Proceedings of the 9th International Conference on Physical Modelling in Coastal Engineering (Coastlab24)
Delft, Netherlands, May 13-16, 2024

Conference Paper**,** DOI: 10.59490/coastlab.2024.703

PHYSICAL MODELLING OF ROCK BAGS UNDER WAVE ATTACK

IAN COGHLAN[1], JAMES CARLEY[2] & DAN MESSITER[3]

1 Water Research Laboratory, School of Civil and Environmental Engineering, UNSW Sydney, Australia, i.coghlan@wrl.unsw.edu.au
2 Water Research Laboratory, School of Civil and Environmental Engineering, UNSW Sydney, Australia, james.carley@unsw.edu.au
3 Royal HaskoningDHV, Australia, dan.messiter@rhdhv.com

ABSTRACT

In 1987, rock bags were developed by Kyowa in Japan to protect against erosion from hydraulic processes in riverine, lake, coastal and marine environments. Since 2020, rock bags have been used as a temporary or emergency coastal protection unit for seawalls on some beaches in Australia (e.g. Wamberal Beach and Collaroy Beach). These structures have typically been built against a pre-existing dune or eroded dune scarp for protection of landward coastal assets. While a limited amount of hydraulic scale modelling has been undertaken for some rock bag applications, their behaviour in shallow water, coastal environments under wave forces had not previously been quantitively evaluated. A two-stage physical modelling program was carried out to assess the behaviour of rock bags when used in this emerging erosion protection application.

As a result of the scale laboratory tests for shallow water seawalls constructed from rock bags, specific results were obtained for a proposed structure at Stockton Beach (Newcastle, Australia) as well as producing generic design information that could be applied at other locations. As expected, the design wave height for rock bag damage (displacement) was found to be inversely proportional to wave period. Preliminary design stability curves were developed for rock bags under monochromatic and irregular wave attack. In addition to displacement, settlement of the rock bags was observed during the modelling, and it is recommended that consideration should be given to vertical settlement over the design life of these structures. Wave runup was also found to be high; this is also an important consideration for future rock bag seawalls in either establishing the design crest level to prevent wave overtopping or adopting and managing a permissible amount of wave overtopping during a design event.

KEYWORDS: Rock bag, filter unit, seawall, armour stability, wave runup.

1 INTRODUCTION

Rock bags (alternatively referred to as “filter units”) are a product developed in 1987 by Kyowa in Japan to protect against erosion from hydraulic processes in riverine, lake, coastal and marine environments. Within Australasia, the rock bags are imported and distributed by Bluemont Pty Ltd (hereafter “Bluemont”). Site-specific physical modelling studies have previously been undertaken to examine the stability of a seawall comprising rock bags in a port, with scour by vessel propeller side thrusters (Messiter *et al.*, 2019), and the stability of wind turbine scour protection comprising rock bags at the seabed forced by indirect wave action (transitional water depth) and tidal currents (HR Wallingford, 2012). A generic (i.e. non site-specific) physical modelling study was also previously undertaken to assess the stability of deep water breakwaters/seawalls comprising 8 t rock bags under direct wave attack (Mizutani, *et al.*, 2007). However, the behaviour of rock bags in shallow water, coastal environments had not previously been assessed in a physical modelling study. This is despite the use of rock bags since 2020 as a temporary or emergency coastal protection unit for seawalls located at the back of some beaches in Australia (e.g. Wamberal Beach and Collaroy Beach). Since this is an emerging erosion protection application for rock bags, Bluemont wanted to assess their hydraulic stability in this arrangement using scale model laboratory tests.

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney was engaged by Haskoning Australia (hereafter “Haskoning”) to partner in carrying out a physical modelling study for Bluemont for shallow water seawalls constructed from rock bags. It was envisaged that seawalls made from rock bags would typically be built with a geotextile underlayer against a pre-existing dune or eroded dune scarp for protection of landward coastal assets. Haskoning directed the two-stage study and WRL designed and operated the scale physical model tests in the laboratory. The focus of the physical modelling program was the stability of the rock bags under wave attack, rather than assessment of the durability of the rock bag mesh/netting material.

Stage 1 of testing focused on assessing the hydraulic stability of 2 t and 4 t Ecogreen rock bags under generic design conditions that were representative of many coastal locations across Australasia. Direct wave impacts on the rock bags were the primary damage mechanism being investigated. To isolate this failure mechanism, the model rock bag seawall structures were built very high so that they were not significantly overtopped (to avoid displacement of rock bags at the crest via overtopping) and the bottom course of rock bags was mechanically restrained (to prevent toe slip). The maximum wave runup extent on the model rock bag seawalls was also observed during testing.

Stage 2 of study involved site-specific testing for an interim seawall design comprising Ecogreen rock bags at Stockton Beach (Newcastle, Australia). Prior to Stage 2 testing, the generic design information from Stage 1 was used to select a rock bag mass (4 t) which was expected to be stable on the main part of the structure slope. In addition to rock bag damage by direct wave impacts, physical modelling of the Stockton Beach interim seawall also examined potential displacement of rock bags at the crest (via overtopping) and the toe.

Unless otherwise specified, data represented are given in prototype (full scale, real-world) equivalent units. Reduced levels refer to the present day, local Mean Sea Level (MSL) datum.

2 STAGE 1: TESTING WITH GENERIC DESIGN CONDITIONS

2.1 Physical Model Setup

2.1.1 Testing Facility

Two-dimensional testing was undertaken in a flume at UNSW-WRL measuring approximately 1.2 m in width, 1.6 m in depth and 44 m in length. The wave generator is a hydraulic, piston-type paddle.

2.1.2 Design and Scaling

Model scaling was based on geometric similarity with an undistorted length scale of 1:19.4 being used for all tests. Selection of the length ratio was primarily based on the size of available model rock bags previously tested at UNSW-WRL. The scaling relationship between length and time was determined by Froudian similitude. Both the prototype and model water and fill rock densities are summarised in Table 1. The scaling relationship for fill rock mass (1:8,000) considered the differences between the water and fill rock densities in the prototype and the model based on the method of Hudson (1979).

Table 1. Prototype and model density values.

Parameter	Value		Units
	Prototype	Model	
Water density	1,025	998	kg/m^3
Fill rock density (average)	2,650	2,630	kg/m^3

2.1.3 Environmental Design Conditions

The objective of Stage 1 was to assess rock bag stability under a range of wave and water level conditions. While only a single design scour level (-1.0 m MSL) was tested, the use of multiple water levels meant that the model covered a range of effective/relative scour levels. Five wave periods (5, 7.5, 10, 15 and 20 s) were considered representative of the range experienced by many Australasian beaches for testing. Permutations of wave height, period and still water level were combined to expose the rock bag seawalls to unbroken, breaking and broken waves. Testing was undertaken using two different methods: monochromatic and irregular wave attack.

2.1.4 Bathymetry

A generic offshore bathymetric profile was adopted which was representative of the steeper envelope of Australasian beaches (detailed in Coghlan *et al.*, 2022). This was selected to ensure that wave heights at the model seawall were maximised (but realistic), which was desirable for modelling conservative but not unrealistic rock bag displacement. The adopted rock bag seawall toe was -1 m MSL which is a commonly adopted toe elevation for seawall structures in microtidal regions of Australasia (Nielsen *et al.*, 1992). The model bathymetry, constructed from water-resistant plywood, extended 379 m seaward of the model structure with the following characteristics:

- intersected the structure at -1.0 m MSL;
- 1V:20H slope from -1.0 m MSL to -6.4 m MSL;
- 1V:50H slope -6.4 m MSL to -11.8 m MSL;
- seaward of -11.8 m MSL false floor sloped at 1V:10H until it intersected the permanent flume floor at -16.1 m MSL.

2.1.5 Seawall Backfill Material

The prototype backfill material on which a geotextile underlayer would be placed (and the generic seawall itself) was assumed to be sand fill which is impermeable at wave period timescales for modelling purposes. Since it is not possible to correctly scale such fine material in the model, the batter slope for the model seawall was constructed with an impermeable hollow plywood core. This frame was covered with geotextile material to approximately model the friction interface that would exist between the rock bags and the geotextile underlayer in the prototype. The modelling approach for the backfill material was expected to have yielded realistic or conservative (due to the planar interface) stability results for the rock bags.

2.1.6 Rock Bags

Two different sizes of Ecogreen type model rock bags were fabricated by Kyowa and provided to UNSW-WRL for testing as summarised in Table 2 (see also Figure 1). The rock bags have a conical shape during lifting (Figure 2B and 2C) but resemble a torus shape (without a hole) when at rest (Figure 2D and 2E). The mass and diameter of the model rock bags matched the product specifications (Bluemont, 2016) well. A good match was also achieved for the average installed height of the 2 t Ecogreen rock bags (0.39 m) which have a specified value of 0.4 m. However, the average installed height of the model 4 t Ecogreen rock bags (0.46 m) was less than the value in the product specification (0.6 m).

Table 2. Properties of model rock bags tested.

Type	Mass (t)	Diameter (m)	Average Installed Height (m)
Ecogreen	2	1.9	0.39
Ecogreen	4	2.4	0.46

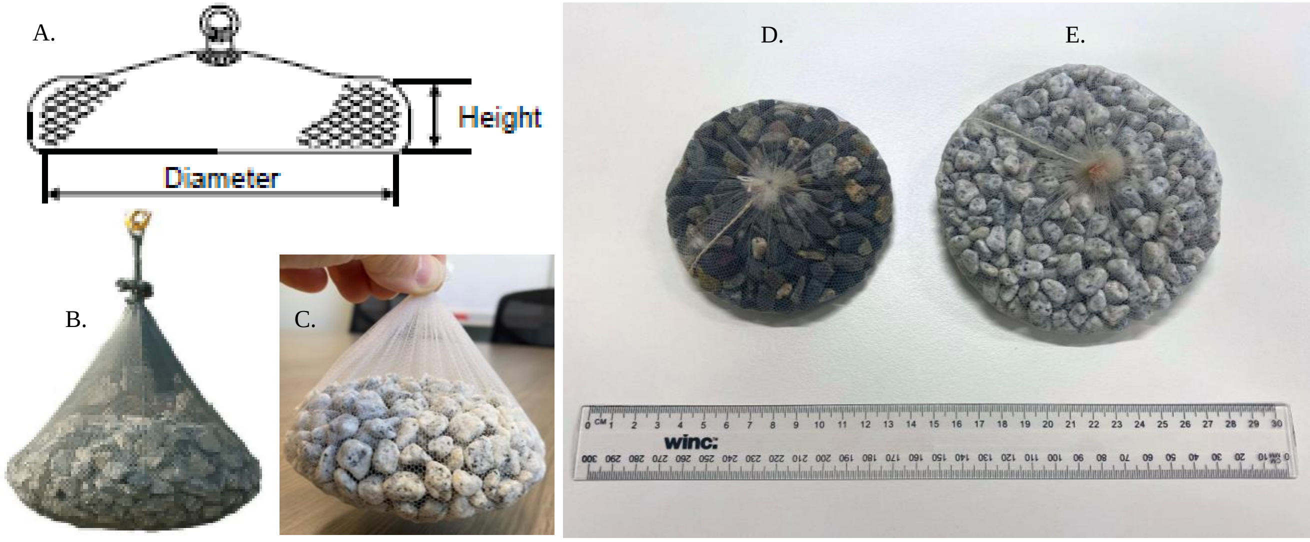


Figure 1. Rock bag dimensions (A), Ecogreen during lifting: prototype (B) and model (C), model bags at rest: 2 t (D) and 4 t (E).

The protype mesh/netting material used to make the rock bags is recycled polyester. The material used by Kyowa in the fabrication of the model rock bags is unknown to UNSW-WRL, but appeared "wedding veil"-like and qualitatively similar in porosity to the prototype material.

The individual rocks within the 2 t and 4 t model rock bags had masses varying between 1.0 kg and 13.0 kg, which corresponds to nominal rock diameters of between approximately 75 and 170 mm. This complied with the product specification for a maximum fill rock diameter of 200 mm (Bluemont, 2016). Note that due to the small size of the model fill rock at a scale of 1:19.4, flow through the rock bags was not fully turbulent ($Re < 3\times10^4$ which is the minimum value recommended in HYDRALAB III, 2007) for all test conditions. That is, the onset of fill rock movement within the rock bags in the model was likely at a lower wave height than in the prototype (real-world). However, due to the relatively large size of the model rock bags, the onset of rock bag displacement was expected to be well reproduced in the model.

The model rock bags were placed directly on top of each other (i.e. horizontal orientation) up the batter slope in a "stretcher bond" fashion so that the vertical "joints" (e.g. where the sides of two rock bags meet) were staggered each course by half a bag diameter. To preserve the "stretcher bond" geometry, "half" rock bags were fabricated and placed at the edges of the model seawall where required. To prevent their displacement and preclude edge effects in the stability results, chain was laid against both sides of the flume on top of the rock bags (since they do not have the stabilising effect from adjacent rock bags on both sides) and “half” rock bags (since their stability was not of interest).

2.1.7 Structure Configurations

Table 3 summarises the four structure configurations that were modelled. The structure slope was 1V:1.5H, the toe elevation was -1 m MSL (as mentioned in Section 2.1.4) and the elevation of the crest of the plywood backing board (covered with geotextile) was 15 m MSL for all configurations. Note that crest elevation of the rock bag structures exceeded the elevation of the backing board for Configurations A and D. Example photos of Configuration A (prior to testing) are shown in Figure 2.

To focus on rock bag displacement by direct wave impacts only, a piece of timber (approximately ½ the height of a rock bag) was screwed down directly seaward of the bottom course of rock bags to prevent toe slip for all configurations (except Configuration B). This simulated a real-world toe being excavated during installation and then buried in sand. However, it is acknowledged that this failure mechanism could occur in the real-world if the sand level at the structure was eroded down to the toe elevation of the structure.

All model seawall structures tested comprised a single layer of rock bags (except Configuration B which had two layers at the toe but no timber restraint).

For the tests with the 4 t Ecogreen rock bags, three different crest elevations were used on the basis of observed results from earlier tests. Following the first test on Configuration B, the second layer of rock bags at the toe was replaced with a timber restraint in Configuration C. This change allowed an extra course of rock bags to be added to the crest. During the first test on Configuration C, 4 t Ecogreen rock bags at the crest were displaced by wave runup/rundown. To prevent failure by this mechanism for subsequent tests, 8 courses of S type 4 t rock bags were added on top of the Ecogreen units to form Configuration D (no further model 4 t Ecogreen rock bags were available). The S type rock bags had the same mass, diameter and height as the Ecogreen units but had an internal restraining rope from top to bottom through the centre (which was absent in the Ecogreen units). It is acknowledged that this allowed the possibility of “contamination” of the damage results by testing with a mix of rock bag types (which occurred in two tests on Configuration D).

Table 3. Summary of rock bag seawall configurations tested.

Structure Configuration	Rock Bag Mass (t)	Structure Slope	Test Types	No. of Rock Bag Courses High (-)	Crest Elevation (m MSL)	Toe Elevation (m MSL)
A	2	1V:1.5H	Monochromatic	43	15.7	-1
B	4	1V:1.5H	Irregular	29#	12.3	-1
C	4	1V:1.5H	Monochromatic, Irregular	30	12.7	-1
D	4	1V:1.5H	Monochromatic, Irregular	38*	16.1	-1

For Configuration B, the toe comprised 2 layers of 4 t Ecogreen rock bags (i.e. a timber restraint was not used)

* For Configuration D, 8 courses of S type 4 t model rock bags were used at the crest

Figure 2. Top view (left) and side view (right) of Structure Configuration A: 2 t Ecogreen rock bags (before testing).

2.1.8 Monochromatic Wave Climates

Wave packets containing five individual waves were generated for each wave period (T). The aim was to identify the steepness limited or depth limited wave height (H) for each T which caused the first rock bag to be displaced from the seawall. Each test began with very small H. Each wave packet was run four times (with a gap between each packet to allow reflected energy in the flume to reduce) exposing the rock bag seawall to a total of 20 waves at each H. The wave height was then increased and the 20 wave sequence repeated. No repairs were made to the model seawall in between each 20 wave sequence. This was undertaken iteratively at each tested water level until the first rock bag was displaced or waves at the structure became steepness limited or depth limited. Once the wave height causing displacement was identified, the 20 wave sequence was generally repeated to examine the progression of damage. The model seawall was re-built for each wave period test series. The still water levels (SWL) varied between 1.5 and 5.0 m MSL (2.5 to 6.0 m water depth at the structure). Note that for the 7.5 s and 10 s tests with 2 t rock bags, multiple water levels were used because the waves were depth limited and not causing displacement at the initial water level. The monochromatic wave test conditions for the 2 t and 4 t rock bag structures are summarised in Table 4.

Table 4. Summary of test conditions for monochromatic wave attack.

Rock Bag Mass (t)	Wave Period, T (s)	Still Water Level (m MSL)	Wave Height at Structure, H (m)	No. of 20 wave sequences (-)
2	5	3.0	0.9 to 2.7	5
2	7.5	3.0	0.9 to 4.1	6
2	7.5	4.0	3.5 to 4.6	5
2	7.5	5.0	5.5	4 (inc. 3 repeats)
2	10	1.5	0.8 to 3.4	7
2	10	3.0	0.6 to 4.2	10 (inc. 2 repeats)
2	15	1.5	0.4 to 3.2	8 (inc. 2 repeats)
4	5	4.0	0.8 to 3.4	9
4	10	4.0	0.6 to 5.4	15 (inc. 4 repeats)
4	15	3.0	0.5 to 5.1	11 (inc. 2 repeats)
4	20	3.0	2.3 to 4.3	14 (inc. 2 repeats)

2.1.9 Irregular Wave Climates

Synthetic 1,000 wave time series were generated using a JONSWAP spectrum (Hasselmann *et al.*, 1973) based on a random seed, a peak enhancement factor of 3.3, peak spectral wave periods (T_P) of 10, 15 and 20 s, and the largest significant wave height (H_S) the wave paddle could generate for each T_P and water level combination. These corresponded to prototype storm durations of 2.2, 3.4 and 4.6 hours, respectively. The aim was to identify the depth limited H_S for each T_P which caused "initial damage" to the rock bag seawall. The model seawall was re-built for each irregular wave test. The water levels varied between 1.5 and 4.0 m MSL (2.5 to 5.0 m water depth at the structure). Note that for the 10 s and 20 s tests, multiple water levels were used to increase the displacement of rock bags. The irregular wave test conditions for the 4 t rock bag structures are summarised in Table 5, including $H_{10\%}$ (the height exceeded by 10% of waves at the structure).

Table 5. Summary of test conditions for irregular wave attack.

Rock Bag Mass (t)	Peak Spectral Wave Period, T_P (s)	Still Water Level (m MSL)	Significant Wave Height at Structure, H_S (m)	$H_{10\%}$ at Structure (m)
4	10	3.0	2.7	3.1
4	10	4.0	3.4	3.9
4	15	3.0	2.8	3.3
4	20	1.5	1.8	2.1
4	20	3.0	2.4	2.9

2.2 Physical Model Data Collection and Analysis

2.2.1 Wave Data

Waves that reflected from model structures towards the wave generator were not actively absorbed by the wave generator. Instead, test wave climates were first calibrated both in deep water (-16.1 m MSL) and at the seawall toe (-1 m MSL), without a model structure in place. Wave conditions were set in deeper water near the model boundary and then allowed to shoal and break across the model bathymetry. Reflections from the far end of the of the wave flume (without a model structure in place) were minimised using low gradient, dissipative materials. The same calibrated test wave climates were then reproduced with the model structures in place. The wave conditions measured at -16.1 m MSL during structural tests were then compared with the measurements without a structure in place to ensure that the influence of wave reflections from the structure were minimal. For brevity, only wave climate data recorded at the seawall toe (-1 m MSL) has been reported throughout this paper.

For the monochromatic wave climates, only the first one to four waves in the packet at both probe locations were assessed because wave reflections from the absorptive foam installed at the landward end of the wave flume began to interfere with the incident waves before the whole packet had passed. The largest wave height (by up-crossing or down-crossing) measured by the probe in these first waves was considered the representative wave height for each five wave packet.

For the irregular wave climates, waves were measured using two, three probe arrays to allow for the separation of incident and reflected waves using the method of Mansard and Funke (1980). Use of this technique further reduced the influence of reflected waves on the calibrated irregular wave climates.

2.2.2 Wave Runup Data

During testing (irregular waves only), it was visually noted if waves overtopped the top course (crest) of the rock bag seawall. If waves didn’t overtop the top rock bag course, the approximate maximum wave runup extent was visually noted.

2.2.3 Rock Bag Damage Assessment

Front-view and side-view (Figure 3) video footage was recorded for each test and used to support visual observations in assessment of rock bag damage. Still photos were also taken of each structure prior to and following each stability test.

Displacement of a rock bag was defined as occurring when a single rock bag was dislodged from the rock bag matrix.

In addition to displacement, the other key rock bag damage parameter was settlement. This either resulted in an overall change to the packing density of the rock bag matrix (typically a general compaction of the whole matrix) or localised changes to the packing density within the matrix (loosening in one region and tightening in another). To quantify the extent of settlement during testing (irregular waves only), the crest level of the top course of rock bags was measured at three locations (centre of the flume and against both walls) using a dumpy level and a levelling staff before and after each test.

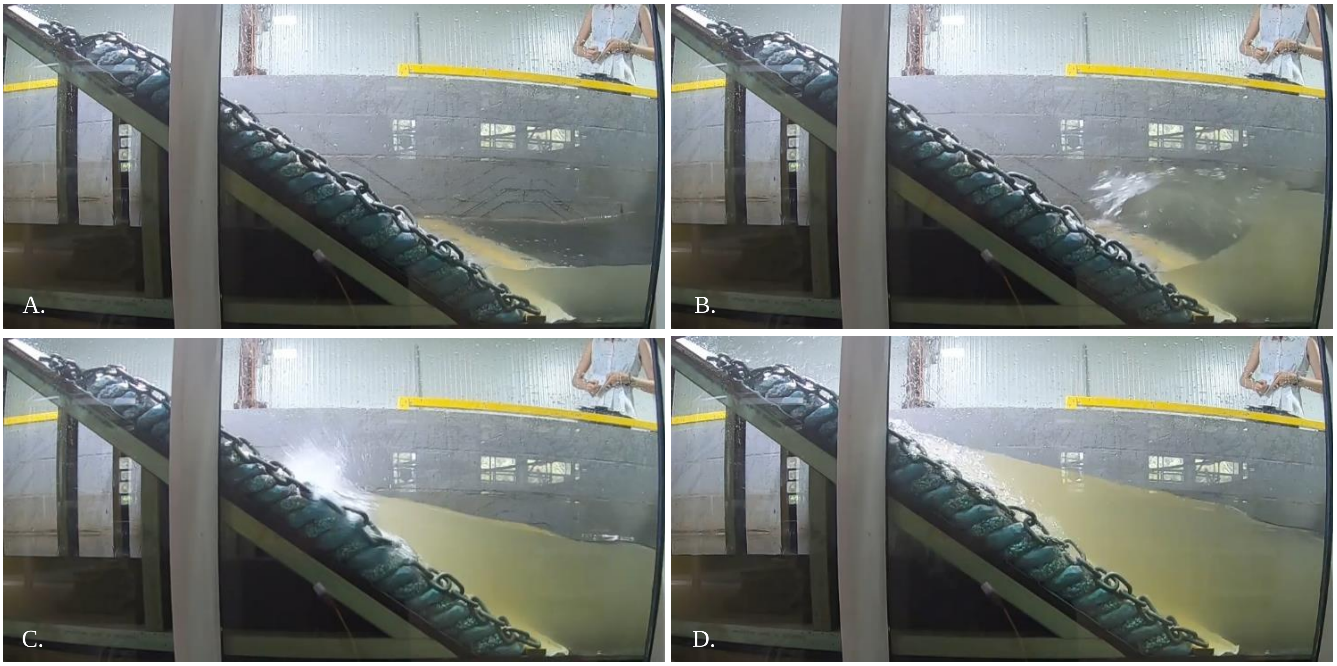


Figure 3. Example side-view sequence (frames A to D) for 4 t Ecogreen rock bags under irregular wave attack (T_P = 15 s).

2.3 Physical Model Results

2.3.1 Monochromatic Wave Tests

A summary of the hydraulic stability results from the monochromatic wave tests on 2 t and 4 t Ecogreen rock bags is detailed in Table 6. It includes the highest wave height which caused no rock bag displacement and the smallest wave height which caused displacement for each wave period tested. Note that waves were steepness limited for the 5 s period tests. As such, it was concluded that 2 t and 4 t Ecogreen rock bags cannot be displaced by 5 s waves of any physically possible height.

Table 6. Hydraulic stability test results for monochromatic wave attack on 2 t and 4 t Ecogreen rock bags.

Structure Configuration	Rock Bag Mass (t)	Wave Period, T (s)	Still Water Level (m MSL)	Maximum Wave Height at Structure with No Displacement (m)	Minimum Wave Height at Structure with Displacement (m)
A	2	5	3.0	2.7	-
A	2	7.5	5.0	4.6	5.5
A	2	10	3.0	3.6	4.1
A	2	15	1.5	2.5	3.2
C	4	5	4.0	3.4	-
C	4	10	4.0	4.7	5.4
D	4	15	3.0	4.5	4.9
D	4	20	3.0	4.1	4.3

Based on the results in Table 6, preliminary design monochromatic wave curves were prepared for non-overtopped seawalls comprising 2 t and 4 t Ecogreen rock bags with 1V:1.5H slope (interlocking, “stretcher bond” placement) as shown in Figures 4 and 5, respectively. The lower line “no damage” is based on the highest wave height which caused no displacement and the upper line “damage initiation” is based on and the smallest wave height which caused displacement. Since the 5 s waves were steepness limited, there is no data point for the “damage initiation” line for this wave period. From these test results, it is apparent that the design monochromatic wave height for both damage lines is inversely proportional to wave period (except for steepness limited 5 s waves).

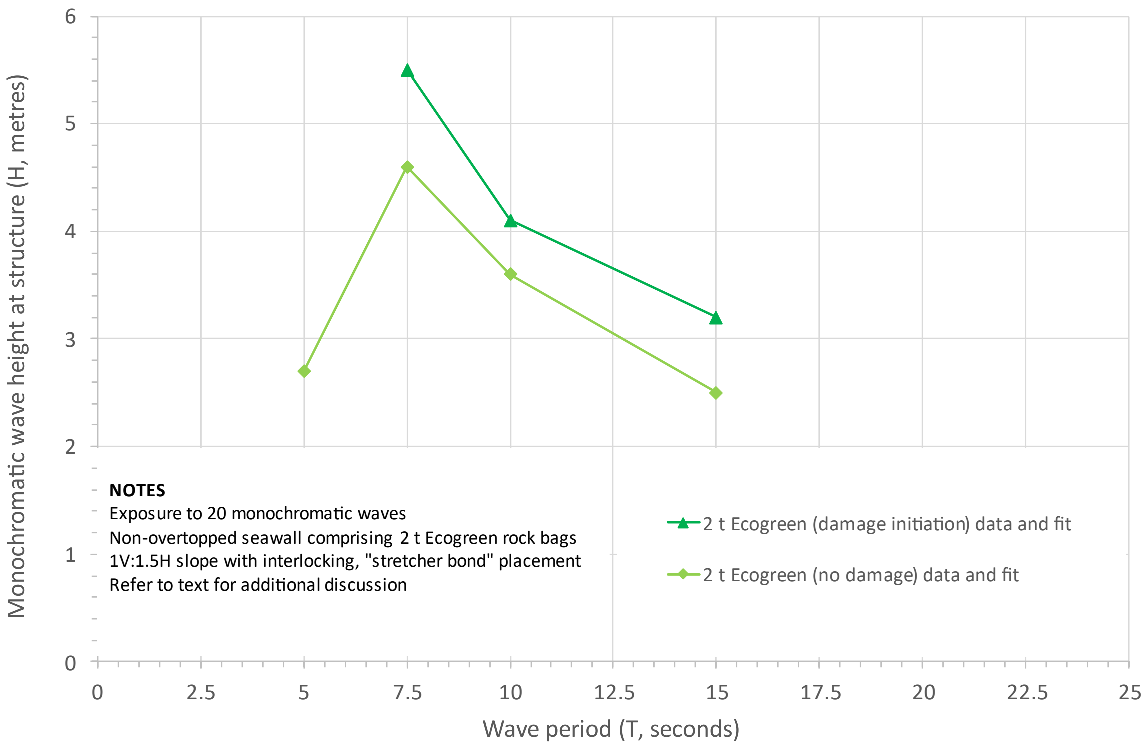
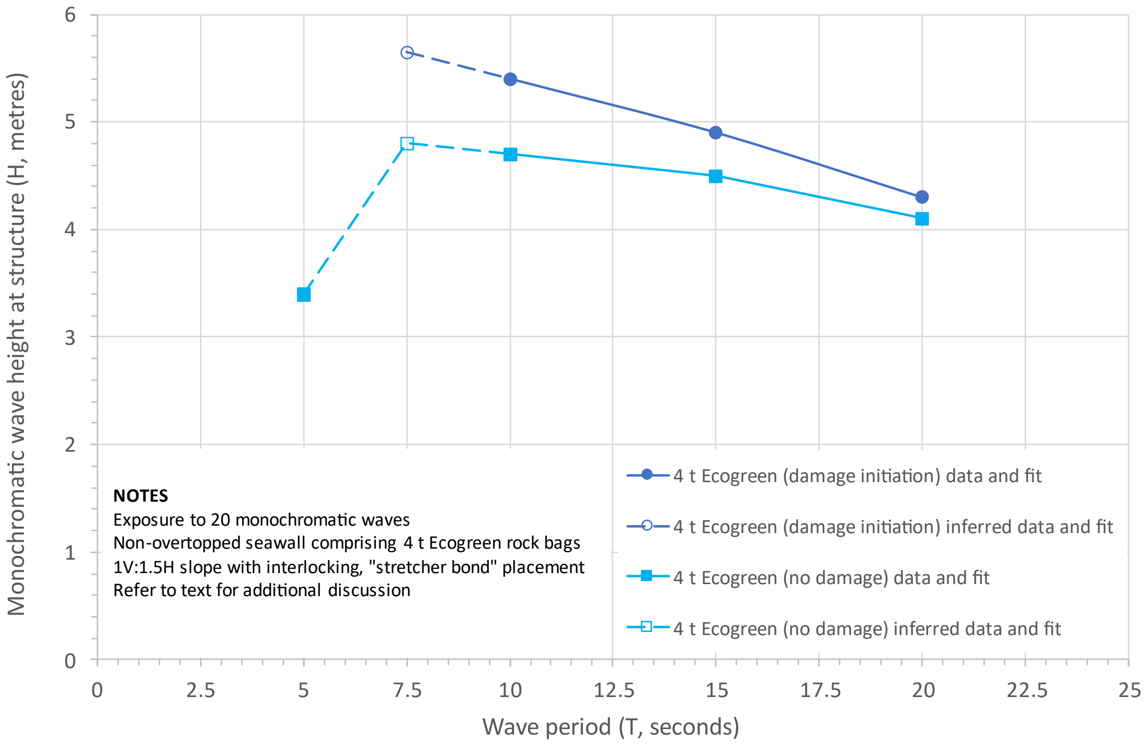


Figure 4. Preliminary design monochromatic wave height for 2 t Ecogreen rock bags.



NOTES
Exposure to 20 monochromatic waves
Non-overtopped seawall comprising 4 t Ecogreen rock bags
1V:1.5H slope with interlocking, "stretcher bond" placement
Refer to text for additional discussion

Figure 5. Preliminary design monochromatic wave height for 4 t Ecogreen rock bags.

Since no tests were undertaken with 7.5 s waves for 4 t Ecogreen rock bags, values in Figure 5 at this wave period were inferred for both the “no damage” and “damage initiation” lines to ensure that they sit above the 2 t Ecogreen values. If additional testing is undertaken in the future for 4 t Ecogreen rock bags with 7.5 s monochromatic waves, damage may not be able to be initiated for this wave period (steepness limited waves).

These monochromatic preliminary design curves should be used with greater caution than the subsequent irregular wave test curves for prototype coastal structure design as the test duration was considerably shorter.

2.3.2 Irregular Wave Tests

A summary of the results from the irregular wave tests on 4 t Ecogreen rock bags regarding rock bag damage (displacement and settlement) and wave runup/overtopping is detailed in Table 7. Damage is expressed as the number of displaced rock bags divided by the total number of rock bags within a reference area from three significant wave heights at the structure above and below the still water level × 100%. A reference area of ± 3 H_S was adopted as almost all displaced rock bags were in this region. Note that the toe elevation for the structures was higher than the water level minus 3 H_S; as such, the reference area effectively extended from the toe up to 3 H_S above the water level. Example photos before and after the test with T_P = 15 s (SWL = 3.0 m MSL) are shown in Figure 6.

Table 7. Test results for irregular wave attack on 4 t Ecogreen rock bags.

Structure Configuration	Rock Bag Mass (t)	Test Conditions at Structure [T_P / SWL / H_S / $H_{10\%}$]	Damage ± 3 H_S (%)	Vertical Settlement (m)	Overtopping Seawall Crest?	Maximum Runup / H_S (-)
B	4	10 / 3.0 / 2.7 / 3.1	0.0	1.5 – 3.5	Yes	>3.4
D	**4**	**10 / 4.0 / 3.4 / 3.9**	**1.8#**	**2.1 – 4.5**	**Yes**	**>3.6**
C	**4**	**15 / 3.0 / 2.8 / 3.3**	**2.8***	**3.3 – 4.7**	**Yes**	**>3.5**
D	**4**	**20 / 1.5 / 1.8 / 2.1**	**5.3**	**0.0**	**No**	**≈6.6**
D	4	20 / 3.0 / 2.4 / 2.9	15.5#	0.1 - 0.3	Yes	>5.5

For these tests, some of the displaced 4 t rock bags included in the damage % were S type units

* For this test, rock bags at the crest which were displaced by wave runup/rundown were excluded from the damage %

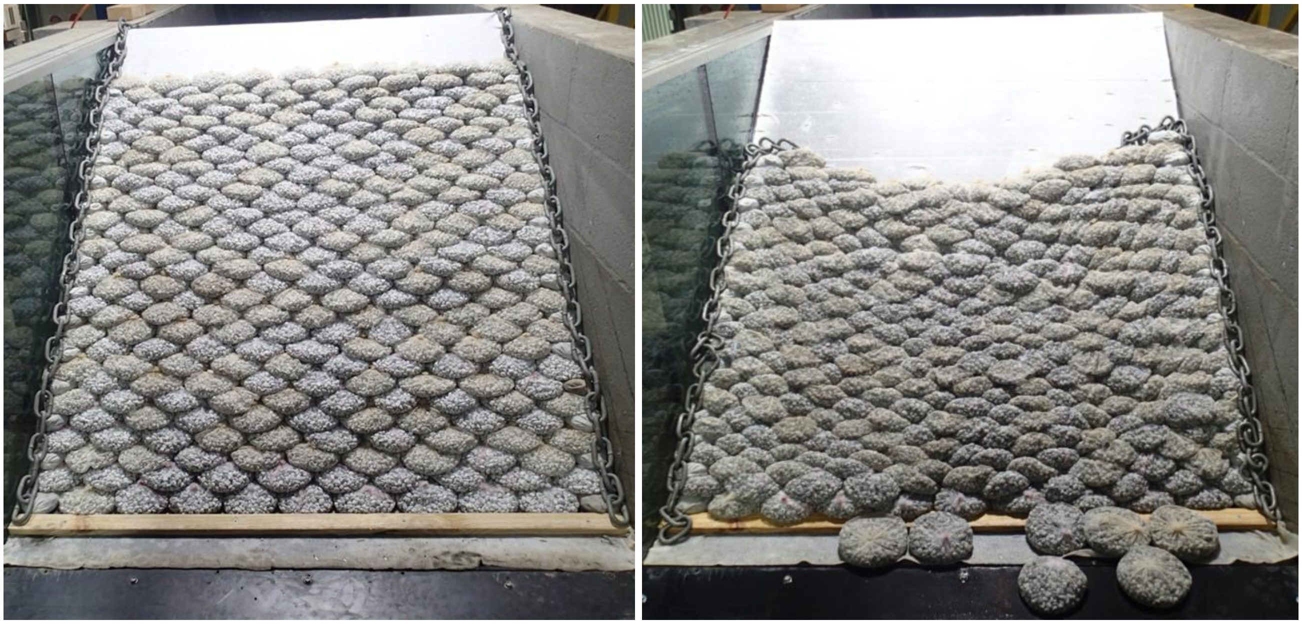


Figure 6. Front view of model 4 t Ecogreen rock bag seawall before (left) and after (right) irregular wave test with 2.8% damage and substantial (3.3 to 4.7 m) vertical settlement (T_P = 15 s, SWL = 3.0 m, H_S = 2.8 m, $H_{10\%}$ = 3.3 m).

For qualitative assessment of rock bag damage (displacement) for design purposes, 2% displacement within the ± 3 H_S reference area was proposed as "initial damage". Based on the three test results shown in bold in Table 7, preliminary design irregular wave curves were inferred for non-overtopped seawalls comprising 4 t Ecogreen rock bags with 1V:1.5H slope (interlocking, "stretcher bond" placement) in terms of H_S (Figure 7) and $H_{10\%}$ (Figure 8). It is acknowledged that the inferred 2% damage line is based on a small range of tests and further irregular wave testing for a wider range of damage results is recommended to develop more comprehensive irregular wave design curves.

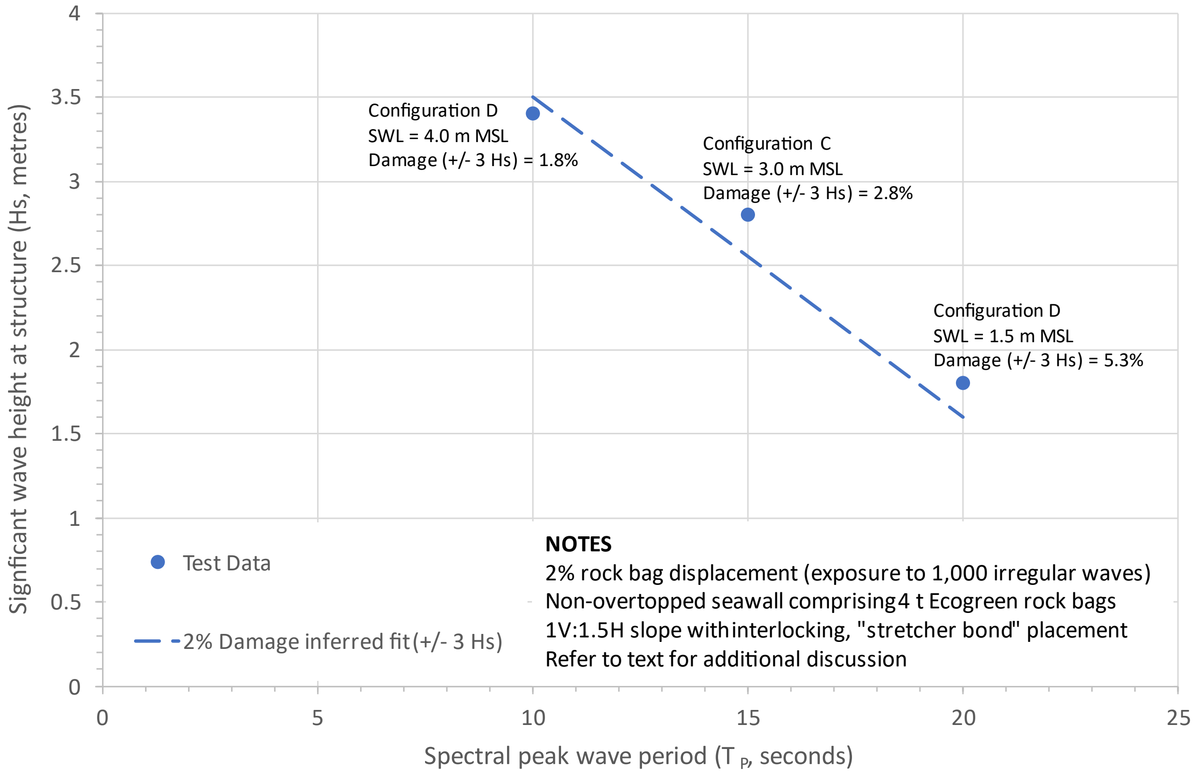


Figure 7. Preliminary design H_S for 2% damage criterion for 4 t Ecogreen rock bags.

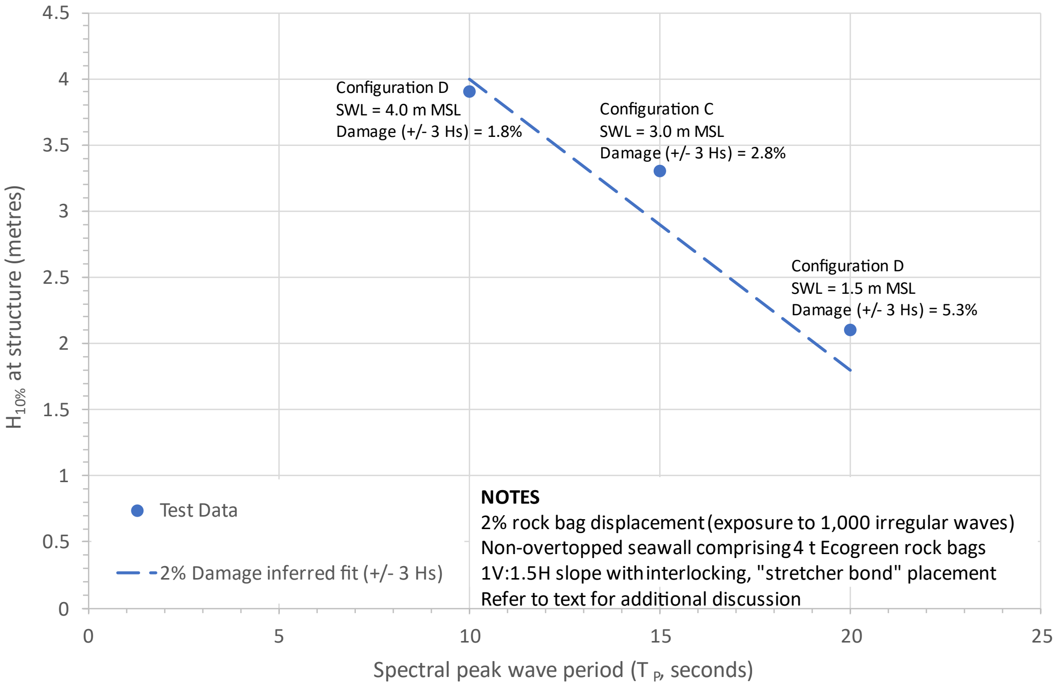


Figure 8. Preliminary design $H_{10\%}$ for 2% damage criterion for 4 t Ecogreen rock bags.

For the 10 s and 15 s T_P tests, uniform vertical settlement (measured at the crest) was quite large; between 1.5 and 4.7 m. However, for T_P of 20 s, vertical settlement at the crest was negligible. Instead, a gap was created in the rock bag matrix via vertical settlement of a lower portion of the structure for both 20 s tests, resulting in increased packing density for those rock bags below the gap. Vertical settlement appeared to primarily occur because the rock bags rotated from their installed horizontal orientation to close to perpendicular to the slope.

Maximum wave runup as a proportion of significant wave height was high. It was greater than 3.4 for tests where the structure was overtopped and was approximately 6.6 for the 20 s T_P test which didn't overtop the seawall.

3 STAGE 2: TESTING WITH SITE-SPECIFIC DESIGN CONDITIONS (STOCKTON BEACH)

3.1 Preamble

Following the conclusion of Stage 1 testing with generic design conditions, Stage 2 testing with site-specific design conditions for an interim seawall design comprising 4 t rock bags at Stockton Beach (Newcastle, Australia) was carried out. While Stage 1 deliberately focused on rock bag damage by direct wave impacts only, it was important in the Stage 2 tests to also examine potential displacement of rock bags at the crest (via overtopping) and the toe (via slip). For succinctness, subsequent discussion of the Stage 2 physical modelling program in this paper is more concise than Stage 1 and focuses on findings which may be relevant for the design of rock bag seawall structures at other locations.

3.2 Physical Model Setup and Results

Testing was undertaken in the same facility and at the same scale as Stage 1 using 4 t Ecogreen rock bags. The same model bathymetry was used without making any physical changes, however, the vertical datum in the model was raised by 0.6 m (prototype) so that the toe of the seawall specific to Stockton Beach was at -0.4 m MSL. The interim seawall has a 5 year design working life and the 18 year average recurrence interval (ARI), equivalent to 5.4% annual exceedance probability (AEP), was selected for both design wave conditions (height, period and direction) and water level conditions (tide plus anomaly). An irregular wave climate was used for the 18 year ARI tests with T_P of 12 s and duration of 3.1 hours (1,000 wave time series). The water level was 1.38 m MSL (1.78 m water depth at the structure). The depth limited wave statistics at the structure toe were H_S of 1.7 m and $H_{10\%}$ of 2.1 m. By comparing these values with the inferred 2% damage lines from Stage 1 (Figures 7 and 8), rock bag damage on the main slope of the Stockton Beach interim seawall was anticipated to be less than 2%.

Two structure configurations were modelled with a structure slope of 1V:1.5H (note that additional Stage 2 tests were undertaken with a steeper structure slope of 1V:1.25H but these have been omitted for brevity). As previously mentioned, the toe elevation was -0.4 m MSL and the elevation of the crest of the plywood backing board (covered with geotextile) was

5.4 m MSL. The difference between the two configurations was crest elevation: one structure was 13 rock bag courses high (5.6 m MSL crest level; Configuration E) and the other 14 courses high (6.1 m MSL crest level; Configuration F). Both configurations comprised a single layer of rock bags, except for the bottom two courses which had two layers.

A summary of the results from the Stage 2 tests is detailed in Table 8. In contrast to Stage 1, damage is expressed as the total number of displaced rock bags with a breakdown provided in brackets by location (crest course, main slope and toe courses). Settlement was again recorded and the mean wave overtopping rate (q) was also measured using a catch tray placed leeward of the seawall crest. Example photos before and after the test on the configuration with 14 rock bag courses high (6.1 m MSL crest level) are shown in Figure 9.

Table 8. Test results for irregular wave attack on Stockton Beach seawall comprising 4 t Ecogreen rock bags.

Structure Configuration	No. of Rock Bag Courses High (-)	Crest Elevation (m MSL)	Test Conditions at Structure [T_P / SWL / H_S / $H_{10\%}$]	No. of Rock Bags Displaced (-)	Vertical Settlement (m)	Mean Wave Overtopping Rate, q (L/s/m)
E	13	5.6	12 / 1.38 / 1.7 / 2.1	5 (crest 1, slope 1, toe 3)	0.1 – 0.5	34.9
F	14	6.1	12 / 1.38 / 1.7 / 2.1	5 (crest 0, slope 0, toe 5)	0.0 – 0.3	26.6

Figure 9. Front view of model Stockton Beach seawall (Configuration F) before (left) and after (right) irregular wave test.

A total of five rock bags were displaced for both configurations; most of these were located in the toe but slip failure did not occur. One crest rock bag was displaced with a lower crest (q of 34.9 L/s/m), but none were displaced for the higher crest (q of 26.6 L/s/m). A single rock bag was displaced from the main part of the structure slope for the test with the lower crest only (Configuration E); this corresponds to 1% damage in this area (consistent with Stage 1 expectations). Finally, vertical settlement was much smaller (0.5 m or less) for the Stage 2 tests compared to the Stage 1 tests.

4 DISCUSSION

4.1 Vertical Settlement

During Stage 1 testing with very high seawall structures (10 s and 15 s T_P), uniform vertical settlement was approximately equivalent to up 10 rock bag heights (4.6 m) for a 30 course high 4 t Ecogreen structure (13.7 m installed height). However, for Stage 2 testing with a more typical seawall height (constrained by typical land levels), uniform vertical settlement was equivalent to only one rock bag height (0.5 m) for a 13 course high 4 t Ecogreen structure (6.0 m installed height). On this basis, settlement is likely a function of the number of rock bag courses in height as each course has voids (as installed) which may contribute to cumulative compaction when exposed to a storm event. The smaller vertical settlement for the Stage 2 tests was also considered to be influenced by the reduced forcing from wave run-down, compared to the Stage 1 tests, due to a substantial proportion of waves overtopping the Stage 2 seawall structures.

It is considered that the use of a rigid plywood backing for the geotextile underlayer in the model may also have contributed to exacerbation of the magnitude of this settlement relative to the expected prototype scenario. Real-world conditions are likely to involve a sand backed geotextile. During construction, the placement of the rock bags against the sand backed geotextile may deform the slope to form a “sawtooth” backing slope with increased resistance to the sliding/rotation mechanism identified in the model, mitigating settlement. Nevertheless, during the design of 4 t Ecogreen rock bag seawalls, consideration should be given to vertical settlement over the structure’s intended design life. For a typical seawall height, this may mean adding an extra course during initial construction, or planning to add an extra course at some point during the structure’s working life (once cumulative compaction is sufficient) to preserve the minimum design crest level.

4.2 Wave Runup and Overtopping

For Stage 1 tests, maximum wave runup as a proportion of significant wave height was high for 4 t Ecogreen rock bags. This is an important consideration for future rock bag seawalls in either establishing the design crest level to prevent wave overtopping, or adopting and managing a permissible amount of wave overtopping during a design event. Based on the results from these tests, it is not possible to estimate the roughness factor of rock bags for input into EurOtop (2018) empirical wave runup and wave overtopping equations. However, in the absence of further wave runup/overtopping test results, it is recommended that rock bag structure designers conservatively adopt a roughness factor of 1.0 (equivalent to a seawall with a smooth, impermeable slope).

Stage 2 test results indicate that 4 t Ecogreen rock bags at the crest of a seawall may withstand a mean wave overtopping rate of approximately 30 L/s/m without being displaced. This q threshold should not be considered universal for all rock bag seawalls, however, as different stability results are possible with other H_S, T_P, water depth and freeboard combinations.

5 CONCLUSIONS

As a result of the two-stage physical modelling program for shallow water seawalls constructed from rock bags, specific results were obtained for a proposed structure at Stockton Beach (Newcastle, Australia) as well as producing generic design information that could be applied at other locations. The design wave height for rock bag damage (displacement) was found to be inversely proportional to wave period. Preliminary design curves were developed for 2 t and 4 t Ecogreen rock bags (non-overtopped seawalls with 1V:1.5H slope and interlocking, “stretcher bond” placement) under monochromatic wave attack for “no damage” and “damage initiation” criteria. Preliminary design curves were also inferred for 4 t Ecogreen rock bags under irregular wave attack for “initial damage” (2% displacement) in terms of H_S and $H_{10\%}$. In addition to displacement, vertical settlement of the rock bags was high for the tall Stage 1 model structures but much smaller for the shorter Stage 2 structures. On this basis, it is recommended that consideration be given to vertical settlement over the intended design life of an Ecogreen rock bag seawall. Since wave runup on Ecogreen rock bags was relatively high, it is also an important consideration for future rock bag seawalls in either establishing the design crest level to prevent wave overtopping, or adopting a permissible amount of wave overtopping during a design event.

The authors encourage future research efforts to build on these preliminary test results for hydraulic stability and wave runup/overtopping for shallow water seawalls constructed from Ecogreen rock bags.

ACKNOWLEDGEMENT

Funding for this modelling investigation was provided by Bluemont and the City of Newcastle. The authors also acknowledge the contribution of several other people to this project, including from within UNSW-WRL and Haskoning.

REFERENCES

Bluemont, 2016. *Rock Bags: Natural protection for the environment*, Brochure/Product Specifications, pp. 1-7.

Coghlan, IR, Lumiatti, G and Carley, JT, 2022. *Kyowa rock bag seawall physical modelling*, WRL Technical Report 2020/42, 11 April.

EurOtop, 2018. *Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application*, Van der Meer, JW, Allsop, NWH, Bruce, T, De Rouck, J, Kortenhaus, A, Pullen, T, Schüttrumpf, H, Troch, P and Zanuttigh, B, www.overtopping-manual.com.

Hasselmann, K, Barnett, TP, Bouws, E, Carlson, H, Cartwright, DE, Enke, K, Ewing, JA, Gienapp, A, Hasselmann, DE, Kruseman, P, Meerburg, A, Muller, P, Olbers, D, Richter, K, Sell, W and Walden, H, 1973. *Measurements of wind-wave growth and swell decay during the Joint North Sea Wave Project (JONSWAP)*, Deutches Hydrographisches Institut, Hamburg.

HR Wallingford, 2012. *Filter Unit Scour Protection: Physical model test data report*, Report no EX6789, Release 3, 10 August.

Hudson RY, Herrmann FA, Sager RA, Whalin RW, Keulegan GH, Chatham CE and Hales LZ, 1979. *Coastal Hydraulic Models*, Special Report No. 5, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

HYDRALAB III, 2007, Wolters, G, Van Gent, MRA, Mühlestein, D, Kirkegaard, J, Allsop, W, Gironella, X, Fortes, J, Capitão, R, Sousa, I, Pinheiro, L, Santos, J, Hamm, L & Bonthoux, L. *Guidelines for Physical Model Testing of Breakwaters: Rubble Mound Breakwaters*, Deliverable NA3.1-2, Final Report (Version 1.3), European Commission.

Mansard, EPD and Funke, ER, 1980. The Measurement of Incident and Reflected Spectra Using a Least Squares Method, *17th International Conference on Coastal Engineering*, Sydney, Australia, pp. 154-172.

Messiter, D, Miller, BM, Simpson, JH and Lumiatti, G, 2019. Super Cruise Vessel vs Rock Bags, *Australasian Coasts and Ports 2019 Conference: Future directions from 40°S and beyond*, Hobart, Australia, pp. 882-888.

Mizutani, N, Shimabukuroi, H and Koyama, H, 2007. Study on a Wave Force Acting on a Filter Unit on Sloping Breakwater and Its Stability, *17th International Ocean and Polar Engineering Conference*, Lisbon, Portugal, pp. 2575–2581.

Nielsen, AF, Lord, DB and Poulos, HG, 1992. Dune Stability Considerations for Building Foundations, *Australian Civil Engineering Transactions*, The Institution of Engineers, Australia, Volume CE34, Number 2, pp. 167–174.