PHYSICAL MODELING SUPPORTING DESIGN AND CONSTRUCTION OF LOW CRESTED BREAKWATER FOR THE AYIA NAPA MARINA, CYPRUS

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ABSTRACT

SmithGroupJJR undertook the design a new marina and accompanying land development at Ayia Napa, Cyprus. The marina features a 600-slip mega-yacht harbour framed by a large shoreline protection scheme comprised of wave absorbing block walls, revetments, breakwaters, and pocket beaches. Significant upland development, including two, 25-story towers and residential villas are also included in the design, with some of the new development near the sheltering breakwater. An innovative one-kilometer long, low-crested breakwater with tetrapod armor and a wide rock berm was designed to protect the harbor and land development. One of the primary design goals was maintaining a crest height low enough to provide the villa owners and marina users with unobstructed views of the sea. Therefore, a key element in the design was to limit the amount of wave overtopping that could pass over the low crested structure and potentially threaten the villas, yachts, cars and people on the lee side of the breakwater. The maximum overtopping flow rate was of interest rather than the mean time-averaged flowrate, since the maximum flow rate is more closely linked to risks to people and property.

A physical model study of a revised breakwater design was carried out at the National Research Council of Canada (NRC). A two-dimensional physical model of an idealized foreshore at the project site was constructed at a geometric scale of 1:42.2 in a 63m long by 1.22m wide wave flume. Scale models of two breakwater cross-sections (one in shallow water, the other in deeper water), due to the variable bathymetry, were constructed and exposed to scaled reproductions of the design-wave conditions forecast for the site. The physical model provided a good simulation of the important hydrodynamic processes influencing the stability and overtopping of the tetrapod armor layer, including nearshore wave transformation, wave breaking, wave run-up, and interstitial flows through the armor and filter layers.

The Ayia Napa breakwater is currently under construction, with approximately 50% of the breakwater constructed to date and completion expected by April 2019. Breakwater construction has been closely supervised, assuring that it meets the conditions specified by the design and observed in the physical model. The innovative double berm, low crested design approach of the Ayia Napa Marina breakwater provides a casebook example of how to achieve a harmonic, high-performance breakwater integrated with its landscape and environmental context, as well as highlighting the value of using a physical model to deal with design changes that arise during construction.

1 INTRODUCTION

The Ayia Napa Marina breakwater was designed based on calculations and physical model tests carried out in March 2015 at Wallingford, England (Boshek 2015). Flume physical model tests at scale 1:45.1 were done to define a stable breakwater cross-section consisting of 8 m^3 Tetrapod armor units as armor layer, with a 10.2 m wide berm, and a 7.8 m-high crown wall, which produced the desired overtopping rate. The cross-section design was confirmed in a 3D physical model test, where head and toe instability were observed. The cross-section was adapted implementing a trenched toe solution with larger 10 m^3 tetrapod units at the head of the breakwater.

The master plan of the marina was further developed, locating villas within the marina, its commercial areas, and two residential towers. The height of the crown wall, 7.8 m above the low-water tide level, became a problem obstructing the views of the marina and the villas. Once construction began in October 2016, the owner requested the analysis of a possible alternative that would reduce the height of the breakwater to improve the views from the residential villas and the marina. SmithGroupJJR developed two possible solutions to reduce the height of the crown wall while maintaining the design overtopping rate: a submerged reef in front of the breakwater, or a wide-berm breakwater. The wide-berm breakwater was selected as the most feasible solution based on the environmental and permitting conditions of the project.

A key design goal for the breakwater was to maintain a crest height low enough to provide the landside villa owners and marina users with unobstructed views of the ocean. Therefore, it was important to limit the amount of wave overtopping that could pass over the structure and pose a threat to the villas, yachts, cars and people on the lee side of the breakwater. The maximum-overtopping flow rate rather than the mean time-averaged flow rate was utilized as a critical design criterion, since it is more closely linked to the risks posed to people and property.

The first step in designing the new low-crested breakwater cross-section was to use the Neural Network Overtopping design tool from TU Delft to estimate the berm width and structure geometry. The berm stone sizes were determined based on the work completed by Van Gent (2013). The newly proposed low-crested breakwater cross-section consists of two layers of 8 m^3 tetrapod armor units placed on a structured grid with a front slope of 1:1.33. The crest of the breakwater extends to an elevation of +4.6 m above the design waterline and a 20.5 m wide berm, 10 meters of a Tetrapod Berm and 10.5 meters of a 4-ton rock berm, backed by a crown wall at the same +4.6 m elevation (see Figure 1).

Due to construction issues while trying to excavate a toe trench in calcarenite bottom, the feasibility of eliminating the toe trench was also investigated in the physical model. Given the various water depths where the breakwater's toe trench would be located and recognizing the reduction in wave forces with increasing water depth, a shallow-water portion and a deep-water portion of the breakwater were physically modeled with alternative solutions to the toe trenching. The need to secure the toe of the breakwater was verified by the physical model.

In order to investigate these two proposed design changes—lowering the crown wall height and removing the toe trench—physical model tests at a scale of 1:42.2 were carried out at the Ocean, Coastal and River Engineering Research Centre of the National Research Council of Canada. The performance of the breakwater's cross-sections was assessed by observing the stability of the armor units and amount of overtopping during exposure to a series of irregular wave conditions and elevated water levels representing design storms. The effects of different widths of the top "berm" on structural stability and overtopping rates was explored using a double-berm-width tray system to optimize use of the laboratory time. Each test series generated much information with respect to the interaction of the extreme design waves with the foreshore and the breakwater (wave breaking, run-up, and overtopping), and the response of the breakwater to this forcing (stability of the armor and the resulting overtopping discharges). This physical modeling was crucial in refining and confirming the proposed design changes developed to accommodate the site conditions encountered during construction of the breakwater. The efficiency of the model study led to reduced downtime in the field while these design sections were verified and optimized.

Several large storms have been encountered to date during construction, which have allowed for verifying the design parameters as observed in the physical model tests. A wave gauge was installed at the project site in a location corresponding to the location of the wave paddle in the physical model. Overtopping rates have been measured, the behaviour of the trenched tetrapods have been documented, and the observed performance of the portions of the breakwater built thus far are in agreement with the physical model results and design expectations.

2 PROPOSED DESIGN OPTIMIZATIONS

2.1 Design Conditions

The site's deep-water wave and wind conditions were obtained from Mediterranean hindcasts and reported in the SmithgroupJJR 2013 Wave Conditions Report. The analysis showed the occurrence of waves from different directions ranging from east to west with 100-year-return-period significant wave heights of up to 7.2 m. The wave conditions for the deep-water conditions are summarized in Table 1.

Table 1: Deepwater Design Wave Conditions

Offshore Wave	1-year		10-year		25-year		50-year		100-year	
	Hs(m)	Tp(s)	Hs(m)	Tp(s)	Hs(m)	Tp(s)	Hs(m)	Tp(s)	Hs(m)	Tp(s)
West	3.5	8.4	4.9	9.4	5.5	9.7	6.0	10.0	6.5	10.3
WSW	4.5	9.1	5.9	10.0	6.4	10.2	6.8	10.5	7.2	10.7
SW	3.4	8.3	4.5	9.1	4.9	9.4	5.2	9.6	5.5	9.7
SSW	2.2	7.2	3.8	8.6	4.4	9.0	4.9	9.4	5.3	9.6
South	1.2	5.9	2.0	7.0	2.4	7.4	2.6	7.6	2.8	7.8
SSE	1.0	5.5	1.7	6.6	2.0	7.0	2.2	7.2	2.4	7.4
SE	1.0	5.5	2.1	7.1	2.5	7.5	2.8	7.8	3.1	8.1
ESE	0.8	5.1	1.3	6.0	1.5	6.3	1.7	6.6	1.8	6.7
East	1.0	5.5	1.5	6.3	1.6	6.5	1.8	6.7	1.9	6.8

The deep-water wave conditions were numerically modeled to the project site using state-of-the-art, steady state spectral model SWAN. The physical model boundary conditions were developed based on the transformed wave heights obtained from the spectral model results. Wave conditions in the physical model were measured at six specific locations using capacitance wave probes, including one probe at the -20m depth contour. The waves used in the physical model were calibrated by adjusting the command signals used to drive the wave generator so that the wave conditions measured at the -20 m depth contour were in agreement with target-wave conditions derived from numerical wave modelling completed by SmithGroupJJR. The target wave conditions for each design storm at the 20 m water depth contour are shown in Table 2. The other five wave gauges were placed at specific depths along the model's bathymetry, including one gauge placed near the toe of the breakwater structure.

Table 2. Specified wave conditions at -20m contour.

Return Period	H_s (m)	T_p(s)	SWL (m CD)
1	2.58	8.3	0.91
10	3.56	8.8	0.91
50	4.51	9.7	0.91
100	4.85	9.9	0.91
20%	5.82	10.8	0.91

2.2 Marina Layout Design

Conceptual design for the basin and breakwater at the Ayia Napa Marina was first evaluated and refined by a series of 2D flume and 3D basin physical model tests at the Laboratories of HR Wallingford, in England. In addition to the physical modeling, numerical modeling was performed at the same time by SmithGroupJJR. The testing addressed; the sizing and stability of the breakwater and the armor units, wave penetration and berthing tranquility in the basin, wave overtopping of the breakwater, wave absorption and reflection damping methods within the basin, and water quality and circulation in the marina.

Due to the observed results of the 3D physical model, additional numerical modeling was conducted to address structural changes to the design of the basin, which either corrected unexpected deficiencies or in some cases further enhanced performance. The original basin plan remains essentially as envisioned in both size and configuration, with minor changes undertaken to assure safety and performance of the facility.

Figure 1. 3D physical model conducted by HR Wallingford for the Marina layout design.

The numerical model NOWT-PARI (Hiraishi 2002), developed by the Port and Airport Research Institute of Japan, was utilized to numerically transform the extreme wave events for modeling the design changes. The events modeled in the 3D physical laboratory were used to calibrate and validate the numerical model. NOWT-PARI is a state-of-the-art, completely non-linear, Boussinesq-type wave transformation model that calculates the water-surface elevation for every time-step instance; this model also considers partial wave absorption-reflection, wave diffraction, wave shoaling and refraction, and wave breaking and nonlinear interactions. The first stage of the numerical modeling study was to reproduce the observed results in the 3D physical model for validation purposes. The same wave input conditions used in the physical model were also used in the numerical model by implemented an irregular, directional wave maker located at the south boundary of the model. The numerical model was set up using a 4 by 4 m regular grid with a partial reflection coefficient of 0.5 applied to the structures. The partial reflection coefficient was then calibrated to reproduce, as closely as possible, the wave conditions as measured in the physical model tests. A reflection coefficient of 0.3 was specified at the beach areas, and sponge energy-absorbing boundaries were implemented at the lateral extents of the numerical model domain. The model time-step calculation was set to 1/600 of the peak spectral period. The resulting numerical model was run for each extreme event for the same duration as the physical model. The numerical model results are shown in figure 2.

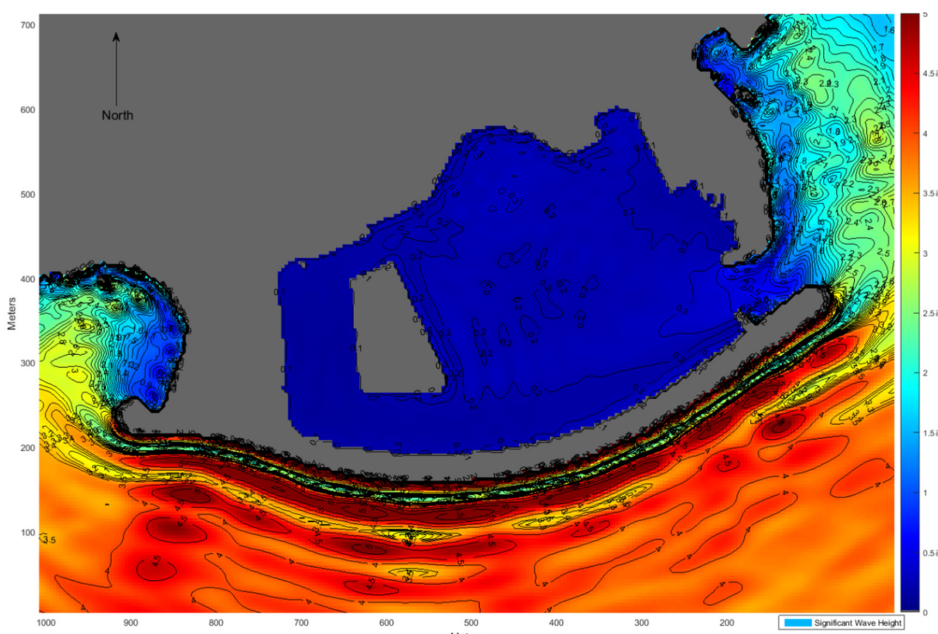

Figure 2. Validated Numerical Model of the Marina Layout

The wave-calmness conditions within the basin of the revised marina layout were obtained and verified with the calibrated and validated state of the numerical model. The resulting layout changes included a geometrical modification to the south breakwater, a revised curvature of the eastern breakwater, modified island location, and complete closure of the west circulation channel, which will affect how waves enter and behave within the marina basin. The final modified marina layout is shown in Figure 3.

The results from the numerical model showed that deep-water waves approaching from the SSW direction produced the largest wave agitation in the basin. The SSW waves had the largest wave heights and periods closest to the project site and presented the largest amount of wave energy in the basin; these results were similar to what was found in the physical model study. To minimize the wave energy entering the basin, a special wave-energy-absorbing block wall was created and implemented throughout the basin perimeter.

The resulting wave agitation in the basin was compared to the tranquility criteria to determine if the basin would comply with the climate targets. The critical wave gauging locations, where the wave climate criteria were compared with the modeled waves, were located on the slips closest to the marina entrance.

Figure 3. Final Marina Layout Design

2.3 Breakwater Cross Section Design

The preliminary design of the breakwater was tested in a physical model conducted at HR Wallingford (Boshek 2015). A detailed analysis on the overtopping rates produced by Accropode armor and Tetrapod armor units was undertaken. The tests revealed that 19.2-ton Tetrapod armor units provided smaller overtopping rates compared to the Accropode units. The conditions at the site present variable water depths ranging from 4.5m to 12m with a sharp slope between the 8m and 5m contours. This condition creates plunging waves which break on the breakwater slope, significantly increasing the overtopping rates. Due to these breaking conditions, smaller waves were observed to ride up the filter layer through the pores in the armor units. While not visible, this same process occurred for larger waves, which suggests that runup height, and therefore overtopping, was much larger in one-layer systems. This finding is also consistent with the predictions of overtopping behavior for different armor types by Cox and Clifford (2014), who developed a means to correlate overtopping volumes to wave transmission; they found that tetrapod armoring produced the least overtopping compared to single layer elements or even rock armor for the same overall breakwater cross-section. Based on the results of both tests and the perceived changes that would be required to effectively reduce the overtopping, the breakwater cross-section was designed with a double-layer of 19.2-ton Tetrapod armor units and a crest height of 7.8 m above the low water level.

The Tetrapod placement specified for the breakwater was based on a recommended pattern by FUDO TETRA, reproduced in figure 4. This placement pattern provides increased interlocking between the units, which adds an increased amount of stability to the weight of the armor units. Placing Tetrapods in this specific pattern required extra attention by the contractor compared to the traditional random Tetrapod placement.

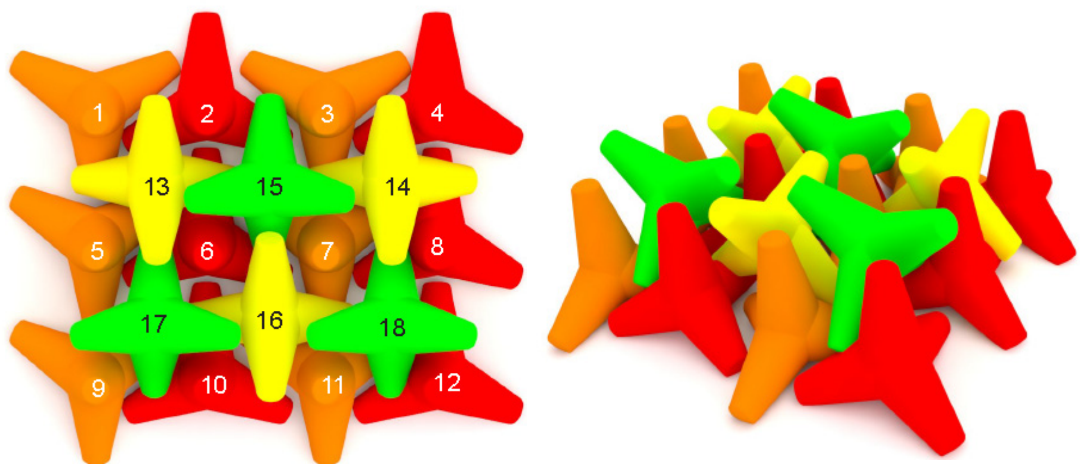

Figure 4. Tetrapod Placement Pattern

Two issues with the breakwater cross-section were observed during the 3D Physical model tests. The first issue was observed at the shallow sections of the breakwater (sections where the water depth is less than 7 m) where the breaking waves created a down rush causing the toe Tetrapods to slide seaward. This issue was not observed in the deeper water sections of the breakwater where the wave drawdown was not as severe. The second issue was poor interlocking of Tetrapods at the head of the breakwater. The original design specified 19.2-ton Tetrapods at the head of the breakwater, and these units were pulled out of position during the model tests. Furthermore, green-water overtopping was observed at some sections of the breakwater due to three-dimensional wave propagation effects due to the shape and variable bathymetry; as a result, any intent to reduce the crown height of the breakwater was abandoned.

The toe sliding had not been observed in the first 2D flume cross section tests, as the breakwater section modeled at HR Wallingford was chosen where the highest waves were observed, in the deeper water section. The issue only became apparent during the 3D physical model tests, where down rush from breaking waves at the shallower portion caused sliding of the Tetrapods. The toe sliding was corrected by introducing a toe trench that can contain the two toe Tetrapod units. The instability of the Tetrapods in the head of the breakwater was corrected by increasing the size of the Tetrapods to 10 m^3, or 24 tons. The crown height of the breakwater remained at 7.8 m above the water level.

Construction of Ayia Napa Marina started in September 2016. During the initial stages of construction, the excavation of the toe trench proved to be a difficult task due to the nature of the soil and the method used. The crown height of the breakwater also obstructed the views from marina, villas and commercial areas. SmithgroupJJR was asked to analyze the possibility of reducing the crown height of the breakwater.

SmithGroupJJR developed two alternative designs to reduce the height. The first alternative created a wider breakwater with a berm to reduce wave overtopping. The second alternative created an offshore, semi-submersed reef to cause wave breaking that would reduce the wave energy at the breakwater. Due to navigation, environmental, and permitting concerns, the submerged reef alternative was abandoned. The wide berm alternative was preliminary designed and a second physical model was carried out at the National Research Council of Canada, NRC.

Based on the results of the initial 2D and 3D physical models conducted by HR Wallingford, calculations were carried out to determine a revised breakwater cross-section including a wide berm. Ideally, a wide berm with a lower crest height would provide the same overtopping rates as the initial design with the higher crest. A target breakwater crown-wall elevation of +5.5m from the low tide water level was selected and overtopping for different berm widths was estimated, the low crested breakwater cross section tested in the physical model is shown in figure 5.

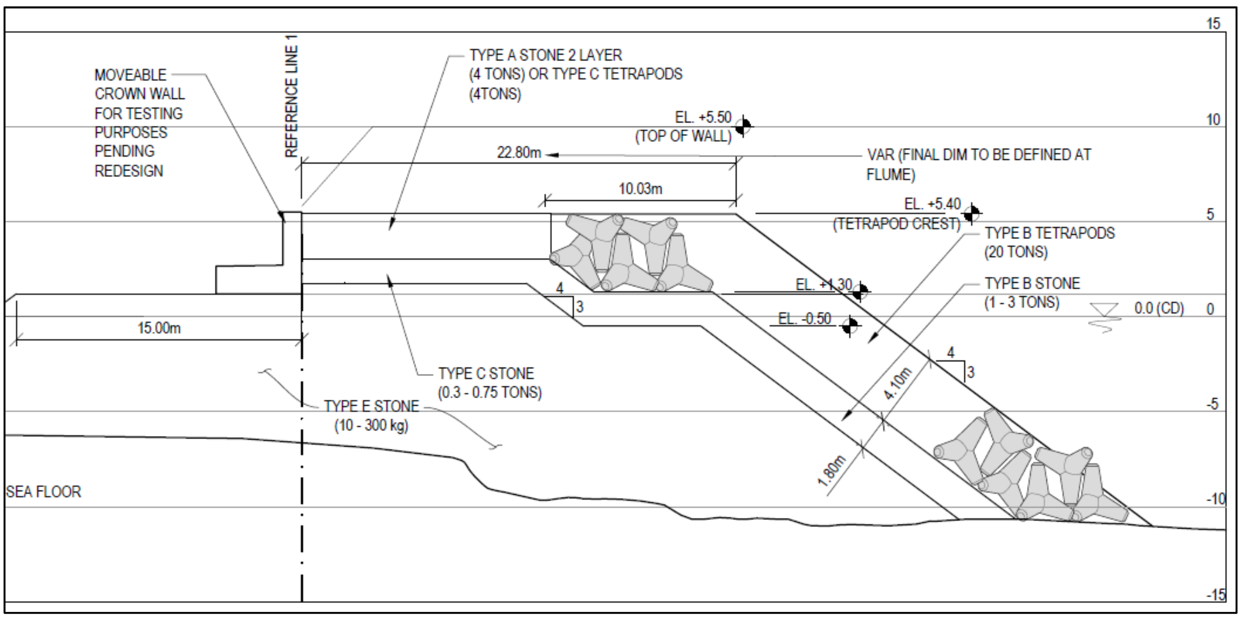

Figure 5. Preliminary Wide Berm Low Crest Breakwater Cross Section

The specific berm-type breakwater section selected for design is unprecedented and out of the range of validity for most commonly used design equations. Thus, a Neural Network method was used to estimate an initial cross-section for the revised breakwater. Even with this method, mathematical uncertainty about the results was significant, making physical model tests required for designing the breakwater. The estimation of mean overtopping was done using the Neural Network software developed by WL | Delft Hydraulics. The Neural Network predictions were only used as first estimates of mean overtopping discharges.

The initial results of the Neural Network software indicated that a 25 m wide berm with Tetrapods on the slope and stone on the inner berm area would produce a lower overtopping discharge than the original 7.8m high cross section. The calculated overtopping discharge rate for the original cross section was estimated at 1.64 l/s/m, while the 25 m berm was estimated to be 1.09 l/s/m. This indicated that a berm width of 25 m should produce similar amounts of overtopping compared to the +7.8m crest height of the previous designed cross-section. However, the 95% confidence-level calculation still showed a slightly higher overtopping rate. The 95% overtopping for the existing 7.8 m crest elevation was 5.31 l/s/m, and for a 5.5m crest elevation with a 25 m berm was 6.45 l/s/m. It was expected that higher individual waves might produce higher individual overtopping rates.

Van Gent's (2013) method to determine the stability of rubble mound breakwaters with a berm was applied to estimate the size of stone required in the horizontal berm behind the Tetrapods, shown in figure 5. This is a horizontal berm located at the crown of the breakwater, where the usual stability equations are inapplicable; the stones must withstand the force of jets of water from breaking waves. The calculation indicated that the size of the berm stone can be reduced by a factor of 3.9 to the equivalent required stone on the slope. Considering that the Ayia Napa berm is out of the validity range stated in Van Gent (2013), an additional safety factor was applied and 4-ton stones where selected. Given the horizontal placement, where interlocking can't be taken into account, they can be substituted with 4-ton armor units (horizontal placement).

3 PHYSICAL MODEL TO SUPPORT BREAKWATER CROSS-SECTION DESIGN CHANGES

Additional physical model tests were commissioned at the National Research Center of Canada to finalize the design changes to the breakwater cross section in November 2016, while the construction of the marina proceeded by creating the access to the breakwater. The available time to finalize the design changes was short as the Contractor progressed and therefore an expedited physical modelling study was undertaken.

3.1 TESTING PROGRAM AND RESULTS

The principal objective of the physical model study was to assess the performance of breakwater toe stability and overtopping in extreme conditions associated with storms of varying return periods. Breakwater performance was primarily assessed by observing the stability of the armor units and the amount of overtopping for various crest widths and corresponding crown wall positions. The overtopping criterion that was applied to this study is as follows:

- 1:100 year mean discharge < 0.1 L/s/m
- 20% overload storm mean discharge < 1.0 L/s/m

In addition to the mean discharge criteria, a maximum single overtopping event of 5000 L/m was also imposed to examine the breakwater performance.

Two simple, accurate and reliable overtopping measurement systems were developed for use in the model. The overtopping system consisted of a water storage reservoir, a capacitance wave gauge to measure the level of the water in the reservoir and a tray that collects all of the overtopped water over a portion of the breakwater cross-section and carries it into the water storage reservoir. The reservoirs were placed on the leeside of the model breakwater and ballast to prevent them from moving. The collection trays were placed immediately behind (and slightly below) the crown wall to capture all overtopping along the width of the conveyance trays. This system is able to capture single overtopping events, which was of importance to the testing program as the maximum overtopping flowrate was used as a critical design criterion compared to the mean time-average flowrate.

A photographic damage analysis system comprising of two remotely-operated digital cameras was used in this study to monitor the movement of armor units on the surface of the breakwater. The two cameras were securely mounted above the flume and aimed to view the seaward breakwater slope and the breakwater crest. Since each camera remained fixed throughout a test series, the movement of armor units could be detected by comparing photographs taken at different times. In addition, a video camera with remote pan, tilt and zoom capabilities was installed outside the flume (looking through the flume's glass viewing windows towards the model breakwater) and was used to digitally record all tests.

3.2 Shallow Water Cross-Section

The shallow breakwater cross-section, shown in figure 6, was tested first. The first tested design features two layers of 20-ton Tetrapod units and a single row of 25-ton Tetrapods at the toe of the breakwater. The initial breakwater design called for a toe trench to be constructed in which the Tetrapods along the toe would be placed. However, during early stages of constructing the breakwater, the hard rock experienced at the site led to difficulties in excavating the trench. One of the driving forces of the physical modelling studies was to investigate the possibility of eliminating the toe trench and instead use larger

Tetrapod units to secure the toe. The crest height of the breakwater was +5.4 m, the height of the crown wall was +5.5 m, and the toe of the breakwater was located near the -6 m depth contour. The design and a photo of the final constructed shallow water cross-section breakwater are shown in **Figure**. The pink Tetrapods represent the 25-ton toe armor units.

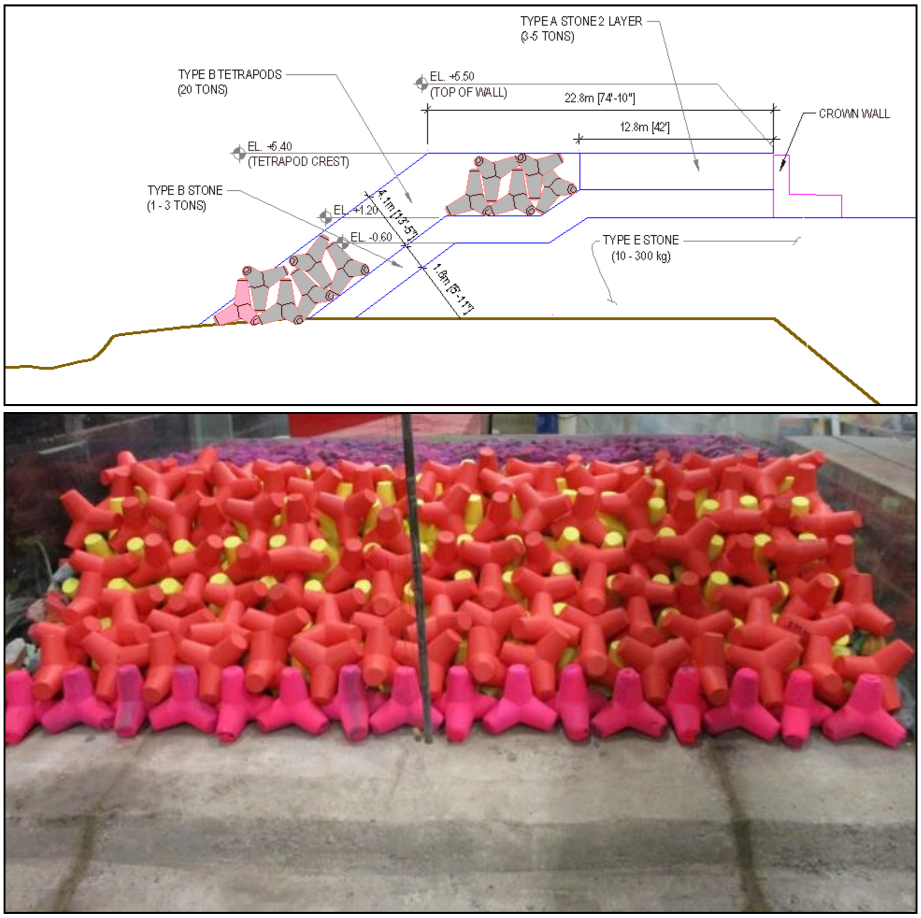

Figure 6. Top – shallow water cross-section design; bottom - constructed shallow-water cross-section.

3.2.1 Toe Stability

The breakwater was exposed to the 1-year and 10-year return period wave conditions shown in table 2. During the course of the 1-year storm, the toe units near the center of the structure shifted seaward, likely caused by drawdown from the larger waves dragging the toe units. After the 3-hour storm duration, the toe row of Tetrapods appeared to have moved approximately 2.1m (0.05m model scale) seaward in the center of the structure. The breakwater was then subjected to the 10 year storm during which the toe shifted even further seaward and caused the armor units on the slope of the breakwater to slump (see Figure 6) and a Tetrapod unit was plucked from the second layer on the toe and was deposited offshore of the breakwater. The amount of armor unit displacement was considered a structure failure and therefore the structure was not tested any further.

The breakwater was rebuilt using larger 30-ton Tetrapod units for the toe to try and increase the toe stability. The larger 30-ton toe units appeared to have been more stable under the 1-year return period storm compared to the previously tested 25-ton units, showing no movement during the storm. Seaward movement of the toe was observed during the 10-year storm. The rundown from a large wave initially pulled two to three of the toe units seaward and subsequent large events caused further displacement of the toe. After the 10-year storm, there were a total of four toe units that were displaced approximately

0.5m seaward. This toe displacement caused a slight slumping of the second layer armor units, directly above the displaced toe units. The 50-year storm caused further displacement of the toe; the toe units that moved seaward during the 10-year storm were dragged even further offshore (resulting in a total displacement of approximately 1m). The maximum wave rundown on the face of the structure would almost fully expose the toe units during large wave events. This rundown appeared to exert a large slope-normal force on the toe units, causing the large 30-ton units to be displaced seaward, which in-turn caused additional slumping of the Tetrapods on the slope of the breakwater particularly in the first two to three rows. The 100-year storm caused a failure of the breakwater section with the toe being displaced approximately 3.8m offshore. The increased toe displacement furthered the slumping of the armor units on the slope of the breakwater. The slumping was significant enough to open gaps through the armor, which exposed some of the filter layer stone. The structure was not tested for the overload case as failure had already been deemed to have occurred under the 100-year storm.

Figure 6 – Displaced toe and slumping of breakwater armor.

Based on these initial tests it was concluded that the toe trench which was initially part of the prototype design was required for a stable breakwater cross-section. The amount of force exerted on the breakwater toe from the drawdown of the waves would require an impractically large Tetrapod size without a toe trench to key in that unit. A section of the model bathymetry was removed and a trench section was re-cast in concrete at the correct elevation. A small portion of the trench, which represents an approximately 6m wide section, was not cast in concrete and was backfilled with small stone (see Figure 7). This was done in order to simulate a potential construction method that would be used in the prototype; if the toe trench is dug too deep the trench will need to be backfilled with stone to the proper elevation. The purpose of including a small section of exposed stone in the model was to investigate if the backfilled trench stone may be pulled out and through the toe armor units. After constructing the toe trench, the shallow water breakwater section was reconstructed using 20-ton Tetrapod units (the same size units used in the armor layer) placed in the trench. It was found that the toe trench greatly increased the stability of the armor layer. No movement of the toe was observed during or after the 50 year, 100 year or the 20% overload storm. In addition, the backfill stone in the trench remained stable throughout all tests.

Figure 7. 1m deep toe trench with exposed backfill stone.

3.2.2 Armor Unit Stability

The only significant armor unit movements observed during the shallow water tests was slumping of the armor layer due to the offshore shifting of the toe units. During the testing of the cross-section with the toe trench, no significant movement of the Tetrapods was observed. Since the cross-section was located in relatively shallow waters, the larger waves would break offshore, reducing the amount of wave energy that the structure was exposed to. The specified armor unit placement pattern seemed to perform well under all wave conditions up to and including the 20% overload scenario.

3.2.3 Overtopping

There was limited overtopping of the crown wall observed during the shallow water cross-section tests. This was likely due to the fact that the larger waves would break in the deeper waters offshore, limiting the amount of wave energy that reached the breakwater. The three largest storm events, the 50-year, 100-year and overload scenario all produced only a fine spray over the crown wall and onto the collection tray, however there was not enough water to provide any measurable overtopping. A number of large waves during the overload scenario produced green water landing approximately 10.5m onto the breakwater crest, which fell short of the crown wall which was located 12.8m landward of the offshore crest line.

3.3 Deep Water Cross-Section

The deep-water cross-section has a similar profile as the shallow-water section except that the toe of the breakwater was in deeper water, near the -10m contour (opposed to the shallow-water section where the toe was near the -6m contour) and the toe was not recessed into a 1m deep trench. The deep-water section design also featured two layers of 20-ton Tetrapod units installed following the same ordered placement pattern presented in figure 4. The crest height and height of the crown wall both remained the same at +5.4m and +5.5m, respectively. Figure 8 presents both the design drawing and the final constructed structure for the deep-water cross-section.

The focus of the deep-water cross-section was to try and optimize the crown wall location by reducing the breakwater crest width while at the same time ensuring the overtopping amounts remained within the design limits. The stability of both the toe units and the armor layer were observed during testing. The crest of the deep water cross-section was split into two halves in order to efficiently test two different crown wall offsets simultaneously. A rectangular piece of sheet metal was placed between the two different crown wall offsets to ensure that no cross-over splashing would interfere with the measurement of the individual overtopping rates.

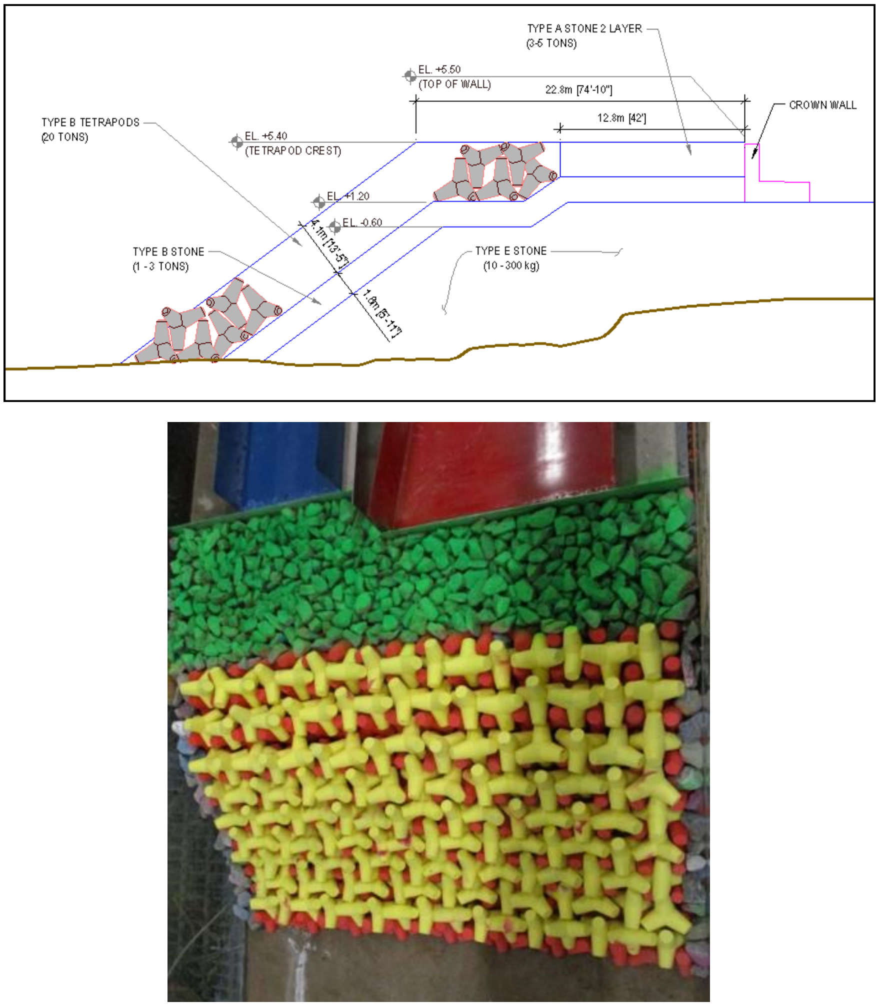

Figure 8. Top – deep water cross-section design; bottom - constructed deep water cross-section breakwater.

3.4 Toe Stability

The deep-water cross-section was constructed with 20-ton Tetrapod units both on the face and at the toe of the structure. The deeper waters at the toe of the structure reduced the drawdown forces acting on the toe and the toe units remained stable during all tested storms, including the 20% overload scenario.

3.4.1 Armor Unit Stability

The deep-water cross-section remained stable throughout the entire testing series, which included 17 different storm segments ranging from the 10-year return period storm to multiple overload storms. Since the main focus of the testing for this section was to examine the overtopping for different crest widths and crown wall locations, the crown wall was moved and the structure was exposed to multiple severe storms without rebuilding the structure. Small movements (shifting in place) of approximately half of the Tetrapod units was observed during the 100-year return period and the overload storms, particularly the Tetrapods located along the SWL. There was no significant movement, rotation, or displacement of any of the Tetrapod units.

3.4.2 Overtopping

Overtopping measurements were taken for a total of five crown wall positions which resulted in testing five breakwater crest widths; 3.5m, 6.5m, 8.2m, 10.5m and 12.8m. Both the average overtopping discharge and single overtopping events were measured and Equation 6.13 of the Eurotop guidelines was applied as a scale and model effect correction factor (Eurotop, 2016) to all recorded overtopping discharges.

The 3.5m crest width produced significant amounts of overtopping during the 100-year storm and the overload scenario (see Figure 9), with single overtopping events in the overload scenario reaching 24000 L/m. This level of overtopping could potentially cause significant damage or sinking of larger yachts in the marina on the leeside of the breakwater (Eurotop, 2016). The average overtopping discharge was 12.18 L/s/m during the overload storm, well above the 1.0 L/s/m criterion set by SGJJR. As the crown wall was moved further from the front slope of the breakwater, thereby creating an increasing crest width, the overtopping discharges and maximum single events decreased, as expected. After completing the testing of the five crest widths, the optimum crest width was found to be 10.5m. Placing the crown wall 10.5m back from the breakwater offshore crest line produced overtopping discharges of 0.05 L/s/m and 0.62 L/s/m for the 100-year return period storm and the overload scenario, respectively. The maximum event recorded during the 100-year storm was 110 L/m, and 2000 L/m for the overload scenario.

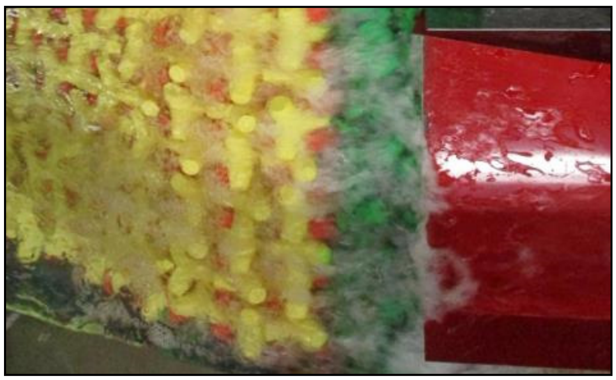

Figure 9. Overtopping of the 3.5m crest width.

Finally, due to the possible problems with the supply of sufficient amount of 4-ton armor stones, a 4.8 ton concrete armor unit was used to replace the stones in the berm to revise the overtopping. Two layers of small 4.8-ton Core-loc units were installed on the breakwater crest, replacing the 4-ton rock. The design of the prototype breakwater structure included smaller 4-ton Tetrapod units on the crest of the structure. However, due to the expedited nature of the physical model, there was not enough time to procure smaller Tetrapod units. Since the stability of the Tetrapods on the crest was previously tested by others and deemed stable, similarly sized rock was used on the breakwater crest for modelling purposes. Using rock on the crest may decrease the friction and size of the voids when compared to using Tetrapods, potentially causing larger overtopping rates compared to what may be observed when using Tetrapods. Therefore, 4.8-ton Core-loc units were installed on the breakwater crest (Figure 10) to investigate the difference between using rock and using armor units on the crest with regards to overtopping. Overtopping tests were repeated for the 10.5m and 12.8m crest widths.

Figure 10. Overhead view of the breakwater crest with a 12.8m crest width and a Core-loc armor layer.

In general, using Core-loc units for the armor crest layer appeared to reduce the amount of overtopping for both the tested 100-year and overload storms. This can be attributed to the increase in roughness and porosity on the breakwater crest provided by the armor units. Table 3 summarizes overtopping results for the 12.8m crest width exposed to the overload storm for both the 4-ton rock and the 4.8-ton Core-loc armor layers used on the breakwater crest.

Table 3. Overtopping summary for 12.8m crest width exposed to overload storm.

Crest Armor Layer	Total Overtopping Volume (L/m)	Overtopping Discharge (L/s/m)	Maximum Event (L/m)
4-ton rock	9659	0.89	1700
4.8-ton Core-loc	5761	0.55	1500

The overtopping results for all the different breakwater cross sections tested are shown in table 3. To determine the allowable overtopping discharges the consequences of the overtopping must be identified. Considering that there are permanently occupied structures behind the breakwater, overtopping at the Ayia Napa Marina can cause a hazard, injury or death to people behind the defense. Overtopping can cause damage to the comfort station, service docks and beach club, and it can cause damage to the small and medium sized boats moored behind the structure. The characteristics of the observed jets showed that when heavy green water went over the crown wall a large amount of water plunged into the parking and roadway increasing the speed of the flow, furthermore the crown wall obstructs visibility of the incoming water jet, providing no warning to the event. Based on the considerations above a level of protection of less than 0.1 l/s/m is required for the 100-year return period. Considering the dramatic change in overtopping rate in a couple of meters along the berm, and the existence of the building elements behind the breakwater the overtopping rate for the overload condition not to exceed 1 l/s/m.

Based on these provisions, the inner rock berm width is set to 10.5m for the final design. The obtained mean discharge rates and maximum water volume for an individual wave for each tested cross section for the different return periods is shown in table 3. The total berm width in these tables include the 10m of Tetrapods the additional tested rock berm. For a rock berm of less than 10m, total berm width 20 meter, there is a sharp increase in the maximum individual volume of water during the test. The reason for this sharp increase is due to green water overtopping. The green water overtopping on top of the wall and into the road way is considered very dangerous as the people will not see the water and it can create a thick sheet of water that would have the potential to displace persons and heavy objects into the Marina Basin. The design avoids such events for 100 year return periods. Figure 12 exhibits the reduction in overtopping rates when the additional rock berm width is increased.

Table 4: Individual and Maximum Overtopping Discharges for variable Berm Widths

Cross Section Condition	TEST	ROCK	10yr		50yr		100yr		1.2x100yr	
		BERM (m)	Q(l/s/m)	V(l/m)	Q(l/s/m)	V(l/m)	Q(l/s/m)	V(l/m)	Q(l/s/m)	V(l/m)
DEEPWATER	A	12.8	0	0	0	0	0.007	NA	0.005	NA
DEEPWATER	B	8.2	0	0	0.1	NA	0.2	200	2.9	8000
DEEPWATER	B	8.2	x	x	x	x	x	x	1.95	7600
DEEPWATER	C	6.5	x	x	x	x	x	x	1.47	7400
DEEPWATER	D	3.5	x	x	0.04	NA	0.54	3550	12.18	24000
SHALLOW WATER	C	6.5	x	x	0.05	200	0.08	270	2.12	7000
SHALLOW WATER	B	8.2	x	x	0.01	90	0.07	280	2.36	9700
DEEPWATER	A	12.8	x	x	x	x	0	NA	0.17	1000
DEEPWATER	E	10.5	x	x	x	x	0	NA	0.23	1000
SHALLOW WATER	A_s	12.8	x	x	0	NA	0.05	250	0.89	1700
SHALLOW WATER	E_s	10.5	x	x	0	NA	0.03	70	0.7	2200
SHALLOW WATER Corelock	A_s	12.8	x	x	x	x	0.03	NA	0.55	1500
SHALLOW WATER Corelock	E_s	10.5	x	x	x	x	0.05	110	0.62	2000

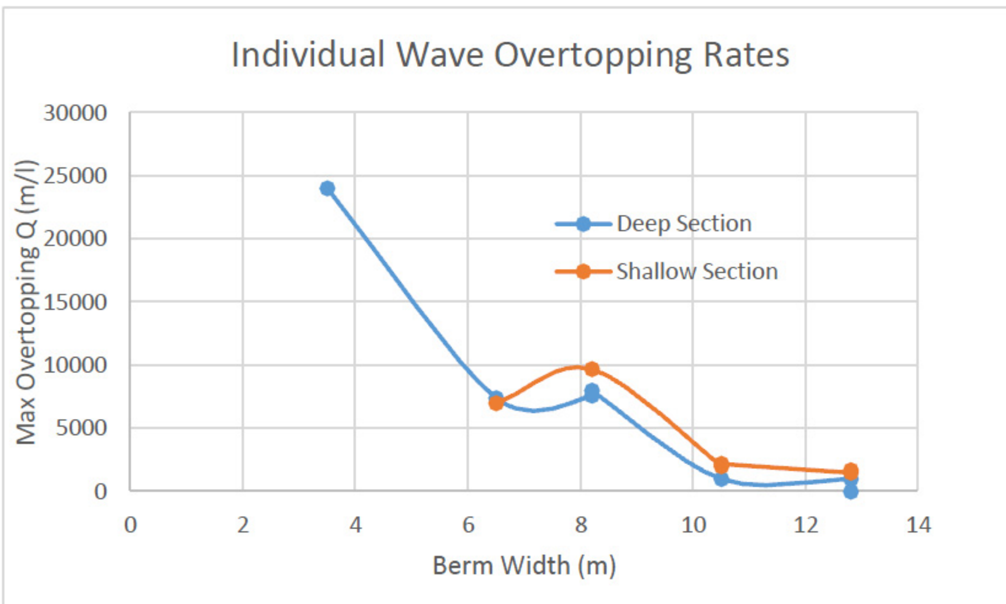

Figure 11. Individual overtopping volume for various berm widths

4 CONSTRUCTION

The final construction drawings with all the design changes, after the physical models carried out the NRC, was completed at the end of December 2016, before the contractor finished the approach to the breakwater. Previous to the placements of the Tetrapods on the breakwater, a test cross section was done in land to review the tolerances and packing density as required in the specified special placement pattern. Tetrapod placement in the water initiated in early March 2016. The redesigned wide berm breakwater reduced the overall cost by more than 3 million euros, further the increase in available area eased the construction since additional turning bays for the trucks transporting the stone materials are not needed and a "bottle neck" effect is not created for the 1 km long breakwater, the total width considering the road way and parking of the crest allows for different equipment and parallel construction.

Figure 12. Test Cross Section in Land and Tetrapod Placement in the water.

4.1 Toe Trench Construction

The results from the second physical model at the NRC proved that the breakwater's toe trench is very important for the stability of the breakwater and could not be avoided. The toe trench construction at the beginning of the project was been done with a 5-ton heavy chisel that was dropped from a height of 10m into the water to loosen the hard calcarenite and then a clam shell bucket with teeth was used to scrape the calcarenite. This initial method proved to be difficult to attain the trench precision and tolerances as required by the design, less than 100m were excavated with this method. The construction methodology of the trench was changed to attain a better precision; a large backhoe dredger was mobilized to complete the toe trench excavation with better results. Nevertheless, a small gap between the leeward row Tetrapods and the trench's vertical wall was inevitable, as shown in figure 13. During construction, large 5m waves impacted the breakwater and the Tetrapods in the trench. The shallow water section had a minor slide towards the front wall of trench, validating the conclusion found in the physical model, which is that a Toe Trench is of utmost importance for the stability of the shallow water section of the breakwater.

Figure 13. Prototype excavated trench and confinement of Tetrapods by the trench at the shallow section.

4.2 Tetrapod Placement Pattern

The performance of the Tetrapod stability with the specified special placement pattern has proven to be stable. During construction the placement pattern was easily achieved by the contractor (figure 14) and no movement or rocking of the Tetrapods has been recorded.

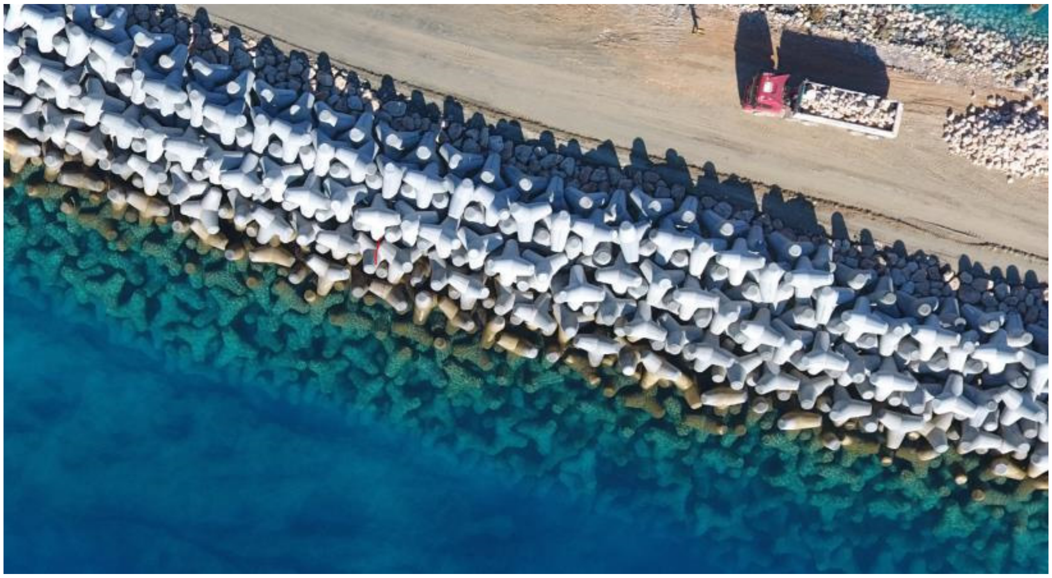

Figure 14. Prototype Tetrapod placement pattern

The construction of the breakwater has progressed based on the redesigned cross section after the physical model carried out at the NRC. Approximately 500 meters of the 1-kilometer long breakwater has been constructed thus far. A wave gauge was installed at a depth of 17.5 m to monitor the performance of the breakwater. The 17.5 m depth matches the approximate depth of where the waves were generated in the physical model. Several large storms have impacted the breakwater during the construction comparable to the 50-year return period event. Since the berm and the crown wall have not yet been built the performance of overtopping is still indicative, nevertheless the design has proven satisfactory under these conditions.

5 CONCLUSION

The crest-height reduction of the breakwater of the Ayia Napa Marina was achieved through creating a wide-berm breakwater section. The combination of an outer Tetrapod section with an inner rock berm to reduce the overtopping rates, proved to be the key to lowering the crest height of the breakwater creating an innovative solution to the obstruction of views by a common breakwater section. The additional length of the required top berm was determined through physical model tests at the NRC, carrying out 14 variations of the berm width and water depth. The additional width of 10.5 meters allowed for a reduction of 2.5m of the crown height, allowing for unobstructed views for the resort-style development on the lee side of the breakwater.

The two-dimensional physical model studies carried out to support the design changes of the breakwater were crucial to obtain a safe reliable solution. The low crested wide berm breakwater will protect the new mega-yacht marina for 100-year return period storms. An expedited physical model was used to investigate key issues that were raised during early stages of construction, including potential elimination of the toe trench and lowering of the breakwater crest. The model was used to assess these changes to the initial breakwater design, and the knowledge and results gained from the physical model study have been used to support and optimize the final design of the new breakwater.

The lowering of the breakwater's crest height provided more harmonic integration with the development's landscape and users, and allowed for substantial cost savings for the owner. Design performance and development economic performance were both enhanced as a result. The design changes have also eased the construction impacts, providing better access and additional valuable, developable areas for the Ayia Napa Marina.

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