Wharves At The Edge Of The World: Physical Modelling Of Zero Maintenance Breakwaters In Remote Locations

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WHARVES AT THE EDGE OF THE WORLD: PHYSICAL MODELLING OF ZERO MAINTENANCE BREAKWATERS IN REMOTE LOCATIONS

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ABSTRACT

During the design process for a breakwater, construction and maintenance considerations usually play a major part in determining the final geometry of the structure. This is particularly the case in remote locations with limited availability of materials, plant and machinery combined with the tyranny of distance. This paper presents the design development to upgrade existing wharves at two sites within, the Chatham Islands archipelago. These islands are populated by only 600 people, and the wharves on the two inhabited islands, Chatham and Pitt, provide a lifeline for their communities.

The physical modelling of both wharves led to a significant reduction of design risks, and constructability improvements. Empirical techniques were found to have mixed results for estimating the stability of concrete primary armour (Xbloc® and Hanbar units) and rock toe armour. Secondary armour stability tests with the breakwater in an "under construction" state also provided insights for construction planning.

On Pitt Island, wave overtopping processes were very three-dimensional such that they could only be robustly estimated using the physical model. On the head of both wharves, it was necessary to extend the crown wall normal to the long axis of the breakwater to improve overtopping and armour stability on the leeward side.

KEWORDS: Breakwater, Xbloc®, Hanbar, armour stability, wave overtopping, Chatham Islands.

1 INTRODUCTION

The Chatham Islands archipelago is located approximately 800 km to the east of mainland New Zealand, at latitude 44°S. The wharves on the two inhabited islands, Chatham and Pitt (Figure 1), provide a lifeline for their respective communities through the provision of everyday goods and export earnings. Significant upgrades were necessary at both wharves to increase their resilience to large storm events, and to improve usability of the facilities and functionality of wharf operations. While both wharves are in very remote locations and in relatively close proximity to each other, their design requirements are quite distinct, particularly due to differences in local wave climate and water depth.

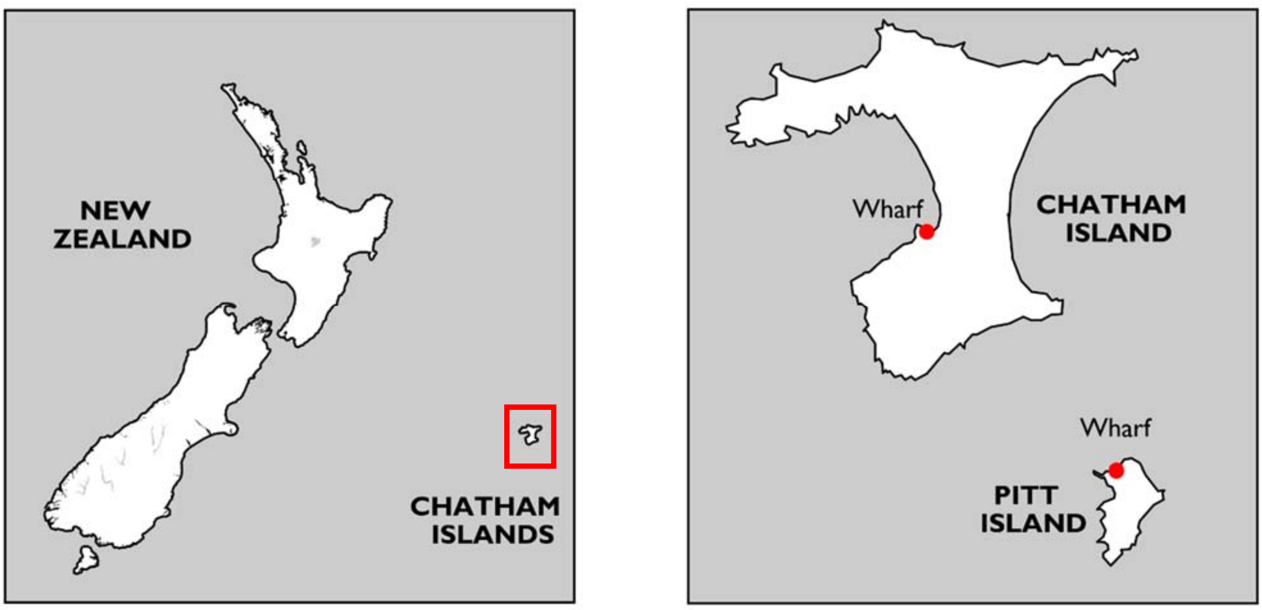

Figure 1. Locations.

The Memorial Park Alliance (MPA) was engaged by the New Zealand central government to prepare "zero maintenance" breakwater designs for upgrades to the wharves on Chatham and Pitt Islands. Following the development of preliminary designs based on empirical techniques by MPA, the UNSW Water Research Laboratory (WRL) undertook physical modelling to refine the structural geometry, to ensure that the final designs were hydraulically stable (without unnecessary conservatism) and reduce the ongoing risk of the design criteria being exceeded during the working life of the wharves. The physical modelling programs were integrated within the design process, rather than undertaken as a final design validation. As such, a range of structural arrangements were tested, aspects of which were incorporated into the final design geometries. Following the conclusion of the physical modelling program, the authors from Tonkin + Taylor (one of the MPA partners) and WRL worked collaboratively to review the assumptions in the preliminary empirical design to learn lessons for future projects.

The dominant wave generation source at both sites is large westerly swells, produced by strong westerly winds from the "Roaring Forties" weather systems in the South Pacific and Southern Oceans (Foster *et al.*, 2017). Other wave sources include ex-tropical weather systems descending from the north and onshore sea breezes. Swell waves dominate the local wave climate year-round. Tides are classified as semi-diurnal, microtidal with mean spring tide ranges of 0.83 m (Chatham) and 1.0 m (Pitt).

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

2 CHATHAM ISLAND WHARF

2.1 Preamble

The existing wharf on Chatham Island was constructed in 1980 and consisted of a reinforced concrete, suspended deck jetty and tee head supported by open piles. In 2014 it was deemed to be at risk of structural failure and significant upgrades were necessary. Wave motions at the existing wharf meant that it was unsafe for operations almost 50% of the time. Cargo handling operations at the port associated with the existing wharf, were also inefficient.

For the upgraded wharf, 9,500 m^2 of land was to be reclaimed to improve the reliability and usability of port operations (Figure 2). A 180 m long breakwater was necessary to protect the reclamation from erosion, wave overtopping and reduce the wave climate in its lee to increase the number of safe berthing days at the upgraded wharf.

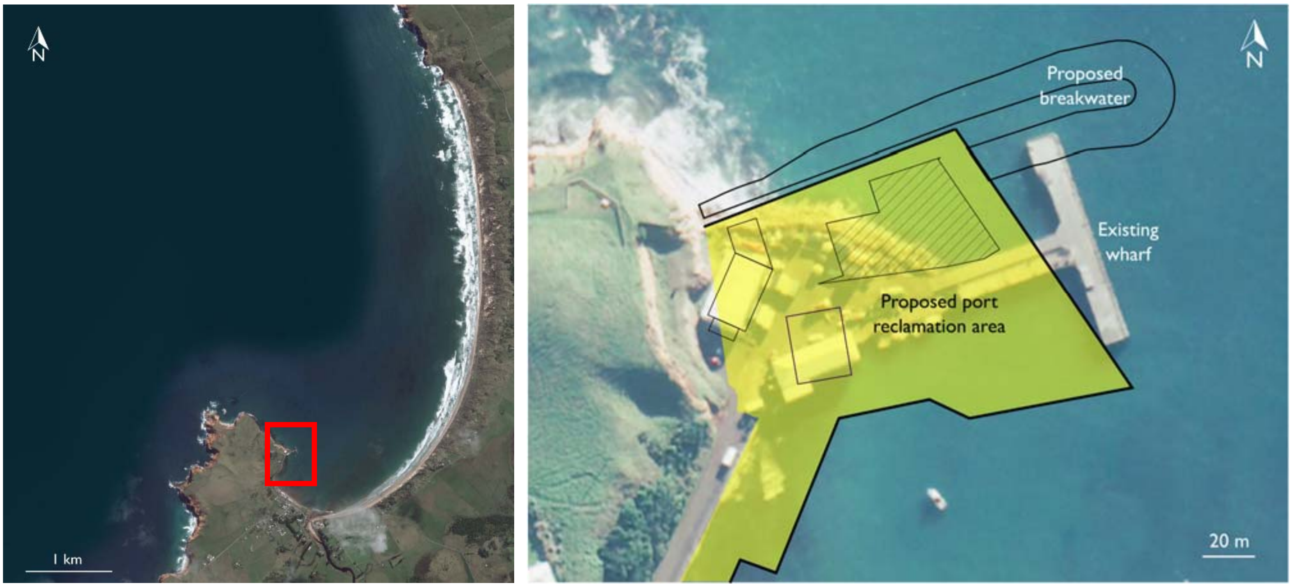

Figure 2. Existing Chatham Island Wharf and proposed breakwater and reclamation area.

2.2 Design Conditions

2.2.1 Planning Horizon

A design life of 50 years was adopted for the breakwater protecting the upgraded wharf. Due to the very remote location, "zero maintenance" (or as close as practically possible) was adopted as the basis of the design. Based on these requirements, the 1,000 year average recurrence interval (ARI), equivalent to 0.1% annual exceedance probability (AEP), was selected for both wave conditions (height, period and direction) and water level conditions (tide plus anomaly). Accordingly, the probability that the design event will be exceeded over the design life is approximately 5%.

2.2.2 Offshore Wave Conditions

Westerly swell waves are the dominant source of wave energy on the Chatham Island coast. However, due to the sheltered location of the wharf (in the lee of a large headland; See Figure 1), the majority of direct swell wave energy propagates past it. Accordingly, the design wave conditions for the upgraded wharf are only refracted and diffracted swell waves.

At a bed elevation of -6.5 m MSL approximately 80 m seaward of the existing wharf, the central estimate for the 1,000 year ARI significant wave height (H_S) was 2.24 m based on extreme value analysis (EVA) of a 35 year (1979-2013) numerical wave hindcast (MPA, 2016). Cognisant of the need for "zero maintenance", this value was increased by 20% to 2.7 m because the EVA confidence interval was wide. A peak spectral wave period (T_P) of 13.9 s and mean wave direction of 335°TN (at the same bed elevation) were adopted for the design.

2.2.3 Still Water Levels

The present day (2015) 1,000 year ARI extreme still water level (excluding wave setup) adopted for the design was 1.05 m MSL based on a 14 year (2001-2014) water level record at the site (MPA, 2016). Allowing for 0.5 m of sea level rise (SLR) over its design life, the design still water level at the end of the life (2065) of the upgraded wharf is 1.55 m MSL. Representative, present day low tide (-0.5 m MSL) and high tide (0.5 m MSL) levels were also used for investigation of toe armour stability and "under construction" secondary armour stability tests.

2.3 Design Objectives and Constraints

Qualitatively, no primary or toe armour damage was to occur during the design event. Quantitatively, guidance for single layer concrete armour with up to 3% rocking units and 0.5% displaced units, and up to 0.5% damage for toe rock armour was adopted to correlate to the qualitative "no damage" criteria (Muttray and Reedijk, 2008).

In addition to the goal of minimal maintenance, an acceptable design mean overtopping rate of 200 L/s/m was also adopted to prevent damage to concrete pavement in the lee of the breakwater during the design event. The crest level of the filter rock was to be at least 2.5 m MSL (approximately Mean High Water Springs tidal level plus 2 m freeboard) to allow for a safe working platform for plant during construction. This constraint influenced the minimum finished crest elevation for the primary armour.

Basalt rock (density = 2,800 kg/m^3) is available from quarries on Chatham Island, but masses for armour rock are typically less than 400 kg due to local shear plane limitations.

2.4 Preliminary Design

2.4.1 Primary and Secondary Armour Sizing

Due to the lack of suitable size local rock and very high transport costs to Chatham Island, Xbloc® concrete armour units cast on-site were adopted as the primary armour for the breakwater. The required Xbloc® volumes (V) on the trunk and roundhead of the breakwater was estimated to be 0.59 and 0.74 m^3 respectively, using the design guidance provided by DMC (2014) in Equation (1). The mass densities of concrete (ρ_c = 2,400 kg/m^3) and seawater (ρ_w= 1,030 kg/m^3), and two correction factors, C_1 (value 1.5 for deep water) and C_2 (value 1.0 for trunk, 1.25 for roundhead), were used as input parameters to this equation.

$$V = \left[\frac{H_S}{2.77\times\left(\frac{\rho_c-\rho_w}{\rho_w}\right)}\right]^3 \times C_1 \times C_2 \tag{1}$$

The smallest commonly produced Xbloc® unit which has a volume of 0.75 m^3. Accordingly, this size Xbloc® unit (mass 1.8 t) with a structure slope of 4H:3V (thickness 1.3 m), overlying two layers of 120 kg locally sourced secondary armour rocks (thickness 0.8 m), was adopted for preliminary design. It is acknowledged that the adopted armour size on the trunk is relatively conservative but smaller units are not advised by the licence holder.

The formula of Van der Meer (Equation 5.137 from CIRA, 2007) was also used to estimate the threshold wave height at which the 120 kg secondary armour would be displaced if exposed to direct wave attack during construction. For a typical ambient swell peak wave period of 13.0 s, initiation of damage to the secondary armour is expected for H_S = 0.94 m (for P = 0.4, S_d = 2, N = 7,500, tanα = 0.75, T_m = 10.8 s).

An additional benefit of selecting Xbloc® primary armour was that the required secondary armour mass (1/15th of the Xbloc® mass) was small enough to be sourced from quarries on Chatham Island. For equivalent hydraulic stability, larger rock primary armour (5-10 t) or double layer concrete primary units would have required secondary armour rocks with mass (typically 1/10th of the primary) exceeding that available locally.

A filter layer consisting of rocks with median mass of 12 kg (thickness 0.8 m) was also included between the secondary armour and the structure core.

2.4.2 Toe Armour Sizing

The required size for rock armour in the breakwater toe (d_{n50}) was estimated using the design guidance provided in CIRIA (2007) in Equation (2). The mass density of rock armour (ρ_a = 2,800 kg/m^3) and a damage number (N_{od} = 0.5, as recommended by DMC, 2014) were used as input parameters to this equation. For the critical low tide (-0.5 m MSL) case, calculations were undertaken on the breakwater trunk with bed elevation of -5 m MSL. For a toe armour crest level of -3.4 m MSL in this section (thickness 1.6 m), the water depth on top of the toe structure (h_t = 2.9 m) and the water depth in front of the toe structure (h = 4.5 m), the required nominal rock diameter is 0.45 m (m_{50} = 251 kg).

$$\frac{H_S}{\left(\frac{\rho_a-\rho_w}{\rho_w}\right)\times d_{n50}} = \left(2 + 6.2\left(\frac{h_t}{h}\right)^{2.7}\right) N_{od}^{0.15} \tag{2}$$

On this basis, locally sourced 300 kg rock (thickness 1.6 m, approximately 3×d_{n50}) was adopted for the preliminary toe armour design. While design guidance suggests a minimum thickness of 2×d_{n50} (DMC, 2014), an increased thickness was adopted due to the objective of minimal future maintenance.

2.4.3 Crest Level and Wave Overtopping

Since construction constraints set the crest level of the filter rock to 2.5 m MSL, the minimum finished crest elevation for the Xbloc® units was 4.6 m MSL (secondary armour thickness 0.8 m, Xbloc® thickness 1.3 m). The mean wave overtopping rate (q) for this crest level was estimated to be 8.6 L/s/m during the design event using the deterministic Equation (3) from the EurOtop (2008) "Overtopping Manual".

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \times \exp(-2.3\frac{R_c}{H_{m0}\gamma_f\gamma_\beta}) \tag{3}$$

In this equation, H_S was considered equivalent to H_{m0} since deep water conditions exist. Crest freeboard (R_c = 3.05 m), acceleration due to gravity (g = 9.81 m/s^2), an Xbloc® roughness factor (γ_f = 0.45) and a wave obliquity factor (γ_β = 1) were also as input parameters to this equation. The crest level was not increased further as this preliminary value was below the acceptable design mean overtopping rate.

A mass concrete crown wall was also included to provide backing to the Xbloc® units at the crest of the breakwater and reduce overtopping flows onto the reclamation. In the preliminary design, the crest level of the crown wall was equivalent to the crest level of the Xbloc® units (4.6 m MSL) and had a height of 1.6 m.

2.4.4 Preliminary Design for Physical Modelling

Figure 3 shows a cross-section of the preliminary design of the breakwater roundhead which was subsequently physically modelled. A paved road was included in the lee of the crown wall to facilitate vehicle access. The sandy seabed was proposed to be excavated by 1.6 m below the toe armour to incorporate secondary armour and filter layers built down to -6.6 m MSL. The preliminary design of the trunk (seaward of the reclamation area), was identical to Figure 2 except that it was a revetment-type structure (i.e. the paved road and leeward armour were not included).

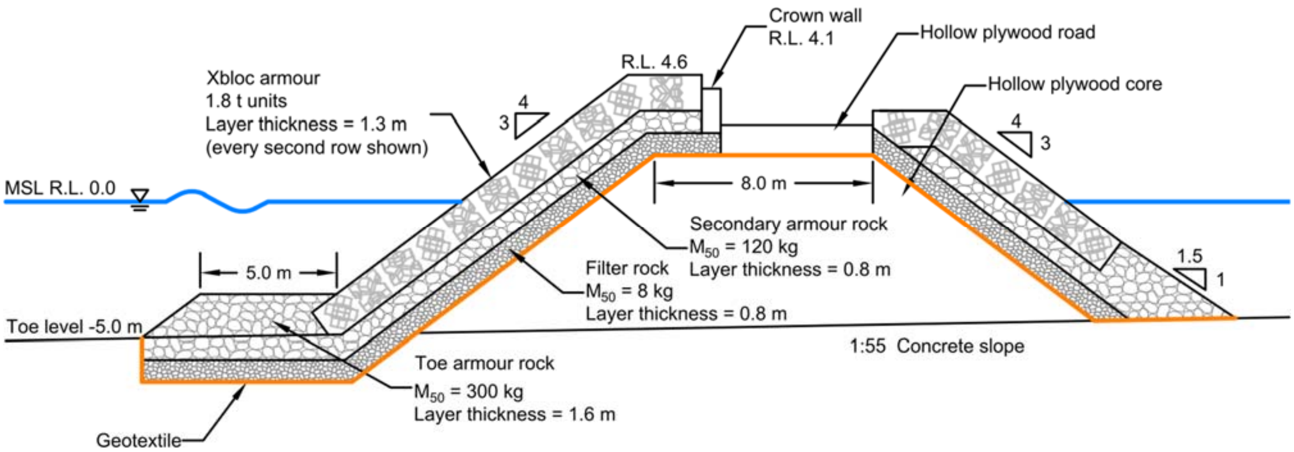

Figure 3. Cross-section of preliminary breakwater roundhead design

2.5 Physical Modelling Setup

2.5.1 Testing Facility and Model Selection

Two-dimensional (2D) and quasi three-dimensional (Q3D) testing was undertaken in a flume measuring approximately 32 m in length, 3.0 m in width and 1.3 m in depth. The wave generator is a hydraulic, piston-type paddle.

2D tests were undertaken to assess stability of Xbloc® units and toe armour, and wave overtopping on the breakwater trunk and head under perpendicular wave attack. These were undertaken using the centre of three × 1 m wide mini flumes built internally within the wider 3.0 m flume, restricting the model breakwater crest length to 1 m.

Q3D tests used the full flume width and were undertaken to assess 3D aspects of stability of Xbloc® units and toe armour on the breakwater roundhead under oblique wave attack (but with simplified 2D bathymetric profile). The ability of the breakwater to reduce the wave climate in its lee was assessed separately using numerical wave modelling rather than a full 3D physical model. The stability of the secondary armour under wave attack during construction was also assessed in the Q3D physical model.

2.5.2 Design and Scaling

Model scaling was based on geometric similarity with an undistorted scale of 1:33.2 being used for all tests. Selection of the length ratio was primarily based on fitting a suitable length of bathymetry (4 wavelengths, or 430 m, seaward of the test structures) within the length of the flume (Howe *et al.*, 2015). The scaling relationship between length and time was determined by Froudian similitude. The model Xbloc® unit density (2, 330 kg/m^3) was reduced to preserve the prototype ratio between Xbloc® units and water in the model (998 kg/m^3). The model rock armour density

(2, 650 kg/m^3) was less than that required to preserve the prototype ratio between rock armour and water (2,726 kg/m^3). Accordingly, rock armour stability results were conservative in the model. The selected scale was large enough to ensure that flow through primary and secondary layers remained turbulent, eliminating viscous scale effects on armour stability.

2.5.3 Bathymetry

The model bathymetry was constructed from aggregate/gravel fill overlain with concrete capping with the following characteristics:

- intersected structure at -5.0 m MSL;
- 1V:55H slope from -5.0 m MSL to -11.3 m MSL; and
- 1V:5H slope from -11.3 m MSL to -26.8 m MSL (where false floor slope intersected the permanent flume floor).

While the bathymetry intersected the structure at -5.0 m MSL (the bottom of the toe armour layer) for the 2D tests, the model structure incorporating the secondary armour and filter layers was built down to -6.6 m MSL (to simulate placement within an excavated sandy seabed).

2.5.4 Breakwater Core Material

The prototype breakwater core material will be a sub-300 mm, coarse, clean, granular fill material. For model design purposes, this core material was considered to be largely impermeable. In the 2D model, the batter slope for the breakwater was constructed with an impermeable hollow timber frame. This frame was covered with geotextile material to separate the core material and the filter layer, and to provide an appropriate interface friction. The modelling approach for the core material was expected to yield conservative stability results for the primary armour since the model has lower permeability relative to the prototype (higher reflections off the core material in the model will lead to higher seaward loads on the armour layers). In contrast, for the Q3D model, the model core was constructed from fine gravel to more readily achieve the 3D geometry of the core material on the breakwater roundhead.

2.5.5 Roadway Pavement and Crown Wall

The roadway pavement on the breakwater crest was constructed from plywood and the mass concrete crown wall was built with two hollow aluminium box sections.

2.6 Physical Model Data Collection and Analysis

2.6.1 Wave Sequence Preparation and Generation

The 1,000 year ARI deep water wave sequence to be reproduced by the wave generator was assumed as a JONSWAP spectrum with a peak enhancement factor of 3.3. The same wave sequence (with adjustment of flume water level) was used with for both the future 1,000 year ARI water level (with SLR) and the present day low tide level.

2.6.2 Wave Data

Waves that reflect from model structures towards the wave generator were not actively absorbed by the wave generator. Instead, test wave climates were first calibrated both where the design wave climate was defined (-6.5 m MSL) and at the breakwater head (-5 m MSL), without a model structure in place. Reflections from the far end of the wave flume (without a model structure in place) were minimised using low gradient, dissipative materials. 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 wave climates. The same calibrated test wave climates were then reproduced with the model structure in place. The wave conditions measured at -6.5 m MSL during the 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. Each test included 1,000 incident waves; depending on each test condition, this related to a prototype storm duration of 2.9 to 3.1 hours.

2.6.3 Overtopping Data

Throughout 2D tests when overtopping was measured on the breakwater trunk, the volume of water overtopping the crest of the breakwater (4.6 m MSL) was collected using a catch tray placed directly leeward of the crown wall crest for a portion of the mini-flume width. This setup allowed the measurement of mean overtopping discharge, q.

2.6.4 Concrete Armour Layer Damage Assessment

The damage percentage for Xbloc® units was defined in terms of both the number of units rocking (oscillation without displacement) and displaced as a proportion of the total number of units in the primary armour layer. Prior to armour stability testing of each model breakwater section for the first time, a settlement test was run with the wave climate at 50% strength for a prototype duration of approximately 1 hour to allow the Xbloc® units to “bed” into place (i.e. find an equilibrium position).

2.6.5 Rock Toe Armour Layer Damage Assessment

Damage was defined as the total number of rocks which were displaced a distance greater than the d_{n50}. The damage percentage for toe armour rock was determined by relating the number of rocks displaced as a proportion of the total number of rocks in the toe armour layer.

2.7 Physical Modelling Program

A total of six separate tests were undertaken as part of the 2D modelling. Five tests were on the breakwater trunk section; one for the design event and four toe armour stability tests with the same wave climate at the present day low tide level. The toe stability tests involved testing two rock masses (300 kg and 120 kg) with two thicknesses (1.6 and 3.2 m). The two thicknesses represented excavated and non-excavated toe arrangements, respectively. One test for the design event on the breakwater roundhead section was carried out to examine leeward Xbloc® stability.

Five Q3D tests were undertaken on the breakwater roundhead. 2 tests with the design event and 1 toe stability test on a test structure with a non-excavated toe comprising 120 kg rock. 2 additional toe tests were carried with to simulate non-excavated and excavated toe arrangements with 300 kg rock.

Four Q3D tests with gradually increasing H_S were carried out on a structure in an "under construction" state (Figure 4).

Figure 4. Secondary Armour Stability During Construction: Before (Left) and After (Right) Testing. Arrows indicate filter layer visible at the breakwater head and the end of the trunk.

2.8 Physical Modelling Results

2.8.1 Wave Overtopping (2D)

During the design event, the mean wave overtopping rate was measured to be 113.4 L/s/m. While this is less than the acceptable design mean overtopping rate of 200 L/s/m, it is an order of magnitude larger than the empirical estimate of 8.6 L/s/m. Subsequent use of the Artificial Neural Network (ANN) from EurOtop (2008) indicated that the rate measured in the physical model was between the ANN percentiles of 25% (105.3 L/s/m) and 50% (216.3 L/s/m).

2.8.2 Armour Stability (2D)

No Xbloc® units were displaced from the seaward side of the breakwater during the design event. For the breakwater roundhead, one Xbloc® unit was displaced from the top of the leeward side of the breakwater crest. This resulted in a total of 0.2% displaced Xbloc® units, which complies with the design criteria of less than 0.5% displaced units. During the same tests, up to 1.4% of Xbloc® units were observed to be rocking, which complies with the design criteria of less than 3% rocking units.

For the 300 kg toe armour (1.6 m thickness, crest level -3.4 m MSL), 0.2% of rocks were displaced into the Xbloc® matrix during the design event at present day low tide, which complies with the design criteria of less than 0.5% damage. To optimise the design further, additional tests were undertaken with a higher 300 kg toe (3.2 m thickness, crest level -1.8 m MSL to avoid the need for seabed excavation) and lighter 120 kg toe armour (1.6 m thickness, crest level -3.4 m MSL) to increase the yield from the local quarries. However, rock toe armour damage for these cases exceeded the design criteria, with damage of 2 and 3%, respectively. A final toe stability test with a higher 120 kg toe (3.2 m thickness, crest level -1.8 m MSL) resulted in 8% damage.

2.8.3 Armour Stability (Q3D)

For the breakwater roundhead under oblique wave attack during the design event, 0.5% of Xbloc® units (total 5) were displaced and an additional 0.6% were rocking. Four of the Xbloc® units were displaced from the crest of the breakwater head at the end of the paved roadway. The fifth Xbloc® unit was displaced from the third bottom row at around 135° from normal (a typically vulnerable location on breakwater roundheads).

During another test with the design event at present day low tide, 120 kg toe armour was displaced from the same region (135° from normal). The dislodgment of this toe armour contributed to the displacement of a sixth Xbloc® unit from the bottom row (0.6% cumulative Xbloc® displacement).

Following the toe stability test, the design event was repeated without re-building the model breakwater. During this test, the Xbloc® matrix progressively failed on the leeward side of the breakwater head. Clearly, the 120 kg toe armour was not suitable for supporting the Xbloc® units on the breakwater head.

The model breakwater was rebuilt with 300 kg toe armour (1.6 m thickness) and the design event with present day low tide repeated. However, the toe armour was extensively damaged on the leeward side of the breakwater head again leading to partial failure of the Xbloc® matrix.

Finally, the model structure was rebuilt with 300 kg toe armour (1.6 m thickness), but tested with the design wave condition with the water level raised by 1 m above the present day low tide to simulate a deeper toe level. Minor displacement of the toe armour occurred but no Xbloc® units were displaced.

2.8.4 Secondary Armour Stability During Construction (Q3D)

While the Xbloc® primary armour was verified to withstand the design event, the secondary armour layer will withstand a lower threshold wave height before damage begins to occur. Using a typical ambient swell peak wave period of 13.0 s and a high tide water level (0.5 m MSL), an "under construction" breakwater model was built and tested with progressively increasing significant wave height from 0.55 to 1.20 m. Damage to the 120 kg secondary armour layer began at H_S = 1.0 m (initiation of movement) and significant reshaping occurred at H_S = 1.2 m with the underlying filter layer becoming exposed around the breakwater head (Figure 4). The extent of wave overtopping halfway across the 2.5 m MSL secondary armour crest level for H_S = 1.2 m can also be seen in Figure 4 (right).

2.9 Discussion

As a result of the physical modelling program for the Chatham Island wharf, no changes were necessary for the crest level. However, the wave overtopping was higher than estimated during preliminary design. This confirmed the need to pave the land behind the breakwater and locate any buildings well back from the crown wall.

In general, the physical model confirmed that the 1.8 t Xbloc® units were suitable for the design event. Based on displacement observed in the Q3D tests, the crown wall should be extended along the end of the roadway to provide support for the adjacent Xbloc® units on the breakwater crest. Unravelling of the Xbloc® matrix on two occasions, highlighted the critical importance of toe armour stability.

Toe armour consisting of 120 kg rock does not have adequate stability for the breakwater head; 300 kg rock should be used. This is important not only for supporting the Xbloc® matrix, but to avoid fracturing the Xbloc® units with impacts from armour rocks thrown up from the toe. Also, the sandy seabed should be excavated to 1.6 m below the existing level, so that the crest level of the toe armour rock does not exceed -3.4 m MSL around the breakwater head.

The construction engineer on the project witnessed the Q3D secondary armour stability tests with the breakwater in an "under construction" state. Given that low swell waves would persist throughout construction, observing these tests provided insights for construction planning and repair methodologies in a very remote location. The threshold for initiation of movement of the 120 kg secondary armour, H_S = 1.0 m, matched well with the preliminary estimate. This significant wave height is exceeded approximately every two weeks for a duration of 1-3 days at the wharf location.

3 PITT ISLAND WHARF

3.1 Preamble

The existing wharf on Pitt Island was constructed in 2014 and consists of a concrete decked, vertical sheet pile structure with a 1.5 m high concrete crown wall. The seaward side of the wharf is protected by an ad-hoc rock rubble revetment that consisted of widely graded limestone rock (with many rocks only 0.2-0.4 m in diameter) on a flat slope overlying a shallow rocky reef (Figure 5). Shortly after construction of the ad-hoc revetment in March 2015, Cyclone Pam mobilised rocks in the revetment and caused damage to the wharf, highlighting its lack of resilience to large storm events. Wave motions on the leeward side of the wharf, due to waves refracting and diffracting around the end of the breakwater and overtopping the wharf, were also often unsafe for functional operations while vessels were at berth.

Figure 5. Existing Pitt Island Wharf and proposed breakwater upgrade (red shaded area in right panel).

For the upgraded wharf, the revetment armouring will be upgraded and also extended to include a breakwater, to reduce wave overtopping volumes during wharf operation, prevent existing rock armour from being displaced over the wharf structure during extreme conditions and reduce the wave climate at the wharf to the greatest extent that is technically and economically feasible. To reduce wave overtopping at the end of the wharf, the crown wall will be extended normal to the existing wall on the seaward side of the wharf.

3.2 Design Conditions

3.2.1 Planning Horizon

A design life of 50 years with "zero maintenance" was again adopted for the breakwater protecting the upgraded wharf. However, more frequent ARIs were selected for the design wave (100 year) and water level (50 year) conditions respectively, indicating the acceptability of a slightly higher level of risk (if it fails, the consequences are less significant).

3.2.2 Offshore Wave Conditions

Pitt Island is exposed to the same dominant westerly swell waves as Chatham Island. However, the wharf on Pitt Island is much more exposed to direct wave energy; approximately double that at Chatham Island Wharf.

At a bed elevation of -10.5 m MSL, approximately 280 m seaward of the existing wharf, the central estimate for the 100 year ARI H_S was 4.8 m based on EVA of a 38 year (1979-2016) numerical wave hindcast (MPA, 2017). The hindcast indicated that swell wave energy at this location has two predominant mean wave directions, 305°TN and 355°TN which were adopted for the design process. A T_P of 13.0 s was also adopted for both wave directions.

3.2.3 Still Water Levels

The present day (2015) 50 year ARI extreme still water level (excluding wave setup) adopted for the design was 0.95 m MSL based on the water level gauge on Chatham Island (no gauge is available on Pitt Island). Allowing for 0.5 m of SLR over its design life, the design still water level at the end of the life (2065) of the upgraded wharf is 1.45 m. Note that since the toe of the existing breakwater is located in shallow water (-0.5 to -2.5 m MSL), wave breaking offshore is expected to further elevate extreme water levels at the wharf through wave setup.

A representative, present day high tide (0.5 m MSL) level was used for tests of wharf operational limits.

3.3 Design Objectives and Constraints

Qualitatively, no primary armour damage was to occur during the design event. Quantitatively, displacement of up to 2% of primary armour units was generally adopted for the design. Due to the limited construction plant available on the island after completion of the upgrade project, ensuring that during storm conditions no armour units were displaced to a location that would impede navigation and berthing to the wharf was a critical consideration, as there was no viable way to relocate displaced units.

An acceptable design mean overtopping rate of 200 L/s/m was again adopted to prevent damage to the concrete wharf deck in the lee of the breakwater during the design event. When the wharf may be operating (defined as $H_S < 1.5$ m), the mean overtopping rate for the upgraded wharf was to be reduced (improved) compared to the existing wharf.

3.4 Preliminary Design

3.4.1 Primary Armour Sizing

Due to the lack of suitable size local rock on Pitt Island, Hanbar concrete armour units (a three-legged unit developed in the late 1970s by an Australian state government agency as discussed in Foster, 1985) cast off-site were adopted as the primary armour for the upgraded breakwater. Although requiring a larger quantity of concrete, double layer Hanbar units were preferred over single layer Xbloc® units due to reduced placement complexity and because materials savings on the relatively small structure would likely be offset by the substantial work required to prepare the foundation and underlayer if Xbloc® units were to be used. For damage of up to 2% on double layer Hanbars, a damage coefficient (K_d) of 5 was adopted based on physical modelling results presented in Blacka et al. (2005). This guidance was based on a combination of results from tests using monochromatic and irregular waves on both breakwater trunk and roundhead sections.

The required Hanbar mass (M) on the trunk and roundhead of the breakwater was estimated to be 2 and 4 t, respectively, using the formula of Hudson (Equation 5.133 from CIRA, 2007). The mass densities of concrete ($\rho_c = 2{,}400$ kg/m^3) and seawater ($\rho_w = 1{,}030$ kg/m^3) and structure slope ($\cot\alpha = 2$ for trunk, 3 for roundhead) were used as input parameters to this equation. The wave height at the toe of the structure (H) was considered equivalent to the depth limited significant wave height ($H_S = 2.7$ m for trunk, 3.7 m for roundhead). These wave heights were estimated based on a conservative breaker depth index (ratio of H_S to water depth, d_b) of 0.8 and bed elevations of -1.9 m MSL (depth 3.35 m) and -3.2 m MSL (depth 4.65 m) on the trunk and roundhead, respectively.

The existing limestone ($\rho_a = 2{,}200$ kg/m^3) rock rubble will be excavated, rocks smaller than 0.3 m discarded and the remainder replaced as secondary armour underneath the Hanbar units.

3.4.2 Crest Level and Wave Overtopping

Based on observations of overtopping of the existing wharf during previous storm events, wave overtopping of the wharf is extremely complicated due to three-dimensional effects of nearshore rocky reefs on incident waves, the geometry of the structure, and the varying elevation of the wharf and crown wall. As such, deterministic overtopping estimates were not prepared; instead, the physical model would be used to define the overtopping rate on the existing wharf.

The levels of the existing concrete deck and crown wall are 1.8 m and 3.3 m MSL, respectively, and were not changed in the upgrade. The finished crest elevation for the Hanbar units was 3.9 m MSL, with double layer thickness of approximately 1.8 m for the trunk (2 t) and 2.3 m for the roundhead (4 t). Note that the crest level of the existing rock rubble breakwater (which will be excavated) is approximately 2.4 m MSL.

3.4.3 Preliminary Design for Physical Modelling

Figure 6 shows a cross-section of the preliminary design of the breakwater trunk, which was subsequently physically modelled. The preliminary design of the breakwater roundhead did not have a crown wall and concrete desk but was entirely composed of 4 t Hanbar units with structure slopes of 1V:3H (seaward)and 1V:1.5H (leeward). The breakwater planform remained largely unchanged from the existing structure; separate numerical modelling indicated that a sufficient extension of the breakwater offshore to reduce wave motions on the leeward side of the wharf would exceed the available project budget.

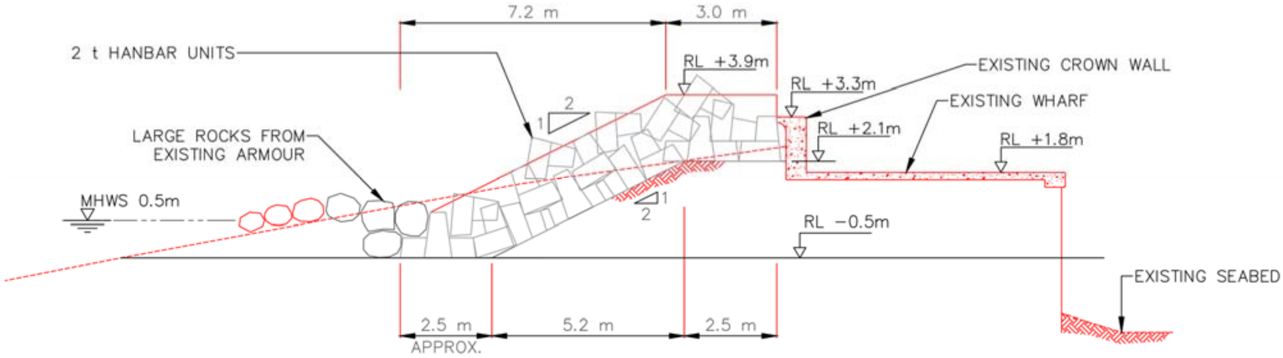

Figure 6. Cross-section of preliminary trunk design

3.5 Physical Modelling Setup

3.5.1 Testing Facility and Model Selection

Three-dimensional (3D) testing was undertaken a wave basin measuring approximately 29 m in length, 17 m in width and 0.8 m in depth. The wave generator is an 8 m wide electro-mechanical, segmented piston-type paddle. 3D tests were undertaken to assess stability of Hanbar units, wave overtopping and wave penetration on the leeward side of the wharf from the two predominant mean wave directions.

3.5.2 Design and Scaling

Model scaling was based on geometric similarity with an undistorted scale of 1:40 being used for all the tests. Selection of the length ratio was primarily based on wave basin area and the availability of existing model Hanbar armour units in specific sizes (Modra et al., 2018). The model Hanbar unit density (2, 330 kg/m^3) was reduced to preserve the prototype ratio between Hanbar units and water in the model. The selected scale was large enough to ensure that the flow through the primary armour layer remained turbulent, eliminating viscous scale effects on armour stability.

3.5.3 Bathymetry

The model bathymetry was constructed using a template with concrete capping technique. Profiles were taken at regular intervals from the scaled bathymetry and laser cut in plywood. Templates were typically spaced at 1 m in the model, but additional templates were added in areas of the reef immediately seaward of the wharf where additional detail was required. The templates were filled with aggregate and capped with a thin layer of concrete. The bathymetry extended at least 320 m (4.4 wavelengths) seaward of the test structure to approximately -15.5 m MSL.

3.5.4 Reef

The areas of nearshore reef seaward and around the head of the wharf were constructed separately from the main bathymetry. The roughness of the reef and shallow bathymetric features required a more detailed construction technique. For this section of seabed, coarse rock aggregate was placed throughout the reef area to visually match aerial photos (Figure 7). The aggregate was then covered in grout to set the final reef profile in place. Following grouting, sand-cement was brushed across the reef to ensure that the roughness of the reef was lower than the prototype. During this process particular attention was paid to reproducing the features of a rock platform adjacent the head of the wharf, which would later form the foundation for placement of the new breakwater head armour.

3.5.5 Wharf Structure

The wharf, including the crown wall, was constructed from expanded PVC sheeting.

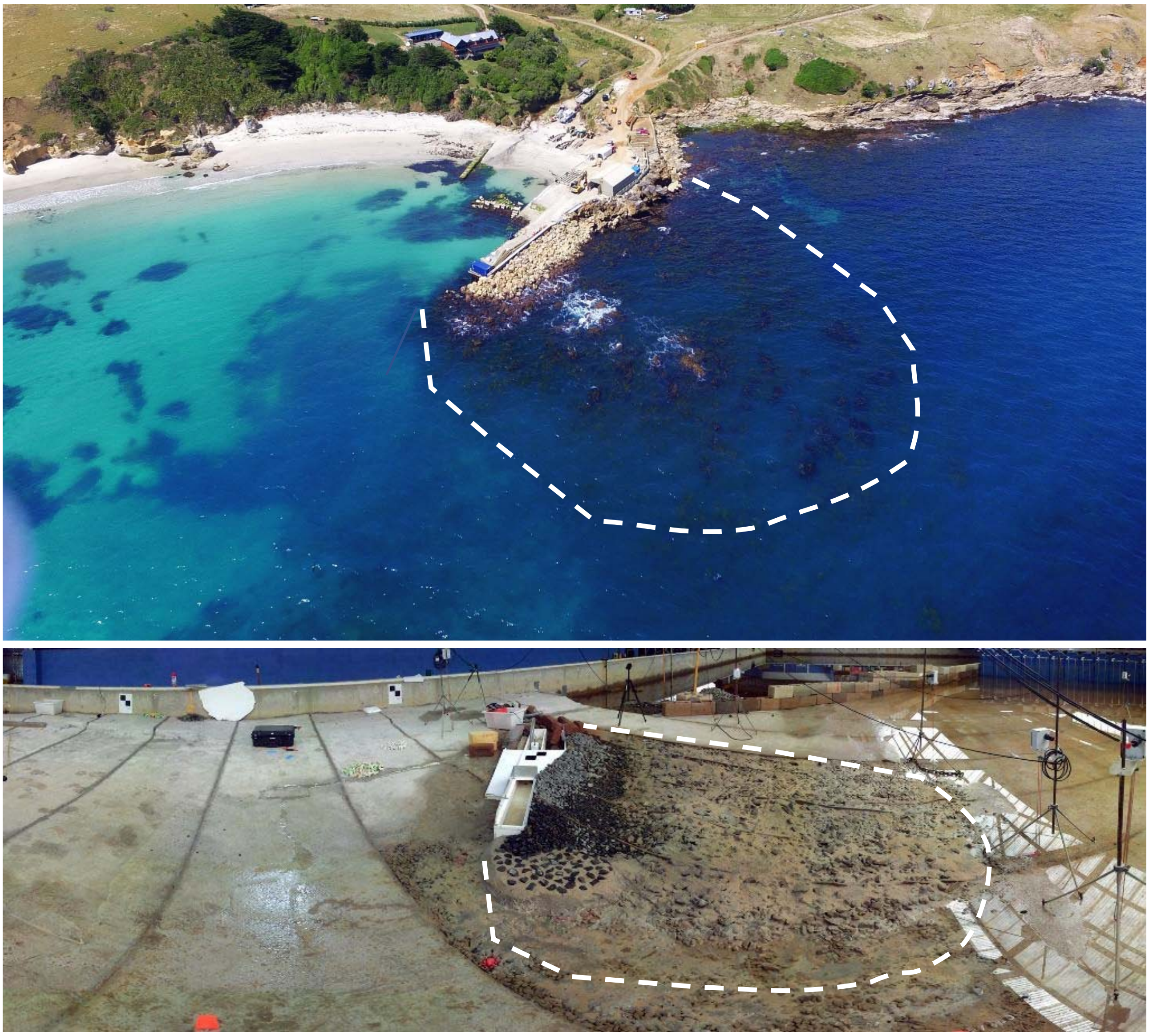

Figure 7. Nearshore reef seaward and around the head of the existing wharf in prototype (Top) and in the model (Bottom).

3.6 Physical Model Data Collection and Analysis

3.6.1 Wave Sequence Preparation and Generation

The 100 year ARI deep water wave sequence to be reproduced by the wave generator was assumed as JONSWAP spectrum with a peak enhancement factor of 3.3. The same sequence (with adjustment of basin water level) was used with both the future 50 year ARI still water level (with SLR) and the present day high tide level (0.5 m MSL).

3.6.2 Wave Data

Test wave climates were calibrated for both wave directions (305°TN and 355°TN) using a three probe array where the design wave climate was defined (-10.5 m MSL), without a model structure in place. The same calibrated test wave climates were then reproduced with the model structure in place. Wave conditions were also measured using single wave probes immediately behind the wharf (to measure wave penetration) and near the wave break point just off the reef. Each test included at least 1,000 incident waves.

3.6.3 Overtopping Data

Throughout the tests, the volume of overtopping water arriving on the concrete deck of the wharf (1.8 m MSL) was collected using an overtopping tray integrated into the end of the wharf structure. The measurement of mean overtopping discharge was normalized to the estimated length of crest which was typically being overtopped.

3.6.4 Concrete Armour Layer Damage Assessment

The damage percentage for Hanbar units was defined in terms of the number of units displaced as a proportion of the total number of units in the primary armour layer.

3.7 Physical Modelling Program

A total of 12 separate tests were undertaken as part of the 3D modelling. Four wave overtopping tests were carried out on the existing wharf (Figure 8, Left) for two offshore wave heights (1.5 and 4.8 m) and two wave directions (305°TN and 355°TN). The preliminary design (2t and 4 t Hanbars) was tested for the same wave conditions from 305°TN. These two tests were repeated with 6.5 t Hanbars (minimal planform) replacing 4 t units on the head. Finally, four tests (2 wave heights and 2 directions) were undertaken on a structure composed of 6.5 t Hanbars with the maximum achievable planform (Figure 8, Right). Hanbars were placed in a random orientation on the 4 t and 6.5 t (minimal planform) structures, but the 6.5 t (maximum planform) were purposely placed with care to maximise stability. Stability and wave overtopping were measured for each of the model Hanbar structures.

Figure 8. Model structures: Existing wharf (Left) and upgraded wharf with 6.5 t Hanbars on a broadened head (Right).

3.8 Physical Modelling Results

3.8.1 Wave Overtopping

Measured mean wave overtopping rates are summarised in Table 1. Wave overtopping for the existing wharf was greatest for waves from 355°TN. However, once the crown wall was extended in conjunction with the addition of Hanbar units, wave overtopping was higher for waves from 305°TN.

During the design event, the mean wave overtopping rate for all model wharf structures was less than the acceptable design mean overtopping rate of 200 L/s/m. The addition of Hanbar units and extension of the crown wall reduced the mean overtopping rate during this event by at least 70%.

For the limit of wharf operations (H_S = 1.5 m and present day high tide), there was no improvement in wave overtopping for the 4 t Hanbar structure compared with the status quo. Relative to the existing wharf, the design with a broadened breakwater head with 6.5 t Hanbars provided the greatest wave overtopping reduction during wharf operations.

Table 1. Mean wave overtopping measurements.

Structure Description	Mean Overtopping Rate, q (L/s/m)			
	305°TN Wave Direction Limit of Wharf Operations Present Day	305°TN Wave Direction 100 year ARI Waves Future with SLR	355°TN Wave Direction Limit of Wharf Operations Present Day	355°TN Wave Direction 100 year ARI Waves Future with SLR
Existing Wharf	0.054	79.648	1.105	85.050
4 t Head Hanbars	0.029	24.137		
6.5 t Head Hanbars	0.065	23.097		
6.5 t Head Hanbars (Broadened)	0.008	19.970	0.000	7.212

3.8.2 Armour Stability

A summary of measured damage to the Hanbar units is presented in Table 2. Percentage damage to the 2 t Hanbar units on the trunk is represented as cumulative (total overall damage) as these units were not repaired when changes were made to armouring on the roundhead. Percentage damage for the 4 t and 6.5 t Hanbar units on the roundhead is the damage induced per test. With reference to the design criteria of damage not exceeding 2% under design conditions, the 2 t Hanbar units on the trunk were found to be suitable. For the roundhead, damage to 4 t Hanbar units exceeded the design criteria but was acceptable for both configurations with 6.5 t units.

Table 2. Summary of primary armour damage measurements.

Structure Description	Wave Direction (°TN)	Trunk Hanbar Cumulative % Damage	Roundhead Hanbar Per Test % Damage
4 t Head Hanbars	305	1	8
6.5 t Head Hanbars	305	2	2
6.5 t Head Hanbars (Max Planform)	305	2	0
	355	2	1

3.8.3 Wave Penetration

Comparison of wave measurements immediately behind the wharf for the existing and upgraded wharf structures, indicated a negligible reduction in wave penetration for all cases.

3.9 Discussion

One of the most unique aspects of the Pitt Island wharf 3D physical model, was the replication of its complex bathymetry and reef. This was necessary to capture the interaction of waves with the natural rocky reef features, and for the detailed designer (witnessing tests and participating in model construction) to integrate the armouring design and placement such that the structure was complemented by the natural features wherever practical. This aspect of the modelling was a critical component in the structure design, as the modelling highlighted the difficulty in ensuring that no displaced armour units would impede the approach navigation channel for the wharf following storm events. Constructability limitations meant that careful interfacing of armour units with the underlying rock platform was a requirement in order to meet this design objective.

The physical modelling program for the Pitt Island wharf quantified the complex 3D wave overtopping volumes on the existing wharf. During the design event, damage to the concrete deck from wave overtopping is not expected to occur. At the limit of wharf operations, a broadened round head with 6.5 t Hanbar units provided the greatest improvement in pedestrian safety. The physical model demonstrated that the preliminary design with 4 t Hanbar units on the roundhead was insufficient to withstand the design event, however, heavier 6.5 t Hanbars were stable when carefully interfaced with the underlying rock reef. This demonstrates the limitations of the present empirical formulation to estimate Hanbar mass for roundheads, particularly because the measured H_S near the breakwater head (3.5 m) was less than that assumed in the preliminary design (3.7 m). Use of 2 t Hanbar units on the breakwater trunk was confirmed as suitable.

4 CONCLUSIONS

To prepare breakwater designs in very remote locations, a delicate balance must be struck between the requirement to minimise maintenance (ongoing costs) and the risk of over-design (initial cost).

As a result of the 2D, Q3D and 3D physical modelling for the Chatham Islands wharves, several significant risks associated with the preliminary empirical designs were identified and addressed. The outcomes from the Q3D and 3D tests particularly underscored the importance of examining armour stability and crown wall overtopping on breakwater roundheads under oblique wave attack prior to design finalisation. The modelling also assisted with breakwater construction planning including placement requirements for Xbloc® and Hanbar units and managing damage to secondary armour from storm events during construction. Use of physical modelling early in the design process allowed rapid changes to made without disrupting parallel aspects of the wharf upgrade projects.

The authors encourage future research efforts to focus on modifying the Hudson equation to provide conservative estimates of required Hanbar mass on breakwater roundheads.

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