CARMEL RIVER LAGOON SCENIC ROAD PROTECTION

PRELIMINARY 30% DESIGN

DRAFT REPORT

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1. INTRODUCTION

1.1 Background

The Carmel River drains an approximate 250 square mile watershed. The river enters the Pacific via the Carmel River Lagoon (Figure 1-1), which serves as rearing habitat for juvenile steelhead and supports a number of threatened species.

The lagoon forms during times of low or no river flow, when waves build a barrier beach across the mouth of the river. Over time the extent of the lagoon grows due to continued inflow from the Carmel River, up to a point where the lagoon breaches into the Pacific. In the past the lagoon has breached naturally or been breached mechanically to the west directly to the Pacific, over the beach south of the lagoon, and via the beach north of the lagoon. When the river channel migrates along the northern or southern beach, a longer, meandering path is taken which reduces the flow during breaching. From a juvenile fish rearing perspective, it’s preferable that the lagoon remains in place for as long as possible, and is allowed to meander north and south, which reduces the hydraulic gradient during breaching and prevents fish from being flushed out to sea too early before they are ready to follow their natural pattern of migration.

The rate of the lagoon drawdown during a breach and the post-breach lagoon water surface elevation are dependent on several factors, including breach location, channel length and width, tidal conditions, and the presence or absence of a rock sill along the outflow channel.

During the period when the mouth of the river is closed, lagoon water levels rise between late fall and spring seasons and threaten private properties along the northern edge of the lagoon, the County wastewater treatment plant, as well as a parking lot and restroom facility operated by the State Parks.

Figure 1-1 Aerial view of Carmel River Lagoon and Beach.

1.2 PURPOSE

The present project focuses on protection of the Scenic Road, which is located immediately north of the Carmel River mouth. It is a publicly maintained road at the top of the bluff, which is periodically threatened by erosion when the river takes a northerly route along the beach before discharging into the Pacific Ocean. The road runs around the northern headlands of Stewart’s Cove, southwest to intersect with Carmelo Street near the Carmel River State Beach parking lot and restrooms. Scenic Road provides recreational access to the State Beach, sole access to six private homes, and has a sanitary sewer pipe under the roadway.

Figure 1-2 shows an example condition where the lagoon has migrated north along the toe of the bluff carrying the Scenic Road. The public parking lot is visible at the right edge of the picture at the shore of the lagoon.

Figure 1-2 Example of lagoon migration north along the beach at the toe of the bluff carrying Scenic Road.

1.3 SCOPE OF WORK

Moffatt and Nichol has been retained by Denise Duffy & Associates, Inc. (DDA) to provide a 30% design of a revetment that provides erosion protection of the bluff along Scenic Drive while enabling the lagoon to form and breach without interference.

This 30% Design Report provides an abbreviated basis of design and the methodology adopted for the design of the revetment.

2. DESIGN BASIS

Key parameters adopted as a basis for the revetment design are summarized in the following.

2.1 DESIGN BASIS SUMMARY

Table 2-1 provides a summary of parameters adopted for the revetment design of the erosion protection for the Scenic Road.

Parameter	Design	Reference	Comment
Coordinate System			
Elevations, dimensions	General	Imperial Units	Some SI units utilized where required by some methods of application.
Planar coordinate system	CA	NAD83	California State Plane System (NAD83), U.S. Survey Feet, Zone 404.
Vertical Datum	US	NAVD88	North American Vertical Datum of 1988 (NAVD88), U.S. Survey Feet.
Topographic data		[1]	2012 aerial survey.
Benchmark	Reference	GU3232	NAVD 1988 elevation: 85.62' (Vertical First Order Class 1).
Bathymetric data		[2]	NOAA Chart 18686, RMC 2006-2007 survey data.
Recurrence Interval of Design Conditions			
Project design life	30 years		
Recurrence interval	100 years		Corresponding to 40% risk of exceedance.
Tides and Extreme Water Levels			
Coastal BFE	N/A	FEMA	FEMA Base Flood Elevation not determined.
MHHW	+5.48 ft	[3]	Mean Higher High Water
MHW	+4.78 ft		Mean High Water
MTL	+3.01 ft		Mean Tide Level
MLW	+1.24 ft		Mean Low Water
MLLW	+0.14 ft		Mean Lower Low Water
NAVD88	+0.00 ft		North American Vertical Datum of 1988
Wave Action			
Significant wave height	32.2 ft	[6]	WIS 100-year offshore wave condition.
Peak wave period	19.5 s		
Wave setup	3.5 ft	Current study	Wave setup due to wave breaking.
Design wave height	21.6 ft	Current study	Nearshore significant wave height for revetment design.
Carmel River and Lagoon			
River BFE	+16.0 ft		FEMA Base Flood Elevation.
Lagoon water level	+15.4 ft		100-year water surface elevation.
Peak discharge	18,000 cfs	[7]	USGS Gauging Station 11143250
Design flow velocity	13.4 fps	Current study	Flow velocity parallel to revetment produced by lagoon breach or river discharge.
Carmel River thalweg	+6.0 ft		Elevation relative to NAVD88.
Sea Level Rise and Water Level Variations			
SLR	2.0 ft	[5]	Mid-century upper-bound sea level rise estimate.
Storm Surge	N/E		Included in total water level analysis of Monterey Bay water level data.

Parameter	Design	Reference	Comment
El Niño	N/E		Included in total water level analysis of Monterey Bay water level data.
1% TWL	+8.29 ft	NAVD88	100-year total water level determined from Monterey Bay, CA, water level data.
Beach Morphology			
Slope	1V:8H to 1V:3.6H	[3]	Beach slope varies from 7.1° at the north end to 15.5° at the south end.
Berm elevation	+14 to +17 ft	[3]	Low point is around +14 feet, typically south of the river mouth. High point is generally in the middle of the beach.
Shoreline variability	± 30 ft	[3]	Seasonal variability in shoreline position.
Short term shoreline change rate	-3.3 ft/yr	[8]	Northern part of beach subject to short term and long term accretion, estimated 1.6 ft/yr.
Long term shoreline change rate	-2.6 ft/yr		
Geotechnical Data			
Sediment grain size	0.01" to 0.03"	[4]	Grain size estimated based on mean beach slope.

Table 2-1 Summary of design parameters.

2.2 STORM PROFILE

Since bathymetric and topographic data immediately following a storm event was not available, a typical beach storm profile was developed to predict the redistribution of sediment offshore of the beach. This profile was estimated using the dune erosion prediction model developed by Vellinga (1986), ref. [9]. This methodology calculates dune erosion based on the sediment size and the design storm wave height. A comparison of the estimated profile vs. the available survey data is shown in Fig. 2-2.

2.3 DESIGN WATER LEVEL

Water level data was extracted from the NOAA Tide Station #9413450 at Monterey, CA. Analysis conducted in the Coastal Engineering Analysis, ref. [3] showed that very little spatial variation in water levels existed between the Station #9413450 and the project site. An extreme water level analysis was performed on the 41 years of hourly water level measurements at Monterey to determine the anticipated return period water levels, shown in Table 2-2. Due to the extended period of record of this tidal station, it is assumed that all storm surges and El Niño events have been captured in the return period water levels.

2.4 SEA LEVEL RISE

The Sea Level Rise (SLR) projection utilized in this analysis was taken as 2 feet by mid-century. This value corresponds to the maximum of the 2050 range for central to southern California in the recently published study by the National Academy of Sciences (2013), ref. [13].

Extreme event water levels for both current conditions and future SLR conditions can be found in Table 2-2.

NOAA # 9413450 Tide Station		
Return Period	Water Elevation (ft. NAVD88)	Water Elevation (ft. NAVD88) with SLR = 2'
1	7.25	9.25
2	7.41	9.41
5	7.61	9.61
10	7.77	9.77
25	7.97	9.97
50	8.13	10.13
100	8.29	10.29

Table 2-2 Extreme Water Levels

2.5 DESIGN WAVE HEIGHT

Since the revetment will be designed based on a 30 year life, an extreme event wave height is necessary to ensure stability throughout the life of the project. Long-term wave measurements required for this analysis are not present in the vicinity of the project area. For the Coastal Engineering Analysis, 5 years of wave measurements taken between 2008 and present by the National Data Buoy Center (NDBC) Station #46239 at Point Sur located 30 miles south of the project area was implemented into the MIKE 21 Spectral Wave Model to determine wave conditions in the project vicinity. This analysis was used to develop swell transformation coefficients based on swell direction and swell period. Further detail is given in the Coastal Engineering Analysis Report, ref. [3].

The long-term wave statistics were taken from hindcast wave data developed by the US Army Corps of Engineers Wave Information Studies (WIS). WIS Station 83074 is located approximately 15 miles from the project site and contains 31 years of hindcast wave data between 1980 and 2011. The Swell Transformation Coefficients were applied to this data per directional bin and peak period bin to account for diffraction, refraction, and shoaling of waves as they move from deep water to shallow water at the project site. An extreme wave height analysis was then performed on this 31 year time series of transformed swell waves, shown in Table 2-3:

Transformed Extreme Wave Heights (Significant Wave Heights in feet)				
Return Period (Years)	Carmel Point 7: Depth= -25.7 ft. NAVD88		Carmel Point 5: Depth= -14.2 ft. NAVD88	
	Hs	Direction	Hs	Direction
1	14.4'	W	15.9'	W
2	14.9'	W	17.5'	W
5	18.7'	W	19.3'	W
10	20.1'	W	20.6'	W
25	21.8'	W	22.6'	WSW
50	23.2'	WSW	24.3'	WSW
100	24.7	WSW	25.8'	WSW

Table 2-3 Extreme Wave Heights Relative to Offshore Angle of Incidence

The location of wave points, Carmel 7 and Carmel 5, are shown in Figure 2-2.

2.6 DETERMINATION OF A DESIGN EXTREME EVENT

To enable stability of the revetment for the 30 year project life, a 1% annual chance of occurrence event was chosen as the design criteria. The total water level (TWL), defined as the sum of the still water level and the 2% run-up (run-up elevation reached by the largest 2% of incoming waves), was calculated based on this design criteria. Additionally, this criterion was used to establish the wave height at the toe of the revetment and the necessary size and weight of rock for the revetment.

The 1% annual chance of occurrence event is a combination of both wave height and water level. For simplicity, it was assumed that these two parameters are independent of each other so that their probabilities could be directly multiplied to determine the overall chance of occurrence. Based on this assumption, the following combinations of water levels and wave heights result in an event with a 1% annual chance of occurrence.

Extreme Event Combination							
Wave Height RP*	Carmel Point 7		Carmel Point 5		Water Level RP*	Water Level (NAVD88)	SLR Water Level (NAVD88)
	Direction	Wave Height	Direction	Wave Height			
1	W	14.4’	W	15.9’	100	8.3’	10.3’
2	W	14.9	W	17.5’	50	8.1’	10.1’
10	W	20.1’	W	20.6’	10	7.8’	9.8’
50	WSW	23.2’	WSW	24.3’	2	7.4’	9.4’
100	WSW	24.7’	WSW	25.8’	1	7.3’	9.3’

RP= Return Period (years)

Table 2-4 Combination of Extreme Water Level and Wave Events

2.7 RUN-UP ANALYSIS

The guidelines outlined by the *Technical Advisory Committee on Flood Defense* (TAW) for determining wave run-up on a structure require the wave height experienced at the toe of the revetment. This elevation was chosen as +2 ft. NAVD88 based on the assumed minimum beach erosion profile. The extreme waves presented in Table 2-3 were further shoaled using the methodology developed by Goda (2000) to estimate wave heights within the surf zone, ref. [10].

Applying the water levels for each extreme event, the wave height at the toe of the structure was determined. In each case, the wave was shown to break prior to reaching the toe of the revetment. Since this is a depth-limited condition, the water depth affects the design wave height. Based on the 1% annual chance water level, the maximum significant wave height at the toe of the structure was found to be 4.9 ft. However, the mid-century condition with SLR= 2’, the additional water depth will result in a higher wave at the toe of the structure, 6.4 ft.

The maximum wave run-up on the structure was found to occur for the combination of the 1% annual chance water level and a wave height of 6.4 ft. at the toe of the structure. Depending on the profile location and revetment configuration, wave run-up ranged between 12 and 18 ft. above the SWL.

2.8 ROCK SIZE

The necessary size of rock was determined using the Van der Meer equations for Armor Stone Sizing, ref. [11]. This methodology is dependent on the wave height, storm duration (number of waves), and the allowable rock displacement. A design wave height of 6.4 ft. was used to determine the required rock properties based on the Caltrans Rock Slope Protection (RSP) guidelines.

Rock Revetment			RSP-Class**			
Armor Layer Rock Weight	Diameter	Min. No. of Armor Layers	Outside Layer RSP-Class	Inner Layer RSP-Class	Backing Class	RSP Fabric Type
1,600 lbs.	2.1 ft.	2	1 Ton	Light	None	B

***Based on California Layered Rock Slope Protection and Section 72-2.02 Material of the Caltrans Standard Specifications, ref. [12].*

Table 2-5 Revetment Properties Required to Withstand Design Conditions with 2' SLR

3. REFERENCES

[1] *Aerial Survey*. Aerial topographic survey flown on August 29, 2012.

[2] *NOAA Chart 18686 Pfeiffer Point to Cypress Point.* Supplemented with 1970 bathymetric survey data providing depth contours between 20 to 100 feet depth; and survey data acquired by RMC in 2006 and 2007.

[3] *Coastal Engineering Analysis*. Carmel River Lagoon Biological Assessment. Prepared by Moffatt & Nichol under contract to Denise Duffy & Associates, Inc. for Monterey County Water Resources Agency. August 2, 2013.

[4] *Coastal Stabilization*. Advanced Series on Ocean Engineering, Volume 14. Silvester and Hsu (1993).

[5] *State of California Sea-Level Rise Guidance Document.* Developed by the Coastal and Ocean Working Group of the California Climate Action Team (CO-CAT), with science support provided by the Ocean Protection Council's Science Advisory Team and the California Ocean Science Trust. March 2013 update.

[6] *Wave Information Studies.* WIS, US Army Engineer Research & Development Center. Pacific Station 83074: Lat: 36.500º, Lon: -122.170º. Depth: 1353 m. Storm Event Return Period of 32-yr (1980-2011) Wave Hindcast. Linear fit to top 21 events: $H_{mo} = 6.8721 + 0.63996 \cdot \ln[\text{ Return Period (years) }]$. Event Peak H_{mo} in meters.

[7] *USGS Station 11143250 – Carmel River Near Carmel, CA*. National Water Information System Web Interface: waterdata.usgs.gov/nwis/uv?site_no=11143250. U.S. Geological Survey, U.S. Department of the Interior.

[8] *National Assessment of Shoreline Change Part 3: Historical Shoreline Change and Associated Coastal Land Loss Along Sandy Shorelines of the California Coast.* U.S. Geological Survey Open File Report 2006-1219. U.S. Geological Survey, Coastal Field Station, Department of Geosciences, University of Rhode Island, Kingston, RI 02881. U.S. Geological Survey, Pacific Service Center, Santa Cruz, CA 95060. Dept. of Geosciences, Oregon State University, Corvallis, OR 97331. U.S. Geological Survey, Woods Hole Science Center, Woods Hole, MA 02543.

[9] Vellinga, Pier, 1986. *Beach and Dune Erosion during Storm Surges.* Delft Hydraulics Laboratory, Department of Public Works.

[10] Goda, Y. 2000. *Random Seas and Design of Maritime Structures*. Advanced Series on Ocean Engineering- Volume 15.

[11] Van der Meer, Jentsje W. (1995) *Conceptual Design of Rubble Mound Breakwaters*.

[12] California Department of Transportation Engineering Service Center, Office of Structural Foundations, Transportation Laboratory (2000). *California Bank and Shore Rock Slope Protection Design.* Final Report No. FHWA-CA-TL-95-10.

[13] *Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future* (2013). Committee on Sea Level Rise in California, Oregon, and Washington; Board on Earth Sciences and Resources; Ocean Studies Board; Division on Earth and Life Studies; National Research Council.

[14] Technical Advisory Committee on Flood Defense (2002). *Wave Run-up and Wave Overtopping at Dikes.* Road & Hydraulic Engineering Institute, The Netherlands.

Figures

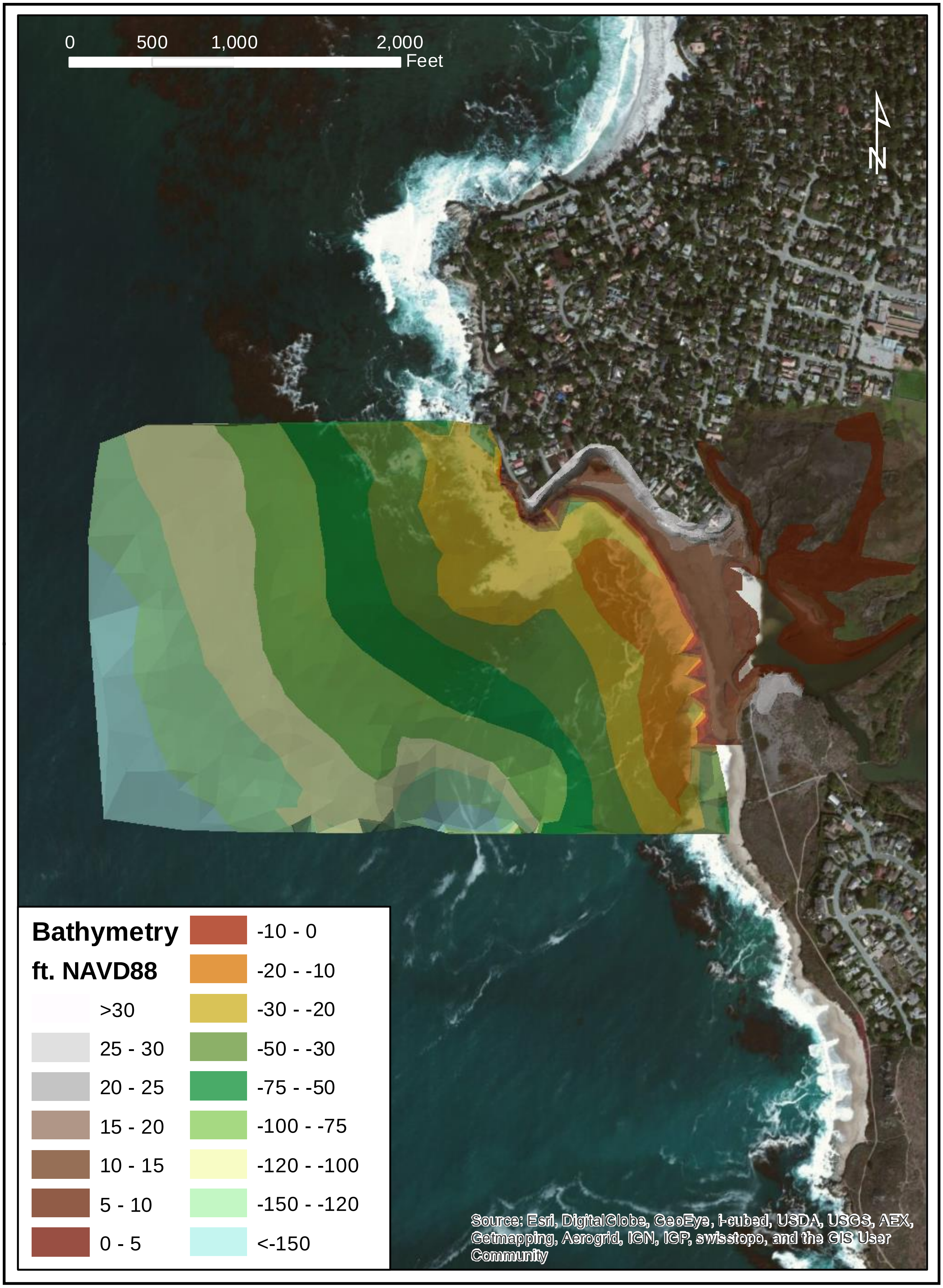
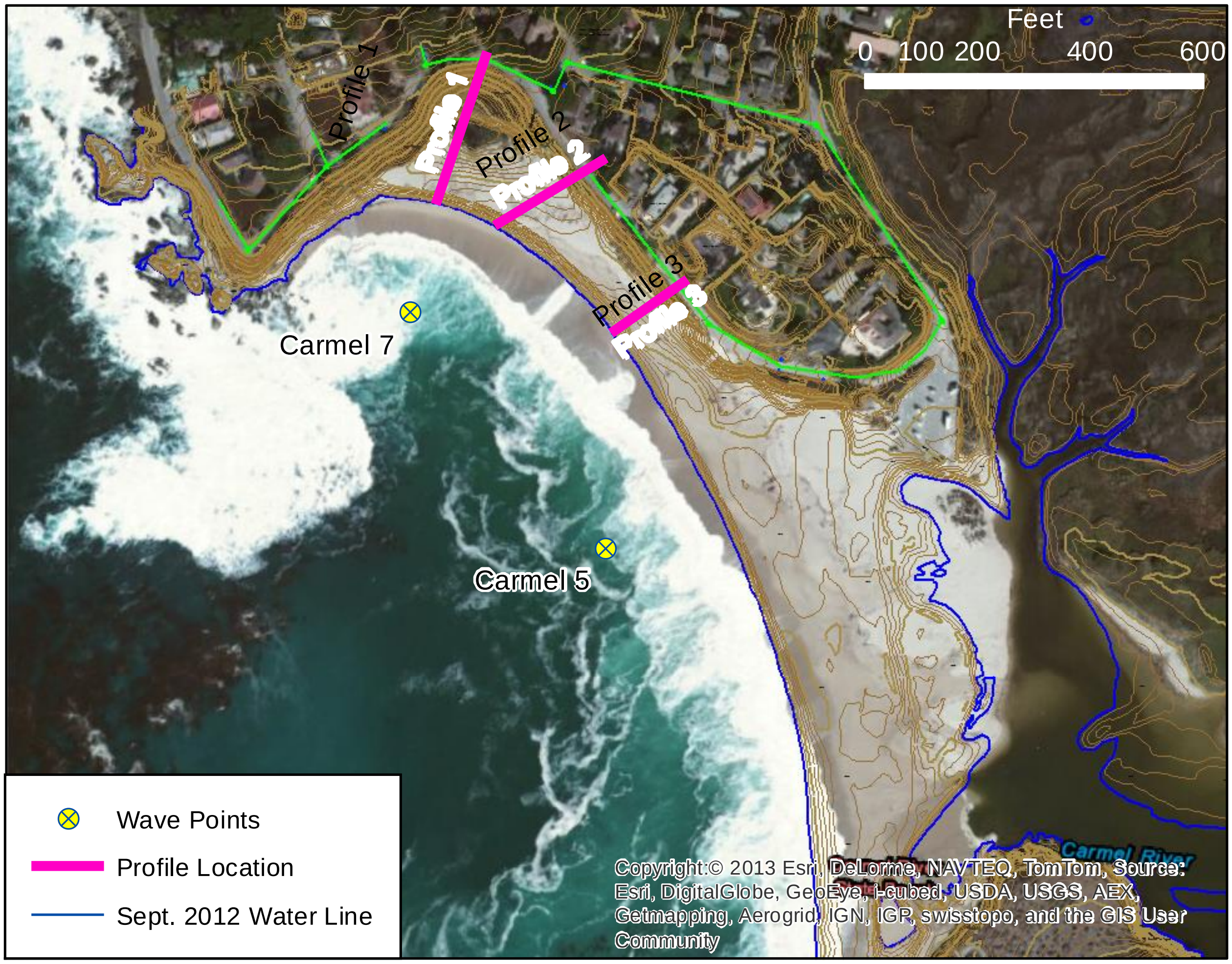


Figure 2-1 Bathymetry and Topography in Project Vicinity



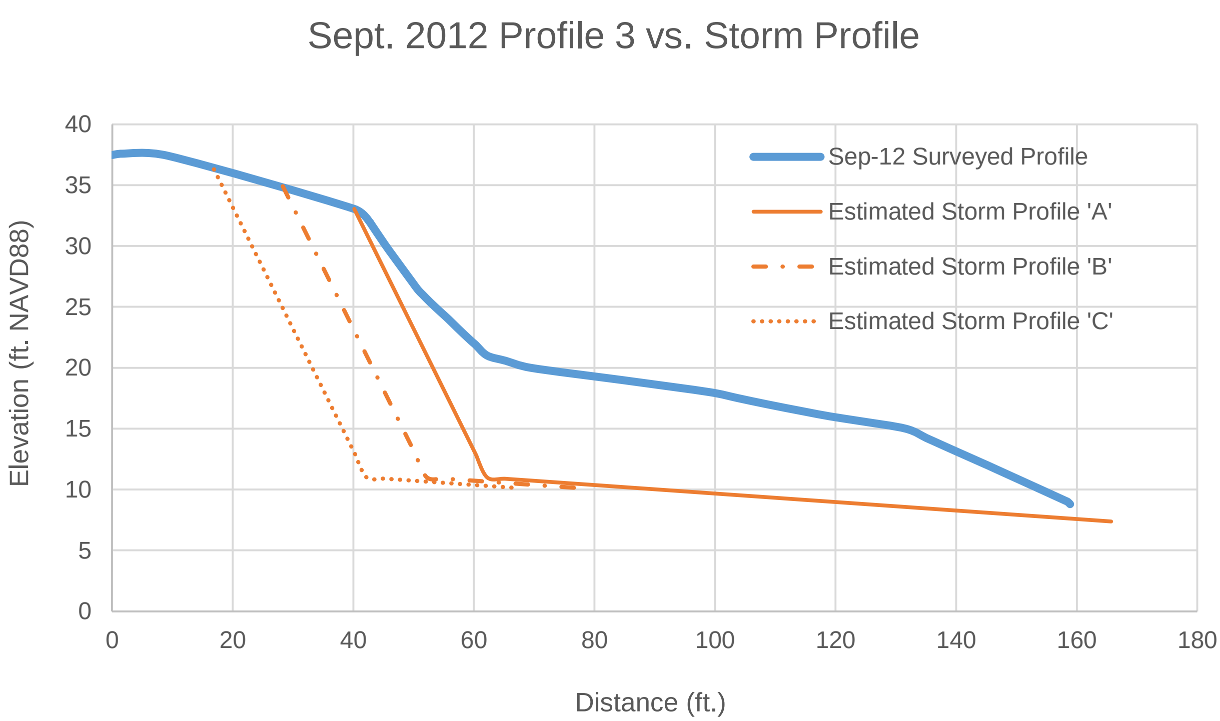


Figure 2-2 Foreshore Profile

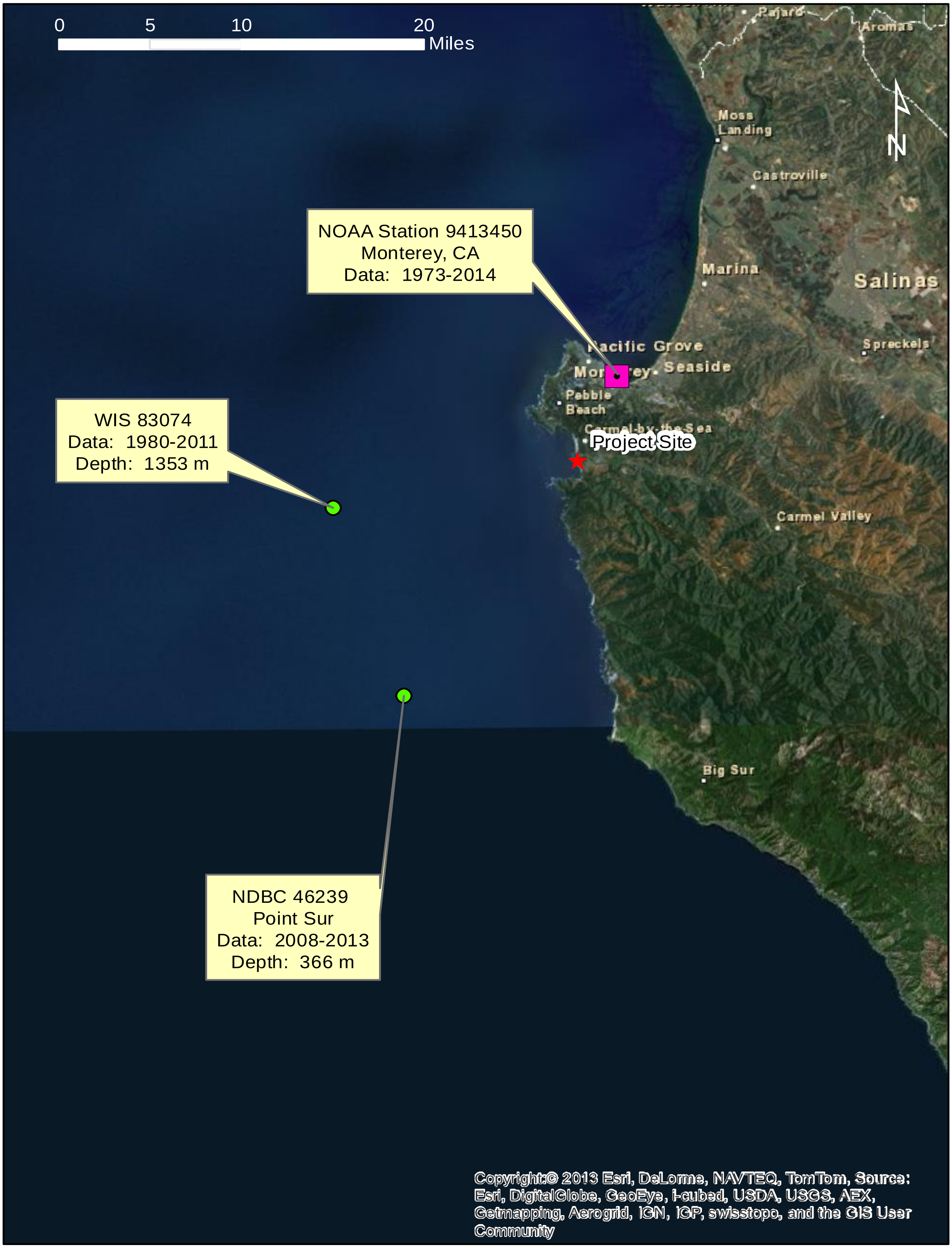
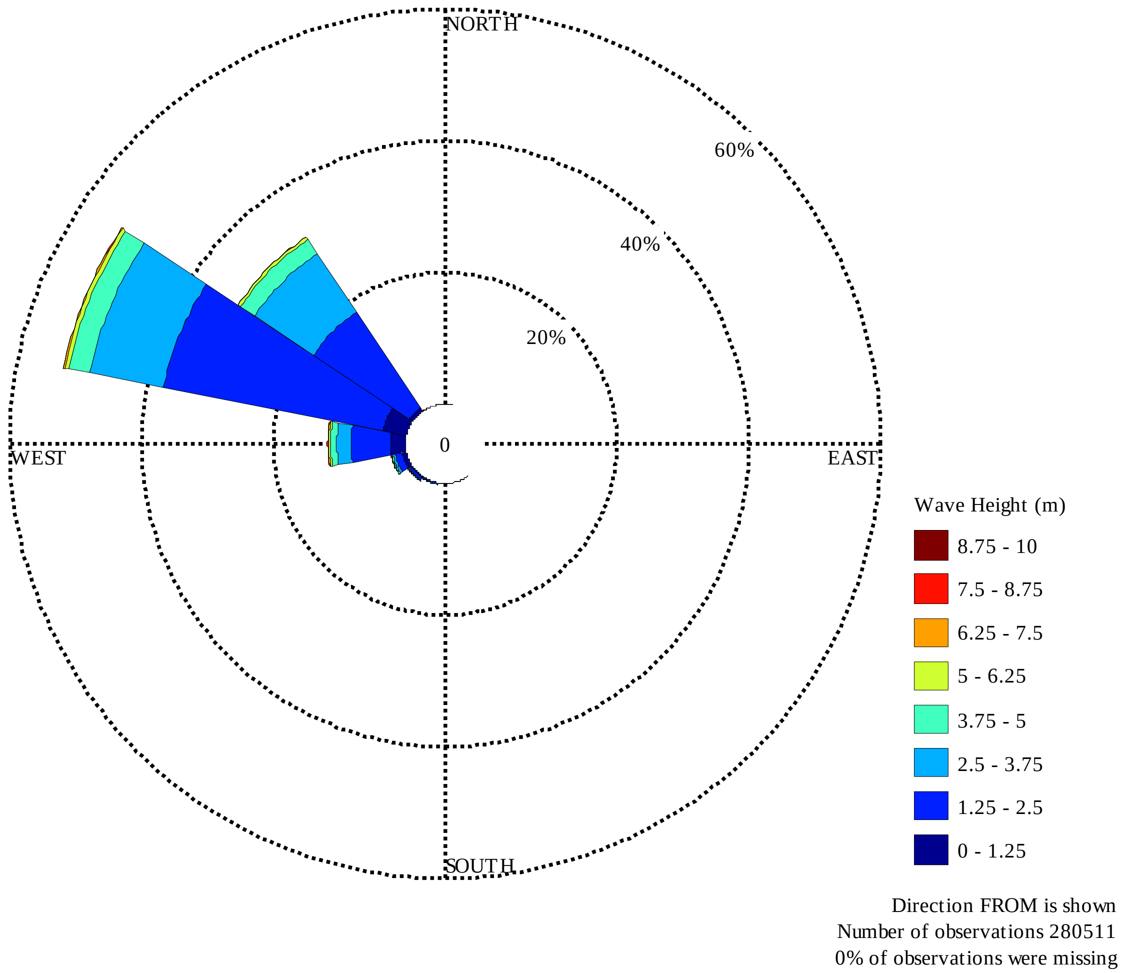


Figure 2-3 Tide and Wave Gauges

Joint Frequency Distribution (Annual)
WIS 83074 Total Wave
Period of Observations: 1980-2011



Percentage of Occurrence

Wave Height (m)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total									0.04	0.26	0.76	2.38	11.8	52.9	31.8	0.03	100
10 - 11.25																	
8.75 - 10																	
7.5 - 8.75												0.01	0.02	0.01			0.05
6.25 - 7.5												0.04	0.11	0.08	0.02		0.25
5 - 6.25											0.02	0.06	0.31	0.63	0.43		1.45
3.75 - 5										0.02	0.05	0.18	0.93	3.33	2.58		7.08
2.5 - 3.75									0.01	0.08	0.15	0.37	2.17	11.3	10.6		24.7
1.25 - 2.5									0.02	0.13	0.36	1.06	5.93	33.9	17.6	0.02	59
0 - 1.25										0.03	0.18	0.67	2.35	3.63	0.54		7.42

Figure 2-4 WIS 83074 Wave Rose