Port of Brownsville Floating Breakwater

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Abstract

The present paper describes the technical aspects of a new floating breakwater concept that was installed at the Port of Brownsville Marina located in Port Orchard Bay, Washington State, in 1999. The breakwater incorporates submerged porous treated timber "wave fences" installed on either side of the concrete pontoons to reduce wave transmission. Eight concrete pontoon units are connected to each other by a patented rubber cushion shear tube and bolt assembly on each side of the pontoon, and are held in place with steel anchor piles. A numerical model based on an eigenfunction expansion method was developed to study floating breakwaters equipped with pairs of wave fences of different porosity. A physical model test was conducted to study the performance of the breakwater under the action of oblique waves. A structural model was used to compute the design force envelopes for the concrete floats, using transient wave loads from the model studies as input. The unique solution developed in this project provides a cost-effective option for coastal and marina protections using floating breakwaters.

Introduction

Floating breakwaters have been widely used to provide wave attenuation in many coastal areas, especially in marinas, where fixed breakwaters may not be cost-effective due to deep water, poor soil conditions and lack of availability of suitable materials. Among the diverse types of floating breakwaters, the rectangular concrete pontoon is the most common. A number of physical and numerical studies have been carried out to evaluate the performance of floating breakwaters (References 4 and 11). Some key factors that affect the performance have been identified, including the ratio of width of pontoon to wave length, mass per unit length of pontoon and ratio of draft to water depth. A disadvantage of pontoon floating breakwaters is the poor attenuation in cases of deep water and long period waves.

Recent environmental requirements limit the shading effect on the seabed by the breakwater. The resulting narrow breakwater has a reduced efficiency. Wave fences have been found to improve the performance of floating breakwaters and have been tested by the Canadian Hydraulics Centre (CHC), National Research Council Canada in Ottawa (References 5 and 6) and installed in a number of applications. A floating concrete pontoon with a single porous timber fence was installed by CeFer Floating Structures Ltd. at Salt Spring Island, British Columbia, Canada (Reference 8).

The Port of Brownsville Marina has a capacity of approximately 300 boats, and is located in Port Orchard Bay, on the east shore of the Kitsap Peninsula in the State of Washington, near the naval base at Bremerton (see *Figure 1*). The new breakwater is 290 meters long and replaces an aging timber floating breakwater. The width of the breakwater is limited to 4.26 meters to satisfy the environmental shading requirement. To meet the specified wave climate in the marina, CeFer Floating Structure Ltd. proposed a design using a porous timber fence on both the upwave and downwave sides of the pontoon to enhance wave energy dissipation. To test the viability of the concept, a numerical model was developed by Westmar Consultants Inc., and physical model tests were then conducted in the CHC wave tank.

Westmar is a consulting engineering company specializing in coastal structures, and designed the floating breakwater and the pile anchoring system. Manufactured by CeFer Floating Structural Ltd., the breakwater was successfully installed in July 1999 (see *Figure 2*).

This paper discusses the technical aspects of the Port of Brownsville floating breakwater project including the design criteria, design concept, numerical and physical models, and the finite element structural model that was used to develop the force envelopes in the concrete pontoons.

Design Criteria

The Port of Brownsville developed the design criteria for incident waves from the northeast and the southeast directions from wave hindcasting techniques. The incident waves, the specified wave climate in the marina, and the required performance of the floating breakwater in terms of a wave transmission coefficient, are given in the following table:

Wave Direction	Incident Significant Wave Height	Peak Wave Period	Transmitted Significant Wave Height	Required Transmission Coefficient, K_t
Northeast	0.76 m	3.1 seconds	0.3 m	0.4
Southeast	0.58 m	2.5 seconds	0.3 m	0.5

Significant wave height is defined as the average height of the highest 33% of the waves in a wave train. Peak wave period is defined as the period of the most energetic waves. The transmission coefficient is defined as follows:

$$K_t = \frac{\text{Wave Height Inside in the Lee of Breakwater}}{\text{Incident Wave Height}}$$

The smaller the value of K_t, the more effective the breakwater.

The water depth is about 10 meters at high water and 6 meters at low water.

For the structural design, forces generated by the wave height H_5 were specified by the Port, and is consistent with the wave heights recommended in the US Army Corps of Engineers "Shore Protection Manual" (SPM, Reference 10) for this type of structure. H_5 is defined as the average height of the highest 5% of the waves in a wave train, which is approximately 1.37 times the significant wave height. In addition to the wave loads, a 2,920 N/m design wind load on the breakwater was specified by the Port.

Design Concept

The Port had an existing supply of galvanized steel pipe piles that were suitable for anchoring the breakwater. The final design required 52 of the 610 mm diameter, 13 mm wall thickness piles driven 10 meters into the seabed through pile wells in the concrete floats. The breakwater was constructed of six 38 meter long and two 29 meter long post-tensioned concrete units that were filled with expanded polystyrene foam. These units were connected by rubber cushion shear tube and bolt assemblies on each side of the pontoon. The general layout of the float units is shown in *Figure 3*. The elevation of the floating breakwater is shown in *Figure 4*.

A typical cross-section of the breakwater is shown in *Figure 5*. A timber fence was attached to the concrete pontoon on both the upwave and downwave sides, extending 1 meter below the bottom of the pontoon. The timber fence on the upwave side had 40% openings, while the timber fence on the downwave side had 10% openings. The purpose of these differing openings was to equalize the wave forces on the fences, so that a standard size and spacing of the vertical fence member could be attached to the pontoon using bolts extending through the width of the pontoons, as shown in *Figure 5*.

Physical Model Tests

Physical model tests were conducted at the CHC in Ottawa, to study the performance of this new breakwater concept, particularly under the action of oblique waves. Tests were carried out on a 1:6.72 scale breakwater model in CHC's coastal wave basin for regular and irregular wave conditions. Wave transmission was measured using wave gauges, and wave-induced forces exerted by the breakwater on the piles were measured using a pylon dynamometer that was attached to one of two anchor piles.

A 5.65 meter long single breakwater unit was tested in a water depth of 0.91 meters. A general layout for the model tests is shown in *Figure 6* and a photograph of a model test is shown in *Figure 7*. The model was constructed of a foam filled aluminum and galvanized sheet metal box, the fences were constructed of wood lattice, and the piles were made from steel pipes. The instrumented model pile was connected to a plate at the bottom of the test tank, and to the dynamometer at the top.

Irregular waves were modelled using the spectrum developed in the Joint North Sea Wave Project (JONSWAP, Reference 1), with a peak enhancement factor of 3.3. The waves were generated and calibrated using CHC's GEDAP (Reference 7) software package, which controls the wave paddle motions and analyses the data. For the design waves from the northeast, a measured K_t of 0.3 was achieved, which is less than the specified K_t of 0.4. *Figure 8* shows the measured incident and transmitted wave spectra for the northeast wave design case. For the design waves from the southeast approaching the two shorter units of the breakwater at a normal angle, a K_t of 0.4 was achieved, which is less than the specified K_t of 0.5.

The clearance between the piles and pile wells was set for the full-scale dimensions of 50 mm and 5 mm. The results showed that the forces on the piles were significantly lower for the smaller clearance, due to the reduction in impact loading.

Design Approach

The complexity of this floating system, which utilized flexible connectors and porous fences, and which is subject to waves from oblique angles, required a sophisticated design approach that utilized physical, numerical and finite element models.

The physical model tests provided wave transmission coefficients and pile forces. The data obtained was then used to calibrate the numerical model and verify the following design approach:

- Determination of the wave transmission coefficients and wave forces on the piles for a 0^o, or normal, wave approach angle using the physical and numerical model tests. Two gaps between the piles and the pile wells were run in the physical model tests, while a zero gap was assumed in the linear numerical model.
- It was determined that the numerical model accurately predicted the zero gap wave forces on the piles as extrapolated from the physical model tests.
- It was determined that the wave transmission predicted by the numerical model agreed with the physical model tests for a range of wave heights and periods, and for regular and irregular waves.
- The physical model was then tested for waves at an approach angle of 42^o. Two gaps between the piles and pile wells were run in this case, and the wave transmission coefficients and pile forces were measured.
- The physical model tests demonstrated that the specified performance of wave transmission was achieved, and the information was used to calibrate the numerical model for varying wave approach angles.
- The wave forces generated by the numerical model for an oblique H_5 wave moving along the breakwater were applied to the finite element model. The pile forces predicted by the combined numerical and finite element models were verified by the physical model tests.
- Having calibrated the numerical and finite element models, the design of the floating breakwater structure was commenced. The maximum internal forces in the breakwater structure and on the piles, for the oblique H_5 waves applied anywhere along the length of the breakwater, were generated using the finite element model and are shown in *Figure 9*.
- The effect of clearance between the pile and the pile well was incorporated, based on the results of the physical model tests.

The physical model tests provided the basis for the design of the structure through the process of calibration and verification of the numerical and finite element models described below.

Numerical Model

To numerically predict wave transmission and forces on the fences, a numerical model was developed based on an eigenfunction expansion method. The wave forces on the fences were computed by separating the fluid field into three wave regimes that satisfy the Laplace Equation and matching boundary conditions both on and beneath the fences (Reference 12).

Finite Element Model

To determine the internal forces in the pontoon structure, and the forces on the pile groups restraining the breakwater, a finite element structural model was developed using STAAD III. In the model, each concrete pontoon was represented by the following beam and spring elements:

- A beam element representing the concrete pontoon, located at the centroid of the pontoon.
- Two beam elements connected to the pontoon beam element, to engage the couplers between the pontoons.
- Springs under the pontoon beam elements to simulate buoyancy.

A time history was generated for the H_5 wave design case as shown in *Figure 9*. The envelopes of vertical shear, horizontal shear, torsion, horizontal moment, and vertical moment were generated by the STAAD model. The resulting envelope of vertical bending moments is shown in *Figure 10*.

Summary

A floating breakwater concept incorporating timber fences on both sides of a concrete pontoon and flexible connections between pontoon elements was developed, tested, designed, manufactured and installed. A sophisticated design approach using physical, numerical and finite element models was used to verify the performance and to design this complex system.

Cefer Floating Structure Ltd. manufactured the pontoons in their plant located in Richmond, BC, Canada, and assembled and installed the floating breakwater at the Port of Brownsville Marina in July 1999, on time and on budget.

The successfully designed and installed floating breakwater provides coastal engineers with a cost-effective option for protecting marinas.

References

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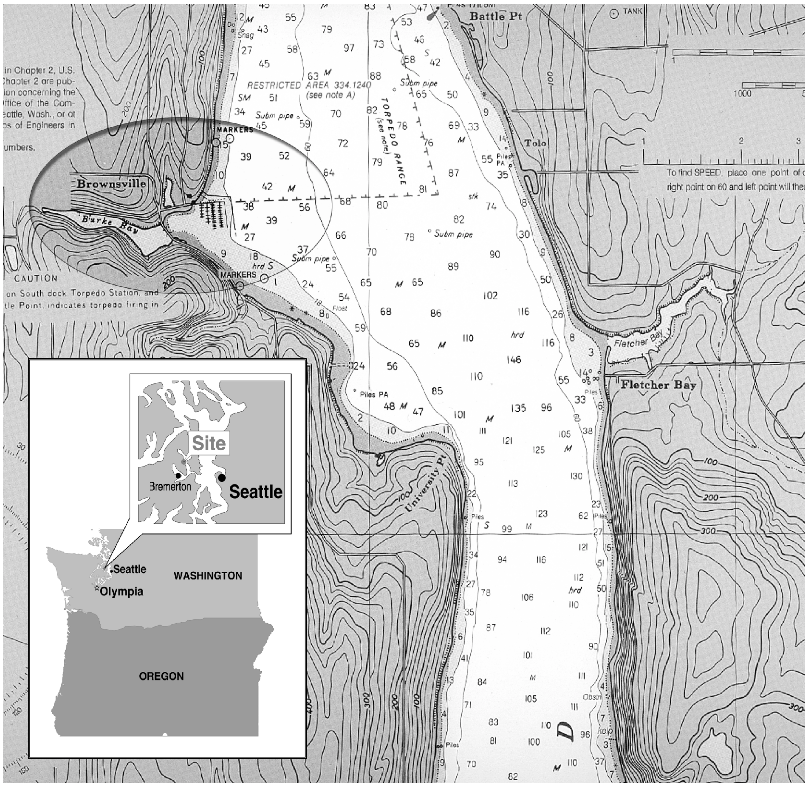

Figure 1. Site Location.

Figure 2. Photograph of the Marina With the Installed Floating Breakwater.

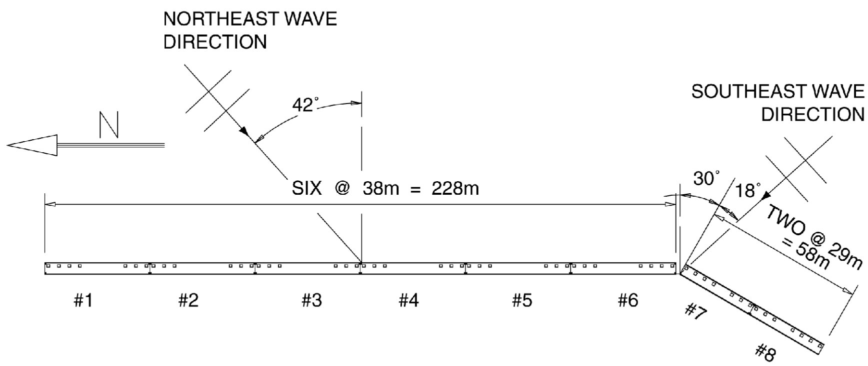

Figure 3. General Layout

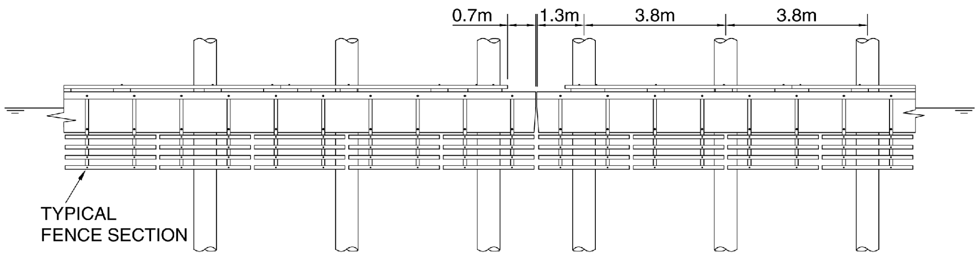

Figure 4. Elevation Showing the Floating Breakwater and Pile Restraints.

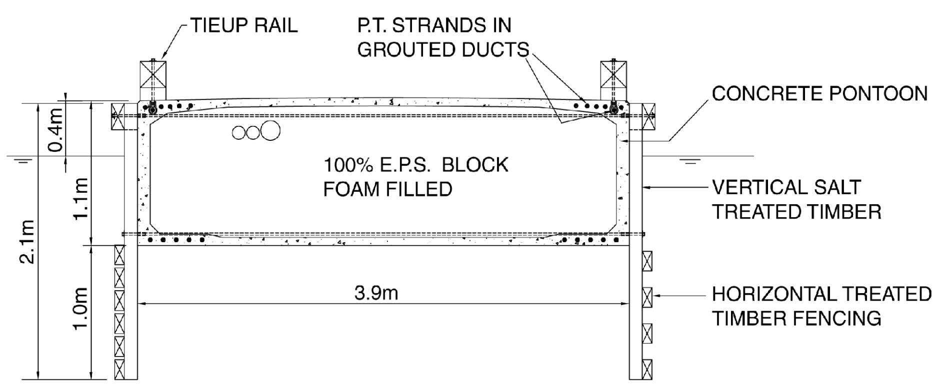

Figure 5. Typical Cross Section

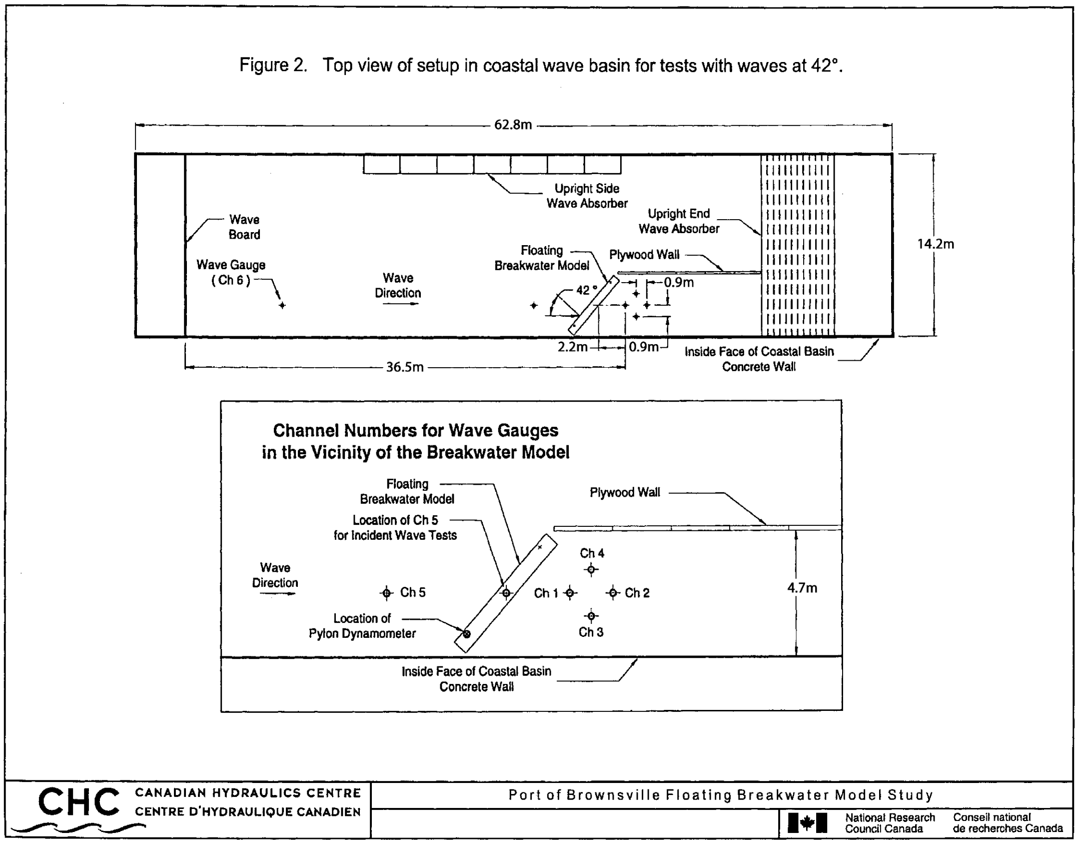

Figure 6. Physical Model Test Setup.

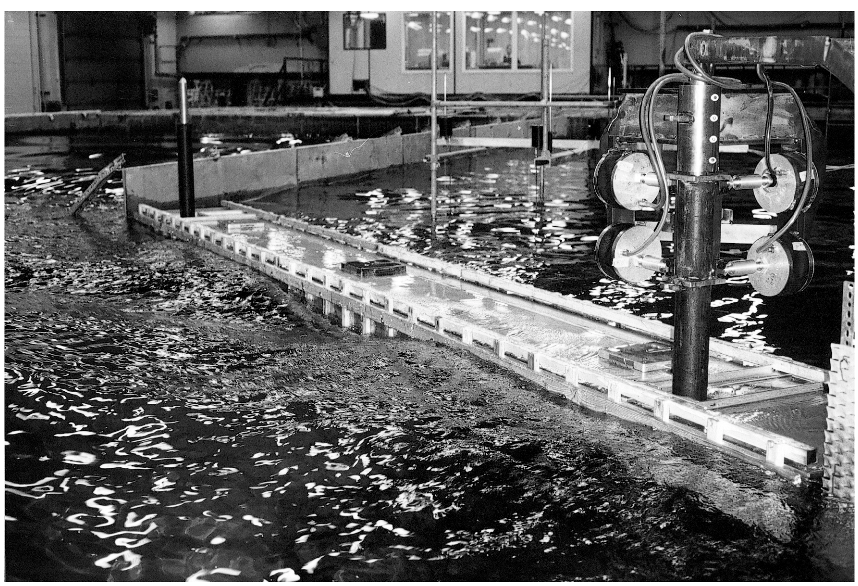

Figure 7. Photograph of the Model Test of the Northeast Design Waves

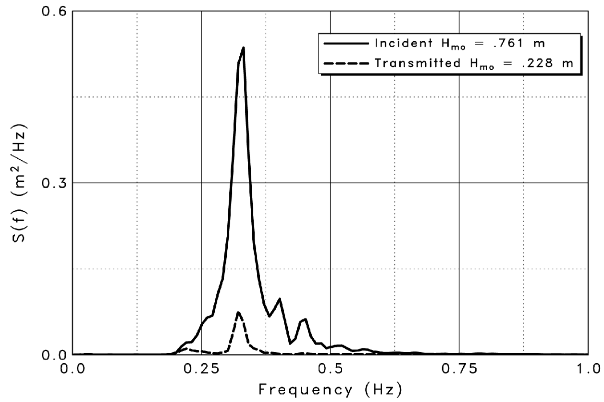

Figure 8. Model Test Results on Transmission of Northeast Design Waves

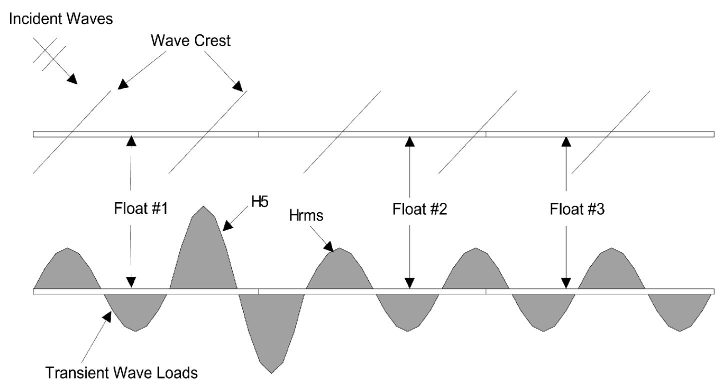

Figure 9. Transient Wave Loads on Northeast Design Waves

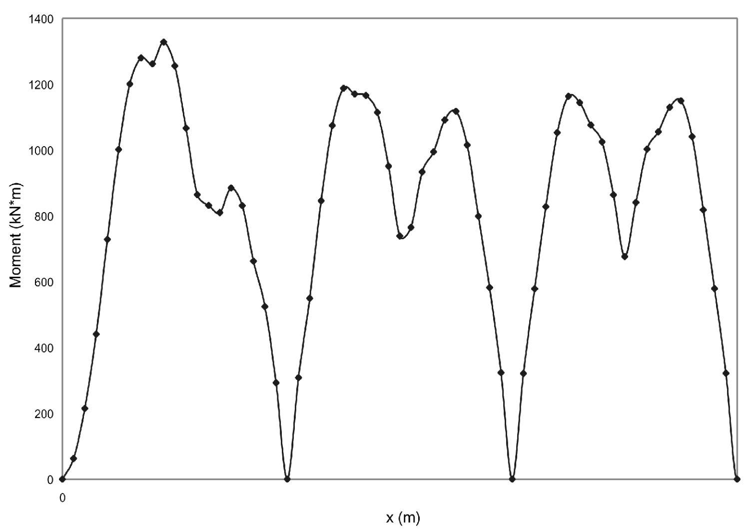

Figure 10. Envelope of Vertical Moment for Northeast Design Waves.