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Optimization of Breakwater Reconstruction at Mont Louis, Québec

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Abstract: The Port of Mont Louis is located on the south shore of the Gulf of St. Lawrence, roughly 120 km west of Gaspé, Québec. A 450 m long breakwater was constructed on the east side of the bay in 1955 to form a sheltered harbour basin and to serve as a commercial wharf. Since this time, the structure has been exposed to numerous severe storms that have caused considerable damage, and today this important infrastructure asset is in urgent need of repair. In December 2006 the National Research Council of Canada (NRC) was commissioned by Public Works and Government Services Canada (PWGSC) to conduct numerical and physical model studies to guide the design of repair works for the wharf and surrounding breakwater. The proposed repair works were rather complex and non-conventional, as they involved constructing a new rubble-mound breakwater on top of the badly damaged structure. They also involved integrating a dynamically re-shaping (berm-style) armour layer design in some areas together with more conventional statically stable designs in others. Desktop and numerical modeling was performed to help define the nearshore wave conditions used as inputs for the physical modeling. A three-dimensional physical model was subsequently constructed and used to investigate the performance of the proposed repairs under a range of site-specific design conditions, including extreme water levels and harsh wave conditions. This paper summarizes the numerical and physical modelling studies and their important role in optimizing and verifying the breakwater reconstruction proposed for Mont Louis. The physical modelling allowed the proposed designs to be optimized to improve their hydraulic performance (stability), enhance their constructability, and reduce costs where possible.

1 Introduction

The Port of Mont Louis is located on the south shore of the Gulf of St. Lawrence on the Gaspé Peninsula (see Figure 1). The port features a harbour basin protected on its eastern side by a 450 m long breakwater that includes a commercial wharf. The wharf and breakwater were constructed in 1955, consisting of a ~300 m long rubble-mound causeway leading out to a series of eight connected steel sheet-pile cells. A ninth sheet-pile cell, separated from the others, served as a mooring dolphin. In the 1970's, armour stone and Dolos concrete armour units were added to help repair the breakwater and protect the wharf from further damage due to wave attack. The structure has suffered considerable damage over the years. At the time of this study, all the steel sheet-piles had been weakened by corrosion and the three outer cells were partially collapsed. The mooring dolphin, located immediately west of the tip of the wharf, was damaged to the point that none of the structure remained emergent above chart datum. The Dolos units and armour stone placed along the sea-side of the wharf were also significantly damaged in numerous locations. A photograph of the site (in 1995) and a more recent photograph showing the damage and relatively shorter pier emergent from the water are shown in Figure 1.

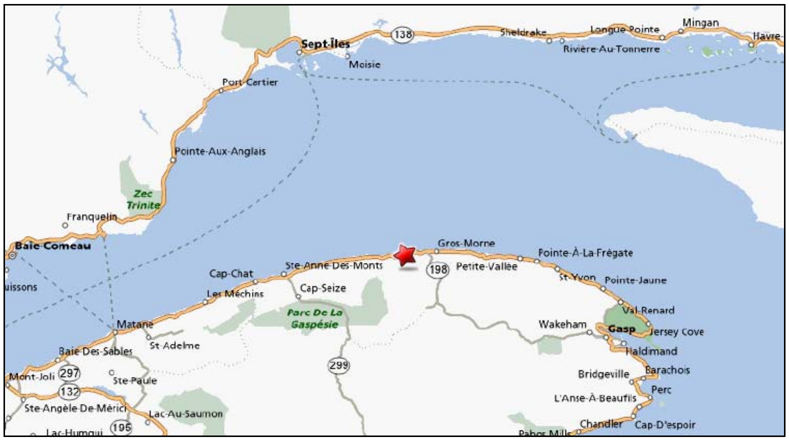

Figure 1. Mont Louis is located on the north shore of the Gaspé Peninsula.


Figure 2. Photographs of the undamaged (left) and damaged (right) east pier at Mont Louis.

This paper describes numerical and physical hydraulic model studies conducted by the National Research Council of Canada to support the optimization and verification of designs proposed by Public Works and Government Services Canada (PWGSC) for repair and refurbishment of the east breakwater and wharf at the Port of Mont Louis, Québec.

Estimates of the nearshore wave climate, including extreme events, were developed through a combination of desktop analysis and numerical simulations of coastal wave propagation. A three-dimensional physical model of the damaged breakwater/wharf and the surrounding bathymetry was constructed at a geometric scale of 1:33.8 in the NRC's Multidirectional Wave Basin. The physical model was used to model and assess five different design alternatives. The model structures were exposed to scaled reproductions of storm wave conditions (extreme wave conditions and water levels) approaching from three different directions, west, north-northwest, and northeast. The testing focussed on assessing the hydraulic stability of the proposed designs in extreme conditions, and optimizing the designs as much as possible to increase stability, improve constructability and reduce costs.

These investigations generated a large quantity of valuable information on the hydraulic performance of various alternative design concepts for the rehabilitation of the pier. This information was used to identify recommended alternatives for implementation, to help PWGSC refine and de-risk the chosen designs, and to assist in optimizing construction efforts.

2 Modelling and Analysis of Wave Conditions

Investigations of the water levels common to Mont Louis show a 2.9m range in tides, and a potential for storm surge of +/- 1m. Through discussions with representatives from PWGSC and NRC, water levels ranging from 0.3m CD to +3.7m CD were selected for use in the model studies. This considers the full tide range and also the combined occurrence of a mean high tide with a severe 1m positive surge.

Preliminary estimates of the extreme offshore wave conditions near Mont Louis were obtained using a simple fetch limited Jonswap hindcast as outlined in Kamphuis (2000). The wind data from two wind stations were used (Mont Jolie and Harve-aux-Maisons) and an overland correction of 1.2 was applied to the hindcast wind speeds. Preliminary estimates of wave height and period were determined from each data set for the North, Northeast, Northwest, and West directions. Table 1 shows the estimates derived from the Harve-aux-Maisons data set.

		Wind (m/s)		Wave Data	
Direction	Fetch (km)	Max	Land Corr. (1.2)	H_{m0} (m)	T_p (s)
North	90	18.9	22.7	3.5	7.9
NE	130	20.3	24.3	4.5	9.2
NW	100	20.3	24.3	3.9	8.4
West	180	20.3	24.3	5.3	10.2

Table 1. Jonswap fetch-limited waves obtained using wind data from Harve-aux-Maisons

PWGSC commissioned a report by Ouellet (2002) in which the extreme wave conditions at Mont Louis were estimated. This report states that significant wave heights in deep water can exceed 6m, and waves with Hs=4.75m can occur near the tip of the structure. Peak wave periods can range up to 11s, and offshore storm waves from the west through northeast directions are common. Ouellet also found that using the wind data from de Harve-aux-Maisons in a wave hindcast provided a better match with wave data recorded by a wave buoy stationed offshore of Mont Louis. This wave buoy (station C45138) has been recorded continuously for 17 years; however, the buoy is removed from the water due to ice concerns from December to March. Investigations by NRC of storm wave data recorded by the buoy revealed that there were 47 individual storms during a recent 17 year period that produced a significant wave height of 3.5m or higher. A peak over threshold method of extreme value analysis was used to estimate extreme wave conditions for various return periods. In this case, the Weibull distribution provided a good fit to the wave height data and was considered suitable for long-term extrapolation. Once the Weibull distribution was fitted to the measured wave height data, the distribution was used to estimate significant wave heights for various return periods (see Table 2). There is an asterisk in this table on the 100-year return period data as it is generally unadvisable to extrapolate data over durations longer than 2-3 times the duration of the measured data set (17 years in the present case). Because of this, and the fact that the buoy did not record data when there was open water for approximately 6 weeks of the year (before ice formed in Mont Louis Bay), a factor of safety of 1.05 was applied to estimate offshore significant wave heights recommended for use in design (grey shaded cells).

Return Period (years)	10	25	50	100*
H_{m0} (m) Weibull	5.9	6.4	6.8	7.2
H_{m0} (m) Factor of Safety	6.2	6.8	7.2	7.6

Table 2. Offshore wave heights for various return periods.

This offshore wave climate was used as the input to a numerical wave transformation model that was developed and applied to obtain the combinations of wave height, period and direction to be simulated in the physical model. In order to estimate the variation of the extreme offshore waves with direction, the maximum storm wave was assumed to come from the west direction, as this was the direction of maximum fetch and also the direction of three of the top four storm waves as recorded by the buoy. The fetch-limited maximum wave heights presented in Table 1 were used to develop ratios describing how the extreme offshore wave heights (see Table 2) might vary with direction. For example the wave height ratio for the northwest direction was obtained as 3.9m/5.3m = 0.74, and the 100-yr return period significant wave height for the northwest direction was obtained as 0.74*7.6m = 5.6m. A similar approach was used to establish maximum significant wave heights for each direction and return period.

The numerical wave transformation model SWAN (Simulating Waves Nearshore) was applied to simulate the propagation and transformation of storm waves from the offshore buoy to the pier, estimate the wave conditions near the project site, and establish target wave conditions for use in the physical modelling. SWAN is an advanced 3rd generation spectral wave transformation model that is particularly effective at modeling wave fields where the processes are highly non-linear. SWAN includes most of the nearshore processes that affect waves as they propagate from deep to shallow water (refraction, diffraction, shoaling, wave generation by winds, wave breaking and bottom friction, non-linear interactions, transmission through and reflection from obstacles). A brief summary of this work is presented in what follows.

Table 3 summarizes the 16 different storm conditions that were modelled using SWAN. The 25-year and 50-year offshore conditions were modelled at both high and low water levels and in combination with two different wave periods: 9s and 11s, as these periods were representative of the storms measured by the offshore wave buoy. Since the waves are locally generated waves, the wind direction was always assumed to coincide with the wave direction, and the wind data from the Harve-aux-Maisons tower was used. SWAN was used to obtain estimates of the nearshore wave conditions for each of the sixteen different offshore wave conditions shown in Table 3.

Test Number	Direction	Water Level (m CD)	Wave Height (m)	Wave Period (s)	Wind (m/s)
1	W	0.3	6.8	9	24.3
2	W	0.3	7.2	11	24.3
3	W	3.7	6.8	9	24.3
4	W	3.7	7.2	11	24.3
5	NW	0.3	5	9	24.3
6	NW	0.3	5.3	11	24.3
7	NW	3.7	5	9	24.3
8	NW	3.7	5.3	11	24.3
9	N	0.3	4.5	9	22.7
10	N	0.3	4.7	11	22.7
11	N	3.7	4.5	9	22.7
12	N	3.7	4.7	11	22.7
13	NE	0.3	5.7	9	24.3
14	NE	0.3	6.1	11	24.3
15	NE	3.7	5.7	9	24.3
16	NE	3.7	6.1	11	24.3

Table 3. Offshore wave conditions for numerical wave transformation modelling.

Numerical results for case 16 are presented in Figure 2. The top figure shows results for the 90 km x 40 km regional domain while the lower figure shows results for the 12 km x 8 km local domain surrounding the project site (the boxed area from the top figure). The lower figure displays a series of points very close to the pier at Mont Louis, and the wave height, period, and direction data from a collection of these points were used as inputs for the physical modelling.

Discussions with PWGSC were held to review the various estimates of the nearshore wave conditions and develop a strategy for simulating the wave climate in the physical model. It was decided that the extreme wave conditions at Mont Louis would be simulated in the physical model as idealized storms approaching from three directions: north-northwest, west, and northeast. The wave conditions and water levels making up the three storms are summarized in Table 4. A series of undisturbed wave tests (wave calibrations) were conducted to both prepare and test the wave machine control signals required to generate these seastates in the physical model, and also to investigate the transformation of the waves as they propagated through the project site. The undisturbed wave tests were conducted with the model bathymetry in place but without the model pier or breakwater. Hence, the measured wave conditions were not contaminated by wave reflections cast from these structures.

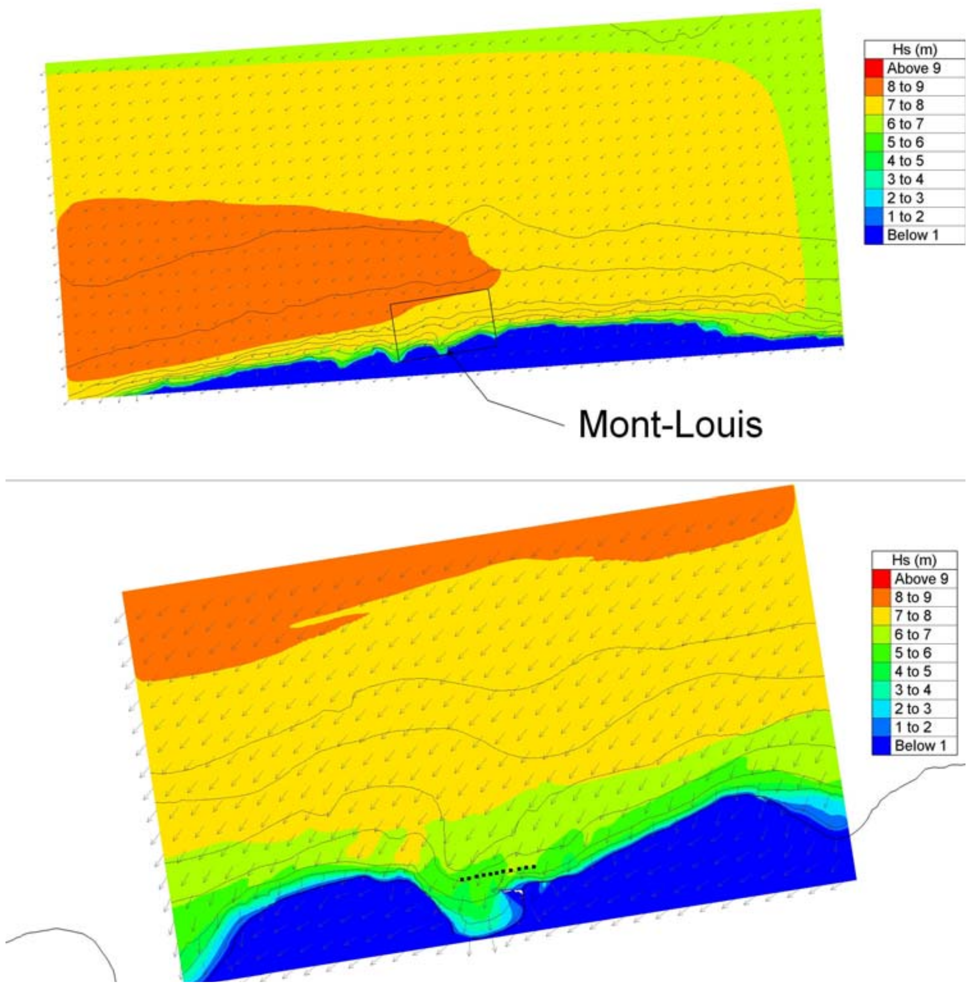

Figure 3. Nearshore significant wave heights and directions predicted by SWAN for case 16: a) regional domain; b) local domain.

Test Description	WL	Target Wave Condtions		
		Dir	H_{m0}	Tp
	(m, CD)	(deg, Az)	(m)	(s)
NNW_1yr_MWL	2	-15	2.5	8
NNW_10yr_MWL	2	-15	4.3	9
NNW_25yr_MWL	2	-15	5.2	9
NNW_50yr_short_MWL	2	-15	6	9
NNW_50yr_long_MWL	2	-15	6	13
NNW_100yr_LWL	0.3	-15	6.1	11
NNW_100yr_short_HWL	3.7	-20	6.4	9
NNW_100yr_long_HWL	3.7	-20	6.4	13
NNW_100yr_MWL	2	-15	6.4	11
NNW_Overload	3.7	-20	7	13
W_1yr_MWL	2	-45	2	8
W_10yr_MWL	2	-45	3.5	9
W_25yr_MWL	2	-45	4.1	9
W_50yr_short_MWL	2	-45	4.5	9
W_50yr_long_MWL	2	-45	4.5	13
W_100yr_LWL	0.3	-45	4.7	11
W_100yr_short_HWL	3.7	-45	4.8	9
W_100yr_long_HWL	3.7	-45	4.8	13
W_100yr_MWL	2	-45	4.8	11
W_Overload	3.7	-45	5.3	13
NE_1yr_MWL	2	15	2.5	8
NE_10yr_MWL	2	15	4.5	9
NE_25yr_MWL	2	15	5.1	9
NE_50yr_short_MWL	2	15	5.7	9
NE_50yr_long_MWL	2	15	5.7	13
NE_100yr_LWL	0.3	15	5.8	11
NE_100yr_short_HWL	3.7	20	6.2	9
NE_100yr_long_HWL	3.7	20	6.2	13
NE_100yr_MWL	2	15	6	11
NE_Overload	3.7	15	6.8	13

Table 4. Wave conditions and water levels for physical model testing.

3 Proposed Repair Works

Two quite different preliminary designs for repair works were developed by PWGSC, and both design alternatives were modelled, tested and optimized in the physical model. The first design, Option A, features an L-shaped planform and includes a series of new revetments protecting the northern face of the pier and a new breakwater arm extending at right angles (southwards) from the old pier. In this scenario, the western 150m of the existing wharf and the detached turning dolphin would be demolished to the -4m elevation. The general layout for this scheme is shown in the upper part of Figure 3. The second design alternative, Option B, involves adding a series of rock berms to reinforce and protect the entire length of the existing pier (except for the turning dolphin caisson). The general layout for Option B is shown in the lower part of Figure 3. The geometry, composition, and cross-sectional details of these two schemes were optimized through the scale model testing in order to improve breakwater stability, constructability, and reduce construction costs (if possible).

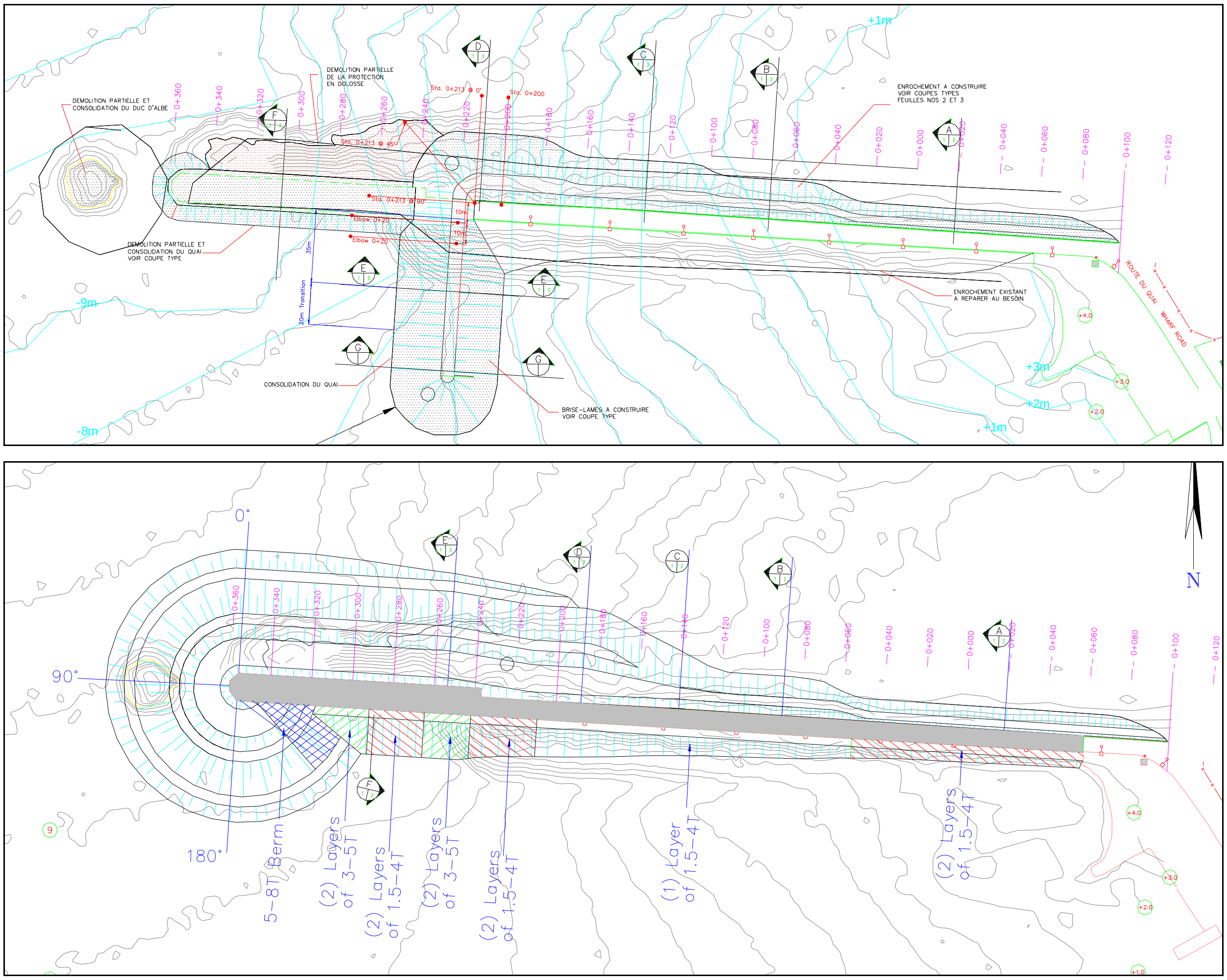

Figure 4. Preliminary design of repair works for the east breakwater at Mont Louis. Top: option A, bottom: option B.

4 The Physical Model

A 1:33.8 scale three-dimensional physical model of the east breakwater/wharf and the adjacent bathymetry was constructed in the 36m by 26m Multidirectional Wave Basin at NRC's facilities in Ottawa. The model was designed such that the orientation of the wave generator was along the 75º-255º plane (waves generated perpendicularly from the wave machine would approach from 345º). Results from the numerical modeling effort showed the nearshore wave directions to be used at the wave machine ranged from 15º for the northeast waves to 315º for the western waves. A rigid impermeable model bathymetry was constructed to imitate the seabed and shoreline at the site between the -15m and +2m contours. The bathymetry sloped down at a 1:10 grade from the -15m CD contour to the toe of the wave machine. Accurate scaled reproductions of the existing breakwater/wharf structure and the proposed repair works were constructed in the model using rock materials that were carefully prepared and selected to replicate the behaviour of the prototype materials. Highly effective wave absorbers that are able to absorb almost all of the incident wave energy were placed on the down-wave side of the model to minimize unwanted wave reflections as much as possible.

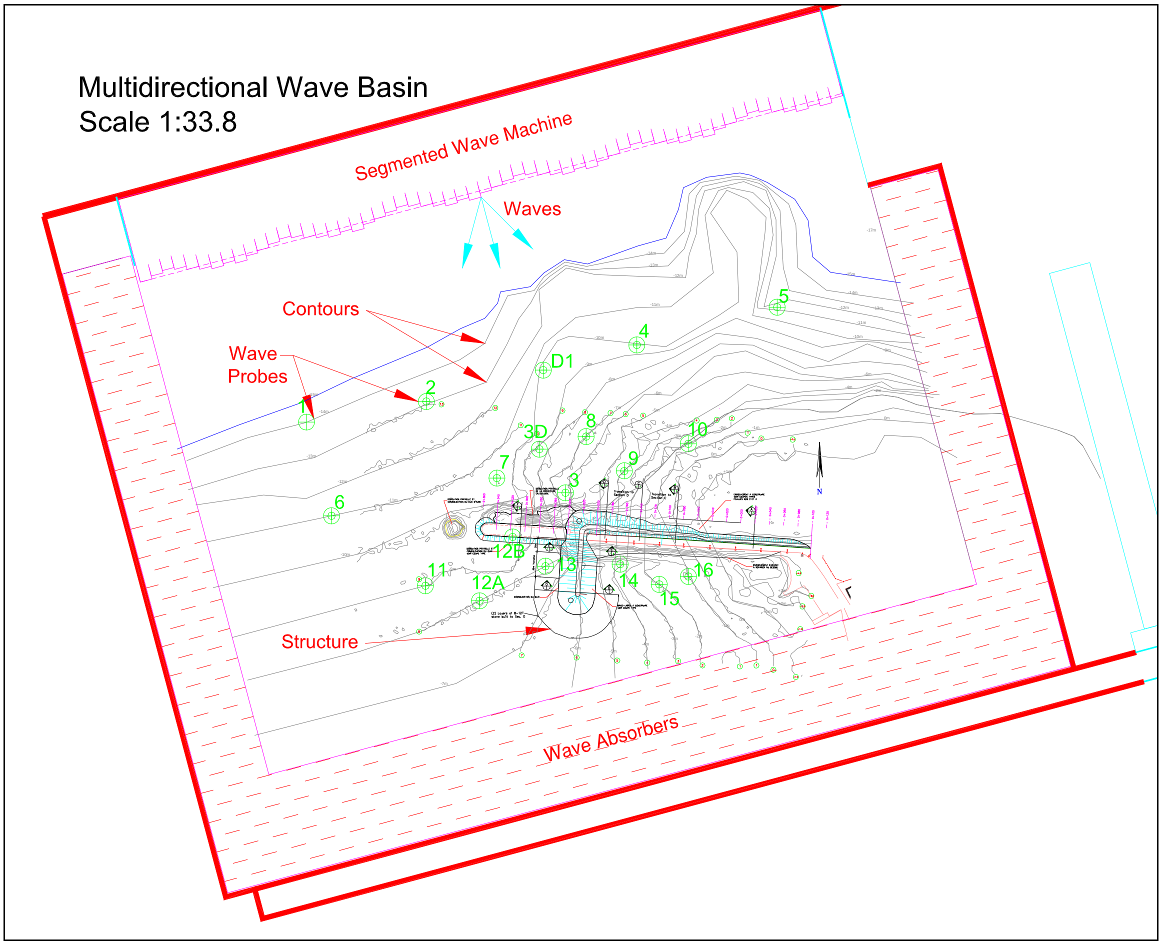

Figure 5. Layout of the physical model in NRC's wave basin.

Scaled reproductions of the existing damaged pier and the various proposed repair works were constructed in the physical model to match preliminary designs provided by PWGSC. Carefully designed templates and precise surveying techniques were employed to control the layout, geometry and composition of the breakwater structures constructed in the physical model. The aim was to replicate, as closely as possible, the layout, geometry, composition and hydraulic performance of the prototype structures. The stability of the breakwater was assessed visually and recorded with numerous digital cameras. Also, an electro-mechanical profiler was used to obtain elevation surveys at many stations where the repair works included re-shaping rock berms and rock-mass armouring. Reshaping of the rock-berms was quantified by differencing pre- and post-test surveys.

Five series of tests, named Test Series A-E, were undertaken to study the stability and performance of the repair works in storm waves arriving at the site from three wave directions: the west, north-northwest, and northeast, and to optimize the initial designs. Test series A-C focused on the L-shaped design alternative shown in Figure 3a, while the second scheme shown in Figure 3b was modelled in test series D and E. While the physical model focused on optimizing the proposed breakwater designs, with each test series improving and re-testing some modifications, the individual results and performance are not presented as part of this paper. Instead, a brief discussion on several challenging aspects of the project is discussed below.

4.1 Test Series A

The main design features and modifications to the existing pier and breakwater investigated in test series A are as follows:

- The west end of the existing breakwater and turning dolphin were partially demolished down to -4m CD.
- The remainder of the existing breakwater pier caissons is fronted by a series of conventional armour stone breakwater sections and a berm breakwater on the western end where the water depths and wave heights are greatest.
- The alignment of the breakwater turns 90º and the leeward trunk berm cross-section continues for another 35m before transitioning down to a conventional breakwater on the roundhead.
- The leeward trunk section then transitions down to a conventional armour stone breakwater on the roundhead.

A brief summary of the observations and performance of the various breakwater sections are discussed below:

- The conventional breakwater sections fronting the pier sustained minimal damage suggesting either the size of the armour stone could be reduced or the sections could be extended into deeper waters.
- The berm section was relatively stable and the reshaping depth relatively shallow indicating the berm elevation could likely be lowered in an effort to reduce the amount of berm material
- Several of the transitions from berm to conventional breakwater or from one section to another section experienced high wave focusing and damage (see Figure 5). The transitions were likely too abrupt and should be more gradual for better performance.
- The end of the roundhead experienced high damage and the filter layers were exposed indicating either larger stones or shallower slopes are necessary in the current configuration.

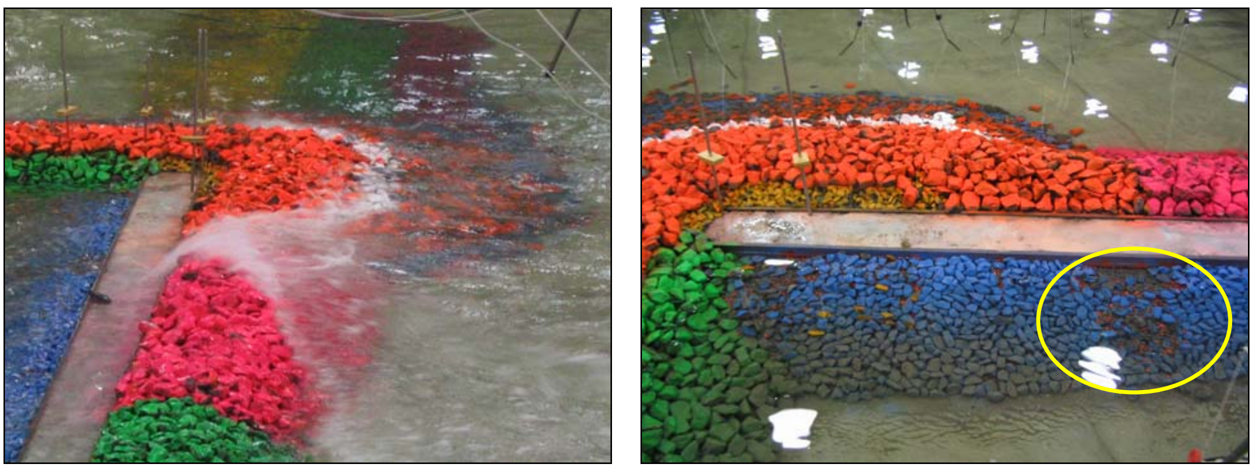

Figure 6. Test series A – left: wave focusing on the eastern berm transition, right: resulting damage to the leeward slope.

4.2 Test Series B

An optimized version of the structure investigated in test series A was studied in test series B. The main design features are as follows:

- The conventional breakwater sections at the root of the breakwater were extended into deeper waters.
- The main berm elevation was dropped from +6m CD to +4.5m CD, and the lower toe berm was removed.
- The rock covering the demolished pier sections and turning dolphin was reduced in size to investigate the stability of smaller stones here.
- The roundhead was reconstructed with larger stone.
- The transitions between sections were lengthened giving a smoother plan shape to the breakwater.

- The method of constructing the berm breakwater sections was slightly different as the entire berm was placed en masse to investigate if a looser method of placement affected stability.

Notes on the performance of the breakwater used in test series B are discussed below:

- The conventional breakwater sections sustained minimal or minor damage.
- The berm experienced minor reshaping and the berm bench remained either partially or fully intact along the length of the structure. Profiles showed the berm reshaping was relatively shallow and only 1-2 stones deep. The reshaped profile remained relatively stable through the overload tests.
- Areas of concern: one layer of leeside slope protection is not enough as holes opened in some sections; the crest stones on the north side of the road should be lowered such that the curb provides some resistance to overturning.
- Large waves were realized near the partially demolished turning dolphin and crib structures indicating a potential navigation concern.

4.3 Test Series C

Test series C was a brief set of tests performed to investigate the effect of entirely removing the existing western caissons instead of partially demolishing them down to the -4m CD level. The submerged portions of the road caisson and the circular turning dolphin were removed from the model and the breakwater was repaired to match the designs investigated in test series B. The main findings from these tests include:

- The wave heights near the berm breakwater were increased slightly showing the partially demolished submerged sections were offering some sheltering of wave energy to the structure.
- The damage to the berm and roundhead increased slightly, again indicating that the submerged sections were offering some benefit to stability of the breakwater.

4.4 Test Series D

The layout of the structure investigated in test series D was very different from what had been studied previously. Instead of partially demolishing the wide section of roadway and caissons that comprised the western portion of the pier, these were preserved and fronted by a new berm breakwater. The main design features of the breakwater in test series D are as follows:

- The location and design of the conventional breakwater sections at the eastern root of the breakwater were unchanged from the previous test series.
- Two different berm sections were used on the western portion of the breakwater with two different sizes (classes of stone), 5-8t stone and 8-12t stone.
- Different sizes of lower berm (toe) stone and also leeside slope stone were investigated to discern stability limits on these locations.

Notes on the performance of the breakwater sections used in test series D are presented below:

- The conventional breakwater sections performed similarly to the previous test series.
- The 5-8t berm experienced a moderate amount of reshaping, particularly near the offshore (western) end of this section. This indicates the western limit of this section could be moved inshore to shallower waters to try to reduce the amount of stone motion here.
- The 8-12t berm stone used on the roundhead of the breakwater experienced excessive movement and reshaping of the berm, particularly on the outer quadrant of the roundhead where only a thin layer of berm stone remained atop the underlayers by the end of the testing. This indicates larger stone is likely needed here (see Figure 6).
- Leeward slope damage was still realized at some locations indicating either increasing the size or number of layers of the armour stone is necessary.

Figure 7. Test series D – overhead view of the 8-12t berm roundhead before (left) and after (right) the testing.

4.5 Test Series E

An optimized version of the structure investigated in test series D was studied in test series E. The main design features are as follows and an overview of the model is shown in Figure 7:

- The conventional breakwater sections and eastern-most berm sections of the breakwater remained unchanged from the previous tests.
- The 8-12t berm section extended more to the east (into shallower depths) to try to reduce stone motion here.
- Larger (10-16t) berm stone was used on the outer portions of the roundhead, and 5-8t stone used on the rear quadrant.
- The thickness of armour stone courses was increased from one to two layers where the leeward slopes experienced damage in previous tests.

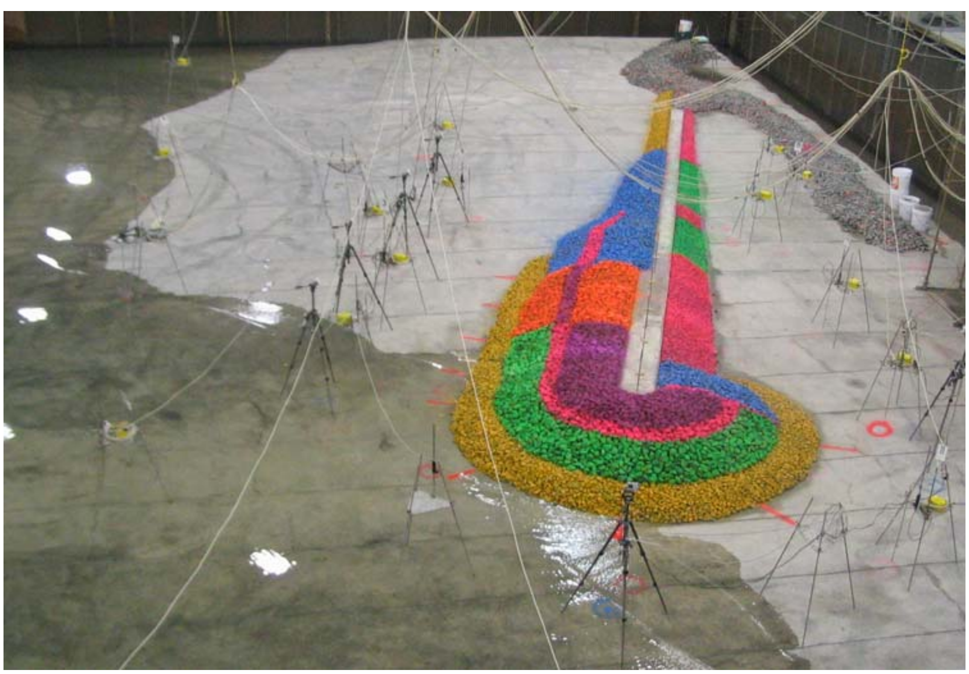

Figure 8 View of the model breakwater prior to the start of test series E.

Stability and performance notes of the structure tested in Test Series E are presented below:

- The conventional breakwater sections sustained minimal or minor damage.
- The berm sections experienced some reshaping, though the berm bench remained either partially or fully intact along the length of the structure. Generally the reshaped profile remained relatively stable through the overload tests. Post-testing deconstruction showed the lowest depression in the reshaped berm still maintained 2-3 stones of cover above the underlayers.
- The two-layered armour stone design on the leeward slopes was relatively stable.

5 Conclusion

Through the careful integration of desktop design, numerical modelling and physical modelling, two potential alternatives for repairing and rehabilitating the main (east) breakwater at the Port of Mont Louis have been developed, tested, optimized and verified. The outputs from analysis and numerical modelling of the winds and waves near Mont Louis were used as inputs to the physical model study. Large scale physical model studies remain the best approach to optimize the layout and design of rubble-mound breakwaters to suit local conditions, and validate the performance of proposed designs prior to construction. Physical modelling is especially valuable for optimizing complex and non-conventional rubble-mound structures such as the repair works proposed for the east breakwater at Mont Louis. These repair works feature dynamically re-shaping berm-type rock armouring integrated with more conventional statically stable armour layers; they also feature new construction built on top of a partially damaged and/or demolished structure. Physical modelling played a key role in developing engineering solutions to cope with these unusual design challenges, and ensuring that these solutions are robust and well adapted to extreme local conditions, and can therefore be expected to perform well for many years. The physical model provided design solutions for two different breakwater layouts, both of which were optimized in order to improve their hydraulic performance and stability. In this case, as in most others, the value of the optimizations, efficiencies and assurances provided by the physical modelling greatly outweighed the costs of the model study, and lead to a lower total life-cycle costs for the project, when capital, maintenance, environmental and societal costs are all included.

6 Acknowledgement

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