Combined Numerical and Physical Modelling of Waves for Ōpōtiki Harbour Entrance Design

Eddie Beetham[1], Tom Shand[1], Grant Pearce[1], Jonathan Clark[1], Patrick Knook[1], Francois Flocard[2] and Lance Partner[1]
[1]Tonkin + Taylor Ltd, Auckland, New Zealand; ebeetham@tonkintaylor.co.nz
[2]Water Research Laboratory, School of Civil and Environmental Engineering, UNSW Sydney

Abstract
The Ōpōtiki Harbour Development Project involves stabilising the entrance of the Waioeka River to allow reliable and safe access for maritime activity. This project is the first major river training works to be designed in New Zealand in over 100 years and includes twin 400 m long training wall breakwaters, dredging of a navigable channel into the harbour, and closing the natural river mouth. Accurate definition of wave height reaching the structure is a key design parameter for armour sizing, setting crest elevation and determining wave penetration into the harbour. To model wave processes for the design, a high-resolution numerical wave model was required to resolve nearshore transformation, refraction, diffraction, and reflection off the structure. The fully non-linear Boussinesq model Funwave-TVD was used to for this work, in conjunction with physical modelling in the wave basin with WRL.

This paper discusses how numerical and physical modelling methods were used in a complementary and iterative manner to inform and test the design. Reflection was a key consideration during the modelling work. Reflection and any resulting convergence needed to be accounted for within the breakwater channel, however, amplification from reflection radiating out to the open sea needed removing to optimise the unit sizing. Wave reflection in the numerical model was assessed using a range of linear and directional spectral methods, with limited success. Improved handling of reflection for the design objective was achieved by repeating simulations with and without the breakwater structures. Reflection off the structures was controlled in the numerical model using a local friction on the breakwater face that achieved a reflection coefficient of 0.3-0.4 to match physical modelling observations. Physical modelling results were also used to validate and calibrate the numerical model. A scaled version of the final design was tested in a 3D physical model for confirmation of stability.

Keywords: Wave modelling, coastal structures, Funwave, reflection.

1. Introduction

The Ōpōtiki harbour mouth is a highly dynamic environment, with channel migration and shifting sand bars preventing safe maritime navigation. The goal of the Ōpōtiki Harbour Development Project is to train the harbour mouth and dredge a channel that allows reliable access that will enable new industries in offshore aquaculture. The harbour mouth improvement design includes twin training wall breakwaters separated by a 120 m wide channel. The breakwaters will be positioned east of the present-day river mouth, which will be closed with beach fill as part of the design.

The breakwater training walls extend 300 m offshore of the shoreline, to a depth of -4 m relative to mean sea level (MSL). Design of the breakwaters in the dynamic and wave exposed setting presents a significant coastal engineering challenge for New Zealand, with no similar structure being designed or constructed in the last 100 years. The breakwater design utilises concrete cast Hanbar armour units for protecting the core, and rock armour for protecting the toe. More detail on the overall breakwater design is presented in a complementary paper [1]. The breakwater orientation was chosen to mirror the natural orientation of the river mouth as it reaches the coast, and to avoid sharp changes in direction of the river where it impacts on the breakwaters and the increased scour effects that would result.

The focus of this paper is the application of a fully nonlinear Boussinesq wave model to inform design wave height and water level around the breakwaters The project required an optimal design in the context of a 500-year return period wave height and sea level 1 m above the present day 1% annual exceedance probability storm tide level. Important considerations in the wave modelling scope were to:

- Resolve wave transformation in the nearshore.
- Resolve surf-zone processes and feedbacks such as wave setup and wave driven currents (rips and alongshore).
- Realistically represent wave reflection off the breakwater structures and resulting wave convergence in the channel.

2. Field setting

Ōpōtiki township is in the Eastern Bay of Plenty on the North Island of New Zealand. The town is located at the confluence two rivers (the Orata and Waioeka) that converge 1.5 km from the coast before flowing out to sea. Coastal and fluvial processes interact at the river mouth to create a dynamic and ever-changing environment. A shifting nearshore delta exists at present, and a spit extends

from the eastern shoreline and directs the river to flow out on a northwest angle (Figure 1).
The river training works will significantly modify the harbour entrance. The lower panel of Figure 1 shows the layout of the breakwaters, mouth closure and dredge channel with associated changes to the elevation contours. Adjusted contours also include how the nearshore ebb tide bar is expected to smooth out following construction.

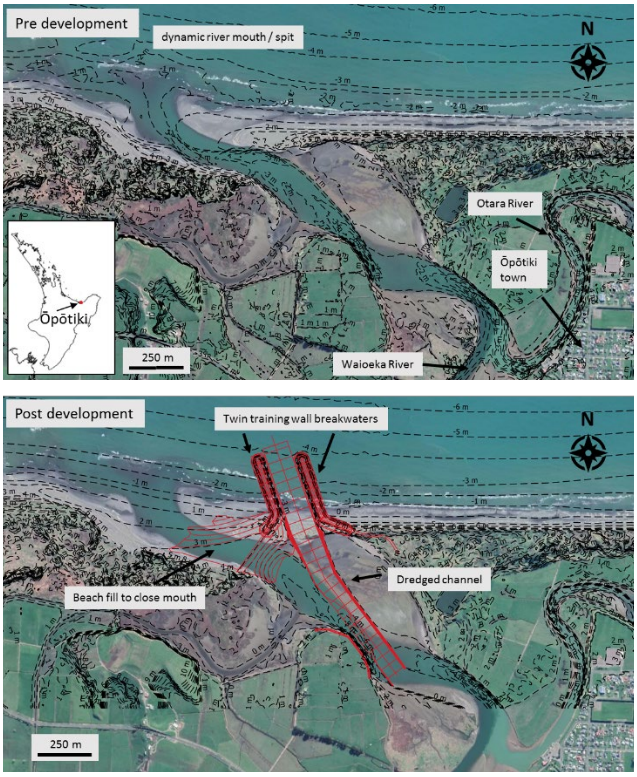

Figure 1: Site overview showing the Ōpōtiki river mouth at present (pre-development) and with the design works to control the river entrance.

2.1 Metocean conditions

The coast at Ōpōtiki is north facing and is exposed to a combination of distant swell waves and regionally wind driven waves. Hindcast modelling by MetOcean [5] identified a 100-year return period wave height at the -20 m depth contour of 4.26 m, and a 500-year wave height of 6.2 m. The extreme wave climate is associated with wave periods of 10 – 12 s [5]. Wave direction at the -20 m contour is variable within a window 20° degrees either side of north. MSL at Ōpōtiki is 0.14 m, with respect to MVD-53 (RL) and the spring tidal range is 1.7 m [6]. Mean high water spring (MHWS) is 0.94 m RL and the highest astronomical tide is 1.14 m RL. Storm surge, due to wind effects and low atmospheric pressure can raise water level above astronomic tide levels. When considering astronomical tides, medium term sea-level variability and storm surge, the present day 1% AEP water level at Ōpōtiki is 1.63 m RL.

3. Methods

3.1 Outline of modelling approach

Two numerical modelling approaches were utilised to understand wave processes at Ōpōtiki and physical modelling in the wave basin at WRL was used as to calibrate model and test a scaled version of the full design. This process is conceptualised in Figure 2 to show the iterative process between different numerical and physical models.

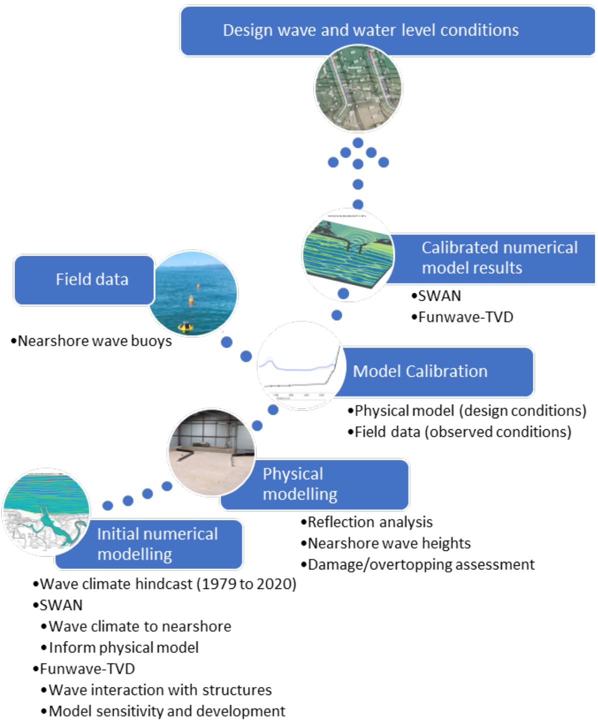

Figure 2: Outline of the modelling process used to define design wave heights and water levels.

The first step was to understand wave transformation from the -20 m depth contour to the harbour mouth using a third-generation spectral wave model, Simulating Waves in the Nearshore (SWAN). The SWAN model was primarily used to propagate the design wave height from the wave hindcast location through the nearshore, using different design water levels. This was important for understanding any water level controls on wave processes at the physical model boundary (-15.5 m).

Figure 3: Physical model of wave transformation at WRL with the channel and not breakwaters.

The next step was simulating wave transformation processes in a wave basin at Water Research Laboratory, University of New South Wales (WRL) to obtain data for calibrating the Boussinesq model. The physical modelling undertaken for model calibration was undertaken at a scale of 1:40.5 and included the Ōpōtiki bathymetry and channel, but not the breakwater structures (Figure 3). Breakwater structures were only included in the physical model when armour unit sizing was confirmed from the numerical model outputs.

Physical model results of wave height around the nearshore and structures were used to calibrate the numerical model using different combinations of wave height at water level. This was used to identify suitable variables for bed friction, breakwater friction for reflection, and different approaches for representing wave breaking. Calibration simulations in the numerical model were undertaken using a like-for-like domain that included a battered slope with an artificially steep transition from -6m RL to – 15 m RL (due to space constraints in the wave basin) and no breakwaters (refer to Figure 5 for the wave basin bathymetry used in the numerical model). The incident wave signal from the physical model was also used as an input condition in the numerical model, where the offshore incident signal was filtered to remove reflection.

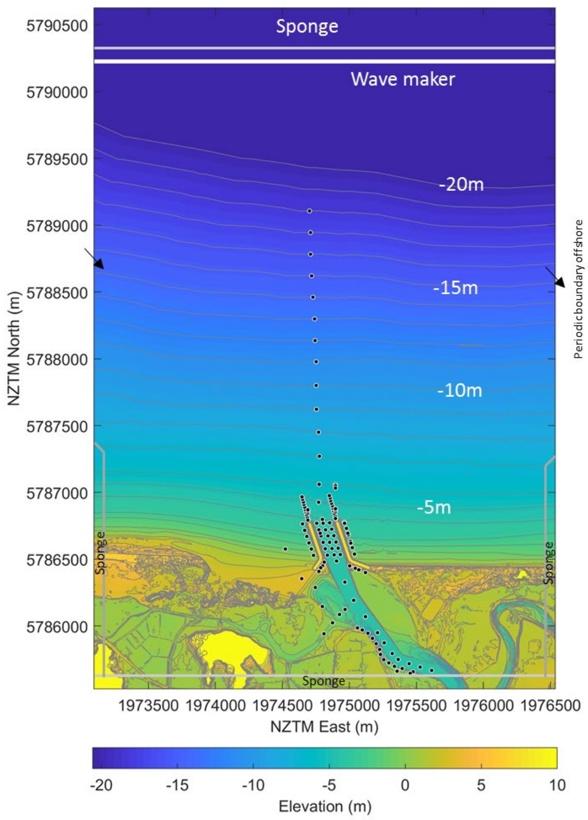

Figure 4: Model domain used in the Funwave-GPU model.

The physical model was constrained to single direction waves and an offshore depth of -15 m (at prototype scale). Therefore, once calibrated, the numerical model was expanded to extend further offshore and alongshore for representing different wave angles with directional spreading (using the 2D JONSWAP boundary condition). This larger domain is presented in Figure 4 and was used to simulate design conditions. The final step was to include a scaled design of the model of the breakwaters in the physical model at WRL to test for damage and stability.

3.2 Funwave-TVD

Funwave-TVD is a fully nonlinear Boussinesq model that resolves the free-surface motion of individual waves, and how these waves interact with each other and the nearshore bathymetry [7]. TVD refers to the Total Variation Diminishing implementation that was developed to improve handling of wave breaking and shoreline interaction from the original Funwave model [3]. Surf-zone processes, such as wave-averaged rips and alongshore currents are resolved, in addition to mean water level changed due to wave setup and set-down. These processes have a dynamic and instant feedback on the motions and behaviour of individual waves. A shock-capturing method is implemented to represent wave breaking and ensure surf-zone and shoreline stability [7]. Wave breaking can be represented 'natively' using the default shock-capturing method or using a more traditional eddy viscosity method. Funwave-TVD is an industry standard wave model that has been comprehensively benchmarked by the developers and by the US Army Corps of Engineers, including application in beach and harbour environments. The model can be run in one horizontal dimension (1D profile), or in in two horizontal dimensions where velocity is depth averaged (2DH). The spatial resolution recommended by developers for resolving wind and swell frequency waves motions is a cell size of Δx and Δy = 2 m. Funwave-TVD is optimised for parallel computing with MPI (message passing interface) and 2D simulations for this project were initially undertaken using 16 CPU cores with equal computational split between *x* and *y* domain directions. Run times on this hardware are in in the order of 12 – 24 hours of clock time for a 40-minute simulation on a domain with 5.8 million cells and a time-stepping CLF condition of 0.5. At the field scale of interest for this project, a lower CLF condition (0.2) was found to be more stable but resulted in run times exceeding 48 hours. This limitation was resolved by switching to the recently released and benchmarked GPU implementation of Funwave-TVD [8], which reduced run times to a manageable 14 hours on available hardware. Model behaviour and outputs were tested to be consistent between GPU and CPU implementation of Funwave-TVD. For this work, the initial calibration work was undertaken using the CPU implementation before switching to the GPU model, where additional calibration and testing was undertaken. Full scale design scenarios were simulated using the GPU model with a 6GB NVIDIA GeForce GTX 1060.

3.3 Data processing

Funwave-TVD model outputs include timeseries of the surface signal measured at wave gauge locations across the domain, and grid output of wave height, mean water level, max water level and velocity. Wave gauge outputs were used to calculate H_s (significant wave height) and H_{10} (10% exceeded wave height) using the zero-downcrossing method where a 0.04 Hz bandpass filter was used to separate infragravity waves and sea-swell frequency waves.

3.4 Design conditions

Three primary design scenarios were used for calculating armour size. Toe armour size was calculated for the LAT (lowest astronomical tide) and MSL condition, using significant wave height around the structure. The design scenario for primary armour unit sizing was modelled using the 1% exceeded water level plus 1 m of sea level rise. Each combination of wave height and water level was simulated in Funwave-TVD using three directions (346°, 0° and 14°) that represent the window of extreme wave exposure. A representative wave period of 12 seconds, based on a joint probability of extreme conditions was used in all simulations [5].

Table 1: Wave height and water level used in the three design scenarios

Scenario	Hs (m)	WL (m)
1) Toe armour: LAT	5.50	-0.84
2) Toe armour: MSL	5.65	0.00
3) Primary armour size: 1% AEP + SLR	6.20	2.63

4. Results

4.1 Model calibration

4.1.1 Wave height

An example of how the physical model was replicated in the numerical model is presented in Figure 5 to show the calibration domain and output wave gauge locations. Model behaviour was generally consistent across scenarios, with wave breaking initiating on the battered slope and a surf-zone developing well seaward of the channel.

Each wave condition was simulated using a range of friction and breaking parameters, where friction was varied by changing the friction coefficient (C_d) and breaking was varied by changing the breaking ratio. The Manning formulation was used in most Funwave simulations, and the C_d value is a model specific friction factor, not Manning's n. The shock capturing handing of breaking is initiated when the ratio between the surface (η) and depth (h) exceeds a threshold, where the default is 0.8. The model was found to be consistent across breaking scenarios and moderately sensitive to increasing friction. The main observation in the model calibration process was that dissipation in the numerical model was more pronounced across the inner surf-zone compared to the physical model (Figure 6). This resulted in wave heights at inner domain locations being typically under-predicted in the numerical model.

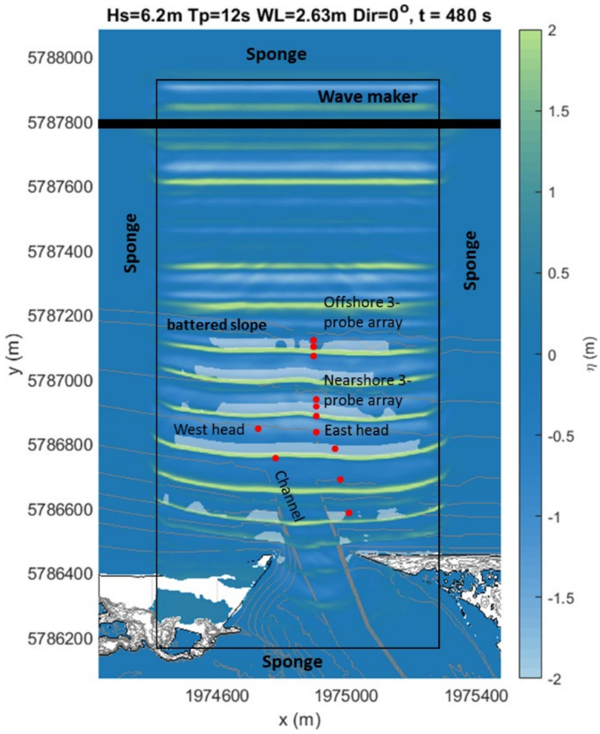

Figure 5: Example model calibration scenario for the Hanbar scenario showing the wave profile (colour), areas of breaking (grey shade) and outline of the wave maker and sponge layers. Wave gauge's locations used for calibration are presented in red.

Comparing the numerical model to the physical model for the higher sea level scenario, wave heights offshore of the channel were slightly over predicted, compared to wave heights at the inner surf-zone being slightly under predicted (Figure 6). This bias is likely a result of the Boussinesq model underestimating wave setup when compared to the physical model. The best representation of wave transformation for the higher sea level scenario was achieved using a breaking threshold of 0.8 and C_d = 0.05, resulting in model skill of 0.87 (Brier Skill Score).

The lower water level scenarios of LAT and MSL consistently underpredicted wave heights measured in the wave flume (Figure 6). This is also likely attributed to the Boussinesq model under predicting wave setup. The under-prediction was consistent across friction and breaking values, and a minimal friction value of C_d = 0.01 was for both lower sea level scenarios. Notably, completely removing friction did not change the prediction or model behaviour at the lower sea level scenarios. Model behaviour was otherwise consistent with the physical model. Therefore, a calibration multiplier was used to adjust the numerical model output for LAT (1.48) and MSL (1.29) at inner surf-zone locations (Figure 6).

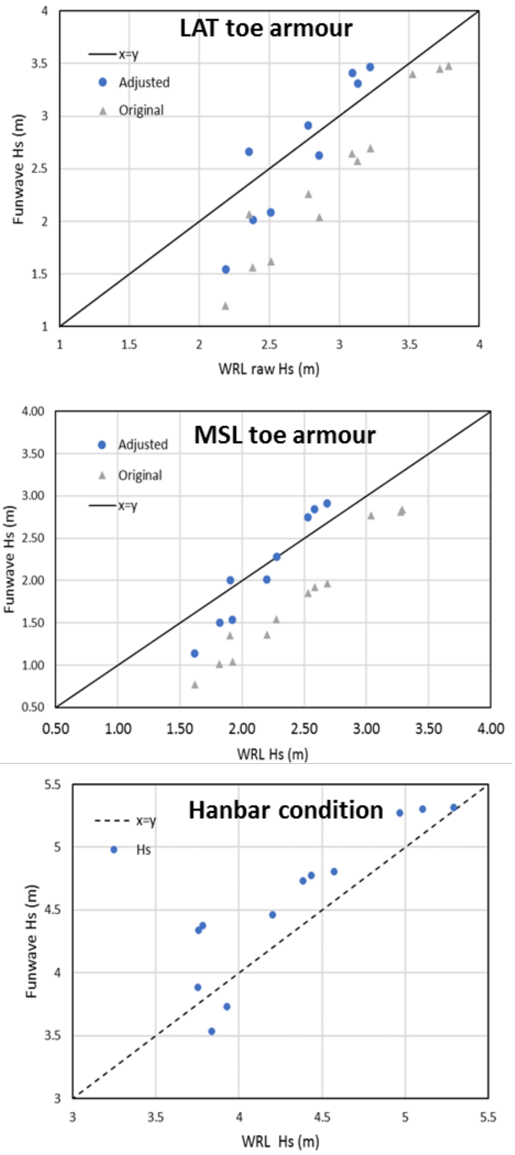

Figure 6: Wave height from the numerical model compared to wave height from the physical model, showing the default outputs (blue) and calibrated outputs (grey).

4.1.2 Reflection

Wave reflection off the breakwater structures was an important consideration in using the numerical model. The aim was to represent realistic reflection of the concrete armour units to resolve wave convergence in the channel. To do this in Funwave, a spatially variable friction layer was used. This layer gradually increased friction from the breakwater toe until a select value was achieved on the sloping face of the breakwater. This is different to the specific 'breakwater' friction in Funwave-TVD that can be used to represent a semi permeable breakwater as a sponge layer without the bathymetry. In a set of additional experiments, a 1D across shore profile was used test different friction values, and a 3-probe array was used to calculate the reflection coefficient using the Mansard and Funk method [4], as conceptualised in Figure 7. A specific reflection coefficient (k_r) was not available for Hanbar armour units, but a review of reflection from a range of concrete cast armour units by [2] was used to identify a target reflection coefficient of k_r between 0.3 to 0.4.

Gradually increasing the friction coefficient across the breakwater face resulted in an expected decrease in reflection. However, diminishing returns were achieved if friction is increased too much, and a minimum reflection coefficient of 0.35 was achieved using $C_d > 5$ with a Chézy formulation, and $C_d > 0.55$ for the Manning formulation.

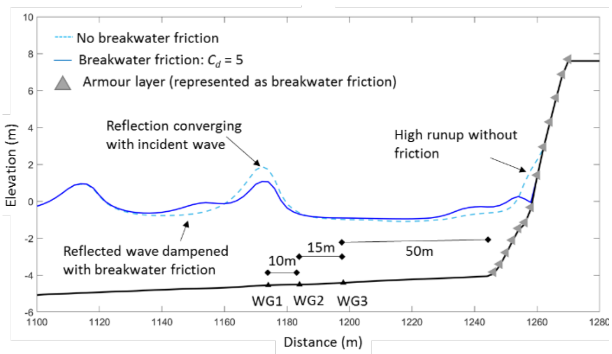

Figure 7: Schematic showing how reflection off the breakwaters can be controlled using a local friction layer

4.2 Design scenarios

The model calibration and testing phase provided confidence for using the model in larger scale simulations to represent design conditions. These design simulations were not constrained to the flume domain and extended seaward, and alongshore so different wave angles could be represented, along with directional spreading. The design simulations also included the breakwater structures in the bathymetry, with a friction layer on the breakwater face to dampen reflection. An example of the model that highlights wave behaviour around the nearshore and breakwater channel is provided in Figure 8 and example outputs of wave height and maximum water level are presented in Figure 9 for the future sea level scenario. These outputs show how wave reflection and convergence are resolved around the breakwaters and channel and highlight the wave dissipation patterns around the breakwaters.

4.3 Design wave heights and reflection

Selecting the design wave height used in armour unit size calculations was not a simple process of outputting values from a single model run. Specific consideration was given to identifying the critical wave direction for each location around the breakwater and removing seaward directed reflection. Careful consideration of reflection was important because seaward directed waves that result in H_s being amplified were considered unnecessary for the design calculations. However, reflected waves within the breakwater channel were considered very important for including in the design wave height.

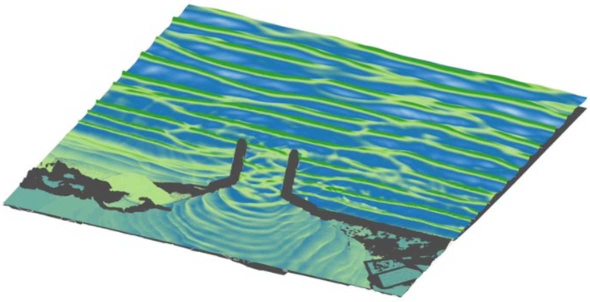

Figure 8: Example snapshot of wave behaviour around the breakwaters from the full domain simulation.

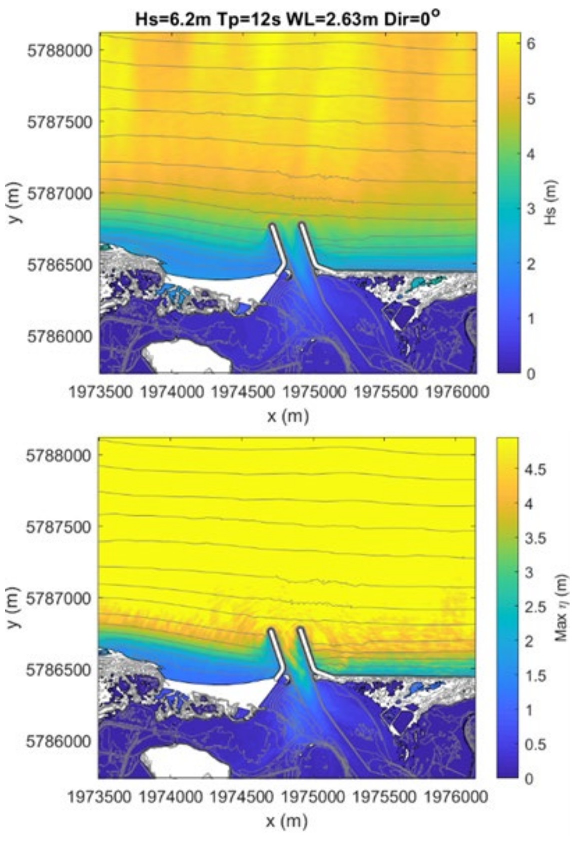

Figure 9: Significant wave height and maximum water level for the full model domain for the Hanbar design condition.

Using the numerical model to visualise wave interaction with the structures indicates that a radial reflection occurs off both breakwater heads, which propagates seaward in an ark. Waves also reflect off the outside wall and the interference can result in H_s values that do not represent the incident wave height that was desired for optimising the design.

A side wave also peels off the on both sides of the inside channel and reflects off the wall on an angle that converges with the next wave, forming a cross pattern (Figure 8). Reflection patterns within breakwater channel was considered important for including in the design calculations and were slightly different depending on wave direction.

Locations in the model domain where reflection was to be included, such as the breakwater channel, could be assessed by calculating H_s and H_{max} from the wave probe location in the numerical model. However, locations where the incident wave was desired were more complex. The three-probe array method only works on a shore normal transect and is not suited for the outside trunk or radial pattern of reflection. Further, a representative reflection coefficient was not considered appropriate for calculating incident wave height from the significant wave height. Directional spectra were also considered as an option for calculating wave heights associated with different frequencies and directions. This was tested using the velocity and surface signal at each wave probe location, then calculating the wave height from spectral moments for each partition. This method was promising, but also sensitive to long period waves and mean flow conditions. There was also a second step required to convert the spectral wave height to a shallow water H_{10} value for the calculating Hanbar design size.

A more suitable method for identifying the incident wave height was achieved by repeating all model simulations using the same domain, but without the breakwaters (Figure 10). The channel and all other components of the bathymetry were consistent. A benefit of the 2D JONSWAP boundary in Funwave is that the resulting boundary condition is deterministic and was therefore identical for simulations when simulations were repeated with and without the breakwaters.

The design wave height at each location was then selected based on the maximum height across the three directions, with careful selection of whether the incident or breakwater influenced wave height was used. As a first step, the lower wave height was selected when comparing the simulations with and without breakwaters. This effectively removed amplification due to seaward reflection, and accounts for wave shadowing in the channel and outside the western trunk (Figure 10). However, some wave gauge were located at a node where reflection reduced the wave height, in which case the incident wave was used. This process required each output location to be reviewed in detail to compare the six different wave heights, before the most appropriate one was selected.

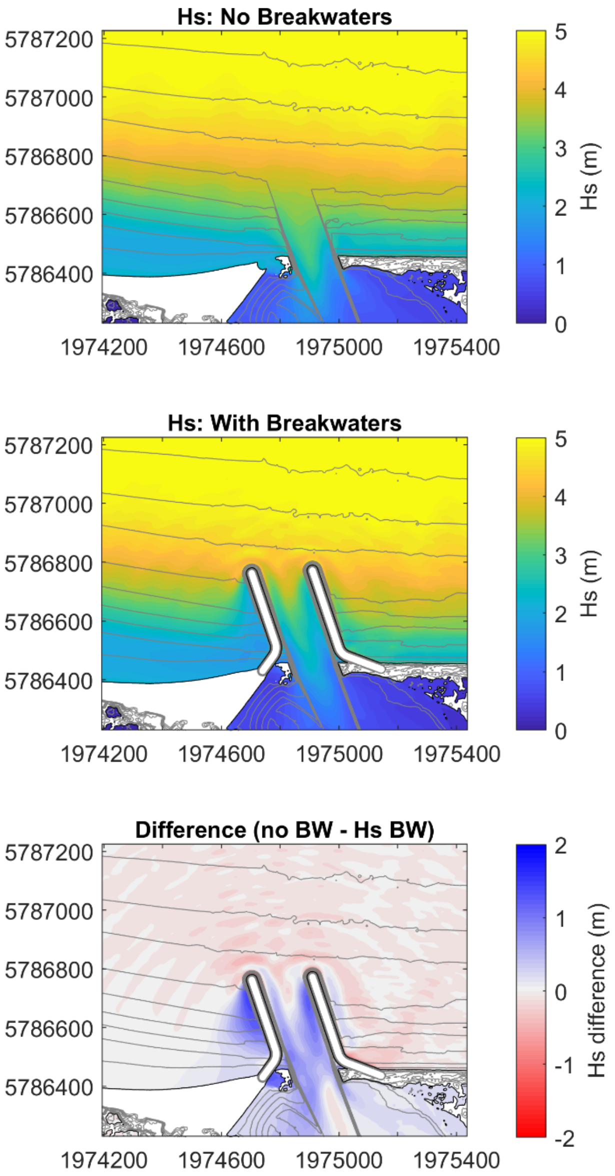

Figure 10: Comparison of wave height with and without breakwaters for the higher sea level scenario with waves from 0 degrees.

5. Summary

Wave processes were carefully considered for designing armour size around the breakwater training walls. This nuanced approach was necessary for optimising transitions in armour size to provide the required stability, without overdesigning the structure.

Wave heights generated using the numerical model Funwave-TVD were used to design a scale model of the breakwaters in the wave basin at WRL. Physical modelling of the same wave conditions were undertaken to test for damage and stability using scaled armour units. Physical modelling confirmed the design was stable for all water level scenarios.

Funwave-TVD was found to be a reliable and efficient numerical model for simulating large domains and extracting detailed information for design purposes. This is especially true for the complex wave behaviour that was resolved in the breakwater channel. However, the under-prediction of setup and therefore inner surf-zone wave height is a limitation for using the model without site specific calibration data.

A benefit of using an efficient numerical model was being able to repeat scenarios with and without breakwater structures. This approach proved to be the best available method for identifying locations in the domain where an incident H_s without influence of structure reflection could be used in design calculations.

6. Acknowledgements

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7. References

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