USE OF PHYSICAL/NUMERICAL MODELING IN DESIGN OF AN EXPOSED QUAY AT PORT OF ASHDOD

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ABSTRACT

This paper addresses key considerations for the design of a ca. 450 m long exposed container quay serving as part of a port expansion in the Port of Ashdod, Israel. The selected quay structure consists of a steel pipe pile supported concrete platform, with a combination stone/ Antifer cube revetment below the platform for wave absorption.

In developing the detail design, numerical wave modeling was carried out to assess the wave climate both in the construction area and at the existing (operating) port, considering the different stages of construction.

To verify estimated wave forces and potential damage to the revetment and quay during both construction and operational stage, two-dimensional physical model testing was carried out in a laboratory wave flume. Pressure gauges were installed to measure the wave induced pressures on the model deck slab, pile caps, (optional) fascia beam, and approach slab.

INTRODUCTION

The project involves construction of an approximately 600 m long extension to the existing Main Breakwater as well as an approximately 1,500 m long new Lee Breakwater. The quay is being constructed near to the existing operational port, and therefore the type of structure and sequence of construction had to consider its potential effects on downtime in the existing port due to wave reflection.

The quay was designed to accommodate Panamax type vessels and was intended to be partially constructed early in the project to provide protection from waves in the main construction area. In addition, two additional quays (Quay 27 and 28), the Temporary Retaining Structure are being constructed for EEE class container vessels and small vessels (such as tugs), respectively. This paper addresses key considerations for the design of the new exposed container quay (Quay 28) serving as part of the expansion of the port (Figure 1).

Three different pre-determined construction stages, namely, after 24 months (Phase 24), after 30 months (Phase 30), and after 42 months (Phase 42), as well as the existing port layout were considered in the analysis. The layout details of these stages are shown in Figure 2. These stages assume that ground improvement, for the Main Breakwater Extension which is required to mitigate liquefaction under seismic loading, will be performed in parallel to the Staging Harbor construction. These timelines are all with respect to completion of the Staging Harbor at 18 months before NTP.

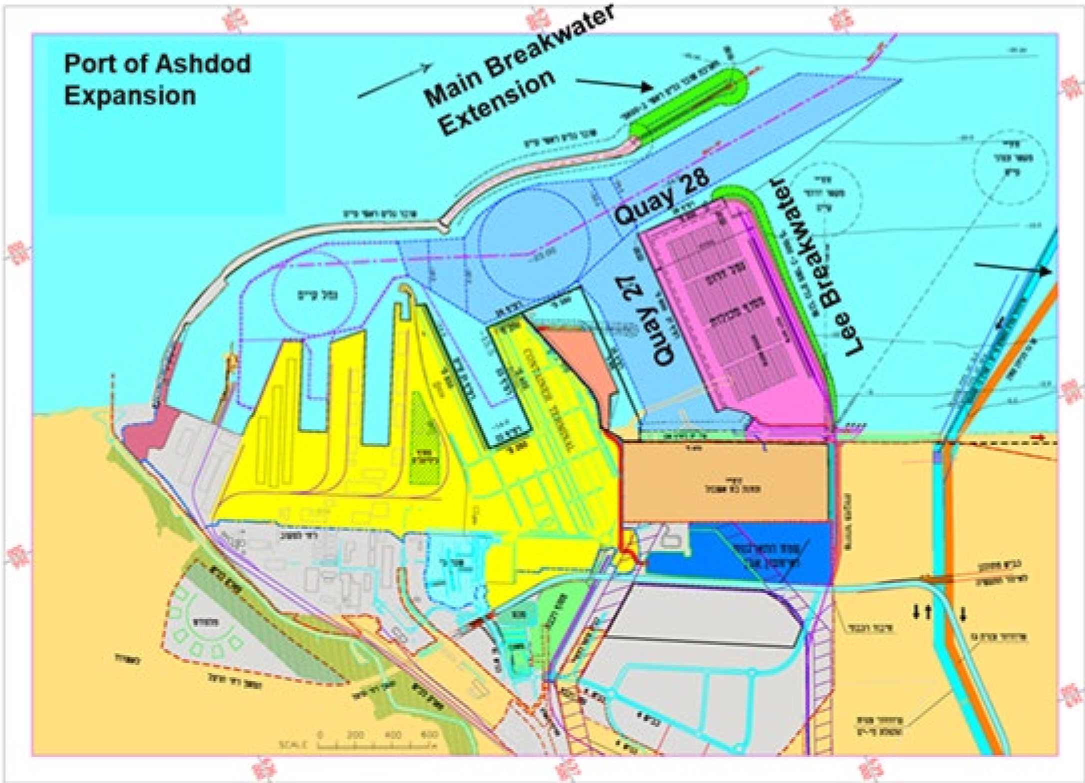


Figure 1. Existing layout (yellow) and new extension (shaded in purple and green) plan layout-Port of Ashdod

This paper describes the use of numerical and physical modeling studies for the design of the Quay 28 structure. This study focuses on the wave conditions at critical locations within the construction area during the various stages of construction, including downtime assessment for the existing port, and wave pressures on the new quay structure and revetment.

The numerical modeling was carried out to determine wave conditions in front of the quay structure and within the port development, including the Entrance Channel, shown on the Figure 1 in blue. The physical model study provided wave pressures which were used to refine the design of the quay structure. It also compared wave pressure results between semi-empirical relationships and the physical modeling.

NUMERICAL MODELING

In developing the preliminary design, numerical modeling was carried out to assess the wave climate both in the construction area and at the existing port, considering the different stages of construction. The selected structure was a steel pipe pile supported platform, with a combination stone and Antifer cube revetment below it for wave absorption. However, to provide protection to the construction area, it was determined that a concrete caisson structure would first be built immediately landward of the platform. Furthermore, in order not to subject the quay deck to excessive wave loading, construction of the quay deck would have to await substantial completion of the Main Breakwater Extension.

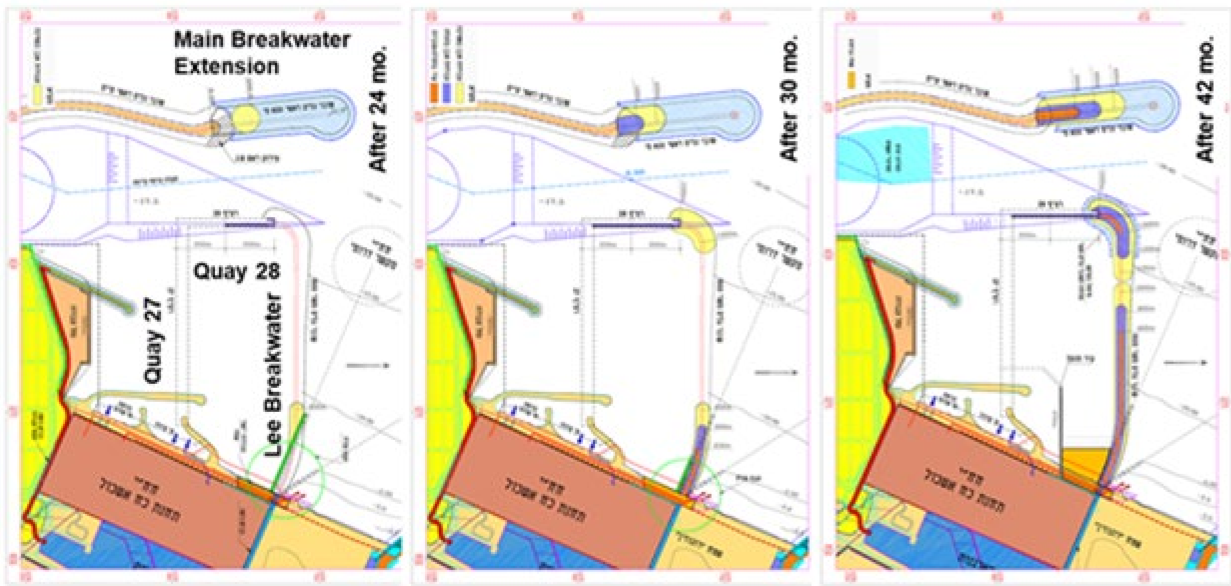


Figure 2. Construction phase layouts at 24, 30 and 42 months for the Port of Ashdod

DHI's MIKE21 Boussinesq Wave (BW) model was used to investigate the wave agitation conditions within the approach channel, in the existing port, and in the construction areas during various stages of construction. The model is capable of reproducing combined effects of shoaling, refraction, diffraction, wave reflection from structure and wave-wave interactions. In the model, a time-series of wave trains was created using wave conditions at the model boundary. To simulate partial reflection from, and transmission through structures, a porosity layer, to absorb wave energy sponge layers, was defined at model open boundaries.

The purpose of modeling the existing port layout is to provide a comparison of wave conditions to be experienced during the construction stages with the current port layout. The offshore boundary of the model has set approximately at the 27 m depth contour and extended in a rectangular grid to include the entire footprint of the Port of Ashdod. A grid resolution of 5m x 5m was used to cover 5 to 10 computational points per wave length for the shortest wave period of 6.0s considered in the simulations. Figure 3 depicts the model bathymetry and boundary for the existing layout, after 24 and 30 months of construction. The effect of the turning basin dredging sequence, the western half to be dredged first and then the eastern half, onto the wave agitation levels for the final layout (i.e. at 42 months) is shown in Figure 4.

In order to monitor the anticipated wave conditions which might occur during the various stages of the construction, twelve wave conditions were simulated. The occurrences given in Table 1 are based on a statistical analysis of measured waves for respective directions at an offshore wave buoy (in 23 m water depth). Prior to input into the BW model, the selected wave conditions were back refracted and shoaled to the BW model offshore boundary.

Table 1 Wave conditions identified for the testing of the construction stages

Occurrence	WNW		NW		NNW	
	H_{m0} (m)	T_p (s)	H_{m0} (m)	T_p (s)	H_{m0} (m)	T_p (s)
10 % exceedance/yr	2.9	8.5	1.2	6.0	1.0	6.0
5 % exceedance/yr	3.4	9.2	1.6	6.3	1.1	6.0
1% exceedance/yr	4.4	10.5	3.0	8.7	1.5	6.1
10 yr Return Period	5.9	12.1	4.8	10.9	3.1	8.8

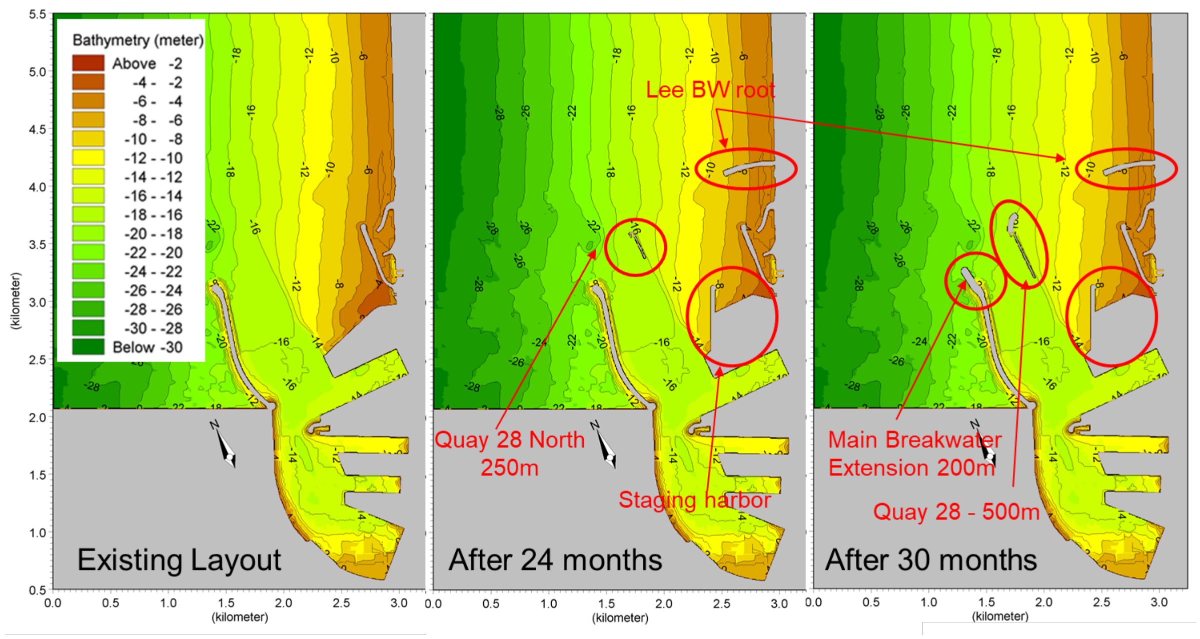


Figure 3. BW model bathymetry and boundary for existing, after 24 months and after 30 months of construction

Figure 5 shows wave height maps for 10% exceedance/yr wave conditions from west-northwest (WNW) for the existing layout and at 24-months. A standing wave pattern is observed in front of Quay 28 at the 24-month construction stage due to wave reflections from the Quay 28 caissons, which are close to fully reflective at this stage. Figure 6 depicts the wave height map for the same wave conditions at the 30 month and 42-month construction stages. The wave conditions are worse at the Quay 28 location for construction stages after 24 months and after 30 months, compared to the existing condition. This is more pronounced for west-northwest waves than other directions. For northwest and north-northwesterly waves, the impact of construction is relatively small, as waves approach Quay 28 more obliquely and thus the reflected waves propagate more towards the southwest direction. As a result, wave amplification zones are more towards the southwest of Quay 28. At 42 months, the wave conditions were improved due to further progress of the Main Breakwater construction. Wave conditions at the Lee Breakwater roundhead are slightly worse during the three construction stages compared to the current condition, due to wave reflection from the caisson structure.

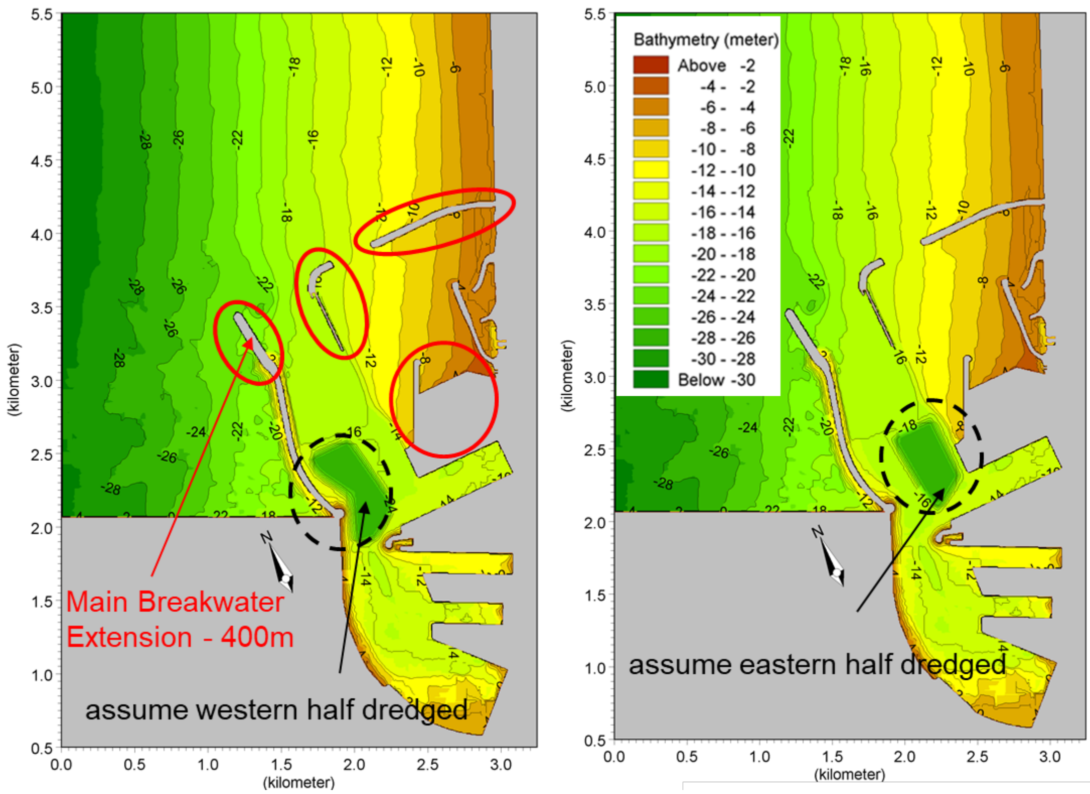


Figure 4. BW model bathymetry and boundary after 42 months of construction

Model results were extracted in front of Quay 28 to input into the physical model tests. Analyses of wave conditions at this location were used as the basis for determining the representative wave conditions to be considered in the physical model testing program for wave induced pressure measurements.

PHYSICAL MODELING

A comprehensive two-dimensional physical model study was carried out to verify estimated wave forces and potential damage to the revetment and quay deck both during construction (prior/during breakwater construction) and once the port goes into operation (following breakwater construction), and to design the quay structural elements for wave loading during both the construction and operational stages. Two-dimensional physical model testing was carried out in a wave flume at the Coastal and Marine Engineering Research Institute (CAMERI) in Haifa, Israel. A 1:25 Froude number scale model of the quay structure was bolted to the floor of the flume and instrumented to provide direct measurements of wave loading on the deck and pile cap elements. Tests were performed using irregular waves, and pressure gauges were installed on the deck, pile caps and optional fascia beam, to measure wave induced pressures logged at a frequency of 1,000 Hz.

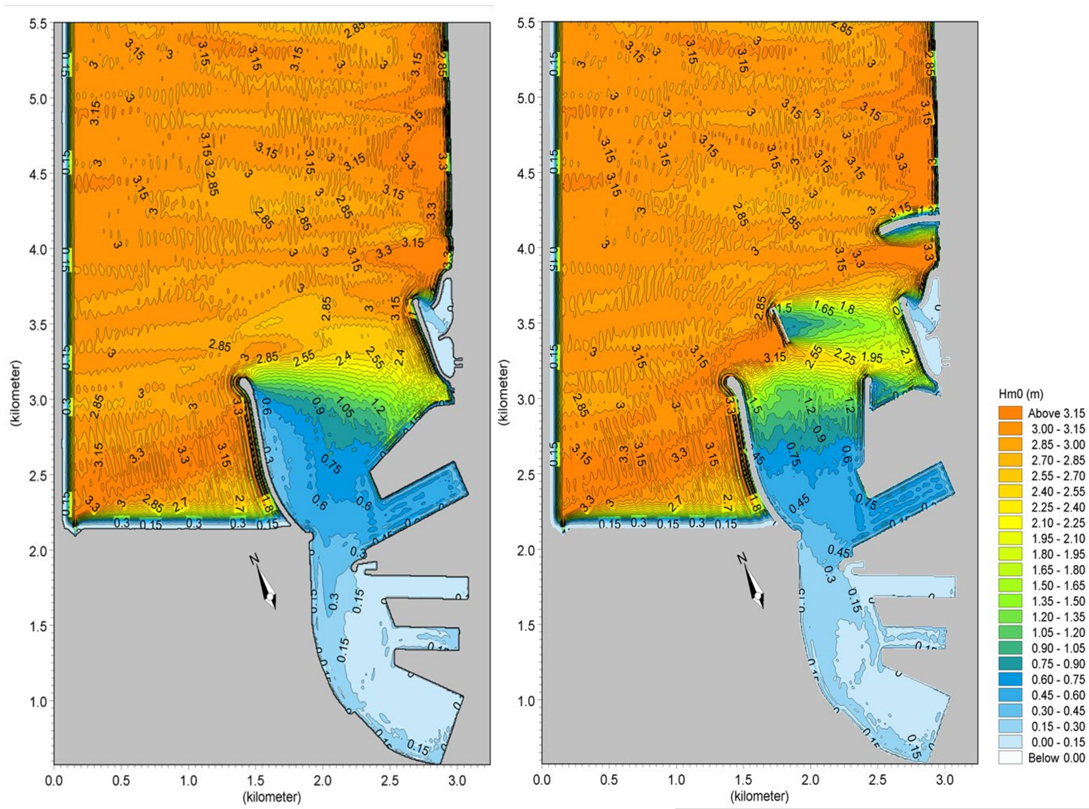


Figure 5. Wave height maps for the existing layout (left panel) and at 24 months (right panel) during 10% exceedance/yr from WNW wave case

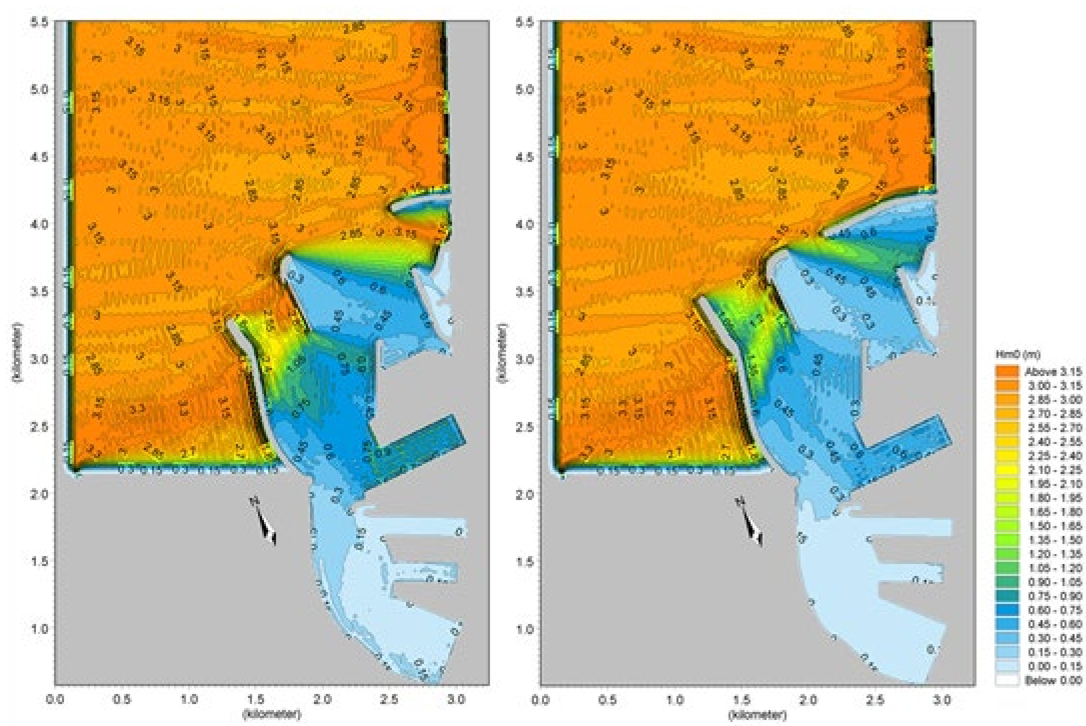


Figure 6. Wave height maps at 30 months (left panel) and at 42 months (right panel) during 10% exceedance/yr from WNW wave case

Figure 7 shows cross-section of the Quay 28 structure in the wave flume. In the model the seabed at the wave board has been represented by a rigid horizontal bed at a depth of -24m to ensure sufficient water depth for proper wave generation. A transition slope of 1:10 is used, beginning at the 4m in front of the wave board, to bring the seabed to a depth of -17.5 m. Two series of tests, namely, continuous and partially open (discontinuous) fascia beam were carried out (see Figure 7), and pressure time histories at each sensor for impulsive and quasi-static loads were measured.

In order to assess the spatial distribution of vertical pressures on the deck and on the pile cap, eight pressure transducers were installed on the deck, and another three on the pile caps. An additional transducer was located on the fascia beam, and it measured horizontal pressures at the seaward side, as shown in Figure 8. Pressure transducer 12 (PT12), measuring the horizontal pressure, was adjusted laterally during continuous and discontinuous fascia beam test cases.

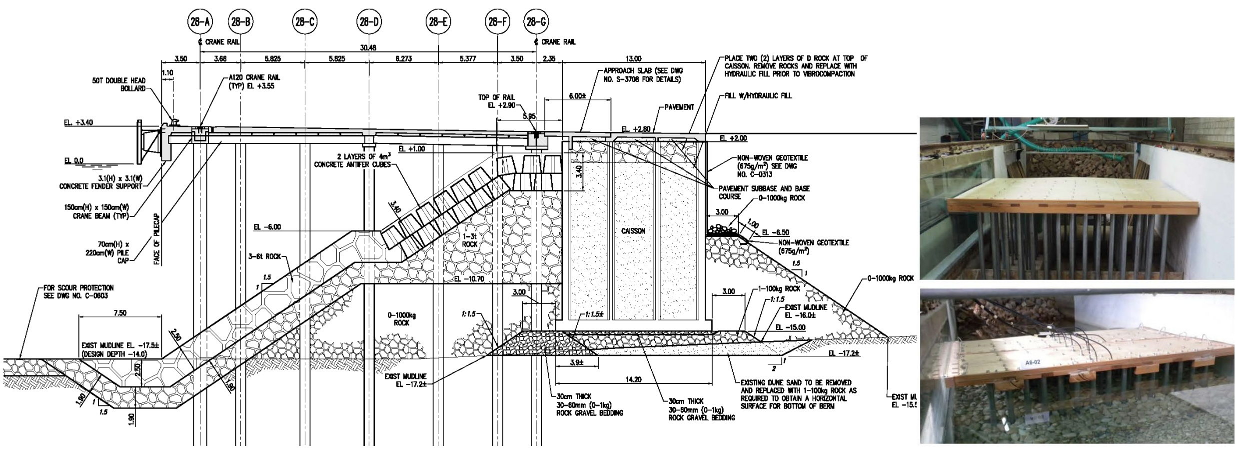


Figure 7. Cross-section of Quay 28 and view of fascia beam (continuous top and discontinuous bottom) and deck in the model flume

The revetment slope was designed as a submerged flat berm at -6.0 m using a composite rock (3-6 tonne class stone) on the lower part and 4.0 m^3 (9.6 tonne) Antifer cubes on the upper part of the berm. To assess damage to the Antifer cube revetment, three different construction stages were evaluated; rock (3.0-6.0 tonne) in place without Antifer cubes on the upper part of the slope; rock and Antifer cubes in place but deck of the quay not yet constructed; and rock and final stage Antifer cube armor in place and deck of quay constructed.

Table 2 summarizes the range of environmental conditions that were modeled in the study. Tests were conducted with ten-hour long duration of irregular wave statistics, which contained approximately 3,000 individual waves and a realistic distribution of extreme wave height, period, and water level combinations. Pressure values were estimated at $P_{0.4\%}$ statistical level (i.e., the 250-statistical level; $P_{0.4\%}$ is the average of the four highest values recorded during each test of 1,000 waves). It is considered that the $P_{0.4\%}$ pressures are realistic values to use in the analysis and design of the deck, as the peak values are unlikely to act over a wide enough area to be used for the design of the entire deck slab. Furthermore, a review of the duration of the peaks (see attached) indicates that they are ca. 0.01 sec., which is considerably less than the calculated 0.12 sec natural period of the deck slab. Therefore, the slab would not be able to fully respond to impacts of such short duration.

Table 2 Physical model test matrix

Test Case	Return Period (yr)	Water level (m)	Wave height, H_{m0} (m)	Peak wave period, T_p (s)	Main Breakwater Completion
Shakedown		0.50	1.0	10.0	
A1	1	0.72	4.30	10.5	×
A2	10	0.95	5.75	12.2	×
A3	100	1.18	4.18	12.5	√
A4	100 (SLR)§	1.58	4.18	12.5	√
A5	100+overload	1.58	5.32	12.5	√
A6	100	-0.54	4.21	12.5	√

Note: § included a 0.4 m Sea Level Rise (SLR)

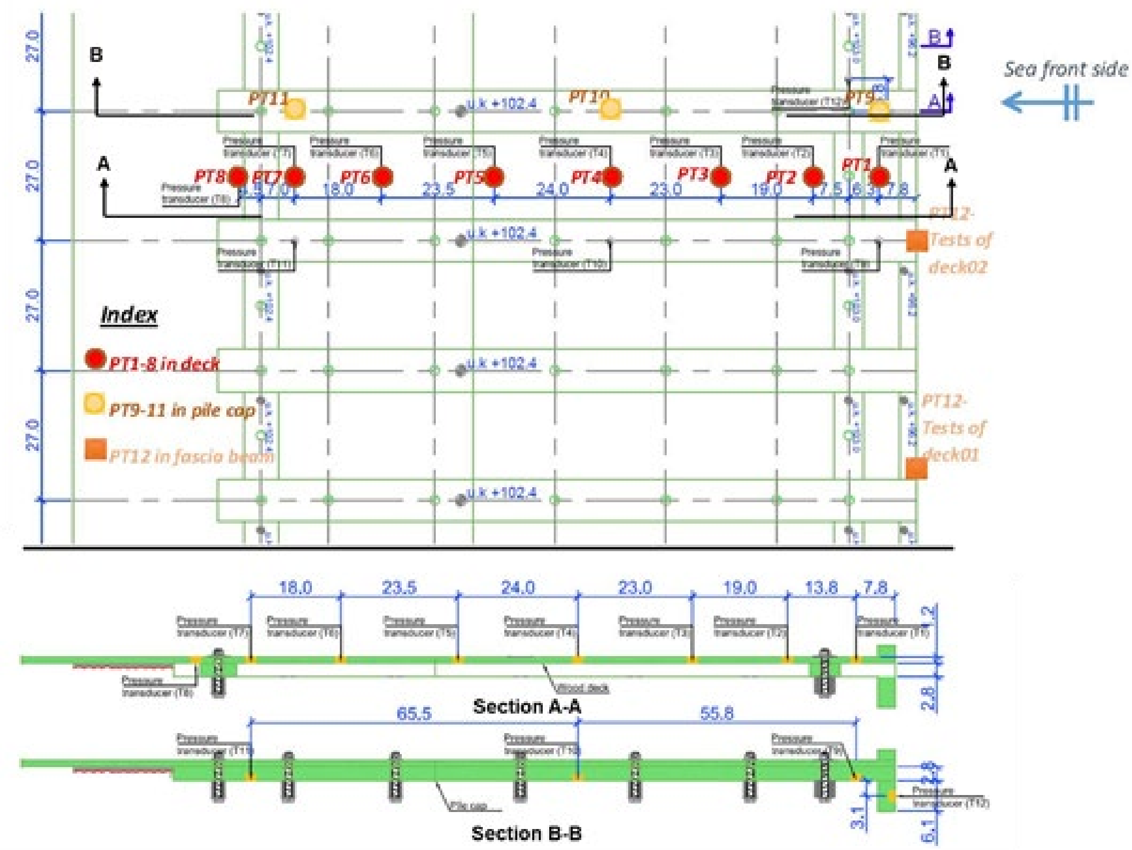


Figure 8. Pressure sensor layout for continuous and discontinuous fascia beam

For the case of 3-6 tonne rock armor in place without Antifer cube, this situation was evaluated for the one (1) year return period event, without the Main Breakwater Extension in place. The results were that 5% of the 1-3 t rocks were displaced or extracted. For the case of rock and Antifer cube armor in place but deck of quay not yet constructed, this situation was evaluated for the one (1) year and ten (10) year return period events, without the Main

Breakwater Extension in place. Following the ten (10) year return period event, cube movements of about 1D were noted for about 9% of the elements, and cube movements greater than 1D were noted for about 2% of the elements. The damage numbers represent percent damage in a cumulative sense.

For the case of rock and Antifer cube armor in place and quay deck constructed, this situation was evaluated for the one hundred (100) year return period event with the Main Breakwater Extension in place. The results were that 4.5% of the cubes underwent movement of less than 1D and 1% of the cubes underwent movement of more than 1D. Similar to the previous construction scenario, the damage numbers were assessed in a cumulative sense. In both cases movement of cubes more than 1D was less than 5.0% of allowable criteria.

Variation of dimensionless maximum vertical pressures ($P_{max}/\rho g H_s$) along the structure for the non-continuous fascia beam case are plotted in Figure 9. The maximum uplift pressures on the deck slab were recorded at pressure transducer PT6 for the 100-year return period event with a water level of +1.58 m (Test Case A4). For the case of low-water, the highest pressure was recorded at PT4 for the A6 case, which corresponds to the 100-year return period event with a water level of -0.54 m. As anticipated, wave impact is concentrated between the deck slab and Antifer Cube units on the slope. The maximum uplift pressures on the pile cap were recorded at pressure transducer PT9 for the A3 case. It was found that the peak pressure on the deck slab increases with decreasing deck clearance in a non-linear manner.

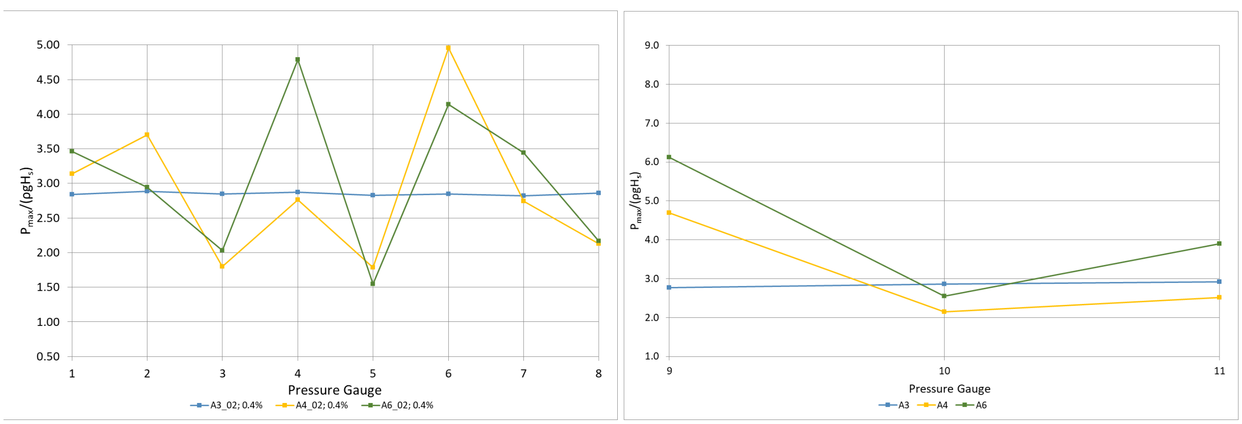
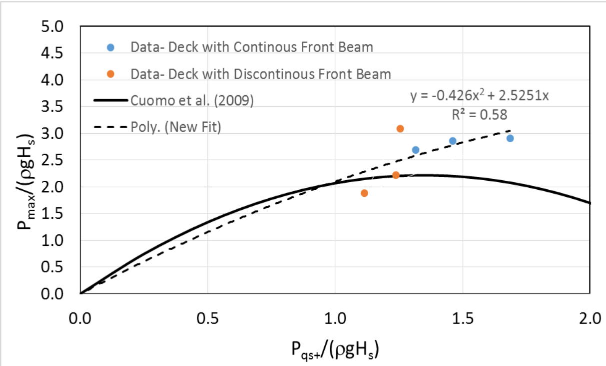


Figure 9. Non-dimensional quasi-static pressure on the deck slab (left) and pile cap (right) for non-continuous fascia beam case

Dimensionless uplift impact pressure on the deck slab (left) and pile cap (right) are shown in Figure 10 as a function of normalized quasi-static pressure. When compared to predictions by Cuomo et al. (2009) for impact pressures acting on the deck, the test data shows significant higher pressures impacting the deck slab. The maximum impact pressure is as much as twice high as the quasi-static pressure both on the deck slab and pile cap for the continuous and discontinuous fascia beam cases. The results of the physical model testing for the revetment indicated that the proposed design was generally sufficient to resist the wave loads with acceptable levels of damage. Wave uplift forces on the deck structure were significantly higher than what was predicted by using methodologies available in the literature. This is due to the fact that these methodologies apparently did not account for a sloped revetment beneath the quay combined with a closed structure behind the quay platform. This required some modification to the structural design.



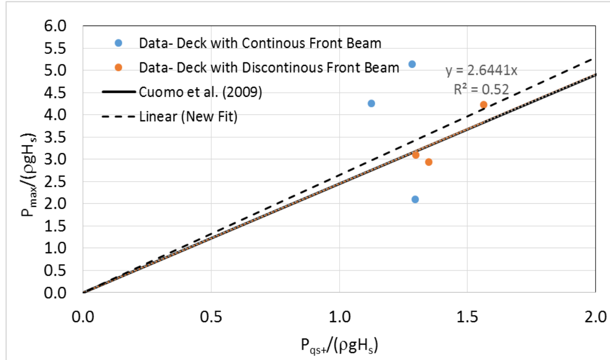


Figure 10. Non-dimensional maximum vs. quasi-static pressure on the deck (left) and on the pile cap (right) for continuous and discontinuous fascia beam case

CONCLUSIONS

The results of numerical and physical model studies have resulted in the following conclusions:

(a) a standing wave pattern is observed in front of Quay 28 for all the construction stages due to wave reflections from Quay 28, which (as a caisson structure in its initial construction stage) is close to fully reflective at these stages. Thus, wave conditions are worse at the Quay 28 location for construction stages after 24 months and after 30 months, compared to the current condition. This is more pronounced for WNW waves. At 42 months, the wave conditions will improve due to further progress of the main breakwater construction,

(b) downtime for the construction equipment within the construction areas (Quay 28) are estimated to be 18% and 19% at 24 and 30 months, respectively. However, there is no downtime for the inner harbor (southern portion of the port) regardless of the construction stage.

The results of the model testing indicated that some reconstruction of the rock slope for the revetment will be required in the event of a one (1) year return period event, and an occasional Antifer cube may have to be re-set in case of a ten (10) year return period event during construction. Although, the number of test cases was limited, it was found that measured maximum uplift forces can be twice as high the quasi-static pressures. The expressions shown in Figure 10 can be used to establish a preliminary estimate of maximum wave pressures on a deck for a pile supported quay structure similar to the one considered herein. However, more physical model test results are required to provide a generalized pressure formula. The wave uplift forces on the deck structure were significantly higher than what was predicted by using semi-empirical equations. Further model tests are required to draw solid conclusions and provide robust maximum and quasi-static pressure empirical formulas.

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