Elim Subsistence Harbor Feasibility Study

Appendix C: Hydraulics & Hydrology

Elim, Alaska

November 2020

U.S. Army Corps of Engineers
Alaska District

This page was intentionally left blank.

Preface

The Tentatively Selected Plan (TSP) design quantities and wave reduction/moorage analysis assumed the offshore extent of the breakwaters terminated at the -7 foot (ft) Mean Lower Low Water (MLLW) contour. After the TSP decision milestone, 16 December 2019, it was determined that the offshore extent of the breakwaters would need to be to the -8-ft MLLW contour to decrease the percent of the time that breaking waves are anticipated at the breakwaters nose. The change would apply to the four alternatives at Elim Beach equally and would not affect plan formulation. The TSP received an endorsement from the HQUSACE Chief of Planning and Policy Division to become the Recommended Plan on 09 July 2020. The Recommended Plan was optimized before the release of the final report to reflect the change in the offshore extent of the breakwaters and design adjustments made due to comments received during the District Quality Control, Agency Technical Review, and Policy and Legal Review. Changes to the design based on the review comments were only applied to the Recommended Plan since reformulation was not necessary. Section 10 of this Appendix presents the designs used for screening of alternatives and Section 11 of this Appendix presents the optimized design.

This page was intentionally left blank.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

This page was intentionally left blank.

1. INTRODUCTION

This hydraulics and hydrology appendix describes the technical aspects of the Elim, Alaska Subsistence Harbor Feasibility Study. It provides the background for determining the Federal interest in the construction of navigation improvements at Elim, Alaska. To determine the feasibility of a project, hindcast data offshore of Elim was used to determine wave conditions at the alternative project locations. The Storm-Induced Water Level Prediction Study for the Western Coast of Alaska (Chapman & Mark 2014) for Golovin was used as an estimate of the storm surge at the alternative project locations.

1.1. Project Purpose and Need

The purpose of the project is to improve the safe accessibility of marine navigation to the community of Elim, Alaska. The need for the project is to reduce hazards in order to provide safer navigation for subsistence vessels, fuel barges, cargo vessels, and a limited commercial fleet, all of which are critical to the long term viability of the community and the mixed subsistence-cash economy at Elim.

1.2. Description of Project Area

Elim is a Central Yupik native village on the northern shore of Norton Sound on the Seward Peninsula (Figure 1). It is located 460 air miles northwest of Anchorage at 64º37’ N, 162º15’ W. (Sec. 15, T010S, R018W, Kateel River Meridian) and is 96 miles east of Nome.

Figure 1. State of Alaska Location Map

Elim is located along the northern shore of Norton Bay (Figure 2) on a pocket beach (Figure 3), characterized by a low-lying shoreline that runs east-west with high, rocky headlands at either end (Figure 4 and Figure 5). Surface material at the pocket beach consists of coarse-grained soils varying in size from sand to gravel with some cobbles (Alaska District 2018). Elim Creek divides the village near the sewage outfall line. There is no harbor or dock in Elim.

Elim is accessible year-round by regularly scheduled flights from Nome to the State-owned gravel airstrip located at the west end of the village (Figure 3). Cargo and fuel are transported to Elim on small regional barges based out of Nome in the summer after the ice is out. The cargo barges wait offshore of Elim for high tide to land on the beach and offload goods and materials with heavy equipment (Figure 5). Residents have reported cargo barges getting stuck on the beach due to low water. The fuel barge anchors offshore of Elim, and a floating pipeline is run from the barge to the fuel header and pumped to the fuel tanks near the airport. In the winter, cargo and fuel can be transported to Elim by plane, which is more costly. Transportation between Elim and the subsistence areas outside of Elim is by boat in the summer and snowmachine in the winter.

Commercial and subsistence vessels currently use the unimproved Kwiniuk River for safe moorage at Moses Point (Figure 6), which is approximately 9 miles east of Elim by road (Figure 2). Access to Kwiniuk River is dependent on the wave and water level conditions. Residents have reported increased shoaling at the mouth of the Kwiniuk River (Figure 7), making access to the safe moorage less predictable. During large storm surges, residents pull their boats higher onshore. If there is a short notice of a storm, vessels can get swamped due to the distance between Elim and Moses Point and the limited number of residents with trucks and trailers.

The Norton Sound Economic Development Corporation (NSEDC) operates a portable fish-buying station (Figure 8) out of Moses Point. Residents sell whole fish to NSEDC, which then packs the fish into iced fish totes and lighters the fish totes to an anchored tender offshore. There are no utilities at Moses Point, and the only ice machine is in Elim. Ice has to be transported from Elim to Moses Point for NSEDC to buy fish.

Figure 2. Google Earth Image of Norton Sound and Norton Bay

Figure 3. Elim, Alaska with Select Project Points of Interest

Figure 4. Elim Shoreline Looking West at Airport Point (August 2018)

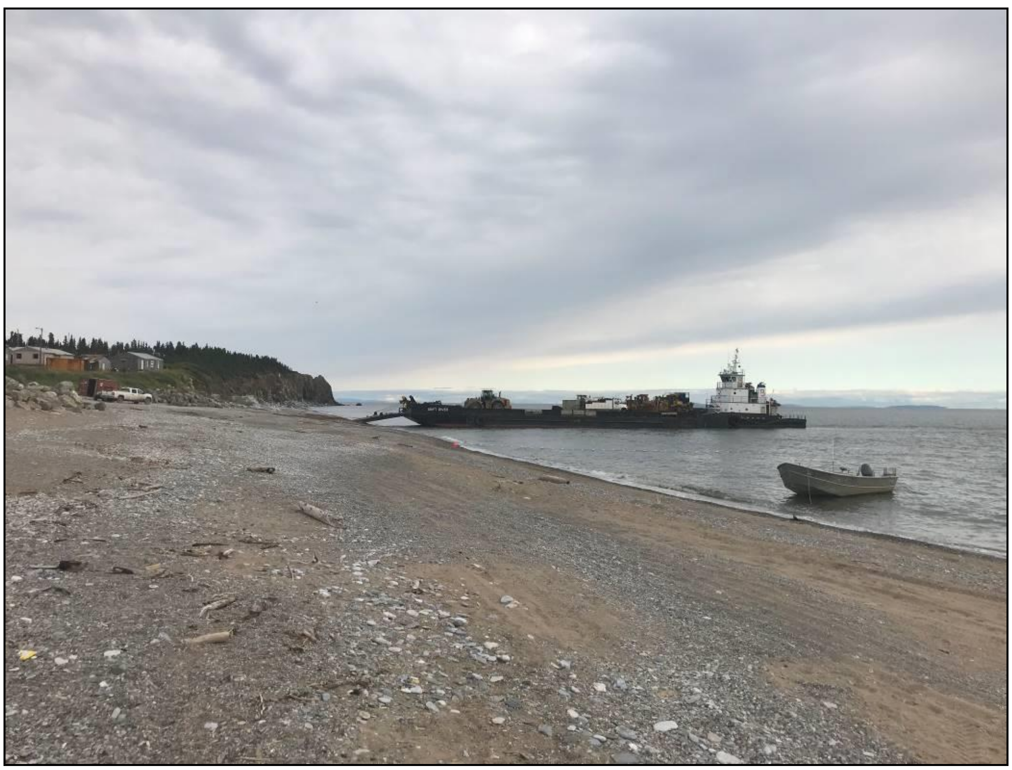


Figure 5. Elim Shoreline Looking East with a Skiff Anchored Onshore and a Materials Barge Unloading (August 2018)

Figure 6. Moses Point, Alaska

Figure 7. The Mouth of Kwiniuk River at Moses Point (August 2018)

Figure 8. NSEDC Fish Buying Station and Safe Moorage up Kwiniuk River at Moses Point (August 2018)

2. STUDY CONSTRAINTS

During the feasibility study, constraints were identified, including the following:

- Avoid or minimize impacts to existing commercial and subsistence fisheries;
- Avoid or minimize impacts to historic sites and/or sites of cultural importance;
- Avoid or minimize impacts to critical infrastructure including the airport, access roads, fuel header, and tank farm; and
- Avoid or minimize impacts to environmental resources and environmental quality.

3. CLIMATOLOGY, METEOROLOGY, AND HYDROLOGY

3.1. Temperature

Elim falls within the transitional climate zone in the sub-Arctic, characterized by tundra interspersed with boreal forests, and weather patterns of long, cold winters and short, warm summers. Elim is generally ice-free between late May and mid-November. Summers are cold and rainy with average temperatures between 40°F and 60°F; winters are cold and dry with average temperatures between -5°F and 15°F (Figure 9). Figure 9 shows the daily average high (red line) and low (blue line) temperature, with 25th–75th and 10th–90th percentile bands. The thin dotted lines are the corresponding average perceived temperatures (Cedar Lake Ventures, Inc., n.d.). Average daily summer temperatures vary slightly due to maritime influence. Total average annual precipitation (rain and melted snow water) is 16.1 inches, with 60 inches of snow (HDR 2012).

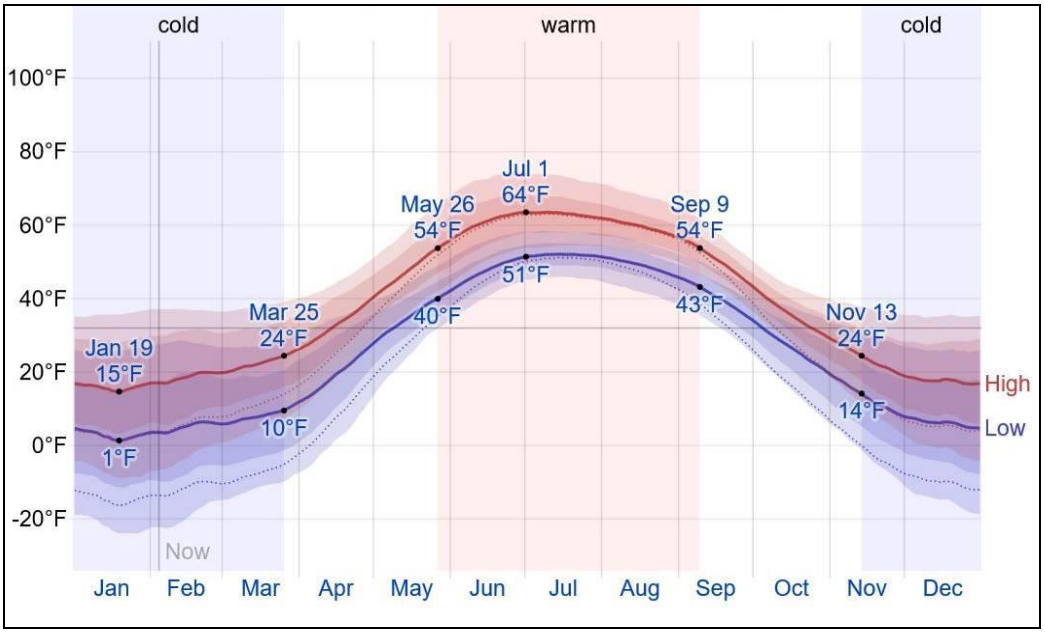


Figure 9. Average Temperature at Elim

3.2. Ice Conditions

Ice conditions within the project area include sea ice and shorefast ice. For Norton Sound, sea ice formation typically occurs in mid-November each year; however, there have been years in which freeze-up in Norton Sound took place in mid-October or as late as December. Spring break-up typically occurs in late May to mid-June. Shorefast ice is sea ice of any origin that remains attached to shoreline features or is grounded on the seabed along the coast and, as a result, does not drift with currents and wind. Based on observations in Nome (Ettema & Kennedy 1982), shorefast ice typically extends out from 0.5 miles to approximately 7 miles depending on seasonal conditions. Nearshore, the ice tends to be relatively smooth to about 0.25 miles. From there, the ice buckles offshore where the influence of pressure ridges are evident. Early winter ice sheet thicknesses of approximately 1 foot (ft) are common. During years where pressure ridges are formed, estimated ice thicknesses at the ridges have been as high as 30 ft.

3.3. Tide

Elim has semi-diurnal tides with two high waters and two low waters each lunar day. The tidal parameters in Table 1 were determined using the National Ocean and Atmospheric Administration (NOAA) Tidal Benchmarks at Elim, Norton Bay (Station ID 9468863). The tidal datum was determined over a 1-month period in September 2012 based on the 1983–2001 tidal epoch. The highest and lowest water level observations were not reported. They could be much higher and lower than the determined Mean Higher High Water (MHHW) and Mean Lower Low Water (MLLW), due to storm surge and/or isostatic (inverted barometer) effects.

Table 1. Elim Tidal Datum Elevations Relative to MLLW (NOAA Tidal Benchmark 9468863 - NOAA 2020)

Parameter	Elevation [ft]
Mean Higher High Water (MHHW)	2.62
Mean Sea Level (MSL)	1.45
Mean Tide Level (MTL)	1.28
Mean Lower Low Water (MLLW)	0.00

3.4. Currents

Measured current data is not available for Norton Sound offshore of Elim. In the summers of 2018 and 2019, Alaska Ocean Observing System deployed a Waverider Buoy to collect ocean current data off the coast of Nome in a water depth of 59.7 ft (AOOS 2019). It is assumed that current data at Nome is an appropriate estimate of current speeds at Elim. Average current velocities are in the range of 0.5 knots (Figure 10), with a maximum observed current speed of 2.3 knots, with a predominant direction from the west (Figure 11). The predominant current direction at Nome is from the west. It is assumed that the predominant current direction at Elim is from the southwest due to the sheltering effect of Cape Darby to the west of Elim (Figure 2).

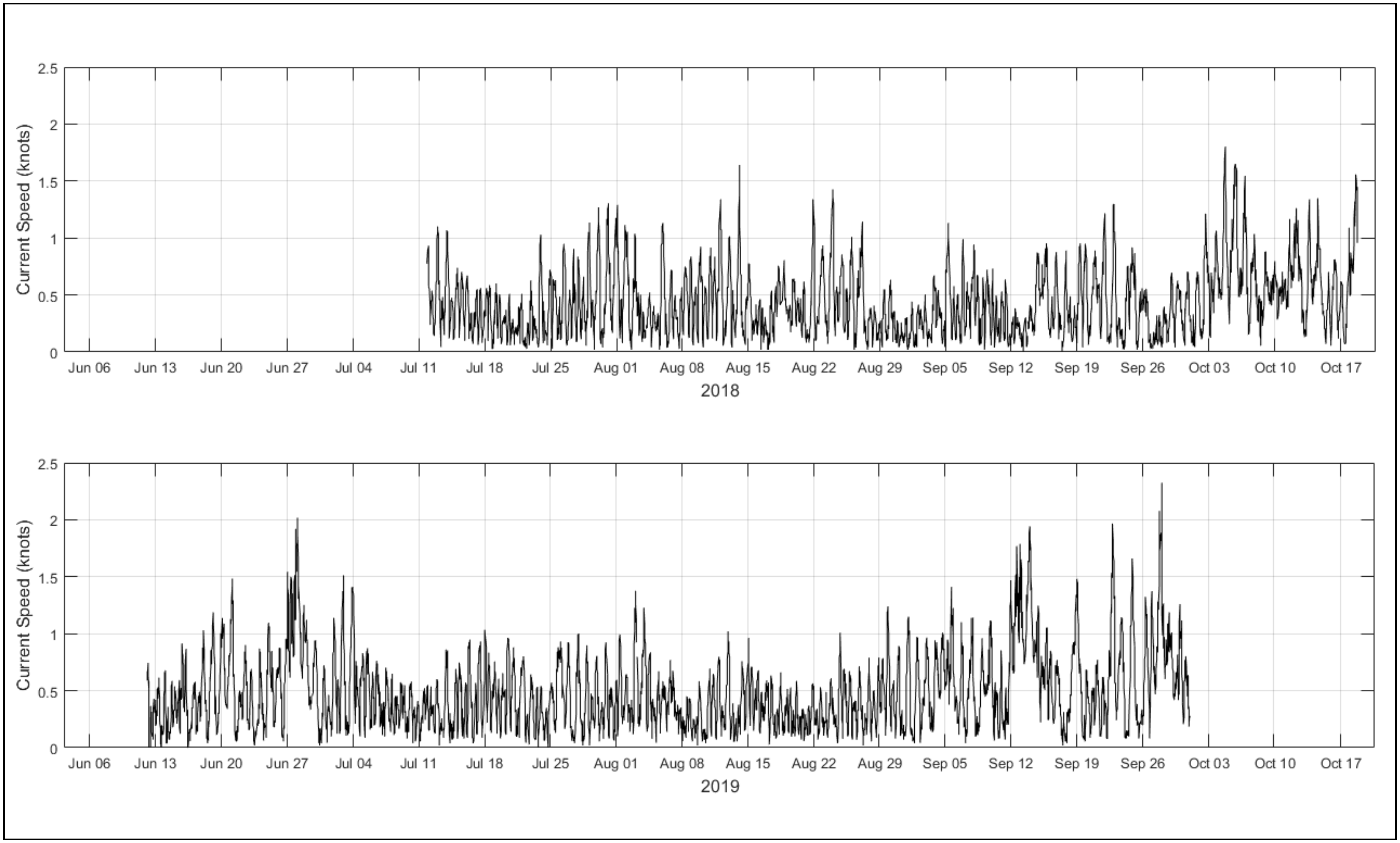


Figure 10. Current Speeds in Knots Offshore Nome During the Summers of 2018 (Top) and 2019 (Bottom)

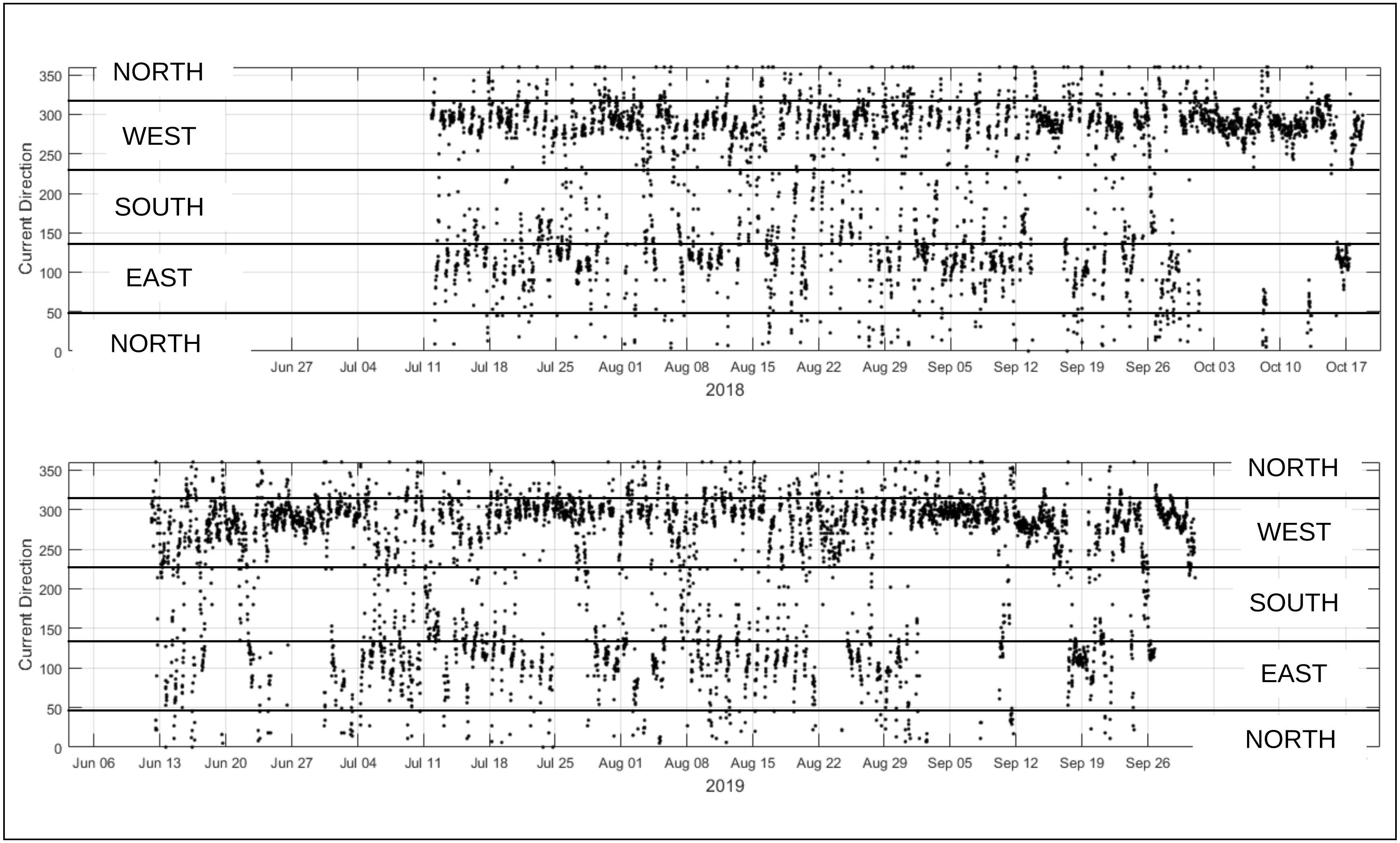


Figure 11. Current Directions Offshore of Nome during the Summers of 2018 (Top) and 2019 (Bottom)

3.5. Wind

Elim Airport hosts a weather station that has been operational since 2012. The average wind speed is approximately 8 mph (7 knots), predominantly from the south-southwest, in the summer (June through August) and approximately 10 mph (8.7 knots), predominantly from the north, in the fall (September through November) (Figure 12).

3.6. Rivers and Creeks in the Project Area

Elim Creek runs south through the center of the town and empties into Norton Bay (Figure 3). Elim Creek is not gaged, and there are no sediment load estimates. It is not anticipated to impact any of the proposed alternatives. The Hazard Impact Assessment for Elim (HDR 2012) states that during snowmelt in spring and heavy rain in the fall, there is elevated creek flow, but that the main flood risk for the community is due to storm surge.

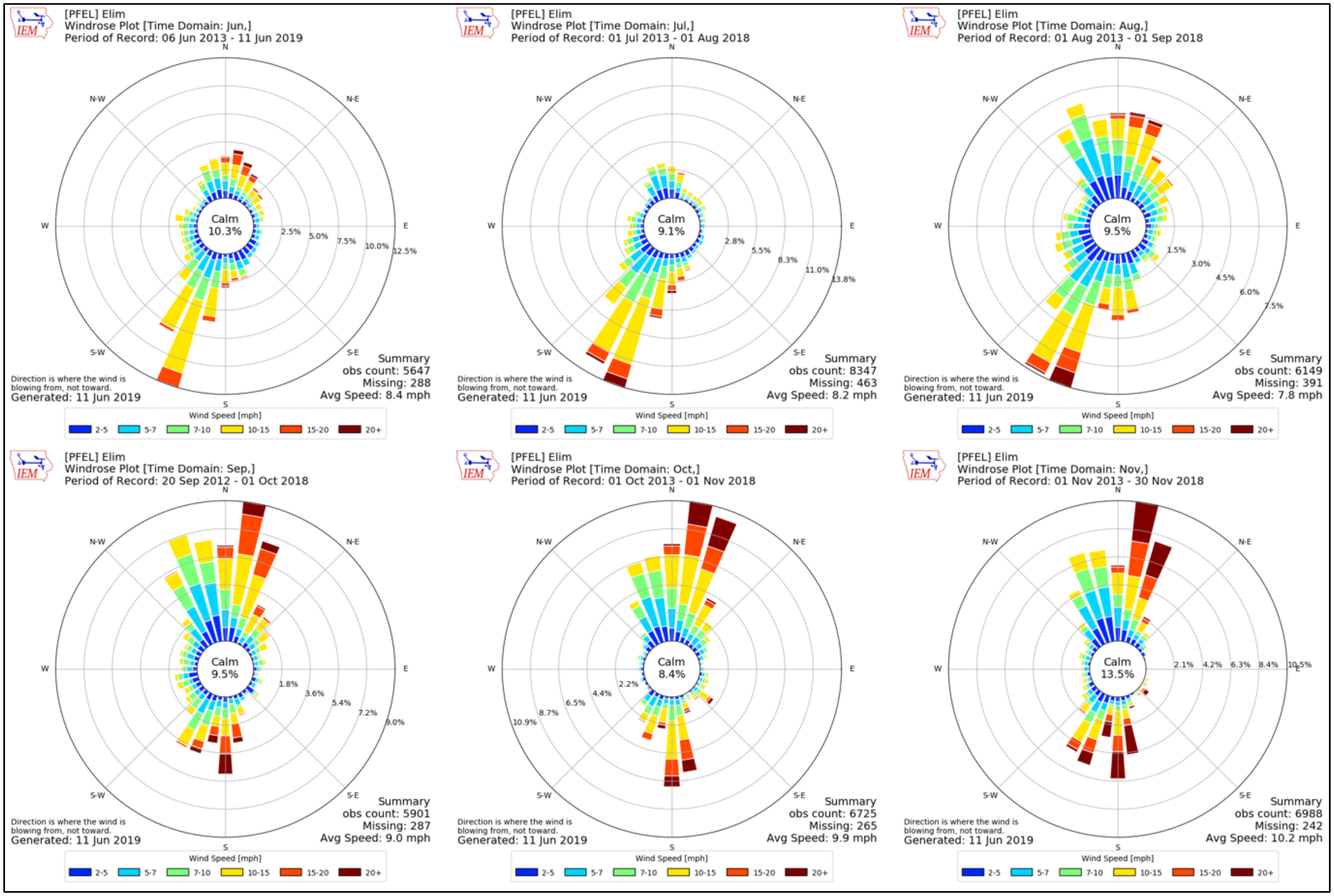


Figure 12. Monthly Wind Roses from Elim Airport for June through November Based on Data from July 2013 to August 2018 (Iowa State University of Science and Technology, n.d.)

4. WAVE ANALYSIS

Elim is protected to the west by Cape Darby, which prevents long-period swell from entering Norton Bay (Figure 14). The wave analysis for navigation improvements at Elim requires an analysis of the deep water waves entering Norton Bay from Norton Sound and fetch-limited waves generated within Norton Bay. Community input indicated that the most destructive waves – spilling and plunging waves – come from the southwest while collapsing and surging waves come from the east (Alaska District 2020). The separation between Norton Sound and Norton Bay is assumed to be the stretch between Cape Darby and Cape Denbigh. Waves impacting Elim from 55°–126° are fetch limited, and waves propagating from 126°–215° are deep water waves (Figure 13). The deep water waves can be characterized using the 30 years of Wave Information Study (WIS) Station 82107 (ST82107) hindcast data from 1985–2014. It is assumed that the 30 years of hindcast conditions are representative of the historic and future conditions impacting Elim.

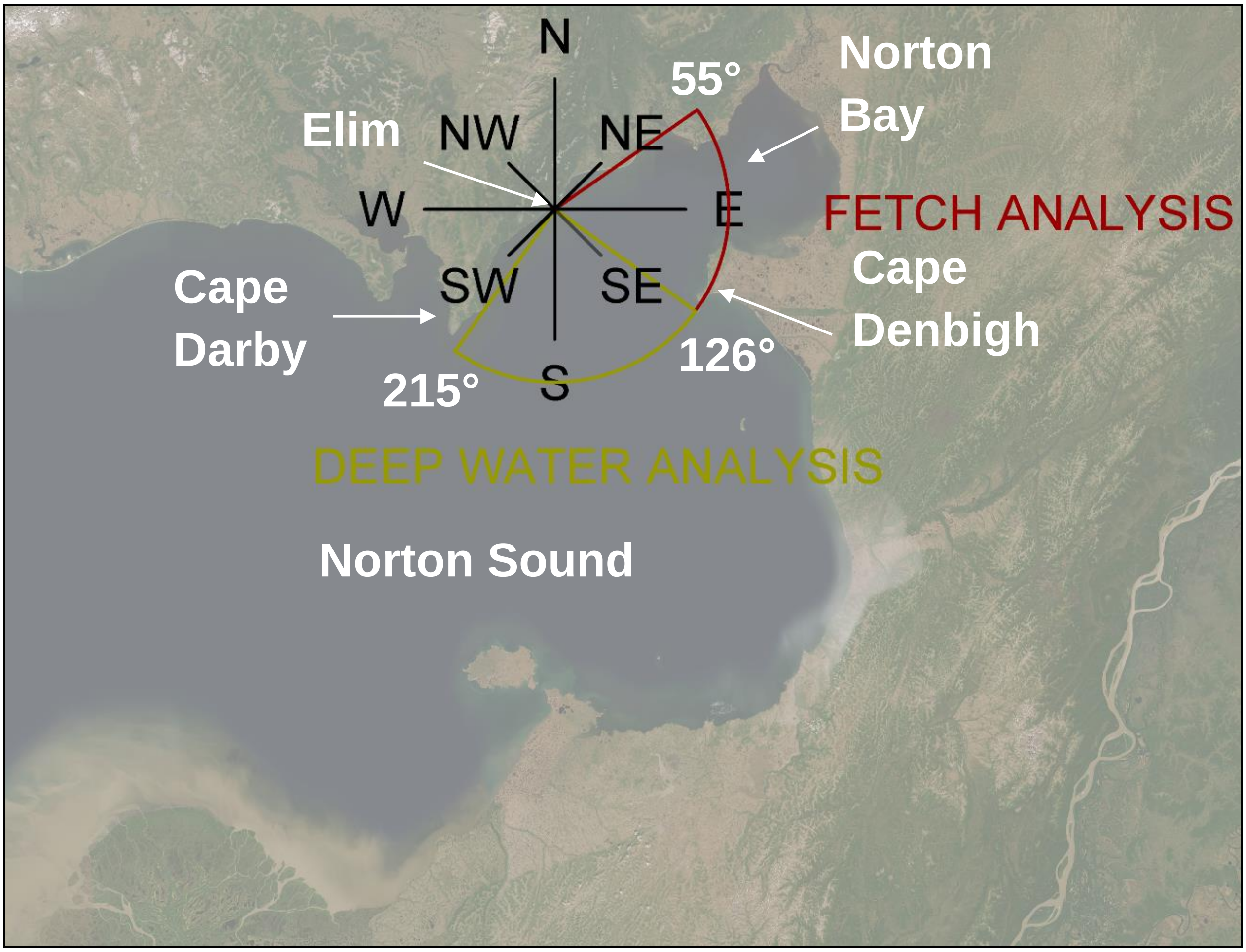
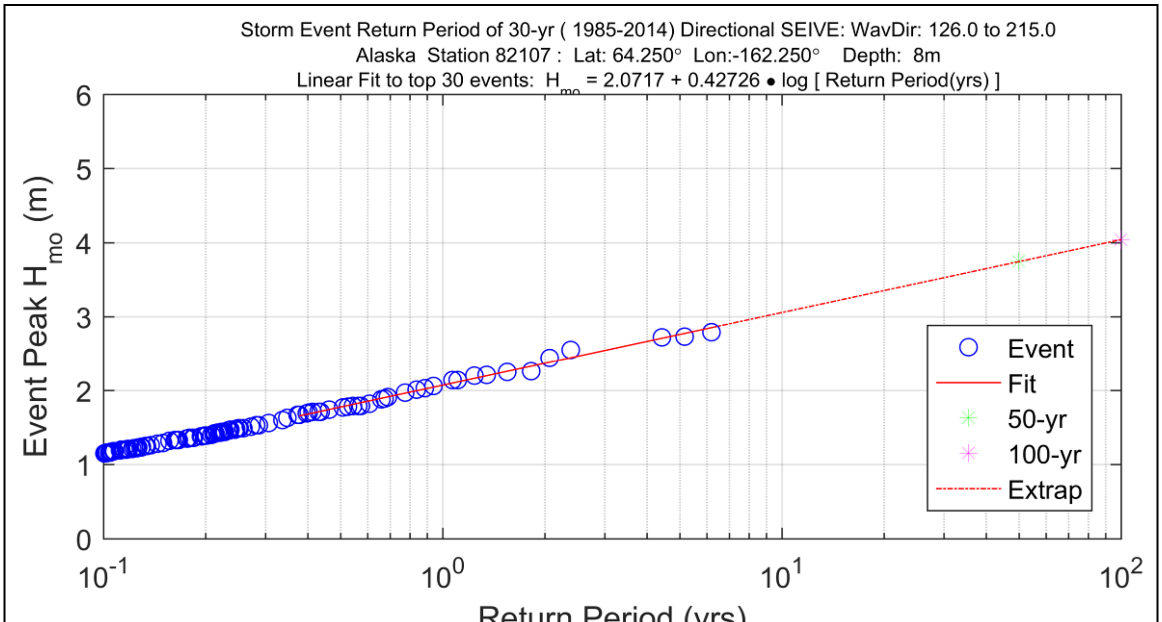


Figure 13. Wave Analysis Method Based on Direction

Figure 14. Google Earth Image of Norton Sound and Norton Bay Showing Location of WIS ST82107 which Is Labored Relative to Elim, Cape Darby, and Moses Point

4.1. Deep Water Waves

A deep water extreme wave analysis was performed on the hindcast wave data at WIS ST82107 to determine the design wave conditions impacting navigation improvements at Elim. To determine the design conditions at Elim, only data for the ice-free periods (assumed to be May 01 to November 31) was used. When the mean monthly ice concentrations for a WIS hindcast grid cell reaches 70% or greater, the model effectively turns that grid cell into the land, causing outputs to contain error values for the entire month. Because the model returns error values when ice concentrations in the area are too high, May and November were included in the analysis, and if the mean monthly ice concentration was 70% or greater, it was automatically removed from the filtered data. The WIS wave data was then filtered for waves propagating from 126°-215° from Norton Sound towards Elim. The extreme wave analysis of the filtered data resulted in a 2% annual exceedance probability (AEP) wave height of 3.7 meters (12.3 ft), with a peak period of 6.0 seconds (Figure 15). Based on the breaking criteria of $(H/L)_{max} = 0.142\tanh(kd)$, this wave height would break in 17 ft of water. The -17.0-ft MLLW contour is offshore of the proposed navigation improvements at Elim Beach and Airport Point (Figure 16).



Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_{pp}	θ_{wave}	Event	Date/Time(UTC)	H_{mo}	T_{pp}	θ_{wave}
5	1985/10/15 07:00	2.79	6.93	156.0	17	2005/09/23 12:00	2.26	5.73	151.0
6	1989/11/16 03:00	2.73	8.39	201.0	20	1992/10/06 13:00	2.25	6.31	162.0
7	1990/09/06 12:00	2.72	6.93	161.0	23	2013/11/14 02:00	2.21	6.30	152.0
13	2003/11/09 01:00	2.55	5.73	145.0	25	2009/11/12 07:00	2.20	7.63	209.0
15	1998/08/19 19:00	2.44	6.93	199.0	28	2002/04/26 19:00	2.14	6.30	206.0

An event is defined as any period when $H_{mo} > 0.98$m θ_{wave} is direction that waves are arriving from

Figure 15. WIS ST82107 Wave Height Extreme Analysis for Wave Directions 126°–215°

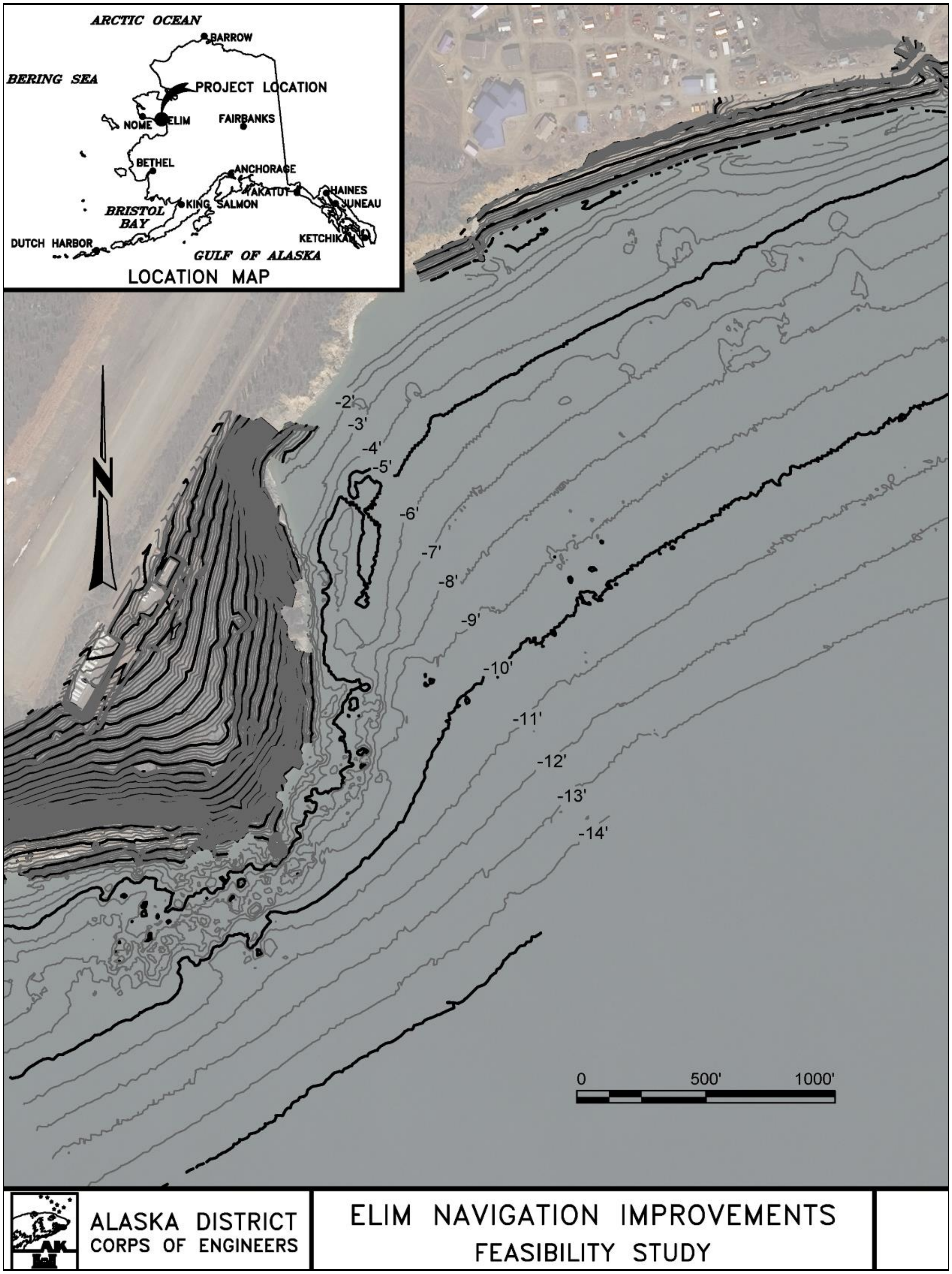


Figure 16. Bathymetry at Airport Point and Elim Beach (Datum is 0 ft MLLW)

Due to the 2% AEP wave breaking before reaching the project site under most water level conditions; an alternative design wave was determined using a duration exceedance analysis of the continuous wave hindcast for the waves traveling towards Elim based on the 30 years of data from WIS ST82107. The duration analysis looked at how often the hindcast wave height was greater than 0.3-meter (about 1 ft) incremental wave heights to create a wave height exceedance curve (Figure 17). A 99% duration exceedance level was used as the design threshold. An analysis of the same wave direction window as the extreme analysis resulted in a design wave of 1.9 meters (6.2 ft) being exceeded by 99% of the hindcast waves propagating from 126° to 215° (Figure 17). A 6.2-ft wave with the same peak period of 6.0 seconds is anticipated to break in a water depth of 8 ft.

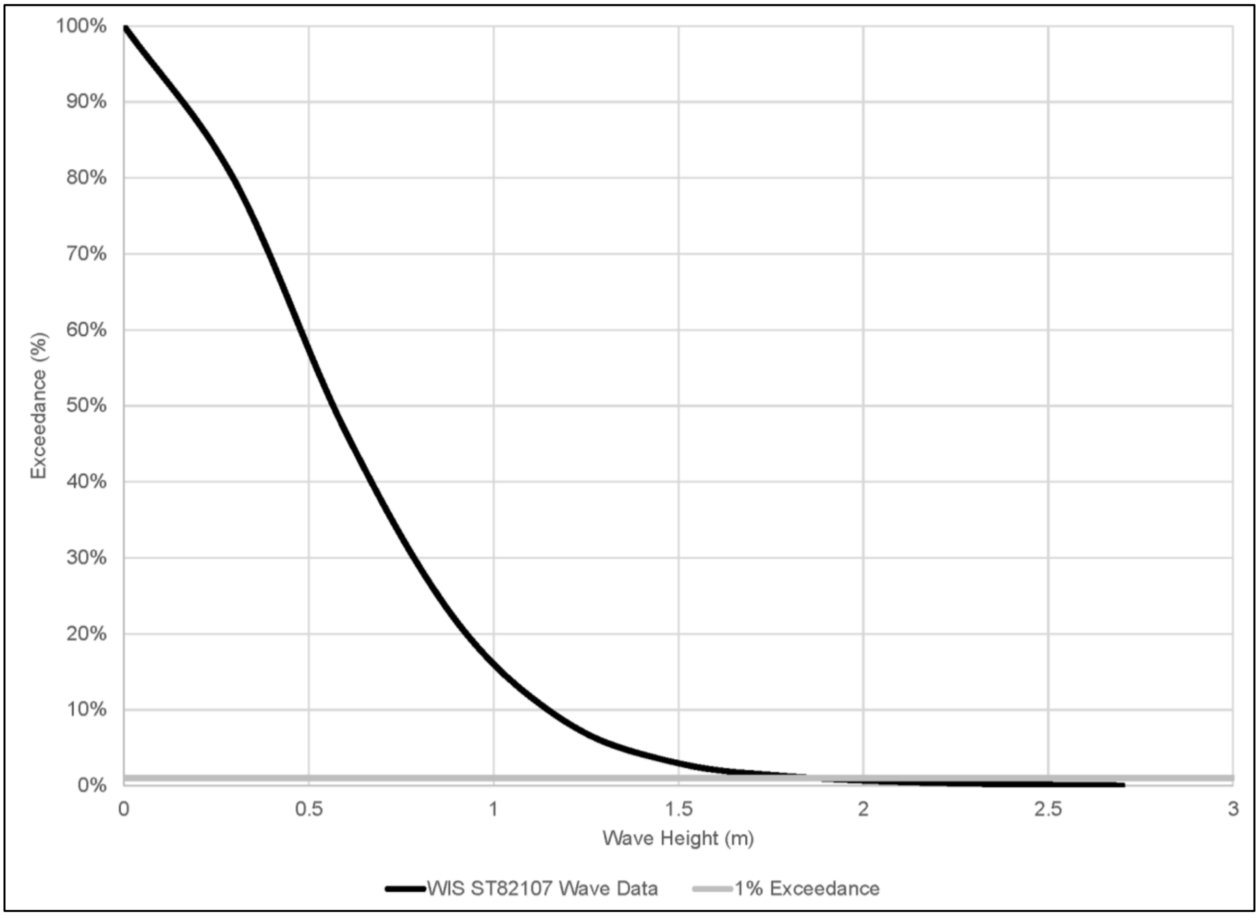


Figure 17. WIS ST82107 Wave Height Relative Duration Exceedance Analysis (filtered for Waves Traveling towards Elim during Ice-free Periods)

4.2. Fetch Limited Waves

Not all waves impacting the proposed navigation improvement project will be deep water waves from Norton Sound. A fetch limited wave analysis was performed to determine what the design wave height would be for waves generated locally within Norton Bay.

Fetches were calculated using the average length of the radial lines at 3° spacing for acres representing each cardinal and ordinal direction. The radial lines used to determine the fetch are shown in Figure 18.

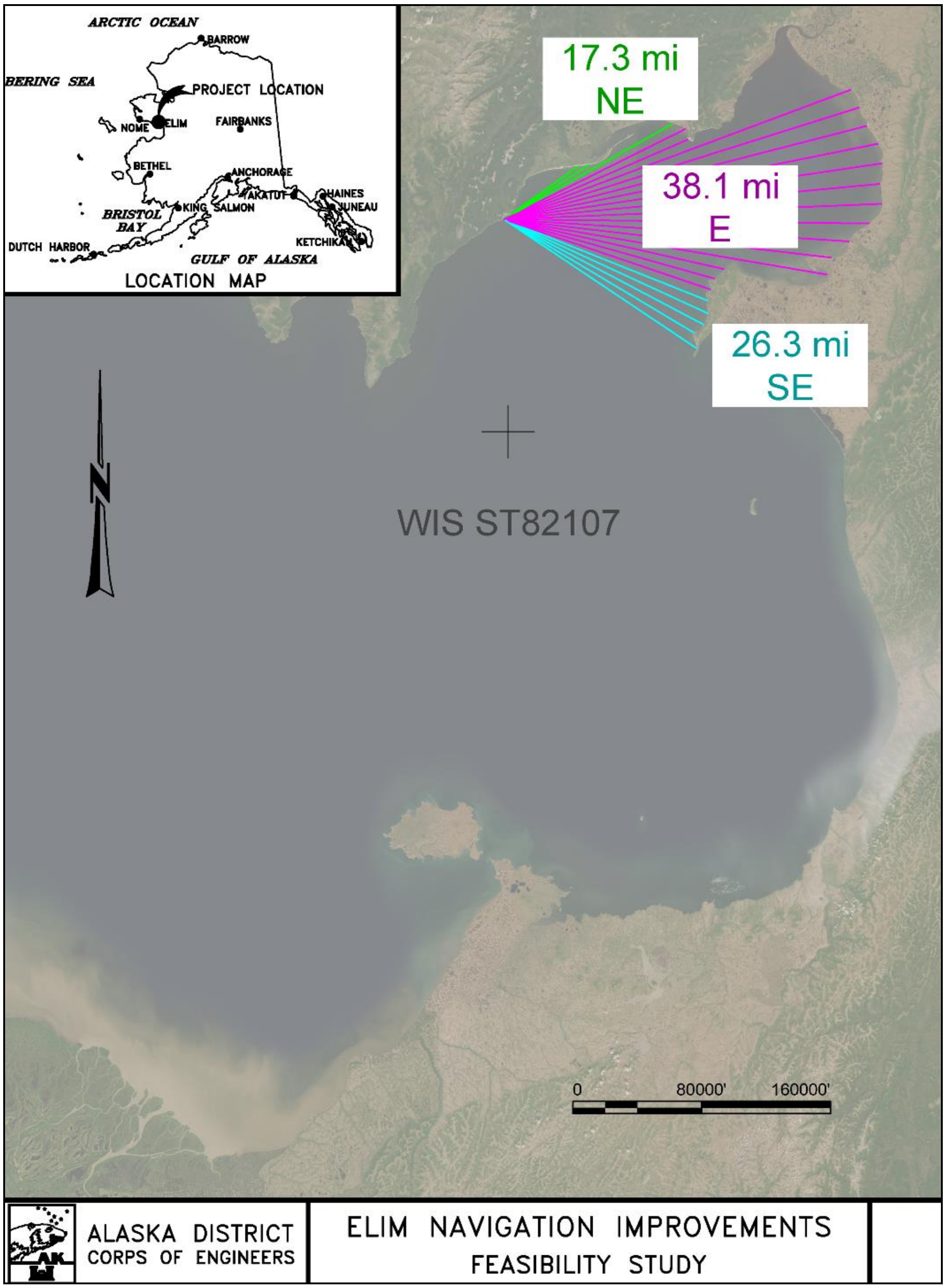


Figure 18. Radial Lines for Fetch Analysis in Norton Bay

Methods described in the Coastal Engineering Manual (CEM), EM 1110-2-1100 (USACE 2002), Part II, Chapter 2 Meteorology and Wave Climate were used to predict wave heights. The CEM equations predict wave heights based on fetch distances and wind speeds. Due to the limited wind data available from Elim Airport (Section 3.1 Wind), the hindcast wind data from WIS ST82107 was used to predict wave heights at the project site. The waves were assumed to be fully-developed for the analysis. A duration exceedance analysis was performed on the hindcast wind data to determine the design wind speed for the fetch analysis. The design wind speed was determined to be 13.1 m/s (25.4 knots) for the wind blowing from 55° to 126° (Figure 19), with 99% of hindcast wind speeds being less than the design speed. This wind speed is lower than the 2% AEP wind speed for winds blowing from 55° to 126° for WIS ST82107 of 23.1 m/s (Figure 20) but was determined to be reasonable with the assumption that 13.1 m/s would create fully-developed waves.

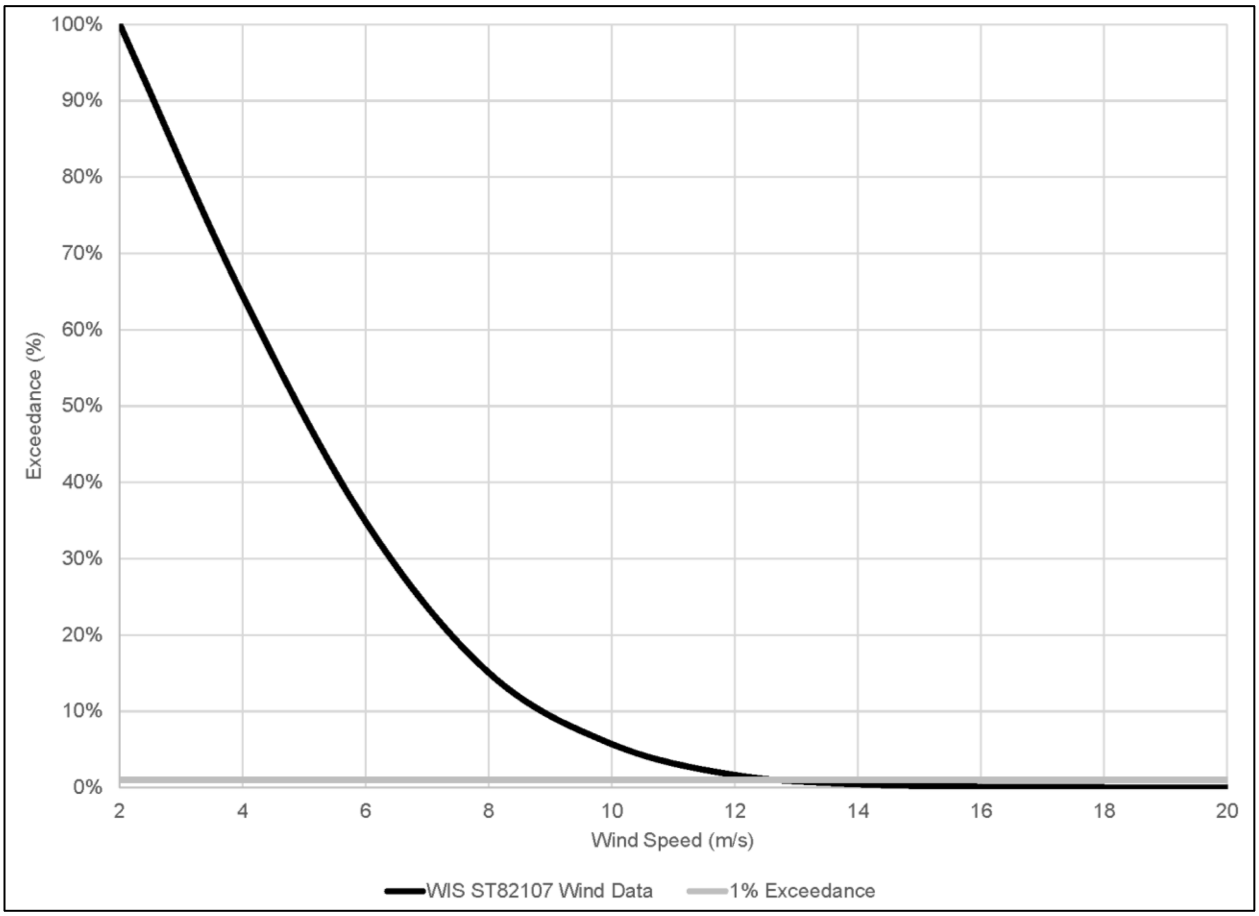


Figure 19. WIS ST82107 Wind Speed Relative Duration Exceedance Analysis (filtered for Winds Blowing towards Elim during Ice-free Periods)

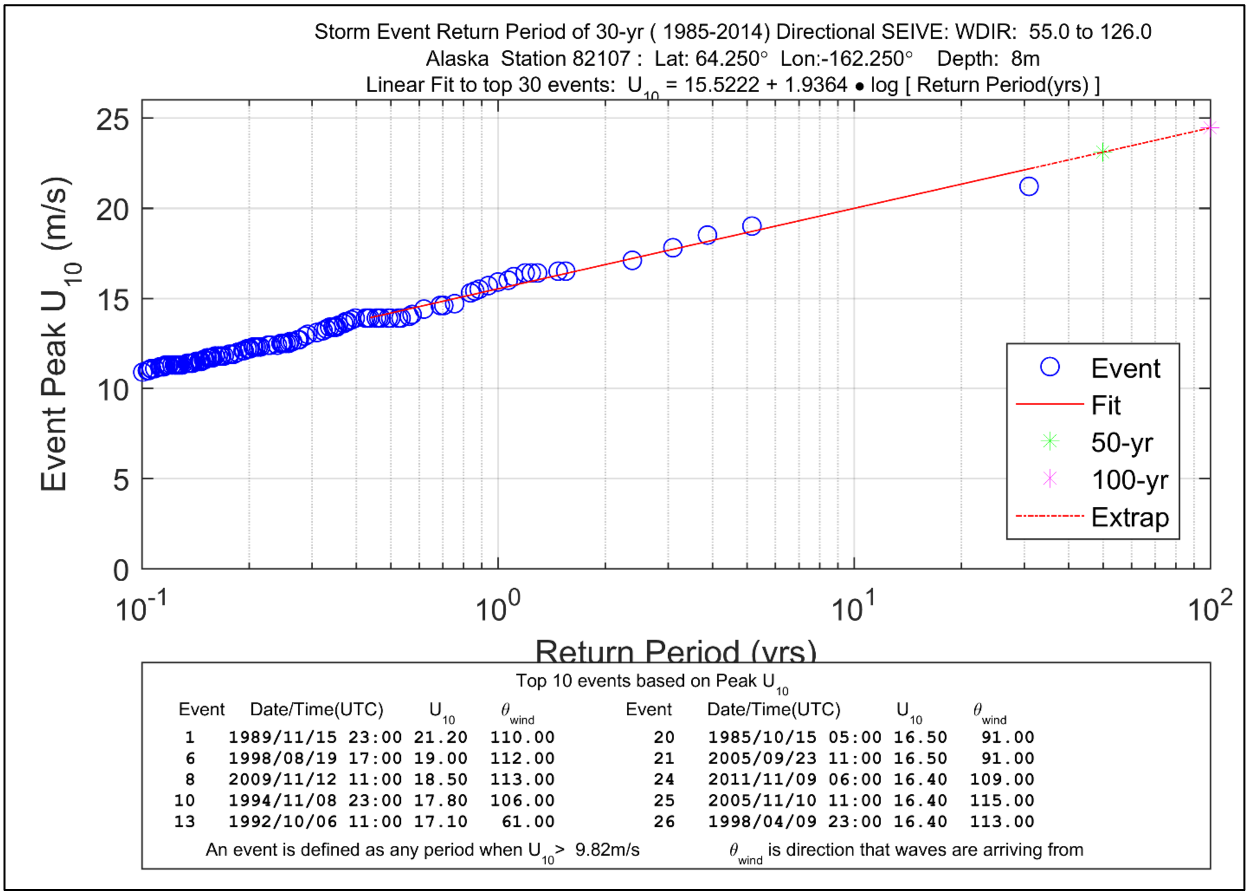


Figure 20. WIS ST82107 Wind Speed Extreme Analysis for Wave Directions 55° to 126°

The design wind speed results in fetch-limited design waves of 2.8 ft with a 3.7 second period from the northeast, 4.5-ft with a 4.9 second period from the east, and 4.2 ft with a 4.4 second period from the southeast. A 4.5-ft wave with a 4.9 second period would break in a water depth of 6 ft.

4.3. Design Condition

For this study, the design wave is the 6.2 ft, 6.0-second deep water wave from the southeast through the southwest. Where appropriate, other wave heights (Table 2) were used to support the analysis of the navigation improvements design.

Table 2. Wave Analysis: Wave Design Conditions

Direction	Wave Height [ft]	Wave Period [s]	Method of Analysis
NE	2.8	3.7	Fetch
E	4.5	4.9	Fetch
SE	4.2	4.4	Fetch
SE-SW	6.2	6.0	Deep Water
SE-SW	12.3	6.0	Extreme

5. WATER LEVEL

The effect of an increase or decrease in total water level needs to be evaluated when designing a navigation project. Water level increase is typically a result of the wave setup, storm surge, and tide. Relative sea level rise is a longer-term increase in water level, and its effects on a project is an additional factor that needs to be considered in a breakwater design. Water level decrease is typically the result of offshore winds and/or high-pressure atmospheric conditions.

5.1. Wave Setup

Wave setup is the superelevation of mean water level caused by wave action, as described in the CEM Part II Chapter 4 Surf Zone Hydrodynamics (Figure 21). Assuming linear wave theory, the maximum lowering of the water level, wave setdown, occurs near the breakpoint (η_b). The nose of the breakwaters terminates farther offshore than the depth of breaking (d_b), the location of maximum setdown, for the design wave height of 6.2 ft. The wave setup at the shoreline (η_s) was included in the total water level estimate to take into account potential wave setup along the structure towards the shore. The wave setup at the shoreline is anticipated by being approximately 0.74 ft for the design wave height based on $\eta_s \approx 0.15d_b$ when the breaking index is assumed to be 0.8.

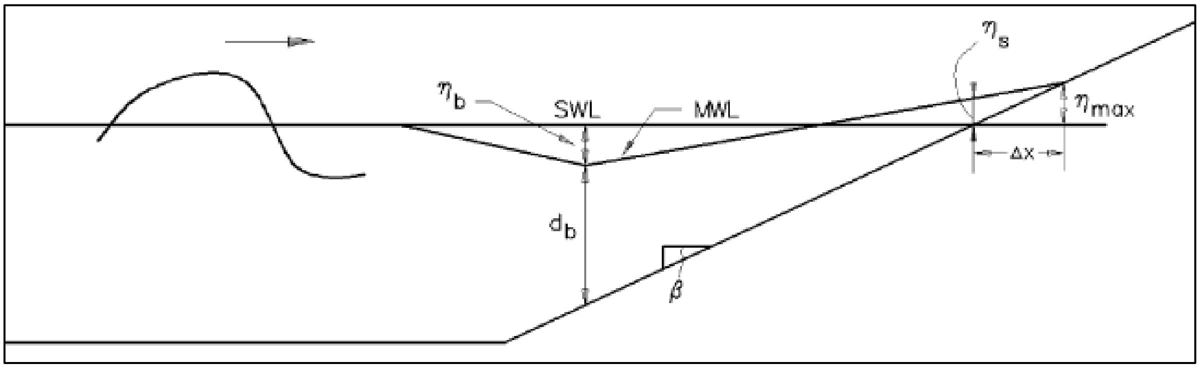


Figure 21. EM 1110-2-1100 Figure II-4-7 Definition Sketch for Wave Setup (USACE 2002)

5.2. Storm Surge

Storm surge is an increase in water elevation caused by a combination of relatively low atmospheric pressure and wind-driven transport of seawater over relatively shallow and large unobstructed waters. Inverted barometer effects cause an increase or decrease in the water surface elevation of 1 ft for every 30 millibars of change in atmospheric pressure. Friction at the air-sea interface is increased when the air is colder than the water, which causes more wind-driven transport. Storm-induced surge can produce short-term increases in water level, which can rise to an elevation considerably above tidal levels. Elim experiences severe storm surge that contributes to flooding along Elim Beach and up Elim Creek (HDR 2012). There is no water level gage or known storm surge model for Elim. As a result, the Western Alaska Storm Surge Modeling Study (Chapman & Mark 2014) results for Golovin, which is 23 air miles west of Elim (Figure 2), were used for storm surge estimates at Elim. The Western Alaska Storm Modeling Study results for Golovin were converted from ft above MLLW to the raw storm surge residual presented in Table 3.

Table 3. Annual Exceedance Probability of Storm Surge in Golovin

AEP [%]	Surge[ft]
20	5.7
10	7.7
7	8.7
5	9.3
4	9.7
2	11.1
1	13.1

5.3. Setdown

Setdowns occur in the Elim area during periods of north winds and/or high-pressure atmospheric conditions. The result is a lowering of the water surface elevation below that of the predicted astronomical tide level. Setdowns typically occur during the ice-free months of September through November when north winds are more prevalent (Figure 12). The duration of setdown water surface elevations varies. These are usually associated with north winds of approximately 20 knots and atmospheric pressures of 1,000 millibars and greater. A duration exceedance analysis was performed on the Nome NOAA Water Level Station 9468756 to determine how often the water level was below the predicted astronomical tide level to predict how prevalent setdown is at Elim. Though Nome has smaller storm surges than those experienced at Elim, Nome is the most appropriate, readily available water level data that can be analyzed for Elim. For June, July, and August, for approximately 93% of the data in the historical record at Nome, the water level was above 0 ft MLLW. For September, October, and November, between 72% and 87% of historic water levels were above 0 ft MLLW. For the season, including May through November, approximately 86% of historic water levels were above 0 ft MLLW (Figure 22). In May and November, which are shoulder open water months with unpredictable ice coverage, approximately 90% of historic water levels were above 0 ft MLLW. It was determined that since between 86% and 90% of the historic water levels are at or above the predicted astronomical tide level, that setdown would not be taken into account for the navigation improvements design.

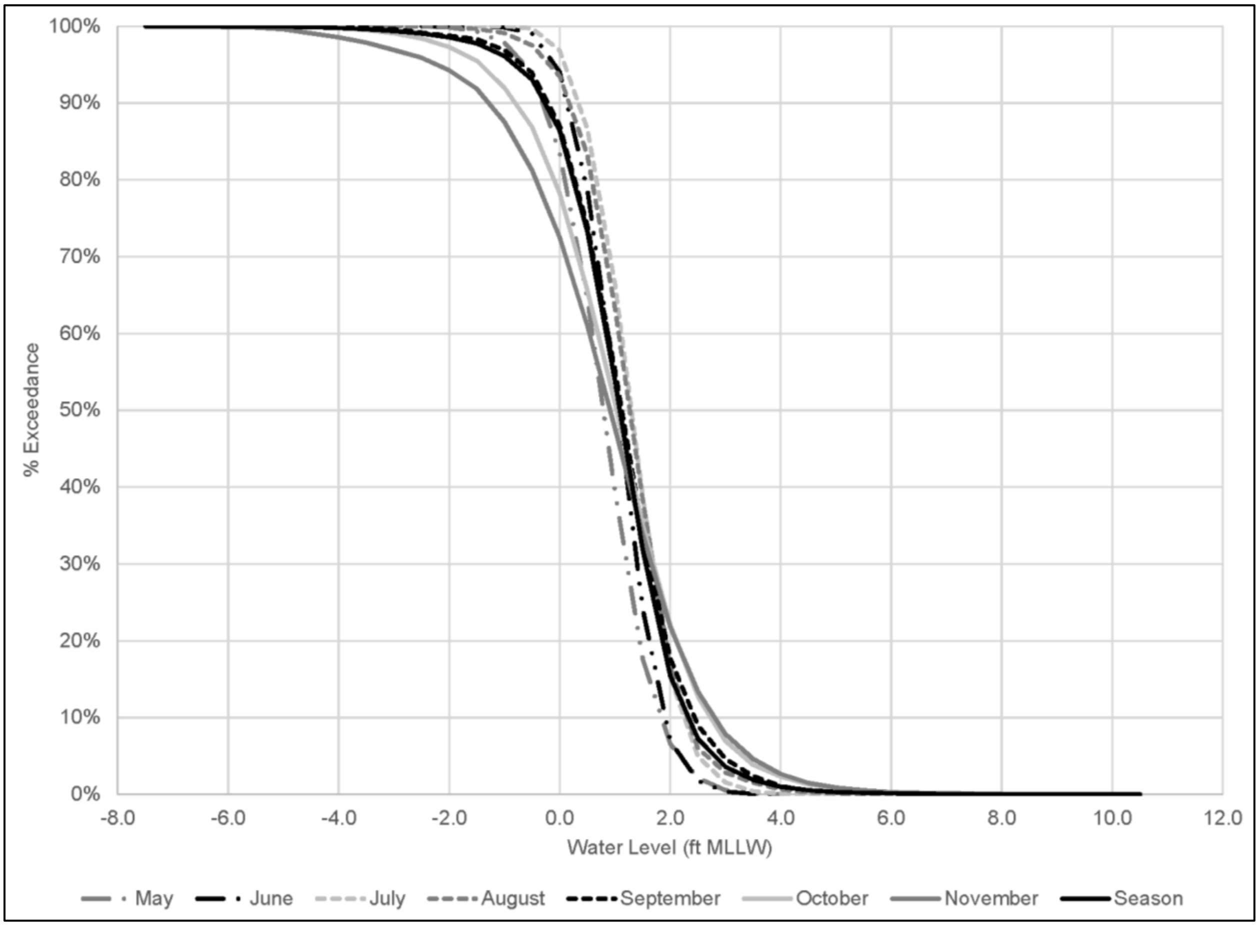


Figure 22. Nome Water Level Analysis NOAA Station 9468756 - October 1992 through December 2019 for May through November and the Season (the Water Level is in Reference to 0 ft MLLW)

5.4. Relative Sea Level Change

The U.S. Army Corps of Engineers (USACE) requires that planning studies and engineering designs consider alternatives that are formulated and evaluated for the entire range of possible future rates of sea level change (SLC). Designs must be evaluated over the project life cycle and include evaluations for the three scenarios of “low,” “intermediate,” and “high” sea level change. According to Engineer Regulation (ER) 1100-2-8162 (USACE 2019) and Engineer Pamphlet 1100-2-1 (USACE 2019), the SLC “low” rate is the historic SLC. The “intermediate” and “high” rates are computed by:

- Estimating the “intermediate” rate of local mean sea level change using the modified National Research Council (NRC) Curve I, the NRC equations, and correcting for the local rate of vertical land movement (VLM).
- Estimating the “high” rate of local mean sea level change using the modified NRC Curve III, NRC equations, and correcting for the local rate of VLM. This

"high" rate exceeds the upper bounds of the Intergovernmental Panel on Climate Change (IPCC) estimates from both 2001 (IPCC 2001) and 2007 (IPCC 2007) to accommodate the potential rapid loss of ice from Antarctica and Greenland.

The 1987 NRC described these three scenarios using the following equation:

$$\boldsymbol{E(t) = 0.0012t + bt^2} \qquad \text{Equation 1}$$

in which *t* represents years, starting in 1986, *b* is a constant, and *E(t)* is the eustatic sea level change, in meters, as a function of *t*. The NRC committee recommended, "projections be updated approximately every decade to incorporate additional data." At the time the NRC report was prepared, the estimate of global mean sea-level (GMSL) change was approximately 1.2 mm/year. Using the current estimate of 1.7 mm/year for GMSL change, as presented by the IPCC (IPCC 2007), results in this equation being modified to be:

$$\boldsymbol{E(t) = 0.017t + bt^2} \qquad \text{Equation 2}$$

The three scenarios proposed by the NRC result in global eustatic sea level rise values (by the year 2100) of 0.5 meters, 1.0 meters, and 1.5 meters. Adjusting the equation to include the historic GMSL change rate of 1.7 mm/year and the start date of 1992 (which corresponds to the midpoint of the current National Tidal Datum Epoch of 1983-2001) results in updated values for the variable b being equal to 2.71E-5 for modified NRC Curve I, 7.00E-5 for modified NRC Curve II, and 1.13E-4 for modified NRC Curve III.

Manipulating the equation to account for it being developed for eustatic sea level rise starting in 1992, while projects will be constructed at some date after 1992, results in the following equation:

$$\boldsymbol{E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2)} \qquad \text{Equation 3}$$

where t_1 is the time between the project's construction date and 1992 and t_2 is the time between a future date at which one wants an estimate for sea-level change and 1992 (or $t_2 = t_1$ + the number of years after construction). Using the three *b* scenarios required by ER 1100-2-8162 (USACE 2019) results in the following three GMSL rise scenarios depicted in Figure 23.

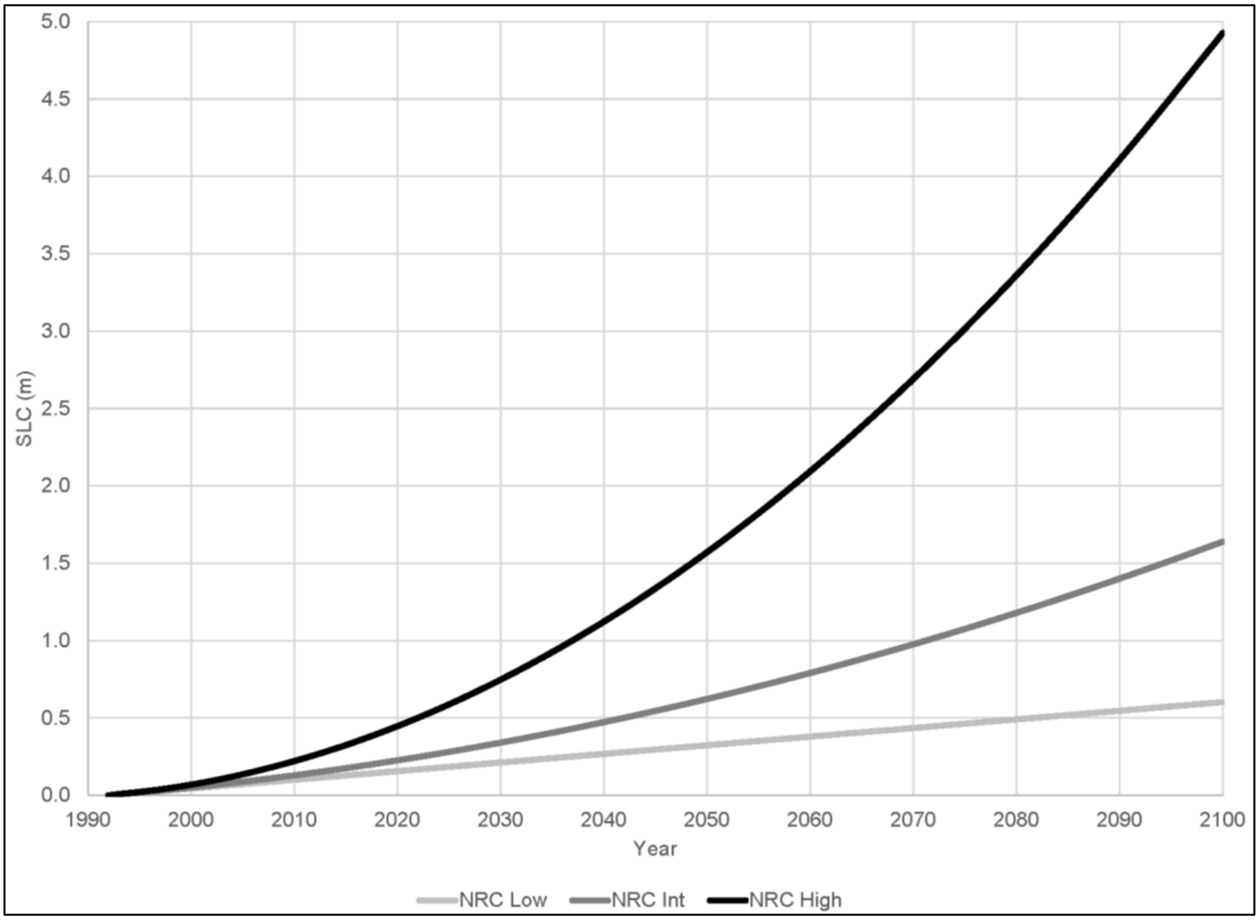


Figure 23. Scenarios for GMSL Rise

There is no sea level trend data for Elim. Due to Elim's location along the sub-Arctic and the lack of data and analysis in this region available for the IPCC estimated GMSL change, the GMSL Rise was deemed an inappropriate base SLC to use to estimate the Relative Sea Level Change (RSLC) in Elim. Several factors contribute to variations in sea level change across geographic areas, including the distributions of changes in ocean temperature, salinity, winds, and ocean circulation (IPCC 2007). The current estimate for GMSL change, as presented by the IPCC, is based on satellite altimetry, thermosteric data (changes in ocean temperature), and tide gages (IPCC 2007). The geographic distribution of TOPEX/Poseidon satellite altimeters (IPCC 2007), thermosteric sea level change estimates (IPCC 2007), and network of tide gages available for the analysis (IPCC 2007) did not cover the sub-Arctic.

Nome is approximately 96 miles west of Elim (Figure 25). It has the closest and longest NOAA-NOS tide gage record in Norton Sound from 1992 to present (NOAA Station 9468756), approximately 26 years, shorter than the recommended 2 tidal epoch duration of about 40 years. NOAA's policy is to develop RLSC trends when there are

30+ years of record for a water level station. NOAA's Relative Sea Level Team approved the release of the trend values for the Nome Station 4 years early due to the highly consistent long-term trend and the high request of trend data for the region (Kinsman 2019). NOAA Center for Operational Oceanographic Products and Services published sea level trend for Nome is +0.0102 ft/year, with a 95% confidence interval of ±0.00958 ft/year, as of December 2019.

The Alaska District used the SLR Rates POA Spreadsheet Tool, created by the Army Geospatial Center, to estimate a running RSLC trend for the water level record at Nome. From the SLR Rate tool provided to the Alaska District, a sea level trend for Nome was determined to be +0.0149 ft/year, using continuous data from August 1997 through August 2019 (Figure 24). Data prior to 1997 were excluded due to a large gap in data collection from March 1994 to August 1997. This rate is greater than the rate published by NOAA and was used as the basis for the Elim RSLC analysis due to the uncertainty of using a trend based on data set that does not cover 40 years.

To estimate RSLC at Elim, the local rate of VLM, published by NASA Jet Propulsion Laboratory, for Nome was subtracted from the Nome sea level trend to estimate a regional sea level trend. The local rate of VLM for Elim was then added to the regional sea level trend. The local rate of VLM for Nome is -0.00156 ft/year ±0.00121 ft/year (NASA Jet Propulsion Laboratory, n.d.), and the local rate of VLM for Elim is -0.00173 ft/year ±0.00065 ft/year (NASA Jet Propulsion Laboratory, n.d.). The estimated sea level trend for Elim is +0.0150 ft/year (Figure 26). Based on the 50-year period of analysis, as defined in ER 1105-2-100 (USACE 2000), for a project in Elim, the project could be exposed to as much as +3.80 ft (Table 4) of sea level rise after construction, assuming construction in 2025. For the 100-year adaptation horizon, a project in Elim could be exposed to as much as +8.56 ft (Table 4) of sea level rise after construction.

As stated in ER 1100-2-8162 and EP 1100-2-1, additional intermediate or high rates may also be included. To evaluate alternative methods of determining future RSLC, the Coastal Assessment Regional Scenario Working Group (CARSWG) low, medium, and high curves for the National Guard Elim Armory were considered. These values, published for years 2035, 2065, and 2100, are based on global sea level rise values of 1.6 ft for the low scenario, 3.3 ft for the medium scenario, and 4.9 ft for the high scenario. They also take into account site-specific adjustments for VLM, ocean circulation, and ice melt effects (Hall et al. 2016). To compare the values published for the Elim Armory, they were compared to the results for the three scenarios required under ER 1100-2-8162 and EP 1100-2-1 (Figure 26 and Table 4).

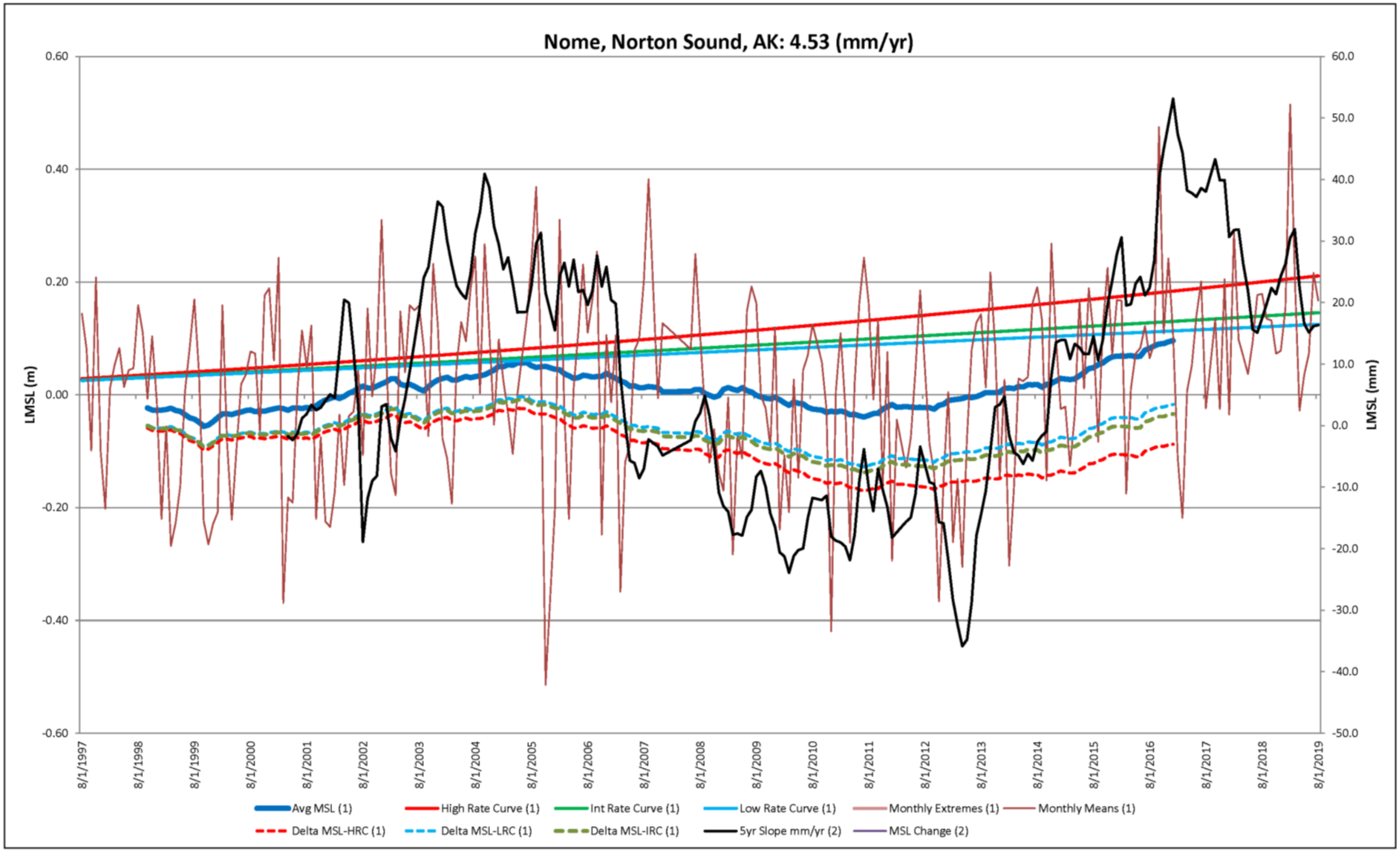


Figure 24. Sea Level Change Rate Model for Nome, AK for August 1997 through August 2019

Note: Sea level change rate of 4.53 mm/yr (0.0149 ft/yr) is based on using continuous water level data from August 1997 through August 2019

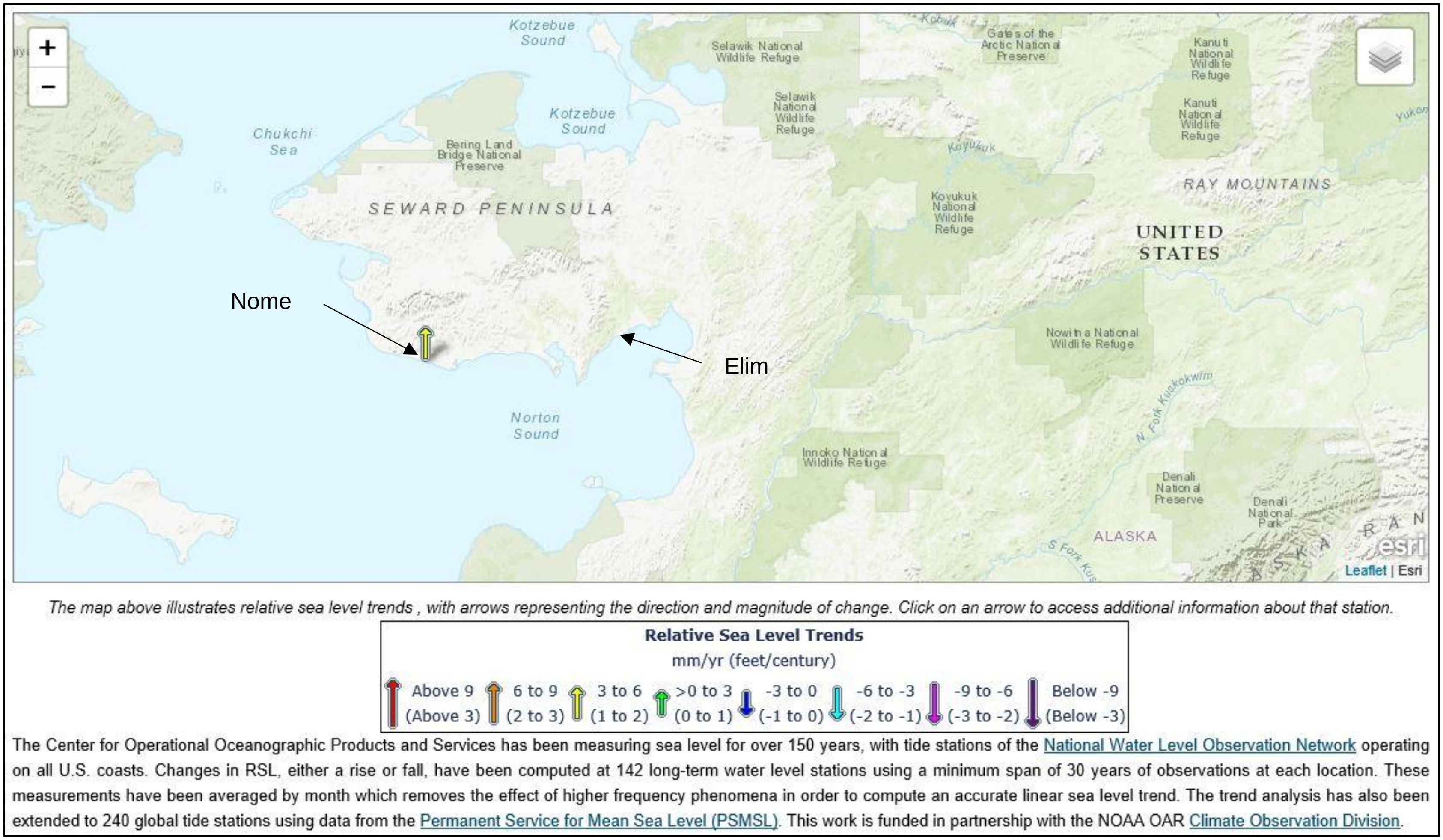


Figure 25. Location of the Nome NOAA Station 9468756 Relative to Elim

The Nome SLR Rates POA Spreadsheet Tool estimated RSLC with VLM adjustments and was used as the basis for estimating RSLC at Elim (Figure 26). The 50-year intermediate RSLC estimate of +1.86 ft (Table 4) was included in the total water level used for the analysis of design for all alternative formulations. Table 4 shows the results of ER 1100-2-8162 requirements for analysis compared to CARSWG published values at the Years 2035, 2065, and 2100. The 50-year RSLC estimates were used for the design and plan formulation of the alternatives. The 100-year RSLC estimates were used to analyze the design's resiliency and potential adaptation strategies, presented in Section 9.4 Climate Change, Resiliency, and Adaptation. Though the average MSL never goes above the low curve, the intermediate curve was used for the final design due to the short period of record and the high variability of the MSL 5-year slope (Figure 24).

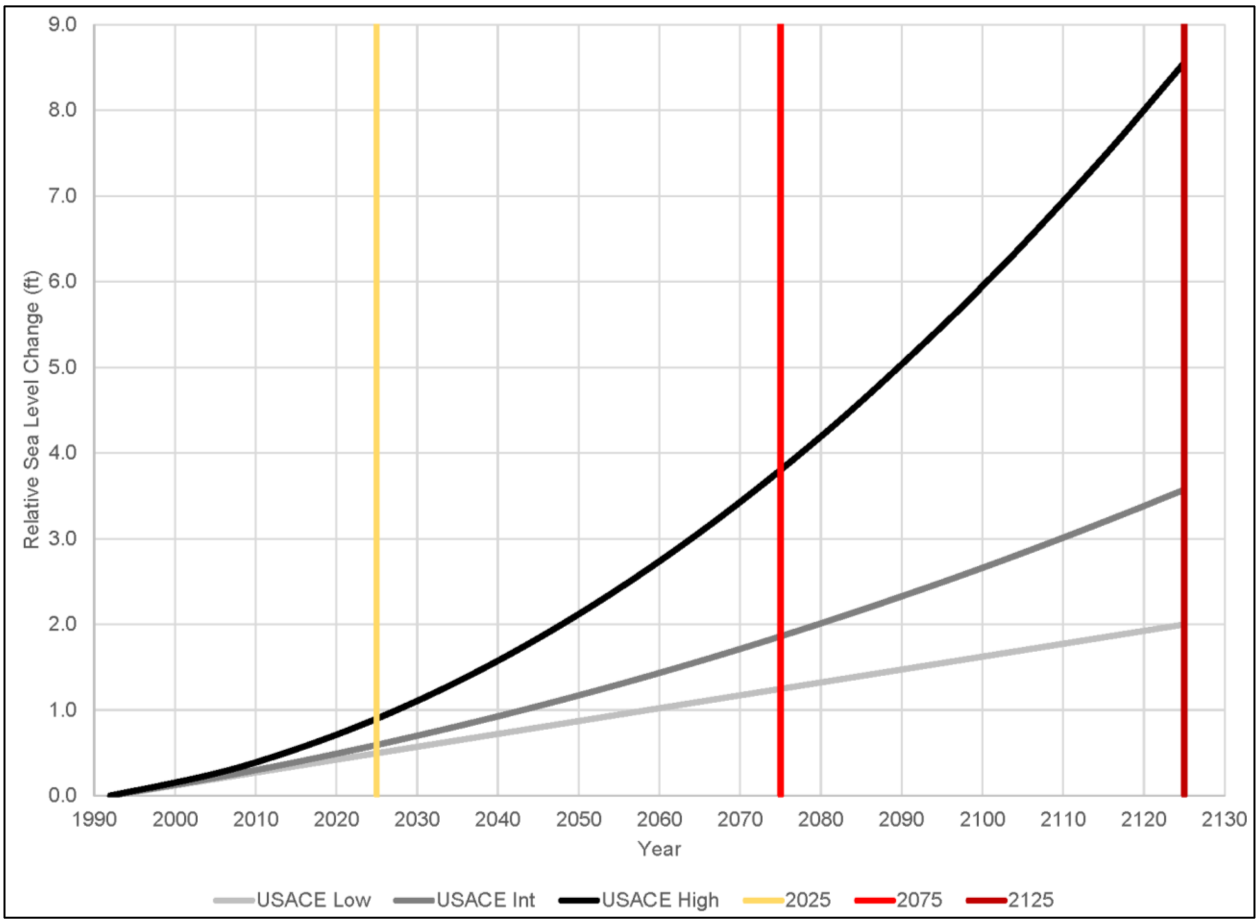


Figure 26. Relative Sea Level Change Projections in Elim (Assumed Year of Construction in 2025 (Yellow), End of Period of Analysis in 2075 (Red), and End of Adaptation Horizon in 2125 (Dark Red))

Table 4. Relative Sea Level Change Projections for the 50-Year Period of Analysis, the Year 2100, and a 100-Year Project Adaptation Horizon assuming Project Construction in 2025

Year	ER 1100-2-8162			CARSWG		
	Low [ft]	Int [ft]	High [ft]	Low [ft]	Medium [ft]	High [ft]
2025	0.50	0.59	0.90			
2035	0.65	0.81	1.33	0.3	0.7	0.4
2065	1.10	1.57	3.07	0.4	1.0	2.0
2075	1.25	1.86	3.80			
2100	1.62	2.66	5.95	1.3	2.9	4.5
2125	2.00	3.57	8.56			

6. DESIGN CRITERIA

6.1. Design Fleet

The economic analysis generated the vessel demand for this study. It was assumed that the existing Elim fleet plus two transient tenders and one barge and tug operation could use navigation improvements at Elim. The characteristics of the fleet proposed to occupy the various alternatives are shown in Table 5. Proposed harbor plans were laid out to accommodate all or a subset of the identified vessels, depending on the alternative.

Table 5. Fleet Characteristics

Vessel	Vessel Length [ft]	Design Beam [ft]	Design Draft [ft]
Freight Barge	159	52	7
Tug	86	28.5	8
Tender	66	24	6
Commercial	32	12	5
Subsistence	18	7	2

6.2. Entrance Channel and Maneuvering Area

The entrance channel width (Table 6) was determined by criteria given in EM 1110-2-1615 (USACE 1984) Table 3-1. The following assumptions were made:

- Poor vessel controllability;
- Two-way straight trench entrance channel for Alternatives 2-4 and 6;
- One-way tug and barge straight trench entrance for Alternatives 5 and 7

Table 6. Entrance Channel Width Required for Each Alternative

Alternative	Vessels	Channel Width [ft]
2	2 Commercials	75
3	Commercial and Tender	115
4	Commercial and Tender	115
5	Barge and Tug	250
6	2 Commercials	75
7	Barge and Tug	250

The maneuvering areas and the fairway widths were set to the American Society of Civil Engineers (ASCE) minimum recommendation of 75 ft (ASCE 2012).

6.3. Entrance Channel and Maneuvering Depth

The entrance and maneuvering depth is dependent on the vessel fleet included for the alternative design. Vessels were assumed to be loaded when entering the harbor, so loaded drafts were used to calculate the required depths for the entrance and mooring basin depth requirements (Figure 27) as outlined in EM 1110-2-1615 (USACE 1984). For the small crafts, the subsistence and commercial vessels, moving at reasonable speeds, squat was taken to be 1 ft for the entrance channel and 0.5 ft for interior channels, moorage basins, and turning basins. For larger vessels, the tender and barge, the squat was calculated, taking into account the anticipated vessel speed, characteristics of the channel and vessel, and interactions with another vessel. Pitch, roll, and heave was estimated based on the rule-of-thumb of half the operation limit wave height for smaller crafts and two-thirds the operational limit wave height for a barge. For hard bottoms, a safety clearance of 2 ft is required.

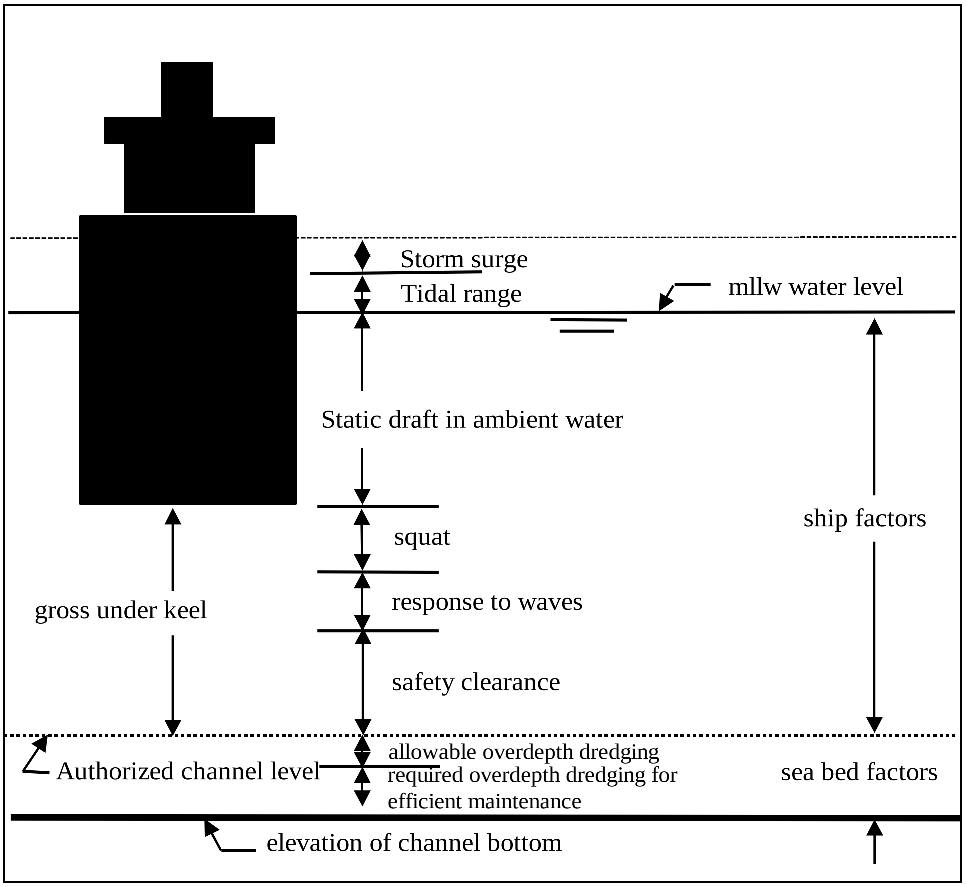


Figure 27. Vessel Factors that Influence Dredge Depth

Setdown below MLLW was not considered when determining design depths due to the infrequency of water levels being below the predicted astronomical tide (Figure 28). For the purpose of this study, it was assumed that vessels impacted by setdown events will wait offshore until conditions permit channel transit.

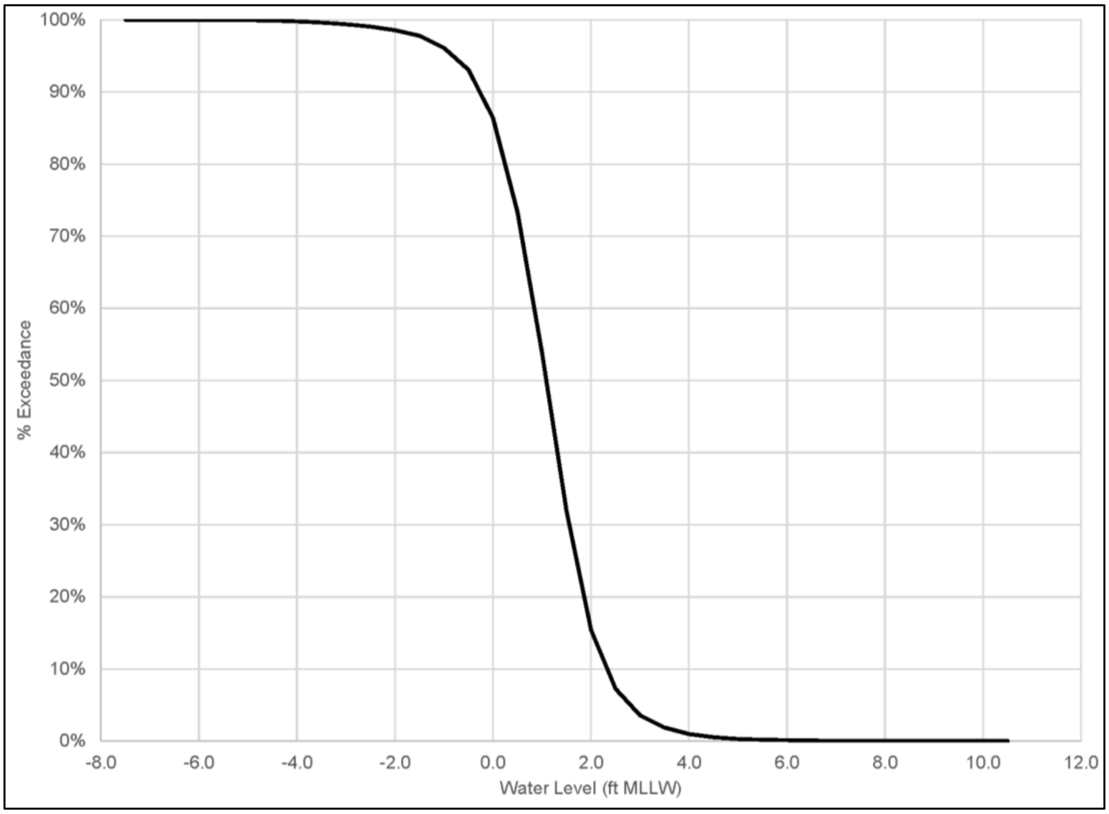


Figure 28. Nome Water Level Analysis NOAA Station 9468756 October 1992 through December 2019 for the Ice-Free Season of May through November (the Water Level is in Reference to 0 ft MLLW)

6.3.1. Subsistence Design Vessel

For subsistence vessel access to the navigation improvements, 86% of the ice-free season (Figure 28), with vessel draft of 2 ft plus an additional 0.5 ft of squat and 0.5 ft for pitch, roll, and heave in a 1-ft wave, the required depth would be -3 ft MLLW. The dredged channel is anticipated to have a hard bottom, which requires an additional 2 ft for safety, bringing the minimum required depth to -5 ft MLLW for safe operations of the subsistence vessels within the entrance channel (Table 7).

Table 7. Subsistence Vessel Minimum Required Depth, Relative to MLLW

	[ft]	[ft MLLW]
Design Water Level		0
Vessel Draft	2	-2
Squat	0.5	-2.5
Pitch, Roll, Heave	0.5	-3
Safety Clearance	2	-5
Minimum Required Depth		-5

6.3.2. Commercial Design Vessel

For commercial vessel access to the navigation improvements, 86% of the ice-free season (Figure 28), with vessel draft of 5 ft plus an additional 0.5 ft of squat and 0.5 ft for pitch, roll, and heave in a 1-ft wave, the required depth would be -6 ft MLLW. The dredged channel is anticipated to have a hard bottom, which requires an additional 2 ft for safety, bringing the minimum required depth to -8 ft MLLW for safe operations of the commercial vessels within the entrance channel (Table 8).

Table 8. Commercial Vessel Minimum Required Depth, Relative to MLLW

	[ft]	[ft MLLW]
Design Water Level		0
Vessel Draft	5	-5
Squat	0.5	-5.5
Pitch, Roll, Heave	0.5	-6
Safety Clearance	2	-8
Minimum Required Depth		**-8**

6.3.3. Tender Design Vessel

For tender vessel access to the navigation improvements, 86% of the ice-free season (Figure 28), with vessel draft of 6 ft plus an additional 1 ft for pitch, roll, and heave in a 2-ft wave, the required depth would be -7 ft MLLW. The dredged channel is anticipated to have a hard bottom, which requires an additional 2 ft for safety, bringing the minimum required depth to -9 ft MLLW for safe operations of the tender vessel within the entrance channel (Table 9).

Table 9. Tender Vessel Minimum Required Depth, Relative to MLLW

	[ft]	[ft MLLW]
Design Water Level		0
Vessel Draft	6	-6
Squat	0	-6
Pitch, Roll, Heave	1	-7
Safety Clearance	2	-9
Minimum Required Depth		**-9**

6.3.4. Barge and Tug Design Vessel

For barge and tug access to the navigation improvements, 86% of the ice-free season (Figure 28), with vessel draft of 8 ft plus an additional 2 ft for pitch, roll, and heave in a 3-ft wave, the required depth would be -10 ft MLLW. The dredged channel is anticipated to have a hard bottom, which requires an additional 2 ft for safety, bringing the minimum required depth to -12 ft MLLW for safe operations of the barge and tug within the entrance channel (Table 10).

Table 10. Barge and Tug Minimum Required Depth, Relative to MLLW

	[ft]	[ft MLLW]
Design Water Level		0
Vessel Draft	8	-8
Squat	0	-8
Pitch, Roll, Heave	2	-10
Safety Clearance	2	-12
Minimum Required Depth		**-12**

6.4. Wave Reduction

Diffraction analysis using the Wiegel diagrams for semi-infinite rigid, impermeable breakwaters (Wiegel 1962) with the fetch-limited waves from the northeast and east and deep water design wave from the southeast through southwest (Section 4.3 Design Condition) were used to determine the wave height reduction in the harbor for each alternative. The operational limit wave height in the harbor for each design vessel is shown in Table 11. The acceptable wave heights were based on conversations with barge operators and NSEDC.

Table 11. Operational Limited Wave Height

Vessel	**Wave Height [ft]**
Subsistence	1.0
Commercial	1.0
Tender	2.0
Barge and Tug	3.0

The diagrams were converted into TIFF files and loaded into the design drawings to utilize the Wiegel diagrams. The diagrams where then scaled based on the wavelength associated with the governing wave conditions from each direction. The diagrams had to be mirrored and/or rotated to align the nose of the diagram’s semi-infinite breakwater nose with the nose of the designed breakwater that would interrupt the wave train first (Figure 29). The reduction value resulting in the smallest wave reduction impacting an area of the basin was then multiplied by the incoming wave height to determine the wave height within the harbor. The second breakwater was not considered in the wave reduction analysis.

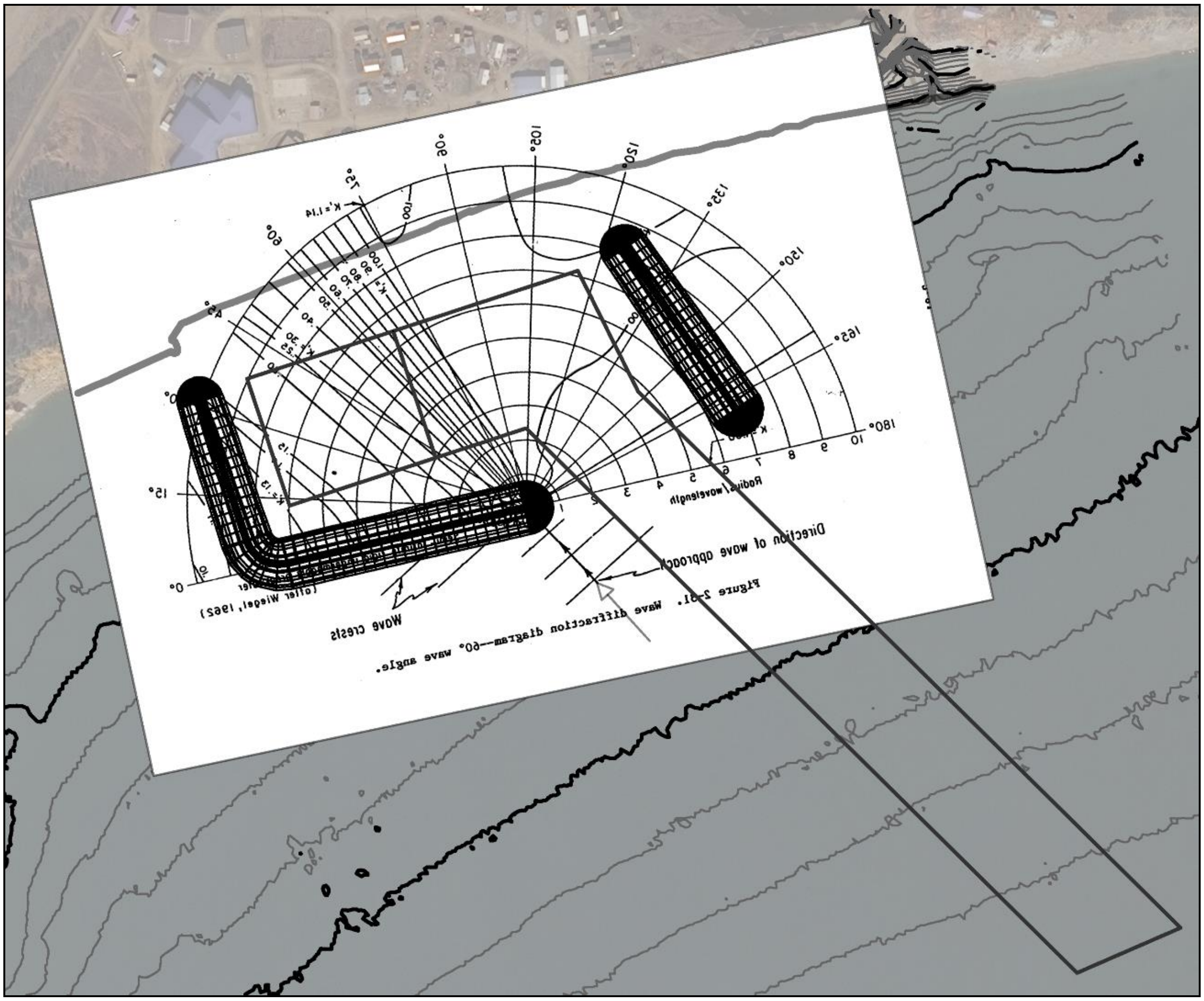


Figure 29. Wiegel Diagram Application for Wave Reduction Analysis

7. NAVIGATION IMPROVEMENT OPTIONS

Options considered for vessel protection during launching and landing include:

- Floating breakwater
- Dolos breakwater
- Rubble-mound breakwater

7.1. Floating Breakwater

A floating breakwater consists of a floating structure that can provide wave protection for short period waves with heights up to 4 ft. A floating breakwater is anchored with chains or piles. Because there is significant sea ice within Norton Sound, a floating breakwater was dropped from further consideration.

7.2. Dolos Breakwater

A Dolos is reinforced concrete blocks in a complex shape used in a number of harbor and shore protection projects by USACE and around the world. Dolos has been used for harbors in the Great Lakes, which also experience severe ice conditions. They have also been used for a shore protection project in Kodiak, AK, that does not experience significant ice conditions but has not been used within Norton Sound. The closest known location that makes Dolos is Seattle, WA, approximately 2,700 sea miles from Elim. Due to the lack of a closer concrete plant and the high mobilization cost from Seattle, Dolos were not considered further.

7.3. Rubble-Mound Breakwater

The use of a rubble-mound breakwater to provide wave protection is a proven concept in the sub-Arctic environment. Rubble-mound breakwaters have been successfully used throughout Norton Sound. Three permitted rock quarries are within 1,300 sea miles, with the closest being the Cape Nome quarry, which is only approximately 100 sea miles from Elim. Because rubble-mound breakwaters have a proven history in similar environments, the decision was made to pursue a rubble-mound breakwater option.

8. BREAKWATER DESIGN PARAMETERS

8.1. Total Water Level

There is a significant storm surge observed in the northeast portion of Norton Sound, which influences the total water level impacting coastal projects within this area. The total water level at the project site was determined using the following equation:

$$\boldsymbol{Total\ Water\ Level = Tide + Wave\ Setup + Storm\ Surge + RSLC} \qquad \text{Equation 4}$$

The tidal component used for the total water level was MHHW, which is +2.62 ft MLLW (Section 3.3 Tide) to take into account that storm surge can last a tidal cycle. The wave setup of 0.74 ft at the shore for the design wave was added on top of MHHW. The Golovin 2% AEP storm surge of 11.1 ft (Section 5.2 Storm Surge) was added to MHHW plus wave setup. Lastly, to account for the effect of RSLC, based on ER 1100-2-8162, at the 50-year economic horizon for the intermediate curve of +1.86 ft (Section 5.4 Relative Sea Level Change) was added on top of the MHHW, wave setup, and the 2% AEP storm surge, to result in a total water level for the design of +16.3 ft MLLW.

8.2. Run-up

Run-up with 2% exceedance level ($R_{2\%}$) was calculated using the following equation for a permeable rock armored slope with irregular head-on waves shown in the CEM Part VI Chapter 5 Fundamentals of Design:

$$R_{2\%} \begin{array}{ll} = 0.96\xi_{om} \times H_s & for\ 1.0 < \xi_{om} \leq 1.5 \\ = 1.17(\xi_{om})^{0.46} \times H_s & for\ 1.5 < \xi_{om} \leq 3.1 \\ = 1.97 \times H_s & for\ 3.1 < \xi_{om} < 7.5 \end{array} \qquad \text{Equation 5}$$

where ξ_{om} is the mean surf-similarity parameter and is dependent on the mean wave period, significant wave height, and slope of the structure, and H_s is the significant wave height.

The run-up elevation was added to the total water level to obtain the crest elevation of the breakwaters that would prevent overtopping. $R_{2\%}$ would be 10.4 ft on the armored breakwater with a slope of 1:1.5 with the design wave height of 6.2 ft and a mean period of 4.8 seconds.

8.3. Armor Stone

Due to the hazard posed by ice riding up onto the breakwater, armor stone sized for both the design wave and the ice conditions were considered. The larger stone requirement was used for the design.

8.3.1. Design Wave

The armor stone was analyzed for being impacted by a nonbreaking wave traveling onshore. Based on the deep water duration analysis, the design wave height is 6.2 ft (Section 4 Wave Analysis). Using the Hudson equation, armor stone sized for specially placed armor being impacted by a 6.2-ft nonbreaking wave would have a median weight of 0.9 tons.

If the total water level at the toe of the breakwater is higher than 9 ft, the 2% AEP wave of 12.3 ft could impact the breakwater. Using the Hudson equation, armor stone sized for specially placed armor being impacted by a 12.3-ft nonbreaking wave would have a median weight of 6.5 tons.

8.3.2. Ice

A physical model was performed in 1982 by the Iowa Institute of Hydraulic Research for the Port of Nome to provide data on ice-structure interactions from which design criteria for ice forces within Norton Sound and the eastern Bering Sea could be attained (Ettema & Kennedy 1982). Based on field observations and community input, ice thickness at Nome was determined to be between 36 and 54 inches. This information, coupled with analytical results, provided strength conditions for the modeled ice. The ice was grown in an ice tank and, once formed, was warmed and weakened until it reached the prescribed strength for each test. The ice was then pushed against the stationary, hand-built 1:20 Froude-scaled sideslope. Twenty tests were performed, with the ice impacting the sideslope normal to structure or at a 45° angle. The conclusion of the Ice Study for the Port of Nome was that 8-ton armor stone could withstand ice ride-up (Ettema & Kennedy 1982) and that ice over-ride was not likely to result in significant structural damage. An 8-ton armor stone size should be able to withstand 13.2-ft waves, compared to the design wave height of 6.1 ft. A 13.2-ft wave is unlikely to occur at Elim based on the hindcast conditions (Figure 15) and is larger than the 2% AEP wave height of 12.3 ft.

8.4. Typical Cross-Section

The crest width was set at 14.0 ft for overtopping conditions, based on the combined width of three armor stones. The crest height determined by total water level and run-up would be set at +26.7 ft MLLW. The final design crest elevation was set to +20 ft MLLW, with the cross-section designed for moderate overtopping conditions. The armor stone being sized for ice impacts also creates a more resilient design that could dissipate more energy during events that have both large waves and high water. A typical cross-section is shown in Figure 30.

Neglecting RSLC and based on the crest elevation, it is anticipated that the breakwaters would begin overtopping once the wave height exceeded 1.8 ft if it occurred during a 2% AEP storm surge, or the storm surge exceeded 4.4 ft when combined with the design wave height.

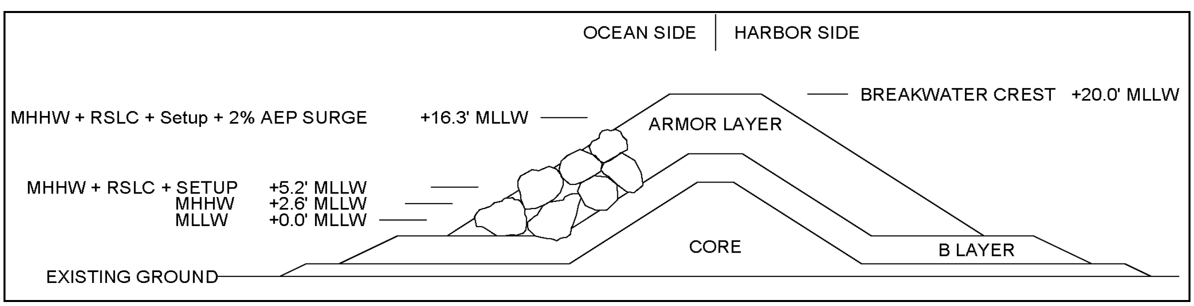


Figure 30. Typical Breakwater Section with Water Level Components

9. HARBOR DESIGN CONDITIONS

9.1. Water Quality and Circulation

The circulation was evaluated against recommendations outlined in the *Planning and Design Guidelines for Small Craft Harbors* (ASCE 2012).

The aspect ratio is a measure of the length divided by the width of the basin. The aspect ratio should be close to unity for peak flushing efficiency (Figure 31). The areas of potentially low exchange in the corners of the basin can be checked to ensure that no more than 5% of the total areas have exchange coefficients less than 0.15. A maximum basin aspect ratio of 1:4 will minimize possible zones of stagnation and short-circuiting of circulation cells within the basin.

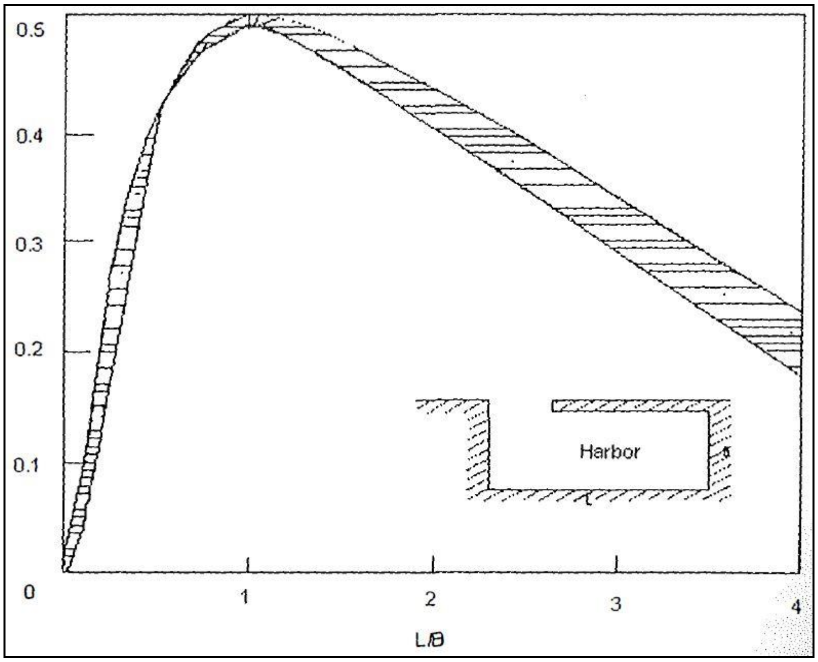


Figure 31. Flushing Exchange Coefficient as a Function of Basin Aspect Ratio (L/B) (ASCE 2012)

The area ratio (AR) is the ratio of the basin area (A) to channel cross-sectional area (a). The size of the fleet and mooring density determines the basin size, and the vessel draft, beam, wave conditions, and tides determine the channel cross-section. A large area ratio (greater than 200) is required, ideally 400 or greater, for good flushing.

9.2. Dredge Material

Based on the fall 2018 geotechnical site investigation (Alaska District 2018), surface material at Elim Beach varies from poorly to well-graded gravel with sand, cobbles, and boulders (Figure 32 through Figure 34). Bedrock outcrops consisting of weathered limestone are located at the east and west ends of the beach. The volume of cobbles and boulders observed from the surface range from 10% to 75% of the total sediment volume at various locations along the beach. Additional geophysical measurements were completed in summer 2019 (Golder 2019), which found that there is some spatial variability in depth to bedrock or other hard/non-rippable subsurface material. There are numerous visible bedrock outcrops on the beach, which are surrounded by very shallow (a few feet thickness) of soft alluvium or soil. The depth of soft alluvial is generally found to be no thicker than 9 ft throughout the subaerial portion of Elim Beach, according to two seismic refraction lines and additional ground-penetrating radar data. These data also suggest that the very dense material, which is likely to be the bedrock surface, is no deeper than 12 ft below the surface in this region.

The summer 2019 geophysical investigation also found that offshore of the site, there appears to be only a thin layer of soft material varying in thickness from 3 to 7 ft. Side scan imagery indicates that there are large boulders or outcrops along the shoreline of Airport Point but few features away from shore at either Airport Point or along Elim Beach. The report states that jet probing and/or drilling would be needed to provide geotechnical information necessary to further the design of any navigational improvements at Elim.

Figure 32. Surface Material Onshore Located at Elim Beach (October 2018)

Figure 33. Onshore Material Sample at Four Inches Depth at Elim Beach (October 2018)

Figure 34. Onshore Material Sample at One-Foot Depth at Elim Beach (October 2018)

9.2.1. Sediment Transport

There are no existing sediment transport models or studies of the Elim area. Generally, wave-driven processes are the predominant mechanism of sediment transport and morphology change in shallow nearshore and beach environments. Wave processes can cause sediment to move either in the cross-shore direction (onshore or offshore) or along the coast. In most coastal systems, longshore sediment transport processes are more important for changes to the local sediment budgets on long time scales (greater than annual) relative to cross-shore processes. CERC Formula[1] (CEM Part III Chapter 2 Longshore Sediment Transport) in the form of a volume transport rate[2] (Equation 6) was

[1] The CERC Formula (based on publications in the 1970's from the US Army Corps of Engineers Coastal Engineering Research Center and contained in the Coastal Engineering Manual), is a widely-used method of computing longshore sediment transport rates based on a correlation of the longshore component of wave energy flux to volumetric longshore sediment transport rate. The CERC formula includes a dimensionless empirical sediment transport parameter usually denoted as K.

[2] The volume transport rate equation uses metric units. Results were computed in metric [m^3/year] and then converted to English units [cy/year] to be consistent with the rest of the appendix.

applied to get an estimate of the longshore sediment transport rate at Elim. The CERC formula is as follows:

$$\boldsymbol{Q_l = K\left(\frac{\rho\sqrt{g}}{16\kappa^{1/2}(\rho_s-\rho)(1-n)}\right)H_b^{5/2}\,\sin(2\alpha_b)}$$ Equation 6

where K is the dimensionless empirical sediment transport parameter, ρ and ρ_s are the density of water and sediment taken as 1,025 kg/m^3 and 2,650 kg/m^3 respectively, n is the in-place sediment porosity taken as the standard value of 0.4, κ is the breaking index taken as 0.78 assuming shallow-water waves, H_b is the breaking wave height, and α_b is the breaking wave angle relative to the shoreline.

For application at Elim, the WIS ST82107, continuous-wave data, propagating from 126° through 215° and wind data blowing from 55° to 126°, was used for the analysis of the waves as described in Section 4 Wave Analysis for the wave forcing in water depths associated with MLLW. The breaking wave height and breaking wave angles were determined using the basic wave transformation procedure outlined in the CEM Part III, Chapter 2 Longshore Sediment Transport, assuming the shore normal angle is 130°.

The soil classification from beach test pits along Elim Beach (Alaska District 2018) indicates that the available sediment is all coarse-grained soils. With Unified Soil Classification System (USCS) classifications ranging from poorly graded sand to well-graded gravel. The test pits had a large range of D_{50} sizes, 1 ft to 1 1/3 ft below the surface, from approximately 2.00–12.70 mm, which are above the limits used to determine K values for the CERC Formula (Figure 35).

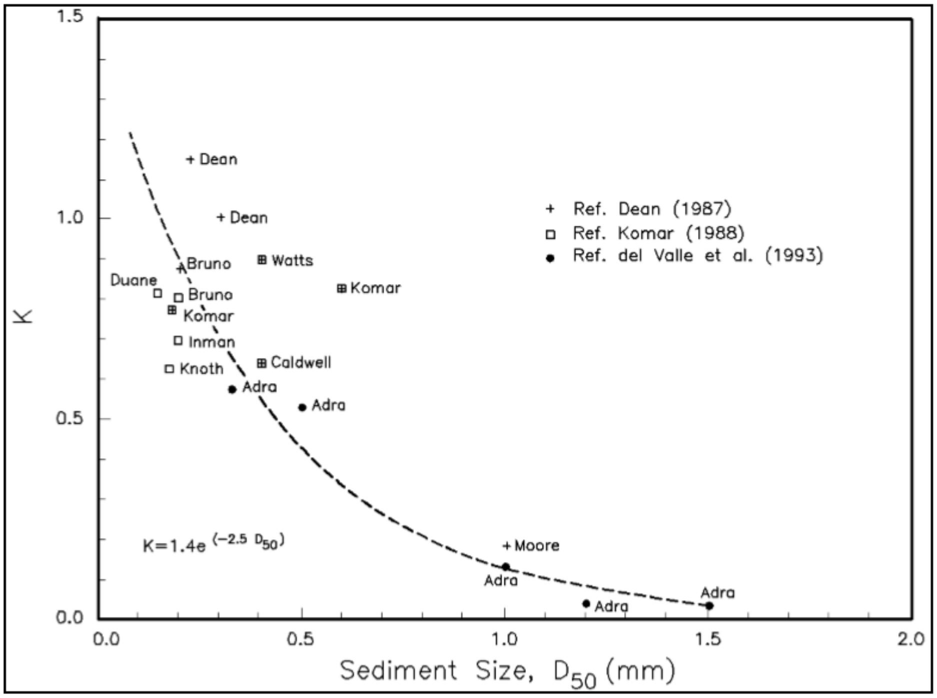


Figure 35. EM 1110-2-1100 Figure III-2-6 Coefficient K versus Median Grain Size D_{50} (del Valle, Medina, and Losada 1993) (USACE 2002)

Based on the empirical equation developed by del Valle, Medina, and Losada (del Valle, Medina, & Losada 1993), plotted in Figure 35, the values of K (the dimensionless empirical sediment transport parameter) for Elim Beach vary from 0.00 to 0.01. The values for Elim Beach are significantly lower than the standard values of K of 0.77 if waves are defined using RMS wave height statistics, or 0.39, if waves are defined using significant wave heights. Using the largest K value for Elim Beach of 0.01, associated with a test pit with the lowest D_{50} value of 2.00 mm, resulting in a net longshore sediment transport rate of 5,205 cy/year towards the northeast (Table 12). Using the CERC Formula, the estimated longshore sediment transport rate also decreases with the increase in water depth.

Table 12. Longshore Sediment Transport Rate Based on Test Pit D_{50} Values. Test Pit Values Based on Samples Taken Between 1'-1 1/3' Below Surface

Test Pit #	D_{50} (mm)	Q_l (cy/yr)
TP-02	4.75	5
TP-03	2.00	5,205
TP-04	12.70	0

Analysis of all available satellite imagery available for Elim from the L5, L7, L8, and S2 missions between 1995 to present with the CoastSat (Vos et al. 2019) tool does not indicate any significant trends in shoreline change at Elim. Although, there are large uncertainties in the rectification of satellite imagery datasets at far northern latitudes, which may limit the identification of small to moderate shoreline change rates. Unfortunately, adequate, repeat morphology data is not available to assess long-term shoreline change rates for the area.

Cross-shore sediment transport is the movement of sediment on and offshore due to wave action which, can also contribute to shoreline changes and/or inputs or exports to the littoral zone. No quantitative analysis of the cross-shore sediment transport was performed for this study, although a qualitative assessment of cross-shore processes for the field site is provided here. The beach material is coarse and heterogeneous (Alaska District 2018), which is not typical of low energy systems, but could indicate a local source of the material such as the headlands to the east and west or ice-rafting of material during the winter. Based on the analysis of the available survey data, there is a shallow sandbar located at or below the intertidal zone, at approximately -1–0 ft MLLW. Still, there is no indication of any complex offshore morphology or steep foreshore indicative of high energy gravel beaches. During major storms, which are likely infrequent based on the geometry of Elim Beach and the fairly uniform alongshore morphology, there could be cross-shore morphology changes. Based on the qualitative data available, it is assumed that longshore sediment transport dominates sediment transport along Elim Beach over inter-annual or longer time scales and, therefore, likely obscures any signal of cross-shore sediment transport.

Based on the longshore sediment transport rate, an inner depth of closure depth of approximately -20 ft MLLW assuming the effective wave height is the 1% AEP wave height of 4.04 meters (13.3 ft) (Hallermeier 1978), and the blockage caused by the breakwaters, estimated maintenance dredging rates were determined for each alternative. It was assumed that maintenance dredging would not be initiated until a project condition survey indicated that the average depth across the harbor and channel was at the required depth.

9.3. Offshore Extent of Breakwaters

To manage costs associated with large breakwater quantities, mobilization, and the number of construction seasons, the breakwater offshore extent was limited to the -8 ft MLLW contour. The -8 ft MLLW contour was chosen to ensure the breakwater noses would be located beyond the approximate location of breaking for the design wave during MLLW in order to provide safe entrance into the basin. Based on the longshore sediment transport rate, maintenance dredging would be more cost-effective than armoring the entrance channel.

9.4. Climate Change, Resiliency, and Adaptation

The NOAA began publishing an annual, peer-reviewed Arctic Report Card in 2006. The Report Card is a “source for clear, reliable, and concise environmental information on the current state of different components of the Arctic environmental system relative to historical records” (Osborne, Richter-Menge, & Jeffries 2018). The 2018 Report Card states that the Arctic sea ice cover is continuing to decline in the summer maximum extent and winter minimum extent (Perovich et al. 2018). The minimum sea ice extent usually occurs in late September. In 2018, the ice cover was 26% lower in late September than the average coverage between 1981 and 2010 and was tied for the 6th lowest ice cover since 1979 (Perovich et al. 2018). With a decreased sea ice extent, there is an increase in time that the sub-Arctic (i.e., Norton Sound) is ice-free or has limited sea ice coverage.

According to the Fourth National Climate Assessment (Wuebbles et al. 2019), a warming trend relative to average air temperatures was recorded from 1925 through 1960. A trend of increasing temperatures starting in the 1970s has been identified and is projected to continue throughout the state of Alaska. The largest temperature increases have been found in winter months with average minimum temperature increases of around 2° F statewide. Carbon emission models project variable increases in statewide temperatures across the state; for the Seward Peninsula region, forecast temperature increases are in the 6 – 8°F range for an intermediate model (RCP4.5) and in the 12 – 14°F range for a high model (RCP8.5) (Figure 36).

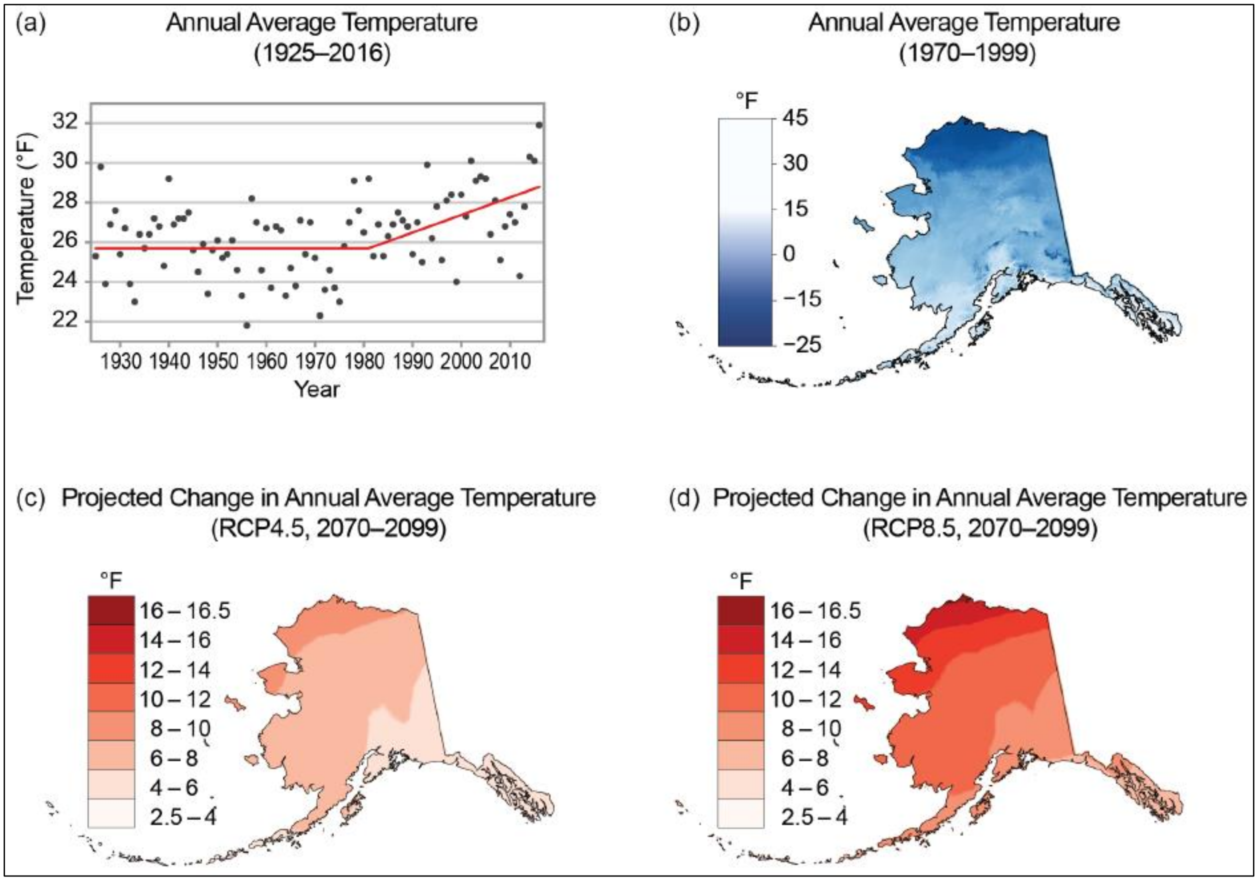


Figure 36. Alaska Temperature Data and Projections (Figure 26.1 from (Wuebbles et al. 2019)

Note (Annotation truncated from the report): (a) Alaska statewide annual temperatures for 1925–2016. The record shows high variability from 1925–1976, but from 1976–2016 a clear trend of +0.7°F per decade is evident. (b) 1970–1999 annual average temperature. (c) Projected changes from climate models in annual average temperature for the end of the 21st century (compared to 1970–1999 average) under a lower scenario. (d) The map is the same as (c) but for a higher scenario. Sources: (a) NOAA and USGS, (b-d) USGS.

An increase in winter temperatures in the region could decrease the period of shorefast and sea ice formation in Norton Sound. The site could be impacted by waves and storm surge in later parts of the year than the season of analysis used for this study. Changing sea ice conditions and potential sea level rise (Section 5.4 Relative Sea Level Change) at the project site could result in unknown changes to the storm conditions and increased depth limited wave height. These non-stationarities could result in increased overtopping of the breakwaters during high water events. The change in sea ice conditions is not anticipated to affect the armor stone size due to the necessity to oversize armor stone for current ice conditions (Section 8.3 Armor Stone). The wave analysis (Section 4 Wave Analysis) does not take into account non-stationarities caused by climate change due to the limited understanding of these impacts that will impact the design conditions. The breakwater cross-section is designed for moderate overtopping.

To evaluate potential damages due to different RSLC curves, an overtopping analysis was performed using equation 6.6[3] from the EurOtop Manual (Van der Meer et al. 2018), rearranged to solve for the average discharge results in the following equation:

$$\boldsymbol{q} = \left[\mathbf{0.1035} \times \mathbf{exp}\left(-\left(\mathbf{1.35}\frac{\boldsymbol{R_c}}{\boldsymbol{H_s\gamma_f\gamma_\beta}}\right)^{\mathbf{1.3}}\right)\right]\sqrt{\boldsymbol{gH_s^3}} \qquad \text{Equation 7}$$

where R_c is the freeboard (as defined in the EurOtop Manual: the difference between the crest of the breakwater and the total water level), H_s is the significant wave height, γ_f is the influence factor for the permeability and roughness of the slope (taken as 0.40 per Table 6.2 in the EurOtop Manual), and γ_β is the influence factor for oblique wave attack (assumed to be 1.0 for perpendicular wave attack).

Damages are anticipated (Figure 37) to be initiated in the year 2082 for the high RSLC curve (Figure 38) when average overtopping discharge reaches the threshold for initiation of damage to a revetment, 50 liters/s per m. Damage to the structure due to overtopping could be managed by placing two rows of 8-ton capstones along the crest of the breakwaters; increasing the breakwater crest height by approximately 4 ft. The addition of capstones would decrease the average overtopping discharge for the high RSLC curve at the year 2082 from approximately 50 liters/s per m to approximately 2 liters/s per m. The adapted breakwater is anticipated to begin experiencing damages at the year 2109 with overtopping rates exceeding 50 liters/s per m and reaching 166 liters/s per m by the year 2125, the end of the adaptation horizon (Figure 39).

---

[3] Equation 6.6 from the EurOtop Manual is in metric and gives results in m^3/s per m length of structure. The equation was used in the standard metric form and results were converted to liters/s per m in order to utilize Table VI-5-6 Critical Values of Average Overtopping Discharge from EM 1110-2-1100 (USACE 2002).

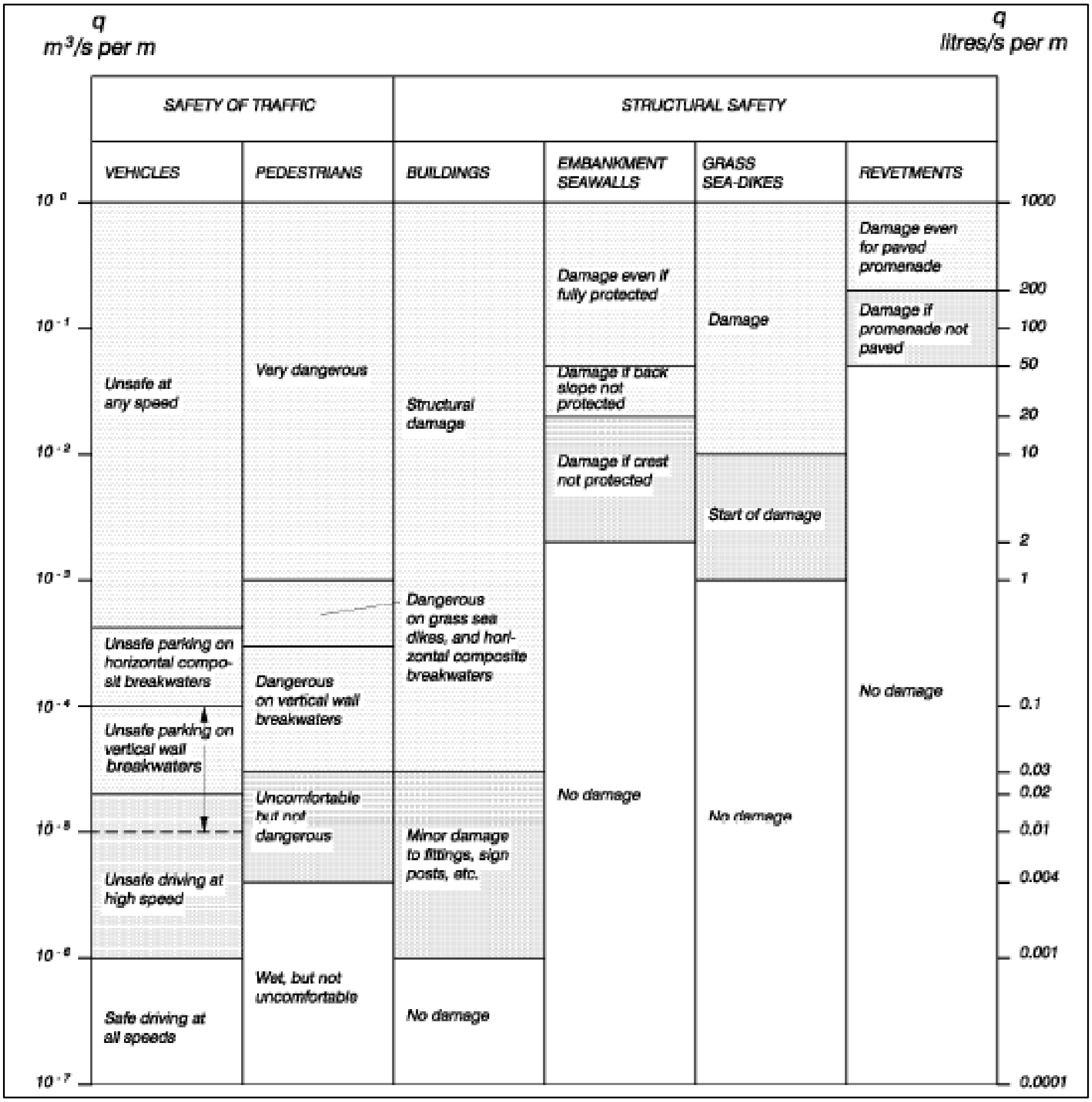


Figure 37. EM 1110-2-1100 Table VI-5-6 Critical Values of Average Overtopping Discharges (USACE 2002)

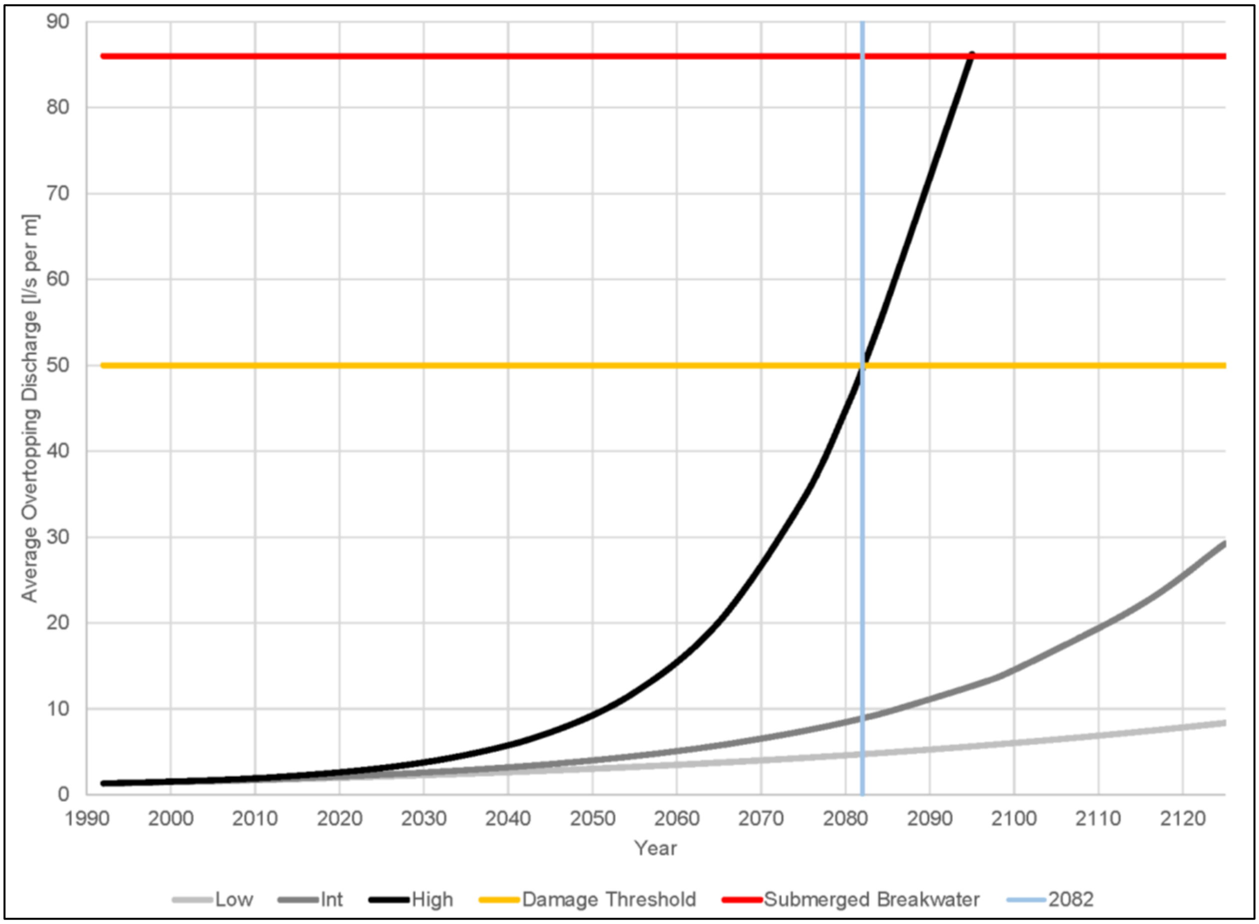


Figure 38. Adjusted Average Overtopping Discharge in Liters/s per m of Length of Breakwater, Response to RSLC Assuming 2% AEP Storm Surge and Design Wave Conditions Are Constant over Time, Looking at a 100 Year Project Adaptation Horizon

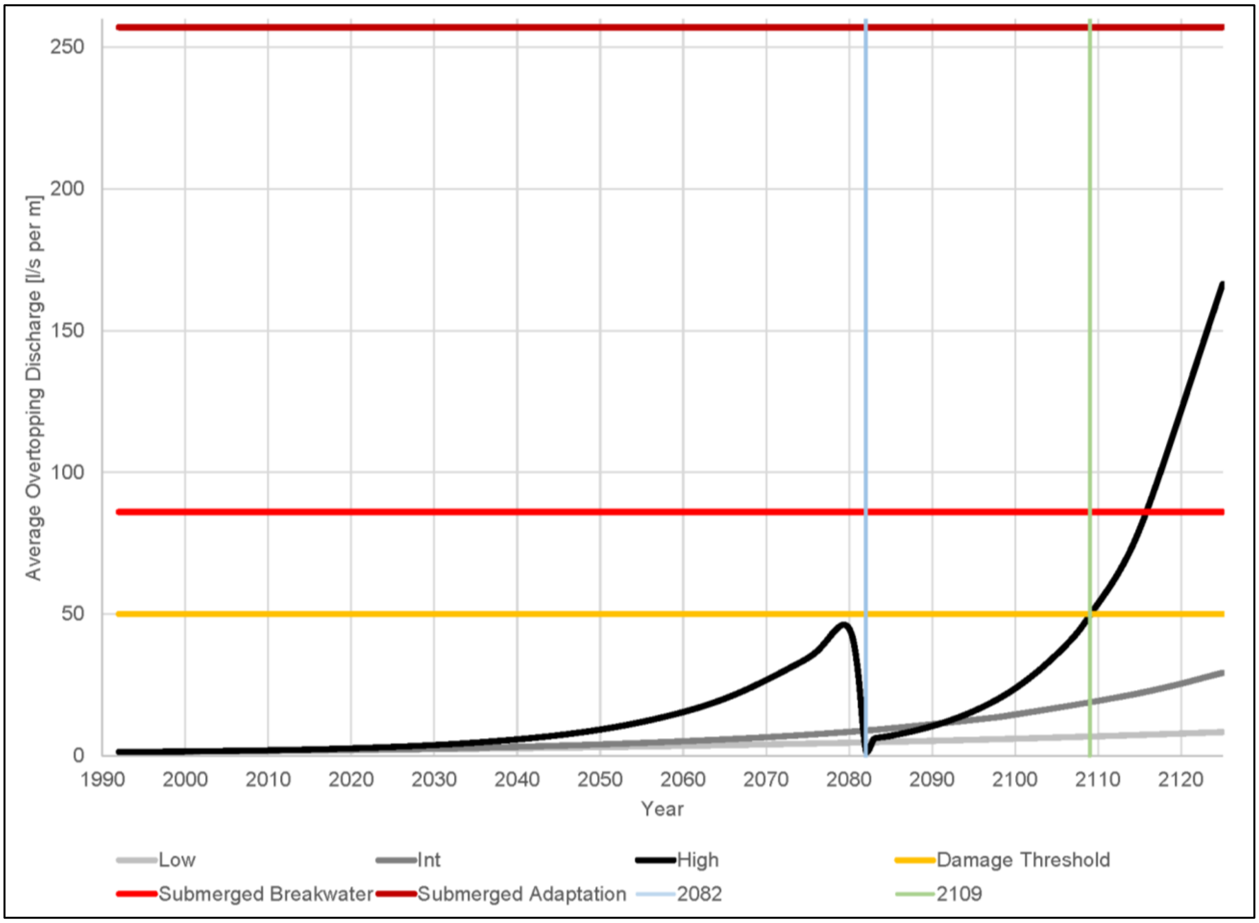


Figure 39. Adjusted Average Overtopping Discharge, in Liters/s per m of Length of Breakwater, Response to RSLC with Adaptive Measures at the Year 2082 for the High RSLC Scenario

The proposed local features have a maximum elevation of +16 ft MLLW. As indicated by the total water level (Section 8.1 Total Water Level), these features are anticipated to be flooded during 2% AEP storm surge events with the intermediate RSLC estimate by the year 2095 for the low curve, 2065 for the intermediate curve, and 2039 for the high curve (Figure 40). Front Street, which would connect the harbor access road to the community roads, has an elevation of approximately +21 ft MLLW. The flood of record occurred in October 1945 due to storm surge. A flood gage and high water elevation sign were placed on a utility pole that is no longer standing. The approximate extents of the flood of record are shown in Figure 41. Based on the survey completed in the summer of 2019 and Figure 41, the flood of record has an approximate elevation of +20 ft MLLW, which likely includes total water level and wave run-up. There is no indication of what the AEP of that flood event was. Based on the design information available, Front Street is not anticipated to be overtopped with a 2% AEP storm surge event except for the high RSLC curve in the year 2106 (Figure 40). In Figure 40, The horizontal yellow line indicates the elevation of the proposed uplands (+16.0 ft MLLW), and the horizontal red line indicated the approximate elevation of Front Street (+21 ft MLLW) where the access road ties into the local road system. The vertical blue line indicates the anticipated year of construction (2025), and the vertical green line indicates the end of the 50-year period of analysis (2075).Front Street and the local features may be flooded more frequently as RSLC is realized, and storms change as a result of climate change. There is an indication that the current total water level elevations combined with wave run-up inundate the low-lying area of Elim.

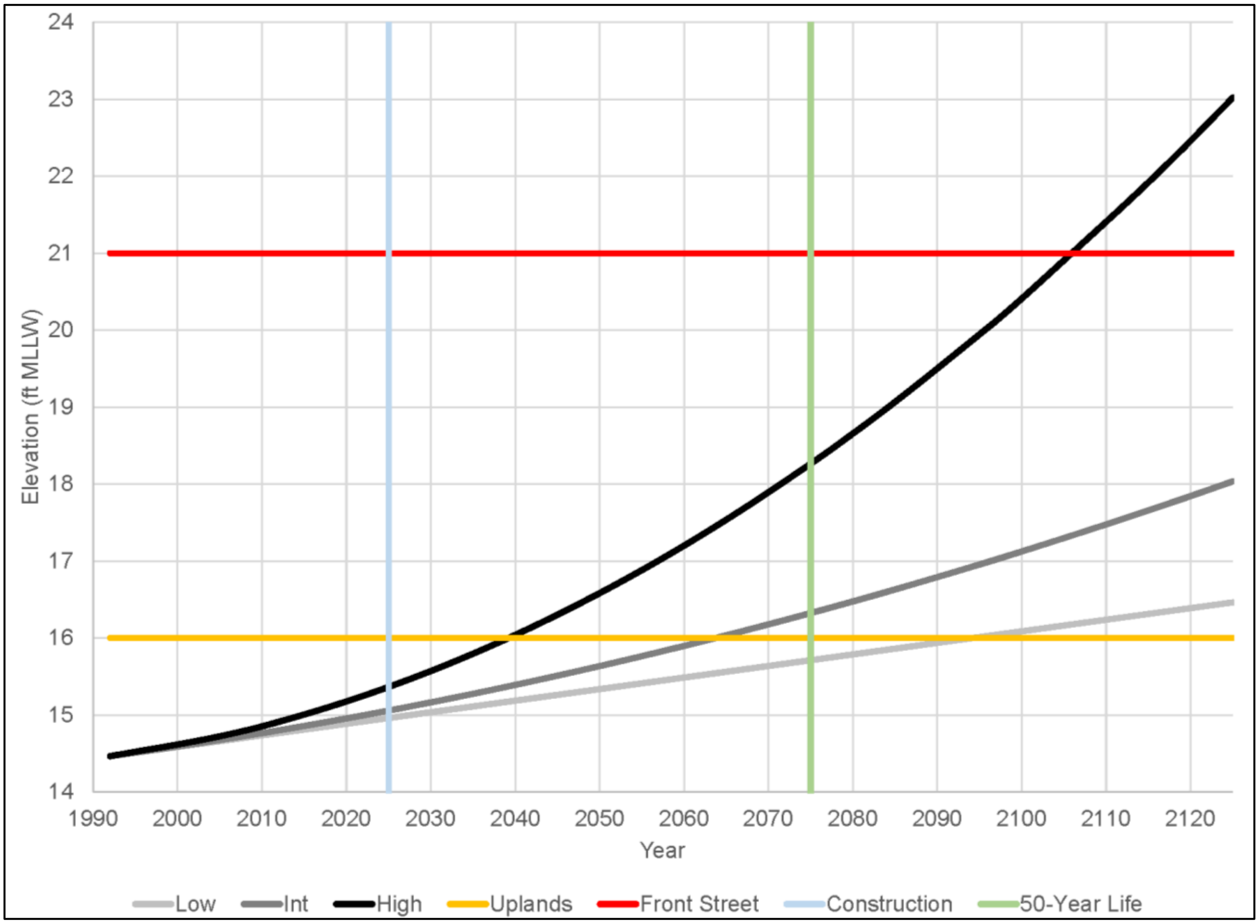


Figure 40. The Impact of RSLC on Total Water Level and Its Impact on Flooding of Local Features

This page was intentionally left blank.

Figure 41. Community Map of Elim with Approximate Flood of Record Traced in Cyan (Vertical Datum Unknown)

This page was intentionally left blank.

Figure 42. Elim Baseline Erosion Assessment - Linear Extent of Erosion (Alaska District 2009)

This page was intentionally left blank.

According to the Alaska Baseline Erosion Assessment, Elim Erosion Information Paper (Alaska District 2009), most erosion occurs where the shore is at its lowest elevation. The region of erosion is about 800 ft along the coast and inland to an estimated 50 ft above the high water line (Figure 42), experiencing 1–2 ft of shoreline loss over the few years prior to the assessment. However, it should be noted that the Elim Erosion Information Paper was completed based on a survey completed by the community; no data collection or data analysis was completed for the Elim Erosion Information and it represents a snapshot in time from 2008. The Alaska Baseline Erosion Assessment placed Elim in the group of communities that needed to be monitored for future erosion. Based on field observations and the CoastSat analysis (Section 9.2.1 Sediment Transport), the erosion anticipated in the Elim Erosion Information Paper has not been realized and the shoreline appears to be stable. Changing sea ice conditions and potential sea level rise (Section 5.4 Relative Sea Level Change) at the project site could result in changes to wave regime impacting the shoreline adjacent to the project site and proposed local features. The proposed navigation improvements could cause sheltering to the east of the breakwater and an increase in wave loading on the bluffs to the west. Per a comment received during the public review period, it is anticipated that there will be a decrease in erosion east of proposed navigation improvements, where erosion was noted in the Elim Erosion Information Paper. The combination of potential changes to the wave conditions due to climate change and diffraction and reflection to the west of proposed navigation improvements could result in an increase in wave loading on the bluffs to the west of Elim Beach.

Changes in shoreline would be limited to the pocket beach in front of Elim between Airport Point and headlands east of the community. The response of the shoreline to the three RSLC scenarios were analyzed using the Bruun Model (Bruun 1962) to evaluate the stability of the uplands and access roads. The Bruun Model predicts the horizontal recession of sandy shorelines due to RSLC (Figure 40) using the following equation:

$$\boldsymbol{R = \frac{L_*}{B+h_*} RSLC} \qquad \text{Equation 8}$$

where L_* is the cross-shore distance to depth h_*, and B is the height of the area on land that is eroded. The cross-shore distance (L) was assumed to go from the 10% AEP storm surge on top of MHHW to the depth of closure. The height of the land that is eroded (B) was assumed to be the vertical difference in MLLW and a 10% AEP storm surge on top of MHHW. The Bruun Model was only applied at Elim Beach since the shoreline at Airport Point is made up of rock outcrops and bedrock.

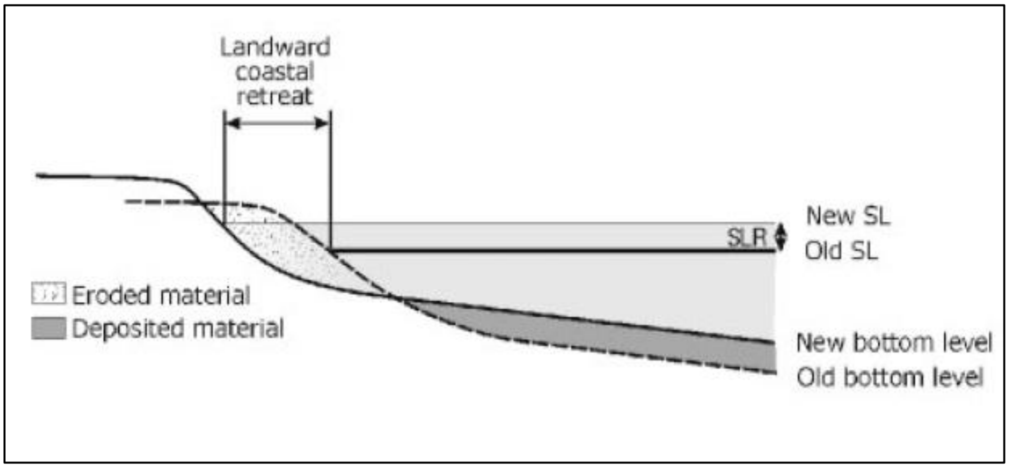


Figure 43. Schematic Illustration of the Bruun Model of Profile Response to Sea Level Rise Showing Erosion of the Upper Beach and Offshore Deposition (Figure 1 from (Davidson-Arnott 2005))

The Bruun Model indicates that the shoreline recession at Elim could be between 80 and 300 horizontal ft by the end of the period of analysis for the project (year 2075). For the 100-year adaptation horizon (USACE 2019), the Bruun Model indicated that the shoreline recession could be as much as 737 ft for the high RSLC scenario projected to 2125. Based on the presence of dense to very dense alluvium or weather rock at approximately 15–20 ft below existing ground on the upper beach and 10–15 ft below existing ground on the lower beach (Golder 2019), it is anticipated that the Bruun Model predictions are highly unlikely. Another limitation of the Bruun Model is that it was developed for sandy shorelines. As indicated in the beach test pits (Alaska District 2018), the Elim shoreline consists of sediment ranging from sand to cobble. The large beach sediment size indicates that the Bruun Model would over predict the shoreline recession at Elim.

The presence of less erodible material at approximately 15 ft below the existing ground is anticipated to limit the horizontal recession of the shoreline to 20–80 horizontal feet for the low and high RSLC scenario in the year 2075, respectively. The 100-year adaptation horizon indicated that shoreline recession could be 40–195 ft. With the absence of readily erodible material, RSLC is also anticipated to increase the actual depth of the entrance channel, turning basin, and moorage basin, decreasing maintenance dredging requirements. To reduce the likeliness of erosion of the access road and uplands, the edges of these features will be armored with rip rap. If the shoreline recession exceeds 120 ft, it is anticipated the Front Street will be damaged.

The potential impacts of increased overtopping of breakwaters, flooding of local features, erosion of the shoreline, and decrease of maintenance dredging are anticipated to impact any design at Elim Beach or Airport Point in the same way. To limit the impact that climate change may have on future conditions at the proposed harbor, the breakwater is designed for overtopping, shoreline features are armored, and discussions about the potential flooding of local features have occurred with the community. If shoreline adjustments are observed during monitoring efforts to the west of the proposed navigation improvements are observed, USACE and the Village of Elim could evaluate the potential placement of maintenance dredged material or local material along the toe of the bluff for beach nourishment.

10. ALTERNATIVES CONSIDERED IN DETAIL

Four potential sites for navigation improvements at Elim were selected during the charrette process at the beginning of the project. Potential sites were then screened down to sites that were within Elim. Several alternatives were considered for navigation improvements while working with two potential sites. Six plans were evaluated, along with a no-action alternative.

The following alternatives presented were used for screening of alternatives to determine the Tentatively Selected Plan (TSP). No updates were made to the alternatives based on review comments and no reformulation was completed. The updates suggested during the district quality control (DQC) review, agency technical review (ATR), public review, and policy and legal review would impact each alternative to the same magnitude and would not change the Recommended Plan. The alternatives presented in this section reflect the designs used for screening of the alternatives. All updates made based on review comments are presented in Section 11 Alternative 5 – Recommended Plan Optimized Design.

10.1. Alternative 1 – No-Action

This alternative would leave the community without any navigation improvements. Vessels would continue to use Moses Point and sustain time lost due to increased shoaling within the mouth of Kwiniuk River and low water events. Vessels would continue to experience damages during large storm surges due to vessels being pulled onto the beach or left in the water and swamped due to short storm notice. Time for subsistence and commercial fishing would continue to be lost, and navigation to safe moorage would continue to be inconsistent.

The fish buying station would stay in Moses Point, and ice would continue to be transported from Elim to Moses Point. The iced fish totes would continue to be lightered to an anchored tender offshore. Barges would continue to beach east of Elim Creek for offloading freight and anchor offshore on the west side of Elim Beach with a floating pipeline to deliver fuel. Wait time due to shallow water would not change or get worse due to shoaling, and environmental associated with fuel delivery would not be addressed.

10.2. Alternative 2 – Elim Beach: Commercial and Subsistence Fleet

This plan consists of the west rubble-mound breakwater, which would be 985 ft, and the east rubble-mound breakwater, which would be 457 ft, that would provide a 3.9-acre basin for 50 boats ranging in length from 18 ft to 32 ft. This plan would provide shelter from waves propagating out of the west through southwest and from the east (Figure 44).

The fish buying station would likely move to Elim, per conversations with NSEDC, but iced fish totes would continue to be lightered to an anchored tender offshore. Barges would continue to beach east of Elim Creek for offloading freight, and fuel delivery operations may need to change due to the location of fuel header and the presence of the harbor. Wait time for barges due to shallow water would not change, and environmental risk associated with fuel delivery would not be addressed.

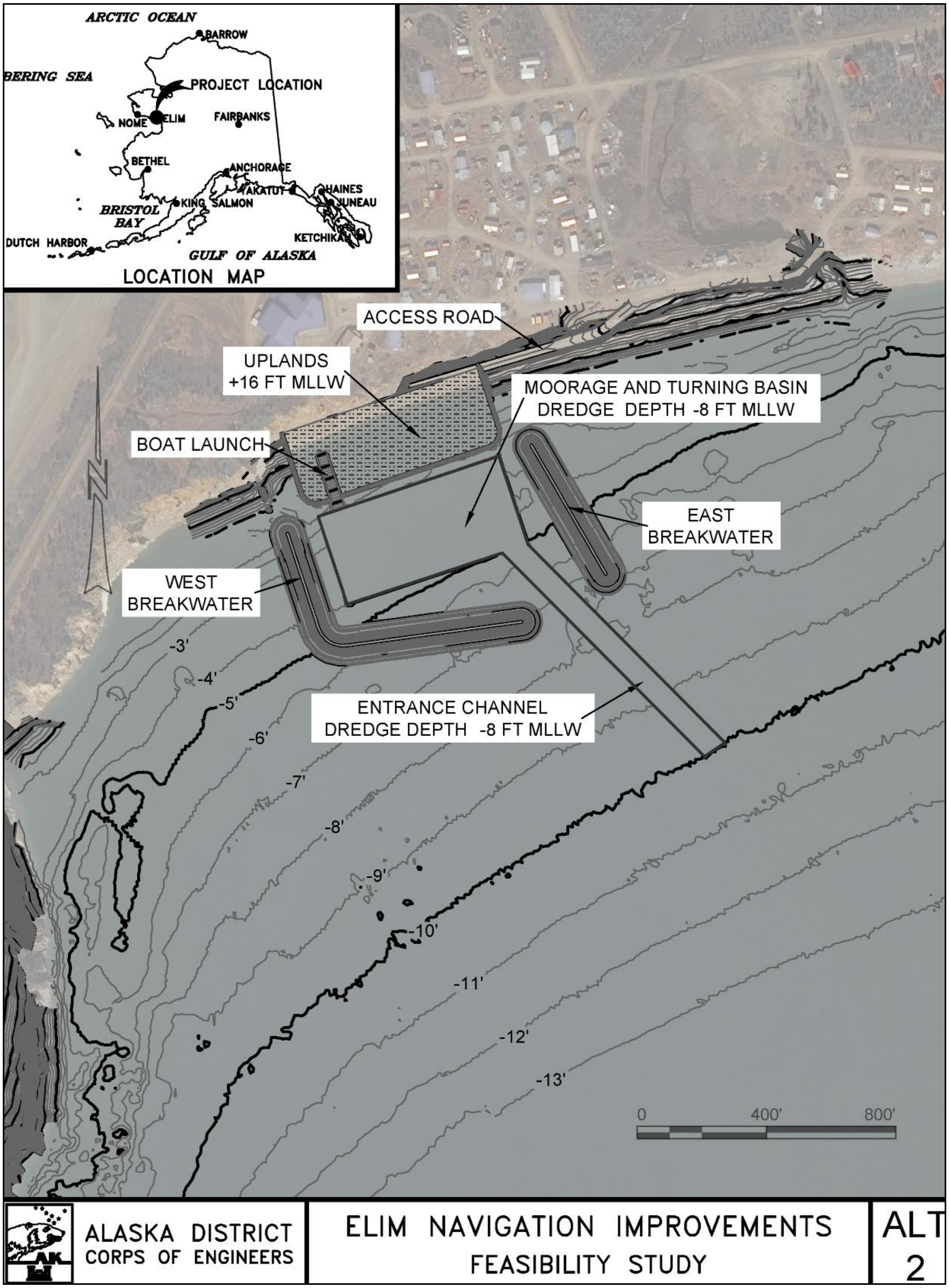


Figure 44. Alternative 2 Layout

10.2.1. Breakwaters

Stone size and crest elevation are described in Section 8 Breakwater Design Parameters. The breakwater would require approximately 25,600 cubic yards of core rock, 27,700 cubic yards of B rock, and 30,300 cubic yards of armor stone. The typical breakwater cross-section is shown in Figure 45.

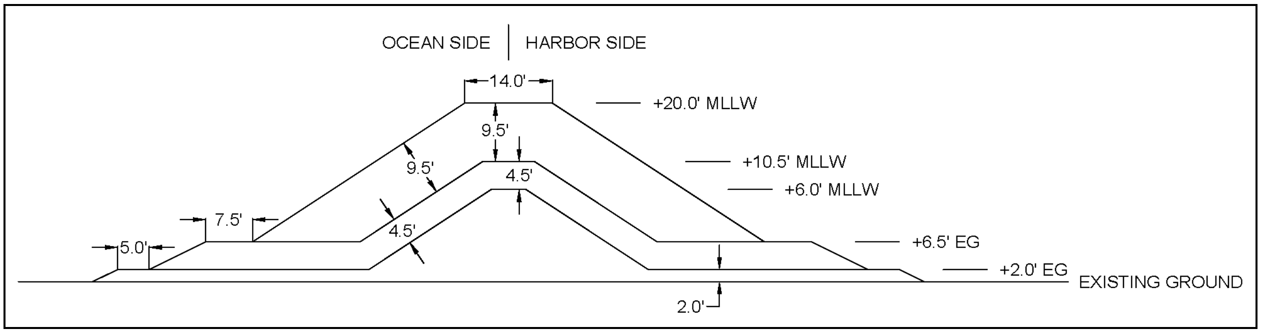


Figure 45. Typical Breakwater Cross-Section

10.2.2. Entrance Channel and Basin

The design vessel used for this alternative is the commercial vessel (Table 5). The entrance channel and moorage basin would be dredged to -8 ft MLLW (Table 8). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The entrance channel and basin would require approximately 46,800 cubic yards of dredging; of this, 19% is expected to be difficult mechanical dredging through weathered bedrock (Appendix B: Geotechnical).

10.2.3. Wave Reduction

The wave height impacting the project site from the northeast through the southwest was determined by applying the deep water design wave and fetch-limited wave. A diffraction analysis was then used to determine the wave heights within the harbor. For Alternative 2, the worst wave conditions within the harbor occur when the sustained wind is blowing from the southeast, which could cause wave heights of up to 6.2 ft in the moorage area and 4.6 ft at the boat launch. When waves are from the east, south, and southwest, the waves in the moorage area are anticipated to be 4.6–0.7 ft, and waves at the boat launch are anticipated to be 2.5–0.4 ft, close to or above the allowable maximum wave height.

10.2.4. Water Quality and Circulation

The flushing analysis (Section 9.1 Water Quality and Circulation) results in an aspect ratio of 1:2.1 and an area ratio of 280. Both meet the minimum recommendations for a basin design, which are 1:4 and greater than 200, respectively.

10.2.5. Maintenance

It is anticipated that approximately 18,000 cubic yards of maintenance dredging will be required every 25 years. There may be minor stone movement along the outside of the breakwaters due to ice ride up. It is anticipated that approximately 2.5% of the armor stone, 750 cubic yards, will need to be replaced every 25 years.

10.2.6. Local Features

The final design and construction of all local features would be the responsibility of the local sponsor. A 3.2-acre uplands would be required to provide parking, boat storage, and turn-around for the boat launch. For the feasibility level design, it was assumed that the elevation of the uplands would be +16 ft MLLW and include armored slopes. An 800-ft long gravel road would be required from Front Street to the harbor uplands. A single boat launch would be required to allow for the launching of boats. It would also enable boat owners to pull their boats onshore if the storm surge and wave conditions are anticipated to be greater than the harbor design conditions. For the feasibility level design, it was assumed that the boat launch would have a 13% slope from the +16 ft MLLW of the uplands to -5 ft MLLW enabling use during most tide conditions.

10.3. Alternative 3 – Elim Beach: Commercial and Subsistence Fleet with One Tender

This plan consists of the west rubble-mound breakwater, which would be 1,068 ft, and the east rubble-mound breakwater, which would be 463 ft, that would provide a 4.6-acre basin for 51 boats ranging in from 18 ft to 66 ft. This plan would provide shelter from waves propagating out of the west through southwest and from the east (Figure 46).

Barges would continue to beach east of Elim Creek for offloading freight, and fuel delivery operations may need to change due to the location of fuel header and the presence of the harbor. Wait time for barges due to shallow water would not change, and environmental risk associated with fuel delivery would not be addressed.

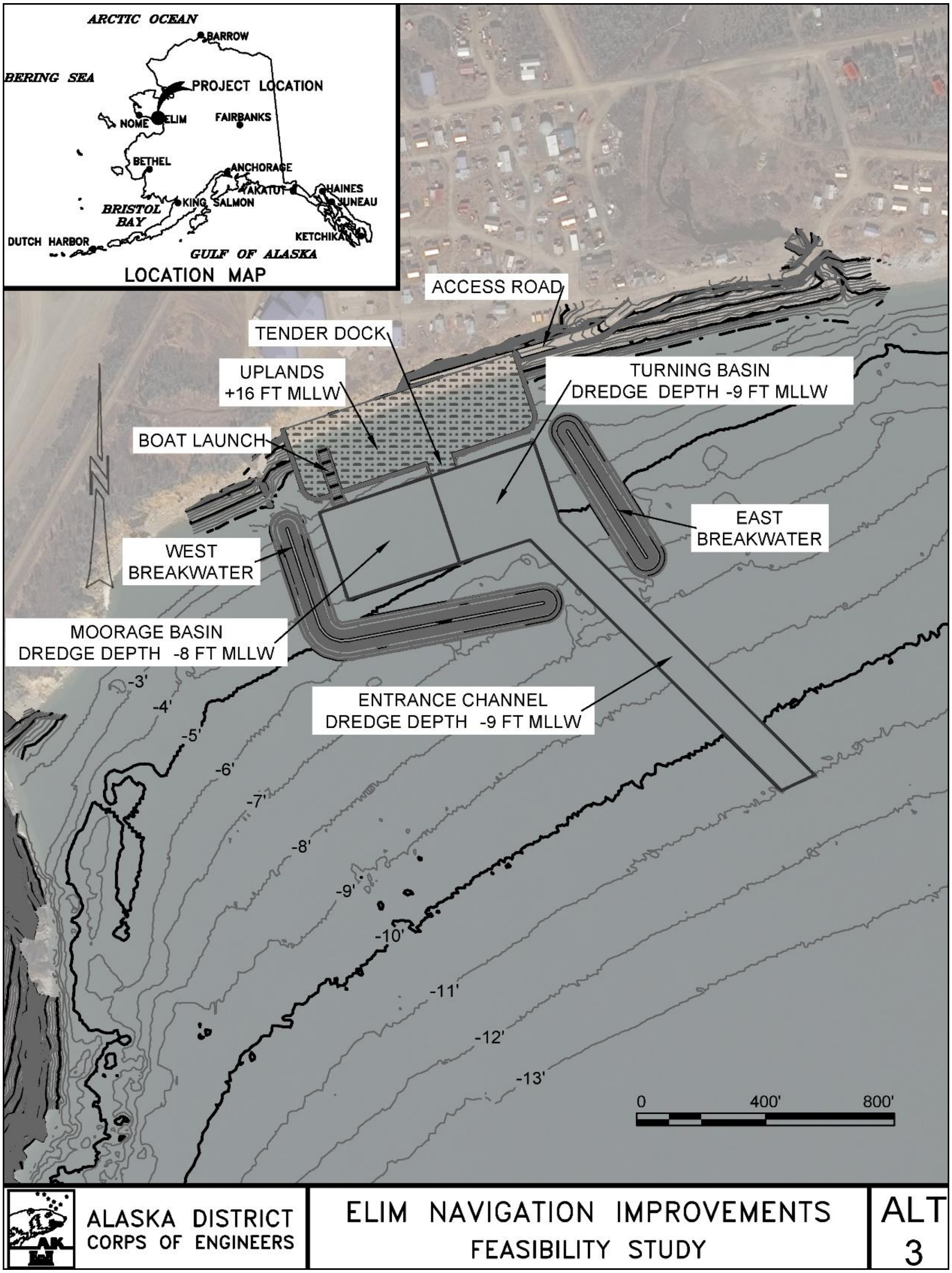


Figure 46. Alternative 3 Layout

10.3.1. Breakwaters

Stone size and crest elevation are described in Section 8 Breakwater Design Parameters. The breakwater would require approximately 26,500 cubic yards of core rock, 29,900 cubic yards of B rock, and 32,100 cubic yards of armor stone. The typical breakwater cross-section is shown in Figure 45.

10.3.2. Entrance Channel and Basin

The design vessel used for this alternative's entrance channel is the tender (Table 5). The entrance channel would be dredged to -9 ft MLLW (Table 9). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The entrance channel would require approximately 26,200 cubic yards of dredging, of this 48% is expected to be difficult mechanical dredging through weathered bedrock (see Appendix B: Geotechnical).

The design vessel used for this alternative's moorage basin is the commercial vessel (Table 5). The basin would be dredged to -8 ft MLLW (Table 8). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The basin would require approximately 26,700 cubic yards of dredging, of this 9% is expected to be difficult mechanical dredging through weathered bedrock (see Appendix B: Geotechnical).

10.3.3. Wave Reduction

The wave height impacting the project site from the northeast through the southwest was determined by applying the deep water design wave and fetch-limited wave. A diffraction analysis was then used to determine the wave heights within the harbor. For Alternative 3, the worst wave conditions within the harbor occur when deep water waves from the southeast and sustained wind from the southeast impact the navigation improvements, causing wave heights of up to 6.2 ft at the tender dock, 5.0 ft in the moorage area, and 4.6 ft at the boat launch. When waves are from the east, south, and southwest, the waves at the tender dock are anticipated to be below the allowable maximum wave height. The waves in the moorage area are anticipated to be 4.4–0.7 ft, and waves at the boat launch are anticipated to be 2.5–0.4 ft, close to or above the allowable maximum wave height.

10.3.4. Water Quality and Circulation

The flushing analysis (Section 9.1 Water Quality and Circulation) results in an aspect ratio of 1:2.5 and area ratio of 209. Both meet the minimum recommendations for a basin design, which are 1:4 and greater than 200, respectively.

10.3.5. Maintenance

It is anticipated that approximately 23,000 cubic yards of maintenance dredging will be required every 25 years. There may be minor stone movement along the outside of the breakwaters due to ice ride up. It is anticipated that approximately 2.5% of the armor stone, 800 cubic yards, will need to be replaced every 25 years.

10.3.6. Local Features

The final design and construction of all local features would be the responsibility of the local sponsor. A 3.9-acre uplands would be required to provide parking, boat storage, and turn-around for the boat launch. For the feasibility level design, it was assumed that the elevation of the uplands would be +16 ft MLLW and include armored slopes. An 800-ft long gravel road would be required from Front Street to the harbor uplands. A single boat launch would be required to allow for the launching of boats. It would also enable boat owners to pull their boats onshore if the storm surge and wave conditions are anticipated to be greater than the harbor design conditions. For the feasibility level design, it was assumed that the boat launch would have a 13% slope from the +16 ft MLLW of the uplands to -5 ft MLLW enabling use during most tide conditions.

A single tender dock would be required to allow for the loading of fish totes onto an NSEDC tender from the fish buying station. The tender dock would be an 87-ft long sheetpile dock at an elevation of +16 ft MLLW with the sheetpile driven to bedrock, approximately 12 ft below existing ground.

10.4. Alternative 4 – Elim Beach: Commercial and Subsistence Fleet with Two Tenders

This plan consists of the west rubble-mound breakwater, which would be 1,099 ft, and the east rubble-mound breakwater, which would be 463 ft, that would provide a 5.1-acre basin for 52 boats ranging in length from 18 ft to 66 ft. This plan would provide shelter from waves propagating out of the west through southwest and from the east (Figure 47).

Barges would continue to beach east of Elim Creek for offloading freight, and fuel delivery operations may need to change due to the location of fuel header and the presence of the harbor. Wait time for barges due to shallow water would not change, and environmental risk associated with fuel delivery would not be addressed.

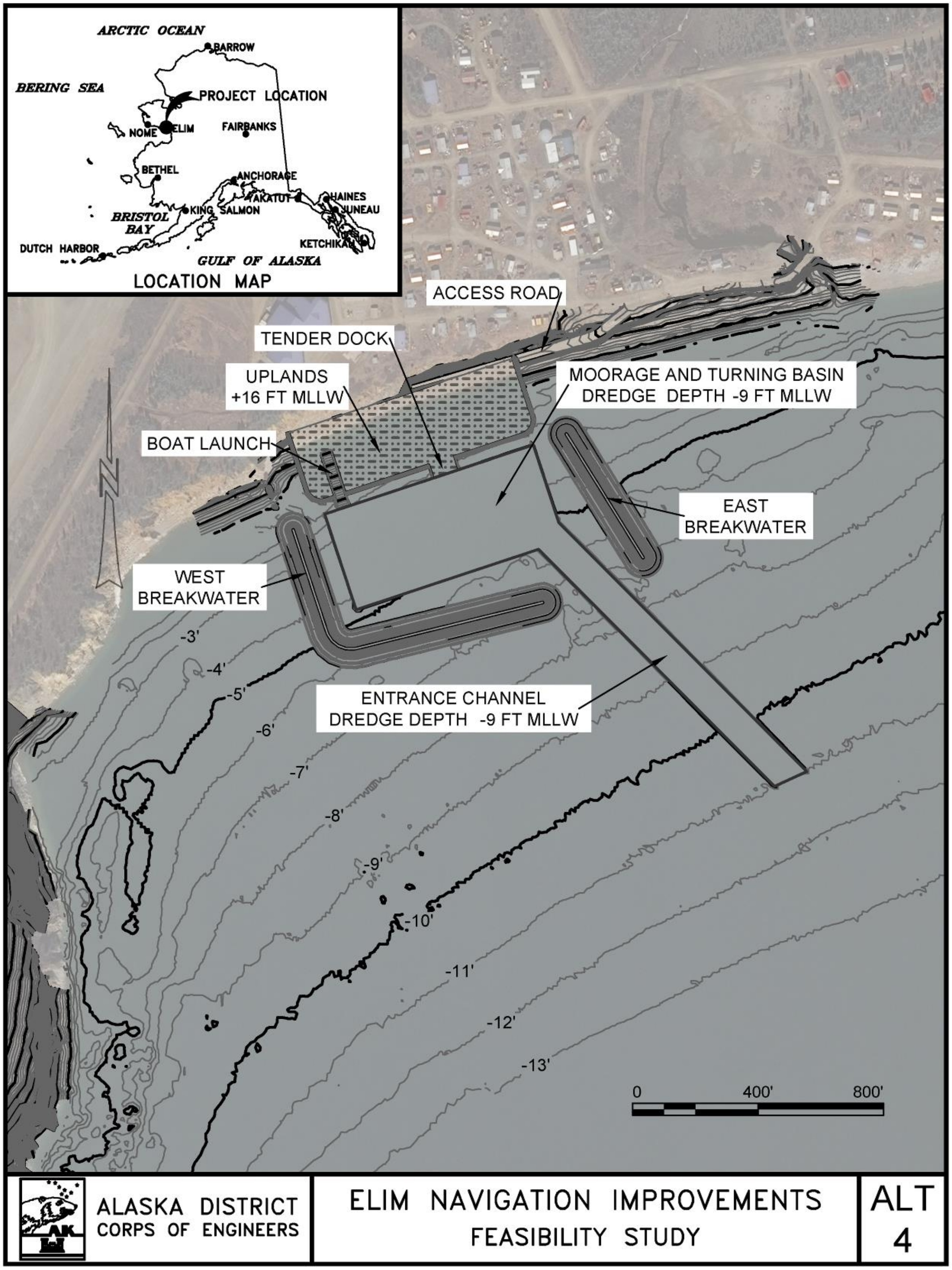


Figure 47. Alternative 4 Layout

10.4.1. Breakwaters

Stone size and crest elevation are described in Section 8 Breakwater Design Parameters. The breakwater would require approximately 28,000 cubic yards of core rock, 29,700 cubic yards of B rock, and 32,800 cubic yards of armor stone. The typical breakwater cross-section is shown in Figure 45.

10.4.2. Entrance Channel and Basin

The design vessel used for this alternative is the tender (Table 5). The entrance channel and moorage basin would be dredged to -9 ft MLLW (Table 9). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The entrance channel and basin would require approximately 72,100 cubic yards of dredging; of this, 27% is expected to be difficult mechanical dredging through weathered bedrock (see Appendix B: Geotechnical).

10.4.3. Wave Reduction

The wave height impacting the project site from the northeast through the southwest was determined by applying the deep water design wave and fetch-limited wave. A diffraction analysis was then used to determine the wave heights within the harbor. For Alternative 4, the worst wave conditions within the harbor occur when deep water waves from the southeast and sustained wind from the southeast impact the navigation improvements, causing wave heights of up to 6.2 ft at the tender dock, 5.6 ft in the moorage area, and 4.6 ft at the boat launch. When waves are from the east, south, and southwest, the waves at the tender dock are anticipated to be below the allowable maximum wave height. The waves in the moorage area are anticipated to be 4.4–0.7 ft, and waves at the boat launch are anticipated to be 2.5–0.4 ft, close to or above the allowable maximum wave height.

10.4.4. Water Quality and Circulation

The flushing analysis (Section 9.1 Water Quality and Circulation) results in an aspect ratio of 1:2.2 and an area ratio of 186. The aspect ratio meets the minimum recommendations for a basin design, 1:4, but the area ratio is less than the minimum recommendation of 200. As described in the *Planning and Design Guidelines for Small Craft Harbors* (ASCE 2012), wider entrance channels do not lead to uniform flushing. Still, alternative flushing analysis indicates that there will be good flushing within the proposed harbor. The harbor may have stagnant water in the corners of the harbor that would likely flush with storm surge recession.

10.4.5. Maintenance

It is anticipated that approximately 25,000 cubic yards of maintenance dredging will be required every 27 years. There may be minor stone movement along the outside of the breakwaters due to ice ride up. It is anticipated that approximately 2.5% of the armor stone, 800 cubic yards, will need to be replaced every 25 years.

10.4.6. Local Features

The final design and construction of all local features would be the responsibility of the local sponsor. A 3.9-acre uplands would be required to provide parking, boat storage, and turn-around for the boat launch. For the feasibility level design, it was assumed that the elevation of the uplands would be +16 ft MLLW and include armored slopes. An 800-ft long gravel road would be required from Front Street to the harbor uplands. A single boat launch would be required to allow for the launching of boats. It would also enable boat owners to pull their boats onshore if the storm surge and wave conditions are anticipated to be greater than the harbor design conditions. For the feasibility level design, it was assumed that the boat launch would have a 13% slope from the +16 ft MLLW of the uplands to -5 ft MLLW enabling use during most tide conditions.

A single tender dock would be required to allow for the loading of fish totes onto an NSEDC tender from the fish buying station. The tender dock would be an 87-ft long sheetpile dock at an elevation of +16 ft MLLW with the sheetpile driven to bedrock, approximately 12 ft below existing ground.

10.5. Alternative 5 – Elim Beach: Commercial and Subsistence Fleet with Two Tenders and Barge Access

This plan consists of the west rubble-mound breakwater, which would be 1,082 ft, and the east rubble-mound breakwater, which would be 468 ft, that would provide a 6.2-acre basin for 54 boats ranging in from 18 ft to 160 ft. This plan would provide shelter from waves propagating out of the west through southwest and from the east (Figure 48).

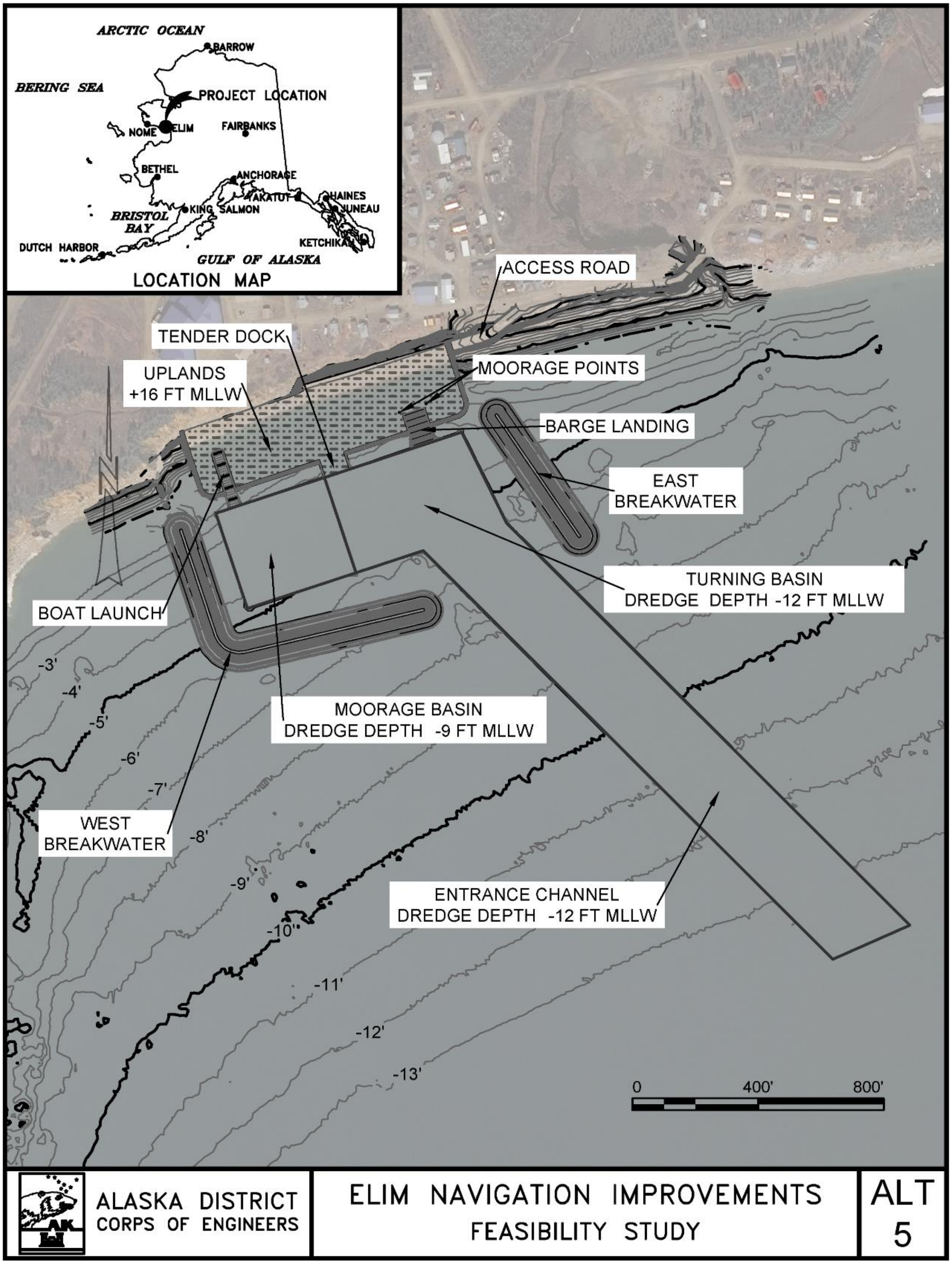


Figure 48. Alternative 5 Layout

10.5.1. Breakwaters

Stone size and crest elevation are described in Section 8 Breakwater Design Parameters. The breakwater would require approximately 27,700 cubic yards of core rock, 29,500 cubic yards of B rock, and 32,600 cubic yards of armor stone. The typical breakwater cross-section is shown in Figure 45.

10.5.2. Entrance Channel and Basin

The design vessel used for this alternative's entrance channel is the barge and tug (Table 5). The entrance channel would be dredged to -12 ft MLLW (Table 10). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The entrance channel would require approximately 124,100 cubic yards of dredging; of this, 46% is expected to be difficult mechanical dredging through weathered bedrock (see Appendix B: Geotechnical), and 1% is expected to require drilling and blasting.

The design vessel used for this alternative's moorage basin is the tender (Table 5). The basin would be dredged to -9 ft MLLW (Table 9). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The basin would require approximately 34,200 cubic yards of dredging, of this 15% is expected to be difficult mechanical dredging through weathered bedrock (see Appendix B: Geotechnical).

10.5.3. Wave Reduction

The wave height impacting the project site from the northeast through the southwest was determined by applying the deep water design wave and fetch-limited wave. A diffraction analysis was then used to determine the wave heights within the harbor. For Alternative 5, the worst wave conditions within the harbor occur when deep water waves from the southeast and sustained wind from the southeast impact the navigation improvements, causing wave heights of up to 6.2 ft at the tender dock and in the moorage area and 4.6 ft at the boat launch. When waves are from the east, south, and southwest, the waves at the tender dock are anticipated to be below the allowable maximum wave height. The waves in the moorage area are anticipated to be 4.2–0.7 ft, and waves at the boat launch are anticipated to be 2.9–0.4 ft, close to or above the allowable maximum wave height.

10.5.4. Water Quality and Circulation

The flushing analysis (Section 9.1 Water Quality and Circulation) results in an aspect ratio of 1:2.6 and an area ratio of 88. The aspect ratio meets the minimum recommendations for a basin design, 1:4, but the area ratio is less than the minimum recommendation of 200. As described in the *Planning and Design Guidelines for Small Craft Harbors* (ASCE 2012), wider entrance channels do not lead to uniform flushing.

Still, alternative flushing analysis indicates that there will be good flushing within the proposed harbor. The harbor may have stagnant water in the corners of the harbor that would likely flush with storm surge recession.

10.5.5. Maintenance

It is anticipated that approximately 51,000 cubic yards of maintenance dredging will be required every 30 years. There may be minor stone movement along the outside of the breakwaters due to ice ride up. It is anticipated that approximately 2.5% of the armor stone, 800 cubic yards, will need to be replaced every 25 years.

10.5.6. Local Features

The final design and construction of all local features would be the responsibility of the local sponsor. A 4-acre uplands would be required to provide parking, boat storage, and turn-around for the boat launch. For the feasibility level design, it was assumed that the elevation of the uplands would be +16 ft MLLW and include armored slopes. An 800-ft long gravel road would be required from Front Street to the harbor uplands. A single boat launch would be required to allow for the launching of boats. It would also enable boat owners to pull their boats onshore if the storm surge and wave conditions are anticipated to be greater than the harbor design conditions. For the feasibility level design, it was assumed that the boat launch would have a 13% slope from the +16 ft MLLW of the uplands to -5 ft MLLW enabling use during most tide conditions.

A single tender dock would be required to allow for the loading of fish totes onto an NSEDC tender from the fish buying station. The tender dock would be an 87-ft long sheetpile dock at an elevation of +16 ft MLLW with the sheetpile driven to bedrock, approximately 12 ft below existing ground.

A barge landing that is 70 ft wide with a 1:4 slope from -5 ft MLLW up to +16 ft MLLW would be required to allow for barge loading and unloading. It is assumed that the style of barge and the method of offloading and loading, driving a loader and/or telescopic handler from the barge onto land via the barge ramp (Figure 49), would not change due to the existence of a harbor. Two moorage points would be required for the barge to moor to while offloading or loading.

Figure 49. Offloading of a Materials Barge on Elim Beach (August 2018)

10.6. Alternative 6 – Airport Point: Commercial and Subsistence Fleet

This plan consists of the west rubble-mound breakwater, which would be 819 ft, and the east rubble-mound breakwater, which would be 418 ft, that would provide a 3-acre basin for 50 boats ranging in from 18 ft to 32 ft. This plan would provide shelter from waves propagating out of the west through south and from the east (Figure 50).

The fish buying station would likely move to Airport Point, per conversations with NSEDC, but ice would need to be transported from Elim to Airport Point. Iced fish totes would continue to be lightered to an anchored tender offshore. Barges would continue to beach east of Elim Creek for offloading freight and anchor offshore on the west side of Elim Beach with a floating pipeline to deliver fuel. Wait time for barges due to shallow water would not change, and environmental risk associated with fuel delivery would not be addressed.

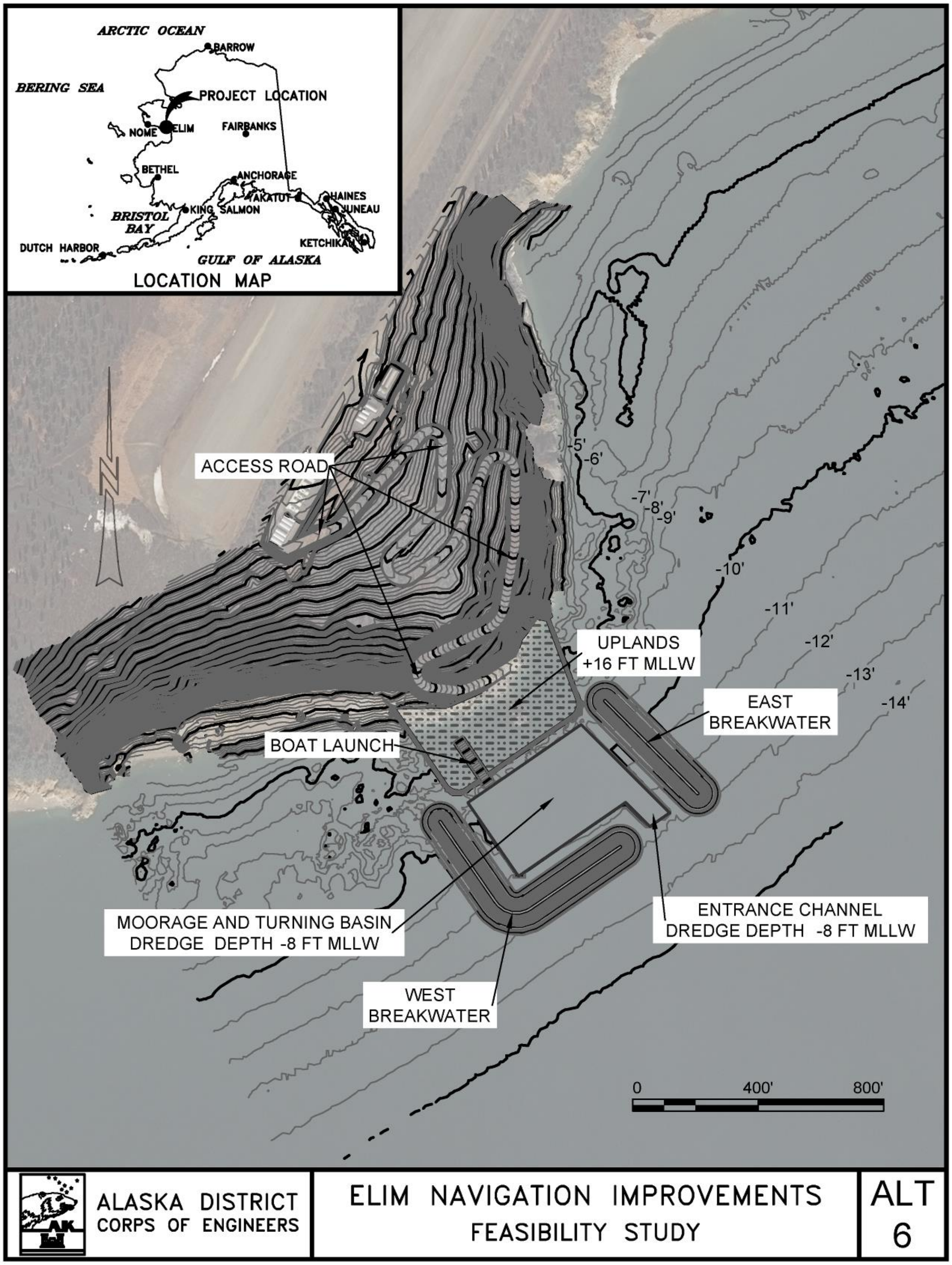


Figure 50. Alternative 6 Layout

10.6.1. Breakwaters

Stone size and crest elevation are described in Section 8 Breakwater Design Parameters. The breakwater would require approximately 37,100 cubic yards of core rock, 29,400 cubic yards of B rock, and 37,100 cubic yards of armor stone. The typical breakwater cross-section is shown in Figure 45.

10.6.2. Entrance Channel and Basin

The design vessel used for this alternative is the commercial vessel (Table 5). The entrance channel and moorage basin would be dredged to -8 ft MLLW (Table 8). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The entrance channel and basin would require approximately 2,600 cubic yards of dredging, of this 2% is expected to be difficult mechanical dredging through weathered bedrock (see Appendix B: Geotechnical).

10.6.3. Wave Reduction

The wave height impacting the project site from the northeast through the southwest was determined by applying the deep water design wave and fetch-limited wave. A diffraction analysis was then used to determine the wave heights within the harbor. For Alternative 6, the worst wave conditions within the harbor occur when deep water waves from the south reach the navigation improvements, which could cause wave heights of up to 6.8 ft in the moorage area and 3.7 ft at the boat launch. When waves are from the east, southeast, and southwest, the waves in the moorage area are anticipated to be 4.5–0.9 ft, and waves at the boat launch are anticipated to be 2.7–0.5 ft, close to or above the allowable maximum wave height.

10.6.4. Water Quality and Circulation

The flushing analysis (Section 9.1 Water Quality and Circulation) results in an aspect ratio of 1:1.5 and the area ratio of 209. Both meet the minimum recommendations for a basin design, which are 1:4 and greater than 200, respectively.

10.6.5. Maintenance

No maintenance dredging is anticipated due to the natural depths observed at Airport Point. There may be minor stone movement along the outside of the breakwaters due to ice ride up. It is anticipated that approximately 2.5% of the armor stone, 900 cubic yards, will need to be replaced every 25 years.

10.6.6. Local Features

The final design and construction of all local features would be the responsibility of the local sponsor. There is a limited flat accessible area at Airport Point. A 3.3-acre uplands would be required to provide parking and turn-around for the boat launch. For the feasibility level design, it was assumed that the elevation of the uplands would be +16 ft MLLW and include armored slopes. A 0.6-mile-long gravel road would be required from the tank farm south of Elim Airport to the harbor uplands. A single boat launch would be required to allow for the launching of boats every spring and removal every fall. It would also enable boat owners to pull their boats onshore if the storm surge and wave conditions are anticipated to be greater than the harbor design conditions. For the feasibility level design, it was assumed that the boat launch would have a 13% slope from the +16 ft MLLW of the uplands to -5 ft MLLW enabling use during most tide conditions.

There are no existing utilities located at Airport Point. To provide electricity and/or water for a harbor at Airport Point, utilities would need to be run from Elim Airport down to the harbor or along the bluff from the school out to the harbor.

10.7. Alternative 7 – Airport Point: Commercial and Subsistence Fleet with Two Tenders and Barge Access

This plan consists of the west rubble-mound breakwater, which would be 1,137 ft, and the east rubble-mound breakwater, which would be 594 ft, that would provide a 6-acre basin for 54 boats ranging in from 18 ft to 160 ft. This plan would provide shelter from waves propagating out of the west through south and from the east (Figure 51).

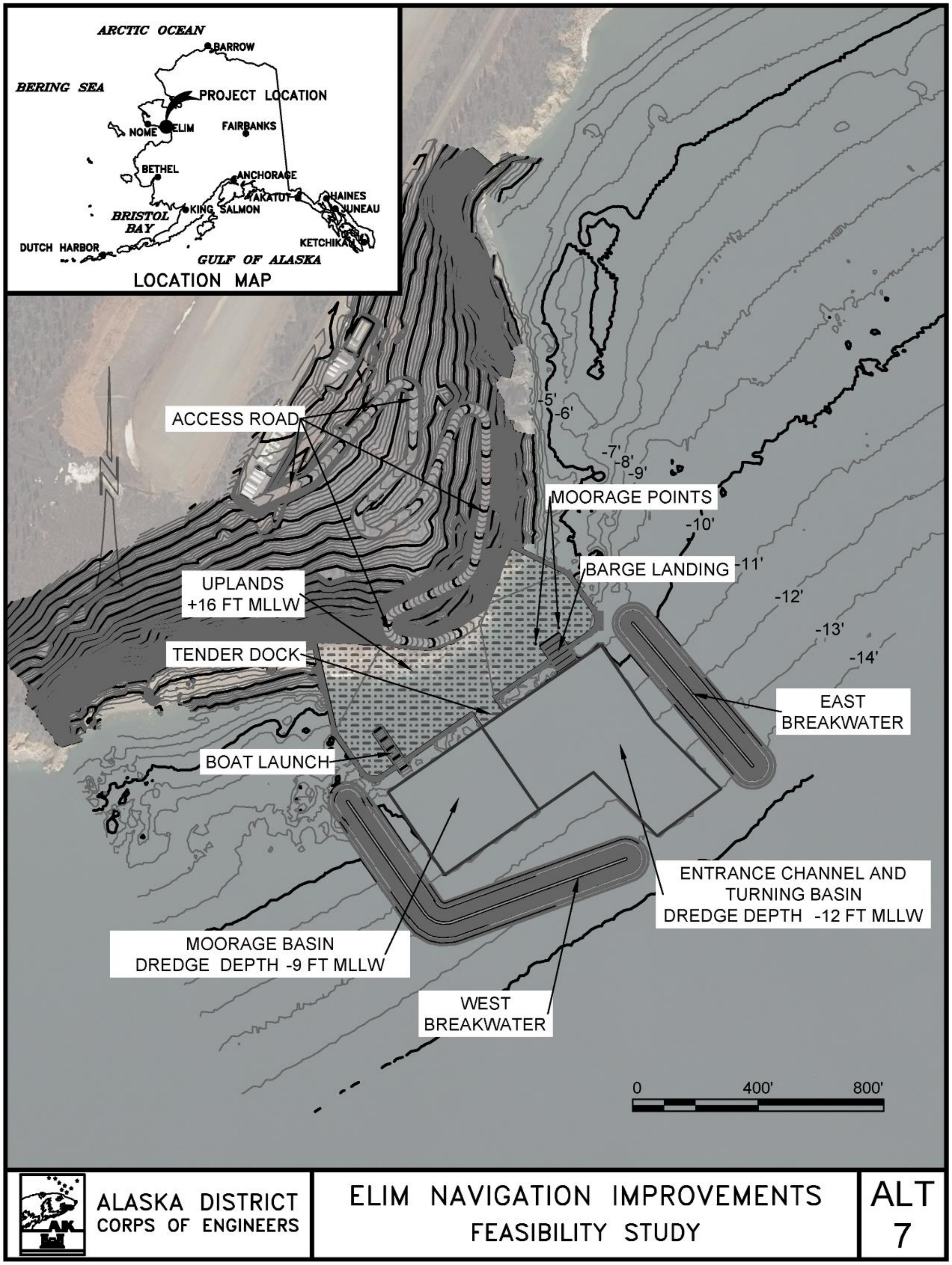


Figure 51. Alternative 7 Layout

10.7.1. Breakwaters

Stone size and crest elevation are described in Section 8 Breakwater Design Parameters. The breakwater would require approximately 55,200 cubic yards of core rock, 42,500 cubic yards of B rock, and 56,000 cubic yards of armor stone. The typical breakwater cross-section is shown in Figure 45.

10.7.2. Entrance Channel and Basin

The design vessel used for this alternative's entrance channel is the barge and tug (Table 5). The entrance channel would be dredged to -12 ft MLLW (Table 10). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The entrance channel would require approximately 22,400 cubic yards of dredging, and 90% is expected to be difficult mechanical dredging through weathered bedrock (see Appendix B: Geotechnical), and 0.5% is expected to require drilling and blasting.

The design vessel used for this alternative's moorage basin is the tender (Table 5). The basin would be dredged to -9 ft MLLW (Table 9). A 2-ft over-dredge allowance would be provided to ensure minimum depth is attained. The basin would require approximately 2,900 cubic yards of dredging, of this 32% is expected to be difficult mechanical dredging through weathered bedrock (see Appendix B: Geotechnical).

10.7.3. Wave Reduction

The wave height impacting the project site from the northeast through the southwest was determined by applying the deep water design wave and fetch-limited wave. A diffraction analysis was then used to determine the wave heights within the harbor. For Alternative 7, the worst wave conditions within the harbor occur when fetch-limited waves from the east reach the navigation improvements, which could cause wave heights of up to 4.9 ft in the moorage area and 3.6 ft at the boat launch. When waves are from the east, southeast, and southwest, the waves at the tender dock are anticipated to be 4.3–0.6 ft. The waves in the moorage area are anticipated to be 3.7–0.5 ft, and waves at the boat launch are anticipated to be 1.9–0.4 ft, close to or above the allowable maximum wave height.

10.7.4. Water Quality and Circulation

The flushing analysis (Section 9.1 Water Quality and Circulation) results in an aspect ratio of 1:2.6 and the area ratio of 88. The aspect ratio meets the minimum recommendations for a basin design, 1:4, but the area ratio is less than the minimum recommendation of 200. As described in the *Planning and Design Guidelines for Small Craft Harbors* (ASCE 2012), wider entrance channels do not lead to uniform flushing. Still, alternative flushing analysis indicates that there will be good flushing within the proposed harbor. The harbor may have stagnant water in the corners of the harbor that would likely flush with storm surge recession.

10.7.5. Maintenance

It is anticipated that approximately 10,000 cubic yards of maintenance dredging will be required every 30 years. There may be minor stone movement along the outside of the breakwaters due to ice ride up. It is anticipated that approximately 2.5% of the armor stone, 1,400 cubic yards, will need to be replaced every 25 years.

10.7.6. Local Features

The final design and construction of all local features would be the responsibility of the local sponsor. There is a limited flat accessible area at Airport Point. A 6.2-acre uplands would be required to provide parking and turn-around for the boat launch. For the feasibility level design, it was assumed that the elevation of the uplands would be +16 ft MLLW and include armored slopes. A 0.6-mile-long gravel road would be required from the tank farm south of Elim Airport to the harbor uplands. A single boat launch would be required to allow for the launching of boats every spring and removal every fall. It would also enable boat owners to pull their boats onshore if the storm surge and wave conditions are anticipated to be greater than the harbor design conditions. For the feasibility level design, it was assumed that the boat launch would have a 13% slope from the +16 ft MLLW of the uplands to -5 ft MLLW enabling use during most tide conditions.

A single tender dock would be required to allow for the loading of fish totes onto an NSEDC tender from the fish buying station. The tender dock would be an 87-ft long sheetpile dock at an elevation of +16 ft MLLW with the sheetpile driven to bedrock, approximately 12 ft below existing ground.

A barge landing that is 70 ft wide with a 1:4 slope from -5 ft MLLW up to +16 ft MLLW would be required in order to allow for barge loading and unloading. It is assumed that the style of barge and the method of offloading and loading, driving a loader and/or telescopic handler from the barge onto land via the barge ramp (Figure 49), would not change due to the existence of a harbor. Two moorage points would be required for the barge to moor to while offloading or loading.

The existing fuel header for Elim is located on the bluff above the proposed location for a harbor at Elim Beach (Figure 3). In order for a fuel barge to use a harbor at Airport Point, the existing fuel header would need to be relocated to Airport Point.

There are no existing utilities located at Airport Point. To provide electricity and/or water for a harbor at Airport Point, utilities would need to be run from Elim Airport down to the harbor or along the bluff from the school out to the harbor.

11. ALTERNATIVE 5 – RECOMMENDED PLAN OPTIMIZED DESIGN

11.1. Plan Selection and Endorsement

Alternative 5 was presented as the TSP at the Agency Decision Milestone on 09 July 2020 and received an endorsement from the HQUSACE Chief of Planning and Policy Division to be carried forward as the Recommended Plan. Alternative 5 serves the full vessel fleet, 25 subsistence vessels, 25 commercial vessels, two tenders, and one barge and tug at Elim Beach.

Comments provided during the policy and legal review suggested that based on plan formulation, the required benefits could be achieved with a smaller or lower cost harbor. Through coordination with the community, it was determined that smaller uplands would be able to achieve the same level of benefits for a lower cost to the community (Alaska District 2020). With these two goals in mind, the optimization of the Recommended Plan was performed. The modifications to the Recommended Plan would not impact the fleet that would be served or change the benefits realized with project construction.

11.2. Design Criteria and Modifications

Design criteria and modifications were made based on district design quality control, agency technical review, policy and legal review comments received, and coordination between the design team and reviewers. All design criteria updates would impact harbor effectiveness and quantities for the full suite of alternatives to a similar extent.

11.2.1. Channel and Basin Depths and Widths

During the DQC review, it was recommended that the basin dimensions be re-evaluated. Through the re-evaluation, it was determined that the entrance channels were narrower than recommended by EM 1110-2-1615 (USACE 1984) Table 3-1. For Alternative 5, the entrance channel width was widened from 250 ft to 300 ft. It was also determined that the required safety clearance for channels with a hard bottom is 3 ft (USACE 1984), not 2 ft as cited throughout Section 6.3 Entrance Channel and Maneuvering Depth. As stated in Section 6.3 Entrance Channel and Maneuvering Depth, for the small crafts moving at reasonable speeds, squat was taken to be 1 ft for the entrance channel and 0.5 ft for interior channels, moorage basins, and turning basins. For larger vessels, the squat was calculated, considering the anticipated vessel speed, characteristics of the channel and vessel, and interactions with another vessel. The subsistence and commercial depths were set based on the use of the interior channel and moorage basin, and the tender and barge and tug were set based on the use of the entrance channel. The minimum required depths for each vessel class were updated as follows:

Table 13. Updated Subsistence Vessel Minimum Required Depth, Relative to MLLW

	[ft]	[ft MLLW]
Design Water Level		0
Vessel Draft	2	-2
Squat	0.5	-2.5
Pitch, Roll, Heave	0.5	-3
Safety Clearance	3	-6
Minimum Required Depth		**-6**

Table 14. Updated Commercial Vessel Minimum Required Depth, Relative to MLLW

	[ft]	[ft MLLW]
Design Water Level		0
Vessel Draft	5	-5
Squat	0.5	-5.5
Pitch, Roll, Heave	0.5	-6
Safety Clearance	3	-9
Minimum Required Depth		**-9**

Table 15. Updated Tender Vessel Minimum Required Depth, Relative to MLLW

	[ft]	[ft MLLW]
Design Water Level		0
Vessel Draft	6	-6
Squat	0	-6
Pitch, Roll, Heave	1	-7
Safety Clearance	3	-10
Minimum Required Depth		**-10**

Table 16. Updated Barge and Tug Minimum Required Depth, Relative to MLLW

	[ft]	[ft MLLW]
Design Water Level		0
Vessel Draft	8	-8
Squat	0	-8
Pitch, Roll, Heave	2	-10
Safety Clearance	3	-13
Minimum Required Depth		**-13**

11.2.2. Typical Cross-Section

During the district design quality control review, it was determined that an armored toe would be beneficial to the resiliency of the breakwater. Once shorefast ice is formed along Elim Beach, the movements of the shorefast ice could destabilize the ocean side of the breakwater. To reduce the risk of damage to the breakwater, an armored toe was added to the ocean side of the breakwater (Figure 52).

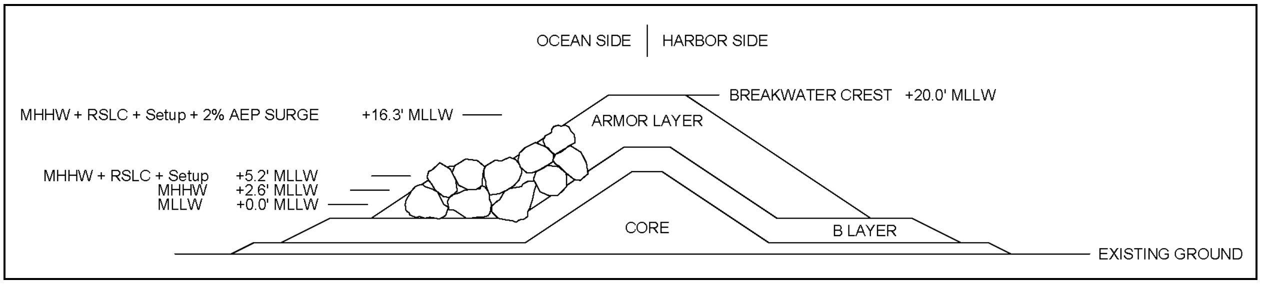


Figure 52. Updated Typical Breakwater Section with Water Level Components

11.2.3. Sediment Transport

During the policy and legal review, the team received comments concerning maintenance dredging quantities. These comments led to a review of the longshore sediment transport evaluation and informed the decision to expand the discussion on how the longshore sediment transport rate was used to estimate maintenance dredging quantities.

With an inner depth of closure of approximately -20 ft MLLW assuming the effective wave height is the 1% AEP wave height of 4.04 meters (13.3 ft) (Hallermeier 1978) and an assumed maximum onshore transport elevation of +11 ft MLLW. The width of active longshore transport is approximately 4,200 ft. Based on the longshore sediment transport rate and the blockage caused by the breakwaters results in a deposition rate of approximately 3.7 yd^2/yr. With the estimated deposition rate and the length of exposed channel maintenance, dredging rates were determined for each alternative. It was assumed that maintenance dredging would not be initiated until a project condition survey indicated that the average depth across the harbor and channel was at the required depth. The recurrence intervals calculated for maintenance dredging was assumed to be an underestimate due to experience with previous projects showing that fillets typically form along an exposed channel, narrowing the channel and requiring maintenance dredging rather than slowing infilling evenly across the channel. The maintenance dredging recurrence interval and quantities were both decreased based on engineering judgment and are presented under Section 11.3.4 Quantities for the Recommended Plan optimized design.

11.2.4. Local Features

Based on community feedback, it was determined that smaller uplands would provide the same level of benefits to the community at a lower cost (Alaska District 2020). The community indicated that for large freight shipments, they receive five 20-ft Connexes. For temporary uplands boat moorage and vehicle and trailer parking, the community only wants about 1 acre. It was also expressed that the community is already planning on building a laydown pad on the bluffs towards the west end of where Front Street runs along the beach, and they would prefer the uplands to tie into their planned laydown pad. The boat launch does not need to run to the uplands. Instead, the community prefers an access road from the uplands to the boat launch along the beach.

The uplands elevation of +16 ft MLLW was determined to be adequate, with local feedback indicating that it may get inundated occasionally but only during large, infrequent storms. The uplands would need to be armored, with the armor rock size to withstand the force of the largest waves expected within the harbor. The access road from the uplands to the boat launch would be approximately 250-ft long and run along the base of the bluffs. A single boat launch would be required to allow for the launching and retrieving of vessels. It would also enable boat owners to pull their boats onshore if the storm surge and wave conditions are anticipated to be greater than the harbor design conditions. For the feasibility level design, it was assumed that the boat launch would have a 13% slope from the base of the bluffs, at approximately +10 ft MLLW, to -5 ft MLLW enabling use during most tide conditions (Section 5.3).

A single tender dock would be required to allow for the loading of fish totes onto an NSEDC tender from the fish buying station. The tender dock would be an 87-ft long sheetpile dock at an elevation of +16 ft MLLW with the sheetpile driven to bedrock, approximately 12 ft below existing ground.

A barge landing that is 100-ft wide with a 1:4 slope from -8 ft MLLW up to +16 ft MLLW would be required to allow for barge loading and unloading. It is assumed that the style of barge and the method of offloading and loading, driving a loader and/or telescopic handler from the barge onto land via the barge ramp (Figure 46), would not change due to the existence of a harbor. Two moorage points would be required for the barge to moor to while offloading or loading.

11.3. Optimized Design and Harbor Effectiveness

During the optimization process, the modifications described above were incorporated into the design before navigation feature layout adjustments.

11.3.1. Breakwaters

To address policy and legal review comments, navigation feature layout adjustments were investigated to determine if features could be moved or removed without impacting harbor effectiveness. One such adjustment was to remove the east breakwater. The fetch limited wave analysis, using WIS ST82107 hindcast data, indicates that the waves out of Norton Bay could be as large as 7.5 ft. If the east breakwater was not constructed, calculated fetch limited wave heights would exceed the design conditions at the tender dock over 20% of the time and at the boat launch over 50% of the time that waves are approaching the navigation improvements out of Norton Bay. Without the east breakwater, wave height design exceedance at the tender and boat launch would be significantly higher than the less than 5%, and 20% of the time for the tender dock and boat launch with the east breakwater is in place. Based on the lack of wave information, limited wind data (Figure 53) for a fetch analysis, assumptions made during the fetch limited wave analysis (Section 4.2 Fetch Limited Waves), and limited information on local observations indicating otherwise, it was determined that removal of the east breakwater would be an unacceptable risk.

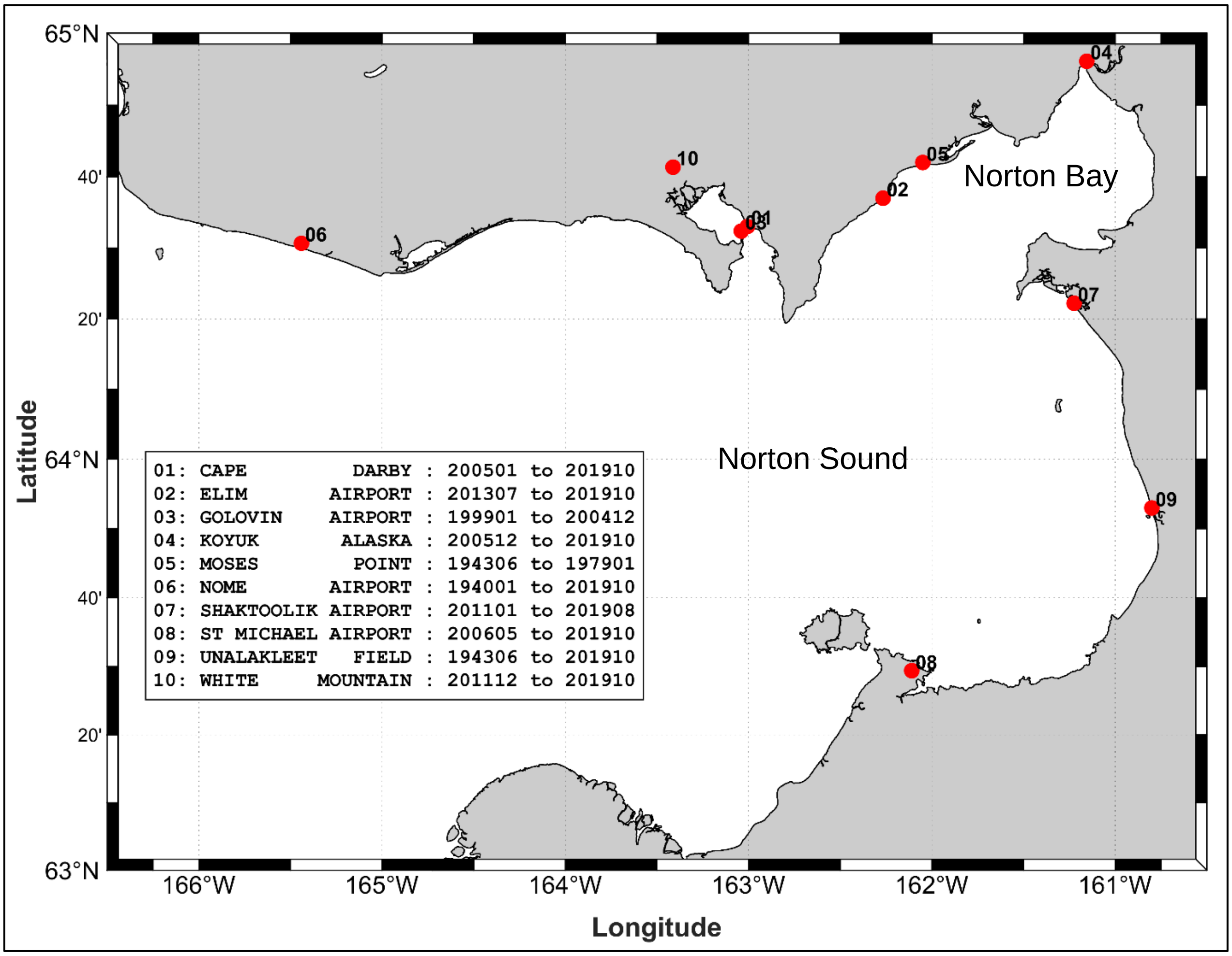


Figure 53. Meteorological Data Available around Norton Sound and Norton Bay

During the optimization of the breakwaters, the east breakwater was straightened and attached to shore for ease of construction and to limit circulation impacts within the harbor. The Recommended Plan optimized design consists of the west rubble-mound breakwater, which would be 986 ft, and the east rubble-mound breakwater, which would be 820 ft. That would provide a 1.4-acre moorage basin for 50 boats ranging in length from 18 ft to 32 ft, an interior channel to provide access to the boat launch, and a 2.5-acre turning and maneuvering basin for the tenders and tug and barge. This plan would provide shelter from waves propagating out of the west through southwest and from the east (Figure 54). Figure 55 and Figure 56 show the updated typical cross-section with dimensions for the west and east breakwater, respectively.

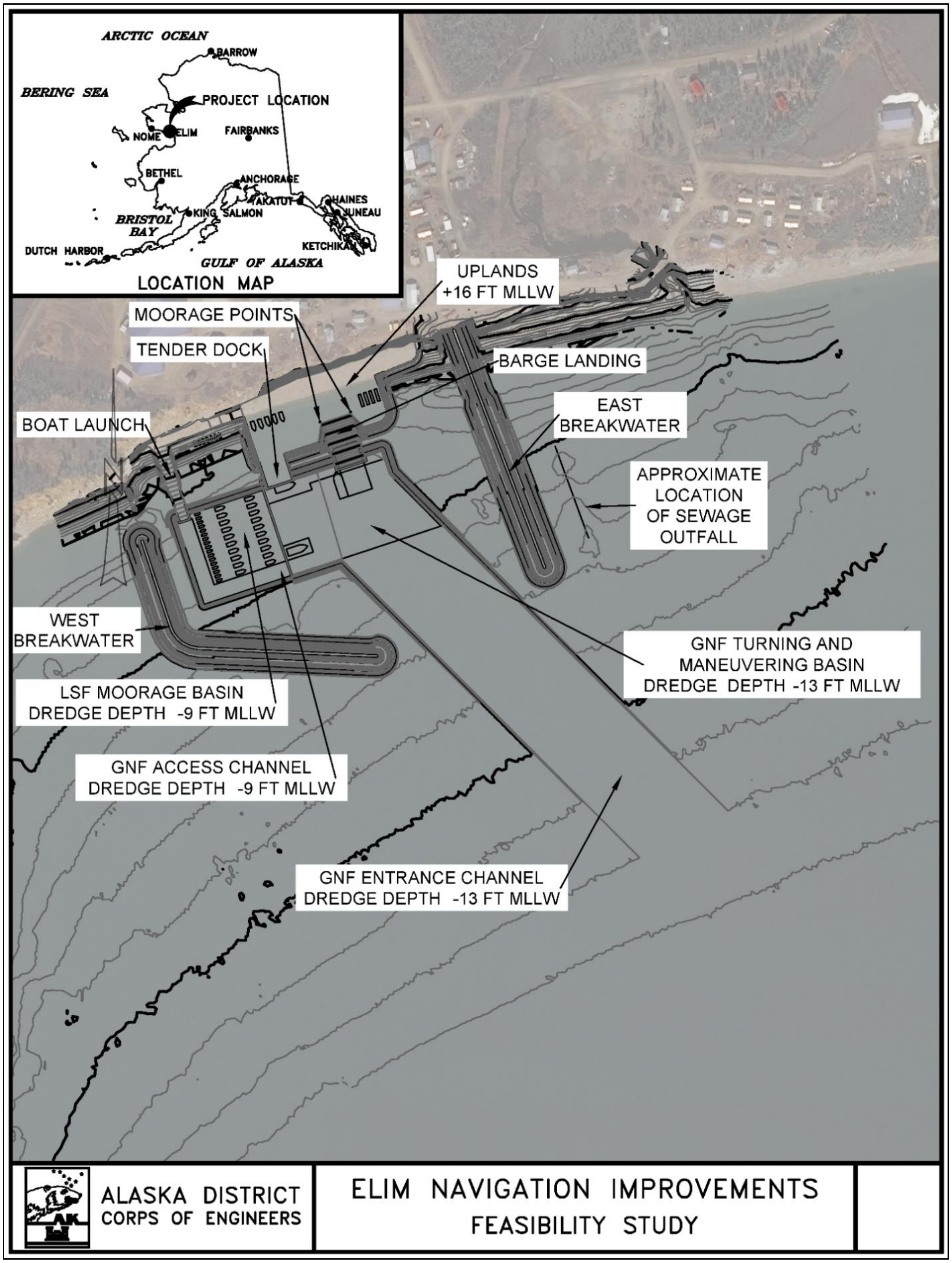


Figure 54. Recommended Plan Optimized Design

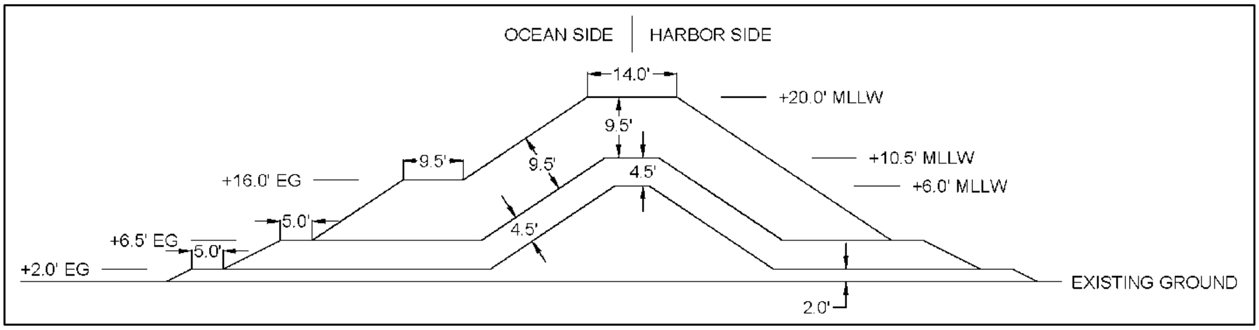


Figure 55. Updated Typical West Breakwater Cross-Section

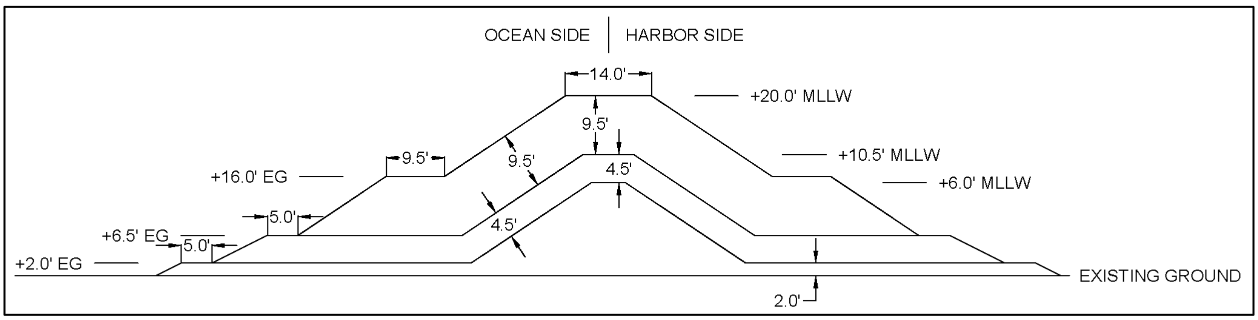


Figure 56. Updated Typical East Breakwater Cross-Section

11.3.2. Wave Reduction

The wave height impacting the project site from the northeast through the southwest was determined by applying the deep water design wave and fetch-limited wave. A diffraction analysis was then used to determine the wave heights within the harbor. For the Recommended Plan optimized design, the worst wave conditions within the harbor occur when deep water waves from the southeast and sustained wind from the southeast impact the navigation improvements, causing wave heights of up to 6.5 ft at the tender dock, 6.2 ft in the moorage area, and 4.4 ft at the boat launch. When waves are from the east, south, and southwest, the waves at the tender dock are anticipated to be below the allowable maximum wave height. The waves in the moorage area are anticipated to be 3.8–0.7 ft, and waves at the boat launch are anticipated to be 2.1–0.5 ft, close to or above the allowable maximum wave height.

11.3.3. Water Quality and Circulation

The flushing analysis (Section 9.1 Water Quality and Circulation) results in an aspect ratio of 1:2.6 and an area ratio of 88. The aspect ratio meets the minimum recommendations for a basin design, 1:4, but the area ratio is less than the minimum recommendation of 200. As described in the *Planning and Design Guidelines for Small Craft Harbors* (ASCE 2012), wider entrance channels do not lead to uniform flushing. Still, alternative flushing analysis indicates that there will be good flushing within the proposed harbor. The harbor may have stagnant water in the corners of the harbor that would likely flush with storm surge recession.

11.3.4. Quantities

Breakwaters, dredging, and upland construction volumes for the Recommended Plan optimized design are presented in Table 17.

Table 17. Recommended Plan Optimized Design Breakwater, Dredge, and Upland Quantities

Material	Quantities [cy]
West Breakwater	
Armor Rock	26,576
B Rock	18,872
Core Rock	17,128
East Breakwater	
Armor Rock	20,501
B Rock	14,705
Core Rock	11,423
Dredging of General Navigation Features	
Mechanical Dredging	46,654
“Ripping”	107,751
Blasting	6,713
Dredging of Local Service Features	
Mechanical Dredging	5,752
“Ripping”	17,621
Blasting	1,154
Uplands	
Fill	50,149
Armor Rock for Revetment	1,558
B Rock for Revetment	1,371

The breakwater and dredging maintenance volumes and upland maintenance requirements for the Recommended Plan optimized design are presented in Table 18.

Table 18. Recommended Plan Optimized Design Maintenance Quantities and Intervals

Material	Quantities	Interval [yr]
Offshore Breakwater Maintenance		
Armor Rock	1,177 [cy]	25
Maintenance Dredging		
Dredging	40,000 [cy]	20
Uplands Maintenance		
Regrading	127,116 [sf]	1
Riprap	78 [cy]	25
Sheetpile	276 [ea]	50
Boat Launch	1 [ea]	50

12. NAVIGATION AIDS

Initial coordination with the US Coast Guard has indicated that the final breakwater plan must include 10-ft by 10-ft poured concrete pads at the offshore nose (Seris 2020). The US Coast Guard would install Federal Aid to Navigation (ATON) on the concrete pads. Coordination with the US Coast Guard would continue during the preparation of the plans and specifications and construction.

13. CONSTRUCTION CONSIDERATIONS

The breakwater construction and dredging are anticipated to take 3 years to complete. Construction can occur during the ice-free period from mid-May until mid-November. It is recommended that the project be advertised early in the year to maximize the number of contractors to bid on this project.

14. FUTURE WORK TO BE COMPLETED IN PED

To more accurately determine the amount of blasting required for the selected plan, borings are required to ground-truth the geophysical investigation that was performed during the Feasibility Study and recalculate quantities if necessary.

The constructability of connecting the west breakwater to the shore at the rocky outcrop, west of the boat launch, will be investigated during PED to decrease the risk of sedimentation at the boat launch.

A phase-averaged spectral wave model, such as STWAVE or SWAN, would be required in PED to determine a more accurate design wave and wave conditions inside the harbor to optimize breakwater alignment. An investigation into existing breakwater and revetment projects that are subject to ice ride-up and a mild wave climate should be completed in PED to determine if smaller armor rock could be used at Elim.

A value engineering study would be required in PED to determine if cost savings could be achieved without impacting harbor effectiveness.

15. REFERENCES

(AAOS) Alaska Ocean Observing System. 2019. Waverider Buoy Station 46265, deployed 2018-2019. Nome, AK: NOAA NDBC.

(Alaska District) United States Army Corps of Engineers Alaska District. (2009). Alaska Baseline Erosion Assessment. Anchorage, Alaska: United States Army Corps of Engineers.

(Alaska District) United States Army Corps of Engineers Alaska District. (2018). Geotechnical Site Assessment Elim Navigation Improvements. Anchorage, AK: Not Published.

(Alaska District) United States Army Corps of Engineers Alaska District. (2020, May 12). Meeting Minutes. Elim Webex Meeting with Sponsors.

(ASCE) American Society of Civil Engineers Task Committee of Marinas 2020. (2012). Planning and Design Guidelines for Small Craft Harbors. Reston, VI: American Society of Civil Engineers.

Bruun, P. (1962). Sea level rise as a cause of shore erosion. Journal of Waterways and Harbor Division, 88, 117-130.

Cedar Lake Ventures, Inc. (n.d.). Average Weather at Elim Airport. Retrieved from Weatherspark: https://weatherspark.com/y/149674/Average-Weather-at-Elim-Airport-Alaska-United-States-Year-Round

Chapman, R. S., & Mark, D. J. (2014). Storm Damage and Flooding Evaluation: Western Alaska Storm Surge Modeling Study: Bering Sea and Bristol Bay Site Calibration and 2005-2011 Extreme Water Level Statistics, Executive Summary. ERDC/CHL. Vicksburg, MS: Not Published.

Davidson-Arnott, R. G. (2005, November). Conceptual Model of the Effects of Sea Level Rise on Sandy Coasts. Journal of Coastal Research, 21(6), 116-1172.

del Valle, R., Medina, R., & Losada, M. (1993, September/October). Dependence of Coefficient K on Grain Size. Journal of Waterway, Port, Coastal, and Ocean Engineering, 119(5), 568-574.

Ettema, R., & Kennedy, J. F. (1982). Ice Study for the Port of Nome, Alaska. The University of Iowa, Iowa Institute of Hydraulic Research. Iowa City: Not Published.

(Golder) Golder Associates Inc. (2019). Final Report of Results Geophysical Survey Report Navigation Improvements Study, Elim, Alaska. Anchorage, AK: Not Published.

Hall, J. A., Gill, S., Obeysekera, J., Sweet, W., Knuuti, K., & Marburger, J. (2016). Regional Sea Level Scenario for Coastal Risk Management: Managing the Uncertainty of Future Sea Level Change and Extreme Water Levels for Department of Defense Coastal Sites Worldwide. U.S. Department of Defense, Strategic Environmental Research, and Development Program.

Hallermeier, W. A. (1978). Uses for a calculated limit depth to beach erosion. Proceedings, Coastal Engineering, 1493-1512.

(HDR) HDR Alaska, Inc. (2012). Elim Hazard Impact Assessment. Elim, AK: City of Elim.

Iowa State University of Science and Technology. (n.d.). AK_ASOS Elim. Retrieved October 2019, from Iowa Environmental Mesonet: https://mesonet.agron.iastate.edu/sites/site.php?station=PFEL&network=AK_ASOS

IPCC. (2001). Climate Change 2001: The Scientific Basis. Contribution of Working Group I to the Third Assessment Report of the. Cambridge, United Kingdom, and New York, NY, USA: Cambridge University Press.

IPCC. (2007). Climate Change 2007: The Physical Science Basis. Contribution of Working Group I to the Fourth Assessment. Cambridge, United Kingdom and New York, NY, USA: Cambridge University Press.

Kinsman, N. (2019, April 10). E-mail Personal Correspondance: USACE - Silver Jackets Non-structural Proposal - Relative Sea Level Change Guidance for Western and Arctic Alaska. Not Published.

NASA Jet Propulsion Laboratory. (n.d.). GPS Time Series. (M. Helfin, Producer) Retrieved October 2018, from NASA Jet Propulsion Laboratory California Institute of Technology: https://sideshow.jpl.nasa.gov/post/series.html

Osborne, E., Richter-Menge, J., & Jeffries, M. (2018). Arctic Report Card 2018. National Oceanic and Atmospheric Administration. Retrieved from https://www.arctic.noaa.gov/Report-Card

Perovich, D., Meier, W., Tschudi, M., Farrell, S., Hendricks, S., Gerland, S., . . . Webster, M. (2018). Sea Ice [in Arctic Report Card 2018]. National Oceanic and Atmospheric Administration. Retrieved from https://www.arctic.noaa.gov/Report-Card

Seris, D. (2020, June 03). E-mail Personal Correspondence: Elim Alaska Navigation Improvements – Conceptual Level Design. Not Published.

(USACE) United States Army Corps of Engineers. (1984, September 25). Hydraulic Design of Small Boat Harbors. (EM 1110-2-1615). Department of Army.

(USACE) United States Army Corps of Engineers. (2000, April 22). Planning Guidance Notebook. (ER 1105-2-100). Department of Army.

(USACE) United States Army Corps of Engineers. (2002, April 30). Coastal Engineering Manual. (EM 1110-2-1100). Department of Army.

(USACE) United States Army Corps of Engineers. (2019, June 15). Incorporating Sea Level Change in Civil Works Programs. (ER 1100-2-8162). Department of Army.

(USACE) United States Army Corps of Engineers. (2019, June 30). Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation. (EP 1100-2-1). Department of Army.

Van der Meer, J. W., Allsop, N. H., Bruce, T., De Rouch, J., Kortenhaus, A., Pullen, T., . Zanuttigh, B. (2018). Manual on wave overtopping of sea defenses and related structures. An overtopping manual largely based on European research, but for worldwide application. EurOtop. EurOtop. Retrieved from www.overtopping-manual.com

Vos, K., Splinter, K. D., Harley, M. D., Simmons, J. A., & Turner, I. L. (2019, December). CoastSat: A Google Earth Engine-enabled Python toolkit to extract shorelines from publicly available satellite imagery. Environmental Modelling & Software, 122.

Wiegel, R. L. (1962, January). Diffraction of Waves by a Semi-infinite Breakwater. Journal of the Hydraulics Division, 88, 27-44.

Wuebbles, D. J., Fahey, D. W., Hibbard, K. A., Dokken, D. K., Stewart, B. C., & Maycock, T. K. (2019). Climate Science Special Report: Fourth National Climate Assessment, Volume I. Washington, D.C.: U.S. Global Change Research Program. DOI:10.7930/J0J964J6