Design of a Floating Breakwater for the Port of Bremerton

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Abstract

A 1,400 ft floating concrete breakwater has been developed to expand and upgrade the Port of Bremerton's marina in downtown Bremerton on Puget Sound in Washington State. The location of the marina presents a unique set of challenges for harbor protection. Extensive data collection and numerical and physical modeling were conducted to determine the optimum geometric shape of the structure. Unique concepts were developed to analyze the hydrodynamic behavior of the structure, and several analyses were conducted to arrive at a final mooring and structural design. The result is a multi-functional state-of-the-art structure that serves as a prominent example of durable, cost-effective harbor protection.

Introduction

The Port of Bremerton owns and operates two recreational marinas in Sinclair Inlet of Puget Sound, which is located approximately 17 miles west of Seattle. The smaller of these two marinas, a fifty-slip transient moorage facility, is located along the shoreline of the City of Bremerton. Bremerton is a city that had suffered decades of economic decline starting in the 1970's. A major political and community commitment was planned to implement a vision of economic revitalization for the harbor district which is the downtown core of the city. A key element of that plan was the development of a world-class yachting marina to replace the existing transient moorage marina in the harbor.

The existing marina, constructed in the early 1990's, had originally been conceived as a small harbor marina. The site for the facility was located immediately adjacent to a ferry terminal and even included joint-use facilities shared between the marina and the ferry agencies.

The design of the original boat basin had provided for harbor wave protection using a shallow draft floating breakwater. The existing breakwater, which was physically oriented to attenuate waves generated by the prevailing winds, was a medium-duty post-tensioned concrete pontoon system anchored to the sea floor. This floating breakwater system performed satisfactorily for defending against wind-generated storm waves. Unfortunately, very little consideration had been given to the effects of ferry boat wakes and the resulting wake wave conditions within the marina. The breakwater, which had been developed to attenuate wind waves, was not effective for attenuating the longer-period wake waves generated by the ferry operations. Ultimately, the original marina had very limited success in attracting boaters. The lackluster performance of the marina operations was likely the combination of a number of issues. The root cause was attributed to ferry wakes and the lack of attractions within the city's decayed downtown harbor district. The ferry wakes, in some cases, caused damage to boats moored in the marina. The wake problem, coupled with a city that lacked amenities that the yachting community covets resulted in a marina that was not particularly popular.

The Port of Bremerton launched an initiative in 2003 with the programmatic objectives to: (1) develop a world class marina facility; and (2) contribute to the economic vitality in the downtown harbor district and thereby create demand that would attract interesting businesses into the area. The success of the first objective required solving the ferry wake problems. The success of the second objective was tied directly to the success of the first objective and partnering with other local agencies and private developers to redevelop the harbor uplands. The vision of the Port's plan is illustrated in the artist's rendering shown as Figure 1.

Figure 1. Artist Rendering of Bremerton Marina and Harbor District

An initial literature search conducted during early stages in planning revealed that although there is a plethora of information on attenuation of wind waves, there is very limited data or information on studies that have been published regarding the attenuation of wake waves. The authors of this paper were retained by the Port of Bremerton to solve the wake wave attenuation problem, and in the process, have gained a wealth of knowledge while developing an engineering solution to the wake challenge.

This paper presents the science and experimentation to quantify the vessel wake problem and its affect on the existing and proposed marinas, the engineering and analysis that were performed to understand the effectiveness of the proposed floating breakwater system, the engineering analysis and computations that were performed to understand the physical demands on the structure and predict the structural loads and forces on the system that occur during storms that may be encountered throughout its service life, and the design engineering necessary to deliver a project that performs as predicted and maintains its structural integrity throughout its 50-year design-life cycle.

At the time of authorship of this paper, the outcomes of the Port of Bremerton's programmatic objectives are not fully realized, but are well underway. A "supply contract" to manufacture the breakwater, a 1,400-foot long floating structure that will double as an over-water public park, is well underway. A marina installation contract has been advertised for bid solicitation to construct the marina illustrated in Figure 1. The new marina is being developed based on market studies to provide most, if not all, of the amenities that are desired by yachters seeking moorage for their 40 to 100-foot vessels. Although not a focus of this paper, it may be interesting to the reader to note that nearly $1/2 billion dollars of capital investments have been made in the Bremerton harbor district since the time the marina expansion initiative was planned.

Data Collection and Physical Modeling

Coast and Harbor Engineering, Inc. under contract with Art Anderson Associates, conducted field data collection, numerical and physical modeling to determine the optimum geometrical shape of the breakwater. A field data collection program was designed and implemented to calibrate and verify the analytical and modeling tools to be used for the one-dimensional analysis and design of cross sectional configuration of the breakwater. Two wave measurement stations, (outer and inner) equipped with pressure gauges were deployed to record synchronous wave parameters outside and inside of the marina breakwater. The wave gauges recorded wave data during 42-day period. The wave gauges were configured to enable recording nearly continuously at a sampling rate of 4 Hz (4 samples per second). Pressure data were converted to instantaneous water level information that could be analyzed with standard software. In addition to the wave data, the filed data collection program was able to collect vessel traffic data (vessel type, speed, and distance to the gauges) for most of the passing vessels during field measurements.

Using measured incident and transmitted wave data, the methodology was developed to compute cumulative hours per year that wave height in the marina exceeds the design criteria. The designed breakwater performance criteria are established in coordination with Art Anderson Associates and the Port of Bremerton as follows:

- Hs = 2 ft should not be more frequent than the 50-year return period.
- Hs = 1 ft should not be exceeded for more than 2 hr/year.
- Hs = 0.5 ft should not be exceeded for more than 50 hrs/year

Using this methodology and economical analysis it was determined that to provide sufficient conditions in the marina, the coefficient of transmission should be less than or equal to 0.40. To meet this criterion, the floating breakwater beam (width) and draft (depth) were preliminarily determined to be 22.5 ft and 8.0 ft, respectively (Ref. 5).

To verify the preliminary breakwater configuration, two-dimensional (2-D) physical modeling (section) was conducted. This was conducted in the 2-D wave flume at the O.H. Hinsdale Wave Research Lab (WRL) at Oregon State University in Corvallis. The flume, with dimensions of 342 feet long, 12 feet wide and 15 feet deep, is equipped with a fully-programmable multimode wave generator with active wave absorption, and can generate waves up to 5.25 feet high with periods of 3.5 seconds. A scale factor sf = 4.9 was selected as the best compromise for meeting scale similarity in the breakwater weight, while staying within the facility limitations. Wave parameters for the physical modeling were derived from the wave measurements in the field. Physical modeling was conducted for two types of waves; regular (monochromatic) and irregular (wake bursts). Regular waves in the model were presented by twelve different monochromatic wave parameter combinations (heights and periods)

The physical model results confirm the effectiveness of the proposed box breakwater to provide reduction of wave energy in the marina. It showed that the dimensions of the proposed breakwater would provide coefficient of transmission approximately equal to 0.4.

Breakwater Hydrodynamic Analysis

Over the course of the study of the breakwater a number of different concepts were considered. Initially it was expected that the four legs of the breakwater could be designed as separate bodies connected together with fenders or other flexible connections in order to break up the total length into shorter segments, thereby reducing the bending moments. These concepts were found to be problematic, due in large part to the approximately 20-foot tidal range, deep water and limited mooring line scope.

The breakwater structure consisted of four segments rigidly connected together in an arc-like fashion to form a large, single body as shown in the sketch in Figure 2. The structure was restrained by 16 mooring lines attached to the first leg, 18 on the second leg, and 16 on the third leg making a total of 50 catenary mooring lines. The use of catenary moorings was dictated by the large water depth (75 feet) and 20 foot tidal variations.

The concrete breakwater studied had a typical cross-section of 24.5 feet beam, 10.0 feet vertical dimension and floated at a draft of 8.0 ft. The loads applied to the breakwater included wind loads, current loads and wave loads. The analysis was carried out by use of a numerical model based on the MORA computer program suite for studying the hydrodynamic/dynamics of multiple, three-dimensional floating bodies. In view of the nonlinear behavior of the structure due primarily to the nonlinear catenary mooring lines, the time-domain computer program which allowed nonlinear moorings was considered necessary.

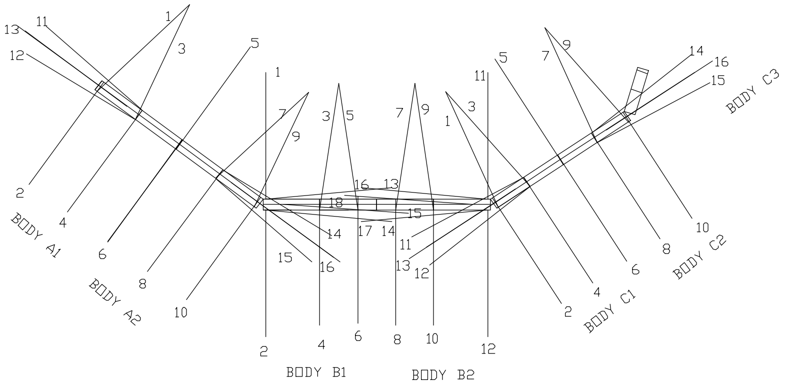

Figure 2. Breakwater and Mooring Lines Layout

In the hydrodynamic analysis the breakwater was treated as seven, separate, rigid, independent bodies each of which interacted hydrodynamically with each other. More specifically, an imaginary cut was made at the center of each of the three longest legs of the structure and also at each of the three "dog-leg" angles as described in Figure 3. This redefined the total structure as seven separate, rigid segments. Each rigid-body segment was connected to the adjacent one by a stiff elastic connection composed of seven springs arranged in such a fashion as to make the connections as rigid as required. In this way, the connection could have any degree of rigidity and, in addition, the six-degree-of-freedom loads at each of the six connections could be measured by virtue of the spring force/deflections. The typical elastic connection based on an arrangement of seven tension/compression springs is described Figure 4. The stiffness of the connections was designed to approximate the same overall deflection of the breakwater to an applied force as the concrete structure. This approximate elastic equivalence was based on linear beam theory together with the sectional properties of the breakwater.

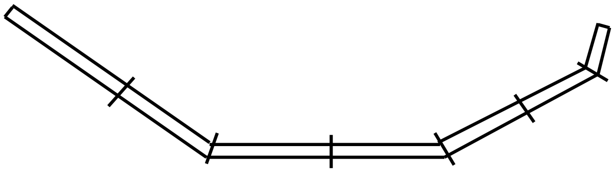

Figure 3. Definition of 7 Rigid Bodies with Elastic Connections Identified by Crossbars

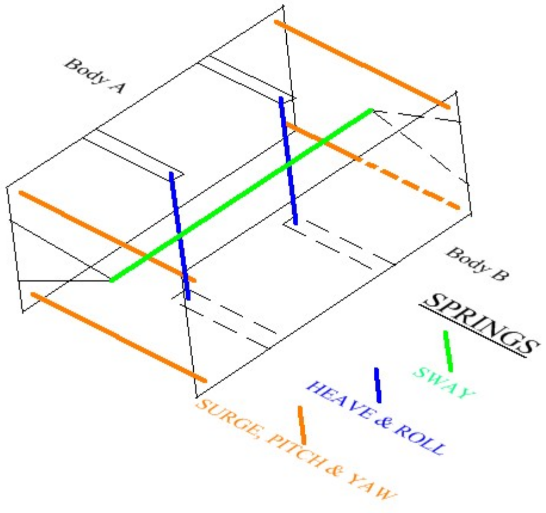

Figure 4. Typical Tension/Compression Spring Connection Between Rigid Bodies

The mooring lines were of the catenary type, composed of a chain segments at both ends and a long wire segment between the chain segments. The overall mooring line length was typically 285 ft. In the numerical modeling the mooring lines were treated as quasi-static elastic lines.

The wave excitation loads, added mass and radiation damping of the breakwater bodies were computed by the multiple-body MORHALN module in MORA and based on linear theory. This program is based on distributed three-dimensional sources on the immersed surface and accounts for the hydrodynamic interaction of all of the seven bodies. The incident waves were represented by JONSWAP spectrum with cosine-squared directional spreading.

The viscous damping of the breakwater was modeled by use of a pair of slender members running the length of the bodies at the location of the bilge corners. The drag coefficient in the horizontal direction was selected to model the mean drag force due to the current of 3 kts (5.06 ft/s). Also, the vertical drag coefficient was sized to model roll damping according to the method based on model testing results of Tanaka (1960). The loads on the members were computed by use of nonlinear Morison's equation at each time-step in the time-domain analysis based on the instantaneous relative velocity vector which included the effect of current, wave-induced velocity and the velocity due to the motion of the body itself.

The mean wind loads on the structure were based on an 80 mi/hr wind speed. These loads were considerably larger than that due to the breakwater alone in order to allow for the likelihood that a number of boats may tie-up to the breakwater during storm conditions.

The results of time-domain analyses tend to be voluminous, especially in comparison to frequency-domain analyses since all parameters are defined by a time-series. These results included the displacement response of each of the seven bodies in six degrees of freedom, the 50 catenary mooring line tensions and the six degree of freedom loads at the six connections between bodies, as well as the tension in each of the seven springs that were used to make the connection between each pair of bodies.

The catenary mooring line loads were defined by time-series for each of the 50 mooring lines. A plot of three of the more heavily loaded mooring lines denoted by 2,4,6 and corresponding to the case noted is shown in Figure 5. The rather large mean tension is due to the wind and current forces.

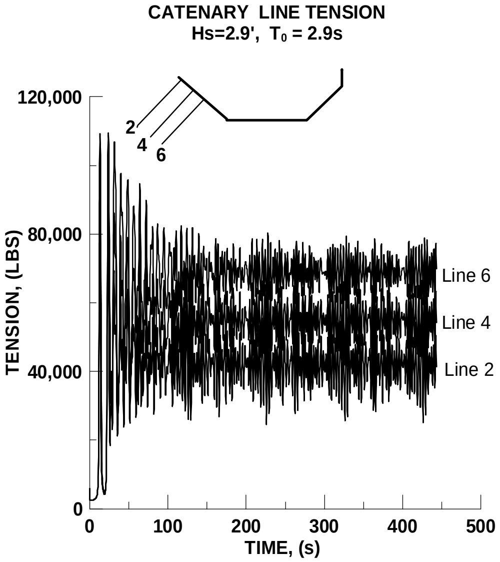

Figure 5. Catenary Mooring Line Tensions

It was not practical to run the computer simulation for extremely long time periods to cover the maximum value occurring in a typical 3 hour storm which would represent the straightforward approach to dealing with a nonlinear simulation. However, the response of the breakwater to waves, as well as the mooring line tensions, was actually quite linear once the mean equilibrium operational offsets were reached. This was due to the fact that the mean loads were fairly large and tended to tension the more heavily loaded catenaries sufficiently to put them in the linear elastic range. The nonlinear mooring analysis was needed primarily because of the nonlinear behavior of the catenaries over large mean offsets. In view of this the maximum value of the loads was obtained from the time-series by stochastic analysis of the responses.

The largest and most critical load was the bending moment in the horizontal plane. For the example case the value of the maximum value for this moment is shown in Figure 6. As expected, the maximum value of this moment occurs in the approximate center of the structure. The figure also shows the moment in the vertical plane for comparison with the horizontal value.

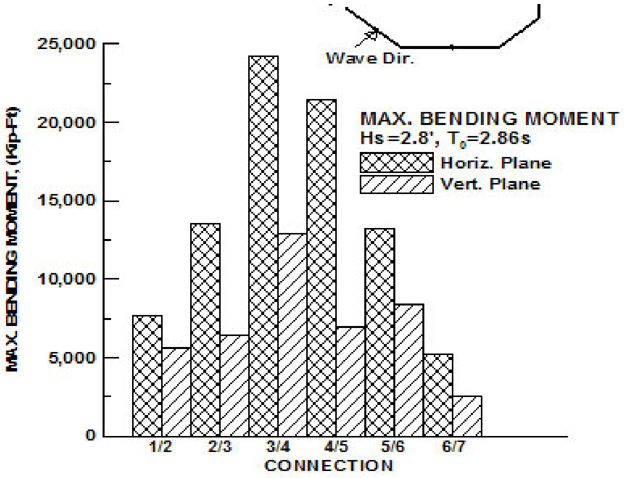

Figure 6. Maximum Value of the Horizontal and Vertical Moment

Mooring Analysis and Structural Design

Several three-dimensional time-transient pseudo-static mooring analyses were carried out to select an optimum mooring arrangement of mooring lines by using the mooring analysis software application OPTIMOOR Plus. As shown in Figure 2, the optimum arrangement was selected for hydrodynamic analysis, stake pile and hawse pipe designs. A brief description of the environmental criteria used is as follows:

- Site wind driven waves (80 mph, fastest mile extreme event) causing 4 feet significant wave height with 3.7 second period and 68 feet of wave length in SW and NE direction
- Maximum current velocity = 3 knots

Throughout several iterations of mooring and hydrodynamic analyses, the level of pretension, length and size of mooring lines were determined to keep the breakwater safely in position against harsh marine environmental loadings.

Based on the suggested dimensions throughout the model tests, mooring and hydrodynamic analyses, the shape of a typical cross section of the breakwater was determined and followed by reinforcement design. Figure 7 describes this typical section. Stability analysis was also carried out and internal walls were arranged. All compartments include EPS (Expanded Poly-Styrene foam), except end compartments at each individual body module. The open cavities are going to be used so that technicians have access to connect the bodies together. It was designed to have sufficient reserve buoyancy to remain stable and afloat when the cavity is damaged and flooded (Ref. 4).

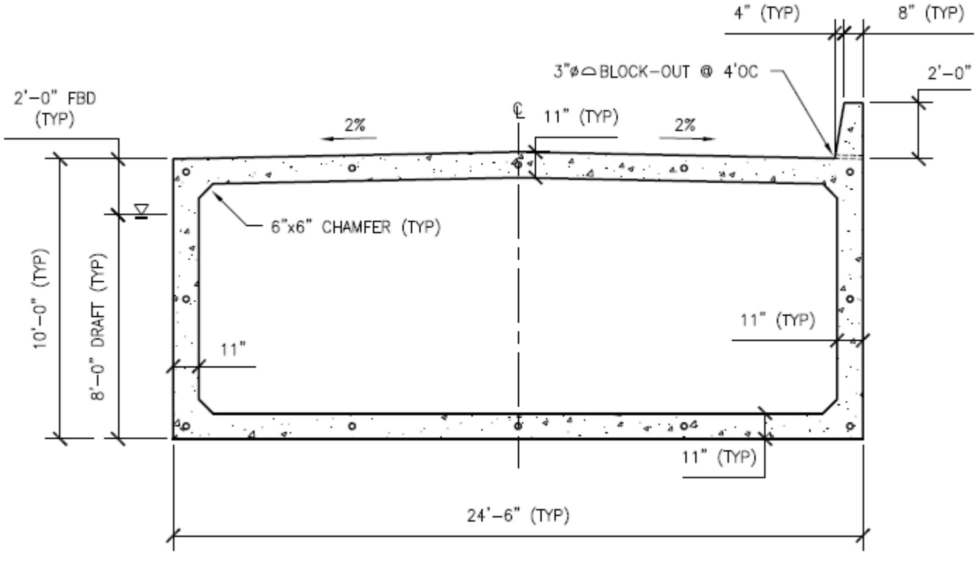

Figure 7. Typical Breakwater Section

Unlike upland structures or small-size floating structures, it was important to set the correct overall project plan to complete design, construction and installation. Floating marine concrete structures can be fabricated in many different ways depending on manufacturer facilities. It was an engineering challenge to provide construction documents satisfying different construction procedures and methodologies of various manufacturers. Considering various facilities and float launching methods, the maximum length of an individual body module is limited to 150 ft., and four different concrete mix designs are specified for either precast or ready-mixed type. It was predicted that several individual bodies might have difficulty in correcting uneven drafts (levels) problems due to center of gravity locations of the bodies away from their own geometric center locations. Utilization of lightweight fine aggregate (imported from Colorado) enables the creation of ballast spaces large enough to correct any uneven draft situations for the stability of the individual breakwater bodies. The lightweight fine aggregate is to be blended with normal weight aggregates. Figure 8 and Table 1 show the breakwater plan of all individual body modules connected and concrete mix designs (Refs. 2 and 3). Quality control of ready-mixed concrete is quite difficult for marine floating concrete structures from past case histories. Therefore, it is necessary to specify the wide range of concrete unit weight and amount of mineral admixtures to meet the needs of strength, workability, durability, and stability (Ref. 1).

Figure 8. Twelve Individual Body Modules

	Mix 1	Mix 2	Mix 3	Mix 4
28 day min compressive strength (psi)	5,000	5,000	5,000	5,000
Min. concrete clear cover	2 in exterior / 1 in interior	2 in exterior / 1 in interior	2 in exterior / 1 in interior	2 in exterior / 1 in interior
Cement	Type I or II	Type I or II	Type I or II	Type I or II
Max water/binder ratio	0.35	0.35	0.35	0.35
Unit weight (pcf)	136 - 144	133 - 141	128 - 136	123 - 131
Entrained air	4% - 7%	4% - 7%	4% - 7%	4% - 7%
Amount of mineral admixtures (Class F fly ash or blast furnace slag)	15% by weight of cement	15% by weight of cement	15% by weight of cement	15% by weight of cement

Table 1. Details of Concrete Mixture Design

The each individual breakwater body is independently post-tensioned along a longitudinal direction by seventy-two (12 ducts x 6 strands/duct) strands. Ten, 1-3/4-inch diameter, high-strength (150 ksi ultimate strength) post-tensioning bars are used for connecting two the individual bodies.

It was a structural and hydrodynamic engineering challenge to provide effective design of the three corners that are prone to high environmental loading. Several iterations of structural, mooring, and hydrodynamic floating body analyses has been made to provide optimum corner design satisfying structural soundness and smooth public flow on the breakwater deck as a function of the over-water park.

Several corner design options were studied to reduce or eliminate high stress applied at the corner section without disturbing the public path. A rigid connection was selected throughout the analyses and feasibility study, including stability and reinforcement arrangement. The rigid connection withstands high marine environmental loading and does not disturb the path of public flow at all.

Geometrical shapes of all the corner bodies were determined and designed differently from straight bodies. The sharp corner is reinforced by high strength – low carbon - chromium steel reinforcement called MMFX2 without post tensioning. High strength (minimum yield strength = 100 ksi) and corrosion resistance of the reinforcement compensates for the lack of post-tensioning at corner locations.

Summary

The overall design of floating concrete breakwaters require the wide knowledge of many disciplines, including project planning, float manufacturing methods, materials, mooring and floating body dynamics to design and construct a long-term high-performance breakwater. Unlike previous large floating structures built in the US, the new breakwater for the Port of Bremerton has innovative construction documents applicable to many different manufacturing and launching methods. The breakwater is also utilized as an over-water park for the public. It will make an impact on other ports, local governments, engineering firms and manufacturers to develop future durable and cost-effective floating breakwaters.

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