HYDRODYNAMIC AND SEDIMENT TRANSPORT MODELING STUDY – MUKILTEO FERRY TERMINAL

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COAST & HARBOR ENGINEERING

Technical Report

Hydrodynamic and Sediment Transport Modeling Study – Mukilteo Ferry Terminal

Executive Summary

A numerical modeling study and engineering analysis were conducted to evaluate possible changes to bottom scour and sediment transport along the Mukilteo shoreline resulting from four proposed modification or relocation alternatives of the Mukilteo Ferry Terminal and due to proposed removal of the existing Tank Farm Pier. Numerical modeling and analysis were also conducted to evaluate possible impacts on water salinity at the NOAA (National Oceanic and Atmospheric Administration) Fisheries Service seawater intake due to the proposed alternatives.

The project alternatives were provided by BergerABAM with details sufficient for evaluation by modeling and analysis. These alternatives are identified as: No-Build; Existing Site Improvements; Elliot Point 1; and Elliot Point 2. A slightly modified version of Elliot Point 2 Alternative has been selected as the Preferred Alternative. Analyses presented for the Elliot Point 2 Alternative in this report apply to the Preferred Alternative.

Data for wind, waves, currents, littoral processes, geomorphology, and water salinity were compiled from available sources and processed to quantify existing conditions and to develop a sufficient database for numerical modeling and analysis. Hydrodynamic, littoral, and geomorphic conditions vary along the Mukilteo shoreline, resulting in diverse coastal features that are complicated by artificial structures. To describe the shoreline conditions in the project area and objectively identify possible impacts associated with the various project alternatives, the Mukilteo shoreline was divided into three reaches, each with similar littoral and geomorphic conditions. The existing littoral and geomorphic conditions at each of these reaches and possible impact on these conditions due to the proposed ferry terminal alternatives are characterized and described in this report.

Under the current study, the possible changes to bottom scour and sediment transport from propwash, vessel wakes, and wind-wave effects were investigated. Propwash was analyzed with numerical modeling to determine and quantify the potential effects from ferry propeller-induced currents on nearby bottom slope and bottom sediments at the proposed ferry terminal alternative locations. Propwash was modeled in several steps using the reliable and industry-accepted 2-D numerical model JETWASH.

Results of propwash and scour depth modeling for all alternatives are summarized in Table ES-1. Modeling for the No-Build Alternative showed that a relatively small amount of scour (depth approximately 2.9 ft) may occur at the bottom slope at the natural depth of approximately 20 to 25 ft below MLLW. This scour would be localized and would not produce any measurable adverse impact on shore erosion at the existing Mukilteo terminal.

Localized scour with a maximum depth of approximately 4.5 ft may develop for the Existing Site Improvements Alternative at the natural depth of approximately 15 to 20 ft below MLLW. This scour area would be located seaward from the limit of profile change by littoral processes, and would not produce any measurable adverse impact on shore erosion at the Mukilteo Terminal.

Modeling results also showed that localized scour (maximum depth approximately 4.5 ft) may develop at the bottom slope at the Elliot Point 1 Alternative. The possible location of a scour hole and its distance from the shoreline may result in adverse, but localized, impact on shore erosion in the vicinity of the terminal area.

Modeling results and analysis show that a small and localized scour area with a maximum depth of 1.4 ft may develop at the lower part of the slope for the Elliot Point 2 Alternative. This scour would occur at the natural depth of approximately 20 to 25 ft MLLW. The small depth of the scour and location at the natural depth of more than 20 ft preclude any detectable impact from this scour on the adjacent shoreline. Bottom scour by propwash at either of the Elliot Point alternatives can be easily controlled by placement of coarser sediment of the size to resist movement by the calculated maximum bottom velocity.

Table ES-1. Results of propwash and scour depth modeling for all alternatives

Alternative	Maximum Scour Depth (ft)	Location of Scour Depth on the Bottom Slope	Possible Effect on Shoreline Erosion
No-Build	2.9	20 to 25 ft MLLW	No detectable impact
Existing Site Improvements	4.5	15 to 20 ft MLLW	No detectable impact
Elliot Point 1	4.5	10 to 15 ft MLLW	Localized impact
Elliot Point 2	1.4	20 to 25 ft MLLW	No detectable impact

Vessel wakes were modeled and analyzed to determine and quantify potential effects on shore erosion from ferry wake wash generated at locations of the four terminal alternatives. These potential erosion effects are defined based on modeled changes of vessel wake and sediment transport parameters at the shoreline relative to existing conditions. Vessel wake parameters are specified in the analysis through spectral moment wake height and peak wake period in ferry wake trains. Sediment transport parameters are investigated through potential swash sediment transport characteristics. (Swash is the oscillating flow from wave action on the beach face, and the transport parameter indicates a potential shoreline response to change of wave or wake energy delivered to the shoreline).

Modeling and analysis has determined that ferries of both the existing Issaquah and proposed New 144 Auto Classes generate wake wash that arrives at the shoreline with relatively small amounts of energy for all modeled alternatives. The maximum spectral moment wake height to reach the shoreline is not expected to exceed 0.60 ft. In comparison, wave heights during yearly storms are estimated to be in the range of 2 ft to 3 ft.

The New 144 Auto Class ferry would generate smaller wake wash than the Issaquah Class Ferry. Therefore, a smaller amount of wake energy than is currently generated by the existing ferry would be delivered to the shoreline if the New 144 Auto Class Ferry replaces the Issaquah Class Ferry.

As expected, the amount of wake wash energy and potential sediment transport in the swash zone at the shoreline is directly correlated to the location of the ferry terminal and to the presence of the Tank Farm Pier that provides a sheltering effect from the ferry wakes. The Existing Site Improvements Alternative has the greatest increase in wake wash energy (compared to existing conditions) in one reach (Reach 1) of the shoreline. However, the Existing Site Improvements Alternative would also result in a reduction of vessel wake energy relative to existing conditions at the other two reaches of the shoreline (Reaches 2 and 3). No impact or increase of vessel wake energy is expected for the Elliot Point 1 or Elliot Point 2 Alternative, excluding the area of shoreline that was originally sheltered by the pier. Local increase of wake energy in this area would occur and would be predominately due to removal of the Tank Farm Pier.

Analysis and numerical modeling were conducted to determine and quantify potential changes in longshore sediment transport trends along the shoreline that may happen upon removal of the existing Tank Farm Pier. Possible changes to longshore sediment transport are identified here through changes of wind-wave patterns and potential longshore sediment transport components. Analysis and modeling showed that removal of the Tank Farm Pier would cause a measurable localized change of wave conditions at the shoreline to the east of the pier from storms that arrive from the west (260 deg N) and northwest (310 deg N) directions. Removing the Tank Farm Pier would change wave patterns along the shoreline for approximately 1,800 ft to the east of the pier.

The analysis also showed that removal of the Tank Farm Pier may cause a localized increase of potential longshore sediment transport at the shoreline and possible erosion of shoreline sediment to the east of the pier. It is likely that the extent of pier removal impact on potential longshore sediment transport would be limited within approximately 1,400 ft from the head of the pier.

A conclusion from numerical modeling and analysis is that removal of the Tank Farm Pier will result in some mobilization of fine bottom sediments in the mound beneath the pier in the event a 10-year return period or larger storm occurs. However, because the sediment composing the mound includes a significant amount of coarse size sediment, an armoring layer with coarse sand and gravel would form on the surface that would limit the depth of scour. If no extreme storm occurs, it is likely that the rate of erosion would be slow and probably would not be detectable during the first 5 to10 years.

Three-dimensional (3-D) flow velocity numerical modeling has demonstrated no detectable velocity change relative to existing conditions at the NOAA water intake caused by ferry operations (for both inbound and outbound ferry runs) at any of the proposed alternative modifications of the terminal. The same conclusions regarding salinity were made from analysis of 3-D salinity modeling. Results of salinity modeling in the project area have demonstrated that there would be no detectable impact on salinity at the NOAA water intake from any of the proposed terminal modification alternatives.

COAST & HARBOR ENGINEERING

Technical Report

Hydrodynamic and Sediment Transport Modeling Study – Mukilteo Ferry Terminal

1. Introduction

1.1. General

This Technical Report presents results of the hydrodynamic and sediment transport modeling study conducted by Coast & Harbor Engineering, Inc. (CHE) under Contract No Y-10058, Amendment 04-FAWAT-07-859 with BergerABAM.

The Federal Transit Administration (FTA) and Washington State Department of Transportation (WSDOT) Ferries Division (WSF) are evaluating the impacts associated with a new ferry terminal located primarily within the City of Mukilteo, Washington, in Snohomish County (Figure 1-1). Currently, WSF operates two 124-vehicle Issaquah Class Ferries at 30-minute intervals. This operation and existing terminal constitute the existing conditions. Four alternatives and existing conditions are evaluated in this report. Proposed project alternatives include minor modifications to the existing terminal, major modifications - including realignment - to the existing terminal, or relocation of the terminal eastward to one of two locations at the former U.S. Department of Defense Tank Farm property.

The modeling study discussed here was conducted to evaluate propeller wash and ferry wakes for each of the terminal alternatives for their effects on bottom scour and sediment transport along the study area shoreline, and velocity and salinity patterns at the NOAA seawater intake. The study also evaluated possible impacts from removal of the entire existing Tank Farm Pier. Prior to conducting the modeling, CHE developed a modeling plan and coordinated the hydrodynamic and sediment transport modeling scenarios with the project team and WSF.

1.2. Project Alternatives

The project alternatives include No-Build, Existing Site Improvements, Elliot Point 1, and Elliot Point 2. The alternatives are described in this report in terms of changes that are pertinent to sediment processes at the nearshore bottom slope and shoreline.

The No-Build Alternative (Figure 1-2) would replace the existing ferry terminal structures; the offshore end of the transfer span would remain in the same location. The existing slip alignment is approximately 45 degrees west of north. The Existing Site Improvements Alternative (Figure 1-3) would extend seaward from approximately the same point on the shoreline, but the alignment of the slip would be

due north (rotated 45 degrees to the north) and the slip would be approximately 50 ft farther offshore.

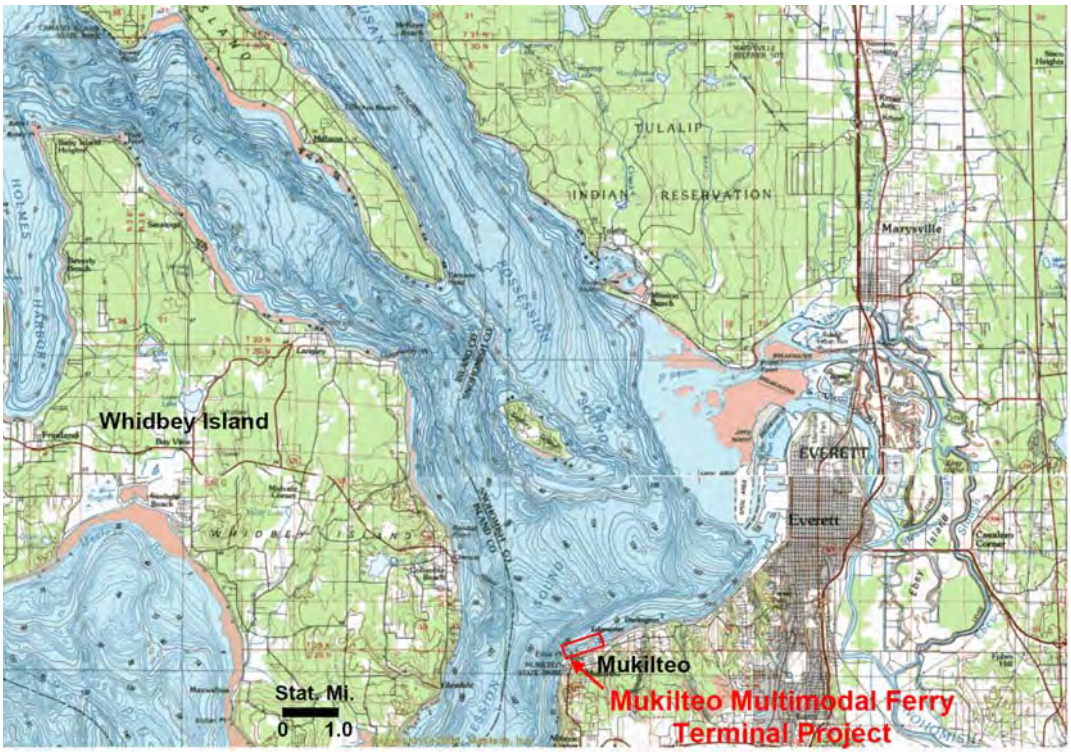


Figure 1-1. Regional map with project location

The Elliot Point 1 Alternative (Figure 1-4) would relocate the terminal approximately 2,700 ft eastward along the shoreline from the current location. In this new location the slip would be approximately 190 ft farther offshore than the existing slip. And the new slip would be angled approximately 50 degrees west of north.

The Elliot Point 2 Alternative (Figure 1-5) would relocate the terminal approximately 1,700 ft eastward from the existing terminal (1,000 ft west of Elliot Point 1). The slip would be approximately 90 ft closer to shore. It would be angled 20 degrees west of north. The base of the transfer span would be essentially against the shoreline.

A slightly modified version of Elliot Point 2 Alternative was selected as the Preferred Alternative in the Final Environmental Impact Statement. The modifications consist primarily of reconfiguring some upland components and shifting the ferry slip slightly to the west, but the overall footprint is unchanged. The analyses and results described in this report for Elliot Point 2 Alterantive apply to the Preferred Alternative as well.

Figure 1-2. No-Build Alternative layout[1]

Figure 1-3. Existing Site Improvements Alternative layout

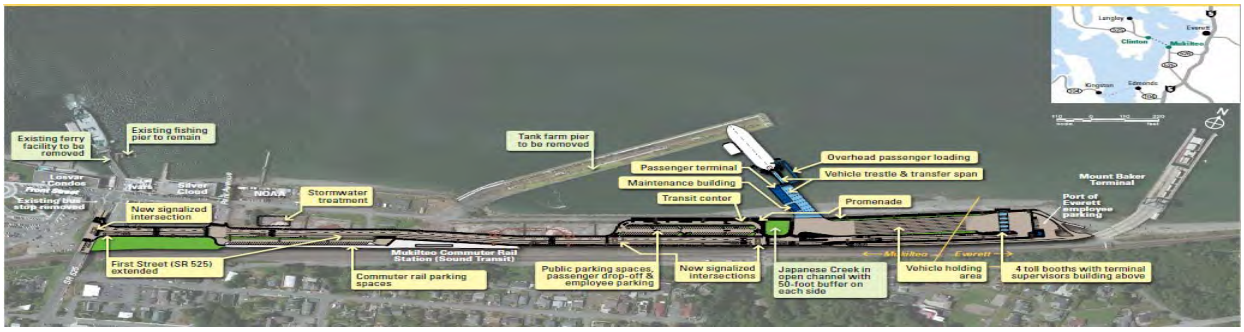


Figure 1-4. Elliot Point 1 Alternative layout

[1] Source for Figures 1-2, 1-3, 1-4 and 1-5 is Washington State Ferries: http://www.wsdot.wa.gov/projects/ferries/mukilteoterminal/multimodal).

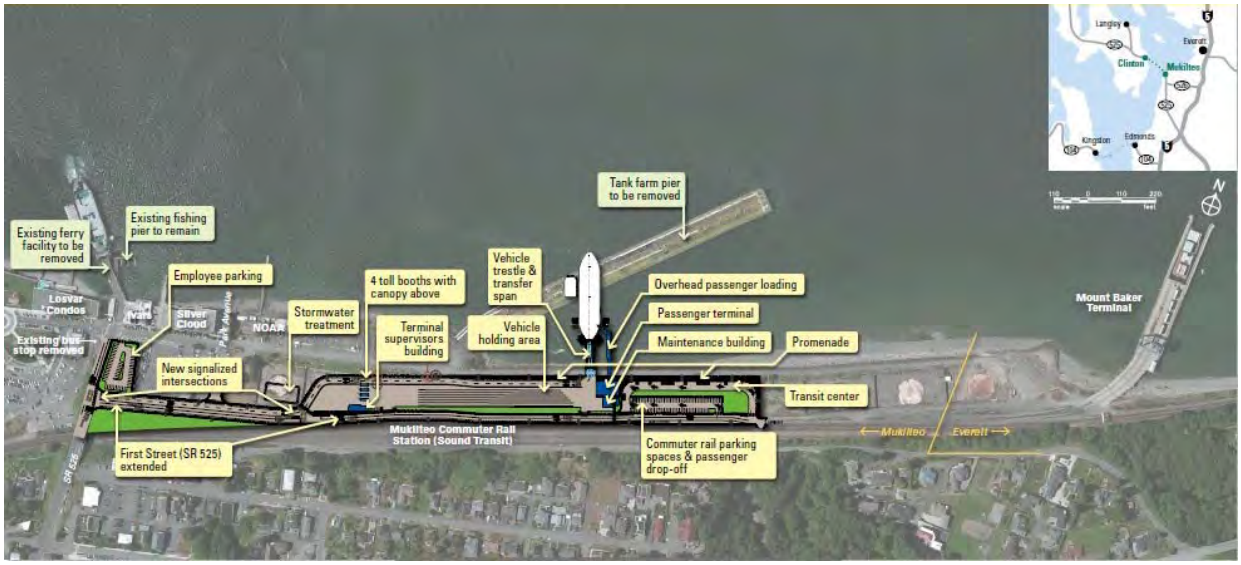


Figure 1-5. Elliot Point 2 Alternative layout

1.3. Study Content and Methodology

The CHE scope of work includes specifying terminal location, water depths, and other parameters, and developing model input for simulating propeller wash and ferry vessel wake hydrodynamics for existing conditions and the four project alternatives. Propeller wash and vessel wake hydrodynamics were simulated for all cases with two-dimensional (2-D) and three-dimensional (3-D) numerical models. CHE also performed modeling and analysis of possible shoreline changes due to full removal of the existing Tank Farm Pier, taking into account the effects from propeller wash, vessel wakes, and the ambient wave climate.

Specified cases of propeller wash were simulated for the four alternative conditions. The results were analyzed to determine effects under each scenario on bottom scour and on sediment transport, sediment budget, and shoreline shape.

Vessel wakes were modeled for all four of the proposed alternatives and existing conditions. Results of simulations were analyzed to determine possible effects on transport potential at the shoreline. In addition, numerical modeling and analysis was performed to determine changes in longshore sediment transport that would result from removing the Tank Farm Pier. Results of modeling and analysis of possible effects of changes in propwash, vessel wakes, and longshore sediment transport for all proposed modifications to the ferry terminal are discussed in detail in Sections 3, 4, and 5.

2. Site Characteristics- Physical Setting and Shoreline Morphology

This section describes the existing physical conditions in the project area. Providing existing conditions establishes a basis against which project-related changes can be compared. Existing physical conditions in the vicinity of the project were derived mostly from published

data sources. Existing wind-wave and current regimes were estimated using published data and a variety of numerical models.

Geographic features, tidal variations, tidal circulation, wind and wind-waves are presented to provide the existing setting for the proposed alternatives. The geomorphology of the shoreline is described and sediment sources and potential sinks are discussed.

2.1. Location

The planned location of the Mukilteo Multimodal Ferry Terminal would be either at the present site or in the Tank Farm Pier area. Figure 2-1 shows the project area, anemometer station, and locations for National Oceanic and Atmospheric Administration (NOAA) tidal data sites in the area.

2.2. Tides

Tidal data for the project site are based on results from NOAA's program VDATUM, which computes site-specific relationships between tidal and vertical geodetic datums. The resulting relationships for vertical datums at the existing Mukilteo Ferry Terminal are presented in Table 2-1.

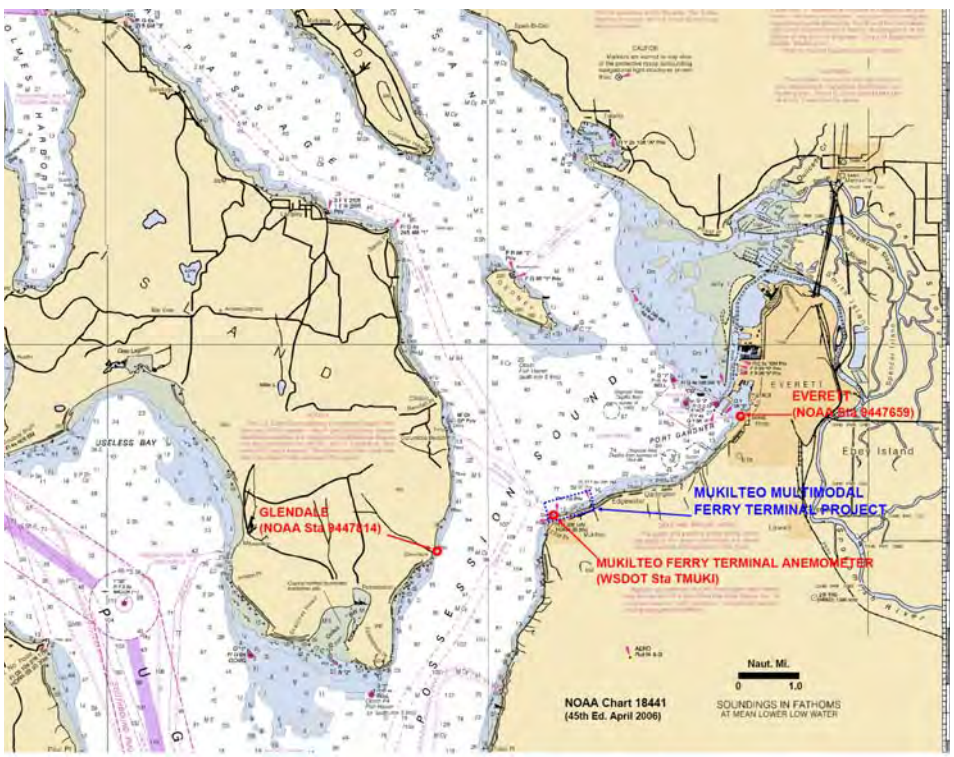


Figure 2-1. Location of Mukilteo Multimodal Ferry Terminal project, WSDOT anemometer, and NOAA tide stations

Table 2-1. Mukilteo Ferry Terminal tidal and geodetic datums

MUKILTEO FERRY TERMINAL, WA TIDAL DATUMS Tidal Epoch: 1983-2001				
	Elevation (m-MLLW)	Elevation (ft-MLLW)	Elevation (m-NAVD88)	Elevation (ft-NAVD88)
ESTIMATED HIGHEST WATER LEVEL	4.280	14.05	3.665	12.02
MEAN HIGHER HIGH WATER (MHHW)	3.369	11.05	2.751	9.03
MEAN HIGH WATER (MHW)	3.107	10.19	2.490	8.17
MEAN TIDE LEVEL (MTL)	1.979	6.49	1.362	4.47
MEAN SEA LEVEL (MSL)	1.971	6.47	1.354	4.44
MEAN LOW WATER (MLW)	0.851	2.79	0.234	0.77
NORTH AMERICAN VERTICAL DATUM 1988 (NAVD88)	0.617	2.03	0.000	0.00
MEAN LOWER LOW WATER (MLLW)	0.000	0.00	-0.617	-2.03
ESTIMATED LOWEST WATER LEVEL	-1.521	-4.99	-2.138	-7.02

Note: Elevations determined with NOAA program VDATUM for Mukilteo location

Tides are semi-diurnal, with a diurnal range of 11.05 ft and mean range (difference in height between mean high water and mean low water) of 7.40 ft. The nearest NOAA tide station with published tidal datums is at Glendale, approximately 2.2 nautical miles south-southwest of Mukilteo. Published tidal datums from NOAA for Glendale are presented below in Table 2-2 for comparative purposes.

Table 2-2. Glendale, WA tidal datums based on NOAA published data

GLENDALE, WA TIDAL DATUMS (NOAA Station 9447814) Tidal Epoch: 1983-2001		
	Elevation (m-MLLW)	Elevation (ft-MLLW)
ESTIMATED HIGHEST WATER LEVEL	4.280	14.05
MEAN HIGHER HIGH WATER (MHHW)	3.364	11.04
MEAN HIGH WATER (MHW)	3.108	10.20
MEAN TIDE LEVEL (MTL)	1.984	6.51
MEAN SEA LEVEL (MSL)	1.976	6.48
MEAN LOW WATER (MLW)	0.860	2.82
MEAN LOWER LOW WATER (MLLW)	0.000	0.00
ESTIMATED LOWEST WATER LEVEL	-1.521	-4.99

NOAA does not publish the NAVD88 datum for Glendale, but does provide NAVD88 datum for Everett and Seattle, which are 2.03 ft MLLW and 2.35 ft MLLW, respectively. NAVD88 calculated for Mukilteo using VDATUM is 2.03 ft MLWW, which compares well with an interpolation between Everett and Seattle.

2.3. Winds and Waves

Wind data were compiled from the WSDOT anemometer station (TMUKI) located on top of the west tower of the transfer span, shown in Figure 2-2.

Figure 2-2. WSDOT Anemometer at Mukilteo Ferry Terminal

Wind data from October 2002 through 2010 were processed and analyzed. An annual wind rose based on these data was constructed and is shown in Figure 2-3. It is evident that the prevailing wind directions at the existing Mukilteo Terminal are from the southwest, north, and east-northeast sectors, which are mostly due to topography channelizing the wind. The percent frequency of wind speeds and maximum measured wind speeds in the period 2003 through 2010 recorded at the Mukilteo Ferry Terminal station are shown in Table 2-3.

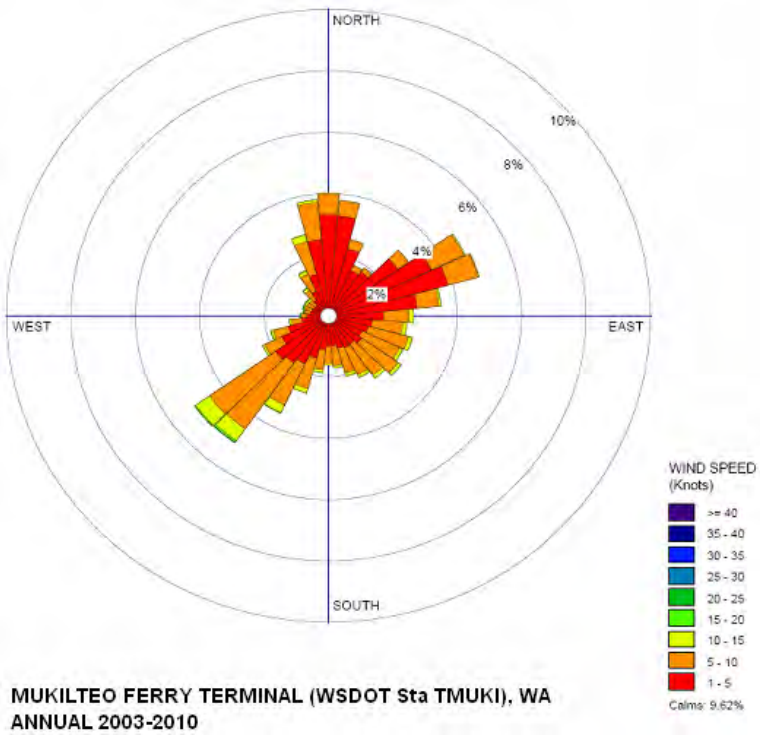


Figure 2-3. Mukilteo Ferry Terminal annual wind rose

Table 2-3. Mukilteo Ferry Terminal percent frequency of wind speed versus wind direction

MUKILTEO FERRY TERMINAL (WSDOT Sta TMUKI), WA
PERCENT FREQUENCY OF WIND SPEED VERSUS WIND DIRECTION (ANNUAL)
Data Record Range: 1/1/2003-12/31/2010
Total No. Cases: 184285

RANGE	DEGREES TRUE																																				
(kts)	0	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	TOTAL
1-5	3.31	3.27	2.27	1.63	1.75	2.64	3.52	3.84	2.76	1.71	1.36	1.28	1.16	1.26	1.14	1.16	1.09	0.97	0.97	1.11	1.43	1.72	1.88	2.00	1.59	1.30	0.84	0.67	0.57	0.54	0.49	0.49	0.64	0.89	1.41	2.50	57.16
5-10	0.66	0.50	0.31	0.19	0.16	0.40	1.12	0.97	0.68	0.79	0.96	1.30	1.28	1.38	1.23	0.96	0.84	0.67	0.58	0.66	1.02	1.49	2.62	2.51	0.60	0.48	0.32	0.16	0.15	0.21	0.25	0.24	0.39	0.62	1.11	1.22	29.01
10-15	0.03	0.02	0.01	0.005	0.004	0.004	0.04	0.03	0.06	0.15	0.13	0.13	0.11	0.12	0.11	0.10	0.11	0.08	0.07	0.10	0.15	0.28	0.48	0.53	0.07	0.08	0.07	0.03	0.04	0.05	0.08	0.08	0.08	0.12	0.22	0.09	3.85
15-20	0	0	0	0	0.001	0	0	0.001	0.003	0.002	0.004	0.001	0	0.002	0.002	0.001	0.002	0.001	0	0.003	0.01	0.02	0.07	0.04	0.004	0.008	0.01	0.02	0.02	0.02	0.02	0.02	0.02	0.01	0.01	0.002	0.30
20-25	0	0	0	0	0	0	0	0	0	0	0	0	0.001	0	0	0	0	0	0	0	0	0.001	0.01	0.004	0	0.001	0	0.005	0.01	0.005	0.004	0.004	0.002	0.003	0	0	0.05
25-30	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.002	0	0	0	0	0.001	0.004	0	0	0.001	0	0	0.001	0.01
30-35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.001	0	0	0	0	0.001	0	0	0.001	0	0	0	0	0.001	0.003
35-40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.001	0	0	0	0	0	0	0	0	0.001
≥ 40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	4.00	3.79	2.59	1.83	1.92	3.04	4.68	4.84	3.51	2.64	2.45	2.70	2.55	2.76	2.48	2.22	2.04	1.72	1.62	1.87	2.61	3.50	5.05	5.09	2.27	1.87	1.23	0.89	0.79	0.84	0.84	0.83	1.13	1.64	2.75	3.80	90.38
																																				CALMS	9.62
																																				TOTAL	100.00

Return period winds for the Mukilteo Ferry Terminal location were computed using a Weibull distribution fit to data of monthly maximum wind speeds for every 10 degrees of wind direction from each year from 2003 to 2010. This method of using monthly maximum speeds is suggested for a data record length of less than 10 years. A wind speed data record length greater than 10 years is necessary for estimating return periods using annual maxima for each wind direction. The correlation coefficient for the fit of the data to the Weibull distribution ranged from 0.888 to 0.995. The computed return period speeds are shown in Figure 2-4. Because there were 8 years of data, return period wind values are statistically reliable to

approximately 25 years using the monthly maxima method for each 10 degrees of wind direction.

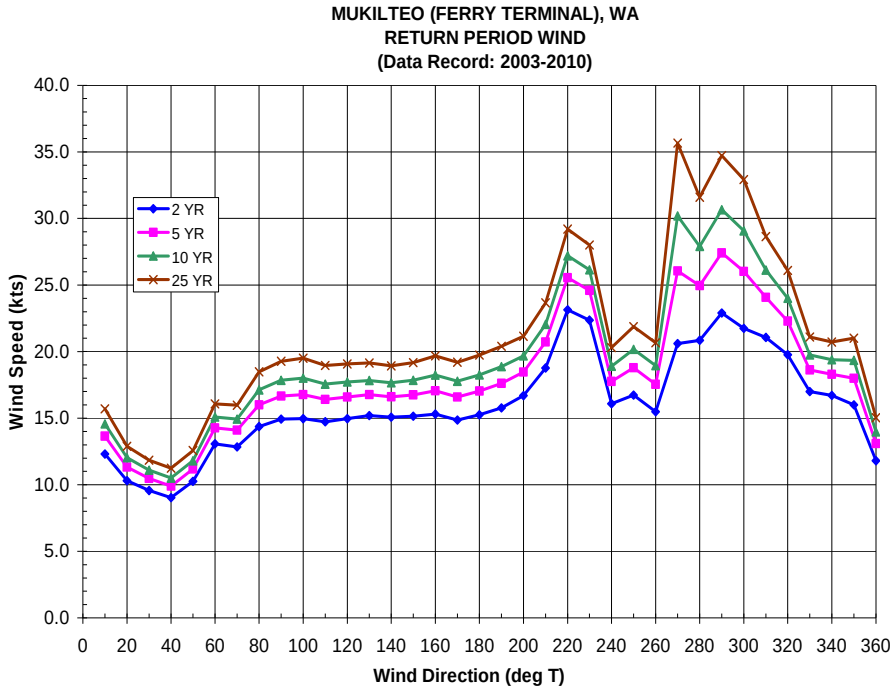


Figure 2-4. Mukilteo Ferry Terminal return period wind speeds

The figure shows that winds from the southwest direction (280-340 degrees) are stronger than from other directions. These winds are channeled up through the entrance to Possession Sound and presumably diverge with the terrain at the location of the Mukilteo Ferry Terminal. The southwest winds contribute significantly to a southwesterly wave climate at Elliot Point. The orientation of the shoreline and presence of Elliot Point provide some protection to the terminal at the existing site and the more easterly alternatives from this wave climate.

Winds from the north and to some extent from the northeast contribute potentially significant wave climates to the alternative sites. Wind-waves from these directions and their potential for sediment transport are addressed in Section 5.

2.4. Ambient Currents

The description of ambient currents at the Mukilteo site is based on a previous study (CHE, 2003). The study consisted of two weeks of field measurements of current velocities and directions using an Acoustic Doppler Current Profiler (ADCP) deployed in 30 ft of water in August and September 2003. The current meter was deployed approximately 100 ft offshore of the base of the Tank Farm Pier (Figure 2-5). Field data were analyzed to determine general patterns of depth-averaged current velocities.

Current measurements were processed to calculate statistical characteristics of depth-averaged velocity. A time series of speed and direction data was processed from current meter recordings, sorted into categories, and graphed as speeds according to frequency of occurrence and direction, called a current rose. The current rose of depth-averaged velocities, representing the frequency, magnitude, and direction of currents, is plotted in Figure 2-5.

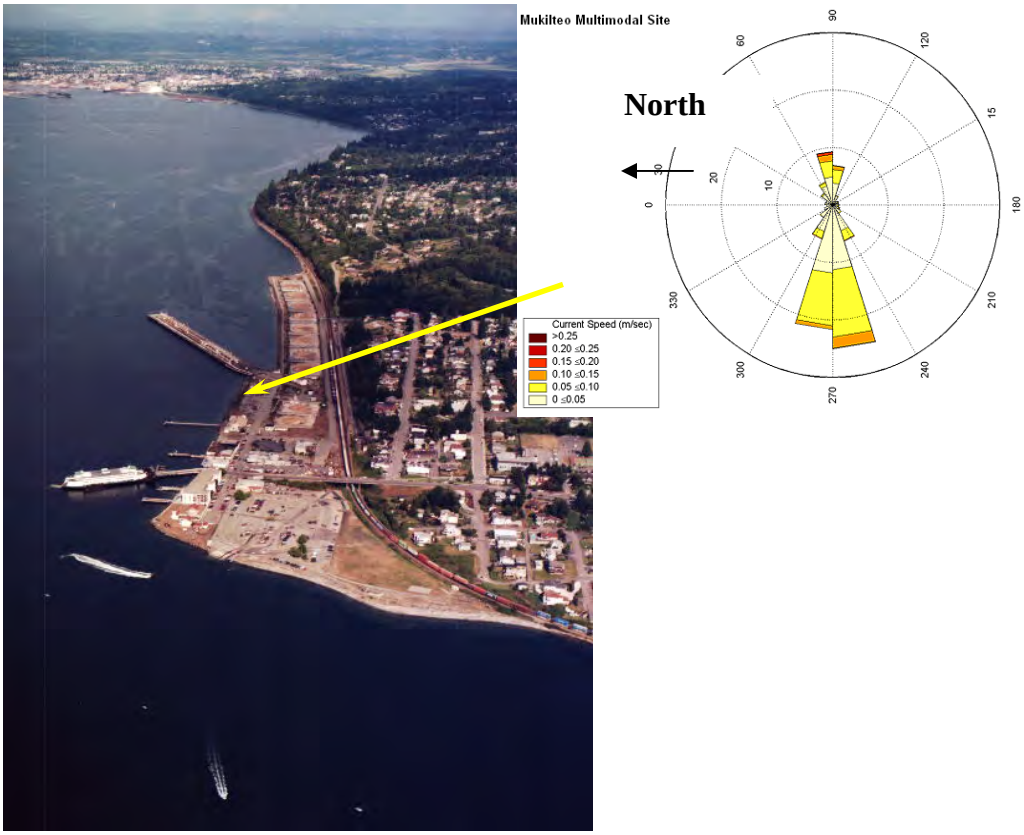


Figure 2-5. Current rose representing current velocities and directions at Mukilteo site

Current measurements showed that the predominant depth-averaged current velocities at the base of the Tank Farm Pier are relatively small, with average velocities in the range of 0 to 0.5 foot/second (ft/sec) (0 to 15 centimeter/second [cm/sec]). Statistical analysis shows that depth-averaged current velocities were measured in the range of 0.2 to 0.3 ft/sec (5 to 10 cm/sec) 47 percent of the time. The occurrences of speeds in the intervals 1.0 to 1.3 ft/sec (30 to 40 cm/sec) and 1.3 to 1.6 ft/sec (40 to 50 cm/sec) were estimated to be 1 percent and 0.6 percent, respectively. Depth-averaged velocities larger than 1.6 ft/sec (50 cm/sec) were not observed in the field data.

Data analysis showed that velocities are mostly bi-directional, east and west. Currents flowing towards the east occur during flood tide, while currents flowing towards the west occur during ebb tide. Depth-averaged velocities during flood tide are stronger than during ebb tide. Current velocities in other directions are small and

are of short duration. Maximum peak velocities in the landward direction, from north to south, did not exceed 0.2 ft/sec for all observed events.

2.5. Shoreline Morphology and Sediment Transport

The Mukilteo Ferry Terminal is located on the southeastern shoreline of Possession Sound as shown in Figures 2-1 and 2-6. The southern end of Possession Sound forms a passage or strait which lies between the southern tip of Whidbey Island (to the west) and the mainland (to the east). At the north end of this passage, the Sound widens abruptly as the eastern shoreline turns sharply to the east. The ferry terminal is positioned just past this eastern turn in the shoreline.

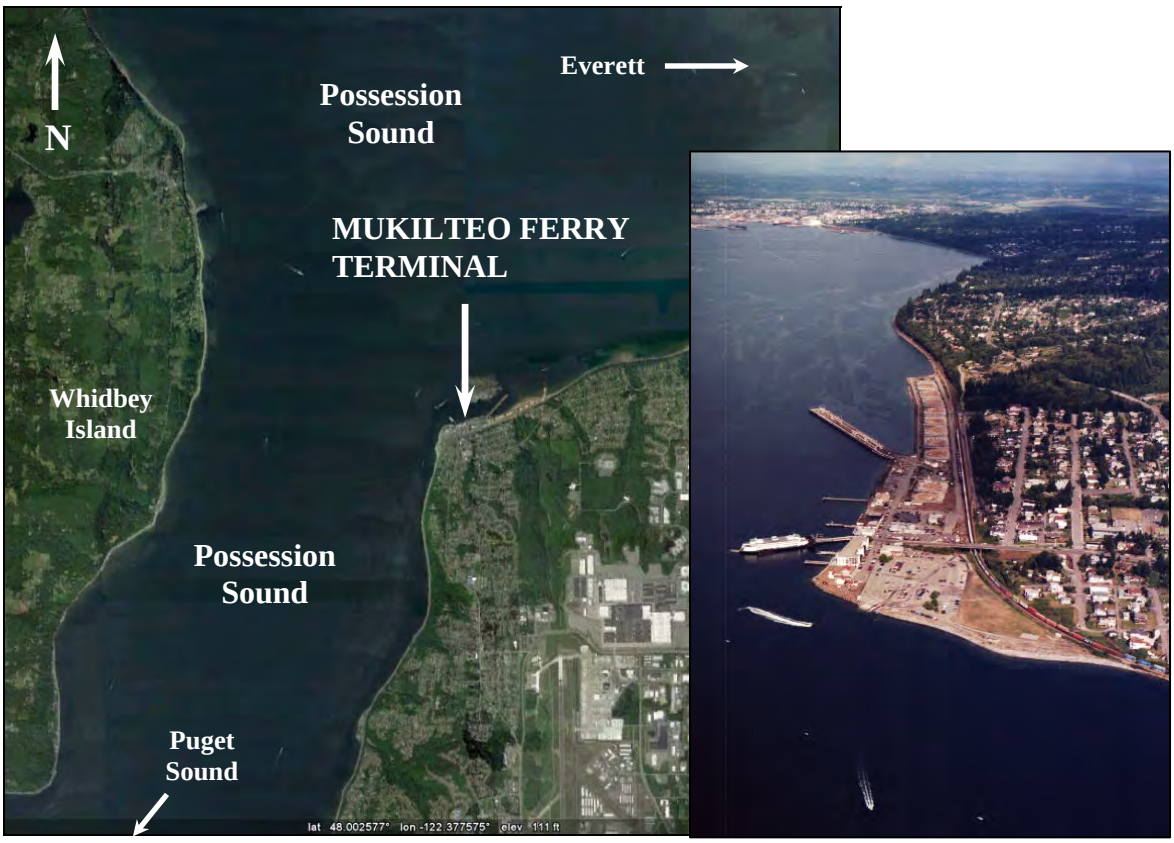


Figure 2-6. Shoreline morphology in Possession Sound and in vicinity of Mukilteo Ferry Terminal

The inset in Figure 2-6 shows that the Mukilteo Ferry Terminal was built on fill, as were the alternate locations discussed in this report. The bathymetry will therefore tend to be deeper closer to shore at the alternative sites than might otherwise be expected, and sediments moved offshore of the project area may be more likely to remain offshore.

Figure 2-7 is an excerpt from NOAA Chart 18443 which was produced by the NOAA Office of the Coast Survey. It shows a relatively broad shallow shelf along the shoreline. Historically, fill was placed in the tidelands at the project site to support

industrial development along this shoreline reach. The fill does not appear to reach beyond the edge of the shallow shelf into deeper water.

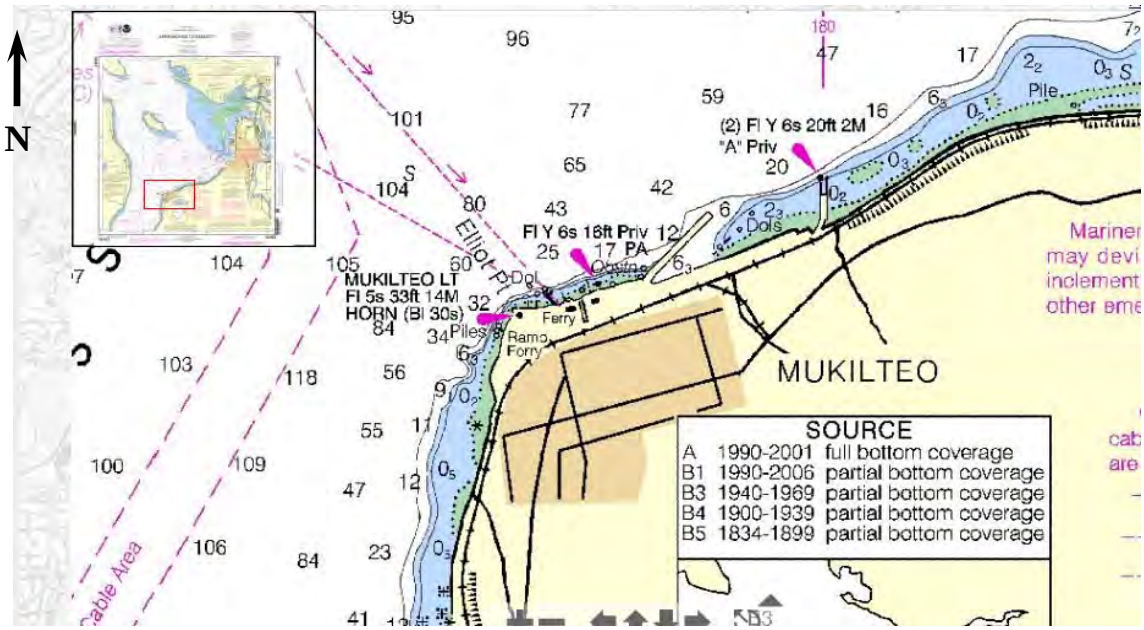


Figure 2-7. Nearshore bathymetry showing fill area at ferry terminal and areas to the east, including the Tank Farm Pier. Excerpt from NOAA Chart 18443; soundings are in fathoms from mean lower low water.

The project area shoreline was analyzed based on site assessments, previous studies (CHE 2005, 2006), review of historical aerial and ground photographs, and available bathymetric surveys. The shoreline along the project area is altered with various shore structures that have various levels of effects on coastal processes.

The shoreline is schematically divided into three reaches (Figure 2-8) for the current specific analysis of probable effects of propwash and wakes. Each reach represents a section of shoreline having relatively similar material composition, structures, and forces affecting the shoreline[2]. Sediment processes at the shoreline are expected to be affected in the post-project stage by propwash and wakes of the new 144 Auto Class Ferry, as discussed below.

[2] It should be noted that this division is simplified and is applicable only for the purpose of developing a background for comparison of the various structures' effects on sediment transport. One may find different types of shorelines, structures, sediment characteristics, and forces acting on the shoreline in each distinct reach and should rearrange the divisions to be applicable for one's purposes.

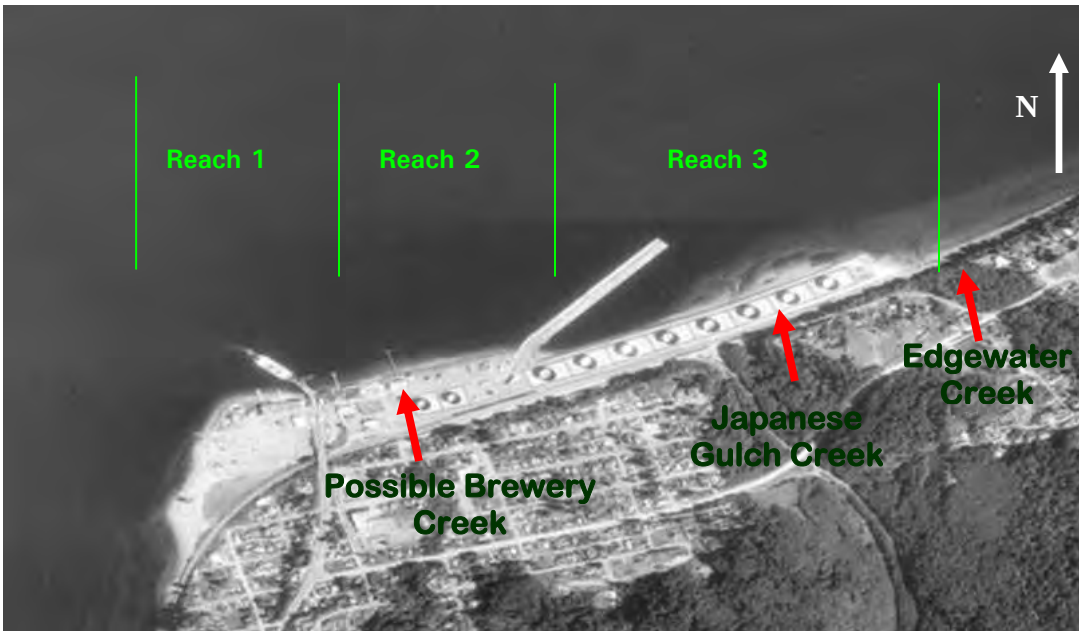


Figure 2-8. Shoreline reaches within sediment budget analysis area

The 2010 Draft Shoreline Master Program Restoration Plan for the City of Mukilteo lists three creeks that enter Puget Sound within the approximate project area. These include:

- Edgewater Creek which enters Puget Sound at the eastern end of the tank farm area, approximately at the Rail/Barge Transfer Facility (Reach 3);
- Japanese Gulch Creek which enters Puget Sound at the location of the delta near the east end of the Tank Farm and near the offshore end of the Tank Farm Pier (Reach 3); and
- Brewery Creek which enters Puget Sound east of the existing ferry terminal (Reach 2). The location is not shown on the Restoration Plan and there is no clearly identifiable delta in this area; however, water was observed flowing from a culvert which daylights on the beach, near the public beach entrance.

2.5.1. Shoreline Reach 1

Reach 1 extends from Elliot Point to the existing ferry terminal, and is approximately 500 ft long. Reach 1 is characterized as a temporary sink for beach sediments (gravelly sand) moved from the south around Elliot Point and towards the east.

Figure 2-9 shows the shoreline that is typical of this reach. The upper beach is armored or backed by foundation walls. Two small piers are present, but do not extend seaward of the intertidal beach. This reach is subjected to wind-waves (which will not be altered by the project) and wakes generated by arriving ferries (which will cease under the Alternatives that relocate the existing ferry terminal, as described in Section 1).

Figure 2-9. View toward the southeast of Reach 1 and Elliot Point

This part of shoreline was historically formed by filling a part of the nearshore. Filling and extending the shoreline into the Sound has resulted in disrupting dynamic equilibrium among wave energy, sediment transport, and bottom profile. The shoreline has since been exposed to larger wind-waves, and later, to vessel wake impacts. No detailed study on shoreline conditions at this area has been identified. It is likely that a long-term trend of shore erosion has resulted from the disruption of a balance between wave energy and the bottom slope since filling displaced the shoreline seaward. This trend may not be uniform along Reach 1, and likely varies according to seasonal variation of shoreline position in the western part of Reach 1. Relocation of the existing ferry terminal will reduce ferry wake energy delivered to the shoreline, and may positively affect coastal processes.

2.5.2. Shoreline Reach 2

Reach 2 extends from the existing ferry terminal to the Tank Farm Pier (Figure 2-10), a distance of approximately 1,300 ft. Wind-waves, ferry wakes, and propwash are thought to be agents in moving beach material eastward in this reach. Wind-waves may also transport beach material westward infrequently and in small amounts.

Figure 2-10 shows the beach at a relatively low tide. The intertidal portion of the beach is clearly widest in the center of the reach and tapers to a narrower beach width both to the west and the east. This implies loss of sediment at both ends of Reach 2. Figure 2-10 also shows the location of the discharge end of a culvert which may be where Brewery Creek enters Possession Sound.

Figure 2-10. Reach 2 intertidal beach narrowing to both the west and the east implies loss of sediments to both the ferry terminal and under the Tank Farm Pier

Reach 2 contains two light-duty piers. The westernmost of the two piers is approximately 140 ft east of the ferry terminal and extends seaward to a bottom elevation of about -5 ft. The easternmost and larger of the two piers (known as the NOAA Pier) is approximately 530 ft east of the ferry terminal (790 ft west of the Tank Farm Pier) and extends seaward to about the -30 ft contour.

Buildings are built on piles over the beach from the western end of Reach 2 eastward for about 375 ft. Eastward from that point the shoreline of Reach 2 is hardened with a rock revetment. Figure 2-11 illustrates the shoreline configuration representative of approximately the center of Reach 2. The western 420 ft of this armored shoreline is fronted with an intertidal beach approximately 100 ft wide. For the eastern 460 ft of Reach 2, the beach tapers to just the armoring with little or no intertidal beach in front.

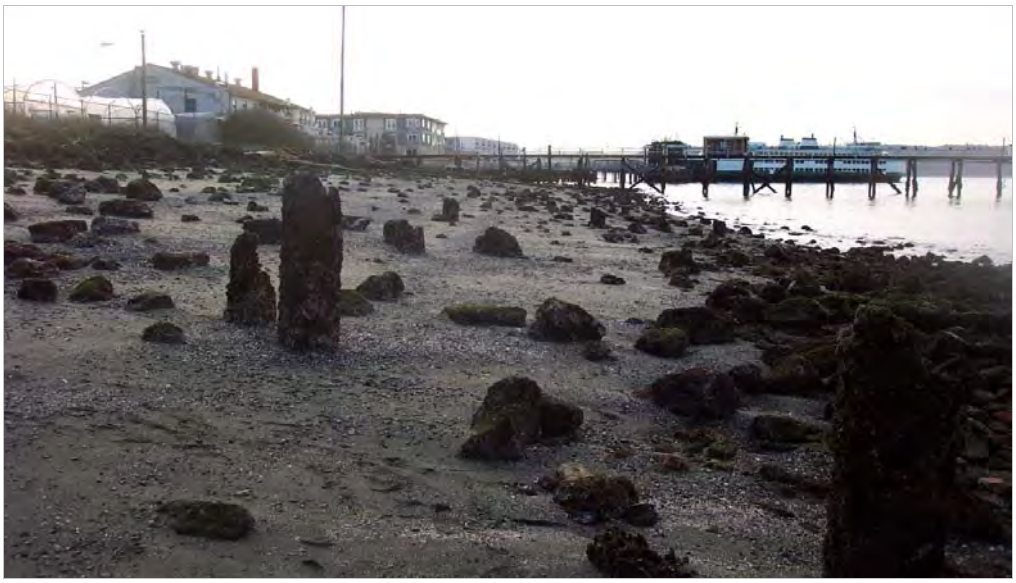


Figure 2-11. Project shoreline, eastern end of Reach 2

Beach sediments in Reach 2 transition from sandy gravel at the west to gravelly sand and riprap towards the eastern part of the reach. The eastern portion of Reach 2 - ending at the Tank Farm Pier - is armored with riprap for most or all of the beach profile.

The width of the intertidal beach is much reduced at either end of Reach 2. It is possible that beach width in the center of this reach is being maintained by some of the sediment discharged from Brewery Creek. If the creek discharges sediment at this location, some of that sediment is probably being lost in both west and east directions.

2.5.3. Shoreline Reach 3

Reach 3 extends from the base of the Tank Farm Pier to the east end of the Tank Farm property, a distance of about 2,500 ft (Figure 2-12). The shoreline is backed by the Tank Farm property fill, and is armored for the full distance of the reach (Figure 2-13). At the eastern end of the reach the Tank Farm fill ends and the shoreline follows the railroad embankment that fronts a steep bluff. All of the exposed Tank Farm fill is stabilized with a rock revetment.

Figure 2-12. Reach 3 from Tank Farm Pier to the west to Rail/Barge Transfer Facility on the east

Figure 2-13. Project shoreline, Reach 3 showing typical armored section

The delta of Japanese Gulch Creek forms a beach in front of the armor stone revetment in the eastern part of Reach 3. This beach narrows towards the west of the reach and pinches out against the revetment. Some beach material is observed to flow around the corner of the Tank Farm fill and continue eastward.

Elevation contours in Figure 2-14 show the beach width narrowing to the west approaching the Tank Farm Pier and a shallow subtidal beach prograding out from the shoreline at the seaward side of the pier. Figure 2-14 also shows shallower water underneath the pier; the center of the mound under the offshore end of the pier is at approximately -14 ft MLLW.

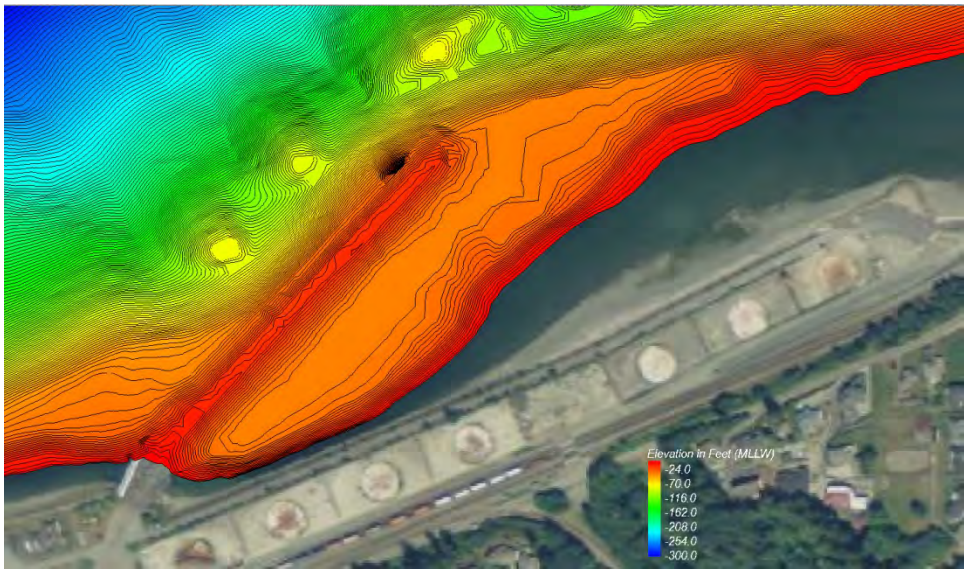


Figure 2-14. Bathymetry under and on either side of Tank Farm Pier with 2-ft contour intervals and landward edge of bathymetry shown at MLLW contour

Navigation charts and hydrographic surveys indicate that a ship's berth was dredged on the east side of the Tank Farm Pier in the late 1940s. Figure 2-14 shows that the dredged berth bottom elevations are not much changed from original construction, with depth approaching 38 ft below MLLW. Persistence of this depth is not expected and might be due to dissipation of wave energy by the piles under the Tank Farm Pier. Dissipation of wave energy by the pier would reduce transport of sediment from the west, under the pier, and into the berth. This would also cause deposition of transported sediment under the Tank Farm Pier and/or accumulation of sediment on the seaward side of and under the pier, as is evident in Figure 2-14.

It appears therefore that the Tank Farm Pier traps sediment that moves eastward along the shoreline on the west side of the pier and prevents it from filling the dredged berth.

The presence of the pier would not slow sediment moving from east to west along the shore, yet there is no evidence from surveys that sediment from an eastern source fills in the berth. Examining the wind rose in Figure 2-3 provides some insight into transport intensity and direction. Some wind energy comes directly from the north and slightly east of north. Waves generated by these winds would be partially blocked by the angled pier, but should cause some westward migration of nearshore sediment until the wave shadow slows transport. The wind rose shows that one of the

three major wind directions essentially parallels the shore. Resulting waves from the northeast would contribute to sediment transport from east to west. It appears however that this transport volume is minor in this location.

2.6. Section Summary

Hydrodynamic, littoral, and geomorphic conditions vary along the Mukilteo shoreline, resulting in diverse coastal features that are complicated by artificial structures. Development of this shoreline involved filling at the shoreline and steepening the shallow subtidal bottom slope. Not accounting for the effects of nearshore structures, this action has allowed more wave energy to reach the shoreline, thus causing greater loss of beach sediment, and resulting in shore erosion.

Ferry and passing deep-draft vessel operations in the western portion of the project shoreline, designated here as Reach 1, might add energy to the shoreline, which intensifies sediment transport at the shoreline, to a minor extent. Longshore sediment transport in the project area is controlled by wind-wave storms. The predominant direction of longshore transport is from west to east.

The Tank Farm Pier and the revetment that protects the Tank Farm fill are dominant influences on the shore processes in the eastern portion of the project area, Reaches 2 and 3. The dense pattern of piling under the Tank Farm Pier appears to interfere with longshore sediment transport. Evidence is a prograding subtidal beach on the seaward (western) side of the pier, and maintenance of initially dredged depth in the berth on the inshore (eastern) side of the pier.

3. Propwash Modeling

3.1. Methodology

The objective of propwash modeling was to determine and quantify the potential effects from ferry propeller-induced currents on nearby bottom slope and bottom sediments at the proposed ferry terminal alternative locations. These quantitative estimates of potential effects will be further used in evaluation of the alternatives and Environmental Impact Statement (EIS) discipline reports. Propwash modeling included simulation of current velocities generated by the propeller of appropriate ferries and the computation of the extent of bottom scour where it occurs. Propwash modeling was conducted for the following four alternatives: No-Build, Existing Site Improvements, Elliot Point 1, and Elliot Point 2.

Propwash modeling and analysis were conducted with several steps. First, simulation of current velocities generated by the ferry propeller was conducted using the 2-D numerical model JETWASH (Shepsis and Simpson, 2000). Second, analysis and computations of potential scour and estimates of depth of scour were conducted for areas where the propwash hydrodynamic force exceeded sediment stability criteria. Sediment stability criteria are assumed to be equivalent to sediment critical shear stress (threshold) criteria (Vanoni, 2006).

The JETWASH numerical model is based on a well-established and empirically verified theory of flow produced by a momentum jet. It computes flow velocities

during propagation of a propeller jet through the water column and interaction of this jet with the bottom. JETWASH is able to compute a plan view propwash velocity pattern as well as a vertical plane of velocities. Figure 3-1 shows an example computation of propwash velocities in a vertical plane.

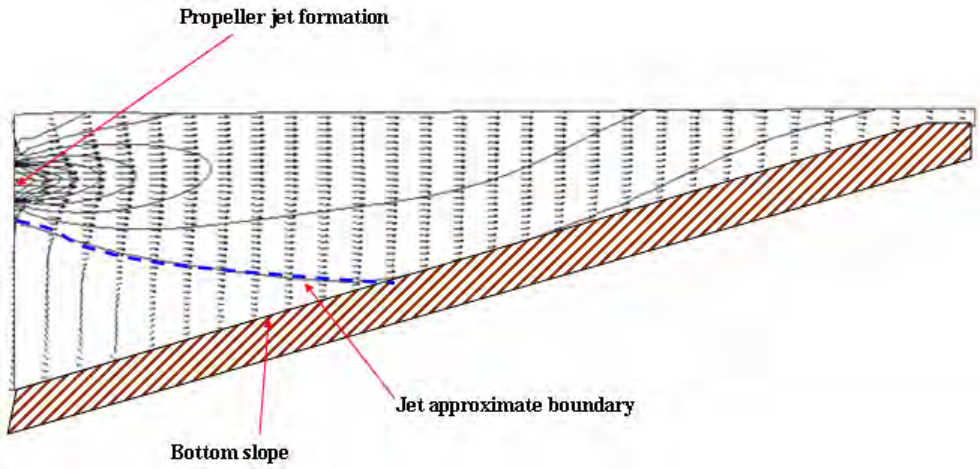


Figure 3-1. Example pattern of propwash velocity in vertical plane output by JETWASH

Please note that a propeller jet is relatively small in diameter at the propeller and increases in size (diffuses) while propagating away from the propeller. Depending on the bottom slope and clearance between the propeller and the bottom, propwash velocity may impinge on the bottom slope at various (sometimes significant) distances from the propeller. Peak propwash bottom velocity at the existing Mukilteo terminal is observed on the bottom at a distance of 66 ft from the propeller when the ferry docks at a water level of MLLW (see Section 3.3.1.2),.

The JETWASH model has been tested and proven to be a reasonable engineering tool for propwash analysis. The model has been accepted by EPA Region 8 and the U.S. Army Corps of Engineers for analysis of sediment stability under impact from propwash of vessels ranging in size from small recreational boats to large ships.

Critical shear stress for scour analysis was computed using modeled velocity at a specified height above the bottom (0.85 ft) and assuming a logarithmic velocity profile develops in the propwash flow near the bottom (USACE, 2002). Depth of scour was computed using a software package assuming that particles larger than the threshold size for motion in the sediment mixture remain on the bottom after the smaller particles are washed away (Borah, 1989). In essence, the procedure assumes that when a propeller jet impacts the bottom, it washes away all particles smaller than a minimum stable size within a zone at the surface. The larger particles in the sediment matrix concentrate on the bottom surface, where they form a layer that effectively armors the bottom, preventing further scour. The scour depth computational procedure that includes armoring effect was further validated during this study in Section 3.3.2.1.

Data and input parameters required for propwash modeling and analysis include ferry vessel propulsion characteristics, bottom sediment characteristics, tide elevations, and bathymetry. These data and input parameters are discussed below.

3.2. Propwash Modeling Input Parameters

3.2.1. Ferry Vessels

Two ferry vessels were used in analysis and modeling of propwash and bottom scour; the existing Issaquah Class Ferry (124 auto capacity) and a future New 144 Auto Class Ferry. Propulsion and other parameters relevant for propwash analysis and for characterizing the vessels were compiled from various sources and are shown in Table 3-1.

Table 3-1. Specifications for present and future ferry classes operating at Mukilteo

Item	Issaquah Class Ferry	New 144 Auto Class Ferry
Length (Overall)	328 ft	362.3 ft
Beam	78.7 ft	83.3 ft
Light Ship Weight	2,228 Long Tons	Not Available
Design Load Displacement	2,950 Long Tons	Not Available
Draft at Light Ship	13.25 ft	Not Available
Draft at Design Load	15.5 ft	16.8 ft
Service Speed	16 kts	17 kts
Rudders	Conventional	High Lift
Propulsion	Geared Diesel Engines	Geared Diesel Engines
Propeller Type (1 ea end)	Controllable Pitch	Controllable Pitch
Propeller Diameter	11.5 ft	11.5 ft
Shaft Horsepower	5000 HP	6000 HP
Propeller Rotational Speed in Landing	120 RPM	120 RPM
Propeller Thrust	36,160 lbf	43,350 lbf
Power per Propeller (kW)	1254 kW	1500 kW
Dist Prop CL to Design Draft WL	10.0 ft	9.95 ft
Propeller Shaft Slope	0 degrees	0 degrees

3.2.2. Bottom Sediment

Results of sediment sampling and grain size analysis from a previous study[3] (WSDOT, 2005) were used to develop bottom sediment input parameters for propwash and bottom effects modeling. Twenty sediment samples were taken at four

[3] No sediment sampling or grain size analyses were available directly at the existing terminal and the three alternative sites. Sediment data from in-water sampling in the project vicinity were used from a 2005 report to Washington State Ferries.

bore holes east of the existing terminal at various depths in that study. Grain size distributions show variability among the samples as a result of sampling at different locations. Variability among these samples is not great, however. Samples were selected for applicability in this analysis by their location relative to the terminal alternatives, comparable water depth, distance from existing structures, and shallow sampling depth below the bottom surface[4]. Figure 3-2 shows grain size distributions for the selected samples.

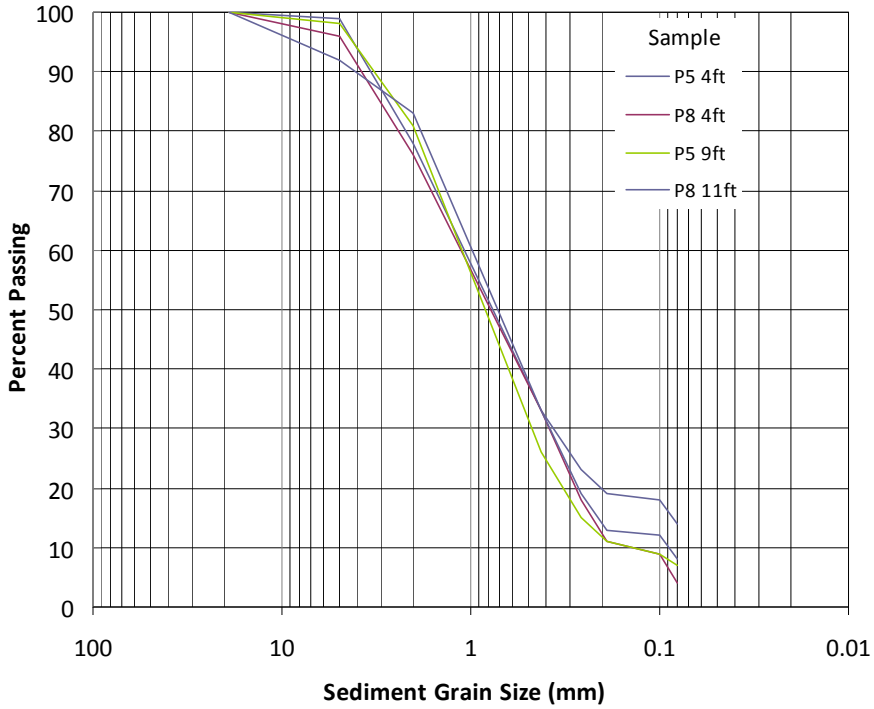


Figure 3-2. Grain size distributions of sediment collected in vicinity of Mukilteo terminal at bore holes in 2005 sampling

Representative grain size characteristics for propwash effects analysis were obtained by averaging data for the applicable distributions. A representative gradation curve was derived by averaging the percent of sample by weight passing at each size category for each sample. Figure 3-3 shows the data and representative grain size distribution that was used for propwash and scour analysis. The gradation curve shows that the median grain size is 0.8 mm, and with maximum size as large as 20 mm.

[4] Sediment was sampled beneath the Tank Farm Pier in 2012, but those data were not used for propwash effects analysis because they result from a unique depositional environment, not representative of a slip at a terminal.

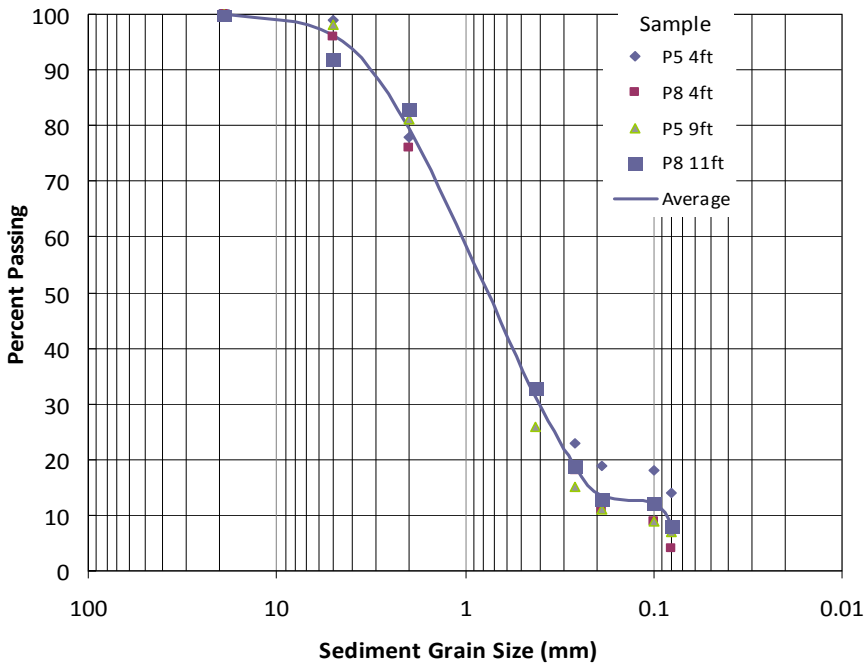


Figure 3-3. Representative grain size distribution for propwash modeling and analysis

3.2.3. Tide Elevations

Propwash scour is a cumulative effect which reflects processes that occur at all tide levels. During high tide, propwash velocities are reduced by the greater distance between the velocity source and bottom. Bottom scour is less likely or scour depth is smaller at times of high tide. Conversely, during low tide propwash velocities are greater at the bottom; thus, bottom scour is more likely. For localized scour in an unconsolidated sand/gravel/shell bottom, there will also be some recovery of bottom elevation between extreme events. Therefore, it is reasonable to use a more frequent event in the analysis of scour conditions. However, to provide a conservative estimate of possible scour, a MLLW water elevation was selected as a design water elevation for the propwash analysis.

3.2.4. Bathymetry

Available bathymetric survey data were compiled and processed to develop input for propwash numerical modeling at the locations of the four terminal alternatives. For the No-Build and Existing Site Improvements Alternatives, data of a September 2008 bathymetric survey (Figure 3-4) was used to develop a propwash modeling grid.

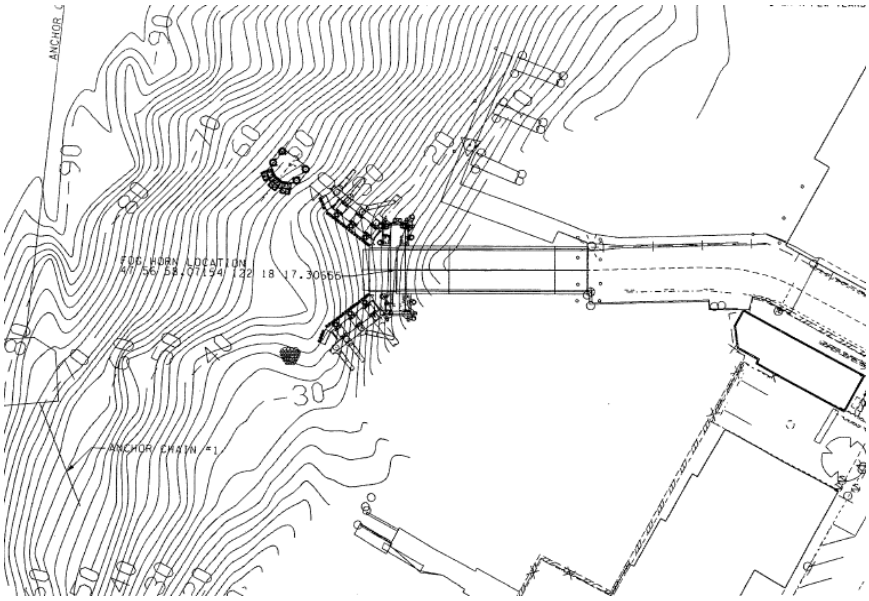


Figure 3-4. September 2008 bathymetric survey

Based on data from this survey, the numerical modeling grids were constructed using bathymetry shown for the No-Build Alternative in Figure 3-5.

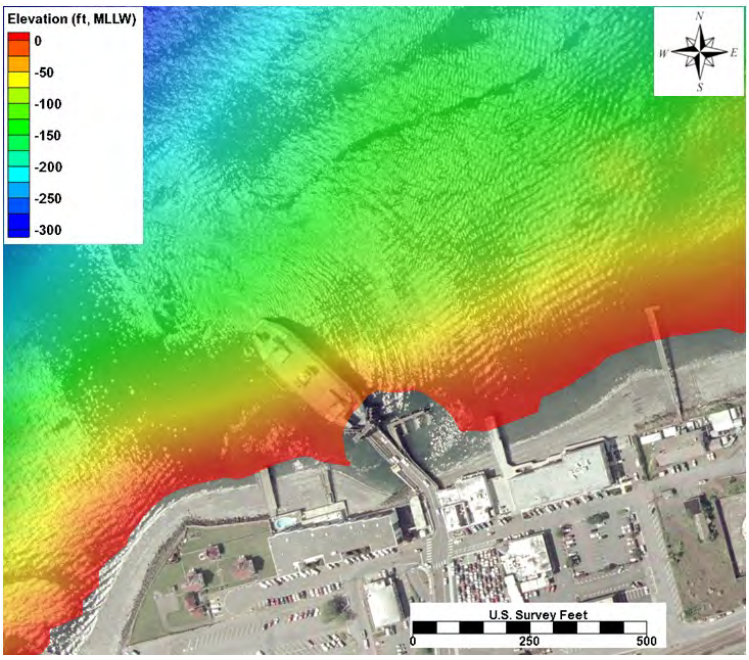


Figure 3-5. No-Build Alternative propwash numerical modeling bathymetry

Bathymetric survey data from a previous study were used for constructing the numerical modeling grids for Elliot Point 1 and Elliot Point 2 Alternatives. Numerical modeling grids for these alternatives also included removal of the pier and dredging the bottom under the pier at the intersection of the ferry route to accommodate safe navigation of the ferry. A dredged cut was built into the modeling

grid with the dimensions: Depth = 26.0 ft MLLW, Width = 330 ft, and dredged side slopes at 3H:1V. Figure 3-6 shows the bottom configuration for Elliot Point 1.

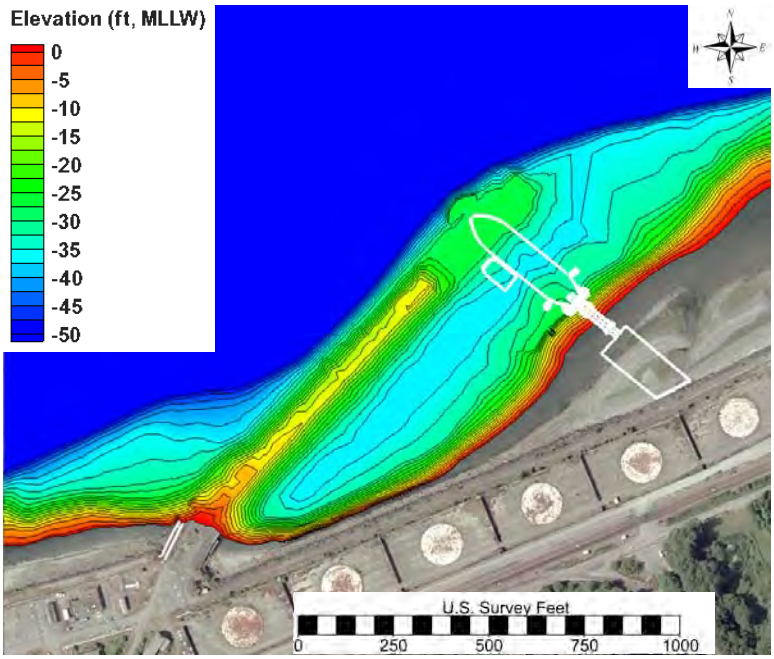


Figure 3-6. Elliot Point 1 Alternative propwash numerical modeling bathymetry

A numerical modeling grid for Elliot Point 2 was developed from the bathymetry data, with a dredged cut having dimensions of depth = 26 ft MLLW, width = 330 ft, and dredged slope of 3H:1V. This bottom configuration is shown in Figure 3-7. The modeled depth and width of the dredged cut are based on dimensions maintained at the existing terminal and represent worst case scenario of propwash scour at the future terminal. A shallower depth results in smaller under-keel clearance and greater scouring velocity due to propwash. Dimensions of the dredged cut described in the FEIS are larger than those input to the model. During final design the dimensions could possibly be made larger than modeled dimensions. If the dredged cut is designed to be deeper and wider than the modeled cut, propwash velocity impinging on the dredged surface will be less than the modeled velocity.

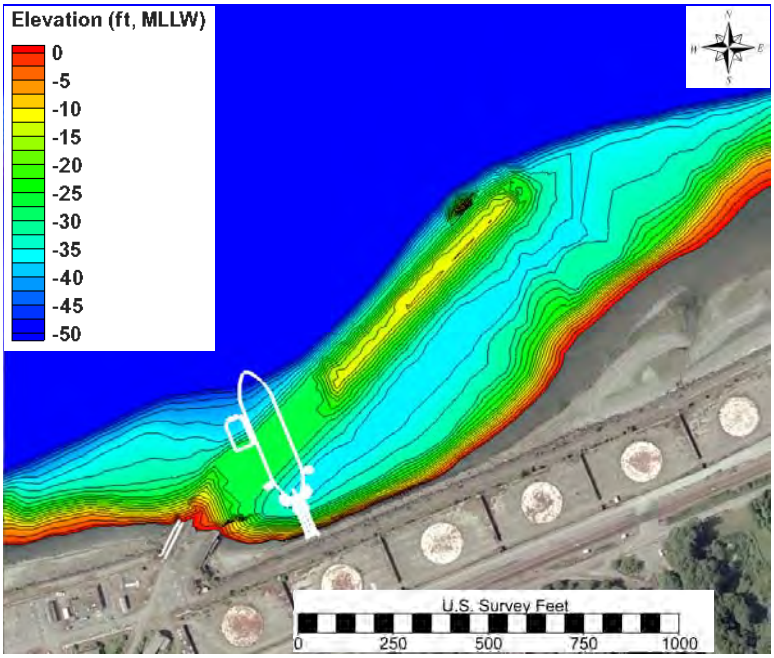


Figure 3-7. Elliot Point 2 Alternative propwash numerical modeling bathymetry

3.3. Modeling Results and Analysis

3.3.1. No-Build Alternative

3.3.1.1. Physical Setting

Propwash modeling and analysis of the existing terminal was conducted for two purposes: to validate and calibrate the methodology to compute propwash depth of scour, and to develop a baseline for the alternatives analysis. Modeling of existing conditions was conducted for two ferry vessels: Issaquah Class Ferry (existing vessel); and a future New 144 Auto Class Ferry. Propwash of the existing ferry at the existing terminal was modeled to validate and calibrate the depth of scour modeling approach. Modeling of the existing terminal and future vessel was conducted as part of the comparative analysis procedure for alternatives evaluation.

The critical maneuver of a ferry under any alternative is docking when the vessel approaches the dock and propeller thrust is directed toward shore. Results of JETWASH modeling for existing conditions and the existing ferry are shown in Figure 3-8. The figure shows a plan view of bottom velocities behind (downflow from) the propeller, in color format at MLLW tide level. Graphs of bottom velocity pattern in the following figures do not show values between horizontal distance from the propeller of zero to about 40 ft because the jet has not fully formed in such close proximity to the propeller (note zone of jet formation in Figure 3-1).

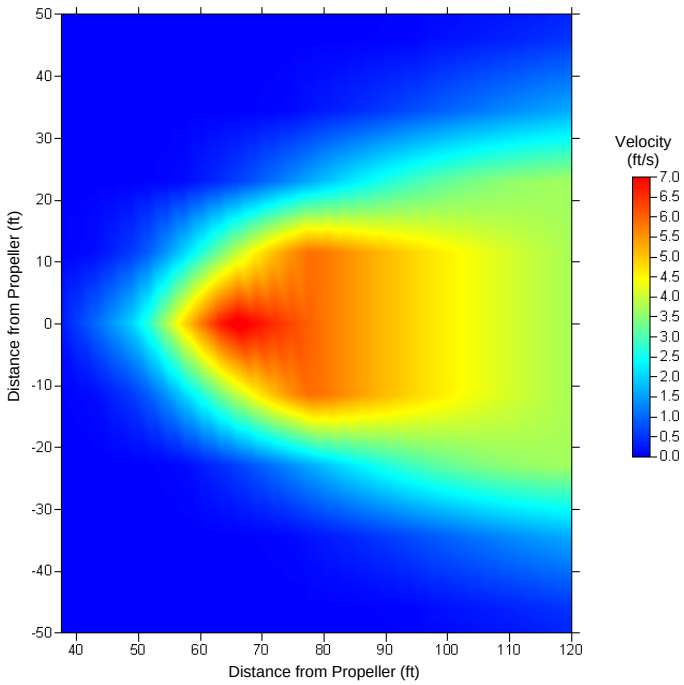


Figure 3-8. Existing conditions bottom velocities modeled with existing terminal and existing Issaquah Class Ferry

Model output shows that 7.2 ft/sec, the largest bottom velocity and represented by red in Figure 3-8, is at the location of 66 ft from the propeller. This velocity attenuates rapidly with distance from the propeller. For example, at 120 ft downflow the propwash velocity at the bottom is 3.7 ft/sec. The velocity pattern in a vertical plane through the velocity jet is shown in Figure 3-9.

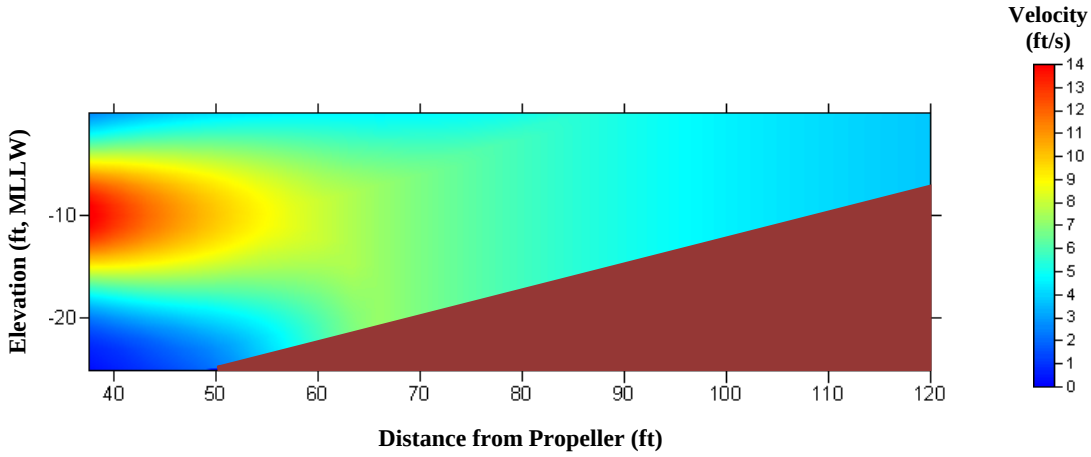


Figure 3-9. Velocity pattern in vertical plane, existing terminal and existing Issaquah Class Ferry

The figure shows velocity variation and the existing bottom slope along the centerline of the terminal. These propwash velocities represent maximum velocity with distance from the propeller because the vertical plane is projected through the jet centerline. Propwash velocities offset from the centerline are lower. Velocity developed in this configuration is further used to verify the modeling approach and specify existing conditions for bottom velocity and scour.

3.3.1.2. Scour Depth Armoring Method Verification and Calibration

Applicability of the procedure to compute scour depth, limited by the process of self-armoring, is verified using conditions at the existing terminal. The calibrated procedure is then used to compute depth of scour for the proposed alternatives. Computations of depth of scour with armoring development is based on the assumption that scour will occur at areas where propwash hydrodynamic force exceeds the surface sediment stability criterion. Depth of scour is computed assuming that when a propeller jet impacts the bottom, it washes away all particles of sediment smaller than a minimum stable size in the surface zone. The larger particles in the sediment mixture then concentrate at the bottom, where they form a layer that effectively armors the bottom, preventing further scour. Armoring layer thickness depends on stable sediment size and the original size distribution, and typically is equal to 3 to 5 diameters of the stable particles. It is assumed that this process changes the size composition of only the surface layer.

This methodology was verified by comparing the results of the scour-with-armoring approach with that of an alternative scour-without-armoring approach. In the alternative approach, it is assumed that armoring does not occur and bottom scour continues to the depth where propwash velocity decreases to a threshold value for the median sediment size. For this analysis, propwash bottom velocities below the centerline of the jet were compared to threshold velocities of sizes at cumulative fractions of the representative bottom sediment. For example, a high velocity might destabilize a grain size at which 90 percent of the sediment is finer, but farther from the propeller the stable size might be the size at which 50 percent is finer. Results of comparing propwash velocity with threshold velocity for the sediment gradation are shown in Figure 3-10. The numerical fraction on the right-hand axis of the figure is not a measure of sediment size, but represents a percent of the sediment mixture. For example, 0.5 corresponds to the median grain size of the distribution. Threshold velocity for suspension of sediment of a given size was determined according to the procedure of Cheng and Chiew (1999).

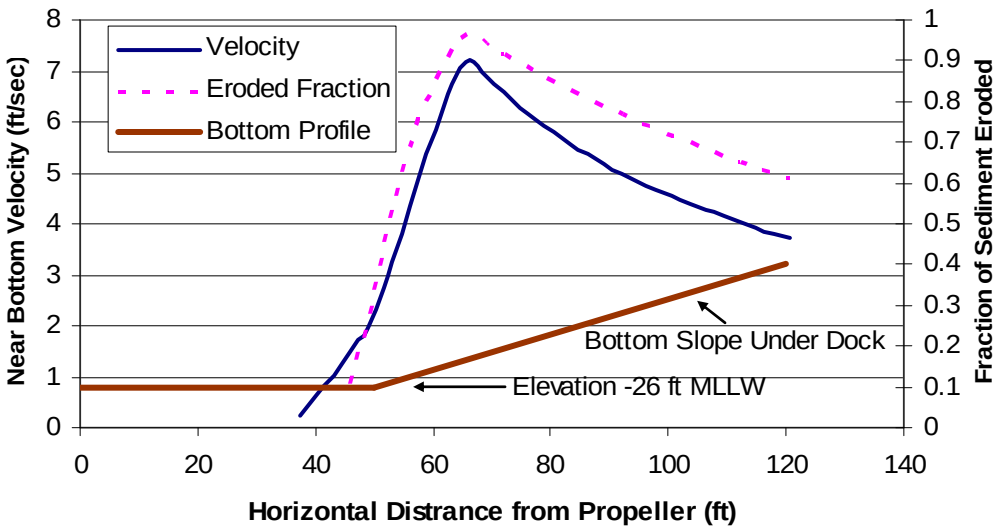


Figure 3-10. Existing ferry propwash velocity near the bottom at existing terminal and fraction of bottom sediment eroded by the propwash

The figure shows the fraction of material that would be eroded on the bottom slope for the modeled propwash bottom velocity (existing ferry at existing terminal) and the representative Mukilteo sediment grain size composition. The figure shows that sediment particles of the median size and smaller are not stable on the slope at locations landward from approximately 50 ft downflow from the propeller. In other words, without self-armoring through selective removal of unstable sizes, the slope should have eroded until the velocity impinging on the bottom was below threshold for suspension. The following text identifies and explains flaws with the scour-without-armoring approach.

The representative grain size distribution shown in Section 3.2.2 indicates the median sediment size is 0.8mm. A threshold velocity for suspended transport of this particle is 2.8 ft/sec. The iterative propwash modeling was conducted by artificial deepening of the slope until bottom velocity at any location on this slope is equal to or smaller than 2.8 ft/sec. Results of analysis show that to achieve this sediment (D_{50}) stability condition, the bottom depth should be at least 58 ft. In this case, the scour depth should have been 20 to 30 ft. However, survey data show that this is not realistic. Lead line soundings taken by WSF in the vicinity of the transfer span towers in 1982, 2008, and 2009 (see Figure 3-11) are the basis of this depth change evaluation.

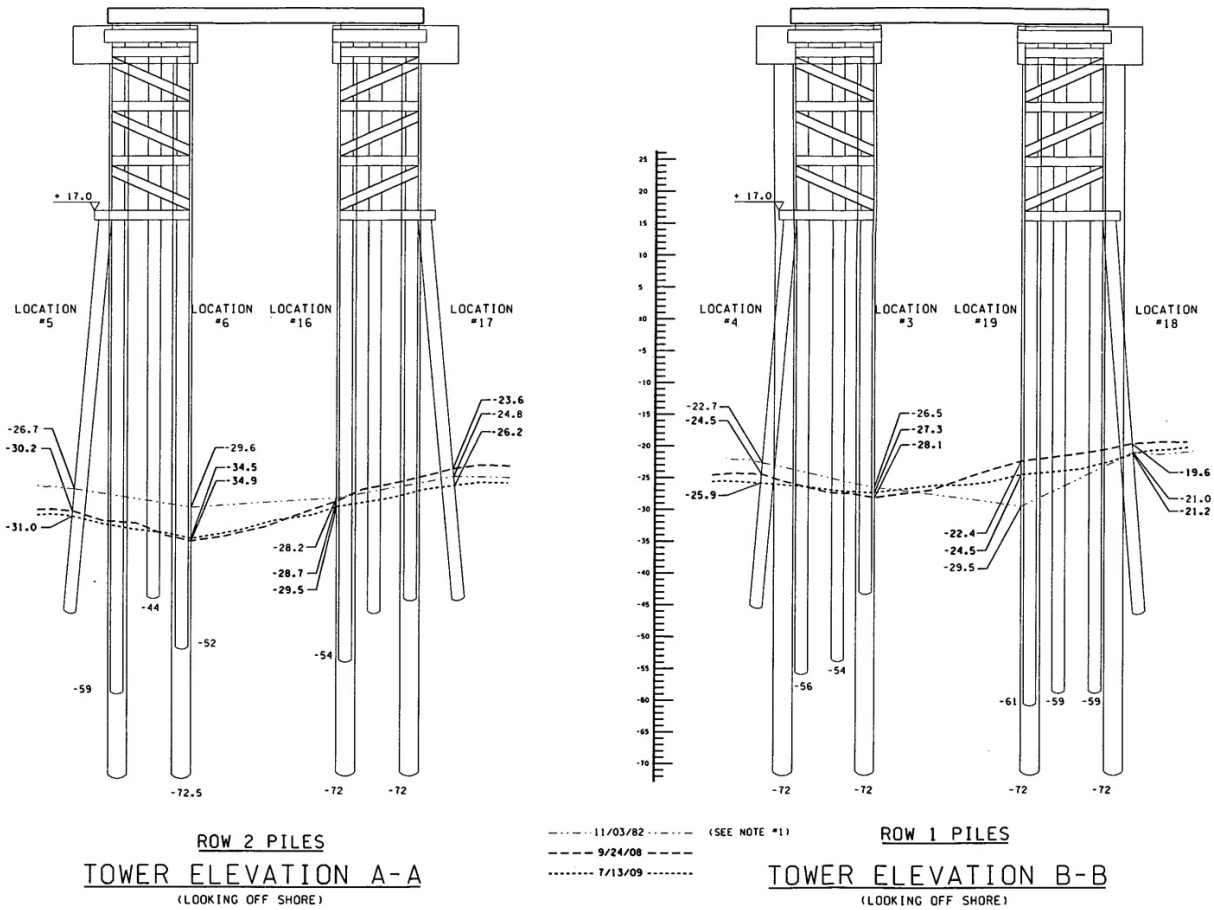


Figure 3-11. Lead line soundings over time at the transfer span towers for the Mukilteo Ferry Terminal; depths are in ft below MLLW

Figure 3-11 shows that maximum bottom depths at different locations did not change during the period from 2008 (the modeled conditions) to 2009. It appears that the bottom slope is in an equilibrium stability condition. The bottom scour of 20 to 25 ft predicted with the concept of neglecting armoring did not occur. In contrast to that, the bottom appears to be stable and no deepening continues to occur, suggesting that the bottom material became armored some time ago and maintains a stable and equilibrium bottom slope at Mukilteo Terminal. Based on the measurement, it appears that the concept of scour-without-armoring is not correct because it contradicts actual data.

Figure 3-11 demonstrates that in 1982 the original bottom depth at Section A-A was 29.6 ft MLLW. Over time, the maximum depth along this section increased and established an equilibrium scoured bottom at elevation -34.9 ft MLLW. Therefore, the historically observed depth of scour at this location may be estimated at 5.3 ft. At Section B-B in 1982 the original bottom depth was 26.5 ft MLLW. Similar to Section A-A, the maximum depth at Section B-B had increased and an equilibrium scoured bottom was established at elevation -28.1 ft MLLW. Therefore, the observed depth of scour at this location may be estimated at 1.6 ft.

Based on measured data, the historical depth of scour at the existing Mukilteo Terminal is estimated to range from 1.6 ft to 5.3 ft. Using the representative grain size distribution, a peak bottom velocity of 7.2 ft/sec and a methodology for determining depth of scour with armoring, the scour depth is computed to be 2.2 ft, with size of armoring material larger than 4.1 mm and thickness of armoring layer of 0.2 ft.

In summary, the verification and calibration of the armoring methodology suggests that bottom scour at the Mukilteo Terminal occurs in areas where propwash velocities exceed threshold velocities for suspension of bottom sediment. At these areas, scour continued until armoring of sufficient thickness developed. Armoring at the existing terminal is calculated to have developed with the size of material that represents 92 percent passing. Thickness of the armoring layer is assumed equal to 3 diameters of the armor material.

3.3.1.3. Propwash Modeling and Scour Analysis for Future Vessel

Propwash modeling and scour depth analysis was conducted for the existing ferry terminal with the New 144 Auto Class Ferry, using input listed in Table 3-1. Figure 3-12 shows modeling results as a plan view of bottom velocities behind the propeller at the existing terminal.

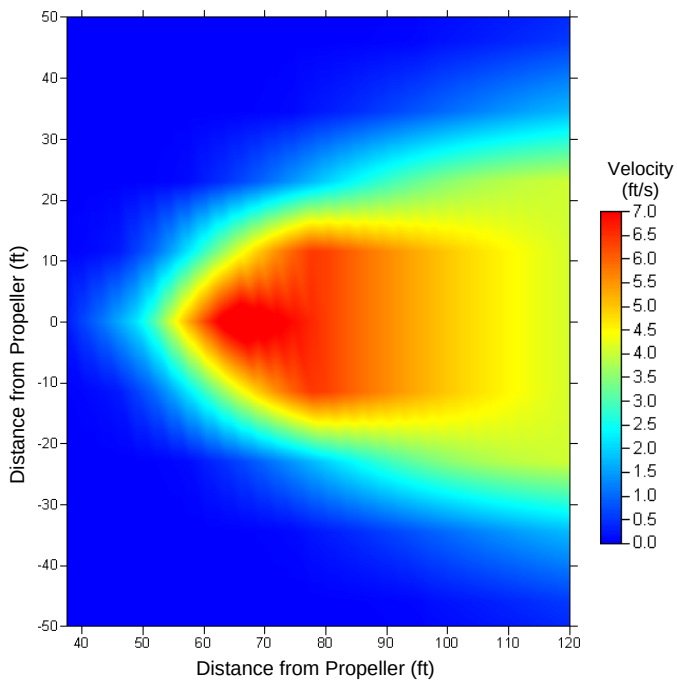


Figure 3-12. No-Build Alternative bottom velocities modeled with existing terminal and New 144 Auto Class Ferry

Bottom propwash velocities below the centerline of the velocity jet are plotted with distance behind the propeller in Figure 3-13 and compared with bottom velocities of

the existing ferry. The figure shows that the New 144 Auto Class Ferry produces slightly greater peak bottom velocity than the existing ferry for otherwise similar conditions. Figure 3-14 shows the modeled velocities of the New 144 Auto Class Ferry at the existing terminal, as a vertical section through the velocity jet. The following figures are scaled to emphasize pertinent physical processes and features. The shoreline position is not shown because it is subjected to propwash velocity that is only lower than the threshold velocity for medium size sediment.

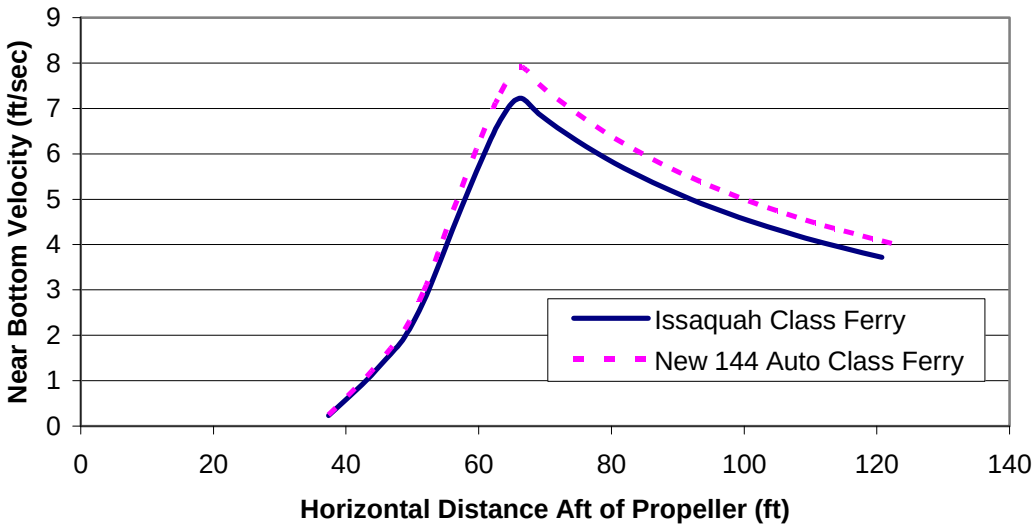


Figure 3-13. Near bottom velocity below propwash jet of New 144 Auto Class Ferry and of Issaquah Class Ferry at existing terminal

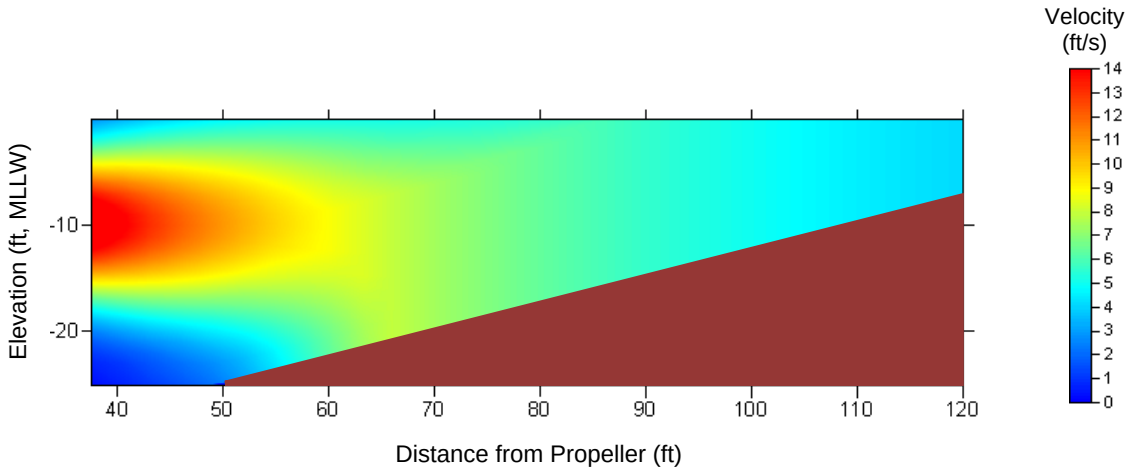


Figure 3-14. Velocity pattern in vertical plane, existing terminal and New 144 Auto Class Ferry

Scour depth was calculated with bottom velocity produced by the New 144 Auto Class Ferry and the self-armoring procedure described above. Scour calculation results are plotted with distance from the propeller in Figure 3-15.

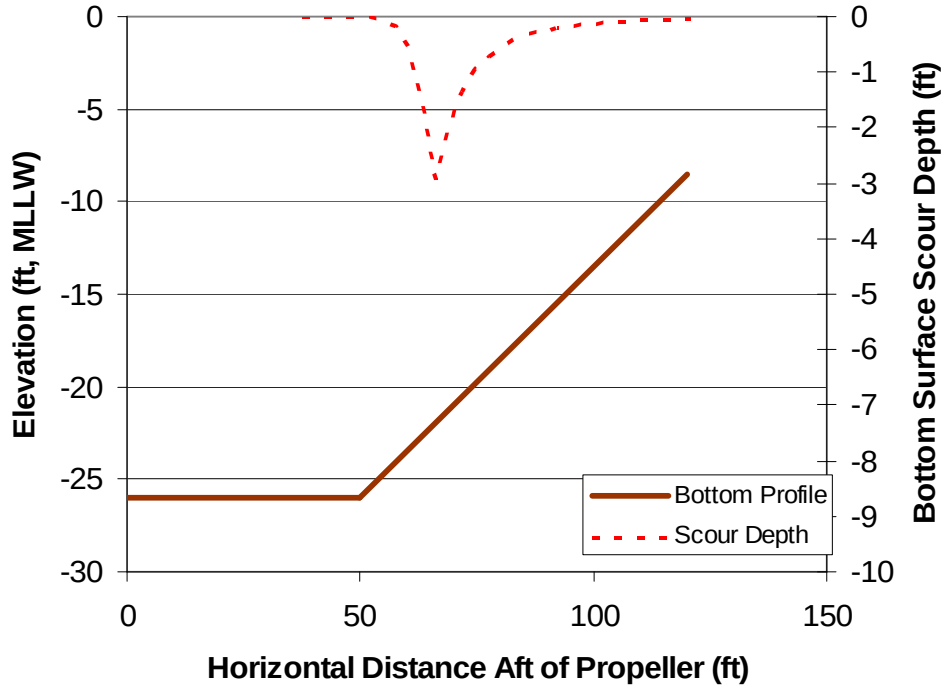


Figure 3-15. No-Build Alternative depth of bottom scour below propwash jet of New 144 Auto Class Ferry at existing terminal

Calculation results show that a localized maximum scour depth of 2.8 ft would develop for the No-Build Alternative upon operation of the New 144 Auto Class Ferry. This scour would occur at the natural depth of approximately 20 to 25 ft MLLW, located 150 to 200 ft offshore from the MHHW shoreline.

Because of the localized area of scour and the location of this area far away from the shoreline, it is likely that this scour depth would not produce any measurable adverse impact on shore erosion at the Mukilteo Terminal. This conclusion is supported by calculating a depth of closure that defines the seaward boundary of sediment transport of beach material. The results of calculations using a procedure presented by Houston (1996) show that depth of closure at the Mukilteo shoreline is approximately 15 ft MLLW, which is shallower than the location of the existing scour hole at the terminal. The implication of this comparison of depths is that sediment to be possibly scoured upon operation of the new ferry is already outside the littoral system, and is not currently available for maintaining the shoreline and beaches. Bottom scour by propwash at this location can be easily controlled by placement of coarser sediment of the size to resist movement by the calculated maximum bottom velocity.

3.3.2. Existing Site Improvements Alternative

Propwash modeling and scour depth analysis were conducted for the Existing Site Improvements Alternative for the New 144 Auto Class Ferry with the same procedures that were used for the existing terminal in Section 3.3.1.3. Figure 3-16 shows a plan view of bottom velocities behind the propeller at the location of the Existing Site Improvement Alternative. Figure 3-17 shows the modeled velocities of

the New 144 Auto Class Ferry at the Existing Site Improvements location, as a vertical section through the velocity jet.

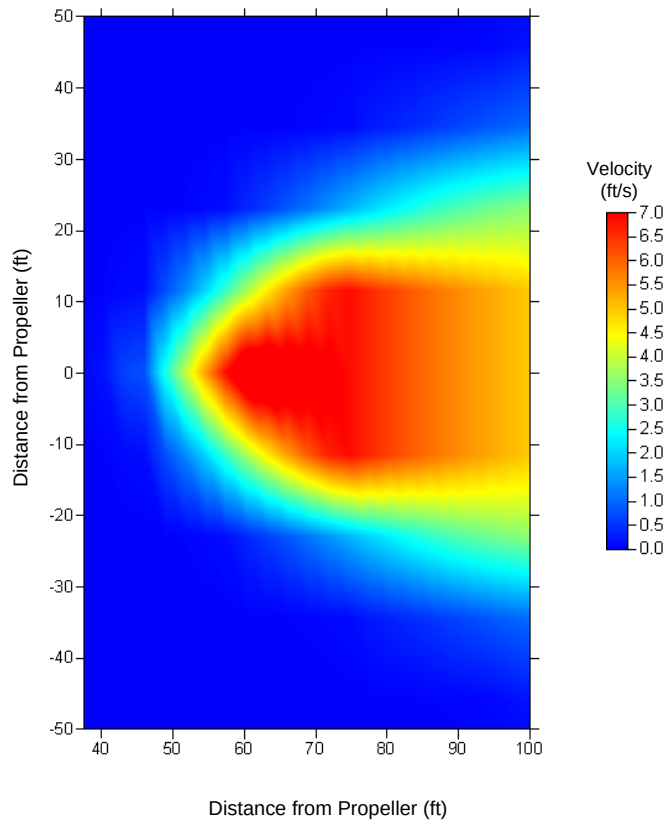


Figure 3-16. Existing Site Improvements Alternative bottom velocities with New 144 Auto Class Ferry

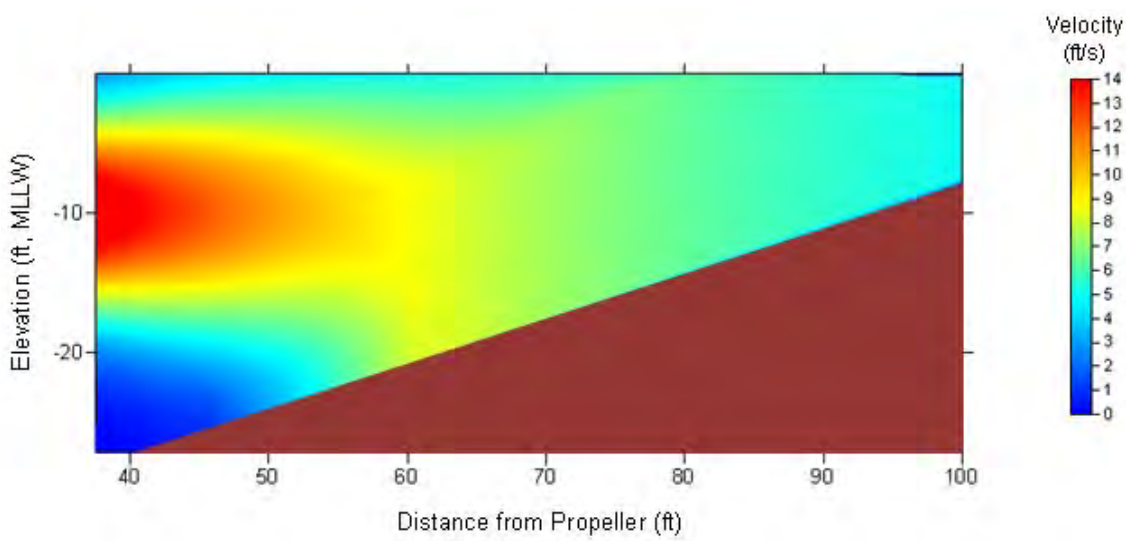


Figure 3-17. Velocity pattern in vertical plane, Existing Site Improvements Alternative with New 144 Auto Class Ferry

Scour depth was calculated with bottom velocity produced by the New 144 Auto Class Ferry and the self-armoring procedure. Scour calculation results for the Existing Site Improvements Alternative are plotted with distance from the propeller in Figure 3-18.

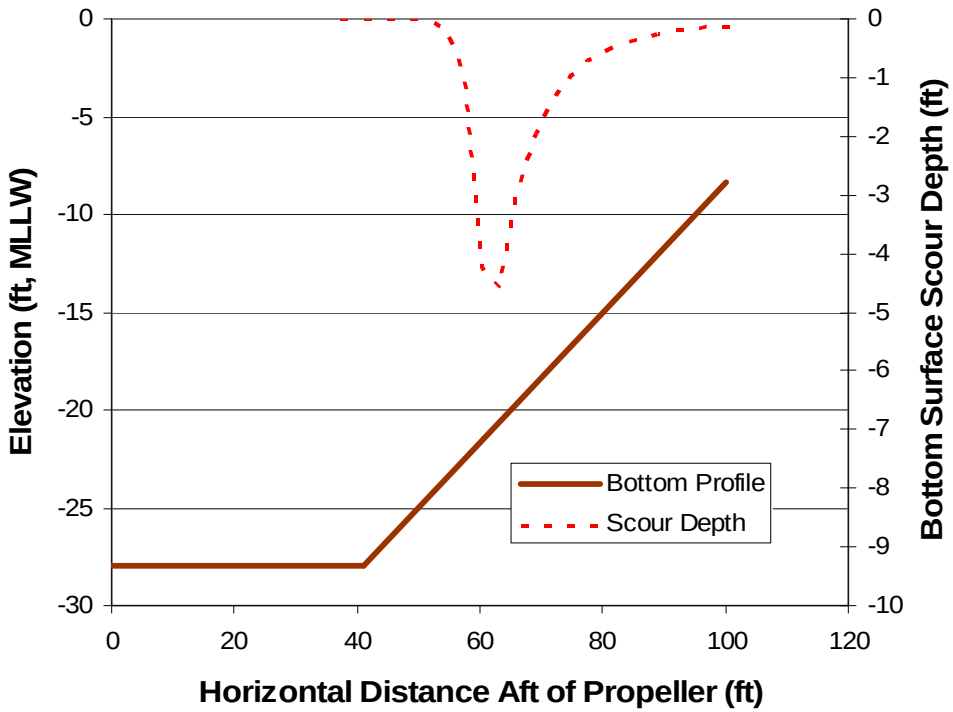


Figure 3-18. Existing Site Improvements Alternative depth of bottom scour below propwash jet of New 144 Auto Class Ferry

Calculation results show that localized scour having a maximum depth of 4.5 ft would develop for the Existing Site Improvements Alternative upon operation of the New 144 Auto Class Ferry. This scour would occur at the natural depth of approximately 15 to 20 ft below MLLW, located 100 to 150 ft offshore from the MHHW shoreline. Similar to the No-Build Alternative (Section 3.3.1) the bottom scour by propwash at this location can be easily controlled by placement of coarser sediment of the size to resist movement by the calculated maximum bottom velocity.

Because of the localized area of scour and location of this area far away from the shoreline, it is likely that this scour depth would not produce a measurable adverse impact on shore erosion at Mukilteo Terminal. This conclusion is supported by calculating a depth of closure that defines the seaward boundary of sediment transport of beach material. The results of calculations using a procedure presented by Houston (1996) show that depth of closure at the Mukilteo shoreline is approximately 15 ft below MLLW, and is shallower than the depth of the scour hole at the existing terminal. The implication is that sediment to be possibly scoured upon operation of the new ferry is already outside the littoral system and currently is not available for maintaining the shoreline and beaches.

3.3.3. Elliot Point 1

Propwash modeling and scour depth analysis were conducted for the Elliot Point 1 Alternative for the New 144 Auto Class Ferry with the same procedures that were used for the existing terminal in Section 3.3.1.3. Figure 3-19 shows a plan view of bottom velocities behind the propeller at the location of the Elliot Point 1 Alternative. Figure 3-20 shows the modeled velocities of the New 144 Auto Class Ferry at the Elliot Point 1 location, as a vertical section through the velocity jet.

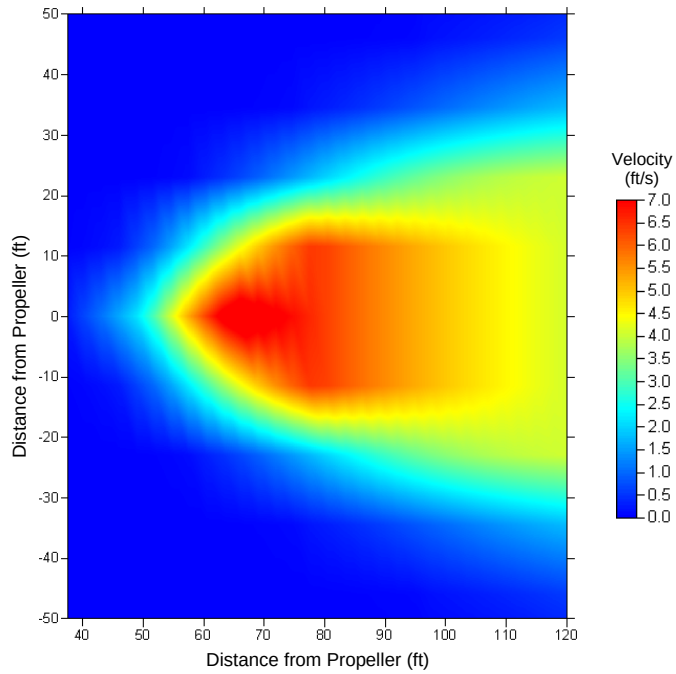


Figure 3-19. Elliot Point 1 Alternative bottom velocities, with New 144 Auto Class Ferry

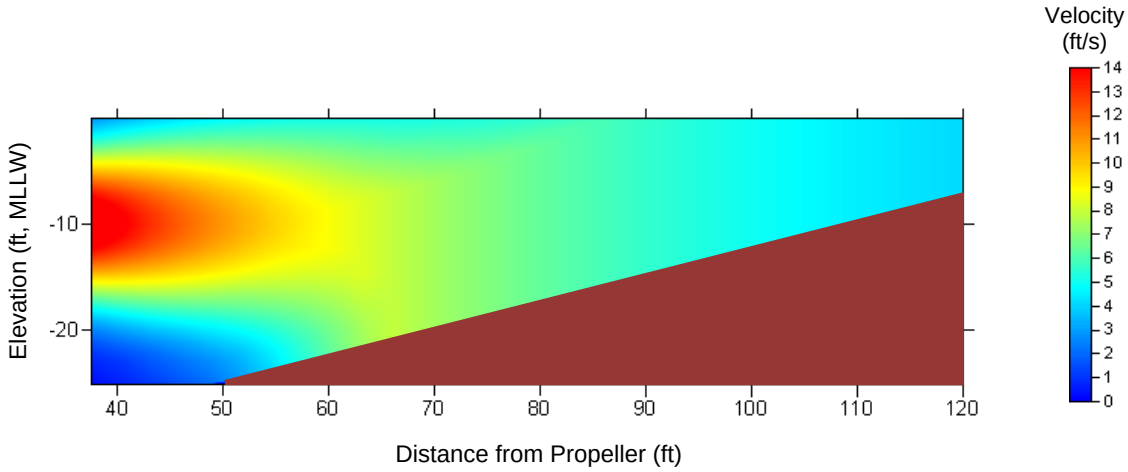


Figure 3-20. Velocity pattern in vertical plane, Elliot Point 1 Alternative with New 144 Auto Class Ferry

Scour depth was calculated with bottom velocity produced by the New 144 Auto Class Ferry and the self-armoring procedure. Scour calculation results for the Elliot Point 1 Alternative are plotted with distance from the propeller in Figure 3-21.

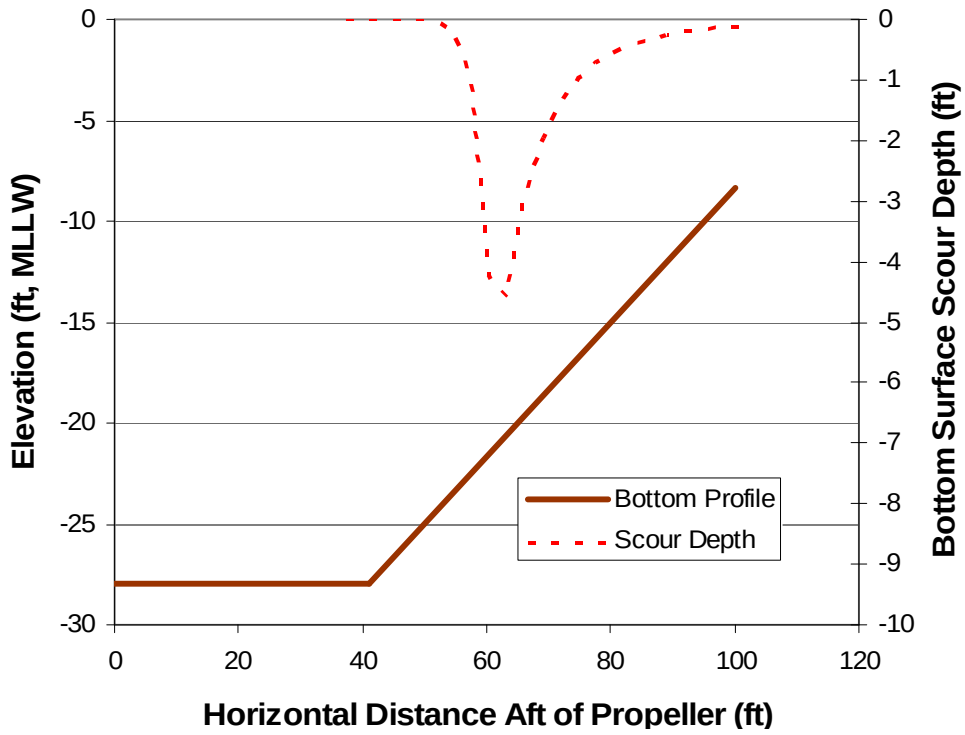


Figure 3-21. Elliot Point 1 Alternative depth of bottom scour below propwash jet of New 144 Auto Class Ferry

Calculation results show that localized scour with maximum depth of 4.5 ft would develop at the bottom slope for the Elliot Point 1 Alternative upon operation of the New 144 Auto Class Ferry. This scour would occur at the depth of approximately 10 to 15 ft MLLW, located 100 to 150 ft offshore from the MHHW shoreline. The location of the scour hole and its distance from the shoreline may result in an adverse impact on shore erosion. However, if this occurs, this erosion will be localized in the vicinity of the ferry terminal. As discussed above (Section 3.3.2) the bottom scour by propwash at this location can be similarly controlled by placement of coarser sediment of the size to resist movement by the calculated maximum bottom velocity.

3.3.4. Elliot Point 2

Propwash modeling and scour depth analysis were conducted for the Elliot Point 2 Alternative for the New 144 Auto Class Ferry with the same procedures that were used for the existing terminal in Section 3.3.1.3. Figure 3-22 shows a plan view of bottom velocities behind the propeller at the location of the Elliot Point 2 Alternative. Figure 3-23 shows the modeled velocities of the New 144 Auto Class Ferry at the Elliot Point 2 location, as a vertical section through the velocity jet.

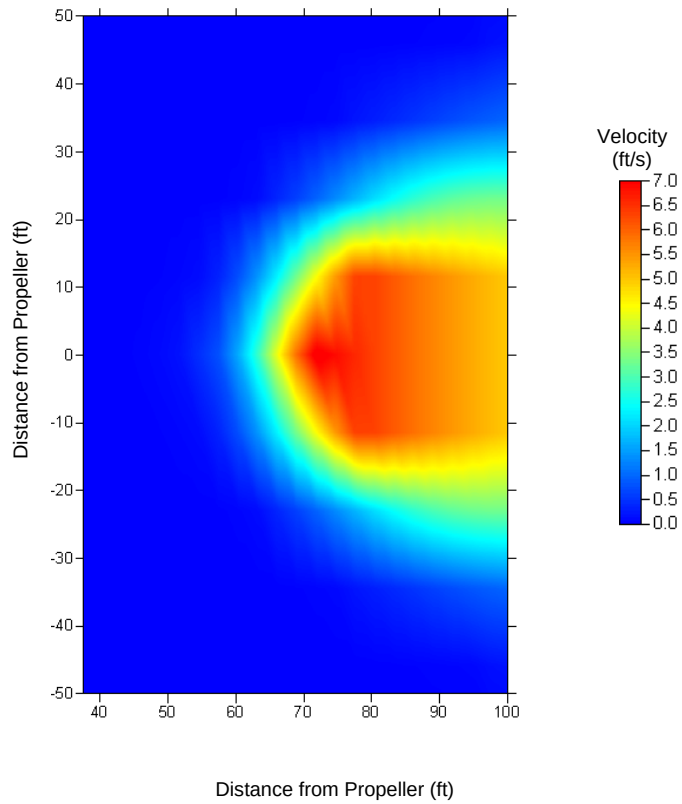


Figure 3-22. Elliot Point 2 Alternative bottom velocities, with New 144 Auto Class Ferry

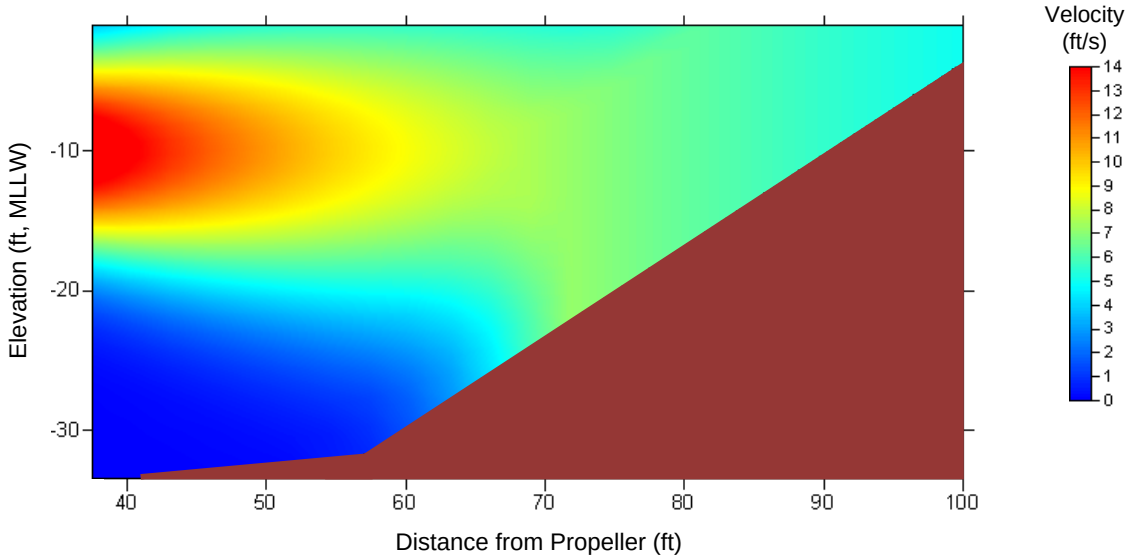


Figure 3-23. Velocity pattern in vertical plane, Elliot Point 2 Alternative with New 144 Auto Class Ferry

Scour depth was calculated with bottom velocity produced by the New 144 Auto Class Ferry and the self-armoring procedure. Scour calculation results for the Elliot Point 2 Alternative are plotted with distance from the propeller in Figure 3-24. The less intense scour at Elliot Point 2 is due to deeper water at the end of the transfer span than for Elliot Point 1. The expected plan-view extent of propeller scour of the bottom for Elliot Point 2 is shown in Figure 3-24b. The scour extent was estimated by modeling the pattern of velocity parallel with the bottom and selecting the contour of velocity that was the threshold for motion of the median size of bottom sediment (0.8 mm diameter). The value of that contour of near-bottom velocity threshold is 1.5 ft/sec. The velocity contour showing the plan extent of bottom scour at Elliot Point 2 is shown in Figure 3-24b.

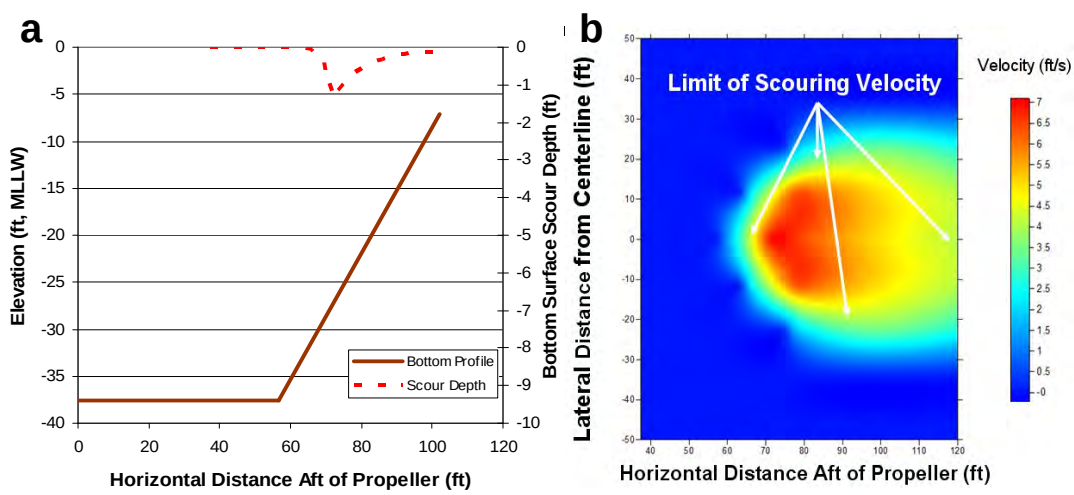


Figure 3-24. Elliot Point 2 Alternative, bottom scour below propwash jet of New 144 Auto Class Ferry: a) depth of scour, b) plan extent of scour

Calculation results plotted in Figure 3-24 show that a small and localized maximum scour depth of 1.4 ft may develop at the lower part of the slope for the Elliot Point 2 Alternative. This scour would occur at the natural depth of approximately 20 to 25 ft MLLW, located 40 to 50 ft offshore from the MHHW shoreline. The small depth of scour and the location at the natural depth of more than 20 ft preclude any detectable impact from this scour on the adjacent shoreline. As discussed above in Section 3.3.3, the bottom scour by propwash at this location can be similarly controlled by placement of coarser sediment of the size to resist movement by the calculated maximum bottom velocity.

3.4. Section Summary

Propwash numerical modeling and analysis were conducted to determine and quantify the potential effects from ferry propeller-induced currents on nearby bottom slope and bottom sediments at the proposed ferry terminal alternative locations. These quantitative estimates of potential effects will be further used in evaluation of the alternatives and Environmental Impact Statement (EIS) discipline reports.

Numerical modeling of propwash was conducted with several steps using the reliable and industry-accepted 2-D numerical model JETWASH.

Results of propwash and scour depth modeling for all alternatives are summarized in Table 3-2. Modeling for the No-Build Alternative showed that a relatively small scour depth (approximately 2.9 ft) may occur at the bottom slope at the natural depth of approximately 20 to 25 ft below MLLW. This scour would be localized and would not produce any measurable adverse impact on shore erosion at the existing Mukilteo Terminal.

Localized scour with a maximum depth of approximately 4.5 ft may develop for the Existing Site Improvements Alternative at the natural depth of approximately 15 to 20 ft below MLLW. This scour would be located seaward from the limit of profile change by littoral processes (depth of closure), and would not produce any measurable adverse impact on shore erosion at the Mukilteo Terminal.

Modeling results show that localized scour (maximum depth approximately 4.5 ft) may develop at the bottom slope at the Elliot Point 1 Alternative. The possible location of a scour hole and its distance from the shoreline may result in adverse, but localized, impact on shore erosion in the vicinity of the terminal area. Material scoured by propwash will be dispersed by the propwash flow and may settle in a lower velocity regime or may enter the littoral system.

Modeling results and analysis show that a small and localized scour area with a maximum depth of 1.4 ft may develop at the lower part of the slope for the Elliot Point 2 Terminal Alternative. This scour would occur at the natural depth of approximately 20 to 25 ft MLLW. The small depth of the scour and location on the profile at the natural depth of more than 20 ft below MLLW exclude any detectable impact from this scour on the adjacent shoreline.

Bottom scour by propwash at these terminal locations can be easily controlled by placement of coarser sediment of the size to resist movement by the calculated maximum bottom velocity.

Table 3-2. Results of propwash and scour depth modeling for all alternatives

Alternative	Maximum Scour Depth (ft)	Location of Scour Depth on the Bottom Slope	Possible Effect on Shoreline Erosion
No-Build	2.9	20 to 25 ft MLLW	No detectable impact
Existing Site Improvements	4.5	15 to 20 ft MLLW	No detectable impact
Elliot Point 1	4.5	10 to 15 ft MLLW	Localized impact
Elliot Point 2	1.4	20 to 25 ft MLLW	No detectable impact

4. Vessel Wake Wash

4.1. Introduction

The objective of vessel wake modeling and analysis was to determine and quantify potential effects on shore erosion from ferry-generated wake wash at the four proposed project alternatives of the new ferry terminal (No-Build, Existing Site Improvements, Elliot Point 1 and Elliot Point 2 (see Section 4.2 for description of alternatives). These potential effects are defined here based on possible changes of vessel wake and sediment transport parameters at the shoreline relative to existing conditions. This information will be further used within other EIS discipline reports in the evaluation of potential environmental effects and alternatives analysis.

Vessel wake modeling and analysis included developing input parameters, numerical modeling of ferry-induced wakes, wake propagation to the shoreline, wake transformation on the bottom slope, evaluation of vessel wake parameters at the shoreline, and vessel wake-generated potential swash sediment transport.

Vessel wake source data represent ferry wake wash in the proximity of the ferry sailing line in the absence of coastal features. Vessel wake source data were used as input to a 2-D spectral wave transformation model (see Section 1.2) to evaluate wake wash propagation and transformation to the shoreline.

Typically, ferry wake wash source data are obtained from specific field measurements, laboratory tests, or with Computational Fluid Dynamics (CFD) modeling results validated by field or laboratory tests. During the present study, only CFD modeling results and stand-alone field measurements (measurements not designed for measuring wake wash sources or for the purpose of CFD validation) were available. CFD modeling was conducted by FRIENDSHIP-System Co. (2004) using a Rankine source SHIPFLOW model. SHIPFLOW is a non-linear, free surface boundary conditions model that simulates wave elevations within the close vicinity of the vessel hull. Further, the CFD computational procedure includes linear extrapolation and analysis of a set of wave cuts behind the hulls for the far field panels (approximately 1,000 ft).

4.2. Methodology

As discussed in Section 4.1, potential ferry wakes effects on shore erosion in relation to the terminal alternatives are defined by possible changes of vessel wake wash and sediment transport parameters relative to existing conditions. Vessel wake parameters are specified in the analysis through spectral moment wake height and peak wake period in ferry wake trains. An increase of spectral moment wake heights indicates larger amounts of wave energy delivered to the shoreline and a possible increase in erosion. Conversely, smaller spectral moment wake heights indicate a reduction of wave energy delivered to the shoreline and a possible reduction in erosion.

Sediment transport parameters are investigated through potential swash sediment transport characteristics. Potential swash sediment transport is a parameter accepted

in coastal engineering that indicates a potential shoreline response to wave or wake energy delivered to the shoreline. Increasing swash sediment transport indicates possible removal or redistribution of sediment in the nearshore zone and exacerbation of shore erosion. Smaller values of swash sediment transport indicate the ability of the shoreline to conserve more sediment in the nearshore zone and reduce or stabilize shore erosion.

Vessel wake parameters (spectral moment wake height and peak wave period) for existing conditions and four proposed alternative terminal conditions were determined using the 2-D spectral wave model SWAN (Holthuijsen *et al.*, 2004). SWAN calculates wave refraction, shoaling, breaking, white-capping, bottom friction, and wave-wave interaction for a range of wave frequencies and propagation directions. Near-field vessel wakes are determined from CFD results described in a previous study (CHE, 2006) and are used as input to the SWAN model (see Section 4.3.4).

Swash sediment transport potential was computed with empirical formulations (Puleo *et al.*, 2003) imbedded into a CHE software package. Swash sediment transport for existing conditions was assumed to be generated by existing ferry traffic[5]. Swash sediment transport for the proposed alternative terminal conditions was assumed to be generated by a new ferry operating at the specific terminal.

Input parameters for vessel wake wash and potential swash sediment transport modeling and analysis include bathymetry data, vessel dimensions, nearshore field of vessel wakes (wake wash source), and ferry operational conditions along the existing and proposed routes. The input parameters for numerical modeling and analysis are discussed in Section 4.3.

4.3. Input Parameters

4.3.1. Bathymetry Data

Available current and historical hydrographic survey data from different sources were compiled and processed to develop a sufficient bathymetry database for numerical modeling and analysis. In addition to localized survey data described in Section 2.5.3, the bathymetric database included that of the NOAA National Ocean Survey (NOAA, 2000) that extends seaward and northeast from the project area. Figure 4-1 shows the bathymetry set compiled from the three different bathymetry sources and used for modeling. Elliot Point Alternatives bathymetry includes dredged depths discussed in Section 3.2.4.

Bathymetry at alternative locations is shown in Figures 4-2 through 4-4. Existing conditions bathymetry is shown in Figure 4-2. Dredging is assumed only for Elliot Point 1 and Elliot Point 2 Alternatives as described below. Figure 4-3 shows bathymetry for simulating wake effects associated with the Elliot Point 1 Alternative. Figure 4-4 shows bathymetry for simulating wake effects associated with the Elliot

[5] Another potential contribution to swash sediment transport is energy of wind-waves. The wind-wave generated swash sediment transport factor, however, is constant for all alternatives, thus it is not included in this analysis.

Point 2 Alternative, including the assumed dredging to accommodate this ferry route. The figures show the assumed dredging through the existing bottom under the Tank Farm Pier, which would be removed with either Elliot Point Alternative.

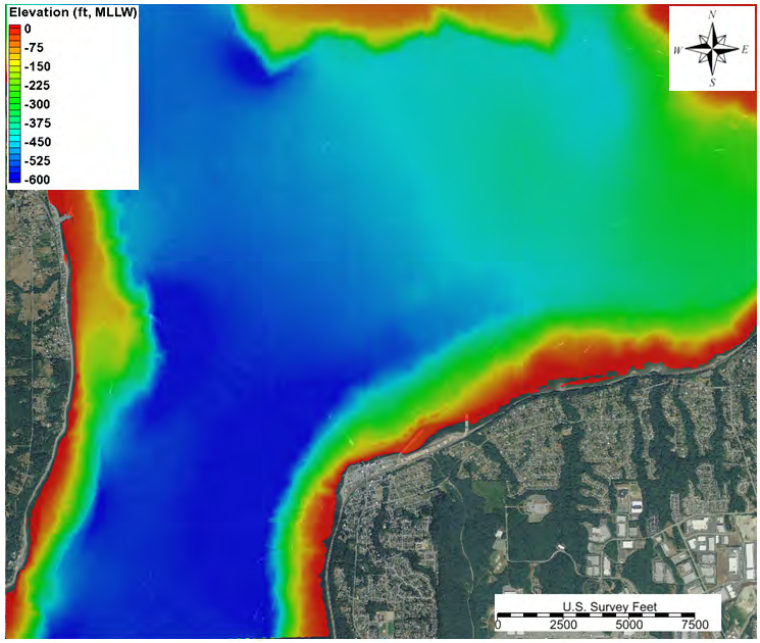


Figure 4-1. Bathymetry set used for modeling

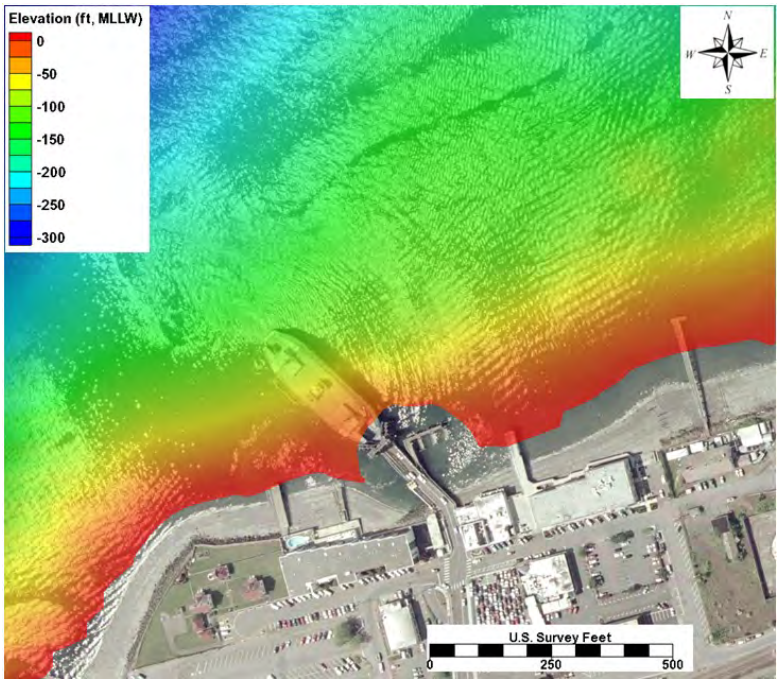


Figure 4-2. Existing conditions bathymetry

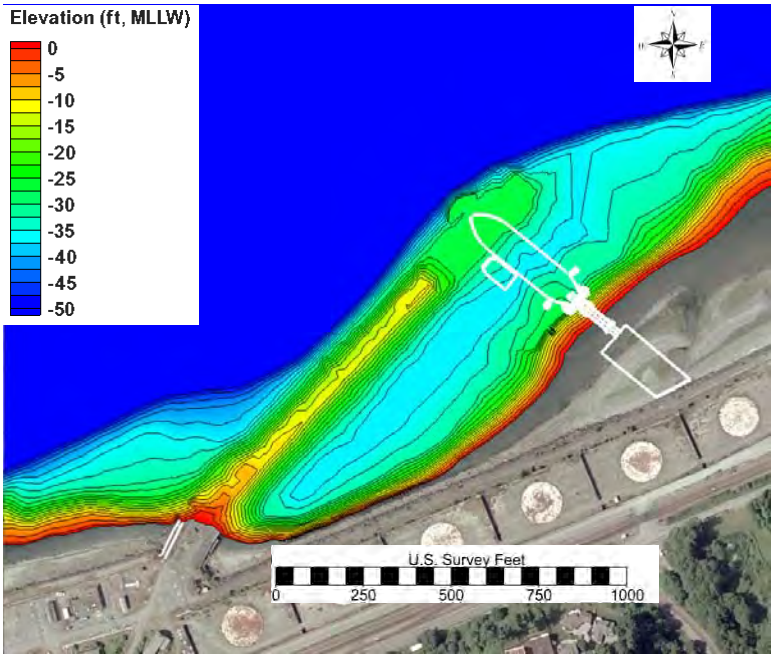


Figure 4-3. Elliot Point 1 bathymetry

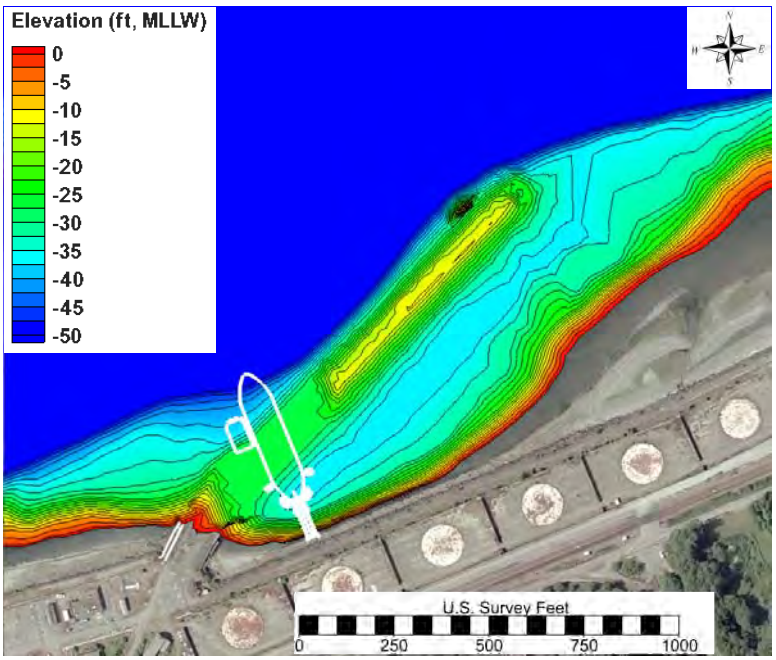


Figure 4-4. Elliot Point 2 bathymetry

4.3.2. Vessel Dimensions

Two ferry vessels were used in analysis and modeling of vessel wake effects: the existing Issaquah Class Ferry and future New 144 Auto Class Ferry. Vessel parameters relevant for the vessel wake study were compiled from various sources and are shown in Table 4-1.

Table 4-1. Specifications for present and future ferry classes operating at Mukilteo

Item	Issaquah Class	New 144 Auto Class
Length (Overall)	328 ft	362.3 ft
Beam	78.7 ft	83.3 ft
Light Ship Weight	2,228 Long Tons	Not Available
Design Load Displacement	2,950 Long Tons	Not Available
Draft at Light Ship	13.25 ft	Not Available
Draft at Design Load	15.5 ft	16.8 ft
Service Speed	16 kts	17 kts
Rudders	Conventional	High Lift

4.3.3. Ferry Operational Parameters

Operational parameters consist of ferry routes approaching the proposed alternative ferry terminals, and vessel speeds. For the purpose of numerical modeling, the most probable ferry route was developed for existing conditions and for the proposed ferry terminal alternatives, as shown in Figure 4-5. The routes are shown up to the assumed locations offshore of the proposed terminals where the ferries start slowing and enter the no-wake zone.

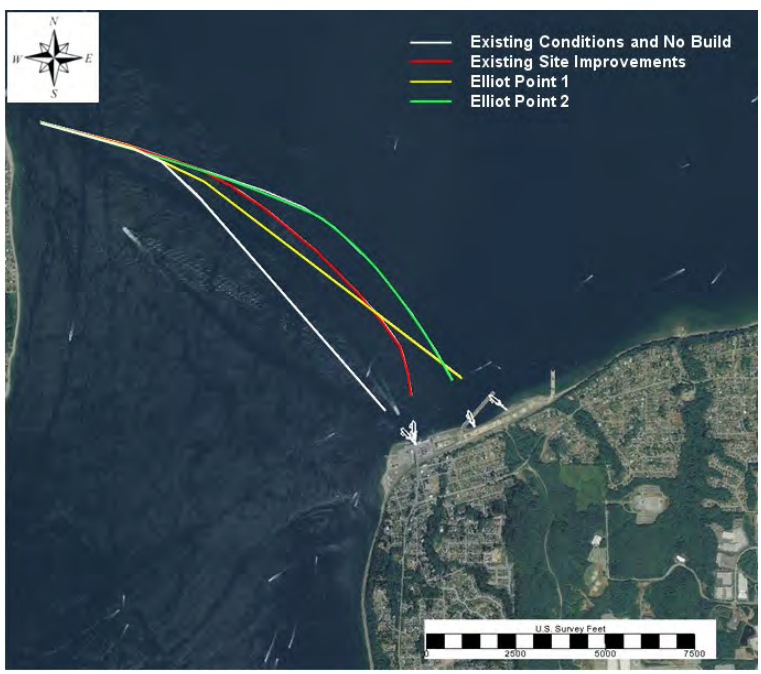


Figure 4-5. Ferry routes specified for wake analysis

4.3.4. Ferry Wake Wash Source Data and Validation

Vessel wake source data represent ferry wake wash in the proximity of the ferry sailing line in the absence of coastal features. Vessel wake source data are used as input into a 2-D spectral wave transformation model to evaluate wake wash

propagation and transformation to the shoreline. Vessel wake source for the existing Issaquah Class Ferry and New 144 Auto Class Ferry were obtained based on the results of a previous CHE study (CHE, 2006). The source of wake data for the existing Issaquah Class Ferry during the previous study was specified based on measurements in the field, in combination with CFD numerical modeling. CFD modeling was conducted by FRIENDSHIP-System Co. (2004) using the SHIPFLOW model. The results of CFD modeling show that spectral moment wake height and peak period calculated for the existing Issaquah Class Ferry are 0.62 foot and 4.9 seconds, respectively.

During the previous study (CHE, 2006) analysis and CFD numerical modeling of wake source data were conducted for the New 130 Auto Class Ferry. A source of vessel wakes for the New 144 Auto Class Ferry was not a part of the previous study. However, sources of wake data for the New 130 Auto Class Ferry and New 144 Auto Class Ferry differ so slightly from each other that they can be considered similar (JJMA, 2007). Therefore, for further analysis of vessel wakes impact, the source of wake data obtained for the New 130 Auto Class Ferry was applied for the New 144 Auto Class Ferry. Following an analytical procedure (see CHE, 2006), the longitudinal cuts of wake data of the New 130 Auto Class Ferry at 1,000 ft from the sailing line were evaluated with zero-crossing analysis. Spectral moment wake height and peak period were calculated in this procedure for the New 144 Auto Class Ferry to be 0.49 foot and 5.0 seconds, respectively.

Wake information obtained through the analysis described above was used for 2-D wake transformation modeling described in Section 4.4.1.

4.4. Vessel Wake Effects Analysis

As discussed in Section 4.1, potential ferry wake effects on shore erosion are defined by possible changes of vessel wake and sediment transport parameters relative to existing conditions. Vessel wake parameters are determined in the analysis through spectral moment wake heights in ferry wake trains. Potential sediment transport quantities are calculated with swash sediment transport relationships.

4.4.1. Vessel Wake Modeling and Analysis

Once a complete set of input parameters was developed, 2-D SWAN wave transformation modeling was performed to simulate vessel wakes propagating from the ferry to the shoreline. The list of numerical modeling scenarios presented in Table 4.2 shows that the Tank Farm Pier was removed from the modeling grids for modeling of Elliot Point 1 and 2 Alternatives.

Table 4-2. Wake wash modeling scenarios

Scenario No.	Alternative
1	Existing Conditions
2	No-Build
3	Existing Site Improvements
4	Elliot Point 1 (No Tank Farm Pier)
5	Elliot Point 2 (No Tank Farm Pier)

Notes: Scenario 1 consists of existing conditions with Issaquah Class Ferry
Scenario 2 consists of existing conditions with New 144 Auto Class Ferry

For the modeling of existing conditions, No-Build, and Existing Site Improvements, the existing Tank Farm Pier pile field (see Figure 4-6) was coded into the modeling domain as a partially-permeable structure. The Tank Farm Pier pile field transmission coefficient of 0.65 was estimated and used in the modeling grid (for more detailed discussion on the Tank Farm Pier, see Section 2.5.3 and Section 5).

Figure 4-6. Existing Tank Farm Pier pile field seaward side (left) and landward side (right)

All modeling scenarios were modeled at a MHHW tide elevation of 11.04 ft above MLLW. The numerical modeling bathymetry domains were constructed using bathymetry data described in Section 4.3.1. Two numerical modeling grids that encompass all four alternatives were built and are shown in Figure 4-7.

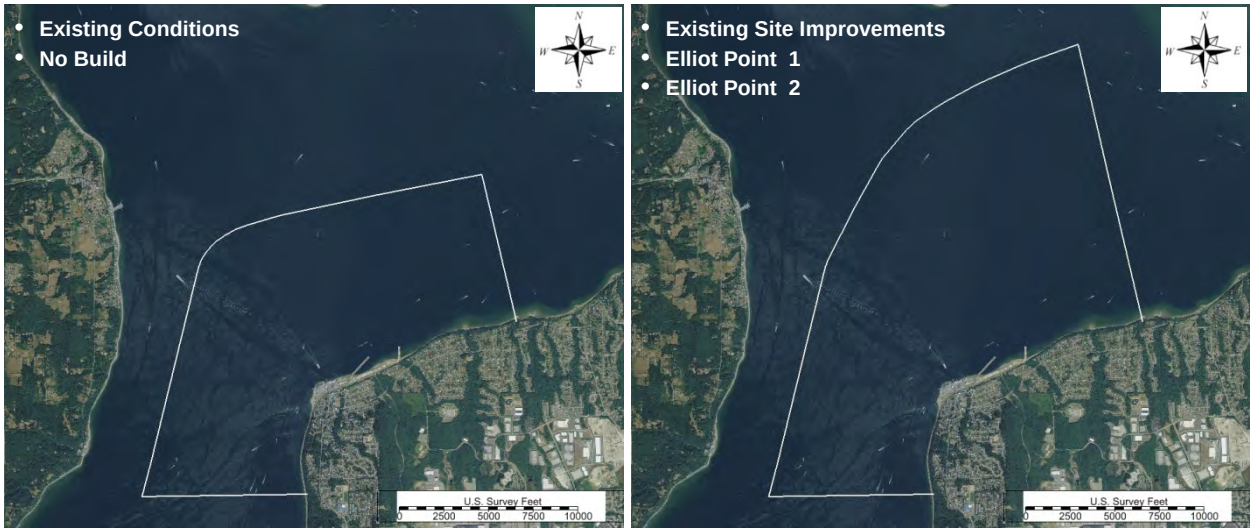
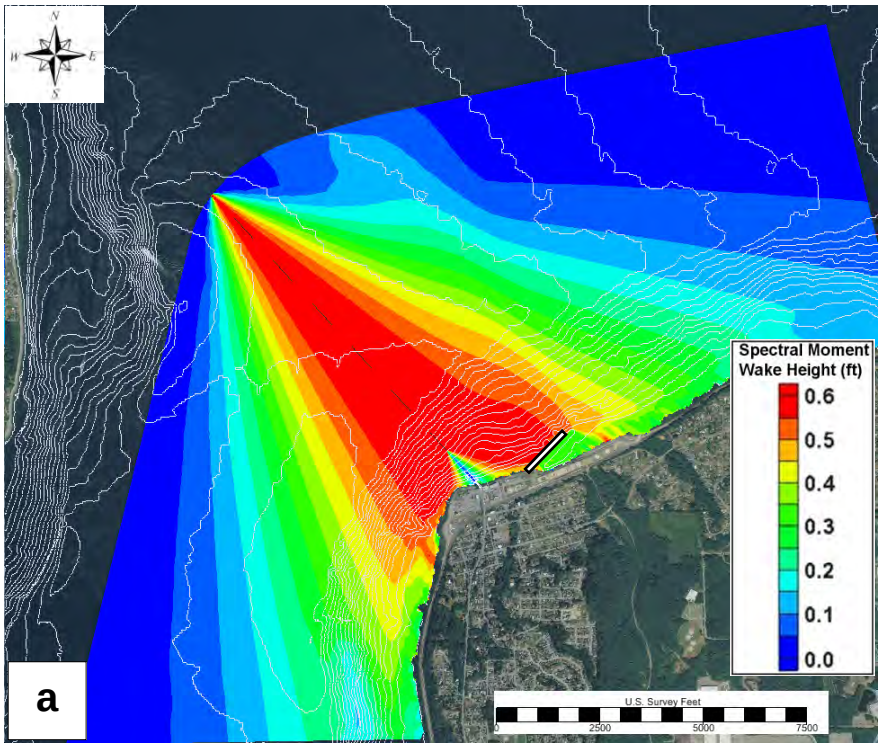


Figure 4-7. SWAN modeling grids

Once the modeling grids were constructed, SWAN numerical modeling was conducted with vessel wake input along the ferry routes and propagation of these input wakes over the modeling grids. Figure 4-8 shows the results of vessel wake modeling for all modeling scenarios in terms of spectral moment wake height.



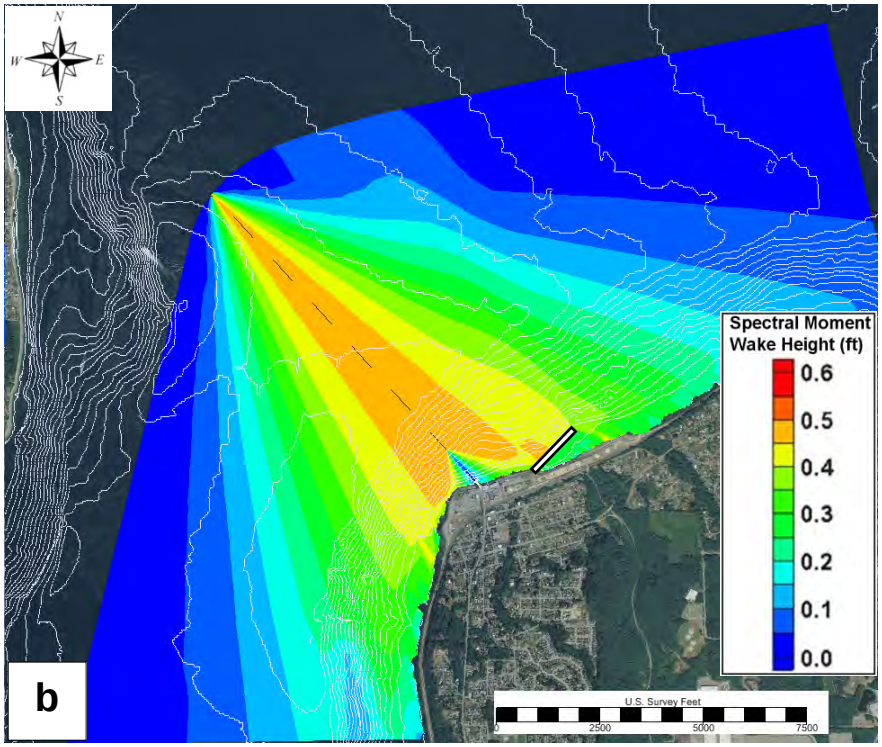
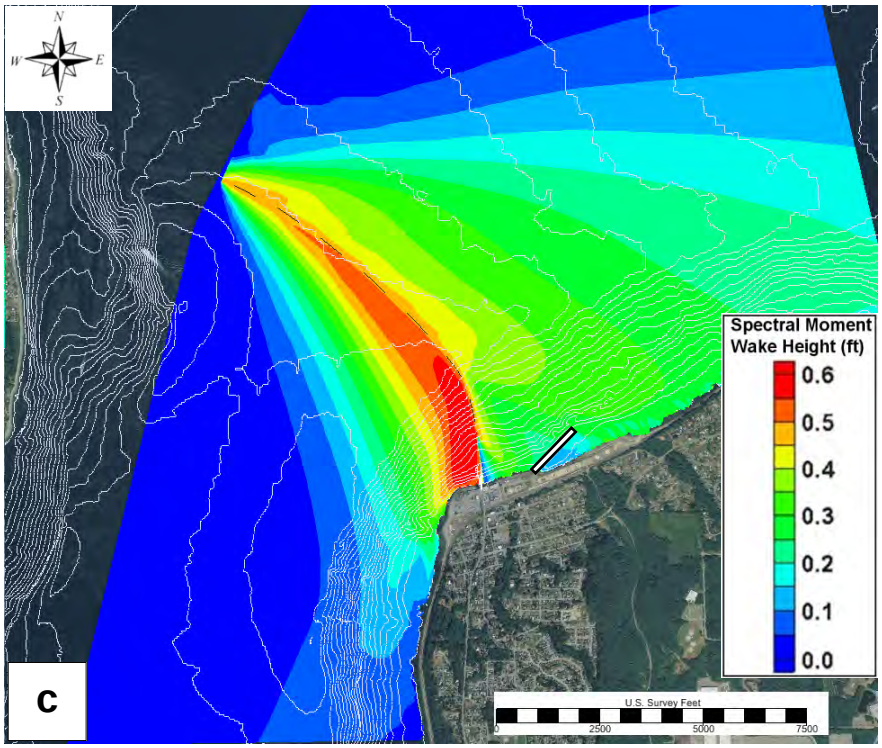


Figure 4-8. SWAN vessel wakes modeling results (spectral moment wake height) for modeling scenarios: a) existing conditions; b) No-Build



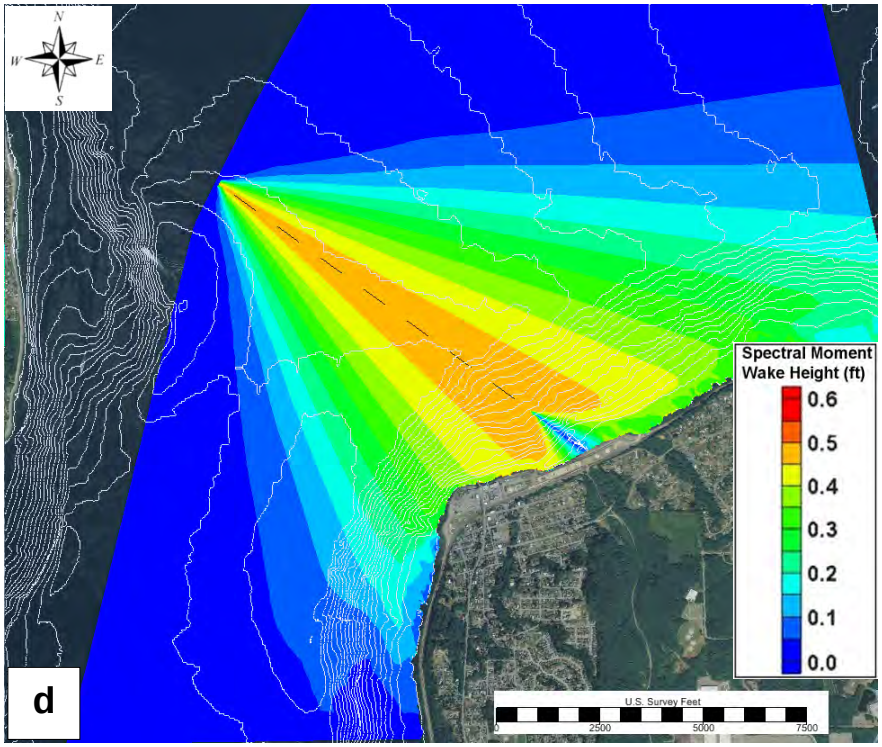


Figure 4-8. SWAN vessel wakes modeling results (spectral moment wake height) for modeling scenarios: c) Existing Site Improvements; d) Elliot Point 1

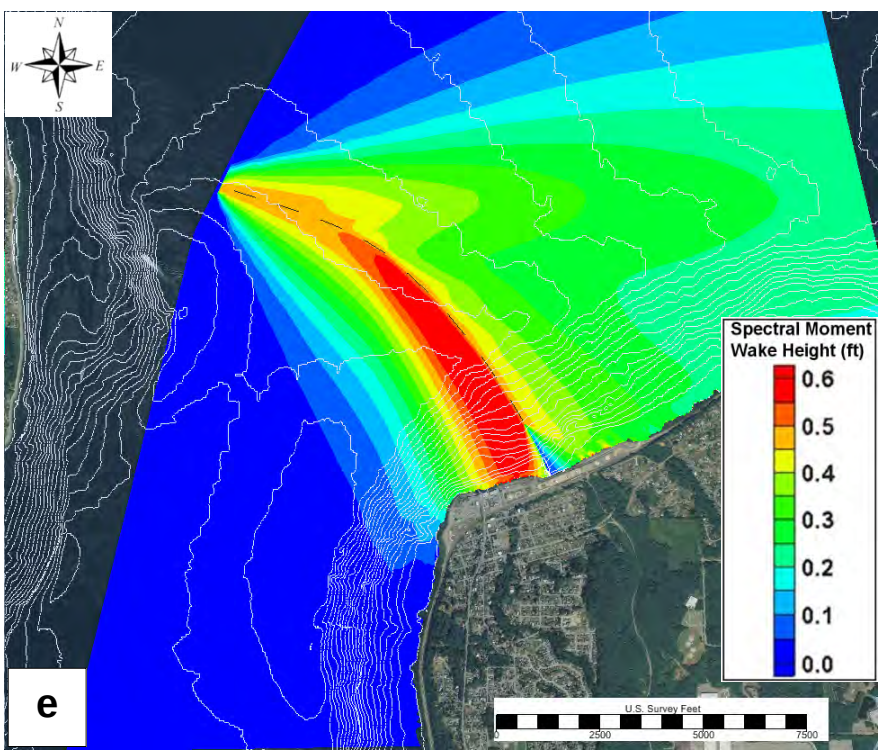


Figure 4-8. SWAN vessel wakes modeling results (spectral moment wake height) for modeling scenarios: e) Elliot Point 2

Figure 4-8 shows wave height distribution in the modeling grids in color format for each of the terminal alternatives. Red indicates large wake height, and blue small wake height or no wakes. The difference in wake patterns illustrated in existing conditions (Figure 4-8a) and No-Build (Figure 4-8b) is attributable to the difference in wakes generated by the existing Issaquah Class and the New 144 Auto Class Ferries, as described in Section 4.3. CFD modeling was the source of the initial wake information for propagation modeling. The wake difference is likely attributable to shape differences in the hulls of the two classes of ferries. A close-up view of modeling results near the terminal locations is shown in Figure 4-9.

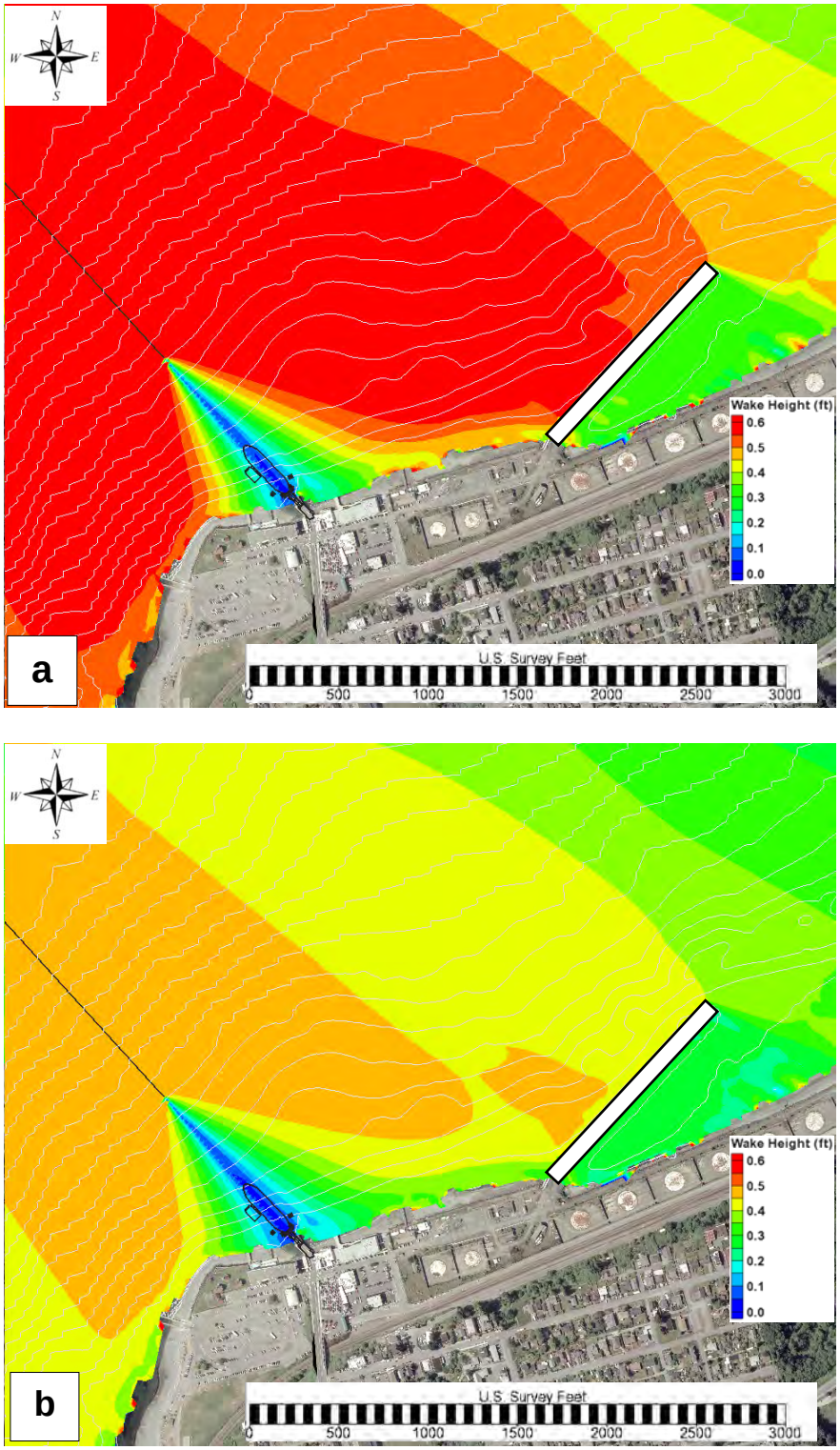


Figure 4-9. Close-up SWAN vessel wakes modeling results (spectral moment wake height) for modeling scenarios: a) existing conditions; b) No-Build

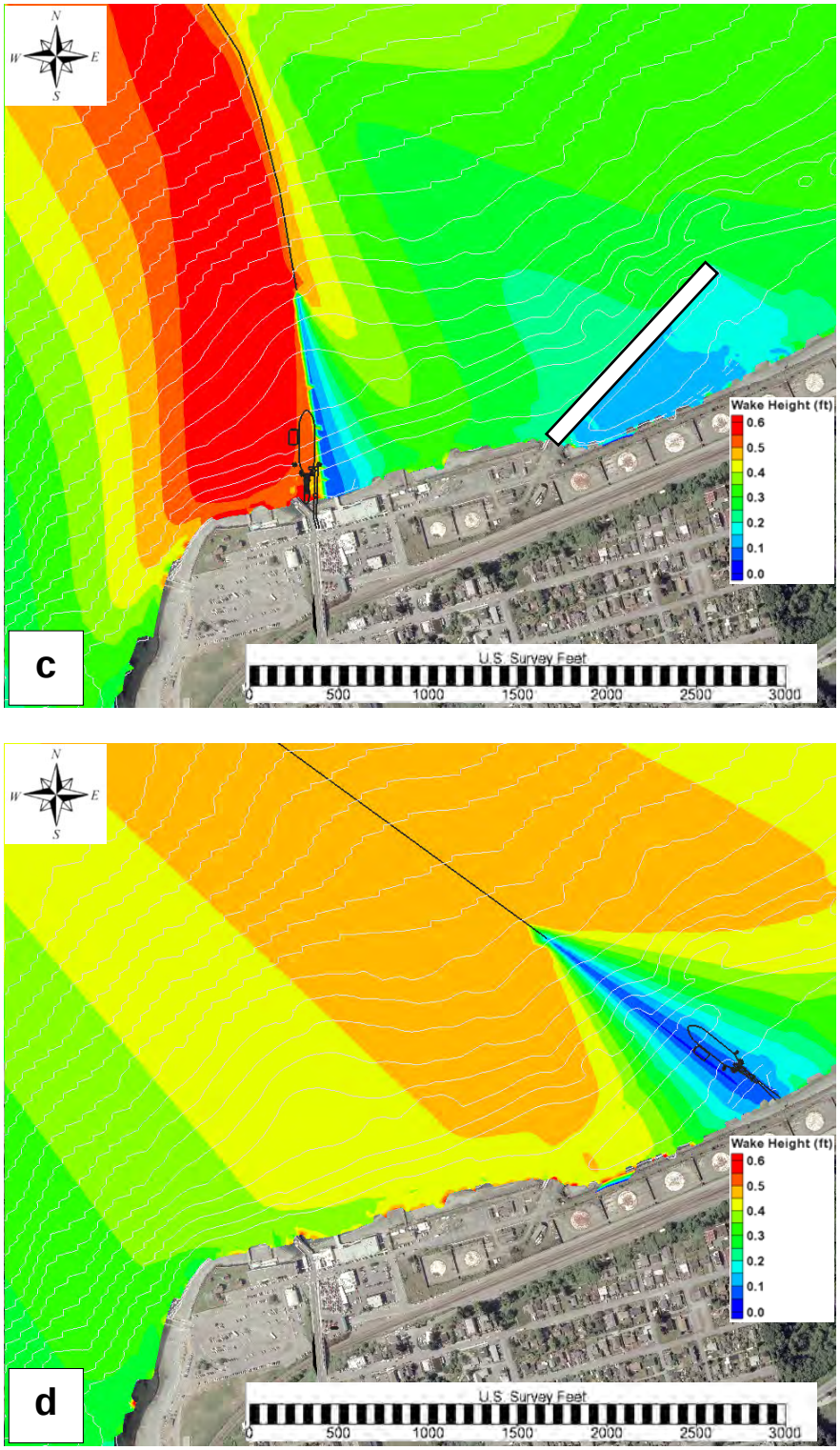


Figure 4-9. Close-up SWAN vessel wakes modeling results (spectral moment wake height) for modeling scenarios: c) Existing Site Improvements; d) Elliot Point 1

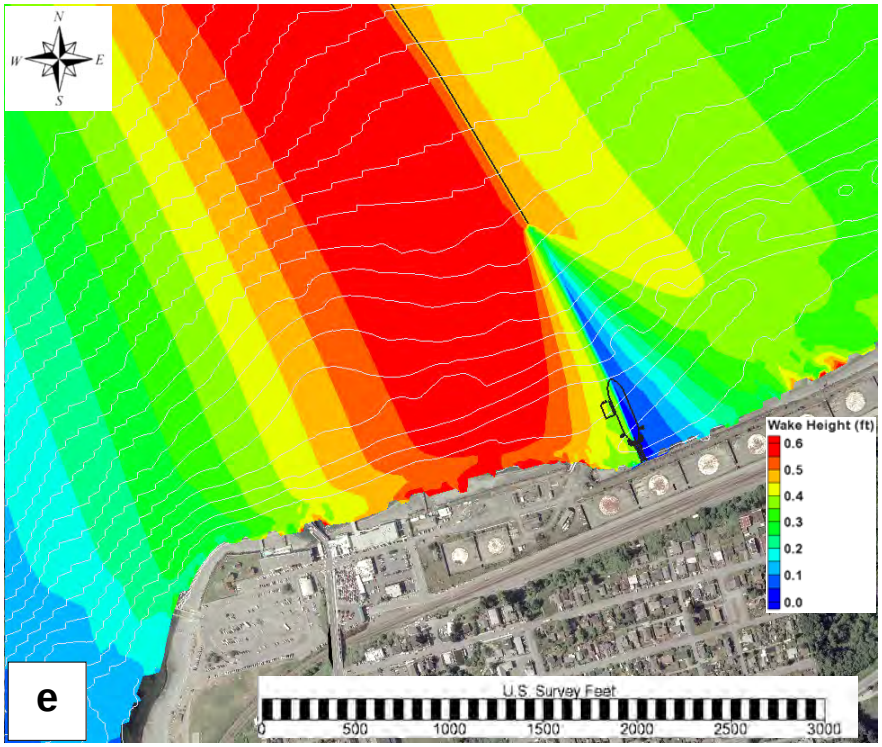


Figure 4-9. Close-up SWAN vessel wakes modeling results (spectral moment wake height) for modeling scenarios: e) Elliot Point 2

The modeling results show that different ferry terminal alternatives are associated with different patterns of wake propagation, resulting in different interaction with the Mukilteo shoreline.

In addition, the figures show areas of lower wake height in the vicinity of the existing and proposed ferry terminals. This is the blue triangle near the terminal in Figures 4.9a through 4.9e. It shows the model's predicted effect of slowing the ferry at the end of the route prior to approaching the terminal. The end of the routes for each of the alternatives is shown in Figure 4-5 above. In reality, the effect of wave reduction in the vicinity of the terminal due to the ferry slowing down would be smaller than that produced by the model. The reduction of wave heights in the field may not be even noticeable or measureable. Exacerbation of the wave reduction effect at the terminal is a limitation of the model, resulting in the artificial blue triangular shapes shown in the figures. In all the other areas of the modeling domain, the results of the modeling are reliable. The model's artificial effect is accounted for further in analysis of difference of wake heights between the proposed and existing conditions (see Section 4.4.1.1).

Two different methods of analyzing modeling results were employed to properly evaluate possible impacts: spatial and stationary. The spatial method includes comparison of ferry-generated wake height spatial distributions (plan view) among all alternatives. The stationary method includes a comparison of ferry-generated wake

heights along the 10-ft (MLLW) depth contour. Considering specifics of shoreline conditions discussed in Section 2.5, the analysis of vessel wakes impacts was conducted individually for each of three geomorphologic reaches specified at the Mukilteo shoreline (see Figure 2-8).

4.4.1.1. Spatial Method

Spatial analysis included comparison of existing conditions spatial distribution of wave heights over the modeling domain with that of each terminal alternative. For comparison, existing conditions wave heights were subtracted from wake heights for each proposed terminal alternative. Figure 4-10 shows the results of this comparison in color format. Red and yellow indicate an increase of wake heights, and blue indicates a reduction of wake heights[6].

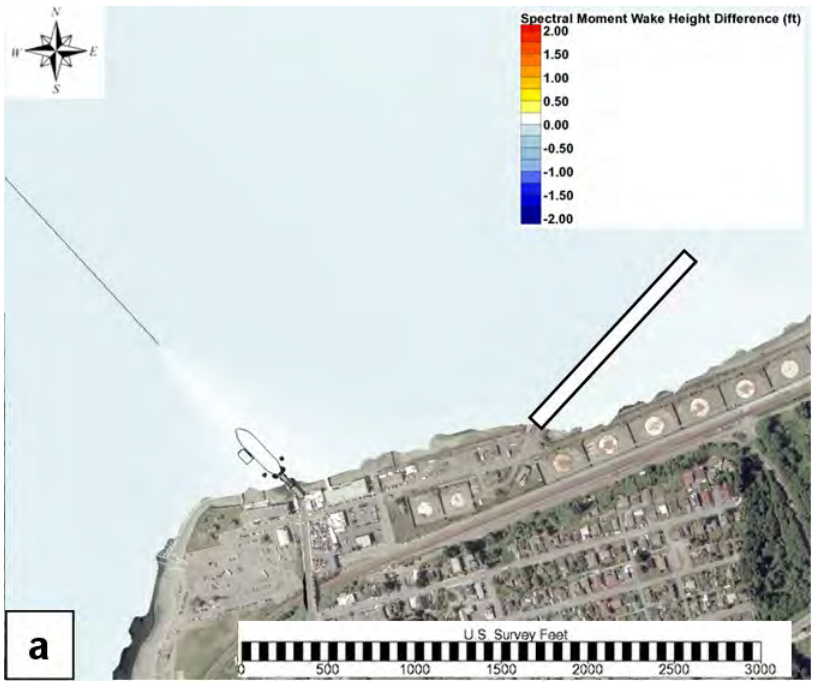


Figure 4-10. Change of spectral moment wake heights inthe nearshore area with respect to existing conditions for: a) No-Build

[6] As discussed above the model artificially exacerbates the effect of wave reduction in the immediate vicinity of the terminal (including existing terminal) due to a slowing ferry at the end of the route. When subtracting the plan view of wake height of the proposed alternatives from that of existing conditions, this results in an artificial increase of wake height in the vicinity of the existing terminal. Please note a similar triangular shape in yellow in Figures 4-10a – 4-10d and blue in Figures 4-9a – 4-9e. This artificial effect is discarded from further analysis.

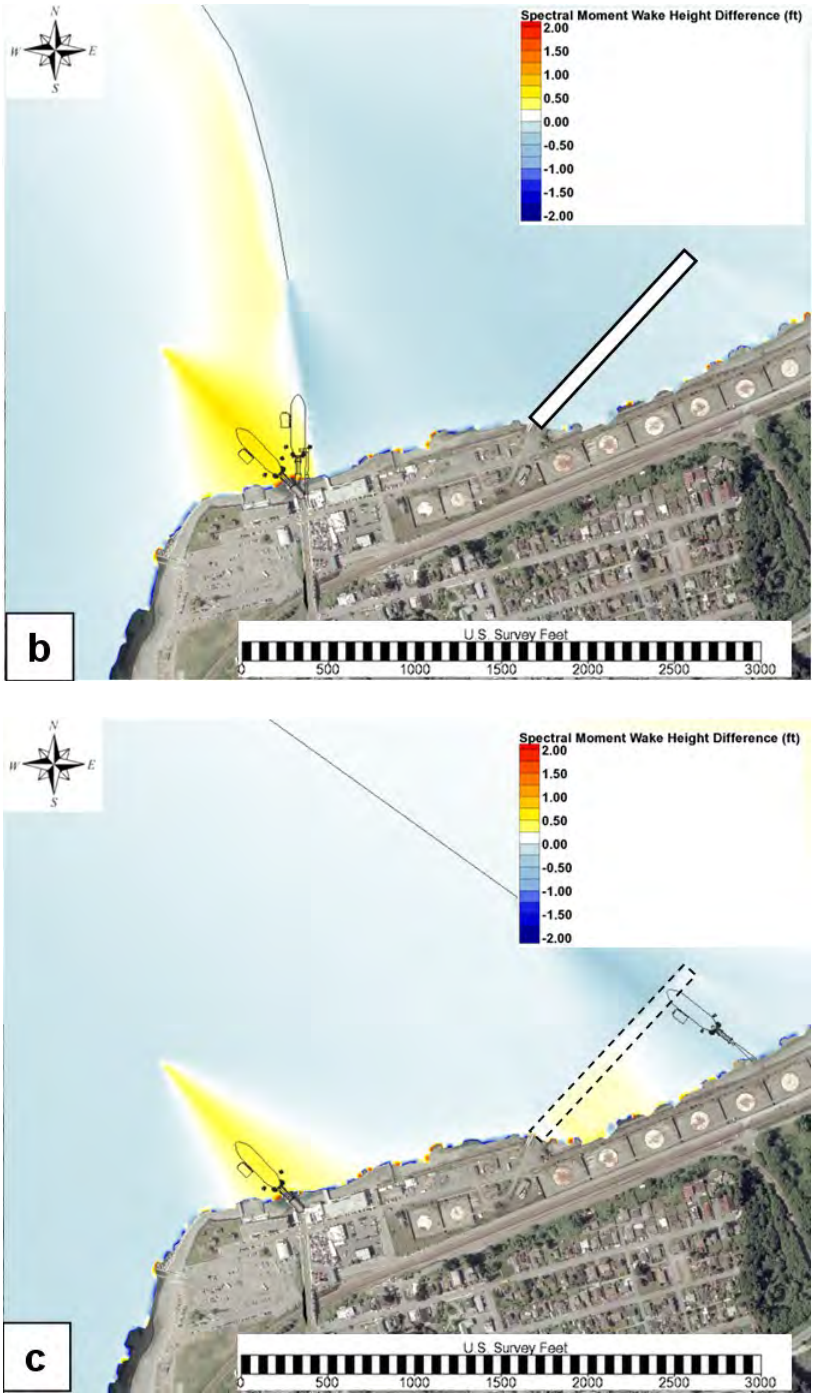


Figure 4-10. Change of spectral moment wake heights in the nearshore area with respect to existing conditions for: b) Existing Site Improvements; c) Elliot Point 1

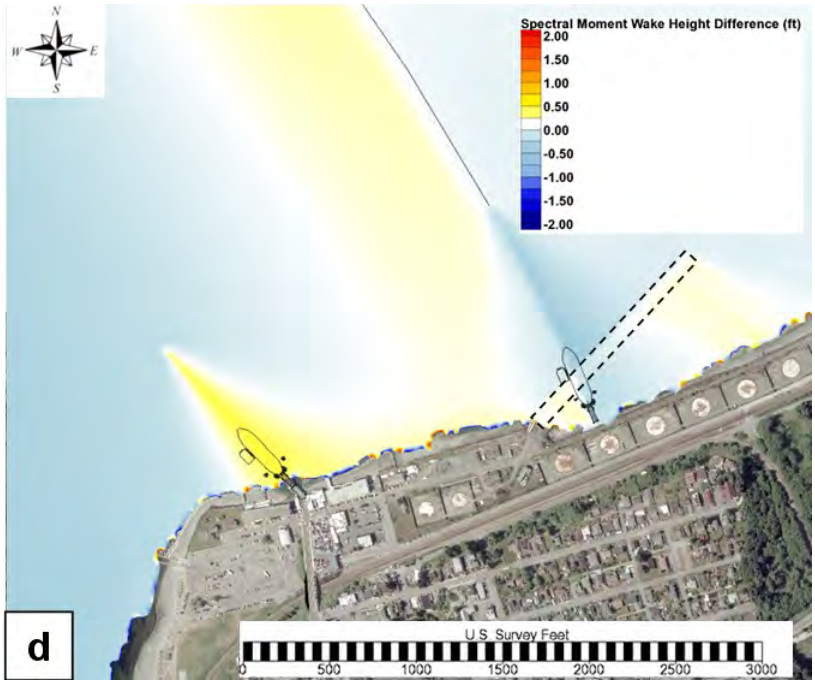


Figure 4-10. Change of spectral moment wake heights in the nearshore area with respect to existing conditions for: d) Elliot Point 2

Figure 4-10b shows that in Reach 1 the largest increase of wake wash energy (spectral moment wake height) with respect to existing conditions is due to the Existing Site Improvements Alternative. However, the same Existing Site Improvements Alternative would result in reduction of vessel wake energy for the rest of the shoreline, including Reach 2 and Reach 3. The Elliot Point 1 and 2 Alternatives are not expected to have any impact through an increase of vessel wake energy, excluding the area of shoreline that was originally sheltered by the pier. A local increase of wake energy in this area (originally sheltered area) would occur predominately due to removal of the Tank Farm Pier, which is included in the model.

4.4.1.2. Stationary Method Analysis

Figure 4-11 shows the location of the -10 ft MLLW contour line used for extraction of wakes for stationary analysis.

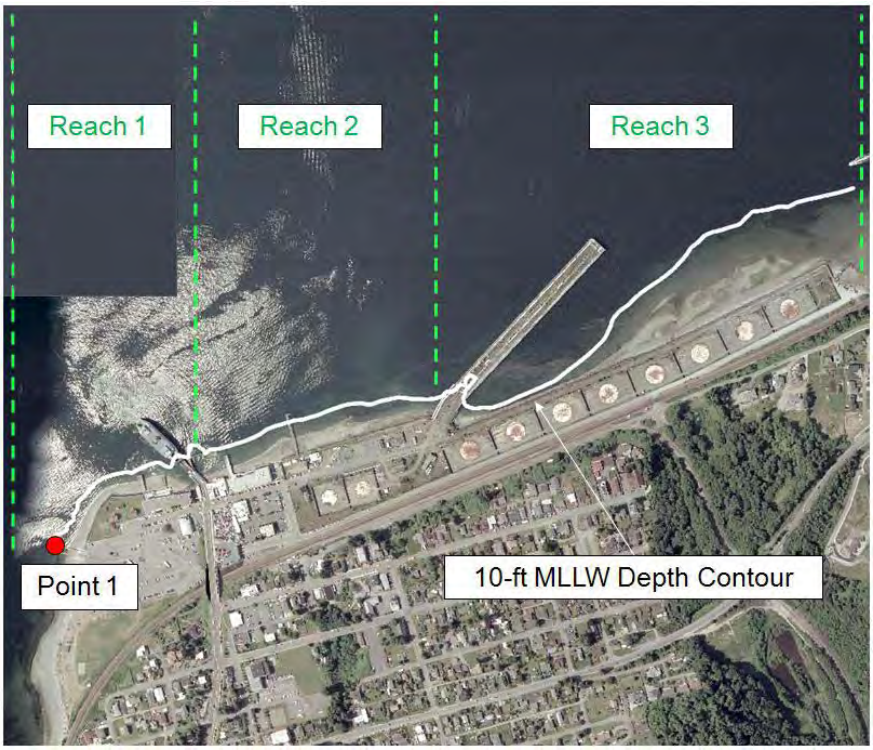


Figure 4-11. Location of -10 ft MLLW contour (white line) used for extraction of wakes for stationary analysis

Figure 4-12 shows wake heights extracted along the 10-ft MLLW depth contour for all alternatives. The x-coordinate is the distance from Point 1 (see Figure 4-11) measured in an easterly direction along the 10-ft MLLW depth contour.