columns, concrete piling, capping beam and sheet pile toe protection.
- Capital cost contingency of +20% for all concept designs covering aspects such as environmental approvals and removal of potential hazardous materials in existing beach fill.
- Ongoing maintenance cost for potential damage to non-rigid rock revetment structures was estimated at 1.0% of total capital costs Gordon (1989). Rigid vertical walls are to be designed for no structural damage under design conditions. However following major storm events, a condition inspection of all coastal protection structures is advised with cleaning and removal of wind-blown sand on promenade areas, estimated as up to 0.2% of total capital costs per annum.

Sand nourishment to maintain beach width amenity including requirements and sources with preliminary unit cost estimates are examined in the accompanying *Stage 4 Sand Nourishment Investigation (MHL2795, 2021)* report and are not included in the preliminary cost estimates in Table 5.1*.* The Stage 4 report documents periodic nourishment requirements to mitigate impacts of underlying recession and future sea level rise recession over a 50-year design period.

Preliminary cost estimates for each seawall concept designs are provided in Table 5.1 including indicative total capital costs and structure maintenance costs. Total capital costs are expressed as a total cost, cost rate per linear meter and for average property frontage of 17 m. Preliminary cost estimates for total capital works range from approximately $25.0M for the more conventional sandstone rock revetment (option 2) to $40.1M for the tiered vertical seawall with foreshore promenade (option 5).

Table 5.1: Summary of preliminary cost estimates of seawall options

Option	Total capital cost ($)	Linear cost rate ($/m)	Typical cost for 17m frontage ($)	Maintenance costs ($/year) [b]
1. Basalt rock revetment	$26,540,000	$19,500	$332,000	$265,400
2. Sandstone rock revetment	$24,990,000 [a]	$18,400	$312,000	$249,900
3. Vertical seawall	$34,010,000	$25,000	$425,000	$34,000
4. Vertical seawall with rock toe	$34,660,000	$25,500	$433,000	$34,700
5. Vertical seawall with promenade	$40,100,000	$29,500	$501,000	$60,100

[a] Preliminary cost estimates for sandstone rock armour have assumed supply from local quarry sources. Should local sandstone sources be deemed not suitable, supply from regional quarries is expected to increase total capital costs for Option 2 by approximately 20%.

[b] Does not include costs for periodic sand nourishment to maintain beach amenity. Please refer to *Stage 4 Sand Nourishment Investigation (MHL2795, 2021)* for sand nourishment requirements to maintain beach amenity including potential sources with preliminary unit cost estimates.

It is noted that the previous Seabee seawall design was costed at $8.2 million in 2004 (MHL, 2004), equivalent to a present value $15.4M assuming an indexing rate of 4% per annum. Preliminary costings for the Seabee option have not been considered in the present study and comparisons should be treated with caution without a more detailed re-evaluation of present-day cost estimates for the former Seabee design with up-to-date information. A comparison of the relative advantages and disadvantages of each concept design option as well as the previous Seabee seawall design (MHL, 2004) for the sake of completeness is provided in Section 5.3.

More recently the Gosford Beaches Coastal Zone Management Study and Plan (CZMP, 2017) estimated costs of $13 million to $15 million for construction of terminal protection at Wamberal Beach based on an indicative rate of $10,000 per linear metre of protection works. These costings were developed for comparative purposes of management options in the CZMP and did not include conceptual design of a terminal protection structure and associated construction preliminary costs. Relatively higher costs in the present study are likely due to these factors with additional capital works items considered including concept design component costs, removal of existing ad-hoc material and rock protection (reused where suitable) on beach, earthworks, wave return structure costs, site establishment and preparation, supervision/surveying and further geotechnical investigations.

5.3 Discussion

5.3.1 Advantages and disadvantages

Advantages and disadvantages of each seawall concept design option are listed in Table 5.2. The rock revetment seawall options are a common, more conventional and typically lower cost option for coastal protection. The sandstone rock armour (option 2) is slightly cheaper (locally sourced) compared to the basalt (option 1), is typically more aesthetically appealing but tends to be less durable than basalt in marine environments. Being non-rigid structures, the rock revetment options are designed to handle a certain degree of initial damage (few displaced units) during a major storm event requiring ongoing maintenance costs. Rock revetments can be adapted to sea level rise through the topping up with primary armour with suitably sized armour units. The rock revetment options, even when located as far landward as possible, are considered to have the highest environmental impact given their large footprint and encroachment into the active beach profile. The sandstone revetment is slightly larger in footprint compared to the basalt revetment due to differences in densities between the two types of rock armour, with the sandstone revetment requiring a larger volume of rock to achieve a similar level of stability as the basalt. After storms, when the beach is eroded these options will impede more frequently and for longer periods of time on alongshore beach access and width available for beach users (Couriel et al., 2020). Due to a more seaward encroachment, these options are likely to interact with wave and swash zone processes more frequently with higher potential impact on the beach profile during storms when exposed to wave activity. Rock revetments can also be buried. However, this sand covering is likely to be eroded during storm events when exposed to wave activity and require ongoing nourishment or beach scraping to keep the structure buried with time, particularly for narrower sections of the beach where more frequent exposure to wave activity is likely.

Comparatively, vertical seawall options when set to the same crest alignment as a rock seawall (aligned as far landward as possible within the physical constraints of the site addressed in Section 4), allow for the greatest room seaward for natural beach processes to occur with the least interaction with and encroachment of hard structures. This also allows for a maximum usable beach width, which after a storm during beach recovery, beach width more quickly returns to provide alongshore access than sloped revetment structures with greater encroachment. As such, provided a landward alignment is adopted, the vertical seawall (option 3) is considered to have the least direct impact on natural beach processes of the options and available beach width, followed by the vertical seawall with rock toe (option 4) and then the tiered promenade option (option 5). It is advised that any existing ad-hoc material and rock protection works seaward of the proposed seawall would have to be removed, or where suitable re-used for design purposes, to enhance beach amenity. Further quantification of the relative impacts on beach amenity of each seawall concept option is provided in the *Stage 2 Coastal Protection Amenity Assessment*.

Vertical seawalls however can be quite visually imposing following storm erosion and cause public safety concerns with large vertical relief. When located in an established foredune (particularly in the northern part of the site) this is likely to be buried most of the time becoming more visible toward the south where the beach is narrower. Options to improve the aesthetics of vertical seawalls include sandstone architectural facing or sand coloured concrete options. The tiered vertical seawall with promenade option reduces public safety concerns and adverse visual impacts associated with the larger vertical relief of the other vertical seawall options.

While the concept designs have accounted for potential sea level rise to 2070, further adaptation of vertical structures can include extension of H columns above design crest level or using stainless steel (or composite) promenade handrail posts to have options to raise the crest height in the future. Added scour protection can also be considered either by adding sheet pile or rock armour at the toe. In contrast, adaptation of rock rubble structures in narrow or restricted foreshore areas will present significantly exacerbated impacts on coastal processes and beach amenity due to 1.5 times seaward encroachment for every increment of rise in response to sea level rise.

Table 5.2: Relative advantages and disadvantages of seawall design options

Option	Advantages	Disadvantages
Option 1: Basalt rock revetment	Conventional Lower cost Non-rigid structure Adaptable to sea level rise	Higher maintenance costs High environmental and social impacts due to wide footprint, including: - Encroachment on usable beach area - Interaction with coastal processes (will increase with sea level rise) - Impede beach access after large storms - Post storm public safety risks (access constraints) High construction impacts (long rock haul/material) Increased risk of more frequent maintenance nourishment due to high beach encroachment and ongoing nourishment costs to keep buried
Option 2: Sandstone rock revetment	Conventional Lower cost Non-rigid structure Adaptable to sea level rise Aesthetic sandstone appeal	Higher maintenance costs High environmental and social impacts due to wide footprint, including: - Encroachment on usable beach area - Interaction with coastal processes (will increase with sea level rise) - Impeded beach access after large storms - Post-storm public safety risks (access constraints) High construction impacts (material) Increased risk of more frequent maintenance nourishment due to high beach encroachment and ongoing nourishment costs to keep buried Sandstone rock armour strength and durability concerns Local sandstone supply limitations
Option 3: Vertical seawall	Lowest environmental and social impacts, due to small footprint, noting: - Low encroachment on usable beach area - Low encroachment on natural beach processes Low construction impacts Opportunities for architectural finishes or artworks to enhance aesthetics. Adaptable to sea level rise	Poor aesthetics of large vertical wall Public safety – vertical drop after erosion Higher reflected wave energy during storms though with limited temporal and spatial extent [a] Potential for graffiti of seawall face

Option 4: Vertical seawall with rock toe	Low environmental and social impacts, due to small footprint, noting: - Relatively low encroachment on usable beach area - Relatively low encroachment on natural beach processes Non-rigid toe design for scour protection Opportunities for architectural finishes or artworks to enhance aesthetics. Adaptable to sea level rise	Poor aesthetics of large vertical wall Public safety – vertical drop after erosion Higher reflected wave energy during storms though with limited temporal and spatial extent [a] Potential for graffiti of seawall face
Option 5: Vertical seawall with promenade	Enhanced foreshore access and amenity Maintenance corridor access via promenade Adaptable to sea level rise Maintained foreshore access after storms (when beach is eroded) Wheelchair accessible Broader community value (i.e. coastal walk) Enhanced foreshore usability Potential tourism drawcard (economic benefits) Enhanced aesthetics/public safety for vertical seawall option Smaller footprint/encroachment (and resulting impacts) than revetment Reuse of existing rock protection works where suitable Opportunity to contain waste material currently embedded in foredune. Opportunities for landscape design along promenade, architectural finishes, or artworks to enhance aesthetics.	Higher cost Requires proactive safety management during large storms Potential privacy issues for low-lying beachfront properties close to scarp Design more complex and less conventional Relatively higher reflected wave energy during storms though with limited temporal and spatial extent [a] Potential for graffiti of seawall face Land tenure issues associated with public promenade areas located on private land
Former Seabee (1998) Design	Potentially lower cost alternative compared to options in present study although subject to current market forces and toe re-design Former detailed design and EIS already completed (albeit no longer valid) Formerly accepted by the community and alignment adopted with agreement of beachfront homeowners (albeit some time ago)	Structural stability highly dependent on efficacy of rigid toe design not included as part of the previous design (flexible gabion mattress toe) Poor tolerance to settlement/movement with potential high maintenance costs Higher wave runup during large storms Not as common in high wave climate settings like Wamberal Beach Relatively high environmental and social impacts due to footprint of sloped structure encroaching on beach and coastal processes. Impeded beach access after storms Ongoing nourishment costs to keep buried Change of beachfront homeowners, property values and community values since previous Seabee design

[a] *Limited to occurrences when the beach is eroded by major storm waves and sections of the seawall are exposed. This effect is mitigated by a more landward cross-shore position of a vertical seawall within the active beach profile and presence of an accreted beach fronting the seawall for most of the time.*

On the other hand, the tiered vertical seawall with promenade (option 5) has a slightly larger footprint than the other vertical options but relatively smaller footprint when compared to the larger rock revetment options. Alternatively to other options, impacts on beach width amenity for the tiered vertical seawall are alleviated via the provision of enhanced foreshore access along the structure promenade. When the beach is eroded following a storm, and other options may inhibit access along the beach, this option maintains a level of access and foreshore usability, allowing beach users to continue to walk along the foreshore even while the beach is narrow after storms. A walkway along the beachside might also aid community pedestrian access between the northern and southern ends of Wamberal, currently connected by a narrow and non-continuous roadside pathway along Ocean View Drive.

Although associated with the highest capital cost, the tiered vertical wall with promenade option is considered to provide the greatest value to the broader community via enhanced access along the beach. Beachfront promenades, both locally at Terrigal Beach and other numerous locations elsewhere (e.g. Manly Beach, Dee Why Beach, Newcastle Beach, Blue Mile Tramway Wollongong) have provided enhanced foreshore amenity and valued public recreational areas for the local community and regional tourism. The *Stage 6 Cost-Benefit Analysis and Distributional Analysis* further explores relative cost and benefit of seawall concept options in monetary terms.

During large storms with high wave overtopping, the promenade would require temporary closure to the public for safety reasons aided by automatic signage and data-driven alerts. The tiered vertical seawall with promenade is considered to be the more complex of the concept design options required to accommodate potentially high wave overtopping onto the promenade during large storms and design condition. The tiered vertical seawall with promenade (option 5) at Wamberal aims to balance community amenity with landowner privacy. The promenade level should be refined during detailed design to prevent visual intrusion of privacy to beach front residents and provide an access point to the public beach area for beachfront properties.

5.3.2 Comparison with former 1998 Seabee seawall design

For completeness, advantages and disadvantages of the former Seabee design are also provided in Table 5.2. The primary advantage of this design is that it had previously been through detailed design and an EIS, albeit a long time ago in 2004 (MHL, 2003). It was also considered to be a relatively lower cost option than concept designs developed previously, however those earlier cost estimates were never market tested and modern day costs are difficult to estimate for the Seabees design as there are no recent examples of large concrete Seabee unit seawalls constructed in Australia. Despite the earlier potential advantages, it has now been 17 years since the EIS for the former design was completed and the area has also seen substantial changes in beachfront home ownership, property values, community values, as well as engineering standards and practice. Present community and homeowner preferences of the seawall make-up are not known and will be re-established by Council from current studies and consultation.

From engineering experience over recent decades, it has been found that the structural stability of the Seabee design (particularly in high wave energy environments like Wamberal Beach) is highly dependent on the efficacy of a rigid toe design due to a low tolerance to settlement and movement. For similar reasons significant expenses were required to fix the damaged toe of the Seabee seawall at Prince St, Cronulla following a series of storms in June 2007. The former Seabee seawall design for Wamberal did not include a rigid toe and should a Seabee design be re-progressed to detailed design, it will require re-design of the toe in the form of capping beam and associated support structures not included in the former design with significant cost implications.

Other disadvantages listed in Table 5.2 include higher wave overtopping than other options and ongoing nourishment costs to keep it buried and vegetated with reoccurring wave exposure (given the relatively wide footprint; being similar to rock). Further consideration of composite designs that incorporate Seabee armour protection with an improved toe design could be developed as part of detailed design should there be ongoing support for this option. It is noted that there are multiple proprietary and alternative designs that may emerge through a tender process following detailed design. Seabee units may for example provide an alternative option to be incorporated into a tiered vertical seawall design for the area landward of the promenade where suitable.

5.3.3 Preferred option selection

A preferred option will be determined in consultation with community, stakeholder groups and Council and also consider outcomes of a beach amenity impact assessment (Stage 2), sand nourishment investigation (Stage 4) and cost-benefit analysis (Stage 6) currently being undertaken in parallel with the present study. It is noted that a preferred option may comprise different concept design options along different sections of the beach to be determined in detailed design stages. As an outcome of consultation with community and stakeholder groups, Council may wish to consider the development for a Master Plan for the Wamberal Beach foreshore that details the alongshore values and uses of the foreshore to inform the preferred option design.

6 Conclusions and recommendations

The study has developed a total of five seawall concept design options for Wamberal Beach as part of Stage 3 of the Wamberal Coastal Protection Assessment, with concept design objectives outlined in Section 3.1.

A review of relevant engineering standards and design parameter selection for the development of concept designs has been provided. A total of five alternative seawall concept designs have been developed with cross-section drawings and footprint mapping for Wamberal Beach comprising:

Option 1: Basalt Rock Revetment
Option 2: Sandstone Rock Revetment
Option 3: Vertical Seawall
Option 4: Vertical Seawall with Rock Toe
Option 5: Tiered Vertical Seawall with Promenade

Preliminary alignment of the concept designs has been proposed following a re-evaluation of the former crest (rear of structure) alignment proposed by Couriel et al. (1998) with regard to:

- present day setback to existing buildings/structures for access
- alignment relative to the foredune erosion scarp present in the beach profile
- alignment relative to the characteristic (natural) shoreline curvature of Wamberal Beach based on measurements of beach width from 1987 to present.

The former alignment was assessed to be suitable for adoption in the present study with only minor changes at regions of stepped crest level changes in the former design.

The adopted crest (rear of structure) alignment has sought to keep the seawall located as far landward as practicable to minimise encroachment into the active beach profile and impacts on amenity, while maintaining uniformity of the previous design within the constraints of adjacent properties and setback requirements. With the adopted alignment (with all options having a common crest alignment and varying seaward toe locations), vertical wall options are considered to have the least impact on beach amenity and coastal processes considering their relatively small footprints compared with the sloped rock revetment designs which encroach further into the active beach profile, impeding more frequently on beach access, public amenity and coastal processes. A more detailed assessment of the impacts of each concept design on amenity and available beach width is provided in the Stage 2 report findings.

Preliminary cost estimates for total capital works and maintenance for each of the five concept design options are listed below:

Option 1: Basalt Rock Revetment $26.5M (equivalent to $19,500 per linear m).
Estimated annual maintenance cost of approximately $265,400 per year. [a]

Option 2: Sandstone Rock Revetment $25.0M (equivalent to $18,400 per linear m). [b]
Estimated annual maintenance cost of approximately $249,900 per year. [a]

---

[a] Maintenance costs give preliminary estimates of potential repairs to damaged rock armour (non-rigid structures), post-storm condition inspections, and promenade maintenance for option 5 (removal of wind-blown sand and public safety control measures during storms).
[b] Capital and maintenance costs for the sandstone rock revetment may vary depending on the source and durability of rock armour selected during detailed design.

Option 3: Vertical Seawall $34.0M (equivalent to $25,000 per linear m).
Estimated annual maintenance cost of approximately $34,000 per year. [a]

Option 4: Vertical Seawall with Rock Toe: $34.7M (equivalent to $25,500 per linear m).
Estimated annual maintenance cost of approximately $34,700 per year. [a]

Option 5: Tiered Vertical Seawall with Promenade: $40.1M (equivalent to $29,500 per linear m).
Estimated annual maintenance cost of approximately $60,100 per year.

Preliminary costings for capital works include allowances for structural components supply & install, wave return wall construction, removal of existing ad-hoc rock protection on beach (reused where suitable), earthworks, site establishment and preparation, supervision/surveying, further geotechnical investigations as well as contingency covering environmental approvals and removal of potential hazardous materials in existing beach fill. Seawall construction is also to be coupled with the provision of sand nourishment for all options as recommended in the Gosford Beaches CZMP (2017). Costs for sand nourishment to maintain beach width amenity fronting the seawall are addressed separately in *Stage 4: Sand Nourishment Investigation (MHL2795, 2021)*.

Preliminary cost estimates for total capital works range from approximately $25.0M for the more conventional sandstone rock revetment (option 2) to $40.1M for the tiered vertical seawall with foreshore promenade (option 5). Although the most capital cost intensive structure, the tiered vertical seawall with promenade (option 5) is considered to provide the greatest value to the broader community via enhanced access and foreshore amenity.

A preferred option will be determined following consultation with community, stakeholder groups and Council and also consider outcomes of the beach amenity impact assessment (Stage 2), sand nourishment investigation (Stage 4), cost-benefit analysis and distributional analysis (Stage 6) currently being undertaken in parallel with the present study. It is noted that a preferred option may comprise different concept design options along different sections of the beach as may be determined from consultation and subsequent detailed design stages. As an outcome of consultation with community and stakeholder groups, Council may wish to consider the development for a Master Plan for the Wamberal Beach foreshore that details the alongshore values and uses of the foreshore to inform the preferred option design.

Wherever practicable and suitable, existing ad-hoc and emergency rock protection present on the beach will be utilised in the proposed new seawall, with details depending on the final design of the preferred option. For example, existing emergency rock bag protection may form suitable fill material landward of the promenade under option 5. Where not suitable, allowance has been made for any existing ad-hoc material and emergency rock toe protection works seaward of the proposed new seawall to be removed to enhance beach amenity.

Any existing rock protection removed with construction should be replaced by sand nourishment wherever possible to extend the level of the natural beach berm or foredune seaward of the seawall. All sand excavated during the construction of the proposed seawall should be screened (to remove any oversized materials) and placed seaward of the works with any necessary fill landward of the seawall comprised of the separated materials (if suitable) and/or suitable clean fill that would be imported to the site. This will maximise the amount of sand added to the beach area as a result of the works.

As part of the concept design development, a number of issues have been identified that will require further consideration as part of detailed design of a proposed option. These include (but are not limited to):

- Estimates of bearing capacity of any underlying material remaining in the dune substrate.
- Review of the adequacy of available geotechnical knowledge of the study area for detailed design which is dependent on the preferred seawall concept progressed to detailed design.
- Refined design crest (and promenade if adopted) levels with alongshore variability in wave exposure, overtopping and characteristic foredune/berm elevations along the structure. Three-dimensional physical modelling should be considered to refine assumptions and reduce costs as part of detailed design including refining structural dimension, wave overtopping estimates and crest levels along the structure considering safety aspects for people and property. This is particularly important for the promenade design where the promenade level and structural details can be significantly refined to accommodate high wave over topping rates during design storm conditions without damage.
- Land tenure matters and detailed design refinements related to specific concept design options (described in this report) including refinement of adopted thickness and steel reinforcement of concrete panels, length of supporting piles, alongshore details of the seawall toe design, drainage design and promenade elevations.
- Detailed property by property assessment of alignment including buildings, decks, patios and other structures, as well as consultation with beachfront homeowners.
- Associated landscaping and design of beach access points within the structure.
- Environmental approvals including environmental impact assessment, crown licences and further geotechnical assessment.
- Detailed design of termination points at Wamberal and Terrigal Lagoon entrances.

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Appendix A Engineering design standards for Wamberal terminal coastal protection

21 September 2020

WRL Ref: WRL2020014 JTC LR20200921

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Dear Matt

Re: Engineering design standards for Wamberal terminal coastal protection

1. Introduction

Establishing the basis of design (BoD) is an essential step of any coastal engineering project. Ideally, the whole life cycle of a coastal engineering project from conception to decommissioning (if appropriate) should be considered during the planning and design phases. Technical aspects should be integrated together with social, environmental, economic and other factors (CIRIA, 2007).

With differing viewpoints on acceptable design probabilities for coastal structures in Australia, Gordon, Carley and Nielsen (2019) attempted to reconcile these. Potentially the most sensitive seawall design parameter, and least understood, is the adopted design scour level, as this determines the depth limited wave height that may reach a structure. A review of available methods for estimating this is provided in Carley et al. (2015).

This letter is restricted to the BoD for the concept design of coastal engineering structures, to assist in the comparison of options and project costings. Numerous standards also exist for the detailed design and specification of specific materials (if they are selected) e.g. standards for strength and durability testing of rock, standards for concrete. These are not covered in this work, but would be addressed in the detailed design and contract documentation phase of the project.

2. 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.

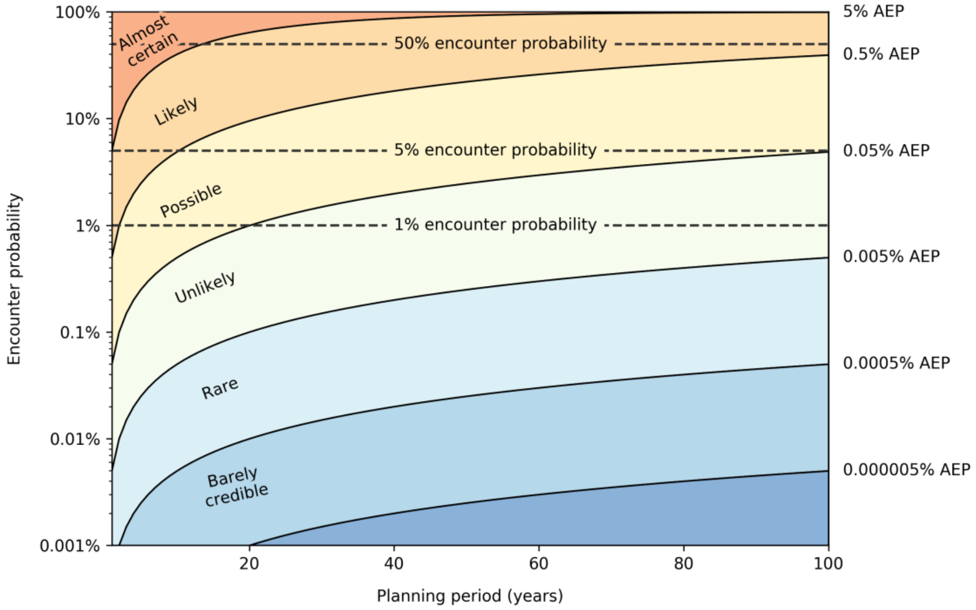
Table 2-1: Encounter Probability (Probability of Exceedance) for Given ARI and Project Life

		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 2-2: 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

Figure 2-1 shows qualitative descriptions of likelihood for a range of encounter probabilities and planning periods (design lives).



Note: Figure adapted from AGS, 2007

Figure 2-1: Likelihood descriptions of encounter probabilities over a 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 2-1), 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% 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.

3. Design working life

Establishing the design working life of a coastal structure is critical for determination of subsequent design parameters. The design life of coastal structures depends on their purpose.

Temporary structures, such as a geotextile groyne, might have a design working life of 2 to 10 years, while a rigid coastal structure, such as a concrete vertical seawall, might have a design working life of 50 to 100 years. It is also important to note that many coastal structures outlive their design working lives.

Some guidance on design working life is provided in literature (discussed in Section 4.3), however, the process remains somewhat subjective. It should be emphasised that establishing the design life is **not exclusively an engineering decision** and should involve numerous stakeholders.

4. Establishing the design event and acceptable risk of failure

4.1 Overview

Once the design working life of a structure has been established, it is prudent to select an appropriate level of design risk (encounter probability) and assign a design event (design waves and water levels). An annual probability of exceedance for significant wave height and still water level forms the design "event" or design conditions. The design event needs to be considered as a component of the overall risk within a project. Structures which are designed for a short/frequent ARI event, or which are retained in excess of their design life will incur substantial costs, which may be in the form of maintenance, repairs, consequential damage, reputational risk or political consequences. Structures which are designed for high/rare ARI events will have low maintenance costs and/or costs due to the risk of failure, but will involve high upfront capital costs. The economic viability of maintenance is strongly determined by the discount rate adopted for future expenditure. This is typically 3 to 10% in Australia, with 7% the default discount rate. It may range from 0 to 15%. This is illustrated in Figure 4-1.

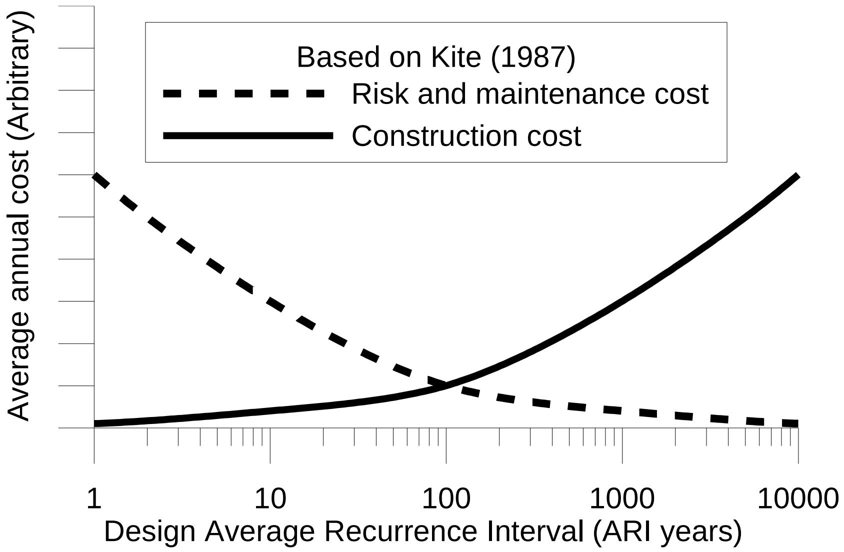


Figure 4-1: Balance between Risk, Maintenance and Capital Cost

The outcome of a successful integrated design should be a structure that delivers the required performance and which is (CIRIA, 2007):

- Robust;
- Easy to build and maintain;
- Socially and aesthetically acceptable;
- Cost-effective; and
- Produces the fewest negative impacts on it environment.

In practice, any project will have to achieve an appropriate balance between all of these requirements. Project economics generally aim to balance the **value** of the project, mainly dictated by functional performance and the impact on the environment, and **cost**, dictated by technical or engineering aspects and by construction.

4.2 Historical practice

Explicit formal guidance is not readily available for selection of an appropriate design event for flexible maritime structures. Conventional coastal engineering practice in Australia is to allocate a design ARI which may range from the design life of the project (e.g. a 1 year design life structure would use a minimum 1 year ARI design event) up to that suggested in Australian Standard AS 4997-2005. However, the most common engineering practice in Australia to date has been to use the 100 year ARI (1% AEP) event for design of "permanent" coastal structures. The 100 year ARI event is the generally accepted balance between risk and initial capital cost, however, specific projects need to be assessed individually (see below).

It is noteworthy that many of the coastal defences of the Netherlands are designed for a 10,000 year ARI (0.01% AEP) event, which has a 1% chance of being exceeded over 100 years (Delta Committee, 1962).

Climate change and sea level rise further complicate the calculation of encounter probability over the asset life.

4.3 Standards

4.3.1 Building Code of Australia

The acceptable likelihood or acceptable risk for private dwellings is considered in several documents, but well accepted or legislated values for coastal hazards are not presently available. However, the Building Code of Australia (2016) lists the following acceptable design probabilities for freestanding detached private houses (note that the design working life is undefined in the code itself but is defined in many feeder documents as 40-60 years):

- Water entry into building: 100 year ARI (1% AEP);
- Wind Load: 500 year ARI (0.2% AEP); and
- Earthquake load: 500 year ARI (0.2% AEP).

4.3.2 Australian Standard AS 4997-2005

Australian Standard (AS) 4997-2005 *Guidelines for the Design of Maritime Structures* recommends design wave heights based on the function and design life of the structure as reproduced in Table 4-1 Note that while this standard covers rigid maritime structures (e.g. wharves and concrete seawalls), it specifically excludes the design of flexible "coastal engineering structures such as rock armoured walls, groynes, etc." However, in the absence of any other relevant Australian Standard, it is commonly considered in the assessment of probability in contemporary Australian coastal engineering practice. AS 4997 recommends that the design water levels accompanying these waves should not be below Mean High Water Springs (MHWS).

Table 4-1: Annual Probability of Exceedance of Design Wave Events (source AS 4997-2005)

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

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

(b) Inferred by WRL

(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%.

4.3.3 International Standard ISO 21650:2007

ISO 21650:2007 "Actions from Waves and Currents on Coastal Structures" contains some guidance on design life and probability and provides the following commentary regarding a range of four (4) "safety classes" for coastal structures: "*Temporary and small coastal structures would belong to the very low safety class. Larger coastal structures such breakwaters in deep water and exposed seawalls protecting infrastructure would belong to the low safety class. Breakwaters protecting an LNG terminal or a power station would belong to the high safety class."* This guidance is summarised in Table 4-2.

ISO 21650:2007 quotes two tentative methods for specifying a probability of failure, namely the Spanish ROM 0.0 method and that of Burcharth (1999). Both of these methods provide a probability for "serviceability" (performance under commonly encountered conditions) and "limit state" (ultimate failure) which are shown in Table 4-2. ISO 21650:2007 only provides the extreme range for the probability of failure in Table 4-2, however, intermediate values using the Burcharth method are presented in Burcharth (2003). ISO 21650:2007 does not prescribe water level encounter probabilities to accompany these wave height encounter probabilities.

Table 4-2: Example of safety classes for coastal structures (ISO 21650:2007)

Safety Class	Consequence of Failure	Probability of Failure (Encounter Probability)	
		ROM 0.0	Burcharth (a)
Very low	No risk of human injury. Small environmental and economic consequences	Serviceability 20% Limit state 20%	Serviceability 40% Limit state 20%
Low	No risk of human injury. Some environmental and economic consequences	not provided	Serviceability 20% Limit state 10%
Normal	Risk of human injury and/or significant environmental pollution or high economic or political consequences	not provided	Serviceability 10% Limit state 5%
High	Risk of human injury and/or significant environmental pollution or very high economic or political consequences	Serviceability 7% Limit state 0.01%	Serviceability 5% Limit state 1%

(a) as quoted in Burcharth (2003)

ISO 21650:2007 suggests the following design working life of coastal structures:

- Temporary coastal structure: 1 to 5 years;
- Permanent coastal structure: 50 to 100 years.

Note that AS 4997-2005 makes no differentiation between "serviceability" and "limit state" design conditions.

4.3.4 Gordon et al (2019)

Synthesising the above information, Gordon et al. (2019) suggested the design life and design event ARI shown in Table 4-3: Design life and design event ARI for various asset categories (Gordon et al., 2019).

Table 4-3: Design life and design event ARI for various asset categories (Gordon et al., 2019)

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

5. Damage versus failure

5.1 Difference between damage and failure

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."

That is, *damage* does not necessarily equate to *failure*.

The most common reasons for the failure/damage of a coastal defence structure are (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 or exceedance of the design conditions;
- ***Construction failure***: this can be caused by unsuitable construction techniques or poorly suited construction materials in which the design capacity of the structure is not achieved;
- ***Deterioration failure***: this failure is the result of structure deterioration and lack of project maintenance such that the intended design capacity of the structure no longer prevails.

Design failure is generally characterised by a relatively large response (damage) that is generated by a minor increase in loading (wave forces).

5.2 Damage and failure for rock rubble structures

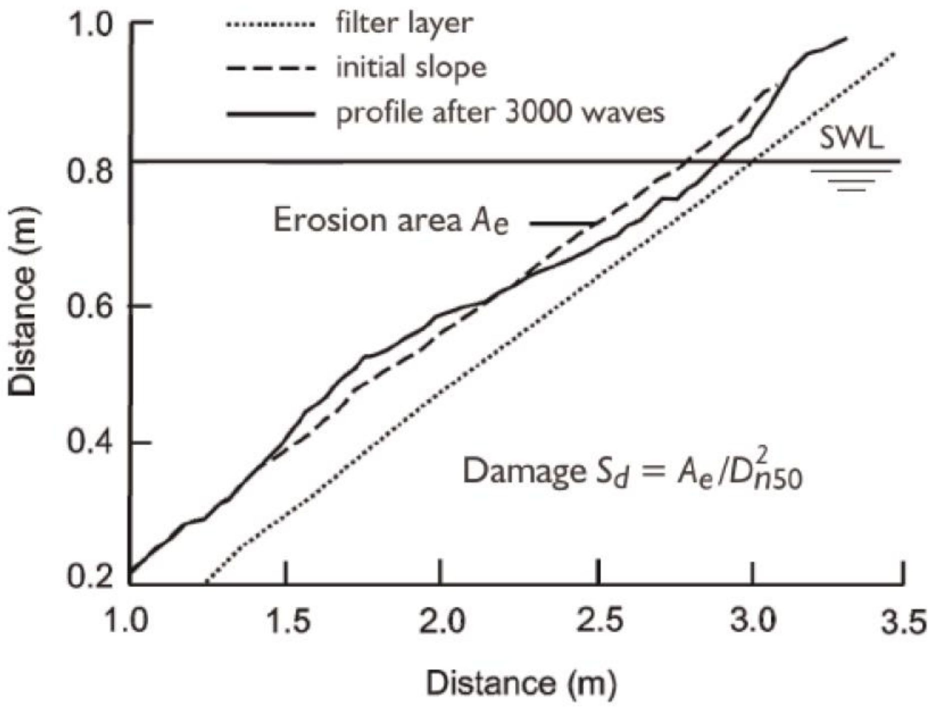
Many commonly used coastal structures, such as rubble mound breakwaters, are inherently flexible. That is, the structure can tolerate a reasonable amount of damage before failure occurs. Therefore it may be acceptable for some coastal structures to be designed to experience some degree of damage during the design working life.

In calculations for rock rubble breakwaters and seawalls, a damage level parameter Sd (Van der Meer in Delft, 1990) or a Hudson percent damage (SPM, 1977, 1984; USACE, 2006) is used to quantify the damage sustained by a structure.

The Hudson percent damage is defined as the proportion of rocks in a given region which move by more than one rock diameter (during the course of a storm).

The Van der Meer Sd damage level parameter is illustrated in Figure 5-1, where D_{n50} is the median armour size (cube side equivalent in metres). Photographs of various Sd values following physical modelling in a wave flume are shown in Figure 5-2. The approximate equivalence between the two methods of damage measurement is shown in Table 5-1.

Although a damage level of 0-5% is often used for design purposes, it may be feasible to apply higher values of 15-20% depending on the desired life of the structure (CIRIA, 2007). By considering initial damage and failure, two design criteria for rock damage/displacement may be adopted. That is, initial damage (serviceability) is assumed for a low ARI event. Failure (limit state) would occur at a high ARI. Ultimately, the adoption of a particular design event is dependent on the *acceptable risk.*



(Source: CIRIA, 2007)

Figure 5-1: Definition of Damage Level (S_d)

Table 5-1: Design values of S_d and equivalent percent damage
(for armour stone in a double layer with slope 1V:1.5H, USACE, 2006; CIRIA, 2007; Delft, 1990)

Damage designation	**Van der Meer S_d**	**Hudson Percent damage**	**Description**
No damage	>0 (a)	0	No units displaced
Initial damage	2 to 3	0 to 5%	Few units are displaced (c)
Intermediate damage	3 to 5	5 to 10% (b)	Units are displaced, but without causing exposure of the under or filter layers
Failure	8	$\geq$20% (b)	The underlayer or filter layer is exposed to direct wave attack

(a) Settlement may result in S_d>0 even for "no damage"

(b) For 1V:2H slope (USACE, 2006)

(c) Previous versions (SPM, 1984, 1977) defined this as "no damage"

Initial profile- No Damage

S_d=2

Initial profile- No Damage

S_d= 4

Initial profile- No Damage

S_d= 6

Notes: Photos are from physical modelling in a wave flume
Right hand panel illustrates rocks displaced by wave action in an extreme storm

Figure 5-2: Illustration of damage level (S_d)

6. Summary of engineering design standards

Based on consideration of AS 4997-2005 and ISO 21650:2007, **encounter probabilities of 5-40%** are suggested for most types of coastal structures but can be as low as 1% for high consequence situations such the Dutch dykes or nuclear power stations. However, it is again emphasised that establishing the acceptable risk of failure (encounter probability) is **not exclusively an engineering decision** and should involve numerous stakeholders.

The following values are suggested for a terminal protection structure at Wamberal:

Initial design life:	50 years
Initial damage for rubble structure:	100 to 200 year ARI
Failure for rubble structure or rigid structure:	500 to 2000 year ARI

Numerous standards also exist for the detailed design and specification of specific materials (if they are selected) e.g. standards for strength and durability testing of rock, standards for concrete. These would be considered in the detailed design and contract documentation phase of the project.

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

Yours sincerely,

Grantley Smith
Director, Industry Research

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Appendix B Geotechnical data review

B.1 Preamble

The geotechnical conditions of the study site are an important component in the design and assessment of foundations of a coastal protective structure. Building on the investigations of previous studies, the following sections provided an outline of the broader coastal geomorphology setting and review of geotechnical data available for the study site.

B.2 Coastal Geomorphology Setting

Terrigal-Wamberal Beach is a sandy embayment situated within the Central Coast sediment compartment. The coastal geomorphology and topography of the embayment is described by Hudson (1997) and MHL (2003) and illustrated in Figure B.1. The sandy embayment is classified as stationary-receded coastal barrier system containing two entrances at Wamberal and Terrigal Lagoons (Hudson, 1997). The embayment is composed of fine to medium grained quartz sand with a carbonate fraction deposited during the mid-Holocene with stabilising sea levels and backed by estuaries infilled with fluvial sediments (Thom and Roy, 1985). Dune elevations reaching typically up to +8 to +10 m AHD increase from south to north in the embayment with increasing exposure to the predominant south to south-easterly wave climate. Much larger transgressive cliff-top dunes reaching up to +100 m AHD are also present north of Wamberal Point and are believed to be deposited during earlier geological time (Hudson, 1997).

The embayment is bounded by interbedded sandstone and shale (Terrigal Formation) headlands at Terrigal in the south and Wamberal Point in the north. Rocky reefs are situated offshore of the embayment in water depths of 20 – 25 m (MHL, 2003). The offshore reefs are relatively shallow compared with those of neighbouring embayments, suggested to have a more pronounced impact on sediment transport processes by trapping offshore storm deposits and containing embayment sediment in a relatively closed system (NSW OEH, in draft ; MHL, 2003). Rock outcrops are also prevalent north of Wamberal Lagoon entrance with smaller outcrops located offshore of the Terrigal Lagoon entrance. Figure B. 2 shows a claystone outcrop temporarily exposed during storms, situated just above mean sea level on the beach in front of 73 -87 Ocean View Dr.

The entrances of Wamberal and Terrigal Lagoons act as sediment sinks in the embayment, intermittently closing with the infilling of marine sand and beach growth depending catchment rainfall and ocean wave conditions.

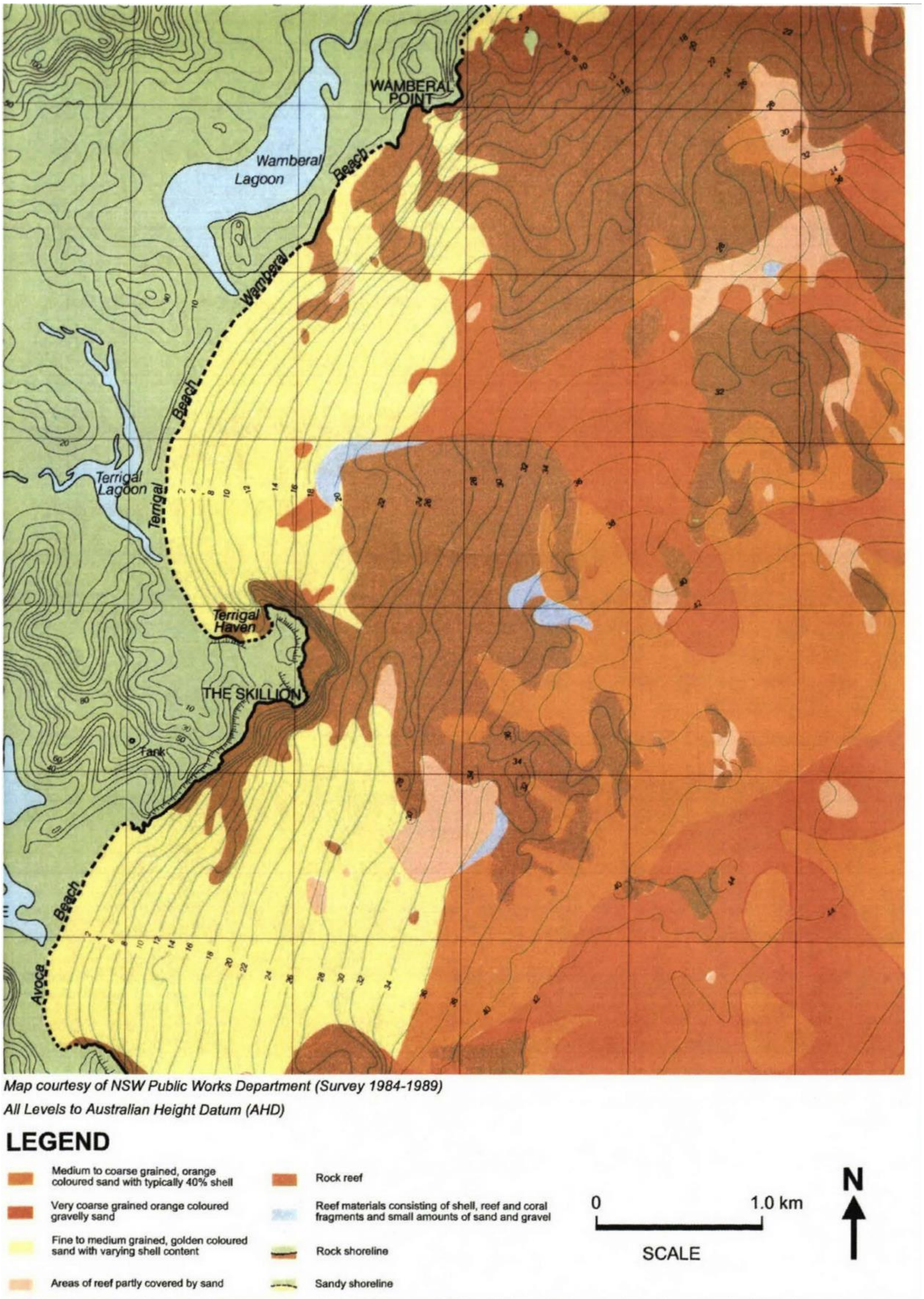


Figure B.1: Terrigal-Wamberal Beach coastal geomorphology. From PWD (1984).

Figure B. 2: Claystone outcrop located between 73-87 Ocean View Dr. Imagery from 19 Jul 2020 Drone Survey.

B.3 Geotechnical Data

Geotechnical studies of the study area have been undertaken by Hudson (1997) and as part of various development applications (DAs) for private land owners within the study area.

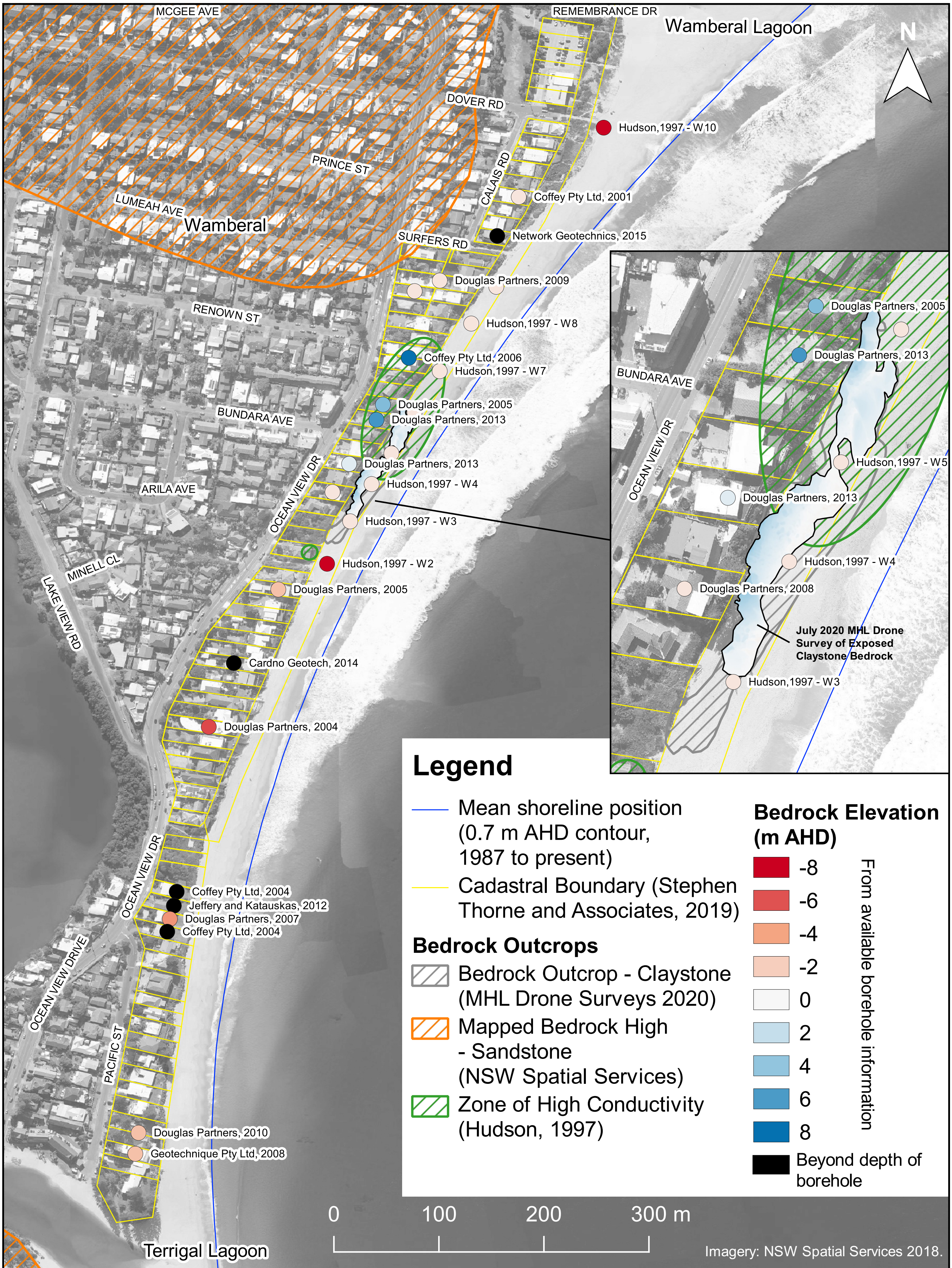
Hudson (1997) carried out geotechnical investigations at Avoca, Wamberal and Forresters Beach for Gosford City Council to identify physical constraints such as bedrock to inform estimates of dune recessions and coastal erosion hazard zones. The study included conductivity measurements along the beach and dune, and drilling samples in the beach face. Additionally, studies undertaken for private residents as part of development applications have typically used drilling samples in the incipient foredune underlying private lots (NSW OEH, in draft). Post-storm georeferenced drone surveys in 2020 undertaken by MHL for NSW Engineering Emergency Management also capture exposed bedrock units in certain sections of the beach.

The geotechnical information from these studies presented in Figure B.3 and described in Table B.1. Typical stratigraphy is characterised by:

- Underlying weathered sandstone; overlaid by
- Cohesive very stiff to hard silt and clay deposits of varying thickness including siltstone, claystone and ferruginous sandstone. Classified by Hudson (1997) as weathered to fresh bedrock basement (Pleistocene); overlaid by
- Fine to medium grained quartz sand (Holocene) with thin layers of gravelly sand and sandy clay typically near the underlying siltstone/claystone boundary.

Borehole stratigraphy from Hudson (1997) is presented in Figure B.4 and Figure B.5. Similar stratigraphy is described for the Wamberal barrier in the Gosford 1:25,000 Geological Map Sheet Series.

Typical depths to the siltstone/claystone unit have been found to vary along the study site as shown in Figure B.3 and Table B.1. In the south section of the study site (south of 73 Ocean View Dr, Figure B.3), this is situated between -2 to below -10 m AHD. In the mid-north of the site, a 400 m section of elevated siltstone/claystone is situated north of 73 Ocean View Dr with shallower depths of -2 to +1 m AHD. This unit is shown in Figure B. 2 and is temporarily exposed during erosion event. The claystone bedrock returns to lower depths in the north of the study site. The geotechnical data indicates that the foredune is predominantly unconsolidated quartz sand from the surface to below 0 m AHD other than a small region between Bundara Ave and Renown St where elevated siltstone/claystone of up to +6 to +8 m AHD has been identified.



SUMMARY OF AVAILABLE GEOTECHNICAL INFORMATION
WAMBERAL BEACH.

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Figure
B.1

Figure B.1.pdf

Table B.1: Borehole information from private property development applications. From (NSW OEH, in draft)

Property	Date	Geotech Firm	Details
Lot 73 DP13304 (7 Calais Rd)	30/05/2001	Coffey Pty Ltd	Bore logs indicate soil profile typically consists of medium dense Aeolian and marine sands to at least -1.5 m AHD (at BH 1), overlying very stiff to hard residual clays and weathered sandstone.
1 Calais Rd	25/01/2015	Network Geotechnics	2 Bore holes, deepest to approx. RL 3.5 m AHD where medium dense/dense marine sand encountered. Only marine sand encountered.
105 Ocean View Drive	6/04/2009	Douglas Partners	Loos to medium dense sand down to -0.6 m AHD. Stiff clay becoming hard from -0.6 to -1.3 m AHD with cemented bands. Weathered bedrock at depth of -1.9 m AHD.
103 Ocean View Drive	31/10/2007	Jeffery and Katauskas	Sand to 0 m AHD. Sandy clay zero to -1.5 m AHD, sandstone below -1.5 m AHD.
93 and 95 Ocean View Drive	20/03/2006	Coffey Pty Ltd	Sand from surface to RL 8.9 m AHD. Clay between RL 8.9 and 8.4 m AHD. Silty Clay between RL 8.4 and 8.4 m AHD. Ironstone banding at around 4.4 m AHD.
87 Ocean View Drive	1/04/2005	Douglas Partners	Sand above RL 6.0 m AHD, underlain by very stiff to hard clay with weathered rock inferred below -3.1 m AHD.
85 Ocean View Drive	4/12/2013	Douglas Partners	Sand above RL 6.4 m AHD. Very stiff to hard clay between -2 and 6.4 m AHD. Weathered rock inferred below -2.0 m AHD.
79 Ocean View Drive	26/09/2013	Douglas Partners	Sand above RL 0.5 m AHD, underlain by very stiff to hard clay to -3.8 m AHD (limit of investigation). Bedrock inferred below.
75 Ocean View Drive	10/07/2008	Douglas Partners	Sand above RL zero m AHD. Very stiff to hard clay between zero and -10.8 m AHD. Weathered rock inferred below -10.8 m AHD.
63 Ocean View Drive	11/11/2005	Douglas Partners	Sand above RL -3.0 m AHD, underlain by very stiff to hard clay.
51 Ocean View Drive	11/09/2014	Cardno Geotech	Loose to medium dense, fine to medium grained sand.
41 Ocean View Drive	10/05/2004	Douglas Partners	Sand above RL -7.4 m AHD, underlain by very stiff to hard clay.
33 Pacific Street	2/11/2004	Coffey Pty Ltd	Sand from surface to RL zero m AHD where borehole terminated. No information below this level.
25C Ocean View Drive	10/12/2004	Coffey Pty Ltd	Sand from surface to RL 0.85 m AHD where borehole terminated. No information below this level.
23B Ocean View Drive	10/01/2012	Jeffery and Katauskas	Thick layer of silty sand and silty clay over relatively clean sandy soils to termination depth (approx. -6.5 m AHD)
23A Ocean View Drive	26/09/2007	Douglas Partners	Sand from surface to -4 m AHD, underlain by stiff clay to -9 m AHD then apparent rock.
7-9 Pacific Street	18/11/2010	Douglas Partners	Sand from surface to -2 m AHD, underlain by stiff to very stiff clay to -15 m AHD (limit of investigation).
5 Pacific Street	24/11/2008	Geotechnique Pty Ltd	Sand from surface to -3 m AHD, underlain by stiff to very stiff clay to -5 m AHD (limit of investigation).

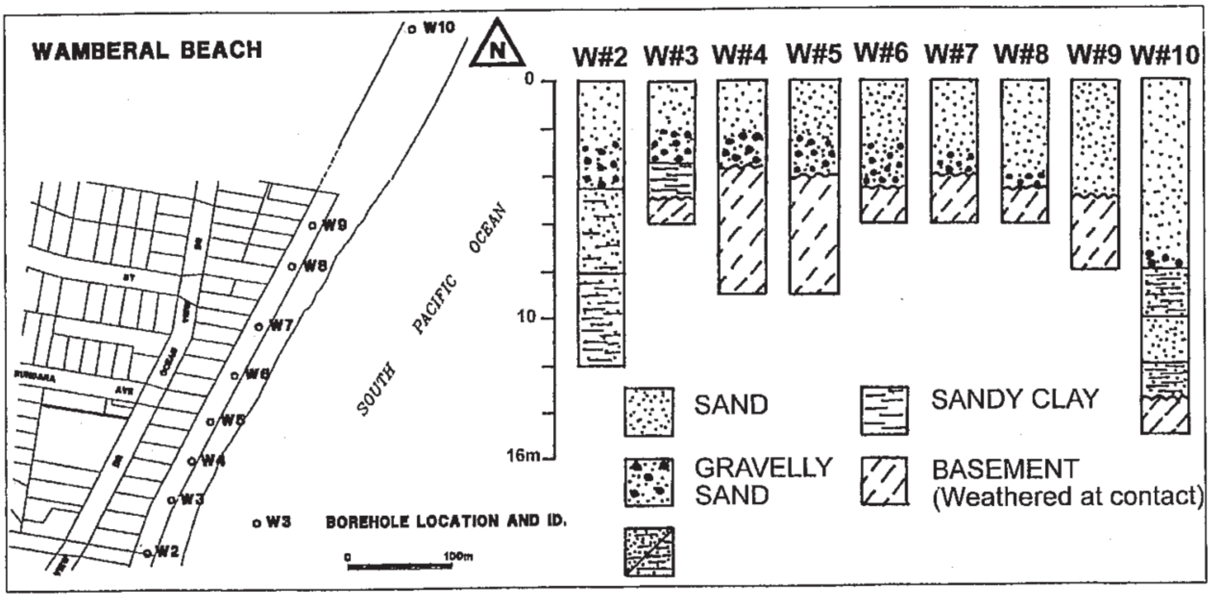
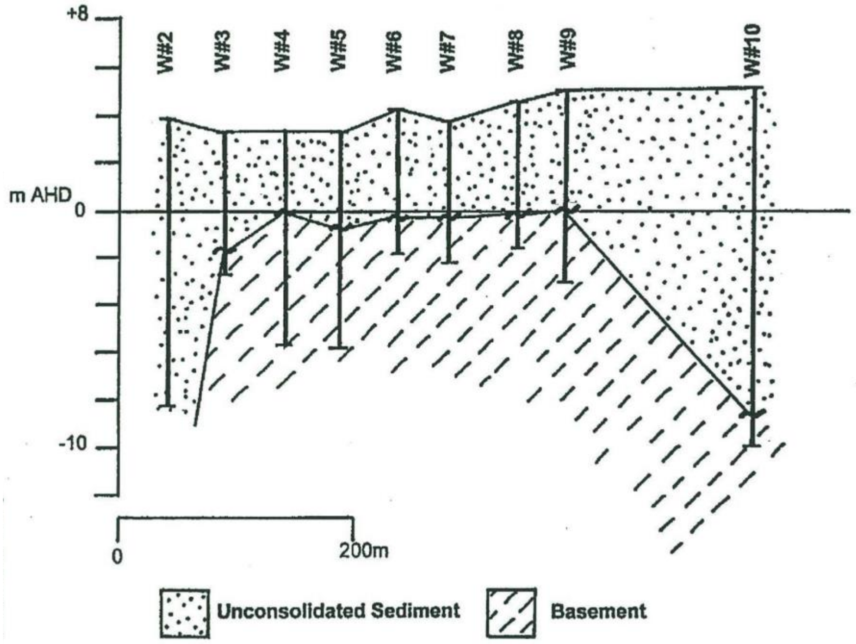


Figure B.4: Wamberal Beach borehole samples from Hudson (1997). Siltstone/claystone unit interpreted as bedrock.



Borehole ID	Ground Level (m. AHD)	Interpreted Depth to Bedrock (m. AHD)
W#2	3.95	-8.05 (NE)
W#3	3.30	-1.70
W#4	3.36	-0.14
W#5	3.34	-0.66
W#6	4.24	-0.26
W#7	3.72	-0.28
W#8	4.41	-0.19
W#9	5.00	0.00
W#10	5.03	-8.47

NE: Bedrock not encountered to depth shown

Figure B.5: Wamberal Beach Geological Section from Hudson (1997). Siltstone/claystone unit interpreted as bedrock.

B.4 Other material in beach and foredune substrate

It is also noted that various ad-hoc material exists in the beach and foredune substrate. In 2017 WorleyParsons described existing materials located on Wamberal beach noted in previous studies and from field inspections at the time (WorleyParsons, 2017). These are listed in Table B.2.

Table B.2: Materials in beach and foredune substrate as documented by WorleyParsons (2017)

Material	Known locations
1974 Rock Protection	43-45, 51-67, 73-75, 81-83, 87-103 and 105 Ocean View Drive. 2 and 4 Surfers Road; 1-9 Calais Road.
1978 Rock Fill and Ballast	23a-b Ocean View Drive
Rock fill/rubble/bricks	9,13-19 Pacific St. 55, 65, 67, 69-71, 75, 81, 85, 91, 97, 101, 103, 105 Ocean View Drive.
Large rock (0.7-2m diameter)	25, 33 Pacific St 25c, 27, 49, 57, Ocean View Drive
Concrete walls (various)	19 Pacific St 35, 53 Ocean View Drive 1 Calais Rd
Terracotta Seabee with rock wire basket toe	59-61 Ocean View Drive
Concrete capping/pieces	29 Pacific St. 37 Ocean View Drive
Corrugated iron	Not specified
Rubber tyres	7 Calais Rd
Septic Tanks fill with sand/gravel	25 Pacific St
Timber retaining walls	21-23,31 Pacific St 27, 31, 41, 75, 79, 81, 83, 93, 95, 97 Ocean View Drive 3, 5, 7 Calais Rd

Additional material was recently placed as part of emergency protection works following substantial storm erosion in July 2020. The works were placed on the beach over a period extending from 26 July to early August 2020. The works are summarised in Table B.3. At total of 2660 tonnes of rock armour and 2120 tonnes of rock bags armour was placed on the beach as part of the works. Concrete blocks were also placed by a resident in front of 73 Ocean View Dr.

Materials visible on the beach following the July 2020 storm erosion were mapped by MHL drone surveys and are presented in Figure B.6. Materials included emergency rock protection and rock bags outlined in Table B.3 as well as ad-hoc materials existing in the beach prior to the July 2020 storm including former rock protection works, failed concrete structures and rubble, gabion rock mattresses, terracotta seabee units and other objects.

Preliminary costs for offsite removal and/or onsite replacement/reuse of various ad-hoc rock protection and materials present on the beach has been considered in Section 5.2.

Table B.3: Emergency protection works at Wamberal Beach July-Aug 2020 as of 8 Aug 2020 (per comms, Royal Haskoning 2020)

Material	Approximate tonnes placed (t)	Locations
Stage 1A Rock armour	Total: 1080t 60t per property 120t at 51 Ocean View Dr	25B-45, 51, 55-63 Ocean View Dr
Stage 1A Kyowa Rock Bags (2t)	Total: 520t 20-70 bags per property	47, beach access adjacent to 65, 69-71 Ocean View Dr
Stage 1B Rock armour	Total: 1580t 60t per property 100t at 95-97 Ocean View Dr 120t at 51 Ocean View Dr	27-45, 49-51, 55-65, 67, 81-83, 93-97 Ocean View Dr
Stage 1B Rock bags (4t)	Total: 1600t 30 bags per property 10 bags at 75 Ocean View Dr	47, beach access adjacent to 65, 69-79, 85-91 Ocean View Dr

NSW GOVERNMENT

EXISTING MATERIALS PRESENT ON BEACH POST JULY 2020 STORM EROSION

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Figure

B.2

Figure B.2.pdf

B.5 Summary

The existing geotechnical information summarised above for the Wamberal Beach study area was considered sufficient to undertake seawall concept design development and costings. The adequacy of available geotechnical information is to be revised for detailed design purposes. A preliminary allowance for further geotechnical investigations as part of detailed design has been considered in preliminary cost estimates in Section 5.2.

During construction, it is strongly recommended that any existing ad-hoc material and emergency rock toe protection works seaward of the proposed new seawall be removed to enhance beach amenity, or where suitable used as fill where required landward of the seawall. Any existing rock protection removed with construction should be replaced by sand nourishment wherever possible to extend the level of the natural beach berm or foredune seaward of the seawall. All sand excavated during the construction of the proposed seawall should be screened (to remove any oversized materials) and placed seaward of the works with any necessary fill landward of the seawall comprised of the separated materials (if suitable) and/or suitable clean fill that would be imported to the site. This will maximise the amount of sand added to the beach area as a result of the works.

B.6 References

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