OPTIMISATION OF GUGGENHEIM ABU DHABI DETACHED BREAKWATERS BY PHYSICAL MODELS

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ABSTRACT

This paper focuses on the physical modelling performed at Artelia’s hydraulic laboratory to optimise the design of the detached breakwaters of the Guggenheim Abu Dhabi museum. The museum is built on a landmass retained by a vertical seawall with varying top elevations. One of the challenges of the study was to limit wave overtopping and negative wave pressures on the seawall while minimising the visual impact of the detached breakwaters. Empirical formulae in terms of wave pressures, wave forces and mean wave overtopping discharges were developed using a 1:15 two dimensional physical model. The stability of the detached breakwaters and different breakwater geometries were investigated in a separate model at a scale of 1:25. Finally, modifications to the design were tested in 3D at a scale of 1:38. These modifications included reducing the lengths and widths of the breakwaters, and raising certain parts of the vertical seawall. From the results of the 2D and 3D physical models, design optimisations could be made resulting in a total construction cost saving of 5.75 million Euros. This case study illustrates the importance of a combined 2D and 3D physical modelling campaign to support the design and allow optimisation of marine infrastructures.

KEYWORDS: Detached breakwaters, wave agitation, wave overtopping, optimisation.

1 INTRODUCTION

The new Guggenheim Abu Dhabi (GAD) museum, designed by Gehry Partners Architects, will be situated on a peninsula at the north-western tip of Saadiyat Island near Abu Dhabi in the United Arab Emirates. The museum will be built on a landmass retained by a vertical seawall with varying top elevations. Wave exposure along the vertical seawall will be limited by four detached breakwaters surrounding the GAD landmass.

An artistic impression of the project layout is shown in Figure 1. The design of the breakwaters was conducted with respect to the wave overtopping and wave pressure criteria set along the landmass perimeter. This paper presents the 2D and 3D physical modelling campaign performed to develop the design of the breakwaters.

2 PROJECT REQUIREMENTS

The Architects of the GAD museum required a minimal visual impact of the detached breakwaters. This meant adopting the smallest possible crest configuration (lowest elevation and narrowest width) and also minimising the overall length of the structures.
The project criteria required the detached breakwaters to sustain minor damage during the 100 year return period (RP) event. A damage parameter ($N_{od}$) of 0.5 was used for the toe and a value ($S_d$) of 2 was used for the breakwater armour (CIRIA/CUR/CETMEF, 2007). 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.

The finished ground level near the water edge varied between +2.5 and +10 m MSL (Figure 2). During the 100 year RP event, the lowest ground levels were only 9 cm above the predicted water level, which highlights the importance of the breakwaters to limit wave exposure and the need of a seawall in some areas. Public access to some areas has to be guaranteed, therefore a maximum allowable mean wave overtopping discharge of 0.1 l/s/m was considered for the 5 year RP event. Some areas also contained building and landscaping elements for which no damage was tolerable. Consequently, maximum allowable mean wave overtopping discharges of 1 and 2 l/s/m were respectively considered.

At the time of the study, the diaphragm wall that retained the GAD museum landmass had partially been constructed to a level of +1.5 m MSL. The seawall was designed to allow maximum positive wave forces of +123 kN/m (landward directed). The negative wave forces (seaward directed) had to be limited to -60 kN/m for persistent design conditions (RP of 100 year) and -70 kN/m for accidental wave conditions (overload conditions).

3 SITE CONDITIONS

For aesthetic reasons, the area between the GAD landmass and the detached breakwaters will be dredged to a level of -3 m MSL. The design high water levels for the 1, 5 and 100 year RP events are approximately +1.8, +2.0 and +2.4 m MSL given the small tidal range at the project site (MHHW = +0.7 m MSL, MLLW = -0.6 m MSL), storm surges in the range of 0.6 to 1.2 m and a sea level rise for a design life of 100 years of +0.5 m.

The south coast of the Arabian Gulf is exposed to north-westerly winds that blow with speeds of up 16 m/s for RP of 1 year. For the project, offshore extreme waves (RP of 1, 5 and 100 years) were estimated via a metocean hindcast model calibrated with altimetric data. The extreme conditions were then transformed to the project site using a SWAN numerical model. Depth-limited wave conditions prevailed at the project site during extreme conditions as shown in the following figure for the 100 year RP event. The graph shows the measured (in the 3D physical model) wave height distribution which deviates significantly from the Rayleigh distribution.

The breakwaters were exposed to a maximum incident spectral significant wave height of $H_{m0} = 3.0$ m associated with a peak wave period of $T_p = 8.5$ s for the 100 year RP event, at a seabed level of -4.5 m MSL. However, as shown in Table 1, wave conditions varied across the directions of incidence and as a function of the water depth in front of each breakwater.
Table 1. Incident wave conditions

<table>
<thead>
<tr>
<th>RP (years)</th>
<th>Location</th>
<th>H\textsubscript{m0,i} (m)</th>
<th>T\textsubscript{p} (s)</th>
<th>Direction (°N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>In front of the west and south-west breakwater, at -3.5 MSL</td>
<td>2.6</td>
<td>8.0</td>
<td>300</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td>2.7</td>
<td>8.5</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>In front of the north breakwater, at -4 m MSL</td>
<td>2.2</td>
<td>7.0</td>
<td>340</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td>2.8</td>
<td>8.5</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>In front of the east breakwater, at -3 m MSL</td>
<td>1.4</td>
<td>7.0</td>
<td>35</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td>2.2</td>
<td>7.5</td>
<td></td>
</tr>
</tbody>
</table>

4 DESKTOP BREAKWATER DESIGN

The rock grading forming the armour protection and elevated rock aprons on the sea-side of the breakwaters was design using the Van der Meer and Van Gent shallow water equations (Van Gent et al, 2004) and Van der Meer equations for toe structure stability respectively (Van der Meer et al., 1995). The method of Van Gent and Pozueta was applied to assess the stability of the rear-slope armour (Van Gent et al., 2005). The required rock grading for the armour protection and apron of the detached breakwater was 3-6 tonne and 1-3 tonne rocks respectively.

The armour protection is placed directly on top of the breakwater core material. The core consists of 0.3 to 1 tonne rocks in a slope of 1V:3H and 1V:2H for the front and the rear of the section respectively. An elevated apron, with a top elevation of -1.2 m MSL, was designed in front of the breakwater to initiate wave breaking and therefore limit wave transmission. The width of the elevated apron was initially set to 30 m, which is approximately half of the 100 year RP wave length. The crests of the breakwaters were set at +3.0 m or +3.5 m MSL, which is about 0.6-1.1 m above the 100 year RP water level. The crests were 9.5 m wide based on a conceptual design and 2D physical model tests by others. Figure 4 shows a typical section of the breakwaters.

![Figure 4. Typical detached breakwater cross-section](image)

The penetrated wave energy was modelled by means of a Mike21-BW model. This numerical model accounts for wave penetration through the gaps between two breakwater heads, but does not consider wave transmission through and above the breakwaters. The transmitted wave energy was estimated using a relationship between the dimensionless breakwater freeboard and the incoming wave height, based on available 2D physical model test results.

5 METHODOLOGY

The objective of the physical model test campaign was to evaluate and optimise the desktop design in terms of seawall top elevations, 2D breakwater geometry and 3D layout and rock stability.

First, 2D measurements of wave loads and wave overtopping at the vertical seawall were performed at a Froude scale of 1:15 for spectral significant wave heights in the range of 0.5 to 1.2 m associated with the 100 year RP events, without the protection of the detached breakwater. No impulsive wave forces were expected on the wall therefore Cauchy similitude was not required. A tensor scale measured wave forces and moments on a finite slice of the seawall. Pressure sensors were also used to provide the vertical distribution of the force. The wall instrumentation setup is shown in Figure 5. Piezoresistive...
sensors (Ø 19 mm, fr > 1000 Hz) measured pressure loads with an achieved precision of +/- 0.1 kPa. The tensor scale measured with an achieved precision of +/- 0.2 kN. Peaks and lows were identified using an algorithm using threshold values of ∆P = 0.1-0.2 kPa, ∆F = 0.5-2 kN, ∆M = 2-6 kN.m and ∆T = 0.2 s. Background noise was removed if identified. The number of peaks and lows and several statistical values were calculated, and each variable was also plotted at the time of the peak wave force. The goal was to estimate the maximum significant wave height allowable at the front of the seawall to keep negative and positive wave forces under the maximum threshold values set for the project. A relationship between the wave force applied on the wall and the total significant wave height measured at the front of the wall was also developed: F_{2D} = function (H_{m0\text{total}}).

The measurements of wave overtopping were conducted at two seawall elevations. A relationship between the mean wave overtopping discharges and the ratio between the significant wave height and the seawall freeboard was derived based on the results. The objective was to define a minimum seawall top elevation to comply with the project overtopping criteria considering the maximum permissible wave height in terms of wave loads. A relationship between the mean wave overtopping discharge at the wall and the total significant wave height measured in front of the wall was also developed: Q_{2D} = function (H_{m0\text{total}}).

A second set of 2D tests examined the stability of the breakwater cross-section at a Froude scale of 1:25. Reynolds numbers were larger than 30,000 in the armour protection and in the exposed layer of the rock berm. A turbulent regime was therefore maintained in these rock layers which guaranteed limited model effects due to the increased amount of viscous forces at model scale. Damages were evaluated in terms of the N_{dd} damage parameter for the elevated apron and S_{d} parameter for the armour. The levels of accumulated damages were compared with the project stability criteria. These tests were also used to measure the transmitted waves in the lee of the detached breakwater section in order to achieve a design which complied with the maximum permissible wave height in terms of wave loads.

A 3D model of the entire museum landmass and breakwater layout was then constructed with the correct bathymetry at a scale of 1:38 in a 30 by 40 m wide wave basin. Long crested waves were simulated for three nearshore incident directions of wave propagation: 300, 340 and 35 °N directions.
In a Froudian model, excessive energy damping occurs in the breakwater core in the case of laminar and intermediate regimes because of the increased amount of viscous forces compared to reality. Particular attention was given to limit this kind of model effect which would result in an underestimation of the transmitted waves in the lee of the structures. Maximum prototype velocities were estimated in the range of 0.2 to 0.3 m/s using IH2VOF RANS model (Figure 6). Hydraulic gradients were analytically estimated and compared with the prototype values which varied in the range of 0.2 and 0.3 (Eugelund, 1953). If the model values exceeded the prototype values by more than 5%, larger rocks were used (Klinting and Jensen, 1983). A scale of 1:34 instead of the Froude scale of 1:38 was applied for the 3D but no correction was necessary for the 2D.

6 DISCUSSION

6.1 Breakwater stability

The 2D tests demonstrated that all rocks were stable, and showed that smaller rocks could potentially be adopted for the berm apron as a maximum $N_{od}$ value of 0.1 was recorded. This was confirmed by the 3D tests at all breakwaters. The 3D tests also demonstrated that the degree of damages was much smaller for the three breakwaters partially sheltered by the north breakwater and located in shallower waters. However, the use of smaller rocks in these parts of the breakwaters was not found to result in cost savings based on construction considerations for this project.

Figure 7 presents the maximum accumulated damage level which was recorded across all directions at completion of the 100 year test.

![](image)

6.2 Wave agitation around the museum platform

Wave agitation around the museum platform was the result of wave transmission through and over the breakwaters, and wave penetration through the gaps between the breakwaters. For the 100 year RP event, the proportions of each component were respectively estimated to be in the range of $H_{m0} = 0.2, 0.6$ and 0.8 m by comparison between the 2D tests with and without transmission over the breakwater and the 3D tests.

The 2D reflection coefficients were estimated to be $C_r = 0.79 - 0.84$ for $H_{m0\ total} = 0.6 - 1.6$ m in the case of no overtopping, using the 3 points method of Mansard et al (1980). In the 3D tests, reflections were multidirectional. They originated from the GAD walls with various orientations and to a lesser degree, from the inner slopes of the breakwaters. Therefore, in 3D the contribution of wave energy directed toward the wall in $H_{m0\ total}$ was smaller than in 2D.

The wave pattern was irregular, formed by the superposition of the incident waves and the multiple reflected waves. The total spectral significant wave height ($H_{m0\ total}$) varied in space and several peak frequencies of energy were identified. The $H_{m0\ total}$ values were therefore seen as indicators of the wave agitation. The following figure shows the measured
spectrum in the lee of the breakwaters for the 100 year RP event (300°N) and a photo showing the sea state in the model.

Figure 8. 2D spectra at the lee of the breakwaters for the 100 year RP event (300°N)

A total of 14 wave probes were deployed to monitor the wave agitation around the museum platform. \( H_{\text{mf}} \) varied in the range of 0.1 to 0.7 m for the 5 year wave conditions, 0.1 to 1.4 m for the 100 year conditions, and 0.7 to 2.2 m for the overload conditions. A stronger agitation was observed at the interception between wall sections as a result of the wave energy focus, as documented in (Goda, 1985). An area semi-enclosed by wall sections also experienced further agitation because of a resonance phenomenon.

6.3 Pressure & forces that apply on the seawall

Loads on the walls were pulsatile (no air entrapment). A comparison of the linearly-integrated pressure value (I) with the force measured by the tensor scale (F) was carried out at times of maximum forces. I-values were within 10 % of the F-values, which indicated that the recorded pressure loads (magnitude and location) provided a correct representation of the force distribution on the wall. The 10 % difference was allocated to measurement error and the slight non simultaneity between the maximum pressures and forces. Measurement quality was controlled by comparison with analytical values (Goda, 1985).

The following figure shows maximums and time-variations of the pressure (Ps), Force (F) and moment (M) with the water level (WL, measured at the front of the wall) at time of the peak force (F\(_{\text{max}}\)) for \( H_{\text{mf}} \) incident = 1.0 m (equivalent to \( H_{\text{mf}} \) total = 1.3 m) .
Figure 9. Maxima of Ps and F (left) and time variations at the time of $F_{\text{max}}$ for $H_{\text{m0 incident}} = 1.0$, $T_p = 8.5$ s

The 2D results allowed the development of relationships between $F$, $P$ and $M$ and $H_{\text{m0 total}}$ as shown in the following figure.

2D $H_{\text{m0 total}}$ threshold values were calculated to comply with the project criteria in terms of forces on the wall. The negative hydrodynamic forces were found to be the more critical with $H_{\text{m0 total}} < 1.2$ m for the persistent extreme design conditions (100 year RP conditions or less) and $H_{\text{m0 total}} < 1.4$ m for the accidental design conditions (overload conditions). $H_{\text{m0 total}} < 1.6$ m satisfied the criteria in terms of positive hydrodynamic force.

6.4 Wave overtopping at the vertical seawall

Wave overtopping consisted of green water with rare occurrence of sprays and no jets. On the basis of the 2D test results, the following empirical relationship was developed ($R^2 = 0.98$):

$$ Q = 0.03 \times \exp(-5.5 \times \frac{R_c}{H_{\text{m0 total}}}) \times \sqrt{g \times H_{\text{m0 total}}^3} $$

Where: $Q$ (l/s/m) is the mean wave overtopping discharge, $H_{\text{m0 total}}$ (m) is the total spectral significant wave height (along the vertical seawall) and $R_c$ (m) is the wall freeboard (Figure 11).

The minimum seawall elevations were calculated to comply with the overtopping project criteria considering the maximum permissible wave heights in terms of wave forces, which led to a revision of the wall elevations for some areas.
In the 3D model, which incorporated the revised seawall elevations, overtopping discharges were measured at 11 points along the wall. They were generally significantly lower than the 2D predictions as expected because of wave obliquity and to the lesser amount of wave energy directed toward the wall in 3D compared to in 2D as the results of the multiple reflections. Further optimisation to the detached breakwaters was therefore possible.

An exception was at the wall interceptions, where a minimum wall elevation of +4.5 m MSL was required because of a strong wave focus in these areas as mentioned previously.

7 DESIGN EVOLUTION

A minimum required seawall top level of +3.3m MSL was determined based on the 2D overtopping model results. 2D modelling of the wave transmission allowed selecting one breakwater geometry out of four tested geometries which investigated the effect of the crest configuration (+3 and +3.5 m MSL) and apron berm configuration (30 and 15 m length, -1.2 m or -1.2 & +0.6 m MSL). The stability of the breakwater section was validated in 2D and demonstrated the potential of using smaller rocks. The breakwater layout was then validated in 3D. Five potential layout changes which consisted of shortening the breakwaters by up to 50 meters and some aprons by 10 meters were elaborated on the basis of the results of the main programme. In the meantime, walls were raised at some vulnerable areas exposed to strong 3D effects.

The total breakwater construction cost savings allowed were of approximately 5.75 million Euros compared to the initial desktop design, which is about 69 and 34 times the costs for the 2D and 3D tests respectively. Most importantly, the physical campaign demonstrated shortcomings in the wall design which could have resulted in costly remediation measures in the future.

The following figure shows the final project layout:
8 CONCLUSION

This paper focused on the extensive physical modelling campaign performed to support the design of the detached breakwaters that will protect the new Guggenheim Abu Dhabi museum. One of the challenges of the study was to limit wave overtopping and negative wave pressures on the seawall while limiting the size of the breakwaters. The mathematical relationships derived using the 2D test results were critical in defining the wave loads and wall elevations along the entire museum platform perimeter. The 3D model allowed for significant cost savings by reducing the lengths and widths of the breakwaters and also highlighted some vulnerable points in terms of overtopping. This case study illustrates the importance of combining the 2D and 3D model results to develop an optimised design for complex seaside structures.

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