ROLE OF PHYSICAL MODELLING IN DEVELOPING A NEW CRUISE SHIP TERMINAL AT AN EXPOSED SITE

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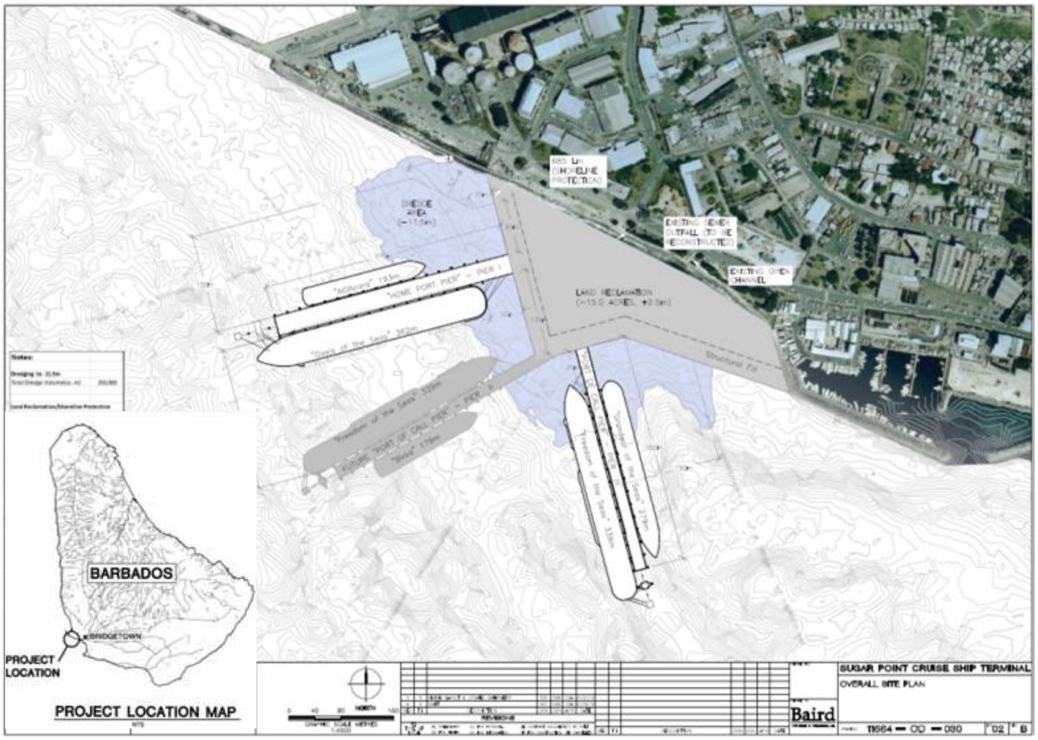
This paper describes the role of physical modelling in the design of a new cruise ship terminal at an exposed site on the coast of Barbados, outside the Port of Bridgetown. Large scale 3D hydraulic model studies were conducted to focus on two of the key technical challenges surrounding the project: the risk of downtime due to excessive ship motions forced by the prevailing winds, seas and swells; and the extreme wave loads and overtopping associated with waves generated by hurricanes. The physical modelling was separated into two phases. The first phase investigated the moored ship response of two different model cruise ship vessels under a range of operational wind and wave conditions. The results of this phase helped determine the range of conditions where the motions of the ships and the associated loads on the portside elements were within acceptable limits, and showed that the expected downtime for the design vessels was satisfactory. The second phase of the study focused on wave-structure interactions, and in particular the impact of extreme waves on the proposed structures, including wave-induced loads on the pier decks, and the wave overtopping and flooding of the landside development. Several innovative measures were developed and tested to accommodate / mitigate the loads on the pier decks as well as reduce the wave overtopping. These physical model studies played a key role in the front end engineering design of the new port, and their results were crucial in assessing various alternatives, optimizing preliminary designs, and validating the layout, costing and construction of the new facility. Due to space limitations, this paper focuses on the second phase of the study, in particular the hydrodynamic loads on the pier decks.

Keywords: physical modelling; wave uplift loads and pressures, pile support pier decks

INTRODUCTION

A joint venture of SMI Infrastructure Solutions and Royal Caribbean Cruise Lines (RCCL) proposed a new cruise ship terminal for an exposed site on the shore of Carlisle Bay, just outside the existing port of Bridgetown, Barbados. Baird & Associates (Baird) were subsequently retained by SMI/RCCL to undertake a Front End Engineering Design study on their behalf. The project, called the Sugar Point Cruise Ship Terminal, encompasses three new cruise ship piers/jetties, dredging and land reclamation works, with associated upland improvements that will both attract people to the site as well as support the cruise terminal operations. The proposed terminal, which is being designed to accommodate the largest cruise ships in the world, would alleviate congestion in the existing port (which currently serves both commercial and cruise operations) and would significantly enhance the experience of cruise passengers. The site is exposed to persistent seas generated by the prevailing trade winds, as well as intermittent low-amplitude swells generated by winter storms in the North Atlantic Ocean, and is occasionally exposed to large waves generated by passing hurricanes and tropical storms. The recommended project layout includes three 350m long pile-supported piers with berths for six large cruise ships, 415,000m^3 of dredging, 15 acres of land reclamation and multi-use landside development (see Figure 1). The project will be implemented as a design-build contract, with construction of Phase I to begin in late 2014 and to be operational for the 2016-17 cruise season. The Phase I marine works are estimated at approximately $120M USD, including dredging, land reclamation, Piers I and III, as well as relocation of a sewer outfall that is within the project footprint.

Figure 1: Location and layout plan of the Sugar Point Cruise Ship terminal.



As part of the design process, Baird commissioned the National Research Council of Canada's Ocean, Coastal and River Engineering (OCRE) portfolio (formerly the Canadian Hydraulics Centre) to conduct three-dimensional physical model studies of the proposed cruise ship terminal. The main goals of the studies were to help define the range of wave, wind and water level conditions that would allow for the safe and comfortable berthing and mooring of cruise ships, and also to help design the marine and coastal structures to withstand the wave induced loads, pressures, run-up and overtopping produced by large storms.

PHYSICAL MODELLING

A 1:50 scale three-dimensional physical model of the new terminal, complete with the surrounding bathymetry, the existing shoreline, the new land reclamation area, the new dredging and two of the new piers was constructed in a 36m by 30m wave basin at the National Research Council laboratory in Ottawa. The model bathymetry was formed in concrete and was based on a combination of high-resolution soundings from the project site and the proposed dredging plan. The bathymetry was faithfully replicated from the -30m contour up to the land's edge. The layout of the model in the testing basin is shown in Figure 2 and Figure 3.

Figure 2: Physical model layout.

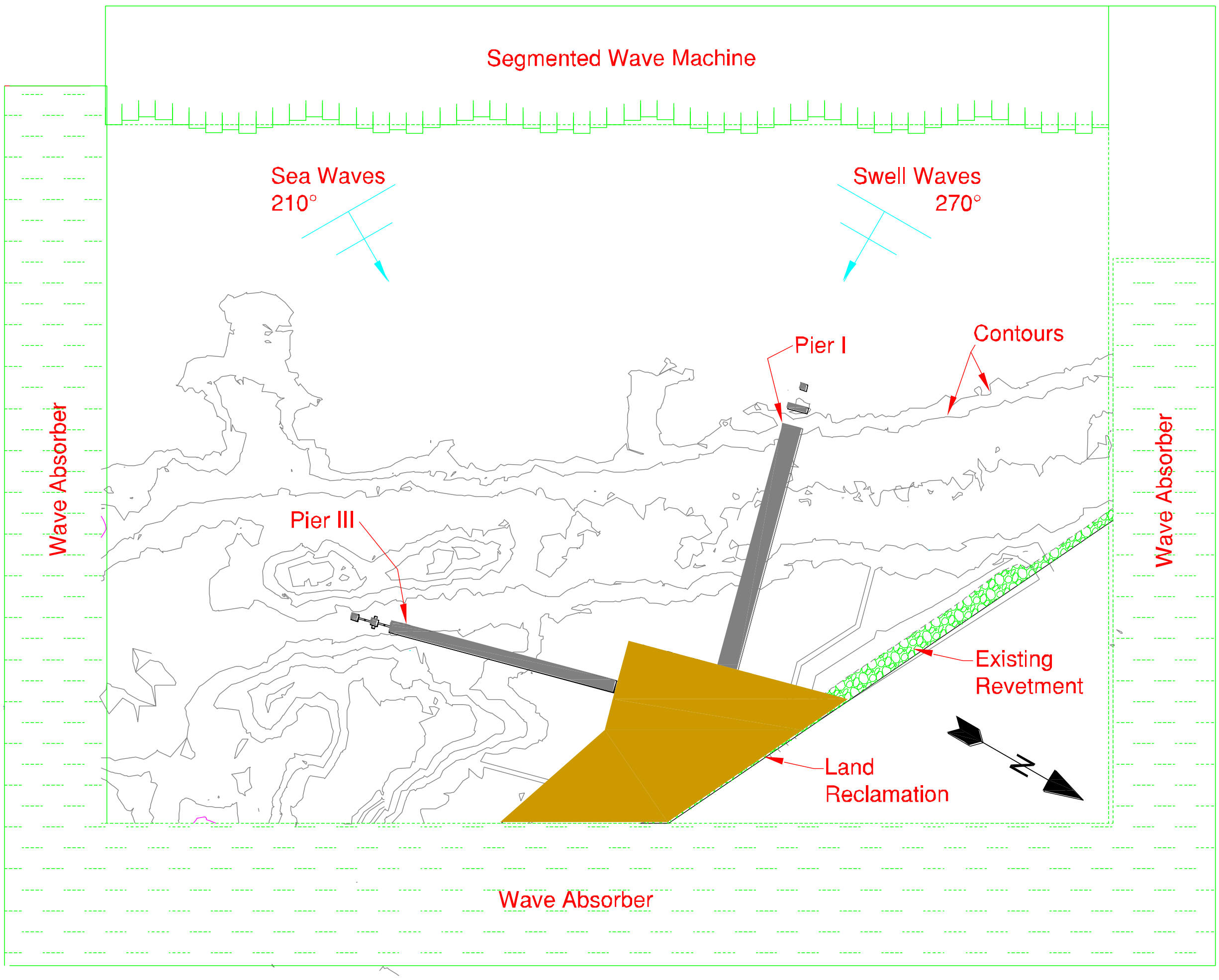
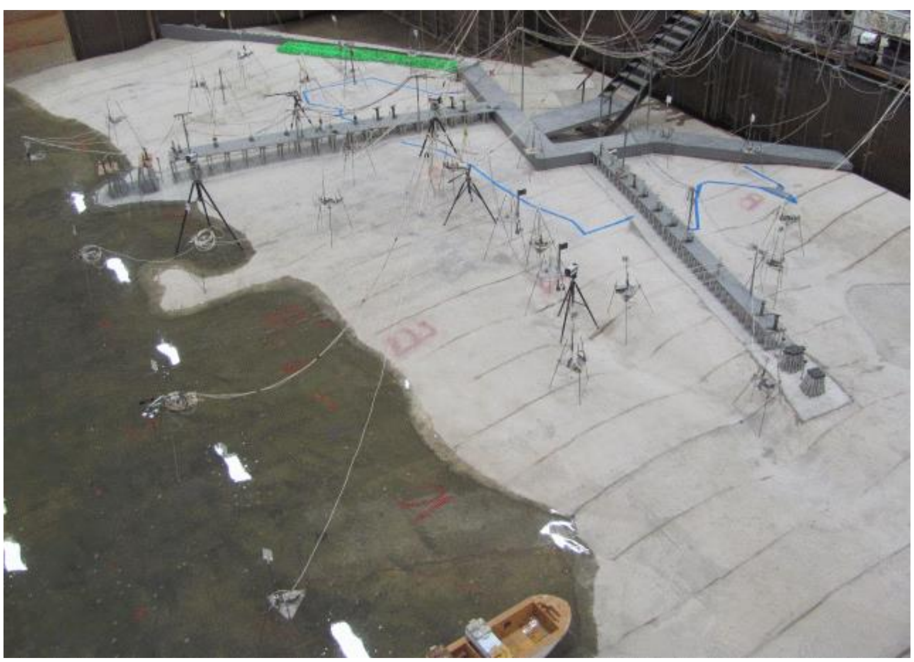
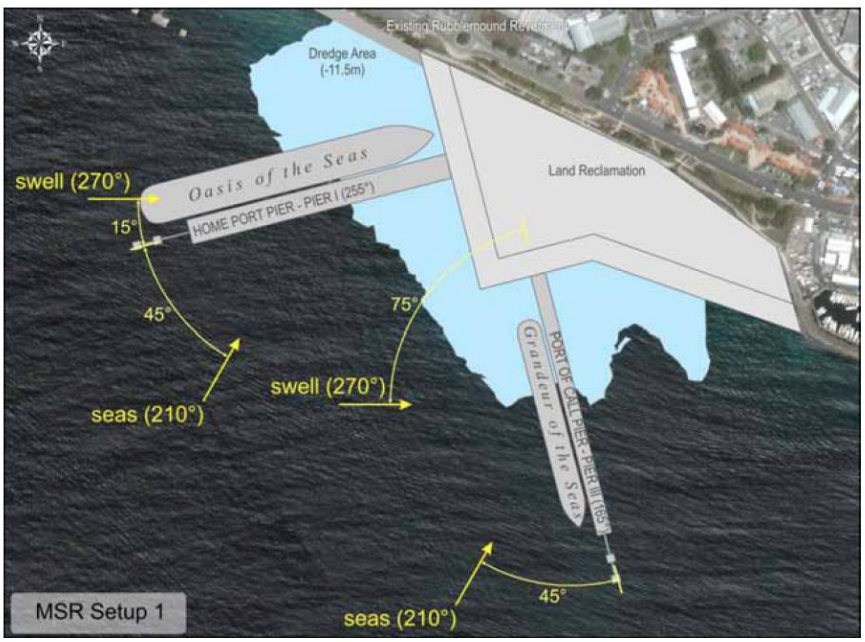


Figure 3: Physical model after bathymetry and port construction.



The operational, storm, and extreme wave climates near the project area were studied and used to develop a set of sea and swell waves representing operational conditions, and another set representing extreme conditions (storm waves). In general, the wave conditions selected for modelling span the 0.1%, 1%, and 10% exceedance probability range. At the project planning phase, the physical model design was optimized to provide a balance between using a large model scale (yielding more reliable data) and using a scale small enough to model the entire project site. Among the considerations influencing the model design was the ability of the wave generator to produce short-crested seas over a ~60° range of mean directions. In reality, swells are expected to approach the site from 270°N to 315°N, while operational seas are expected to approach from 160°N to 180°N. Hurricane waves may approach the site from a range of directions between 200°N and 270°N. Unfortunately, the full directional range of seas and swells at the site could not be generated in one physical model arrangement. Since the primary focus of the physical model was the extreme loads associated with hurricanes, and the longer period (westerly) swells were anticipated to be more important to moored ship response than the shorter period (southerly) seas, the wave machine orientation was set to 240°N so that waves approaching from 210°N to 270°N could be modelled. A schematic and table illustrating the set of wave conditions developed for use in the physical model is shown in Figure 4.

Figure 4: Wave conditions for the Sugar Point physical model study.



H_{m0}	T_p	Wave Direction
(m)	(s)	(°N)
Operational Waves		
Seas		
0.65	9	210
0.8	9	210
1.15	6	210
Swells		
0.2	12	270
0.4	13	270
0.7	14	270
1	14	270
Combined Sea/Swell		
0.8 & 0.4	9 & 13	210 & 270
Storm Waves		
4.5	8	210/240/270
4.5	10	210/240/270
5.5	10	210/240/270
6.5	10	210/240/270
6.5	12	210/240/270
7.5	12	210/240/270

MODEL SETUP AND PROCEDURES

Phase 1 – Modelling and Assessment of Moored Ship Behaviour

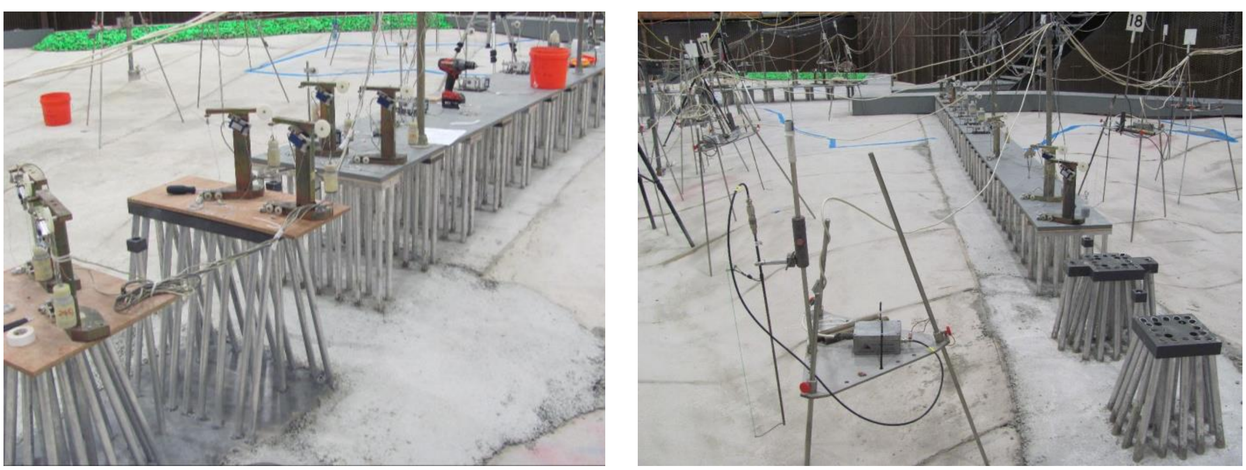
The physical modelling study was conducted in two phases. The first phase focused on the operational considerations of the port, including the behaviour of moored cruise ships under operational sea and swell conditions. Two different ships were modelled in this study as representative vessels that would frequent the port. The target ships were selected from RCCL's fleet, and were the 279m long Grandeur of the Seas as well as their flagship vessel, the 360m long Oasis of the Seas. Models of both ships were designed and fabricated at NRC, and were ballasted to replicate the mass properties and the dynamic characteristics of the prototype vessels. The Grandeur was ballasted to replicate a displacement of 36,000 tonnes and draft of 7.8m, while the Oasis was ballasted to 105,500 tonnes and 9.2m draft.

The port structures in the physical model were designed and constructed to closely replicate the preliminary prototype designs. All model structures were laid out with precision on the model bathymetry, and surveyed into place. A portion of the existing rubblemound revetment and vertical wall along the shoreline north of the new port site was replicated in the model, although the armour stone was slightly oversized to withstand the larger hurricane waves in the second phase of the model testing. Although the upland design details were largely undefined when this study was undertaken, the main features of the land reclamation area were reproduced in the model including a vertical sea wall and 30m wide "hardscape" promenade around its seaward perimeter. The top elevation of the land reclamation area was originally set to +2.5m CD, although it was later raised to higher elevations and fitted with various design elements to mitigate wave overtopping.

Pier I, intended to be the home port pier, is 370m long, 30m wide, has a top deck elevation of +2.9m CD, and is supported by double pile bents capped with a 6.5m wide by 1m thick pile cap. The double pile bents have two rows of seven 1.2m diameter steel piles, and are spaced at 20m intervals along the pier. Two smaller mooring dolphins supported on battered piles are located beyond the outer end of the pier. The pier and dolphin structures were fabricated in the model using aluminum tubing piles, and a combination of PVC and timber for the pile caps and pier decking. The decking was rigidly connected to the pile caps and piles, and a cement mortar was used to replicate the seabed below the pier. Pier III was designed and modelled in a similar fashion, with the main differences being that the pier deck was 350m long by 18m wide, and that the substructure comprised single rows of five 1.2m diameter piles spaced at 10m intervals. Two mooring dolphins supported on battered piles were also modelled beyond the outer end of Pier III.

The mooring line and fender simulators used in the model to replicate the behaviour of the prototype mooring lines and fenders were installed on each model pier to match initial mooring layout designs. Photographs of the two model piers during preparations for the operational testing are shown in Figure 5. The mooring simulators were each configured to simulate the non-linear load-elongation behaviour of one or more prototype mooring lines. Mooring line tensions were recorded using shear beam load cells, and pre-tensions were applied using counter weights. The fender simulators were configured to simulate the non-linear load-deflection response of one or more prototype fenders, including the fender buckling at high loads. Shear beam load cells were used to measure the loading on each model fender.

Figure 5: Photographs of Pier I (left) and Pier III (right) in the physical model during preparations for the moored ship response phase.



The reactions at each fender and the tension loads in all mooring lines were measured continuously during the study and compared with safe working limits. Tests were conducted with both vessels moored at their berths simultaneously, the Oasis at Pier I and the Grandeur at Pier III. Additional tests were performed with the Grandeur moored at Pier I and the Oasis removed from the physical model.

The 6-axis motions of the vessels were measured using two high-precision motion tracking systems manufactured by Qualysis Inc. The Qualysis cameras use infra-red light reflections from reflective markers fixed to the model ships to determine the position and orientation of the vessels in real time with very high precision. The ship motions were compared against thresholds for passenger comfort and safety.

The effect of a steady wind blowing the vessels, either directly on or directly off their berths, was simulated in the model as a steady horizontal force. For each ship, the wind force was applied through a horizontal string that was attached at the center of windage. The steady force was generated by running the string through a pair of low friction pulleys and suspending a weight from the other end of the string.

A segmented directional wave machine was used to generate short-crested waves matching the operational sea and swell wave conditions in Figure 4. Seas and swells approaching from different directions were simulated together in some cases. Twenty-four capacitance wave probes were positioned throughout the model to measure the wave agitation levels in locations of interest. A summary of the test conditions and model setup for the different moored ship test series is shown in Table 1, while Figure 6 shows the Oasis of the Seas moored at Pier I.

Table 1. Summary of test conditions for the moored ship response phase of the study.

Test Series	Wave Directions	Pier I - North		Pier III - West	
	(°N)	Vessel	Wind	Vessel	Wind
Cal	210, 240, 270	N/A	N/A	N/A	N/A
A	210, 270	Oasis	Off Berth	Grandeur	Off Berth
B	210, 270	Oasis	None	Grandeur	None
C	210, 270	Oasis	On Berth	Grandeur	On Berth
D	210, 270	Oasis	Off Berth	Grandeur	None
E	210, 270	Oasis	None	Grandeur	None
F	210, 270	Grandeur	None	None	None

Figure 6: Oasis of the Seas moored at Pier I.

Additional information on the modeling and assessment of moored ship behaviour is provided in Knox *et al* (2014). The remainder of this paper focuses on the modeling and assessment of structure performance.

Phase 2 – Modelling and Assessment of Structure Performance

After the first phase of the study was completed, the setup of the model was changed so that the wave-structure interactions at the port under more intense storms could be investigated (without the vessels in the port). The main change to the model was removing the pier decks, mooring simulators, fender simulators and related instrumentation systems that had been used in the Phase 1 study. These were replaced with pier decks that provided a more detailed simulation of the conceptual designs and which included instrumentation to measure wave uplift pressures and forces. Also, revised designs for the land reclamation area, including scour protection at the foot of the vertical wall around the perimeter of the reclamation area, as well as various rock and concrete armour unit revetments, were simulated and studied in the structure performance assessment phase of the study.

One of the primary concerns facing the design team was designing for, and if possible, mitigating the uplift forces and pressures on the pier decks and their support elements. This concern was exacerbated by the unfavourable subsurface conditions beneath the piers; specifically, the presence of calcareous sediments limits the tensile (uplift) capacity of conventional driven piles. As such, the wave uplift loads were the controlling factor with respect to pier design. The deck of both model piers was initially constructed to represent solid concrete decking; however, in order to mitigate the wave uplift forces, Pier I was also modelled with the central portion of the deck comprised of an open grating. The overall ratio of solid to open decking in this case was approximately 50%.

Selected portions of both model pier decks were designed so they could be fitted with instrumentation to measure either the wave forces acting on a complete deck panel (including the pile cap), or the local wave pressures on specific deck elements. Specially designed deck panels fitted with pressure and force sensors were installed in these locations. As shown in Figure 7, the locations where the instrumented deck panels could be installed were concentrated at the nearshore end, the offshore end, and in the middle of each pier. In each location, the model was designed so that a single pressure panel could be installed between a pair of force panels (see Figure 8). The set of three instrumented deck panels were sometimes moved to new locations as a set between different test series. Dummy deck panels without instrumentation were inserted whenever the instrumented panels were removed. Information on the wave induced pressures and loads acting on the inner, outer and central portions of both pier decks was obtained in this way

The pressure panels were rigidly connected to the pile caps and each fitted with several pressure sensors facing both down and up. The force panels were free floating elements (they did not touch the piles or pile caps) and were each suspended from a 6-axis load cell located above the centre of the force panel. Each load cell was mounted to a rigid beam supported on adjacent pile caps by threaded rods.

Figure 7: Location of force panels and pressure panels in the physical model.

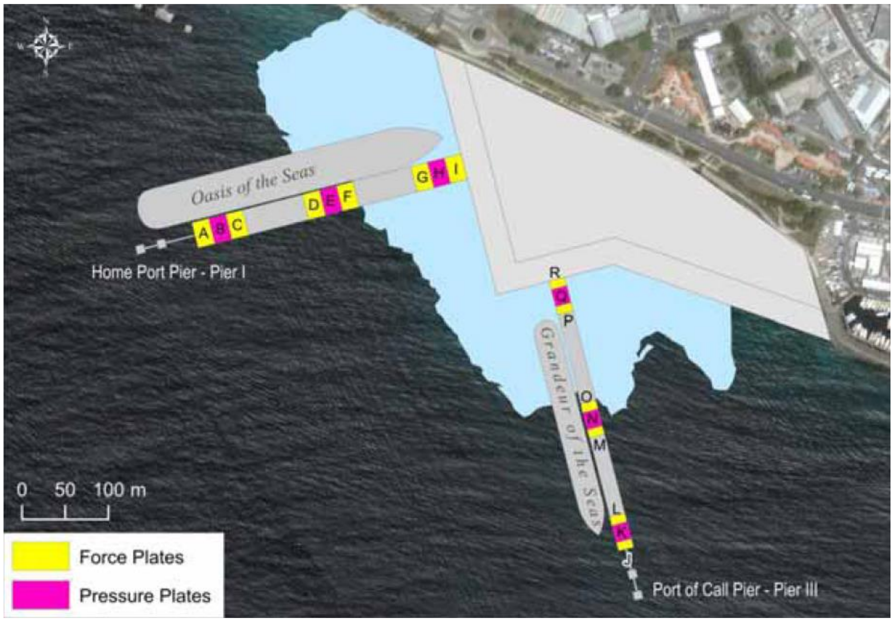
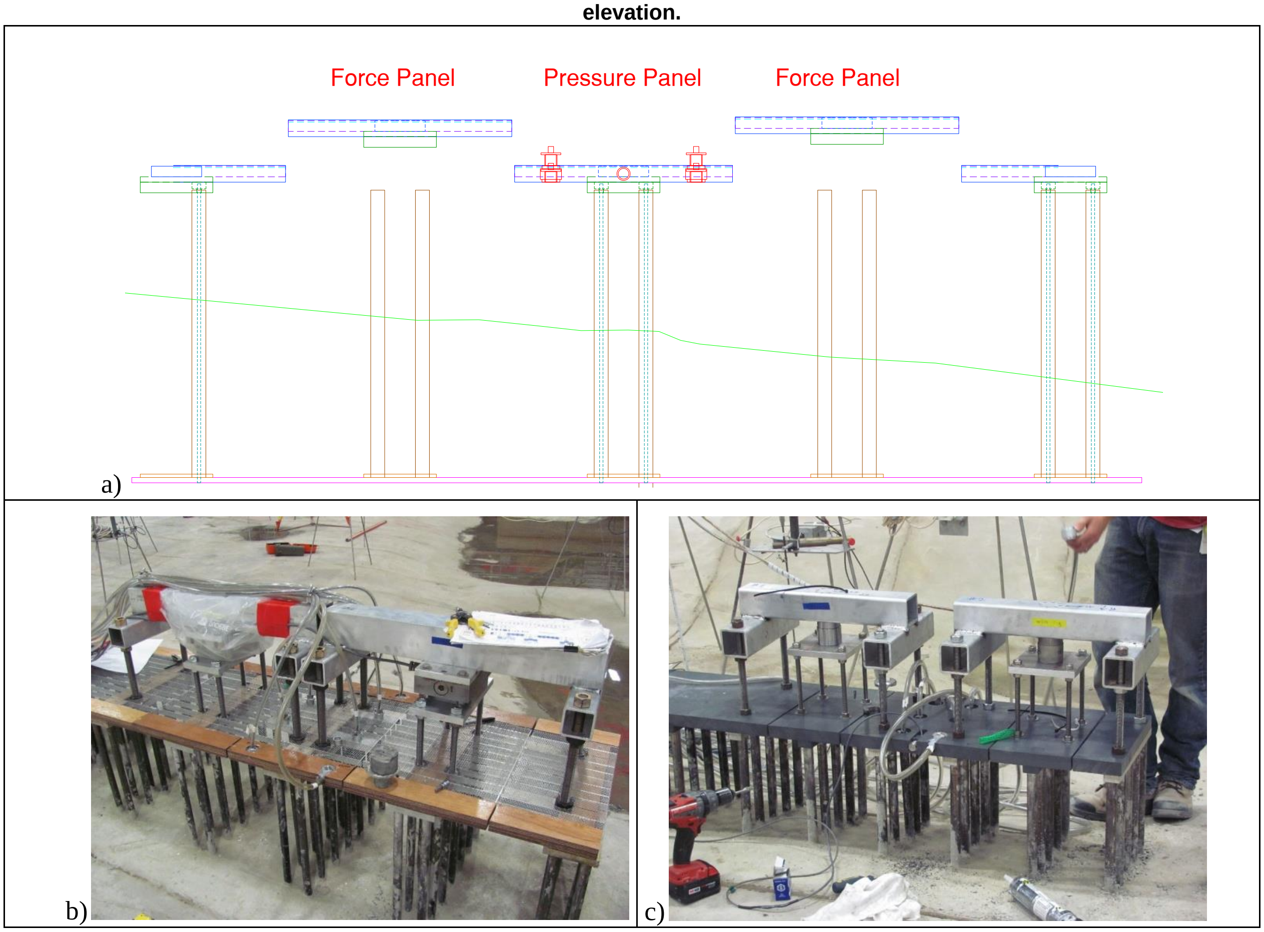
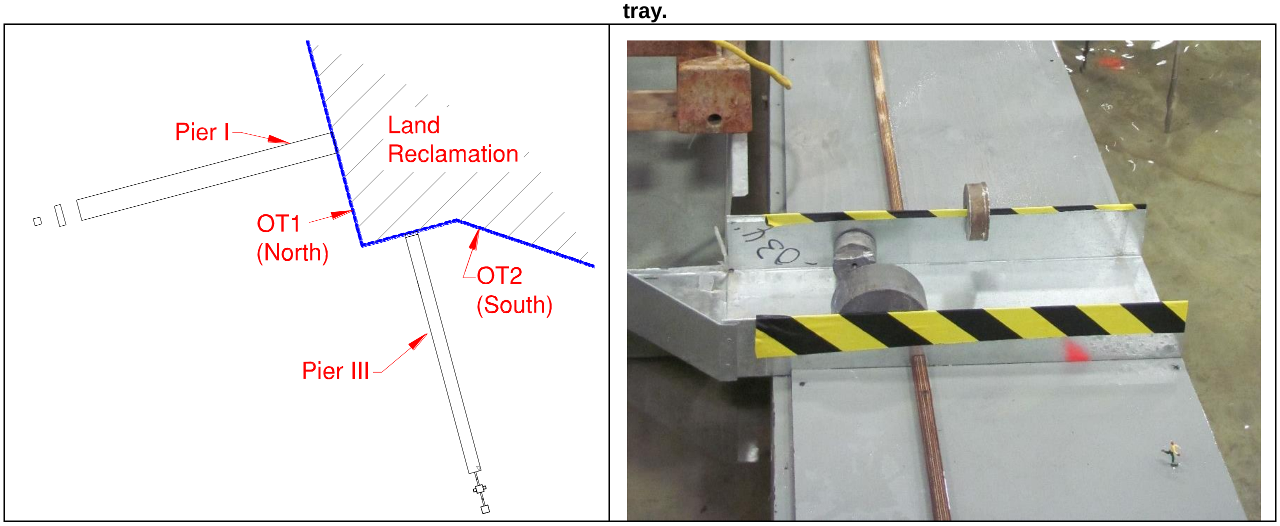


Figure 8: Conceptual design of the force and pressure panels (top) and realization in the physical model: b) Pier I with 50% open deck; c) Pier III with solid deck. Note, the force panels shown in the schematic (a) are shown above their true elevation.



The seaward edge of the land reclamation area was initially modelled as a vertical wall, with a 30m wide ”hardscape” promenade at +2.5m CD around the perimeter of the land reclamation area. During the Phase 2 tests, wave overtopping rates onto the promenade area were measured at two locations (see Figure 9). The wave overtopping measurement system consisted of a collection tray set at a certain elevation, conveying the overtopping flows into a reservoir fitted with a water level sensor. After the initial tests, the elevation of the promenade area was raised to +3.15m CD. In addition, several modifications were made during the testing program to reduce wave overtopping onto/beyond the promenade, including: raising the elevation of the collection tray (to ascertain the overtopping flows at various elevations); adding a flood wall set back 20m or 30m inland from the seaward edge of the promenade (to various elevations); and installing a 0.5m high sea wall along the seaward edge of the promenade. The sea wall was constructed in 10m sections with 1m gaps to allow water to drain back to the sea. The effects of building a 3:1 sloping berm in front of the flood wall (to simulate a landscaped berm) were also investigated.

Figure 9: Wave overtopping at the land reclamation area – measurement locations (left) and photograph of the catchment tray.

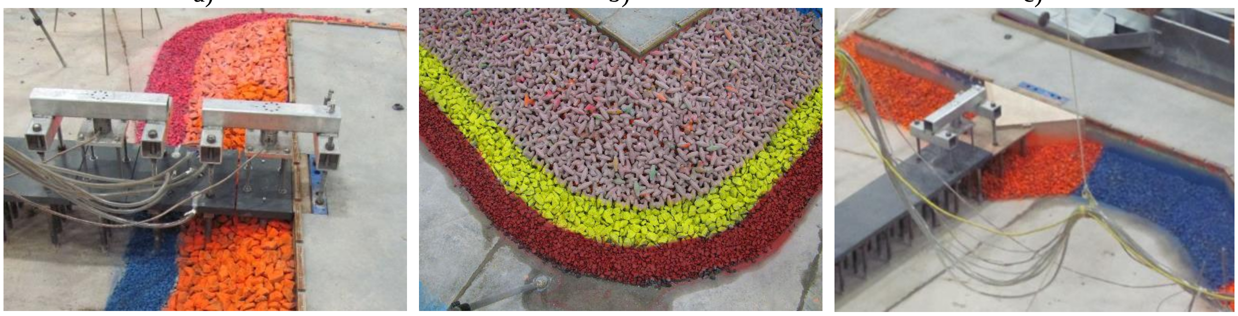


Tests were completed with several types of protective structures around the seaward perimeter of the land reclamation area. Initially, the vertical wall was protected with different designs of low profile scour protection mats using armour stones varying in size from 0.5 tonnes to 4 tonnes. The stability and performance of three different revetments fronting the land reclamation area was also investigated. The revetment’s ability to reduce wave overtopping and uplift forces on the adjacent pier decks was studied, as well as the performance of the revetment armour layers to withstand wave attack. Initially, a rock revetment with 5-10t armour stone was investigated at two different crest elevations: +2m and +3m CD. The second type of revetment used Core-Loc® units that represented ~5.2m^3 units at full scale. The third revetment design assessed in the model was a “hybrid” design featuring a vertical wall fronted by a low-crested rock berm with armour stone sloping down at 1:3 from -5m CD to the dredge elevation of -11m CD. This hybrid design featured a triangular plan form configuration at the landward end of the piers that was intended to reflect wave energy away from the piers and potentially lower the wave induced loads on the pier deck. Several different sizes of armour stone were investigated for the hybrid design. The three different model revetments are shown in Figure 10.

The loading and response of the port structures in extreme conditions was assessed using short-crested realizations of the storm wave conditions summarized in Figure 4. The water level was varied from +0m CD (LAT) to +1.7m CD (estimated range in extreme water levels) to understand its effect on the wave loads and overtopping flows. Twenty-two capacitance wave probes were positioned around the basin to measure the wave conditions in key locations.

Figure 10: Several different revetment designs were modelled and assessed: a) rock armour, b) Core-Loc® armour, c) hybrid low crested rock armour structure with triangular wall at the root of the pier.

a) b) c)



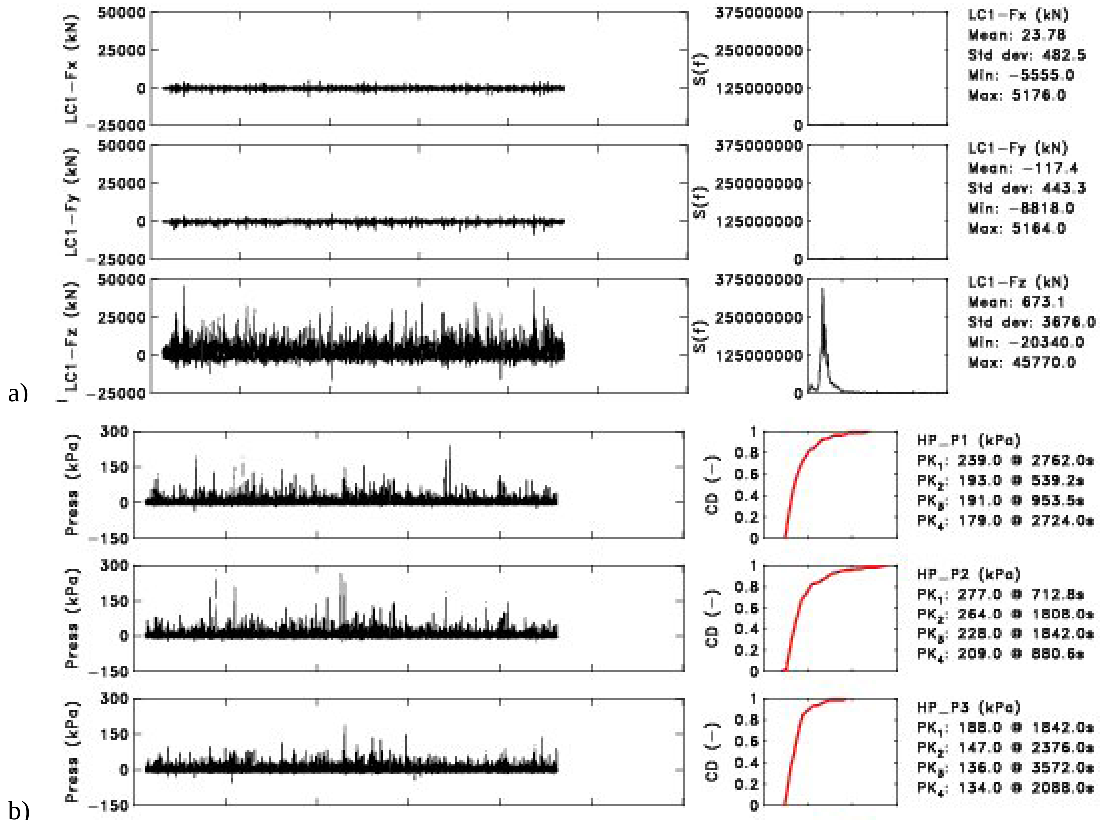
DATA ANALYSIS

NRC's GEDAP software was used for all primary analysis of the measured data. GEDAP is a general-purpose software system for the synthesis, analysis and management of laboratory data that also includes modules for real-time experiment control and data acquisition functions. Standard GEDAP time-domain, frequency-domain, peak detection and statistical analysis algorithms were applied to analyse in considerable detail the wave conditions and overtopping levels measured in the model, as well as the loads on the force panels and the pressures on the pressure sensors.

Forces and Pressures

The 6-axis load cell and pressure sensor outputs were sampled at 500 Hz and 1000 Hz (71 Hz and 143 Hz prototype scale), respectively. The instruments used for measuring force and pressure also captured the inertial loads due to vibrations that were proportional to the mass and accelerations of the force and pressure panels. To separate these spurious inertial loads from the hydrodynamic loads, free vibration tests were undertaken on the force panels and low-pass filtering with a cut-off frequency of 20 Hz was implemented. The pressure sensor data was not filtered, as the spurious signals due to vibration were insignificant. The 6-axis load cell outputs were resolved to determine the characteristic forces (F_x, F_y, and F_z) and moments (M_x, M_y, and M_z) on each force panel, as well as the overall response - the horizontal force (F_h), total force (F) and the overall moment (M_o). The pressure fluctuations recorded at each sensor were analyzed individually, and also combined to obtain estimates of the spatially averaged pressure (and force) acting upwards and downwards on the deck panel, as well as the net (differential) pressure. The GEDAP analysis routines produced graphical presentations of the model data, and also allowed for the tabular organization of many key parameters derived from analysis of the measured time-histories. Examples of the GEDAP analysis plots displaying the pressure and force fluctuations recorded during a typical 3-hour long test with short-crested waves are shown in Figure 11.

Figure 11: Typical graphical outputs from the GEDAP force and pressure analysis: a) time series and spectra for Fx, Fy, and Fz; b) time histories and cumulative distributions for three pressure sensors).



RESULTS AND DISCUSSION

A total of 277 tests were conducted to assess and optimize the design of the new port structures in Phase 2 of the study. Initially, the effectiveness that different features on the land reclamation area had in mitigating wave overtopping was investigated. The focus then shifted to defining the wave uplift forces and pressures on the two pier decks. Lastly, the performance of different revetments fronting the land reclamation area was investigated, both in terms of their own stability, but also their effect on reducing the hydrodynamic forces on the pier decks and the wave overtopping volumes at the land reclamation area. The outputs from the physical model were used to help the design team select the most preferred types of coastal structures and their features, and also provide valuable data leading to an optimized detailed design.

Wave Uplift Response Signature

Many studies have investigated the hydrodynamic forces on similar pile-supported deck structures under wave action. El-Ghamry (1965) and also Wang (1970) investigated the effects of waves imparting lateral and vertical pressures on horizontal decks. These authors describe the force-time signature of a single event typically having a large initial peak (impact) followed by a slowly-varying quasi-static load. Kaplan (1979) and Kaplan et al (1995) developed a semi-empirical model for estimating wave loads on horizontal and vertical members. Shih and Anastasiou (1992) investigated these hydrodynamic loads at small scales. Much research undertaken as part of the Exposed Jetties research project in the United Kingdom has given additional information on wave-in-deck loads through the research of Tirindelli et al, (2002), and Cuomo et al (2003, 2004). More recently, efforts to describe the very complex phenomena by using carefully calibrated CFD models have been undertaken, such as in Cornett et al (2013). Despite all of this research, the wave-in-deck loads in the surf and splash zone deals with such complex hydrodynamic issues such as non-linear waves, fluid viscosity/turbulence effects, and air gap/entrainment, to name a few, making reliable estimates of design pressures and forces very challenging. The specific characteristics of the time-series response are dependent on many factors including the structure geometry and freeboard, as well as the incident wave conditions. The characteristics (magnitude and duration) of the peak impact impulse generally shows greater variation than the quasi-static pressure, and this variation also depends on many factors, such as the shape of the wave at impact and the presence (or lack thereof) of entrained air or air pockets. Figure 12 shows typical time series data recorded at the Force-Pressure-Force arrays at the outer end of each pier during a test with energetic wave conditions. The three lines in each figure show the average uplift pressure derived from each of the two force panels (i.e. P=F/A), and the average uplift pressure obtained by combining the signals from the individual pressure sensors mounted on the adjacent pressure panel. The small offset in the timing of the forcing at Pier I (compared with Pier III) is due to the fact that the waves in this test approached the site from a southerly direction, propagating roughly parallel to the axis of Pier 1 but roughly broadside to Pier III.

Clearly, the complex and dynamic nature of the wave uplift loads, in particular the large spatial and temporal variation in these loads as the waves interact with the structure, is an important consideration in the development of design loads to be used in the structural analysis and design of the piers. This issue is discussed in more detail below, along with a discussion of the reduction in wave uplift loads achieved with the open deck concept.

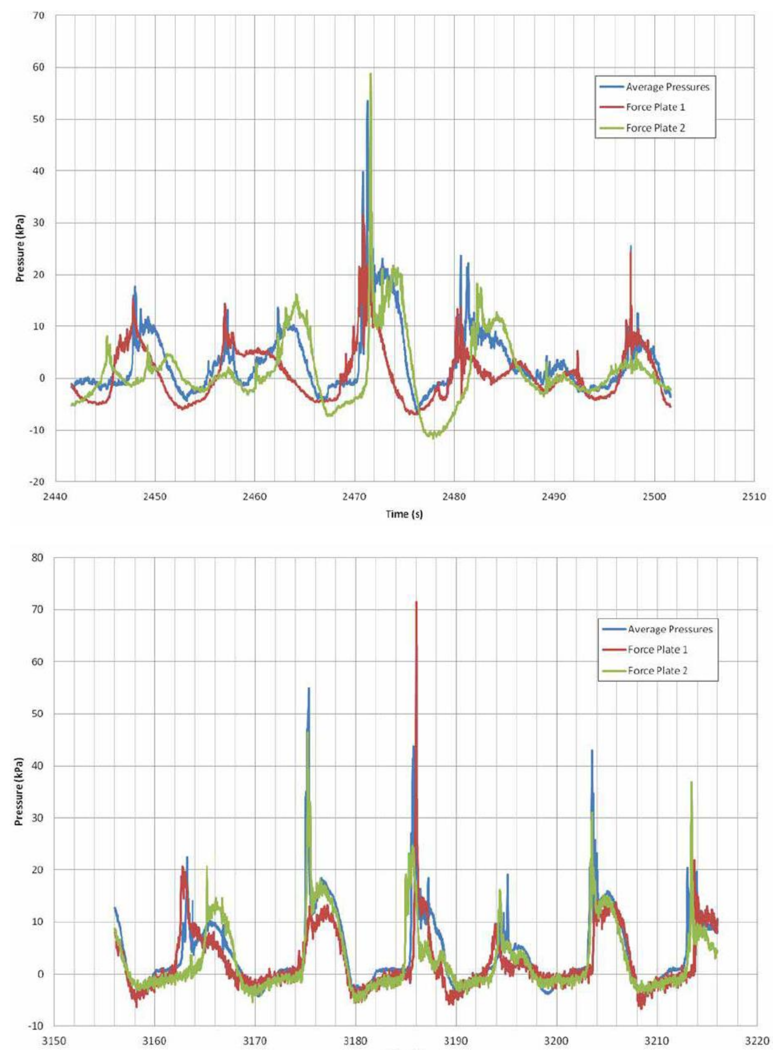


Figure 12: Example time series of spatially averaged uplift pressure from the F-P-F panels: a) the outer end of Pier I; b) the outer end of Pier III.

Application of Physical Model Results to Structural Design

The selection of the design event for a marine structure is ultimately a cost versus risk decision; in general, designing to resist a more severe event will cost more, but will result in a reduced risk of damage. The acceptable risk of damage must consider the consequences of damage/failure. Ultimately, it is the Client's responsibility to select/define the design life and acceptable risk of failure for the structure; the return period of the design event is dictated by the selection of these two parameters. Frequently, it is necessary for the Engineer to guide the Client though the process of selecting an appropriate design event.

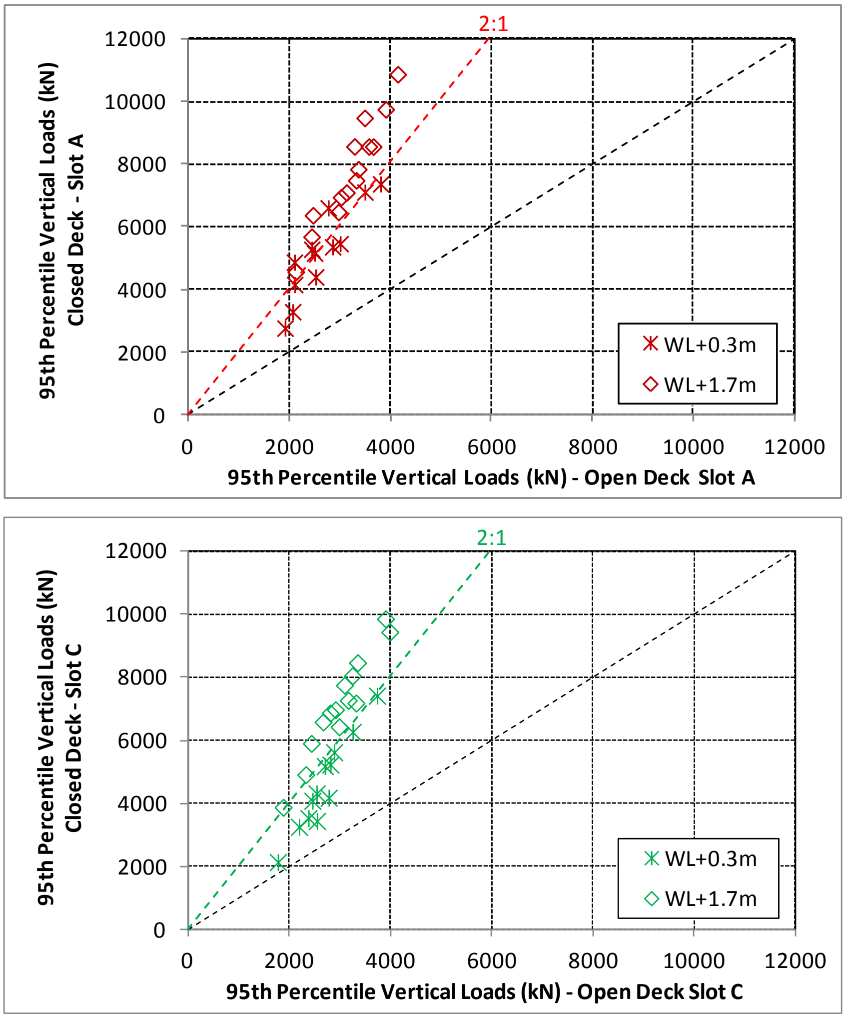
In this case, the design event is characterized by the wave and water level conditions generated by a hurricane with the specified return period. Putting aside the question of selecting the appropriate return period for the design event, the design (wave uplift) loads must also consider the temporal and spatial variation in loads generated by the specified "design wave". As noted above, the uplift loads generated by an individual wave are characterized by an initial impact load of significant magnitude but very short duration, followed by a slowly varying (quasi-static) load. The structural analysis and design of the pier must consider the nature of the applied loads, as well as the response of the structure to these loads, including anticipated damage mechanisms and failure modes. Based on discussions with the structural design team, and review of guidance provided in international design standards and codes, the following approach was adopted for the structural design of the piers:

- Design for "collapse prevention" under the specified extreme design event;
- Use "smoothed data" from the physical model results as input to a quasi-static structural analysis (a smoothing interval in the order of 0.25 s was considered appropriate);
- Lower duration/higher frequency loads shall be considered in the design of the pier superstructure (deck), but are not relevant to the pier substructure (piles) due to structural damping effects.

Influence of Semi-Open Deck on Wave Uplift Loads

One of the key objectives of the physical modelling study was to determine the reduction in wave uplift forces associated with semi-open (porous) deck structures compared with decks that are 100% solid. An alternative deck design that was ~50% solid and ~50% open was modelled and tested at Pier 1 (see Figure 8b). For the semi-open deck, a stiff porous wire mesh was used to replace roughly 50% the solid decking. The edges of the deck and the pile caps remained solid, while the inner part of the deck was porous.

The peak wave uplift forces recorded at the offshore end of Pier I were compared for like wave conditions with semi-open and solid deck structures (see Figure 13) . The data indicates that for these conditions, the wave uplift loads on the instrumented deck panels were roughly half for the semi-open deck as compared to the solid deck, with the increase being more pronounced at higher water levels. The reduction in wave uplift loads at the root of Pier I was also investigated. The test results showed that for this location, the peak loads on the semi-open deck were up to a three or four times smaller as compared to the solid deck; however, this result was found to be highly dependent on the length of the averaging duration assumed in determining the peak load. Additional insight into the performance of the semi-open deck concept was obtained through review of the pressure sensor data for the sensors located within the semi-open part of the deck versus those embedded within the solid portion of the pier deck. Lower instantaneous pressures were observed at the sensors embedded within the porous grating, showing a release of the wave pressure through the pores in the decking.



Wave Conditions			
WL	Dir	T_p	H_{m0}
m, CD	°N	s	m
0.3	240	8	4.5
0.3	210	10	4.5
0.3	240	10	4.5
0.3	270	10	4.5
0.3	210	10	5.5
0.3	240	10	5.5
0.3	270	10	5.5
0.3	210	10	6.5
0.3	240	10	6.5
0.3	270	10	6.5
0.3	240	12	6.5
0.3	240	12	7.5
1.7	240	10	2.5
1.7	240	10	3.5
1.7	240	8	4.5
1.7	210	10	4.5
1.7	240	10	4.5
1.7	270	10	4.5
1.7	210	10	5.5
1.7	240	10	5.5
1.7	270	10	5.5
1.7	210	10	6.5
1.7	240	10	6.5
1.7	270	10	6.5
1.7	240	12	6.5
1.7	240	12	7.5

Figure 13: Reduction in peak uplift loads for the semi-open deck structure: a) force panel A; b) force panel C.

The physical model results demonstrate that a significant reduction in the wave uplift loads may be achieved with a semi-open deck concept, with the potential for some cost savings associated with a reduced uplift demand on the piles. The primary disadvantage with the open deck concept is increased maintenance requirements. At this time, the base design for the project is based on a solid deck concept; however, discussions continue regarding the potential application of a semi-open deck concept on this project.

Variation in Wave Uplift Loads Along the Pier

Tests were undertaken with the Force-Pressure-Force panels located in three locations along each pier – an inner (landward), central (mid-length), and outer (seaward) location. In general, the highest hydrodynamic loads were measured on the panel closest to the vertical seawall (the landward location). The peak loads on the panel immediately adjacent the wall were up to 20% higher than those measured at the central and outer locations for Pier III, and up to 30% higher for Pier I. The peak uplift forces at the next force panel (close but not immediately adjacent the sea wall) were significantly lower, suggesting a possible node/anti-node standing wave reflection pattern underneath the pier.

SUMMARY AND CONCLUSIONS

A large new cruise ship terminal is being developed on the open coastline outside the Port of Bridgetown Barbados. The new facility will include three large pile-supported ship piers with berths for six large cruise ships, approximately 15 acres of land reclamation, and associated landside development. Two key challenges for the project were: the risk that the prevailing seas and swells would cause downtime due to excessive moored ship motions; and designing the marine and coastal structures to resist the significant wave forces and overtopping flows generated by passing hurricanes.

Large scale physical hydraulic modelling is an excellent approach for investigating and developing solutions to these types of complex hydrodynamic problems. In a well designed physical model, the complex wave conditions near a project site and the propagation and transformation of the waves over complex nearshore bathymetries and around port structures can be simulated with good accuracy. The behaviour and response of moored ships to winds and waves can be faithfully replicated in a physical model. A physical model can also be used to reliably simulate the interaction of extreme waves with port structures and measure the loading, and response of these structures with precision. Critical issues such as the uplift pressures and forces on pier decks, the stability of various rubble-mound structures, and the performance of overtopping mitigation measures can be studied and design solutions can be developed and tested. Moreover, as demonstrated in this study, physical modelling is a valuable tool for assessing the performance of alternative layouts and designs, and for optimizing designs to suit site-specific local conditions. Physical modelling represents the state of the art in understanding moored ship response and wave interactions with complex marine structures, and is recognized as the standard of care for large coastal engineering projects.

The results of the moored ship response phase of the study were used to define a range of wind and wave conditions where ship motions, mooring line loads, and fender forces were within acceptable limits. Also, the moored ship response data was used to calibrate and validate a numerical model that was then used to develop downtime estimates for the proposed facility under a wider range of conditions. The second phase of the study, which focused on the performance of the port structures in storm conditions, generated a large body of knowledge and data that allowed the design team to advance and improve the port structure designs in several important ways. First, the design of the land reclamation was revised to incorporate several measures to reduced wave overtopping and improve flood protection. Second, a rubble-mound revetment protecting the toe of the land reclamation was enlarged, optimized and validated. Third, the study was used to establish design loads for two alternative pier deck designs, one with a solid deck slab and the other with large grated openings. These results were used by the design team to develop detailed structural designs for the new pier structures. These physical model studies played a key role in advancing the design of the new port, and their results were crucial in assessing various alternatives, optimizing preliminary designs, and validating the layout, costing and construction of the new facility.

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