Guidance for coastal protection works in Pacific island countries

DESIGN GUIDANCE REPORT

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Abbreviations

AEP	annual exceedance probability
ARI	annual recurrence interval
EbA	ecosystem-based approaches
CMB	concrete masonry “besser” construction blocks
d	depth
ENSO	El Niño–Southern Oscillation
GPS	Global Positioning System
GSC	sand-filled geotextile container
H_s	significant wave height
IPCC	Intergovernmental Panel on Climate Change
IPO	Interdecadal Pacific Oscillation
km	kilometer
li m	linear meter
m	meter
mm	millimeter
MPa	megapascal
MSL	mean sea level
NGO	nongovernment organization
PIC	Pacific Island Countries
PRIF	Pacific Region Infrastructure Facility
s	second
SEAFRAME	SEA-Level Fine Resolution Acoustic Measuring Equipment
SLR	sea level rise
T+T	Tonkin & Taylor International
WRL	Water Research Laboratory

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Executive Summary

Coastal erosion is a perpetual and serious concern for Pacific island countries. Coastal erosion may include various consequences from natural disasters, such as high water levels, sediment impacts on coral reefs, coastal sand extraction on beaches and rivers, and sediment traps. The effects of climate change, such as the rise in sea level, degradation of coral reefs, and increased frequency and intensity of storms, also increase the risk of erosion.

Erosion and accretion are natural processes that can potentially affect high-value assets, placing road, maritime, and aviation infrastructures at risk with significant potential cost implications. Erosion is of particular concern to the transport sector which, through its infrastructure, provides a critical lifeline for these geographically dispersed nations. While a range of measures may be applied to mitigate erosion hazards, including a departure from perilous locations or the relocation of assets, such options often are not viable when the availability of land is limited or the infrastructure in place is expensive to relocate. In such cases, land and assets must be protected.

Based on a study, this guidance report aims to add to the existing knowledge by developing innovative solutions for coastal protection to maximise the use of local materials and labour, while minimising the requirements for imported materials and equipment. This document provides an overview of the design process that includes a description on how to identify a soil erosion issue, determine the most appropriate mitigation solution, and assess design conditions. It also outlines the required steps for ensuring a robust engineering design, assessing environmental effects, obtaining the necessary construction documentation, and monitoring throughout the process.

This report also provides information on the design of selected 'affordable' coastal protection options, including a description of each, as well as a concept-level design guideline for selected conditions, general specifications, design drawings, proposed construction methodology, and information on maintenance requirements. It aims to assist government public works departments, coastal managers, consultants, nongovernment organisations and contractors, among others.

This document neither includes detailed designs for complex or high-priority structures, nor does it intend to replace codes, standards, or guidance manuals that are in existence.

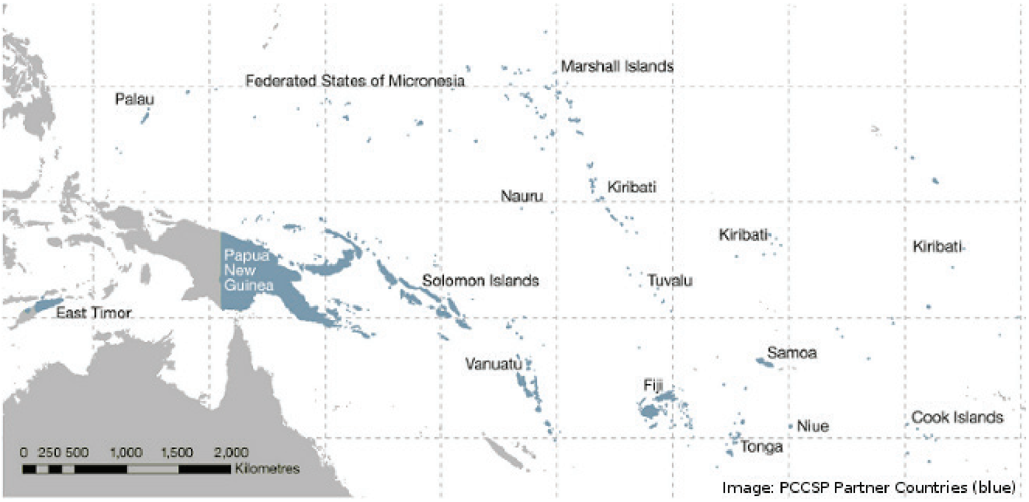
It is recommended that a suitably qualified professional be involved in the assessment and design process to ensure a robust and reliable outcome. Furthermore, a full understanding of the local setting is critical to the development of any design solution.

1 Introduction

1.1 Background

Coastal erosion is a constant threat to Pacific island countries (PIC) (Figure 1-1) as a result of storms and high water levels, increased sediment on coral reefs, mining of coastal sands on beaches and rivers, and entrapment of sediment. Many of these issues are the effects of climate change, which raise the risks on coastal protection.

Figure 1-1 Location of Pacific Island Countries



Source: www.pacificclimatefutures.net/en

While erosion and accretion are natural processes, when they affect road, maritime, or aviation infrastructures, these high-value assets are placed at risk, with significant potential cost implications. Erosion is of particular concern to the transport infrastructure sector, which provides a critical lifeline for these nations that are geographically dispersed.

While a range of measures are available to mitigate the effects of erosion, including avoiding locations that are potentially threatened or relocating assets, these options are often not feasible in the face of limited land availability and the high cost to relocate infrastructure. As such, land and assets must be guaranteed protection.

Traditional responses to coastal erosion include rock or concrete revetments and seawalls. These structures are typically engineered to withstand scour, wave impact, and overtopping, and formal design guidance is available. The major obstacles of coastal protection construction in PICs are the lack of design expertise and contractors, construction plants, and local materials that are suitable (e.g., rock of sufficient size and quality), as well as the high cost of importing materials.

A range of "non-engineered" pilot projects for coastal (land) protection have been carried out throughout the region, with varying levels of success. These have included the use of gabion baskets, sandbags, grout-filled bags, stacked coral rock, grouted coral rock, concrete-filled pipes, and other materials. Such methods, however, have exacerbated coastal erosion, due to the use of local beach sand and coral sand aggregate to produce lightweight and low-strength concrete. This component has damaged backshores due to the overtopping of walls and loss of material from within the wall. Many of these challenges, nevertheless, can be resolved by modifying the design or materials.

1.2 Study Overview

Pacific Region Infrastructure Facility (PRIF) engaged Tonkin & Taylor International (T+TI), in association with the Water Research Laboratory (WRL) of the University of New South Wales to undertake coastal engineering research on affordable options for coastal protection. The objective of the study is to add to the existing knowledge by developing innovative solutions for coastal protection, at the same time attempting to maximise the use of local materials and labour and minimise the need for imported materials and equipment.

The study has been undertaken in two phases. The first incorporated a desktop review to catalogue and critically evaluate existing approaches to shoreline protection in the Pacific, based on technical, social, and environmental criteria. Selected approaches were assessed in terms of annual costs for various locations and wave climate regimes, taking into account design wave height, availability of suitable materials, transportation, and design life.

It was evident from the study that conventional approaches, such as rock revetments, typically have long design lives and moderate construction costs in areas where materials and construction plants are available. Costs significantly increase when materials are to be transported, particularly over long distances, to remote island locations. It may be more cost effective to use local material and labour for alternative protection measures, such as structures that require lower material volumes or those that can use local materials and labour, despite the potential for having shorter design lives.

Recommendations were provided on preferred approaches (depending on location and material cost), as well as modifications to existing approaches to improve performance. Various alternatives were also proposed, including the use of small, manually placed, geosynthetic containers and material that is locally available, such as concrete masonry (Besser®) blocks that can be placed in a specific pattern in low energy environments. Given the lack of design guidance for these approaches, it was necessary to carry out hydraulic model testing to determine threshold wave conditions.

The second phase of this study relates to the guidelines developed from this study to enable the application of selected approaches identified from the desktop review.

1.3 Development of the Guidance Report

This coastal infrastructure design guidance report provides an overview of the design process (Figure 1-2), and it includes the methods used to identify an erosion issue, the most appropriate mitigation solutions, and an assessment of design conditions. It also outlines the steps necessary to ensure an engineering design that is robust, environmental effects to be taken into account, construction documentation, and monitoring during and after construction.

The document also presents general design information on selected "affordable" coastal protection options. These include a description of the options, concept-level design guidance for selected conditions, general specifications, design drawings, proposed construction methodology, and information on maintenance requirements.

The guideline is intended for a range of end users. These include public works departments, coastal managers, consultants, donors and NGOs, and contractors.

1.3.1 Document Outline

The document is structured as follows:

Section 1 Introduction and overview of the guidance document

Section 2 Identification of an erosion issue

Section 3 Overview of coastal protection structures

Section 4 Optional assessment for coastal protection

Section 5 Assessment of design conditions

Section 6 Fundamentals of engineering design

Section 7 Assessment of environmental effects

Section 8 Documents required for construction

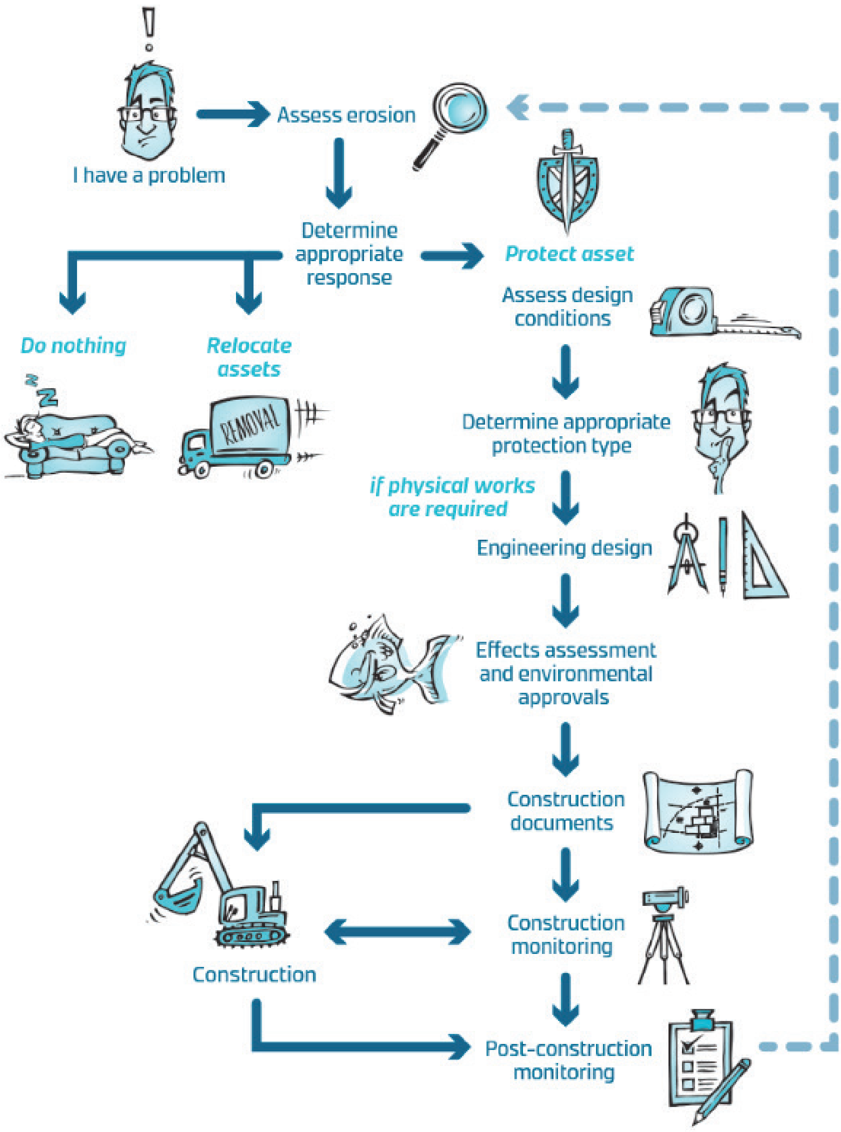
Appendix A Concept designs for a range of coastal protection works

1.3.2 Limitations

This study presents an overview of the design process, and assists in an option comparison and the selection of concept designs that relate to remote PICs. It is neither intended to guide (i) detailed designs, (ii) designs of complex or high-priority structures, nor (iii) replace existing codes, standards, and guidance manuals.

It is essential that a suitably qualified professional be involved in the assessment and design process to ensure a robust and reliable outcome. In-depth understanding of the local setting is critical when developing any design solution.

Figure 1-2 Typical Assessment, Design, and Construction Flow



2 Identifying an erosion issue

2.1 Coastal Erosion

Coastal erosion refers to the landward movement of the coastal edge. Erosion occurs on all coastal types, including unconsolidated beaches, soft estuarine shorelines, and a harder cliffed coastline. The mechanism responsible for erosion and the rate of erosion, however, will vary.

On unconsolidated coastlines, the shoreline position fluctuates in the short term as a result of storms and calm periods and, over longer periods, can recede or accrete as a function of sediment supply and demand. Erosion of a cliffed coastline is generally a one-way process that is caused by marine and bio-erosion, as well as the weathering of the cliff toe and surface, causing cliffs that have been oversteepened by waves to collapse. Estuarine shorelines are highly dependent on water level, with the shoreline position typically a function of water level with exposure to wave energy a secondary control.

In order to solve an erosion issue, local coastal processes and the drivers of erosion must be fully comprehended. Otherwise, the solution may not prove to be successful or may have unintended consequences.

2.2 Site Inspection

The first step in diagnosing a potential erosion issue is to inspect the site and surrounding areas to evaluate the local setting and potential erosion mechanisms. Key indicators of erosion (Figure 2-1) include:

- low sand levels on the beach with saturation during low tidal levels;
- a steep scarp on the upper beach, with water reaching the scarp toe;
- structures or vegetation, once on land, now on the beach; and
- exposure of beach rock on the beach face, with beach rock forming below the beach surface, indicating past beach position.

Anecdotal information provided by local inhabitants, particularly those who have lived or visited the location for a long period, can assist in understanding changes over time. Questions may include:

- Where was the original beach/shoreline?
- Have recent or historical storms caused erosion and damage to the beach?
- Do sand levels fluctuate and, if so, are they event based (e.g., following storms), seasonal, or decadal type changes?
- Has sand been building up somewhere else nearby?
- Do waves wash over the beach crest/berm and, if so, how often?
- In which direction do currents typically go? Does this change at different times of the year or during storm events?
- Does sand mining occur on the beach (Figure 2-2) or has it previously done so?

Figure 2-1 Example of Low Beach Levels

(A) (B) (C) (D)

(A) Backshore Scarp; (B) Structures; (C) and Trees (D) Remaining on Beach Face.

Photo credits: T. Shand.

Figure 2-2 Examples of Local-Scale Beach Mining

Photo credits: T. Shand.

2.3 Historic Aerial Imagery

Historic aerial photographs provide a useful way of quantifying changes along a length of coastline. Aerial photographs are often available from the 1940s or 1950s and satellite images since the late 1990s or early 2000s, depending on location. Images must typically be georeferenced to a consistent spatial location, using geographic information system (GIS) software, and then the position of the historical shoreline is digitised. The position of previous shorelines allows long-term changes to be quantified and trends identified. Figure 2-3 shows historical shoreline positions at Manase, Samoa, which reflect erosion since 1954 at the eastern and western ends of the beach and accretion since this time in the central part of the beach.

Figure 2-3 Summary of Historic Shorelines Overlaid on Satellite Image, July 2012

Source: Tonkin + Taylor, 2014.

2.4 Sediment Budget

A sediment budget is used to identify the origin of the sediment (source), where it goes to (sediment sinks), and whether the coastline has a surplus (accretion) or a deficit (erosion). In PICs, most sediment is derived from:

- mechanical and biological weathering of the top of the reef;
- biological production within the lagoon;
- erosion of the shoreline (i.e., dunes and cliffs); and
- erosion of the inland areas, with material carried down streams and rivers to the coast.

Sediment is lost due to:

- the mechanical wearing of individual particles by wave action;
- being transported offshore under wave action and nearshore currents to areas where it cannot return to the coast (i.e., offshore of a reef edge and through reef passages); and
- anthropogenic causes, such as beach mining and impoundment, when sediments are trapped behind hard structures or against breakwaters and groynes.

A conceptual model of the shoreline processes should be developed (Figure 2-4), with identified sources, transport pathways, and sinks. Critical questions when developing such a model include:

- Where does the sediment come from?
- How is it moved and to where?
- Where is the sediment being lost?
- What may have changed or what may be changing in the system to cause an erosion issue?

The sediment budget may be either qualitative or quantitative in the presence of data.

Figure 2-4 Conceptual Model of Coastal Processes Occurring at Manase Beach, Samoa

Source: Tonkin + Taylor, 2014.

2.5 Climate Change Effects

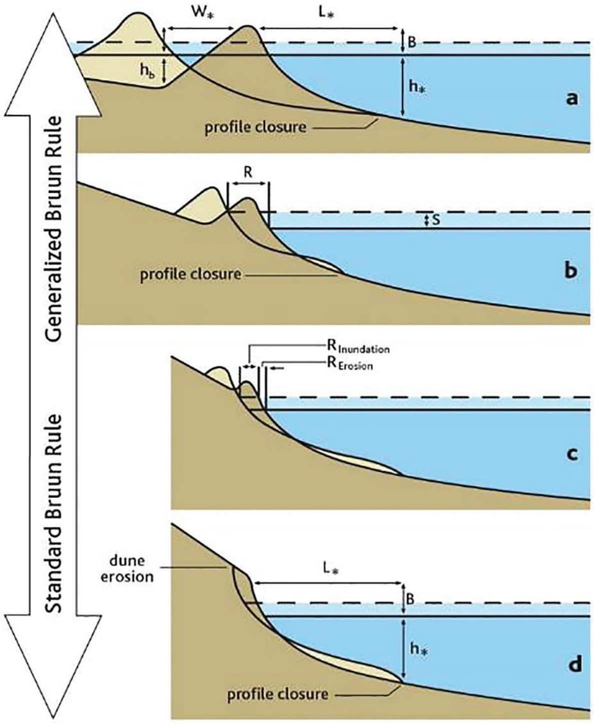
Future climate change effects in the Pacific may include changes in (Australian BoM and CSIRO, 2011):

- temperature (air and sea)
- rainfall patterns
- in wind-speed and tropical cyclone strength and distribution
- in ocean salinity and acidification
- in sea level.

While these may have direct or indirect effects on sediment processes and budget, the most pertinent in terms of coastal erosion are changes in sea level. While many atoll islands are dynamic and respond to changing climate by altering their form and location, rising sea levels will generally result in a landward migration of the coastal edge. This may signify that accreting coasts may slow, stable coasts may begin to retreat, and eroding coasts may do so at a faster rate.

The most widely known model for this beach response is that of Bruun (1962). The Bruun model assumes that as the sea level rises, the equilibrium profile moves upward and landward, conserving mass and original shape (Bruun, 1962; Cowell and Kench, 2001; Komar et al., 1999). This profile translation effectively results in a recession of the coastline. While some recent studies have observed increases in total land area on Pacific islands over the past decades (Webb and Kench, 2010), they have generally occurred on more mobile reef-top islands where there is biogenic sand production. Coupled with this, Hoegh-Guldberg et al. (2011) suggest that ocean acidification over the twenty-first century will compromise carbonate accretion, with corals becoming increasingly rare on reef systems, thus reducing an important source of sediment for Pacific beaches. The only offset to this may be a projected increase in rainfall (BoM, 2012), bringing more volcanic sediment from the catchment, although this process is restricted to mountainous islands and does not apply to atolls.

Figure 2-5 Schematic Diagrams of the Bruun Model Modes of Shoreline Response



Source: Cowell and Kench. 2001. The Morphological Response of Atoll Islands to Sea-Level Rise. Part 1: Modifications to the Shoreface Translation Model. JCR, SI 34 ICS(2000). Pp. 633–644.

2.6 Consequence and Hazard Risk

It is important to recognise that erosion is a natural process. Uninhabited islands may change form and position significantly (Webb and Kench, 2010), with parts of the island experiencing erosion and some accretion, with no adverse impact on human values and assets.

In order to select the most appropriate response, erosion—or predicted future erosion— and the effect or consequence on human values must be considered. Hazard risk (R) is a function of the likelihood of an event occurring (P) and consequence of that event occurring (C) (AGS, 2000).

$$R = P \times C$$

The likelihood of hazard occurrence (Table 2-1) ranges from "Almost Certain", where the event is on-going or is expected to occur during the next year, to "Rare" events, with return periods in excess of 200 years, thus having a low probability of occurring in a person's lifetime. The likelihood of occurrence at the current time, along with the future likelihood over a selected period, should be assessed.

Measures of consequence may include values of economic importance (e.g., roads, pipelines, and structures), as well as social and cultural values (e.g., meeting areas, graveyards, and village or tribal land), examples of which are provided in Table 2-2. Consequence may range from Insignificant, where there is minor inconvenience and some delays (hours) (e.g., a temporary road closure) to catastrophic (e.g., major volcanic eruption or tsunami), where widespread infrastructure is completely destroyed and there are multiple fatalities and extreme delays (years) due to the reconstruction of multiple infrastructure.

Table 2-1 Measures of the Likelihood of Hazard Occurrence

Descriptor	Description	Estimated Event Return Period	Examples
Almost certain	The event is ongoing or is expected to occur during the next few months	Monthly	High spring tides; minor erosion
Very likely	The event is expected to occur intermittently	~Annual Event	Tropical storm; small-scale erosion or flooding
Likely	The event is expected to occur under adverse conditions (i.e., several occurrences per generation)	2–10 years	Category 1–2 cyclone coinciding with high tides; moderate erosion or flooding
Possible	The event is expected to occur under adverse conditions (i.e., likely to occur at least once per generation)	10–50 years	Minor tsunami (<2 metres); direct impact from Category 2–3 cyclone. Major coastal flooding or erosion
Unlikely	The event is expected to occur under high to extreme conditions (i.e., possibility of occurring in a generation)	50–200 years	Moderate tsunami (2–5 metres); direct impact from Category 3–4 cyclone. Extreme coastal flooding or erosion
Rare	The event could occur under extreme conditions (i.e., low probability of occurring during a generation)	>200 years	Large tsunami >5 metres; direct impact from Category 5 cyclone.

Table 2-2 Measures of Consequence

Descriptor	Description
Catastrophic	Infrastructure completely destroyed, multiple downstream infrastructure severely affected. Multiple fatalities. Extreme delays (years) as reconstruction of multiple infrastructures take place.
Major	Extensive damage to the infrastructure and moderate effects on downstream infrastructure. Widespread injury with possible fatalities. Extensive delays (months) as significant repairs take place.
Medium	Moderate damage to infrastructure and significant delays (days to weeks) with minor effects to downstream infrastructure. Some injury probable, with possible fatalities (e.g., landslip over major road; wave damage to house foundations).
Minor	Limited damage to part of the infrastructure. Some delays (days). Isolated injuries possible (e.g., minor seawall damage during storm; water ponding within dwelling).
Insignificant	Minor inconvenience. Some delays (hours) (e.g., road closure due to combined heavy rainfall and high tide inundation; flooding of land surrounding dwelling).

Once the likelihood of the erosion "event" and the consequences of that erosion are determined, the risk can be assessed (Table 2-3). Only then should an informed decision be made on what, if any, action should be taken.

For example, if there is a moderate likelihood of future erosion but the consequence is the loss of unused land (i.e., a minor consequence), the risk may be assessed as low and only a small amount of effort in addressing the erosion issues may be justified. If there is a high likelihood of erosion that threatens roads, pipelines, and dwellings (major consequence), however, the risk is considered high and a substantially higher amount of effort can be justified.

Table 2-3 Risk Matrix

		Consequences				
		1: Catastrophic	2: Major	4: Medium	5: Minor	6: Insignificant
Likelihood	A – Almost certain	Very High	Very High	High	High	Moderate
	B – Very likely	Very High	High	High	Moderate	Low
	C – Likely	Very High	High	Moderate	Low	Low
	D – Possible	High	High	Low	Low	Very Low
	E – Unlikely	High	Moderate	Low	Very Low	Very Low
	F – Rare	Moderate	Low	Very Low	Very Low	Very Low

Risk may differ between the present and future as hazard likelihood increases (e.g., rising sea levels increase coastal flooding). An example of such a hazard assessment is presented in Table 2-4 below.

Table 2-4 Risk Assessment Example for Present and Future Scenarios

Infrastructure	Hazard	Timeframe	Likelihood	Consequence	Risk
Low-lying houses	Coastal flooding	Present	Likely	Minor: flooding of existing dwelling floor levels	Low
		2100	Very likely		Medium
Access road	Coastal erosion	Present	Likely	Minor: ongoing erosion closes road to vehicles, forcing use of alternative route	Low
		2100	Almost Certain		High
New beachside resort	Moderate tsunami	Present	Unlikely	Major: buildings in hazard zone destroyed; fatalities or severe injury if no evacuation	Medium
		2100	Unlikely		Medium

Risk level implications are shown in Table 2-5 although, in general, risks of low or very low magnitude can be managed at the village level in the short term and monitored in the longer term. Medium risks and above require a treatment plan to reduce risks to acceptable levels, with the risk level guiding the type of solution deemed appropriate. Changes to hazard over time provide a good indication of future risk and present an opportunity for preventative (‘*top of cliff*’) action.

Table 2-5 Risk-Level Implications

Risk Level		Implications for Risk Management
VH	Very high risk	Immediate solution required to reduce risk to acceptable levels. May require detailed investigation, design, planning, and implementation of solution. May involve high costs.
H	High risk	Short-term solution required to reduce risk to acceptable levels. May require investigation, design, planning, and implementation of solution. Moderate costs likely.
M	Moderate risk	Broadly tolerable in short term, provided treatment plan is implemented to reduce risks; may require investigation and planning of treatment options.
L	Low risk	Acceptable in short-to-medium term. Long-term treatment requirements to be defined to maintain or reduce risk.
VL	Very low risk	Acceptable. Manage by normal local procedures.

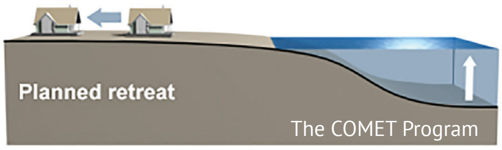
3 Overview of Coastal Protection Methods

3.1 Introduction

While erosion and accretion are natural processes, when they affect road, maritime, or aviation infrastructure, these high-value assets are put at risk with significant potential cost implications. Erosion is of particular concern for transport infrastructure, which provides critical lifelines for these geographically dispersed nations.

A range of measures may be used to mitigate the erosion hazard, including avoidance of hazardous locations or relocation of assets. These options are often not feasible when land availability is limited or when infrastructure is expensive to relocate. In these cases, the land and assets must be protected. Principles of coastal protection are described below, together with examples of specific protection measures trialled within the Pacific.

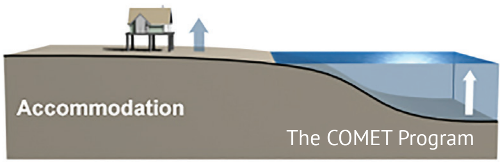
Table 3-1 Options for Managing Coastal Hazards

<table>
<tr><td rowspan="4">Avoid/Retreat</td><td colspan="2">Avoiding or Retreating from the hazard through either planning restrictions or by relocating assets out of the hazard-affected area eliminates the likelihood and, therefore, the risk.</td></tr>
<tr><td>Examples</td><td>▪ Development restrictions
▪ Relocation out of hazard zone

</td></tr>
<tr><td>Benefits</td><td>▪ Long-term security
▪ No/low maintenance requirements or future costs
▪ No risk of immediate failure of protection system
▪ Reduces risk to ecosystems and the environment
▪ Can increase public space in high-use amenity areas (e.g., beaches)
▪ Increases tourism potential
▪ Low regrets</td></tr>
<tr><td>Barriers</td><td>▪ Perceived (or real) loss of land or use of land
▪ Land unavailability for relocation or high purchase cost
▪ Services required to relocation areas
▪ May not be economically viable or compensation required
▪ Legacy issues; community, social, and political inertia
▪ Land ownership, usage rights, legislation</td></tr>
</table>

<table>
<tr><td rowspan="4">Accommodate</td><td colspan="2">Accommodate the hazard by reducing the likelihood or magnitude of the hazard or reducing the consequence of the hazard.</td></tr>
<tr><td>Examples</td><td>

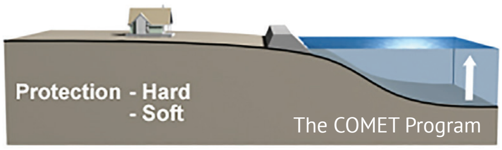
Structure maintenance
Raising building platform levels
Early warning systems + Disaster Risk Reduction, reducing loss of life
Ecosystem-based approaches (EbA)
</td></tr>
<tr><td>Benefits</td><td>

Typically lower capital costs than protection
Less regrets than protection
Generally lower impact on ecosystems and environment than protection (or improvement in case of EbA)
Minimal social disruption
Lower long-term impact and reduced opportunity for “flow-on” impacts (e.g., seawall end effects)


</td></tr>
<tr><td>Barriers</td><td>

May not be sustainable in long term (e.g., seawall repairs)
May take time to become effective (e.g., EbA)
Risk may be reduced, although not eliminated
</td></tr>
<tr><td rowspan="4">Protect</td><td colspan="2">Construction of physical works to Protect against a particular threat or range of threats. Options may be “hard”, such as a revetment or seawall; “soft”, including beach nourishment; or a combination of the two. Protection options should be used in combination with Accommodation options, in order to broaden benefits and minimise adverse effects, and with Avoidance options to ensure long-term resilience.</td></tr>
<tr><td>Examples</td><td>

Beach nourishment
Offshore structure
Groyne
Revetment
Seawall


</td></tr>
<tr><td>Benefits</td><td>

Immediately beneficial for intended purpose (i.e., protection of land from erosion)
If adequately designed, effective against intended hazard
If adequately designed, benefits for years to decades
Can incorporate ecologically beneficial aspects or minimise damage
Can improve amenity (e.g., beach nourishment or walkway on concrete capped seawall)
</td></tr>
<tr><td>Barriers</td><td>

High regrets if it fails (e.g., seawall collapse)
Perceived level of protection may encourage development, increasing consequence and therefore risk after the works’ protective lifespan has passed
High ongoing maintenance and replacement costs
May adversely affect ecology and the environment
May adversely affect recreational amenity or other uses of the area
</td></tr>
</table>

3.2 Alternatives to Hard Protection

A range of alternatives to hard coastal protection may be considered to provide more economic or long-term solutions to coastal erosion. These are discussed in the sections below.

3.2.1 Avoidance of, or Retreat from Hazard

The most effective way to manage the risk from a hazard (where feasible) is to avoid the hazard altogether. Development restrictions place limits on the development that can occur in locations deemed to be hazardous or place requirements for measures to be undertaken to avoid the hazard. These restrictions may apply to new developments only or may include modifications to existing developments. Restrictions on developments in one area require the provision of alternative sites, with infrastructure such as roads, power, and water in place to minimise negative social impacts.

Asset relocation involves the progressive abandonment or movement of assets located in hazardous zones or not built to withstand hazardous events to non-hazardous areas. Such relocation may be required immediately, when the hazard is high and protection or accommodation is not feasible, or may only occur in the future when climate change increases the hazard to a point where retaining the asset is not sustainable. Site-specific negative social impacts relating to involuntary resettlement losses must be considered.

3.2.2 Maintaining Sediment Budgets

A sediment budget refers to the sediments entering and leaving a coastal system. Where more material leaves the system than enters, the system is in deficit, and erosion/recession of the backshore occurs. Changes in the sediment budget may be due to natural changes in the environment or due to human activities such as degradation of the reef, trapping of sediment by structures, or direct removal of material from the coastal zone through sand mining.

Sand mining has historically occurred throughout the Pacific on a commercial and domestic scale. Sand can be derived from river systems, lagoons, and the fringing coral reef. Some of these sources are naturally replenished and sustainable; however, excessive removal of material, or removal from the wrong locations, can lead to an eventual deficit of sand on the beach and increase erosion potential. While much of the commercial mining has ceased in recent years, observation by the authors suggests smaller-scale domestic mining continues in many PICs.

Alternatives to coastal sand mining are to mine on land where sands have been historically deposited or at sediment sinks where material has left the coastal system. An example of this is the European Union funded *Environmentally Safe Aggregates to Tarawa* (ESAT) project, at South Tarawa in Kiribati, where offshore lagoon material is dredged and sorted to provide a sustainable aggregate source.

Reducing sand mining in some locations may minimise the potential for shoreline erosion and the requirement for coastal protection works. It also improves the amenity value of the coastline for the local community and tourist operations. In some locations, however, reduction of sand mining could lead to reduced navigability and an infill of ports.

3.2.3 Ecologically Based Approaches

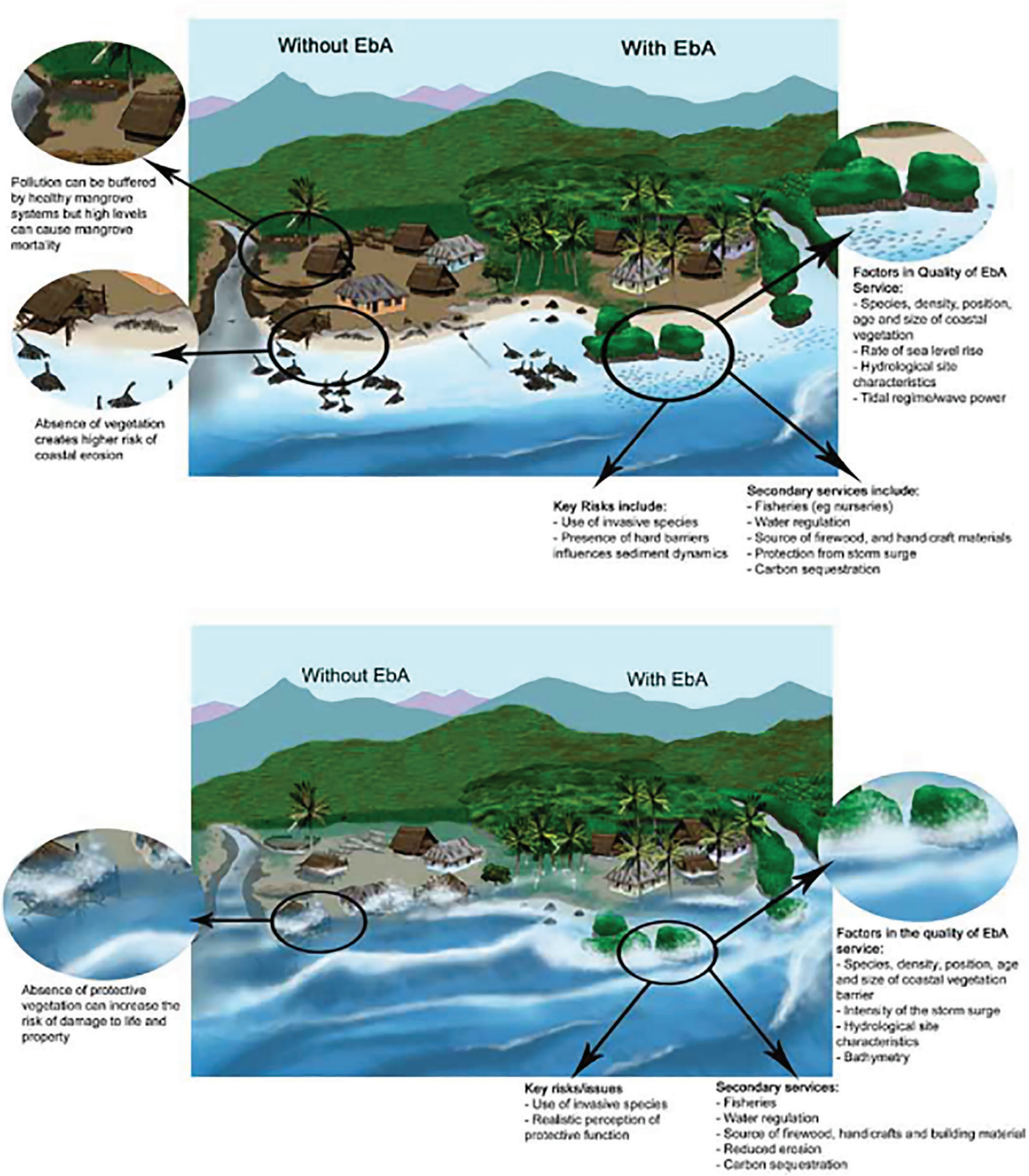
Ecosystem-based approaches (EbA) aim to protect the shoreline from wave-induced erosion by maintaining healthy ecosystems. This may include the following:

- establishment of offshore vegetation, such as mangroves, to dissipate wave energy before it reaches the shoreline and traps fine sediment, while maintaining habitats for juvenile fish and marine species;
- establishment of backshore vegetation to reduce wave run-up extent and damage potential, trap wind-blown sand, and improve ecological connectivity between land and sea; and
- improvement of coral reef health to ensure coral production is maintained.

The use of EbA for coastal protection and as a method to offset the impacts of climate are described extensively in the literature (e.g., World Bank, 2010; Hills et al., 2011), including techniques to combine EbA with conventional protection structures (DECCW, 2009). Although economic analyses of ecological approaches often identify high benefit-cost ratios compared to coastal protection structures, this is generally a function of low implementation costs with modest improvements in the protection provided.

Such improvements are unlikely to achieve desired outcomes when erosion is directly and immediately threatening coastal infrastructure or assets.

Figure 3-1 Concept Sketches of the Use of Ecologically Based Approaches to Reduce the Effects of Coastal Erosion and Flooding



Source: Hills, T., A. Brooks, J. Atherton, N. Rao, and R. James. 2011. Pacific Island Biodiversity, Ecosystems and Climate Change Adaptation: Building on Natures' Resilience. Apia, Samoa: Secretariat for the Pacific Regional Environment Programme.

3.3 Rigid Structures

Rigid structures protect the land by resisting coastal processes. They may be vertical (Figure 3-2), sloping, or stepped, and are traditionally constructed of mass concrete or reinforced concrete, grouted rock or blocks, timber or steel sheet piling, or timber posts. They require a well-founded toe, preferably on hard substrate or deeply piled to avoid scour and undermining. Additional toe protection, using a semi-rigid structure, may be required to prevent scour and undermining. The structures must be robust due to the high wave loading and, therefore, they tend to be either massive structures or better suited to low-to-medium wave environments where wave loading is moderate. Runup and overtopping is similarly high, as rigid structures do not effectively dissipate wave energy. Backshore protection is often required to limit damage by wave overtopping.

Figure 3-2 Example of a Rigid Vertical Seawall

Photo credits: J. Carley

3.4 Semi-Rigid Structures

Semi-rigid structures (Figure 3-3) are able to move under wave loading, allowing some energy to be dissipated and for the structure to settle as the seabed or backshore changes form due to erosion or settlement. Semi-rigid structures are, therefore, often better suited to higher wave environments and to dynamic environments such as sandy beaches compared to rigid structures. Semi-rigid structures are generally sloped revetments and, therefore, use more space than rigid structures . Examples of semi-rigid structures include:

- rock revetments
- concrete armour unit revetments
- articulated blocks and blanket structures
- cut and stacked blocks
- sand-filled geotextile bags held under gravity.

Due to the flexibility of the outer layer, a filter layer is required to contain the fine land material behind. This filter may be smaller aggregate or a geotextile fabric, and it essentially forms the barrier between land and sea, with the armour providing protection to the filter from wave attack.

Figure 3-3 Semi-Rigid Geotextile Container at Manase, Samoa and a Rock Revetments on South Tarawa, Kiribati

Sources: Tom Shand (right).

3.5 Dynamic Shoreline Protection

Dynamic structures respond to the incoming waves, altering in shape to effectively absorb energy without compromising the integrity of the structure. Examples of dynamic protection include the following:

- Reshaping revetments, whereby rocks are mobile under wave attack and form a more stable profile (Figure 3-4)
- Sand replenishment—also referred to as beach nourishment—is the artificial addition of sand or gravel to the coast to improve the capacity of a beach to act as a buffer against storm erosion, coastal recession, or tidal inundation to protect the land behind.

Dynamic materials may continue to be moved over time with some expected losses from the system. Coastal protection, using dynamic materials, must therefore include sufficient material to protect against wave attack and gradual material loss over time. Rock and gravels are generally less mobile than sands, and require less ongoing maintenance and replenishment. Control structures, such as groynes and offshore structures, are also used to limit material loss from the system.

Figure 3-4 A stacked coral block wall in Kiribati collapses, forming a "Dynamically Stable Revetment"(left) and sand replenishment at Manase, Samoa (right)

3.6 Offshore Structures

Offshore structures protect the shoreline by reducing the wave energy arriving at the shore and rotating incoming wave crests. On a sandy coast, this can reduce longshore drift gradients and encourage sand deposition in the lee of the structure (Figure 3-5). Offshore structures may be emergent, partially-emergent, or submerged. Submerged and semi-submerged structures act by breaking or refracting the waves rather than absorbing or reflecting them to dissipate energy. While less visually intrusive, they are less effective than emergent structures, particularly during high water level and wave conditions that can result in beach erosion. Structures may be constructed from rock, pre-cast concrete armour units, or geosynthetic containers (GSC) and must be stable under wave attack while also reducing transmitted wave energy to a desirable level.

Figure 3-5 Salients Created in the Lee of Offshore Breakwaters at Manase Beach, Samoa

Source: Quilter, 2015.

3.7 Low-Cost Protection Works

The South Pacific Applied Geoscience Commission reviewed and discussed coastal protection measures in the South Pacific (SOPAC, 1994). It found that beach mining and the reclamation of shorefront land has exacerbated natural erosion processes. It established that conventional rubble mound structures have been used widely throughout the Pacific. These have consisted of basalt and granite where available (Samoa and Cook Islands), coral boulders in other locations, and some concrete armour units where deepwater protection is required. Some standard designs have been used, such as coastal road protection in Samoa, although many walls are based on rock availability rather than being formally designed.

Hand-placed rocks are widely used due to the ease of construction, although they often fail through undermining, overtopping, or the rock being undersized. Gabion baskets are similarly popular due to the relative ease of construction and availability of small rock. These have been relatively successful, especially when placed at the back of the beach where they are not frequently exposed to wave action, although once the wire coating is damaged and corrosion occurs, failure is rapid. Likewise sand and cement-filled bags are popular, although degradation of the fabric from UV exposure, abrasion from coral, and vandalism can occur, so it is suggested that use is restricted to temporary works.

Small pattern-placed armour units, such as Seabees, have reportedly been trialled on Onotoa, Kiribati. No results, however, have been reported. While these units are deemed effective and economic, they require care with foundation preparation, toe detailing, and unit placement to ensure satisfactory interlocking. Larger concrete armour units have been used and are restricted to deepwater locations where design waves are large.

SOPAC (1994) cited a lack of suitably sized materials as a major limitation in constructing conventional coastal protection structures, and where coral boulders or concrete units are used, physical modelling is generally required. Paeniu et al. (2015) presented a review of the typical coastal protection works used within different PICs, based on information provided by local stakeholders. They found that rock riprap seawalls are widespread in volcanic and coral islands, along with vertical concrete walls, grouted stone and sandbag walls, and gabion baskets. Other types of protection include rubber tyres, tree trunks, scrap metal and machinery, and drums filled with concrete. Concrete armour units have been used but are generally limited to ports or areas of high value. Examples of failed interventions are presented and comprise mostly of collapsed seawalls with apparent undermining, structural failure, and overtopping.

A desktop review was undertaken by PRIF (PRIF, 2017) to catalogue and critically evaluate the range of coastal protection methods used throughout the Pacific islands. Major issues identified with the commonly used solutions within PICs (Figure 3-6) include:

- use of local beach sand, exacerbating shore sediment deficit;
- use of low-strength and lightweight concrete, leading to structural failure;
- failure of structural members such as gabion wire;
- not extending the walls sufficiently deep to prevent scouring of the base of the wall and loss of material from behind;
- no use of geotextile (or other filter) behind the wall, resulting in loss of material when the wall cracks or is undermined;
- walls are under height or lack an upstand wall, allowing waves to overtop; and
- lack of backshore protection, resulting in land damage, among others.

It was identified that local approaches can be improved through use of alternative materials and design and construction methodologies to increase design life and improve hydraulic performance. Examples include:

- use of higher quality polyester geotextile bags rather than the low-cost woven polypropylene bags currently used;
- use of alternative bag placement and bonding patterns to improve hydraulic performance (this requires additional hydraulic testing to extend guidance);
- use of pre-cast blocks, rather than bags in grouted seawalls, to improve the unit material quality and the bond between units;

- use of more durable and robust gabion basket materials, subject to cost and design life guarantee by manufacturers;
- extension of structure toes to a firm substrate or below expected scour depth to prevent undermining and toe failure;
- use of a suitable geotextile behind structures to retain backshore soils, including with partial failure of a rigid structure; and
- extension of structures sufficiently high to prevent frequent overtopping events; or placement of stabilising materials, such as natural vegetation or armouring, within the overtopping zone.

Figure 3-6 Typical Failure Mechanisms

(A)

(B)

(C)

(D)

(A) Outflanking; (B) overtopping and loss of fines; (C) undermining and (D) structural failure

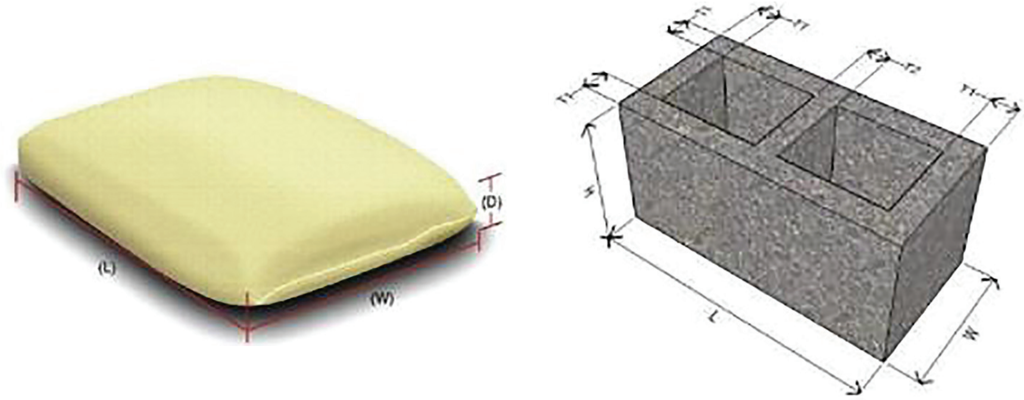
Photo credits: T. Shand, 2015.

The PRIF (2017) review identified alternative affordable coastal protection methods that have potential for use on low-energy coastlines, including smaller hand-placed and sand-filled GSCs and concrete masonry Besser® construction blocks (CMB), both placed on a sloping revetment. These innovative protection options have the benefit of being either widely available, having existing established supply chains, are cheaper to import, and/or can be placed without the need for heavy construction equipment. Without previous application or testing, however, there is little to no engineering design guidance available.

3.7.1 Physical Model Testing

A physical modelling study was undertaken to investigate the performance of these alternative coastal protection methods. The modelling program considered the stability and runup/overtopping characteristics of 40 kilogram sand-filled GSCs and concrete masonry blocks (Figure 3-7), when placed on a 1 (vertical):1.5 (horizontal) revetment slope. A range of placement configurations and wave conditions were investigated for armouring options to determine the threshold of unit stability, as well as wave runup characteristics.

Figure 3-7 Geotextile Bag and Concrete Masonry Block Dimensions



Source: Blacka, M., J. Carley, R. Cox, W. Hornsey, and S. Restall. 2007. Field Measurements of Full Sized Geocontainers. Proceedings of the Coasts and Ports Conference, Engineers Australia.

Figure 3-8 Model Concrete Masonry Block and Hand Placed Geotextile Container Revetment in Wave Flume

Photo credits: T. Shand.

The stability of concrete masonry blocks in four alternative placement configurations was tested, and for all wave periods modelled (3–10 seconds), the blocks were found to be stable in waves up to 1 metre significant wave height, including with up to 5% damage (blocks broken or displaced). With higher damage (10%), the revetment rapidly degraded. The revetment toe is critical to maintaining integrity and should be either founded on hard subsurface or beneath potential scour depths, or have a robust concrete or rock toe cap. Once the crest of the revetment was overtopped by waves (>1 linear/second/metre (l/s/m)), the upper courses of blocks became unstable. Therefore, such revetments should be designed as non-overtopped structures (q <1 l/s/m) or with a cap to hold the top layer in place. The structural strength or life expectancy of the blocks in the marine environment are unknown and must be tested by field trials.

The stability of small GSCs in two alternative placement configurations was tested, and for all wave periods modelled (3– 10 s), the containers were found to have a stability limit of approximately 0.4 metre (m) significant wave height. Waves in excess of this height resulted in rapid displacement of the containers from the revetment face slope. Only slight improvement was gained by placing GSCs with long axis offshore compared to the traditional along-shore placement. Therefore, such revetments are likely better suited to temporary works or in very low wave environments.

4 Optional Assessment

Once an erosion risk is identified, a decision should be made as to how the risk should be treated. If the decision is to protect the shoreline, a range of factors must be considered. The desktop review report (PRIF, 2017) presents a summary of frequently used structures and evaluates these against:

- engineering
- social
- environmental
- cost considerations.

4.1 Technical Considerations

Results of the technical analysis are presented in Table 4-2 and findings demonstrate that revetments constructed of conventional materials are the most effective at protecting land and they have typically long design lives. They are moderately complex to design and construct with the exception of geosynthetic containers and Seabees (depending on construction methodology), which requires a substantial construction plant. They are moderately resilient to climate change and can often be raised, although care needs to be taken that units are adequately designed for increased wave climate and height. Social effects are typically average to poor, without specific design consideration for access, although some methods, such as GSCs, provide reasonable coastal access. Environmental impacts are, likewise, average to poor as the natural system is being interrupted by a fixed structure with generally a large occupation area.

Conventional vertical structures are also moderately effective at protecting land, although they dissipate less wave energy, are more vulnerable to toe scour and overtopping, and have limited resilience to climate change, thus being difficult to raise or otherwise upgrade. They have poor social and environmental effects, restricting access to the shore (unless stairs or ramps are integrated) and may increase end-effect erosion through wave reflection, although they occupy a smaller area than revetment structures.

Low-cost solutions, using local materials, are typically simple and scalable with good opportunities for local labour. However, they typically have short design lives and limited effectiveness in protecting land. They can have poor environmental effects and may release material (e.g., sand bags, rock, tyres) into the marine environment as they deteriorate and fail or if they are inadequately designed. Some potential opportunities were found in using commonly available materials, such as concrete Besser® blocks in lower energy environments and by applying alternative placement configurations.

Ecologically based approaches, such as coastal planting and replenishment, tend to have the best environmental outcomes; however, beach replenishment is highly site specific and dependent on an available supply of appropriate replenishment material. Furthermore, design life can be short if erosion is ongoing and replenishment material is rapidly lost. Protection of land is not guaranteed with high water levels, often allowing erosion of the backshore despite replenishment. Replenishment is often combined with harder "backstop" protection structures to improve effectiveness while maintaining environmental benefits. Coastal planting can be moderately beneficial in the long-term as plants mature—particularly in dissipating overtopping flows which occur on an infrequent basis—and reducing wind-blown sand, although it is more limited in preventing erosion and continued loss of beach material, being subject to wave forces on a frequent basis. Furthermore, planting can restrict access and views of the coast for locals and tourists, thereby disconnecting people from the coast.

4.2 Material Availability

The geology of the Pacific islands is a mixture of dense volcanic rock and less dense sedimentary and coronus rocks. Neall and Trewick (2008) suggest five major processes are involved in the formation of islands (Figure 4-1):

(i) formation of active large shield volcanoes as a result of basaltic magmas rising through the lithosphere to the surface;

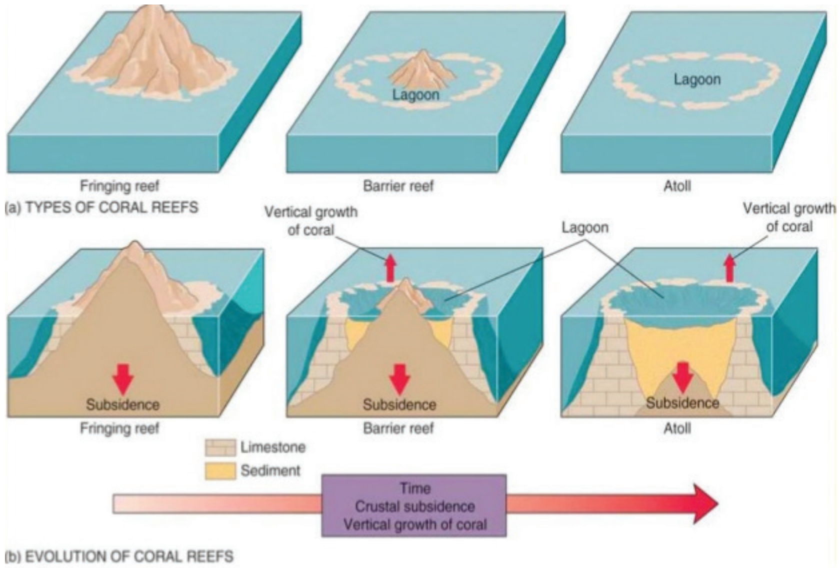
(ii) growth of coral around the volcano, forming fringing reefs;

(iii) gradual subsidence of the volcano, as it moves away from its area of generation with reefs moving progressively offshore to become barrier reefs;

(iv) complete subsidence, leading to development of atolls built on subsided volcanos; and

(v) further subsidence, leading to submerged seamounts.

Figure 4-1 Development Sequence of Coral Reefs



Source: Sumich, J.L. and J.F. Morrissey. 2009. Introduction to the Biology of Marine Life. Ninth edition, Jones and Bartlett Publishers.

PRIF (2015) describes the geology of PICs and indicates the available aggregate resources. Most PICs are volcanoes or have a central volcanic core where volcanic rock is found. This rock is a hard, well-cemented, massive volcanic breccia/rock strata (PRIF, 2015) well suited for use as armour rock. Coronus is coralline material that originates from uplifted coralline deposits, and coral rock originates from live or dead material from either fringing barrier or atoll reef formation. These coral-based aggregates are generally less dense, have a shorter design life, and are less well suited for armour rock compared with larger rock. Corals are available only in Kiribati, the Republic of the Marshall Islands, and Tuvalu. It should be noted that in countries where certain material is present, not every island will contain such material (e.g., the remote northern atolls of the Cooks Islands will not contain volcanic rock, while the southern islands do), and the availability of material will be dependent on social, technical, and regulatory criteria.

Selection of the most appropriate coastal protection method is highly dependent on the local availability of material. Where volcanic materials are present and available for use in sufficiently large sizes, rock revetments are likely to be the most technically robust and cost-efficient solution. On islands without such rock (including some islands within countries with volcanic material present), there are other protection materials that are potentially more efficient.

Table 4-1 Geological Description and Material Types Present in Pacific Island Countries

Country	Material Type			Geological Description
	Coral	Coronus/ Sedimentary	Volcanic	
Cook Islands	✓	✓	✓	Located in the central-southern Pacific Ocean, the Cook Islands form two distinct geographic groups. In the north are six coral atolls, while the southern islands are mostly volcanic in origin, usually with distinct central cores. Most have an elevated coral reef platform adjacent to the coast, as well as recent coral reefs.
Fiji	✓	✓	✓	Located in the central Pacific Ocean, Fiji includes more than 320 islands, islets, and reefs. The two main islands are ruggedly mountainous, made up of volcanic and sedimentary rock with limited alluvial plains, uplifted limestone, and raised shorelines, with extensive coral reefs in shallow areas.
Kiribati	✓	×	×	Kiribati includes three island groups which lie across the Equator. Apart from Banaba, which rises to 80 metres above sea level, the islands are low-lying coral atolls, often enclosing a central lagoon. The thin layer of sandy coral supports only sparse vegetation.
Marshall Islands, Republic of (RMI)	✓	×	×	RMI has scattered, low-lying coral atolls forming the eastern-most group of the Micronesian archipelago. Some atolls enclose very large lagoons.
Micronesia, Federated States of (FSM)	✓	✓	✓	Located in the west-central Pacific Ocean, FSM has more than 600 tiny islands and atolls. There is a mixture of mountainous islands of volcanic origin, low coral atolls, and isolated reefs.
Nauru	✓	✓	×	A single island in the southern Pacific Ocean, Nauru is an uplifted coral limestone atoll, with a terraced rim containing caves and sinkholes, and an inland plateau of phosphate bearing rock.
Niue	✓	✓	✓	A raised atoll southeast of Samoa, Niue has its former reef and lagoon uplifted to about 60 metres above sea level. The central plateau in the middle of the island is edged with steep slopes. A coral reef fringes parts of the coastline.
Palau	✓	×	✓	Palau is an archipelago of approximately 340 islands in the northwest Pacific Ocean. Only nine of them are inhabited. There are two volcanic islands with high centres, although most of the remaining islands are raised coral atolls.
Papua New Guinea (PNG)	✓	✓	✓	Located just below the Equator in the western South Pacific Ocean, PNG has 600 islands and coral atolls which are mostly of younger volcanic origin, although the mainland is a massive rugged cordillera (i.e., Central Highlands) with wide and very fertile alpine valleys and ice-capped peaks.
Samoa	✓	✓	✓	Located to the west of American Samoa, Samoa has two large islands and six smaller islets formed from volcanic cones, with several peaks and deeply eroded canyons. Coastal beaches ring the main islands.
Solomon Islands	✓	✓	✓	Located southeast of Bougainville (Papua New Guinea), the Solomon Islands are a series of high, rugged islands located along a northwest/southeast trending fault system, with some raised coral reefs. Soils range from extremely rich volcanic to relatively infertile coral limestone.
Timor-Leste	✓	×	✓	Timor-Leste is part of the island of Timor, the largest and eastern-most of the Lesser Sunda Islands. Most of the country is mountainous.
Tonga	✓	✓	✓	Tonga has 169 islands in an archipelago in two almost parallel chains. The eastern islands consist of low coral islands, with a covering of volcanic ash. The western islands consist of tall, recently formed volcanic islands.
Tuvalu	✓	×	×	Located north of Fiji and south of the Equator, the islands and atolls of Tuvalu are of coral formation and very low lying.
Vanuatu	✓	✓	✓	The young volcanic islands of Vanuatu, some of which are still active, were formed from belts of older sedimentary rock which were repeatedly uplifted.

Source: Brij, V.L., & Fortune, K. (Eds). 2000. The Pacific Island: An Encyclopaedia. Honolulu: University of Hawai'i Press.

4.3 Transport Costs

Transport is a major component of coastal protection costs at remote locations. Transport can occur by road across land masses, although this is typically less than 50-100 kilometres (km) in PICs due to their small size. Road transport costs are usually in the range of A$0.50 to $1.00/cubic metre ($m^3$)/km; however, road conditions can be poor and travel times high.

Scheduled container shipping runs between major ports. Shipping containers are typically capable of transporting 18-20 tonnes of material (up to 33 m^3 by volume). Shipping costs depend on the specific ports but generally range from A$3000 to A$6000 per container.[1] Costs such as taxes and duty are additional.

For transport to remote locations without scheduled shipping or transporting large shipments of bulk cargo, such as armour rock, chartered barges may be required or may be the most cost-effective option. Where no docking facilities are available at remote locations, barges with roll-on/roll-off capability are generally required, or cargo must be transferred to smaller local boats, which takes time and incurs high costs. The following approximate costs for transport are assumed:

Base: Material is produced locally and transported by road within 30 km. An example would be Suva, Fiji, where cement is produced locally and good quality volcanic aggregate is available.

Local transport: Local transport within 200 km by road or barge, including one handling. Assume A$150/$m^3$.

Primary port: Loaded at a primary port, transported up to 3,000 km, unloaded and transported locally to site. An example is South Tarawa, Kiribati. Based on typical freight costs, assume a cost of A$500/$m^3$, although this is likely to fluctuate with location and local import tax and duty.

Remote location: Loaded barge at primary port, transported up to 2,000 km, and unloaded at wharf, jetty, or directly onto land using a ramp. Mechanical plant is typically required to facilitate the offload. Assume A$1,000/$m^3$, based on typical barge hire rates.

1 This is based on personal communications with Go Logistics NZ Ltd. in December 2015.

Table 4-2 Results of Technical Analysis

Technical Criteria		Rating			Revetments							Vertical Seawalls				Offshore		Low Cost or Local Materials							Ecosystem-Based ApproachesA			
		1	3	5	Rock revetment	Geo-containers	Articulating concrete blocks	Tetrapod armour units	Samoa Stone	Seabees	COPED	Mass concrete	Reinforced concrete	Timber retaining wall	Sheet pile wall	Sand saver	Reef Balls	Grout-filled sand bags	Gabion baskets and mattresses	Grouted rock	Stacked coral	Rubber tyres	Concrete pipes	Filled drums	Beach replenishment	Brush structures	Biorock	Planting mangroves and vegetation
Engineering	Design wave characteristics	Single wave height/ period	Range of heights	All heights and periods up to design height	5	3	2	3	4	4	3	4	3	2	2	1	3	1	1	2	1	1	2	1	4	1	3	1
	Design life	<2 years	5-20 years	>50 years	5	3	3	4	4	4	4	4	4	3	3	3	3	2	2	2	1	1	2	1	2	1	3	4
	Time period to become effective	> 2 years	Within 2 years	Immediate	5	5	5	5	4	4	5	5	5	5	5	3	2	5	5	5	5	5	5	5	4	2	1	1
	Effectiveness at protecting land	Limited protection of backing land, or often fails	Moderate protection, or sometimes fails	High level of protection of backing land reported in all cases	5	4	4	5	5	5	3	4	4	3	3	2	2	2	3	2	1	1	1	1	3	1	1	2
	Effect on overtopping	Large overtopping volumes or runup level high	Moderate overtopping volumes	Low overtopping volumes or decrease in runup	4	2	3	5	4	3	3	1	1	1	1	1	3	1	3	2	3	3	1	1	5	3	3	4
	Toe scour	High toe scour occurs	Moderate toe scour	Low levels of toe scour occur	3	2	3	3	3	3	3	1	1	1	1	3	5	1	2	1	3	2	1	1	5	3	3	4
	Design guidance available	None	Some	Complete	5	4	3	4	4	5	2	4	4	4	3	3	2	2	3	2	1	1	1	1	4	1	3	2
	Resilience to climate change	No adaptation—replacement required	Modification required	Provision for a specified level of climate change	3	2	2	3	2	3	3	1	1	1	1	2	2	1	1	1	1	1	2	2	4	4	5	4
	Construction complexity	Requires international contractor	Requires local contractor	Semi-skilled or unskilled local labour	3	3	3	2	2	3	2	3	2	3	3	4	2	4	4	4	5	4	3	4	3	5	2	5
	Construction plant required	Large and/or expensive plant required	Some construction plant required	No construction plant required	2	4	3	2	2	3	2	4	4	3	2	3	2	4	5	4	5	3	3	4	3	5	4	5
	Scalability	Highly site-specific	Site-specific modification required	Very generic	4	4	2	2	3	3	2	3	2	3	3	3	2	4	4	4	4	3	3	3	1	4	2	4
Social	Use of local labour	Local labour cannot be used	Some local labour	May be completed using local labour	2	3	3	2	2	4	2	3	3	3	2	2	3	4	4	4	5	3	3	4	2	5	3	5
	Beach access	Prohibits access	Access unchanged	Enhances access	3	4	4	2	4	4	2	2	1	2	1	4	3	2	3	2	2	1	1	1	5	2	4	2
	Aesthetic	Significantly differs from existing	Slightly differs from existing	In keeping with existing environment	3	3	3	2	3	3	2	1	1	3	1	3	3	2	3	3	3	1	1	1	5	2	5	2
	Cultural acceptability	Never used	Sometimes used already	Already widely used	4	3	3	2	3	3	3	2	2	2	1	3	3	3	3	3	4	1	1	1	5	4	5	4
Environmental	Occupation of seabed	Large occupation area (<3 x design wave height)	Moderate occupation area (1-2 x design wave height)	Small occupation area (less than 1 x design wave height)	1	2	1	1	2	2	1	4	4	5	5	3	1	3	3	4	4	4	4	4	1	4	1	1
	End effects	Enhances erosion of adjacent land	Neutral	Reduces erosion of adjacent land	3	3	3	3	3	3	3	1	1	2	1	3	4	2	2	2	3	2	2	2	5	3	3	4
	Effect on sediment budget	Depletes sediment budget	Neutral	Enhances sediment budget	2	2	2	3	3	3	3	2	2	2	2	4	4	2	2	2	2	3	2	2	4	5	5	4
	Effect on ecosystems	Significant adverse effect	Neutral	Significantly improves ecosystems	3	2	3	2	2	2	2	1	1	1	1	2	4	1	2	1	2	2	1	1	4	3	5	5
	Impact of construction activities	Significant and/or long-term adverse effects	Some and/or short-medium-term effects	Negligible adverse impacts	4	3	4	4	4	4	4	4	4	3	2	4	4	2	2	2	2	2	3	2	4	4	4	5

4.4 Cost Considerations

Results of the cost analysis are presented in the desktop review report (PRIF, 2017). The methodology employed is to:

(i) determine typical costs of materials used for the construction of coastal protection works;

(ii) undertake generic designs and costing, assuming all materials are available locally (Table 4-3);

(iii) determine typical transport costs for a range of scenarios;

(iv) determine cost of protection options incorporating transport costs (where necessary); some options include the use of local materials and, therefore, transport costs are not added in these cases;

(v) convert to a cost/year basis on a typical (well-contracted) design life of the specific option;

(vi) convert resultant costs/years to relative costs compared to a locally produced rock revetment to assess the most cost efficient protection options for transport and wave height scenarios (Table 4-4).

Conclusions of the analysis were as follows:

- While hand-placed sand bags have the lowest initial capital cost, they are limited in their design life and wave height. Alternative bag materials, however, may provide longer design lives, making them more attractive options for temporary works and remote locations.
- Rock (where locally available) has the lowest annual cost, with higher density volcanic rock requiring smaller rock (and, therefore, lower seawall volumes and cost) than the lower density limestone and coronus material.
- Where rock is locally unavailable and must be transported, the initial capital cost of rock revetments increases substantially, although annual cost remains lower than many shorter design life local options.
- Low cost, local solutions often have low to moderate initial capital cost that does not increase substantially with remoteness, as most materials are available locally. Short design lives (typically, two to ten years), however, substantially increase the annual cost and whole-of-life cost.
- Small hand-placed concrete armour units, such as Seabees, are typically two to three times more expensive than rock (where rock is available locally) although, being lower in volume, become more cost-efficient as transport costs increase. Furthermore, the larger the design wave height, the lower the transport cost, whereby such units become cost effective.
- Large geosynthetic containers are more expensive where rock is locally available; however, due to relatively low transport costs, they become less expensive in remote locations, comparable with single-layer armour units, although the shorter design lives increase annual cost.
- Beach replenishment costs are highly dependent on material availability (affecting capital cost) and ongoing material loss (which affects design life). Where a low-cost supply of sand or gravel is available and ongoing losses are not likely to be high (or control structures are used to extend the life), such approaches may be cost effective compared to other methods, particularly in remote locations.
- CMBs are relatively inexpensive to construct, and transport costs add moderately to cost. Their modular nature and established supply chains, however, are likely to reduce transport costs below alternative materials. Furthermore, their design life is unknown and will drastically affect their annualised cost.

Table 4-3 Indicative Cost/Metre (Linear) for Coastal Protection Works, Assuming Local Materials

Protection Method	Details	Design Life[2] (years)	A$/Metre for Low Wave Energy (H_s = 0.7 metre)	Moderate Wave Energy (H_s = 1.5 metre)	A$/Metre for High Wave Energy (H_s = 3 metres)
1a. Rock revetment–high density	Assumes basalt or similar >2600 kilogram (kg)/cubic metre (m^3)	50	675	3,000	10,700
1b. Rock revetment–low density	Assumes limestone, coral or similar) ~2200 kg/m^3	30	850	4,200	N/A
2. Mass concrete	Assumes local aggregates are used	30	2,500	10,000	N/A
3. Reinforced concrete	High strength (50 Megapascal (Mpa)) marine-grade concrete	25	1,700	6,700	N/A
4. Grout-filled bag wall	Bags secured with a grout mix	5	950	N/A	N/A
5a. Geosynthetic container–1 layer	Assumes 0.75 m^3 containers for low wave and 2.5 m^3 for moderate wave	10	1,900	3,900	N/A
5b. Geosynthetic container–2 layers		20	3,350	7,100	N/A
6a. Seabees–imported materials	Includes concrete cap and rock toe	25	1,200	3,300	12,500
7a. Tetrapods–imported concrete	Includes rock toe	30	N/A	5,100	31,000
8. Grouted coral wall	Assumes 1:3 ratio concrete:coral block	10	900	N/A	N/A
9. Beach replenishment	Assumes 1:12 slope and 20% loss of material/year	5	1,000	4,200	17,500
10. Timber wall	Assumes piles driven and H6 Marine grade timber	15	2,400	N/A	N/A
11. Gabion basket	Assumes local aggregates and PVC coated wire	7	650	N/A	N/A
12. Terrafix® blocks	Assume T60 blocks	15	1,300	N/A	N/A
13. Small hand-placed bags	Assumes good quality polyester geotextile	24	3,505	N/A	N/A
14. Small hand-placed blocks	390 x 190 x 190 mm concrete masonry blocks	54	200	N/A	N/A

Notes:

(i) Costs are indicative for comparative purposes only and should not be used for project costing;
(ii) design life assumes typical term of effectiveness in Pacific environment with no or minimal maintenance;
(iii) N/A indicated method is not suitable for that wave climate;
(iv) design life is unknown and is subject to field trial; and
(v) small, hand-placed sand-filled geotextile container only suitable for H_s <0.4 m.

Table 4-4 Relative Cost/Year for Low Wave Environment

Protection Option	Design Life (years)	Costs/Year (proportion of local rock revetment)			
		Base	Local	Primary Port	Remote Location
1a. Rock revetment—volcanic	50	1.0	2.1	4.6	8.2
1b. Rock revetment—limestone	30	2.1	2.8	4.3	6.6
2. Mass concrete—local concrete	30	6.1	6.5	7.4	8.7
3. Reinforced concrete	25	5.0	5.7	7.5	10.0
4. Grout-filled bag wall	5	13.9	15.3	18.4	22.8
5a. Geocontainer—single layer	10	13.9	14.5	15.9	17.9
5b. Geocontainer—double layer	20	12.4	13.0	14.1	15.8
6a. Seabees—imported materials	25	2.7	3.6	5.7	8.8
6b. Seabees—local materials	15	6.2	6.3	6.7	7.3
8. Grouted coral	10	6.6	6.9	7.7	8.7
9. Beach replenishment	5	14.8	14.8	14.8	14.8
10. Timber wall	15	11.9	13.6	17.8	23.7
11. Gabion basket	7	6.9	8.5	10.2	12.0
12. Terrafix® blocks	15	6.5	7.3	9.1	11.8
13. Small hand-placed sandbags	21	12.7	13.7	16.0	19.4
14. Small hand-placed blocks	51	3.1	5.0	9.5	15.9

Notes: (i) Insufficient information available on design life of new methods; and (ii) conservative values used and field trials required.

4.5 Comparison Methodology

Selection of an appropriate option is generally not as simple as selection of the lowest cost (capital or annual); rather, it will include trade-offs with social and environmental impacts and between capital cost and design life. An approach may be as follows:

(i) apply a rapid screening of technical suitability, using information such as provided in Table 4-2, to identify feasible options and likely environmental and social impacts;

(ii) estimate the cost, based on wave height, material availability, transport cost and design life (PRIF, 2017);

(iii) determine the whole-of-life cost or annual cost, based on design life;

(iv) assess suitability for site, based on technical and cost considerations; and

(v) rank options in consultation with wider stakeholder group.

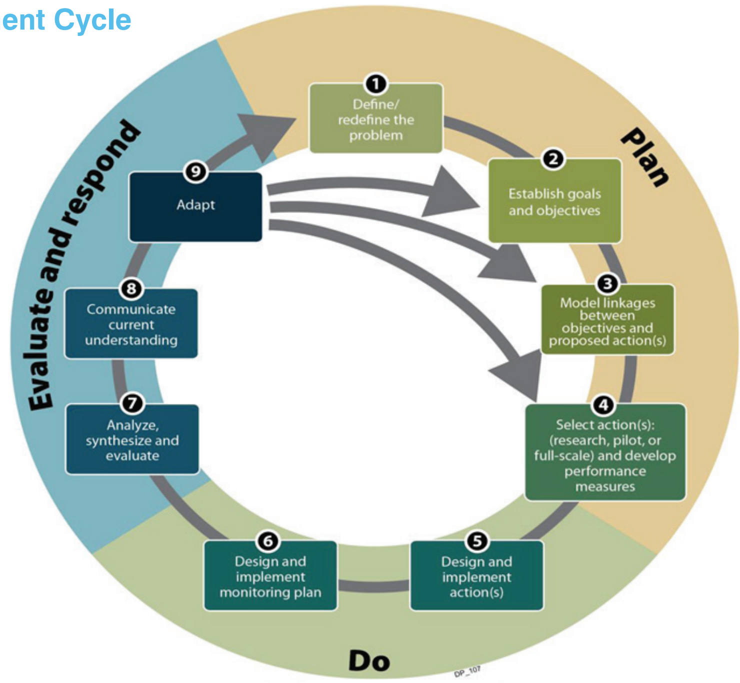
Table 4-5 Example of Table Used for Comparison and Selection of Options

Option Rank	Protection Option	Social Impact	Environmental Impact	Estmated Capital Cost ($)	Design Life (years)	Annual Cost (A$/year)	Suitability for Site
1							
2							
3							

4.6 Adaptive Management

Adaptive management follows the philosophy that in the natural environment, information on which to base decisions is uncertain or incomplete. By defining a problem and establishing goals and objectives, the design and implementation of the most appropriate approach can be undertaken, based on the best information available. By monitoring and evaluating outcomes, the initial plan can be adapted, going forward, in response to observed conditions that initially were rather uncertain (Figure 4-2). This "learning by doing" approach works well, provided ongoing monitoring and evaluation is undertaken. It is not suited, however, for "one-off investment" type projects.

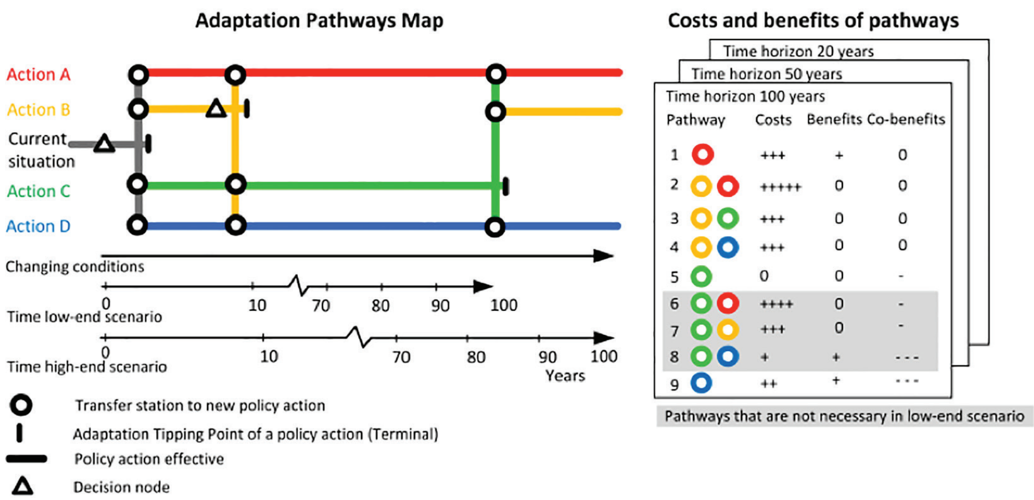
Figure 4-2 Adaptive Management Cycle



Source: What is California EcoRestore? Adaptive Management. http://resources.ca.gov/ecorestore/what-is-california-ecorestore

Given the uncertainty in future climate change and its effect on the natural and built environment, adaptive management techniques are particularly well suited. One example of this is the dynamic adaptation pathways model (Figure 4-3), where multiple possible actions are identified (e.g., managed retreat, sand replenishment, hard protection) and the costs and benefits of different pathways, incorporating one or more actions, are evaluated. Trigger points are set that would necessitate a new action, with recognition given to the fact that the timing of this change in approach is governed by ambiguity in climate change predictions.

Figure 4-3 Example of an Adaptation Pathways Map and Scorecard for Each Pathway



Source: https://www.deltares.nl/en/adaptive-pathways/

5 Assessment of Design Conditions

5.1 Datum and Coordinate System

Vertical levels should be referenced to a specific land-based datum to allow an accurate setout of design to a particular "Reduced Level". Often, such land data are based on mean sea level at a particular time or based on a tide gauge (e.g., in Kiribati the SEAFRAME[2] Tide Gauge Zero 1992, STGZ92, is often used as a land datum). Alternatively, a local datum can be established. All site surveys and water level calculations should be referenced to this datum or reduced level.

Defining a coordinate system to be used in the design is also essential, and may be based on the World Geodetic System (WGS 84; degree latitude/long), a Transverse Mercator (kilometre x, y), or a local coordinate system (x,y from a particular benchmark). Drawings and setout points should be provided, based on this system.

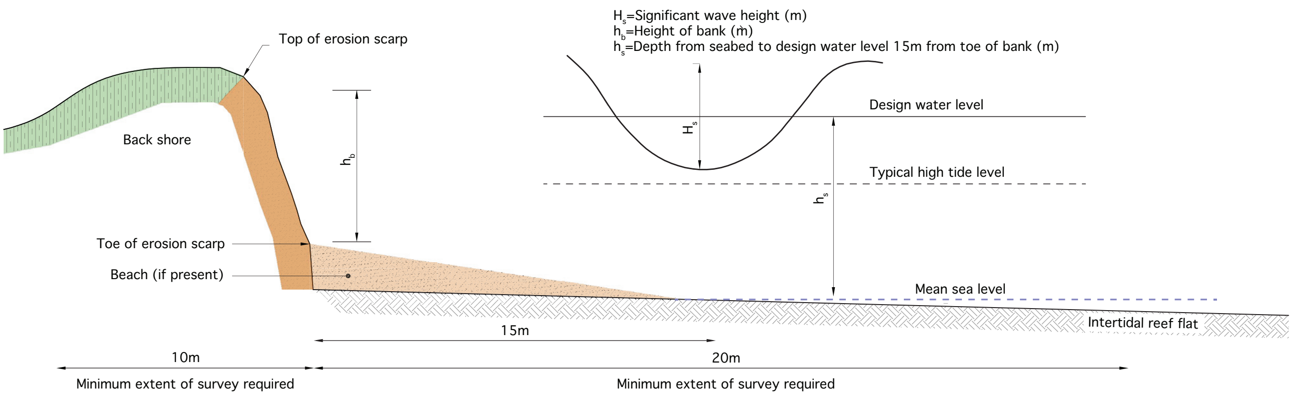
5.2 Site Survey

Topographic surveys can be undertaken, using ground-based methods (e.g., dumpy level, theodolite or global positioning system (GPS)) or aerial survey methods (e.g., photogrammetry or Lidar). While aerial-based surveys can cover a wider area more swiftly, they must be accurately corrected to ground level, using surveyed "ground control points".

Surveys should extend at least 10 m landward of the shoreline to 20 m seaward of the shoreline and at least 50 m either side of the anticipated shoreline extant. A topographic survey should pick up (at a minimum):

- changes in grade, including top and toe of bank, dune, or scarp;
- transition of sand to reef;
- location of man-made structures, including roads, stairs, utility pipes and chambers, and existing coastal protection works; and
- the location of debris or high tide mark.

Figure 5-1 Survey Definition Sketch



Note: m = metre.

A bathymetric survey is undertaken from a boat and utilises a sonar or mechanical measurement to determine depths to seabed. Bathymetric surveys are not often required for coastal protection works where the reef or lagoon is exposed at low tide, but may be required for coastal protection in deeper environments. Such works generally will require a level of design outside the scope of this study, and professional assistance should be sought.

2 Defined as SEA-Level Fine Resolution Acoustic Measuring Equipment.

5.3 Ground Conditions

The ground conditions are important in determining the most appropriate protection type as some methods, such as rigid structures, are ill-suited to sandy environments without a firm base. Important factors include the surface material and extants (i.e., silty, sandy, or rocky; and does it transition further down the beach), depth to reef (i.e., if sandy, and reef exists at depth), or changes in soil firmness with depth. The ground conditions can be assessed using:

- observation by an experienced practitioner;
- Scala (or Dynamic Cone) penetrometer—observe number of blows to penetrate in 50 millimetre (mm) intervals;
- hand auger or machine borehole—observe excavated material; and
- test pit using mechanical or manual digging methods.

5.4 Water Level

The water level, at any time, is determined by the combination of several components including:

- astronomical tides;
- barometric and wind effects, generally referred to as storm surge;
- medium-term fluctuations, including El Niño–Southern Oscillation (ENSO) and Interdecadal Pacific Oscillation (IPO) effects;
- inputs from catchment flows;
- long-term changes in sea level;
- long-term changes in land level (tectonic movement and subsidence); and
- wave breaking, which can also contribute to water level through wave setup and runup.

Design flood levels should also incorporate a freeboard. This accounts for uncertainties in prediction and other local effects.

5.4.1 Mean Water Levels

Shorter-period fluctuations, such as tides and meteorological effects, fluctuate around a mean water level. As described above, this mean water level is also likely to change over time, and its exact value will depend on the period the level is averaged over. The mean sea level, at a certain point in time, is often adopted as a land datum although, over time, this datum is likely to deviate from the existing mean level.

Since 1993, sea level measurements have been continuously recorded by the SEAFRAME tide gauges on the Pacific Islands (BoM, 2011). The tide gauge and satellite altimeter data show an increasing trend of the mean water level from 1993 to 2010 for the majority of PICs.

5.4.2 Tides

Astronomical tide is the periodic rise and fall of the level of the sea surface, caused by the gravitational interaction of the earth, sun, and moon on the Earth's waters. Tides within the southwest Pacific basin are semi-diurnal, with a typical tidal range (difference between high and low waters) from 1 m to 2.5 m (Figure 5-2).

It is important to determine the mean sea level and approximate mean high water level for a site to inform design. Methods to enable this (from most accurate to least) include:

(i) Deployment of water level measuring instrumentation for a period of at least 28 days and tidal harmonic analysis to determine local tidal levels. Instruments should be surveyed into a local benchmark to allow the water levels to be tied to topographic survey levels.

(ii) Survey from a known benchmark, allowing translation from a nearby tidal gauge.

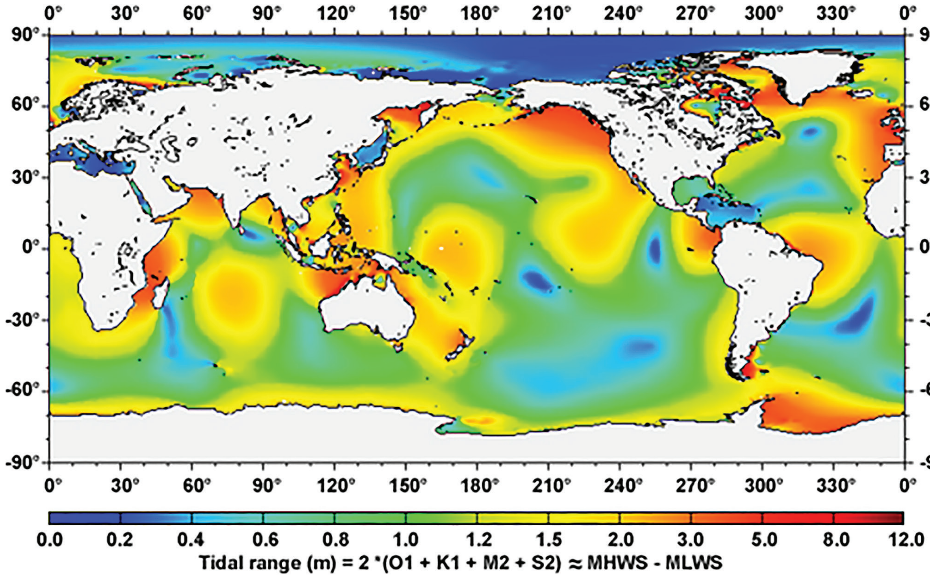
(iii) Precise GPS measurement, allowing comparison with nearby tidal gauge.

(iv) Measurement of recent high water marks compared to a land benchmark. Note that local wave effects should be considered in any swash measurement.

Selection of an appropriate method will depend on the stage of design (preliminary option assessment through to detailed design) and the value/importance of the works. Sources of tidal information include:

- Admiralty charts for a number of primary and secondary ports (e.g., Table 5-1);
- SEAFRAME Sea Level Monitoring Project;[3] and
- Numerical and tidal forecasts, although some tidal analysis may be required.[4]

Figure 5-2 Global Tidal Range



Source: National Tidal Centre, Australian Bureau of Meteorology.

Table 5-1 Example of Tidal Levels in Suva, Fiji

Tide State	Suva, Fiji (metre Tide Gauge Zero)
Highest Astronomical Tide (HAT)	2.1
Mean High Water Springs (MHWS)	1.8
Mean High Water Neaps (MHWN)	1.6
Mean Sea Level (MSL)	1.1
Mean Low Water Neaps (MLWN)	0.6
Mean Low Water Springs (MLWS)	0.4
Chart Datum at Suva (CD)	0.1

Source: UK Admiralty Tide Tables, 2009.

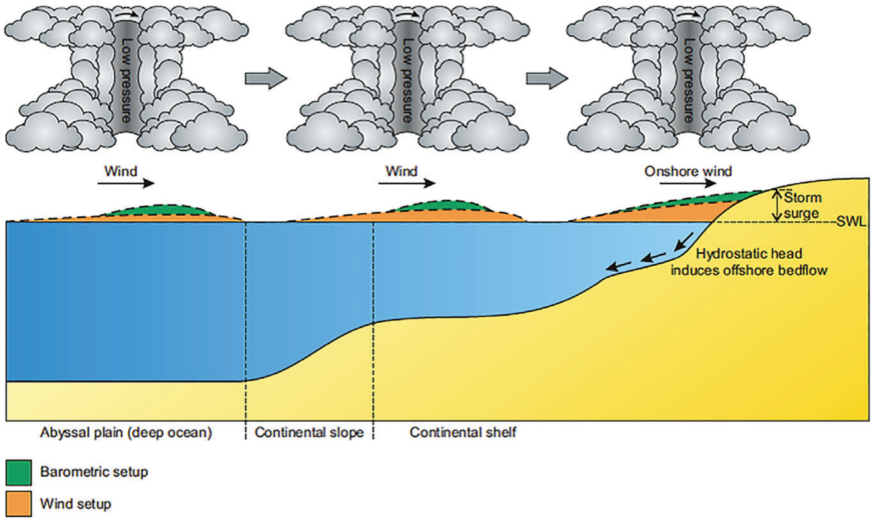
5.4.3 Storm Surge

Storm surge results from the combination of barometric setup from low atmospheric pressure and wind stress from winds blowing along or onshore, which elevates the water level above the predicted tide (Figure 5-3). Cyclones are particularly effective at generating storm surge due to their very low central pressure and high winds; however, their small size means that the cyclone must pass very close to the observation point for the surge to be significant. Additionally, storm surge is amplified in shallow coastal waters and within embayments, implying that islands surrounded by relatively deep water are less vulnerable to large surge heights.

3 See http://www.bom.gov.au/oceanography/projects/spslcmp/data/index.shtml

4 See https://tidesandcurrents.noaa.gov/tide_predictions.html

Figure 5-3 Processes Contributing to Storm Surge



Source: Shand, T.D., J.T. Carley, G.P. Smith and W.L. Peirson. 2010. Review of Storm Surge and Coastal Inundation Modelling. Technical Report 2010/18, University of New South Wales Water Research Laboratory.

The combined elevation of the predicted tide, climatic cycles, and storm surge is known as storm tide. The elevation of the storm tide at any particular island group will depend on the mean water level at the time, the astronomical tidal level, the magnitude and proximity of a cyclonic system, and the strength and direction of cyclonic winds. Generally, storm tide levels are assessed in terms of their return period or average annual recurrence interval (ARI).

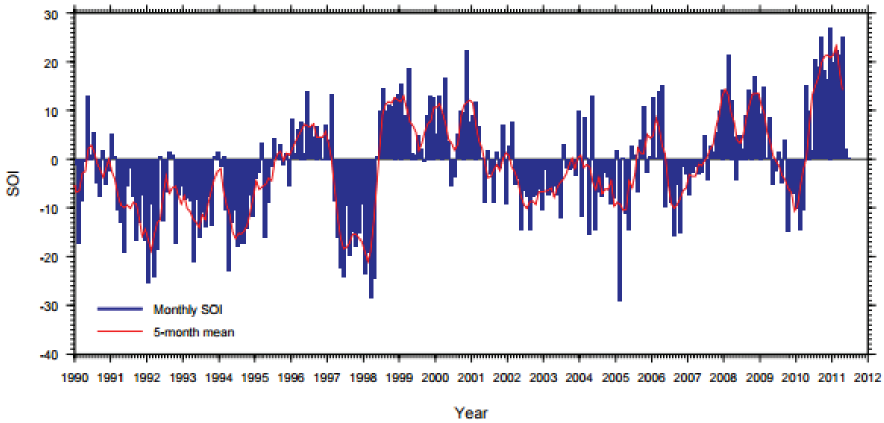
The assessed storm tide levels, therefore, may range from one-year ARI for a relatively frequent event, such as a tropical storm or depression, to 100-year ARI for a Category 1-3 storms (depending on location) to >1,000-year ARI for intense Category 4-5 storms. Selection of an appropriate event for a design water level will depend on the type of structure constructed (Section 5.1).

5.4.4 Annual to Decadal Fluctuations

The principal driver of annual to decadal fluctuation within the Pacific is the ENSO, measured using the Southern Oscillation Index (SOI) (Figure 5-4). El Niño's impact on sea level is mostly felt along the South Pacific Convergence Zone due to changes in the strength and position of the Trade Winds—which have a direct bearing on sea level—and along the Equator due to related changes in ocean currents. These impacts affect virtually every aspect of oceanic and climatic fluctuations, including sea level, winds, precipitation, and air and water temperature.

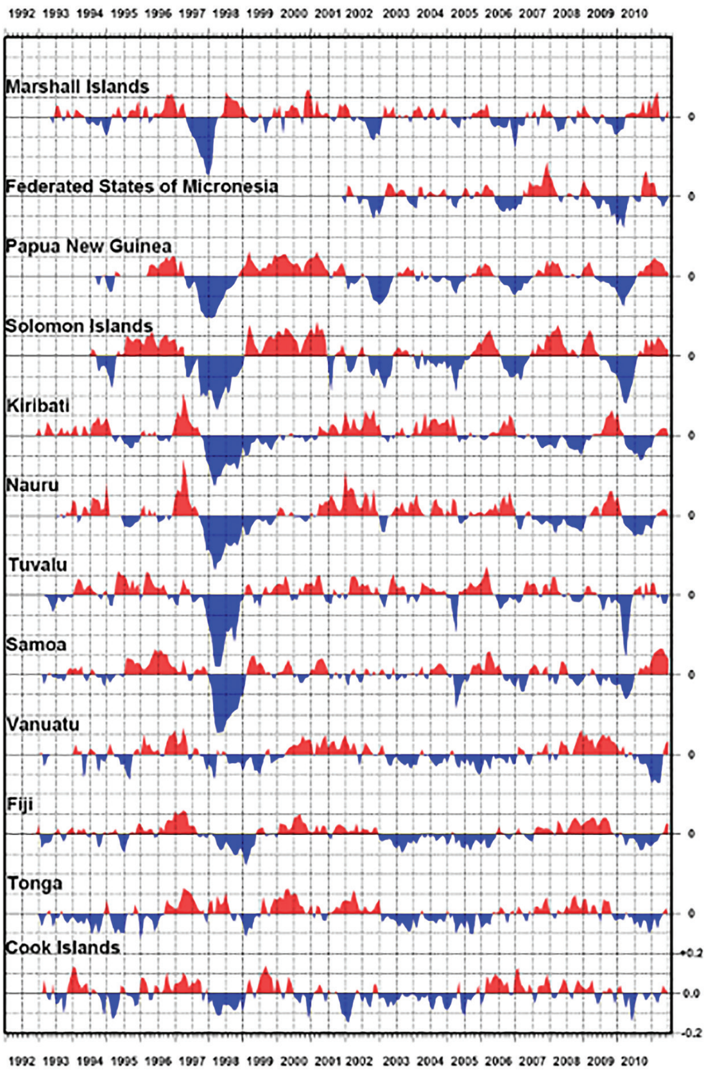
As part of the AusAID-sponsored South Pacific Sea Level and Climate Monitoring Project (Pacific Project) for the Forum region, SEAFRAME gauges were installed in 12 locations around the Pacific. Figure 5-5 illustrates the monthly mean sea level anomalies derived from removing tidal levels from the measured data. The mean sea level variation across years can vary by up to 0.3 m, with most sites showing the lowest recorded sea levels during the 1997/1998 El Niño years.

Figure 5-4 Southern Oscillation Index



Source: BoM (2012) South Pacific Sea Level and Climate Monitoring Project: Sea Level Data Summary Report, July 2010 to June 2011

Figure 5-5 Monthly Mean Sea Level Anomalies, 1992-2011



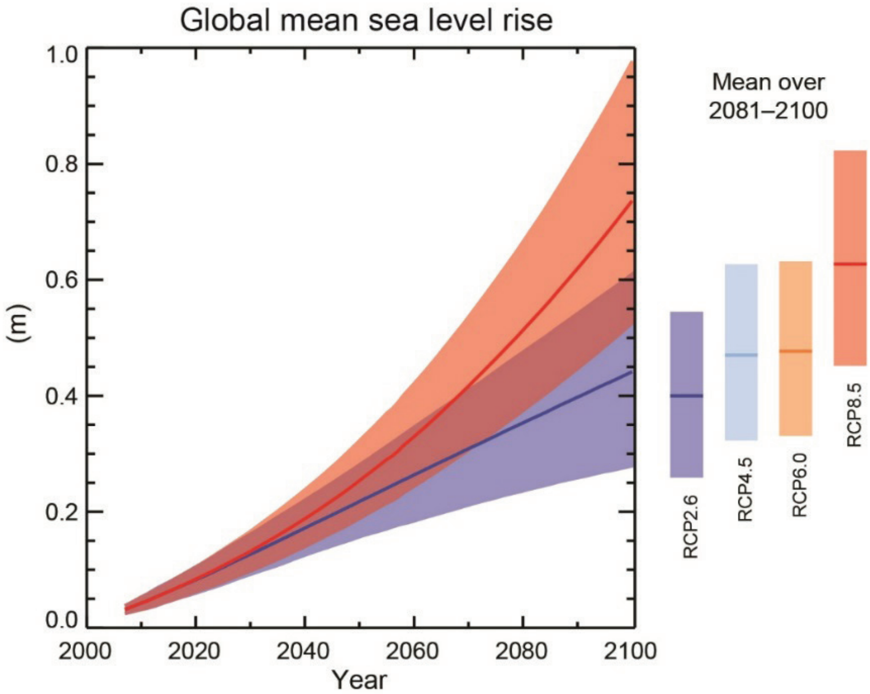
Source: BoM (2012) South Pacific Sea Level and Climate Monitoring Project: Sea Level Data Summary Report, July 2010 to June 2011

5.4.5 Sea Level Rise

The mean level of the sea has been rising over the last decades, with the Australian Bureau of Meteorology observing trends of relative sea levels ranging from 3.6 mm/year to 17 mm/year between 1993 and 2010 across the southwest Pacific, based on SEAFRAME tide gauge data. This is higher than the global average of sea level rise (SLR) of 3.3 mm/year over the same period (Cazenave and Llovel, 2010) and likely indicates tectonic movement, as well as a rise in the actual mean sea level.

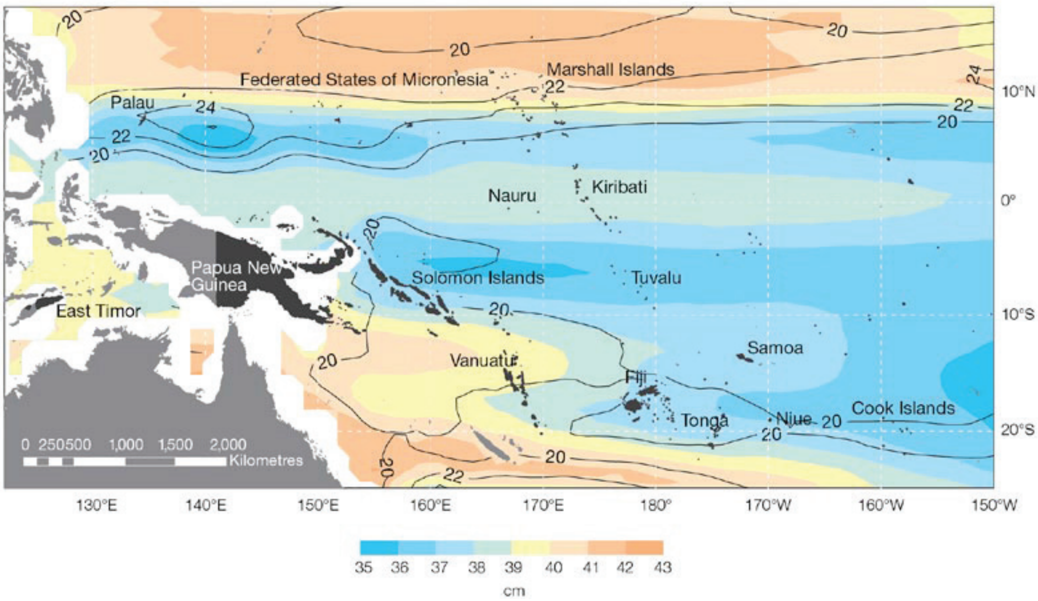
Modelling, presented within the most recent report (AR5) of the Intergovernmental Panel on Climate Change (IPCC) (IPCC, 2014), shows projects global SLR values by 2100 to range from 0.27 m to 1 m, depending on the emission scenario adopted (see Figure 5-6). Based on recent rates of SLR, those within the Pacific could be higher than this global average projection. Within the Pacific, projections of SLR also vary, with Figure 5-7 showing sea level projections for the "A1B" scenario (based on IPCC AR4 modelling) for 2081-2100 to vary by up to 10 centimetres.

Figure 5-6 Projections of Potential Future Sea Level Rise Presented within the Fifth Assessment Report of the Intergovernmental Panel on Climate Change



Source: IPCC. 2014. Climate Change 2014: Synthesis Report. Contribution of Working Groups I, II, and III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change. Geneva: IPCC.

Figure 5-7 Sea-Level Projections for the A1B (Medium) Emissions Scenario in the Pacific Climate Change Science Program for the Pacific Island Region for 2081-2100* (in centimetres)



* *Relative to 1981-2000, based on the Fourth Assessment Report of the Intergovernmental Panel on Climate Change and indicated by shading.*

5.5 Wave Climate

As winds blow over a water surface, energy is transferred into the water column to form waves. There are typically four sources of waves in the Pacific:

(i) Waves generated locally within lagoons. These waves will typically be "'fetch-limited" or limited in height by the distance that wind can blow over. In larger lagoons (e.g., Tarawa, Kiribati), these waves may be over 1 m high with periods of 3 to 5 s, but are typically less than 0.5 m with periods of 1-3 s.

(ii) Wind sea waves associated with local trade winds. Waves are typically less than 2 m with less than 10 s periods. These waves affect all Pacific islands between +30° and -30°.

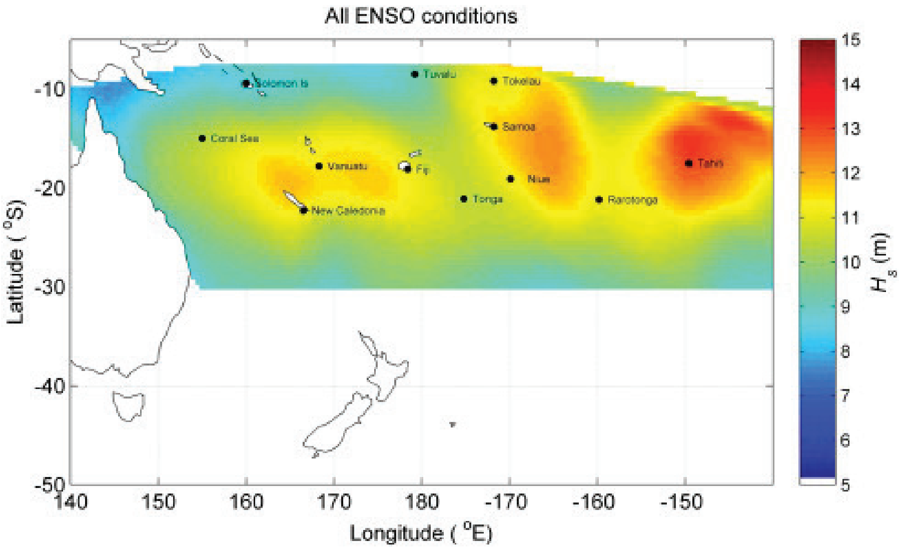
(iii) Swell waves generated by large extratropical storms in the 40°-50° belt of the southern and northern Pacific Ocean. These waves typically affect all Pacific island coasts facing them, with waves up to 5 m or more and long periods between 13 and 20 s (Kruger et al., 2011). Swell may propagate through the entire Pacific, however, with swells from the southern area reaching Hawaii in the north and swells generated in the north of the Pacific reaching Tonga in the south.

(iv) Tropical cyclone and storm-induced waves are generated locally. These waves are generally responsible for the largest waves and can be combined with significant storm surge.

(v) Stephens and Ramsay (2014) have assessed tropical cyclones in the South Pacific and found significant deep water wave heights of 6-9 m, 8-12 m, and 10-14 m, respectively, for the 10-, 50-, and 100-year ARI cyclonic events. An example of the significant wave height (m) associated with 50-year ARI tropical cyclones in the southwest Pacific is shown in Figure 5-8.

The actual wave height reaching a particular coastline is highly affected by local bathymetry and presence of offshore fringing reefs, which cause breaking and refraction of incoming wave energy. Local nearshore wave modelling is typically required to resolve nearshore wave processes.

Figure 5-8 Significant Wave Height Associated with 50-Year Annual Recurrence Interval Tropical Cyclones (in metres)



Notes: °S = degrees south; H_s = significant wave height; m = metre; °E = degree east.

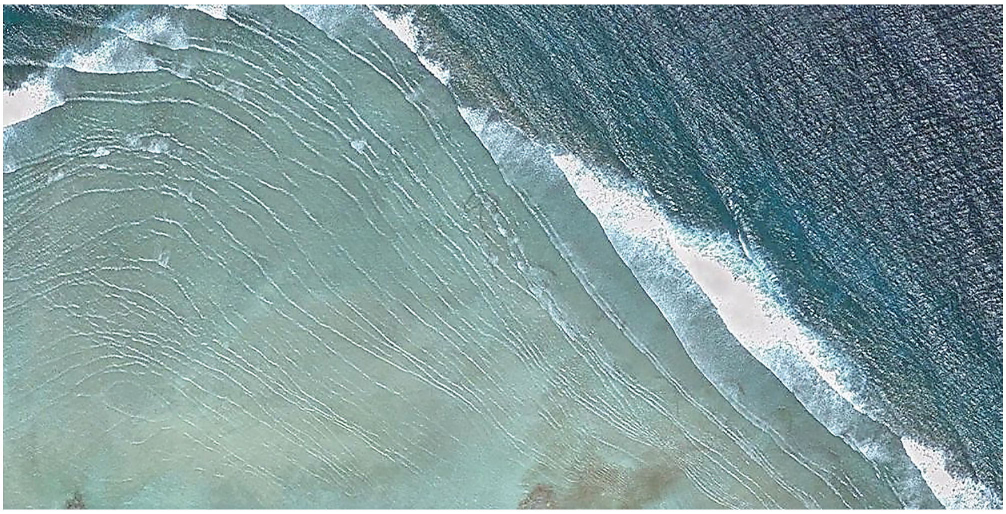
Source: Stephens, S.A. and D.L. Ramsay. 2014. Extreme Cyclone Wave Climate in the Southwest Pacific Ocean: Influence of the El Niño Southern Oscillation and Projected Climate Change. Global and Planetary Change, 123, pp. 13-26.

5.5.1 Reef-Top Processes

As waves approach a coral reef, they shoal and change in height and direction before breaking on the reef crest. They then decay as they move across the reef flat due to dissipative breaking processes and bed friction. Studies have shown that following the initial breaking process, the maximum size of waves on reef flats is controlled by water depth, with the maximum significant wave length (H_s) approximately 0.6 times the water depth (d), including any wave-generated setup (Gourlay, 1994; Kench and Brander, 2006). Recent laboratory studies (Killalea et al., 2017) have shown that closer to the reef edge, larger waves may be encountered as waves are breaking and dissipating their excess energy, while surf beat energy is high. Some general guidance may be adopted as follows:

<50 m from ocean reef edge	$H_s \approx 1.0 \times d$	(1)
50-100 m from ocean reef edge	$H_s \approx 0.75 \times d$	(2)
>50 m from ocean reef edge	$H_s \approx 0.6 \times d$	(3)

Figure 5-9 Example of Harmonic Decomposition of Incident Wave Forms after Breaking and Reforming into Oscillatory Waves in North Tarawa, Kiribati



Source: DigitalGlobe (https://www.digitalglobe.com).

Nearshore water level can also be modified by wave processes. Wave setup occurs due to the onshore momentum flux that occurs during wave breaking. Without breaks in the reef, to allow the seaward escape of elevated water within the lagoon, setup can be significant. Empirical models derived by Gourlay (1994) suggest that wave setup at the shoreline (η_{max}) may be up to 15% of offshore breaking wave height (H_0) (Equation 4).

$$\eta_{\max} \approx 0.15 \times H_0 \qquad (4)$$

An associated process is wave runup, which varies with breaking wave characteristics and beach and backshore slope and composition. Wave runup occurs on the beach face and causes periodic wave swash above the static water level. It may contribute to flooding, causing risk to public safety and damaging structures on the coastal edge. Kruger et al. (2011) noted wave runup of 2-5 m above mean sea level (MSL) on the Fijian south coast, associated with a distant swell event. The formula of Hedges and Mase (2004) can be used as an initial estimate of the runup level, exceeded by 2% of waves ($R_{2\%}$) (Equation 5).

$$R_{2\%} = (0.37 + 1.38\xi_0)H_S \qquad (5)$$

Where ξ_0 is the Irribarren number $\xi_0 = \tan\beta / \sqrt{H_S / L_0}$

Where H_S is the offshore significant wave height, L_0 is the offshore wavelength and β is the backshore slope.

5.6 Design Event

5.6.1 Design Life

Design life describes the life expectancy of a structure if properly maintained. Different types of structure have different design life expectancies based on their material strength and durability. The required design life of a structure varies depending on whether the structure is a temporary structure (i.e., typically less than a 5-year design life), interim measure (5-20 year design life), or long-term solution (20-100 year design life). The required design life will affect the choice of structure and the selected design event (i.e., whatever event the structure is designed to withstand).

Factors that can reduce the expected design life of a structure include:

- incorrect construction (e.g., failure to adequately lap geotextile);
- lack of maintenance (e.g., failure to reinstate a displaced rock or concrete armour unit);
- vandalism (e.g., illegal mining of beach nourishment material); and
- events exceeding design (e.g., a 500-year event impacting a structure designed for a 50-year event).

5.6.2 Probability of Event Occurrence

The design event describes the magnitude of event, generally relating to wave height, that a structure is designed to survive with no or minimal damage. As the occurrence of large storm events are inherently random, events are described according to their average ARI, more commonly known as return period. Another way to describe events is according to their average annual exceedance probability (AEP). For example, by the following relation, an event with a 100-year ARI or return period has a 1% chance of AEP.

$$AEP = 1 - \exp\left(\frac{-1}{ARI}\right) \qquad (6)$$

The likelihood of a design event impacting a structure is a function of the probability of event occurrence and the time frame being considered. Over a longer period, the probability of a rare event occurring is higher. For example, over a time frame of 50 years, an event with an AEP of 1% (or a 100-year ARI) has a 39% probability of occurring (Table 5-3). Therefore, if the certainty of a particular event not being exceeded is required over a long time frame (i.e. >100 year ARI), a very low probability design event must be selected (i.e., ~1,000-year ARI).

Table 5-2 Probability of Event Occurrence within a Specified Time Frame

Design Event Occurrence	ARI (years)	AEP (%)	Probability (%) of Event Occurrence Within:					
			1 Year	5 Years	10 Years	20 Years	50 Years	100 Years
	1	63	63.2	99.3	100	100	100	100
	5	18	18.1	63.2	86.5	98.2	100	100
	10	9.5	9.5	39.3	63.2	86.5	99.3	100
	20	5	4.9	22.1	39.3	63.2	91.8	99.3
	50	2	2.0	9.5	18.1	33.0	63.2	86.5
	100	1	1.0	4.9	9.5	18.1	39.3	63.2
	1,000	0.1	0.1	0.5	1.0	2.0	4.9	9.5

Notes: ARI = annual recurrence interval or return period; AEP = annual exceedance probability.

5.6.3 Selection of Appropriate Design Event

Selection of an appropriate design event depends on the anticipated design life of the structure and the risk of failure, which is considered tolerable. For high importance structures, such as those protecting a major road or critical building, this should be a low probability event (i.e., a 50-500+ year ARI event, depending on design life, such as a Category 3-5 cyclone). For less important structures, this may be a much more frequent 5-50 year ARI event, such as a tropical storm or Category 1-2 cyclone. In this case, more frequent maintenance and a higher likelihood of replacement should be expected. An example table, adapted from the Australian Standard 4997 Guidelines for the design of maritime structures, is presented in Table 5-3.

Table 5-3 Recommended Design Event Average Recurrence Interval

Function Category	Category Description	Design Working Life (Years)		
		5 or less (temporary works)	25 (interim structures)	50 (long-term structures)
1	Structures presenting a low degree of hazard to life of property	5	20	50
2	Normal structures	20	50	100
3	High property value or high risk to people	50	100	500

Source: Adapted from Australian Standard™: Guidelines for the Design of Maritime Structures. AS 4997–2005.

5.6.4 Calculating Design Wave Height

In Pacific environments, wave height is generally depth limited with waves having either broken on an offshore reef crests or being developed locally within a lagoon. In these cases, it is recommended to determine a depth limited wave height based on water levels corresponding to the appropriate design event. An example of the calculation of nearshore water level, seabed depth, and nearshore wave height for a range of return period events is shown in Table 5-4 and illustrated in Figure 5-3. For nondepth limited sites, additional analyses of wave conditions will be required.

Table 5-4 Example of Design Water Levels and Wave Height (metres above mean sea level[a,b])

Average Recurrence Interval (ARI)	1	25	50	100	1000
Likelihood over 50-year design life	*100%*	*86.5%*	*63.2%*	*39.3%*	*4.9%*
Astronomical tide (MHWS)	1.0	1.0	1.0	1.0	1.0
Storm surge	0.25	0.4	0.55	0.7	1.3
Offshore wave height (H_0, m)	*2*	*3.5*	*5*	*5.5*	*6*
Wave setup ($\eta_{max} \approx 0.15 \times H_0$)	0.3	0.5	0.75	0.8	0.9
Total water level = tide + storm surge + setup (m MSL)	1.55	1.9	2.3	2.5	3.2
Depth 15m offshore of structure[b] (d, m)	**1.55**	**1.9**	**2.3**	**2.5**	**3.2**
Depth-limited wave height at toe ($H_s \approx 0.6 \times d^a$)	*1.0*	*1.15*	*1.4*	*1.5*	*1.9*
Depth + 0.5 m SLR (m MSL)	**2.05**	**2.4**	**2.8**	**3.0**	**3.7**
Depth-limited wave height at toe including SLR ($H_s \approx 0.6 \times d^1$)	*1.3*	*1.4*	*1.7*	*1.8*	*2.2*

[a]These values are for example only and NOT intended to be used for design.

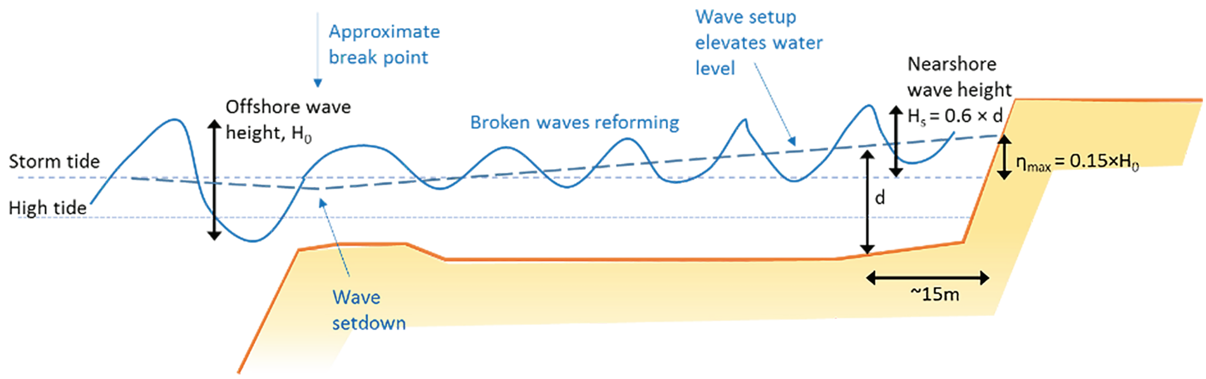
[a]MSL = mean sea level, to be defined for site.

[b]Assume lagoon bed 15 m offshore of toe = 0 m MSL.

Notes:

MHWS = mean high water spring; m = metre; ηmax = wave setup at the shoreline; H0 = offshore breaking wave height; d = depth; H_s = significant wave length; SLR = sea level rise.

Figure 5-10 Reef-Top Processes Affecting Nearshore Wave Height



6 Fundamentals of Engineering Design

6.1 Design Overview

Coastal protection structures have a principal function of protecting the shoreline from erosion caused by wave, current, or tidal effects. These may include structures that are built on and directly armour the shoreline or structures that are located offshore and indirectly protect the land by reducing wave heights. Coastal protection structures are varied in their form and construction material, and are vulnerable to different failure mechanisms, thus exerting different pressures on the environment.

Key design factors considered in coastal protection include:

- structure alongshore length
- structure cross-shore location (backstop wall or in active beach)
- required height of structure to limit overtopping to desired levels
- slope of structure
- seawall toe detail
- seawall end detail
- material size and density
- filter material and geotextile
- crest width
- allowance for settlement and/or later crest raising
- backshore protection.

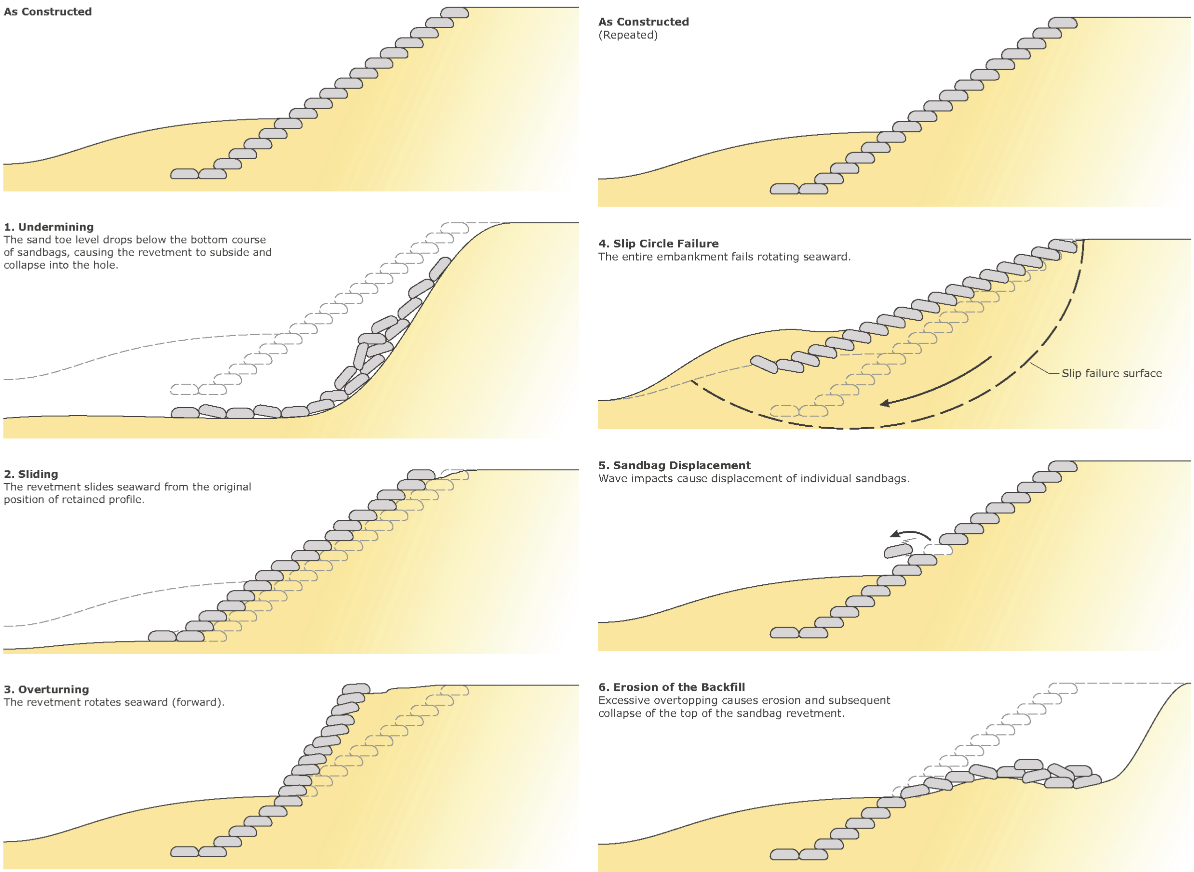
6.2 Typical Failure Mechanisms

Typical failure mechanisms, as defined within the U.S. Army Corp of Engineers (2006, Figure 6-1), include:

- undermining, in which the sand or rubble toe level drops below the footing of the wall, causing the wall to subside and collapse into the hole;
- sliding, in which the wall moves away from the retained profile;
- overturning, in which the wall topples over;
- slip circle failure, in which the entire embankment fails;
- loss of structural integrity, due to wave impact;
- erosion of the backfill, caused by wave overtopping, high water table levels, or leaching through the seawall;
- corrosion, abrasion, and impact damage; and
- outflanking and end scour.

Failure mechanisms can differ for coastal protection types, with rigid structures tending to be more vulnerable to catastrophic failures. Semi-rigid and flexible structures tend to fail with progressive actions.

Figure 6-1 Examples of Seawall Failure Mechanisms



Source: Water Research Laboratory (2015) Coastal Engineering Short Course, Coasts & Ports 2015, Auckland New Zealand

6.3 Cross-Shore Location of Structure

The location of the structure on the beach system (i.e Figure 6-2) influences the extent to which the structure interacts with the active beach system. Structures located in the active surf zone will be subject to higher wave energy and will need to be constructed of larger and more robust materials, while a structure located further landward will be subject to lower and less frequent wave energy and can be smaller.

Generally, the initial location of the structure will be determined by the current coastal form, with the coastal works being located directly in front of the current coastal edge. If the structure is located further seaward than the current coastal edge, additional fill material may be required to infill the reclamation. If the structure is located further onshore, excavation may be required.

Over time, the position of the structure with respect to the beach may change. On an accreting coast, the structure may gradually be covered in sand and become a “backstop” seawall (i.e., only exposed during extreme storm conditions). On an eroding coast, the beach width and sand level in front of the structure will gradually reduce, exposing the structure to higher and more frequent wave energy (Figure 6-3). Such changes should be taken into account during initial design.

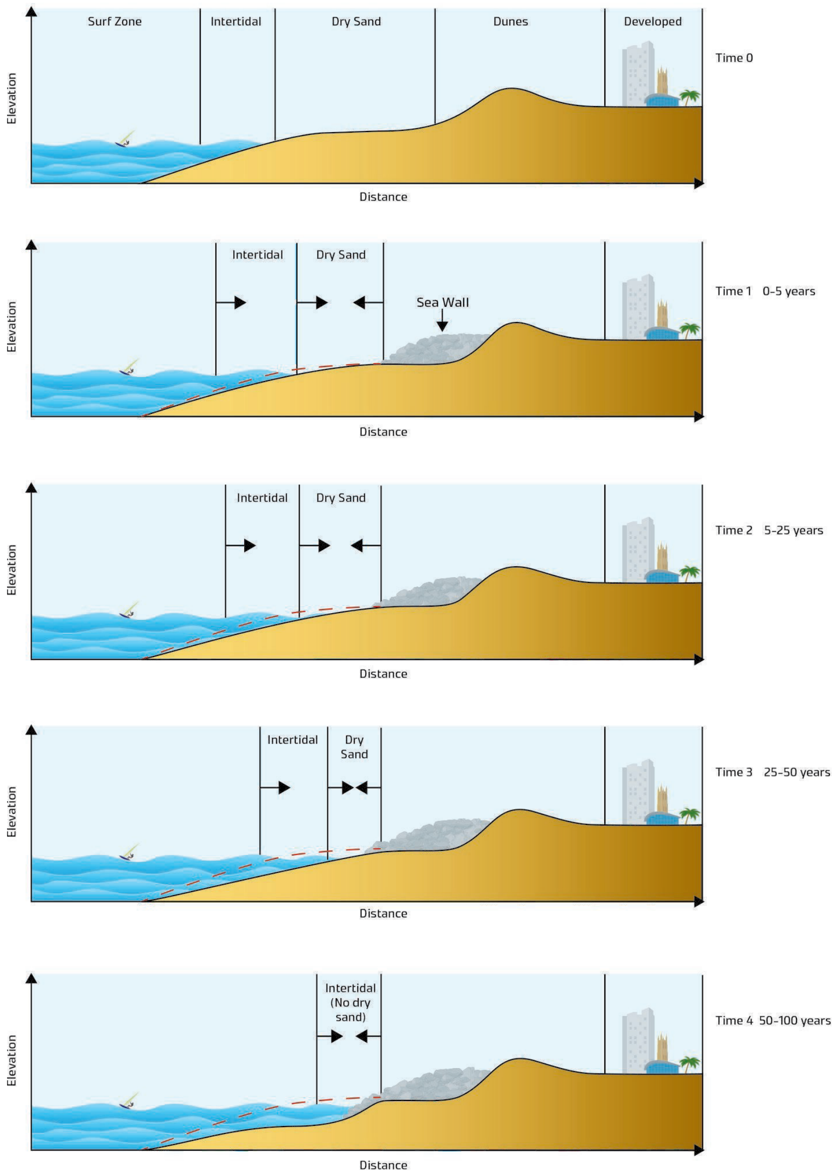
Steeper structures (e.g., a vertical concrete wall) will occupy a smaller cross-shore footprint than lower, gently sloping structure. The location and footprint of a structure can affect public access along a beach and should be considered in the selection of an appropriate protection type.

Figure 6-2 Comparison of a Seawall Located at the Back of A Beach and an Active Surf Zone: An Example

Back of a beach (left) **Active surf zone (right)**

Photo credits: T. Shand.

Figure 6-3 Example of Change in Cross-Shore Beach Width over Time on an Eroding Coast



6.4 Structure Length and End Detail

The structure length should extend sufficiently alongshore to protect the land of interest. The structure will not provide protection to adjacent land. If the coastline is in an overall state of "passive" erosion, as is often the case where a protection structure is required, the adjacent unprotected land will continue to migrate landward, potentially outflanking the structure (i.e. Figure 6-4). Additionally, seawalls may induce additional erosion at their ends ("end effect erosion") due to the interaction of the wall with local coastal processes Figure 6-5). Although the exact mechanisms and magnitude of this "active" end erosion continue to be debated, laboratory studies (McDougal et al., 1987) have discovered that the depth (r) and length (S) of end erosion could be related to a seawall length (l_s) by the following ratios:

$$r = 0.1 \times l_s \qquad (7)$$

$$S = 0.7 \times l_s \qquad (8)$$

Shand et al., (2010), however, found that this likely overestimates end-effect erosion for long seawalls (longer than 100 m). End erosion length and depth of 70 m and 10 m, respectively, are therefore practicable upper limits.

A coastal protection structure must terminate at some point. In order to avoid damage due to outflanking, the structure should be terminated as follows (in order of preference):

(i) at a rocky headland or adjacent coastal protection structure;

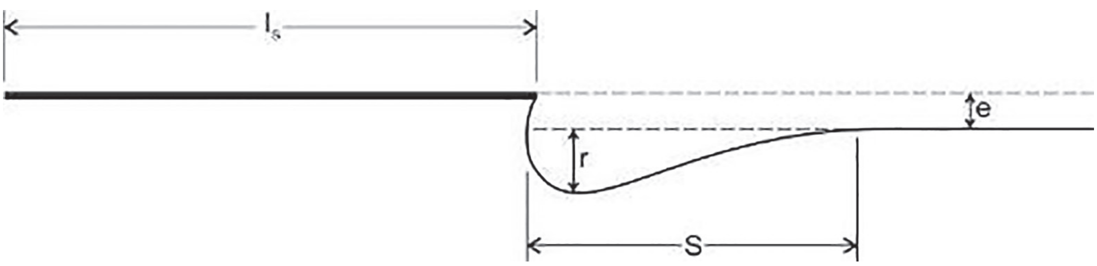
(ii) extending beyond the area of active erosion to a more stable part of the coast; and

(iii) having a return beyond the expected future passive erosion (e) and active end erosion (r).

Figure 6-4 Example of a Seawall Being Outflanked in Tarawa, Kiribati

Photo credit: T. Shand.

Figure 6-5 Example of Erosion at the End of a Structure of Length, Including Passive Erosion and End-Effect Erosion



Notes: l_s = length; e = passive erosion; r = end-effect erosion; S = length of end effect.

Source: McDougal., W.G., Sturtevant, M.A., and Komar, P.D. 1987. Laboratory and Field Investigations of the Impact of Shoreline Stabilization Structures on Adjacent Properties. Proceedings Coastal Sediments '87, American Society of Civil Engineers, 961-973.

6.5 Hydraulic Stability

A coastal protection structure is designed to resist wave forces to protect the land behind. The removal of units from the face of the structure will occur when the wave forces exceed the restraining forces that result from:

(i) submerged weight

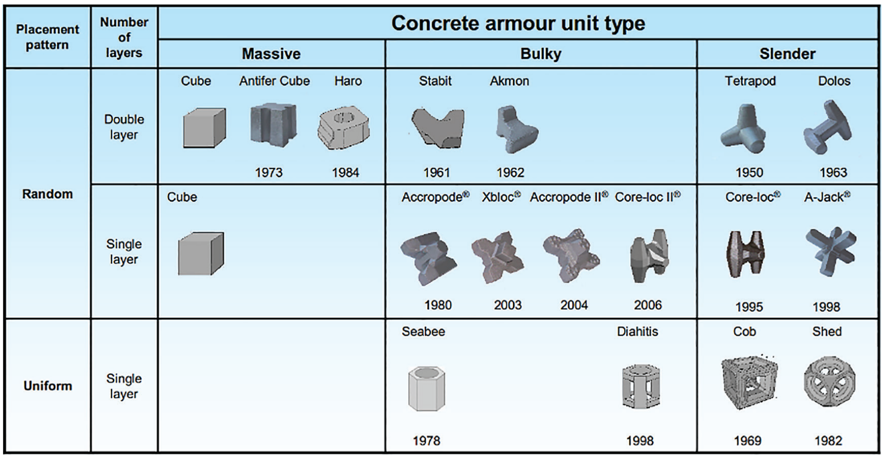
(ii) friction

(iii) interlocking.

Some materials, such as rock or massive concrete armour units (Figure 6-6), rely primarily on their mass to resist wave forces. Other materials, such as slender dolos or tetrapod units, rely more heavily on interlocking to resist wave forces. Other materials, such as Seabee units or concrete masonry blocks, are pattern-placed in a layer and rely on friction between units.

There are several formulae available to calculate the required armour unit size for design wave characteristics. These include the Hudson (1974) formula and the Van der Meer (1988) formula, which include more flexibility in the permeability of the structure, number of waves, and damage level.

Figure 6-6 Example of Concrete Armour Units



Source: Muttray, M. and Reedijk, B. (2008) Design of Concrete Armour Layers. DMC internal report.

Hudson Formula (USACE, 2006: Equation VI-5-67)

$$M = \frac{\rho H_s^3}{K_D \Delta^3 \cot\alpha} \qquad (9)$$

Where:

M = unit mass

Hs = design significant wave height

Δ = relative concrete density (ρc- ρw/ ρw)

ρw = mass density of seawater (assumed at 1,030 tonnes/m^3)

ρc = mass density of concrete (assumed 2,400 tonnes/m^3)

K_D = damage coefficient, which is dependent on the allowable damage, type and shape of armour, number of layers, among others. Higher K_D values occur in armour units with more interlocking.

Van Der Meer (1988) (USACE, 2006: Equations VI-5-68 and 69)

For plunging waves ($\xi_m < \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = c_{pl} P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \tag{10}$$

For surging waves ($\xi_m \geq \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = c_s P^{-0.13} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \sqrt{cot\alpha}\,\xi_m^P \tag{11}$$

Where:

H_s = design significant wave height of the incident waves at the toe of the structure

D_{n50} = nominal cubic equivalent rock diameter = $(W_{50}/\Delta)^{1/3}$

N = number of incident waves (equilibrium assumed at N = 7,500)

P = notional permeability of the structure (P = 0.4)

ξ_m = surf similarity parameter ($\xi_m = tan\alpha/\sqrt{(\frac{2\pi}{g}.H_s/T_m^2})$

α = slope angle = 1.5(H):1(V)

s_d = damage number (S_d = 2)

Δ = relative buoyant density (ρs/ρw-1)

ρ_w = mass density of seawater (assumed at 1,030 kilograms/m^3)

ρ_s = mass density of rock (typically 2,300-2,800 kilograms/m^3)

c_{pl} = constant = 6.2

c_s = constant = 1.0.

6.6 Underlayers and Filters

Underlayers are used to provide a transition between the larger armour rock or units and finer core or native backshore material. These layers provide dissipation of wave energy below the armour layer and limit loss of fine material from the core. This can be achieved using granular filter layers, a geotextile fabric, or a combination of the two. Generally, for coastal revetments, a single underlayer is used below the primary armour, with a geotextile placed below.

The underlayer rock and subsequent rock filter layers should be sized to prevent a loss of material through the above layer, based on a "geometrically closed"' principal. Complete guidance to filter layer sizing is provided in the Rock Manual (CIRIA, 2007) although, in general, the median weight of the underlayer ($W_{underlayer}$) should be equivalent to $W_{armour}/15$.

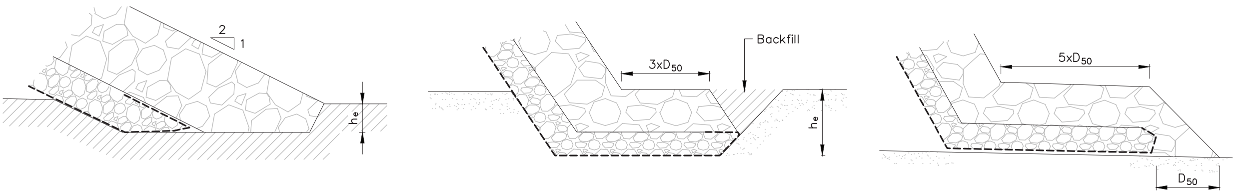
Geotextile fabrics should be sized according to the rock size being placed directly on them, with heavier geotextiles being used for larger rock to prevent puncture and damage (refer to manufacturer's specifications for individual geotextiles). The geotextile should be wrapped into the underlayer to prevent exposure at the toe or crest and potential for damage.

6.7 Toe Detail

The termination of the protection structure at the toe is an important consideration, as it provides a foundation for the above structure, and damage to the toe may result in failure of the entire structure. This is particularly true for rigid or pattern-placed structures.

Several options (Figure 6-7) for the toe detail of a coastal structure are presented in USACE (2006) and CIRIA (2007) although, in brief, these include excavating a toe trench in a hard substrate, excavating in soft seabed to a "design scour" depth where the toe is unlikely to be undermined, or placing a large toe on a soft seabed to enable the structure to "self-heal" as scour occurs. These are described below.

Figure 6-7 Options for Toe Detail on a Rock Revetment



Excavate into sandy seabed

Sandy seabed levels can fluctuate under storm conditions, with reflection from coastal structures causing additional scour. Van Rijn (2006) presents a number of methods to evaluate potential scour at the toe of coastal structures in an equilibrium system. In an unconsolidated, sandy seabed, scour depth is found to be a function of wave characteristics. Simplified estimates of scour are as follows:

- reflective structures (e.g., concrete seawalls) ~1.0 x H_s
- nonreflective structures (e.g., rock revetments) ~0.7 x H_s

Note that these depths are for equilibrium systems. If the average beach profile moves landward in the long term (i.e., the beach is in a state of long-term recession or recedes under SLR), the beach level can drop in front of the structure. This should be allowed for in the design.

The seabed should be excavated to the design depth with sand, rock, or GSCs placed on the seaward side to form a bund (refer Figure 6.8)

A pump may be required to dewater the trench. The toe of the structure should be placed in the trench with a geotextile, overlain by filter and armour layers.

Rafted on sandy seabed

If it is not feasible to excavate to the design scour depth due to a high water table, a larger toe can be placed. The toe will settle into the scour hole which forms in front, eventually reaching the design scour depth and supporting the above structure. The required size of the toe will depend on the likely resultant scour depth. This type of toe is more suitable for rock revetments than for armour or blanket-type structures that are more susceptible to damage by toe movement.

Keying the toe into rock

On a hard substrate where scour is unlikely, the armour should be keyed into the seabed to avoid sliding of the lowest units. The trench depth should be at least 0.5 x the armour unit diameter (D) and 2 x D wide. The trench should have as steep a front slope as possible and, in the case of pattern-placed units such as Seabees and concrete blocks, should be backfilled with concrete.

Figure 6-8 Example of Xbloc® Armour Units

Xbloc® armour units being placed in an excavated rock trench (left); a sand toe being excavated in Kiribati (right).

Photo credit: R Craven, 2014.

6.8 Crest Detail

The crest of a structure is important, as it provides the transition between the structure and the land behind and influences the volume of water which is able to overtop the structure. The crest must be effectively terminated to ensure that the structure is not damaged, that backshore material cannot migrate through the structure, or that overtopping flows are not sufficiently high to cause damage to the backshore. Examples of methods for terminating the crest of a structure are shown in Figure 6-9 and include:

Placement of a rock crest

This is the standard crest detail for a rock revetment and includes placement of armour rock above the level of the backing land. Generally, the crest is 2 x D_{50} high above the level of the backshore and 3 x D_{50} wide. The additional height and width of the armour rock will dissipate more wave energy than lower crest details, resulting in lower overtopping values, although the level of the geotextile (i.e., the impermeable layer) defines the structure crest height rather than the top of the rock.

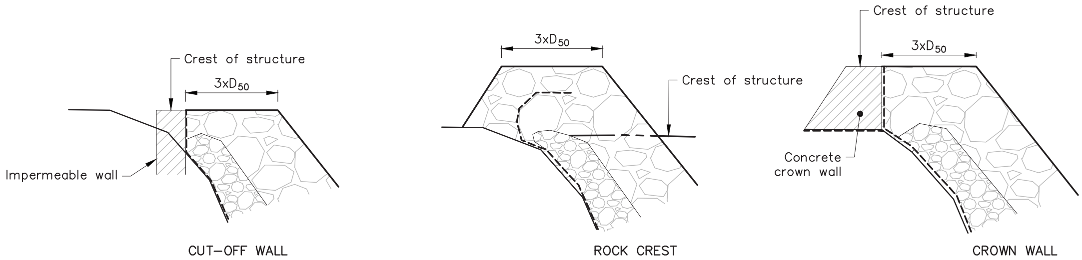
Installation of an impermeable wall behind the structure

This crest detail limits the loss of fines through a structure and generally allows for a low revetment crest at or below the level of the backshore. The wall is essentially a retaining wall and should be designed for geotechnical stability. The effective crest level is equivalent to the top of the impermeable wall and all overtopping calculations should be based on this level rather than the backshore level, if higher.

Installation of a crown wall

A crown wall is an impermeable wall placed behind the structure crest to constrain the structure and limit overtopping flow. A crown wall is useful where a higher structure than the level of the backshore land is required to reduce overtopping to a tolerable level. Crown walls can be mass or reinforced concrete, or can include steps and promenades for public access. The wave forces placed on a crown wall may be important, especially in relation to larger waves (H_s <1 m) and should be taken into account in the design.

Figure 6-9 Options for the Crest Detail on a Rock Revetment



6.8.1 Overtopping

The crest elevation of a structure, relative to the design water level (freeboard), determines the volume of water that can overtop under wave impact. Wave overtopping can be hazardous to pedestrians and vehicles; it can cause damage to structures, backshore land or pavement, and the coastal protection structure itself. While wave overtopping generally occurs as infrequent events associated with large waves, it is measured in terms of a mean overtopping discharge (q). Tolerable discharge values are provided for a range of activities within the Coastal Engineering Manual (USACE. 2006) (Figure 6-10).

In general, overtopping should be limited to less than 1 l/s/m for pedestrian safety in smaller wave environments (H_s <2 m) and to limit damage of unprotected backshore areas. Overtopping of up to 10 l/s/m can be tolerated for better protected backshores (grass or paved). Tolerable overtopping for high-value structures, such as buildings, should be assessed on a site-specific basis.

The general formula for overtopping provided within the EurOtop Manual (EurOTop, 2016: Equation 4.3) is

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.023}{\sqrt{tan\alpha}} \gamma_b \xi_{m-1,0} \cdot exp\left[-\left(2.7 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)^{1.3}\right] \quad (12)$$

Where:

q = overtopping rate [l/s/m]

g = acceleration due to gravity

R_c = crest freeboard of structure

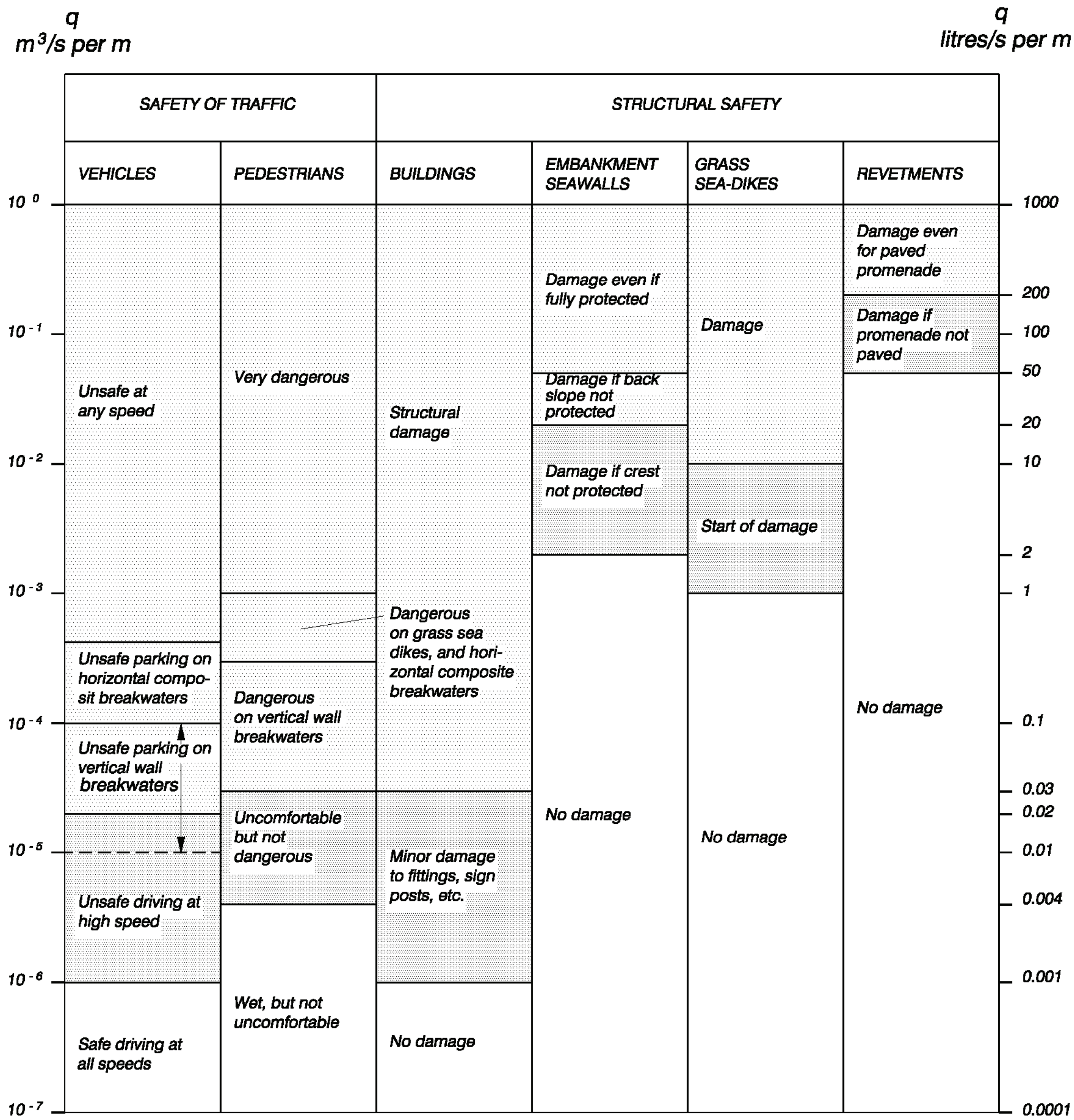
H_{m0} = significant wave height (m) from spectral analysis

α = structure slope

$\gamma_{b,f,\beta,v}$ = correction factors for the permeability and roughness of the slope and oblique wave angle.

It is recommended to calculate overtopping by using the neural network calculator provided as part of EurOTop (2016) at www.unibo.it/overtopping-neuralnetwork

Figure 6-10 Critical Values of Average Overtopping Discharges



Source: USACE. 2006. Coastal Engineering Manual. Washington, DC: U.S. Army Corps of Engineers. Section VI-5-20.

6.9 Geotechnical Considerations

Geotechnical considerations include:

Global stability

Global stability considers the potential for a landslip to occur beneath the structure, as illustrated in Figure 6-11. This can be improved by adequate drainage of the backshore (reducing build-up of porewater pressures) and flatter slopes. This is not typically an issue on reef flats.

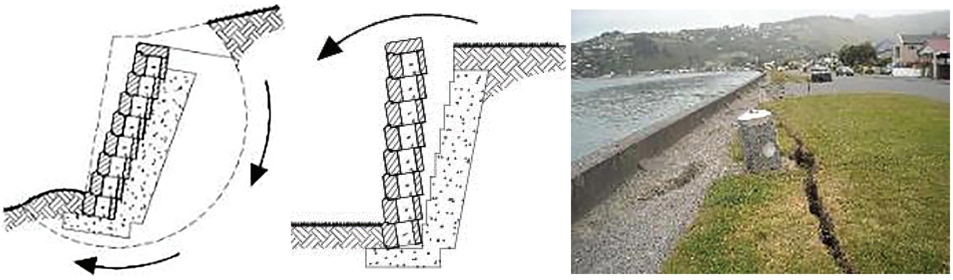
Sliding and overturning

Sliding and overturning occurs when the structure has insufficient support at the toe and slides or topples forward. It should be assessed based on the retained height and active soil pressure. It can be improved with adequate drainage and sufficient structure mass or toe support.

Liquefaction and lateral spreading

Liquefaction is where loose sands below the groundwater level completely lose strength in response to an applied cyclic force, such as the shakes of an earthquake or wave loading. The structure can either suffer a bearing failure and/or be laterally displaced.

Figure 6-11 Example of Geotechnical Failure Mechanisms



Global failure (left); toppling failure (middle); lateral spreading at the Avon-Heathcote Estuary (right)

Source: M. Esslemont, 2010.

6.10 Use of Concrete in Pacific Island Countries

6.10.1 Use of Coral Aggregates

In many PICs, the dense and durable volcanic aggregates typically used in concrete are not available. Only coral and coronus materials are obtainable and are typically less dense and durable.

Howdyshell (1974) reviewed concrete methods and examples since World War II and found that coral has successfully been used as an aggregate for concrete, provided the quality of coral is uniform and of high quality, and the mix design is carefully prepared and complied with. The only significant type of deterioration observed has been the cracking and spalling associated with corrosion of steel reinforcement. This may be attributable to the salts present in unwashed coral aggregates that destroy the passivity of embedded steel and lead to corrosion; however, similar corrosion occurs in many conventional concrete structures situated in the marine environment, particularly where the reinforcement is close to the surface and/or cracks are present.

Yodsudjai et al (2002) found that while the strength and durability of concrete is influenced by the quality, strength, and durability of the low-quality coarse aggregate used, the use of low cement-water ratios (i.e., increase in the amount of cement in the mix) lessens the negative effect of coral aggregate. Thus, reasonable compressive strength can still be achieved.

6.10.2 Use of Salt Water

A number of experimental investigations have been carried out on concrete using sea water for mixing and/or curing. Kaushik and Islam (1998) reported that seawater used as mixing water in concrete decreases setting time and increases early (~7 days) strength. A decrease in strength of 5-10%, however, was observed after 18 months. Mixing and curing in salt water had minimal effect on concrete alkalinity. Mbadike and Elinwa (2011) reported an 8% strength decrease and Islam et al. (2012) reported a 10% loss in strength when concrete was mixed and cured with sea water.

Mohammed, Hamada, and Yamaji (2004) reported earlier strength gain and no difference in long-term strength when sea water was used for mixing, and they found no indication that seawater mixed concrete is less durable. Maniyal and Patil (2005) found no major difference in compressive strength when sea water was used for mixing and curing, and they suggest that sea water is safe to use for mass concreting without any change of the concrete strength properties.

Nishida et al. (2014) carried out a literature review and experimental testing. They found 50% of papers reviewed had a positive opinion of the use of sea water in concrete mixing, with the addition of minimal additives such as blast furnace slag or fly ash. They indicated the potential of using sea water in reinforced concrete.

Literature is limited on the effect of sea water on concrete that is reinforcement with alternative materials, such as glass fibres or basalt. Information from manufacturers suggests that this may be feasible, although it would require further investigation.

6.10.3 Concrete Mix Design

The National Building Code of Kiribati (GoK, 2010) provides standard mix ratios for low-strength concrete (Table 6-1). Mix ratios for higher-strength concrete should be specifically designed, tested, and approved.

Table 6-1 Mix Ratios for Concrete and Grout

Compressive Strength	Mix Ratios by Volume			
	Water	Cement	Sand	Coarse Aggregate
10 MPa concrete	0.9	1.0	3.2	3.0 of 20 mm agg.
17.5 MPa concrete	0.8	1.0	2.7	3.0 of 20 mm agg.
20 MPa concrete	0.7	1.0	2.3	2.7 of 20 mm agg.
17.5 MPa grout	1.0	1.0	2.0	4.0 of 10 mm agg.

Notes: (i) The quantity of water is the maximum allowable and must be reduced with increase in moisture content of sand or aggregate; (ii) the compressive strength value (MPa = megapascal) in the table is Indicative only, and depends on the quality of the materials used; all concrete requiring a specified strength must be tested; and (iii) sand and aggregate should be washed prior to use.

Source: GoK. 2010. National Building Code of Kiribati. Government of Kiribati.

6.11 Consideration of Additional Site-Specific Factors

Other aspects that require specific design consideration include the following:

6.11.1 Long-Term Shoreline Trends

Long-term shoreline trends are important in determining where coastal protection works should end and how far inland tie-backs should extend. This includes toe depth, future water depth, and wave height in front of the structure.

Such trends are assessed using historical shoreline imagery (Section 2.3), beach profile data, or anecdotal information of past shoreline position. This should be combined with the anticipated effects of SLR and potential short-term erosion extents to determine a "design erosion" distance, behind which the ends of the wall should be located with the toe located beneath.

6.11.2 Stormwater Outlets

The location and dimensions of existing stormwater outlets should be noted and allowed for in the design, including likely maximum discharge rates. Failure to do so can result in damage to the protection structure and/or backshore flooding.

6.11.3 Rivers and Streams Discharging at the Site

The locations of river and stream outlets are typically unstable and dynamic. Specific attention should be given to rivers and streams that may be intersected by coastal protection works.

6.11.4 Requirement for Public Access

The requirement for public access onto or along the foreshore will influence the choice of coastal protection structure. Narrower and more vertical structures tend to provide better alongshore access by taking up less room on the foreshore, while flatter structures tend to provide improved access across them and onto the foreshore. Stairs, however, can be installed on vertical structures at additional expense.

6.12 Climate Change Adaptation

While future climate change effects in the Pacific may include changes in temperature, rainfall patterns, ocean salinity, acidification, wind speed, and tropical cyclone strength and distribution, the most notable effect influencing coastal structures is likely to be an increase in mean sea levels. Specifically, increased SLR may induce higher design water levels and waves at the structure, increasing overtopping flows and wave loading. It may also cause landward retreat of the shoreline potential by exposing the structure to outflanking and undermining. These effects and options for retrofitting and adaptation are shown in Table 6-2.

Table 6-2 Summary of Potential Climate Change Effects on Coastal Structures and Options for Retrofitting and Adaptation

Effect of Climate Change	Effect on Structure	Adaptation Option
Sea level rise causing increased design water levels at structure	Increased overtopping volumes	Raise crest to reduce overtopping or armour backshore to tolerate higher flows
Larger waves caused by higher wind speeds or deeper water at structure toe	Higher wave loadings on armour units	Place larger armour over existing units or create berm in front of structure to induce early wave breaking and dissipation
	Increased wave overtopping volumes	Raise crest to reduce overtopping or armour backshore to tolerate higher flows
Landward retreat of shoreline under rising sea level	Outflanks end of structure	Extend structure alongshore or recurve end to behind new shoreline
	Beach level lowers in front of structure	Extend toe using rock or sheet piling

6.13 Monitoring and Maintenance

Once a structure is constructed, it should be subject to ongoing monitoring and maintenance to ensure that the structure remains in good condition over its design life. If adequately maintained, structures can often significantly exceed their intended design life. Conversely, structures which are not maintained may fail prior to their intended design life.

Seawall monitoring can typically be divided between:

- **condition monitoring** to assess the condition of the structure, including:
    - superficial inspections multiple times a year and reporting of defects, changes, or unusual features of the seawall, such as cracks, displaced elements, or scour;
    - special inspections carried out following specific events, such as extreme events, floods, storms, or when other inspection indicates a cause for major concern;
- **performance monitoring**, focusing on the assessment of the principal function of the structure, such as:
    - limiting overtopping to tolerable levels; and
    - protecting the land and structures behind the seawall.

Monitoring would typically be undertaken during large wave and high sea level conditions, or reports should be collated of such conditions. Results of monitoring may be to:

(i) take no action/continue monitoring when no problems are identified or issues are minor;

(ii) rehabilitate all or part of the structure, where steps are taken to correct a problem before the structure functionality is significantly degraded (e.g., grouting holes in a seawall before fine particles of materials are lost from behind the wall, Figure 6-12);

(iii) repair all or part of the structure after damage has occurred and structural functionality is significantly reduced (i.e., after a seawall failure).

(iv) Upgrade or retrofit the structure to achieve a higher functionality or to withstand modified design conditions (e.g., as the climate changes or new information is available to define design conditions).

Figure 6-12 Rehabilitation of a Seawall in South Tarawa

Applying grout to degrading sandbags (left); repair a collapsed seawall by placing tribar units in Majuro, Marshall Islands (right)

Source: R. Craven, 2014 (left photo).

6.14 Further General Design Guidance

The following documents provide design guidance on coastal structures:

- CIRIA. 2007. *The Rock Manual: The Use of Rock in Civil Engineering*. 2nd edition, Publication C683. London: CIRIA.
- DoA. 1984. *Shore Protection Manual: Volume 1*. Vicksburg, MS: Coastal Engineering Research Center, United States Army Engineers Waterways Experiment Station. http://ft-sipil.unila.ac.id/dbooks/S%20P%20M%201984%20volume%201-1.pdf
- DoA. 1995. *Design of Coastal Revetments, Seawalls, and Bulkheads*. EM 1110-2-1614, Vicksburg, MS: United States Department of the Army.
- EurOtop. 2007. *Wave Overtopping of Sea Defences and Related Structures: Assessment Manual*. United Kingdom: Environment Agency. http://www.kennisbank-waterbouw.nl/DesignCodes/EurOtop.pdf
- EurOtop. 2016. *EurOtop II—Manual on Wave Overtopping of Sea Defences and Related Structures. An Overtopping Manual Largely Based on European Research, but for Worldwide Application*. 2nd edition Pre-release. J.W. Van der Meer, N.W.H. Allsop, T. Bruce, J. De Rouck, A. Kortenhaus, T. Pullen, H. Schüttrumpf, P. Troch, and B. Zanuttigh. www.overtopping-manual.com/docs/EurOtop%20II%202016%20Pre-release%20October%202016.pdf
- ISO. 2007. *Actions from Waves and Currents on Coastal Structures*. #21650, 1st edition, published 2007-10-15. Geneva: International Organization for Standardization.
- Oliver, J., D. Plotkin, J. Lesnik, and D. Pirie. 1998. Condition and Performance Rating Procedures for Rubble Breakwaters and Jetties. Technical Report REMR-OM-24. Champaign, IL: United States Army Construction Engineering Research Laboratory.
- USACE. 2006. *Coastal Engineering Manual*. Washington, DC: United States Army Corps of Engineers.

7 Assessment of Environmental Effects

An assessment of environmental effects—or environmental impact assessment—identifies the potential effects of proposed works on the environment. Exact requirements are generally established within local guidelines that relate to the acquiring of licences, approvals, and/or permits for work. In general, the assessment would consider the elements below.

7.1 Requirements for Works

The requirement for works describes and justifies the proposed works. It includes current and potential coastal processes and impact on human activities.

7.2 Consideration of Alternatives

A range of options are generally available to mitigate a hazard. While establishing why the proposed works is the best option, including its economic, environmental, and social/cultural impacts, consideration is given to alternatives.

7.3 Short-Term Construction Effects

This should consider the following:

- potential impacts on others due to noise, dust, vibration, or temporary loss of access;
- potential impacts on other coastal structures (e.g., damage during access or excavation); and
- potential for discharge of contaminants and effects on water quality and marine ecology.

Mitigation measures to limit the likelihood of severity of these effects should be proposed. For example, this could be in the form of a construction management plan.

7.4 Long-Term Effects

This should consider the following:

- effects on coastal processes, including changes to waves, currents, and sediment transport, as well as the effects on adjacent coastline (Section 7.4.1)
- effects on ecology, including loss of habitat and marine-terrestrial connectivity;
- social impacts, including loss of access and amenity and visual aesthetics; and
- positive effects, such as security of community and private assets and improved access.

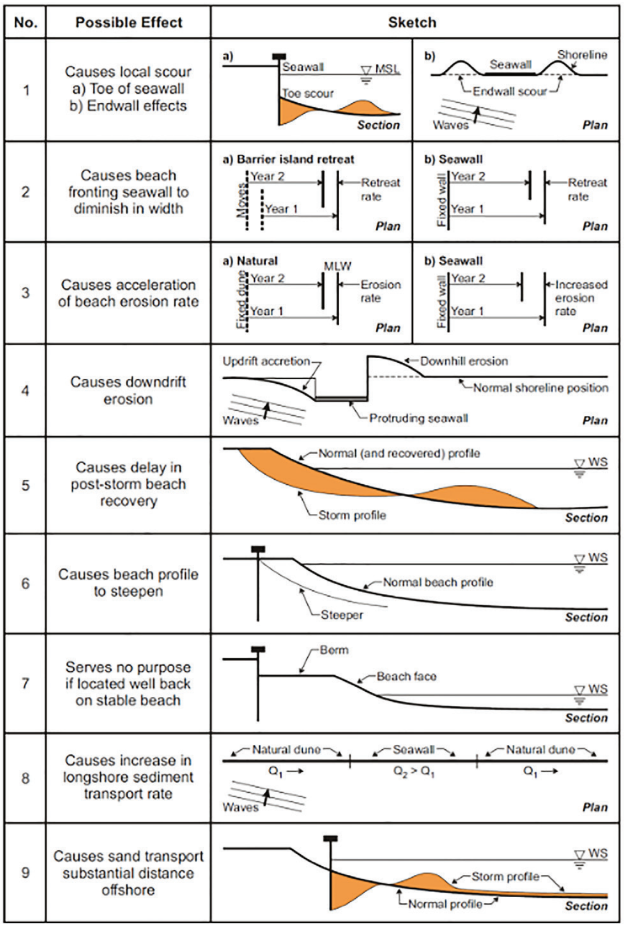
7.4.1 Effects of Seawalls on Beaches: A Note

It is important to note that coastal protection structures, such as seawalls and revetments, are intended only to protect the land behind the structure. They do not protect the fronting beach and, if the coast is in a state of recession, the beach will gradually be lost in front of a wall. Similarly, they will not protect adjacent land from ongoing erosion/recession. If recession continues, the challenges of erosion will continue adjacent to any constructed wall. This land must be monitored and, should erosion persist alongshore into other high-value areas, the seawall may need to be extended and/or additional management options considered.

Kraus and McDougal (1996) attributed much of the controversy about the potential adverse effects of seawalls on beaches to a lack of distinguishing between “passive erosion” and “active erosion” (Pilkey and Wright, 1988; Griggs et al. 1991; Griggs, Tait, and Coronoa, 1994). Passive erosion is defined as being caused by “tendencies which [that] existed before the wall was in place” and active erosion as being “due to the interaction of the wall with local coastal processes”. Of passive erosion, Griggs, Tait, and Coronoa (1994) stated that whenever a seawall is built along a shoreline undergoing long-term net erosion (i.e., recession), the shoreline will eventually migrate landward behind the structure, resulting in the gradual loss of beach in front of the seawall as the water deepens and the shore face profile migrates landward.

Dean (1986) presented a list of nine possible and often suggested effects of seawalls on adjacent shorelines and beaches (Figure 7-1). He then critically examined these postulations and concluded (Basco, 2006) the following:

Figure 7-1 Commonly Stated Effects of Seawalls on Adjacent Shorelines and Beaches



Notes: MSL = mean sea level; MLW = mean low water; WS = sediment fall velocity; Q1 = rate of longshore sediment transport

Source: Dean, R. C. 1986. Coastal Armoring: Effects, Principles and Mitigation. Coastal Engineering 1986, pp. 1843-1857.

Dean (1986) found that the armouring of a beach does not cause (numbers in parentheses are potential effects from Figure 7-1) the following:

- profile steepening (6)
- delayed beach recovery following storms (5)
- increased longshore transport (8)
- sand transport further offshore (9)
- increased long-term average erosion rate (3).

Dean (1986) also found that armouring of the beach will contribute to:

- frontal effects (e.g., toe scour, depth increases (1a))
- end-of-wall effects (e.g., flanking (1b))
- blockage of littoral drift when projecting in surf zone (i.e., groyne effect (4))
- reduced beach width fronting armouring (2).

7.5 Mitigation Measures

Consideration for what can be done to mitigate adverse effects include the following examples:

- limit work hours to reduce disturbance;
- designate plant set down and fill areas to minimise likelihood of spills;
- import sand to offset sediment impounded behind seawall;
- add a promenade in the seawall crest to mitigate loss of access along beach;
- add stairs and boat ramps to assist public access onto the foreshore; and
- include holes and irregular surfaces in vertical walls to provide habitat.

8 Documents for Construction

8.1 Drawings

Engineering drawings show the location and form of the works. These would likely include the following:

- drawing list;
- site drawing showing the location of the works, extent of work area, set down area, hazards, and areas of cultural significance;
- plan showing existing land and seabed contours, mean high water line, location and extent of the structure, and grades, among others;
- cross section(s) showing the existing topography and water levels (typical and extreme) and the extent, location, and geometry of the structure;
- details such as connections, transitions, and complex features (e.g., stairs, stormwater outlets, toe, and crest detailing); and
- setout plan that includes a site benchmark and either locations or distances to specific setout points on the structure.

8.2 Specification

The engineering specification defines the quality and performance of the materials which make up the works. This is likely to include:

- quality and characteristics of materials (e.g., rock gradings and tests);
- construction directions (i.e., what the contractor must and must not do);
- performance of the resultant product (e.g., construction tolerances);
- method of measurement to ensure sufficient materials are brought to site and accurate payment can be made; and
- standards and compliance that should be adhered to.

Manufacturers of proprietary products, such as geotextile or GSCs, may have their own additional specifications.

8.3 Schedule of Quantities

The schedule of quantities defines the units and quantity of each component of the physical works. Provision items may be included, such as rates for day work and plant and labour hire.

It is important to accurately define quantities, given that the costing provided by the contractor is generally based on this. A contingency of 20-30% is generally included to allow for unexpected items or variations that may arise during construction.

8.4 Engineer Estimate

An engineer's estimate provides rates for each item in the schedule of quantities to provide guidance and allow comparison with tendered prices. Engineer estimates may be based on cost guidance, such as construction handbooks or manufacturer's rates, experience with similar recent projects, or may be undertaken by specialist quantity surveyors. A suitable contingency should be included.

9 Concept Design Guidance

Concept-level design guidance for a selection of protection structures is provided in Appendix A. This guidance includes specific information on:

- suitability of the protection type
- required materials and construction plant
- design life
- typical costs
- design considerations
- material specifications
- typical construction methodology
- monitoring and maintenance requirements
- options for climate change adaptation
- typical drawings.

Concept guidance is provided in Appendix A for the following protection structures:

A-1 Rock Revetment

A-2 Seabee Revetment

A-3 Concrete Block Revetment

A-4 Geosynthetic Container Revetment

A-5 Grouted Rock or Mass Concrete Wall.

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Appendix A: Concept Designs

A-1. Rock Revetment

Description	Rock revetments are conventional land protection structures that have been used extensively throughout the Pacific. A rock revetment is formed using a geotextile filter fabric placed on a formed backshore slope, overlain by a cushioning layer of small rock, and protected from wave energy by suitably large rock armour. The high porosity provided by the voids between the rock, together with the slope, provide a form of wave energy dissipation, reducing the reflected wave and wave overtopping. Rock armour slopes typically range from 1.5(Horizontal (H)):1(Vertical (V)) to 3(H):1(V), with lower slopes requiring more construction material but enabling the use of smaller rock and resulting in less overtopping. The revetment should be extended sufficiently deep that the toe is not undermined by scour or erosion, and sufficiently high to reduce overtopping to tolerable volumes. Rock density makes a large difference in required size, with lighter rocks such as limestone (coral) requiring much larger sizes for similar wave height.  Rock revetment at South Tarawa, Kiribati
Suitability	▪ All wave climates (provided large enough rock can be sourced) ▪ Sandy and rocky seabed.
Materials required	▪ High quality, nonwoven, geotextile fabric ▪ Rock of suitable density, quality, and size (dependent on wave climate).
Construction plant	▪ Excavator (size dependent on rock mass) ▪ Barge or truck to deliver (unless rock sourced from site).
Design life	▪ 50+ years (basalt or similar volcanic rock) ▪ 10-30 years (coral/limestone or similar less durable materials).
Typical costs	Typical costs for various coastal protection works as a function of design wave height, material availability, and transport costs are set out in PRIF (2017), along with design and transport assumptions. A summary of **costs per linear metre** (A$/li m) for rock revetments is set out below.

Supply location	Low Wave Energy (Hs = 0.7 m; A$)	Moderate Wave Energy (Hs = 1.5 m; A$)	High Wave Energy (Hs = 3 m; A$)
Locally available material	700	3,000	10,000
Regional transport	1,400	6,500	22,500
International—primary port	3,100	15,000	50,000
International—remote location	5,500	25,000	90,000

Notes:

(i) Costs are indicative only and will fluctuate with material availability, market forces, and local taxes and tariffs;

(ii) costs should not be used for detailed project costing without input and review from qualified practitioner.

Design considerations

- Armour rock and underlayer should be sized to withstand design wave height (refer to table below)
- Geotextile should be used to limit loss of fine soil particle through structure and should be wrapped into or beneath armour layers
- Toe should be designed to withstand scour by excavating and/or placing additional toe armour
- Crest elevation should be set to limit overtopping to tolerable level for use and backshore material
- Crest should be backed by impermeable wall, if below backshore level, by armour rock 3xD_{50} wide or by crown wall if crest is higher than backshore level desired
- Structure should be protected from flanking by extending alongshore beyond areas of active erosion, by tying into adjacent nonerodible structure, or by landward return at moderate angle beyond likely extent of future erosion.

Wave Height H_s (m)	Armour Layer			Underlayer			Toe Depth		Crest Width	Crest Freeboard		Minimal Return
	W_{50} (kg)	D_{50} (m)	t_a (m)	W_{50} (kg)	D_{50} (m)	t_a (m)	h_e (m)		c_w (m)	h_c (m)		Ls (m)
							Reef	Sand		Ocean	Lagoon	
0.7	220	0.52	0.8	15	0.2	0.3	0.3	0.5	1.2	1.2	0.7	2
1.0	650	0.75	1.15	45	0.3	0.45	0.4	0.7	1.7	1.6	1.2	3
1.5	2200	1.1	1.7	150	0.45	0.7	0.6	1.1	2.6	2.3	1.4	5
2.0	5200	1.5	2.3	350	0.6	0.9	0.8	1.4	3.5	2.9	1.8	7
>2.0	*Obtain specialist advice*											

Notes:

- Based on rock density >2,600 kg/m³, add 50% rock mass for rock 2,400-2,600 kilogram (kg/cubic metre (m³), add 100% rock mass for rock 2,200-2,400 kg/m³
- Based on a front slope of 1(V):2(H), increase rock mass by 40% for 1(V):1.5(H) slope and decrease by 40% for 1(V):3(H) slope
- Based on shallow water wave conditions with no-damage criteria (<2% damage during design conditions).
- Toe depth based on excavation to 0.5 x D_{50} in reef or 0.7 x Hs in sand
- Crest freeboard based on mean overtopping flows of less than 1 linear/second for pedestrian safety and to minimise damage to backshore. If backshore paved or vegetated, it likely will tolerate higher overtopping rates (EurOtop, 2016). Values given for Pacific Ocean coasts (Tp ,10s) and Lagoon (Tp <5s)
- Structure return length minimum of 3 x H_s but consider other erosion drives and long-term shoreline trends.

Further design guidance

See the following:

- USACE (2006)
- CIRIA (2007)
- EurOtop, 2016.

Material specifications

Rock

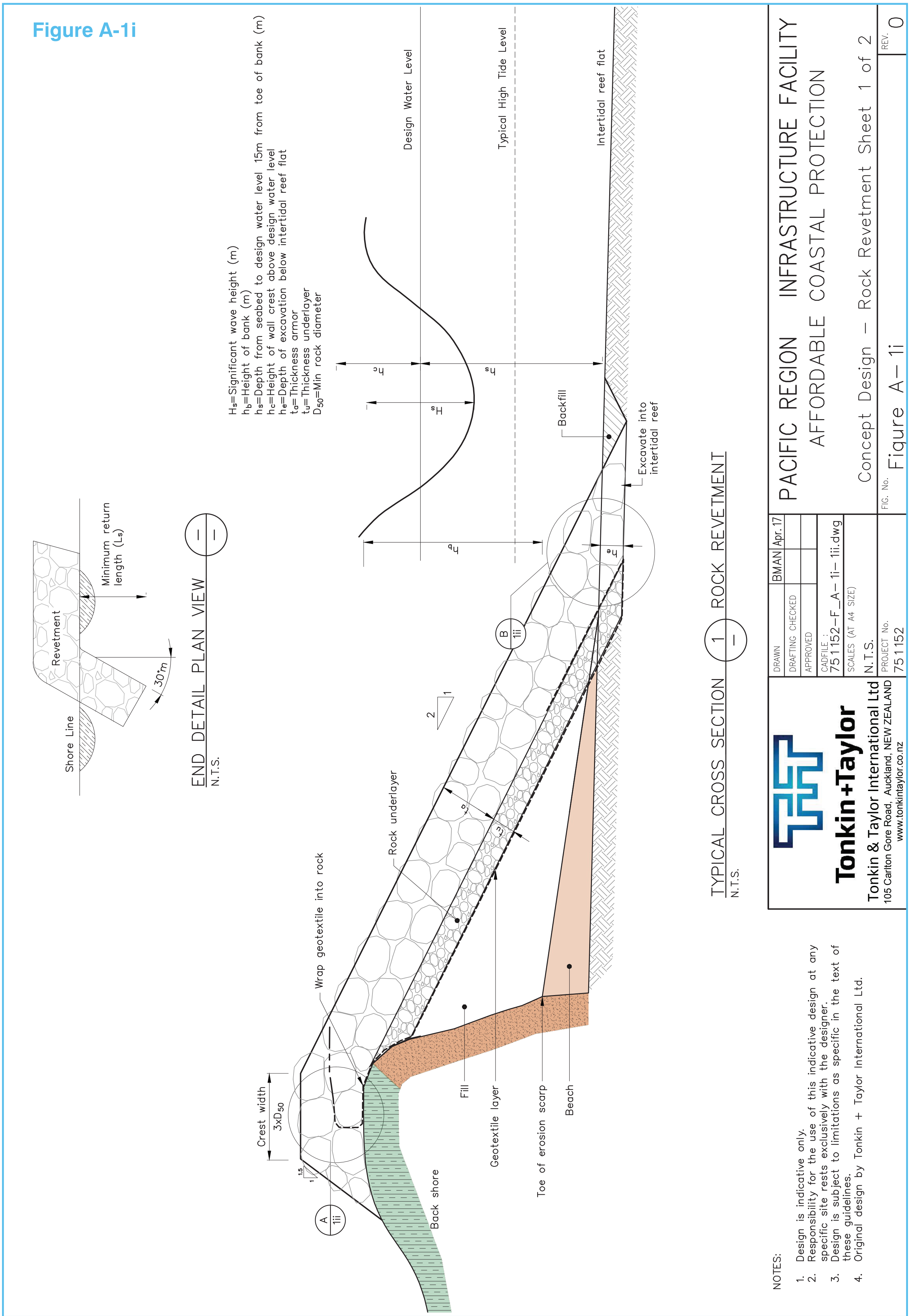
- 90% of the rock should have a density of at least 2,600 kg/m³ or as designed
- Grading should be 0.5xW_{50} <W_{50} <2W_{50}
- Maximum rock dimension should not exceed 3 x minimum rock dimension
- Rock shall be free from visually observable cracks, veins, fissures, laminations, unit contacts, cleavage planes, or other such flaws that could result in breakage during loading, unloading, or placing
- Rock generally shall be visually clean and free from impurities such as clays and soils

Geotextile

- Geotextile filter fabric shall be a nonwoven, needle-punched, continuous filament polyester or polypropylene geotextile
- The geotextile shall be Texcel® 900R or equivalent, approved by supervising engineer
- Fabric should be stored out of direct sunlight and not in contact with ground
- Torn or punctured fabric shall not be used.
- Note that full construction specifications should be prepared prior to construction.

Typical construction methodology	1. Establish site working area, including temporary fencing (landward of high tide) and signage 2. Set out rock revetment, including footprint and levels with stakes and string lines 3. Remove existing sandy sediment from the revetment footprint and stockpile for later replacement 4. Excavate toe and place fill to achieve target slopes and levels; earthworks shall be undertaken in a controlled manner so that erosion of disturbed areas is kept to a practical minimum and eroded material is confined on site as far as possible 5. Protect toe trench and slope, as necessary, from tide and wave action, using temporary rock or sandbag bunds 6. Place geotextile against prepared slope with minimum of 0.5 m laps in all directions 7. Place underlayer rock on geotextile and wrap geotextile as shown 8. Place rock armour on underlayer to form rock revetment to target profile. Do not end tip, roll, or drop rocks. 9. Place or cast concrete cap or crown wall (if required) 10. Replace the removed sandy sediment along toe of rock revetment or on adjacent beach 11. Remove debris from site and set down area.
Monitoring and maintenance requirements	Physical inspections should be undertaken annually and following large wave events. Inspections should include photographs, and observation and maintenance implications may include the following: ▪ Evidence of rock displacement: rock may be undersize, requiring larger rock to be placed or slope flattened ▪ Evidence of toe scour: additional toe armour may be required if scour is excessive or additional sand is placed ▪ Exposed geotextile: geotextile should be repaired or replaced if damaged, and additional rock placed to cover ▪ Scour behind the wall: wall may need to be raised and/or erosion-resistant material laid behind ▪ Scour at wall ends: wall may need to be extended alongshore or inland to prevent outflanking or additional sand placed to counter erosion.
Climate change adaptation	Future sea level rise may result in higher water levels at the structure and higher waves reaching the structure. ▪ Undersized rock can be upgraded by placing an additional layer of larger rock over the undersize rock. ▪ Increased overtopping can be reduced by raising the crest of the structure, using either additional geotextile and rock armour or by installing or upgrading a crown wall.

Figure A-1i



Shore Line
Revetment
Minimum return length (Ls)
30°m
END DETAIL PLAN VIEW
N.T.S.
Hs=Significant wave height (m)
hb=Height of bank (m)
hs=Depth from seabed to design water level 15m from toe of bank (m)
hc=Height of wall crest above design water level
he=Depth of excavation below intertidal reef flat
ta=Thickness armor
tu=Thickness underlayer
D50=Min rock diameter
Design Water Level
Typical High Tide Level
Intertidal reef flat
hc
Hs
hs
hb
he
Backfill
Excavate into intertidal reef
Crest width 3xD50
1.5
1
A 1ii
B 1ii
2
1
ta
tu
Wrap geotextile into rock
Rock underlayer
Back shore
Fill
Geotextile layer
Toe of erosion scarp
Beach
TYPICAL CROSS SECTION 1 ROCK REVETMENT
N.T.S.
NOTES:
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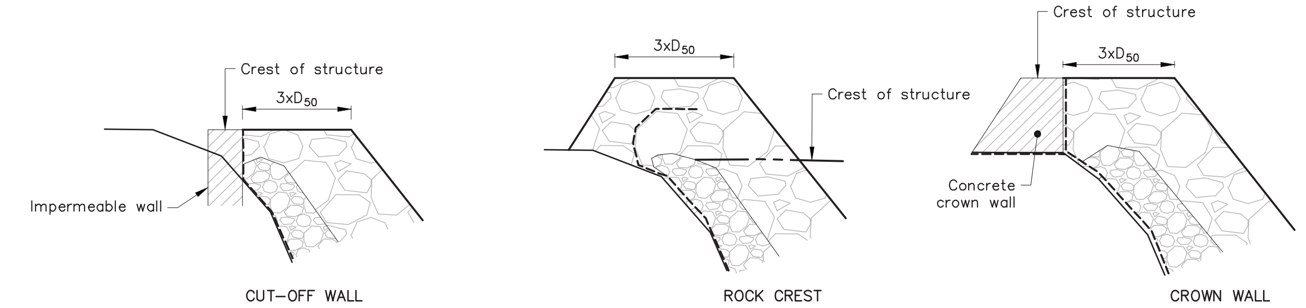
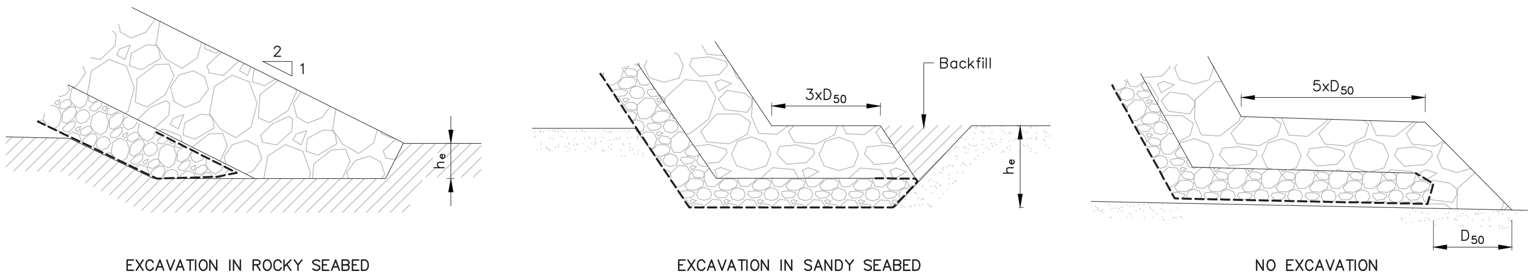
AFFORDABLE COASTAL PROTECTION
Concept Design – Rock Revetment Sheet 1 of 2
FIG. No. Figure A-1i
REV. 0

Figure A-1ii



TYPICAL DETAILS (A / 1i) CREST DETAILS
N.T.S.



TYPICAL DETAILS (B / 1i) TOE DETAILS
N.T.S.

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Concept Design – Rock Revetment Sheet 2 of 2

FIG. No. Figure A–1ii

REV. 0

A-2 Seabee Revetment

Description

Seabees are pattern-placed, hexagonal, interlocking units. Once interlocked, the units act as a blanket with a high structural integrity to mass ratio compared to random placed concrete armour units. Stability is dictated by layer thickness and, therefore, the size (width) of units can vary dependent on specific site requirements (placed by hand or machinery). While runup for this type of blanket structure is typically higher than for rock, runup can be reduced by using a "paired upstand" design, whereby every third unit is elevated, thus increasing roughness characteristics. The toe and ends of such blanket walls also require consideration as scour of the toe or outflanking of the ends may unravel the entire revetment. Seabees have been successfully used in high-energy environments (H_s >3m) in Australia, Argentina, Kuwait, and the United Kingdom with units of over 4,000 kilograms (kg) produced. The earliest walls were constructed in 1978 (initially ceramic units), with concrete units first used in 1982 at Abbot Point, Australia. These walls apparently remain in good repair. With units constructed of 35 megapascal (MPa) concrete, adequate toe and crest detailing, and wall ends protected from outflanking, such revetments should have design lives of 30+ years.

Seabee seawall Boigu Island, Torres Strait (Source: P. Riedel, 2005)

Suitability

- All wave climates (provided sufficiently large underlayer rock can be sourced)
- Sandy and rocky seabed.

Materials required

- Seabee units: concrete moulds or prefabricated
- Underlayer rock of suitable quality and size (dependent on Seabee size)
- High-quality, nonwoven geotextile fabric.

Construction plant

- Barge or truck to deliver underlayer rock and/or Seabees (unless rock sourced from site and Seabees cast onsite)
- Concrete batching or mixing plant (unless Seabees prefabricated)
- Excavator (if Seabees large or required toe is deep).

Design life

- 10-30 years, depending on concrete strength.

Typical costs

Typical costs for various coastal protection works as a function of design wave height, material availability, and transport costs are set out in PRIF (2016), together with design and transport assumptions.

A summary of **costs per linear metre** (A$/li m) for Seabee revetments is set out below.

Supply Location	Low Wave Energy (Hs = 0.7 m; A$)	Moderate Wave Energy (Hs = 1.5 m; A$)	High Wave Energy (Hs = 3 m; A$)
Locally available material	900	3,400	12,000
Regional transport	1,200	4,400	15,000
International—primary port	1,900	6,700	23,000
International—remote location	2,900	10,000	33,500

Notes:

(i) Costs are indicative only and will fluctuate with material availability, market forces, and local taxes and tariffs;

(ii) costs should not be used for detailed project costing without input and review from qualified practitioner.

Design considerations

- Seabee size and underlayer should be sized to withstand design wave height (refer to table below)
- Geotextile should be used to limit loss of fine soil particles through structure and should be wrapped into or beneath armour layers
- Toe should be designed to withstand scour by excavating into reef and backfilling with concrete or excavating into sand and placing toe armour rock or a geosynthetic gabion mattress
- Crest elevation should be set to limit overtopping to tolerable level for use and backshore material
- Crest should terminate in a concrete capping beam, crown wall, or gabion basket/mattress cap; otherwise, risk of displacement
- End of structure should terminate in in-situ case concrete beam or rock armour to lock in Seabee units
- Structure should be protected from flanking by extending alongshore beyond areas of active erosion, by tying into adjacent nonerodible structure, or by landward return at shallow angle beyond likely extents of future erosion.

Wave height H_s (m)	Seabee (Normal Concrete)				Underlayer			Toe Depth		Crest Freeboard		Minimal Return
	W (kg)	D (mm)	R (mm)	d (mm)	W_{50} (kg)	D_{50} (mm)	t_a (mm)	h_e (m)		h_c (m)		Ls (m)
								Reef	Sand	Ocean	Lagoon	
0.7	4	165	165	94	0.5	50	100	0.3	0.5	1.6	1.2	2
1.0	11.5	235	235	135	1	75	150	0.4	0.7	2.3	1.7	3
1.5	40	350	350	200	4	120	250	0.5	1.1	3.4	2.4	5
2.0	92	470	470	270	10	150	300	0.7	1.4	4.6	3.2	7
>2.0	*Obtain specialist advice*											

Notes:

- Based on shallow water wave conditions
- Assume D/R ratio of 1. D/R ratios can range from 0.4 to 2.5 with R held and D and W adjusted accordingly
- Upstand units 1.5 x height (R) and mass of normal Seabee units and used every third unit
- Assume porosity of 40%; can be adjusted from 35% to 45% with d changed
- Based on concrete density >2,300 kg/m^3: add 50% Seabee mass for concrete 2,100-2,300 kg/m^3, add 150% Seabee mass for concrete 1,900-2,100 kg/m^3
- Based on a front slope of 1(V):1.5(H)
- Toe depth based on excavation to 1.5 x R in reef or 0.7 x Hs in sand
- For toe and end armour rock sizing, refer to rock revetment specification
- Crest freeboard based on mean overtopping flows of less than 1 l/s for pedestrian safety and to minimise damage to backshore. If backshore paved or vegetated, it will likely tolerate higher overtopping rates (EurOtop, 2016). Values given for Pacific Ocean coasts (Tp , 10s) and Lagoon (Tp < 5s)
- Structure return length minimum of 3 x H_s but consider other erosion drives and long-term shoreline trends.

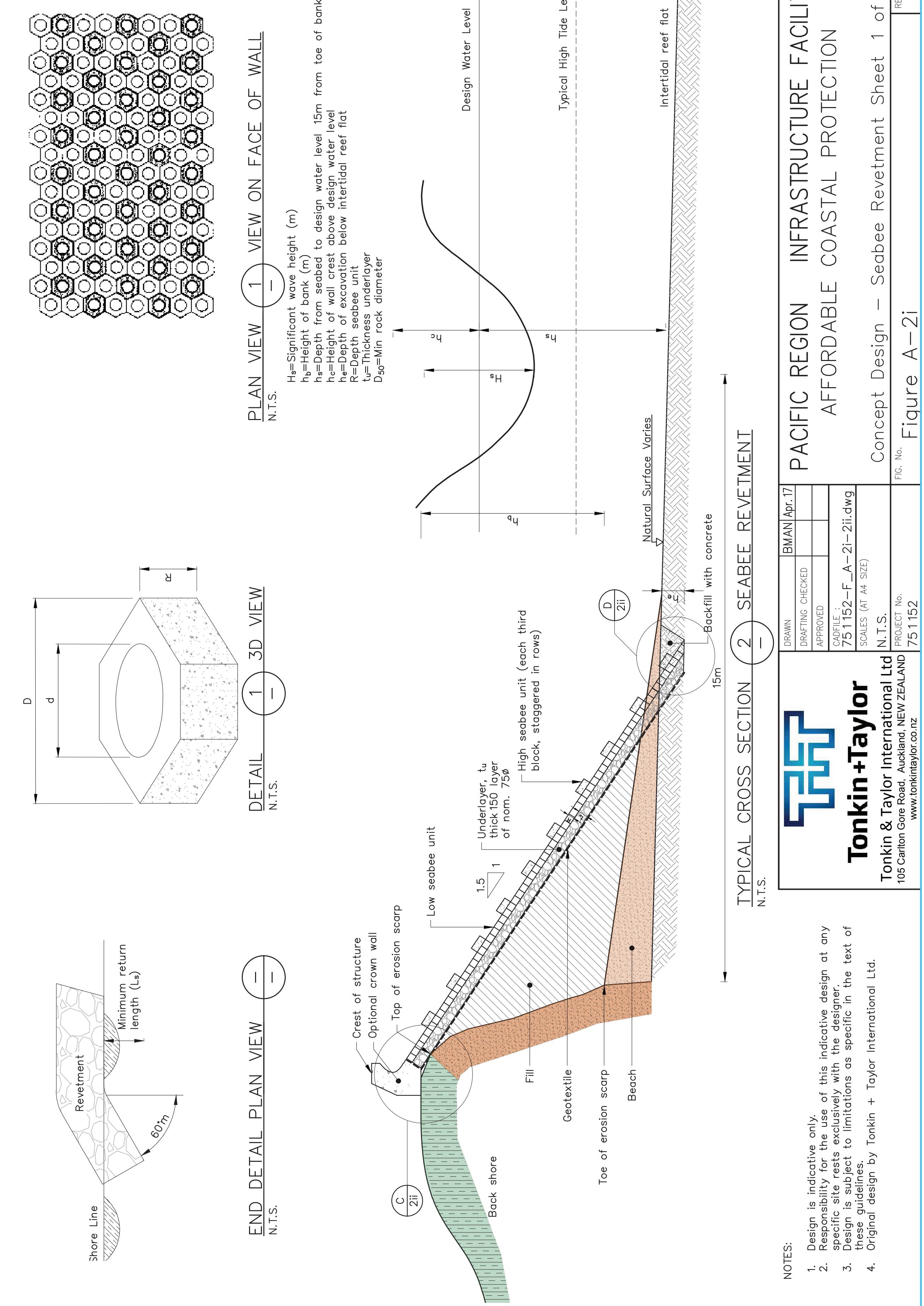
Further design guidance

- WRL (1997)
- EurOtop, 2016.

Material specifications	**Seabees** ▪ Concrete should have density of at least 2,300kg/m^3 or adjust unit size ▪ The concrete compressive strength should be at least 25 megapascal (MPa) after 28 days or lower if reduced design life accepted ▪ Maximum size of coarse aggregates should be 20 millimetres (mm) ▪ Concrete slump should be 100 mm ▪ Cement shall be type GP: general purpose Portland cement or equivalent approved ▪ A mix design should be submitted for approval prior to casting ▪ Units should be free of defects, such as honeycombing and cracks. The surface of the concrete should be smooth and dense ▪ Ceramic units can be used with sizing guidance from WRL (1997). **Rock** ▪ 90% of rock should have a density of at least 2,400 kg/m^3 or as designed ▪ Grading should be $0.65xW_{50} < W_{50} < 1.7W_{50}$ ▪ Maximum rock dimension should not exceed 3 x minimum rock dimension ▪ Rock shall be free from visually observable cracks, veins, fissures, laminations, unit contacts, cleavage planes, or other such flaws that could result in breakage during loading, unloading, or placing ▪ Rock shall generally be visually clean and free from impurities, such as clays and soils. **Geotextile** ▪ Geotextile filter fabric shall be a nonwoven, needle-punched, continuous filament polyester or polypropylene geotextile ▪ The geotextile shall be Texcel® 600R or equivalent, approved by supervising engineer ▪ Fabric should be stored out of direct sunlight and not in contact with ground ▪ Torn or punctured fabric shall not be used. ▪ Note that full construction specifications should be prepared prior to construction.
Typical construction methodology	1. Establish site working area, including temporary fencing (landward of high tide) and signage 2. Set out revetment, including footprint, slopes, and levels with stakes and string lines 3. Remove existing sandy sediment from the revetment footprint and stockpile for later replacement 4. Excavate toe and place fill to achieve target slopes and levels; earthworks shall be undertaken in a controlled manner so that erosion of disturbed areas is kept to a practical minimum and eroded material is confined on site as far as possible 5. Protect toe trench and slope as necessary from tide and wave action, using temporary rock or sandbag bunds 6. Place geotextile against prepared slope with minimum of 0.5 m laps in all directions 7. Place underlayer rock on geotextile and wrap geotextile as shown; form toe berm if toe is in sand 8. Place sand on underlayer to form smooth slope 9. Place Seabees on sand-covered underlayer to achieve target profile. Units should be ±30% normal to the sloping surface and ±15% between adjacent units. Maximum horizontal gap between units is 30 mm. Grout larger gaps 10. Place toe armour rock (if in sand) or backfill toe excavation with concrete (if in rock) 11. Place rock or cast concrete beam along exposed ends of revetment. 12. Place or cast gabion, concrete capping beam, or crown wall along crest. 13. Replace the removed sandy sediment along toe of rock revetment or on adjacent beach 14. Remove all debris from site and set down area.

Monitoring and maintenance requirements	Physical inspections should be undertaken annually and following large wave events. Inspections should include photographs, and observation and maintenance implications may include the following: ▪ Evidence of unit displacement: units may be undersize or original placement poor. Replace units by removing overlying and rebuilding. If displacement persists, use larger units or flatter slope ▪ Evidence of toe scour: additional toe rock armour may be required if scour is excessive or additional sand is placed ▪ Exposed geotextile: geotextile should be repaired or replaced if damaged, and additional rock placed to cover ▪ Scour behind the wall: wall may need to be raised and/or erosion-resistant material laid behind ▪ Scour at wall ends: wall may need to be extended alongshore or inland to prevent outflanking or additional sand placed to counter erosion.
Climate change adaptation	Future sea level rise may result in higher water levels at the structure and higher waves reaching the structure. ▪ Unit size generally cannot be increased with rebuild, so allow for increased future wave height in design ▪ Increased overtopping can be reduced by raising the crest of the structure, using a crown wall or earth bund and additional Seabee units.

Figure A-2i


PLAN VIEW 1 — VIEW ON FACE OF WALL
N.T.S.
H_s=Significant wave height (m)
h_b=Height of bank (m)
h_s=Depth from seabed to design water level 15m from toe of bank (m)
h_c=Height of wall crest above design water level
h_e=Depth of excavation below intertidal reef flat
R=Depth seabee unit
t_u=Thickness underlayer
D_{50}=Min rock diameter
DETAIL 1 — 3D VIEW
N.T.S.
D
d
R
END DETAIL PLAN VIEW — —
N.T.S.
Shore Line
Revetment
Minimum return length (L_s)
60°m
Design Water Level
Typical High Tide Level
Intertidal reef flat
Natural Surface Varies
h_c
H_s
h_s
h_b
h_e
Crest of structure
Optional crown wall
Top of erosion scarp
Low seabee unit
1.5
1
Underlayer, t_u thick 150 layer of nom. 75ø
High seabee unit (each third block, staggered in rows)
Backfill with concrete
15m
C 2ii
D 2ii
Back shore
Fill
Geotextile
Toe of erosion scarp
Beach
TYPICAL CROSS SECTION 2 — SEABEE REVETMENT
N.T.S.
NOTES:
1. Design is indicative only.
2. Responsibility for the use of this indicative design at any specific site rests exclusively with the designer.
3. Design is subject to limitations as specific in the text of these guidelines.
4. Original design by Tonkin + Taylor International Ltd.
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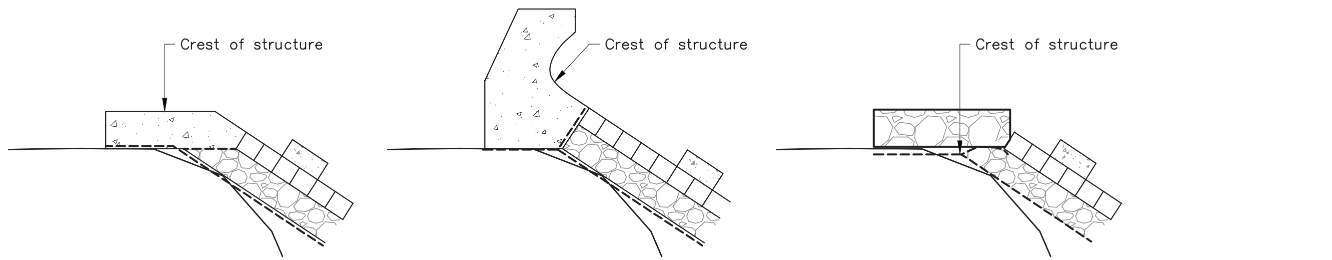
Concept Design – Seabee Revetment Sheet 1 of 2
FIG. No. Figure A-2i
REV. 0

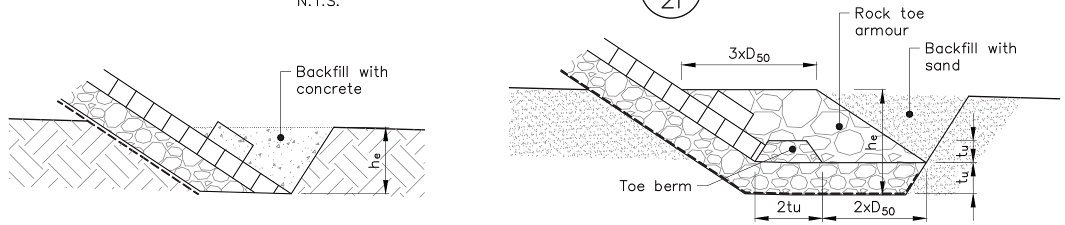
Figure A-2ii



CONCRETE SLAB — CROWN WALL — GABION BASKET OR MATTRESS

TYPICAL CREST DETAILS (C / 2i)

N.T.S.



EXCAVATE INTO ROCKY SEABED — EXCAVATE IN SANDY SEABED

TYPICAL TOE DETAILS (D / 2i)

N.T.S.

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PROJECT No. 751152		

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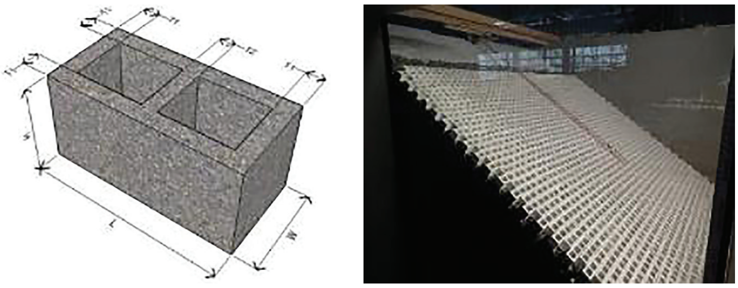
Concept Design – Seabee Revetment Sheet 2 of 2

FIG. No. Figure A–2ii — REV. 0

A-3 Concrete Block Revetment

Description

A revetment constructed of standard concrete masonry blocks (CMB) or Besser® blocks is proposed as a low-cost alternative to the more established blanket systems, such as Seabees and Terrafix® blocks. CMBs have the advantage of being widely available and/or having existing established supply chains. They can be placed without the need for heavy construction equipment. Disadvantages are that standard blocks are relatively small and low strength, limiting their stability under wave attack and design life. Furthermore, cost efficiencies are gained by using standard, widely available blocks, and a wide range of block quality is likely across the Pacific. Without previous application or testing, however, there is little to no engineering design guidance available. A series of physical model tests were undertaken at the Water Research Laboratory, Sydney, to provide guidance on stability under unidirectional wave attack. Show blocks were stable in up to 1 metre (m) significant wave height (limit of testing). Guidance presented below is based on this testing, although no information is available on likely design life of the typically low-strength concrete blocks. This must be determined by field trials.



Schematic drawing of concrete masonry blocks and model revetment slope (Source: WRL, 2017)

Suitability

- Wave climates <1 m (limits of testing)
- Sandy and rocky seabed.

Materials required

- Concrete masonry blocks prefabricated
- Underlayer rock of suitable quality and size
- High-quality, nonwoven geotextile fabric.

Construction plant

- Barge or truck to deliver underlayer rock and/or CMBs (unless rock sourced from site)
- Concrete mixing plant if blocks to be capped/toed in concrete.

Design life

- Unknown, although estimated at 5-10 years, depending on CMB strength.

Typical costs

Typical costs for various coastal protection works as a function of design wave height, material availability, and transport costs are set out in PRIF (2016), together with design and transport assumptions. A summary of costs per linear metre (A$/li m) for CMB revetments is set out below.

Supply location	Low wave energy (Hs <1.0 m; A$)
Locally available material	200
Regional transport	350
International—primary port	650
International—remote location	1,100

Notes:

(i) Costs are indicative only and will fluctuate with material availability, market forces, and local taxes and tariffs;

(ii) costs should not be used for detailed project costing without input and review from qualified practitioner.

Design considerations

- Design wave height should be less than 1 m or design water depth less than 1.6 m
- Concrete masonry block should be underlain by a rock filter
- Geotextile should be used to limit losses of fines through structure and wrapped into or beneath armour layers
- Toe should be designed to withstand scour by excavating 0.4 into reef and backfilling with concrete or excavating 0.7 m into sand and placing toe armour rock or a geosynthetic gabion mattress
- End of structure should terminate in in-situ case concrete beam or rock armour to lock in CMB units.
- Structure should be protected from flanking by extending alongshore beyond areas of active erosion, by tying into adjacent non-erodible structure, or by landward return at shallow angle beyond likely extents of future erosion

Wave height H_s (m)	Concrete Masonry Block			Underlayer			Toe Depth		Crest Freeboard		Minimal Return
	W (kg)	R (mm)	R (mm)	W_{50} (kg)	D_{50} (mm)	t_a (mm)	h_e (m)		h_c (m)		Ls (m)
							Reef	Sand	Ocean	Lagoon	
0.7	16	190	390	1	75	150	0.3	0.5	1.9	1.5	2
1.0	16	190	390	1	75	150	0.4	0.7	2.8	2.3	3
>1.0	*Use alternative material*										

Notes:

- Based on shallow water wave conditions
- Based on a front slope of 1(V):1.5(H)
- For toe and end armour rock sizing, refer to rock revetment specification
- Crest freeboard based on mean overtopping flows of less than 1 linear/second for pedestrian safety and to minimise damage to backshore. If backshore paved or vegetated, it may tolerate higher overtopping rates (EurOtop, 2016). Values given for Pacific Ocean coasts (Tp , 10s) and Lagoon (Tp <5s)
- Structure return length minimum of 3 x Hs but consider other erosion drives and long-term shoreline trends.

Further design guidance

- Blacka, M., How, D. and Coghlan, I.R. (2017)
- EurOtop, 2016.

Material specifications

Concrete masonry blocks

- Blocks should be 390 x 190 x 190 mm and approximately 16 kg
- Should comply with AS/NZS 4455:1997 or equivalent
- Should withstand drop from 0.5 m onto hard ground (not concrete).

Rock

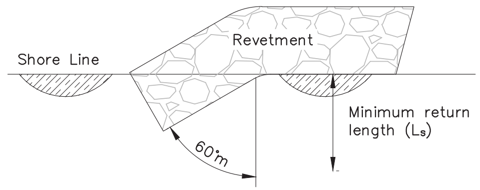
- 90% of the rock should have a density of at least 2,200 kg/m^3 or as designed
- Grading should be 40-120 mm, with 50% of rock larger than 70 mm
- Maximum rock dimension should not exceed 3 x minimum
- Rock shall be free from visually observable cracks, veins, fissures, laminations, unit contacts, cleavage planes, or other such flaws that could result in breakage during loading, unloading, or placing
- Rock generally shall be visually clean and free from impurities, such as clays and soils.

Geotextile

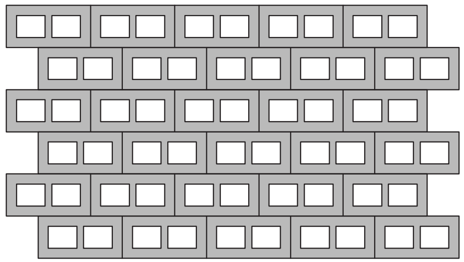
- Geotextile filter fabric shall be a nonwoven, needle-punched, continuous filament polyester or polypropylene geotextile
- The geotextile shall be Texcel® 600R or equivalent, approved by supervising engineer
- Fabric should be stored out of direct sunlight and not in contact with ground
- Torn or punctured fabric shall not be used.
- Note that full construction specifications should be prepared prior to construction.

Typical construction methodology	1. Establish site working area, including temporary fencing (landward of high tide) and signage 2. Set out revetment, including footprint, slopes, and levels with stakes and string lines 3. Remove existing sandy sediment from the revetment footprint and stockpile for later replacement 4. Excavate toe and place fill to achieve target slopes and levels; earthworks shall be undertaken in a controlled manner so that erosion of disturbed areas is kept to a practical minimum and eroded material is confined on site as far as possible 5. Protect toe trench and slope, as necessary, from tide and wave action, using temporary rock or sandbag bunds. 6. Place geotextile against prepared slope with minimum of 0.5 m laps in all directions 7. Place underlayer rock on geotextile and wrap geotextile as shown; form toe berm if toe is in sand 8. Place sand on underlayer to form smooth slope 9. Place CMBs on sand-covered underlayer to achieve target profile. Maximum horizontal gap between units is 30 mm. Grout larger gaps. Maximum vertical displacement between units is 50 mm; modify under layer to achieve. 10. Place toe armour rock (if in sand) or backfill toe excavation with concrete (if in rock) 11. Place rock or cast concrete beam along exposed ends of revetment 12. Place or cast gabion, concrete capping beam or crown wall along crest 13. Replace the removed sandy sediment along toe of rock revetment or on adjacent beach 14. Remove all debris from site and set down area.
Monitoring and maintenance requirements	Physical inspections should be undertaken every six months and following large wave events. Inspections should include photographs, and observation and maintenance implications may include the following: ▪ Evidence of unit breakage: units may be subject to high wave loads or may be structurally weak, as information on structural integrity of these units is lacking. Replace units by removing overlying units and rebuilding. If displacement persists, wave climate is likely too large for this type of revetment ▪ Evidence of unit displacement: units may be undersize or original placement poor. Replace units by removing overlying and rebuilding. If displacement persists, wave climate is likely too large for this type of revetment ▪ Evidence of toe scour: additional toe rock armour may be required if scour is excessive or additional sand is placed ▪ Exposed geotextile: geotextile should be repaired or replaced if damaged and additional rock placed to cover ▪ Scour behind the wall: wall may need to be raised and/or erosion-resistant material laid behind ▪ Scour at wall ends: wall may need to be extended alongshore or inland to prevent outflanking, or additional sand placed to counter erosion.
Climate change adaptation	While the design life of this structure is not expected to be long, future sea level rise or a change in El Niño–Southern Oscillation (ENSO) conditions may result in higher water levels at the structure and higher waves reaching the structure. ▪ Unit size cannot be increased, so allow for increased future wave height in design or use alternative material ▪ Increased overtopping can be reduced by raising the crest of the structure, using a crown wall or earth bund and additional CMB units.

Figure A-3i

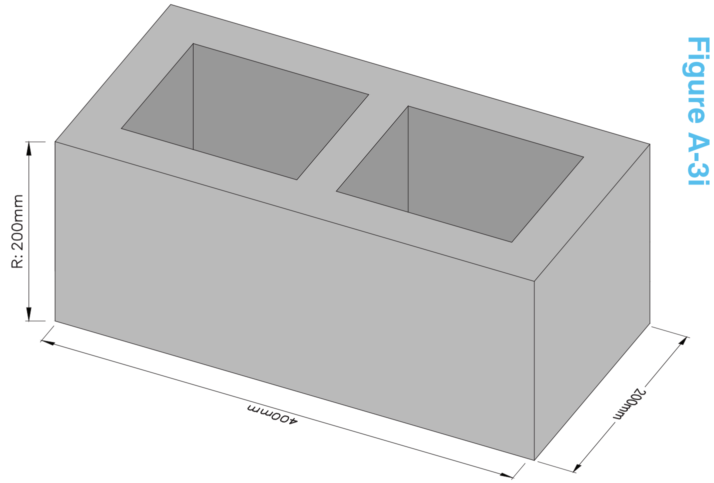


END DETAIL PLAN VIEW
N.T.S.

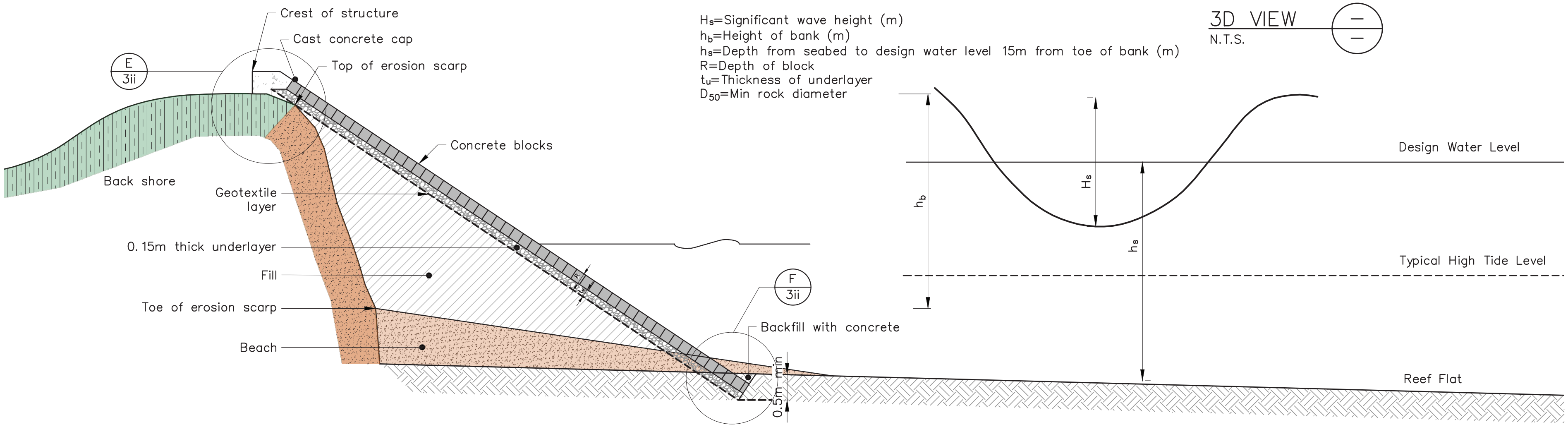


Block placement
Long axis shore parallel

PLAN VIEW
N.T.S.



3D VIEW
N.T.S.



TYPICAL CROSS SECTION 3 CONCRETE BLOCK REVETMENT
N.T.S.

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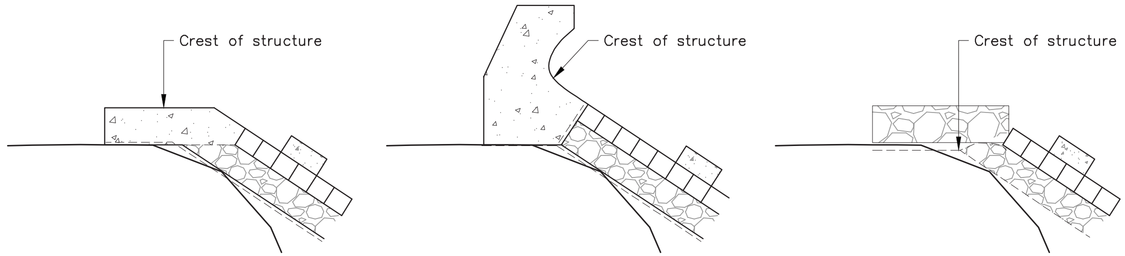
Concept Design – Concrete Block Revetment Sheet 1 of 2

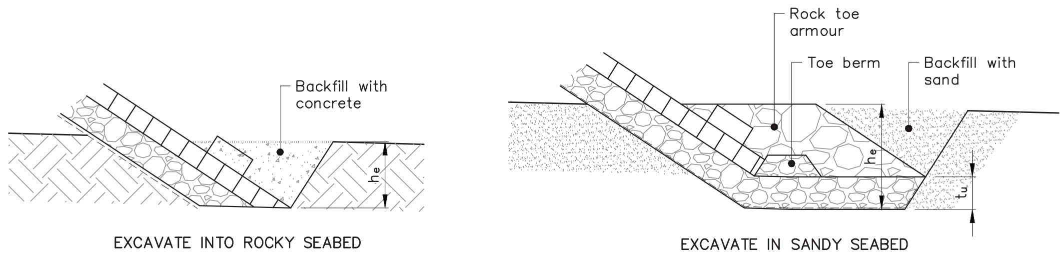
FIG. No. Figure A–3i

REV. 0

Figure A-3ii



TYPICAL CREST DETAILS
N.T.S.
E
3i



TYPICAL TOE DETAILS
N.T.S.
F
3i

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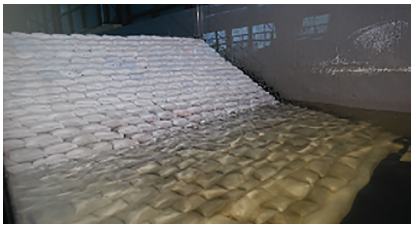
AFFORDABLE COASTAL PROTECTION
Concept Design – Concrete Block Revetment Sheet 2 of 2
FIG. No. Figure A–3ii
REV. 0

A-4 Geosynthetic Container Revetment

Description	Geotextile containers (GSC) are commonly referred to as "geobags". They comprise a geotextile pillow filled with sand. Their use in Australia has been documented in Coghlan et al. (2009) and Hornsey et al. (2011). They have been widely used throughout the world. Commonly available sizes in Australia are 2.5 cubic metres (m^3) and 0.75 m^3, although smaller bags can be manufactured. Empty containers are light and can be transported readily; however, larger bags require filling frames and slurry pumps with mechanical plant to assist in placement. Durability for high-quality geotextiles is typically 10 to 20 years, although this can be reduced due to debris damage or vandalism. The modular nature of these structures is such that they will remain structurally coherent when up to 2% of individual containers are damaged or removed, especially if a double layer is used. For 10-second spectral peak wave periods, 2.5 m^3 containers can withstand significant waves of approximately 1.7 m, while 0.75 m^3 containers can withstand significant waves of approximately 1.3 m. The idea of using smaller hand-placed but high-quality GSCs was raised during the desktop review (PRIF, 2016). Testing and construction to date primarily has been two layers with the long axis placed alongshore. While rescaling of model results can yield threshold wave conditions for smaller bags, alternative placement configurations, such as long axis cross-shore, were investigated in a series of physical model tests undertaken at the Water Research Laboratory in Sydney. Show bags were stable in waves up to H_s = 0.4 m, with some damage occurring above this height and complete failure between H_s = 0.5 and Hs = 0.6 m. This is generally in keeping with previous results, although bags placed cross-shore were slightly more stable than alongshore, with the single layer reducing redundancy. An advantage of the small hand-placed bags is that following damage, they can be replaced easily.  2.5 m3 Elcorock® revetment (James Carley, WRL UNSW)  Scale testing of small hand-placed geotextile bags at WRL
Suitability	▪ Sandy and rocky seabed, though case should be taken when used on rocky seabed to prevent damage ▪ 2.5 m^3 GSC up to H_s = 1.7 m ▪ 0.75 m^3 GSC up to H_s = 1.3 m ▪ 40 kilograms (kg) GSC up to H_s = 0.4 m.
Materials required	▪ Geosynthetic containers ▪ Sand.
Construction plant	▪ Filling frame (0.75 and 2.5 m^3 GSCs) ▪ 20 tonne excavator and J-Bin (0.75 m^3 GSCs) ▪ Slurry pump (2.5 m^3 GSCs) ▪ Hand-held sewing machine.   
Design life	▪ 15-20 years given by manufacturer for 0.75 and 2.5 m^3 Elcorock® GSCs. Single layer and hand-placed bags may be shorter.

Typical costs

Typical costs for various coastal protection works as a function of design wave height, material availability, and transport costs are set out in PRIF (2016), together with design and transport assumptions.

A summary of costs per linear metre (A$/li m) for single layer GSC revetments is set out below assuming sand is available locally.

Supply Location	Low Wave Energy (Hs < 0.5 m) Hand-Placed GSC	Moderate Wave Energy (Hs < 1.3 m) 0.75 m3 GSC	Moderate Wave Energy (Hs < 1.7 m) 2. 5 m3 GSC
Locally available material	350	1,800	3,300
Regional transport	370	1,960	3,500
International-primary port	430	2,150	3,800
International-remote location	530	2,420	4,200

Notes:

(i) Costs are indicative only and will fluctuate with material availability, market forces, and local taxes and tariffs;

(ii) costs should not be used for detailed project costing without input and review from qualified practitioner.

Design considerations

- GSC should be sized to withstand design wave height (refer to Suitability)
- Long axis of bags should be placed parallel to slope, if double layer, and perpendicular to slope if single layer used
- Geotextile should be used beneath GSC to limit loss of fine soil particles through structure
- Geotextile should be wrapped around lowest GDC as “dutch toe” or if special ‘toe GSCs’ used
- Toe should be designed to withstand scour by excavating below scour level and/or placing special toe GSCs.

Water Depth ds (m)	Wave height H_s (m)	Geosynthetic Container					Crest Width	Crest Freeboard		Minimal Return
		V (m3)	M (kg)	L (m)	W (m)	D (m)	c_w (m)	h_c (m)		Ls (m)
								Ocean	Lagoon	
1.1	0.4	0.03	40	0.57	0.47	0.13	1	1,2	0.9	1.5
2.1	1.3	0.75	1500	1.8	1.5	0.4	3	3.8	3.2	4
2.8	1.7	2.5	4600	2.6	1.9	0.6	4	4.9	4.2	5
>2.8	>1.7	*Use alternative material*								

Notes:

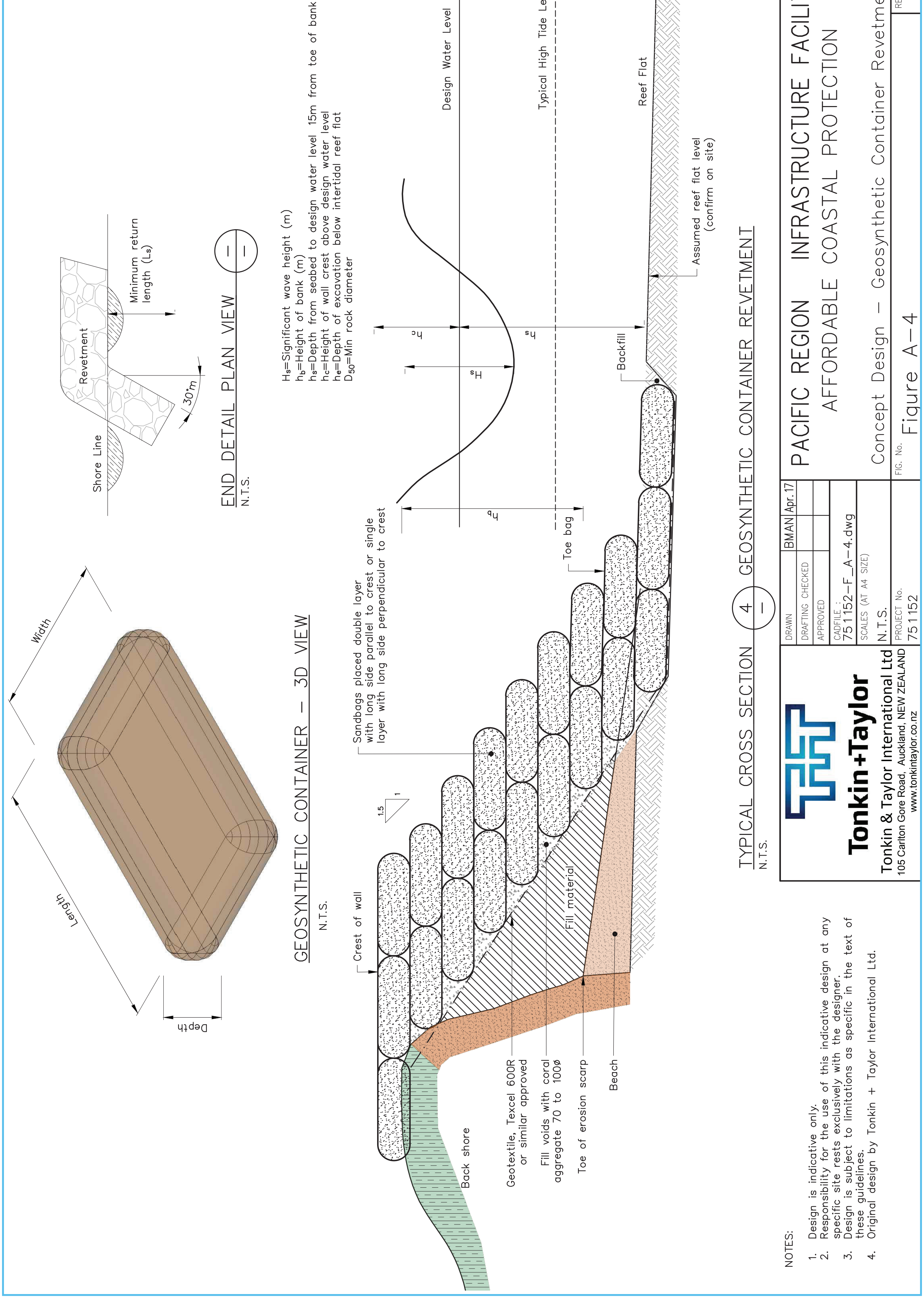
- Based on a front slope of 1(V):1.5(H), steeper revetments are slightly more stable under wave attack, although they are difficult to construct in dry sand
- Revetment can be single or double layer, although single layer is more vulnerable to failure
- Crest freeboard based on mean overtopping flows of less than 1 l/s for pedestrian safety and to minimise damage to backshore. If backshore is paved or vegetated, it may tolerate higher overtopping rates (EurOtop, 2016). Values given for Pacific Ocean coasts (Tp , 10s) and Lagoon (Tp <5s)
- Structure return length minimum of 3 x H_s but consider other erosion drivers and long-term shoreline trends
- Structure should be protected from flanking by extending alongshore beyond areas of active erosion, by tying into adjacent nonerodible structure, or by landward return at moderate angle beyond likely extents of future erosion.

Further design guidance

- Blacka, M., How, D. and Coghlan, I.R. (2017)
- Coghlan et al. (2009)
- Hornsey et al. (2011)
- EurOtop, 2016.

Material specifications	**Geosynthetic containers** ▪ GSCs shall be constructed of a nonwoven, needle-punched, continuous filament polyester or polypropylene geotextile ▪ Sand used to fill GSC should have density greater than 1,380 kg/m^3 ▪ Sand used for filling should not be larger than 10 millimetres to prevent puncture of the GSC ▪ GSC should be stored out of direct sunlight and not in contact with ground ▪ Torn or punctured GSC shall not be used ▪ GSC shall be filled, closed, and placed in accordance manufacturer's specifications. **Geotextile** ▪ Geotextile filter fabric shall be a nonwoven, needle-punched, continuous filament polyester or polypropylene geotextile ▪ Geotextile shall be Texcel® 600R or equivalent, approved by supervising engineer ▪ Fabric should be stored out of direct sunlight and not in contact with ground ▪ Torn or punctured fabric shall not be used. Note that full construction specifications should be prepared prior to construction.
Typical construction methodology	1. Establish site working area, including temporary fencing (landward of high tide) and signage 2. Set out revetment, including footprint, and levels with stakes and string lines 3. Remove existing sandy sediment from the revetment footprint and stockpile for later replacement 4. Excavate toe and place fill to achieve target slopes and levels; earthworks shall be undertaken in a controlled manner so that erosion of disturbed areas is kept to a practical minimum and eroded material is confined on site as far as possible 5. Protect toe trench and slope, as necessary, from tide and wave action, using temporary rock or sandbag bunds 6. Place geotextile against prepared slope with minimum of 0.5 m laps in all directions. 7. Fill, machine stitch, close, and place toe GSC, and wrap geotextile as shown 8. Fill, close, and place GSCs in rows 9. Replace the removed sandy sediment along toe of revetment or on adjacent beach 10. Remove all debris from site and set down area.
Monitoring and maintenance requirements	Physical inspections should be undertaken regularly and following large wave events. Inspections should include photographs, and observation and maintenance implications may include the following: ▪ Evidence of GSC slumping or displacement: slumping should be monitored and bags replaced as needed (difficult for larger GSCs) ▪ Evidence of wearing, split, or cut in GSC: repair immediately using patch, as directed by manufacturer ▪ Evidence of toe scour: additional GSC bags at the toe may be required if scour is excessive or additional sand is placed ▪ Scour behind the wall: wall may need to be raised and/or erosion-resistant material laid behind ▪ Scour at wall ends: wall may need to be extended alongshore or inland to prevent outflanking, or additional sand placed to counter erosion.
Climate change adaptation	Future sea level rise may result in higher water levels at the structure and allow higher waves to reach the structure. ▪ GSCs cannot generally be upgraded, except by placing a larger GSC in front. Increased wave height should be allowed for in design ▪ Increased overtopping can be reduced by raising the crest of the structure, using additional GSCs.

Figure A-4



Length
Width
Depth
GEOSYNTHETIC CONTAINER – 3D VIEW
N.T.S.
Shore Line
Revetment
Minimum return length (Ls)
30°m
END DETAIL PLAN VIEW
N.T.S.
1
–
Hs=Significant wave height (m)
hb=Height of bank (m)
hs=Depth from seabed to design water level 15m from toe of bank (m)
hc=Height of wall crest above design water level
he=Depth of excavation below intertidal reef flat
D50=Min rock diameter
Design Water Level
Typical High Tide Level
Reef Flat
Assumed reef flat level (confirm on site)
Backfill
Toe bag
Sandbags placed double layer with long side parallel to crest or single layer with long side perpendicular to crest
Crest of wall
1.5
1
Back shore
Geotextile, Texcel 600R or similar approved
Fill voids with coral aggregate 70 to 100ø
Fill material
Toe of erosion scarp
Beach
TYPICAL CROSS SECTION
N.T.S.
4
–
GEOSYNTHETIC CONTAINER REVETMENT
NOTES:
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Concept Design – Geosynthetic Container Revetment
FIG. No. Figure A-4
REV. 0

A-5 Grouted Rock Wall

Description	Grouted rock walls are constructed using stone or coral blocks, which are stacked and set in mortar with a geotextile and drainage layer behind, as well as drains through the structure They provide protection from wave impact and support the backing ground by using their weight and having a broad foundation base to prevent sliding and overturning. These structures require a well-founded toe, ideally on a hard stratum, since undermining has the potential to cause rapid and catastrophic failure. Alternatively, a deeply embedded toe or rock toe protection can be considered, although these require special design consideration. Unlike rock revetments, these structures provide impermeable barriers to waves, and rather than dissipating wave energy, waves are either reflected offshore or deflected upward, potentially causing substantial wave overtopping. Backshore protection is often required to limit damage by wave overtopping, and these structures are better suited to low wave environments. They are typically a lower-cost alternative to a mass-concrete wall, although with good quality rock and mortar (e.g., left photo below), they may have a moderate to long design life (30–50 years). The walls have much shorter design life (<10 years) using lower-quality rock (coral) and lower-strength mortar (right photo below).  **A well-constructed grouted rock wall in New Zealand (left) and informal grouted rock wall in the Republic of the Marshall Islands (right)**
Suitability	▪ Typically wave climates <1.5 m ▪ Hard seabeds (>3 Scala blows/50 mm) ▪ Additional toe protection, using a semi-rigid structure, may be required to prevent scour and undermining of the wall foundations. Alternatively, foundations can be embedded to deeper depths or deeply founded.
Materials required	▪ Supply of suitable stone or coral blocks ▪ Cement, water ▪ Concrete aggregate ▪ Drainage aggregate typically 20-30 millimetre grading ▪ Geotextile filter fabric.
Construction plant	▪ Barge or truck to deliver aggregates and cement (unless rock sourced from site) ▪ Concrete mixing plant.
Design life	▪ 30-50 years using volcanic rock and higher strength concrete (>30 megapascal (MPa)) ▪ 10-20 years using limestone and/or lower strength concrete (>20 MPa.)

Typical costs

Typical costs for various coastal protection works as a function of design wave height, material availability, and transport costs are set out in PRIF (2016), together with design and transport assumptions.

A summary of costs per linear metre (A$/li m) is set out below, assuming all materials are imported except where indicated as locally available. Use of local rock and concrete aggregate and imported cement will be slightly higher than local cost.

Supply Location	Low Wave Energy (Hs = 0.7 m) 1.5 m retained height	Low Wave Energy (Hs = 0.7 m) 3m retained height
Locally available material	400	1,350
Regional transport	600	1,900
International-primary port	1,100	3,150
International-remote location	1,800	5,000

Notes:

(i) Costs are indicative only and will fluctuate with material availability, market forces, and local taxes and tariffs;

(ii) costs should not be used for detailed project costing without input and review from qualified practitioner.

Design considerations

- Design wave height should be less than 1.5 m or design water depth less than 2.5 m
- Toe should be founded on hard seabeds (>3 Scala blows/50 mm) and below scour depth
- Geotechnical stability should be confirmed during detailed design, with front face slightly sloped to improve stability
- Geotextile should be used behind structure to limit loss of fine soil particles through structure and wrapped into or beneath armour layers
- Crest elevation should be set to limit overtopping to tolerable level for use and backshore material
- Structure should be protected from flanking by extending alongshore beyond areas of active erosion, by tying into adjacent non-erodible structure, or by landward return at shallow angle beyond likely extents of future erosion

ds (m)	Hs (m)	Crest Freeboard h_c (m)	
		Ocean	Lagoon
1.1	0.7	2	1.4
1.6	1.0	2.9	2.0
2.5	1.5	4.3	2.9
>2.5	>1.5	*Obtain specialist advice*	

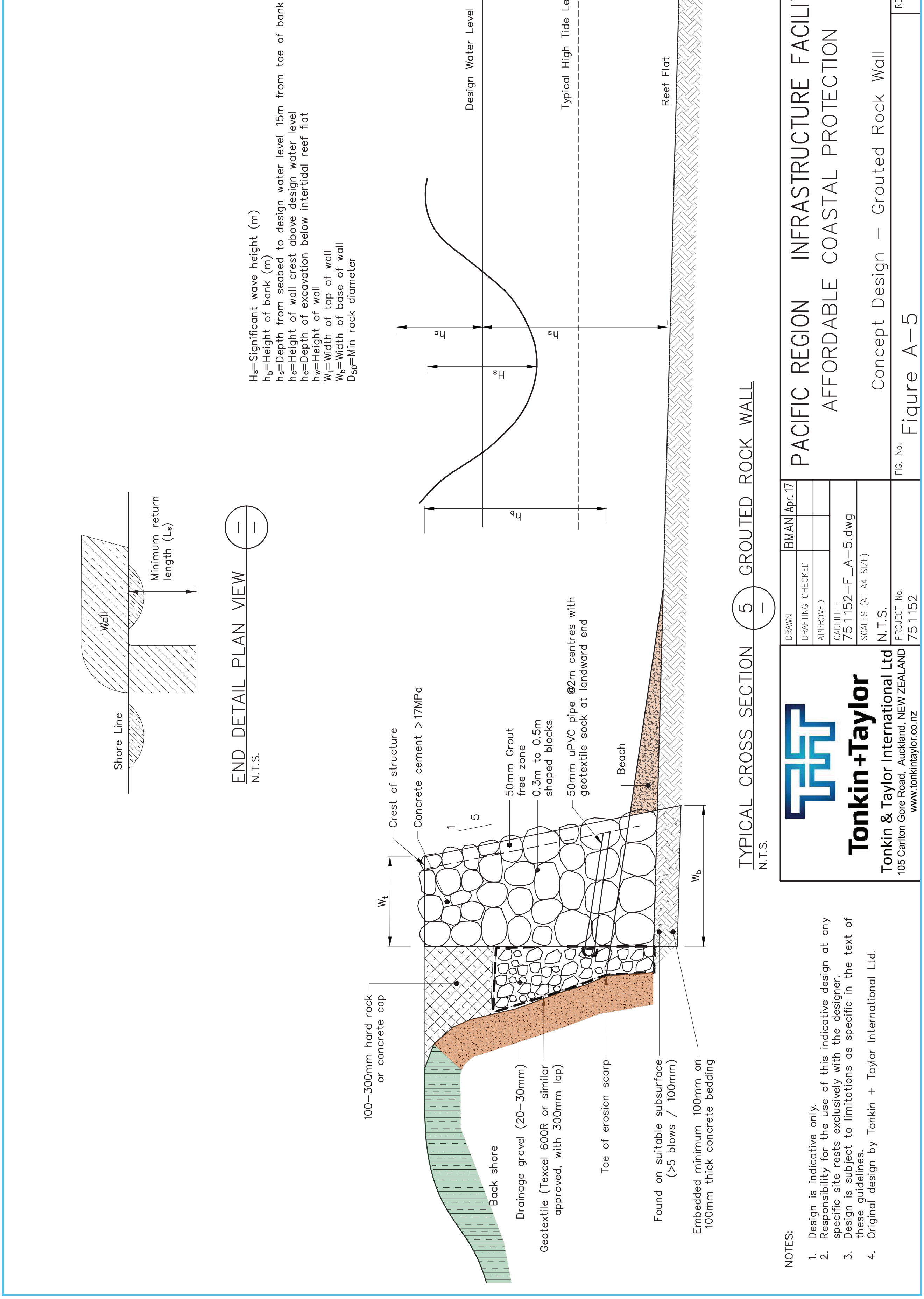
Retained Height (hw)	Wall Width	
	Base W_b (m)	Top W_t (m)
1.5	0.7	0.4
2.0	0.95	0.55
2.5	1.2	0.7
3.0	1.5	0.9
>3m	*Obtain specialist advice*	

Further design guidance

- USACE (2006)
- EurOtop, 2016.

Material specifications	**Rock** ▪ Rock ideally should be 300-500 millimetre (mm) shaped blocks (or smaller if approved) ▪ Rock generally shall be visually clean and free from impurities, such as clays and soils ▪ Rock shall be free from visually observable cracks, veins, fissures, laminations, unit contacts, cleavage planes, or other such flaws that could result in breakage during loading, unloading, or placing. **Cement mortar** ▪ Mortar for bedding and pointing shall consist of one part by volume of Portland cement to two parts by volume of clean fine aggregate, maximum aggregate size of 10 mm ▪ Compressive strength should be tested at 28 days, using 50 mm dia x 100 mm cylinders, and shall be at least 20 megapastal (MPa) (or as designed) ▪ Seawater is permissible but temperature should not exceed 35°C ▪ Mix design and testing should comply with local standard specifications. **Geotextile** ▪ Geotextile filter fabric shall be a nonoven, needle-punched, continuous filament polyester or polypropylene geotextile ▪ Geotextile shall be Texcel® 600R or equivalent, approved by supervising engineer ▪ Fabric should be stored out of direct sunlight and not in contact with ground ▪ Torn or punctured fabric shall not be used. Note that full construction specifications should be prepared prior to construction.
Typical construction methodology	1. Establish site working area, including temporary fencing (landward of high tide) and signage 2. Set out wall alignment and levels with stakes and string lines 3. Remove existing sandy sediment from the wall footprint and stockpile for later replacement 4. Excavate to required foundation depth 5. Pour 100 mm thick concrete bedding layer directly onto prepared undercut subgrade 6. Place rock into fresh concrete bedding layer so that the depth of embedment is between 30 mm and 50 mm, ensuring that a firm interlocking action is achieved. The voids shall be filled with 20 MPa cement-sand mortar, leaving the upper 20 mm of rock exposed and free from mortar, ready for the next placement of rock 7. Subsequent placement of rock layers should be into fresh mortar with voids being hand-filled with mortarm leaving 20 mm of exposed rock free of mortar. Ensure good contact between rocks and concrete/mortar is achieved 8. If rock placement has reached a suitable height, install pipe drainage as shown 9. Continue Steps 7 and 8 until the design wall height has been reached. Ensure a 50 mm grout-free zone is established for rocks lining the exposed wall face and that they are also placed to provide a relatively smooth surface following the design racked back angle (see appropriate figure) 10. Place geotextile against the prepared slope surface and up against the back of wall surface, ensuring sufficient overlap exists to wrap over the top the infilled drainage gravel 11. Place drainage gravel onto geotextile and lightly compact. Wrap over geotextile fabric as shown 12. Cover drainage gravel with an erosion resistant layer or larger rocks 13. Remove debris from site and set down area.
Monitoring and maintenance requirements	Physical inspections should be undertaken annually and following large wave events. Inspections should include photographs, and observation and maintenance implications may include the following: ▪ Evidence of unit displacement: grout may be weakened and blocks displaced. Blocks should be reset using 20 MPa sand mortar ▪ Evidence of toe scour: additional toe rock armour may be required if scour at toe is evident ▪ Scour behind the wall: wall may need to be raised and/or erosion-resistant material laid behind ▪ Scour at wall ends: wall may need to be extended alongshore or inland to prevent outflanking or additional sand placed to counter erosion.
Climate change adaptation	Future sea level rise may result in higher water levels at the structure and higher waves reaching the structure. Increased overtopping can be reduced by raising the crest of the structure, using a crown wall.

Figure A-5


END DETAIL PLAN VIEW
N.T.S.
Shore Line
Wall
Minimum return length (L_s)
H_s=Significant wave height (m)
h_b=Height of bank (m)
h_s=Depth from seabed to design water level 15m from toe of bank (m)
h_c=Height of wall crest above design water level
h_e=Depth of excavation below intertidal reef flat
h_w=Height of wall
W_t=Width of top of wall
W_b=Width of base of wall
D_{50}=Min rock diameter
Design Water Level
Typical High Tide Level
Reef Flat
h_c
H_s
h_s
h_b
100–300mm hard rock or concrete cap
Back shore
Drainage gravel (20–30mm)
Geotextile (Texcel 600R or similar approved, with 300mm lap)
Toe of erosion scarp
Found on suitable subsurface (>5 blows / 100mm)
Embedded minimum 100mm on 100mm thick concrete bedding
W_t
W_b
Crest of structure
Concrete cement >17MPa
1
5
50mm Grout free zone
0.3m to 0.5m shaped blocks
50mm uPVC pipe @2m centres with geotextile sock at landward end
Beach
TYPICAL CROSS SECTION 5 GROUTED ROCK WALL
N.T.S.
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Concept Design – Grouted Rock Wall
FIG. No. Figure A–5
REV. 0

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