REPORT

RICH PASSAGE WAVE ENERGY EVALUATION

Beach Response to In-situ Testing of Rich Passage 1

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EXECUTIVE SUMMARY

Wake wash from high-speed ferries can potentially cause adverse effect to shorelines in confined waterways and environmentally sensitive areas. Repeated attempts to establish regular passenger only fast ferry (POFF) service on the Seattle to Bremerton route in Puget Sound have been constrained by the effect of POFF wake wash on the shorelines of private property along the narrowest portion of the route. This report presents results from the Rich Passage Shore Response Study, a multi-disciplinary study designed to evaluate the environmental feasibility of re-introducing high speed passenger-only ferry service on the Seattle to Bremerton route.

The performance of *Rich Passage* 1 (RP1), a new low-wake design foil-assisted catamaran, has been tested to determine the wake wash it creates, and the effects of that wake wash on nearby beaches and structures. The results were compared against wake wash and impact criteria that were developed from a synthesis of extensive data, including physical and biological studies in Rich Passage, results from full-scale field trials of the vessel, and a system of integrated numerical models that simulate the coastal processes and response of the system at a range of space and time scales. This report summarizes the beach response to RP1 operations from *25* June to 2 November 2012, and compares the beach response to baseline conditions recorded between 2004 and 2012 when no POFF service was operating. Baseline studies measured beach morphology, sediment composition, and how these conditions changed between seasons and between years.

Beach Response at Specific Sites

Observations, measurements, and modeling indicate that the physical processes responsible for shoreline change in Rich Passage are highly site-specific. Shoreline change is a function of local topography and bathymetry, exposure of the site to vessel sailing line, size, speed, and operating frequency of vessels, speed and direction of tidal currents, exposure to wind-waves, sediment characteristics, sediment supply and beach slope. The beaches within Rich Passage change significantly each season and each year depending on regional variations in climate as well as the variations in the local site-specific factors. In general, the magnitude of beach response observed during RP1 operations is within the scale of observed long-term seasonal and inter-annual beach response. The intensive beach observations during the RP1 testing program reveal a temporal variability of the beach response that was not previously revealed during the less intensive baseline monitoring. At some sites within the study area, beach slope was slightly reduced and sediments were redistributed during the RP1 testing interval. The pattern is characteristic of the response anticipated for a POFF operation based on historical observations. However, there are also significant temporal variations in beach response during the testing interval that can only be attributed to natural forces (e.g., wind-waves and tidal current).

High-resolution, three-dimensional surveys of selected beaches were obtained using a laser scanning system before, during, and after the RP1 test interval. These surveys enable detailed mapping of the foreshore at an unprecedented level of detail. Differences between laser scanning surface maps over time indicate that Point Glover beaches are dominated by alongshore transport. The transport is primarily confined between groins and headland promontories in discrete cells or pockets. Sediment transport direction at Point Glover responds primarily to wind direction, which is from the northeast in the summer and from the southwest in winter. Although there was no direct correlation between beach response and vessel wake wash from RP1 along Point Glover, the wake wash measurements indicate RP1 generates more wake power at the shoreline while travelling from Seattle to Bremerton than vice versa. The latter could increase the net transport from northeast to southwest along this shoreline and should be monitored closely during any future POFF operations.

Beach response along Pleasant Beach and Port Orchard appears to be more closely associated with an increase in wind speeds and a shift in wind direction from northeast to south-southwest than the start and end of RP1 operations. At most sites along Point White, beach elevations have decreased overall, with a steady decline in beach volumes since 2004 that is unrelated to POFF operations. Although the beach response during RP1 testing varies from site to site along Point White, the changes were not significant in comparison to the seasonal trends, inter-annual variability and the long-term trends.

Beach Response to Vessel Operating Parameters

The beach response during RP1 testing did not show direct correlation with distance from the RP1's sailing line. However, there is a correlation between the wake-wash energy received at the shoreline relative to the curvature of the sailing line; beaches along straighter portions of the sailing line (Point White, East Bremerton, and Port Orchard) and on the outside curvature of the sailing line (Pleasant Beach) generally receive lower energy wake wash than beaches along the inside curve of the sailing line (Point Glover). These curved areas that focus wake wash energy may be more susceptible to wake-wash effects that occur particularly when the wakes interact with shore protection structures; these effects include wave loading on the structure, scour at the toe of the structure and overtopping of the structure. Overtopping discharge of structures was not observed during RP1 operations, but predictions show only a very few structures would be susceptible to overtopping from RP1 at extreme high water levels. In addition, a 2-year storm event would cause significantly more overtopping of the structure than RP1 at extreme high water levels.

Sections of shoreline along portions of the route where fast ferries would be speeding up, slowing down, or traveling at hump speed (the specific speed of a boat that creates the largest wave height) may also be subject to increased energy. Operating RP1 at speeds that are not in the optimum range of 34 to 37 knots (kts), as well as acceleration and deceleration, may create more significant wake wash, as noted during RP1 testing in Port Orchard Reach and along East Bremerton. However, the magnitude of the beach response during the testing interval along East Bremerton was within the scale of the seasonal changes that were observed during the baseline studies.

Comparison of the RP1 testing intervals with 40 trips per week as compared to 60 trips per week showed there was no trend in sediment transport patterns or rates directly correlated to number of RP1 sailings. In general, the changes in volume are more closely correlated to the presence and absence of wind-wave events than to the number of vessel sailings. However, several sites did exhibit a subtle (small scale) but characteristic response to both POFF operations (beach flattening) and cessation of operations (beach steepening). It is therefore important not to overlook the potential for long-term cumulative effects in the planning and implementation of any future operations with a POFF such as RP1 or its equivalent. However, the research indicates that the potential for long term effects from future operations that are consistent with the test program operations appears to be insignificant.

Guidelines for the Future

This report concludes by providing guidelines and criteria for future POFF planning. These guidelines are intended to minimize the potential for long-term, cumulative effects through monitoring plans and specifying operating conditions for future fast ferries. For instance, semi-annual monitoring is recommended to maintain the baseline, and quarterly monitoring is recommended during future POFF operations. Future POFF operations, including the RP1 or any similar vessel, should be run at speeds that will minimize wake wash within Rich Passage and the Port Orchard reach. In addition, POFF schedules should start at frequencies tested during RP1 in-situ testing during months that are not correlated with seasonal change.

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ACRONYMS AND ABBREVIATIONS

AST	acoustic surface tracking
AWAC	Acoustic Wave and Current (sensor)
BMAP	beach morphology analysis package
CEM	Coastal Engineering Manual
CY	cubic yard
db	Decibel
dBA	A-weighted decibel
ft	Feet
FT-1	Fisher Tippett Type 1
GPS	global positioning system
HFMS	hull and foil monitoring system
H_{max}	Maximum Wave Height
Hz	Hertz
IPCC	International Panel on Climate Change
J/m	Joules per meter
kg	Kilogram
kt	Knot
lb	Pound
LSV	Lagrangian super-critical vessel (model)
m	meter
mm	millimeter
m/s	meters per second
MHW	mean high water
MHHW	mean higher high water
MLW	mean low water
MLLW	mean lower low water
MTL	mean tide level
NAIAD	Naiad Maritime Group, Inc.
NOAA-NOS	National Oceanic and Atmospheric Administration, National Ocean Service
NOAA CO-OPS	NOAA Center for Operational Oceanographic Products and Services
obs	observations
OT	Overtopped structure
PIT	passive integrated transponder
PB	Pleasant Beach
PG	Point Glover

PO	Port Orchard
POF	passenger only ferry
POFF	passenger only fast ferry
POT	peaks over threshold
psia	pounds per square inch ambient
PUB	pop up buoy
PW	Point White
RBR	Richard Brancker Research Ltd.
RFID	radio frequency identification
Rich Passage	Bainbridge Island and Kitsap Peninsula
RP1	*Rich Passage 1* (test vessel)
RPM	revolutions per minute
RTK-DGPS	Real Time Kinematic Differential Global Positioning System
s	seconds
SEA-TAC	Seattle-Tacoma International Airport
SNH	Snohomish
SPT	Spirit
SLR	sea-level rise
UW CIG	University of Washington Climate Impacts Group
WAC	Washington Administrative Code
WG	wave gauge
WSDOT	Washington State Department of Transportation
WSF	Washington State Ferries

1.0 INTRODUCTION

The Rich Passage Shore Response Study is a research project designed to evaluate the environmental feasibility of providing environmentally-sound passenger only fast ferry (POFF) service between the cities of Seattle and Bremerton, Washington (Figure 1-1). Fast passenger ferry operations that reduce travel time for commuters are often fundamental to the development and economic well-being of water-based communities. However, surface waves (wakes) generated by high speed ferries can potentially cause adverse effects to shorelines and properties in confined waterways and environmentally sensitive areas (Parnell and Kofoed-Hansen 2001; Bauer et al. 2002). Repeated attempts have been made to establish passenger fast ferry service over the past two decades on the 13.7-mile-long Seattle-to-Bremerton route that passes through a narrow channel between Bainbridge Island and Kitsap Peninsula (Rich Passage) in Puget Sound; however, these efforts have met with limited success as a result of such effects along the shoreline. Since 2004, federally funded research has been conducted to understand the response of Rich Passage shorelines to wake wash from high speed vessels, other vessels, and natural processes, and to establish new performance criteria for evaluating potential future high speed passenger only operations. The study has developed extensive physical and biological baseline data, assembled and validated an integrated system of predictive tools to assess the relative effects of alternative POFF operations, and conducted full-scale testing with candidate vessels on the ferry route in order to monitor shoreline response. A new low-wake vessel prototype was designed, built, and tested in a simulated operation to evaluate whether shoreline effects could be minimized through decreased wave action while regularly completing the transit in approximately 30 minutes.

In 2011, *Rich Passage* 1 (RP1), a new high-speed foil-assisted catamaran optimized for low wakes, was tested and validated against the wake criterion and wake specification developed for Rich Passage (Golder 2013a, 2013b). Wake wash acceptance of RP1 was the first step in the process of application of the vessel in a simulated passenger only fast ferry service between Seattle and Bremerton. The second step is to use the RP1 to simulate a high-speed passenger-only ferry service, and perform in-situ testing of how beaches respond to the RP1's wakes. This beach response testing is performed to quantify wake effects and to understand wake behavior from RP1 near the shorelines along the Seattle-to-Bremerton route.

A large amount of physical and biological monitoring data have been collected and assimilated to develop baseline conditions during intervals without POFF operations (Fall 2004 to Spring 2012) in Rich Passage. Baseline studies conducted from 2004 to 2007 are documented in a series of reports published in 2007 (PIE 2007a, 2007b). Physical monitoring includes measurements of waves, wakes, water levels, wind, current, gravel transport, and beach morphology. Biological monitoring includes towed underwater video surveys for eelgrass and bull kelp, quadrat surveys of fauna, macrophytes, and substrate, core sampling for benthic organisms and grain size distribution, and eelgrass mapping.

The purpose of this report is to update the baseline studies to document physical and morphologic data collected and analyzed between Fall 2007 and Spring 2012. Furthermore, this report outlines the vessel operations, physical data collection, data analysis, and application of numerical models during the in-situ beach response testing of RP1 from 25 June to 3 November 2012. Summaries of the biological studies conducted since 2007 and new studies conducted before and during beach response testing are provided in separate reports (Golder 2012; 2013c).

This report is organized into the following sections:

- Section 2 presents the methodology used to collect and process data including measurements of waves, vessel-generated wake wash, wind, beach morphology, gravel tracers, noise, and an inventory of shoreline structures from 2007 to 2012. The measurements include the interval of in-situ testing of RP1 between June and November 2012. This section also outlines the methodology for work completed between 2004 and 2007 to summarize the baseline hydrodynamic forcing and beach response.
- Section 3 is a synthesis of the results from baselines studies from 2004 to 2012 quantifying baseline hydrodynamic forcing and the associated morphologic response with an emphasis on seasonal trends. Section 3 also provides baseline information on shoreline structures and ambient noise levels.
- Section 4 presents the results of the hydrodynamic forcing and morphologic response during the interval of operations with RP1 as compared to intervals without fast ferry operations (baseline conditions). The structural response to wake wash and environmental noise generated during RP1 testing is also discussed.
- Section 5 provides conclusions and guidelines for operations and monitoring of any future fast-ferry service. These guidelines are intended to limit the potential for long-term beach response to any future fast-ferry service on the Seattle-to-Bremerton ferry route with a vessel such as RP1 or its equivalent.

Figure 1-1: Map of the Seattle to Bremerton route through Rich Passage

2.0 METHODOLOGY

Previous studies have shown that beach response in Rich Passage is dominated by the wind-wave climate in the winter and by vessel-generated wake wash in the summer. This section describes methods used in previous work to establish baseline wind-wave and wake wash climates as well as the current work that extends the interval of the climate studies and the collection of wake wash data during in-situ testing of RP1.

Measurements of beach morphology have been collected through beach profile and beach photo observations since 2004, and new methods were implemented in 2011. Baseline measurements of gravel transport were collected at two sites for 12 months (August 2006 through August 2007) and are being repeated to quantify any change in the gravel transport regime as a result of the operation of RP1 (May 2012 through May 2013). An inventory of shoreline structures was conducted in 2006 and updated in 2012 to determine existing locations, extents along the shorelines, crest elevations, relative condition, and potential areas of concern. Environmental noise levels were monitored in Rich Passage on 26 June 2012 to measure noise from RP1 operations and baseline conditions. A timeline for the data collected in 2004 through 2012 is provided in Figure 2-1.

Figure 2-1: Rich Passage – Data Timeline

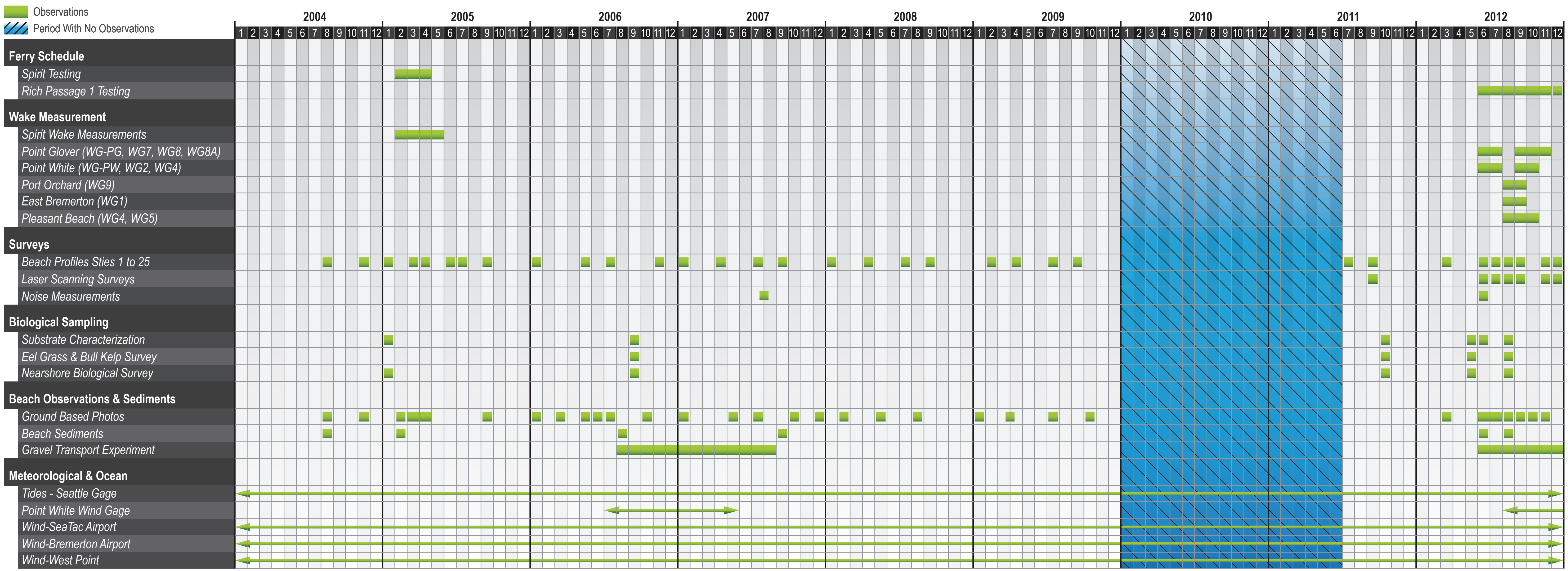


FIGURE **2-1**
RICH PASSAGE - DATA TIMELINE

2.1 Hydrodynamic Forcing

In previous work, physical measurements of the Rich Passage area were collected to develop comprehensive wind-wave, wake wash, and tidal current climatologies (PIE 2007a). The wind-wave climate analysis was repeated to document storm events during baseline conditions from 2004 to 2012. Wake wash and wind measurements were collected during beach response testing in 2012 in a similar manner to previous studies. This section describes the wind record used, the methods and instruments used to collect data on waves and wake wash (both baseline and resulting from RP1), and the processing and modeling of wave data.

2.1.1 Wind and Wind-Generated Waves

Previously a detailed analysis of wind climate was performed based on three permanent meteorological stations within the region (Seattle-Tacoma International Airport, West Point, and Bremerton Airport) and wind measured in the local project area on Point White Navigation Aid (PIE 2007a, 2007b). Wind speed data are generally similar between Point White and both Bremerton Airport and Seattle-Tacoma International Airport (SEA-TAC). However, the Bremerton wind record has a higher percentage of missing data than SEA-TAC, and Bremerton is sheltered from northerly winds blowing across Puget Sound, which might affect the east entrance to Rich Passage. Therefore for this study the SEA-TAC wind record is used, as it is considered to offer the best representation of the overall wind climate in Rich Passage, independent of local forcing caused by topographic influences, and the data provides the longest and most continuous wind record in the area.

The SEA-TAC wind record was used in an extremal analysis to determine probability distributions of wind speeds and their associated return periods. In order to assess the wind-generated wave climate in Rich Passage during RP1 operations in 2012, the local wind climate was updated through the beginning of 2013. The results of this analysis are used to compare wind events during 2012 beach response testing to wind conditions during baseline periods of monitoring in Rich Passage (Fall 2004 to Spring 2012). In addition, wind speed and direction were measured at the Point White Navigation Aid for a majority of the 2012 RP1 operations to determine the relative influence of wind-wave events on beach response.

2.1.1.1 Extremal Analysis – SEA-TAC

Historical wind records were updated for SEA-TAC (WBAN_ID 24233) through 2013. SEA-TAC has the longest established wind record in the region with continuous measurements (2-minute averages reported hourly) available beginning in 1976. Figure 2-2 shows a wind rose for the hourly record at SEA-TAC from 1976 through 2013. Wind speeds are filtered for speeds greater than 3 knots (kts). Wind directions are binned in 18° increments. In general, the wind direction at SEA-TAC is north-south aligned with the north-south axis of the local topography (and the channel on the western entrance of Rich Passage) for maximum wind speeds. A significant portion of winds also occur from the southeast in alignment with the eastern entrance to Rich Passage.

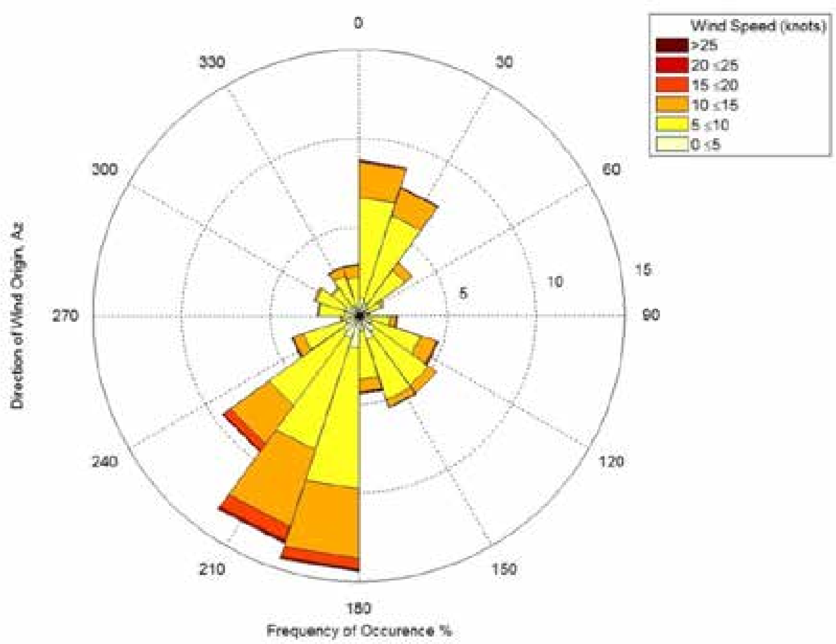
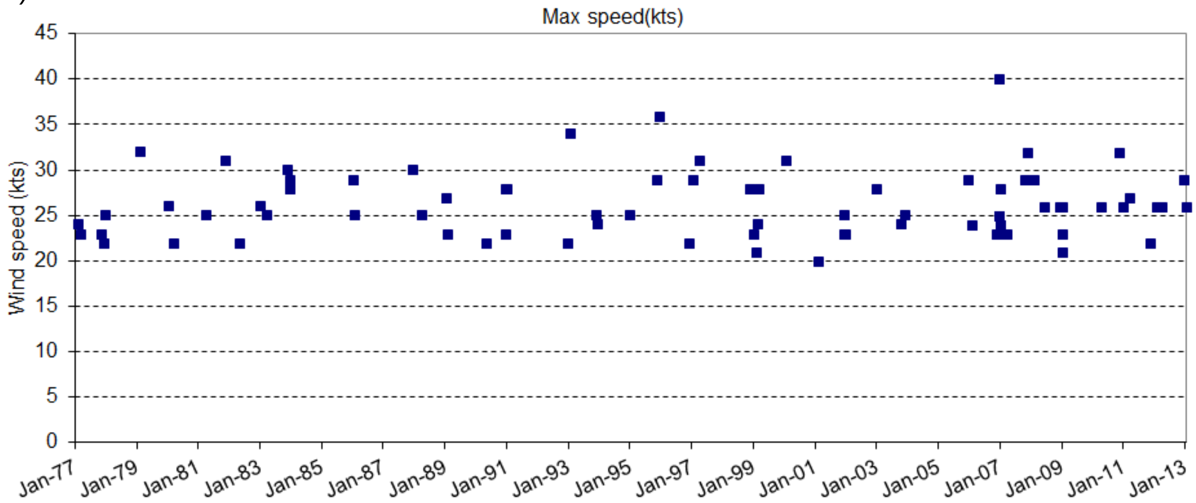


Figure 2-2: Wind rose from SEA-TAC International airport (1976-2013)

Extreme wind speeds recorded at SEA-TAC between 1976 and 2013 were used to determine probability distributions of wind speeds and their associated return periods. A peaks-over-threshold (POT) analysis was used to calculate the peak wind speeds of the largest storms during the 36-year SEA-TAC record. Extreme wind speeds were determined by filtering the record for peak wind speeds during storms with speeds greater than 20 kts sustained for at least four consecutive hours. Time of maximum wind speed, primary direction of wind, and duration of each storm were determined and summarized. Figure 2-3 shows a time series of the maximum wind speed and direction for discrete wind events generated by the POT analysis. The figure shows that most of the maximum wind speeds are directed from the SSW.

a)



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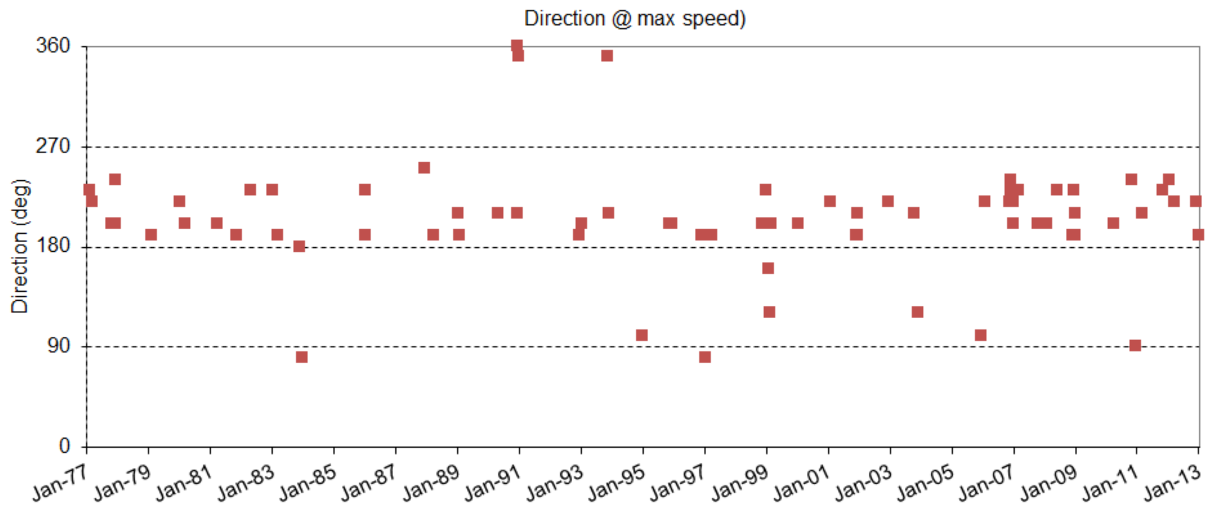

Figure 2-3: Time series of a) maximum wind speed and b) direction from POT analysis

An extremal analysis following Leenknecht et al. (1992) was applied to the station record of extremes in order to determine the 5-, 10-, 25-, 50- and 100-year wind speeds at the site. A time series of 77 maximum wind speeds measured during discrete storms between 1976 and 2013 was input to the extremal analysis. The analysis included the application of Fisher Tippett Type 1 (FT-1) and Weibull distributions to the peak water level time series. A FT-1 distribution produced the highest correlation (0.994) between the theoretical and measured distributions. Results of the analysis are summarized in Table 2-1 and a plot of the FT-1 distribution is shown in Figure 2-4. The 100-year wind speed is 40.0 kts and the 50-year wind speed is 38.0 kts.

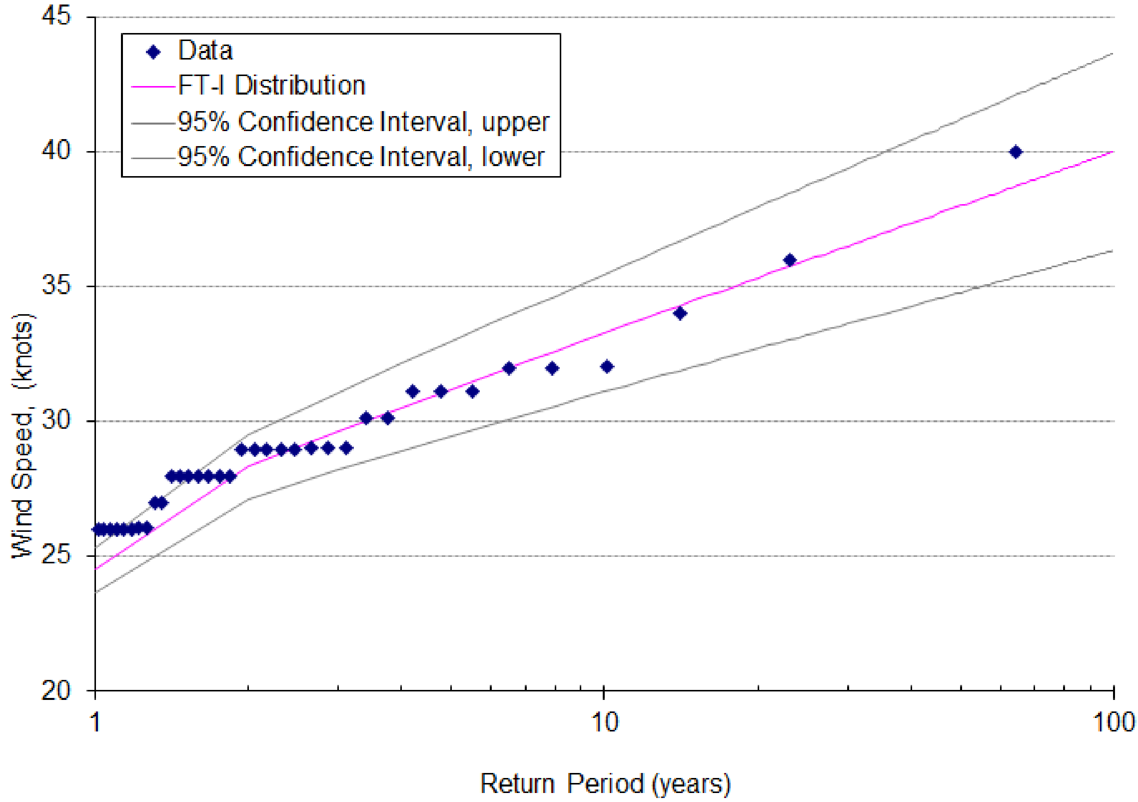

Figure 2-4: Wind speed as a function of return period for a FT-1 distribution

Table 2-1: Extreme wind speeds and associated return periods for SEA-TAC (1976-2013)

Return Period (year)	Wind Speed (knots)	95% Confidence Level
5	31.2	29.5 - 33.0
10	33.3	31.1 - 35.5
25	36.0	33.2 - 38.7
50	38.0	34.8 - 41.2
100	40.0	36.3 - 43.6

2.1.1.2 Local Wind Analysis – Point White

Local measurements of wind were collected from 22 August 2012 to 10 December 2012 by installing a wind station on Navigation Aid "10" near Point White (Figure 2-5 and 2-6). Wind speed and direction were measured by an RM Young propeller vane and helicoil integrated with electronics and software manufactured by Coastal Leasing, Inc. to provide internal power and data recording. The wind vane was mounted on an aluminum pipe approximately 10 feet (ft) above the Navigation Aid platform and approximately 40 ft above Mean Lower Low Water (MLLW). During the installation, the coupling of the wind vane was manually sited toward three individual bearings based on identifiable landmarks in the area while the directional reading from the gage was monitored to establish true north.

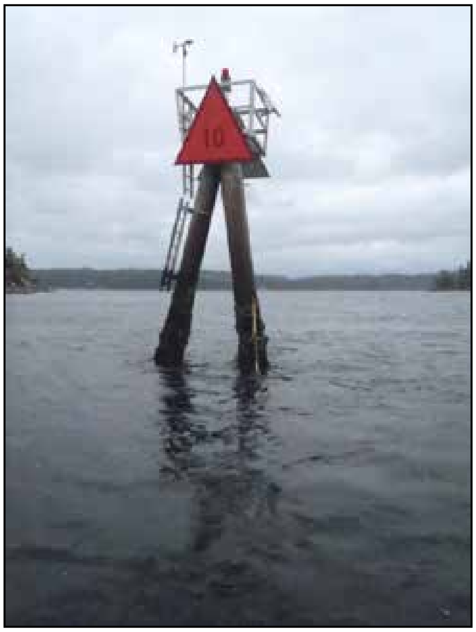

Figure 2-5: Navigation aid at Point White with anemometer installed

The wind station measured a 2-minute average of wind speed, direction, and temperature sampled at 1 Hz and recorded every 10 minutes. The data were filtered to remove wind speeds less than 3 kts (~1.5 meters per second [m/s]), and thus bias was removed towards a single direction at very low wind speeds. The data were then analyzed for individual storm events and compared to the SEA-TAC and Bremerton airport wind records, as discussed in Section 4.1.1.

2.1.2 Vessel Operations

RP1 operated from 25 June to 2 November 2012 carrying passengers to simulate a commercial-scale ferry service. For the first 10 weeks (25 June to 4 September), the operation ran 4 trips in the commuting hours of the morning and 4 trips in the commuting hours of the evening (8 one-way trips) 5 days a week (40 trips per week). During the following 8 weeks, RP1 ran an additional 2 trips mid-day and 10 trips on Saturdays (10 trips per day, 6 days a week) resulting in 60 trips per week. In general, the vessel operated at speeds of 35 to 37 kts within Rich Passage.

The wake wash produced by a high-speed vessel is influenced by a number of factors, including its loading, speed, hull shape, and depth of water. The hull shape is particularly important, and its influence is difficult to assess, since the heave and trim varies with speed and loading. The hull and foil monitoring system (HFMS) aboard RP1 was set up to log vessel performance data for each one-way trip including pitch angle, roll angle, heave, pressure distribution on the hull and foil angle as well as fuel consumption and engine RPM data, global positioning system (GPS) position, heading, and speed data. Data from the HFMS that record the dynamic motion of the vessel (heave, pitch, roll) and its horizontal position in time are used to correlate the vessel's speed and position with wave measurements.

In 2011, iterative testing was conducted to verify the wake wash from RP1 met the wake wash performance criterion and design specification for fully laden conditions (Golder 2013a). The testing was continued in 2012 under half laden and light laden conditions, to identify optimal foil and interceptor settings for these loads, and this testing overlapped with in-situ beach response testing (Golder 2013b). During the first two months of in-situ beach response testing RP1 was operated for all passenger loads with the foil and interceptor settings of 0.6 degrees and 15% determined to minimize the wake wash generation for fully laden conditions. From 27 August to 5 November, RP1 was operated with foil and interceptor settings of 0.6 degrees and 15% for passenger loading of 50 to 100% and settings of -0.6 degrees and 15% for passenger loading of 0 to 50%. It was later determined that the optimal foil and interceptor settings for wake wash generation under half laden conditions were -0.2 degrees and 15%. However, RP1 is less sensitive to foil and interceptor settings under half laden conditions and RP1 met the wake wash criterion under half laden conditions for both of the settings used during in-situ beach response testing (Golder 2013b).

The operations plan for RP1 during in-situ beach response testing included the following restrictions:

1. No operation could occur through Rich Passage at less than 34 kts; ideally the RP1 traveled at 37 kts or greater. If RP1 could not operate at speeds between 34 and 37 kts, then speeds were reduced to less than 12 kts.
2. If vessel traffic within Rich Passage made it unsafe to operate at speeds of 34 to 37 kts, then the RP1 crew waited until traffic cleared before transiting through Rich Passage.
3. From 25 June to 26 August, the Naiad Maritime Group, Inc. (NAIAD) automated foil and interceptor controls were set to 0.6 degrees foil angle and 15% interceptor for all passenger loads while operating between Bremerton and Clam Bay (eastern entrance to Rich Passage). The NAIAD system switched to 0.6 degrees and 0% interceptor for fuel efficiency between Clam Bay and Seattle.
4. From 27 August to 5 November, RP1 was operated with foil and interceptor settings of 0.6 degrees and 15% for passenger loading of 50% to 100% and settings of -0.6 degrees and 15% for passenger loading of 0 to 50%. The NAIAD system switched to 0.6 degrees and 0% interceptor for fuel efficiency between Clam Bay and Seattle for all loadings.
5. At the beginning of in-situ testing, RP1 was to operate along the center channel vessel sailing line whenever safe for navigation. After the first week of wave measurements and observations, the sailing line for RP1 was adjusted to north of the center channel line (closer to Point White), to decrease the wake wash received along Point Glover and balance the potential response between Point White and Point Glover along the narrowest stretch of Rich Passage.

When travelling through Port Orchard Reach, RP1 operated at the lower end of the testing speed range (34 kts) for fuel efficiency. Wake acceptance testing showed that the vessel still met the wake acceptance criterion at this speed, although it was not the optimal speed to minimize wake wash (Golder 2013b). However, RP1 often operated at non-optimal speeds when maneuvering through Port Orchard Reach between Bremerton and the western entrance to Rich Passage (between Point Glover and Point White) to avoid oncoming traffic in the passage.

2.1.3 Wake Wash

Non-directional wave data were collected during the RP1 test operations from 25 June 2012 to 3 November 2012 to measure the wake wash produced by RP1 at different locations along the shorelines of Rich Passage. The time series of wave properties were analyzed to characterize RP1's wake wash characteristics at each. Table 2-2 summarizes the wave gauge (WG) sites and dates where wave data was collected during the test operations, and Figure 2-6 shows the locations of the sites within the research area.

Table 2-2: Summary of wave data collection location and dates

Site No.	Type of Gauge	Start Date	End Date
WG-PW	RBR	25-Jun-12	17-Jul-12
WG-PW (2nd)	RBR	22-Oct-12	missing
WG-PG (1st)	RBR	25-Jun-12	17-Jul-12
WG-PG(2nd)	RBR	15-Oct-12	3-Nov-12
WG1	RBR	24-Aug-12	22-Sep-12
WG2	RBR	24-Sep-12	14-Oct-12
WG3	RBR	17-Jul-12	8-Aug-12
WG4	AWAC	29-Aug-12	26-Sep-12
WG5	MacroWave	24-Sep-12	14-Oct-12
WG7	RBR	24-Sep-12	14-Oct-12
WG8	RBR	18-Jul-12	8-Aug-12
WG8A	MacroWave	13-Sep-12	23-Sep-12
WG9	AWAC	23-Aug-12	22-Sep-12

Notes: The instrument deployed on October 22 was missing at the time of recovery.

Figure 2-6: Location of wave gauges and wind station in 2012

2.1.3.1 Wave Measurement Instrumentation

Two different types of bottom-mounted instruments were used to measure waves in the research area: internally logging pressure sensors wave gauges (two models) and an up-looking 1000-kHz Nortek Acoustic Wave and Current (AWAC) sensor. The pressure-sensing wave gauges are used to measure wave height, but not wave direction, while the AWAC can measure both wave height and wave direction.

The two wave gauge types, MacroWave pressure-sensing wave gauges manufactured by Coastal Leasing, Inc. and RBR*duo* T.D wave gauges manufactured by Richard Brancker Research Ltd. (RBR), were fastened to weighted underwater frames and deployed at approximately 0 ft MLLW. The frames were deployed and recovered at low tide. Several additional deployments of wave gauges were performed by attaching the sensors to the pilings of two different navigation aids in Rich Passage at approximately -2 to 0 ft MLLW. Figure 2-7 shows a wave gauge deployed at WG3.

Figure 2-7: RBR Wave and Tide Recorder (left), Coastal Leasing MacroWave (center), and Nortek AWAC (right)

The MacroWave wave gauges recorded nearly continuously at a sampling rate of 4 Hz (four samples per second). To enable the measurement of as many wake wash events as possible, the gauges recorded continuously for 34 minutes (8,192 samples), and shut down for 2 minutes to write the data before starting the next 34-minute burst. The MacroWave is equipped with strain gauge pressure sensors with an accuracy of 0.1% of full scale (30 pounds per square inch ambient [psia]).

The RBR wave gauges recorded continuously at a sampling rate of 4 Hz with no breaks in the data. The RBR pressure sensor has an accuracy of 0.01% of full scale (20 meters [m]) and a resolution of <0.001% of full scale.

The Nortek AWAC measures the water surface position using acoustic surface tracking (AST), a special mode where the instrument acts as an inverted echo sounder. The AWAC was installed on an aluminum bottom-mounted trapezoidal frame. The frame is trawl-resistant, protecting the instruments from fishing gear and debris, and the frame was weighted with 90 kg (200 pounds [lbs]) of lead brick to provide a stable platform for measurements. The instrument head was gimbal-mounted to allow for self-leveling and accurate measurement of the water surface position. The frame had no surface expression; it was deployed and recovered using a combination of divers and a Pop Up Buoy (PUBs) which is released from a vessel using a hydrophone.

Non-directional wave data were collected from 25 June 2012 to 3 November 2012 at the 11 stations shown in Figure 2-6. Table 2-2 summarizes the 13 wave gauge deployments, including the start and end date for each deployment and the type of instrument used at each station.

In general, the wave gauges recorded continuously during the interval listed in Table 2-2 with two exceptions:

- WG-PG: Instrument software failure resulted in data gaps.
- WG-PW: The wave gauge was missing upon recovery, resulting in no recovered data.

2.1.3.2 Wake Wash Time Series Processing

Surface gravity waves that propagate as a result of a passing vessel are collectively termed wake or wake wash and can be characterized by wave height and period. Figure 2-8 shows an idealized sine wave consisting of a series of crests and troughs. One wavelength comprises one crest and one trough; the wave height is the vertical distance between the trough and the crest; amplitude is one half the wave height. The wave period is the time for a single wavelength to pass a stationary position. Surface gravity waves propagate energy in the direction of wave travel; the energy density of one wave is a function of the wave height squared times the wavelength, the wave power (or energy flux) is a function of the wave height and wave celerity. The power or energy flux of a wake is an important parameter because it can be related to the potential of a wave to transport sediment. When the waves break on the shoreline the energy is transferred to the substrate and can result in particle movement, which can lead to erosion and accretion.

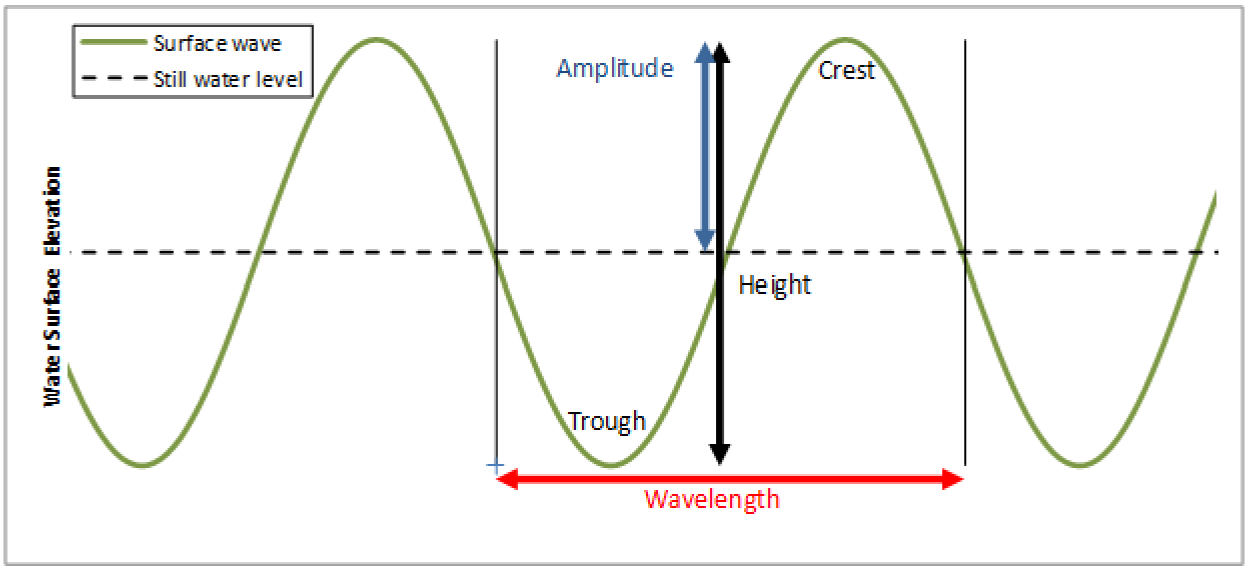

Figure 2-8: Plot of idealized surface wave with wave properties defined

The processing of the wave gauge data involves using existing MATLAB© routines as the framework for computing, summarizing, and visualizing wave time series and wave properties. The time series of wave properties are then analyzed to develop wake wash characteristics. A description of the processing routines follows:

- MATLAB routines are used to convert pressure to water depth above the instrument using an average barometric pressure and water density during the deployment period. Except for the data collected with the AWAC which measures the water surface position directly using AST, a special mode where the instrument acts as an inverted echo sounder.
- The processing routines convert static water depth to a water surface elevation time series suitable for zero-crossing (up- and down-crossing) analysis to estimate wave height, period, and energy. The conversion includes a correction for pressure attenuation as a function of depth and wave frequency. Corrections are applied in the frequency domain and parameters are converted to the time domain for analysis. The attenuation correction factor is based on the linear wave theory dispersion relationship, and the maximum frequency cutoff is based on that reported by Earle et al. (1995) dependent on the depth of water and the response characteristics of the sensor. A zero-crossing analysis of the water surface elevation time series is used to generate a series of wave height, period and energy for each burst.
- A sensitivity analysis was conducted for the data collected during for the RP1 testing by limiting the frequency cutoff of 0.6 Hz, which was used for processing the vessel wake wash. This removes energy at frequencies greater than 0.6 Hz.
- The highest frequency wave that can theoretically be measured for a particular sampling frequency is determined by the Nyquist frequency. The Nyquist frequency is one half the sampling frequency; the upper bound for the data sampled at 4 Hz is thus 2 Hz, equivalent to a wave period of 0.5 s (minimum measurable wave period).
- Automated MATLAB software is used to identify wakes based on the results of the zero-crossing analysis and produce a time series of vessel wake events at the instrument location. The RP1 vessel position is used to determine when RP1 passed an instrument location.

Wave statistics for the wake wash from each vessel sailing were output to a summary file. The important statistics for each wake wash event used for further analysis are as follows:

- H_{max} – maximum wave height in the wake wash time series
- $T@H_{max}$ – wave period of the maximum wave height
- Power – wave power of the wake wash time series (energy density integrated over time)
- F_p – peak frequency (Hz) of the wake wash
- T_p – peak period of the wake wash/mean period of the highest one-third of waves

Following the initial processing, the wake wash time series were analyzed to develop summary statistics of the wake wash generated by RP1 at each site. Wake wash data were screened during the initial processing step for the following:

- Incomplete or missing time series due to
  - Instrument failure
  - Data gaps resulting from burst sampling protocol
- Wake wash contaminated by other waves such as
  - Washington State Ferry vessel wake wash

- Other vessel wake wash
- Wind waves
- Invalid data caused by
  - Pressure sensor depth too deep to resolve high frequency wave correctly
  - Mean water level at or below the level of the instrument sensor

A secondary step was taken to remove wake wash time series generated by RP1 when not operating under standard protocols; i.e. operating at speeds less than 30 kts within Rich Passage or decelerating within the passage. Following the screening process, aggregate statistics were compiled to develop a wake wash climate for each site. Detailed tables of statistics for every wave measured are provided in Appendix A.

2.1.4 Integrated Modeling

The hydrodynamic environment for the study area was developed by combining the results of separate models for tides, wind-waves and vessel wakes.

Tidal flows and levels were computed using a calibrated ADCIRC finite element tidal model of the study area. Results of model validation simulations showed that the tidal model accurately reproduced the current speed and directions at each of four measurement locations in Rich Passage, and captured the flood dominance along the north shore of Rich Passage caused by the re-circulation gyre during the ebb tide. For impact assessment, long-term tidal records at each of the 27 study sites were extracted from the ADCIRC results for a neap tide to spring tide cycle. These results were used to synthesize two 6-month simulations (one each for summer and winter conditions).

Summer and winter wave climates were developed using the WABED (now known as CMS-Wave) spectral wave model. Since measured wind conditions were used to drive the wave models, three separate models were established; each model was oriented to match dominant wind directions (north, southwest and southeast). A matrix of 75 wave parameter combinations were modeled, covering the three dominant wind directions, five wind speeds, and five water surface elevations. Using these runs, a wind-wave climate was constructed for each of the 27 study sites.

Wake climates for a number of vessels that operate in the study area were developed using the LSV wake model (MacDonald 2005; PIE 2007b). This model provides accurate descriptions of the temporal and spatial wake train patterns, including their generation, propagation and transformation by currents and bathymetry. The model accurately described the processes of wake train dispersion, as well as shoaling, refraction, and breaking. Simulations were performed for a number of vessels, including:

- Washington State Ferry (WSF) car ferry
- Large, earlier generation POFF, M/V Snohomish (SNH)

- An earlier candidate POFF, M/V Spirit (SPT)
- Low-wake POFF, Rich Passage 1 (RP1)
- Miscellaneous other vessels (e.g., small craft) that ply the area

Since the LSV model is too computationally expensive to be used for simulations of large areas over long periods of time, a set of representative runs were modeled for each vessel type. These results were then sampled and used as input to the 27 detailed ProfileAnalysis cross-shore models.

ProfileAnalysis is a one-dimensional, profile-based model that provides a long-term integrated assessment of the relative effects of the major forcing mechanisms (PIE 2007b). ProfileAnalysis was the primary tool for investigating the relative effects of tides, waves and wakes on the shores of the study area. The model was developed specifically for the Rich Passage project and provides semi-quantitative, efficient and integrated predictions of the effects of wind-waves, tides and wakes on the shores of the region. Unlike the models described in the previous paragraphs, ProfileAnalysis can be used for simulations lasting months at a time, which is required in order to assess the processes in Rich Passage that affect morphological change. A separate ProfileAnalysis model simulation was performed at each study site. The model uses input from the tide, wave, and wake models at the offshore boundary (5-m contour), to transform the hydrodynamic forcing mechanisms across the profile to the shore, while computing sediment transport and profile evolution. The sediment transport routines include asymmetrical wave-induced transport, which is important for the modeling of coarse material, such as gravels and cobbles, which make up the beaches of the area. Appendix B contains a detailed report on the numerical modeling Coldwater Consulting performed to simulate the wake wash and beach response from RP1 operating through Rich Passage.

2.2 Morphologic Measurements

The methods described in this section were used to collect data that helped to describe beach morphology and gravel transport between 2004 and 2012 during intervals of baseline studies and in-situ beach response testing to RP1.

2.2.1 Beach Profiles

The beach profile monitoring network was initiated in April 2000 in response to beach effects from WSF-operated POFF vessels along the Seattle to Bremerton route. The initial monitoring network consisted of 14 profiles which were surveyed through 2001, and then re-surveyed as part of the existing research program starting in August 2004. There is a significant gap in the data between 2001 and 2004. Therefore the baseline period used to describe the beach morphology along Rich Passage shorelines is from August 2004 to May 2012. A period with no beach observations occurred between September 2009 and July 2011 when research funding was being conserved.

Beach profile monitoring surveys are conducted with a Trimble Real Time Kinematic Differential Global Positioning System (RTK-DGPS). Profiles are surveyed from the top of bulkheads to wading depth at low tide. Total profile length was established at 200 ft but most sites are surveyed on foot out to a distance of 100 ft from the bulkhead. The RTK-DGPS measurements have a horizontal accuracy of approximately +/-0.33 ft and vertical accuracy of +/- 0.16 ft. The profile network is shown in Figure 2-9 and dates for the profile monitoring surveys are shown in Figure 2-1.

Figure 2-9: Beach profile and laser scanning survey sites

Beach profiles were processed and analyzed using the Beach Morphology Analysis Package (BMAP) (Sommerfeld et al. 1993 1994; Wise 1995), to compute beach volume changes above and below the mean tide level (MTL) for beach monitoring sites. Figures in Appendix C show the above and below MTL beach volume change from August 2004 to December 2012 for representative sites along the shorelines of Rich Passage.

2.2.2 Laser Scanning Surveys

Laser scanning topographic surveys of three sections of shoreline were made using a Leica Scan Station 2 and RTK-DGPS starting in 2011. The Leica Scan Station is used to capture high resolution three-dimensional ground terrain data of the beach from the bulkhead to low water with an accuracy of 0.1 ft. Surveys were conducted in September 2011 and June 2012 in conjunction with beach profile monitoring to validate the technique and provide a baseline survey for comparison to laser scans collected on the same schedule as other morphologic measurements during RP1 beach response testing. One laser scanning survey site is on Point Glover (Site 9) and the other two sites are on Point White (Sites 3 and 4) (Figure 2-9). Laser scan surveys were performed on the dates shown on the timeline in Figure 2-1, which generally correspond to intervals when beach profiles were measured. A more complete description of the laser scanning survey methodology is provided in a report by Triad Associates, who were contracted to conduct the surveys (Appendix D).

Three-dimensional point data was provided by Triad Associates to Golder for processing in Surface Modeling System v10.1. The cloud data were interpolated onto a grid with a resolution of 0.25 ft by 0.25 ft with boundaries as represented in Figures 2-10 through 2-12. The gridded data from each survey interval was used to create difference plots by subtracting the elevation at each point in the grid from the elevation at the same point measured at a subsequent time interval. The elevation differences were color contoured to show morphological changes in the beach at each site such that hot colors (red to yellow) represent accretion, cool colors (blues) represent erosion, and white indicates no change.

Morphologic change maps were calculated for each survey interval at each of the three sites (Appendix E). In addition, morphologic change maps were developed for key time intervals: September 2011 to June 2012 to represent the baseline condition, June to November 2012 to represent the interval of RP1 operations, and November 2012 to December 2012 to depict the post-operation interval.

A systematic analysis of beach volume change was conducted after the morphological change surfaces were developed from the laser scanning surveys. First, a series of calculation mask boundaries were developed to constrain the area by the extent of the survey with the least coverage. The calculation mask boundaries at Site 3 and Site 4 on Point White and at Site 9 on Point Glover had a cross-shore extent that spanned the beach from the bulkheads to approximately -2.67 ft MLLW contour. The mask boundaries cover nearly the full survey domain of each of the sites, but are truncated along the boundaries of the survey to eliminate data gaps associated with uneven boundaries and limited scanning survey site lines. The mask boundaries were further subdivided to analyze cross-shore and alongshore exchanges of beach volume within each of the survey boundaries. Sites 3 and 4 on Point White were divided into four subdivisions by splitting the mask laterally at approximately MTL and longitudinally at approximately half the length of the mask boundary. The survey area at Site 9 on Point Glover is not as uniform in shape and orientation relative to the shoreline compared to Sites 3 and 4 on Point White. Site 9 also contains several groins that disrupt alongshore transport. Therefore, Site 9 was divided into 8 subdivisions by splitting the mask laterally at approximately MTL and longitudinally into approximately quarters where the boundary of the cells is located at a groin if one is present in the cell. The mask boundaries are shown in Figures 2-10 through 2-12. The mask boundary coordinates are presented in Appendix E.

Figure 2-10: Volume calculation mask boundary for Point White – Site 3

Figure 2-11: Volume calculation mask boundary for Point White – Site 4

Figure 2-12: Volume calculation mask boundary for Point Glover – Site 9

After the volume mask boundaries were selected, the volume eroded or accreted was calculated by selecting the elevation change surface for each survey period and multiplying the mask area by the positive elevation change to yield accretion volume, and multiplying by the negative elevation change to yield erosion volume. The total volume change is calculated by summing the positive changes (accretion) and the negative changes (erosional).

The volume change analysis provides the total volume change over the high-resolution survey area in order to quantify the movement of sediment both in the cross-shore direction and in the alongshore direction; this data can be used to analyze the development of bars or berms, sediment transport patterns, and morphologic recovery. The data also supports the development of a sediment budget to determine the volume gains or losses and potential sediment pathways. A sediment budget is the balance of the sediment volume entering and exiting a defined area, in this case the volume mask boundaries. A sediment budget may be defined by the following:

$$\sum Q_{source} - \sum Q_{sink} - \Delta V + P - R = Residual$$

Where Q is the transport flux into (source) or out of (sink) a pre-defined calculation cell boundary, ΔV is the volume change, and *P* and *R* are volume source and sink terms representing the mechanical placement of sediment volume or removal of sediment volume, respectively. The results of the volume change analysis for each of the three survey sites (Site 3, 4, and 9) will be summarized and discussed in Section 4; the complete set of calculations is included in Appendix E.

2.2.3 Beach Photos

Observations of beach condition and beach elevations at bulkheads have been made along the shorelines of the ferry route since June 2004. Beginning in January 2005, geo-referenced and time-stamped photographs were acquired approximately quarterly at a number of reference locations throughout the study area. The resulting time series of photographs and beach elevations relative to bulkheads for each beach sub-section are shown in Appendix F. A summary of the beach observations in each of the beach sub-sections is provided in Section 3.0.

2.2.4 Gravel Tracers

Direct measurements of pebble, gravel and cobble (hereafter collectively referred to as gravel) transport were obtained from three sites in the study area (Sites 3, 4, and 5; Figure 2-9) by means of a particle tracing technique that involves Radio Frequency Identification (RFID) technology, Passive Integrated Transponder (PIT) tags, and RTK-DGPS surveying. Gravel transport studies in Rich Passage were first conducted in 2007 and details of the RFID system were documented as part of the previous studies

(PIE 2007a). Results have been published in peer reviewed journals by Curtiss et al. (2010) and Osborne et al. (2011).

Beach gravels were sampled from the surface armor layer at each of the gravel tracer sites in May 2012 using pebble count methods (Wolman 1954) and a gravelometer. The gravels were sampled by placing a 50-ft measuring tape on the beach surface parallel to the shore, and acquiring a pebble at 1-ft increments along the length of the tape, yielding a sample of 50 particles. The size class for each particle was determined by passing the particle through the smallest possible opening in the gravelometer. The tape was then moved in increments down the beach face, and the sampling process was repeated to obtain additional samples. This process was repeated between +8.5 ft and +2 ft MLLW at Site 3, between +7.2 ft and +2 ft MLLW at Site 4, and between +8.7 ft and +2 ft MLLW at Site 5 until at least 350 particles were sampled from each deployment site. These data were analyzed to determine grain size distribution at each of the three sites. Plots of grain size distribution at each size are provided in Appendix G. A set of 48 to 50 gravel tracer particles were chosen to most accurately represent the grain size distribution at each site. The particles in each set are classed into four size classes with nominal mean sizes of 16 mm, 22.6 mm, 32 mm and 45 mm. Particle sizes less than 12 mm are not represented in the tracer sample because of the limitation imposed by the minimum size of the PIT tags.

Three sets of tracers were deployed at monitoring Sites 3, 4, and 5 on Point White on 7 June 2012. Each of the three sets of tracers were placed in an 8-ft-by-6-ft grid centered at approximately +9 ft MLLW with 1-ft spacing between tracers. This elevation was deemed to be most appropriate for maximum exposure to forcing mechanisms that could act to transport the tracers.

The tracer set at Site 3 was originally deployed on 7 June 2012, the same day as Sites 3 and 4, but the tracer set was collected by local residents between 7 June and 21 June and redeployed by Golder staff by scattering the tracers around the intertidal zone on 21 June. The three sets of tracers were recovered every two weeks for eight weeks after deployment and monthly thereafter (Table 2-3). On 5 July, an equipment failure (batteries) resulted in the survey not being completed at Site 3 for that survey period.

Recovery rates were generally above 70% at Sites 3 and 5 prior to the November and December surveys (Figure 2-13). The November and December surveys were conducted at night due to the timing of low tide, resulting in lower recovery rates because of the challenges of surveying at night. At the time of tracer deployment, the upper beach at Site 4 was mostly sand and lacked a surface layer of gravel. This may have caused a substantial portion of the tracers to move in the offshore direction, as part of a sorting or equilibration to the cross-shore variation in native sediment distribution.

Table 2-3: Deployment and monitoring dates of each group of tracers placed on Point White (PW)

Site 3 Deployment	Site 3 Surveys	Site 4 Deployment	Site 4 Surveys	Site 5 Deployment	Site 5 Surveys
6/7/2012		6/7/2012		6/21/2012	
	6/21/2012		6/21/2012		7/5/2012
	7/19/2012		7/5/2012		7/19/2012
	7/31/2012		7/19/2012		7/31/2012
	8/29/2012		7/31/2012		8/29/2012
	9/24/2012		8/29/2012		9/24/2012
	11/13/12		9/24/2012		11/13/12
	12/9/12		11/13/12		12/9/12
			12/9/12		

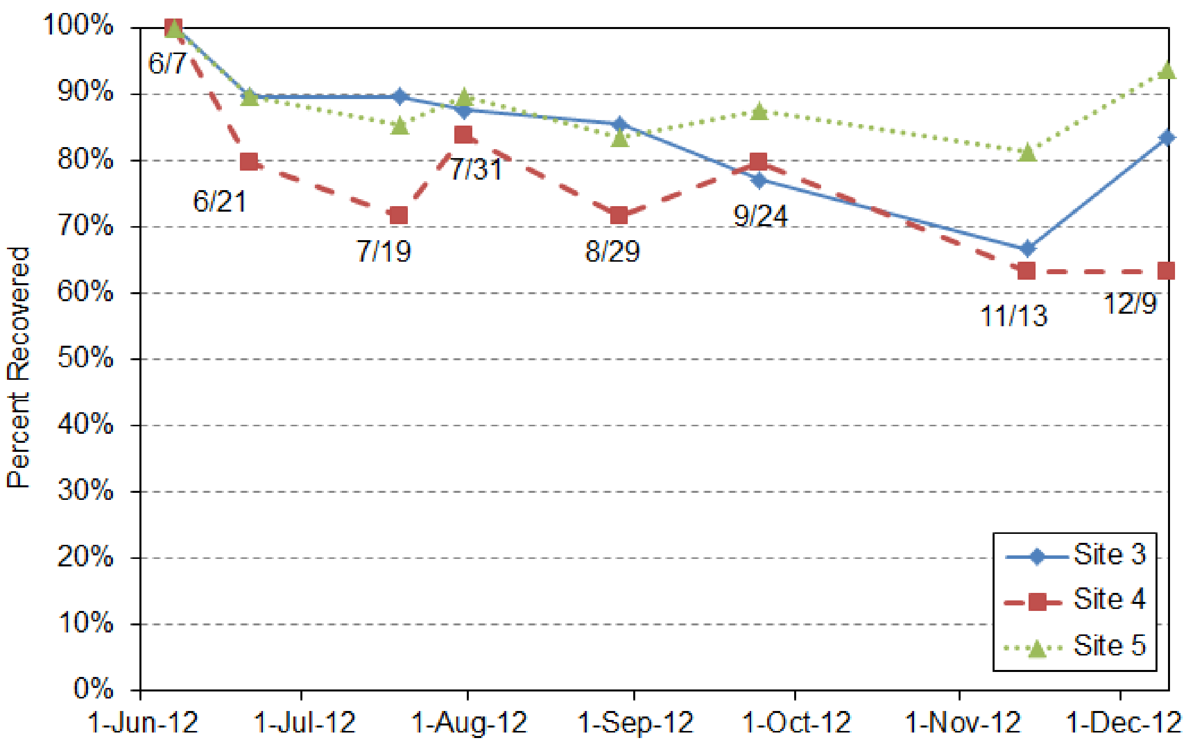

Figure 2-13: Recovery statistics for Sites 3, 4, and 5

2.3 Structures

Several structures, such as bulkheads, retaining walls, boat ramps, groins, and revetments, were surveyed in August and September 2006 to determine both their current locations and extents along the shorelines (PIE 2007a). The general condition of structures, especially the bulkheads was assessed during this survey. The survey of the bulkheads produced a detailed summary of crest elevations of these structures, which were used as the basis for an analysis of their susceptibility to overtopping due to both wind-generated and vessel-generated waves (PIE 2007b). Bulkhead surveys also included measurement of the elevation at the toe of the bulkhead and beach slope in front of the bulkhead. These data were used to calculate the relative position of the bulkheads with respect to tidal datum and the run-up and potential for overtopping of the structures by various wind-wave and vessel-generated wave conditions. In addition, the bulkhead survey provided data to assess the susceptibility of various beaches to enhanced scour caused by wave reflections from structures. The structure survey was repeated by Golder in March and April 2012 to document any changes in condition and elevation since the 2006 survey.

2.4 Overtopping

An analysis of susceptibility of coastal structures in Rich Passage to overtopping from wind-waves and vessel-generated waves was conducted in 2007 at several high water level conditions (PIE 2007b). Bulkheads were analyzed for runup and overtopping from wind-generated waves, fast ferry vessel wake wash, and varying water levels. The analysis was updated in 2013 to reflect changes to extreme water level conditions using updated water level data and to include wake wash parameters for RP1 using output from the LSV model (MacDonald 2005; PIE 2007b). The following updates were made to the analysis:

- The extremal analysis of water levels was updated to include the historical water level data at the Seattle tide station through March 2013.
- The assessment was performed for two additional water levels that account for low probability and high probability projections of sea-level rise (SLR) in Puget Sound to 2050.
- The assessment was completed using the modeled RP1 wake wash parameters.
- Bulkhead crest and toe elevations were updated based on the 2012 structure survey results.

The purpose of the analysis was to identify those bulkheads that might be susceptible to overtopping at high water levels and wake wash from RP1. It is important to note that wake wash and wave parameter output from the LSV model are not specific to each structure, but are output at several (27) nodes distributed evenly throughout the study area and used to estimate wave conditions at the nearest structure locations to the respective output nodes. In general, the results reflect broadly the patterns of anticipated wave overtopping and susceptibility in the study area. However, the results should not be interpreted or applied as detailed site specific information except in those cases where the prediction node coincides with a particular structure location. A more detailed study and analysis would be necessary to assess the overtopping discharge at each specific location and property in the study area.

2.4.1 Water Level Analysis

The water level in Rich Passage is primarily influenced by astronomical tides which result in a predictable rise and fall of the water surface. Non-tidal fluctuations in water level may also be caused by a number of mechanisms including storm surge, which is measured by a rise in water surface as a result of the variation of barometric pressure across the path of a storm and wind shear stress on the water, as well as tsunamis, long waves, long-term sea level change, thermal expansion or contraction, pressure gradients from large scale oceanic processes (boundary currents, kelvin waves, etc.), precipitation and surface water inputs, and other non-cyclical astronomical conditions. These mechanisms are considered collectively in order to develop an analysis of extreme water levels from actual long term measurements of water level at a location near the project site.

Tides in Puget Sound are mixed semi-diurnal, meaning two high tides and two low tides of differing heights occur each day. For each tidal cycle there exists a lower low, low, high, and higher high tide. A record of water levels is available from the Seattle National Oceanic and Atmospheric Administration, National Ocean Service (NOAA-NOS) station 9447130 (47° 36.1' N 122° 20.3' W) tide gauge (NOAA Center for Operational Oceanographic Products and Services [NOAA CO-OPS] 2013). The tide gauge was established in 1899; however, records of monthly maximum water levels for the station are available from 1955 to 2013. Long term historical water level extremes were used to conduct an extremal analysis of the highest water levels and determine the probability distribution of water levels and their associated return periods.

The elevations for tidal datums relative to mean lower low water (MLLW) and North American Vertical Datum 1988 (NAVD88) based on the tide gauge are presented in Table 2-4. The tidal datums are based on the most recent epoch (1983 to 2001) which occurs in multiples of the 19-year metonic cycle. The 19-year epoch accounts for the 18.6-year lunar nodal period, but is extended to 19 years to prevent datum bias due to local seasonal variation in sea level. MLLW is the average of all the daily lower low tides in the epoch and the mean higher high water (MHHW) is the average of all the daily higher high tides in the epoch. Mean low water (MLW) and mean high water (MHW) are averages of all lows and highs, respectively. The mean tide range at Seattle (NOAA NOS tide station 9447130), the difference between MHW and MLW, is 7.66 ft. The highest water level at Seattle was 14.48 ft, MLLW and was recorded twice on 27 January 1983 and again on 17 December 2012.

Table 2-4: Tidal datum elevations for Seattle, WA relative to MLLW and NAVD88.

Datum	Water Level (ft, MLLW)	Water Level (ft, NAVD88)	Description
MHHW	11.36	9.02	Mean Higher High Water
MHW	10.49	8.15	Mean High Water
MSL	6.64	4.30	Mean Sea Level
MLW	2.83	0.49	Mean Low Water
MLLW	0	-2.34	Mean Lower Low Water
NAVD88	2.34	0	North American Vertical Datum 1988
Maximum	14.48	12.14	Highest Water Level on Record: 27 January 1983; 17 December 2012
Minimum	-5.04	-7.38	Lowest Water Level on Record: 04 January 1916

Datum data obtained from NOAA Station ID: 9447130 (47° 36.1' N 122° 20.3' W) tide gauge

Extreme water levels recorded at NOAA NOS station 9447130 between 1955 and 2013 were used to determine probability distributions of water levels and their associated return periods. An extremal analysis following Leenknecht et al. (1992) was applied to the station record of extremes in order to determine the 2, 5, 10, 25, 50 and 100-year water levels at the site. A time series of 58 maximum water levels measured as discrete events between 1955 and 2013 were input to the extremal analysis. The analysis included the application of Fisher Tippett Type 1 (FT-1) and Weibull distributions to the peak water level time series. A Weibull distribution with a shape factor of k = 2.00 produced the highest correlation (0.96) between the theoretical and measured distributions. Results of the analysis are summarized in Table 2-5 and a plot of the Weibull distribution with k = 2.00 is shown in Figure 2-14. Table 2-5 gives the extreme water levels relative to both MLLW and NAVD88; the 100-year water level is 14.76 ft, MLLW.

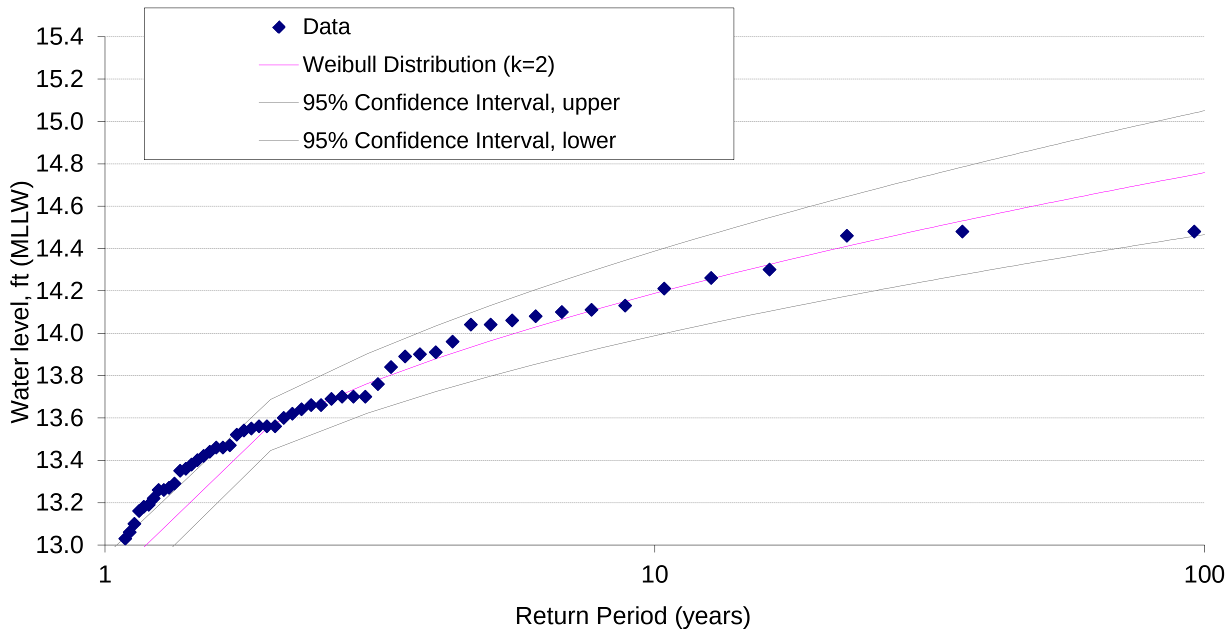

Figure 2-14: Water level as a function of return period for Weibull Distribution (k=2.00)

Table 2-5: Extreme water levels and associated return periods for Seattle, WA relative to MLLW and NAVD88.

Return Period (year)	Water Level (ft, MLLW)	Water Level (ft, NAVD88)	95% Confidence Interval (ft, MLLW)
2	13.57	10.90	13.4 – 13.7
5	13.96	11.29	13.8 - 14.1
10	14.19	11.52	14.0 - 14.4
25	14.44	11.77	14.2 - 14.7
50	14.61	11.94	14.3 - 14.9
100	14.76	12.09	14.5 - 15.1

2.4.2 Sea-Level Rise Projections for Puget Sound

Local SLR in Puget Sound can be attributed to the combined effects of global SLR and local factors such as vertical land deformation and seasonal ocean elevation changes due to atmospheric circulation effects (Mote et. al 2008). Projections for 21st century global SLR are provided by the United Nations International Panel on Climate Change (IPCC) using climate change models for different emissions and model scenarios. In the most recent report, the IPCC projects global SLR to be between 0.6 to 1.25 ft for the lowest emissions scenario and 0.8 to 1.9 ft for the highest emissions scenario (IPCC 2007). The highest emissions scenario anticipates a potential rapid loss of ice mass from Antarctica and Greenland.

The University of Washington Climate Impacts Group (UW CIG) provides estimates for local SLR in Puget Sound that consider the IPCC projections as well as local factors of tectonic movement and atmospheric circulation (Mote et al. 2008). Table 2-6 summarizes the projections from the IPCC and the UW CIG. The low-probability, high impact scenario of SLR for the Puget Sound was estimated to be 1.83 ft by 2050, the high-probability, medium impact scenario was estimated to be 0.5 ft, and the low-probability, low impact scenario was estimated to be 0.25 ft by 2050.

Table 2-6: Summary of Global and Local Sea Level Rise (SLR) Projections for Puget Sound

Source	**SLR Projection (ft)**	**Notes**
IPCC 4th Assessment Report (lowest emissions scenario) - 2100	0.6 to 1.25	Global
IPCC 4th Assessment Report (highest emissions scenario) - 2100	0.8 to 1.9	Global; anticipates a potential rapid loss of ice mass from Antarctica and Greenland
UW CIG (low probability, high impact scenario) - 2050	1.83	Local projection for Puget Sound; anticipates a potential rapid loss of ice mass from Antarctica and Greenland
UW CIG (high probability, medium impact scenario) - 2050	0.5	Local projection for Puget Sound
UW CIG (low probability, low impact scenario) – 2050	0.25	Local projection for Puget Sound

2.4.3 Wave Structure Interaction

The wave structure interaction for each section of bulkhead was analyzed using the same methodology as described in a previous report (PIE 2007b). The overtopping and runup were calculated using formulas from the Coastal Engineering Manual (CEM) (USACE 2002): the Goda (2000) formula for vertical walls, the van der Meer and Janssen (1995) formula for overtopping of revetment structures, and the Hunt formula for wave runup on a pebble beach. The formulas were applied depending on the type of structure as classified during the 2006 assessment. The different types of structures used in the classification are described below:

- Type 1 – Revetment structure (slope less than 1:1)
- Type 2 – Smooth Vertical wall
- Type 3 – Perforated vertical wall (representative of a vertical faced armor stone retaining wall with no grouting)

Structure elevations were updated in 2012 as described below (Section 2.4.3.2). An additional formula proposed by Franco & Franco (1999) was used to support the results from the formulae used in the previous report.

2.4.3.1 Wave Runup and Overtopping

In addition to the overtopping and runup equations used in the previous analyses, calculations using the method developed by Franco & Franco (1999) were performed to verify the previous methodology against a formulation that incorporates permeability of the structure. The Franco & Franco formula calculates overtopping discharge, used for impermeable and permeable vertical walls.

$$q = 0.082 \exp\left(-3.0 \frac{\mathrm{Rc}}{\mathrm{Hs}} \frac{1}{\gamma_\beta \gamma_s}\right)$$

Where R_c is the freeboard, H_s is the wave height at the structure, γ_β is the reduction factor for angle of attack and γ_s is the factor defining the permeability of the wall face. The units of the discharge are liters per second per meter width.

For the purpose of this analysis a positive discharge $q > 0\ gal/min$ per ft, where gal/min is gallons per minute, means that the structure is overtopped. The discharge results were compared to the runup results in order to validate the methods used in the previous report (PIE 2007b). The number of overtopped structures and the qualitative magnitude of discharge during each water level and wave/wake forcing condition were in agreement between all methods.

2.4.3.2 Structure Survey Update

For the updated analysis a combination of the 2006 and 2012 survey data were used. During the 2012 survey a majority of crest elevations were not surveyed because of access issues to the waterfront properties; in general only the toe of structures was surveyed for each structure. Approximately 90 crest elevations were measured to confirm the 2006 measurements on the same structures throughout the study area. Based on the measurement comparison and an analysis of the survey controls and monuments, the 2006 crest elevations for bulkheads along the Point White and Pleasant Beach shorelines were adjusted by -0.571. Survey data from 2012 was used without the need for adjustment

2.4.3.3 RP1 Wake Wash Parameters

Wave parameters (height, period, and direction) at each study site (27 in total) were predicted using the LSV model (Section 2.1.4) for RP1 traveling from Seattle to Bremerton and Bremerton to Seattle. The wave parameters are an average of model runs within a 0.5 m water elevation window around MSL. For vertical wall bulkheads the design wave required estimation of the wave height at the toe of the structure. ProfileAnalysis numerical model (PIE 2007b) was used to calculate wave parameters just before wave breaking on the beach. The wind-wave parameters for the 2-year return period storm (33 kt winds) used in the 2007 analysis (PIE 2007b) were repeated.

2.4.3.4 Sources of Error

The results of the overtopping analysis were generated using empirical methods for predicting the precise amount of wave runup on beaches and structures. Complications inherent to runup prediction that may introduce error include non-linear wave transformation, wave reflection, three-dimensional effects (bathymetry, infragravity waves), structure geometry, porosity, roughness, permeability, and groundwater elevation. By not accounting for these effects, the calculated wave heights at certain locations may differ from measured wave heights. There are locations where walls are set at 90 degrees, and amplification of wave height will occur near the intersection of these walls due to reflections. This can cause localized wave heights in excess of those calculated. Furthermore, nearshore bathymetry will affect the shoaling and breaking locations of waves, and this can significantly change the overtopping with respect to the calculated overtopping values.

A depth of -17 ft MLLW at the base of the nearshore slope was assumed for the purpose of incident wave estimates used in the calculations of wave runup on beaches and revetment structures. Overtopping of vertical wall structures was determined using wave transformation performed with two dimensional (2-D) transects that were surveyed at select locations. This was necessary as the overtopping formula for vertical walls does not account for wave period, and requires a design wave height at the wall instead of an offshore wave height. The nearshore bathymetry was not surveyed for all structures, and thus it was necessary to make the assumption that the local bathymetry was constant between surveyed profiles.

The change in the monument elevation reported by Washington State Department of Transportation (WSDOT) from 2000 to 2008 represents another source of error for the overtopping calculations of Point White and Pleasant Beach. The reported shift was incorporated in the 2006 survey data; however there is an uncertainty in the elevations measured due to the change in the monument elevation, and it has not been confirmed by WSDOT what produced this shift in the monument elevation.

2.5 Noise Measurements

Environmental noise levels were monitored in Rich Passage during RP1 operations on 26 June 2012. Noise levels were measured using sound meters near Site 4 on Point White and Site 8 on Point Glover (Figure 2-9).

A SPER Scientific data logging sound meter (model 840013C) was placed 1.5 m above the beach on a level tripod (Figure 2-15) and set to collect one sample per second (1 Hz) for the duration of each measurement. The range of the meter was 30 to 130 decibels (dB) and was set to auto-range to increase the sensitivity of the measurements as needed. The meter was positioned approximately 3 ft above the beach slope/bottom to measure the ambient noise environment and avoid focusing on sound from a particular direction. Measurements of noise levels were collected during the passage of RP1, the

Washington State Ferries (WSF) Walla Walla and Kaleetan car ferries, and fishing and recreational boats. Noise levels were also measured during three periods of ambient conditions when no vessels were present.

Sections 3 and 4 of this report summarize the noise level measurements performed on 26 June 2012 for the different vessels and ambient conditions in Rich Passage.

Figure 2-15: Noise recording device located on Point Glover beach

3.0 BASELINE CONDITIONS

This section summarizes the analysis of physical data and numerical modeling conducted between August 2004 and May 2012 to quantify baseline hydrodynamic forcing and the morphologic and structures response to this forcing with an emphasis on seasonal trends.

3.1 Hydrodynamic Forcing

During previous studies, existing vessel traffic, wind-wave energy, and tidal currents were established as the baseline hydrodynamic forcing mechanisms to be used as a basis for comparison with future POFF operations in Rich Passage (PIE 2007b). This section describes baseline data collected on wind records, wind-generated and vessel-generated waves.

3.1.1 Wind and Wind-Generated Waves

Wind records were analyzed to identify extreme events during the baseline interval because previous observations have shown that the beaches in the study area experience significant erosion and accretion during large storm events (PIE 2007a). Since 2004 there have been 26 recorded storms that exceeded the threshold wind speed of 20 kts with duration exceeding 4 hours. Table 3-1 lists these storms by date with maximum wind speed and direction and the associated return period. The forcing and beach response occurs on a seasonal basis rather than the calendar year so, the shading in the table groups the events by winter season and summer season. For the purpose of this report, winter is considered the period between October through March and summer is considered April through September. The storm analysis shows that the majority of the storms are produced during winter season by winds from the south-southwest as measured at SEA-TAC.

The fetch distances in the study area are relatively short; therefore wind-wave conditions are fairly mild for most shorelines in the study area as compared to regions of central Puget Sound where the fetches are much longer. However, numerical modeling of wind-waves has shown that the largest breaking wave heights with the potential to drive sediment transport in the study area occur along the western side of Point White and along the shores of Port Orchard. In addition, wind-driven transport in the winter was found to be greater than in the summer months and at profiles that are at the end of the longest fetches in the area (PIE 2007b). Based on previous collection of wind and wave data, a typical annual storm (return period of 0 to 1 year) with maximum wind speed of 20 kts was shown to result in significant wave heights of 0.75 to 1 ft along Point White. The waves associated with such storms are capable of mobilizing and transporting sediment on the beaches in the study area. A 1-in-100-year storm occurred in December 2006, and generated significant wave energy and a surge in alongshore transport at Site 3 on Point White during the 2006-2007 gravel tracer studies (PIE 2007a).

In general, winters with a larger number of storms will correlate with larger changes in beach volumes through both cross-shore and alongshore transport. Storms are not as common in summer months. The events in Summer 2010 and Summer 2012 both occurred on 1 April and could be categorized as winter storms as they are at the very beginning of the summer season. The storm in June 2008 was a seasonal anomaly; the storm slowed the recovery of the beaches compared to typical recoveries observed in the summer months. Further discussion on the summer seasonal beach response and anomalies is provided in Section 3.2.

Table 3-1: Discrete wind events (storms) generated by the POT analysis since 2004 and the associated return periods

Season	Date and Time (UTC)	Maximum Wind Speed (knots)	Wind Direction at Maximum (deg)	Return Period (year)
Winter 2005-2006	12/18/2005 18:00	29	100	2.5
	2/4/2006 22:00	24	220	0.7
Summer 2006	NO EVENTS			
Winter 2006-2007	11/16/2006 4:00	23	220	0.6
	12/13/2006 14:00	25	240	0.8
	12/15/2006 9:00	40	230	100
	1/2/2007 18:00	24	200	0.7
	1/6/2007 5:00	28	200	1.8
	1/9/2007 22:00	24	200	0.7
	3/12/2007 1:00	23	230	0.6
Summer 2007	NO EVENTS			
Winter 2007-2008	10/18/2007 21:00	29	200	3.2
	11/12/2007 20:00	32	200	6.7
	2/7/2008 9:00	29	200	2.5
Summer 2008	6/10/2008 3:00	26	230	1.1
Winter 2008-2009	12/12/2008 22:00	26	190	1.1
	12/29/2008 21:00	26	230	1.1
	1/6/2009 21:00	23	190	0.6
	1/7/2009 8:00	21	210	0.5
Summer 2009	NO EVENTS			
Winter 2009-2010	NO EVENTS			
Summer 2010	4/2/2010 21:00	26	200	1.1
Winter 2010-2011	11/16/2010 6:00	32	240	6.7
	12/18/2010 20:00	26	90	1.1
	3/11/2011 11:00	27	210	1.4
Summer 2011	NO EVENTS			
Winter 2011-2012	11/17/2011 17:00	22	230	0.5
	1/25/2012 16:00	26	240	1.1
Summer 2012	4/1/2012 21:00	26	220	1.1
Winter 2012	12/17/2012 22:00	29	220	2.5

3.1.2 Vessel-Generated Wake Wash

Between 2004 and 2007, the summer wave climate was shown to be dominated by vessel-generated wake wash primarily resulting from WSF car ferries and other recreational and commercial traffic sailing through Rich Passage (PIE 2007b). The vessel traffic patterns from Fall 2007 to Spring 2012 are also dominated by WSF car ferries since no other regular operations have been implemented along this route. Although the baseline time interval has been classified as a time interval when POFF vessels were not operating, a passenger only ferry (POF) service operated four times per day in Rich Passage in 2006 and 2007, slowing in the Passage to a maximum speed of less than 30 kts. As many as three other POF vessels have run one return trip per day through the Passage to Bremerton servicing the Puget Sound Naval Shipyard and the US Naval Ship *Everett* for intervals of six months during the baseline interval of Fall 2004 to Spring 2012. These vessels also operate at slow speeds through environmentally sensitive wake wash areas. In February to April 2005, the foil-assisted catamaran, *Spirit,* was operated along the Seattle-to-Bremerton route to provide data for direct validation of the numerical wake wash and shoreline response models (Osborne et al. 2009; PIE 2007b). *Spirit* was operated eight round trips per day, Monday through Friday (80 trips per week) operating at speeds between 27 and 35 kts under fully laden conditions simulated with ballast. The wake wash climate from WSF car ferries, *Spirit* and other vessels was used to calibrate wake wash models to generate a prediction of the beach response.

In the summer months, WSF wake wash energy was found to be of the same order of magnitude as the energy from wind-generated waves along Point White, Pleasant Beach, and Point Glover, but much less along Port Orchard. In winter months, wind-wave energy exceeds WSF wake wash energy along the western side of Point White and along the shores of Port Orchard (PIE 2007b). The wake wash simulations with POFF vessels showed that, overall, *Spirit* has a smaller contribution to hydrodynamic forcing of sediment transport than the POFF vessels that operated prior to the baseline interval, such as WSF POFFs in 1999-2001. In-situ measurements and numerical modeling of the wake wash generated by *Spirit* showed a minor increase to the baseline wake wash climate when a simulated *Spirit* operation of 80 trips per week is assumed (PIE 2007a, 2007b).

Numerical modeling showed the alongshore sediment transport potential is dominated by wind-wave energy and the WSF car ferry wake wash during intervals of POFF operations and non-POFF operations. Long-period wake wash generated by WSF POFF vessels tends to reach shore in a more shore-normal orientation, thereby only weakly contributing to alongshore transport (PIE 2007b). However, shorter period wake wash generated by RP1 (Golder 2013a) is less affected by refraction than wake wash from WSF POFF vessels, therefore the waves come in at an angle, which can provide a mechanism for alongshore transport (Appendix B).

Visual observations and modeling of wakes indicate that POFF wakes in particular are amplified near the shoreline to the east of Point Glover as a result of constructive wake-wake interactions on the inside corner of the vessel sailing line combined with the refractive focusing induced by the relatively shallow bathymetry of the Point. Although morphologic response to wake wash is less of a concern at this location because of the limited sediment supply, the potential for toe scour and structure overtopping should not be overlooked. There is also a marked asymmetry in the wake energy distribution between Seattle-Bremerton and Bremerton-Seattle transits. These patterns are explored in more detail in Section 4.

3.2 Morphologic Response

Beaches on East Bremerton, Point White, Pleasant Beach, Port Orchard and Point Glover exhibit some similar characteristics and patterns of behavior, as well as some important differences. The majority of the beaches, except those in Lynwood Bay, are backed by bulkheads of varying construction and location with respect to tidal water levels. The beach foreshore is generally steep with gravel overlying mixed sand and gravel that varies in thickness and grain size with increasing distance from the bulkheads.

Beaches along Point Glover, including all sites from the southwest corner of Waterman Point progressing northeast around Point Glover, are typically isolated gently sloping pocket beaches of loose sand, silt, and broken shell overlying a hard-bottomed mudstone terrace; these beaches transition abruptly to deep water. Some of the pocket beaches are sheltered from wind-waves and wakes by rock headlands, while others are exposed and more susceptible to forcing mechanisms.

In previous studies it was shown that the beach exhibited a minor to insignificant response to the in-situ testing from the research vessel *Spirit*; however testing was only conducted over a 6-week interval and occurred during winter when wind-waves might dominate transport (PIE 2007a). In general, for the baseline period of Fall 2004 to Spring 2012, the measurements indicate the beach volume change at most of sites behaves similarly in the areas above and below MTL. This means that when there was a decrease of beach material above MTL there was a corresponding decrease of beach material below MTL, and vice versa. This can be an indication that sediment transport patterns are dominated by alongshore movement driven by seasonal shifts in the relative dominance of WSF car ferry wake wash and wind-waves. Cross-shore sediment transport is interpreted as the predominant mode of beach volume change when the changes above MTL are a mirror image of the changes below MTL, such that material is being eroded from the upper beach and being deposited on the lower beach, and vice versa.

Seasonal and short-term cycles are also evident in the volume changes on the study area’s beaches. Calculations of volume change over intervals of no fast ferry operations provide a scale in which to measure the relative effects of beach response to new fast ferry operations. All sites show seasonal fluctuations in beach volume above and below the MTL of approximately +/- 1.0 to 1.5 cubic yards per foot (unit width alongshore) of beach.

The study area is divided into five major sub-sections for analysis: East Bremerton (Manette to Enetai Creek), Port Orchard (east to Waterman Point), Point Glover (Waterman Point to Manchester State Park), Point White (northeast to Lynwood Center), and Pleasant Beach (Lynwood Center to Fort Ward State Park). A composite of graphics of the most representative profiles for the shoreline sections of East Bremerton, Port Orchard, Point White, Pleasant Beach and Point Glover are provided in the following sections. Each graph of beach volume change depicts the change in the volume of beach material above and below MTL relative to the beach volume in August 2004. The blue shaded region in 2005 represents the interval of high speed trials with *Spirit*.The results of beach volume change analysis, including baseline and RP1 operations for other sites are provided in Appendix C.

The beach profiles and beach volume change analysis provide quantitative measurements of changes in the cross-shore from the bulkhead to MLLW. Similarities and differences between the trends in beach volume change on the upper beach compared to the lower beach can suggest alongshore transport patterns, but both cross-shore and alongshore transport patterns are more clearly defined by the laser scanning difference maps.

The East Bremerton shorelines exhibit seasonal fluctuations in beach volume change on an annual basis whereby the upper beach (above MTL) shows a decrease in beach volume during winter months (October through March) followed by an increase in volumes on the upper beach during the summer months (April through September) (Figure 3-1). The lower beach (below MTL) exhibits episodic changes in beach volume in response to wind-wave energy (e.g., 2007 and 2011). During the gap between 2009 and 2011 beach erosion occurred above and below MTL. However, from September 2011 through June 2012 the beach volumes have recovered, resulting in zero net loss over time. Comparison of the beach profiles from the beginning of the baseline interval (November 2004) to the end of the baseline interval (June 2012) shows there has been very little change in beach slope or the location of the slope break (transition between two distinct beach slopes) over time. The beach volume change during the testing with *Spirit* in 2005 is consistent with seasonal variability at this site.

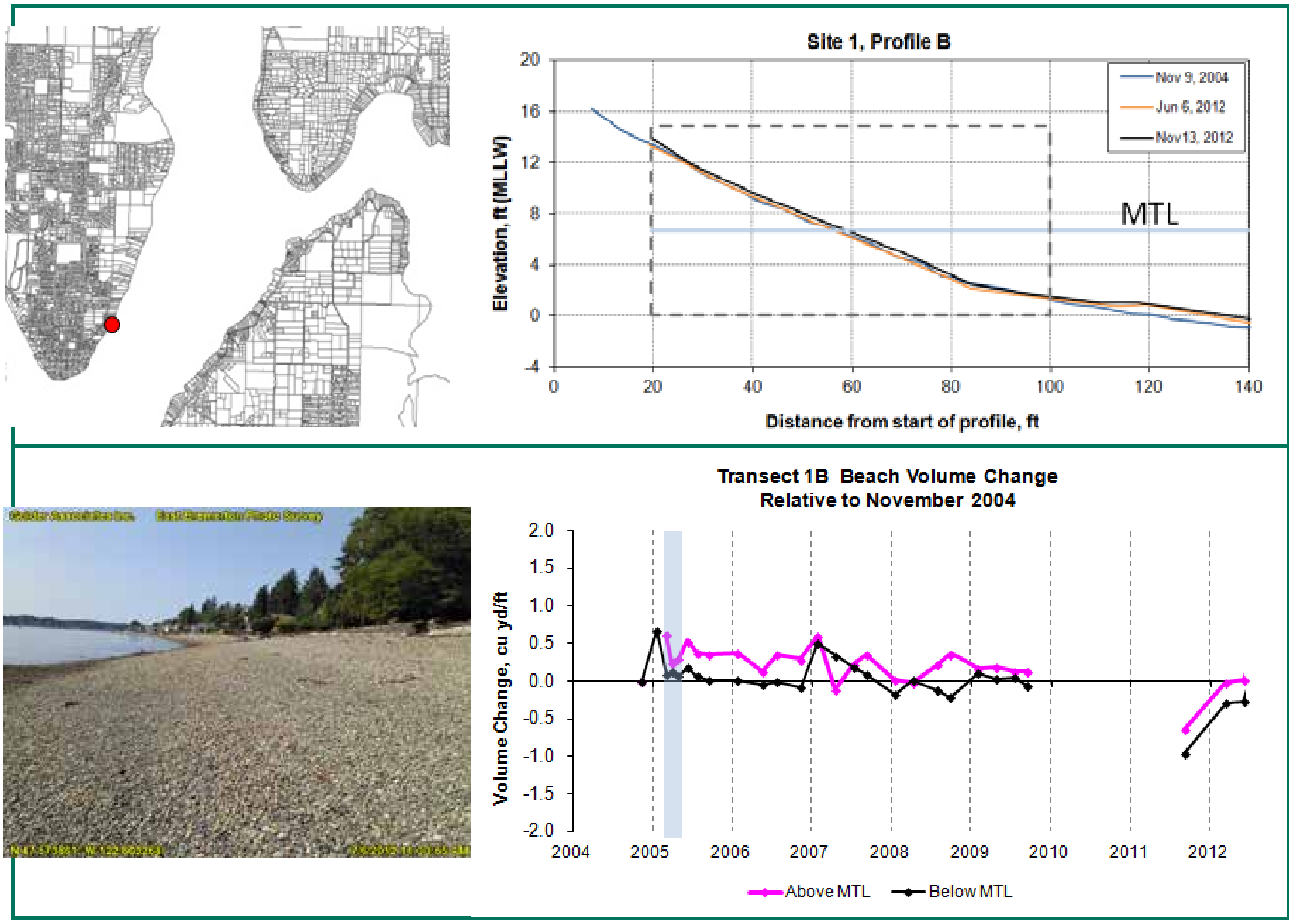

Figure 3-1: Beach volume change at Transect 1B on East Bremerton

3.2.1 Port Orchard

During most of the baseline interval, the Port Orchard beaches exhibited similar behavior to East Bremerton with seasonal fluctuations in beach volume change (Figure 3-2). A clear cross-shore transport pattern resulting in a flattening of the beach slope is shown by the beach volume change at the end of 2008 through 2009, where a loss in beach volume above MTL is reflected by a gain in beach volume below MTL. In June 2008, there was a significant wind-wave event which disrupted the normal seasonal cycle of several beaches in the project area. In addition, there were four significant wind-wave events during the Winter of 2008-2009, two of which were from the south (rather than the southwest). The anomalous cross-shore beach volume change patterns along East Bremerton can be attributed to the anomalies in the wind-wave climate in 2008 to 2009. Similar to East Bremerton, there was a decrease in beach volumes between 2009 and 2011 followed by a recovery of beach volumes in 2012 along Port Orchard. During the beach recovery in 2012, there was a steepening of the beach profile as volumes increased on the upper beach faster than on the beach below MTL. Comparison of the beach profiles from the beginning of the baseline interval (November 2004) to the end of the baseline interval (June 2012) shows there has been little to no net change in beach slope over time.

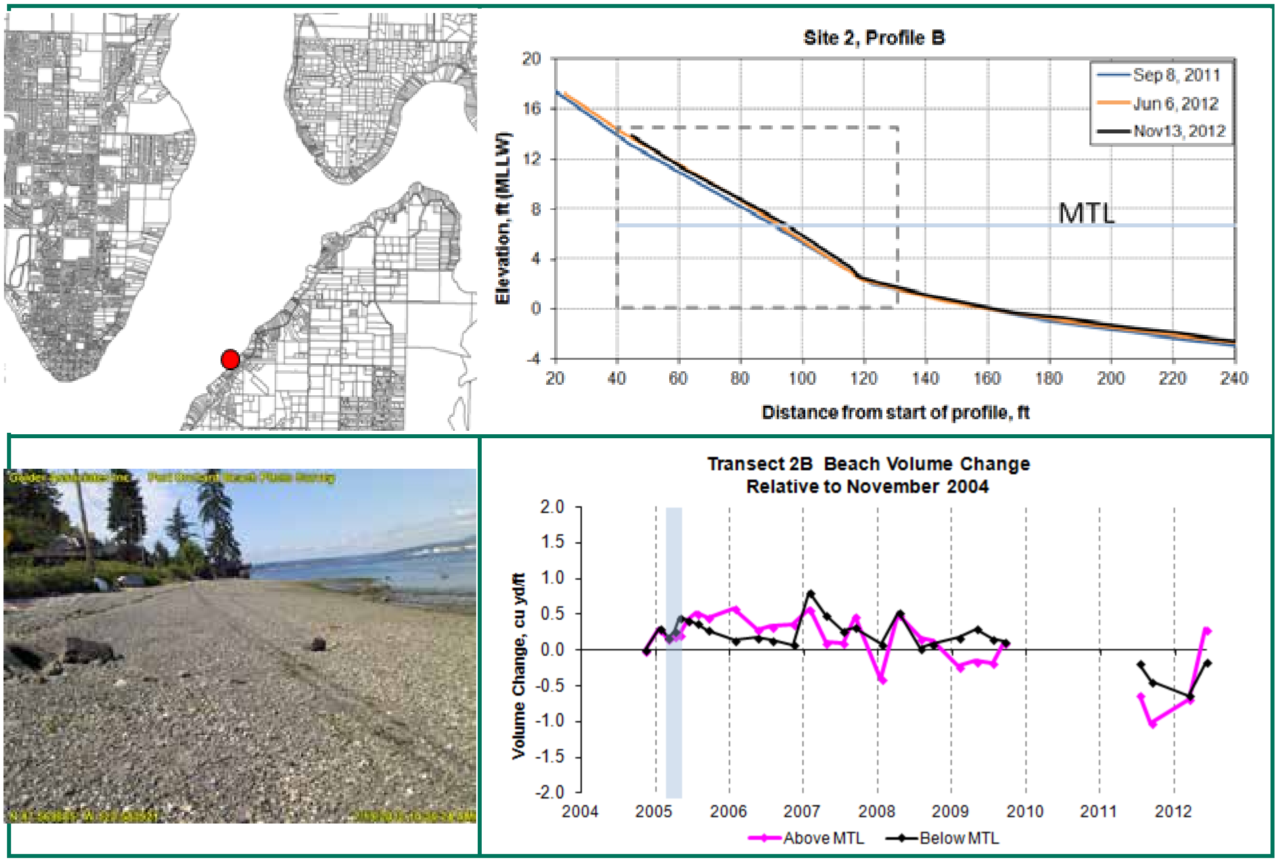

Figure 3-2: Beach volume change at Transect 2B on Port Orchard

3.2.2 Point White

Since 2004, the beaches along Point White have changed significantly. Comparison of beach profiles measured in 2004 and 2012 show the elevation near the bulkheads has decreased by 1 ft at Site 3C (Figure 3-3) and by as much as 3 ft at Site 4A (Figure 3-4). At most sites along Point White, there has been an overall decrease in elevations and steady decline in beach volumes both above and below MTL as seen in Figure 3-3. The beach volume change analysis at Site 4A shows an overall decrease in the beach volume on the upper beach, but an increase in beach volumes below MTL. At Site 4A, the elevations below MTL have increased by approximately 1 ft over most of the profile resulting in a decrease in beach slope (flattening). The increase in beach volume below MTL at this site is related to local changes in the curvature of the shoreline and adjacent properties and structures inhibiting the alongshore transport of material. Seasonal variability is characterized by beach volumes reaching an annual peak in the summer and gradually declining throughout the winter. At Site 4A, large inter-annual variations are evident during the baseline period indicated by the divergence of beach volume change patterns above and below MTL on approximately a 4-year cycle. The volume below MTL begins increasing in 2004 thru 2008, decreases significantly in 2009 and then increases again through 2012 (Figure 3-4). While the volume of material below MTL is increasing (2004-2008), the volume of material above MTL is relatively constant with seasonal fluctuations. However, the volume above MTL also decreased significantly in 2009. The data suggests that a new 4-year cycle is starting with a decline in beach volumes at the end of 2012. No significant changes occurred during trials of *Spirit* in 2005.

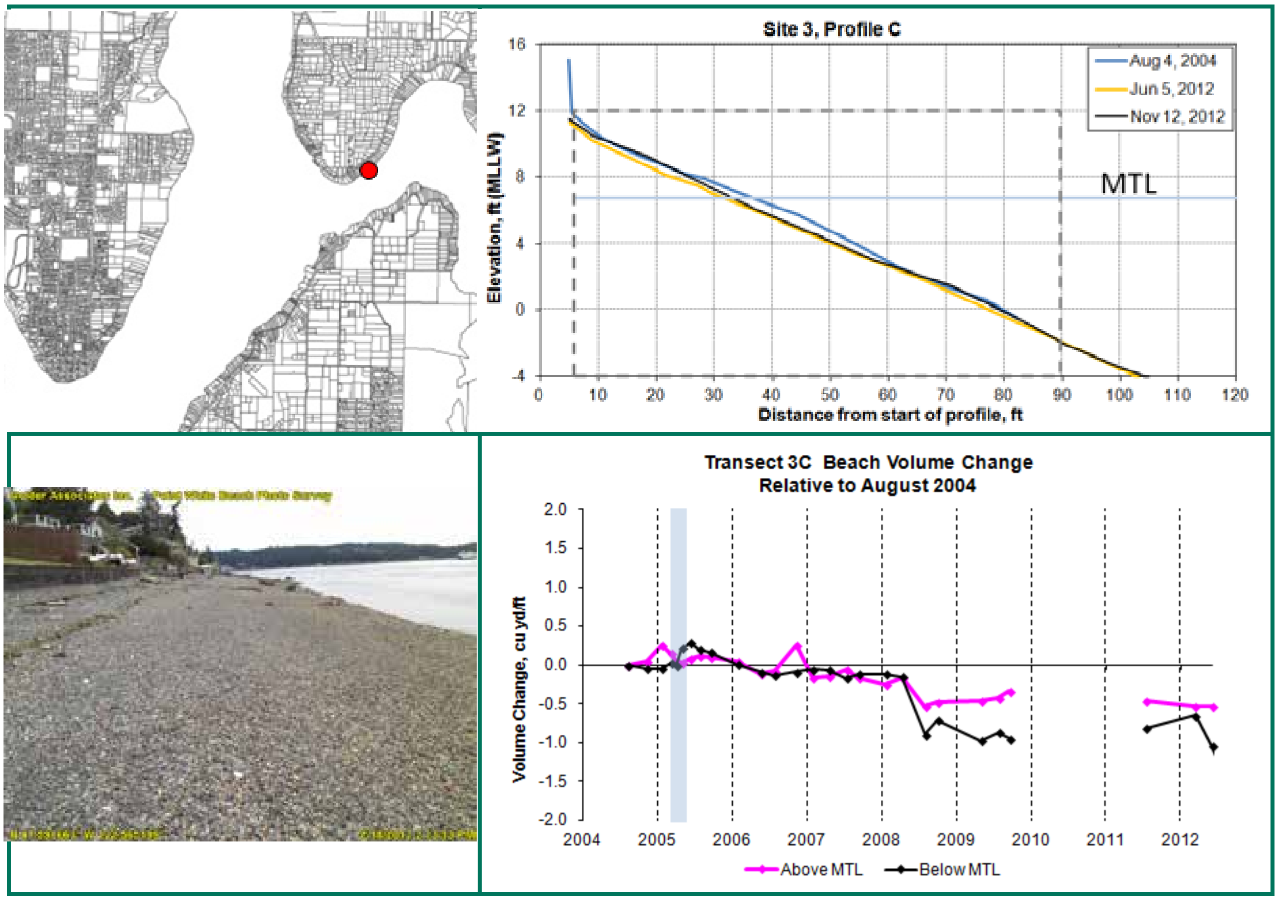

Figure 3-3: Beach volume change at Transect 3C on Point White

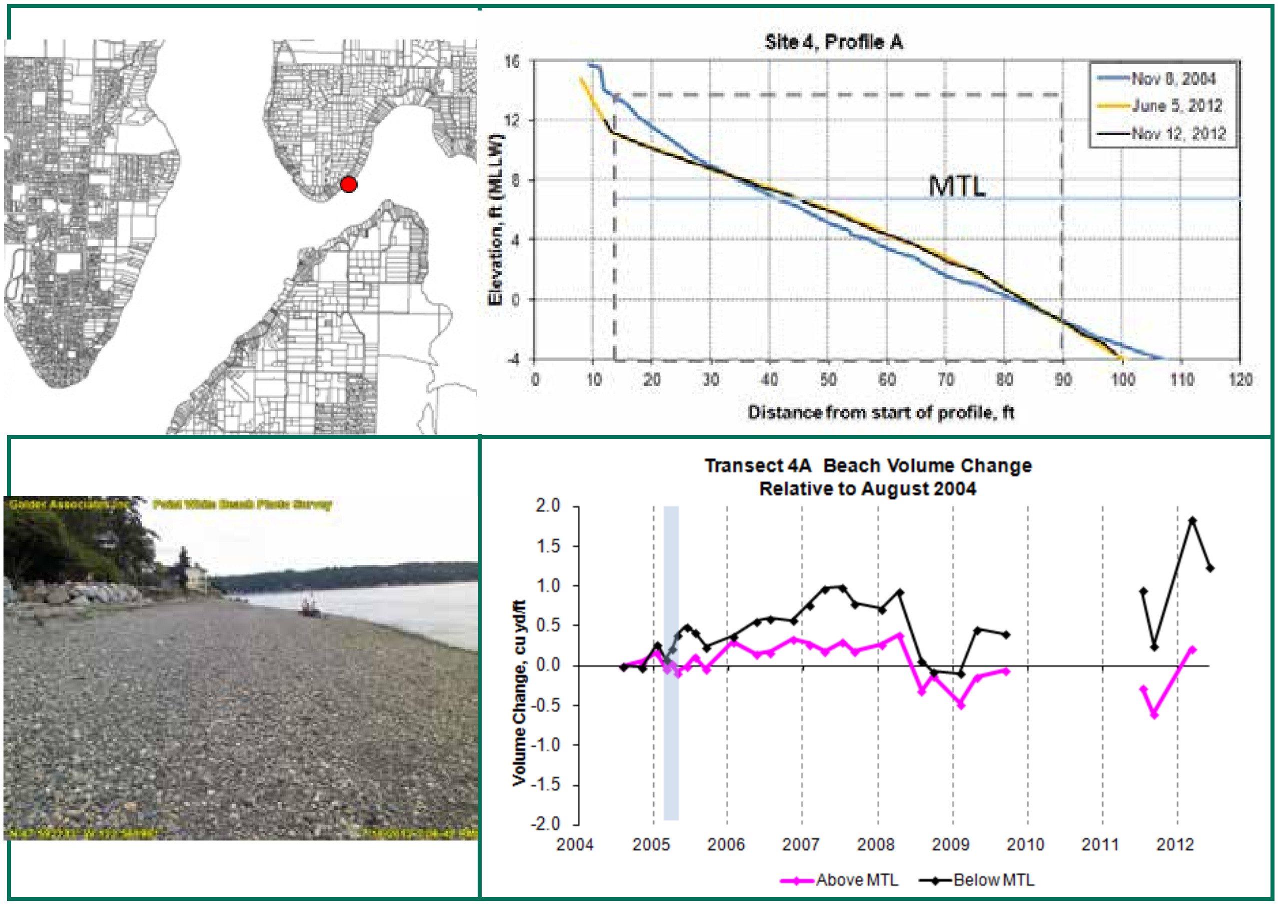

Figure 3-4: Beach volume change at Transect 4A on Point White

The long-term decline of beach volumes along Point White is further supported by the difference maps created from the laser scanning surveys measured at Sites 3 and 4 between September 2011 and early June 2012. Figure 3-5 shows there are more areas that have experienced erosion (cool colors) than accretion (warm colors) at Site 3. In addition, the alternating pattern of warm colors and cool colors indicate sediment moves alongshore in waves, particularly at the interface of the beach and the bulkhead at Site 3. The difference map for the laser scanning surveys at Site 4 further supports erosion of the upper beach and the accretion of material on the lower beach (Figure 3-6). However the difference map also shows site specific changes such as the erosion of material below MTL on the southern end of this survey area and a slight accretion of material above MTL on the northern end. Beach dynamics at Site 4 are also significantly influenced by runoff from a creek that discharges onto the beach immediately south of the Site 4A profile. A small creek-mouth delta contributes a supply of sediment to the beach; this sediment is gradually re-distributed both across the beach and alongshore to the north.

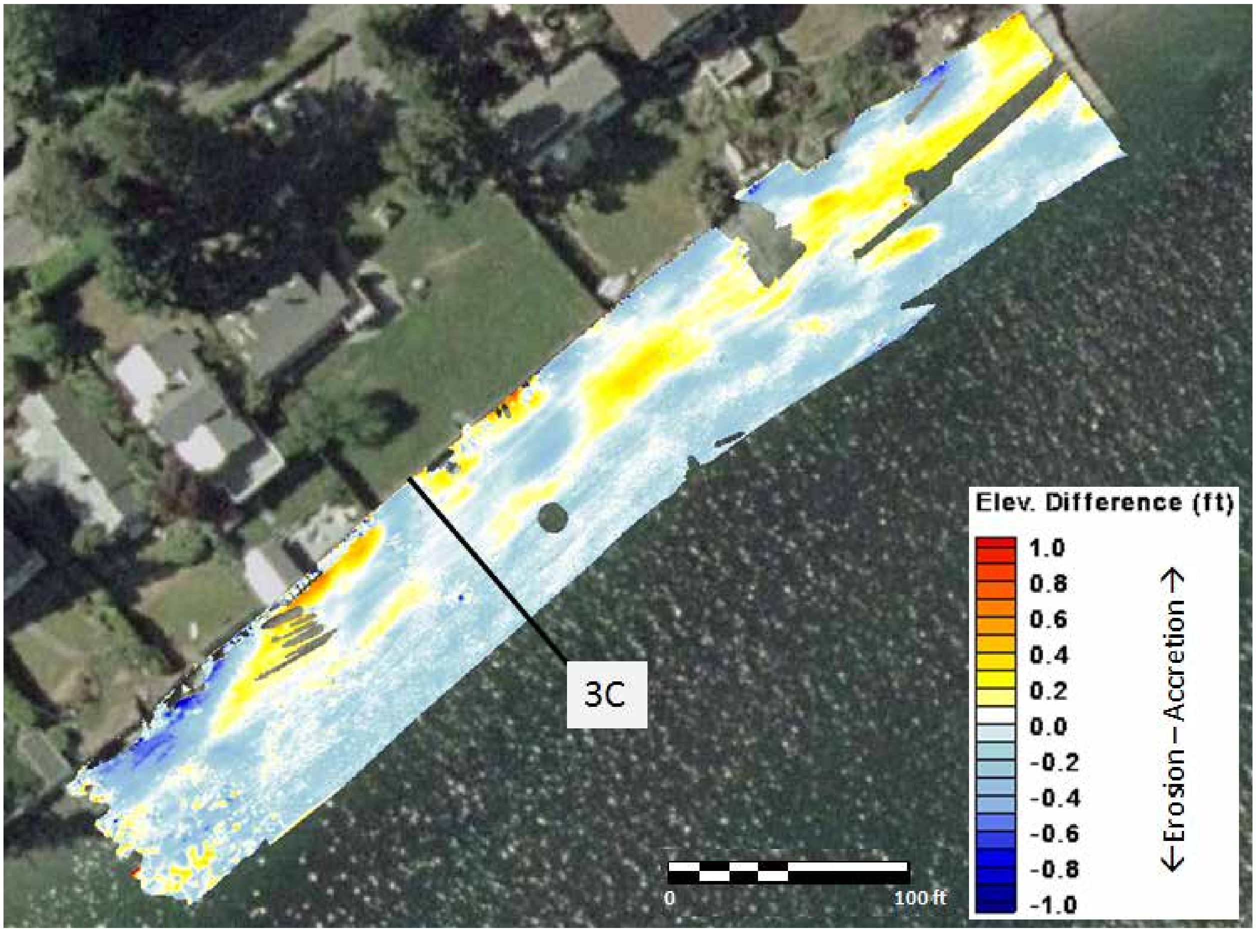

Figure 3-5: Difference map of laser scanning surveys measured at Site 3 on Point White between 9 September 2011 and 4 June 2012

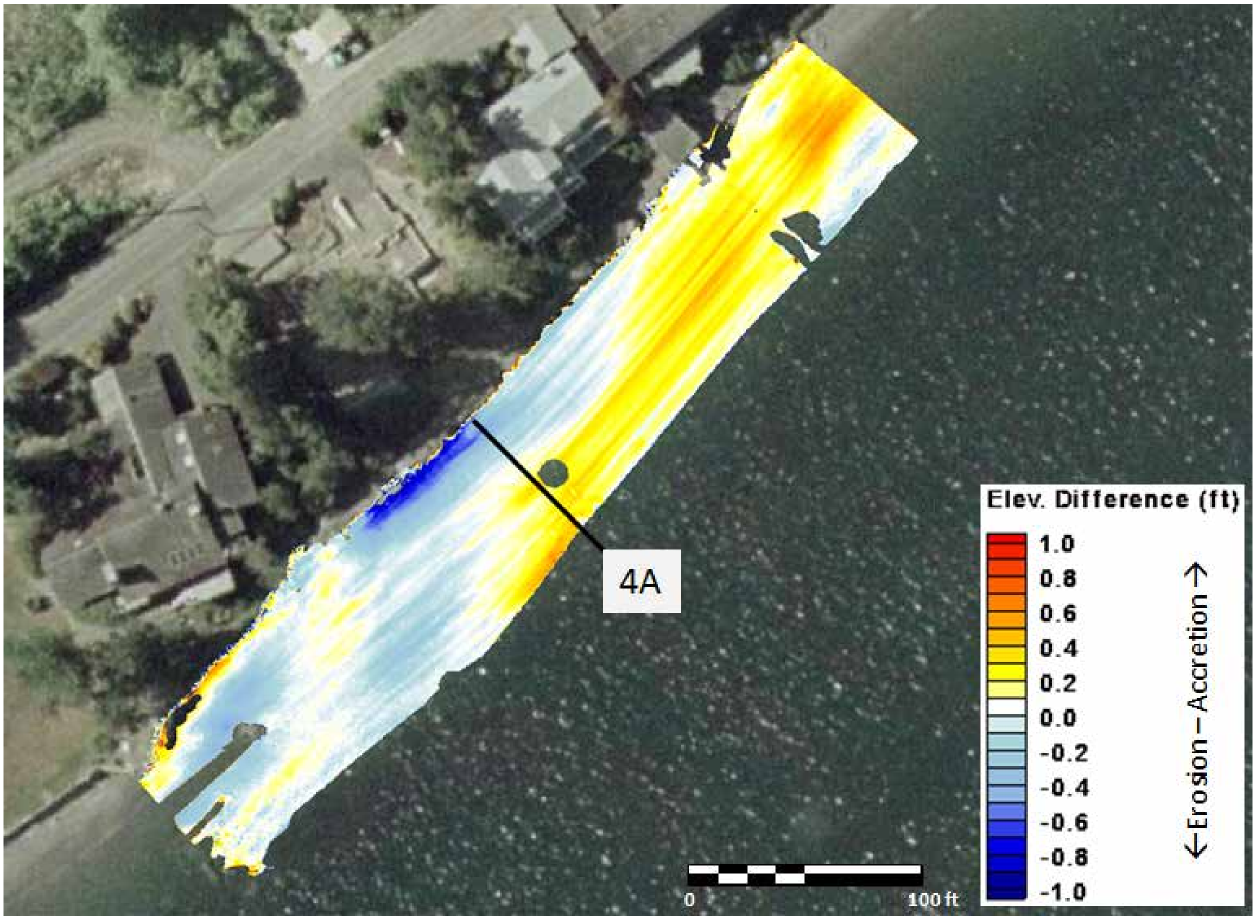

Figure 3-6: Difference map of laser scanning surveys measured at Site 4 on Point White between 9 September 2011 and 4 June 2012

Gravel tracer data collected in 2006 and 2007 were analyzed to determine the magnitude of alongshore and cross-shore transport at Sites 3 and 4 on Point White (Curtiss et al. 2009). During the 2007 study, the most noticeable feature is the amplified alongshore movement to the northeast at both sites. The alongshore movement is greater at Site 3 than at Site 4.

Tracers at Site 3 move alongshore at a rate of approximately 0.065 m/day between April and November, while at Site 4 the rate of movement alongshore is only 0.005 m/day in the same interval. The daily transport rate at Site 3 increases by a factor of 6 in the period between November and January. At Site 4 the alongshore transport rate increases by a factor of 90 during the month of December. The transport observed during December is largely a result of a 64-year storm that occurred in December 2006. The magnitude of alongshore movement in December is similar at both sites. However, the magnitude of alongshore movement during the non-storm intervals is higher at Site 3 because Site 3 is more exposed to wakes, wind-waves, and tidal currents than Site 4.

The cross-shore movement of the tracers is a small fraction of alongshore movement. The tracer movement at both Site 3 and Site 4 is slightly offshore from January 2007 to April 2007 and then changes to slightly onshore from April 2007 through October 2007. At Site 4 there is a trend of offshore movement of the tracers from February 2007 to April 2007 and a gradual return onshore of the smallest tracers in the following months.

The distribution of different sized tracer particles on the beach varied during the study, between a well-mixed state where all particle sizes are observed equally on the upper and lower beach, and a well-sorted state, where larger (smaller) particles tend to accumulate on the lower (upper) beach. Two events were observed that caused the particles to become better sorted. First, initial sorting at both sites occurred within one to two weeks of tracer deployment. The largest particles, 45 mm, at mid-beach at Site 4 were larger than the sediment distribution at the deployment location and were quickly transported to lower elevations. The tendency for coarser particles to accumulate at lower elevations reflects a low energy regime in which the onshore transport due to waves is not strong enough to overcome the downslope component of gravity for sediment particles of a given size. In the second instance, the tracers at both sites became well-mixed across the beach from January 2007 to April 2007 during the storm interval, reflecting a significant increase in wave energy relative to gravitational effects. The tracers at Site 3 re-sorted into the reverse-graded pattern during the following non-storm interval. Site 4 remained poorly sorted (in a spatial sense) with the exception of the largest tracers which remained concentrated on the lower beach.

Cross-shore variations in the alongshore transport patterns are also observed in the tracer patterns. The tracers highest on the beach were transported farther than tracers lower on the beach. The shape of the resulting distribution of tracers was elliptical, with the longest axis in the direction of alongshore movement. At Site 4, between 10 and 15 of the tracers moved alongshore in a gravel berm in the upper foreshore where the beach and revetment meet. Larger and more immobile tracers were located lower on the beach. The tendency for greater alongshore transport at higher beach elevations reflects the higher percentage of time that the upper beach is exposed to breaking waves relative to lower tide elevations because of the diurnal inequality in the tides in Puget Sound.

A comparison of the daily alongshore transport rate for each survey interval in 2006 to 2007 revealed a trend in transport to the southwest from May to September. This short-term reversal of transport in the alongshore direction indicated a change in the relative influence of forcing mechanisms at the site. The reversal was most likely caused by the dominance of the flood tidal currents that flow to the southwest during an interval without significant wind-wave activity from the southwest which drives northeast alongshore transport.

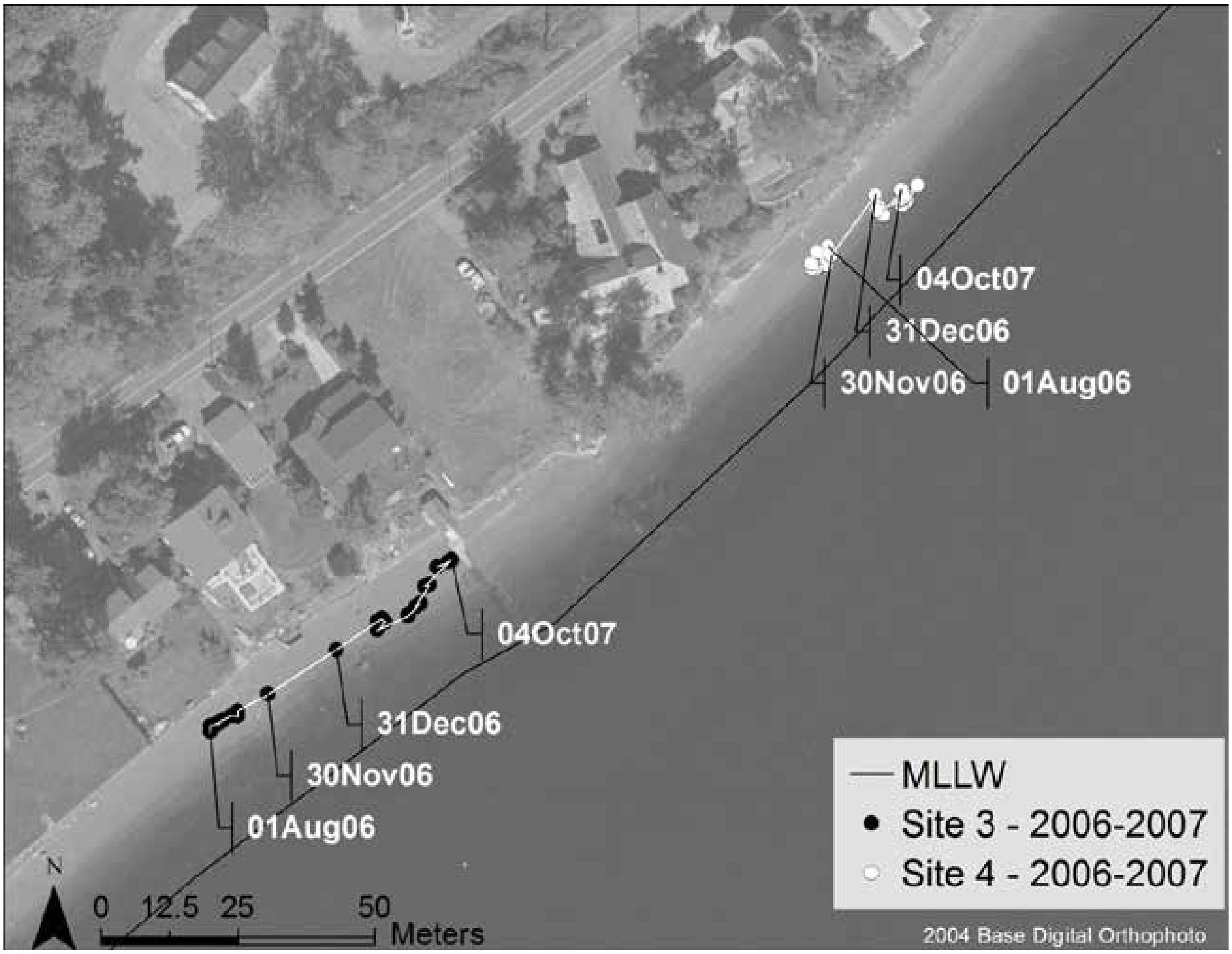

Figure 3-7: Location of tracer centroids for the 2006-2007 survey at sites 3 and 4

3.2.3 Pleasant Beach

Pleasant Beach exhibits a long-term decrease in sediment volumes since May 2006 similar to the decrease in beach volumes observed along Point White (Figure 3-8). Between April 2008 and February 2009, five storm events contributed to the significant decline in beach volumes across the entire profile. Beach volumes have partially recovered since early 2009, and beach volumes have been relatively constant from Summer 2009 to Summer 2012 with small seasonal fluctuations. Although beach profile measurements at this location began in January 2006, adjacent sites on Pleasant Beach have been measured since November 2004 and showed no response to *Spirit* test operations.

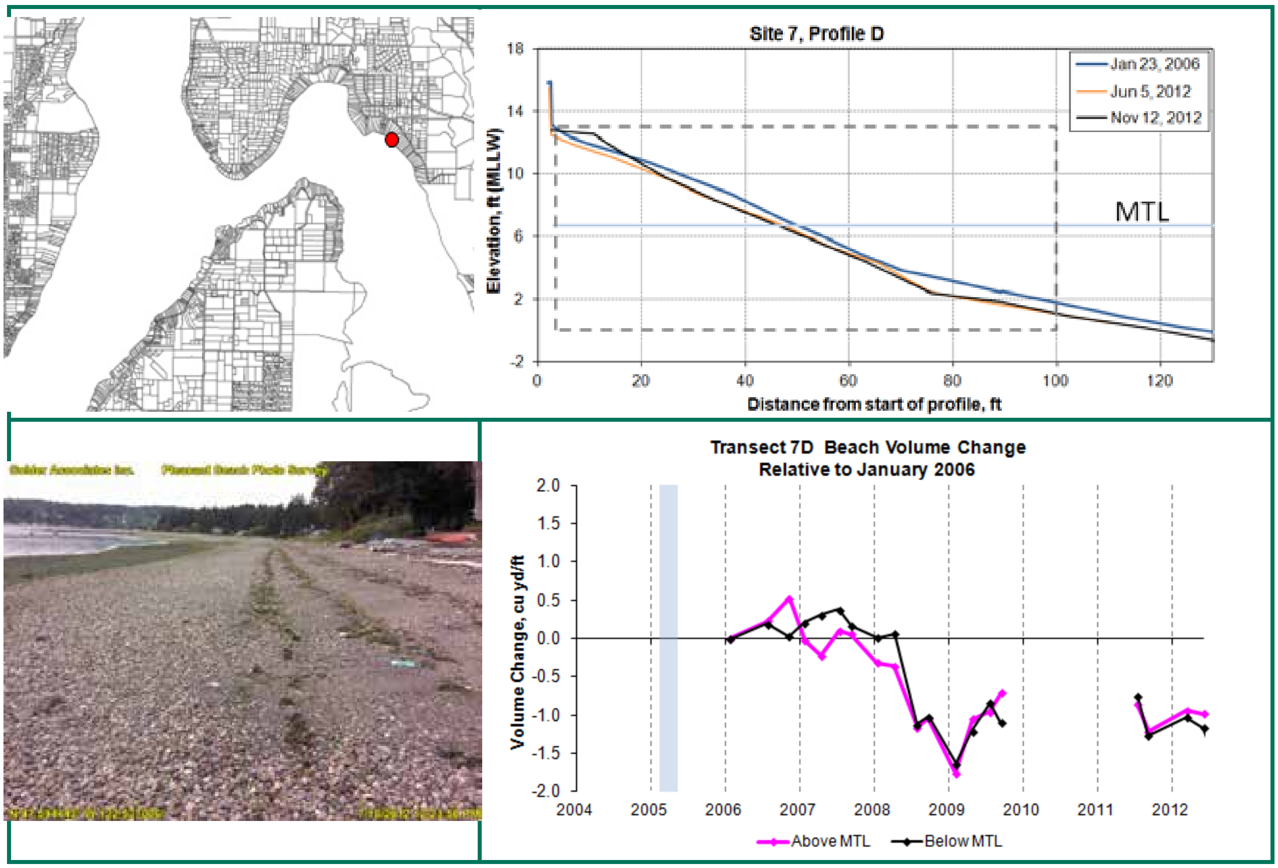

Figure 3-8: Beach volume change at Transect 7D on Pleasant Beach

3.2.4 Point Glover

This transect is in the middle of a pocket beach along Point Glover. This beach is consistent with the majority of other pocket beaches along the Point: it has a thin layer of loose sand, silt, and broken shell overlying a hard-bottom mudstone terrace, and the beach transitions abruptly to deep water. Although beach volumes at this site show a slight decline over the baseline interval with the largest changes occurring between 2008 and 2009, there is very little sediment to measure at this site (Figure 3-9). This site also has a system of cross-shore groins which limit the alongshore transport of the beach sediment. Plan views showing elevation differences from laser scanning surveys at Site 9 show the changes observed on the upper beach are due to the alongshore movement of sediment within the two groins (Figure 3-10). The alternating patterns of cool colors and warm colors in the laser scanning difference map indicates northerly alongshore transport occurred in Winter 2011-2012 and the extent of the sediment transport is somewhat, but not entirely limited by the riprap groins. Additional discussion of the results from the laser scanning surveys measured at this site is provided in Section 4 and additional graphics are shown in Appendix E. Site 9D was first surveyed in January 2006, after the trials of *Spirit*.

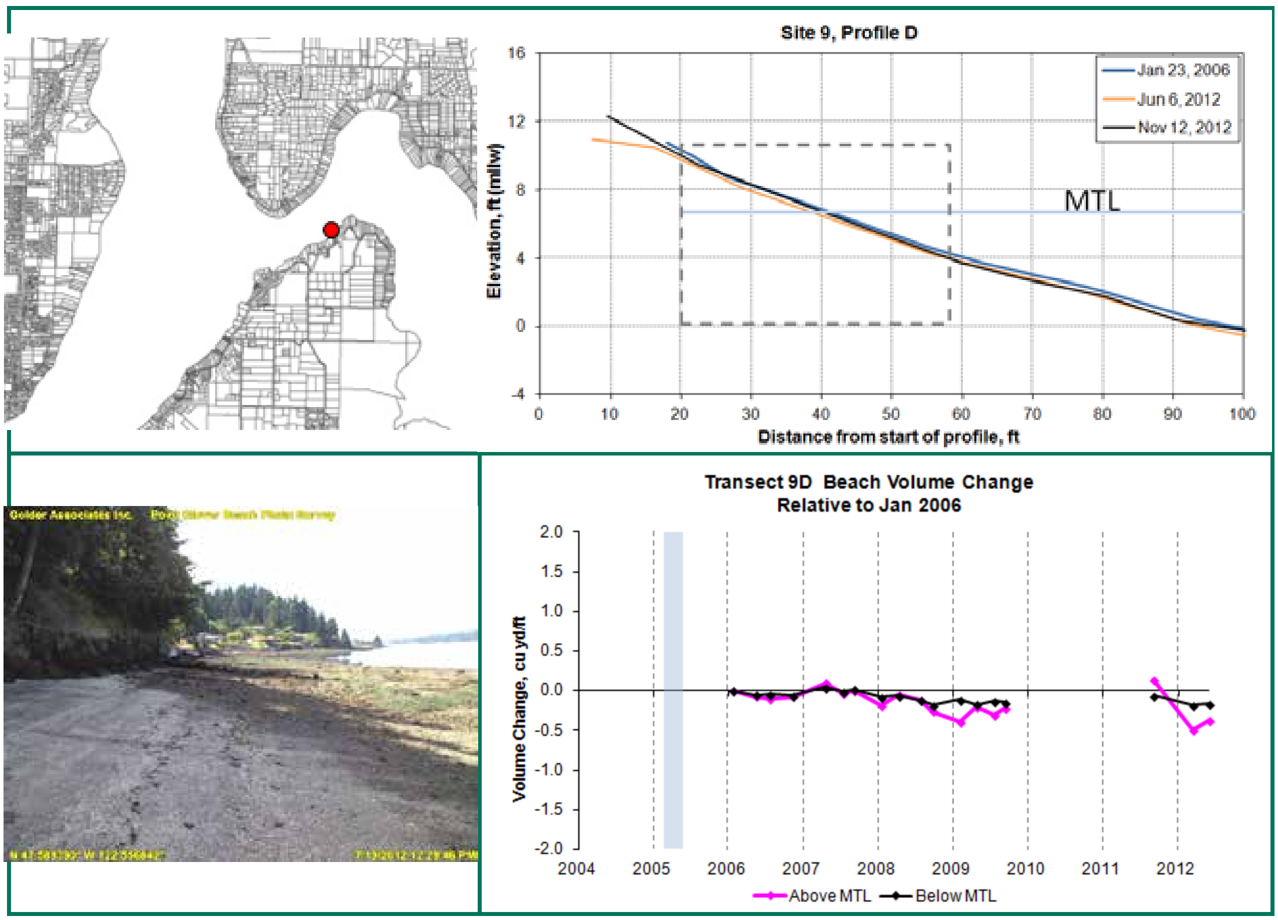

Figure 3-9: Beach volume change at Transect 9D on Point Glover

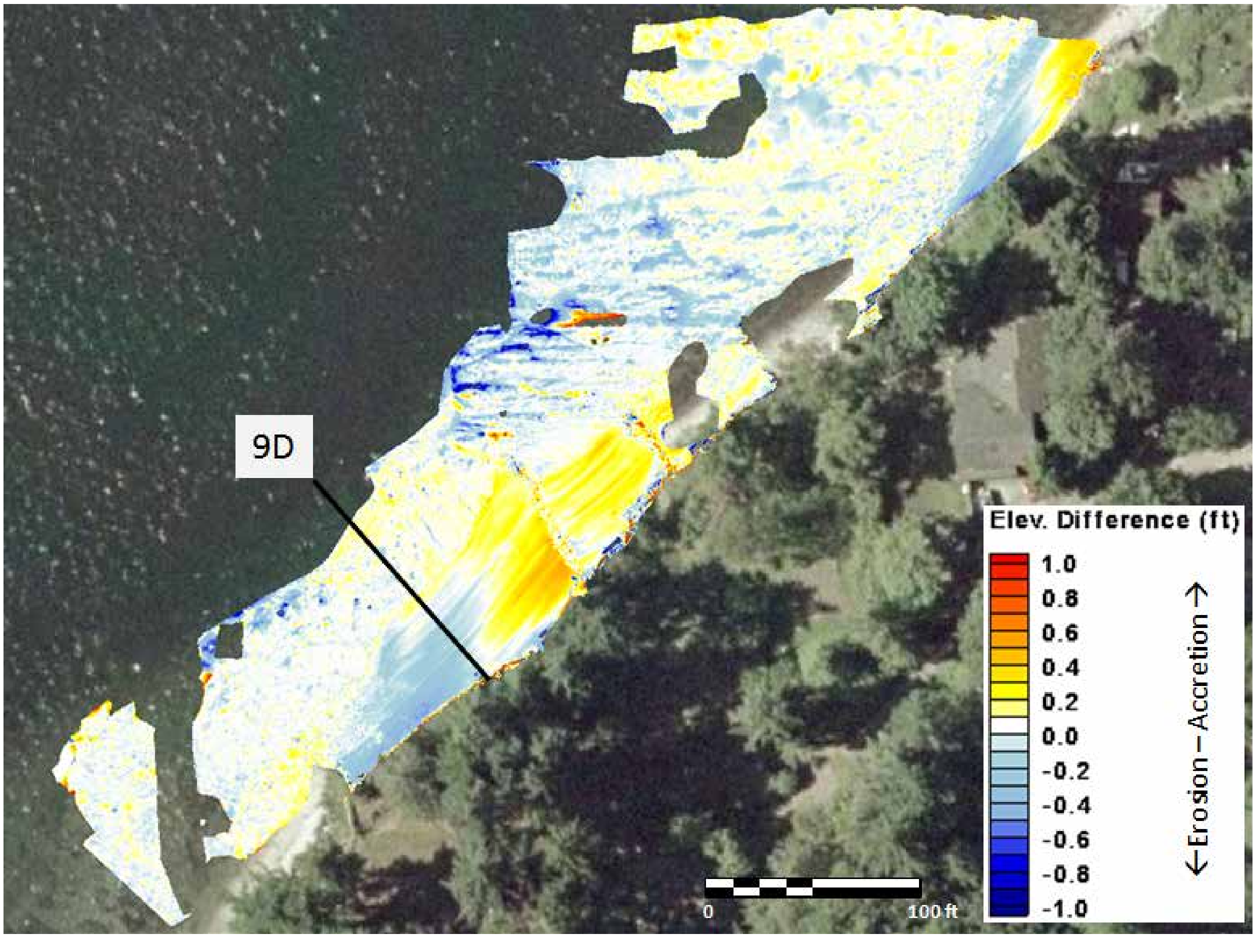

Figure 3-10: Difference map of laser scanning surveys measured at Site 9 on Point Glover between 8 September 2011 and 5 June 2012

3.3 Structure Response

The objective of the structure response survey and inventory was to provide data for comparison with an earlier survey in 2006 (PIE 2007a) in order to note any significant or substantial changes in the condition of bulkheads. Although several properties have had upgrades (or complete reconstruction) of shore protection structures in the intervening years, there was no evidence of new damage or any significant progression of damage in 2012.

The structure inventory and regular beach photo observations provide an opportunity for systematic observation and development of a photographic record that can indicate the scale of beach change at the toe of bulkheads in the study area. Figure 3-11 contains an example of a photo time series with relative beach elevations derived from the photographs of reference location EB_16 on the East Bremerton shoreline. The time series suggests that seasonal fluctuations of +/- 0.5 ft in the beach elevation occur at this location, with beach elevations being relatively lower in late winter and spring and higher in late summer and fall. Beach elevations also fluctuate by +/- 1.5 ft on a two-year cycle. The sequence of photographs illustrates seasonal changes in beach gravel cover that accompany the fluctuations in beach elevation. Lower elevations and flatter slopes are associated with a sandy upper foreshore while steeper slopes and higher relative beach elevations are associated with gravel on the upper foreshore. Beach elevation changes for all beach photo monitoring sites are included in Appendix F.

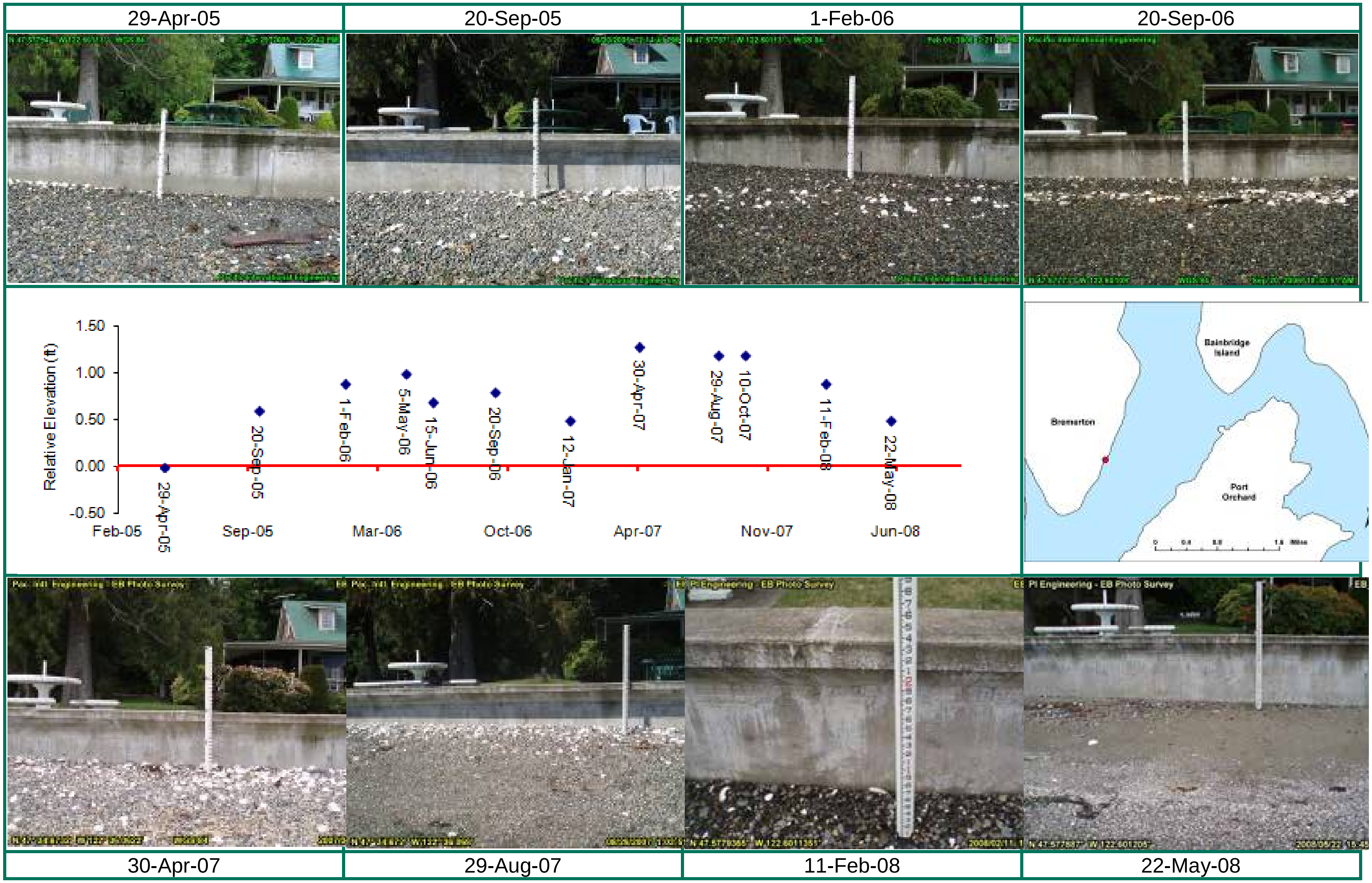

Figure 3-11: Photo time series and relative beach elevation change at Site EB_16 on East Bremerton between 2005 and 2008

3.4 Noise

The Washington Administrative Code (WAC) 173-60-040 sets maximum permissible environmental noise levels. The maximum permissible noise level for Class A property (defined as lands where human beings reside and sleep) is a decibel limit of 55 dB between the hours of 7:00AM and 10:00PM and 45 dB between the hours of 10:00PM and 7:00AM. In addition, these thresholds may not be exceeded by more than:

- 5 dBA for a total of 15 minutes in any one-hour period
- 10 dBA for a total of 5 minutes in any one-hour period
- 15 dBA for a total of 1.5 minutes in any one-hour period

All of the measurements of vessels in Rich Passage resulted in noise levels exceeding the 55-dB threshold, but were within the allowable limits of exceedence for the duration of the noise (Table 3-2). Measurements of ambient conditions did not exceed the 55-dB threshold. For example, the Bremerton-bound Walla Walla was measured at Point White and exceeded the 55-dB threshold by 5.7 dB, but only for 84 seconds (s) or 48.6% of the measurement, and the maximum noise level was 60.7 dB. Figure 3-12 shows time series of noise measured near Site 8 (Point Glover) for various vessels and ambient conditions on 26 June 2012.

The maximum noise level was measured from the Bremerton-bound Kaleetan at Point Glover. The measurement exceeded the 55-dB threshold by 17.5 dB for 54 seconds, which is still under the maximum permissible limits.

Table 3-2: Summary of noise level measurements in Rich Passage; duration of the measurement at various decibel levels above the 55-dB threshold

Site	Vessel	Length of Observations (obs) (s)	Maximum Noise Level (dB)	Duration of obs > 55 dB (s)	Duration of obs >60 dB (s)	Duration of obs >65 dB (s)	Duration of obs >70 dB (s)
Point Glover	Ambient 1	51	52	0	0	0	0
Point White	Ambient 2	61	47.5	0	0	0	0
Point White	Ambient 3	67	53.5	0	0	0	0
Point Glover	Small Fishing Boat	306	65.7	139	26	2	0
Point Glover	Recreational Motor Yacht	175	64.8	112	47	0	0
Point Glover	*Kaleetan* BRE-SEA	392	72.5	213	102	30	0
Point White	*Kaleetan* SEA-BRE	187	58.5	82	0	0	0
Point White	*Walla Walla* BRE-SEA	218	60.7	106	5	0	0

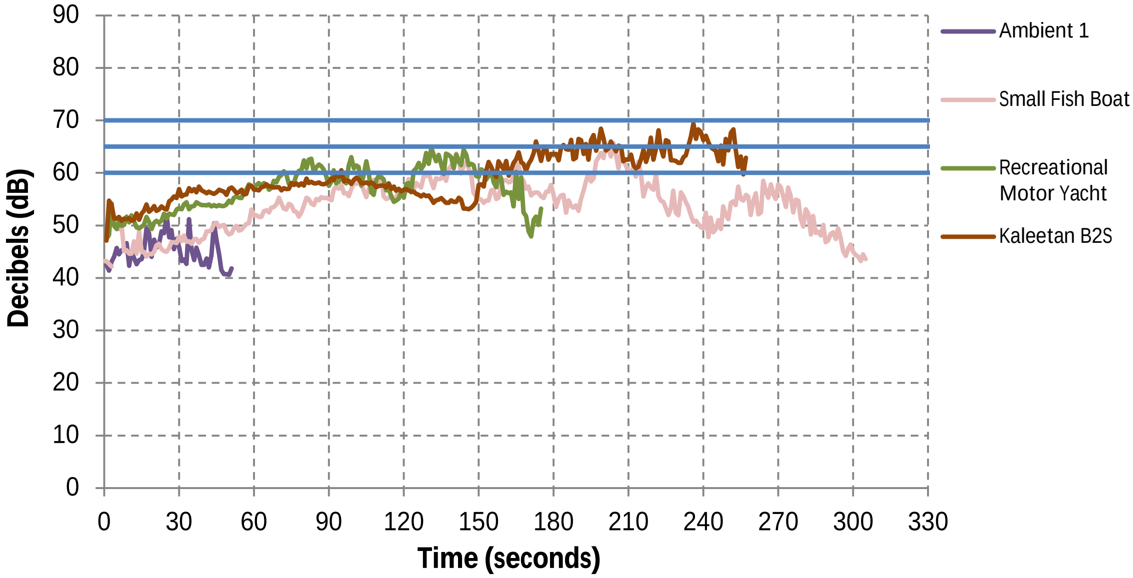

Figure 3-12: Measurements of noise levels near Site 8 (Point Glover) for various vessels and ambient conditions in Rich Passage on 26 June 2012

4.0 RP1 OPERATIONS

This section summarizes the analysis of wind- and vessel-generated wake wash measured during the operation of RP1. As described in Section 2.1.2, RP1 operated from 25 June to 2 November 2012 carrying passengers to simulate a commercial-scale ferry service. From 25 June to 4 September 2012, RP1 operated 40 trips per week which was increased to 60 trips per week weeks from 4 September to 2 November 2012. During the RP1 operations interval, other vessel traffic, wind-wave energy, and tidal currents were also acting on the beaches. The baseline hydrodynamic forcing and beach response to vessel traffic and typical wind-wave climates was discussed in Section 3.0. This section focuses on the additional hydrodynamic forcing from RP1 operations and the morphologic and shoreline response to the total hydrodynamic forcing including wake wash from existing vessels, RP1 operations, wind-waves, and tidal currents.

4.1 Hydrodynamic Forcing

4.1.1 Wind

Winds were measured from 22 August 2012 to 10 December 2012 at Point White. Figure 4-1 shows a time series of wind speed and direction. The time series of wind measured at Bremerton airport is provided in Figure 4-2 to provide wind data for the earlier interval of RP1 testing not covered by the wind record at Point White (approximately 1 July through 22 August 2012). The measurements show a shift in wind direction from the northeast to south-southwest on approximately 11 October 2012. After this date there is marked increase in the number of wind speeds observations higher than 20 kts; there are 17 observations greater than 20 kts after 11 October 2012 compared to only 1 before that date. There were no events that exceeded 20 kts sustained for at least 4 hours, which would qualify as an extreme event. The shift in wind speed and direction reflects the annual seasonal shift from summer-time dominated by northeast winds to winter season dominated by southwest winds.

Figures 4-3 to 4-5 show wind rose diagrams for Point White, SEA-TAC, and Bremerton Airports. The wind roses are constructed for the time periods before and after 11 October 2012 for which measurements were recorded at Point White. The roses show the difference between the typical summer wind conditions dominated by winds out of the northeast and winter (south-southwest) wind conditions. The shift in wind direction is measured at all three stations.

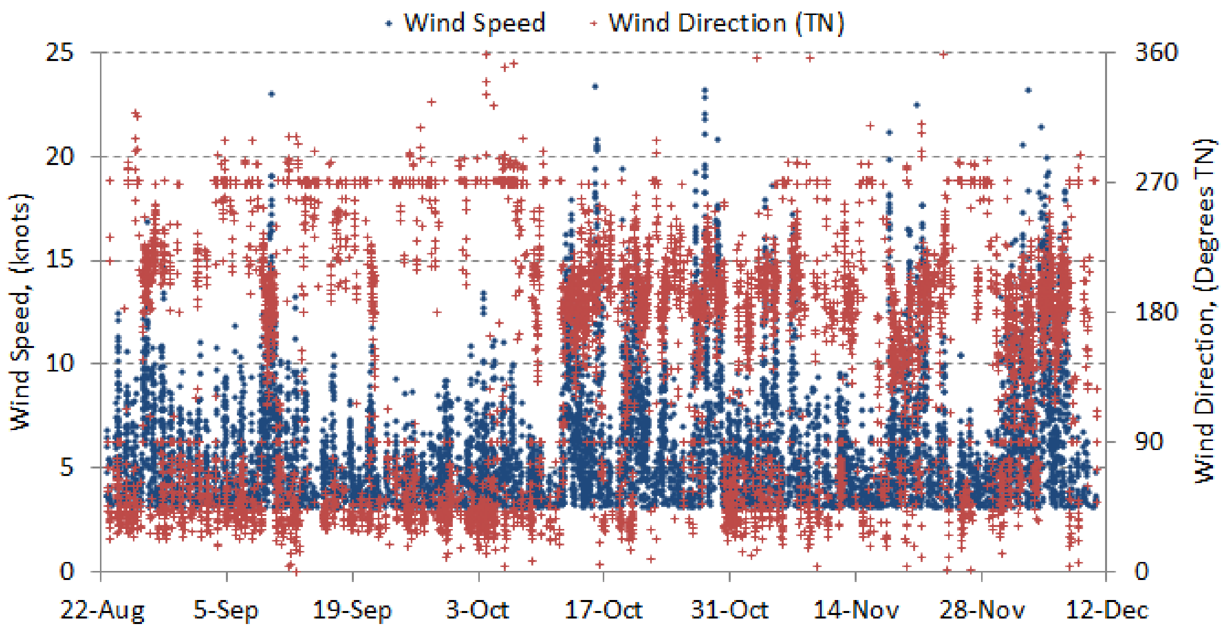

Figure 4-1: Time series of wind speed and direction at the Point White Navigation Aid Wind station

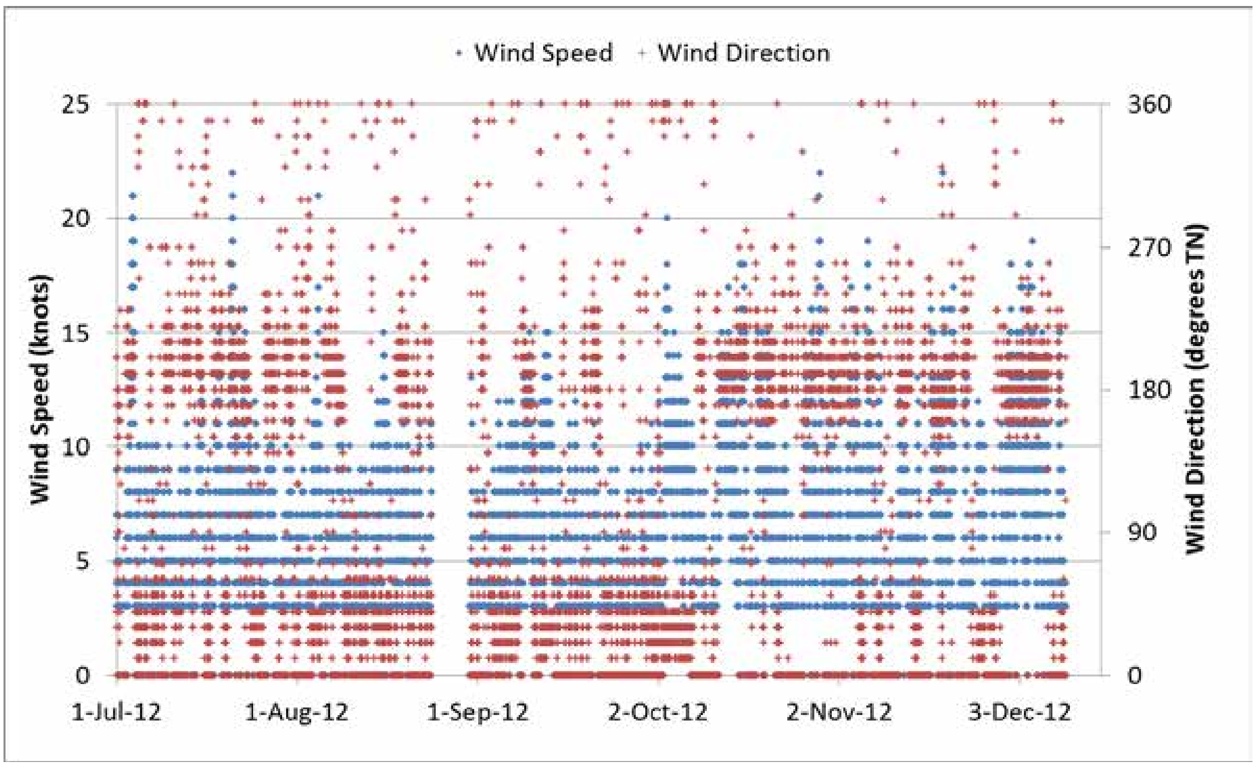

Figure 4-2: Time series of wind speed and direction at the Bremerton Airport

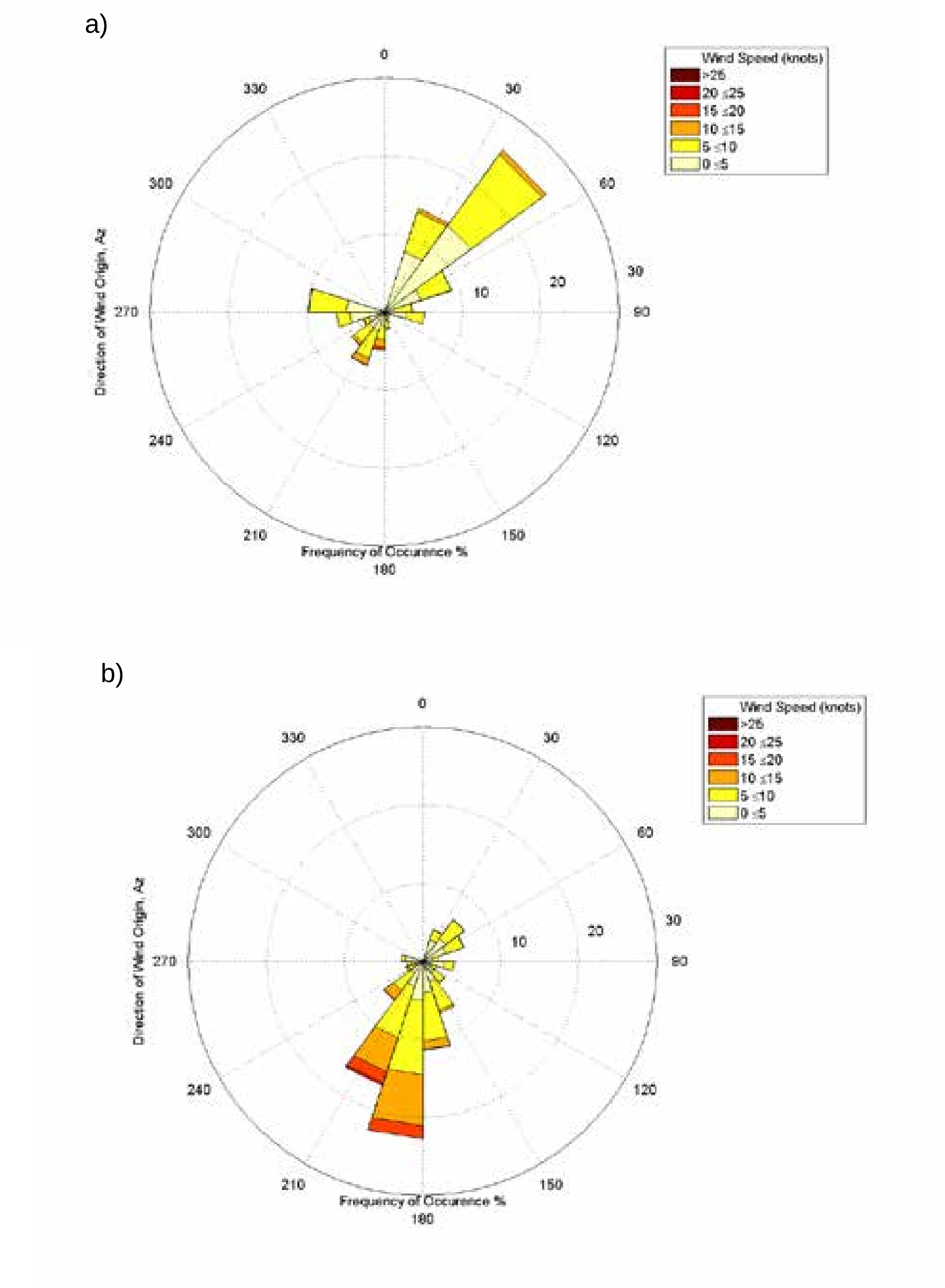

Figure 4-3: Wind roses for the Point White wind station for two periods, a) 22 August 2012 to 11 October 2012, and b) 12 October 2012 to 10 December 2012

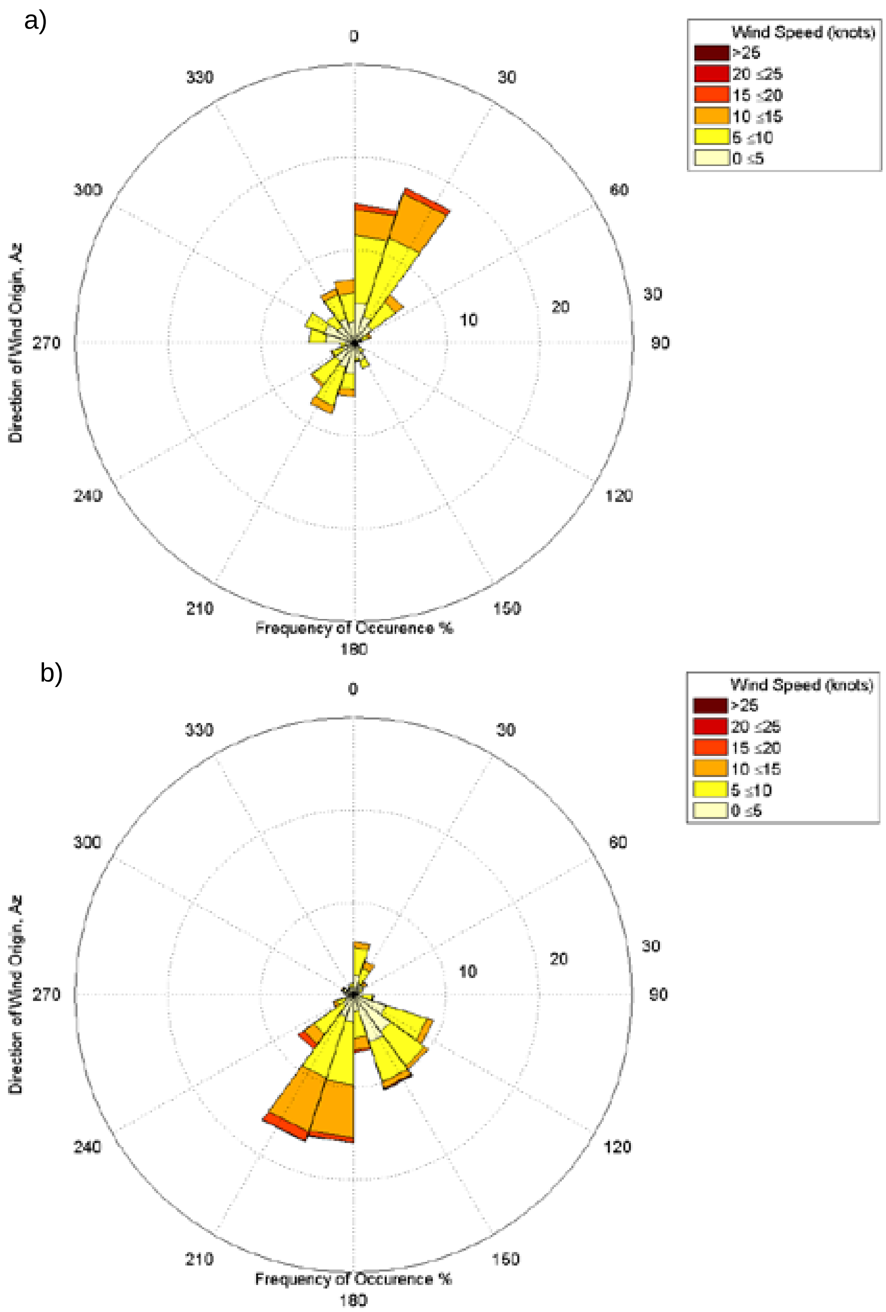

Figure 4-4: Wind roses for SEA-TAC for two periods, a) 22 August 2012 to 11 October 2012, and b) 12 October 2012 to 10 December 2012

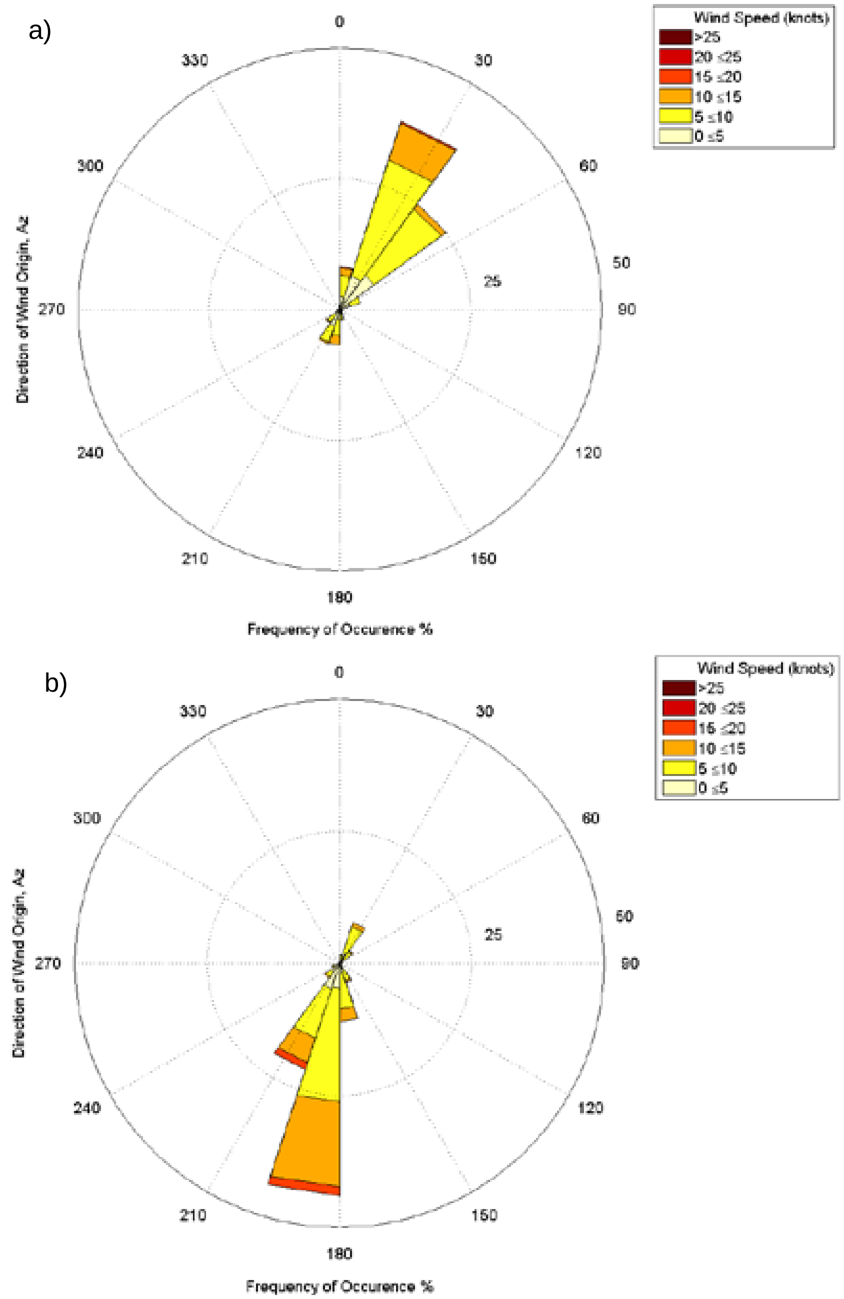

Figure 4-5: Wind roses for the Bremerton for two periods, a) 22 August 2012 to 11 October 2012, and b) 12 October 2012 to 10 December 2012

4.1.2 Wake Wash Generated by RP1

The LSV wake model (MacDonald 2005) was calibrated for RP1 to compare the wake-wash from RP1 to the wake-wash from other vessels transiting in Rich Passage and candidate vessels for fast ferry service. A detailed report on the the wake wash modelling for RP1 is included in Appendix B, and can be summarized as follows:

- The wave height and wave energy generated by RP1 is significantly less than the wake wash generated by previous POFF, M/V Snohomish and measurably less than the previous candidate vessel, *Spirit*, and approxaimtely equal to WSF car ferry wake wash.
- The breaking waves from Snohomish are concentrated around a wave height of 0.6 m and wave period of 10 s; whereas wave breaking from RP1 is concentrated around wave height and period of 0.2 m and 5 s respectively.
- The shorter smaller waves of RP1 are much less energetic than the larger longer waves of Snohomish and less likely to increase cross-shore sediment transport.
- Long-period wake wash generated by Snohomish tends to reach shore in a more shore-normal orientation, thereby only weakly contributing to alongshore transport (PIE 2007b). However, shorter period wake wash generated by RP1 (Golder 2013a) is less affected by refraction than wake wash from WSF POFF vessels, therefore the waves come in at an angle, which can provide a mechanism for alongshore transport.

The numerical modeling tools provide controlled scenarios to compare the wake wash from one vessel to another and to provide indicators of locations in Rich Passage that might be more susceptible to sediment transport from vessel wakes than other. The wake wash model validation is conducted with controlled measurements of vessel generated wave parameters in deep water. The results of the wake wash modeling are difficult to compare directly to measurements of wake wash along the shorelines of Rich Passage due to the local variability of tidal currents, water levels, other vessel generated waves, local topography and wind-generated waves. However, the spatial variability in wave energy and patterns in sediment transport potential predicted by the integrated modeling of RP1 is comparable to the measurements of wake wash and beach response discussed in the following sections.

The wake wash generated from RP1 was measured 72% of the time on average after screening the data for invalid wakes as described in Section 2.1.3. Table 4-1 provides aggregate statistics for all valid wakes during each wave gauge deployment. The statistics provided are mean values of maximum wave height ($H_{max)}$, wave period at H_{max} (T@H_{max}), wave power for the wake wash, and sailing distance to the wave gauge. Vessel information was not necessarily available for every valid wake measurement as a result of problems with the HFMS. However, average vessel speed is calculated from the runs where vessel data was recorded. Vessel position along the route was also recorded by the HFMS.

Table 4-2 provides aggregate statistics for all valid wakes during each wave gauge deployment. The data have been sorted by direction of vessel travel: Bremerton to Seattle (BRE-SEA) and Seattle to Bremerton (SEA-BRE). The statistics provided are mean values of H_{max}, wave power for the wake, and sailing distance to the wave gauge. Figure 4-6 shows histograms of wave power for all wake wash measurements at sites on each shoreline. Figure 4-7 and 4-8 show histograms of wave power for the Bremerton to Seattle and Seattle to Bremerton sailing directions measured at each wave gauge location. A sample of the vessel sailing lines operated by RP1 is shown in Figure 4-9. Histograms of wave power and maximum wave height for each station and complete (more detailed) tables of wake statistics for each valid measured wake for each deployment are provided in Appendix A. Separate tables in Appendix

A are provided for bulk wake statistics of each deployment and for wake statistics sorted by direction of vessel travel.

Measurements of wake wash from RP1 can be summarized as follows:

- Despite significant differences in distances from the sailing line, average H_{max} is approximately 20 centimeters (cm) at all sites except for the two wave locations measured along Pleasant Beach (WG4 and WG5), where the average H_{max} is 13 cm or less (Table 4-1).
- The mean wave power is also the smallest along Pleasant Beach (WG4 and WG5). Both sites are located along a section of straight shoreline that typically experiences minimal wake energy due to longer distances from the vessel sailing line and flatter beach slopes with extensive sub-tidal shoals which dissipate more wave energy before the waves reach the shoreline. In addition to distance decay, waves are also subject to refraction (focusing and spreading) of energy depending on the curvature of the sailing line and the bathymetry between the sailing line and measurement point. Pleasant Beach is on the outside curve of the sailing line so energy is spread along a lengthening wave crest as it propagates to the shore.
- Mean wave power and highest percentage of wave power is greater than 10,000 Joules per meter (J/m) along East Bremerton (WG1).
- Along East Bremerton and Port Orchard, RP1 generated wake wash at higher powers more frequently when travelling to Seattle (Figure 4-7). The wave power generated by RP1 traveling to Bremerton (Figure 4-6) is larger than the wave powered generated traveling to Seattle along Pleasant Beach and Point Glover.
- Along Point White, Point Glover, and Pleasant Beach, RP1 generated wake wash at higher powers more frequently when travelling to Bremerton (Figure 4-8).
- There is more variability in the distribution of wave power generated by RP1 traveling to Bremerton than traveling to Seattle along Point White, Point Glover, and East Bremerton.
- Average vessel speeds were between 37 and 39 kts at most wave gage locations, except East Bremerton and Port Orchard where average vessel speeds were 28 and 34 kts respectively.
- The average operating speed for RP1 as it passed the wave gage on East Bremerton was 28 kts, but ranged from 17 to 38 kts. RP1 was often accelerating or decelerating between Bremerton and the entrance to Rich Passage to avoid other vessel traffic and to save on fuel. Based on the wake acceptance testing, RP1 will generate a significantly larger wake operating at 17 kts than at 38 kts (Golder 2013b).
- There was significant variability in the sailing line of RP1 with respect to the channel centerline throughout in-situ beach response testing (Figure 4-9).

Visual observations and measurements of wakes confirm the amplification of RP1 wakes near the shoreline to the east of Point Glover when RP1 is travelling to Bremerton. The increased wave energy along the eastern shore of Point Glover was also predicted in the numerical modeling (Appendix B). This amplification occurs as a result of constructive wake-wake interactions on the inside corner of the vessel sailing line combined with the focusing of the wake train as it shoals and refracts across the relatively shallow bathymetry of the Point. Observations of the wake-wash from RP1 show the vessel generated waves between Manchester State Park and Point Glover (northeastern portion of Rich Passage) take a longer time to arrive at the shoreline of Point Glover than the vessel generated waves between Point Glover and Waterman's Point (southwestern portion of Rich Passage). Therefore, two different wave trains from wake-wash generated by RP1 on the sailing from Seattle to Bremerton are received at Point Glover at approximately the same time. This phenomenon is not observed when RP1 travels to Seattle, because the wake-wash created between Waterman's Point and Point Glover dissipates before the wake created between Point Glover and Clam Bay reaches the shoreline.

The amplification of wakes and energy along East Bremerton is mostly a function of the proximity of the vessel sailing line to the shore combined with the change in wake properties resulting from variations in vessel sailing speed as RP1 approaches and departs from Bremerton. The morphological implications are addressed in Section 4.2.

Table 4-1: Aggregate statistics of wave measurements during RP1 beach response testing

Site No.	Valid Wake Events Measured	Mean H_{max} (m)	Mean T@H_{max} (s)	Mean Power (J/m)	Passing Speed (kts)	Mean Sailing Distance (m)
WG-PW	76	0.20	3.15	5442	38.74	199
WG-PG (1st)	51	0.22	3.22	7925	36.61	376
WG-PG (2nd)	117	0.18	3.16	7022	36.94	507
WG1	104	0.19	4.06	11359	27.94	657
WG2	146	0.21	3.31	6491	37.77	190
WG3	89	0.21	3.06	7831	38.48	242
WG4	99	0.11	3.38	3790	37.51	426
WG5	61	0.13	2.81	4486	38.16	653
WG7	130	0.20	2.92	8922	37.28	582
WG8	83	0.22	3.19	8845	38.07	251
WG8A	19	0.19	3.84	6791	37.34	430
WG9	149	0.17	3.40	7611	34.44	523

Notes: The instrument deployed on October 22 was missing upon recovery.

Table 4-2: Statistics of wave measurements during RP1 testing based on sailing direction

DIRECTION	BREMERTON-SEATTLE			SEATTLE-BREMERTON		
Site No.	Mean H_{max} (m)	Mean Power (J/m)	Mean Sailing Distance from Wave Gauge (m)	Mean H_{max} (m)	Mean Power (J/m)	Mean Sailing Distance from Wave Gauge (m)
WG-PW	0.21	5539	210	0.20	5346	188
WG-PG (1st)	0.23	8617	356	0.20	7146	398
WG-PG (2nd)	0.15	4527	476	0.21	8820	529
WG1	0.19	11157	580	0.20	11577	741
WG2	0.21	6313	172	0.21	6665	207
WG3	0.22	8153	245	0.21	7529	240
WG4	0.10	3426	433	0.13	4427	414
WG5	0.13	4508	651	0.13	4448	657
WG7	0.19	8103	568	0.21	9740	598
WG8	0.20	8291	262	0.23	9411	241
WG8A	0.15	3738	423	0.23	10990	440
WG9	0.17	8882	569	0.17	6214	472

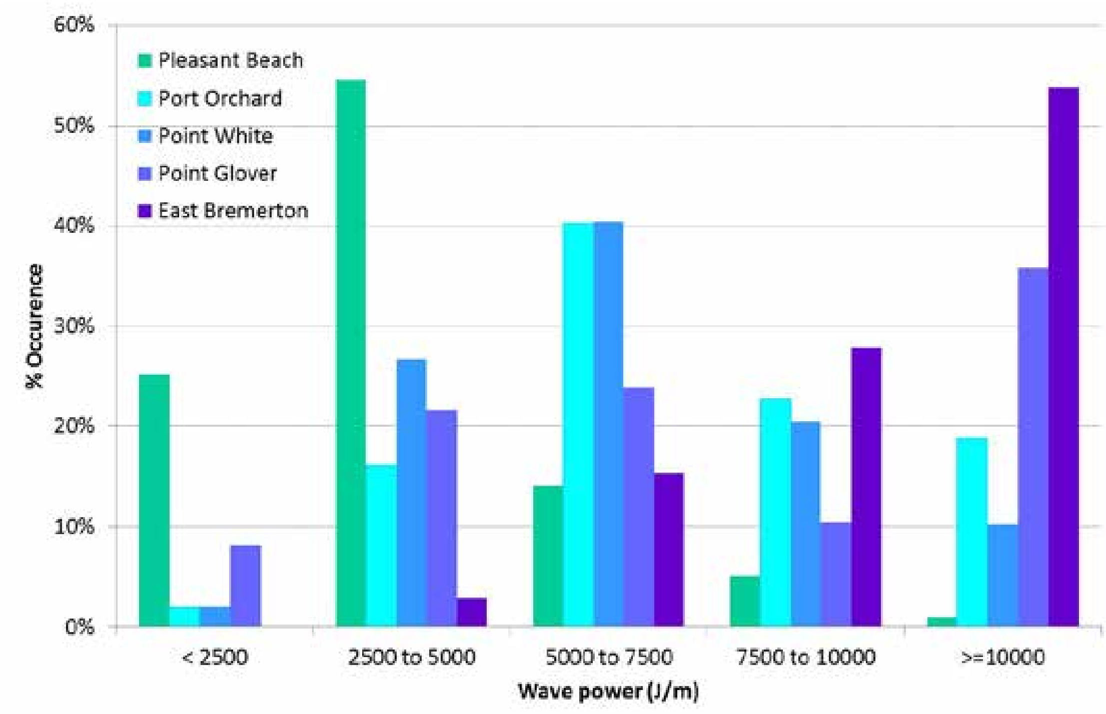

Figure 4-6: Histogram of wave power from RP1 measured along each shoreline

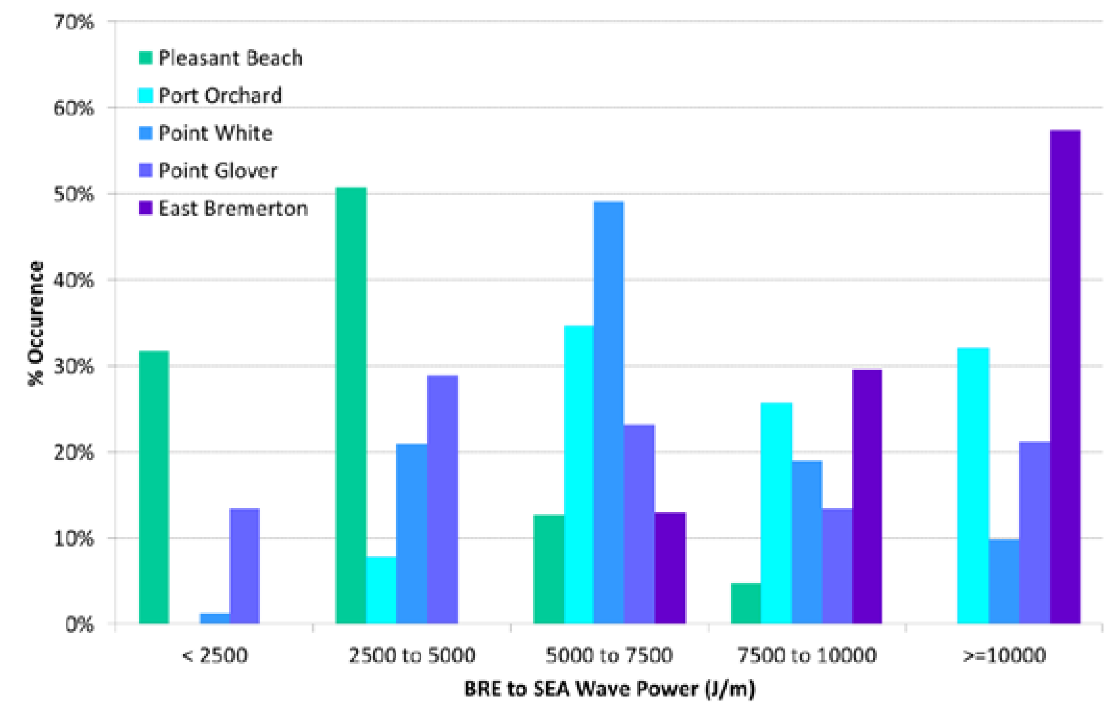

Figure 4-7: Histogram of wave power from RP1 while operating Bremerton to Seattle measured along each shoreline

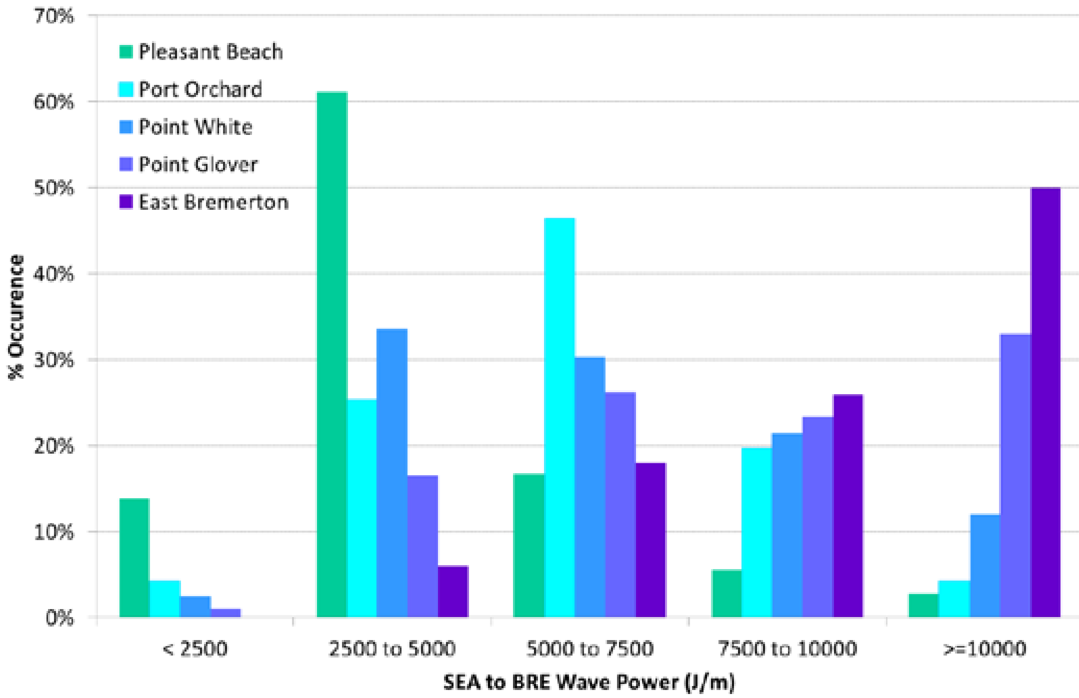

Figure 4-8: Histogram of wave power from RP1 while operating Seattle to Bremerton of RP1 measured along each shoreline

Figure 4-9: Vessel sailing lines from RP1 during in-situ beach response testing

4.2 Morphologic Response

The beach response to RP1 was measured with beach profiles, laser scanning surveys, gravel tracer measurements and beach photo observations. In general, the magnitude of beach volume change was within the scale of typical seasonal beach volume change. However, the beach response observed during the RP1 testing varies spatially and temporarily depending on the relative exposure to wakes (e.g., Section 4.1.) and the local size distributions and supply of sediment.

On East Bremerton, the beach volume change analysis and beach photo observations suggest a minor flattening of the beach slope after the onset of operations, which has also been seen in previous POFF operations. The volume of sediment below MTL has increased at a greater rate than the beach volume above MTL (Figure 4-10). This beach response is attributed to RP1 operating at less than optimal speeds for minimizing wake wash along Port Orchard reach as well as accelerating and decelerating to wait for vessel traffic to clear before entering Rich Passage. Despite this response, the elevation of the beach along the entire profile following the conclusion of the RP1 tests at Sites 1B and 1C is not significantly different from the November 2004 baseline condition (Appendix C).

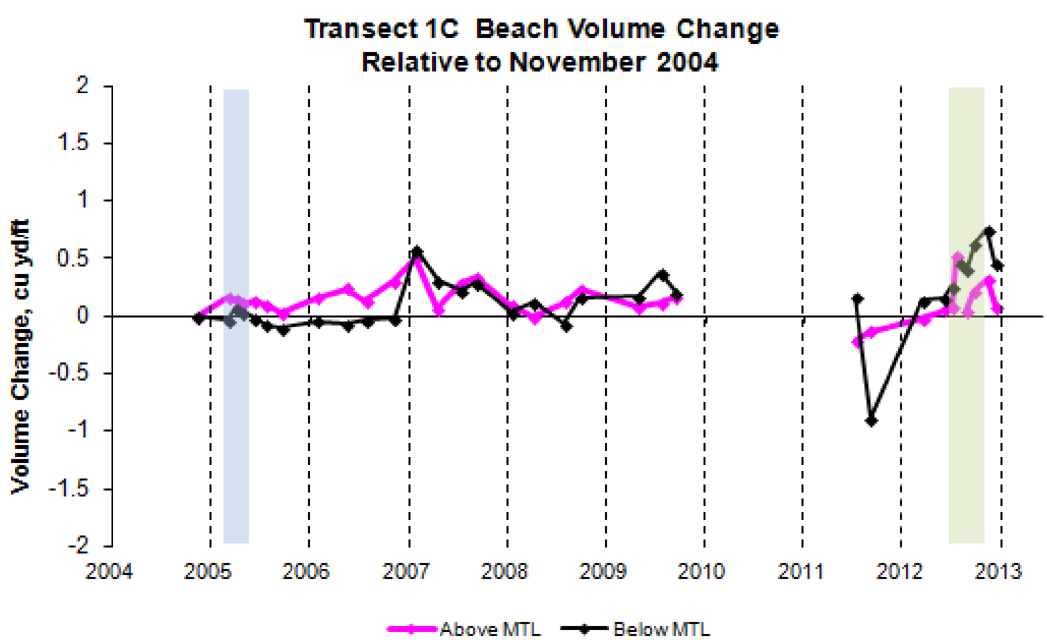

Figure 4-10: Beach volume change at Transect 1C on East Bremerton

On Port Orchard, beach volumes had begun to increase prior to the start of RP1 testing as is typically observed during the summer months (Figure 4-11). The beach volume both above and below MTL started decreasing in October and continued decreasing through December. However, the largest decline in beach volumes occurred between November and December, just after RP1 testing had stopped. The pattern appears to be more closely associated with the increase in wind speeds and shift in wind direction from northeast to south-southwest in October than the start and end of RP1 operations. The magnitude of the beach response during RP1 testing was within the seasonal response observed in 2007 to 2008 and 2008 to 2009.

Pleasant Beach responded similarly to Port Orchard. The upper beach volumes show accretion throughout the summer, followed by decrease in beach volumes above the MTL between October and December 2012 similar to the seasonal changes in beach volumes observed in 2006 and 2007 (Figure 4-12).

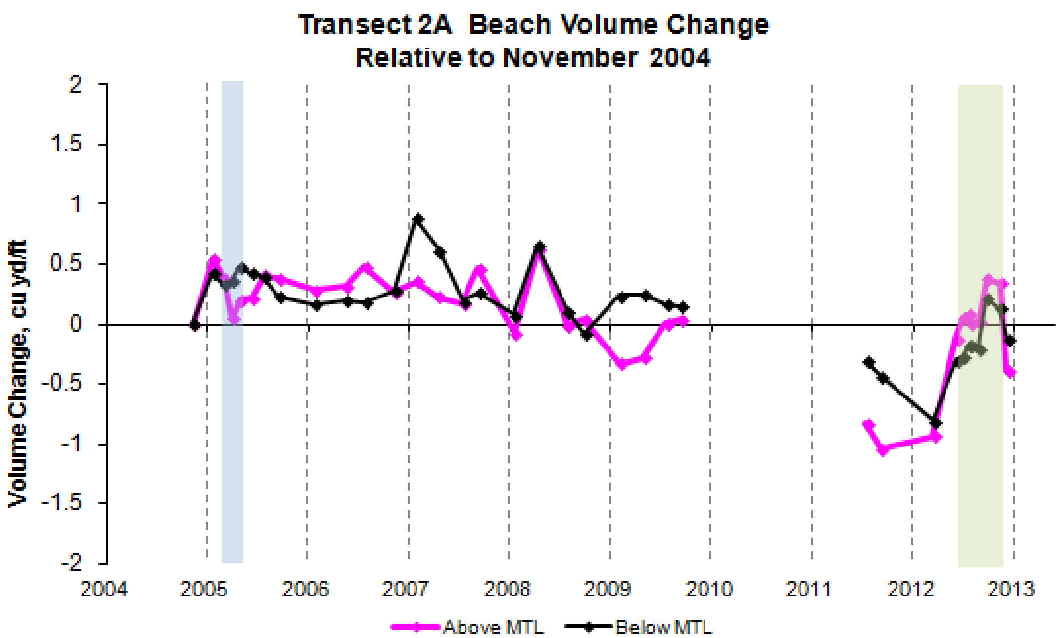

Figure 4-11: Beach volume change at Transect 2A on Port Orchard

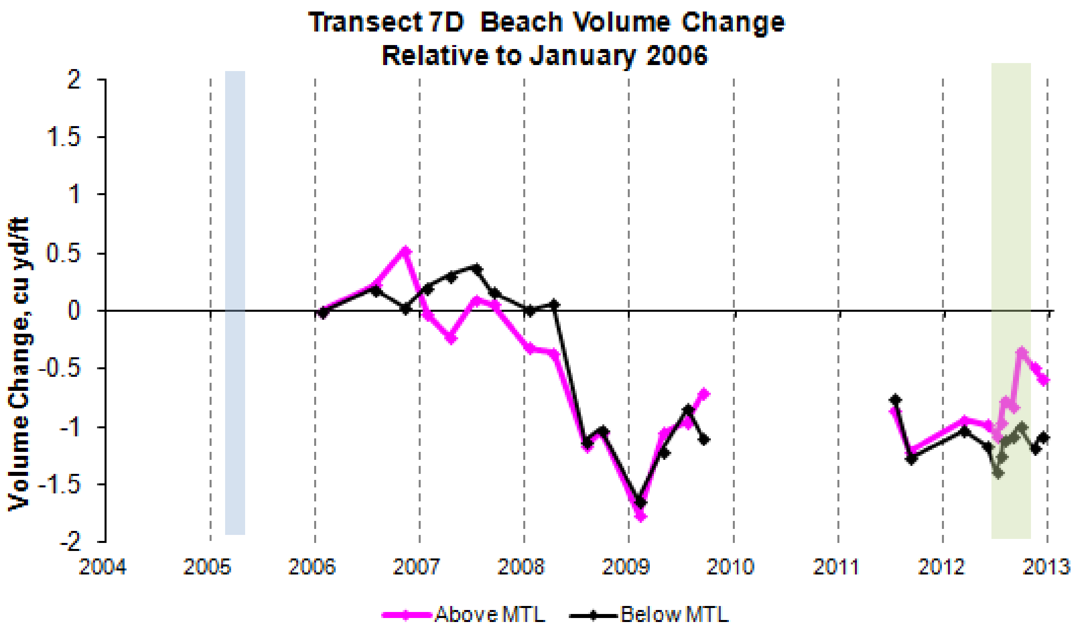

Figure 4-12: Beach volume change at Transect 7D on Pleasant Beach

Although the beach response during RP1 testing varies from site to site along Point White, the changes were not significant in comparison to the seasonal trends and inter-annual variability. There was no systematic trend in beach volume change along Point White during RP1 operations, and therefore no net increase or decrease in beach volumes (Figure 4-13).

The beach volume change analysis for Point Glover shows insignificant change in the volume of sediment in the cross-shore during the testing interval. However, the alongshore patterns of beach response along Point Glover are discussed in the following section.

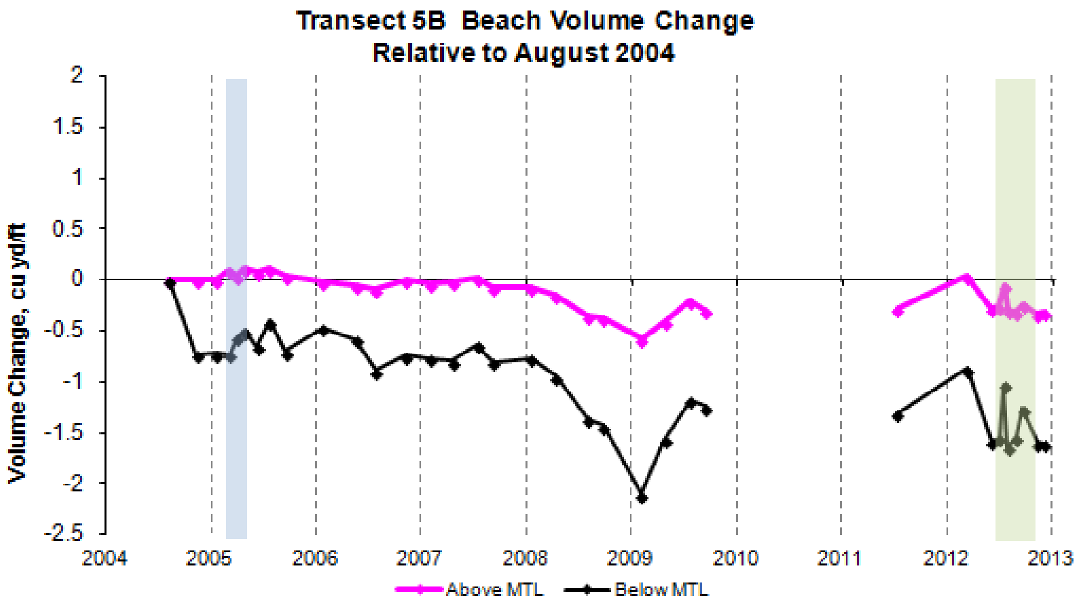

Figure 4-13: Beach volume change at Transect 5B on Point White

4.2.1 Sediment Transport Patterns

The beach response to RP1 was further quantified through laser scanning surveys measured at two sites along Point White and one site on Point Glover. A complete set of difference maps between each survey interval is presented in Appendix E. A subset of these results for the interval of RP1 operations is discussed below.

The difference plot presented in Figure 4-14 shows the beach change during the entire RP1 testing interval (June to November 2012) at Site 3 on Point White. The hot colors (red) near the bulkhead indicate there was net accretion on the upper beach during this interval. The alternating pattern of warm colors (accretion) and cool colors (erosion) indicate sediment moves in waves alongshore, particularly at the interface of the beach and the bulkhead, similar to the patterns seen during the Winter 2011 to 2012 (Figure 3-5), but with more accretion in the summer and more erosion in the winter. Figure 4-15 shows a one-month difference plot representing the post-RP1 operation period for November to December 2012. The gravel mounds at the bulkhead have moved cross-shore away from the bulkheads but remain in the upper beach. This is a typical winter seasonal shift resulting from the increasing wind-waves and higher tides during the day such that wake wash and wind-waves have a more significant effect on the upper beach.

Figure 4-14: Elevation difference at Point White – Site 3 between 04 June 2012 and 12 November 2012

Figure 4-15: Elevation difference at Point White – Site 3 between 12 November 2012 and 11 December 2012

At Site 4 on Point White, subtle gravel bars form parallel to the shoreline, which indicates cross-shore sorting of gravels in response to wave forcing on the lower beach (Figure 4-16). During RP1 operations there is a small decrease in beach elevation on the upper beach and accretion of sediment on the lower beach. The area with the most accretion is on the upper beach near the mouth of a creek near Profile 4A. The runoff from the creek contributes sediment to the beach at this location, but the curvature of the shoreline can cause sediment moving alongshore to be trapped at the creek mouth. Beach sediments are transported alongshore to the north as well as to the south at Site 4. From November to December, there is erosion of sediment at the mouth of the creek, which is deposited on the lower beach (Figure 4-17). The gravel bars present on the lower beach in the summer have moved up the beach during this winter interval, essentially reversing the erosion and accretion patterns between the summer and winter season.

The summer and winter seasonal transport processes have been measured using gravel tracers in 2006 to 2007 and were repeated in 2012 during RP1 testing. Figures 4-18 through 4-20 show photo-maps of individual tracer particle positions at their deployment locations in June 2012 and after the most recent monitoring in December 2012 for Sites 3, 4, and 5. Particle tracers are differentiated based on size classes. In general, the tracers have spread to the northeast along the beach and have remained largely on the inter-tidal beach indicating a predominance of alongshore transport.

No clear pattern of sorting has occurred at Site 3, however the tracer path moves initially offshore, then onshore along a north-northeast trajectory. At Site 4, the absence of grains coarser than 45 millimeter (mm) suggests an offshore movement of the coarsest particles and therefore they were not within the survey area. Grain sizes less than 20 mm appear to have been transported predominately alongshore to the northeast relative to the larger size classes. At Site 5, sizes coarser than 32 mm and 45 mm are located on the lower beach, while the smaller particles are located higher on the beach in the reverse-graded pattern that is characteristic of a lower energy regime.

Figure 4-16: Elevation difference at Point White – Site 3 between 04 June and 12 November 2012

Figure 4-17: Elevation difference at Point White – Site 3 between 12 November and 11 December 2012

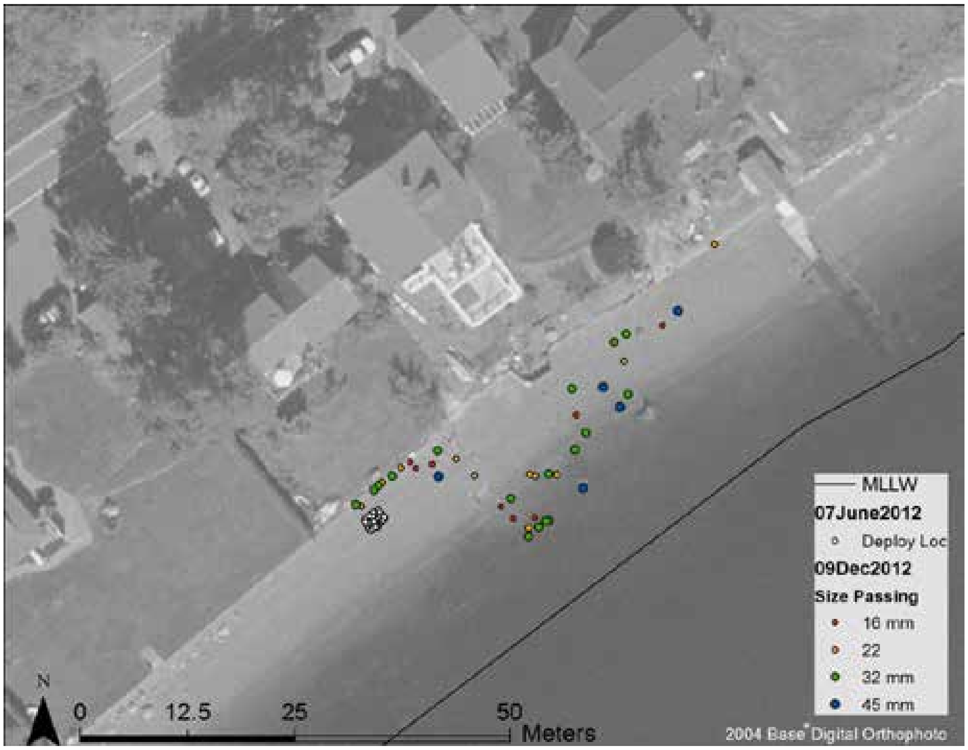

Figure 4-18: Tracer locations from first to last survey dates at Site 3

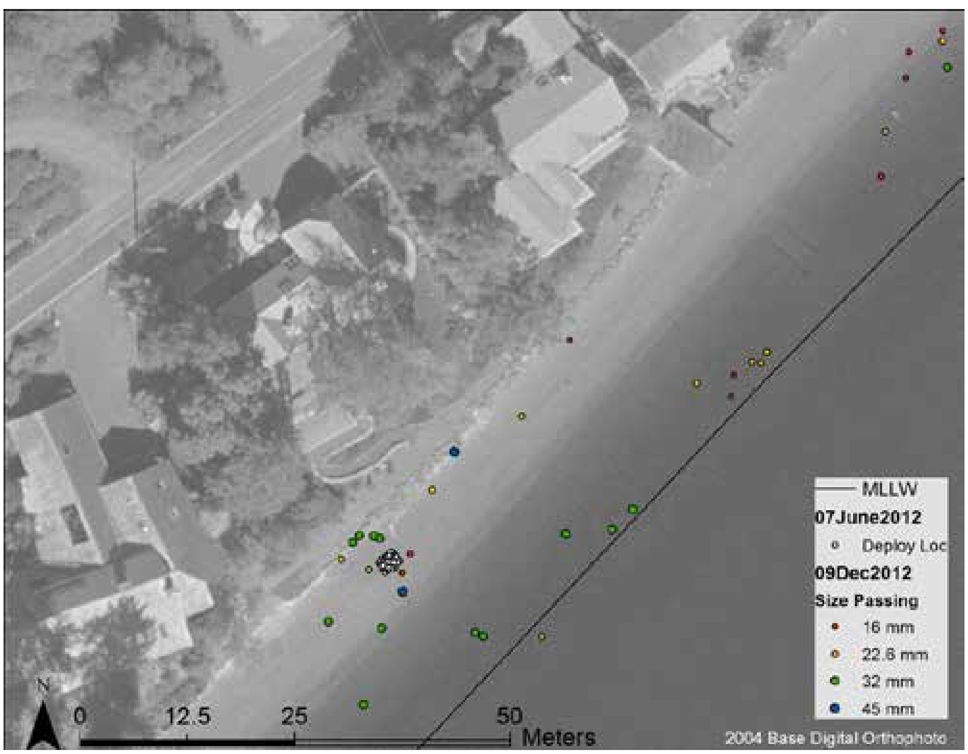

Figure 4-19: Tracer Locations from first to last survey dates at Site 4

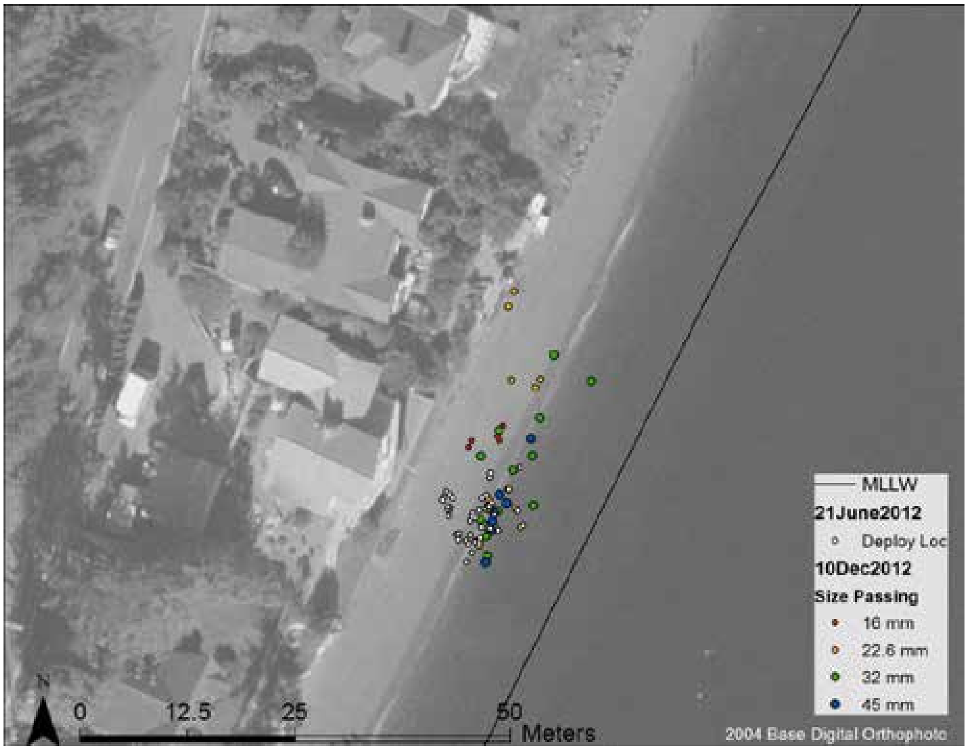

Figure 4-20: Tracer Locations from first to last survey dates at Site 5

As with the baseline study, alongshore movement towards the northeast is the most noticeable feature at the three sites (Figure 4-21 and 4-22). However, movement is greater at Site 4 than Site 3 and substantially greater at Site 3 and Site 4 than Site 5. Several short-term reversals in alongshore transport occurred at all three sites in July and August and another reversal was observed at Site 4 that lasted from September to November, spanning several survey periods. Tracer movement at Site 3 and Site 5 averaged 0.07 and 0.03 meters per day (m/day), respectively during the entire survey period while tracer movement at Site 4 averaged 0.15 m/day in June and July and decreased to an average of 0.05 m/day for the remainder of the survey. Average tracer movement at Site 3 in 2006 was approximately equal to average tracer movement in 2012 (Figure 4-21). Transport rates were 0.065 m/day in the Summer and Fall of 2006 as compared to 0.07 m/day during the Summer and Fall of 2012. However, average tracer movement at Site 4 was 0.005 m/day in 2006 as compared to 0.05 m/ day in 2012. This discrepancy in transport magnitude at Site 4 is examined in more detail below. Gravel tracer studies were not conducted at Site 5 in 2006.

Figures 4-23 and 4-24 show a comparison of the size distribution of the tracer particles relative to the resident grain size distribution of the beach measured in 2012 and 2007. The tracer distribution is approximately equivalent to the beach surface samples at Site 3 in 2012. However, the tracer distribution is significantly coarser than the resident grain size distribution of the beach measured at Site 4 in 2012, whereas the grain size distribution at Site 4 was more closely matched to the tracer distribution in 2006.

The same set of gravel tracers, and therefore grain size distribution of tracers, was used in 2006 as in 2012. At Site 3, the rapid transport rates in June and July (0.15 m/day) are explained by the incompatibility in grain size distribution between the gravel tracers and the beach sediments at this site. In general, the median grain size of the beach gravel increases with decreasing elevation on the beaches along Point White (PIE 2007a) and most of Puget Sound (Finlayson 2006). This pattern is more pronounced at Site 4 due to the weak hydrodynamic forcing combined with gravitational affects as compared to Site 3, which is more exposed to wind-waves and wake wash. As a result, the larger size tracer particles were transported to lower elevations to match the distribution of the resident beach sediment. The sorting of beach sediments by grain size was also observed in 2006, but occurred in the first one to two weeks of deployment.

In the baseline study, transport increased by at least a factor of 6 during the storm interval from November to January. The final gravel tracer monitoring was on 9 December 2012, which was prior to any significant storm events that might have shown significant increase in transport rates. Gravel tracers will be monitored again in Spring 2013.

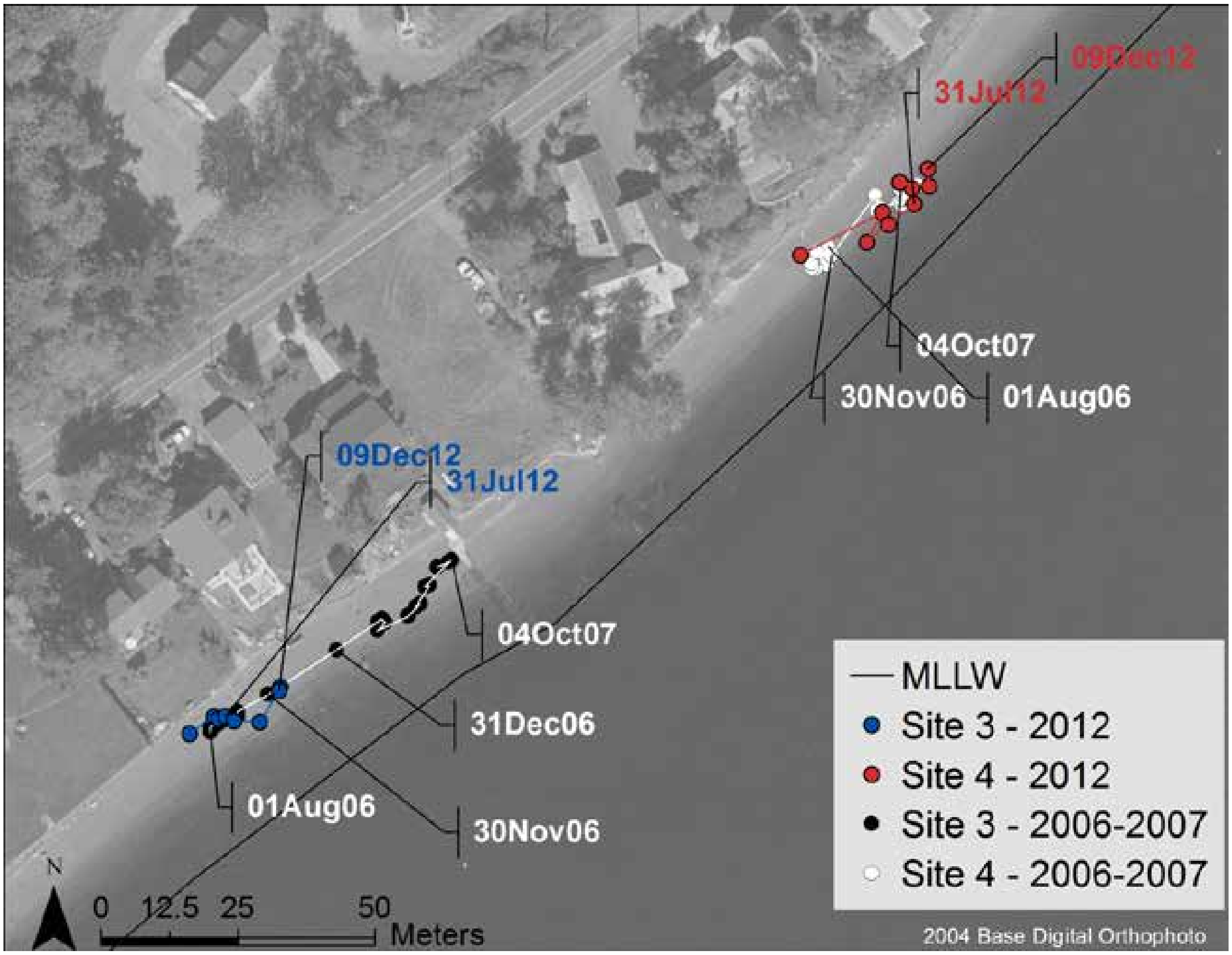

Figure 4-21: Location of tracer centroids for the 2006-2007 and 2012 survey at Sites 3 and 4

Figure 4-22: Location of tracer centroids for the 2012 survey at Site 5

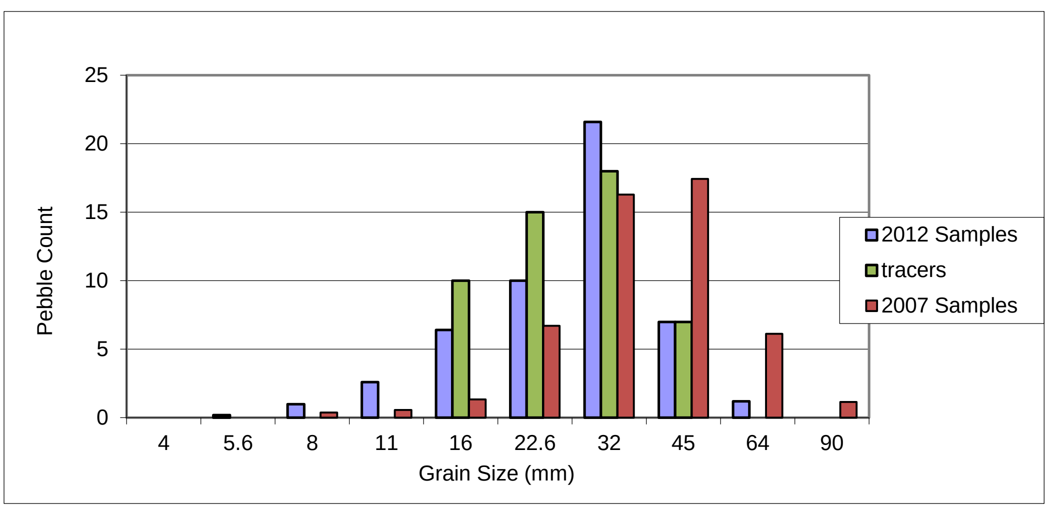

Figure 4-23: Comparison of the normalized frequency of the 2012 and 2007 beach samples and tracer sediments at Site 3 at 2’ to 8.5’ MLLW

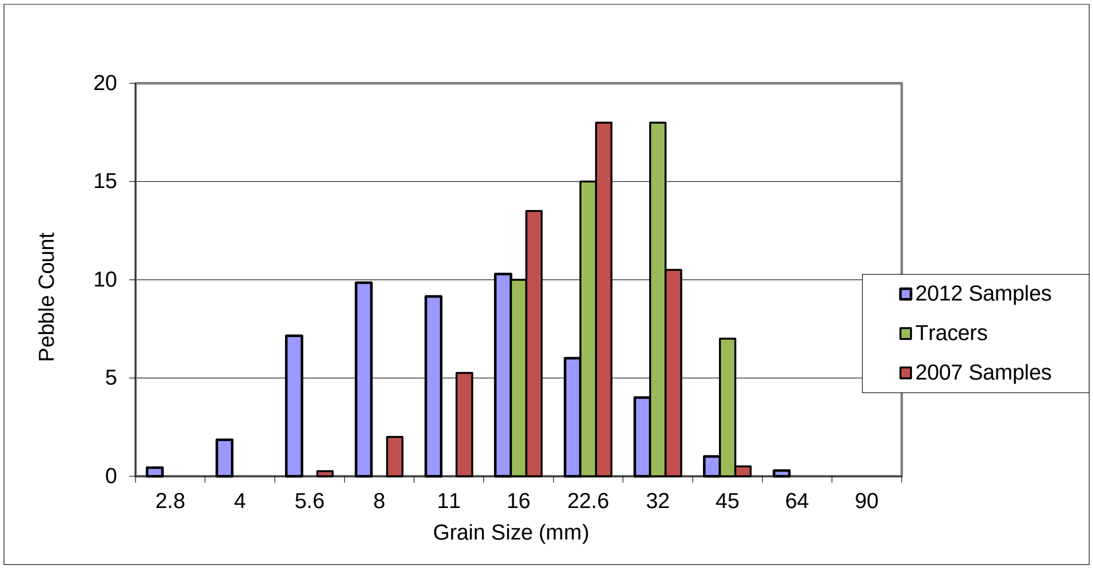

Figure 4-24: Comparison of the normalized frequency of the 2012 and 2007 beach samples and tracer sediments at Site 4 at 2' to 7.2' MLLW

Laser scanning surveys conducted on Point Glover showed alongshore transport of material during the RP1 testing period (June to November 2012). Figure 4-25 shows an elevation difference plot implicating a net southwesterly transport of sediment along this shoreline. This pattern is more pronounced near the northeast corner of the survey area, where sediment is eroding and moving southwest along the bulkhead. A reversal of this transport pattern is observed during the post RP1 testing period (between November and December 2012). The south-to-north, alongshore transport pattern observed along Point Glover between November and December 2012 (Figure 4-26) is consistent with the change observed during the winter of 2011-2012. This pattern may be more a reflection of the seasonal shift in wind patterns and the relative wake to wind-wave dominance at this site rather than solely attributed to RP1 operations

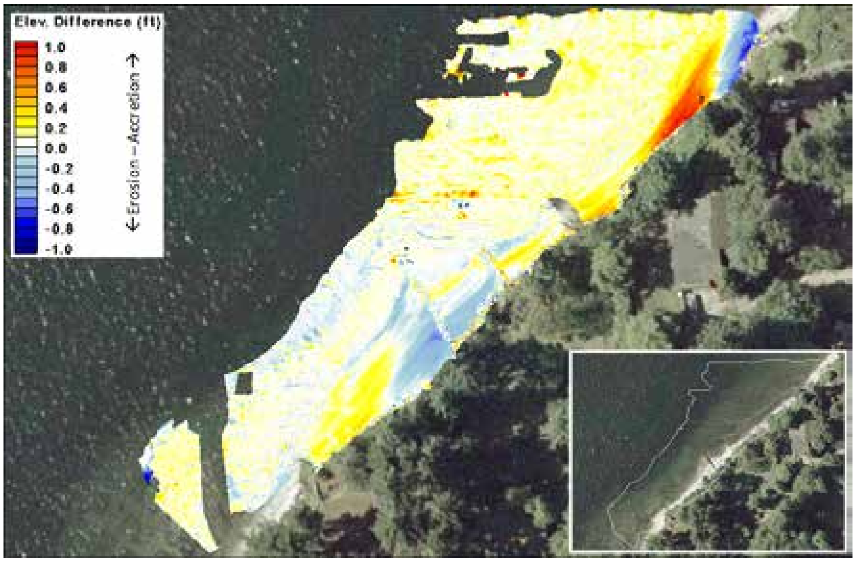

Figure 4-25: Elevation difference at Point Glover – Site 9 between 05 June 2012 and 13 November 2012

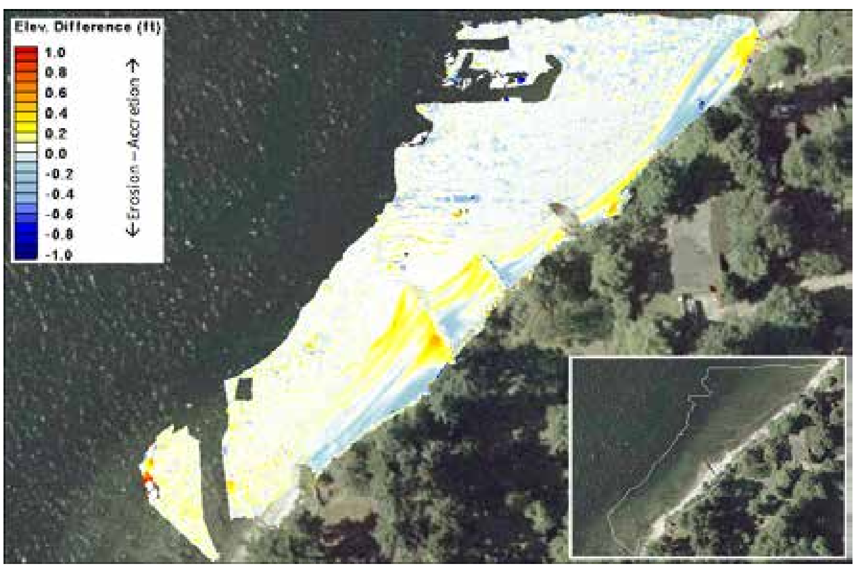

Figure 4-26: Elevation difference at Point Glover – Site 9 between 13 November 2012 and 13 December 2012

4.2.2 Beach Volume Changes

This section summarizes the results of the systematic analysis of beach volume change; this analysis was conducted on morphological change surfaces developed from the laser scanning surveys. A complete summary of calculations from the volume change analysis for each of the three survey sites (Site 3, 4, and 9) is included in Appendix C.

When net volume change is averaged over every survey period (September 2011 through December 2012) these data show that Site 3 experiences an average net accretion of 0.3 cubic yards (CY) per day (107 CY/year), Site 4 experiences an average net erosion of -0.1 CY/day (-38 CY/year), and Site 9 experiences an average net accretion of 0.2 CY/day (72 CY/year). In general, these annual volume changes are small relative to the total length and area of the survey. At Site 9 on Point Glover, there is a positive volume (accretion) bias stemming from the incomplete survey of the drift cell at the northeast corner of Site 9. The drift cell at this site is a pocket beach with the distal end close to the Point where the shoreline changes orientation. When only the four southwestern subdivisions of the boundary mask on Point Glover are analyzed between the first groin on the southwest end of the site and the last groin at southern terminus of the survey, the average net change is 0.0 CY/day, or no net change. If feasible, it is recommended that future surveys of Site 9 at Point Glover be extended or shifted approximately 100 ft northeast in order to capture the entire southwestern drift cell.

Although there is significant variability in the volume change from one survey to the next, over the longer term the general trend is convergence to zero net change. This general trend is evident in Figure 4-27 and Figure 4-28. Figure 4-27 shows the erosion and accretion rates for each site over entire interval of the laser scanning surveys. This figure depicts significant scatter at shorter survey intervals (e.g., < 100-day intervals) and considerably more consistent rates of change as the survey intervals increases (e.g., > 100-day intervals). In general, the erosion curves are the inverse of the accretion curves, indicating that the majority of the changes may be explained by local movement of sediment within the cells. Figure 4-28 shows the net volume change rates for each site during the interval of the laser scanning surveys. This figure also depicts significant scatter during the shorter survey intervals (e.g., < 100-day intervals) and considerably more consistent rates of change during the longer survey intervals (e.g., >100-day intervals). This indicates that although significant changes may occur over short time frames at all three sites, the periods of net volume loss are followed by periods of net volume recovery, resulting in only minor net changes over time.

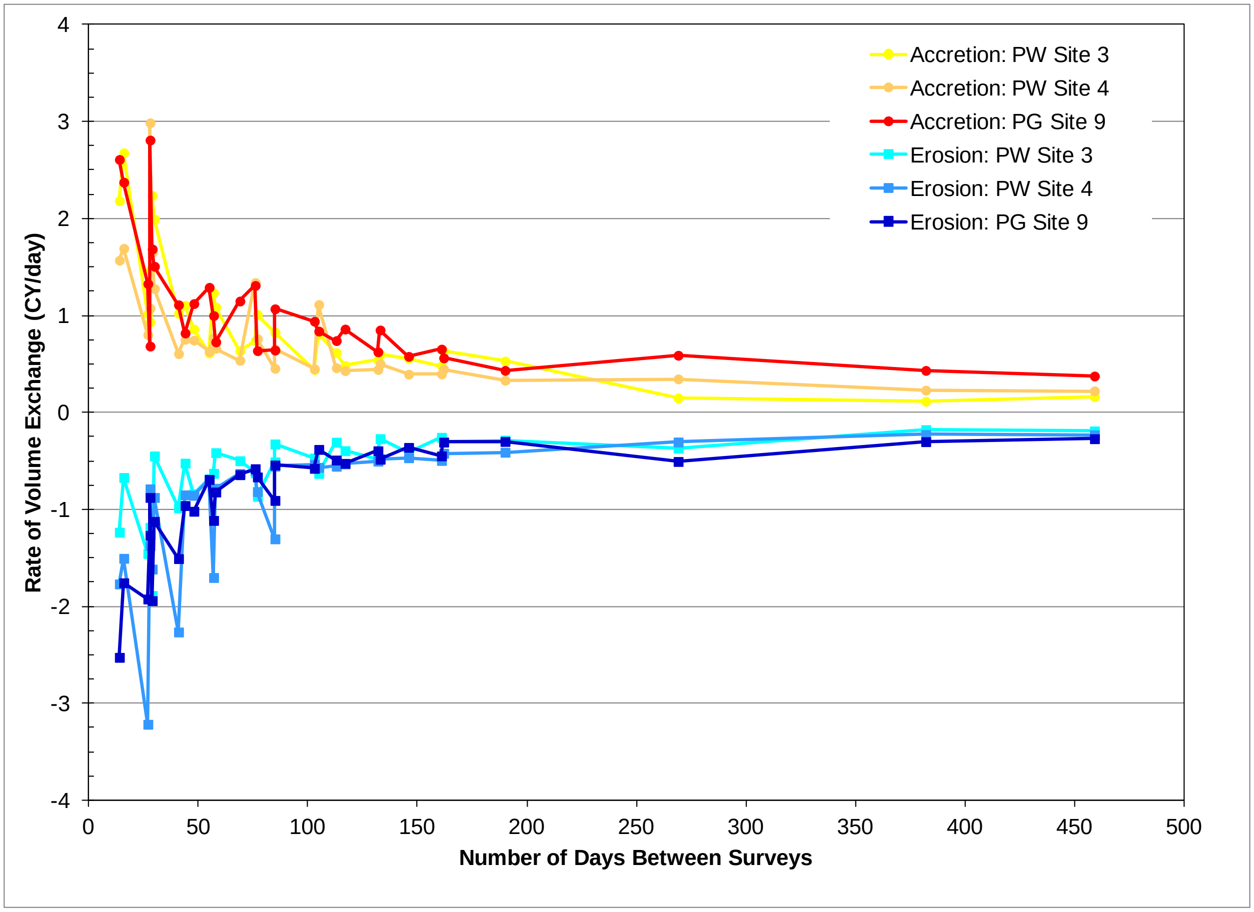

Figure 4-27: Calculated rates of erosion (negative volume; cooler color lines) and accretion (positive volume; warmer color lines) at each site for every survey period

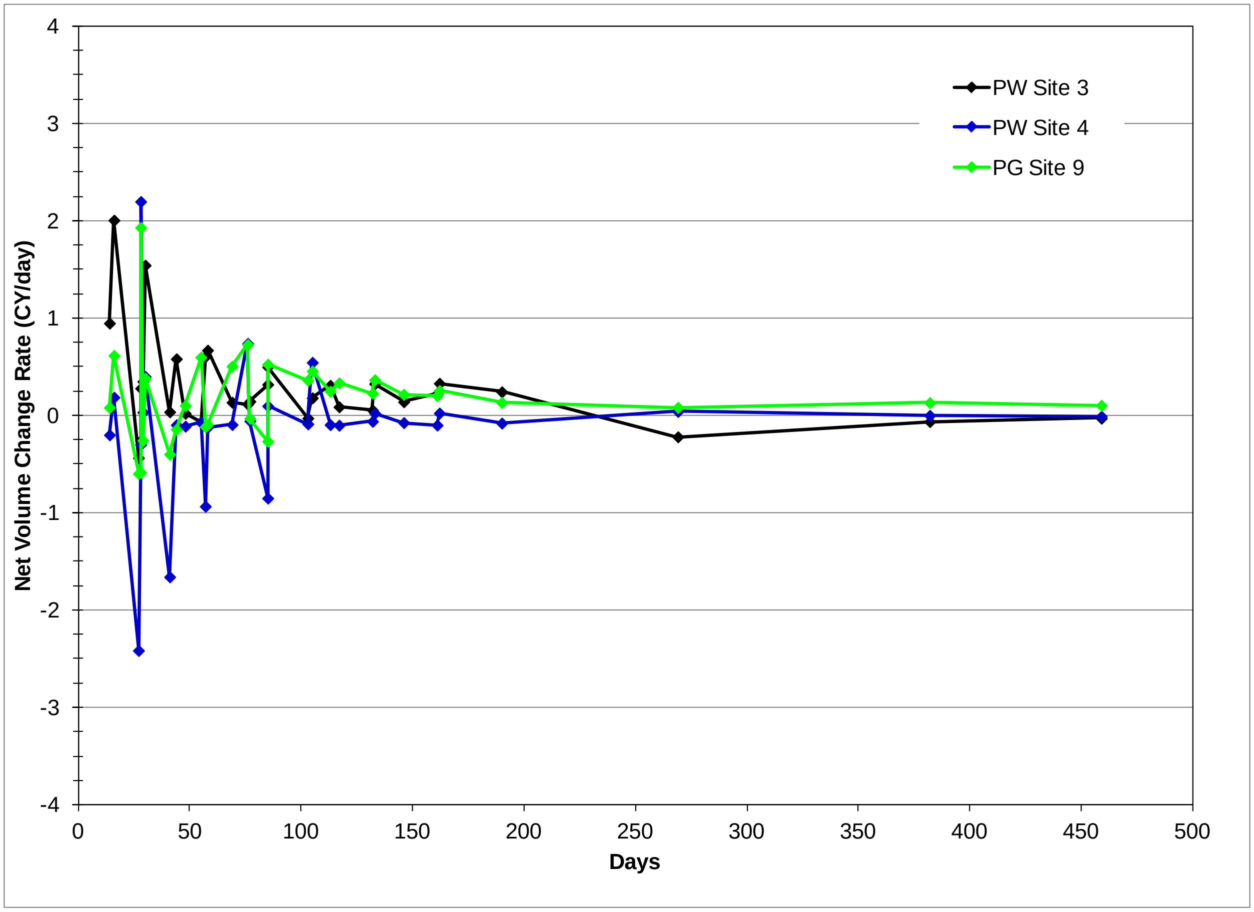

Figure 4-28: Calculated rates of volume change at each site for every survey period

Table 4-3 presents the calculated volume change between September 2011 and the start of the RP1 operations in June 2012 for Site 3, Site 4, and Site 9. The volume change at Site 3 is 42% of the gross transport within the boundary cell. Therefore, the volume loss of -59 CY is a significant \volume loss and net erosion. The bulk of this lost volume is most likely transported alongshore with the potential for some cross-shore loss. The volume loss may be explained by seasonal transport of material due to wind-wave activity throughout the fall and winter. The volume change at Site 4 was 11 CY of accretion and only 6% of the gross transport within the boundary cell. Although Site 4 is adjacent to Site 3 on Point White, the two sites' different shoreline orientations result in different exposures to wave-driven transport. This difference in exposure has also been observed in differences in transport rates measured through the gravel tracer studies. Site 4 is also located towards the down drift end of the same littoral transport cell as Site 3 and is therefore more likely to exhibit net depositional characteristics. Site 9 experienced a -23 CY volume loss between September 2011 and June 2012, which is 22% of the gross transport during that period. This volume loss is likely explained by seasonal change.

Table 4-3: Calculated volume change for the interval between September 2011 and June 2012 (Winter 2011-2012)

Survey Site	Time Range (days)	Mask Area (ft^2)	Accretion (CY)	Erosion (CY)	Gross Transport (CY)	Volume Change (CY)	Volume Change Rate (CY/day)
Site 3	269	26970	40.08	-99.16	139.24	-59.08	-0.22
Site 4	269	25450	93.00	-82.06	175.06	10.94	0.04
Site 9	271	21709	39.67	-62.67	102.34	-23.00	-0.08

Table 4-4 presents the calculated volume change over approximately a 1-year interval between September 2011 and September 2012 for Site 3, Site 4, and Site 9. The volume change of -24 CY at Site 3 is 22% of the gross transport within the boundary cell. Although this still shows a net volume loss over the course of the year, it is significantly less than the volume loss from September 2011 to June 2012 (-59 CY). The volume change at Site 4 is 0% of the gross transport within the boundary cell and therefore essentially no net change for the 1-year interval. Site 9 experienced a 50-CY gain between September 2011 and September 2012, which is 18% of the gross transport during this interval. However, if the drift cell on the northeastern end of the survey area is removed since it is not being surveyed to the distal end of the cell, the volume change reduces to a -25 CY loss of material over the 1-year interval. This indicates more material is being transported alongshore to the north than to the south over this 1-year interval, with a net loss from the southern cells.

Table 4-4: Calculated volume change for the interval September 2011 to September 2012 (1 year)

Survey Site	Time Range (days)	Mask Area (ft.2)	Accretion (CY)	Erosion (CY)	Gross Transport (CY)	Volume Change (CY)	Volume Change Rate (CY/day)
Site 3	382	26970	43.70	-67.84	111.54	-24.14	-0.06
Site 4	382	25450	87.63	-87.39	175.02	0.24	0.00
Site 9	382	39,660	165.41	-115.27	280.68	50.15	0.13

Figures 4-29 through 4-31 show the calculated rates of volume change for the upper beach (above MTL; in red) and the lower beach (below MTL; in black) at each site during each month surveyed throughout the interval of RP1 in-situ testing. The figures show no clear trend in cross-shore volume exchange between the upper beach and lower beach. The analysis also did not show a clear trend in alongshore exchange between the east and west subdivision cells for each of the three sites. Site 3 exhibits the largest accretion rates in both the upper and lower beach during July (Figure 4-29). This may be a result of increased wind speeds that occurred on 3 July 2012 with 21 kts south-southwesterly winds measured at Bremerton airport. While Site 4 (Figure 4-30) and Site 9 (Figure 4-31) also exhibit some accretion during July, it is not as pronounced as Site 3 since these sites are less exposed to wind-waves. Significant net erosion occurs at all three sites in August particularly on the upper beach. August is a period of relatively low energy winds, almost zero precipitation, and low tides such that wake wash energy more actively produces sediment transport particularly in the absence of wind-wave activity. It is possible that the August erosion particularly on the upper beach was produced by RP1 wake wash. However, the greatest accretion rates at Site 4 (Figure 4-30) and Site 9 (Figure 4-31) in both the upper and lower beach occur in September, countering most of the beach loss in August. Site 3 also shows accretion during September, but it is not as pronounced as Site 4 and Site 9. September also had a significant wind event with 19 kts out of the south-southeast measured at Point White on 10 September 2012. These patterns indicated Site 3 is more susceptible to wind-waves out of the southwest aligned with the western entrance to Rich Passage, while Site 4 is more susceptible to transport from wind-waves out of the southeast aligned with the eastern entrance to Rich Passage. Winds occur most frequently out of the northeast during the months of August and September, which would drive southerly alongshore transport at site 9, resulting in the net accretion in September. After RP1 operations stopped in November, all three sites show differing response. Site 3 indicates accretion across the entire area. Site 4 and 9 shows erosion of the lower beach and net accretion of the upper beach. These patterns are potential the response to the stopping of RP1 operations. Historically, an erosion of the upper beach and accretion of the lower beach has been observed in response to fast ferry wakes. This pattern is not as clear during RP1 operations, because the wake wash is relatively small in comparison to previous POFF operations, and wind-waves were driving alongshore transport at the same time as RP1 operations were driving cross-shore transport and are expected to be a significant contributor to gross and net beach response relative to RP1 based on historical (baseline) data and modeling. There is an indication of flattening of the beach slope in August when wind-waves are not present followed by a reversal of this trend in November when RP1 operations were stopped particularly at Site 4. However, it is important to also note that the net change over the 1-year interval of September 2011 and September 2012 was zero at Site 4, which included the August interval with the largest potential for transport from RP1 wake wash. Although RP1 operations were increased from 40 trips per week to 60 trips per week at the beginning of September, the largest net accretion occurred in September; the latter response is generally the opposite of expectations for an increase in POFF wake wash energy. This shift from erosion to accretion between August and September corresponds with the reversals in alongshore tracer transport noted above in Section 4.2 and may represent a wave of sediment moving first in one direction and then the other in response to the reversal in dominant wind direction from northeast in August and September to southwest after 11 October 2012.

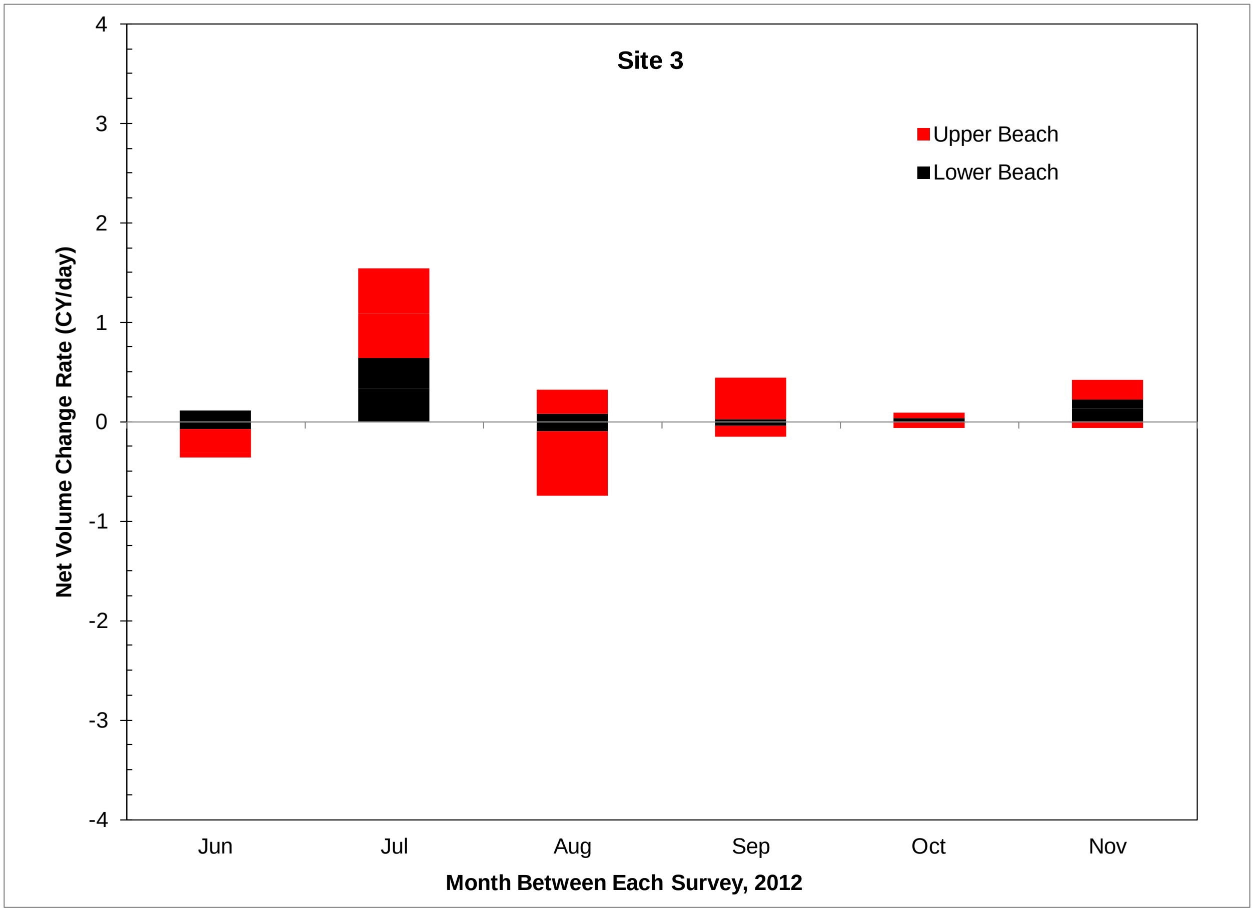

Figure 4-29: Calculated rates of volume change at Site 3 for each approximately month-long survey span during the Summer through Fall 2012 operations

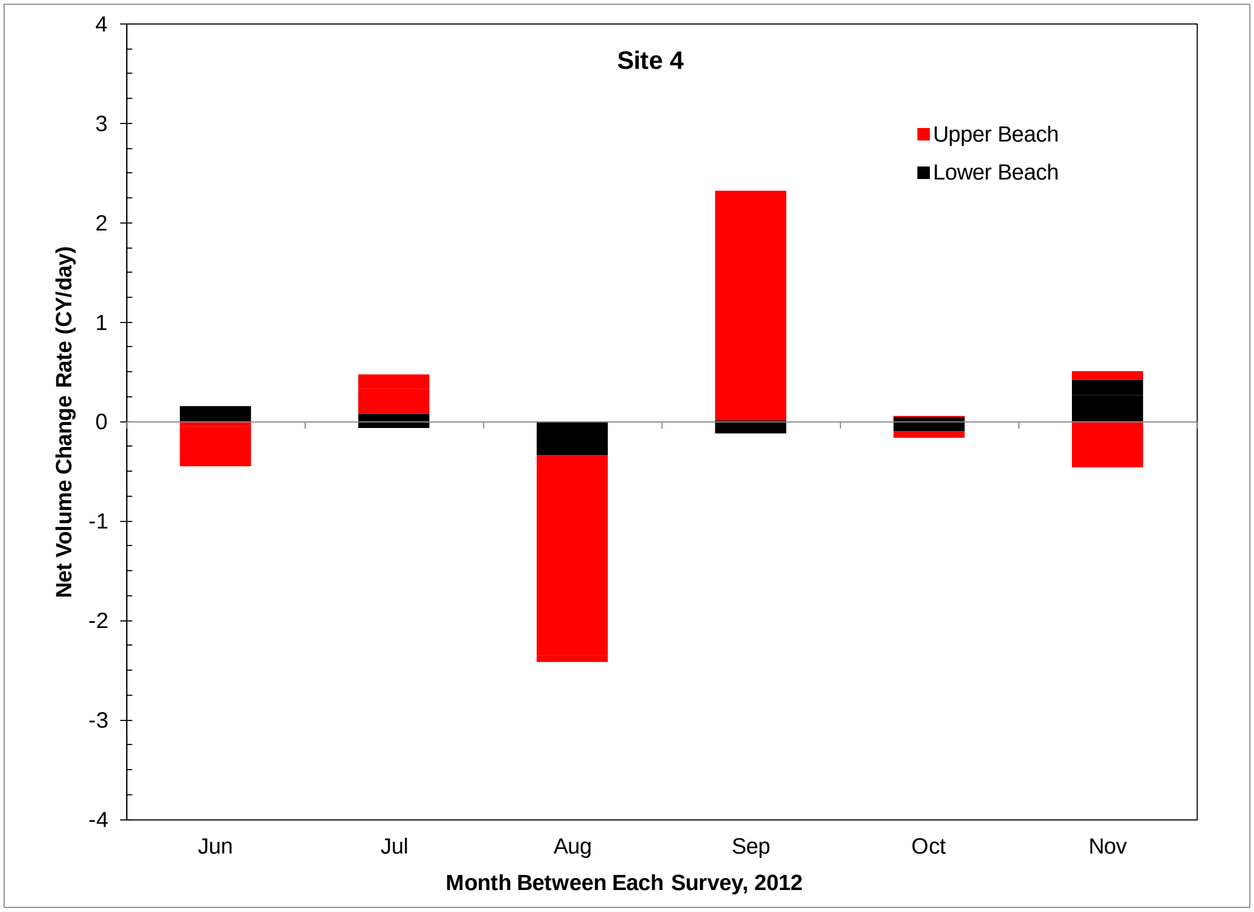

Figure 4-30: Calculated rates of volume change at Site 4 for each approximately month-long survey span during the Summer through Fall 2012 operations

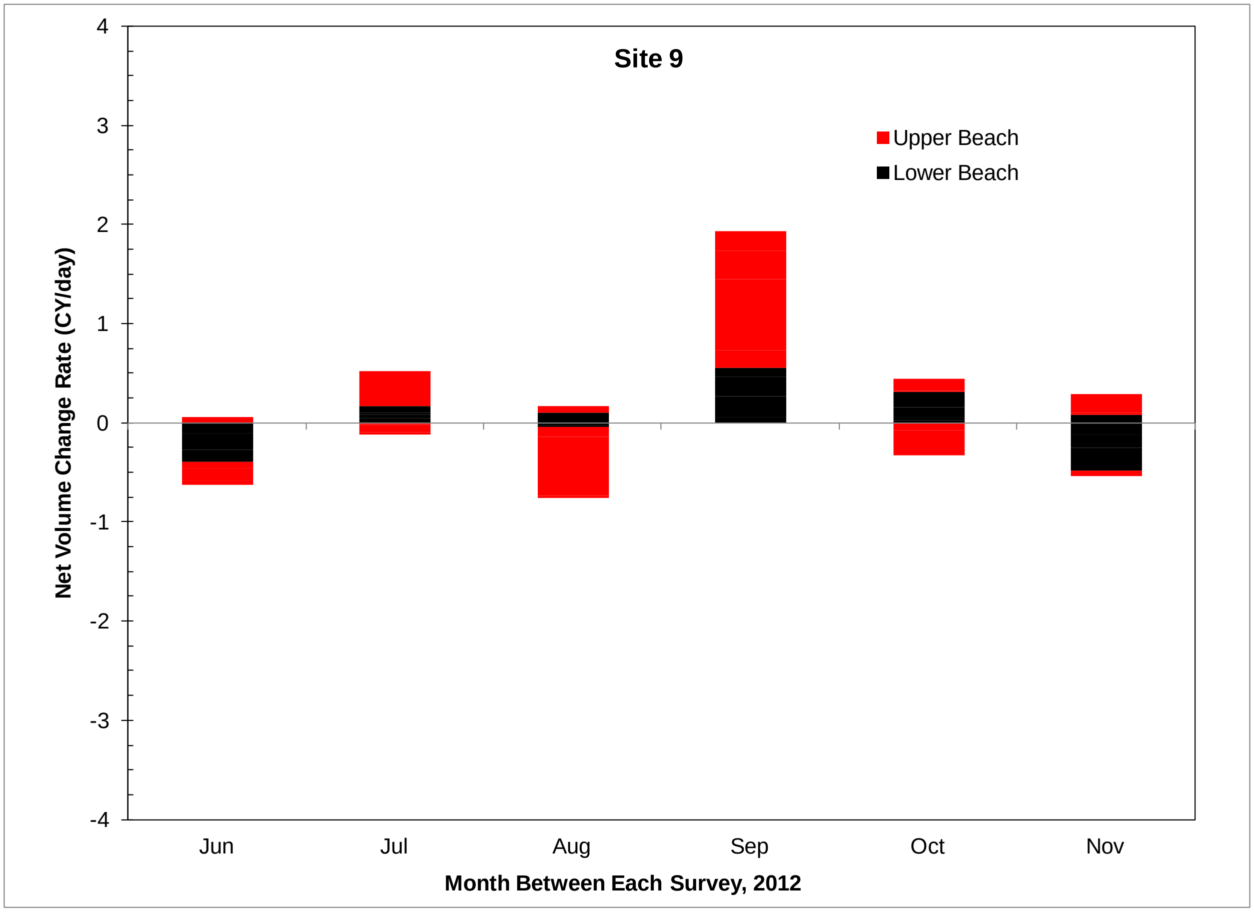

Figure 4-31: Calculated rates of volume change at Site 9 for each approximately month-long survey span during the Summer through Fall 2012 operations

Figure 4-32 shows the calculated rates of volume change at all three sites during each month surveyed before, during, and after the RP1 operations period (June to December 2012). The survey intervals are representative of monthly intervals since they range from 27 to 30 days each for all months except for October 2012, which was a 48-day interval. This figure shows alternating pattern of accretion and erosion at Sites 3 and 4, which is an indication of sediment moving alongshore in waves. Site 9 shows alternating patterns of erosion and accretion through August followed by consistent net accretion from September through November. The alongshore transport at Site 9 was observed to be from north to south through the laser scanning surveys. As the Site 9 survey area does not capture the full extent of the northern portion of this drift cell, it is expected that the erosion at the northern end would balance the accretion within the rest of the drift cell for a net volume change of close to zero if this survey area was extended.

Erosion of the upper beach and accretion on the lower beach occurred at both sites on Point White (Site 3 and 4) during the month of June when RP1 had only been operating for approximately 5 days. This suggests the cross-shore response was largely a result of forcing within the baseline hydrodynamic climate rather than RP1 wake wash.

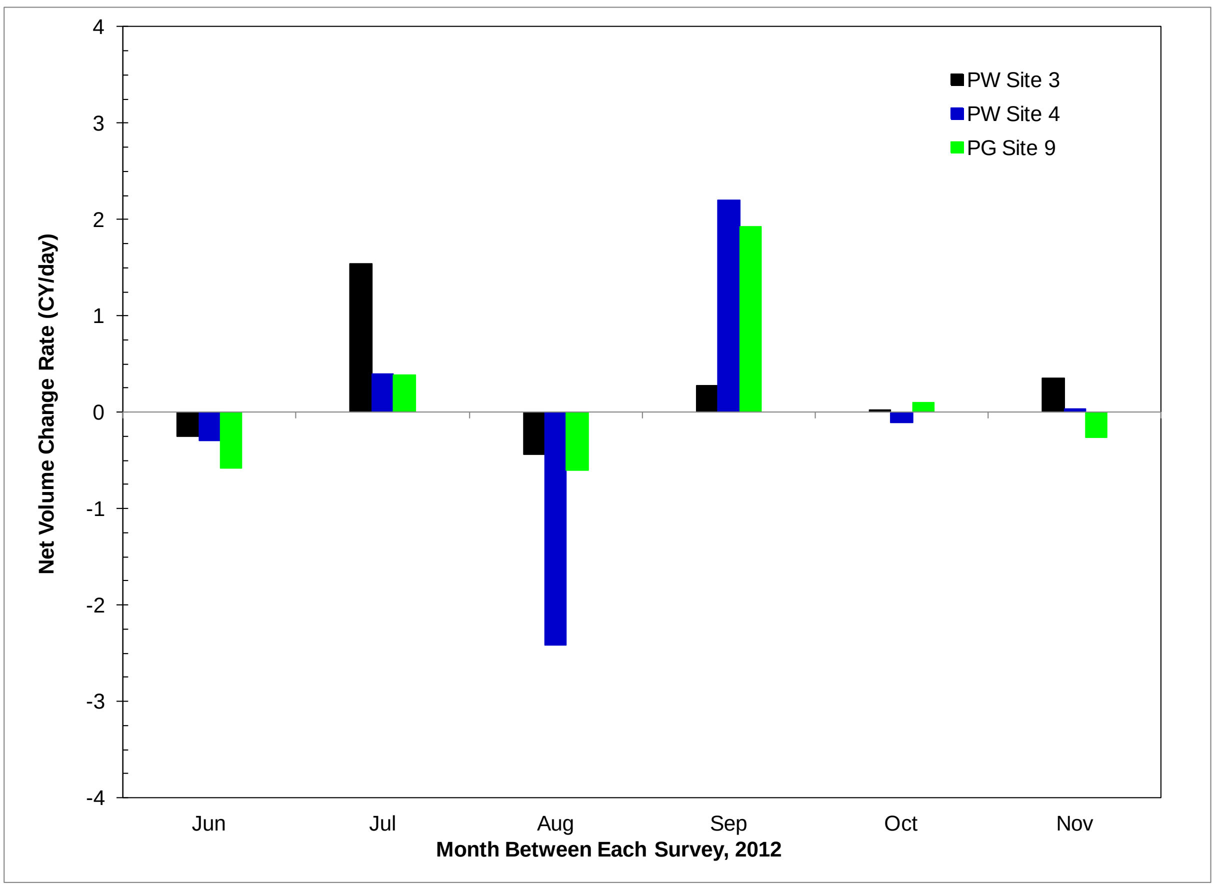

Figure 4-32: Calculated rates of volume change at all sites for each approximately month-long survey span during the Summer through Fall 2012 operations

Between 25 June and 31 August 2012, RP1 operated 8 one-way trips five days a week (40 trips per week). Between 4 September and 2 November 2012, RP1 operated 10 one-way trips five days a week (60 trips per week). The volume change calculated during these intervals is shown in Table 4-5. Site 4 and Site 9 show a similar pattern of net accretion during the more frequent (60 trips per week) sailing interval and net erosion during the less frequent (40 trips per week) sailing interval. Site 3 shows net accretion during both testing intervals with greater net accretion (0.61 CY/day) during the less frequent (40 trips per week) sailing interval. These results follow a similar pattern to the observed changes between sites described by the monthly data analysis presented above. No increase in erosion was observed at any sites during the interval of increased sailing frequency, and no uniform increase in accretion was observed at any of the three sites. The patterns of volume change are more closely correlated with the presence and absence of wind-wave events. No significant differences in cross-shore or alongshore volume exchange patterns were identified when comparing the two sailing frequency intervals.

Table 4-5: Calculated volume change for the two vessel sailing frequencies

	RP1 Operating 40 trips/week 25 Jun - 31 Aug 2012	**RP1 Operating 60 trips/week 4 Sep - 2 Nov 2012**
Survey Site	Volume Change (CY)	Volume Change Rate (CY/day)
Site 3	0.61	0.12
Site 4	-0.93	0.74
Site 9	-0.12	0.73

4.3 Structure Response and Beach-Structure Interactions

Regular beach observations provide an opportunity for photographic record of the condition of structures in the study area, as well as a measure of the relative scale of beach change at the toe of these structures. Beach photo observations were recorded on a similar timeline to beach profile and laser scan surveys during in-situ beach response testing of RP1. Selected sites characterizing the changes observed along each shoreline are discussed in this section, and the full set of beach photo time series is provided in Appendix F.

Changes in beach elevation along the southern portion of East Bremerton were small and within the error of the measurement (Figure 4-33). However, there was approximately a 1-ft increase in the beach elevation at the toe of the bulkheads at sites along the northern portion of East Bremerton between 25 June and 13 November 2012 (Figure 4-34). In general, the amount of accretion at the bulkhead increases from south to north along East Bremerton. No beach observations were recorded between 13 November 2012 and the date this report was issued to determine changes or reversals of this pattern after RP1 completed operations. However, beach elevation changes at this site typically fluctuate on the order of +/1 ft per year such that the elevation at the bulkhead increases through the summer and decreases in the winter (Figure 4-35). The patterns are consistent with a seasonal shift in transport direction driven by variations in the prevailing winds from southwest in winter to northeast in summer. There is a long-term alongshore drift of sediment from southwest to northeast along East Bremerton, where sediment is being eroded from the southern end and deposited on the northern end of this drift cell.

Beach photo observations recorded along Pleasant Beach, Point White, Port Orchard, and Point Glover indicate seasonal variability on the order of +/- 0.5 ft, but there were no significant changes in the elevation at the toe of the bulkhead during RP1 testing (Figures 4-36 through Figure 4-39).

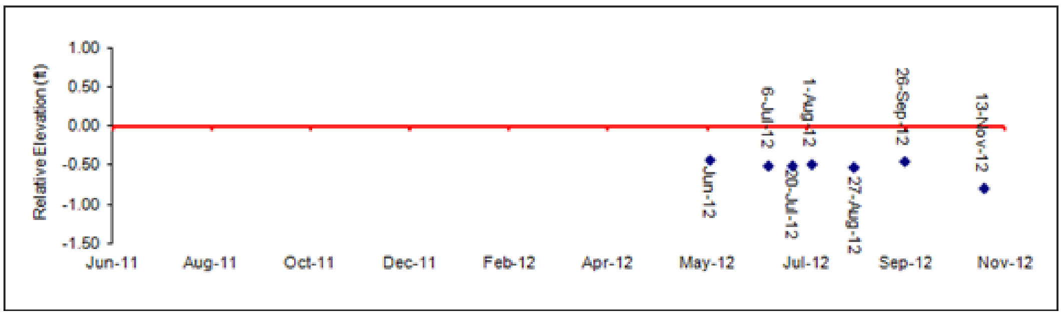

Figure 4-33: Beach elevation measured at the toe of the bulkhead relative to February 2005 at Site EB_01 on the southern end of East Bremerton

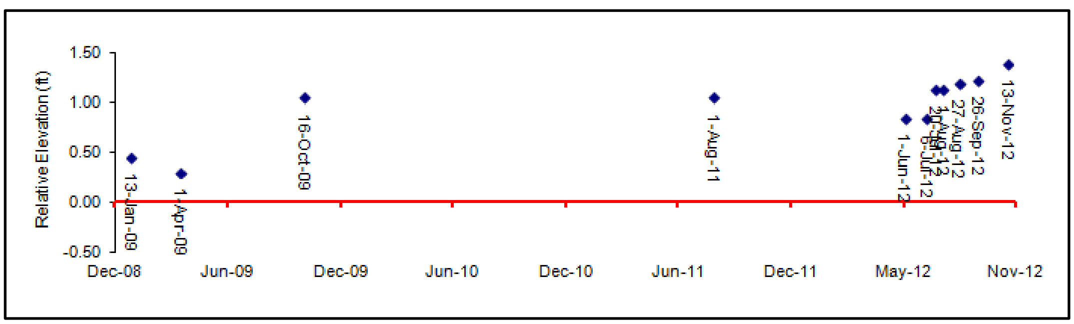

Figure 4-34: Beach elevation measured at the toe of the bulkhead from January 2009 to November 2012 relative to February 2005 at Site EB_16 on the southern end of East Bremerton

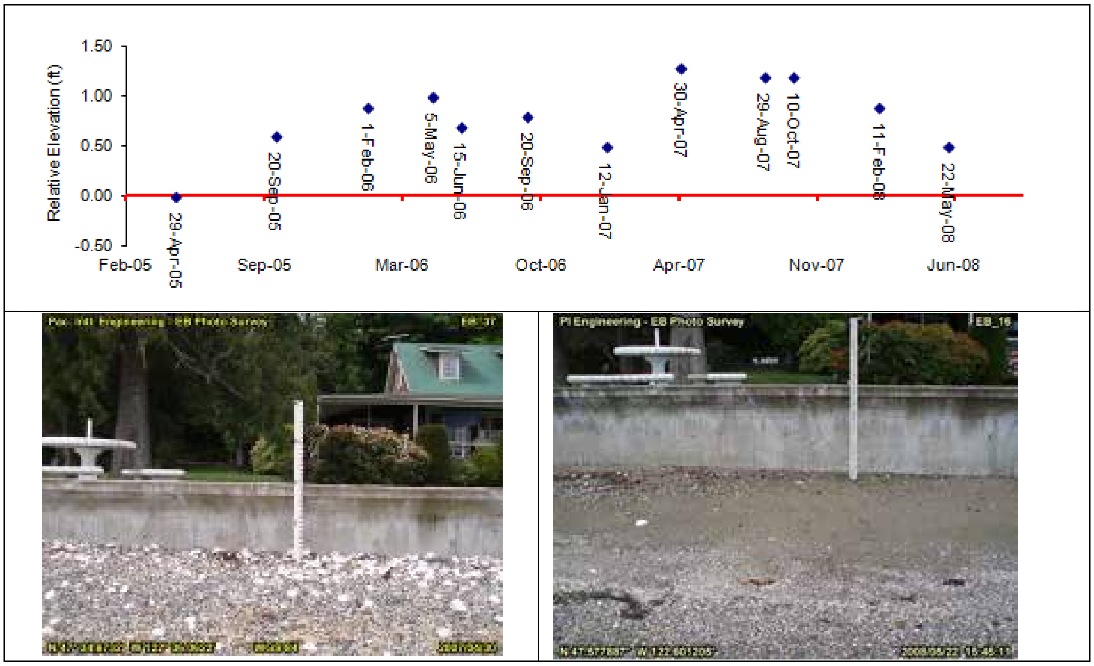

Figure 4-35: Beach elevation measured at the toe of the bulkhead from April 2005 to May 2008 relative to February 2005 at Site EB_16 on East Bremerton; Photograph of Site EB_16 on 29 April 2005 (bottom left) and photograph of Site EB_16 on 22 May 2008

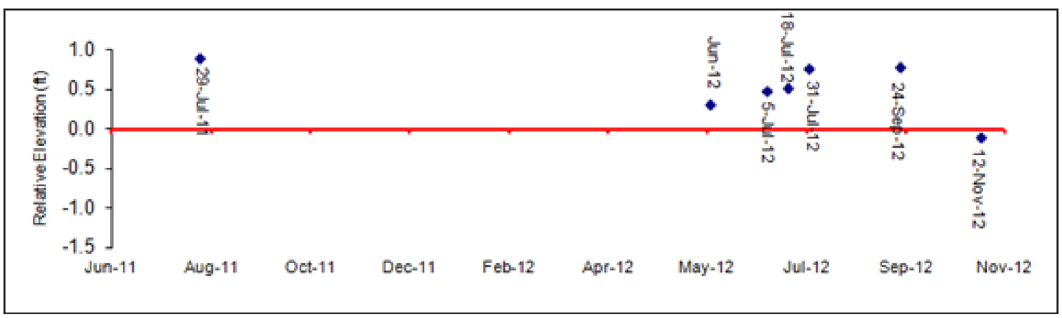

Figure 4-36: Beach elevation measured July 2011 to November 2012 at the toe of the bulkhead relative to February 2005 at Site PB_07 on the Pleasant Beach

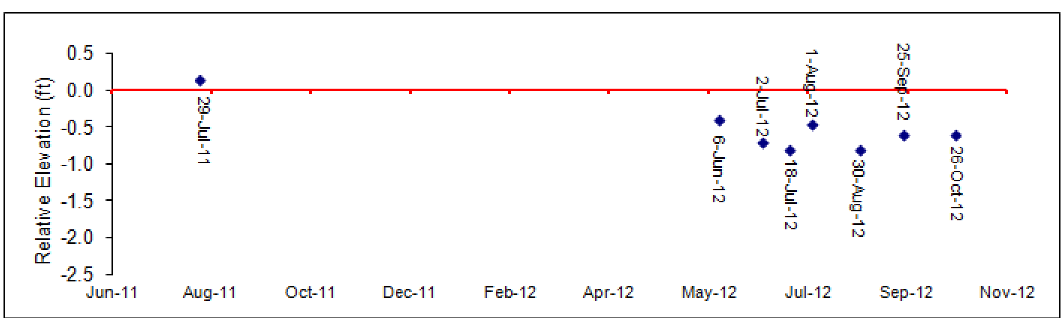

Figure 4-37: Beach elevation measured July 2011 to October 2012 at the toe of the bulkhead relative to February 2005 at Site PW_11 on Point White

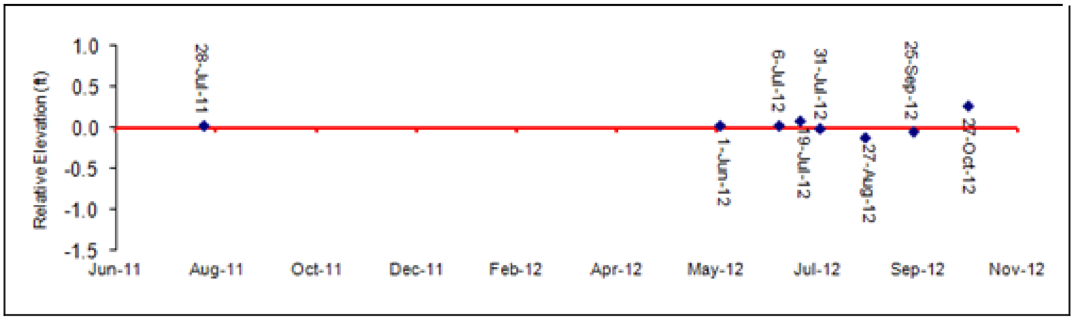

Figure 4-38: Beach elevation measured July 2011 to October 2012 at the toe of the bulkhead relative to February 2005 at Site PO_02 on Port Orchard

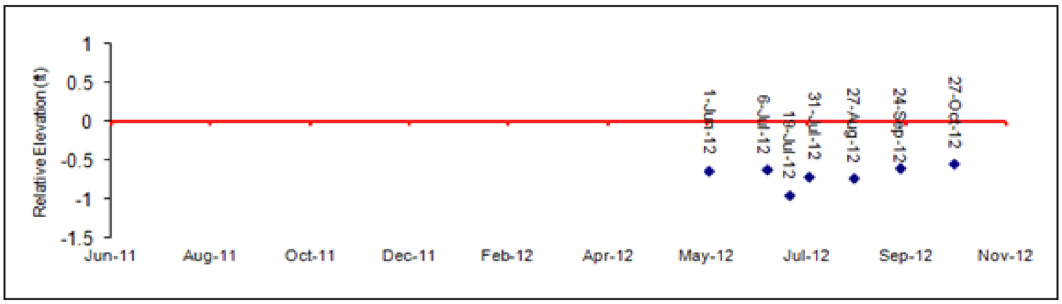

Figure 4-39: Beach elevation measured in June to October 2012 at the toe of the bulkhead relative to February 2005 at Site PG_07 on Point Glover

4.4 Overtopping

Three water levels were used in the 2007 overtopping analysis (PIE 2007b) to calculate the potential for wind-waves and vessel wakes of overtopping shoreline structures. Two water levels were analyzed for overtopping of fast ferry wake wash and were based on a frequency analysis of water levels during fast ferry operations in 2000 to 2001. The following water levels were examined for fast ferry wake wash:

- 99.0% of ferry traffic occurred below 11.90 ft MLLW
- 99.9% of ferry traffic occurred below 12.75 ft MLLW

The first water level, 11.90 ft MLLW, represents MHHW and the second water level, 12.75 ft MLLW, represents extreme high tide. The third water level condition was a 2-year return period water level coupled with the 2-year return period wind, which results in a 4-year return period event for wind-waves applied to the overtopping analysis.

In the 2012 analysis, local sea-level rise projections for 2050 were applied as an elevated water level in addition to the updated water level extremal analysis. A summary of the water level and wave scenarios used for the 2012 analysis is provided in Table 4-6. The UW CIG low impact (0.25 ft increase) and medium impact (0.5 ft increase) projections for 2050 were considered in the analysis. The low-probability, high-impact projection (1.83 ft increase) was not included in the analysis because 36% of the coastal defense infrastructure inventoried along Rich Passage have crest elevations of less than 15.4 ft MLLW (2-year return period water elevation + low probability, high impact SLR case), and therefore will require repair or modifications in order to prevent regular flooding due to water level alone. By comparison, only 2% of the structures have crest elevations less than the 14.07 ft MLLW (the 2-year return period water elevation + medium probability, medium impact SLR case).

Table 4-6: Water Levels and Wave Scenarios Applied to the Overtopping Analysis in Rich Passage

Water level description	Water level (ft, MLLW)	2050 SLR, UW CIG low impact (ft, MLLW)	2050 SLR, UW CIG medium impact (ft, MLLW)
Water level 99.0% of ferry traffic occurs below (RP1 wake wash only)	11.90	12.15	12.40
Water level 99.9% of ferry traffic occurs below (RP1 wake wash only)	12.75	13.00	13.25
2-year return period (coupled with 2-year wind-wave event)	13.57	13.82	14.07

For the overtopping calculation, an overtopping value was calculated for every surveyed point taken on a given bulkhead crest and was classified as “not overtopped (No OT)”, “minor overtopping (minor OT)” or “overtopped (OT)”. Minor OT is assessed for overtopping values greater than 0.0 ft and less than 0.1 ft above the crest; OT is assessed for values greater than or equal to 0.1 ft. For the purpose of this assessment a condition of overtopped structure (OT) was assigned when any of these points of each structure was overtopped. In other words, for some scenarios at certain locations, Figures 4-40 to 4-45 will show that the whole structure is overtopped when only a portion of the structure is overtopped.

Tables 4-7 to 4-10 provide the number of bulkheads per shoreline section that are at risk of overtopping for a given water level. The first two rows are for RP1 wake wash conditions and the last three rows are for a given wind-wave condition and water level. Appendix H provides detailed tables and maps of properties that are at risk for overtopping for each scenario.

Table 4-7: Number of bulkheads overtopping for various test conditions along Point White

Point White	Water Level (ft MLLW)			Water Level Incorporating Projections for 2050 SLR (ft MLLW)		
(38 Bulkheads)	11.9	12.75	13.57	12.4	13.25	14.07
S à B (RP1)	0	3	–	0	9	–
B à S (RP1)	0	0	–	0	7	–
N wind	0	0	3	0	0	3
SE wind	9	10	17	9	11	25
SW wind	2	13	29	8	21	30

Notes: Dash (-) indicates case not evaluated for RP1.

Table 4-8: Number of bulkheads overtopping for various test conditions along Pleasant Beach

Pleasant Beach	Water Level (ft MLLW)			Water Level Incorporating Projections for 2050 SLR (ft MLLW)		
(24 Bulkheads)	11.9	12.75	13.57	12.4	13.25	14.07
S à B (RP1)	0	0	–	0	0	–
B à S (RP1)	0	0	–	0	0	–
N wind	0	0	0	0	0	0
SE wind	5	13	22	8	21	24
SW wind	0	5	18	5	12	23

Notes: Dash (-) indicates case not evaluated for RP1.

Table 4-9: Number of bulkheads overtopping for various test conditions along Port Orchard

Port Orchard	Water Level (ft MLLW)			Water Level Incorporating Projections for 2050 SLR (ft MLLW)		
(92 Bulkheads)	11.9	12.75	13.57	12.4	13.25	14.07
S à B (RP1)	0	2	–	1	4	–
B à S (RP1)	0	1	–	0	4	–
N wind	0	2	10	0	6	17
SE wind	1	10	27	2	22	32
SW wind	1	10	19	3	16	26

Notes: Dash (-) indicates case not evaluated for RP1.

Table 4-10: Number of bulkheads overtopping for various test conditions along Bremerton

Bremerton	Water Level (ft MLLW)			Water Level Incorporating Projections for 2050 SLR (ft MLLW)		
(80 Bulkheads)	11.9	12.75	13.57	12.4	13.25	14.07
S à B (RP1)	0	0	–	0	1	–
B à S (RP1)	0	0	–	0	1	–
N wind	0	1	10	0	4	16
SE wind	0	3	16	0	12	28
SW wind	0	5	21	3	18	38

Notes: Dash (-) indicates case not evaluated for RP1.

The results of the 2012 bulkhead overtopping analysis indicate that there are very few structures susceptible to overtopping from RP1 wake wash at the existing water levels. The amount of overtopping caused by RP1 is less than the amount caused by a 2-year return period wind wave event (33 kt) for a given water level. The number of bulkheads susceptible to overtopping increases for both RP1 wake wash and the wind-wave scenarios when projections of sea level rise are considered in the analysis. The number of properties predicted to be overtopped by RP1 at present water level conditions is 2; this increases to 14 when the medium impact SLR scenario water levels are considered for 2050.

4.4.1 Point White

The overtopping results along the Point White shoreline section showed that the SW and SE wind waves scenarios produce the largest overtopping impact, since these directions have the longest fetch with respect to the shoreline alignment. The analysis shows that for RP1 (traveling from Seattle to Bremerton) wake wash at the 12.75 ft MLLW water level, three bulkheads, shown in Figure 4-40, were predicted to overtop (3698, 3780 and 3804 Point White Dr.). Also, these bulkheads are predicted to be overtopped for water level alone at or above 13.57 ft MLLW. These three are shown in in Table 4-7 in the case for wind-waves from the north; although this shoreline is not exposed to the north wind-waves, the crests are lower than 13.57 ft MLLW for each of the three bulkheads.

Figure 4-40: Overtopped bulkheads from RP1 wake wash at 12.75 ft MLLW water level. From left to right: 3698, 3780 and 3804 at Point White Dr.

From the results, three additional bulkheads, shown in Figure 4-41, are predicted to overtop at 13.25 ft MLLW from RP1 (traveling in both directions) wake wash scenarios; they are located at 3192, 3204 and 3284 Point White Dr.

Figure 4-41: Overtopped bulkheads from RP1 wake wash at 13.25 ft MLLW water level. From left to right: 3192, 3204 and 3284 at Point White Dr.

4.4.2 Port Orchard & Point Glover

Only two bulkheads on properties (6601 Beach Dr. E. and 3262 Beach Dr. E.) along Port Orchard and Point Glover were calculated to be susceptible to overtopping from RP1 at the 12.75 ft MLLW water level (Figure 4-42). Both of these properties were also predicted to be overtopped by *Spirit* in the earlier analysis. The bulkhead analyzed at 3262 Beach Dr. E. is backed by a second, higher bulkhead which is of sufficient height to protect the house on the property; thus, overtopping does not pose a significant threat to the upland property. At lower water levels no overtopping is predicted.

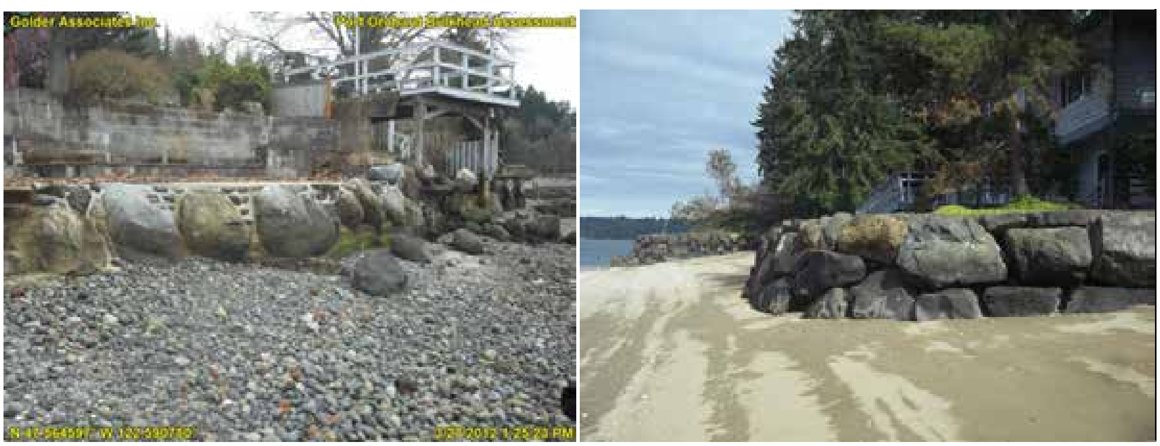

Figure 4-42: Overtopped bulkheads from RP1 at 12.75 ft MLLW water level. From left to right: 3262 and 6601 Beach Dr E.

Two additional bulkheads are predicted to be overtopped at 13.25 ft MLLW; a water level which considers projections for SLR. The bulkheads are located at 6613/6671 Beach Dr. E. and 6089/6101 Watauga Beach Dr. E., shown in Figure 4-43. The bulkhead at 6089/6101 Watauga Beach Dr. E. is a vertical cast in-place concrete retaining wall with no toe and protrudes onto the inter-tidal beach. It has been closely monitored because it is located on the inside curve of the RP1 sailing line and waves tend to arrive from two directions along the vessel sailing line when RP1 travels to Bremerton. Overtopping could pose a potential problem at these properties, and should be closely monitored. The bulkheads at 6613 and 6671 Beach Dr. E. are located next to the bulkhead at 6601 Beach Dr. E, which is overtopped by wakes from

RP1 at a water level of 12.75 ft MLLW. These three bulkheads have good drainage and heavy vegetation behind them, suggesting that occasional overtopping could represent very low risk to damage of the structure.

Figure 4-43: Overtopped bulkheads from SLR projection at 13.25 ft MLLW. From left to right: 6089/6101, 6613, 6671 Beach Dr E.

4.4.3 Pleasant Beach

Very few properties on Pleasant Beach are susceptible to overtopping. No overtopping is calculated from RP1 wakes for any water level condition. The overtopping results showed that for a wind speed of 33 kts from the SE or SW, a significant number of bulkheads along Pleasant Beach will experience overtopping during elevated water levels. Some of the lowest crested bulkheads overtopped during the SE or SW wind-wave scenarios are shown in Figure 4-44.

Figure 4-44: Examples of overtopped bulkheads from SE and SW wind waves. From left to right: 3385, 3861 and 3863 located at Pt. White Dr.

4.4.4 Bremerton

Very little overtopping is expected along this section of shoreline. The analysis showed that one bulkhead (1624 Jacobsen Blvd.) shown in Figure 4-45, is predicted to overtop at 13.25 ft MLLW for RP1 traveling in both directions. It is a low-crested cast in-place concrete retaining wall, and the presence of logs on the top of the bulkhead could be indicative of attempts to protect the property from overtopping. Generally, greater levels of overtopping will occur from a 33 kt wind than from the RP1 ferry wake wash at a given water level.

Figure 4-45: Overtopped bulkhead from RP1 at 13.25 ft MLLW (1624 Jacobsen Blvd).

4.5 Noise

Measurements of noise recorded on 26 June 2012 from RP1 and ambient conditions in Rich Passage are provided in Table 4-11.

RP1 noise levels were measured twice at Point White: once when RP1 was traveling towards Seattle and once when it was traveling towards Bremerton. The noise levels exceeded the 55-dB threshold for a maximum of 167 seconds, but only exceeded 70 dB for 16 seconds. The maximum allowable environmental noise as defined by Washington State must not exceed 70 db for more than 90 seconds. Therefore, the noise generated by RP1 and received at Class A properties is below the Washington State maximum allowable limits.

Figure 4-46 shows a plot of noise level versus time for three different vessels, and includes separate measurements for the RP1 travelling Seattle-bound and Bremerton-bound. Although the noise generated by RP1 was higher than the other vessels when measured at Point White, a maximum noise of 72.5 dB was received from Bremerton-bound Kaleetan along Point Glover.

Table 4-11: Summary of noise level measurements in Rich Passage; duration of the measurement at various decibel levels above the 55-dB threshold

Site	Vessel	Length of Observations (s)	Maximum Noise Level (dB)	Duration of obs >55 dB (s)	Duration of obs >60 dB (s)	Duration of obs >65 dB (s)	Duration of obs >70 dB (s)
Point Glover	Ambient 1	51	52	0	0	0	0
Point White	Ambient 2	61	47.5	0	0	0	0
Point White	Ambient 3	67	53.5	0	0	0	0
Point White	RP1 BRE-SEA	100	69.3	84	52	25	0
Point White	RP1 SEA-BRE	267	71	167	91	21	3

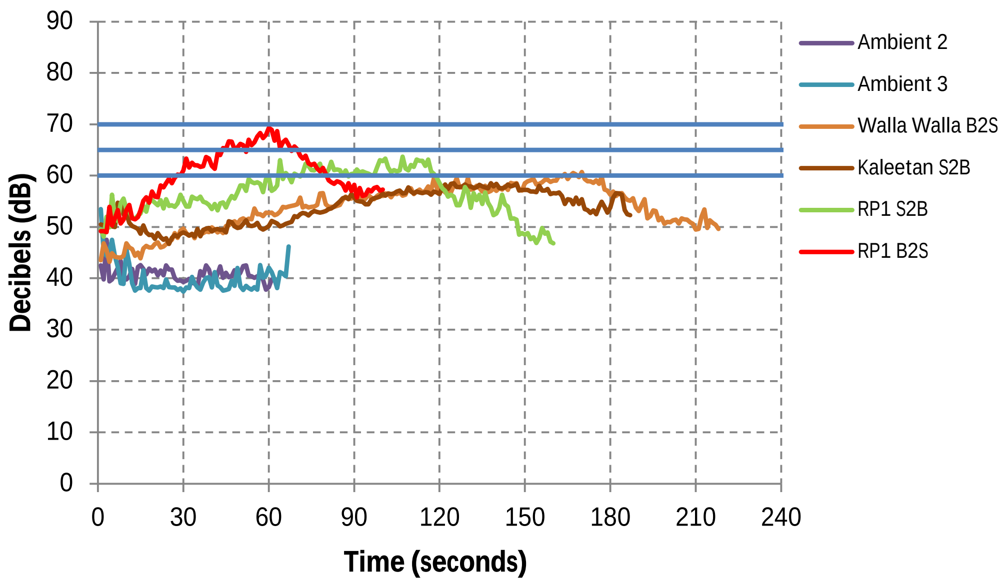

Figure 4-46: Measurements of noise levels near Site 3 (Point White) for various vessels and ambient conditions in Rich Passage on 26 June 2012

5.0 CONCLUSIONS AND GUIDELINES FOR FUTURE POFF OPERATIONS

This section discusses the observations of the beach response as related to site-specific factors, seasonality, and RP1 contributing factors (distance from the sailing line, vessel speed, and frequency of sailings), along the primary shorelines in the study area: East Bremerton, Point Glover, Pleasant Beach, Port Orchard, and Point White. The section also discusses effects of sailing frequency, and provides recommendations for future POFF operations.

The hydrodynamic forcing within Rich Passage is site-specific based on local bathymetry and exposure of the site to vessel wake wash, tidal currents, and wind-waves. The beach responses to these forces are also site-specific based on the dominant hydrodynamic forcing mechanism affecting the beach, the sediment grain size distribution, sediment supply, and beach slope. During RP1 testing, the morphological response of the beaches within Rich Passage varied temporally and varied spatially from shoreline to shoreline as well as between specific sites within each shoreline. However, the magnitude of response observed is within the seasonal and inter-annual beach response that already occurs at each site. Furthermore, much of the variability in beach response that occurred during testing appears to be attributable to variations in wind-generated waves, tidal currents, and stream run-off.

RP1 testing was conducted primarily during summer months (July through September), with only one month of testing during the winter season (October). It has been previously hypothesized that wake wash energy is more likely to dominate the beach response during the summer as compared to wind-wave energy, which is expected to dominate in winter (PIE 2005, 2007a). The more intensive beach observations during the RP1 testing program reveal a sensitivity of the beach response to variability in forcing that was not previously revealed by less intensive quarterly monitoring. Although some sites showed distinctive patterns of response during the testing interval that are characteristic of POFF wake-wash forcing, there are also marked temporal variations in beach response observed at the sites can only be attributed to variations in baseline forces (e.g., wind-waves and tidal currents).

During previous studies, authors hypothesized that proximity to the sailing line played a significant role in how beaches responded to vessel wake wash (PIE 2007a, 2007b). This study's in-situ beach response testing found very little direct correlation between beach response and distance from RP1's sailing line. However, the amount of wake wash energy received at the shoreline was correlated with the curvature of the sailing line; beaches along the straighter portions of the sailing line and on the outside curvature of the sailing line generally receive lower energy wake wash than beaches along the inside curve of the sailing line. These curved areas that focus the wake wash may be susceptible to wake-structure interactions including wave loading, toe scour and overtopping.

Vessel speed can also affect the energy of wake wash. Sections of shoreline along portions of the fast ferry route where the vessel would be accelerating, decelerating, or traveling at the vessel hump speed may be subject to increased energy. Wake acceptance testing showed that RP1 would meet the wake acceptance criterion at speed of 32 to 40 kts; however, the optimal speeds to minimize wake wash were 36 kts and greater (Golder 2013b).

Along the East Bremerton shoreline, operating RP1 at non-optimal speeds as well as acceleration and deceleration through Port Orchard Reach created more significant wake wash and beach response than anticipated. During the testing interval, the elevation of the beach increased at the toe of the bulkhead and the beach slope flattened, due to larger volumes of sediment accretion on the lower beach than the upper beach. The magnitude of this beach response along East Bremerton was within the seasonal changes observed during the baseline interval; however, the recommended restrictions on POFF speeds and operations within Rich Passage should also be implemented within the Port Orchard reach, to decrease the potential for long-term cumulative affects along East Bremerton shorelines.

At Point Glover, differences in laser scanning surface maps suggest the Point is dominated by alongshore transport patterns, which are primarily confined between groins and promontories. In general, the transport is northeast to southwest in the summer and southwest to northeast in the winter. However, these transport patterns may reverse in the short-term based on the presence or absence of wind-wave energy and wind direction. Point Glover is more susceptible to sediment transport from wind-waves out of the southeast aligned with the eastern entrance to Rich Passage (originating in central Puget Sound). Although there was no direct correlation between beach response and vessel wake wash from RP1 along Point Glover, the wake wash measurements indicate RP1 generates more power travelling from Seattle to Bremerton than vice versa, which could increase the net transport from northeast to southwest along this shoreline. Additional laser scanning surveys are planned for the Spring and Fall of 2013 to measure the beach volume change in the absence of RP1 and document baseline changes in transport volumes and patterns along Point Glover.

Beach response along Pleasant Beach and Port Orchard appears to be more closely associated with an increase in wind speeds and a shift in wind direction (from northeast to south-southwest in October) than the start and end of RP1 operations. Pleasant Beach is characterized by relatively straight shorelines parallel to the sailing line and on the outside curvature of the sailing line. These shorelines typically experience minimal wake wash energy due to longer distances from the vessel sailing line, wave dispersion or spreading, and flatter beach slopes with extensive sub-tidal shoals which dissipate more wave energy before the waves reach the shoreline. Although the beach response to RP1 along Port Orchard was insignificant, the potential for long-term cumulative beach response on this shoreline will be minimized by limiting the total number of POFF weekly transits along Port Orchard reach.

At most sites along Point White, beach elevations have been decreasing overall, and beach volumes have been steadily declining since 2004; these changes are unrelated to POFF operations. Although the beach response during RP1 testing varied from site to site along Point White, the changes were not significant in comparison to the seasonal trends, inter-annual variability, and the long-term trends in beach elevation and volume. No systematic trend in beach volume change along Point White was observed during RP1 operations, and therefore no net increase or decrease in beach volumes occurred. Site-specific beach response may reflect differences in shoreline orientation. The change in Point White's shoreline orientation between approximately Site 3 and Site 4 creates a difference in the exposure of the sites, particularly to wave-driven transport. In general, Site 3 is more susceptible to wind-waves out of the southwest aligned with the western entrance to Rich Passage, while Sites 4 is more susceptible to transport from wind-waves out of the southeast aligned with the eastern entrance to Rich Passage.

Results from the overtopping analysis indicate that a few structures are susceptible to overtopping from RP1 wake wash, but only at extreme high tides of 12.75 ft MLLW or greater. The amount of overtopping discharge caused by RP1 is less than the amount caused by a 2-year return period wind event at extreme high tide and 2 year return period water levels. The number of bulkheads susceptible to overtopping increases for both RP1 waves and the wind-wave scenarios when projections of sea level rise are considered in the analysis. The number of properties predicted to be overtopped by RP1 at existing high water level conditions is 2; this increases to 14 when the medium impact SLR scenario water levels are considered for 2050. Comparison of the frequency of RP1 sailings (between RP1 testing intervals with 40 trips per week and those with 60 trips per week) showed there was no trend in sediment transport patterns or rates directly correlated to the increase in the number of sailings. In general, the patterns of volume change are more closely correlated to the presence and absence of wind-wave events than to the number of vessel sailings. However, as noted above, several sites did exhibit a subtle (small magnitude) but characteristic response to both POFF operations and cessation of operations. However, the research results suggest that the potential for long term effects from future operations that are consistent with the test program operations appears to be insignificant. Planning for future POFF operations with RP1 or equivalent should consider the potential for long-term cumulative effects if more intensive (frequent) transits than were tested in the research program are required. The following sections outline several guidelines and criteria for consideration that are intended to minimize the potential for such effects.

5.1 Ongoing Monitoring

The beaches in Rich Passage are changing on a seasonal and inter-annual basis due to the combined effects of natural forcing, wakes from car ferries, and structural modifications that may limit the supply of sediments. Continued periodic monitoring of beach volume change at selected locations is important, to provide an ongoing baseline for comparison with any future POFF operations. The following guidelines outline a minimum monitoring program to be maintained:

- Perform laser scanning surveys on a semi-annual basis at one site each on the following four shorelines: East Bremerton, Point White, Pleasant Beach, and Point Glover. This reduced monitoring program will continue documenting seasonal and inter-annual variability.

- Take beach photo observations quarterly, to provide more extensive spatial coverage.
- Perform biological surveys in Fall 2013, repeating previous work, to provide a post-operations survey of biological communities and to document a lag in response from RP1 operations.
- Continue communications with property owners; this includes maintaining current contacts, and providing a vehicle for communication and response to concerns.

5.2 Guidelines for POFF Operations

The variability and magnitude of wake wash and beach response can be reduced through operations planning and effective implementation. The following guidelines are intended to minimize the potential for long-term beach response to POFF operations:

- Limit initial operations with a POFF vessel such as RP1 or equivalent to previously tested conditions (60 trips per week) for at least 3 months then increase the frequency of trips. Operations and modifications to operations should be accompanied by a beach observation and monitoring program.
- Begin operations during a month that does not typically correspond to a seasonal shift (i.e., avoid starting in June or October).
- Pre-program HFMS with optimized settings for three loadings, rather than two loadings as was done during in-situ beach response testing.
- Operate RP1 at optimal speed (36 to 40 kts) starting as close to Bremerton as possible rather than waiting until the vessel reaches Rich Passage), and avoid accelerating and decelerating in Port Orchard Reach and Rich Passage.
- Monitor beach response using laser scanning surveys and beach photo observations monthly for first 3 months and quarterly thereafter.

6.0 CLOSING

This is a final report being submitted to Kitsap Transit for review and comment. The content of this final report will be made publicly available for comment.

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7.0 GLOSSARY

Accretion. The gradual buildup of beach or land by sediment deposition or accumulation. Synonym: aggradation; Antonym: *erosion*.

Acoustic Surface Tracking (AST). A vertically oriented acoustic beam on an AWAC that tracks the water surface elevation directly.

Acoustic Wave and Current (AWAC). An oceanographic instrument used for acoustically measuring waves and water current profiles.

Alongshore. The direction parallel to the shoreline.

Bathymetry. The underwater equivalent to topography. Bathymetric charts are designed to present accurate, measurable description and visual presentation of the submerged terrain.

Beach Morphology. The external structure, shape, or form of a beach.

Beach Morphology Analysis Package (BMAP). Software package used for analysis of cross-sectional beach volumes.

Benthic. Relating to the ecological region at the lowest level of a body of water, including the organisms that live there. *.*

Bulkhead. For this report a bulkhead is generally defined as any shore-parallel, man-made structure used to prevent waves and water from intruding beyond the beach area. Includes seawalls, retaining walls, and revetments.

Cross-shore. The direction perpendicular to the shoreline.

Crest Elevation. The elevation of the uppermost portion of a shore structure.

Decibel. Unit of measurement for sound level above a standard reference level.

Drift Cell (Littoral). Discrete divisions of a shoreline that are independent of one another and for which distinct sediment sources and sinks can be identified.

Draft. The vertical distance between the waterline and the bottom of the vessel hull.

Extremal Analysis. A statistical method dealing with the extreme deviations from the median of probability distributions. It seeks to assess, from a given ordered sample of a given random variable, the probability of events that are more extreme than any previously observed.

Erosion. The gradual lowering or loss of beach or land by sediment weathering, removal, or transport to another location by natural forces (current, waves, wind). Synonym: degradation; Antonym: *accretion*.

Foil. Wing-like structure mounted under the hull of a vessel. As the vessel speed increases the foil helps lift the hull out of the water and reduce the wetted area and drag of the vessel.

Heave. The vertical (up/down) motion of a vessel.

Hertz (Hz). A frequency unit (cycles/second).

Hull and Foil Monitoring System (HFMS). The system installed on RP1 to record vessel data parameters.

Hump speed. For any given vessel and constant water depth, an increase in speed will lead to an increase in the height of the maximum wave in a wake train (H_{max}) up to a certain speed. Beyond that speed, the maximum wave height will decrease. The speed at which H_{max} occurs is often referred to as the *hump* speed. The hump occurs when the ship produces a wake with a wavelength that is one-half the length of the ship.

Hydrodynamic Forcing. Application of liquid flow and pressure energy over a known distance or area. Examples of this include waves, waves, and tides.

Laser Scan Surveys. Surveys using a scanning laser to measure high resolution three-dimensional data of the terrain.

Littoral Transport or Drift. The process by which beach sediment is moved along the shoreline. Drift results primarily from the oblique approach of wind-generated waves and can therefore change in response to short-term (daily, weekly, or seasonal) shifts in wind direction.

Lagrangian Super - Critical Vessel (LSV) model. A two-dimensional model that provides a description of wake wash properties in plan view.

Interceptor. Transom-mounted blades that are hydraulically actuated and controlled in a vertical direction. The interceptor generates an increase in pressure on the hull bottom directly ahead of the transom by intercepting the water flow in continuous proportion to vessel motions. The resulting forces make it possible to simultaneously damp both roll and pitch motions, and optimize running trim, by automatically varying the deployment of each interceptor in response to vessel motion.

Macrophyte. An aquatic plant that grows in or near water and is either partially underwater, entirely underwater, or floating. Examples include macroalgae and eelgrass and bull kelp.

Mask Boundary. A region or area defined by a data analyst to determine the selection area or subset of a dataset being analyzed. Only data within the mask boundary are selected and applied for the specific calculation.

Maximum Wave Height (H_{max}). The maximum wave height in a collection of waves.

Mean Lower Low Water (MLLW). A tidal datum. The average of the lower low water height of each tidal day observed over the National Tidal Datum Epoch.

Mean Tide Level (MTL). A tidal datum. The average of mean high water and mean low water.

Morphology Change. The change in a beach's shape or form over time.

Wave Overtopping. Occurs when waves meet a structure and water passes over the crest of the structure.

Passive Integrated Transponder (PIT). A type of RFID tag that is inductively charged by the reader and does not have a battery.

Peak Frequency. The spectrally determined frequency of peak energy in group of waves.

Pitch. An angular measurement of rotation about the lateral axis of a vessel.

Quadrat. A plastic or metal square (in this case, 0.25 m^2) used in ecology to isolate a sample for quantitative observations during surveying.

Radio Frequency Identification (RFID). The use of a wireless non-contact system that uses radio-frequency electromagnetic fields to transfer data from a tag attached to an object, for the purposes of automatic identification and tracking.

Real Time Kinematic – Global Positioning System (RTK GPS). A satellite navigation technique used to enhance the precision of position data derived from satellite-based positioning systems. It relies on a single reference station to provide real-time corrections, providing up to centimeter -level accuracy.

Roll. An angular measurement of rotation about the longitudinal axis of a vessel.

Sediment Tracers. Particles similar or the same as native sediment particles which are released into the environment and tracked through time to measure transport patterns.

Scour. Localized erosion at the toe of a structure.

Sediment Budget. The mass balance of sediments or the sum of the sediment input and output within a defined region or area.

Surface Gravity Wave. Waves generated at the interface between the atmosphere and the free surface of ponds, lakes, seas, or oceans.

Toe Elevation. Refers to the elevation where a bulkhead intersects the beach.

Transom. The surface that forms the stern of a vessel.

Tracer Centroid. Geometric center of sediment tracers; calculated as the arithmetic mean of all the tracer coordinates.

Trim. The difference between the forward draft and aft draft of a vessel.

Wavelength. The horizontal distance between a wave's crest and its trough.

Wave Energy. A function of the wave height squared times the wavelength.

Wave Height. The vertical distance between a wave's crest and its trough.

Wave Period. The time it takes for a single wavelength to pass a stationary position.

Wave Power. The propagation of wave energy in time. Also known as wave energy flux.

Wake Wash. A collection of surface gravity waves caused by the displacement of water by a passing vessel.

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