Guidelines for seawall adaptation

Science and Synthesis for NSW

NSW Office of Environment and Heritage's Coastal Processes and Responses Node - Technical Report

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Climate Change Adaptation Options for Seawalls

WRL Research Report 264
September 2018

by
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Bibliographic Data Sheet

Report Title	Climate Change Adaptation Options for Seawalls
Report Author(s)	D Howe and R J Cox
Report No.	WRL Research Report 264
Report Status	Final
Report Date	27 September 2018
ISBN-13	978-0-7334-3777-9
Number of Pages	
Distribution Statement	
Price	
Sponsoring Organisation	Sydney Institute of Marine Science - ACCARNSI
Keywords	coastal erosion, coastal hazards, climate change, coastal adaptation

Document Status

Version	Reviewed By	Approved By	Date Issued
Final Draft	J. T. Carley	G P Smith	24 October 2017
Final	J. T. Carley	G P Smith	27 September 2018

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Abstract

Coastal structures in NSW are typically designed for depth-limited breaking wave conditions. With a projected sea level rise of up to 1 m by 2100, the design wave height for these structures is expected to increase. Many of these structures will require significant armour upgrades to accommodate these new design conditions. Increasing the armour mass is the simplest way to improve the stability of a coastal structure, but this may not be practical in all situations.

Many breakwaters and seawalls in NSW are armoured with quarry rock, but these structures are difficult to upgrade because of the low availability of large rock material in NSW (at present it is not economic to supply quarry rock larger than approximately 7 t).

Other structures are constructed with concrete armour units. Concrete armour units do not have the same size limitations as quarry rock, but their stability depends on interlocking between units. Stacking layers of different-sized units on top of each other can create weakness planes in the structure.

This study investigated the effectiveness of retrofitting existing rock and concrete-armoured coastal structures with additional (and more stable) primary armour. Physical modelling was used to enhance the understanding of unconventional designs that may arise when coastal structures are upgraded in response to sea level rise.

Executive summary

Coastal structures in NSW are typically designed for depth-limited breaking wave conditions. With a projected sea level rise of up to 1 m by 2100, the design wave height for these structures is expected to increase. Many of these structures will require significant armour upgrades to accommodate these new design conditions (for example, a 25% increase in wave height may require the armour mass to be doubled).

The Water Research Laboratory (WRL) of the School of Civil and Environment Engineering, University of New South Wales Sydney, was engaged through Australian Climate Change Adaptation Research Network for Settlements and Infrastructure (ACCARNSI) with funding from the NSW Office of Environment and Heritage (OEH), as part of the Coastal Processes and Responses Node of the Climate Adaptation Research Hub to investigate the effectiveness of retrofitting existing rock and concrete-armoured coastal structures with additional (and more stable) primary armour. Physical modelling was used to enhance the understanding of unconventional designs that may arise when coastal structures are upgraded in response to sea level rise.

Rock structures

Three upgrade strategies for rock-armoured structures were identified in this study, as shown in Figure 1, and described below.

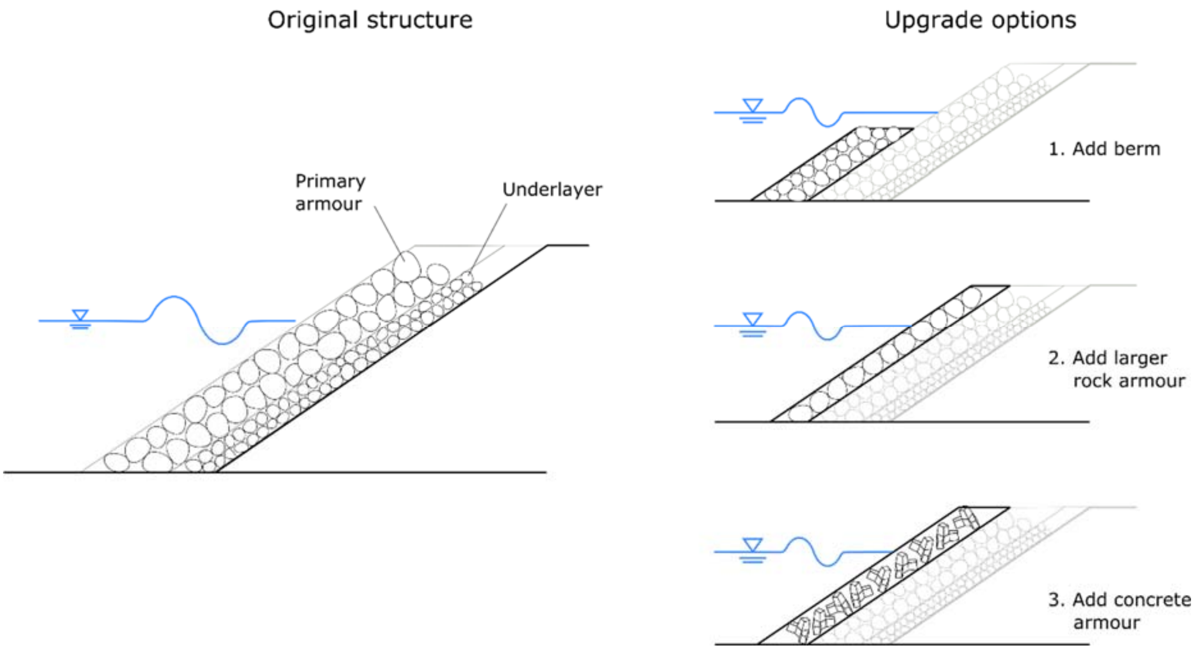

Figure 1. Upgrade options for structures with rock armour

Rock option 1. Add a berm to the seaward side of the structure

Berms cause waves to break offshore, reducing the amount of energy that reaches the main part of the structure. Icelandic berm breakwaters are designed to optimise the placement of armour when large rocks are in short supply. Their rock volume requirements and footprints are larger than conventional structures, but they may be more economical due to their efficient use of available rock.

Rock option 2. Add larger rock armour

Armour stability is proportional to mass, but it is often difficult to obtain large rock (greater than 7 t) economically in NSW, because quarries are usually set up to produce small aggregates for construction.

Rock option 3. Add concrete armour units (CAUs)

Concrete armour is more stable than rocks of the same mass, because the individual units interlock more with each other. Concrete armour units may be preferred in situations where rock armour of sufficient size is difficult to obtain locally.

Hanbar CAUs which are commonly used for coastal protection in NSW have been shown in model testing to be unstable for placing in a single layer directly onto existing rock structures. Upgrades with double layer Hanbar designs are stable but may not be cost-effective.

A summary of the upgrade options for rock structures is provided in Table 1.

Table 1. Upgrade options for rock structures

Option	Advantages	Drawbacks
1. Berm	• Makes efficient use of available rock.	• Large footprint requirements. • Comparatively difficult to design and construct (for Icelandic berm).
2. Larger rock	• Simple to design.	• Large rock may not be available for all locations.
3. Concrete armour	• Concrete units are smaller than equivalent rock armour. • Concrete is often easy to supply.	• Public access is restricted. • Aesthetic appeal is reduced.

Concrete structures

Three upgrade strategies for structures with concrete armour units (CAUs) were identified in this study, as shown in Figure 2 and described below.

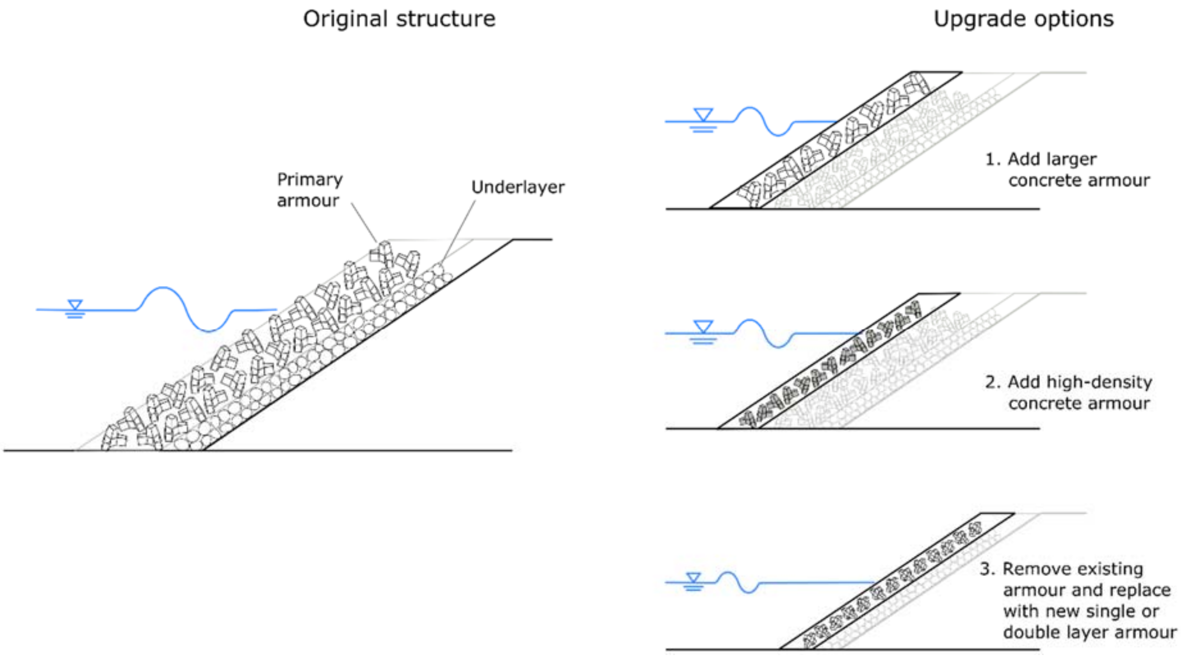

Figure 2. Upgrade options for structures with concrete armour units

Concrete option 1. Add larger concrete armour units

Larger concrete armour would increase the stability of a structure, but it may be difficult to achieve good interlocking with existing armour. A single layer of larger Hanbars performs poorly when placed on top of smaller Hanbars. A two layer upgrade may be possible, but would have large concrete and spatial footprint requirements.

Concrete option 2. Add high-density concrete armour units

High-density concrete can be used to produce concrete armour units with increased stability, while retaining the same dimensions as the existing armour on a coastal structure (therefore ensuring good interlocking). High density concrete has been made possible by recent advances in geopolymer concrete technology and use of steel furnace slag aggregates.

Concrete option 3. Remove existing armour and replace with alternative armour units

Placing additional armour above existing concrete armour may not always be possible, especially if the structural footprint is restricted. In these cases it may be appropriate to remove the existing concrete armour entirely, and replace it with new concrete armour units with enhanced stability. It would be desirable to use concrete armour with a higher stability coefficient than the existing armour (rather than larger units of the same type), to prevent washout of the underlayer. This option would be the most challenging to construct, because the structure would be vulnerable to storm damage while the primary armour is absent.

A summary of the upgrade options for structure with concrete armour is provided in Table 2.

Table 2. Upgrade options for concrete structures

Option	Advantages	Drawbacks
1. Larger concrete armour	• Relatively simple to design.	• Single layer upgrade may perform poorly (for Hanbars). • Double layer upgrade would be expensive, and have large footprint.
2. High-density concrete armour	• Stability is increased without compromising interlocking.	• High-density concrete is still under development.
3. Remove and replace armour	• Overall footprint is reduced.	• Structure is vulnerable to storms during construction. • Existing armour must be disposed of.

Conclusion

1. Sea level rise will cause increased design wave heights for coastal structures in NSW.
2. For quarries in NSW to economically produce large volumes of rock, the maximum individual rock mass is approximately 7 t.
3. Hanbar armour units should not be placed in a single layer, except for on top of existing Hanbars of similar size.
4. High-density concrete armour can potentially be placed on existing armour with the same dimensions, to provide enhanced stability while retaining good interlocking.
5. Physical modelling should be used to ensure satisfactory performance of a structure during detailed design.

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1 Introduction

The Intergovernmental Panel on Climate Change (IPCC) projects that mean global sea levels will rise significantly during the 21st century (Figure 1.1).

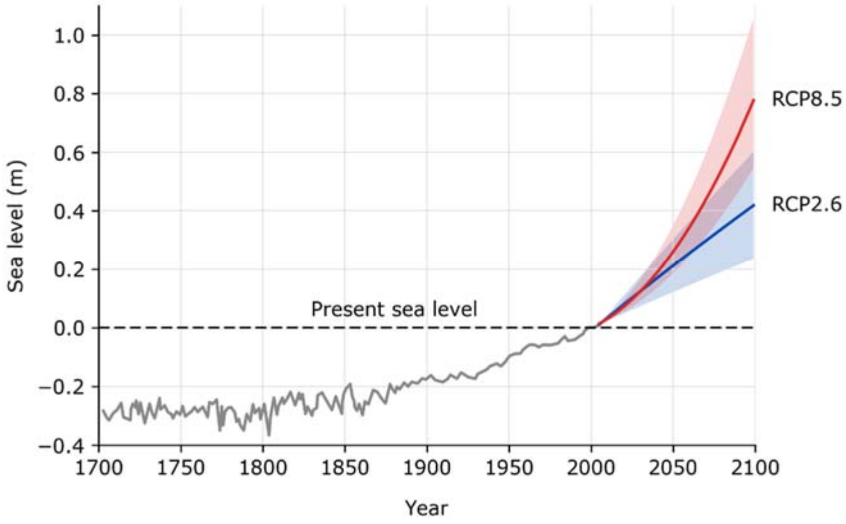


Figure 1.1. Historical sea levels projections of global mean sea level rise (after IPCC, 2014)

Sea level rise (SLR) on the NSW coastline is projected to be between 0.24 m (under RCP2.6, a low emissions scenario) and 1.06 m (under RCP8.5, a high emissions scenario) by 2100, compared to 2000 levels (Table 1.1).

Table 1.1. SLR projections above 2000 levels for NSW, in metres (Glamore et al., 2015)

Scenario	2050	2100
Lowest estimate (RCP2.6, minimum)	0.14	0.24
Low estimate (RCP2.6, mean)	0.22	0.42
High estimate (RCP8.5, mean)	0.27	0.78
Highest estimate (RCP8.5, maximum)	0.36	1.06

Sea level rise will have a significant impact on coastal infrastructure. With 80% of the Australian population living within 50 km of the coast (Hugo et al., 2013), much of the important social and economic infrastructure is located in coastal areas, including (NCCARF, 2012):

- Local government and community assets and recreational open space;
- Coastal residential property;
- Water supply, wastewater, electricity and telecommunication networks; and
- Transport infrastructure, such as roads, bridges, airports, and harbours.

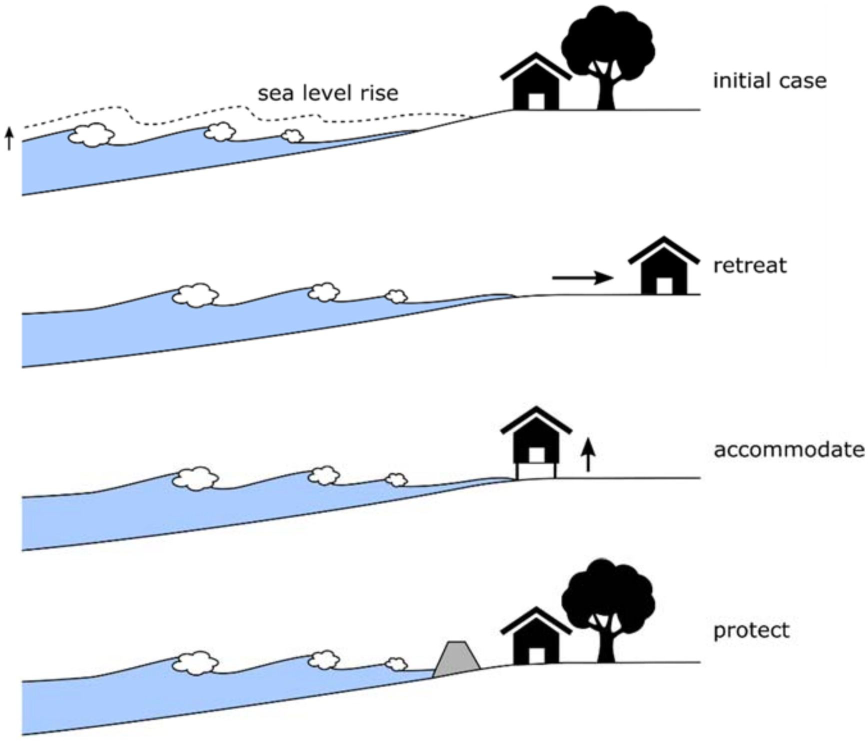


Figure 1.2. Adaptation options for coastal settlements

Adaptation options for coastal settlements and infrastructure in response to sea level rise include retreat, accommodate, and protect (Figure 1.2). For developed shorelines with high asset value the protect option is frequently pursued. The most common choices to support this option are sand nourishment and seawalls.

Seawalls and breakwaters are designed with heavy primary/outer armour rock or concrete units that are sized to resist the impact forces of waves that can break upon them. With the required mass of the armour rock/units being proportional to the cube of the impacting waves, existing structures can be expected to undergo major damage and/or even destruction with projected increases in design waves arising from climate projections for increases in storm intensity and sea level rise. Information is needed to enable decision makers using a risk-based approach to optimise upgrades for existing structures and design for staged construction of resilient and adaptive new seawalls.

WRL was engaged through Australian Climate Change Adaptation Research Network for Settlements and Infrastructure (ACCARNSI) with funding from the NSW Office of Environment and Heritage (OEH) as part of the Coastal Processes and Responses Node of the Climate Adaptation Research Hub to investigate the effectiveness of retrofitting existing rock and concrete-armoured coastal structures with additional (and more stable) primary armour. Physical modelling was used to enhance the understanding of unconventional designs that may arise when coastal structures are upgraded in response to sea level rise.

2 Impacts of sea level rise

Maximum wave heights along the NSW coastline are typically depth limited close to shore (Short, 1999), so the design wave heights for coastal structures can be determined using the local water depth. Sea level rise relaxes the depth limitation of waves, resulting in larger wave heights and runup levels (Arns et al., 2017). As sea levels increase, the surf zone moves towards the land, and fixed coastal structures can be exposed to larger waves (Figure 2.1).

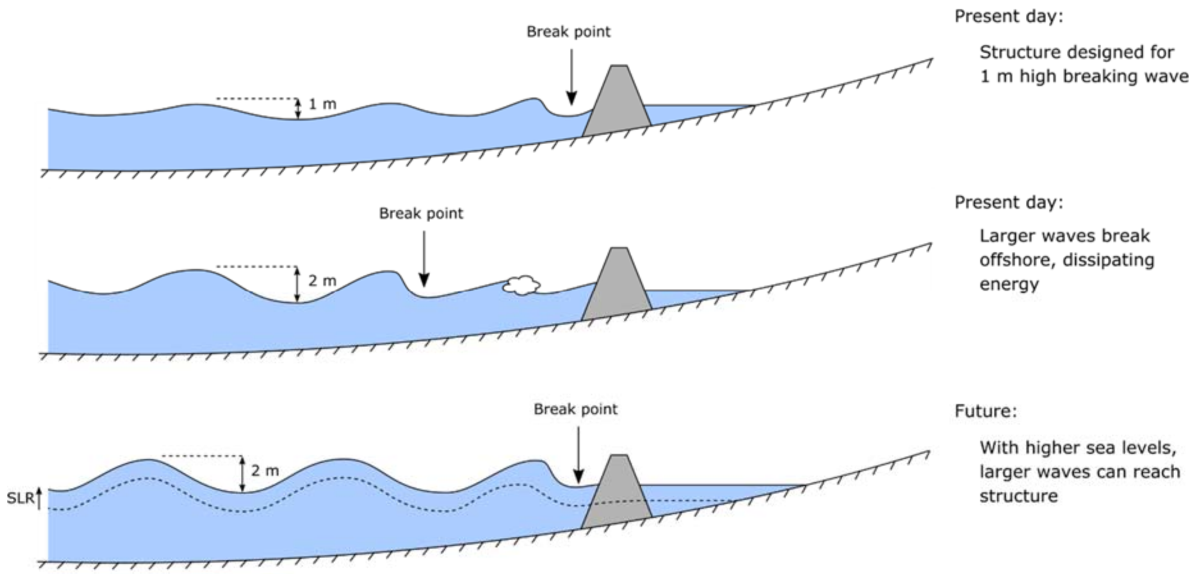


Figure 2.1. Impact of sea level rise on depth-limited waves

2.1 Breaker index in NSW

The height of a breaking wave (H_b) is related to the water depth (d_b) by the breaker index (γ):

$$\gamma = \frac{H_b}{d_b} \quad (1)$$

McCowan (1891) used solitary wave theory to calculate a shallow water breaker index of 0.78, but laboratory experiments using irregular waves have found this value to be overly conservative for low gradients, and suggest a maximum breaker index of approximately 0.55 (Nelson, 1985; Riedel and Byrne, 1986; Dack and Peirson, 2005).

Goda (2010) developed a general formula for calculating a breaker index for irregular waves on a sloping bottom:

$$\gamma = \frac{H_{b,1/3}}{d_b} = \frac{0.12}{d_b/L_0}\left\{1 - exp\left[-1.5\,\pi\frac{d_b}{L_0}\left(1 + 11s^{\frac{4}{3}}\right)\right]\right\} \quad (2)$$

where:

$H_{b,1/3}$	Significant breaker height	(m)
L_0	Deep water wavelength	(m)
s	Bottom slope	(-)

Nearshore seabed slopes in NSW usually range from 1V:20H to 1V:50H (Short, 2007), and wave periods during storm conditions are typically around 12 s (Figure 2.2). Using these representative values, the Goda (2010) equation yields a breaker index of 0.6 for the NSW coastline (Figure 2.3).

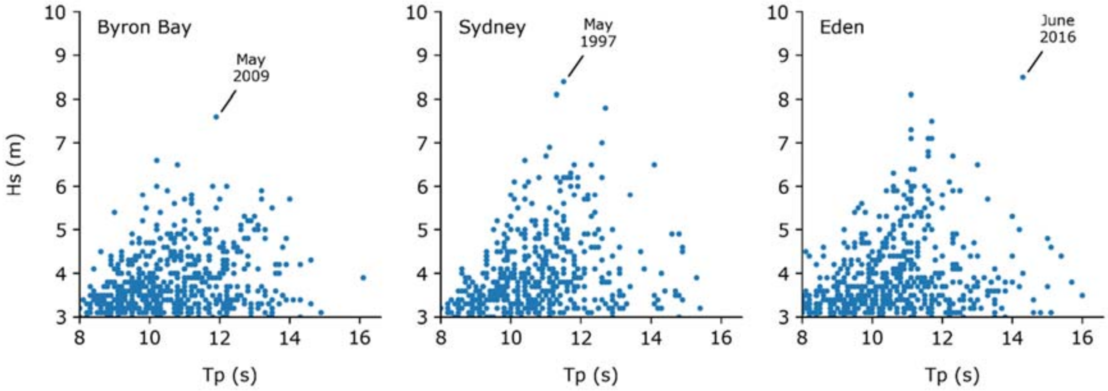


Figure 2.2. Wave conditions observed using Waverider buoys during storms (MHL, 2017)

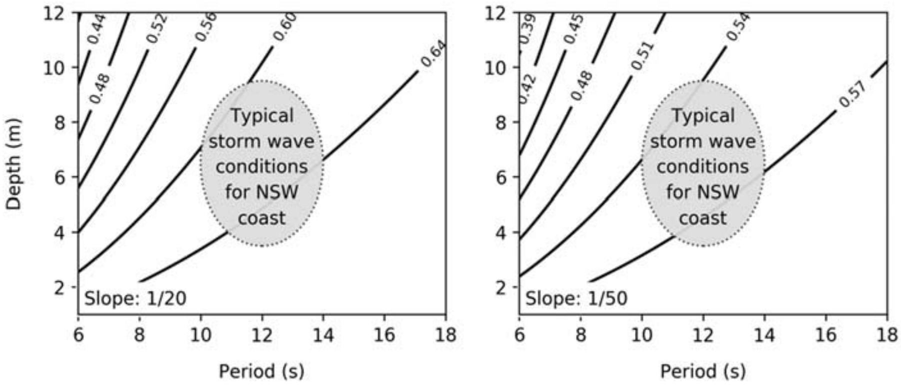


Figure 2.3. Breaker index values calculated using Goda (2010)

2.2 Increased armour requirements for coastal structures

The required mass of breakwater armour is proportional to the wave height cubed (see Section 3). Larger depth-limited wave heights driven by sea level rise will increase the armour mass requirements for fixed coastal structures, especially those in shallow water (Figure 2.4)

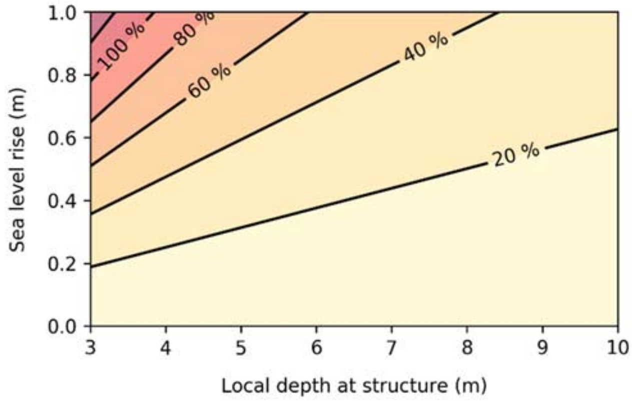


Figure 2.4. Increased armour mass requirements, based on a breaker index of 0.6

3 Breakwater design theory

3.1 Conventional rubble mound structures

Stable rubble mound structures are constructed from stones which are large enough to be stable when exposed to wave action in the design event. A structure consisting exclusively of large stones is not practical for two reasons (USACE, 2006):

1. Large stones are expensive to supply; and
2. Mounds of large stones are highly porous, which allows wave energy to penetrate through the structure.

As a result, conventional rubble mound structures are constructed with a core of fine quarry run material beneath a layer of large primary armour of fairly uniform size. An underlayer (or secondary armour layer) of intermediate rock sits above the core, to prevent the fine material from washing out through the armour layer (Figure 3.1).

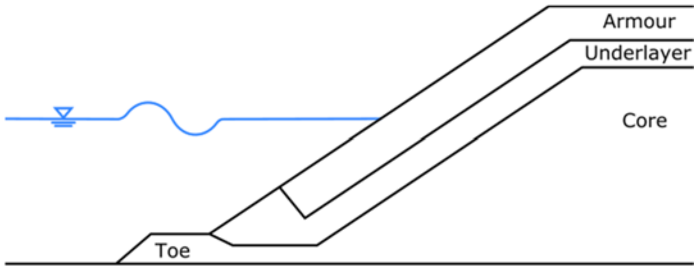


Figure 3.1. Conventional breakwater design

3.2 Unconventional rubble mound structures

3.2.1 *Low-crested structures*

Some early low-crested structures were created accidently. The Rosslyn Bay breakwater in Queensland sustained significant damage during cyclone David in 1976, but still performed effectively as a submerged breakwater for more than two years before repairs were completed (Foster et al., 1978). Experience gained from Rosslyn Bay was used to design an offshore breakwater in Townsville, with the ability to reshape or fail under extreme wave conditions (Figure 3.2), while still providing protection to the harbour behind it (Bremner et al., 1980).

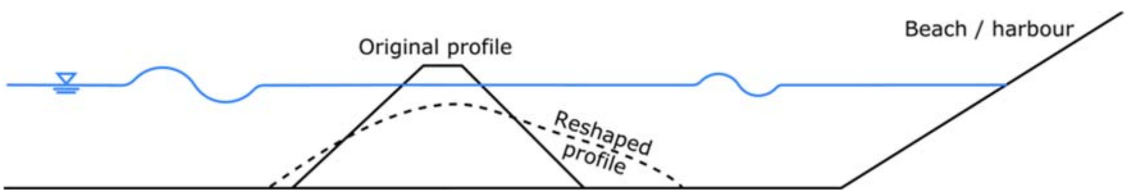


Figure 3.2. Reshaping reef breakwater

Conventional breakwaters are typically designed to minimise overtopping. In contrast, low-crested structures allow a large amount of wave energy to pass over them. This means smaller armour can be used for a low-crested structure, compared with the armour on the front of a non-overtopping structure (Burcharth et al., 2006). In addition, low-crested structures require much less material to build because the fill volume increases rapidly with the height of the crest.

3.2.2 Berm breakwaters

In contrast to conventional rubble mound structures with armour of uniform size, berm breakwaters use a thick layer of smaller armour rock with a wide mass grading. This allows berm breakwaters to optimise the use of locally available quarry rock (Figure 3.3), especially where large rock is difficult to produce (van der Meer and Sigurdarson, 2016).

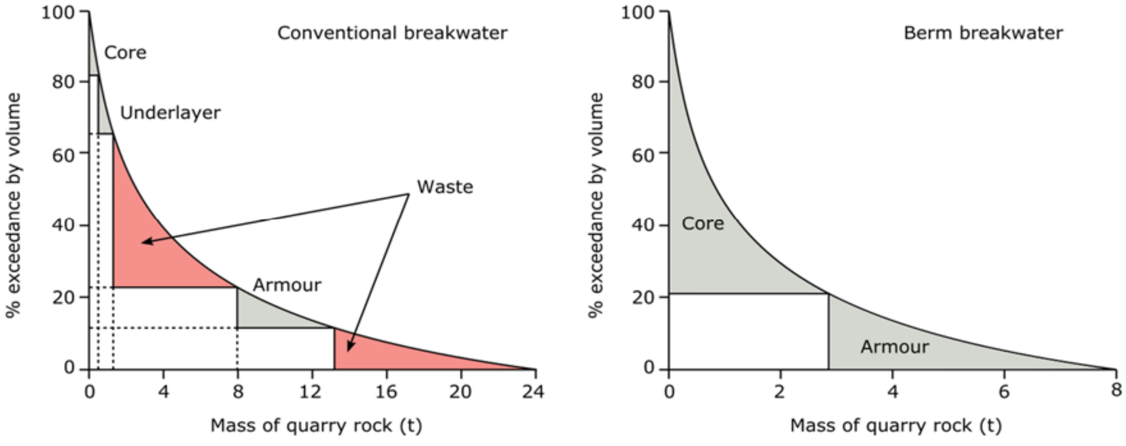


Figure 3.3. Quarry yield and rock masses required for breakwaters (after Baird et al., 1984)

Sigurdarson and van der Meer (2012) classify berm breakwaters into two types:

1. **Mass-armoured berm breakwaters**, characterised by a homogenous berm with a single rock class that can be partially or fully reshaped under wave action (Figure 3.4); and

2. **Icelandic breakwaters**, characterised by a structure with multiple rock classes that is designed to experience only minor reshaping (Figure 3.5)

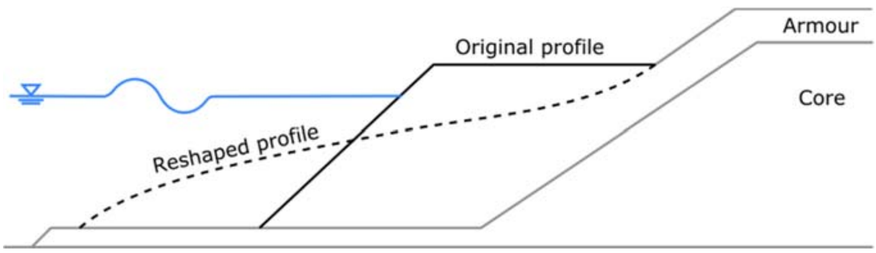


Figure 3.4. Mass-armoured berm breakwater

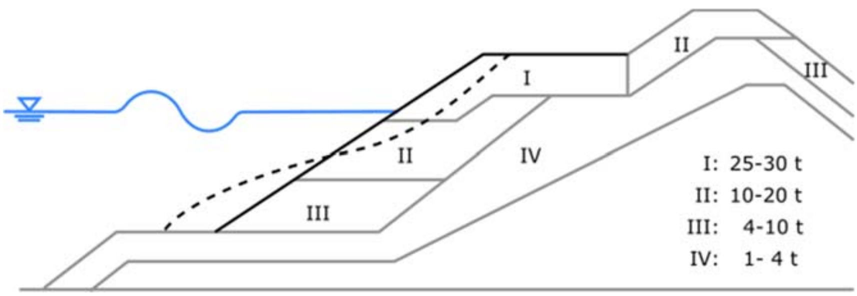


Figure 3.5. Icelandic berm breakwater, with four armour classes

Berm breakwaters tend to form an S-shaped profile when they are reshaped. This results in a local flattening of the breakwater which causes the waves to break further offshore, dissipating energy before they impact directly on the structure.

Over time designers have tended to favour breakwaters with more armour classes and less reshaping (van der Meer and Sigurdarson, 2016). Icelandic berm breakwaters are recommended over mass-armoured berm breakwaters (even if just two classes are used), because:

- Larger rocks can be placed in the region of wave attack, maximising overall stability; and
- Narrow rock gradings result in higher permeability and better wave energy dissipation.

While relatively uncommon, some berm breakwaters have been constructed in Australia, including:

1. Mackay, QLD; and
2. Shell Cove, NSW.

The Mackay small craft harbour breakwater sustained significant damage during Tropical Cyclone Ului in 2010 (Colleter et al., 2011). Sigurdarson et al. (2012) suggest that the extensive damage could have been avoided if efforts had been made to sort the larger armour stone during construction, and place it in the most exposed areas of the breakwater cross section.

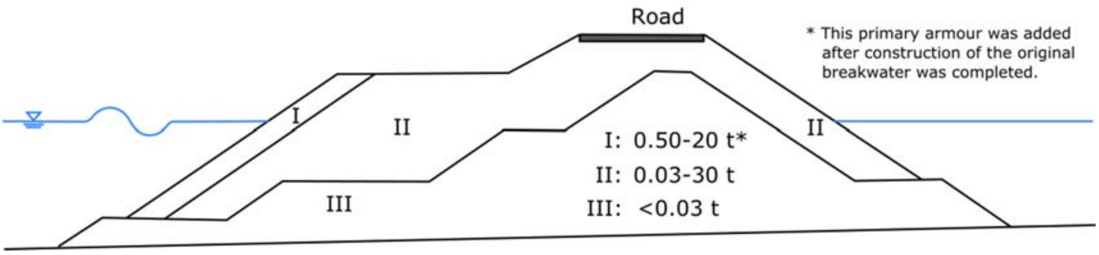


Figure 3.6. Cross section of Mackay small craft harbour breakwater (Johnson et al., 1999)

An Icelandic berm design was chosen for the Shell Cove breakwater (Figure 3.7) instead of a concrete armour unit structure. The following advantages of an Icelandic berm breakwater were identified (Britton et al., 2017):

- The individual armour units are smaller;
- Wave run-up and overtopping are reduced;
- The structure can sustain high levels of damage before failure;
- Rock structures can be more aesthetically pleasing than concrete armour units; and
- Maintenance is simpler due to smaller armour units.

Some limitations of Icelandic berm breakwaters (compared to conventional concrete armour structures) include:

- Construction is more complex because careful sorting and placing is required;
- Larger volumes of rock are required;

- Overall footprint is larger; and
- Local quarries are required to supply rock armour.

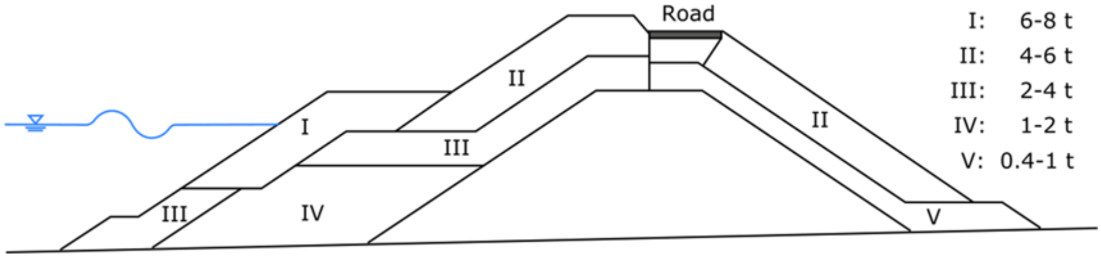


Figure 3.7. Typical cross section of Shell Cove breakwater (Britton et al., 2017)

3.3 Stability parameter sensitivity

The relationship between wave height and armour mass was described by Iribarren (1938), and was later developed by (Hudson, 1959), who derived this equation:

$$\boldsymbol{M = \frac{\rho_a H^3}{K_D \Delta^3 \cot\theta}}, \tag{3}$$

and

$$\boldsymbol{\Delta = \frac{\rho_a}{\rho_w} - 1}, \tag{4}$$

where:

M	Armour mass	(kg)
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
H	Design wave height	(m)
K_D	Stability coefficient	(-)
Δ	Relative submerged density	(-)
θ	Seaward slope	(rad)

Other design formulae have been developed (e.g. Van der Meer, 1987), but Hudson's equation provides practical insight into the factors that determine the overall stability of breakwaters exposed to wave attack (Figure 3.8).

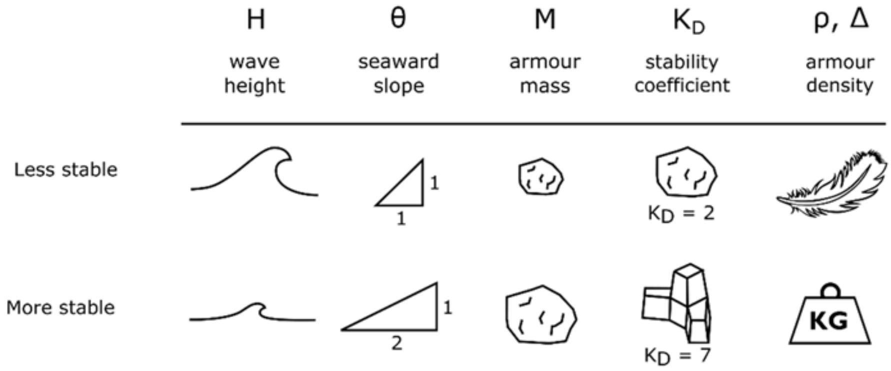


Figure 3.8. Hudson's equation: impact of individual terms on overall stability

3.3.1 Wave height

The wave climate at a coastal structure is usually beyond the designer's control, but in some cases incident wave heights can be reduced before they reach the coastline by constructing a reef offshore. Submerged reefs (especially coral fringing reefs in tropical regions) have long been recognised for their ability to provide natural protection from waves. Waves first break on the reef when they experience depth-limited conditions, then continue to lose energy as they propagate across a dissipation zone before reaching the shoreline. In the same way, low-crested (or submerged) breakwaters can be created in the nearshore to provide coastal protection to beaches or structures behind them (Ahrens and Cox, 1990).

3.3.2 Seaward slope

Milder seaward slopes of the structure are generally more stable than steeper slopes in units without high interlocking (e.g. rock). The angle of repose of rock tipped underwater can be as high as 1V:1.2H, but most rock armour breakwaters are constructed with a slope of 1V:1.5H (CIRIA, 2007, p.795). Milder slopes can generally be used to improve stability, at the cost of increased material requirements and a larger footprint (Figure 3.9), but milder slopes do not provide increased stability for highly-interlocking single layer concrete armour units (such as Accropode, Core-loc, and Xbloc; Table 3.1).

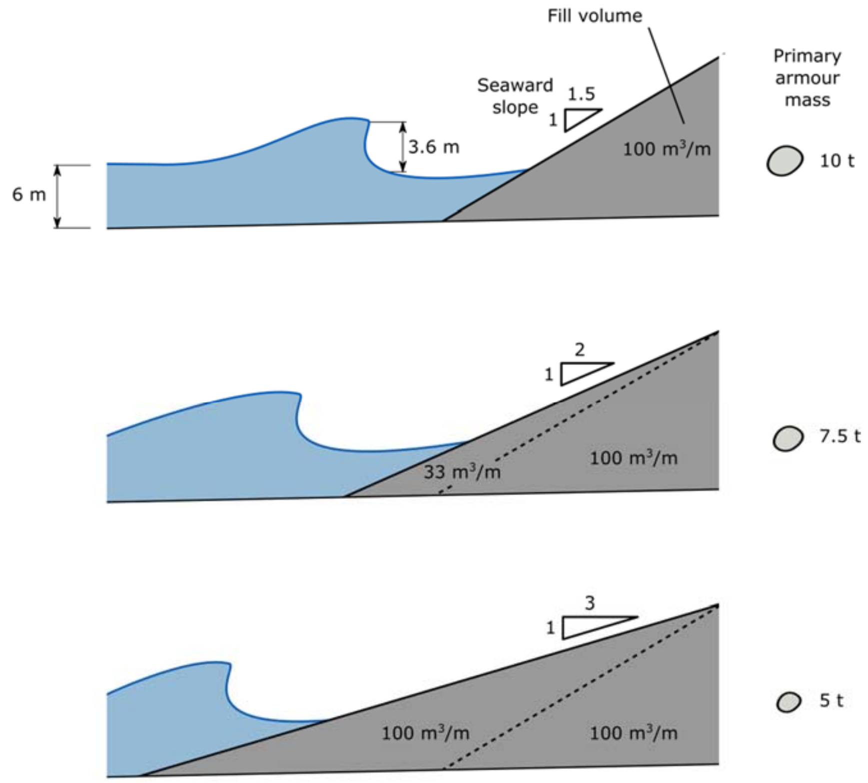


Figure 3.9. Impact of seaward slope on armour size, volume, and structural footprint

3.3.3 Armour mass

Iribarren (1938) established that the required armour mass of rubble mound breakwater is proportional to the wave height cubed, i.e. a 25% increase in wave height will require a doubling of the armour mass. Increasing the armour mass is the simplest way to improve the stability of a breakwater.

Rock availability varies widely across different regions. The maximum class designation in the European Standard EN 13383 is 10-15 t (CIRIA, 2007), but some Norwegian quarries can provide larger material, up to 50 t (Mibau, 2017). In NSW it is difficult to economically obtain large quantities of rocks with individual mass greater than 7 t (Coghlan et al., 2013).

The maximum mass of concrete armour is dependent on the geometry of the armour unit. Concrete cubes up to 150 t (side length of approximately 4 m) have been used as breakwater armour (Arquero, 2008), but complex concrete armour units are limited to smaller masses, typically 40-60 t (Bosman, 1980; DMC, 2014; CLI, 2015). Hanbars up to 28 t have been used in NSW (Blacka et al., 2005)

3.3.4 Stability coefficient

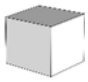
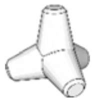
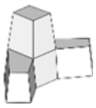
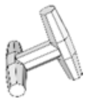
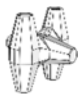
Hudson’s stability coefficient relates to the ability of the armour to interlock. There are three broad classes of armour (Table 3.1):

1. Massive;
2. Slender; and

3. Blocky.

Massive armour (such as rock) obtains stability from its mass. Early concrete armour units (CAUs) were typically slender shapes placed in two layers, and relied on interlocking (rather than mass) for their stability. However, if slender units rock back and forth under wave action there is a risk that their legs will break off and their interlocking ability will be compromised.

Table 3.1. Selection of breakwater armour

	Name	Origin	Year	Type	Layers	k_D[1]
	Rock	-	-	Massive	2	2
	Cube	-	-	Massive	2	6
	Antifer	France	1973	Massive	2	7
	Tetrapod	France	1950	Slender	2	7
	Hanbar	Australia	1979	Slender	2	7 [2]
	Dolos	South Africa	1963	Slender	2	16
	Tribar	U.S.A.	1958	Bulky	1	12 [3]
	Seabee	Australia	1978	Bulky	1	- [4]
	Accropode	France	1980	Bulky	1	15 [3]
	Core-loc	U.S.A.	1996	Bulky	1	16 [3]
	Xbloc	Netherlands	2003	Bulky	1	16 [5]

Notes:
1. k_D values are for the structure tunk exposed to breaking waves, up to 5% damage. Source: USACE (1984) unless otherwise noted.
2. Source: Blacka et al. (2005).
3. Source: USACE (2006).
4. No k_D value available – different design method to be used for Seabees.
5. Source: Muttray et al. (2003).

Following a number of catastrophic failures of Dolos structures in the 1970s and 1980s (Maddrell, 2005), there has been a decline in the use of slender units, and a return to massive units, particularly the Antifer Cube. Recent development of concrete armour has been focussed on single layer bulky units, which are not susceptible to breakage from rocking (USACE, 2006). Single layer bulky units can be highly stable, but the units require careful placement during construction. In general, the more sophisticated the armour, the more catastrophic the failure may be, and the more difficult to retrofit and repair (Gordon, 2014).

3.3.5 Armour volumetric mass density

Like Hudson's equation, the (van der Meer, 1987) equation also uses the dimensionless number Δ to represent the submerged relative density of the armour material:

$$\frac{\boldsymbol{H_s}}{\boldsymbol{\Delta D_{n50}}} \times \sqrt{\boldsymbol{\xi}} = \mathbf{6.2}\ \boldsymbol{P}^{\mathbf{0.18}} \left(\boldsymbol{S}/\sqrt{\boldsymbol{N}}\right)^{\mathbf{0.2}}, \tag{5}$$

where:

H_s	Significant wave height	(m)
D_{n50}	Armour diameter	(m)
ξ	Surf similarity parameter	(-)
P	Core permeability	(-)
S	Damage level	(-)
N	Number of waves	(-)

Both equations take slightly different forms, but they can both be rearranged in terms of H/D:

van der Meer:

$$\left(\frac{\boldsymbol{H_s}}{\boldsymbol{D_{n50}}}\right)^3 = \boldsymbol{\Delta}^3 \left(\mathbf{6.2}\ \boldsymbol{P}^{\mathbf{0.18}} \left(\boldsymbol{S}/\sqrt{\boldsymbol{N}}\right)^{\mathbf{0.2}} / \sqrt{\boldsymbol{\xi}}\right)^3, \tag{6}$$

Hudson:

$$\left(\frac{\boldsymbol{H}}{\boldsymbol{D}}\right)^3 = \boldsymbol{\Delta}^3\, \mathbf{K_D} \mathbf{cot}\, \boldsymbol{\theta}, \tag{7}$$

so that both equations give:

$$\left(\frac{\boldsymbol{H}}{\boldsymbol{D}}\right)^3 \propto \boldsymbol{\Delta}^{\boldsymbol{x}} \tag{8}$$

where $x = 3$. Since Hudson's work, several authors have investigated the effects of submerged relative density, and have generally suggested that x is less than 3 (Table 3.2).

Table 3.2. Investigations into the effect of submerged relative density

Investigation	Material	SG_{min}	SG_{max}	x
Hudson (1959)[1]	Rock	2.60	3.10	3.0
van der Meer (1987)	Rock	1.94	3.05	3.0
Helgason et al. (2000)	Rock	2.65	3.05	2.7
Scholtz et al. (1982)	CAU (Dolos)	1.81	3.02	2.3
Triemstra (2000)	CAU (cube)	2.20	3.90	3.0
Howe and Cox (2017)	CAU (Hanbar)	2.35	2.80	3.0

Notes: 1. Hudson did not investigate the effects of material density in isolation.

The most extensive study was that of Scholtz et al. (1982), who used model Dolos armour units with specific gravity (SG) values of 2.31, 2.41, and 2.57 (Zwamborn, 1980), and then conducted a follow-up investigation with SG values of 1.81, 2.39, and 3.02 (Zwamborn and van Niekerk, 1982). They concluded that $x = 2.3$, (for the extended range of SG from 1.81 to 3.02) which is considerably less than the original value of $x = 3$ given by Hudson.

Howe and Cox (2017) reanalysed the results from Zwamborn (1980) and Zwamborn and van Niekerk (1982) and concluded that their results were probably overly conservative, because it is unlikely that a coastal structure would be constructed from low-density concrete. If the results from the lower-density Dolos units ($SG < 1.8$) are ignored, a power curve fits the remaining data best with $x = 2.9$.

WRL completed an investigation into the relative stability of Hanbar units with different specific gravity values (Appendix A), and found that $x = 3$ (Figure 3.10).

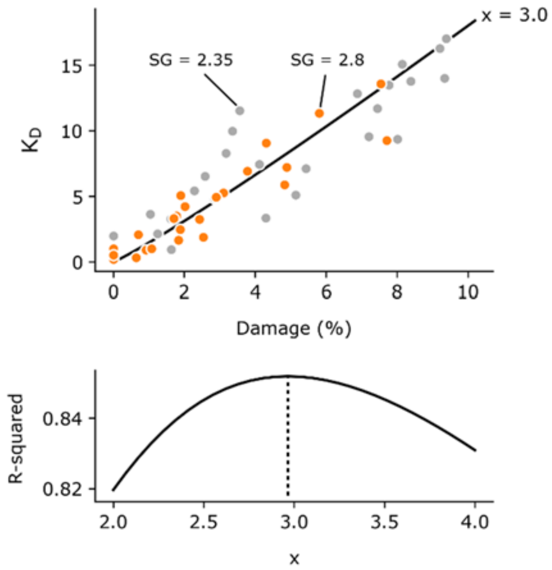


Figure 3.10. Results from experiments C1 and C2 (Appendix A)

When all of the x values are compared, with revised values for Scholtz et al. (1982), it is clear that some uncertainty remains as to the impact of relative submerged density on breakwater armour stability (Figure 3.11).

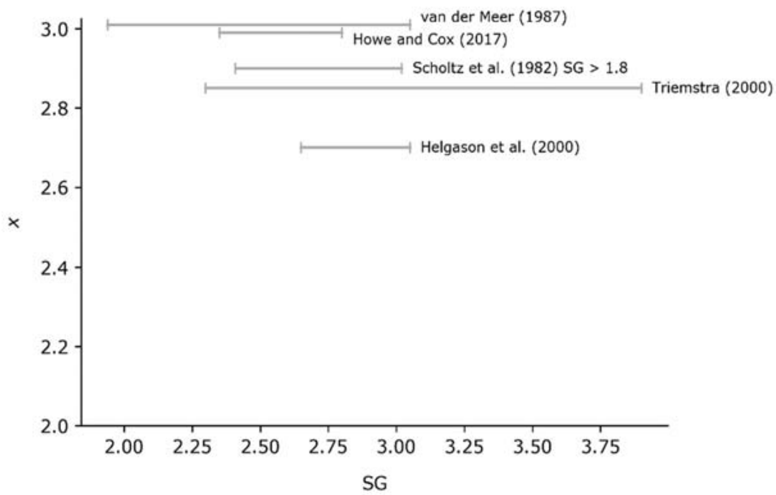


Figure 3.11. Results, and ranges of specific gravity used in experiments

For preliminary design of rock and CAU structures, a value of $x = 3$ should be adopted. If high-density armour is used to improve the stability of a structure, physical modelling should be conducted during the detailed design phase to confirm the effects on overall stability.

3.4 Failure modes

Hudson's equation can predict the stability of armour on the front face of a seawall or breakwater near the mean water level, but there are other failure mechanisms that must be considered (Figure 3.12).

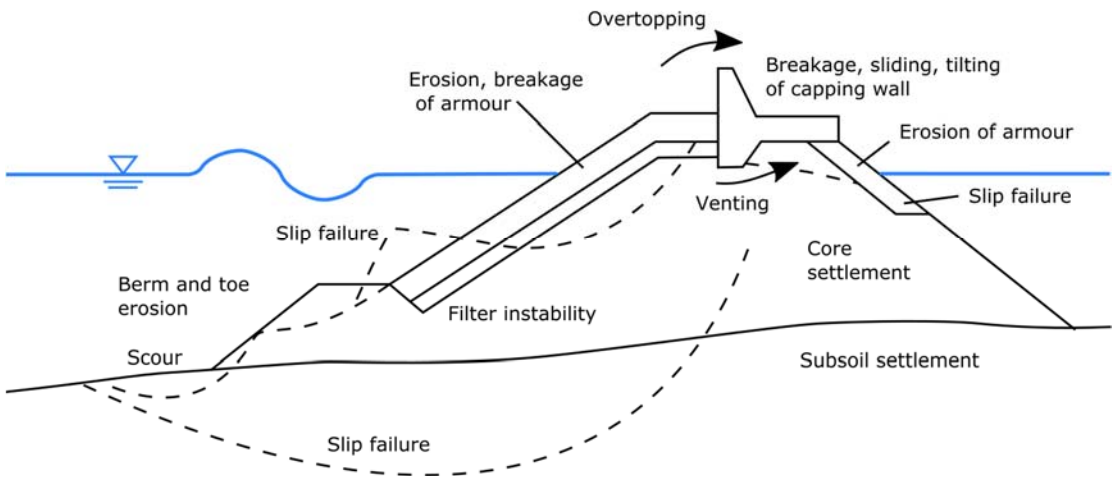


Figure 3.12. Breakwater damage mechanisms (after Burcharth and Liu, 1995)

A detailed discussion of breakwater failure modes is provided in Burcharth and Liu (1995) and CIRIA (2007).

4 Breakwaters and training walls in NSW

Australia's first breakwaters were constructed by tipping rocks into the sea and allowing them to settle at their angle of repose, similar to a railway embankment (Kraus, 1996). Additional rock was placed on these structures to repair storm damage, where required. Coastal structures have been designed more carefully since 1970 to reduce maintenance costs, and physical modelling has often been used to optimise designs (Foster, 1984).

The NSW Breakwater Asset Appraisal evaluates the state of breakwaters in NSW (MHL, 1994). It contains descriptions of 33 different sites managed by NSW Crown Lands, most of which consist of two or more separate structures. Many of these structures are armoured with rock obtained from nearby quarries (Figure 4.1).

Figure 4.1. Map of breakwaters (MHL, 1994) and quarries (DPWS, 1997) in NSW

Technically most of the structures contained in the NSW Breakwater Asset Appraisal would be classified as training walls (designed to guide river entrances, often constructed perpendicular to the coastline) rather than breakwaters (designed to reduce wave action in a shore area or harbour, typically constructed parallel to the coastline). In this report, training walls and similar structures are referred to as breakwaters, as the design principles and damage mechanisms are similar.

Some sites are omitted from the NSW Breakwater Asset Appraisal because they are managed by councils or other authorities. These include:

- Newcastle harbour;
- Port Botany;
- Sydney Airport;
- Port Kembla;
- Barrack Point;
- Shell Cove; and
- Bass Point.

The 1994 Breakwater Asset Appraisal was used for this study. Since it was published, training walls have been constructed at Lake Illawarra Entrance in 2000, and a number of breakwaters have been repaired or upgraded, including:

- Ballina;
- Clarence;
- Coffs Harbour;
- Forster; and
- Eden.

Even though the present condition of each structure is not up to date, the appraisal provides a useful overview of the design conditions and construction history of each structure. Estimates of required armour masses calculated using Hudson's equation show that the primary rock armour on many of the breakwaters in NSW is undersized for present-day conditions (Figure 4.2). Most sites are designed for depth-limited conditions, so sea level rise will lead to larger discrepancies between required armour size and actual armour size.

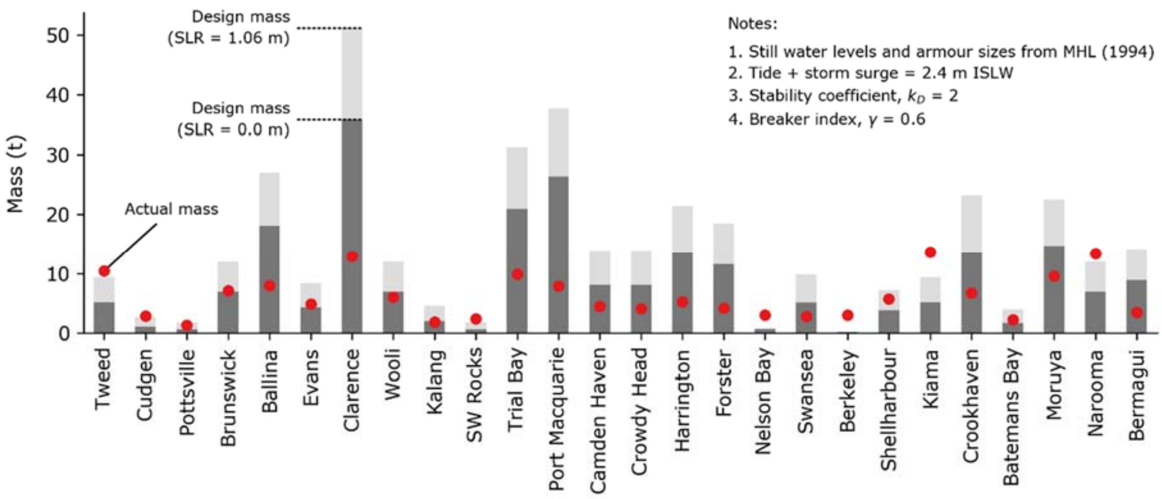


Figure 4.2. Required and actual rock armour masses for trunk sections of NSW breakwaters

MHL (1994) remarked that many of the breakwaters in NSW have performed well despite having undersized armour, but also noted that the structures may not yet have experienced a 100 year ARI design storm event.

4.1 Rock supply in NSW

One sea level rise adaptation strategy is to upgrade a structure's primary armour with larger rock, but this is not possible in all cases. Sourcing large quantities of rock with mass greater than 7 t in NSW can be difficult, as quarries target their operations to produce aggregates for concrete and road construction (Britton et al., 2017; Coghlan et al., 2013; DPWS, 1997; Russell et al., 2013). This rock mass limitation means that structures in deeper water will need to consider alternative adaptation strategies (Figure 4.3).

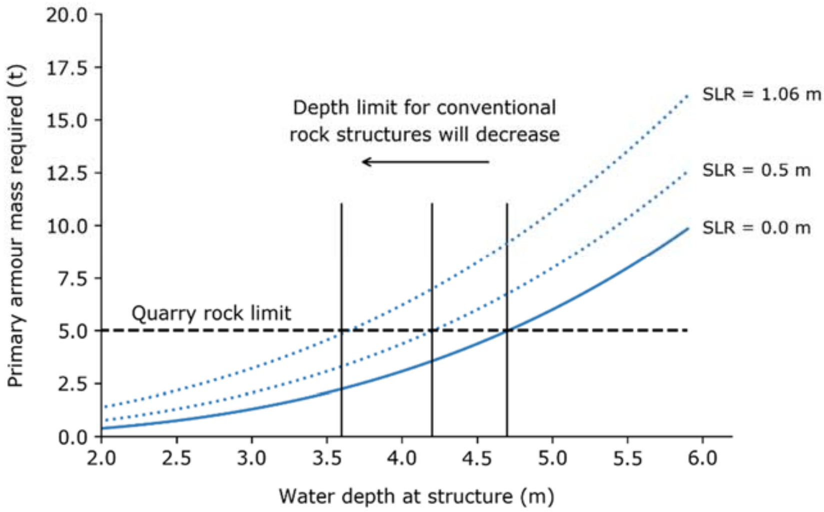


Figure 4.3. Impact of sea level rise on the maximum design depth for rock structures

4.2 The Hanbar concrete armour unit

The Hanbar was developed by NSW Public Works in the late 1970s and is the most commonly used concrete armour unit in NSW (Blacka et al., 2005). It only requires a single-piece mould, which makes it much simpler to cast than alternative units that were available at the time, such as the Dolos, Tetrapod, and Tribar (Figure 4.4).

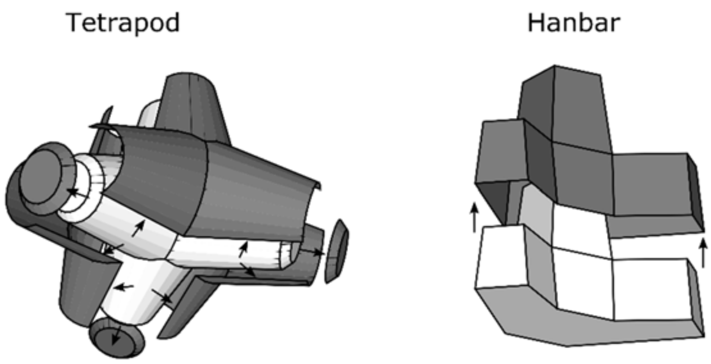


Figure 4.4. Concrete moulds required for Tetrapod and Hanbar concrete armour units

No formal design guidelines are available, but Hanbars are usually placed in two layers, similar to other slender concrete armour units (Table 3.1). Hanbars were placed in a single layer for the repair of the Forster breakwater (Figure 4.5) but this single layer design may have contributed to the structure sustaining significant damage in subsequent storm events (Figure 4.6).

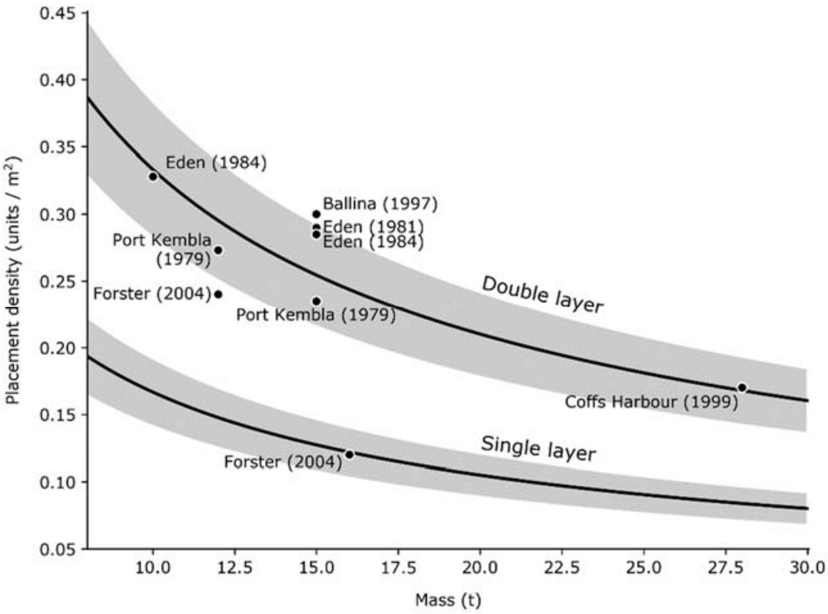


Figure 4.5. Hanbar placement densities. After Blacka et al. (2005)

Figure 4.6. Forster breakwater head before and after June 2016 storm. Source: Crown Lands

5 Practical considerations for seawall upgrades

5.1 Economics

Much of the literature on seawall and breakwater adaptation is focussed on the economics of upgrades. Headland et al. (2011) found that an adaptive management approach could be appropriate in some cases. This allows a structure to be upgraded incrementally in response to sea level rise triggers, rather than receive a single upgrade for the maximum anticipated sea level rise over the life of the structure.

Harrison and Cox (2015) examined five different scenarios for upgrading an existing rubble mound breakwater, with upgrades broken into one, two, or three stages over a 100 year planning period (Figure 5.1). The scenarios were said to either 'lead' or 'lag' sea level rise:

- Leading (a) scenarios had their crest level and armour size set for the end of the design period.
- Lagging (b) scenarios had their crest level set for the end of the design period, but the armour size set for the start of the design period.

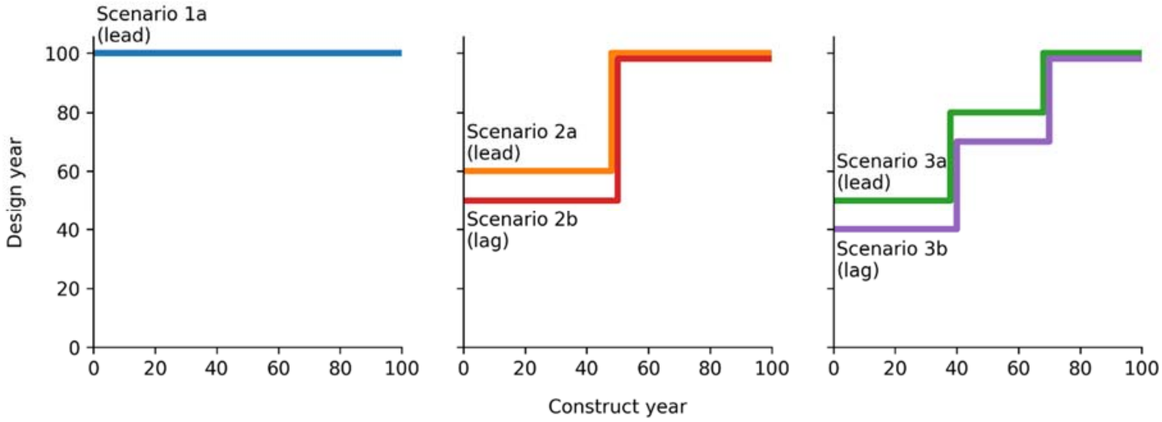


Figure 5.1. Scenarios for timing of seawall upgrades. Adapted from Harrison and Cox (2015)

The most economical upgrade scenario was identified for a range of different interest rates, depending on the value of the assets that the seawall/breakwater was protecting. Publicly-funded infrastructure planning in NSW is based on a discount rate of 7% (±3% for sensitivity tests). At a discount rate of 7%, Scenario 2a (a structure with a 50-year design life, to be upgraded and rebuilt 50 years in the future) was the most economical option, except where the value of the breakwater is less than the value of the asset it is protecting (Figure 5.2). For a discount rate of 4%, Scenario 1a (a structure with a 100-year design life) was the most economical option.

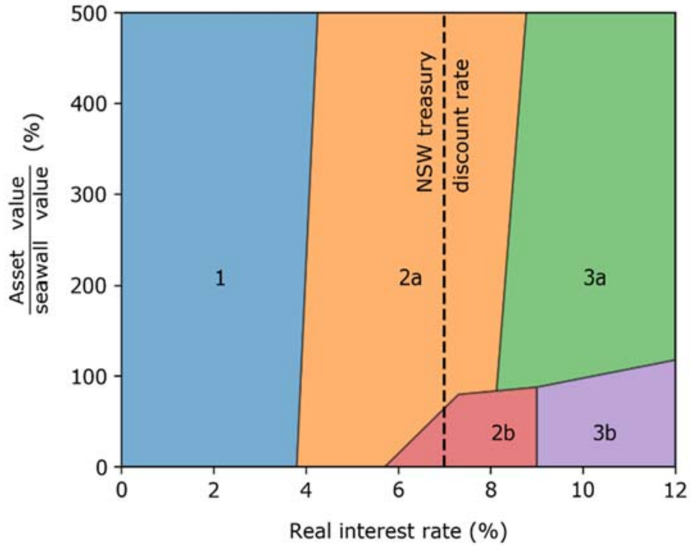


Figure 5.2. Lowest net present value for upgrades. Adapted from Harrison and Cox (2015)

5.2 Seawall upgrade strategies

Burcharth et al. (2014) identified a number of options for upgrading seawalls in response to sea level rise, but did not perform model testing to validate their findings. Design guidance for retrofitting existing seawalls with rock or concrete armour units is limited. Each site is different, with its own constraints and its own history of design, construction, and repair.

Most retrofitting strategies will include placing additional armour on an existing structure. This approach comes with two challenges:

1. The M/10 rule; and
2. Interfaces between different concrete armour types.

5.2.1 The M/10 rule for underlayers

Conventional design guidelines recommend that the primary armour mass should be approximately 10 times larger than the mass of the underlayer (USACE, 1984). It is impractical to follow the "M/10" rule when upgrading an existing structure, because the new upgraded armour would often be too large. A structure may only need a 1.5x increase in stability, so a 10x increase would be overdesign (Figure 5.3).

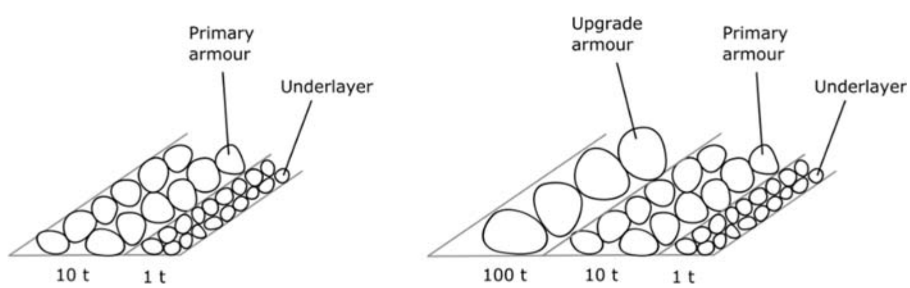


Figure 5.3. Sizing rules for armour and underlayer, Shore Protection Manual (USACE, 1984)

A realistic upgrade design is likely to break the M/10 rule for underlayer mass. CIRIA (2007, p.581) states that relatively large underlayer material has two advantages:

1. Larger rocks will make the surface of the underlayer less smooth, which increases friction between the armour layer and the underlayer.
2. Permeability is increased, which increases the stability of the armour layer.

This suggests that upgrading a structure with similar-sized armour will be effective, but the impacts of this are not well documented.

5.2.2 Armour of different types

Concrete armour units rely on interlocking for the their stability, and this interlocking is different for each type of unit. Junctions between different armour types create a plane of weakness (Foster, 1984). This includes concrete armour units of the same type, but of different sizes.

5.3 Physical model testing

During the present investigation, WRL completed several physical model tests to gain a better understanding of potential upgrade strategies (Appendix A).

6 Upgrade options for rock structures

Three upgrade strategies for rock-armoured structures were identified in the present study (Figure 6.1):

Option 1. Adding a berm to the seaward side of the structure;
Option 2. Adding larger rock armour; and
Option 3. Adding concrete armour.

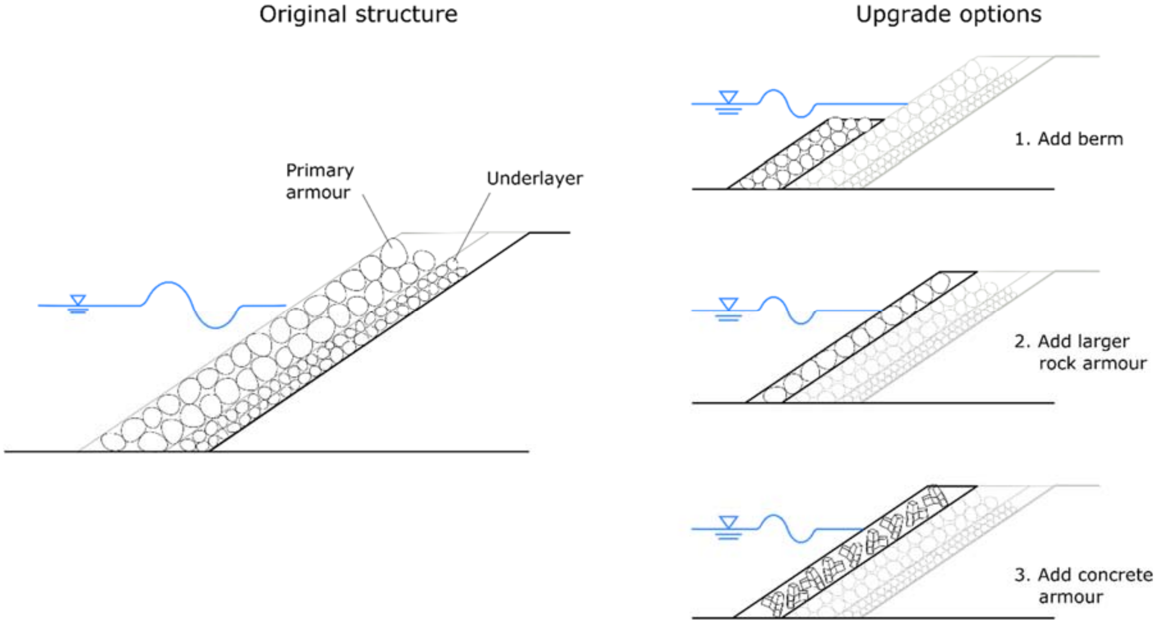


Figure 6.1. Upgrade options for rock structures

6.1 Berm structures

Some rock structures in NSW may require rock armour that is larger than can be supplied by local quarries. In these cases, an unconventional design may be adopted.

6.1.1 Seaward face berm

Some structures may be upgraded by placing a berm on the seaward side of the structure. This approach was adopted when upgrading an existing rock revetment at the Damai Lagoon Resort in Malaysia (Figure 6.2).

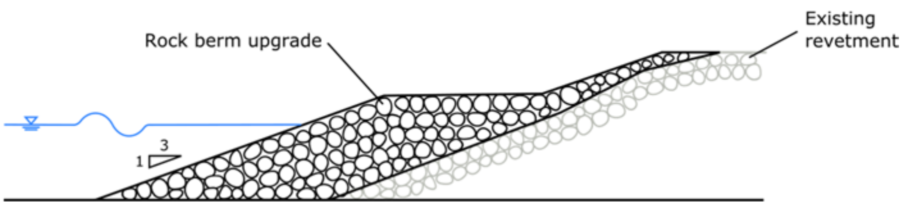


Figure 6.2. Cross section of Damai Lagoon Resort revetment upgrade (Walker et al., 1999)

6.1.2 Icelandic berm

The original design for the Shell Cove breakwater included Hanbars, because local quarries were unable to supply enough large rock (greater than 6 t) for a conventional rock-armoured structure. However, an Icelandic Berm breakwater design was adopted, because the armour mass requirements were lower than for a conventional breakwater design. In addition, the designer negotiated directly with a local quarry to produce armour specifically for the breakwater project in two ways (Britton et al., 2017):

1. Some blasts at the quarry produced 'rogue' rocks much larger than 6 t. Instead of breaking them down into smaller aggregate, the quarry stockpiled them for use as primary armour in the breakwater.
2. The operator agreed to selectively work one area of the quarry to specifically target production of larger rock.

Berm breakwaters have a larger footprint, and greater armour volume requirements than conventional structures. But compared with an alternative of concrete armour, the rock berm design offers advantages in public access and aesthetics. Some structures could potentially be improved by optimising the placement of existing armour, and relocating it to the most vulnerable part of the structure.

6.2 Larger primary rock

Harrison and Cox (2015) investigated the effectiveness of upgrading an existing rock structure with a single layer of larger rock armour. They found a single layer upgrade provided effective coastal protection, but only if the upgraded armour was placed with a tight packing density (or low porosity). This result was validated in the present study (Models R1, R2, and R3; Appendix A).

Rock armour is typically placed in two layers, but the testing results suggest a single layer rock armour upgrade is likely to be effective. Rock armour relies on mass rather than interlocking, so the interface between smaller and larger rock armour is unlikely to cause a point of weakness in the upgraded structure. Care must be taken to place upgrade armour so that an appropriate packing density is obtained. Model testing of a structure with a loosely packed armour (porosity=45%) compared with a structure with tightly packed armour (porosity=40%) resulted in a 25% decrease in armour stability (Figure 6.3).

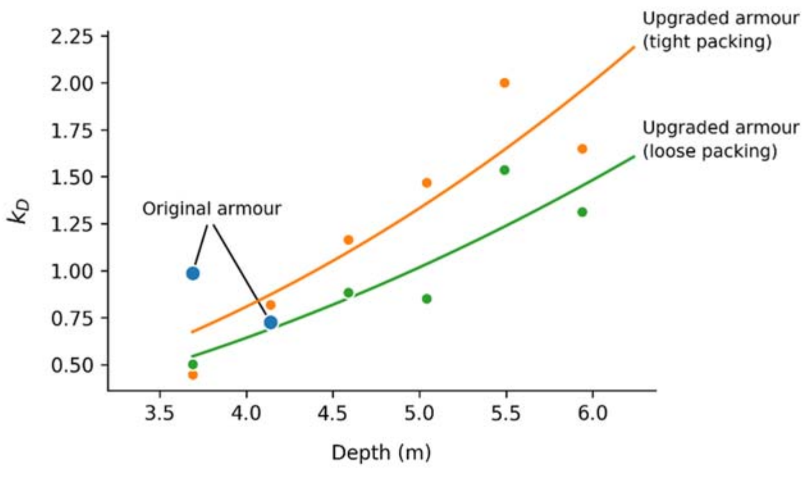


Figure 6.3. Results of rock armour testing with different porosity (see Appendix A)

6.3 Concrete armour units

In the case where larger rock is not available, a rock armour structure may be upgraded with concrete armour units. Physical model testing was undertaken to examine the effectiveness of a single layer Hanbar upgrade of an existing rock structure (Models C3, C4, C5, and C6; Appendix A). The models were built with different mass ratios between the existing rock armour and the additional Hanbars to see if there was an optimum ratio where stability was maximised.

Hanbars were found to be relatively more stable when their mass was similar to that of the existing primary rock armour but this effect is negated when the overall performance of a single layer Hanbar design is considered. The experiments gave stability coefficient values of approximately $k_D = 2$ at 5% damage for the single layer Hanbar upgrades. This value is much lower than expected, considering that the accepted stability coefficient for a two-layer Hanbar structure is $k_D = 7$ (Figure 6.4), and means that a single layer Hanbar structure is no more stable than a rock armour structure with the same unit mass (Figure 6.5)..

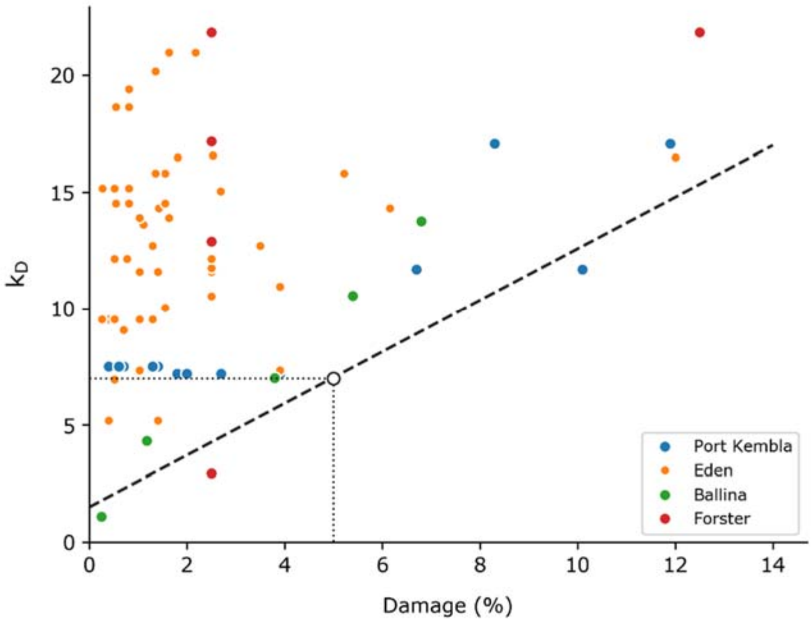


Figure 6.4. Results from previous Hanbar physical modelling tests (Blacka et al., 2005)

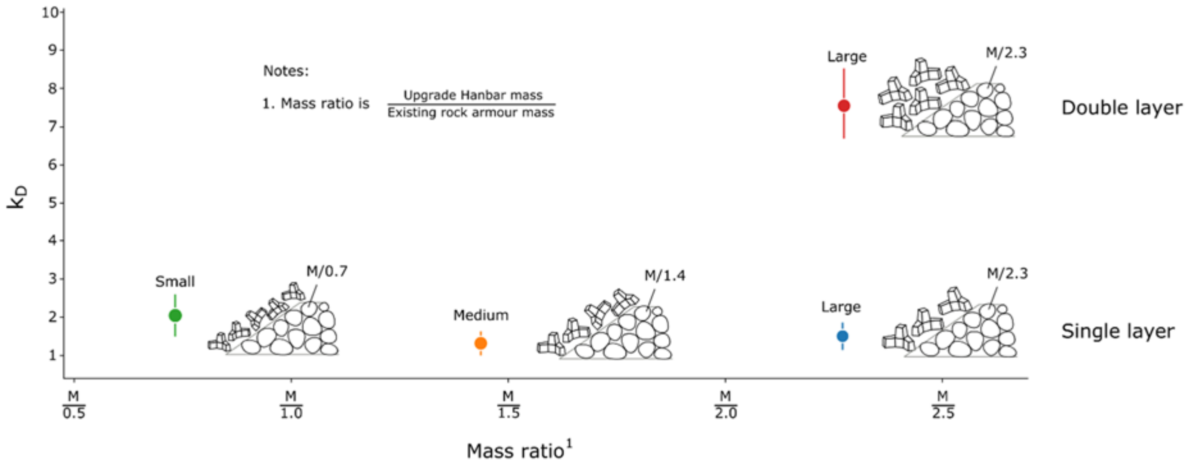


Figure 6.5. Stability of existing rock armour upgraded with Hanbars

The Hanbar may not be a suitable concrete armour unit for upgrading existing rock structures, for these reasons:

1. A single layer structure would require large units to compensate for its poor interlocking performance.
2. A double layer structure would require a large number of armour units, resulting in a large spatial footprint and high costs for armour placement and concrete supply.

Different concrete armour units may offer a more efficient alternative to Hanbars (Figure 6.6), but model testing is essential to verify the theoretical performance.

	Single layer Hanbar	Double layer Hanbar	Alternative armour
Unit mass	8.2 t	2.3 t	1.5 t
Stability coefficient	2	7	16
Units per 100 m²	20	90	45
Mass per 100 m²	164 t	206 t	70 t

Figure 6.6. Comparison of Hanbar and alternative CAU upgrades for an existing rock structure

7 Upgrade options for concrete-armoured structures

Three upgrade strategies for concrete-armoured structures were identified in the present study (Figure 7.1):

1. Adding larger concrete armour units;
2. Adding high-density concrete armour units; and
3. Removing the existing armour and replacing it with alternative armour.

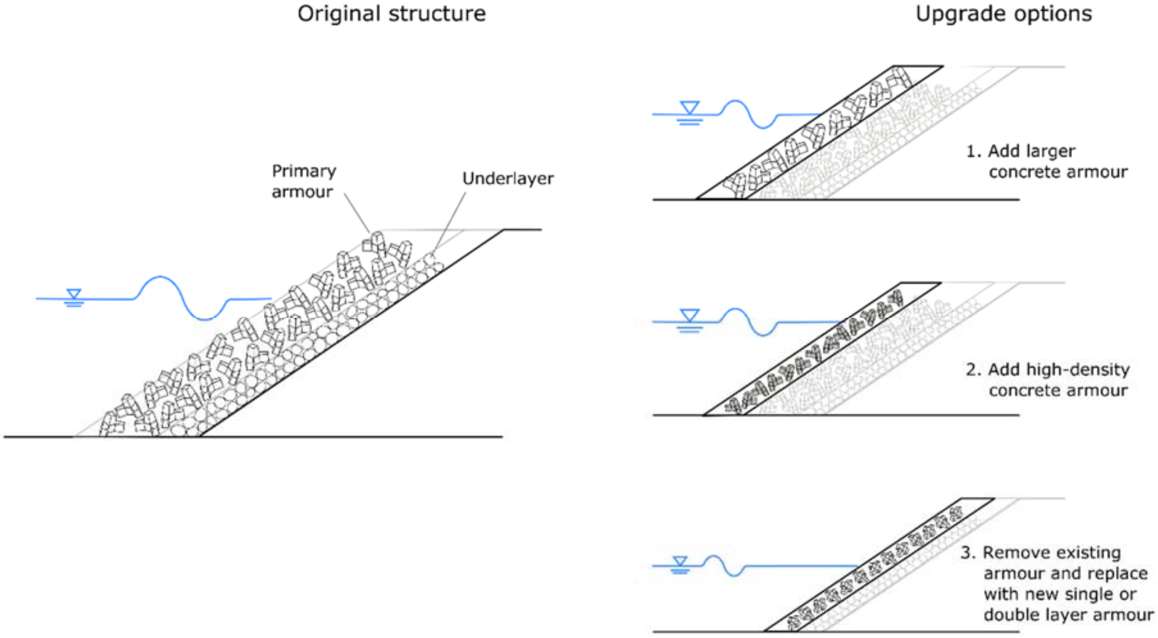


Figure 7.1. Upgrade options for concrete-armoured structures

7.1 Larger units

Li and Cox (2013) placed a single layer of larger Hanbars on an existing two-layer Hanbar structure. They found the single layer upgrade performed well for low levels of damage, but eventually failed rapidly. The performance was also sensitive to the placement density of the Hanbars (Figure 7.2).

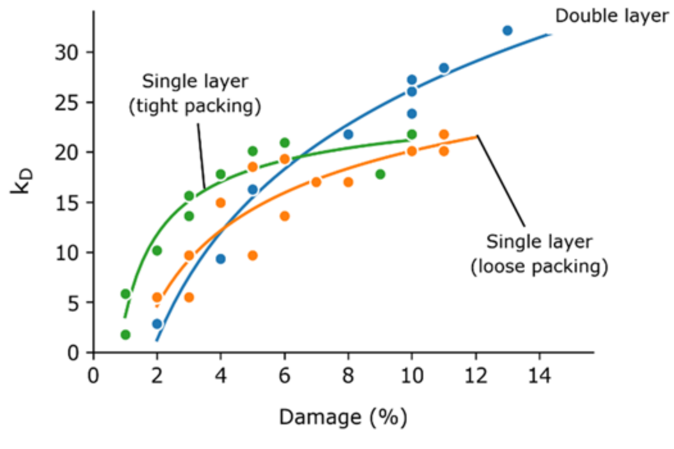


Figure 7.2. Stability of two-layer Hanbar structure upgraded with Hanbars (Li and Cox, 2013)

7.2 High-density concrete armour units

Howe and Cox (2017) tested conventional (SG=2.35) and high-density (SG=2.8) Hanbar units of different sizes, but equivalent stability as predicted by Hudson's equation (Models C1, and C2; Appendix A). Physical model testing confirmed that the performance of the different units was similar (Figure 7.3).

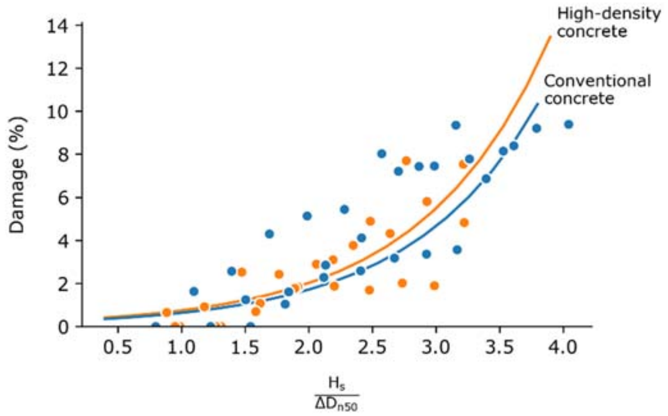


Figure 7.3. Performance of conventional and high-density Hanbars

Recent advances in materials technology have allowed the development of high-density concrete. By replacing gravel aggregate with steel furnace slag (a by-product from steelmaking process), the specific gravity of concrete can be increased to 2.7, or higher (Khan et al., 2016).

The original concrete armour moulds can be reused to produce high-density armour. This high-density armour will have increased stability, while retaining identical dimensions to the existing armour, to ensure good interlocking. In this way a single layer of high-density concrete Hanbar would theoretically be effective when placed on an existing two-layer structure; it would simply behave as a three-layer structure (Figure 7.4). Physical modelling is required to confirm the stability of this specific configuration (it was not tested in the present study).

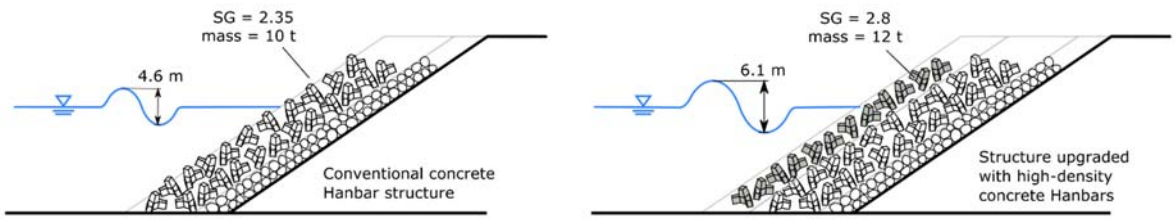


Figure 7.4. Upgrading an existing structure with one layer of high-density concrete Hanbars

7.3 Remove armour and rebuild

In some cases it might be appropriate to remove the existing concrete armour, repair the underlayer if necessary, and place new concrete armour units on the structure. Simply placing larger armour of the same type may not be appropriate, depending on the size of the underlayer rock. If the ratio between the primary armour mass and the underlayer mass is too large (USACE (1984) recommends a ratio of 10:1), the underlayer rock can wash out between the voids in the primary armour. In these cases it may be necessary to choose a new armour unit with a larger stability coefficient, to reduce the overall dimensions of the unit. Hanbar armour could be removed and upgraded with alternative concrete armour units (see Table 3.1 for options).

Careful consideration must be given when selecting an armour unit. Single layer pattern-placed structures are generally less robust than double layer designs, especially in depth-limited conditions along the NSW coast where design wave conditions may be encountered frequently (often more than once per year). Single layer pattern placed structures can also be difficult to construct in even mild wave conditions.

The main challenge with armour replacement is the construction process. The structure will be vulnerable to storms once the original armour is removed. Costs associated with removal and disposal of the original armour must also be considered.

8 Conclusion

Sea level rise will cause the design wave height for many coastal structures in NSW to increase. WRL completed an investigation into the options for adapting existing breakwaters and seawalls in NSW to accommodate these new design conditions.

Upgrading rock armoured structures is challenging because the size of quarry rock in NSW is usually limited to approximately 7 t, but this can be overcome in some cases by adopting unconventional designs and negotiating with local quarries to maximise their production of large rock.

Physical model testing of rock and concrete armour yielded the following observations:

1. A rock armour structure can be upgraded with a single layer of larger rock armour, provided a high packing density can be maintained.
2. Hanbar armour units should not be placed in a single layer, except for on top of existing Hanbars of the same size.
3. High-density concrete armour can be placed on existing armour with the same dimensions, to provide enhanced stability while retaining good interlocking.

For some concrete armour structures it may be desirable to remove the existing armour and replace it with new armour of enhanced stability to reduce the concrete requirements and spatial footprint of the structure. Construction planning is essential to overcome any difficulties in placement of pattern placed CAU options.

Physical modelling should be used to ensure satisfactory performance of a structure during detailed design.

9 Acknowledgements

This project was undertaken on behalf of the Australian Climate Change Adaptation Research Network for Settlements and Infrastructure (ACCARNSI) with funding from the NSW Office of Environment and Heritage (OEH) as part of the Coastal Processes and Responses Node of the Climate Adaptation Research Hub. Support for the high-density concrete Hanbar testing was provided by Fibrecon and Australian Steel Mill Services.

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Appendix A. WRL experimental work

A.1 Rock armour experiments

The rock armour testing was undertaken in the WRL 3 m flume, using Froudian similitude with a length scale of 1:45. The flume was divided into three 1 m wide sections. The model was located in the left section, and three capacitance wave probes were placed in the right section to measure wave heights at the structure. Three additional wave probes were located close to the wave paddle to measure wave heights in deep water (Figure A.1).

Figure A.1. WRL 3 m wide wave flume

Three different rock armour configurations were tested (Figure A.2):

R1. Original structure, with 2.4 t primary rock armour (Figure B.1);
R2. Upgraded structure, with additional 6.2 t rock armour with 40% porosity (Figure B.2); and
R3. Upgraded structure, with additional 6.2 t rock armour with 45% porosity(Figure B.3).

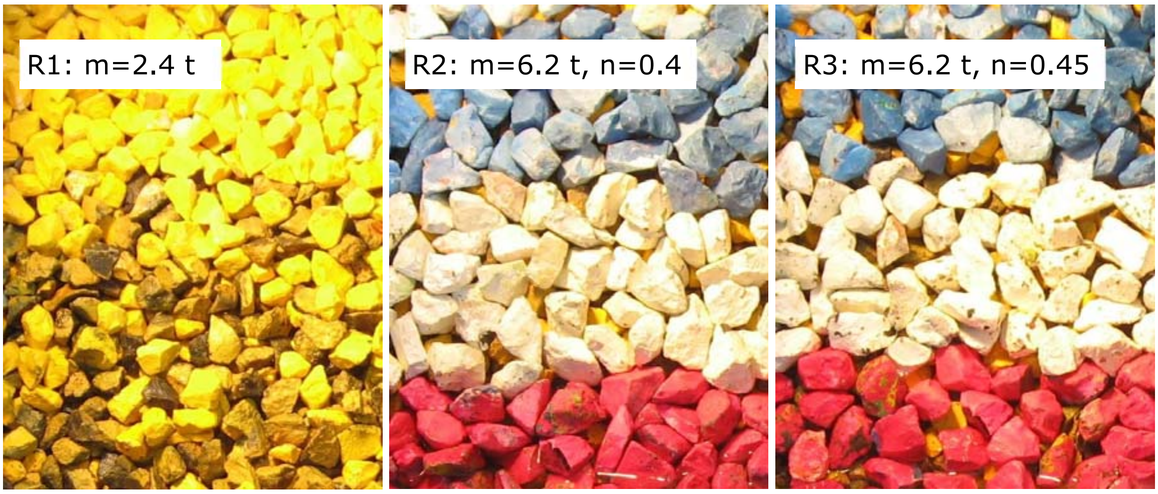

Figure A.2. Armour for rock models

Each model was subjected to tests of 1000 waves. The initial wave height was small enough to cause no damage to the structure. After each test the model was photographed to check for damage (defined as a rock moving a distance of more than one diameter during the test).

Damage was calculated based on the number of units displaced, divided by the total number of units in the region extending one wave height above and below the still water level (SWL). If only 0 or 1 rocks were displaced, the wave height was increased for the next test. This process was repeated until either:

- the structure sustained more than 20% damage; or

- depth-limited conditions were reached.

After the structure failed or was found to be stable under depth-limited conditions, the model was rebuilt, the water level was raised, and the wave height was reset. Model R1 was only tested for the lowest two water levels (the model would have failed under non-breaking waves for the higher water levels).

It was difficult to compare the relative performance of the single layer upgrade armour compared with the exiting double layer primary armour because of the difference in size of the design waves, but it is clear that the tightly packed armour performed better than the loosely packed armour (Figure A.3).

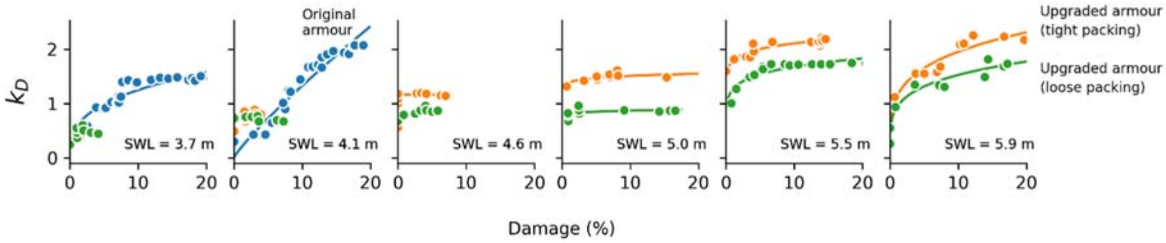

Figure A.3. Results from rock testing

A.2 Concrete armour experiments

The concrete armour testing was undertaken in the WRL 1.2 m flume, using Froudian similitude with a length scale of 1:33. An array of capacitance wave probes was used to measure wave heights in deep water. After the testing was completed the model was removed and a second array of wave probes was used to measure wave heights in the same location as the structure (Figure A.4).

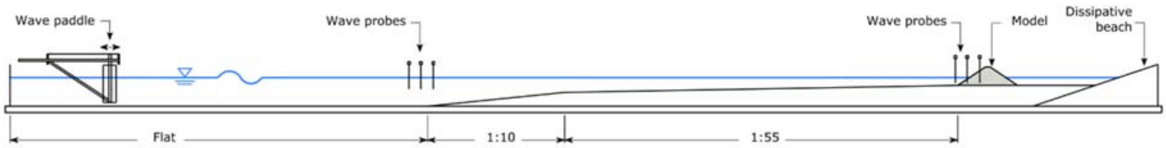

Figure A.4. WRL 1.2 m wide wave flume

Two different test programs were completed to investigate high-density concrete, and single layer armour upgrades.

A.1.1 High-density concrete armour units

Two different concrete armour configurations were tested (Figure A.5):

C1. Two layers of 8.3 t conventional concrete Hanbar armour (Figure C.1); and
C2. Two layers of 4.3 t high-density concrete Hanbar armour (Figure C.2).

The high-density concrete Hanbars were designed to have the same theoretical stability as the conventional Hanbars, based on Hudson's equation (Figure A.6).

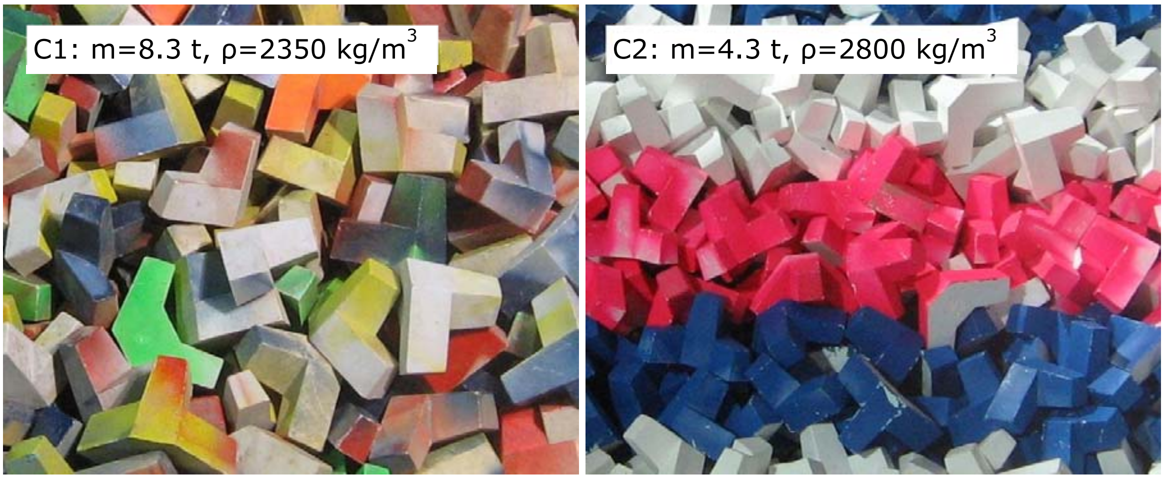

Figure A.5. Armour for high-density concrete testing

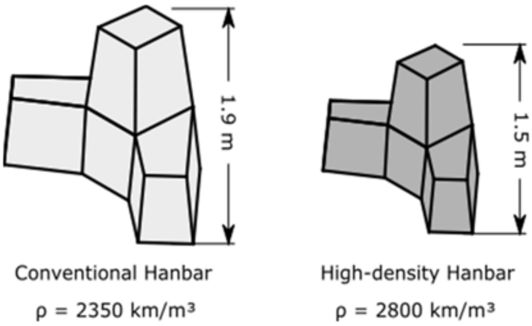

Figure A.6. Hanbar units used for models C1 and C2

Wave heights were progressively increased in the same way as with the rock armour experiments, and the tests were repeated for three different wave periods (Figure A.7). The conventional and high-density concrete Hanbars were found to have roughly equivalent performance for the conditions tested.

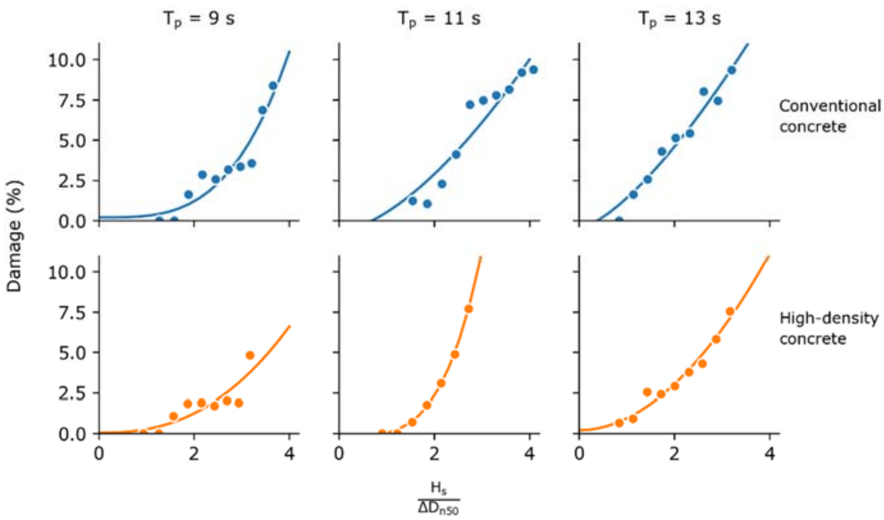

Figure A.7. Results from high-density concrete Hanbar tests

A.1.2 Single layer concrete armour units

Four different concrete armour configurations were tested (Figure A.8, Figure A.9):

C3. Rock armour upgraded with a single layer of small Hanbars (Figure C.3);
C4. Rock armour upgraded with a single layer of medium Hanbars (Figure C.4);
C5. Rock armour upgraded with a single layer of large Hanbars (Figure C.5); and
C6. Rock armour upgraded with a double layer of large Hanbars (Figure C.6).

Figure A.8. Underlayer rock ratios for single layer Hanbar models (after failure)

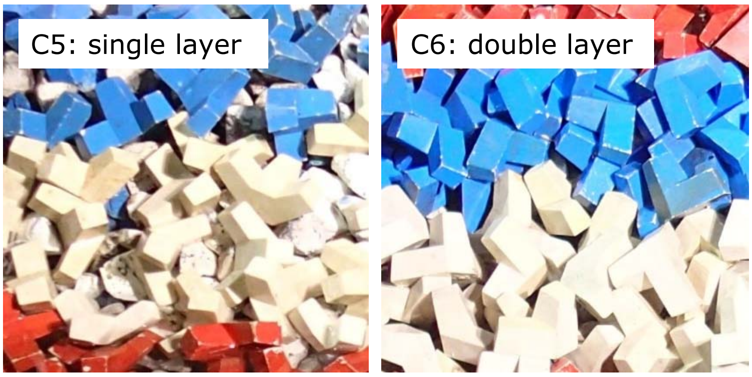

Figure A.9. Comparison of single and double layer Hanbar armour

The same Hanbar units were used for each model to keep the scale of the waves consistent. The small/medium/large descriptions refer to the relative mass of the Hanbars compared with the existing rock armour beneath (Table A.1).

Table A.1. Mass ratios for single and double layer Hanbar tests

	Description	Underlayer mass (t)	Mass ratio $\left(\frac{M_{\text{armour}}}{M_{\text{underlayer}}}\right)$:
C3	Small	6.0	0.7
C4	Medium	3.0	1.4
C5	Large	1.9	2.3
C6	Double	1.9	2.3

The small Hanbars performed slightly better than the medium and large Hanbars, but the overall performance of the single layer armour was very poor. The stability coefficient at the 5% damage level was approximately 2 for the single layer armour (compared with $k_D = 7$ for conventional double-layer hanbar armour; Figure A.10).

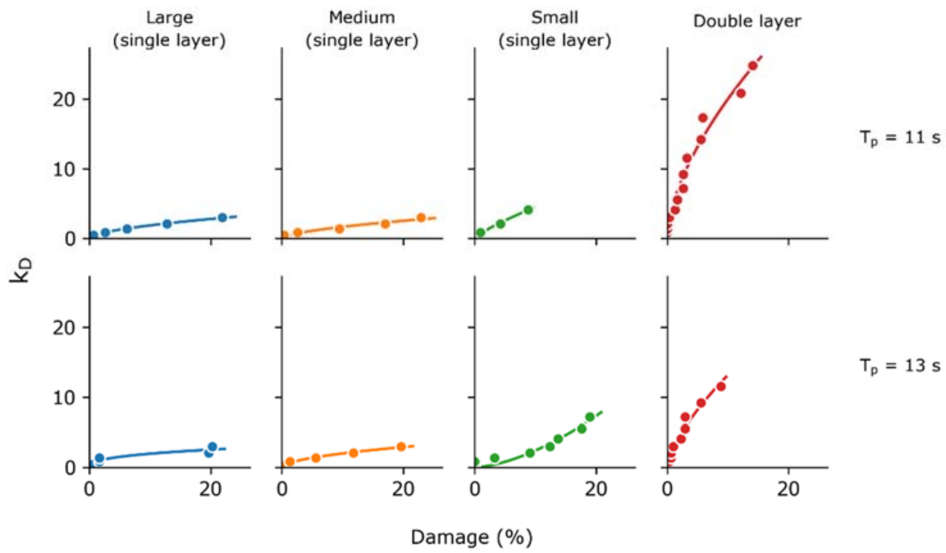

Figure A.10. Results of single and double layer Hanbars with different rock underlayers

Appendix B. Model design for rock armour

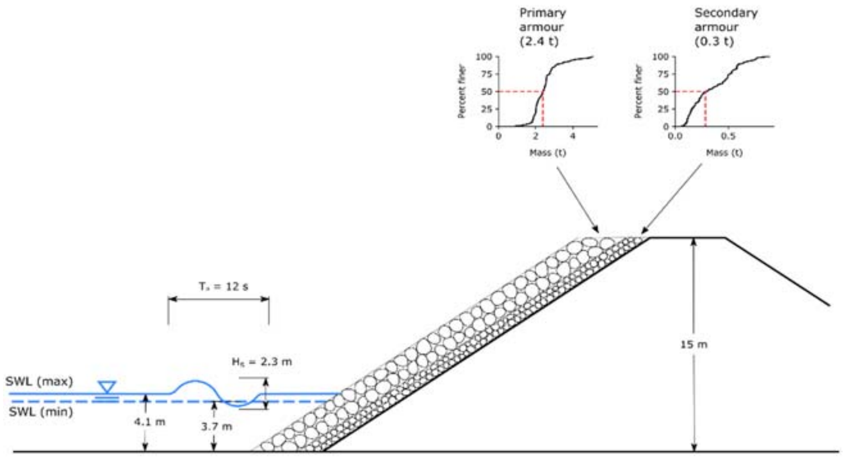

Figure B.1. Model R1 (scale=1:45)

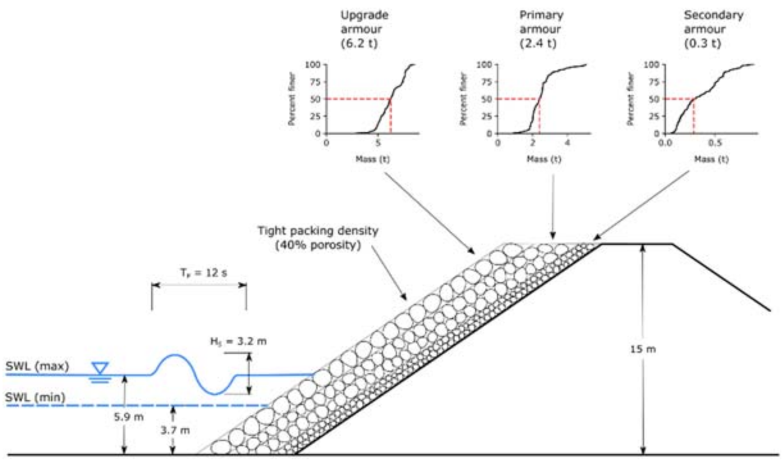

Figure B.2. Model R2 (scale=1:45)

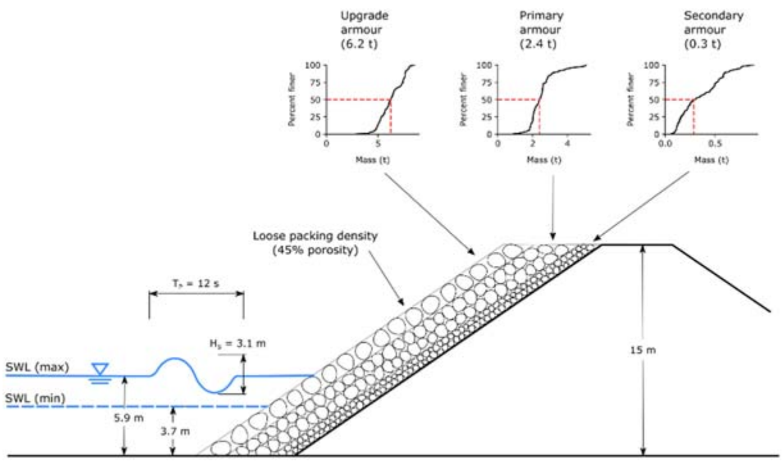

Figure B.3. Model R3 (scale=1:45)

Appendix C. Model design for concrete armour

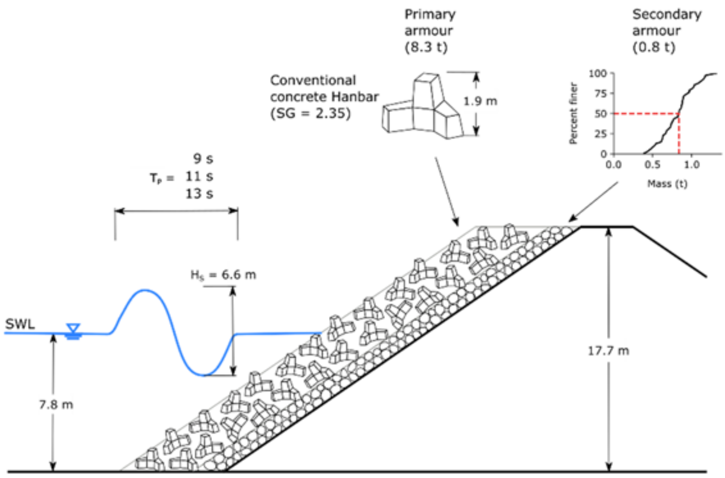

Figure C.1. Model C1 (scale=1:33)

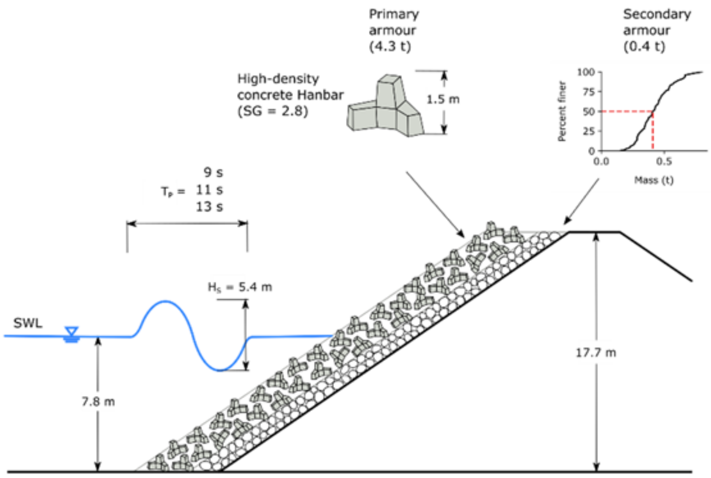

Figure C.2. Model C2 (scale=1:33)

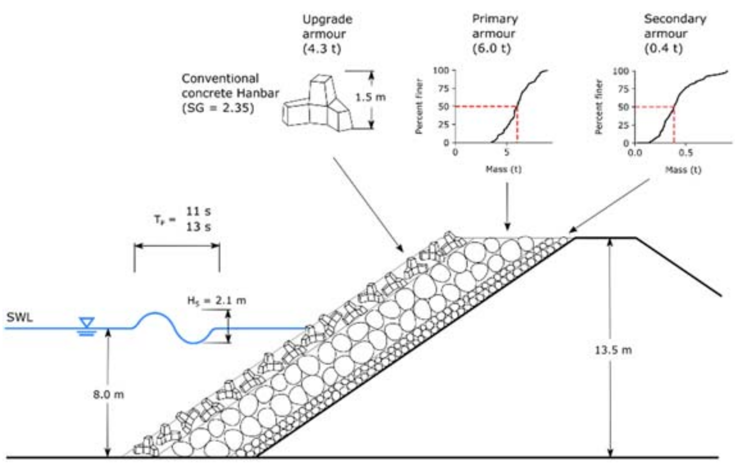

Figure C.3. Model C3 (scale=1:33)

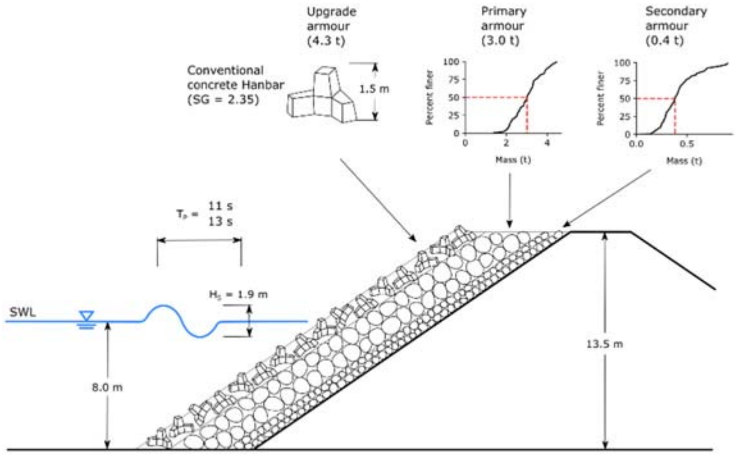

Figure C.4. Model C4 (scale=1:33)

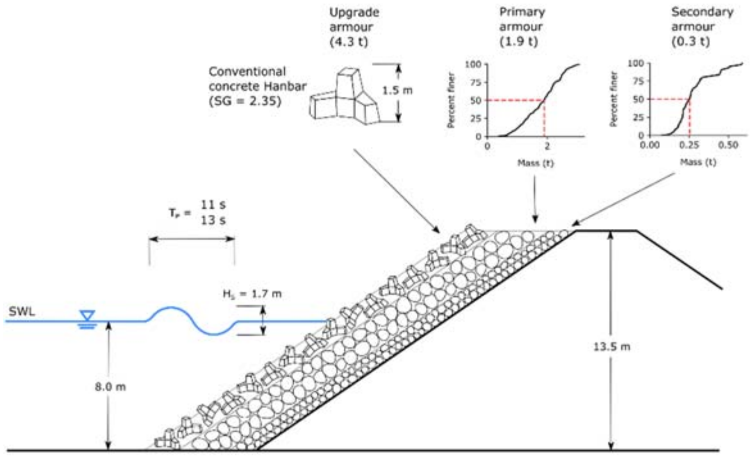

Figure C.5. Model C5 (scale=1:33)

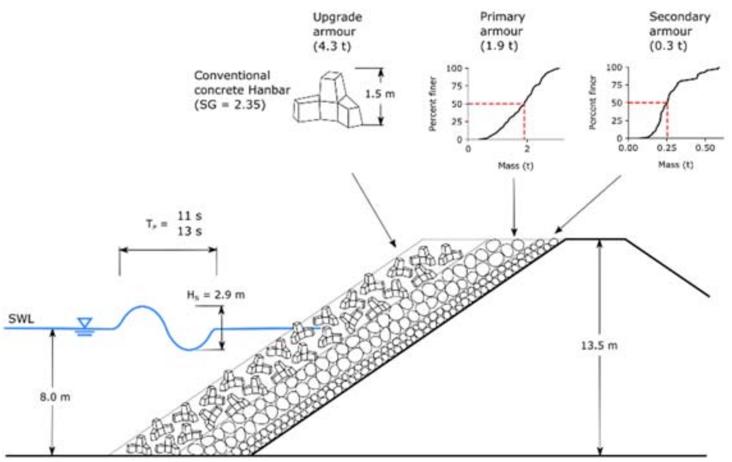

Figure C.6. Model C6 (scale=1:33)

Appendix D. Test conditions

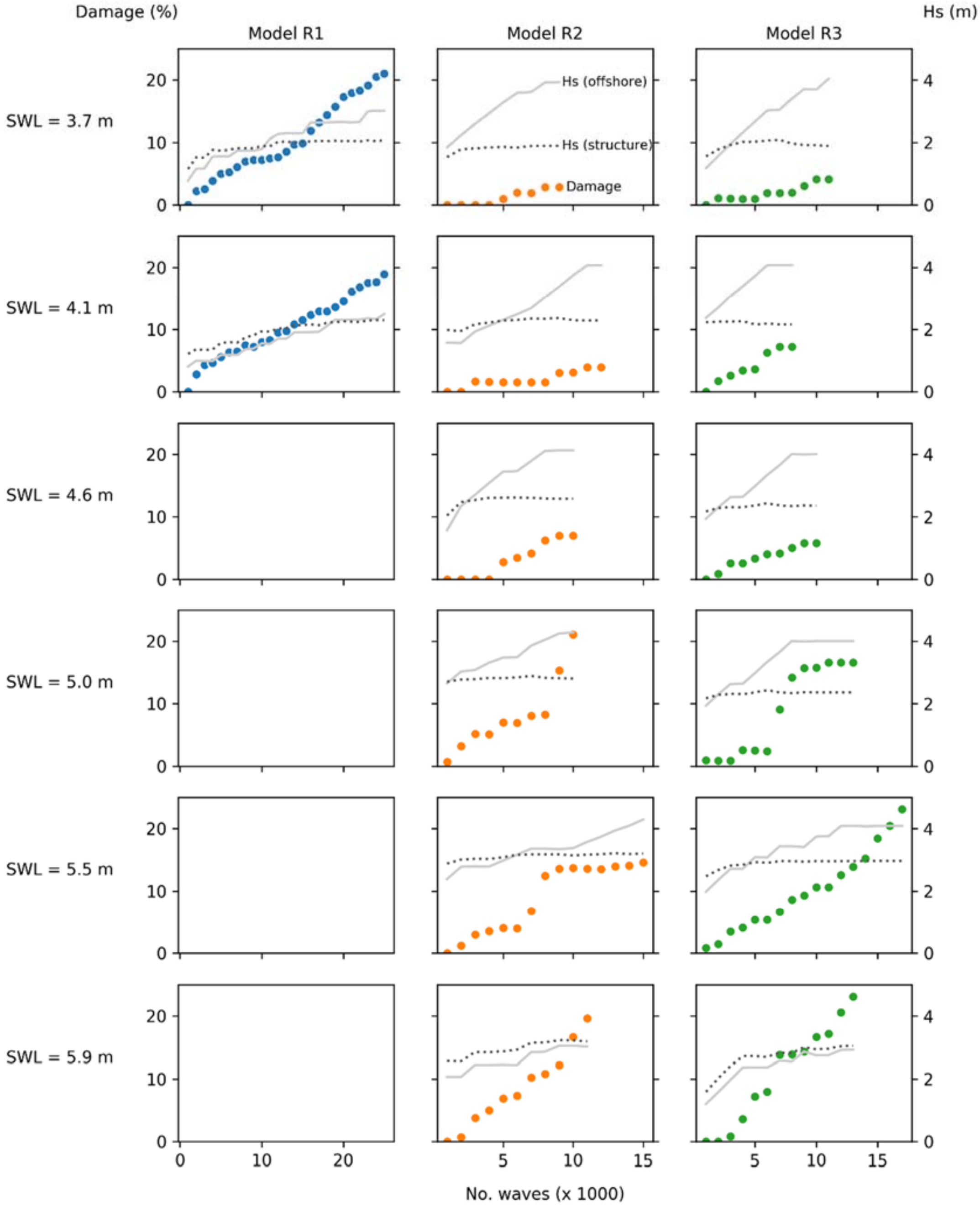

Figure D.1. Measured wave heights and damage for tests on models R1, R2, and R3

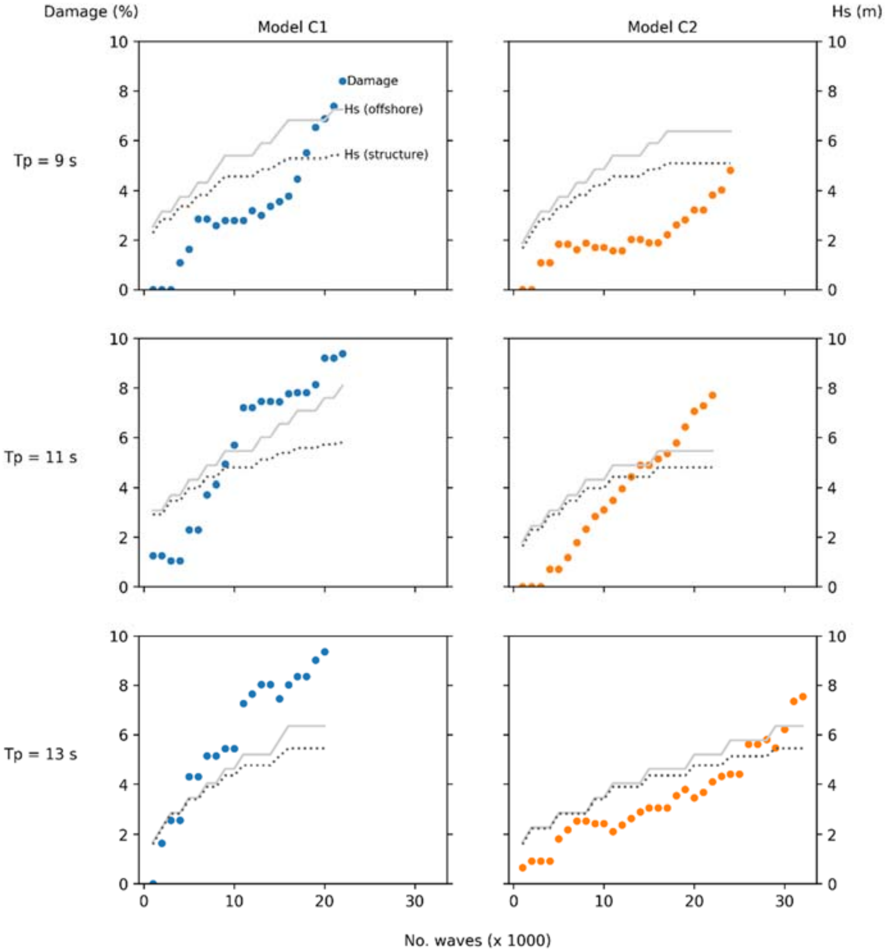

Figure D.2. Measured wave heights and damage for tests on models C1 and C2

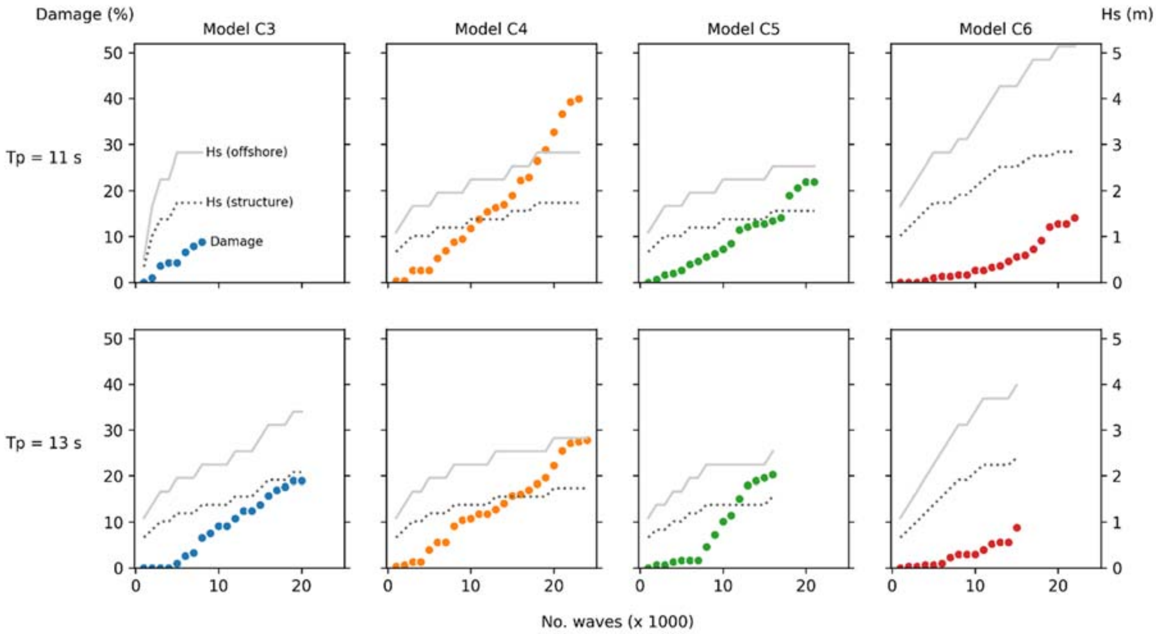

Figure D.3. Measured wave heights and damage for tests on models C3, C4, C5, and C6

Appendix E. Test program

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
1	R1	2.4	2650	12	3.7	0.8	1.1	0	0
2	R1	2.4	2650	12	3.7	1.2	1.5	7	7
3	R1	2.4	2650	12	3.7	1.2	1.5	1	8
4	R1	2.4	2650	12	3.7	1.5	1.8	6	14
5	R1	2.4	2650	12	3.7	1.6	1.7	4	18
6	R1	2.4	2650	12	3.7	1.6	1.8	1	19
7	R1	2.4	2650	12	3.7	1.7	1.8	4	23
8	R1	2.4	2650	12	3.7	1.7	1.8	3	26
9	R1	2.4	2650	12	3.7	1.7	1.8	1	27
10	R1	2.4	2650	12	3.7	1.8	1.9	1	28
11	R1	2.4	2650	12	3.7	2.1	1.9	1	29
12	R1	2.4	2650	12	3.7	2.3	2.0	3	32
13	R1	2.4	2650	12	3.7	2.3	2.0	4	36
14	R1	2.4	2650	12	3.7	2.3	2.0	4	40
15	R1	2.4	2650	12	3.7	2.3	2.0	1	41
16	R1	2.4	2650	12	3.7	2.6	2.0	9	50
17	R1	2.4	2650	12	3.7	2.6	2.0	6	56
18	R1	2.4	2650	12	3.7	2.6	2.0	5	61
19	R1	2.4	2650	12	3.7	2.6	2.1	6	67
20	R1	2.4	2650	12	3.7	2.7	2.0	6	73
21	R1	2.4	2650	12	3.7	2.6	2.0	3	76
22	R1	2.4	2650	12	3.7	2.7	2.0	1	77
23	R1	2.4	2650	12	3.7	3.0	2.1	5	82
24	R1	2.4	2650	12	3.7	3.0	2.0	5	87
25	R1	2.4	2650	12	3.7	3.0	2.1	3	90
26	R1	2.4	2650	12	4.1	0.8	1.2	0	0
27	R1	2.4	2650	12	4.1	1.0	1.4	8	8
28	R1	2.4	2650	12	4.1	1.0	1.3	4	12
29	R1	2.4	2650	12	4.1	1.0	1.4	1	13
30	R1	2.4	2650	12	4.1	1.2	1.6	5	18
31	R1	2.4	2650	12	4.1	1.2	1.6	3	21
32	R1	2.4	2650	12	4.1	1.2	1.6	0	21
33	R1	2.4	2650	12	4.1	1.4	1.7	6	27
34	R1	2.4	2650	12	4.1	1.4	1.8	0	27
35	R1	2.4	2650	12	4.1	1.5	1.9	5	32
36	R1	2.4	2650	12	4.1	1.5	1.9	1	33
37	R1	2.4	2650	12	4.1	1.7	2.0	7	40
38	R1	2.4	2650	12	4.1	1.7	2.0	1	41
39	R1	2.4	2650	12	4.1	1.9	2.1	7	48
40	R1	2.4	2650	12	4.1	1.9	2.1	3	51
41	R1	2.4	2650	12	4.1	1.9	2.2	4	55
42	R1	2.4	2650	12	4.1	1.9	2.1	2	57
43	R1	2.4	2650	12	4.1	2.1	2.2	2	59
44	R1	2.4	2650	12	4.1	2.3	2.2	4	63
45	R1	2.4	2650	12	4.1	2.3	2.3	5	68
46	R1	2.4	2650	12	4.1	2.3	2.2	7	75
47	R1	2.4	2650	12	4.1	2.3	2.2	3	78
48	R1	2.4	2650	12	4.1	2.4	2.3	5	83
49	R1	2.4	2650	12	4.1	2.3	2.3	1	84
50	R1	2.4	2650	12	4.1	2.5	2.3	6	90

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
51	R2	6.1	2650	12	3.7	1.8	1.5	0	0
52	R2	6.1	2650	12	3.7	2.2	1.8		0
53	R2	6.1	2650	12	3.7	2.6	1.8		0
54	R2	6.1	2650	12	3.7	2.9	1.8	0	0
55	R2	6.1	2650	12	3.7	3.3	1.9	1	1
56	R2	6.1	2650	12	3.7	3.6	1.8	1	2
57	R2	6.1	2650	12	3.7	3.6	1.9	0	2
58	R2	6.1	2650	12	3.7	3.9	1.9	1	3
59	R2	6.1	2650	12	3.7	3.9	1.9	0	3
60	R2	6.1	2650	12	4.1	1.6	2.0	0	0
61	R2	6.1	2650	12	4.1	1.6	2.0	0	0
62	R2	6.1	2650	12	4.1	1.9	2.2	2	2
63	R2	6.1	2650	12	4.1	2.1	2.2	0	2
64	R2	6.1	2650	12	4.1	2.3	2.3	0	2
65	R2	6.1	2650	12	4.1	2.5	2.3	0	2
66	R2	6.1	2650	12	4.1	2.7	2.3	0	2
67	R2	6.1	2650	12	4.1	3.0	2.3	0	2
68	R2	6.1	2650	12	4.1	3.4	2.4	2	4
69	R2	6.1	2650	12	4.1	3.7	2.3	0	4
70	R2	6.1	2650	12	4.1	4.1	2.3	1	5
71	R2	6.1	2650	12	4.1	4.1	2.3	0	5
72	R2	6.1	2650	12	4.6	1.6	2.0	0	0
73	R2	6.1	2650	12	4.6	2.3	2.5	0	0
74	R2	6.1	2650	12	4.6	2.7	2.5	0	0
75	R2	6.1	2650	12	4.6	3.1	2.6	0	0
76	R2	6.1	2650	12	4.6	3.4	2.6	4	4
77	R2	6.1	2650	12	4.6	3.5	2.6	1	5
78	R2	6.1	2650	12	4.6	3.8	2.6	1	6
79	R2	6.1	2650	12	4.6	4.1	2.6	3	9
80	R2	6.1	2650	12	4.6	4.1	2.6	1	10
81	R2	6.1	2650	12	4.6	4.1	2.6	0	10
82	R2	6.1	2650	12	5.0	2.7	2.7	1	1
83	R2	6.1	2650	12	5.0	3.0	2.8	4	5
84	R2	6.1	2650	12	5.0	3.1	2.8	3	8
85	R2	6.1	2650	12	5.0	3.3	2.8	0	8
86	R2	6.1	2650	12	5.0	3.5	2.8	3	11
87	R2	6.1	2650	12	5.0	3.5	2.8	0	11
88	R2	6.1	2650	12	5.0	3.9	2.9	2	13
89	R2	6.1	2650	12	5.0	4.0	2.8	0	13
90	R2	6.1	2650	12	5.0	4.2	2.8	11	24
91	R2	6.1	2650	12	5.0	4.3	2.8	9	33
92	R2	6.1	2650	12	5.5	2.4	2.9	0	0
93	R2	6.1	2650	12	5.5	2.8	3.0	2	2
94	R2	6.1	2650	12	5.5	2.8	3.0	3	5
95	R2	6.1	2650	12	5.5	2.8	3.0	1	6
96	R2	6.1	2650	12	5.5	3.0	3.1	1	7
97	R2	6.1	2650	12	5.5	3.2	3.2	0	7
98	R2	6.1	2650	12	5.5	3.4	3.2	5	12
99	R2	6.1	2650	12	5.5	3.4	3.2	10	22
100	R2	6.1	2650	12	5.5	3.3	3.2	2	24
101	R2	6.1	2650	12	5.5	3.4	3.1	0	24
102	R2	6.1	2650	12	5.5	3.6	3.2	0	24
103	R2	6.1	2650	12	5.5	3.7	3.2	0	24
104	R2	6.1	2650	12	5.5	3.9	3.2	1	25
105	R2	6.1	2650	12	5.5	4.1	3.2	0	25

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
106	R2	6.1	2650	12	5.5	4.3	3.2	1	26
107	R2	6.1	2650	12	5.9	2.1	2.6	0	0
108	R2	6.1	2650	12	5.9	2.1	2.6	1	1
109	R2	6.1	2650	12	5.9	2.4	2.9	5	6
110	R2	6.1	2650	12	5.9	2.4	2.9	2	8
111	R2	6.1	2650	12	5.9	2.5	2.9	3	11
112	R2	6.1	2650	12	5.9	2.4	2.9	1	12
113	R2	6.1	2650	12	5.9	2.8	3.1	6	18
114	R2	6.1	2650	12	5.9	2.9	3.2	1	19
115	R2	6.1	2650	12	5.9	3.1	3.2	3	22
116	R2	6.1	2650	12	5.9	3.1	3.2	8	30
117	R2	6.1	2650	12	5.9	3.0	3.2	5	35

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
118	R3	6.1	2650	12	3.7	1.2	1.6	0	0
119	R3	6.1	2650	12	3.7	1.6	1.8	1	1
120	R3	6.1	2650	12	3.7	1.9	1.9	0	1
121	R3	6.1	2650	12	3.7	2.3	2.0	0	1
122	R3	6.1	2650	12	3.7	2.7	2.0	0	1
123	R3	6.1	2650	12	3.7	3.0	2.1	1	2
124	R3	6.1	2650	12	3.7	3.0	2.1	0	2
125	R3	6.1	2650	12	3.7	3.4	2.0	0	2
126	R3	6.1	2650	12	3.7	3.7	1.9	1	3
127	R3	6.1	2650	12	3.7	3.7	1.9	1	4
128	R3	6.1	2650	12	3.7	4.0	1.9	0	4
129	R3	6.1	2650	12	4.1	2.4	2.2	0	0
130	R3	6.1	2650	12	4.1	2.7	2.2	2	2
131	R3	6.1	2650	12	4.1	3.1	2.2	1	3
132	R3	6.1	2650	12	4.1	3.4	2.3	1	4
133	R3	6.1	2650	12	4.1	3.7	2.2	0	4
134	R3	6.1	2650	12	4.1	4.1	2.2	3	7
135	R3	6.1	2650	12	4.1	4.1	2.2	1	8
136	R3	6.1	2650	12	4.1	4.1	2.2	0	8
137	R3	6.1	2650	12	4.6	1.9	2.2	0	0
138	R3	6.1	2650	12	4.6	2.3	2.3	1	1
139	R3	6.1	2650	12	4.6	2.6	2.3	2	3
140	R3	6.1	2650	12	4.6	2.6	2.3	0	3
141	R3	6.1	2650	12	4.6	3.0	2.4	1	4
142	R3	6.1	2650	12	4.6	3.3	2.4	1	5
143	R3	6.1	2650	12	4.6	3.6	2.4	0	5
144	R3	6.1	2650	12	4.6	4.0	2.3	1	6
145	R3	6.1	2650	12	4.6	4.0	2.4	1	7
146	R3	6.1	2650	12	4.6	4.0	2.4	0	7
147	R3	6.1	2650	12	5.0	1.9	2.2	1	1
148	R3	6.1	2650	12	5.0	2.3	2.3	0	1
149	R3	6.1	2650	12	5.0	2.6	2.3	0	1
150	R3	6.1	2650	12	5.0	2.6	2.3	2	3
151	R3	6.1	2650	12	5.0	3.0	2.4	0	3
152	R3	6.1	2650	12	5.0	3.3	2.4	0	3
153	R3	6.1	2650	12	5.0	3.6	2.4	8	11
154	R3	6.1	2650	12	5.0	4.0	2.3	6	17
155	R3	6.1	2650	12	5.0	4.0	2.4	2	19
156	R3	6.1	2650	12	5.0	4.0	2.4	0	19
157	R3	6.1	2650	12	5.0	4.0	2.4	1	20
158	R3	6.1	2650	12	5.0	4.0	2.4	0	20
159	R3	6.1	2650	12	5.0	4.0	2.4	0	20
160	R3	6.1	2650	12	5.5	2.0	2.5	1	1
161	R3	6.1	2650	12	5.5	2.4	2.7	1	2
162	R3	6.1	2650	12	5.5	2.7	2.8	3	5
163	R3	6.1	2650	12	5.5	2.7	2.8	1	6
164	R3	6.1	2650	12	5.5	3.1	2.9	2	8
165	R3	6.1	2650	12	5.5	3.1	2.9	0	8
166	R3	6.1	2650	12	5.5	3.4	3.0	2	10
167	R3	6.1	2650	12	5.5	3.4	3.0	3	13
168	R3	6.1	2650	12	5.5	3.4	2.9	1	14
169	R3	6.1	2650	12	5.5	3.7	3.0	2	16
170	R3	6.1	2650	12	5.5	3.8	3.0	0	16
171	R3	6.1	2650	12	5.5	4.1	3.0	3	19
172	R3	6.1	2650	12	5.5	4.1	3.0	2	21

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
173	R3	6.1	2650	12	5.5	4.1	3.0	2	23
174	R3	6.1	2650	12	5.5	4.1	3.0	5	28
175	R3	6.1	2650	12	5.5	4.1	3.0	3	31
176	R3	6.1	2650	12	5.5	4.1	3.0	4	35
177	R3	6.1	2650	12	5.9	1.2	1.6	0	0
178	R3	6.1	2650	12	5.9	1.6	2.0	0	0
179	R3	6.1	2650	12	5.9	2.0	2.4	1	1
180	R3	6.1	2650	12	5.9	2.4	2.7	4	5
181	R3	6.1	2650	12	5.9	2.4	2.7	5	10
182	R3	6.1	2650	12	5.9	2.4	2.7	1	11
183	R3	6.1	2650	12	5.9	2.6	2.8	9	20
184	R3	6.1	2650	12	5.9	2.6	2.8	0	20
185	R3	6.1	2650	12	5.9	2.9	3.0	2	22
186	R3	6.1	2650	12	5.9	2.7	2.9	3	25
187	R3	6.1	2650	12	5.9	2.8	3.0	1	26
188	R3	6.1	2650	12	5.9	2.9	3.0	6	32
189	R3	6.1	2650	12	5.9	2.9	3.0	4	36

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
190	C1	8.3	2350	9	7.8	2.5	2.3	0	0
191	C1	8.3	2350	9	7.8	3.1	2.8	0	0
192	C1	8.3	2350	9	7.8	3.1	2.8	0	0
193	C1	8.3	2350	9	7.8	3.7	3.4	2	2
194	C1	8.3	2350	9	7.8	3.7	3.4	1	3
195	C1	8.3	2350	9	7.8	4.3	3.8	3	6
196	C1	8.3	2350	9	7.8	4.3	3.8	0	6
197	C1	8.3	2350	9	7.8	4.9	4.2	0	6
198	C1	8.3	2350	9	7.8	5.4	4.6	1	7
199	C1	8.3	2350	9	7.8	5.4	4.6	0	7
200	C1	8.3	2350	9	7.8	5.4	4.6	0	7
201	C1	8.3	2350	9	7.8	5.4	4.6	1	8
202	C1	8.3	2350	9	7.8	5.9	4.9	0	8
203	C1	8.3	2350	9	7.8	5.9	4.9	1	9
204	C1	8.3	2350	9	7.8	6.4	5.1	1	10
205	C1	8.3	2350	9	7.8	6.8	5.3	1	11
206	C1	8.3	2350	9	7.8	6.8	5.3	2	13
207	C1	8.3	2350	9	7.8	6.8	5.3	3	16
208	C1	8.3	2350	9	7.8	6.8	5.3	3	19
209	C1	8.3	2350	9	7.8	6.8	5.3	1	20
210	C1	8.3	2350	9	7.8	7.2	5.4	2	22
211	C1	8.3	2350	9	7.8	7.2	5.4	3	25
212	C1	8.3	2350	11	7.8	3.1	2.9	2	2
213	C1	8.3	2350	11	7.8	3.1	2.9	0	2
214	C1	8.3	2350	11	7.8	3.7	3.5	0	2
215	C1	8.3	2350	11	7.8	3.7	3.5	0	2
216	C1	8.3	2350	11	7.8	4.3	4.0	3	5
217	C1	8.3	2350	11	7.8	4.3	4.0	0	5
218	C1	8.3	2350	11	7.8	4.9	4.4	4	9
219	C1	8.3	2350	11	7.8	4.9	4.4	1	10
220	C1	8.3	2350	11	7.8	5.4	4.8	3	13
221	C1	8.3	2350	11	7.8	5.4	4.8	2	15
222	C1	8.3	2350	11	7.8	5.4	4.8	4	19
223	C1	8.3	2350	11	7.8	5.4	4.8	0	19
224	C1	8.3	2350	11	7.8	6.0	5.1	2	21
225	C1	8.3	2350	11	7.8	6.0	5.1	0	21
226	C1	8.3	2350	11	7.8	6.6	5.4	1	22
227	C1	8.3	2350	11	7.8	6.6	5.4	1	23
228	C1	8.3	2350	11	7.8	7.1	5.6	1	24
229	C1	8.3	2350	11	7.8	7.1	5.6	0	24
230	C1	8.3	2350	11	7.8	7.1	5.6	1	25
231	C1	8.3	2350	11	7.8	7.6	5.7	4	29
232	C1	8.3	2350	11	7.8	7.6	5.7	0	29
233	C1	8.3	2350	11	7.8	8.1	5.8	1	30
234	C1	8.3	2350	13	7.8	1.7	1.6	0	0
235	C1	8.3	2350	13	7.8	2.3	2.2	2	2
236	C1	8.3	2350	13	7.8	2.9	2.8	2	4
237	C1	8.3	2350	13	7.8	2.9	2.8	0	4
238	C1	8.3	2350	13	7.8	3.4	3.4	4	8
239	C1	8.3	2350	13	7.8	3.4	3.4	0	8
240	C1	8.3	2350	13	7.8	4.0	3.9	3	11
241	C1	8.3	2350	13	7.8	4.0	3.9	0	11
242	C1	8.3	2350	13	7.8	4.6	4.3	2	13
243	C1	8.3	2350	13	7.8	4.6	4.3	0	13
244	C1	8.3	2350	13	7.8	5.2	4.8	6	19

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
245	C1	8.3	2350	13	7.8	5.2	4.8	1	20
246	C1	8.3	2350	13	7.8	5.2	4.8	1	21
247	C1	8.3	2350	13	7.8	5.2	4.8	0	21
248	C1	8.3	2350	13	7.8	5.8	5.1	0	21
249	C1	8.3	2350	13	7.8	6.3	5.4	3	24
250	C1	8.3	2350	13	7.8	6.3	5.4	1	25
251	C1	8.3	2350	13	7.8	6.3	5.4	0	25
252	C1	8.3	2350	13	7.8	6.3	5.4	2	27
253	C1	8.3	2350	13	7.8	6.3	5.4	1	28

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
254	C2	4.3	2800	9	7.8	1.9	1.7	0	0
255	C2	4.3	2800	9	7.8	2.5	2.3	0	0
256	C2	4.3	2800	9	7.8	3.1	2.8	3	3
257	C2	4.3	2800	9	7.8	3.1	2.8	0	3
258	C2	4.3	2800	9	7.8	3.7	3.4	3	6
259	C2	4.3	2800	9	7.8	3.7	3.4	0	6
260	C2	4.3	2800	9	7.8	4.3	3.8	0	6
261	C2	4.3	2800	9	7.8	4.3	3.8	1	7
262	C2	4.3	2800	9	7.8	4.9	4.2	0	7
263	C2	4.3	2800	9	7.8	4.9	4.2	0	7
264	C2	4.3	2800	9	7.8	5.4	4.6	0	7
265	C2	4.3	2800	9	7.8	5.4	4.6	0	7
266	C2	4.3	2800	9	7.8	5.4	4.6	2	9
267	C2	4.3	2800	9	7.8	5.4	4.6	0	9
268	C2	4.3	2800	9	7.8	5.9	4.9	0	9
269	C2	4.3	2800	9	7.8	5.9	4.9	0	9
270	C2	4.3	2800	9	7.8	6.4	5.1	2	11
271	C2	4.3	2800	9	7.8	6.4	5.1	2	13
272	C2	4.3	2800	9	7.8	6.4	5.1	1	14
273	C2	4.3	2800	9	7.8	6.4	5.1	2	16
274	C2	4.3	2800	9	7.8	6.4	5.1	0	16
275	C2	4.3	2800	9	7.8	6.4	5.1	3	19
276	C2	4.3	2800	9	7.8	6.4	5.1	1	20
277	C2	4.3	2800	9	7.8	6.4	5.1	4	24
278	C2	4.3	2800	11	7.8	1.8	1.6	0	0
279	C2	4.3	2800	11	7.8	2.4	2.3	0	0
280	C2	4.3	2800	11	7.8	2.4	2.3	0	0
281	C2	4.3	2800	11	7.8	3.1	2.9	2	2
282	C2	4.3	2800	11	7.8	3.1	2.9	0	2
283	C2	4.3	2800	11	7.8	3.7	3.5	2	4
284	C2	4.3	2800	11	7.8	3.7	3.5	2	6
285	C2	4.3	2800	11	7.8	4.3	4.0	3	9
286	C2	4.3	2800	11	7.8	4.3	4.0	2	11
287	C2	4.3	2800	11	7.8	4.3	4.0	1	12
288	C2	4.3	2800	11	7.8	4.9	4.4	3	15
289	C2	4.3	2800	11	7.8	4.9	4.4	2	17
290	C2	4.3	2800	11	7.8	4.9	4.4	2	19
291	C2	4.3	2800	11	7.8	4.9	4.4	2	21
292	C2	4.3	2800	11	7.8	4.9	4.4	0	21
293	C2	4.3	2800	11	7.8	5.4	4.8	3	24
294	C2	4.3	2800	11	7.8	5.4	4.8	1	25
295	C2	4.3	2800	11	7.8	5.4	4.8	2	27
296	C2	4.3	2800	11	7.8	5.4	4.8	3	30
297	C2	4.3	2800	11	7.8	5.4	4.8	3	33
298	C2	4.3	2800	11	7.8	5.4	4.8	1	34
299	C2	4.3	2800	11	7.8	5.4	4.8	2	36
300	C2	4.3	2800	13	7.8	1.7	1.6	1	1
301	C2	4.3	2800	13	7.8	2.3	2.2	1	2
302	C2	4.3	2800	13	7.8	2.3	2.2	0	2
303	C2	4.3	2800	13	7.8	2.3	2.2	0	2
304	C2	4.3	2800	13	7.8	2.9	2.8	3	5
305	C2	4.3	2800	13	7.8	2.9	2.8	1	6
306	C2	4.3	2800	13	7.8	2.9	2.8	1	7
307	C2	4.3	2800	13	7.8	2.9	2.8	0	7
308	C2	4.3	2800	13	7.8	3.4	3.4	1	8

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
309	C2	4.3	2800	13	7.8	3.4	3.4	0	8
310	C2	4.3	2800	13	7.8	4.0	3.9	0	8
311	C2	4.3	2800	13	7.8	4.0	3.9	1	9
312	C2	4.3	2800	13	7.8	4.0	3.9	1	10
313	C2	4.3	2800	13	7.8	4.0	3.9	1	11
314	C2	4.3	2800	13	7.8	4.6	4.3	2	13
315	C2	4.3	2800	13	7.8	4.6	4.3	0	13
316	C2	4.3	2800	13	7.8	4.6	4.3	0	13
317	C2	4.3	2800	13	7.8	4.6	4.3	2	15
318	C2	4.3	2800	13	7.8	4.6	4.3	1	16
319	C2	4.3	2800	13	7.8	5.2	4.8	0	16
320	C2	4.3	2800	13	7.8	5.2	4.8	1	17
321	C2	4.3	2800	13	7.8	5.2	4.8	2	19
322	C2	4.3	2800	13	7.8	5.2	4.8	1	20
323	C2	4.3	2800	13	7.8	5.8	5.1	2	22
324	C2	4.3	2800	13	7.8	5.8	5.1	0	22
325	C2	4.3	2800	13	7.8	5.8	5.1	6	28
326	C2	4.3	2800	13	7.8	5.8	5.1	0	28
327	C2	4.3	2800	13	7.8	5.8	5.1	1	29
328	C2	4.3	2800	13	7.8	6.3	5.4	0	29
329	C2	4.3	2800	13	7.8	6.3	5.4	4	33
330	C2	4.3	2800	13	7.8	6.3	5.4	6	39
331	C2	4.3	2800	13	7.8	6.3	5.4	1	40

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
332	C3	3.9	2350	11	8.0	0.5	0.3	0	0
333	C3	3.9	2350	11	8.0	1.7	1.0	3	3
334	C3	3.9	2350	11	8.0	2.2	1.4	8	11
335	C3	3.9	2350	11	8.0	2.2	1.4	2	13
336	C3	3.9	2350	11	8.0	2.8	1.7	0	13
337	C3	3.9	2350	11	8.0	2.8	1.7	7	20
338	C3	3.9	2350	11	8.0	2.8	1.7	4	24
339	C3	3.9	2350	11	8.0	2.8	1.7	3	27
340	C3	3.9	2350	11	8.0	3.4	2.1	15	42
341	C3	3.9	2350	11	8.0	3.4	2.1	11	53
342	C3	3.9	2350	11	8.0	3.4	2.1	10	63
343	C3	3.9	2350	13	8.0	1.1	0.7	0	0
344	C3	3.9	2350	13	8.0	1.4	0.8	0	0
345	C3	3.9	2350	13	8.0	1.7	1.0	0	0
346	C3	3.9	2350	13	8.0	1.7	1.0	0	0
347	C3	3.9	2350	13	8.0	2.0	1.2	3	3
348	C3	3.9	2350	13	8.0	2.0	1.2	5	8
349	C3	3.9	2350	13	8.0	2.0	1.2	2	10
350	C3	3.9	2350	13	8.0	2.2	1.4	10	20
351	C3	3.9	2350	13	8.0	2.2	1.4	3	23
352	C3	3.9	2350	13	8.0	2.2	1.4	5	28
353	C3	3.9	2350	13	8.0	2.2	1.4	0	28
354	C3	3.9	2350	13	8.0	2.5	1.6	5	33
355	C3	3.9	2350	13	8.0	2.5	1.6	5	38
356	C3	3.9	2350	13	8.0	2.5	1.6	0	38
357	C3	3.9	2350	13	8.0	2.8	1.7	4	42
358	C3	3.9	2350	13	8.0	3.1	1.9	6	48
359	C3	3.9	2350	13	8.0	3.1	1.9	4	52
360	C3	3.9	2350	13	8.0	3.1	1.9	2	54
361	C3	3.9	2350	13	8.0	3.4	2.1	4	58
362	C3	3.9	2350	13	8.0	3.4	2.1	0	58

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
363	C4	3.9	2350	11	8.0	1.1	0.7	1	1
364	C4	3.9	2350	11	8.0	1.4	0.8	0	1
365	C4	3.9	2350	11	8.0	1.7	1.0	7	8
366	C4	3.9	2350	11	8.0	1.7	1.0	0	8
367	C4	3.9	2350	11	8.0	1.7	1.0	0	8
368	C4	3.9	2350	11	8.0	2.0	1.2	8	16
369	C4	3.9	2350	11	8.0	2.0	1.2	5	21
370	C4	3.9	2350	11	8.0	2.0	1.2	6	27
371	C4	3.9	2350	11	8.0	2.0	1.2	2	29
372	C4	3.9	2350	11	8.0	2.2	1.4	7	36
373	C4	3.9	2350	11	8.0	2.2	1.4	6	42
374	C4	3.9	2350	11	8.0	2.2	1.4	5	47
375	C4	3.9	2350	11	8.0	2.2	1.4	3	50
376	C4	3.9	2350	11	8.0	2.2	1.4	2	52
377	C4	3.9	2350	11	8.0	2.5	1.6	6	58
378	C4	3.9	2350	11	8.0	2.5	1.6	10	68
379	C4	3.9	2350	11	8.0	2.5	1.6	2	70
380	C4	3.9	2350	11	8.0	2.8	1.7	11	81
381	C4	3.9	2350	11	8.0	2.8	1.7	7	88
382	C4	3.9	2350	11	8.0	2.8	1.7	12	100
383	C4	3.9	2350	11	8.0	2.8	1.7	12	112
384	C4	3.9	2350	11	8.0	2.8	1.7	8	120
385	C4	3.9	2350	11	8.0	2.8	1.7	2	122
386	C4	3.9	2350	11	8.0	3.1	1.9	11	133
387	C4	3.9	2350	11	8.0	3.1	1.9	8	141
388	C4	3.9	2350	11	8.0	3.1	1.9	7	148
389	C4	3.9	2350	11	8.0	3.1	1.9	6	154
390	C4	3.9	2350	13	8.0	1.1	0.7	1	1
391	C4	3.9	2350	13	8.0	1.4	0.8	1	2
392	C4	3.9	2350	13	8.0	1.7	1.0	2	4
393	C4	3.9	2350	13	8.0	1.7	1.0	0	4
394	C4	3.9	2350	13	8.0	2.0	1.2	8	12
395	C4	3.9	2350	13	8.0	2.0	1.2	5	17
396	C4	3.9	2350	13	8.0	2.0	1.2	0	17
397	C4	3.9	2350	13	8.0	2.2	1.4	11	28
398	C4	3.9	2350	13	8.0	2.2	1.4	4	32
399	C4	3.9	2350	13	8.0	2.2	1.4	1	33
400	C4	3.9	2350	13	8.0	2.2	1.4	3	36
401	C4	3.9	2350	13	8.0	2.2	1.4	0	36
402	C4	3.9	2350	13	8.0	2.5	1.6	3	39
403	C4	3.9	2350	13	8.0	2.5	1.6	4	43
404	C4	3.9	2350	13	8.0	2.5	1.6	5	48
405	C4	3.9	2350	13	8.0	2.5	1.6	1	49
406	C4	3.9	2350	13	8.0	2.5	1.6	3	52
407	C4	3.9	2350	13	8.0	2.5	1.6	4	56
408	C4	3.9	2350	13	8.0	2.5	1.6	4	60
409	C4	3.9	2350	13	8.0	2.8	1.7	8	68
410	C4	3.9	2350	13	8.0	2.8	1.7	10	78
411	C4	3.9	2350	13	8.0	2.8	1.7	5	83
412	C4	3.9	2350	13	8.0	2.8	1.7	1	84
413	C4	3.9	2350	13	8.0	2.8	1.7	1	85

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
414	C5	3.9	2350	11	8.0	1.1	0.7	0	0
415	C5	3.9	2350	11	8.0	1.4	0.8	2	2
416	C5	3.9	2350	11	8.0	1.7	1.0	3	5
417	C5	3.9	2350	11	8.0	1.7	1.0	1	6
418	C5	3.9	2350	11	8.0	1.7	1.0	2	8
419	C5	3.9	2350	11	8.0	2.0	1.2	4	12
420	C5	3.9	2350	11	8.0	2.0	1.2	2	14
421	C5	3.9	2350	11	8.0	2.0	1.2	3	17
422	C5	3.9	2350	11	8.0	2.0	1.2	2	19
423	C5	3.9	2350	11	8.0	2.2	1.4	3	22
424	C5	3.9	2350	11	8.0	2.2	1.4	4	26
425	C5	3.9	2350	11	8.0	2.2	1.4	9	35
426	C5	3.9	2350	11	8.0	2.2	1.4	2	37
427	C5	3.9	2350	11	8.0	2.2	1.4	2	39
428	C5	3.9	2350	11	8.0	2.2	1.4	0	39
429	C5	3.9	2350	11	8.0	2.5	1.6	2	41
430	C5	3.9	2350	11	8.0	2.5	1.6	2	43
431	C5	3.9	2350	11	8.0	2.5	1.6	15	58
432	C5	3.9	2350	11	8.0	2.5	1.6	5	63
433	C5	3.9	2350	11	8.0	2.5	1.6	4	67
434	C5	3.9	2350	11	8.0	2.5	1.6	0	67
435	C5	3.9	2350	11	8.0	2.8	1.7		67
436	C5	3.9	2350	11	8.0	2.8	1.7		67
437	C5	3.9	2350	11	8.0	2.8	1.7		67
438	C5	3.9	2350	11	8.0	2.8	1.7		67
439	C5	3.9	2350	13	8.0	1.1	0.7	0	0
440	C5	3.9	2350	13	8.0	1.4	0.8	2	2
441	C5	3.9	2350	13	8.0	1.4	0.8	0	2
442	C5	3.9	2350	13	8.0	1.7	1.0	2	4
443	C5	3.9	2350	13	8.0	1.7	1.0	1	5
444	C5	3.9	2350	13	8.0	2.0	1.2	0	5
445	C5	3.9	2350	13	8.0	2.0	1.2	0	5
446	C5	3.9	2350	13	8.0	2.2	1.4	9	14
447	C5	3.9	2350	13	8.0	2.2	1.4	8	22
448	C5	3.9	2350	13	8.0	2.2	1.4	9	31
449	C5	3.9	2350	13	8.0	2.2	1.4	4	35
450	C5	3.9	2350	13	8.0	2.2	1.4	11	46
451	C5	3.9	2350	13	8.0	2.2	1.4	9	55
452	C5	3.9	2350	13	8.0	2.2	1.4	3	58
453	C5	3.9	2350	13	8.0	2.2	1.4	2	60
454	C5	3.9	2350	13	8.0	2.5	1.6	2	62
455	C5	3.9	2350	13	8.0	2.5	1.6	4	66
456	C5	3.9	2350	13	8.0	2.5	1.6	7	73
457	C5	3.9	2350	13	8.0	2.5	1.6	9	82
458	C5	3.9	2350	13	8.0	2.5	1.6	5	87
459	C5	3.9	2350	13	8.0	2.5	1.6		87

Test	Model	Mass	Density	Tp	Depth	Hs offshore	Hs structure	Displaced	Total
460	C6	3.9	2350	11	8.0	1.7	1.0	0	0
461	C6	3.9	2350	11	8.0	2.0	1.2	0	0
462	C6	3.9	2350	11	8.0	2.2	1.4	0	0
463	C6	3.9	2350	11	8.0	2.5	1.6	1	1
464	C6	3.9	2350	11	8.0	2.8	1.7	2	3
465	C6	3.9	2350	11	8.0	2.8	1.7	1	4
466	C6	3.9	2350	11	8.0	2.8	1.7	0	4
467	C6	3.9	2350	11	8.0	3.1	1.9	1	5
468	C6	3.9	2350	11	8.0	3.1	1.9	0	5
469	C6	3.9	2350	11	8.0	3.4	2.1	3	8
470	C6	3.9	2350	11	8.0	3.7	2.2	0	8
471	C6	3.9	2350	11	8.0	4.0	2.4	2	10
472	C6	3.9	2350	11	8.0	4.3	2.5	1	11
473	C6	3.9	2350	11	8.0	4.3	2.5	3	14
474	C6	3.9	2350	11	8.0	4.3	2.5	3	17
475	C6	3.9	2350	11	8.0	4.6	2.6	1	18
476	C6	3.9	2350	11	8.0	4.8	2.7	4	22
477	C6	3.9	2350	11	8.0	4.8	2.7	6	28
478	C6	3.9	2350	11	8.0	4.8	2.7	9	37
479	C6	3.9	2350	11	8.0	5.1	2.8	2	39
480	C6	3.9	2350	11	8.0	5.1	2.8	0	39
481	C6	3.9	2350	11	8.0	5.1	2.8	4	43
482	C6	3.9	2350	11	8.0	5.4	2.9	2	45
483	C6	3.9	2350	11	8.0	5.4	2.9	15	60
484	C6	3.9	2350	11	8.0	5.4	2.9		60
485	C6	3.9	2350	13	8.0	1.1	0.7	0	0
486	C6	3.9	2350	13	8.0	1.4	0.8	1	1
487	C6	3.9	2350	13	8.0	1.7	1.0	0	1
488	C6	3.9	2350	13	8.0	2.0	1.2	1	2
489	C6	3.9	2350	13	8.0	2.2	1.4	0	2
490	C6	3.9	2350	13	8.0	2.5	1.6	1	3
491	C6	3.9	2350	13	8.0	2.8	1.7	4	7
492	C6	3.9	2350	13	8.0	3.1	1.9	2	9
493	C6	3.9	2350	13	8.0	3.1	1.9	0	9
494	C6	3.9	2350	13	8.0	3.4	2.1	0	9
495	C6	3.9	2350	13	8.0	3.7	2.2	3	12
496	C6	3.9	2350	13	8.0	3.7	2.2	4	16
497	C6	3.9	2350	13	8.0	3.7	2.2	1	17
498	C6	3.9	2350	13	8.0	3.7	2.2	0	17
499	C6	3.9	2350	13	8.0	4.0	2.4	10	27
500	C6	3.9	2350	13	8.0	4.3	2.5	13	40
501	C6	3.9	2350	13	8.0	4.3	2.5	5	45
502	C6	3.9	2350	13	8.0	4.3	2.5	5	50
503	C6	3.9	2350	13	8.0	4.3	2.5	4	54