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Physical Modelling and Design Optimizations for a New Container Terminal at the Port of Moin, Costa Rica

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ABSTRACT

The role of physical modelling in advancing port design is demonstrated by means of a case study in which two large-scale 3D physical modelling studies were conducted to support the design of a new container terminal at the Port of Moin, Costa Rica. The first model focused on validating the stability of the new rubble-mound structures in extreme conditions. Several 2D and 3D models of portions of the north, east, and stub breakwaters were constructed at 1:43 scale. The study helped to develop and confirm the final design of the breakwaters, including their configuration and the sizing of armour units and armour stone. The second model was used to assess wave agitation and moored ship motions at the new container terminal. A 1:82 scale model of the existing port, the surrounding bathymetry, the proposed dredging, and the Phase 1 expansion was constructed, and then modified to simulate several alternative port layouts. The study also investigated the potential impacts on operations at the existing port due to the new terminal.

PROJECT INTRODUCTION

Existing port facilities at Moin are the largest in Costa Rica, handling 1.05 million TEUs in 2013, but are limited to vessels of up to 2,500 TEU capacity. The first phase of expansion (refer to Figure 1) calls for a dredged access channel and turning basin, construction of a 1.5km breakwater with a 40ha container yard, 650m of quay, and 2 berths equipped with 6 post-*Panamax* cranes. Upon completion of the final phase, the new terminal will have an area of 80ha, with 1,500m of quay, 5 berths, a 2.2km breakwater, and deeper access channel, serving as a shipping hub for the Caribbean and Central America. The expansion project represents an overall investment of approximately 1 billion USD. Two large-scale physical models were used to assess and optimize the preliminary designs for the first phase of expansion.

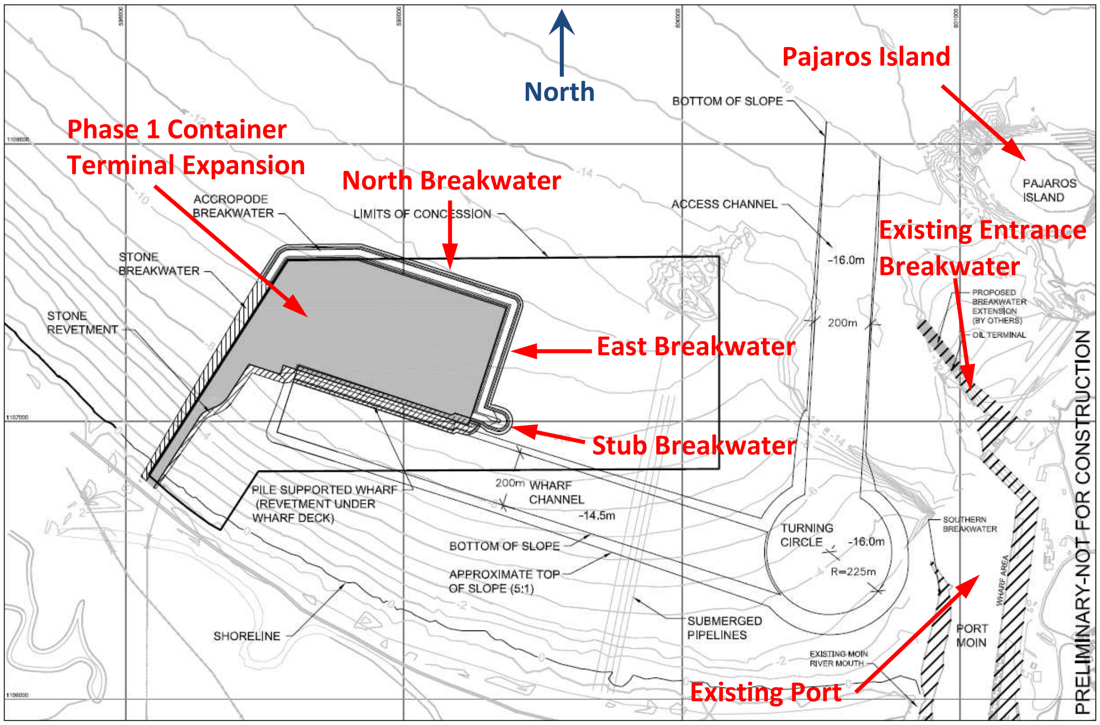

Figure 1. Concept layout plan for the first phase of the proposed expansion.

PHYSICAL MODELLING TO ASSESS BREAKWATER STABILITY

The first physical modelling study focused on validating the stability of the new rubble-mound structures in extreme conditions and optimizing their designs. Several 2D & 3D breakwater models were constructed and tested in the 63m by 14m by up to 1.5m deep Coastal Wave Basin (CWB) of the National Research Council of Canada (NRC). The CWB is equipped with a powerful computer-controlled multi-mode wave machine capable of generating irregular long-crested waves with significant wave heights up to ~0.35m. A 1:43 model scale was selected as it suited model concrete armour units that were available and represented a reasonable compromise between minimizing scale effects and boundary effects without exceeding the limits of the CWB facility. The armour stone and model Accropode units (man-made unreinforced concrete blocks designed to resist the action of waves on breakwaters and coastal structures, see CLI 2012) were selected to closely match the submerged stability of the prototype units and materials in a realistic manner, taking into account the density differences of the armour units and the fluid in the model and prototype conditions.

Sea level fluctuations at the port due to tides are minor. Extreme design waves at the test site are estimated to exceed 6m (significant wave height) approaching mainly from the north-easterly direction. The wave conditions selected for stability testing included significant wave heights ranging from 1.7 to 7.4m with peak periods between 6.0 and 15.0s.

A simplified model of the seabed bathymetry extending offshore from the north breakwater was constructed in the basin for these studies. This bathymetry was required to produce model waves that would be similar to the conditions in nature.

This was particularly important since the height and energy of the waves reaching the breakwaters would be limited by the shallow local water depth. The basin floor was assigned a depth of -28.3m. A 1:20 sloping bathymetry was constructed up to an elevation of -14.2m, where the slope changed to ~1:250 and rose gradually to a depth of -9.8m. This gently sloping bathymetry presents a good representation of the typical seafloor slope offshore from the proposed port structures.

Five different series of tests, each with different model structures, were conducted in this study. Careful attention was given to the location, dimensions, and composition of each structure to ensure that it accurately replicated the proposed designs. All model armour units were placed individually by hand according to the methods specified by Concrete Layer Innovations (see CLI 2012). A photographic damage analysis system comprising six remotely-operated digital cameras was used to monitor the movement of Accropode units and armour stones on the surface of the breakwater. The cameras were mounted on tripods and positioned to monitor different sections of each model structure. Since each camera remained fixed throughout a test series, the movement of individual stones and armour units could be detected by comparing photographs taken before and after each test segment.

All wave conditions were verified during undisturbed wave trials conducted prior to building the model structures. The command signals were tuned so that the average measured wave conditions were in close agreement with specifications.

The first test series sought to establish appropriate Accropode unit sizing for the north and stub breakwaters. In contrast to the design concept shown in Figure 1, the original layout for the container terminal development featured the stub breakwater extending eastward from the NE corner of the north breakwater. At the time of model construction, this older layout was used (see Figure 2a). The model was oriented such that the angle of wave attack was most critical for the north breakwater and stub breakwater configuration. The model structures were exposed to a series of increasingly severe seastates and higher water levels in order to assess their relative performance. An overtopping measurement system was deployed behind the north breakwater crest. The system consisted of a water storage reservoir, a capacitance wave gauge to measure the depth of water in the reservoir, and a metal tray to convey the overtopping flow from the crest of the breakwater into the reservoir (see Figure 2b).

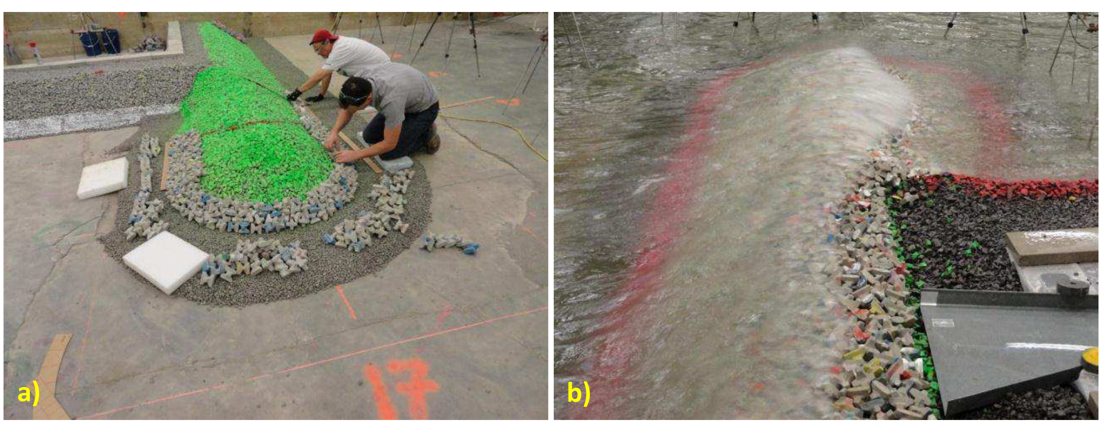

Figure 2. Stability assessment for the north and stub breakwaters.

Test results showed that 7.3m^3 Accropode armour units remained stable on the stub breakwater roundhead, while 5.3m^3 units performed adequately on the main breakwater trunk and on the lee side of the stub breakwater. These two unit sizes satisfied the CLI damage performance criteria. In general, the toe armour performed very well, sustaining only slight damage along most of the structure, with additional (but acceptable) movement at the rear of the roundhead.

During the course of the first test series, an updated design for the container terminal was received in which the stub breakwater was relocated further south (into shallower water), leaving the east side of the reclamation exposed to wave conditions similar to those along the north side (refer to Figure 1). The earlier test results indicated that it may be possible to use the smaller 5.3m^3 Accropode units on both the north breakwater and the stub breakwater roundhead in its new location. Therefore, the second test series sought to investigate the use of the smaller armour units on the east and stub breakwaters. The new location of the stub breakwater was in slightly shallower water, hence the model structures were constructed at a different location on the model bathymetry.

It was necessary to assess and verify the design for waves approaching from two dominant directions, so the basin was partially divided into two halves and two model structures were constructed, one on each side of a dividing wall. Two model structures that were mirror images of the interior corner formed by the intersection of the east and stub breakwaters were modelled and tested (see Figure 3a). The model structure on one side was oriented such that incoming waves approached from 23°, while the other model was oriented to simulate waves approaching from 53° (such that wave energy was focused directly into the interior corner). The 53° model featured an overtopping measurement along the east breakwater just before the interior corner.

The model structures were similarly exposed to a series of increasingly severe seastates and higher water levels in order to assess their relative performance. Based on the test results, it was evident that 5.3m^3 armour units could perform adequately as primary armour along the east breakwater and at the transition between the east breakwater and the stub breakwater. It was clear that waves approaching from a more north-easterly direction (53°) were able to focus more energy at the interior corner than identical waves approaching from a more northerly direction (23°). This concentration of wave energy led to large and frequent wave runup and overtopping at the transition between the east breakwater and the stub breakwater.

Using the same setup, an additional round of testing focused on assessment of the entire stub breakwater for the more critical incident angle (53°, see Figure 3b). The lee quadrant of the stub breakwater roundhead showed some general settlement of the 5.3m^3 Accropode units in shakedown testing, and showed only minor unit movements up to and during the overload event. However, during exposure to the 200-year wave event at high water, a unit in the 3rd row was removed from the structure. Based on the test results, it was evident that 5.3m^3 units could perform adequately along the east breakwater, at the interior corner, and on the lee side of the structure. However, larger armour units were recommended for armouring the roundhead portion of the stub breakwater.

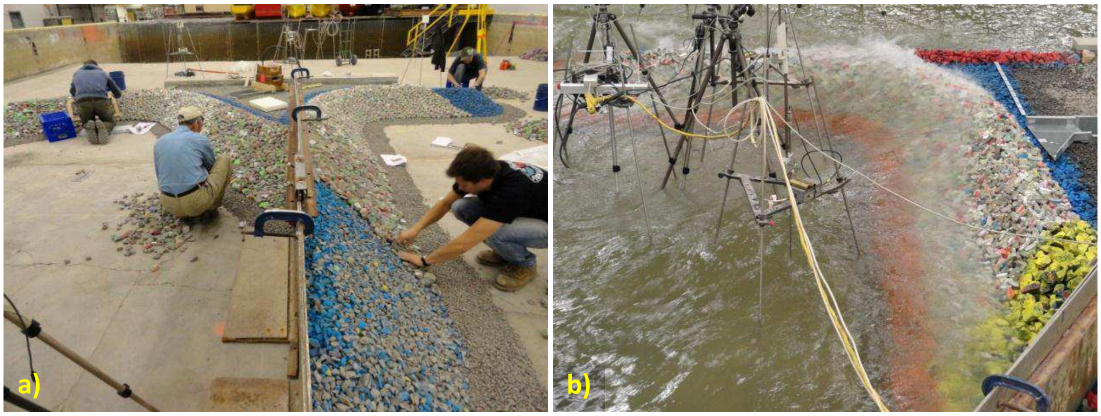

Figure 3. Testing stub breakwater stability for 2 different incident wave angles.

Due to project schedule constraints, a 2D approach was used to further model and assess the performance of the north breakwater. Six parallel concrete block walls were constructed in the basin to form five 50m (full scale) wide flumes in which five alternative cross-sections for the main breakwater could be tested at the same time (see Figure 4a). One model was built according to the base design, whereas the four other models were used to assess the performance of minor variations on the base design, mainly by increasing the crest height or increasing the crest width. An overtopping collection system was located behind each cross-section.

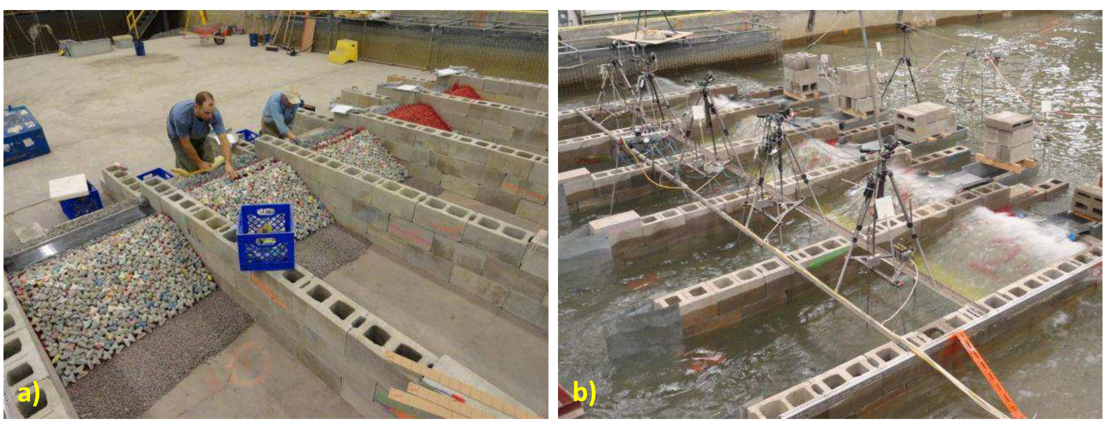

Figure 4. Testing breakwater stability for 5 alternatives of the north breakwater.

The model structures were similarly exposed to a series of increasingly severe seastates and higher water levels in order to assess their relative performance (see Figure 4b). The 5.3m^3 Accropode units performed well on the base design; however, runup and overtopping were greater than desired. The base design for the lee side of the main breakwater crest did not provide sufficient protection against the erosion of fill material by overtopping flows. Therefore, an improved design which extended the scour pad filter stone further behind the crest was recommended.

Of the five alternative design sections studied, cross-sections #3 (1.5m increased crest height, see Figure 5a) and #4 (same crest height but 7m wider crest width, see Figure 5b) performed best at minimizing the frequency and severity of overtopping discharges, thereby mitigating the potentially harmful effects of wave overtopping.

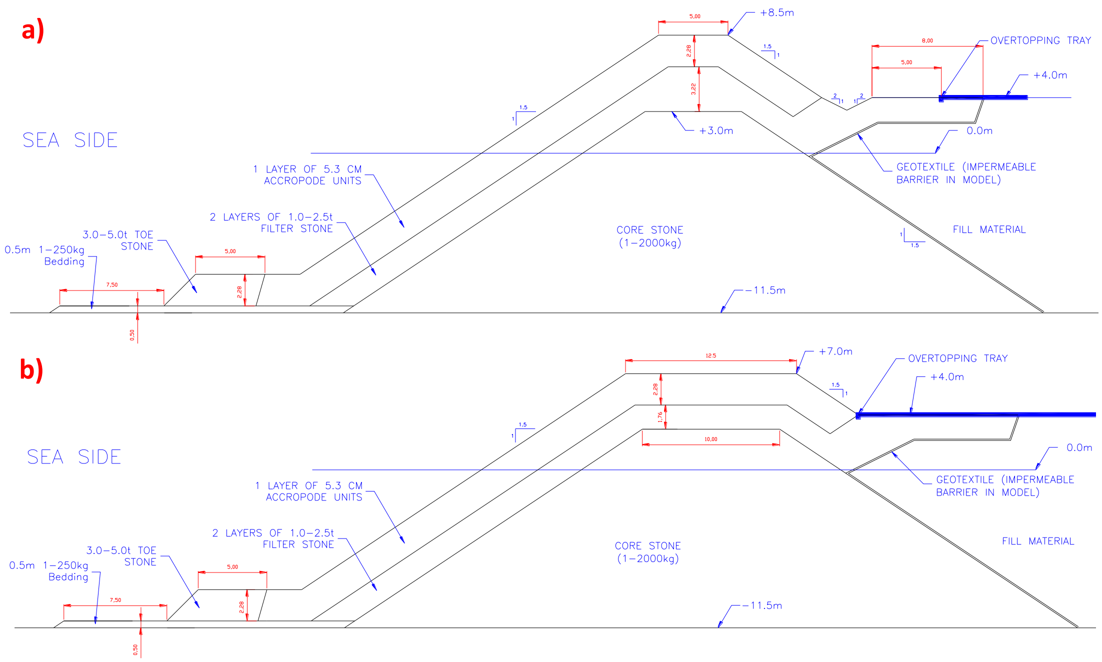

Figure 5. Recommended cross-section alternatives for the north breakwater.

PHYSICAL MODELLING TO ASSESS WAVE AGITATION AND MOORED SHIP MOTION

The second physical modelling study was conducted primarily to assess wave agitation and moored ship motions at the new berths in operational conditions, estimate the reduction in efficiency for (un)loading operations due to adverse weather, and to predict the downtime for a range of alternative terminal layouts and configurations. The study also investigated the potential impacts on operations at the existing port due to the new terminal.

A large-scale 3D model study of wave agitation and moored ship motions was conducted in NRC’s 47m by 30m by up to 0.8m deep Large Area Basin (LAB), and was constructed at an undistorted scale of 1:82. At this scale, the model represented a 9.5km^2 rectangular area (3,854m by 2,460m) of prototype terrain. The model included scaled reproductions of the existing port, the existing shoreline, and the entire proposed container terminal. The model included faithfully scaled reproductions of the proposed revetments, breakwaters, and dredging affecting wave propagation and reflection at the site. The bathymetry was modelled as a fixed bed made of smooth concrete.

The physical model was designed so that processes due to waves approaching the port from directions ranging from 20° to 55° (approximately north to northeast) could be studied. Sea level fluctuations at the site due to tides are minor. Extreme design waves at the test site are estimated to exceed H_s = 6m approaching mainly from the north-easterly direction. The wave conditions selected for agitation testing included significant wave heights from 0.5 to 3.5m with peak periods between 6.5 and 12.5s.

Waves were generated in the model using combinations of two or three computer-controlled wave generators. A second-order wave generation technique was applied to reduce the generation of spurious long waves. The portable wave machines were relocated during the study to generate waves from different directions. Portable wave guides were used to direct the waves from the wave machines up towards the model port. The wave guides prevented the loss of wave energy through diffraction effects and helped to compensate for the finite length of the wave machines. Pajaros Island (see Figure 1) was not included in the physical model domain; however the effect of this island on the incident waves was simulated in the model through reduced incident waves in the lee of the island. Most of the perimeter of the basin was lined with wave-absorbing beaches to minimize unwanted wave reflections.

Scaled replicas of the existing port structures were first constructed. Then a series of incident wave calibrations were conducted to tune and measure the incident wave conditions generated in the physical model (without reflections from the proposed container terminal). Afterwards, the proposed berths and port structures for the first phase of the container terminal expansion were faithfully reproduced (see Figure 6a). The model structures were designed and built to have geometries and wave reflectance properties that were very similar to the prototype designs. Precision surveying techniques were applied to guide and check the horizontal location and elevation of all port structures. The breakwaters were constructed using simplified cross-sections that matched the overall geometry of the preliminary designs. Large armour stone was used to represent the specified Accropode armour units specified for the north, east, and stub breakwaters. The size of the armour material used in the model was purposely oversized to avoid deformation during testing. A second physical model (described above) was conducted to validate the stability of the rubble-mound structures.

Wave conditions were measured in the model using twenty-four capacitance-wire wave gauges. Four of the gauges were arranged in a compact array in order to collect the information necessary to resolve the directional properties of the wave field at one location.

Most tests were conducted with a model *Panamax* class container vessel (59,800 DWT at 1:82 scale) moored at one of the new container terminal berths. Some tests were also conducted with a 230m model tanker vessel at the existing petroleum products terminal. Ballasting weights were distributed within the model ships in order to replicate several desired vessel characteristics at model scale. The total amount of weight added was governed by the required draft, while the spatial distribution of the weight was optimized to match target values for the metacentric height (GM), and the vessel radii of gyration about the x- and y-axes (k_{xx} and k_{yy}).

Fourteen mooring line simulators (eight for the container vessel, and six for the tanker vessel) were used to model the mooring lines in this study. These specialized devices made it possible to reproduce the non-linear elastic properties of the mooring lines while recording the tension using a shear-beam type load cell. In addition, the mooring line pretension could be set using a simple counterweight.

Fourteen fender simulators (eight for the container vessel, and six for the tanker vessel) were used in the model to represent the action of the fenders and measure the compressive loads exerted on the fenders by the moored ships. The specified cell fenders were represented using two-stage simulators that were able to simulate buckling.

Two optical motion tracking systems were employed to measure the motions of the moored vessels in this study. Each system consists of a pair of special cameras and an array of reflective targets mounted on each ship (see Figure 6b). Specialized software was then used to calculate the exact position and orientation of the marker array, and thus the vessel, in three-dimensional space.

A reduction in efficiency for (un)loading operations is experienced whenever the significant vessel motions in any direction, or the maximum tensions in any mooring line, or the maximum loads in any fender exceeds the appropriate safe working limit. The reduction in efficiency (or berth availability) is estimated by summing the occurrence frequency for all seastates in which any of the safe working limits are exceeded. Less certain are the appropriate safe working limits, and the methods that should be used to define the extreme motions and maximum loads. In this study, Baird estimated the reduction in efficiency for (un)loading operations based on results from the analysis of vessel motions and mooring forces provided by NRC.

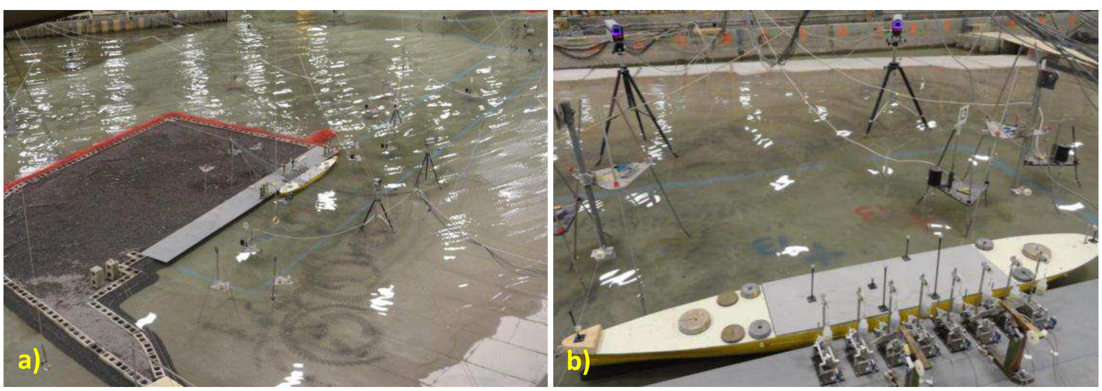

Figure 6. 3D model of wave agitation and moored vessel motion.

Testing commenced with the base case design and continued in pursuit of an optimized design. Throughout the testing program, the effects of several key variables were studied, including: stub breakwater length, berth location, wind acting on the moored ship, mooring line stiffness, mooring line pretensions, and incident wave direction, period, and height.

Throughout the course of testing, the length of the stub breakwater on the east side of the new container terminal was optimized by defining the sensitivity of vessel motions and mooring forces to changes in stub breakwater length. In total, five different lengths were tested for the stub breakwater, varying from 0m (no stub breakwater) to 250m in length (see Figure 7a). There was a significant reduction in wave agitation near the port going from no stub breakwater to a 100m breakwater. With further increased breakwater lengths, the effect was somewhat lessened. Wave heights observed closer to the vessel berths were only mildly affected by breakwater length. Subsequently, greater breakwater lengths provide a notable reduction in maximum mooring line and fender loads. Increasing the stub breakwater length had a strong influence on decreasing critical ship motions, particularly for lengths up to 150m. Further increases in stub breakwater length beyond 150m brought diminishing returns.

Wave heights observed near the west berth were slightly smaller than those measured near the east berth, especially as the length of the stub breakwater was increased. As expected, the results indicated that the east berth was the more exposed location where ship motions and mooring loads are greatest.

For the case of north-easterly waves, the model testing showed that the proposed container terminal did cause some increase in wave agitation at the entrance to and inside the existing port area. This increased agitation resulted in slightly higher mooring forces and larger moored ship motions, which could impact operations negatively. An eastward extension of the proposed north breakwater was investigated as a possible measure to reduce the influence of the proposed container terminal on the existing port facilities (an extension of the north breakwater would later form part of the Phase 2 expansion). In total, four different north breakwater lengths were tested, varying from 0 m (no extension) to 1,000 m in length (see Figure 7b). The study determined that the proposed breakwater extensions caused only small reductions in wave agitation, mooring forces, and moored ship motions within the existing port. However, the study confirmed that these same breakwater extensions provided a notable reduction in the levels of wave agitation, mooring forces, and moored ship motions at the proposed container terminal berths. The study also found that extending the existing port entrance breakwater by 225m generally provided minor reductions in the level of wave agitation, mooring forces, and moored ship motion at the existing petroleum products berth, as well as within the existing port.

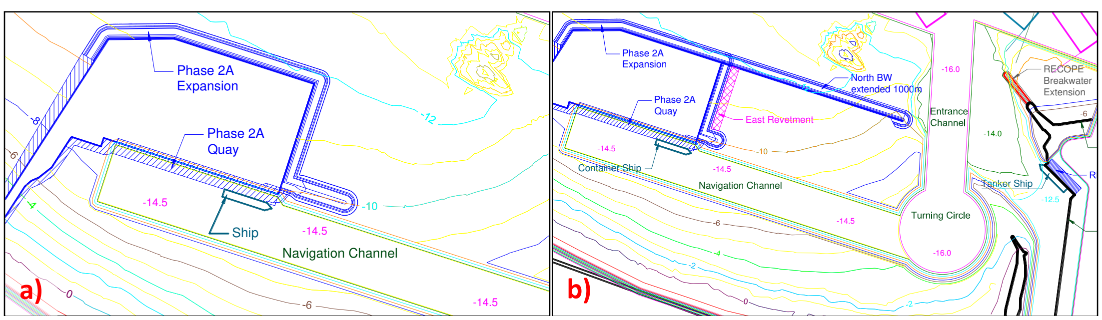

Figure 7. Assessment of stub breakwater length and north breakwater extension.

CONCLUSIONS

These physical modelling studies have generated a large quantity of information concerning breakwater stability and wave overtopping to support the design of the various rubble-mound structures. Furthermore, data describing the wave disturbance and wave-induced motions for moored container ships will support the design of the new container terminal planned for the Port of Moin, Costa Rica. Many important influences have been investigated and quantified, notably: wave height, wave period, wave direction, cross-section design, breakwater length, vessel location, mooring line type, and wind.

The large-scale 2D and 3D stability studies determined that $5.3m^3$ Accropode units are sufficient for protecting the north and east breakwaters, while $7.3m^3$ units are recommended for the stub breakwater roundhead. As expected, the lee quadrant of the stub breakwater roundhead appears to be a critical location due to the combination of downrush from large overtopping events, wave breaking, and the diffraction of wave energy around the head of the breakwater. The proposed bedding stone and toe armour appears to be sufficient for all locations along the breakwater.

This study found that the original design for the lee side of the north breakwater crest did not provide sufficient protection against the erosion of fill material by overtopping flows. Therefore, an improved design which extends the scour pad filter stone further behind the crest is recommended.

As expected, this study determined that a wider crest or taller crest, as compared to the base design, can greatly reduce both the frequency and intensity of wave overtopping, and mitigate the potentially harmful effects wave overtopping flows can cause. Other methods, such as including a roadway and seawalls behind the crest, could also reduce the level of overtopping reaching the work area.

The large-scale 3D agitation and motion study determined that a stub breakwater of ~100 – 150m in length provides a notable reduction in levels of wave agitation and mooring forces, and a significant reduction in moored ship motions. Longer stub breakwater lengths provide only marginal improvements.

The (softer) polypropylene mooring lines resulted in notably lower mooring line tensions, but slightly increased fender forces and notably increased moored ship motions, as compared to the (harder) polyester mooring lines.

The results of this study have been used to estimate the reduction in efficiency for (un)loading operations for the new terminal (due to adverse wave conditions), and to predict the variation of downtime with changes in stub breakwater length.

REFERENCES

CLI (2012). "Accropode – the benchmark in single-layer armouring technology." *Accropode Brochure*, < http://concretelayer.com/documentation>.

Cornett, A. (2014). "Physical modelling of moored ships for optimized design of ports and marine terminals." Proc., 5th Int. Conf. on Physical Modelling in Port and Coastal Engineering (CoastLab 2014), Varna, Bulgaria.