Engineer Research & Development Center / San Francisco District
U. S. Army Corps of Engineers

US Army Corps of Engineers®
Engineer Research and Development Center

North Half Moon Bay Shoreline Improvement Project,
Pillar Point Harbor, CA
Coastal Engineering Appendix

Photograph courtesy of U.S. Army Corps of Engineers, 2011

Lihwa Lin, Honghai Li, James Zoulas, Lisa Andes, and Frank Wu

FINAL January 2015

North Half Moon Bay Shoreline Improvement Project, Pillar Point Harbor, CA

Lihwa Lin[1], Honghai Li[1], James Zoulas[2], Lisa Andes[3], and Frank Wu [4]

*[1]Coastal and Hydraulics Laboratory, U.S. Army Engineer Research and Development Center,
3909 Halls Ferry Road, Vicksburg, MS 39180-6199*

[2]San Francisco District, U.S. Army Corps of Engineers, 1455 Market Street, San Francisco, CA 94103

[3]Saint Louis University, andes@slu.edu

[4]Retired from San Francisco District, U.S. Army Corps of Engineers

Final Report

Contents

Figures and Tables

Figures

Tables

Unit Conversion Factors

Multiply	**By**	**To Obtain**
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
knots	0.5144444	meters per second
miles (nautical)	1,852	meters
miles (U.S. statute)	1,609.347	meters
miles per hour	0.44704	meters per second
pounds (force)	4.448222	newtons
pounds (force) per foot	14.59390	newtons per meter
pounds (force) per square foot	47.88026	pascals
square feet	0.09290304	square meters
square miles	2.589998 E+06	square meters
tons (force)	8,896.443	newtons
tons (force) per square foot	95.76052	kilopascals
yards	0.9144	meters

1 Introduction and Purpose

This report summarizes the findings of a coastal engineering study undertaken to support the development of a Detailed Project Report for the North Half Moon Bay CAP 111 Shoreline Improvement Project. The objectives of this study are to investigate potential impacts of the Pillar Point Harbor east breakwater on the shoreline directly to the south, and if impacts exist, to evaluate design measures to mitigate those impacts. The findings in this report represent the culmination of a collaborative effort involving the U.S. Army Corps of Engineers San Francisco District (SPN), the Coastal and Hydraulics Laboratory at the Engineering Research and Development Center (CHL-ERDC), and the San Mateo County Harbor District. This report includes an evaluation of previous studies in the project area, analyses of existing data, additional field data collection, and application of predictive modeling technology (Coastal Modeling System [CMS]) created by the U.S. Army Corps of Engineers Coastal Inlets Research Program (CIRP).

1.1 Background

Half Moon Bay is located approximately 18 miles (29 kilometers) south of San Francisco on the Pacific coast side of San Mateo County, California. Half Moon Bay derives its name from its crescent shaped coastline, which extends from the headland that comprises Pillar Point to the coastal bluffs, plains and dunes to the southeast (U.S. Army Corps of Engineers [USACE], 1984).

In 1948, Congress authorized the construction of two breakwaters (west and east) in order to create the 245 acre (1 square kilometer) Pillar Point Harbor (Figure 1). Design and construction commenced in 1954 with the project being completed in 1961 (USACE, 1986). The harbor is primarily used by commercial and recreational fishing interests, along with other recreational vessel traffic. In addition, Pillar Point Harbor is the only harbor of refuge along the 75 miles (121 kilometers) of coastline from San Francisco to Santa Cruz, California (USACE, 1996).

The west breakwater is located on the west side of Pillar Point Harbor, with the northernmost point of the west breakwater connecting to Pillar Point. In 1965, a 1,050 ft (320 m) log dogleg section was added to the west breakwater in an effort to further reduce wave energy from entering the harbor (USACE, 1986). The east breakwater, which is the focus of this study, is located southeast of Princeton, and separates the harbor from the coastal bluffs to the southeast. Dimensions of the east and west breakwaters are presented in Table 1. In addition, Pillar Point Harbor has three interior breakwaters, which were constructed in 1982 in order to enclose a marina development at the east end of Pillar Point Harbor (USACE, 1981).

Table 1: Dimensions of East and West Breakwaters of Pillar Point Harbor		
Parameter	West Breakwater	East Breakwater
Length	2,620 ft (800 m)	4,420 ft (1,350 m)
Crest Width (m)	18 ft (5.5 m)	18 ft (5.5 m)
Crest Elevation (m)	11 to 15 ft (3.4 to 4.6 m)	(11 to 13 ft (3.4 to 4.0 m)

The study area is located adjacent and southeast of the east breakwater and extends 0.9 mi (1.4 km) along the shoreline. Construction of the east breakwater occurred between 1959 and 1961 (USACE, 1961), and rates of coastal erosion within the study area shoreline have increased since construction of the harbor (Griggs et al., 2005). It has been postulated that the construction of the east breakwater has disrupted the natural movement of sediment that supplied sand to the adjacent shoreline and further south (USACE, 1996). Thus, this study will utilize a variety of techniques to determine the extent to which the east breakwater has enhanced erosion in the study area, and evaluate potential erosion mitigation measures.

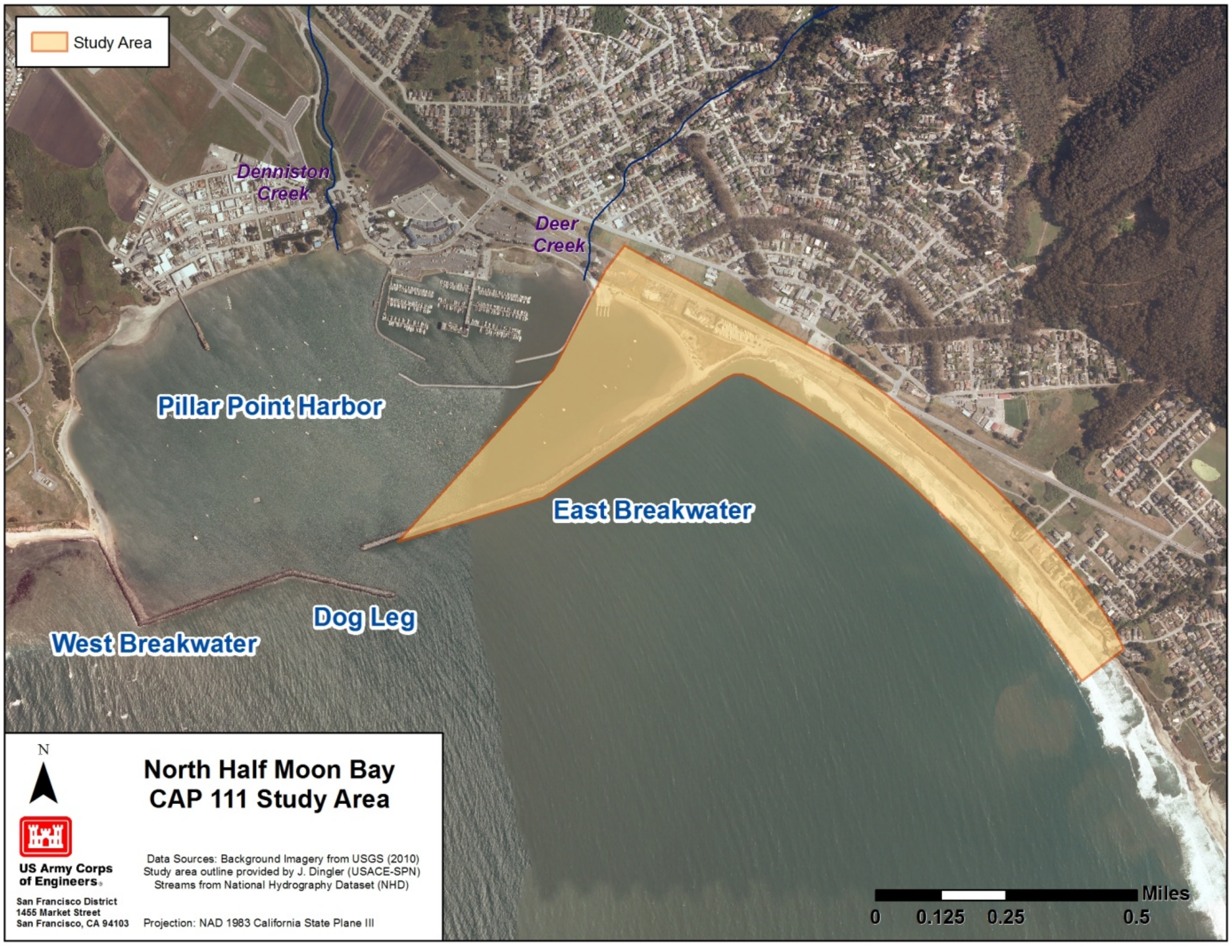

Figure 1. North Half Moon Bay study area

1.2 Objectives

The objectives of this study were formulated to support the development of a Detailed Project Report, and involved evaluating the impacts of the east breakwater on the study area (Figure 1):

1. Review findings from previous studies and historical data collected in the study area.
2. Establish rates of bluff erosion, and determine if and the extent to which increased rates of bluff retreat can be attributed to impacts from the east breakwater.
3. Conduct a field data collection effort to identify physical processes occurring in the study area for use in numerical model calibration/validation.
4. Estimate sediment transport rates under present (without-project) conditions in the vicinity of the east breakwater and section of shoreline experiencing erosion.
5. Estimate background sediment transport rates of the study area prior to breakwater construction.
6. Compare sediment transport rates pre- and post-breakwater construction to determine changes in net sediment transport incurred by construction of the east breakwater.
7. Simulate the impacts of proposed design measures on sediment transport and nearshore bed morphology.

1.3 Methods

In order to meet the study objectives, SPN staff reviewed previous studies and data, analyzed field data collected from the project area, and conducted a GIS-based analysis of coastal bluff retreat. In addition, CHL-ERDC and SPN staff utilized the Coastal Modeling System (CMS) to simulate sediment transport and nearshore morphology (bed) change under various without-project and design measure scenarios. Detailed descriptions of the methods employed by this study are presented in Section 3 of this document.

1.4 Scope of Report

This report is organized into 7 sections, which culminate in a summary of key findings along with several recommendations regarding design measures. The report begins with a description of physical conditions of the site including wind, waves and sediment transport, which is followed by a discussion of the methods utilized to meet the study objectives. The report also includes a discussion of the anticipated without-project conditions based on coastal bluff erosion analysis and CMS modeling of pre- and post-breakwater scenarios. This is followed by a discussion of the design measure formulation process, the results of the engineering analyses (including numerical modeling) of the proposed design measures, and the final summary section.

2 Physical Characteristics

Physical characteristics evaluated at this site include wind, wave, water surface elevation (WSE), currents, and sediment composition. Physical characteristics data were obtained from a number of sources: nearby National Buoy Data Center (NDBC) buoys (Figure 2), National Oceanic Atmospheric Administration NOAA tide stations (Figure 2) and field measurements collected by the U.S. Geological Survey (USGS) (Figure 3). The offshore wind and wave information described herein is primarily based on the Monterey (46042) and Half Moon Bay (46012) wave buoys, with additional data from the Bodega Bay buoy (46013) utilized to further characterize physical conditions during the numerical modeling simulation period (see Section 3.3 and Table 3).

Tidal information described herein is based on the NOAA tide station (9413450) located at Monterey, California. Information used to describe nearshore wave, current and WSE was collected by the USGS as part of the field data collection effort of this project between May 2 and June 2, 2011 (Hoover, 2011). Four moorings were deployed as part of this effort, and referred to as the ‘Harbor Mouth’, ‘Nearshore’, ‘Offshore’ and ‘Romeo Pier’ Moorings (Figure 3; Table 3). Sediment transport, sources and characteristics described herein are based on a number of previous studies conducted in the project area.

Table 2: NDBC wave buoys in the study area

Buoy ID	Latitude	Longitude	Water Depth
46012 - Half Moon Bay	37.363	-122.881	685 ft (208.8 m)
46013 - Bodega Bay	38.242	-123.301	382 ft (116.4 m)
46026 - San Francisco	37.755	-122.839	174 ft (53 m)
46042 - Monterey	36.785	122.469	6,883ft (2,098 m)

Table 3: Field Deployment Locations

Mooring ID	Latitude	Longitude	Water Depth
Offshore	37.47675	-122.49538	89 ft (27 m)
Nearshore	37.49218	-122.47270	30 ft (9 m)
Harbor Mouth	37.49515	-122.48698	26 ft (8 m)
Romeo Pier	37.50008	-122.49075	10 ft (3 m)

Figure 2. Select NDBC and NOAA stations in the study area region, which were used to characterize physical conditions in the study area

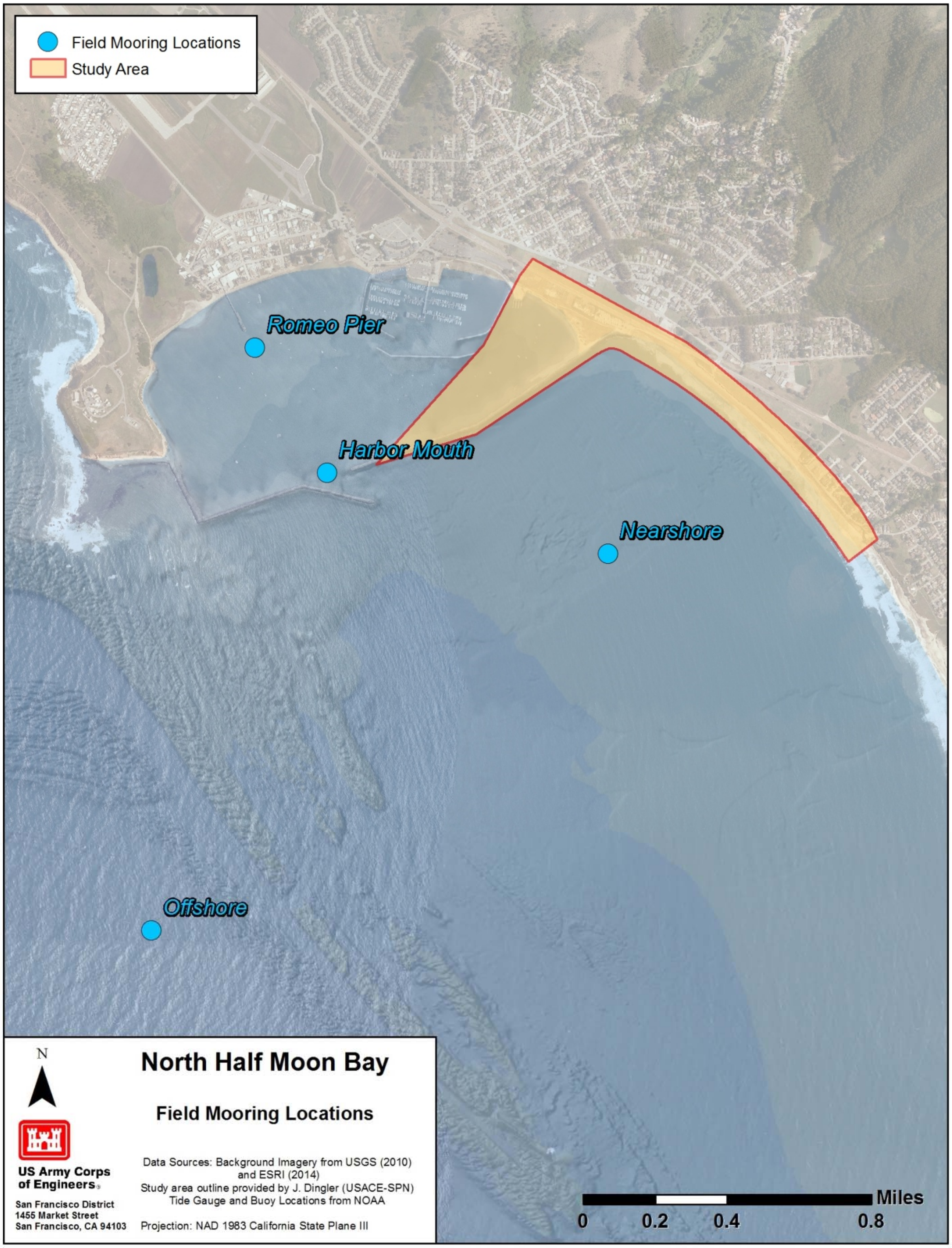

Figure 3. Field Mooring Locations at Romeo Pier, Harbor Mouth, Nearshore and Offshore

2.1 Water Surface Elevation (WSE)

2.1.1 Tides

All WSE data was derived from measurements at the Monterey tide station (9413450), which is located at the southern end of Monterey Bay, approximately 60 nautical miles (111 kilometers) from the project area (Figure 2). Hourly water level data was archived from 1974 to the start of this analysis in 2011, which spans a period of 37 years. Figure 4 illustrates a representative tidal data set of predicted, verified and residual WSE from May 2 to June 2, 2011.

Predicted tides are computed based on a number of tidal constituents that account for the impact of lunar, solar and other physical forces on water levels. Verified tides represented measured WSEs, which have been subjected to quality control by the NOAA National Ocean Service. The verified (measured) water level includes predicted tide and atmospheric effects, such as storm surge, sea level rise and El Niño contributions. Although there may be small errors in the predicted tide computation, for the purposes of this analysis it is assumed that the predicted tide record is correct and the differences between measured and predicted tide are all due to atmospheric effects.

Figure 4 illustrates the mixed semi-diurnal tidal signal commonly observed on the California coast, which is characterized by 2 unequal sets of daily highs and lows that vary in amplitude over time. Tidal amplitude also has a distinct seasonal signal along the California coast, with the largest tidal ranges typically occurring during spring tidal cycles in the winter months (Figure 5; Figure 6; Figure 7; Figure 8). As a result, coastal infrastructure is often more vulnerable to damage from inundation and wave attack when strong storm events (e.g., 27 January 1983) coincide with the peak of spring and other longer-term tidal cycles (Griggs and Johnson, 1983).

In order to provide a better understanding of how water levels may vary in the immediate study area, field measurements of water levels from May 2 to June 2, 2011 were compared with verified WSEs from the Monterey (9413450) tide station (Figure 9). Qualitatively, the Monterey tide station and the local field measurements of WSE show good agreement, suggesting minimal change in WSE between the tide station at Monterey and the project area. The WSEs measured at the field moorings had a maximum range of 5.6 ft (1.7 m), which is reasonably close to the difference of 5.33 ft between the Mean Lower-Low Water (MLLW) and Mean Higher High Water (MHHW) datums at the Monterey tide station (Table 4).It should be noted that there was a relatively high amplitude tidal cycle from May 14 to 20, which can account for some of the difference between the field mooring and long-term tide station ranges.

Table 4: Tidal datum of NOAA tide station 9413450 in the Monterey, CA and North Half Moon Bay, CA

Datum	WSE* (ft, NAVD 88)	WSE* (ft, MLLW)	North Half Moon Bay** (ft, MLLW)
Maximum Recorded Tide (01/27/1983)	8.02	7.88	9.02
Mean Higher High Water (MHHW)	5.46	5.33	5.45
Mean High Water (MHW)	4.76	4.63	--***
Mean Tide Level (MTL)	2.99	2.86	--***
Mean Sea Level (MSL)	2.96	2.83	2.99
Mean Low Water (MLW)	1.22	1.09	--***
Mean Lower Low Water (MLLW)	0.13	0.00	--***
Minimum Recorded Tide (01/11/2009)	-2.26	-2.40	-2.49

*National Oceanic Atmospheric Administration, 2011
**U.S. Army Corps of Engineers, 1996
***Data not available to report

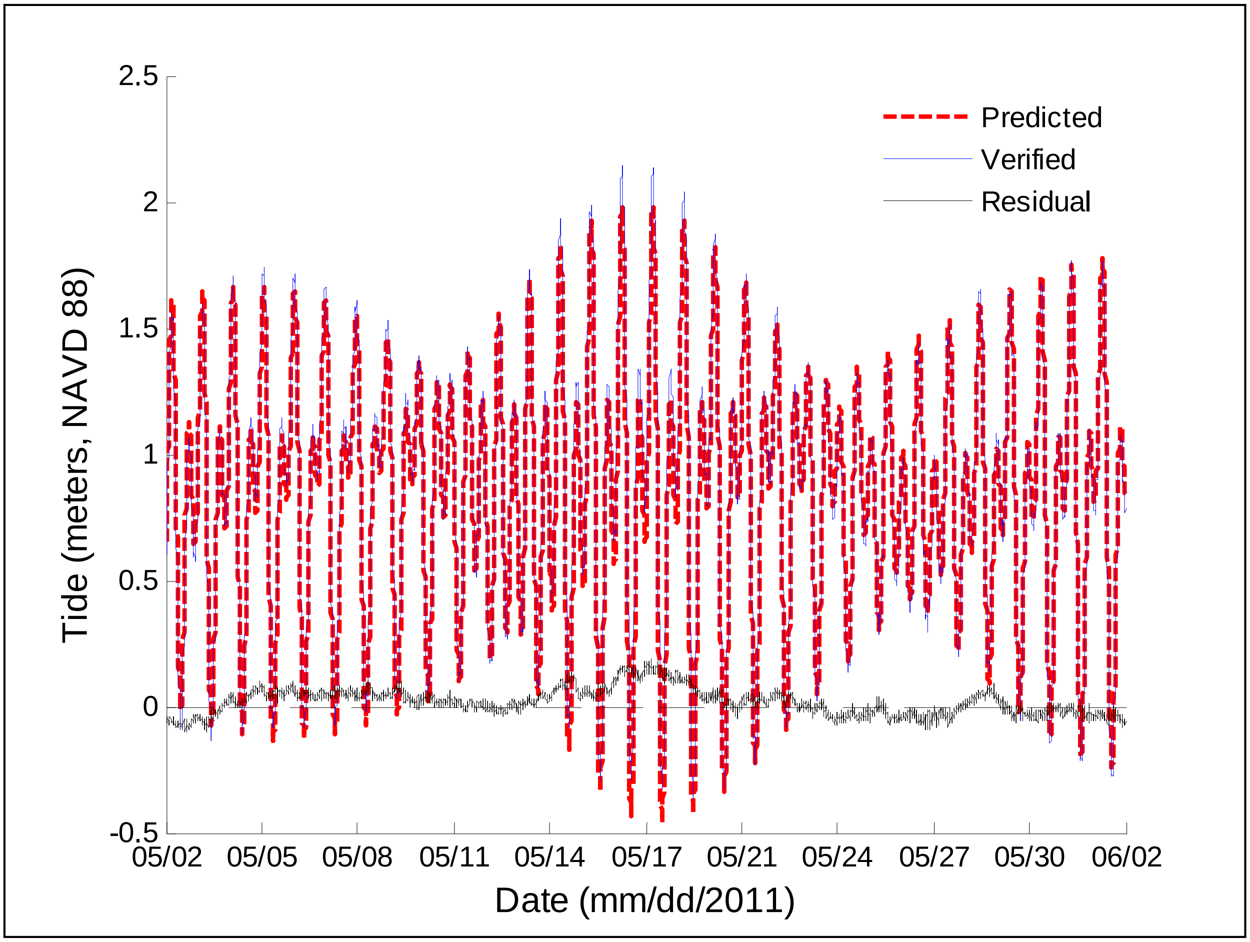

Figure 4. Time series of Monterey tide station (9413450) data from 05/02~06/02/2011 (NOAA, 2011)

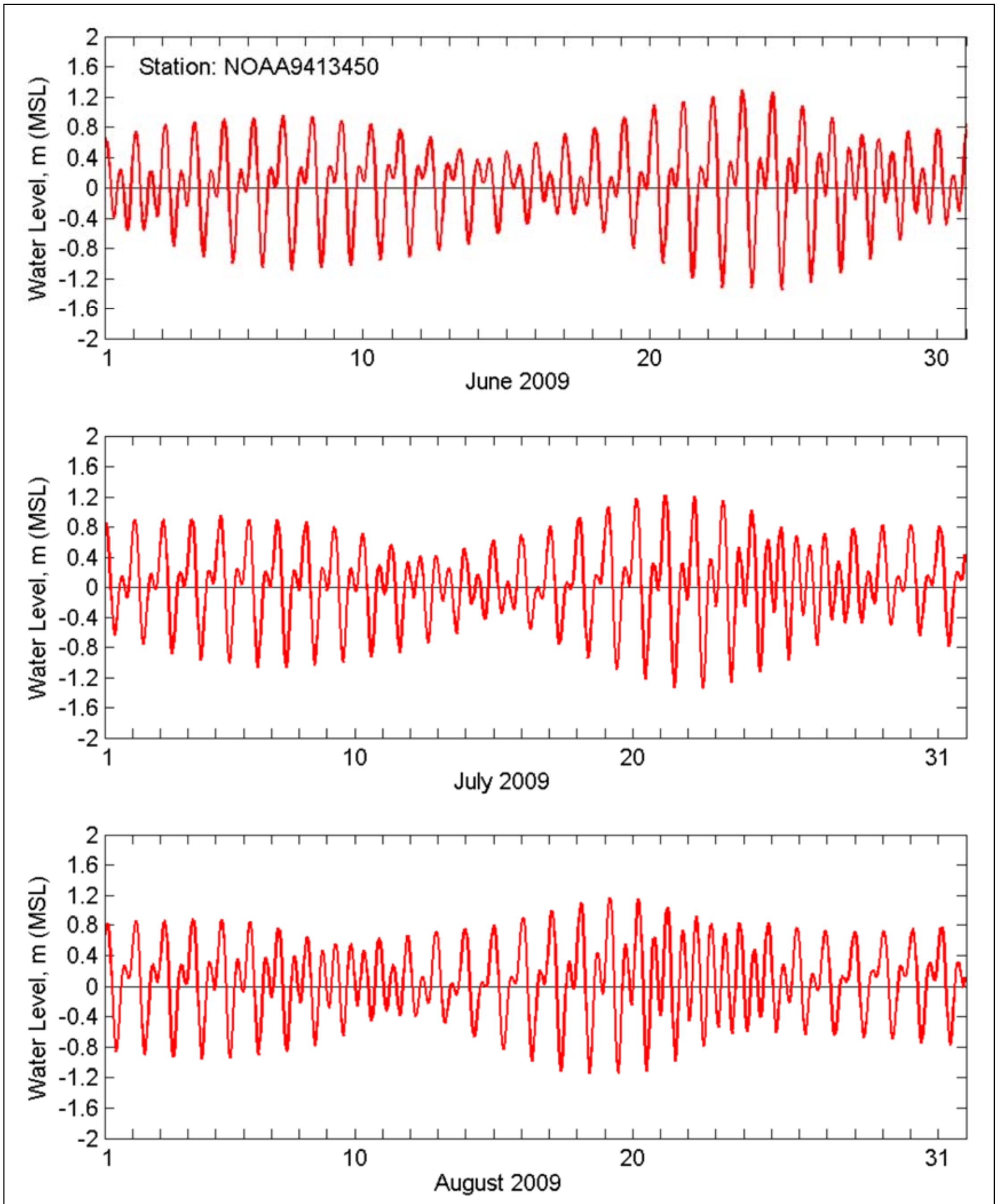

Figure 5. Water level data at NOAA Station 9413450 for June to August 2009

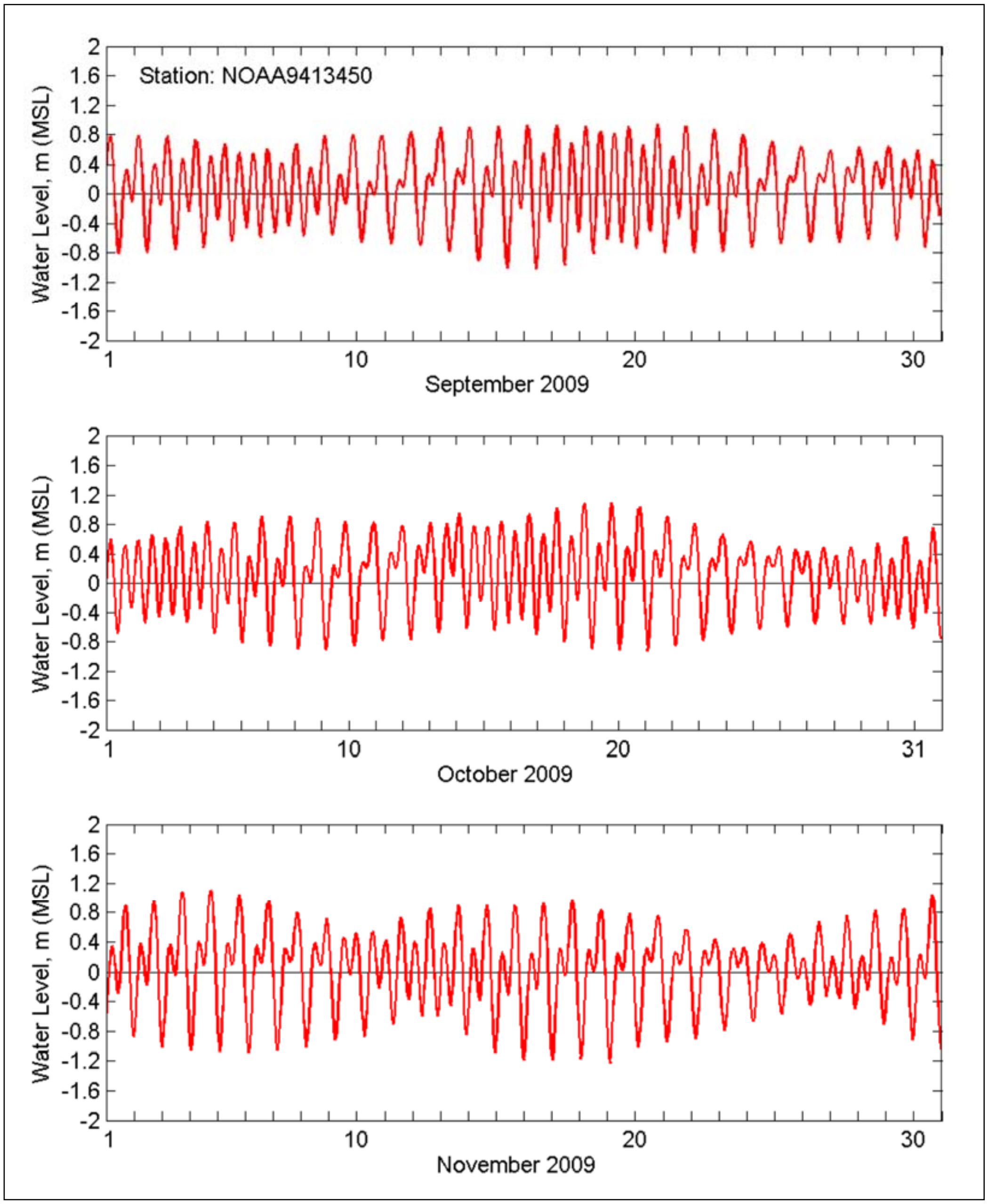

Figure 6. Water level data at NOAA Station 9413450 for September to November 2009

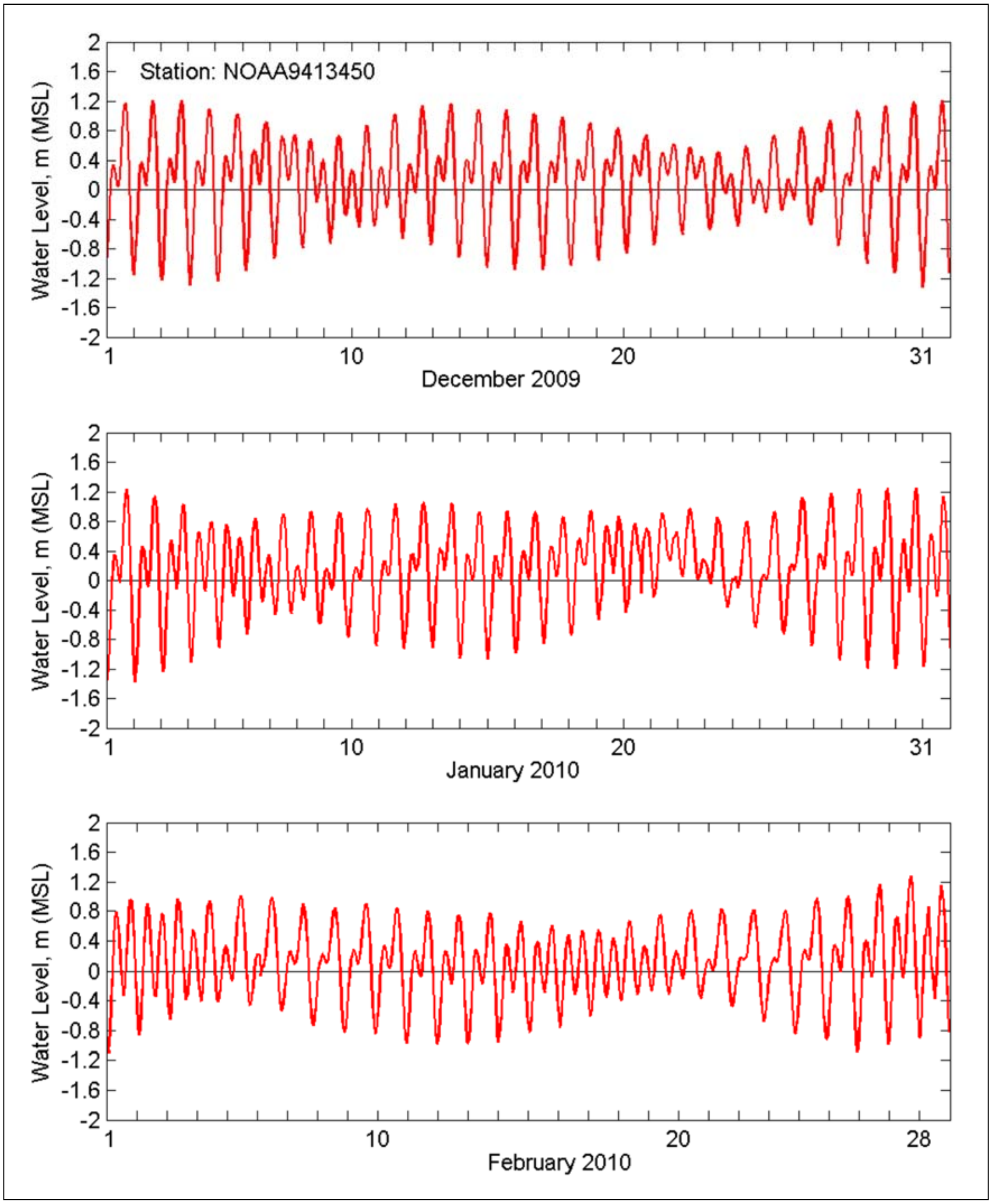

Figure 7. Water level data at NOAA Station 9413450 for December 2009 to February 2010

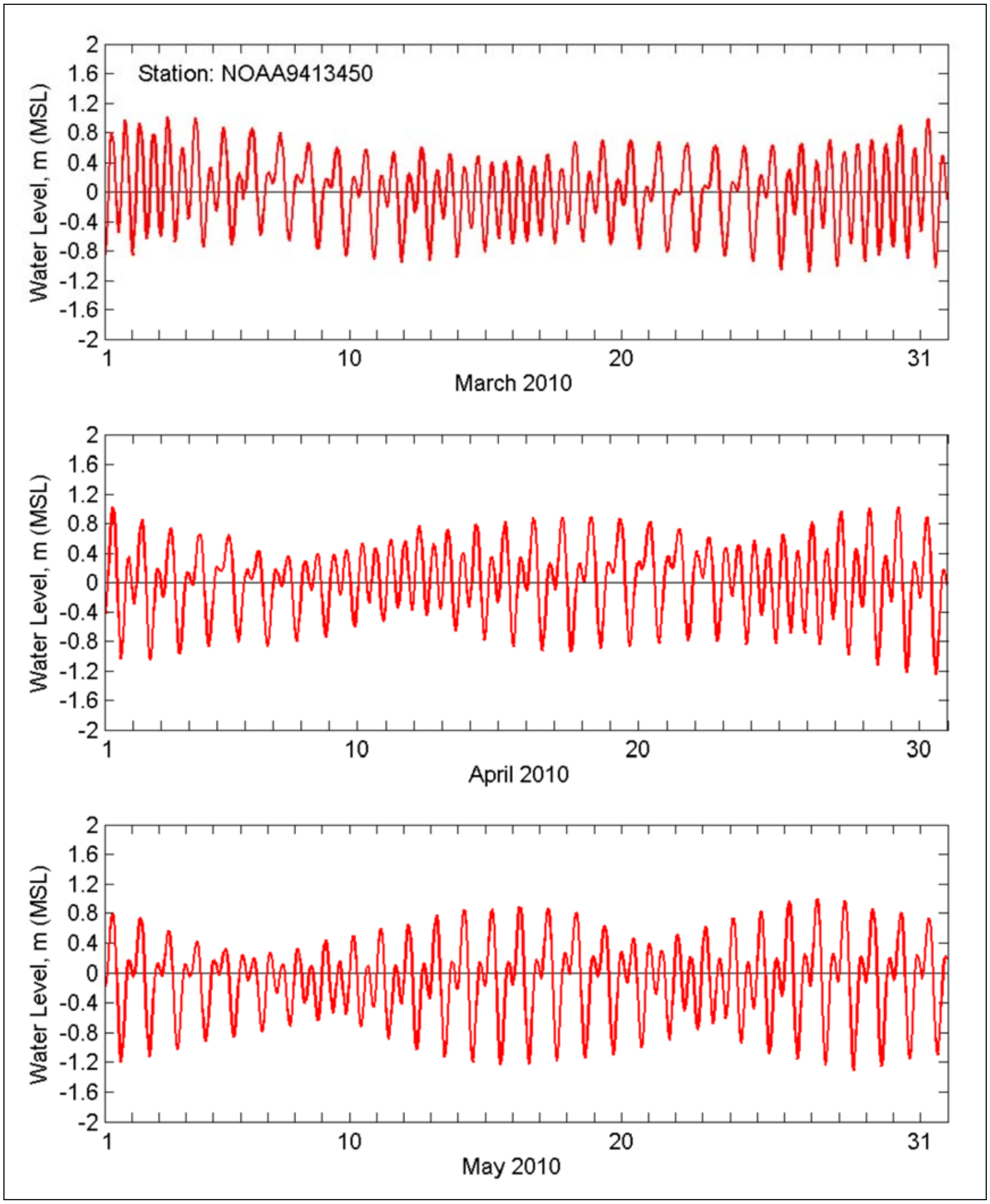

Figure 8. Water level data at NOAA Station 9413450 for March to May 2010

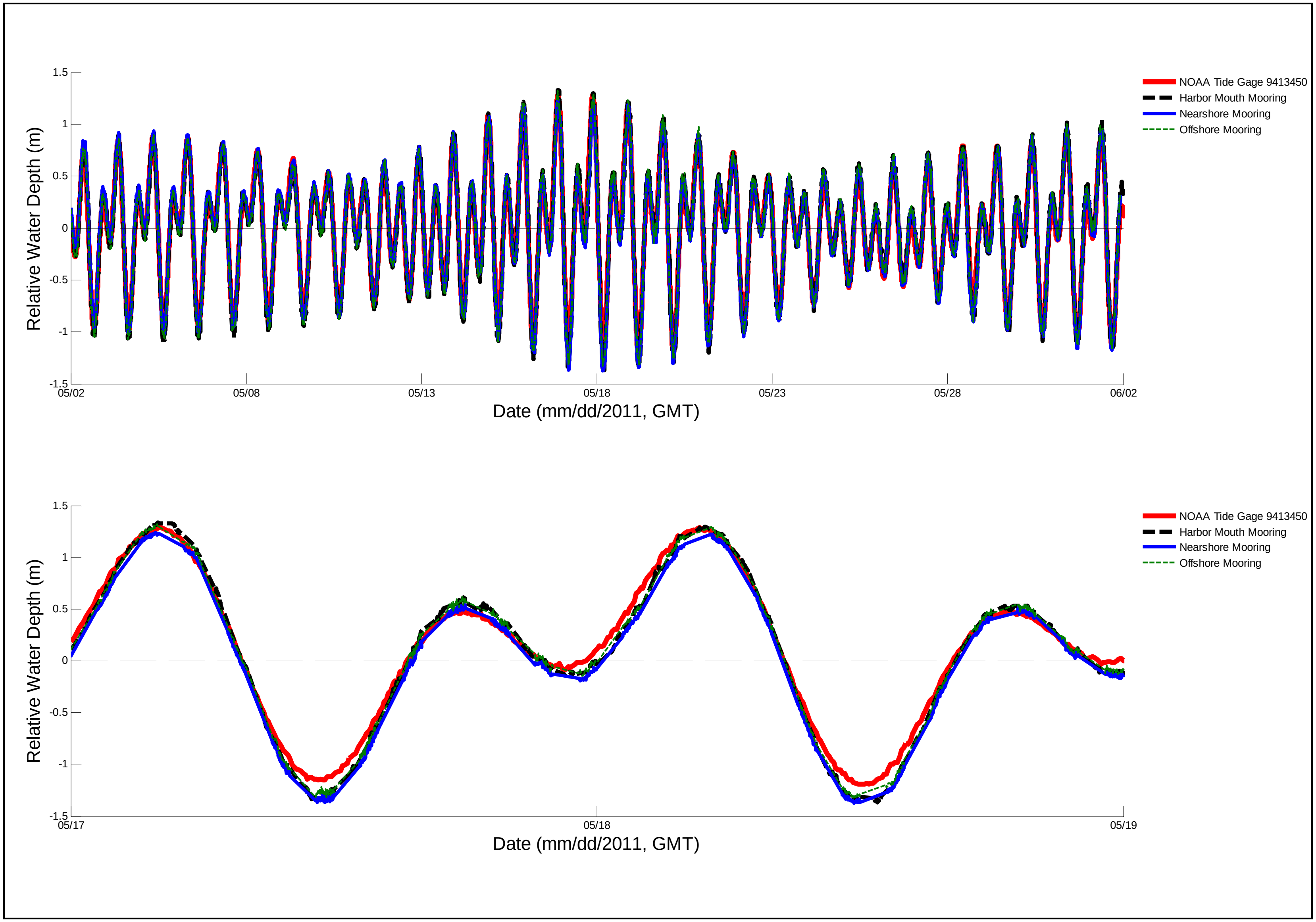

Figure 9. Comparison of measured WSEs at the NOAA tide gage, Harbor Mouth Mooring, Nearshore Mooring, and Offshore Mooring

2.1.1.1 Storm Surge

Storm surge is induced by storm wind stress and depressed atmospheric pressure at the center of a given storm. On the west coast, central pressure depression is the major contributor to the surge, due to the relatively narrow continental shelf and steep beach profiles. The residual WSE was computed by subtracting the predicted tide from the verified tide. The maximum tidal residual presented in Figure 4 was 0.7 ft (0.2 m) and the maximum recorded tidal residual at the Monterey tide station is on the order of 2.0 ft (0.6 m). To the north, the maximum tidal residual recorded at the San Francisco tide station was approximately 3 ft (0.9 m). The expected tidal residual (coastal storm surge) at the study location likely falls somewhere between the Monterey and San Francisco tide stations, as it is located on the open coast between these tide stations.

2.1.1.2 Sea Level Rise (SLR)

Future projections of SLR can be described with local and eustatic (global) historical sea level rise (SLR) rates. Local historical SLR projections in the vicinity of the project site are estimated to be approximately 0.44 ft (0.13 m) in a 100 year period or 1.34 millimeters per year (mm/yr) +/- 1.35 mm/yr (NOAA, 2011). The local SLR rate is based on mean monthly sea level data collected at the Monterey tide station between 1973 and 2006 (Figure 10). Global historical SLR is estimated to be 1.7 mm/yr, at a slightly higher rate than the local SLR trend, which was computed based on the USACE guidance (Engineer Regulation [ER] 1100-2-8162, Engineer Technical Letter [ETL] 1100-2-1).

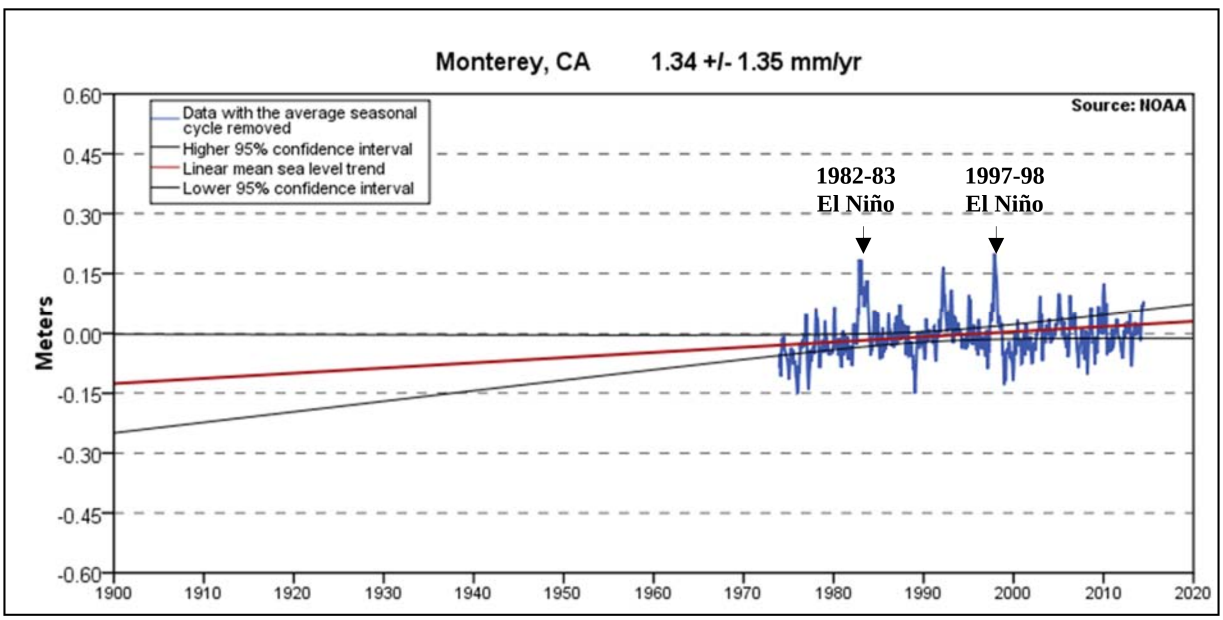

Figure 10. SLR trend at the Monterey tide station 9413450 (NOAA, 2011)

ER 1100-2-8162 specifies that USACE projects shall be formulated and evaluated for a range of possible rates of future sea level change, which can be represented by 3 scenarios of “Low”, “Intermediate”, and “High” sea level change rates. The Low rate is to be based on the local historical rate, which was derived from the Monterey tide station for this study (Figure 10), with the Intermediate and High rates based on modified National Research Council (NRC) Curves I and III, respectively. The Intermediate and High rates were computed based on equations referenced in ETL 1100-2-1, and all rates were projected out 50 years from a Year 0 starting at 2011 (Table 5).

Table 5: Projected sea level rise

50 - Year Projection	Sea Level Rise Curve Elevation
Year 50, Low (Local Historical)	0.23 ft (0.07 m)
Year 50, Eustatic Historical	0.30 ft (0.09 m)
Year 50, Intermediate (Modified NRC I)	0.66 ft (0.20 m)
Year 50, High (Modified NRC III)	1.94 ft (0.59 m)

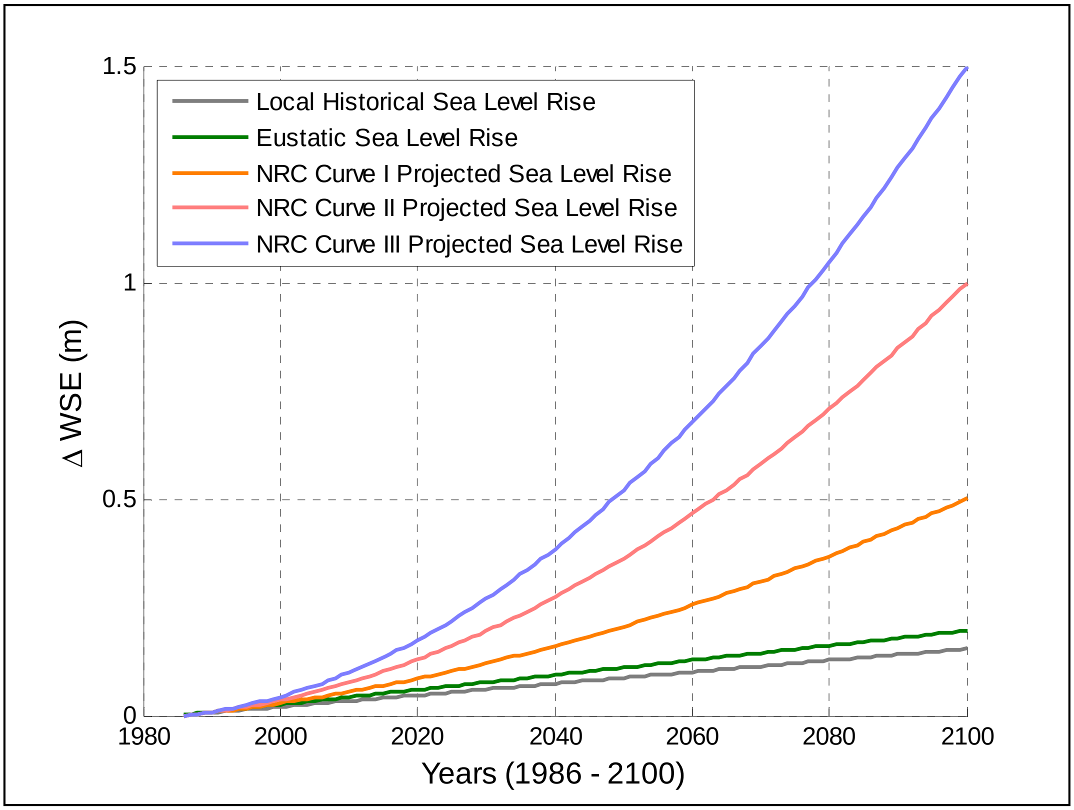

Figure 11. Local historical (USACE Low), eustatic historical, NRC Curve I (USACE Intermediate), NRC Curve II, and NRC Curve III (USACE High) sea level rise trends

2.1.1.3 El Nino

As shown in Figure 10, WSEs also fluctuate over inter-annual and longer time periods in concert with ocean-atmosphere oscillations such as the El Niño Southern Oscillation (ENSO). Strong El Niño conditions in the Pacific are associated with increased storm activity along the California coast, with storms following a more southerly track (Seymour, 1998; Storlazzi and Griggs, 2000; Griggs et al., 2005). As a result, the relatively low atmospheric pressure and strong waves associated with these events serve to elevate WSEs (National Research Council, 2012), with tidal residuals order of 0.2 m or 0.7 ft (Figure 10). El Niño conditions generally occur every 3 to 7 years, although the particularly intense and damaging El Niño events (e.g., 1982 - 1983, 1997 - 1998) tend to occur on the scale of every 10 to 20 year (Storlazzi and Griggs, 2000; Table 6). Recent research also suggests that the frequency of strong El Niño could double under current global warming projections (Santoso et al., 2013).

Table 6. *El Niño events between 1950 and 2012 (Null, 2014)*		
El Niño		
Weak	Moderate	Strong
1952-53	1951-52	1957-58
1953-54	1963-64	1965-66
1958-59	1968-69	1972-73
1969-70	1986-87	1982-83
1976-77	1991-92	1987-88
1977-78	1994-95	1997-98
2004-05	2002-03	
2006-07	2009-10	

Following the strong and damaging 1982-83 El Niño event, USACE investigated the meteorological patterns which lead to the severe wave conditions at Half Moon Bay during this event (USACE, 1983). The report indicated that seven severe storm events occurred during the winter months, with offshore wave heights ranging between 5 and 7 m (16 and 23 ft) with periods 8 to 16 seconds. Nearly all of these storms originated as strong extratropical cyclones, which approached the coastline from the northwest.

In contrast to a number of other structures along the California coast, the breakwaters at Pillar Point Harbor were not damaged by any of these storm events. The lack of damage was likely due to variations in local bathymetry (Figure 24), with large rocky outcrops serving to dissipate a considerable amount of the wave energy approaching from the west and northwest (USACE, 1983). However, waves approaching directly from the south may bypass these rock formations resulting in more direct wave energy reaching the shoreline and breakwaters.

2.2 Wind

Offshore winds in the study area are characterized by measurements collected at the Half Moon Bay buoy (NDBC 46012) (Figure 2). Continuous wind data has been collected by the Half Moon Bay buoy since 1980, resulting in a record of over 30 years. Wind speeds are greatest in the late winter and early spring, with mean monthly wind speeds between 5 and 8 m/s (10 to 15 knots). Wind speeds are weakest during the summer and fall months (July through September) when monthly mean wind speeds are between 2 and 5 m/s (5 to 10 knots). The predominant wind direction is from the northwest with 65% to 75% of the wind approaching from this direction in the spring through fall (Table 7; Figure 12). The only exception to this pattern occurs during the winter season, when the wind is from the northwest only 42% of the time.

Table 7. Seasonal wind speed statistics for NDBC buoys in the vicinity of the project area.

Season	U_{wind} (m/s)	U_{wind} (kts)	Northeast (0 to 90 degrees) (%)	Southeast (90 to 180 degrees) (%)	Southwest (180 to 270 degrees) (%)	Northwest (270 to 360 degrees) (%)
Annual	5.1	9.9	9	16	12	63
Winter	5.5	10.7	21	24	12	42
Spring	6.1	11.9	3	13	12	73
Summer	4.6	8.9	2	9	14	75
Fall	4.6	8.9	8	16	11	65

Measurements from the buoys indicated that periods of relatively strong winds occurred throughout the model simulation period (see Section 3.3) with the strongest winds recorded further offshore at the Bodega Bay (46013) buoy (Figure 14; Figure 15). As in the case of the annual statistics (Table 7), the predominant wind direction was from the north-northwest from the spring through the summer and fall, with a distinct shift during storm events in the winter months.

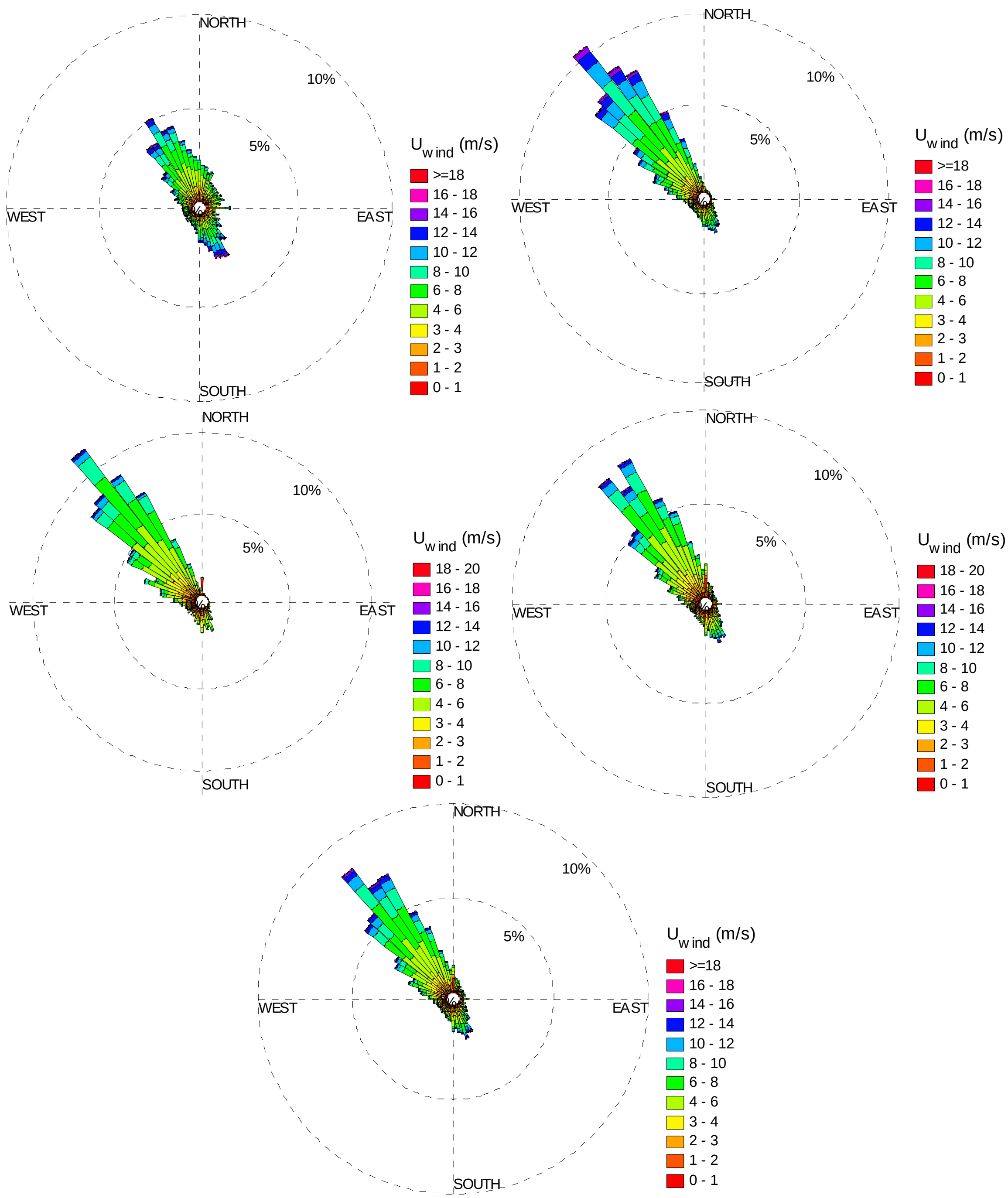

Figure 12. Wind rose of annual and seasonal directional wind speed (m/s). Winter (top-left), spring (top-right), summer (middle-left), fall (middle-right), annual (bottom).

2.3 Waves

Both offshore and nearshore wave fields can often induce currents, which influence sediment transport in the study area. As a result, a good understanding of local wave conditions is necessary to characterize potential sediment transport patterns and associated morphologic change in the study area.

2.3.1 Offshore Waves

Offshore wave conditions and wave climate for the study area were characterized by compiling long-term seasonal wave statistics from the Half Moon Bay (46012) and Monterey (46042) buoys. Wave data from these buoys provide a good overview of the wave climate in the region, and cover most of the directions from which waves can approach the study area.

The closest buoy is NDBC buoy 46012, or the ‘Half Moon Bay buoy’, which is located approximately 21 nautical miles (nm) (39 kilometers) west-southwest of the project site over relatively deep water (209 m or 685 ft). The Half Moon Bay buoy has collected continuous wave data since 1980 for a duration of 30 years; however, directional wave information was not collected until 2010 (National Oceanic Atmospheric Administration, 2011 (a)). As a result, the Monterey buoy, NDBC 46042, also provides valuable information on offshore wave directionality in the project vicinity.

Seasonal wave statistics from the Half Moon Bay buoy depicting seasonal trends in significant wave heights (H_s) and peak period (T_p) and are shown in Table 8. Mean seasonal H_s range between 5.2 and 9.2 ft (1.6 and 2.5 m), with the maximum recorded H_s of 31.8 ft (9.7 m) occurring during a winter storm event in February 2004. Continuous wind direction and magnitude has been collected since 1998 for a period of 12 years and is discussed below.

NDBC buoy 46042, or the ‘Monterey buoy’, is located approximately 48 nautical miles (89 kilometers) south of the study area over the deep waters (6,883 ft or 2,098 m) offshore of Monterey Bay (Figure 2). This buoy has collected continuous wave data since 1987, with the record of directional wave data in starting in 1991 (19 year period). Wind speed and direction were collected shortly thereafter. As in the case of the to the Half Moon Bay buoy, there are also clear seasonal trends with larger H_s in the winter and smaller H_s in the summer months. The maximum recorded H_s of 32.5 ft (9.9 m) occurred during a winter storm in January 2008. The mean H_s for winter, spring, summer and fall seasons are 8.9 (2.7), 7.5 (2.3), 5.9 (1.8) and 6.9 (2.1) ft (m), respectively. While the measured H_s at this buoy are the larger of the 2, there is little difference in the observed seasonal mean period (National Oceanic Atmospheric Administration 2011 (b)).

Wave direction has been collected at the Monterey buoy since 1991, with continuous wind direction and magnitude collected since 2002. Wave roses indicate that the dominant wave direction is from the northwest, as in the case of other nearby buoys (Figure 13). However, there appear to be seasonal shifts in the direction of wave approach, with waves approaching from a more westerly direction during the winter season and northerly direction during the transitional period in spring. The buoy measurements are consistent with previous coastal analyses at the study area (USACE, 1996), which indicated that the predominant longshore transport is to the south due to northwesterly wave approach, with intermittent northward transport when waves approach from the southwest.

Table 8: Seasonal Wave statistics for NDBC buoys in the vicinity of the project area (National Buoy Data Center, 2011).

Season	Half Moon Bay		Monterey	
	H_s	T_p (s)	H_s	T_p (s)
Winter	8.2 ft (2.5 m)	13	8.9 ft (2.7 m)	13
Spring	7.5 ft (2.3 m)	11	7.5 ft (2.3 m)	12
Summer	5.6 ft (1.7 m)	10	5.9 ft (1.8 m)	10
Fall	6.6 ft (2.0 m)	12	6.9 ft (2.1 m)	12
Annual	6.9 ft (2.1 m)	11	7.2 ft (2.2 m)	12

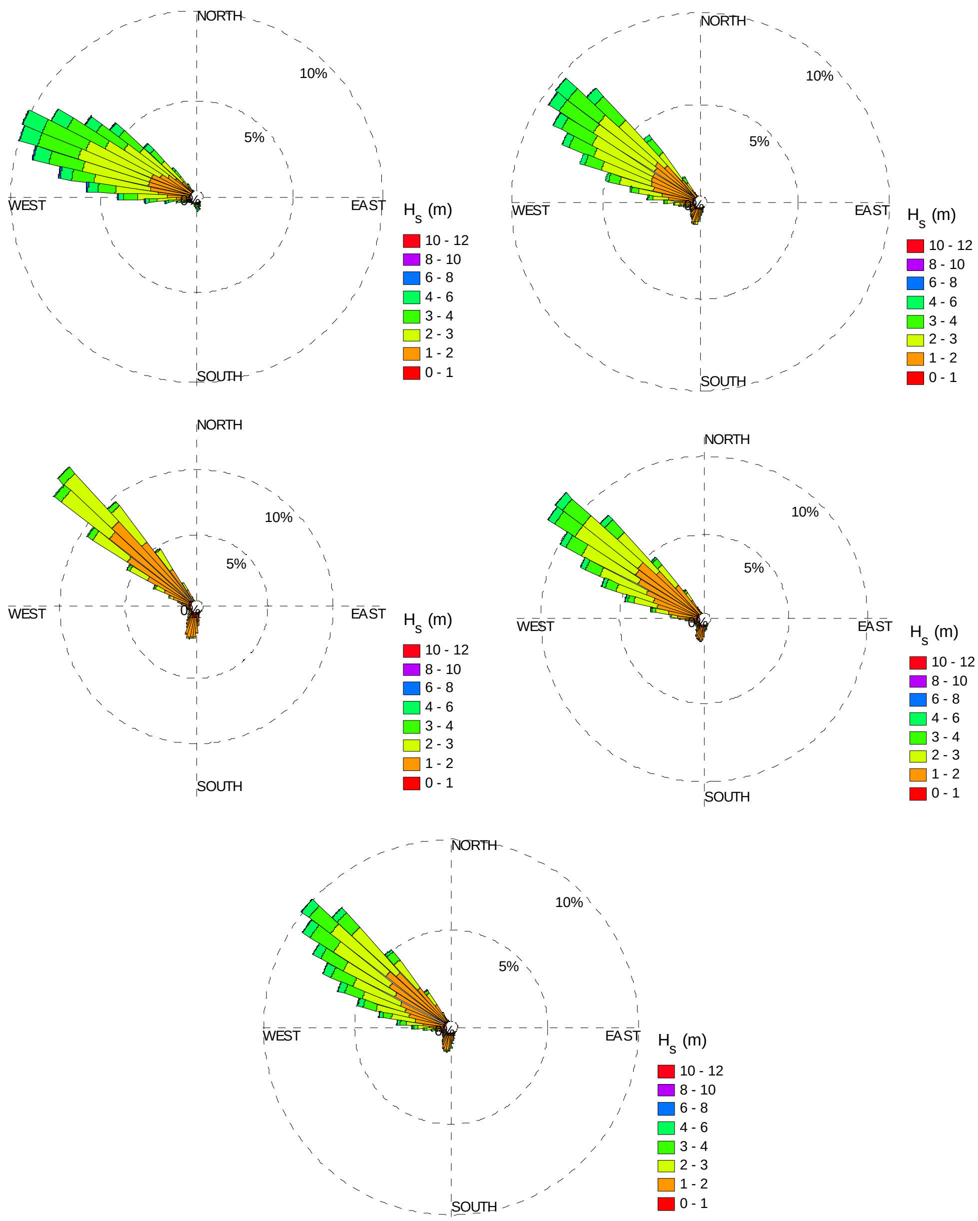

Figure 13. NDBC Buoy 46042 wave rose of annual and seasonal directional wave height. Winter (top-left), spring (top-right), summer (middle-left), fall (middle-right), annual (bottom).

In terms of extreme wave conditions, relatively large wave events occur almost every winter, with maximum annual H_s values approaching 20 to 25 ft (6 to 7.6 m) during the strongest storm of a given winter (Table 9). While a detailed statistical analysis of exceedance probability was not performed, the mean annual H_s plus 2 standard deviations is in the range of 30 to 34 ft. This suggests that the 0.01 percent annual chance exceedance (100 yr) H_s could be close to 35 ft (10.7 m). However, it should be noted that the both buoys did not collect data for much of winter season during the 1997-1998 El Niño event, and therefore the extreme statistics might underestimate peak H_s in the offshore region of the study area.

Table 9. Maximum annual H_s for NDBC wave buoy's in the project area

	Half Moon Bay (NDBC 46012)	Monterey (NDBC 46042)
Period of Record	1981 to 2008	1987 to 2008
Mean	22.7 ft (6.9 m)	24.5 ft (7.5 m)
Maximum	31.8 ft (9.7 m) in Feb 2004	32.5 ft (9.5 m) in Jan 2008
Standard Deviation	3.8 ft (1.2 m)	4.6 ft (1.4 m)
Mean + (2 * Standard Deviation)	30.4 ft (9.3 m)	33.8 ft (10.3 m)

During the model simulation period (June 2009 to May 2010), the monthly mean significant wave heights (H_s) at the 3 buoys (46013, 46026, and 46042) ranged from approximately 1.5 m (5 ft) in the summer months to just over 3 m (10 ft) in the winter months (Table 11). As expected, the greatest H_s were recorded during distinct storm events in the winter months (Figure 14), with the greatest H_s (8.2 m or 27 ft) recorded at the Monterey (46042) buoy on 20 January 2010 (Figure 15). Overall, the measured wave heights during the simulation period appeared to be slightly greater than the annual statistics for the buoys in the region (Table 8), which is consistent with the expected impacts of the observed moderate El Niño conditions (Table 6) on wave climate.

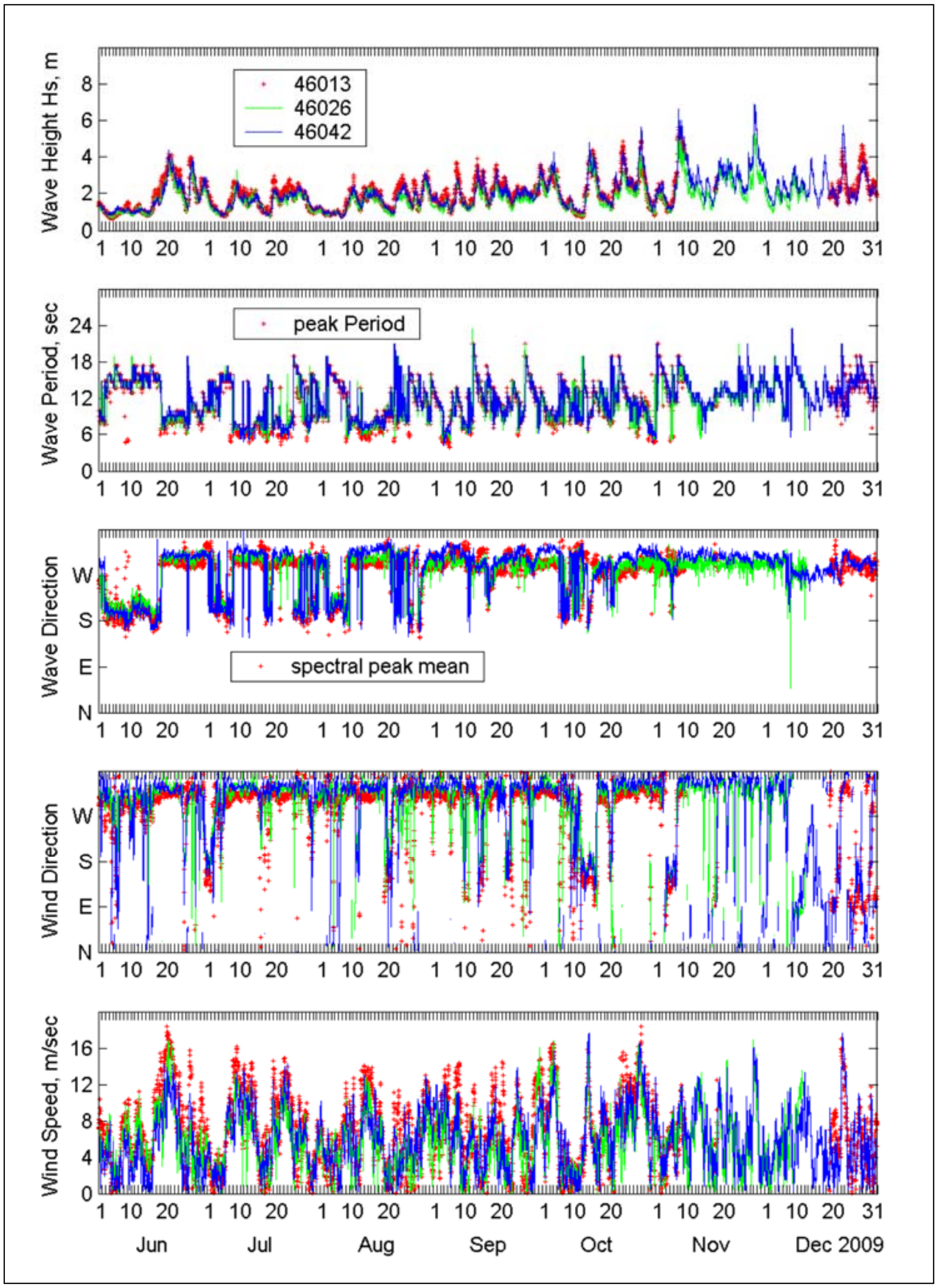

Figure 14. Wind and wave data at Buoys 46013, 46026, and 46042 for June to December 2009

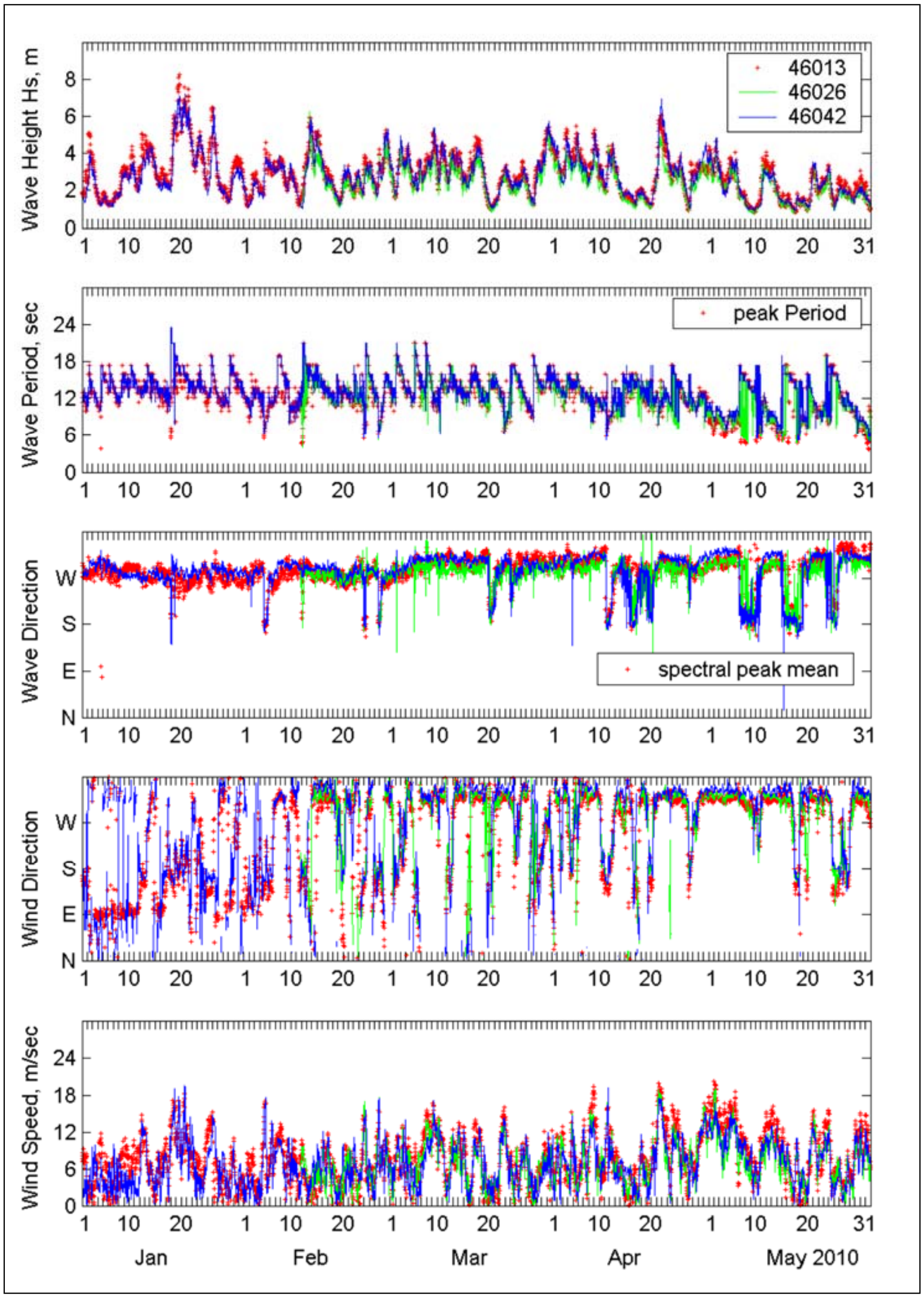

Figure 15. Wind and wave data at Buoys 46013, 46026, and 46042 for January to May 2010

2.3.2 Nearshore Waves

As the offshore waves propagate shoreward, wave energy can be focused or dispersed by shoreline irregularities, submarine canyons, headlands, and localized shoals. In the case of this study area, offshore waves typically refract around the Pillar Point headland before reaching the breakwaters and adjacent shoreline. Thus, it can be expected that nearshore wave heights in the study area will be smaller than at the offshore buoys, due to the dispersion of wave energy associated with refraction. Wave height is expected to be further reduced by the dissipation of wave energy on offshore reefs in the area.

This section describes wave data collected by USGS between May 2 and June, 2011 as part of the effort to quantify the nearshore wave environment in the study area. This data collection effort included equipment moorings at the Pillar Point Harbor inlet, in the nearshore area adjacent to the study area and offshore of the study area south of the west breakwater (Figure 3). The largest waves in the vicinity of the study area were measured at the Offshore Mooring (Figure 16), which is located just offshore of the rocky outcrops (shallow reefs), that surround much of the North Half Moon Bay study area.

At the Offshore Mooring, wave height ranged between 2.6 and 9.8 ft (0.8 and 3.0 m) with a mean wave height of 5.6 ft or 1.7 m (Figure 16). The wave heights measured at the Nearshore Mooring were smaller than the waves measured at the Offshore Mooring, and ranged from to 1.3 to 5.6 ft (0.4 to 1.7 m) with a mean wave height of 2.6 ft or 0.8 m (Figure 16). The decrease in wave height between the offshore and Nearshore Mooring could be related to wave dissipation over the reef structures. The smallest wave heights were measured at the Harbor Mouth Mooring, with a mean wave height of 0.3 ft (0.1 m), and a range of 0.2 to 1.0 ft (0.05 and 0.3 m) (Figure 16).

The wave period was fairly consistent between the three moorings, with a mean period of 13 seconds and a range of 1 and 23 seconds. However, there was more variability in wave period at the Harbor Mouth Mooring, possibly due to uncertainty associated with the very small wave energy measurements (Figure 17).

Measurements from the three moorings outside of the harbor suggest that there is considerable variability in wave direction in the vicinity of the study area (Figure 18). At the Offshore Mooring, wave direction was primarily from the northwest with sporadic changes in direction to the southwest. At the Nearshore Mooring, the wave direction was primarily from the southwest, as wave crests aligned parallel to the shoreline. Wave direction at the Harbor Mouth Mooring was more variable, perhaps due to the influence of tidal currents and the uncertainty associated with the insignificant wave energy measured at this location. However, it appears that the predominant direction of wave approach Harbor Mouth Mooring is from the southeast.

A review of the field deployment data and NOAA records indicate that several storm events occurred from May 15 to May 30 (NOAA, 2011). During these storm events, there was a notable shift in wave direction from the northwest (290 degrees) to the southwest (230 degrees) at the Offshore Mooring. This directional change is consistent with the understanding that waves typically approach from a more southerly direction during storm events. It should also be noted that this shift in wave direction was not observed at the other moorings closer to shore.

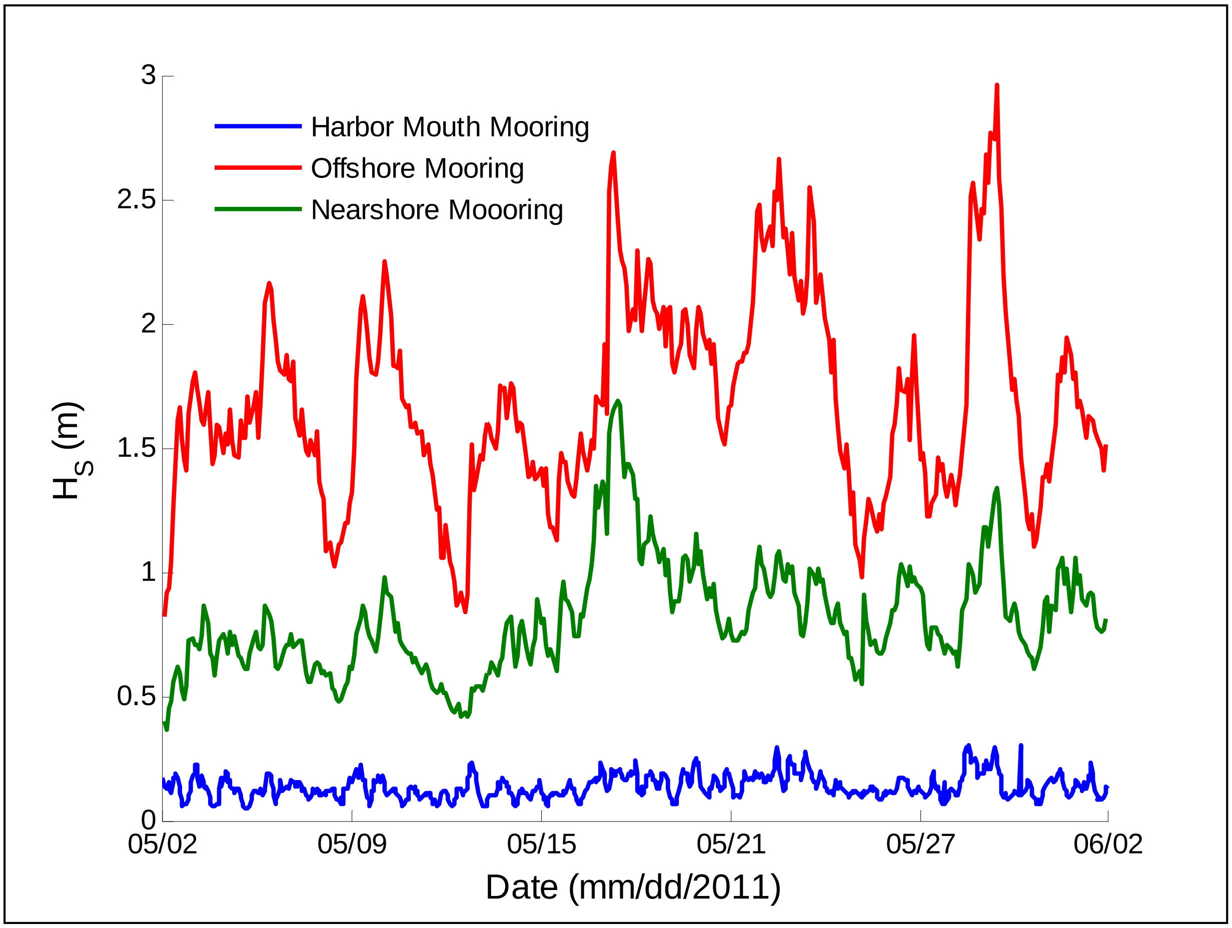

Figure 16. Nearshore wave height measured at the Harbor Mouth, west of Surfer's Beach and east of the West Breakwater

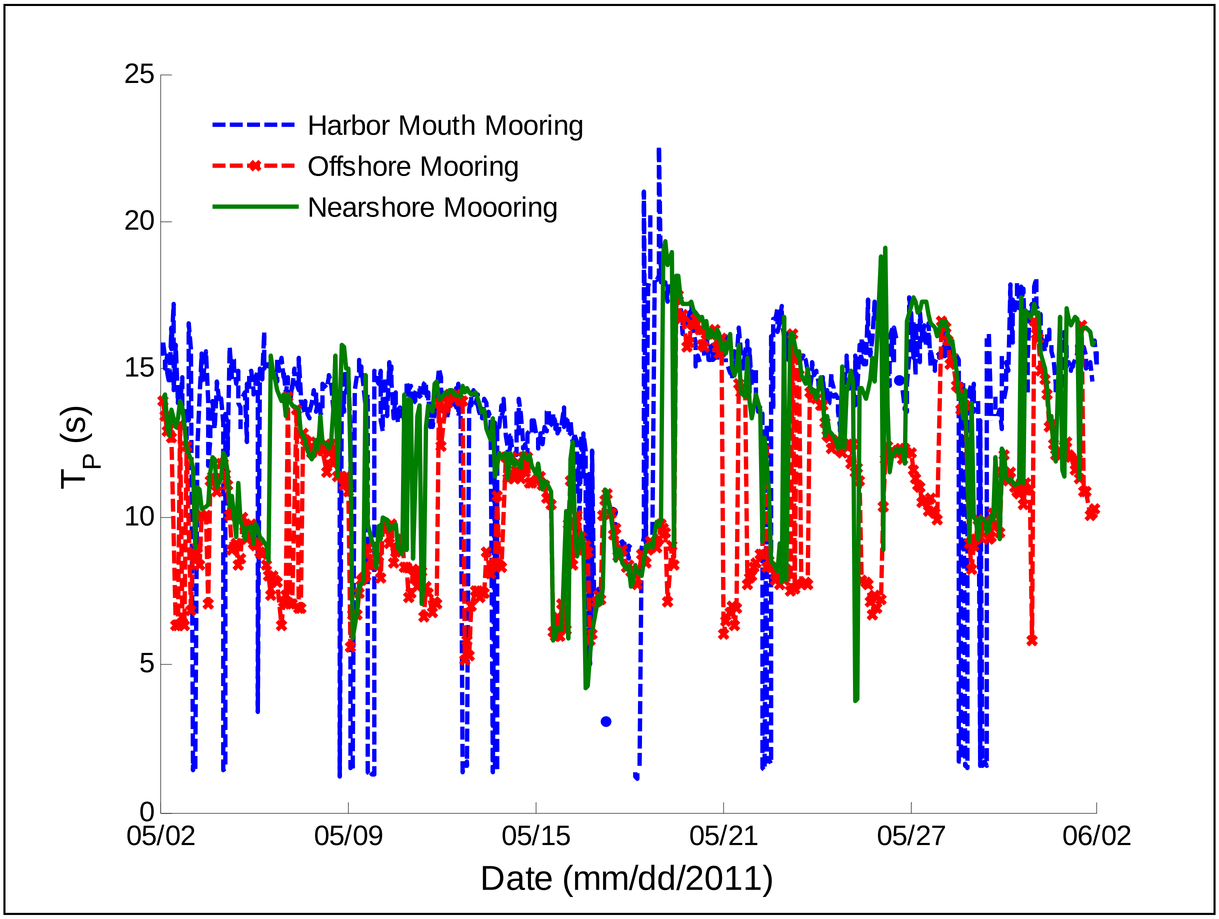

Figure 17. Nearshore peak period measured at the Harbor Mouth, west of Surfer's Beach and east of the West Breakwater

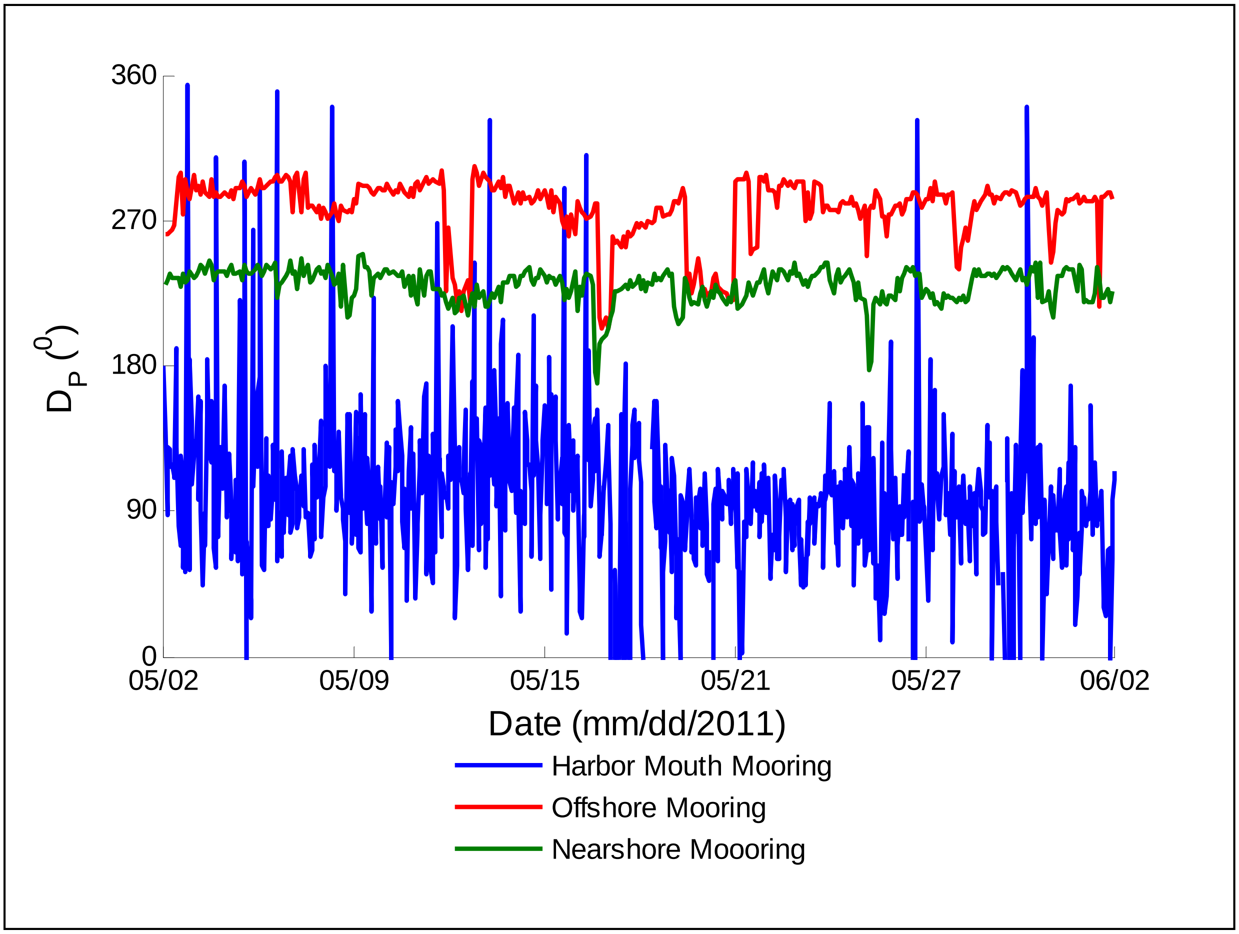

Figure 18. Nearshore wave direction measured at the Harbor Mouth, west of Surfer's Beach and east of the West Breakwater

2.4 Currents

Circulation patterns in the vicinity of the study area can be a function of waves, winds, and tides. 3 moorings were deployed in the study area to describe current magnitude and direction vertically in the water column (Figure 3). However, further analysis will be necessary to decouple the current magnitude and direction in order to identify the respective contributions of waves, tides and winds. In addition, these field measurements were used to calibrate the hydrodynamic and sediment transport model used to estimate erosion rates described in Section 3.3. Finally, it should be also noted that the primary offshore current in the study area, the California Current, is too weak and far offshore to mobilize sediment in the project area (USACE, 1996).

Figure 19 summarizes the vertical distribution and depth averaged direction of currents at the Harbor Mouth Mooring. The blue and turquoise colors shown in the figure represent current magnitudes between 0 and 0.49 ft per second (0 and 0.15 m/s). The average current speed at the Harbor Mouth Mooring during the field deployment was 0.36 ft/s (0.11 m/s) with a maximum observed current speed of 4.6 ft/s (1.4 m/s). Tidal currents at the harbor mouth can generate large eddies within Pillar Point Harbor. 2 clockwise rotating eddies typically form in the harbor during flood tides, in the western and eastern sections of the harbor (USACE, 1996). Current direction generally aligns with the axis of the dogleg of the west breakwater and inlet to Pillar Point Harbor, as the currents oscillate to the northwest and southeast. The storm event of May 15 to May 19 was not detected in the current profile, suggesting that the Harbor Mouth Mooring may be more sheltered from storm effects that the moorings on the open coast.

Figure 20 summarizes the vertical distribution and depth averaged direction of currents at the Nearshore Mooring. This section of the study area had the weakest current magnitudes, with an average current speed of 0.2 ft/s (0.06 m/s) and a maximum observed current speed of 0.6 ft/s (0.19 m/s). The current rose does not indicate a dominant current direction, as it appears that the currents were measured from all directions at the Nearshore Mooring. The storm event of May 15 to May 19 was observed in the vertical profile of the current measurements, and is highlighted by the bright red colors of the current profile between May 17 and 18. During this event, the measured depth averaged currents increased from approximately (0.2 ft/s) 0.06 m/s to 0.8 ft/s (0.24 m/s).

Figure 21 summarizes the vertical distribution and depth averaged direction of currents at the Offshore Mooring. This section of the study area had the strongest current magnitudes, with an average current speed of 0.36 ft/s (0.11 m/s) and a maximum observed current speed of 2.0 ft/s (0.60 m/s). As in the case of the mooring at the harbor inlet, the depth averaged currents appear to be directed towards the northwest and southeast. The storm event of May 15 to May 19 was most pronounced at this mooring, as highlighted by the dark red colors in the vertical profile. During this storm event, the measured depth averaged currents increased from approximately 0.36 ft/s (0.11 m/s) to 1.4 ft/s (0.44 m/s).

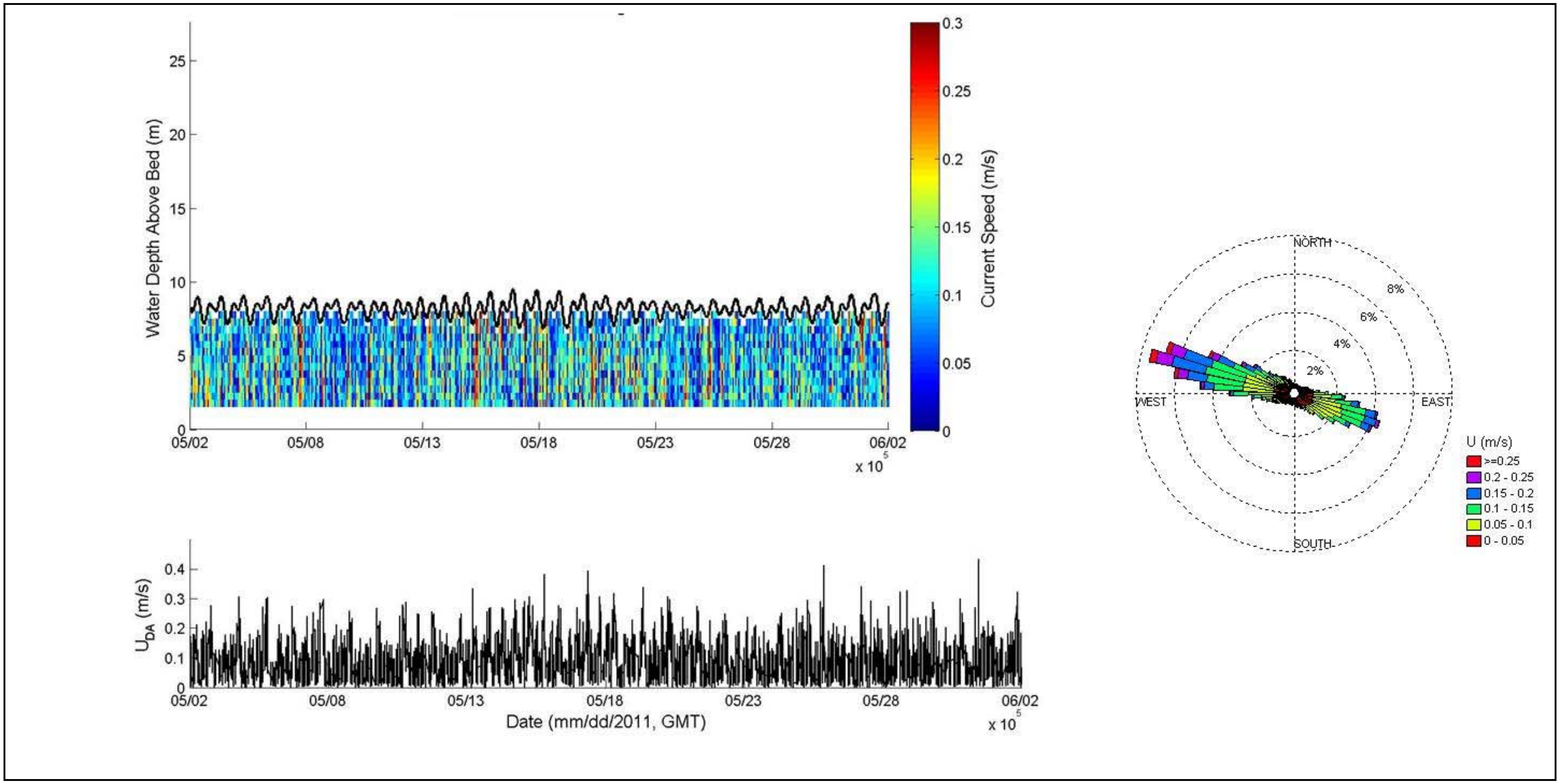

Figure 19. Vertical profile of currents at the Harbor Mouth Mooring between May 2 and June 2, 2011

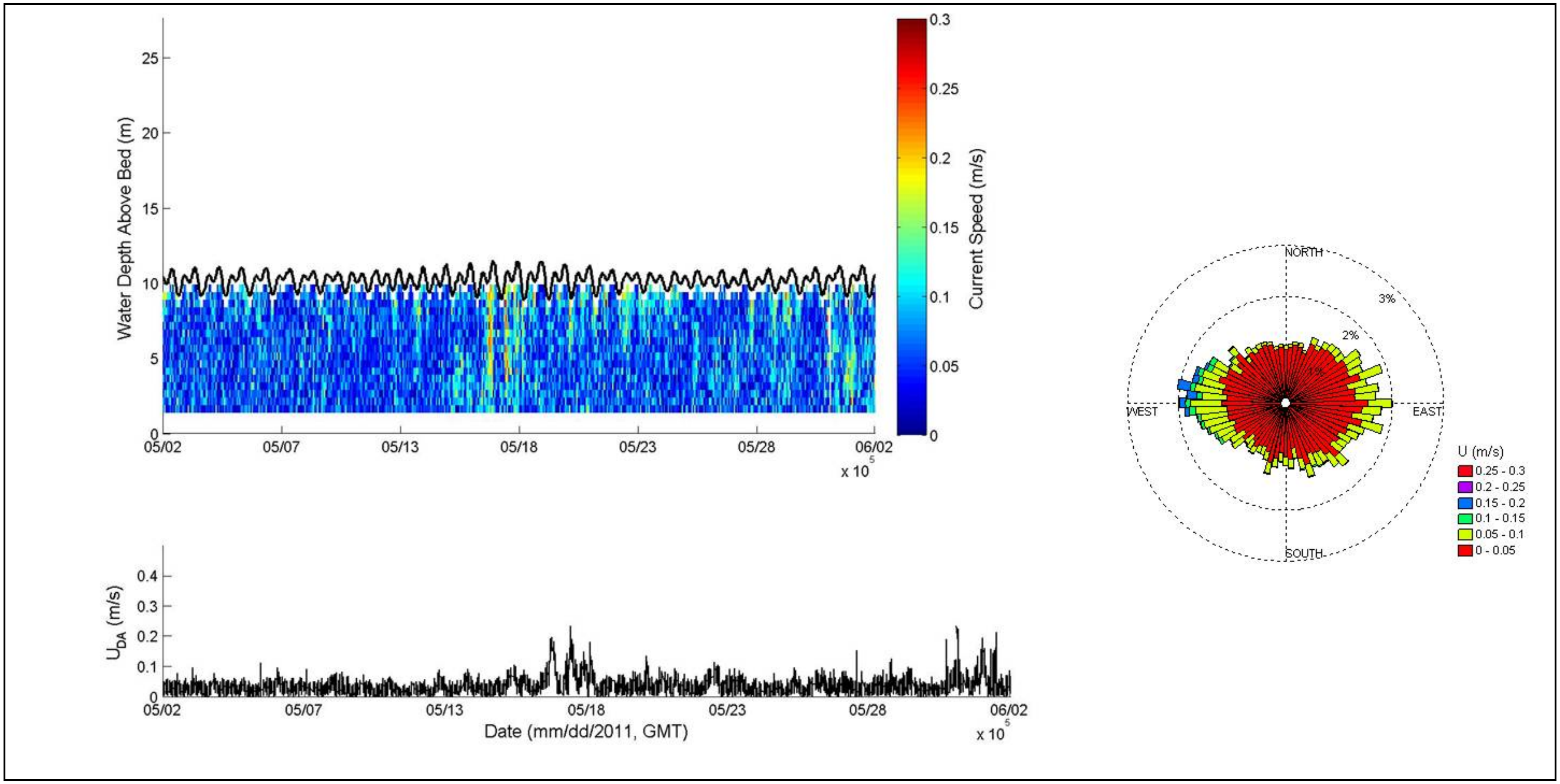

Figure 20. Vertical profile of currents at the Nearshore Mooring between May 2 and June 2, 2011

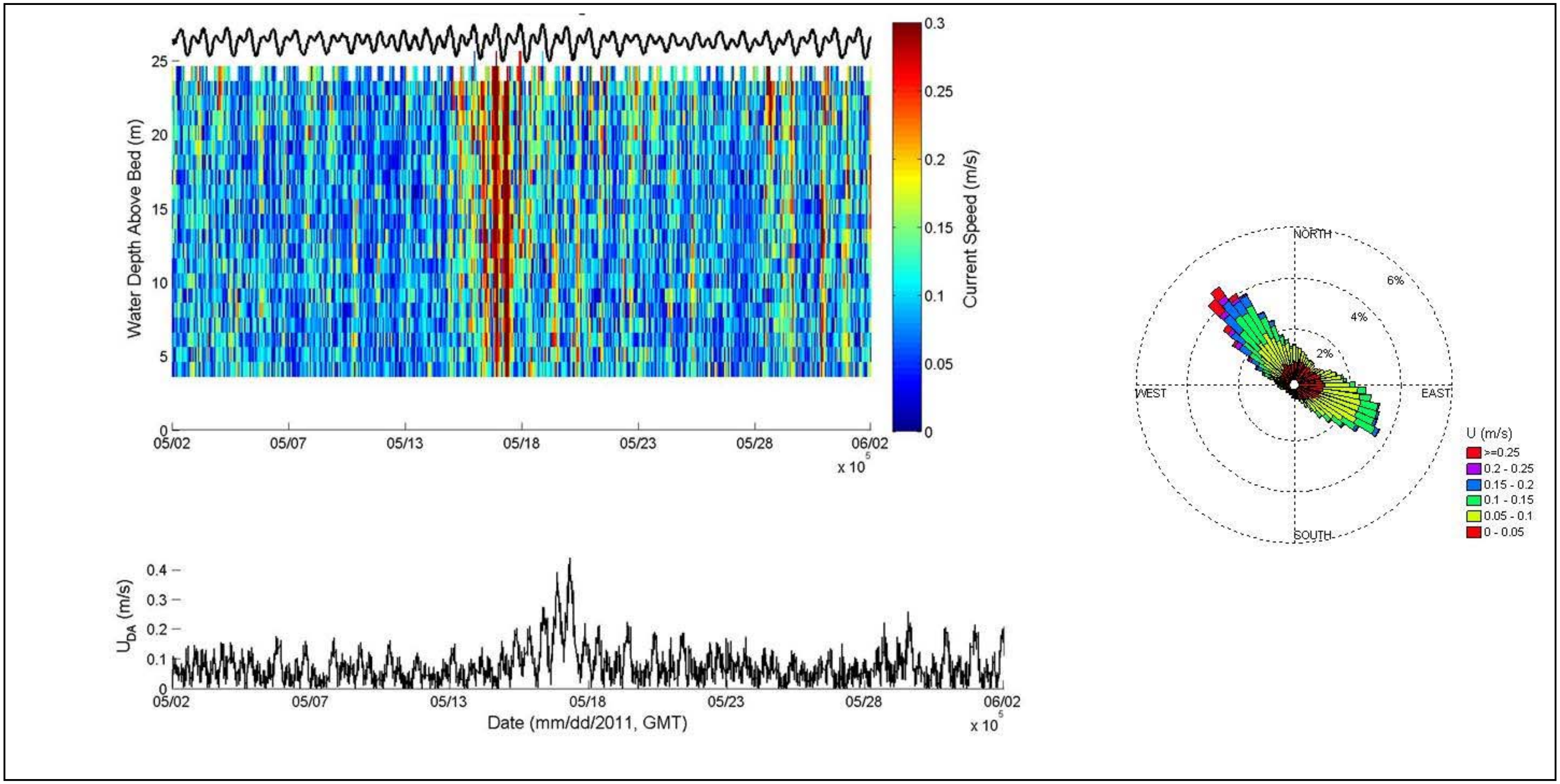

Figure 21. Vertical profile of currents at the Offshore Mooring between May 2 and June 2, 2011

2.5 Sediment Characteristics, Sources and Transport

While sediment bed characteristics have not been extensively studied in the region, there is limited data that can employed to draw conclusions about the study area. Multibeam backscatter data collected by the USGS provides a good overview of the study area seafloor, and clearly shows rocky outcrops and coarse and fine grained sediments in the in the nearshore areas (USGS, 2011; Figure 22).

This dataset is consistent with results from several USACE-sponsored sediment sampling analyses in the 1960s and 1970s (USACE, 1996). These analyses suggest that there are several layers of sediment underlying the study area, with the overall sediment thickness ranging from 20 ft (6 m) along the beach to 160 ft (49 m) in deeper sections of Half Moon Bay. The top layer of sediment is comprised of loose silty sand, which overlays a layer of dense clayey sands and sandy clays. This sandy clay layer overlays a 7 to 20 ft (2.1 to 6.1 m) layer of dense fine sand, which overlays a seaward dipping fracture sedimentary rock (grey siltstone).

An analysis of sediments underlying the east breakwater foundation indicated the bed was composed of beach quality sands that extended to a depth of approximately 21 ft (6 m) at the shore ranging up to 147 ft (45 m) approximately 1,000 ft offshore (USACE, 1986). A stiffer sediment layer of consolidated marine sediments was encountered below the sand, and it is thought to be of similar composition to the exposed cliffs of Pillar Point (USACE, 1986). It is also likely that the sediment bed characteristics found in Pillar Point Harbor and below the east breakwater extend to study area.

Sediment is supplied to the study area by both fluvial delivery from local creeks and longshore transport of sediment eroded from coastal bluffs. Prior to the construction of the east and west breakwaters, the primary sediment source to the study area was erosion of nearby coastal bluffs due to wave action and sediment output from local creeks (Winzler and Kelly, 1984). There are 5 creeks are in the vicinity of the study area, with 3 of the creeks (Deer Creek, Denniston Creek, and a small unnamed creek) draining into the interior of Pillar Point Harbor. Dennison Creek is the only significant source of littoral material in the harbor, with an estimated contribution of 1600 cy/yr (USACE, 1981; 1996). Dennison Creek and the other creeks within Pillar Point Harbor are believed to account for approximately14 percent of the total amount of material shoaling in the interior of the harbor, with the remaining 86 percent thought to come from offshore sources (USACE, 1996).

Sediment transport is primarily driven by hydrodynamic conditions created by incident waves. Incident waves can be focused by shoreline irregularities, submarine canyons and coastal promontories through wave refraction and diffraction. The rocky outcrops and headlands offshore of the project area can cause wave focusing, which in turn may induce sediment transport. In the study area, strong wave-induced longshore currents frequently occur producing a sediment flux in the dominate wave direction (USACE, 1996). The prominent wave direction in the study area (and along much of the California coast) is from the northwest, and there is a general consensus that the net direction of sediment transport is from the north to south (Griggs and Savoy, 1985). The majority of sediment transport in the study area is carried out in the breaker zone, which can range down to depths of 25 ft (7.6 m). However, it is thought that sediment transport can occur at offshore depths as great as 50 ft (15.2 m) (USACE, 1996).

The assumption of net southward directed transport is further supported by photographs taken during construction of the east breakwater in 1959, which show sand accumulation on the harbor (north) side of the breakwater while construction was underway. However, it is believed that there are occasional reversals in sediment transport, as waves associated with strong storms often approach from the south-southwest direction and cause the sediment transport direction to reverse to the north. This northward transport can cause sediment to enter the harbor from the inlet or through the east breakwater (USACE, 1996).

Sediment can also be mobilized by flood and ebb tidal currents and carried into or out of the harbor, respectively. Tidal currents greater than 0.36 ft/s (0.11m/s) are considered strong enough to mobilize the sediments in the study area (USACE, 1996), and the mean measured current at the Harbor Mouth Mooring is at this threshold (0.36 ft/s) with greater velocities during ebb and flood tides. A more detailed discussion of the implications of the construction of the harbor on sediment transport is included in Sections 3 and 4.

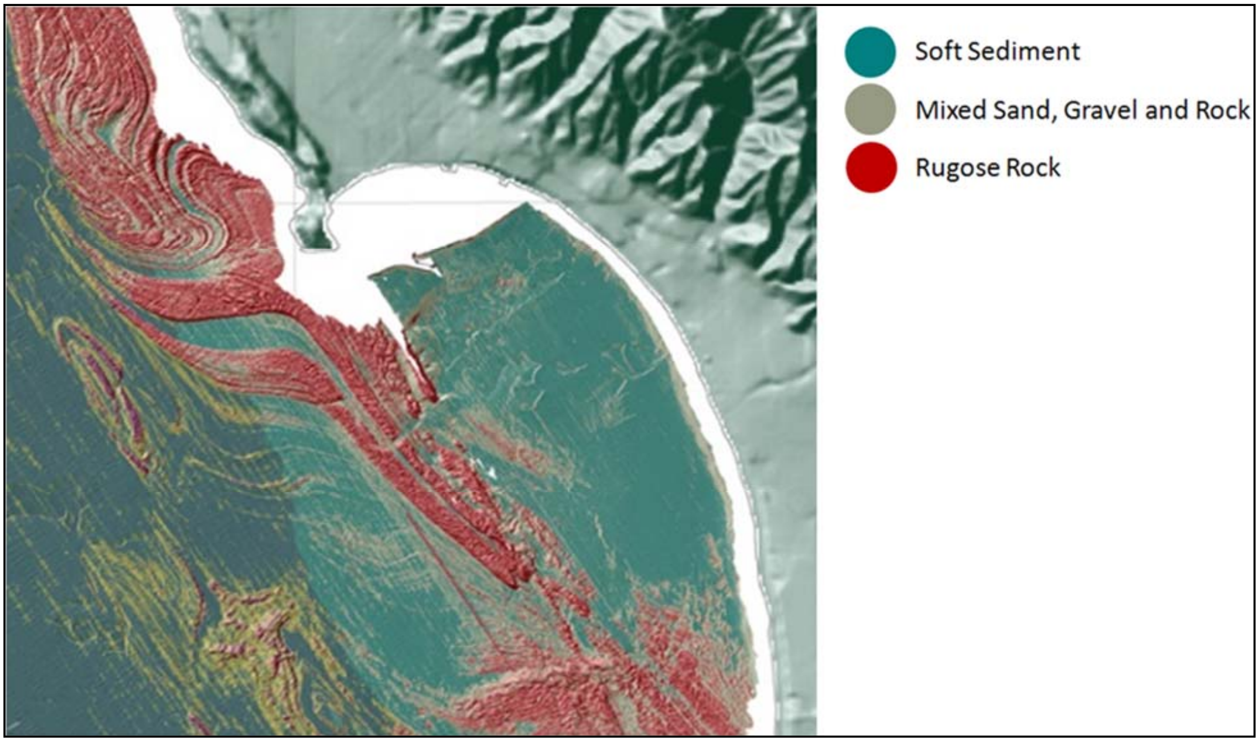

Figure 22. Seafloor sediment characteristics of the project area (USGS, 2011)

2.6 Coastline and Bathymetry

This section briefly describes the major bathymetric features and data sources that were evaluated and used as input for the numerical modeling effort described in Section 3.3.

The extensive rocky outcrops surrounding the north and west sides of Half Moon Bay are well known bathymetric features which dominate the bathymetry in the study area (USACE, 1983). The outcrops appear to be a southeast underwater extension of the formation that culminates in the cliffs of Pillar Point. One formation runs north to south between Pillar Point and Sail Rock and has been described to have the shape of an inverted hammer. A second formation lies immediately south-southwest of the bend of the west breakwater, with another outcrop (Southeast Reef) located approximately 1.5 miles south of the harbor entrance. It is important to note that these outcrops are relatively shallow and can be exposed at extreme low tides.

As discussed in section 2.1.1.3, these rock outcrops provide considerable protection to Pillar Point Harbor and the adjacent shoreline from wave energy approaching from every direction except from the south. This has implications for both the maintenance of the breakwaters, and the wave climate of the study area outside of the protection of the breakwaters. Thus, it is important that the bathymetric input to the numerical model reflect these features that exert a significant influence over wave climate and associated erosion and sediment transport patterns.

The bathymetric data used as input in the numerical modeling effort was derived from several sources, and combined into one bathymetric grid dataset (Figure 23). Much of the nearshore bathymetry in the immediate vicinity of Pillar Point Harbor was derived from a fall 2009 hydrographic survey of the harbor and surrounding areas undertaken by USACE (via a contractor) as part of the Coastal Structure Inspection Program (Towill, Inc., 2011; Figure 24). Additional nearshore bathymetry was derived from surveys undertaken by the California Sea Floor Mapping Program (California State University Monterey Bay, 2011; Figure 25). Offshore bathymetric data were obtained from the GEOphysical Data System (GEODAS) database, which has been developed and managed by the National Geophysical Data Center (NOAA, 2011(c)).

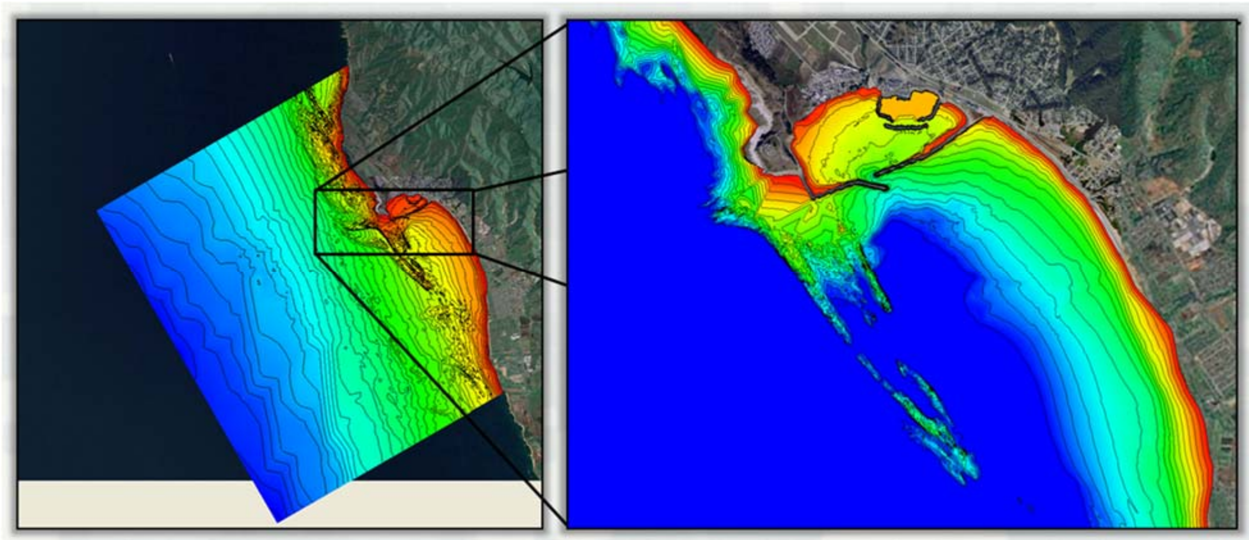

Figure 23. Composite bathymetry from all data sources

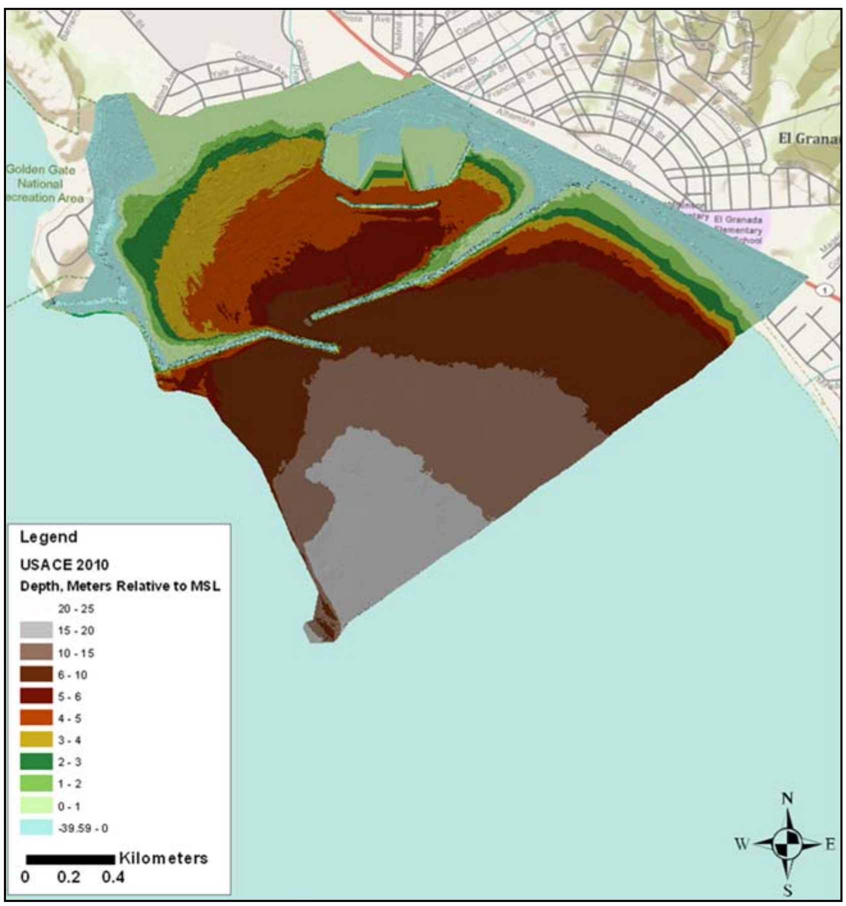

Figure 24. Bathymetry of the Pillar Point Harbor and North Half Moon Bay study area (Towill Inc., 2011)

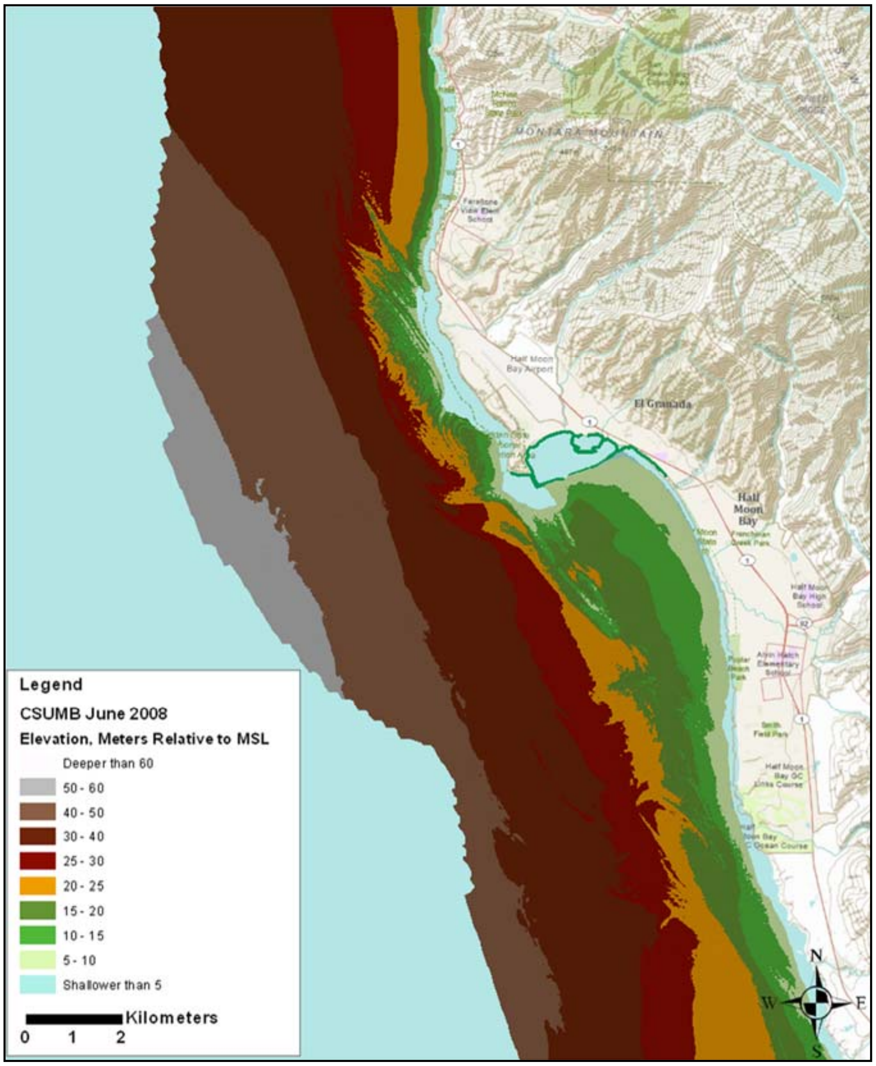

Figure 25. Bathymetry of the offshore extents of North Half Moon Bay study area (California State University Monterey Bay, 2011)

3 Methods of Analysis

This section describes the methods that were utilized to achieve the study objectives.

3.1 Review of Previous Studies and Compilation of datasets

SPN and ERDC Staff reviewed a number of previous studies of coastal processes and beach erosion in the vicinity of Pillar Point Harbor. The review included both USACE studies and reports prepared by other federal government agencies (e.g., USGS) and various consultants, often working on behalf of local municipalities and agencies. SPN and ERDC staff also compiled datasets that characterized physical conditions in the study area including: wave, water level and wind data from NOAA; field measurements of waves, water levels and currents at 4 locations; and nearshore bathymetry from several sources. Please see Sections 2.1 through 2.6 for a detailed description of these data sources.

3.2 GIS Based Analysis

SPN Staff utilized GIS Software (ArcGIS 10.1) for a number of tasks which involved characterizing without-project conditions in the study area. These tasks included an aerial photography based analysis of coastal bluff retreat and computations of quantities of sand available for removal from the harbor under two scenarios.

3.2.1 Coastal Bluff Retreat, 1993 – 2012

In order to quantify and determine the spatial extent of the influence of the east breakwater on bluff retreat, three sections of bluff were selected for this analysis (*Figure* 26). Two of the sections (North and South) were located relatively close to the east breakwater; with a third section (Background) located approximately 3.1 miles (5 kilometers) to the southeast of the breakwater.

The section located closest to the east breakwater is referred to as the North Section, and extends 2,215 ft (675 m) from the Highway 1 revetment southeast to the Mirada Road revetment. This section has been subject to significant bluff retreat since the construction of the east breakwater, and is the primary focus of this study.

The South Section extends approximately 1,560 ft (475 m) southeast from the Mirada Road revetment to the mouth of the small stream adjacent to San Pablo Avenue. The northernmost half of this section is characterized by a low (< 16 foot (5 m) high) bluff, which then transitions into low, ice plant covered dunes south of Alcatraz Avenue.

The Background section extends approximately 3,450 ft (1,050 m) south from Kelly Avenue to Poplar Street. This section is comprised of approximately 33 foot (10 m) high unprotected bluffs, and was selected to establish the “natural” background bluff retreat rate based on recommendations from USACE staff with significant geologic field experience in the study area.

Figure 26. Map depicting the three sections of coastal bluffs addressed by the analysis

Bluff retreat rates for the three sections were established utilizing GIS software to measure changes in the positions of bluff edges from 1993 to 2012. The time period of 1993 to 2012 was selected in order to provide an estimate of current bluff retreat rates, which can be extrapolated to establish a conservative baseline estimate of future retreat rates. Previous studies have documented relatively high bluff retreat rates (1 to 2 m per year) in the two to three decades following the construction of the east breakwater in 1959. However, the current rates of bluff retreat may have slowed in the past two decades, as the shoreline geomorphology may have approached equilibrium with the altered wave conditions. If this is the case, then using the higher rates from previous studies could cause erroneous conclusions to be made regarding the economics and Federal interest viability of the project (based on the future 50-year planning horizon).

The GIS-based analysis involved three steps: (1) selection and georeferencing of aerial imagery, (2) digitization of bluff edges, and (3) analysis of bluff retreat rates using the USGS Digital Shoreline Analysis (DSAS) software (Thieler et al., 2009):

(1) Images from seven vertical aerial surveys from 1993 to 2012 were selected to provide a sufficient sample size to establish current bluff retreat rates (Table 10). Aerial images were selected based on spatial and temporal coverage, and the visibility of the bluff edge. Google Earth served as the primary source of aerial imagery, as this service provided convenient access to relatively high quality aerial imagery. However, there were no images available from Google Earth between 1993 and 2002, and time constraints did not allow for a more thorough search for aerial imagery.

Aerial images were cropped to provide high resolution coverage for the three respective sections, and then georeferenced to a base image with ESRI ArcMap software. This analysis used a USGS High Resolution Orthoimage (HRO) from October 2005 as a base image in order to minimize measurement errors associated with aerial image distortion. The base image had a resolution of 0.15 m, and was projected to the North American Datum (NAD) 1983 California State Plane III (US Feet) coordinate system. The georeferencing process involved selecting approximately 20 common ground control points between the respective aerial image and base image, with the goal of minimizing the reported Root Mean Square (RMS) error (Table). Typical ground control points included road intersections, traffic markings on roadways, corners of buildings, and other features that were assumed to remain in a static location over time. Additional quality control was accomplished by overlaying and comparing the georeferenced images to the base image.

(2) The process of digitizing bluff edges involved a considerable amount of interpretation of aerial imagery, including frequent checks against high resolution oblique aerial photos from the California Coastal Records Project. In general, the tops of coastal bluffs in the study area were

vegetated, with vegetation typically extending all the way to or slightly overhanging the bluff edge. Bluff faces were generally devoid of vegetation, with the relatively light color of the exposed sediments providing a notable contrast with the adjacent vegetation. Thus, the edge of vegetation was often used a proxy for the bluff edge, and digitized at a scale of 1:300 or smaller.

Digitized bluff edges from different dates were also checked against each other to ensure overall consistency in the analysis.

(3) The analysis of bluff retreat rates for the three sections involved using USGS DSAS software to create shore-normal transects, which intersected the time series of bluff edges. Transects were cast from shore-parallel baselines at intervals of 25 m, and a number of shoreline change statistics were computed along those transects. This analysis utilized end point rate (EPR) and linear regression rate (LRR) statistics to characterize bluff retreat rates from 1993 to 2012.

It should be noted that there can be considerable measurement error associated with utilizing aerial imagery and GIS software to measure bluff retreat. This issue has been addressed by several USGS-led studies (Hapke, 2004; Hapke and Reid, 2007; Hapke et al., 2009), which have provided detailed methods for quantifying measurement uncertainty. This analysis utilized the methods detailed in Hapke and Reid (2007) to determine the total bluff position error associated with each bluff edge and annualized end point rate (EPR) errors (Table 10 and Table 15):

$$\textit{Total Bluff Position Error} = \sqrt[2]{Georeferencing\ Error^2 + Digitizing\ Error^2} \quad \textit{(Equation 3.1)}$$

Georeferencing Error = RMS Error

Digitizing Error = Resolution + Bluff Edge Placement Error *(Equation 3.2)*

In addition, DSAS also calculated standard errors, which represent the uncertainty that can be assigned to linear regression rates (Table 15).

Table 10: Aerial imagery used in bluff retreat analysis.							
Date	Source	Section	Approximate Resolution (m)	Ground Control Points	RMS Error (m)	Total Bluff Position Error (m)	Total Bluff Position Error (ft)
19 April 1993	California Coastal Records Project	North	0.6	24	0.49	1.86	6.10
		South	0.6	20	0.26	1.87	6.14
		Background	0.6	22	0.30	2.46	8.07
10 Oct 2003	United States Geological Survey	North	0.3	-	-	0.91	2.99
		South	0.3	-	-	1.52	4.99
		Background	-	-	-	-	-
11 October 2005	United States Geological Survey (Used as Base Image)	North	0.15	-	-	0.46	1.51
		South	0.15	-	-	1.37	4.49
		Background	0.16	-	-	0.76	2.49
18 February 2007	Google Earth (DigitalGlobe)	North	0.3	20	0.15	2.14	7.02
		South	0.3	20	0.13	2.13	6.99
		Background	0.6	22	0.25	2.45	8.04
30 September 2009	Google Earth (DigitalGlobe)	North	0.3	24	0.14	0.92	3.02
		South	0.3	26	0.14	1.52	4.99
		Background	0.6	24	0.27	1.85	6.07
1 May 2011	Google Earth	North	0.3	22	0.15	0.92	3.02
		South	0.3	20	0.14	1.53	5.02
		Background	0.6	22	0.26	1.85	6.07
19 May 2012	Google Earth	North	0.4	26	0.15	1.13	3.71
		Background	0.6	22	0.26	1.85	6.07

3.2.2 Computations of Available Sand and Beach Fill Volumes

Recent hydrographic surveys and visual observations indicate that a large shoal has formed on the northwest (harbor side) of the east breakwater (Gahagan & Bryant Associates, 2007), and there has been considerable interest in removing sand from this shoal and placing it on Surfers Beach. As a result, GIS software was utilized to define potential sand removal and placement footprints and compute the quantity of sand that could be removed from this shoal down to a depth (-10 ft, NAVD 88) that approximately matched the adjacent harbor bed (Figure 27). This analysis was run for 2 sediment removal scenarios based on the total amount of sand that could be removed, and a more “realistic” sand removal strategy (see Sections 6.1 and 6.2).

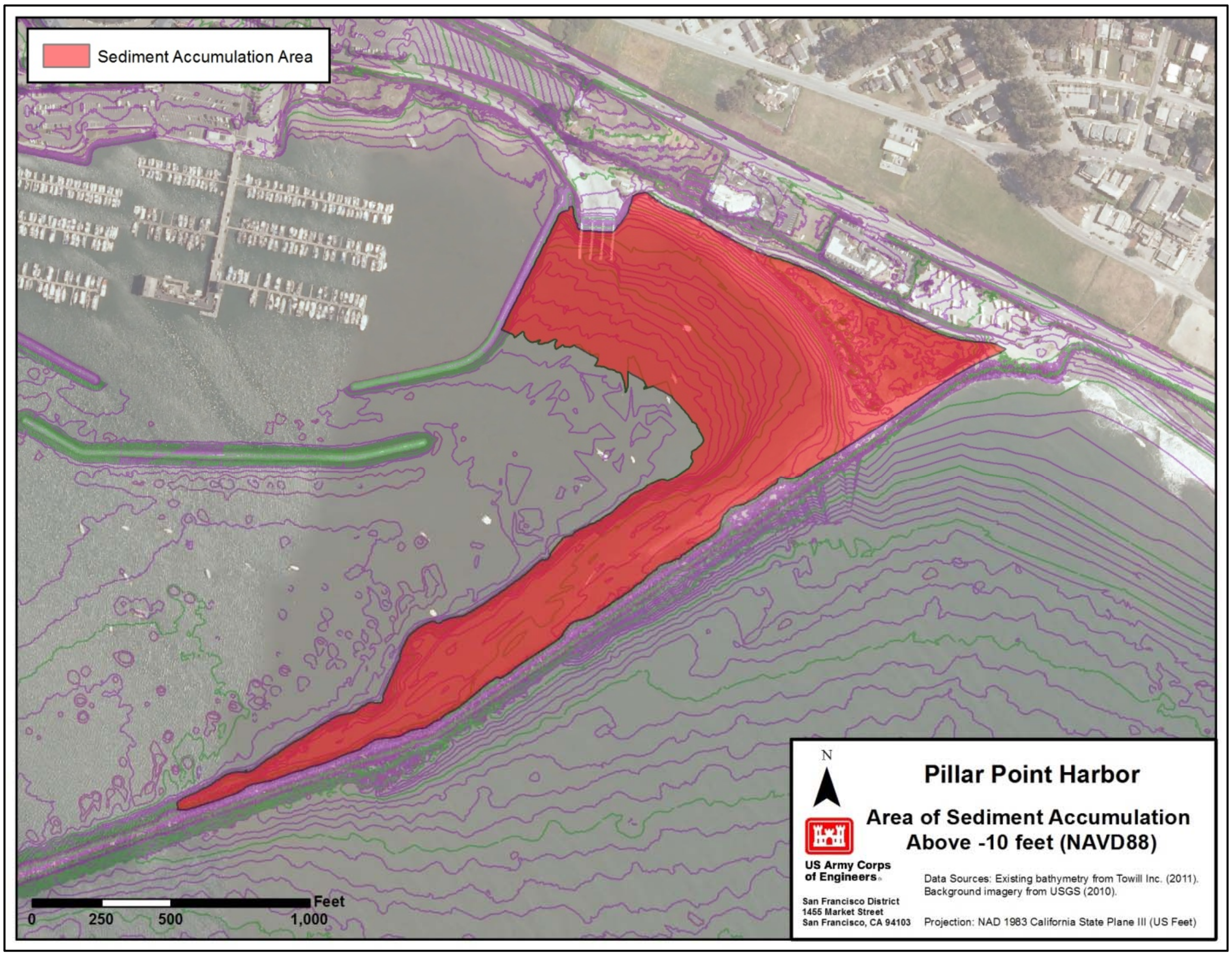

Figure 27. Area of excessive sediment accumulation and shoaling in Pillar Point Harbor

In the first step, contours derived from a fall 2009 hydrographic and topographic survey (Towill Inc., 2011) were utilized to create a triangulated irregular network (TIN) surface of the study area. This TIN was then used to delineate the shoal area above the -10 foot (NAVD 88) contour, which was further defined into sand removal footprints (polygons) associated with the 2 scenarios. These polygons and the TIN were then converted into raster files, with the polygons assigned an elevation of -10 foot (NAVD88). In the final step, the existing surface and -10 ft (NAVD 88) plane rasters were utilized to perform a cut-fill analysis, which defined the volume of available sand above the -10 foot (NAVD 88) plane. In addition, GIS software was also used to develop the layout and geometry of two potential beach fill placements, and the details of this process are described in Section 6.1.

3.3 CMS Model Analysis

The Coastal Modeling System (CMS) is a suite of major multidimensional numerical models integrated to simulate waves, currents, water levels, sediment transport and morphology change in coastal inlets, estuaries, and harbors. The CMS, developed by the U.S. Army Corps of Engineers Coastal Inlets Research Program (CIRP), consists primarily of two modeling modules, CMS-Wave and CMS-Flow (USACE, 2014c). Two-way coupling between CMS-Flow and CMS-Wave can be operated through a steering module within the Surface-water Modeling System (Zundel, 2007) to dynamically simulate sediment transport and morphology change (Buttolph et al. 2006; Lin et al. 2008). CMS can also be coupled with a particle tracking model (PTM) (Demirbilek et al. 2008) to compute the fate and pathways of sediment and other waterborne particles from the simulated wave environment, flow field and water exchange via CMS-Wave and CMS-Flow.

CMS-Wave is a two-dimensional (2D) wave spectral transformation model implemented in the Coastal Modeling System. The model employs a forward-marching, finite-difference method to solve the wave action conservation equation. It is a phase-averaged model, which averages changes in the wave phase to calculate wave properties and is based on the wave-action balance equation as:

$$\frac{\partial(C_x N)}{\partial x}+\frac{\partial(C_y N)}{\partial y}+\frac{\partial(C_\theta N)}{\partial \theta}=\frac{\kappa}{2\sigma}[(CC_g\cos^2\theta N_y)_y-\frac{CC_g}{2}\cos^2\theta N_{yy}]-\varepsilon_b N-S$$

(Equation 3.3)

Where $N=\frac{E(\sigma,\theta)}{\sigma}$

is the wave-action density that is a function of frequency σ and direction θ; x and y are the horizontal coordinates; C and C_g are the wave celerity and group velocity; C_x, C_y and C_θ are the characteristic velocity with respect to x, y and θ; N_y and N_{yy} denote the first and second derivatives of N with respect to y; κ is an empirical parameter representing the intensity of diffraction effect; ε_b is the parameterization of wave breaking energy dissipation; and S denotes the additional sources and sinks such as wind forcing and bottom friction loss.

CMS-Wave has theoretically derived approximations for wave refraction, shoaling, diffraction, reflection, and wave-current interaction, and therefore, is appropriate for conducting wave simulations at coastal inlets with jetties and in harbor entrances with breakwaters. It employs a forward-marching, finite-difference, steady-state (time-independent) Eulerian method to solve the wave action conservation equation. Wave diffraction is implemented by adding a diffraction term derived from the parabolic wave equation to the energy-balance equation (Mase et al. 2005). CMS-Wave can operate either on a coastal half-plane or full-plane with primary waves propagating from the seaward boundary toward shore. Shoreward and seaward reflections are treated using a mirror reflection principle.

CMS-Flow is a three-dimensional (3D) finite-volume model that solves the mass conservation and shallow-water momentum equations of water motion on a non-uniform Cartesian grid. The model simulates currents, water level and sediment transport to characterize the water circulation pattern and morphology change in the coastal zone. The model can be executed in a 2D mode based on the depth-integrated continuity equation, which was applied in the present study. The 2D depth-integrated continuity and momentum governing equations are:

$$\frac{\partial(h+\eta)}{\partial t}+\frac{\partial q_x}{\partial x}+\frac{\partial q_y}{\partial y}=0 \qquad \textit{(Equation 3.4)}$$

$$\frac{\partial q_x}{\partial t}+\frac{\partial u q_x}{\partial x}+\frac{\partial v q_x}{\partial y}+\frac{1}{2}g\frac{\partial(h+\eta)^2}{\partial x}=\frac{\partial}{\partial x}D_x\frac{\partial q_x}{\partial x}+\frac{\partial}{\partial y}D_y\frac{\partial q_x}{\partial y}+fq_y-\tau_{bx}+\tau_{wx}+\tau_{sx}$$

(Equation 3.5)

$$\frac{\partial q_y}{\partial t}+\frac{\partial u q_y}{\partial x}+\frac{\partial v q_y}{\partial y}+\frac{1}{2}g\frac{\partial(h+\eta)^2}{\partial y}=\frac{\partial}{\partial x}D_x\frac{\partial q_y}{\partial x}+\frac{\partial}{\partial y}D_y\frac{\partial q_y}{\partial y}+fq_x-\tau_{by}+\tau_{wy}+\tau_{sy}$$

(Equation 3.6)

where q_x and q_y are the flow per unit width parallel to the x and y axis; η is the water surface elevation from the still water level; h and t are the still water level and time; u and v are the depth-averaged current velocities parallel to the x and y axis; D_x and D_y are the diffusion coefficients; f is the Coriolis parameter; τ_{bx} and τ_{by} are the bottom stress parallel to the x and y

axis; τ_{wx} and τ_{wy} are the surface stress parallel to the x and y axis; and τ_{sx} and τ_{sy} are the wave stress parallel to the x and y axis.

The wave radiation stress and wave information entering the flow and sediment transport formulas are supplied to CMS-Flow through coupling with CMS-Wave. Calculated currents and water level changes from CMS-Flow are in turn input to the wave model to increase the accuracy of the wave transformation prediction (Buttolph et al. 2006).

3.3.1 Model Domain

Figure 28 shows the CMS domain for this modeling study. The area covers Pillar Point Harbor (the inner and the outer breakwaters), the entire Half Moon Bay, and the offshore region. The model domain extends approximately 6 kilometers (3.7 miles) from east to west and 12 kilometers (7.5 miles) from north to south and the western open boundary reaches a depth of 40 m (130 ft). A non-uniform rectangular grid system with more than 40,000 grid cells was created to depict the nearshore and the offshore region. The grid system permits much finer resolution (20 m or 66 ft) in areas of high interest such as the harbor area. The coarser grid of 150 m (490 ft) has been used near the offshore boundaries. The outer breakwaters were specified as permeable structures that allow for wave transmission and flow seepage (surging) into the harbor.

3.3.2 Simulation Period and Model Forcing

To investigate shoreline changes related to the harbor construction, CMS simulations under the existing (post-harbor) and the pre-harbor conditions were set up for a one-year period from June 2009 to May 2010. The CMS calibration was performed for May 2011, during which four current meters were deployed near Romeo Pier inside the harbor, at the harbor mouth, at the shallow area outside the harbor, and at an offshore area (Figure 3 and Table 3). These gages collected water surface elevation, waves, and current measurements, and the results of the calibration effort are summarized in Section 3.3.3.

The CMS input included measurements of wind, waves, and water surface elevation at selected stations. Incident wave spectra were transformed from three buoys (46012, 46026, and 46042) in the study area to the CMS-Wave grid offshore boundary using weighted averages and linear wave theory with a simple assumption of shore-parallel depth contours (Figure 2). The resulting model H_S and T_P inputs from the incident wave spectra generally followed their measured counterparts well, and were within 1 standard deviation of the measured data (Figure 29, Figure 30, Table 11; Table 12).

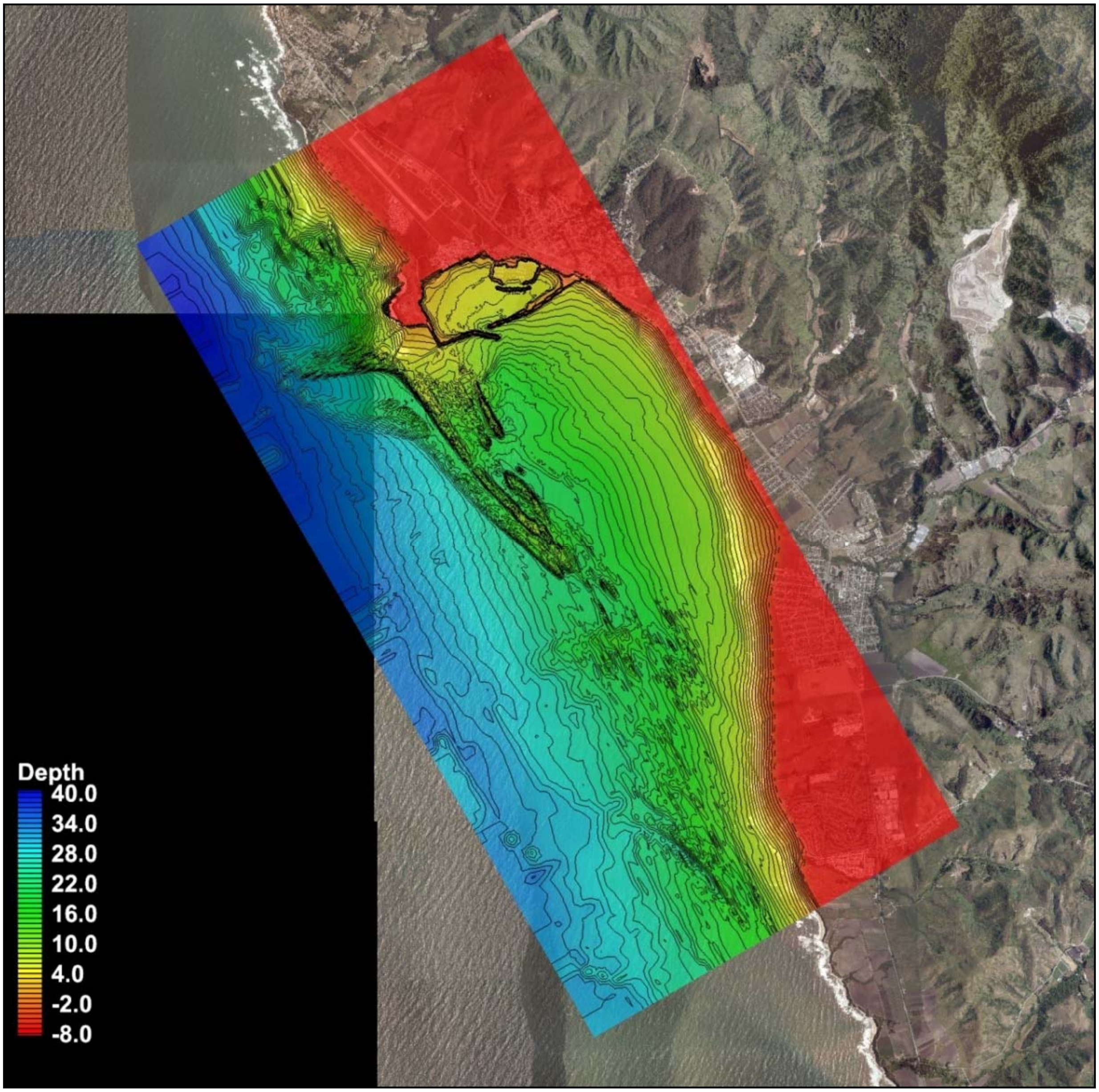

Figure 28. CMS domain and water depth (m). Note that all depths in the CMS model are referenced to Mean Sea Level (MSL) at the Monterey Tide Station (9413450).

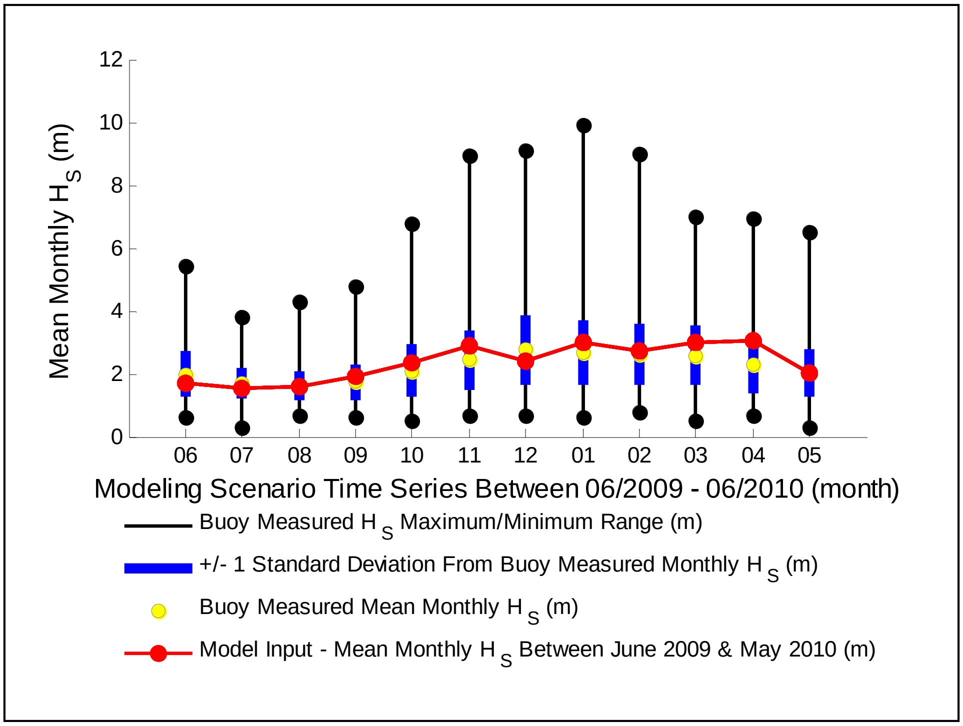

Figure 29. Mean monthly H_S from NBDC Buoy 46042

Table 11: Mean Monthly H_S (NBDC Buoy 46042) Compared with Mean Monthly Modeled H_S

Month of Modeling Scenario	Buoy Measurement - 1 Standard Deviation (m)	Buoy Measurement	Buoy Measurement + 1 Standard Deviation (m)	Modeling Scenario - Buoy Measurement(m)
06/2009	1.29	2.00	2.71	1.71
07/2009	1.19	1.69	2.18	1.54
08/2009	1.14	1.61	2.08	1.60
09/2009	1.17	1.74	2.31	1.92
10/2009	1.27	2.11	2.94	2.34
11/2009	1.47	2.44	3.40	2.92
12/2009	1.65	2.77	3.88	2.38
01/2010	1.62	2.65	3.68	3.02
02/2010	1.66	2.64	3.62	2.74
03/2010	1.63	2.59	3.55	3.00
04/2010	1.40	2.29	3.19	3.03
05/2010	1.28	2.03	2.78	2.03

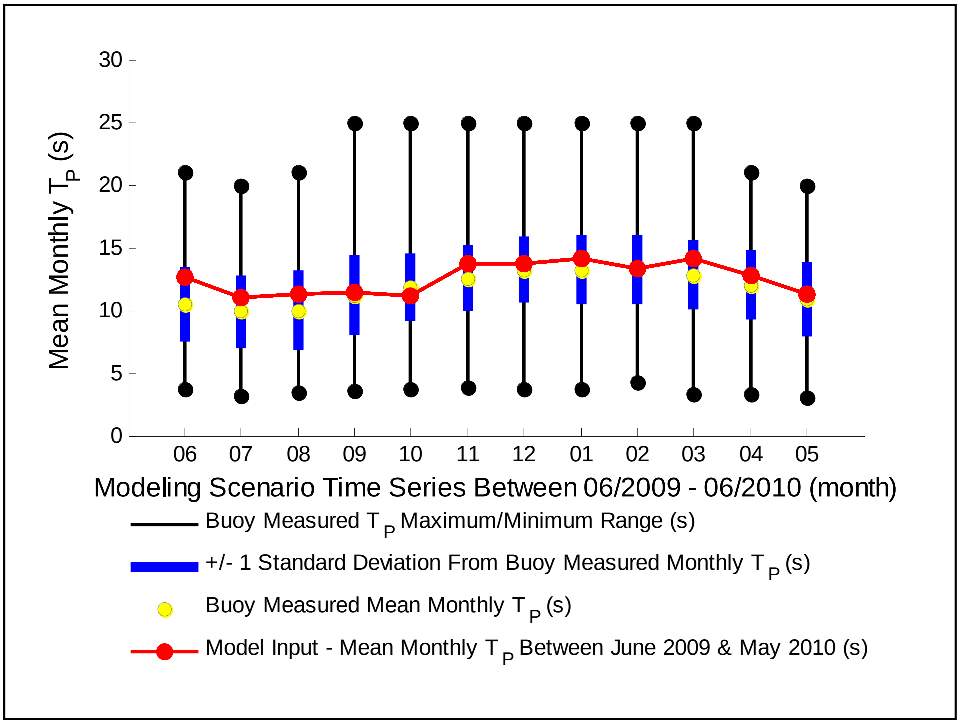

Figure 30. Mean monthly T_P from NBDC Buoy 46042

Table 12: Mean Monthly T_P Compared with Mean Monthly Modeled T_P.

Month of Modeling Scenario	Buoy Measurement - 1 Standard Deviation (s)	Buoy Measurement (s)	Buoy Measurement + 1 Standard Deviation (s)	Modeling Scenario - Buoy Measurement
06/2009	7	10	13	13
07/2009	7	10	13	11
08/2009	7	10	13	11
09/2009	8	11	14	11
10/2009	9	12	15	11
11/2009	10	13	15	14
12/2009	11	13	16	14
01/2010	10	13	16	14
02/2010	10	13	16	13
03/2010	10	13	16	14
04/2010	9	12	15	13
05/2010	8	11	14	11

Measured water surface elevations were extracted from the NOAA station (9413450) at Monterey (Figure 2), which provides a reasonable representation of water level conditions in the study area despite it's somewhat distant location (see Section 2.1). Wind input information was acquired from two different datasets, with the offshore wind derived from NDBC Buoy 46012 and the coastal wind derived from the Half Moon Bay airport (USAF 720646). The wind conditions varied between the offshore and coastal stations, with greater wind speeds recorded in the offshore region due to essentially no fetch limitations (Figure 31). Thus, it is expected that model simulations utilizing the offshore wind data will not necessary verify all that well against field measurements closer to shore in the study area.

3.3.3 Calibration

CMS-Wave was calibrated at 3-hour intervals with incident wave spectra and water surface elevations, which were specified at the open ocean boundaries. Wind inputs from NDBC Buoy 46012 and Half Moon Bay Airport (USAF 720646) were applied to the entire model domain. The wave diffraction intensity in (Equation 3.3) was set to 4.0 and the bottom friction was neglected in the calibration, with wave run-up specified at the inner and the outer breakwaters and along the shoreline of the bay.

Figure 32, Figure 33, and Figure 34 shows the comparisons of wave height, wave period, and wave direction between the CMS-Wave results and field measurements at the offshore, the nearshore, and the harbor mouth deployment locations. Table 13 presents the model bias in model prediction compared to the field measurements of wave height. This comparison indicates that calculated wave height, period, and direction agree very well with the field data although CMS-Wave slightly under-predicts the wave height. At the harbor mouth, swells are mostly blocked by the harbor structures, and random noises due to wind disturbance are shown in the period and direction measurements (Figure 34). In contrast to the three other field moorings, the percent difference is relatively large at this location. However, this relatively high percentage difference is expected, given that this small wave height (0.09 m) is approaching the random noise levels (error) in the measurements.

Table 13: Comparison of Calculated Wave Height at Mooring Locations

Mooring	Calculated Hs (m)	Measured Hs (m)	Model Bias
Offshore Mooring	1.63	1.69	-0.06 m (-0.20 ft)
Nearshore Mooring	0.80	0.81	-0.01 m (-0.03 ft)
Harbor Mouth Mooring	0.09	0.14	-0.05 m (-0.16 ft)

Figure 35 shows the comparison of modeled and measured water surface elevations (WSEs) at the four instrument deployment locations for May 2011. As Figure 35 suggests, the CMS performed well in predicting WSEs during this time period. The current measurements exhibited different flow patterns inside and outside the harbor (Figure 36). The depth averaged current at the Offshore Mooring has an average speed of about 0.11 m/s (0.36 ft/s) and the peak speed during the period is about 0.45 m/s (1.48 ft/s). These measurements and the CMS outputs show that the offshore currents are tidally dominated and affected by wind, which predominantly flows from northwest to southeast. However, the tidal signal in the CMS current output is not as clear as the signal from the measurements (Figure 36).

The depth averaged current at the Harbor Mouth Mooring has a speed of 0.11 m/s (0.36 ft/s), and clearly shows the flood and ebb tidal current pattern in the entrance channel. Table 14 and Figure 36 show the comparison of the calculated and measured average current speeds at the mooring locations, and suggest that CMS overestimates the current speeds outside the harbor and underestimates current speeds at the Romeo Pier inside the harbor.

Table 14: Comparison of Calculated and Measured Current Speed at Mooring Locations

Gage	Calculated Average Current Speed (m/s)	Measured Average Current Speed (m/s)	Model Bias
Offshore	0.12	0.11	0.01 m/s (0.03 ft/s)
Nearshore	0.06	0.06	0.00 m/s (0.00 ft/s)
Harbor Mouth	0.07	0.11	-0.03 m/s (-0.10 ft/s)
Romeo Pier	0.03	0.04	-0.01 m/s (-0.03ft/s)

Figure 37 and Figure 38 depict snapshots of the depth-averaged water circulation field inside Pillar Point Harbor during the flood and ebb tide cycles, respectively. These figures demonstrate that strong tidal currents occur at the harbor entrance during the peak flood and ebb cycles, with current speeds ranging from 20 and 30 cm/sec (0.66 to 0.98 ft/sec). Within the harbor, the flood current splits into two branches as it approaches Romeo Pier. One branch flows eastward, passes the inner harbor, and turns back towards the entrance along the eastern breakwater. Another branch flows westward and turns south towards the entrance along the western breakwater. Under the ebb tide condition, the harbor water flows out along the eastern and western breakwaters, and through the middle of the harbor. The circulation pattern generated by the CMS is consistent with that found in a Pillar Point Harbor circulation study by Wuertz et al. (2011).

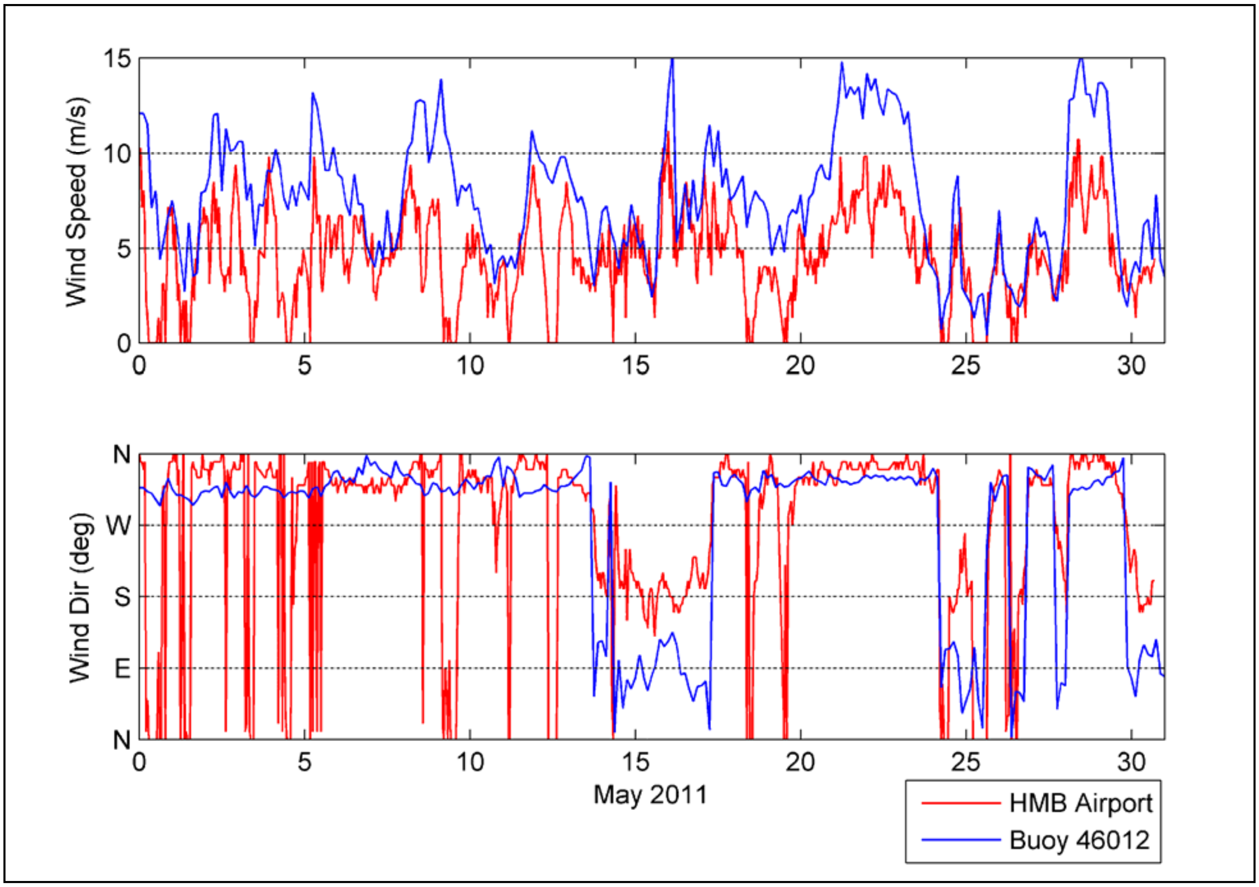

Figure 31. Offshore and coastal wind measurements

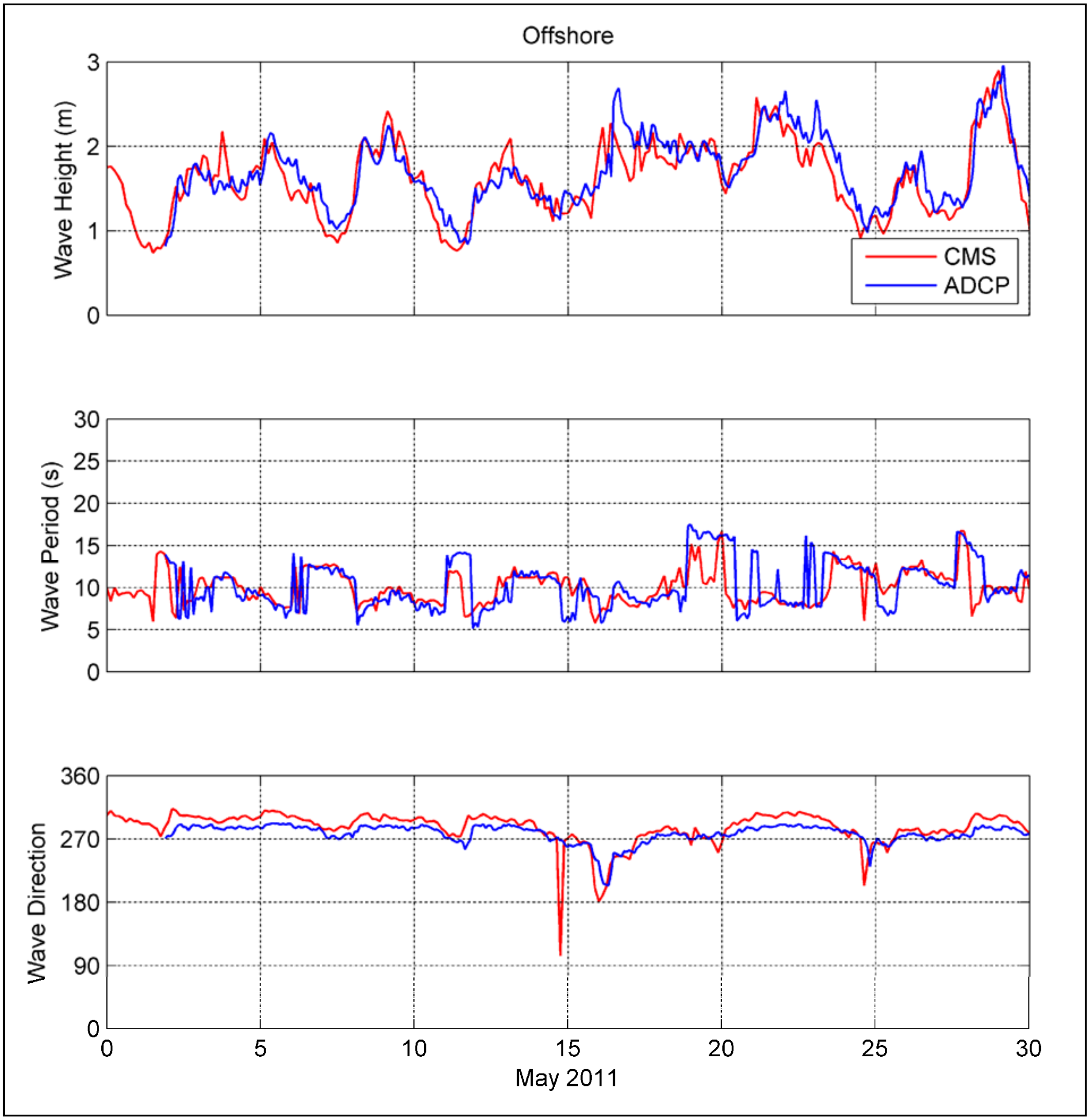

Figure 32. Wave parameter comparisons at the Offshore Mooring

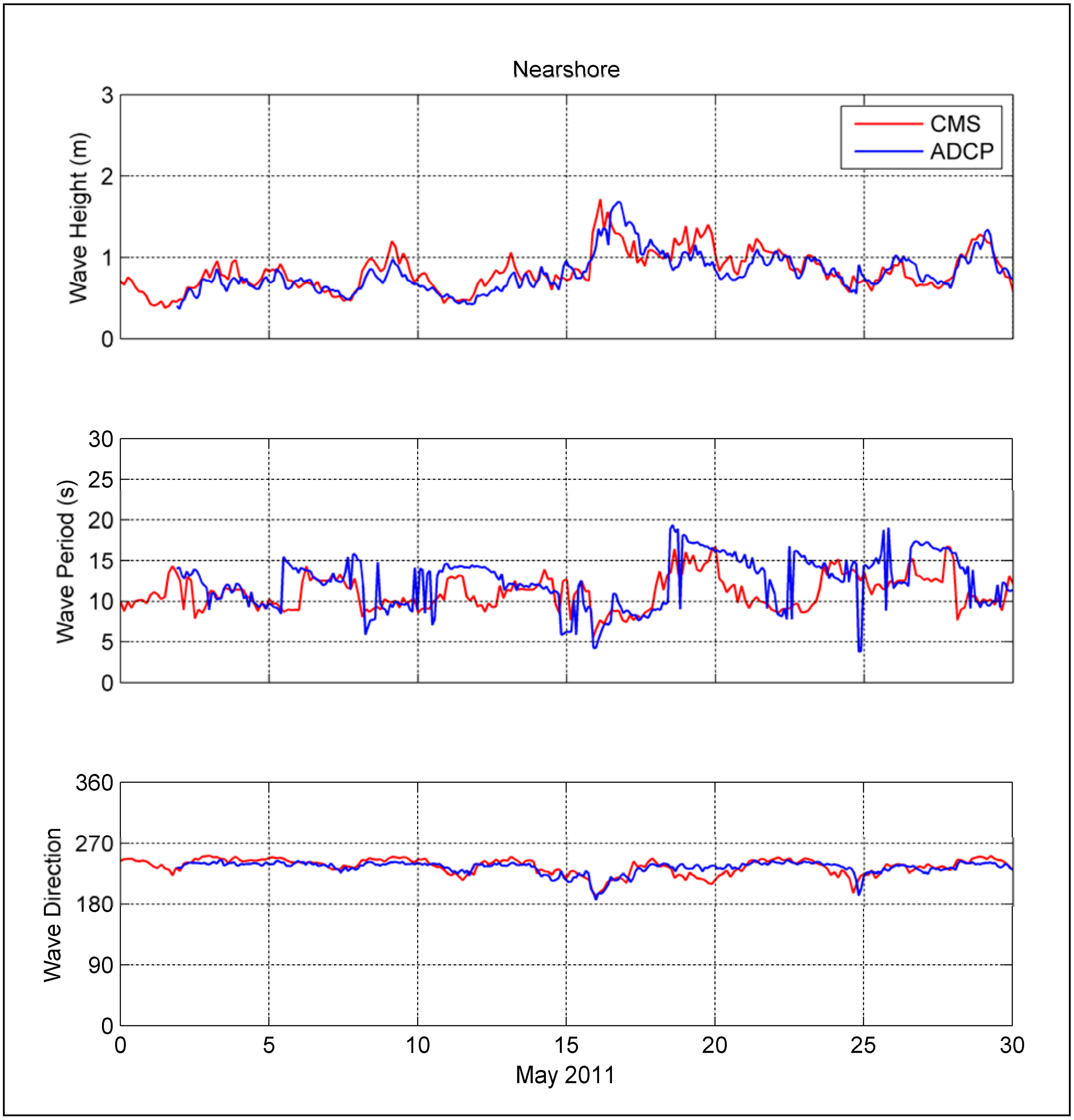

Figure 33. Wave parameter comparisons at the Nearshore Mooring

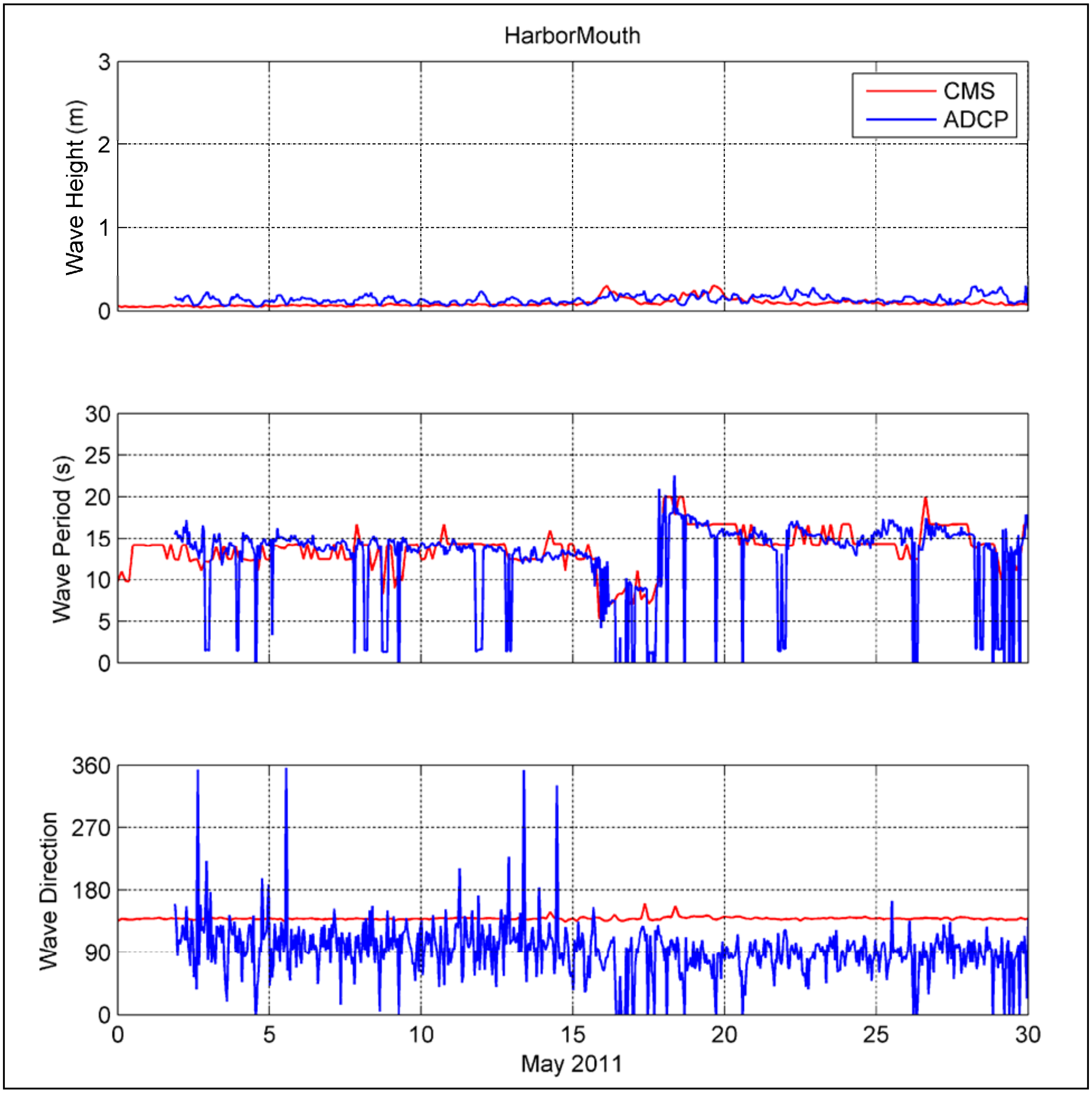

Figure 34. Wave parameter comparisons at the Harbor Mouth Mooring

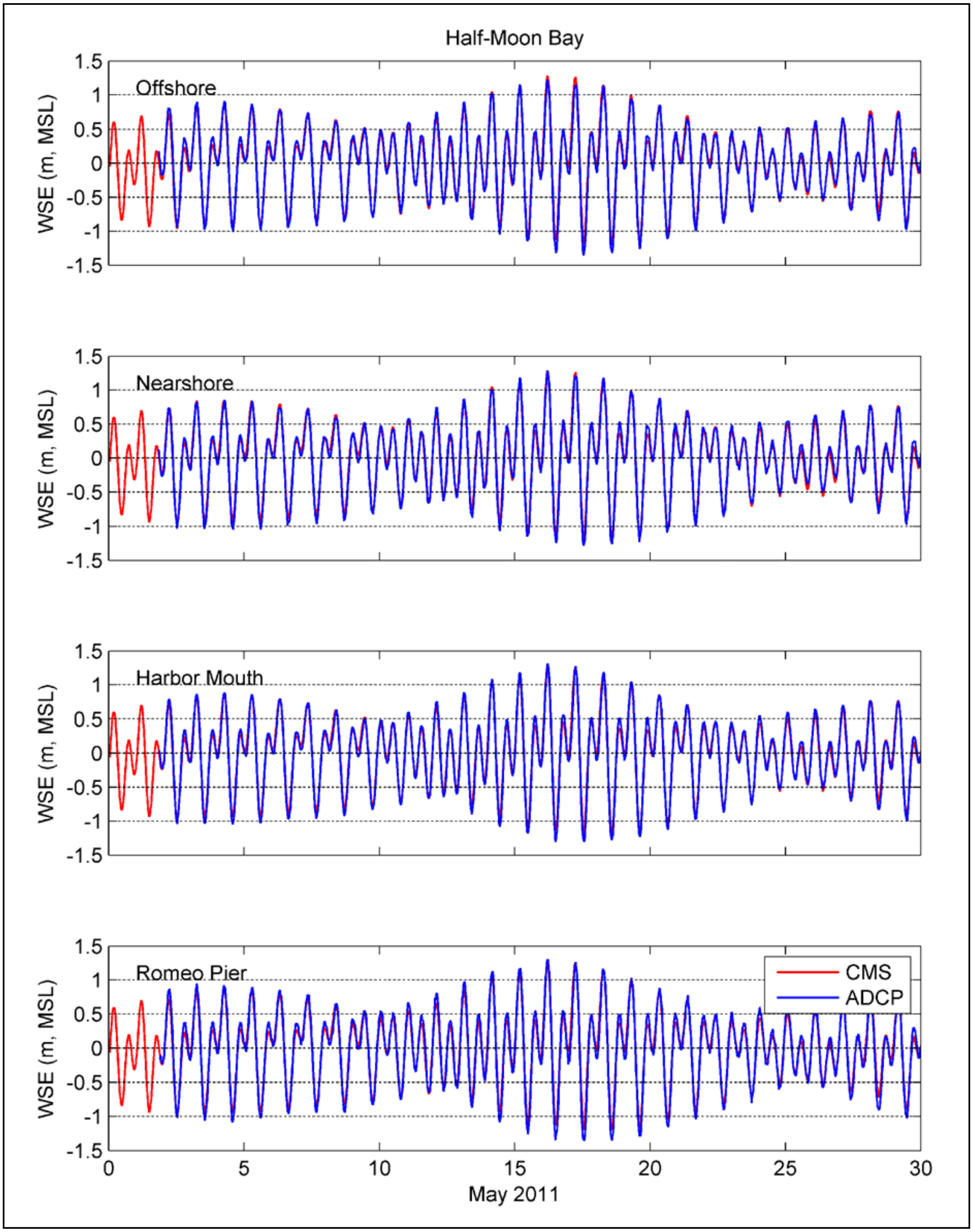

Figure 35. Water surface elevation comparisons at the instrument deployment locations

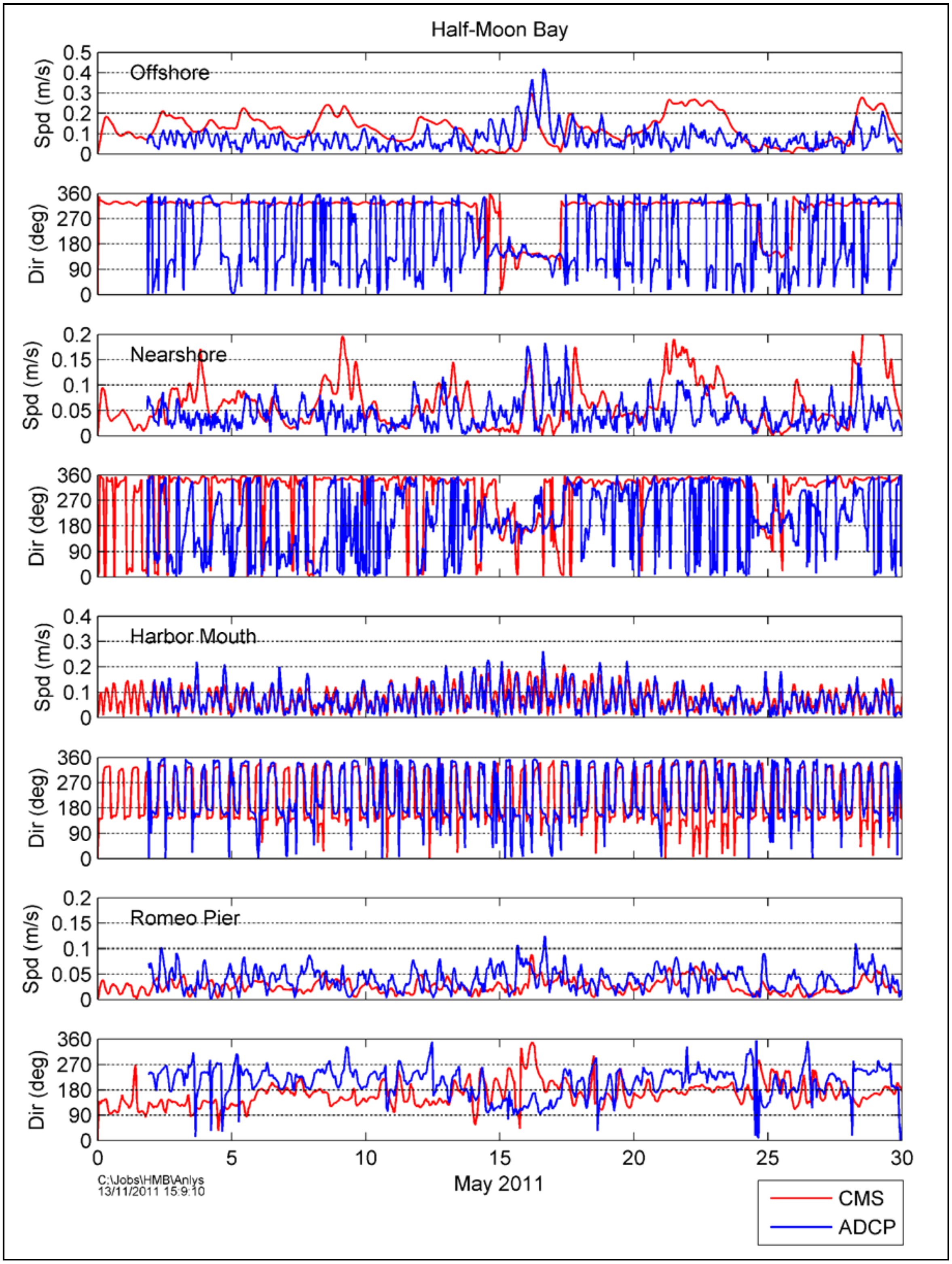

Figure 36. Current comparisons at the instrument deployment locations.

Figure 37. Circulation in Half Moon Bay during the flood tide

Figure 38. Circulation in Half Moon Bay during the ebb tide

3.3.4 Uncertainty and Interpretation of Model Results

In order to provide some context for understanding the CMS model results, it is necessary to consider the uncertainties associated with these results. Model uncertainties can be quantified through a validation process, where the modeled results for a given parameter (e.g., wave height) are compared to field measurements at the study site. Field calibration with measurements from four deployment locations was completed for several hydrodynamic parameters in 2011 (see Section 3.3.3), but not for sediment transport or morphology (bed) change.

Of the modeled hydrodynamic parameters, the water level was the most accurate and closely followed measurements from the four instrument deployment locations (Figure 3). The modeled wave height was also reasonably accurate, with the model slightly under-predicting wave height (by less than 4%) outside of the harbor (Figure 32; Figure 33). Predicted wave heights inside of the harbor were less accurate, possibly due to random noise in the field measurements from wind (Figure 34) and the small signal. The modeled current speeds were less accurate than wave heights, with the model slightly over-predicting current measurements outside of the harbor (< 10%), and under-predicting the current speeds in the harbor and at the mouth by approximately 20 to 30% (Figure 36).

Sediment transport and morphology change outputs were not validated against field measurements. The sediment transport formulas utilized by the model were based on empirical data from laboratory work conducted at ERDC (Buttolph et al., 2006), although there can be significant differences between the laboratory and "real world" conditions (Lin, pers comm., 2014). Field based validation of modeled nearshore morphology change has not been completed to date, and the interpretation of the predicted morphology change often requires the application of significant engineering judgment (Lin, pers comm., 2014). Thus, it is not possible to quantify the uncertainty associated with the morphology change results presented in this document.

Given the above uncertainties, there are two important factors that should be considered when interpreting the modeling results presented in this document. First, the tendency of the model under-predict current speeds in the harbor suggests that the model will also over-predict shoaling rates. Thus, the in-harbor shoaling rates predicted by the model can be considered a conservative (worst-case scenario) prediction of shoaling and associated impacts to navigation. Second, the predicted morphology change should be viewed as a conceptual level prediction of major morphology trends such as shoaling and the erosion of recently placed beach fill. In addition, the comparison of design measures against each other should be more accurate than an absolute quantification of each measure because the model bias errors should cancel out when comparing measures against each other. Thus, the relative magnitudes of the morphology changes associated with different design measures can provide valuable information regarding the expected performance of these measures.

4 Results of Without-Project Condition Analysis

The objective of the without-project condition analysis is to characterize present and future physical conditions at the project site in the absence of any Federal action to address beach and bluff erosion. 2 methods of analysis were utilized to accomplish this objective, including GIS-based DSAS analysis of recent bluff erosion rates, and numerical modeling of present and future without-project condition scenarios. These modeled scenarios include a hypothetical pre-breakwater scenario, which was compared to the existing (with-breakwater) scenario to evaluate the impacts of the breakwaters on erosion and accretion at the site. In addition, the potential impacts of sea level change over the next 50 years were evaluated using modeled scenarios based on the USACE “intermediate” and “high” sea level change rates.

4.1 Bluff Erosion Rates in the Project Area

North Section: The DSAS analysis indicates that the bluff located between the Highway 1 revetment and Mirada Road revetment (North section) has retreated at a rate of 1.64 ft/yr (0.5 m/yr) from 1993 to 2012 (Table 15). This retreat rate is significantly greater than the annualized and standard errors, which demonstrates that there is a definite erosional trend in this area. However, this retreat is not necessarily spatially uniform, with bluff failure hotspots accounting for higher rates up to 2.3 ft/yr (0.7 m/yr) in some locations (Figure 39).

South Section: The results indicate that the bluff located immediately south of the Mirada Road revetment (South section) has retreated at a rate of less than 0.16 ft/yr (0.05 m/yr) from 1993 to 2011 (Table 15). This rate of retreat is rather small, and falls well within the annualized and standard errors, indicating this section may be stable and in equilibrium with current wave conditions. This is in contrast to the erosional North section, which suggests that the effects of the east breakwater do not extend south to this section (Figure 40).

Background Section: The results indicate that the bluff located between Kelly Ave and Poplar Street (Background section) has retreated at a rate of 0.23 ft/yr (0.07 m/yr) from 1993 to 2012 (Table 15). This rate falls within the annualized and standard errors, but visual inspection of aerial imagery in this section indicates that some erosion has occurred during the analysis period (Figure 41). Thus, this analysis will assume that the values calculated by DSAS represent the best available estimate of the “natural” background rate in the study area, and show a minor natural retreat rate for Northern Half Moon Bay.

Table 15: Bluff retreat rates calculated with USGS Digital Shoreline Analysis System (DSAS) Software				
Section	Mean End Point Rate (m/yr), 1993 to 2012 (2011 for South Section)	Annualized Error for EPR* (m/yr)	Mean Linear Regression Rate (m/yr)	Standard Error for LRR** (m/yr)
North	0.49	0.11	0.50	0.13
South (Bluff)	0.04	0.13	0.03	0.12
Background	0.07	0.16	0.07	0.11

* Annualized error was calculated using methods outlined in Hapke and Reid (2007).
** Standard error is based on a 90% confidence level, per methods outline in Hapke et al. (2009)

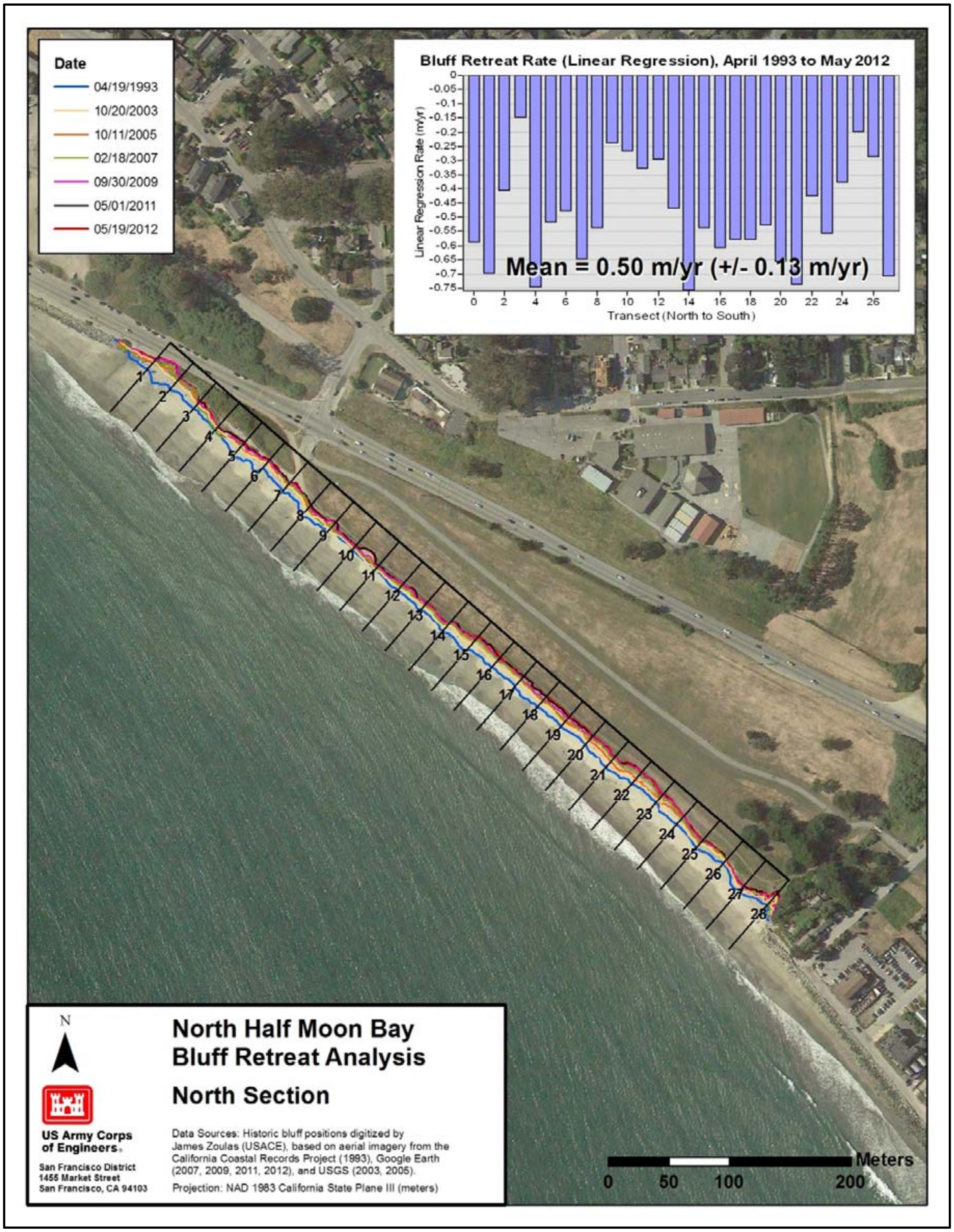

Figure 39. Bluff retreat rates along the section of shoreline between the Caltrans and Mirada Road revetments. Background imagery from Google Earth (2012)

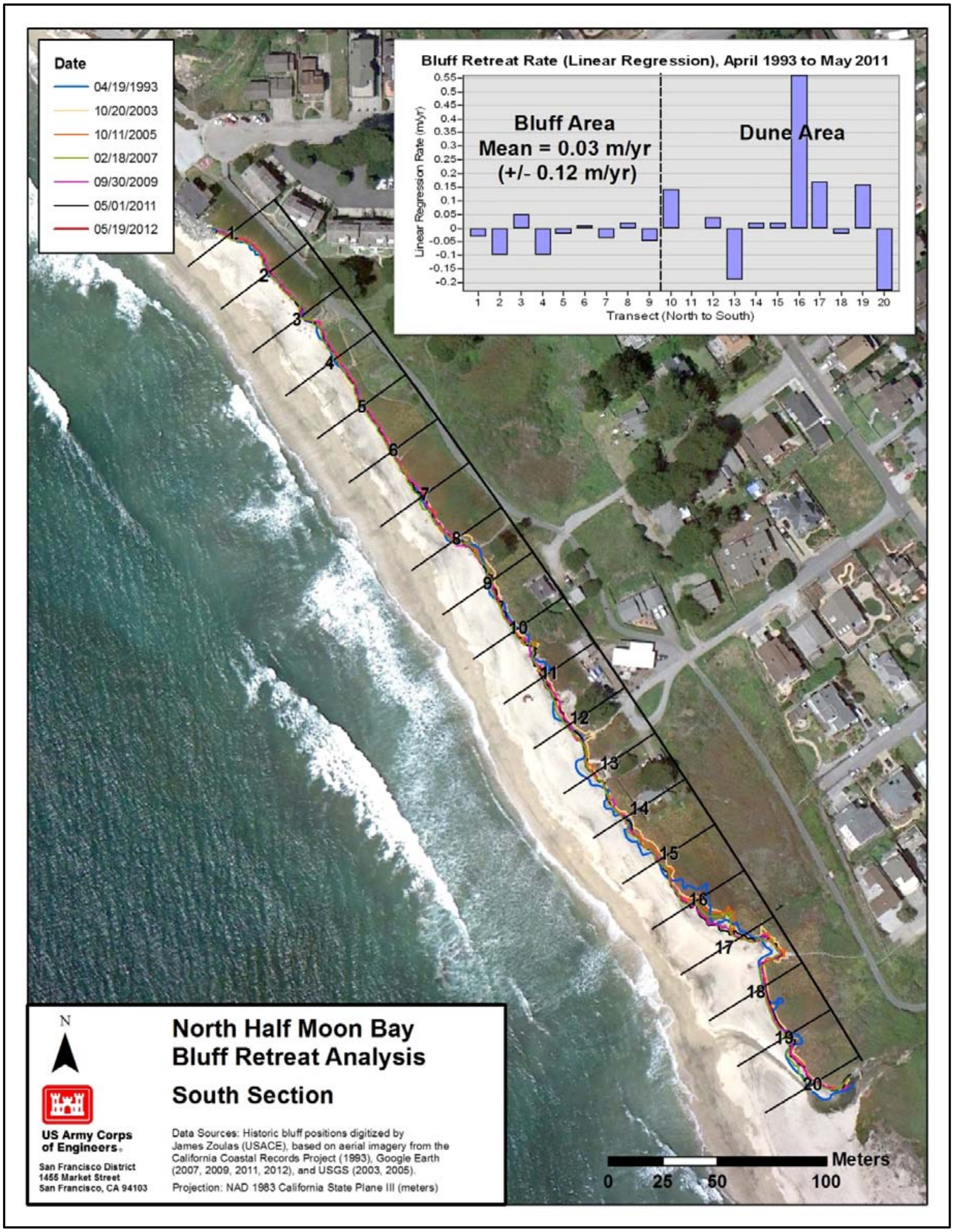

Figure 40. Bluff and dune retreat rates just south of the Mirada Road Revetment. Background imagery from Google Earth (2011)

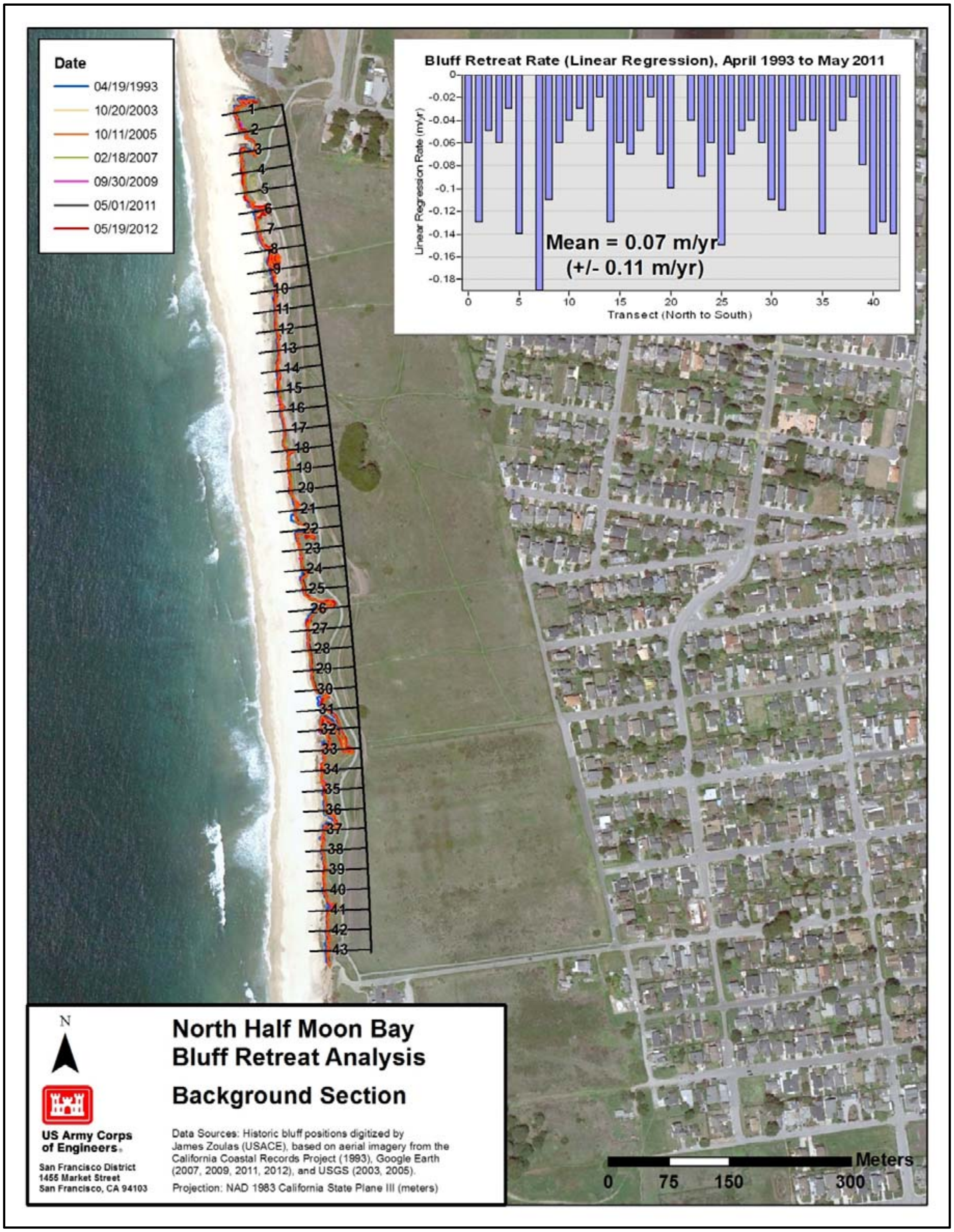

Figure 41. Bluff retreat rates along the section of shoreline used to determine the "natural" background erosion. Background imagery from Google Earth (2011)

There are several conclusions that can be drawn from the bluff erosion analysis. First, the current bluff retreat rates in three sections exhibit considerably variation (up to 1.64 ft/yr), with the bluff in the North section retreating at a significantly faster rate than the other two sections. Second, the difference between the North and Background bluff retreat rates is approximately 1.3 ft (0.4 m) per year. Thus, it can be inferred that the east breakwater is inducing up to an additional 1.3 ft/yr (0.4 m/yr) of bluff retreat in the North section. The effects of the east breakwater do not appear to extend south of the Mirada Road revetment, given the slow bluff retreat rate to the south of the revetment.

4.2 Without-Project Condition: CMS Modeling of Pre-Breakwater Scenario

The pre-breakwater scenario was developed in order to gain a better understanding of physical conditions at the study site prior to construction of Pillar Point Harbor. The pre-harbor bathymetry was reconstructed as a logarithmic spiral beach with a concave shoreline from the up coast headland (Figure 42). This shoreline was exposed to both local seas and swell, which facilitated unimpeded longshore transport of sediment throughout the current location of the harbor. Thus, more significant changes in nearshore bathymetry are expected to occur under this scenario, given the open nature of this system when compared to the present (post-breakwater) scenario.

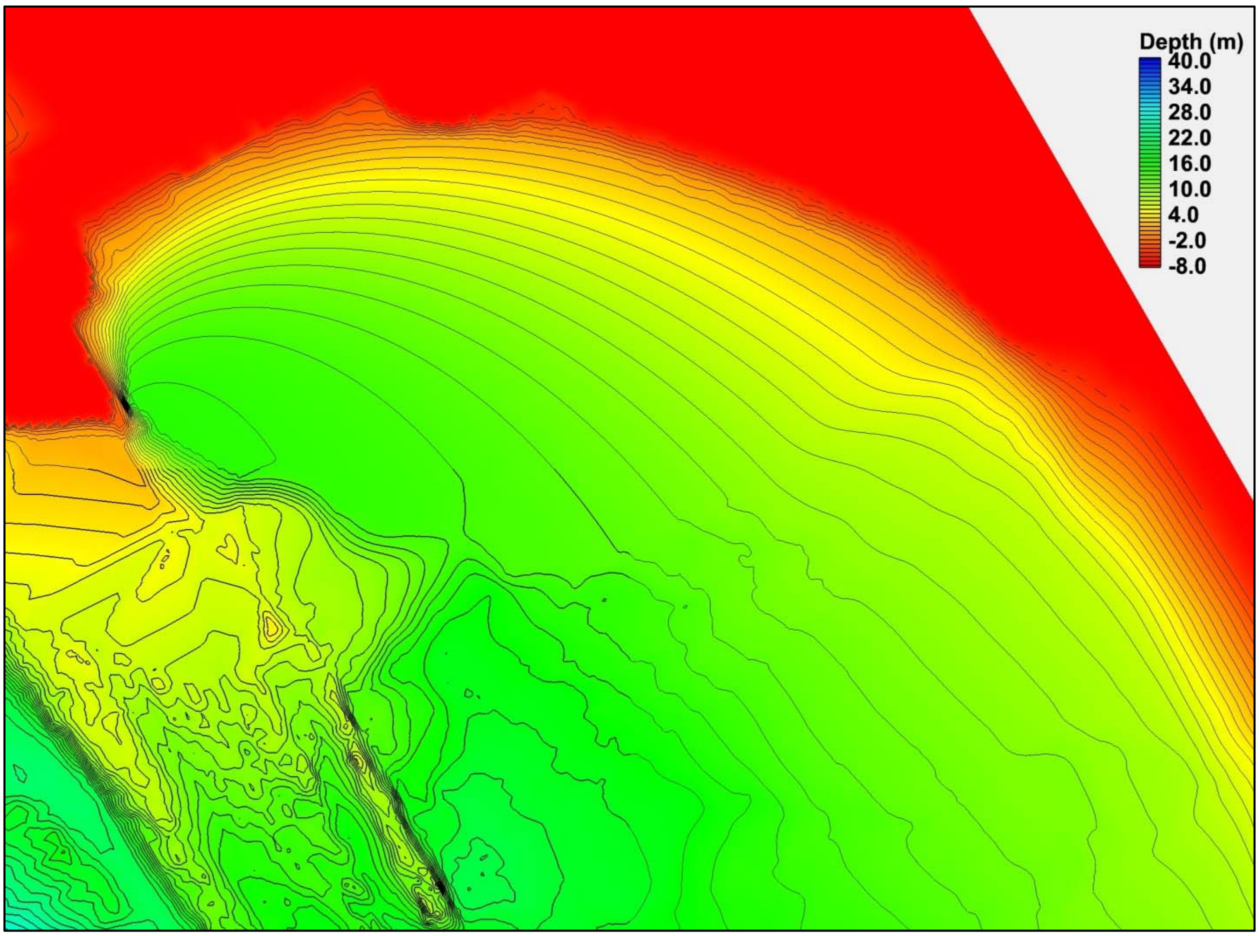

Figure 42. Bathymetric grid depicting pre-breakwater conditions

In order to provide a consistent methodology for comparing the without-project conditions to the results of the analysis of design measures (Section 6), the model outputs (primarily volumetric changes) were examined in several boxes. The boxes were formulated to capture the impacts of various design measures on erosion and accretion at: (1) the nearshore and beach in front of the unprotected bluff at Surfer's Beach, (2) the nearshore and beach in front of the Mirada Road Revetment, and (3) the section of Pillar Point Harbor adjacent to the east breakwater which has been subject to significant shoaling (Figure 43).

The box (1) encompassing Surfer's Beach extends 3,200 ft along the shoreline and 1,200 ft out to a depth of approximately 15 to 20 ft (NAVD 88). The Mirada Road box (2) extends 1,600 along the length of the entire revetment, encompasses the mouth of Arroyo Medio, and extends out 1,200 ft to a similar depth. Previous study suggests that the breaker zone extends out to a depth of 25 ft and with sediment transport occurring in depths as great as 50 ft (USACE, 1996). However, an analysis of modeled volumetric changes indicated that there was very little erosion or accretion (< 5 yd^3 per approximately 4,000 ft^2 cell) at and below a depth of approximately 15 ft (NAVD 88) over the simulation period. Thus, a lower bounding depth of 15 to 20 ft (NAVD 88) was selected in order to focus the analysis on the range of depths with the most significant erosion and accretion.

The results of the yearlong simulation of the pre-breakwater scenario suggest that conditions within and southeast of Pillar Point Harbor were erosional prior to construction of the harbor (Table 16, Table 17, and Table 18). The most significant net erosion occurred between the present roots of the east breakwater and Inner Breakwater, where approximately 6,500 yd^3 of sand were lost from the "box" over the course of the year. This result is consistent with a previous study of the general area (Krumbein, 1947), which concluded that a significant quantity of sand (30,000 yd^3) moved offshore annually. However, a previous shoreline change study (USACE, 1947) suggests that the shoreline within this area of the harbor was modestly accretional from 1861 to 1946. This is because the nearby bluffs are highly erosional, with eroded bluff material accumulating on the beach at a slightly greater rate than offshore loss rate.

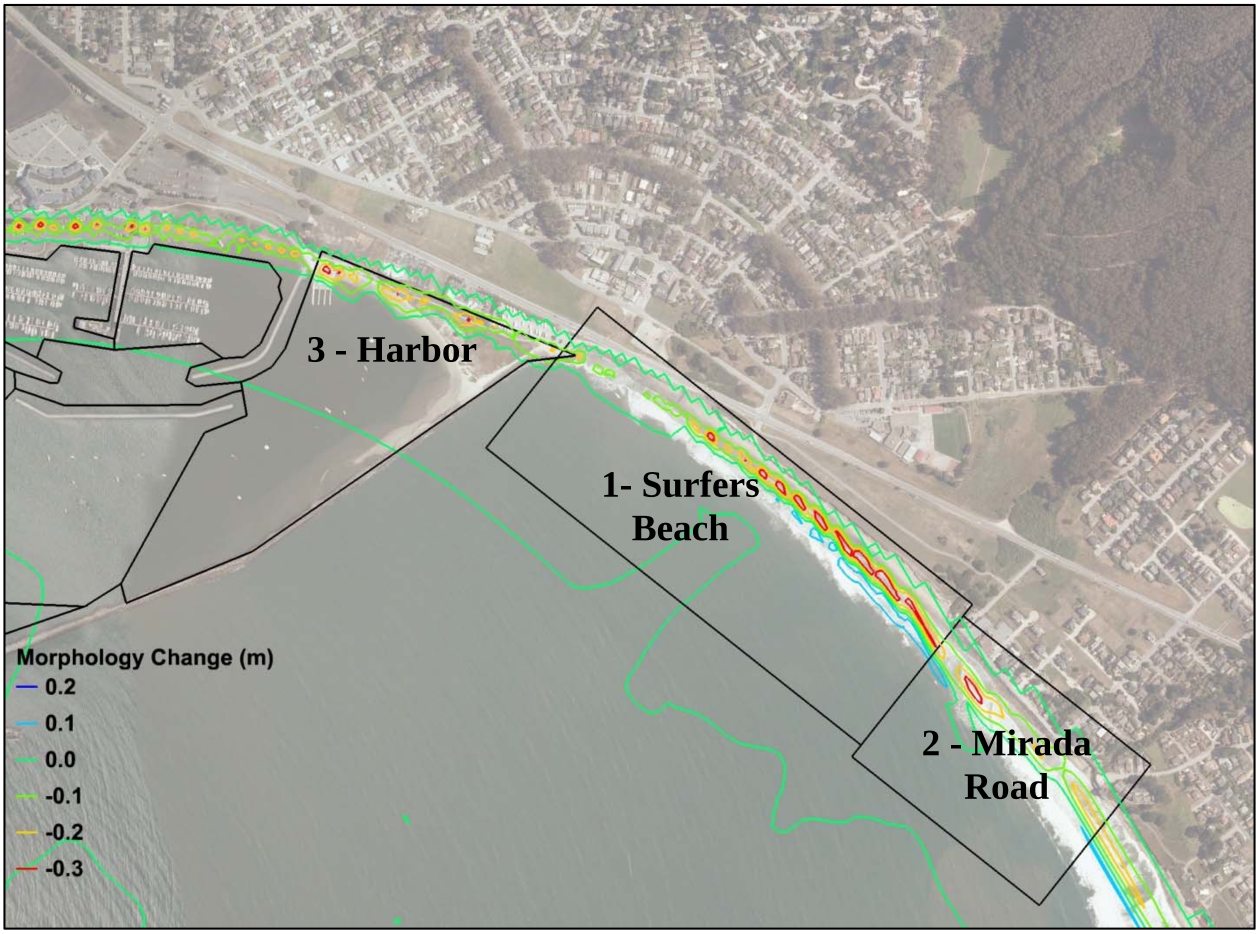

Figure 43. Erosion and accretion in vicinity of Surfer's Beach from June 2009 to November 2009 (6 months). The boxes (1 through 3) outline the analyses areas

Table 16. Erosion and accretion in the immediate vicinity of the fill placement area (box 1)

	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)
Jun to Nov 2009	-8,200	+5,900	-2,200
Dec 2009 to Mar 2010	-6,000	+4,900	-1,100
Apr to May 2010	-2,100	+1,400	-800
Total (Morph Change)	-16,300	+12,200	-4,100

Table 17. Erosion and accretion in the vicinity of the Mirada Rd revetment (box 2)			
	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)
Jun to Nov 2009	-5,500	+2,500	-3,000
Dec 2009 to Mar 2010	-4,300	+2,300	-2,000
Apr to May 2010	-1,800	+900	-1,000
Total (Morph Change)	-11,600	+5,700	-6,000

Table 18: Erosion and accretion in Pillar Point Harbor (box 3)			
	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)
Jun to Nov 2009	-4,500	+1,600	-3,000
Dec 2009 to Mar 2010	-2,900	+500	-2,300
Apr to May 2010	-1,500	+300	-1,200
Total (Morph Change)	-8,900	+2,400	-6,500

4.3 Without-Project Condition: CMS Modeling of Existing (with Breakwater) Scenario [Baseline Conditions]

This scenario differs from the pre-breakwater scenario, in that it depicts the current configuration of the Pillar Point Harbor coastal infrastructure, including both the inner and outer breakwaters. Thus, this scenario represents the actual present without project-condition, and can thereby serve as the baseline scenario against which the efficacy of the with-project design scenarios could be evaluated.

There were several key findings from the year-long simulation. The primary finding was that there is considerable net erosion along and offshore of Surfer's Beach (-4,200 yd^3/yr) and the revetment fronting Mirada Rd (-3,900 yd^3/yr) (Figure 44, Table 19, Table 20). The most significant erosion was concentrated at the southeast end of Surfer's Beach at approximately mean lower low water (MLLW). There was some modest accretion in depths of 3 to 7 ft; however this accretion was not sufficient to offset the erosional trend in the nearshore zone. There was also significant accretion within Pillar Point Harbor (Table 21), with approximately 2,000 yd^3 of sand accumulating over the course of 1 year.

When these results are compared to those from the pre-breakwater simulation, there are several notable changes in erosion and accretion patterns. First, the nearshore environment within Pillar Point Harbor transitioned from erosional to accretional after the construction of the breakwaters (Table 21), and this result is consistent with the well documented shoaling within this section of the harbor. Second, the rate of erosion in the nearshore adjacent to the Mirada Rd revetment decreased following construction of the breakwaters. This suggests that impact of the east breakwater on wave patterns and sediment transport extends to the Mirada Road Revetment. However, it should be noted that the bluff retreat analysis (see Section 4.1) indicated that the east breakwater's influence on bluff erosion did not extend south of this revetment.

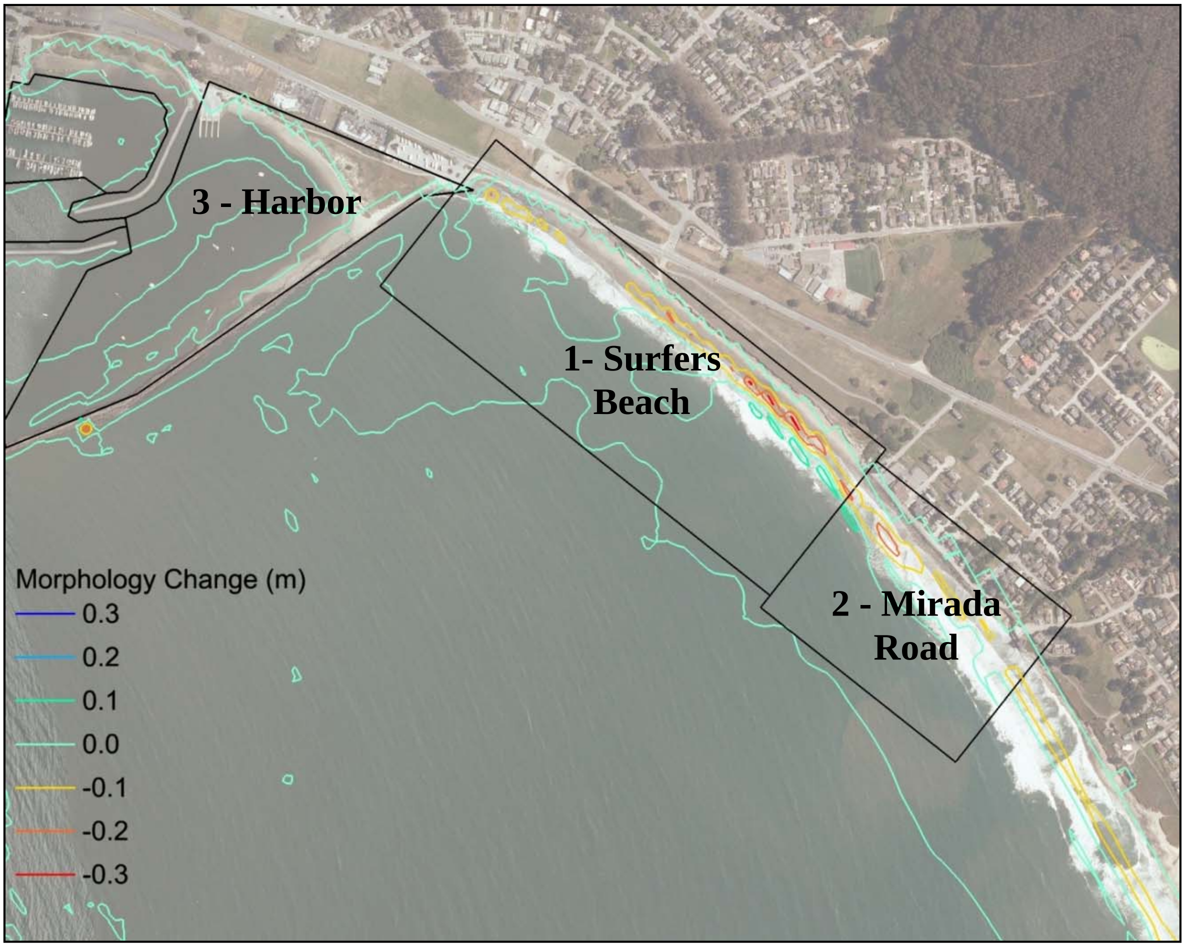

Figure 44. Calculated morphology change in the vicinity of Surfers Beach from June 2009 to November 2009 (6 months). The boxes (1 – 3) outline the analysis areas

Table 19: Erosion and accretion in the immediate vicinity of Surfers Beach (box 1)

	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Pre-Breakwater (yd^3)
Jun to Nov 2009	-5,900	+3,700	-2,200	+20
Dec 2009 to Mar 2010	-5,500	+4,500	-1,000	+50
Apr to May 2010	-2,100	+1,200	-900	-170
Total (Morph Change)	-13,500	+9,400	-4,200	-100

Table 20: Erosion and accretion in the vicinity of the Mirada Road revetment (box 2)				
	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Pre-Breakwater (yd^3)
Jun to Nov 2009	-3,300	+1,500	-1,800	+1,200
Dec 2009 to Mar 2010	-3,400	+2,000	-1,400	+600
Apr to May 2010	-1,300	+700	-700	+300
Total (Morph Change)	-8,100	+4,200	-3,900	+2,100

Table 21: Erosion and accretion in Pillar Point Harbor (box 3)				
	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Pre-Breakwater (yd^3)
Jun to Nov 2009	-750	+1,630	+880	+3,800
Dec 2009 to Mar 2010	-710	+1,610	+900	+3,200
Apr to May 2010	-180	+440	+260	+1400
Total (Morph Change)	-1,640	+3,680	+2,040	+8,500

4.4 Without-Project Condition: Impact of Sea Level Rise

In order to account for potential impacts of sea level rise, 2 additional without-project conditions (with breakwater) scenarios were modeled with the CMS. The scenarios were based on projected water surface elevations associated with the USACE Intermediate and High rates at year 50, in accordance with ER 1100-2-8162 (see Section 2.1.1.2). Due to the resource constraints of this CAP study, CMS simulations of projected sea level change were limited to these 2 rates for the without-project condition. This was done under the assumption that the effects of the more extreme sea level change rates (Intermediate and High) under the without-project condition would likely translate to the potential with-project scenarios.

4.4.1 Intermediate Rate

If sea level were to increase following the USACE Intermediate rate, the water surface would rise by 0.71 ft (0.22 m) in the study area over the course of 50 years. As a result, it is expected that rates of erosion of beach sands and adjacent bluffs would increase as the higher water levels exposed the upper beach and bluff toes to increased wave attack. This hypothesis is supported by the modeling results, which show a clear increase in erosion rates in the nearshore outside of the harbor (Figure 45; Table 22; Table 23).

In contrast to the unprotected areas, there was a slight increase in the sediment accretion rate inside of the harbor (Table 24). This suggests that higher water levels may increase the net volume of water and sand that is transported through the east breakwater via surging, given that breakwater structures were specified as “permeable” in this and other modeling scenarios.

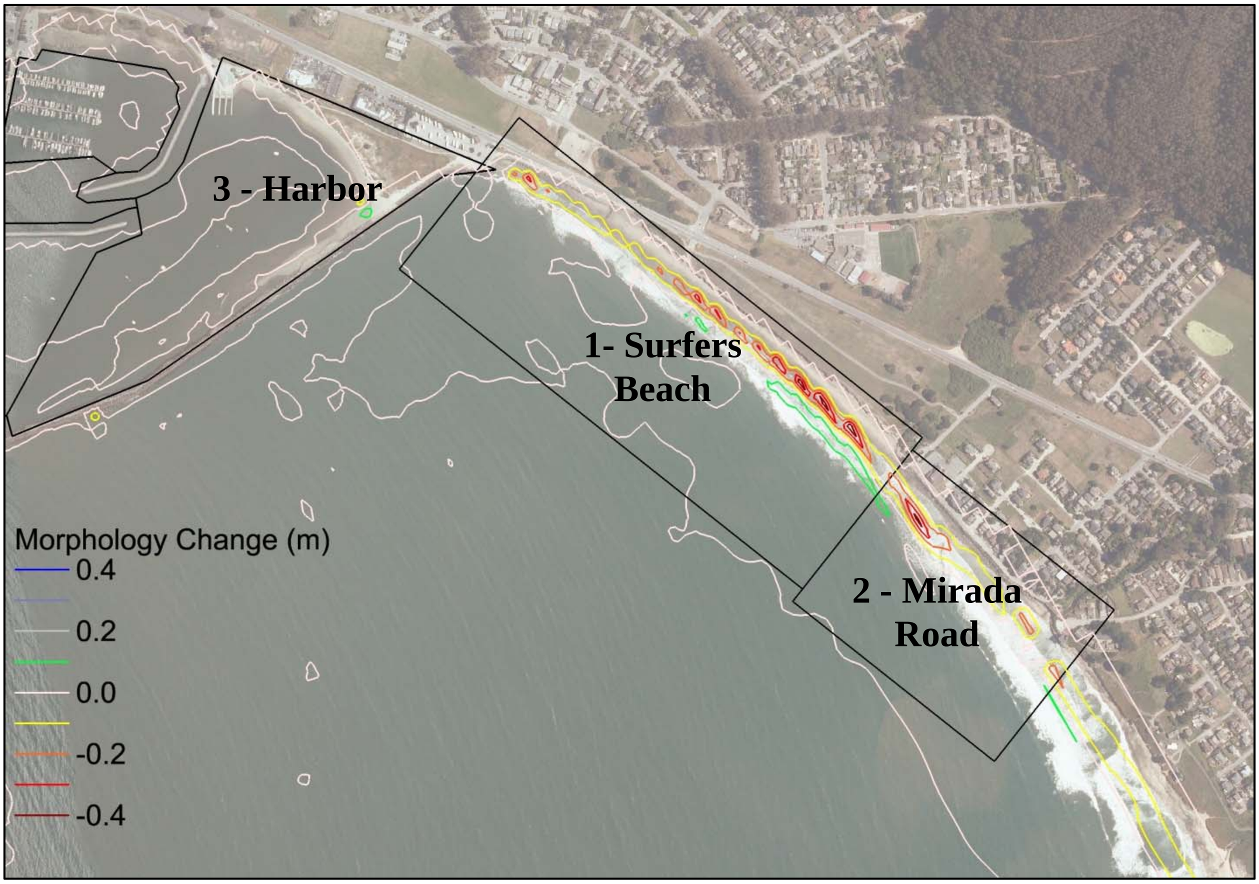

Figure 45. Calculated morphology change in the vicinity of Surfers Beach from June 2009 to November 2009 (6 months) based on a sea level rise of 0.71 ft (0.22 m) in 50 years (USACE Intermediate Rate). The boxes (1 – 3) outline the analysis areas

Table 22: Erosion and accretion in the immediate vicinity of Surfers Beach (box 1)

	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Existing Condition (yd^3)
Jun to Nov 2009	-8,000	+4,800	-3,200	-1,000
Dec 2009 to Mar 2010	-7,000	+5,000	-2,000	-1,000
Apr to May 2010	-2,500	+1,200	-1,300	-300
Total (Morph Change)	-17,500	+11,000	-6,500	-2,300

Table 23: Erosion and accretion in the vicinity of the Mirada Road revetment (box 2)

	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Existing Condition (yd^3)
Jun to Nov 2009	-4,400	+2,000	-2,400	-600
Dec 2009 to Mar 2010	-4,300	+2,500	-1,800	-400
Apr to May 2010	-1,700	+800	-900	-200
Total (Morph Change)	-10,400	+5,300	-5,100	-1,200

Table 24: Erosion and accretion in Pillar Point Harbor (box 3)				
	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Existing Condition (yd^3)
Jun to Nov 2009	-800	+1,800	+1,000	+130
Dec 2009 to Mar 2010	-700	+1,700	+1,000	+100
Apr to May 2010	-200	+500	+300	+50
Total (Morph Change)	-1,700	+4,000	+2,300	+280

4.4.2 High Rate

This scenario is based on the assumption that sea level will rise by 2.06 ft (0.63 m) at the end of 50 years. As anticipated, erosion rates increased in the nearshore areas outside of the harbor, with a significant increase in erosion compared to the Intermediate scenario (Figure 46, Table 25, and Table 26). There was also modest increase in accretion in the harbor, particularly in the summer and fall months (Table 27).

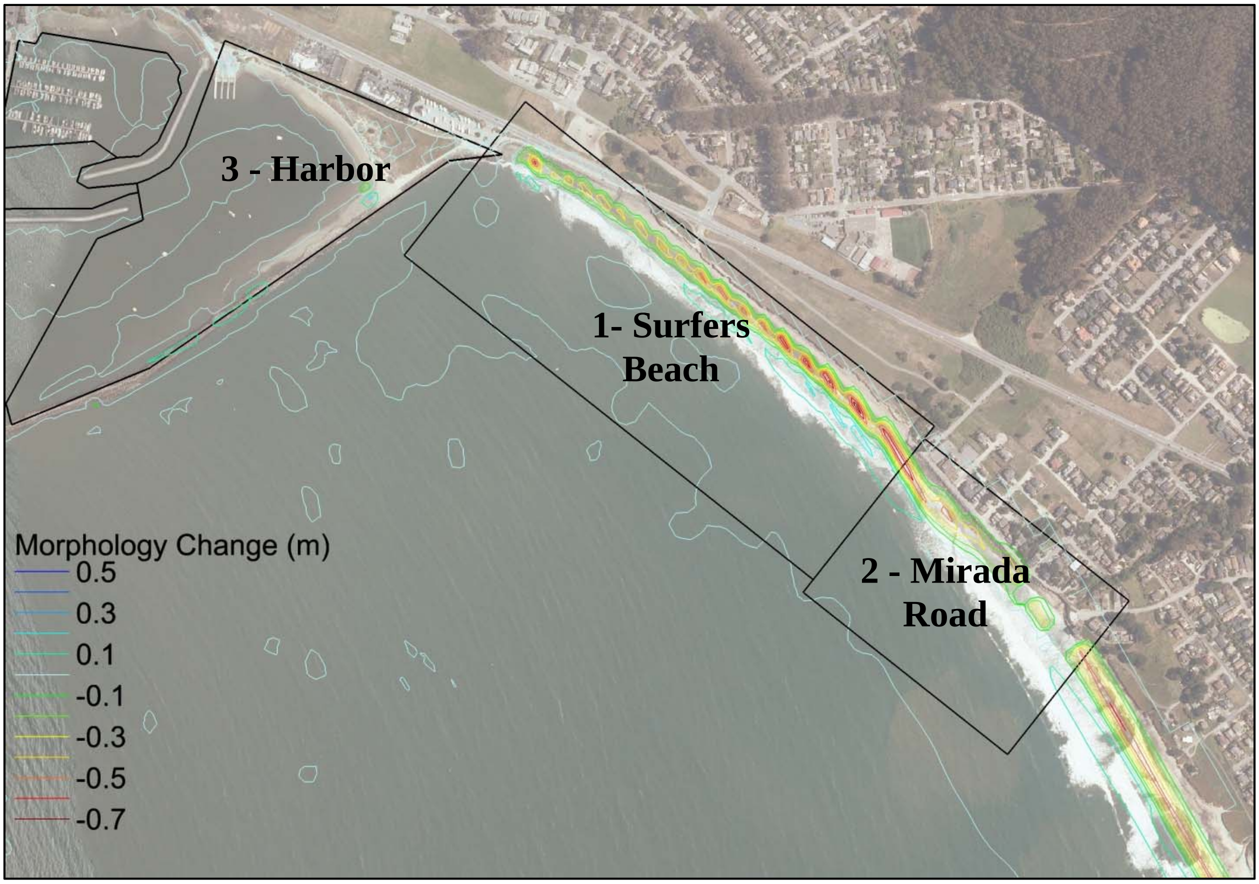

Figure 46. Calculated morphology change in the vicinity of Surfers Beach from June 2009 to November 2009 (6 months) based on a sea level rise of 2.06 ft (0.63 m) in 50 years (USACE "High" Rate). The boxes (1 – 3) outline the analysis areas

Table 25: Erosion and accretion in the immediate vicinity of Surfers Beach (box 1)

	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Existing Condition (yd^3)	Difference from Curve I (yd^3)
Jun to Nov 2009	-13,900	+7,400	-6,500	-4,300	-3,300
Dec 2009 to Mar 2010	-11,600	+6,800	-4,900	-3,800	-2,900
Apr to May 2010	-4,300	+1,900	-2,400	-1,400	-1,100
Total (Morph Change)	-29,800	+16,100	-13,700	-9,500	-7,300

Table 26: Erosion and accretion in the vicinity of the Mirada Road revetment (box 2)					
	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Existing Condition (yd^3)	Difference from Curve I (yd^3)
Jun to Nov 2009	-8,000	+3,500	-4,500	-2,700	-2,100
Dec 2009 to Mar 2010	-8,100	+3,500	-4,600	-3,200	-2,800
Apr to May 2010	-3,000	+1,100	-1,900	-1,200	-1,000
Total (Morph Change)	-19,100	+8,100	-11,000	-7,100	-5,900

Table 27: Erosion and accretion in Pillar Point Harbor (box 3)					
	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Existing Condition (yd^3)	Difference from Curve I (yd^3)
Jun to Nov 2009	-1,000	+2,700	+1,700	+760	+630
Dec 2009 to Mar 2010	-1,800	+2,300	+500	-370	-460
Apr to May 2010	-400	+700	+300	+20	-30
Total (Morph Change)	-3,200	+5,700	+2,500	+410	+140

5 Formulation of Design Measures

The formulation of design measures involved reviewing the information compiled in Sections 2 through 4 to develop an adequate understanding of the physical environment of the site and associated without-project conditions. This understanding was then employed to identify site specific problems that served as the basis for formulating the design objectives. Potential design measures were then formulated to meet those objectives, and screened through coordination with the Project Delivery Team (PDT), Non-Federal Sponsor, and other stakeholders.

5.1 Statement of the Problem

The rapid erosion of the coastal bluffs extending south from the east breakwater has been a source of concern over the past several decades. A recent analysis of bluff retreat from 1993 to 2012 suggests that the approximately 2,200 ft long unprotected section of bluff between the root of the east breakwater and the Mirada Road revetment is retreating at a rate of 1.64 ft (0.5 m) per year (Figure 39). There is considerable uncertainty regarding the respective contributions of terrestrial (runoff) or marine (wave attack) processes to the observed bluff erosion, with evidence of both processes playing a role (Hampton, 2002; Griggs et al., 2005). However, the rate of retreat in the unprotected section closest to the breakwater was significantly greater than the rate at a geologically similar unprotected section of bluff further down coast. This suggests that the locally higher rate of erosion can be at least partially attributable to changes in local hydrodynamic (wave and current) conditions and sediment supply related to the construction of the east breakwater.

In addition to the problem of bluff erosion, the construction of the east breakwater has also been associated with the accumulation of a significant amount of sediment (primarily sand) within Pillar Point Harbor (Figure 27). Some of this sand has formed a beach near the root of the breakwater, which includes a fairly well developed set of vegetated dunes. Additional sand has accumulated along the harbor side of the east breakwater to form a shoal that is 200 to 250 ft wide and close to 2,000 ft long. This shoal could present a potential navigation hazard for vessels utilizing the nearby small boat launch ramp. Hydrographic surveys from 1994 and 2007 suggest that shoaling rates adjacent to the east breakwater have exceeded 3,000 yd^3 per year (Gahagan & Bryant Associates, 2007). A comprehensive hydrographic and topographic survey from 2011 suggests that much of this shoal is at least 10 ft thick (Towill Inc., 2011), and a GIS-based analysis indicated it could yield at least 150,000 $yd^3$of sand.

Both of the above problems trace their roots to changes in hydrodynamic conditions and sediment transport wrought by the construction of the east breakwater. The current scientific understanding is that prior to the construction of the harbor, sediments (including sand) traveled in a generally southeast direction owing to longshore currents induced by wave energy approaching from the northwest (Griggs et al., 2005). However, this southeastward transport of sand was effectively disrupted upon completion of the harbor, resulting in a sand deficit in the beaches and nearshore area directly southeast of the root of the east breakwater. Numerical modeling (see sections 4.2 and 4.3) with the Coastal Modeling System (CMS) suite confirms this hypothesis, and suggests that prior to harbor construction there was a node (at Surfers Beach) where southeast and northwest longshore currents might have converged to deposit sand (Figure 47). This node then disappeared following construction of the harbor, with much of the sand that would have been transported to Surfers Beach now trapped in the shoal on the harbor side of the east breakwater.

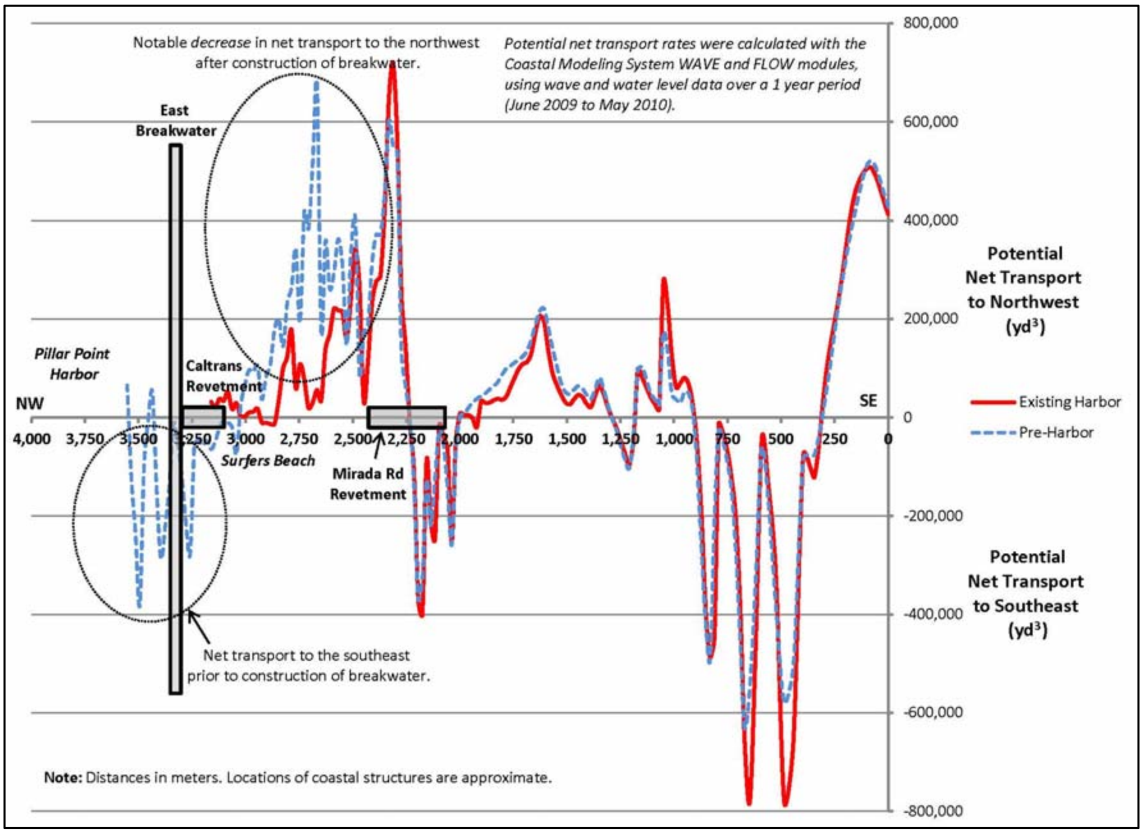

Figure 47. Potential net sediment transport based on Coastal Modeling System (CMS) simulations.

In addition to disrupting the southeastward transport of sand, the east breakwater has served as a semi-porous structure that allows sand to enter the harbor from the southeast via voids in the structure (Figure 48 and Figure 49). A shoaling analysis performed in 1996 suggests that wave surging through the breakwater accounts for a significant portion of sand accumulating in the harbor (USACE, 1996). The strongest wave surging is typically associated with intense storm events approaching from the south and southwest, which can cause sediment transport reversals from the predominant southeastward littoral drift. Once in the harbor, it appears that the sand generally settles out adjacent to the breakwater in the shoal, although currents induced by the wave surging may further redistribute the sand in the harbor (USACE, 1996).

Figure 48. Photograph looking into Pillar Point Harbor showing surging of water through the east breakwater (Source: Craig Conner)

Figure 49. Photograph showing areas of scour due to surging along the harbor side of the east breakwater (Source: James Zoulas)

5.2 Design Objectives

Given that there are two significant (but related) problems, there are two distinct design objectives. The first design objective is to reduce damages associated with the ongoing erosion of Surfer's Beach and the unprotected bluff backing the beach. Meeting this objective should result in the reduction of the rate of beach and bluff erosion to the "background" rate prior to construction of the east breakwater. The second design objective is to improve navigation in Pillar Point Harbor by removing sand that has shoaled in undesirable locations. This design objective is strongly supported by the San Mateo County Harbor District (non-Federal sponsor) given the impact of shoaling on navigation in the harbor.

Under ideal circumstances, a well formulated design measure should address both objectives by fixing the underlying cause(s) of the problems. The current understanding is that the presence of the harbor structures, particularly the east breakwater, is the primary driver behind both problems. However, it is not feasible to completely remove these structures, and doing so would directly contradict the second design objective. Thus, the design measures are more likely going to serve only as partial solutions to the problems at hand, which must be incorporated into a longer-term comprehensive coastal management plan for the project area.

5.3 Formulation of Design Measures

The design measures discussed in this document were formulated with the intention of addressing both of the design objectives in an economical manner that would be acceptable to local stakeholders. The measures were formulated to cover a wide array of structural and non-structural actions including beach fill, dredging, alteration of the east breakwater, construction of a spur breakwater, and managed retreat. The design measures were initially formulated by the coastal engineer, with significant input from the other project team members. Public involvement also played a key role in formulating and refining the design measures, with a public meeting on 8 November 2013 providing a forum for the USACE project team and the public to exchange ideas.

There were a number of potential design measures that were screened out due to concerns regarding engineering feasibility, economic viability, safety, and environmental impacts. Notable screened out measures included a rubblemound revetment/seawall (economic viability, environmental impacts), series of groins (environmental impacts), and an offshore reef structure (environmental impacts, safety). In the end, a final array of seven design measures was selected for Feasibility level coastal engineering analysis, and the results of these analyses are presented in Section 6.

6 Results of Design Measures Analysis

6.1 Design Measure 1: Maximum Beach Fill

This design measure involves the one-time placement of 200,000 to 250,000 yd^3 of sand on approximately 3,100 foot long section of Surfer's Beach. The placement area would extend from the root of the east breakwater to the northwestern terminus of the revetment fronting Mirada Rd. The extensive shoal and beach that has formed on the north side of the east breakwater would serve as the exclusive source of sand for this measure. The sand would mitigate bluff erosion by providing a buffer that would reduce the impacts of elevated water levels and wave attack on the bluff toe.

This design measure was formulated under the assumption that virtually all of the sand that has accumulated inside of the east breakwater would be available for removal and placement on Surfer's Beach (Figure 50). Volumetric calculations performed with ArcGIS software indicated that approximately 260,000 yd^3 of sand could be available if: (1) the shoal along the east breakwater were removed down to -10 ft (NAVD88), and (2) the beach extending from the boat launch ramp to the east breakwater was shifted shoreward with a slope starting 15 ft seaward of the existing pedestrian path. The cross-shore morphology of this beach would be based on existing surveyed beach profiles, in order to ensure a stable beach profile (Figure 51).

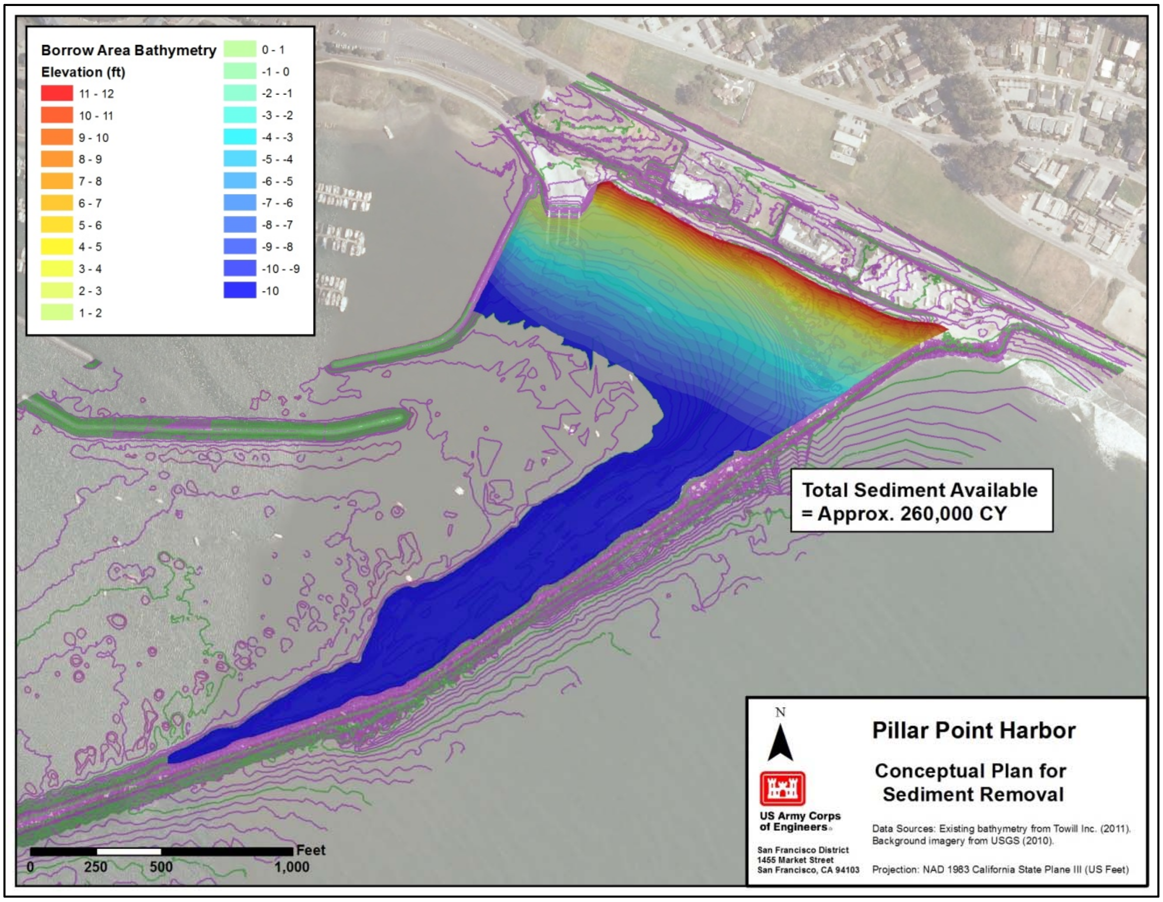

Figure 50. Conceptual plan for removing the maximum amount of available sediment from inside Pillar Point Harbor

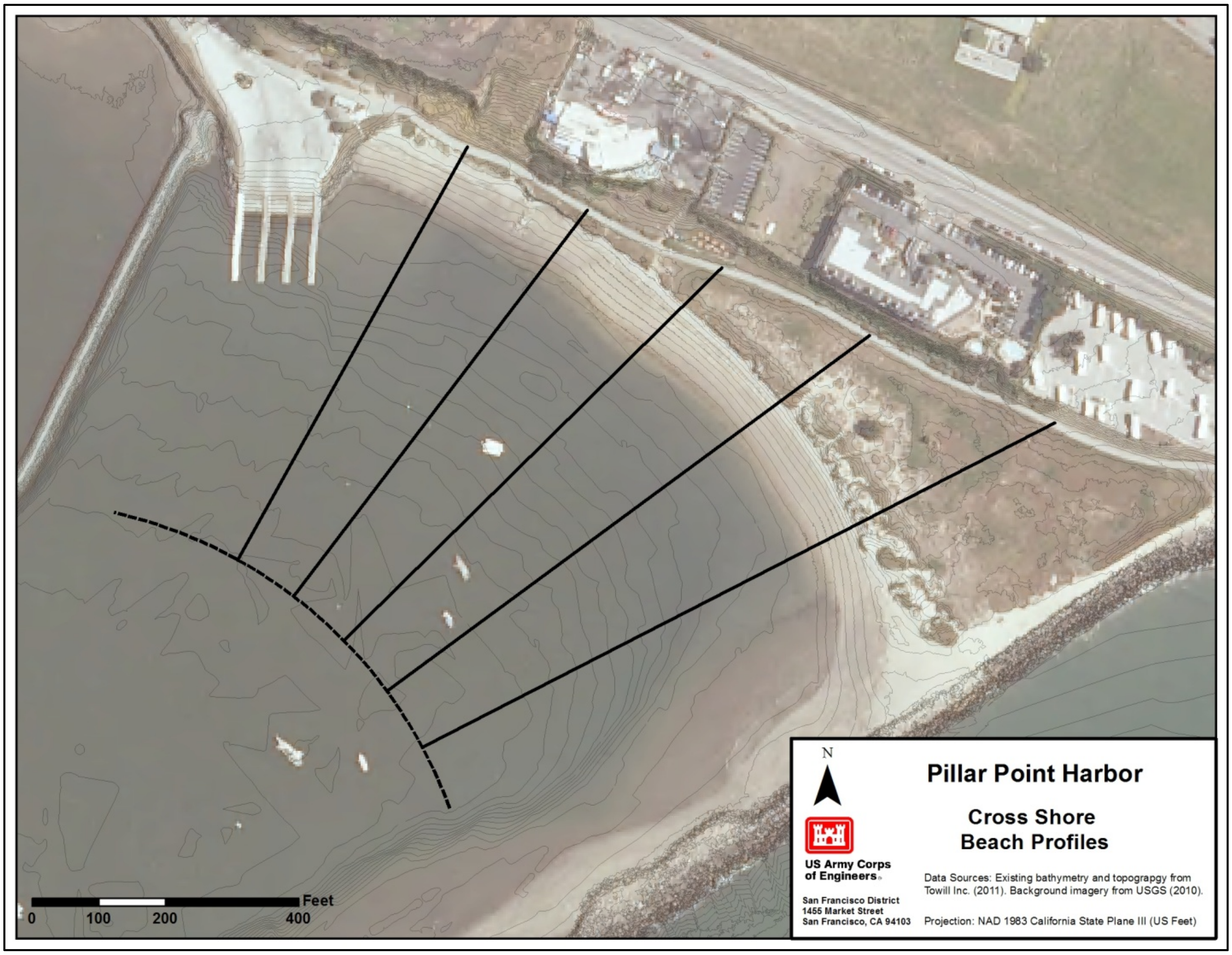

Figure 51. Locations of beach profiles utilized to determine morphology (slope) of proposed remaining beach after removal of sand

A similar design measure was a key component of several of the alternate plans considered by a previous USACE Beach Erosion Control Report (USACE, 1971). The 1971 report evaluated the efficacy of beach fill as part of two alternatives and as a stand-alone alternative. The stand-alone alternative would have involved the construction of an approximately 100 foot wide protective beach extending 4,600 ft from the east breakwater to the Arroyo en Medio. Ultimately, the 1971 report concluded that the alternatives involving beach fill would be too costly due to high annual maintenance costs. Thus, this design measure was formulated to only include a one-time placement of a large amount of sand, and assumes that there will be no maintenance (additional sand placement) in the near future.

A cost estimate prepared per guidance in Engineering Regulation 1110-2-1302 (USACE, 2008) indicates that this measure would cost approximately $6.4 million with a unit cost of $19.19 per yd^3 of sand. This estimate is based on the assumption that the sand would be dredged and pumped onto the beach using a pipeline dredge, and shaped with equipment including a small lightweight dozer and low ground pressure scraper. The estimate also included mobilization and demobilization cost of approximately $760,000 and a 20 percent contingency (So, pers comm., 2014). The high cost of mobilization/demobilization strongly suggests that a one-time removal and placement of sand at Surfers Beach will be more cost effective measure than periodic nourishment episodes.

This beach fill design was formulated based on guidance from the USACE Coastal Engineering Manual (USACE, 2002) and consideration of site specific constraints. In order to minimize potential impacts to the nearshore zone and surfing, this design measure was formulated to maximize the amount of fill placed on the sub-aerial beach. This design was also based on the assumption that a protective beach would be most effective in front of unprotected sections of bluff and in the immediate vicinity of the east breakwater, where erosion has been most severe. Thus, the beach fill would only extend approximately 3,100 ft from the root of the east breakwater to the northwest terminus of the Mirada Road revetment, and would not extend further southeast to the mouth of the Arroyo en Medio.

The beach fill would initially be placed in an “over-built” berm (Figure 52), which is expected to narrow as the new beach profile approaches equilibrium with existing hydrodynamic conditions (USACE, 2002). The berm crest elevation was determined using criteria outlined in the CEM (USACE, 2002) including examination of existing beach profiles and consideration of water level datum information (Table 4). Existing profiles in the proposed fill area indicated a very steep upper beach with no natural berm or defined backshore beach **(**Figure 53), and profiles from a nearby beach suggested that a “natural” beach berm tends to form at an elevation of 15 ft (NAVD88). However, the “natural” berm elevation is quite high, as it is nearly 10 ft higher than MHHW (5.48 ft), and 7 ft higher than the highest recorded still water level (8.01 ft) at Monterey. Given the constraints (desired beach width, available fill quantity) associated with a berm elevation of 15 ft, a target berm elevation of 9 to 10 ft was selected, based on the highest recorded water level and elevation of the toe of the bluff backing the beach.

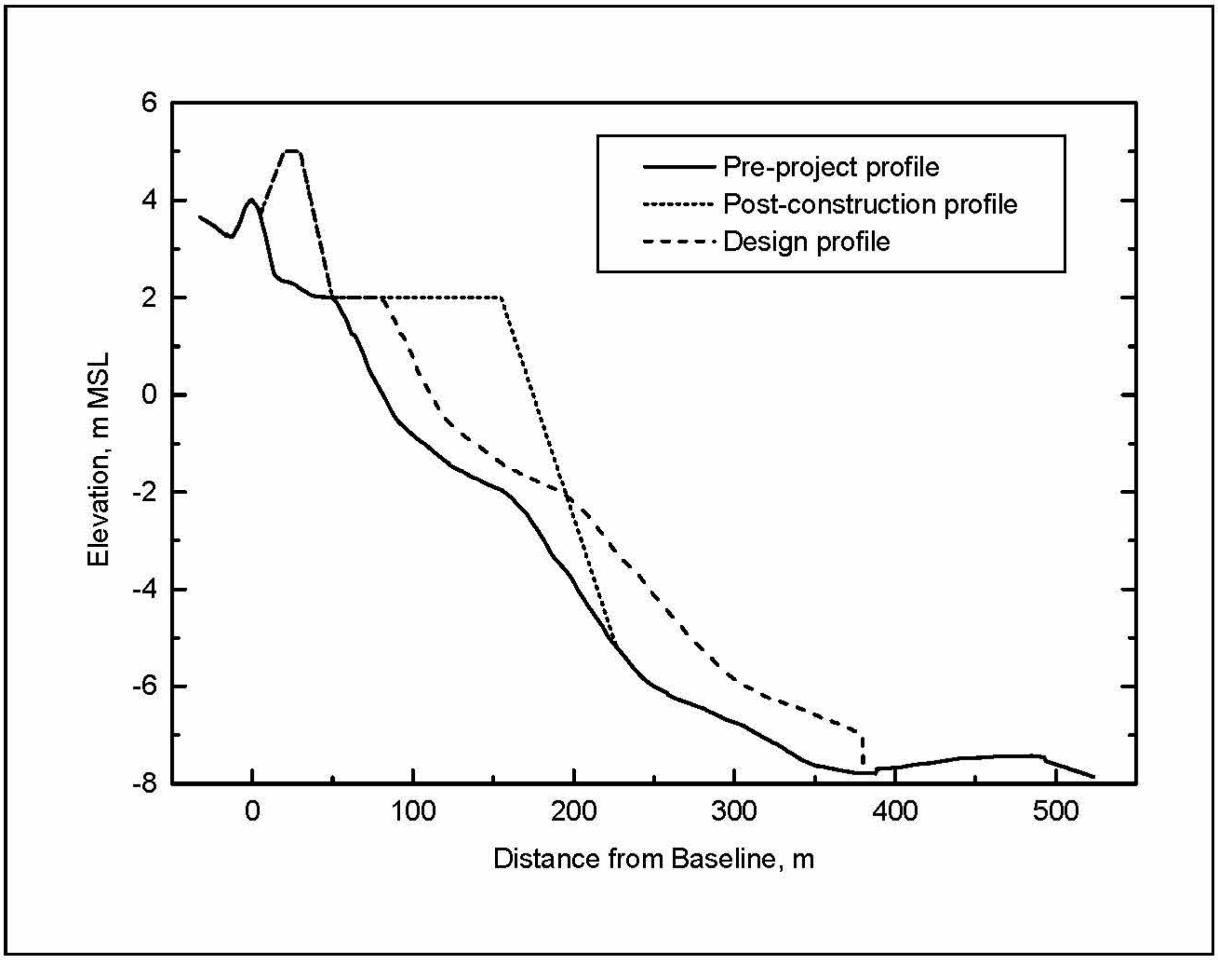

Figure 52. Schematic illustration of a typical pre-project, post-construction and design beach profiles from the CEM (USACE, 2002)

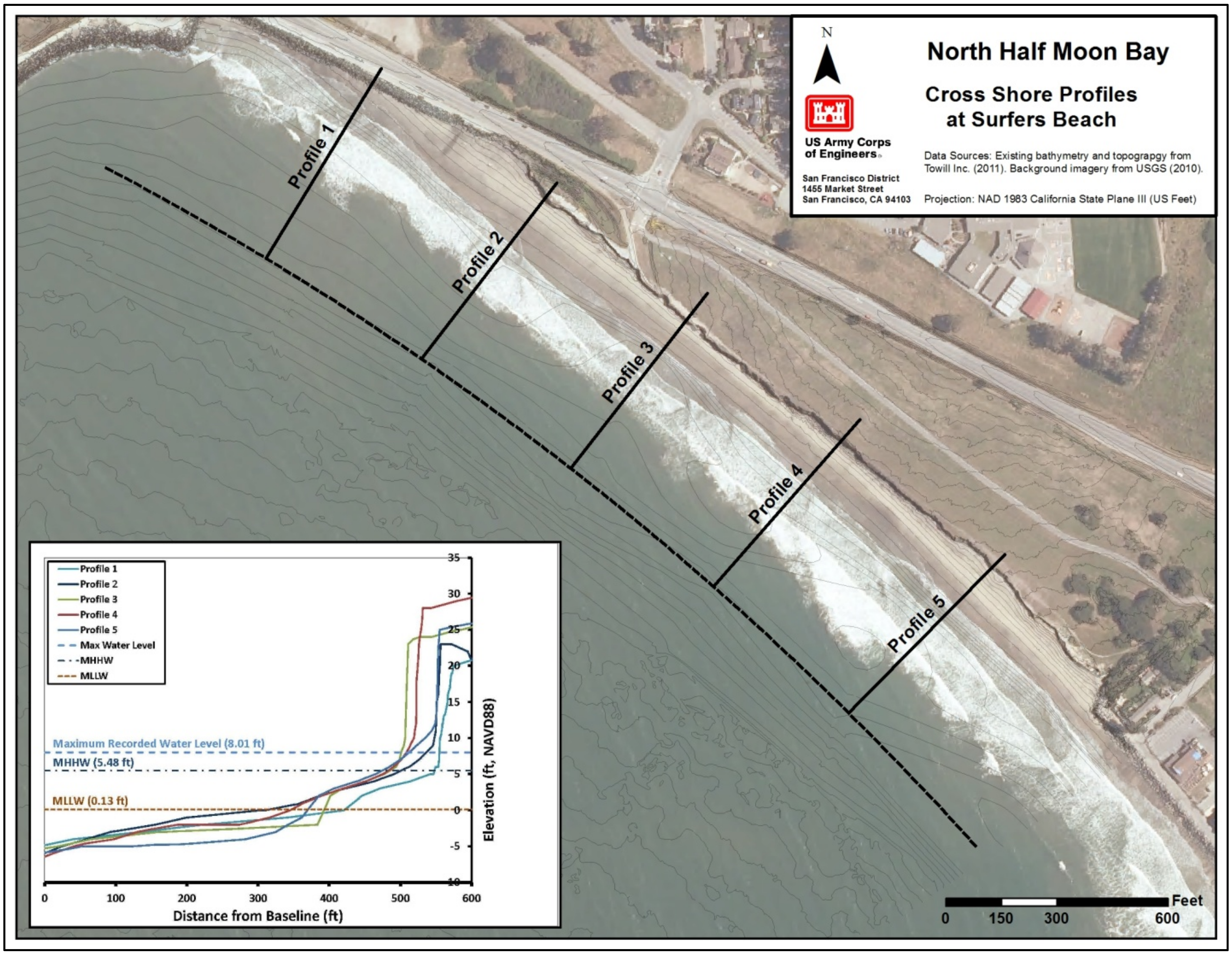

Figure 53. Existing beach profiles and water level datums utilized to formulate the maximum beach fill design measure

After the target berm elevation was determined, iterative volume computations were carried out with ArcGIS software to determine the maximum berm width that could be constructed with the available sand. These computations assumed that up to 250,000 yd^3 of sand would be available for placement, and that there would be no overfill factor, as past sediment sampling has indicated that the sand in the proposed borrow area is very similar in gain size to sand in the proposed fill area (USACE, 1996). The computations indicated that an approximately 180 foot wide beach berm could be constructed with 200,000 to 250,000 yd^3 of sand. The berm crest would have an elevation of 9 ft, which will rise up to 10 ft at the intersection with the bluff toe (Figure 54). This gentle rise in the berm profile will help prevent ponding and reduce the impacts of overtopping. The beach face would then slope down at 12H:1V, until it contacts the existing nearshore bathymetry.

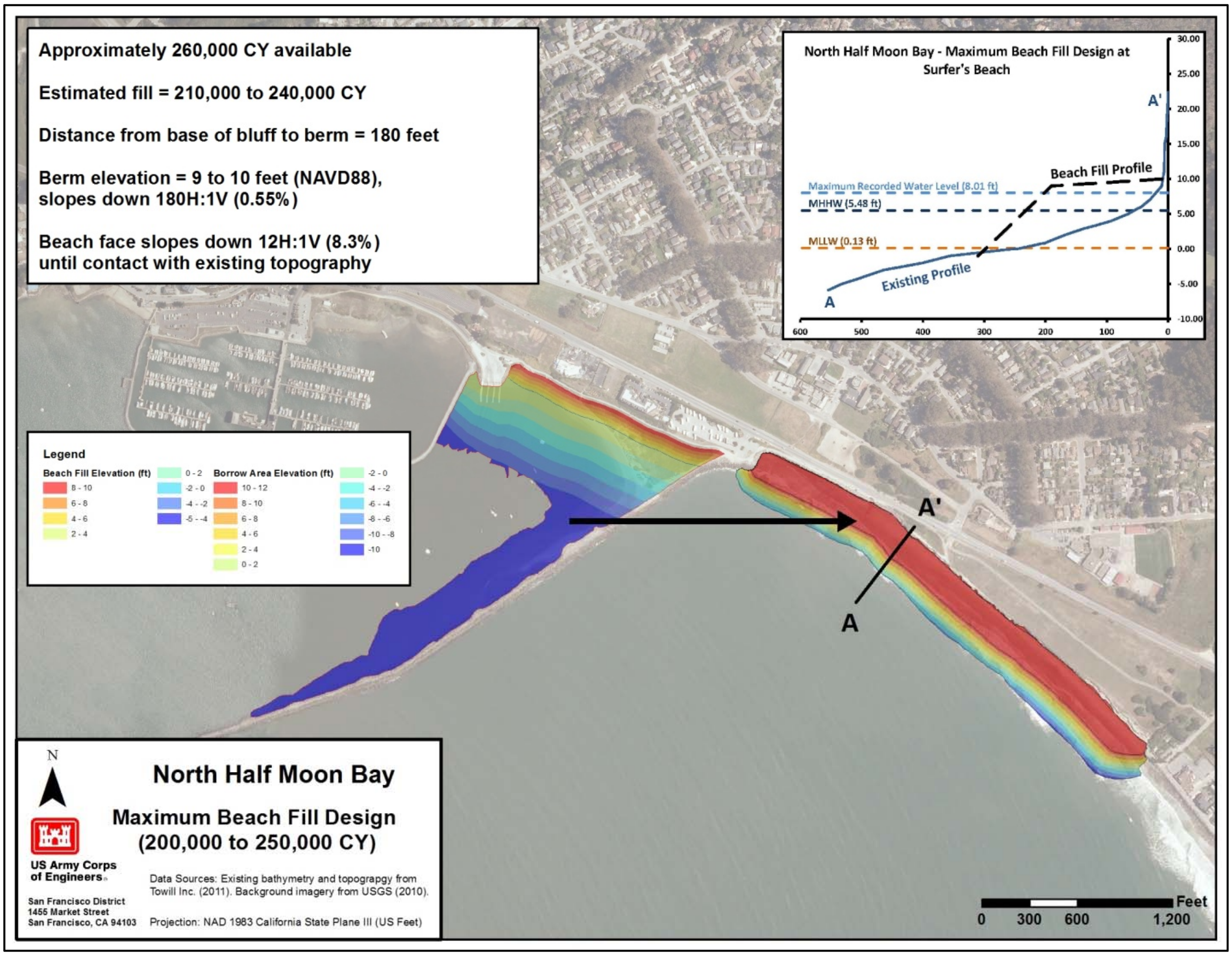

Figure 54. Schematic of maximum beach fill design scenario

In order to simulate the performance of this design scenario, the existing CMS bathymetry grids were modified to include an approximately 180 foot (55 m) wide beach berm (Figure 55). This was accomplished by importing points representing the morphology of the proposed fill area (developed in ArcGIS) into the SMS modeling suite, and creating a new bathymetric surface of 20 m x 20 m (66 x 66 ft) cells in the fill area. The cells above a depth of 1.5 m (4.9 ft) in the fill area were designated to allow for wave runup processes to contribute to erosion and accretion. It should also be noted that this simulation did not include any changes to the proposed borrow area inside the harbor, as this simulation focused on the performance of the beach fill. However, the subsequent Medium Beach Fill (Design Measure 2) simulation included changes to the borrow area, in order to determine if there could be immediate significant shoaling following the borrow event.

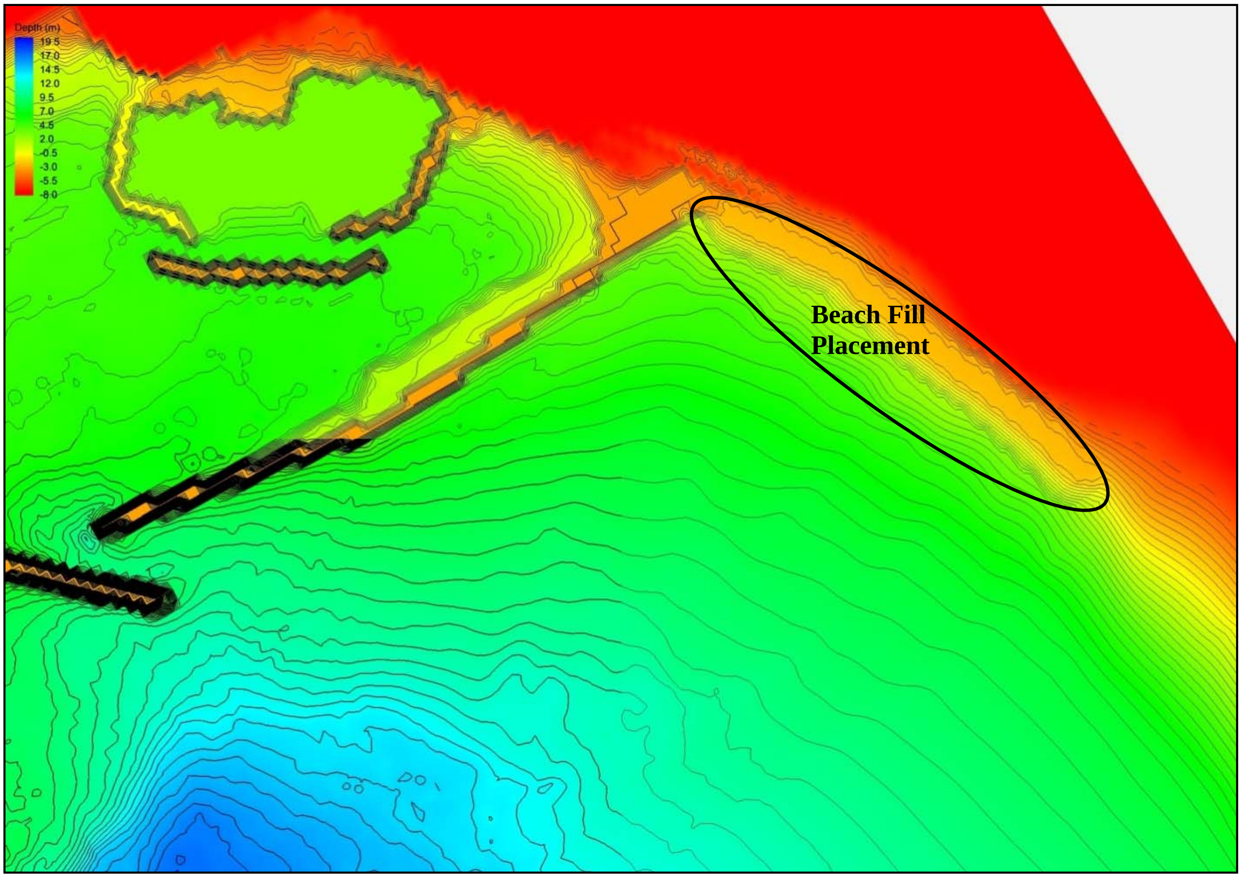

Figure 55. Depth grid used to simulate maximum beach fill design (200,000 to 250,000 yd3) scenario

There were several significant findings from the maximum beach fill design model simulation (Figure 56; Table 28, Table 29, Table 30). As in the case of the baseline conditions simulation, there was net erosion in the vicinity of the placement area (box 2) and in front of the Mirada Road revetment (box 3). The net erosion quantities were relatively close to the baseline conditions scenario, although the net erosion seaward of the Mirada Road Revetment decreased by almost 900 yd^3 (Table 30). This relatively small decrease suggests that only a modest amount of sand from the placement area will be transported to the southeast.

Within the fill placement footprint (box 1), approximately 10 to 15% of the sand would erode in 1 year, with nearly all of this sand (80 to 90%) moving seaward to depths of 3 to 10 ft in the adjacent nearshore zone. If it is assumed that this rate can be extrapolated into the future, then this (mostly) visible fill placement will have a lifespan of approximately 7 to 8 years. This assumption of a linear trend of erosion is conservative, as it is expected that the erosion rate will decrease after the newly formed beach profile approaches equilibrium with ambient hydrodynamic conditions.

However, the results suggest that significantly smaller quantity of sand (4,900 yd^3/yr) left the overall project area (Box 2) when compared to the erosion in the immediate fill area. This suggests that the total residence time of the sand in the project area will be much longer than 7 to 8 years, and on the order of 40 to 50 years. Thus, this sand in the nearshore zone could provide relatively long-term erosion mitigation benefits by dissipating wave energy and effectively reducing the amount of wave energy reaching the sub-aerial beach.

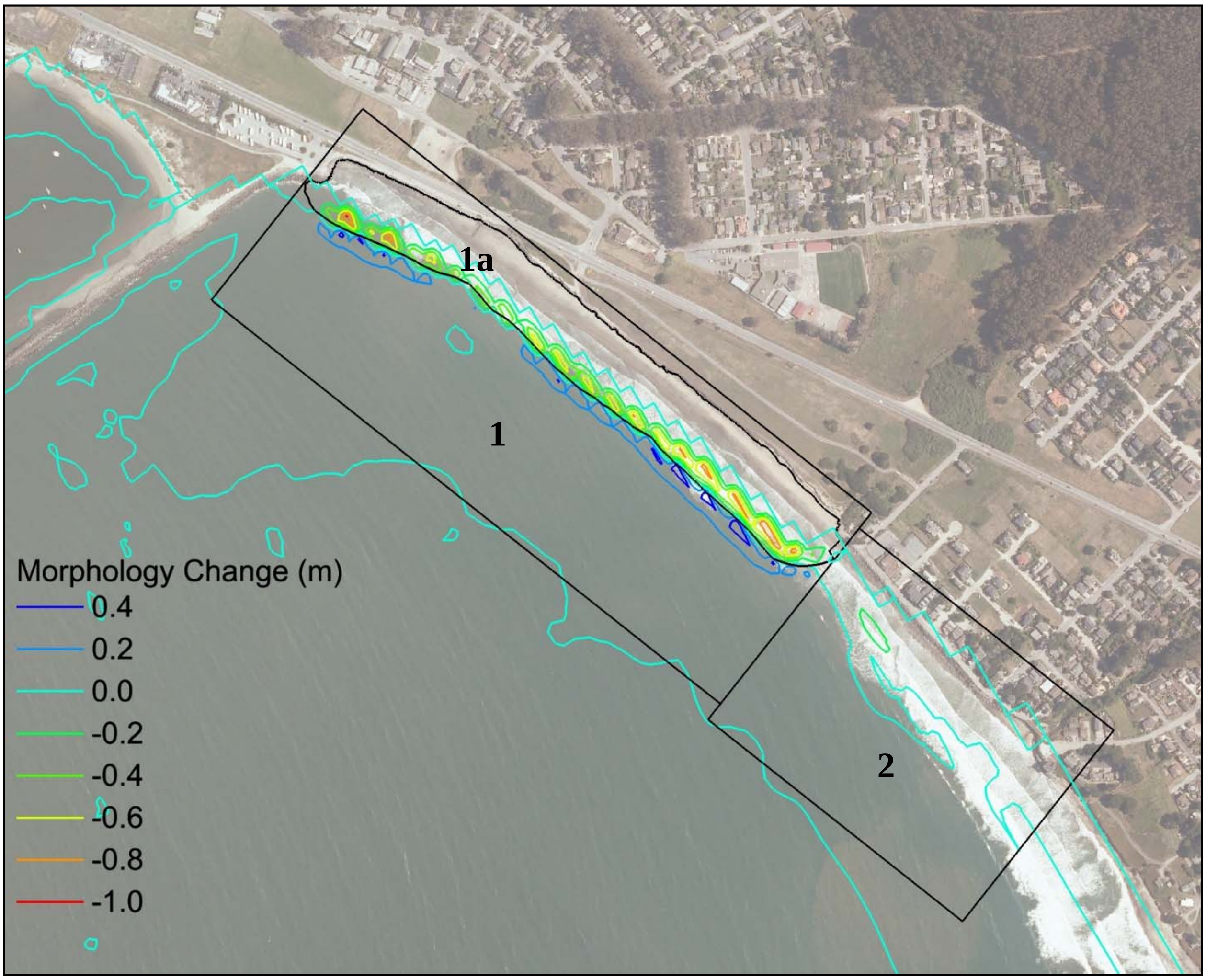

Figure 56. Erosion and accretion in vicinity of beach fill placement (240,000 yd3) from June to November 2009. The boxes outline the analyses areas

Table 28: Erosion and accretion in the fill placement footprint (box 1a).			
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)
Jun to Nov 2009	-16,570	+40	-16,530
Dec 2009 to Mar 2010	-10,750	+170	-10,580
Apr to May 2010	-3,280	+70	-3,210
Total (Morph Change)	-30,600	+280	-30,320

Table 29: Erosion and accretion in the vicinity of the fill placement area (box 1)				
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-16,700	+13,800	-2,900	-690
Dec 2009 to Mar 2010	-11,100	+9,900	-1,200	-230
Apr to May 2010	-3,400	+2,600	-800	+135
Total (Morph Change)	-31,300	+26,300	-4,900	-780

Table 30: Erosion and accretion in the vicinity of the Mirada Rd revetment (box 2)				
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-3,300	+1,500	-1,700	+60
Dec 2009 to Mar 2010	-3,200	+2,500	-700	+680
Apr to May 2010	-1,200	+700	-500	+130
Total (Morph Change)	-7,700	+4,700	-3,000	+870

6.2 Design Measure 2: Medium Beach Fill

As in the case of Design Measure 1, this design measure would involve a one-time placement of sand on an approximately 3,100 foot long section of Surfer's Beach. However, this design measure was developed to represent a more modest sand removal effort from inside the harbor, due to potential environmental and recreational concerns associated with removing large sections of the vegetated sub-aerial beach. Thus, this design measure was formulated under the assumption that only the sand that had accumulated along the east breakwater can be used for placement on Surfer's Beach (Figure 57).

Volumetric computations indicated that the shoal along the east breakwater could yield approximately 140,000 to 150,000 yd^3 of sand if the shoal area is dredged down to a depth (-10 ft, NAVD88) that approximately matches the surrounding bathymetry. As in the case of the maximum beach fill scenario, a target berm crest elevation of 9 ft was used, and iterative volume computations were carried out to determine the maximum berm width that could be constructed with 140,000 to 150,000 yd^3 of sand. As a result, the medium beach fill design will include a 125 ft wide beach berm, with a beach face that will slope down at 12H:1V, until it contacts the existing nearshore bathymetry.

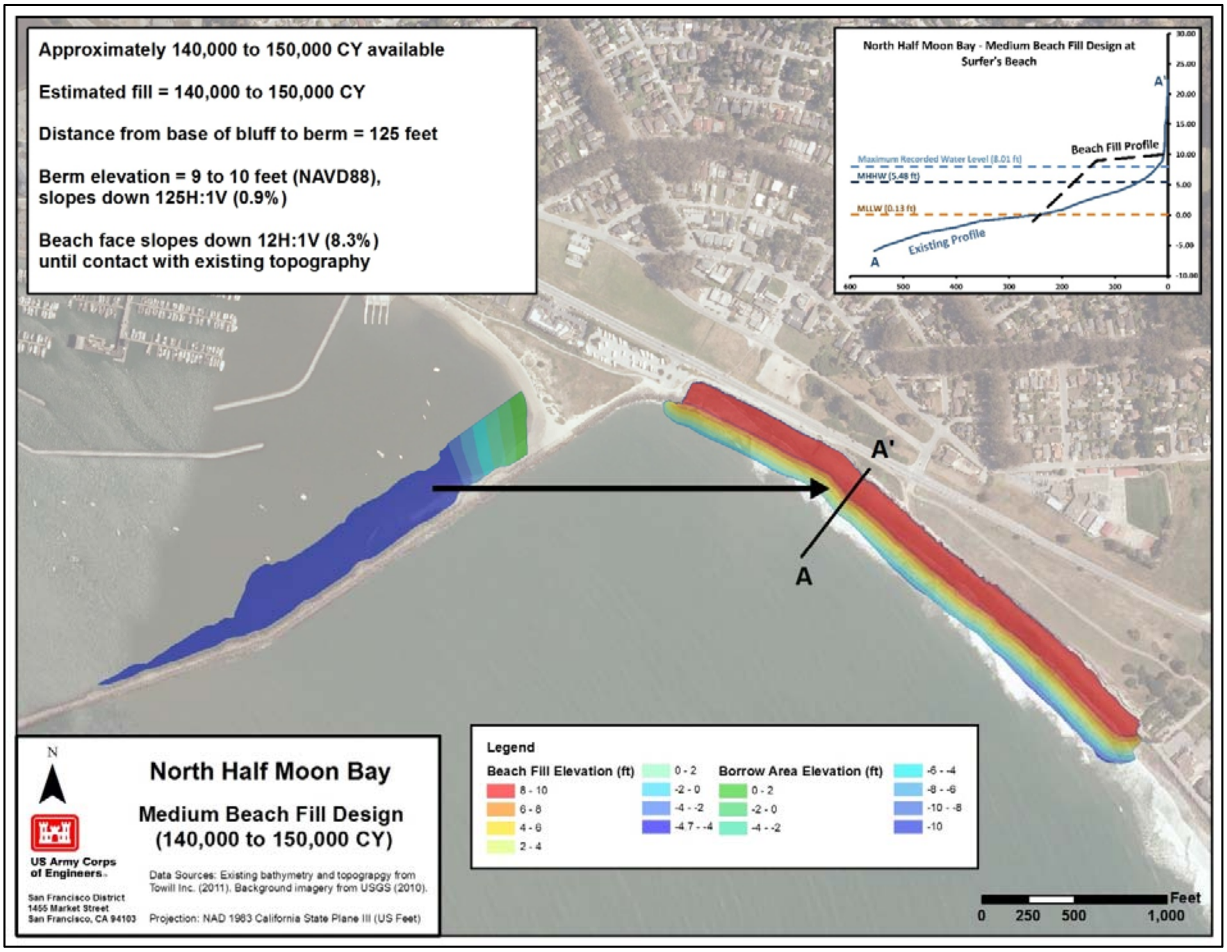

Figure 57. GIS schematic of medium beach fill design scenario including a borrow area along the east breakwater

As in the case of Design Measure 1, the performance of the medium beach fill scenario was simulated over the course of 1 year with the CMS modeling suite. However, this modeling simulation also included changes to the proposed borrow area, in order to evaluate if and at what rate sand might accumulate in the newly dredged shoal area (Figure 58).

It should be noted that SMS (Data Calculator tool) indicated that this beach fill design would require 147,000 yd^3. This is quite close to value (144,000 yd^3) calculated in GIS. However, the volume calculations for the borrow area were quite different in GIS (144,000 yd^3) and SMS (89,000 yd^3).This difference is likely due to the relatively coarse representation of the shoal and adjacent features (e.g., east breakwater) in SMS. In any case, this model run simulated the impact of removing the shoal down to -10 ft or -3.048 m (NAVD88), as the GIS computations (which utilized higher resolution bathymetry) indicated that dredging to this depth will yield 144,000 yd^3.

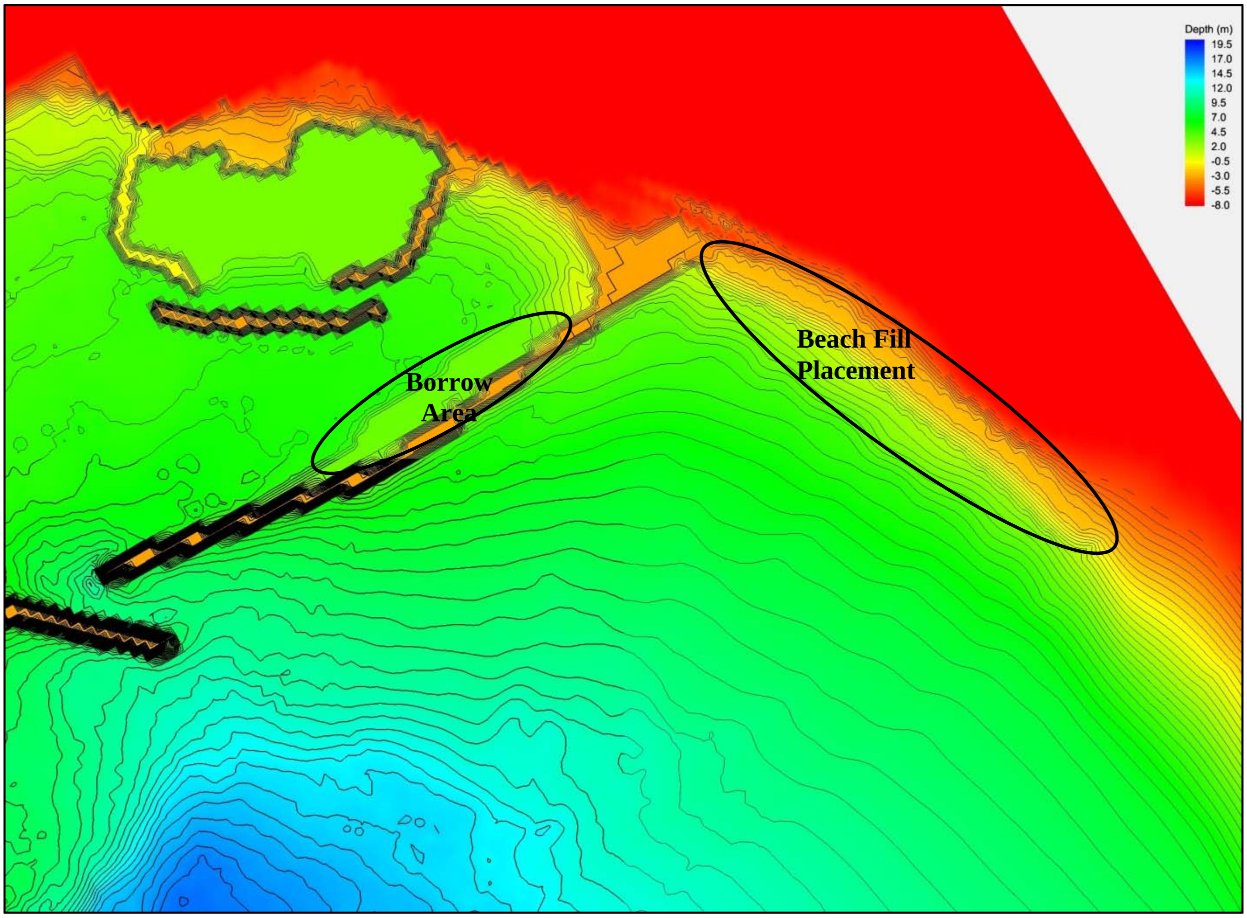

Figure 58. Depth grid used to simulate medium beach fill design (140,000 to 150,000 yd^3) scenario. Note that this scenario includes a borrow area from the shoal adjacent to the east breakwater

The results from the medium beach fill design modeling effort were fairly similar to the results from Design Measure 1 (Figure 59, Table 31, Table 32, Table 33, Table 34). As in the case of the Design Measure 1 simulation, there was net erosion in the vicinity of the fill placement area (box 2) and in front of the Mirada Road revetment (box 3). The net erosion quantity in the fill placement area (box 2) was slightly smaller (+900 yd^3) than in the Design Measure 1 simulation, and there was slightly more net erosion (-500 yd^3) near the Mirada Road revetment (box 3).

The results were also very similar for the fill placement footprint (box 1), with approximately 10 to 15% of the sand eroding within 1 year and the vast majority of this sand moving seaward to the adjacent nearshore zone. A simple linear extrapolation of this erosion rate (24,000 yd^3/yr) would yield a lifespan of approximately 6 years for this (mostly) visible fill placement. However it is anticipated that this erosion rate will decrease after an initial period of adjustment to ambient hydrodynamic conditions, so the expected lifespan of the visible placement is likely longer than 6 years. In addition, the total residence time of the placed sand in the project area could be on the order of 30 to 40 years, given the net erosion rate of 4,000 yd^3/yr in the vicinity of the beach fill placement (box 2).

In terms of the borrow area (box 4), there was little change with only modest erosion of approximately 300 yd^3. This result suggests that sand will not necessarily immediately accumulate in the borrow area under the hydrodynamic conditions simulated by this model run. However, the period of time simulated by this model run did not include extreme wave conditions (often associated with El Niño events), which are thought to be responsible for transporting sand through and over the east breakwater into the shoaling area (USACE, 1996).

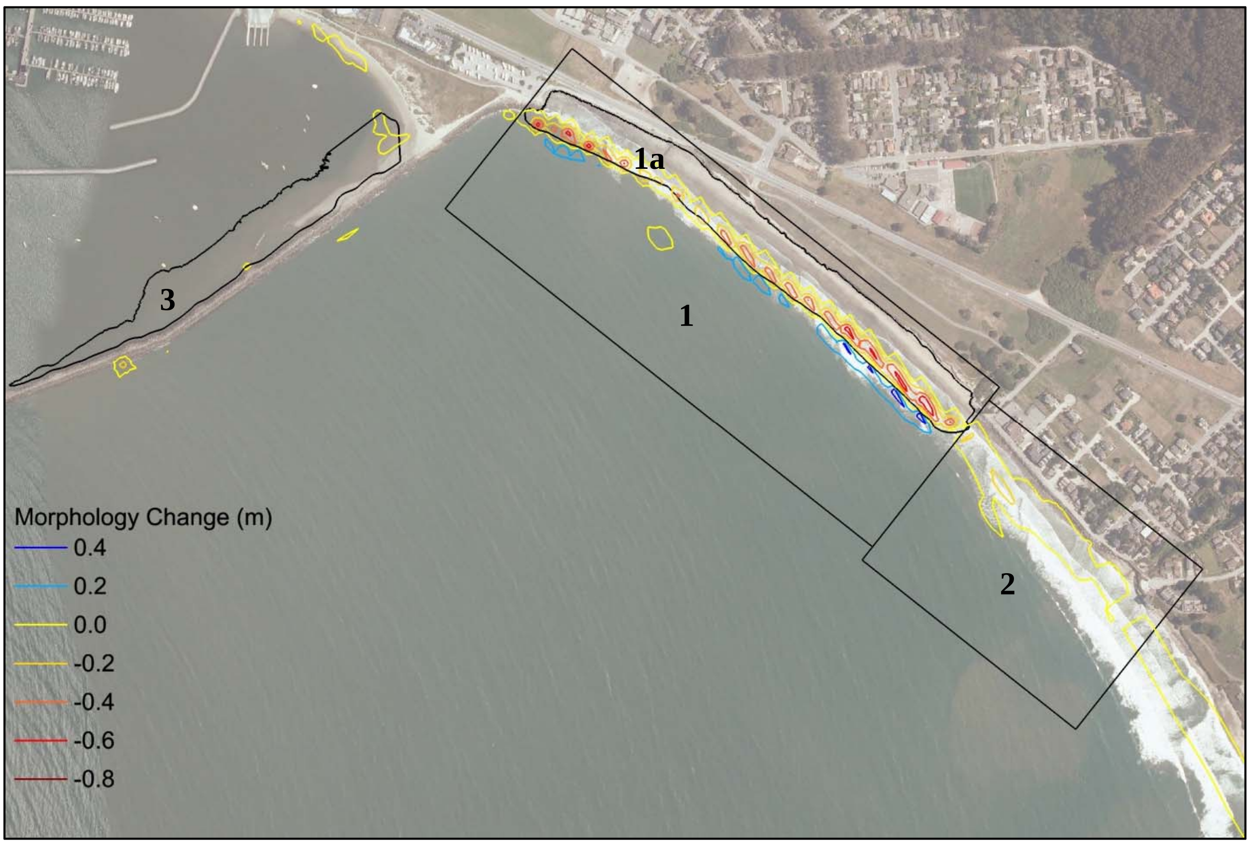

Figure 59. Erosion and accretion in vicinity of beach fill placement (140,000 to 150,000 yd3) and borrow areas from June 2009 to November 2009 (6 months). Boxes outline the analyses areas

Table 31: Erosion and accretion in the fill placement footprint (box 1a)			
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)
Jun to Nov 2009	-12,300	+90	-12,200
Dec 2009 to Mar 2010	-9,000	+300	-8,700
Apr to May 2010	-2,800	+90	-2,700
Total (Morph Change)	-24,100	+500	-23,600

Table 32: Erosion and accretion in the immediate vicinity of the fill placement area (box 1)				
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-12,500	+10,300	-2,200	+60
Dec 2009 to Mar 2010	-9,500	+8,500	-1,000	+20
Apr to May 2010	-3,000	+2,100	-900	+70
Total (Morph Change)	-25,000	+21,000	-4,000	+150

Table 33: Erosion and accretion in the vicinity of the Mirada Rd revetment (box 2)				
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-3,500	+1,700	-1,800	+40
Dec 2009 to Mar 2010	-3,400	+2,300	-1,100	+320
Apr to May 2010	-1,300	+700	-600	+80
Total (Morph Change)	-8,200	+4,700	-3,500	+360

Table 34: Erosion and accretion in the borrow area inside Pillar Point Harbor (box 3). Note that the extent of the borrow area is significantly smaller than Zone 4, which was utilized in the without-project and breakwater modification analyses			
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)
Jun to Nov 2009	-140	+160	+30
Dec 2009 to Mar 2010	-620	+270	-350
Apr to May 2010	-50	+40	-10
Total (Morph Change)	-810	+470	-330

6.3 Design Measure 3: Seal East Breakwater

This design measure would involve modifying a section of the east breakwater in order to prevent sediment from surging through voids in the breakwater into the harbor. While there is considerable uncertainty regarding the pathway(s) of sand into the harbor, this measure was formulated under the assumption that wave induced surging was responsible for a significant amount of sediment transport through the east breakwater. The shoaling analysis from the 1996 USACE Reconnaissance Report indicated that at least a 1,200 ft section of the east breakwater (station 12+00 to 24+00) was subject to significant surging (USACE, 1996). The 1996 analysis further recommended that a 600 ft long section (station 18+00 to 24+00) of the breakwater be sealed in the case that "present shoaling rates are found to be unacceptable or future shoaling rates are to be minimized".

The proposed modification would involve sealing a larger 2,500 ft section of the east breakwater, under the assumption that surging poses a potential problem from near the root of the breakwater (station 10+00) to the bend at approximately station 35+00 (Figure 60). Cost estimates from the 1978 repair of the east breakwater indicated that placing approximately 600 yd^3 of concrete to seal holes along a 1,600 foot section would cost $346,000 (2014 dollars) not including include mobilization/demobilization (USACE, 1978). Given the uncertainty regarding the quantity of concrete needed to seal the breakwater, it is assumed that the cost will likely be in the range of $400,000 to $600,000.

As in the case of Design Measures 1 and 2, the performance of sealing a section of the east breakwater was simulated over the course of 1 year with the CMS modeling suite. It should be noted that the original CMS-Wave and Flow files specified the breakwater cells as part of a "permeable" structure, based on an analysis of site conditions depicted by aerial imagery (Lin, personal communication, 2013). Thus, the CMS-Wave and Flow bathymetry grids and associated files were then altered to make a section of the breakwater "impermeable" to sediment transport.

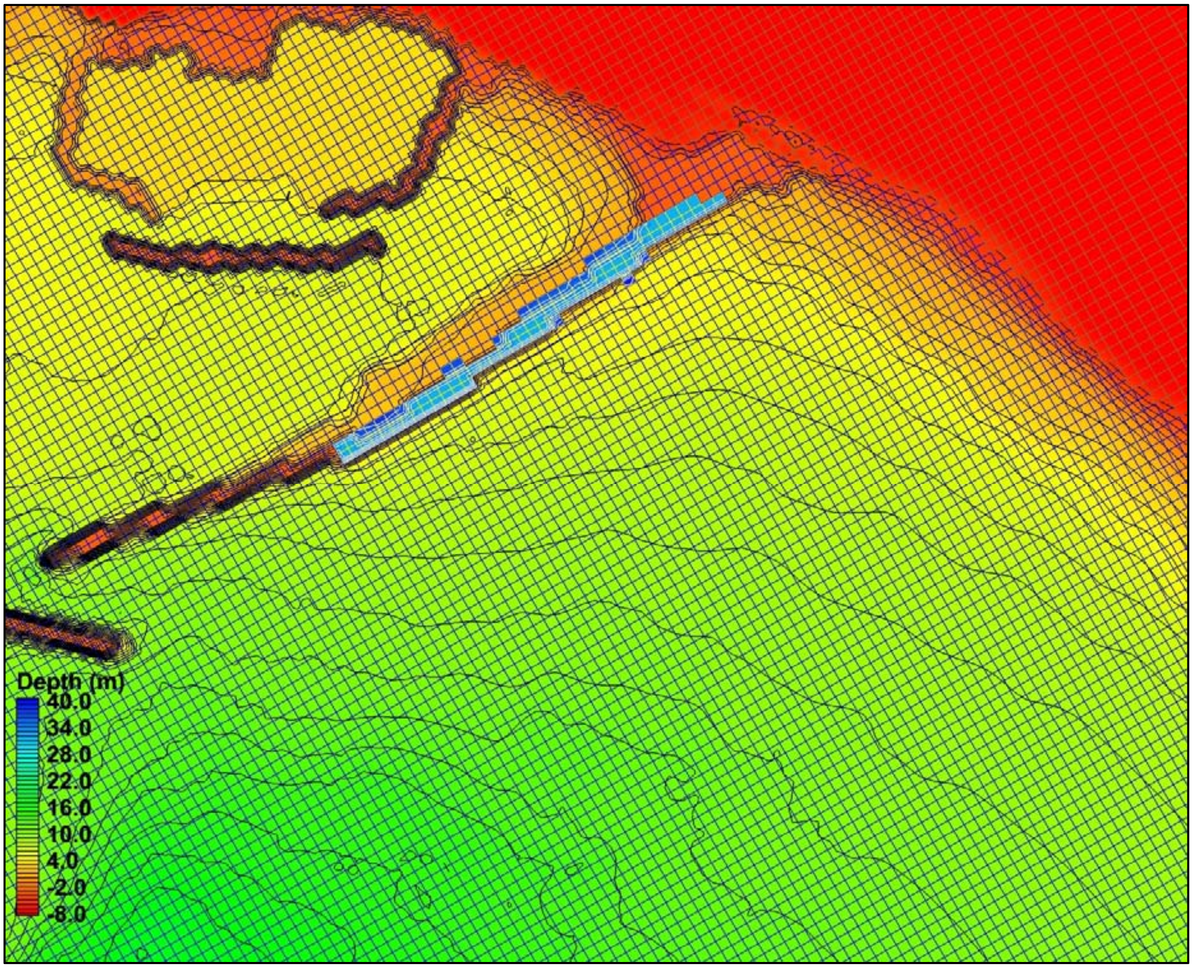

Figure 60. Modified grid with "impermeable" cells highlighted in blue

The results from this scenario (Figure 61) indicated that the sealing of the east breakwater would result in a relatively minor decrease in erosion rates (400 to 500 yd^3 per year) in the vicinity of Surfers Beach (Table 35) and in front of the Mirada Road revetment (Table 36). On the other hand, the rate of accretion inside the harbor was significantly reduced from approximately +2,000 yd^3 per year in the baseline case to approximately +500 yd^3 per year with Design Measure 3 (Table 37). Thus, this model simulation strongly suggests surging through the breakwater is an important sediment transport process, and that sealing the breakwater should reduce shoaling in the harbor.

However, the prevention of shoaling inside of the breakwater (box 3) does not appear to provide immediate (within 1 year) protective benefits along Surfers Beach (box 1) and in front of the Mirada Rd revetment (box 2). The model outputs showed no clearly defined areas of significant accretion, which prompts the question of the fate of the sand that would have been transported through the east breakwater. This question will require a more detailed analysis of the model results, which is not recommended at this time as this design measure does not appear to hold much promise for significantly mitigating erosion in the project area.

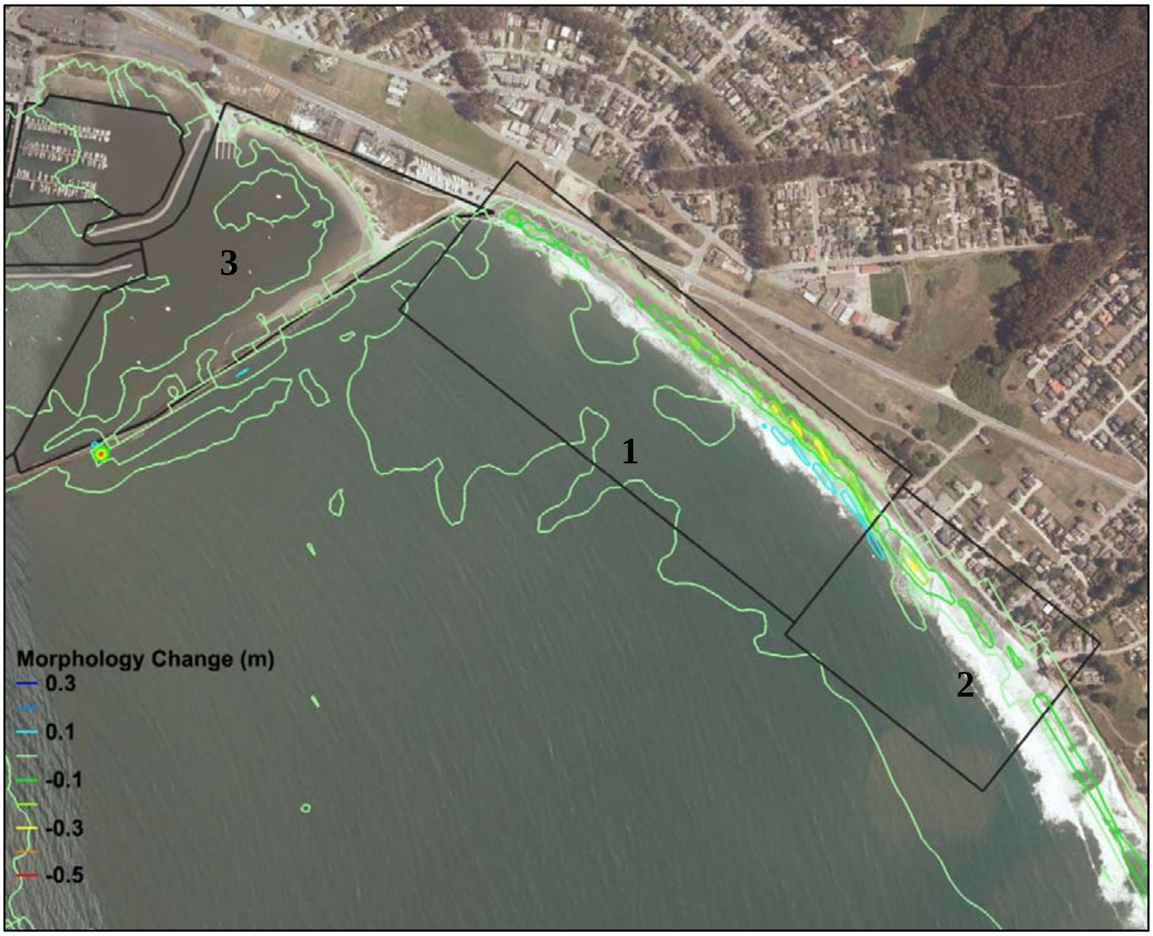

Figure 61. Erosion and accretion from June 2009 to November 2009 (6 months). The boxes outline the analyses areas

Table 35: Erosion and accretion in the immediate vicinity of the fill placement area (box 1)				
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-6,200	+4,200	-2,000	+250
Dec 2009 to Mar 2010	-5,100	+4,000	-1,100	-80
Apr to May 2010	-2,000	+1,400	-600	+290
Total (Morph Change)	-13,300	+9,600	-3,700	+460

Table 36: Erosion and accretion in the vicinity of the Mirada Rd revetment (box 2)				
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-3,500	+1,700	-1,800	+40
Dec 2009 to Mar 2010	-2,900	+1,800	-1,100	+300
Apr to May 2010	-1,300	+700	-600	+60
Total (Morph Change)	-7,600	+4,100	-3,500	+400

Table 37: Erosion and accretion in Pillar Point Harbor (box 3)				
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-160	+340	+180	-700
Dec 2009 to Mar 2010	-150	+310	+160	-750
Apr to May 2010	-80	+130	+50	-210
Total (Morph Change)	-440	+320	+390	-1,660

6.4 Design Measure 4: Notch in East Breakwater

This design measure would involve removing a short (~200 foot long) section of the east breakwater from approximately station 18+00 to 20+00 (Figure 62). This opening in the east breakwater would then allow currents (longshore and tidal) to move sand from the harbor to just offshore of Surfers Beach, where the sand would presumably provide some degree of protection to the beach and bluff toe. This design measure was previously briefly evaluated in the USACE Initial Appraisal (USACE, 2009), and comments during the public meeting on 8 Nov 2013 indicated that there is some local stakeholder support for this measure.

Cost estimates from the east breakwater Repair Design Documentation Report indicate that removing 170 linear ft of the breakwater would cost $230,000 with an additional $39,000 for mobilization/demobilization (USACE, 2006). This would translate into a cost of approximately $1,600 per linear foot of section removed, which could range from $240,000 to $320,000 if 150 ft and 200 ft are removed, respectively. Thus, this measure would be significantly less expensive than the beach fill design measures.

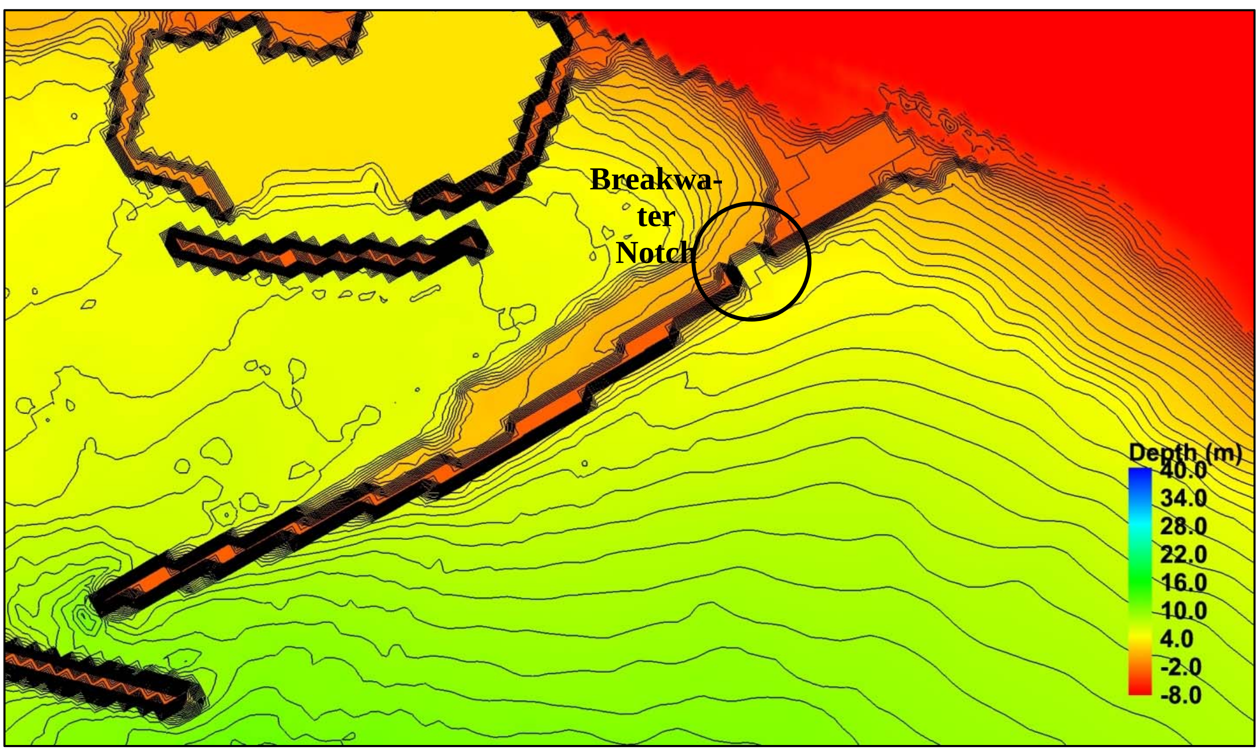

Figure 62. Bathymetric grid depicting the ~ 200 foot notch in the east breakwater

As in the case of Design Measures 1 through 3, the performance of a 200 foot long notch in the east breakwater was simulated over the course of 1 year with the CMS modeling suite. This was accomplished by altering the bathymetric Wave and Flow grids to include a notch with a depth that matched the bathymetry on the ocean side of the breakwater (Figure 63). In addition, the relevant lines were removed from the associated advanced cards and structure files.

The cells depicting the shoal along the inside of the east breakwater were specified as "limited erodible" with a limit of 10 ft (~3 m) in order to allow for up to 10 ft of erosion of the sand (Figure 64). This limit was based on the assumption that the sand in this area was at least 10 ft deep per hydrographic surveys (Towill Inc., 2011), and would be subject to erosion under the certain hydrodynamic conditions (e.g., high current velocities). In addition, the cells depicting the bottom of the newly created notch were specified as "limited erodible cells" with a limit of 2 ft (Figure 65), in order to ensure that the model would not be overly aggressive in depicting scour (Lin, personal communication, 2013).

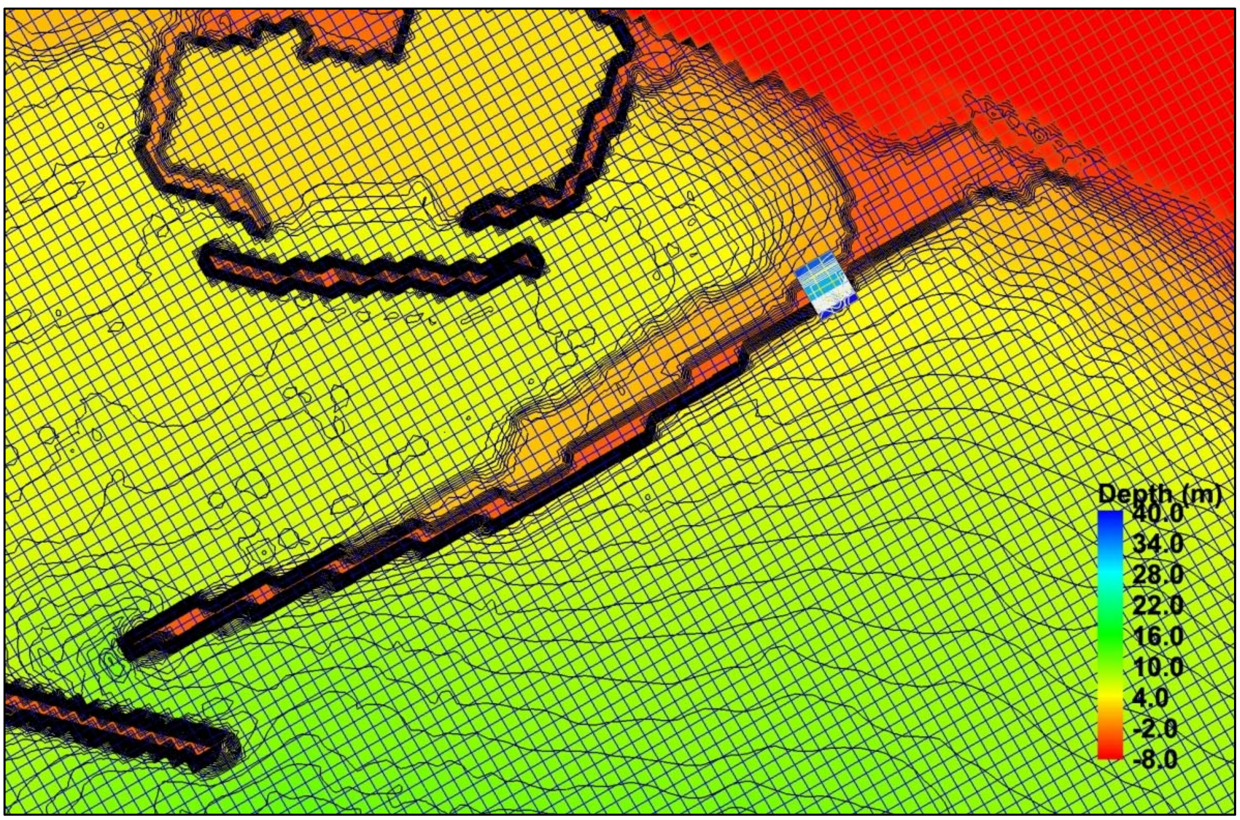

Figure 63. Bathymetric cells that were altered to depict a breakwater notch (cells highlighted in blue)

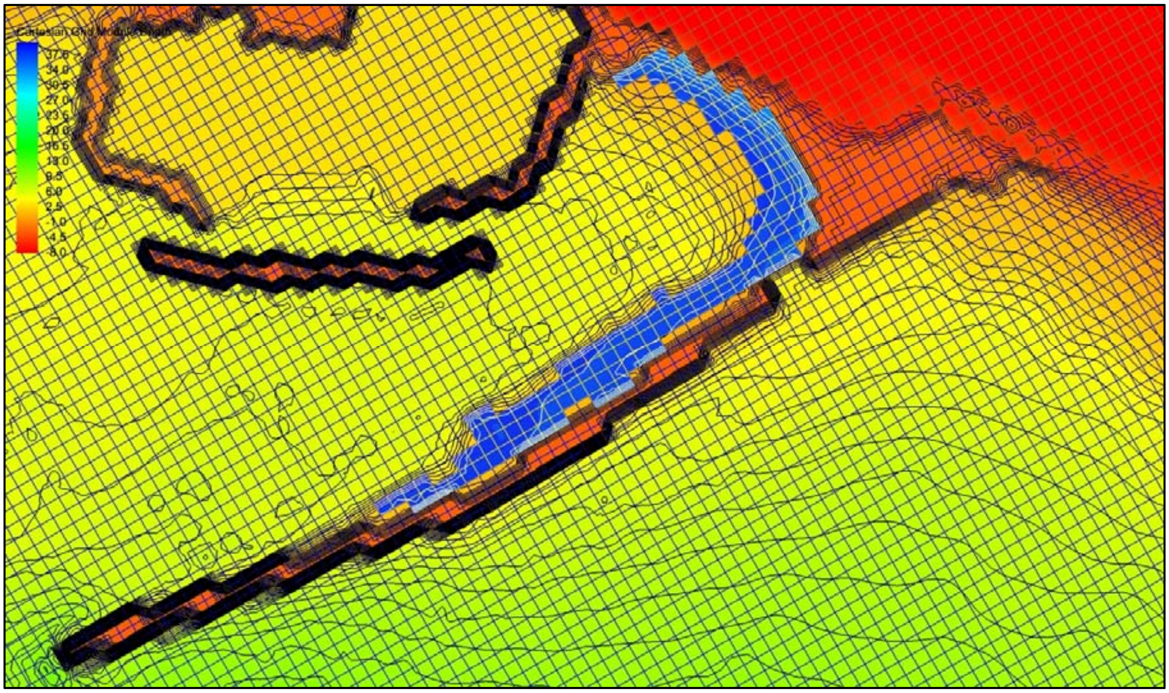

Figure 64. Cells specified as "limited erodible cells" with a limit of 3 m, as it was assumed that sand in these areas was at least 3 m (10 ft) deep

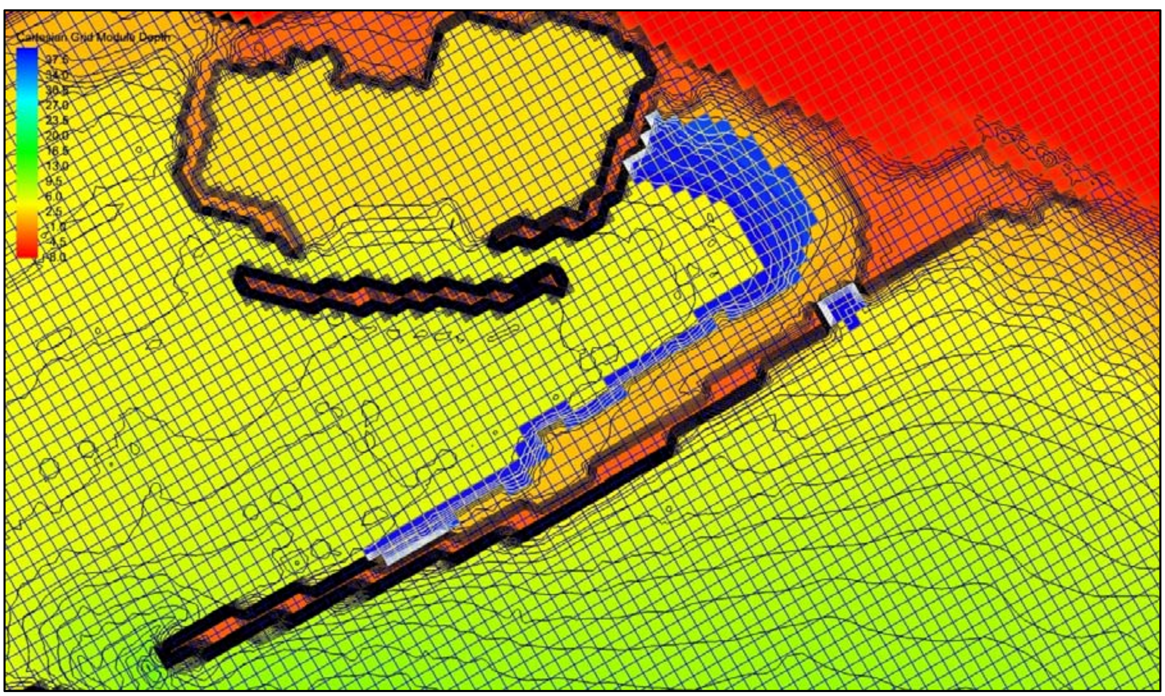

Figure 65. Cells specified as "limited erodible cells" with a limit of 0.6 m, as it was assumed that sand in these areas was at least 0.6 m (2 ft) deep

The results from the simulation of this design measure (Figure 62) indicated that creating a small opening in the breakwater could have some (albeit minor) impacts on erosion rates at Surfers Beach and seaward of the Mirada Road revetment (Table 38 and Table 39). Net erosion at both locations was decreased by 500 to 600 yd^3 per year, with net accretion decreasing by 700 yd^3 inside the harbor (Table 40). In addition, the model depicted a relatively small area of scour just inside of the notch (Figure 66), which suggests that the notch in the breakwater facilitates the formation of relatively strong currents.

The above results suggest that at least a small quantity of sand, which would have been deposited in the harbor, was transported through the notch from inside the harbor to the project area. However, this quantity of sand (500 to 1000 yd^3) is likely not great enough to immediately mitigate the present erosion concerns. In addition, while there was a decrease in the rate of accretion inside the harbor, there was not a definitive transition to an erosional regime within harbor. Thus, the placement of a notch in the breakwater will probably not result in the immediate erosion of material from the shoal, and therefore will not achieve the second design objective.

In addition, the opening of a notch could alter hydrodynamic conditions inside the harbor in such a way to potentially create navigation hazards. For example, a cursory analysis of model outputs indicates that wave heights could slightly increase adjacent to the inner breakwaters in the event of construction of the notch (Figure 67 and Figure 68). As a result, there will need to be detailed analysis of potential hydrodynamic impacts associated with this design measure, which could add to the overall design cost. Thus, these factors provide sufficient justification for not further pursuing this design measure, given the need for immediate erosion mitigation benefits.

Figure 66. Morphology change associated with a breakwater notch from June to November 2009

Table 38: Erosion and accretion in the immediate vicinity of Surfers Beach (box 1)

Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-6,100	+4,150	-1,950	+270
Dec 2009 to Mar 2010	-4,900	+3,900	-1,000	+ <10
Apr to May 2010	-2,000	+1,350	-650	+280
Total (Morph Change)	-13,000	+9,400	-3,600	+560

Table 39: Erosion and accretion in the vicinity of the Mirada Rd revetment (box 2)

Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-3,450	+1,700	-1,750	+50
Dec 2009 to Mar 2010	-2,900	+1,800	-1,100	+300
Apr to May 2010	-1,250	+650	-600	+60
Total (Morph Change)	-7,600	+4,150	-3,450	+410

Table 40: Erosion and accretion in Pillar Point Harbor (box 3)				
Time Period	Erosion (yd^3)	Accretion (yd^3)	Net (yd^3)	Difference from Baseline (yd^3)
Jun to Nov 2009	-1,240	+1,800	+550	-330
Dec 2009 to Mar 2010	-1,600	+2,200	+600	-300
Apr to May 2010	-540	+720	+180	-80
Total (Morph Change)	-3,380	+4,710	+1,330	-710

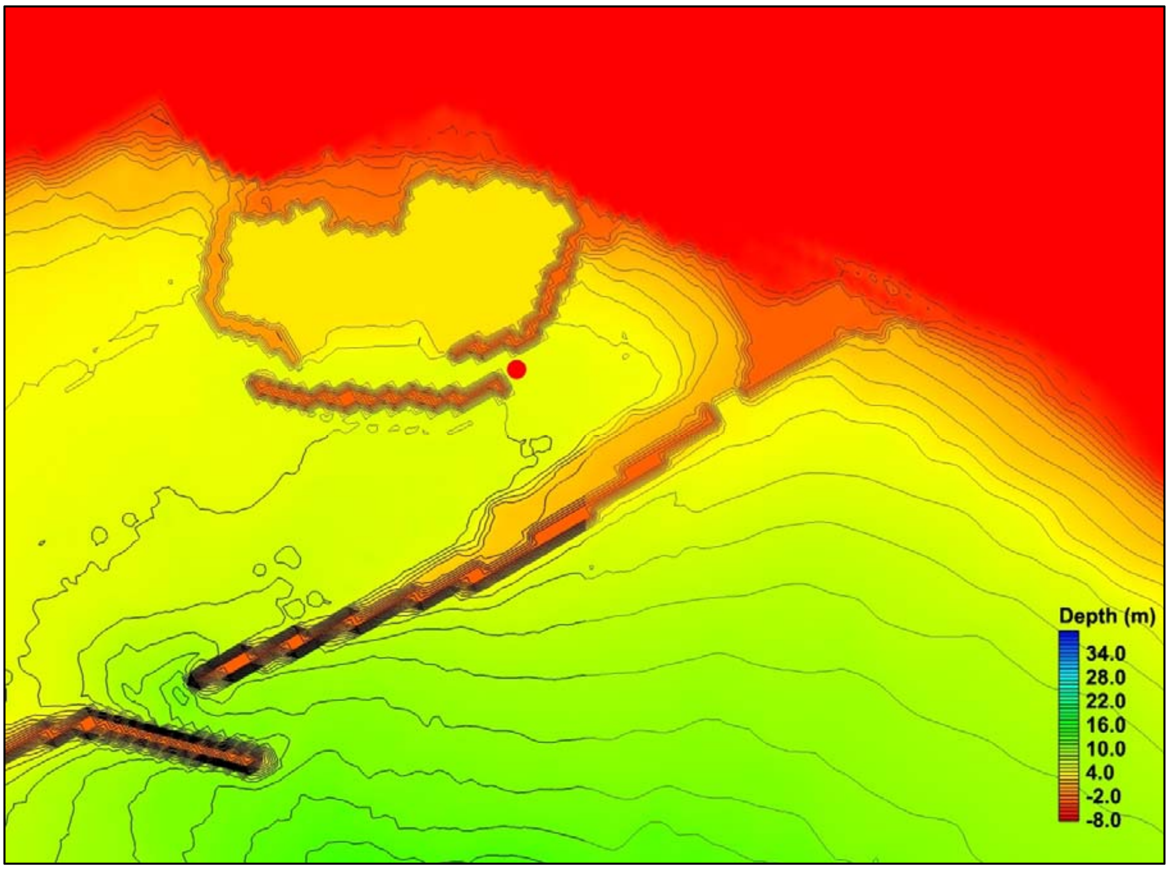

Figure 67. Location of time series of wave heights derived from the baseline condition and breakwater notch model runs

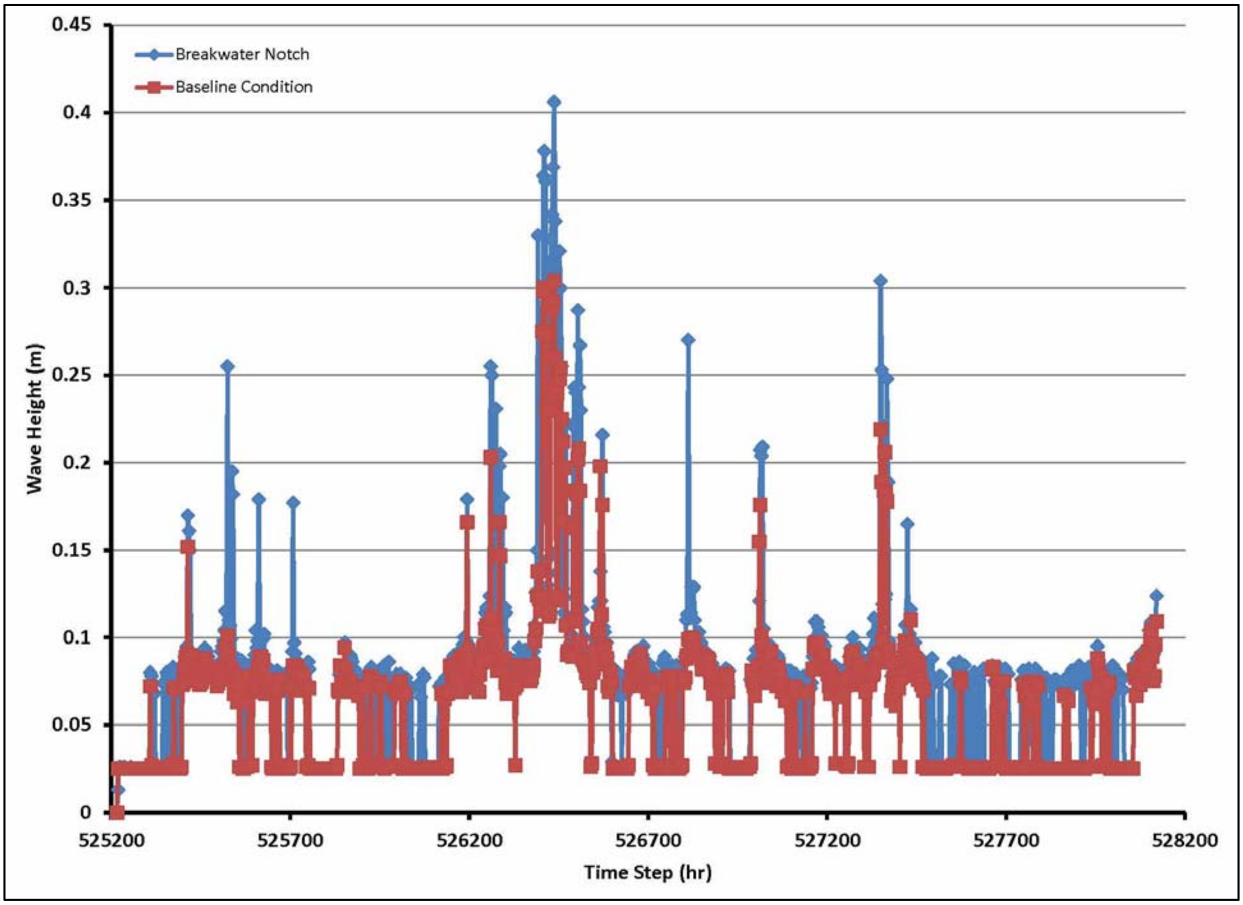

Figure 68. Wave heights near the inner breakwaters for the breakwater notch design measure and baseline condition from Dec 2009 to March 2010

6.5 Design Measure 5: Alternative Dredged Material Placement

This design measure would involve the continuous removal of sand from the shoal along the east breakwater, and placement of the sand near the harbor entrance. Strong ebb currents in the harbor entrance would then transport the sand out of the harbor, and into adjacent areas outside of the harbor mouth, thereby returning the sand the local littoral cell. Once in the local littoral cell, a portion of the sand could move shoreward, and perhaps provide some degree of protection to Surfer's Beach by dissipating wave energy offshore.

This design measure was formulated based on input from a local stakeholder following the public meeting on 8 November 2013, and had not been investigated by USACE in previous studies. Thus, this measure represents a novel approach to addressing the shoaling problem in the harbor, and this analysis will determine if the measure holds sufficient promise to justify further numerical modeling, design and cost estimating efforts.

There is considerable uncertainty regarding the cost of this measure, as it will require establishing a more or less permanently operating dredge and pipeline. Cost information from a nearby hydraulic dredging and pumping operation (Oakland Middle Harbor Enhancement Area) indicates that dredging and redistributing sediment would cost approximately \$11 per yd^3, with an additional \$640,000 for mobilization/demobilization (USACE, 2011b). This would translate into a cost of \$2.3 million if 150,000 yd^3 of sand is moved from the shoal to the vicinity of the harbor entrance. In addition, cost information from Moss Landing Harbor suggests that dredging operations in this type of coastal harbor could cost up to \$25 per yd^3, which translates to a cost of \$4.7 million including mobilization/demobilization (USACE, 2013). Thus, it is estimated that this design measure will cost between \$2.3 and \$4.7 million, with additional uncertainty regarding the cost of operating a more or less permanently operating pipeline.

In order for sand to be transported, local currents must exceed a given threshold velocity. Calculations performed as part of the 1996 USACE shoaling analysis indicated that the threshold velocity for sand within the harbor (assuming a d_{50} of 0.1 mm) is approximately 0.37 ft/s or 0.11 m/s (USACE, 1996). Current measurements from the harbor mouth indicate that this threshold is frequently exceeded under "typical" hydrodynamic conditions, although the strongest measured currents were associated with flood tides at this particular location (Figure 3 and Figure 19). This suggests that placement of sand directly in the harbor mouth would likely result in the sand being circulated back into the harbor by the strong flood tide currents. Thus, further analysis is required to determine if there are other suitable locations for sand placement, particularly along the east breakwater.

In addition to the field measurements, the CMS model simulation of baseline conditions provided a basis for visualizing how sand might be transported from a placement area to outside the harbor. An analysis of modeled current direction and magnitude suggest that sand placed along the harbor side of the east breakwater near the entrance would likely be transported out of the harbor (Figure 69 and Figure 70). This hypothesis is supported by a recent harbor circulation study, which utilized dye distribution and identified a zone of scour just inside of the head of the east breakwater (Wuertz et al., 2011).

However, it appears that most of this sand would be transported to the relatively deep (~30 ft) water directly offshore of the entrance (Figure 71). If the sand were to settle on the seafloor at this depth, it is unlikely that the relatively small waves associated with beach building would be able to mobilize and transport this sand onshore. Thus, this cursory examination of modeled currents suggests that placement of sand along the east breakwater will not necessarily provide shoreline erosion mitigation benefits for Surfers Beach.

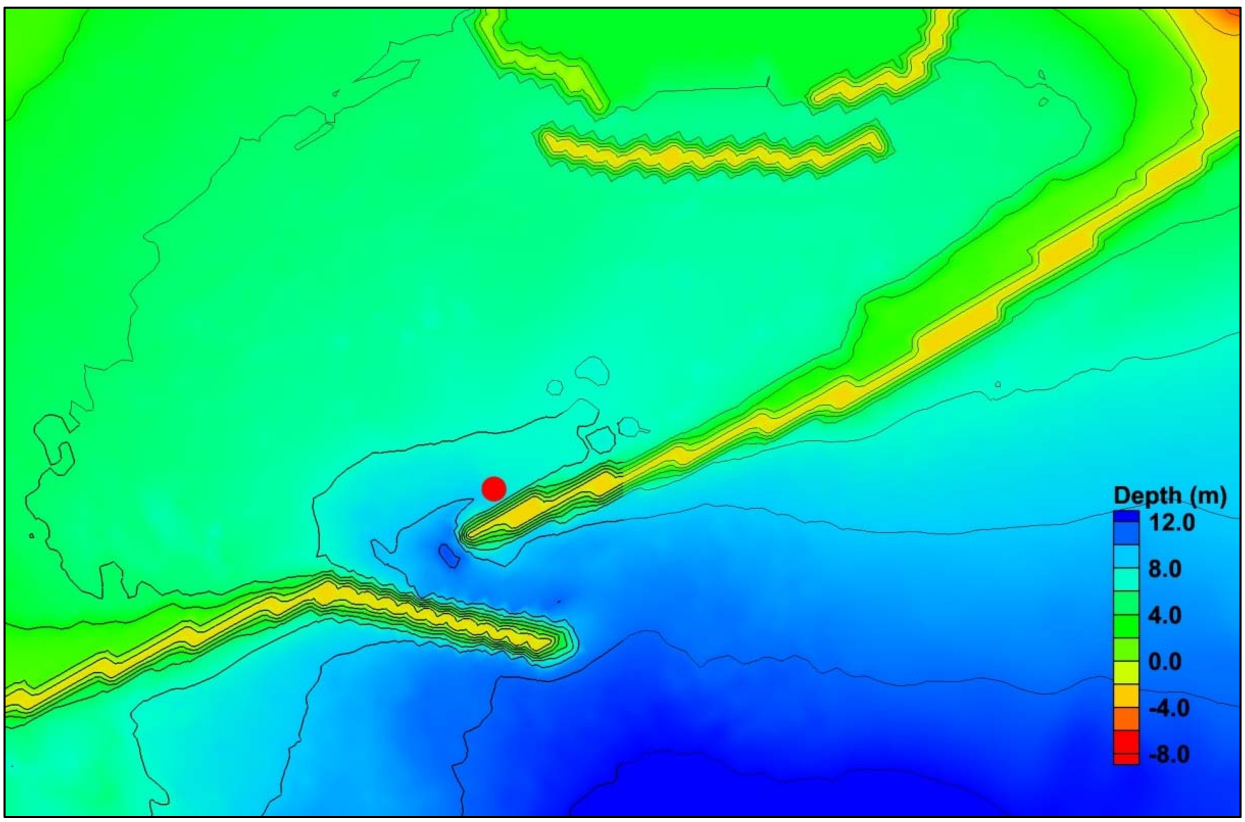

Figure 69. Location of modeled current magnitude and direction data used to evaluate Design Measure 5

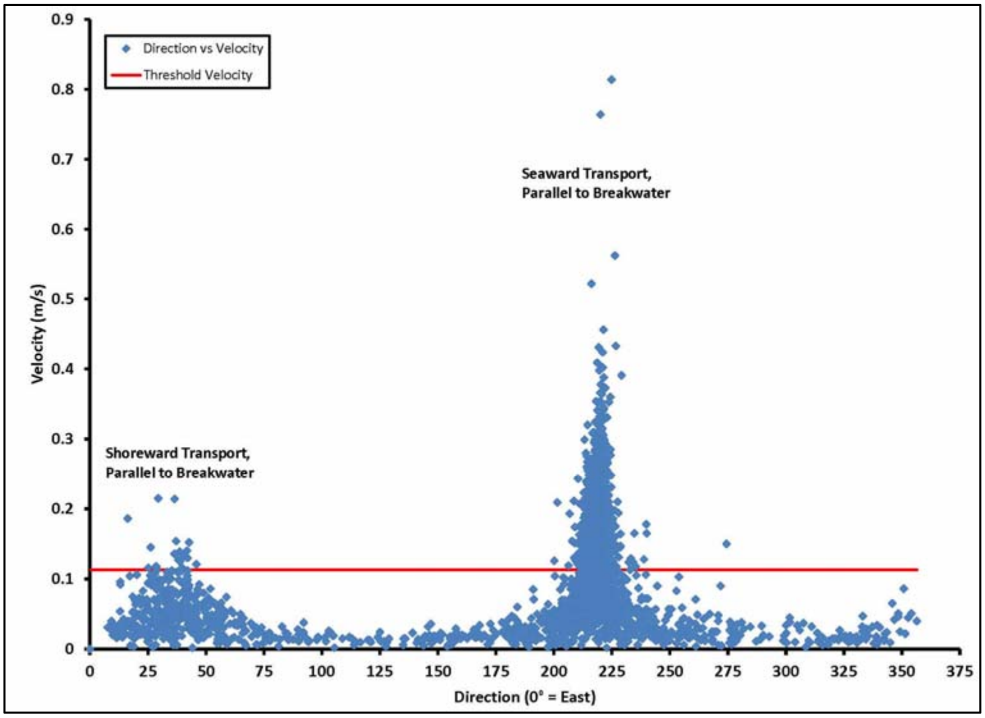

Figure 70. Modeled current magnitude and direction data
(see Figure 3 for location)

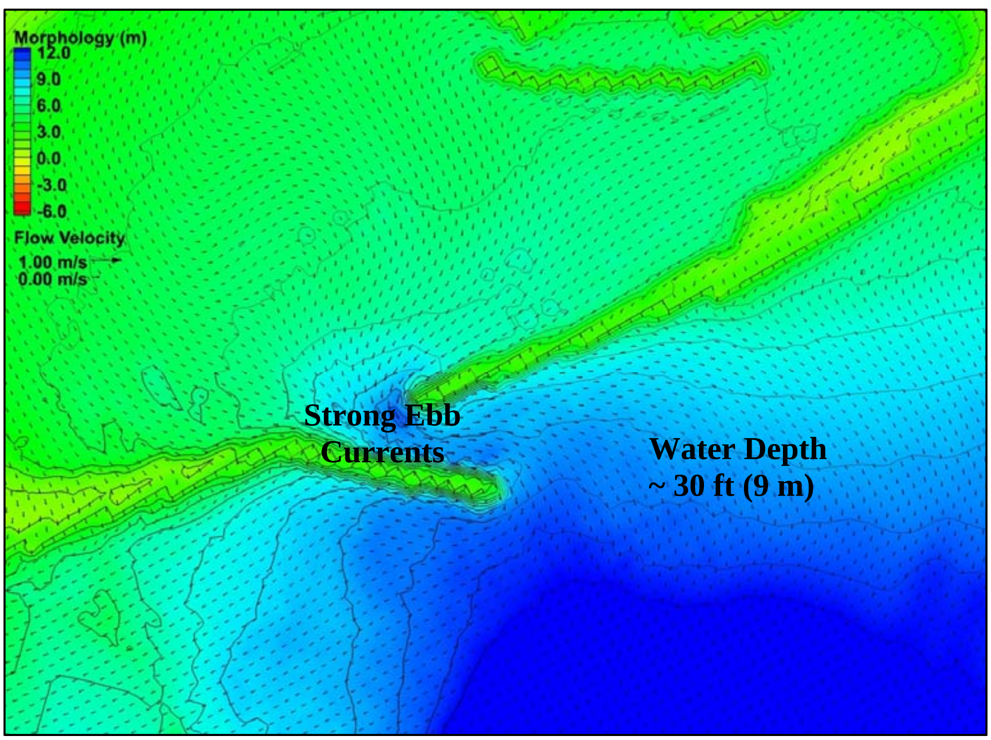

Figure 71. Modeled flow field associated with the typical ebb tide conditions

In addition to the inferences drawn from the measured and modeled current data, there are two other factors that add to the uncertainty of the efficacy of this design measure. First, the placement of sand near the entrance channel has the potential of having unintended consequences, including unexpected shoaling and associated navigation hazards. Second, transporting sand from the shoal to the placement area would require a more or less permanently operating pipeline. The operation of this pipeline would result in significant maintenance costs, particularly if this measure is compared to the single beach fill placement design measures, which would more efficiently directly place sand in the desired location (e.g., Surfers Beach). Given the uncertainties associated with this design measure and the significantly greater chances of achieving the design objectives with other measures (e.g., beach fill placement), no additional analyses are recommended at this time.

6.6 Design Measure 6: Spur Breakwater

This design measure would involve constructing a spur (deflector-arm) breakwater that would extend southeast from the existing east breakwater. The spur breakwater would be oriented parallel to the shoreline, where it would significantly reduce the wave energy reaching the shoreline and potentially induce accretion of sand in the protected area. This sand would in turn reduce the impacts of elevated water levels and wave attack on the bluff toe.

This design measure was a key component of several of the alternate plans considered by the previous USACE Beach Erosion Control Report (USACE, 1971). The 1971 report evaluated the efficacy of a 600 foot long rubble-mound spur breakwater, in conjunction with other measures such as groins, a seawall, and beach fill (Figure 72). This evaluation involved the formulation of design criteria, cost estimates and an economic analysis. The report did not evaluate the spur breakwater as a stand-alone measure, and assumed that it would be charged with 230,000 yd^3 of beach fill.

The 1971 report also included a summary (Appendix D) of findings from an investigation of several deflector-arm breakwater configurations with a 1:100-scale hydraulic (physical) model. The investigation found that the presence of a 500 foot long deflector-arm breakwater “causes formation of desired eddy effect behind this structure and also diverts the attack of the littoral currents down beach about 1,000 ft from the junction of the east breakwater and shore.” It should be noted that the physical modeling appendix did not provide an estimate of how much sand would be retained by a spur breakwater structure.

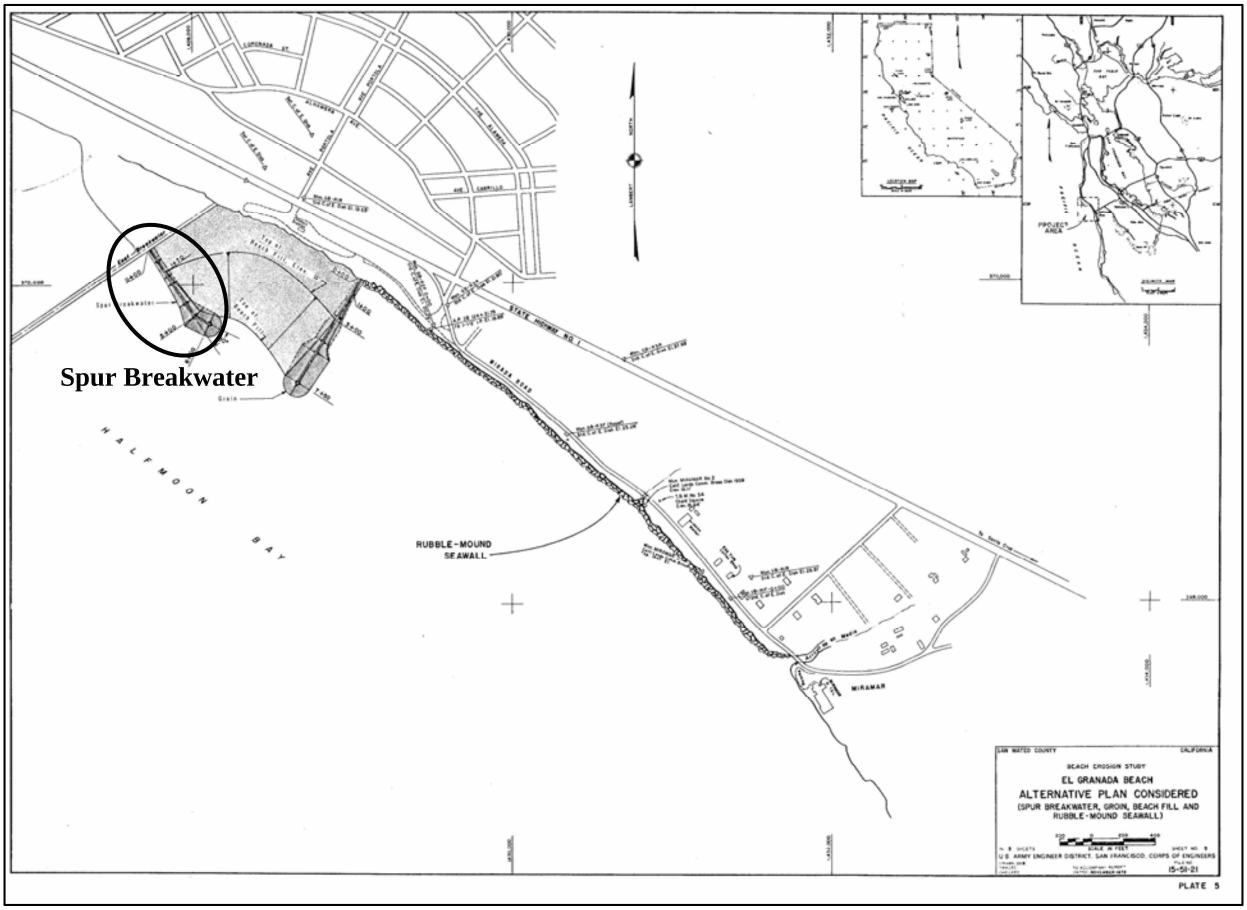

Figure 72. Schematic from the El Granada Beach Erosion Control Report showing the proposed spur breakwater and other design measures (USACE, 1971)

The proposed spur breakwater would have been constructed with a crest elevation sloping down from 10 ft (MLLW) at the east breakwater to 7 ft (MLLW) at the head. The crest would be 15 ft wide with side slopes ranging from 1.75:1 to 3:1, and a layer of 8 to 10 ft of A1 armor stone (4 to 6 tons) over B2 and C underlayer stone (Figure 73). The total first cost of the spur breakwater was estimated to be $3.2 million if the proposed 230,000 yd^3 of beach fill is not included in the estimate.

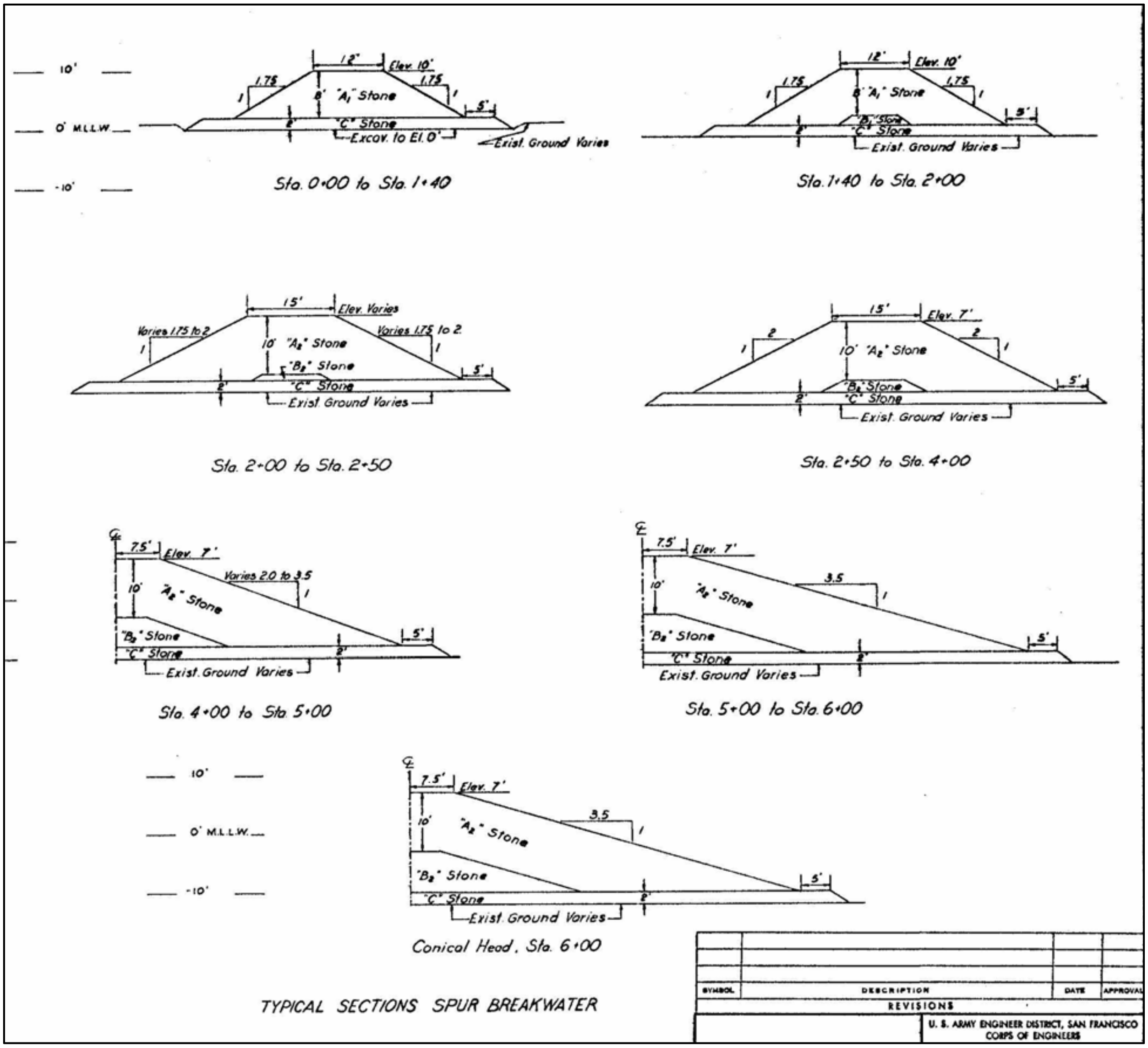

Figure 73. Sample cross-section of proposed spur breakwater, from the El Granada Beach Erosion Control Report (USACE, 1971)

In the end, the report concluded that the alternatives which included the spur breakwater would be too expensive, given the high total first cost associated with the beach fill component. Thus, the report recommended the construction of a 4,600 foot long rubblemound seawall (revetment). Cost continues to be a concern, as a more recent cost estimate (USACE, 2006) for the repair of a 170 foot long section of the east breakwater suggests that breakwater construction could cost up to $4,200 per linear foot (USACE, 2006). This would translate into a construction cost of approximately $2.5 million for a 600 foot long spur breakwater, and this cost would not include beach fill in the newly protected area.

Given the estimated high cost of this design measure and the potential erosion mitigation benefits being limited to a small area, it is unlikely that there will be a favorable benefits-to-cost ratio associated with this measure. This measure would also likely have significant impacts on the popular surf break adjacent to the breakwater, and the recent public meeting (8 November 2013) suggests that there is only limited support for this measure. In addition, construction of this measure would involve a significant alteration to the sea floor, which could raise serious environmental concerns, given that that the footprint of this measure is in the Monterey Bay National Marine Sanctuary (MBNMS).

Thus, additional analyses and modeling of this design measure are not recommended, as the limited resources available for this study should be devoted to pursuing more feasible design measures.

6.7 Design Measure 7: Managed Retreat

This design measure would involve the relocation of infrastructure to accommodate future erosion of the unprotected marine terrace (bluff) backing Surfer's Beach. This design measure was formulated under the assumption that the current erosion mitigation measures (revetments) would be maintained by the respective agencies (Caltrans, County of San Mateo) perpetually into the future. Thus, only the infrastructure that is not currently protected by well-maintained revetments would need to be relocated under this design measure.

The extent of infrastructure that would need to be relocated was evaluated by utilizing an estimated bluff retreat rate (1.64 ft/yr) to project the location of the bluff edge 10 and 50 years into the future (Figure 74). This retreat rate was estimated using a series of georeferenced aerial images from 1993 to 2012, and a detailed description of the bluff analysis is provided in Section 4.1 of this appendix. The 10 and 50 year projections were based on the assumption that the bluff retreat rate would continue into the future at a constant rate. However, in reality bluff erosion tends to be more episodic with periods of very little erosion punctuated by major storms and other forcing factors. Thus, the projections represent an approximate estimate based on the past 20 years, and retreat rates could exceed this estimate if a series of anomalously "stormy" (e.g., strong El Niño conditions) winters were to occur in the near future.

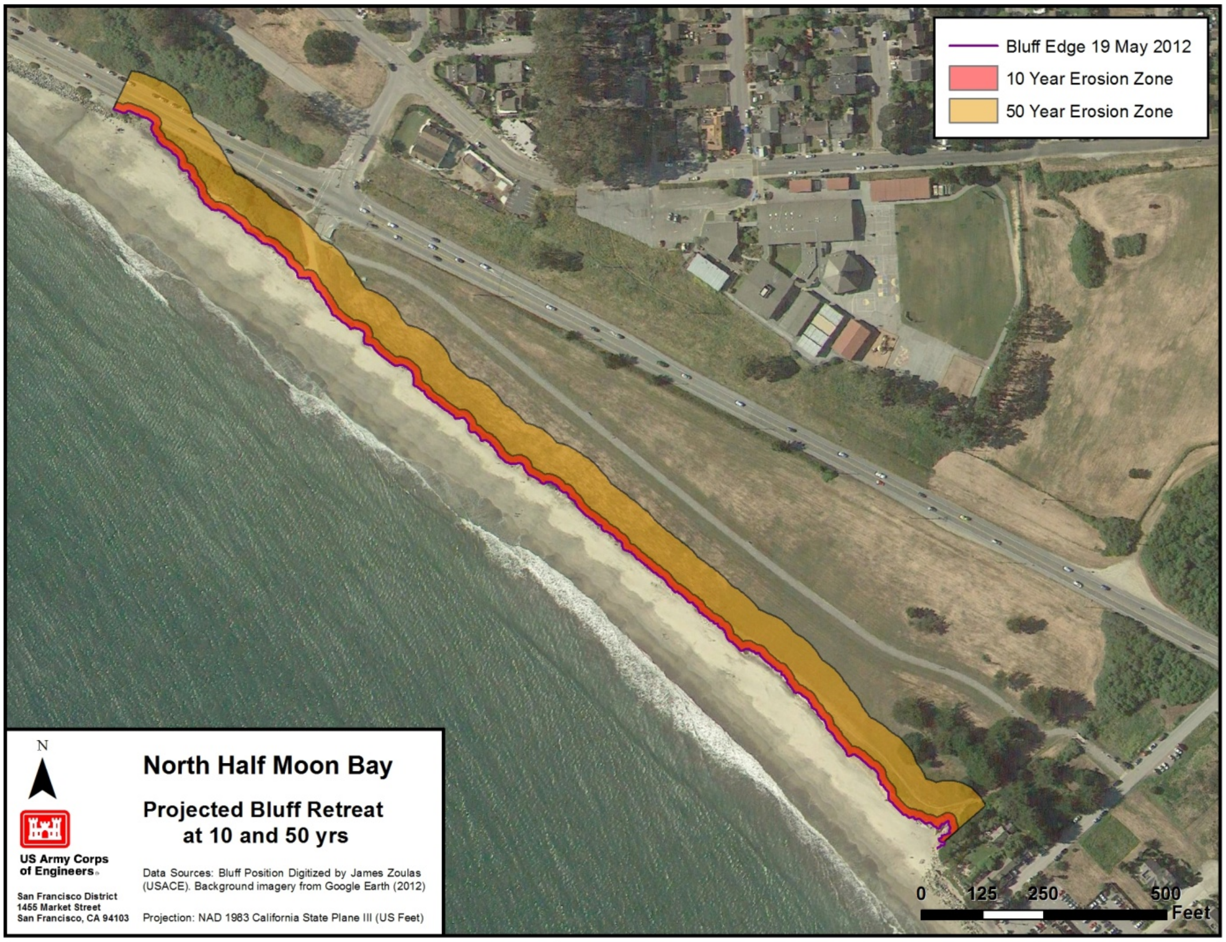

Figure 74: Extent of projected bluff retreat at 10 and 50 years, based on the estimated retreat rate of 1.64 ft (0.50 m) per year

The projected bluff retreat rate indicates that an approximately 80 foot long section of the southbound shoulder of Highway 1 is expected to be undermined in the next 10 years, with approximately 250 ft at risk in the next 50 years. Sections of the Coastal Trail located seaward of the highway are also at risk, with a 25 foot long section of the pathway at the terminus of the revetment already being actively undermined by erosion. Thus, these sections of highway and recreational pathway will likely need to be relocated if no other erosion mitigation measures are implemented in the near future.

However, it is unlikely that only a small section of highway would be relocated, as moving even a small section would require realignment of a much longer section of roadway. For example, the Highway 1 Safety and Mobility Improvement Study (Local Government Commission.et al., 2010) proposed several Highway 1 realignment alternatives, all of which would involve realigning an approximately 4,400 foot long section of highway (Figure 75). This study only proposed conceptual level plans and did not include detailed cost estimates. However, a study of a comparable situation, the Great Highway at Ocean Beach in San Francisco, indicates that relocating a coastal highway could cost $3,700 per linear foot (Moffatt & Nichol, 2007). Thus, relocating 4,400 ft of highway could cost over $16 million, not including the additional costs associated with obtaining the necessary real estate and right-of way.

Figure 75. One of three conceptual plans for realigning Highway 1 from Capistrano Road to Coronado Street (from Local Government Commission et al., 2010)

Therefore, it is recommended that no additional analysis of this design measure be performed at this time, given the complexity and high cost of this design measure.

6.8 Comparison of Design Measures

In order to provide a basis for comparison, the estimated costs, anticipated effectiveness, and potential concerns for each design measure are presented in Table 41. The cost estimates for the beach fill design measures were prepared by the SPN Cost Engineering section in accordance with Engineering Regulation 1110-2-1302 (USACE, 2008). All other estimates were performed by SPN Water Resources staff based on information from similar past projects, and are not official USACE cost estimates. Effectiveness is a qualitative evaluation of whether a given design measure meets both of the design objectives and the overall planning objective(s) of the project (USACE, 2000). In addition, the table also includes potential concerns associated with each design measure, which might need to be taken into account during the plan formulation process.

There is a wide range of anticipated effectiveness for the respective design measures, with a couple (Alternative Dredged Material Placement, Managed Retreat) not expected to meet both of the design objectives. Several of the measures (Breakwater Sealing, Breakwater Notch, Spur Breakwater) might be effective at addressing one, but not both, of the design objectives. Thus, there are only two measures (Maximum and Medium Beach Fill) that will likely meet both of the design objectives.

The respective efficiency of the two beach fill measures can be further evaluated by examining the relationship between the quantities of fill and the anticipated residence time of the sand placed in the nearshore environment. This additional sand in the nearshore environment is expected to mitigate beach and bluff erosion by dissipating wave energy further offshore. The Maximum Beach Fill measure would require up to 250,000 yd^3 of sand, and provide up to 50 years of erosion mitigation benefits to the beach and bluffs. The Medium Beach Fill measure would require 150,000 yd^3 of sand and provide up to 40 years of erosion mitigation benefits. This translates into approximately 5,000 yd^3 per year of erosion mitigation benefits for the Maximum Beach Fill measure, and 3,750 yd^3 per year for the Medium Beach Fill measure.

The comparison can be carried further by examining the ratio of estimated costs to the anticipated erosion mitigation benefits. The Maximum Beach Fill measure has an estimated cost of $6.4 million, which would translate into a cost of approximately $128,000 per year of erosion mitigation benefits. The Medium Beach Fill has an estimated cost of $5.0 million, which would translate to approximately $125,000 per year of erosion mitigation benefits. This comparison suggests that could be some (albeit minor) degree of “diminishing returns” of erosion mitigation associated with additional fill beyond 150,000 yd^3. Thus, the Medium Beach Fill measure will likely be the most effective and efficient from an engineering perspective.

However, it can be assumed that the equipment mobilization and demobilization costs associated with the Medium and Maximum Beach Fill measures will essentially be the same. Thus, the unit cost per yd^3 will be considerably higher (approximately $6) for the Medium Beach Fill measure. As a result, the Medium Beach Fill measure will be the most cost effective if the greatest amount of sand (150,000 yd^3) is removed from the shoal.

Table 41: Comparison of design measures with estimated costs and anticipated effectiveness

Design Measure	Description	Type of Analysis	Estimated Cost (2014 Dollars)	Effectiveness	Concerns
Baseline Condition (see Section 4.3)	Existing condition at project site (June 2009 to May 2010).	Numerical modeling of scenario with Coastal Modeling System (CMS).	Not Applicable.	None.	Beach and bluff erosion will continue to threaten infrastructure. Shoaling will impact navigation in Pillar Point Harbor.
1. Maximum Beach Fill	Remove 200,000 to 250,000 yd^3 from Pillar Point Harbor and construct 180 foot wide beach berm at Surfer's Beach.	GIS-based computations of dredge and fill quantities. Numerical modeling of measure with CMS.	$6,386,000*, assumes use of pipeline dredge and 20 percent contingency. Unit cost of per yd^3	High. Meets both design objectives. Might provide protective benefits to Surfer's Beach for up to 50 years.	Environmental concerns regarding removal of sand from vegetated sub-aerial beach in Pillar Point Harbor. Relatively high cost.
2.Medium Beach Fill	Remove 140,000 to 150,000 yd^3 from Pillar Point Harbor and construct 125 foot wide beach berm at Surfer's Beach.	GIS-based computations of dredge and fill quantities. Numerical modeling of measure with CMS.	$5,009,000*, assumes use of pipeline dredge and 20 percent contingency	High. Meets both design objectives. Might provide protective benefits to Surfer's Beach for up to 40 years.	Relatively high cost and uncertainty regarding performance of beach fill under persistent stormy conditions (e.g., El Niño event).
3. East Breakwater Modification: Seal Voids	Seal voids along a 2,500 ft long section to prevent sand from surging through the breakwater.	Numerical modeling of measure with CMS.	$400,000 to $600,000 to seal breakwater with concrete, based on a previous estimate (1978) to seal a 1,600 ft long section.	Medium. Meets the design objective of improving navigation in the harbor, but does not mitigate beach and bluff erosion.	Constructability. Past experience demonstrates that it will be very difficult to completely seal the breakwater.
4. East Breakwater Modification: Notch	Modify the East Breakwater by removing a 200 foot long section to create a notch that facilitates transport of sand.	Numerical modeling of measure with CMS.	$240,000 to $320,000 to remove 150 to 200 ft of breakwater, based on a previous estimate (2006) to remove 170 linear ft of breakwater.	Low to Medium. Modest decrease in rate of shoaling in harbor, but does not mitigate beach/bluff erosion.	Changes in hydrodynamics in harbor, which could interfere with navigation.
5. Alternative Dredged Material Placement	Continuous removal of sand from the shoal along the east breakwater with placement near the harbor entrance.	Utilized outputs from CMS simulation of baseline condition.	$2.3 million to $4.7 million, based on costs from Oakland MHEA and Moss Landing Harbor. Note uncertainty associated with continuously operating pipeline.	Low. Likely does not meet both design objectives.	Interference with navigation, high maintenance costs and project complexity.
6. Spur Breakwater	Construct 600 ft long spur breakwater extending from east breakwater.	Review of design developed by USACE (1971), which was based on physical modeling.	$2.5 million to $3.2 million, based on 2006 and 1971 estimates, respectively	Low to Medium. Limited benefits to a small section of beach. Uncertainty regarding impact (if any) on navigation in harbor.	Impacts to surfing resources and nearshore environment.
7. Managed Retreat	Remove infrastructure from areas vulnerable to erosion. Realign a 4,400 ft long section of Highway 1	GIS-based analysis of projected bluff retreat. Review of conceptual plans and infrastructure data from Ocean Beach.	Over $16 million to realign Highway 1, assuming a cost of $3,700 per linear ft of highway.	Low. Does not meet the design objective of improving navigation in harbor, and does not reduce rate of beach/bluff erosion.	High cost. Complex planning process involving multiple stakeholders.

* Costs estimated by USACE San Francisco District Cost Engineering Section based on guidance in ER 1110-2-1302

7 Conclusions and Recommendations

There are 2 key findings from this study, which informed development of the design objectives and formulation of design measures.

First, a GIS-based analysis of coastal bluff retreat supports the well established hypothesis that the construction of the east breakwater has induced additional erosion of the unprotected coastal bluff and beach between the Caltrans and Mirada Road revetments. The analysis suggests that bluff erosion rates in the immediate vicinity of the east breakwater are more than 1 foot per year greater than the background erosion rates in nearby geologically similar sections of coastal bluff. Therefore, it is reasonable to assume that presence of the east breakwater will continue to induce additional erosion in the absence of any erosion mitigation efforts.

Second, construction of the breakwaters has resulted in deposition of at least 150,000 yd^3 of sand along the harbor side of the east breakwater, which has limited the available maneuvering and anchoring area in the harbor. Previous studies, observations, and numerical modeling with the CMS suite strongly suggest that most of this sand originates outside of the harbor, and is deposited on the harbor side of the east breakwater via surging of waves through and over the structure. It is anticipated that sand will continue accumulate in this shoal in the absence of a measure to prevent sand from surging through and over the structure.

These 2 findings served as the basis for developing the two design objectives, which in turn informed the design measure formulation process. As a result, 7 design measures were evaluated to determine which would be the most effective at addressing the design objectives of mitigating beach and bluff erosion between the 2 revetments and improving navigation in the harbor. The evaluation process involved simulations with the CMS modeling suite for 4 of most promising the measures, and other methods (e.g., GIS-based analysis) for the other 3 measures.

Of the 7 measures, the Medium Beach Fill design (150,000 yd^3) will likely be the most effective from an engineering perspective as it directly addresses both of the design objectives. However, detailed cost and economic analyses are necessary to determine if this and other design measures are economically viable and should be carried forward in the Detailed Project Report.

8 References

Buttolph, A. M., C.W. Reek, N.C. Kraus, N. Ono, M. Larson, B. Camenen, H. Hanson, T. Wamsley, and A.K. Zundel (2006). Two-dimensional depth-averaged circulation model CMS-M2D: Version 3.0, Report 2 Sediment Transport and Morphology Change. *Coastal and Hydraulics Laboratory Technical Report ERDC/CHL-TR-06-7.* Vicksburg, MS: U.S. Army Corps of Engineers Engineer Research and Development Center.

California State University, Monterey Bay, Sea Floor Mapping Lab (2011). *Data Library*: http://seafloor.csumb.edu/SFMLwebDATA_SURVEYMAP.htm, accessed on 7 November 2011.

Demirbilek, Z., K.J. Connell, N.J. MacDonald, and A.K. Zundel (2008). Particle Tracking Model (PTM) in SMS10: IV. Link to Coastal Modeling System, *ERDC/CHL CHTEN-IV-71.* Vicksburg, MS: U.S. Army Corps of Engineers Engineer Research and Development Center.

DeTemple, B., Battalio, R., and D. Ruttenberg (1999). *A Feasibility Study of sediment Retention Basins for Deer Creek, El Granada, California,* 30 June 1999.

ESRI (2012). *ESRI ArcMap 10.1 (Build 3035) software*. Esri, 380 New York Street, Redlands, CA 92373-8100, USA.

Gahagan & Bryant Associates (2007). *Summary Report: Pillar Point Harbor Bathymetry Study & Outer Channel Design,* prepared for San Mateo County Harbor District, 400 Oyster Point Blvd., Suite 300, So. San Francisco, CA 94080.

Griggs, G.B., and R.E. Johnson (1983). Impact of 1983 Storms on the Coastline of Northern Monterey Bay, Santa Cruz County. *California Geology*, Volume 36, Number 8, pp 163-174.

Griggs, G. and L. Savoy (1985). *Living with the California Coast*. Duke University Press. Durham, NC.

Griggs, G., Webber, J., Lajoie, K. R., and S. Mathieson (2005). *San Francisco to Año Nuevo,"* in Griggs, G., Patsch, K., and Savoy, L. (eds.). *"Living With the Changing California Coast,"* University of California Press, Berkeley, 540 pp.

Hampton, M.A. (2002). Gravitational Failure of Sea Cliffs in Weakly Lithified Sediment, *Engineering & Environmental Geoscience*, Vol. VIII, No. 3, August 2002, pp.175-191.

Hapke, C. (2004). The Measurement and Interpretation of Coastal Cliff and Bluff Retreat, In Hampton, M. and Griggs, G. (eds.), Formation, Evolution, and Stability of Coastal Cliffs – Status and Trends, *U.S. Geological Survey Professional Paper 1693*. pp. 39-50.

Hapke, C.J., and D. Reid (2007). National Assessment of Shoreline Change, Part 4: Historical Coastal Cliff Retreat along the California Coast: *U.S. Geological Survey Open-file Report 2007-1133.*

Hapke, C.J., Reid, D. and B. Richmond (2009). Rates and Trends of Coastal Change in California and the Regional Behavior of the Beach and Cliff System. *Journal of Coastal Research,* Vol. 25, No. 3, p. 603 - 615.

Hoover, D. (2011). Pillar Point Instrument Deployment Data, 18 July 2011.

Krumbein, W. C. (1947). Shore Processes and Beach Characteristics. *Beach Erosion Board, Tech. Memo*. No. 3, 34 pp.

Lajoie, K. R. and Mathieson, S. A. (1985). *"San Francisco to Año Nuevo,"* **in** Griggs, G. and Savoy, L. (eds.). *"Living with the California Coast,"* Duke University Press, Durham, NC, 394 pp.

Lin, L., Z. Demirbilek, F. Wu, J.T. Jackson, and A.T. Shak (2007). Coastal Numerical Modeling of Peninsula Beach, California. *Proceedings of 10th Estuarine and Coastal Modeling Conference*. Newport, RI (2007).

Lin, L., and Z. Demirbilek. (2008). CMS: A Coastal Modeling System for inlets and navigation projects. *Proceedings of the 5th International Ocean-Atmosphere Conference (2008).*

Lin, L., Z. Demirbilek, H. Mase, J. Zheng, and F. Yamada (2008). CMS-Wave: A nearshore spectral wave processes model for coastal inlets and navigation projects. *Coastal and Hydraulics Laboratory Technical Report ERDC/CHL-TR-08-13.* Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Lin, L. (2013-2014). *Various personal communications with J. Zoulas via phone and e-mail*, August 2013 through March 2014.

Local Government Commission, Glatting Jackson/Walkable Communities, Opticos Design, Inc., Kittelson and Associates, Inc. (2010). *Highway 1 Safety and Mobility Improvement Study, San Mateo County Midcoast Communities: Princeton, El Granada, and Miramar*.

Moffatt & Nichol (2007). *Ocean Beach, California: Guidance for a Beach Nourishment Study under Section 933.* Prepared for City & County of San Francisco, July 2007, M&N Job Number 5087-04.

National Oceanic and Atmospheric Administration (2008). *Tides & Currents Monterey, CA 9413450.* http://tidesandcurrents.noaa.gov/geo.shtml?location=monterey+ca, accessed on 7 November 2011.

National Oceanic and Atmospheric Administration (2011). May 2011 Regional Climate Summary. http://www.wrh.noaa.gov/mtr/climate/monthly_summary/2011/0511summary.pdf

National Buoy Data Center (2011a). *Station 46042 – Monterey – 27 NM West of Monterey Bay, CA*: http://www.ndbc.noaa.gov/station_page.php?station=46042, accessed on 7 November 2011.

National Buoy Data Center (2011b). *Station 46012 – Half Moon Bay – 24 NM South Southwest of San Francisco, CA:* http://www.ndbc.noaa.gov/station_page.php?station=46012, accessed on 7 November 2011.

National Oceanic Atmospheric Administration, National Geophysical Data Center (2011c). http://www.ngdc.noaa.gov/mgg/geodas/geodas.html, accessed on 7 November 2011.

National Research Council (2012). *Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future.* National Academic Press, Washington, D.C.

Null, J. (2014). El Niño and La Niña Years and Intensities – Based on Oceanic Niño Index (ONI): http://ggweather.com/enso/oni.htm, accessed on 3 October 2014.

Santoso, A.; McGregor, S.; Jin, F-F.; Cai, W.; England, M.H., An, S-I; McFaden, M.J.; and E. Guilyardi (2013). Late-twentieth-century emergence of the El Niño propagation asymmetry and future projections. *Nature*: www.nature.com/doifinder/10.1038/nature12683

Seymour, R. J., 1998, Effects of El Niños on the west coast wave climate. *Shore and Beach*, Vol. 66, No. 3, pp. 3 - 6.

So, York (2014). Personal communication, 21 October 2014.

Storlazzi, C. D., C. M. Willis and G. B. Griggs (2000). Comparative Impacts of the 1982-83 and 1997-98 El Niño Winters on the Central California Coast, *Journal of Coastal Research* Vol. 16, No. 4., pp. 1022-1036.

Thieler, E.R., Himmelstoss, E.A., Zichichi, J.L., and Ayhan Ergul (2009). Digital Shoreline Analysis System (DSAS) version 4.0— An ArcGIS extension for calculating shoreline change:

U.S. Geological Survey Open-File Report 2008-1278. http://woodshole.er.usgs.gov/project-pages/dsas/version4/index.html (Note: USGS DSAS version 4.3.4730 was used in this analysis)

Towill Inc. (2011). *Coastal Structures Program – Pillar Point Breakwaters: Pillar Point Harbor Survey.* Project report prepared for US Army Corps of Engineers, San Francisco District, 15 April 2011.

United States Army Corps of Engineers (1947). *Survey Report on Pillar Point, Halfmoon Bay, San Mateo County,* 1 August 1947.

United States Army Corps of Engineers (1963). Marine Advisers, *A Sea-Swell Recording Study at Half Moon Bay, California,* November 1963.

United States Army Corps of Engineers (1971). *Beach Erosion Control Report on the Shores of El Granada Beach, San Mateo County, California,* District Engineer, San Francisco District, 35 pp plus appendices.

United States Army Corps of Engineers (1976). *Shoreline Protection and Earthwork Grading for Pillar Point Marina El Granada California*, 25 October 1976.

United States Army Corps of Engineers (1978). *Half Moon Bay Breakwater Repair – Final Inspection of East Breakwater Repairs*, 19 December 1978.

United States Army Corps of Engineers (1981). *Plan Formulation Document Section 107 Small Harbor Project Pillar Point Marina San Mateo County, California*. August, 1981.

United States Army Corps of Engineers (1983?). *Half Moon Bay Lessons Learned*.

United States Army Corps of Engineers (1986). *Comprehensive Condition Survey for Half Moon Bay Breakwaters, California.* November, 1986.

United States Army Corps of Engineers (1996). *Pillar Point Harbor San Mateo County, California Reconnaissance Report,* March 1996.

United States Army Corps of Engineers (2000). *Engineer Regulation 1105-2-100, Planning Guidance Notebook*, April 2000.

United States Army Corps of Engineers (2002). *Engineer Manual 1110-2-1100, Coastal Engineering Manual – Part V, Chapter 4: Beach Fill Design.* Change 2, 1 August 2008.

United States Army Corps of Engineers (2006). *Pillar Point Harbor East Breakwater Repair, Design Documentation Report,* March 2006.

United States Army Corps of Engineers (2008). *Engineer Regulation 1110-2-1302, Civil Works Cost Engineering,* 15 September 2008.

United States Army Corps of Engineers (2009a). *North Half Moon Bay Shoreline Improvement Project, Pillar Point Harbor, CA. Section 216: Review of Completed Projects, Initial Appraisal,* July 2009.

United States Army Corps of Engineers (2009b). *Engineer Circular 1165-2-211, Water Resource Policies and Authorities Incorporating Sea-level Change Consideration in Civil Works Programs,* July 2009

United States Army Corps of Engineers (2012). *Engineer Manual 1110-2-1304, Civil Works Construction Cost Index System*, 31 March 2012. Includes *Amendment #3: Tables Revised as of 30 September 2013*.

United States Army Corps of Engineers (2013-2014). *San Francisco District eCoastal GIS Database: Enterprise GIS for Coastal Engineering and Sciences.* GIS files accessed on 6 January 2014.

United States Army Corps of Engineers (2013). *Engineer Regulation 1100-2-8162, Incorporating Sea-Level Change in Civil Works Programs,* 31 December 2013.

United States Army Corps of Engineers (2014a). *Engineer Technical Letter 1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation,* 30 June 2014.

United States Army Corps of Engineers (2014b). "Sea-Level Change Curve Calculator" from *Climate Change Adaptation, Comprehensive Evaluation of Projects with Respect to Sea-Level Change*, URL: http://corpsclimate.us/ccaceslcurves.cfm.

United States Army Corps of Engineer (2014c). *Coastal Modeling System (CMS) Website*, URL: http://cirpwiki.info/wiki/CMS

United States Geological Survey (2011). *National Seafloor Mapping and Benthic Habitat Studies: Pacific,* http://walrus.wr.usgs.gov/research/projects/benthic_hab.html, accessed November 7, 2011.

United States Geological Survey (2013). *High Resolution Orthoimagery (HRO) Production Information*, https://lta.cr.usgs.gov/High_Resolution_Orthoimagery.

Winzler and Kelly Consulting Engineers (1984). *Sediment Transport Study West Shoreline Park Pillar Point Harbor Half Moon Bay, California,* October 1984.

Wuertz, S.; Wang, D.; Zamani, K. and F. Bombardelli (2011). *An Analysis of Water Circulation in Pillar Point Harbor, Half Moon Bay, California, based on the Dye Distribution Study of September 27, 2008.* Draft Report prepared by Dr. Wuertz (U.C. Davis) under contract from the San Mateo County Resource Conservation District (SMCRCD), May 2011.

Zundel, A. K. (2006). Surface-water Modeling System Reference Manual – Version 9.2. Provo, UT: Brigham Young University Environmental Modeling Research Laboratory.