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Coastal and Hydraulics Laboratory

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Coastal Inlets Research Program

August 2004

South Jetty Sediment Processes Study, Grays Harbor, Washington: Processes Along Half Moon Bay

Philip D. Osborne and Michael H. Davies

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by

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Draft report

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Preface

This report describes an evaluation of the performance of engineering measures that have been implemented to control breaching of the South Jetty and shoreline erosion in Half Moon Bay at Grays Harbor, WA. The report presents a set of hypothetical alternatives to improve the performance of the existing condition. A preliminary evaluation of alternatives is performed leading to recommendations for a concept design that provides a long-term solution to breaching at the South Jetty. The work was conducted as part of activities of the Coastal Inlets Research Program (CIRP). CIRP is administered at the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL) under the Navigation Systems Program for Headquarters, U.S. Army Corps of Engineers (HQUSACE). Mr. Barry W. Holliday is HQUSACE lead technical monitor for CIRP. Dr. Sandra K. Knight, CHL, is Technical Director for the Navigation Systems Program. Dr. Nicholas C. Kraus, Senior Scientists Group, CHL, is CIRP Program Manager.

The mission of CIRP is to conduct applied research to improve USACE capability to manage federally maintained inlets, which exist on all coasts of the United States (including the Atlantic Ocean, Gulf of Mexico, Pacific Ocean, and Great Lakes regions). CIRP objectives are to (a) make management of channels – the design, maintenance, and operation – more effective to reduce the cost of dredging, and (b) preserve the adjacent beaches in a systems approach that treats the inlet and beach together. To achieve these objectives, CIRP is organized in work units conducting research and development in hydrodynamic, sediment transport, and morphology change modeling; navigation channels and adjacent beaches; inlet scour and jetties; laboratory and field investigations; and technology transfer.

This report was prepared by Dr. Philip D. Osborne and Dr. Michael H. Davies of Pacific International Engineering, PLLC. Dr. Osborne was Principal Investigator for Pacific International Engineering. Technical assistance at PI Engineering in conducting the study was provided by Drs. Paul Tschirky, Nels J. Sultan, and Wei Chen, Messrs. David Hericks, Kenneth Gund, Robert Osborne, Ryan Freke, and Adam Skalenakis, in support of field data collection; and data processing. Dr. Kraus performed technical review of this report. Work was performed under the general administrative supervision of Mr. Thomas W. Richardson, Director, CHL.

Dr. James R. Houston was Director of ERDC, and COL James R. Rowan, EN, was Commander and Executive Director at the time of publication.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
miles	1.60934	kilometers
square miles	2.58999	square kilometers
feet	0.3048	meters
inches	2.54	centimeters
cubic yards	0.76455	cubic meters

1 Introduction

Background

Grays Harbor is located on the southwest Washington coast at the mouth of the Chehalis River, about 45 miles[1] north of the Columbia River mouth. The harbor is 13 miles wide at its broadest point, and 15 miles long from Aberdeen, Washington, to the entrance on the Pacific Ocean. The water surface area is 91 square miles at mean higher high water (mhhw) and 38 square miles at mean lower low water (mllw). The estuary is enclosed on the ocean side by spits, Point Brown on the north, and Point Chehalis on the south. The spits are separated by a 2-mile wide opening, which forms the natural harbor entrance. Two convergent rock jetties, North Jetty and South Jetty, extend seaward from the spit points. The jetties are part of the Grays Harbor Navigation Project, which is a federally constructed and maintained navigation channel that allows deep-draft shipping through the outer bar, Grays Harbor estuary, and the Chehalis River to Cosmopolis (Figure 1-1).

The development of the channels and facilities at Grays Harbor has been a continuing process since the Rivers and Harbors Act of June 1896 authorized the construction of the South Jetty. Maintenance dredging has been required after the 1990 Grays Harbor Navigation Improvement Project was completed. Erosion on South Beach and Half Moon Bay prompted the disposal of a portion of this dredged material in these areas. In December 1993, persistent shoreline erosion near the South Jetty culminated in the formation of a breach between the jetty and the adjacent South Beach. The U.S. Army Engineer District, Seattle (NWS), filled the breach in 1994 with 600,000 cu yd of sand dredged from the navigation channel as a temporary measure to protect the Grays Harbor Navigation Project and public facilities located south and east of the breach area.

The fill was originally expected to be effective in protecting the project for 3 years. During the seventh winter that the fill was in place (2001-2002), a series of storms damaged the South Beach and modified the Half Moon Bay shoreline, re-emphasizing the temporary nature of the sand fill. Further damage to the breach fill was caused by storms in the winter of 2003-2004.

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[1] This study involves analysis of historic and recent engineering documents with values expressed in American customary (non-SI) units. To maintain continuity with the previous body of work, the original units are retained in their context. Measurements and calculations made as part of the present study are expressed in SI units. A table of factors for converting non-SI units of measurement to SI units is presented on page xii.

Relevant engineering and maintenance measures in the area include the maintenance dredging and disposal program for the Grays Harbor and Chehalis River Navigation Project, the Point Chehalis Revetment fill, the South Beach breach fill with gravel transition beach, and the South Jetty wave diffraction mound. Each of these measures was designed to prolong the life of the breach fill and provide beach erosion protection. The purpose of the maintenance dredging and disposal program is to reduce the rate of beach erosion by periodically reintroducing sediment into the littoral system. The Point Chehalis Revetment fill is a Section 111 project designed to lessen the active shoreline erosion, providing periodic renourishment at year four and year eight of the ten-year project. The gravel transition beach was designed to slow erosion of the beach directly adjacent to the south side of the jetty and to eliminate the 8 ft high scarp at that location. In 2000, a wave diffraction mound and gravel transition beach were constructed to reduce erosion caused by wave action in western Half Moon Bay.

Purpose of Study

The U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory (CHL) coordinated with NWS to develop a Plan of Action to evaluate the engineering facilities and maintenance measures in the vicinity of South Jetty in a project technical meeting held at NWS on 15 January 2002. The South Jetty Sediment Processes Study was developed and keyed to elements of the Plan of Action. The purpose of the study was to evaluate the performance of engineering measures that have been implemented to control breaching of the South Jetty and shoreline erosion in Half Moon Bay. Additionally, the study assessed the risk of future breaching and erosion. The results of the study were documented in a report by Osborne, Wamsley, and Arden (ERDC/CHL TR-03-4). Subsequent reports document the analysis of a breached condition, assessment of the risk of future breaching, and development of a large-scale 3-dimensional physical model at ERDC/CHL for simulation of nearshore processes in Half Moon Bay ERDC/CHL TR-04-XX.

The purpose of this study is to update the evaluation of engineering measures implemented at Half Moon Bay through the analysis of recent field measurements of coastal processes, and beach and nearshore morphological change in Half Moon Bay. The evaluation provides a basis for the development of a set of hypothetical engineering alternatives to improve the performance of existing measures. A preliminary set of alternatives was developed by the project team in meetings at Seattle District in March 2002, following a public meeting at Westport to discuss the project. Alternatives were further discussed and refined with the project team on 25 February 2004 in a meeting at Seattle District. A preliminary evaluation of alternatives with planshape analysis and numerical modeling leads to the development of a concept design that would potentially provide a long-term solution to breaching at the South Jetty.

Chapter 2 of this report reviews the history of breach occurrence at South Jetty and the related engineering measures to prevent re-breaching. Chapter 2 includes a review of the breach fill, dredging and disposal activities associated with maintenance and new work dredging, analysis of the wave diffraction mound performance, analysis of upland and intertidal topography and nearshore bathymetry surveys, analysis of shoreline position changes, analysis of beach planshape, analysis of nearshore waves and currents, and transport of gravel, cobble and sand, and identification of sediment pathways, and a sediment budget

is developed. The performance of the engineering and maintenance measures is then evaluated based on these results.

Chapter 3 presents the results of a wave climate and water levels analysis for Half Moon Bay. The objective is to determine the frequency of occurrence of wave height period and direction and water levels (tide elevations), including the joint frequency of occurrence. The results are used to develop test runs for physical models, ERDC/CHL-TR-04-XX and numerical models of Half Moon Bay (Chapter 5).

Chapter 4 outlines the evaluation criteria and development of hypothetical functional alternatives to improve the performance of existing engineering measures. A preliminary screening of some alternatives is accomplished by application of planshape equilibrium analysis.

Chapter 5 describes the Coastal Gravity WAVE (CGWAVE) model and its implementation at Half Moon Bay, followed by discussion and interpretation of model calculations. The wave model CGWAVE provides input for calculation of sediment transport potentials in the Half Moon Bay nearshore. The existing condition and a preliminary screening of alternatives is accomplished in terms of the changes to waves and longshore sediment transport potential relative to the existing condition.

Chapter 6 develops a preliminary conceptual design that would potentially provide a long-term solution to breaching at the South Jetty. The conceptual design incorporates the most promising alternatives to reduce shoreline erosion in Half Moon Bay developed in the report, as well as those alternatives that would address erosion of South Beach

Chapter 7 provides an integrated summary of the report.

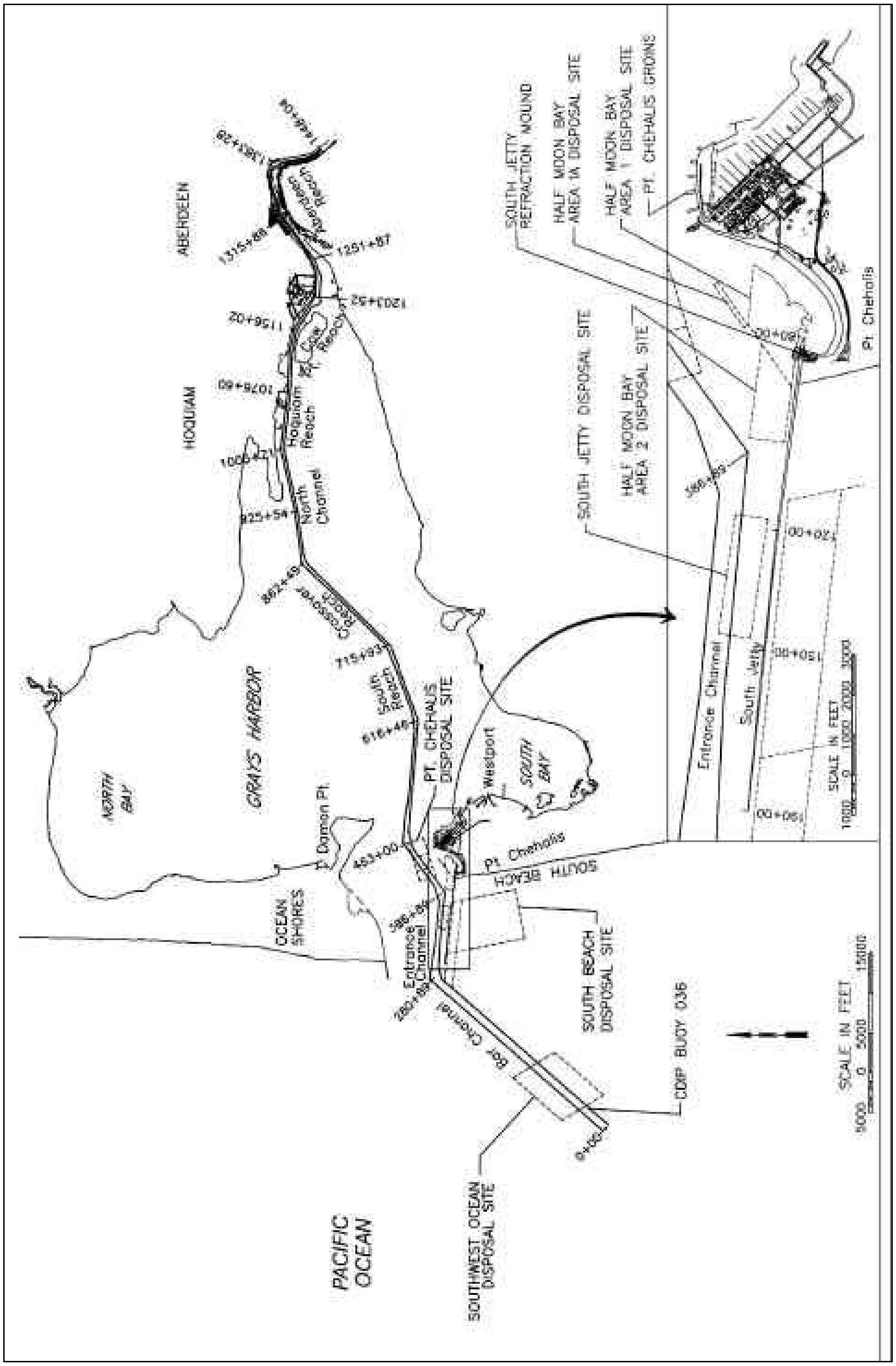

Figure 1-1. Grays Harbor Navigation Project

2 Physical Setting and Present Conditions

This chapter describes the physical setting and present conditions at Half Moon Bay and South Beach near the South Jetty at Grays Harbor. A brief history of breach occurrence at South Jetty, reviews the engineering measures taken to prevent breach re-occurrence, and analyses recent field measurements of coastal processes, and beach and nearshore morphological change in Half Moon Bay are presented. Engineering measures taken by the US Army Corps of Engineers, Seattle District, to prevent re-breaching at South Jetty include placement of 600,000 cu yd of sand dredged from the navigation channel to fill the breach in 1994, and construction of the wave diffraction mound and a gravel transition beach in 1999 and 2000. Sand dredged from the navigation channel was also placed in the nearshore of Half Moon Bay and at dredged sediment disposal sites in the vicinity of Half Moon Bay to help relieve the sediment deficit in this area.

This chapter includes an update of the performance evaluation of the engineering measures by Osborne et al (2003). That report concluded that the breach control efforts have been effective, but are not necessarily an efficient long term solution. The performance evaluation is based on analysis of upland and inter-tidal topography and nearshore bathymetry surveys, maintenance dredging and disposal volumes, shoreline position changes, beach planshape analysis, and direct measurements of waves, currents, suspended sand, gravel and cobble transport. A sediment budget for the Half Moon Bay shoreline and nearshore has been developed. The budget is in general agreement with a sediment budget developed by Osborne, et al (2003) for a larger region north of the South Jetty.

History of Breach Occurrence at South Jetty

Grays Harbor is located on the southwest Washington coast at the mouth of the Chehalis River, about 45 miles north of the Columbia River mouth. The harbor has one of the largest tidal prisms in the United States, with large tidal currents that dominate the movement of fine grained sand sediment around the harbor and entrance. In addition, the harbor is exposed to the large, long period waves which are typical of the Pacific northwest coast. The harbor is 13 miles wide at its broadest point, and 15 miles long from Aberdeen, Washington, to the entrance. The water surface area is 91 square miles at mean higher high water (mhhw) and 38 square miles at mean lower low water (mllw). The estuary is enclosed on the ocean side by spits, Point Brown on the north and Point Chehalis on the south. The spits are separated by a 2-mile wide opening, which forms the natural harbor entrance. Two convergent rock jetties, North Jetty and South Jetty, extend seaward from the spit points. The jetties and navigation are part of the federal Grays Harbor Navigation Project maintained by the US Army Corps of Engineers. The channel allows deep draft navigation and is maintained to a depth of 46 ft at the outer bar, decreasing to 32 ft at Aberdeen and Cosmopolis on the Chehalis River.

In December 1993, persistent shoreline erosion near the South Jetty culminated in the formation of a breach between the South Jetty and the adjacent South Beach (Figure 2-1). The City of Westport, Grays Harbor County, and the Port of Grays Harbor were alarmed by the potential for a rapid acceleration of the erosion and subsequent catastrophic damage to the jetties and navigation channel, in addition to the potential loss of upland water wells, sewage treatment plant, and other facilities in and near the City of Westport. Under the direction of the Department of the Army, the U.S. Army Corps of Engineers, Seattle District, filled the breach in the fall of 1994 with 600,000 cu yd of sand dredged from the navigation channel at a cost of $3,730,000 as a temporary measure to protect the Grays Harbor Navigation Project. Additional sediment placement efforts and other measures to reinforce the breach area are discussed further in the following section. An analysis of the breach and accompanying shoreline recession has been documented in a recent separate report (Chapter 2, Wamsley and Cialone, 2004). It concludes that the Grays Harbor breach resulted from shoreline erosion on both the ocean (South Beach) and bay (Half Moon Bay) side of Point Chehalis.

Figure 2-1.   Breached area at South Jetty Grays Harbor, 1994

Engineering Measures

Breach Fill

The breach was filled in the fall of 1994 with 600,000 cu yd of sand dredged from the bar channel. The breach was filled to temporarily protect the navigation project while plans for long-term management were developed. Figures 2-2 through 2-4 show aerial photographs of Half Moon Bay and South Beach taken between November 1994 and February 1996. The photographs illustrate the shoreline response after the breach was filled. The bayside shoreline receded rapidly toward the south during this interval. In November 2001, storms began to overtop the breach fill, causing concern to local interests and the Seattle District. The breach fill was originally expected to be effective in protecting the project for 5 to 10 years. Throughout the seventh winter that the fill was in place (2001-2002), a series of storms with sustained periods of high rainfall and high waves further damaged the South Beach and accelerated recession of the Half Moon Bay shoreline in the lee of the diffraction mound. In addition, it was found that the breach fill surface elevation at the narrowest area between Half Moon Bay and the Pacific Ocean was decreased during previous (summer-fall 2001) construction activities at the South Jetty. As a result, wave runup from the ocean side overflowed the fill, which channelized it and contributed to the scouring of the fill. The Seattle District placed 135,000 cu yd of dredged sand in May 2002 to restore the breach fill and raise the crest elevation to +26 ft mllw. Also, in November 2002, approximately 50,000 sprigs of native American dune grass were planted on 3 acres of the breach fill to help resist wind and storm wave erosion (Arden 2003). A further 30,000 cu yd was placed in February 2004, 25,000 cu yd placed in the southwest corner of Half Moon Bay to alleviate erosion of the shoreline at the end of the transition gravel beach, and 5,000 cu yd to restore the breach fill on the South Beach side.

Figure 2-2. 2 November 1994 during placement of breach fill

Figure 2-3. 3 January 1995 following placement of 600,000 cu yd of dredged sand in the breach

Figure 2-4. 1 February 1996 approximately 13 months following placement of 600,000 cu yd of dredged sand in the breach

Dredged Sediment Disposal in Half Moon Bay

Dredged material has been placed at a number of sites in and near Half Moon Bay, in part to mitigate ongoing erosion. In May 1992, a submerged berm was constructed by the Seattle District in Half Moon Bay to evaluate the use of dredged material to mitigate bayside erosion. Approximately 200,000 cu yd of sediment was placed in the form of a submerged berm just inshore of the -18 ft mllw contour. In May 1994, an additional 146,000 cu yd of dredged sand was placed on the berm at elevation –20 ft mllw.

In January 1995, the City of Westport placed 82,000 cu yd of sand along the eroded shoreline of Half Moon Bay to prevent further damage and protect the sewer outfall. Nearly all of this sediment was eroded by the end of the 1995 winter storm season. In the fall of 1995, under authority of Section 111 of the River and Harbor Act of 1968, the Seattle District placed 300,295 cu yd directly along the Point Chehalis beach (Chapter 1, Figure 1-1). The 300,295 cu yd quickly eroded causing termination of the Section 111 project by February 1996. Observations of nearshore placement confirm that seasonal placement in May results in net onshore transport compared to the erosion that occurred after placement in the fall of 1995.

Dredged material from the navigation channel is typically placed at six disposal sites in Grays Harbor and the open ocean. Currently, the Seattle District uses disposal sites at Point Chehalis, Half Moon Bay, South Beach, South Jetty, and the Southwest site; other sites are also permitted. Sites in Half Moon Bay and on Point Chehalis are designated for the disposal of dredged material that benefits beach nourishment and shore protection at Point Chehalis and Half Moon Bay. The volume of dredged material placed at Half Moon Bay and South Beach sites is summarized in Table 2-1. The table also lists the source of dredged material. Sites in Half Moon Bay receive dredged material predominately from South Reach, Point Chehalis, and Entrance Channel, characterized by sand material typical of Half Moon Bay beach material. Approximately 340,000 cu yd/year on average over the past 13 years has been disposed in the Half Moon Bay and South Beach areas (Chapter 1, Figure 1-1).

Table 2-1
Sand Placement Volumes and Sources 1991-2004
(Adapted from Osborne et al 2003, Table 6)

	Disposal Sites, Annual Volumes (cu yd)					
Year	**Half Moon Bay Nearshore (In water)**	**Half Moon Bay Direct (In water)**	**Westport Fill (Upland)**	**Breach Fill (Upland)**	**South Beach (In water)**	**Total**
1991	0	0	0	0	0	0
1992	200,000	0	0	0	0	200,000
1993	0	0	0	0	373,000	373,000
1994	146,000	0	0	600,000	265,000	865,000
1995	0	0	300,295 82,000	0	0	300,295
1996	274,780	0	0	0	0	274,780
1997	308,508	0	0	0	0	308,508
1998	441,474	0	0	0	0	441,474
1999	228,470	228,963	0	0	76,187	533,620
2000	0	0	0	0	0	0
2001	0	0	0	0	0	0
2002	378,441	135,706	0	135,000	75,219	589,366
2003	0	382435	0	0	137,689	520,124
2004	0	289652	0	30,000	262,176	581,828
Total volume (cu yd)	1,977,673	901,756	382,295	765,000	1,118,271	4,987,995
Reaches Dredged	Entrance, South	Entrance, South	South	Entrance, South	Bar	

Wave Diffraction Mound

In 2000, a wave diffraction mound structure was completed at the terminus of the South Jetty in Half Moon Bay. The purpose of the diffraction mound was to modify the wave approach angle along the shore and reduce or spread wave energy, thereby reducing erosion by longshore transport. The wave diffraction mound project included a number of features that decreased the effectiveness of the design, including removal of stone from the remaining portion of the South Jetty terminus, and modification of the slope and shape of the mound structure to conform to requests from environmental permitting agencies. These features were modifications to the original concept intent.

The City of Westport contracted PI Engineering to analyze the shore erosion problem at South Jetty and Half Moon Bay and identify possible engineering solutions. In a draft report dated November 1998, PI Engineering proposed a concept that included a wave diffraction mound added to the inshore end of South Jetty, sand tightening of a section of South Jetty, construction of a buried revetment extending through the former breach area from the flank of the South Jetty, and a beach fill placed in the first 1,000 ft south of the jetty[1].

In November 1998, Seattle District requested that the US Army Corps of Engineers, Engineering Research and Development Center (ERDC) conduct physical model tests of the proposed modifications to the South Jetty and Half Moon Bay. The model tests were conducted in the idealized inlet physical model operated by the Coastal Inlets Research Program (CIRP) (Seabergh, 1999). Different alternatives were tested including the existing condition (jetty remnant), and the Seattle District modified design with a wave diffraction mound. Test results indicated that the modified Seattle District design with the wave diffraction mound was the most effective for protecting the breach fill immediately adjacent to the jetty.

The diffraction mound was constructed from December 1999 to February 2000. The core of the diffraction mound was constructed at a 1 vertical to 3 horizontal (1:3) slope. The core was constructed of approximately 1,500 tons of jetty rock removed from the eastern 250 ft of the jetty. The outer layer of the diffraction mound was constructed with 30,000 tons of rock ranging in size from 100 to 10,000 lbs with side slopes ranging from 1:5 to 1:10 on the north side, and from 1:7 to 1:10 on the south side. The exposed northern face of the mound was constructed with 300 to 10,000 lb graded riprap and the southern face was constructed with 100 to 1,000 lb quarry spalls. The maximum elevation of the diffraction mound was approximately +17 ft mllw.

At the time of mound construction, the Seattle District removed existing jetty rock over the eastern 250 ft of the jetty. The jetty extension top elevation of +8 ft mllw was lowered to about +2 ft mllw during construction of the mound. The removal of the jetty extension caused the diffraction mound to be constructed at a point approximately 250 ft west of the position identified in the original concept proposed by PI Engineering. The diffraction mound concept and as-built condition are further discussed in Chapter 4. Shifting the diffraction mound to the west effectively caused the diffraction point for incident waves to be shifted to the west. The implications of this shift to shoreline erosion in Half Moon Bay are examined in the sections below, by planshape analysis in Chapter 4, and by numerical modeling in Chapter 5.

Gravel Transition Beach

In addition to constructing the diffraction mound, a gravel transition beach was placed as a transition material between the diffraction mound and the sandy shore of Half Moon Bay. Gravel 1 to 2 inches in size and was placed between the diffraction mound and a point 400 ft south of the center of the mound along the Half Moon Bay shoreline. Approximately 17,358 tons of gravel material was

[1] Pacific International Engineering, PLLC. (1998). "Grays Harbor Navigation Project South Beach stabilization analyses," Draft report submitted to the City of Westport, Pacific International Engineering, PLLC, Edmonds, WA.

placed at the time of construction of the mound, between December 1999 and February 2002.

The purpose of the gravel transition material was to protect the breach fill from erosion adjacent to the diffraction mound and to eliminate the dangerous 8-foot-high scarp on the Half Moon Bay side of the breach fill. The exact gradation of the material used is uncertain. The modified design dated 1 September 1999 for the South Jetty repairs is ambiguous concerning the specification for the gravel transition beach. Reference is made in construction documents to the use of naturally occurring rounded gravel and cobble material (+3/8 inch size). However, the material specifications called for 12 inch minus cobbles with up to 50 percent by weight larger than 3 inches.

Field observations and shoreline positions interpreted from aerial photographs reveal that the transition gravel was successful in stabilizing the shoreline in the location where it was placed. However, in the 2001-2002 winter, the sandy shoreline at the southern terminus of the transition gravel receded up to 4 m. High waves occurred at times of high water early in the storm season, which is inferred to have been significant to the shoreline recession. Rain saturation and channelization of the fill upland of the shoreline is also a probable factor. It is not known to what extent each of the factors is responsible for the observed erosion.

From December 2001 to January 2002, the transition gravel was extended eastward around the Half Moon Bay shoreline terminating approximately 1,000 to 1,200 ft from the diffraction mound. The purpose of this additional gravel was to stop the shoreline erosion that was progressing towards the access road to the State Park. A sustained period of strong rainfall and high waves in the winter of 2001-2002 deteriorated the breach fill and led to further recession south of the wave dissipation mound. Shortly afterward, the Seattle District performed emergency repairs consisting of fill placement, and gravel transition rehabilitation and extension. Unfortunately, the details regarding the extent and position of placement are limited; no as-built condition surveys were completed at the time of the gravel placement.

Performance Evaluation of Engineering Measures

The South Jetty Sediment Processes study (Osborne et al. 2003) evaluated the performance of engineering and maintenance measures that have been implemented to control breaching of the beach adjacent to the South Jetty, and erosion at Half Moon Bay and the Point Chehalis revetment area. The evaluation covered the maintenance dredging and disposal program for the Grays Harbor and Chehalis River Navigation Project, the Point Chehalis revetment, the South Beach fill, and the South Jetty wave diffraction mound with gravel transition zone. This report provides an update to the performance evaluation by Osborne et al (2003) based on the following new data and analyses:

- Shoreline and beach profile change
- Beach planshape analysis
- Measurements of gravel and cobble transport
- A preliminary sediment budget

The analyses are based on upland and inter-tidal topography, nearshore bathymetry surveys collected since the previous report, maintenance dredging and disposal volumes, shoreline position changes, beach planshape analysis, and direct measurements of gravel and cobble transport.

Shoreline and Beach Profile Change at South Beach and Half Moon Bay

Osborne et al (2003) digitized shoreline positions between 1996 and 2002 from ortho-rectified aerial photographs. The shoreline and scarp position database has been updated with shoreline and scarp positions for 2003 and 2004 (Figures 2-5 and 2-6). Figure 2-7 shows the change in shoreline position defined as the Average High Water Line (AHWL) at Half Moon Bay Transect 3 (HMB3) relative to the 1996 shoreline position. The location of transects at Half Moon Bay and South Beach is shown in Figure 2-8. Transect HMB3 is located in the southwest corner of the bay where the highest rates of erosion have occurred. The shoreline position measurements indicate the shoreline has receded at an average rate of 13.0 m/year at HMB3 between 1996 and 2004. A change in recession rate starting in 2000-2001 is apparent in Figure 2-7. The recession rate increases from 8.1 m/year for the interval 1997 to 1999 to 18.5 m/year for the interval 2000 to 2004. This shift in the recession rate coincides with the year following the removal of the existing rock over the eastern 250 ft of the jetty and with the completion of the diffraction mound. Table 2-2 and Figure 2-9 show the rate of shoreline position change calculated for each of the 9 transects over the three time intervals shown in Figure 2-7. The shoreline position data between 1996 and 2004 reveal a pattern of net shoreline recession of between 1.8 to 13.0 m/year at the western end of Half Moon Bay west of Transect HMB7 and net shoreline advance of approximately 7 m/year east of Transect HMB7. Figure 2-7 also reveals that the position of maximum erosion shifted from HMB5 to HMB3 for the periods 1997-1999 and 2000-2004, respectively. The westward shift in the location of maximum erosion is consistent with the shift in the diffraction control point caused by the removal of the jetty remnant for construction of the diffraction mound. This concept is explored further in Chapter 4 and Chapter 5.

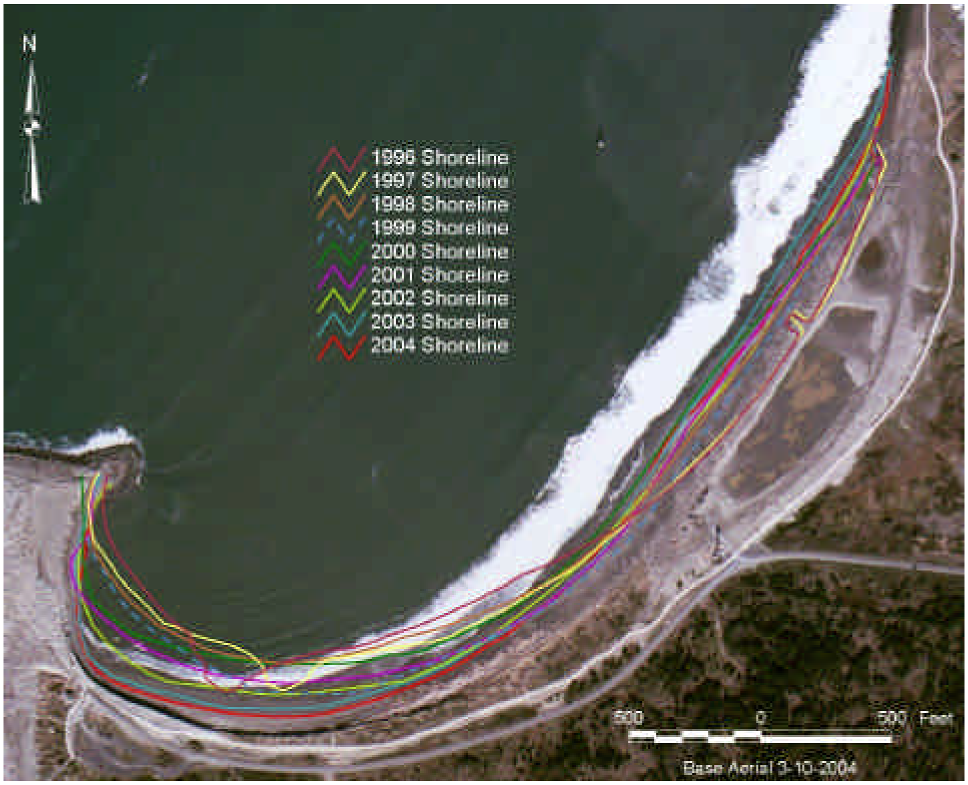

Figure 2-5. Half Moon Bay shorelines digitized from annual aerial photographs

Figure 2-6. Half Moon Bay scarp positions digitized from annual aerial photographs

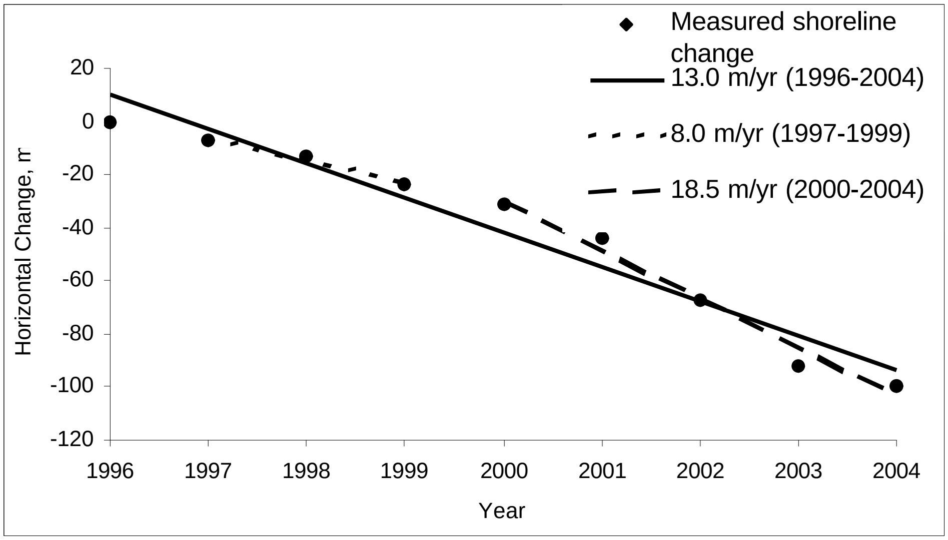

Figure 2-7. Measured shoreline recession at Half Moon Bay Transect HMB3 relative to its position in 1996

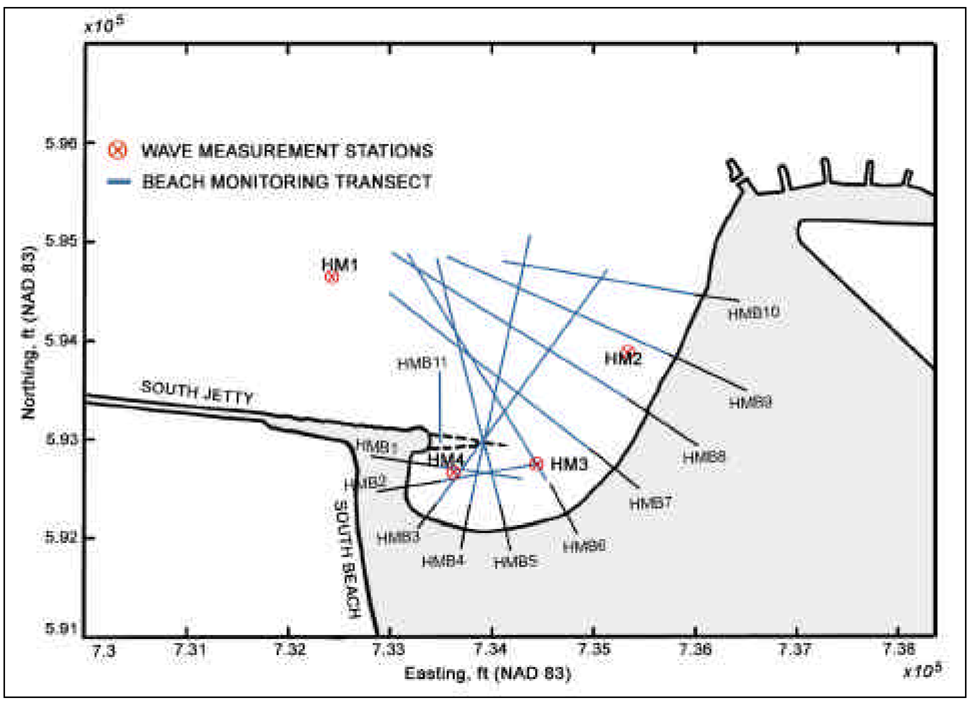

Figure 2-8. South Beach/Half Moon Bay survey transect locations

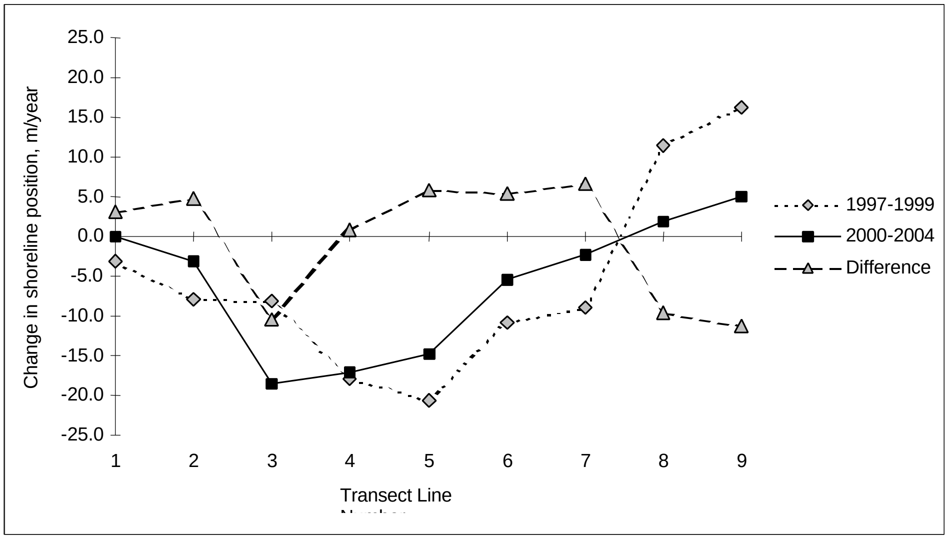

Figure 2-9. Measured shoreline recession at Half Moon Bay Transects relative to their positions from 1997 to 1999 and 2000 to 2004

Table 2-2
Average recession rates for Half Moon Bay shoreline at transect locations HMB 1-9, m/year

Transect	1997-1999	2000-2004	Difference	1996-2004
1	-3.2	-0.1	3.1	-1.8
2	-7.9	-3.2	4.7	-9.6
3	-8.1	-18.5	-10.4	-13.0
4	-17.9	-17.1	0.8	-7.0
5	-20.6	-14.8	5.8	-8.5
6	-10.9	-5.4	5.5	-5.9
7	-9.0	-2.3	6.7	0
8	11.6	2.0	-9.6	7.1
9	16.2	4.9	-11.1	6.8

Beach profile measurements have also been analyzed in addition to shoreline position trends derived from aerial photography. Figure 2-10 shows rectified profiles at HMB1 to HMB9. The beach elevation rises and steepens with distance east from the diffraction mound around Half Moon Bay. The profile at HMB1 has been relatively stable with minor erosion occurring on the upper portion of the gravel beach. Significant and persistent erosion has occurred at HMB2 through 5.

The most severe erosion occurred at HMB3 where approximately 70 m of horizontal recession occurred between December 2001 and October 2003. The profile surveys at HMB3 indicate that a steep erosional scarp developed above approximately 2 m elevation (NAVD88). The measurements indicate that the scarp receded rapidly between June 2002 and February 2004, and also that the beach surface beneath the elevation of mean higher high water (equivalent to 2.47 m NAVD88) lowered by approximately 1 m in front of the scarp since 2001. The beach lowering permits more wave energy to directly impact the scarp in 2004 for the same water level in 2001. The profiles indicate that the scarp receded at a higher rate than the average shoreline recession rate since June 2002. The scarp retreated more than 21 m between June 2002 and March 2003 (9 months). The scarp retreated a further 11 m between March 2003 and 15 October 2003. Almost all of the recession between March 2003 and October 2003 occurred during a single storm on 12-13 October 2003 when wave heights reached 10 m offshore from Grays Harbor.

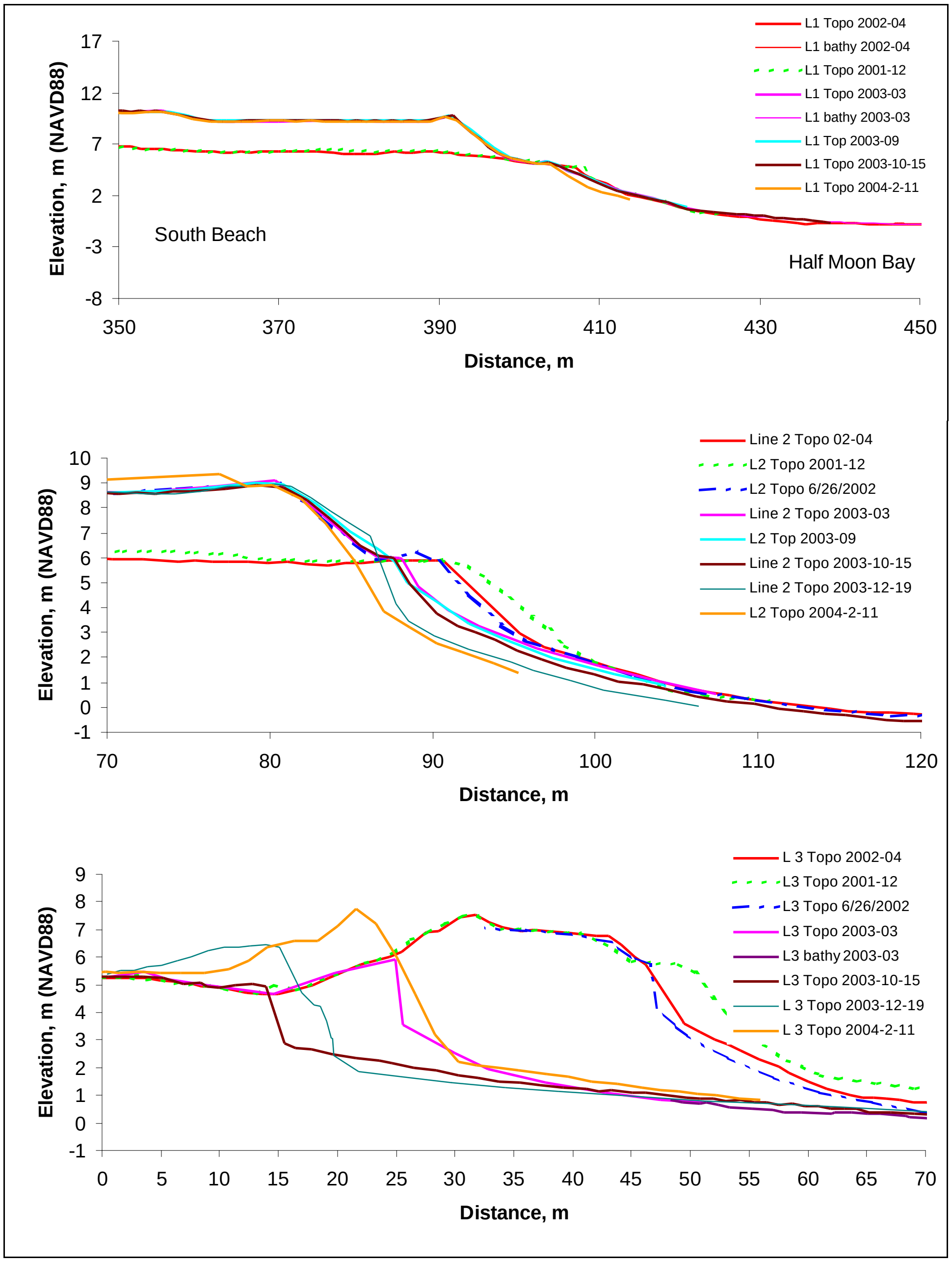

Figure 2-10. Half Moon Bay profiles (sheet 1 of 3)

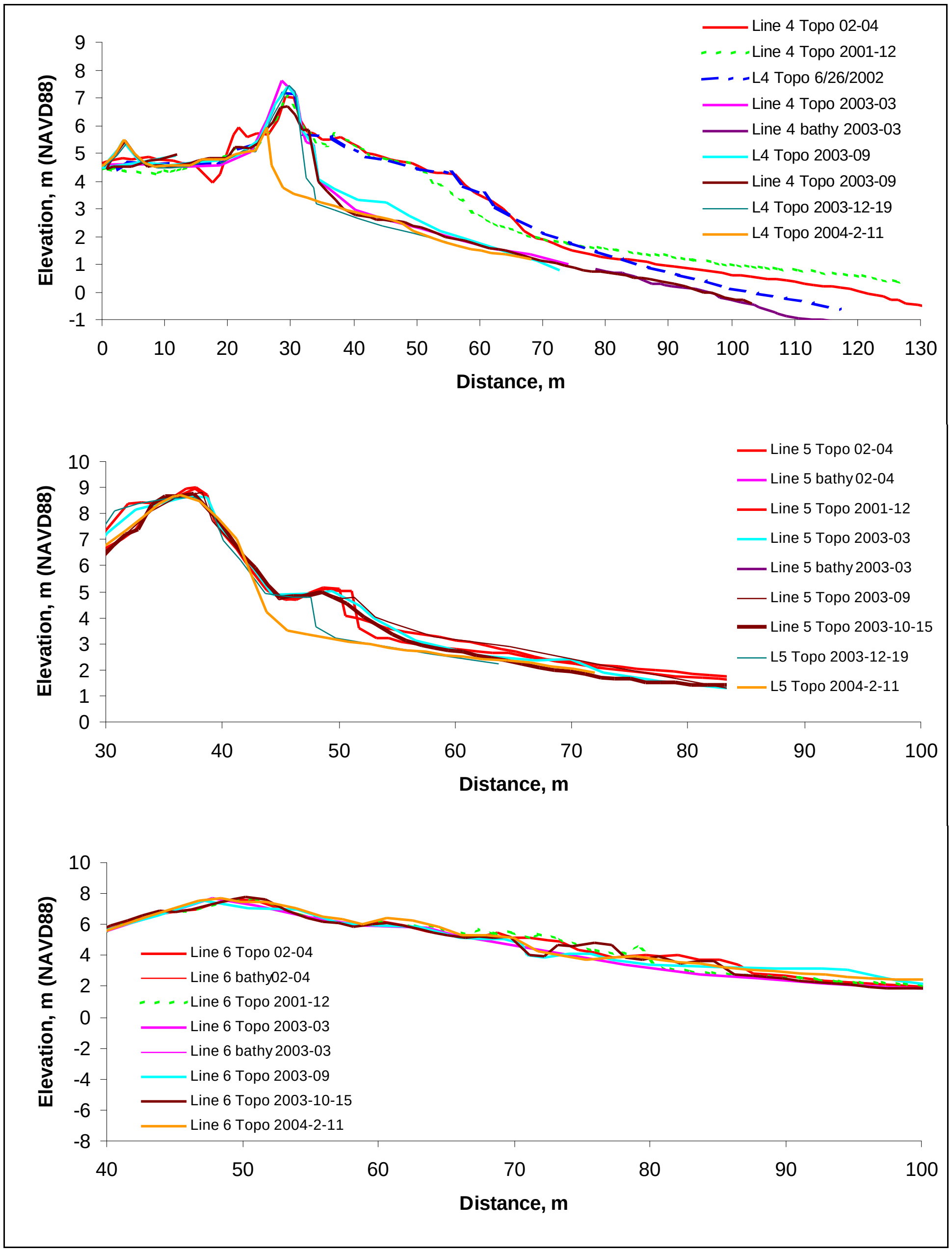

Figure 2-10. Sheet 2 of 3

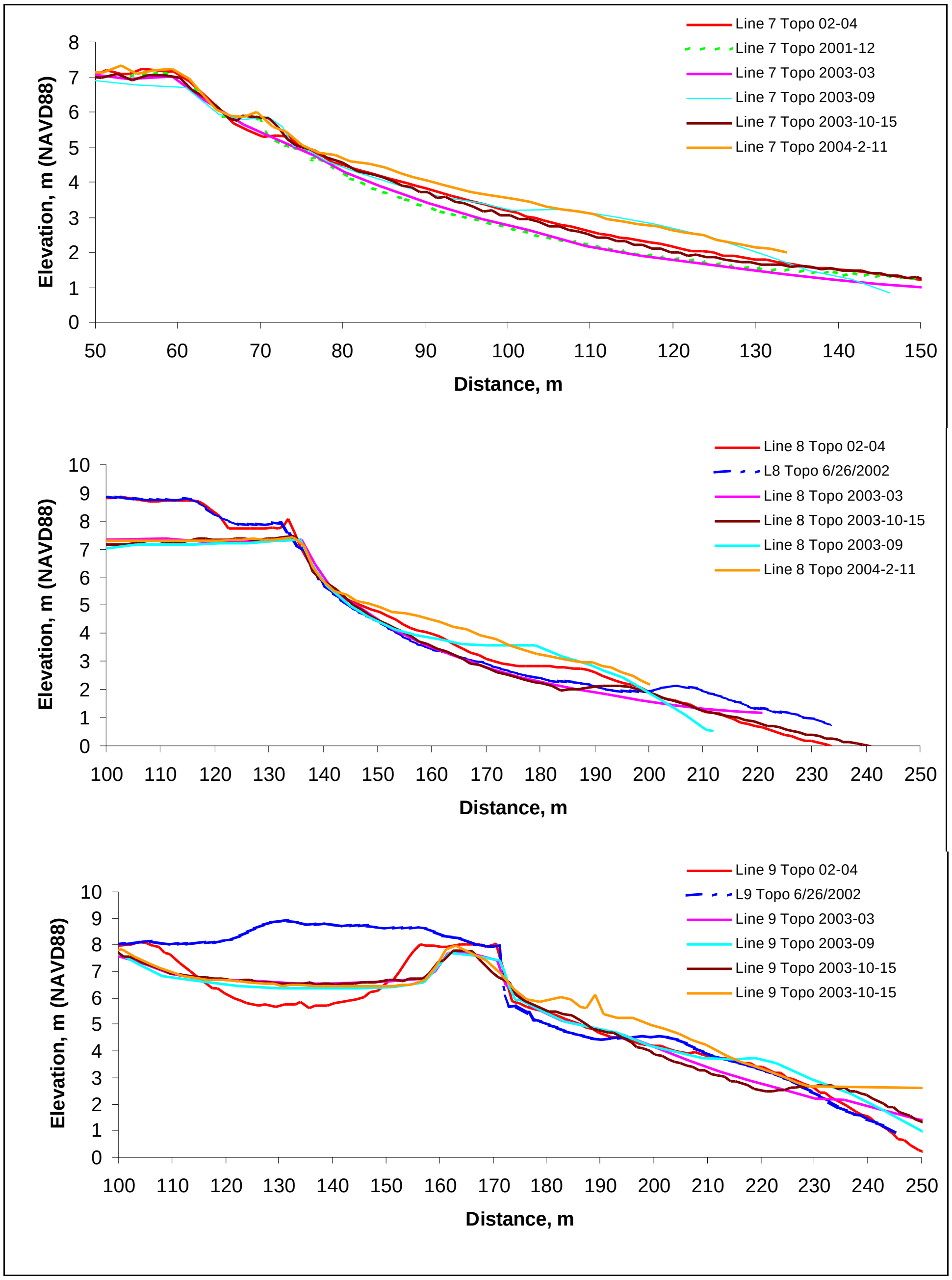

Figure 2-10. Sheet 3 of 3

In response to the erosion in October 2003, the City of Westport installed temporary erosion protection consisting of a double layer of concrete “ecology” blocks, approximately 7,000 cu yd of sand fill, and a geotextile fabric to help retain the sand fill. A series of high spring tides occurred between 3 December and 13 December 2003 that caused some further erosion of the beach scarp and resulted in damage to the temporary erosion protection work installed by the City of Westport. Figure 2-11 is a photograph of the temporary erosion protection on 23 October 2003 shortly after installation. Figure 2-12 shows the condition of the temporary protection on 6 December 2003 during the series of high spring tides.

Some end-effect erosion of the scarp and damage to the shore protection occurred despite the moderate to small ocean wave heights that were prevalent during the high tides in this interval. The damage is likely to have been more severe had the high tides been coincident with large ocean wave heights. On 8-9 December 2003, the City of Westport reconstructed the ecology blocks in a new alignment about 15 feet shoreward of the first placement. Only three days later, the center of the temporary erosion protection began to fail. Figure 2-13 shows the condition of the temporary erosion protection on 11 December 2003.

Although the temporary structure was damaged and caused end-effect erosion, the structure was successful in preventing extensive and deep scarp recession and loss of public facilities. On 5 February 2004, a federal court judge lifted a temporary restraining order and permitted the Seattle District to place approximately 25,000 cu yd of sand on the shoreline. The sand was obtained from the Point Chehalis stockpile. An additional 5,000 cu yd was placed on the breach fill adjacent to the South Jetty on the south beach side of the fill. The profile surveys at HMB 3 for 19 December 2003 and 11 February 2004 in Figure 2-10 include these emergency sand placements by the City of Westport and the Seattle District. The sand placements were intended as temporary measures to protect the Half Moon Bay shoreline, and public facilities including a footpath, jetty, State park access road, and public restroom from erosion related damage. The 25,000 cu yd placement temporarily restored the shoreline to the March 2003 position.

Erosion decreases with distance eastward along the Half Moon Bay shoreline to HMB6 where the beach profile has been relatively stable since 2002. Seasonal fluctuations in profile position and some accretion have occurred at HMB7 through 9 but no significant trend of net erosion is evident along this Point Chehalis section of the beach in the last 2 to 3 years. The longshore trend in profile change suggests sediment is transported from west to east along the shoreline.

Figure 2-11. Temporary erosion protection, 23 October 2003

Figure 2-12. Temporary erosion protection, 6 December 2003

Figure 2-13. Temporary erosion protection, 11 December 2003

In addition to analysis of topographic data along the Half Moon Bay shoreline, topographic data and change has been analyzed in the breach area and adjacent shoreline along South Beach. Sultan and Osborne (2003) analyzed profile measurements from transects on South Beach and surface map surveys collected by the Southwest Washington Coastal Erosion Study. The northern end of South Beach shows a trend of erosion. The erosion is at a maximum (approximately 5 m/year) immediately adjacent to the South Jetty, transitioning to a more stable shoreline position approximately 3,500 ft south of the jetty.

Note that the Seattle District maintains the shoreline at this location with direct sand placements above mhhw, and also with nearshore disposal of dredged sediment. As a consequence, the short-term trend in shoreline position within 500 m of South Jetty is nearly stable. A trend analysis since 1940 by Sultan and Osborne (2003) estimates that 20,000 cu yd/year of beach nourishment are necessary to maintain the beach volume above mllw within 3,500 ft of the South Jetty. The trend analysis by Sultan and Osborne does not include profile changes below mllw. Bathymetry volume changes below mllw between 1955 and 2001 reveal erosion of 500,000 cu yd/year over the same approximate length of shoreline and between depth contours –4.5 m and –21 m, mllw. While the volume change below mllw is larger than the volume change above mllw, the surface area is also larger. The bathymetry changes below mllw are equivalent to 0.13 ft/year while the changes above mllw are equivalent to 0.21 ft/year (total loss per sq ft/year).

Beach Planshape Analysis

In addition to analyzing shoreline change at transect locations, shoreline planshapes at Half Moon Bay derived from aerial photography have been analyzed using equilibrium beach planform analysis. This work involves fitting the existing beach planshape to the analytic planshapes predicted using the method of Hsu and Evans (1987) and then interpreting the effects of the gravel transition fill on expected future equilibrium beach planshapes.

At the western-most limit of the bay where the beach is composed of the gravel transition fill, the beach planshapes observed at Half Moon Bay do not follow the equilibrium planshapes predicted by the Hsu and Evans technique. The oval area highlighted in Figure 2-14 is the location of the gravel transition fill. The shoreline position is east of the predicted equilibrium position in this area. The shoreline makes an abrupt change in direction to conform with the equilibrium shape at the south end of the transition region in the southwest corner of the bay.

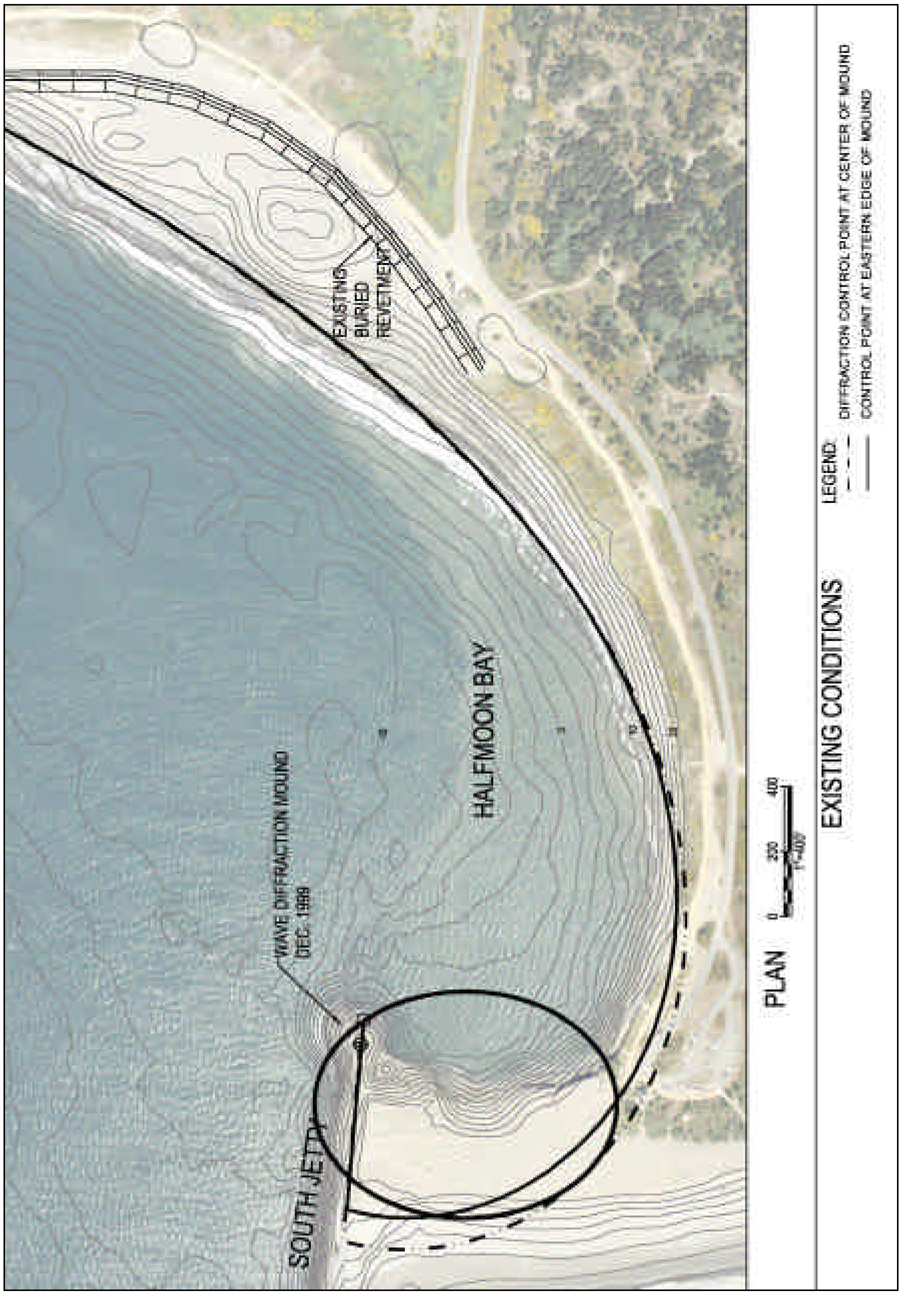

Figure 2-14. Half Moon Bay equilibrium shoreline planshapes

It is reasonable to expect that a beach composed only of fine-grained sand would follow a planshape similar to that marked by the dash-dot and solid lines in Figure 2-14. However, the transition gravel fill has reduced erosion and has prevented the shoreline from developing the predicted curved planform. The existing beach shape represents a balance between sediment mobility and equilibrium planshape. The equilibrium planshape is the beach shape that would be expected to occur if all sediments on the beach were mobilized by wave action. For a typical sand beach, the resulting planform is one where the combined effects of refraction and diffraction result in a near-zero angle of incidence of waves all along the beach. In the case where gravel and cobble materials are also present in significant quantity, the transport threshold is also a decisive factor. Generally, sand is mobile anywhere breaking waves are present. However, this is not necessarily the case for gravel. The implication is that the sediments in the gravel transition zone are below threshold under typical waves and therefore would not be expected to move toward the equilibrium planshape.

The two limiting processes of sediment mobility and equilibrium planshape work together to control the beach shape at Half Moon Bay. The eastern portion of the bay is composed of fine sand and appears to conform well to the predicted equilibrium planshape. However, the western portion of the bay is composed of gravel and cobble and is therefore controlled by sediment mobility. These relationships are examined in more detail in the next sections of this chapter and in Chapters 4 and 5 of this report.

Waves, Currents, and Suspended Sand Transport

The directional wave measurements provide data useful for verification of the numerical and physical model. Direct comparisons between the field measurements and models are described in Chapters 3 and 5. Direct measurements of waves heights, periods, and directions, wave orbital velocities, steady currents, and suspended sediment concentrations are also useful for elucidating sediment transport paths in the bay, for interpretation of gravel and cobble transport data and morphological changes leading to the development of a sediment budget.

Field measurements obtained between 9 December 2003 and 19 February 2004 are described in detail in Appendix A. Figure 2-8 shows the location of the instrumented tripods deployed in Half Moon Bay.

a. Waves. Time series of significant wave height, H_s, measured at the 4 inshore stations (Stn HM1 to HM4) and the Coastal Data Information Program (CDIP) Buoy 3601 located 5/8 nautical miles southwest of Grays Harbor Entrance at 41.5 m depth are shown in Figure 2-15. The CDIP record indicates that offshore H_s was greater than 7 m in early December 2003. Figure 2-16 shows monthly average H_s, T_p, and Dir based on measurements at the CDIP buoy 3601. Monthly average H_s, T_p, and Dir for the 2003-2004 deployment interval are also shown in Figure 2-16 illustrating that the waves during the measurement interval are representative of the long term statistics.

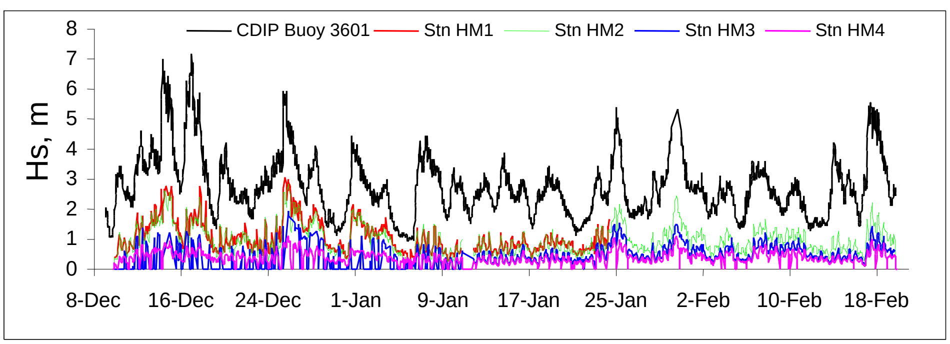

Figure 2-15. Time series of H_s measured at Stn HM1 to HM4 and the CDIP Buoy 3601 during the deployment interval in 2003-2004

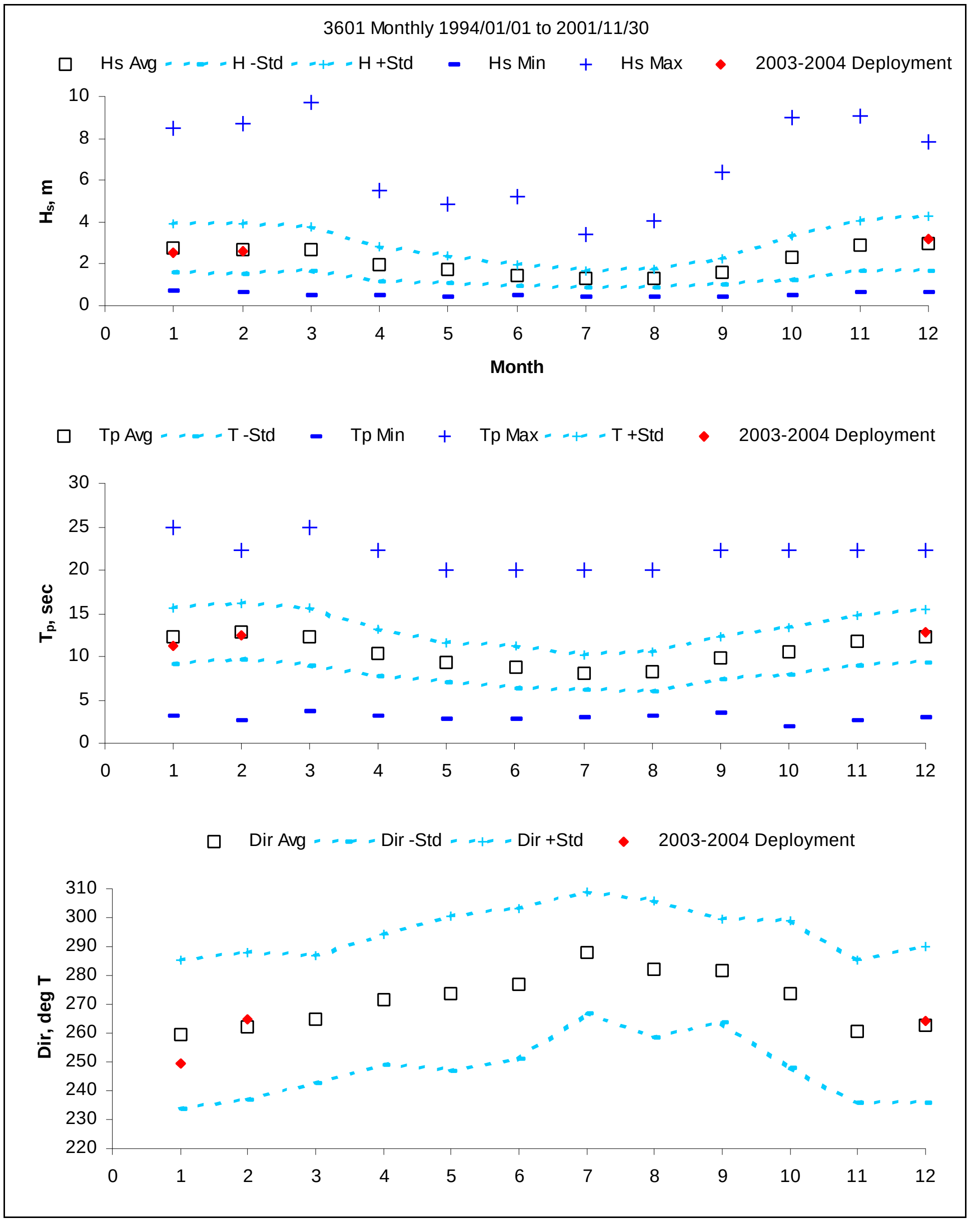

Figure 2-16. Monthly statistics of Hs, Tp, and DIR from 1994-2001 at CDIP Buoy 3601 compared with statistics for the deployment interval in 2003-2004

Wave roses derived from time series of H_s and peak wave direction are shown in Figure 2-17. Waves measured at the CDIP buoy 3601 show most of the ocean waves during the deployment arrived from the direction band between 270 to 292.5 degrees. Most of the larger storm waves also occurred in this band. A smaller percentage of waves arrived from the southwest between 211.5 and 270 degrees.

Waves measured at Stn HM1 occur within the range between 270 and 315 degrees with slightly more energy in the band between 292.5 and 315 degrees. There is a significant reduction in wave energy between the CDIP buoy and Stn HM1. Wave height is reduced by a factor of approximately 2 to 3 between the

CDIP and Stn HM1. There is no significant reduction in wave height between Stn HM1 and HM2. At Stn HM2, the waves occur almost entirely within the 292.5 to 315 degree band. The predominant angle of approach continues to shift progressively more to the north at Stn HM3 and HM4 while the wave energy also declines at these locations. The systematic rotation of wave approach angle and reduction in wave height evident in the directional roses illustrates both the refraction of waves as they enter shallower water approaching Half Moon Bay (Stn HM1 to HM2) as well as the effects of wave diffraction induced by the eastern terminus of the South Jetty (Stn HM2 to HM4).

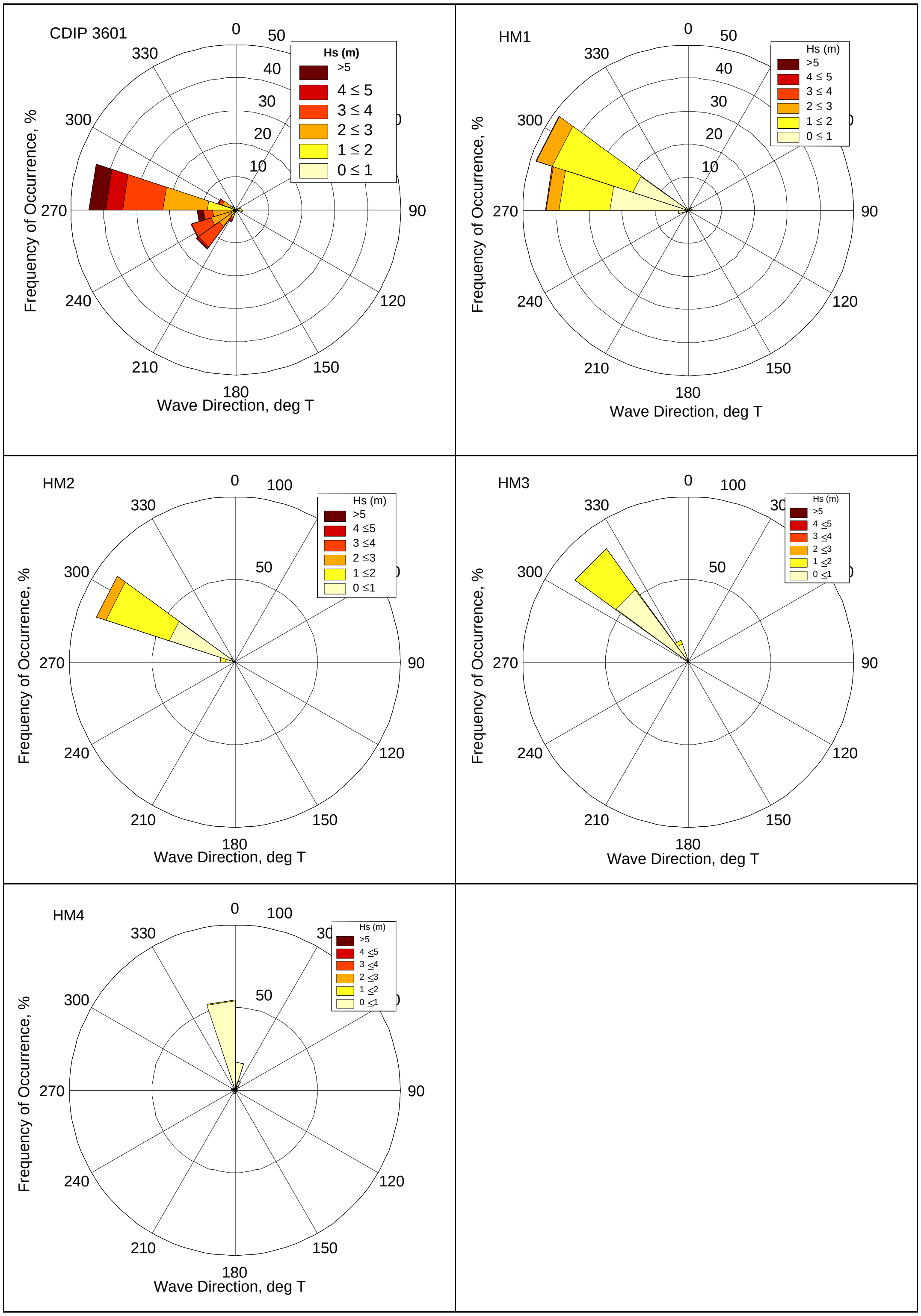

Figure 2-17. Wave roses from wave gauges deployed in Half Moon Bay and the CDIP Buoy 3601 for the deployment interval (9 December 2003 through 10 January 2004)

b. Currents. Time series of near bed current vectors at Stn HM1 through 4 during a storm between 12 December 2004 and 18 December 2004 are shown in Figure 2-18. Corresponding time series of Hs and water depth, h, at Stn HM1 are shown in Figure 2-19. The near bed mean current at Stn HM1 is dominated by tidal forcing and oscillates between east-northeast and west-southwest on flood and ebb tide, respectively. Peak ebb currents are capable of eroding sand from the bed and wave orbital velocities at this location are almost always large enough to ensure that fine sand particles are mobile at most times during the tidal cycle. The current at Stn HM2 is tide modulated but current speeds also increase and decrease with wave height at this location. The current at Stn HM2 is directed predominately alongshore to the north towards Point Chehalis and the navigation channel. At Stn HM3, the mean current is mainly wave dominated and directed predominately offshore and alongshore to the north and north west. Mean currents at Stn HM4 are tide modulated, relatively weak (typically < 0.25 m/sec) and directed approximately east-west (normal to shore). During periods of larger waves, the southerly (shore parallel) component of current increases at Stn HM4.

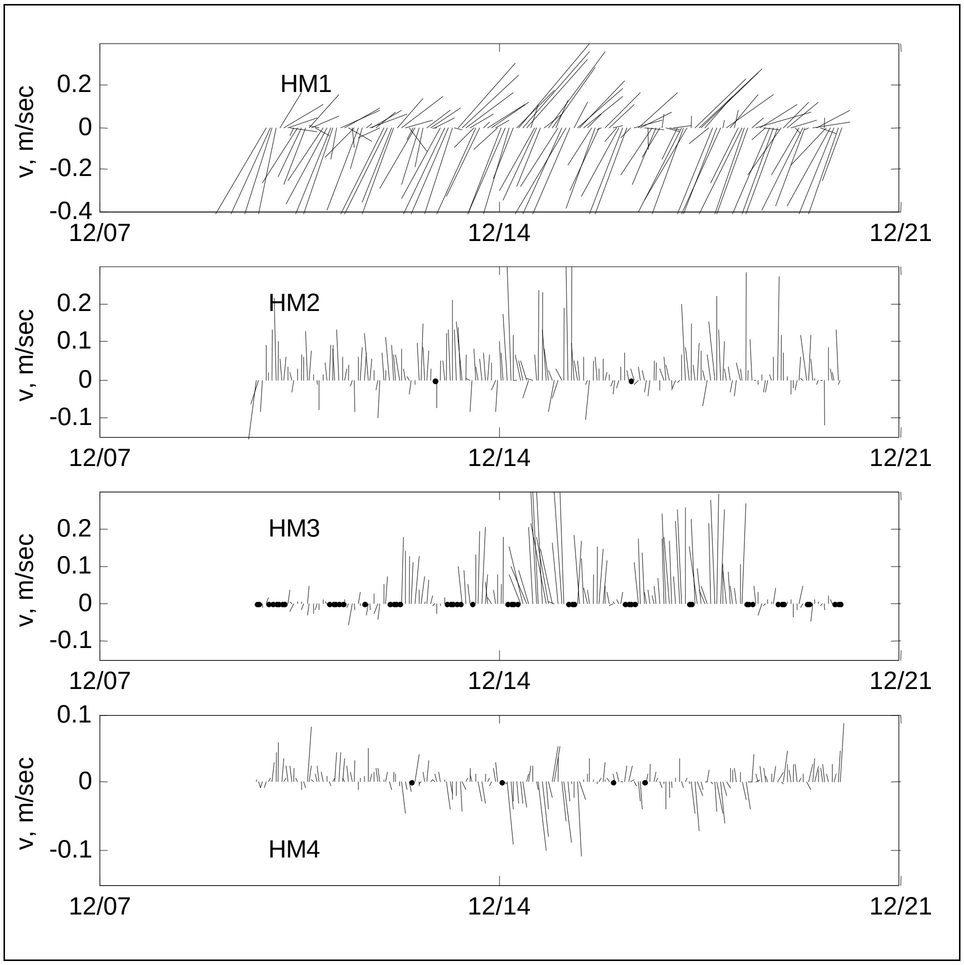

Figure 2-18. Time series of current vectors at Stn HM1 through HM4 between 12 December 2003 and 18 December 2003

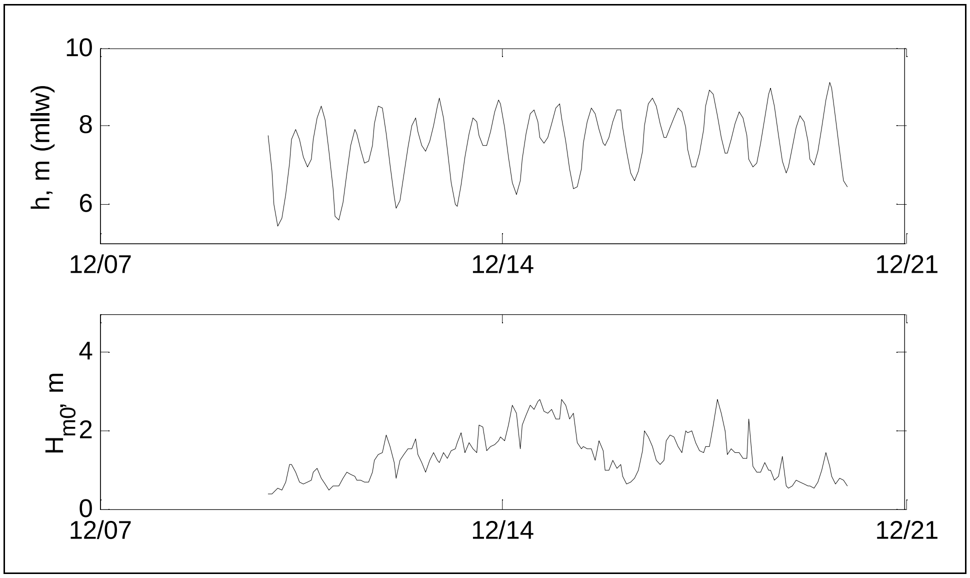

Figure 2-19. Measured h and Hs at Stn HM1 between 12 December 2003 and 18 December 2003

c. Suspended Sand Transport. Time series of near bed suspended sediment concentration, *SSC*, at Stn HM2 through HM4 corresponding with the large waves occurring between 12 December 2003 and 18 December 2003 are shown in Figure 2-20. Comparison with Figure 2-19 reveals that SSC at Stn HM2 and HM3 is strongly correlated by wave height whereas the SSC at Stn HM4 in the lee of the diffraction mound is correlated with tidal current and more weakly correlated with wave height than at the other locations. Multiplying the *SSC* with mean current vectors provides a point estimate of the suspended sediment flux. Suspended sediment flux roses compiled for the period 9 December 2003 to 10 January 2004 are shown in Figure 2-21. The transport associated with mean currents is consistent with mean current patterns and is predominately alongshore to the north at Stn HM2 and Stn HM3 and shore perpendicular at Stn HM4. It is likely that waves also contribute significantly to the net transport at each of these locations.

The sediment transport patterns revealed by the most recent process measurements from Half Moon Bay are consistent with the previous conceptual model developed by Osborne et al (2003). Wave refraction and diffraction in the bay create alongshore and cross-shore currents. The longshore current adjacent to the Half Moon Bay shoreline typically flows from the west end of the bay to the northeast. An off-shore component of current is usually also present in the nearshore under breaking waves and this results in a north and north-west directed mean current in the nearshore along the shoreline. The longshore current transports suspended sand entrained by wave action to the north, out of the bay where it is entrained by tidal currents.

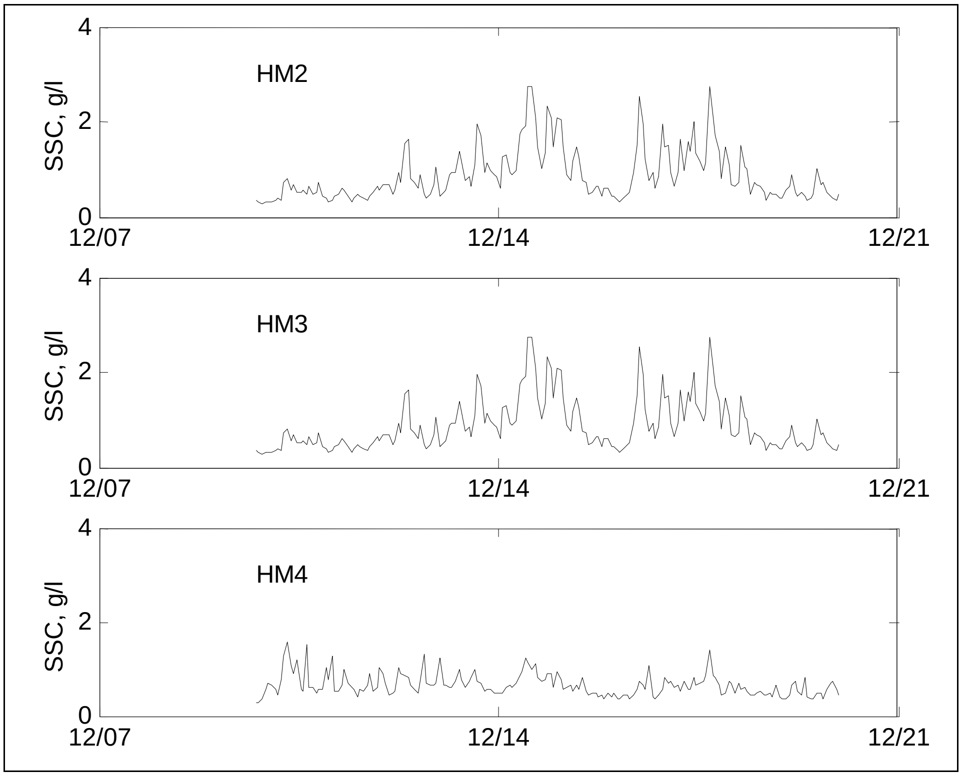

Figure 2-20. Time series of near bed suspended sediment concentration, *SSC*, 12 December 2003 to 18 December 2003

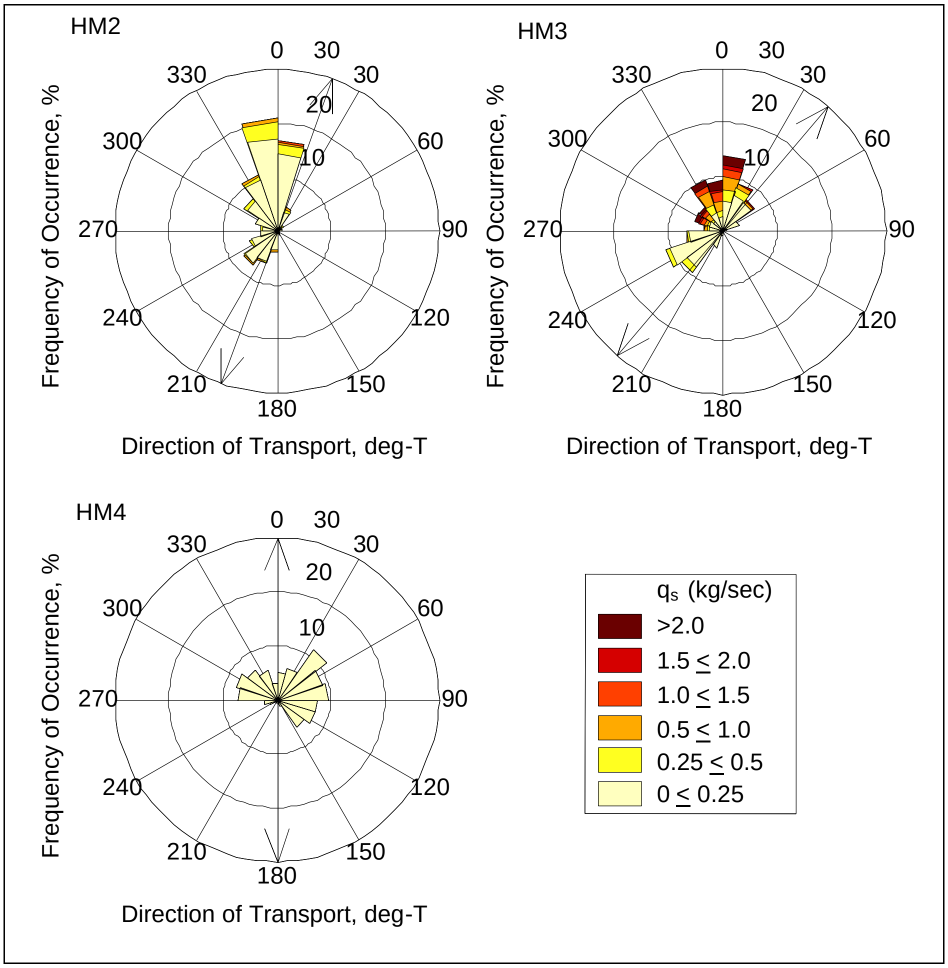

Figure 2-21. Suspended sediment flux roses for Stn HM2 through HM4 for the period 9 December 2003 to 10 January 2004

Cobble and Gravel Transport

A series of direct measurements of gravel and cobble particle transport on the transition beach at Half Moon Bay were collected during two sets of particle tracer measurements. The tracer measurements permit an assessment of cross-shore and alongshore transport modes and patterns and an assessment of the stability and mobility of the transition cobble and gravel beach. The particles are tracked by placing a magnet inside the particle to enable finding the particle with a metal detector. The location of the particles is then mapped over time using survey equipment.

Modes and mechanisms for transport of coarse bedload material, including gravel and cobble sized particles, on beaches under waves are poorly understood. Predictions of gravel and cobble transport are often unreliable mainly because of a lack of high quality data with which to develop, test, and verify sediment transport formulae. Therefore, the field data is valuable for analyzing the transport pathways and fate of gravel and cobble, and for evaluating the performance of the gravel transition fill.

The first set of tracer measurements was conducted during two successive high tides between 17 December and 19 December 2003. The second set of measurements was conducted over four successive high tides between 9 February and 13 February 2004. Details of the sampling methodology for bed material characterization, particle tracer preparation, tracer placement and recovery are described in Appendix B. Figure 2-22 shows the placement location of tracer particles during the two tracer deployments on 17 December 2003 and 9 February 2004. The rectangles in Figure 2-22 show the initial positions of the particles. Each particle was placed on a grid, equally spaced within a 6.1 m (20 ft) by 7.3 m (24 ft) approximately sized matrix. During the first deployment, a single set (Set 1) of thirty tracer particles (five particles in six size classes) was deployed. During the second deployment two sets (Sets 2 and 3) of thirty tracer particles were deployed.

The measured wave height, period, and direction during the two tracer deployments are summarized in Tables 2-3 and 2-4. Waves during the deployments are characteristic of average winter conditions. However, the wave heights and periods were greater during the first deployment, which contributed to increased particle displacement.

Table 2-3
Average Wave Parameters During Particle Tracer Analysis: December 17-19, 2003

Station	Average Hs, m	Average Tp, sec	Average Dp, deg-T
CDIP	3.38	15.5	268.9
HM1	1.23	14.6	279.7
HM2	1.14	14.3	294.6
HM3	0.73	17.2	320.3
HM4	0.45	16.6	166.3

Table 2-4
Average Wave Parameters During Particle Tracer Analysis: February 9-13, 2004

Station	Average Hs, m	Average Tp, sec	Average Dp, deg-T
CDIP	2.01	12.8	271.1
HM2	0.87	13.0	291.0
HM3	0.57	13.1	312.2
HM4	0.38	12.9	309.4

Figure 2-23 shows the particle paths for transport that occurred during the two successive high tides between 17 December and 19 December 2003. The transport paths indicate a net transport alongshore to the south and east. The net alongshore transport is generally greater than the net cross-shore transport. Figures 2-24 and 2-25 show the particle paths for transport that occurred during the second tracer deployment between 9 February and 13 February 2004. The different particle sizes are listed in Table 2-5.

Table 2-5
Tracer particle size classes and characteristics

Size Class		**Average size of particles in class**					**Description**
		L, mm	**I, mm**	**S, mm**	**M, kg**		
Set 1	1	126	107	69	1.471	-6.7	Large Cobble
	2	91	68	44	0.414	-6.1	Medium Cobble
	3	70	52	38	0.201	-5.7	Small Cobble
	4	49	37	20	0.055	-5.2	Pebble - Cobble
	5	38	29	17	0.030	-4.8	Large Pebble
	6	32	20	14	0.012	-4.3	Small Pebble
Set 2	1	134	108	67	1.392	-6.7	Large Cobble
	2	93	69	38	0.376	-6.1	Medium Cobble
	3	66	50	31	0.152	-5.7	Small Cobble
	4	52	38	22	0.061	-5.3	Pebble - Cobble
	5	42	30	20	0.039	-4.9	Large Pebble
	6	33	21	13	0.013	-4.4	Small Pebble
Set 3	1	138	108	74	1.732	-6.8	Large Cobble
	2	103	79	41	0.453	-6.3	Medium Cobble
	3	81	54	36	0.228	-5.8	Small Cobble
	4	50	39	22	0.069	-5.3	Pebble - Cobble
	5	39	29	21	0.034	-4.8	Large Pebble
	6	26	19	12	0.008	-4.3	Small Pebble

L - length of long axis, I - length of intermediate axis, S - length of short axis, M - mass, ϕ - phi-size of intermediate axis, Description - Udden-Wentworth Classification

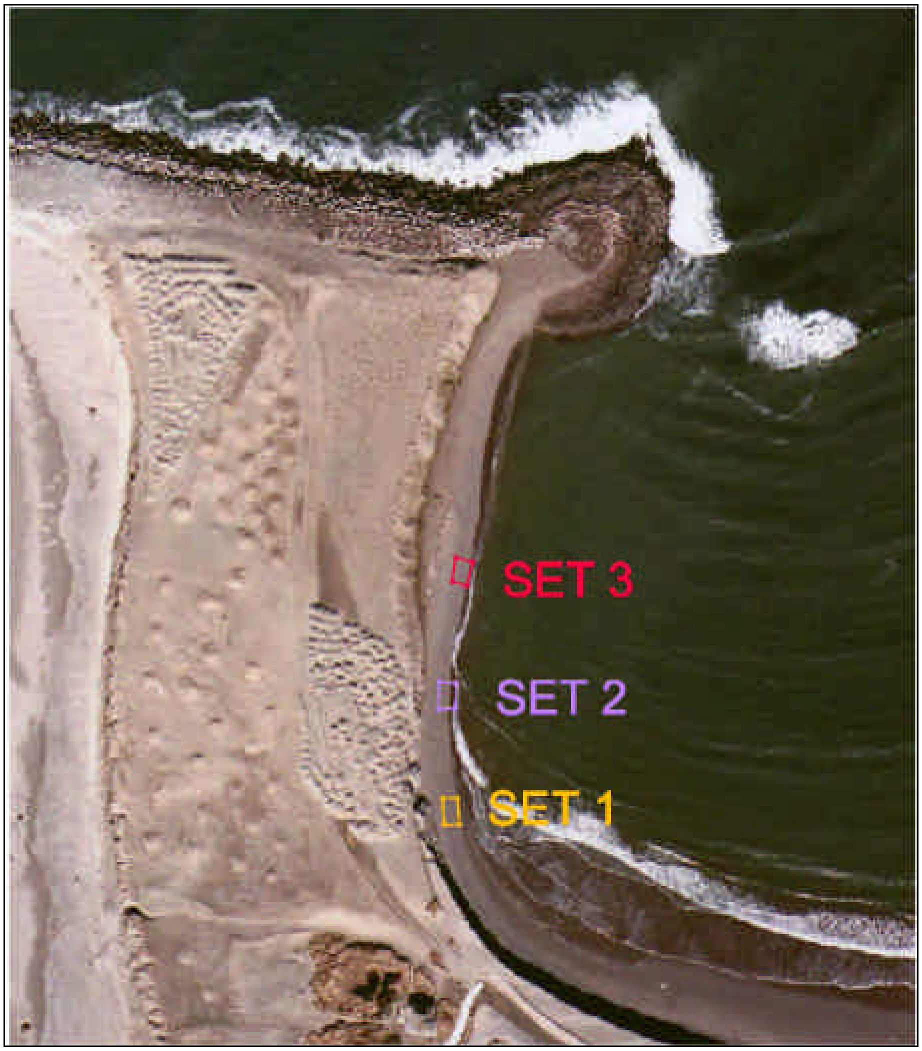

Figure 2-22. Initial location of tracer particles

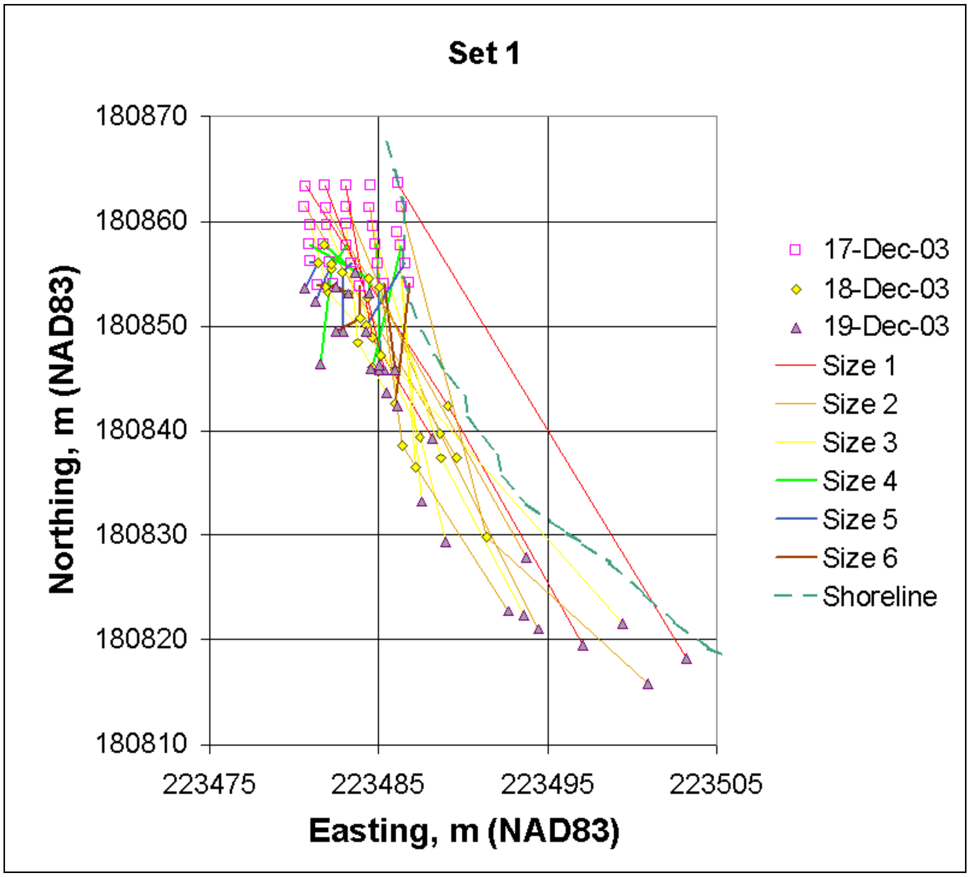

Figure 2-23. Particle transport paths for Set 1 (17 December 2003 to 19 December 2003)

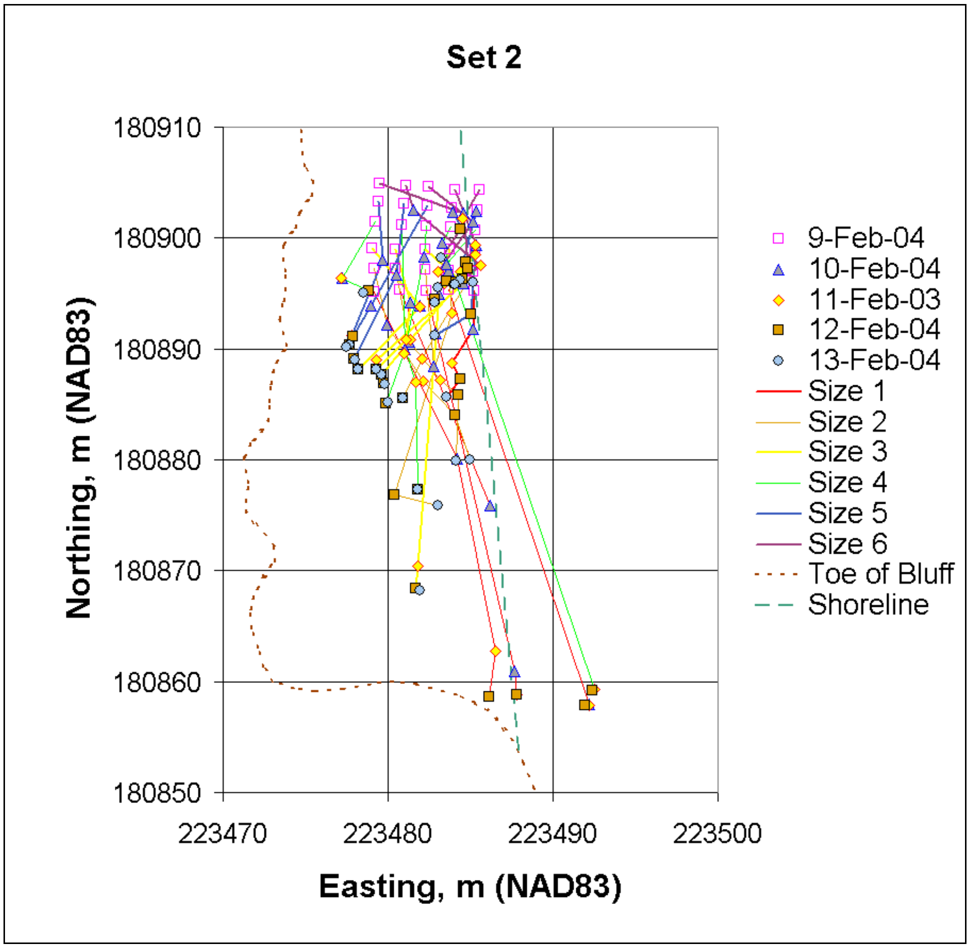

Figure 2-24. Particle transport paths for Set 2 (9 February 2004 to 13 February 2004)

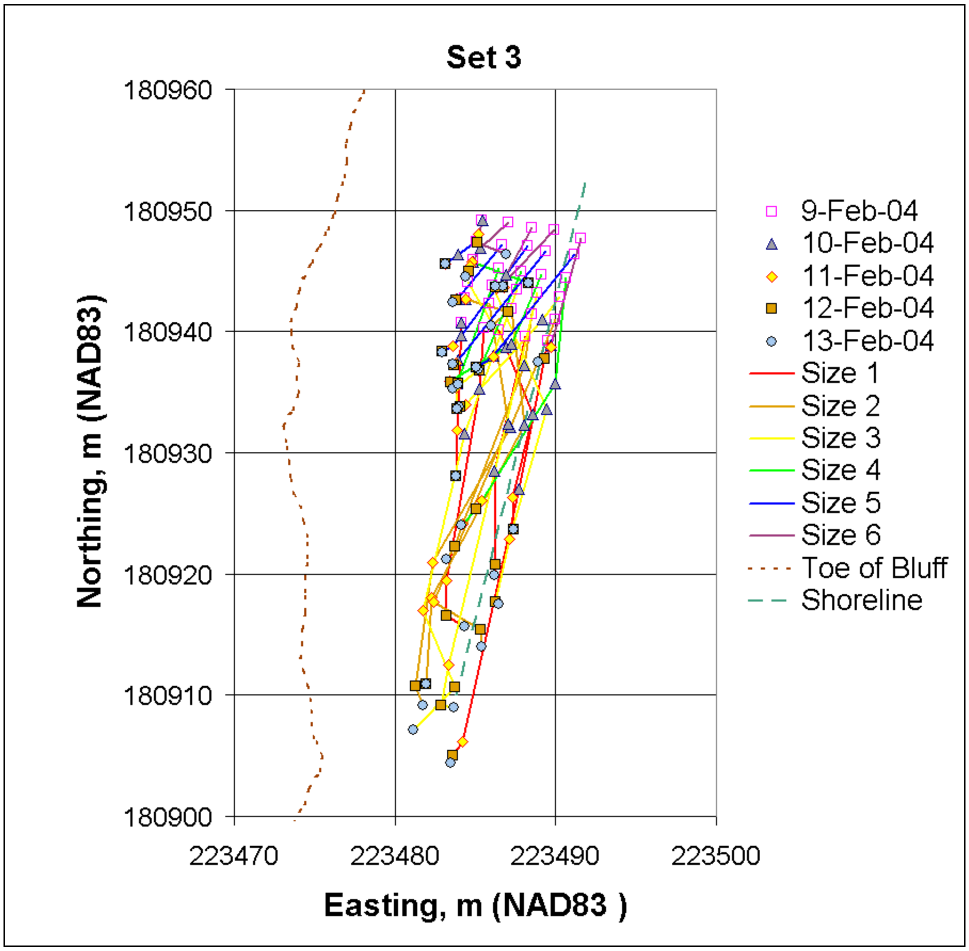

Figure 2-25. Particle transport paths for Set 3 (9 February 2004 to 13 February 2004)

Figure 2-26 shows the mean particle transport distance per diurnal tidal cycle as a function of the particle mass. The particle transport distance is an inverse function of particle size (or mass), with larger particles tending to be transported greater distances. This counter-intuitive relationship may reflect the selective entrainment of larger particles, which are more exposed to fluid forces on the surface than smaller particles, which are sheltered within the matrix of larger particles. At a point between the 0.2 kg (54 mm) and 0.4 kg (70 mm) size particles, the particle transport rate begins to decrease with increasing size most likely reflecting the decreased capacity of the fluid forces to transport larger and heavier particles.

Figure 2-27 shows the mean depth of burial of particles as a function of particle mass. The results are consistent between sets and illustrate that the smaller particles are more susceptible to burial than the larger particles. The smaller particles in Set 1 were buried to greater depth than similar sized particles in Set 2 and 3 most likely owing to the more energetic waves during the Set 1 measurements. The large burial depth associated with the largest particle size in Set 2 is an outlier from the remainder of the data set. The largest particles in Set 2 were transported into the new beach fill area and were deeply buried in the sand fill.

The exposed cobble and gravel particles on the surface of the transition beach are relatively mobile alongshore under average winter waves. The particle transport results suggest a significant mass flux of cobble is occurring out of the transition beach area to the east along the Half Moon Bay shoreline. This mass flux is confirmed by visual observations and surveys of beach profiles in the transition beach area. Beach profiles (Figure 2-10) in the transition beach indicate a net loss of cobble-gravel of 9.4 m^3/m of beach above mllw between mid-October and mid-December 2003. Visual observations during the same interval indicated that cobble migrated east from the transition beach area and was deposited at the base of the temporary erosion protection installed by the City of Westport.

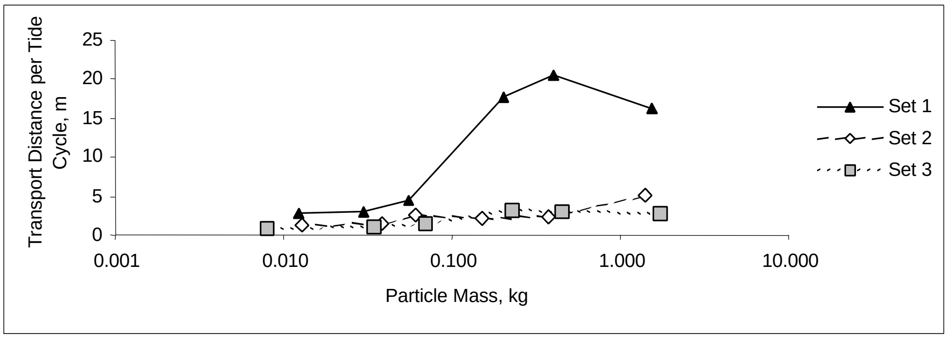

Figure 2-26. Transport distance per diurnal tidal cycle as a function of particle mass

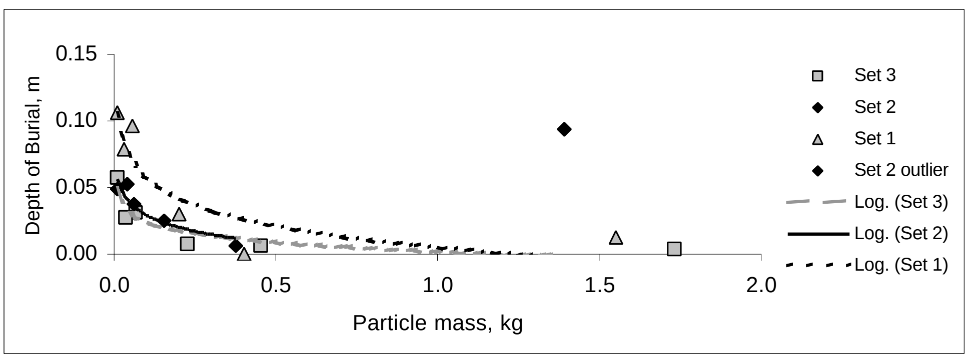

Figure 2-27. Depth of burial as a function of particle mass for Sets 1, 2 and 3

Figures 2-28 to 2-30 show the daily average transport distance per diurnal tidal cycle for each particle size. Figures 2-28 and 2-30 clearly illustrate the significant transport distances associated with the 3 largest particle sizes for Sets 1 and 3. Figure 2-26 shows that the largest particle sizes were subject to much higher transport distance per diurnal tidal cycle in the Set 1 deployment owing to greater wave energy during this interval. Overall, the three largest sizes showed significantly higher transport rates than the three smallest sizes.

The differentiation of transport distance with particle size is not as evident for Set 2 as it is for Sets 1 and 3 (Figure 2-28). During the deployment interval, the Seattle District began beach replenishment operations in the area adjacent to Set 2. The larger particles of Set 2 exhibited similar transport rates compared to the other sets between 9 February 2004 and 11 February 2004. However, when encountering the freshly deposited sand they were immediately subject to burial. While locating the particles on 12 and 13 February 2004, it was common to find the larger particle buried at depths of 0.2 m (9 in) to 0.4 m (15.75 in). The location of the beach nourishment can be described with reference to Figure 2-24. In Figure 2-24, the shoreline and toe of bluff are primarily in a north-south orientation until approximately 180860 Northing, m (NAD83). At this point, the toe of the bluff changes direction approximately 90 degrees and intersects the shoreline. The outcropping of the bluff toe is the area where the new beach fill was placed in February 2004.

The particle trajectories in Figure 2-24 reveal that the larger particles did not transport beyond the sand fill area on the final two days of measurements. These particles were therefore filtered from the analysis for 12 February 2004 and 13 February 2004 because of the interaction between the freshly deposited sand and the particle transport.

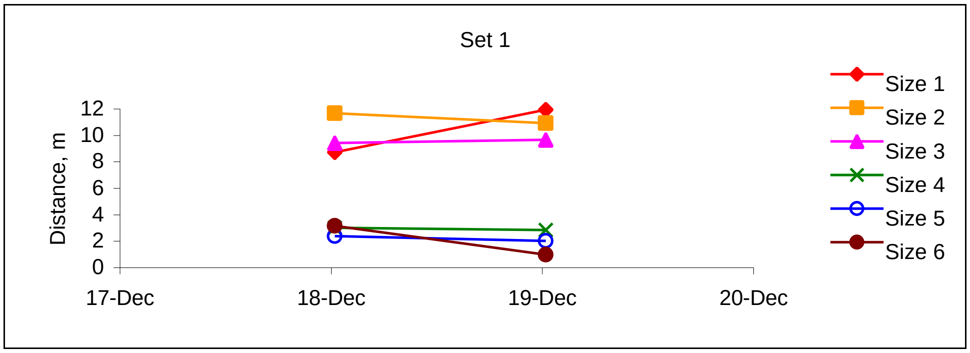

Figure 2-28. Daily average transport distance per diurnal tidal cycle - Set 1 (17 December 2003 to 19 December 2003)

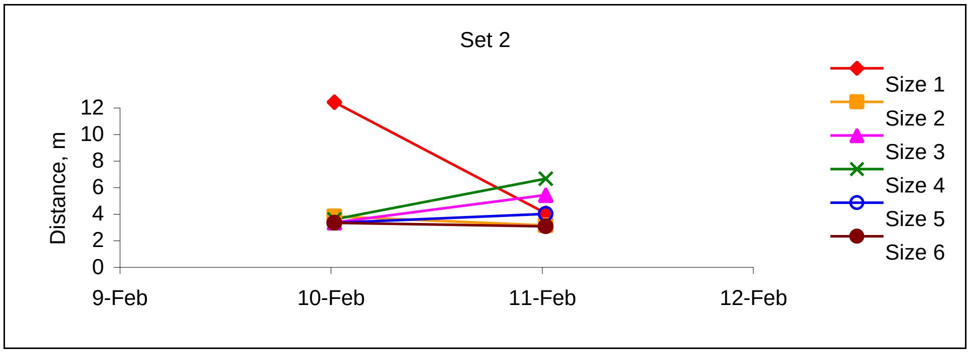

Figure 2-29. Daily average transport distance per diurnal tidal cycle Set 2 (9 February 2004 through 13 February 2004)

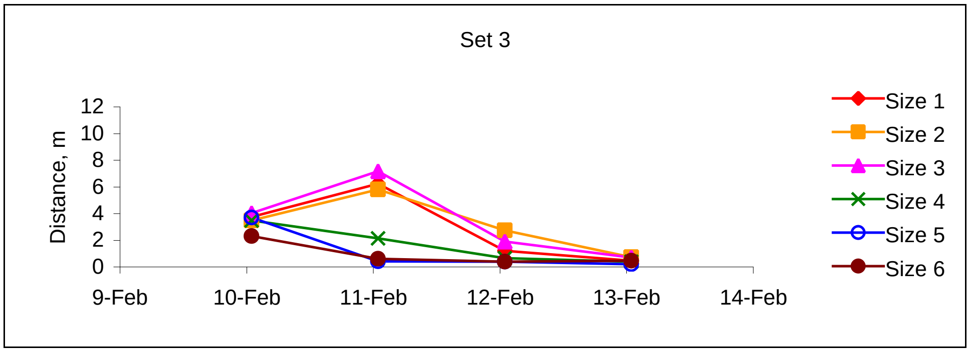

Figure 2-30. Daily average transport distance per diurnal tidal cycle Set 3 (9 February 2004 through 13 February 2004)

The angle between the transport direction and the relative shoreline was calculated using Cartesian vector geometry to analyze the relationship between particles mass and transport direction. The angle was calculated for each particle, then averaged for each particle size over the respective periods. Figure 2-31 shows the relationship between particle trajectory angle and particle mass. An apparent trend is evident in which the particles of larger mass tend to move long-shore more than the smaller particles, while the particles with smaller mass have more tendency to move in the cross-shore direction.

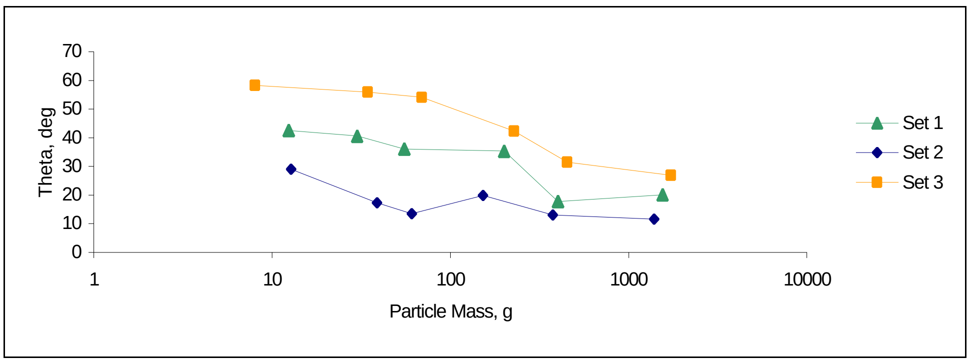

Figure 2-31. Particle trajectory angle with the shoreline as a function of particle mass (9 February 2004 through 13 February 2004)

Sediment Budget

A preliminary sediment budget has been developed for Half Moon Bay based on beach profile data, bathymetric surveys, and dredging records for the interval 1996 to 2004. The results are shown in Figure 2-32. The sediment budget is developed by calculating the volume change in each of the cells shown in Figure 2-33.

The beach profiles within cells "Shore_1" to "Shore_5" are analyzed by calculating the difference in cross-sectional area between successive profile surveys. The distance between profiles is then applied to calculate the volume change. This method is an "average end areas" type of calculation. Adjustments and extrapolations have been made to account for the non-uniform coverage of the profiles and the irregular shape of the budget cells.

The volume change in cell “Nearshore_HMB” is calculated by gridding nearshore bathymetry surveys acquired by the Seattle District then calculating the volume difference between the surfaces. The volume changes in cell “Inner HMB” are calculated using the same data that was analyzed previously and reported in the South Jetty Sediment Processes Study (Osborne, et al, 2003). The volume change in cell “South Beach” is calculated from the results reported in the South Beach Shoreline Change Analysis report by Sultan and Osborne (2003). In addition, sediment placement volumes connected to Grays Harbor dredging are included in the analysis. The volume input to “Breach Fill” is calculated using this data.

The volume changes that are calculated are usually not absolute differences between 1996 and 2004. The time series for each cell typically span different time periods. Instead, the volume change trend over the available data is calculated. Figure 2-34 shows a time series of volume change for each cell, and two trend lines, one for cell Shore_1 and one for cell Shore_2. The trend of cubic meters change per year is multiplied by 8 years to get the inferred volume change for the period 1996-2004. This method has the additional advantage of smoothing the short term fluctuations in some of the time series. The net volume change in each cell is shown in Table 2-6.

The sediment budget patterns and cell divisions are consistent with the sediment budget developed over a larger region near the South Jetty and reported in Osborne, et al (2003). It is clear that the overall sediment transport pattern in this area is erosion along the shoreline to the northeast and north at Half Moon Bay. The sediment is transported through the nearshore cells and ultimately into the Grays Harbor Navigation Channel where it acted on predominately by ebb tidal currents.

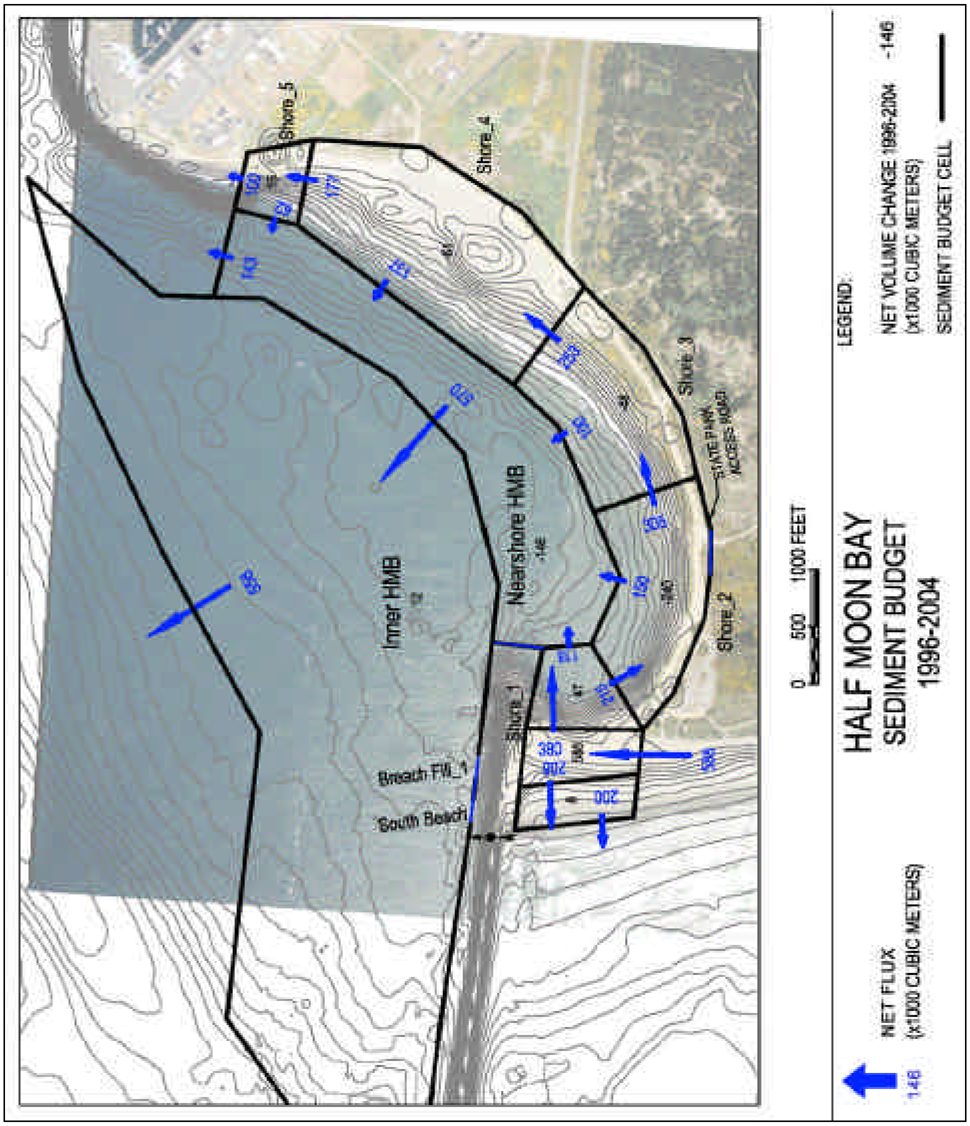

Figure 2-32. Half Moon Bay preliminary sediment budget

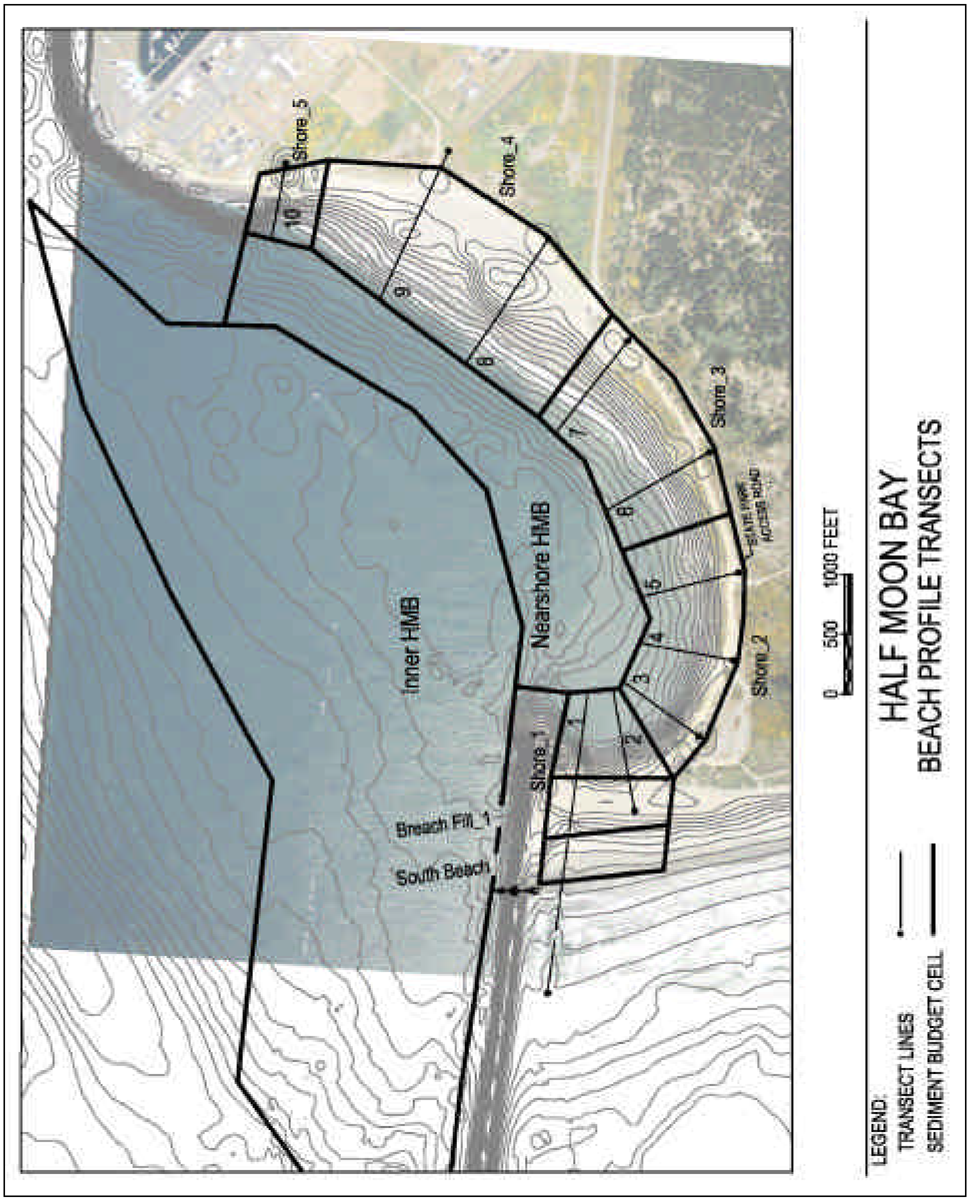

Figure 2-33. Half Moon Bay beach profile transects and sediment budget cells

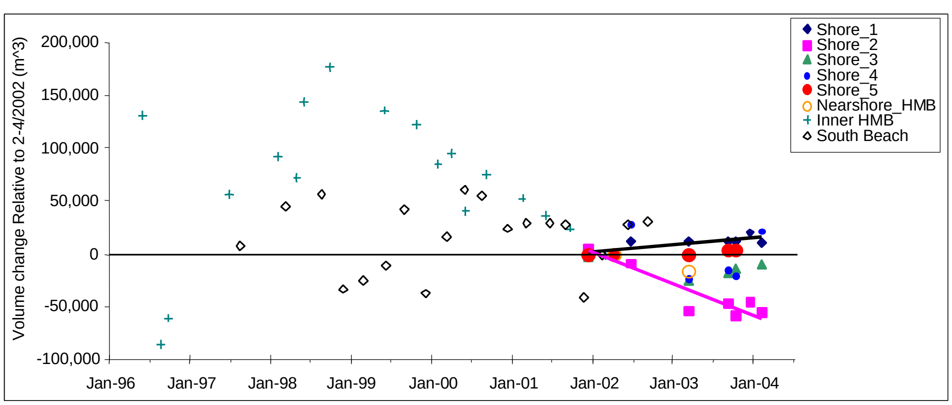

Figure 2-34. Sediment budget cells volume change relative to 15 April 2002

Table 2-6 Net Volume Change in Sediment Budget Cells - 1996 to 2004		
Cell	**Volume Trend (m^3/year)[1]**	**Volume Change 1996-2004 (m^3)[1]**
Shore_1	6,000	47,000
Shore_2	-30,000	-240,000
Shore_3	-6,000	-48,000
Shore_4	-8,000	-61,000
Shore_5	2,000	15,000
Nearshore_HMB	-18,000	-146,000
Inner HMB	1,000	12,000
South Beach	1,000	8,000
Net Change	-52,000	-413,000
Breach_Fill Input	74,000	588,000

[1]Positive is accretion, negative is erosion.

Summary

This chapter has summarized the physical setting and present conditions at Half Moon Bay. Engineering measures have been implemented over the years to prevent the recurrence of a breach between Half Moon Bay and the Pacific Ocean. It is concluded that the efforts taken to prevent re-breaching and to place sediment in a manner beneficial to the Half Moon Bay shoreline have been effective, but are not necessarily an efficient long-term solution. The ongoing erosion in the southwest corner of the bay is an indicator that the diffraction mound and gravel-cobble transition beach are not optimized in terms of performance.

A number of analyses have been conducted to characterize the physical setting and conditions and to evaluate the performance of the engineering measures. Some of the analyses are updates to previous work that is reported in Osborne et al (2003). The history of placement of dredged sediment at Half Moon Bay is summarized. The work includes analysis of upland and inter-tidal topography and nearshore bathymetry surveys and analysis of beach planforms. The movement of gravel and cobbles is measured directly in a field study and a preliminary sediment budget is developed.

Analysis of shoreline positions and beach profile measurements indicate that the erosion rate in the southwest corner of the beach has accelerated since the wave diffraction mound was constructed in 1999. The position of maximum erosion rate shifted approximately 200 to 300 m to the southwest following construction of the wave diffraction mound which also coincided with the removal of 250 ft of jetty rock from the remnant eastern terminus of the jetty. Removal of the jetty remnant effectively shifted the diffraction point 250 ft to the west. The erosion pattern that has occurred in the southwest corner of the beach is consistent with a westward shift in the diffraction control point. The placement of gravel and cobble in the transition beach area has caused the shoreline position to be further east than the predicted equilibrium position according to the model of Hsu and Evans (1987). The shoreline abruptly changes direction to join the predicted equilibrium shape near the end of the gravel and cobble transition beach.

Direct measurements of waves, current, and suspended sand transport indicate that fine sand is highly mobile throughout the Half Moon Bay sub-tidal areas under most wave and tide conditions. The exception is the sub-tidal area in the lee of the diffraction mound and jetty remnant where wave-induced currents and tidal currents are generally weaker than elsewhere in the bay. The longshore current adjacent to the Half Moon Bay shoreline typically flows from the southwest end of the bay to the north and northeast. An offshore current is usually also present in the nearshore under breaking waves. This results in a north and northwest directed mean current in the nearshore of Half Moon Bay. The mean current transports suspended sand entrained by wave action and eroded from the southwest corner of the beach to the north out of the bay where it is entrained by tidal currents and removed from the area.

Gravel and cobble sized particles are highly mobile along the upper inter-tidal shoreline in the southwest corner of the bay during periods of large waves and water levels above mid tide level. Significant quantities of gravel and cobble have been transported from the transition beach area on the west side of the bay shoreline to the southwest end of the bay. The redistribution of the gravel alongshore in this area has reduced the effectiveness of the transition beach to provide protection to the breach fill, particularly in the areas between transects HMB2 and 4.

A sediment budget based on beach profiles and bathymetry surveys for the beach and nearshore areas of Half Moon Bay indicates a negative budget for the period 1996 to 2004. This negative budget contrasts with the small positive budget in outer Half Moon Bay (that includes the dredged sediment disposal areas) reported by Osborne et al (2003). Patterns and pathways of net transport are consistent between the detailed nearshore sediment budget and more regional budget reported by Osborne et al (2003).

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Hsu, J.R.C. and Evans, C. 1987. "Parabolic bay shapes and applications." Proceedings of the Institution of Civil Engineers. Vol. 87, Part 2, pp. 557-570.

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Seabergh, W.C. 1999. "Physical model for coastal inlet entrance studies", Coastal Engineering Technical Note CETN IV-19, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

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3 Wave Climate and Water Levels

Introduction

This chapter presents the results of a wave climate and water level analysis for Half Moon Bay. The objective is to determine the frequency of occurrence of wave height, period, and direction, and water levels (tide elevations), including the joint frequency of occurrence in the entrance to Grays Harbor. The results will be used to develop appropriate wave and water level test conditions for physical and numerical models of alternatives within Half Moon Bay. PI Engineering in cooperation with the US Army Corps of Engineers is evaluating several engineering alternatives to control erosion at Half Moon Bay. Some alternatives will be evaluated using a large scale physical model at the US Army Corps of Engineers Engineer Research and Development Center (ERDC). In addition to the physical model, the results will be applied to verification of numerical models, including the CGWAVE model discussed in Chapter 5.

Approach

Waves and water levels in Half Moon Bay were analyzed and evaluated following the approach outlined below:

1. Data from the Grays Harbor offshore buoy and two wave measurement stations within Half Moon Bay were analyzed and used to develop descriptive statistics for the offshore and nearshore wave climates.
2. Tide elevation data were- analyzed to determine the frequency of occurrence of extreme water levels.
3. The Grays Harbor offshore buoy data was transformed by numerical modeling to nearshore data points in and near Half Moon Bay. Numerical models were applied to develop a “wave transformation matrix”. The matrix was developed by running a coupled STWAVE and ADCIRC model for certain representative, evenly distributed, combinations of offshore wave height, period and direction, water level, and tide stage (and associated current velocity). The output for each model run was the wave height, period, direction, and water

level at select points around Half Moon Bay. The transformation matrix developed can then be applied to an arbitrary combination of the offshore wave height, period, and direction to determine the corresponding wave height, period, and direction at certain Half Moon Bay nearshore points. The measured wave buoy and tripod field data for Half Moon Bay were used to verify and calibrate the wave transformation matrix.

4. Test conditions for physical and numerical modeling of the engineering alternatives were determined from the analyzed wave and water level data.

Measured Wave and Water Level Data

The field data applied in this analysis includes measured wave data from the Grays Harbor offshore buoy, wave data measured at two stations deployed in Half Moon Bay, and water surface elevation measured by a tide gage deployed at Westport during 1999-2001.

Grays Harbor Offshore Buoy

Grays Harbor offshore wave data are available from the Coastal Data Information Program Buoy No. 036 (CDIP, 2004). The buoy is currently a Datawell directional buoy located in a water depth of 38.4 m, approximately 1000 m southwest of the entrance to Grays Harbor. Wave height and period measurements are available for November 1981 to July 1982, sporadically from July 1982 to January 1985, and from January 1985 to the present. Directional data became available in August 1993. There are 15.3 years of total data and 9.2 years of directional data available for this site.

Half Moon Bay Wave Measurements

On two occasions, Pacific International Engineering deployed wave measurement platforms in the Half Moon Bay region. One station was operational in 1999 and the other in 2002. Both stations collected wave, current, and water level data. The locations of these two platforms are shown in Figure 3-1.

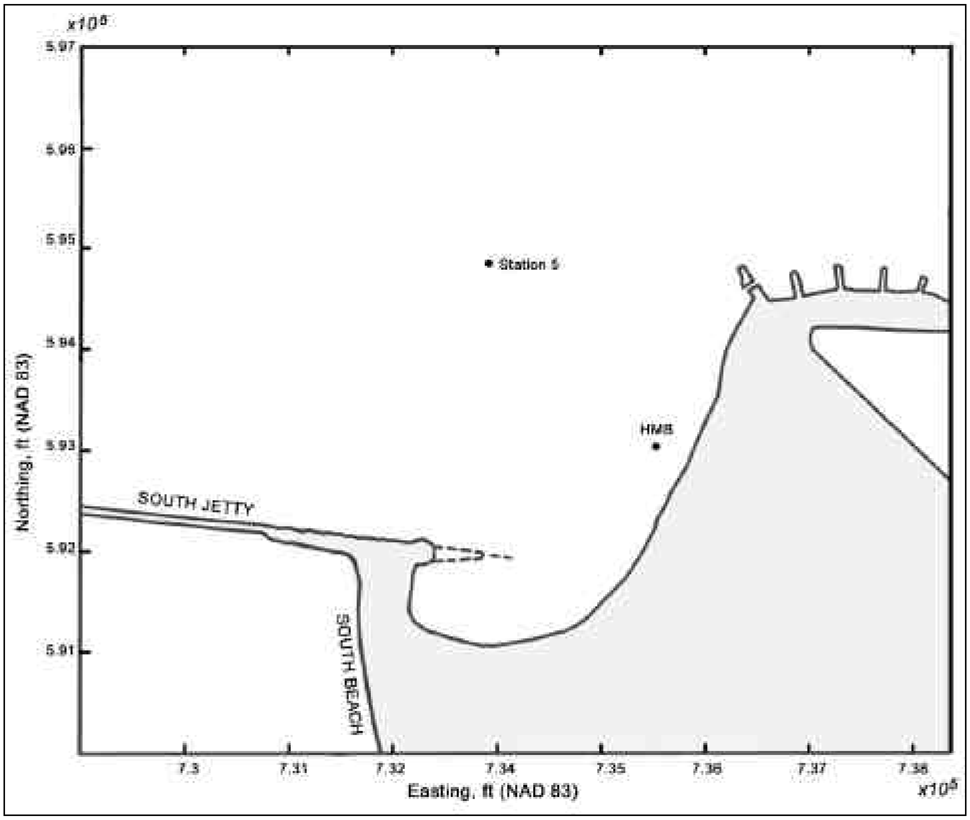


Figure 3-1. Half Moon Bay wave measurement locations

Station 5 was deployed between September and November, 1999 at 223698.0 E, 181850.5 N (WSP South, NAD83, meters). The platform was located in 10.1 m of water, mean tide level (mtl). Instruments on the platform included a 1500 kHz Sontek Acoustic Doppler Profile and a Sontek Hydra with Acoustic Velocimeters, and two Optical Backscatterance Sensors. A time series of the wave height and depth at Station 5 is shown in Figure 3-2.

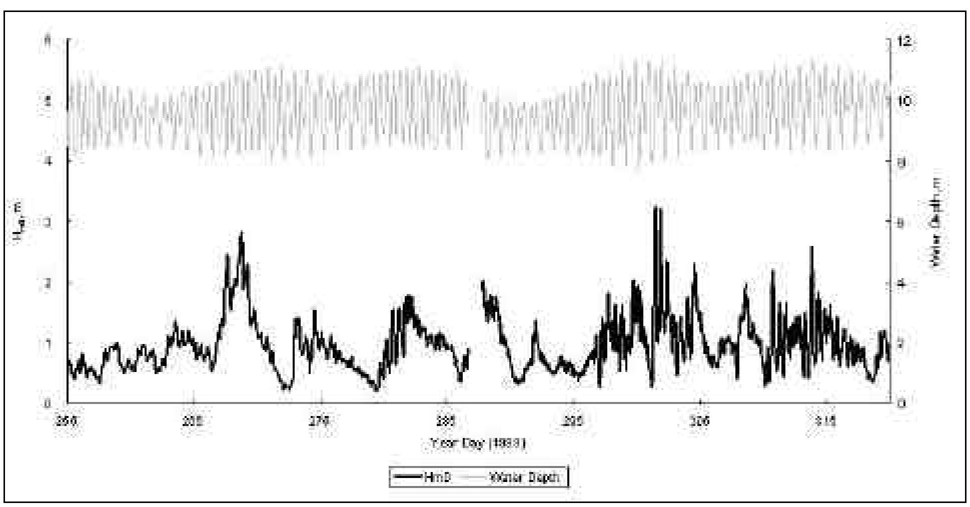


Figure 3-2. Water depth and wave height time series at Station 5 (13 September to 17 November 1999)

The second measurement station, HMB1, in 3.27 m of water (mlt) was located at 224161.9 E, 181343.0 N (WSP South, NAD83, meters) and deployed between 6 March and 29 April 2002. Instruments at HMB1 consisted of a SonTek Hydra with an ADV-Ocean current meter and a Druck pressure sensor. A time series of the depth and wave height measured at HMB1 is shown in Figure 3-3.

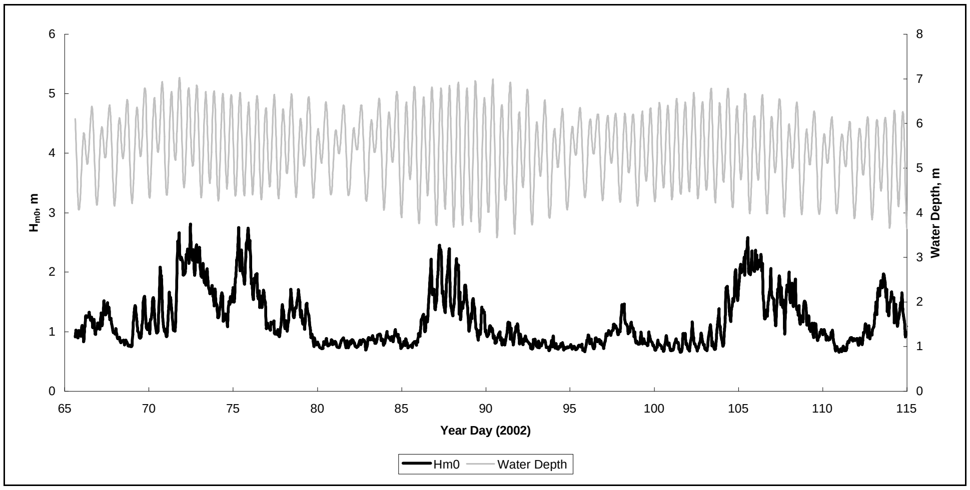


Figure 3-3. Water depth and wave height time series at HMB1 (6 March to 29 April 2002)

Westport Tide Gauge

Water level data from October 1999 to June 2001 was obtained from the tide gauge at Westport. Water levels for the physical model will be baed ona preliminary frequency of occurrence analysis of the data. A longer, unprocessed water level record exists spanning from October 1999 to present (May 2004) and data collection at the site is ongoing. More detailed analysis including separation of the tidal harmonics at the site, should be undertaken when the complete water level time series is available.

Analysis of Measured Waves and Water Levels

To characterize conditions at the study site, the collected wave and water level data were analyzed. The following sections summarize these analyses.

Offshore Wave Data Analysis

Joint wave height and period analysis was performed on the entire available data set for the Grays Harbor offshore wave buoy, CDIP No. 036. The annual frequencies of occurrence for the full 15.3 years of data, independent of direction, are shown in Figure 3-4. The analysis indicates that the most frequent waves at the site have significant wave heights (H_s) and peak periods (T_p) in the order of 1.5 m and 9 sec, respectively. For the present analysis, the spectral estimate of the significant wave height, H_{m0} was used as a measure of H_s. The measured data illustrates that waves with H_s > 8.0 m, as well as those with T_p > 20 sec can be expected offshore of Grays Harbor.

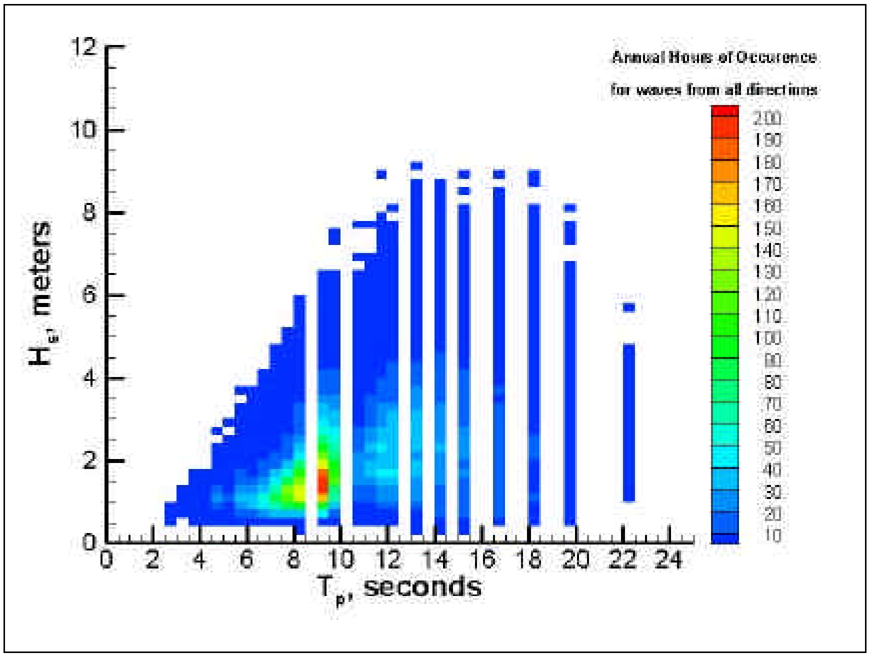


Figure 3-4. Annual hours of occurrence for waves at CDIP Buoy 036

Extreme value analysis was also performed on the CDIP Buoy 036 data. A three-parameter Weibell distribution with a = 1.5 was selected as the best match for the measured wave heights (R^2 = 0.977). The results of this analysis are provided in Table 3-1. Based on this analysis, the one-year return period H_s is 7.67 m. The highest H_s recorded at the buoy of 9.75 m measured on 12 October 2003, has a computed a return period of approximately 25 years.

Table 3-1
Results of extreme value analysis for CDIP Buoy 036

Return Period, years	H_s, m
1	7.67
2	8.18
5	8.77
10	9.19
25	9.70
50	10.06
100	10.41

Since wave direction is often fundamental to coastal processes, joint wave height, period, and direction analysis was also completed. Directional data at the buoy were only available from 1993 to 2003. The wave data were divided into five primary direction bins as given in Table 3-2. Waves from the 270.0 and 292.5 deg bins make up 76 percent of the annual climate, with less than ten percent of the waves coming from the 213.75 and 326.24 deg bins combined. Thirteen percent of waves come from directions within the 247.5 deg bin.

Table 3-2 Direction bins used for joint height, period, and direction analysis				
Bin #	**Mean Direction and Bin Label**	**Hours per Year**	**Lower Limit, deg**	**Upper Limit, deg**
1	213.75	691.96	180.00	230.60
2	247.50	1127.78	230.60	258.75
3	270.00	3495.79	258.75	281.25
4	292.50	3164.08	281.25	309.40
5	326.25	158.14	309.40	360.00

A graphical representation of results from the joint height, period, and direction analysis is provided in Figure 3-5. As shown in Figure 3-5, the height-period distribution for bin 1 (213.75 deg) is bimodal, with one peak occurring at H_s = 2.5 m and T_p = 9 sec and the second peak at H_s = 0.75 m and T_p = 15 sec.

The waves with the highest frequency of occurrence from the 247.5, 270.0, and 292.5 deg bins are very similar with H_s = 1.5 m and T_p = 9 sec. At CDIP Buoy 036, 40 percent of the waves come from the 270.0 deg bin, including 92 percent of waves with periods over 20 sec. With the exception of the second largest storm on record (24 November 1998), all of the top ten recorded storms had directions between 258.0 and 280.0 deg. The 24 November 1998 storm came from 234 deg. While the waves from 270.0 deg typically have H_s = 1.5m and T_p = 9s, those with periods greater than 10 sec have an average height of 2.5 m. 40 percent of the longer period waves (T_p > 13 sec) from 247.5 deg have wave heights less than 1.0 m.

Waves from 292.5 deg account for 36 percent of the recorded CDIP Buoy 036. These waves are narrowly distributed, with 75 percent occurring within ± 1.0 m and ± 3.0 sec of the dominant significant wave height (1.5 m) and peak wave period (9 sec).

Only two percent of records at the CDIP buoy come from directions greater than 310 deg. These waves typically have H_s = 1.0 m and T_p = 4.5 sec, with no recorded wave heights and periods greater than 4.0 m and 10 sec, respectively.

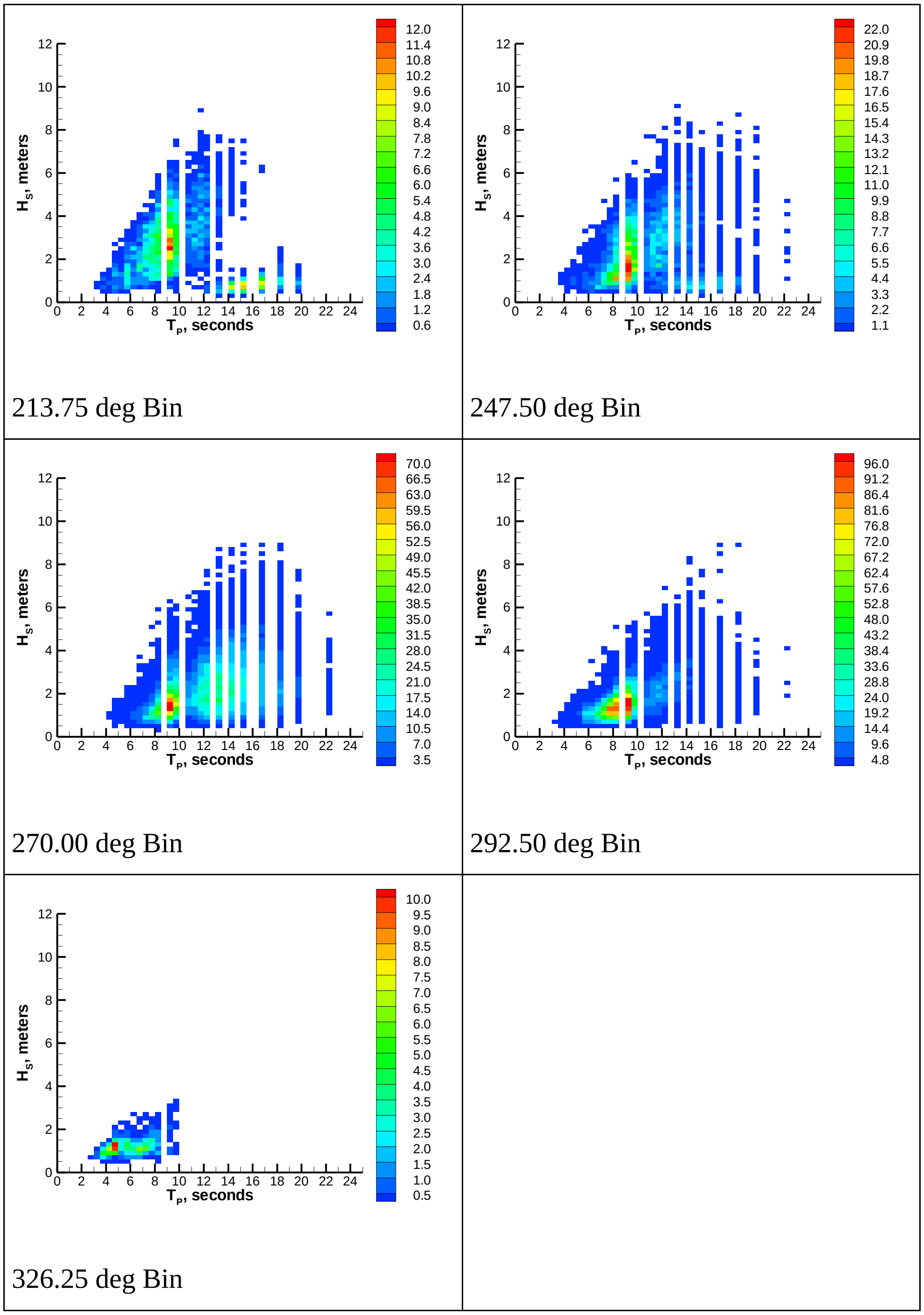


Figure 3-5. Percent frequency of occurrence plots of joint wave height, period, and direction for CDIP Buoy 036

Another means of evaluating the measured offshore waves conditions is by examining the wave energy. Annual wave energy at the CDIP buoy for each direction was calculated by applying the following formula from linear wave theory (Dean and Dalrymple, 1991):

$$\overline{E} = \frac{1}{K}\sum_{i=1}^{N} P_i t_i = \frac{1}{K}\sum_{i=1}^{N} (E_i C_{g_i}) t_i = \frac{1}{K}\sum_{i=1}^{N} \frac{1}{8} rgH_{rms\,i}^{2} C_{g_i} t_i$$

where

P = wave power by direction, W/m/year

E = wave energy by direction, J/m/year

E = average annual wave energy by direction, J/m/year

N = total number of wave records

H_{rms} = root mean square wave height = $0.706H_s$, m

C_g = group celerity, m/s

t = duration over which the record is valid, seconds

K = number of years on record, years

This calculation allows comparison of the nearshore locations and determination of the energy lost during wave transformation. The directional wave energy rose for CDIP Buoy 036 is shown in Figure 3-6. This rose was created using the directional data available from 1993 to 2003. The energy weighted direction is centered about waves coming from 263.6 deg, while the mean wave direction is 272.2 deg. The total annualized wave energy at the buoy is 831.3 GJ/m/year.

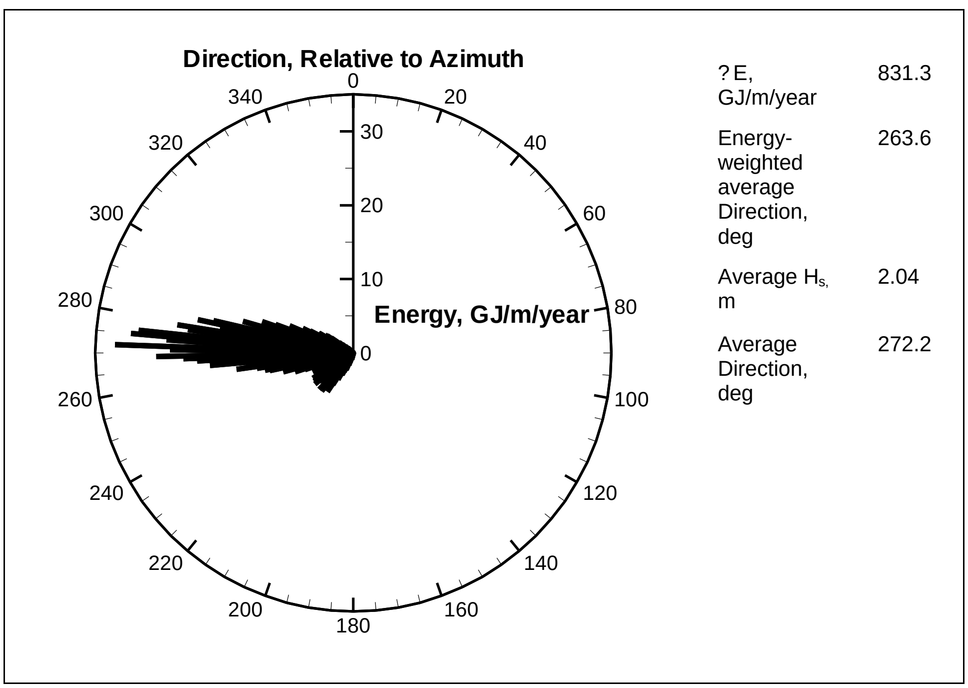


Figure 3-6. Annualized wave energy rose for CDIP Buoy 036

Nearshore Wave Data Analysis

Recent short-term wave measurements at Half Moon Bay (December 2003 and January 2004) were discussed in Chapter 2 with further detail provided in Appendix A. This data provided valuable wave information near the shoreline but the record is of insufficient length to provide statistical information on the long-term wave climate, which is needed to gain useful insight for numerical and physical models of the site and proposed alternatives. For this reason, the longer duration offshore wave record must be transformed to within Half Moon Bay.

Water Level Analysis

An assessment of available water level data in the vicinity of Half Moon Bay is required to determine the appropriate high water levels for physical and numerical modeling. The nearest available water level record at Westport (October 1999 to June 2001) was analyzed to extract the semi-diurnal high water levels (see Figure 3-7). This resulted in a dataset of 1500 observed high water levels as shown in Figure 3-8. These were analyzed to determine the statistical distribution of high water levels at Westport.

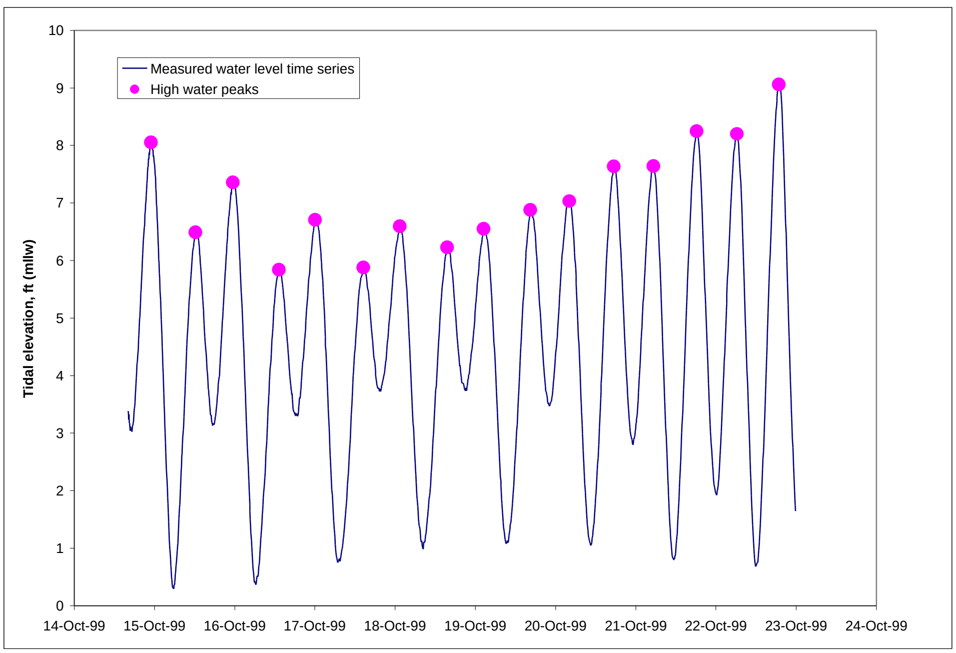


Figure 3-7. Sample of time series of measured water levels at Westport

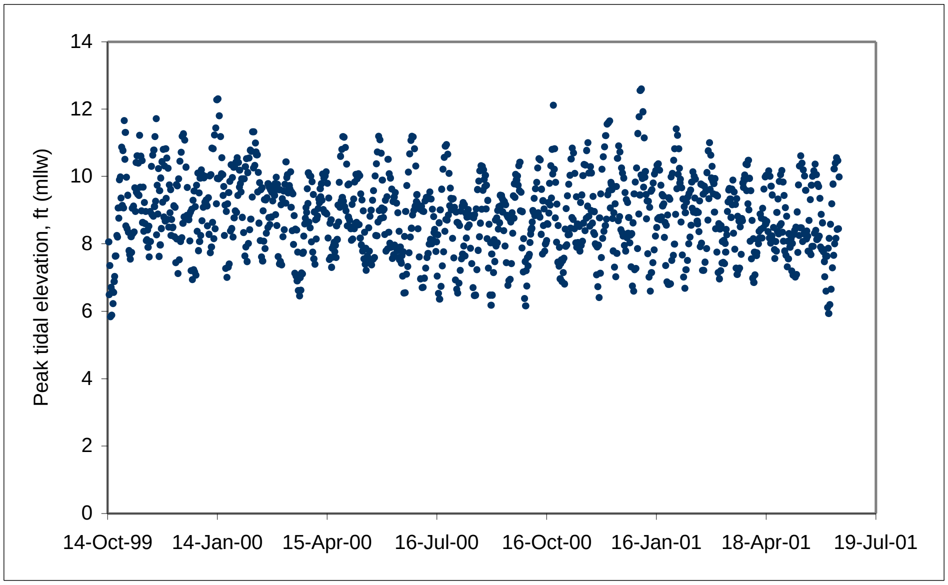


Figure 3-8. Peak measured tidal water levels at Westport

The cumulative exceedance curve of peak water levels at Westport is shown in Figure 3-9. The inset figure shows the upper end of the distribution plotted on semi-log axes.

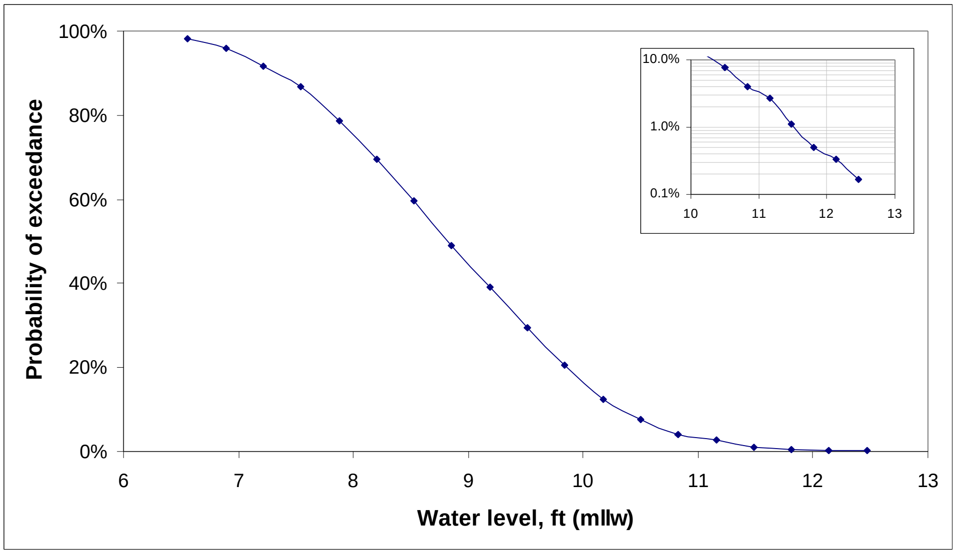


Figure 3-9. Peak water level exceedance

Figure 3-9 shows that the median high water level (50 percent exceedance level) was 8.9 ft. The maximum-recorded water level in the 1.75 years of record was 12.6 ft. An exceedance plot, presented in terms of the number of times a given level is exceeded annually, is shown in Figure 3-10.

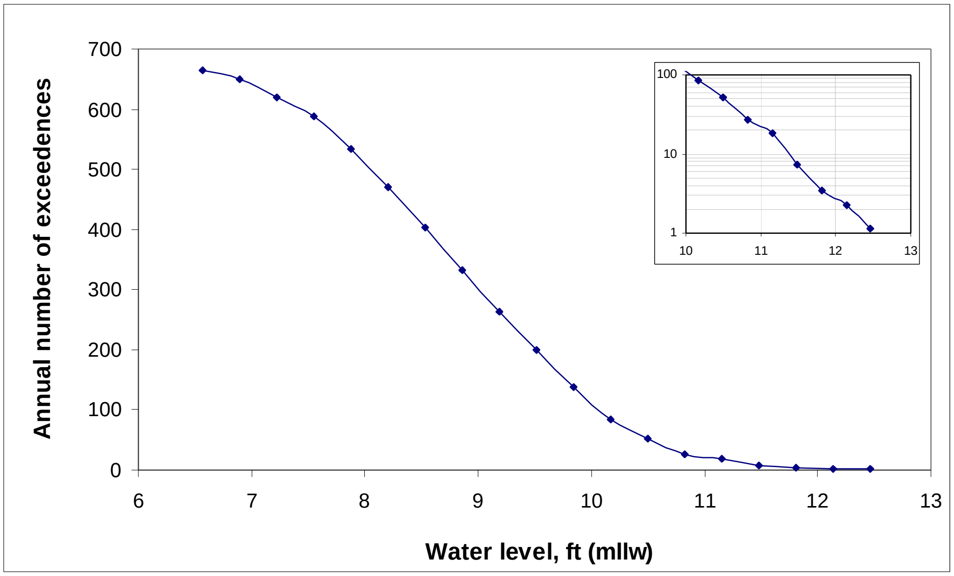


Figure 3-10. Annual exceedance for peak water levels at Westport

Water levels above 11 feet occur approximately 20 times per year and levels above 12 feet occur approximately twice a year. A level of 11.5 feet would have an expected exceedance frequency of approximately 7.5 times per year. Based on this analysis, a water level of +11.5 ft mllw is recommended for further work in the physical model.

Wave and Tidal Current Modeling

Numerical modeling was required to determine the nearshore waves at Half Moon Bay. The available measured wave data at this location is insufficient in duration to develop wave statistics and make conclusions about the long-term wave climate.

The interaction of the waves with the tidal currents can significantly alter wave transformation in regions near the shore, especially in harbors, bays, and estuaries. Waves were transformed from the CDIP buoy to Half Moon Bay with numerical wave and tidal circulation models for Grays Harbor.

Tidal currents were simulated with the ADvanced CIRCulation model (ADCIRC) (Luettich, Westerink, Scheffner, 1992), a two-dimensional, depth-integrated hydrodynamic circulation model was used. The model is designed to compute long wave circulation in coastal oceans associated with tides, winds, and density-driven flows. The model uses a reformulation of the depth-averaged shallow water wave equations for conservation of mass and momentum. It can be applied to a wide range of time and depth conditions, from simulations of months to years, and in deep ocean to estuarine systems. An ADCIRC model can have variable element sizes, allowing the use of fine-scale, detailed elements in regions of interest and large-scale elements elsewhere. Output from the model can be used to define tide and storm surge elevations, as well as velocities at every node in the mesh.

An ADCIRC tidal circulation model previously developed for Grays Harbor, was applied in the present study. The Grays Harbor ADCIRC model was developed and calibrated by M. Cialone of the US Army Corps of Engineers Engineer Research and Development Center (ERDC). The ADCIRC model simulations completed for the period of 7 to 22 June 1999 were applied in this study. Details regarding the Grays Harbor ADCIRC model can be obtained from North Jetty Performance and Entrance Navigation Channel Maintenance, Grays Harbor, Washington, Volume 1: Main Text (US Army Corps of Engineers, ERDC/CHL TR-03-12).

The Spectral Transofrmation WAVE model (STWAVE) (McKee, Smith et al, 2001) is a finite difference, steady-state spectral wave model based on the wave action balance equation, capable of calculating radiation stresses and regions of active wave breaking. STWAVE "simulates depth-induced wave refraction and shoaling, current-induced refraction and shoaling, depth- and steepness-induced wave breaking, diffraction, wind-wave growth, and wave-wave interaction and whitecapping that redistribute and dissipate energy in a growing wave field" (McKee Smith et al, 2001).

STWAVE works with the following assumptions:

- mild bottom slope and negligible wave reflection,
- spatially homogeneous offshore wave conditions,
- steady-state waves, currents, and winds,
- linear refraction and shoaling,
- depth-uniform current, and
- negligible bottom friction.

The STWAVE and ADCIRC models can be coupled through the Surface Water Modeling System, SMS (Brigham Young University, 2003). SMS provides a Steering Module utility for passing ADCIRC calculated currents to STWAVE. The effects of the currents on the wave field can then be taken into account.

A new series of coupled STWAVE-ADCIRC model simulations were performed to calculate the wave transformations and aid in the development of the inshore wave climate within Half Moon Bay. Four tidal stages were selected from the ADCIRC simulations for input to the STWAVE simulations; the peak flood, high-water slack, peak ebb, and low-water slack. To select steady-state currents for use in the STWAVE model, a time series of water surface elevation was extracted from the ADCIRC grid representative of tidal conditions in the vicinity of Half Moon Bay. This time series is plotted in Figure 3-11. The selected ADCIRC simulation times for each of the flood, high-water slack, ebb, and low-water slack are 175500 sec, 189000 sec, 199800 sec, and 214200 sec, respectively.

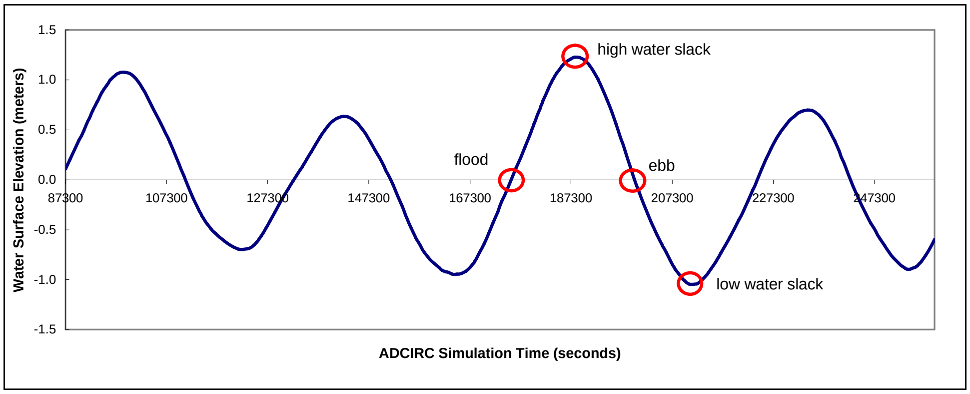


Figure 3-11. Time series of water surface elevation from the Grays Harbor ADCIRC simulation

STWAVE Modeling

An STWAVE model of the immediate offshore region, the entrance to Grays Harbor, and Half Moon Bay was developed to transform representative offshore wave conditions to sites within Half Moon Bay. This modeling serves as the basis for developing a wave transformation matrix, which will allow estimation of the nearshore wave conditions for any set of observed offshore waves. Longer available offshore wave measurements can then be applied to generate a longer-term nearshore wave climate.

Model Bathymetry and Set-up

The STWAVE model grid is shown in Figure 3-12, relative to the Grays Harbor ADCIRC bathymetry. The grid has a regular 50 m spacing, and is oriented at 260 deg relative to Azimuth. It is based on the STWAVE model grid developed for the Ocean Shores-Westport Ferry Route Wave Study, completed in 2002 (Cialone et al, 2002 and Cialone, Krause, 2001) The red box in Figure 3-12 indicates the region covered by the Half Moon Bay physical model, while the red dot is the location of CDIP Buoy 036. For clarity, only every tenth grid line is shown.

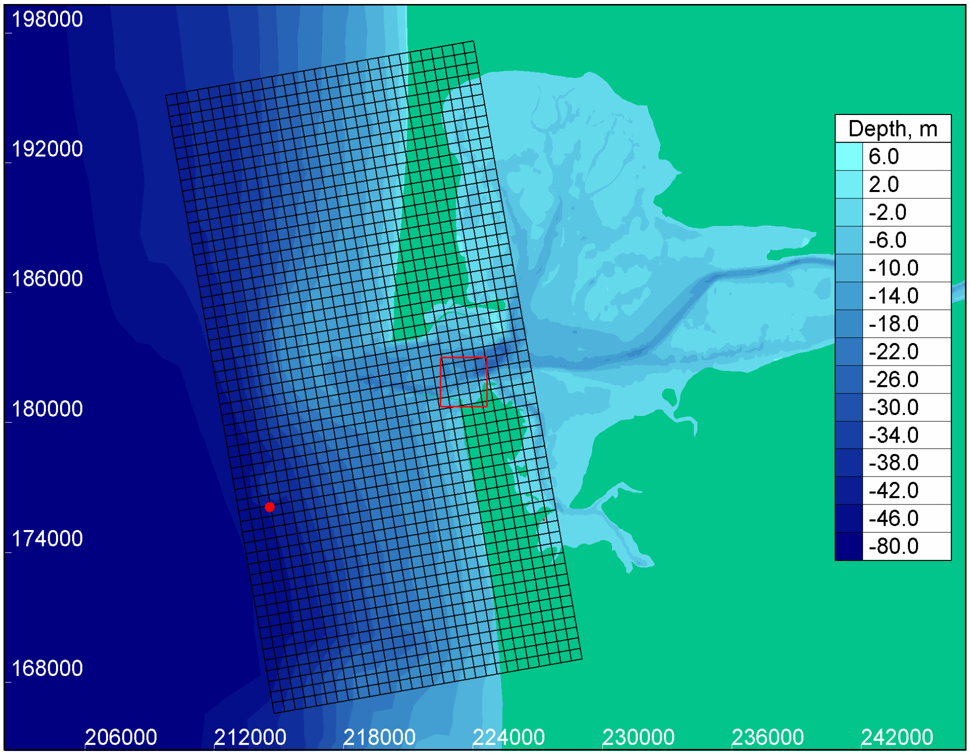


Figure 3-12. Half Moon Bay STWAVE grid, over ADCIRC bathymetry

The bathymetric grid for the STWAVE simulations is a mosaic created from three separate grids dating from 2001 to 2003. Before being combined, all bathymetric data was converted to mean tide level (mtl) standard adopted for the ADCIRC mesh. Table 3-3 lists the sources of bathymetric data used to develop the STWAVE grid.

Table 3-3
Source bathymetric grids used to develop STWAVE model

Bathymetric Source	File Type	Source Information	Date Last Modified	Datum
1	STWAVE Depth file.	STWAVE model completed by Mary Cialone of ERDC-CHL for the Ocean Shores-Westport Ferry Route Wave Study (Cialone et al, 2002).	25/10/2002	mtl, m
2	ADCIRC bathymetric and surface elevation mesh.	Grays Harbor 1999 ADCIRC model developed by Mary Cialone of ERDC-CHL (Cialone et al, 2002).	01/06/2001	mtl, m
3	Regular 25 ft bathymetric grid.	PI Eng. composite of 2003 Seattle District inlet and outer Half Moon Bay surveys, 2002 Department of Ecology nearshore bathymetry, and the 1996 Jetty survey and design drawings (Osborne et al, 2003).	27/05/2003	mllw, ft

The difference between mtl and the water surface elevations at high and low water slack were determined using the Grays Harbor ADCIRC surface elevation file. These differences were added or subtracted from the STWAVE mtl grid to create the high and low water slack bathymetric grids. The water surface elevation was assumed to be mtl (0 m) for the ebb and flood conditions. The resulting bathymetric grid is shown in Figure 3-13.

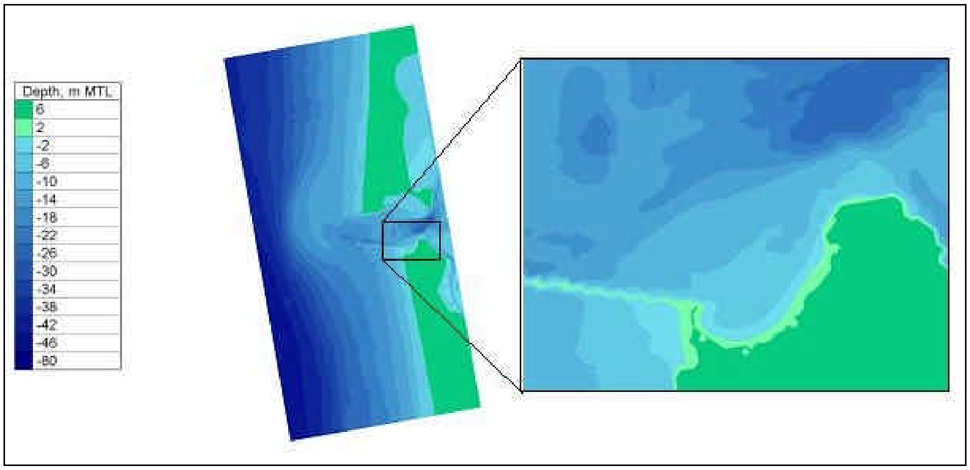


Figure 3-13. STWAVE bathymetric grid (close-up of the Half Moon Bay region)

Model Simulations

STWAVE simulations were run for 19 wave cases, 4 tidal phases, and 5 directions. The wave cases selected for simulation are shown by the red dots in Figure 3-14, which overlay the annualized CDIP joint wave height and period data. These input wave conditions are assumed to be spatially homogeneous over the offshore boundary of the model. The 4 tidal phases simulated were flood, high water slack, ebb, and low water slack. The wave directions used in the simulations were the mean CDIP buoy direction bins of 213.75, 247.50, 270.00, 292.50, and 326.25 deg. These conditions result in 380 individual STWAVE model simulations.

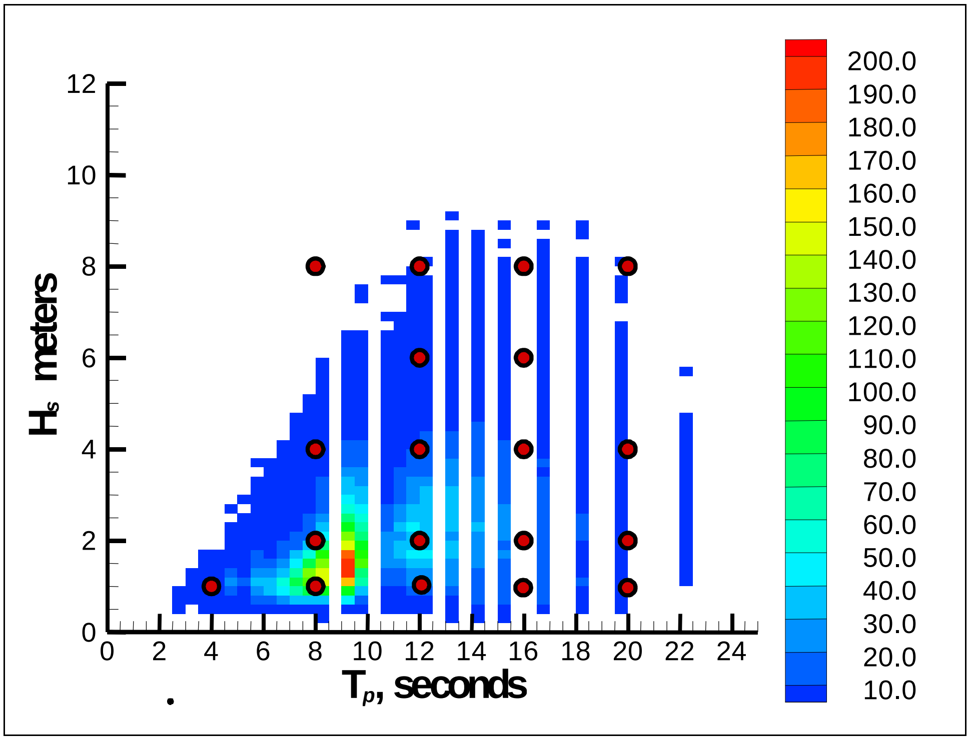


Figure 3-14. CDIP Buoy 036 annual hours of occurrence and selected model wave conditions (red dots)

Each of the modeled waves represents a different percentage of the total annualized wave climate at the CDIP buoy. These percentages, and their hourly equivalents, are categorized according to median wave direction and summarized in Table 3-4.

Table 3-4
Annualized data, by direction, from CDIP Buoy 036 (1994-2003)

Direction Bin		213.75	247.50	270.00	292.50	326.25
Direction Ranges		180.0 - 230.6	230.6 - 258.75	258.75 - 281.25	281.25 - 309.4	309.4 - 360
Hours per Year		691.96	1127.78	3495.79	3164.08	158.14
Percentage of Year		7.9%	12.9%	39.9%	36.1%	1.8%
Equivalent Duration of Nominal Wave Direction, hours	1m 4s	39.86	26.18	32.86	185.93	96.19
	1m 8s	57.91	245.64	696.25	1458.53	45.52
	1m 12s	26.96	67.01	346.32	122.37	0.00
	1m 16s	60.77	41.37	105.29	6.56	0.00
	1m 20s	4.09	3.53	9.81	0.28	0.00
	2m 8s	34.86	12.89	36.66	34.06	0.00
	2m 12s	34.86	156.58	809.66	383.96	0.00
	2m 16s	1.74	12.89	299.12	27.69	0.00
	2m 20s	0.06	1.35	36.66	1.01	0.00
	4m 8s	128.30	97.09	69.68	34.06	0.06
	4m 12s	60.10	134.76	311.79	89.18	0.00
	4m 16s	0.62	11.94	190.54	21.47	0.00
	4m 20s	0.00	0.56	14.35	0.45	0.00
	6m 12s	31.67	21.02	25.67	3.92	0.00
	6m 16s	0.62	6.67	29.70	2.41	0.00
	8m 8s	0.11	0.00	0.00	0.00	0.00
	8m 12s	1.12	1.79	1.01	0.22	0.00
	8m 16s	0.39	0.95	2.91	0.34	0.00
	8m 20s	0.00	0.28	0.34	0.00	0.00
Equivalent duration expressed as a percentage:	1m 4s	0.455%	0.299%	0.375%	2.122%	1.098%
	1m 8s	0.661%	2.804%	7.948%	16.650%	0.520%
	1m 12s	0.308%	0.765%	3.953%	1.397%	0.000%
	1m 16s	0.694%	0.472%	1.202%	0.075%	0.000%
	1m 20s	0.047%	0.040%	0.112%	0.003%	0.000%
	2m 8s	0.398%	0.147%	0.419%	0.389%	0.000%
	2m 12s	0.398%	1.787%	9.243%	4.383%	0.000%
	2m 16s	0.020%	0.147%	3.415%	0.316%	0.000%
	2m 20s	0.001%	0.015%	0.419%	0.012%	0.000%
	4m 8s	1.465%	1.108%	0.795%	0.389%	0.001%
	4m 12s	0.686%	1.538%	3.559%	1.018%	0.000%
	4m 16s	0.007%	0.136%	2.175%	0.245%	0.000%
	4m 20s	0.000%	0.006%	0.164%	0.005%	0.000%
	6m 12s	0.362%	0.240%	0.293%	0.045%	0.000%
	6m 16s	0.007%	0.076%	0.339%	0.028%	0.000%
	8m 8s	0.001%	0.000%	0.000%	0.000%	0.000%
	8m 12s	0.013%	0.020%	0.012%	0.003%	0.000%
	8m 16s	0.004%	0.011%	0.033%	0.004%	0.000%
	8m 20s	0.000%	0.003%	0.004%	0.000%	0.000%
	Totals	5.525%	9.618%	34.459%	27.083%	1.618%

Special model observation points were used to output the resultant waves along the western boundary and shoreline of Half Moon Bay. The model output at these points included wave height, period, direction, and directional wave energy spectra. Twenty special output points were used for the Half Moon Bay model runs, ten along the western model boundary, and ten along the shoreline. The ten points along the western edge correspond to the location of the proposed physical model boundary. The coordinates of these points are given in Table 3-5, and their locations are shown in Figure 3-15.

Table 3-5 Coordinates of STWAVE special observation points		
Special Observation Point	X Coordinate, m (WSP South)	Y Coordinate, m (WSP South)
1	222887	182461
2	222855	182353
3	222881	182206
4	222858	182049
5	222875	181951
6	222852	181794
7	222878	181647
8	222855	181490
9	222872	181392
10	222898	181244
11	223135	181337
12	223350	181273
13	223564	181209
14	223680	181128
15	223836	181105
16	223975	181180
17	224057	181296
18	224187	181421
19	224268	181536
20	224340	181701

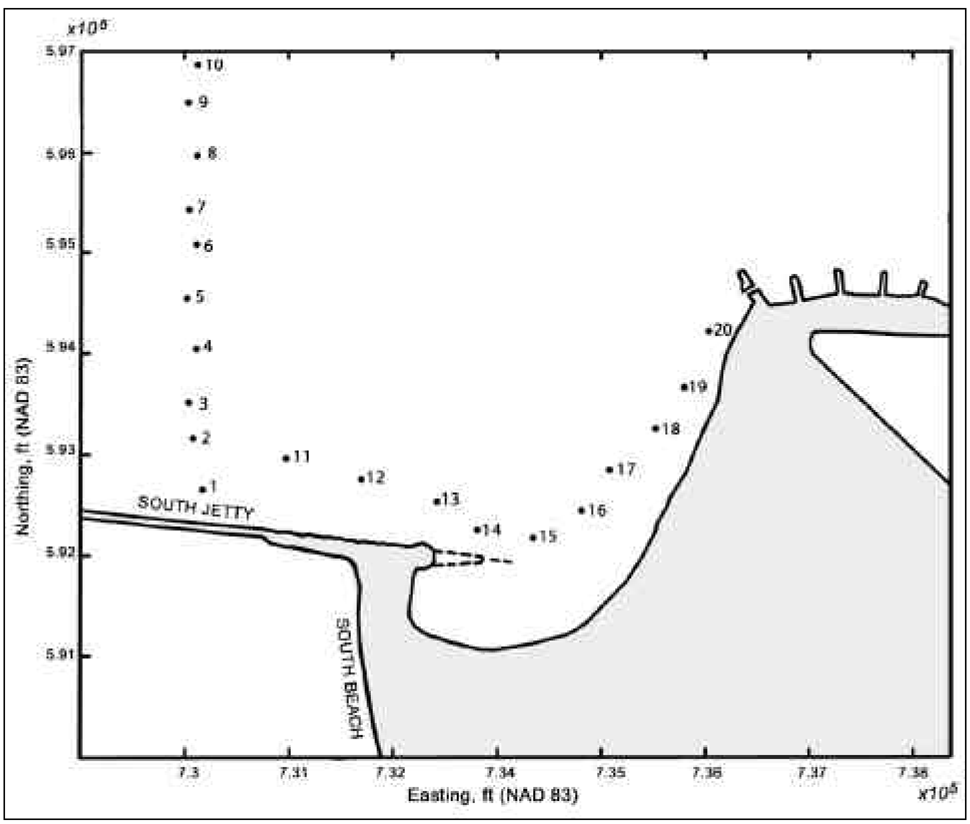


Figure 3-15. Half Moon Bay STWAVE model special observation points

Model Verification

The time series for each of the Half Moon Bay wave measurement stations and the CDIP Buoy 036 are shown in Figures 3-16 and 3-17. Offshore data for the model assumed zero time lag between the CDIP buoy 036 and the two inshore Half Moon Bay wave stations for the purpose of comparison between measurements and modeled results.

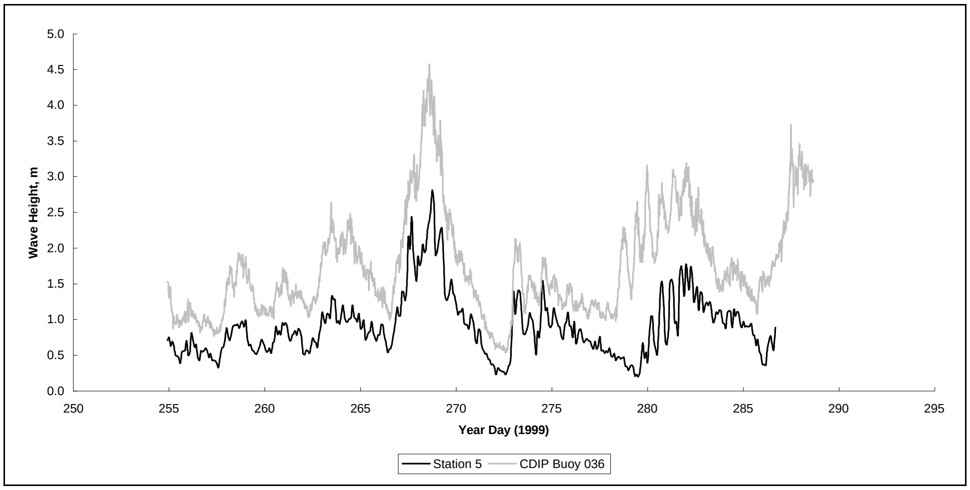


Figure 3-16. Time series of wave height data (12 September to 5 November 1999)

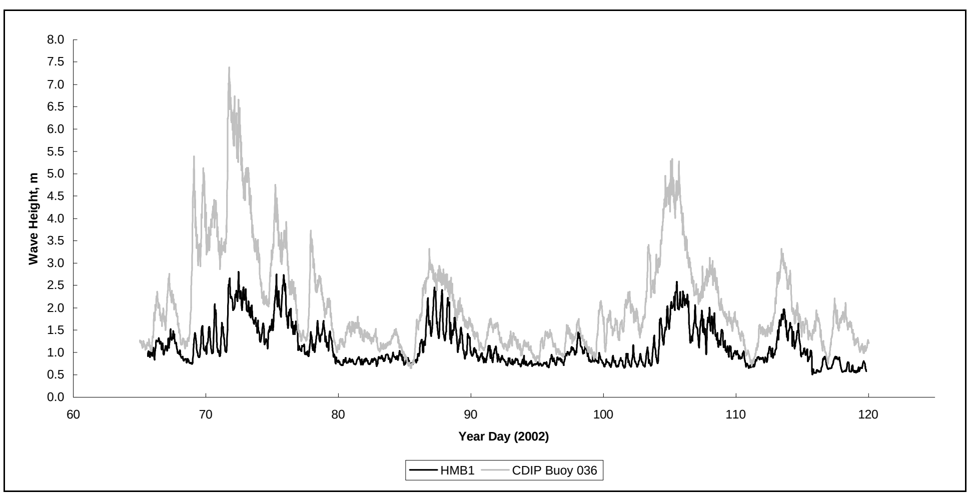


Figure 3-17. Time series of wave height data (6 March to 29 April 2002)

The time intervals between measurements were not the same for CDIP Buoy 036 and the two inshore locations. Measurements were made every half-hour offshore, every hour at HMB1, and every two hours at Station 5. The H_{m0}, T_p, and a measured inshore were linearly interpolated in time to match the CDIP buoy recording interval of every half-hour to that the measurements could be directly compared with modeled results.

Preliminary verification of the STWAVE model was completed using the SMS V8.1 ADCIRC-STWAVE Steering Module (Zundel et al, 2002). The Steering Module allows the user to couple the two models, so that date and time-specific current files can be generated using ADCIRC, and run with a coordinated time series of waves in STWAVE. The ADCIRC-STWAVE Steering Module was used to simulate wave conditions from 12 September 1999 to 30 September 1999, a time period that corresponds to when Station 5 was operational. The input wave parameters for the STWAVE model were taken from the CDIP buoy measurements for this period. Current fields were predicted by ADCIRC using the LeProvost tidal constituents toolbox. A time series of the measured and predicted wave heights is provided in Figure 3-18. The standard error of the estimate is used to compare the measured and predicted results. This value is calculated using the following formula:

$$\sigma_{est} = \sqrt{\frac{\sum_{i=1}^{N}(Y_i - Y_i')^2}{N}}$$

where

Y = the predicted value

Y′ = the measured value

N = total number of values being compared

As can be seen in Figure 3-18, the predicted waves show close agreement with those measured at Station 5. The standard errors of the estimates of H_{m0} and a for this simulation are 0.2 m and 6.4 deg.

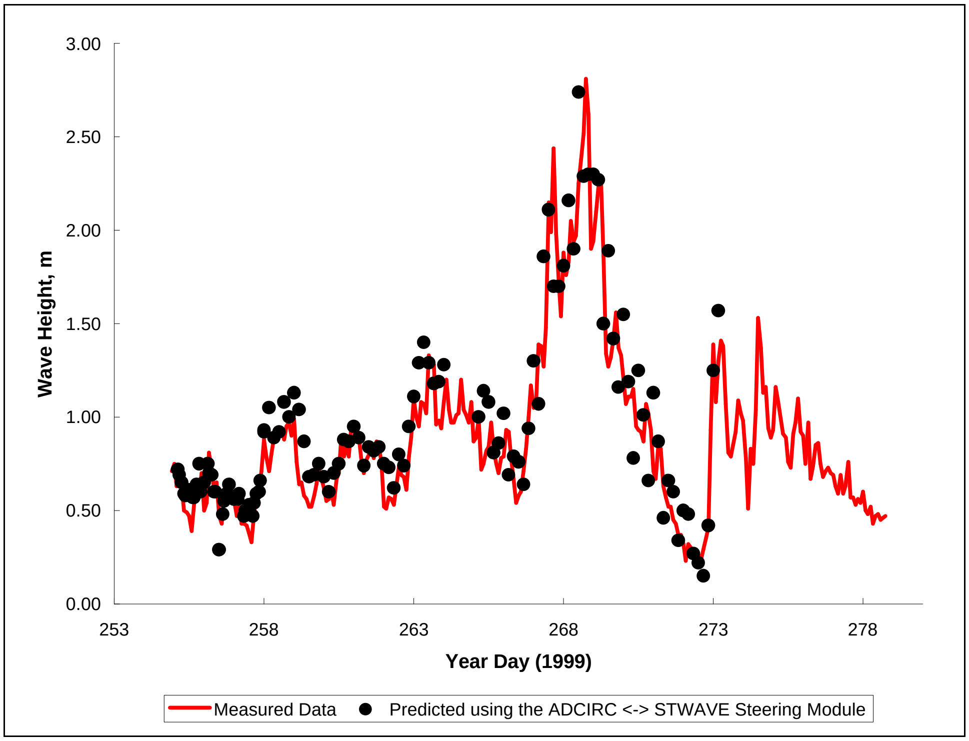


Figure 3-18. Time series of measured and ADCIRC-STWAVE predicted wave heights (12 September to 30 September 2003)

Model Results

The results of the STWAVE modeling provide key information regarding wave conditions within Half Moon Bay. Of primary importance are the wave heights and directions expected along the western and shoreward boundaries of the bay. Wave properties in these regions, including two-dimensional spectra, were output at the special observations points (see Figure 3-15). Analysis of the wave heights and directions at these points reveal distinct patterns in the expected values.

Figure 3-19 presents a graphical summary of the predicted wave conditions at observation point 5. In comparison to Figure 3-5 (offshore wave conditions at CDIP Buoy 036), extreme wave heights are significantly reduced at this location, particularly for waves from the southwest. The majority of waves occur in the 270 deg bin.

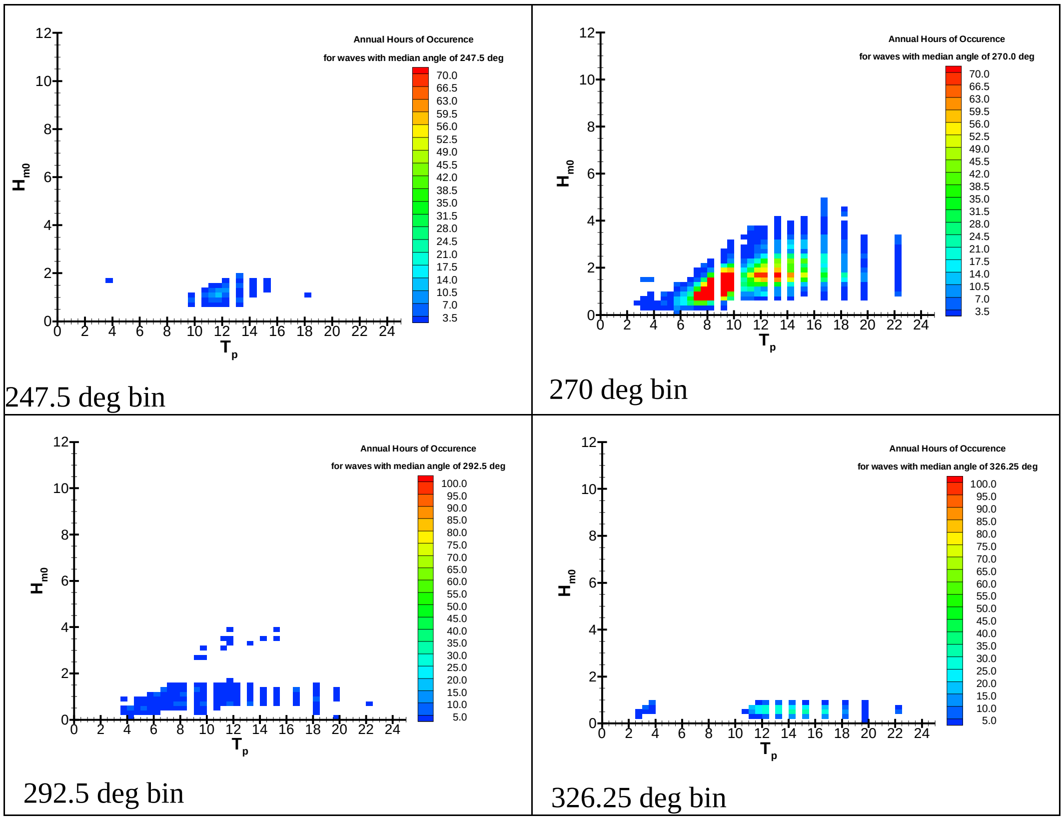


Figure 3-19. Joint wave height, period and direction plots for observation point 5 (see Figure 3-15 for location). Note: No waves from the 231.75 deg bin reach this location

Figure 3-20 illustrates the annual frequency of occurrence of the waves, independent of direction, for observation point 5, the midpoint of the western boundary. The most frequently occurring waves have significant wave heights (H_s) of approximately 1 m with peak periods (T_p) of 9 sec. The dominant wave heights are slightly reduced with fewer occurrences of larger waves compared with the offshore conditions at CDIP Buoy 036 (Figure 3-4).

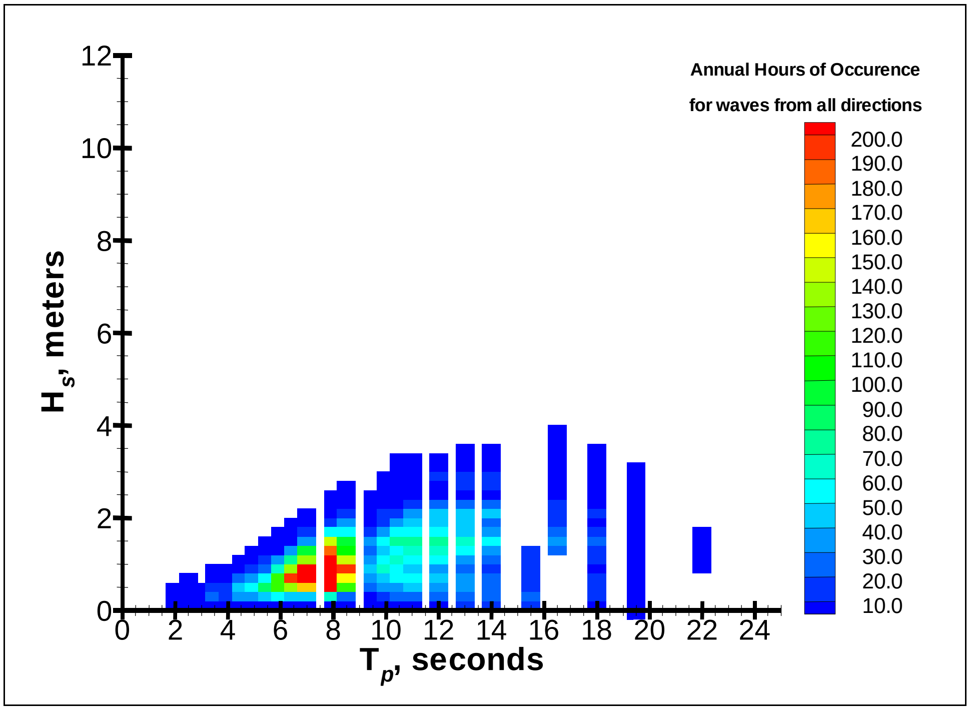


Figure 3-20. Annual hours of occurrence for waves at model observation point 5

Wave direction can be expected to remain approximately constant at any observation point for a given tidal stage, regardless of offshore wave conditions. This is especially true for observation points 11 through 20, which follow the shoreline. This can be seen in Figure 3-21, which illustrates the wave direction at the selected observation points for various offshore directions and wave conditions.

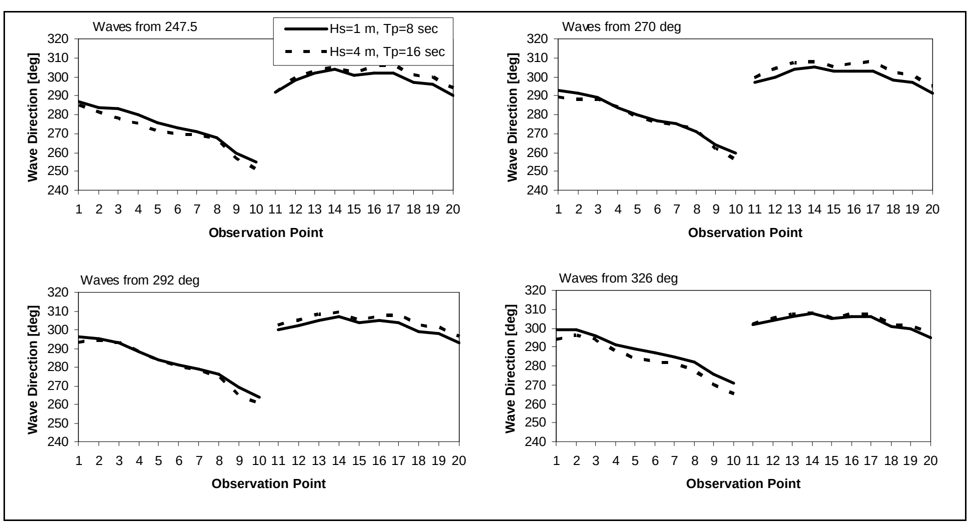


Figure 3-21. Comparison of wave directions at the observation points at high water slack

However, wave directions do differ with tidal condition, as shown in Figure 3-22. This plot shows that the variation in direction due to tidal stage is less than ten deg.

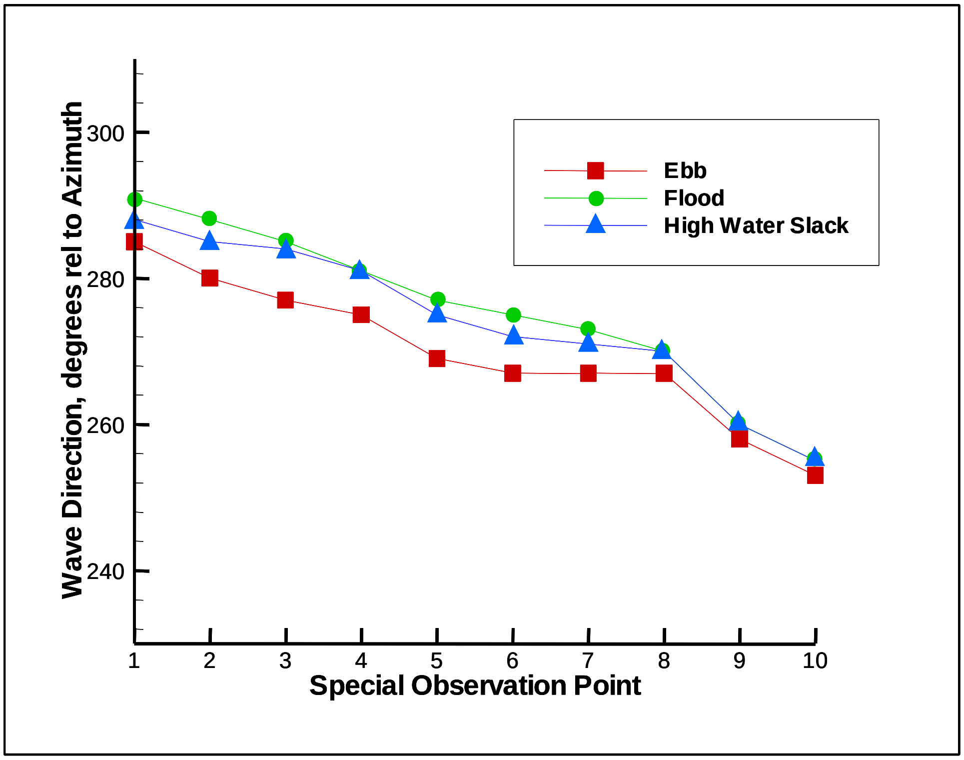


Figure 3-22. Wave direction (2m, 16s offshore waves from 270 deg)

Trends in modeled wave heights at each of the observation points are not as clear as those for wave direction. In general, waves from the southwest are attenuated more than those from the northwest. As shown in Figure 3-23, waves from 292.50 deg can be expected to maintain their height more than from other directions.

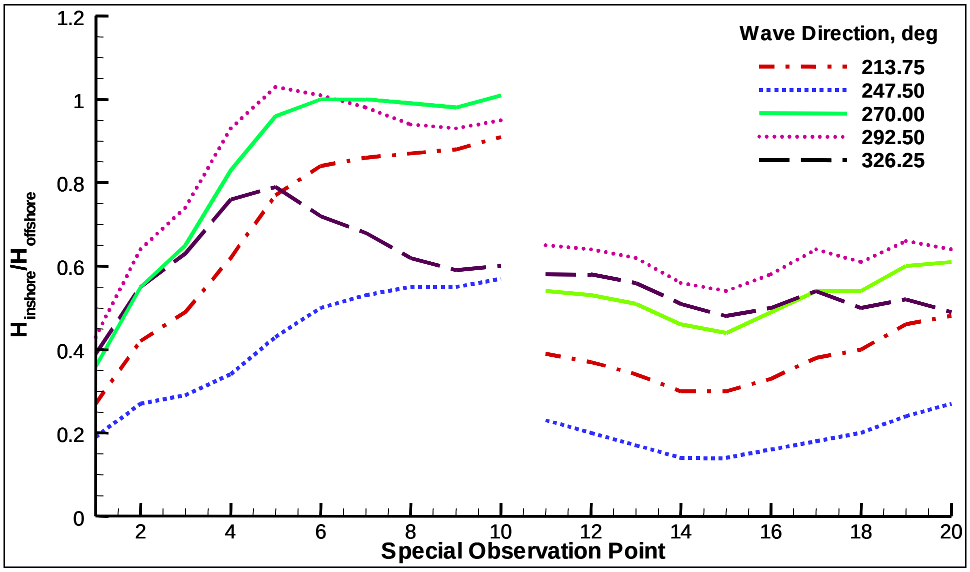


Figure 3-23. Wave attenuation (1m, 8s waves, ebb tide)

Wave Transformation Model

A numerical interpolation method was developed to transform the CDIP Buoy 036 waves inshore to the western boundary of Half Moon Bay, based on the STWAVE model results. The STWAVE results developed a discrete data set of nearshore H_{m0}, T_p, and a as a function of offshore conditions. This data set was used to build a simplified wave transformation model.

A series of transformed height and direction matrices were assembled according to offshore wave direction. Twenty matrices, corresponding to the five modeled directions and four tidal stages, were created for each point of interest in the model domain. Each matrix contains transformed wave height and direction data for any height and period combination between the modeled values. When an offshore wave condition is transformed according to this method, the corresponding nearshore height and direction are determined using the transformation matrices. The transformed wave height and direction are found by linearly interpolating between the transformation matrices for the wave directions greater and lesser than the input offshore condition. If an input wave direction falls above or below the modeled bin limits, the wave is transformed using a linear scaling of its nearest neighbor within the model limits. The transformed wave conditions are output for each of the modeled tidal stages, and can be used to approximate specific water level conditions. Figure 3-24 graphically depicts an example of the wave transformation matrices. The five directional matrices for a flood tide at observation point 5 are shown.

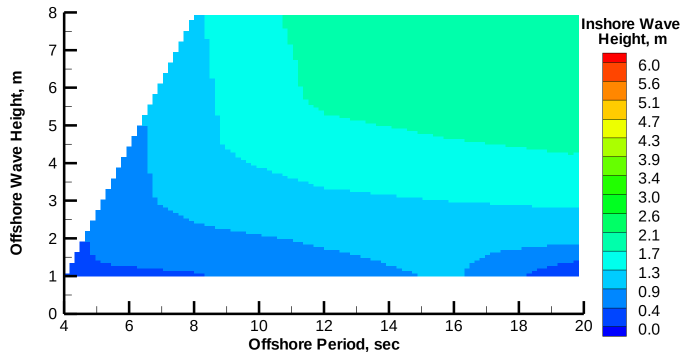
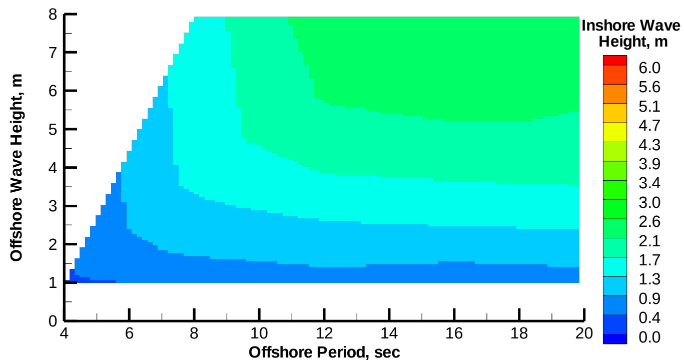
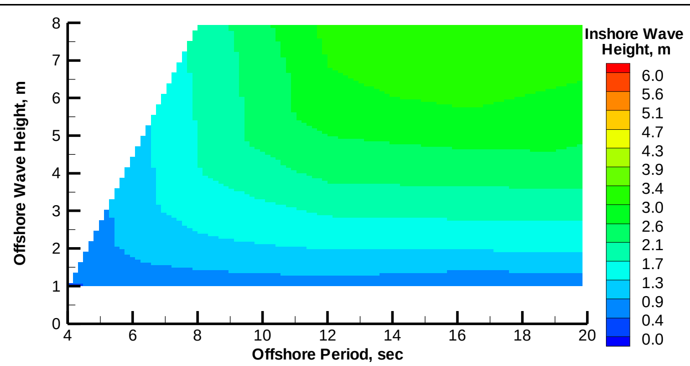
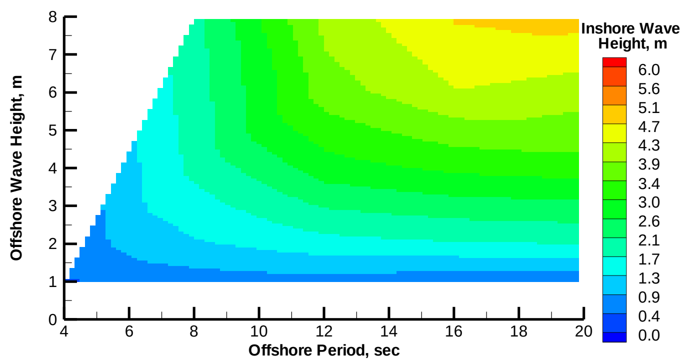
<table>
<tr><th>Matrix</th><th>Mean Offshore Wave Direction, deg</th></tr>
<tr><td>
</td><td>213.75</td></tr>
<tr><td>
</td><td>247.50</td></tr>
<tr><td>
</td><td>270.00</td></tr>
<tr><td>
</td><td>292.50</td></tr>
</table>

Figure 3-24. Directional matrices of transformed wave heights at observation point 5, flood tide conditions

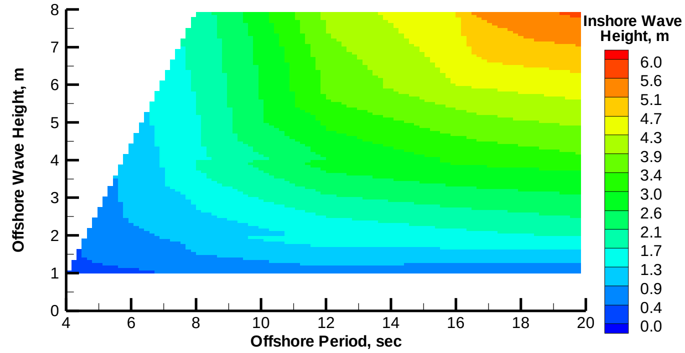
Matrix	Mean Offshore Wave Direction, deg
	326.25

Figure 3-24. (continued from previous page) Directional matrices of transformed wave heights at observation point 5, flood tide conditions

To validate the interpolation model, transformed waves were once again compared to the conditions measured at Station 5 and HMB1. The CDIP Buoy 036 time series data that corresponded to the recording periods of the inshore wave measurement stations were transformed using the model. Depth data from the inshore measurement stations was used to determine the approximate tidal stage for each of the waves. The standard error of estimate was calculated for both the wave heights and directions. Comparisons of measured and predicted results are summarized in Table 3-6.

Table 3-6
Standard estimate of the error for the predicted wave heights and directions

	Standard Error of the Estimate			
Measurement Station	**Predicted Using the Steering Module**		**Predicted using the Simplified Wave Transformation Model**	
	H_{m0}, meters	**Direction, deg**	**H_{m0}, meters**	**Direction, deg**
HMB1	N/A	N/A	0.22	15.2
Station 5	0.20	6.4	0.25	13.7

The transformation model appears to provide a reasonable prediction of the waves conditions in Half Moon Bay. The differences in the standard error of the estimate between the predicted waves using Steering Module and the simplified transformation model at Station 5 are 0.05 m in wave height and 7.3 deg in direction. Modeling of a complete annual series of wave conditions using the SMS ADCIRC-STWAVE Steering Module would be extremely time-consuming and impractical.

Another method of assessing the transformation model is to compare the annual wave energies, measured and predicted, at Station 5 and HMB1. In order to do this, transformed data from ebb, flood, high water, and low water tidal stages were averaged, assuming that all conditions have an equal probability of occurring at each of the four tides. This data is summarized in Figure 3-25.

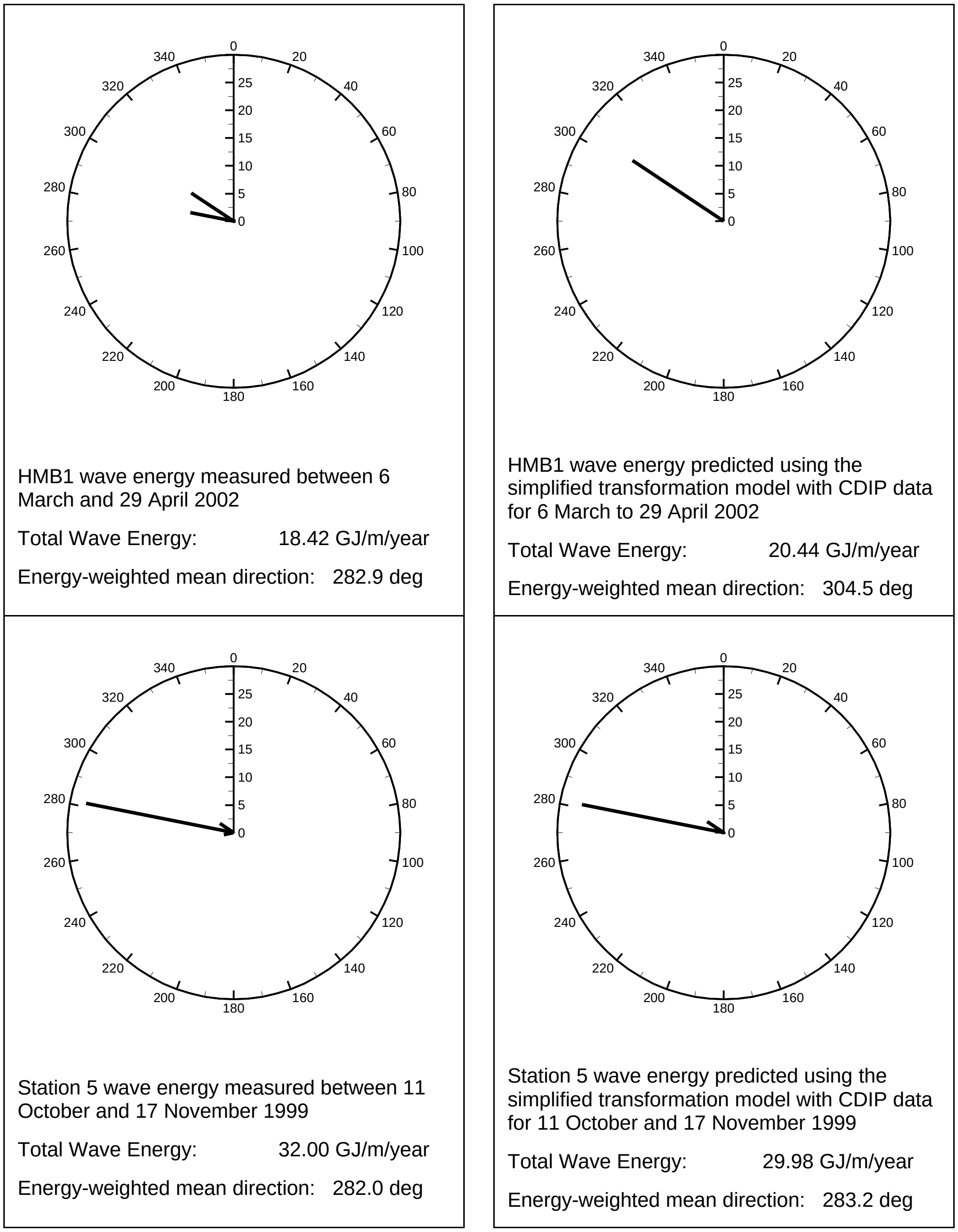


Figure 3-25. Comparison of energy roses of measured and transformed data, values in GJ/m

The simplified wave transformation model can also be used to study expected annual wave energies along the western boundary of Half Moon Bay, as shown in Figure 3-26. All waves along the western boundary of Half Moon Bay have refracted, arriving from 255 to 290 deg. The largest average annual energies occur at observation points 10 and 7, both of which are less than half that measured at CDIP Buoy 036. Analysis of predicted energies for all observation points indicate that the average annual energy is 231.1 GJ/m/year and the energy-weighted mean wave direction is 272.6 deg.

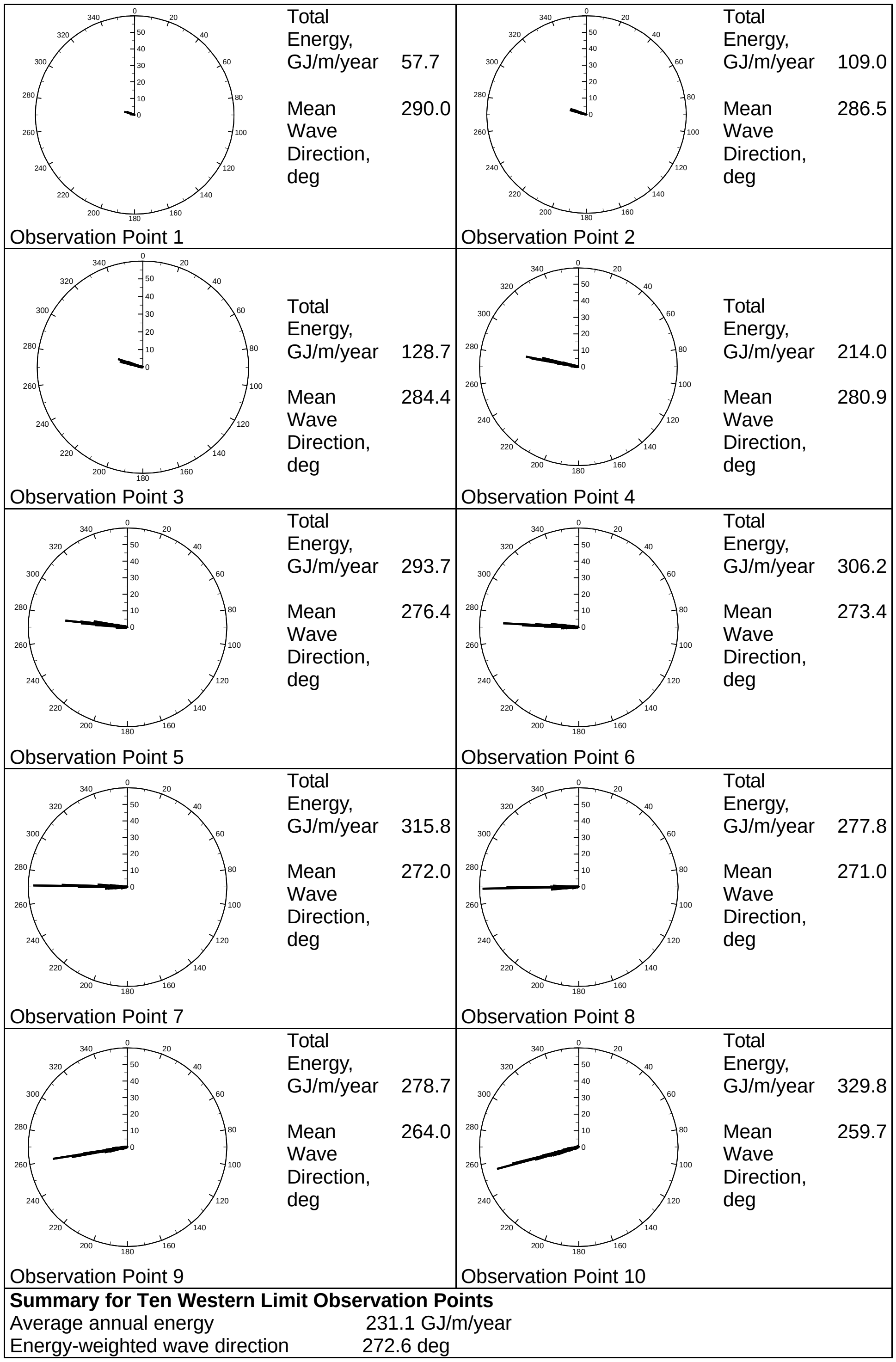


Figure 3-26. Computed wave energies and directions at observation points along the western Half Moon Bay boundary

Summary and Discussion

The limited nearshore wave measurements performed at Half Moon Bay were supplemented with the longer term wave record measured by CDIP Buoy 036, located just offshore of the entrance to Grays Harbor. The measured offshore wave conditions can be transformed into the bay using numerical wave and current models. The resulting transformed wave climate serves as the basis for model test boundary conditions for detailed physical and numerical models of Half Moon Bay. These models, in turn, can be used to evaluate alternatives to preventing erosion of the bay shoreline and breaching of the South Jetty.

Since the water level influences the wave and current climate near the shore, the water levels at the site were assessed, especially the anticipated extreme levels, which are typically associated with periods of highest shoreline damage. The Half Moon Bay region has a typical tidal range of 2.2 m (7.2 ft) with a spring tide range of 2.8 m (9.2 ft). Based on the available water level data near Half Moon Bay, median high water levels are approximately 9 ft (mllw) with water levels above 12 ft occurring approximately twice per year. Water levels 11.5 ft and above occur about one percent of the time.

Analysis of measured wave data just offshore of Half Moon Bay at CDIP Buoy 036 indicates the most frequent waves are from the quadrant centered around 270 deg with typical significant wave heights of 1.5 to 2 m and peak periods of 9 to 11 s. The computed average wave energy is 831.3 GJ/year. The largest recorded storm waves in the past 20 years had a significant wave height of 9.75 m, a peak period of 16.7 sec, and a peak direction of 265 deg.

Both measured and modeled waves at the boundary and into Half Moon Bay are reduced substantially from these offshore measured values.

The wave conditions along observation points 1 through 10 in the present analysis can be used to evaluate the required input wave conditions for the physical model of Half-Moon Bay and establishment of an equivalent wave climate for the model. In the physical model, several discrete wave conditions are being used which are representative of the storm conditions at the bay in order to simulate the observed response of the Half-Moon Bay shoreline over a period of months and years. One way of achieving the effect of storms over time on the physical model is to generate waves of equivalent energies seen by the shoreline. Averaging the annual wave energies over observation points 1 through 10, the approximate physical model boundary, yielded 231.1 GJ/m/year and an energy-weighted average direction of 273 deg. The equivalent wave energy can be obtained in the physical model by running a combination of 400 hours of waves of H_s=2 m, T_p=12 sec and 180 hours of waves of H_s=4 m and T_p=12 sec. This corresponds to a model storm duration of 57 hours of the 2m waves followed by 26 hours of the larger, 4 m waves (using a physical model scale of 1:50).

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Smith, J.M., Sherlock, A. R., and Resio, D.T. 2001. STWAVE: Steady-state spectral wave Model User's Manual for STWAVE Version 3.0. CE-ERDC/CHL IR-01-1, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS

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4 Functional Alternatives for Prevention of Breach Recurrence

Introduction

This chapter outlines the evaluation criteria and development of functional alternatives for the prevention of breach recurrence at the South Jetty. Over the first year of study, technical criteria were established and numerous alternatives were proposed. These alternatives were screened to identify engineering feasibility and physical constructability. Various “soft” and conventional engineered alternative solutions to breaching were discussed at a stakeholders meeting organized by the Seattle District Corps of Engineers and held at the City of Westport on 12 March 2003. A preliminary list of alternatives was developed by the project team in a follow-up meeting at Seattle District. Subsequent modeling and analysis lead to refinement of the list of alternatives by the project team in February 2004. This chapter documents the preliminary engineering and analysis of each alternative. The purpose is to develop a preliminary design of each alternative for possible evaluation by physical and numerical modeling. Included are the approximate layout and dimensions that may be required for each alternative. A preliminary screening of a portion of the alternatives is accomplished by application of planshape equilibrium analysis. In addition, a review of the engineering alternatives developed in 1994 is included in Appendix C.

Criteria

The objective is to prevent the threat of a recurrence of breaching of South Beach at the South Jetty. Meeting this objective requires solution of the ongoing erosion of Half Moon Bay on a long-term basis through the most efficient and practical combination of engineering solutions and sediment placement management. Prevention of a breach essentially requires keeping South Beach shoreline away from Half Moon Bay. Since a breach can originate from either the South Beach or Half Moon Bay side, a proper solution to the breach threat will also include consideration of management of South Beach. The work described herein focuses on increasing the buffer between Half Moon Bay and the South Beach and stabilizing the shoreline position of Half Moon Bay.

A solution to the erosion on the Half Moon Bay side of the south spit must meet the following criteria:

1. have a project life of at least 25 years.
2. prevent breaching of the south spit (protect jetty, navigation channel, backbay including Westport).
3. achieve a predictable dynamic equilibrium shoreline position in Half Moon Bay.
4. have a constructible initial cost.
5. have a constructible maintenance cost.
6. present an acceptable risk, or low risk of failure.
7. be permittable (with respect to endangered species, other environmental concerns, recreational concerns, and aesthetic concerns).
8. be compatible with potential future changes in the Federal navigation project (including, for example: channel relocation northward; deepening).
9. preserve native sediments and an inter-tidal area.
10. limit cross-shore and longshore sediment transport so that infill to the Grays Harbor Navigation Channel is not increased and seasonal beach profile changes are acceptable.

Prior Studies

Dean, et al (1994), prepared a report for the US Army Corps of Engineers, District Seattle in response to the December 1993 breach that included engineering alternatives to remedy the breach.

Table 4-1 lists the alternatives considered in 1994. For ease of reference, illustrated Figures from the 1994 report are provided in Appendix C.

Table 4-1
1994 Engineering Alternatives

No.	Alternative	Description
1	Beach revetment.	2,500 ft revetment along Half Moon Bay shoreline and 5,000 ft revetment along South Beach.
2	Revetment and jetty extension.	Extend South Jetty and existing Half Moon Bay revetment until they intersect.
3	South Jetty spur groin.	2,000 ft spur groin perpendicular to South Jetty and 2000 ft offshore from the South Beach mhhw contour.
4	Reinforcement of the jetty.	Increase the cross-section area of the 2,000 ft eastern end of the South Jetty.
5	Nearshore berms.	Regular placement of dredged sand in nearshore berms between the –20 and –40 ft contours off the South Beach and in another berm in Half Moon Bay.
6	Direct beach nourishment.	Regular placement of dredged sand directly in the breach area.

7	Relocate entrance channel.	Relocate the entrance channel from its current location adjacent to the South Jetty to mid-way between North and South Jetty.
8	Relocate bar channel.	Consider relocating the bar channel if entrance channel relocation is selected.

Adopted Measures

As documented in Chapter 2, the interim solution adopted by Seattle District to prevent re-breaching at the South Jetty includes a combination of several of the above alternatives. The interim solution includes the following measures: placement of 600,000 cu yd of sand dredged from the navigation channel in 1994, extension of the Point Chehalis revetment and burial with a sand fill in 1998 and 1999, and construction of the wave diffraction mound and gravel transition beach in 1999 and 2000. Sand dredged from the navigation channel has also been placed in the nearshore at Half Moon Bay and on South Beach.

Alternatives

Numerous alternatives were proposed and screened in the first year of the present study to identify feasible engineering and physically constructible alternatives within the broad criteria defined above. Eleven alternatives passed preliminary screening and are listed in Table 4-2. This section documents the preliminary engineering and analysis for each alternative. The intent is to develop the layout and dimensions of each alternative so that it may be tested using physical and numerical modeling. Each alternative is also shown in Figures 4-1 through 4-11.

Alternative A – Modification of the eastern terminus of the South Jetty

The first set of alternatives involves modifications to the eastern terminus of the South Jetty based on the concept of shifting the control point for wave diffraction and reducing the wave energy on the Half Moon Bay shoreline. The alternatives in this class include raising the submerged portion of the South Jetty to the east of the present diffraction mound (Alt A1), increasing the size of the wave diffraction mound (Alt A2) and adding a spur to the diffraction mound (Alt A3).

Alt A1. Raise submerged portion of South Jetty east of mound. This alternative involves raising the submerged portion of the South Jetty to elevation +20 ft mllw, which is the present design elevation of the crest of the South Jetty. Two sub-alternatives are proposed for further analysis, a 250 ft extension (Alt A1_250) and a 500 ft extension (Alt A1_500). The two alternatives are shown in Figures 4-1 and 4-2. In 1999, stone was removed from the eastern terminus of the South Jetty to an elevation of approximately +2 ft mllw. The removed stone was used to construct the diffraction mound. Restoring this portion of the South Jetty would decrease the wave energy reaching the western edge of Half Moon Bay. This could result in a change in wave energy reaching the eastern end of Half Moon Bay, including the area that is protected by the buried revetment and the hardened shoreline at Point Chehalis.

This alternative is similar to solutions that have been developed at other inlets with inner-bank erosion, such as St. Andrews Bay Inlet at Panama City, Florida (Seabergh, 2002). Jetty extension has been effective at protecting the shoreline immediately adjacent to the jetty terminus at other inlets with similar crenulate shaped shorelines. Jetty extension has the potential to shift the erosion problem further into the inlet. At Grays Harbor's South Jetty, the armored shoreline of Point Chehalis provides a downdrift control that would offset any easterly shift of the shoreline.

This alternative has the advantage of having minimal impact on sub-tidal areas. The 250 ft extension would overlay the existing jetty remnant presently at +2 ft mllw. The 500 ft extension would lie over older portions of the remnant jetty. However, the effect on wave, current, and sediment transport patterns needs to be assessed. This alternative is analyzed further with physical and numerical modeling as documented in ERDC/CHL TR-04-XX and Chapters 3 and 5 of this report.

Table 4-2
Alternatives for Half Moon Bay

No.	Alternative	Description
A	Modify east end of the South Jetty.	
A1_500	Raise submerged jetty to elevation +20 ft, 500 ft extension.	Restore the submerged portion of the jetty to its original elevation, for a length of 500 ft.
A1_250	Raise submerged jetty to elevation +20 ft, 250 ft extension.	Restore the submerged portion of the jetty to its original elevation, for a length of 250 ft.
A2	Diffraction mound modification – increase mound size with flatter slope.	Modify the wave diffraction mound to better conform to the original design, and improve wave diffraction and energy dissipation.
A3	Diffraction mound modification – add diffraction spur.	Construct a 300-ft spur from the eastern end of the South Jetty towards the northeast.
B	Point Chehalis control point.	Construct a shore-normal structure at the eastern end of Half Moon Bay to act as a groin to block longshore transport to the east.
C	Submerged berm.	
C1	Segmented submerged breakwater.	Construct structures offshore to reduce wave energy reaching the shoreline and act as a partial barrier to offshore sediment transport.
C2	Continuous nearshore berm.	Place dredged sediment from the Grays Harbor Navigation Channel offshore in Half Moon Bay to nourish the shoreline.
D	Geotube alternatives.	
D1	Geo-terraced revetment.	Protect the shoreline at Half Moon Bay with a series of stepped (terraced) revetments.
D2	Geo-tube perched beach.	Place beach nourishment fill and retain the fill with an offshore, shore-parallel, submerged sill.
E	Gravel/cobble beach.	Place rounded gravel and cobble along the shoreline to reduce erosion and to form a dynamically stable profile.
F	Sand nourishment.	Place sand fill as needed to maintain a stable shoreline location.

Alt A2. Increase size of diffraction mound with reduced slope. This concept involves modifying the existing wave diffraction mound to better conform to the original design and to improve the performance of the mound in terms of energy diffraction and dissipation; the top of the mound is extended seaward and the slope is reduced. The intent is to use wave refraction over the slopes of the mound to re-direct the wave energy diffracting around the end of the South Jetty. The incident wave angle at the shoreline of Half Moon Bay would potentially be transformed to a more perpendicular direction thereby decreasing longshore sediment transport and erosion. In addition, a diffraction mound will dissipate wave energy, resulting in a reduction in wave height at the shoreline and reduced cross-shore sediment erosion. A description of jetty terminus diffraction mound physical models, including the previous physical model for the South Jetty, is contained in Seabergh (2002). This alternative is shown in Figure 4-3. This modified mound geometry is consistent with the size originally proposed in the PI Engineering South Beach Stabilization Analysis (1998). That report recommended a wider mound located at the terminus of the jetty remnant approximately 250 ft east of the present mound. Figure 4-4 shows the geometry of the originally proposed diffraction mound reproduced from PI Engineering (1998). The originally proposed geometry is referred to as Alt A2_98 in further analysis.

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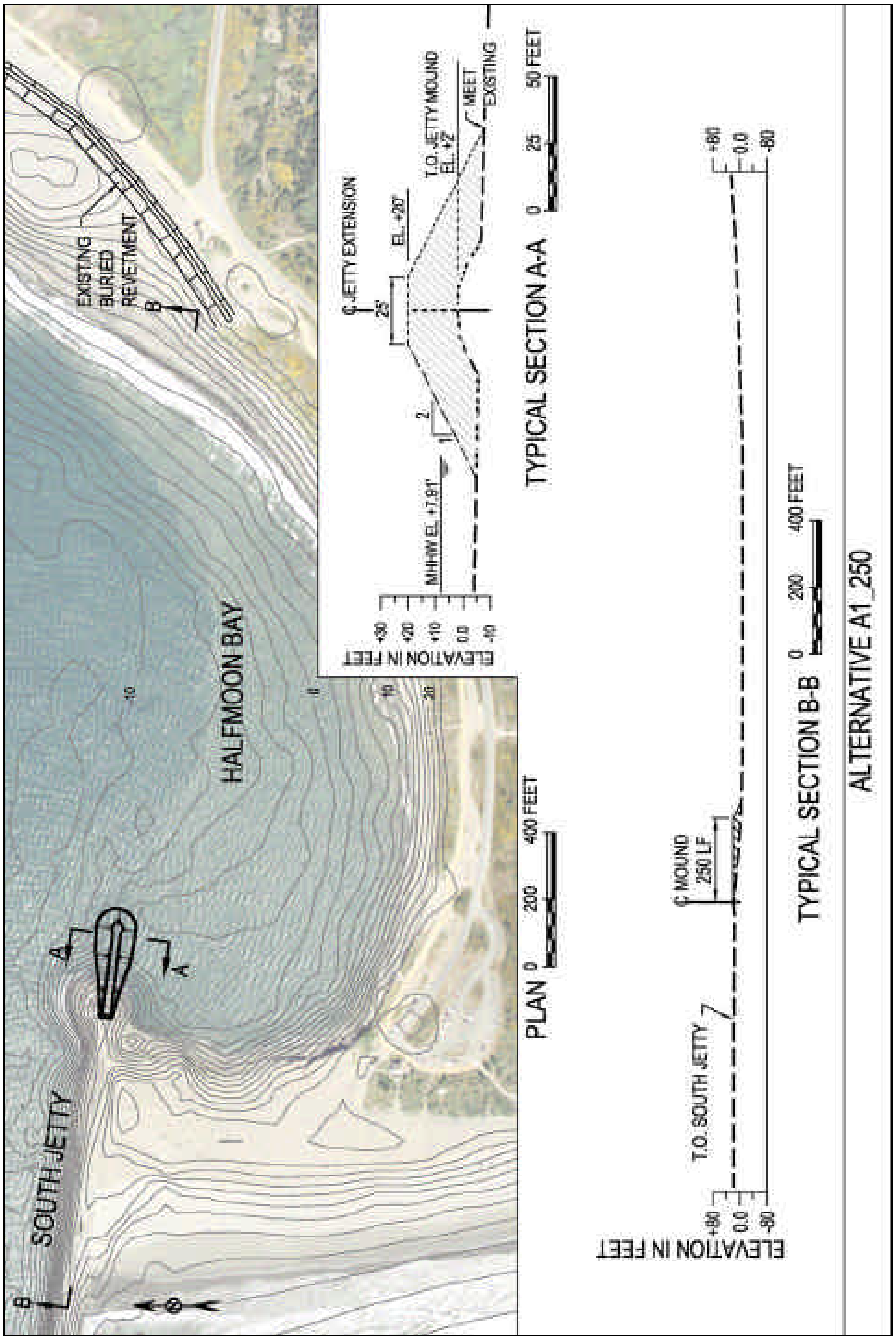

Figure 4-1. Alternative A1_250 raise submerged portion of South Jetty

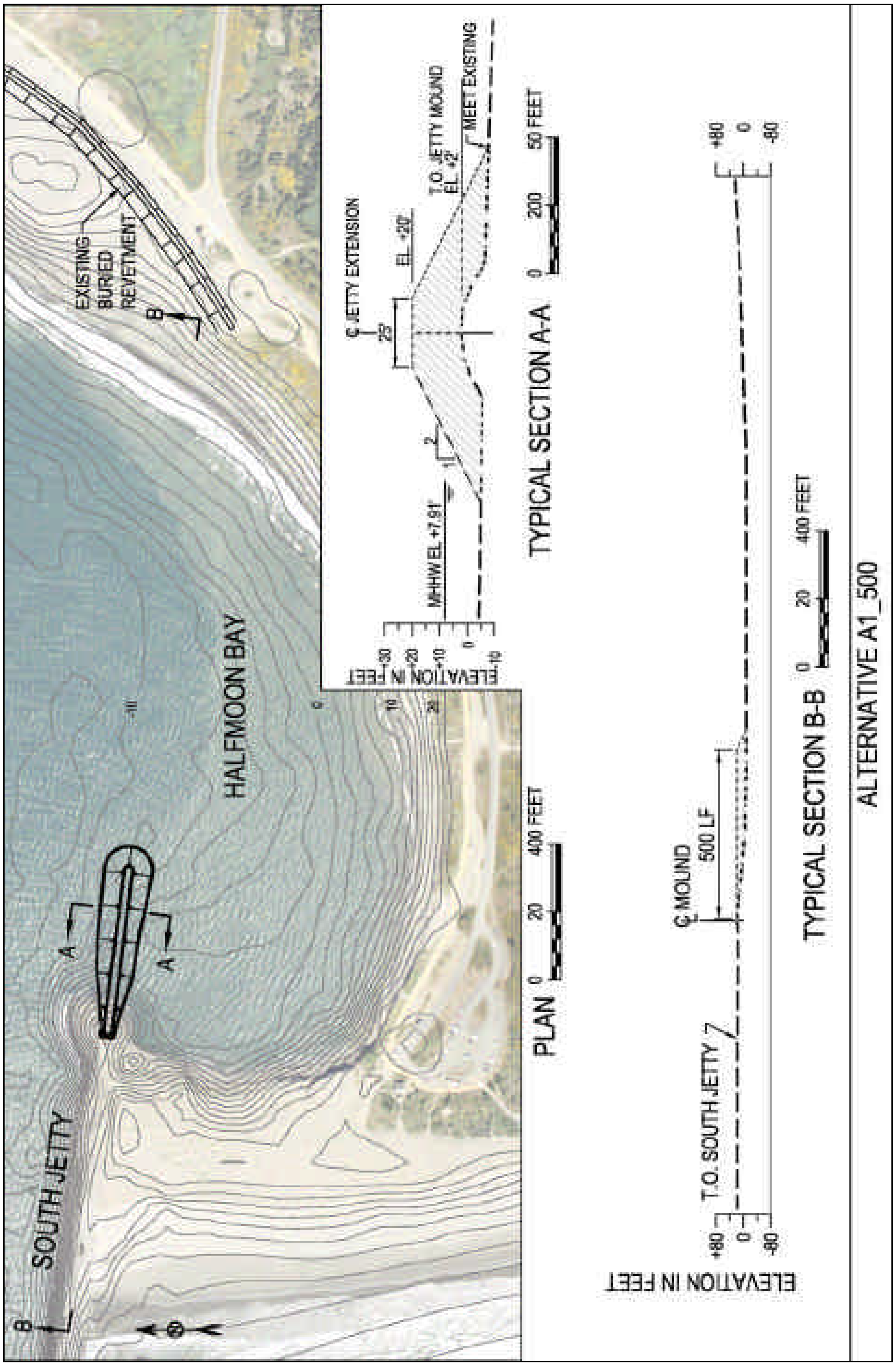

Figure 4-2. Alternative A1_500

Alt A3. Diffraction spur. This alternative consists of building a spur from the base of the South Jetty towards the northeast. A preliminary design consists of a spur 300-ft long with a crest elevation of +20 ft mllw (Figure 4-5). A spur will decrease the amount of wave energy transmitted to Half Moon Bay from outside Grays Harbor and alter the pattern of wave approach to the shoreline at Half Moon Bay, to potentially create more favorable sediment transport conditions.

The structure is a potential hazard to navigation and could be vulnerable to scour at the toe. The alternative is analyzed further by planshape analysis. There is a risk that this structure would deflect any sediment transport heading east along the South Jetty out toward the navigation channel. This can be assessed using both the physical model and circulation modeling.

Alternative B – Point Chehalis control point

This alternative consists of building a shore-normal structure at the eastern end of Half Moon Bay to act as a groin to block longshore transport to the east and thereby prevent or reduce erosion of the shoreline. In theory, it is possible to design a headland control point and beach fill such that the shoreline is in an equilibrium position, with waves arriving normal to the shoreline at all points on the periphery, resulting in no net littoral drift. See USACE (1992a) for a description of the concept and list of references. The concept design is for a shore-normal jetty with a length of 300 ft located on the downdrift beach towards Point Chehalis (Figure 4-6).

This alternative is analyzed later in this chapter using planshape analysis. The long-term sediment transport response to this structure is uncertain and would need to be addressed using both physical and numerical modeling. However, it is a potentially efficient solution that minimizes the impact to sub-tidal and inter-tidal areas.

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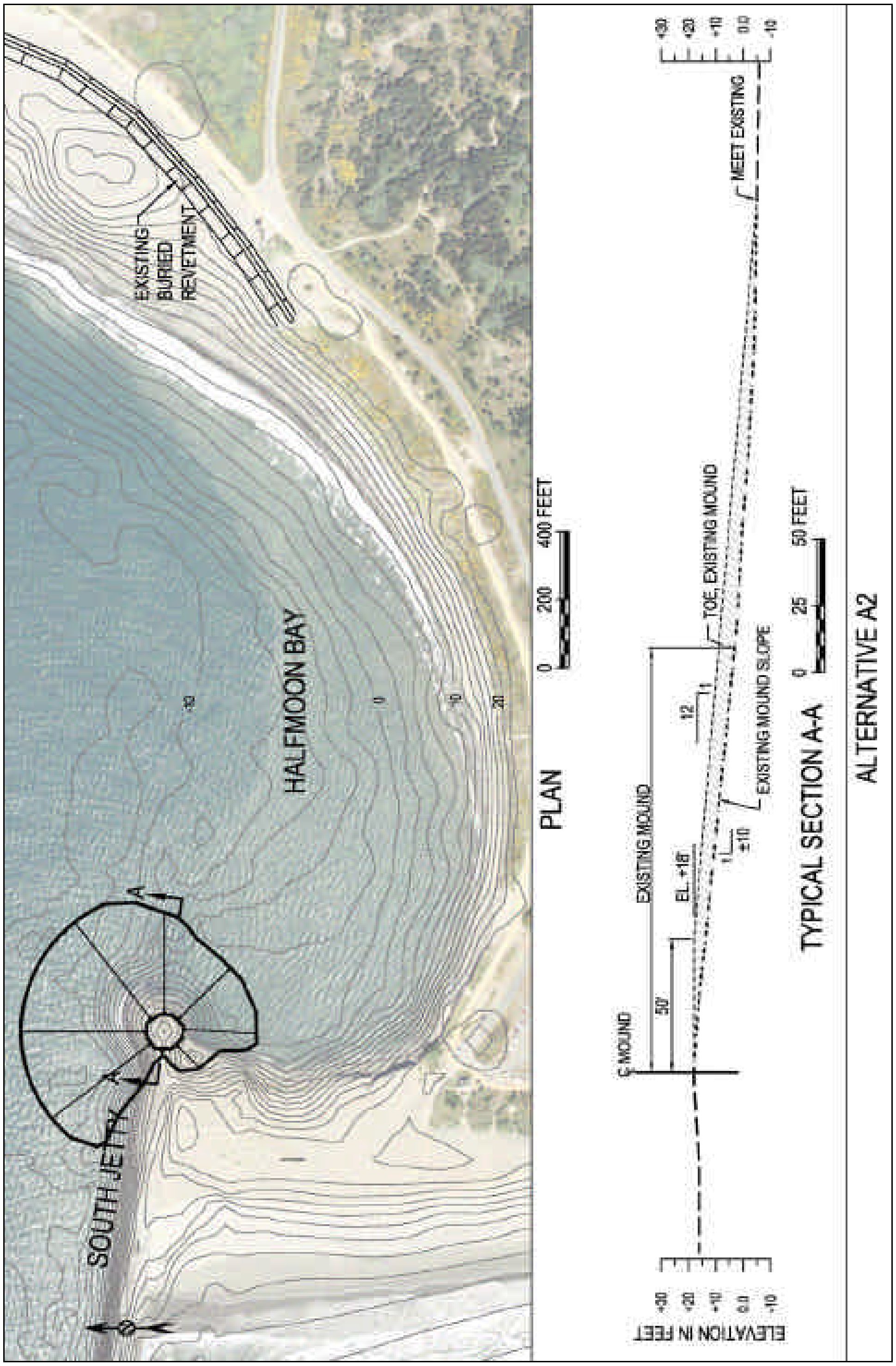

Figure 4-3. Alternative A2 increase size of diffraction mound with reduced slope

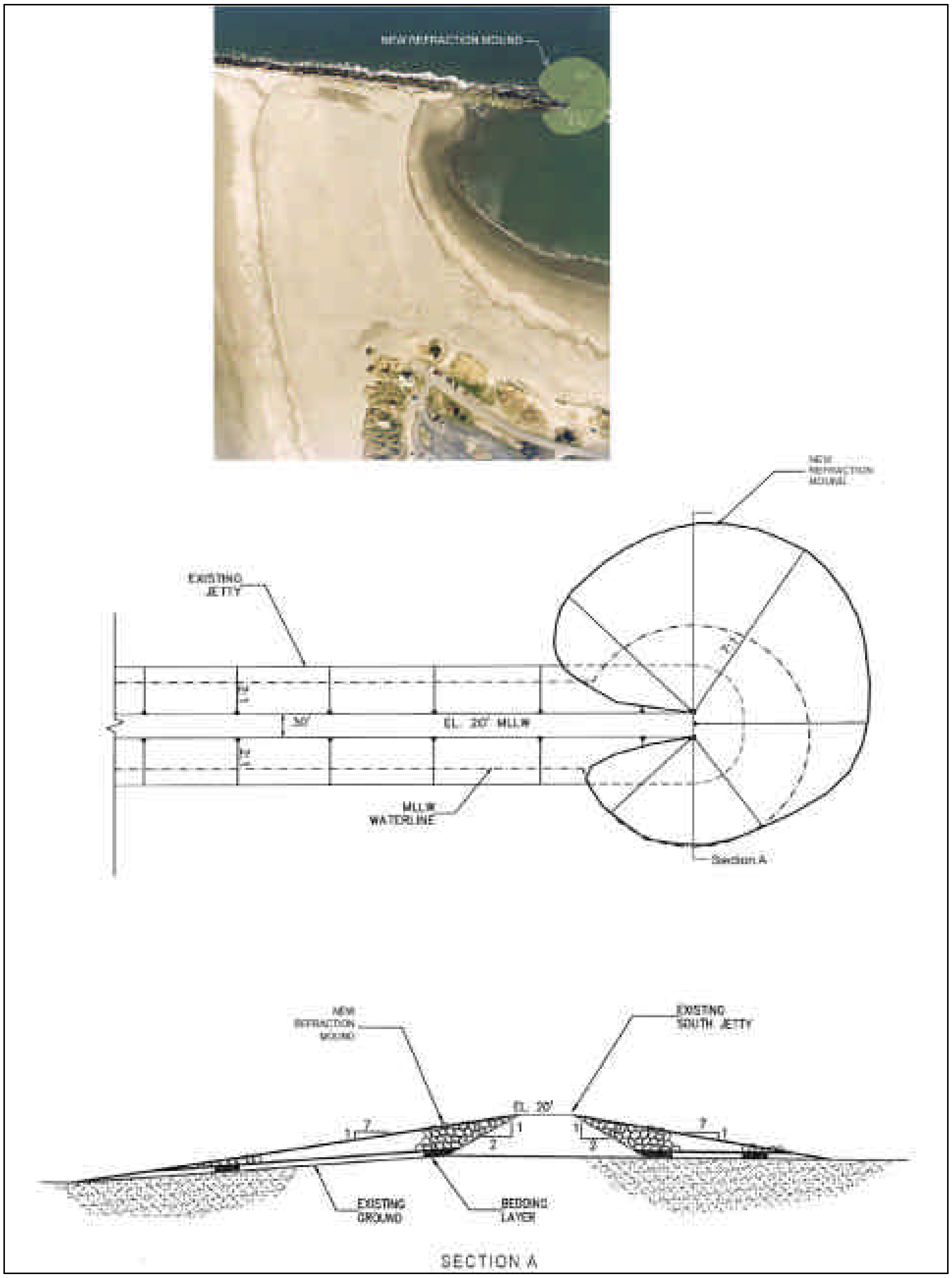

Figure 4-4. Alternative A2_98 Geometry and configuration of the proposed diffraction mound as originally proposed (Figure 30 from PI Engineering, 1998)

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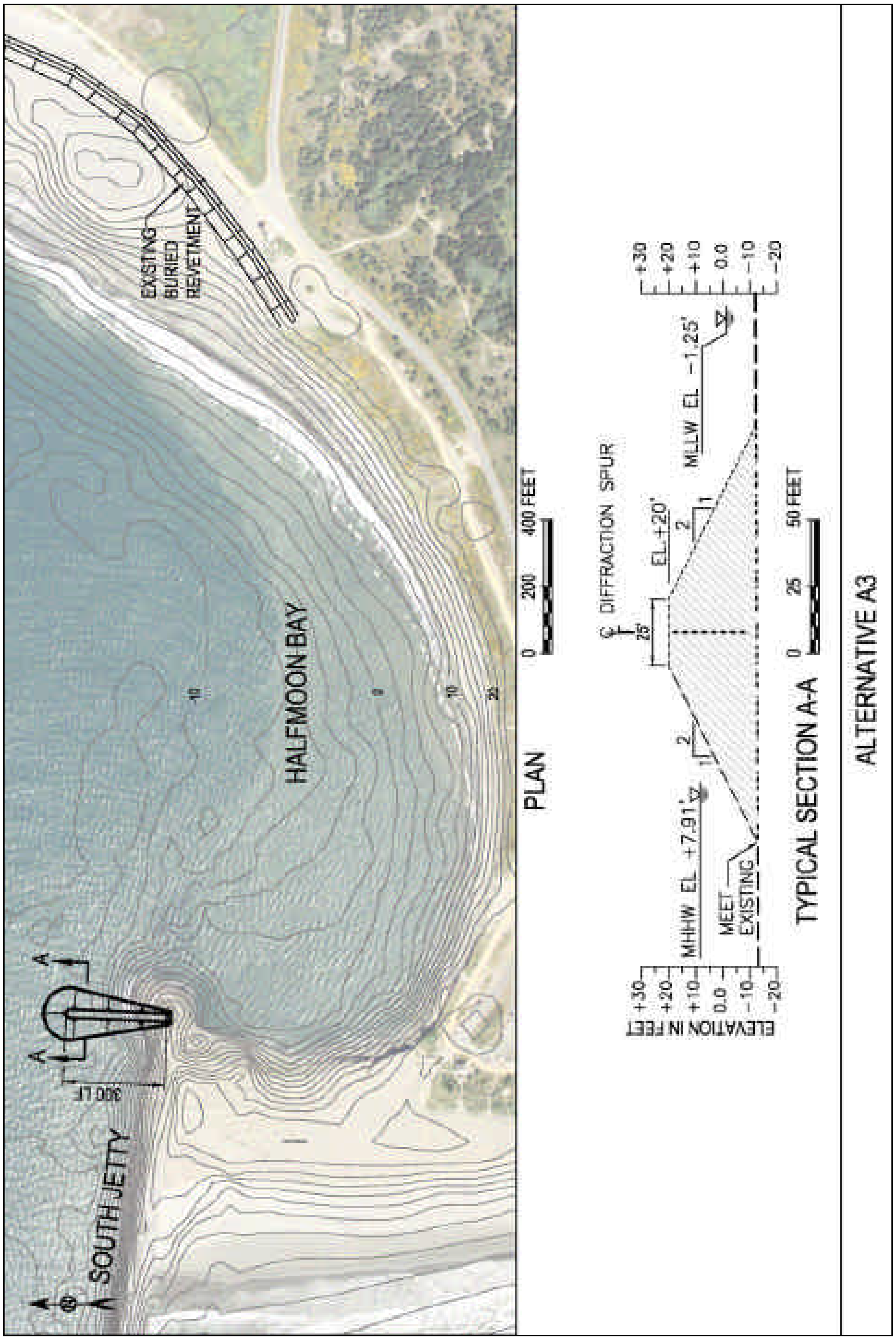

Figure 4-5. Alternative A3 diffraction spur

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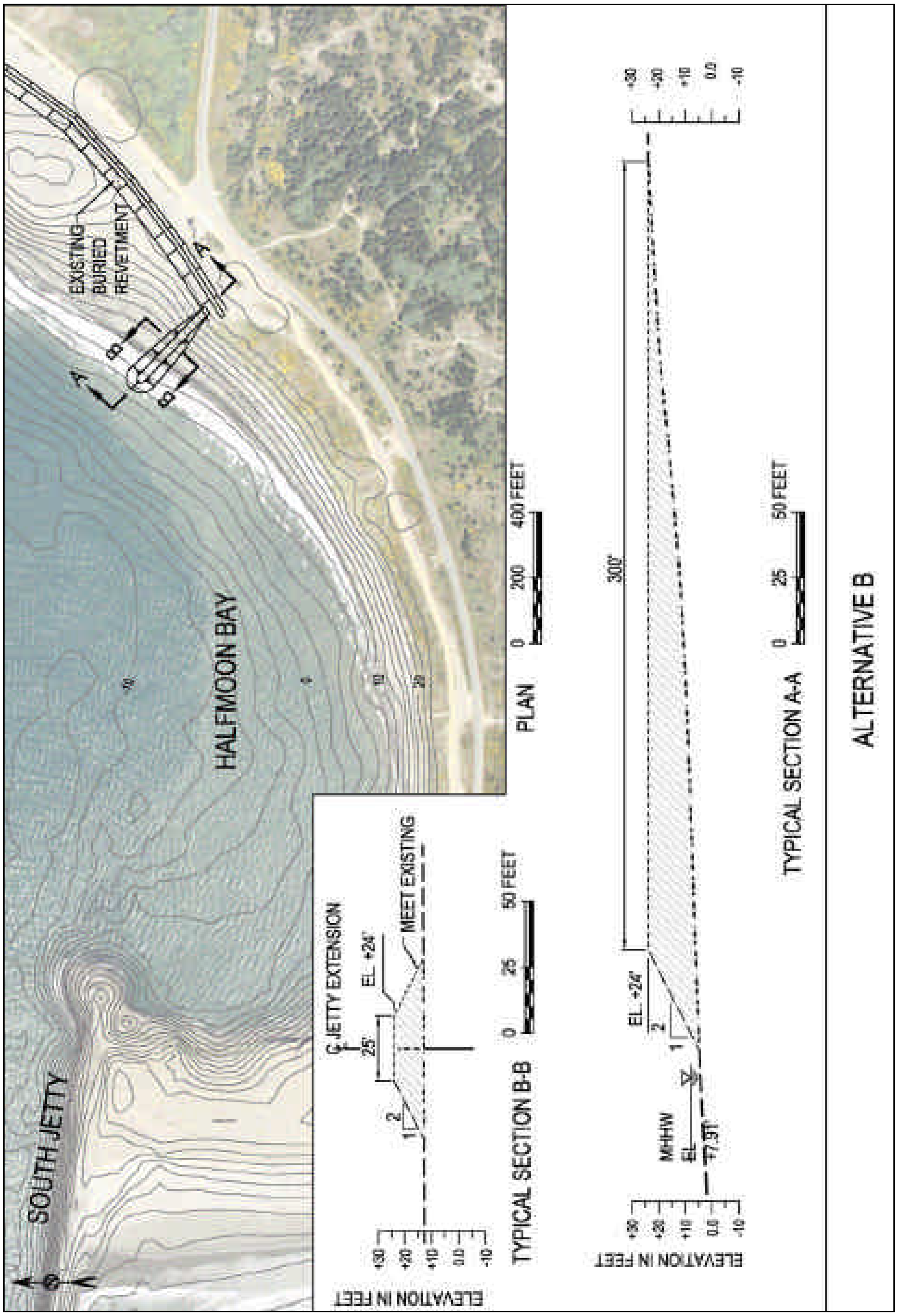

Figure 4-6. Alternative B Point Chehalis control point

Alternative C – Submerged berm or breakwater

Alt C1. Segmented submerged breakwater. The segmented submerged breakwater concept consists of placing structures offshore to reduce the wave energy reaching the shoreline along Half Moon Bay. The breakwaters will also function as a barrier to shore-normal sediment transport. The intent is to prevent erosion and encourage natural sediment accumulation to form a stable beach profile. An important goal is to avoid adverse impacts to surfing conditions in Half Moon Bay. It may be possible to develop a submerged offshore structure that improves the surfing conditions, while also achieving the primary goal of a stable shoreline. A submerged profile is desired to minimize visual impacts and to allow better control of the shape of the beach by allowing a more uniform distribution of wave energy to be transmitted to the shoreline.

A number of articles and publications contain information on segmented breakwaters. Review and design guidance are contained in the US Army Corps of Engineers manual "Coastal Groins and Nearshore Breakwaters" (USACE, 1992a) and technical note "Empirical Methods for the Functional Design of Detached Breakwaters for Shoreline Stabilization" (USACE, 1991). A recent review paper on the design of low-crested and submerged structures is in Pilarczyk, K.W. (2003).

The preliminary design is for five breakwaters with crest lengths of 200 ft, spacing between segments of 200 ft, crest widths of 20 ft, and side slopes of 2:1 (Figure 4-7). The preliminary design configuration will be tested using numerical and physical models. Selection of a stable rock size for the submerged breakwater is beyond the scope of this preliminary design. It is possible that the offshore breakwaters will be constructed using geo-tubes, or some material other than rock. It is anticipated that the structures will need to have a relatively low degree of submergence in order to be effective at reducing wave energy at the shoreline during high tides and storm induced water levels. The high tide range at the project site necessitates a berm that is above water most of the time. Also, long period waves may result in a "pumping" action that increases longshore transport, erosion, and scour at the gaps in the breakwaters. However, the effectiveness of an offshore breakwater increases if it is built high enough, with large enough rocks and small enough gaps between the breakwater segments. The effectiveness of a submerged breakwater scheme at protecting against breach recurrence would have to be assessed using physical and numerical modeling.

Alt C2. Continuous nearshore berm. The nearshore berm concept consists of placing dredged sediment from the Grays Harbor Navigation Channel in Half Moon Bay. The dredged material is fine sand and the design concept is for the material to nourish the shoreline by moving onshore slowly under natural wave action. The preliminary design is for a berm that will raise the elevation of Half Moon Bay to -6 ft mllw (Figure 4-8). Alt C2 would require approximately 50,000 cu yd of sand for construction.

A berm may be able to provide protection to the shore from erosion by reducing the height of waves, functioning in part as a submerged breakwater (Alt C1), provided the submergence is relatively low. A nearshore berm will ideally cause higher energy storm waves to break, while allowing lower energy waves to propagate onto the beach. A nearshore berm may act as a "feeder" berm, providing a sacrificial source of sediment to reduce the movement of sediment offshore or alongshore. During times of accretion, the berm may act as a sand source for the beach profile.

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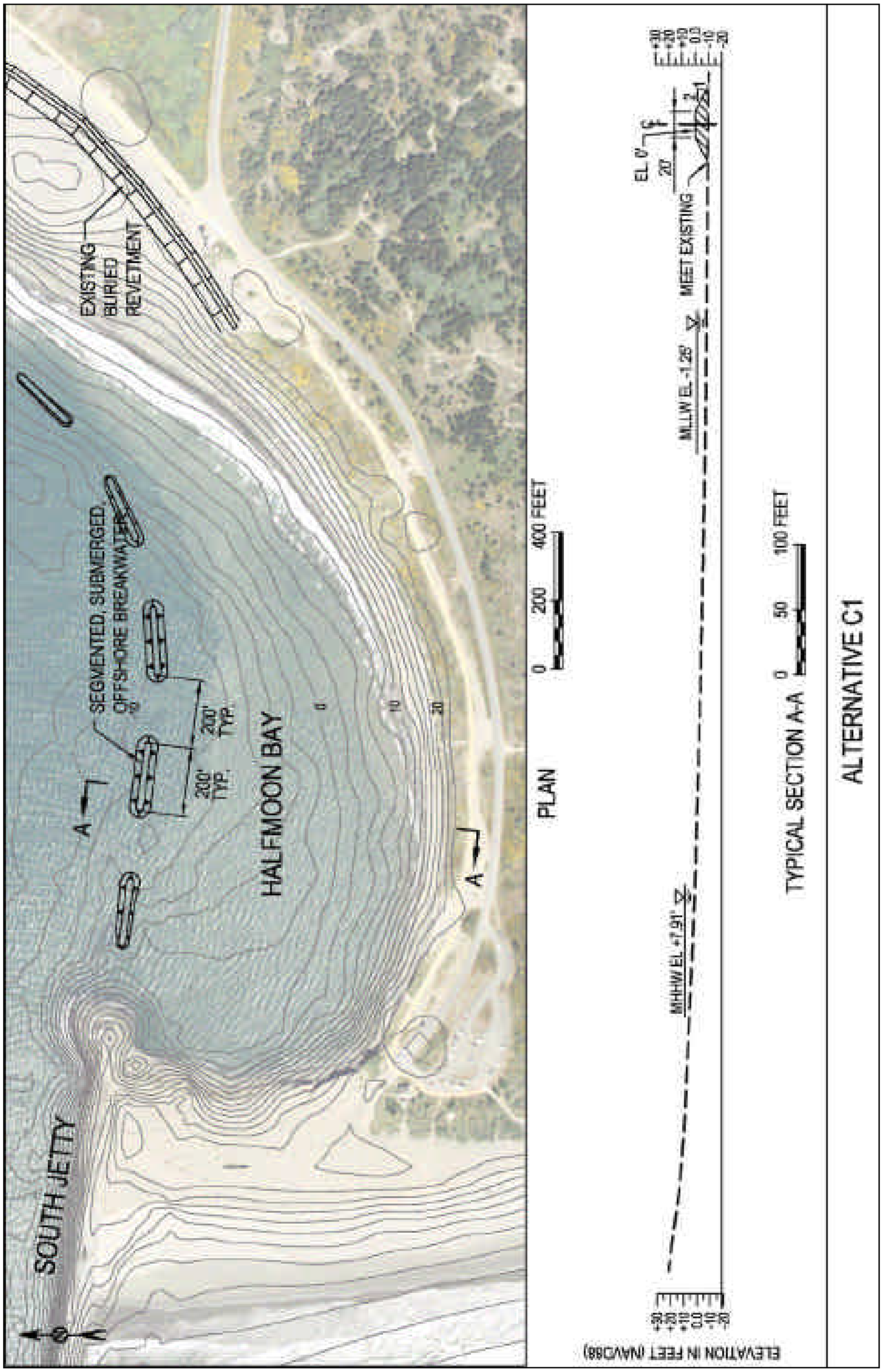

Figure 4-7. Alternative C1 Segmented submerged offshore breakwater

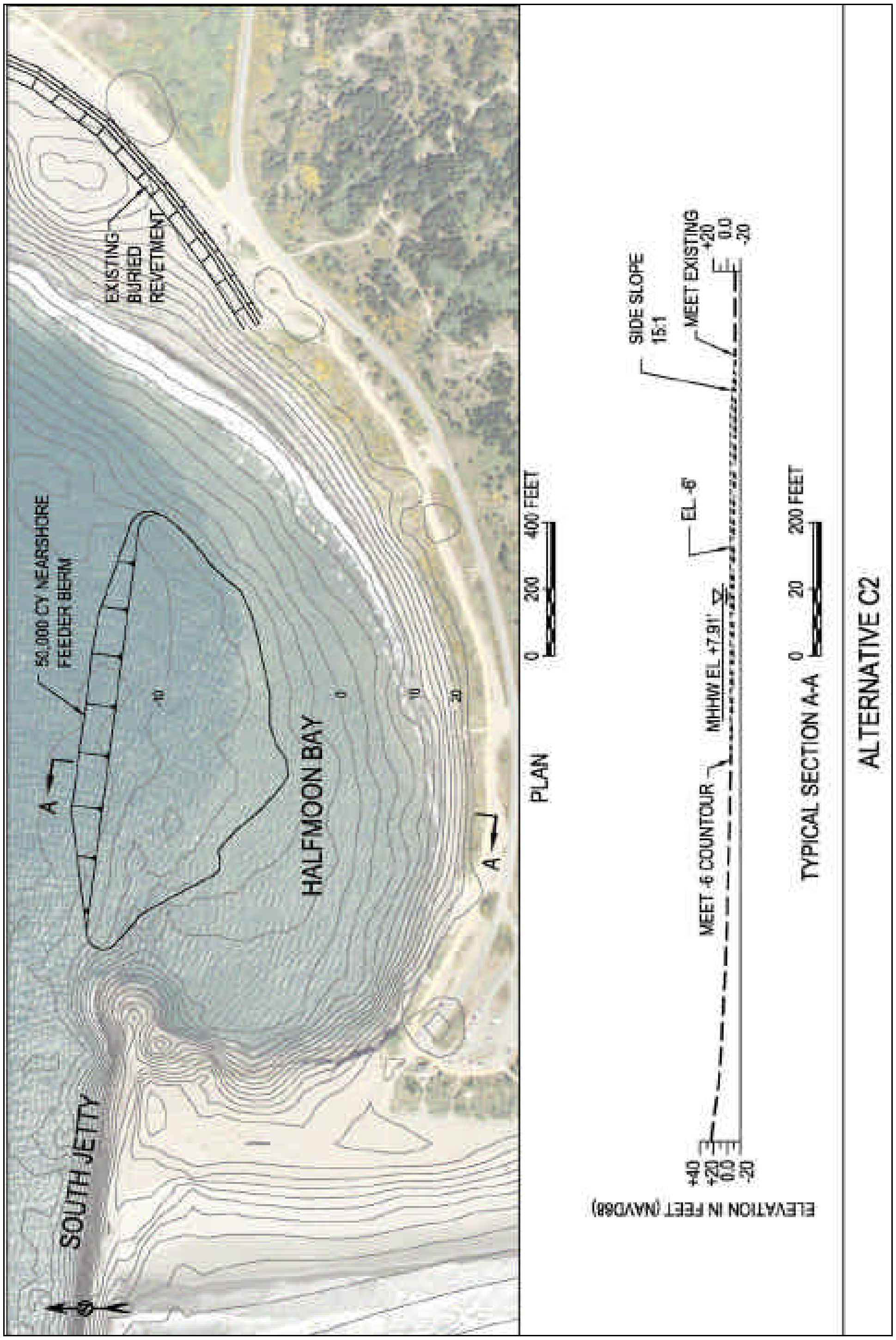

Figure 4-8. Alternative C2 Continuous nearshore berm

Nearshore berms have been applied at a number of locations. A description of existing projects along with engineering design guidance is included in a series of Technical Notes from the US Army Corps of Engineers Dredging Research Program (USACE, 1990a, 1990b, 1992, 1993). An analytical model to predict the response of nearshore berms is presented in Larson and Ebersole (1999). Nearshore berm response has also been predicted in Foster, et al (1996) using the techniques of Kraus, et al. (1991) and Hands and Allison (1991). Monitoring results for nearshore mounds are reported in a number of articles, including Work and Otay (1996), Junke, et al (1989), and US Army Corps of Engineers (1990c).

Predicting the performance of nearshore berms is uncertain. To be effective the berm should be in shallow water, for which direct placement on the beach would probably cost the same and be more effective as shore protection. The likelihood of success of such a scheme increases if coarse sand is used and if placement is done in late spring or early summer. One time placement is unlikely to be effective as a long-term solution. However, a nearshore berm is a beneficial use of dredged material with potentially minimal impact to the environment.

Alternative D – Geo-tube alternatives

Alt D1. Geo-terraced revetment. This alternative will protect the shoreline at Half Moon Bay with a series of "stepped," or terraced revetments. The concept can be understood as a modification of a conventional revetment where geo-tubes are used instead of a rock slope for the revetment. Also, instead of one continuous relatively steep revetment slope, the slope is broken into two or more components with a nearly flat, natural sediment slope or terrace between each geo-tube.

The terracing concept has been successfully applied on the open coast north of Grays Harbor at Ocean Shores using a design developed by ERDC's contractor. The design at Ocean Shores used rock instead of geo-tubes for the revetment slopes. However, geo-tubes were initially used successfully at Ocean Shores as a temporary structure. The alternative shown in Figure 4-9 includes three geo-tubes, intended to prevent erosion between elevations +20 ft and +5 ft NAVD88. Further work is needed to determine the number of geo-tubes required to protect the cross-shore profile at Half Moon Bay, including their elevations and locations.

It is possible that terraced revetments made of rock will be necessary in order to provide adequate durability and resistance to vandalism. Design guidance for coastal engineering with geo-tubes is in Davis and Landin (1997).

As documented in Chapter 2, the City of Westport placed concrete blocks with 7,000 cu yd of sand back-fill as a temporary emergency measure to stop erosion in the southwest corner of Half Moon Bay in October 2003. The temporary concrete block revetment was successful in preventing general scarp recession during a series of storms and high tides that occurred between October 2003 and mid-February 2004, although the structure was damaged and end-effect erosion occurred during its short period of installation. The temporary block revetment was removed in February 2004 to satisfy environmental permit requirements. Nonetheless, the experience indicates that a terraced revetment may be effective as a long-term solution.

Geo-tubes are vulnerable to vandalism, and a terraced slope is a relatively new application for geo-tubes. Design guidance is limited. However, geo-tubes are relatively low cost and may be effective as a type of erosion "insurance" to protect the shoreline during episodes of high tide and high waves. This concept may be most effective in combination with sand nourishment and re-vegetation to maintain a layer of beach-fill and grass cover over the geo-tubes.

Alt D2. Geo-tube perched beach. This alternative consists of a perched beach behind a submerged sill constructed using geo-tubes. The intent is to nourish the shoreline at Half Moon Bay with beach-fill, and to reduce the rate of offshore transport of the beach-fill with a submerged geo-tube sill. The geo-tube helps to confine the material to the upper beach profile, reducing erosion to the offshore. The alternative includes shore parallel geo-tubes near the mean lower low water elevation. As shown in Figure 4-10, the geo-tubes retain beach-fill between elevation +10 ft mllw to the top of the geo-tubes at elevation +9 ft mllw.

Perched beaches have not been studied extensively and design guidance is limited. A review of the literature on perched beaches and a list of references is in USACE (1992b). Information is also contained in a recent review article on low-crested offshore structures by Pilarczyk (2003).

A sill will inhibit offshore sediment transport caused by storm waves. However, a sill will also reduce onshore movement of sediment during long period, swell-type wave conditions. A sill's net effect on onshore/offshore sediment transport has not been quantified (USACE, 1992b).

The perched beach sill design concept can be distinguished from the submerged (and/or segmented) breakwater concept and nearshore berm concept. A sill is primarily intended to retain beach-fill, while a submerged breakwater is primarily intended to reduce the wave energy that reaches the shoreline and a nearshore berm is primarily intended to provide a source of sediment to nourish the littoral system. Nonetheless, all three concepts overlap in function to some extent, especially at Half Moon Bay given the unique morphology and wide range of concepts being considered for the site.

Note that if the sediment transport at Half Moon Bay is primarily longshore, a sill structure will be less applicable. Consideration should be given to end structures to completely enclose the beach-fill. Stauble and Tabar (2003) report some success reducing erosion along the New Jersey shoreline using narrow-crested offshore breakwaters that completely enclose beach-fill between shore-normal groin structures. Geo-tubes are vulnerable to vandalism and design guidance is limited. However, a perched beach is relatively low cost and can enhance the formation and maintenance of a sandy beach profile.

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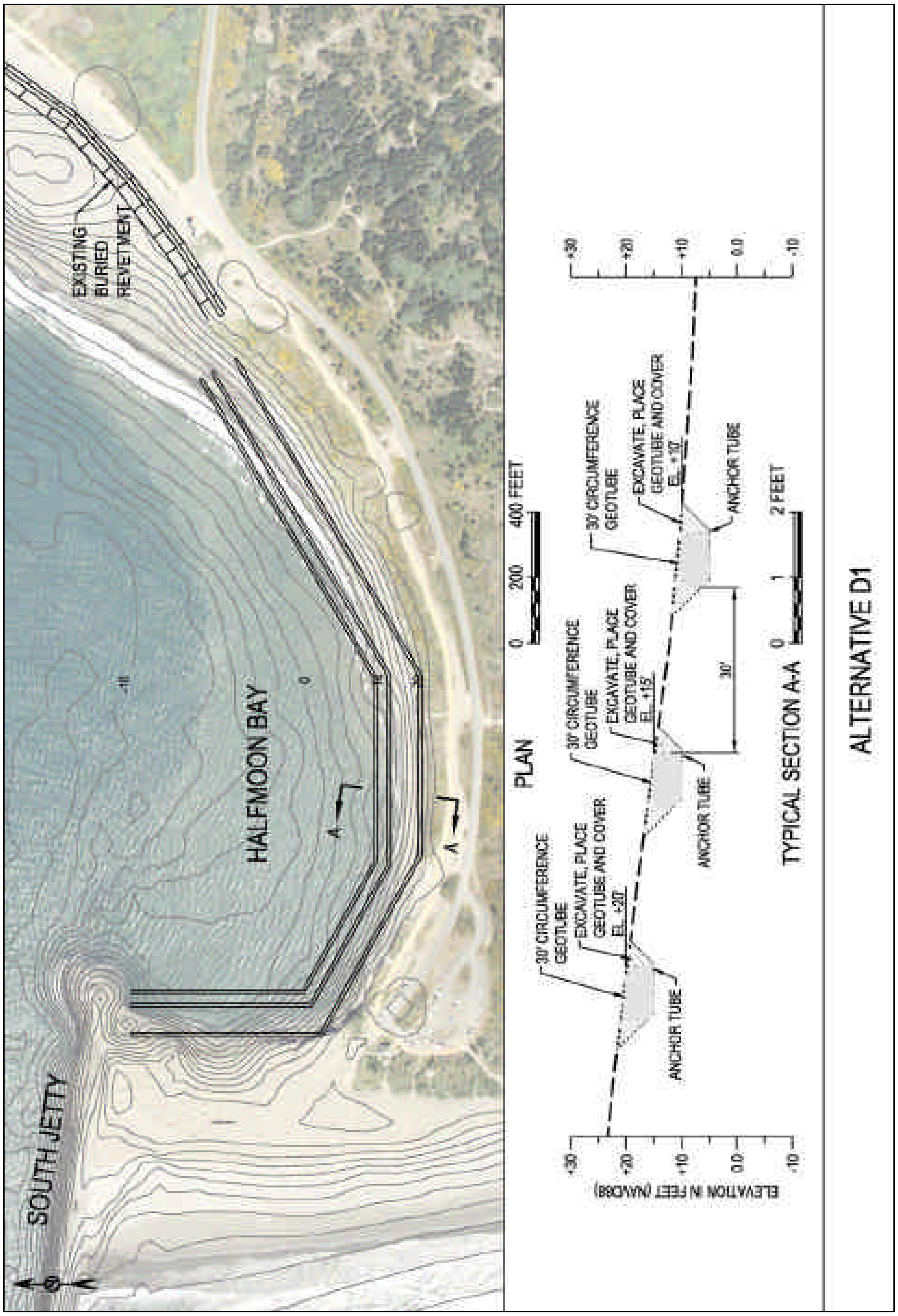

Figure 4-9. Alternative D1 Geo-terraced revetment

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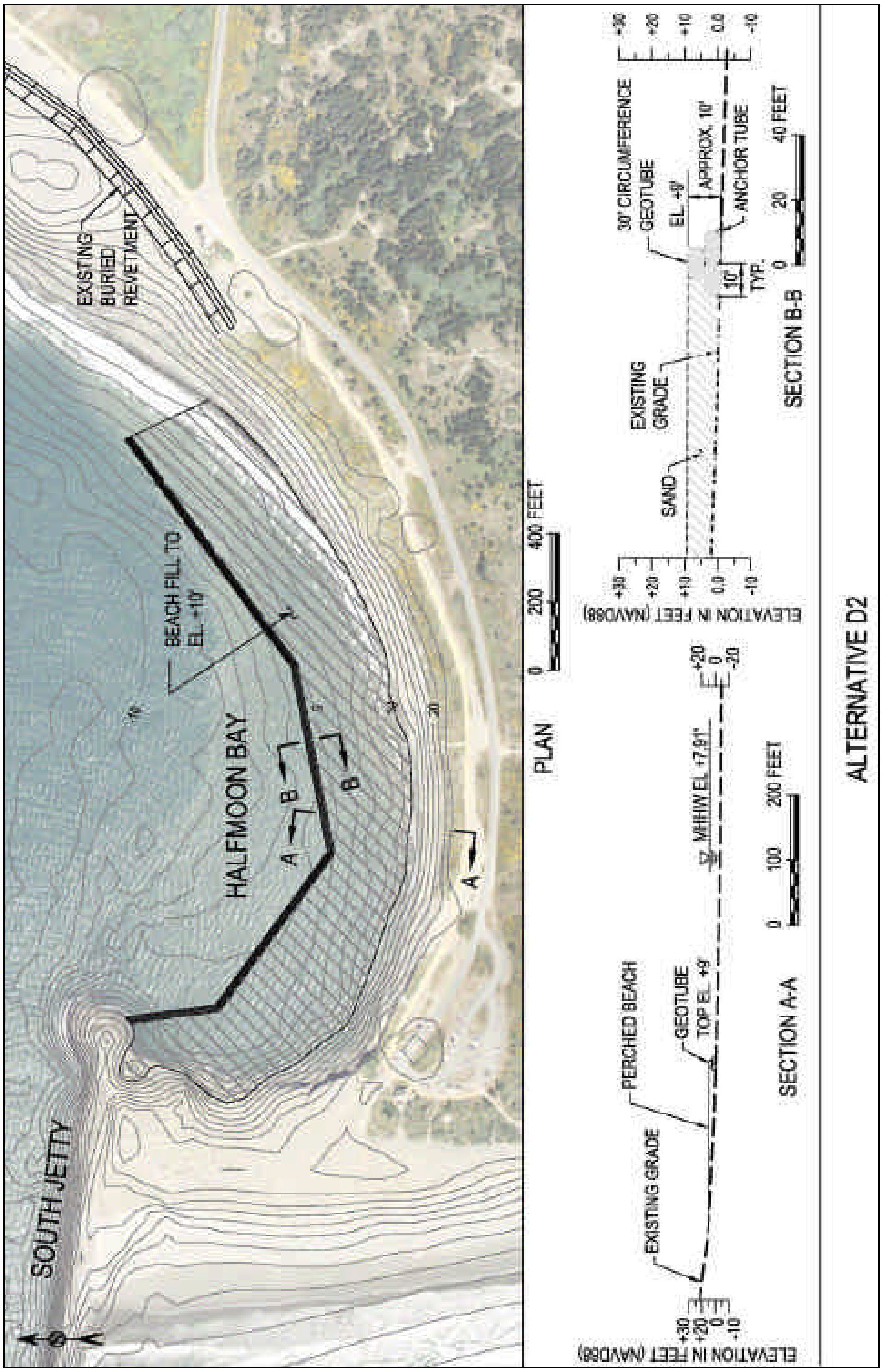

Figure 4-10.   Alternative D2 Geotube perched beach

Alternative E – Gravel and cobble transition beach

The gravel and cobble transition beach alternative involves placement of a volume of gravel and cobble along the shoreline to the south of the eastern terminus of South Jetty. This would provide protection to the sandy breach-fill and an adequate transition from the hard point at the jetty terminus to the natural sandy beach of the Half Moon Bay shoreline. A preliminary design template for gravel berm placement consists of 40 tons of gravel/cobble per ft of shoreline, placed adjacent to the scarp between elevations +20 ft NAVD88 and mhhw +7.91 ft NAVD88 (Figure 4-11).

Gravel and cobble has previously been used effectively at Half Moon Bay to stabilize a portion of the shoreline. A gravel/cobble beach is relatively low-cost and consists of natural beach sediments. However, the coarsening of the beach sediment relative to the native sand material is undesirable. The performance is uncertain and design guidance is limited.

Alternative F – Sand nourishment.

This alternative involves periodic placement of sand in the southwest end of Half Moon Bay to replace sandy material lost by erosion. It is anticipated that an initial placement volume of 30,000 cu yd of sand would be required for the initial placement to restore contours to March 2003 condition. Measurements of shoreline position published by Osborne, Wamsley, and Arden (2003), indicate that the shoreline in the southwest corner of the beach has receded at an average rate of 34.4 ft/year (10.5 m/year). A series of storms in October 2003 caused bank recession of approximately 33 ft and loss of approximately 30,000 cu yd of sand.

Significantly larger volumes would be required to restore the beach contours to their 2001 condition. Storms in November and December 2001 caused severe erosion of the shoreline, a temporary construction haul road used to transport armor rock to the South Jetty was breached by erosion, and three large rainwater runoff gullies were cut through the breach-fill.

The alternative recommended for further testing and analysis is shown in Figure 4-12 and consists of placing 40 cu yd of sand fill per linear ft of shoreline between the scarp and mean higher high water. The sediment placement spans 1,200 ft of shoreline, including a transition region at the end of the placement area.

It may be feasible to combine sand nourishment with a control structure alternative, such as a submerged sill or groin. The result would be a “hybrid” alternative that may be the final preferred alternative. Further analysis and consideration is recommended of hybrid alternatives, based on the results of model testing and analysis of alternatives.

Sand nourishment is desirable because it provides a natural sandy beach profile. However, it will likely require frequent maintenance and re-nourishment without the use of control structures (groins, sills, breakwaters) to retain the beach-fill.

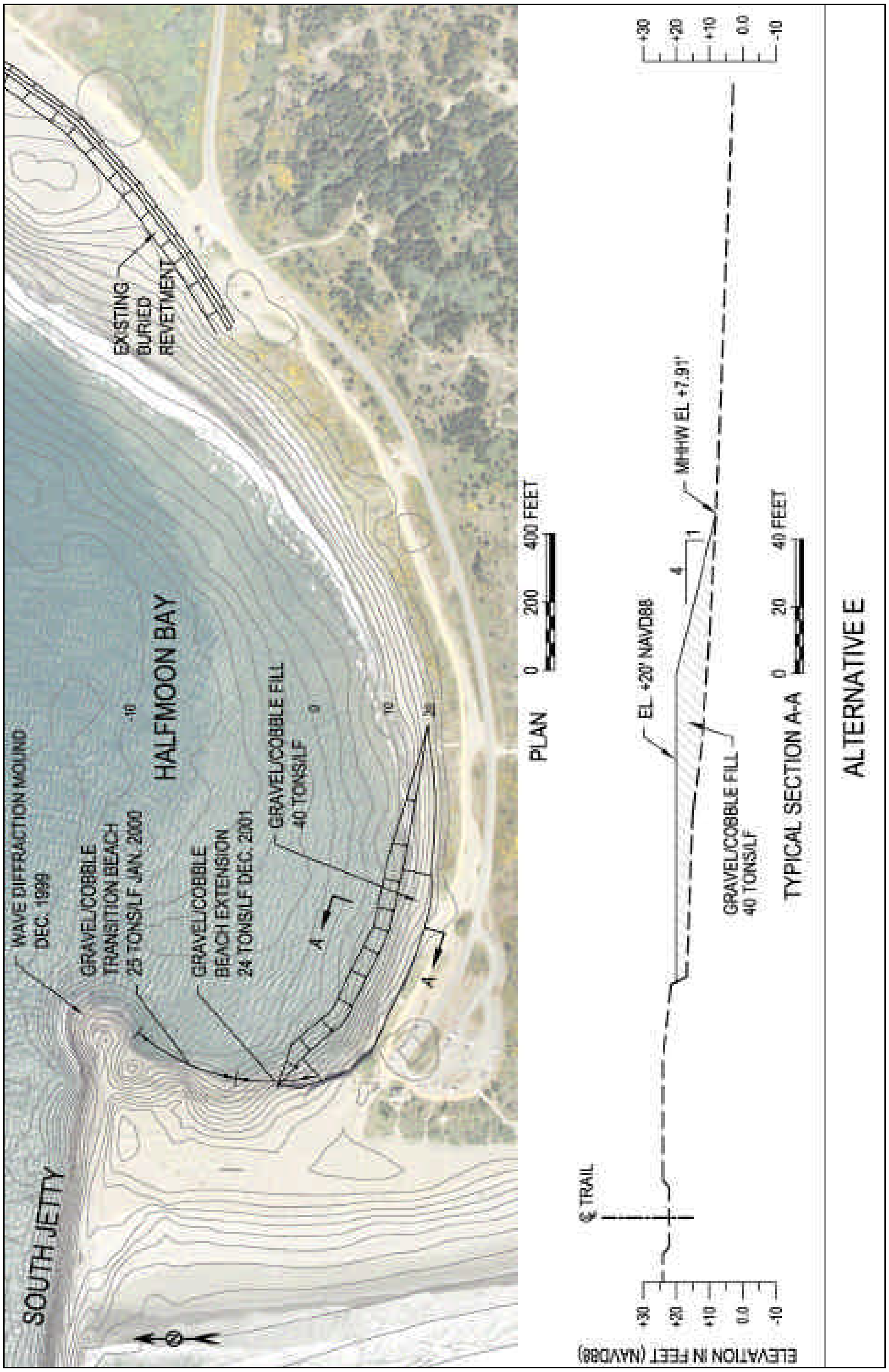

Figure 4-11. Alternative E Gravel-cobble beach

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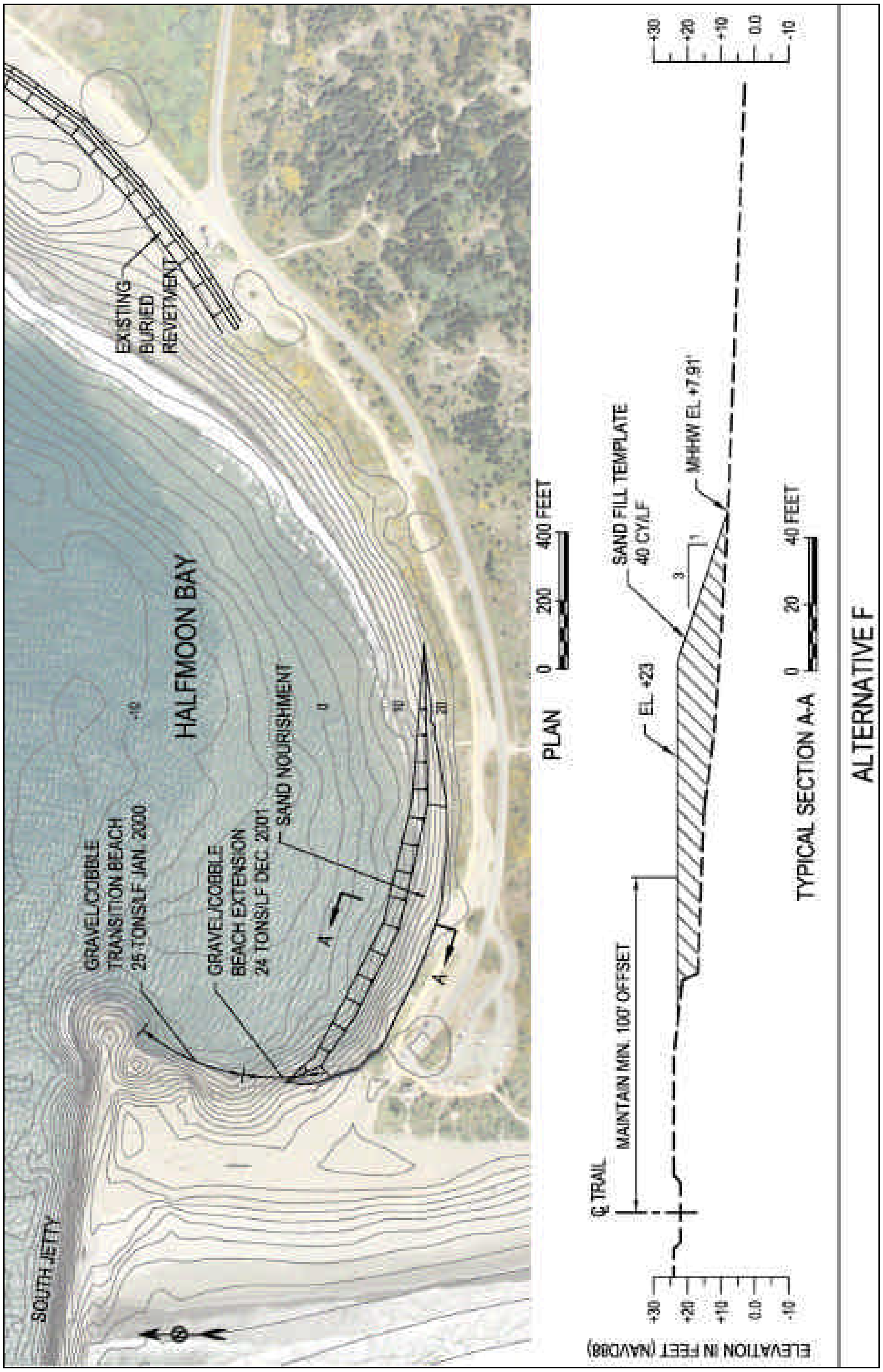

Figure 4-12. Alternative F Sand nourishment

Planshape Analysis Of Alternatives

Analysis of erosion and beach shapes at Half Moon Bay has been conducted using equilibrium beach planshape analysis. The purpose of the analysis is to provide a preliminary screening of the alternatives amenable to planshape analysis. This will assist with prioritization of alternatives for further analysis with physical modeling. The analysis involves fitting the existing beach planshape to the analytic planshapes predicted using the method of Hsu and Evans (1989), and interpreting the effects of the gravel transition fill and the various engineering alternatives on expected future equilibrium beach planshapes. In addition, numerical modeling and sediment mobility calculations can be done based on the existing planshape to predict wave transformations and sediment transport.

The expected beach planshape was computed for alternatives A1_500, A1_250, A2, A3, and B. The equilibrium beach planshape predictions were based on Hsu and Evans (1989). As noted here and in Chapter 2, this analysis does not consider the influence of mobility of the gravel fill on the planshape of the western end of the beach. Beach mobility and transport along the shore are addressed in more detail in Chapter 5 of this report. Alternatives C, D, E, and F are not amenable to planshape analysis.

Figures 4-13 through 4-17 show the predicted equilibrium beach planshapes for alternatives A1_500, A1_250, A2, A3, and B. Analysis of each of those alternatives results in an eastward shift in the beach planshape. This result suggests that raising the submerged portion of the South Jetty over a length of 250 to 500 ft, or adding a spur to the diffraction mound in conjunction with beach nourishment, would likely provide a significant benefit in terms of maintaining the transition beach and breach-fill between Half Moon Bay and the South Beach. These alternatives would also eliminate or assist in reducing the erosion in the southwest corner at the end of the transition beach. Increasing the footprint of the diffraction mound might also provide a similar benefit but would likely be much less effective than the jetty or spur alternatives.

Summary

Functional alternatives have been developed for the prevention of breach recurrence in the South Beach region. Prevention of a breach essentially requires keeping the South Beach/Half Moon Bay shorelines from encroaching. Since a breach can originate from either the South Beach or Half Moon Bay side, a proper solution to the breach threat will also include consideration of management of South Beach. The alternatives identified here are intended to provide a solution from the Half Moon Bay side of the area. Table 4-3 outlines each alternative developed, and provides a preliminary summary of the pros and cons.

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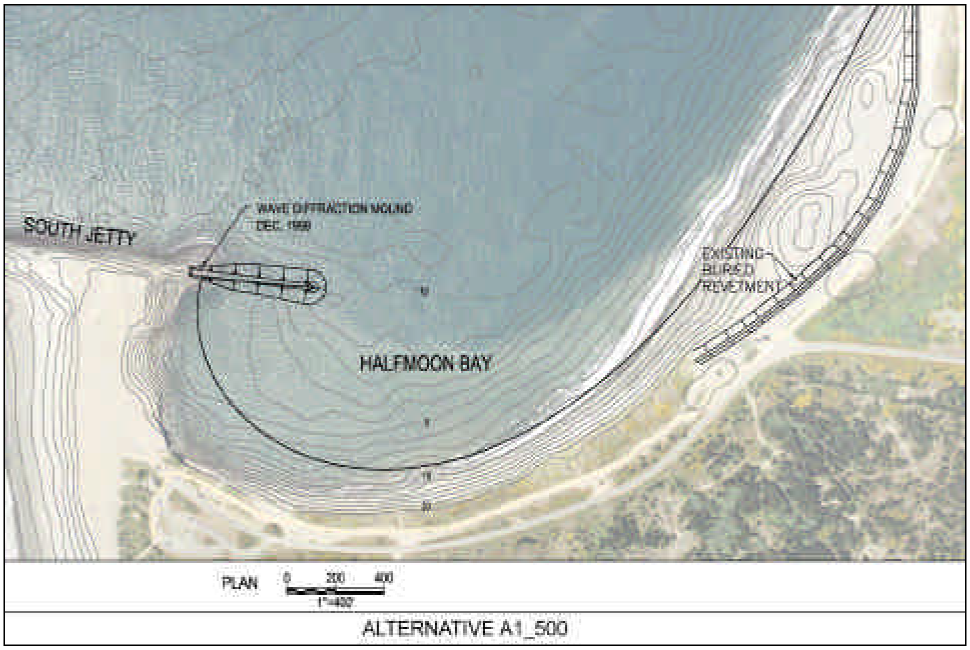

Figure 4-13. Equilibrium planshape for a control point consistent with Alt A1_500

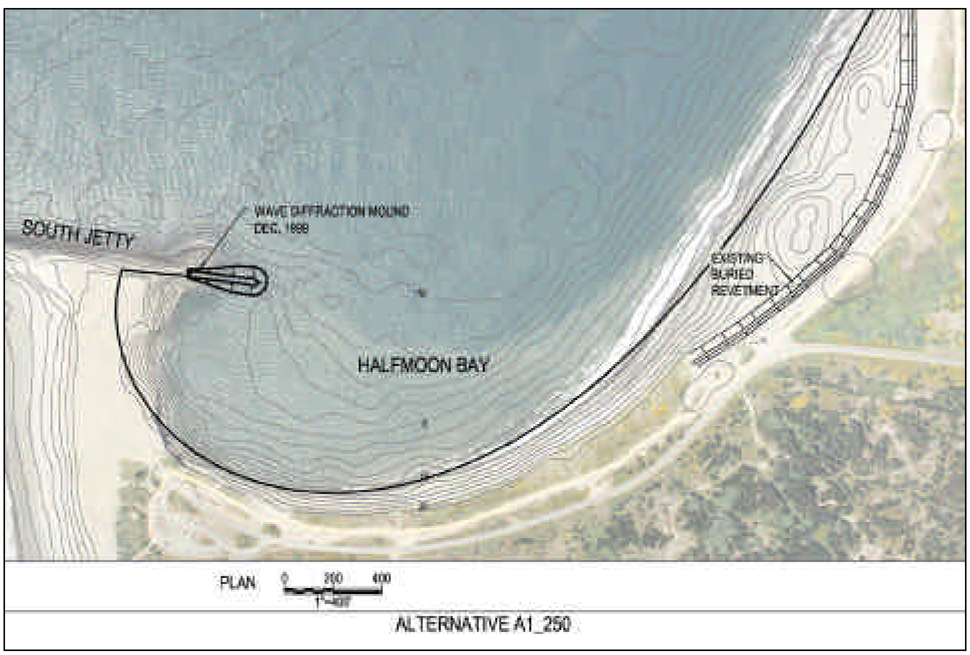

Figure 4-14. Equilibrium planshape for a control point consistent with Alt A1_250

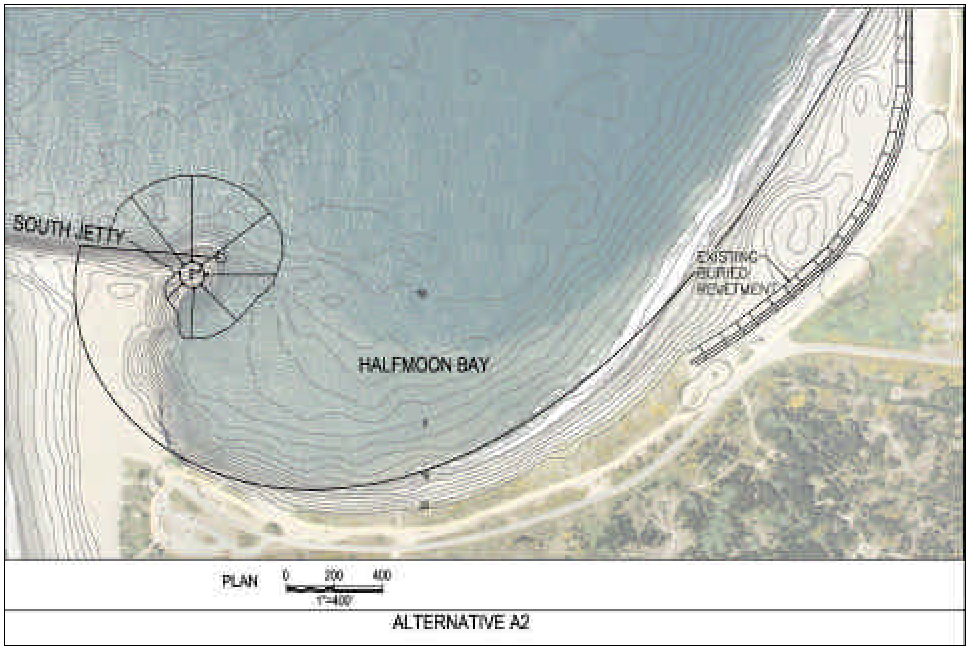

Figure 4-15. Equilibrium planshape for a control point consistent with Alt A2

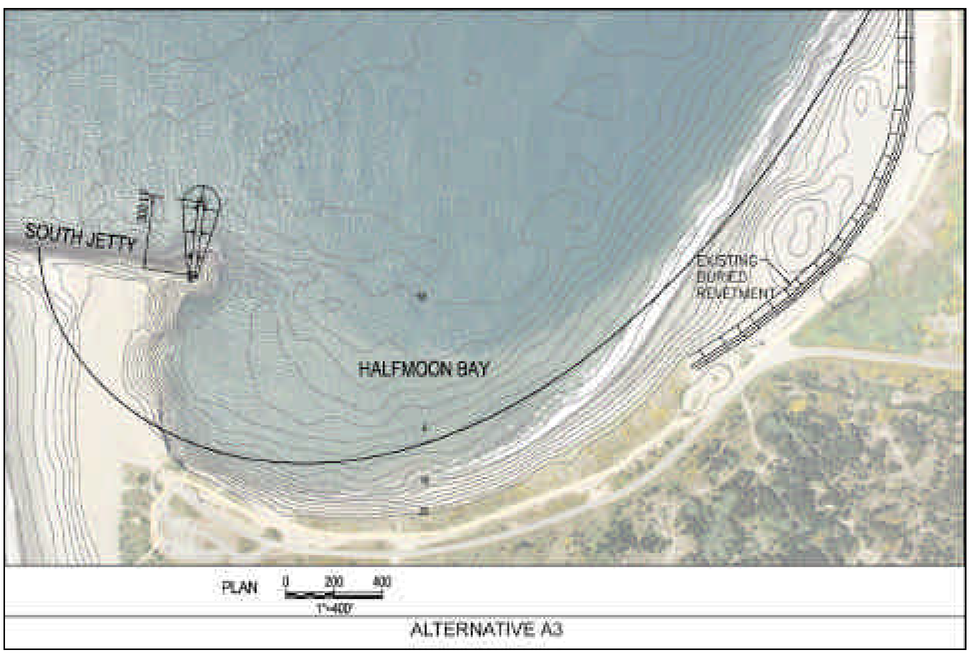

Figure 4-16. Equilibrium planshape for a control point consistent with Alt A3

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Figure 4-17. Equilibrium planshape for a control point consistent with Alt B

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Table 4-3
Alternatives for Half Moon Bay

Alternative	Confidence in providing breach protection	Downdrift effects	Environmental aspects	Construction issues	Construction costs ($)	Maintenance costs ($)	Analysis methods, issues, notes
A1_500. Raise submerged jetty to +20 feet, 500 feet length.	High, (High provided that beach nourishment also occurs).	Reduced sediment supply to Chehalis – possible downdrift erosion.	Approx. 250 ft extension to existing remnant footprint on sub-tidal area.	Minimal.	60,000 tons of stone. $4.1M	Minimal for structure; beach nourishment requirements may change.	No new impact to sub-tidal area. Planshape analysis indicates it will result in improved shoreline position. Needs physical & numerical modeling to determine potential reductions in waves and transport.
A1_250. Raise submerged jetty to +20 feet, 250 feet length.	Medium – High (Med-High provided that beach nourishment also occurs).	Similar to A1_500; possibly a smaller downdrift impact.	No new impact to sub-tidal area.	Same as A1_500	22,500 tons of stone. $1.6M	Minimal for structure; beach nourishment requirements may change.	Planshape analysis indicates it will result in improved shoreline position. Needs physical & numerical modeling to determine potential reductions in waves and transport.
A2. Diffraction mound modification – Increase mound size, flatter slope.	Medium – Low.	Reduced sediment supply to Chehalis – possible downdrift erosion.	Minor increase to footprint on sub-tidal area.	Stone stability needs to be addressed – increased exposure as size of mound increases – could affect cost.	52,000 tons of stone. $3.2M	Minimal for structure; beach nourishment requirements may change.	Past experience at this site with this technique has been less than fully successful. The benefit to shoreline position is probably less than Alt A1 Needs physical & numerical modeling to determine potential reductions in waves and transport.

Table 4-3
Alternatives for Half Moon Bay

Alternative	Confidence in providing breach protection	Downdrift effects	Environmental aspects	Construction issues	Construction costs ($)	Maintenance costs ($)	Analysis methods, issues, notes
A3. Diffraction mound modification – Add diffraction spur.	Medium.	Redirection of sediment supply to Chehalis, possible deflection of eastward transport along South Jetty to navigation channel.	Minor increase to footprint on sub-tidal area – increase habitat for predator species.	Same as above, higher exposure could lead to expensive round-head, possible hazard to navigation.	27,000 tons of stone. $1.9M	Minimal for structure; beach nourishment requirements may change.	A potentially efficient solution with minor impact to inter-tidal and sub-tidal areas. A potential hazard to navigation and vulnerable to scour at the toe. Needs physical & numerical modeling to determine potential reductions in waves and transport.
B. Point Chehalis control point.	Low.	Reduced sediment transport downdrift.	Moderate – footprint of riprap in nearshore, good updrift beach effects, possible adverse circulation effects.	None. Land-based construction.	20,400 tons of stone. $1.4M	Minimal for structure; beach nourishment requirements may change.	Minor effect on breach but potentially an important component of overall solution – ensuring stability of downdrift beach. Needs physical & numerical modeling to determine potential reductions in waves and transport.

Table 4-3
Alternatives for Half Moon Bay

Alternative	Confidence in providing breach protection	Downdrift effects	Environmental aspects	Construction issues	Construction costs ($)	Maintenance costs ($)	Analysis methods, issues, notes
C1. Segmented submerged breakwater.	Medium – depends on configuration.	Minimal. Possible reduced circulation – could be issue with respect to Westport outfall.	Footprint on seabed – potential habitat for predator species.	Fairly simple, requires marine construction.	(5 - each 200 ft long – total of 40,000 tons of stone). $2.8M	Minimal for structure; beach nourishment requirements may change.	High tide range at the site necessitates a berm that is above water most of the time. A berm crest elevation below mllw will likely not reduce wave energy sufficiently. Long period waves may result in a “pumping” action that increases erosion and scour at the gaps in the breakwater. Needs physical & numerical modeling to determine potential reductions in waves and transport.
C2. Nearshore berm.	Low.	Very good, depending on circulation could lead to increase in sand entering navigation channel.	Very good.	Need to place as close to shore as possible and preferably in spring/early summer to encourage onshore transport.	$.74M for 50,000cy.	Continuous maintenance is key to success.	A beneficial use of dredged material with potentially minimal impact to the environment. To be effective, the berm should be in relatively shallow water. Direct placement on the beach would probably cost the same and perform better. One-time placement would likely not be adequate as a long-term solution.

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Table 4-3
Alternatives for Half Moon Bay

Alternative	Confidence in providing breach protection	Downdrift effects	Environmental aspects	Construction issues	Construction costs ($)	Maintenance costs ($)	Analysis methods, issues, notes
D1. Geo-terraced revetment.	Medium – improved if placement widens buffer zone.	Reduced sediment supply to Chehalis – possible downdrift erosion.	If exposed, geotextile tubes might be an eyesore Minor impact to benthic invertebrates.	Issues regarding stability, scour, removal of fill, risk of vandalism.	$1.5M	Uncertain.	Relatively low-cost, and allows a stepped, sandy beach profile. Experience with similar structures indicates potential for success in this environment. Geo-tubes are vulnerable to vandalism. A relatively new application for geo-tubes with uncertain performance. Needs further analysis to determine potential reductions in cross-shore/longshore transport potential.
D2. Geo-tube perched beach.	Unknown – perched beach only useful when cross-shore transport is key design parameter.	Reduced sediment supply to Chehalis – possible downdrift erosion.	If exposed, geotextile tubes might be an eyesore. Minor impact to benthic invertebrates.	Issues regarding stability, scour, removal of fill, risk of vandalism.	$1.9M	Uncertain.	Relatively low-cost, and allows a sandy beach profile. Geo-tubes are vulnerable to vandalism. Design guidance for perched beaches is very limited. Perfomance is uncertain. Needs further analysis to determine potential reductions in cross-shore/longshore transport potential.
E. Gravel/cobble beach.	Medium – dependent on maintenance, there can be longshore transport problems if wave conditions are too severe.	Reduced sediment supply to Chehalis – possible downdrift erosion.	Gravel moving downshore not suitable for recreational beach use. Minor temporary impact to benthic invertebrates.	Longshore transport of gravel will diminish effectiveness.	$2.1M	Potential periodic nourishment.	Relatively low-cost. Natural rounded beach sediment preferred environmentally over a hard structure. Needs analysis to determine potential reductions in cross-shore/longshore transport potential.

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Table 4-3
Alternatives for Half Moon Bay

Alternative	Confidence in providing breach protection	Downdrift effects	Environmental aspects	Construction issues	Construction costs ($)	Maintenance costs ($)	Analysis methods, issues, notes
F. Sand nourishment.	Low.	Maintains sand supply to Point Chehalis.	None.	None.	$0.54M for 60,000cy.	Ongoing maintenance is critical to success.	A natural beach profile with native sediment is desirable and has the least environmental impact. Coarsening of shoreline sediment is undesirable. Performance is uncertain. Frequent maintenance and beach re-nourishment would be required unless controls structures (sills, groins, breakwaters) are included.

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5 Modeling and Analysis of Nearshore Waves and Sediment Transport Potential

A local-scale numerical wave transformation model was established for Half Moon Bay Grays Harbor, WA, and then applied in the present study. This chapter describes the Coastal Gravity WAVE (CGWAVE) (Demirbilek, Xu and Panchang, 1996; Demirbilek and Panchang, 1998; Panchang, Xu and Demirbilek 1999; Panchang and Demirbilek, 2001) model and its implementation at Half Moon Bay, followed by discussion and interpretation of model calculations. The wave model CGWAVE provides input for calculation of waves and sediment transport potentials in the Half Moon Bay nearshore. The existing condition and six alternatives (Alt A1_250, Alt A1_500, Alt A2, Alt A2_98, Alt C1, and Alt C2) as described in Chapter 4 in terms of the changes to waves and longshore sediment transport potential. Other alternatives outlined in Chapter 4 are not readily amenable to analysis with the CGWAVE model. Topics contained in this Chapter include previous wave model studies of Grays Harbor entrance and Half Moon Bay, model description, grid development, linkage of the local CGWAVE model with the regional STWAVE model, model verification, modeling the existing condition, sediment transport calculations, and application of the model to evaluate alternatives.

Wave Modeling at Half Moon Bay, Grays Harbor, WA

STWAVE - Regional Wave Transformation Modeling

The STeady-state spectral WAVE model (STWAVE) (Resio 1987; Smith, Sherlock, and Resio 2001) was operated to transform waves at and around the Grays Harbor entrance as part of the North Jetty Performance and Entrance Navigation Channel Maintenance study (Cialone, Davies, and Osborne, 2003). The STWAVE model assumes spatially homogeneous offshore wave conditions and accounts for linear refraction and shoaling processes. The model was linked with the ADvanced CIRCulation (ADCIRC) (Luettich, Westerink, and Scheffner, 1992) tidal circulation model within the Surface-water Modeling System (SMS)

(Zundel, Cialone and Moreland, 2002) to account for the effects of tidal currents on wave transformations.

The offshore boundary of the STWAVE grid is at the Grays Harbor Coastal Data Information Program (CDIP) buoy at a depth of 41 m. The model was applied to transform waves from offshore to the harbor entrance. The STWAVE grid resolution was 50 m with 341 cells across shore and 588 cells alongshore. The model was extensively validated against field measurements of directional waves inside and outside the harbor entrance and showed good correlation with the measurements (Cialone et al. 2003).

The purpose of the STWAVE modeling was to determine wave conditions in the inlet entrance and in the vicinity of the Half Moon Bay nearshore, to provide radiation stresses for the prediction of wave induced currents for circulation modeling and to provide boundary conditions for physical modeling (ERDC/CHL TR-04-XX) and local scale wave numerical modeling (this Chapter).

Relatively few wave transformation models have the capability to simulate the relatively strong diffraction and refraction that occurs as waves propagate past the eastern terminus of the South Jetty. Therefore, a more specialized wave model is required to simulate nearshore waves in Half Moon Bay at Grays Harbor.

The CGWAVE Model

CGWAVE is a state-of-the-art wave prediction model applicable to estimation of wave fields in harbors, open coastal regions, coastal inlets, around islands, and around fixed or floating structures. CGWAVE is a finite-element model that operates in the Surface-water Modeling System (SMS) interface for graphic input, output, and efficient implementation (Demirbilek and Panchang, 1998).

The model is a phase-resolving, finite element, coastal wave model based on the two-dimensional elliptic mild slope equation. CGWAVE simulates combined refraction-diffraction-reflection-dissipation (breaking and friction) caused by structures and bathymetry of arbitrary shape. The user is required to specify boundary reflectivities and input wave conditions on a semi-circular outer boundary. Output from the model includes wave heights, phases, directions, velocities, and pressures at selected locations or over the entire grid (Demirbilek and Panchang, 1998)

.

Previous Application of CGWAVE at Grays Harbor, WA

Physical model tests of the wave diffraction mound and modifications to the South Jetty were conducted by CHL in November 1998 at the request of the Seattle District. The Seattle District took advantage of an existing base jetty configuration and capabilities of the Coastal Inlets Research Program (CIRP) physical model facility to investigate the proposed modifications at the Grays Harbor South Jetty (Seabergh, 1999). Questions arose regarding the validity of the physical model results because the configuration tested did not include the jetty remnant extending east into Half Moon Bay from the diffraction mound.

The CGWAVE model was applied to Half Moon Bay (Osborne et al. 2003) to evaluate the performance of the wave diffraction mound with the jetty remnant in place and compare the results to the design modeled in the laboratory.

Simulation with the remnant structure showed that the waves approach the shoreline more perpendicular than without the remnant at a +8 ft mllw water level. The remnant has little or no effect on wave direction for a +12 ft mllw simulation. Overall, the structure remnant has little or no effect on wave height along the Half Moon Bay shoreline for either the +8 ft mllw water level or +12 ft mllw water level simulations. The model calculations indicate that the remnant structure has not adversely affected the functioning of the wave diffraction mound. Only a limited set of wave and water level conditions were simulated in the previous work and the model grid was based on the somewhat idealized base jetty and basin configuration of the CIRP physical model facility.

Bathymetry grid

The wave model requires a computational bathymetric grid over which to transform waves from the outer boundary to the nearshore. The CGWAVE grid was generated with the SMS grid generator using available digital bathymetry.

Figure 5-1 shows the CGWAVE model domain and bathymetry contours. The domain extends offshore from the Half Moon Bay shoreline to the Entrance and Point Chehalis reaches of the navigation channel. The model domain did not extend beyond the southeast margin of the navigation channel to avoid the need to include wave-current interaction in the model. Wave-current interactions in the inlet entrance are accounted for in the regional STWAVE model. A review of field measurements (Chapter 2) indicates that tidal currents in Half Moon Bay are relatively weak in comparison with wave-induced currents. The model domain was also designed to coincide approximately with the Half Moon Bay physical model domain. Bathymetry for the CGWAVE model was based on the same 2002 and 2003 surveys acquired by Seattle District, that were used in the development of the Half Moon Bay physical model (ERDC/CHL TR-04-XX). Particular attention was paid to developing the grid for the numerical model in the nearshore and inter-tidal zones of Half Moon Bay, and around the wave diffraction mound and jetty remnant area at the eastern terminus of South Jetty. Figure 5-2 shows profiles along a bathymetry survey line that transects the submerged jetty remnant in the north-south direction. The wave diffraction mound and jetty remnant were not well represented in survey data. Existing survey data were supplemented with data derived from aerial photographs and crest elevation and side slope information from historical documentation. Figure 5-3 shows an oblique view of the jetty remnant contours reconstructed from survey data and aerial photography. Nearshore and inter-tidal bathymetry were supplemented with additional topographic survey information obtained from RTK-GPS surveys and shoreline positions interpreted from aerial photographs. A rectangular bathymetry grid with 25 ft horizontal grid spacing provided the basis for developing the computational grids.

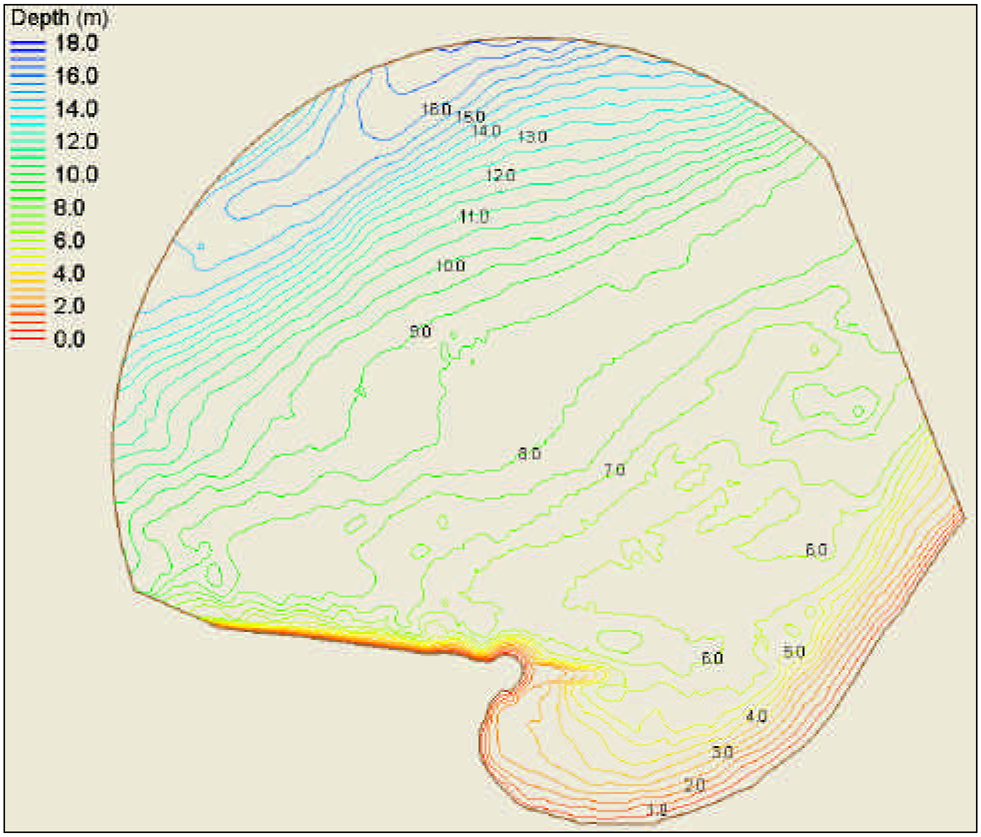


Figure 5-1. CGWAVE model domain and bathymetry contours

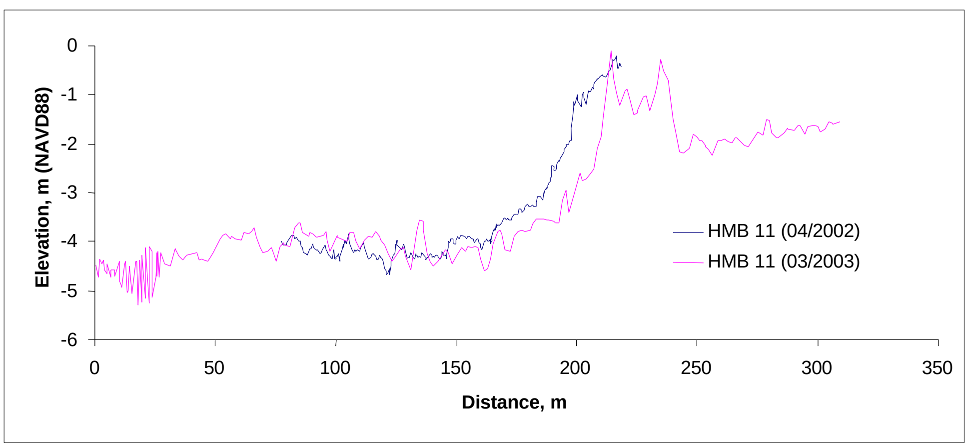


Figure 5-2. Bathymetry survey crossing the submerged jetty remnant in the north-south direction

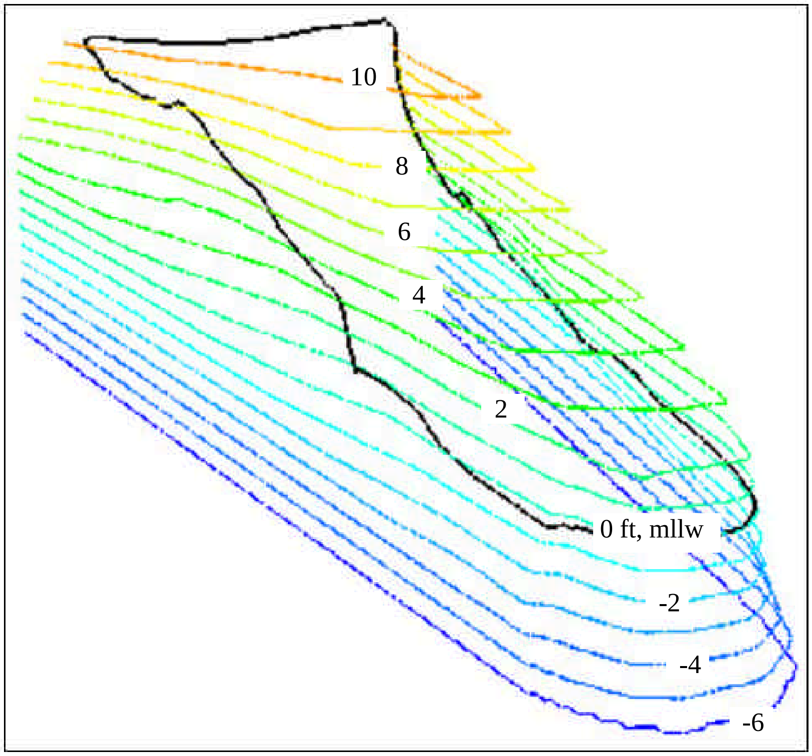


Figure 5-3. Contours of the jetty remnant (black line indicates the intersection with the bathymetric grid)

Finite Element (FE) mesh

An unstructured FE mesh was generated in SMS based on the 25 ft bathymetry grid and the selected model domain. The sizes of the mesh elements in a CGWAVE grid vary solely with local water depth because they are wavelength dependent. Two FE meshes were developed: one for mean high water, mhw (mllw +2.48 m) and one for mean low water, mlw (mllw +0.13 m). Element resolution for each mesh was set to 12 nodes per wavelength based on a wave period of 12 sec and resulted in approximately 26,000 computational nodes. Figure 5-4 shows the detail of the FE mesh near the wave diffraction mound. The largest element size for the domain was approximately 7.8 m and the smallest element size was approximately 1.35 m. The minimum depth in the model was set to 0.5 m.

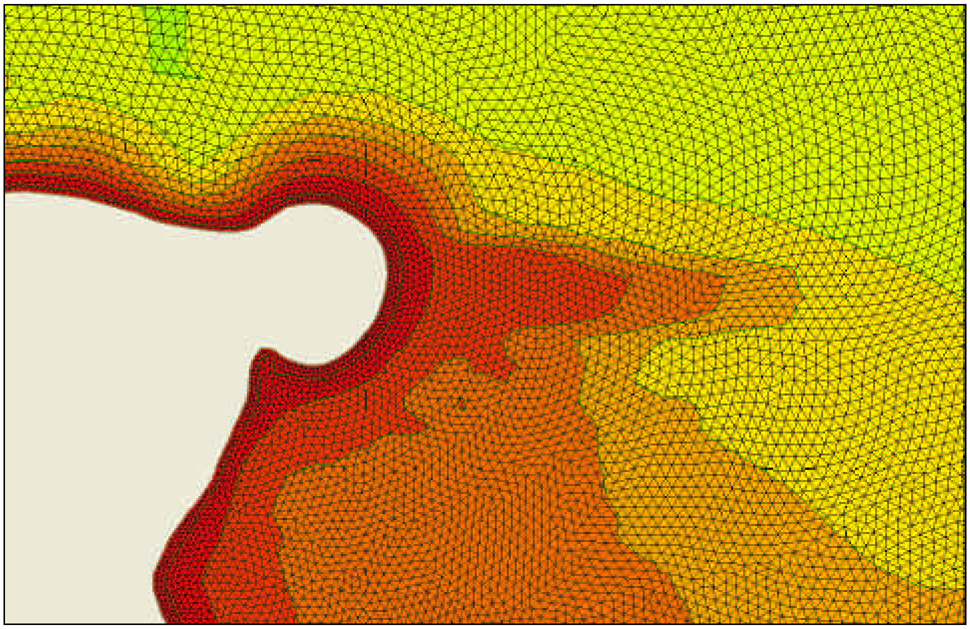


Figure 5-4. FE mesh in the vicinity of the wave diffraction mound and jetty remnant

Specification of input waves at the outer boundary

Wave height, period, and direction need to be specified along the offshore open boundary of the CGWAVE model domain. In this study, simulations involved only monochromatic waves. This is clearly an approximation, but local spectra at the outer boundary of the model domain were shown to be relatively unidirectional and narrow-banded (Chapters 2 and 3).

In a typical application of CGWAVE, the user is required to assume that the wave condition beyond the modeling domain is unilaterally one-dimensional (invariant bathymetry alongshore). In monochromatic mode, a single representative wave height, period, and direction are specified at some point offshore and then transformed onto the semi-circular open boundary by a one-dimensional wave model embedded in the CGWAVE model. This assumption is not appropriate for Half Moon Bay owing to the presence of the South Jetty on the southern boundary of the model domain and somewhat irregular bathymetry in the inlet entrance, and the strong tidal currents that waves encounter as they propagate into the inlet entrance.

Modifications to the CGWAVE source code were made to account for variable wave heights along the outer boundary of the CGWAVE model domain. Lateral variations in wave height, period, and direction along the outer boundary were determined from output from the regional STWAVE model that accounts for refraction, shoaling, and wave current interaction. Wave heights and directions were obtained from STWAVE output at several points that defined a parallelogram in the vicinity of the outer boundary of the CGWAVE domain. A lateral weighting function (Figure 5-5) was applied to the variation in wave height around the perimeter of the parallelogram with unity applied to the wave height closest to the apex of the semicircular boundary.

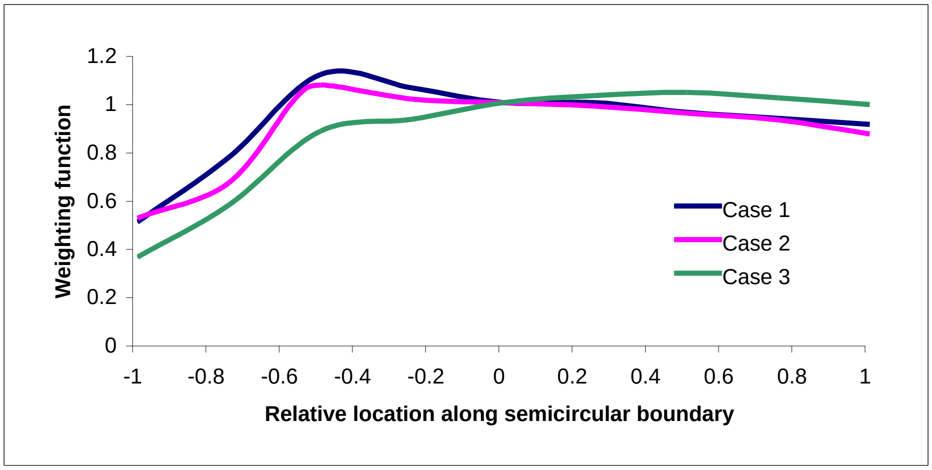


Figure 5-5. Weighting curves obtained from STWAVE output

Specification of boundary reflection

A fully-absorbing outgoing wave boundary was defined along the northern portion of the model domain from the semicircular outer boundary to the intersection with the Half Moon Bay shoreline near Point Chehalis. The boundary condition was defined following the method defined by Chen et al (2002). The boundary condition is dependent on wave direction and bottom bathymetry. The method prevents spurious boundary reflections that may arise with more conventional boundary conditions.

Partial reflection (reflection coefficient = 0.2) was specified for the boundary along the South Jetty and diffraction mound. Wave reflection at incident wave frequencies can be neglected on the Half Moon Bay shoreline as the majority of incident wave energy is dissipated by breaking in the surf zone and turbulence in the swash zone.

CGWAVE Verification

Two months of directional wave measurements were collected between December 2003 and February 2004 at four stations in the Half Moon Bay area as described in Chapter 2. The location of the measurement stations is shown in Figure 2-8.

Time series of wave height, period, and direction measured at the four stations and the offshore CDIP buoy for a three-day interval between 25 December 2003 and 27 December 2003 are shown in Figure 5-6. A winter storm occurred during the interval in which offshore significant wave height exceeded 6 m. As discussed in Chapter 2, average winter H_s at this location is between 2 to 3 m. Three wave height, period, direction, and water level combinations representing typical wave and tidal level combinations were chosen based on this segment of the measurements for the purpose of model verification (Table 5-1).

Table 5-1
Summary of CGWAVE verification test cases

Case	Description		H, m	T, sec	Dir, deg-T	Level (m) mllw
1	high waves high tide	CDIP Buoy	4.5	15.5	282	2.48
		CGWAVE	2.6	16	295	
2	low waves high tide	CDIP Buoy	2.3	13	289	2.48
		CGWAVE	1.3	13	290	
3	high waves low tide	CDIP Buoy	5.3	15.5	278	0.13
		CGWAVE	3.8	16	286	

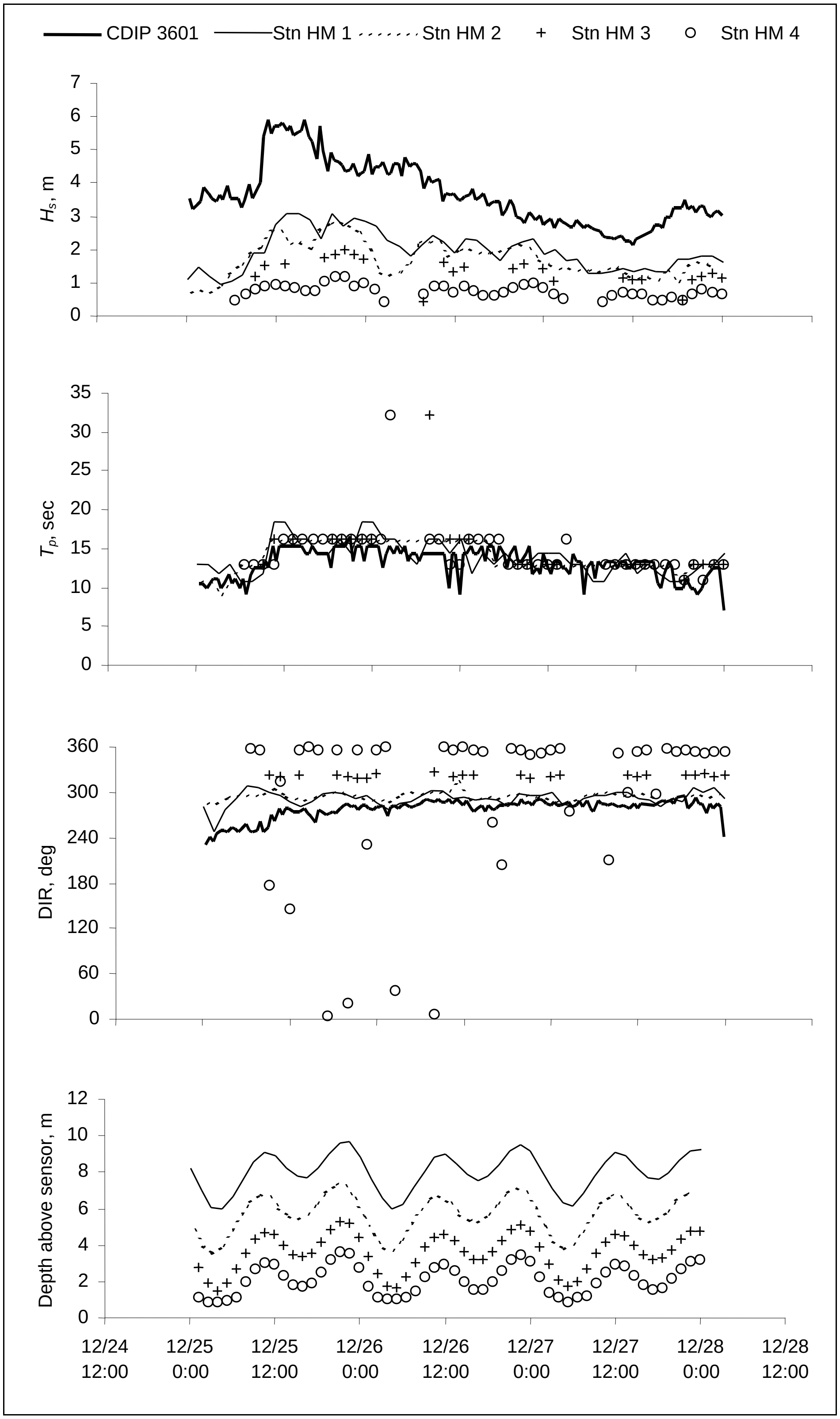


Figure 5-6. Time series of H_s, T_p, DIR, and Depth above sensor (24 December 2003 and 27 December 2004)

Figures 5-7 though 5-9 show maps of significant wave height and phase as computed by the model for all three verification cases. In these simulations CGWAVE has modeled the prescribed significant wave height, H_s, as a monochromatic wave of equal height. The model results produced wave height and phase patterns accounting for the effects of wave refraction, diffraction, and reflection. The phase plots are indicative of the wave crest and trough direction and spacing. The bending of wave crests as waves propagate past the diffraction mound and into Half Moon Bay is evident. Wave height is significantly reduced in the diffraction zone for all conditions. The model reproduces the reduction of wave heights in the diffraction zone and illustrates the variation along the shoreline of Half Moon Bay. The phase map for Case 3 (low tide) indicates that waves approach the shoreline more perpendicular than they do in the high tide cases, a result that is consistent with previous application of the CGWAVE model. Outgoing waves approach the model boundaries at realistic angles with no evidence of distortion caused by the boundary condition.

Table 5-2 provides a comparison of the modeled wave heights with the measurements as well as with STWAVE predictions at the four measurement stations. Stn HM1 provided calibration for incident waves, while Stn HM2 to 4 provided verification data. There is excellent agreement between CGWAVE model results and measurements for all three cases at all stations. The relative errors in wave height are within 10 percent for Case 1 and 2, and the wave direction is within 10 degrees. The wave height errors for Case 3 are somewhat larger (15 percent) but the wave angles agree within 10 degrees.

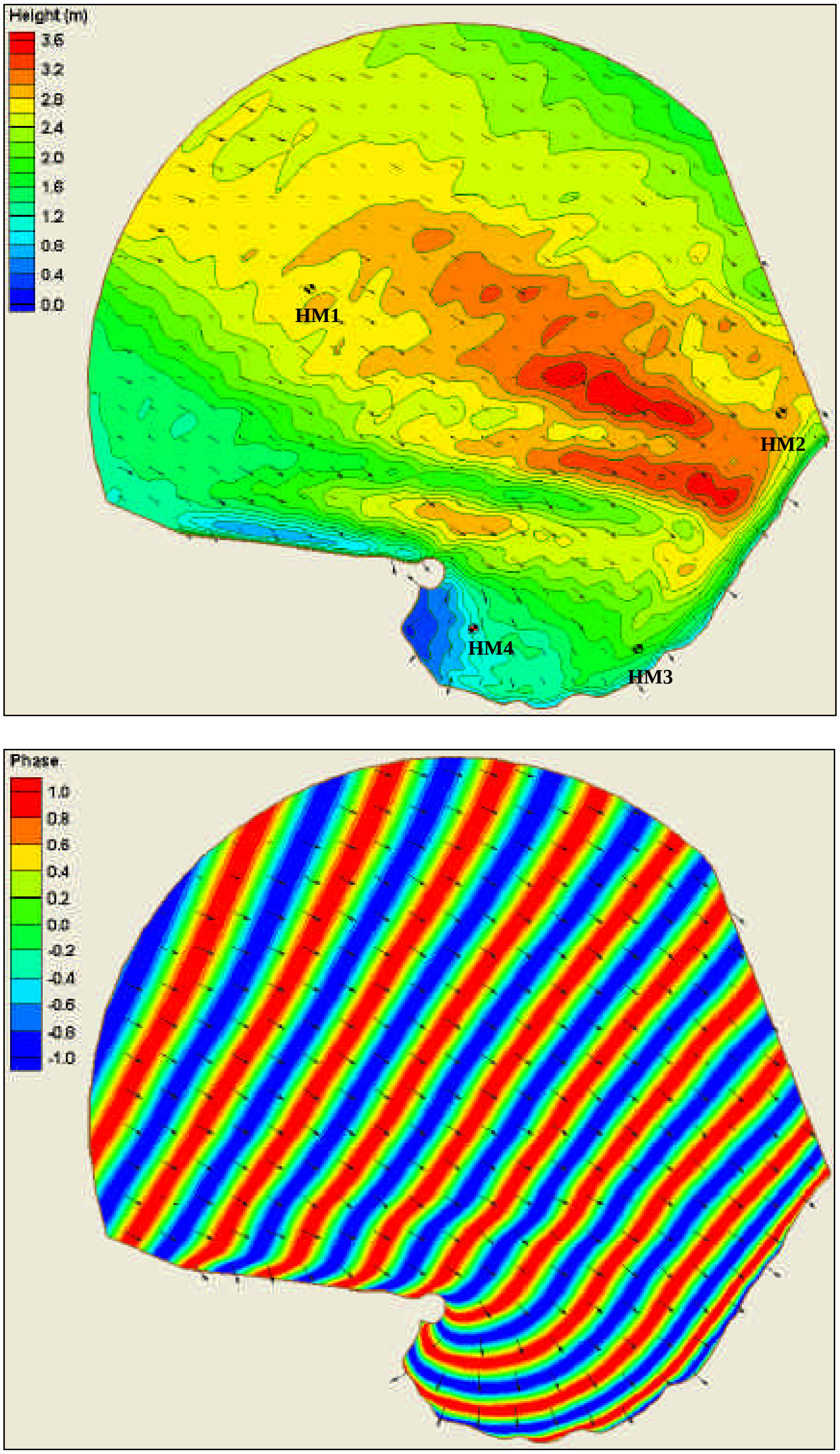


Figure 5-7. Maps of wave height (top) and phase (bottom) for test case 1

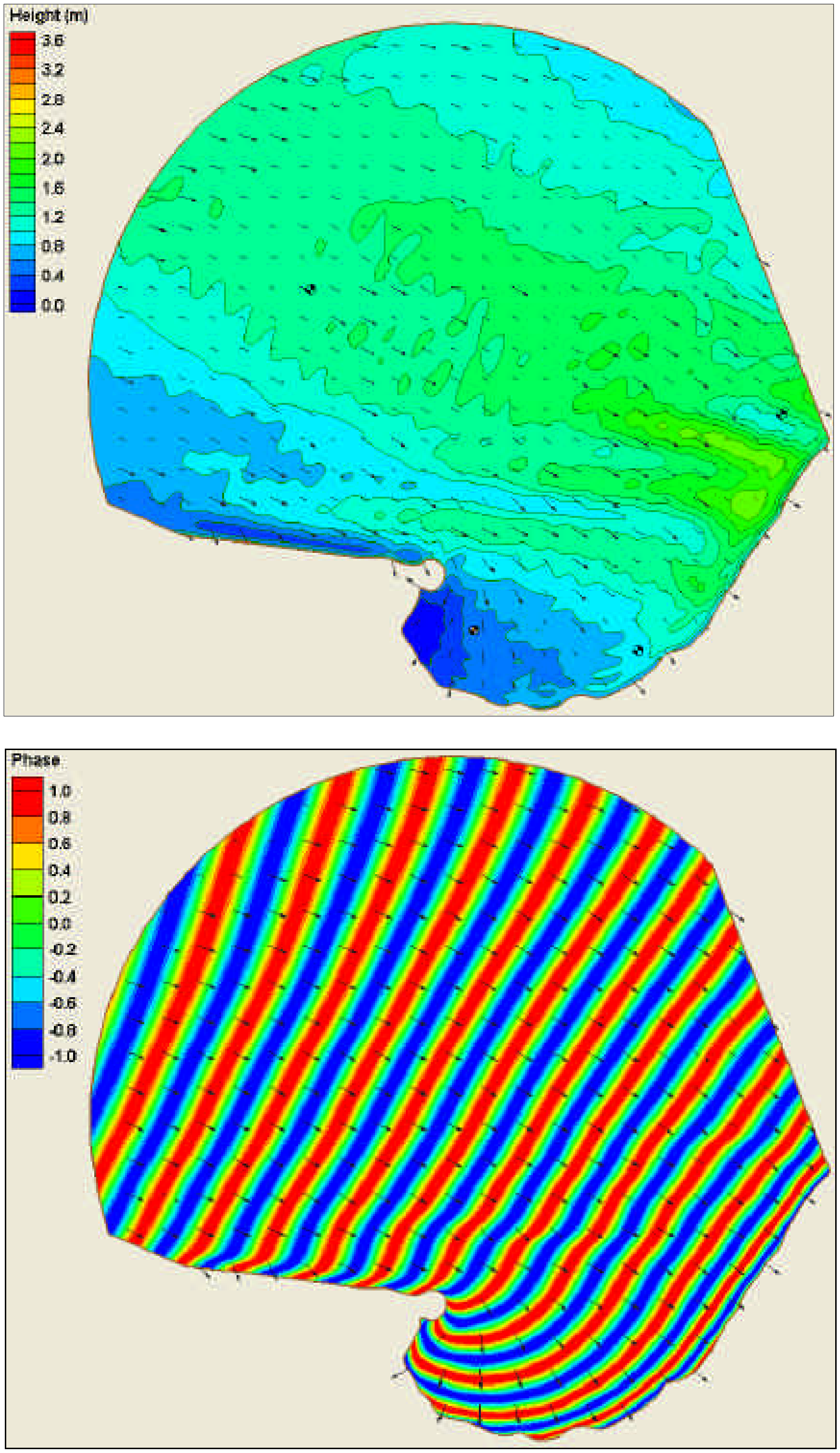


Figure 5-8. Maps of wave height and phase for test case 2

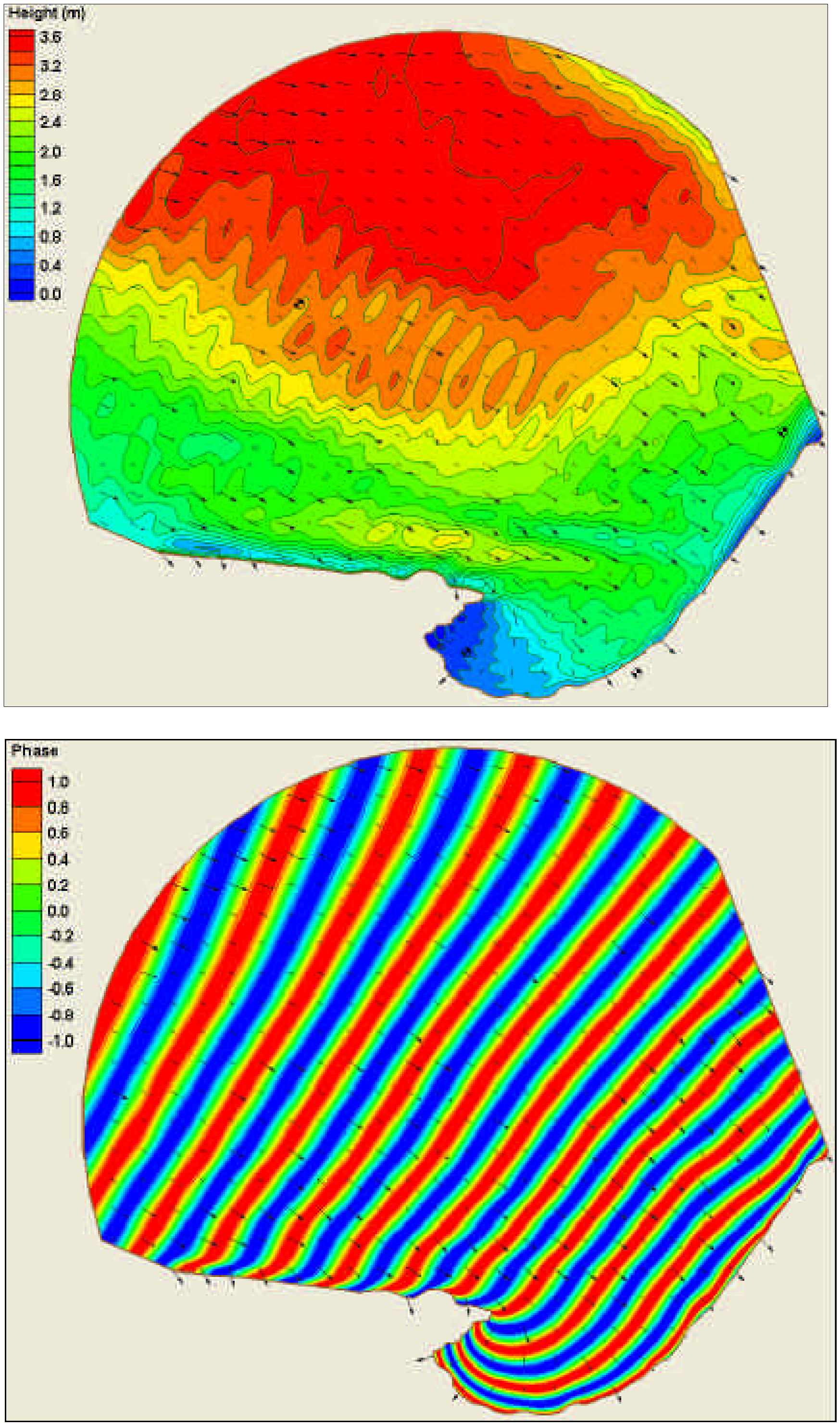


Figure 5-9. Maps of wave height (top) and phase (bottom) for test case 3

Table 5-2
Summary of CGWAVE verification results

			Stn HM1		Stn HM2		Stn HM3		Stn HM4	
Case	**Description**		**H, m**	**DIR**	**H, m**	**DIR**	**H, m**	**DIR**	**H, m**	**DIR**
1	high waves high tide	Measured	2.63	289	2.83	296	1.92	316	1.10	355
		CGWAVE	2.69	303	2.85	312	1.96	323	1.08	6
		STWAVE	3.04	334	2.91	322	1.08	303	0.44	296
2	low waves high tide	Measured	1.37	288	1.27	299	1.02	320	0.62	353
		CGWAVE	1.38	298	1.31	309	0.92	320	0.54	358
		STWAVE	1.77	335	1.63	323	1.08	303	0.25	296
3	high waves low tide	Measured	3.05	286	2.18	289	N/A	N/A	0.71	358
		CGWAVE	3.00	295	1.89	305	N/A	N/A	0.33	36
		STWAVE	2.37	336	2.02	319	N/A	N/A	0.01	305

Model Application

The verified CGWAVE model was applied to evaluate the existing condition and the following alternatives:

- Raise submerged jetty (Alt A1_250 and Alt A1_500)
- Modify diffraction mound (Alt A2)
- Original diffraction mound concept (Alt A2_98)
- Segmented submerged breakwater (Alt C1)
- Submerged nearshore berm (Alt C2)

Comparative analysis was performed on a selection of representative wave conditions. It included comparison of spatial patterns in wave height and phase and variation of wave and longshore transport parameters with distance along the breaker line around the Half Moon Bay shoreline.

Selection of Waves

An extensive review and analysis of the wave climate at Grays Harbor entrance is provided in Chapter 3. The review identifies typical and extreme storm conditions for waves approaching Half Moon Bay. A set of discrete storm conditions in the entrance area is required as input to numerical and physical models to simulate the nearshore waves, currents, and sediment transport that cause erosion at the Half-Moon Bay shoreline. Table 5-3 summarizes the representative set of wave and water level conditions at the outer boundary of the CGWAVE domain.

Table 5-3 Summary of waves parameters for CGWAVE modeling of nearshore waves in Half Moon Bay				
		Entrance Waves (Model Boundary)		
Case	Description	H, m	Tp, sec	Dir, deg
1	4m, 16 sec waves offshore	2.32	16	295
2	2m, 16 sec waves offshore	1.10	16	295
3	6m, 16 sec waves offshore	3.48	16	295
4	4m, 12 sec waves offshore	2.32	12	295
5	4m, 16 sec waves offshore	2.32	16	277

Case 1 was identified as a typical storm wave for Grays Harbor, an offshore significant wave height of 4 m, local wave direction 295 deg, and T_p of 16 sec. Cases 2 through 4 provided variants either by wave height or by period. Case 5 was performed for a local wave direction of 277 degrees to assess the effects of varying wave angle. All cases were run with a high tide water level.

Cases 1 through 5 were simulated for the existing condition. A preliminary comparative analysis of the alternatives and existing condition was conducted with simulations of Case 1.

The lateral variations of wave height for the selected wave conditions were not re-computed by STWAVE; instead they were selected, based on the wave direction and period determined from three weighting functions derived for the verification cases. The lateral variation is predominately a consequence of wave refraction by the bottom topography when spectral waves propagate toward the nearshore, which for a given bathymetry, is mostly affected by the offshore wave direction and secondly the peak period, rather than by the wave height.

Wave height, wave direction and longshore flux at the break point

Wave height and wave angle relative to the shoreline at the breakpoint, H_b and α_b respectively, are two parameters that are considered fundamental to nearshore morphodynamics and sediment transport processes. When waves approach the shoreline at an angle, a portion of the waves onshore-directed momentum flux is directed alongshore. Wave breaking results in a cross-shore gradient in the momentum flux. The gradient in the momentum flux constitutes a force or thrust on the water column, which is a major factor in the generation of longshore currents (e.g. Longuet-Higgins, 1970). Sediment transport resulting in shoreline and beach morphological change may be driven by longshore currents and swash processes that occur at an oblique angle to shore normal.

Komar and Inman (1970) for example, suggest an empirical formula to estimate the magnitude of longshore current that is based solely on H_b and α_b:

$$V_l = 0.585\sqrt{gH_b}\,\sin 2a_b$$

Longshore transport rate is also commonly estimated following the energy flux method (e.g. USACE, 1998). The longshore energy flux potential, P_l, a measure proportional to the longshore mass transport, is calculated as follows:

$$P_l = \frac{\rho g}{16} H_b^2 C_{gb} \sin 2\alpha_b$$

Wave breaking in the CGWAVE model is calculated with the Battjes (1973) formula. Identifying the break-point in CGWAVE output is possible by locating the point at which a significant reduction of wave height occurs. Plots of wave height as a function of distance offshore are useful in this process. The series of breakpoints with distance alongshore defines the position of the breaker line. The simple breaking criterion

$$H_b = 0.76h$$

where h is the water depth, was found to provide an adequate approximation of the position of the breaker line on the Half Moon Bay shoreline.

Figure 5-10 shows an example of a breaker line position superimposed on the Half Moon Bay bathymetry. Distances along the breaker line from the wave diffraction mound are also indicated in Figure 5-10.

Once the location of breaking is known, H_b can be retrieved directly from CGWAVE output, while α_b is calculated as the angle between local wave direction and the direction of the local seabed slope. Longshore current speeds and potential energy flux may then be estimated with the equations as described above.

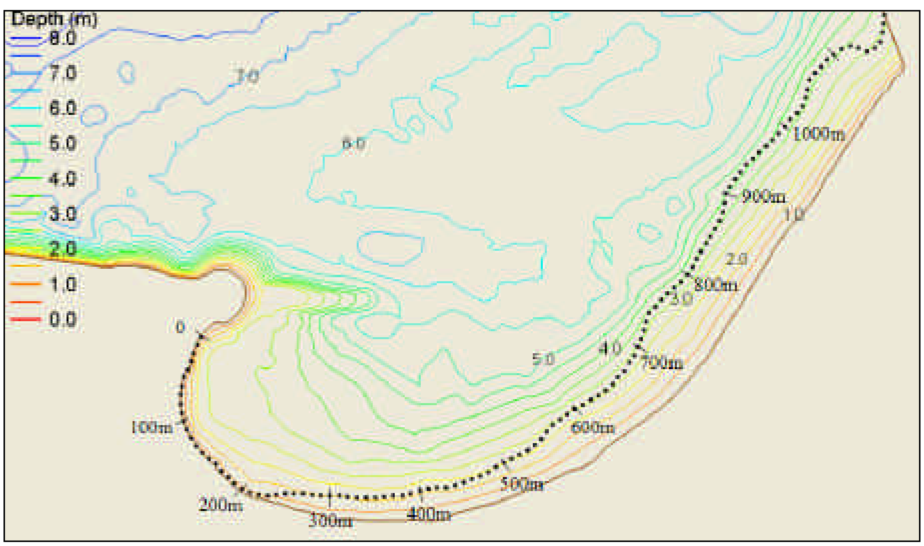


Figure 5-10. Location of the breaker line for the existing condition

Existing Condition

Figure 5-11 shows maps of wave height, phase, and direction for Cases 1 and 5. The maps illustrate the effects of variation of wave angle at the outer boundary of the model domain between 277 deg and 295 deg. The analysis was performed to determine the most appropriate angle for positioning the wave maker in the physical model (ERDC/CHL TR-04-XX) and also to provide an estimate of the sensitivity of physical and numerical model results to incident wave angle. Figure 5-12 shows the variation in H_b, α_b, V_l, and P_l with distance along the breaker line for the two cases shown in Figure 5-11

The wave height maps and the plots of H_b along the breaker line indicate a region of reduced wave height in the southwest corner of the bay (in the lee of the diffraction mound). Progressively higher waves occur with distance away from the mound around the Half Moon Bay shoreline and reach a maximum in the nearshore area south of Stn HM2 and the USCG Front Range tower (approximately 800 to 1000 m from the mound). The wave phase maps and variation in α_b with distance along the breaker line indicate that waves approach the shoreline at a steep angle in the lee of the diffraction mound. Wave approach is generally more shore perpendicular along the Point Chehalis shoreline. The high wave angles in the southwest corner of Half Moon Bay have the potential to create a strong longshore current and large longshore flux potential. However, as indicated in Figure 5-12, the longshore flux potential is reduced in the immediate vicinity of the mound owing to the very small waves heights in that region. Flux potential and current speeds increase significantly with distance in the southwest corner of the bay as wave height increases.

The results in Figures 5-11 and 5-12 indicate sensitivity to the angle of wave approach in the entrance to Grays Harbor. A shift in the incident angle from 277 deg to 295 deg results in a significant increase in the wave heights reaching the diffraction zone in the lee of the diffraction mound and therefore, greater V_l and P_l in the southwest corner of the bay.

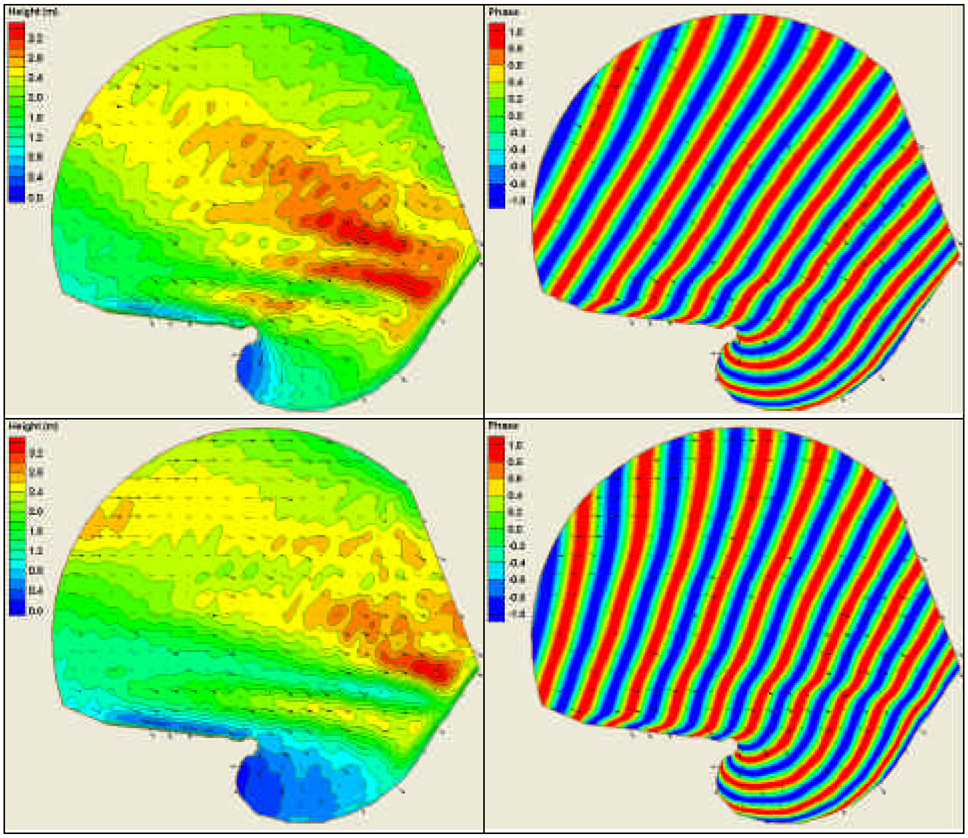


Figure 5-11. Maps of wave height and phase for offshore H_s = 4 m, T = 16 sec for Local DIR = 295 deg (top) and Local DIR = 277 deg (bottom) for the existing condition

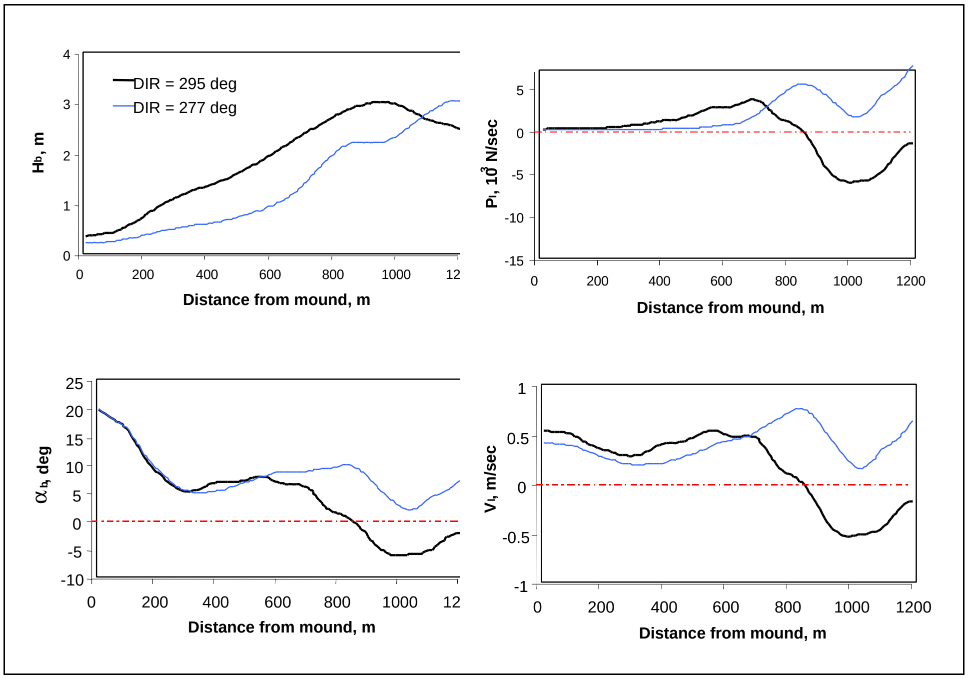


Figure 5-12. Variation in H_b , α_b, V_l, and P_l with distance along the breaker line for for offshore Hs = 4 m, T = 16 sec. (a) DIR = 277 deg; (b) DIR = 295 deg for the existing condition

Figure 5-13 shows maps of wave height, phase, and direction to illustrate the effect of varying incident wave height and period for the existing condition with a local wave angle of 295 deg. Figure 5-14 shows the variation in H_b, α_b, V_l, and P_l with distance along the breaker line for the cases shown in Figure 5-14.

Wave angle relative to the shoreline (α_b) remains positive up to 850 m from the diffraction mound. The positive wave angle combined with a progressive increase in wave height with distance generates an increasing longshore flux potential toward the east. The strong flux gradient in the southwest corner of the bay is the most likely cause of the beach erosion in that area.

The combination of long period waves and large wave heights is clearly the condition that generates the highest breaking wave heights and longshore flux potentials in the southwest corner of the bay.

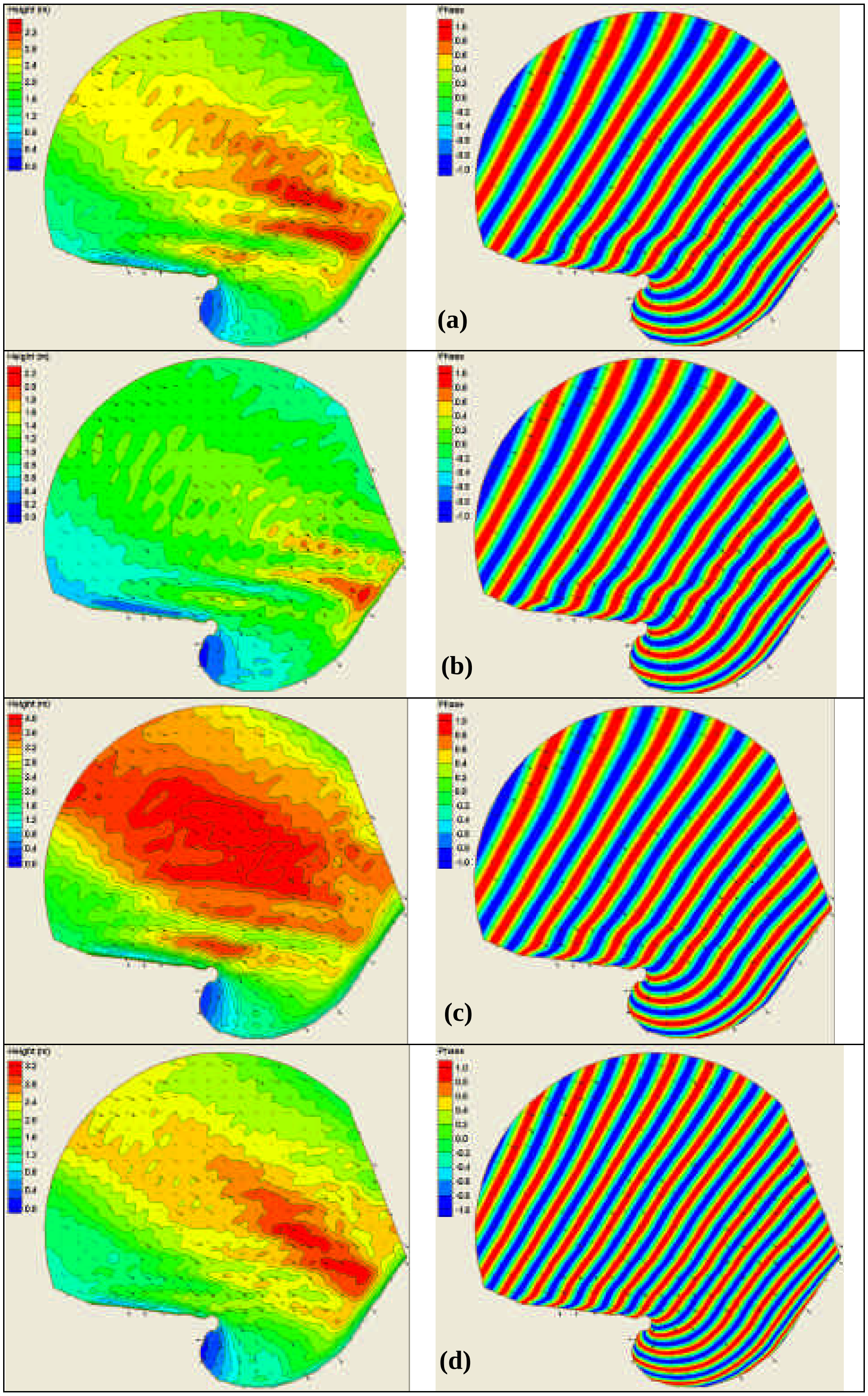
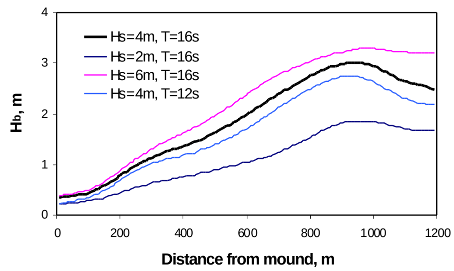
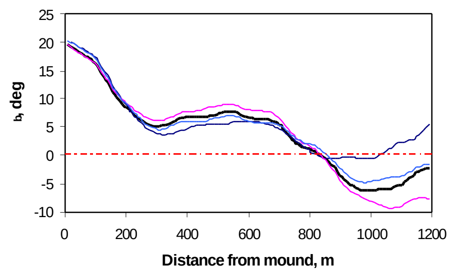


Figure 5-13. Maps of wave height and phase for (a) H_s = 4 m, T = 16 sec; (b) H_s = 2 m, T = 16 sec; (c) H_s = 6 m, T = 16 sec; and (d) H_s = 6 m, T = 12 sec for Local DIR = 295 deg for the existing condition



Hs=4m, T=16s
Hs=2m, T=16s
Hs=6m, T=16s
Hs=4m, T=12s
Hb, m
Distance from mound, m



b, deg
Distance from mound, m

6 Conceptual Design of a Long Term Solution for Prevention of Breaching

This chapter provides a preliminary screening of the alternatives developed in Chapter 4 based on the planshape analysis and numerical modeling presented in Chapters 4 and 5. The preliminary screening leads to the development of a conceptual design, intended for further study, that would potentially provide a long-term solution to breaching at the South Jetty.

Pre-Screening of Conceptual Alternatives

Chapter 4 and Appendix C document a number of conceptual alternatives for prevention of breaching of South Beach at the South Jetty. Since a breach can originate from either South Beach or Half Moon Bay side, a proper solution to the breach threat will include consideration of management of Half Moon Bay and South Beach. A preliminary screening of a portion of the alternatives that address the erosion of the Half Moon Bay shoreline was accomplished by application of planshape equilibrium analysis in Chapter 4. The analysis is extended in Chapter 5 with the application of the CGWAVE model to predict nearshore wave heights and directions as well as longshore transport potential and sediment mobility in the Half Moon Bay nearshore. The various alternatives are summarized in Table 4-3. Table 6-1 provides a possible ranking of the potential alternatives. The ranking system contains some elements of subjectivity but the ranking is useful in providing an overall comparison of the potential alternatives and aiding in the selection of the most suitable choices. A score of 1 to 5 (5 being best) is assigned to each alternative in six categories covering various aspects of the concepts with confidence in providing breach protection and potential environmental issues receiving greater weighting. The weighted total was then used to rank the alternatives.

The ranking process results in Alt A1_500 (raising the jetty remnant to +20 ft over a length of 500 ft) being the top ranked alternative. The numerical and planshape analysis also indicates that the original refraction diffraction mound concept (Alt A2_98) with the mound located 250 ft further east might perform as well as Alt A1_500.

Table 6-1
Ranking of Alternatives for Half Moon Bay*

Alternative	Confidence in providing breach protection	Downdrift effects	Environmental aspects	Construction issues	Construction costs ($)	Maintenance costs ($)	Weighted score	Rank
Weighting Factor	**1.0**	**0.5**	**1.0**	**0.5**	**0.5**	**0.5**		
A1_500. Raise submerged jetty to +20 feet, 500 feet length	5	4	5	4	3	4	17.5	1
A1_250. Raise submerged jetty to +20 feet, 250 feet length	3	4	5	4	4	4	16	2
A2. Diffraction mound modification – Increase mound size, flatter slope	2	4	4	4	2	4	13	6
A3. Diffraction mound modification – Add diffraction spur	3	4	4	4	4	4	15	4
B. Point Chehalis control point	1	1	3	5	4	3	10.5	10
C1. Segmented submerged breakwater	3	5	2	4	2	3	12	8
C2. Nearshore berm	1	5	5	5	5	1	14	5
D1. Geo-terraced revetment	5	3	3	4	4	4	15.5	3
D2. Geo-tube perched beach	3	3	2	2	4	3	11	9
E. Gravel/cobble beach	3	2	1	4	3	1	9	11
F. Sand nourishment	2	5	5	5	1	1	13	6

* Notes: (1) In applying scores: 5 is best, 1 is worst. (2) Unknowns are treated as neutral - assigned a score of 3. (3) Construction costs for Alt. F are anticipated to be high due to 2003 observed erosion rates.

Proposed Conceptual Design of a Long Term Solution

While the focus of this report has been on the Half Moon Bay shoreline, a complete solution must also consider the recession of the South Beach shore since breaching of South Jetty is a combination of the two. When South Beach erodes and the western portion of Half Moon Bay is cut back, the two shorelines meet resulting in a breach. Appendix C contains some discussion of alternatives for the protection of South Beach. The most promising of these would appear to be the use of a revetment, preferably buried, which would provide the shoreline with protection against extreme erosion events while at the same time providing a beach access and an aesthetically acceptable solution. The design of a buried revetment or other protection of South Beach requires further study as a integral part of a total solution.

Based on the preceding analysis and evaluation, a proposed project to prevent breach recurrence at the South Jetty at the entrance to Grays Harbor, Washington is described. The proposed project consists of three components:

- Protection of the southwest portion of the Half Moon Bay shoreline. Of the alternatives identified in Chapter 5, Alternatives A and D offer the most cost-effective and reliable means of preventing the ongoing erosion at this area.
- Buried revetment and sand nourishment between Half Moon Bay and South Beach, and
- Realignment of the Grays Harbor Navigation Channel to eliminate the "dogleg" portion between the North and South Jetties.

Figures 6-1 to 6-4 show the three project elements.

The proposed project is a preliminary concept design, intended for evaluation and testing. Included in this chapter are concept drawings, preliminary analysis, and estimated quantities and costs.

Three project elements are needed to effectively prevent breach recurrence and to maintain the quality of the local beach environment. Individual elements alone will not solve the problem in the long term. Erosion of the breach fill at Half Moon Bay is intermittent, coinciding with episodes of high water levels combined with large waves entering Grays Harbor from the ocean. Tidal currents and wave-driven currents transport eroded sediment out of Half Moon Bay. Table 6-2 lists the cost estimates, and Table 6-3 lists the area of land affected by each element's "footprint". Table 6-4 summarizes each element of the project.

Element 1: Shoreline protection at Half Moon Bay

Two markedly different options for protection and stabilization of the Half Moon Bay shoreline seem most likely to succeed out of the range of possible options. The first option is the restructuring of the eastern terminus of the South Jetty. Alt A1_500, for example, would provide shore stabilization in this area by reducing wave penetration into the southwest corner of the bay. The wave and beach planshape modeling described in Chapters 4 and 5, indicate that this approach would reduce sediment transport in the southwest corner of the bay and stabilize the shoreline position approximately 200 ft east of its current position. The second option is to reinforce the shoreline to enable it to better withstand the waves and high water levels to which it is presently exposed. A beach reinforcement concept similar to Alt_D1, described in Chapter 4, but using conventional rock revetment rather than geotubes is recommended. A terraced rock revetment offers a better combination of long term cost effectiveness and reliability than a geotube option. The final choice for shoreline protection will depend on a number of factors including cost, reliability, environmental permitting, and maintenance considerations.

An overview of these alternatives is presented as follows:

Element 1a: Rebuilding eastern terminus of South Jetty. This alternative (Figure 6-1) will reduce the wave impacts to the southwest portion of Half Moon Bay and shift the updrift control point for the beach to the east thereby allowing a stabilized beach approximately 200 ft east of its present location. This has the desirable benefits of widening the buffer between Half Moon Bay and the Pacific Ocean as well as preserving a natural shoreline. Reconstruction of 250 ft of the jetty (Alt A1_250) would result in a sand shoreline with equilibrium orientation as shown in Figure 6-13.

A 250 ft reconstruction of the jetty would require 22,500 tons of stone and would cost approximately $1.6 M to construct. A 500 ft reconstruction would require 60,000 tons of rock at cost approximately $4.1 M.

Sand nourishment would be required to build the shoreline out to its predicted equilibrium position. It is estimated that a 500 ft reconstruction would require NNN cu yds of sand nourishment and 250 ft reconstruction would require NNN cu yds to restore the shoreline to the predicted equilibrium position. A preliminary sediment budget presented in Chapter 2 revealed a sediment deficit in Half Moon Bay. Analysis of volume trends along the Half Moon Bay shoreline for the period January 2002 to January 2004 indicates that approximately 50,000 cu yds/year of sand is required to maintain the Half Moon Bay shoreline at the existing location. The jetty reconstruction alternatives would reduce this volume but ongoing monitoring would be required and additional maintenance would likely still be required.

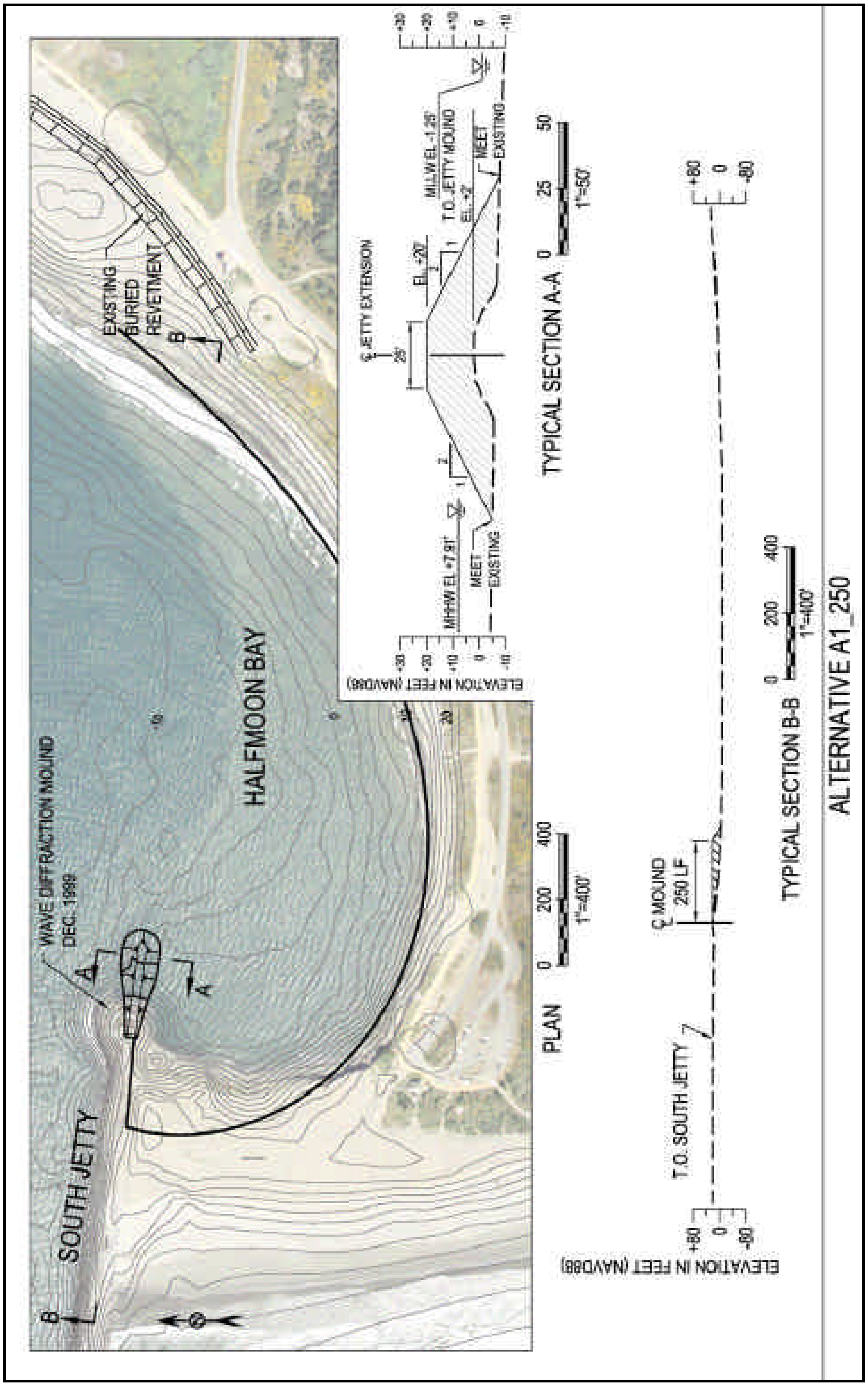


Figure 6-1. Reconstruction of the eastern terminus of South Jetty (Alt A1_250)

Element 1b: Terraced revetment and sand nourishment - Half Moon Bay. The terraced revetment concept was introduced as Alt D1 in Chapter 4. The possibility of vandalism and concerns regarding the long term durability of a terrace constructed with geotextile tubes perhaps favors a terraced rock revetment. The terraced revetment and sand nourishment concept is a direct approach to addressing the erosion of the breach fill and adjacent Half Moon Bay shoreline. A revetment relatively high on the profile will prevent erosion and consequent scarp formation during storms. The existing scarp formed by erosion of the Half Moon Bay shoreline is a potential public safety hazard and significantly detracts from the aesthetic quality of the shore. A terraced, or stepped, profile will provide a more gradual transition to the shore, facilitate beach access for the public, as well as provide effective dissipation of incident wave energy. Aesthetics, environmental quality, and erosion resistance can be further improved by incorporating erosion resistant vegetation in the design of the shore protection. The terraces shown in Figure 6-2 are parallel. However, it is possible that non-parallel terraces or smooth transitions between terraces can be designed to create a relatively large and level "perched beach" area that will enhance recreation benefits and aesthetics.

Periodic sand nourishment will also be required to maintain beach levels and prevent erosion downdrift from the project. A preliminary sediment budget in Chapter 2 concluded that a sediment deficit exists in Half Moon Bay. A preliminary estimate is that approximately 50,000 cu yds/year of sediment is required to maintain the shoreline of Half Moon Bay (Figures 2-32 through 2-34).

Sand nourishment is not included in the concept design depicted in Figure 6-2. However, an initial sediment placement of 250,000 cu yd is included in the cost estimate in Table 6-3. Further study is needed to determine the required volumes and frequency of sand nourishment, and the effect of nearshore bathymetry changes on sand nourishment requirements. The sand nourishment should be placed at an "updrift" location, near the South Jetty and terraced revetment. The sediment would be transported to the east by longshore currents.

The cost of the terraced revetment outlined in Figure 6-2 is estimated to be $2,413,000, or $3,220 per ft of shoreline. The cost estimate is listed in Table 6-2.

Element 2. Buried Revetment and Sand Nourishment - South Beach

A buried revetment in the middle of the historic breach area will provide a long-term solution to breaching. Analysis of historic charts and aerial photos reveals the long-term trend is shoreline retreat on the ocean side in the region immediately south of the South Jetty. However, this erosion from the South Beach side of the breach area (the ocean side) has been slowed in recent years by sand placement and maintenance activities of the Seattle District, including placing dredged sand in nearshore berms (PI Engineering, 2003). A buried revetment will provide an element required for long-term protection of the breach area. Periodic sand nourishment will also be required at South Beach to help maintain beach levels and prevent erosion downdrift from the project.

Sediment budget analysis and shoreline change analysis (Sultan and Osborne, 2003) concludes that a sediment deficit exists. A preliminary estimate is that approximately 80,000 cu yd/year of sediment is required to maintain the shoreline of South Beach. The 80,000 cu yd/year is based on analysis of short-term and long-term shoreline trends, the history of sand fill placement in the breach area, and nearshore bathymetry changes.

Sand nourishment is not included in the concept design depicted in Figure 6-3. However, an initial sediment placement of 400,000 cu yds is included in the cost estimate in Table 6-2. Further study is needed to determine the required volumes and frequency of sand nourishment, and the effect of nearshore bathymetry changes on sand nourishment requirements. The sand nourishment should be placed at an "updrift" location, adjacent to the South Jetty.

The cost of the buried revetment outlined in Figure 6-3 is estimated to be $6,898,000, or $8,630 per ft of revetment. The cost estimate is listed in Table 6-2.

Element 3. Channel Realignment

The Grays Harbor Navigation Channel is adjacent to and parallel to the South Jetty. Before reaching Half Moon Bay, the channel alignment changes, forming a "dogleg" in the area between the North and South Jetties. It has been hypothesized that channel realignment would result in less wave energy reaching Half Moon Bay, this would lead to reduced shoreline erosion. The channel may act as a "wave guide", channeling wave energy along its axis and enhancing the height of waves that subsequently diffract around the terminus of the South Jetty and enter Half Moon Bay.

Channel realignment will likely have other benefits, including improved navigability and reduced wave induced damage to the South Jetty. Channel realignment may also make it easier to dredge and maintain the channel depth. The proposed realignment would reduce the area of seafloor habitat affected by dredging. Further study is needed to test these hypotheses and concepts, including numerical wave transformation modeling and sediment transport and morphological change modeling. Additional questions to be addressed include whether channel realignment would persist, and the effects of sand wave migration. Further study is needed, including a dredging demonstration project.

The cost of the channel realignment shown in Figure 6-4 is estimated to be $2,115,000. This is the initial cost for dredging a new channel and is based on bid prices for maintenance dredging of navigation channels in the Pacific Northwest, and an initial dredging volume of 750,000 cu yds. The cost estimate is included in Table 6-2. The dredging volume is estimated assuming that dredging will be required over a 4,000 ft length of channel. The volume is based on the geometry of a prism with the following dimensions; a trapezoidal channel cross section, channel width of 700 ft, 3:1 side slopes, a channel depth of –40 ft, an existing seafloor elevation of –35 ft and 2 ft of advanced maintenance dredging.

Table 6-2
Preliminary Cost Estimate for Proposed Conceptual Alternatives for Half Moon Bay

1a. Jetty extension (500 ft)	_	**Quantity**		**Unit Cost**	**Total**
Mobilization		1	EA		$290,000
Site Preparation		1	EA		$15,000
Armor Stone		60,000	Ton	$48.00	$2,880,000
Contingency	30%	1	EA		$956,000
				Total:	**$4,141,000**
1a. Jetty extension (250 ft)	_	**Quantity**		**Unit Cost**	**Total**
Mobilization		1	EA		$110,000
Site Preparation		1	EA		$15,000
Armor Stone		22,500	Ton	$48.00	$1,080,000
Contingency	30%	1	EA		$362,000
				Total:	**$1,567,000**
1b. Terraced Revetment - HMB	-	**Quantity**		**Unit Cost**	**Total**
Mobilization		1	EA		$169,000
Site Preparation		1	EA		$15,000
Armor Stone		27,000	Ton	$48.00	$1,296,000
Geotextile		7,500	SY	$6.60	$50,000
Excavation and Placement		1,000	CY	$5.00	$5,000
Sand Fill		250,000	CY	$1.00	$250,000
Plantings		7,100	SY	$10.00	$71,000
Contingency	30%	1	EA		$557,000
				Total:	**$2,413,000**
2. Buried Revetment - South Beach	-	**Quantity**		**Unit Cost**	**Total**
Mobilization		1	EA		$482,000
Site Preparation		1	EA		$15,000
Armor Stone		56,500	Ton	$48.00	$2,712,000
Underlayer Stone		22,500	Ton	$48.00	$1,080,000
Geotextile		12,000	SY	$6.60	$79,000
Excavation and Placement		107,500	CY	$5.00	$538,000
Sand Fill		400,000	CY	$1.00	$400,000
Contingency	30%	1	EA		$1,592,000
				Total:	**$6,898,000**
3. Channel Realignment	_	**Quantity**		**Unit Cost**	**Total**
Mobilization		1	EA		$500,000
Dredging		750,000	CY	$1.50	$1,125,000
Contingency	30%	1	EA		$490,000
				Total:	**$2,115,000**

Table 6-3
Half Moon Bay Project Element Footprint

	Area (SF) Above MHW	**Area (SF) MHW-MLLW**	**Area (SF) Below MLLW**	**Area (SF) Total**
1a. Jetty extension (500 ft)	7,000	9,700	10,000	26,700
1a. Jetty extension (250 ft)	7,000	9,700	42,000	58,700
1b. Terraced Revetment - HMB	101,000	4,600		105,600
2. Buried Revetment - South Beach	116,000			116,000
3. Channel Realignment			2,968,000	2,968,000

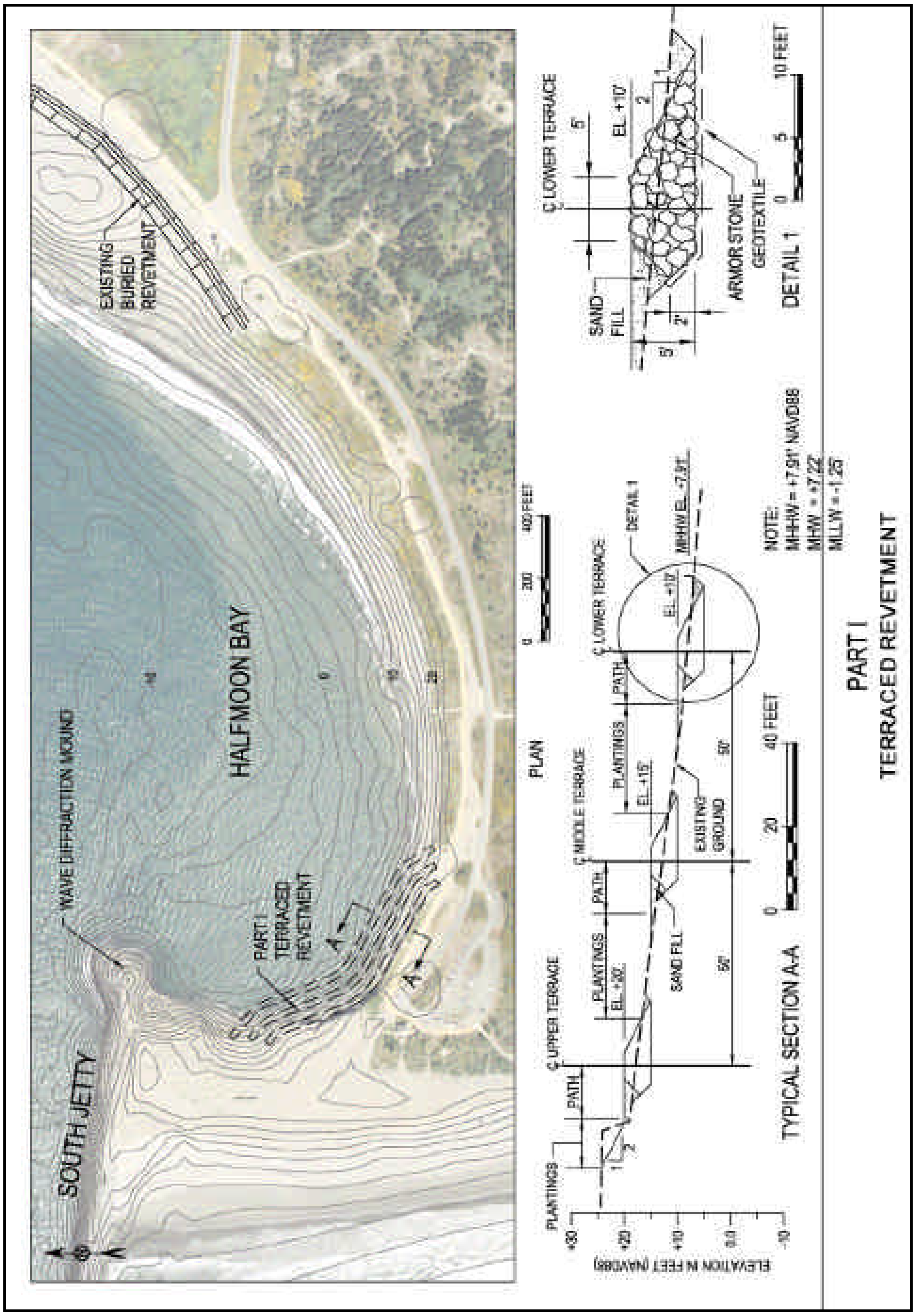


Figure 6-2. Terraced Revetment

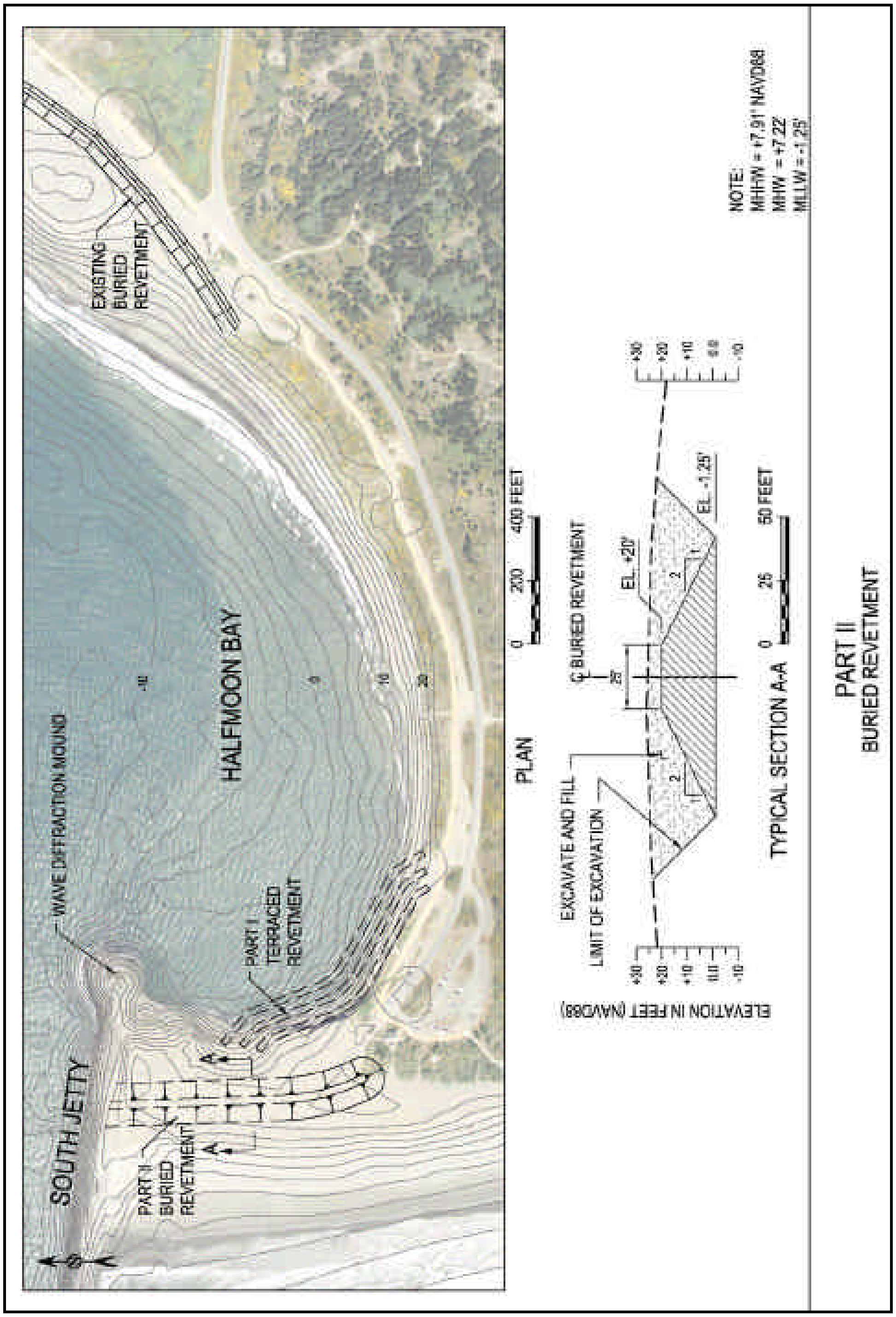


Figure 6-3. Buried Revetment

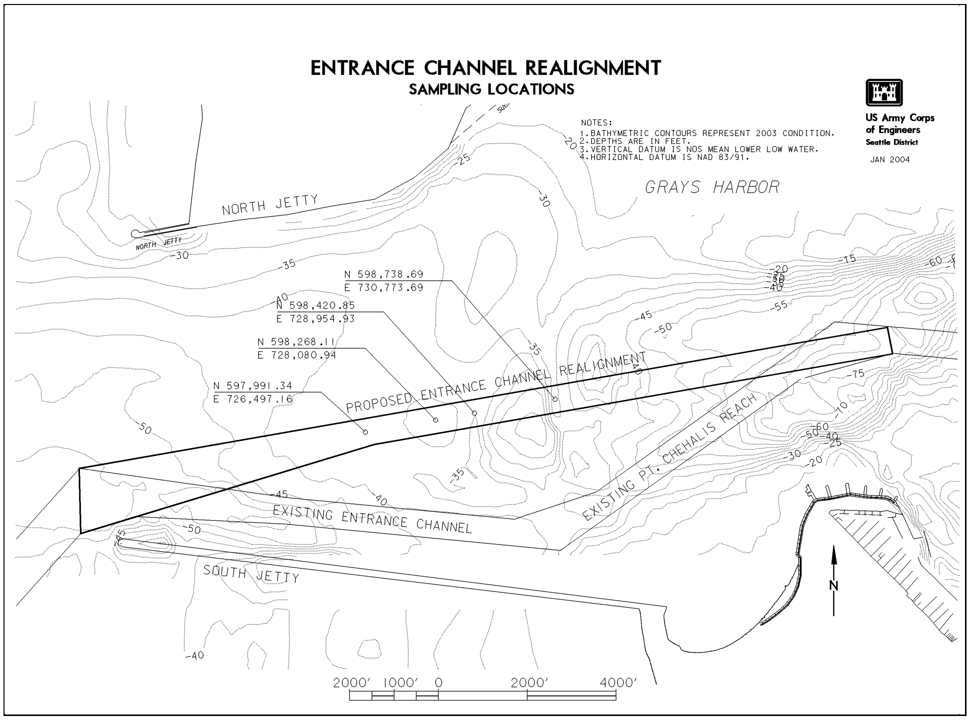


Figure 6-4. Seattle District Proposal for Realignment of the Grays Harbor Navigation Channel

Table 6-4
Summary - Half Moon Bay Project to Prevent Breach Recurrence

Element	Description	Environmental aspects	Construction issues	Cost Estimate	Notes
1a Rebuilding eastern terminus of South Jetty.	Move control point for southwest beach eastward by rebuilding jetty remnant over a distance of between 250 to 500 ft.	No impact to inter-tidal area nor to land above MHW. Extension will lie over existing remnant footprint on sub-tidal area.	Minimal.	22,500 to 60,000 tons of stone may be required. $1.6 to $4.1 Million depending on length of extension.	No new impact to sub-tidal area. Planshape analysis indicates it will result in improved shoreline position. Needs physical and numerical modeling to determine potential reductions in waves and transport.
1b Terraced revetment and sand nourishment.	Construct a terraced revetment to prevent erosion of the Half Moon Bay shoreline. Nourish Half Moon Bay to maintain seabed level and shoreline position relative to revetments.	Approximately 4,570 SF of inter-tidal area will be affected, and 100,860 SF of land above MHW.	None.	27,000 tons of stone may be required. $2.5 Million.	No new impact to sub-tidal area, and minimal impact to inter-tidal area. Needs physical and numerical modeling to determine potential reductions in longshore and cross-shore sediment transport .
2. Buried revetment and sand nourishment.	Construct a buried revetment to prevent a breach between Half Moon Bay and the Pacific Ocean. Nourish South Beach to maintain beach level and prevent erosion of downdrift beaches.	No inter-tidal or sub-tidal habitat will be affected. The total footprint area is 116,000 SF, entirely above MHW.	A stockpile area will be required for temporarily excavated sand.	79,000 tons of stone may be required $6.9 Million.	No new impact to sub-tidal and inter-tidal areas.

Table 6-4
Summary - Half Moon Bay Project to Prevent Breach Recurrence

Element	Description	Environmental aspects	Construction issues	Cost Estimate	Notes
3. Channel realignment	Dredge a new entrance channel to eliminate the “dogleg” portion between the North and South Jetties.	Approximately 3 million SF of sub-tidal area will be dredged to form the new channel. Approximately 12 million SF of sub-tidal area occupied by the existing channel will no longer be affected by dredging.	Conventional hopper dredging can accomplish all work.	Approximately 750,000 cu yds of dredging required. $2.2 Million	Model testing and analysis is required to quantify the effect of channel realignment on wave energy reaching Half Moon Bay.

References

PI Engineering (2003). “South Beach Shoreline Change Analysis”, A report to the Southwest Washington Coastal Communities, Pacific International Engineering, Edmonds, Washington.

7 Integrated Summary

This study has provided an up-to-date evaluation of engineering measures implemented at Half Moon Bay through an analysis of recent field measurements of coastal processes, and beach and nearshore morphological change in Half Moon Bay. The evaluation provides a basis for the development of a set of conceptual engineering alternatives to improve the performance of existing breach prevention measures. A preliminary evaluation of alternatives with planshape analysis and numerical modeling aided in the development of the concept design, presented in this section, that would potentially provide a long-term solution to breaching at the South Jetty.

The evaluation indicates that the efforts taken to prevent re-breaching and to place sediment in a manner beneficial to the Half Moon Bay shoreline have been effective, but are not necessarily an efficient long-term solution. The ongoing erosion in the southwest corner of the bay is an indicator that the diffraction mound and gravel-cobble transition beach are not optimized in terms of performance. The accelerated rate of erosion in the southwest corner of the bay since the construction of the diffraction mound correlates with removal of the jetty rock from the eastern terminus of the jetty (remnant) and a westward shift of the diffraction control point. The placement of gravel and cobble in the transition beach has caused the transition beach shoreline position to be further east than the predicted equilibrium position according to the model of Hsu and Evans. However, significant quantities of gravel and cobble have been transported out of the transition beach area reducing the effectiveness of the gravel transition beach to provide protection to the breach fill.

Local wave climate analysis indicates that the waves approach the boundary of Half Moon Bay predominately from westerly through northwesterly directions. The present wave transformation patterns does not sufficiently redirect the incoming waves to result in a beach planshape compatible with the present or desired shoreline in order to prevent breaching of the South Jetty. The alternatives presented in this report addressed protection of the Half Moon Bay shoreline by creating a more favorable wave climate, fixing the shoreline position, nourishing the beach or a combination thereof. Alternatives A1_250 and A1_500 modeled for nearshore wave response using CGWAVE provide increased sheltering and reduced sediment transport potential along the western portion of Half Moon Bay thereby reducing the risk of breaching.

In Chapter 6 a preliminary conceptual design is developed to prevent breach recurrence at the South Jetty at the entrance to Grays Harbor, Washington. The conceptual design incorporates the most promising alternatives for reducing shoreline erosion in Half Moon Bay developed in this report as well as conceptual alternatives for reducing the threat of breaching originating from the Pacific Ocean side of South Beach. The proposed project intended for further evaluation and testing consists of three project components:

- Protection of the southwest portion of the Half Moon Bay shoreline. Of the alternatives identified in Chapter 5, Alternatives A and D offer the most cost-effective and reliable means of preventing the ongoing erosion at this area.
- Buried revetment and sand nourishment between Half Moon Bay and South Beach, and
- Realignment of the Grays Harbor Navigation Channel to eliminate the "dogleg" portion between the North and South Jetties.

Appendix A – Field Measurements of Waves, Currents, and Suspended Sediments in Half Moon Bay

Introduction

This appendix provides background information on the field data collection in Half Moon Bay, Grays Harbor, Washington between 9 December 2003 and 19 February 2004, as part of the Grays Harbor South Jetty Performance Study. Pacific International Engineering, PLLC conducted field measurements, data processing, and analysis for the Engineer Research and Development Center – Coastal Hydraulics Laboratory (ERDC-CHL) under its Broad Agency Announcement contract DACW42-01-C-0011. An overview of the data collection program is followed by descriptions of the data collection methods and equipment, deployment methods, data recovery, data processing and quality checks, and time-series plots of the measured parameters. Further information on the platform design, instrument configuration, and deployment methods can be found in Osborne, Hericks and Kraus (2002b).

High-quality field measurements are an integral part of the design process for new and existing coastal engineering projects. A fundamental element to success in modeling is a field measurement program designed to obtain as much information as possible about crucial input parameters and especially model output parameters. Calibration and data verification assist in substantially reducing uncertainty of model outputs so that final results are useful, quantitative approximations. Carefully collected, high-resolution, field measurements yield valuable insights to aid in the interpretation of processes active in a project area.

The measurement program was designed to obtain detailed field measurements of directional waves in Half Moon Bay. The objective of the measurements is to provide information to verify numerical models for wave transformation, thereby advancing their value as design tools to aid in the optimization of project performance.

Overview

The field data collection effort involved the deployment of four instrument platforms to measure waves, currents, and suspended sediment concentrations (SSC) in and around Half Moon Bay. The data collection was scheduled to occur during two months of the winter storm season, with the platforms being deployed for two consecutive 30-day periods. Deployment 1 encompasses the period from 9 December 2003 through 10 January 2004. Deployment 2 encompasses the period from 11 January 2004 through 19 February 2004. In each deployment, stations were identified as HM1 through HM4. Table A-1 indicates deployment and retrieval dates along with time, location, and elevations.

Table A-1
Tripod Deployment Locations

Station ID	Deployment Date	Position				Elevation	Retrieval Date
		Latitude[1]	Longitude[1]	Easting[2]	Northing[2]		
HM1	12/9/03 1/11/04	N 46 54.5790 N 46 54.5711	W 124 07.9999 W 124 07.9942	E 732658 E 732680	N 595744 N 595695	-21 -21	1/10/04 2/19/04
HM2	12/9/03 1/11/04	N 46 54.4810 N 46 54.4632	W 124 07.3180 W 124 07.3303	E 735469 E 735412	N 595018 N 594913	-10 -10	1/10/04 2/19/04
HM3	12/9/03 1/11/04	N 46 54.2360 N 46 54.2648	W 124 07.4870 W 124 07.5185	E 734697 E 734574	N 593563 N 593744	+1 -5	1/10/04 2/19/04
HM4	12/9/03 1/11/04	N 46 54.2450 N 46 54.2476	W 124 07.7320 W 124 07.7014	E 733680 E 733808	N 593665 N 593675	-4 -4	1/10/04 2/19/04

[1] Format is D ddd mm.mmmm, where D = N, S, E, or W; ddd = 1 to 3 digits, degrees; mm.mmmm = two digits and four decimal places, minutes; Referred to North American Datum of 1983

[2] Referred to North American Datum of 1983 – Washington South 4602 (in feet).

[3] In feet referred to mean lower low water (mllw).

During both deployments, station HM1 was deployed inside the entrance of Grays Harbor between the South Jetty and the Point Chehalis Reach of the shipping channel at an elevation of approximately -6.4 m (-21 ft) mllw. Station HM2 was deployed near the USCG Point Chehalis Front Range Tower at an elevation of approximately -3 m (-10 ft) mllw. Station HM3 was deployed near the beach in the southeast portion of Half Moon Bay. Because of difficult weather conditions and the boat crew's unfamiliarity with the type of deployments used, the HM3 platform was first deployed at an elevation of approximately +0.3 m (+1 ft) mllw, rather than the planned elevation of approximately -1.6 m (-5 ft) mllw. This deployment location resulted in the sensors emerging above the waterline at tide elevations below approximately +1 m (+3 ft) mllw. Evidence of the intermittent surfacing of the sensors can be seen in Figure A-20, which show the water depth to sensor.

During Deployment 2, the HM3 station was placed at a more favorable location with a bed elevation of approximately -1.6 m (-5 ft) mllw. The difference in average sensor depth between the two deployments can be seen in Figure A-18. Station HM4 was deployed in the southwest portion of Half Moon Bay, at an elevation of approximately -1.3 m (-4 ft) mllw. This location was chosen to situate the platform in the lee of the diffraction mound, south of the eastern terminus of the South Jetty. Figure A-1 illustrates the location of each station.

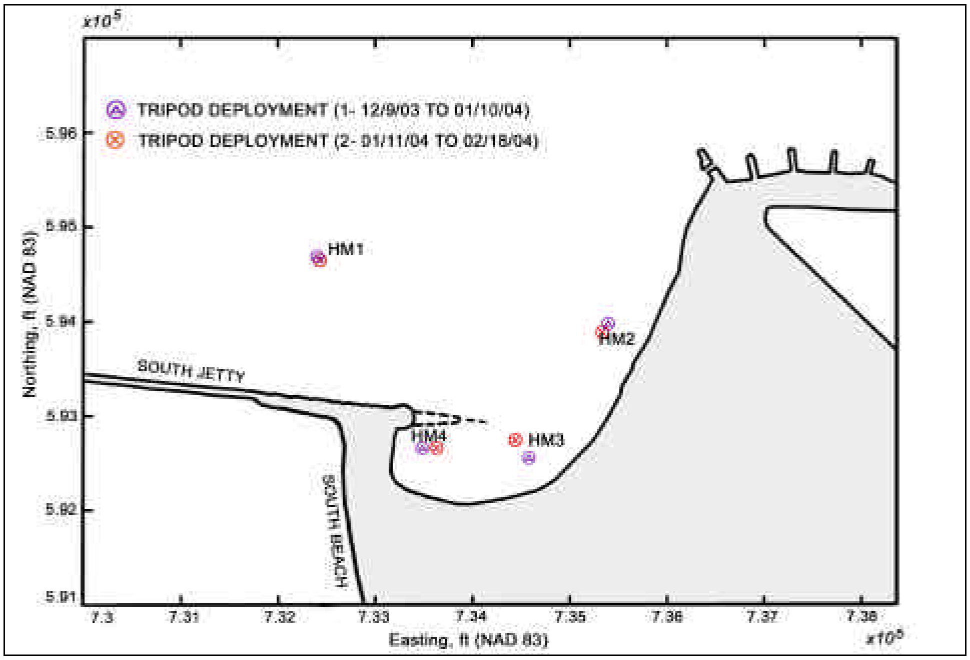


Figure A-1. Location of instrument deployment in Half Moon Bay

PI Engineering processed the recorded instrument data with SonTek ViewHydra, WavesMon, WaveView, and in-house software. Initial quality checks of the ADCP data at HM1 indicated that it was moved and overturned on 24 January 2003. However, useful data was recorded between 11 January 2003 and 24 January 2003, approximately 35 percent of the total deployment cycle. January is the peak crab season in Grays Harbor and there were numerous crab traps near the platform. It is likely that the platform was accidentally snagged and pulled into deeper water when crabbers retrieved their traps. Upon recovery, debris was found on one side of the platform, indicating that it had been on its side at one time.

A high percentage of quality data was recovered from the other instrument platforms during both deployments.

Data Collection Methods and Equipment

Instrument platforms consisted of two tripod frames (HM1 and HM2) and two trapezoidal frames (HM3 and HM4). Wooden pads were placed on the base of the platform legs and attached with lag bolts. The pads, designed to break free during recovery, prevented the platforms from sinking into the seabed. The HM1 configured platform is shown in Figure A-2.

Figure A-2. HM1 tripod instrument platform with ADCP prior to deployment

All equipment was transported from Seattle on a flatbed truck and assembled upon arrival in Westport. After all equipment was attached to the sensor platforms, function checks of all instruments were conducted. The ADVO sensors housing the Hydra compasses were aligned vertically with an “up-looking” orientation on the shallow water trapezoidal frames and a “down-looking” orientation on the tripod station for HM2. The compass and tilt sensors were positioned within the sensor head to provide correct heading, pitch and roll data. The sampling method for all instruments is shown in Table A-2.

Table A-2
Data Sampling Method

Equip.	Data Type	Recorded Data	Start Recording	Record Interval (min)	Record Duration (min)	Sample Frequency (Hz)	Samples Recorded/ Burst
ADCP (HM1)	Current velocity profiles	Velocity	Every 90 min.	6	3	N/A	4080 (1 avg.)
ADCP (HM1)	Depth, waves	Pressure, Current, Dist. To Surface	Every 90 min.	36	34.13	2	4080
Hydra (HM2-4)	Directional wave burst	Orbital velocity & pressure burst	Every 80 min.	40	34.13	4	8192
Hydra (HM2-4)	OBS suspended sediment concentration	Counts	Every 80 min.	40	34.13	4	8192

Stations HM2 to HM4 were instrumented with a SonTek Hydra configured with a high-resolution pressure sensor, an Acoustic Doppler Velocimeter Ocean (ADVO), and two optical back-scatterance sensors (OBS). The combined velocity measured near the beach surface by the ADVO along with suspended sediment measurements enables the calculation of suspended sediment flux. The combined measurements of the ADVO and pressure sensor enables the calculation of directional wave information. Instrumentation on HM1 consisted of an RDI 1,200 kHz Acoustic Doppler Current Profiler (ADCP) equipped with a pressure transducer and configured to record directional wave data, water levels, and current speed and direction through the water column in 0.65 m bins starting at 1.20 m above the transducer for a total number of 18 bins.

Deployment Method

Three different commercial fishing vessels were used to deploy and recover the platforms on this project. A mooring line, with an intermediate anchor weight and two surface buoys, was deployed with each platform. On each platform, a "ground" line was attached to one end and an 80 – 140 lb (36 - 64 kg) anchor weight on the other. A second "buoy" line was also shackled to the anchor weight with a 14 inch crab float and a 12 inch spherical trawl float at the opposite end. On HM1, the "ground" line was 100 ft long. On the shallower platforms (HM2, HM3), the ground lines were 50 ft long. If the surface buoys or buoy-lines disappeared during deployment, the ground lines would provide a good target for grappling. Mooring with an intermediate anchor also has the advantage of de-coupling the wave-induced motion and strain the platforms place on the buoys and buoy-lines.

The platforms were deployed using the available rigging on the vessel. The typical deployment procedure employs a line-controlled quick-release on the end of the vessel's main boom winch-line. The intermediate anchor is first suspended over the side of the vessel on a second quick-release and all mooring lines are laid out to minimize entanglement. The platform quick-release is attached to a shackle at the apex of the platform; the platform is then lifted off the deck and over the side (guided by deck crew) and lowered to the bottom. Once the platform is situated on the bottom, a crewmember triggers the platform quick-release. The vessel's winch-line and release-line are then recovered while a second (and possibly third) crewmember pays out the ground line, releases the intermediate anchor and pays out the buoy-line while slowly lowering the anchor to the bottom. When the ground line is taught, the second crewmember releases the anchor. The buoys and the buoy-line are then tossed over the side.

Recovery of the platforms is accomplished using a large "crab-block" capstan winch and the vessels boom/crane winch. The deployment platforms' mooring line is put in the crab-block, and the vessel is maneuvered to a location above the platform. The platform is raised along side the vessel until the boom/crane line can be attached to the platform and then it is lifted over the vessels' side-rail and lowered to the deck.

During each deployment and recovery, a differential GPS and navigation computer are set up on the ships' bridge. Planned location "targets" are used to position the vessel at the desired deployment location and targets of the platform position and anchor weight are recorded. The vessels echo sounder is typically checked with a sounding line at dockside in the marine to determine the echo sounder draft setting. Prior to deployment the "echo sounder reported depth" for each platform is calculated using the planned instrument deployment depth, predicted tide, estimated deployment time, and vessel echo sounder offset. This report depth allows the ships captain to quickly check for the appropriate depth when positioned at the deployment site.

Calibration of Optical Backscatter Sensors

All OBS sensors deployed with Hydra systems were calibrated for suspended sediment concentration. OBS analog voltage signals are converted to digital levels and recorded by the Hydra system in "counts" ranging from 100 to 65,000, which must be converted to the desired units during post processing. Calibrations were performed in a turbidity chamber following the specifications recommended by the manufacturer. Instrument gains were set prior to deployment using tap water (minimum) and 800-nepthelometric turbidity units (ntu) and Formazin standard solution (maximum). OBS sensors deployed on the instrument platforms were calibrated over a range of 0 to 25 grams per liter (g/l) at four concentrations (2, 8, 16 and 25 g/l) with sediment previously collected in Grays Harbor near Half Moon Bay.

OBS calibration data (n = 1028 at 4 Hz) was extracted from the Hydra files. Statistics including the average, standard deviation, minimum, maximum, and coefficient of variation were computed and the series plotted for a visual quality check. Coefficient of variation was typically 10 percent, and not more than 15 percent for acceptable calibration results. Second-order polynomial curves fit to the calibration data yielded high correlations (average $R^2 = 0.9993$). Calibration coefficients are summarized in Table A-3. In processing the measured time series, the calibration formula was applied to each sample to convert OBS sensor counts to suspended sediment concentration in g/l. They are the same for both deployments.

Table A-3
OBS Calibration Results 2003

Station No.	Serial No.	Elevation above bed (m)	Calibration coefficients for 0-24 g/l range			R^2
			A	B	C	
HM 2	1404	0.40	4.157394E-09	2.063481E-04	8.084542E-03	9.993642E-01
HM 2	1402	0.23	4.281899E-09	2.036476E-04	-6.397939E-02	9.988544E-01
HM 3	1399	0.30	3.173032E-09	2.636814E-04	-3.694748E-01	9.997223E-01
HM 3	1398	0.20	3.375241E-09	2.370680E-04	-2.324662E-01	9.992140E-01
HM 4	1406	0.30	3.872221E-09	2.270801E-04	-1.005294E-01	9.994976E-01
HM 4	1403	0.20	3.817009E-09	2.141522E-04	-8.378580E-02	9.984898E-01

Calibration Formula: $y = Ax^2 + Bx + C$
Where y is suspended sediment concentration in g/l, x is OBS sensor counts, and A, B, and C are the calibration coefficients.

Data Recovery

Data recovered from all instruments during both deployments are summarized in Tables A-4 through A-7. Tables show the deployment and recovery times as well as the total amount of recorded and processed data. Pressure and velocity data loss occurred at HM3 because of instrument emergence at low tide during Deployment 1.

Table A-4
Deployment 1 SonTek Hydra Data Recovery

Deployment (12/9/03 to 1/10/04)			
Hydra Data Recovery	**HM2**	**HM3**	**HM4**
Station/File Name	HM2V1001.adr	HM3V1001.adr	HM4V1001.adr
Original File Size (bytes)	13784672	139190473	137824701
Number of Recorded Bursts (8192 samples/burst)	580	585	579
Time of First Recorded Burst	12/9/03 - 1800	12/9/03 - 1800	12/9/03 - 1800
Time of Last Recorded Burst	1/10/04 - 2200	1/11/04 - 0440	1/10/04 - 2040
Number of Processed Bursts	580	585	579
Number of Usable Bursts (Pressure)	579	584	578
Number of Usable Bursts (Velocity)	579	394	578
Percent Data Recovery (Pressure/Velocity)	99.8% 99.8%	99.8% 67.4%	99.8% 99.8%

Table A-5
Deployment 2 SonTek Hydra Data Recovery

Deployment (1/11/04 to 2/19/04)			
Hydra Data Recovery	**HM2**	**HM3**	**HM4**
Station/File Name	HM2V2001.adr	HM3V2001.adr	HM4V2001.adr
Original File Size (bytes)	167583219	167583219	167583219
Number of Recorded Bursts (8192 samples/burst)	705	705	705
Time of First Recorded Burst	1/11/04 - 1900	1/11/04 - 1900	1/11/04 - 1900
Time of Last Recorded Burst	2/19/04 - 2140	2/19/04 - 2140	2/19/04 - 2140
Number of Processed Bursts	705	705	705
Number of Usable Bursts (Pressure)	704	704	704
Number of Usable Bursts (Velocity)	704	704	704
Percent Data Recovery (Pressure/Velocity)	99.8% 99.8%	99.8% 99.8%	99.8% 99.8%

Table A-6
Deployment 1 ADCP Data Recovery

Deployment (12/9/03 to 1/10/04)	
ADP Data Recovery	HM1
Station/File Name	DPL1_001.000
Original File Size (Kbytes)	168048
Number of Recorded Bursts	514
Time of First Recorded Burst	12/9/31 - 1900
Time of Last Recorded Burst	1/10/04 - 1730
Number of Processed Bursts	511
Number of Usable Bursts (Pressure)	509
Number of Usable Burst (Velocity)	509
Percent Data Recovery (Pressure)	99%
Percent Data Recovery (Velocity)	99%

Table A-7 Deployment 2 ADCP Data Recovery	
Deployment (1/11/04 to 2/19/04)	
ADP Data Recovery	HM1
Station/File Name	DPL2_001.000
Original File Size (Kbytes)	204594
Number of Recorded Bursts	627
Time of First Recorded Burst	1/11/04 - 1900
Time of Last Recorded Burst	2/19/04 - 2030
Number of Processed Bursts	627
Number of Usable Bursts (Pressure)	626
Number of Usable Burst (Velocity)	203
Percent Data Recovery (Pressure)	99%
Percent Data Recovery (Velocity)	32%

As stated earlier, the reason for the low percentage of velocity data recovery shown in TableA-7 is the overturning and relocation of HM1 during deployment 2.

Data Processing and Quality Checks

A preliminary visual data quality check was performed on raw Hydra data using SonTek ViewHydra software. Data was extracted from raw data files (*.adr) using SonTek Hydra extraction software and written to ASCII time series (*.ts), header (*.hdr) and control (*.ctl) files. All remaining processing and post-processing was accomplished using in-house PI Engineering software.

Processing and quality checking of extracted time series files consisted of the following steps:

a. plotting header file parameters (*.hdr): Heading, pitch and roll angles, mean temperature, and mean pressure, and bed position (if relevant) were plotted as time series as a step in the data quality check process to identify periods when instruments may have shifted position or been subject to burial or fouling.

b. calibration and conversion of time series: A Matlab routine (preproc.m) and associated subroutines processed the extracted time series files and produced corrected and calibrated ASCII time series files (*.tsc). The routine accomplished the following:

    1) horizontal components (E, N) of ADVO velocities were corrected from magnetic north to true north direction using the magnetic declination for the location and time of deployment.

    2) pressure measurements were converted to static water depth above the ADV using mean barometric pressure and water density (calculated from temperature and estimated salinity) during the deployment period (p2h.m). Measured water temperature and estimated salinity were converted to water density using the International Equation of State of Seawater 1980 (IES80) (Fofonoff and Millard 1983). Bursts were eliminated from post-processing analyses that were above the water surface through comparing the height of the bin with the water depth mean.

3) water surface elevation series suitable for wave height and period calculation were calculated using static water depths and horizontal components of velocity by correcting for pressure attenuation and orbital velocity attenuation as a function of depth and wave frequency (p2eta.m; uv2eta.m). Corrections were carried out in the frequency domain and converted to the time domain for output. The attenuation correction factors for pressure and velocity are based on the linear wave theory dispersion relation and the maximum frequency cutoff is based on algorithms developed by PI Engineering that were reported by Earle, McGehee, and Tubman (1995), which is dependent on the water depth.

4) Optical Back-Scatterance Sensor (OBS) counts were converted to suspended sediment concentration using laboratory calibration coefficients (obscal-03-12-04.txt). OBS sensors were calibrated in a turbidity tank with bed sediment from the deployment site prior to the deployment . Data were inspected for evidence of bio-fouling and sensor burial. Bio-fouling and the approach to burial are indicated in the SSC signal by rising background or change in the sensor offset. Complete burial is indicated by a significant change in sensor offset. Subjective estimates of when bio-fouling or burial became significant and affected data were discarded from analysis.

5) processed time series data were output to *.tsc files identical in format to the extracted *.ts format.

6) burst-averaged summary statistics files (*.sts) were generated by taking the mean and variance of the processed data from each 8192-point burst.

Data post-processing was performed on the velocity (E, N, U) data to remove poor quality or erroneous data. Poor quality data is typically a result of environmental conditions, which cause poor acoustic signal return and poor signal correlation.

Recovery of the RDI Acoustic Doppler Current Profiler (ADCP) data was completed by Orders Associates Research Systems, LLC in Edmonds, Washington, who had leased the instrument to PI Engineering. The data extraction was accomplished by PI Engineering using RDI WavesMon Version 2.01 software, which processed the individual bursts into a log file (*.000) and a wave file (*.wvs). RDI WaveView software was used to read the wave files and perform initial quality checks on time series and directional and autospectral estimates. The loss of current data and increase in platform depth was discovered during this initial quality check.

A standard wave parameter file was then created from the WaveView software containing burst averaged wave height, period, direction, depth to sensor, current magnitude, and direction. Both the wave parameter and log files were imported into Excel for post-processing analysis. Extraneous data was filtered and the remaining data checked for consistency.

Data Quality

The quality of the data obtained at stations HM2 to HM4 was verified by plotting pressure and velocity data of the individual bursts using the SonTek ViewHydra software. Header files (*.hdr) produced by the SonTek ADV software during initial raw data processing contain burst statistics for evaluating: mean heading, pitch and roll angles, E, N, U velocity, mean signal correlation, mean temperature, velocity boundary range, and mean pressure. Recorded heading, pitch, and roll angles were inspected to determine if there were significant changes in instrument platform orientation and tilt.

The mean signal correlation is another important parameter used to check data quality. SonTek ViewHydra computes the Northing, Easting, and Upward correlation components of a selected burst as a percentage. In post processing, the mean signal correlation is calculated from these three components. Bursts with a percentage of less than 70 percent are considered poor quality and are filtered from the rest of the bursts. For example, Figure A-34 shows the mean signal correlation for HM3 during the first deployment. The significant number of bursts with a value of less than 70 percent is attributed to the intermittent emergence of the instrument platform during low tide. These bursts were excluded during post-processing analysis.

Sensor movement, intermittent or continuous emergence of the sensor near low tide is inferred to cause poor data quality. In post-processing, usable velocity data, velocity ambiguities including spikes greater than ±2 m/sec were removed by iterative linear interpolation between adjacent good data points.

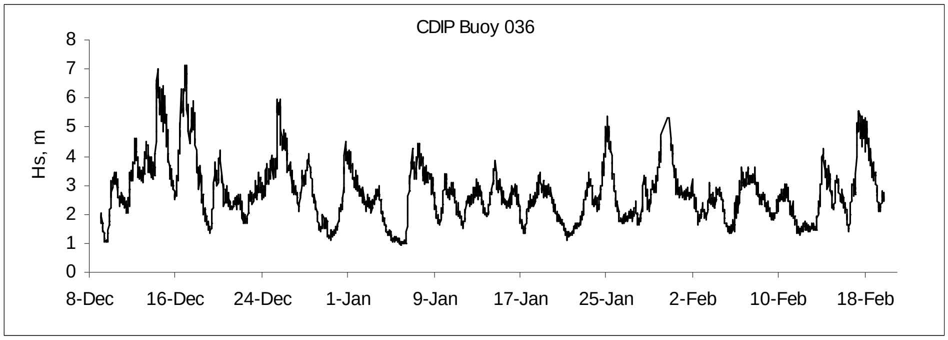
The velocity boundary range represents the approximate distance from the ADVO to the seabed. One should be cautioned that this is not always an accurate portrayal of the seabed level. Migrating sand waves or other objects could distort the true value. In addition, platform movement would also misrepresent the seabed depth. The velocity boundary range figures can offer a rough illustration of whether the seabed is accreting or eroding. HM2 is the only station to show the velocity boundary range because HM3 and HM4 had their ADVO sensors orientated upwards due to deployment in shallower water.

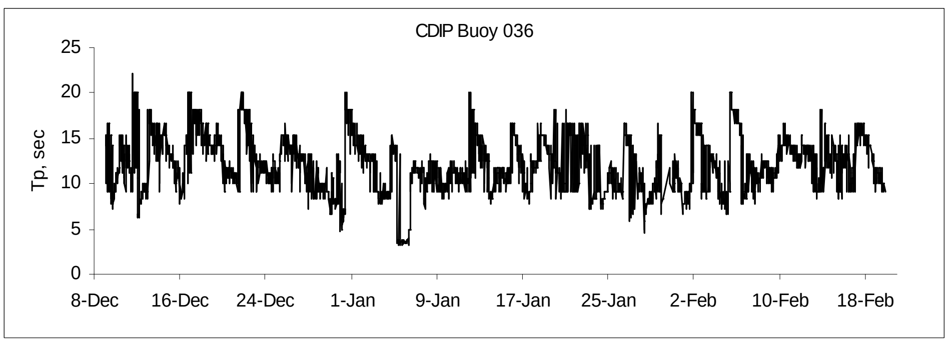
The RDI WavesMon software does not create a header file, however the data was checked for quality and integrity during post-processing analysis.

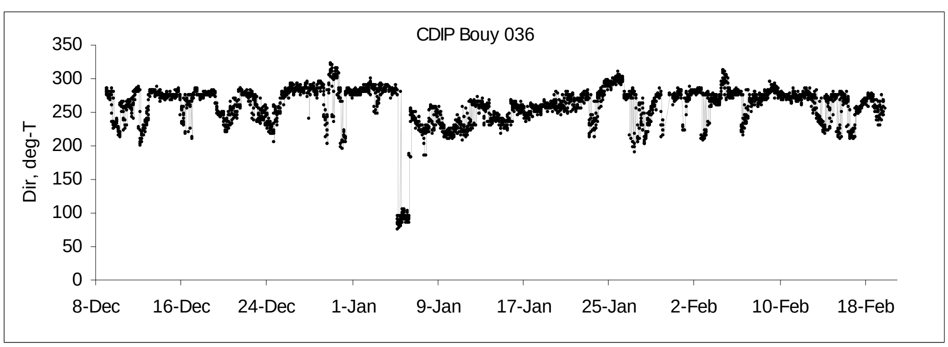
Time Series of Burst-averaged Parameters

Time series were created following the completion of the data processing. Figures A-3 to A-5 show the significant wave height (H_s), peak period (T_p) and direction (D_p) for the same period as deployments 1 and 2 at the Coastal Data Information Program (CDIP) buoy 036 located approximated 5/8 of a mile southwest of the entrance to Grays Harbor, Washington.

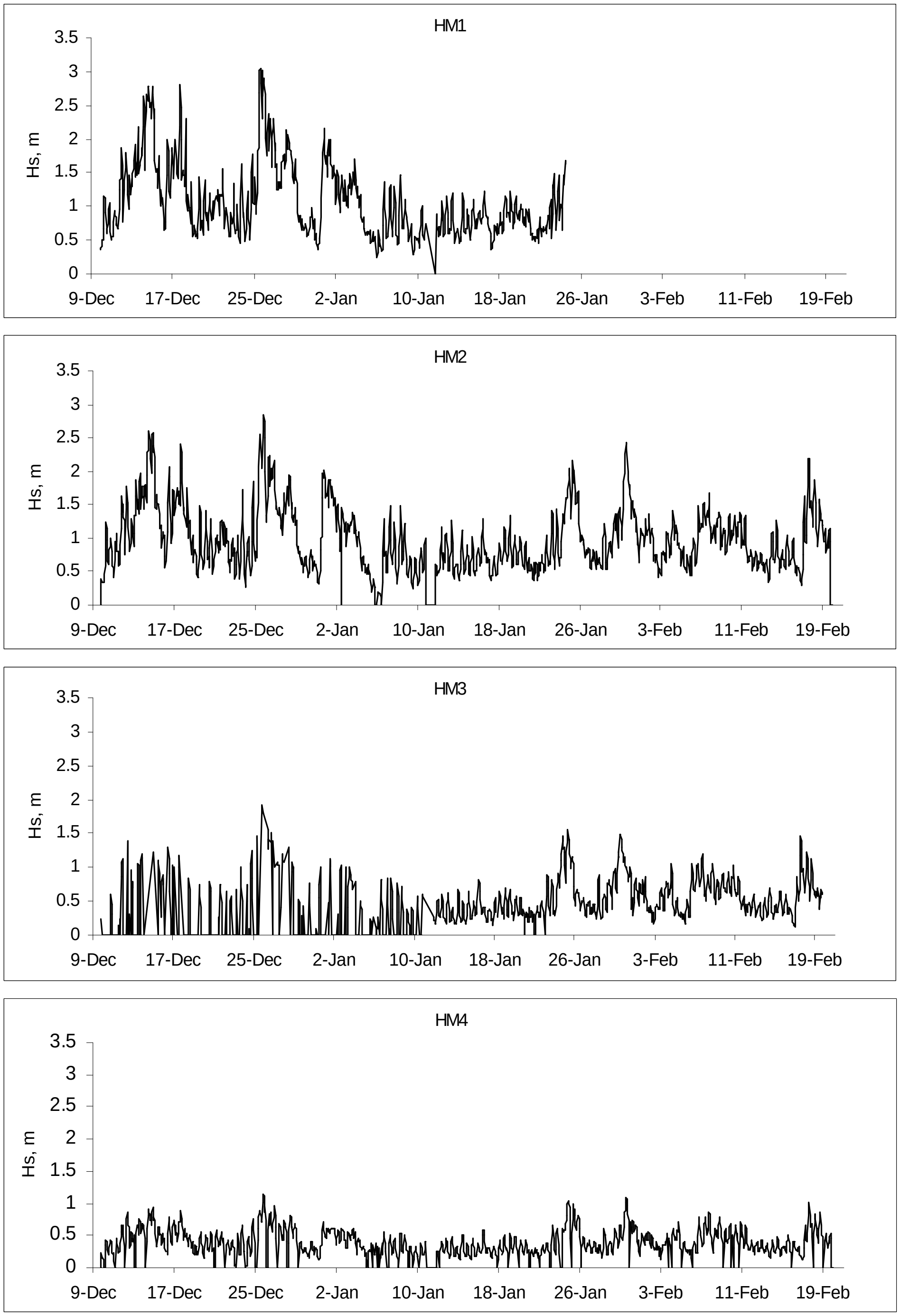
Figures A-6 to A-21 contain plots of H_s, T_p, D_p and depth to sensor for stations HM1 – HM4. Plots of Northing and Easting mean current velocities ($V_n$mean,$V_e$mean) for stations HM3 to HM4 are shown in Figures A-22 and A-23. Time series of suspended sediment concentrations at top and bottom OBS for stations HM2 to HM4 are shown in Figures A-24 to A-29. Measured burst-averaged header parameters for stations HM2 to HM4 subjected to post-processing are provided in Figures A-30 to A-43.



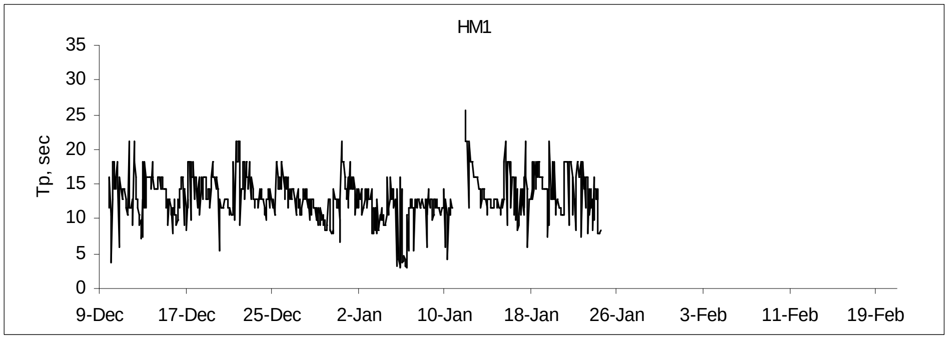


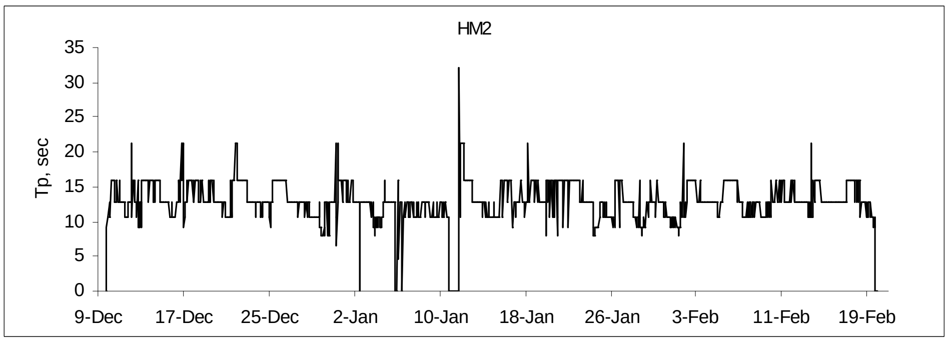


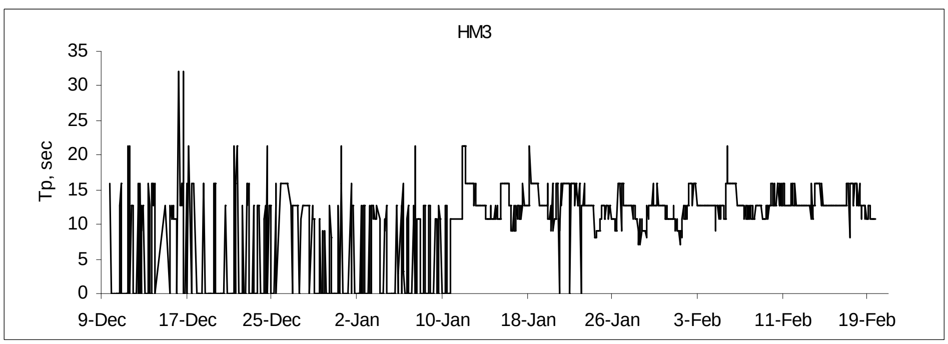
Figures A-3 to A-5. Time series of significant wave height (H_s), peak period (T_p) and dominant direction (D_p) measured at Grays Harbor CDIP Buoy (036) during instrument deployments (9 December 2003 through 19 February 2004)

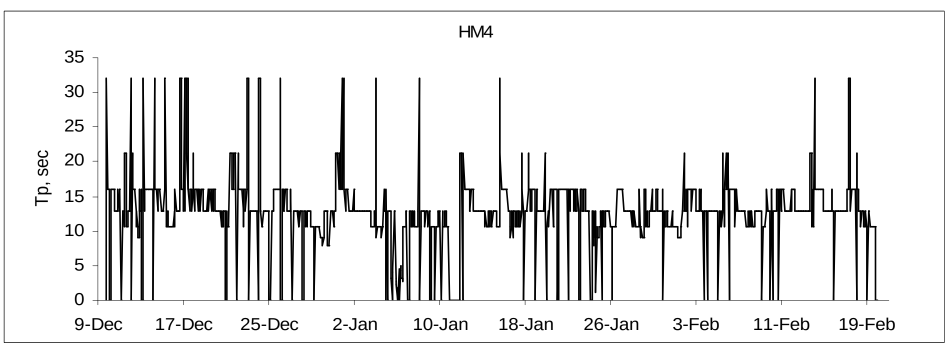


Figures A-6 to A-9. Time series of H_s for stations HM1 to HM4 (9 December 2003 through 19 February 2004)

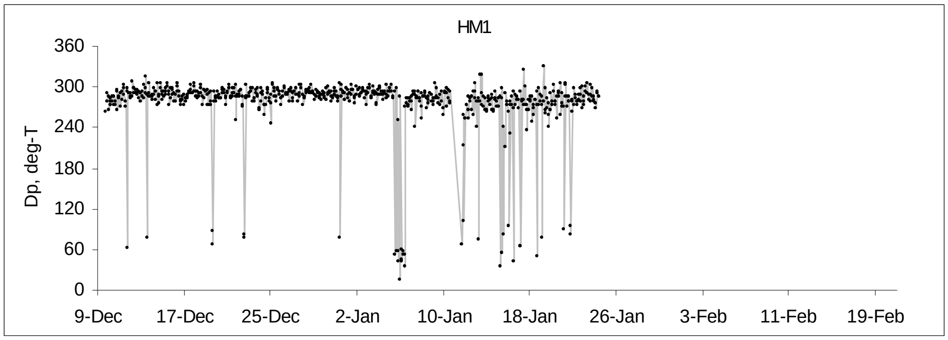


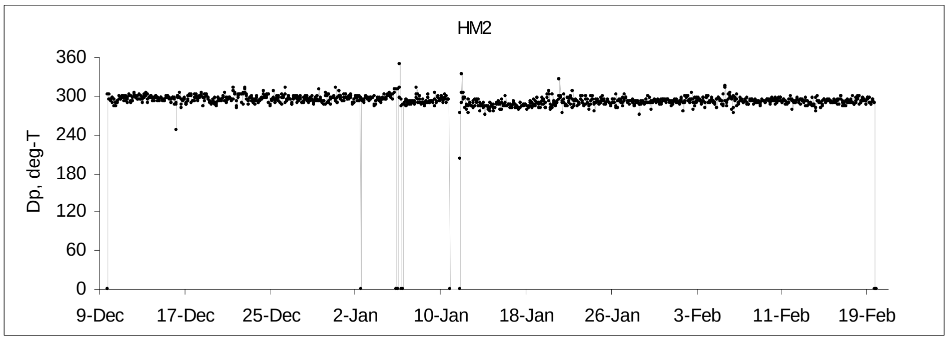


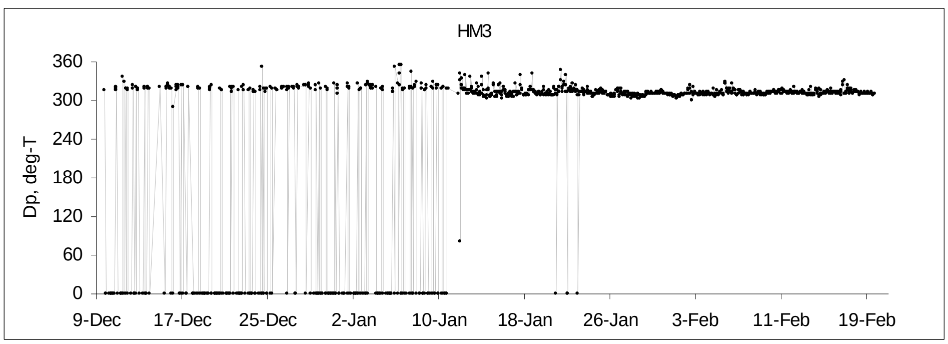


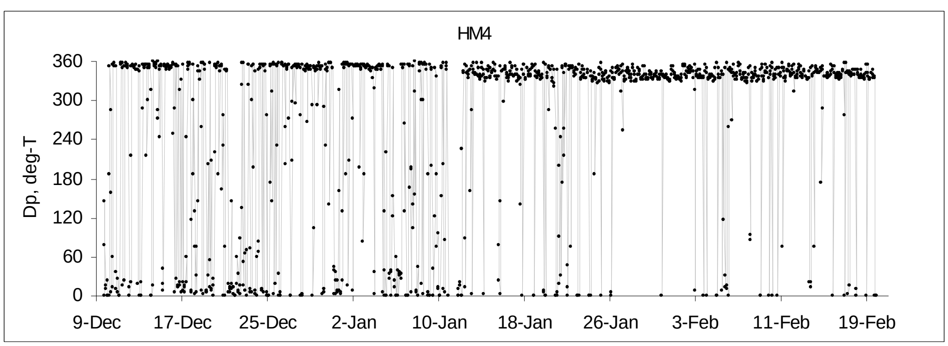


Figures A-10 to A-13. Time series of T_p for stations HM1 to HM4 (9 December 2003 through 19 February 2004)

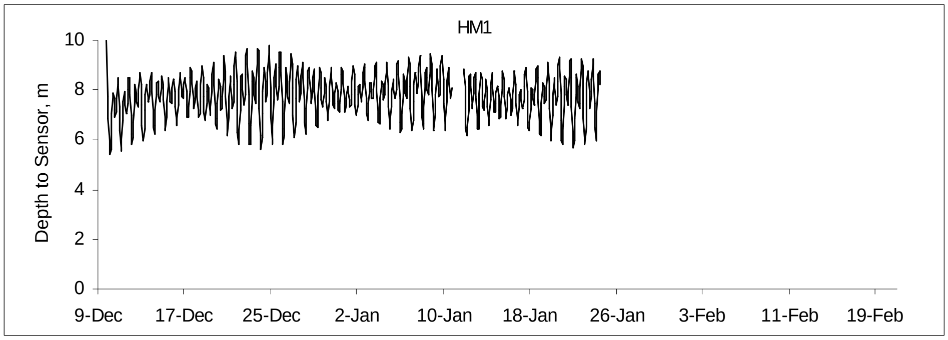


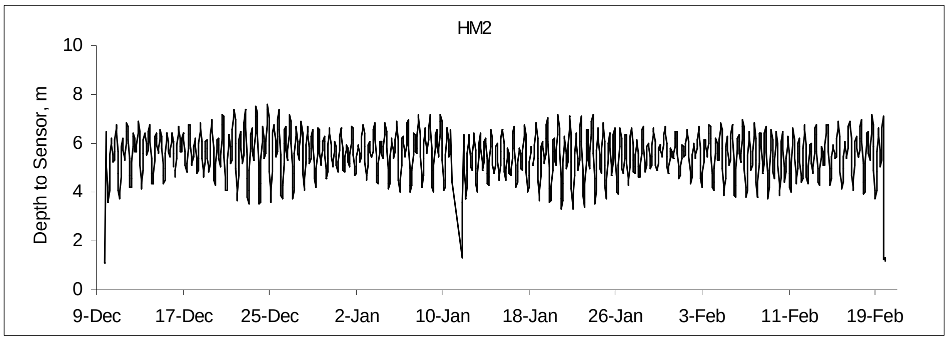


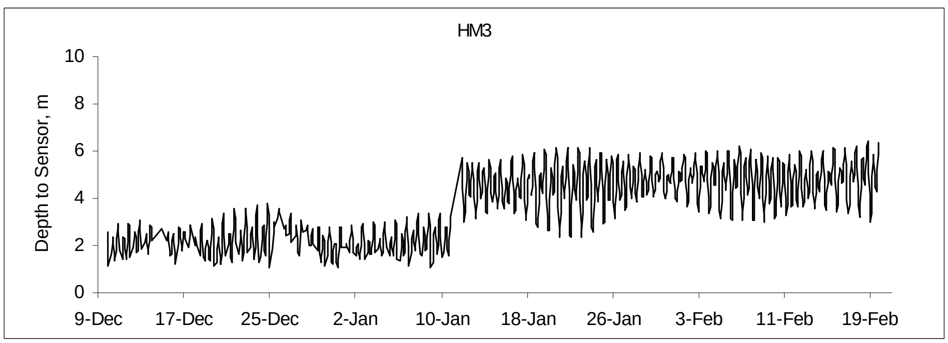


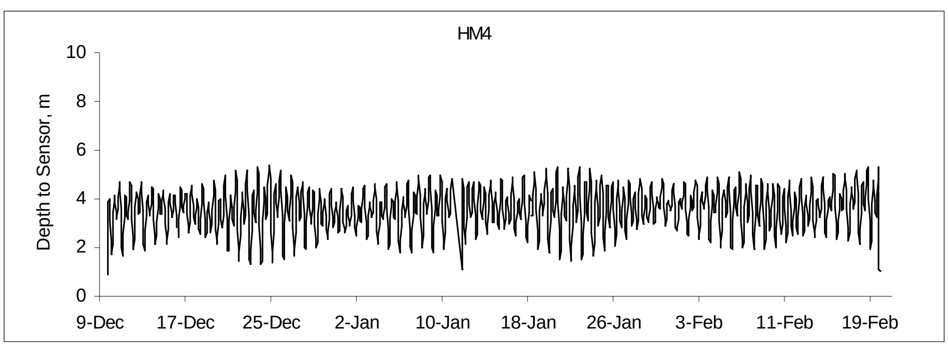


Figures A-14 to A-17. Time series of D_p for stations HM1 to HM4 (9 December 2003 through 19 February 2004)

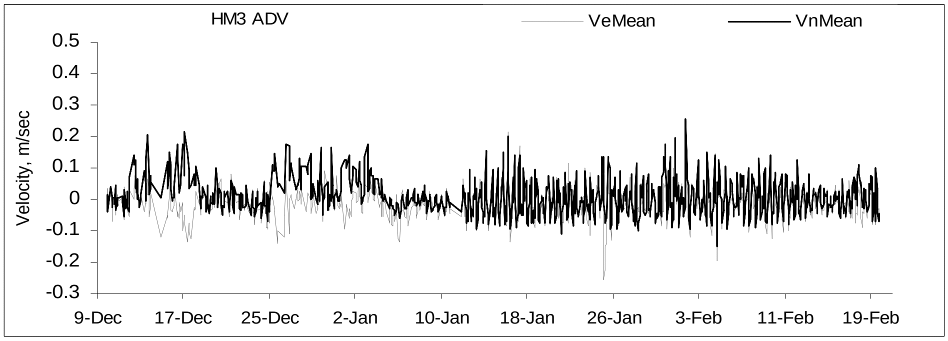


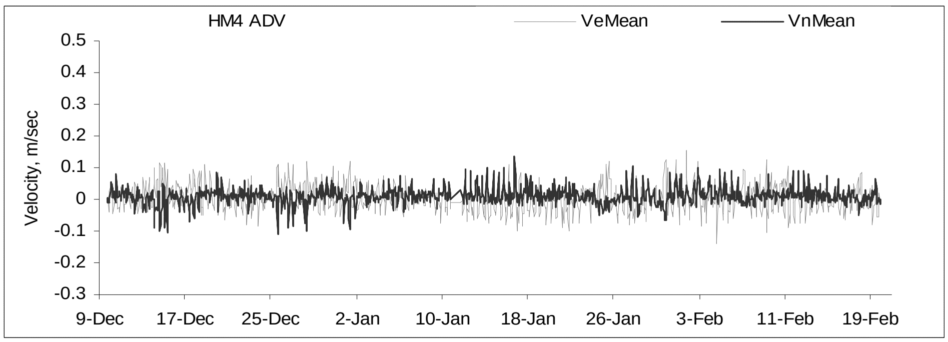




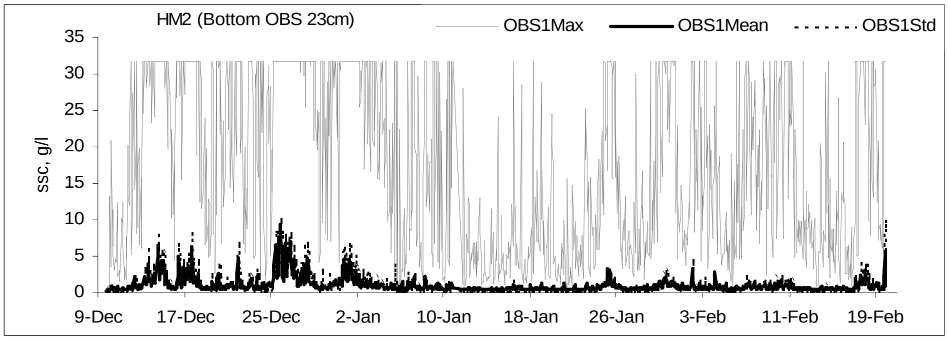


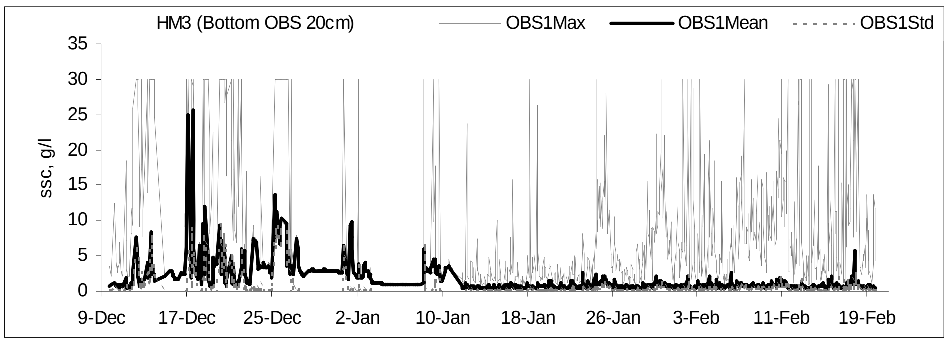
Figures A-18 to A-21. Time series of water depth to sensor for stations HM1 to HM4 (9 December 2003 through 19 February 2004)

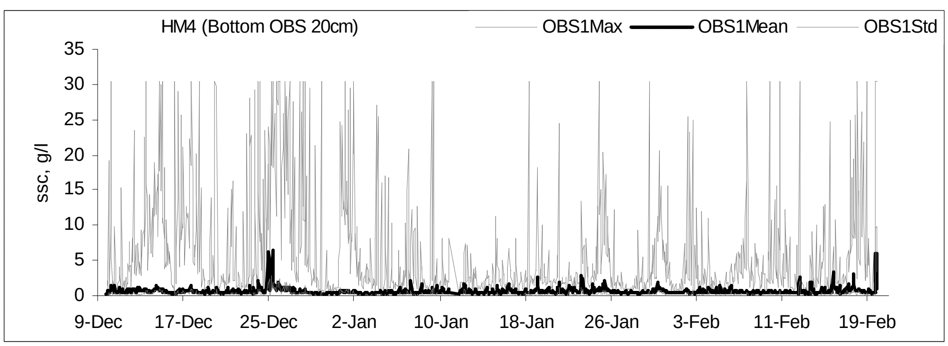




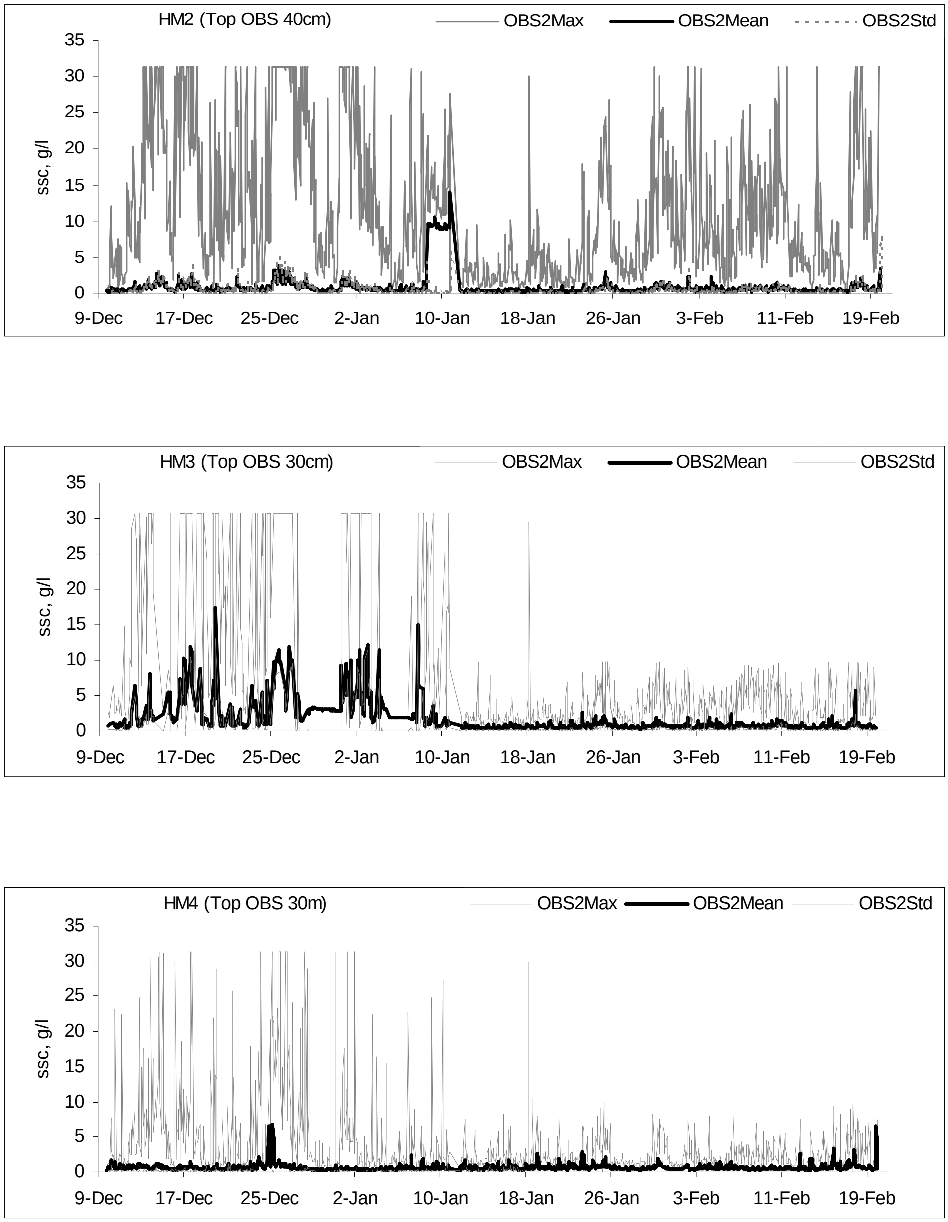
Figures A-22 to A-23. Time series of Northing and Easting mean current velocity (Ve mean, Vn mean) for stations HM3 and HM4 (9 December 2003 through 19 February 2004)



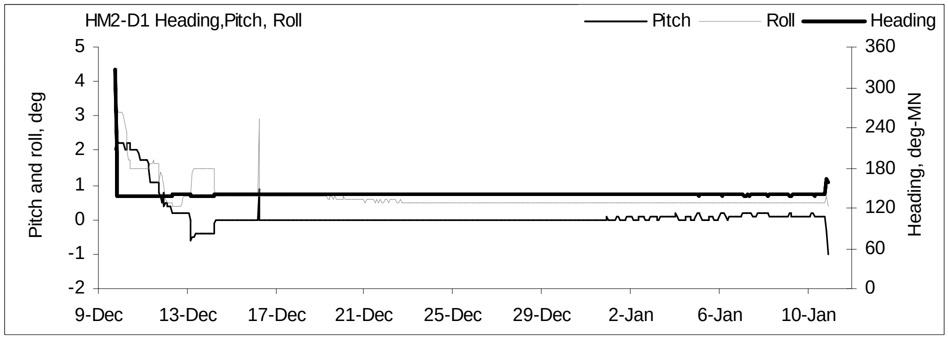


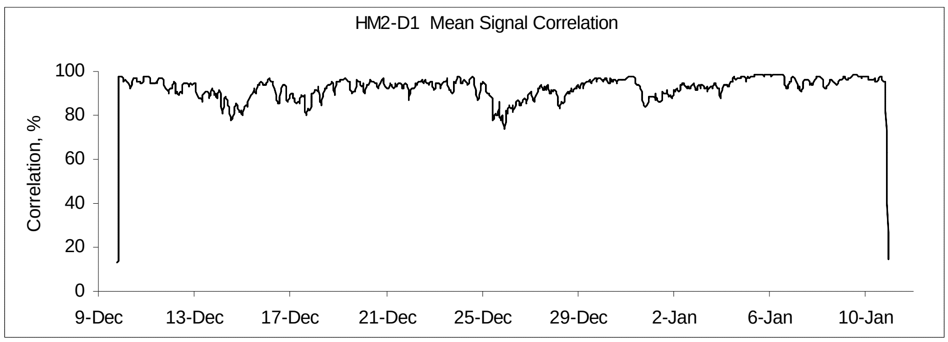


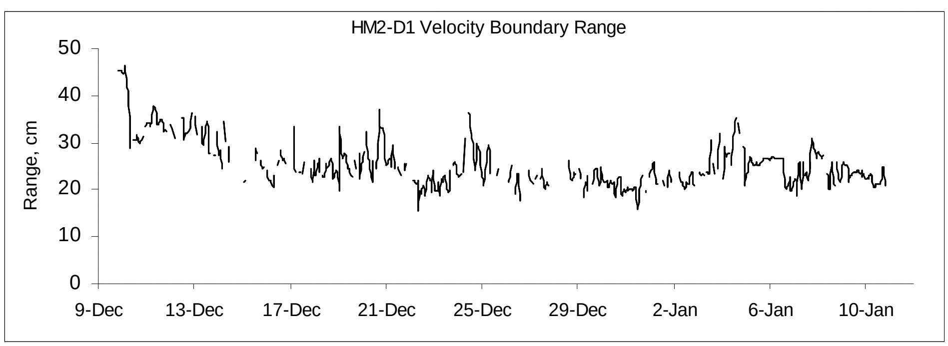
Figures A-24 to A-26. Time series of SSC at bottom OBS for stations HM2 to HM4 (9 December 2003 through 19 February 2004)



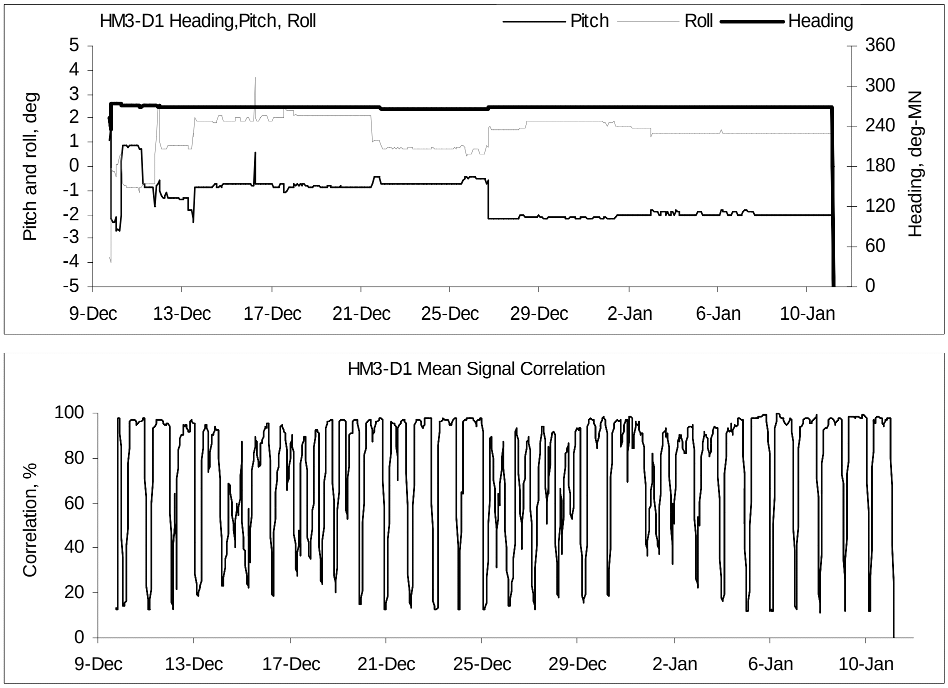
Figures A-27 to A-29. Time series of SSC at top OBS for stations HM2 to HM4 (9 December 2003 through 19 February 2004)



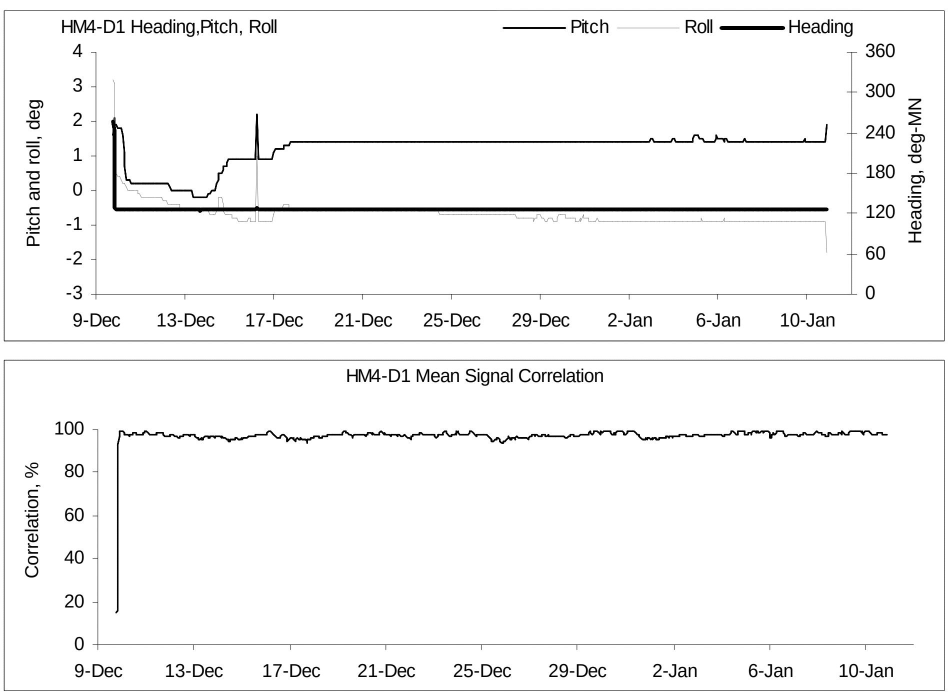




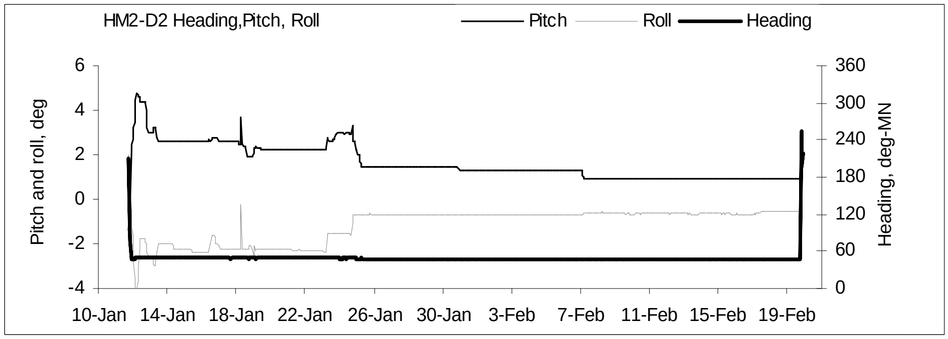
Figures A-30 to A-32. ADVO data quality parameters for station HM2 during Deployment 1 (9 December 2003 through 19 February 2004)

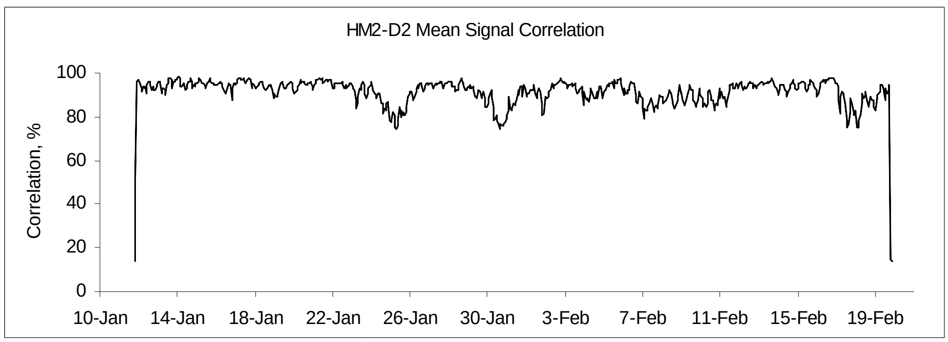


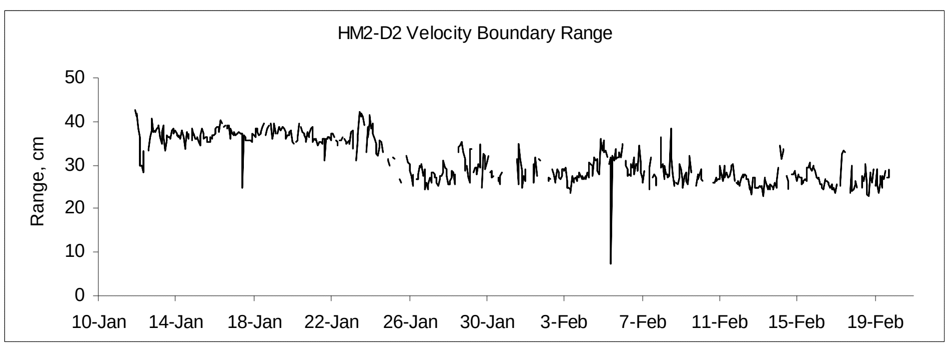
Figures A-33 to A-34. ADVO data quality parameters for station HM3 during Deployment 1 (9 December 2003 through 19 February 2004)



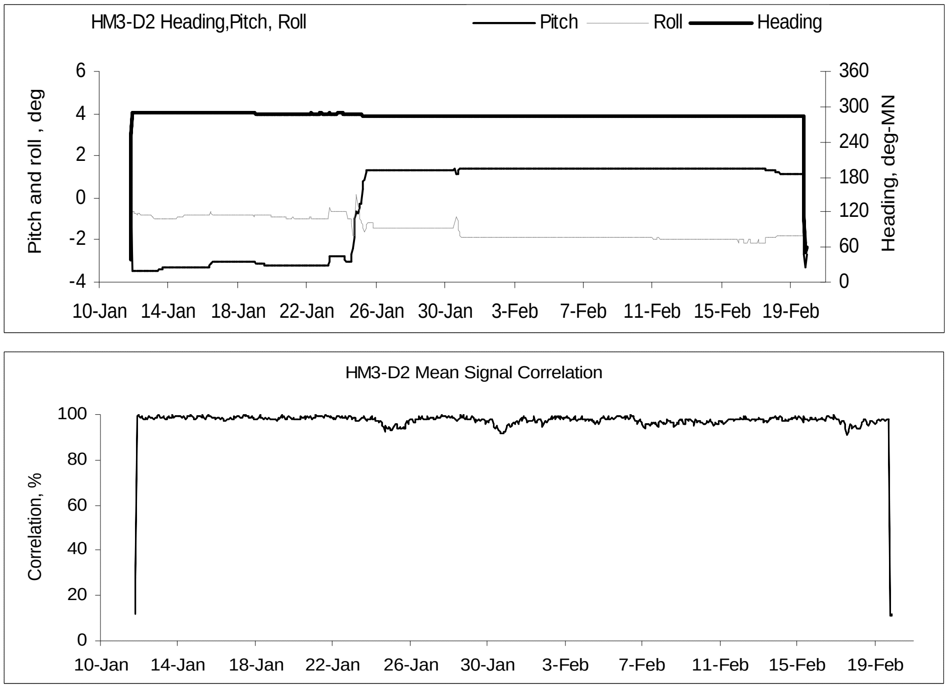
Figures A-35 to A-36. ADVO data quality parameters for station HM4 during Deployment 1 (9 December 2003 through 19 February 2004)



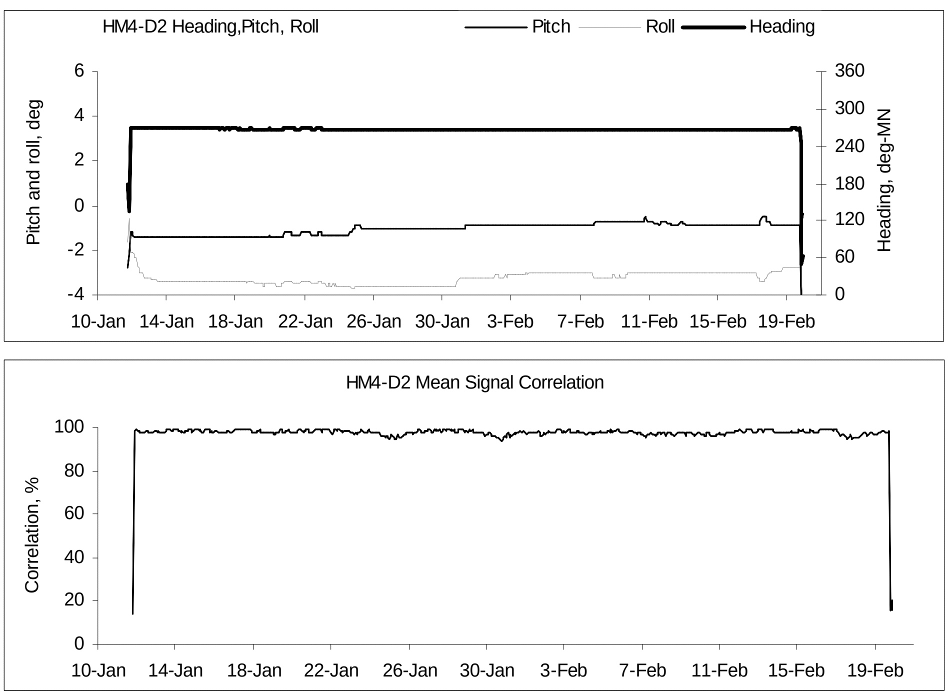




Figures A-37 to A-39. ADVO data quality parameters for station HM2 during Deployment 2 (11 January 2003 through 19 February 2004)



Figures A-40 to A-41. ADVO data quality parameters for station HM3 during Deployment 2 (11 January 2004 through 19 February 2004)



Figures A-42 to A-43. ADVO data quality parameters for station HM4 during Deployment 2 (11 January 2004 through 19 February 2004)

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Appendix B - Sampling and Analysis of Gravel and Cobble Sediments in Half Moon Bay

Introduction

This appendix provides background information on the collection and analysis of gravel and cobble samples in Half Moon Bay, Grays Harbor, Washington as part of the Grays Harbor South Jetty Performance Study. Pacific International Engineering, PLLC conducted field measurements, data processing, and analysis for the Engineer Research and Development Center – Coastal Hydraulics Laboratory (ERDC-CHL) under its Broad Agency Announcement contract DACW42-01-C-0011.

Bed Material Characterization and Tracer Grain Preparation

Size distributions of the transition gravel and cobble beach sediments were characterized by sampling the surface and subsurface sediments. Surface samples were acquired to obtain tracer particles and to examine any horizontal variations in size and shape characteristics on the transition beach. Subsurface sampling was conducted to examine any vertical variations in bed material composition. Because of the large number and volume of samples required for characterization of gravels and cobbles, sieve analysis was not practical for all surface samples of the material.

The design specification for the gravel and cobble transition beach is shown in Table B-1. The size distribution of a surface sample (N=414) of gravel and cobble from the transition beach is shown as a histogram and cumulative frequency curve in Figure B-1.

Surface gravel and cobble material were sampled using the grid-by-number method (Wolman, 1954). The grid-by-number method is the most widely used sampling technique for coarse bed surfaces (e.g. Rice and Church, 1996) and involves the determination of the relative area covered by grain sizes rather than their relative weight. The method involves establishing a grid on the beach surface and sampling the grains lying directly beneath each grid point. Distance between successive samples is no less than three times the diameter of the largest particles and individual grains are characterized by the length of the intermediate axis. Precision of percentile estimates using the Wolman procedure is achieved at the expense of a large sample size. Rice and Church (1996) showed that once sample sizes reach 400 particles the appropriate 95 percent confidence limits of +/-0.1 ϕ are reached.

The size distribution of the surface samples was determined by measurement of the long, intermediate, and short axes of the particles. The size distribution of the surface sample was divided into six size fractions based approximately on the Udden-Wentworth size classification scale and the ϕ size of the particles, where $f = -\log_2(DI_{mm})$, and DI_{mm} is the length of the intermediate axis in millimeters.

Five particles from each of six size classes were sampled at random from the overall surface sediment sample. Small but powerful magnets were attached to each tracer particle with an epoxy and a label for grain identification. Each particle was coated with epoxy paint that was color coded by size fraction for easier identification of tracer particles on the beach. The magnetic tracer particles are shown in Figure B-2.

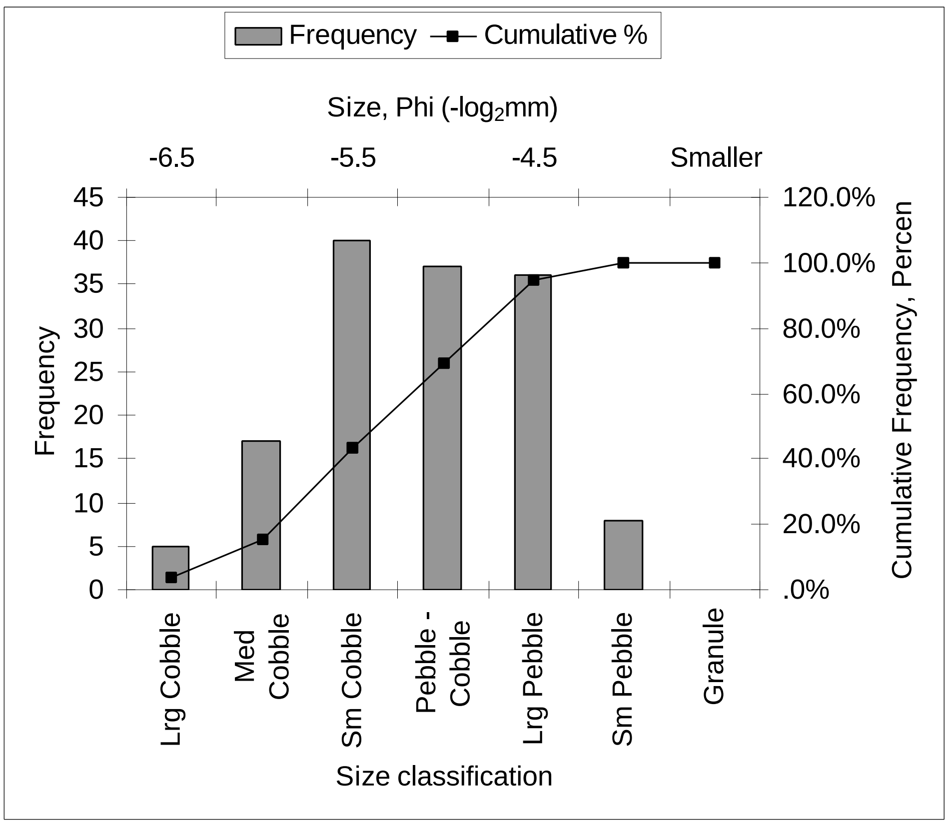

Figure B-1. Size distribution of a surface sample (N=414) of gravel and cobble from the transition beach (August 2003). Size is based on the length of the intermediate axis

Table B-1 Size distribution of the gravel and cobble for the transition beach	
US Standard Sieve size mm (inches)	**Percent Passing by Weight**
304.8 (12)	100
152.4 (6)	85-100
76.2 (3)	50-85
19.1 (3/4)	0-40
9.5 (3/8)	0-6
0.074 (0.003)	0-3

Figure B-2. Photo showing magnetized tracer particles prior to deployment

Surface and subsurface bed material was also characterized by volumetric bulk sampling analysis using field sieving for the coarse cobble fraction and standard laboratory sieving for the finer fraction. Sediment samples were acquired from the transition beach in June 2002 for analysis. Shore material gradation was determined at a series of transects coinciding with regular beach profile monitoring surveys (Figure 2-8 in Chapter 2). The gradation of material at transects HMB2, HMB3, and HMB4 in the transition beach region are listed in Table B-2. The proportion of sand varies with distance below the surface and also with position on the profile. At lower elevations on the beach, the higher percentage of sand results in a lower beach slope. Higher on the beach profile, there is a higher proportion of gravel and the beach steepens.

Table B-2 Gradation of Shore Material in Western End of Half Moon Bay					
Transect	**Source**	**Elevation (ft)**	**Gravel (%)**	**Sand (%)**	**Fines (%)**
2	Surface	3	33.9	65.9	0.2
2	Subsurface	3	21.4	77.8	0.8
3	Surface	3	5.2	93.3	1.5
3	Subsurface	3	0.2	98.8	1
4	Surface	3	13.1	86.6	0.3
4	Subsurface	3	2.4	97.1	0.5
Note: Date of sampling is 26 June 2002					

Cobble Transport Measurements

Direct field measurements of the transport of pebble and cobble beach sediments have been obtained through particle tracing experiments using painted, magnetized, or tagged particles. Sediment tracing experiments using painted particles often have relatively poor recovery rates (much less than 50 percent) that limit the value of quantitative information that can be derived from the effort. Recently, the development of methods based on magnetized particles has allowed the motion of pebble and cobble particles to be studied with more precision (e.g. Hassan and Church, 1992; Nicholls and Webber, 1987; Voulgaris et al., 1999). A metal detector can be used to locate individual grains resulting in a higher recovery rate. In this study, the tracer technique involved a combination of magnetic tracer particles and surveying with Real Time Kinematic Global Positioning System (RTK-GPS).

The tracer experiments involved three sets of magnetic particle tracer deployments on 17 December 2003 and 9 February 2004. The first set (Set 1) was deployed on 17 December 2003 and re-surveyed during the two successive high tides between 17 December and 19 December 2003. Sets 2 and 3 were deployed on 9 February 2004 and measured during the successive high tides between 9 February and 13 February 2004.

Tracer Grain Placement

A series of six approximately shore normal transects were established at approximately 2 m alongshore intervals within the upper inter-tidal zone of the transition beach for the placement of tracer particles. The five particles from each size class were placed along each of the six shore-normal transects at elevations between +2 ft mllw and +6 ft mllw at intervals of approximately 1 ft vertical. At each position, a particle of approximately equal size was removed from the bed and replaced with the tracer particle, with care to minimize bed disturbance. Initial location of each particle was surveyed using a Real Time Kinematic Global Positioning (RTK-GPS) survey system (Figure B-3).

Figure 2-22 (Chapter 2) shows the placement location of tracer particles during the two tracer deployments on 17 December 2003 and 9 February 2004. The rectangles represent the matrix outlines used to situate the particles at their initial positions. Each particle was placed an equal distance apart in a 6.1 m (20 ft) by 7.3 m (24 ft) approximately sized matrix. During the first deployment, a single set (Set 1) of 30 tracer particles (five particles in six size classes) was deployed. During the second deployment, two sets (Sets 2 and 3) of 30 tracer particles were deployed.

Figure B-3. Surveying the initial placement of tracer particles with an RTK-GPS

Tracer Recovery

The tracer particles were re-located visually and with a magnetic detector after each diurnal tidal cycle (Figure B-4). Each particle was positively identified by its painted color and unique numeric label. The particle depth below surface was recorded with a graduated scale. Particle positions were determined by surveying with RTK-GPS.

Upon re-location and identification, the particle was lifted to the surface at the new location and left on the surface for the next cycle. Beach profiles were measured using RTK-GPS to record significant changes in beach profile shape and volume displacements of sediment.

Two tracer particles were not recovered during the trials; recovery was 93 percent.

Figure B-4. Relocation of tracer particles using a magnetic detector, RTK-GPS survey system. Depth below the sand surface was also recorded in cases where particles were buried

Cobble Size and Shape Analysis

Samples of the cobble in the gravel/cobble transition were obtained on 29 May 2003 for the purpose of examining the horizontal variation in cobble size and shape in the transition beach area and along Half Moon Bay beach.

Cobbles were sampled at approximately the high water mark at three locations around Half Moon Bay coinciding approximately with cross-sections HMB1, HMB3, and HMB5. Note that HMB1 is 146 ft along the shoreline from the diffraction mound at the terminus of South Jetty, HMB3 is 771 ft from the mound, and HMB5 is 1,540 ft from the mound. HMB5 is outside the area of placement of the original transition cobble, although it is possible that the cobbles at HMB5 were transported along the beach from the transition beach - anecdotal evidence (e.g. Nelson, pers. comm. 2003; Chapman, pers. comm. 2003) suggests that large quantities of cobble may have been present near the east end of the South Jetty and that these cobbles may have been redistributed from west to east in Half Moon Bay at the time of the breach in 1994. Erosion of the beach fill and redistribution of the cobble since that time may have re-exposed these older deposits. It is unknown whether these older cobble deposits are from construction of the South Jetty around 1900 or are native material.

Cobbles were sampled by selecting each of the surface particles along a number of randomly selected 2 ft transects at each cross-section. A total of 37 particles were measured at HMB1, 53 particles at HMB3, and 56 particles at HMB5. The length of each principal orthogonal axis (L = long, I = intermediate, and S = short) was measured with a scale. Sphericity of the particles is determined according to the method of Sneed and Folk (1958). Maximum-projection sphericity, ψ, according to Sneed and Folk (1958) is calculated as:

$$y = \left(\frac{S^2}{IL}\right)^{0.33}$$

Sphericity indicates how closely a particle approximates a sphere by calculating the ratio between the maximum-projection area of the particle to that of a perfect sphere.

Table B-3 is a statistical summary of the cobble sample dimensions and sphericity at each transect. The summary indicates that there is a significant increase in the average size of the cobbles from HMB1 to HMB5, as indicated by the 95 percent confidence intervals about the mean. In contrast, there is a progressive decrease in ψ from HMB1 to HMB5 indicating that the particles are less spherical as well as larger with distance eastward from the diffraction mound. This corresponds with the visual observation that particles become flatter and more disk-shaped between HMB1 and HMB5. Note that the standard deviation, an indicator of the relative sorting, increases from HMB1 to HMB5 indicating that particles are better sorted at HMB1 than at HMB5. Figure B-5 shows the relationship between particle size, represented by the intermediate axial length, and ψ for the three cross-sections.

Table B-3
Statistical Summary of Cobble Measurements at Half Moon Bay

		L (mm)	I (mm)	S (mm)	y	L/S
HMB1	Mean	40.35	28.81	17.51	0.64	2.37
	Median	40	28	17	0.64	2.32
	Standard Deviation	8.35	6.77	3.31	0.11	0.60
	Count	37	37	37	37	37
	95 % CI	2.69	2.18	1.07	0.04	0.19
HMB3	Mean	62.92	46.68	26.47	0.62	2.61
	Median	61	44	26	0.62	2.50
	Standard Deviation	12.99	12.42	9.10	0.14	0.91
	Count	53	53	53	53	53
	95 % CI	3.50	3.34	2.45	0.04	0.24
HMB5	Mean	92.09	70.71	35.13	0.57	2.77
	Median	89	67	33	0.57	2.70
	Standard Deviation	25.07	21.63	11.70	0.12	0.83
	Count	56	56	56	56	56
	95 % CI	6.57	5.66	3.06	0.03	0.22

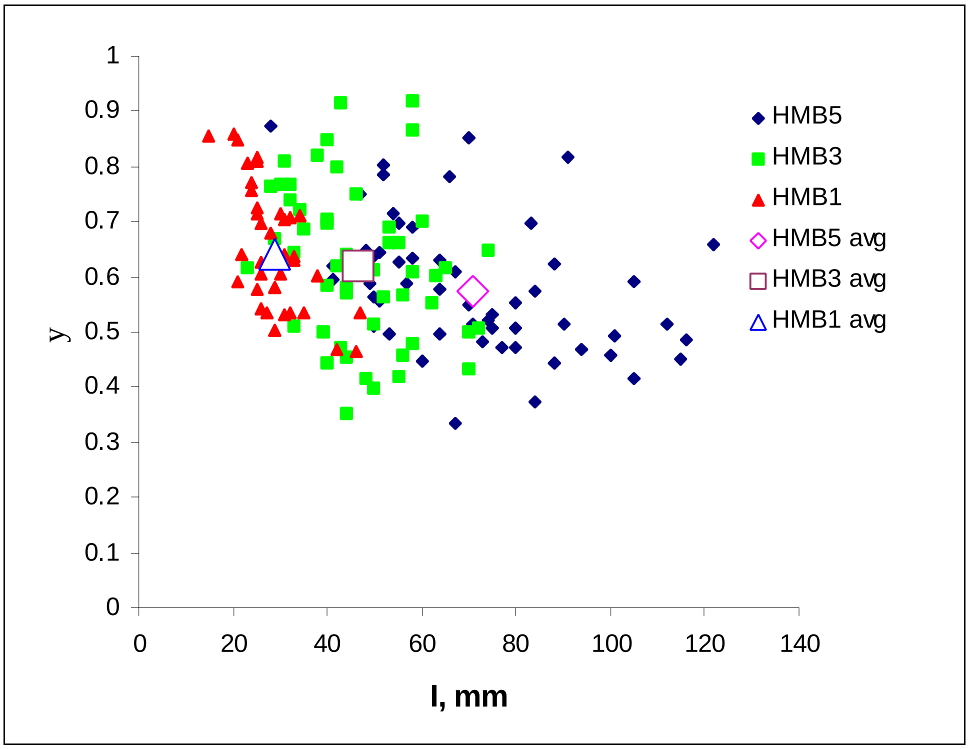

Figure B-5. Particle sphericity as a function of intermediate axial length

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Appendix C - Description of 1994 Alternatives

This Appendix provides a summary of eight alternatives identified in a report by Dean et al (1994) for the Seattle District. The alternatives were intended to provide a solution to prevent breach recurrence.

Beach Revetment

This alternative consists of a conventional multi-layer revetment to armor the shoreline at South Beach and Half Moon Bay. The original design called for a 5,000 ft revetment on South Beach and 2,500 ft on the Half Moon Bay shoreline (Figure C-1). The revetment cross-section extends from elevation +25 ft mllw to below mllw. The revetment has an outer armor layer of 2 to 12 ton concrete DOLOS units and a rock underlayer and toe protection. It is likely that a stone armor layer would be a more efficient design solution than concrete DOLOS units and that a revetment entirely above mllw would be adequate. If concrete armor units were to be considered now, a single-layer armor such as CORE-LOC would be preferable to DOLOS units. The cross-section should be modified in a number of ways, including adding a geo-textile and improving the toe design. This alternative would result in a heavily armored shoreline and would likely create a severe disruption of nearshore sediments. It is unlikely that exposed rock revetments would be permitted at this location. The cost for construction is also likely to be prohibitive.

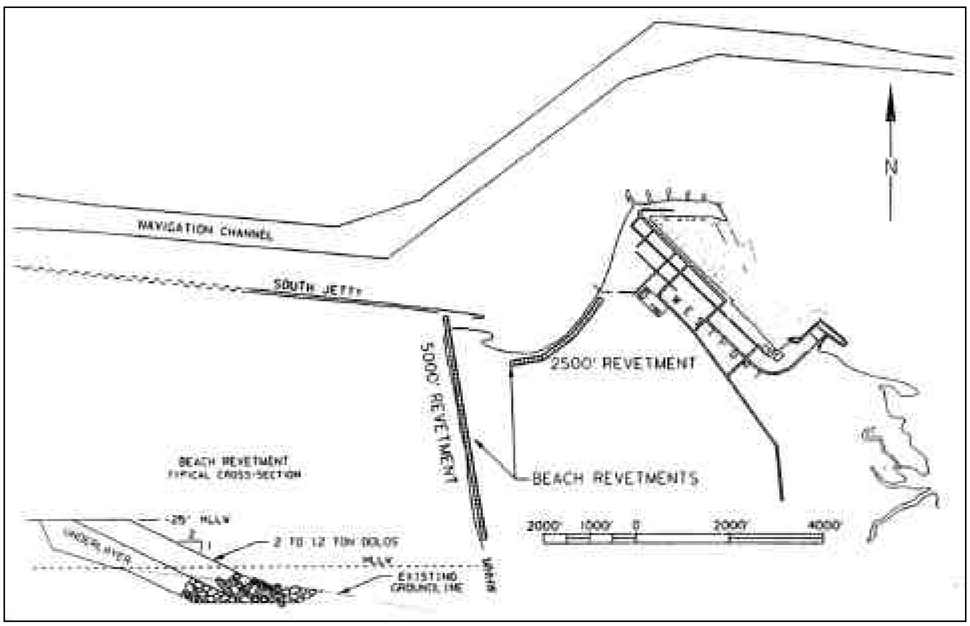

Figure C-1. Beach revetment

Revetment and Jetty Extension

This alternative consists of extending the South Jetty east to connect to Point Chehalis (Figure C-2). This plan ensures that no further breach will occur and that the entrance channel cannot migrate south to a breach and possibly "jump the jetty". However, no shore protection for South Beach is included. Also, it will be difficult to obtain agency approval and permits for any large expanse of new rock revetment.

This alternative would likely require fill south of the jetty extension to prevent Half Moon Bay becoming a lagoon of stagnant water. Otherwise, a gap could be included in the jetty extension to allow tidal circulation in Half Moon Bay. It is possible that a constructed wetland with a dendritic drainage pattern would connect to the gap in the jetty extension. Erosion on the ocean side of South Beach could possibly be managed or eliminated through a program of beach nourishment and/or nearshore berm sediment placement.

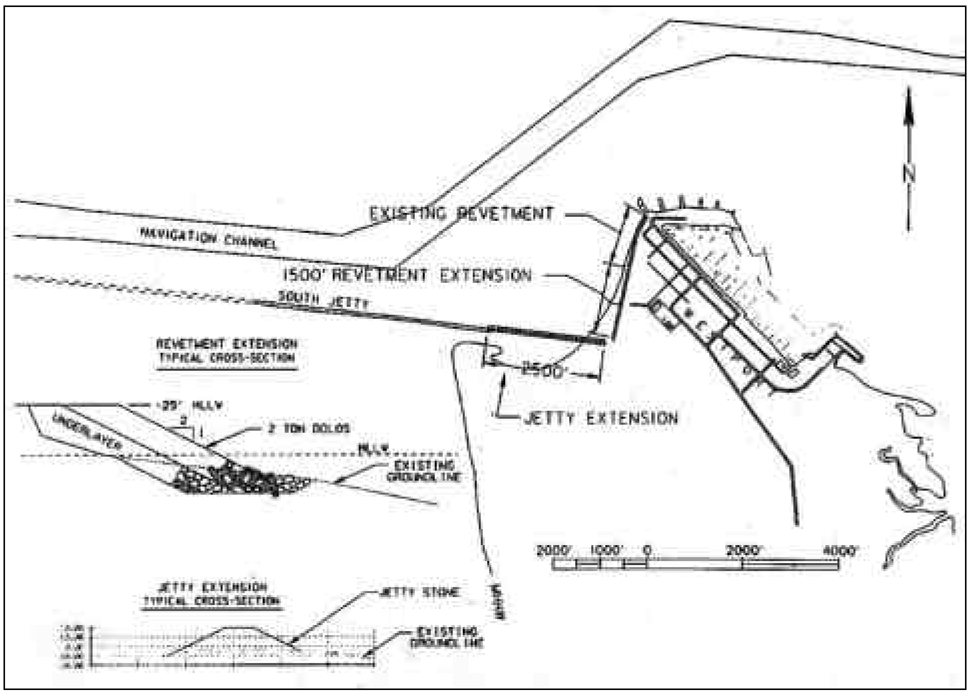

Figure C-2. Revetment and jetty extension

South Jetty Spur Groin

This alternative consists of an emergent spur placed perpendicular to the South Jetty (Figure C-3) and extending approximately 2,000 ft to the south of the jetty. The function of the spur groin would be to disrupt the seaward current flowing along the south side of the jetty, and thereby reduce the seaward transport of sediment and consequent erosion of South Beach. Further work would be necessary to determine the mechanism and patterns of currents and sediment transport at South Beach. It is not certain that a seaward current along the jetty is the cause of South Beach erosion. Also, a spur jetty does not address the problem of erosion within Half Moon Bay.

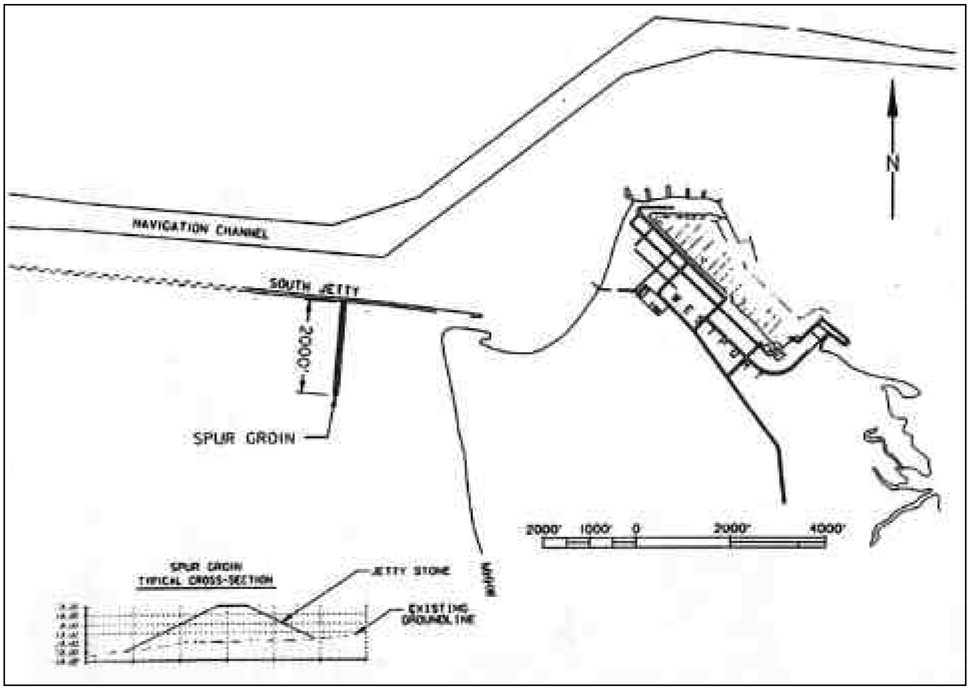

Figure C-3. South Jetty spur groin

Reinforcement of the Jetty

This alternative would allow a breach to continue to exist and evolve while reinforcing the jetty to withstand the increased wave and current forces resulting from it being more exposed (Figure C-4). The South Jetty would become an "island". There is a risk that the breach would continue to grow, the shipping channel would migrate further south parallel to the South Jetty, and eventually the channel may jump the jetty. Breaching would also increase exposure of back-bay areas to ocean waves. This alternative was considered unacceptable by the City of Westport and the Port of Grays Harbor in 1999.

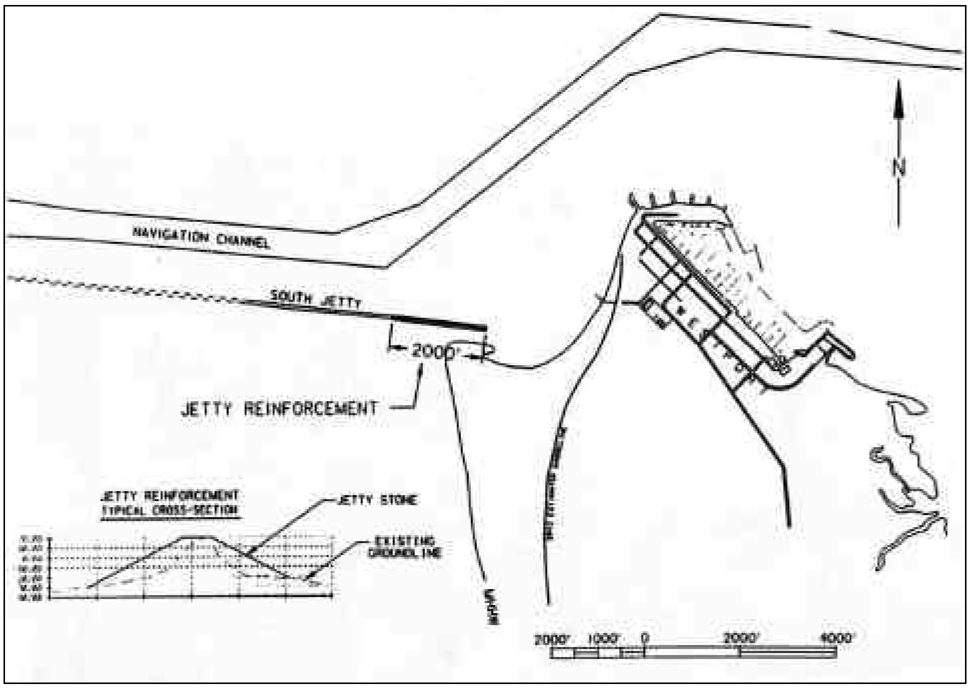

Figure C-4. Reinforcement of the jetty

Nearshore Berms

This alternative consists of placing dredged sediment in submerged berms in Half Moon Bay and near South Beach (Figure C-5). The extra cost for nearshore berm placement is relatively small compared to offshore disposal and in some circumstances may cost less. Nearshore berms have been applied both before and after the 1993 breach. During September and August 1993, a 2,000 ft long berm containing 385,000 cu yd of sediment was placed at the 40 ft contour at the South Beach. In 1992, 185,000 cu yd was placed in Half Moon Bay and another 200,000 cu yd was placed there in 1994.

Nearshore berms require placement of sediment within profile closure depth to provide effective nourishment of beach. Sediment placed in berms outside the closure depth does not migrate shoreward to nourish the beach. Hopper dredges and barges have a draft that precludes placement close to shore. Placement by hydraulic pipeline is more expensive, on the order of $5 more per cu yd, than simply placing sediment offshore from hopper dredges. Placement of sand directly on the beach with a hydraulic pipeline is probably preferred to placement in a submerged berm.

It is not known whether the nearshore berm placement that has been applied since 1994 has been effective at reducing the erosion at South Beach and Half Moon Bay. Nearshore berm placement by itself is unlikely to be a solution to the erosion problems at Half Moon Bay, considering that a significant portion of the erosion at Half Moon Bay is concentrated at the upper part of the beach profile and coincides with storms that elevate wave and water levels. A program of regular nearshore berm placement at South Beach may be effective at slowing or stopping erosion from the ocean side of South Beach.

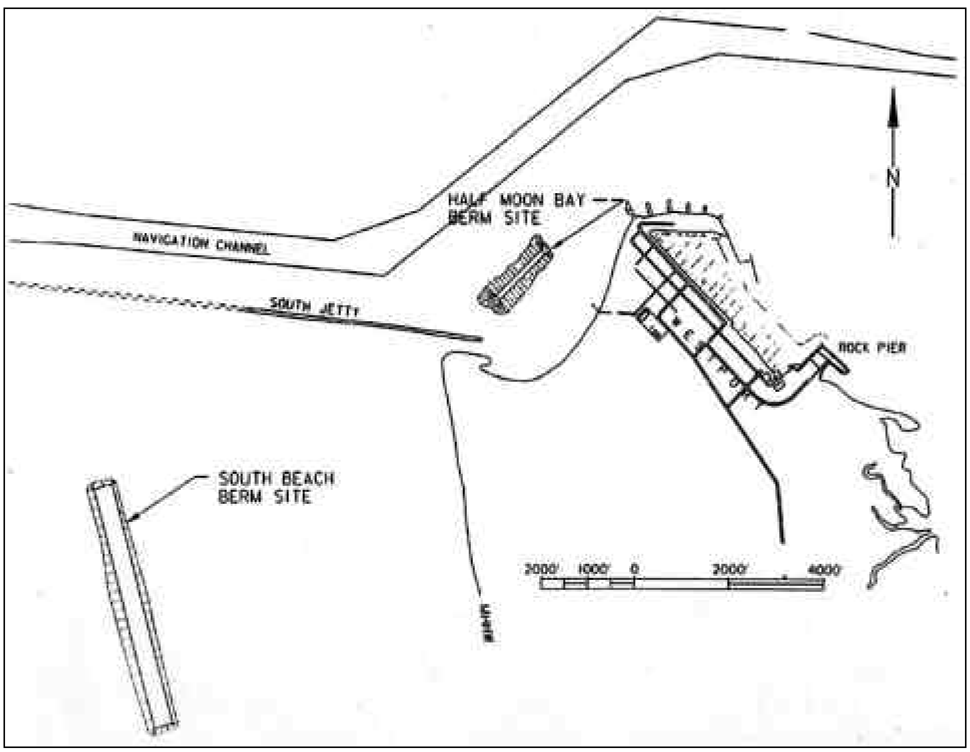

Figure C-5. Nearshore berms

Direct Beach Nourishment

Placement of sand directly in the breach area has been applied a number of times since the breach in 1993. Approximately 500,000 cu yd of suitable sediment is dredged annually from the Entrance and Bar Channels of the Grays Harbor Navigation Project. This sand can be placed by hydraulic pipeline in the project area at an incremental cost, of approximately $5/cu yd, compared to regular offshore disposal. Beach nourishment requires a program of regular re-nourishment in order to be an effective long-term solution. The direct re-nourishment concept is illustrated in Figure C-6.

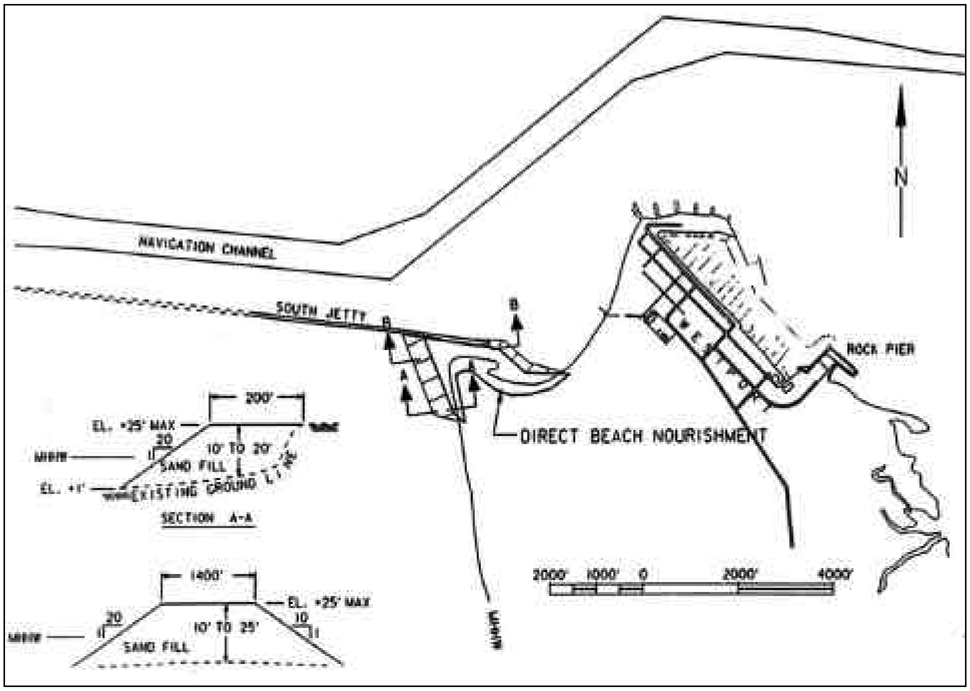

Figure C-6. Direct beach nourishment

Relocate Entrance Channel

Relocation of the entrance channel midway between the jetties has been proposed to reduce the exposure of Half Moon Bay and Point Chehalis to ocean waves. Further study is needed to determine the optimum alignment of a relocated channel and the effect this would have on waves, currents, and sediment transport. Additional structures and sediment placement may be necessary to control further migration of the channel. Relocation of the channel may have some benefit in the long term (20 years), but is unlikely to be a short to medium term solution or to provide complete solution to the erosion problems at Half Moon Bay.

Relocate Bar Channel

In addition to the entrance channel, relocation of the bar channel has been considered. A new bar channel location should be studied as part of any entrance channel relocation study.

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