Design-Construction of a Breakwater/Pier Structure at U.S. Naval Station Everett

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This $25.8 million combination Breakwater/Pier Structure was constructed in two phases for the United States Navy Homeport facility at Naval Station Everett in the state of Washington. The facility includes a 1500 ft long x 144 ft wide (457 x 43.9 m) pier and a 150 ft long x 144 ft wide (45.7 x 43.9 m) approach trestle. Precast/prestressed concrete was used cost effectively for the piles, deck panels and utilidor lids for the pier and vertical panel elements for the breakwater structure. Prior to construction, model testing was carried out to verify the function and strength of the breakwater panels. The complexity of the project stems from the dual functionality of a breakwater that had to be capable of upgrading to a useable berthing pier in the future. The purpose of this article is to present the conceptual design, structural design considerations for the breakwater, model testing, and construction highlights of the project, concentrating in particular on the precast concrete elements.

In 1985, BERGER/ABAM Engineers, Inc., Federal Way, Washington, began to work with the United States Navy in developing a berthing facility for its ships at Naval Station Everett in the state of Washington. Fig. 1 shows a rendering of the outer harbor as it was initially planned. The naval base is located in the City of Everett, about 25 miles (40 km) north of Seattle.

The original intent of the plan was to have berthing facilities within the inner harbor, with only a Carrier Pier, South Wharf and a rubble-mound breakwater located in the outer harbor area. However, due to Navy downsizing and environmental concerns with the contaminated dredge material discovered in the inner harbor, the focus was shifted to the outer harbor area for all of the berthing facilities.

The South Wharf was constructed with the Carrier Pier during the period of 1989 to 1992. During this time, the need for additional berthing space in the outer harbor was discussed. The constraints of the site and proximity of the Snohomish River navigation channel clearly identified that the footprint of the proposed rubble-mount break-

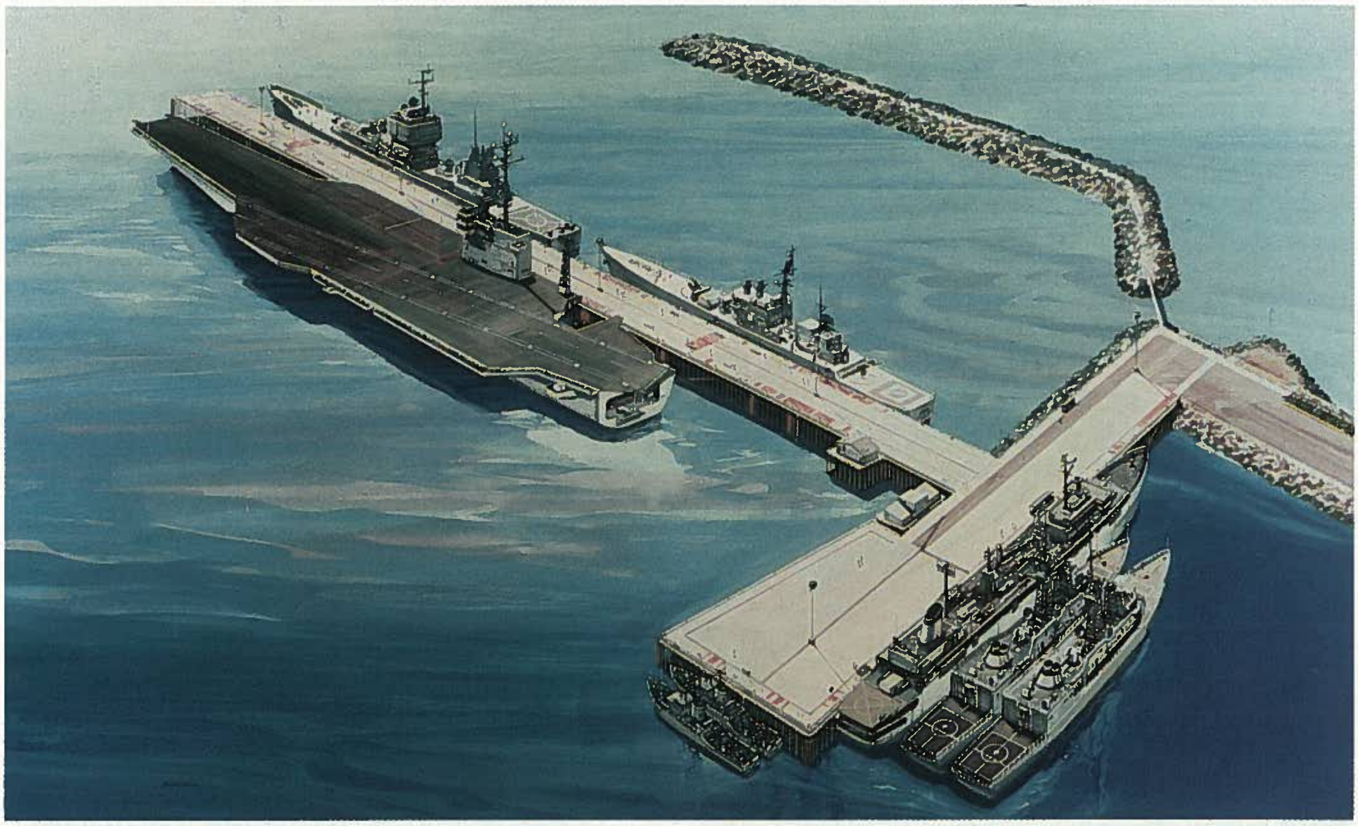


Fig. 1. Rendering of the outer harbor at Naval Station Everett as it was initially planned in 1985. Proposed rubble-mound and Carrier Pier are shown.

water was the only visible space for additional berthing spaces. The challenge now became one of integrating the need for wave protection with a berthing pier.

It was recognized that of these two "needs," wave protection for the South Wharf and Carrier Pier needed to be provided first in order to meet the planned arrival date of the first carrier to be homeported in Everett.

Available construction funds would also impact the process. A rubble-mound breakwater had been programmed and funded. This cost had to be adhered to in the first phase of a two-phase process. Phase I was to build a breakwater with the available programmed funds, which could then be turned into a berthing pier during Phase II when funding was made available.

Concept development on the Structural Breakwater/Breakwater Pier began in 1988. BERGER/ABAM provided support to the Navy throughout the concept study phases, permitting and Environmental Impact Statement (EIS) work, model studies, and designed the Structural Breakwater constructed in Phase I.

Once funding was procured, BERGER/ABAM was awarded the contract to prepare the final design for the breakwater upgrade to a functional pier in Phase II, complete with utilities. Fig. 2 is an aerial view of the nearly completed Structural Breakwater and Fig. 3 is an aerial view of the completed Breakwater Pier.

This article describes the planning of the facility, which considered the environmental impacts and the effects of the breakwater on the adjacent navigation channel. Breakwater alternatives are also discussed.

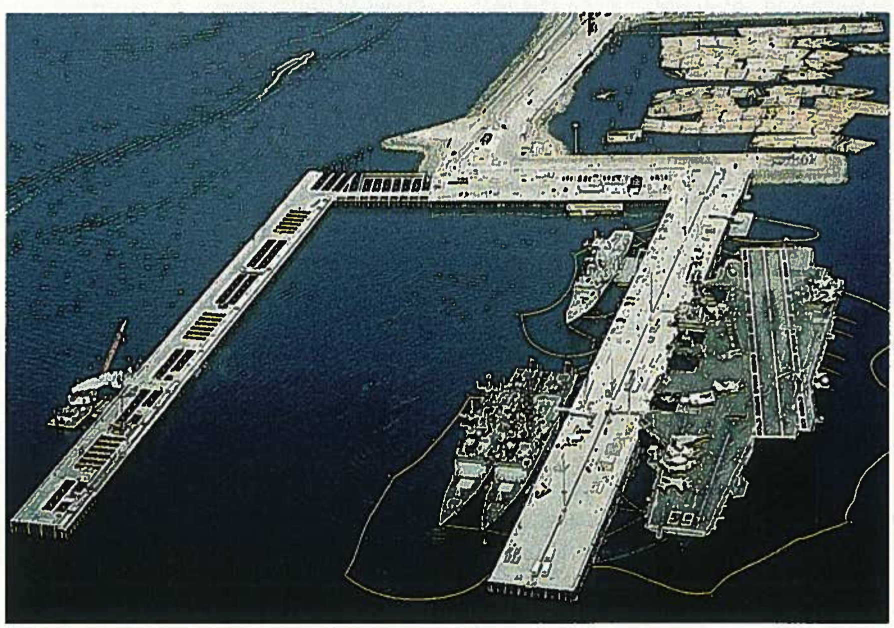


Fig. 2. Aerial view of Breakwater, with South Wharf and Carrier Pier.

Fig. 3. Aerial view of completed Breakwater Pier, with South Wharf and Carrier Pier.

Two key results (in addition to staying within the programmed costs) to be achieved were:

- To keep waves being reflected back in to the navigation channel to a maximum of 1 ft (0.305 m) (any wave and small craft in the channel would be affected adversely).
- Limit the wave height entering the berthing area to a maximum of 3 ft (0.91 m).

The concept that met these criteria and deemed to be most economical by virtue of materials of construction and ease of constructability was a permeable vertical panel arranged along two lines approximately 90 ft (27.4 m) apart, thus creating not only a wave energy attenuation at each line, but also a baffled chamber between the lines where wave energy can be safely dissipated.

To verify the breakwater design, a 1:10 scale model of the structure was fabricated and tested in a wave tank under various wave heights and wave periods. From this model test, wave attenuation performance was determined, as well as expected wave loads on the breakwater elements and support structure. The lateral support for these breakwater elements formed the structural framework for the functional berthing pier.

DESCRIPTION OF FACILITIES

Fig. 4 shows the layout of the facilities at the south end of Naval Station Everett. The Structural Breakwater/Breakwater Pier includes a 1500 ft long x 90 ft wide (457 x 27.4 m) pier with a 150 ft long x 144 ft wide (45.7 x 43.9 m) approach trestle.

The Breakwater Pier is designed to accommodate a variety of Navy vessels, mainly destroyers and frigates, along the east face. This structure also protects the vessels berthed at the Breakwater Pier and at the adjacent facilities by acting as a breakwater.

The South Wharf is designed to accommodate smaller support vessels, such as minesweepers and barges. The approach trestle for the Breakwater Pier would also be designed for berthing of these smaller support vessels. The Carrier Pier is designed to accommodate a variety of Navy vessels, ranging from a Nimitz-class aircraft carrier to frigates, cruisers, and destroyers.

The Breakwater Pier and the Carrier Pier provide mechanical utilities such as potable water, steam, air, sewer, and condensate return. These utilities are housed in a covered utilidor with removable lids. Primary electrical power is supplied to the berthing structures through deck-mounted substation buildings at the Breakwater Pier and below-deck electrical vaults on the Carrier Pier.

Electrical conduits are buried within the deck ballast to the electrical transformers and switchgears located inside the buildings or vaults on the pier. Power to the ships is distributed to electrical mounds at the east face of the Breakwater Pier and both east and west faces of the Carrier Pier.

DESIGN CONSIDERATIONS FOR BREAKWATER (PHASE I)

The design team had worked with the U.S. Navy on projects in Everett since the early planning stages in 1985. Being familiar with the local environmental process and Navy funding and development policies, BERGER/ABAM was able to bring continuity and expertise to the table to assist the Navy in establishing the following performance criteria for the Structural Breakwater project:

- Provide wave protection for vessels berthed at the facility.
- Limit the reflected wave height.
- Provide for future opportunities for upgrading to a berthing pier.
- Furnish a breakwater at a cost that was within the Navy's funding limits.
- Provide a structure that would be environmentally acceptable.

Environmental design criteria for the structures had been developed as part of the early studies in 1985. Based on area geometry and wind data, the incident wave was determined to be 6 ft (1.83 m) in height with a period of approximately six seconds. Using this criterion, a mooring analysis of destroyers moored at the west side of the Carrier Pier determined that beam-on incident waves would produce excessive sway and roll motions.

From this analysis, it was concluded that a breakwater would be required at this site to provide safe and unrestricted moorage at the Naval Station Everett berthing facilities. The mooring study established a maximum long period wave height of less than 3 ft (0.91 m) as being acceptable for safe mooring.

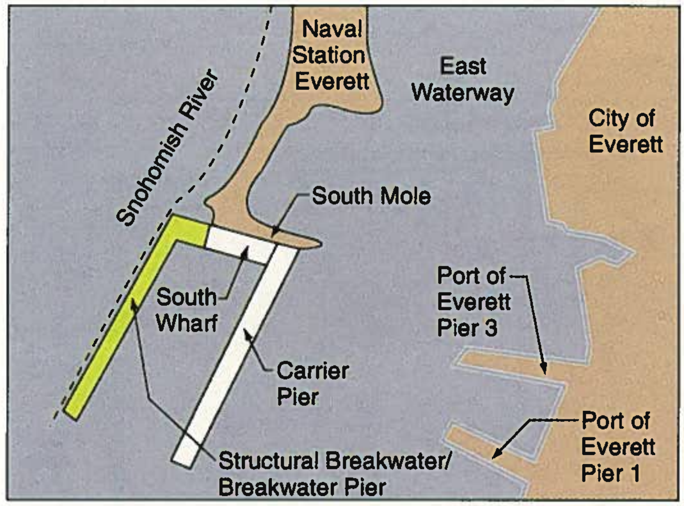


Fig. 4. Site plan showing outer harbor at Naval Station Everett and nearby Port of Everett facilities.

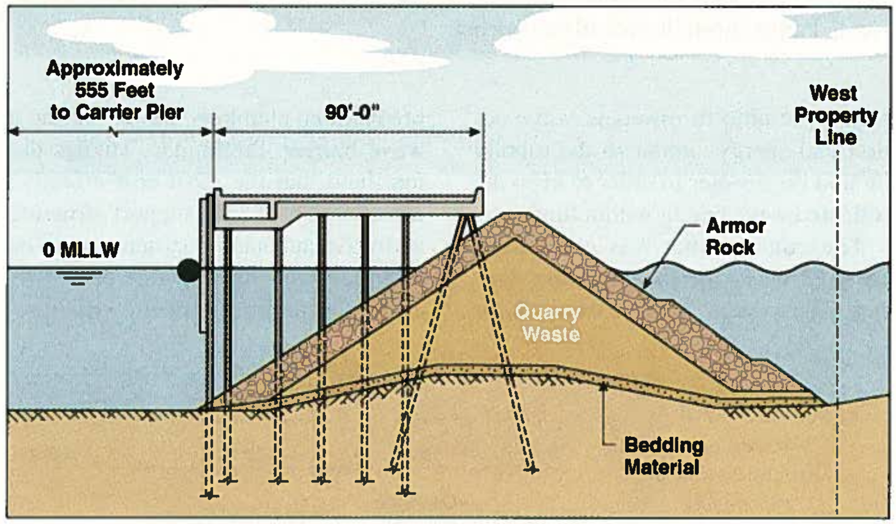


Fig. 5. Rubble-mound breakwater concept.

CONCEPTUAL DESIGN OF BREAKWATER

The following concepts for a breakwater structure were considered and are discussed below:

- Rubble-mound
- Partial sheetpile cofferdam
- Permeable wave barrier

The first concept considered was a rubble-mound breakwater and is illustrated in Fig. 5. This concept was found to be the most economical solution for a breakwater at this site. A rubble-mound breakwater provides 100 percent reduction in wave height while allowing virtually no wave reflection due to the wave runup on the rock armored structure.

The design also allows the breakwater structure to be environmentally friendly as it is constructed of natural materials. The rubble mound would be constructed first to act as a breakwater, followed by a conventional pile supported pier.

In the analysis, it was found that rubble mound breakwaters require a relatively large footprint for stability of the side slopes. This would result in a space between the future Breakwater Pier and the existing Carrier Pier that is too narrow for the required vessel mooring and maneuverability clearances. Therefore, the rubble-mound concept was not a viable option for the breakwater, with future upgrade to a functional pier.

The next concept studied is shown in Fig. 6. This concept was a partial sheetpile cofferdam. The top of the sheetpile is established at an elevation of -8 ft (2.44 m) to allow near-shore fish migration. This type of cofferdam functions very well for wave attenuation but a mathematical analysis showed that it would perform very poorly for wave reflection at low tide. Consequently, this concept was not pursued.

The environmental impact statement identified that any breakwater scheme should limit the reflected wave height to approximately 1 ft (0.305 m) so as to not create objectionable conditions for recreational boat users in the Snohomish River. The performance requirement relative to wave attenuation of the breakwater structure was to reduce the 6 ft (1.83 m) high incoming wave to a 3 ft (0.91 m) transmitted wave. Additionally, the wave barrier

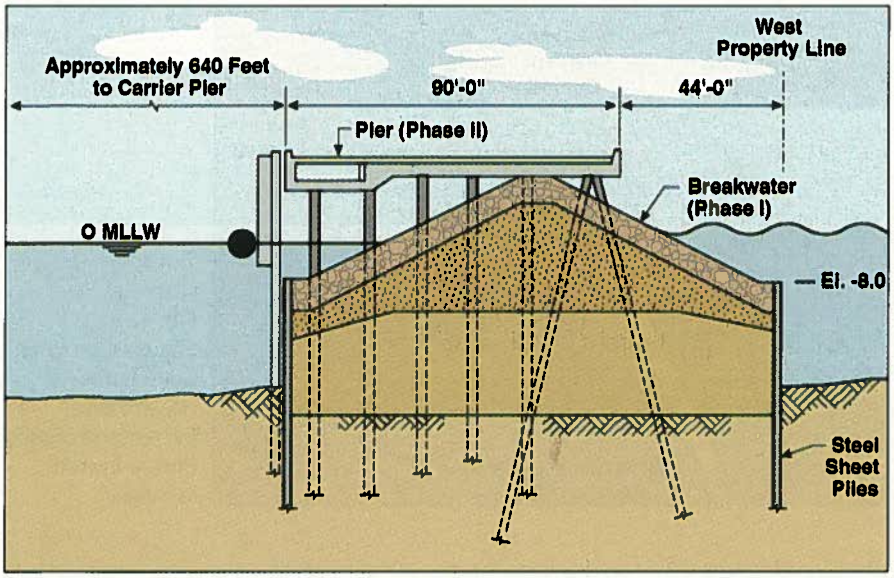


Fig. 6. Partial sheetpile cofferdam concept.

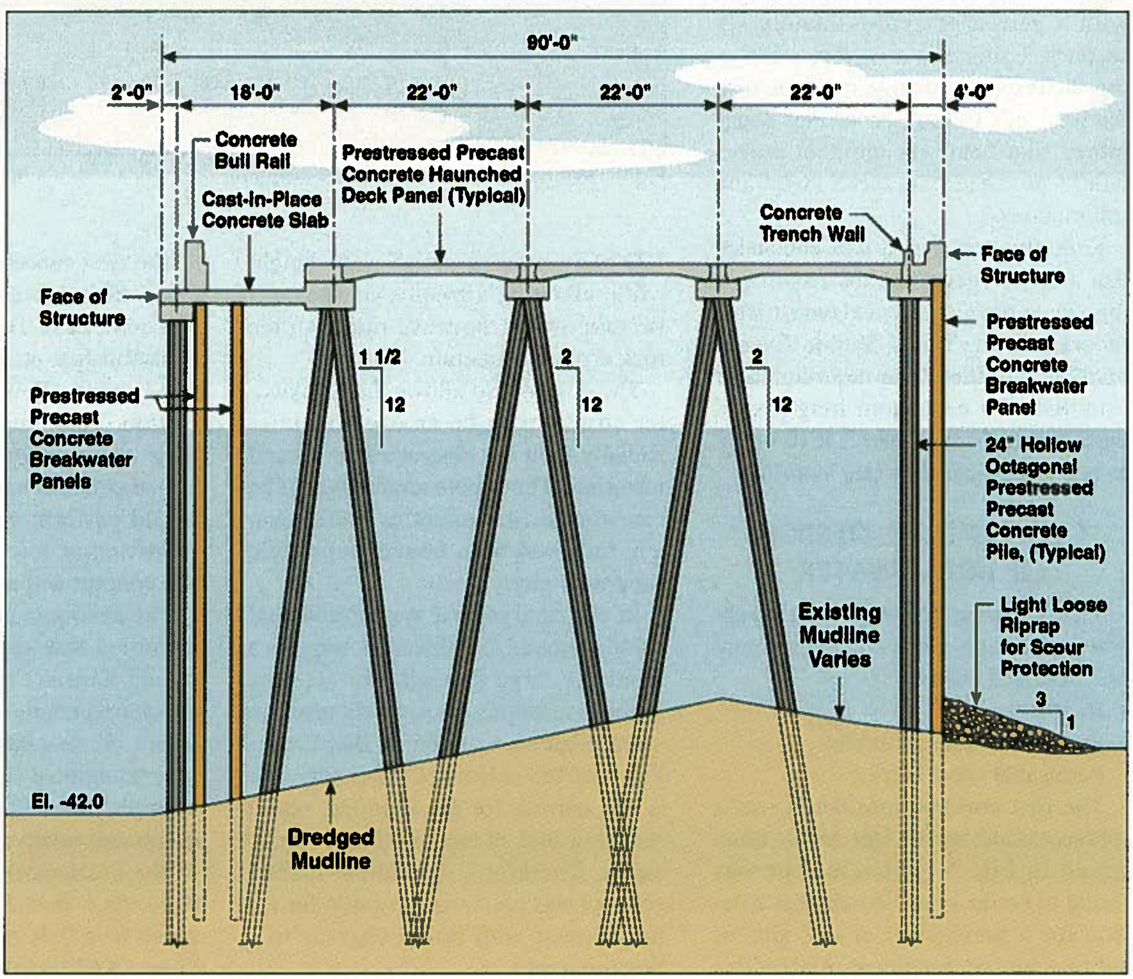
should be able to dissipate wave action and energy similar to the rubble-mound breakwater in order to keep the reflected wave height within limits.

The concept that was eventually adopted was a permeable wave barrier. The system consists of walls, or breakwater elements, which act as a wave barrier. Preliminary studies determined that the most cost effective combination of pier support structure and wave attenuation system would be to locate the breakwater elements along the perimeter of the structure. This type of breakwater would allow some transmission of the wave to minimize the reflected wave.

In addition, it was also thought that a permeable breakwater would be considered environmentally friendly to fish passage, allow water circulation, and prevent shoaling in the inner harbor of the berthing area. The selected breakwater element layout is shown in Fig. 7.

BERGER/ABAM worked with the Navy's environmental consultant, and their specialty consultant, Dr. Robert G. Dean, on the permeable breakwater concepts. It was determined by mathematical analysis that two potential solutions would satisfy the transmitted and reflected wave criteria as well as the geometric site constraints for this future pier structure. The first concept was to construct two walls spaced 45 ft (13.7 m) apart and aligned in plan with each other. The second concept was to build two walls spaced 90 ft (27.4 m) apart, with the first line of walls offset in plan from the second line of walls.

Fig. 7. Typical section of selected pier structure with permeable breakwater system.



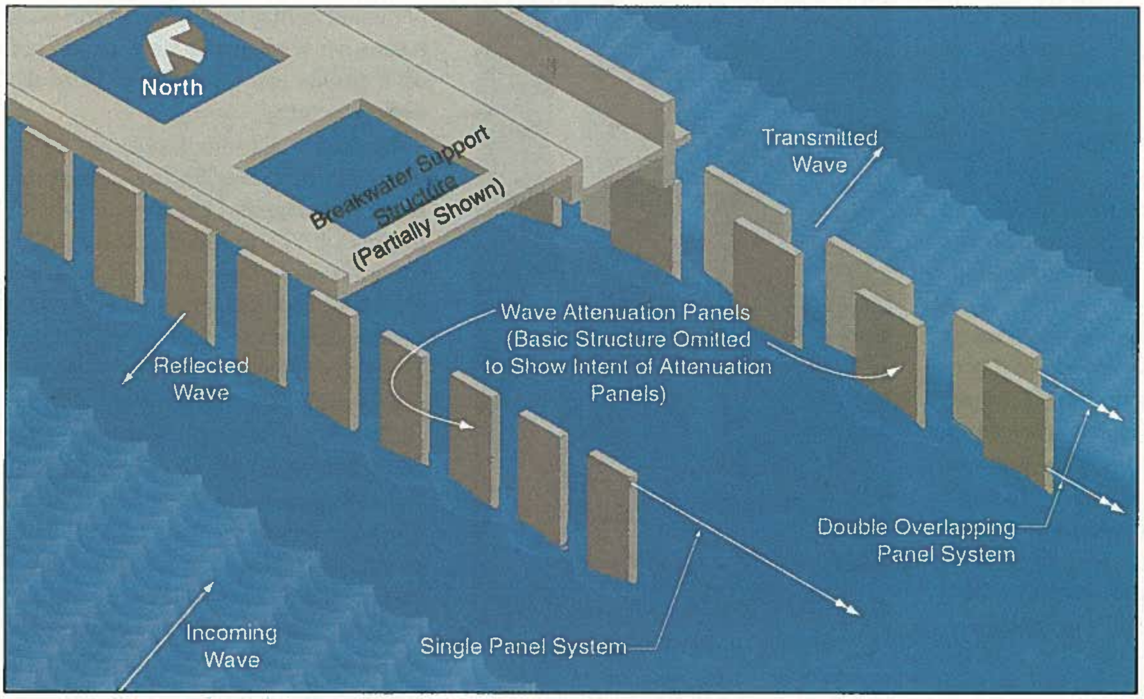


Fig. 8. Schematic plan of proposed permeable breakwater system, shown with a partial deck system.

Fig. 8 shows a schematic plan of the proposed permeable breakwater system. It consists of closely spaced wall elements, one set at each face of the breakwater structure. Mathematical analysis and prior experience with other breakwater structures show that the breakwater elements would need to extend quite deep in order to be effective for the long period waves.

LAYOUT OF STRUCTURAL BREAKWATER

It would be relatively straightforward to design a breakwater if its sole function is to protect vessels at adjacent berths. The real engineering challenge was to integrate the breakwater elements within a structure that would facilitate upgrade to a future berthing pier. As such, the structure was designed backwards by looking at the support structure for a completed pier and then looking at how to fit the breakwater elements into the final pier configuration. The typical pier section is illustrated in Fig. 7.

The key issue was the pile layout, particularly with regard to the Phase II constructability whereby the breakwater structure would become the structural framework for the future pier structure. It was deemed difficult and not cost effective to construct only a portion of the structural piling required for the future pier during breakwater construction. Consequently, all piling was installed during the Phase I construction of the structural breakwater.

Fig. 9 shows the typical layout of the piling and breakwater element. As noted previously, the attenuating effect of the vertical panel does not need to extend all the way to the mudline; however, it is required for a significant depth below the waterline. Many similar permeable wave walls consist of timber logging or steel framing or a combination of the two. These panels can be costly to install and require frequent maintenance. From a constructability standpoint, the equivalent of a concrete sheet pile proved to be most economical on a final-cost basis, and has the least cost on a life-cycle-cost basis as well. The precast and prestressed concrete panel, design for the marine environment, became the preferred breakwater element.

A single concrete panel system was used at the upwave or west face of the pier, adjacent to the Snohomish River

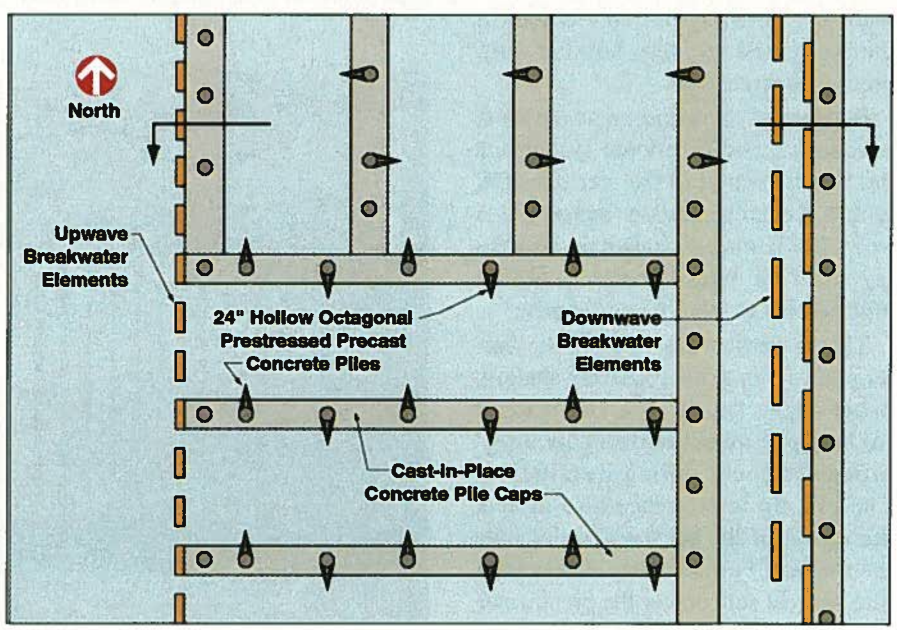


Fig. 9. Proposed plan layout of piling, pile cap, and breakwater elements.

Channel. A double overlapping concrete panel system was developed for the downwave or berthing side of the pier. Based on the site geometry, the incoming wave or upwave would strike the single panel system, with a significant portion reflected back and the remainder coming through the double overlapping panels as transmitted waves.

The upwave breakwater elements were 4 ft (1.22 m) wide and the downwave elements were 8 ft (2.44 m) wide. Both types of panels were 14 in. (0.35 m) thick, and their lengths varied from 70 to 94 ft (21.3 to 28.6 m). These precast concrete panels were designed for 6000 psi (41 MPa) at 28 days and axially prestressed to a final stress level of 1680 psi (11.6 MPa).

Piling layouts were developed for the ultimate function of the breakwater structure as a berthing pier. The structural system chosen for the breakwater pier was an assemblage using precast, prestressed concrete piles, cast-in-place concrete pile caps, precast, prestressed deck panels, gravel ballast, and fiber-reinforced concrete pavement.

This system is known as the Puget Sound System of pier construction. It was previously used on the Carrier Pier and the South Wharf, and is well known to all major waterfront contractors in the Northwest.

Precast, prestressed concrete piles were the obvious choice at this site, with its cost effectiveness and ease of installation. Lengths of 180 ft (54.9 m) were not an issue and contractors in the Northwest are very familiar with precast concrete piles.

Batter piles were chosen as the most economical and functional system for the lateral support of the pier structure with a relatively shallow batter of 1.5 to 2:12. The major challenge with the layout of the piles is constructability with respect to pile tip interference.

This is particularly true for the batter piles. Even at the relatively shallow batter angle, the tip of a 180 ft (54.9 m) long pile moves horizontally a significant amount during installation. The pile tip interference also affects the layout of the breakwater elements themselves. Further study determined that vertical support for the breakwater elements could be achieved in an intermediate silty sand layer. The batter piles would then be below the tips of the breakwater panels. Piles are battered in both transverse and longitudinal directions to resist lateral forces.

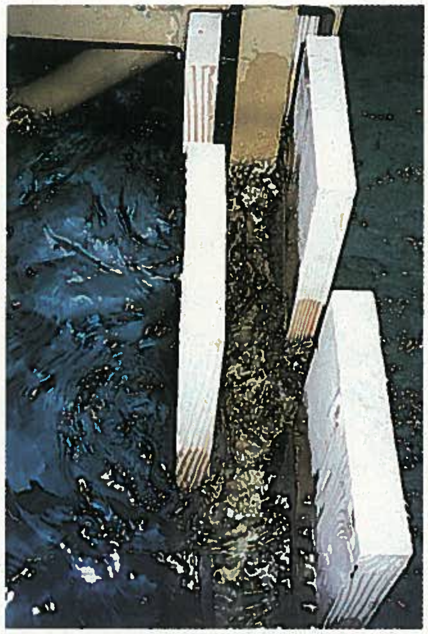


Fig. 10. Plan of model breakwater elements in wave tank.

The lateral support for the breakwater panels would be provided through the deck structure, consisting of cast-in-place concrete pile caps acting as struts and precast concrete deck panels acting as diaphragms. By constructing a limited deck structure (see Fig. 8), the Phase I construction costs could be kept within the budget with relatively easy upgrade to a functional berthing pier. Phase II construction would simply entail filling in the rest of the deck structure with precast, prestressed deck panels and pile caps to form the deck structure.

The batter and plumb piles are 24 in. (0.61 m) diameter hollow octagonal precast, prestressed concrete units with lengths of 172 to 188 ft (52.4 to 57.3 m). Piles were designed with 8000 psi (55 MPa) concrete strength with a pile bearing and uplift capacity of 250 and 130 tons (227 and 118 t), respectively.

The deck panels are typically 15 in. (0.38 m) deep at midspan and haunched to 24 in. (0.61 m) deep at the ends and are 8 ft (2.44 m) wide. The edge panels are 4 ft (1.22 m) wide prismatic sections, and $^1/_2$ in. (12.7 mm) prestressing strands are used for all deck panels. Concrete strengths for deck panels and utilidor lids are specified at 6000 psi (41 MPa). All prestressing steel was uncoated, Grade 270, low relaxation, $^1/_2$ in. (12 mm) diameter, seven-wire strands.

For lateral load resistance as a breakwater, a 26 ft (7.93 m) width of deck structure is constructed, continuous throughout the 1500 ft (457 m) length of the breakwater. This could then be used in Phase II as a work platform to facilitate the installation of the remaining deck panels and pile caps.

With the chosen breakwater element layout, the question of whether they would perform as intended with respect to wave reduction and wave reflection still remained. Additionally, the design loads imparted onto the pier

Fig. 11. Model of downwave breakwater elements at low tide in wave tank.

structure by the breakwater elements could also be verified. To address these concerns, a scaled model of the breakwater structure and layout was built and tested.

MODEL TESTING

The model testing was conducted at Chicago Bridge and Iron (CBI) Technical Services Company, Marine Research Department in Plainfield, Illinois. The testing was performed in the spring of 1993. The purpose of testing the specific breakwater model configuration was to answer questions in these areas:

- How much of the incident wave is transmitted through the breakwater?
- How much reflection will occur into the Snohomish River Channel?
- What loads are applied on the breakwater elements?

A 1:10 scale model of the breakwater structure and layout was constructed and tested in a 33 ft wide x 250 ft long x 18 ft deep (10.1 x 76.2 x 5.49 m) wave tank. Fig. 10 shows the plan view of the model breakwater panels in the wave tank. Fig. 11 shows the model downwave breakwater elements at low tide in the wave tank.

Waves were generated corresponding to both high water levels (deep water) and low water levels (shallow water). The deep water represented a tide level of extreme high water elevation of +14.5 ft (4.42 m) MLLW datum. The shallow water represented a tide level of extreme low water elevation of –4.5 ft (1.37 m) MLLW. Three wave heights at each period were tested, varying from three to seven seconds in one-second increments. The longer period waves were tested at 2, 4 and 6 ft (0.61, 1.22 and 1.83 m) heights. A random wave was tested at each wave height.

Wave heights were measured by wave probes at various points in the tank. Load cells were attached to the top and bottom of the model breakwater panels to determine the load on the individual panels and to the breakwater support frame. Differential pressure transducers were installed to measure the dynamic pressure on the panels. This measurement was also used to establish the shape of the applied load function.

Figs. 12 and 13 show graphic representations of the breakwater system's effectiveness with regard to reflected and transmitted waves. It should be emphasized that the curves were drawn as a best fit using only three data points collected for each wave period. This was considered sufficient to estimate the performance of the configuration as tested.

In Fig. 12, the transmission coefficient (C_t) is computed as the ratio of the transmitted wave height to the incident wave height. The reflection coefficient (C_r) is computed as the ratio of the reflected wave height to the incident wave height for the data shown in Fig. 13.

The following conclusions can be made with respect to the transmission coefficient:

- C_t decreases as the wave period decreases.
- C_t decreases as the wave height increases.
- C_t decreases as the water depth decreases.

Fig. 12 shows curves representing the limit for transmitted wave heights of 3.0 and 3.5 ft (0.91 and 1.07 m). The 3 ft (0.91 m) limit was defined as the desirable maximum transmitted wave height. The plotted data show

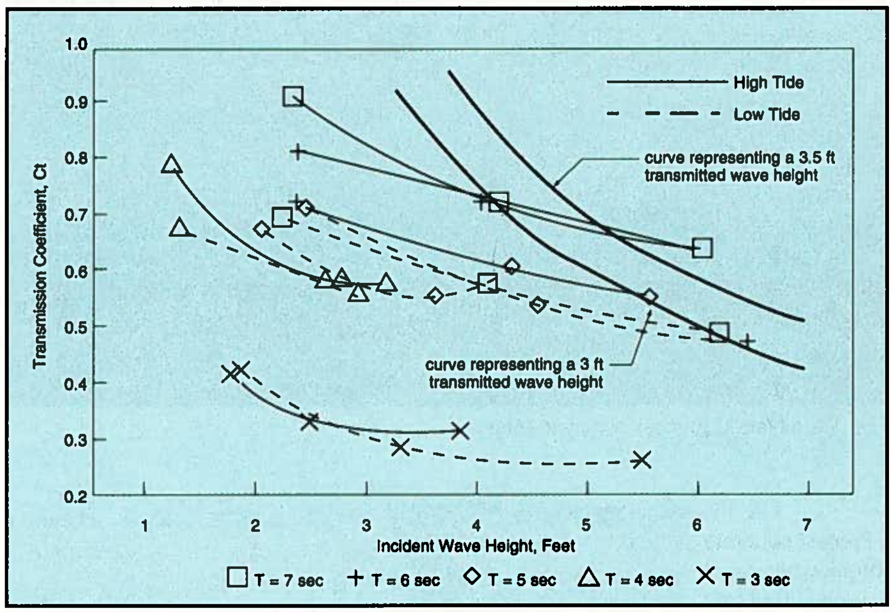


Fig. 12. Graph plotting transmission coefficient C_t versus incident wave height.

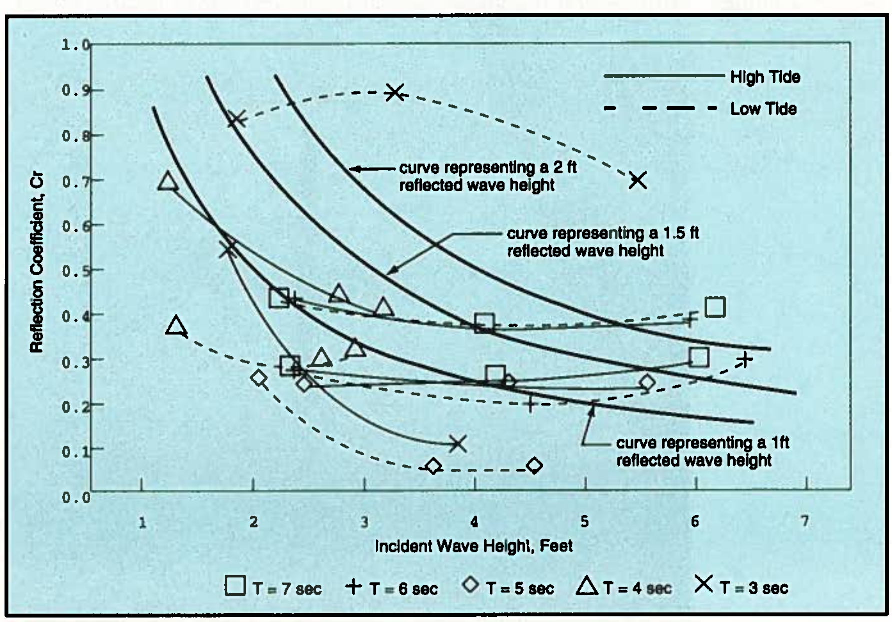


Fig. 13. Graph plotting reflective coefficient C_r versus incident wave height.

that the breakwater configuration satisfies the criterion of 3 ft (0.91 m) for all wave periods less than 6 seconds. For the longer wave periods of 6 and 7 seconds, the criterion of 3 ft (0.91 m) wave transmission is met for all wave heights less than 4 ft (1.22 m).

Subsequent independent verification of the wave period determined that the maximum period at this site is 4.4 seconds. Consequently, the breakwater layout will satisfy the revised wave transmission criteria for all wave heights.

Based on the data and curves shown on Fig. 13, it is difficult to draw general conclusions regarding the relationship between the reflection coefficient C_r, incident wave height, and wave periods, wave heights, or water depths. Acceptable reflected wave heights have not formally been established other than the approximately 1 ft (0.305 m) limit criterion as stated previously.

To illustrate the sensitivity of the reflected wave height criteria versus obtained test data, curves representing 1, 1.5, and 2 ft (0.305, 0.46, and 0.61 m) reflected wave heights are shown on Fig. 14. Most waves satisfy the maximum reflected wave height criterion of 1.5 ft (0.46 m).

Fig. 14. Installed precast concrete piles.

Fig. 15. Precast concrete breakwater panel being lifted into position for placement into template.

Wave forces and moments on the breakwater panels were determined based on output from the load cells in the model test. The wave force distribution was assumed to follow a cosh decay function similar to the linear wave theory acceleration term (i.e., the forces are mainly inertial). By integrating this function over the height of the panel, the location of the total force resultant, i.e., reaction centroid, was found.

Note that the panels were assumed pinned supported at the top (deck level) and bottom (mudline) of the panel. The maximum reactions and simple span moments in the panels were determined based on the calculated reaction centroid.

It should also be noted that this bending moment is calculated assuming a pinned support at the mudline elevation, while the actual bottom panel reaction centroid is several feet below the mudline elevation. This assumption is reasonable because the tip of the panel will actually be installed well below the mudline, which will provide some fixity. This in turn reduces the bending moment between the support points, offsetting any increase in bending moment resulting from an assumed lower bottom reaction support point.

Calculated unfactored design bending moments in the breakwater elements were summarized as follows. These measured bending moments have been increased approximately 25 percent to account for the variability in test results and wave loading. These bending moments will be applied to the breakwater elements in both directions, as the waves will act from both directions:

- Upwave Breakwater Row = 20 ft-kips/ft of width (89.0 kN-m/m)
- Second Breakwater Row = 25 ft-kips/ft of width (111.2 kN-m/m)
- Third Breakwater Row = 30 ft-kips/ft of width (133.5 kN-m/m)

The model test results indicated that the established design parameters relative to wave reduction and reflection could be achieved. Estimated design loads on the breakwater elements were also verified.

Fig. 16. Close-up view of installation of concrete breakwater panel.

Fig. 17.
Top view of upwave breakwater panels.

TIME LINE AND CONSTRUCTION COSTS

The Phase I Structural Breakwater project was fast tracked with regard to plans and specifications developed. Bids were opened in May 1994, with an awarded construction cost of $15.5 million. Construction was completed in September 1995, within budget and four months ahead of schedule.

Fig. 14 shows the installed precast concrete piles and Fig. 15 shows a breakwater panel being lifted into position for placement into the template. Fig. 16 shows the breakwater panels in-place. Figs. 17, 18 and 19 show the breakwater elements along the east face of the pier and Fig. 20 shows the elements installed along the west face of the pier.

The original bid documents had an option of using two tongue and groove 4 ft (1.22 m) wide breakwater panels in place of one 8 ft (2.44 m) wide panel for weight considerations. The contractor chose to use the 8 ft (2.44 m) panels since the equipment was available for handling the larger size panels. The weight of these panels ranged from 26 to 70 tons (24 to 63 t). Fig. 21 shows the partially completed deck system. See Fig. 2 for the completed Structural Breakwater.

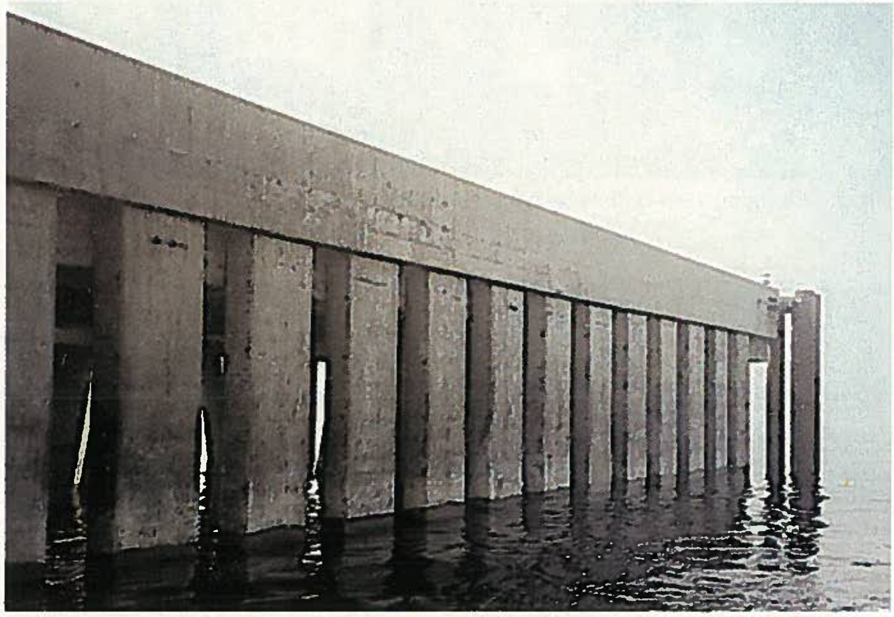


Fig. 18. Elevation view of upwave breakwater panels.

The design of Phase II, namely, upgrading the Structural Breakwater to a functional Berthing Pier, was performed during the summer of 1996 and the bids were opened in November 1996. The Phase II award construction cost was $9.6 million, including utilities. Construction was completed in January 1998, within

Fig. 19. Panoramic view of upwave breakwater panels.

Fig. 20. Elevation view of downwave breakwater panels.

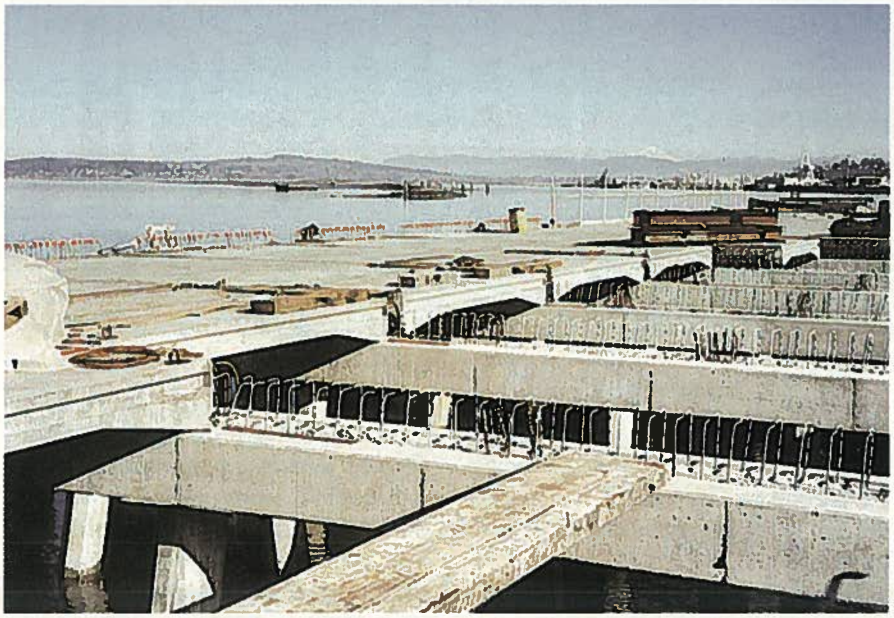


Fig. 21. View of partially complete deck system.

budget and on schedule. See Fig. 3 for an aerial view of the completed Breakwater Pier.

The precast/prestressed concrete components for the Phase I part of the project (piles, deck panels, and breakwater panels) were manufactured and shipped by barge to the site [a distance of about 60 miles (100 km)] by Concrete Technology Corporation in Tacoma, Washington.

The precast/prestressed concrete components for the Phase II part of the project (deck panels and lids) were manufactured and trucked to the site [a distance of about 60 miles (100 km)] by Bellingham Marine in Bellingham, Washington.

The facility required 842 24 in. (610 mm) hollow octagonal precast, prestressed concrete piles, measuring 24 in. (610 mm) in diameter and 172 to 188 ft (52.4 to 57.3 m) to support the structure.

The breakwater consists of 423 14-in. (356 mm) thick precast, prestressed vertical concrete panels. For the pier, there were a total of 985 precast concrete deck panels, about half of which were haunched and the other half were of constant depth. Table 1 summarizes the quantities and principal dimensions of the precast/prestressed products used on this project.

CONCLUDING REMARKS

This project is recognized as a state-of-the-art berthing facility that is being used routinely and successfully by the United States Navy.

After completion of this facility, the U.S. Navy presented BERGER/ABAM

Table 1. Quantities and principal dimensions of precast concrete components.

Product	Quantity	Description	Length
Piles	842	24 in. diameter hollow octagonal prestressed concrete	172 to 188 ft
Breakwater Panels	203	14 in. thick x 4 ft wide precast, prestressed concrete	70 to 94 ft
	220	14 in. thick x 8 ft wide precast, prestressed concrete	70 to 94 ft
Deck Panels	515	6 ft wide precast, prestressed concrete, 1 ft 3 in. deep at midspan haunched to 2 ft at ends	18 ft 3 in.
	428	6 ft wide precast, prestressed concrete, 1 ft 3 in. deep at midspan haunched to 2 ft at ends	20 ft 3 in.
	18	1 ft 6 in. deep x 4 ft wide precast, prestressed concrete, prismatic	18 ft 3 in.
	9	1 ft 6 in. x 6 ft wide precast, prestressed concrete, prismatic	18 ft 3 in.
	15	2 ft deep by varying width precast, reinforced concrete	Length varies
Utilidor Lids	276	1 ft 6 in. deep by varying width precast, reinforced concrete	Length varies

Note: 1 ft = 0.3048 m; 1 in. = 25.4 mm.

with the Design Excellence Award for exemplary performance of design service on the Breakwater/Pier Structure at Naval Station Everett.

The structure was also the recipient of the 1998 PCI Design Award for Best Miscellaneous Structure. The jury citation read:

"Precast/prestressed concrete provided a cost effective solution that is both functional and aesthetic for a naval berthing facility situated in a difficult and environmentally sensitive coastal area. This solution can be adapted to similar structural applications in aggressive marine environments."

BERGER/ABAM worked closely with the U.S. Navy to efficiently handle the challenges associated with the development of the Breakwater-Pier Structure under difficult site constraints along with intensive interaction with the environmental agencies. Additionally, expertise in the Navy funding and programming process allowed the provision of workable solutions to the required phasing. A cooperative and skillful contractor ensured that the Structural Breakwater was completed ahead of schedule and under budget.

Another important reason for the success of this project is that high quality precast/prestressed concrete products, manufactured under controlled factory conditions, were delivered by the precast producers to the site on schedule.

The project needs were skillfully and cost-effectively met. The Breakwater/Pier Structure performs as a functioning berthing pier and provides a breakwater for ships at berth. It was designed within the U.S. Navy's funding limitations for each phase of the work. The breakwater concept was a cost effective solution that allowed for easy conversion to a complete berthing pier.

The use of precast/prestressed concrete for this Navy project provided the following benefits:

- An aesthetically pleasing structure that ties into the existing facilities.
- A highly functional structure that is low in maintenance.
- Limited environmental impact.
- Durability in an aggressive marine environment.
- Easy access to utilities, while supporting heavy crane loads.
- A fast track construction schedule.
- A cost effective structural system with easy integration of Phase I and Phase II work.

Since completion, the Breakwater/Pier Structure has performed extremely well having withstood several severe storms and strong tidal surges. With the experience gained, this project can serve as a model for future breakwater/berthing facilities around the world.

CREDITS

Owner: United States Navy, Everett, Washington

Designers:

- BERGER/ABAM Engineers Inc., Federal Way, Washington — Prime designer
- HNTB Corporation, Bellevue, Washington — Mechanical and electrical utilities, architecture
- Norton Corrosion Limited, Woodinville, Washington — Cathodic protection
- Chicago Bridge and Iron Technical Services Co., Plainfield, Illinois — Wave tank testing

Contractors:

- Phase 1 – Structural Breakwater – Manson Construction & Engineering Co., Seattle, Washington
- Phase 2 – Breakwater Pier – Robison Construction Inc., Sumner, Washington

Precast Concrete Manufacturers:

- Phase 1 – Structural Breakwater – Concrete Technology Corporation, Tacoma, Washington — Deck panels, breakwater panels, piling
- Phase 2 – Breakwater Pier – Bellingham Marine, Bellingham, Washington — Deck panels, utilidor lids