Ōpōtiki Harbour Development Project - Design of New Zealand's first river training works in over 100 years

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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 involves construction of twin 400 m long training wall breakwaters, dredging a navigable channel into the Harbour, and closing the natural river mouth.

The design solution chosen involves conventional rubble mound breakwaters armoured with Hanbar concrete armour units and includes a wide rock armoured toe apron. Design of the Harbour entrance breakwaters has involved a complex process of defining both coastal and river design parameters that input into the detailed design of the structures. Key aspects of both coastal and river processes were modelled numerically and physically with the results of the modelling feeding into the detailed design of the structures. Data obtained from site investigations was used to inform and calibrate the modelling and design decisions alongside predicted climatic changes to the coastal and river hydrology over the design life of the structures. Compared to the engineers of 100 years ago we have a greater understanding of the construction environment and more design tools, however this creates additional challenges.

This paper discusses how the respective models were used to calibrate and evaluate the design parameters from both coastal and river processes. It also discusses some of the design philosophy and decisions made during detailed design particularly in relation to design wave height and the effects of waves against currents, the choice of K_D value for stability design of the armour units, calibration of the calculated and modelled wave overtopping flows, requirements and feasibility for ground improvements, and the design philosophy behind the choice of toe apron design.

Keywords: coastal structures, breakwater design, harbour entrance, concrete armour units, hanbars.

1. Introduction

The Ōpōtiki Harbour Development Project will improve maritime access for the Ōpōtiki region by stabilising the Waioeka River mouth using twin training wall breakwaters ("training walls"). These will be located to the east of the existing river outlet and extend out to approximately 350m from the existing shoreline, with dredging to provide a navigable channel from the nearshore towards the town.

This work will allow improved passage for vessels and will stimulate regional economic development in industries such as aquaculture. The current physical environment at the Waioeka River mouth is shallow and highly dynamic, with mobile coastal spits and nearshore bars creating navigation hazards and unreliable access (Figure 1).

The decision to route boat traffic and a significant river system through the same gateway created a number of design and operational challenges when coupled with an already dynamic wider coastal environment.

Figure 1: Existing Waioeka River entrance and proposed including river training works (source: ODC|stuff.co.nz)

This paper summarises the main elements of the detailed design that resulted in changes to the initial concept design (Figure 2) and provided increased confidence in the performance and constructability of the structures.

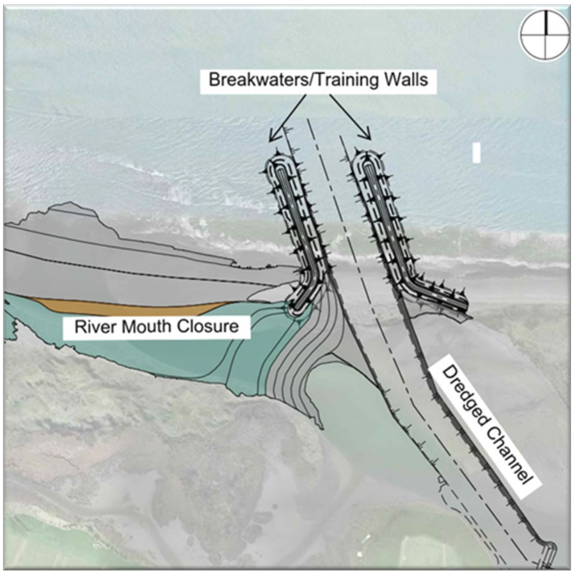

Figure 2: Key features of the harbour development relative to the existing shoreline and Waioeka Estuary

2. Data collection and design environment

For the concept design, data collection was limited, there was uncertainty around the location and extent of structures, and an understanding that given the dynamic environment the morphology of the coastline and estuary was likely to change prior to construction. The detailed design required the collection of a number of additional datasets, including high resolution topographic and bathymetric surveys that reflected key changes to the environment including; a migration of the river mouth several hundred metres to the west, changes to the river delta, and fluctuations in seabed elevations at the seaward ends of the training walls. Changes in river geometry were also supplemented with river cross sections. These data sets provided key inputs to updated numerical models, the baseline topography for the physical model, and ground levels for the training wall design.

An updated wave hindcast model was produced and calibrated against wave buoy data collected at the 10m and 40m contours. These provided the inputs to the numerical and physical models used to resolve the nearshore wave environment and wave-structure interactions.

A number of assumptions had been made as part of the concept design relating to ground conditions, and both nearshore and estuarine sediment characteristics, based on limited investigations. Prior to detailed design process a comprehensive programme of sediment sampling and ground investigations was conducted. Collection of data in the surf zone was problematic and involved the collection of CPT and borehole data from a raised platform along the proposed training wall extents. This information was critical for the geotechnical assessment and design of the structures and provided valuable inputs to the morphological modelling of river scour and shoreline evolution.

3. Investigations

3.1 Numerical Modelling

Two modelling approaches were utilised to assess wave processes at Ōpōtiki [1]. First, a third-generation spectral wave model Simulating Waves in the Nearshore (SWAN) to understand wave transformation from the 20 m depth contour to the harbour mouth. The SWAN model was used to connect wave climate (hindcast) points available at the 20 m and 10 m depth contours, with nearshore wave heights around the shoreline and structures. SWAN results also provided a first order assessment of design scenario wave heights around the structure.

To understand wave processes impacting the structures at a higher resolution, the fully nonlinear Boussinesq model, Fun wave-TVD, was also used. Fun wave-TVD is a free-surface model and therefore resolves water level and velocity motions of individual waves, including how these waves interact with each other, the bathymetry, the shoreline, and structures. This modelling approach also resolves surf-zone processes such as currents and wave setup, which provide an instant feedback to the propagation, refraction, diffraction, reflection, and breaking processes. Results from Fun wave-TVD provided a more detailed understanding of wave heights and surf-zone water levels that are required for design.

Modelling of the river system was undertaken separately in order to gauge the effects on peak flood levels, and scour depths in the vicinity of the training walls. The river model needed to be capable of simulating morphological bed mobilisation and sediment transport. A numerical model was developed in TUFLOW FV to undertake this assessment.

Scour generated during river flood events is critical to the design, both in terms of conveyance of flood flows and peak water levels which have to be shown to not exceed the present day flood risk, and for assessing the requirements for structure toe protection and stability during these events. The model also provides key information on current speeds at different stages of the tidal cycle. Confidence in the modelled response was developed through sensitivity testing.

3.2 Physical Modelling

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney was engaged for the physical modelling of the Ōpōtiki Harbour Development works. Modelling was split in to four stages:

- Stage 1: 3D modelling of nearshore processes using natural bathymetry.
- Stage 2: 2D modelling of key sections of the training wall trunks
- Stage 3: Quasi-3D modelling of the breakwater head
- Stage 4: Full 3D modelling of complete structures

For Stage 1 a full 3D model bathymetry was fabricated in WRL's wave basin, representing the existing bathymetry at the site in the vicinity of the future training walls. The bathymetry included the existing coastline as well as the entrance of the future dredged channel, and extended seaward to approximately the -6 m RL contour. During this stage, the model was used to calibrate and investigate a range of wave conditions, with measurements taken at several locations within the nearshore zone.

For Stages 2 and 3, a series of 2D and Quasi 3D tests were undertaken to examine the training wall design at several key locations such as armour transitions and roundheads. These tests utilised the same model bathymetry within the wave basin, with temporary wave guide walls set up to form 2D test flumes on the bathymetry. During these tests, the focus was on understanding the characteristics of armour stability and overtopping under both perpendicular and oblique wave attack.

For Stage 4, the full 3D training walls were constructed (Figure 3) on either side of the dredged entrance channel. The focus of the testing during Stage 4 was the confirmation of the training wall armouring design at key locations for extreme wave conditions, and investigation of wave penetration within the entrance channel for smaller operational wave conditions.

A separate physical modelling exercise was also conducted by the University of Auckland [7] to assess the performance of the falling toe apron and different construction options.

3.3 Geotechnical modelling

One of the key requirements of the design of the breakwaters is that they are geotechnically stable during static conditions, storm scour conditions and seismic conditions. Due to the liquefaction risk at the site, the primary geotechnical risk is related to the seismic performance of the breakwater.

For modelling purposes the geotechnical modelling assumed a worst case scenario whereby a river flood event had resulted in scour of the channel, the falling apron had dropped in to the channel, and the river channel had only partially infilled following the flood. This constituted a retained height of up to 10m.

Training wall stability was assessed using SlopeW and FLAC analysis (Figure 4) and showed that in order to achieve the maximum allowable displacement criteria, effective dynamic compaction was required to a depth of at least -10m RL.

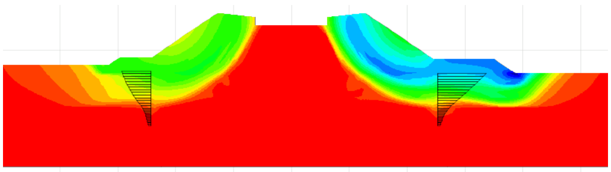

Figure 3 : Example of FLAC Analysis results and predicted displacements illustrating a drop in crest level and horizontal movement of the toe.

Figure 4: Fully constructed 3D model of structures within the WRL wave basin.

3.4 Ground Improvement trials

The concept design presented recommendations for Dynamic Compaction (DC) Ground Improvement and geotextile reinforcement. The depth of improvement has been shown to be critical for the structure stability for the seismic load cases and it is acknowledged that DC is only suitable for some soil types. Therefore, a DC Trial (Figure 5) was completed primarily to understand the suitability of DC for the site, and likely depths of improvement under the training walls. The concept design also incorporated a high strength basal geotextile and the DC Trial assisted in understanding the potential for damage to the geotextile when subject to DC.

The proposed DC method was effective in densifying the upper soils within the trial area which are largely representative of the training wall works. The energy levels applied were determined based on published relationships to achieve the target depth.

Figure 5: A DC trial was conducted on site to assess the likely depth of improvement and potential damage to geotextiles.

Significant densification was observed in the sands in the top 5 to 7 m. Reduced levels of densification were shown to extend to a depth of around 10 to 11 m which is equivalent to -7.5 to -8.5 m RL when DC is completed from 2.5 m RL on the training walls. The target improvement depth of -10 m RL was not achieved. The basal geotextile reinforcement was also shown to be significantly damaged during the DC trial even with a gravel/sand "cushion" above the geotextile. Based on these observations from the DC Trial, it was concluded that the geotextile reinforcement would be removed from the design and alternative reinforcement of sheetpiling was selected to supplement the DC.

4. Key design considerations

Based on the investigations conducted as part of the detailed design process [6] a number of changes were made to the design. The key considerations are outlined below.

4.1 Armour unit stability coefficient

Calculations for hanbar unit sizing were derived based on the calculated design waves along the structure and a stability coefficient. The Hudson Damage Coefficient (K_D) as recommended in design guidance [3] was used in order to allow for comparisons with previous studies, where a conservative .value for the 5% damage level is given as $K_D = 7$ [2]

2D Physical model tests (Figure 6) allowed for the determination of actual K_D numbers, using a placement methodology and density specified for construction, a range of wave conditions and normal incident waves and those approaching at 45^o. Although the actual incident wave angle is likely to be more acute, 45^o chosen as a practical and conservative angle for modelling waves impacting the breakwater trunk sections.

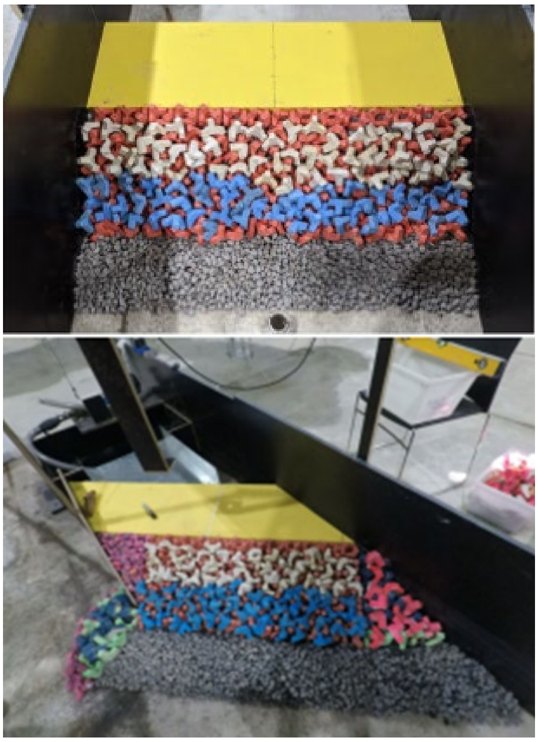

Figure 6: 2D Physical model tests were used to define site specific stability coefficients for waves approaching head on (top)and at 45^o (bottom).

Test results recorded the number of displaced units, but also the number of severely rocking units. The latter were recorded to provide increased confidence in the stability of the structure but have not been used to review the damage coefficient versus percentage relationship as this is not typically considered in the Hudson formula.

Figure 7 shows that for normal incident waves the observed damage (displaced units) scatter points are typically above the black linear line. This shows that for 5% damage (i.e. initial damage based on Hudson) the K_D value is 7 as shown by the red dotted line. This is consistent with the K_D value for Hanbar units recommended by previous studies [2].

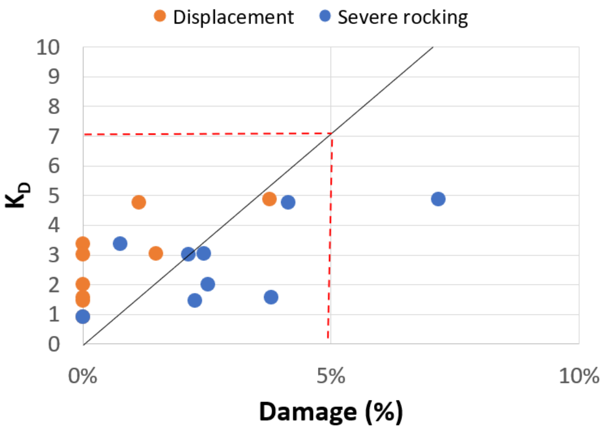

Figure 7: Relationship between percentage damage and stability coefficient for displaced units (orange) and displaced units + severely rocking units (blue) for normal incident waves. Red dotted line indicates appropriate K_D value for 5% damage based on best fit (black line).

The relationship between percentage damage observed and damage coefficient K_D for waves approaching at 45 degrees demonstrated that the hanbars were relatively more stable for the 45 degree approach angle. The same approach was used to assess the relationship, with a 5% damage threshold equating to a K_D value of approximately 9. Due to the limited number of tests completed for the 45 degree approach angle, and to allow for some future proofing of the breakwaters should greater navigation depth in the channel between the breakwaters be required, the position of transitions between Hanbar sizes was based on the more conservative K_D value of 7. Hanbar layouts were tested in the full 3D physical model to validate the performance of a design layout.

4.2 Wave current interaction

The numerical and physical wave modelling set out in the previous sections does not allow for ambient currents or non-wave driven currents entering the model domain. However, as the Waioeka River enters the sea through the breakwater channel, waves propagating towards the breakwaters may interact with the river flow that is present within the channel. This wave-current interaction may locally modify wave processes, influencing wave shoaling, breaking and dissipation. The influence of currents will depend on the interaction of fluvial flow velocity and offshore wave processes. To investigate the influence of current on potentially increasing wave height in the breakwater channel, a combination of empirical calculations and numerical modelling methods were adopted.

Outputs from the river modelling work were used to inform river flow conditions that may influence wave height in the breakwater channel. This shows that the largest current velocities occur at the deeper parts of the river landward of the breakwaters, through the breakwater channel and extending some distance offshore. Water level and velocity timeseries during a flood event show that the peak velocity at the centre of the channel (~3 m/s) occurs at low tide (see Figure 8) and that the velocity during the peak water level is roughly 1 m/s less (i.e. ~2 m/s).

The effect of opposing currents on wave height was assessed using the numerical wave model SWAN. This has been done by running SWAN excluding and including currents, and to assess the percentage increase in wave height. Currents were included in SWAN by exporting the X and Y velocity from the TUFLOW FV model domain, interpolated to the extent of the SWAN model domain. Using this approach, the appropriate velocity for a given water level and known time on the hydrograph was achieved.

Results show the typical increase in wave height in the vicinity of the breakwater structures is in the order of 10% and may go up to 15-20% at the centre of the channel (Figure 8). As a single current field from TUFLOW FV was used, differences in current velocity and direction may change the extents of the zone where wave heights increase and could potentially extend further up the breakwater channel. For this reason, design waves were increased by 10%, for design calculations, all the way up the channels to the elbow of the breakwaters.

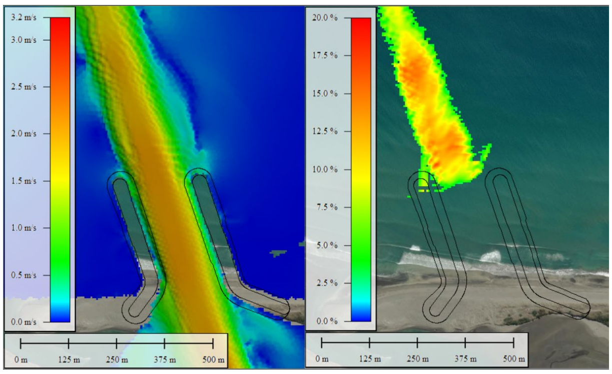

Figure 8: Example of current velocity from the TUFLOW FV model during peak velocity (left figure) and wave-current interaction in terms of percentage difference for Hs = 2m, WL = MSL during a 5% AEP river flood event during peak velocity (right figure)

4.3 Wave Overtopping

In order to assess the wave overtopping risk, calculations were completed using the EurOtop design guide [5]. As published roughness factors for Hanbars were not available, we assumed a layer roughness similar to Tetrapod armour units, on the basis that these are similar in shape and placement interlock and have a similar design stability factor. As the crest Hanbars are to be placed in a single layer of pattern placed units the effective impermeable crest height was also unknown, as depending on the wave angle the gaps between the Hanbar chimneys may allow increased wave penetration and overtopping.

Several overtopping tests were completed in the physical modelling study. Tests were done with waves perpendicular to the training walls, and at an angle of 45 degrees to the walls. Various Hanbar sizes were tested, using a range of wave climates. The results showed that as expected overtopping rates increased as wave climates increased (and crest freeboards reduced), decreased as the Hanbar size and layer thickness increased absorbing more wave energy, and decreased with the more acute wave direction (Figure 9). As an example, the 6.5T Hanbars tested with wave climate WC4 (which is the present day 1% AEP storm surge and a 500 year return period wave climate), had an overtopping flow rate of 23 L/s/m when tested with waves perpendicular to the breakwater and this reduced to zero when tested at an angle of 45 degrees.

Results from the physical modelling study were then used to calibrate the Eurotop formulae using the assumed tetrapod roughness factor and setting the permeable crest elevation at the elevation of the top of the Hanbar chimneys, and good correlation was achieved with the physical model test results. Allowing for the actual angle of incident wave attack the calculated wave overtopping flows for the training wall trunks are as follows:

- For present day 1%AEP storm surge and 500 years return period wave climate (wave climate WC4), the largest overtopping flow rate was 0.08 L/s/m
- When sea level rise of 1 m is allowed for (wave climate WC5), the highest overtopping flows occur along the trunk of the eastern breakwater and are between 10 and 20 L/s/m.

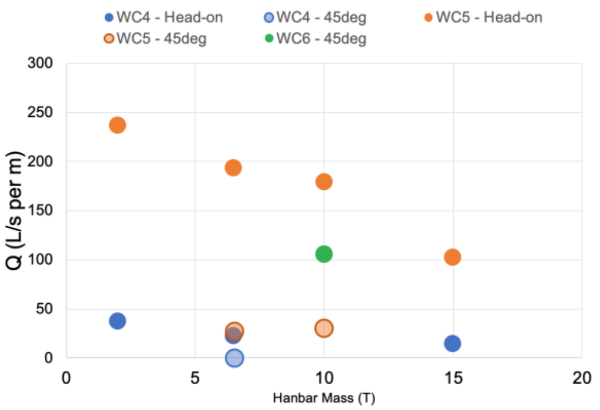

Figure 9: Results of Physical modelling overtopping tests using different hanbar sizes with a 1% AEP still water level and 0.2% AEP wave height (WC4 - Present Day, WC5 - 1m SLR, WC6 - 1.5m SLR)

Hanbar unit placement density, methodology and crest detail all have effect on OT rates that is not encapsulated by a simple permeability/roughness coefficient. Calibrating the results with the physical modelling allowed for increased confidence in predictions of wave overtopping and the potential requirements for future maintenance of the crest surface.

4.4 Geotechnical stability

The DC trial determined that, while the ground improvement in the upper soils reduced potential for liquefaction in these materials, the ground improvement under the toe of the Hanbar slope was not sufficient to prevent a slope stability failure during a seismic liquefaction event. The use of high strength geotextile at the base of the fill was also rejected due to risk of damage.

Sheet piling was selected as the preferred option to supplement DC, with Steel sheet piles at the toe of the concrete armour slope extending down to provide 3 m embedment into the non-liquefiable layer. The sheet piles will provide shear resistance to slope instability during seismic conditions. In addition, the sheet piles may also be used to provide some wave protection during construction prior to driving to design depth.

4.5 Toe Armour

Design of the toe armour and apron had to fulfill three criteria, stability under wave attack, a functional falling apron under river scour and providing geotechnical stability for the structure during a seismic event.

Physical modelling of the performance of the falling apron was conducted by the University of Auckland [7]. Two basic apron designs were tested, one with an underlayer beneath three layers of armour rock, and one consisting of only armour rock. The tests showed that as the riverbed in front of the apron scoured, rock was launched from the outer end of the apron to armour and protect the scoured slope. The resulting protection layer was only one stone thick for both apron designs tested. So if an underlayer was included in the falling apron design the slope scoured by river flood flows would only be protected by a single rock thickness the size of the underlayer rock. This would have resulted in the scoured slope then being venerable to wave scouring of any exposed sections of the slope only protected by underlayer rock.

Based on the performance of the apron configurations tested (Figure 10) a hybrid of two configurations was specified. The inner 6 to 8 m of the apron requires an underlayer and will act more like a coastal scour apron. This will help to keep the apron in place and at a good elevation and protect against wave down rush sucking sand out through armour rock for the toe area closest to the breakwaters. It is important from a geotechnical perspective that the inner part of the apron is kept in place as a buttress to the sheet piles and the toe of the concrete armoured slope. The outer 4 to 6 m is to be constructed using only armour stone, so that in a river scour situation the scoured slope has more even and continuous protection, as is desirable from a river scour perspective.

Figure 10: Example of Physical model test showing the performance of a falling rock apron using three layers of armour rock and an underlayer [7].

Aprons are designed to be constructed three layers thick, with the expectation these will initially fall to an approximately 1.5H:1V slope in response to river scour, and ultimately form a 2H:1(V) slope under extreme scour events whilst maintaining an even and continuous protection of at least one layer thick.

Falling apron widths were sized based on the maximum predicted river scour within the channel defined by the numerical modelling and empirical calculations of predicted scour and fluctuations in bed levels on the seaward side and heads of the training walls.

The rock toe/apron armour was sized using a range of empirical formula [3,4] with the adopted methodology varying depending on water depth. At higher water levels larger waves can reach the structure, but the toe is submerged and wave forces are reduced. Lower water levels allow for wave action to act more directly on the toe, however due to depth limiting waves are smaller. A range of water levels, wave heights and methods were assessed, and the critical water level and associated methodology used in sizing the rock.

Rock sizing was validated with physical model testing at a range of water levels, this included addressing concerns that empirical calculations may underestimate toe armour sizing at the heads (ends) of the training walls, which performed well under testing with damage well below tolerable levels. This is considered the result of taking a conservative approach and adopting the largest specified rock size from a range of formula and conditions.

5. Summary

Detailed design of the Ōpōtiki Harbour training walls relied on the collection of significant amounts of additional data, numerous numerical and physical models, and construction trials on site. The process was often iterative with results from different work streams feeding back in to associated design elements and resulting in significant revisions to the design and a greater confidence in the constructability and ultimate performance of the structures.

6. Acknowledgements

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7. References

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