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J. Brady Richmond for the degree of Master of Ocean Engineering in Ocean Engineering presented on March 11, 2005.

Title: Design and Implementation of a Physical Model for Keystone Harbor, WA

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Daniel T. Cox

Strong tidal currents experienced at Keystone Harbor, WA, have caused navigational hazards and disrupted ferry service in the area. This thesis presents details of the design, construction and implementation of a large scale physical model of Keystone Harbor used to evaluate the effectiveness of proposed jetty design alternatives to improve navigation at the Keystone Ferry Terminal.

A fixed bed, 1:40 Froude scaled model was built in the Tsunami Wave Basin at the O. H. Hinsdale Wave Research Laboratory using a bathymetric survey of Keystone Harbor. A recirculating pump system simulated the peak ebb tidal current and a wavemaker simulated 50 year storm conditions. A series of five tests were conducted to (1) evaluate the performance of existing and proposed jetty extensions and (2) determine whether these changes would have an adverse affect on the neighboring cobble beach. Analysis revealed large velocity gradients for all 4 proposed alternatives at the harbor channel entrance which may have significant impacts on navigational safety. No adverse impacts were observed on the adjacent beaches.

Design and Implementation of a Physical Model for Keystone Harbor, WA

by
J. Brady Richmond

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APPROVED:

Redacted for privacy

Major Professor, representing Ocean Engineering

Redacted for privacy

Head of the Department of Civil, Construction and Environmental Engineering

Redacted for privacy

Dean of the Graduate School

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J. Brady Richmond, Author

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Design and Implementation of a Physical Model for Keystone Harbor, WA

1 Introduction

The Puget Sound region of northwest Washington is a conglomerate of island communities centered around the economic hub of Seattle. Many of the residents living and working in this region rely heavily on the ferry transportation system allowing travel between these islands. The ferry system, operated by Washington State Ferries (WSF), services eight counties in Washington and the province of British Columbia, Canada. Ridership in the fiscal year of 1999 included over 26 million people and 11 million vehicles, making WSF the largest operation of its kind in the United States (WSDOT 2005).

The Keystone Ferry Terminal, the subject of this report, is one of twenty sites serviced by the ferry system. This small, manmade harbor is located approximately 40 miles NW of Seattle, on Whidbey Island. Figure 1 shows the location of Keystone Harbor in relation to surrounding cities.

The ferries operating out of Keystone Harbor provide direct service between Keystone and Port Townsend. Sailings occur 10-12 times daily depending on season. The ferry crossing at Keystone is extremely important for regional mobility to the residents of Whidbey Island. The ferry route provides the most direct link between Whidbey Island and the Olympic Peninsula (Playter 2003). Commute times are approximately 30 minutes by ferry compared to a three hour commute by car.

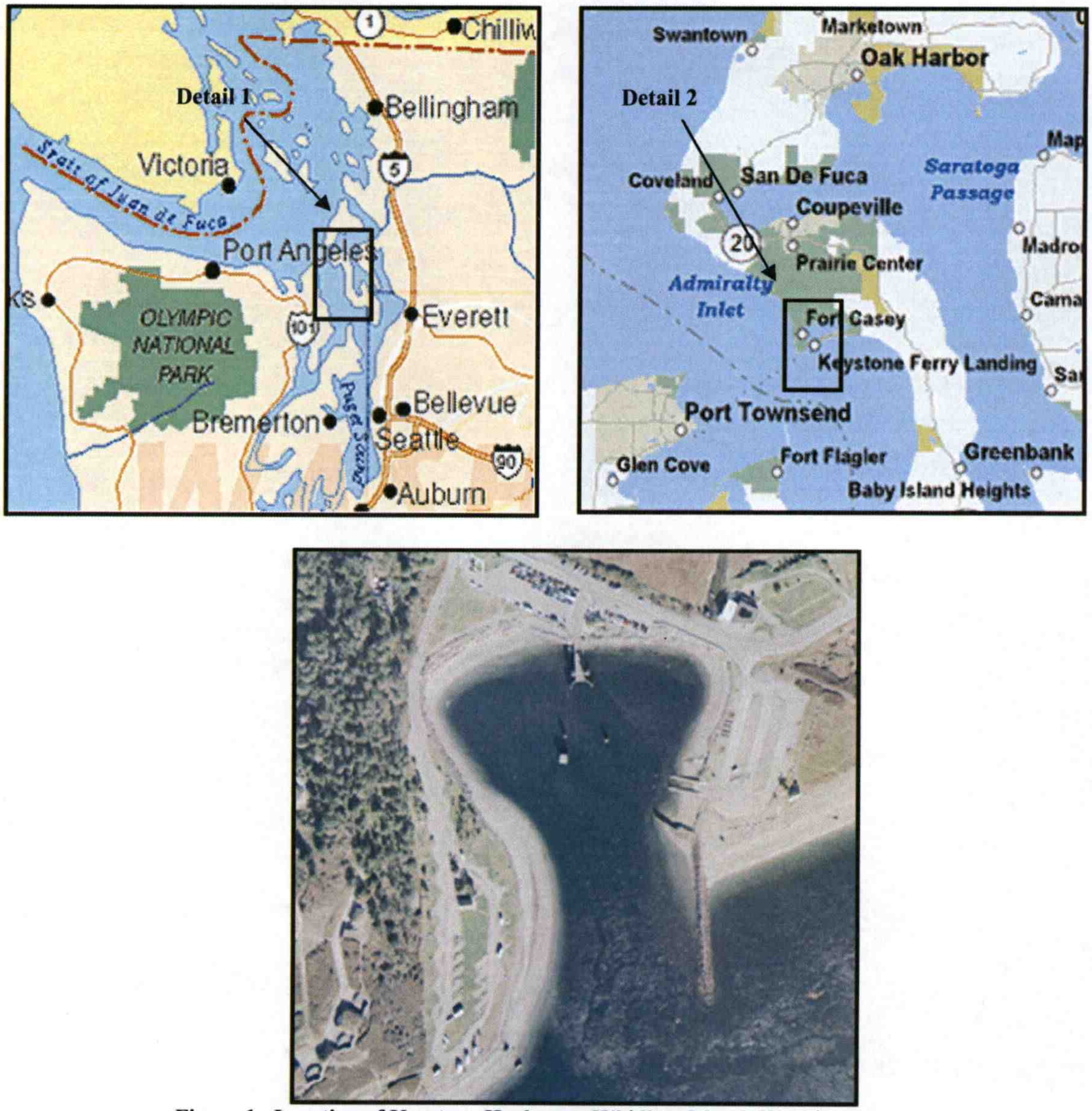

Figure 1. Location of Keystone Harbor on Whidbey Island, Washington.

1.1 History

The Keystone-Port Townsend ferry route originated in the early 1900's. Until 1948, the auto ferry operations were located approximately 1.5 miles east of the existing ferry terminal. During this time ferry service operated on a seasonal schedule, with no service during the winter months due to inclement weather and lower ridership.

The ferry terminal has been at its current location since 1941. At this time, the United States Government, through the Secretary of War, authorized a new harbor to be constructed on the Southwest shore of Crockett Lake. This harbor would allow year-round service, providing a reliable ferry shuttle for troops and supplies between army forts on the Olympic Peninsula and Fort Casey, located adjacent to Keystone Harbor. In 1948, the construction of the ferry terminal, riprap jetty, and berm separating Crockett Lake was completed.

The initial intentions of the riprap jetty extending from the east side of Keystone Harbor were to provide protection for the ferries from waves and longshore currents experienced at the site. It also served as sediment trap, impeding the transport of beach material into the harbor entrance. It was thought that this structure would doubly serve its purpose both by protecting harbor navigation and minimizing maintenance dredging required in the harbor (Shepsis 2004).

1.2 Site Conditions

1.2.1 Waves and Tides

Keystone Harbor resides northeast of Admiralty Head with a harbor channel centerline oriented approximately with a North-South axis. Deep water wave conditions at the site are generally minor due to the interference caused by both the Olympic Peninsula and Vancouver Island, Canada. Certain conditions do allow waves to propagate through the Strait of Juan De Fuca where they eventually encounter Admiralty Head. Wave refraction around the headland results in the presence of minor waves experienced at Keystone.

Locally generated wind waves occur due to frequent winds acting on Admiralty Bay. These winds and resulting waves, generally act in the 180° to 210° (true N) segment. Maximum wave heights recorded during winter storms have been upwards of 1.2 m (4.0 ft) with 5 s periods. Tides at Keystone Harbor and Puget Sound in general, consist of semidiurnal, mixed tides (CHE 2004). Mean tidal range is 2.6 m (8.5 ft).

1.2.2 Currents

Strong ebbing and flooding tidal currents have a significant impact on the local area surrounding Keystone Harbor. Predominant current direction is in the south-westerly ebb direction regardless of tidal state, as can be seen in Figure 2.

Figure 2. Oblique view of Keystone Harbor, WA showing predominant current direction.

Adjacent to the Keystone jetty, a persistent, counterclockwise rotating gyre exists. This eddy only experiences brief periods when the current direction reverses and flow is observed in an easterly direction at much less speed than the ebb flow. Current magnitudes have been recorded to reach as high 1.2 m/s (3.9 ft/s) in the area of interest (Lilly 2002).

1.3 Problem Statement

The combination of low tides and strong longshore currents experienced at Keystone has proved to be a serious problem for safe navigation of ferries entering and exiting the harbor. The problem has resulted in the most canceled ferry sailings of all WSF routes. In 2001, there were 99 canceled sailings due to a combination of tides and currents. Cancellations at Keystone amount to approximately 5% of total scheduled sailings, nearly 4 times the WSF average. These canceled services represent lost revenue to the ferry system and, more importantly, disrupt a vital public service (Playter 2003).

The existing jetty configuration at Keystone does not provide adequate protection against cross currents within the harbor channel entrance. Ferries entering the harbor are subjected to strong hydrodynamic forces on the stern while the bow is in still water in lee of the jetty (CHE 2004). This forces the ferry stern to rotate, causing difficulties in the maneuverability of the ferry. In addition, the ability of ferries to maneuver within the channel entrance is highly restricted due to the size of Keystone Harbor. To minimize longshore drift caused by cross channel currents, the simple solution is for ferries approaching the harbor to increase speed, attempting to power through the cross-currents. However, the required stopping distance necessary for this solution is unavailable with the existing harbor setup. A number of unintentional ferry groundings have resulted due to the cross currents experienced at Keystone, as can be seen in Figure 3.

Figure 3. Grounded state ferry at Keystone Harbor, WA. (source: Seattle Post-Intelligencer, 8/ 23/2002, photo credit: Mary Schenker).

1.4 Possible Solutions

The conditions experienced at Keystone Harbor have revealed a need to improve efficiency and safety of ferry services to Whidbey Island. A feasibility study, undertaken by CH2M-Hill, explored a number of probable options aimed at solving the existing issues. Along with the aforementioned goals, the feasibility study also hoped to incorporate objectives of WSF's strategic plan. In this plan, WSF aimed to replace the existing fleet of Steel Electric vessels with a larger, deeper draft fleet of vessels (Playter 2003). This further complicated the issues surrounding size and maneuverability within Keystone Harbor. Three possible solutions were considered as feasible options:

- Relocation of the harbor and associated ferry terminal
- Cancellation of ferry service in the area
- Extend and reorient existing jetty configuration

Relocation of the existing harbor and cancellation of the Keystone ferry service were scrutinized by citizen groups and ferry users. Nearby residents of proposed relocation sites were opposed to the construction of a new harbor and ferry terminal. Many residents were not receptive to this new terminal impinging on their homes and workplaces. The only option that came under no outward opposition was the extension and possible reorientation of the existing jetty. Further investigations were needed for this option to be heavily considered as probable solution.

2 Methodology

An in-depth examination, investigating the feasibility of jetty extension and reorientation, was undertaken by Coast and Harbor Engineering, Inc. (CHE), as subcontracted through CH2M-Hill. This investigation looked at all aspects necessary to evaluate the effectiveness of alternative jetty modification designs. This evaluation included the following independent actions: field measurement program, numerical simulation, shoreline erosion analysis, and large scale physical modeling. Utilizing information from each of these aspects will provide substantial information necessary to make final improvement recommendations.

2.1 Field Measurement Program

A series of field data collection programs were administered by both CH2M-Hill and CHE to provide data representative of site conditions. Included in this field measurement program was a bathymetric survey providing updated topography of the nearshore regime. Additionally, in-situ current velocity and wave height measurements were made in the vicinity of Keystone Harbor. To measure longshore currents, two RD Instruments, Inc. Acoustic Doppler Current Profiler (ADCP) meters were deployed near the terminal site for a period of 18 days from June 28 to July 15, 2002. ADCP measurements used in the report were analyzed at a depth of 8.25 m (27.1 ft) below the sea surface; this depth corresponded with the mid-draft depth of ferries proposed to operate out of Keystone Harbor (Lilly 2002).

Wave climate was obtained through a combination of corrected offshore buoy data records and use of the US Army Corps of Engineers computational program "Windspeed Adjustment and Wave Growth" (Lilly 2002). Information from field data collection activities was used to establish environmental site conditions for the remaining aspects of the investigation.

2.2 Numerical Simulation and Shoreline Erosion Analysis

Numerical modeling simulated a variety of coastal processes that effect both ferry operation at Keystone Harbor and the environmental aspects of the surrounding area. These simulations included the modeling of longshore currents, wave propagation, refraction, diffraction, reflection, sediment transport, and shoreline morphology. Information from these numerical simulations provided a basis for comparing existing conditions to the changes due to alternative jetty modifications. Numerical simulations widely used in engineering practice, such as ADCIRC, STWAVE, and RMA4, were employed during this phase of the investigation.

2.3 Physical Modeling

In conjunction with goals set forth by CHE, the O.H. Hinsdale Wave Research Laboratory (WRL) at Oregon State University was subcontracted to design and build a scaled physical model of Keystone Harbor. The purpose of the model was to accurately reproduce scaled site bathymetry and environmental conditions, providing grounds to test alternative jetty designs in a controlled laboratory setting. Information from this study was used to aid CHE in the evaluation of alternative jetty design performance and effectiveness.

This thesis will concentrate on the strategies used to design and construct the scaled physical model of Keystone Harbor, WA. It will also focus on experimental methods used to collect and analyze data during the testing of four proposed jetty design alternatives. The results of the experiment were provided to CHE for verification and validation the aforementioned numerical simulations. The study aimed at providing qualitative evaluation of jetty design alternatives with regard to their effect on shoreline changes in the vicinity of the jetty. In addition, the study allowed quantitative assessment of jetty effects on longshore currents near the harbor channel entrance.

2.3.1 Major Objectives

The scope of work provided by CHE outlined four main objectives to be attained while developing the physical model setup and testing method. Design and construction of model scaling was optimized to resolve these complicated issues, providing guidance through the development process. The objectives were as follows:

- Improve the navigational hazards associated with environmental conditions experienced at Keystone Harbor, WA.
- Minimize adverse environmental impacts on the surrounding shoreline adjacent to jetty.
- Determine optimal jetty design length and orientation.
- Test waves and currents to observe wave refraction/diffraction patterns and quantify (where possible) longshore current velocity changes.

3 Background Research and Design Considerations

The following section discusses the sequence of events that lead to the construction and implementation of the Keystone Harbor physical model. The process began with a literature review of past experiments with similar objectives. The literature review provided a foundation for model design. Design of the model involved scaling parameters, bathymetry design and orientation within the basin, and the determination of requirements for longshore current generation.

3.1 Literature Review

Hughes and Schwichtenberg (1998) conducted a physical model study similar in scale and scope to that of Keystone. Their study involved current-induced scour along a breakwater at Ventura Harbor, CA. They developed and constructed a 1:25 (Model: Prototype, scaling factor *sf* = 25) scale movable-bed physical model to replicate the coastal processes that occur at the harbor channel entrance. Although the objective of their investigation was to study sediment scour, whereas the Keystone study focused on longshore current affects, much of their laboratory setup was applicable.

The Ventura model was built on the flat bottom portion of a wave flume that featured a rectangular movable-bed section. Flows representing longshore currents were generated by pumping water through a 3.7 m (12.1 ft) wide manifold. Vertical guide walls were used to aid in the directing of water flow towards the model area of interest. Due to the difficulty involved with modeling sediment transport based on strict similitude, further addressed in Section 5.2.5, calibration of the model was carried out by the reproduction of a prototype event.

One primary model design consideration taken from the Ventura Harbor investigation was the use of a current generation system consisting of a pump, supply line, and discharge manifold. This setup allowed the efficient generation of currents that could be easily controlled by valve adjustments. A design with similar characteristics was proposed as a strong possibility for use in the Keystone model setup.

Further literature review provided design guidance on advantages and disadvantages of various current generation systems. Included are the works of Hamilton et. al. (1996), Hamilton and Ebersole (2001), and Rosati et. al. (1995). Most of these works refer extensively to the experiment of Visser (1980), while their experiments made minor modifications to improving the recirculation system.

The work of Simons, et. al (1995) was also beneficial to application towards the Keystone Harbor currents. Longshore currents were circulated using a series of 4 independently driven, reversible axial flow pumps. Currents were introduced into the facility through 40 outlet flumes, each controlled by its own undershoot weir. A matching set of flumes was located opposite of the outlet flumes to serve as an inlet return to the pumps. Pumps were actuated under a programmable control, allowing time varying longshore currents based on user-defined values. This system utilized a similar delivery system to that of Hughes and Schwichtenberg with the improvement of weirs at both inlet and outlet locations. This study revealed the distinct advantage of utilizing multiple lower capacity pumps to create and control longshore current flow. In addition, the coupling of multiple pumps allowed large flow rates to be achieved at a considerably lower cost than a single high output pump. Other important information gained from the study was the integration of a current guide wall for added control of current direction.

A combination of these methods were considered in the design of the experimental setup of Keystone Harbor. The current generating system of Keystone Harbor was only constructed for this project, rather than as a permanent installation to the facility. Therefore, modifications to previously mentioned methods were made to accommodate this situation. Details are given in Section 3.3.3.

3.2 Site Visit

To provide a sense of site scale to those working on the Keystone physical model, a visit to the project site was taken in early August 2004. The site visit allowed the design team to acquaint themselves with the actual site in question and to gain insight into the scope of the real world problem. Students were able to increase their understanding of the size, topography, environmental conditions, and harbor layout by observing the problem area directly. Notable observations included the similar length scales of the existing jetty and the ferries operating at the terminal. In addition, understanding the size and distribution of the cobble beach material at the site was important. Figure 4 shows the Keystone Harbor jetty and the adjacent cobble beach.

Figure 4. Keystone Harbor jetty and adjacent cobble beach.

3.3 Preliminary Design

3.3.1 Froude Scaling

For initial model design considerations, a number of scaling factors were explored to maximize model size while minimizing adverse laboratory effects (see Hughes for a detailed discussion). Previous studies by Seabergh and Smith (2002) demonstrated that Froude scaling reproduced wave heights and currents occurring at tidal inlets well with large scale physical models (sf = 75 and below). Froude scaling is applicable for processes in which inertial forces are balanced primarily by gravitational forces, as is the case in most gravity wave problems. The Froude number, F, is given by

Eq. 1

$$F = \frac{U}{\sqrt{gl}}$$

where U is a characteristic velocity, g is gravitational acceleration, and l is a characteristic length. Similitude is achieved by setting model and prototype Froude numbers equal to each other. For the case of constant gravitational acceleration, the scaling for velocity is given by

Eq. 2

$$\frac{U_m}{U_p} = \sqrt{\frac{l_m}{l_p}}$$

where subscripts p and m denote prototype and model parameters, respectively. The scaling for wave period, T, is given by

Eq. 3

$$\frac{T_m}{T_p} = \sqrt{\frac{l_m}{l_p}}$$

Using Froude similitude, length dimensions are scaled directly with the scaling factor, while velocity and wave period are scaled as the square root of the scaling factor. Table 1 gives a summary of scaling analysis for the three scaling factor options analyzed.

Table 1. Scaling analysis for scaling factor *sf* = 30, 40, and 50.

Parameter	Prototype		1 : 30, *sf*=30		1 : 40, *sf*=40		1 : 50, *sf*=50	
Depth, D	25	m	83.0	cm	62.5	cm	50	cm
Height, H_{min}	0.9	m	3.0	cm	2.3	cm	1.8	cm
Height, H_{max}	1.2	m	4.0	cm	3.0	cm	2.4	cm
Period, T_{min}	4.0	s	0.73	s	0.63	s	0.56	s
Period, T_{max}	5.0	s	0.91	s	0.79	s	0.71	s
Jetty Length	118	m	3.9	m	2.9	m	2.4	m
Jetty entrance width	61	m	2.0	m	1.5	m	1.2	m
Peak current	1.2	m/s	0.22	m/s	0.19	m/s	0.17	m/s
Cross sectional area	6,800	m^2	7.5	m^2	4.2	m^2	2.7	m^2
Volumetric flow	$8.2*10^6$	ltr/s	$1.6*10^3$	l/s	$7.6*10^2$	ltr/s	$4.6*10^2$	ltr/s

Note: 1.0 ltr/s = 16 GPM

3.3.2 Model Bathymetry Design

To determine the scaling of the Keystone Harbor model that would fit appropriately in the basin and be the most accurate representation of the site conditions, an area of interest needed to be established. The area chosen included the existing jetty, the area of problematic currents, and the surrounding bathymetry. The area of interest extended offshore to the 25 m (82 ft) contour, the depth beyond which the incoming waves would not be significantly affected by the bottom boundary. To estimate the required volumetric flow, the cross-sectional area of a typical bathymetry transect was calculated. This value was then multiplied by the current velocity yielding the volumetric flow required for each scaling factor. It was determined that a 1:40 Froude scale physical model was most reasonable for the necessary application. The scale would allow a sufficient area of interest to be examined while the required volumetric flow rate was achievable with a series of commercial pumps.

The physical model of Keystone Harbor was constructed in the tsunami wave basin (TWB) at the WRL. Basin dimensions are as follows: Length 48.8m, Width 27.8m, and Depth 2.1m (160 ft, 87 ft, 7 ft). A bathymetric survey provided by CHE was scaled according to the dimensional similitude described above. A contour plot of the scaled survey data was created using the surface mapping program Surfer. This base map was then manipulated in the drafting program AutoCAD for final design application. The model orientation within the wave basin was aligned so that waves incident from the wavemaker face would correspond with a wave heading of 195° (true N) as observed at the actual site. The chosen wave heading is the mean of dominant wave directions as stated in the CH2M-Hill Feasibility Report (Lilly 2002).

Figure 5 shows a layout of the design for the Keystone Harbor bathymetry in the TWB. Details on the construction methods is given in Section 4.1.

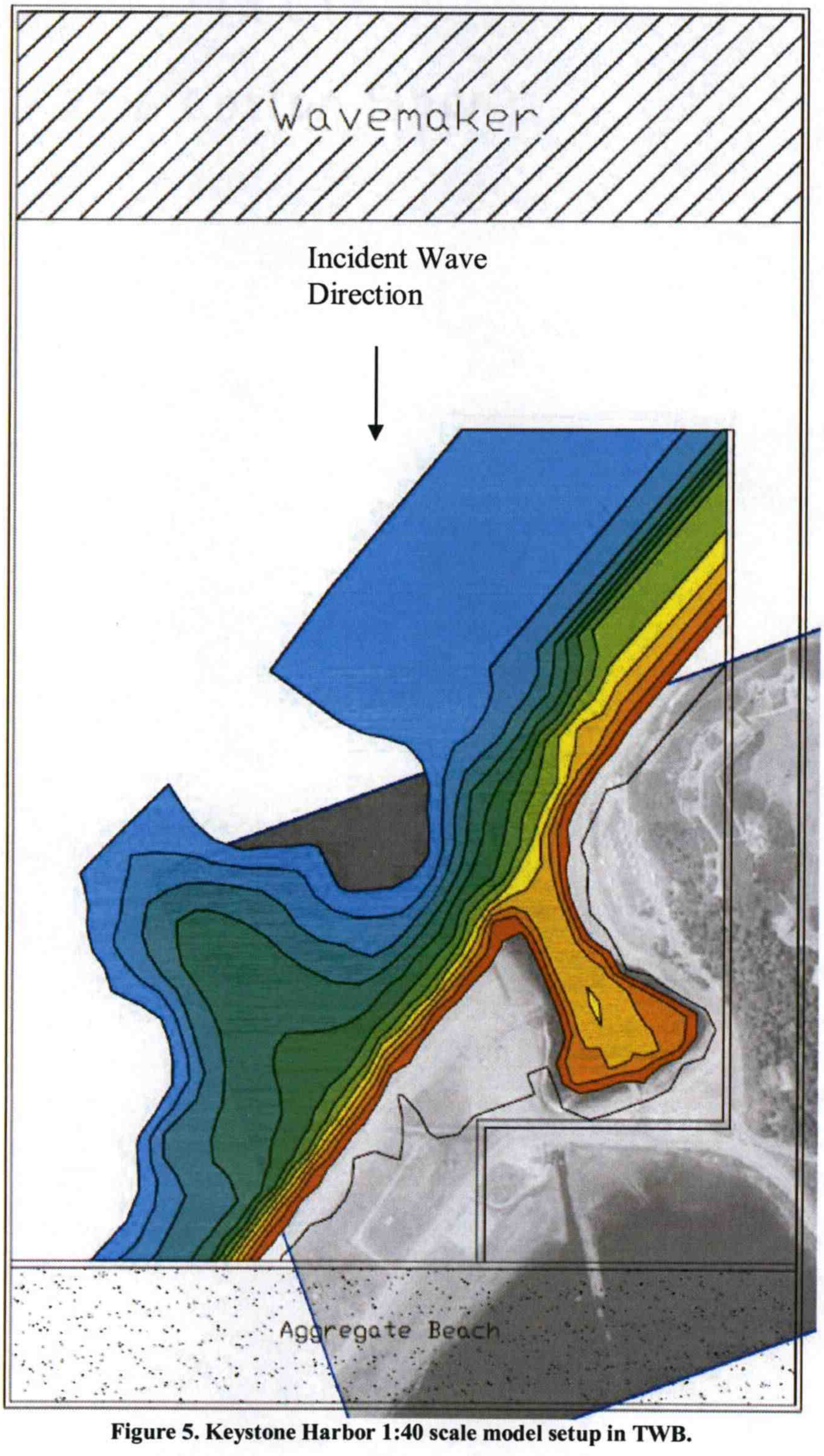

Figure 5. Keystone Harbor 1:40 scale model setup in TWB.

3.3.3 Current Generation Design

Two recirculation systems were considered for reproducing scaled tidal currents in the model. The first system would consist of open channel flow at both the inlet and discharge ends of the setup. A recirculation pump would be used to supply flow to the system and a series of weirs would be used to control flow. Advantages of this setup were ease of installation and the entire system being subjected to atmospheric pressure, minimizing head losses. The major disadvantage of this system was the lack of adjustable flow control, leading to difficulty in the establishment of a uniform flow field.

The second system would utilize a pump and discharge manifold setup similar to that of Hughes and Schwichtenberg. This system would consist of an open channel inlet supplying water to a pump. Attached to the pump, a discharge manifold with valve controls would be used as an outlet for the charged flow. This setup had the distinct advantage of flow control from the valves located on the discharge manifold. Disadvantages of this system included the expense of the piping and manifold setup, as well as head losses within the system.

A pump and discharge manifold setup was chosen since flow control was critical for the study. This setup differed from that of Hughes and Schwichtenberg because it used open channel flow at the intake return. Previous designs involved both manifold inlets and outlets. The goal was to use an open channel return to minimize headloss associated with long piping distances. By developing a setup with open channel flow at the intake, pump capacity requirements were less than if an intake manifold and piping system had been used. This design also decreased costs associated with the longshore current generation system.

During the testing of longshore currents, a current guide wall was constructed of stacked concrete cinder blocks. This guide wall began at the terminal end of the discharge manifold and extended towards the southeast corner of the TWB. The guide wall would minimize gyres and limit viscous effects between still and moving water. Its installation would allow additional control of current direction during this phase of testing.

4 Implementation and Construction

This chapter describes the construction methods used to implement the physical model designs described previously. In addition to constructing the required model bathymetry, a longshore current generation system was fabricated and installed. Upon completion of the construction process, a verification of model accuracy was carried out to ensure the quality model erection. A list of vendors used throughout the duration of the project is supplied in Appendix A.

4.1 Bathymetry Construction

Prior to the construction of the model bathymetry, a survey origin was established at the northwest corner of the TWB, as can be seen in Figure 7. Establishment of this control point provided a consistent method of lateral referencing of position within the basin. All succeeding surveys utilized this point as the horizontal origin (0,0). The basin floor was chosen as the vertical 0-level. After determining this point of reference for the model, a concrete cinder block retaining wall was placed in the basin to create an intake channel for the pumping system. It was also used to separate the existing aggregate beach from the model construction area. Along the intake channel, the wall was built four blocks in height, 0.8 m (2.6 ft), and one block in width, 0.4 m (1.3 ft). The aggregate retaining wall was built to three blocks in height, 0.6 m (1.9 ft) and one block in width. The difference in cinder block wall height allowed for realistic land topography to be built along the intake channel. Figure 6 shows the installed return channel retaining wall.

Figure 6. Return channel retaining wall.

To provide the greatest accuracy in the construction of the model, a series of forms were needed to establish control points along the bathymetry. These control points were determined by dividing the bathymetry in the alongshore direction into a series of transects. Transects were oriented shore-normal and spaced on 2 m (6.6 ft) increments along the beachface. Transect spacing width was chosen to allow access of heavy equipment during construction. The transects were drawn in an AutoCAD file and the intersection points of the transect lines and bathymetry contours were recorded and plotted for each transect. Figure 7 shows the layout of construction transects on the model.

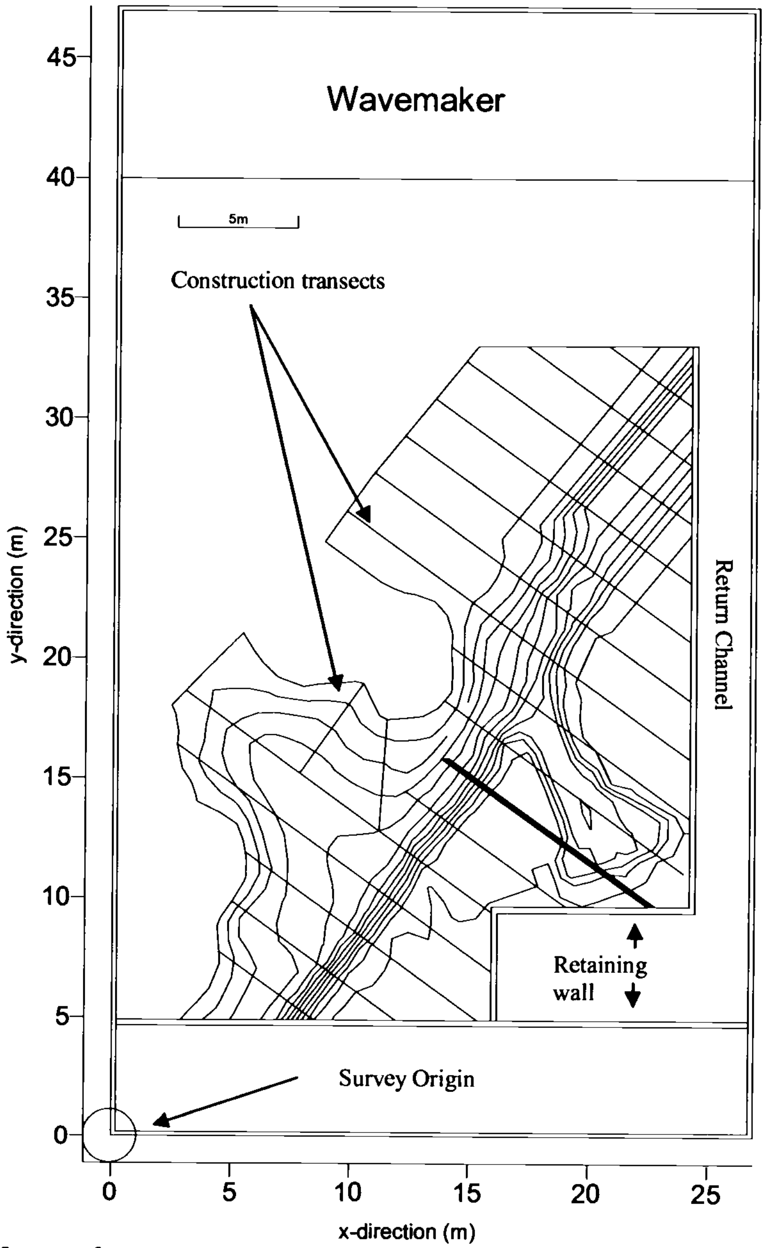

Figure 7. Layout of construction transect forms. Bold indicates Transect 7 plotted in Figure 8.

The profile of each transect was recorded and plotted using the contour plot with transect form overlays (Figure 7). These profiles were used as design blueprints for the construction of each transect form. Figure 8 depicts a schematic of a specific transect form (bold in Figure 7). This transect, located near Keystone Harbor, shows the elevation change through the cross-section of the harbor. The water level would be at -0.07 m in this figure.

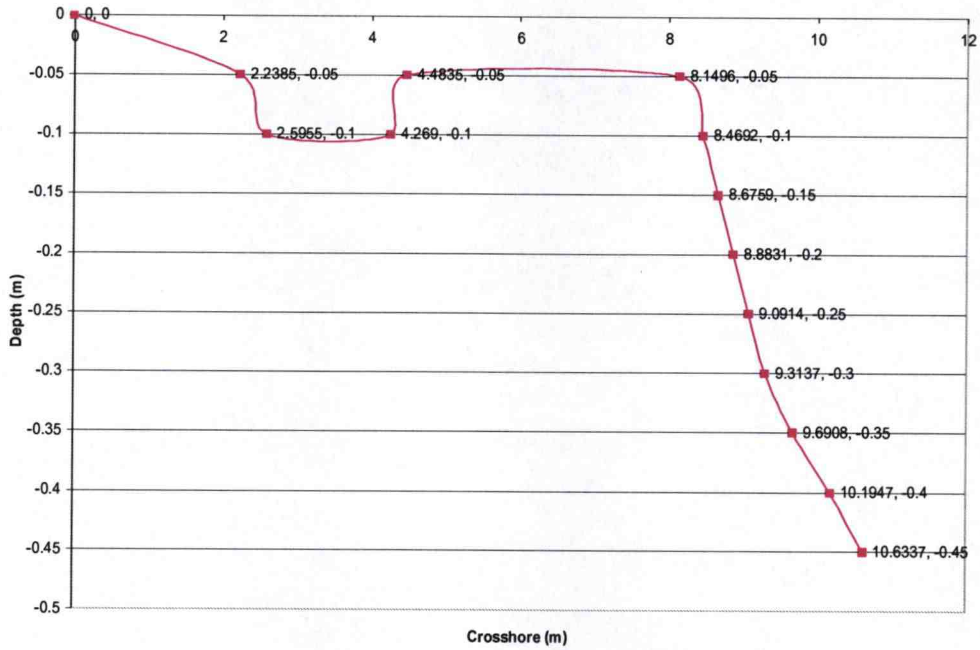

Figure 8. Profile view of Transect 7.

The design of the transect forms with 2 m (6.6 ft) spacing allowed construction equipment to be driven between adjacent forms for the placement of filler sand. Marine grade plywood was cut to the shape of each transect and then surveyed into its appropriate position using a Sokkia, Series 30 R total station. Figure 9 shows the placement of a transect during initial construction. Pictured in this figure are Erin Lucas and James Galloway both of whom participated in the construction of the Keystone model as part of a National Science Foundation (NSF) sponsored Research Experience for Undergraduates (REU). Also pictured is Shingo Ichikawa, a first year graduate student in the Ocean Engineering program.

Figure 9. Placement of a plywood transect form. Pictured from left to right: Erin Lucas, Shingo Ichikawa, and James Galloway.

For stability of the form system, 2 m long, 2" x 4" studs were attached between transects. Transects were then anchored to uni-strut in the basin floor through the use of angled iron brackets. This anchoring provided a solid base for the form system and alleviated shifting of the model structure during sand placement.

After erection of the transect forms, 200 tons of construction grade sand were placed between transects. Sand was placed using a Bobcat 763 skid steer equipped with a 0.6 m^3 (0.8 yd^3) loading bucket. Additionally, the sand was shoveled and raked into place, then compacted using a jumping jack and plate compactor. For final preparation, the sand was screeded and leveled between transects by dragging a 2" x 4" board between transects. Screeding resulted in a 3.8 cm (1.5 in) space between the top level of sand and the top of each transect form. This gap allowed for placement of a concrete cap which would be flush with the top of each form. The area within the harbor was not contoured by forms because it did not directly affect the project area of interest.

The final stage of construction required the placement of the 3.8 cm (1.5 in) concrete cap on top of the existing sand bathymetry. The addition of a concrete layer would maintain the correct elevation and bathymetry of the model while protecting it from erosion by waves, currents, and foot traffic. Double Eagle Construction, a local concrete contractor, was employed to consult and assist the WRL on the placement and finishing of concrete. A concrete pumping truck was used for the placement of 12 m^3 (16 yd^3) of the wet concrete and finishing was undertaken by project staff. Pumping, screeding, and finishing of the concrete took four hours with the help of two experienced finishers, and nine other less experienced workers. A 1 m (3.3 ft) bull float with a 5 m (16.4 ft) long handle was used to create a smooth finish between each transect. Figure 10 shows the finishing of concrete after placement.

Figure 10. Concrete finishing using a 1 m bull float.

The concrete was allowed to cure for a period of 72 hours before additional finishing was undertaken. After this period, rough edges of the concrete were ground down using an 18 cm (7 in) diameter wheel grinder, and the model and basin were pressure washed to prepare for final grout application. A thin set grout was applied to all of the cracks and crevices present on the model. This created smooth transitions along the model surface and minimized any obstructions that might cause adverse effects to the water flow. Figure 11 shows the completed model bathymetry after the application of the concrete cap.

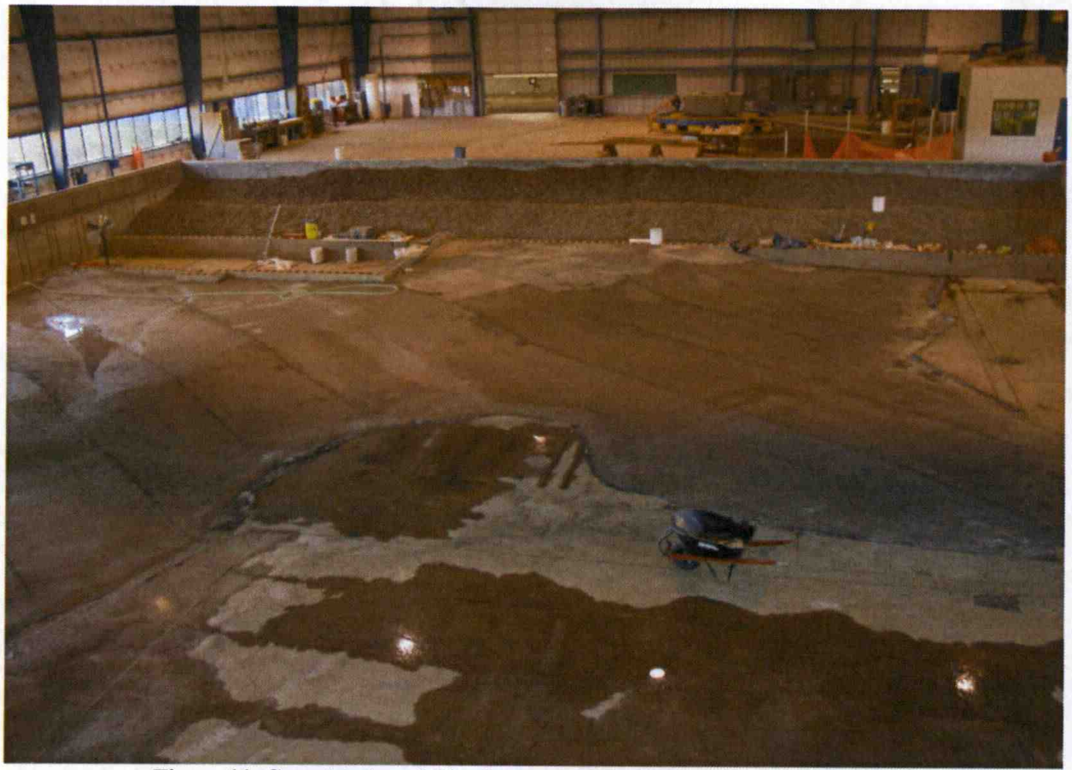

Figure 11. Completed model bathymetry after application of concrete cap.

4.2 Pump and Discharge Manifold Installation

Initial design for the current generating system required a volumetric flow of 12,700 GPM. This flowrate could be achieved by attaching a series of three submersible pumps rated at 5,000 GPM apiece, to an intake manifold. Figure 12 shows a schematic of the pump and discharge manifold system designed for generation of currents. The orientation of the pump and manifold system relative to the basin is depicted Figure 16, below.

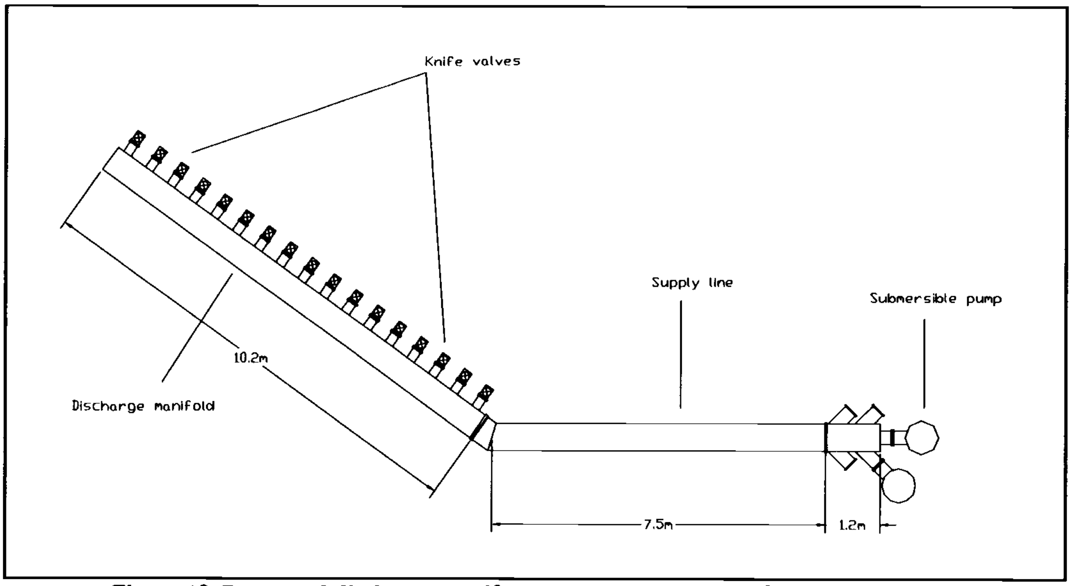

Figure 12. Pump and discharge manifold setup for generation of currents in TWB.

The supply line and discharge manifold were constructed of 0.6 m (24 in) diameter high density polyethylene (HDPE) pipe. The main lines of the discharge apparatus were built by Ferguson Supply Inc., utilizing plastic welding technology to construct the custom built manifold. The supply line was built to a length of 7.5 m (24.6 ft) and fitted with flanged adapters at each end. The discharge manifold was capped on one end and fitted with a flanged adapter on the other. The discharge manifold also consisted of seventeen, 20 cm (8 in) diameter knife valves mounted normally to the main water line. Flanged 22° elbow extensions were fastened to each knife valve, directing water flow tangent to the model surface (Figure 12).

At the closed end of the discharge manifold, a 1.2 m x 2.4 m (4 ft x 8 ft) sheet of galvanized sheet metal was placed under the manifold setup to provide a smooth flow transition from the basin floor to model contours. The sheet metal minimized adverse flow conditions at the bathymetry contour/basin floor seam and prevented scour at the interface.

Figure 13. Current discharge manifold and knife valves.

To prevent movement of the discharge manifold during testing, it was secured to the model bathymetry. The anchoring system consisted of two, 6 mm (0.25 in.) angled iron brackets connected to the bathymetry with concrete anchors. Cable stirrup straps were wrapped around the manifold and joined to the angled brackets with turn-buckles. This design allowed tightening of the retention system and prevented the manifold from torquing due to water discharge.

4.3 Jetty Construction

A footprint of each jetty alternative was surveyed onto the model topography and marked off with waterproof crayon for reference during jetty placement. All riprap jetties were constructed of d_{50} = 54.5 mm angular basalt aggregate. All jetty dimensions were constructed in accordance with plans provided by CHE. Berm width and length, as well as, jetty slopes were verified after placement of each jetty alternative. Details of length dimensions for prototype and modeled jetty alternatives can be viewed in Appendix B. Figure 14 shows the design of proposed jetty Alternative 1A. Additional proposed jetty designs can be viewed in Section 6.1.

Figure 14. Proposed jetty design Alternative 1A. Courtesy of CHE.

Alternative 4A, the pile supported wave barrier, was constructed of fifteen, 2.2 cm (0.8 in) O.D. copper pipe piles. These piles were placed at 30 cm (1 ft) intervals along the wave barrier centerline. The wave barrier cross-section extended to 2/3 the water depth and was constructed from two sheets of marine grade plywood. Guy wires were used at three positions along the wave barrier to add rigidity to the structure. Total width of the wave barrier was 6.0 cm (2.4 in).

4.4 Visual Scale Development

To enhance the connection of the scale model with the real world site, a number of minor characteristics were affixed to the model. A scale model ferry was built from foam insulation board and placed in the model harbor (Figure 15a). Using an aerial photograph of Keystone Harbor, several roads, a parking lot, and campground (Figure 15b) were spray painted onto the model to add a sense of scale. These recognizable objects aided laboratory visitors visualizing model scale and its relation to the real world site.


(a)


(b)

Figure 15. Visual scale development. (a) Model sized ferry in Keystone Harbor,WA. (b) Model of Fort Casey Campground.

4.5 Verification of Model Construction

To ensure the quality and accuracy of the model construction, a verification survey of the model bathymetry was performed. Epic Scan Ltd., an Oregon based survey firm specializing in 3-D data collection, was subcontracted to carry out the survey. Utilizing a CYRAX 2400 laser scanner equipped with LIDAR (LIght Detection And Ranging) technology, Epic Scan surveyed the constructed model. The survey was conducted over a period of four hours and resulted in data collection of more than 4 million data points. Figure 16 shows a dot matrix plot of the data collected during the LIDAR survey. It is important to note that this plot is comprised of data points and is not a photographic image.

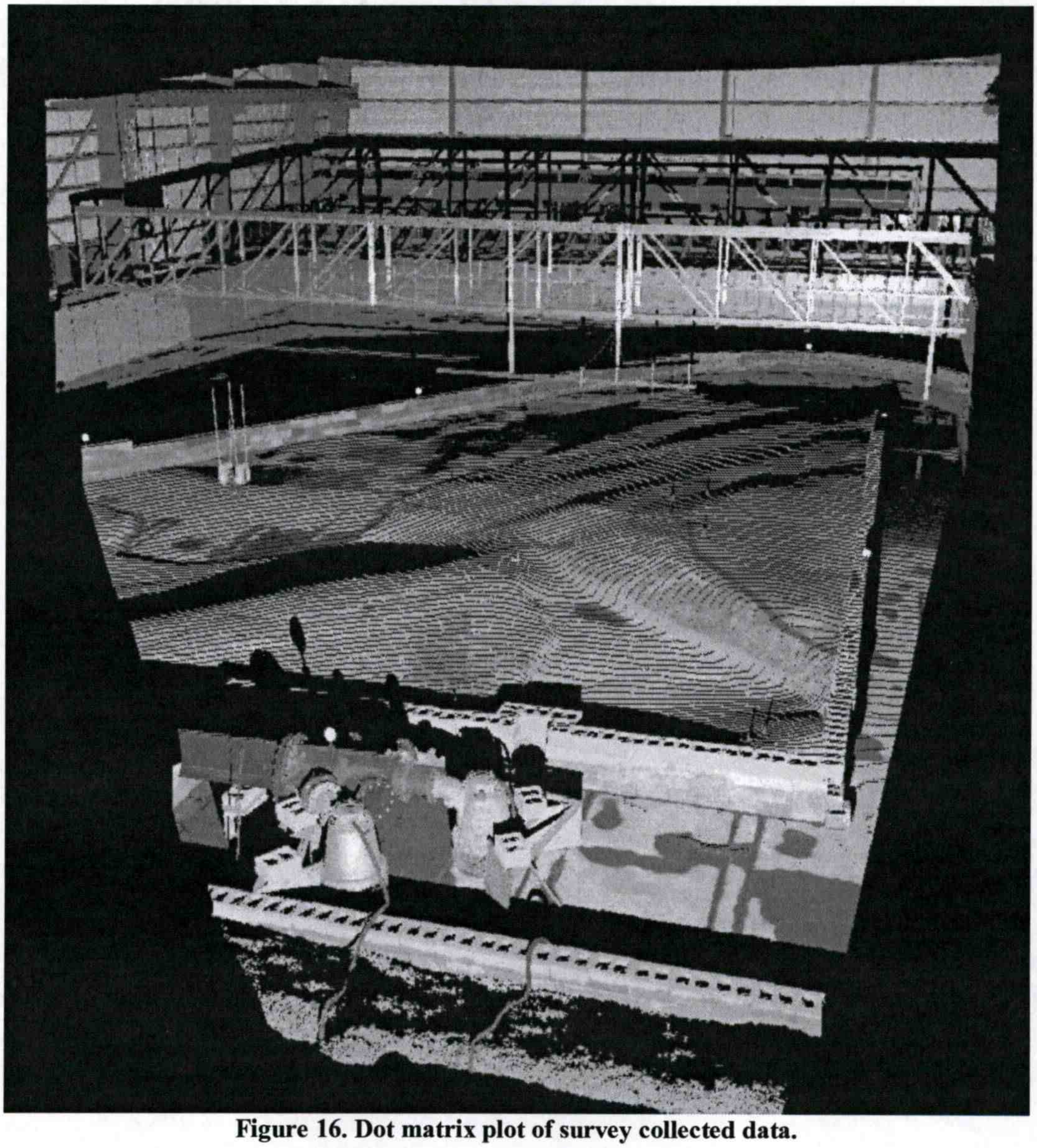

Figure 16. Dot matrix plot of survey collected data.

Figure 17 shows the comparison of model designed and model constructed. The survey results confirmed that the model was built to an acceptable level of accuracy for testing. Average deviations over the compared model surfaces ranged from - 4.8 cm to 3.9 cm. One section that had a large deviation from model design, represented by **A** in Figure 17, was directly below jetty footprints and was not corrected because it would not have affected the experiment. A large deviation area can be seen at the downstream end of the model, denoted **B**. This deviation area is present because provided bathymetric survey data was incomplete at the downstream end of the model. Linear extrapolation of bathymetric survey endpoints was carried out in order to provide a smooth transition to the return channel.

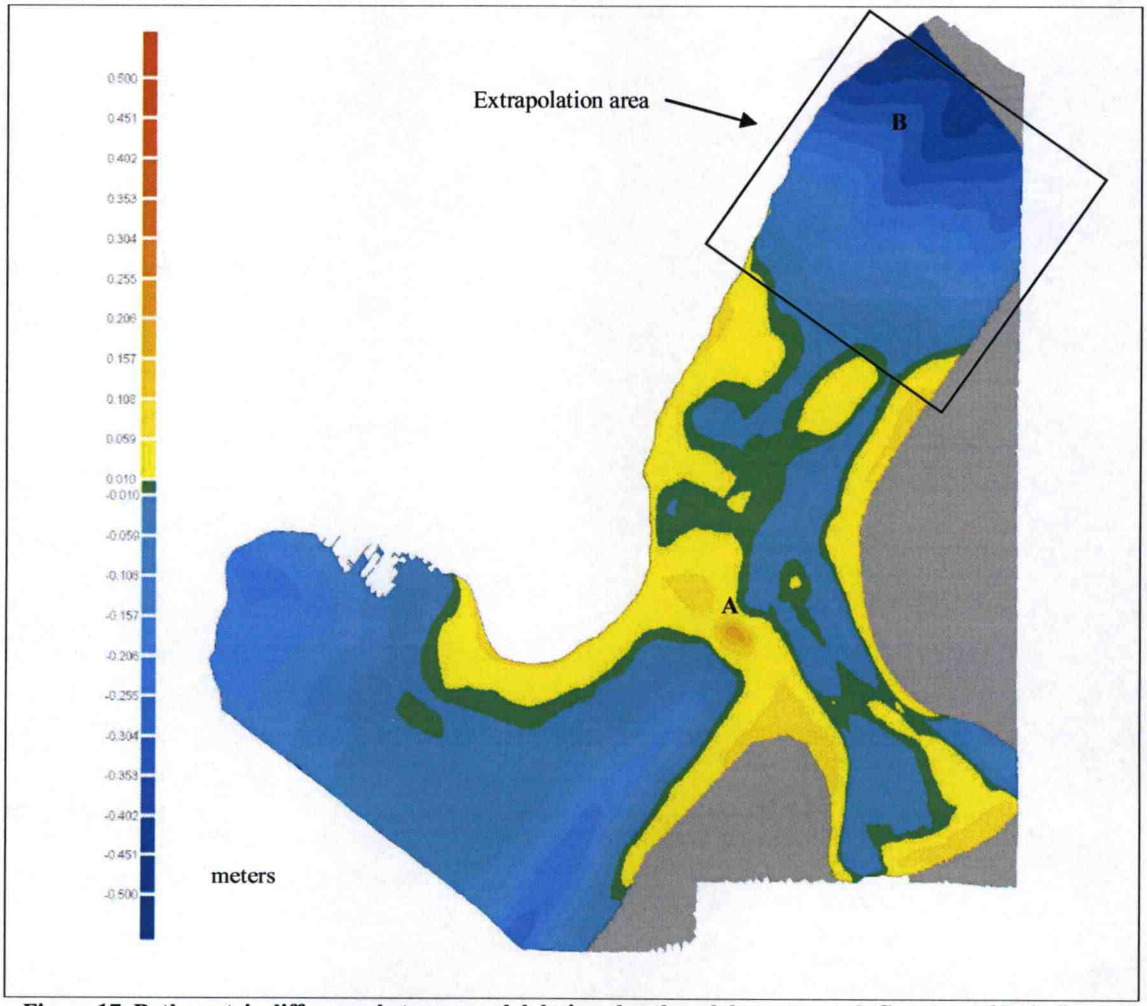

Figure 17. Bathymetric difference between model designed and model constructed. Courtesy of Epic Scan.

Upon completion of the model construction verification, testing of each proposed jetty design alternative began. Figure 18 gives a schematic of the as-built model bathymetry and current generation setup. This figure was created using the survey data collected by Epic Scan during the verification process.

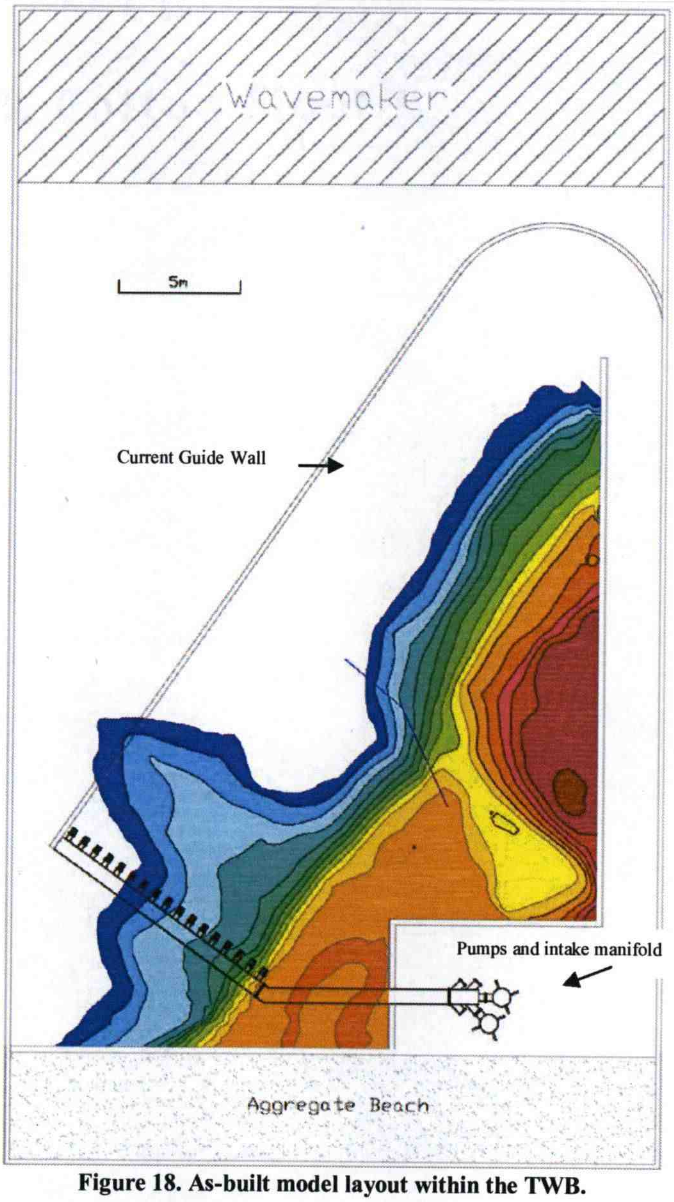

Figure 18. As-built model layout within the TWB.

5 Experimental Setup and Data Collection

The following section discusses the experimental setup and data collection methodology used for the physical model testing. Data were collected in two distinct phases. The first stage of testing resolved longshore current velocity vector fields, while the second determined wave transformation and reflection patterns due to each jetty alternative. The current guide wall was removed prior to wave testing. Table 2 shows the timeline during which alternative jetty testing occurred. A more detailed timeline is included in Appendix C. Current data collection was a quantitative effort. Wave field data was collected quantitatively, however, was used only for qualitative analysis. For both current and wave testing, a water depth of 55 cm was used in the basin, corresponding with Mean Lower Low Water (MLLW) at the project site.

Table 2. Testing timeline for alternative jetty testing.

Jetty Alternative	Description (prototype dimensions)	Current Testing Date	Wave Testing Date
Null Case	No jetty structure	10-08-04	N/A
Existing	Existing jetty structure	10-10-04	10-20-04
1A	600 ft jetty extension, single alignment	10-11-04	10-19-04
3A	600 ft jetty extension, two alignments, emerging dog-leg	10-13-04	10-14-04
3B	600 ft jetty extension, two alignments, submerged dog-leg	10-12-04	10-19-04
4A	600 ft jetty extension, two alignments, wave barrier	10-24-04	10-22-04

5.1 Longshore Current Generation and Measurement

Currents were generated with two BIBO BS-2250 submersible dewatering pumps, rated at 5000 GPM each (Figure 19). It should be noted that initial design calculations called for a total of three pumps; however, preliminary testing revealed a need for only two pumps to create the desired currents. Total volumetric flow for longshore current testing of proposed jetty design alternatives was 10,000 GPM.

Figure 19. BIBO BS-2250 submersible pump used for current generation.

During initial testing, radial flow skirts were placed around each pump to minimize vortexing and water drawdown at the pump intake.

5.1.1 Establishment of Testing Time Interval

During initial calibration, longshore current measurements were collected using a single Sontek, 3-D Acoustic Doppler Velocimeter (ADV). The ADV sampled at 1 Hz, allowing an accurate representation of the current velocities while filtering out high frequency noise.

In order to establish the spin-up time necessary for longshore currents to reach steady state conditions, some initial testing was required. This testing involved the sampling of return channel fluid flow for a period of two hours. The extended testing period was used to determine if any low frequency seiches occurred as a result of the water flow. Data collection began prior to pump startup and continued for the remaining two hours of the test period. Analysis of the data showed no signs of seiching and that pump steady state was reached after approximately 2 minutes of operation. For the remainder of longshore current testing, pumps were operated for a minimum of 5 minutes before data collection took place.

During initial calibration, a determination test was conducted to establish the length of data collection time necessary for accurate representation of current velocity at each location. This process was carried out by collecting longshore current data for a period of 30 minutes and time averaging data over five different intervals. Figure 20 shows a plot of the velocity time record and the different time averaging intervals.

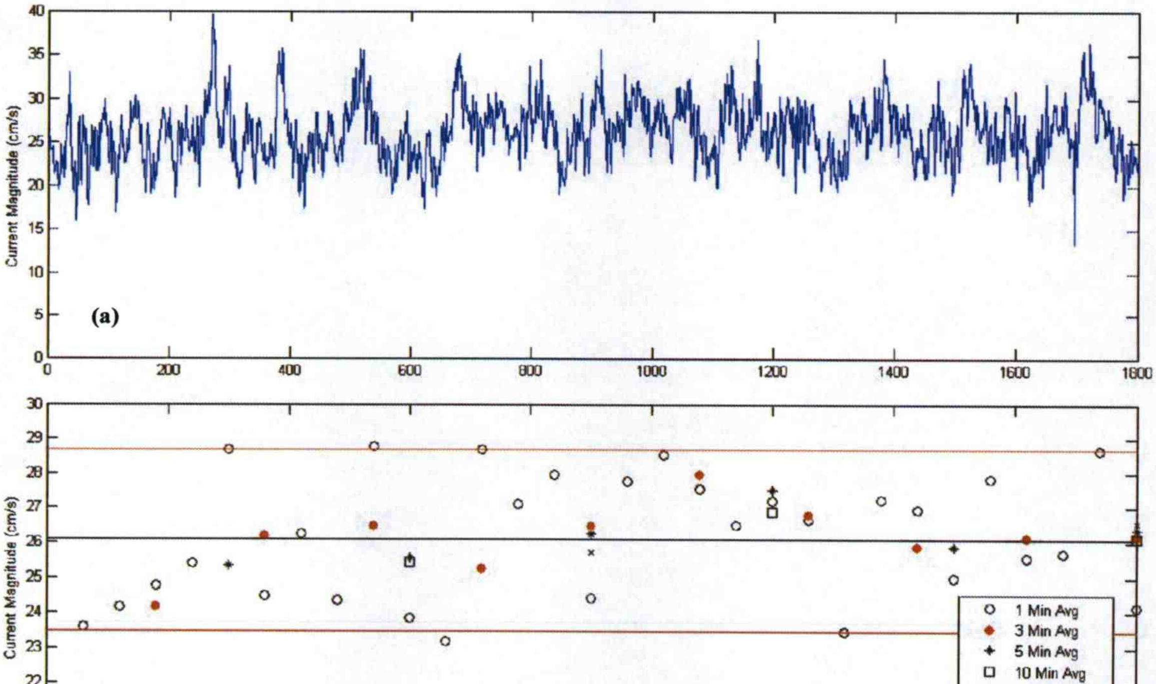

Figure 20. Thirty minute time record of current magnitude data and associated time averaging intervals. (a) raw signal and (b) averaged over 5 time intervals.

Analysis showed that time averaging current data for 3 minute intervals provided values within ± 10% of the mean value. Using a 3-minute time average resulted in a decrease of overall testing time required, yet results would still fall within 10% of the mean in comparison to 30-minute time averaging. The remainder of longshore current testing used a 3-minute time averaging interval for data collection.

5.1.2 Calibration of Longshore Currents

Longshore current calibration of the model was undertaken with no jetty in place. This is referred to as the null case condition. An iterative process of adjusting knife valves and obtaining current velocity measurements was used to determine the discharge manifold settings for the testing of all proposed jetty designs. Much of the initial adjustments were based on visual observation of the current flow and attempting to rid the flow of observable recirculation or eddy formations. The extent to which knife valves were opened was limited in order to force a pressurized system within the pump setup. To ensure the operating system was pressurized, a pressure gage was installed on the supply line. The pressurized system, which operated at 3 psi, allowed for added control of water flow. When adjustments were completed, a final current velocity measurement data set was collected (see Section 6.1). This measurement set utilized the same testing grid used in the testing of all jetty alternatives. Details on the testing setup are given in Section 5.1.3

5.1.3 Measurement of Longshore Currents

An array of four ADV's was used in the measurement of longshore current velocities. Sampling rate was set at 1 Hz and data collection occurred for 3-minute intervals at each data location, as described above. The ADV's were oriented with X-axis positive onshore (parallel to the TWB sidewalls), Z-axis positive up, and positive Y-axis corresponding to a right handed coordinate system.

For the longshore current testing phase, an instrument frame was constructed of wide flange aluminum I-beams. The frame was attached to a movable bridge which spans the entire width of the TWB. This setup would allow simplified data collection in both the cross-shore and long-shore directions. Instruments were deployed off the frames, lower 8 m section of I-beam. This I-beam was oriented parallel to the movable bridges spanning width. Figure 21 shows the instrument frame as mounted on the movable bridge.

Figure 21. Instrument deployment frame mounted to TWB movable bridge.

A data collection grid was created to accurately capture the velocity vector field near the channel harbor entrance (Figure 22). Data collection points were spaced on 1 m (3.3 ft) intervals in both the cross-shore (designated y) and long-shore (designated x) directions. This setup allowed a representative cross section of the velocity field to be sampled for analysis. Each point on the grid represents a point of data collection.

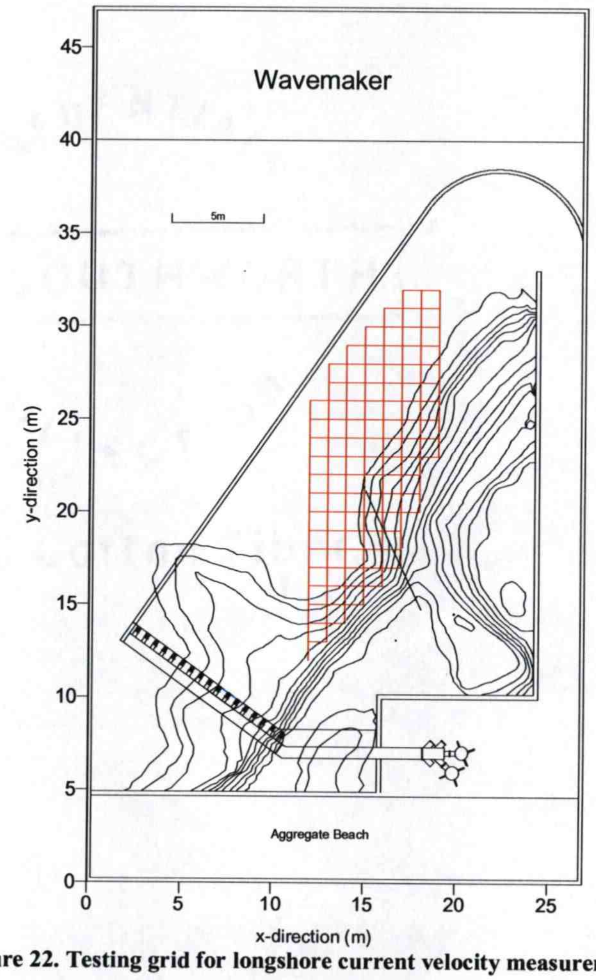

Figure 22. Testing grid for longshore current velocity measurement.

5.1.4 Longshore Current Testing Procedure

During the testing of jetty alternatives, each ADV was individually mounted to the testing frame described above. ADV probe tips were submerged 10 cm (3.9 in) below still water level (SWL), resulting in a sampling volume located at 15 cm (5.9 in) below SWL. Data collection began with a single ADV at the furthest onshore location, y-location: 12 m, and proceeded to move offshore. Additional ADV's were added to complete collection grid and removed as obstructions were encountered. Upon completion of the first sweep of data collection (x-locations: 12-14 m, y-locations: 12-30 m), the ADV's were relocated to x-locations: 15-18 m. For the second sweep of data collection, advancement of the collection array was onshore, from y-location: 32 m to y-location: 16 m. Again, ADV's were removed to avoid interference with any obstacles and added for grid completion. Figure 23 shows the sequence in which current velocity testing occurred.

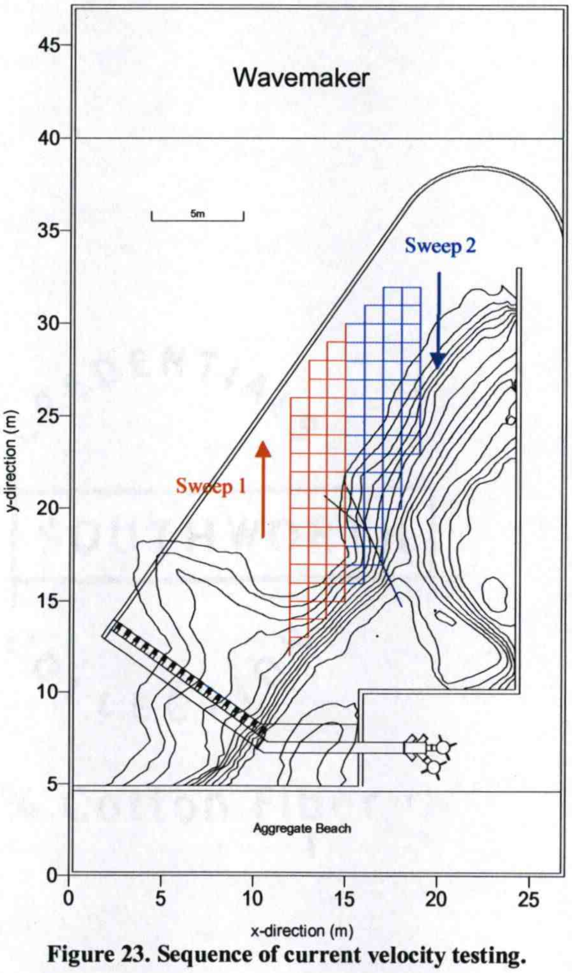

Figure 23. Sequence of current velocity testing.

5.2 Wave Generation and Measurement

Waves were generated using a fully programmable multi-panel, piston-type wavemaker. The wavemaker consists of 29 independent wave boards, driven by 30 electrical actuators. MTS wavemaker control software allowed user input of specified wave conditions to be created. Both monochromatic (regular) and random wave fields were produced by the wavemaker. Random wave spectra were created using GEDAP software. This software used a TMA spectral shape consisting of directional waves and a spectral spreading factor of $\gamma = 3.3$.

Wave testing of each jetty alternative involved data collection of both monochromatic and random wave fields. Table 3 gives details on the characteristics of the target wave for each testing case.

Table 3. Characteristics of 6 wave cases used in wave testing sequence.

Case #	Prototype θ,H,T	Wave Type	Model θ,H,T
1	195°, 0.9 m, 4 s	Regular	0°, 2.3 cm ,0.63 s
2	210°, 1.2 m, 4 s	Regular	-15°, 2.3 cm ,0.63 s
3	195°, 0.9 m, 5 s	Regular	0°, 3.0 cm, 0.79 s
4	210°, 1.2 m, 5 s	Regular	-15°, 3.0 cm, 0.79 s
5	195°, 0.9 m, 4 s	Random	0°, 2.3 cm, 0.63 s
6	210°, 1.2 m, 5 s	Random	0°, 3.0 cm, 0.79 s

5.2.1 Establishment of Testing Time Interval

Collection of wave data required recording 20 minute time records for each of the six wave cases. To obtain an accurate representation of the wave field, data collection was necessary at a number of y-locations along the bathymetry. The duration of wave recoding at each y-location would differ for regular and random waves. For regular waves, data collection would occur for 1-minute at each location.

The random wave time interval was decided based on the recommendations of Goda (2000), for capturing accurate representations of random wave data. It was suggested that at least 200 waves be recorded at each y-location to provide enough information for reconstruction the wave spectrum. This would require a 3-minute recording time interval.

5.2.2 Calibration of Incident Waves

Prior to wave testing, the current guide wall was removed to prevent the impedance of the incident wave field. Additionally, a synthetic matting of wave absorbing material was installed along the discharge manifold and exposed basin wall. This material minimized wave reflection off of these objects during wave testing.

Initial wave height measurements were obtained using three surface piercing, resistance-type wave gages. Data collection was performed using a sampling rate of 50 Hz. Wave gages were mounted at 2 m (6.6 ft) intervals to the same instrument frame described in Section 5.1.3. Figure 24 shows the wave gage data collection array as mounted on the instrument frame.

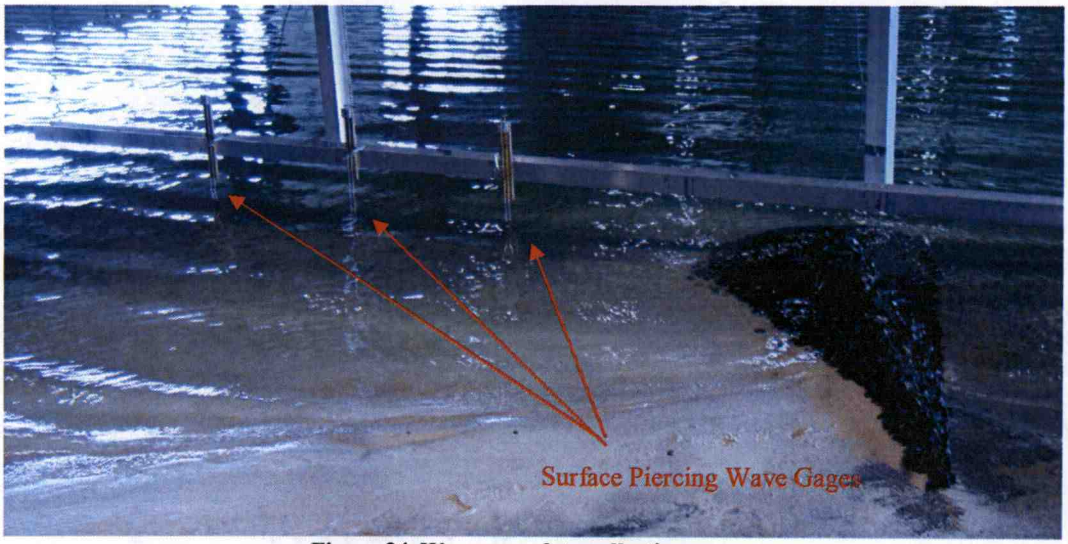

Figure 24. Wave gage data collection array.

The resistance type wave gages were calibrated prior to data collection using a linear calibration curve. This calibration curve related gage wire resistance to submerged depth over a range of 20 cm (7.9 in), in 2 cm (0.8 in) increments. Approximate error of wave gage calibration is sub-millimeter, resulting in wave measurement accuracy of ± 1 mm. Wave gages were recalibrated every three days during the wave testing period.

To calibrate the incident wave field for each wave case, preliminary tests were run to ensure wavemaker output agreed with target values. To verify the output of the wavemaker, 3-minute time series were collected at y-location: 19 m. At this location, deep water conditions existed for all three of the mounted wave gages. Data collection of the free surface elevation began prior to wave propagation, ensuring collection of SWL and the incident wave field. Analysis was undertaken using a zero up-crossing routine to extract significant wave height (H_s) values from each wave gage record. Incident wave conditions were satisfied when the average of the three computed H_s values were within ± 10 % of the target value. This method of verification was undertaken for each of the six wave cases, described in Table 3.

5.2.3 Measurement of Waves

Wave data collection for the six wave cases was undertaken in a similar manor as the calibration of the incident wave field. Three wave gages were mounted to the instrument frame and sampling occurred at 50 Hz. Duration of wave record at each y-location varied for monochromatic and random waves. Regular wave data was collected for 1 minute intervals, while random wave data was collected for 3 minute intervals. To keep the total time duration of data collection constant at 20 minutes/run, the measuring grid density for regular and random waves differed. Figure 25 shows the regular wave testing grid, while Figure 26 shows a schematic of the random wave testing grid. Grid nodes represent the location of each wave gage during the testing sequence.

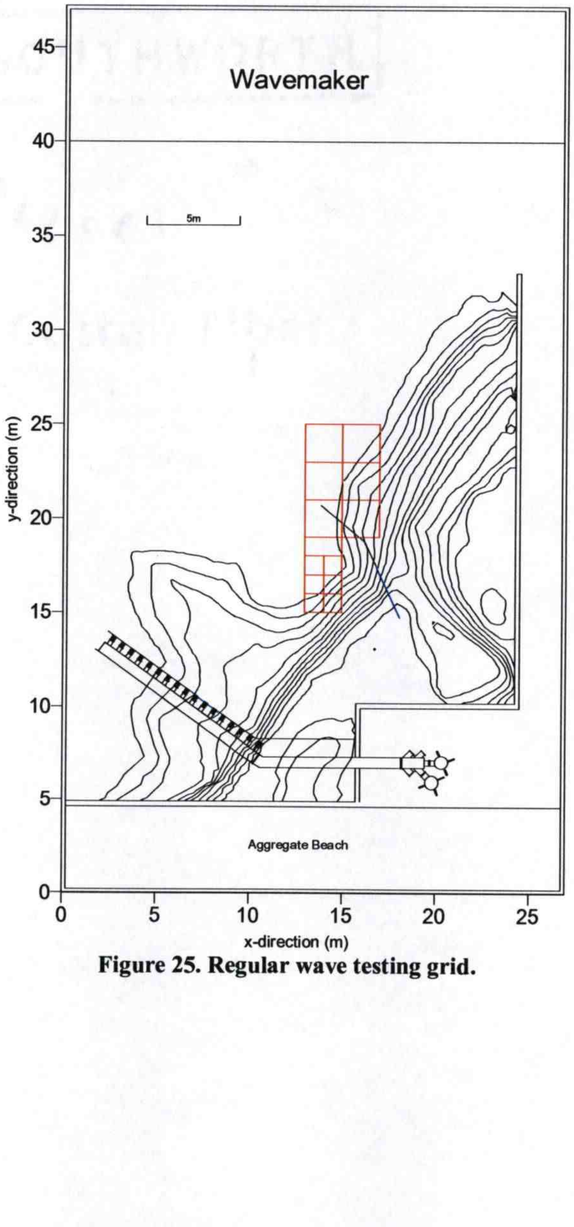

Figure 25. Regular wave testing grid.

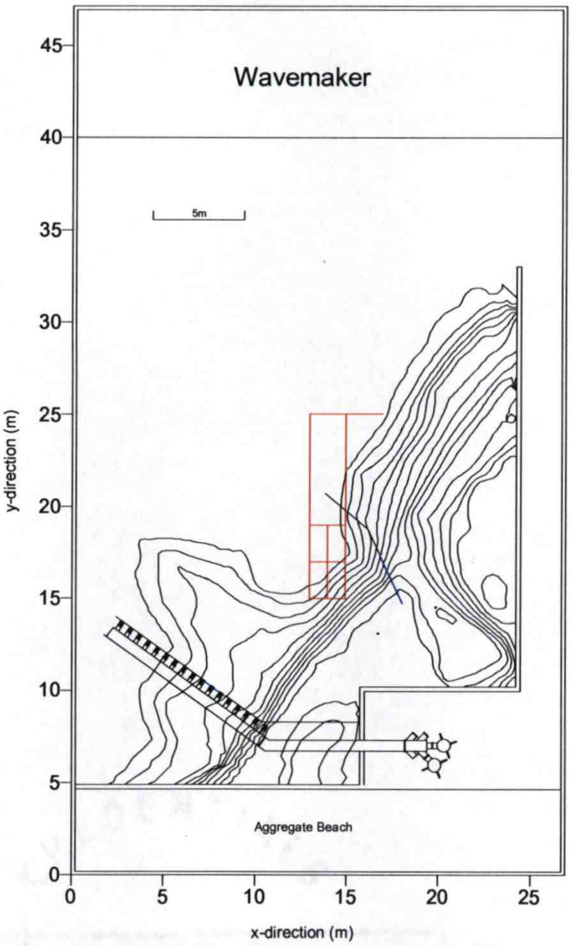

Figure 26. Random wave testing grid.

5.2.4 Wave Testing Procedure

During wave testing, independent wave cases consisting of regular and random wave trains were run at each jetty alternative. Testing was used to measure wave transformation due to realistic bathymetric elevation changes. Data collection began prior to wave generation, allowing the incident wave field to be captured at the start of each run. Wave data were collected by advancing the movable bridge from its furthest offshore position towards the shoreline. Data collection offshore of the jetty extensions was conducted with wave gages in x-locations: 13, 15, and 17 m. Inside the jetty extensions, wave gages were attached at 1 m (3.3 ft) intervals and positioned at x-locations: 13, 14, and 15 m. In addition to wave height measurements, testing allowed qualitative assessment of shoreline changes adjacent to jetty alternatives and the observation of harbor channel diffraction patterns.

5.2.5 Wave Induced Shoreline Change

For the purpose of shoreline change analysis, a 5 cm (2 in) veneer of scaled cobble and sand material was placed adjacent to the jetty toe. This thin veneer of material stretched along the beach face approximately 4 m from the jetty and extended seaward to a water depth of 45 cm. The material was comprised of two sizes of silica sand mixed in a 1:1 ratio. It is important to note that the d_{50} grain sizes of sediments were scaled so that the fall velocity of prototype sediments matched the scaled fall velocity of the model sediments. Because of inherent difficulties in accurately modeling sediment transport with Froude similitude, sediment sizes were not scaled via the length scale. Sediment was scaled using Dean number similitude. The Dean number, H_w, is given by

Eq. 4
$$H_w = \frac{H}{w_f T}$$

where H is wave height, w_f is fall velocity, and T is wave period. Using the method outlined in the Coastal Engineering Manual (2002), fall velocity, w_f, is given by

Eq. 5
$$w_f = \left(\frac{5.4}{S^* + 0.9}\right)^{-1} \sqrt{\left(\frac{\rho_s}{\rho_w} - 1\right) g\, d_{50}}$$

where ρ_s and ρ_w are density of sediment and water, respectively, g is gravitational accerleration due to gravity, d_{50} is nominal diameter of sediment and S^* is the sediment fluid parameter given by

Eq. 6

$$S^{*} = \frac{d_{50}}{4\upsilon}\sqrt{\left(\frac{\rho_s}{\rho_w} - 1\right) g\, d_{50}}$$

where υ is the kinematic viscosity of water.

Additional information regarding sediment scaling can be found in Hughes (1993). Table 4 gives nominal dimensions of silica sand grains used for beach material. Supplemental sieve analysis was performed on the sediment material for the purpose of quality control, as requested by CHE.

Table 4. Nominal dimensions of silica sand used for beach material.

	Prototype		Model (used)		Model (Dean)	
Sediment Type	d_{50} (mm)	w_f (cm/s)	d_{50} (mm)	w_f (cm/s)	d_{50} (mm)	w_f (cm/s)
Sand	30.0	3.5	0.095	0.56	0.094	0.55
Cobble	0.3	76.0	1.05	12.00	1.04	11.90

Before each wave case was run, the sediment beach adjacent to the jetty was groomed to a smooth and uniform finish. Photographs were taken prior to testing, and at 1, 10, and 20 minute time intervals. Additionally, detailed notes on shoreline change observations were taken during testing. These notes included observations of diffraction patterns, sediment transport, and berm formations.

6 Experimental Results – Phase I

The following section discusses the analysis and presentation of experimental results of Phase I testing of the Keystone physical model. Phase I testing included both calibration and testing of the existing conditions and four alternative jetty designs (Table 3). The results of longshore current testing will be presented first, followed by a discussion of wave testing and shoreline change for each jetty design.

6.1 Longshore Current Results

6.1.1 Null Case Condition

The null case condition was used to calibrate a uniform vector field required for subsequent jetty alternative testing. This case was not a proposed alternative solution; however, its information was important for establishing the initial knife valve settings necessary to create the desired flow field. Neither wave testing nor shoreline change analysis was carried out on the null case condition.

Figure 27 shows a velocity vector field record for the null case condition. This data set was collected during the calibration stage of longshore current testing. The contour plot shows semi-uniform current magnitudes throughout the sample domain. Although the current was not completely uniform over the region, the flow field was within a representative margin to warrant continued testing. After obtaining the desired flow field, knife valve openings remained unchanged for the duration of longshore current testing.

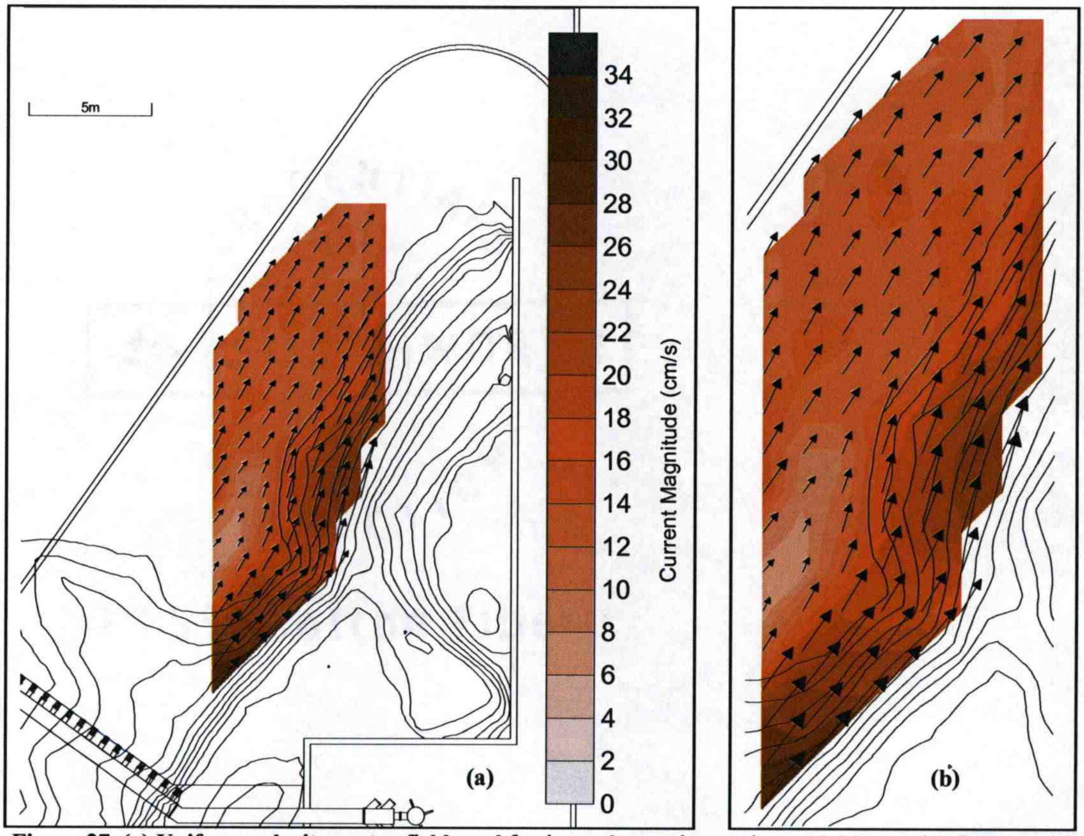

Figure 27. (a) Uniform velocity vector field used for jetty alternative testing and (b) velocity field detail.

6.1.2 Existing Condition

The existing jetty configuration was tested to provide a baseline flow field for comparison with proposed jetty alternative vector fields. The existing jetty extends 118 m (387 ft) above SWL from Keystone Harbor's western shore. The scaled model was constructed of basalt aggregate material with a crest length of 2.9 m (9.5 ft) above SWL. A comparison of existing model jetty and existing prototype jetty can be seen in photographs in Figure 28.

Figure 28. Comparison of existing jetty at the site and in the model.

Collection of longshore current data was performed on the modeled existing jetty conditions, resulting in the velocity vector field shown by Figure 29. Interestingly, construction of the existing jetty forced flow field that was more uniform than the null case and consistent with numerical modeling predictions.

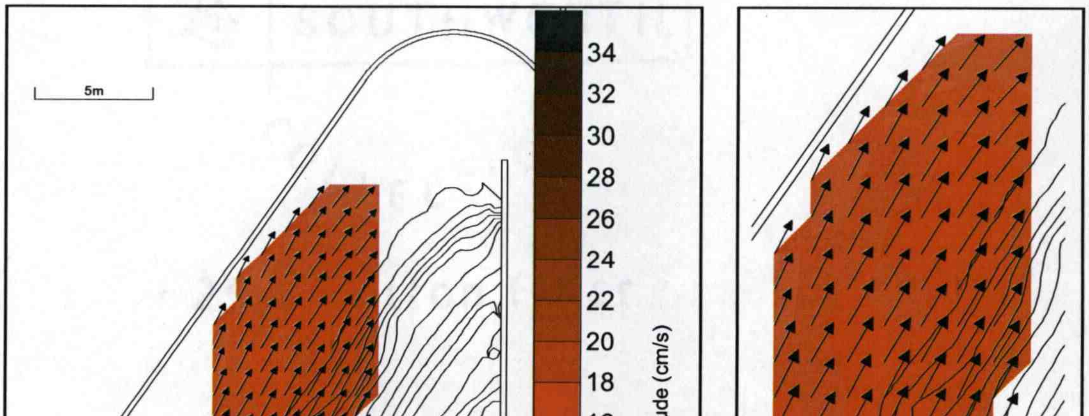

Figure 29. (a) Velocity vector field with existing jetty in place and (b) velocity field detail.

No areas of increased velocity were observed in the data collection region. Maximum recorded velocity for the existing condition was 23.8 cm/s. By comparing the velocity vector field plots of the remaining proposed jetty alternatives with the existing conditions, the performance of each jetty can be evaluated. Comparisons of jetty Alternative testing results are tabulated Section 6.1.7 (Table 5).

6.1.3 Alternative 1A

The first proposed jetty design to be tested was Alternative 1A. The design of this jetty called for a 183 m (600 ft, prototype length) extension to the existing jetty. The jetty extension remained in the same orientation as the original jetty, with a slope of 1.5H:1V. In addition, Alternative 1A remained a surface piercing jetty. Figure 30 gives a design overlay of jetty Alternative 1A at the study site. Figure 31 shows an aerial photograph of the modeled jetty for Alternative 1A, with a modeled length of 4.6 m (15 ft).

Figure 30. Design overlay of proposed jetty Alternative 1A. Courtesy of CHE.

Figure 31. Modeled jetty Alternative 1A.

Visual observation of the longshore currents seemed to show a significant decrease in velocity near the harbor channel entrance and in lee of the jetty. Analysis of the data shows the decrease in current velocities and the shadow region provided by the jetty extension (Figure 32).

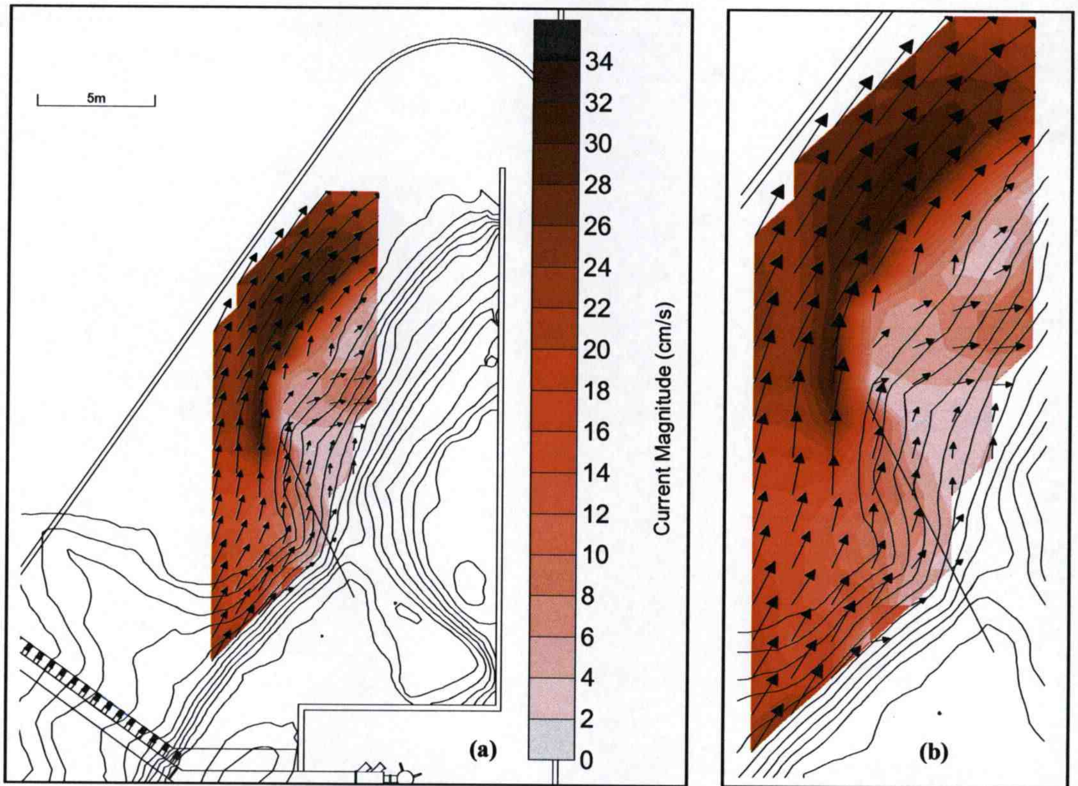

Figure 32. (a) Velocity vector field with jetty Alternative 1A and (b) velocity field detail.

In addition to the decrease in current velocities near the harbor channel entrance, large velocity increases were recorded off the tip of the jetty extension. The increased velocities are due to Venturi effects around the tip of the jetty.

6.1.4 Alternative 3A

The design of jetty Alternative 3A called for a surface piercing jetty extension with two alignments: The first alignment was an extension of 61 m (200 ft, prototype) along the existing jetty alignment. The second alignment was at an angle of 150°

azimuth, with an extension of approximately 122 m (400 ft, prototype). This design is commonly referred to as a dogleg. Jetty extensions were constructed of armor rock at a slope of 1.5H:1V. Figure 33 shows a design overlay of proposed jetty Alternative 3A. An aerial photograph of modeled jetty Alternative 3A can be seen in Figure 34.

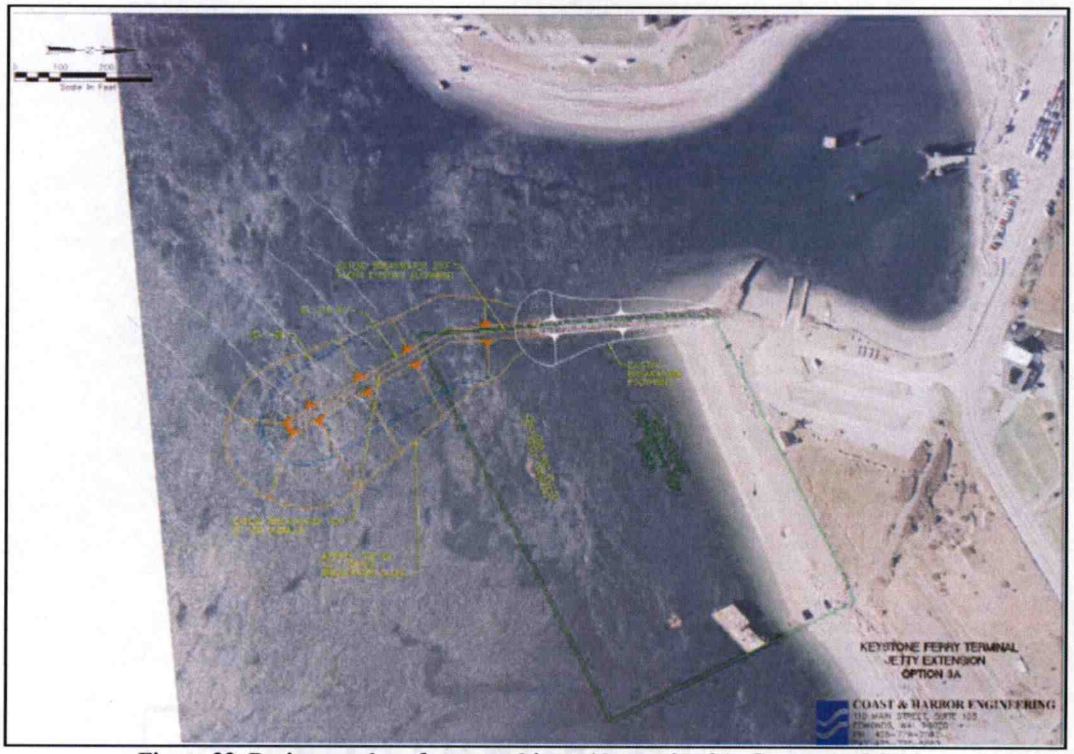

Figure 33. Design overlay of proposed jetty Alternative 3A. Courtesy of CHE.

Figure 34. Modeled jetty Alternative 3A.

Measured current velocities obtained from the testing of Alternative 3A are provided in Figure 35. Again, decreases in current velocities were obtained near the harbor channel entrance and in lee of the jetty.

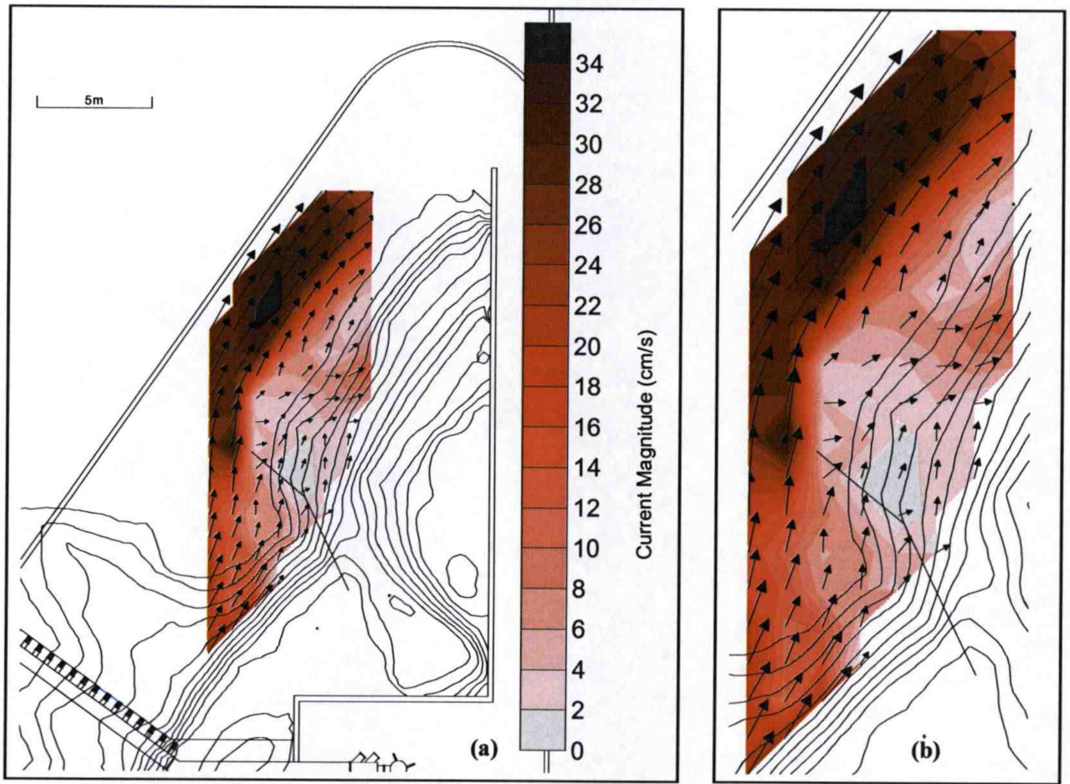

Figure 35. (a) Velocity vector field with jetty Alternative 3A and (b) velocity field detail.

In comparison with Alternative 1A, a larger area near the channel entrance was influenced by the proposed jetty. This larger influence area would provide more protection for ferries entering and exiting the terminal. However, increased velocities were observed off of the jetty extension tip. This accelerated flow could potentially be problematic for ferry operation because of the large velocity gradient. The maximum current velocity observed in the testing of Alternative 3A was approximately 35.2 cm/s, a 12% increase over the maximum of 31.4 cm/s observed in Alternative 1A.

6.1.5 Alternative 3B

The design of jetty Alternative 3B was similar to that of Alternative 3A. It called for extension lengths in the same dogleg orientation; however, the extension was purposed as a submerged jetty design. The submerged crest depth was 1.5 m (4.9 ft, prototype) below SWL. Jetty extensions were constructed of armor rock with a slope of 2H:1V, providing a larger footprint for the jetty design. A design overlay of proposed jetty Alternative 3B and photograph of modeled jetty layout can be seen in Figure 36 and Figure 37, respectively.

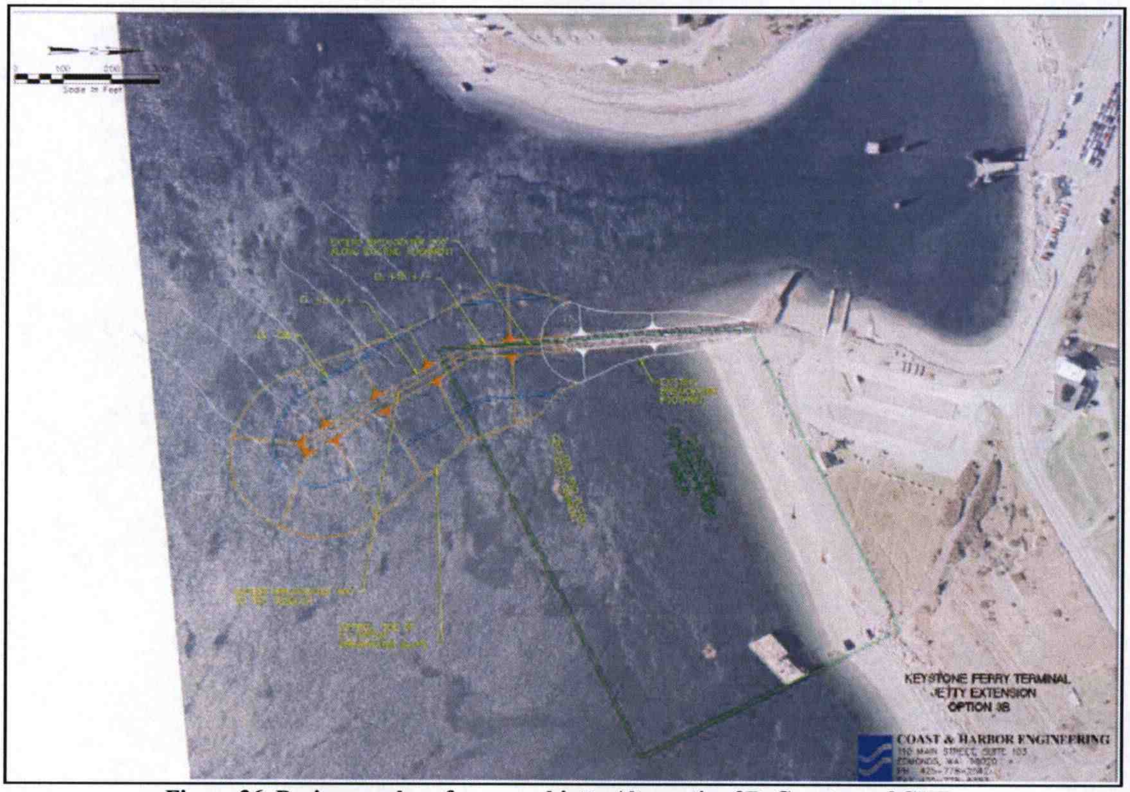

Figure 36. Design overlay of proposed jetty Alternative 3B. Courtesy of CHE.

Figure 37. Modeled jetty Alternative 3B.

The testing of Alternative 3B revealed that the jetty design provided the harbor channel entrance significant protection from longshore currents. In comparison with other proposed jetty designs, Alternative 3B, yielded the largest area of protection in lee of the jetty while minimizing acceleration around the jetty extension tip. Maximum current values recorded near the jetty tip were 31.5 cm/s, an increase of less than 8 cm/s compared to the existing condition. The influence of Alternative 3B provided the most beneficial changes to the velocity vector field. It increased shelter from longshore currents and minimized increases in velocity. Figure 38 shows the collected data for Alternative 3B longshore current testing.

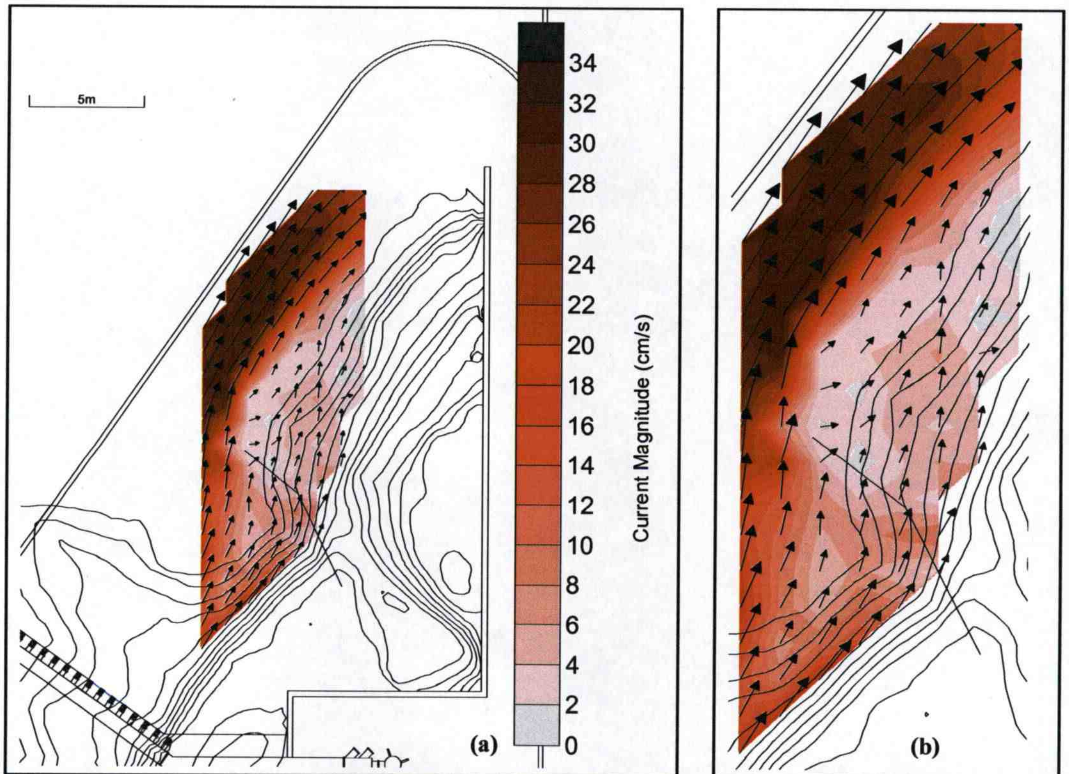

Figure 38. (a) Velocity vector field with jetty Alternative 3B and (b) velocity field detail.

6.1.6 Alternative 4A

The final jetty design tested was Alternative 4A. This proposed jetty design included two jetty orientations, similar to Alternatives 3A and 3B. However, this jetty did not include typical rip-rap armor units. Instead, Alternative 4A was built as a wave barrier extending to approximately 2/3 the depth of the water column. The prototype barrier would be constructed of steel pilings and concrete dividers. The model jetty was built as discussed in Section 4.3. Figure 39 gives design overlay of proposed jetty Alternative 4A. A picture of the modeled jetty Alternative 4A can be seen in Figure 40.

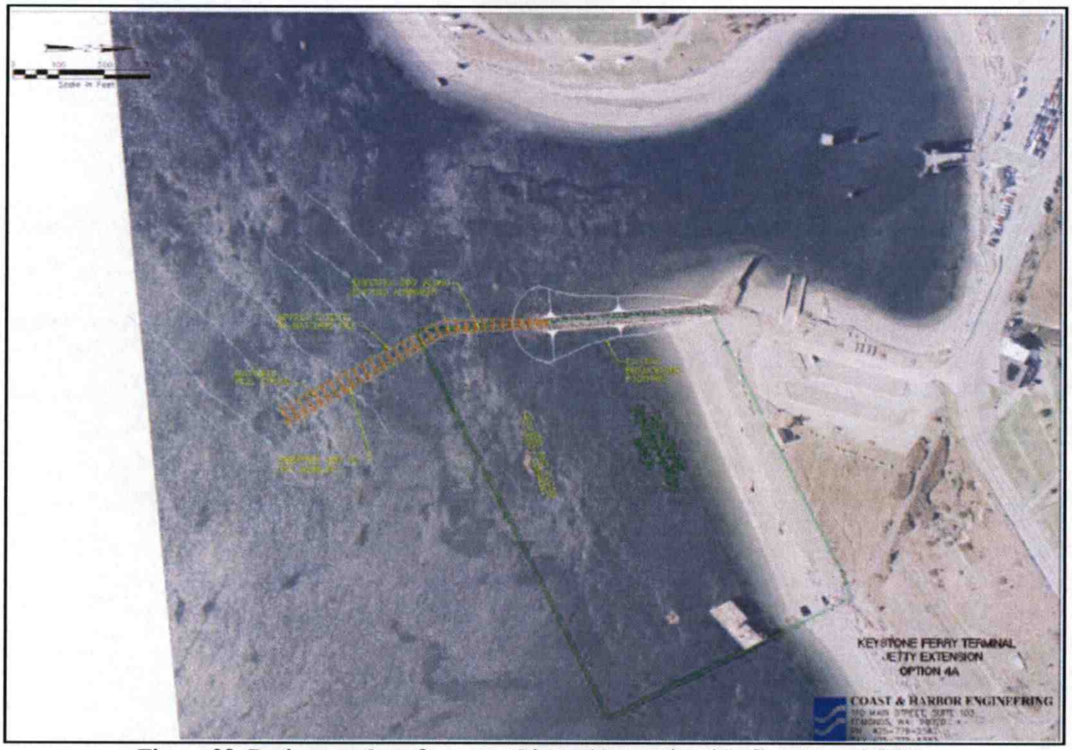

Figure 39. Design overlay of proposed jetty Alternative 4A. Courtesy of CHE.

Figure 40. Modeled jetty Alternative 4A.

During the testing of Alternative 4A minor disturbances in the water surface were noticeable in lee of the jetty. This observation was thought to be caused by the redirection of longshore current under the wave barrier. Analysis of the data confirmed the observation by showing larger cross-channel currents in lee of the jetty. The shelter region provided by Alternative 4A was considerably smaller than in other alternative jetty designs. In addition, the maximum current recorded in testing was 36.3 cm/s, the largest current velocity achieved during testing. Figure 37 shows the velocity vector field for jetty Alternative 4A.

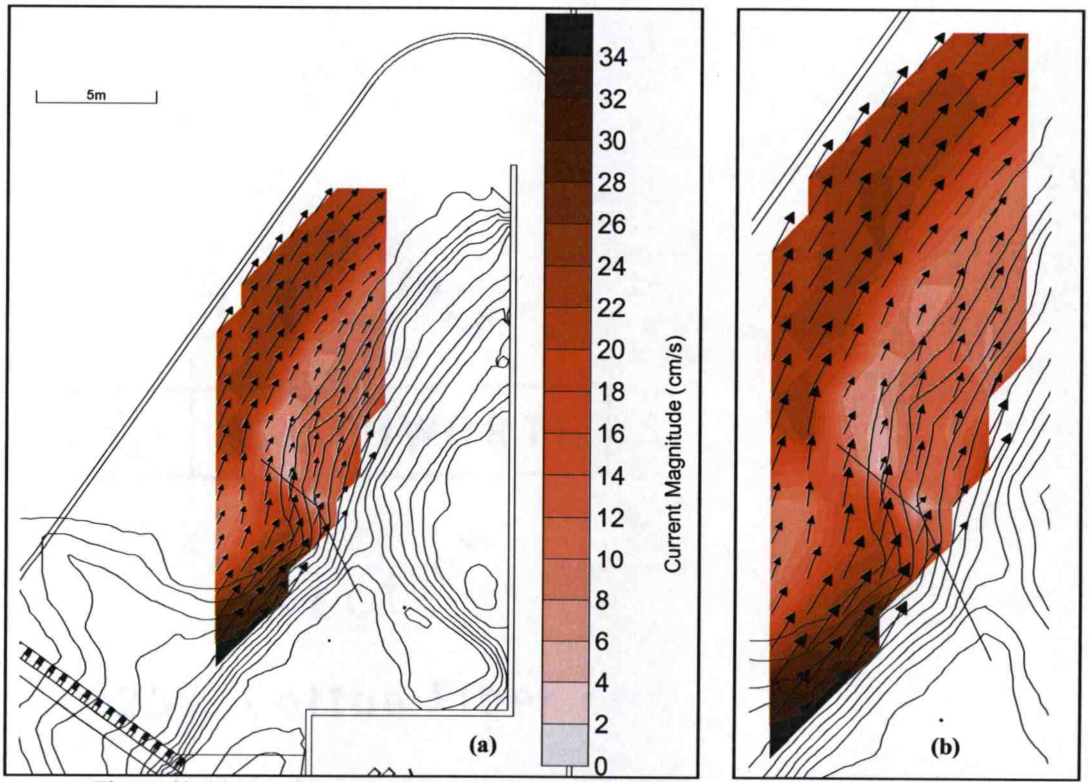

Figure 41. (a) Velocity vector field with jetty Alternative 4A and (b) velocity field detail.

6.1.7 Longshore Current Evaluation Method

To evaluate the performance of jetty alternatives a number of parameters were analyzed along the harbor channel centerline. Figure 42 shows a schematic of the harbor channel centerline used for evaluation.

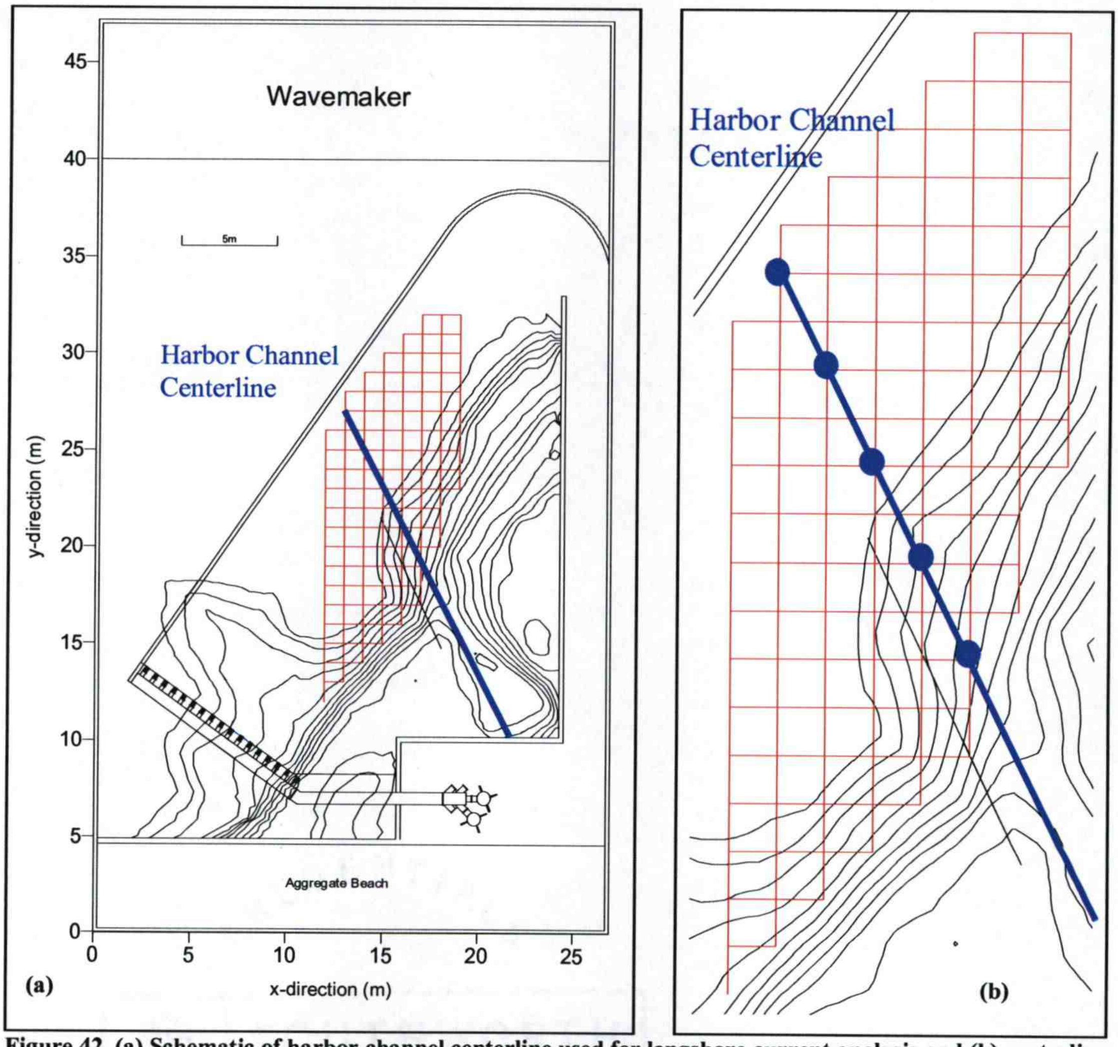

Figure 42. (a) Schematic of harbor channel centerline used for longshore current analysis and (b) centerline detail.

Maximum and average current velocities were calculated along the harbor channel centerline both inside and outside of the jetty reach. Additionally, these maximum values were used to calculate velocity gradients between data collection points along the channel centerline. Velocity gradient, $\nabla \vec{U}_{CL}$, calculations used the following formulation

Eq. 7

$$\nabla \vec{U}_{CL} = \frac{\left|\vec{U_2} - \vec{U_1}\right|}{\sqrt{X_2^2 - X_1^2}}$$

where $\vec{U_2} = u_2\hat{i} + u_2\hat{j}$ and $\vec{U_1} = u_1\hat{i} + u_1\hat{j}$ correspond to current velocities data taken at grid locations X_2 and X_1 respectively.

6.1.8 Summary of Longshore Current Results

The testing of longshore currents provided quantitative information for comparing the effectiveness of proposed jetty designs. The results of this analysis are summarized in Table 5.

Table 5. Summary of longshore current testing results.

	Outside Jetty Reach		Inside Jetty Reach		
Jetty	$U_{CL\,\max}$ (cm/s)	$\overline{U}_{outside}$ (cm/s)	$U_{CL\,\max}$ (cm/s)	$\overline{U}_{lee}$ (cm/s)	$\nabla \vec{U}_{CL\,\max}$ (cm/s/m)
Existing	16.6	14.5	12.7	16.1	1.6
Alternative 1A	28.5	26.3	3.1	4.3	11.0
Alternative 3A	32.1	30.4	2.9	4.4	12.6
Alternative 3B	29.6	29.4	2.5	3.1	12.1
Alternative 4A	23.7	22.2	6.0	7.8	7.9

Note: $U_{CL\,\max}$ = maximum velocity along channel centerline
$\overline{U}_{outside}$ = average velocity along channel centerline outside of jetty reach
$\overline{U}_{lee}$ = average velocity along channel centerline inside of jetty reach
$\nabla \vec{U}_{CL\max}$ = maximum gradient along channel centerline

Alternative 3B provided the most effective harboring in lee of the jetty as can be seen by yielding the lowest velocities inside the jetty reach. Alternative 4A minimized the velocity gradient, however has highest current velocities inside the jetty reach. The testing results of Alternative 1A show the best combination of reducing current velocities in lee of the jetty as well as a providing a low velocity gradient.

Each of the aforementioned jetty designs have aspects of improved protection; however none of these alternatives provide a sufficient combination of improvements to greatly improve the navigational hazards experienced at Keystone Harbor. A coorelation exists between decreased current velocities in lee of the jetty and increased velocity gradients.

6.2 Wave Testing Results

The following section gives an overview of the results from wave measurements collected during the wave testing sequence. It is important to note, that although quantitative values were recorded during wave testing, the major objective of this testing was to qualitatively assess shoreline impact based on incident wave conditions.

6.2.1 Refraction and Diffraction Patterns

The wave testing sequence included subjecting each jetty alternative to six wave cases with varying characteristics, as described in Section 5.2. Wave cases 1 through 4 were characterized by regular waves, while cases 5 and 6 consisted of random wave trains. Data collection during the testing was used to gain a broader understanding of the refraction and diffraction patterns present with each jetty alternative. The collected wave data was post-processed and organized in tabular format. Tables delineating wave transformation for each wave case are located in Appendix D.

6.2.2 Shoreline Change Observations

In addition to wave height measurements, shoreline change was observed to aid in the qualitative assessment of jetty alternatives. These detailed observations outlined the impact of the wave field on the beach veneer adjacent to jetties. Noteworthy items included the formation of offshore berms, sediment transport and sorting, creation of swash zone scarps, and other beach responses. Records were also made describing cross wave patterns in the harbor channel entrance. A table outlining these notes can be found in Appendix E. Shoreline change observations revealed that none of the proposed jetty design alternatives resulted in adverse impacts on the adjacent cobble beach.

6.3 Results Summary

Laboratory testing of four proposed jetty design alternatives was carried out on the Keystone Harbor physical model. The testing procedure involved the data collection of longshore currents, wave transformation, and visual observation of shoreline change. Analysis of the data has provided information applicable for the assessment of jetty design effectiveness. The proposed alternatives were evaluated based on their ability to:

- Alleviate navigational hazards associated with longshore currents experienced at the Keystone Harbor channel entrance.
- Minimize adverse environmental impacts on the surrounding shoreline adjacent to jetty.

Based on this evaluation method, none of the proposed jetty designs were determined to provide sufficient improvement for safe ferry operation. Decreased velocities in lee of the jetty led to increased velocities outside the jetty reach. Due to this result, large velocity gradients were observed and deemed unsatisfactory for navigational safety. Additional testing is suggested to determine an effective and efficient solution to the problems experienced at Keystone Harbor

7 Discussion

Throughout the construction and implementation of this project a number of valuable lessons were learned by those involved. First and foremost, it is extremely important to consider the project in a holistic manner. Taking a broad approach to the solution can prevent encountering additional problems at a later time. For example, while designing the discharge manifold, little attention was paid to the placement of the manifold on the model bathymetry. This lack of foresight led to increased difficulty in the assembly of the discharge manifold and the correct positioning of knife valves. In order to correct the problem wood trough supports had to be constructed. Additionally, flanged 22° elbow adapters were required to correct the offset height and angle forced by the trough supports. Although these seemingly minor details were overlooked in the design phase, they required substantial amounts of time and effort to be corrected. These unforeseen problems caused delays in the progress of construction and testing.

Another lesson learned was the importance of checking and verifying the work of co-workers. Although verification is tedious and time consuming, discovering faulty calculations or measurements at an early stage is significantly less troublesome than making corrections later. During the initial placement of transects, several transect forms were surveyed in to incorrect positions. This error was eventually realized and required the excavation of fill material and significant amounts of time and work to correct the problem. Had the survey been verified to begin with, these mistakes could have been avoided all together.

Strong communication between group members and faculty advisors was extremely important throughout the design, construction, and testing process. Developing a sound basis of communication through weekly meetings and brainstorming sessions allowed the design team to gain from each others' knowledge and experience. This provided a means for determining individual strengths and weaknesses and also allowed assignment to be allocated accordingly. Each person involved with the project was assigned specific tasks to manage, providing a sense of responsibility and lessening the burden of management for any one person.

Building a strong communication pathway between the clients was also of great significance. Establishing this contact early on in the process enabled the project team to learn about the specific criteria or interests that the client wanted addressed. This eliminated miscommunications and helped avoid additional mishaps. Asking clear and concise questions aided tremendously when directions were vague or ambiguous. Determining the clients' expectations and the steps needed to resolve them was extremely valuable in the design and setup of the testing procedure.

As a whole, the design and implementation of the physical model progressed fairly smoothly. The following list outlines a few problem areas in which improvements could be made. This list is also representative of recommendations for future testing endeavors. Progress can be made in the following categories:

- Pre-planning site visit would have allowed an improved understanding of project scope and critical components before any work was carried out. This is an essential component for making connections with the problem characteristics.

- Additional collaboration with sub-contractors during the design and construction stages. Utilizing the experience and expertise of construction professionals would have been highly beneficial for streamlining the construction process. This would have also increased efficiency during construction.
- Making conservative estimates with time scheduling. It is necessary to allocate adequate time for unexpected delays and malfunctions of equipment. Working with consulting agencies requires deadlines to be met, so it is imperative that conservative and realistic estimates be made in original timelines.
- Keeping accurate records of project schedule and inventory of materials. Taking the time to update construction progress and to manage the paperwork. These are both good housekeeping mechanisms that provide valuable information for future projects. Tracking quantities, material costs, and man-hours worked can be useful tools for providing reasonable estimates for bidding purposes.

Although this discussion is not all-inclusive, it outlines the major obstacles experienced during the progression of this project. Likewise, a number of subjects needing enhancement were identified and suggestions made to execute these improvements. One final recommendation for those undertaking similar large scale testing endeavors is to take advantage of the help of others familiar with similar projects. Try to gain as much experience and exposure to the subject matter as possible; build upon the knowledge of fellow researchers, peers, and professors.

8 Conclusions

8.1 General Conclusions

A large scale physical model of Keystone Harbor, WA was designed and constructed at the O.H. Hinsdale Wave Research Laboratory at Oregon State University. Upon the completion of construction, a series of multiphase tests were administered to evaluate the performance and effectiveness of four proposed jetty design alternatives. Proposed jetty alternatives were evaluated based on their ability to deter longshore current effects within the harbor navigation channel and to minimize adverse environmental impacts on surrounding beaches. Quantitative and qualitative data was collected, processed and analyzed, providing concerned parties the necessary information to collectively evaluate proposed jetty designs.

The experience and learning process of the physical modeling design and implementation can be summarized with the following conclusions:

- The construction of realistic bathymetry was recreated at a 1:40 Froude scale in a laboratory environment. Accuracy of model construction ranged between -4.8 cm to 3.9 cm over the averaged model surface when compared with designed model surface.
- The model accurately simulated coastal processes involving waves and tidal currents in a laboratory setting.
- The work of this thesis established the capabilities to produce high capacity longshore currents within the TWB at Oregon State University.
- Comparisons with field data revealed that these longshore current capabilities can provide accurate representations of field processes.

- The data collected herein, was provided to Coast and Harbor Engineering to verify and validate numerical modeling efforts.
- Information and data analysis from the physical modeling study was an integral part in the feasibility analysis of proposed jetty designs for Keystone Harbor, WA.

8.2 Broader impacts

As well as providing information necessary to solve a practical problem, the physical model of Keystone Harbor was utilized for auxiliary educational purposes. The design and construction phase of the project provided a hands-on opportunity for a number of college students to further their understanding of coastal engineering applications. This phase of the modeling process was used as a National Science Foundation, REU (Research Experience for Undergraduates) site. Led by a graduate student mentor, two REU students were involved in the conceptual design and construction development of the Keystone physical model.

In addition, two groups of high school physics students utilized the model and wave laboratory facilities to conduct senior science projects. In one case, the model was used to explore the combined effects of waves and currents on estuarial recruitment of buoyant particles. The second group evaluated the extent of maximum run-up and inundation on the model. By utilizing the Keystone model, realistic situations were recreated for both projects. The model dually served its purpose for both real world problem solving and educational outreach.

8.3 Future Studies

In addition to the work presented in this thesis, a second phase of testing on the Keystone Harbor model will begin in March 2005. Phase II will involve the reconfiguration of the harbor channel entrance. This scenario will simulate the dredging and removal of approximately 91 m (300 ft) of shoreline on the east side of the current harbor channel, a newly proposed alternative for Keystone Harbor. The existing jetty will be relocated approximately 91 m (300 ft) east of its current location. Testing will commence upon completion of the bathymetric changes.

In April 2005, a tsunami inundation and run-up study will be carried out on the model. This study is aimed at using video techniques to resolve maximum run-up and tsunami bore velocities on realistic bathymetry/topography. The study will be conducted by an OSU graduate student and the results will be used in the completion of his thesis degree requirements.

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APPENDICES

Appendix A. Vendor Contact Information

This appendix gives contact information for material suppliers and contractors used throughout the duration of the Keystone Harbor physical modeling project.

Company	Sales Person	Product	Contact Information
Morse Brothers	N/A	Concrete & Sand Materials	541-752-3428
Double Eagle Construction	Anthony Kaumanns	Concrete Construction	541-929-4050
Willamette Graystone	N/A	Concrete Products	541-752-3456
MacDonald Industrial	N/A	Construction Fasteners	541-928-7277
Robnetts Hardware	N/A	Construction Hardware	541-753-5531
Fastenal	N/A	Construction Hardware	541-753-2064
Industrial Welding Supply	N/A	Construction Hardware	541-752-8686
Coastal Farm & Supply	N/A	Construction Hardware	541-928-2511
Benton Electric	Bryan Babbitt	Electrical Services	541-967-1244
Sunbelt Rentals	N/A	Equipment Rentals	541-791-5168
Corvallis Rentals	N/A	Equipment Rentals	541-753-2213
Epic Scan Ltd.	Carlos Velazquez	LIDAR Imaging	541-857-4904
Keith Brown Lumber	N/A	Lumber & Supplies	541-752-1674
Spaeth Lumber & Home Center	N/A	Lumber & Supplies	541-752-1930
FNW/Ferguson	Keith Jorgensen	Pump Rental/Sales	503-287-7781
Pacific Survey Supply	Richard Ash	Survey Equipment	541-754-3488

Appendix B. Proposed Jetty Alternative Designs

This appendix contains proposed jetty alternative design drawings as provided by CHE. Model jetty alternatives were scaled and constructed based on design drawings. Table B1 delineates jetty dimensions in prototype and model lengths.

Table B1. Alternative jetty dimensions in prototype and model length scale.

Jetty	Dimension	Prototype ft	Prototype m	Model ft	Model m
Existing	Length above SWL	386.32	117.75	9.66	2.94
	Length below SWL	435.96	132.88	10.90	3.32
	Footprint width @ 61.0 m	88.27	26.90	2.21	0.67
	Footprint width @ 106.8 m	156.49	47.70	3.91	1.19
	Berm width above SWL	8.00	2.44	0.20	0.06
	Height above SWL	14.00	4.27	0.35	0.11
1A	Length above SWL	986.34	300.64	24.66	7.52
	Length below SWL	1138.16	346.91	28.45	8.67
	Footprint width @ 198.3 m	233.07	71.04	5.83	1.78
	Footprint width @ 281.3 m	323.24	98.52	8.08	2.46
	Height above SWL	15.00	4.57	0.38	0.11
3A	Length above SWL to angle	586.32	178.71	14.66	4.47
	Length above SWL from angle	400.00	121.92	10.00	3.05
	Length below SWL from angle	551.76	168.18	13.79	4.20
	Footprint width @ 106.8 m	156.53	47.71	3.91	1.19
	Footprint width @ 27.9 m from angle	241.29	73.54	6.03	1.84
	Footprint width @ 110.8 m from angle	333.37	101.61	8.33	2.54
	Berm crest width above SWL	30.00	9.14	0.75	0.23
	Berm width at SWL	75.00	22.86	1.88	0.57
	Height above SWL	15.00	4.57	0.38	0.11
3B	Length above SWL to angle	586.32	178.71	14.66	4.47
	Length below SWL from angle	400.00	121.92	10.00	3.05
	Length on floor from angle	563.82	171.85	14.10	4.30
	Footprint width @ 106.8 m	156.53	47.71	3.91	1.19
	Footprint width @ 27.9 m from angle	264.74	80.69	6.62	2.02
	Footprint width @ 110.8 m from angle	357.42	108.94	8.94	2.72
	Submerged depth below SWL	5.00	1.52	0.13	0.04
	Berm crest width below SWL	30.00	9.14	0.75	0.23
	Height above SWL	15.00	4.57	0.38	0.11
4A	Length above SWL to angle	586.32	178.71	14.66	4.47
	Length above SWL from angle	400.00	121.92	10.00	3.05

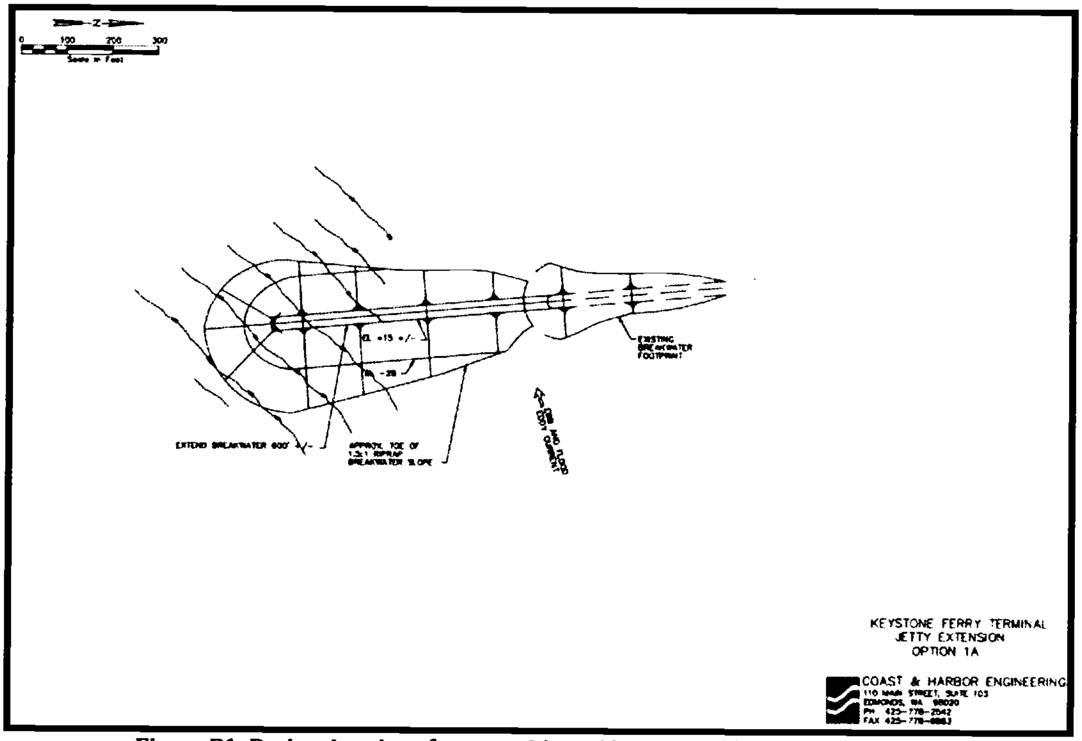


Figure B1. Design drawing of proposed jetty Alternative 1A. Courtesy of CHE.

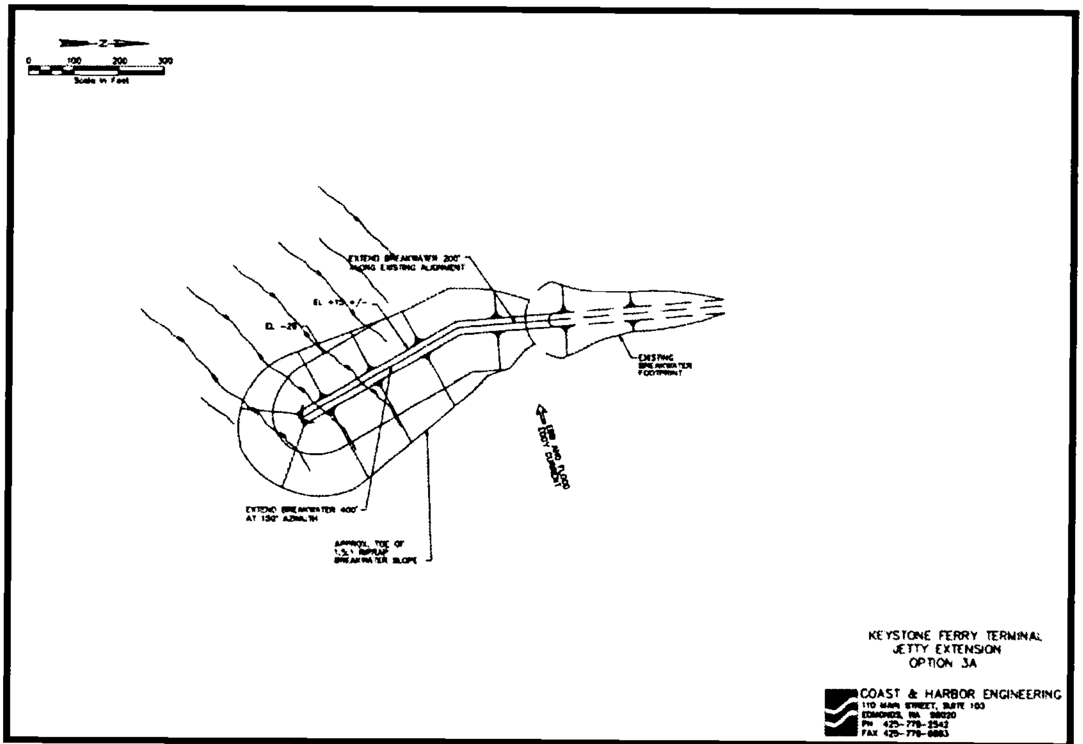


Figure B2. Design drawing of proposed jetty Alternative 3A. Courtesy of CHE.

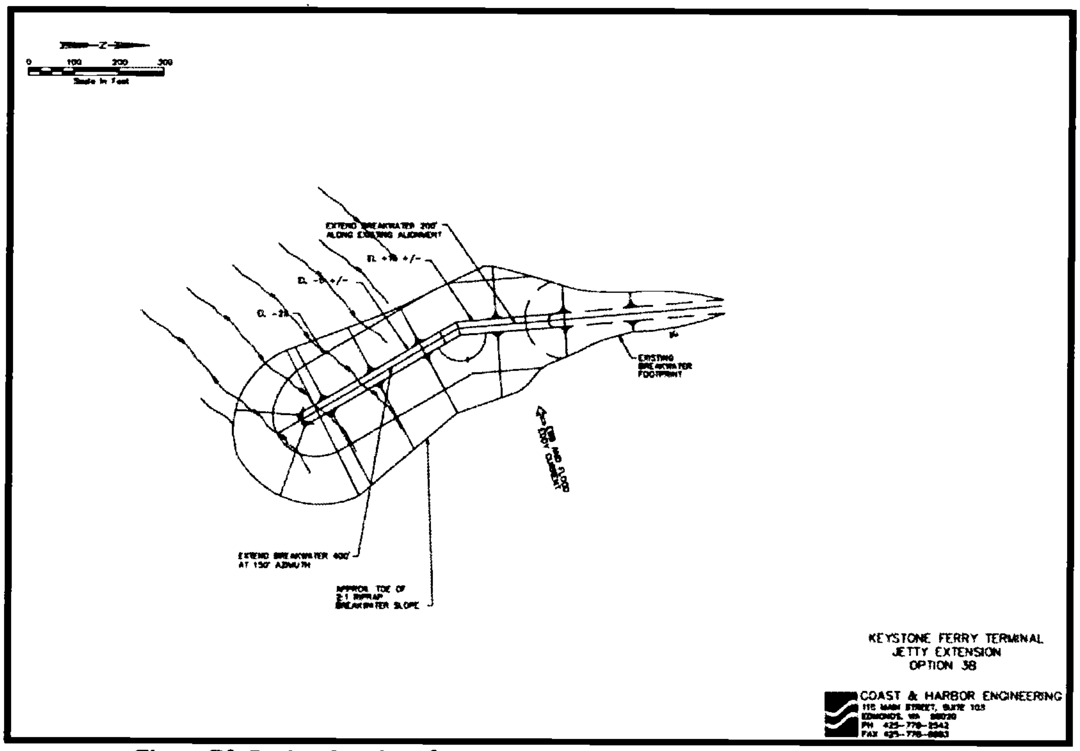


Figure B3. Design drawing of proposed jetty Alternative 3B. Courtesy of CHE.

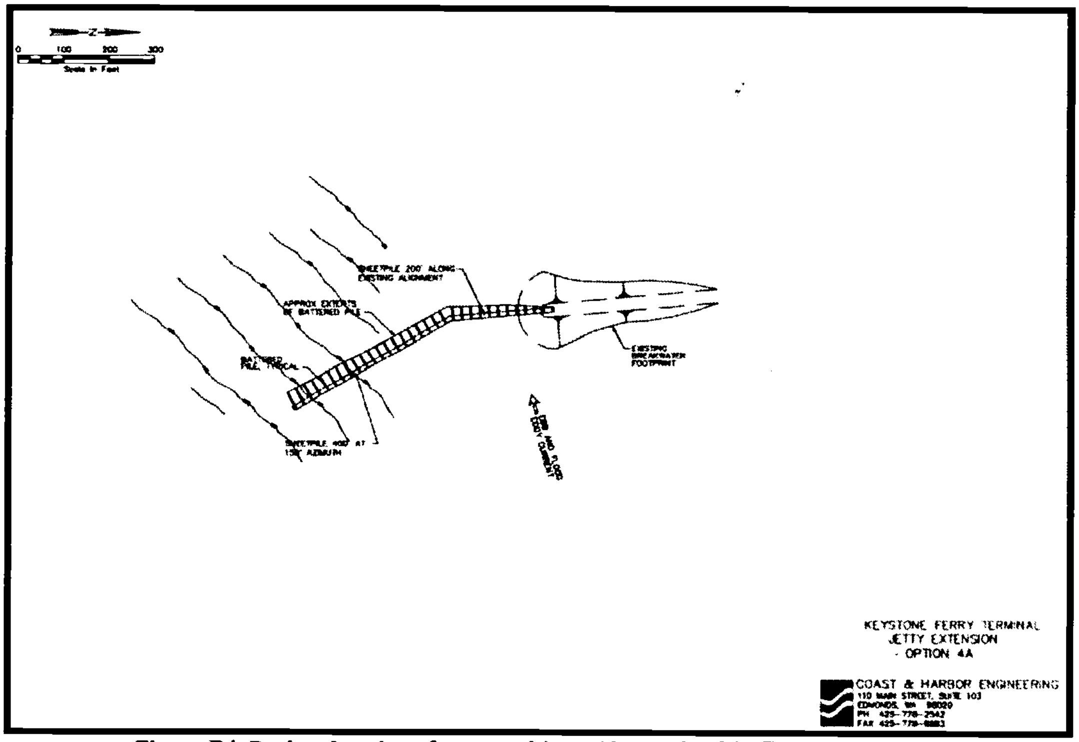


Figure B4. Design drawing of proposed jetty Alternative 3A. Courtesy of CHE.

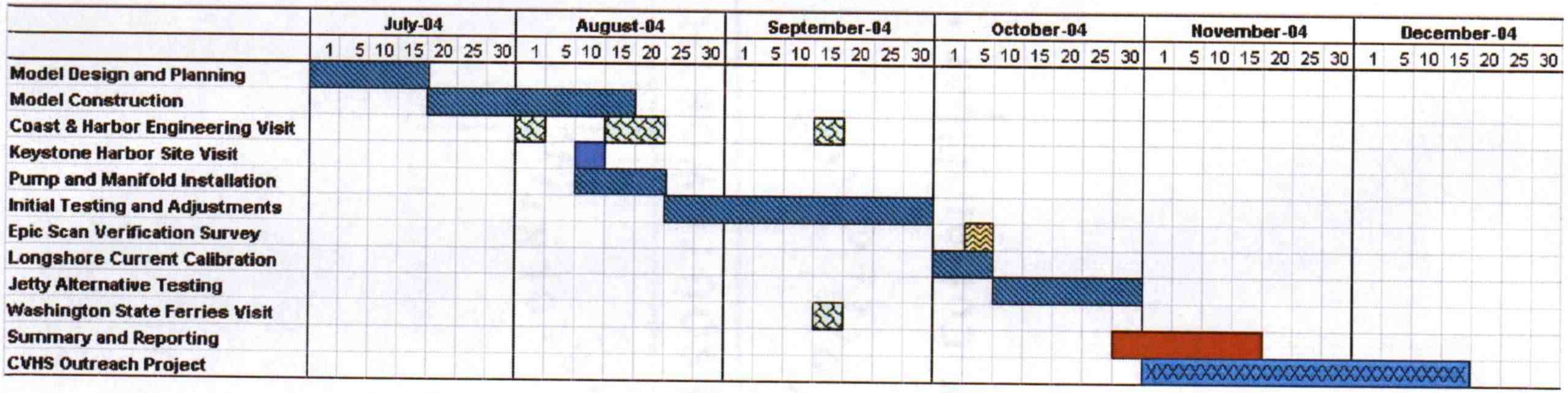
Appendix C. Timeline and Notable Events

This appendix gives details on the construction and testing schedule along with notable events which occurred during the process. Table D1 provides start and end dates for the construction and testing sequence. Table D2 shows a timeline of significant events which took place during the experimental process.

Table C1. Construction and testing schedule for Keystone Harbor physical model.

Task Name	Date Start	Date Finish	Duration (days)
Preconstruction--Order materials	13-Jul-04	14-Jul-04	2
Preparation of staging area	14-Jul-04	15-Jul-04	2
Materials delivery	15-Jul-04	20-Jul-04	6
Drain Tsunami Wave Basin	18-Jul-04	18-Jul-04	1
Cutting/constructing forms	19-Jul-04	20-Jul-04	2
Block placement	19-Jul-04	21-Jul-04	3
Sand Delivery	20-Jul-04	28-Jul-04	9
Skid steer Rental	21-Jul-04	11-Aug-04	22
Sand placement and contouring	21-Jul-04	09-Aug-04	20
Compaction of fill material	28-Jul-04	09-Aug-04	13
Keystone site visit	08-Aug-04	09-Aug-04	2
Placement of concrete veneer	13-Aug-04	13-Aug-04	1
Construct pump manifold	13-Aug-04	17-Aug-04	5
Fill Basin	14-Aug-04	14-Aug-04	1
Initial pump rental	19-Aug-04	27-Oct-04	70
Additional pump rental	28-Sep-04	27-Oct-04	30
Verification Survey--Epic Scan	04-Oct-04	04-Oct-04	1
Initial testing and adjustments	06-Oct-04	08-Oct-04	3
Longshore current testing--Null	08-Oct-04	08-Oct-04	1
Longshore current testing--Existing	10-Oct-04	10-Oct-04	1
Longshore current testing--Alt 1A	11-Oct-04	11-Oct-04	1
Longshore current testing--Alt 3B	12-Oct-04	12-Oct-04	1
Longshore current testing--Alt 3A	13-Oct-04	13-Oct-04	1
Wave testing--Alt 3A	14-Oct-04	18-Oct-04	5
Wave testing--Alt 1A	19-Oct-04	19-Oct-04	1
Wave testing--Alt 3B	19-Oct-04	19-Oct-04	1
Wave testing--Existing	20-Oct-04	20-Oct-04	1
Longshore current testing--Alt 4A	24-Oct-04	24-Oct-04	1
Wave testing--Alt 4A	24-Oct-04	24-Oct-04	1
Completion of testing		24-Oct-04	
Total days of construction and testing			86

Table C2. Timeline and significant milestones achieved during construction and testing of Keystone Harbor model..



Appendix D. Wave Testing Data Tables

The following tables provide wave height values calculated during the wave testing sequence. Data was collected as previously described in section 5.2. Wave height was calculated independently with each wave gage. Incident wave data were collected at y-location: 25 m. Incident wave conditions were met when the mean of the three wave gage values was within ± 10% of the target wave height. The tables included herein report wave height values for each of the three wave gages (denoted WG1, WG2, and WG3) as well as the average of the three values. Calculations of percent difference between target and mean recorded values are also included.

Table D1. Summary of wave field data for Existing Condition.

Case	Y-Location [m]	Theta [deg]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean WG1:WG3	Target [cm]	Difference [%]
1	incident	0	2.1	1.9	2.4	2.1	2.3	-7
1	25	0	2.4	2.1	2.3	2.3	2.3	-1
1	23	0	2.0	2.2	2.3	2.1	2.3	-7
1	21	0	3.7	3.2	2.9	3.3	2.3	43
1	19	0	2.6	2.2	2.1	2.3	2.3	0
1	18	0	2.7	2.8	2.1	2.5	2.3	10
1	17	0	2.7	2.9	1.6	2.4	2.3	5
1	16	0	2.3	3.3	2.1	2.6	2.3	13
1	15	0	2.2	2.4	2.1	2.3	2.3	-1
2	incident	-15	3.4	2.5	1.7	2.5	2.3	10
2	25	-15	4.4	3.2	1.8	3.1	2.3	37
2	23	-15	3.0	3.8	2.4	3.1	2.3	35
2	21	-15	2.9	3.6	4.0	3.5	2.3	53
2	19	-15	1.6	2.9	3.1	2.5	2.3	11
2	18	-15	3.0	1.8	2.8	2.5	2.3	11
2	17	-15	2.2	3.1	3.0	2.8	2.3	21
2	16	-15	3.8	3.0	3.6	3.5	2.3	52
2	15	-15	3.2	2.6	3.0	3.0	2.3	29
3	incident	0	2.6	3.0	3.0	2.9	3.0	-5
3	25	0	3.2	2.8	3.6	3.2	3.0	7
3	23	0	3.0	3.8	2.4	3.1	3.0	3
3	21	0	2.7	3.5	2.6	2.9	3.0	-2
3	19	0	3.4	2.9	3.4	3.2	3.0	6
3	18	0	3.3	3.3	3.4	3.4	3.0	12
3	17	0	3.1	3.1	2.5	2.9	3.0	-4
3	16	0	3.3	3.2	3.2	3.2	3.0	8
3	15	0	2.9	3.2	2.8	2.9	3.0	-2
4	incident	-15	3.5	2.5	3.7	3.2	3.0	7
4	25	-15	3.3	2.8	3.8	3.3	3.0	11
4	23	-15	3.7	3.2	2.9	3.3	3.0	9
4	21	-15	3.3	3.0	2.3	2.8	3.0	-5
4	19	-15	3.8	3.7	3.7	3.7	3.0	24
4	18	-15	2.5	3.4	2.2	2.7	3.0	-10
4	17	-15	3.7	3.4	4.4	3.9	3.0	28
4	16	-15	2.5	2.3	3.1	2.6	3.0	-13
4	15	-15	3.9	1.7	4.0	3.2	3.0	7
5	incident	0	2.2	2.1	2.3	2.2	2.3	-5
5	19	0	2.3	2.2	2.2	2.2	2.3	-2
5	17	0	2.2	2.2	2.3	2.2	2.3	-3
5	15	0	2.2	2.1	2.2	2.2	2.3	-5
6	incident	0	2.9	2.8	2.9	2.9	3.0	-5
6	19	0	3.0	3.0	3.3	3.1	3.0	3
6	17	0	3.1	2.8	3.2	3.0	3.0	1
6	15	0	3.1	2.7	3.1	2.9	3.0	-2

Table D2. Summary of wave field data for proposed jetty Alternative 1A.

Case	Y-Location [m]	Theta [deg]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean WG1:WG3	Target [cm]	Difference [%]
1	incident	0	2.2	2.2	2.4	2.3	2.3	-1
1	25	0	1.9	1.9	1.9	1.9	2.3	-16
1	23	0	1.8	2.7	2.7	2.3	2.3	-1
1	21	0	2.5	0.0	2.6	1.7	2.3	-26
1	19	0	2.2	0.8	1.9	1.6	2.3	-28
1	18	0	2.3	1.6	1.5	1.8	2.3	-22
1	17	0	2.8	1.2	1.1	1.7	2.3	-26
1	16	0	2.7	1.1	1.2	1.7	2.3	-27
1	15	0	2.6	0.7	1.8	1.7	2.3	-27
2	incident	-15	3.2	2.5	1.6	2.4	2.3	7
2	25	-15	4.5	3.7	1.3	3.2	2.3	38
2	23	-15	2.7	4.0	3.1	3.3	2.3	43
2	21	-15	2.2	0.0	2.0	1.4	2.3	-39
2	19	-15	2.6	1.0	1.6	1.8	2.3	-23
2	18	-15	2.6	0.9	2.0	1.8	2.3	-19
2	17	-15	2.3	1.1	2.1	1.9	2.3	-19
2	16	-15	3.2	0.7	1.6	1.8	2.3	-20
2	15	-15	2.0	0.9	0.8	1.2	2.3	-46
3	incident	0	2.8	3.0	3.3	3.1	3.0	2
3	25	0	3.3	3.6	4.1	3.7	3.0	22
3	23	0	3.3	2.6	3.3	3.0	3.0	2
3	21	0	3.3	0.0	3.7	2.3	3.0	-23
3	19	0	3.4	1.2	3.0	2.5	3.0	-16
3	18	0	3.6	1.1	3.0	2.6	3.0	-15
3	17	0	3.3	1.3	2.0	2.2	3.0	-27
3	16	0	3.5	1.8	3.0	2.8	3.0	-8
3	15	0	3.2	1.3	2.4	2.3	3.0	-24
4	incident	-15	3.5	2.3	3.6	3.1	3.0	5
4	25	-15	3.6	2.6	4.1	3.4	3.0	15
4	23	-15	3.3	3.4	3.0	3.2	3.0	7
4	21	-15	3.3	0.0	1.7	1.7	3.0	-45
4	19	-15	2.8	0.5	2.0	1.8	3.0	-41
4	18	-15	1.2	1.0	1.8	1.3	3.0	-56
4	17	-15	2.6	0.9	3.4	2.3	3.0	-23
4	16	-15	1.9	2.2	0.8	1.7	3.0	-45
4	15	-15	2.3	1.7	1.3	1.8	3.0	-41
5	incident	0	2.2	2.1	2.3	2.2	2.3	-6
5	19	0	2.5	0.7	1.9	1.7	2.3	-26
5	17	0	2.4	1.0	1.7	1.7	2.3	-26
5	15	0	2.3	1.0	1.6	1.6	2.3	-30
6	incident	0	2.9	2.7	3.0	2.9	3.0	-4
6	19	0	3.2	1.1	2.3	2.2	3.0	-27
6	17	0	2.9	1.3	2.2	2.1	3.0	-29
6	15	0	2.8	1.3	2.1	2.1	3.0	-31

TableD. Summary of wave field data for proposed jetty Alternative 3A.

Case	Y-Location [m]	Theta [deg]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean WG1:WG3	Target [cm]	Difference [%]
1	incident	0	2.1	2.5	2.1	2.2	2.3	-2
1	25	0	1.9	2.0	1.3	1.8	2.3	-23
1	23	0	1.9	2.0	2.2	2.0	2.3	-11
1	21	0	2.2	2.6	2.4	2.4	2.3	5
1	19	0	1.9	0.1	0.6	0.9	2.3	-62
1	18	0	2.1	0.6	0.5	1.1	2.3	-54
1	17	0	2.4	1.0	1.0	1.5	2.3	-36
1	16	0	2.1	1.1	1.3	1.5	2.3	-35
1	15	0	1.6	1.0	1.5	1.4	2.3	-38
2	incident	-15	1.8	2.8	2.8	2.5	2.3	9
2	25	-15	1.8	3.1	2.5	2.5	2.3	8
2	23	-15	3.4	2.2	4.0	3.2	2.3	39
2	21	-15	4.3	3.0	4.2	3.8	2.3	67
2	19	-15	3.0	0.3	0.5	1.3	2.3	-44
2	18	-15	1.4	0.7	0.5	0.9	2.3	-62
2	17	-15	1.3	0.8	1.7	1.3	2.3	-43
2	16	-15	0.8	0.6	2.2	1.2	2.3	-47
2	15	-15	1.6	0.8	1.6	1.3	2.3	-42
3	incident	0	3.0	2.8	2.9	2.9	3.0	-3
3	25	0	3.2	3.5	3.4	3.4	3.0	13
3	23	0	3.2	2.7	3.3	3.1	3.0	2
3	21	0	2.7	2.4	2.7	2.6	3.0	-13
3	19	0	2.3	0.4	2.3	1.6	3.0	-46
3	18	0	2.1	1.2	1.5	1.6	3.0	-46
3	17	0	1.9	1.3	1.5	1.6	3.0	-48
3	16	0	2.8	1.0	1.5	1.8	3.0	-41
3	15	0	2.3	1.2	0.7	1.4	3.0	-53
4	incident	-15	3.4	2.5	4.0	3.3	3.0	10
4	25	-15	2.9	2.8	3.7	3.1	3.0	4
4	23	-15	3.7	3.1	3.4	3.4	3.0	13
4	21	-15	3.0	3.2	1.7	2.6	3.0	-12
4	19	-15	2.3	0.0	1.3	1.2	3.0	-60
4	18	-15	0.2	0.7	1.9	0.9	3.0	-69
4	17	-15	2.3	1.3	2.6	2.1	3.0	-31
4	16	-15	1.2	2.9	1.2	1.8	3.0	-41
4	15	-15	1.9	1.0	0.8	1.2	3.0	-59
5	incident	0	2.2	2.2	2.3	2.3	2.3	-2
5	19	0	2.0	0.4	0.8	1.1	2.3	-54
5	17	0	1.9	0.8	1.0	1.2	2.3	-47
5	15	0	1.8	0.8	1.2	1.3	2.3	-44
6	incident	0	2.7	2.6	2.8	2.7	3.0	-10
6	19	0	2.4	0.6	1.3	1.4	3.0	-52
6	17	0	2.2	1.2	1.4	1.6	3.0	-46
6	15	0	2.2	1.1	1.5	1.6	3.0	-47

Table D4. Summary of wave field data for proposed jetty Alternative 3B

Case	Y-Location [m]	Theta [deg]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean WG1:WG3	Target [cm]	Difference [%]
1	incident	0	2.1	2.0	2.2	2.1	2.3	-8
1	25	0	2.4	2.1	2.3	2.3	2.3	-2
1	23	0	2.0	2.2	2.3	2.1	2.3	-7
1	21	0	2.3	2.4	2.2	2.3	2.3	-1
1	19	0	2.6	1.7	2.4	2.2	2.3	-5
1	18	0	2.1	1.0	1.8	1.6	2.3	-29
1	17	0	2.0	1.8	1.9	1.9	2.3	-18
1	16	0	2.0	1.9	2.4	2.1	2.3	-10
1	15	0	1.9	1.9	2.6	2.2	2.3	-6
2	incident	-15	3.3	2.2	1.6	2.4	2.3	4
2	25	-15	4.4	3.2	1.8	3.1	2.3	36
2	23	-15	3.1	3.4	2.5	3.0	2.3	28
2	21	-15	3.2	3.7	3.8	3.6	2.3	55
2	19	-15	0.6	3.2	2.8	2.2	2.3	-7
2	18	-15	1.8	1.7	3.1	2.2	2.3	-5
2	17	-15	0.7	2.8	3.9	2.5	2.3	7
2	16	-15	2.9	2.1	3.7	2.9	2.3	26
2	15	-15	3.0	0.8	2.9	2.2	2.3	-3
3	incident	0	2.8	2.8	3.2	2.9	3.0	-2
3	25	0	3.2	2.8	3.6	3.2	3.0	7
3	23	0	2.9	2.4	3.1	2.8	3.0	-6
3	21	0	2.7	2.9	2.4	2.7	3.0	-11
3	19	0	1.7	2.4	3.7	2.6	3.0	-14
3	18	0	2.1	2.3	2.3	2.2	3.0	-26
3	17	0	2.2	3.5	1.8	2.5	3.0	-16
3	16	0	2.1	3.1	1.7	2.3	3.0	-23
3	15	0	1.6	2.7	0.7	1.7	3.0	-43
4	incident	-15	3.2	2.5	3.6	3.1	3.0	3
4	25	-15	3.3	2.8	3.8	3.3	3.0	11
4	23	-15	3.0	3.1	3.1	3.1	3.0	2
4	21	-15	3.3	3.6	2.0	3.0	3.0	-1
4	19	-15	2.1	3.0	4.1	3.1	3.0	3
4	18	-15	1.3	1.9	4.6	2.6	3.0	-14
4	17	-15	2.9	1.8	5.3	3.3	3.0	11
4	16	-15	3.5	1.3	3.2	2.7	3.0	-10
4	15	-15	3.7	0.7	2.3	2.2	3.0	-26
5	incident	0	2.1	1.9	2.3	2.1	2.3	-9
5	19	0	1.9	1.6	2.2	1.9	2.3	-17
5	17	0	1.7	2.1	1.7	1.8	2.3	-20
5	15	0	1.5	2.2	1.5	1.7	2.3	-24
6	incident	0	2.9	2.7	2.9	2.8	3.0	-7
6	19	0	2.2	1.8	2.9	2.3	3.0	-24
6	17	0	2.1	3.0	1.9	2.3	3.0	-22
6	15	0	2.1	2.5	1.8	2.1	3.0	-29

Table D5. Summary of wave field data for proposed jetty Alternative 4A

Case	Y-Location [m]	Theta [deg]	WG1 [cm]	WG2 [cm]	WG3 [cm]	Mean WG1:WG3	Target [cm]	Difference [%]
1	incident	0	2.2	2.2	1.9	2.1	2.3	-7
1	25	0	1.6	2.1	1.9	1.9	2.3	-19
1	23	0	2.2	1.7	2.5	2.2	2.3	-6
1	21	0	2.2	2.1	2.0	2.1	2.3	-9
1	19	0	2.0	0.4	0.3	0.9	2.3	-60
1	18	0	2.5	0.3	0.9	1.2	2.3	-47
1	17	0	2.6	0.4	0.4	1.1	2.3	-51
1	16	0	2.5	0.3	1.6	1.5	2.3	-37
1	15	0	2.0	0.3	0.9	1.0	2.3	-55
2	incident	-15	3.4	2.5	1.3	2.4	2.3	5
2	25	-15	4.3	2.6	0.8	2.6	2.3	13
2	23	-15	4.6	4.2	3.5	4.1	2.3	80
2	21	-15	2.8	3.9	3.2	3.3	2.3	45
2	19	-15	0.9	1.4	0.5	1.0	2.3	-58
2	18	-15	2.4	1.3	1.0	1.6	2.3	-31
2	17	-15	1.4	0.6	1.0	1.0	2.3	-56
2	16	-15	2.7	1.5	0.9	1.7	2.3	-26
2	15	-15	1.2	0.8	0.7	0.9	2.3	-60
3	incident	0	3.1	3.0	2.8	3.0	3.0	-1
3	25	0	3.1	2.7	3.1	3.0	3.0	-1
3	23	0	3.0	2.4	3.3	2.9	3.0	-3
3	21	0	3.2	3.6	3.7	3.5	3.0	18
3	19	0	3.0	1.0	1.5	1.8	3.0	-39
3	18	0	2.8	1.0	1.8	1.9	3.0	-37
3	17	0	2.6	1.7	0.6	1.6	3.0	-46
3	16	0	2.2	1.1	2.1	1.8	3.0	-40
3	15	0	1.9	1.2	2.1	1.7	3.0	-42
4	incident	-15	3.6	2.7	3.0	3.1	3.0	2
4	25	-15	4.1	3.0	3.3	3.5	3.0	14
4	23	-15	4.2	3.7	4.0	3.9	3.0	31
4	21	-15	3.3	2.7	2.6	2.9	3.0	-4
4	19	-15	3.6	0.3	1.0	1.6	3.0	-45
4	18	-15	0.9	0.8	1.2	1.0	3.0	-67
4	17	-15	2.3	1.1	1.5	1.6	3.0	-45
4	16	-15	1.5	2.2	1.0	1.6	3.0	-48
4	15	-15	1.9	1.1	0.5	1.2	3.0	-61
5	incident	0	2.3	2.0	2.0	2.1	2.3	-8
5	19	0	2.4	0.6	0.9	1.3	2.3	-44
5	17	0	1.8	0.7	1.1	1.2	2.3	-47
5	15	0	1.9	0.9	1.1	1.3	2.3	-44
6	incident	0	3.1	2.9	2.8	2.9	3.0	-2
6	19	0	2.9	0.9	1.3	1.7	3.0	-43
6	17	0	2.6	1.2	1.5	1.8	3.0	-41
6	15	0	2.3	1.2	1.5	1.7	3.0	-43

Appendix E. Shoreline Observation Tables

The following tables give details of qualitative shoreline observations noted during the wave testing sequence. Observations include sediment sorting, bar and berm formations, beach cusp and scarp development, as well as other notable observations.

Table E1. Shoreline change observations for Existing Condition.

	Wave Case	Sediment Sorting	Submerged bar	Berm formation	Beach cusps in swash zone	Scarp	Additional notes
Existing Condition	1	Yes	Formed 5-10 cm from beach face	None	None	Minor	Very little protection from incident waves.
	2	Yes	Formed 5-10 cm from beach face	None	None	Minor	Very little protection from incident waves.
	3	Yes	Formed 10-30 cm from beach face. Offshore location increases with distance from jetty.	Yes	Early formation of individual cusps.	None	Substantial sediment transport on and offshore. Eroded sediment exposing concrete at 1.8 m from jetty.
	4	Yes	Formed 10-30 cm from beach face. Offshore location increases with distance from jetty.	Yes	Early formation of individual cusps. Increase width with distance from jetty.	None	Higher Sediment trasport rates than all other conditions.
	5	Minor	None	None	None	None	Minimal sediment transport.
	6	Minor	None	None	None	None	Minimal sediment transport.

Table E2. Shoreline change observations for proposed jetty Alternative 1A.

	Wave Case	Sediment Sorting	Submerged bar	Berm formation	Beach cusps in swash zone	Scarp	Additional notes
Alternative 1A	1	Minor	None	None	None	None	Visibly larger waves in lee of jetty compared to Alternatives 3A and 3B.
	2	Minor	None	None	None	None	Visibly larger waves in lee of jetty compared to Alternatives 3A and 3B.
	3	Yes	None	Berm formation due to merging of individual cusps.	Cusp size increase with distance from jetty. 10 cm wide and 25 cm length.	None	Cusps and berms made predominantly from fine materials.
	4	Yes	Formed 10-30 cm from beach face	Yes	Cusp size increase with	None	Higher Sediment trasport rates than all other conditions.
	5	Minor	None	None	None	None	No major shoreline changes.
	6	Minor	None	None	None	None	No major shoreline changes.

Table E3. Shoreline change observations for proposed jetty Alternative 3A.

	Wave Case	Sediment Sorting	Submerged bar	Berm formation	Beach cusps in swash zone	Scarp	Additional notes
Alternative 3A	1	None	Early formation of bar at 22 cm offshore.	None	None	Minor	No noticeable wave reflection from jetty/wave interaction.
	2	Yes	None	Berm formation due to merging of individual cusps.	Early formation of individual cusps.	Scarp 0.5 cm high on backshore.	Scarp comprised of larger grain material. Sediment sorting at scarp toe.
	3	Yes	Offshore 25 cm from beachface. Comprised of larger grain materials.	Berm formed along entire beach face, 10 cm wide, 0.5 cm high.	Early formation of individual cusps.	None	Cusps and berms made predominantly from fine materials.
	4	Yes	Minor	None	Early formation of individual cusps.	Scarp along backshore.	Minor accreation at jetty toe.
	5	Minor	None	Minor	None	Minor	Finer material dominate swash zone. Less prominent formations than in regular wave cases.
	6	Minor	None	Minor	None	Minor	Finer material dominate swash zone. Less prominent formations than in regular wave cases.

Table E4. Shoreline change observations for proposed jetty Alternative 3B.

	Wave Case	Sediment Sorting	Submerged bar	Berm formation	Beach cusps in swash zone	Scarp	Additional notes
Alternative 3B	1	Minor	None	Berm formation due to merging of individual cusps.	Early formation of individual cusps.	Minor	Jetty shadow zone visibly smaller than Alternative 3A. Offshore migration of sediment.
	2	Minor	None	Berm formation due to merging of individual cusps.	Cusp size increase with distance from jetty.	Minor	More wave energy transmitted across submerged jetty.
	3	Yes	Offshore 30 cm from beachface. Comprised of larger grain materials, 12 cm in width.	Large berm formation 1.6 m from jetty toe. Berm widths up to 15 cm.	Cusp size increase with distance from jetty.	None	Cusps and berms made predominantly from fine materials.
	4	Yes	Minor	Berm formation 2.0 m from jetty toe. Limited cusp merging.	Early formation of individual cusps.	None	Higher Sediment trasport rates than all other conditions.
	5	Minor	None	None	None	None	No major shoreline changes.
	6	Minor	None	None	None	Minor	No major shoreline changes.

Table E5. Shoreline change observations for proposed jetty Alternative 4A.

	Wave Case	Sediment Sorting	Submerged bar	Berm formation	Beach cusps in swash zone	Scarp	Additional notes
Alternative 4A	1	Minor	Early formation of bar at 22 cm offshore.	None	Small beach cusp formations.	None	Noticeably larger waves in harbor channel compared to rubble mound structures.
	2	Minor	None	None	Small beach cusp formations.	None	Strong cross wave pattern in harbor channel. Strong reflections off of wave barrier. More diffraction around jetty.
	3	Yes	None	Merging of cusps. Berm formed along entire beach face, 10 cm wide, 0.5 cm high.	Early formation of individual cusps. 5 cm wide, 30 cm long.	None	Cusps and berms made predominantly from fine materials.
	4	Yes	None	Merging of cusps. Berm formed along entire beach face.	Early formation of individual cusps.	None	Cusps and berms made predominantly from fine materials.
	5	Minor	None	None	None	Minor	Minimal sediment transport.
	6	Minor	None	None	None	Minor	Minimal sediment transport.