St. George Harbor
Feasibility Study

Appendix A - Hydraulic Design

St. George, Alaska

Tentatively Selected Plan

August 17, 2018

U.S. Army Corps of Engineers

Alaska District

CONTENTS

1. INTRODUCTION

1.1 Appendix Purpose

This appendix describes the technical aspects of proposed navigation improvements to support a harbor on St. George Island, Alaska. It provides the engineering background information for determining the Federal interest in the major construction features including causeways, breakwaters, channel improvements, and support facilities. Existing data was gathered and analyzed to determine site characteristics, and numerical modeling was performed to determine the physical impacts of the wave climate for design of the proposed navigation improvements.

1.2 Current Stage of Work

This appendix describes the engineering that has been performed to arrive at the Tentatively Selected Plan (TSP) for this study. The TSP is Alternative N-3 as described in this appendix, which is a small boat harbor on the north side of St. George Island near the Village of St. George which accommodates the local subsistence fleet, fuel delivery barges and fishing vessels to 14 foot draft. The TSP was selected with the analysis available to the team at the time. During the progression of this study, deficiencies in the numerical model being used to analyze harbor responses were revealed. Alterations to the numerical model to compensate for these deficiencies are being pursued, but were not completed prior to selection of the TSP. Numerical modeling and concept design are continuing efforts in the study and further study could require changes to the configuration, construction methods and costs associated with the TSP. Additional numerical modeling is ongoing to show the effectiveness of the harbor configuration of the TSP. As methods improve, results may show improved effectiveness in alternatives that were not selected as the TSP and the relative effectiveness of the alternatives is expected to change. Further numerical model design is required to better refine the scope of some plan features.

1.3 Project Purpose and Needs Assessment

The following objectives were identified for navigation improvements at St. George Harbor.

a. Provide safe and more efficient improvements for the various design fleets.
b. Provide facilities for fuel barges, fishing vessels and freight logistics vessels for which current depths and facilities are not available.
c. Reduce harbor access and moorage delays and increase port operation efficiencies.

The project purpose is to provide a safe and efficient harbor in an environmentally sound manner that satisfies the above objectives.

2. SITE SELECTION

This study encompasses two sites on St. George Island; Zapadni Bay and the North Site, as shown in Figure A-1. Initially, Zapadni Bay was selected through a charrette process that included stakeholders at the Federal, State and local levels. The charrette considered several sites on the island and settled upon the one site evaluated in this study. Zapadni Bay is the location of the existing harbor and upland infrastructure to support harbor operations. As the study progressed, the team decided to investigate a location on the north shore of the island as a potential new harbor site with more favorable wave conditions to Zapadni Bay. The north site is located at the west end of the community of St. George.

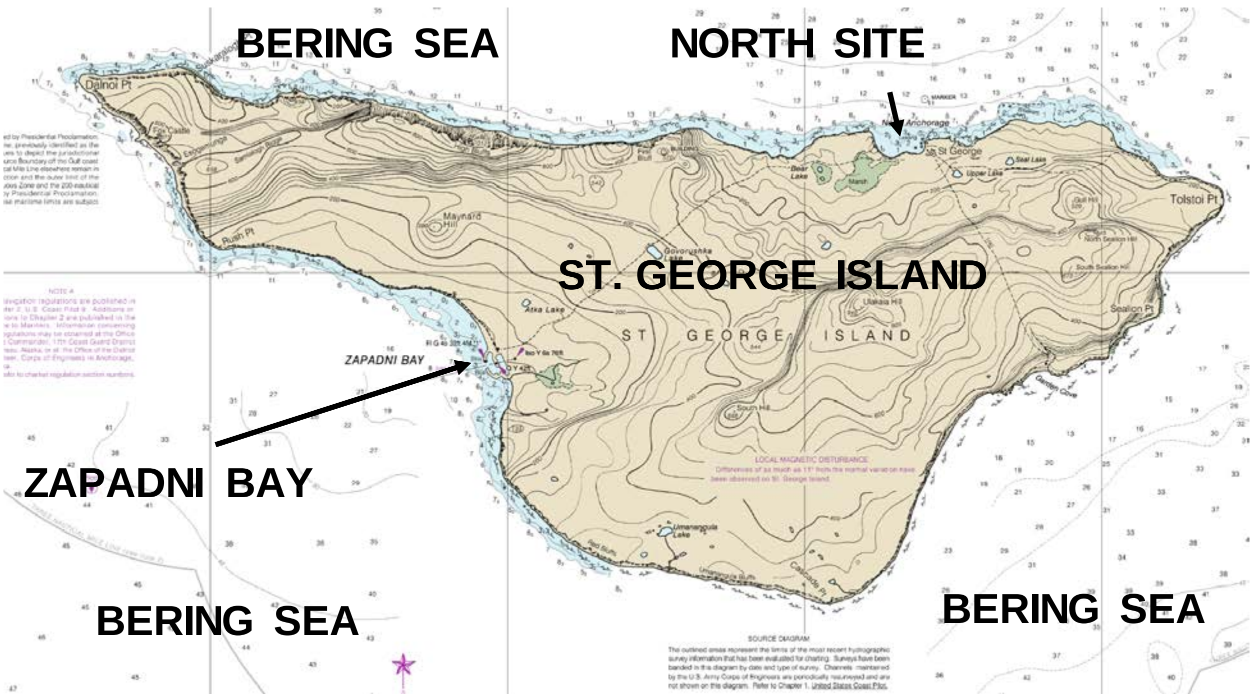


Figure A-1: Site locations. Detail from NOAA Chart 16381, St. George. Annotation Added.

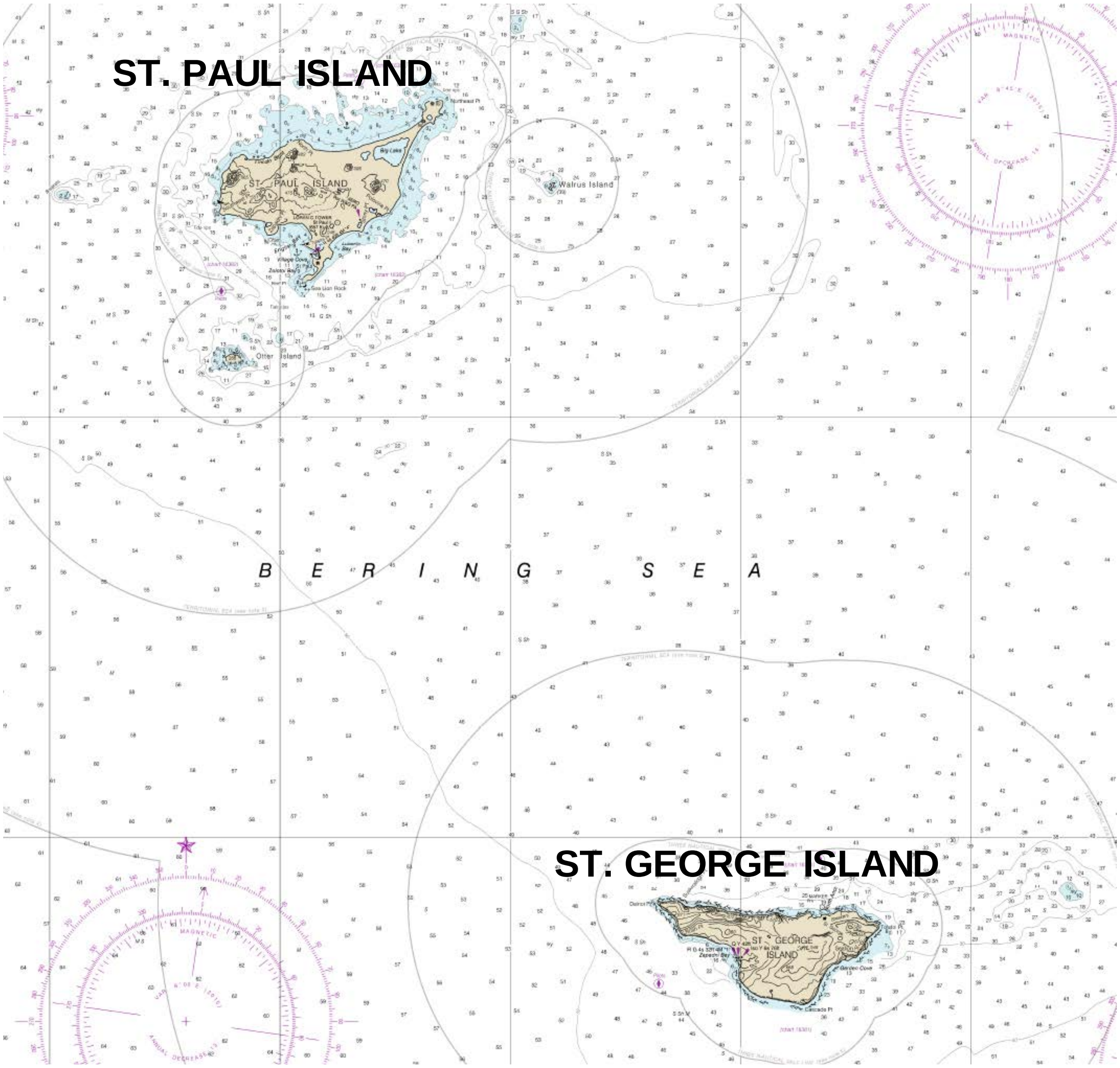


Figure A-2: Pribilof Islands vicinity. Detail from NOAA Chart 16380, Pribilof Islands. Annotation Added. St. George Island is approximately 49 miles to the southeast of St. Paul Island.

St. George Island is one of the Pribilof Islands, which are located in the Bering Sea approximately 225 miles north of Dutch Harbor and 750 miles west of Anchorage. Two of the islands, St. Paul and St. George, are inhabited. St. Paul Island has a harbor constructed by the U.S. Army Corps of Engineers (Corps) at Village Cove, which supports crabbing vessels operating in the Bering Sea. St. George Island has a harbor at Zapadni Bay constructed by the City of St. George with initial dredging performed by the Corps. The harbor includes a navigation channel dredged by the Corps and turning basin protected by three rubble mound berm breakwaters constructed by the City of St.

George. Federal maintenance of the navigation project was suspended in 1996 when the local sponsor was unable to enter a cost sharing agreement to complete construction dredging to reach the authorized depth.

2.1 Existing Facilities

The existing harbor breakwaters at Zapadni Bay were constructed from 1984 to 1987 by the City of St. George with funding from the State of Alaska. The breakwaters were designed as berm structures with 8 ton armor stones produced from a local quarry. The original design called for the berm breakwaters to be built in depths of -30 feet mean lower low water (MLLW) with an inner stub breakwater connected to the south breakwater that would have protected a small basin in deep water. The shoreline inside the breakwater would act as a spending beach to dissipate wave energy and minimize reflection inside the harbor. During construction, the design was changed by moving the breakwaters into shallower water at about -20 feet MLLW and using the rock quarry as an inner harbor basin.

After the breakwaters were constructed, a federal navigation project was constructed to provide navigation depths required for vessels to use the harbor. The St. George Harbor navigation project was authorized on November 17, 1986 by Public Law 86-645 under Section 107 of the Rivers and Harbor Act of 1960. The authorization was to construct dredge a maneuvering area to -18 feet MLLW and an entrance channel to -20 feet MLLW which was estimated to require removal of approximately 176,000 cubic yards (CY) of material. Construction of St. George Harbor and related facilities progressed as follows:

1984 - The City of St. George undertakes construction of three rubble mound breakwaters funded by the State of Alaska.

1987 – Breakwater construction is completed.

1988 – A contract is awarded for dredging the federal project, including a 3-acre boat basin and 2 feet of advance maintenance in the entrance channel to be funded by the City of St. George and the State of Alaska.

1989 – Dredging begins in April and continues through the season until October with 54% of the project reported complete.

1990 – The contractor re-mobilizes in the spring and dredges into August.

1993 – The last project condition survey is completed.

1994 – Insufficient depth in the entrance channel necessitates further construction dredging.

1995 – Under the new local cooperation agreement, the City proceeds with improvements to the project, however project design depth is still not achieved.

1996 – The City of St. George is unable to enter into a new project cost sharing agreement with the Government to complete the project. The federal maintenance obligation is suspended.

2004 – Contract awarded to repair damage to the south breakwater.

2016 – Contract awarded to repair to the south breakwater caused by a December 2015 storm. New 8 ton armor stone is placed on the south breakwater.

2017 – Contract awarded to place a 10 foot seaward berm of 8 ton armor rock on the repaired extents of the south breakwater.

Figure A-3: St. George Harbor aerial image.

2.1.1 North and South Breakwaters

The North and South Breakwaters were designed as berm breakwater structures which are intended to change shape over time in response to wave energy. The original breakwater design was performed by Peratrovich, Nottingham & Drage, Inc. in 1984. The Typical section consisted of primary armor stone weighing between 1.7 and 10 tons. The section includes a 55 foot berm on the seaward face of the breakwater to be constructed to an elevation of +12 feet MLLW and a crest elevation of +26 feet MLLW (Figure A-4). The concept for this section was to allow waves to move the rock in the berm to form a shallower seaward slope over time and function as a beach. Use of this concept has proven that the rock berm formed through this process has required emergency repairs in 2004 and 2016 and has not reduced the harbor wave climate to acceptable levels.

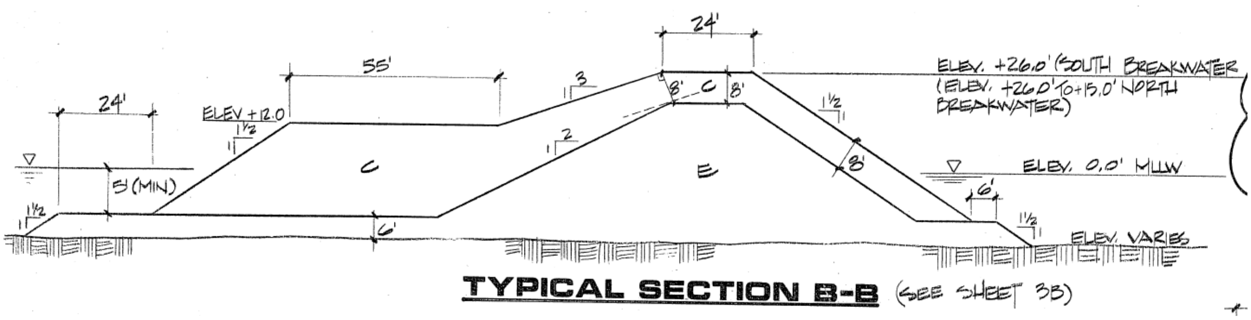


Figure A-4: Typical berm breakwater section.

2.1.2 Inner Breakwater

The inner breakwater was formed by the boundary of the quarry pit and the shoreline. The core material of this structure is solid rock. Additional rock was constructed to increase the crest elevation to protect the inner harbor.

2.1.3 Entrance Channel and Maneuvering Area

The entrance channel and maneuvering area are currently the only federally constructed features at St. George Harbor. The Entrance channel was dredged to depths of -20 and -22 feet MLLW and the Maneuvering basin was dredged to -18 feet MLLW. The north end of the inner harbor was not dredged, but the quarried depths of the basin are -12 feet and -8 feet MLLW. Construction dredging of the entrance channel was never completed; rock pinnacles and shallow areas limit navigable depth at the harbor entrance and at the south breakwater near the inner harbor entrance.

2.2 Climatology

St. George falls within the southwest maritime climate zone, characterized by persistently overcast skies, high winds, and frequent cyclonic storms. The climate of St. George is controlled by the cold waters of the Bering Sea. The summers are cold and windy, the winters are long, freezing, and extremely windy, and it is overcast year round. Over the course of the year, the temperature typically varies from 24°F to 52°F and is rarely below 9°F or above 56°F.

2.3 Water Levels, Currents, and Waves

2.3.1 Tides

Water level data is not recorded at St. George Island. The nearest tidal station is located at Village Cove on St. Paul island, approximately 50 miles away. Due to the similarity of the sites, tidal data from St. Paul was used for this study (Table A-1).

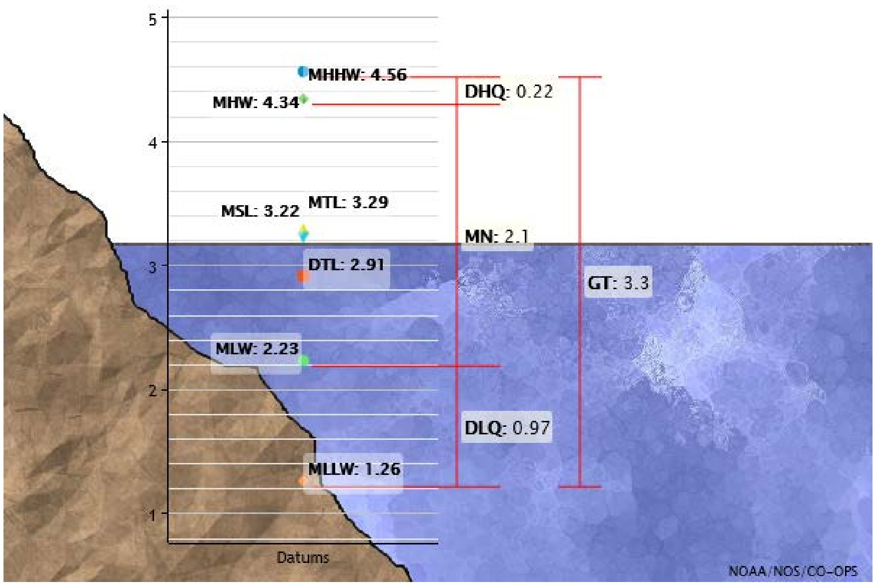


Figure A-5: Datums for 9464212, Village Cove, St Paul Island, AK. Units are in feet, station datum. Note that MLLW is +1.26 feet in station datum.

Table A-1: Published tidal data for Village Cove, St. Paul Island, Alaska. Values in feet, Mean Lower Low Water.

Highest Observed Water Level (12/08/06)	+5.26
Highest Astronomical Tide (HAT)	+4.09
Mean Higher High Water (MHHW)	+3.30
Mean High Water (MHW)	+3.08
Mean Tide Level (MTL)	+2.03
Mean Tide Level (MSL)	+1.96
Mean Low Water (MLW)	+0.97
Mean Lower Low Water (MLLW)	0.00 (datum)
Lowest Astronomical Tide (LAT)	-1.50
Lowest Observed Water Level (12/06/10)	-2.10

Source: NOAA NOS, Tidal Epoch 1983-2001, published 12/12/11.

From the above data, the mean tide level (arithmetic average of the MHW and the MLW) is +2.03 foot. The mean tide range (the difference between MHW and MLW) is 2.11 feet.

2.3.2 Sea Level Change

The Corps 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). Guidance for addressing SLC is in Engineer Regulation (ER) 1100-2-8162 and detailed below. Three scenarios of "low," "intermediate," and "high" SLC are evaluated over the project life cycle. According to the ER, the SLC "low" rate is the historic SLC. The "intermediate" and "high" rates are computed using the following:

> Estimate the "intermediate" rate of local mean sea-level change using the modified NRC Curve I and the National Research Council (NRC) equations. Add those to the local historic rate of vertical land movement.
>
> Estimate the "high" rate of local mean SLC using the modified NRC Curve III and NRC equations. Add those to the local rate of vertical land movement. This "high" rate exceeds the upper bounds of Intergovernmental Panel on Climate Change (IPCC) estimates from both 2001 and 2007 to accommodate potential rapid loss of ice from Antarctica and Greenland.

NRC Equations

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

$$E(t) = 0.0012t + bt^2$$

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 change was approximately 1.2 mm/year. Using the current estimate of 1.7 mm/year for global mean sea level (GMSL) change, as presented by the 2007 IPCC, results in this equation being modified to be:

$$E(t) = 0.0017t + bt^2$$

The three scenarios proposed by the NRC result in global eustatic sea level rise values, by the year 2100, of 0.5 meter, 1.0 meter, 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. The three GMSL rise scenarios are shown in Figure A-6 (Figure 5 from EC 1165-2-212).

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

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2)$$

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 $t_2 = t_1$ + number of years after construction) . For the three scenarios proposed by the NRC, b is equal to 2.71E-5 for Curve 1, 7.00E-5 for Curve 2, and 1.13E-4 for Curve 3.

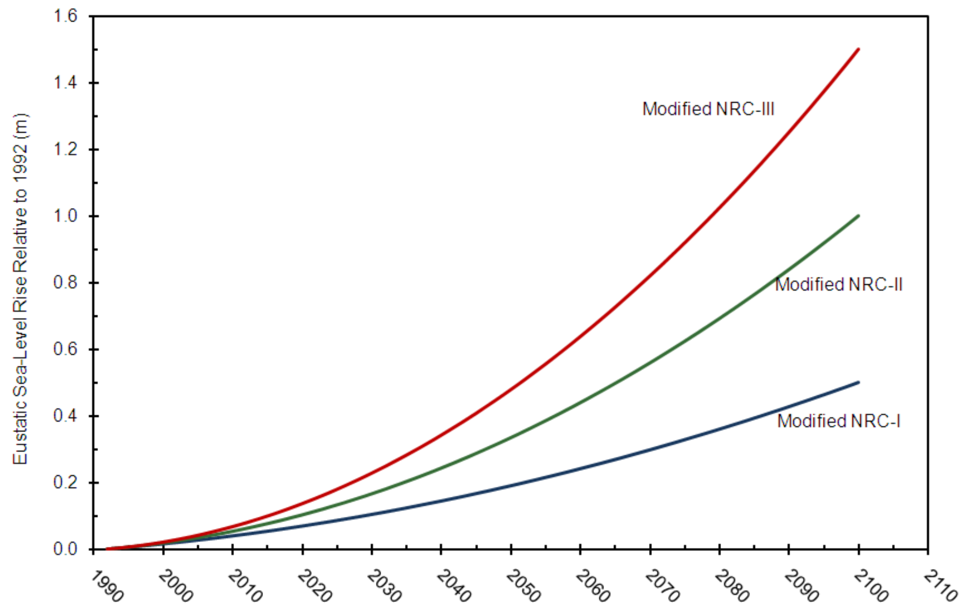


Figure A-6: (Figure A-5 from EC 1165-2-212). Scenarios for GMSL Rise (based on updates to NRC 1987 equation).

The local St. Paul tide station does not have the recommended 40-year period of record for the relative sea level change (RSLC) value. The tide station has a 10-year water level records from 2006. Based on the tide data available, the RSLC would be +0.015mm/yr. Per the guidance recommendation, a U.S. tide station with a 40-year period of record was investigated for use as the RSLC value. The nearest U.S. tide station with the required 40-year period of record is the Unalaska, Alaska station, roughly 225 miles from the site. It has a historic RSLC of -5.58 mm/yr. Due to the distance from St. George, the Unalaska gage was not further investigated. Due to the short period of record at St. Paul, the GMSL rate was used to model sea level change at St. George (Table A-2). Table A-2 assumes a project construction year of 2023 with projected sea levels in 2073.

Table A-2: (Table A-3 per EC 1165-2-212). Sea Level Rise Prediction for a 50-Year Project Life.

Scenario	Low (Historic)	Intermediate	High (Curve III)
GMSL	+0.28 feet	+0.78 feet	+2.35 feet
St. Paul	+0.00 feet	+0.50 feet	+2.08 feet
Unalaska	-0.83 feet	-0.36 feet	+1.13 feet

2.3.3 Water Levels

Water levels at St. Paul are primarily affected by astronomical tides. The difference between predicted astronomical tides and observed water levels are attributable to storm surge and atmospheric pressures. The water level record at St. Paul includes predicted and observed values. The highest recorded water level in the record was +5.223 feet MLLW. The non-tidal residuals were analyzed to determine the range of positive and negative residuals. The highest positive residual in the record occurred on April 7, 2011 where the observed water level was 2.578 feet above the predicted tide. The lowest recorded water level at St. Paul was -2.10 feet MLLW. The maximum non-tidal residuals did not coincide with the maximum high water levels for most events.

The water level record at St. Paul was then used to identify extreme high water events and extreme non-tidal residual events over the 10 year period of record. Extreme events of high water levels and non-tidal residuals were then analyzed with an Extreamal Type I, or Weibull, distribution to estimate probabilistic water levels and residuals. The results estimate a total water level of 5.6 feet MLLW at a 50 year recurrence interval. This compares to a non-tidal residual of 3.0 feet at the same 50 year recurrence interval. The non-tidal residuals were added to the MHHW level of +3.3 feet MLLW to compare to total water level measurements. Water levels calculated using MHHW and non-tidal residuals were 0.5 to 0.7 feet higher than total water levels (Table A-3).

Table A-3: Probabilistic Total Water Level and Non-Tidal Residuals

Return Period (yrs)	MHHW (ft., MLLW)	Non-Tidal Residual (ft.)	MHHW + Non-Tidal Residual (ft., MLLW)	Total Water Level (ft., MLLW)	Delta (ft)
10	3.3	2.5	5.8	5.3	-0.5
20	3.3	2.7	6.0	5.4	-0.6
50	3.3	3	6.3	5.6	-0.7

Historic records of storm surge at St Paul are few. A single event was noted in the Alaska District's Flood Plain Management Files recording a storm surge of 5 feet at St. Paul on December 25, 1966, that flooded a house in the community. Flooding was attributable to wind driven waves and high water.

Shoreline geometry and bathymetry at St. Paul and St. George differ significantly in regards to the potential to produce storm surge. At St. Paul, the shoreline is bounded to the south by Reef Point and to the west by Zapadni Point. Bathymetry between these features is fairly uniform with a gentle slope (Figure A-7). This creates a potential for west and southwest wind and wave events to produce a storm surge at St. Paul. The event reported in 1966 was a result of wind and waves surging into the village at Zolotol Bay, to the south of Village Cove where the harbor is located.

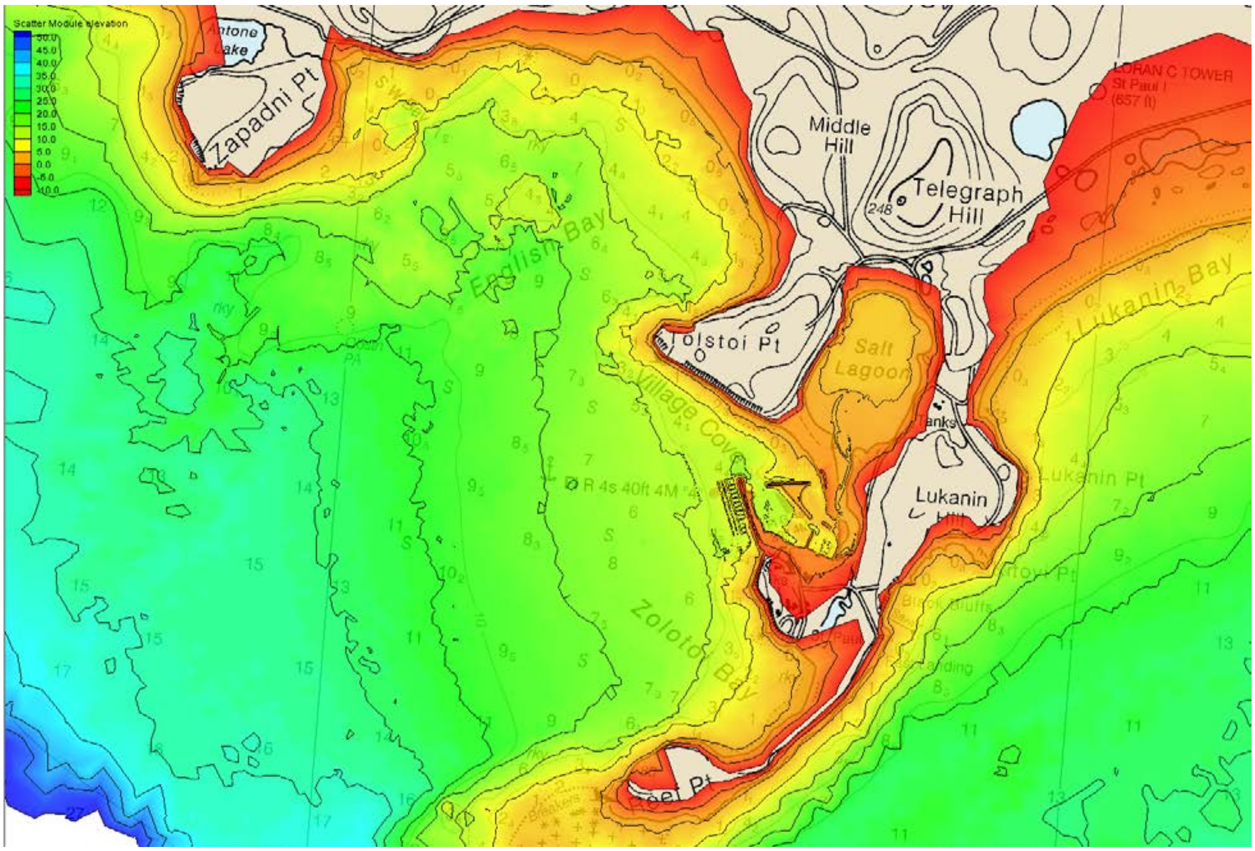


Figure A-7: St. Paul shoreline and bathymetry, detail from NOAA chart 16382 with contour shading from National Ocean Service digital bathymetry and 2009 project condition survey data. Color ramp and contours are in 5 meter intervals. Depth at the edge of the color ramp is -50 meters MLLW.

At St. George, the shoreline is less confining for west and southwest events. Rush Point to the west and the Red Bluffs to the south do not extend as far beyond the harbor site as at the shoreline at St. Paul and nearshore bathymetry is deeper with a steeper slope to the shoreline (Figure A-8). This shoreline geometry and deeper bathymetry allow for a more efficient flow of water around the island which results in a lower potential for storm surge.

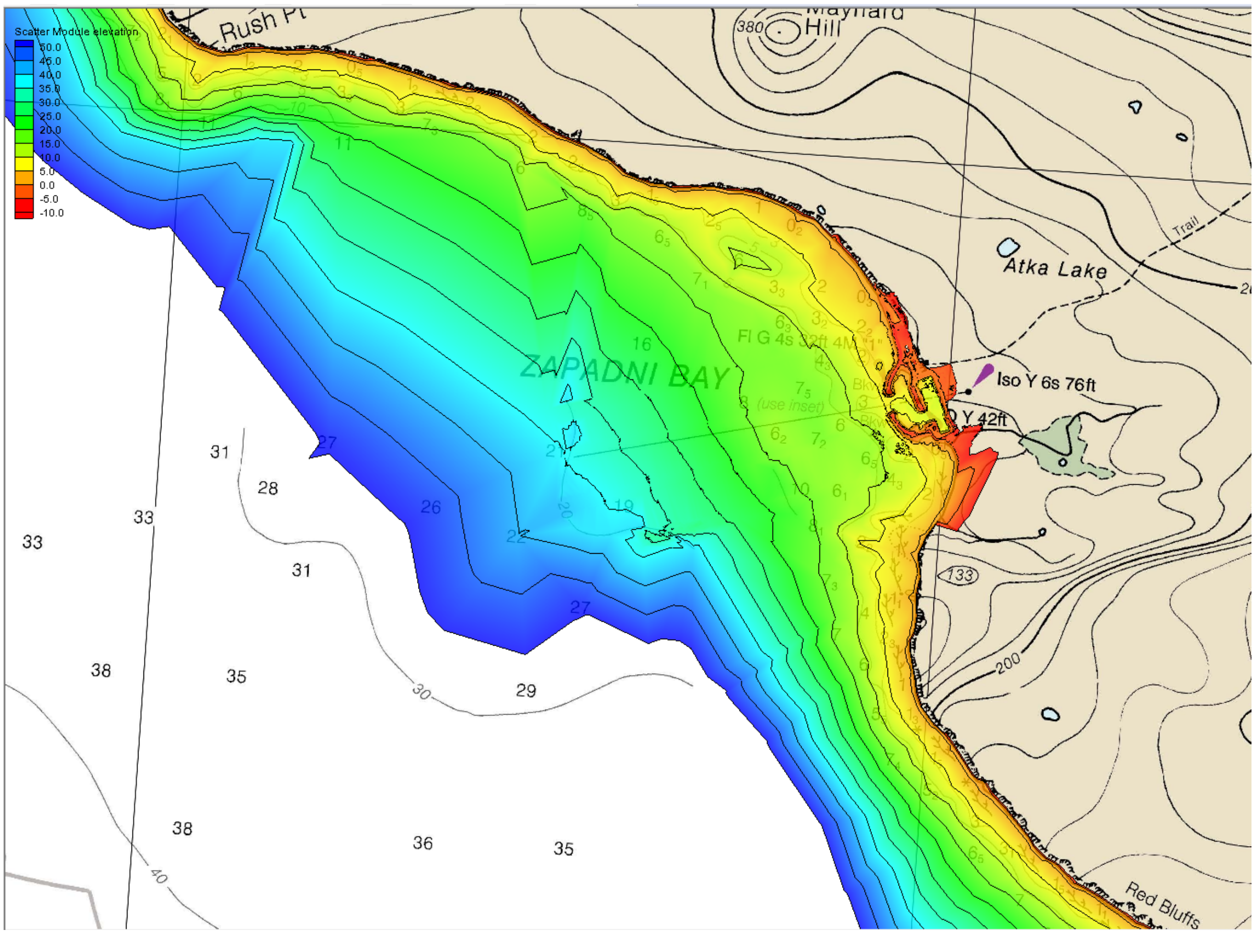


Figure A-8: St. George shoreline and bathymetry, detail from NOAA chart 16381 with contour shading from NOS digital bathymetry and 2013 multibeam survey data. Color ramp and contours are in 5 meter intervals. Depth at the edge of the color ramp is -50 meters MLLW.

For design purposes, two water levels were used. For modeling wave propagation through the harbor and alternative designs, a water level of +5.9 feet MLLW (+1.8 m MLLW) was used for all simulation runs. This water level is above the highest measured data at St. Paul and is representative of the nominal sea surface elevation as storms approach the island. For breakwater design, a water level of +8.5 feet MLLW was used to account for historical surge events. This higher value was selected to ensure that breakwater structures not be overtopped during storm events.

2.3.4 Currents

Measured current data is not available for St. George. Barge operators related experiences navigating through beam-on currents when entering and exiting the harbor at St. George. The predominant current direction is to the north, though it was noted that it sometimes flows to the south. Fishing vessel captains contacted did not report having any concerns for currents at the harbor. Current velocities were not estimated.

2.3.5 Wave Climate

The wave climate at St. George is very similar to that of St. Paul and is controlled by the Bering Sea. Two storm mechanisms were identified producing the most severe effects in the Bering Sea. Typically, winter storms in the Bering Sea are generated in the Sea of Okhtosk and travel east. These storm systems can occur multiple times over the course of a season and sometimes follow one after another for multiple weeks at a time. The most severe wave conditions occur in the winter months as typhoon remnants from the south Pacific blow past the Aleutian chain and generate waves in the Bering Sea. Buoy data to the north of the Chain shows waves in excess of 30 feet on an annual basis. St. George is directly exposed to these waves and energy is only dissipated from these events in the nearshore zone as bathymetry causes these waves to shoal and break before reaching the shore. The nearshore wave climate around the island is depth-limited with wave breaking caused by bottom friction being the only mechanism to reduce wave energy from storms before it reaches the shore.

2.4 Ice Conditions

St. George Island lies at the southern extent of sea ice in the Bering Sea. Typically, Zapadni Bay is ice free. Historical sea ice concentrations have been cataloged and recorded in Alaskan waters from the 1850s to the present. These records were compiled into a Sea Ice Atlas database which maps the Bering Sea in quarter degree increments. This work was done by the International Arctic Research Committee and the University of Alaska Fairbanks. The atlas was accessed at http://seaiceatlas.snap.uaf.edu/ and sea ice concentrations were investigated at 56.75°N, 169.5°W which is to the south of St. George Island. The records show that St. George historically has open waters (ice concentrations of 30% or less) from June through February and greater concentrations of ice from March through May (Figure A-9). The records also show that pack ice (concentrations over 90%) has never been recorded at St. George. The most recent recorded ice at St. George above 30% was in January of 2000 with the next previous event occurring prior to 1980.

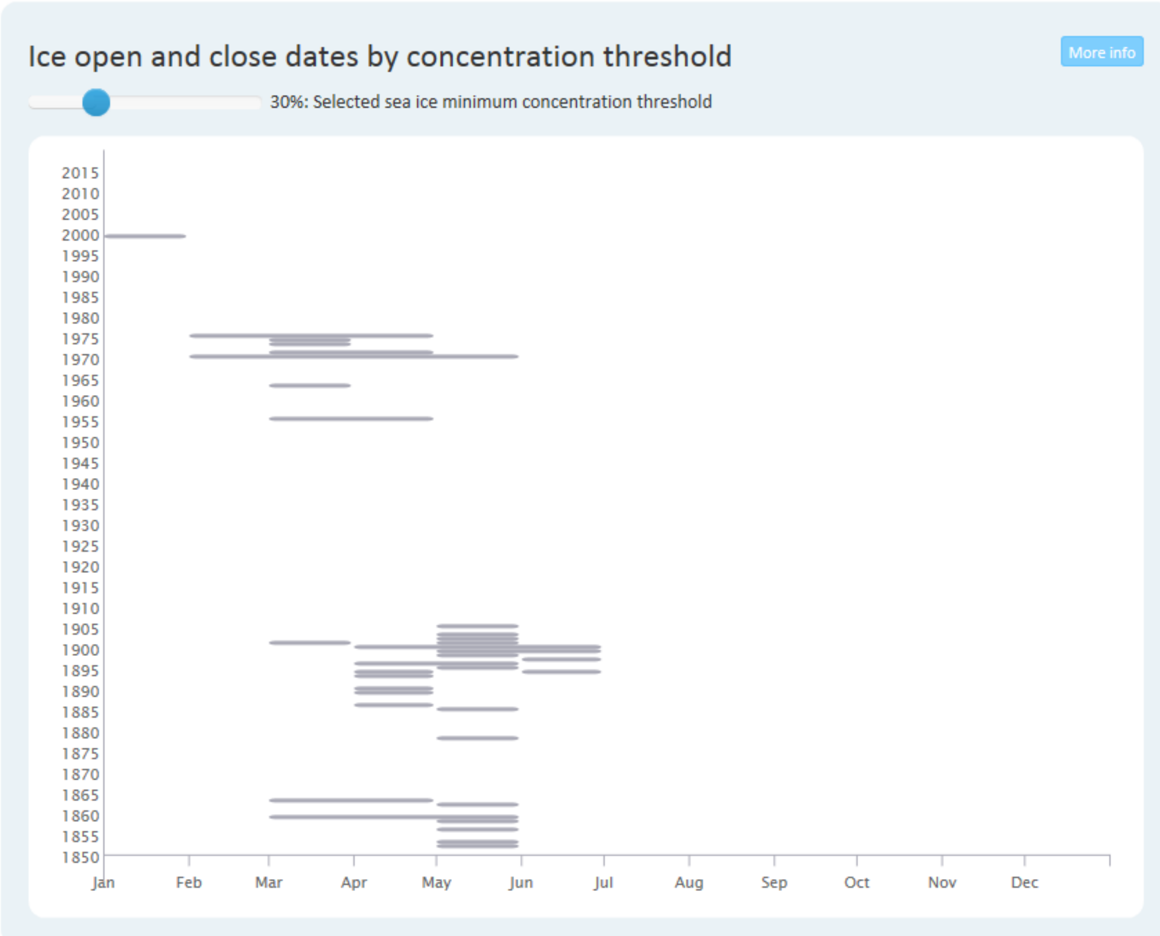


Figure A-9: Historical concentrations of sea ice exceeding 30% at 56.75°N, 169.5°W, near Zapadni Bay, St. George.

Due to the orientation of the harbor and typical ice concentrations in the area, sheet ice is not expected to form and produce ice forces against harbor structures.

For comparison purposes, the historical sea ice concentrations at St. Paul were also investigated. More frequent ice coverage was noted in the records as shown in Figure A-10. The most noticeable difference in ice coverage is from 2000 to 2010 where the St. George data shows no incidents of ice concentrations above 30% and St. Paul shows several events in the January through April timeframe. While this data indicates St. George experiences less ice than St. Paul, there is insufficient detail in the records to determine what impacts this would have on vessels attempting to use a harbor in the Pribilof Islands during this season.

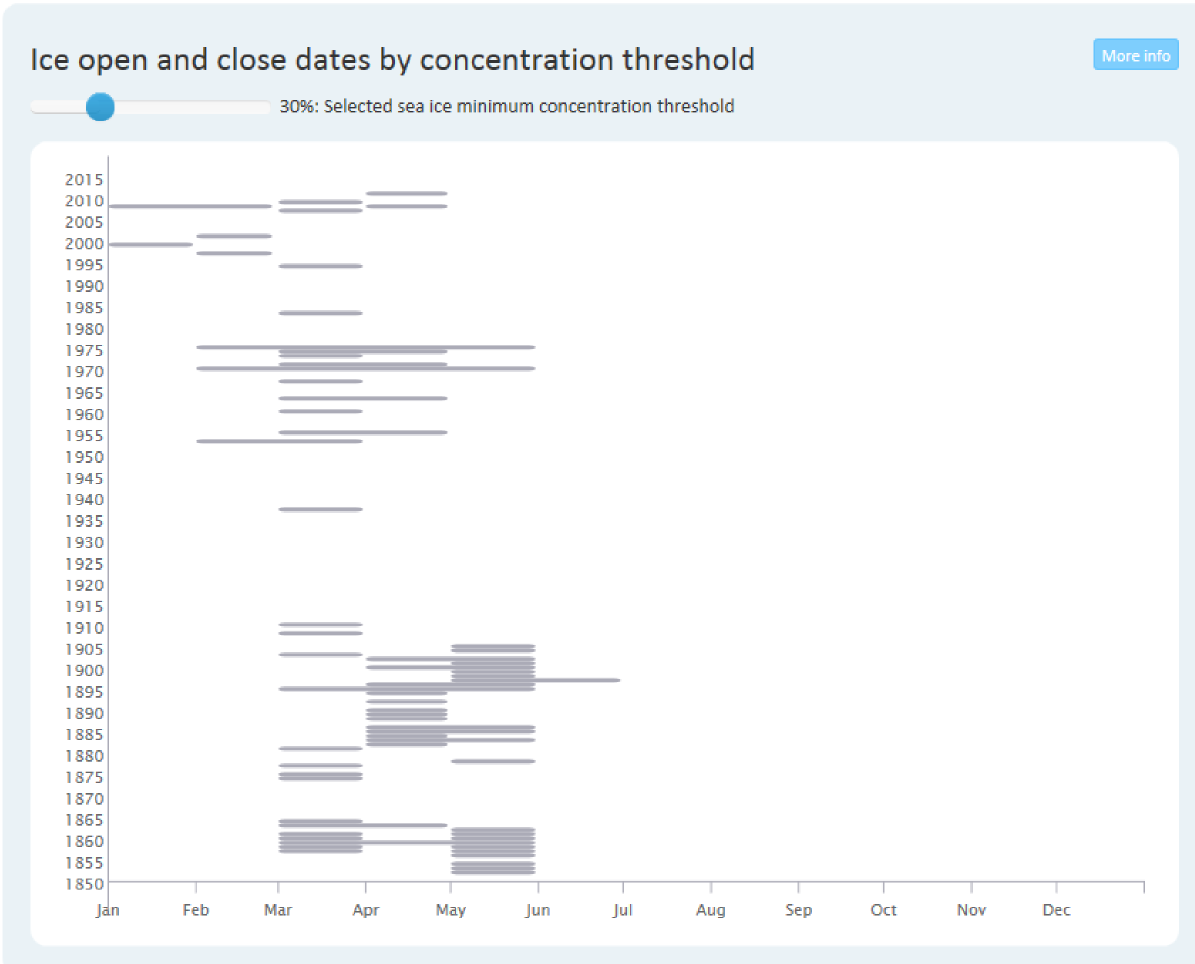


Figure A-10: Historical concentrations of sea ice exceeding 30% at 57.25°N, 170.5°W near Village Cove, St. Paul.

Additional ice coverage analysis was performed to determine the likelihood of ice sheet coverage near the north site of St. George that would indicate the presence of marine mammals that use ice sheets as haul out habitat. For the north site, sea ice concentrations were investigated at 57.0°N, 169.5°W. This would impact winter construction activities at the north site for blasting and dredging, discussed later in this appendix. To account for the presence of ice sheets at St. George, a 60% ice coverage criteria was used. Two events were noted both in March and April in 1970 and 1976 and eight May events were noted from 1859 to 1906 (Figure A-11). Over the 165 year period of record, there were ten occurrences of ice concentration which roughly corresponds to a 6% occurrence of pack ice at the north site. Impacts of these occurrences are likely to represent delays to project construction of up to two months.

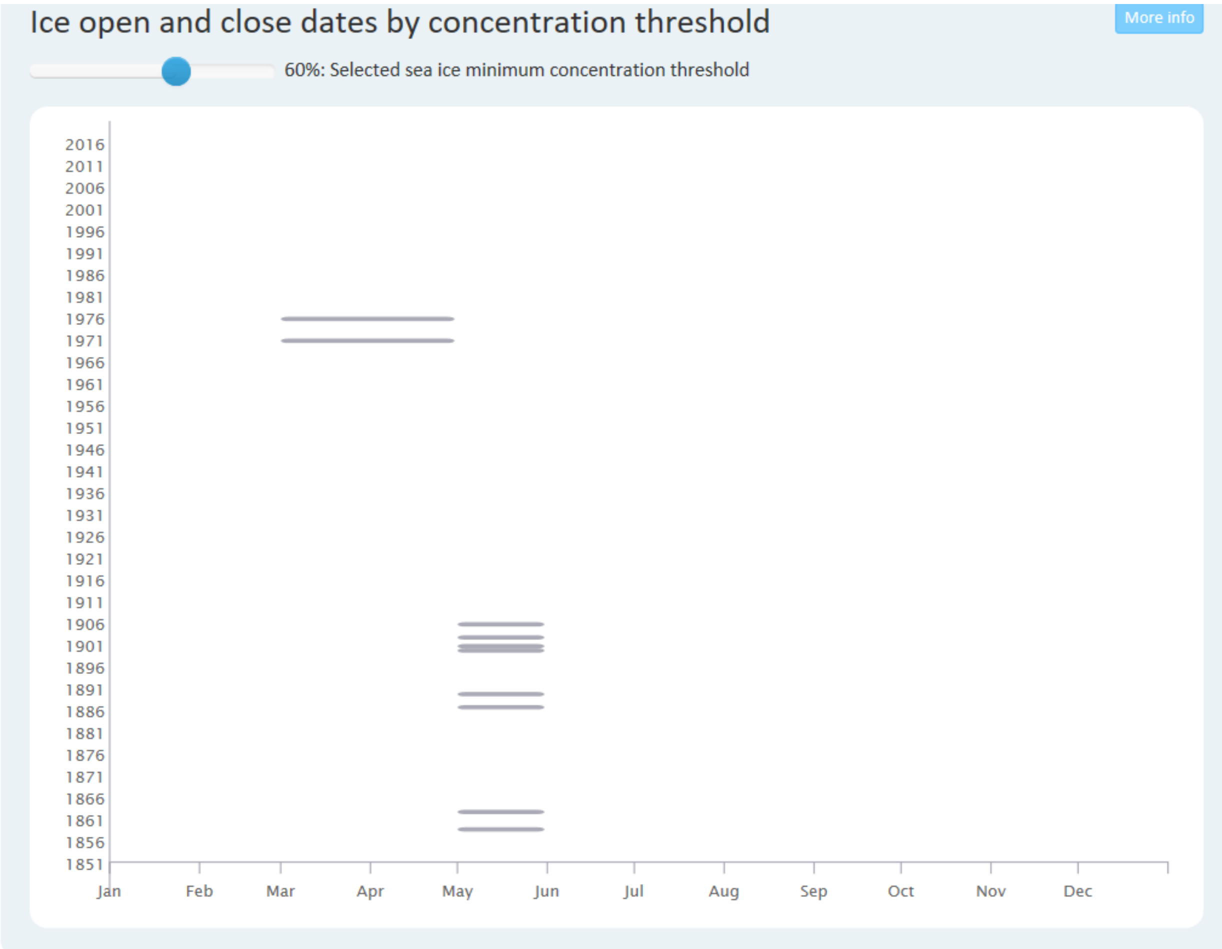


Figure A-11: Historical concentrations of sea ice exceeding 60% at 57.0°N, 169.5°W near the north site, St. George.

2.5 Sedimentation

Sedimentation has been observed to occur in the harbor at Zapadni Bay. USACE maintained the navigation channel into the harbor at Zapadni Bay through 1996. Surveys from the time of construction to that date showed no change in bathymetric conditions in the harbor except for construction activities. Channel depths through this period remained at or below the authorized depth of -22 feet MLLW. Maintenance of the channel was suspended in 1996. No surveys were performed from 1995 when the Tanaq Corporation had the harbor surveyed until 2013 when the City of St. George began to investigate navigation improvements at their harbor. The 2013 survey showed significant shoaling in the channel with the formation of a bar across the outer breakwaters with a minimum elevation of about -14 feet MLLW. A subsequent survey in 2016 showed that this bar had migrated into the harbor at about the same depth. Several large storms occurred over this interval, including one that damaged the south breakwater in December 2015 requiring repairs to be performed in 2016 and 2017.

2.5.1 Sources and Sinks

St. George Island is an isolated sediment transport system and all sediment occurring along its shoreline is likely to have been generated by weathering of the stone cliffs that comprise the island's shoreline. Mariners noted that there is a dominant current at Zapadni Bay from the south to the north, though this current can reverse direction. Given the fact that the island poses only a minor obstruction to the circulation of water in the Bering Sea, it is likely that cross-shore transport of sediment during storm events is a greater contributor to sediment transport than longshore transport movements.

During the data collection phase at Zapadni Bay, one of the Acoustic Doppler Current Profilers (ADCP) sensors was lost inside the harbor. During attempts to recover it, the surveyors noted that the bottom material of the harbor was fine material up to 5 feet deep. This material could be left over from quarry operations during the original harbor construction. Also, there is a potential that fine material is generated as the berm breakwater sized stones shift under storm conditions and as the rock walls of the inner harbor erode.

2.5.2 Sediment Transport Rate

The Limited Reevaluation Report for St. George published in 1993 estimated that to maintain the harbor at Zapadni Bay, 10,000 CYs would need to be dredged from the outer harbor every 2 years. A cursory evaluation of sedimentation was made by quantifying the volumetric change in conditions within the outer harbor between the 1995 and 2013 surveys and between the 2013 and 2016 surveys. Only the outer harbor areas were compared since an extension of the inner harbor was constructed between 1995 and 2013 which would skew results. The volume of bathymetric change was divided by width of the harbor opening measured at 0 feet MLLW between the outer breakwaters to give a unit rate of transport per harbor opening width. The harbor opening at Zapadni Bay by this definition is 300 feet.

2.5.2.1 Volumetric Change 1995 - 2013

Volumetric net change within the outer harbor between 1995 and 2013 was +3,000 CYs. Movement of material was greatest between the outer breakwaters with a maximum increase in elevation of +8 feet along the channel bottom. Change in volume rapidly decreased to below +3 feet within 100 feet of the harbor entrance, then tapered off to less than +1 foot within 250 feet of the opening. The average rate of sedimentation in the harbor over this period is approximately +170 CYs per year.

2.5.2.2 Volumetric Change 2013 - 2016

Volumetric net change within the outer harbor between 2013 and 2016 was +13,300 CYs. Movement of material near the outer breakwaters ranged from -7 feet to + 7 feet as the shoal migrated from across the harbor entrance to about 400 feet inside the harbor. This movement of material accounts for most of the net sediment transport within the harbor during this period. Volume change in the inner harbor was negligible. The average rate of sedimentation in the harbor over this period is approximately 4,400 CYs per year which is consistent with the estimate in the 1993 LRR report.

2.5.2.3 Design Sedimentation Rate for Zapadni Bay

The likely reason for the large difference in sedimentation rates between the two periods analyzed is the time required to form a sediment wedge around the toe of the breakwater. When the harbor was completed in 1990, sediment would have begun to accumulate at the toe of the breakwater. During this period of time, the only material to move in and out of the harbor would have been located directly at the harbor entrance. As the sediment wedge built over time, a new source of material became available to move into the harbor during storm events. It is possible that the majority of the sediment movement found in the period from 1995 to 2013 occurred near the end of this period once the sediment wedges had been formed. Since this source of material is currently the condition of the harbor, it is assumed that future sedimentation will follow the pattern observed from 2013 to 2016 and the maintenance dredging requirement at Zapadni Bay will be 4,400 CYs per year, or 15 CYs per linear foot of harbor opening.

2.5.3 North Site Sedimentation

No time series of surveys of the north site have been performed which would provide a basis for quantifying volumes of sediment transport. Wave analysis around the island indicates that there is greater wave energy and potential for cliff erosion and sediment transport on the southwest side of the island when compared to the north side of the island. Peak spectral energy on the north side during storms is in the 12 to 14 second range while on the southwest side, it is in the 18 to 22 second range. Using the log relationships between the Wave Information Studies (WIS) Stations representative of these coastlines, representative wave energy flux values were calculated based on the design wave height for breakwaters with peak periods in the range of the top ten storm events from each site.

Wave power is calculated by the formula:

$$P = {}^{1}/_{2}\, E_0 C_0 \quad \text{where } E_0 = \frac{\rho g H^2}{8} \text{ and } C_0 = \frac{gT}{2\pi}.$$

The value of the constants in these equations were taken as ρ=1029 kg/m^3 (density of sea water) and g = 9.81 m/s^2 (gravity). The characteristic wave heights and periods used for Zapadni Bay and the north site were 7.1 meters, 20 seconds and 4.5 meters and 13 seconds respectively.

The wave power at the Zapadni Bay site was found to be approximately four times the value of the wave power at the north site. For the purpose of analyzing sediment movement at the north site, it is assumed that the rate of movement is one quarter the rate estimated at Zapadni Bay. The unitized rate of sediment transport into the harbor is assumed to be 4 CYs per linear foot of harbor opening, or approximately 1,000 CYs per year for a 300 foot opening similar to the existing harbor.

It should be noted that this is a high level assumption and does not take into account differences in the availability of sediment, differences in sediment gradation, degradation rates of the coastline and rock structures or sheltering effects of shoreline geometry. To account for these effects, a time series of surveys of the north site, representative sampling of sediment from both sites and laboratory analysis would need to be performed, which is beyond the scope of this study.

3. DESIGN CRITERIA

3.1 Design Vessel and Fleet

A fleet spectrum was developed for the arctic region and is outlined in the Economics Appendix for this study. Expected fleet missions are commercial fishing, subsistence fishing and freight and fuel delivery. Characteristic vessels have been identified to provide the minimum design requirements for port facilities.

3.1.1 Commercial Fishing

Commercial fishing would be accomplished using ocean going vessels of the same type found at St. Paul or Dutch Harbor. Vessel dimensions were obtained for 78 vessels operating with permits in the Bering Sea. This sample was assumed to be representative of the fishing fleet and representative dimensions were taken from this data. Vessels sampled have length dimensions from 80 feet to 170 feet, beam from 24 feet to 41 feet and draft from 8 feet to 17 feet. Since vessel draft for this fleet is a controlling dimension for channel design, a distribution of vessel drafts was created to see what percentage of the vessels in the fleet exceed various draft thresholds (Figure A-12).

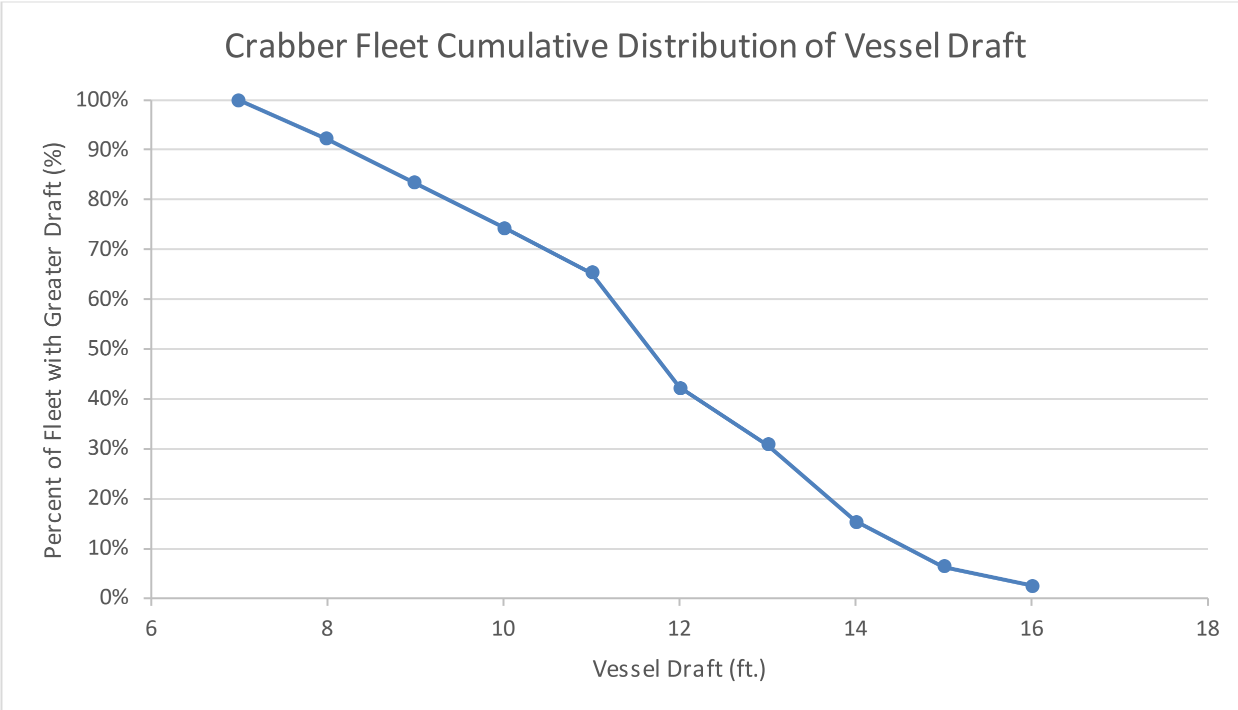


Figure A-12: Distribution of vessel draft of crabber vessels operating in the Bering Sea.

Based on the draft distribution, a design vessel draft of 14 feet was selected for the fleet accessing St. George. This draft includes 85% of the vessels sampled. The deeper draft vessels generally have the longest length and beam dimensions and are less likely to call at St. George as they would not be able to offload their entire hold of product at facilities likely to be operated at St. George. A design vessel draft of 10 feet, which would be the minimum to accommodate the fuel barge, would include 25% of the vessels sampled.

For the purpose of this study, it was assumed that waves at the harbor entrance must be 10 feet or less in height for a crabber to enter the harbor. When analyzing model output, the threshold value for crabbers to enter and exit the harbor is 3 meters. This is based on prescriptive guidance from St. Paul Harbor operations that the harbor is generally closed when waves at the main breakwater reach 10 feet. Some variation in acceptable harbor accessibility conditions are expected depending upon vessel characteristics and crew experience.

3.1.2 Freight Delivery

Freight delivery to St. George is currently carried out by air freight. Infrequent freight barges offload supplies, equipment and material at St. George for construction activities. The vessels chosen to represent this operation were taken from Alaska Marine Lines' fleet data. They operate a 270 foot barge, Western Service which is 270 feet long, 70 feet wide with a draft of 19 feet. The largest tug operated by the same group which has dimensions of 94 feet long, 27 feet wide and 14 feet draft. Another tug in their fleet had a beam of 30 feet, which creates a maximum vessel beam of 100 feet. Recent construction activities to repair the Zapadni Bay South Breakwater was supported by an articulated tug and barge operated by Brice Marine with a length of 245 feet and a loaded draft of 9.1 feet. This vessel navigated to the inner harbor to offload rocks for the repairs.

3.1.3 Fuel Delivery

Fuel deliveries to St. George are currently supplied by Delta Western which uses vessels operated by Cook Inlet Tug and Barge. The barge used for this mission is 180 feet long and 54 feet wide. It is assumed that other shippers would use similar vessels should the service provider for the community change. The loaded draft of this vessel is approximately 10 feet. Crowley Marine uses a 180 foot barge with a width of 52 feet and a loaded draft of 12.25 feet in the region. Tugs for the Crowley fleet can be up to 32 feet in width which would create a maximum vessel beam of 84 feet for a tug on hip.

For all harbor alternatives considered in this study, tug and barge deliveries require the tug to make up alongside the barge outside the harbor. This maneuver requires relatively calm seas ranging from a few feet according to the barge operators to “dead calm” according to the harbormaster at St. Paul. For the purpose of this study, a wave criteria of 1 meter was used to determine whether a tug and barge could make up on hip outside the harbor before navigating to the dock and mooring. For these vessels, the wave climate outside the harbor controls whether or not a delivery can be made.

3.1.4 Subsistence Fleet

Residents of St. George operate boats to harvest sea resources for subsistence. The local fleet is generally comprised of welded trailer able aluminum boats of beams of 8.5 feet or less. Trailer able boats usually have lengths up to 28 feet and drafts up to 4 feet. Wave criteria for these vessels was set at a 4 foot (1.2 meter) wave height. This criteria is based on discussions with vessel operations.

3.2 Vessel Navigation

The ability of the design fleet to navigate the harbor was a key design consideration. The small vessels of the local fleet and the commercial fishing vessels are maneuverable and can handle fairly tight turning scenarios. These vessels have hull designs with a deep vee to help them track a line through waves and have control surfaces that allow them to make these maneuvers. The fuel barge, on the other hand, is a flat bottomed vessel with no control surfaces and is maneuvered by tug thrust. Interviews with the tug and barge operations revealed specific concerns for the existing harbor at Zapadni Bay.

Fuel deliveries to St. George are about 85,000 gallons of fuel per delivery. While this is not a full load for the fuel hold, it is typically the heaviest delivery made to St. George and is in the least maneuverable vessel. The vessel must reconfigure outside the harbor from a tow line configuration to an on-hip configuration to allow the tug to vector thrust against the barge for maneuvering. Sea conditions outside the harbor control whether or not this can be done. The operators stated that a four foot swell creates unusable conditions at the existing harbor. Wind is a key factor fir this vessel due to the low steerage experienced at slow speeds. Barge freeboard, indicative of the sail area of the vessel varies from 2 feet fully loaded to 10 feet empty.

To accomplish the turns described in this section, the vessel must reduce its velocity to nearly dead slow. This affects steerage by reducing flow past the tug's rudder and makes turns slow to accomplish. The low speed also allows wind on the superstructure and the portion of the hull above water to significantly affect the course of the vessel. During these maneuvers, there is a concern that wind gusts will overpower the available thrust and steerage and blow the vessel aground.

To improve navigation safety of the existing harbor, it was suggested by the fuel barge operators that widening the opening at the inner breakwater by a minimum of 15 feet would significantly improve navigation into the harbor and reduce the risk of vessel casualty during this maneuver. In general, reducing the turning requirements on the fuel vessel will improve navigation safety. To maintain fuel supply access at St. George, alternatives were planned to either maintain the current navigation maneuvers, since they currently support fuel deliveries to St. George, or reduce the turning requirements for the fuel vessel.

Entrance navigation into the existing St. George Harbor is shown in Figure A-13. The fuel vessel transits the Bering Sea with the fuel barge on a tow line (1). To navigate the harbor, the tug makes up on hip, on the port side stern of the barge to assist in making the initial turn out of the sea past the outer breakwaters (2). While navigating the outer harbor, the vessel tries to approach the breakwater opening from the northwest to create as straight of a path as possible from the outer breakwaters to the nose of the inner breakwater (3). At the nose of the inner breakwater, the vessel slowly arcs around the breakwater at dead slow speed. The operators note that this is the most difficult stage of navigating to the docks since the distance between the nose of the inner breakwater and the opposite side of the inner harbor opening is very narrow, about 185 feet. Once the barge clears the inner breakwater, it moors across docks 2 and 3 on the back side of the inner harbor to deliver fuel (5).

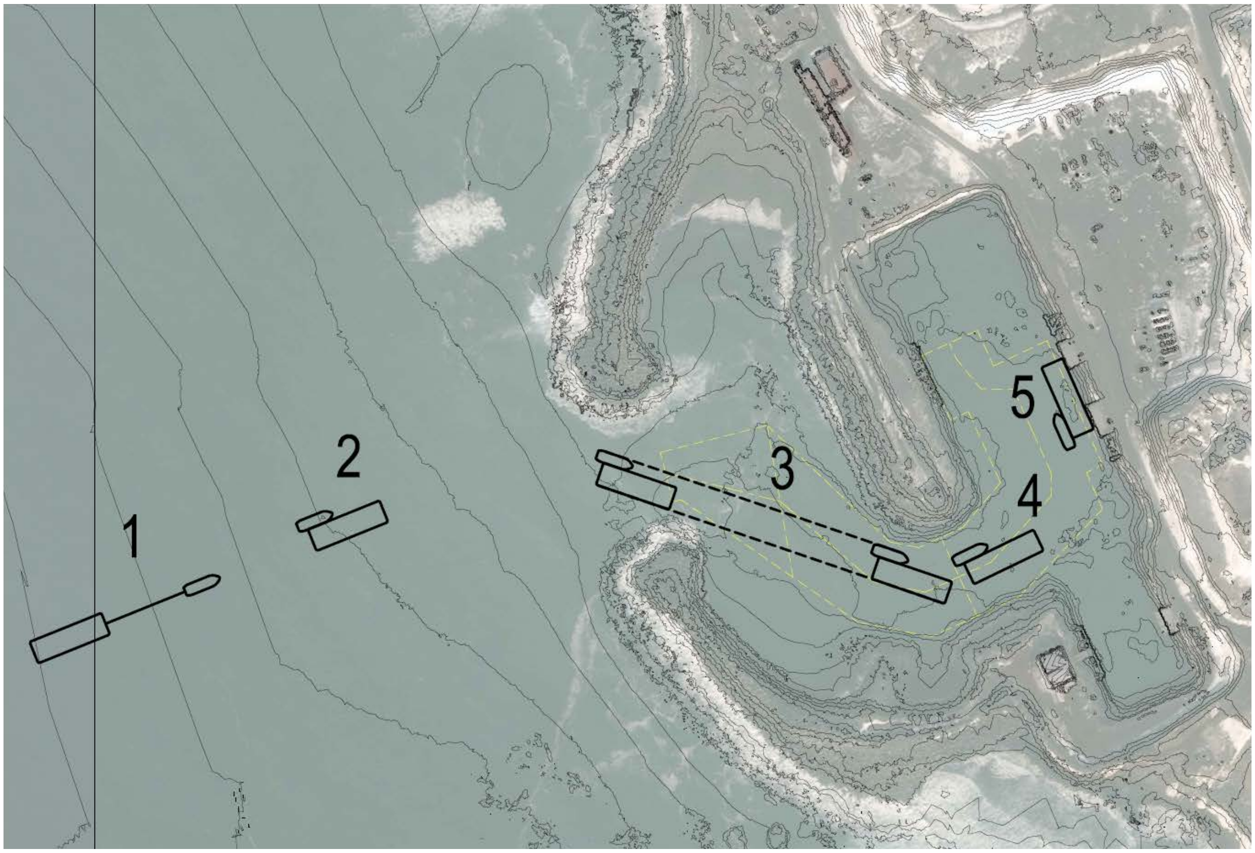


Figure A-13: Fuel Barge Entrance Navigation Diagram

Departure navigation out of the existing St. George Harbor is shown in Figure A-14. After making fuel deliveries at St. George, the fuel barge is nearly empty and rides near maximum freeboard, which is about 10 feet above the water surface. Exiting the harbor, the fuel vessel leaves the docks (1) backs up into the notch near the ice plant building (2) to begin its turn towards the outer harbor. At this point, the vessel is required to turn at a very slow speed to orient itself past the inner breakwater (3). The vessel then makes a slow path through the outer harbor and turns westward towards the outer breakwater opening (4). At this location, the vessel has minimal steerage and becomes most exposed to open ocean winds. Since fuel has been offloaded, the vessel also has maximum freeboard. The concern here for the operators is that a strong wind could overpower thrust and steerage causing the vessel to be blown into the shallows to the north of the harbor entrance. The operators cited this scenario as the cause of ships that have historically run aground at St. George. Once clear of the harbor, the vessel reconfigures back on the tow line for its return transit across the Bering Sea.

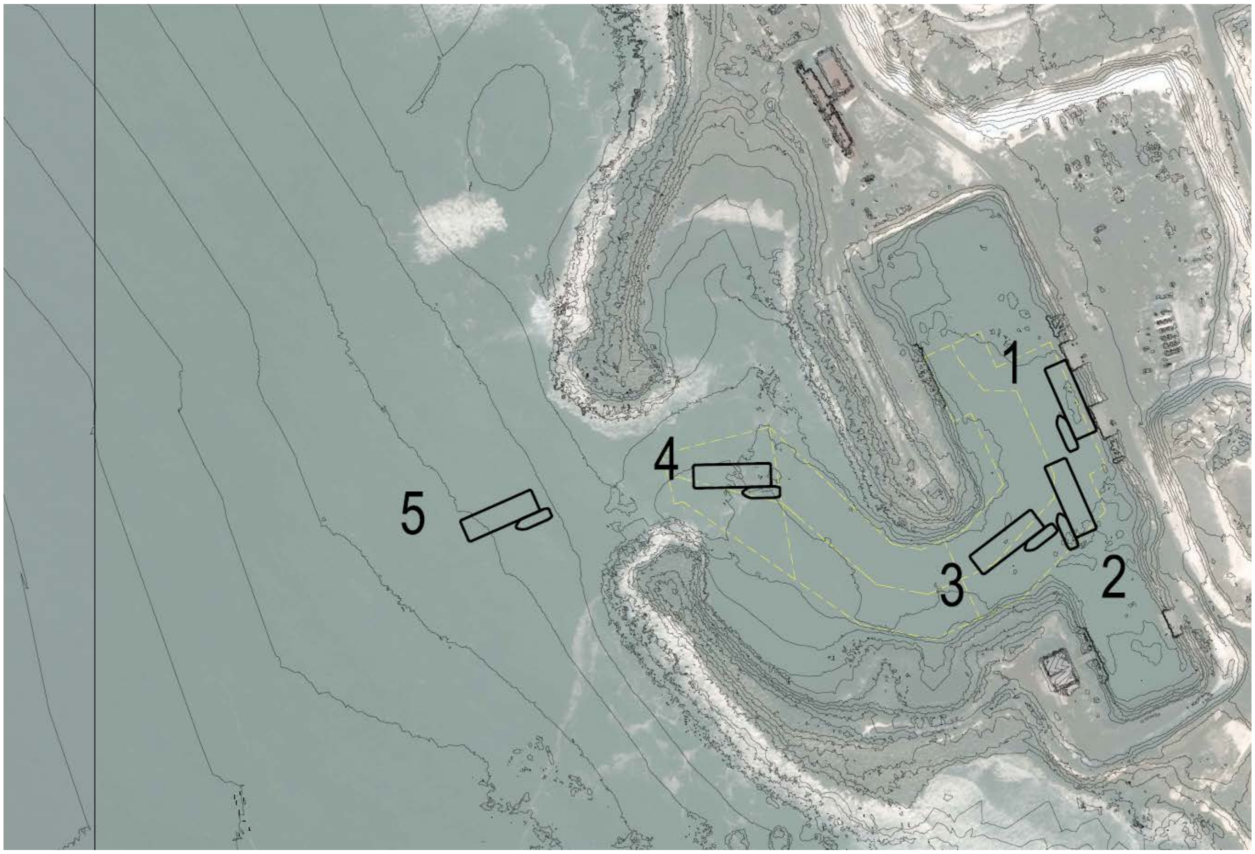


Figure A-14: Fuel Barge Departure Navigation Diagram

3.3 Allowable Wave Heights

3.3.1 Sea Conditions

Since the harbor site has open exposure to Bering Sea waves, there are times when the wave climate outside the harbor is too severe to allow vessels to operate. At St. Paul Harbor, the harbormaster typically closes navigation to and from the harbor when waves outside the harbor exceed 10 feet. Many vessel captains choose to use a lower threshold to decide when to attempt to enter or exit the harbor. The sea outside the harbor at St. George has more directional exposure to the Bering Sea than St. Paul and similar operating constraints are expected.

3.3.2 Outer Harbor

The outer harbor of St. George is used for navigation only and does not require the level of protection needed for a vessel to moor at a dock or raft with other vessels. No target wave conditions were designed for this portion of the harbor; when sea conditions allow for vessels to enter or exit the harbor, the breakwaters provide sufficient protection to allow vessels to navigate to the entrance. When wave conditions outside exceed these thresholds, vessels will not be in this area.

3.3.3 Inner Harbor

The inner harbor is designed to support fuel deliveries and commercial fishing activities. Due to the size of the vessels in the fishing fleet that would use the harbor, wave height criteria have been established accordingly. Wave heights of less than 2.5 feet, (0.75 meters) are assumed to be acceptable for the use of this fleet. This criteria was established in the design of St. Paul harbor where it was recognized that some wave action would transmit through the breakwater and affect vessels inside the harbor. When wave conditions exceed 2.5 feet at the dock, it is assumed that vessels will wait at anchor in the harbor to minimize damage from impacts with the dock. All model output for the alternatives studied at St. George are spectral peak waves. This means that waves larger than those reported by the model are expected to occur. The distribution of these waves in the harbor is not well defined. To account for these higher waves, the model output threshold for mooring was reduced to 1.6 feet, or 0.5 meters.

3.4 Channel and Basin Widths and Depths

3.4.1 Entrance Channel and Outer Harbor

The vessels making fuel deliveries to St. George also serve the community of St. Paul. The harbor configuration at St. Paul is a 250 foot wide channel dredged to -30 feet MLLW. The channel is perpendicular to the shore and makes a 90 degree turn to the south around the nose of the main breakwater. Through the turning section, the channel is 355 feet wide. Beyond the head of the breakwater, vessels pass between the main breakwater and the detached breakwater through a channel with a bottom width of 150 feet. Vessels enter and exit the harbor when waves outside of the harbor are 10 feet or less. Channel design for St. George follows similar criteria to St. Paul to accommodate the same vessels.

The channel depths were determined based on economic evaluations, design vessel draft, vessel motion in waves, squat, tide, safety clearance, advanced maintenance, and dredging tolerance. Pitch, roll and heave requirements are based on the most severe wave conditions in which vessels calling at St. George are expected to operate.

Tidal accessibility of the proposed outer entrance channel depths was based on the information shown Table A-4, which lists a range of channel depths and the percentage of time the channel would be accessible based on an analysis of observed water levels at St. Paul (Figure A-15) and assumed requirements for vessel motions and safety clearances. An entrance channel depths of -20 feet MLLW was determined to be the acceptable channel depth for the 10 foot draft vessels based on percentage of time usable. Design vessels were assumed to be loaded when entering the port for the alternatives at the proposed site. Therefore, loaded drafts were used to calculate required bottom depths for the entrance channel. Costs for construction and economic benefits for the various channel depths were evaluated in the Economic Appendix.

Table A-4: Tidal Accessibility at St. George

Tide Elevation (ft. MLLW)	-2	-1	0	1	2
% Time Accessible	100	99.5	94.3	80.1	56.9

Channel depth optimization procedures are outlined in ER 1105-2-100. The procedure includes evaluation of economic benefits, estimated costs, safety, efficiency, and environmental impacts. Refer the Economics Appendix for discussion of channel depth optimization.

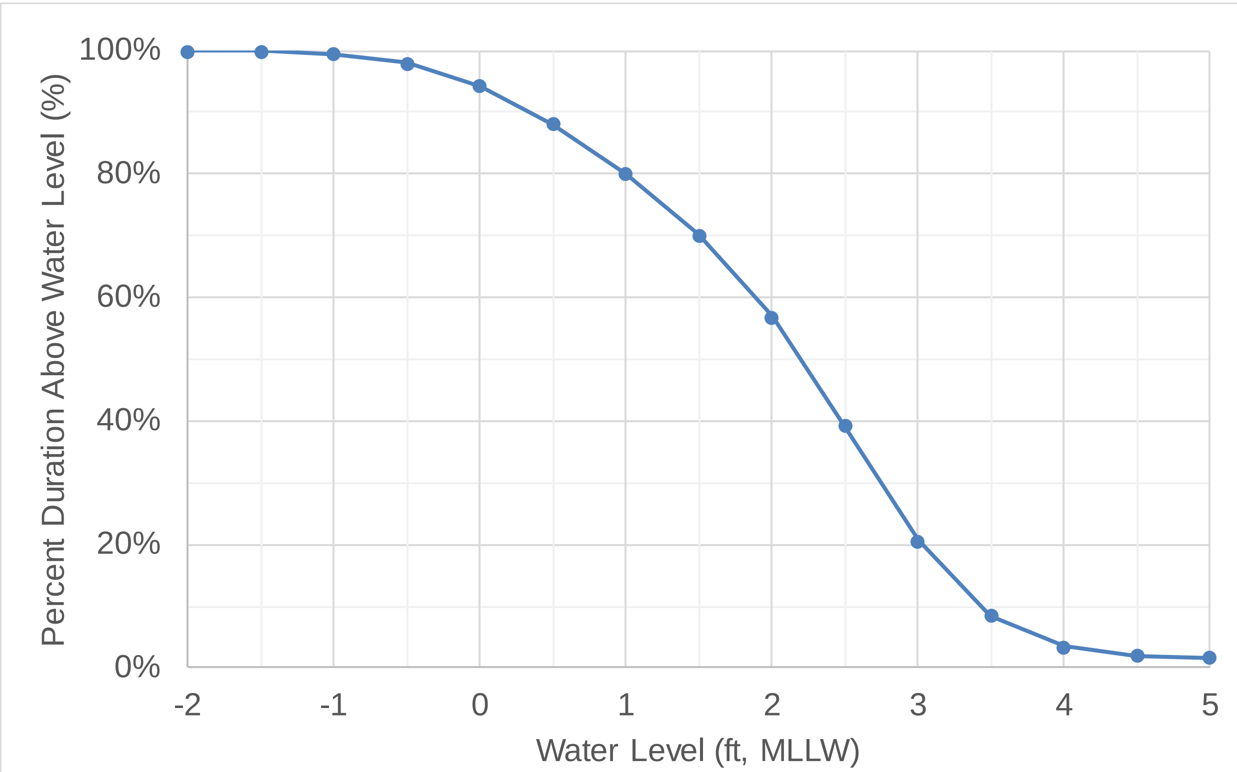


Figure A-15: Frequency of water levels at St. Paul, Alaska based on recorded water levels at St. Paul from October 2006 to October 2016.

Table A-5: Minimum Channel Depth Determination for Crabber Access

	Entrance Channel		Mooring Basin	
Crabber Channel Depth Criteria	Value (ft)		Value (ft)	
Water Level	0.0	ft. MLLW	0.0	ft. MLLW
Vessel Draft	14.0	ft.	14.0	ft.
Pitch, Roll, and Heave (2/3 of allowable wave height)	6.7	ft.	1.7	ft.
Squat	1.0	ft.	1.0	ft.
Safety clearance (based on rocky bottom)	3.0	ft.	3.0	ft.
Minimum Channel Depth	-24.7	ft. MLLW	-19.7	ft. MLLW

Table A-6: Minimum Channel Depth Determination for Barge Access

	Entrance Channel		Mooring Basin	
Fuel Barge Channel Depth Criteria	Value (ft)		Value (ft)	
Water Level	0.0	ft. MLLW	0.0	ft. MLLW
Vessel Draft	10.0	ft.	10.0	ft.
Pitch, Roll, and Heave (2/3 of allowable wave height)	4.0	ft.	1.7	ft.
Squat	1.0	ft.	1.0	ft.
Safety clearance (based on rocky bottom)	3.0	ft.	3.0	ft.
Minimum Channel Depth	-18.0	ft. MLLW	-15.7	ft. MLLW

Table A-7: Minimum Channel Depth Determination for Subsistence Fleet Access

	Entrance Channel		Mooring Basin	
Subsistence Fleet Channel Depth Criteria	Value (ft)		Value (ft)	
Water Level	0.0	ft. MLLW	0.0	ft. MLLW
Vessel Draft	4.0	ft.	4.0	ft.
Pitch, Roll, and Heave (2/3 of allowable wave height)	2.7	ft.	0.7	ft.
Squat	0.0	ft.	0.0	ft.
Safety clearance (based on rocky bottom)	3.0	ft.	3.0	ft.
Minimum Channel Depth	-9.7	ft. MLLW	-7.7	ft. MLLW

Dredging tolerance of 2 feet was assumed for a depth of -20 feet MLLW; therefore, it is anticipated that the construction contract for the deep draft navigation project would specify a required depth of -20 feet MLLW with a maximum pay line of -22 feet MLLW. Additional depth to account for advanced maintenance is not proposed.

3.5 Site Accessibility

To determine access availability for the harbor sites at St. George, hourly hindcast data from WIS stations were analyzed for exceedance of vessel operating thresholds. Hourly wave data was simulated for the period from 1985 through 2014. This wave data approximates sea conditions outside proposed harbors at Zapadni Bay and the North Site. WIS Station 82265 was used to represent Zapadni Bay conditions and WIS Station 82255 was used to represent North Site conditions. Applicability of these stations to their associated sites are discussed in paragraphs 5.3.1 and 0. Directional wave data was filtered to represent the sheltering effect the island has on conditions just outside the harbor sites. For the North Site, WIS Station 82255 was filtered to include waves originating from the north between 270 degrees and 090 degrees. For Zapadni Bay, WIS Station 82265 was filtered to include waves originating from the southwest between 120 degrees and 300 degrees (Figure A-16). Waves originating outside these arcs were assigned a height of 0 meters. The hourly wave data was filtered against access criteria for different classes of vessels in the design fleet to determine what percentage of time the harbor sites would be available for a vessel to enter the harbor. The duration exceedance analysis compares the number of hours in the record that wave heights exceed the vessel threshold criteria to the total number hours in the record (Figure A-17). The analysis generally shows that WIS Station 82255 has a shorter duration of wave heights exceeding any given threshold, though this is most pronounced at lower wave heights.

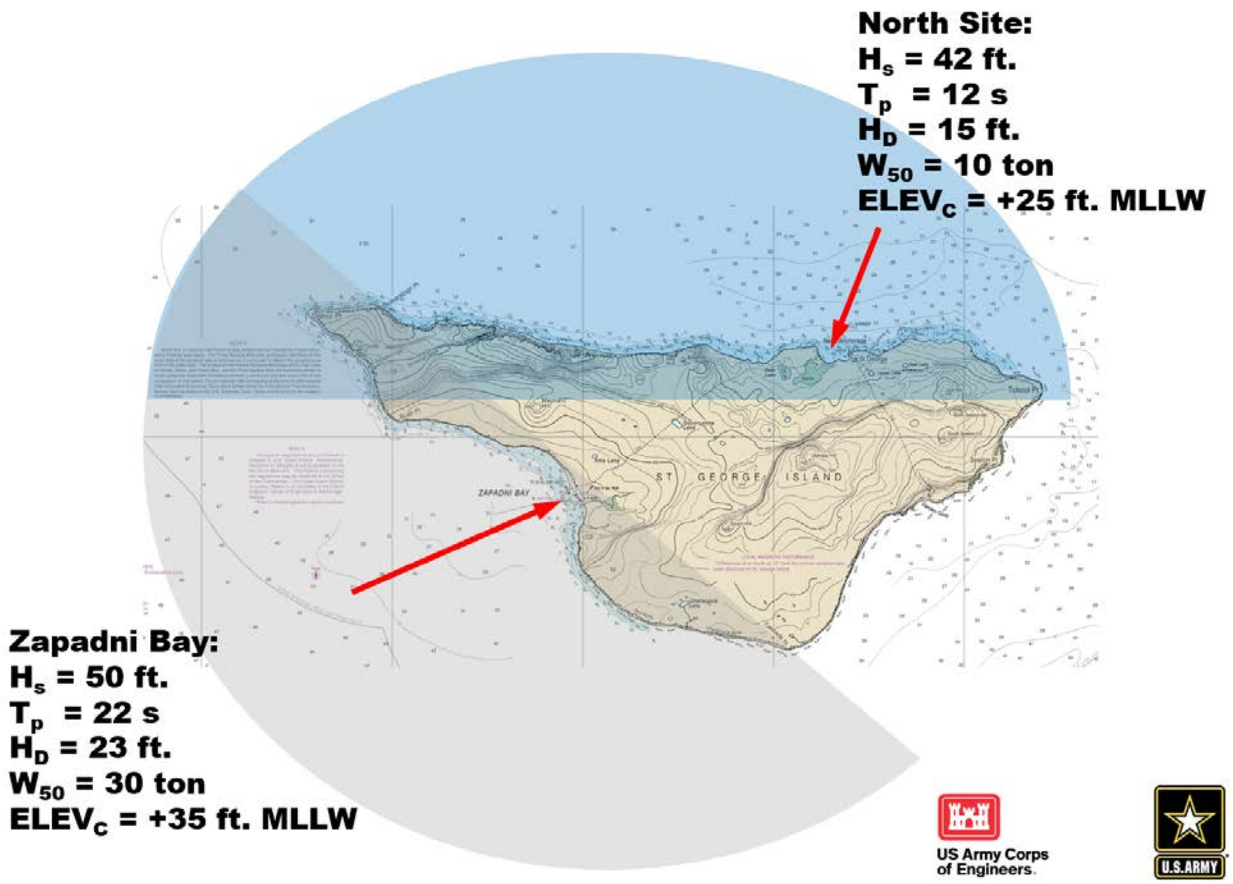


Figure A-16: Site comparison between Zapadni Bay and the North Site. H_s is the design deep water wave, T_p is the design spectral peak period, H_D is the design wave height at the outermost breakwater, W_{50} is the median armor stone weight of the primary breakwater and $ELEV_C$ is the design crest elevation of the outer breakwater.

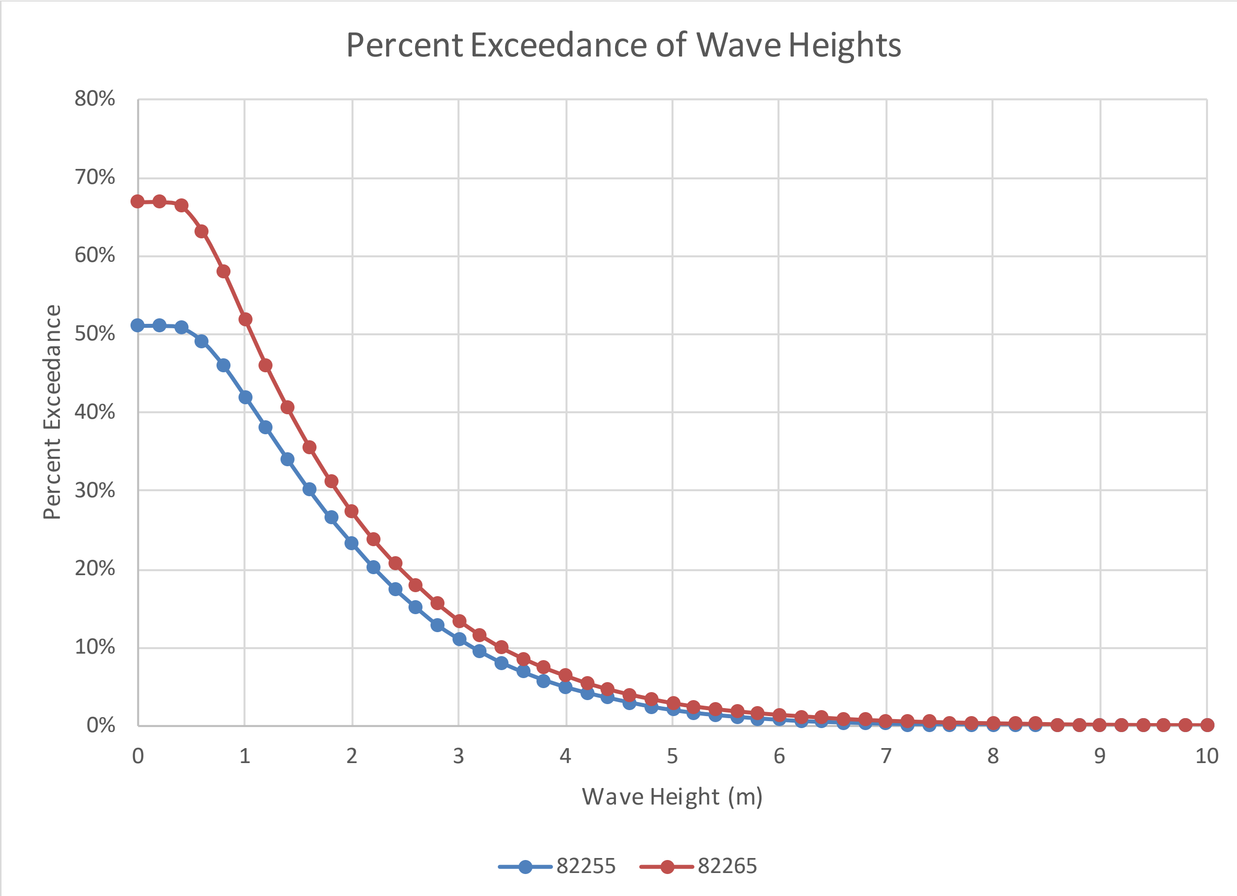


Figure A-17: Wave Height Duration Exceedance at WIS Stations 82255 and 82265.

This computes as a percentage of time which is also shown as number of days per year. Wave criteria for the fleet and wave exceedance durations for both sites are shown in Table A-8.

Table A-8: Vessel Operating Wave Threshold Exceedance at Study Sites

		Annual Harbor Accessibility Duration (%)			Annual Harbor Accessibility Duration (days)		
Vessel	Wave Criteria (m)	Zapadni Bay	North Site	Δ_{North}	Zapadni Bay	North Site	Δ_{North}
Fuel Barge	1	48%	58%	10%	175	211	36
Subsistence Vessel	1.2	54%	62%	8%	197	226	29
Crabber	3	87%	89%	2%	316	324	9

3.6 Circulation

The circulation aspects of the proposed harbors at St. George were evaluated based on guidance given in Engineering Manual (EM) 1110-2-1202 (USACE 1987). Tidal variation, storm surge, wave driven currents, ice effects, and wind stresses are factors that affect water circulation. It is estimated that the predominant mechanism that would drive water circulation would be wave and wind stress induced currents within the maneuvering areas and entrance channel. Tidal variation at St. George is approximately 3.3 feet.

The aspect ratio (length divided by width) guidance for harbor improvements at Zapadni Bay is difficult to determine. The outer basin is an irregular shape and open to westerly waves. This portion of the harbor can readily exchange water with the open ocean. Wave activity within the existing inner basin location of all alternatives remains significant during storm events. Length to width ratios can be taken of the inner basins of harbor configurations. The guidance for harbor circulation can be applied in a general sense for this study to show the relative differences in potential circulation between alternatives. It has been shown that aspect ratios of less than 3:1 reduce the potential for multiple circulation gyres to decrease the gross water exchange between the basin and ambient water. Another parameter used to evaluate harbor circulation is the ratio of the basin planform area (A) to the entrance cross-sectional area (a). Guideline values of A/a and $A/a^{1/2}w$ are given in Nece 1979. Typical values recommended are A/a < 400 and $A/a^{1/2}w < 100$ to ensure optimal basin configuration for flushing. Area ratios for selected alternatives are shown in Table A-9.

Table A-9: Indicator aspect ratios for circulation analysis

Basin Element	Aspect Ratio	A/a	$A/a^{1/2}w$
Without Project Condition	4.0:1	95	19
Alternative 1	4.3:1	85	23
Alternative 2	5.7:1	114	27
Alternative 3	7.8:1	210	44
Alternative 4	5.3:1	129	26
Alternative 5	1.4:1	204	15
Alternative 6	1.8:1	237	21
Alternative 7	2.0:1	383	29
Alternative N1	1.9:1	87	15
Alternative N2	1.5:1	93	13
Alternative N3	2.1:1	137	18

Rounding of basin corners may have some slight benefits in reducing local exchange in the “hot spots.” Also, the orientation and location of a single, central entrance channel is generally favorable in driving harbor circulation.

Typically for deep draft navigation projects, physical and numerical modeling studies are recommended in order to analyze the hydrodynamics of proposed channel improvements. For this study, circulation was evaluated using the best available guidance and analytical techniques. Detention time, volume of water exchange, mixing, dilution, and stratification would not be expected to change significantly any of the harbor alternatives studied.

3.7 Life-Cycle Breakwater Design

Armor stone for the proposed breakwaters at St. George was sized using the 50-year design wave forces expected to impact the structure. This was determined to be the most cost-effective means of protection for port alternatives considered. The average sea side armor stone size for a 50-year design at Zapadni Bay is 30 tons. There is a 2 percent chance of a 50-year design event happening in any given year throughout the 50-year design life. The chance goes up to 4 percent for a 25-year design. The percentage goes down to 1.3 percent for a 75-year design level and to 1 percent if a 100-year design level is used. Due to the depth-limited nature of the coastline at St. George, there is minimal difference in cost between armor stone sized for a 25-year event versus a 50-year event. Rock for the project would likely either be barged from the quarry at Cape Nome to the project location. The Cape Nome quarry is the closest likely source to the project and has the capacity to produce 30 ton armor stone. Replacement costs are estimated to be relatively high because the project location is very remote and mobilization costs are substantial. A 75- or 100-year design would reduce the frequency and magnitude of needed maintenance, however design conditions for these events are not well known due to the period of record of data available at the site and there is less certainty that basing the design on a lower frequency event would produce a structure that would be capable of withstanding events of greater severity than those observed and studied. A 50-year design provides the optimum balance between minimizing maintenance requirements and the cost of procuring the stone for repairs. The loss or damage to a relatively small amount of armor stone over time would have little to no effect on the operation and use of the port; therefore, there was not sufficient justification for basing the design on a life-cycle horizon beyond the 50-year level.

3.8 Dredging

Dredging limits were determined based on vessel maneuvering characteristics as a function of length, beam, whether or not tug assist would be provided, turning radii, traffic, and wind conditions. Side slopes of 3H:1V were assumed based on the character of dredged material anticipated (sands, gravel, cobbles, and glacial till). Such side slopes would be stable and rock slope protection would not be necessary for placement on the side slopes.

A minimum offset bench width distance of 15 feet horizontal between the top of the dredge cut slope and the toe of any causeway or breakwater structure is recommended. For purposes of dredging adjacent to the proposed dock faces, the required depth can abut to the dock faces.

The maximum dredging depth determined for the site was to -25 feet MLLW. Previous studies have indicated a need to drill and blast 2 feet below the design depth to produce an efficient pattern to loosen the material for excavation. Dredging tolerances were assumed to be 2 feet due to the coarse nature of the material around the island and the potential need for blasting to remove it.

4. PHYSICAL MODEL STUDIES

For purposes of this study, physical modeling for wave analysis was beyond the scope, budget, and schedule. Due to the extreme wave climate and harbor resonance problems known to exist at St. George, physical modeling will be required prior to publishing of plans and specifications for harbor construction. This step is necessary to validate numerical model results and to identify harbor-specific hydrodynamic issues that the numerical models are not capable of replicating. This study needs to be performed in a facility dedicated to wave modeling run by full time research engineering staff. The Corps owns and operates the necessary facilities at the Corps' Engineer Research and Development Center (ERDC) Coastal Hydraulics Laboratory in Vicksburg, MS.

Physical modeling studies for design of the Port of Nome were used to validate the breakwater structures' capability to resist ice forces. For the Port of Nome, scale model testing showed that a minimum armor stone size of 8 tons was required on a 2H:1V slope to be stable for ice sheet impact. The design waves found around St. George require larger stones to be needed to survive wave attack, and prescriptively, due to the island's location, ice sheets are expected to be thinner and produce less force than at Nome. For the purposes of this study, design wave height controls armor stone size.

5. NUMERICAL MODEL STUDIES

Numerical modeling of wave conditions at St. George used a three-tiered method that employed three separate models. Deep water wave conditions were analyzed from the WIS results published by the Coastal Hydraulic Laboratory. WIS data was binned and sorted to represent the appropriate approach directions and seasons for finer analysis. The WIS data was then used as the boundary conditions for STWave models to simulate nearshore wave transformation as the deep water waves approach the shoreline. STWave results provided design wave conditions for breakwater design and boundary conditions for harbor response modeling. The last phase of modeling primarily used the FUNWAVE model to simulate wave propagation through the harbor alternatives developed for the study sites. The model development process and run results will be presented for each site separately.

4.1 Elevation Data

A large section of the Bering Sea covering the Pribilof Islands was modeled bathymetrically in SMS to create a baseline scatter set from which numerical models could be run. Bathymetric data was collected from the National Ocean Service (NOS) website. The NOS website contains a database of digitized survey data that can be used to build elevation models of site bathymetry. The surveys used for the elevation model are H07914, H07948, H08002, H08003, H08004, H08072, H08074, H08075, H08121 and H11095. Most of this survey data covers bathymetry around the Pribilof Islands and was collected between 1950 and 1955. H11095 was a coastline survey of St. George Island performed in 2006 using LiDAR, however weather conditions for that season were not favorable and this survey is considered to have a high degree of uncertainty and should be validated with new site survey information as the study progresses. Project condition surveys for St. Paul harbor were added to the NOS data to provide high resolution data of the harbor. Surveys performed by TERASOND at Zapadni Bay in 2013 and 2016 were added to provide better nearshore and harbor bathymetry. Topography for the village of St. George was added by tracing survey contours from the community map published by the State of Alaska.

4.2 Wave Data Collection

Efforts were made to collect wave data at Zapadni Bay prior to this study. Three ADCPs were deployed to the seabed in the harbor and offshore at Zapadni Bay in 2013. These instruments recorded the free water surface and velocity profiles from September 18 to November 17, 2013. The sensors were set to record data at half-second intervals for a 20-minute period over each hour of the deployment period. On November 7, the sensors captured a long period storm event which is shown as run number 17 in Table A-12. This event was used to measure the modelling process effectiveness at reproducing observed problems in the harbor. Water depths measured during the peak event are shown in Figure A-19.

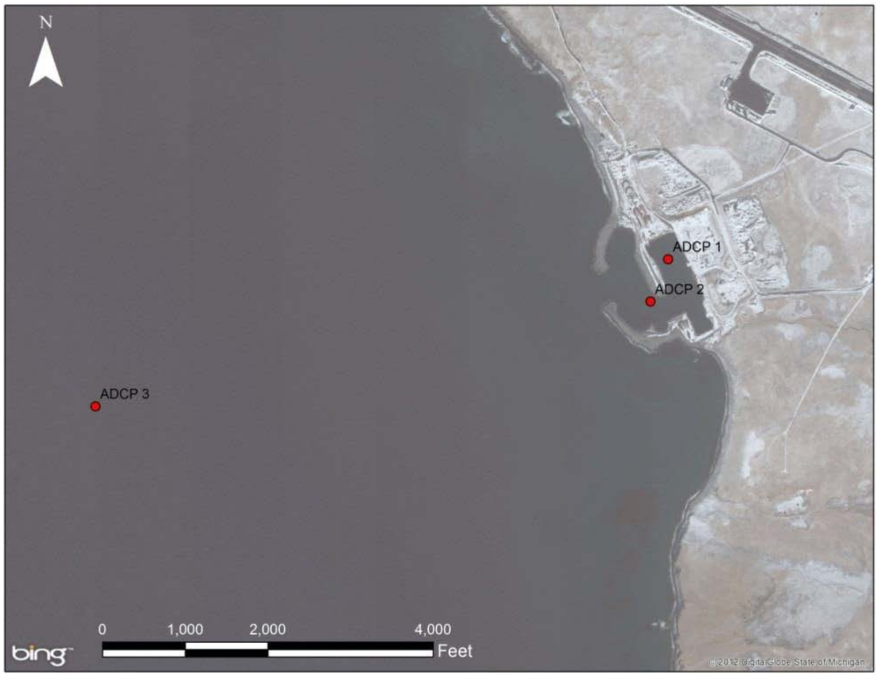


Figure A-18: ADCP deployment locations from HDR Baseline Conditions Report (2014).

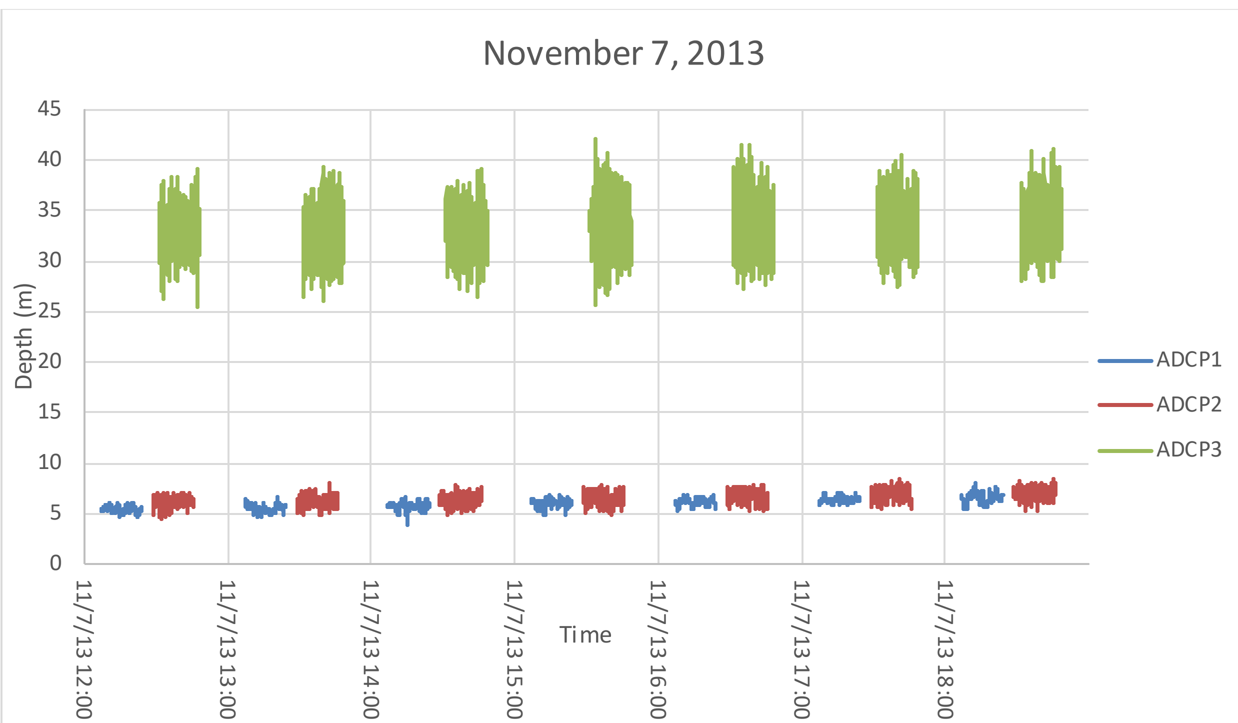


Figure A-19: ADCP measurements during November 7, 2013 storm

4.3 Deep Water Conditions

A wave hindcast was performed by ERDC. Deep water wave conditions have been hindcast at offshore points along the entire coast of Alaska in the Alaska WIS. It should be noted that St. George Island is not represented in the WIS model grid and the sheltering effect of the island for energy generated from different directions is only accounted for at St. Paul. To account for the sheltering effect of St. George Island from different storm directions. WIS points around St. Paul Island were taken from the same relative position to that island as St. George. The wave energy at the WIS points around St. Paul were then applied to locations around St. George Island at the same depths and representing similar orientation relative to the island. Since the two islands are only 50 miles apart and both are more than 200 miles away from the nearest landmasses, it is assumed that storms passing through the region will create very similar deep water conditions around both islands. Also, due to the depth-limited nature of wave energy dissipation along the coastline of St. George, differences in offshore climate between St. Paul and St. George are washed out in the nearshore zone wave transformation process.

4.3.1 Zapadni Bay Offshore Wave Climate (WIS)

Wave height results based on 1985-2014 wind and pressure fields for WIS Station 82265 are applicable for Zapadni Bay. This station is located southwest of St. Paul in water depths of approximately 71 meters. These stations occupy an appropriate relative position to St. Paul Island to represent waves affecting St. George (Figure A-20). The 50-year wave height is estimated at 15.19 meters using the log relationship developed for WIS Station 82265 as shown in Figure A-21. Wave periods are estimated to be in the 18 to 20 second range. The frequency of occurrence relationship for waves at WIS Station 82265 is shown in Table A-10. The location of Zapadni Bay on the southwest coastline of St. George Island exposes the harbor to waves originating from the south, southwest and west sectors and shelters the harbor from northerly and easterly directions. The strongest storm signals in the region tend to originate from the southwest and west directions, so the non-directional wave height frequency of occurrence relationship for this station would adequately represent the west and southwest storms expected to affect the site.

Table A-10: Southern exposure WIS Station analysis, H_{m0} in meters

Return Period (years)	82264	82265	82266	82267	Tp (s)
2	10.24	9.91	9.73	9.77	16
10	12.82	12.55	12.37	12.26	18
20	13.92	13.69	13.50	13.34	18
50	15.39	15.19	15.00	14.76	20
100	16.50	16.33	16.13	15.84	22

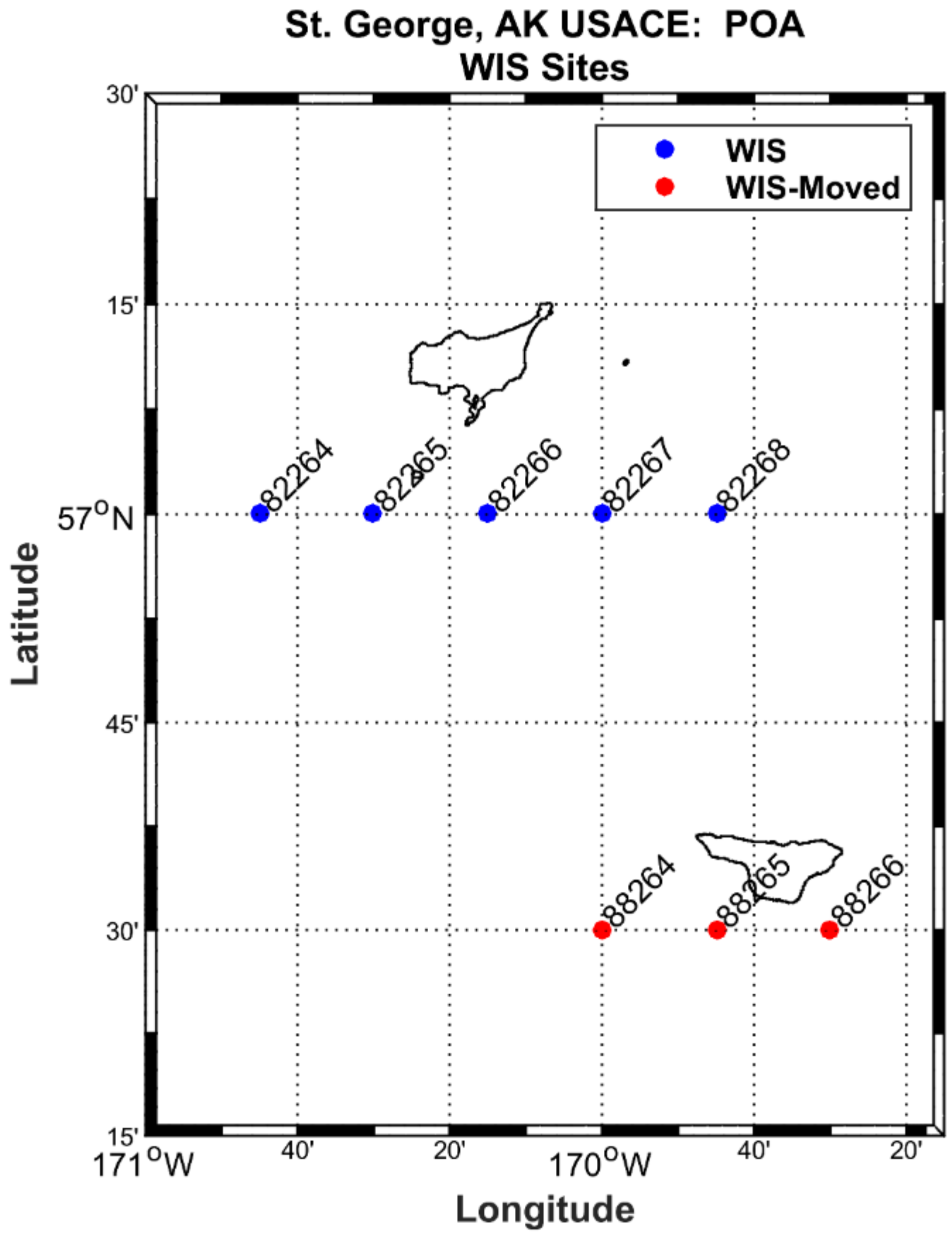


Figure A-20: Location of south exposure WIS station at St. Paul and super positioning to St. George

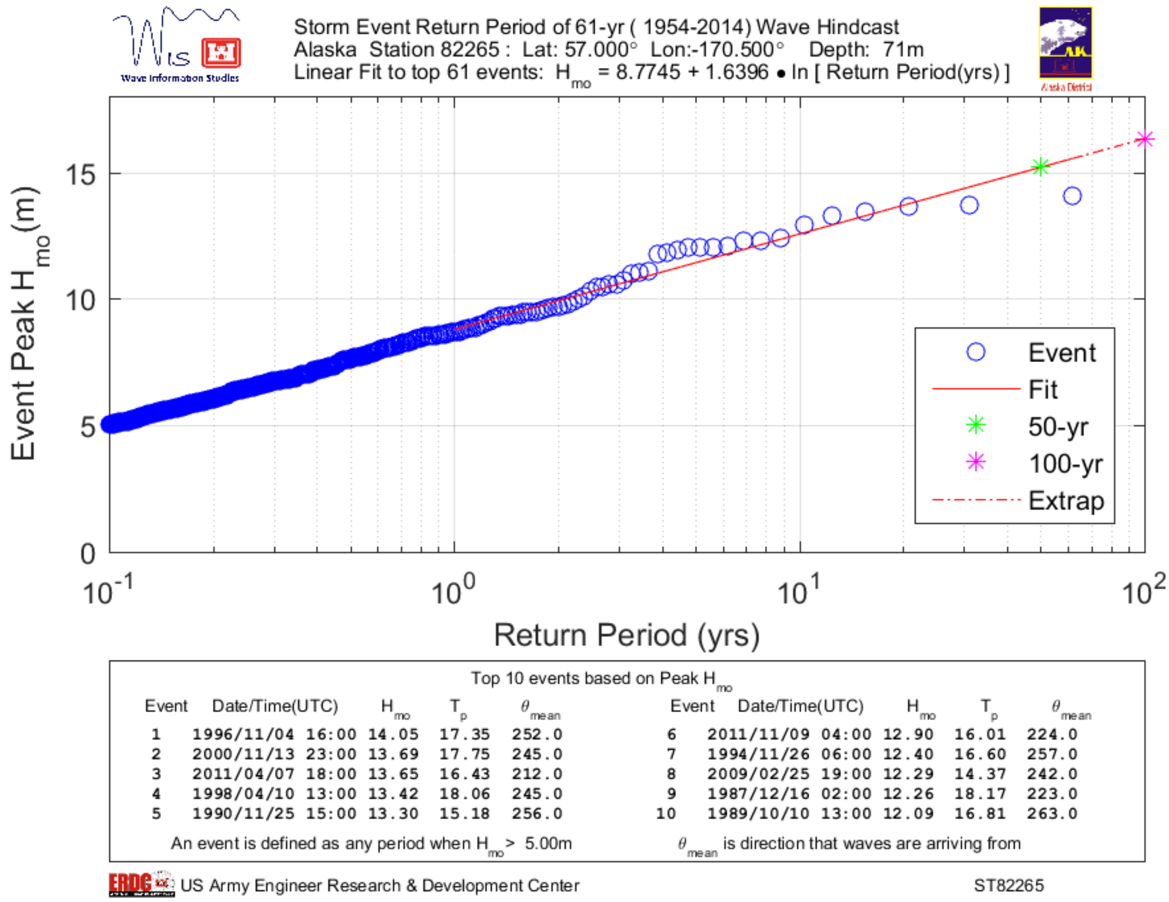


Figure A-21: Analysis figure for WIS station 82265 (Jensen, 2011)

The data from this WIS station was developed in two ways. A series of 19 hindcast storm events affecting the site were extracted from the data and run as storm scenarios for STWave and FUNWAVE modelling efforts. The storms were filtered so that only storms originating from the arc from 180 degrees (due south) to 300 degrees (west-northwest) were considered. Storms originating from other directions were assumed to be filtered by the presence of the island. The storm simulations range in peak spectral height from 8.85 meters to 14.05 meters and peak spectral periods from 13.27 seconds to 19.12 seconds.

4.3.1.1 WIS Station 82265 Duration Exceedance Analysis

A duration analysis of the ONELINES data of WIS Station 82265 was also performed to determine availability of the harbor and docks for navigation and mooring purposes. This was performed by evaluating the hourly data from 1985 to 2014 with a series of thresholds. The data was first binned into 15 degree arcs. The data in the arcs from 180 to 300 degrees were assumed to affect the site at Zapadni Bay. This section describes modeled wave conditions confined to this arc, however as the study progressed, it was decided that long period waves from 120 degrees to 180 degrees would also affect conditions at the harbor entrance and were included in the site comparison analysis.

Data with waves originating from other directions were treated as zeroes. Occurrence of waves were compared to data thresholds of half meter wave height increments. The percentage of time for wave heights exceeding the threshold was calculated in comparison to the duration of the entire record and duration exceedance curves developed for each 15 degree bin and for the whole arc under consideration. The data was sorted by direction and by month to determine any trends in high wave events. Sorting the data by direction, there was a noticeable tendency for waves to originate from the southwest with a noticeable peak between 240 and 255 degrees (Figure A-22).

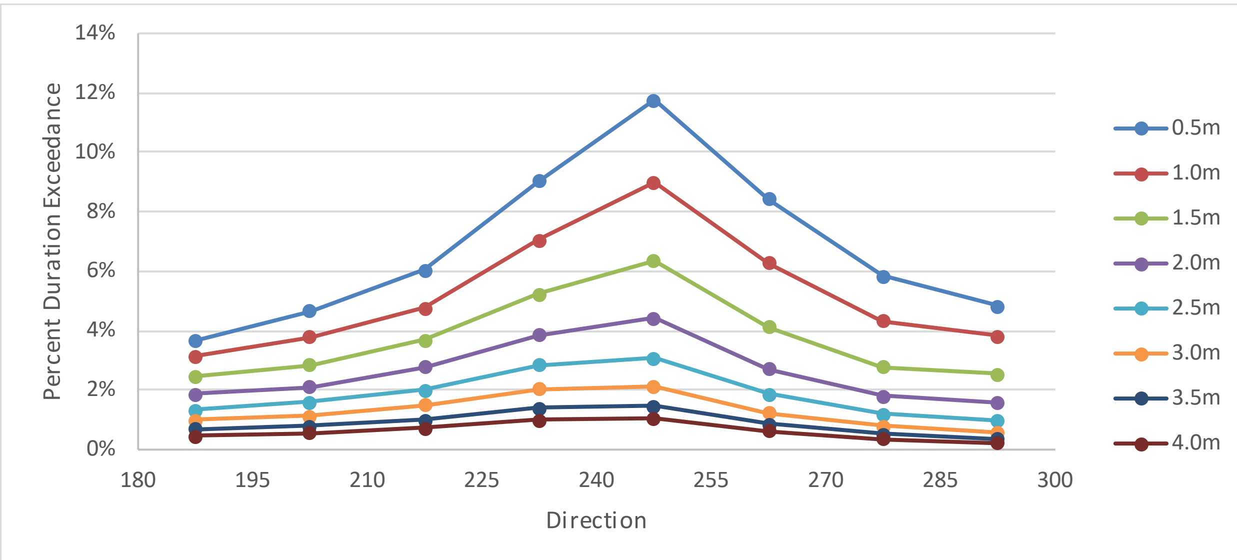


Figure A-22: WIS Station 82265 Wave Height Exceedance from 180 degrees to 300 degrees.

Aggregate wave height exceedance throughout the entire arc was calculated. This analysis is simply a sum of all directional bins within the arc (Figure A-23).

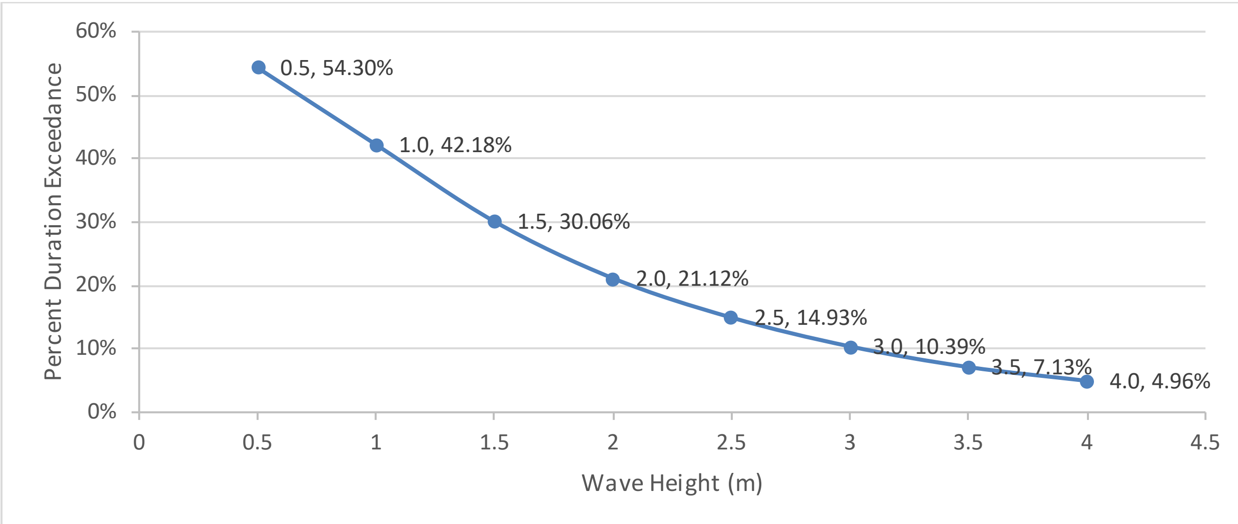


Figure A-23: WIS Station 82265 Annual Wave Height Exceedance from 180 to 300 degrees.

Sorting the data by month, conditions were noticeably rougher in the November and December period and calmer in the June and July period (Figure A-24). This agrees well with local knowledge of the timing of storms which typically occur in the fall and winter.

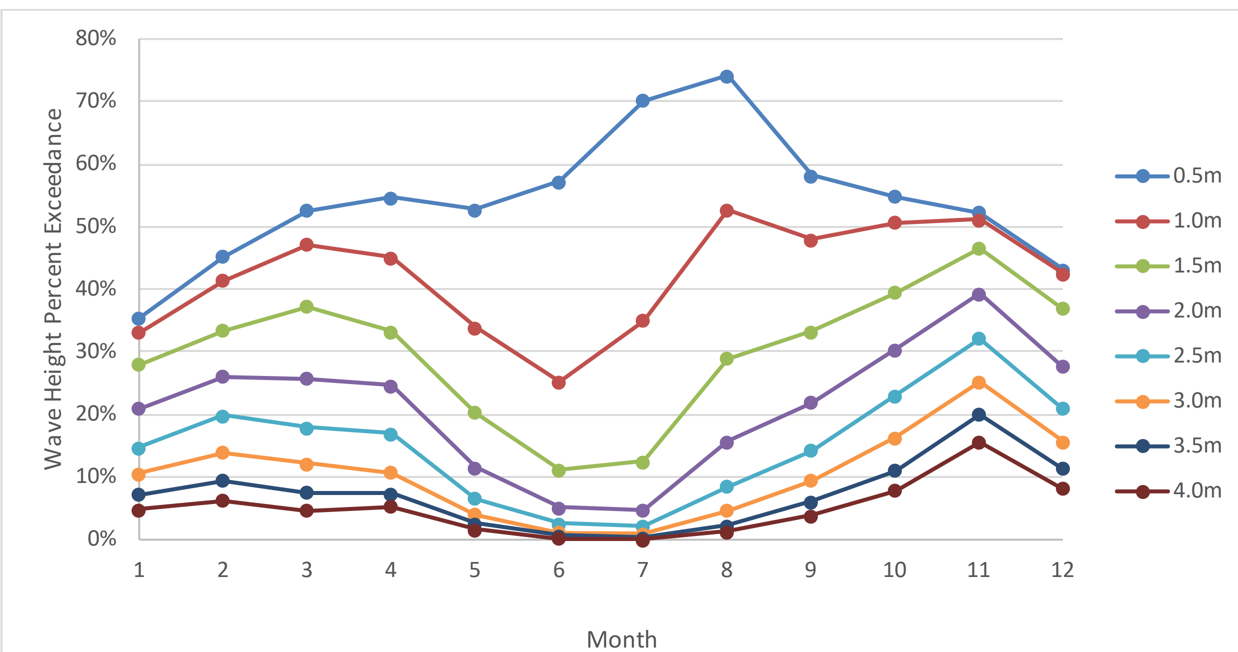


Figure A-24: WIS Station 82265 Wave Height Exceedance by Month from 180 degrees to 300 degrees.

A final duration exceedance analysis was performed of the station from the months of October through March of each year to simulate offshore conditions during the anticipated crabbing season. The results of this curve were slightly different than the annual curve. Wave heights in excess of 4 meters occurred for a longer duration over the crabbing season than over the entire year. The duration exceedance analysis provides information regarding the potential benefits of project alternatives.

4.3.2 North Site Offshore Wave Climate (WIS)

Wave height results based on 1985-2014 wind and pressure fields for WIS Station 82255 are applicable for the north site. This station is located northeast of St. Paul in water depths of approximately 71 meters. These stations occupy an appropriate relative position to St. Paul Island to represent waves affecting St. George (Figure A-25).

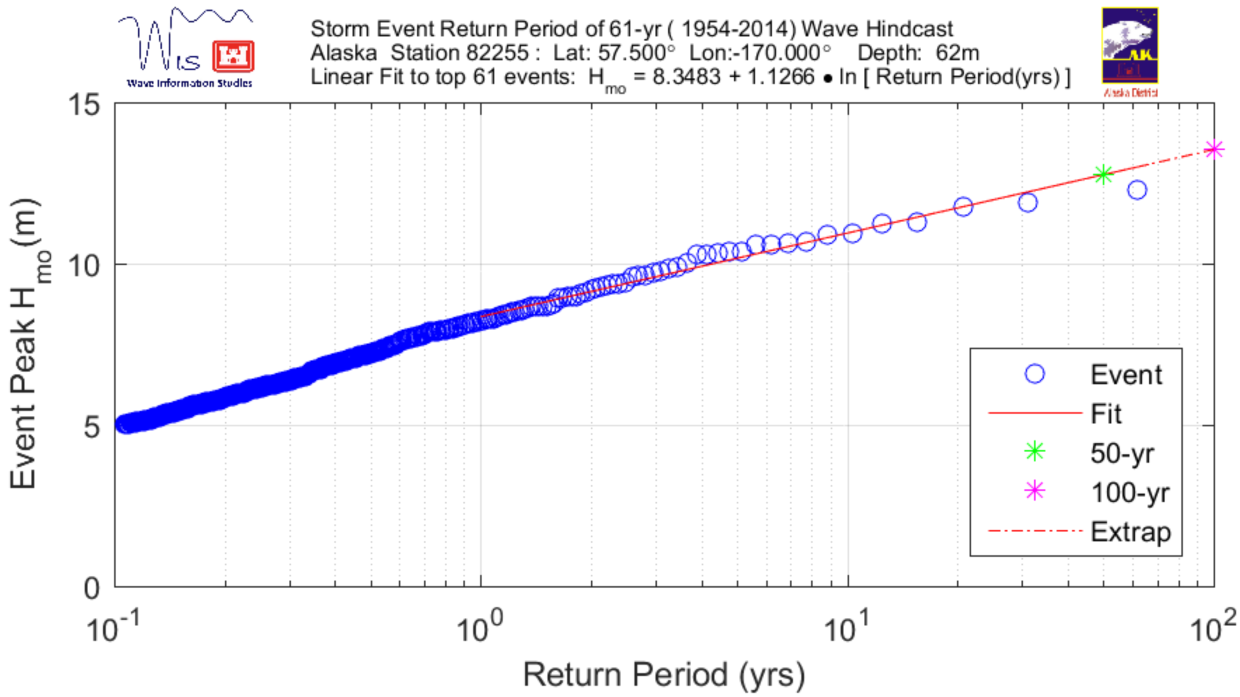
Figure A-25: WIS Station transformation for North Site Analysis.

The 50-year wave height is estimated at 12.76 meters using the log relationship developed for WIS Station 82255 as shown in Figure A-26. Wave periods are estimated to be in the 12 to 16 second range. The frequency of occurrence relationship for waves at WIS Station 82255 is shown in Table A-11. The location of the north site exposes potential harbors to waves originating from the west, northwest and north sectors and shelters the harbor from southerly directions. The strongest storm signals in the region tend to originate from the southwest and west directions. The position of Station 82255 tends to filter some of the southwest energy as storms pass over St. Paul Island, however some long period energy is expected to wrap around the island and converge again before reaching the WIS Station. Directional sorting of the ONELINES data shows a decrease in maximum wave heights when waves from the southerly directions are excluded.

Table A-11: North Site WIS Station analysis, H_{m0} in meters

Return Period (years)	82254	82255	82256
2	9.00	9.13	9.13
10	11.03	10.94	10.90
20	11.90	11.72	11.66
50	13.05	12.76	12.67
100	13.92	13.54	13.43

Periods associated with storm events were interpreted from the storm records. Directional analysis of the distribution of spectral peak periods showed that storms originating from the west generally had periods in the 14 to 16 second range while storms originating from the northwest through the east generally had peak periods in the 12-13 second range.



Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}	Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}
1	1955/10/03 09:00	12.28	16.18	278.0	6	1989/10/10 13:00	10.94	17.16	265.0
2	1990/11/25 15:00	11.88	15.12	259.0	7	1991/12/22 07:00	10.87	12.52	40.0
3	1996/12/08 13:00	11.76	16.13	277.0	8	2000/11/04 08:00	10.67	13.63	109.0
4	1996/11/04 16:00	11.28	17.33	253.0	9	2005/10/17 19:00	10.63	13.40	285.0
5	1994/11/11 21:00	11.25	17.84	265.0	10	2011/11/09 04:00	10.57	15.23	215.0

An event is defined as any period when H_{mo} > 5.00m θ_{mean} is direction that waves are arriving from

ERDC US Army Engineer Research & Development Center ST82255

Figure A-26: Analysis Figure for WIS station 82265 (Jensen, 2011)

The data from this WIS station was developed in two ways. A series of nineteen hindcast storm events affecting the site were extracted from the data and run as storm scenarios for STWave and FUNWAVE modelling efforts. The storms were filtered so that only storms originating from the arc from 270 degrees (due west) to 000 degrees due north) were considered. Storms originating from other directions were assumed to be filtered by the presence of the island or proposed harbor geometry. The storm simulations range in peak spectral height from 8 meters to 11.76 meters and peak spectral periods from 12 seconds to 16.13 seconds

4.4 Nearshore Wave Transformation

The WIS data from stations 82255 and 82265 represent the offshore condition at the study sites. To model harbor response to storm events, the offshore conditions need to be transformed to a nearshore condition to allow a harbor model to run. Using STWave as an intermediate model allows energy from the transformed WIS station location to be propagated over measured bathymetry and transformed into reasonable boundary conditions for a high resolution harbor model to run.

4.4.1 Zapadni Bay Nearshore Wave Transformation (STWave)

The STWave domain for Zapadni Bay covers the area from a depth of about 75 meters offshore to the shoreline of St. George Island. The model grid covered the area from Rush point to the west of Zapadni Bay to the Red Bluffs to the south of Zapadni Bay as shown in Figure A-27.

The model grid used to perform the nearshore wave transformation was extracted from NOS bathymetry of the size and placed on a 20 meter by 20 meter grid. The grid was oriented at 25 degrees in SMS which corresponds to waves approaching from 245 degrees. The model grid has 360 cells in the i direction and 432 cells in the j direction for overall grid dimensions of 7200 by 8640 meters. The model was run in the half-plane mode with propagation of the boundary conditions only, no wind propagation. The water level for all STWave runs was set to +1.8 meters MLLW to account for additional depth of storm surge and setup.

Output stations were selected near the shoreline of Zapadni Bay to allow a smaller finer grid to be developed for the FUNWAVE runs for the harbor. The STWave model produced results as expected in a depth limited environment. While storms of varying wave heights between 8.85 and 14.05 meters were run from the offshore boundary, results at the output points near the harbor all fell within a narrower band of wave heights between 9.18 and 11.34 meters. While input directions varied from 202 to 273 degrees, the wave vectors at the FUNWAVE model boundary generally fell within a 30 degree band centered about waves directly entering the outer breakwaters of the existing harbor. This is caused by diffraction of the incident wave energy over the nearshore bathymetry of the island. Storm simulation run results over the STWave model are shown in Table A-12. Note that while the wave direction from the WAM model is meteorologic, the direction of resultant waves from STWave are based on the orientation of the grid. A positive direction indicates angle counterclockwise of the i direction of the STWave grid. Storm simulations were run at ERDC on the TOPAZ HPC to facilitate processing of the storm events.

Table A-12: Simulated Zapadni Bay Storm STWave model results

RUNNO	HWAM (m)	TWAM (s)	THWAM (degrees)	HSTWave (m)	TSTWave (s)	THSTWave (degrees)
1	14.05	17.35	252	11.18	17.99	10
2	13.69	17.75	245	11.34	19.8	20
3	13.65	16.43	212	10.98	16.34	27
4	13.42	18.06	245	11.18	17.99	17
5	13.3	15.18	256	10.98	16.34	6
6	12.9	16.01	224	11.18	17.99	16
7	12.4	16.6	257	10.98	16.34	7
8	12.29	14.37	242	10.75	14.86	14
9	12.26	18.17	223	11.34	19.8	20
10	12.09	16.81	263	11.18	17.99	3
11	12.03	17.69	261	11.18	17.99	4
13	12.01	17.57	221	11.18	17.99	23
14	11.91	15.86	273	10.98	16.34	1
15	11.79	16.08	241	10.98	16.34	14
16	11.78	16.27	202	10.15	16.34	30
17	11.07	19.12	256	11.34	19.8	12
18	11.02	17.4	246	11.18	17.99	10
24	10.45	15.11	251	10.88	16.34	7
32	9.65	13.5	272	9.23	13.51	3
33	9.62	13.55	227	9.59	13.51	17
36	9.48	15.37	249	10.97	16.34	9
37	9.48	14.92	242	9.18	14.86	15
40	9.38	13.77	256	10	14.86	6
44	9.31	16.39	229	10.85	16.34	15
45	9.31	14.78	256	10.07	14.86	6
47	9.21	14.59	234	10.2	16.34	14
49	9.08	13.27	246	9.31	13.51	8
51	8.99	16.6	223	10.33	17.99	18
52	8.96	16.89	245	10.87	17.99	11
54	8.85	14.45	243	9.59	14.86	11

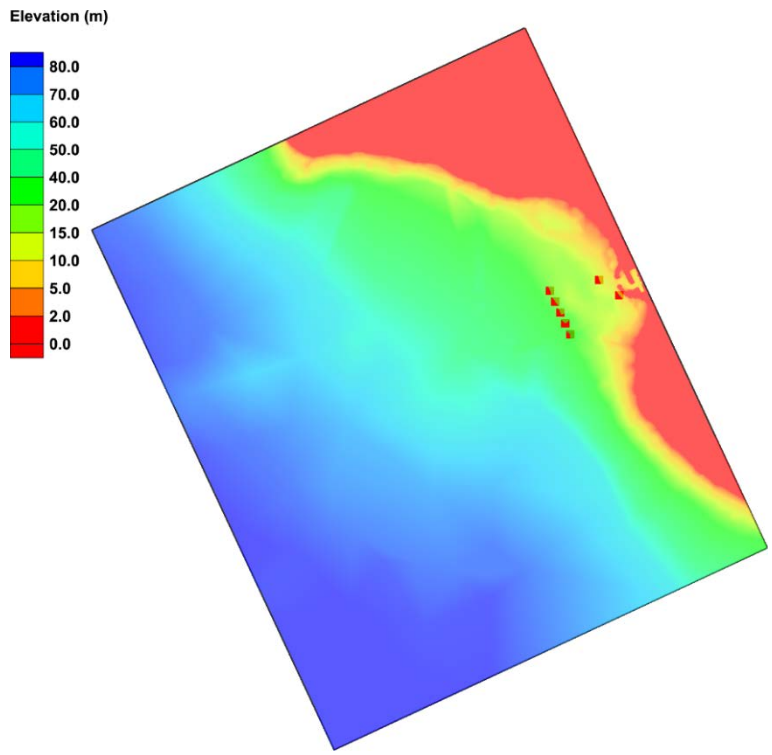


Figure A-27: STWave grid bathymetry and output locations.

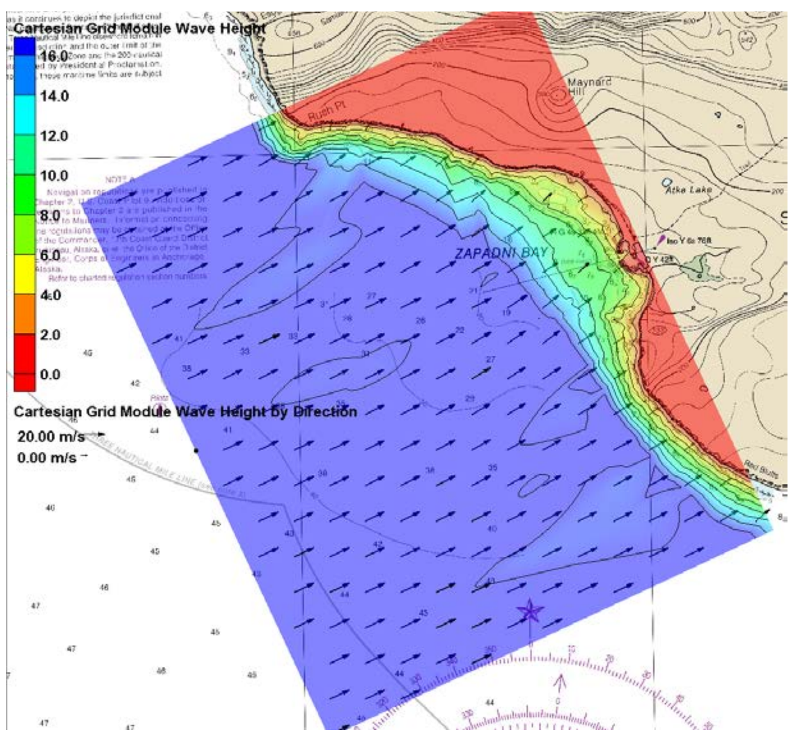


Figure A-28: STWave results for Hm0 = 16.33 m and Tp = 22 s.

4.4.2 North Site Nearshore Wave Transformation (STWave)

WIS data from station 82255 represents the offshore condition at the north site. To model harbor response to storm events, an STWave model was used in the same manner as for developing conditions at Zapadni Bay. The STWave domain for the north site covers the area from a depth of about 75 meters offshore to the shoreline of St. George Island. The model grid was oriented to propagate storms originating from the northwest as shown in Figure A-27.

The model grid used to perform the nearshore wave transformation was extracted from NOS bathymetry of the size and placed on a 25 meter by 25 meter grid. The grid was oriented at 25 degrees in SMS which corresponds to waves approaching from 315 degrees. The model grid has 360 cells in the i direction and 432 cells in the j direction for overall grid dimensions of 7200 by 8640 meters. The model was run in the half-plane mode with propagation of the boundary conditions only, no wind propagation. The water level for all STWave runs was set to +1.8 meters MLLW to account for additional depth of storm surge and setup.

Output stations were selected near the shoreline of the north site to allow a smaller finer grid to be developed for the FUNWAVE runs for the harbor. The STWave model produced results as expected in a depth limited environment. While storms of varying wave heights between 8.9 and 14.1 meters were run from the offshore boundary, results at the output points near the harbor all fell within a narrower band of wave heights between 9.2 and 11.3 meters. While input directions varied from 277 to 341 degrees, the wave vectors at the FUNWAVE model boundary generally bent towards a shore-normal direction approaching perpendicular to proposed harbor entrance. This is caused by diffraction of the incident wave energy over the nearshore bathymetry of the island. Storm simulation run results over the STWave model are shown in Table A-13. Note that while the wave direction from the WAM model is meteorologic, the direction of resultant waves from STWave are based on the orientation of the grid. A positive direction indicates angle counterclockwise of the i direction of the STWave grid. Storm simulations were run at ERDC on the TOPAZ HPC to facilitate processing of the storm events.

Table A-13: Simulated North Site Storm STWave model results.

RUNNO	HWAM (m)	TWAM (s)	THWAM (degrees)	HSTWave (m)	TSTWave (s)	THSTWave (degrees)
1	14.05	17.35	252	11.18	17.99	10
2	13.69	17.75	245	11.34	19.8	20
3	13.65	16.43	212	10.98	16.34	27
4	13.42	18.06	245	11.18	17.99	17
5	13.3	15.18	256	10.98	16.34	6
6	12.9	16.01	224	11.18	17.99	16
7	12.4	16.6	257	10.98	16.34	7
8	12.29	14.37	242	10.75	14.86	14
9	12.26	18.17	223	11.34	19.8	20
10	12.09	16.81	263	11.18	17.99	3
11	12.03	17.69	261	11.18	17.99	4
13	12.01	17.57	221	11.18	17.99	23
14	11.91	15.86	273	10.98	16.34	1
15	11.79	16.08	241	10.98	16.34	14
16	11.78	16.27	202	10.15	16.34	30
17	11.07	19.12	256	11.34	19.8	12
18	11.02	17.4	246	11.18	17.99	10
24	10.45	15.11	251	10.88	16.34	7
32	9.65	13.5	272	9.23	13.51	3
33	9.62	13.55	227	9.59	13.51	17
36	9.48	15.37	249	10.97	16.34	9
37	9.48	14.92	242	9.18	14.86	15
40	9.38	13.77	256	10	14.86	6
44	9.31	16.39	229	10.85	16.34	15
45	9.31	14.78	256	10.07	14.86	6
47	9.21	14.59	234	10.2	16.34	14
49	9.08	13.27	246	9.31	13.51	8
51	8.99	16.6	223	10.33	17.99	18
52	8.96	16.89	245	10.87	17.99	11
54	8.85	14.45	243	9.59	14.86	11

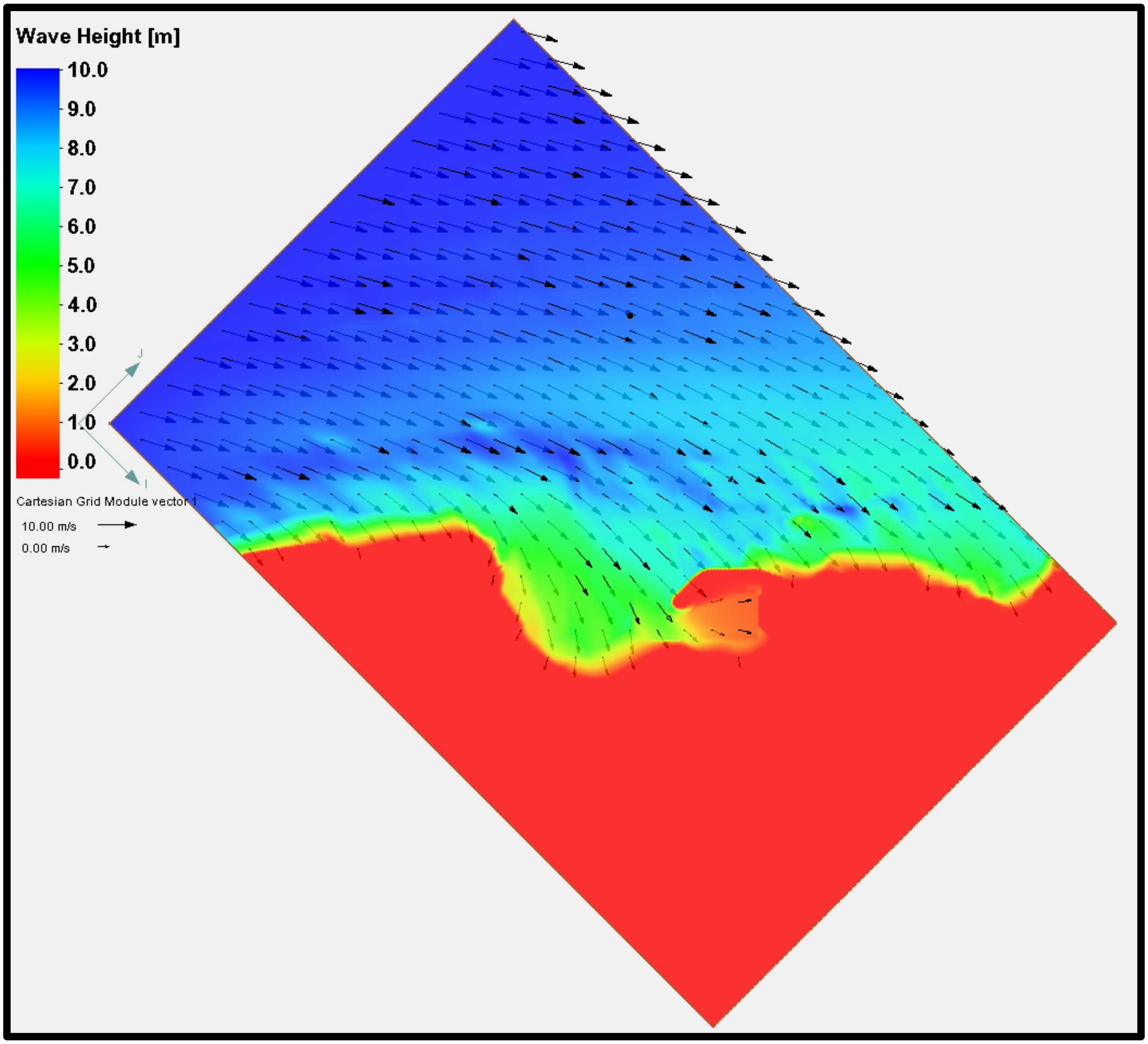


Figure A-29: STWave results for Hm0 = 10 m and Tp = 12 s. Bathymetry representing the proposed harbor location was added to show directional effects around the proposed harbor entrance.

4.5 Harbor Response Modelling

Wave modeling inside the existing harbor and harbor alternatives requires a small model domain with a high resolution grid to adequately calculate wave interaction with the bottom, shorelines and structures. The FUNWAVE model was used for this study to replicate baseline harbor conditions and measure changes from baseline measurements for different harbor configurations. This model uses a Bousinnesq equation to simulate the free water surface and velocity through the water column.

4.5.1 Zapadni Bay FUNWAVE Model

Bathymetry was extracted from the survey data and plotted to a 2 meter x 2 meter grid. Bathymetry was adjusted at the seaward end of the model domain to accommodate a wave maker. Early runs of the model revealed that wave breaking near the wave maker caused model instability and invalidated run results. Since the wave climate is depth limited and the scale of the FUNWAVE model includes only nearshore bathymetry, wave breaking occurred along the seaward edge of most wave scenarios. To compensate for this, the bathymetry along the seaward edge of the domain was deepened to create a wave basin where the waves could develop across a full wavelength before breaking. The net effect of this modification would be an amplification of wave energy outside the harbor, which then breaks when it encounters the natural bathymetry. Model runs were performed at a water level of +1.8 meters MLLW to include the effects of surge and setup.

Wave maker conditions were determined from the results of the STWave analysis of the simulated storm events. For these scenarios, the spectra of the wave maker was modeled as a TMA spectra based on the output wave height and periods found at the monitoring stations. Additional model analysis was performed using 81 auxiliary storms which have a range of peak wave heights and periods to perform sensitivity analysis of the harbor response to differing conditions. These storms had peak wave heights and peak periods as shown in Table A-14.

Table A-14: Auxiliary Storm wave spectral peak wave height and period.

Hm0 (m)	Tp (s)										
	6	8	10	12	14	16	18	20	22	24	26
0.5	X	X	X								
1	X	X	X								
1.5	X	X	X								
2	X	X	X	X	X	X	X	X	X	X	X
2.5	X	X	X								
3	X	X	X								
3.5	X	X	X								
4	X	X	X	X	X	X	X	X	X	X	X
6 (-15°)			X	X	X	X	X	X	X	X	X
6			X	X	X	X	X	X	X	X	X
6 (+15°)			X	X	X	X	X	X	X	X	X
8			X	X	X	X	X	X	X	X	X
10							X	X	X	X	X

While the STWave analysis showed that nearshore processes tend to diffract waves such that the wave direction at the outer breakwater entrance is nearly perpendicular to the shoreline, directional sensitivity was tested using 6 meter waves directed ±15 degrees from a head-on bearing. In total, 100 wave and period scenarios were simulated over the existing bathymetry and run through the existing harbor model to determine design storm cases to be used for alternative analysis.

Model output was measured at selected grid cells to see how the harbor responded to different storm and wave scenarios. Stations of particular interest are shown in Table A-15.

Table A-15: Monitoring Stations of Interest in the FUNWAVE Model.

Station ID	Location
19	Entrance Channel between the Outer breakwaters
17	Entrance Channel at the Inner Breakwater
16	Mooring Basin at the Ice Plant Dock
11	Mooring Basin at the Fuel Docks
18	Mooring Basin at Mooring Pile Structure (Location of ADCP 1)

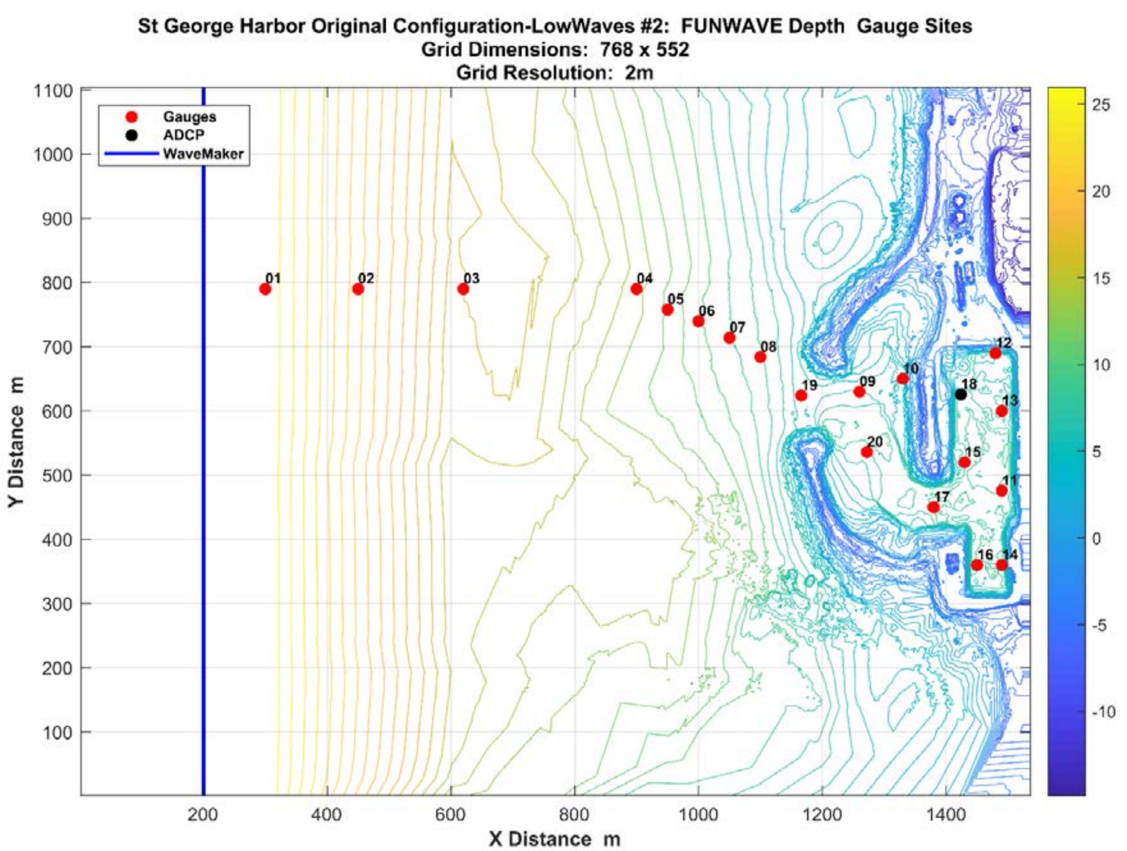


Figure A-30: FUNWAVE model grid and monitoring stations of existing harbor.

4.5.1.1 FUNWAVE Model Calibration

The FUNWAVE model was calibrated to the ADCP data collected in 2013. Storm conditions from the November 2013 event were extracted from the WIS data and run through the STWave grid to determine FUNWAVE boundary conditions. The initial FUNWAVE grid was Initial runs performed with bottom friction set to zero showed good results when the amplitude and frequency of the water surface deviations were compared to the ADCP data for sensors 1 and 2 (Figure A-31). The longer period of the signal in the inner harbor is clearly seen at Station 18 when compared to the signals of stations 17 and 19 just outside the inner basin. This indicates that the FUNWAVE model was able to model the seiche condition observed locally and measured by ADCP 1.

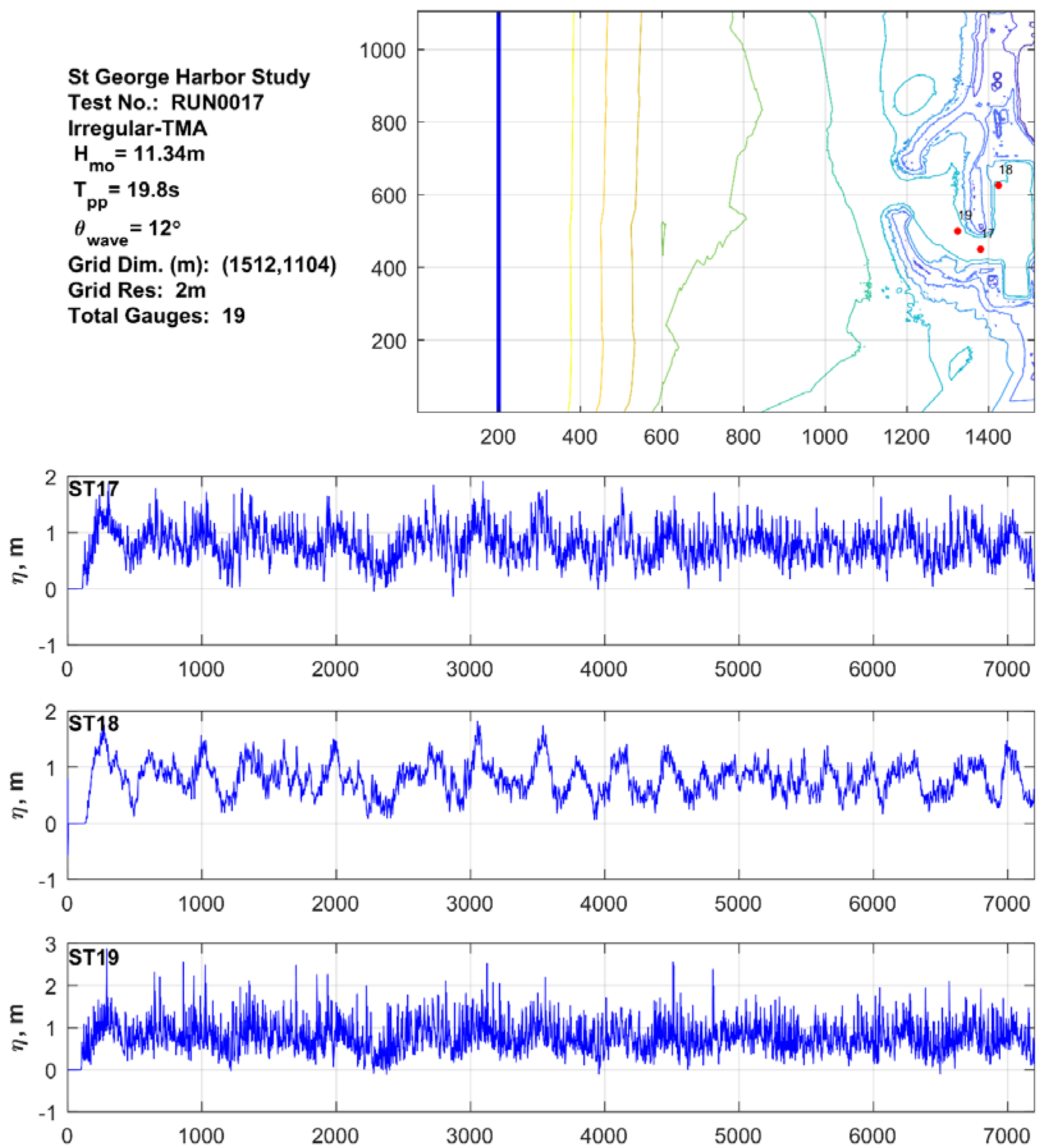


Figure A-31: Free water surface elevation at gages 17, 18 and 19. Gage 18 corresponds to ADCP 1 and Gage 19 corresponds to ADCP 2.

4.5.2 North Site FUNWAVE Model

The North Site is ungagged and there are no data sets available with which to validate model performance. Since the FUNWAVE model of the existing harbor at Zapadni Bay matched measured ADCP data fairly well, it was initially assumed that running the FUNWAVE model at the North Site would produce comparably close approximations to shoreline and harbor responses. Initial testing of North Site alternatives showed considerably less wave attenuation than expected and significant long period harbor response. It was expected that with shallow sloping shorelines, wave energy would dissipate on the shoreline enclosed by proposed breakwaters. The high level of wave response indicated that wave energy was reflecting off these boundaries.

4.5.3 St. Paul Harbor FUNWAVE Model

To test the assumption that wave reflection off inner harbor boundaries was causing the unexpected wave response at the North Site, the team developed a FUNWAVE model of St. Paul Harbor to see if wave reflection would be found at a known site with shallow harbor slopes. St. Paul Harbor includes a spending beach constructed at a 5H:1V slope and has a natural beach along its eastern perimeter. Both of these boundaries are known to absorb and dissipate wave energy inside the harbor from operational experience. When modeled in FUNWAVE, wave amplification was modeled along the constructed spending beach and along the perimeter of the harbor (Figure A-32). To compensate for this effect, a newer version of FUNWAVE with code developed during the course of this study was tested over the St. Paul model grid (Figure A-33). This code allows for adding internal dampening, or sponge boundaries to cells inside the grid. At the time of implementation, no guidance for the effectiveness or application of these sponge boundaries was available. Since St. Paul Harbor has been operating for over 30 years, it was decided to target expected wave conditions for the inner and outer harbor areas (0.3 and 1 meters respectively) as a cursory step towards calibration. A set of sponges was applied to the boundaries of St. Paul Harbor to match the expected wave response conditions (Figure A-34).

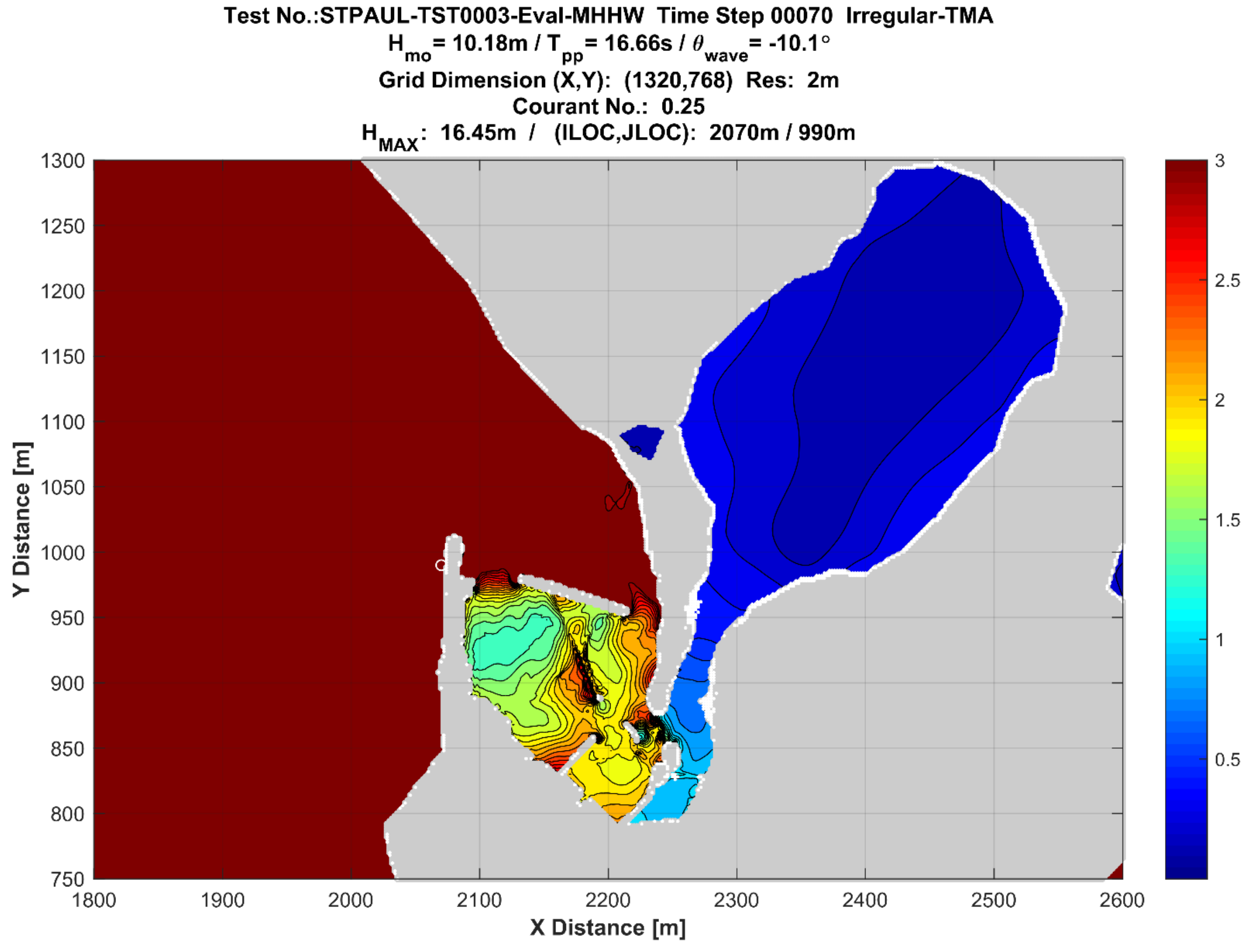


Figure A-32: Significant wave height results of undamped model grid at St. Paul Harbor. The red areas inside the harbor at the south end of the spending beach and the inner breakwater show waves in 2 to 3 meter range indicating reflection modeled off these structures.

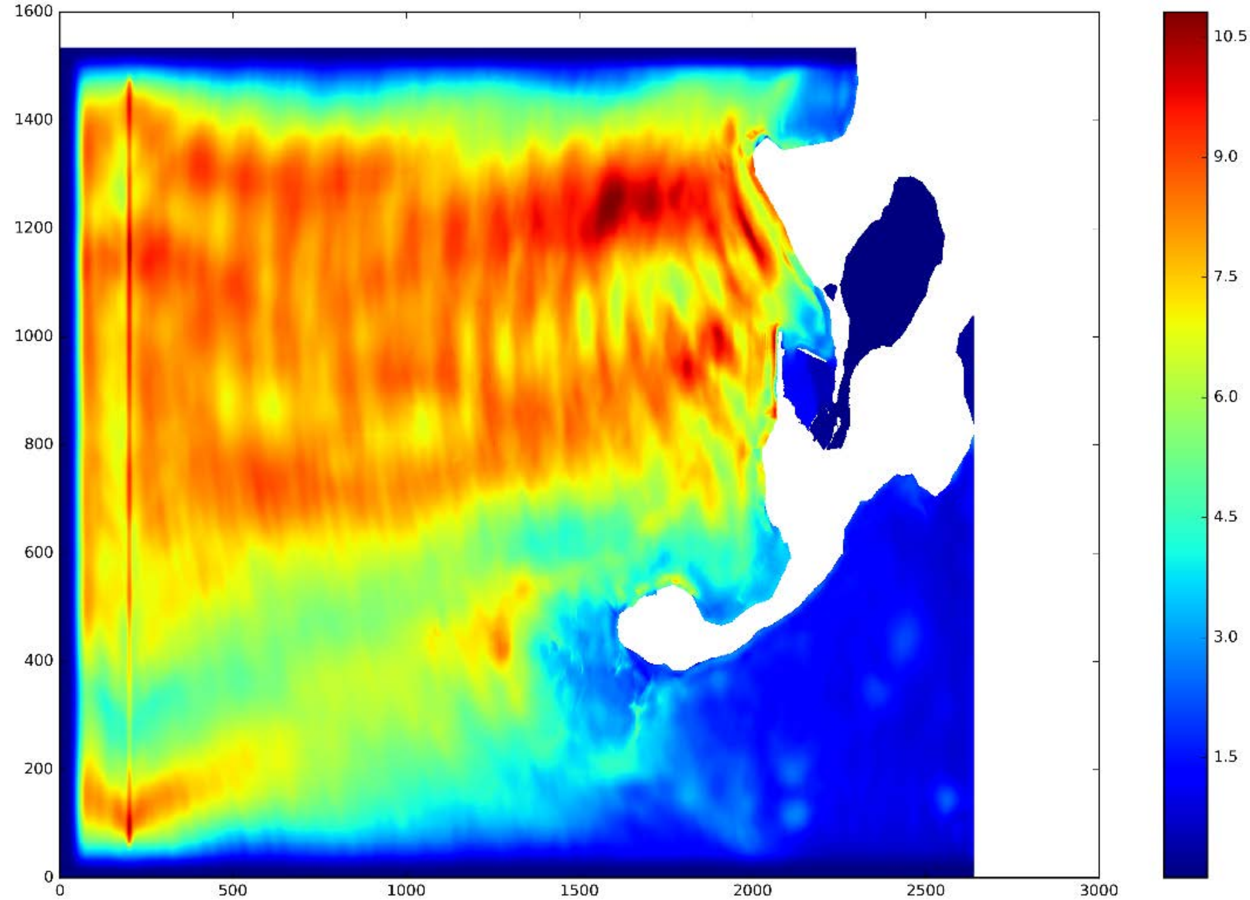


Figure A-33: St. Paul Harbor FUNWAVE model domain. The domain includes the harbor and the Salt Lagoon to the north of St. Paul.

Use of sponge boundaries improved expected model output significantly. Using 15 meter wide sponges on beaches, 10 meter wide sponges on porous rock slopes and 5 meter wide sponges on harbor slopes, the model was able to produce wave responses of about 1 meter in the outer harbor and 0.3 meters in the inner harbor (Figure A-35).

The St. Paul test indicates that the inclusion of internal sponge boundaries can approximate wave energy dissipation on spending beaches and in porous structures. Initial applications of 15 meter, 10 meter and 5 meter sponges to harbors at St. George did not yield reasonable results and the application of internal sponge boundaries for this study is under investigation at the time of the writing of this appendix.

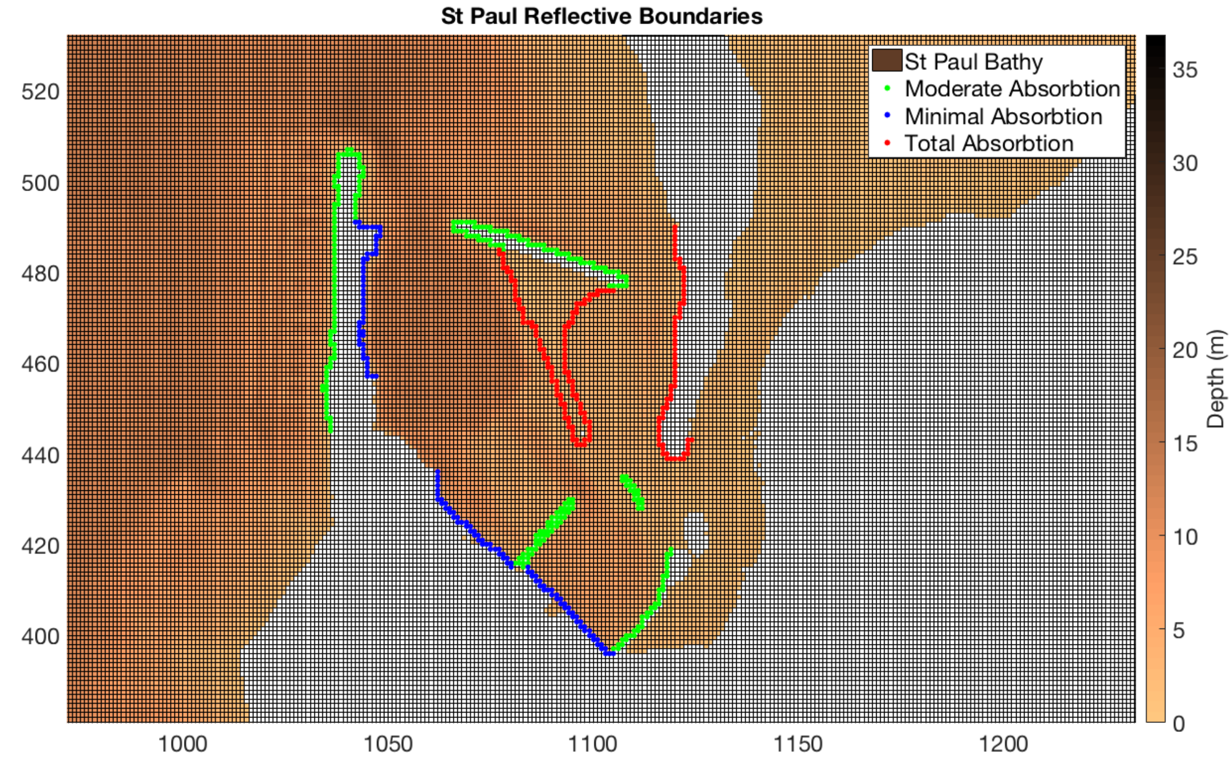


Figure A-34: Internal absorption or sponge boundaries for the St. Paul Harbor grid.

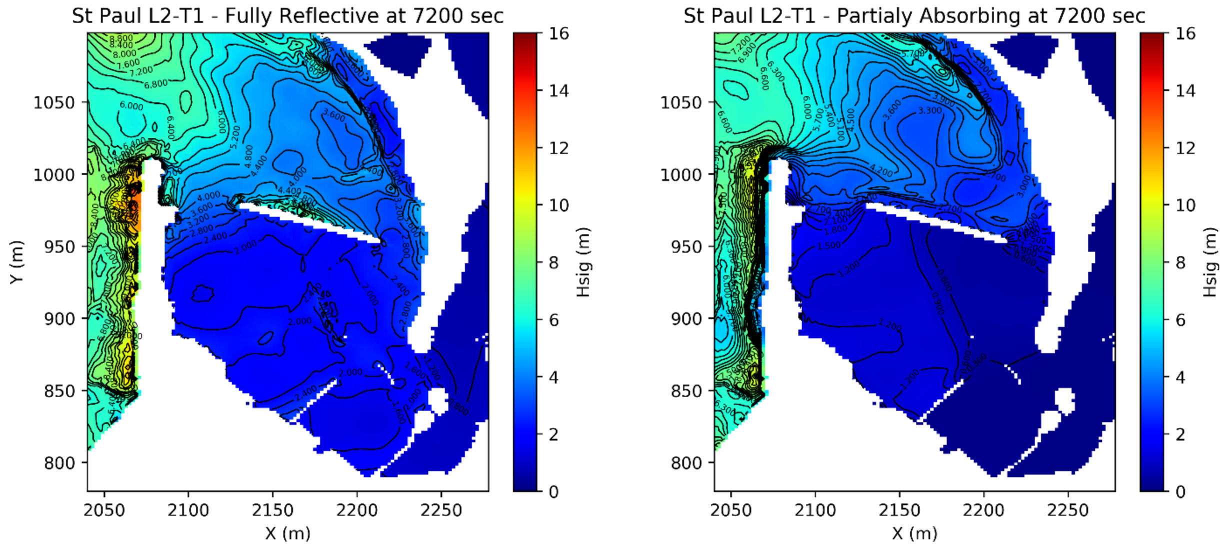


Figure A-35: St. Paul FUNWAVE results without sponge boundaries (left) and with sponge boundaries (right). Waves at the spending beach without sponges are about 2.4 meters whereas with the sponges, wave heights are about 0.9 meters.

6. ZAPADNI BAY ALTERNATIVES

All Zapadni Bay alternatives developed and tested were found to be ineffective at providing the wave environment required for vessels to transfer cargo during storm conditions. Rough order of magnitude costs were developed and it was found that the cost to provide a safe mooring environment was very high. The descriptions of alternatives presented here is for information only to demonstrate the level of effort expended to attempt to find a solution at the existing harbor. None of these alternatives are recommended for further development. Most of these alternatives share the same concept for breakwater design where breakwater modifications were considered. Breakwaters exposed to the open ocean environment were designed as a 3 layer rubble mound breakwater with 30 ton armor stone and a crest elevation of +35 feet MLLW (Figure A-36).

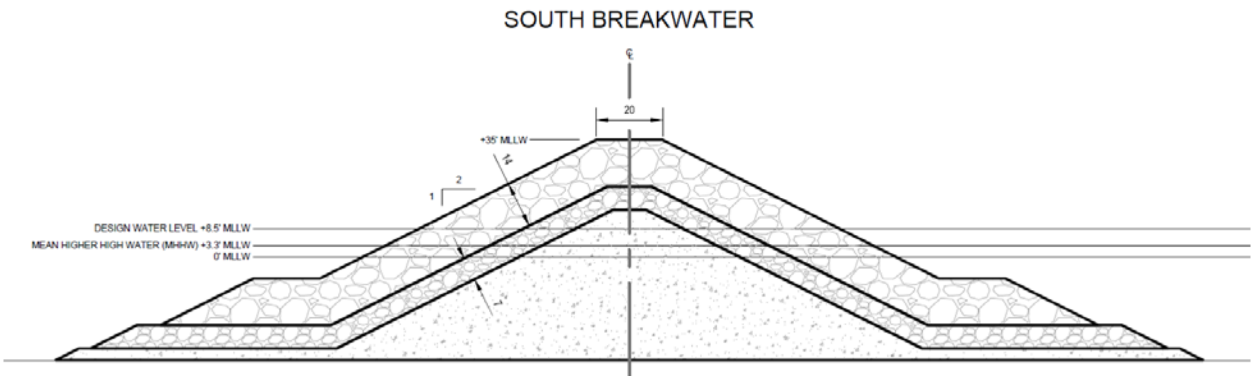


Figure A-36: Typical Breakwater Cross Section for Zapadni Bay Alternatives.

Some variations of this design are indicated in some alternatives.

Quantities and costs for these alternatives are included for comparison of relative effort of construction for each alternative. The costs presented in this appendix are rough order of magnitude project costs using the quantities estimated from CAD three dimensional surface models of the alternatives and assumptions for design and construction administration costs. These cost estimates do not include risk based contingency estimates and differ from the numbers found in the main report.

5.1 Alternative Z-1: South Breakwater Extension

This alternative includes constructing an 800 foot long extension to the existing south breakwater with a crest elevation of +35 feet MLLW, a 500 foot jetty off the existing north breakwater with a crest elevation of +10 feet MLLW, three 1,000 foot long submerged reefs with crest elevations of -12 feet MLLW, a new inner breakwater with a crest elevation of +20 feet MLLW with a spending beach sloped at 10H:1V and a new navigation channel with a depth of -24 feet MLLW and a new turning basin with a depth of -20 feet MLLW. This alternative re-routes vessel traffic to the north end of the harbor in an attempt to reduce the occurrence of storm waves entering the harbor from the southwest direction (Figure A-37). This navigation pattern is expected to improve barge access to the harbor.

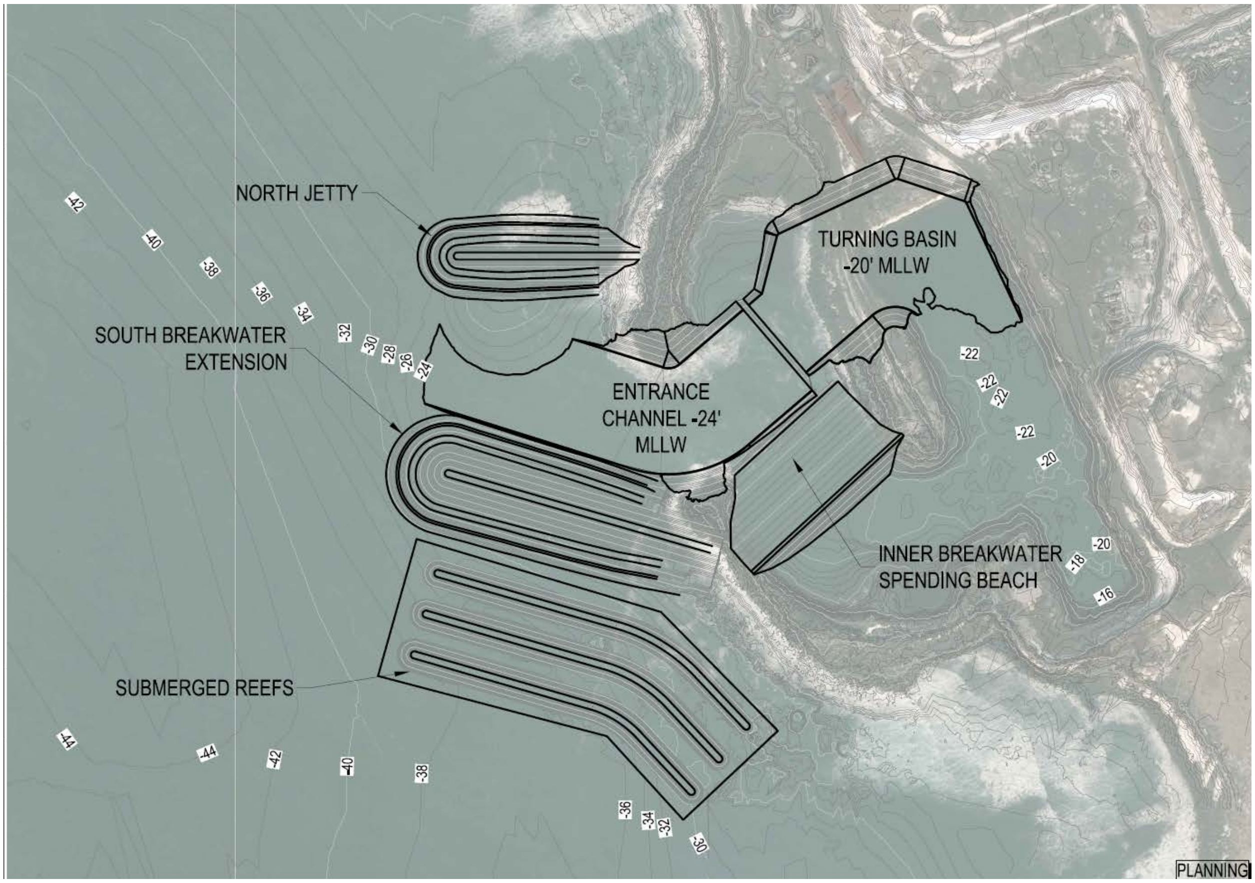


Figure A-37: Alternative Z-1 Concept Plan

5.1.1 Structural Design

The South Breakwater extension and North Jetty are subject to storm waves from the southwest and use a design wave height of 23 feet. This results in an average armor stone weight of 30 tons when constructed at a 2H:1V slope. Due to the long period of the storm waves, energy is assumed to diffract around the breakwater heads and also transmit through the breakwater section requiring both sides of the breakwater and jetty to be armored. The reefs were designed by referencing the existing reefs in place at nearby St. Paul Harbor with a stone size of 1.5 tons.

5.1.2 FUNWAVE Analysis

A FUNWAVE grid off this harbor was created to determine the effectiveness of the navigation features at providing a wave climate usable by vessels for transferring cargo (Figure A-38). The storms used to analyze the existing harbor shown in Table A-12 and Table A-14 were run over this grid to determine harbor response. Selected gages were analyzed to measure harbor response. The critical gages on this grid are gages 11, which is at the existing fuel dock and 16 which is at the existing ice plant dock. The model run results did not produce wave heights at the docks less than 0.5 meters in any scenario and the sea climate outside the harbor required for vessels to safely moor was not found.

5.1.3 Harbor Effectiveness

This harbor configuration did not improve moorage conditions at the existing docks at Zapadni Bay. It is believed that the wider entrance channel and open westerly exposure allows too much wave energy to pass directly into the inner harbor area. Additionally, the rerouting of the navigation channel eliminated area for waves to dissipate after passing through the outer breakwaters at the north end of the existing inner breakwater. Instead of dissipating, energy is channelized into the inner harbor resulting in degraded mooring conditions.

5.1.4 Alternative Quantities and Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-16. The estimated project cost for this alternative is $167 million without contingency cost.

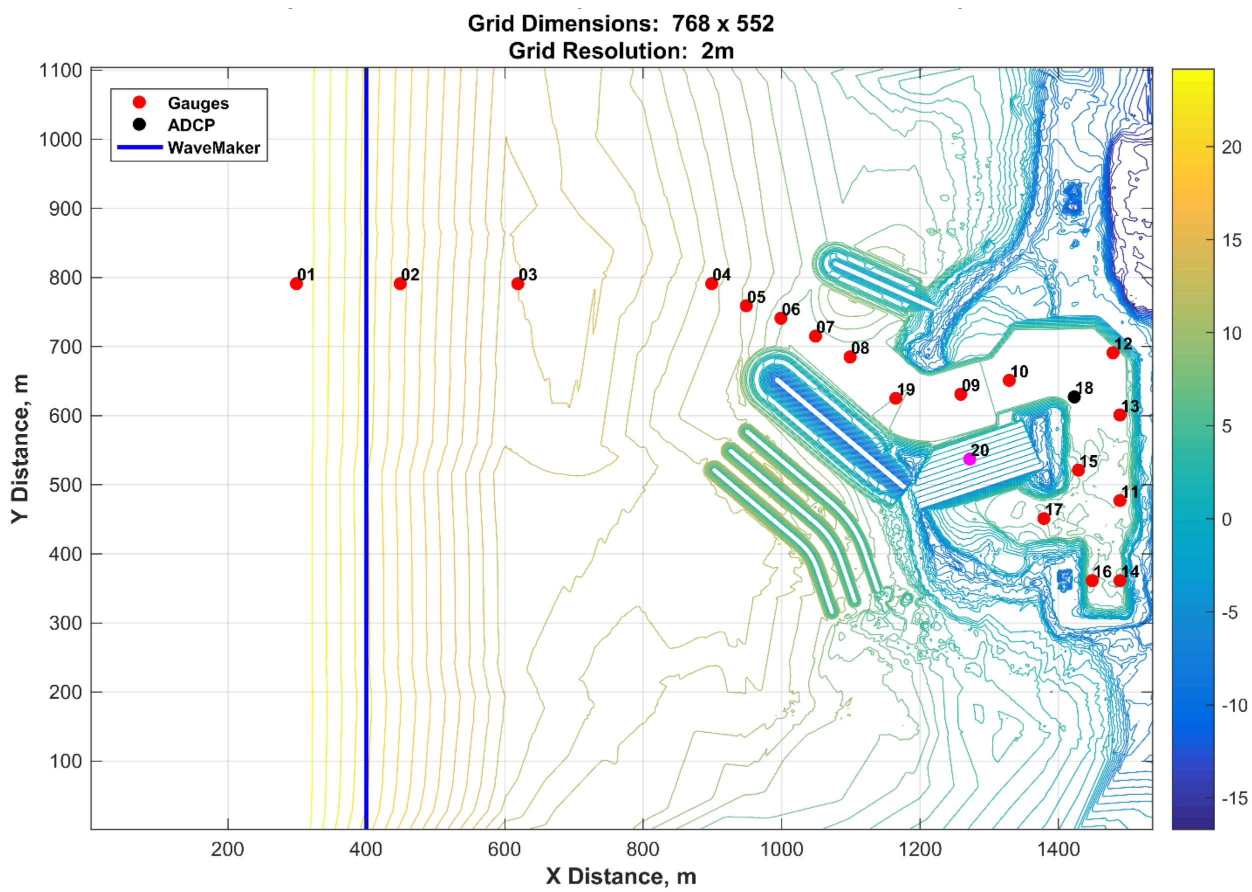


Figure A-38: FUNWAVE model grid for Alternative Z-1. Note, wavemaker location shown is incorrect, the wavemaker is located at X = 200m. Gages 11 and 16 were used to measure wave height at the dock faces.

Table A-16: Alternative Z-1 Quantities

South Breakwater Extension		**North Jetty**	
A-Rock	110,000 CY	A-Rock	14,000 CY
B-Rock	35,000 CY	B-Rock	12,000 CY
C-Rock	57,000 CY	C-Rock	7,700 CY
Inner Breakwater - Spending Beach		**Reefs**	
A-Rock	13,000 CY	Reef 1	37,000 CY
B-Rock	7,800 CY	Reef 2	43,000 CY
C-Rock	14,000 CY	Reef 3	43,000 CY
Rock Spalls	68,000 CY	Bedding Layer	36,000 CY
Dredging			
Drill, Blast and Dredge	230,000 CY		

5.2 Alternative Z-2: South Breakwater Overlap

This alternative includes constructing a 1,050-foot long cap and extension to the existing south breakwater with a crest elevation of +35 feet MLLW, a 400-foot jetty north of the new breakwater breakwater with a crest elevation of +10 feet MLLW and a new navigation channel with a depth of -22 feet MLLW and a new turning basin with a depth of -20 feet MLLW (Figure A-39). The existing north breakwater would be demolished to allow vessels to pass through this area. The construction provides a breakwater overlap of the inner harbor facilities in an attempt to provide improved protection for the existing docks. The new channel alignment includes wider turning sections than the existing harbor. This navigation pattern is expected to improve barge access to the harbor.

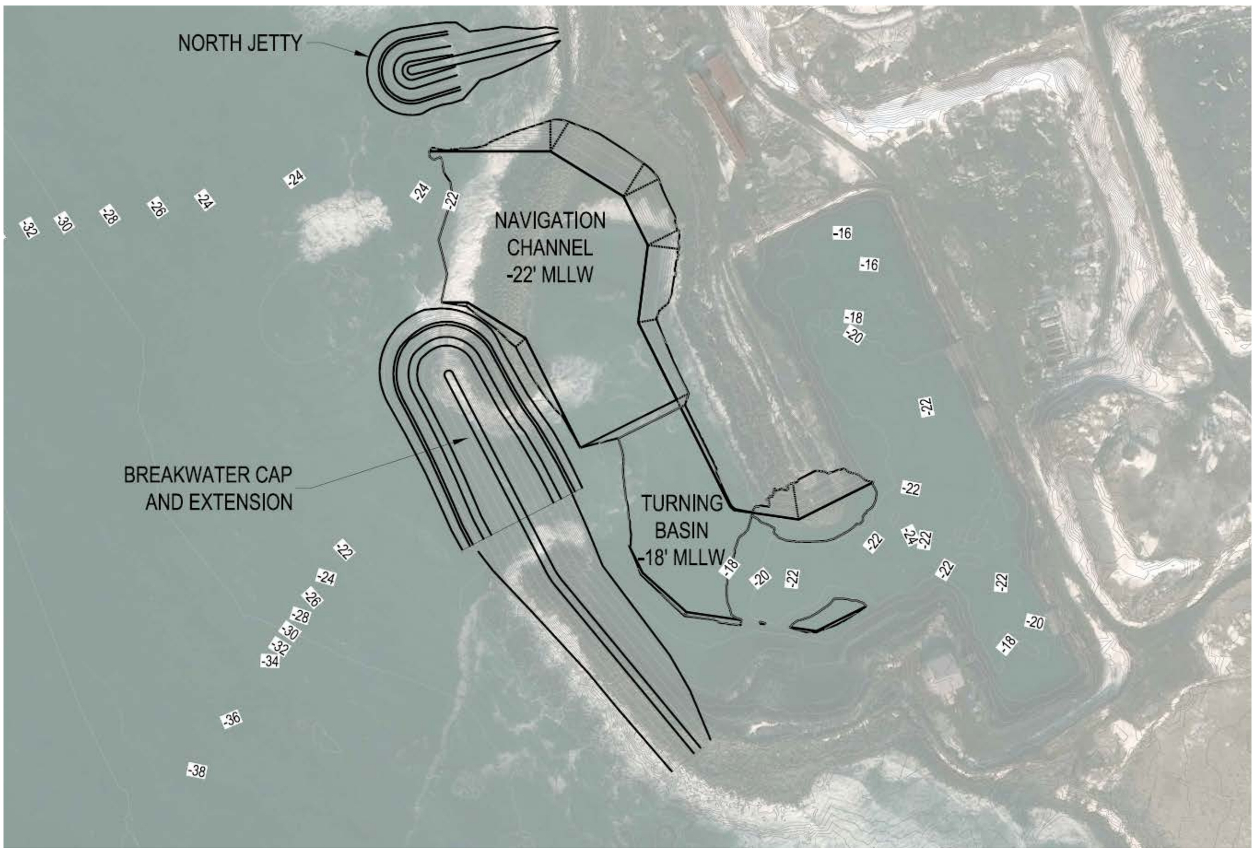


Figure A-39: Alternative Z-2 Concept Plan

5.2.1 Structural Design

The South Breakwater extension and North Jetty are subject to storm waves from the southwest and use a design wave height of 23 feet. This results in an average armor stone weight of 30 tons when constructed at a 2H:1V slope. Due to the long period of the storm waves, energy is assumed to diffract around the breakwater heads and also transmit through the breakwater section requiring both sides of the breakwater and jetty to be armored.

5.2.2 FUNWAVE Analysis

A FUNWAVE grid off this harbor was created to determine the effectiveness of the navigation features at providing a wave climate usable by vessels for transferring cargo (Figure A-40). The storms used to analyze the existing harbor shown in Table A-12 and Table A-14 were run over this grid to determine harbor response. Selected gages were analyzed to measure harbor response. The critical gages on this grid are gages 11, which is at the existing fuel dock and 16 which is at the existing ice plant dock. The model run results indicate that waves outside the harbor at the wavemaker location need to be less than 2.39 meters in height to produce wave heights at the docks less than 0.5 meters. It is estimated that sea conditions exceed this height approximately 19% of the time annually which is within 1% of the existing condition.

5.2.3 Harbor Effectiveness

This harbor configuration did not improve moorage conditions at the existing docks at Zapadni Bay. It is believed that the alignment of the entrance channel and the presence of the jetty to the north channelize incident wave energy causing it to propagate efficiently through the channel to the inner harbor. As with Alternative Z-1, the dissipation area north of the inner breakwater was lost resulting in degraded mooring conditions.

5.2.4 Alternative Quantities and Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-17. The estimated project cost for this alternative is $102 million without contingency cost.

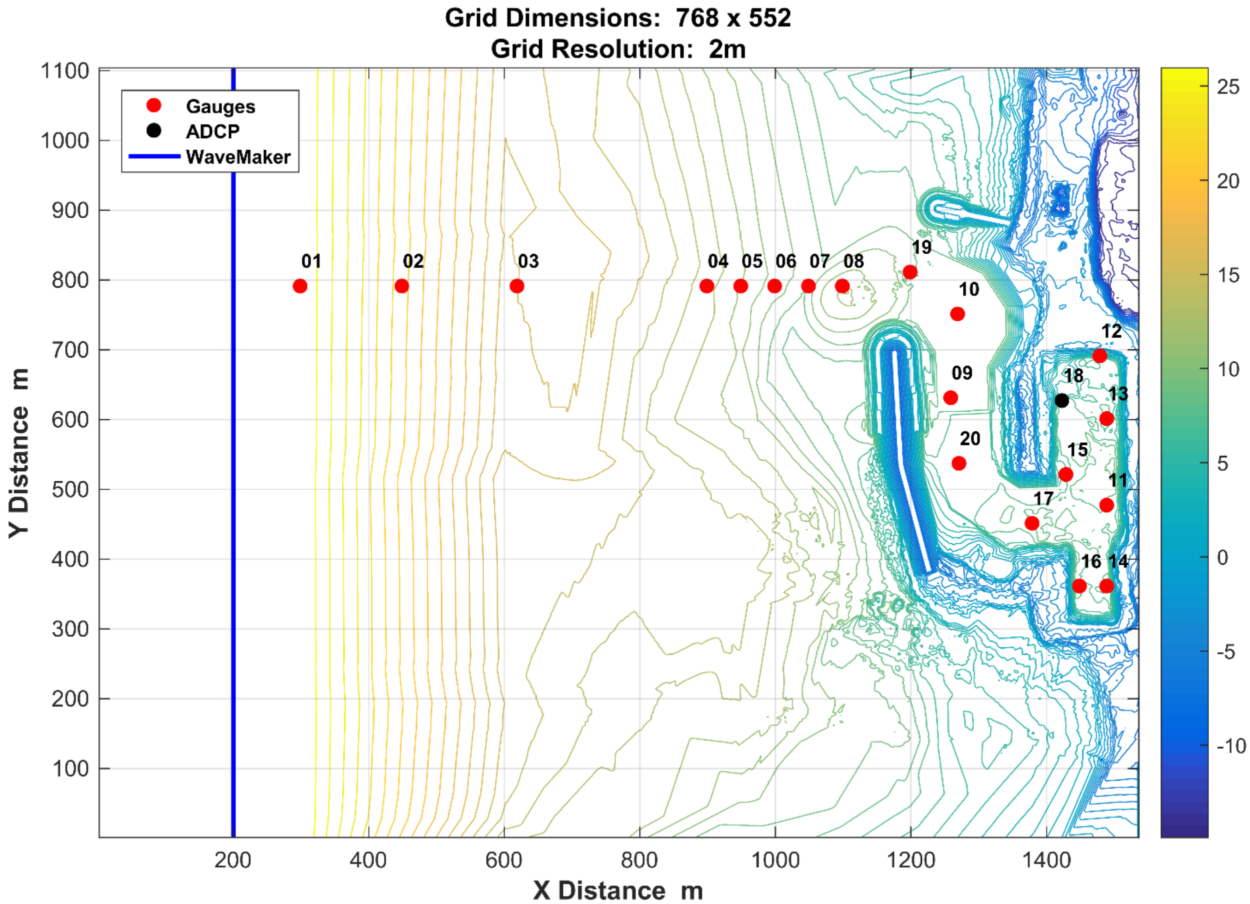


Figure A-40: FUNWAVE model grid for Alternative Z-2. Gages 11 and 16 were used to measure wave height at the dock faces.

Table A-17: Alternative Z-2 Quantities

South Breakwater Extension		**North Jetty**	
A-Rock	110,000 CY	A-Rock	14,000 CY
B-Rock	35,000 CY	B-Rock	12,000 CY
C-Rock	57,000 CY	C-Rock	7,700 CY
Breakwater Nose Demolition			
Rock Removal	140,000 CY		
Dredging			
Drill, Blast and Dredge	150,000 CY		

5.3 Alternative Z-3: Inland Basin

This alternative includes constructing a new 700 foot long by 500 foot wide mooring basin to the northeast of the existing harbor. The new basin would be connected to the existing harbor by a 200 foot wide navigation channel. Excavation of the new mooring basin included excavation to construct a road around its perimeter to allow vehicles to traverse the perimeter of the harbor. The north end of the existing inner basin and the new inner basin would be sloped at 5H:1V to reduce wave reflection within the mooring basins. The existing harbor breakwaters would remain in their existing condition and the existing channel would be widened to a minimum of 200 feet at the head of the inner breakwater and dredged to a depth of -22 feet MLLW (Figure A-41). The navigation channel widens the pinch point around the inner breakwater and is expected to improve barge navigation to the harbor.

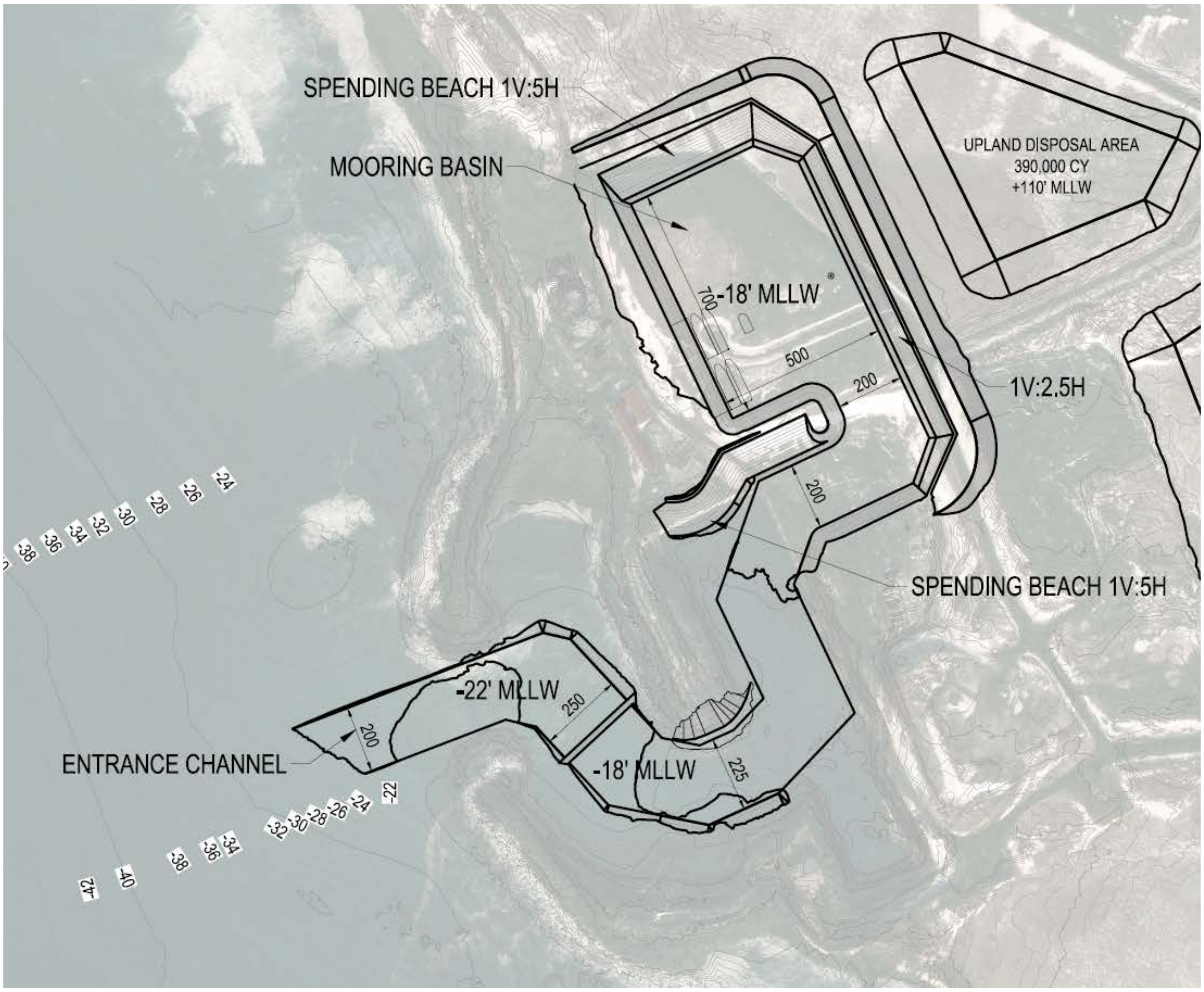


Figure A-41: Alternative Z-3 Concept Plan

5.3.1 Structural Design

Primary construction of this harbor design would be through excavation and dredging. No new rock structures would be placed. Slope protection rock would be provided where the native rock was determined to be too small to provide slope protection under the expected wave conditions inside the harbor under storm conditions.

5.3.2 FUNWAVE Analysis

A FUNWAVE grid off this harbor was created to determine the effectiveness of the navigation features at providing a wave climate usable by vessels for transferring cargo (Figure A-42). To reduce the processing time required to generate wave heights at the docks, a smaller set of three simulated storms and nine auxiliary storms were run through the FUNWAVE grid. Storm definitions are shown in Table A-18 and Table A-19.

Table A-18: Simulated Storm wave spectral peak wave height and period.

Run No.	Hm0 (m)	Tp (s)	Dir (deg)
1	14.05	17.35	252
2	13.69	17.75	245
17	11.07	19.12	256

Table A-19: Auxiliary Storm wave spectral peak wave height and period.

Hm0 (m)	Tp (s)		
	10	20	26
2	X	X	X
6	X	X	X
8	X		
10		X	X

Selected gages were analyzed to measure harbor response. The critical gages on this grid are gages 11, which is at the existing fuel dock and 16 which is at the existing ice plant dock and gage 26 which is the location of a proposed new dock in the new mooring basin. The model run results indicate that waves outside the harbor at the wavemaker location need to be less than 4.14 meters in height to produce wave heights at the docks less than 0.5 meters. It is estimated that sea conditions exceed this height approximately 8% of the time annually.

5.3.3 Harbor Effectiveness

This harbor configuration did not improve moorage conditions at the existing docks at Zapadni Bay. The new dock location in the new basin showed improved wave conditions, however still showed a significant percentage of time where the dock would be unusable. It was also found that there is a secondary seiche in the new basin. It is believed that the seiche conditions in the existing inner harbor create forcing conditions through the new navigation which sets up a secondary seiche in the new mooring basin during storm events. This harbor also requires a significant excavation volume on the order of 2,000,000 CYs of material requiring disposal outside of the harbor area.

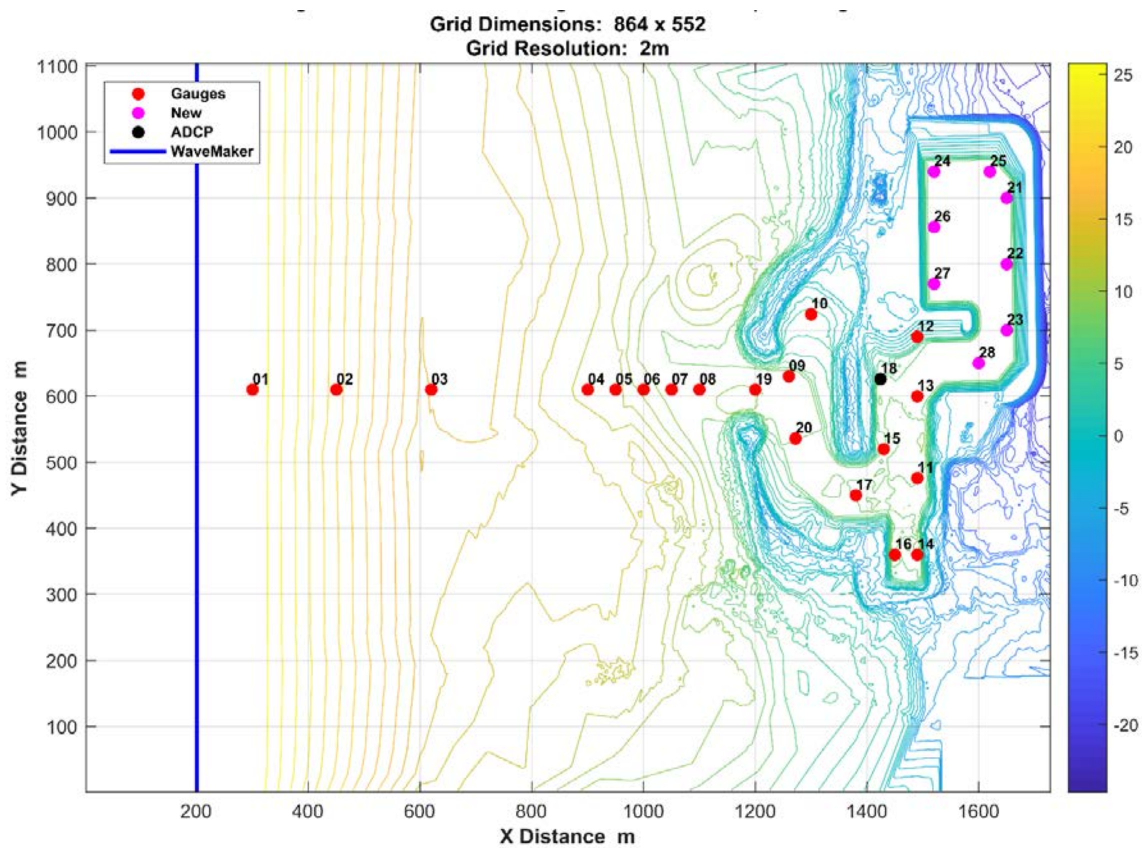


Figure A-42: FUNWAVE model grid for Alternative Z-3. Gages 11 and 16 were used to measure wave height at the existing dock faces. Gage 26 was used to measure wave height at a proposed new dock location.

5.3.4 Alternative Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-20. The estimated project cost for this alternative is $74 million without contingency cost.

Table A-20: Alternative Z-3 Quantities

Dredging		**Breakwater Shortening**	
Mooring Basin	2,000,000 CY	Inner Breakwater	11,000 CY
Entrance Channel	23,000 CY		

5.4 Alternative Z-4: Overall Harbor Concept (OHC)

This alternative was developed by the State of Alaska Department of Transportation and Public Facilities (AKDOT&PF) and HDR Inc. prior to initiation of the USACE feasibility study effort. The AKDOT&PF plan was modified to meet navigation requirements for the fuel barge to enter the harbor, however the parallel jetties would still pose an impediment for the barge to clear the outer breakwaters. This alternative includes constructing a 400-foot-long jetty at the ends of the north and south breakwaters with a crest elevation of +35 feet MLLW, a 500-foot inner north breakwater with a crest elevation of +20 feet MLLW and a north mooring basin with a depth of -10 feet MLLW (Figure A-43). The jetties restrict the available approach headings for barges to enter the harbor and further restrict barge access to make deliveries.

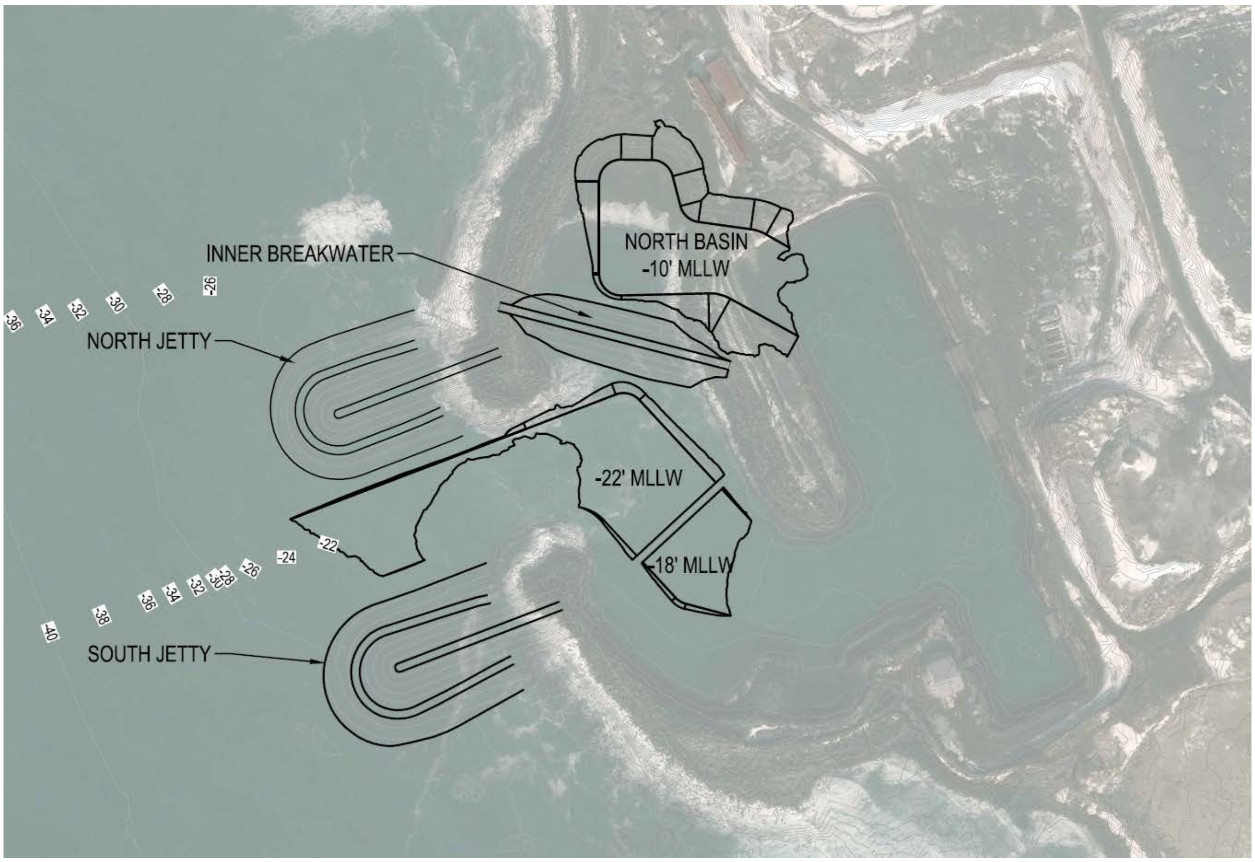


Figure A-43: Alternative Z-4 Concept Plan

5.4.1 Navigation Design

The north and south jetties are subject to storm waves from the southwest and use a design wave height of 23 feet. This results in an average armor stone weight of 30 tons when constructed at a 2H:1V slope. The new inner breakwater is also armored with 30 ton stone due to its proximity to the harbor entrance.

5.4.2 FUNWAVE Analysis

A FUNWAVE grid off this harbor was created to determine the effectiveness of the navigation features at providing a wave climate usable by vessels for transferring cargo (Figure A-44). The storms shown in Table A-18 and Table A-19 were run over this grid to determine harbor response. Selected gages were analyzed to measure harbor response. The critical gages on this grid are gages 11, which is at the existing fuel dock and 16 which is at the existing ice plant dock. The model run results indicate that waves outside the harbor at the wavemaker location need to be less than 2.44 meters in height to produce wave heights at the docks less than 0.5 meters. It is estimated that sea conditions exceed this height approximately 19% of the time annually which is within 1% of the existing condition.

5.4.3 Harbor Effectiveness

This harbor configuration did not improve moorage conditions at the existing docks at Zapadni Bay. Conditions at the existing docks were found to be essentially the same as the existing condition. STWave runs showed that incoming storm waves generally diffract to a shore-normal direction which propagates straight through the existing harbor entrance. The jetties extend this entrance into deeper water, but do little to reduce wave energy from this direction. Allowing the seiche conditions seen in the existing harbor to develop.

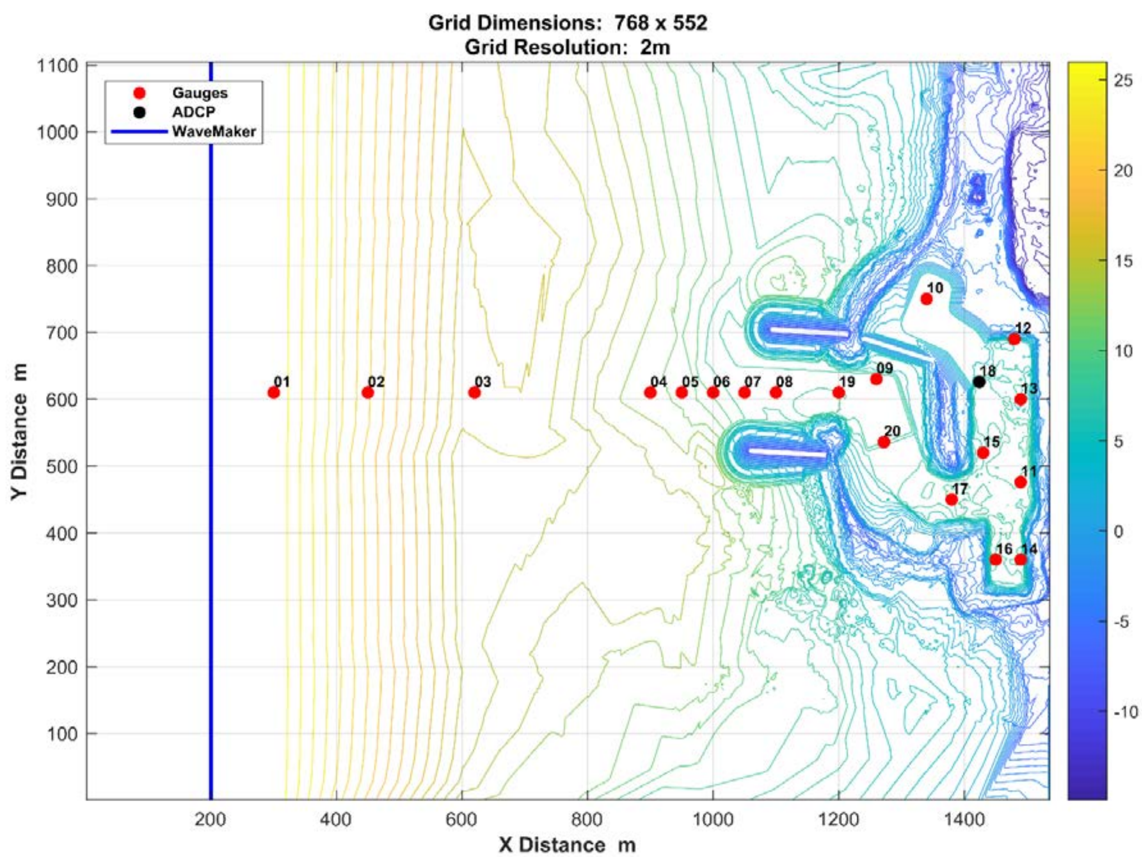


Figure A-44: FUNWAVE model grid for Alternative Z-4. Gages 11 and 16 were used to measure wave height at the existing dock faces.

5.4.4 Alternative Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-21. The estimated project cost for this alternative is $85million without contingency cost.

Table A-21: Alternative Z-4 Quantities

South Jetty		**North Jetty**	
A-Rock	48,000 CY	A-Rock	42,000 CY
B-Rock	31,000 CY	B-Rock	28,000 CY
C-Rock	52,000 CY	C-Rock	44,000 CY
Inner Breakwater			
A-Rock	12,000 CY		
B-Rock	8,000 CY		
C-Rock	13,000 CY		
Dredging		**Upland Fill**	
Drill, Blast and Dredge	96,000 CY	Causeway	31,000 CY

The estimated project cost for this alternative is $85 million without contingency cost.

5.5 Alternative Z-5: Expanded Harbor

This alternative includes demolishing the existing south breakwater and constructing an 3,000 foot long breakwater from the ice plant to an overlap position seaward of the existing north breakwater with a crest elevation of +35 feet MLLW. A 300 foot long extension of the north breakwater would be constructed with a crest elevation of +20 feet MLLW perpendicular to the new breakwater to define the mooring basin behind the new breakwater. New docks would be constructed on the inside of the new main breakwater with the entire basin enclosed by the new breakwaters being dredged to -232 feet MLLW. The back slope of the existing inner harbor would be filled at a 10H:1V slope to provide a spending beach in the new mooring basin (Figure A-45). The navigation pattern for this alternative is very similar to St. Paul Harbor and the wider channel around the breakwater is expected to improve barge navigation to the harbor.

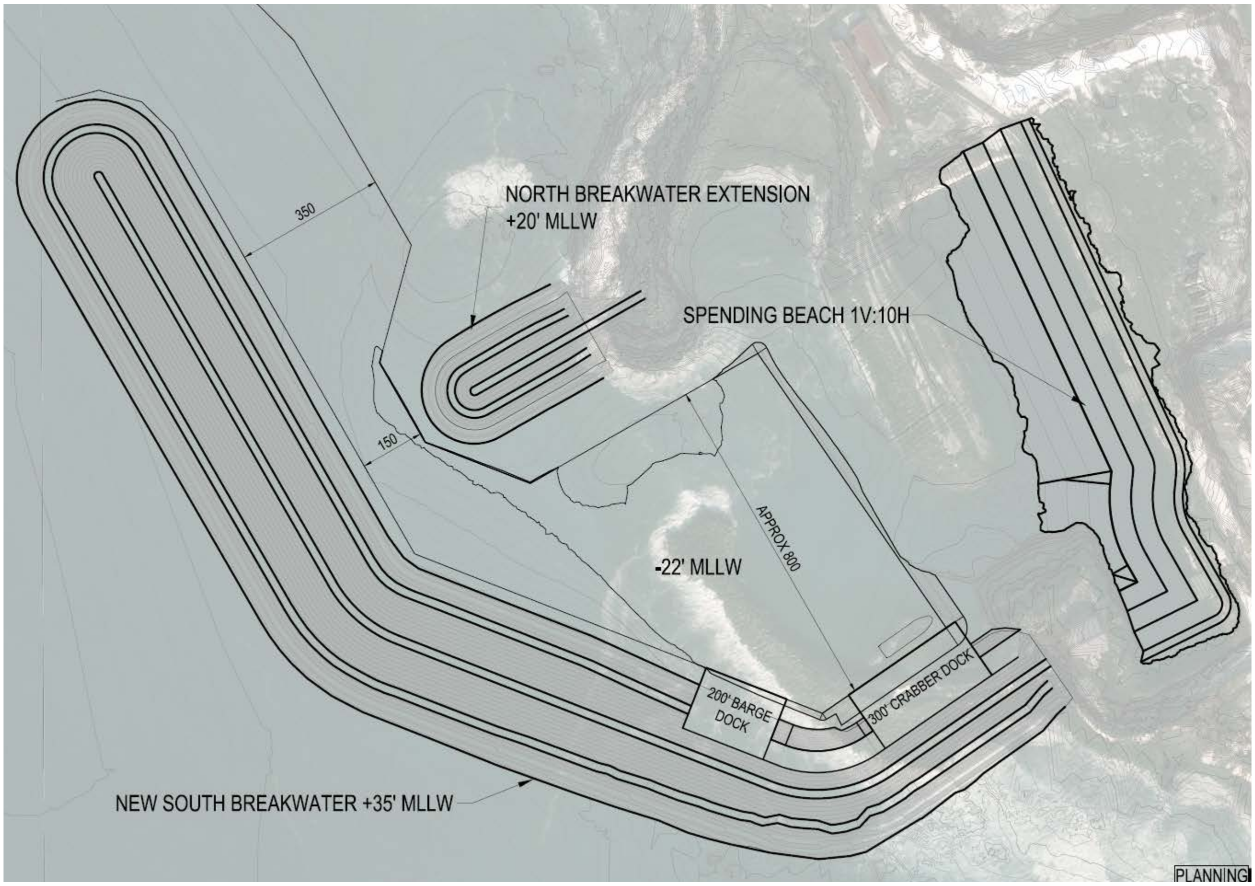


Figure A-45: Alternative Z-5 Concept Plan

5.5.1 Navigation Design

The new breakwaters are subject to storm waves from the southwest and use a design wave height of 23 feet. This results in an average armor stone weight of 30 tons when constructed at a 2H:1V slope. The new inner breakwater is also armored with 30 ton stone due to its proximity to the harbor entrance.

5.5.2 FUNWAVE Analysis

A FUNWAVE grid off this harbor was created to determine the effectiveness of the navigation features at providing a wave climate usable by vessels for transferring cargo (Figure A-44). The storms shown in Table A-18 and Table A-19 were run over this grid to determine harbor response. Selected gages were analyzed to measure harbor response. The critical gages on this grid are gages 11 and 14 which are at the location of proposed new docks. The model run results indicate that waves outside the harbor at the wavemaker location need to be less than 6.9 meters in height to produce wave heights at the docks less than 0.5 meters. It is estimated that sea conditions exceed this height approximately 2 percent of the time.

5.5.3 Harbor Effectiveness

This harbor substantially improves moorage availability but would still require vessels to leave the dock during storm events to avoid damage. While this design is essentially functional, the quantities of rock required to construct the breakwater would take a substantial amount of time to produce and place and phased construction over several years would be required.

5.5.4 Alternative Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-22. The estimated project cost for this alternative is $437 million without contingency cost.

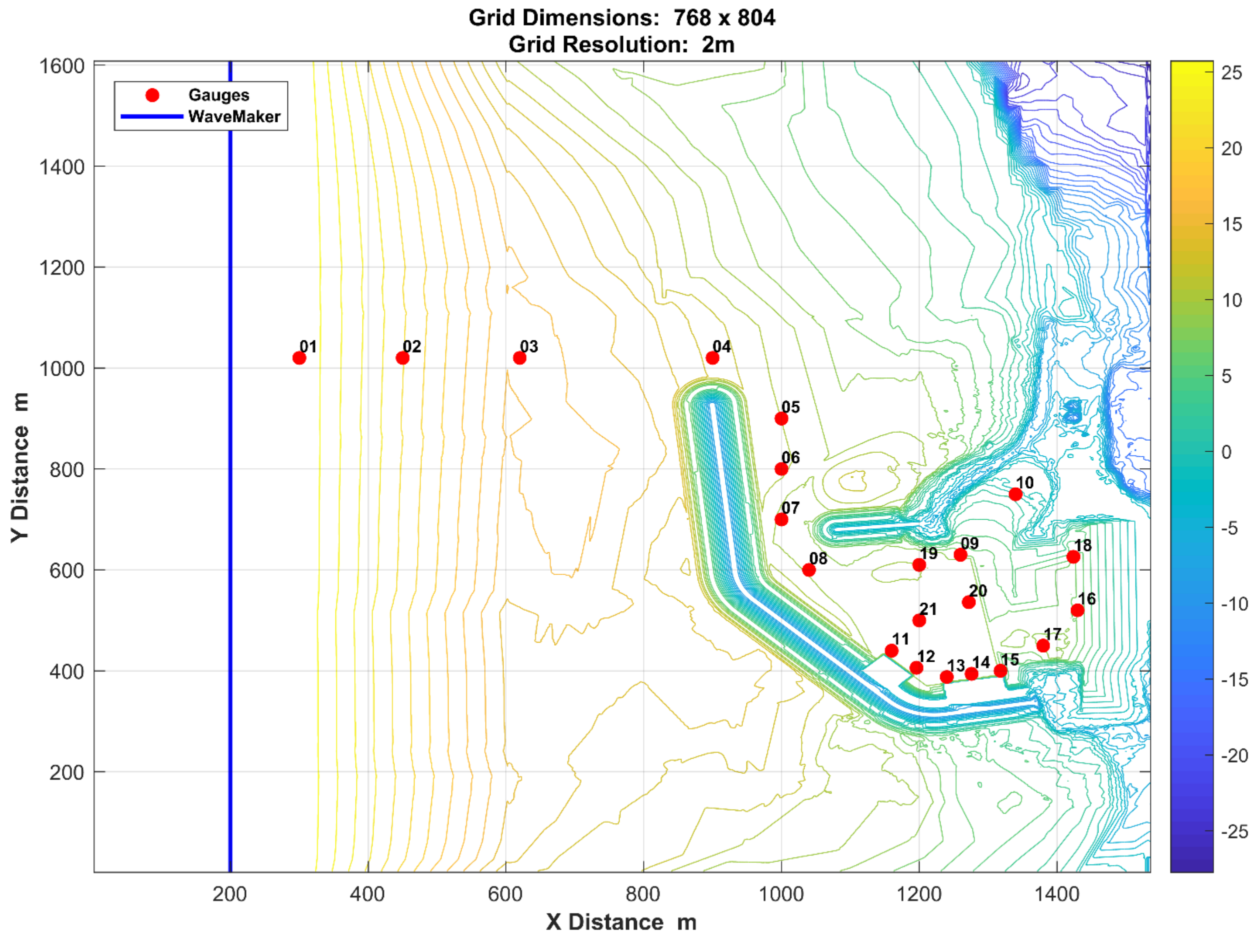


Figure A-46: FUNWAVE model grid for Alternative Z-5. Gages 11 and 14 were used to measure wave height at the new dock faces.

Table A-22: Alternative Z-5 Quantities

New South Breakwater		North Breakwater Spur	
A-Rock	420,000 CY	A-Rock	30,000 CY
B-Rock	250,000 CY	B-Rock	22,000 CY
C-Rock	540,000 CY	C-Rock	23,000 CY
Breakwater Demolition			
South Breakwater	220,000 CY		
Inner Breakwater	130,000 CY		
Dredging		**Upland Fill**	
Drill, Blast and Dredge	241,000 CY	Causeway	31,000 CY

5.6 Alternative Z-6: Half-Moon Harbor

This alternative includes constructing a new 900 foot radius semi-circular mooring basin into the eastern edge of the existing inner harbor. The side slope of the new basin would be 10H:1V to reduce reflection in the mooring area. Excavation of the new mooring basin included excavation to construct a road around its perimeter to allow vehicles to traverse the perimeter of the harbor. The existing harbor breakwaters would remain in their existing condition and the existing channel would be widened to a minimum of 200 feet at the head of the inner breakwater and dredged to a depth of -22 feet MLLW (Figure A-47). The navigation channel widens the pinch point around the inner breakwater and is expected to improve barge navigation to the harbor.

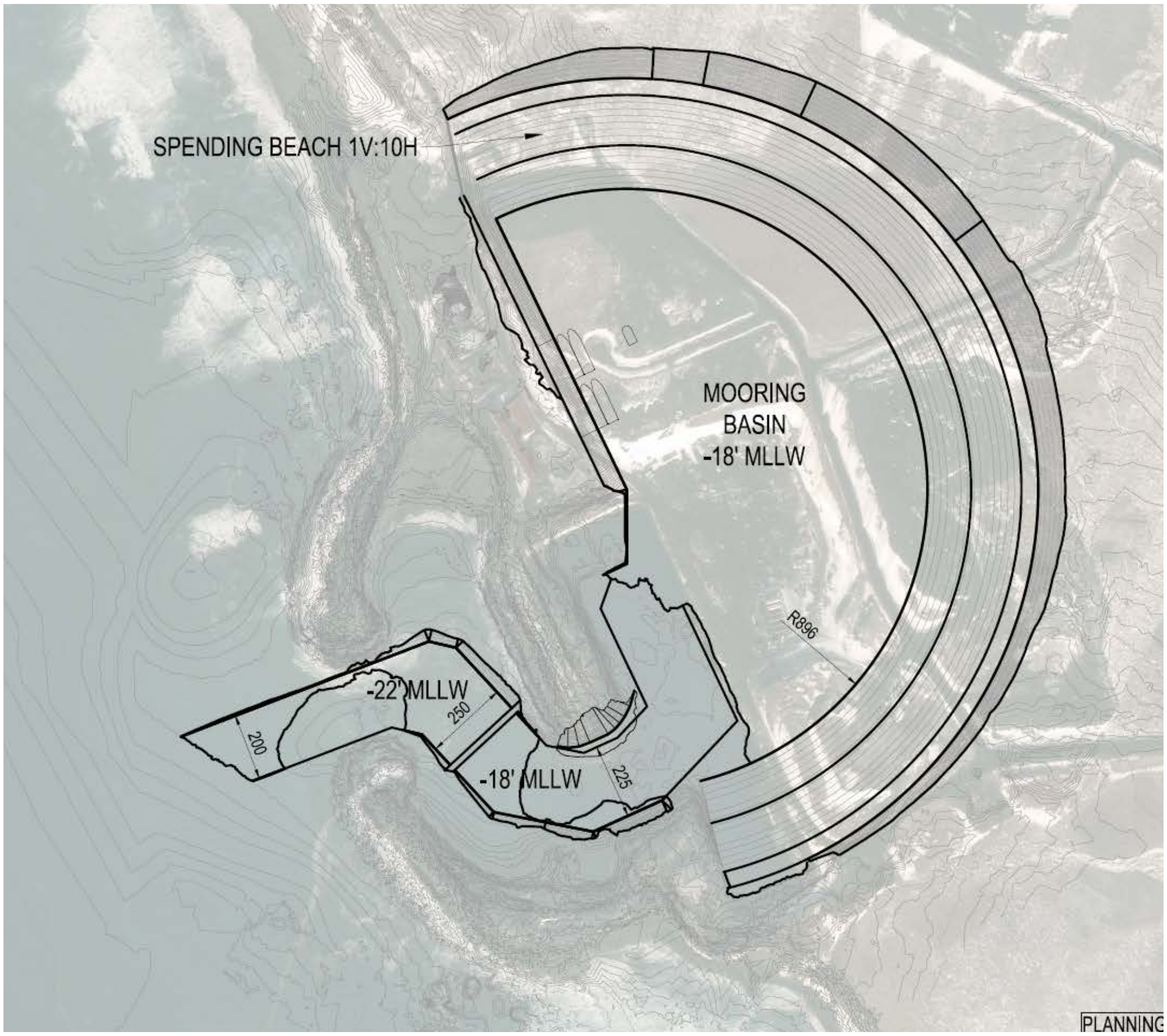


Figure A-47: Alternative Z-6 Concept Plan

5.6.1 Navigation Design

Primary construction of this harbor design would be through excavation and dredging. No new rock structures would be placed. Slope protection rock would be provided where the native rock was determined to be too small to provide slope protection under the expected wave conditions inside the harbor under storm conditions.

5.6.2 FUNWAVE Analysis

A FUNWAVE grid off this harbor was created to determine the effectiveness of the navigation features at providing a wave climate usable by vessels for transferring cargo (Figure A-44). The storms shown in Table A-18 and Table A-19 were run over this grid to determine harbor response. Selected gages were analyzed to measure harbor response. The critical gage on this grid is gage 26 which is at the location of proposed new docks. The model run results indicate that waves outside the harbor at the wavemaker location need to be less than 5.49 meters in height to produce wave heights at the docks less than 0.5 meters. It is estimated that sea conditions exceed this height approximately 4% of the time.

5.6.3 Harbor Effectiveness

The new dock location in the new basin showed improved wave conditions, however still showed a small percentage of time where the dock would be unusable. While the results show that the wave conditions in the mooring basin are improved, there are some responses with peak periods in the 650 to 820 second range indicating that there is still some degree of seiching occurring.

5.6.4 Alternative Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-23. The estimated project cost for this alternative is $176 million without contingency cost.

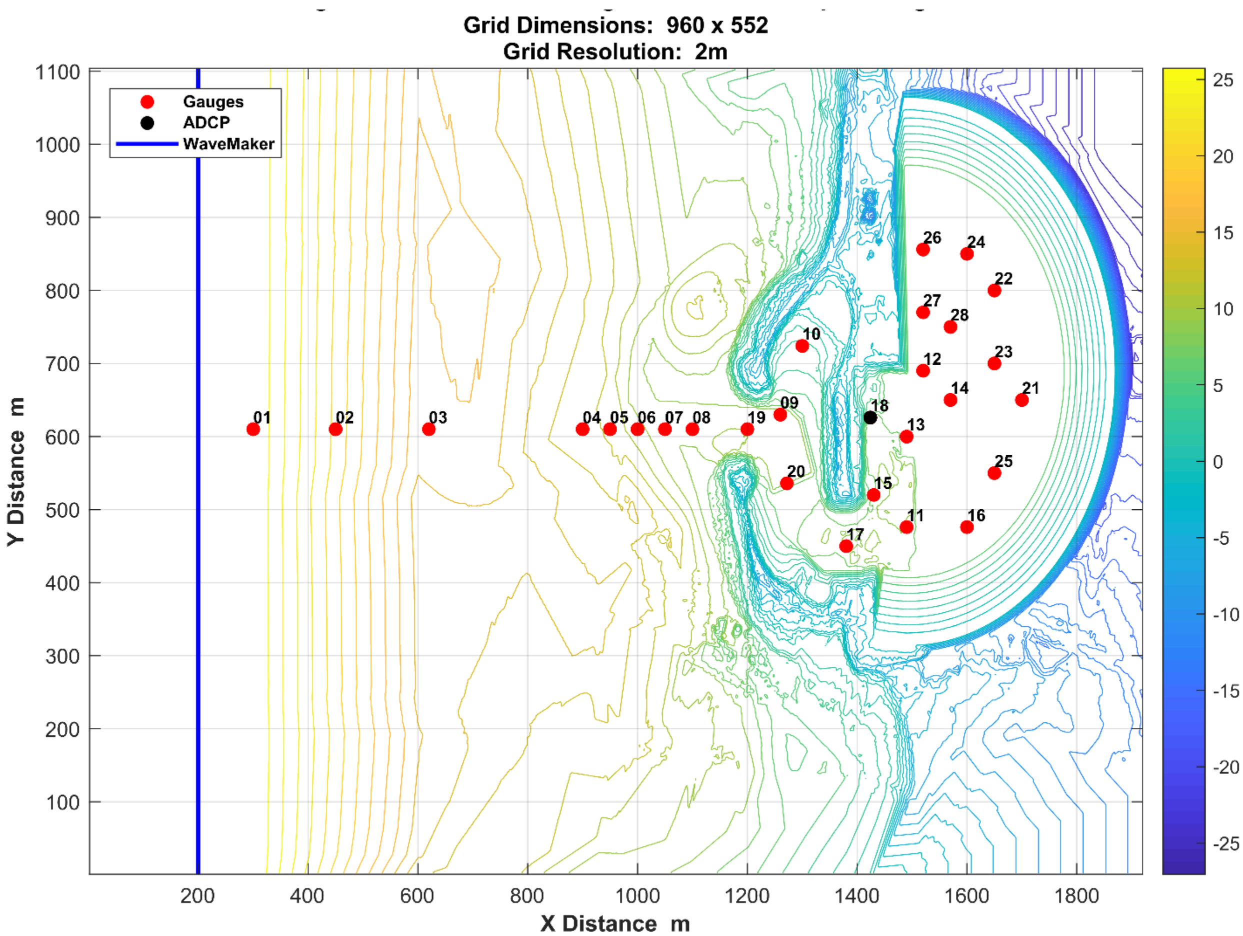


Figure A-48: FUNWAVE model grid for Alternative Z-6. Gage 26 was used to measure wave height at the new dock faces.

Table A-23: Alternative Z-6 Quantities

Dredging		Breakwater Shortening	
Mooring Basin	6,000,000 CY	Inner Breakwater	11,000 CY
Entrance Channel	23,000 CY		

7. NORTH SITE ALTERNATIVES

The North Site was found to have a lower design wave at the location new breakwaters were considered. This resulted in significant differences in the size and quantity of rock needed to protect an area from the open ocean environment. The descriptions of alternatives presented here is for information only to demonstrate the level of effort expended to attempt to find a solution at the existing harbor. One alternative at this site was selected as the Tentatively Selected Plan. Most of these alternatives share the same concept for breakwater design. Breakwaters exposed to the open ocean environment were designed as a 3 layer rubble mound breakwater with 10 ton armor stone and a crest elevation of +25 feet MLLW (Figure A-36). For these breakwaters, the sea side of the breakwater was designed at a 2H:1V slope and the harbor side was designed at a 1.5H:1V slope.

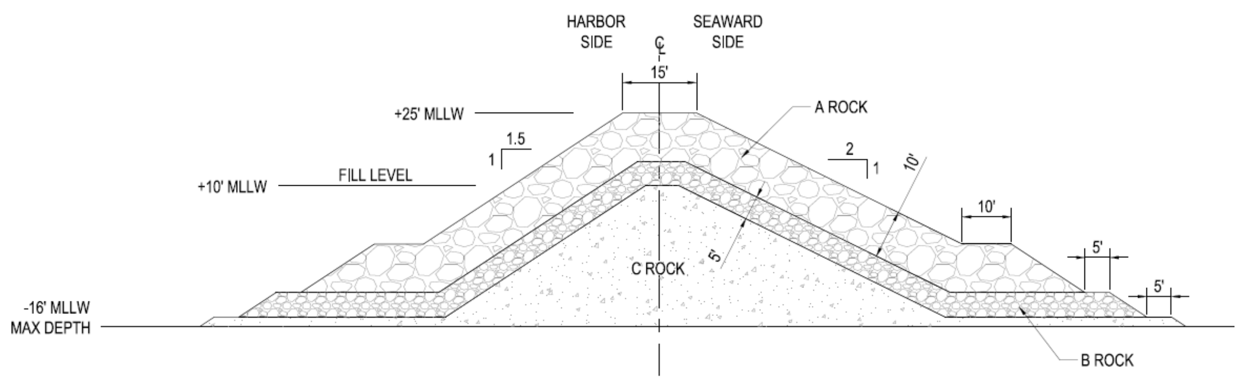


Figure A-49: Typical Breakwater Cross Section for North Site Alternatives.

Some variations of this design are indicated is some alternatives.

Numerical model development to determine the effectiveness of these alternatives is still underway; preliminary results for Alternative N-2 are believed to be high based on the model test performed for St. Paul Harbor.

Quantities and costs for these alternatives are included for comparison of relative effort of construction for each alternative. The costs presented in this appendix are rough order of magnitude project costs using the quantities estimated from CAD three dimensional surface models of the alternatives and assumptions for design and construction administration costs. These cost estimates do not include risk based contingency estimates and differ from the numbers found in the main report.

6.1 Alternative N-1: Subsistence Fleet Launch

This alternative includes constructing protected boat launch and recovery area for the local subsistence fleet. A new 675 foot long breakwater with 10 ton armor stone and a crest elevation of +25 feet MLLW would protect a new concrete launch ramp and launching basin. The launching basin would be dredged to -8 feet MLLW to provide full tide access for the fleet and connected to the Bering Sea with a 50-foot wide channel dredged to -10 feet. New uplands would be constructed inside the breakwater to provide a staging area for the subsistence fleet to launch and recover. Barge and fishing vessel access to St. George would continue to rely on the existing harbor at Zapadni Bay and would be unchanged by this alternative.

6.1.1 Structural Design

The new breakwater is subject to storm waves from the north and use a design wave height of 15 feet. This results in an average armor stone weight of 10 tons when constructed at a 2H:1V slope. The inner slopes of the breakwater would be constructed at 1.5H:1V except at the breakwater nose where the 2H:1V slope is wrapped around and carried through for 50 feet (Figure A-50). Where uplands abut the breakwater, the A rock extends over the crest for the full width but is omitted from the harbor side slope. This results in upland fill being placed against B rock.

6.1.2 FUNWAVE Analysis

A FUNWAVE grid of this alternative has not been developed. Model reflectivity issues encountered when analyzing Alternative N-2 would be an issue for this alternative. Also, no moorage analysis of this alternative is warranted; the harbor is designed for launch and recovery operations only, so the only wave criteria needed to analyze this harbor's effectiveness is the vessel access criteria, which is 4 feet for the subsistence fleet. Since this Alternative was not selected as the TSP, no further analysis is planned.

6.1.3 Harbor Effectiveness

Harbor effectiveness for Alternative N-1 is based solely on changes in vessel access which is a function of the site conditions measured at the WIS Station. Due to the harbor geometry and beach slopes of the coastline, seiching is not expected to be an issue.

Figure A-50: Plan view of Alternative N-1.

6.1.4 Alternative Quantities and Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-24. The estimated project cost for this alternative is $24 million without contingency cost.

Table A-24: Alternative N-1 Quantities

North Breakwater			
A-Rock	19,000 CY		
B-Rock	16,000 CY		
C-Rock	17,000 CY		
Dredging		**Upland Fill**	
Drill, Blast and Dredge	10,000 CY	Fill	22,000 CY

6.2 Alternative N-2: Fuel and Supply Barge Harbor

This alternative includes constructing protected boat launch and recovery area for the local subsistence fleet. A new 1,730-foot long North Breakwater with 10 ton armor stone and a crest elevation of +25 feet MLLW would protect a new 550 foot x 450 foot maneuvering basin, a 300-foot dock and concrete launch ramp. A Spur Breakwater with 10 ton armor stone and a crest height of +20 feet would be constructed inside the North Breakwater from the base of the cliffs along the south edge of the harbor to filter waves diffracted around the nose of the North Breakwater. The maneuvering basin would be dredged to -16 feet MLLW with a transition zone and an entrance channel dredged to -18 feet MLLW. The entrance channel maintains a 300-foot width from deep water to the end of the breakwater and includes widened turning section outside the breakwater nose. The channel narrows to 250 feet wide at the breakwater nose. The wind and wave climate as well as the wider entrance channel are expected to improve barge access to St. George.

6.2.1 Structural Design

The new breakwater is subject to storm waves from the north and use a design wave height of 15 feet. This results in an average armor stone weight of 10 tons when constructed at a 2H:1V slope. The inner slopes of the breakwater would be constructed at 1.5H:1V except at the breakwater nose where the 2H:1V slope is wrapped around and carried through for 50 feet (Figure A-50). Where uplands abut the breakwater, the A rock extends over the crest for the full width but is omitted from the harbor side slope. This results in upland fill being placed against B rock.

The launch ramp will be a precast concrete structure constructed at a 13% slope with vertical curves meeting highway design guidance to allow vehicular launching and recovery operations.

The dock is planned as a concrete deck on steel piles with a marine fendering system. The deck would be precast and post-tensioned in place to minimize the volume of concrete and grout required to be cast in place on site.

6.2.2 FUNWAVE Analysis

A FUNWAVE grid off this harbor was created to determine the effectiveness of the navigation features at providing a wave climate usable by vessels for transferring cargo (Figure A-44). The storms shown in Table A-18 and Table A-19 were run over this grid to determine harbor response. The model run results indicate that waves outside the harbor at the wavemaker location need to be less than 3.41 meters in height to produce wave heights at the docks less than 0.5 meters. It is estimated that sea conditions exceed this height approximately 7% of the time. Through study of a known harbor, it was determined that the FUNWAVE model was reflecting too much energy off the inner surfaces of the harbor and amplifying wave energy inside the protected area. Based on this information, it is assumed that a properly calibrated damped model would show a higher wavemaker wave threshold required to induce unmoorable conditions at the dock and reduce the percent of unmoorable time compared to these results.

6.2.3 Harbor Effectiveness

The new dock location in the new basin showed improved wave conditions compared to the existing harbor at Zapadni Bay, however still showed a small percentage of time where the dock would be unusable. Model results are assumed to show amplified wave conditions inside the harbor due to wave reflection off the inner harbor boundaries.

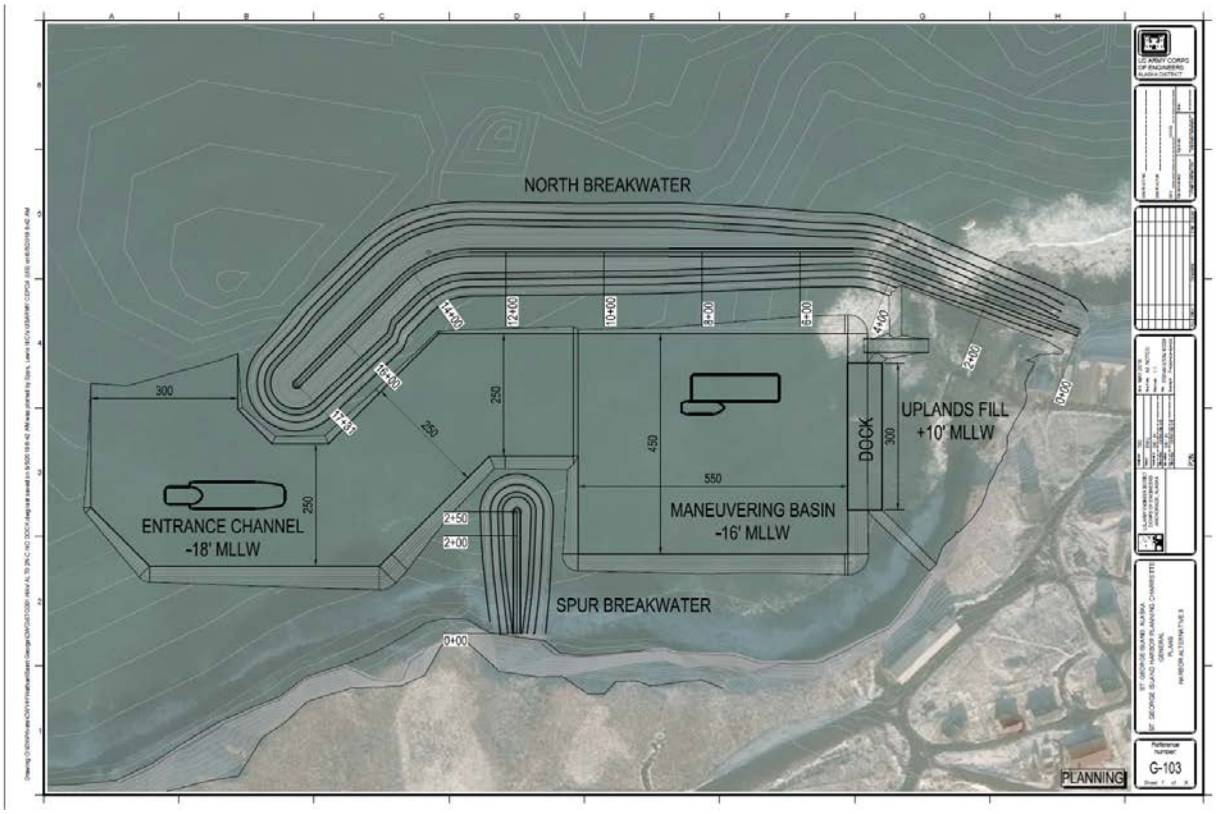


Figure A-51: Plan view of Alternative N-2.

6.2.4 Alternative Quantities and Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-25. The estimated project cost for this alternative is $89 million without contingency cost.

Table A-25: Alternative N-2 Quantities

North Breakwater		**Spur Breakwater**	
A-Rock	85,000 CY	A-Rock	8,900 CY
B-Rock	54,000 CY	B-Rock	6,500 CY
C-Rock	80,000 CY	C-Rock	4,800 CY
Dredging		**Upland Fill**	
Drill, Blast and Dredge	230,000 CY	Fill	44,000 CY

6.3 Alternative N-3: Crabber Fleet Harbor

This alternative is based on the same harbor size and geometry as N-2 with increased channel and basin depths to allow a greater percentage of the fishing fleet to moor. This alternative includes constructing protected boat launch and recovery area for the local subsistence fleet. A new 1,730 foot long North Breakwater with 10 ton armor stone and a crest elevation of +25 feet MLLW would protect a new 550 foot x 450 foot maneuvering basin, a 300 foot dock and concrete launch ramp. A Spur Breakwater with 10 ton armor stone and a crest height of +20 feet would be constructed inside the North Breakwater from the base of the cliffs along the south edge of the harbor to filter waves diffracted around the nose of the North Breakwater. The maneuvering basin would be dredged to -16 feet MLLW with a transition zone and an entrance channel dredged to -18 feet MLLW. The entrance channel maintains a 300-foot width from deep water to the end of the breakwater and includes widened turning section outside the breakwater nose. The channel narrows to 250 feet wide at the breakwater nose. The wind and wave climate as well as the wider entrance channel are expected to improve barge access to St. George.

6.3.1 Structural Design

The new breakwater is subject to storm waves from the north and use a design wave height of 15 feet. This results in an average armor stone weight of 10 tons when constructed at a 2H:1V slope. The inner slopes of the breakwater would be constructed at 1.5H:1V except at the breakwater nose where the 2H:1V slope is wrapped around and carried through for 50 feet (Figure A-50). Where uplands abut the breakwater, the A rock extends over the crest for the full width but is omitted from the harbor side slope. This results in upland fill being placed against B rock.

The launch ramp will be a precast concrete structure constructed at a 13% slope with vertical curves meeting highway design guidance to allow vehicular launching and recovery operations.

The dock is planned as a concrete deck on steel piles with a marine fendering system. The deck would be precast and post-tensioned in place to minimize the volume of concrete and grout required to be cast in place on site.

6.3.2 FUNWAVE Analysis

Alternative N-3 has not been modeled in FUNWAVE. The breakwater geometry is identical to Alternative N-2 with only minor changes in channel and basin depth. While internal dampening of the model is under development, the FUNWAVE results for Alternative N-2 are used to represent the effectiveness of Alternative N-3.

6.3.3 Harbor Effectiveness

The new dock location in the new basin showed improved wave conditions compared to the existing harbor at Zapadni Bay, however still showed a small percentage of time where the dock would be unusable. Model results are assumed to show amplified wave conditions inside the harbor due to wave reflection off the inner harbor boundaries. Modifications to the model to dampen these boundaries are still in progress.

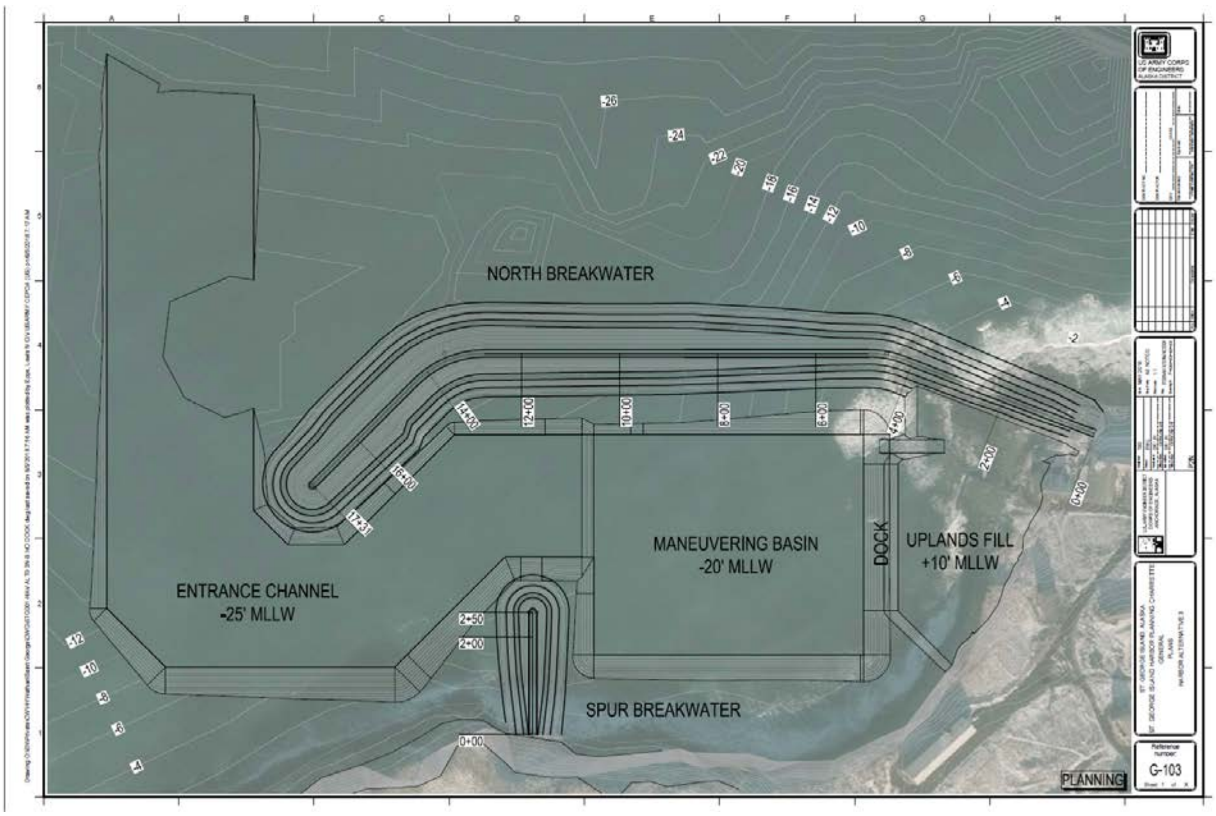


Figure A-52: Plan view of Alternative N-3.

6.3.4 Alternative Quantities and Cost

Quantities for this alternative were based on volumetric calculations of TIN surface modeling of the harbor features in Autodesk Civil3D. These quantities were calculated to the nearest CY, however due to uncertainties in terrain modeling, should only be considered accurate to two significant figures. Rounded quantities for this alternative are shown in Table A-26. The estimated project cost for this alternative is $101 million without contingency cost.

Table A-26: Alternative N-3 Quantities

North Breakwater		Spur Breakwater	
A-Rock	85,000 CY	A-Rock	8,900 CY
B-Rock	54,000 CY	B-Rock	6,500 CY
C-Rock	80,000 CY	C-Rock	4,800 CY
Dredging		**Upland Fill**	
Drill, Blast and Dredge	430,000 CY	Fill	44,000 CY

7 PROJECT IMPLEMENTATION

7.1 Breakwaters

Breakwater and causeway construction would typically be performed under a USACE administered contract to ensure that minimum construction requirements are met as the port alternatives are built. The breakwater and causeways would use several layers of stone armor to achieve wave protection and filtering criteria. All material used in the construction of these project features would be of a self-compacting nature consisting of rock spalls or dredged tailings that can be placed underwater by excavator bucket, skip box, or dump scow. Fill prisms and “C” rock layers would be randomly placed and controlled by construction survey to assure that design elevations and layer thicknesses were met. Larger stone, typically “B” rock and “A” rock layers would be placed selectively by an excavator with an articulated thumb or crane with rock tongs to achieve minimum stone to stone contact requirements. Placement of stone would likely be performed by equipment mounted on a barge until the breakwaters were built up above the tide range, then placement would be with an excavator on the top of the breakwater.

7.2 Dredging

The material at all sites is assumed to require blasting and mechanical dredging equipment to reach design depths. Dredging features typically include a 2-foot allowance for overdredge to ensure that the minimum required depth is met. Blasting also requires a minimum 2-foot depth allowance to ensure that minimum depth is achieved, so blasting patterns would need to be established to loosen material to 4 feet below the minimum required depths designed for the selected plan. The dredge machinery would load a scow, which would deliver the dredged material to an offshore disposal site. Multiple scows may be used to provide for continuous dredging operations.

7.3 Local Service Facilities

For each of the three alternatives, it is assumed that the local service facilities would be constructed under the same contract for the Federal features of the project. Local service facilities include the non-Federal dredging areas, docks, fendering systems, mooring dolphins and bollards, launch ramps, utilities, fuel tanks, access roads, and road bed surfaces. The non-Federal dredging portions of the project are represented by the area adjacent to the proposed dock faces out to an offset distance of approximately two vessel beams in width.

Upland staging and laydown areas are also local service facilities. These would be constructed concurrently with the harbor project.

7.4 Aids to Navigation

As part of the construction of the project, concrete navigation marker bases would be constructed at the heads of the new causeways and/or breakwaters. Coordination with the U.S. Coast Guard Aids to Navigation Office will be conducted to ensure that necessary marking of the new entrance channels are considered. New navigation towers and lights would be incorporated into the head of the new causeways and/or breakwaters for any of the alternatives. The Coast Guard would install the navigation lights and signage after construction is completed. In addition, navigation aid day markers would continue to be installed seasonally by the City of Nome for the Nome alternatives to mark the inner entrance channel limits between the causeway and the main breakwater. These markers are in the form of bottom anchored buoys. Red and green color coding is provided and would correspond with the new signage installed on the causeway extension and existing main breakwater as appropriate. The existing navigation aid marker base on the spur breakwater would be removed. For Alternative 1C, the existing navigation marker base would be repositioned on the re-aligned main breakwater head. The existing range boards and lights located on-shore would likely remain with some possible modifications in elevation to guide navigation in the inner channel/maneuvering area.

7.5 Construction Schedule

Major construction features for the TSP include rubblemound north and spur breakwaters, dredging, pile supported docks, and upland fill areas. The material source for A and B rock would be offsite from an established quarry such as Cape Nome or Granite Cove on Kodiak Island. The material source would most likely be far enough away from the site that rock production would need to significantly lead placement operations to ensure that the construction crew on site has enough material delivered to the site for a full season of work. Stone production in the quarry and delivery to the site would likely be the first project tasks undertaken.

Construction of the North Breakwater is most likely to be performed with land based equipment. The breakwater core would be constructed to above the tide range to allow the placing equipment to drive the breakwater core and place B and A rock layers to protect the work in progress. Core rock would likely be transported and staged on the breakwater with off-road dump trucks, then shaped to the design prism by an excavator.

Near the west end of the breakwater, an excavator on a barge may be required to shape the toe and benches of the breakwater where the seabed is deeper. Uplands would be constructed concurrently with the breakwater to build a staging area for breakwater material.

Dredging could occur concurrently with stone production; initial dredging and blasting is expected to be a winter activity to protect nearby fur seal rookeries. Dredging opportunities during these months are limited due to adverse weather and the blasting program could take three years to complete. Some dredging prior to constructing the breakwaters would provide access for construction barges to the breakwater sites. The total estimated performance period for construction the project is a minimum of 3 years and likely would be 5 years.

8 OPERATIONS AND MAINTENANCE

The non-Federal operator of the Port would be responsible for operation and maintenance of the completed mooring areas and local service facilities portion of the project. The Federal Government would be responsible for maintenance of the causeway extension and breakwaters (except for the road prism and surfaces, and docks and other local service facilities) and the entrance channel portions of the project. Personnel from the Alaska District would visit the site(s) periodically to inspect the causeways and breakwaters and perform hydrographic surveys at 3- to 5-year intervals for the dredged areas. The hydrographic surveys would be used to verify whether the predicted minimal maintenance dredging was warranted for the entrance channel and maneuvering areas. Maintenance requirements for the causeways and breakwaters would be determined from the surveys and inspections. Local and Federal dredging requirements, if necessary, would probably be combined, so there would be only a single mobilization and demobilization cost.

The causeways and breakwaters were designed to be stable for the 50-year predicted wave conditions. Therefore, no significant loss of stone from the rubblemound structures is expected over the life of the project. It is estimated that at the worst case, 2.5 percent of the armor stone would need to be replaced every 25 years. Because stone quality would be strictly specified in the project construction contracts, little to no armor stone degradation would be anticipated. For the TSP, Alternative N-3, a quantity of 2,100 CYs of A-Rock would be required for replacement on the North and Spur Breakwaters at year 25.

Maintenance dredging would be conducted on an estimated 10-year cycle. The entrance channel and maneuvering area would require dredging of approximately 10,000 CYs. A dredged material management plan would be developed for the project in which a long-term disposal option would be identified. For purposes of this study, it is assumed that the entrance channel and maneuvering area material would be disposed of in the offshore disposal area east of the harbor. Clamshell bucket dredging equipment with a scow barge would likely be used for maintenance dredging. Dredged material characteristics should be easier to remove than construction dredging of the area and no blasting would be required for maintenance.

9 REQUIRED FURTHER DESIGN STUDIES

The following are items that require further study in the preconstruction engineering design phase of the project before plans for construction can be published:

Geotechnical investigation and analysis of subsurface materials at the North Site to determine their physical characteristics and chemical composition, dredging methods and equipment requirements, and suitability as foundation materials for the proposed causeways, breakwaters, docks, and upland facilities.

A detailed physical model study in a wave tank that is capable of simulating wave spectra originating from multiple directions of approach. This step is necessary to validate numerical model results and to identify harbor-specific hydrodynamic issues that the numerical models are not capable of replicating. This study needs to be performed in a facility dedicated to wave modeling run by full time research engineering staff. The Corps owns and operates the necessary facilities at the ERDC Coastal Hydraulics Laboratory in Vicksburg, MS.

Detailed design of local service facilities including the proposed docks, fender systems, mooring dolphins and bollards, utilities, access roads, uplands staging and laydown areas and launch ramps.

10 REFERENCES

HDR. 2014. “St. George Breakwater Improvements, Existing Conditions Wave Modeling & Baseline Conditions Report,” AKSAS Project No. 54525.

HDR. 2015. “St. George Breakwater Improvements, Preliminary Design Report, Final DRAFT” AKSAS Project No. 54525.

Moritz, H. P. and Mortiz, H. R. 2008. “Evaluating Potential Implications of Cumulative Storm Power on Coastal Infrastructure Damage and Shoreline Erosion”. U.S. Army Corps of Engineers, Portland District.

Permanent International Association of Navigation Congresses (PIANC), 1995. “Criteria for Movements of Moored Ships in Harbours, a Practical Guide, Report of the Working Group no. 24 of the Permanent Technical Committee II, Supplement to Bulletin No. 88”.

Tippetts, Abbett, McCarthy, Stratton (TAMS) Engineers, 1982. “Port of Nome, Alaska, Design Memorandum”.

U.S. Army Corps of Engineers (USACE). 2008. “Coastal Engineering Manual” - Part II. EM 1110-2-1100.

USACE. 2006. “Hydraulic Design of Deep-Draft Navigation projects,” EM 1110-2-1613.

USACE. 1984. “Hydraulic Design of Small Boat Harbors,” EM 1110-2-1615.

USACE. 1987. “Environmental Engineering for Deep-Draft Navigation Projects,” EM 1110-2-1202.

USACE. 1986. “Engineering and Design, Design of Breakwaters and Jetties,” EM 1110-2-2904.

USACE. 1989. “Water Levels and Wave Heights for Coastal Engineering Design,” EM 1110-2-1414.

USACE. 2011. “Sea-Level Change Considerations for Civil Works Programs,” Engineer Circular EC 1165-2-212.

USACE. 1998. “Design for Navigation Improvements at Nome Harbor, Alaska, Coastal Model Investigation” - Technical Report CHL-98-28.

USACE. 2009. “Storm-Induced Water Level Prediction Study for the Western Coast of Alaska,” ERDC/CHL Letter Report.

USACE. 1984. “Shore Protection Manual,” Fourth Ed.