DETACHED BREAKWATERS DESIGN OPTIMISATION USING NUMERICAL AND PHYSICAL MODELS FOR GUGGENHEIM ABU DHABI MUSEUM

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The museum will be built on a landmass retained by a vertical seawall with varying top elevations, protected by four detached breakwaters. One of the main challenges of the study was to limit wave overtopping and negative wave pressures on the seawall while minimizing the visual impact of the detached breakwaters. Empirical approaches and a series of numerical and 2D/3D physical models were used to validate and optimize the design of the detached breakwaters, while still meeting the project requirements in regards to wave loads and wave overtopping discharges.

Keywords: detached breakwaters, physical modelling, wave agitation, wave overtopping, design optimization

Introduction

The Saadiyat Cultural District is being developed in the southwestern corner of Saadiyat Island in Abu Dhabi (United Arab Emirates) and is set to be the area dedicated for culture and art. Crafted by the world's greatest architectural minds, the area will be a shining beacon on the international arts scene with the Louvre Abu Dhabi, Zayed National Museum and Guggenheim Abu Dhabi. The Guggenheim Abu Dhabi (GAD) Museum is the largest museum in a series of cultural institutions planned as part of the Saadiyat Island Cultural District (Figure 1).

Figure 1. Planned museums within the Cultural District Area

The GAD museum will be developed in the northwestern corner of the Cultural District (Figure 2 and Figure 3). The museum landmass will be protected by means of a diaphragm wall along seaward perimeter. Four detached breakwaters are planned around the Guggenheim Museum to limit the wave exposure to the seawall (Figure 5). Advanced numerical models, empirical approaches and three sets of 2D and 3D physical model tests were performed to design the cross-section and the plan view layout of the breakwaters. Back-analysis of the 3D physical models with empirical and numerical approaches was used to optimize the design and reduce the construction costs.

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Figure 2. Project Location in Abu Dhabi, United Arab Emirates

Figure 3. Project Location at Southwestern corner of Saadiyat Island

GAD Landmass Configuration and Detached Breakwaters Concept Design

The landmass area surrounding the future GAD Museum consists of various platforms with levels between +2.5 and +10 m MSL (see Figure 4). The design high water level for a return period of 100 years is +2.41 m MSL, only 9 cm below the lowest target land level. This highlights the importance of a well-structured design of the detached breakwaters to limit the wave exposure around the GAD landmass.

During the concept design (by others), a plan layout with four detached breakwaters was developed to protect the GAD landmass (see Figure 5). Three different detached breakwater configurations were proposed:

- Wide-crested breakwater with crest-level +2.5 m MSL;
- Narrow-crested breakwater with crest level +3.0 m MSL;
- Narrow-crested breakwater with low crest level of +2.0 m MSL and long underwater apron.

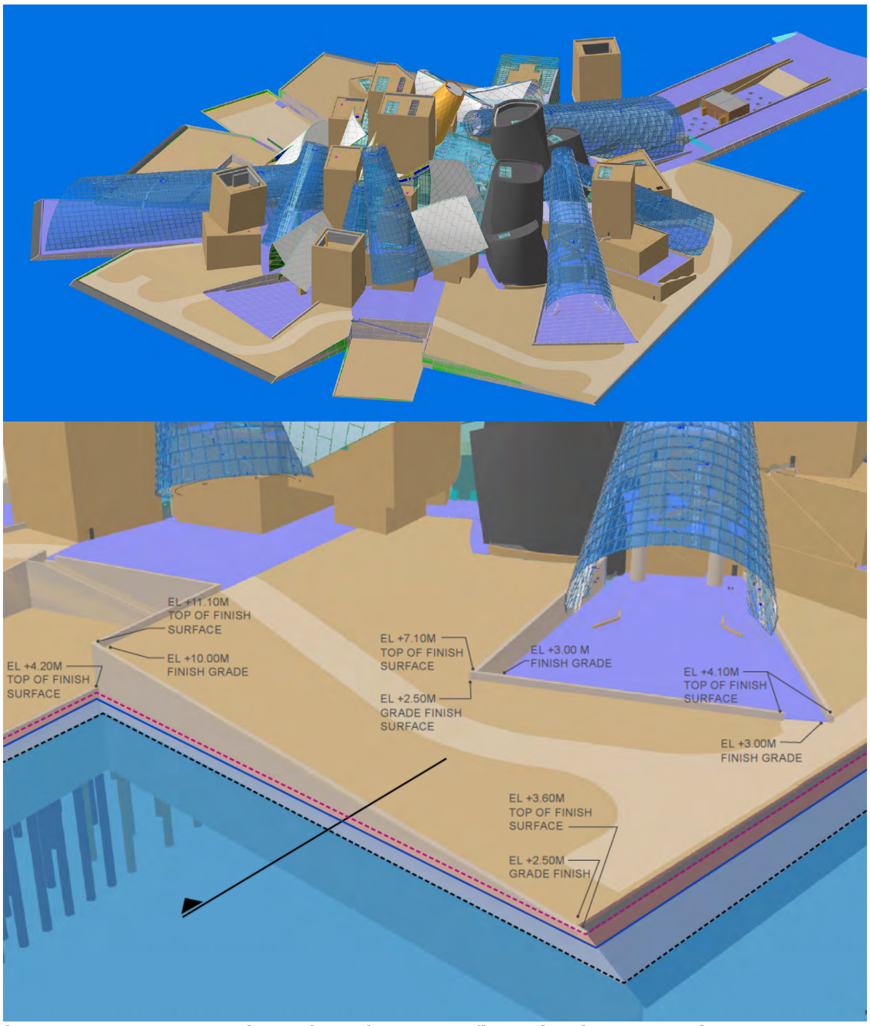

Figure 4. GAD Museum Planned Landmass Configuration (Source: Gehry Partners LLP)

In agreement with the Client the narrow-crested breakwater with long underwater apron was selected as the preferred solution. Based on previous physical model test results (by others), it was concluded that the crest of the breakwater should be increased and that the lowest target landmass level of +2.5 m MSL for the GAD was likely not feasible.

Project Requirements

The project criteria required the detached breakwaters to sustain only minor damage during the 100 year return period event. A damage parameter (N_{od}) of 0.5 was used for the toe and a damage parameter (S_d) of 2 was used for the breakwater armour stability. The apron was allowed to reshape providing that it did not undermine the overall breakwater stability, translating to a damage parameter (N_{od}) of 2.

Buildings and landscaping elements were planned around the landmass area, for which no damage was tolerable. Consequently, maximum allowable mean wave overtopping discharges of 1 and 2 L/s/m were respectively considered. In some areas public access was to be guaranteed, therefore a maximum allowable mean wave overtopping discharge of 0.1 L/s/ was considered for the 5 year return period event, accepting that for events more severe than this, public access would be prevented.

At the time of the studies, the diaphragm seawall around the GAD landmass had partially been constructed to a level of +1.5 m MSL. The seawall was designed to allow maximum positive wave forces (landward directed) of +192 kN/lm for a water level of +2.3 m MSL.

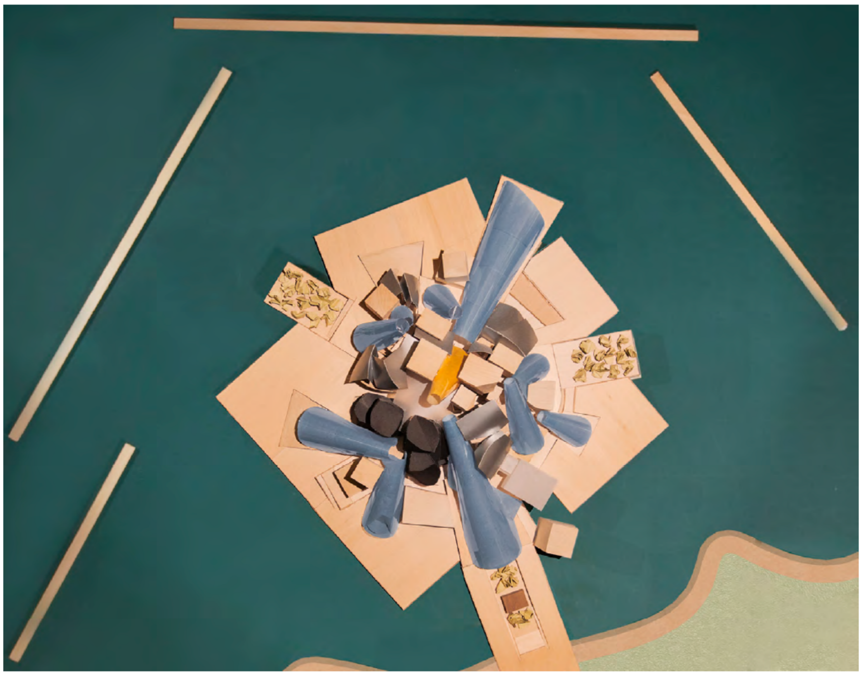

Figure 5. Planview Configuration of Detached Breakwaters

Site Conditions

A series of coastal studies were developed to determine the site conditions. A hindcasting approach was used to develop estimates of wind, waves and storm surge near the Project Site. This involved the application of a series of numerical models driven by atmospheric forcing (primarily wind fields) to develop time series metocean databases spanning a period of 51 years. A statistical analysis of the data was then implemented at a location offshore the Project Site to derive typical climatology and extremes.

The climatic wind rose based on 51 years of data at the offshore point is presented in Figure 6. The majority of winds come from the Northwestern and northeastern sectors. The strongest winds come from the Northwestern sector. The maximum storm surge associated with a north-westerly storm with a return period of 100 year is 1.03 m.

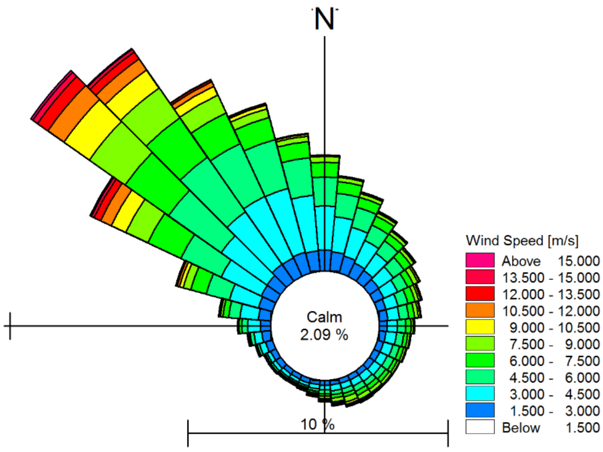

Figure 6. Offshore Wind Conditions

The climatic wave rose based on 51 years of data at the offshore locations is presented in Figure 7. The majority of waves have a direction of between N310° and N340°. A small portion of the waves have a direction of between N0° and N50°. The largest waves originate from directional sector [N300°;N340°] with significant wave heights of 2.8 m and 3.5 m for the return period of 1 and 100 years respectively.

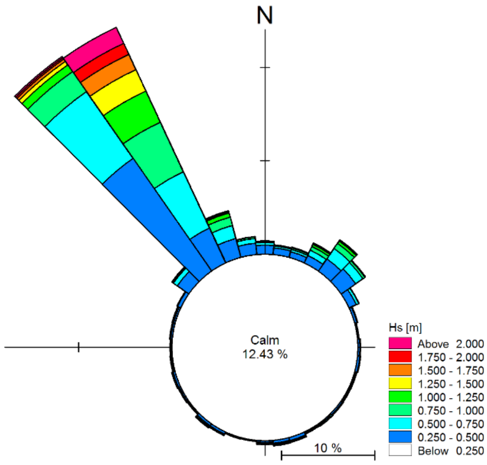

Figure 7. Offshore Wave Conditions

A wave transformation study was implemented using the SWAN numerical model to determine extreme (storm) wave conditions at the project site. Wave heights at the Project Site range between 1.6 m and 2.7 m for the 1 year return period and between 1.9 m and 3.0 m for the 100 year return period. The extreme wave conditions for the various locations around the project site (Figure 8) are presented in Table 1.

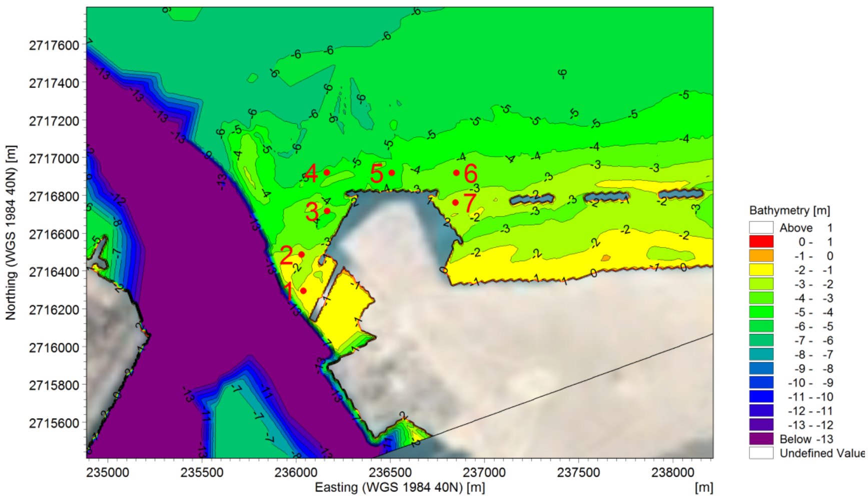

Figure 8. Wave Extraction Points at Project Site

Table 1. Extreme Wave Conditions at the Project Site

Return Period	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6	Point 7
1	1.6	2.0	2.4	2.7	2.5	2.4	2.1
100	1.9	2.2	2.7	3.0	2.8	2.7	2.3

General Design Approach

The general design approach is schematized in Figure 9. The following main steps were followed:

- Determination of site conditions by means of numerical modelling (offshore metocean and wave transformation & penetration);
- Determination of allowable wave conditions in front of GAD landmass seawall, to ensure that maximum positive and negative pressures were not exceeded, by means of 2D physical models;
- Validation of detached breakwater design and measurements of wave transmission through/over breakwaters by means of 2D physical models;
- Determination of detached breakwaters cross-sections and plan layout adopting the results of 2D physical models and wave penetration numerical model;

- Value engineering assessment by applying a combination of 3D physical model and wave penetration numerical model to determine the final and most optimal detached breakwater layout and cross-section.

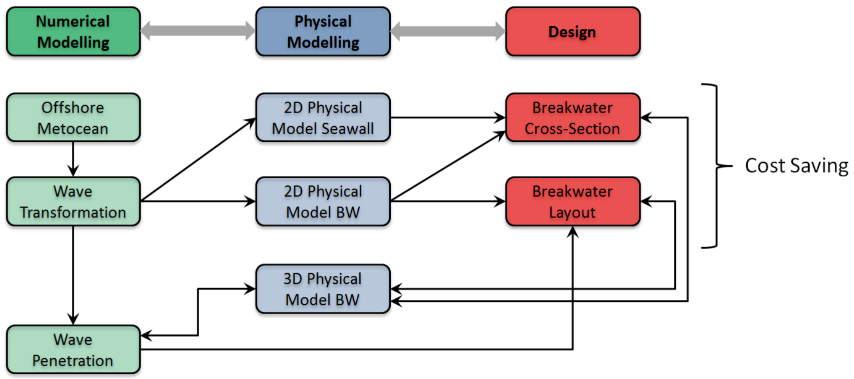

Figure 9. General Design Approach

2D Physical Model Test for GAD Landmass Seawall

Two-dimensional physical model tests of the GAD landmass seawall were conducted at a scale of 1/15 to determine the mean wave overtopping discharges and the pressures and forces on the seawall.

For the overtopping tests, the test section was built with wood considering two different crest levels of +3.3 m MSL and +4.0 m MSL (see Figure 10). Five different wave heights between 0.5 m and 1.2 m were tested with peak wave periods of 8.5 s, representative for conditions with a return period of 100 years.

The wave overtopping discharges for the various conditions for a wall level of +3.3 and +4.0 m MSL are presented in Figure 11. It was observed that the measured mean wave overtopping discharges in the physical model were significantly lower than the discharges estimated by means of the formulae for the probabilistic approach as per EurOtop (EA, ENW and KFKI, 2007). It was concluded that for a seawall top level of +3.3 m MSL a significant wave height of up to 0.9 m can be allowed without exceeding the maximum allowable overtopping discharge of 2 L/s/m.

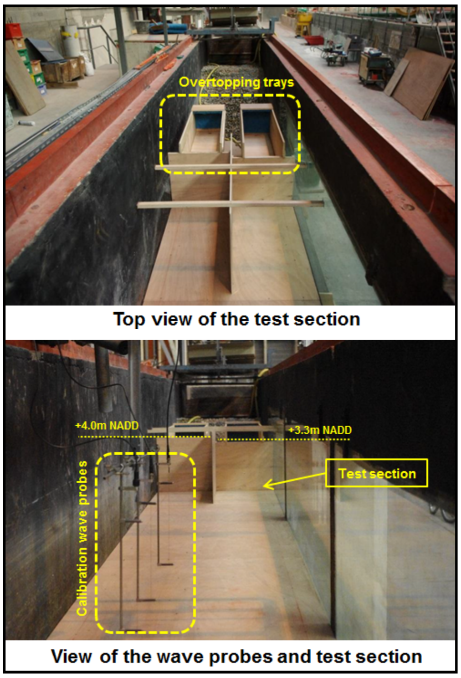

Figure 10. Model Set-Up (Overtopping)

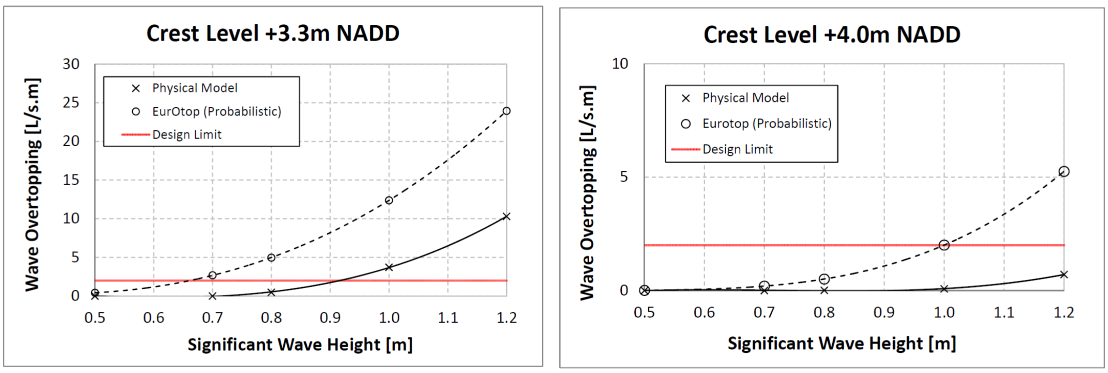

Figure 11. Mean Wave Overtopping Discharge Results

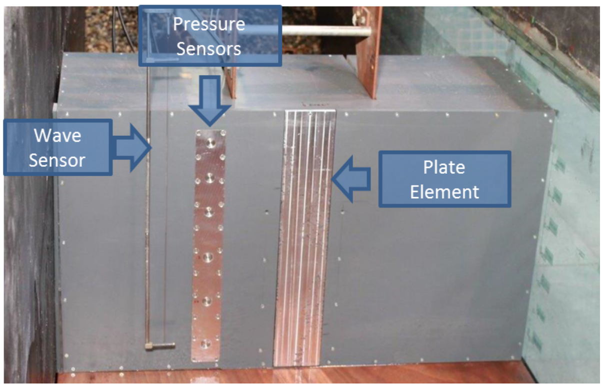

Figure 12. Model Set-Up (Wave Pressures and Loads)

For the wave load tests, the physical model test section was built using rigid PVC elements considering a crest level of +5.6 m MSL, allowing no overtopping (Figure 12). Pressure loads were measured using silicon piezo-resistive sensors at six levels along the wall (-2.10, -0.60, +0.90, +2.41, +3.50 and +4.6 m MSL). Forces were measured on a thin vertical section (free plate element) spreading over the entire height of the wall. A tensor scale was connected to the free plate element, which allowed measurements of the horizontal accelerations which were directly translated into force.

The measured positive (landward directed) and negative (seaward directed) wave forces are presented in Figure 13 and have been compared with estimated wave forces based on empirical equations. The positive wave forces measured in the physical model tests were approximately 12% lower than the values estimated by empirical equations (Goda, 1985). The negative wave forces measured in the laboratory were approximately 12% higher compared to the values estimated by empirical equations (OCDI, 2002).

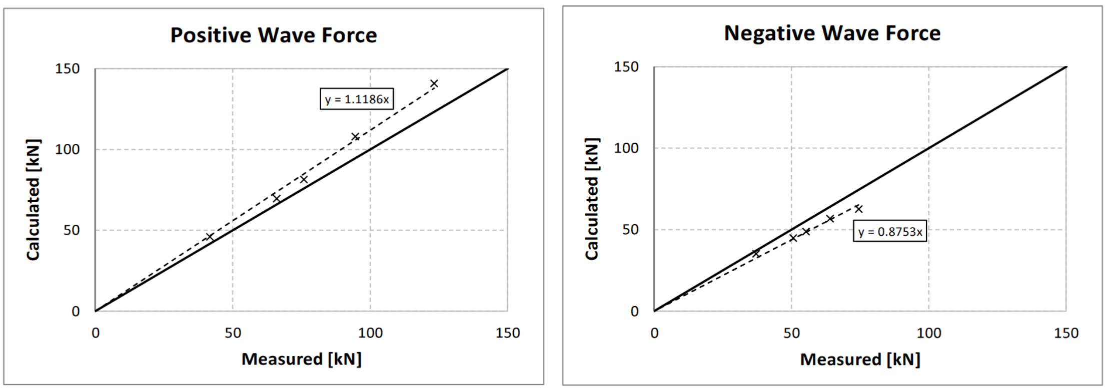

Figure 13. Comparison of measured positive (left figure)) and negative (right figure) wave forces with wave forces estimated using empirical methods

Based on the result of the 2D physical models (Figure 14), it can be concluded that an incoming target wave height of 0.9 m translates to an estimated positive and negative wave force on the seawall of 90 kN/m and 60 kN/m respectively.

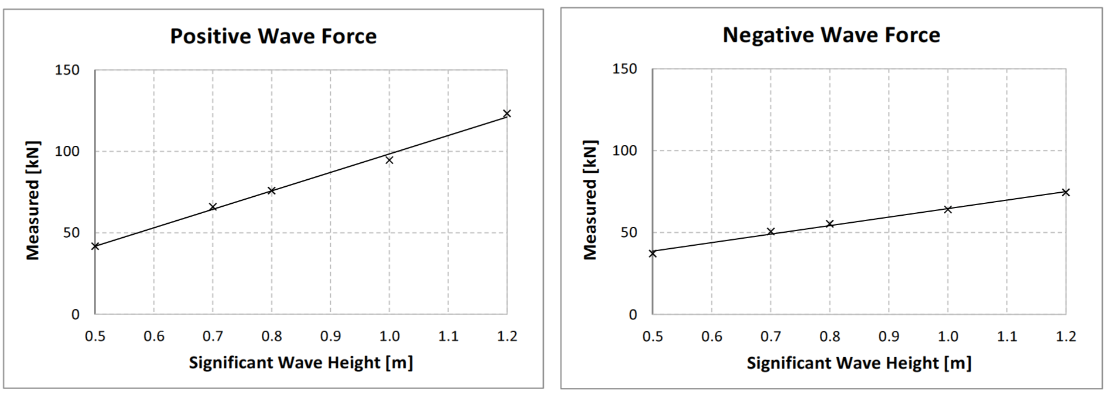

Figure 14. Positive (left figure) and negative (right figure) wave forces in function of significant wave height

2D Physical Model Test for Detached Breakwaters

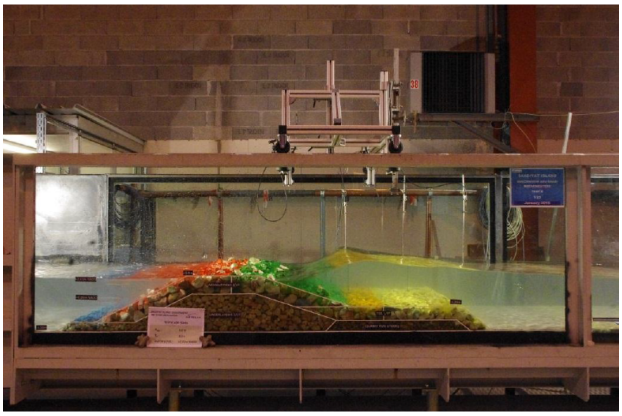

Figure 15. Model set-up (detached breakwater)

Two-dimensional physical model test of the GAD detached breakwater were conducted at a scale of 1/25 to validate the design of the structures and measure the wave transmission through the breakwaters (Figure 15). The rock grading forming the armour protection and elevated rock aprons on the sea-side of the breakwaters were designed using the Van der Meer and Van Gent shallow water equations (CIRIA/CUR/CETMEF, 2007) and Van der Meer equations for the toe structure stability (CIRIA/CUR/CETMEF, 2007). The method of van Gent and Pozueta was applied to assess the stability of the rear-slope armour (Van Gent, M.R.A and Pozueta, B. 2005).

The required rock grading for the armour protection and apron of the detached breakwater was 3-6 tonne and 1-3 tonne rock respectively. The core of the structure consists of 0.3-1 tonne rock. The sea-side and rear-side slops of the breakwaters are 1V:3H and 1V:2H respectively. The crest level is +3.0 m MSL and crest width is 10.5 m. A typical cross-section is presented in Figure 16. The width of the underwater apron was originally designed as 30 m wide.

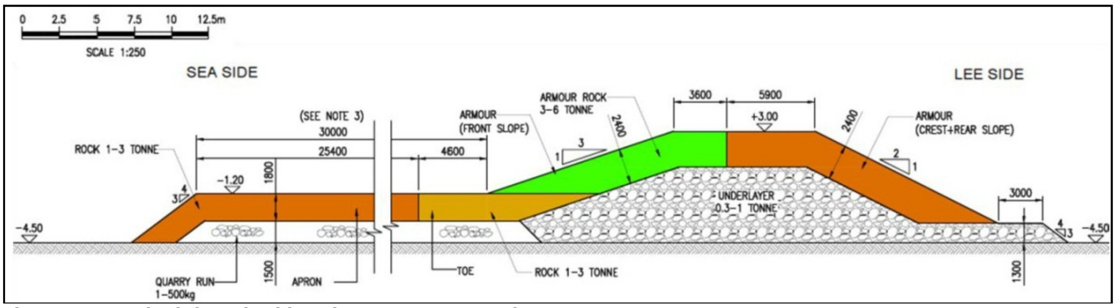

Figure 16. Typical detached breakwater cross-section

A series of nine tests were undertaken to assess the overall stability of the structure and to measure the transmitted wave on the lee side of the detached breakwater. The test series included wave conditions with a return period of 1 year, 100 years and overload conditions (120% of 100 year condition). Furthermore, optimization amendments to the typical cross-section were tested, such as shortening the underwater apron structure to 15 m instead of the original 30 m.

The overall stability of the detached breakwater complies with the design criteria of the stability of the rock armour (S_d<2), toe (N_{od}<0.5) and apron (N_{od}<2). A transmitted significant wave height of 0.7 m was measured in the lee of the breakwater for a storm event with return period of 100 years. For the detached breakwater featuring a reduced underwater apron, the transmitted significant wave height slightly increased to 0.8 m. As reducing the apron by 15 m only had a marginal impact on the wave transmission, this design optimization was validated.

Wave transmission coefficients observed in the laboratory for various design conditions and breakwater configuration varied between 0.06 and 0.29. The measured values show good correlation (see Figure 17) with the estimated values using empirical approaches (Briganti et al., 2003).

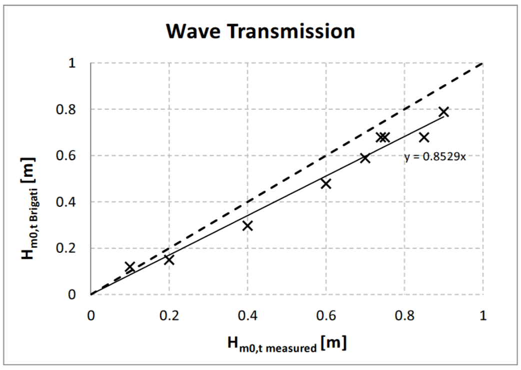

Figure 17. Transmitted wave height comparison between physical model and empirical approaches

Detached Breakwater Plan Layout Design

The wave conditions in front of the GAD seawall are a combination of the wave transmission through the detached breakwaters and the waves penetrating between the gaps of the detached breakwaters. To accurately define the wave conditions along the GAD landmass the MIKE 21 BW (Boussinesq Wave Module) numerical model was implemented to simulate the penetrating waves. The numerical model output from SWAN was used as boundary conditions and wave transmission through the breakwaters was not considered for the MIKE 21 BW modelling.

Area averaged significant wave heights were calculated at various areas along the GAD seawall. The areas were defined at a distance of a few grid cells from the model boundaries to prevent artificial boundary effects and were selected based on the seawall sections (see Figure 18, left side).

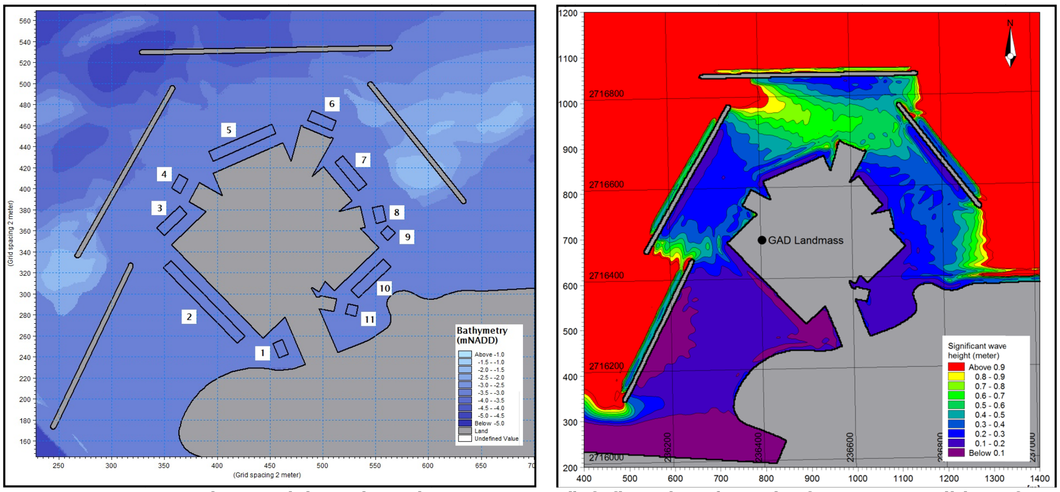

Figure 18. Wave penetration model – selected output areas (left figure) and results for wave conditions from N260° for a return period of 100 year (right figure)

With the exception of Area 6, the penetrated significant wave height at each designated area generally does not exceed 0.4 m and 0.5 m for events with a return period of 1 and 100 years respectively. For Area 6 the penetrated significant wave height is 0.6 m and 0.7 m for events with a return period of 1 and 100 year respectively.

As the combined wave height due to wave transmission and wave penetration will not result in exceedance of the allowable wave overtopping and wave forces on the wall, the breakwater cross-sections and plan layout were fixed for validation and further optimization in the 3D physical models.

Value Engineering by means of 3D Physical Model Tests

A three dimensional physical model was built in a wave basin with an area of 64 m x 40 m with a maximum water depth of 0.5 m at a scale of 1 in 38. The objectives of the 3D physical model were to evaluate the hydraulic stability of the detached breakwaters, measure the wave agitation in the zone between the detached breakwaters and the GAD landmass and to validate and further optimize the breakwater typical cross-section and plan layout. A total of ten main test simulations for three offshore wave directions were conducted with an additional 9 test simulations carried out for two modifications and four optimizations. Two main types of design optimization were tested:

- Shortening of the underwater apron; and
- Shortening of the detached breakwater length.

To validate the shortening of the underwater apron, the measured wave exposure in front of the GAD seawall after the amendment of the cross-section were evaluated in terms of wave overtopping and wave pressures. Prior to testing the shortening of breakwaters in the physical model, the wave exposure in front of the GAD seawall was estimated by combining the energy from wave penetration extracted from the numerical model with the energy from wave transmission determined from the empirical approaches. If the estimated wave exposure was acceptable in terms of wave overtopping and wave pressures, the amended breakwater layout was tested in the 3D physical model. Figure 19 presents the correlation between the observed wave height in the physical model and the wave height determined from a combination of numerical and empirical approaches. A reasonable correlation can be observed with an RMSE of 0.13 m.

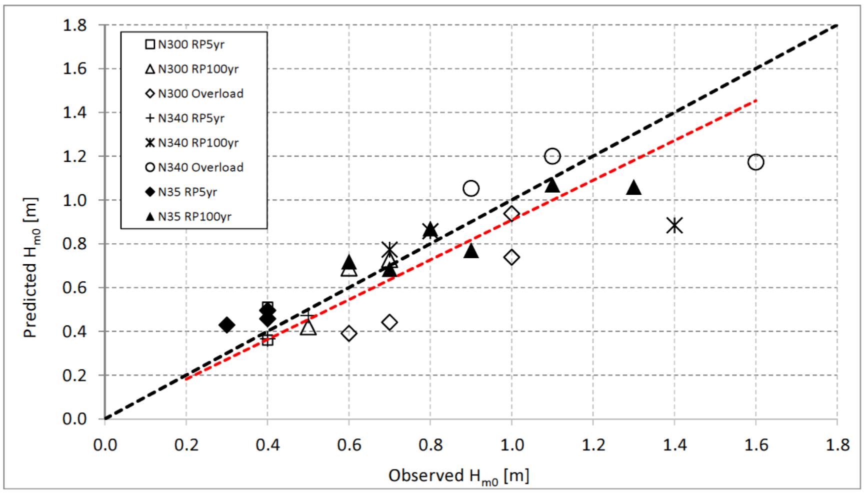

Figure 19. Correlation between wave height observed in 3D physical model and wave height estimated from numerical and empirical approaches.

Conclusion

Through a series of 2D and 3D physical model tests, a number of design optimization were achieved for the detached breakwaters:

- Shortening of the breakwater apron from 30 m to 15 m for all breakwaters based on the 2D physical model;
- Shortening of the breakwater apron from 15 m to 5 m for three detached breakwaters; and
- Shortening the length of one breakwater by 25 m.

A combination of 2D and 3D physical models at various stages of the project allowed the designer to understand and design for various coastal processes around a complex configuration of shore protection. Back-analysis has proven that a combination of numerical and empirical assessments is a suitable tool to target physical model tests efficiently in the value engineering process. Extensive physical modelling allowed the designer to provide the client with the most economical design with an estimated cost saving of 7 million USD.

Acknowledgments, Appendices, Footnotes and the Bibliography

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