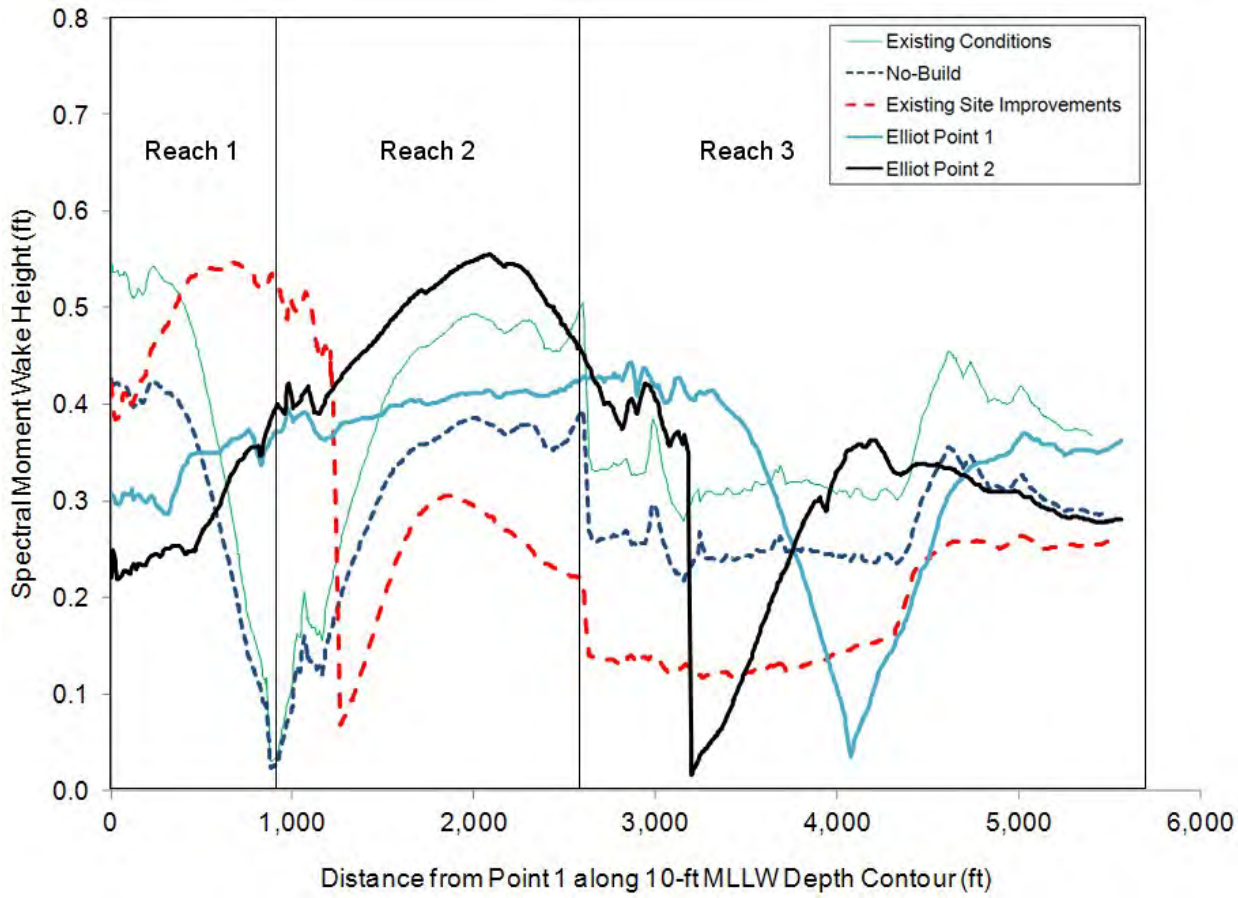


Figure 4-12. Spectral moment wake height along 10-ft-depth (below MLLW) for all alternatives

Figure 4-12 shows that in Reach 1 the largest amount of wake wash energy (spectral moment wake height) is from the existing conditions and Existing Site Improvements Alternative. In Reach 2, the largest amount of wake wash energy is from the existing conditions and Elliot Point 2 Alternative. In Reach 3, the largest amount of wake wash energy is from the existing conditions and Elliot Point 1 and 2 Alternatives.

In summary, the two methods of analysis showed that the amount of wake wash energy (spectral moment wake height) transferred to the shoreline is directly correlated to the location of the ferry terminal, to the presence of the Tank Farm Pier and the sheltering effect of its pile field, to the shape of the ferry route and to the characteristics of the near-field wakes generated by the ferries.

Within Reach 1 the largest amount of wake wash energy would occur with the existing conditions and Existing Site Improvements Alternative. For these alternatives, the terminal location is at the eastern edge of Reach 1. The No-Build Alternative and existing conditions share the same route, but the near-field spectral moment wake height is smaller for the No-Build Alternative. Therefore, the wake wash energy generated and transferred to the shoreline would be smaller for the No-Build Alternative. The Existing Site Improvements Alternative has the same

spectral moment wake height as the No-Build Alternative, but the ferry route is concave, and wake wash energy is focused towards the concave part of the route.

Within Reach 2 the largest amount of wake wash energy is with the existing conditions and Elliot Point 2 Alternative. The reasons are the larger near-field spectral moment wake height generated for the existing conditions and the concave route of the Elliot Point 2 Alternative which tends to increase wake energy at the Reach 2 shoreline more than other alternatives.

For Reach 3 a high level of wake wash energy will occur under existing conditions and both Elliot Point Alternatives. The reason is wakes from the Elliot Point Alternatives propagate directly to the Reach 3 shoreline, and other alternatives' routes are more distant. The Tank Farm pile field more effectively blocks wake energy of the other alternatives than it does for the Elliot Point Alternatives.

4.4.2. Potential Swash Sediment Transport Analysis - Methodology

Potential swash sediment transport parameters due to ferry wakes were computed at the locations of nine controlling stations (points) located in Figure 4-13. Wake heights and periods were extracted at these stations and used as input parameters for computation of swash sediment transport according to a procedure of Puleo *et al*. (2003).

Figure 4-13. Location of controlling stations (points) used for analyzing potential swash sediment transport

Calculated rates of potential swash sediment transport are plotted in Figure 4-14 for points located in Figure 4-13.

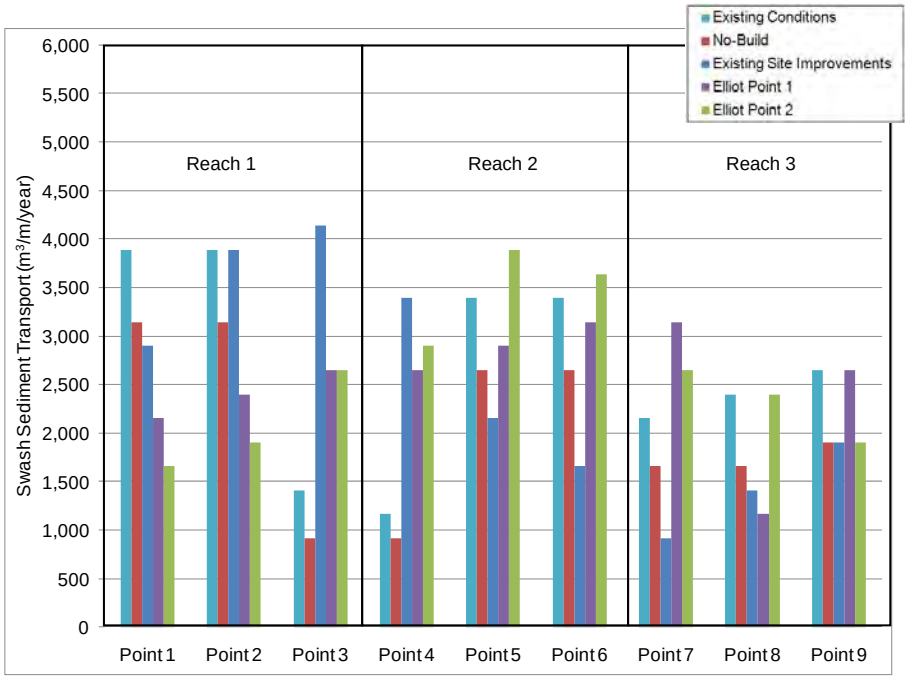


Figure 4-14. Potential swash sediment transport calculated at points 1-9 for all alternatives

Figure 4-14 shows that in Reach 1 the highest potential swash sediment transport rate corresponds to existing conditions and the Existing Site Improvements Alternative. In Reach 2, the highest potential swash sediment transport rate corresponds to existing conditions and Elliot Point 2. In Reach 3, the highest potential swash sediment transport rate corresponds to existing conditions and Elliot Point 1 and 2. Localized rates of potential swash sediment transport by extracting values at single points, and the variability within the same reach is explained by the specific location of the extraction points with respect to the location of the ferry terminal.

4.5. Section Summary

Ferries of both the Issaquah and 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 2 ft to 3 ft. (see Section 2.3).

The New 144 Auto Class Ferry would generate smaller wake wash than the Issaquah Class Ferry. Therefore, a smaller amount of wake energy 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 swash sediment transport at the shoreline is directly correlated to the location of the ferry terminal, to the presence of the Tank Farm Pier and the sheltering effect of its pile field, to the shape of the ferry route and to the characteristics of the near-field (initial) wakes generated by the ferries. The largest amount of wake wash energy and highest potential swash sediment transport rate in Reach 1 corresponds to the existing conditions and Existing Site Improvements. The largest amount of wake wash energy in Reach 2 corresponds to existing conditions and Elliot Point 2. High wake energy for existing conditions is caused by the large near-field spectral moment wake height and proximity to Reach 2, and for Elliot Point 2 is due to the concave route towards shore and proximity to Reach 2. The largest amount of wake wash energy in Reach 3 corresponds to existing conditions and Elliot Point 1 and 2. High wake energy for existing conditions is caused by the large near-field spectral moment wake height, and for Elliot Point 1 and 2 is due to the proximity to Reach 3 and lack of the existing protection provided by the Tank Farm Pier pile field.

5. Tank Farm Pier Removal Impact on Longshore Sediment Transport

5.1. Methodology

The objective of the longshore sediment transport analysis and modeling was 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. Wind-waves are the dominant mechanism to transport sediment alongshore. Therefore, ferry routes and terminal locations are not considered in this analysis. With no new construction but with removal of the Tank Farm Pier assumed for the shoreline reach, this analysis represents a conservative simulation of transport. Constructing a terminal in this reach would result in transport conditions less different from existing conditions. The analysis included development of input parameters, numerical modeling of wind-wave propagation to the shoreline, and computations of potential longshore sediment transport components along the Mukilteo shoreline. Wave propagation was modeled with the 2-D spectral wave numerical model SWAN (Holthuijsen *et al.*, 2004). Potential longshore sediment transport was computed using the CERC formula developed at the Coastal Engineering Research Center (USACE, 2002). The potential longshore sediment transport (PLST) characterizes a part of wave energy that potentially would be transformed in sediment transport along the shoreline if this sediment were available. Therefore, the PLST is a relative parameter that describes possible changes in longshore sediment transport. In other words, PLST is a parameter through which a change in wave conditions can be related to possible changes in sediment transport and related erosion-deposition processes (for example, nearshore wave changes due to removal of Tank Farm Pier).

Modeling and analysis were conducted for two shoreline configurations: Existing Conditions (with Tank Farm Pier) and Post-Project Conditions (with removal of Tank Farm Pier). Potential longshore sediment transport analysis results for Existing Conditions and Post-Project Conditions were used in a comparative study and not in an absolute quantitative analysis.

Input parameters for analysis and numerical modeling of wind-waves and potential longshore sediment transport included bathymetry data and representative wind speed and direction. Development of input parameters for the analysis and modeling are discussed in Section 5.2.

5.2. Input Parameters

5.2.1. Bathymetry Data

As discussed in Section 3.2.4 and Section 4.3.1, bathymetric survey data from various sources were compiled and organized to develop a sufficient data set for numerical modeling of wind-waves and analysis of potential longshore sediment transport. Figure 5-1 shows the compiled bathymetry set (built using four different bathymetry sources) used for modeling and analysis of longshore sediment transport, including the wind-wave large modeling grid.

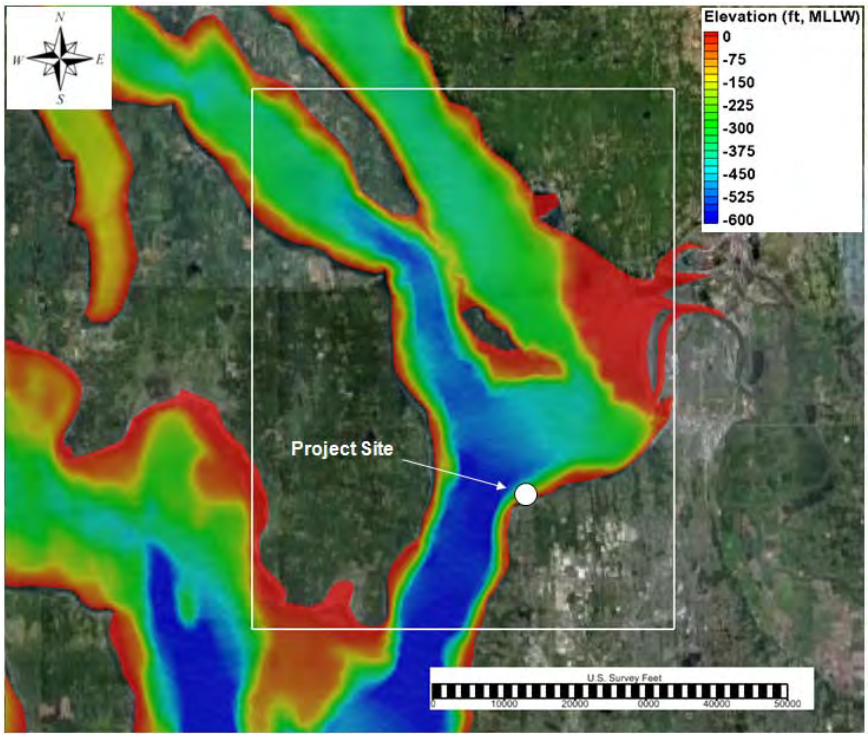


Figure 5-1. Bathymetry set used for modeling; white box shows boundaries of large wind-wave modeling domain

5.2.2. Wind Data

Wind data were compiled and statistically processed to develop wind data input for wind-wave numerical modeling. The detailed descriptions of wind data and results of wind analysis are shown in Section 2.3. The 25-year return period wind storm events from three different directions were selected as the design storms for potential

longshore sediment transport analysis[7]. The relevant wind directions of the 25-year return period wind storm events are defined as: W (260° N), NW (310° N), and NE (30° N), as shown in Figure 5-2.

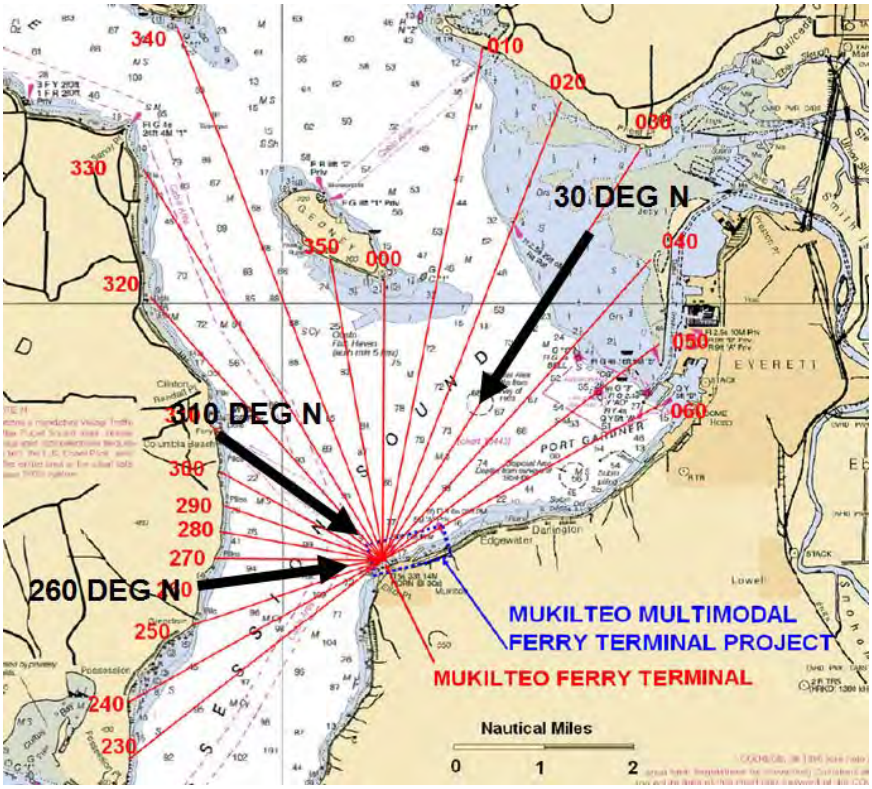


Figure 5-2. Wind-wave fetches at Mukilteo project site and wind directions chosen for SWAN wind-wave modeling

Table 5-1 shows the design wind parameters used for wind-wave modeling.

Table 5-1. Design wind for wave modeling

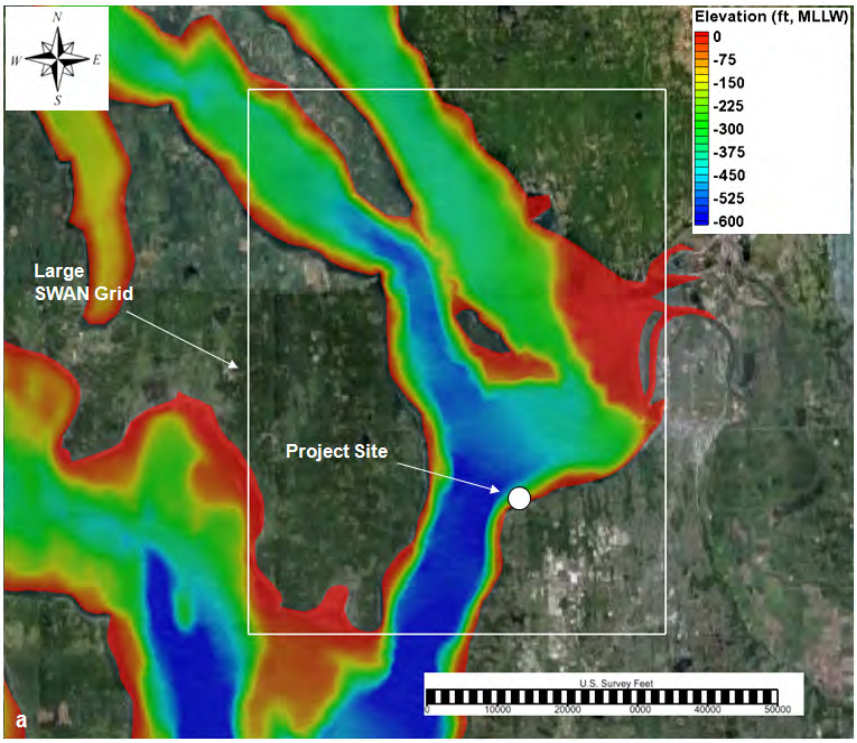
Return Period (yr)	Wind Direction (Deg N)		
	260	310	30
25	28.0 kts	36.5 kts	16.1 kts

All three modeling scenarios shown in Table 5-1 were run with a tide elevation of 6.5 ft above MLLW, which correspond to the local mean sea level. Modeling scenarios were run for Existing Conditions and Post-Project Conditions (with removal of Tank Farm Pier).

[7] Lower return period (less energetic) storm events would not be correct indicators for Tank Farm Pier impact on sediment transport.

5.3. Wind-Wave Modeling Results

SWAN wind-wave generation and propagation modeling was performed with a large-scale modeling grid to generate input design wave conditions in the vicinity of the project site, and then with a small-scale (nested) modeling grid with higher resolution to propagate the waves at the project site (the outlined area in Figure 5-3b). The numerical modeling bathymetry domains were constructed using bathymetry data discussed in Section 5.2.1. The SWAN modeling grids are shown in Figure 5-3.



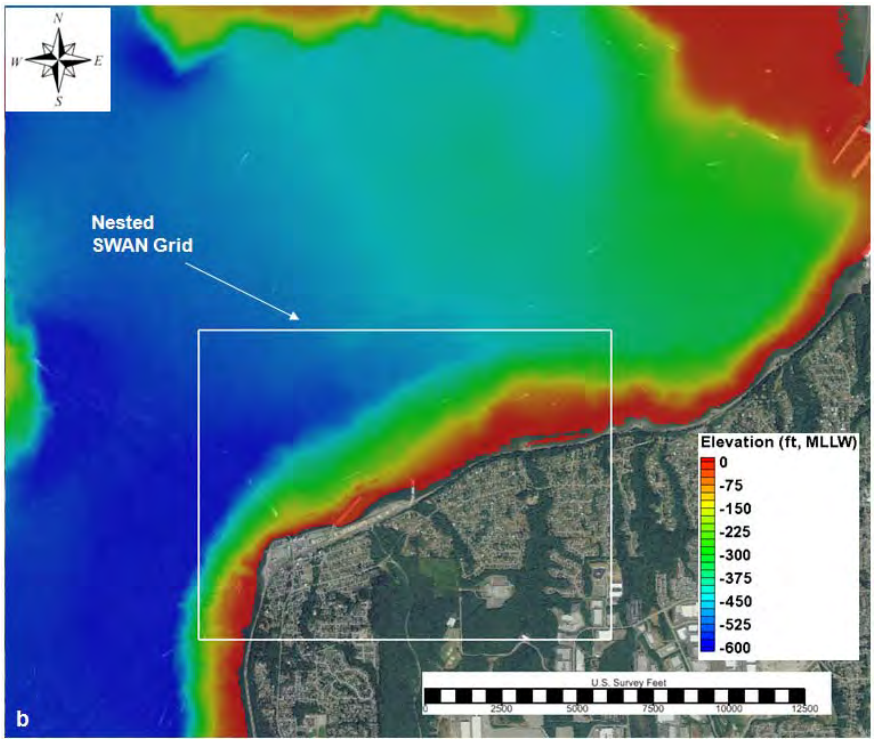


Figure 5-3. SWAN modeling domains; a) large model, b) nested model

The existing Tank Farm Pier pile field was coded into the modeling domain as a partially-permeable structure. The Tank Farm Pier piles are relatively dense, with spacing between piles approximately 3 to 4 ft. The diameter of the majority of the piles is approximately 1 ft, with some pile diameters as large as 2 ft. Figure 5-4 shows close-up views of the pile field. The Tank Farm Pier pile field transmission coefficient of 0.65 was estimated and used in the modeling grid for Existing Conditions. For Post-Project Conditions, the same modeling grid was used, but the Tank Farm Pier pile field transmission coefficient was set equal to 1.0 (to simulate removal of the pier).

Figure 5-4. Existing Tank Farm Pier pile field seaward side (left) and landward side (right)

Modeling was conducted for 25-year return period storm events from the three different directions shown in Figure 5-2 and for two project conditions: Existing Conditions, with the Tank Farm Pier; and Post-Project Conditions, without the Tank Farm Pier. An example of wave modeling results with the design storm from 310° N is shown in Figure 5-5a for Existing Conditions and 5-5b for Post-Project Conditions.

Figure 5-5 shows significant wave height distribution over the modeling domain in color format. Red-yellow indicates high waves; green-blue indicates low heights or no waves. The figure shows that the removal of the Tank Farm Pier would result in some changes of wave height over part of the modeling domain, specifically along the shoreline. In order to evaluate patterns of these differences, wave height results of Existing Conditions (with Tank Farm Pier) were subtracted from wave height results of Post-Project Conditions (without Tank Farm Pier) in the whole modeling grid. Figure 5-6 shows the results of wave height differences for all modeling scenarios in color format (a, wind direction 260° N; b, wind direction 310° N; c, wind direction 30° N). Red-yellow indicates an increase in wave height. Blue color indicates a reduction in wave height.

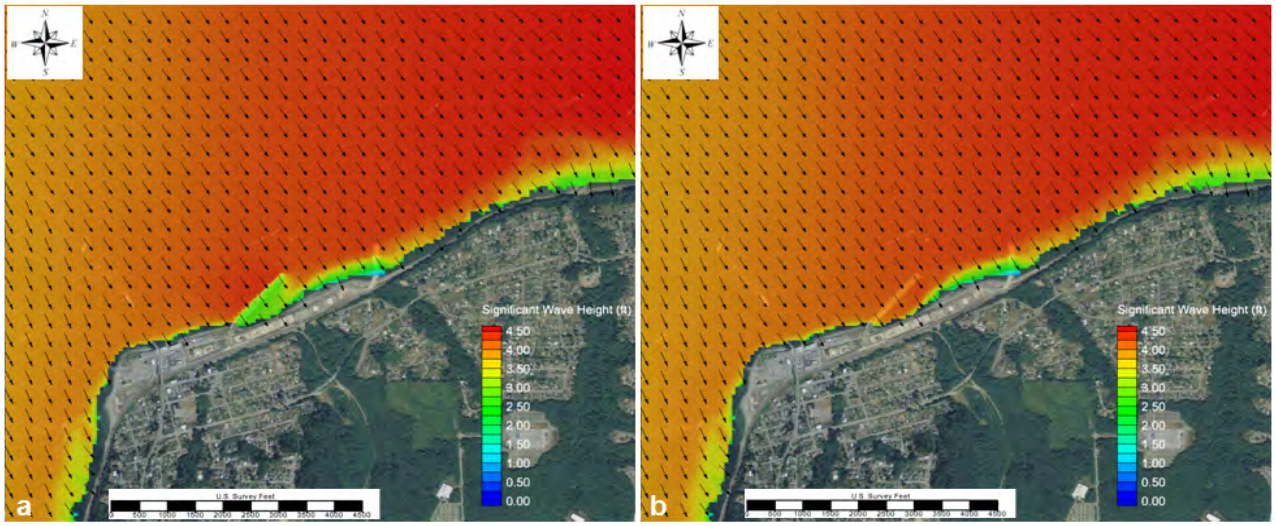


Figure 5-5. Modeled wave height pattern for 25-year return period storm from 310° N, a) with Tank Farm Pier, b) without Tank Farm Pier

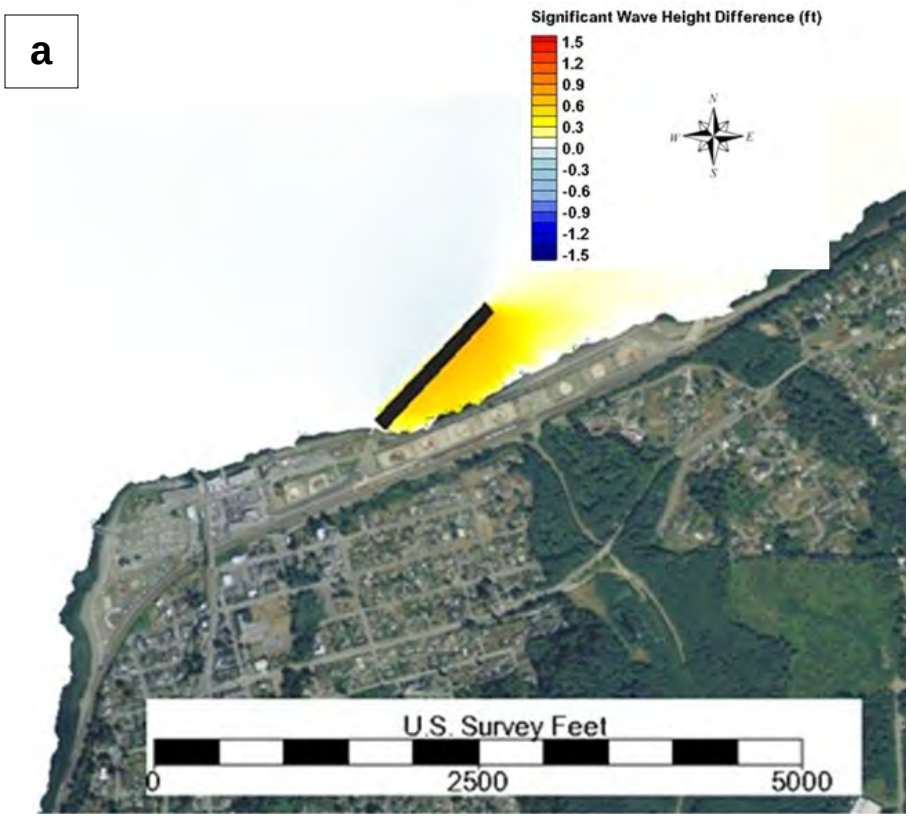


Figure 5-6. Wave height difference (Post-Project Conditions minus Existing Conditions) for: a) west storm

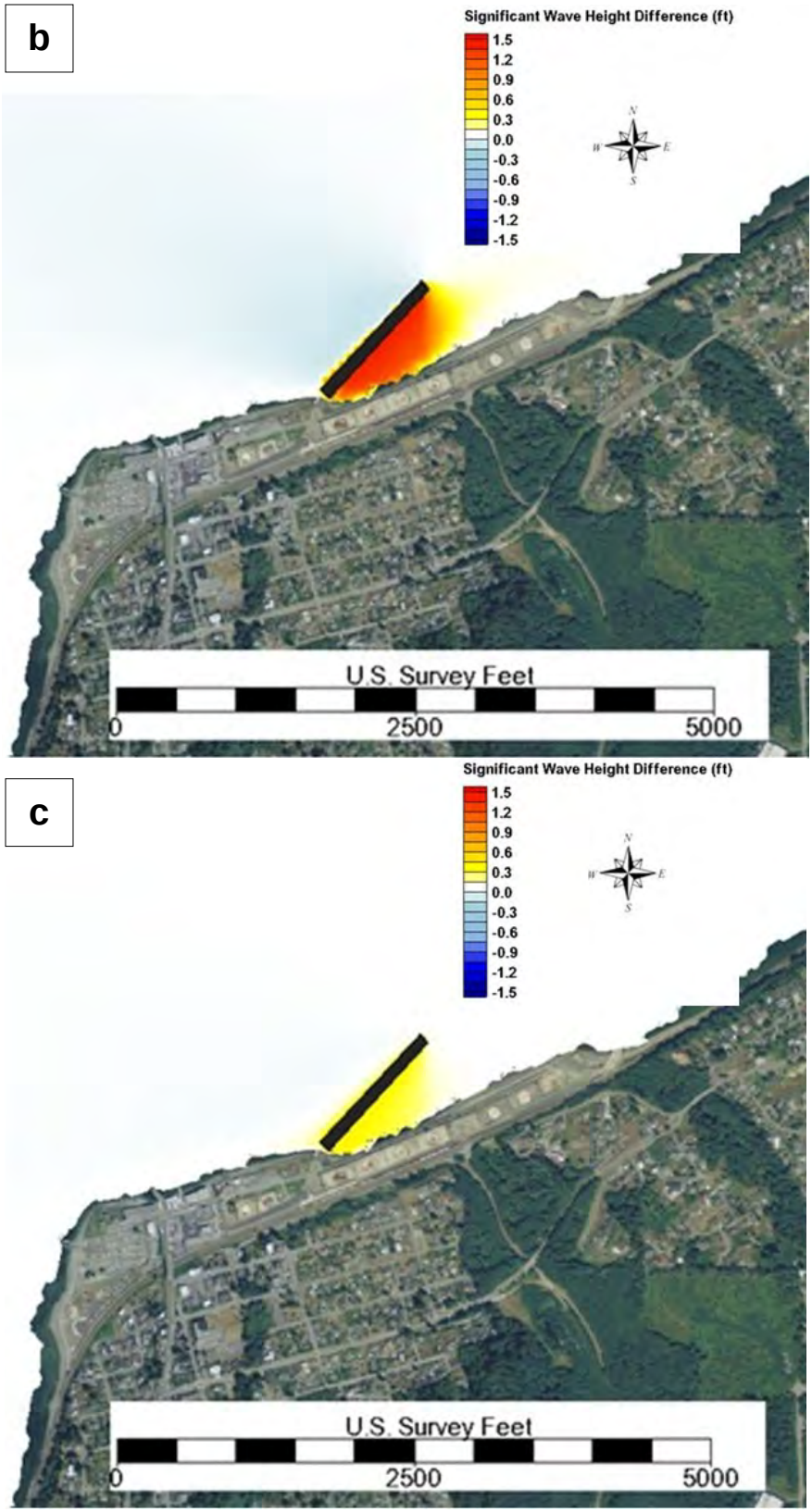


Figure 5-6. Wave height difference (Post-Project Conditions minus Existing Conditions) for: b) northwest storm and c) northeast storm

The analysis shows that for storms coming from the west (260° N) and northwest (310° N), removal of the Tank Farm Pier would cause a measurable change in wave conditions at the shoreline to the east of the pier. For storms coming from the northeast (30° N), impact of the removal of the pier would be less noticeable. The measurable direct impact of Tank Farm Pier removal on change of wave patterns from west and northwest storms would extend eastward to a distance up to 1,800 ft, roughly the eastern end of the Tank Farm. These observed changes in wave conditions would result in changes of potential longshore sediment transport. The next section provides estimates of Tank Farm Pier removal impact on potential longshore sediment transport.

5.4. Relative Effects on Potential Longshore Sediment Transport

Wind-wave modeling results (wave height and wave period) were extracted at seventeen output stations placed in the numerical modeling grid, eastward of the Tank Farm Pier. Station locations were selected with the understanding that the impact of the removal of the pier on potential longshore sediment transport would be measurable only eastward of the pier. Therefore, the stations were located to the east of the pier, starting 100 ft from the base of the pier and spaced approximately 100 ft apart. Figure 5-7 shows the location of the selected stations for sediment transport analysis.

Figure 5-7. Location of controlling stations used for analysis of potential longshore sediment transport

Extracted wave parameters at the controlling stations were used to calculate potential longshore sediment transport at each station location with the CERC method (USACE, 2002). Potential longshore sediment transport was calculated for wind storms coming from the west (260° N) and northwest (310° N)[8]. The calculated relative increase (percent) in potential longshore sediment transport due to the removal of the Tank Farm Pier for the two selected storms is shown in Figure 5-8.

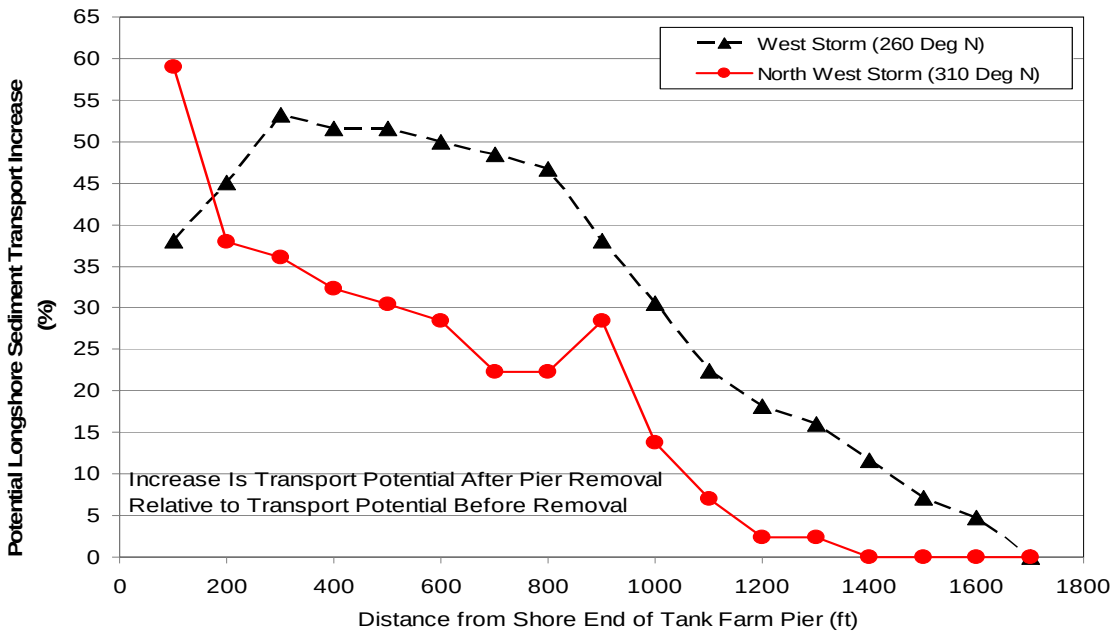


Figure 5-8. Relative increase in potential longshore sediment transport for 25-year return period storms due to removal of Tank Farm Pier

The analysis shows that removal of the Tank Farm Pier may cause a measurable increase of potential longshore sediment transport along the shoreline due to northwest and west storms. An increase of potential longshore sediment transport means that more wave energy would be available for eastward longshore sediment transport. In other words, if sufficient sediment were available, more sediment would be transported from west to east at this part of the shoreline. If the sediment source and supply rate are the same after Tank Farm Pier removal as before, erosion should be expected at areas of increased sediment transport potential. Under those conditions, sediment that originally was stable at the west would be mobilized and transported by waves to the east. Assuming that a change in longshore sediment of less than 10 percent of original value is not detectable, the extent of pier removal impact on longshore sediment transport would be limited to approximately 1,400 ft.

[8] Wave characteristics and induced longshore transport due to wind storms from the northeast direction (30° N) do not depend on the Tank Farm Pier and are not analyzed here.

5.5. Section Summary

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 were identified through changes of wind-wave patterns and potential longshore sediment transport components. Potential longshore sediment transport characterizes a part of wave energy that would be transformed in transporting sediment along the shoreline if sediment were available.

The analysis and modeling showed that removal of the Tank Farm Pier would cause a measurable change of wave conditions at the shoreline to the east of the pier from storms from the west (260° N) and northwest (310° N) directions. The impact of Tank Farm Pier removal on change of wave patterns would extend eastward a distance of approximately 1,800 ft from the shore end of the pier.

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

6. Tank Farm Pier Removal Impact on Stability of Under-Pier Sediments

6.1. Objective and Methodology

The objective of this analysis was to determine existing conditions at the sediment mound under the Tank Farm Pier, estimate the historical and future accumulation rates in the mound, and identify the potential for sediment mobilization from the existing sediment deposits under the pier, after the pier is removed. Conservative assumptions were made for this analysis. No new terminal construction was assumed after removing the Tank Farm Pier.

Input parameters for the analysis included: available bathymetric survey data, wave information, and physical characteristics of bottom sediment. Bottom bathymetry and morphology were analyzed using limited existing data and relevant information developed and presented in previous sections of this report. In addition, a significant number of assumptions were developed and used in the analysis to compensate for the limitations and uncertainties of existing data. Most of the assumptions are based on engineering intuition and previous practical experience. While appropriate for the current conceptual level of study, these assumptions need to be validated or replaced with actual data (once data become available), and results of the analysis should be adjusted accordingly.

6.2. Input Parameters

6.2.1. Bathymetry Data

Two sources of bathymetric survey data were used to determine bottom depth in the Tank Farm Pier area. Lead line soundings conducted by WSDOT in 2007 provided

some bathymetry under the pier. Additional bathymetry in the areas around the pier was collected via depth sounder by WSDOT in 2011. These data are more detailed in the pier footprint than were available for model bathymetry shown in previous sections. Data from both sources were combined to develop the bathymetry map at the project area, as shown in Figure 6-1. The figure shows bottom depth in color format: red represents shallower water depths and blue represents deeper water.

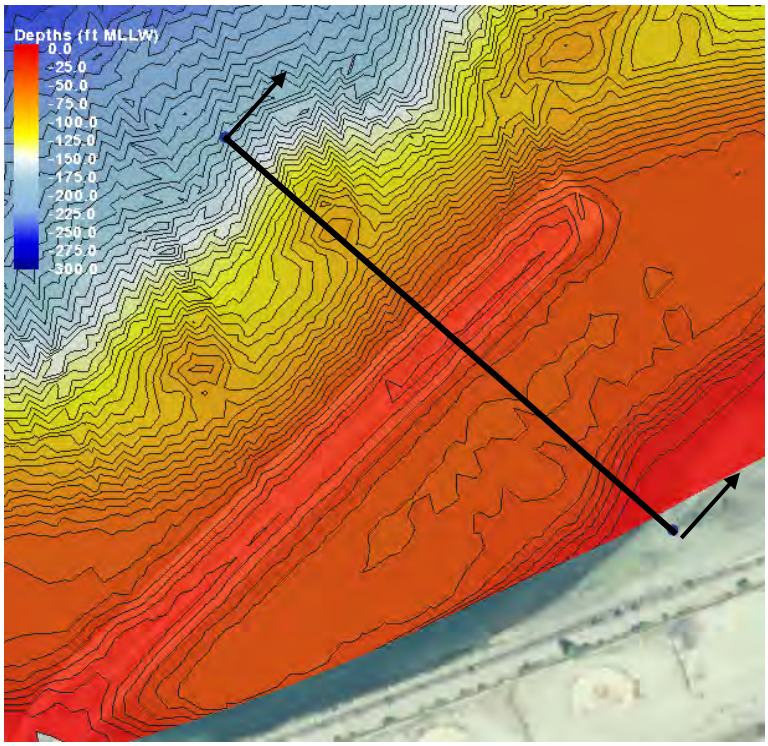


Figure 6-1. Bathymetry in vicinity of Mukilteo Tank Farm Pier and cross-section location

The contours and colors in Figure 6-1 show the accumulative formation (mound) under the full length of the Tank Farm Pier. Crest elevation of this mound varies slightly along the pier, with the maximum relief nearly 20 ft above the surrounding bathymetry.

The figure also shows some irregularities in the bottom slope on the seaward and landward sides of the pier. Figure 6-2 shows the depth configuration at a cross-section taken perpendicular to the pier (location indicated in Figure 6-1) and depicts some of these irregularities. The depression in the bottom on the landward side of the pier is a relict of the historically dredged area. The nature of the bottom hump on the seaward part of the pier is not well understood and may represent a relict bottom formation, submerged debris, dredged material disposal site, or the result of interpolation between limited data points.

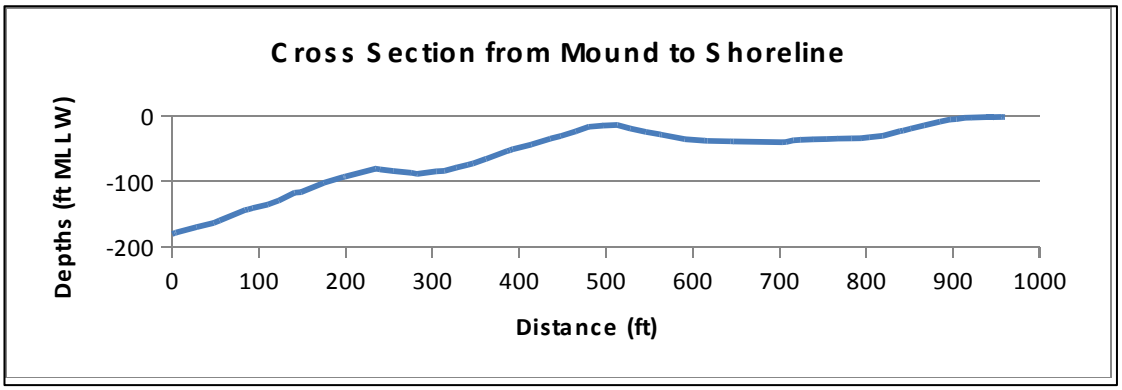


Figure 6-2. Bathymetric relief in vicinity of Tank Farm Pier (pier is centered at approximately 500 ft)

6.2.2. Wind-Wave Information

Detailed information on wind-wave conditions including wind statistics, wave generation, propagation modeling, and other processes is presented in Section 5 of this report. For this analysis, additional numerical wave modeling was conducted to evaluate sediment stability upon removal of the Tank Farm Pier. For this purpose, the pier was removed from the numerical modeling grid and the model was re-run for two modeling scenarios: 10- and 25-year return period storm events.

6.2.3. Sediment Data

Information on physical characteristics of bottom sediment located directly under the Tank Farm Pier became available in May 2012. Three surface sediment samples from the crest of the mound under the pier were collected and analyzed for size gradation. Gradation curves of the three samples, D1, D2, and D3, are similar. The individual gradations and the average of the three are plotted in Figure 6-3. The median size (D50) is 0.8 mm and is classified as coarse sand.

The representative bottom sediment size of 0.8 mm is used for analysis of sediment stability related to removal of the Tank Farm Pier along the entire length of the mound.

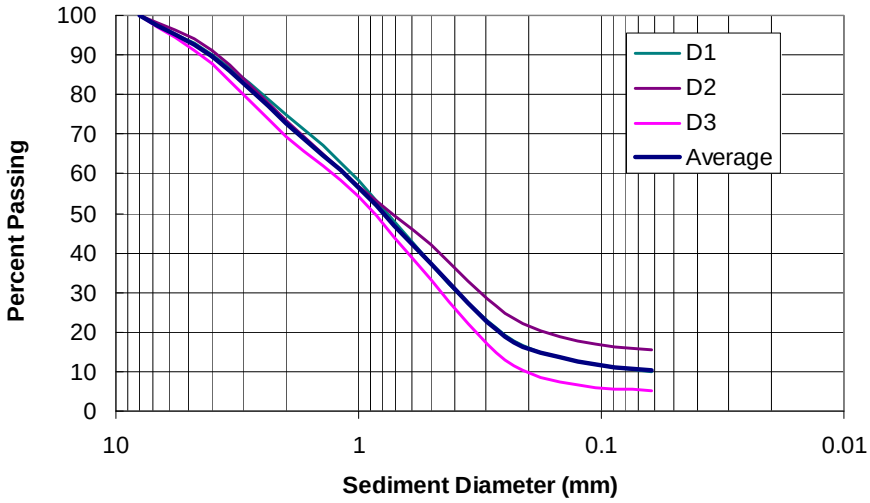


Figure 6-3. Gradation of surface sediment collected under Tank Farm Pier

6.3. Under-Pier Mound Morphology

Mound morphology was analyzed and deposition rates were estimated using bottom survey data discussed above in Section 6.2.1. Bottom elevation data were extracted from this survey at six cross-sections spaced at equal distance along the pier. The locations of the sections are shown in Figure 6-4. Cross-section profiles are plotted in Figure 6-5.

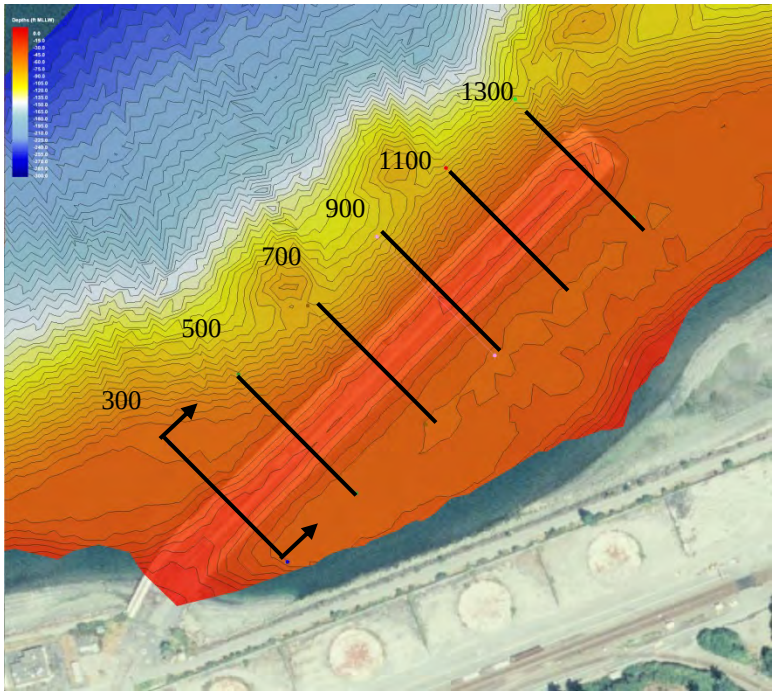


Figure 6-4. Location of sequential cross-sections shown in Figure 6-5

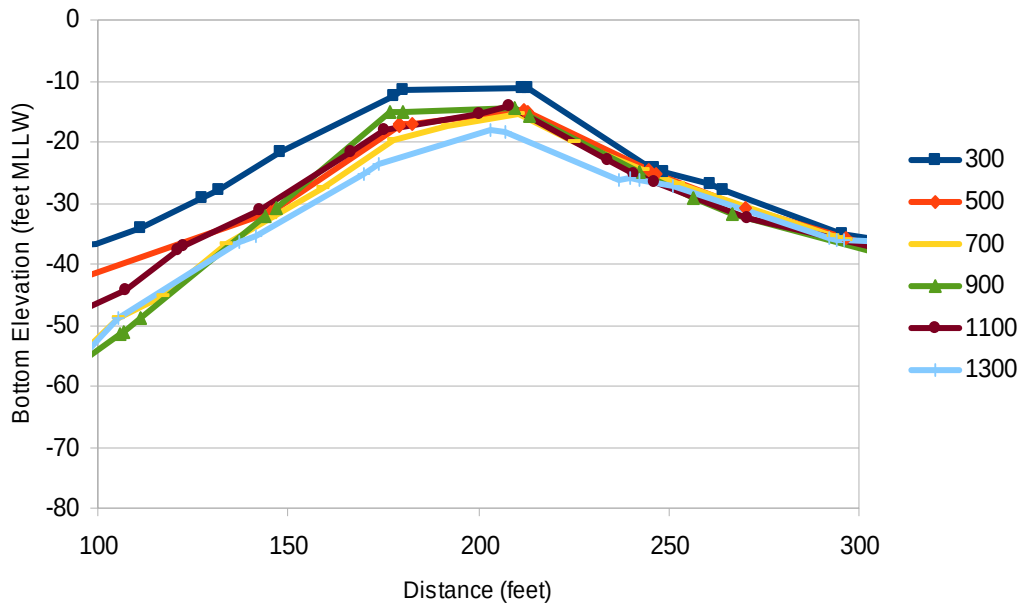


Figure 6-5. Cross-section profiles along Tank Farm Pier

The plotted elevations depict a mound-shape formation under the Tank Farm Pier, which runs nearly the full length of the pier. The highest crest elevation of the mound, approximately -11 ft MLLW, is located at the nearshore area at the most landward part of the pier. Farther seaward, the mound crest elevation lowers and is approximately -19 ft MLLW at the most distal cross-section.

Several hypotheses are possible for explaining the mound formation and evolution. One hypothesis suggests that the mound was formed by deposition of littoral drift (longshore and cross-shore sediment transport induced by nearshore waves and currents). Central to this hypothesis is the attenuation of waves and currents by the system of piles under the pier, resulting in sediment deposition and formation of the mound. The observed distribution of crest elevations along the mound is evidence that supports this hypothesis. Sediment is transported alongshore at a higher rate in the wave breaking zone of the profile, which corresponds to a larger size of the mound at the landward part of the pier. A smaller volume of littoral drift offshore in deeper water corresponds to a smaller size of the mound (also see CHE's 2006 report for other supporting evidence for this hypothesis).

Another hypothesis of mound origin considers a possibility of deliberate placement of sediment under the pier. The relative regularity in height, slope and cross-section, as well as the symmetry of the mound argues for it being a constructed feature. It is conceivable that fill was deliberately placed beneath the future pier to provide lateral support for long, relatively slender timber piling. Or, another possibility is that disposal of material dredged from the slip or berth landward of the pier may have been accomplished, at least partly, by sidecasting or moving the material out of the berth and into the area under the proposed pier.

As discussed above, there is not sufficient information or data to support or reject either of the above hypotheses. Furthermore, there is not sufficient bathymetry data to evaluate the time of mound formation. One may assume the mound had formed immediately after or during construction of the pier and has not changed since then. With the same certainty (or uncertainties), it may be assumed that the mound is a long-term formation and is still in a process of evolution. Considering the lack of data and uncertainties with explanations of mound formation, it is not feasible to estimate sediment depositional rates under the pier. It is therefore also not possible to predict future mound development at any reasonable level of accuracy.

6.4. Pier Removal Sediment Mobilization Analysis

The sediment mobilization analysis was based on computations of orbital bottom velocities generated by wind-waves and comparing these orbital bottom velocities to threshold velocities for initiation of motion of the bottom sediment. The main assumptions of the analysis included the following:

- Bottom surface sediment is mobilized when the orbital water velocity at the water-sediment interface is higher than the threshold velocity of sediment motion. Once mobilized, this surface sediment is moved outside of the pier area.
- Sediment under the mound is not a single-size but is composed of different sizes (has a size gradation). Larger sediment that is not mobilized (threshold velocity for initiation of motion is higher than the orbital water velocity) would be stable and form an armor layer on the surface of the remaining part of the mound. The thickness of this armor layer would increase with time and at some point would provide protection to the mound against future scour.

Based on these assumptions the mobilization analysis was conducted in two steps. First, analysis determined the bottom orbital velocities during design storm conditions and the size of sediment that may be mobilized during these storms. The second step estimated the total volume of material in the mound under the pier and evaluated the volume of material that may be scoured and dispersed during the design storm event.

The first step of analysis (orbital velocity and stable sediment) included 2-D wave refraction-diffraction numerical modeling with the SWAN model. Discussion of the modeling procedure and modeling grid is presented in Section 5 of this report. The model outputs include orbital velocities in the water column above the bottom sediment. These velocities were extracted from the modeling grid at two locations under the pier. Figure 6-6 shows the locations of the extraction points. Location 1 is on the centerline of the pier, approximately one-third of the distance from the seaward end of the pier, where the bottom elevation is at approximately -14.4 ft MLLW. Location 2 is near the centerline of the pier and close to the nearshore end of the pier, where the bottom elevation is at approximately -10.7 ft MLLW.

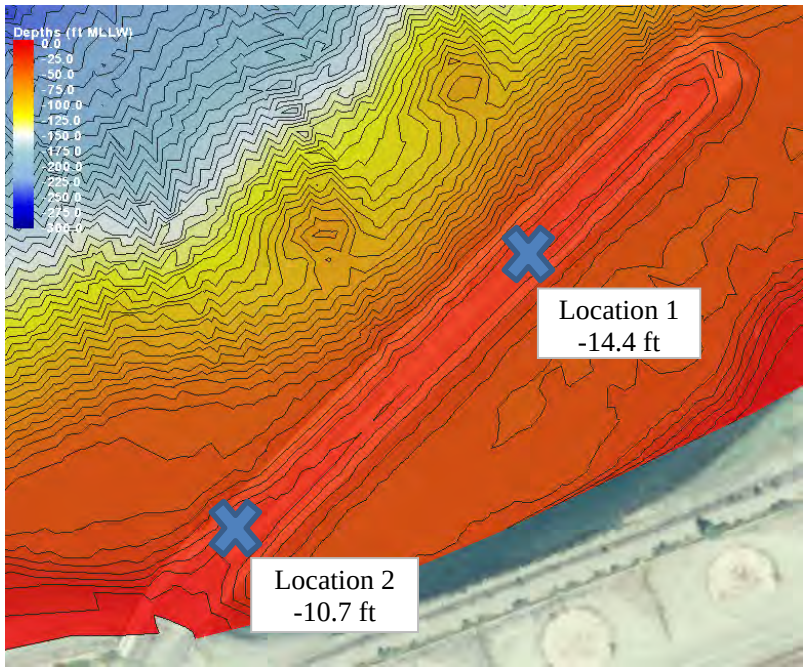


Figure 6-6. Locations of wave-induced velocity extraction points for sediment stability analyses

Sediment stability analysis was conducted for two extreme wind-wave storm events during MHHW conditions. The results of these analyses are shown in Table 6-1 in terms of grain size at threshold of motion for the selected wave cases.

Table 6-1. Grain Size Stability Analysis

Wind speed 33.1 knots (10-yr return period) Wind direction 310 deg Tide Stage MHHW				
Location	**Water depth**	**Wave Height**	**Wave Period**	**Stable grain size**
no.	ft	ft	sec	mm
1	25.5	3.4	4.0	0.8
2	21.7	3.3	4.0	1.2
Wind speed 36.5 knots (25-yr return period) Wind direction 310 deg Tide Stage MHHW				
1	25.5	4.5	4.4	1.7
2	21.7	4.5	4.4	2.5

The data in Table 6-1 demonstrate that the maximum size of sediment to be mobilized during a 10- or 25-year return period storm event at MHHW elevation is in the range between 0.8 and 2.5 mm, depending on the location of this sediment along the mound. For further analysis and computation of volume of scour, the sediment size of 2.0 mm was used as maximum size for mobility for the entire mound, which represents a conservative value estimate.

The second step of analysis is to estimate the volume of sediment that may be scoured during the design storm event. The total estimated volume of sediment in the mound under the pier is approximately 70,000 cubic yards (cy). Sediment data collected under the pier and provided to CHE on May 30, 2012 showed that a significant part (60 percent or more) of sediment in the mound is represented by coarse sand, very coarse sand, and gravel. Figure 6-3 shows the gradation of under pier sediment samples.

The volume of scour was estimated using new grain size data and one storm event with a return period of 25 years and water level of MHHW. Velocity at the sediment surface due to the specified wave and water level indicates sediment with a size of less than 0.8-2.5 mm (depending where this material is located along the pier) could be eroded. As discussed above, the sediment with sizes coarser than 0.8-2.5 mm would be stable and form an armor layer on the surface of the remaining part of the mound. This armor layer would consist of a mixture of sand and gravel, with gravel predominately on the surface. The thickness of this armor layer would increase with time and provide protection to the mound against future scour.

Typically, the thickness of a developed armor layer is on the order of 3 to 5 times the diameter of the stable size of sediment. For the purpose of scour depth calculation, the future armor layer at the mound is assumed to be 3 inches thick. Based on this surface thickness, the depth of mound scour was computed as a function of loss of all sediment particles smaller than 0.8-2.5 mm within the surface layer. The resulting depth of scour averaged over the length of the mound for the design storm at low tide was computed to be 0.3 ft.

The volume of material that may be eroded from the mound during the design storm was computed assuming a linear proportion between depth of scour and volume of scour (plan area of scour remains the same). The maximum volume of sediment estimated to be eroded from the mound during a 25-year return period storm is approximately 1,050 cy. Part of this volume would probably be eroded during a smaller storm (5- to 10-year return period) that may occur prior to the more extreme event. Therefore, in reality sediment eroding from the mound would be transported into the adjacent littoral system in small portions at a low frequency of occurrence, corresponding to occurrences of extreme wave storms. However, to provide a conservative answer to the question “Where will this limited amount of material land?” the entire volume of erosion (1,050 cy) is used, assuming that an extreme 25-year storm event occurs right after removal of the pier.

Figure 6-7 shows the existing bathymetry around the pier and the direction of approach of the design storm (25-year return period). As discussed above, this storm may result in a maximum volume of scour of 1,050 cy of sediment from the mound if the pier has been removed at a time of storm arrival.

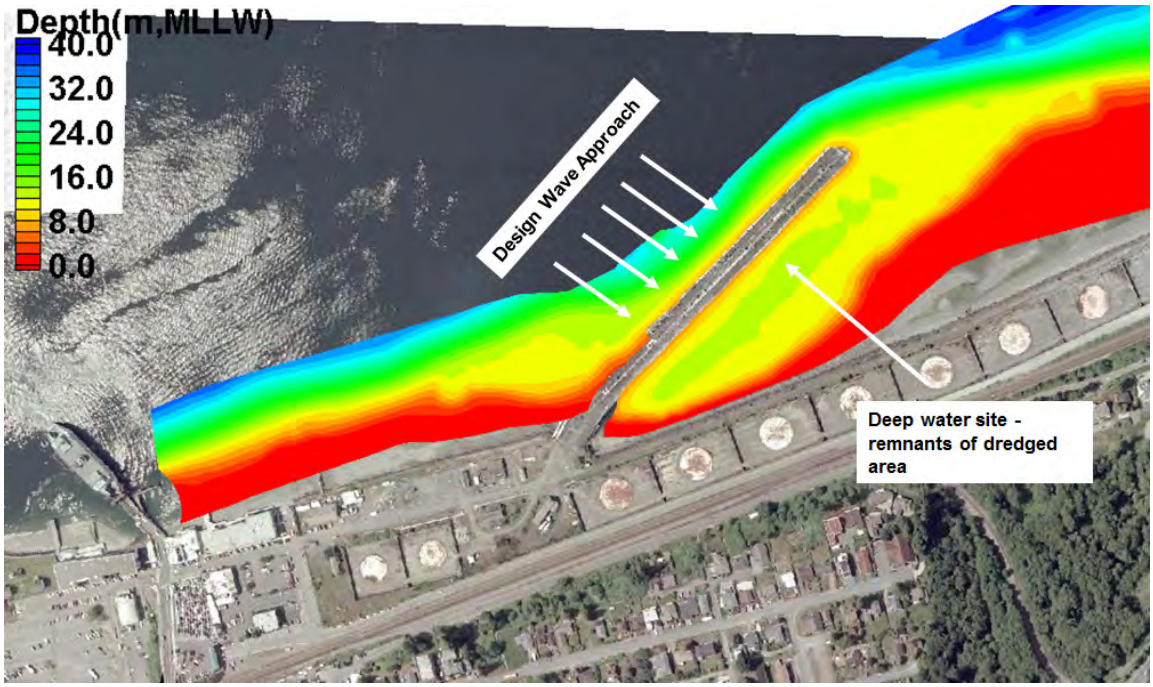


Figure 6-7. Existing bathymetry around the pier

Migration of sediment during a storm typically occurs in the direction of wave propagation that is perpendicular to the pier and toward the shoreline, as shown in Figure 6-7. Figure 6-7 shows a depression with an average depth of 35 to 37 ft MLLW at the shoreward side of the existing pier. The depth at this depression is 15 to 20 ft more than at the adjacent bottom. This depression is the remnant of a formerly dredged area and will act as a settling basin (sediment trap) for sediment that may erode from the mound during the design storm.

As discussed above, the total amount of sediment that may possibly erode from the mound during the design storm event is estimated to be approximately 1,050 cy. Part of this volume would settle in the sediment trap (deepwater depression) at the shoreward side of the pier. Only part of the eroded material from the mound would be able to bypass the depression and deposit somewhere downcurrent from the pier.

A simplified mass balance of eroded sediment was calculated assuming the most conservative scenario: the 25-year storm occurs right after removal of the Tank Farm Pier; no new terminal construction in this shoreline reach; the volume of eroded sediment from the mound would be 1,050 cy; only 50 percent of the eroded material settles in the deepwater depression and 50 percent bypasses the sediment trap, migrates in one direction, and is deposited in areas within 2,000 ft from the pier. The estimated average thickness of deposition was then found to be 0.08 inch. Considering the negligible thickness of deposition, no further analysis or numerical modeling is recommended to evaluate the volumes and areas of sediment deposition.

Based on the analysis, it is concluded that removal of the Tank Farm Pier will result in some mobilization of bottom sediments on the mound beneath the pier during a

10-year or greater return period storm. However, because the grain size of sediment composing the mound includes a significant amount of coarse sediment, an armoring layer with coarse sand and gravel would form on the surface that limits 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.

7. Possible Impact on Water Quality at NOAA Fisheries Service Intake

The objective of the analysis under this task was to determine possible impacts on water salinity at the NOAA intake due to changes in hydrodynamics associated with any of the four proposed Mukilteo Ferry improvement alternatives. The location of the NOAA Fisheries Service intake and the four alternatives are shown in Figure 7-1.

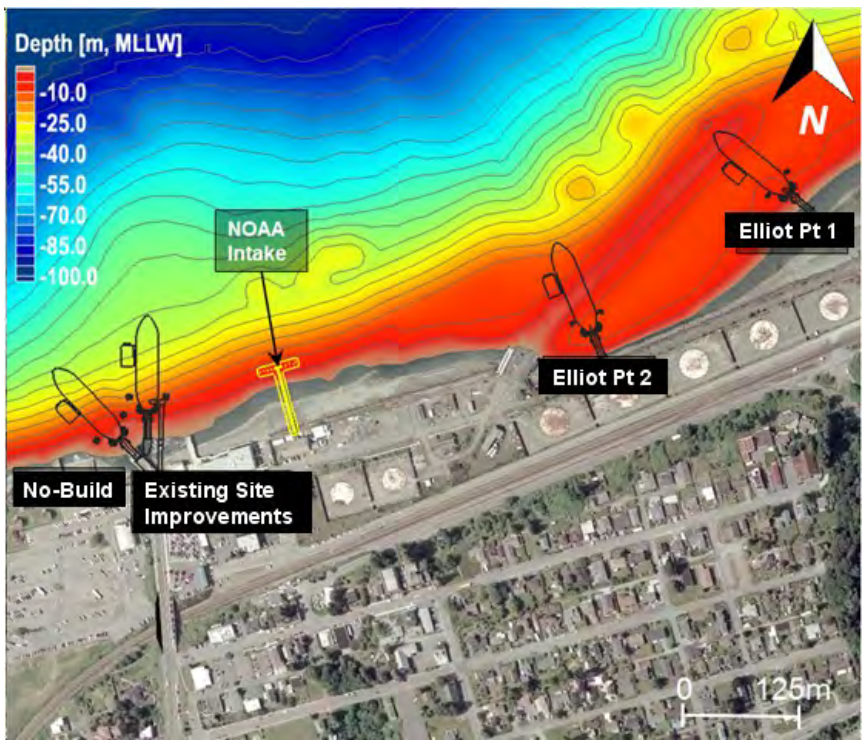


Figure 7-1. Location of NOAA Fisheries Service seawater intake and terminal alternatives

The primary seawater intake is a 6-inch diameter pipeline with the intake extending about 60 ft perpendicular into the nearshore zone beyond the end of the pier (northerly direction). The depth of the intake is approximately 40 ft below MLLW. In discussion with NOAA, the salinity of the seawater from this intake is 30 practical salinity units (psu) plus or minus 2 psu throughout the year (personal communication with Paul Plesha, December 2011). Investigation of change of salinity was based on effects of ferry propeller wash.

Analysis of possible impact on water salinity was determined through 3-D hydrodynamic and salinity modeling using the commercially available model FLOW3D. Modeling was conducted with two methods of analysis. The first method of analysis was to see if the alternatives produced modeled changes of flow velocities in the modeling domain,

specifically at the NOAA Fisheries Service intake. If the model identified any changes of flow velocity at the intake, the second method would involve modeling of salinity. For both methods, a time series for both velocity and salinity results, and with and without the terminal (ferry) would then be extracted and compared at the intake.

Modeling of the propwash impact was conducted for the worst case scenario alternative, the Existing Site Improvement Alternative. This alternative was selected as the worst case scenario due to its proximity to the NOAA Fisheries Service intake. Any possible changes in salinity at the NOAA Fisheries Service intake would most likely be sensitive to propwash impacts that are generated from ferry operation at this alternative.

The FLOW3D modeling domain was constructed to include nearshore survey bathymetric data at the site. The boundary of the domain extended 1,300 ft in the north-south direction and 1,480 ft in the east-west direction. The rectilinear grid consisted of over two million cells with variable resolution. The spacing of the grid ranged from 1.5 to 15 ft. The modeling bathymetry used for the 3-D modeling is shown in Figure 7-2.

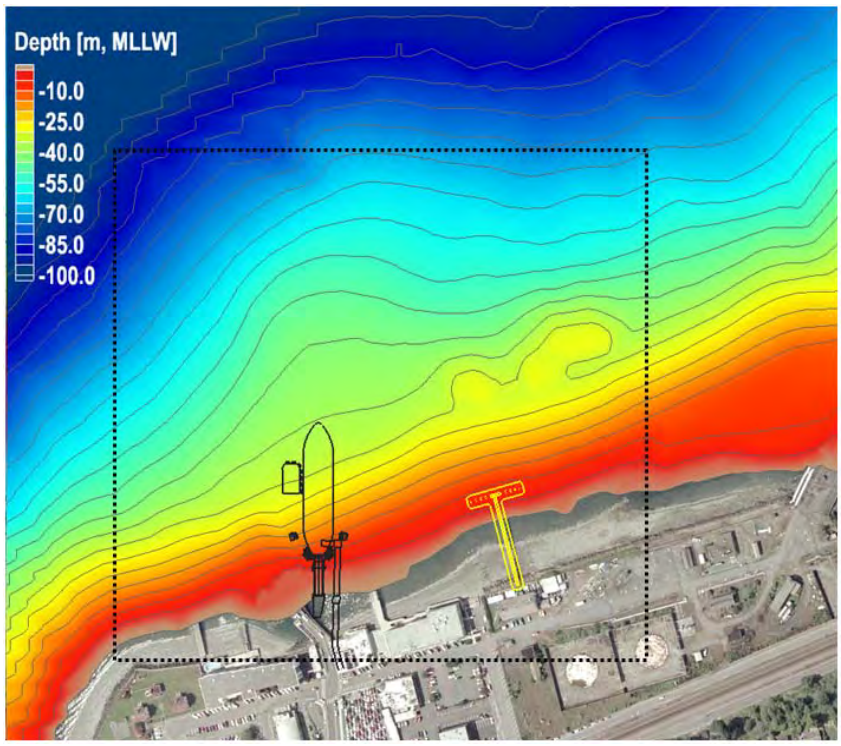


Figure 7-2. Modeling domain (outlined in black) and bathymetry

The New 144 Auto Class Ferry was used for modeling with the Existing Site Improvement Alternative. The depth of the center of the propeller is located 10 ft below the surface of the water with a propeller diameter of 11.5 ft and a maximum propwash velocity at the propeller of 14 ft/sec (CHE 2011).

Possible stratification of salinity through the water column in the modeling domain and at the location of the NOAA Fisheries Service intake was analyzed to develop reliable input

parameters for 3-D salinity modeling. Existing salinity stratification near Mukilteo was investigated by compiling and analyzing a time series of measured conductivity-temperature-depth (CTD) transect data provided by King County Puget Sound Marine Monitoring. The location of the nearest offshore CTD monitoring station is identified as JSUR01 – Point Wells Offshore, as shown in Figure 7-3. Time history transect data is shown in Figure 7-4.

By analyzing over six years of salinity profile data at JSUR01, the data suggests that the water column is weakly stratified. Figure 7-5 shows the average salinity profile over the time period. The data shows that typical salinity variability is within 1 psu. The mean salinity at the propeller depth is approximately 29.8 psu, while the mean salinity at the NOAA Fisheries Service intake depth is approximately 29.2 psu.

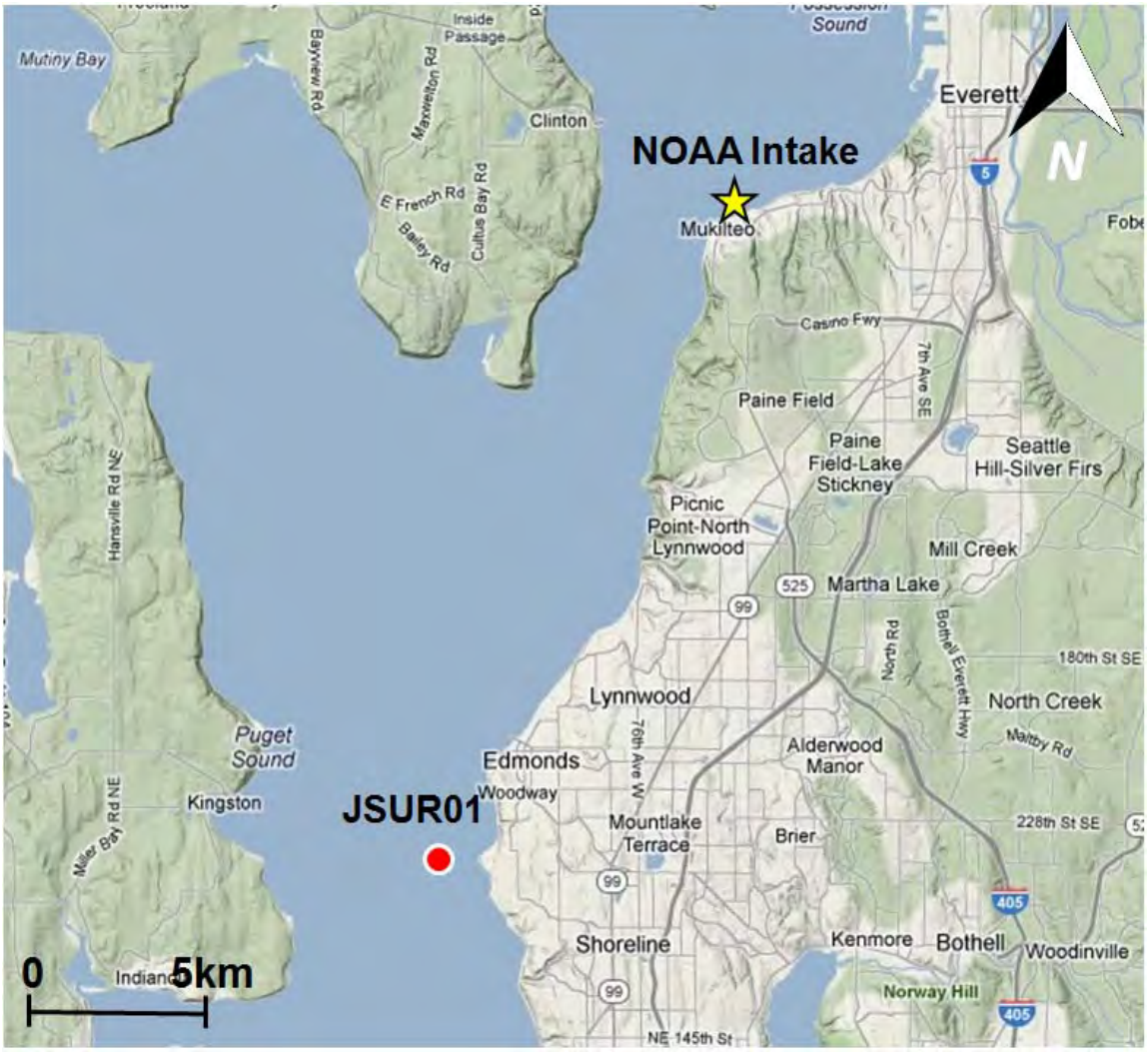


Figure 7-3. Location of nearest offshore CTD monitoring station, JSUR01 – Point Wells Offshore

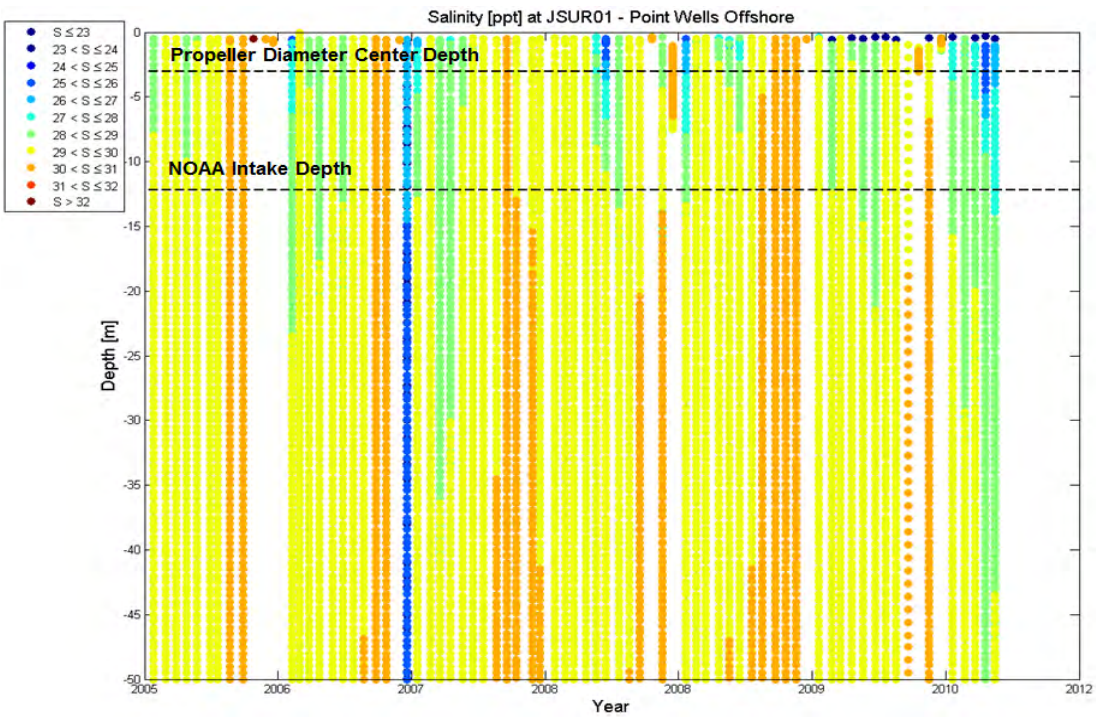


Figure 7-4. Time history of salinity profiles at JSUR01 – Point Wells Offshore

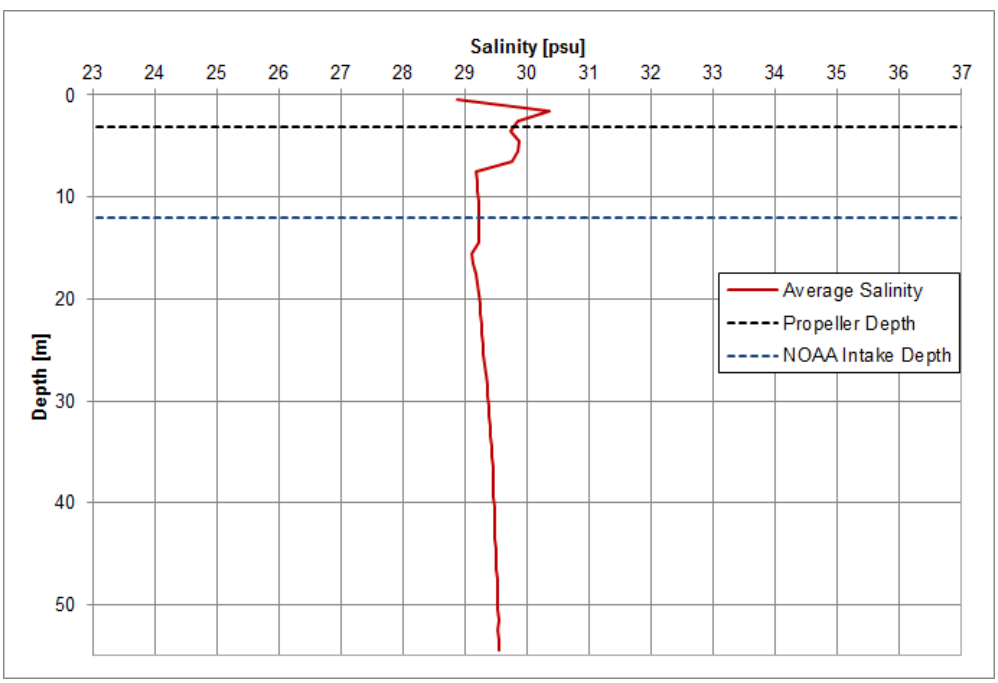


Figure 7-5. Average of salinity profiles at JSUR01 – Point Wells Offshore

The average salinity profile shown in Figure 7-5 was used as an initial salinity condition for the purpose of salinity modeling.

Modeling of impact from propwash on salinity conditions at the NOAA Fisheries Service intake was conducted for both inbound and outbound ferries. For the inbound case, the ferry was situated at the terminal (Existing Site Improvement Alternative) with a constant propwash jet. For the outbound case, the ferry was leaving the terminal at a northerly direction with a constant acceleration. The ferry would start from 0 speed and accelerate to a maximum 6 knots ship velocity over a period of 60 seconds. The exit flow velocity at the jet was computed at 14 ft/s for both cases.

The results of the velocity modeling are shown in Figure 7-6 and Figure 7-7. Figure 7-6 is a snapshot of plan view velocities for an inbound (docked at the terminal) ferry, assuming the ferry propeller rotates at a constant rpm with constant horsepower on the shaft. Figure 7-7 shows plan view velocities for the accelerating outbound ferry.

Both figures show snapshots of plan view velocities in the modeling domain resulting from the propeller in color format. Red indicates high velocity, while blue indicates zero velocity. Because the model is three-dimensional it is able to output velocities at different depths. The figures show plan view velocities at two depths: 10 ft (propeller depth), and 40 ft (NOAA Fisheries Service intake depth) below MLLW.

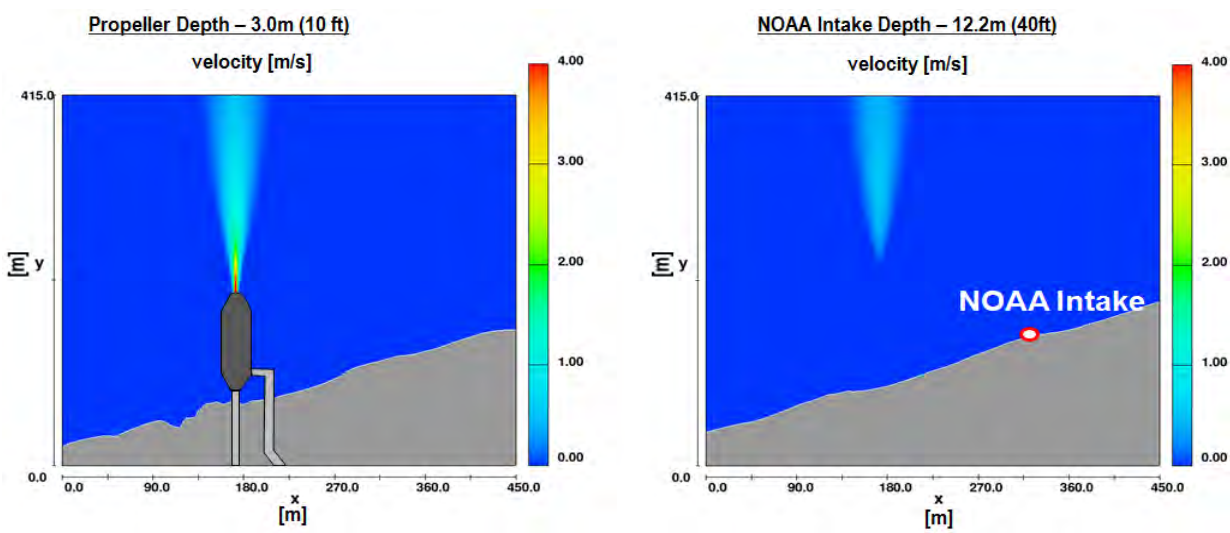


Figure 7-6. Velocity results (shown in m/s) for inbound case at propeller depth (left) and NOAA Fisheries Service intake depth (right)

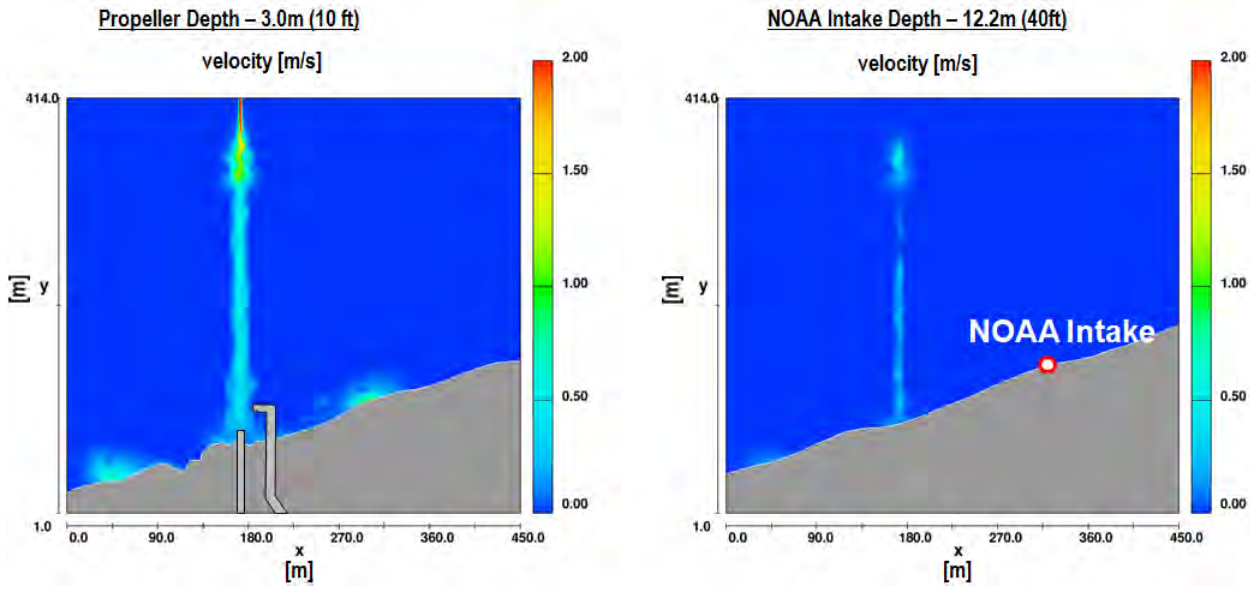


Figure 7-7. Velocity results (shown in m/s) for outbound case at propeller depth (left) and NOAA Fisheries Service intake depth (right)

The analysis of plan view velocities from the modeling results for both inbound and outbound cases demonstrate that no detectable velocity change at the NOAA Fisheries Service intake would occur due to ferry operations at the Existing Site Improvement Alternative terminal. In order to assure this conclusion, a time series of velocities were extracted from the modeling grid directly at the NOAA intake location. Figure 7-8 shows the time series of velocities, extracted directly at the intake location during the period of simulation.

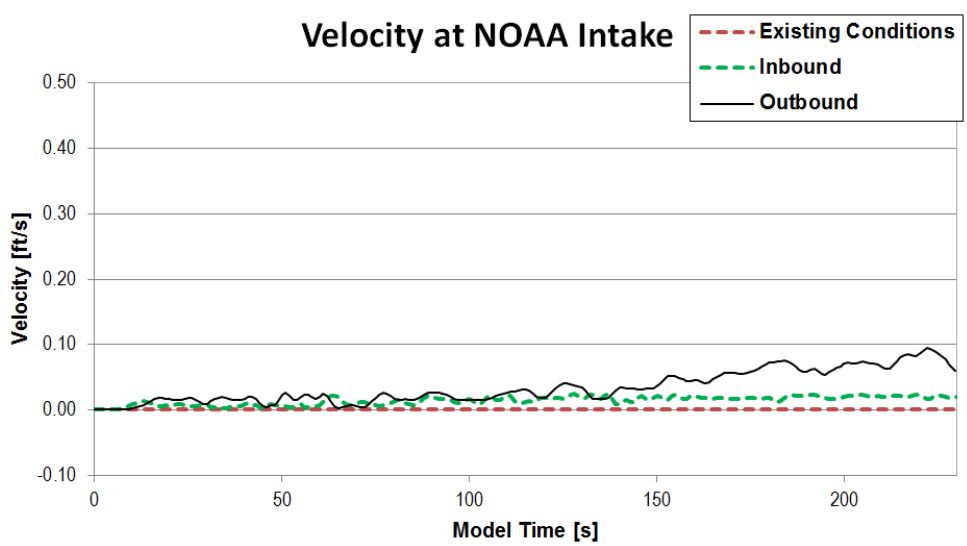


Figure 7-8. Time series of current velocities magnitude extracted at NOAA Fisheries Service intake for both inbound and outbound ferries for Existing Site Improvement Alternative

The figure shows current velocities at the intake during operations of inbound and outbound ferries relative to existing conditions (existing conditions coincide with zero line). The results of the modeling indicate that during operations of the inbound ferry, the possible change of velocities at the intake would be less than 0.03 ft/s (0.001 m/s). However, during operations of the outbound ferry, the model predicts that the maximum increase of velocities may be up to 0.1 ft/s (0.03 m/s). With these results, the changes in velocities are considered negligible and would be very difficult to detect in the real environment. Furthermore, with the presence of strong ambient currents and wind-wave action, any small propwash velocities would be completely masked by ambient turbulence. The velocity modeling results already would be sufficient to conclude that there will be no impact from the ferry operations on salinity at the NOAA intake. Negligible changes of velocities would not be able to generate any detectable change in salinity.

Modeling of salinity also was performed for both inbound and outbound scenarios. As stated previously, the average salinity profile shown in Figure 7-5 was used as an initial salinity condition for the model. The results of the salinity modeling are shown in Figure 7-9 and Figure 7-10.

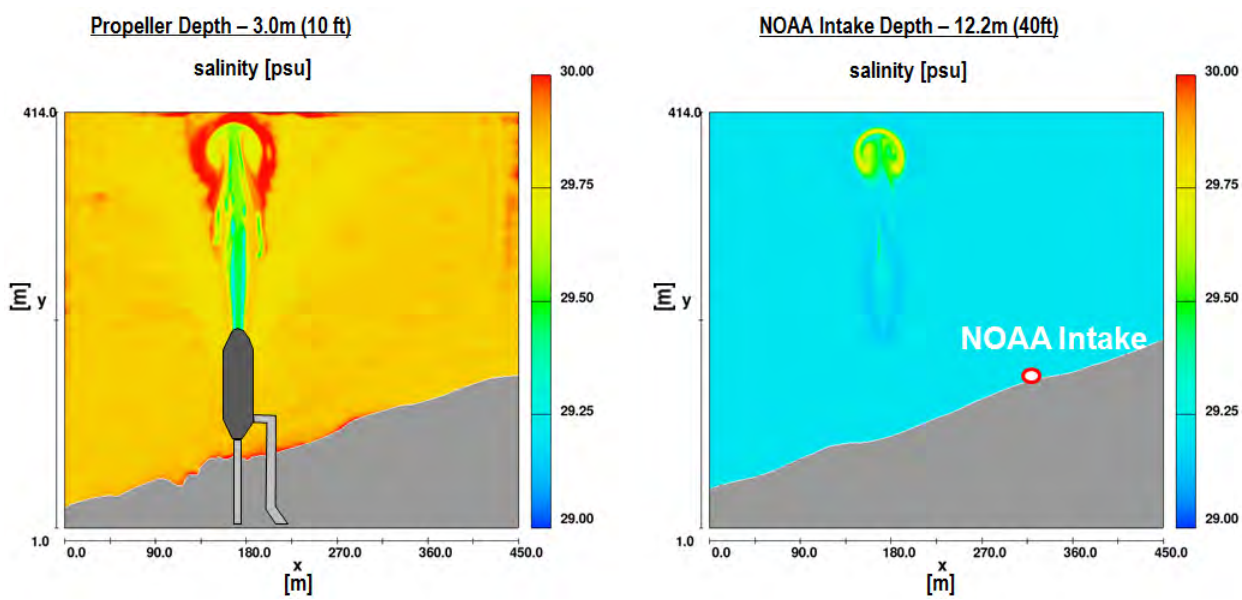


Figure 7-9. Salinity results (shown in psu) for inbound case at propeller depth (left) and NOAA intake depth (right)

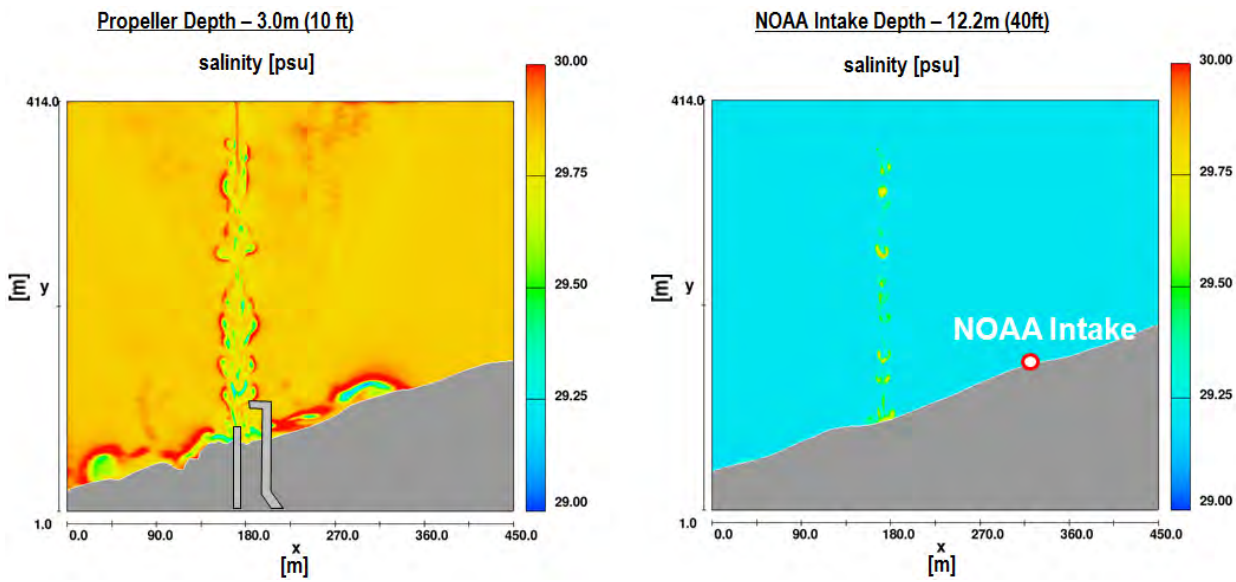


Figure 7-10. Salinity results (shown in psu) for outbound case at propeller depth (left) and NOAA intake depth (right)

The analysis of plan view salinity values from the modeling results for both inbound and outbound cases demonstrate that changes in salinity at the NOAA Fisheries Service intake would be negligible (as in agreement with the velocity changes). For further investigation, time series results of the modeling were extracted at the NOAA intake and are shown in Figure 7-11.

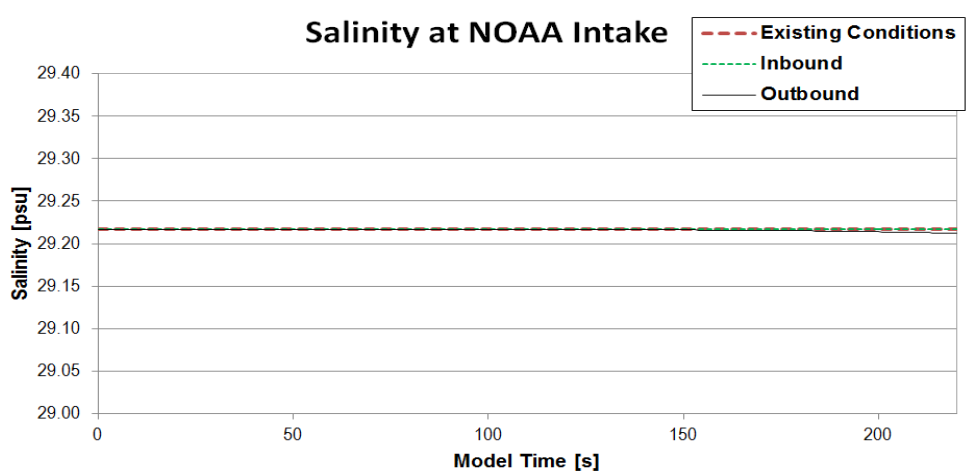


Figure 7-11. Time series of salinity values extracted at NOAA Fisheries Service intake for both inbound and outbound ferries for Existing Site Improvement Alternative

The figure shows deviations of salinity at the NOAA Fisheries Service intake relative to existing conditions during operations of the inbound and outbound ferries. The modeling results demonstrate that maximum change of salinity at the intake due to ferry operations

would not exceed 0.005 psu. This maximum change of salinity is negligible and represents only a small fraction of the natural variability of salinity at the project area.

Numerical modeling of flow velocities and salinities has demonstrated that there would be no detectable impact on salinity at the NOAA Fisheries Service intake associated with the Existing Site Improvement Alternative. The same conclusion is valid for the Elliot Point 1 and 2 Alternatives due to the remote locations of these alternatives relative to the intake. It is concluded that none of the proposed terminal alternatives would result in change of salinity at the NOAA Fisheries Service intake relative to existing conditions.

8. Pile Removal, Dredging, and Stone Column Installation – Temporary Effects on Water Quality for Elliot Point 2

This section of the report presents the results of CHE's analysis and evaluation of the possible temporary effects on water quality (turbidity) caused by dredging, pile removal, and stone column installation during construction of Elliot Point 2 for the Mukilteo Multimodal Project. At the time of this analysis a slightly modified version of Elliot Point 2 Alternative had been selected as the Preferred Alternative for the FEIS. Therefore, the other three alternatives were not modeled for temporary water quality effects. Analysis and results described here for Elliot Point 2 Alternative apply to the Preferred Alternative.

8.1. Project Site Ambient Conditions

This section discusses ambient conditions at the project site that need to be addressed for the analysis of potential water quality changes during dredging, pile removal, and stone column installation in the vicinity of the Tank Farm Pier. Conditions include bottom depths, current velocities, and bottom sediment characteristics. For a complete and detailed description of physical conditions at the project site, see Section 2 of this technical report.

8.1.1. Bathymetry

Figure 8-1 shows the Elliot Point 2 Alternative location and surrounding bathymetry where dredging, pile removal, and stone column installation are designed to take place.

As shown in Figure 8-1, the bottom depths in the vicinity of the area (ferry terminal and Tank Farm Pier) range from approximately 10 to 35 ft relative to Mean Lower Low Water (MLLW). As discussed in Section 6.2 of this report, the area under the Tank Farm Pier is shallow (approximately -10 ft, MLLW) due to the presence of a mound.

8.1.2. Current Velocities

Analysis and evaluation of ambient currents at the project site were conducted based on review of field data (current speed and direction) measured with an Acoustic Doppler Current Profiler (ADCP). During the period of measurements, the ADCP was located approximately 60 ft from the seaward side of the Tank Farm Pier, at a bottom depth of approximately 35 ft below MLLW. Figure 8-2 shows the location of the ADCP measuring station.

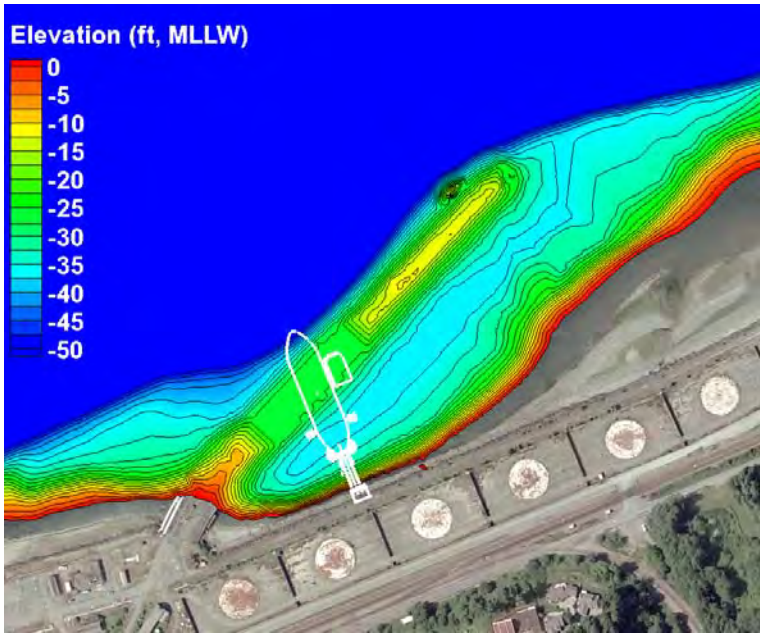


Figure 8-1. Elliot Point 2 Alternative site and dredged bathymetry

Figure 8-2. Location of ADCP measuring station

The ADCP was programmed to measure current velocity profiles from approximately 3.3 ft (1 meter) above the bottom, to the water surface in 3.3-ft increments. The time series of depth-averaged current velocities collected during a two-week period at the ADCP current measuring station is shown in Figure 8-3.

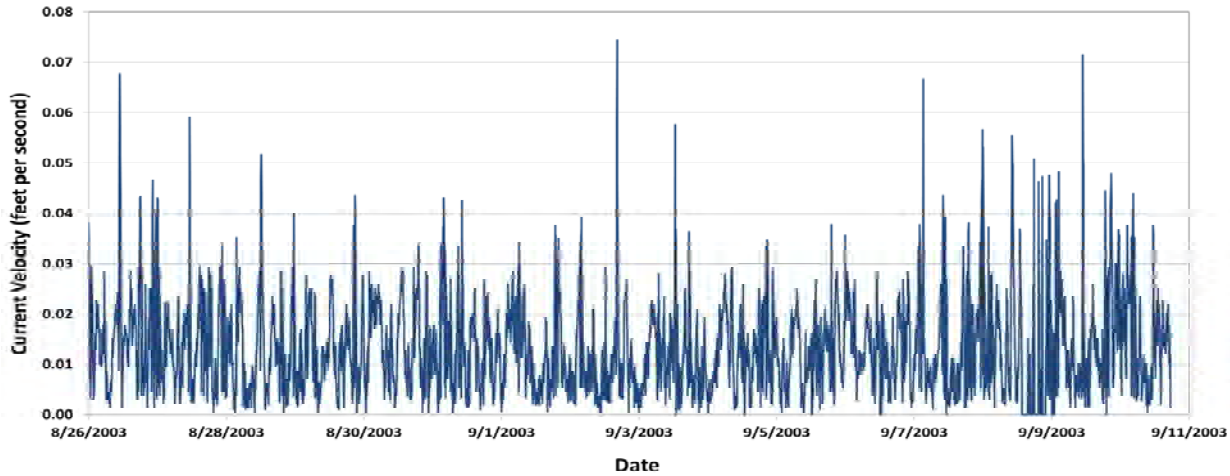


Figure 8-3. Depth-averaged current velocities at ADCP station

Statistical analysis of the measured data shows that depth-averaged current velocities near the project location are relatively small and do not exceed 1 ft per second. For further analysis, two representative values of current velocities were used: average maximum current velocity (0.02 ft per second); and absolute maximum current velocity (0.075 ft per second).

8.1.3. Bottom Sediment

The available sediment data show some spatial variability of sediment characteristics along the project site and under the pier. In general, based on the grain size analysis results, it appears that the median sediment size (D_{50}) ranges from approximately 0.3 mm to 0.8 mm, depending on the location and depth of the sample.

Typically, finer (smaller) grain size material contributes more significantly to formation and propagation of turbidity plumes in the water during in-water construction. Therefore, for a conservative estimate, the sediment sample with the finest grain size characteristics (D_{50}[9] = 0.3 mm) was chosen for the turbidity analysis. Figure 8-4 shows the grain size distribution based on the finest core sample taken from the site. Data plotted in Figure 8-4 show that D_{10} = 0.05 mm. Further, the turbidity analysis was performed for two different grain sizes: D_{50} (0.3 mm) and D_{10} (0.05 mm), to obtain a conservative estimate as a result of in-water construction activities.

[9] D_{50} is the particle size at which 50 percent of the material by weight in a grain size distribution is smaller than the given size. In other words, 50 percent of material in the grain size distribution has a diameter smaller than D_{50}. Similarly, D_{10} is the particle size at which 10 percent of the material in a grain size distribution is smaller.

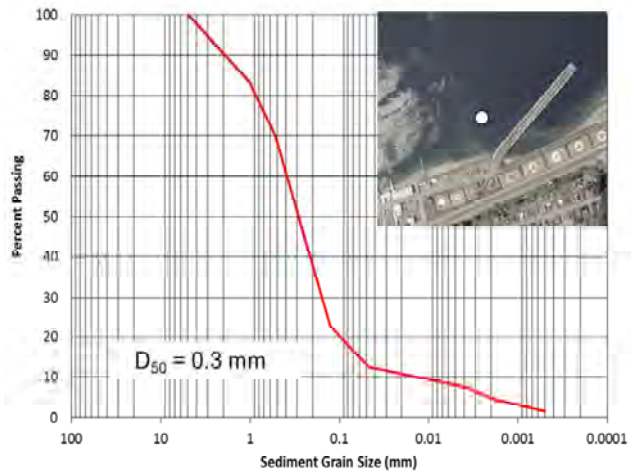


Figure 8-4. Grain size distribution based on finest core sample taken from site

8.2. Water Quality Criteria

During construction operations, water quality and turbidity conditions at the project site will be monitored in accordance with the *401 Water Quality Certification* and *WAC 173-201A-210* (WAC, 2012) for marine water bodies (See Section 8.4 for a detailed description of the monitoring program). Per *WAC 173-201A-210*, a 150-foot radius area of mixing is established from the in-water work location. Per the same *WAC 173-201A-210*, an increase in ambient turbidity outside of the established 150-foot radius area is limited to 5 Nephelometric Turbidity Units (NTUs) when the background is 50 NTUs or less, or a 10 percent increase in turbidity when the background turbidity is more than 50 NTUs. These criteria are applied herein for further analysis and evaluation of possible impact on water quality during dredging, pile removal, and stone column installation operations.

The water quality (turbidity) criteria in *401 Water Quality Certification* and *WAC 173-201A-210* are provided as NTU. Most of the reliable methods of analysis and numerical models operate with TSS units (Total Suspended Solids). The relationship between TSS and NTU is site-specific (Thackston and Palermo, 2000) and is reliable if data measured in a reasonable proximity of the project site are used for the calibration. No turbidity or water quality data were found at the exact location of the project site, but detailed field measurements of TSS (milligrams per liter, mg/L) and turbidity (NTU) were collected from previous studies near Mukilteo and were used to develop the relationship between TSS and NTU[10]. Figure 8-5 shows the linear relationship between TSS and NTU obtained from the measured data. Data were collected at a dredging and pile driving project for constructing terminal improvements at the Port of Everett having similar water depth and sediment type

[10] Measurements were conducted at the Port of Everett, approximately 4 miles away from the project site (Hartman Associates, 1996). Data are not currently available; only the best-fit regression of TSS and NTU values is available.

that exist at the Mukilteo Multimodal Project. TSS versus NTU at the two sites should be similar.

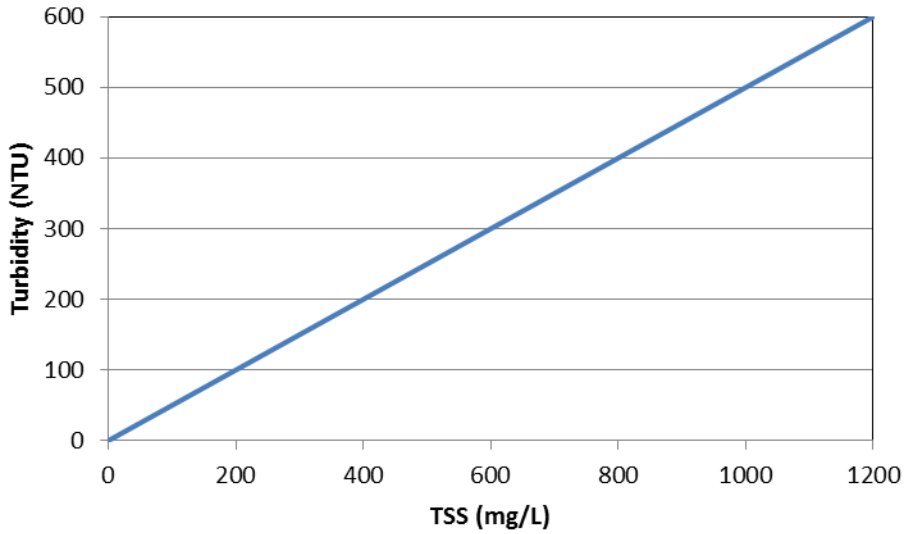


Figure 8-5. Turbidity (NTU) vs. TSS (mg/l) based on locally-measured data

8.3. Potential Construction Impacts on Water Quality

8.3.1. Dredging Impact on Water Quality

The dredging operations at the Elliot Point 2 Alternative will most likely be conducted with a clamshell dredge with flat deck and/or bottom dump (split hull) scow barges. Tugs will be used to position the dredge and barges within the work area. It is expected that dredging and dredged material disposal will be conducted using Best Management Practices (BMPs) and, where appropriate, following recommendations from Technical Guidelines for Environmental Dredging of Contaminated Sediments (Palermo *et al.*, 2008). Detailed instructions for the Contractor regarding bucket type, BMPs, and guide recommendations will be provided in the contract document to be developed during the next phase of the design. Additional sediment sampling and analysis will be conducted in this next phase, and a determination of suitability of dredged sediment for open water disposal will be made. With these results, the BMPs will be adjusted appropriately.

Dredging is expected to be conducted with an environmental type bucket (fully closed, cable arm, or other appropriate type) that minimizes the spill of suspended sediment in the water column. There is a wide-range of bucket sizes in the industry. Typical crawler crane dredge buckets are cable-operated and range in size from 2 to 6 cubic yards and larger. A large size bucket (5.9 cubic yards) was used for the analysis for a conservative evaluation of possible impact on water quality[11].

[11] A larger clamshell bucket creates a larger return effluent flow that may produce a larger turbidity plume relative to a smaller bucket.

Dredging impact on water quality may occur through re-suspension of bottom sediment during initial contact of the bucket with the bottom, filling of the bucket, escape of re-suspended sediment through the water column upon lifting of the bucket or moving the bucket over the water, and escape of re-suspended sediment from the barge, if the barge is not sealed or if return water is allowed to overflow. Turbidity may be generated by other aspects of dredging besides those listed above.

To minimize re-suspension the following BMP measures shall be implemented:

- The bucket shall be lowered at a low speed to reduce impact and minimize re-suspension during initial contact with the bottom.
- The bucket closure shall be conducted smoothly without jerking to minimize re-suspension and avoid incomplete closure.
- Operation of the dredging equipment shall be conducted by an experienced Contractor who is able to determine and avoid incomplete bucket closure. If debris prevents bucket closure the experienced operator shall slightly adjust tension in the closing cable repeatedly, while not lifting the bucket, until the bucket closes.
- A WSDOT representative will be on site to observe pier removal and will be alert to water quality problems caused by debris in the case that the Contractor generates an unnecessary amount of wood fragments in the process.
- The specifications will request that split-hull dredged material barges shall be sealed and prohibit release of untreated water, and other barges shall be managed to prevent overflow of untreated water.

Re-suspended sediment generates a plume that propagates and expands in the water through diffusion and dispersion by ambient currents. The concentration of suspended sediment (observed as turbidity) in the plume reduces with distance from the dredging site. The area or distance (length) where diffusion and dispersion occurs is also referenced as the "mixing zone." Based on the water quality criteria discussed in Section 8.2, the impact on water quality is considered significant when water turbidity exceeds the background conditions by 5 NTUs (or more), or a 10 percent increase in turbidity, when the background turbidity is more than 50 NTUs, at a distance of 150 ft or more from the dredging site.

The U.S. Army Corps of Engineers' steady-state "DREDGE" model was used to calculate suspended sediment concentration (TSS) at the dredging site at the Elliot Point 2 Alternative and determine the length of the mixing zone. This model (DREDGE) is contained in the U.S. Army Corps of Engineers' Automated Dredging and Disposal Alternatives Management System (ADDAMS). The model is a standard tool that calculates TSS concentrations (mg/L) in a turbidity plume extending away from a dredging site. Sub-modules allow for specification of dredge-specific parameters, ambient conditions, and site-specific sediment properties.

DREDGE modeling was conducted using the following input data and assumptions:

- An environmental bucket for a mechanical clamshell dredge, 4.5 cubic meters (5.9 cubic yards) in size.

- Initial water depths ranging from 3 meters (9.8 ft) to 10 meters (33 ft).
- Ambient water velocities ranging from 0.02 to 0.085 ft/sec (See Section 8.2.2[12]). Please note that higher ambient velocities correspond to larger mixing zones, therefore representing more conservative conditions. Smaller ambient velocities correspond to smaller mixing zones.
- Sediment was assumed to be primarily fine sands and silt. Simulations were conducted with various sediment sizes, ranging from 0.3 mm (D_{50}) to 0.05 mm (D_{10}). These sizes corresponded to classifications of sand and silt, respectively.
- In-situ dry density for the sediment was assumed to be 700 kg/m³, as per DREDGE model manual recommendations.
- The sediment re-suspension rate or Turbidity Generation Unit (TGU), defined within the DREDGE manual as the rate sediments are re-suspended into the water column and carried away from the dredging site, was determined to range between 15,000 and 25,000 g/m³ (Nakai, 1978).
- Dispersion coefficients were set at 100,000 cm²/sec (107.63 ft²/sec) in the lateral direction and 10 cm²/sec (0.01 ft²/sec) in the vertical direction (DREDGE manual recommendations).

Using the above input parameters, a total of six DREDGE numerical modeling scenarios were completed. Examples of modeling results (worst-case scenario: depth-averaged current velocity = 0.085 ft/sec, water depth = 9.8 ft, sediment size = 0.05 mm, TGU = 25,000 g/m³) are presented in Figures 8-6 and 8-7.

Figure 8-6 shows a plan view of turbidity dispersion above background conditions in color format. Red color corresponds to a higher turbidity value. Blue color corresponds to lower or no turbidity value. The figure shows a pattern of dispersion of turbidity and decrease of NTU with increasing longitudinal distance away from dredging operations. The figure also plots the distance of 150 ft from the dredging site and indicates no detectable turbidity increase above the ambient level (background conditions).

Figure 8-7 is a longitudinal section through the middle of the modeling domain (lateral distance 0 ft) shown in Figure 8-6. The figure shows a decrease of turbidity along this section with increased distance from the dredging site. It also shows that the increase of turbidity at a distance of 150 ft from the dredging site would be smaller than 3 NTUs. Turbidity would reach ambient conditions at a distance of approximately 300 ft away from the source of dredging.

[12] For conservatism, the maximum value of current velocity chosen for the analysis was 0.085 ft/sec.

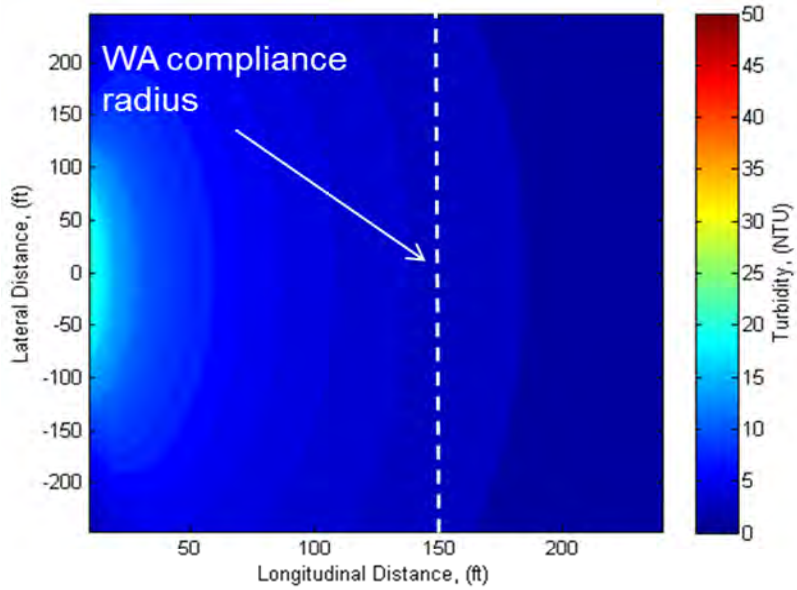


Figure 8-6. Plan view of turbidity above ambient level during dredging operations

Results of numerical modeling for all scenarios, including the most conservative scenarios, indicate that dredging operations for the Elliot Point 2 Alternative would not violate the existing water quality criteria, established by *401 Water Quality Certification* and *WAC 173-201A-210* for marine water bodies in the State of Washington. The results also indicate that turbidity control measures would likely not be needed for dredging operations. It should be noted that dredging work will be conducted using standard BMPs to control spread of turbidity, if indeed some unexpected sediment re-suspension occurs. To further minimize sediment loss back into the water and limit any impact to water quality, other more stringent BMPs could be instituted, if needed. These measures include, but are not limited to:

- Reducing the dredge cycle time and controlling the ascent and descent speeds of the clamshell bucket through the water column.
- Minimizing dredging activities during strong current events.
- Selecting areas to dredge with coarser sediments during periods of ebb and flood tide and finer sediments during periods of slack tide.

Lateral movement of the turbidity plume is dependent upon tidal current velocity and direction. Current direction data show that the flow is mostly bi-directional (east-west), with currents flowing to the east being stronger but less frequent than currents flowing to the west.

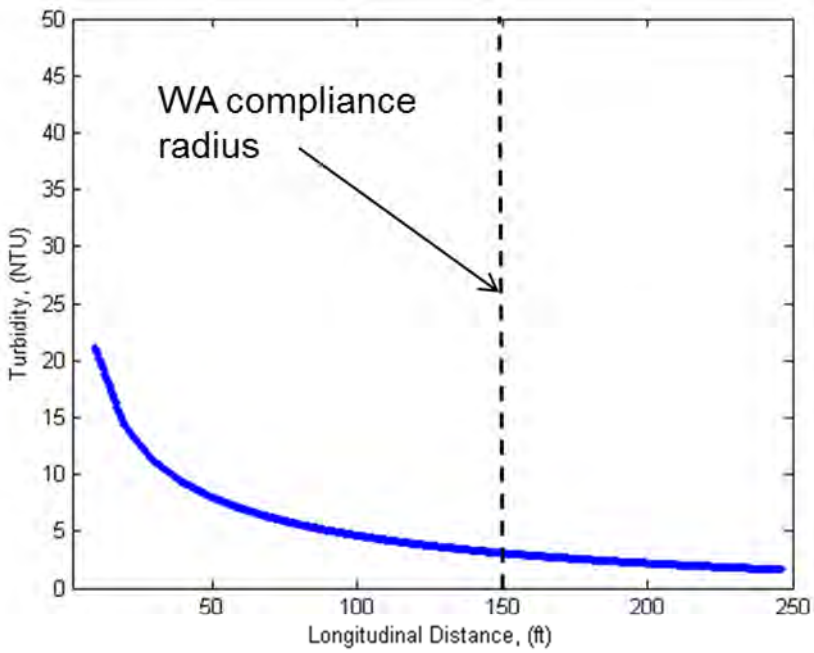


Figure 8-7. Section view through turbidity above ambient level during dredging, modeled at plume centerline

8.3.2. Pier Removal Impact on Water Quality

8.3.2.1. Pile Removal

Impact on water quality from pile removal at the Tank Farm Pier was analyzed based on review of available technical information and data collected from three projects with relevant construction and environmental characteristics. Available design and monitoring data from these projects were compiled and analyzed.

Lower Sequim Bay, WA, Pile Removal Project. The project consisted of removing 99 creosoted timber piles with a vibratory hammer. Monitoring of turbidity over the background conditions was conducted during the construction period. Suspended solids were monitored at fixed stations located at various distances from the piles. In addition to fixed readings, transects were performed during five pulling events. Background turbidity in Sequim Bay was approximately 5 to 10 NTUs at the time of the monitoring activities. Increase in turbidity upon removal of the piles was within 5 NTUs of background within 60 to 150 ft downstream of the piling removal. A detailed description of the monitoring program and results is described in Weston Pascoe (2006).

Port of Anacortes, WA, Former Scott Paper Mill Cleanup Project. Removal of piling and timber wall breakwater was one of the project components. Routine turbidity and water quality monitoring was conducted for pile removal operations. Turbidity (water quality) monitoring was conducted at stations located 150 and 300 ft from the work place. Turbidity measurements were recorded at each station at the surface, mid-depth, and bottom of the water column. No increase of turbidity above 5 NTUs

over background conditions was observed based on the monitoring data. A detailed description of the monitoring program and results is contained in CHE (2009).

Port of San Francisco, CA, Wharf J-10, Pile Removal. The Port of San Francisco repaired the Wharf J11 substructure by removing and replacing some of the existing bearing and fender piles. Monitoring was conducted to determine a possible increase of hydrocarbons and turbidity in the surrounding water during project activities. Water quality monitoring was conducted during a total of three days. Turbidity was measured with a portable nephelometric turbidity meter. Background turbidity was measured at least once per day approximately 20 ft outside of the construction area. The measured data demonstrated that pile removal did not appear to cause significant impact to near-surface bay water turbidity. Background turbidity was always less than 50 NTUs and measured turbidity was less than 8 NTUs above background. No measurable discharge of hydrocarbons was detected during pile removal and pile driving operations. A detailed description of the monitoring program and results is described in Weiss Associates (2009).

The review of available data from these three projects demonstrates that pile removal operations, if performed properly (applying BMPs), would not cause a significant increase of turbidity with respect to ambient conditions, even in close vicinity to construction activities. The review also shows that pile removal/installation/repair activities are likely to generate much less turbidity than dredging operations discussed in Section 8.3.1. In summary, it is concluded that removal of the Tank Farm Pier piles will not generate significant impact on water quality if the work is conducted following proper BMPs. Potential BMPs utilized for pile removal will include a variety of techniques to limit any significant impact to water quality. Pile removal specific BMPs might include the following:

- If the surface and/or subsurface sediment is contaminated, additional water quality monitoring (e.g., chemical analysis), silt curtains and/or surface sediment sampling will be considered.
- Removed treated wood and all adhered sediment will be disposed of in an approved upland location; treated wood from the removed pier will not be reused.
- Hydraulic water jets will not be used for pile removal; other effective and proven low-disturbance methods such as vibratory and cable extraction will be used to remove the piles.
- Grubbing for piling outside the new construction footprint will not be allowed.

8.3.2.2. Deck Removal

Demolition of the deck and bracing is not directly associated with potential increase of water turbidity, although it may be a cause of secondary impact if a violation of standard and BMP practice occurs. To avoid a secondary impact on water quality, the Contractor's management plan submittal shall satisfactorily address the practical prevention of generating sinking and floating debris by the pier removal. The submittal shall provide details on demolition sequencing and selection and placement of equipment to minimize the loss of debris into the water.

8.3.3. Stone Column Installation Impact on Water Quality

The areas of the onshore facilities and the nearshore trestle of the proposed Elliot Point 2 Alternative need to be improved with stone columns to mitigate the potential lateral spread due to widespread liquefaction in the project area. It is likely that stone columns would be installed by air injection or water jetting to advance the stone column probe past localized dense layers in the ground.

Ground improvement with the stone column installation is a standard technique employed to densify soils and mitigate liquefaction potential under seismic conditions. Available installation methods range widely, but a common feature is that liquefiable soils are displaced under pressure and a dense stone (gravel to cobble) mixture fills the space vacated by the soil.

Information is limited on specifics of stone column design and methodology for construction at the proposed Elliot Point 2 Alternative. The use of stone columns for soil improvement on the upland adjacent to or in-water within the footprint of marine structures is increasing in Puget Sound. The technique has been used for several projects at the Port of Tacoma and for replacement of the Seattle US Coast Guard Pier 36 Berth Alpha. The effects of stone column installation generally consist of air releases at the bottom of the water body near the stone column installation site. The air releases can cause temporary, short term, localized turbidity increases.

It is assumed that possible effects on water quality at the project site may occur through re-suspension of bottom sediment due to positive pore pressure at the bottom during air injection or water jet injection associated with placement of the stone column materials. Therefore, the method of analysis includes generating a source of suspended sediment in the lower water column and evaluating dispersion (mixing zone) of this source in the ambient water body. The mechanism of turbidity generation and dispersion due to stone column installation is schematically shown in Figure 8-8.

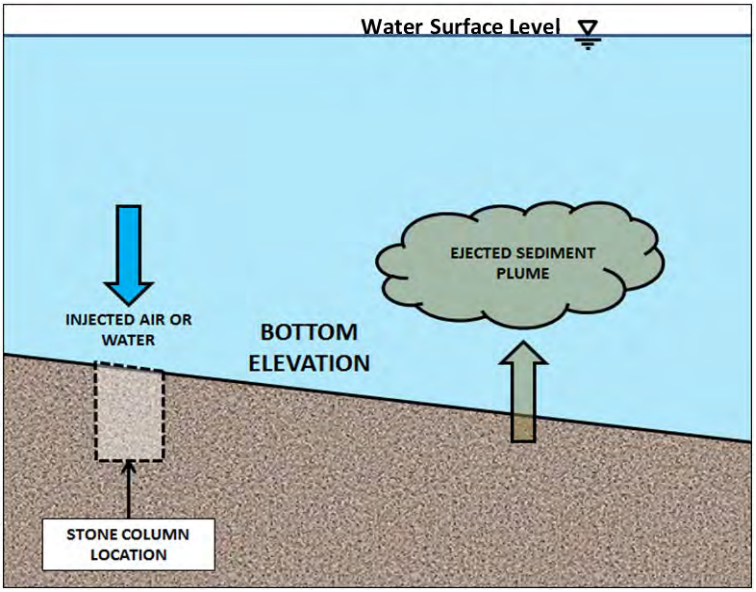


Figure 8-8. Mechanism of turbidity generation and dispersion due to stone column installation

The source of suspended sediment is simulated as a discharge pipeline located at the bottom that releases suspended sediment concentration into the water column. The flow or slurry discharge, velocity at the discharge point, and concentration of suspended sediment would depend on a number of variable factors and are not accurately defined at this stage.

It is assumed that the velocity of discharge is less than the initiation of motion for bottom sediment motion with D_{50} equal 0.3 mm (otherwise, a scour hole would form at the bottom[13]), and that the concentration of suspended sediment is 10 percent of flow discharge. The mixing zone for this scenario was computed using the DREDGE model.

The DREDGE model, the same as was used for analysis of dredging impact on water quality (See Section 8.3.1), was used to determine the mixing zone for turbidity generated by construction of the stone column. The model has a Hydraulic Pipeline module that allows for the computation of suspended sediment discharged by a hydraulic pipeline. The DREDGE model and Hydraulic Pipeline module are appropriate for simulating turbidity spread by stone column installation because the activity generates a flow of suspended sediment from a specified area that can be idealized as the discharge end of a submerged pipe. DREDGE modeling was conducted using the following input data and assumptions:

- Hydraulic discharge pipeline diameter of approximately 1 ft[14].
- Discharge water depths ranging from approximately 10 ft to 33 ft.
- Ambient water velocities ranging from 0.02 to 0.085 ft/sec (See Section 8.2.2 and 8.3.1). As discussed in Section 8.3.1, higher ambient velocities correspond to larger mixing zones, therefore representing more conservative conditions. Smaller ambient velocities correspond to smaller mixing zones.
- Sediment was assumed to be primarily fine sands and silt. Simulations were conducted with various sediment sizes from 0.3 mm (D_{50}) to 0.05 mm (D_{10}).
- In-situ dry density for the sediment was assumed to be 700 kg/m³, as per DREDGE model manual recommendations.
- The sediment re-suspension rate or Turbidity Generation Unit (TGU), defined within the DREDGE manual as the rate sediments are re-suspended into the water column and carried away from the dredging site, was determined to range between 15,000 and 25,000 g/m³ (Nakai, 1978).

[13] A scour hole (if it would occur) would decrease the effectiveness of the stone column support. In other words, it is assumed that the velocity of discharge must be low enough to not create the scour hole.

[14] Sensitivity analysis with different modeling scenarios showed that the diameter of the pipeline does not influence the size of the mixing zone. Increasing (or reducing) the diameter of the pipeline results in reducing (or increasing) flow velocities and/or flow discharge. A pipeline with a diameter of 1 ft was selected in the analysis for simplicity of calculations.

- Dispersion coefficients were set at 100,000 cm^2/sec (107.63 ft^2/sec) in the lateral direction and 10 cm^2/sec (0.01 ft^2/sec) in the vertical direction (DREDGE manual recommendations).

Using the above input parameters, a total of six DREDGE numerical modeling scenarios were completed. Examples of modeling results (worst-case scenario: depth-averaged current velocity = 0.085 ft/sec, water depth = 9.8 ft, sediment size = 0.05 mm, TGU = 25,000 g/m^3) are presented graphically in Figures 8-9 and 8-10.

Figure 8-9 shows a plan view of turbidity due to suspended sediments discharged during stone column installation. The figure shows a pattern of dispersion of turbidity and a decrease of NTU with increasing longitudinal distance away from column installation operations. The figure also plots the boundary of a compliance radius distance from the dredging site which equals 150 ft. Analysis of modeling results indicates there is a slight increase of turbidity above the 5 NTU limit at the 150-ft compliance distance. The actual turbidity value at the 150-ft compliance distance computed by the model is approximately 6.8 NTUs over background. Please note that 1.8 NTU exceedance (modeling results = 6.8 NTUs; required criteria = 5 NTUs) may not be detected visually from Figure 8-9 due to the applied color scale. This exceedance is more detectable in Figure 8-10, which shows a longitudinal section through the middle of the modeling domain (lateral distance 0 ft from the source centerline). Figure 8-10 shows a decrease of turbidity along this section with increased distance from the stone column installation location. The figure shows that turbidity would meet the criteria of 5 NTUs above background, at a distance of approximately 177 ft away from the stone column installation location, or that the mixing zone is approximately 27 ft longer than that specified by *401 Water Quality Certification* and *WAC 173-201A-210*.

Several factors should be taken into consideration when evaluating the significance of the numerical modeling results. One factor that needs to be accounted for in the evaluation of ambient turbidity exceedance is the DREDGE model accuracy. The model's accuracy for predicting dispersion of plumes during stone column installation has a practical limit. Based on modeling experience it is reasonable to expect a maximum accuracy of 20 to 30 ft for establishing the location of a low concentration. In other words, where the model shows compliance at 150 ft, in reality, it may be 120 ft or 180 ft.

A second factor is the conservative approach in selection of the input parameters for the modeling. As discussed above for analysis of turbidity during stone column installation operations, the DREDGE model was run using conservative estimates of several input parameters, including sediment size, tidal flow velocity, and Turbidity Generation Unit (TGU). The combination of these parameters used for the modeling have resulted in conservative estimates of turbidity dispersion, allowing more turbid water propagating at longer distance with less dilution. Therefore, the modeling result showing exceedance of 1.8 NTUs above criteria is likely due to an over-conservative approach to the modeling procedure. Considering the above described compounding of conservatism in modeling, exceeding ambient turbidity at

the 150-ft compliance distance by 6.8 NTUs (with accepted criteria = 5 NTUs) is unlikely to occur in reality, and should be considered as insignificant.

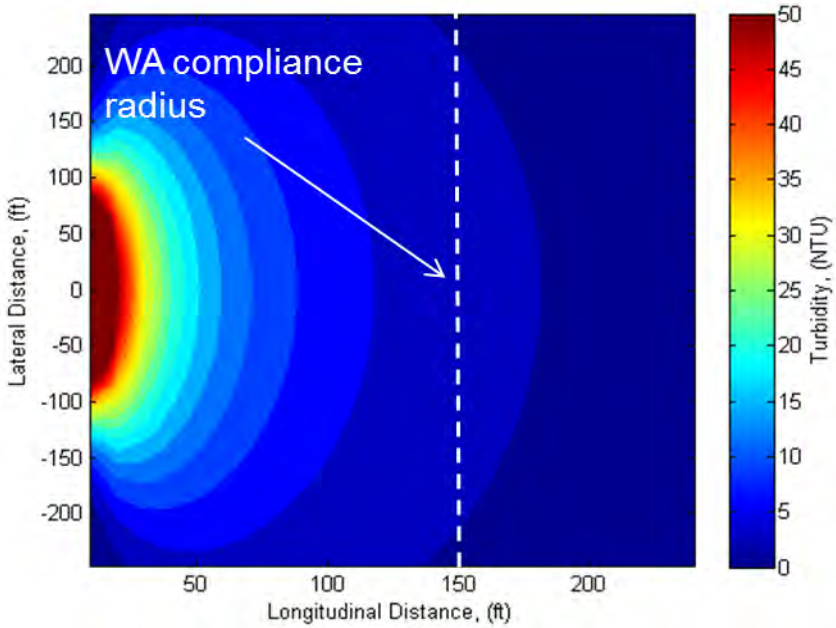


Figure 8-9. Plan view of turbidity above ambient level during stone column installation[15]

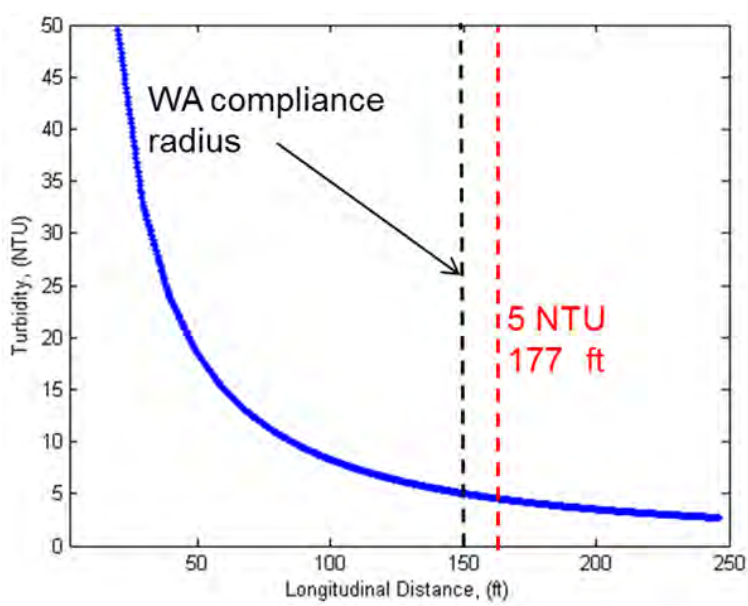


Figure 8-10. Section view through turbidity above ambient level during stone column installation, modeled at plume centerline

[15] Note: The color scale in Figure 8-9 differs from the color scale in Figure 8-6 for clarity purposes.

Therefore, considering all the above, the results of numerical modeling indicate that construction operations pertinent to stone column installation at the Elliot Point 2 Alternative are not expected to violate the existing water quality criteria, established by *401 Water Quality Certification* and *WAC 173-201A-210* for marine water bodies in the State of Washington. To further elimiate any likeihood of impact on water quality during stone column instalation operation, standard BMP measures would be established in combination with the monitoring program (See Section 8.4 below). Based on the modeling results, it appears that no additional measures such as silt curtains would be necessary to control the turbidity inside of the allowed 150-ft radius of the mixing zone, though a final decision may be made upon implementation of the monitoring program. Potential BMP measures to limit any significant impact on water quality during stone column installation operations should include the following:

- Decrease outflow velocity of the water/air jet during column installation if turbidity monitoring indicates a possibility of exceedance over threshold values set by *401 Water Quality Certification and WAC 173-201A-210*.
- Preclude spill of stone column fill material (gravel and small stones) outside of the filling area.
- If the measures described above do not eliminate exceedance of turbidity outside the 150-ft radius, BMPs will include installation and maintenance of silt curtains. A silt curtain weighted at the bottom and suspended from a floating boom is a practical means of controlling the dispersion of suspended sediment in a large range of water depths. Stone column installation would occur in a relatively concentrated work area. The ambient current speed is low and wave height that would interfere with silt curtain effectiveness is rare. Anchors would stabilize silt curtain position so that the bottom of the curtain would always be in contact with the seabed in this tidal range.
- Other appropriate BMPs may need to be developed as more information is developed about stone column location and installation method.

8.4. Water Quality Sampling and Monitoring Plan

Washington State Department of Transportation (WSDOT) will prepare and implement a Water Quality Protection and Monitoring Plan (WQPMP) in accordance with the Washington State Department of Ecology (Ecology) for this location pertaining to dredging, pier removal, and stone column installation work. This plan shall be approved by Ecology and work shall not begin until obtaining Ecology approval. The detailed WQPMP will be prepared during the final design phase of the project. It is understood that the Plan will also be reviewed by EPA. This plan will include and specify the sections as follows:

- Parameter to be Sampled: Turbidity (or suspended sediment concentration) during dredging and stone pile installation.
- Location of Samples: Background and point of compliance samples will be collected at the same stations to be defined by the WQPMP. The point of

compliance samples shall be collected at the boundaries and outside of the established mixing zone. No samples should be collected inside of the mixing zone boundaries, unless WSDOT wishes to know the source of existing turbidity that is within the mixing zone. In that case, a turbidity meter can be deployed from a small boat to collect data that would identify turbidity sources. All samples shall be collected at the same frequency as the point of compliance samples.

  - At a minimum, turbidity sampling and documentation shall occur at 150 ft and 300 ft away from in-water construction activity.

- Sampling Frequency: Water quality measurements for turbidity shall be collected 30 minutes prior to start of work, then every four hours initially during the in-water construction activities and stone column installation. If results show consistent non-exceedance of standards during construction in worst-case conditions (e.g., high currents), frequency of turbidity measurements will be reduced to every eight hours during the time of in-water work.
- Equipment: Sampling for turbidity shall be accomplished using a turbidimeter properly calibrated according to the operator's manual.
- Detection of Exceedances: Water quality standards for turbidity are as follows:
  - Turbidity shall not exceed 5 NTUs over background turbidity when the background turbidity is 50 NTUs or less, or more than a ten (10) percent increase in turbidity when the background turbidity is more than 50 NTUs, at the point of compliance when a turbidimeter is used.
- Activities during Exceedance: WSDOT and contractors shall implement 1-2.2K Responsibility for Environmental Compliance During Construction (*WSDOT Construction Manual M 41-01.13, July 2012).* If exceedances of these standards are detected at the point of compliance, as specified in WAC 173-201A-210, work shall cease immediately and the WSDOT or contractor shall assess the cause of the water quality problem and take immediate action to stop, contain, and correct the problem and/or prevent further water quality exceedances. After such an event, WSDOT shall assess the efficacy of the site BMPs, and update or improve the BMPs used at the work site in an effort to prevent a recurrence of the exceedance. If an exceedance occurs, WSDOT or the Contractor shall follow the protocols and notification procedures below:
- Notification of Exceedances: Notification of exceedances that are detected through water quality sampling shall be made to Ecology within 24 hours of occurrence. Notification shall be made with reference to the project order number. WSDOT shall, at a minimum, provide Ecology with the following information:
  - A description of the nature, extent, and cause of the exceedance.
  - The period of non-compliance, including exact dates, duration, and times and/or the anticipated time when the project will return to compliance.
  - The steps taken, or to be taken, to reduce, eliminate, and prevent recurrence of the non-compliance.

- In addition, within five (5) days after notification of an exceedance, WSDOT (Applicant) shall submit a written report to Ecology that describes the nature of the exceedance, water quality monitoring results and location, photographs, and any other pertinent information.

- Debris Monitoring/Management: Debris will be visually monitored during pier removal as stated in Section 8.3.2. A management plan shall be submitted by the Contractor before starting work, and adjustments to techniques or institution of BMPs will be made if debris causes exceedance of water quality limits or appears to interfere with proper operation of dredging equipment.
- Reporting: Results of the water quality monitoring shall be documented in a report and submitted to Ecology weekly during project construction. The reports must include:
  - Date and time the sample is collected.
  - Sample location.
  - Sample results.
  - Name of person collecting sample.
  - Tide and current conditions.
  - Weather conditions.

- Possible Avoidance: If, after the first four (4) weeks of instrumented water quality monitoring of the project activities, no turbidity exceedances are detected, WSDOT may submit a request to Ecology to convert from physical to visual monitoring. Ecology will review the data and provide a written response to the request. Additionally, Ecology must approve, in writing, any changes to the final WQPMP. Mitigation and/or additional monitoring may be required if water quality standards are not met.

8.5. Section Summary

The analysis shows that dredging, pile removal, and stone column installation at the project site are not likely to generate turbidity levels of concern, according to the criteria described in *401 Water Quality Certification* and *WAC 173-201A-210* for marine water bodies in the State of Washington.

As BMPs, it is recommended that background water turbidity conditions be measured directly prior to initiation of any type of in-water construction, including dredging, pile removal, and stone column installation. The reference stations for turbidity measurements should be established at 150 ft from the project site. The location of reference stations must not be affected by the in-water work, and must have similar hydrodynamic / morphological characteristics as the project site.

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