Alaska Deep Draft Arctic Port System Feasibility Study

Appendix A - Hydraulic Design

Nome & Port Clarence, Alaska

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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 the Port of Nome, Point Spencer and Cape Riley, 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 and ice conditions for design of the proposed navigation improvements.

1.2 Project Purpose and Needs Assessment

The following objectives were identified for navigation improvements at Nome, Point Spencer, and Cape Riley.

a. Provide safe and more efficient improvements for the various design fleets.

b. Provide facilities for fuel barges, US Coast Guard vessels, oil and gas support vessels, and resource extraction vessels for which current depths and facilities are not available.

c. Reduce travel delays and increase port operation efficiencies.

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

2. SITE SELECTION

This study encompasses three sites on the Seward Peninsula; the Port of Nome, Point Spencer at the entrance to Port Clearance, and Cape Riley to the southwest of Teller as shown in Figure 1. These sites were selected through a charrette process that included stakeholders at the Federal, State and local levels. The charrette considered many sites in the region and settled upon the three sites evaluated in this study. The study compares the potential use of these sites as stand-alone projects or combination projects with multiple sites used to cover a range of missions and economic activities. Only one alternative design was considered for each site or combination of sites. In each case the alternatives were considered to be the optimal designs for their intended use by the project delivery team (PDT). The selected alternative will be refined in further detail during the preliminary engineering design (PED) phase.

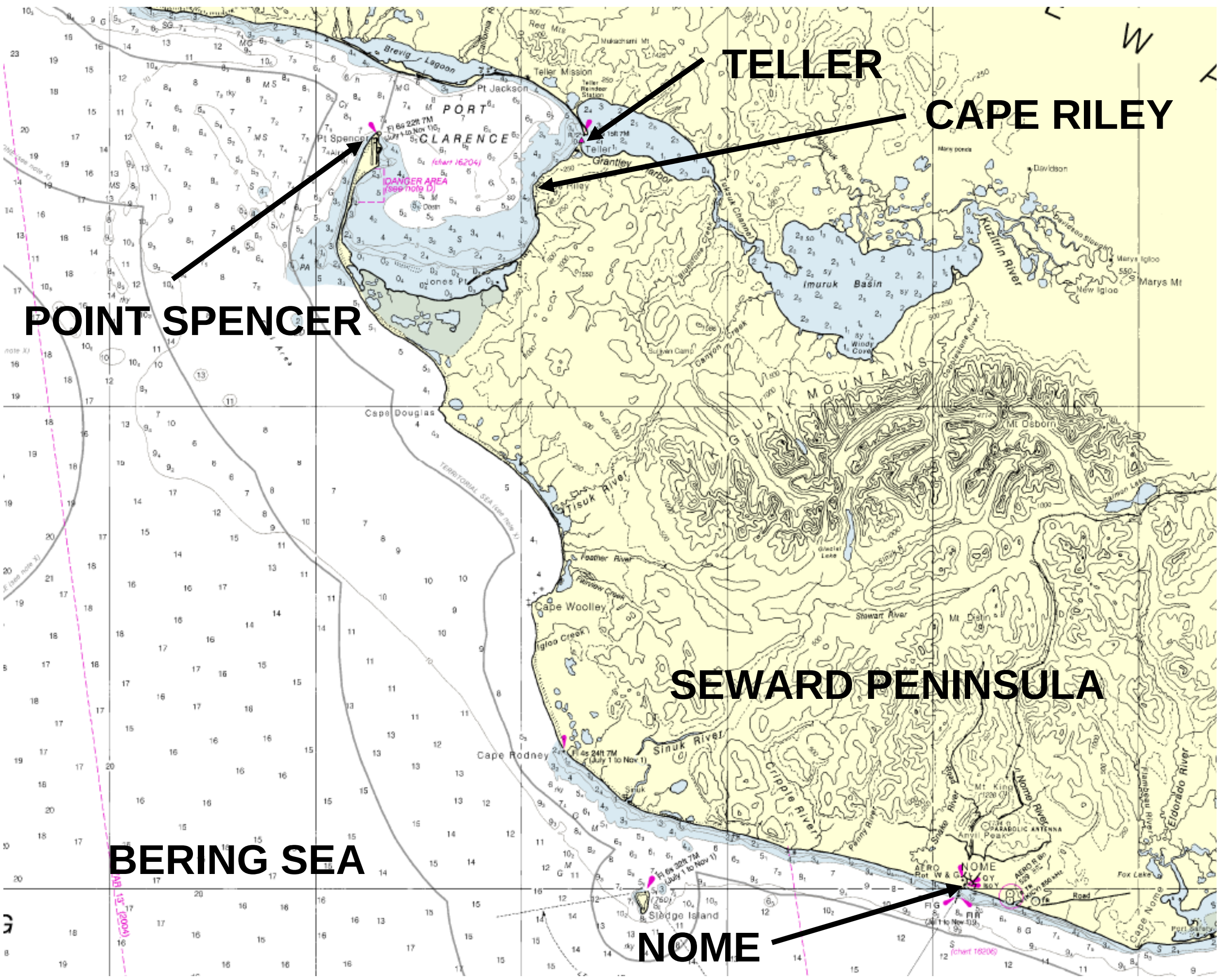


Figure 1: Site locations. Detail from NOAA Chart 16200, Norton Sound to Bering Strait. Annotation Added. The latitude lines on this chart are 30 minutes apart, equivalent to 30 nautical miles or approximately 34.5 statute miles.

3. PORT OF NOME

The Port of Nome is the regional hub for maritime commerce. The port includes a federally maintained navigation channel and turning basin protected by rubble mound breakwaters. Existing facilities at the port accommodate medium draft vessels with maximum moorage depths of -22 feet MLLW.

3.1 Existing Facilities

The Nome Federal navigation project was first authorized by the Rivers and Harbor Act of August 8, 1917. The authorization was to construct jetties, dredge a channel, and armor the banks of the Snake River with a stone revetment. Subsequent construction included modification of the original jetties, construction of a seawall along the Nome shoreline, construction of a rubble mound causeway into Norton Sound, construction of sheet pile docks on the causeway, construction of a breakwater adjacent to the causeway and re-alignment of the Snake River. Construction of Port of Nome and related facilities progressed as follows:

1923 - The original 335 and 460-foot timber and concrete jetties and the revetments are completed.

1940 - The jetties are reconstructed with steel reinforced concrete to modified lengths of 240 and 400 feet.

1949 - Work begins on the seawall and the 400-foot extension to the turning basin. Records indicate annual maintenance dredging.

1951 - Construction of the seawall is completed in June. Extension of the turning basin to 600 feet in length is effectively completed.

1954 - The timber revetment was re-faced with sheet steel piling.

1964 - Contract is awarded for the repair of both jetties in July.

1965 - Repair to the jetties is completed in October.

1982 - The east jetty incurs damage in the spring of 1982; the last 40 feet is detached from the remainder of the structure.

1985 - Construction of 700 linear feet of the Nome Causeway is completed.

1986 - Interim repairs are completed on the sheet pile wall in the entrance channel; Nome Causeway construction completed to 2,700 linear feet.

1989 – 190’ long West Gold sheet pile dock constructed on the Nome Causeway.

1991 – 200’ long City Dock sheet pile dock constructed on the Nome Causeway.

2004 - Construction begins on the new breakwaters and entrance channel.

2005 - The original entrance channel is dredged for the last time in June and closed off in July after construction of the new entrance channel. Construction on the new breakwater and causeway spur continues.

2006 - The new entrance channel is dredged. The breach through the sand spit is armored to prevent sloughing of material into the channel. New steel sheet pile is installed on the south side of the inner harbor.

2007 - Construction of the sheet pile replacement on the south side of the inner harbor is completed.

2008 - Construction of the sheet pile replacement on the Crowley (east) dock is completed.

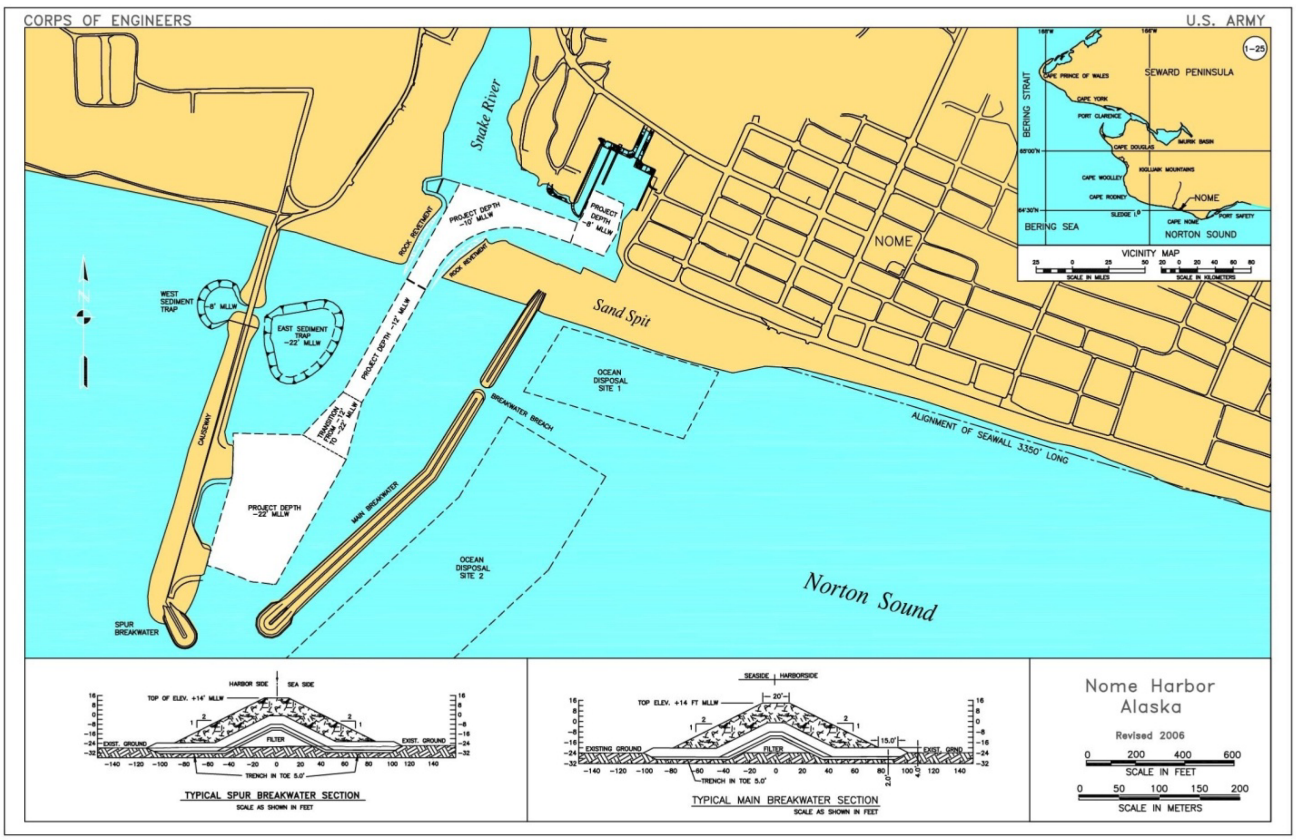


Figure 2: Port of Nome Federal Projects from Project Maps and Index Sheets, 2012.

3.1.1 Causeway

The causeway was designed by Tippetts, Abbett, McCarthy, Stratton (TAMS) Engineers to replace the historic open-water lightering system used for resupply of Nome and outlying villages. The City of Nome began construction of the causeway in November 1985. Construction history and other information is contained in the "1989 Annual Report of Nome Littoral Drift Monitoring and Shore Protection Program," prepared by the city of Nome. This report chronicles the construction sequence as follows.

In November 1985, 700 linear feet of the causeway was constructed. Causeway construction was shut down for the winter season on November 21, 1985. In mid June 1986, construction of the causeway resumed, with work proceeding until November 1, 1986, when the causeway reached its current length of 2,700+ linear feet. Final armor stones were placed in June 1987.

The causeway was constructed with 22-ton, 16-ton, and 8-ton armor rock on the west side of the trunk, 22-ton rock at the head, and 8-ton rock on the east side of the trunk. The side slope of the west side and head is 2H:1V; the side slope on the east is 1.5H:1V. The top elevation of the west face varies from +20 to +28 feet MLLW. The east face top elevation is +15 feet MLLW. The core of the causeway is pit run tailings, which is clean gravel with a maximum unit weight of 100 pounds and not more than 5 percent by weight passing a 200 sieve. Adequate mass was placed to resist up to 110 kips per linear foot of lateral ice thrust.

A breach was left in the causeway for passing fish and other marine life. The breach was established at the -7-foot MLLW depth contour and bridged to allow cargo transfer. This bridge was replaced in 2005 when the main breakwater was constructed with a steel girder bridge supported by Open Cell™ abutments. The bridge has a two-axle load capacity of 120 tons and a three axle load capacity of 130 tons. Heavier loads require a special overload permit from the Alaska District.

After completion of the causeway, two earth-filled circular sheet pile docks were constructed on its east side. The inner or northernmost cell, West Gold Dock, was completed in the fall of 1989. The outer cell, City Dock, was completed in August 1991. Both docks were designed by a consultant and are owned and operated by the City of Nome. The outer dock is used by the large petroleum barges and has the pipeline and headers for this type of cargo transfer.

The causeway structure is in excellent condition. There has been no known movement of armor stone due to wave and ice forces since its initial construction. Cargo handling is curtailed to some degree because the bridge across the fish passage breach controls loads on the causeway road.

3.1.2 Seawall

The original project was constructed during the years 1947 through 1951. The revetment protects the town from damage during the severe coastal storm events that frequently occur in Norton Sound. The total length of the federally constructed, locally maintained portion of the project is 3,350 linear feet beginning just east of Campbell Avenue at the east end and proceeding westerly. As part of a Federal navigation project, a 460-foot extension on the west end was constructed in 2005. The City of Nome is not responsible for O&M on the 460-foot extension of the rock revetment. The Federal portion of the project crosses where the old harbor entrance channel was located and ends approximately aligned with West D Street. The original project consists of a rock revetment with a single layer of armor stone on a 2H:1V side slope and a 15-foot-wide toe apron (Figure 3). The crest elevation of the revetment is +18 feet MLLW. The seawall extension consists of a rock revetment with 8 to 10-ton armor stone on a 1.5H:1V side slope and a 15-foot-wide toe. Its crest elevation is also +18 feet MLLW. Armor stone weights for the original project are assumed to range from 12,000 to 20,000 pounds with some percentage larger stones. Armor stone specified for the 460-foot extension have a median weight of 16,000 pounds and range from 12,000 to 20,000 pounds.

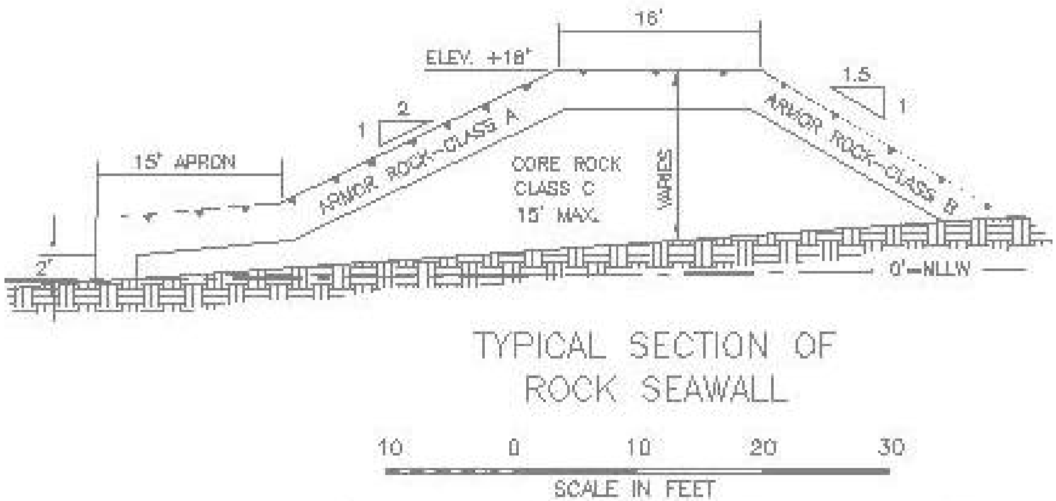


Figure 3: Nome Seawall Typical Section. This section is typical of the 1951 construction. The 2005 construction near the breakwater has side slopes of 1.5H:1V.

3.1.3 Main and Spur Breakwaters

A Corps of Engineers Navigation Improvements project was designed in 1998. The design consisted of a new main breakwater to the east of the causeway, a new spur breakwater extension of the causeway head, and a new dredged entrance channel into the harbor between the main breakwater and spur breakwater/causeway. The spur and main breakwaters were constructed in 2005. The outer layers of the spur and main breakwaters consist of 22-ton, 16-ton and 8-ton armor stone, with the heaviest stone placed at the head of the breakwater and lighter stones placed shoreward. The crest elevation is + 14 feet MLLW; the crest is 20 feet wide and the armor stone was placed on a 2H:1V slope. The main and spur breakwaters are owned by the City of Nome, however O&M responsibility remains Federal. Figure 4 shows the typical section for the spur and main breakwater heads.

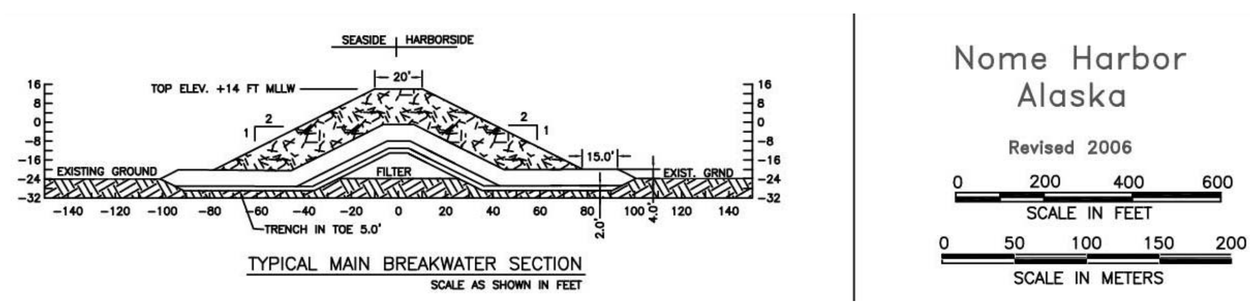


Figure 4: Spur and Main Breakwater Head Typical Section.

3.1.4 Entrance Channel and Turning Basin

The entrance channel to the Port of Nome was initially dredged in 2006 to a maximum depth of -22 feet MLLW. The entrance channel includes the mooring basin that covers the West Gold Dock and City Dock on the Nome Causeway. The entrance channel transitions and continues north past the West Gold Dock to a depth of -12 feet MLLW, and then transitions to -10 feet MLLW at the mouth of the Snake River, where the channel turns to the east towards the small boat harbor. The harbor is maintained at -8 feet MLLW; however, the most recent surveys show depths ranging from -10 to -11 feet MLLW along the south bulkhead inside the harbor.

3.1.5 Small Boat Harbor

The City of Nome upgraded and expanded the small boat harbor and its facilities. The area of upgrade is adjacent to the northern portion of the existing federally authorized turning basin. The upgraded project included constructing sheet pile bulkheads, dredging existing material to achieve depths of -8 to -10 feet MLL W, filling an area of approximately 6 acres for uplands, constructing floating docks and ramps, and placing riprap along the shoreline. The city started construction in 1997 by dredging the west half of the harbor and completed the harbor improvements in 2000.

3.1.6 Harbor Bulkheads

Four sheet pile docks are located within the inner (small boat) harbor: the south bulkhead constructed in 2003, the east bulkhead, the fishery dock, and low level dock. See Figure 8 in Appendix B for existing inner harbor bulkhead facilities layout.

3.1.6.1 South Bulkhead

The south bulkhead is an HZ975B-12/AZ18 combi-wall with cantilevered AZ34 and AZ18 sections on the eastern end of the wall. This bulkhead was completed in 2006 and is a Federal project. The wall was intended to handle 3,000 pounds per square foot (psf) track loads along the combi-wall section; however, subsequent analysis of the wall has led to reduced load ratings.

3.1.6.2 East Bulkhead

The east bulkhead is a tied-back AZ34 sheet pile wall. This was designed to handle 5,000 psf track loads and is primarily used by Crowley Marine for loading and unloading fuel supply vessels. The east bulkhead is also a Federal project.

3.1.6.3 Low Level Dock

An Open Cell™ low level dock is located to the north of the east bulkhead. It serves the smaller barges and low height vessels during normal tide stages and water levels. During storm surge events, the dock is designed to overtop.

3.1.6.4 Fishery Dock

The fishery dock is located on the western edge of the harbor across from the east bulkhead. This dock is also an Open Cell™ structure, and it serves the commercial fishing and crab fleet for offloading of product.

3.2 Climatology

The City of Nome is located along the northern coast of Norton Sound, approximately 545 miles by air northwest of Anchorage. The climate is influenced by both the Norton Sound and Bering Sea maritime conditions. Norton Sound typically has open water from early June to about the middle of November. Storms within the region during the summer and fall months result in extended periods of cloudiness and rain. Average daily summer temperature variation is slight due to maritime influence. July temperatures are typically in the range of 44 to 65 degrees F. Following freeze-up in November, an abrupt change from a maritime to a continental climate is prevalent. Temperatures generally remain well below freezing from the middle of November to the latter part of April with January typically the coldest month of the year. January temperatures range from -3 to 11 degrees F. Average annual precipitation is 18 inches, with 56 inches of snowfall. Precipitation reaches its maximum in late summer and drops to a minimum in April and May.

3.3 Water Levels, Currents, and Waves

3.3.1 Tides

The tidal influence at Nome is relatively small, and the tides are primarily diurnal. Much larger water surface elevation fluctuations occur at Nome due to storm surges than are shown below. Tidal data, referenced to Mean Lower Low Water, are shown in Table 1.

Table 1: Published tidal data for Nome, Alaska

Published tidal data for Nome, Alaska (ft)

Highest Observed Water Level (10/19/04)…	+9.80
Mean Higher High Water (MHHW)	+1.52
Mean High Water (MHW)............................	+1.33
Mean Low Water (MLW)............................	+0.30
Mean Lower Low Water (MLLW)..................	0.0 (datum)
Lowest Observed Water Level (11/11/05)......	-6.69

Source: NOAA NOS, Tidal Epoch 1983-2001, published 10/06/11.

From the above data, the mean tide level (arithmetic average of the Mean High Water and the Mean Low Water) is 0.83 foot. The mean tide range (the difference between Mean High Water and Mean Low Water) is 1.03 feet.

3.3.2 Sea Level Rise

The Corps of Engineers 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 Circular EC 1165-2-212 and detailed below. Three scenarios of “low,” “intermediate,” and “high” SLC are evaluated over the project life cycle. According to the EC, 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 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 GMSL change, as presented by the IPCC (IPCC 2007), 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 5 (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.

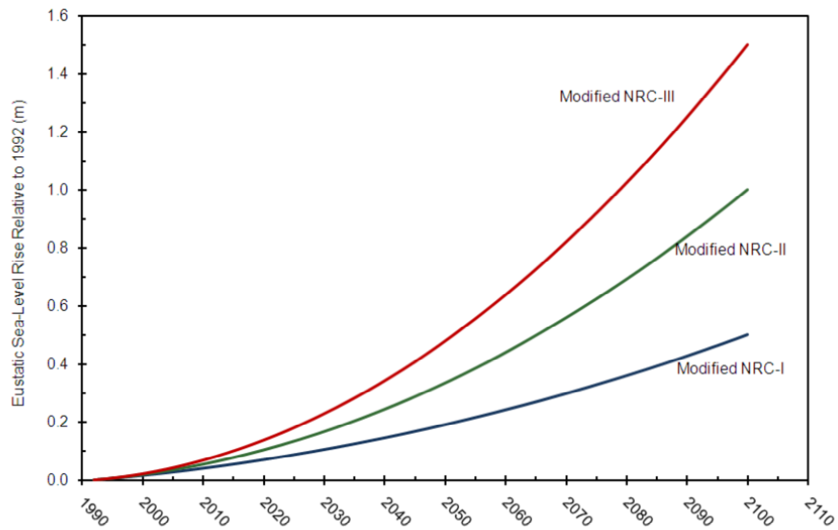


Figure 5: (Figure 5 from EC 1165-2-212). Scenarios for GMSL Rise (based on updates to NRC 1987 equation).

The local Nome tide station does not have the recommended 40-year period of record for the relative sea level change (RSLC) value. The Nome tide station has a 22-year period of record. Based on the tide data available, the RSLC would be +0.02mm/yr. Per the guidance recommendation, a U.S. tide station with a 40-year period of record was investigated for use as the RSCL value. The nearest U.S. tide station with the required 40-year period of record is the Seldovia, Alaska station, roughly 550 miles from the site. It has a historic relative sea level change (RSLC) of -9.45 mm/yr. Due to the distance from Nome, the Seldovia gage was not further investigated. To model sea level change at Nome, two scenarios were identified; the GMSL rate was compared to the data available from Nome (Table 2). While the Nome station does not have the recommended 40 year period of record, it more accurately accounts for isostatic rebound effects in the region which are not well represented by GMSL change. To best model sea level change in the region, the Nome station data was used. The sea level rise prediction used in the formulation of alternatives is the Nome historic prediction.

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

Scenario	Low (Historic)	Intermediate (Curve I)	High (Curve III)
GMSL	+0.28 feet	+0.75 feet	+2.24 feet
Nome	+0.00 feet	+0.47 feet	+1.97 feet

3.3.3 Storm Surge

The northern coastline of Norton Sound is subject to storm surge increases in water surface elevation due to its exposure to a long southwest fetch. Contributing to storm surges are the effects of the mildly sloping offshore shelf and shallow depths in the Nome vicinity. Positive storm surges are characterized as increases in water surface elevation from the normal astronomical tidal elevation. A storm surge consists of the water surface response to wind-induced surface shear stress and atmospheric pressure fields. Storm induced surges can produce short-term increases in water levels to an elevation considerably above mean tide levels.

The "Great Bering Sea" storm of November 12, 1974 was the most severe to hit Nome in the town's recorded history dating back to 1898. The storm surge rise in water level was the greatest on record: about 12 feet above MLLW. The predicted tide level and atmospheric pressure components combined have been estimated to be about 2 feet of the total during the 1974 event. The storm coincided with the highest tides of the month. This storm generated waves that overtopped the seawall with crest elevation of +18 feet MLLW fronting the City of Nome. The storm moved north-northeast from the central Aleutian Islands up through the Bering Strait with winds of 50 to 75 knots occurring within 12 hours of the frontal passage. The southerly fetch in the Bering Sea was about 1,000 miles long. Winds persisted over this fetch for about 36 hours. Along with causing widespread damage in Nome, this storm also caused flooding along the Bering Sea coastline.

Typically, the major storm surges occur in the Norton Sound area during the fall months. Throughout its history the City of Nome has experienced at least 18 occurrences of coastal flooding. With only two exceptions, the flooding occurred during the fall season. NOAA has established a tide gauge on the Nome Causeway, and it has been recording water surface elevations continuously since June of 1992. The water level data is available on NOAA's water level observation network web page.

The Engineer Research and Development Center (ERDC) published a study of predicted storm-induced water levels for the western coast of Alaska (USACE, 2009). The study presents the results of various numerical modeling techniques in the form of frequency of occurrence relationships for water levels at several selected communities in the region. For the Nome area, a 50-year storm surge water level of +9.66 feet MLLW was estimated.

3.3.4 Set-Down

Set-downs occur in the Nome area during periods of north winds and/or high pressure atmospheric conditions. The result is a lowering of the water surface elevation below that of the astronomical tide level. Set-downs typically occur during the fall months when north winds are more prevalent. The duration of set-down water surface elevations varies. Typically, a 2 to 3-day period of low water is observed. The most extreme set-down recorded at Nome of -6.69 feet MLLW was a rare event. More often, set-downs of -2 to -4 feet are observed. These are usually associated with north winds of approximately 20 knots and atmospheric pressures of 1,000 millibars and greater.

3.3.5 Currents

Measured current data is not available for Norton Sound offshore of Nome. Predicted current velocities based on tidal swings for Sedge Island are in the range of 0.5 knot on the ebb tide and

1.0 knot on the flood tide. Such values are likely representative for the Nome area. Sledge Island is approximately 20 nautical miles west of Nome.

Localized current velocities at the entrance to the Port of Nome vary depending on the wind and wave conditions. Local observations of current velocities of 0.5 to 0.8 knot have been reported. Stronger currents may be experienced by vessels navigating into and out of the port entrance channel when wave heights begin to exceed 4 to 5 feet and greater during storms.

The Corps of Engineers conducted a 3-D physical model study for the Nome Navigation Improvements project in 1999. As part of the study, wave induced currents were evaluated using scaled measurements of current velocities in the model. Various wave heights, periods, wave directions, and still water levels were tested. The results are detailed in the Coastal Model Investigation report. Generally, current velocities were measured in the range of 0.4 to 1.3 feet per second at the entrance between the spur and main breakwaters. The highest measured current velocity of 4.4 feet per second was recorded in the model for wave heights of 16 feet from the southwest at a still water level of 1.6 feet MLLW.

3.3.6 Wave Climate

The wave climate at Nome is governed by exposure to conditions in Norton Sound as well as the Bering Sea. During the ice-free season (generally between the first week in June to early November), waves can approach the shoreline from the southwest, south, and southeast depending on the wind direction. Short period wind waves can be generated by local winds in Norton Sound from the various directions of exposure. Longer period swell may also approach Nome from the Bering Sea window of exposure between St. Lawrence and St. Matthew Islands and the mainland. Generally, wave heights are less than 6 feet with periods less than 12 seconds. However, during strong southwesterly, southerly, or southeasterly winds, wave heights can increase to 10 to 15 feet with periods of 12 to 16 seconds. During storms associated with typhoon remnants propagating north toward the Aleutian Islands and into the Bering Sea, waves at Nome can reach 19 feet with periods greater than 18 seconds.

The original causeway design by TAMS was based on a wave analysis performed in the early 1980's. Wind data for the 130 degree to 260 degree directions of exposure were analyzed using stations at Nome Airport (1945-1982), St. Paul Island (1943-1982), and Northeast Cape (1953-1969). A storm with sustained winds of 50 knots over a period of 12 hours was selected as the design event. A design significant wave height of 16.7 feet and period of 8.9 seconds was calculated based on the CERC (1977 Shore Protection Manual) wave forecasting formulas. At the time of the TAMS study, there was no available measured wave data for Norton Sound or the Bering Sea.

A 10-year wave hindcast was developed during the Corps of Engineers' 1998 Feasibility Study for the Nome Navigation Improvements project. Winds derived from atmospheric pressure field data were analyzed and used to predict wave heights, periods, and directions. A storm surge height-frequency interval curve to establish the low and extreme high water levels was used in conjunction with predicted winds to develop the hindcast. Results provided a range of wave heights, periods, directions, and frequencies of occurrence of waves at a point offshore from Nome in water depths of approximately 30 feet. The numerical models WISWAVE and STWAVE were then used to transform the offshore wave characteristics to the navigation improvements project site. Two water levels were used in the analysis: 0 foot MLLW and +13 feet MLLW. Wave directions were consolidated into three groups. The percentage of time of occurrence for each directional group during the 7-month ice-free season was determined for each of the two water levels. The predominant wave directions, after accumulating them into direction bins, were from 15 degrees east and 30 degrees west of due south. Wave heights were predicted to exceed 3.3 feet (1 meter) about 48 percent of the time total (31.8 percent from the southwest, 11.8 percent from the south, and 4.4 percent from the southeast). The wave hindcast also identified extremal wave heights using the sample range of 1976 through 1996 for storm events. The 50-year wave height at a water level of +13 feet MLLW was determined to be 19 feet with wave periods of up to 16 seconds. For purposes of developing test wave conditions for physical model studies of the navigation improvements project, wave periods of 9, 12, and 16 seconds were used.

An updated wave hindcast was performed in 2010 by the Corps' Engineer Research and Development Center (ERDC). Wave height results based on 1985-2011 wind and pressure fields for Station 82100 are applicable for the Nome area. This station is located south-southwest of Nome in water depths of approximately 65 feet. The 50-year wave height is estimated at 21.9 feet as shown in Figure 6. Wave periods in the 10 to 12 second range were estimated.

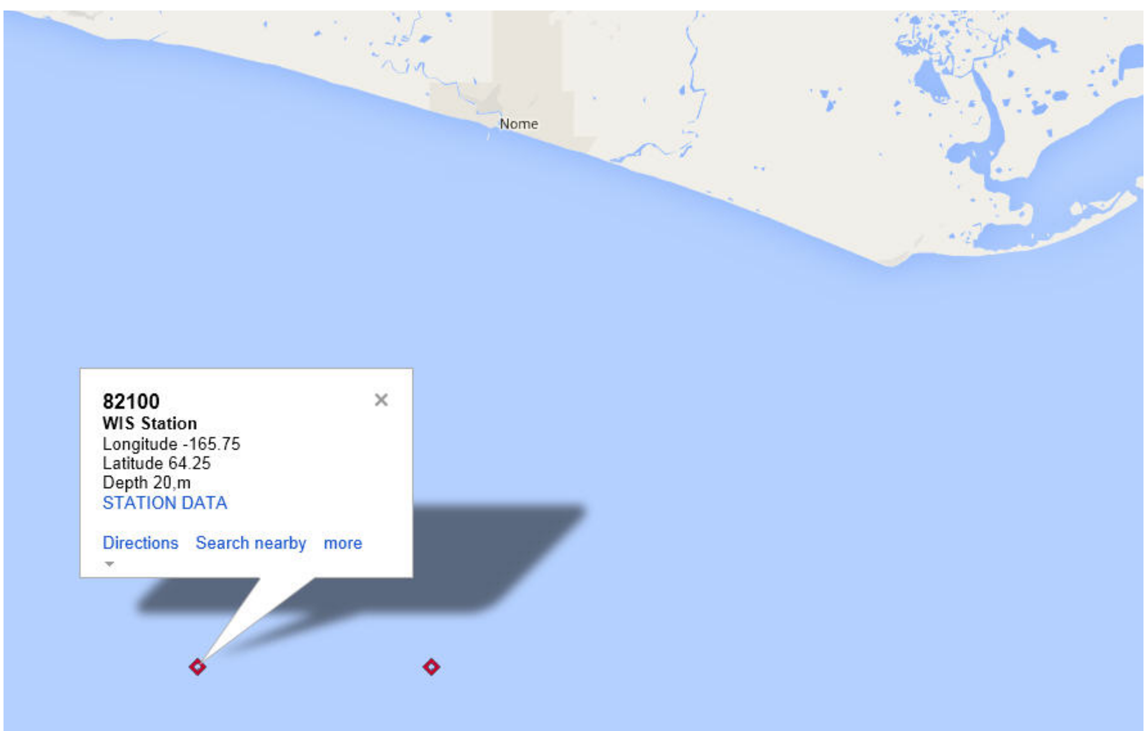
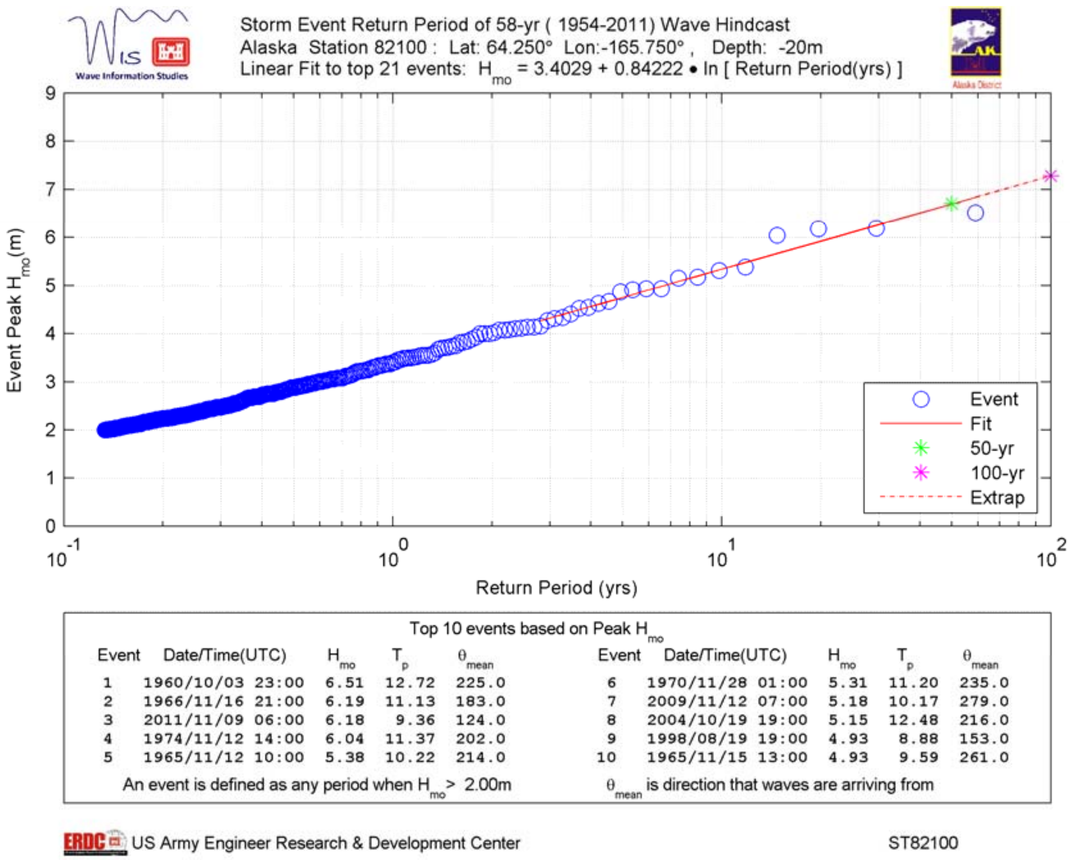


Figure 6: Location of WIS station 82100



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	1960/10/03 23:00	6.51	12.72	225.0	6	1970/11/28 01:00	5.31	11.20	235.0
2	1966/11/16 21:00	6.19	11.13	183.0	7	2009/11/12 07:00	5.18	10.17	279.0
3	2011/11/09 06:00	6.18	9.36	124.0	8	2004/10/19 19:00	5.15	12.48	216.0
4	1974/11/12 14:00	6.04	11.37	202.0	9	1998/08/19 19:00	4.93	8.88	153.0
5	1965/11/12 10:00	5.38	10.22	214.0	10	1965/11/15 13:00	4.93	9.59	261.0

An event is defined as any period when H_{mo} > 2.00m θ_{mean} is direction that waves are arriving from

Figure 7: Analysis figure for WIS station 82100 (Jensen, 2011)

For purposes of this study, the 50-year design wave height of 19 feet used for the navigation improvements project was selected for causeway armor stone stability design. For purposes of evaluating wave diffraction around the proposed causeway head, a range of wave periods varying from 6 seconds to 12 seconds was selected.

3.4 Hydrology

The Snake River is the predominant drainage in the project vicinity. It discharges into Norton Sound through the Spit and the navigation channel between the causeway and the main breakwater. The approximate drainage area of the basin is 86 square miles and the average daily discharge is less than 500 feet per second during the summer months. During breakup, however, the discharge may increase up to between 2,000 and 3,000 cubic feet per second. Dry Creek and Bourbon Creek also drain into the project area and discharge through culverts beneath Seppala Drive into the back portion of the small boat harbor. Both creeks provide an average of less than 20 cubic feet per second discharge contribution to the system during average summer conditions. Similar to the Snake River, their flows increase during breakup with snowmelt conditions.

3.5 Ice Conditions

Ice conditions within the project area include sea ice and shore fast ice. For the Nome area, sea ice formation typically occurs in early November each year; however, there have been years in which freeze-up in Norton Sound took place in mid-October. Spring break-up typically occurs in late May. Fast ice is sea ice of any origin that remains attached to shoreline features along the coast such as the existing breakwater, causeway, and seawall. Fast ice typically extends out from shore from 0.5 mile to approximately 7 miles depending on seasonal conditions. Near shore, the ice tends to be relatively smooth out to about .25 mile. From there the ice tends to become buckled offshore where the influence of pressure ridges are evident. Areas of large pressure ridges and possibly grounded pack ice have been observed in recent years at the entrance to the navigation channel between the spur and main breakwaters. Early winter ice sheet thicknesses of approximately 1 foot are typical. Maximum thicknesses of approximately 4.5 feet are predicted from computed freezing-degree-day estimates of ice growth. During years where pressure ridges are formed, estimated ice thicknesses at the ridges have been as great as 30 feet.

The Snake River typically freezes up during the end of November each year. Earlier freeze-ups can occur during late September to any time in October depending on seasonal weather patterns. The upstream portion of the river tends to freeze up first while the downstream portion near the mouth at the navigation channel freezes last. Spring break-up of the river typically occurs in mid May prior to break-up of the pack ice in Norton Sound. With increased river discharge in May, open leads begin to form in the navigation channel and tend to accelerate the pack ice breakup between the causeway and main breakwater. Little ice from the river itself flows down the channel to the mouth and river ice jams have not been observed in the area.

3.6 Sedimentation

Long-shore sediment transport was evaluated during the 1998 feasibility study. The predominant direction of littoral sediment movement along the shoreline at Nome is from west to east. A volume of approximately 120,000 cubic yards per year (gross) of material transported along the shoreline was estimated. The net west-to-east transport volume of 60,000 cubic yards per year was calculated and represents the deposition of material on the west side of the causeway.

As part of the 2006 navigation improvements project, three features were incorporated into the project for managing sediments: a west sediment trap, an east sediment trap, and an increased bridge span and deepened gap in the causeway. These features were designed in the physical model for the project at ERDC to transport and intercept sediments prior to reaching the navigation channel into the harbor. In addition, sediment management would also prevent the tip shoal at the seaward end of the causeway from growing larger and potentially impacting the wave focusing on the causeway navigation between the heads of the spur and main breakwater.

Since construction of the navigation improvements project, the Corps of Engineers has performed maintenance dredging annually in the navigation channel. The east sediment trap portion of the project has not required annual maintenance but has been dredged on an as needed basis. The west sediment trap has not been maintained. Sediments from the channel maintenance dredging have been discharged on the beach east of the main breakwater since the late 2000's. As a result, the steady buildup of the beach in front of the City of Nome has been observed along and in front of the rock seawall. This is an indication of the net sediment transport from west to east continuing after the completion of the navigation improvements project.

The Snake River's contribution to the sediment load in the system was also analyzed during the 1998 feasibility study. A volume of 5,900 cubic yards of sediment per year was estimated to be contributed to the system by the river. The majority of this material discharges into Norton Sound during spring break-up when ice cover is still present. River sediments are not expected to shoal and accumulate in the navigation channel.

4. PORT CLARENCE

Port Clarence is a large natural harbor on the Seward Peninsula of Alaska 60 miles northwest of Nome. The communities of Teller and Brevig Mission are located on the northeast coastline of Port Clarence. The Point Spencer and Cape Riley sites are located in Port Clarence. For the purposes of this study, site conditions at Point Spencer and Cape Riley will have the same atmospheric, tidal and ice site characteristics. Wave conditions, currents, site hydrology, and sedimentation will be analyzed separately for each site.

4.1 Climatology

The climate at Port Clarence is primarily influenced by Bering Sea maritime conditions. Port Clarence typically has open water from early June to about the middle of November. Storms within the region during the summer and fall months result in extended periods of cloudiness and rain. July temperatures are typically in the range of 45 to 57 degrees F. Following freeze-up in November, an abrupt change from a maritime to a continental climate is prevalent. Temperatures generally remain well below freezing from the middle of November to the latter part of April with January typically the coldest month of the year. January temperatures range from -4 to 9 degrees F. Average annual precipitation is 13 inches, with 47 inches of snowfall. Precipitation reaches its maximum in late summer and drops to a minimum from March through June.

4.2 Water Levels

4.2.1 Tides

Two tidal benchmarks have been established in Port Clarence, one at Teller and one at the mouth of the Lost River. The Teller benchmark was selected for alternative design because it is located within Port Clarence and is more representative of the basin (Table 3).

Table 3: Tidal data for Teller, Alaska

Published tidal data for Teller, Alaska (ft)

Mean Higher High Water (MHHW)	+1.18
Mean High Water (MHW)..............................	+1.06
Mean Low Water (MLW)..............................	+0.26
Mean Lower Low Water (MLLW)..................	0.0 (datum)

Source: NOAA NOS, Tidal Epoch 1983-2001, published 2/06/08.

4.2.2 Sea Level Rise

Due to the lack of local water level data, the sea level rise analysis for Nome was applied to the Port Clarence sites without modification.

4.2.3 Storm Surge

Water levels in the region are measured at two gages: Nome and Red Dog Dock. The period of record for these gages begins in 1992 and 2003, respectively. The gage data does not have sufficient period of record to establish frequency of occurrence relationships for water levels in the region. Probabilistic water levels for the western coast of Alaska were modeled using hindcast storm information applied to an ADCIRC long-wave model (USACE, 2009). The study grid was established to determine frequency occurrence relationships for 17 points along the western Alaska coast. Port Clarence is not a location covered under the Storm-Induced Water Level Prediction Study for the Western Coast of Alaska (WAK). The closest points studied in the WAK report are Nome and Wales.

The water level study was initially performed in 2005 using two separate grids; a grid developed to study water levels at Barrow which covered the Beaufort and Chukchi Sea coastlines which had a southern boundary in Port Clarence and a southern grid developed to capture the coastline from Bristol Bay to the Bering Strait which extended north past Shishmaref. Dr. Ray Chapman, a member of the research team, was contacted and site-specific data for Teller was found on the southern grid (Table 4). The data shows the 2 percent annual exceedance probability (AEP) water level to be 2.36 meters above Mean Lower Low Water, which corresponds to 7.74 feet.

Table 4: Summary of frequency-of-occurrence relationships for hindcast storm induced water level at Teller, Alaska

Return Period (Years)	Water Surface Elevation (ft, MLLW)	Standard Deviation (ft)
5	3.81	0.49
10	5.38	0.46
15	5.97	0.56
20	6.40	0.75
25	6.92	0.82
50	7.74	0.62
75	8.01	0.52
100	8.14	0.52

2005 WAK Study Bristol Bay Grid

4.2.4 Set Down

Local observations of water levels in Port Clarence have not been made. Set down is assumed to be in the -2 to -4-foot range, similar to Nome.

4.3 Ice Conditions

Ice conditions within Port Clarence can generally be characterized as shore-fast ice. Due to the geometry of the shorelines, ice within Port Clarence is not subject to the general movement trends characterized in the Bering Sea. Research on the movement of sea ice in the region suggests that Bering Sea ice interacts with the bottom at depths between 10 and 20 meters (33 and 66 feet) and that ice pack movement is generally parallel to bathymetric contours at these depths. Maps delineating shore-fast ice generally follow this feature with the region to the east of the berm being considered shore fast.

A submarine berm west of the entrance to Port Clarence is located near the atlas delineation of shore-fast ice and may have been formed by pack ice movement. Undeformed shore-fast ice in the region was estimated to be 6 feet thick in protected areas such as Port Clarence (USACE, 1970). Ice movement can create pressure ridges thicker than undeformed ice, however it was assumed that ice movement within Port Clarence was minimal and the ice ride-up issues that have been seen at Nome would not occur on a similar structure in Port Clarence.

Ice concentrations in Port Clarence have historically been above 50 percent from December through May. Ice concentrations in June have historically been at 30 percent. Ice movement in Port Clarence is most likely to occur in November as the ocean freezes when the ice is not thick enough to be shore fast or in the summer during breakup as ice melts and flows out of Port Clarence. Residents of Teller and Brevig Mission noted that vessels and oil rigs have been stored in Port Clarence over winter, however the specific details of these vessels were not known.

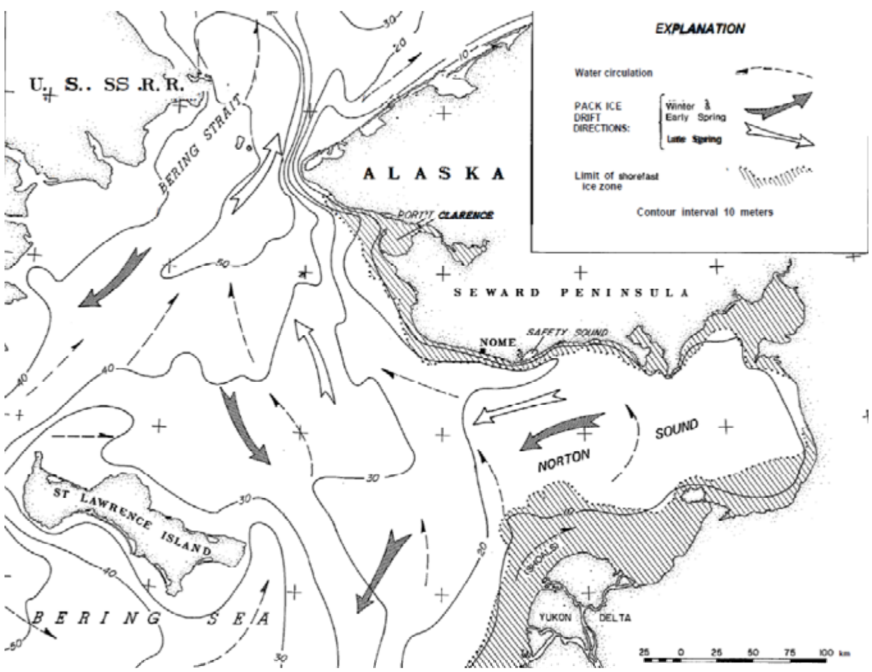


Figure 8: Ice pack movement trends and shore-fast ice zones in the Bering Straits and Norton Sound (Thor & Nelson, 1980). Shore-fast ice zones are shaded in gray.

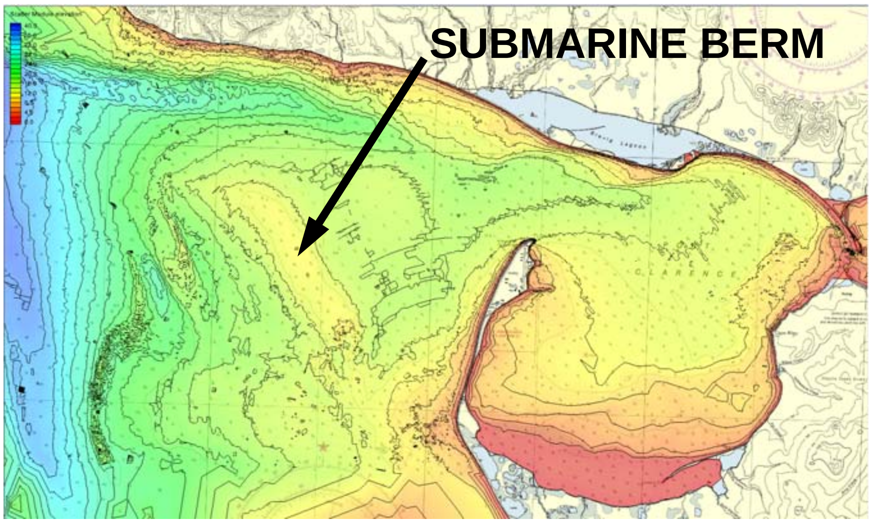


Figure 9: Detail of NOAA Chart 16204 colorized with digital NOS data. Shaded contours are at 2 meter intervals. The northwest-southeast submarine berm has a crest elevation of -10 meters MLLW. This location roughly correlates to the extent of shore-fast ice around Port Clarence.

5. POINT SPENCER

Point Spencer is located on a 7.5-mile-long littoral spit at the western edge of Port Clearance. The site is at the northern tip of the spit, which acts as a 1-mile-wide barrier island connected to the mainland by an 8,000-foot-long spit to the south. This spit overtops during fall storm events, however the site remains connected to the mainland after water levels recede. The point is relatively flat with maximum elevations approximately 10 feet above sea level. The site was used as a LORAN-C Station from 1961 to 2010.

5.1 Existing Facilities

Point Spencer currently has a 4,497-foot-long runway with a 120-foot-wide asphalt surface. The airfield extends an additional 4,000 feet to the north with a gravel surface. The structures supporting the LORAN-C station still exist at the site; however, the 1,350-foot-high transmitter tower was demolished in 2010. Support structures include a transmitter control building, a power house, a dormitory capable of housing 40 personnel with dining hall, and a vehicle maintenance building. The buildings are all connected through a central corridor to allow the station crew to move from one building to the next without going outside. The facility includes 420,000 gallons of fuel storage. The pipelines from the storage tanks to the fuel header are considered to be unusable.

All equipment required to operate the LORAN-C transmitter and maintain a station crew has been removed from the site. The generators are still in place and functional but are reported to be near the end of their usable life. The rest of the facilities have been decommissioned and are maintained at a level of readiness that requires facilities to be usable within 6 months of a decision to reactivate.

A U.S. Coast Guard officer reported that a major drawback to the site was the lack of a potable water source. The station crew obtained water from a melt pond that was pumped into a water storage tank and held over the winter. Occasionally, the melt water pond would become contaminated with seawater during coastal storms. The buildings are reported to be 3 to 4 feet above the highest observed flood elevations at the site.

5.2 Waves and Currents

5.2.1 Currents

Currents have not been measured at the Point Spencer site. Tidal fluctuations cause a current to form along the eastern shore of Point Spencer, especially on the ebb tide as water in Port Clarence is funneled through the outlet. This is evidenced by a shallow channel that runs along the toe of the beach. The speed of these currents is not known.

5.2.2 Wave Climate

Waves in Port Clarence are primarily generated by local winds. Wind data for this analysis was extracted from a catalogued data set compiled to evaluate wind power potential at Point Spencer. The data is based on hourly wind speed measurements from 1973 to 1990 recorded at ICAO Station PAPC. The NCDC data set number is 701190. There is a data gap in the record from 1979 to 1982 where little or no data was recorded. These years were excluded from the analysis. The data represents a 2-minute average wind speed recorded every hour over a 14 year period of record. Peak annual wind speeds were used for this analysis (Table 5).

Table 5: Peak annual wind speeds at Point Spencer from 1973 to 1990.

Date	Wind Speed (knots)
February 3, 1973	64.9
August 20, 1975	45.1
September 25, 1975	59.8
March 17, 1976	44.9
November 25, 1977	49.9
March 5, 1978	44.9
December 10, 1983	45.8
July 30, 1984	40.0
January 23, 1985	42.0
September 5, 1986	31.1
December 23, 1987	45.1
February 4, 1988	49.9
February 3, 1989	49.9

March 29, 1990	34.0

Frequency of occurrence relationships for wind speed were developed using the Gumbel distribution. The distribution was solved with graphical methods using the plotting position formula:

$$F(x_m) = \frac{m - 0.44}{N + 0.12}$$

It should be noted that most of the high wind events occurred between December 1 and April 30 when the port was iced in and not open for vessel traffic. The cumulative distribution function of the data set was modeled as:

$$F(x) = e^{-e^{-\left[\frac{x-42.0773}{7.5815}\right]}}$$

Using this distribution, the 2 percent annual exceedance probability(AEP) wind speed was calculated to be 72.2 knots (37.1 meters per second). This wind speed was used to calculate locally generated wave heights. The highest recorded wind event in the period of record (64.9 knots) produced a return interval of 19.4 years with this distribution, which is reasonable for the period of record. It should be noted that estimating an event with a return period of 50 years using this dataset is problematic due to the short period of record. In the design phase of the project, a more detailed analysis of weather systems over the site should be performed to estimate site wind speeds over a longer period of record prior to defining a design wind speed.

Wind direction was analyzed by the Alaska Energy Authority (AEA) in 2005 as part of their station summary for the site. The frequency distribution rose shows that winds primarily come from the northeast and the speed distribution rose shows that high speed winds normally come from the north and south. The wind speed distribution shows winds of 5 meters per second from the east and 12 meters per second from the north (Figure 10). To approximate this distribution, the 2 percent AEP wind was reduced to 65 percent of the Gumbel distribution value for east winds, which is 24.1 meters per second. Wind speed in directions between north and east were interpolated based on arc deflection from the north (Figure 11).

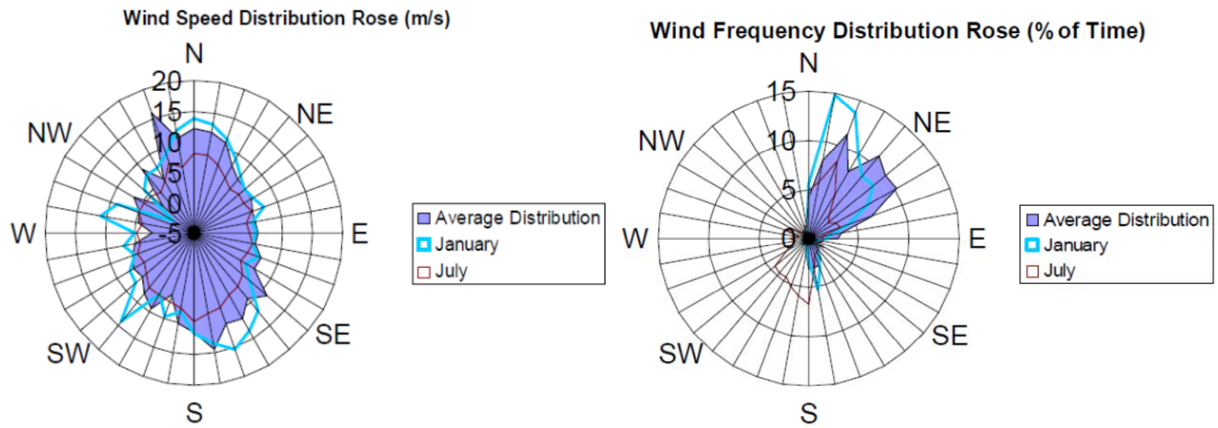


Figure 10: Wind speed distribution and wind frequency distribution for Port Clarence (AEA, 2005)

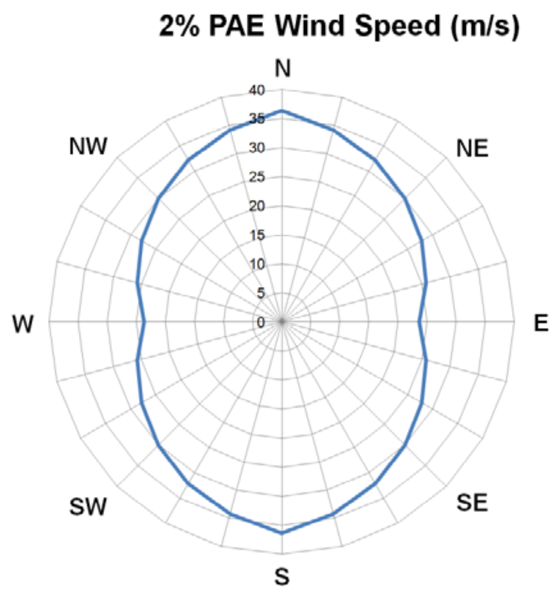


Figure 11: Modeled 2% AEP wind speed distribution at Point Spencer

Local waves were assumed to be limited by fetch and were estimated using the methodology prescribed in EM 1110-2-1100 Part II Chapter 2. Fetch lengths were determined using guidance from the Shore Protection Manual (1984), which prescribes averaging the distance of nine radials centered on the principal fetch direction with 3-degree angles of deflection between radials. Principal fetch directions were determined at 15-degree intervals, and wave heights were determined for each direction (Table 6). It should be noted that the southern fetches include a significant distance over shallow water where bars above +0 feet MLLW are charted (Figure 12).

Waves in the Bering Sea primarily come from the south and southwest. During storm events, long-period waves generally travel to Port Clarence from the south or southwest and diffract around Point Spencer into Port Clarence. The effects of long-period waves on Port Clarence were studied with an STWAVE model, which is described in section 8.2 of this appendix.

5.2.3 Hydrology

Point Spencer has no streams, but there are many small lakes at the south end of the site between the runway and Port Clearance. Surface runoff at the site is all locally generated and can be routed away from project facilities by constructing roadways and storage areas with proper grades and ditches.

5.2.4 Coastal Flooding

Regular coastal flooding of Point Spencer has been reported from Coast Guard personnel familiar with the history of the LORAN-C station. The flooding is reported to inundate the entire spit except for the immediate area surrounding the buildings of the station ant the runway. Flooding was reported to occur within 4 feet of the buildings of the LORAN-C station. The frequency of occurrence of inundation depths on the point is not known at this time and would require a site survey to compare against predicted water levels for Port Clarence.

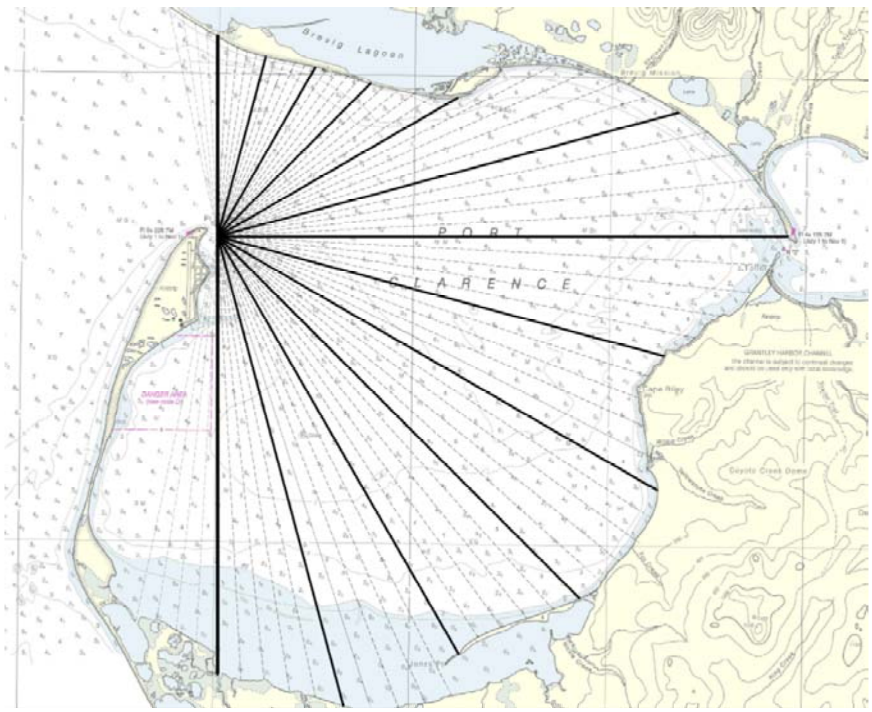


Figure 12: Fetch determination for the Point Spencer site. The blue shaded area in the southern portion of Port Clarence includes several bars which would be above +0 feet MLLW.

Table 6: Fetch-Limited waves at Point Spencer using the 2% AEP wind distribution model.

Direction	Fetch	H_{m0}	T_p
	(m)	(m)	(sec)
0	8085	2.2	4.0
15	7497	1.9	3.8
30	7804	1.8	3.7
45	8766	1.8	3.8
60	12761	1.9	4.1
75	18796	2.1	4.6
90	21085	2.0	4.6
105	19446	2.2	4.6
120	19540	2.4	4.8
135	19987	2.7	5.0
150	19648	2.9	5.1
165	19075	3.1	5.2
180	16930	3.1	5.1

6. CAPE RILEY

6.1 Existing Facilities

The Cape Riley area currently has no upland or sea-side developments, or permanent human presence. The site is completely undisturbed.

6.2 Waves and Currents

6.2.1 Currents

Currents have not been measured at the Cape Riley site. Astronomical tidal fluctuations would cause minor low velocity currents within Port Clarence. Winds and waves events would be responsible for all significant currents in the area. The currents generated by wind and wave events would be dependent on the severity, duration, and directions of the events.

6.2.2 Wave Climate

Wave conditions at Cape Riley were estimated with an STWAVE model using the same data sources as noted for Point Spencer wave estimation. Wind speeds and frequencies used in this analysis were those used for the Cape Spencer analysis.

Waves impacting the Cape Riley site in Port Clarence are primarily generated by local winds. Wind data for this analysis was extracted from a catalogued data set compiled to evaluate wind power potential at Point Spencer. The data is based on hourly wind speed measurements from 1973 to 1990 recorded at ICAO Station PAPC. It should be noted that most of these events occurred between December 1 and April 30 when the port was iced in and not open for vessel traffic.

Local waves were assumed to be fetch limited and were estimated using the methodology prescribed in EM 1110-2-1100 Part II Chapter 2. Fetch lengths were determined using guidance from the Shore Protection Manual (1984), which prescribes averaging the distance of nine radials centered on the principal fetch direction. Principal fetch directions were determined at 22.5-degree intervals, and wave heights were determined for each direction (Table 7).

Long-period waves in the Bering Sea primarily come from the south and southwest. During storm events, long-period waves generally travel to Port Clarence from the south or southwest and diffract around Point Spencer into Port Clarence. The effects of long-period waves on the Cape Riley site were studied with an STWAVE model, which is described in section 8.2 of this appendix. The design wave determined from the STWAVE analysis has a significant wave height of 8.5 feet and approaches the site at an angle of 285 degrees.

Table 7: Fetch-Limited waves at Point Spencer using the 2% AEP wind distribution model.

Direction	Fetch	H_s	T_p
	(mi)	(ft)	(sec)
NNW	11.4	7.4	4.4
NW	18.6	7.9	4.8
WNW	14.3	5.5	4.1
W	13.5	4.1	3.7
WSW	10.9	4.9	3.7
SW	4.8	3.3	3.1
SSW	1.3	2.5	2.1

6.3 Hydrology

The Cape Riley site is located to the south of the estuary at the confluence of Willow Creek and Yellowstone Creek. The access road runs near the southern bank of Yellowstone Creek. The harbor and dock are adjacent to an unnamed stream between Yellowstone Creek and Wild Goose Creek. All of these streams are ungaged. Probabilistic peak stream flows on these creeks were estimated using USGS regional Regression equations. The equations for the northwest region of Alaska are power functions of basin area with coefficients determined to best fit the probabilistic flows of gaged streams in the region. No surveyed cross sections of the creek channels are available, so it is not possible to relate probabilistic flow to water elevations at this time.

Table 8: peak stream flows on streams near the Cape Riley site.

Stream	Peak Flow (cfs)			
	50% AEP	4% AEP	2% AEP	1% AEP
Willow Creek	173	585	672	758
Yellowstone Creek	156	530	609	687
Unnamed Creek	7	27	31	36
Wild Goose Creek	29	105	123	140
Fox Creek	65	232	269	304

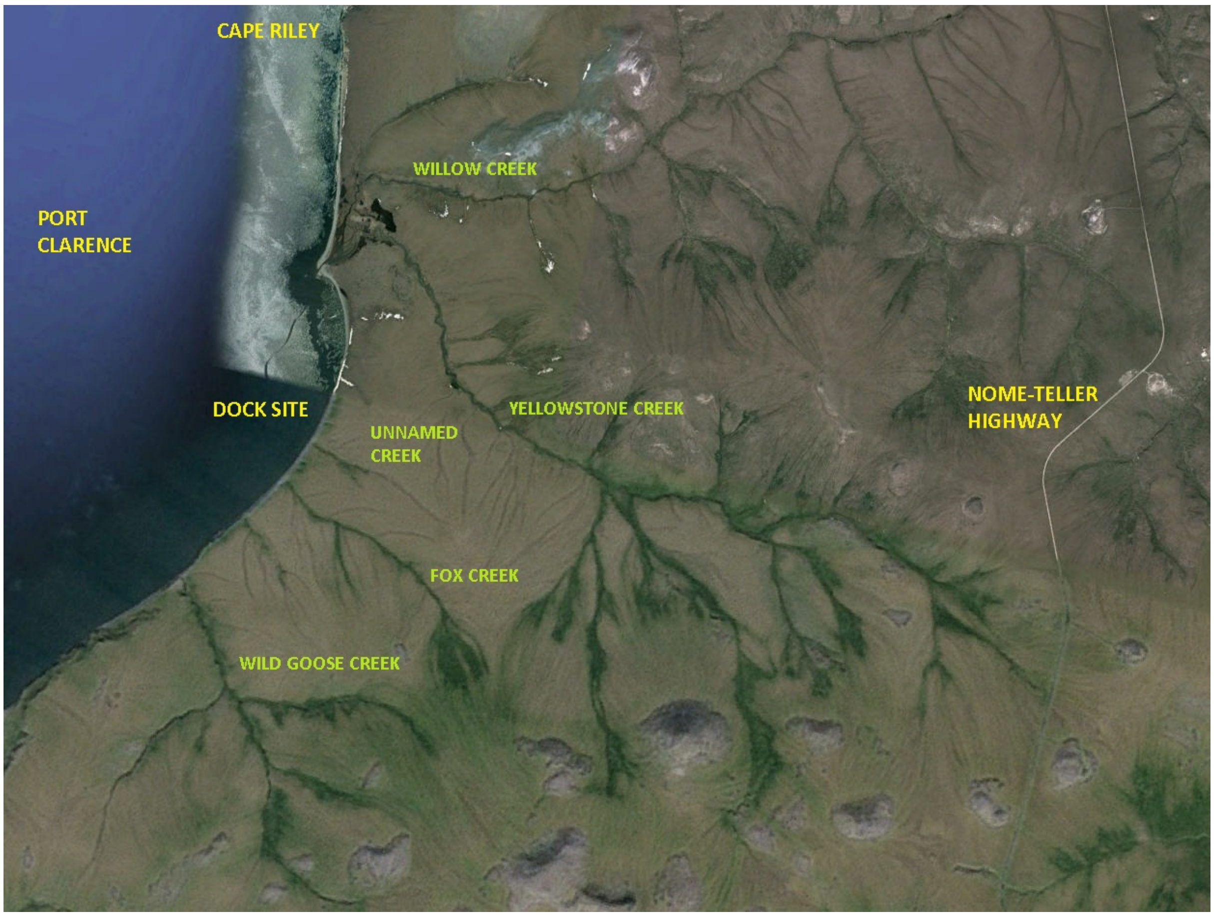


Figure 13: Cape Riley Area Streams

7. DESIGN CRITERIA

7.1 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 oil and gas exploration, mineral resource extraction, and search and rescue. Supporting activities for these missions include the delivery of fuel and supplies to vessels expected to spend extended periods of time in the arctic. Characteristic vessels have been identified to provide the minimum design requirements for port facilities.

7.1.1 Oil and Gas Exploration

Support vessels for oil and gas exploration include the following: general cargo and spill response vessels (length 420 feet, draft 18 feet), up to 30 lighterage barges (length 250 feet, draft 18 feet), and up to 10 offshore support vessels (length 380 feet, draft 28 feet). Heavy lift operations use barges up to 400 feet long pulled by ocean class tugs. In a port, the combined beam of the barge plus the tug on hip can be up to 175 feet for these operations.

7.1.2 Resource Extraction

Resource extraction is expected to use a lightering operation to transfer mined materials to deep draft bulk carriers moored several miles from the dock in the naturally deep portion of Port Clarence. A lightering tug and barge would make multiple trips to load the bulk carrier. The lightering barge dimensions are length 150 feet, beam 50 feet, draft 5 feet. Red Dog Mine uses a similar operation.

7.1.3 Search and Rescue

For the purpose of this study, the arctic search and rescue mission is assumed to require a polar class icebreaker. The United States operates three icebreakers capable of this mission. The Canadian Coast Guard also operates a fleet of polar class icebreakers. Other nations also operate icebreakers in the region. The following vessels have been identified as potential users of port facilities in Port Clarence and Nome areas: USCGC Healy (length 420 feet, beam 82 feet, draft 29 feet), USCGC Polar Star (length 399 feet, draft 28 feet), and USCGC Spar (length 225 feet, draft 13 feet).

7.1.4 Support

Support vessels deliver fuel and supplies to staging areas and vessels at sea. The characteristic vessel identified for fuel delivery is tanker (length 417 feet, beam 67 feet, draft 29 feet, light-loaded draft 17 feet). For freight and supply, a barge (length 400 feet, draft 14 feet) was identified. Various tugs ranging in length from 74 feet to 200 feet with drafts of 12 feet to 20 feet were included in the array of vessels to assist tankers, barges, and other port operations. Two landing craft would also provide support (lengths 180 and 134 feet, drafts 4 and 7 feet, respectively).

7.1.5 Research

Research includes the hydrographic survey vessel “Fairweather” operated by the National Oceanic and Atmospheric Administration (NOAA). Its characteristics are identified as length 231 feet, and draft 16 feet. In addition, the National Science Foundation’s vessel Sikuliaq with length of 261 feet, and draft of 19 feet is included in the fleet spectrum.

7.2 Allowable Wave Heights

7.2.1 Port of Nome

For the Port of Nome alternatives, the proposed causeway extension was positioned to reduce incident wave heights from the southwest and south by more than 50 percent in the outer maneuvering area. Substantially protected moorage for the line haul fuel barge and/or the icebreaker vessel would be provided during their periodic use of the caisson dock facilities. Progressively smaller wave heights would be expected in the inner maneuvering area between the causeway and main breakwater. Improvements to existing wave protection at the sheet pile docks would be realized.

During periods of southeasterly wave exposure, the outer maneuvering area would still be exposed to somewhat reduced wave heights as a result of diffraction around the head of the causeway extension. However, with the short-term transient usage of the caisson dock and low frequency incidence of southeasterly wave conditions, port operations would not be significantly impacted. The inner maneuvering area would benefit from nearly full protection from southeasterly waves due to the diffraction effects of the extended causeway in conjunction with the existing or re-aligned main breakwater. Under storm surge conditions from the southeast, wave overtopping of the existing main breakwater would still be expected to occur due to its relatively low crest elevation (+14 feet MLLW). However, often during storms that generate high water levels, winds shift from southeasterly to southerly. As this shift progresses, the wave protection effects of the extended causeway would be accentuated.

Due to the anticipated periodic use of the outer maneuvering area and the very large design vessel that would use the proposed caisson dock facilities, wave height criteria have been established accordingly. In general, wave heights of less than 3.3 feet (approximately 1 meter) in the maneuvering area for large vessels has previously been applied during the 1998 Nome Navigation Improvements project as the criteria for designing wave protection structures. For this study, an allowable wave height of 3.3 feet during incident wave conditions from the southwest and south was selected for design of the outer channel and maneuvering area. Periodically, wave heights may exceed this value, particularly when the wave is oriented more southeasterly; however, vessel operations at the new caisson dock facilities would not be significantly delayed or impacted. For the inner maneuvering area at the sheet piles docks, wave heights of 3.3 feet were also selected for design; however, due to the enhanced wave protection provided by the causeway extension, estimated wave heights would be less than 1 foot for the majority of the time. During storm events with waves from due southeast, wave heights in the inner maneuvering area would be between 1 and 3.3 feet.

7.2.2 Point Spencer

The causeway and breakwater at Point Spencer was aligned to provide a dockside wave height of 2 feet for all locally generated wave conditions. Diffraction analysis of the design waves was performed around the nose of the breakwater to ensure that dockside wave conditions were satisfied up to two beam widths out from the dock face.

For long-period waves, the dockside wave height was allowed to be up to 3.3 feet for waves entering Port Clarence directly from the west. Due to the way this event was modeled, it is assumed that an event causing this condition has a probability of occurrence far less than 2 percent per year. Typical storm conditions modeled using a southwest wave approach indicate very little influence of long-period waves in the harbor area due to the protection provided by the natural shoreline.

Wave heights in the turning basin are highly dependent upon the approach direction of the wave. The site is naturally protected from the influence of long-period waves; however, locally generated waves approaching from the north may allow waves up to 7 feet on the west side of the turning basin. The average annual distribution of wind indicates that up to 10 percent of wind at the site would come from a direction that could cause this; however, the combined recurrence of high speed wind blowing from this direction is assumed to be infrequent during ice free months. When the area is covered in ice, waves are assumed to be negligible.

7.2.3 Cape Riley

The breakwater for the Cape Riley site was sized and aligned to provide a dockside wave height of 3 feet or less for the 50-year event (2% annual probability of occurrence) locally generated and long-period wave conditions. Diffraction analysis of the design waves was performed around the nose of the breakwater to ensure that dockside wave conditions were satisfied at the dock face.

7.3 Channel and Basin Widths and Depths

7.3.1 Port of Nome

The entrance channel width requirements were determined by criteria given in EM 1110-2-1613 (USACE 2006). For a two-way channel with tidal current velocities of 1.5 to 3.0 knots, the width should be approximately 6.5 times the beam of the design vessel. For the proposed entrance channel at the Port of Nome alternative, a minimum bottom width of 700 feet would allow adequate maneuverability and clearance on each side of the causeway head and main breakwater head. Such channel width also takes into account the turning radius, wind effects, and wave conditions from the south and southeast. While it is not expected that the channel would frequently be used as a two-way traffic entrance for the design vessel, it is likely that traffic of varying beams would enter or exit the port at the same time periodically over the life of the project. In addition, the Nome alternative is configured with a common entrance channel for both the outer and inner maneuvering areas as well as the small boat harbor. Therefore, multiple vessels of varying beam dimensions and port uses would be expected to generate sufficient traffic to warrant the proposed channel width.

The entrance channel width for navigation into the inner maneuvering area was determined based on the design vessel expected to operate in and adjacent to the sheet pile docks. Multiplying the beam dimension of the freight barge (100 feet) by a factor of 4, the required inner channel width was calculated to be 400 feet. Accounting for wind, traffic, and turning movement, a width of 450 feet was determined appropriate for Alternatives 1A and 1B and coincides with the existing width between the causeway and the head of the main breakwater. Similarly, for Alternative 1C an entrance channel width for the inner maneuvering area of 650 feet was determined taking into account two-way traffic for freight barges combined with deeper draft support vessels that will use the inner dock area.

Both outer and inner maneuvering areas would be used for mooring operations at the proposed and existing dock facilities. The proposed entrance channel widths account for such use as well.

Typically for deep draft navigation projects, physical modeling and ship simulator studies are required for channel design. Also, field data collection of ship maneuvering and wave motion is warranted. Due to schedule and budget limitations for this study, channel design was conducted using the best available guidance and analytical techniques.

The outer channel depths were determined based on economic evaluations, design vessel draft, vessel motion in waves, squat, tide, safety clearance, advanced maintenance, and dredging tolerance. For the Nome 1B and 1C alternatives, outer channel depths of -35 feet MLLW were calculated. For the Nome 1A alternative, outer channel depths were reduced to -28 feet MLLW to accommodate smaller and light-loaded vessels.

Table 9: Channel depth determination

Entrance channel	Criteria (ft)
Vessel draft	-29
Pitch, Roll, and Heave	3.5
Squat	0
Access at 96.6% of tide stages (based on predicted low tides)	0.5 MLLW
Safety clearance (based on sand/gravel bottom)	2.0
TOTAL	-35.0 MLLW

Table 10: Channel depth determination Nome 1A alternative

Entrance channel	Criteria (ft)
Vessel draft	-22
Pitch, Roll, and Heave	3.5
Squat	0
Access at 96.6% of tide stages (based on predicted low tides)	0.5 MLLW
Safety clearance (based on sand/gravel bottom)	2.0
TOTAL	-28.0 MLLW

Dredging tolerance of 2 feet was assumed for depths of -35 feet MLLW and -28 feet MLLW; therefore, it is anticipated that the construction contract for the deep draft navigation project would specify a required depth of -35 feet MLLW or -28 feet MLLW with a maximum pay line of -37 feet MLLW or -30 feet MLLW respectively. Additional depth to account for advanced maintenance is not proposed for the Nome alternative.

Tidal accessibility of the proposed outer entrance channel depths was based on the information shown in

Table 11, 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 Nome between June 15 and October 15 (Figure 14) and assumed requirements for vessel motions and safety clearances. An entrance channel depth of –35 feet MLLW was determined to be the acceptable channel depth based on percentage of time usable. Costs for construction and economic benefits for the various channel depths were evaluated in the Economic Appendix.

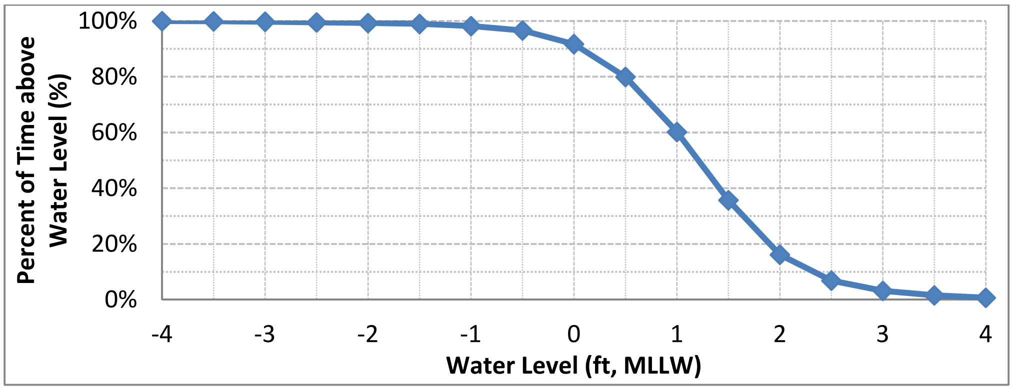


Figure 14: Frequency of water levels at Nome, Alaska based on recorded water levels at Nome from 1992 to 2013 between June 15 and October 15 of each year.

Table 11: Channel depth accessibility for the Port of Nome and Point Spencer

	Entrance Channel Depth (ft MLLW)				
	-37	-36	-35	-34	-33
Tide (ft MLLW)	-2.5	-1.5	-0.5	0.5	1.5
Design Vessel Draft (ft)	-29	-29	-29	-29	-29
Pitch, Roll & Heave (ft)	-3.5	-3.5	-3.5	-3.5	-3.5
Squat (ft)	0	0	0	0	0
Safety Clearance (ft)	-2	-2	-2	-2	-2
% Time Accessible	99.5	99.0	96.6	80.0	35.7

The natural bathymetry offshore of the existing port drops off gradually down to –50 feet MLLW and greater approximately 3,200 lineal feet seaward (south) of the proposed entrance channel. 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.

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.

7.3.2 Point Spencer

Channel and turning basin design for the Point Spencer alternatives was based on guidance from EM 1110-2-1613. The design vessels used to determine channel and basin dimensions are the USCGS Healy with a length of 420 feet and a beam of 82 feet and an articulated tug and barge (ATB) tanker with a length of 605 feet and beam of 78 feet. The Point Spencer Harbor and Point Spencer Dock have differing design characteristics and will be discussed separately.

Since observed tides in the region measured at Nome and Red Dog deviate significantly from predicted tides, it is assumed that wind and atmospheric pressure effects will have a significant effect on water level in Port Clarence. In the absence of measured water level data in Port Clarence, the percent time accessible analysis was directly applied to the Point Spencer alternatives, and a channel and basin depth of -35 feet MLLW was selected.

For the Point Spencer Dock alternative, it is assumed that the dock and basin will have only one user at a time and vessels waiting to use the facility will wait at anchor in Port Clarence if necessary. To minimize wave action, the dock face was placed as close to the shoreline as possible. To minimize dredging quantities for the entrance channel, the approach course parallels the tip of Point Spencer into the basin and makes a 37 degree turn to align the vessel parallel to the dock face. The centerline radius of this turn is 1,350 feet. EM guidance for one way traffic in a channel is 5 times the design vessel beam. The turn requires additional width to allow for vessel motion through the turn. Using the Healy as the design vessel and the PIANC guidelines for a vessel with a length to beam ratio of 5 (the USCGC Healy length to beam ratio is

5.12); this width increase through the turn should be 1.6 times the vessel beam. This produces a channel width of 541 feet. Using the ATB as the design vessel, the width increase is approximately 2.7 times the vessel beam, resulting in a channel width of 600 feet. All of this guidance is valid for wind speeds of 25 knots or less and currents of 1.5 knots or less. Analysis of the wind data suggests that winds above 25 knots occur roughly 10 percent of the time and can be reasonably expected to occur while a vessel is using the channel. Also, this alternative would not significantly alter the tidal currents across the navigation channel and turning basin. While these currents have not been measured, they may exceed the 3-knot threshold while the tide is running out. To be conservative at this stage of the project, a channel width of 700 feet was selected for this alternative. A turning basin width of 1,200 feet and length of 1,800 feet were selected to allow for wind effects and current while maneuvering in the basin.

7.3.3 Cape Riley

The Cape Riley dock was designed for use by a single shallow draft lightering barge. The channel width was determined for one-way traffic from the design vessel. EM 1110-2-1615 guidance provides recommendations for entrance channel width in percent of beam width. For a one-way channel with a bend greater than 40 degrees and vessels with poor maneuverability, the recommended minimum channel width is 610 percent of the vessel beam or 305 feet.

Before or after loading the barge would have to turn around within the basin. EM guidance requires a minimum turning basin width of 1.5 times the design vessel length for locations where currents are less than 1.5 knots and wind speeds are less than 25 knots. The design wind speed exceeds 25 knots so the turning basin diameter was increased to twice the overall length of the tug and barge. Ship simulation studies of the site should be performed during the design phase of the project to ensure that these dimensions are adequate for the intended fleet.

Entrance channel and turning basin depths were calculated using guidance from EM 1110-2-1615, assumed design vessel dimensions, the estimated 2 percent AEP wave conditions, and Nome tidal data. In the absence of measured water level data in Port Clarence, tide data from Nome tide station was used for both tide level and percent accessibility estimates. The design draft of the lightering barge is 5 feet. Vessel response is based on the EM recommendation of one-half the design wave height or 4.35 feet. A safety clearance of 2 feet was used for the sand and gravel bottom conditions of the channel. The dredge tolerance for water depth less than 20 feet is 1 foot.

Basin depth calculation showed that a slightly lesser basin depth was required in the basin versus the entrance channel. The basin depth was increased to that of the entrance channel for purposes of continuity. The increase in basin depth would provide adequate depth at the dock for nearly all expected water levels.

Table 12: Channel depth accessibility for Cape Riley

	Entrance Channel Depth (ft MLLW)				
	-14	-13.5	-13	-12.5	-12
Tide (ft MLLW)	-1.5	-1	-0.5	0	0.5
Design Vessel Draft (ft)	-5	-5	-5	-5	-5
Pitch, Roll & Heave (ft)	-4.5	-4.5	-4.5	-4.5	-4.5
Squat (ft)	0	0	0	0	0
Safety Clearance (ft)	-2	-2	-2	-2	-2
Dredge Tolerance (ft)	-1	-1	-1	-1	-1
% Time Accessible	99.0	98.2	96.6	91.7	80.0

7.4 Circulation

The circulation aspects of the proposed causeway extension at Nome were evaluated based on guidance given in EM 1110-2-1202 (USACE 1987). Tidal variation, storm surge, fresh water input from the Snake River, 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 Nome is relatively small; however, storm surge events pump significant volumes of water into the Snake River estuary. Strong onshore winds and high surf waves are usually associated with storm surges and would represent the larger water circulation component under such conditions. Secondarily, the ebb drawdown of the volume of water in the estuary would also drive circulation.

The aspect ratio (length divided by width) of the existing port at Nome seaward of the sand spit is approximately 3:1. With the causeway extension this would be increased to approximately 3.5:1. With the breaches in both the existing causeway and main breakwater at the -6.5-foot MLLW contour and the entrance channel essentially open to Norton Sound, the outer portion of the Port is not configured as an enclosed harbor. Therefore, planform geometry would not be an integral factor in determining circulation parameters. However, the guidance for harbor circulation can be applied in a general sense for this study. 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. For the Nome alternative, the following values were calculated: $A/a = 125$ and $A/a^{1/2}w = 30$. Area ratios for all alternatives are shown in

Table 13.

Table 13: Indicator aspect ratios for circulation analysis

Alternative	Aspect Ratio	A/a	$A/a^{1/2}w$
Nome	3.5:1	125	30
Point Spencer	1.75:1	200	50
Cape Riley	1.5:1	111	23

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. In addition, the areas of potentially low exchange in the corners of the basin can be checked to ensure that no more than 5 percent of the total areas have exchange coefficients less than 0.15. For the Nome alternative, the northwest and northeast corners are naturally rounded beach areas, and the proposed causeway extension was designed with a radius of 200 feet. The outer maneuvering area would basically be open to Norton Sound to the east.

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 with the Nome causeway extension alternative.

7.5 Ice Forces

7.5.1 Port of Nome Ice Forces

The Port of Nome Design Memorandum by TAMS incorporated an ice engineering investigation. The objectives were to identify, evaluate, and design for ice-sheet and ice-ridge interactions with the proposed port. In addition, ice loadings on the port facilities were estimated. Ice design parameters for the project were determined as follows:

Maximum ice-sheet thickness	
Mobile, mid to late winter	3.0 feet
Landfast, all season, and mobile late spring	4.5 feet
Velocity	
Maximum, moving ice-sheet	2.5 feet per second
Mean	0.7 feet per second
Strength	
Flexural	102 pounds per square inch
Shear	55 pounds per square inch
Loading	110 kips per foot

Ice ride-up accumulations of up to 30 feet on the causeway crest were estimated based on test runs of the physical model.

For this study, the data from the TAMS analysis were applied for the proposed causeway extension and caisson dock design. The existing causeway has performed well since its construction in the mid 1980's, and there have been several significant ice ridge-ups during that period. In the spring of 2001, southeasterly winds created moving ice-sheet conditions that overrode the causeway and built up accumulations of approximately 25 feet in height at the head and south bridge abutment. Also, during the winter of 2005 a large ice pressure ridge formed at the entrance between the causeway head and main breakwater from moving ice-sheet forces. This feature was approximately 30 feet in height above the MLLW elevation and likely was grounded on the seafloor at -25 feet MLLW. A similar ice pressure ridge formation occurred during the winter of 2012 although its magnitude was not a great. Since the proposed causeway extension and caisson dock structure are similar in design to the original port project, it is expected that they will perform equally as well. The "A5" armor stone was shown to be stable in the physical model under design ice loading conditions.

7.5.2 Point Spencer and Cape Riley Ice Forces

Both sites in Port Clarence are protected from movement of large sheets of ice by the spit and island along the western shoreline, shallow bathymetry to the southwest of the entrance, and shore-fast ice conditions during the winter. It is assumed that there is insufficient distance within Port Clarence to develop significant ice forces at the shorelines. Ice forces were not used as a design parameter for these sites.

7.6 Life-Cycle Causeway Extension/Breakwater Design

7.6.1 Port of Nome Life-Cycle Design

Armor stone for the proposed causeway extension at Nome was sized using the 50-year design wave and ice forces expected to impinge on 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 25-year design is 11.7 tons, 50-year design is 22 tons, and for a 100-year design is 26.8 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. 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 or trucked from the local quarry at Cape Nome to the project location. If the construction contractor selected a quarry other than Cape Nome as the rock source, the rock would be barged to the site for placement. The Cape Nome quarry in the project vicinity has the capacity to produce armor stone for either a 25-year event or a 50-year event. Using the 25-year design, it is estimated that overall cost savings throughout the project design life would not be realized due to higher operations and maintenance replacement costs. Replacement costs are estimated to be relatively high because the project location is relatively remote and mobilization costs are substantial. A 75 or 100-year design would reduce the frequency and magnitude of needed maintenance. 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.

7.6.2 Point Spencer and Cape Riley Life-Cycle Design

Separate life-cycle design analysis of the Point Spencer and Cape Riley alternatives were not performed. For the sake of comparison, the same damage ratios assumed for the Port of Nome were applied to the Point Spencer and Cape Riley breakwaters.

7.7 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 for Nome and Cape Riley 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. Side slopes at Point Spencer were designed on a 5H:1V slope to provide a more gradual transition from existing grade to dredged grade because high speed currents appear to form around Point Spencer.

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.

Dredging tolerances vary with depth. For areas where -28 feet MLLW and -35 feet MLLW required depth is specified, a dredging tolerance of 2 feet was used. For areas where -22 feet MLLW required depth is specified, a tolerance of 1 foot was used.

8. MODEL STUDIES

8.1 Physical Modeling

For purposes of this study, additional physical modeling for wave and ice analysis was beyond the scope, budget, and schedule. However, the results of previous modeling have been applied in general toward the proposed causeway extension for the Port of Nome alternative.

As part of the original Port of Nome causeway design by TAMS in 1982, a physical model of ice impacts was conducted at the Iowa Institute of Hydraulic Research (IIHR). The purpose of the ice engineering physical model was the following: (1) to study the ice impingement on the causeway and its head, (2) estimate ice loadings on the causeway and other port improvements, (3) determine armor stone stability under ice loading conditions, (4) determine the likely frequency of ice overtopping of the causeway and dock facilities, and (5) design ice protection and management strategies.

The model was constructed in two components. First, a 1:20 scale side slope 2-dimensional model was constructed to test ice impacts to an armored causeway section for stability. Second, a 3-dimensional 1:30 scale breach model was constructed to evaluate moving ice sheet impacts on the causeway breach. Ice movement in the vicinity of the breach was investigated for both an oblique 45 degree direction and a parallel direction of impact.

Results of the ice modeling showed three modes of ice failure: (1) flexural, (2) buckling, and (3) crushing. Both the causeway and breach armor stone requirements for stability were determined in the model. In addition, the causeway and breach crest elevations were established to accommodate ice overtopping events without sustaining damage.

A 3-dimensional physical model study was conducted at ERDC in 1998 for the navigation improvements project at Nome. The purpose of the physical model was the following: (1) to study wave, current, and shoaling conditions at the existing harbor and with the proposed navigation improvements, (2) determine the impacts the proposed improvements would have on wave-induced current patterns and magnitudes, sediment transport, and wave conditions in the navigation channel, (3) optimize the length and alignment of the new breakwater, (4) optimize the length and alignment of the new causeway extension (spur breakwater), and (5) develop plans for addressing design wave and ice conditions as necessary.

The model was constructed to an undistorted linear scale of 1:90 (model to prototype). It reproduced waves, wave-induced currents, and sediment transport patterns along several thousand feet of shoreline with and without improvement components. The wave generator was designed to produce waves of various heights, periods, and directions in order to model the range of conditions anticipated.

Results of the modeling showed that major decreases in wave activity could be achieved with the proposed improvements. Both the spur and main breakwaters were shown to be required in order to reduce wave heights and currents to criteria levels and to control sedimentation within the navigation channel.

8.2 Numerical Modeling

Previous studies of the wave climate at Nome were performed to define the design parameters of the navigation improvements at the Port of Nome. Deep water wave conditions have been hindcast at offshore points along the entire coast of Alaska in the Alaska Wave Information Study (WIS). Numerical modeling using WIS data from points to the south and west of Port Clarence was performed to simulate the transformation of deep water waves from the Bering Sea as they enter Port Clarence and travel to the project locations at Point Spencer and Cape Riley. Wave modeling was performed over a coarse grid covering all of Port Clarence to reveal general trends in wave transformation, and then finer grids were established around each site of interest to define the design wave heights. All models were run using the STWAVE module of Surface-Water Modeling System (SMS) version 10.1.4 in the half-plane mode. All modeling was performed in SI units over a UTM projection of the bathymetric data. STWAVE was run using propagation mode only as it was found that including source terms (wind) produces smaller wave heights.

8.2.1 Elevation Data

Bathymetric data was collected from the National Oceanographic 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 H02604, H07837, H07838, and H07840. H02604 was a large area survey conducted in 1902 that covered Norton Sound and Port Clarence. The data from this survey is sparse; however, in the southern portions of Port Clarence, this was the only source of digital point data. H07837, H07838, and H07840 were performed in 1950. These surveys covered the approach, entrance, and northern portion of Port Clarence with dense soundings. The digital bathymetry was supplemented with manually digitized soundings and contours from NOAA chart 16204.

Topography of the spit that defines the western edge of Port Clarence was not available; elevations were arbitrarily selected at the airfield and along the coast from Point Spencer to the mainland. Generally, elevations along the coast between Point Spencer and the mountains 5 miles to the southwest were assumed to be 1 meter or 1.5 meters above MLLW. It was assumed that some parts of this coastline would overtop during storm surge events, and this was allowed to occur in the model.

8.2.2 Boundary Wave and Water Level Conditions

All STWAVE models used a still water level of 2.36 meters above MLLW, which corresponds to the 2 percent AEP water level developed by hindcast modeling, which is discussed in section 4.2.3. The STWAVE model was run using the 2 percent AEP significant wave height for deep water waves. Wave boundary conditions were taken from the Alaska Wave Information Study (WIS). The WIS study is a hindcast of hourly wave climate along the western Alaska coast from

1985 to 2009. Storm events from 1954 to 1985 were also hindcast to lengthen the period of record for determining extreme event wave heights. The study was updated to include the years 2010 and 2011 in September 2011, but this data was not available at the time wave modeling for Port Clarence was performed. Ocean wave heights were calculated from the extremal analysis of station 82089, which lies approximately 40 miles due west of Point Spencer. The extremal analysis includes a logarithmic relationship between return period and wave height for each site (Figure 16). The relationship was used to estimate 4 percent, 2 percent, and 1 percent AEP wave heights at this station. Event statistics from the save point show wave periods of 11 to 12 seconds for the top ten modeled events (Figure 16). The 2 percent AEP wave was used as the boundary condition and has a significant wave height of 9.21 meters with a period of 12 seconds.

Wave directions for this station are predominantly from the south (Figure 17). There was an error in the directional plots (wind and wave roses) in the latest update of the Alaska WIS study that resulted in all graphical outputs having a 180 degree error. For the sake of model simplicity, the wave height at the station was assumed to be direction insensitive and boundary waves were assumed to come from the west. This means that the waves considered in the model are not truly 2 percent AEP events since the direction of these waves is not known. Modeling waves from the west increases the wave heights inside Port Clearance above waves coming from the south or southwest and can be considered a conservative assumption. Further study of wind and wave direction at the entrance of Port Clarence will be needed during the design phase to determine the frequency of occurrence relationship of wave heights at the project sites.

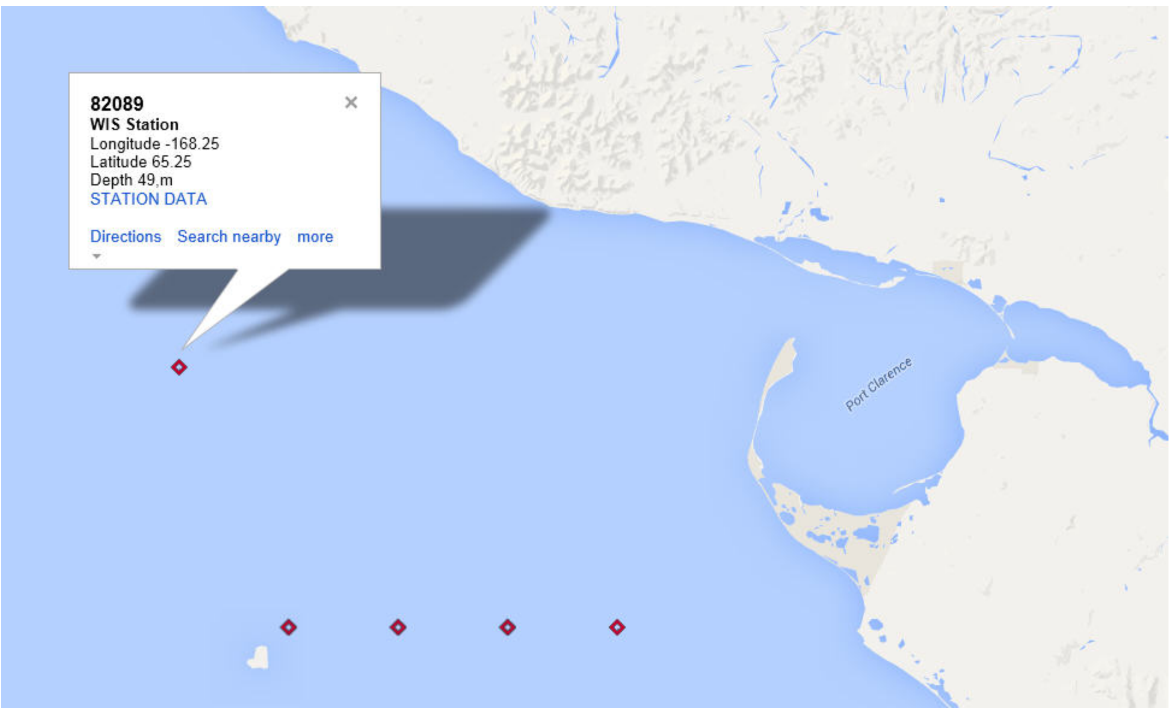


Figure 15: Location of WIS station 82089

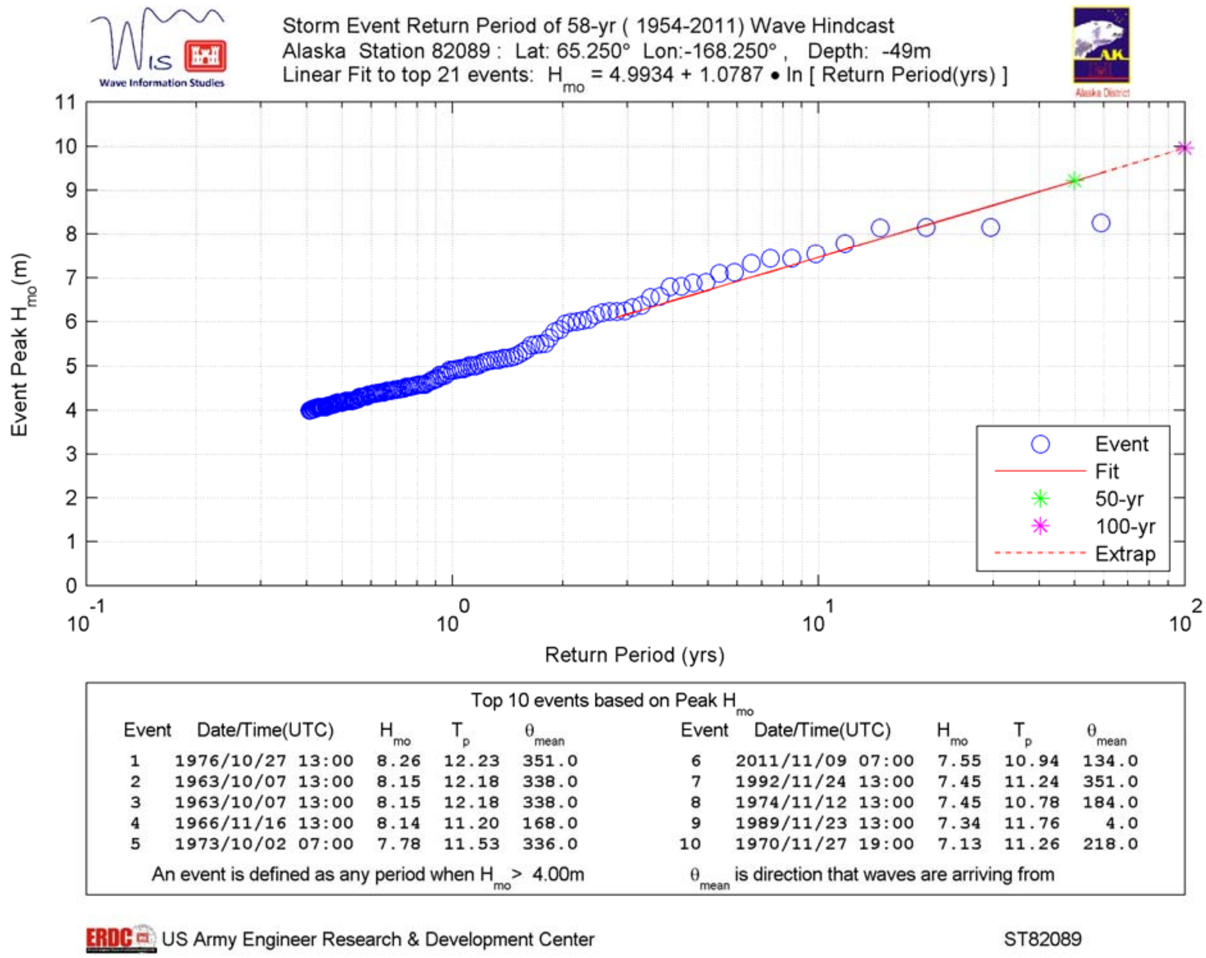


Figure 16: Analysis figure for WIS station 82089

Table 14: Frequency of occurrence relationship for modeled significant wave heights at WIS station 82089.

AEP	H_{mo} (m)	T_p (s)
4%	8.47	12
2%	9.21	12
1%	9.96	12

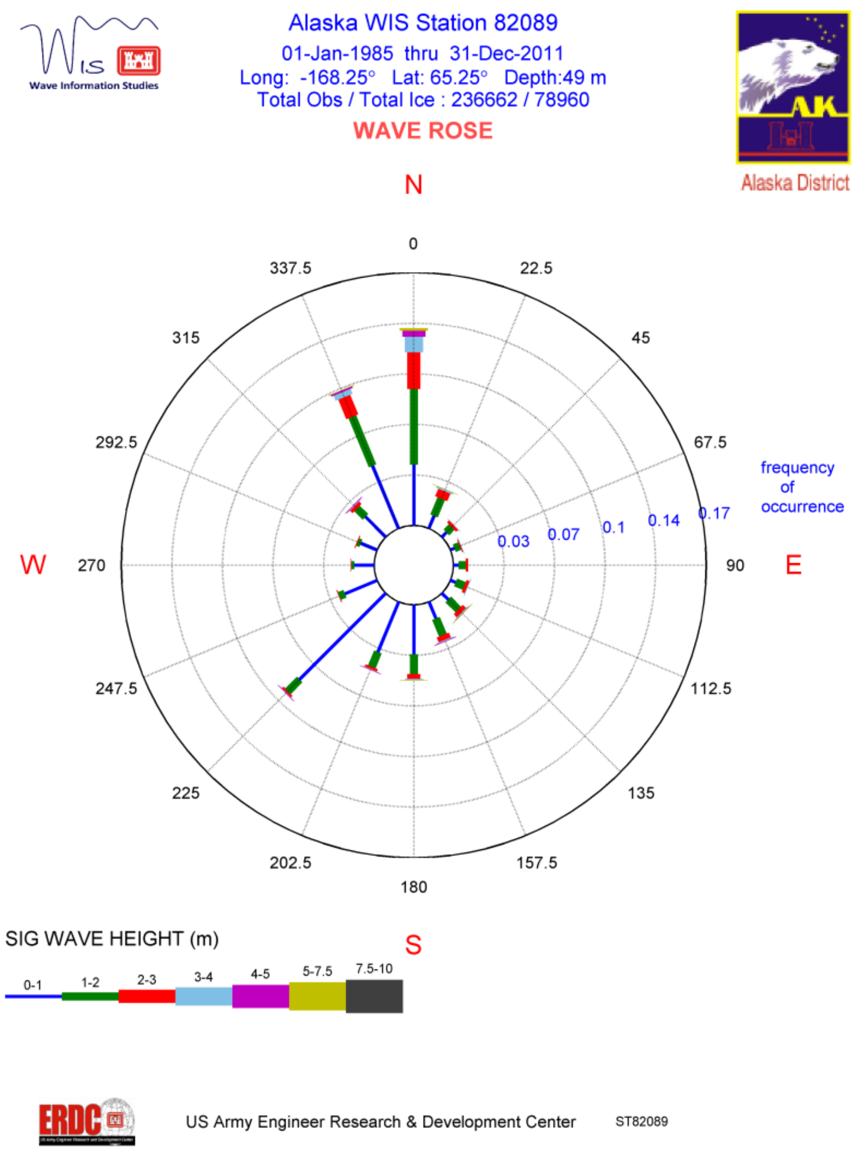


Figure 17: Wave rose for station 82089. A special note for the Alaska WIS project indicates that all directional products including wave roses are plotted with a 180 degree error meaning the predominant wave direction for this site is south, not north.

8.2.3 Port Clarence STWAVE Model

A calibration grid was created from WIS station 82089 to Point Spencer to analyze wave transformation outside Port Clarence. This was done to find a reasonable seaward boundary for grids extending from the Bering Sea to the eastern shore of Port Clarence. It was found that no significant changes in wave height occurred for approximately the first 20 miles. Water in this region is 80 feet deep or greater. The initial Port Clarence grid was drawn from the WIS data point to the eastern shore of the inlet. Due to the size of the area, grid cells were sized at 40 meters by 40 meters. The seaward boundary of this model is approximately 20 miles west of Point Spencer. The preliminary model results produced wave heights of 2.2 meters at the Point

Spencer site and 1.8 meters at the Cape Riley breakwater. The same grid boundaries were then applied to a 20-meter by 20-meter grid to check the sensitivity of the model to grid size. The 20-meter by 20-meter grid found wave heights of 2.2 meters at the Point Spencer site and 2.6 meters at the Cape Riley breakwater. Higher resolution grids for this area could not be run due to the number of grid cells.

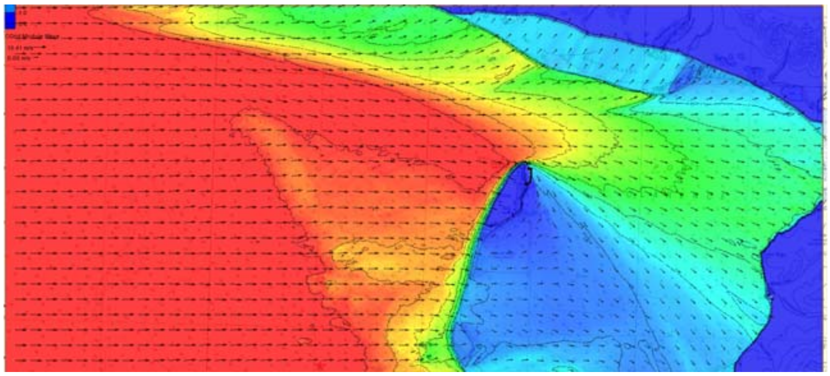


Figure 18: STWAVE 40-meter by 40-meter grid model results using the 2% AEP water level. Wave height contours are in 1-meter intervals. Red areas denote waves above 8 meters.

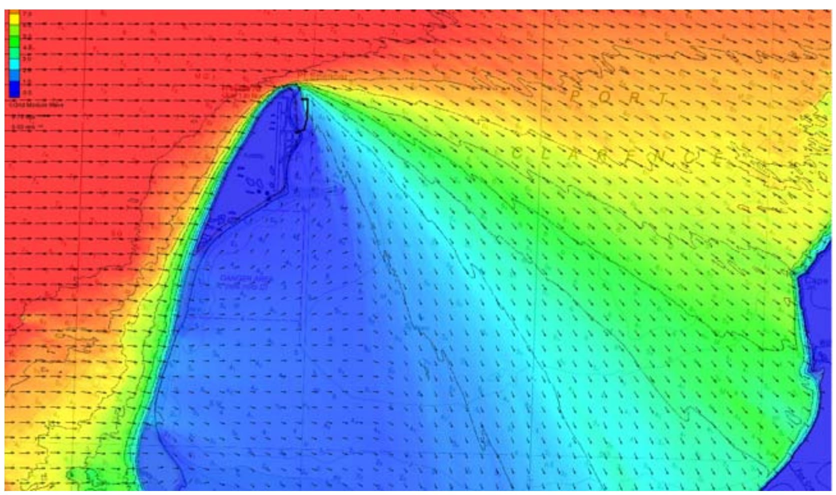


Figure 19: STWAVE 20-meter by 20-meter model results using the 2% AEP water level. Wave height contours are in 1-meter intervals. Red areas denote waves above 8 meters.

Long-period waves usually approach the entrance from the south or southwest. An additional large scale STWAVE model was created to model waves approaching the entrance to Port Clarence from the southwest. Boundary waves were derived from WIS station 82091 to the southeast of Port Clarence, and the 2 percent AEP water level was used. This model was used to establish typical storm conditions in the area and not used to determine design values for breakwater armor stone. Further study of wind and wave direction at Port Clarence will be required during design to establish the typical frequency of occurrence of long-period wave direction at the entrance to establish the design wave.

Table 15: Frequency of occurrence relationship for modeled significant wave heights at WIS station 82091

AEP	H_{mo} (m)	T_p (s)
4%	6.74	11
2%	7.30	11
1%	7.87	11

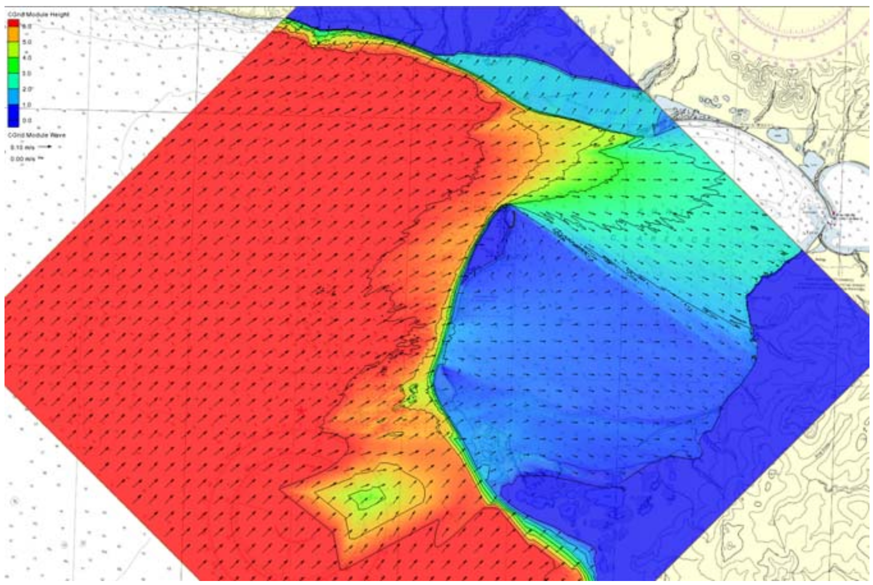


Figure 20: STWAVE 30-meter by 30-meter model results propagating waves from the southwest towards the entrance of Port Clarence. Wave height contours are in 1-meter intervals. Red areas denote waves above 6 meters. Note that the area south of the line between Point Spencer and Yellowstone Creek predicts wave heights of 1 meter or less.

8.2.4 Point Spencer STWAVE Models and Results

A 10-meter by 10-meter model grid was created to determine design wave heights at Point Spencer. A seaward boundary was defined approximately 20 miles west of Point Spencer (at the same location as the larger grids), and the model extended half a mile east of the site. The grid covered a 7,000-meter swath between these boundaries. Results of the 10-meter grid model produced 2.5-meter waves along the site, which are slightly larger than waves produced by the 40 and 20-meter models. Note that these grids were developed under the assumption that a breakwater would be required at the site; wave conditions at the dock face without a breakwater under this scenario are negligible due to the geometry of the shoreline at Point Spencer.

Table 16: STWAVE modeled wave heights at the Point Spencer site.

Grid Size (m x m)	Project Wave Height at outer entrance channel (m)	Project Wave Height at inner entrance channel (m)
40 x 40	2.2	1.2
20 x 20	2.2	1.2
10 x 10	2.5	1.1
30 x 30	0.1	0.1

8.2.5 Cape Riley STWAVE Models

Three small scale grids were generated to evaluate wave transformation of local wind generated waves at the Cape Riley site. The grids were oriented to represent waves coming from the northwest, west-southwest, and southwest. Boundary waves for these grids were taken as the fetch-limited waves calculated in the direction of the grid from the Cape Riley site (Figure 21). Because the site is relatively shallow, it was expected that wave heights would decrease at the breakwater site near the shore. STWAVE model results at the Cape Riley breakwater are in Table 17.

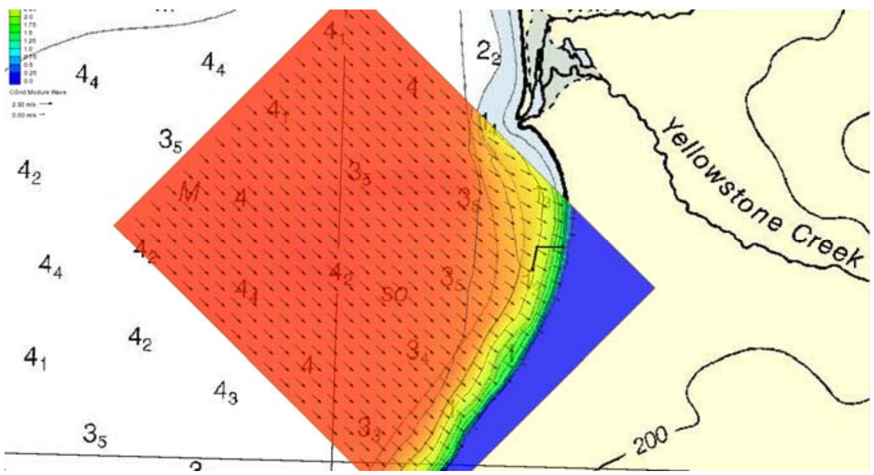


Figure 21: STWAVE 3-meter by 3-meter model results for Cape Riley with waves coming from the northwest. Similar grids were generated for waves coming from the west-southwest and the southwest. Wave height contours are in 0.25-meter intervals. Red areas denote waves above 3 meters. The black line denotes the breakwater alignment.

Table 17: STWAVE results at the Cape Riley site

Grid Direction	Project Wave Height (m)
NW	2.4
WSW	1.5
SW	1.0
W (20 x 20)	2.6

8.3 Point Spencer Unprotected Site Availability

The eastern shoreline of Point Spencer is naturally protected from long-period waves entering Port Clarence. Without protection of a breakwater, the site is exposed to locally generated waves that could potentially impact cargo transfer and mooring. Wind data from the site was analyzed to determine the percent of time that a dock without a breakwater at the site would be available for cargo transfer operations over the period of record. The threshold value for cargo operations was assumed to be 3.3 feet or 1 meter at the dock face. Wave reflection at the dock face was not considered since the moored vessel would reflect the majority of the wave energy before it reaches the dock face.

8.3.1 Wind Analysis

The data set analyzed for site availability was recorded at Point Spencer from 1973 to 1978 and 1983, and consists of wind speed observations and wind direction observations. The data was grouped into 3-hour intervals consisting of an average wind speed and direction over the interval. Wind directions were reported in 10 degree intervals. Fetch lengths calculated for design waves were interpolated to fit this direction array, and threshold wind speeds were found that would produce a wave of 1 meter from each direction. Only winds with an eastern component from 0 degrees (north) to 180 degrees (south) were considered in this analysis. Winds with a western component from 190 degrees to 350 degrees were considered to have no effect on the site. The data set was also filtered for an assumed ice free period of April 1 to December 31 for each year to model an assumed future ice free period where vessels could use the site. The maximum wind speeds for each direction were found and used to calculate the maximum wave height expected at the site over the period of record.

8.3.2 Percent Availability Analysis

The data set was filtered to find events corresponding to wind speeds above the threshold speed for the given direction to find the percent of time the site would experience wave heights too large for transferring cargo from ship to shore. Three work periods for cargo transfer operations were considered for site availability: 3-hour work periods, 24-hour work periods, and daily work periods. For each time window, wind speeds above the threshold speed were considered to be unworkable times. The unworkable times were counted and compared with the overall time in the period of record to determine the percent of time an unprotected facility would be available for cargo transfer operations. The 3-hour interval analysis used the raw data from the set and represents the maximum percent of time the site would have been available over the period of record. The 24-hour interval analysis assumed a vessel would require an uninterrupted 24-hour window at the dock to load and unload cargo. For this analysis, if the period of time between two inoperable periods was less than 24 hours, the entire period of time between the events was considered to be inoperable time. The daily interval analysis assumed that cargo transfer operations would proceed on a day by day basis. For this analysis, any day that experienced any wind speed above the threshold at any time of the day would be considered unavailable. Percent availability of the site for these work periods is shown in Table 18.

Table 18: Percent time availability at Point Spencer April 1 to December 31 based on percent of time wave heights are less than 1 meter at the site.

Time Increment	Percent of Time Available for Cargo Transfer Operations
3-hour work period	98.4 %
24-hour work period	97.9 %
Daily work period	95.7 %

9. ALTERNATIVES

See the Attachments Section at the end of this appendix for plan and section view figures for the alternatives.

9.1 Port of Nome Expansion

A range of alternative designs was considered for deep draft navigation improvements at the existing Port of Nome. A matrix of possible alternatives for consideration was developed in the initial phase of the study that included various configurations of modifications and/or extensions of the existing causeway, main breakwater, and dredging. This phase narrowed down the alternative designs to the basic concept alternative: extension of the causeway offshore with an "L-shaped" segment at the head, dredging an extended entrance channel and outer maneuvering area and inner channel maneuvering area, and construction of new dock facilities to support the fleet. Several minor variations of this basic concept alternative were analyzed and refined to define the final alternatives (1A, 1B, and 1C) selected for Nome. No sites other than the existing port site were explored in detail for consideration.

The alternatives were evaluated using established design guidance given in the appropriate Corps of Engineers Engineering Manuals (EM's) and the Coastal Engineering Manual (CEM). Physical modeling of the alternatives was not included in the scope of this analysis, although previously the physical model study conducted for the 1998 feasibility study and the 1982 ice modeling work prepared for TAMS were used.

Following an evaluation of the wave climate and ice conditions in Norton Sound, it was determined that a rubblemound causeway extension for protection from the southwesterly wave exposure and southeasterly moving ice forces were most appropriate and cost effective. Relatively shallow water depths lend themselves to an economically constructed rubblemound causeway extension for this project.

Vessel traffic conditions, including existing dock and barge operations, were considered in the layout of proposed alternatives. Development of an expanded port at this site would not adversely impact current operations at the City and Westgold Dock areas. Vessels would continue to be able to maneuver and utilize both dock areas and navigate into the existing harbor and coexist with the increased vessel usage in the area.

The current port site has limited uplands; however, the City of Nome has prepared a master plan identifying future uplands development for support of an expanded port. The future uplands development would be sufficient to support the fleet and associated port operations. This site also represents the most practical site for port development due to its existing infrastructure and relative proximity to the City of Nome. The proposed causeway extension would be immediately south of the existing causeway in an area that has natural bottom elevations ranging from –25 feet MLLW to –35 feet MLLW. Such depths in the area of the proposed extension are suitable for cost effective rubblemound causeway construction. The wave climate for the three directions of exposure (southwest, south, and southeast) and ice forces expected to impact the structure are also suitable for cost effective rubblemound causeway construction.   Large armor stone is required for wave protection from the southwest. Several causeway extension alignments were considered and optimized to determine the most effective and least costly alternative at this site. Optimum locations for the proposed dock structures were evaluated for their ability to accommodate the fleet, provide the required wave protection, and maintain sufficient navigation and maneuvering area for vessels. The alternative plans at this site were developed for a 50-year design life. Alternative 1A incorporates the following: a 2,150-foot-long rubblemound causeway extension located south of the existing causeway, a 600-foot- long concrete caisson dock, a 400-foot-long steel sheet pile modified diaphragm dock, a 535-foot- long steel sheet pile modified diaphragm dock, a new outer entrance channel and maneuvering area dredged to -28 feet MLLW with maximum depth of -30 feet MLLW, and an expanded inner maneuvering area dredged to -22 feet MLLW. In addition, the existing utilities such as fuel, water, sewer, and power lines would be extended from their current ends to the new caisson dock at the extended causeway head. The new entrance channel alignment to the port would be oriented with more of an “S-turn” movement around the heads of the new causeway extension and the existing main breakwater and into the inner maneuvering area and navigation channel. This entrance channel configuration is somewhat different from the existing condition but was designed to meet safe navigation criteria under extreme wave and wind conditions. A new navigation marker light would be established at the head of the new causeway extension along with the existing one at the main breakwater head to guide vessels into the port. Alternative 1B includes the same features as Alternative 1A and includes a second concrete caisson dock (450 feet long) located on the inside north-south perimeter of the causeway extension and a new outer entrance channel and maneuvering area dredged to -35 feet MLLW with maximum depth of -37 feet MLLW. Alternative 1C includes the same features as Alternatives 1B, but replaces the existing sheet pile docks in the inner maneuvering area with a new continuous sheet pile dock, an expanded inner maneuvering area dredged to -35 feet MLLW, and a realigned main breakwater head.

9.1.1 Outer Entrance Channel and Maneuvering Area

The new outer entrance channel and maneuvering area for the port would require dredging to -28 feet MLLW for the 1A alternative and to –35 feet MLLW for the 1B and 1C alternatives. The -35 feet MLLW depth is sufficient for the design vessel based on criteria given in Section 7.1 of this appendix. The -28 feet MLLW depth would be sufficient for the smaller vessels of the fleet. The largest vessels of the fleet would only be accommodated while light-loaded. The outer entrance channel would be flared out to daylight at the existing contours and follow the head radius of the extended causeway. The effective channel width would be 700 feet. A total combined outer entrance channel and maneuvering area of approximately 45 acres would be available for the Port of Nome alternatives. This area would accommodate the new caisson dock for alternative 1A and could be appended with an additional caisson dock for Alternative 1B. The inner channel and maneuvering area would have a combined area of 27.2 acres for Alternatives 1A and 1B, and 37.1 acres for Alternative 1C compared with the existing 15.4 acres now available.

9.1.2 Wave Heights

All four alternatives for the Port of Nome site would provide for improved wave protection for the inner channel, maneuvering area, docks, and navigation channel into the harbor. The extended causeway was positioned to reduce incident wave heights from the various directions of exposure to acceptable levels. The maximum wave heights in the maneuvering areas, based on the 50-year design incident waves from three directions (SW, S, and SE), were estimated to be reduced by greater than 50 percent. Progressively smaller wave heights would occur farther into the inner maneuvering area. Estimated wave heights in the outer maneuvering area would be reduced approximately 70 percent when approaching the port from the southwest, approximately 60 percent when approaching from the south, and approximately 50 percent when approaching from the southeast. The outer maneuvering area is not intended to be fully protected from incident wave exposure; however, it will provide for partial protection sufficient to support the proposed port operations at the new caisson dock. The southwest wave is the most severe in terms of wave height, period, and frequency of occurrence and the extended causeway head would fully protect the new caisson dock from wave induced forces.

9.1.3 Circulation

None of the four alternatives would fully enclose the port proper because the proposed causeway extension would be outside and offset from the existing navigation channel. It is estimated that the exchange of water in the new configuration would be similar to that of the existing port during each tide cycle. Since the tide range at Nome is relatively minimal, water exchange due to tidal influence is minor. Wind induced currents and flow from the Snake River are estimated to provide the larger portion of water exchange within the port system. Also, the breaches in the causeway and the main breakwater provide flow paths for wave driven currents and rip currents.

9.1.4 Shoaling

Significant shoaling of the new entrance channel would not be expected since there has historically been no evidence of such in the existing entrance channel. The tip shoal that has been building at the head of the existing causeway has so far not encroached to the east and impacted the navigation channel. It has, however, been a concern. The new causeway extension would provide additional length and orientation that would be expected to further minimize the concern of shoaling in the entrance. In addition, as the buildup of sediments on the beach west of the causeway at the bridge is worked through the system and in conjunction with the existing sediment traps and maintenance dredging, it is estimated that shoaling in the entrance channel would be minimal over the life of the project.

9.1.5 Construction Dredging

Dredging would be required for the Nome alternatives. Dredging quantities and conditions were derived from the most current bathymetry and geotechnical data available. The dredged material would consist of silts, sands, gravel, cobbles, and glacial till. It is anticipated that dredging such material would be difficult but could be performed with mechanical equipment such as a clam shell dredge and would not require drilling and blasting. Construction dredging quantities for the

Port of Nome alternatives are shown in Table 19. Side slopes for the entrance channel and maneuvering area would be dredged to lV:3H. Side slopes would not require slope protection.

Table 19: Construction dredging quantities for Port of Nome Alternatives

Port of Nome Alternative	Outer Channel (cubic yards)	Inner Channel and Maneuvering Area (cubic yards)
1A	153,600	287,400
1B	565,200	287,400
1C	565,200	1,100,800

9.1.6 Dredged Material Disposal

There is an existing offshore dredged material disposal area located at and east of the proposed project in water depths ranging from 0 to -35 feet MLLW. This site was used historically for disposal of maintenance dredged material from the small boat harbor and most recently the material from the Navigation Improvements project in 2005. Dredged material could be disposed of by dump scow barge efficiently using this site. Alternatively, a new offshore disposal site could be designated in deeper water, for example at depths of approximately -50 feet MLLW, to the south of the proposed project. Dredged material could be disposed of by dump scow barge in such a location.

Currently, the maintenance dredged material from the existing channel and harbor at Nome is placed in the near shore area along the beach. This area is approximately 0.5 miles east of the existing main breakwater in front of the existing rock seawall extension west of Front Street. The dredged material for the proposed project could be placed east of this site for purposes of further nourishing the beach in front of and to the east of Nome. Water depths vary from approximately 0 feet MLLW at the beach to -10 feet MLLW offshore. Dredged materials could be placed in the littoral zone in an evenly distributed manner parallel to the beach line progressing in the easterly direction.

Additionally, several possible upland areas within the City of Nome could be used for placement of dredged material for use as fill for site development. Further evaluation of the material within the proposed dredge prism would be required to determine if the material would be suitable for purposes of construction as fill.

9.1.7 Maintenance Dredging

Maintenance dredging is expected to be minimal over the course of the design life of the project. Dredging has not been required in the existing -22-foot MLLW area since its initial construction in 2006. Littoral transport of sediments generally appears to be from west to east under the bridge and into the east sediment trap. The channel cut through the sand spit appears to capture material not deposited in the east sediment trap where it is maintenance dredged annually. An estimated new maintenance dredging requirement is described in section 0 of this appendix.

9.1.8 Causeway and Main Breakwater Extension Design

The positioning of the new causeway extension would create an entrance channel alignment allowing access to the port from the southeast. Maximum depths of water are –33 feet MLLW along the alignment of the causeway extension at the head. Foundation materials would be sand, gravel, and glacial till that would serve as a suitable base for the structure. The existing spur breakwater would be demolished and the causeway head would be removed for tie-in of the new causeway extension.

Methods described in the CEM using Hudson's equation were used to determine armor stone sizes for the new causeway extension, essentially using the same design as the existing causeway by TAMS. Stone size for the outer armor layer was determined using the significant wave height established previously, along with a sea-side side slope of 2H:1V and harbor-side slope of 1.5H:1V, and a K_d value of 12 for selective placement and a breaking wave condition. A stone specific gravity of 2.65 was assumed for the calculations. Armor stone (A1 rock) with a range of sizes from 27-ton maximum weight, 22-ton average weight to 19-ton minimum weight would be used on the seaward face of the causeway extension. Secondary stone (B2 rock) would range from 7,500-pound maximum weight, 4,000-pound average weight to 3,000-pound minimum weight. Core stone (C1 rock) would range from 1,000-pound maximum weight, 300-pound average weight to 150-pound minimum weight. Filter stone (D rock) would be well graded gravel with a gradation of maximum 5 percent greater than 6 inches, and maximum of 15 percent passing the ¾-inch sieve. Sea-side armor stone thickness would be 15 feet, and secondary stone thickness would be 7 feet. For the harbor side, armor stone (A5 rock) with a range of sizes from 10-ton maximum weight, 8-ton average weight to 6-ton minimum weight would be used on the inside face of the causeway extension. Secondary stone (B3 rock) would range from 3,600-pound maximum weight, 1,600-pound average weight to 1,000-pound minimum weight. Core stone (C2 rock) would range from 150-pound maximum weight, 80-pound average weight to 15-pound minimum weight. "F" fill material would be classified fill 3-inch maximum and non-frost-susceptible. "E" fill material would be unclassified fill and could be derived from the various gold dredge tailings sites in Nome. All the armor stone would be placed "selectively" with the long axis of each stone oriented perpendicular to the side slope and with maximum contact with each surrounding stone. The A1 rock would extend down the sea-side slope to a 6-foot dredged-in B2 rock buttress configuration at the base of the causeway extension. This provides for toe stability by anchoring the lower reaches of the side slope into the *in-situ* seafloor material and provides protection from potential scour. The A5 rock is sized to be stable under moving ice pack and ice run-up conditions.

The crest elevation for the sea side of the causeway extension was set to match that for the existing causeway at +28 feet MLLW. It was determined by considering wave run-up, storm surge, and extreme high tides to provide for a non-overtopping structure. Projected sea level rise was originally not taken into account during the initial design of the causeway but has since been evaluated in the 1998 feasibility study and this study. For the harbor side of the causeway extension, the crest elevation was set at +15.5 feet MLLW, again essentially matching the existing causeway. A roadway driving surface width of 30 feet was selected for vehicle access to the proposed precast concrete caisson dock.

For Alternative 1A, a total of 181,600 cubic yards of A1 rock, 29,100 cubic yards of A5 rock, 100,300 cubic yards of B2 rock, 13,350 cubic yards of B3 rock, 30,700 cubic yards of C1 rock, 9,800 cubic yards of C2 rock, 47,725 cubic yards of D filter material, 82,075 cubic yards of F fill material, 367,350 cubic yards of E fill material, and 3,950 of surface course material would be required for construction of the causeway extension. For Alternatives 1B and 1C a total of 181,600 cubic yards of A1 rock, 18,950 cubic yards of A5 rock, 100,300 cubic yards of B2 rock, 8,400 cubic yards of B3 rock, 30,700 cubic yards of C1 rock, 6,200 cubic yards of C2 rock, 43,775 cubic yards of D filter material, 113,450 cubic yards of F fill material, 434,900 cubic yards of E fill material, and 7,100 cubic yards of surface course material would be required for construction of the causeway extension.

For Alternative 1C, the existing main breakwater head for a distance of approximately 500 linear feet would be demolished and re-positioned on a more easterly dog-legged alignment. This would be necessary to provide for additional entrance channel width at -35 feet MLLW into the deeper inner maneuvering area. Stone size for the outer armor layer of the re-aligned breakwater head was determined based on using the same size armor stone (A5) as the existing seaward side of the trunk section of the main breakwater. Since the proposed causeway extension would provide for full wave protection from the southwest and south, the armor stone size requirement for the new main breakwater head would be governed by ice forces instead of wave forces. Therefore, a “transition” section similar to the existing main breakwater head would be used for the re-aligned head with A5 and A6 armor stone transitioning to all A5 armor stone. As established previously, the side slopes of 1.5H:1V would transition to 2H:1V over a distance of 100 linear feet. Armor stone (A5 rock) with a range of sizes from 10-ton maximum weight, 8-ton average weight to 6-ton minimum weight would be used on the seaward face of the transition and full head section of the main breakwater. A6 rock with a range of sizes from 6.5-ton maximum weight, 5-ton average weight to 4-ton minimum weight would be used on the harbor side face of the transition section of the main breakwater. Sea-side armor stone thickness would be 10 feet, and secondary stone thickness would be 5 feet. For the transition section harbor side, armor stone (A6 rock) layer thickness would be 9 feet. All the armor stone would be placed “selectively” with the long axis of each stone oriented perpendicular to the side slope and with maximum contact with each surrounding stone. The A5 rock would extend down the side slopes to a 5-foot dredged-in B3 rock buttress configuration at the base of the main breakwater realignment. This provides for toe stability by anchoring the lower reaches of the side slope into the *in-situ* seafloor material and provides protection from potential scour. The A5 rock is sized to be stable under moving ice pack and ice run-up conditions.

For Alternative 1C, the crest elevation for the re-aligned main breakwater head section was set to match that for the existing main breakwater at +14 feet MLLW. It was established during the navigation improvements project design by considering wave run-up, storm surge, extreme high tides, and ice run-up to provide for a partially overtopping structure during large storm events. The crest width would vary from 12 feet in the transition section to 20 feet at the full head section. A total of 15,275 cubic yards of A5 rock, 3,975 cubic yards of A6 rock, 10,925 cubic yards of B3 rock, 11,775 cubic yards of C2 rock, and 5,750 cubic yards of D filter material would be required for construction of the main breakwater head re-alignment.

Typical sections for the causeway extension and main breakwater modifications are shown in the Attachments Section of this appendix.

9.1.9 Concrete Caisson Dock Design

The proposed concrete caisson docks were designed based on the original TAMS design except that rectangular dock modules were selected instead of circular modules. The caisson concept is applicable to the Port of Nome site due to the dense glacial till characteristics of the *in-situ* seafloor material being suitable for its foundations. Also, it has the advantage of its weight to resist damage and impact forces under moving pack-ice conditions. Individual 200-foot by 50-foot by 30-foot concrete dock modules would be fabricated in the Lower 48 states and transported to Nome for assembly and placement. Initial dredging of a 5-foot trench and placement of 3 feet of gravel bedding would be done prior to positioning and sinking the dock modules into place. Concrete wall thicknesses would be on the order of 12 inches and steel reinforcing would consist of two #10 bars at 12 inches on center for interior walls and #12 bars at 12 inches on center for exterior walls. A 9-foot-high concrete parapet wall section would be placed at the face to cap off the top elevation of the dock face. Two steel pipe pile mooring dolphins would be provided for securing vessel tie-off lines. Final design details for the caisson dock would be further refined during preparation of plans and specifications for the project. This would include concrete specifications, connection details, post-tensioning design, fender system design, and mooring dolphin design. It is anticipated that the dock modules could be floated and towed to Nome by tug from the Lower 48 states.

9.1.10 Steel Sheet Pile Modified Diaphragm Dock Design

Two steel sheet pile modified diaphragm docks are proposed for Alternatives 1A and 1B. The new north dock would be 400 feet in length, 52 feet wide, and consist of PS27.5 steel face sheets and tail wall anchor pile sheets driven into the seafloor. Face sheets would have a tip elevation of -35 feet MLLW, tail wall sheets would be stepped at various tip elevations back to a minimum of approximately -15 feet MLLW, and anchor pile sheets would be driven to -27 feet MLLW. Fenders, mooring bollards, and anodes for corrosion protection would be provided. Prior to construction, the existing A5 rock on the causeway side slope would be removed and salvaged. The north dock would tie into and extend north from the existing Westgold Dock.

The new mid dock would have the same design as the north dock but its length would be 535 feet. It would fill in the gap between the City Dock and the Westgold Dock and tie into both at either end.

For Alternative 1C, the existing sheet pile docks would be removed and replaced with a new continuous sheet pile modified diaphragm dock of 1,350 feet in length, 52 feet wide, and consist of PS27.5 steel face sheets and tail wall anchor pile sheets driven into the seafloor. Face sheets would have a tip elevation of -48 feet MLLW, tail wall sheets would be stepped at various tip elevations back to a minimum of approximately -28 feet MLLW, and anchor pile sheets would be driven to -40 feet MLLW.

9.1.11 Uplands

Onshore uplands development to support the expanded port would be provided by the City of Nome per its master plan. In addition, upland staging and laydown areas of 2.1 acres, 1.3 acres, and 1.7 acres would be created on the causeway with construction of the new caisson, north sheet pile dock, and mid sheet pile dock, respectively. Similar additional upland staging and laydown areas would be created on the causeway with construction of the new caisson docks and new sheet pile dock for Alternative 1C. Such areas would be sufficient for current and future anticipated port operations and support along the causeway.

9.1.12 Entrance Channel Navigation

The proposed causeway extension alignment would create an entrance channel with an effective width of 700 feet at project depth between the new causeway head and existing main breakwater head. This width was established to provide for the design vessel (tanker) or the USCG icebreaker to maneuver into position at the new caisson dock. It would also allow new and existing barge, tug, and support vessel traffic to enter the inner maneuvering area with protection from southwesterly and southerly wave conditions. For Alternatives 1A and 1B, sufficient width and turning radius is provided for the right angle right turn into the inner area. In addition, the effective channel into the inner area is increased to a width of 450 feet due to the removal of the spur breakwater. For Alternative 1C the width of the channel into the inner maneuvering area was widened to 650 feet to accommodate sufficient clearance for the larger, deeper draft vessels to navigate to the new inner deep draft dock face.

9.2 Point Spencer Harbor

There is no existing marine infrastructure at Point Spencer. Six design concepts were created for Point Spencer; three assume a plan independent of the Port of Nome expansion concepts, three are based on a joint Nome and Point Spencer plan.

9.2.1 Site Selection

Several sites were considered for port development: the north bight near the tip of Point Spencer, the south bight east of the runway, and the south shore near the LORAN-C station. Development of port facilities on the west shoreline of Point Spencer was not considered due to the wave and ice environment. Site selection was based on natural depth and distance to the -35-foot bathymetric contour to minimize dredging to accommodate the fleet.

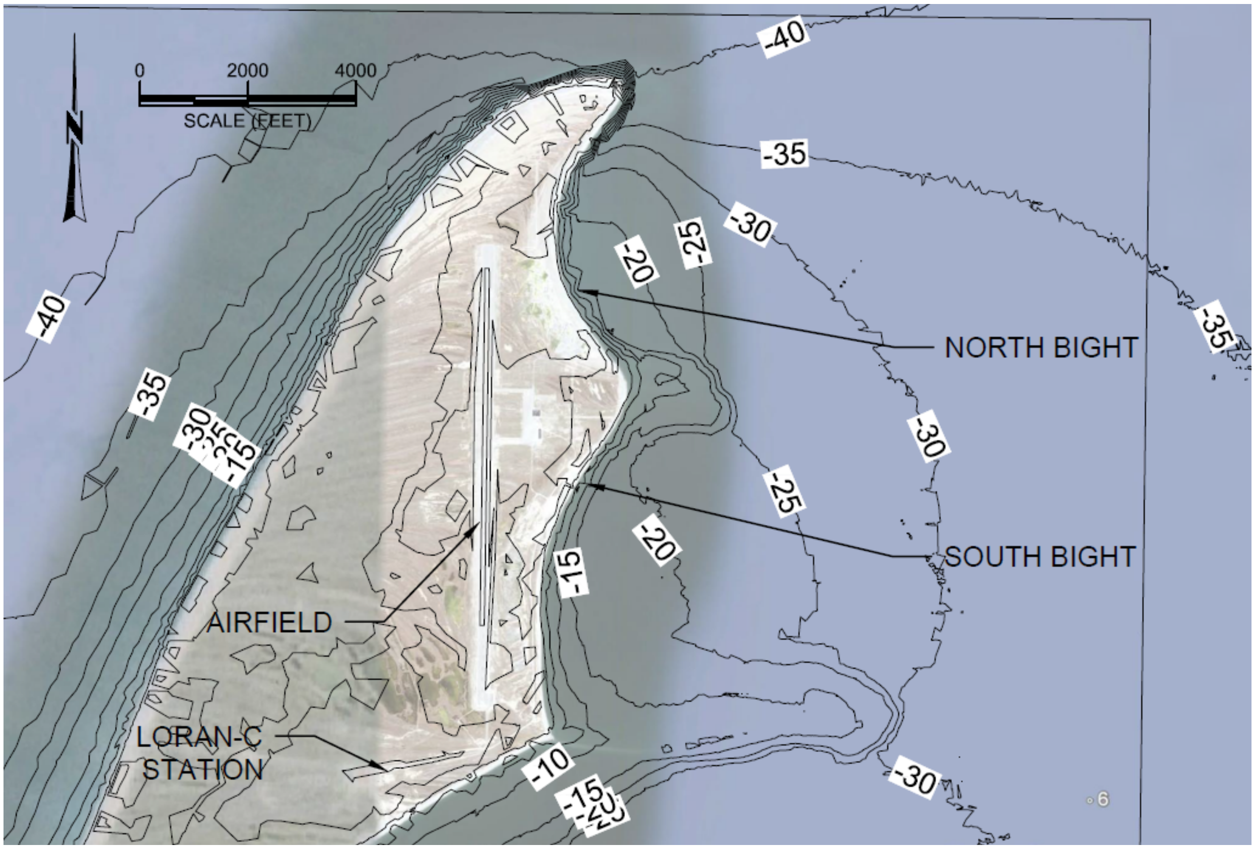


Figure 22: Point Spencer sites and bathymetry

9.2.1.1 North Bight

The north bight includes the shoreline from the tip of Point Spencer to the point opposite the airport apron. Depths at the site are in the 20 to 25-foot range, and it is within 2,000 feet of naturally deep water. A causeway extending from the point between the north and south bight would provide protected moorage for a deep draft dock and turning basin. The north bight site was selected for further consideration because it is the closest to deep water.

9.2.1.2 South Bight

The south bight includes the shoreline from the point near the apron to the point at the south end of the runway. The area is bounded by two underwater ridges with elevations around -15 feet MLLW. Depths at the site are also in the 20 to 25-foot range; however, the distance to -35 feet is 7,000 to 8,000 feet. Since this site would require a longer navigation channel than the north bight, it was excluded from further consideration.

9.2.1.3 LORAN-C Station

The site near the LORAN-C station includes the shoreline from the point at the south end of the runway to the southern end of the island at the end of the spit. The slope of this stretch of shoreline is more gradual than at the north and south bights and would require either a long causeway to reach deep waters or very high dredging volumes to create a basin near the shoreline. Deep water is approximately 30,000 feet from this site. Additionally, construction of facilities would need to account for runway safety zones and would be subject to height restrictions, which could prohibit the use of this site by vessels with high masts or cargoes. This site was excluded from further consideration.

9.2.2 Alternative Design

The alternatives at Point Spencer are based on providing a deep draft dock face with a depth of -35 feet MLLW to support oil and gas exploration activities in the arctic. Several alternative designs were considered for deep draft navigation improvements at Point Spencer. All of the alternatives at Point Spencer share common elements described in this section. The alternatives vary the access to the site and length of the dock.

The alternatives were evaluated using established design guidance given in the appropriate Corps of Engineers Engineering Manuals (EM's) and the Coastal Engineering Manual (CEM). Physical modeling of the alternatives was not included in the scope of this analysis.

The Point Spencer site has no upland area developed to support a port. The natural elevation of land surrounding the airport and LORAN-C station is low and subject to regular flooding during fall storms. An area of relatively higher ground was identified east of the apron that could be raised above the expected 2 percent AEP inundation levels with a minimal amount of fill material. The size of the uplands (15.5 acres) was based on the size of the Northland container yard in Nome. This size would need to be adjusted during the PED phase to account for the needs of oil and gas exploration activities.

Due to the low frequency of occurrence of wave heights exceeding 1 meter at the dock site, wave protection measures were deemed unnecessary for alternative design. Pile supported structures were avoided due to potential ice damage to and jacking forces on the piles during the winter months. The dock was placed as close to the east shore of Point Spencer as possible to maximize the wave protection afforded by the shape of the shoreline.

The alternative plans at this site were developed for a 50-year design life. All alternatives incorporate the following features: a 15.5-acre storage area adjacent to the airport apron, a 2,800-foot road connecting the storage area to the causeway, and a new entrance channel and maneuvering area dredged to -35 feet MLLW with maximum depth of -37 feet MLLW.   Power would be extended from the existing LORAN-C station powerhouse to the dock. Fuel pipelines would be constructed to connect the dock with the fuel storage area located in the new staging area.   Common extended facilities include a water treatment plant, replacement of the diesel generators at the existing powerhouse, a bunkhouse and dining facility, and bulk fuel storage.

The entrance channel alignment to the port would require a 37-degree turn for vessels to enter the turning basin and come alongside the dock. In the turning basin, vessels would execute a U-turn to leave the harbor. Navigation buoy lights would be placed along the range lines of the entrance channel to guide vessels into the harbor.

Plan and section views showing the major features of these alternatives are shown on sheets S-1 through S-5 attached to this appendix. Basic design elements of each alternative are shown in Table 20.

Table 20: Point Spencer Harbor Alternative definition

Point Spencer Harbor Alternative	Dock Length (ft.)	Fuel Storage (gal.)	Airport Improvement	Road Access
2a	200	5,000,000	YES	YES
2b	600	5,000,000	YES	YES
2c	1,000	5,000,000	YES	YES
3a	200	1,000,000	NO	NO
3b	600	1,000,000	NO	NO
3c	1,000	1,000,000	NO	NO

9.2.3 Entrance Channel and Maneuvering Area

The entrance channel and turning basin for the port would require dredging an average 11.6-foot cut to –35 feet MLLW. Such a depth is sufficient for the design vessel based on criteria given in Section 7 of this appendix. The entrance channel would follow the eastern shoreline of Point Spencer to daylight at the -35-foot MLLW contour. The effective channel width would be 700 feet.

9.2.4 Wave Heights

The dock was positioned to reduce exposure to the Port Clarence climate as much as possible. As with the Port of Nome alternative, acceptable waves are defined as 3.3 feet (1 meter) or less at the dock face. Wind and wave analysis shows that wave heights at the dock site have been less than 1 meter approximately 97 percent of the time without construction of additional wave protection features. Maximum waves at the site were calculated to be in the 6 to 8 foot range.

9.2.5 Circulation

The Point Spencer alternatives are all open to Port Clarence and would not significantly alter existing currents at the site.

9.2.6 Shoaling

No estimate of sediment transport around the tip of Point Spencer has been made. Site photographs suggest that the beach material is composed of large cobbles, and the slope of the shoreline from 0 to -40 feet MLLW suggests large particles with a high angle of repose. Surveys conducted by NOAA in 1950 and 2010 were compared to identify trends in sediment movement. In general, the Point Spencer harbor area shows a slight trend towards accretion. Over the 60-year period of record, the change in depth in the basin was within 1 foot. There was a trend of accretion near the entrance channel where depths increased about 0.6 foot over 60 years, and a trend towards scour near the south end of the basin where depths changed on the order of -1.2

feet. The effect of different survey methodologies and changes in datum on these numbers is not known.

It is expected that dredging the channel and basin would increase the rate of sedimentation within the dredged area. Without a barrier to divert material from the site, the natural tidal currents are expected to reduce in speed over the dredged area and release suspended material over the basin. In consideration of this process, it was assumed that sedimentation rates at the site would be on the order of 10 to 20 times as high as the historic rates. For the purposes of comparing alternative maintenance costs, it was assumed that a volume equivalent to 7.5 feet of material covering the basin and channel area would accumulate within the authorized limits of the entrance channel and turning basin. Further study of sediment transport rates and typical particle sizes of the shorelines at Point Spencer would be needed to refine this estimate.

9.2.7 Construction Dredging

Dredging would be required for the Point Spencer alternative. Dredging quantities were derived from bathymetry published by NOAA in 2010. No sampling of the material to be dredged has been performed. It is assumed that the dredged material would consist of silts, sands, gravel, cobbles, and glacial till, trending towards larger particle sizes. It is anticipated that dredging such material would be difficult but could be performed with mechanical equipment such as a clam shell dredge and would not require drilling and blasting. Side slopes for the entrance channel and maneuvering area would be dredged to 5H:1V. Side slopes would not require slope protection. Side slopes may be increased after the material to be dredged is sampled and analyzed. Dredging quantities for the Point Spencer Harbor alternatives are shown in Table 21.

Table 21: Dredge quantities for Point Spencer Harbor alternatives

Material (cubic yards)	Alternative		
	2a, 3a	2b, 3b	2c, 3c
Construction Dredging	1,543,000	1,565,000	1,583,000
Overdredge Allowance	262,000	262,000	262,000

9.2.8 Dredged Material Disposal

Sufficient water depths near the proposed project site for delineating an offshore dredged material disposal site are available. Within 5 miles west of the shoreline at Point Spencer, water depths are -50 feet MLLW and greater. The dredged material could be disposed of in such a site efficiently by dump scow barge. Alternatively, a near shore area along the beach on the west side of the Point Spencer Spit could be used as a dredged material placement area. The material could be placed in a submerged berm configuration to provide a source for nourishing the toe of the beach. This area is just south of the proposed port development site on the west side of the Spit.   Water depths vary from approximately -20 feet MLLW offshore to 0 feet MLLW at the beach. Dredged materials could be placed in the littoral zone by barge in evenly distributed lifts along the beach line progressing in the southerly direction. There is the potential to use some of the dredged material to construct the fill zone of the causeway. Testing of the material in the dredge prism would be required to determine whether the material is suitable for construction.

9.2.9 Maintenance Dredging

Maintenance dredging would be expected to be minimal over the course of the design life of the project. Littoral transport of sediments generally appears to be from south to north along the seaward shoreline of Point Spencer, and material appears to be deposited around the tip. The rate of sediment transport at this site has not been measured and is not known. An estimated maintenance dredging requirement is described in the Operations and Maintenance section of this appendix.

9.2.10 Causeway Design

The causeway is a rock spall fill prism road embankment with armored revetments to protect the side slopes from erosion. The alignment is perpendicular to the shoreline and provides the shortest route to the dock. Armor stone size for the causeway and breakwater was determined using the Hudson's equation ad defined in Table VI-5-22 of the CEM. Stone size for the outer armor layer was determined using a wave height of 6.6 feet and a side slope of 1.5H:1V, and a K_d value of 4.8 for selective placement and a breaking wave condition per the CEM guidance. The higher K_d value used for the Nome alternative was not used because the armor size and slope for the Point Spencer alternative have not been modeled to determine this value. A stone specific gravity of 2.6 was assumed for the calculations. Armor stone (A rock) with a range of sizes from 2,250-pound maximum weight, 1,800-pound average weight to 1,350-pound minimum weight would be used. Secondary stone (B rock) would range from 235-pound maximum weight, 180-pound average weight to 125-pound minimum weight. Core stone (C rock) would range from 14-pound maximum weight, 9-pound average weight to 5-pound minimum weight. Armor stone thickness would be 4.5 feet, and secondary stone thickness would be 2 feet. Fill material would be unclassified fill and could be composed of rock spalls that could be placed underwater without compaction effort. All the armor stone would be placed "selectively" with the long axis of each stone oriented perpendicular to the side slope and with maximum contact with each surrounding stone. The A rock would include a 6.75-foot weighted toe to protect the armored slope from damage. B rock would be extended 3 feet beyond this toe to filter the natural beach material and for constructability. As the existing ground rose above Mean Lower Low Water, the three layered armor section would be replaced with a 4-foot-thick riprap revetment with a median stone weight of 200 pounds.

The crest elevation for the causeway was determined to be +13 feet MLLW by considering run-up and storm surge using the MHHW water level and 2 percent AEP wave heights. Run-up analysis was performed in accordance with EM 1110-2-1100 Part V Section 5. The run-up calculation includes a 2-foot freeboard allowance that accommodates all scenarios of estimated sea level change over the life of the project.

The roadway surface and subbase layers would be composed of aggregates separated from the rock spall fill by a separation geotextile. The caisson units would be filled and paved with the same material as the causeway. Causeway and dock fill quantities are shown in Table 22.

Table 22: Causeway and Caisson Fill Quantities

Material (cubic yards)	Alternative		
	2a, 3a	2b, 3b	2c, 3c
A Rock	16,400	16,400	16,400
B Rock	3,500	3,500	3,500
C Rock	2,300	2,300	2,300
Class II Riprap	1,000	1,000	1,000
Rock Spall Fill	44,900	66,200	87,800
Aggregate Surface Course	1,900	2,700	3,400
Aggregate Subbase	4,000	5,500	7,000
Separation Geotextile (square yards)	6,400	8,800	11,300

9.2.11 Concrete Caisson Dock Design

The proposed dock for Point Spencer uses the same caisson concept used for the Port of Nome alternative described in Section 9.1.9. An array of dock lengths was developed for Point Spencer to accommodate different levels of operations. A 200-foot-long dock can accommodate a single vessel and may require re-positioning of larger vessels during the cargo transfer process. A 600-foot-long dock can accommodate a single design vessel or multiple smaller support vessels. This dock would allow for cargo transfer operations without moving the vessel for the entire fleet. A 1,000- foot-long dock would allow the largest vessels considered in the study, the USCGS Healy and the line haul ATB, to use the dock simultaneously. Dock configurations considered in each alternative are shown in Table 23. The dock would be equipped with pneumatic fenders to protect both the facilities and vessels from damage during operations. The fenders would be chain-mounted to the top of the dock and would be hoisted from the water and stored on the dock when the harbor is frozen.

Table 23: Dock configurations for Point Spencer Alternatives

Item	Alternative		
	2a, 3a	2b, 3b	2c, 3c
200-foot Caissons	1	3	5
Dock Face Length (ft.)	200	600	1,000
Mooring Dolphins	6	4	2
Mooring Length (ft.)	1,200	1,200	1,200

9.2.12 Uplands

Point Spencer currently has no developed upland areas. The natural ground adjacent to the airfield and buildings of the LORAN-C station experiences regular coastal flooding as reported by the Coast Guard. Most of the undeveloped terrain consists of low lying dunes and shallow lakes. Higher ground to the northwest of these sites may provide suitable upland area with the placement of additional fill. A total of 15.5 acres of uplands were designed for all Point Spencer alternatives with the intent of matching the current container yard at the Port of Nome.

9.2.13 Bulk Petroleum Product Storage Facilities

To operate the port, fuel storage for resupply of vessels would be required. The LORAN-C station has 420,000 gallons of fuel storage for its power plant. It is assumed that power for the new facilities would be generated from the existing power plant and that these tanks would be required for power generation. New tanks would be required for the refueling of vessels at the site. Alternatives 2a, 2b and 2c assume that 5 million gallons of fuel storage would be required at Point Spencer to support future operations. Alternatives 3a, 3b and 3c assume that 1 million gallons of fuel storage would be required at Point Spencer to support future operations in conjunction with expanded port facilities at Nome. A new facility would require aboveground storage tanks with a secondary containment area with a recommended volume capacity of 120 percent of the maximum product storage capability of the tanks. The new tanks would be sited next to the LORAN-C station to take advantage of naturally higher ground near the site and to be out of the safety areas of a potential runway expansion project. This location would require building pipelines from the dock around the south end of the runway to the LORAN-C station.

9.2.14 Crew Facilities

It was assumed that the Coast Guard would maintain control over the buildings of the LORAN-C station and that a new support building would be required for all alternatives to maintain personnel at the site. The new building is expected to provide rooms and a dining facility for 50 personnel. A detailed plan of site operations would be needed to design full support facilities for the site.

9.2.15 Water Supply

To support full time operations during the exploration season, a potable water source with the capacity to sustain the Point Spencer station crew and resupply vessels would need to be developed. The existing water supply utilized a water catchment area to fill a storage tank for continuous operations. The Coast Guard noted that the water catchment area would become contaminated with seawater after fall storm events. To provide a reliable supply of fresh water at the site, it was assumed that a reverse osmosis treatment plant would be needed to process water taken from the catchment area. Also, to increase the water supply, it is assumed two new wells would be installed. Treated water would be stored in a 1-million-gallon insulated and heated storage tank to provide water for year-round station tenants and vessels using the port facility when surface water at the site is frozen. Installation of test wells, water quality analysis, and a projected water demand for the life of the project would need to be performed prior to the design of a water treatment facility.

9.2.16 Sewage Treatment and Solid Waste Management

With increased use of the site, it is expected that significant waste products would be generated, and facilities to handle these wastes would be required. It is anticipated that sewage can be treated with a septic system sized to handle the needs of the full time resident staff and seasonal exploration staff needed to manage supply operations. Solid waste would be collected to a central point and buried in accordance with current landfill practices to keep debris from being scattered by the wind. It is assumed that a 3-acre landfill site can accommodate solid waste production over the life of the project.

9.2.17 Airport

Alternatives 3a, 3b and 3c would utilize the existing runway and airport infrastructure to provide site access. The existing runway is capable of handling medium sized cargo planes such as the Lockheed C-130, which has been used by the U.S. Coast Guard at the site. A C-130 aircraft requires a minimum civilian runway length of 4,000 feet (either paved or gravel).

Alternatives 2a, 2b and 2c would require a substantial airport improvement project to improve access for crew changes and delivery of perishable goods. It was assumed that the airport would need to be upgraded to allow jet service. The most common jet aircraft flown to western Alaska airports is the Boeing 737-400. To accommodate this aircraft, it is assumed that the existing runway would need to be shifted north to accommodate 1,000-foot safety areas at each end and completely rebuilt. A new runway concept including a 6,500-foot-long runway and taxiway were provided by the Alaska Department of Transportation and Public Facilities.

9.2.18 Road

Alternatives 2a, 2b and 2c require a road link to Nome. The new road would be approximately 27 miles long and cross mountainous terrain, coastal tundra, a 1.9-mile-long stretch of beach that overtops regularly during storm events, and a 1 to 2 mile connection to the causeway. A planning level road concept (with gravel surface) was provided by the Alaska Department of Transportation and Public Facilities.

9.3 Cape Riley

9.3.1 Site Selection

The area south of Cape Riley, specifically south of Willow and Yellowstone Creeks, was considered for port development. Site selection was based on limiting channel length to natural deep water, the length of breakwater protection, ease of access to suitable upland areas, and the length of access road to the Nome-Teller highway.

9.3.2 Alternative Design

The alternative at Cape Riley is based on providing a shallow draft dock face with a depth of -12.5 feet MLLW to support lightering operations for mineral extraction to the in the area. The design is based on a short navigation channel providing the design vessel with access to a protected loading dock and turning basin. Land access to the dock would be via gravel access road from the Nome-Teller Highway. The Cape Riley alternative was developed for a 50-year design life.

The Cape Riley alternative, shown in Figure 4-1, incorporates the following: a 305-foot-wide entrance channel joining a 550-foot-wide turning basin, a 250-foot-long concrete caisson dock with an attached 1.5-acre staging area. The entrance channel alignment includes a 90 degree turn around the breakwater head to line up the dock approach. In the turning basin, vessels would execute a U-turn after leaving the dock. Wave protection for the turning basin and dock would be provided by a 1,575-foot-long rubblemound breakwater. A new navigation marker light would be established at the head of the breakwater. No upland utilities or fuel storage would be provided.

The alternatives were designed using established guidance given in Corps of Engineers Engineering Manuals (EM's) and the Coastal Engineering Manual (CEM). Physical and numerical modeling of the alternative was not included in the scope of this design.

Due to the wave and ice conditions at the site, a rubblemound breakwater was considered to be the only effective means of protecting the turning basin and dock. Pile supported structures were avoided due to potential ice and jacking forces during the frozen months. The location of the dock was placed as close to shore as possible to minimize breakwater and dock placement quantities while providing sufficient entrance channel width for the design vessel.

The Cape Riley site has extensive upland areas that could be developed to support port and/or mineral extraction operations. The natural elevation of the upland area adjacent to the planned port is significantly higher than any coastal flood level. The size of the upland staging area would likely need to be adjusted to accommodate the specific mineral extraction operation. Additional upland staging area and material storage areas could be added to account for the specific needs of the particular mining operation during the PED phase.

9.3.3 Entrance Channel and Turning Basin

The entrance channel and turning basin for the port would require dredging an average dredge cut of 6.1 feet to establish the –12.5-foot MLLW depth. Such a depth is sufficient for the design vessel based on criteria given in Section 7 of this appendix. The entrance channel would follow the eastern shoreline of Cape Riley to daylight at the -35 feet MLLW contour. The channel width would be 305 feet in both straight and curved sections.

9.3.4 Wave Heights

The causeway and breakwater were positioned to reduce wave heights from the various directions of exposure to acceptable levels at the dock face and within the turning basin. The acceptable wave level for the Cape Riley dock alternative was defined as 3 feet or less at the dock face.

9.3.5 Circulation

The alternative would have a limited impact on currents directly adjacent to the proposed protected basin. Minimal tidal currents are expected near the proposed dock alternative. Currents a short distance away from the protected basin would be unaffected by the proposed alternative.

Tidal circulation within the basin is expected to be adequate. The layout of the breakwater and turning basin provides a low basin aspect ratio with rounded basin corners to provide adequate tidal exchange. Normal tidal forcing in the region is low, with tide ranges less than 1.2 feet, but the basin tidal volume exchanged each tide is still roughly 10 percent of the total basin water volume. The basin configuration lies within guidance parameters to maintain acceptable levels of circulation, and it is expected that an adequate exchange of water between the basin and unprotected areas would occur.

9.3.6 Shoaling

No estimate or study of sediment transport around the tip of Cape Riley has been made. Site photographs suggest that the beach material is composed of coarse gravel and sand at the surface. Near-shore bathymetry slopes vary between 50H:1V and 100H:1V, which generally indicates finer sediments. The shoreline near the port site appears to be fairly stable but historical surveys in the area are very limited so trends cannot be determined.

It is expected that dredging the channel and basin and constructing the causeway and breakwater would increase the rate of sedimentation in the general area. For the purposes of comparing alternative maintenance costs, it was assumed that a volume equivalent to 1.5 feet of material covering the basin and channel area would accumulate within the authorized limits of the entrance channel and turning basin. A study of sediment transport rates, potential sediment sources, transport methods, and sediment characterization of the soils at Cape Riley would be needed to estimate shoaling.

9.3.7 Construction Dredging

Dredging would be required for the Cape Riley alternative. Dredging quantities were derived from bathymetry in the published Port Clarence NOAA chart. No sampling of the material to be dredged has been performed. It is assumed that the majority of the dredged material would consist of gravel, sands, and silts with some cobbles, boulders, and glacial till. It is anticipated that dredging such material would be difficult but could be performed with mechanical equipment such as a clam shell dredge and would not require drilling and blasting. A total of 182,300 cubic yards for the channel and turning basin would be dredged. Up to 29,700 cubic yards of additional material could be dredged to the maximum pay limit. This produces a dredging volume range from 182,300 cubic yards to 210,000 cubic yards. Side slopes for the entrance channel and maneuvering area would be dredged to 3H:1V. Side slopes angles and the need for possible slope protection would need to be addressed after the material was sampled and characterized.

9.3.8 Dredged Material Disposal

The dredged material for the Cape Riley alternative could be placed in the same location as that for the Point Spencer alternatives, i.e. either in an offshore disposal area or in the near shore area along the beach on the west side of the Point Spencer Spit. Both areas are approximately 18.5 miles by barge west and south of the proposed port development site. There is the potential to use some of the dredged material to construct the fill zone of the causeway. Testing the material in the dredging prism would be required to determine whether the material is suitable for construction.

9.3.9 Maintenance Dredging

Maintenance dredging is expected to be minimal over the course of the design life of the project. Although the rate of sediment transport at this site has not been measured, the littoral transport of sediments and maintenance dredging is assumed to be minimal due to the gravel and cobble surface material, lack of significant freshwater drainages nearby, and the low tidal currents expected for the area. Significant transport of sediments is assumed to be only associated with storm events. An estimated new maintenance dredging requirement is described in the Operations and Maintenance section of this appendix.

9.3.10 Breakwater Design

The breakwater was positioned as close to shore as possible while providing the minimum entrance channel width and turning basin dimensions required for the design vessels to use the new dock. The alignment takes advantage of the Cape Riley headland to the north and the curving bay shoreline to the south to provide protection from wind and waves from the north and south sectors. Maximum depths of water are –11 feet MLLW along the outside toe of the breakwater. Foundation materials are assumed to be sand, gravel, and glacial till that would serve as a suitable base for the structure.

Armor stone size for the causeway and breakwater was determined using the Hudson's equation ad defined in Table VI-5-22 of the CEM. Stone size for the outer armor layer was determined using the significant wave height established previously, along with an armor side slope of 1.5H:1V, and a K_d value of 2.0 for random placement and a breaking wave condition per the CEM guidance. A stone unit weight of 165 lbs/ft^3 was assumed for the calculations. The resulting armor stone gradation gives a range of weights from 12,000 pounds maximum, 8,600 pounds average, and 6,000 pounds minimum. Secondary stone (B rock) would range from 1,750-pound maximum weight, 860-pound average weight to 200-pound minimum weight. Core stone (C rock) would range from 200-pound maximum weight, 86-pound average weight to 5-pound minimum weight. Armor stone layer thickness would be 7.5 feet, and secondary stone layer thickness would be 3.5 feet. Fill material would be unclassified fill and could be composed of dredged material if suitable material is encountered.

The crest elevation for the causeway was determined to be +18 feet MLLW by considering run-up and storm surge using the 2 percent AEP water level and 2 percent AEP wave heights. Run-up analysis was performed in accordance with EM 1110-2-1100 Part V Section 5. The breakwater was modeled as permeable slopes due to the coarse, angular rock used for the core.

A total of 48,300 cubic yards of A rock, 29,500 cubic yards of B rock, and 35,900 cubic yards of C rock would be required for construction of the causeway and breakwater.

9.3.11 Concrete Caisson Dock Design

The proposed dock for Cape Riley uses a similar caisson concept used for the Port of Nome alternative described in Section 9.1.9. At Cape Riley, one 250-foot by 40-foot by 26-foot concrete dock modules would be used to create 250 feet of linear dock face.

9.3.12 Uplands

Cape Riley currently has no developed upland areas. The shoreline adjacent to the proposed dock has a mild beach slope with a fairly steep beach berm behind it. The upland areas landward and to the east are relatively flat with only a few small drainages.

9.3.13 Road

A gravel access road is included in this alternative to provide vehicle access from the Nome-Teller Highway. A planning level road concept (with gravel surface) was provided by the Alaska Department of Transportation and Public Facilities. The cross-section includes 5 feet of fill with a half-foot gravel surface course. The road section was designed as all fill to eliminate possible rock excavation of the low hills in the area. Fill slopes were designed as 1V:2H slopes. The shoulder to shoulder width is 22 feet to accommodate light off-road haulers typically used in smaller quarrying and mining operations.

10. PROJECT IMPLEMENTATION

10.1 Breakwaters and Causeways

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. Where road access is provided by the causeways, stone placement could be accomplished from dry ground; however, this may be limited by the reach of the placing equipment.

10.2 Dredging

The material at all sites is assumed to require mechanical dredging equipment to reach design depths. Dredging at Port Clarence would employ the use of cranes with clamshell buckets or excavators mounted on barges. 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. Dredging at Nome may employ either a cutter head or clamshell with materials being disposed of onshore through direct placement, or in the nearshore environment inside of the zone of closure to ensure materials are pushed to the beach through wave action.

10.3 Caisson Dock

The caisson dock represents a specialized construction activity with two distinct phases: caisson fabrication and caisson placement. The fabrication phase would occur outside Alaska, presumably on the west coast of the United States in a controlled precast fabrication facility. Fabrication may occur in a precast fabricator’s graving yard or leased dry dock space with concrete production equipment temporarily moved to the facility to cast the caissons. Production of the caissons would potentially take an entire calendar year due to the height of the caissons, tonnage of reinforcing bars to be assembled and placed, and the volume of concrete to be poured. Once completed, the caissons would be floated from the production facilities and towed to the site by tug or placed on a submersible heavy lift barge and towed to the site. The caissons would be fitted with precast concrete lids to prevent water from filling the units and lowering their draft in transit. The tow or barging operation would be timed to occur in the summer months to minimize the chance of heavy seas in the Gulf of Alaska and minimize the risk of damaging or losing the caissons in transit.

Prior to arrival, the dock locations would need to be dredged to 3 feet below the bottom of the caisson and filled with a level bedding course of rock spalls or aggregate material to provide a solid foundation for the caissons to land on. The caissons would be positioned over their landing sites by tug and flooded with water until the caisson is firmly seated on the bedding layer, then filled with material to final grade. Openings in the interior walls of the caisson would help ensure that the caisson remains level as it is landed in its final position to minimize deformation of the bedding layer. Drain holes cast below the tide range would be opened once the units were grounded to allow rainwater to drain from the caisson once filled. Where multiple caissons would be used to create a single dock face, the units would incorporate a precast grout channel near the seaward face. These channels would be positioned 1 foot apart during the landing operations. Once the caissons have settled on their foundations, the grout channels would be lined with a fabric tube and filled with hydraulic grout. The space between the caissons would be filled once the grout has set.

10.4 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, 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 may or may not be constructed concurrently with the deep draft port project. For the Nome alternative, it is assumed that the City of Nome would incrementally develop upland areas as needed over the course of many years in support of the port.

10.5 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.

10.6 Construction Schedule

10.6.1 Port of Nome Schedule

Major construction features for the Nome alternative include the rubblemound causeway extension, main breakwater head re-alignment, dredging, concrete caisson docks, sheet pile docks, and extension of fuel, water, and power lines. Stone production in the quarry and placement for the rubblemound causeway and main breakwater would likely be the first project features undertaken. Concurrent demolition of the existing spur breakwater and main breakwater head would likely take place with the salvaged armor stone incorporated into the new construction. Work on dredging and disposal would then follow. The caisson dock would be cast in a concrete graving facility on the west coast of the Lower 48 states (likely Los Angeles, or possibly Seattle). The dock would then be towed or barged to the site at Nome in the second or third year of the construction contract. Sheet pile dock construction could begin following completion of the causeway extension; however, the use of the existing City and Westgold docks would need to be maintained during any construction season. The total estimated performance period for construction the project is a minimum of 3 years and likely would be 4 years. The duration of each summer construction season is estimated to be 4 months (mid-June through mid- October). Winter construction is not anticipated. Construction scheduling would be required to avoid conflict with the continued use of the existing port and harbor facilities at Nome. The existing dock facilities, causeway access road, fuel lines, water lines, power, navigation channel, and small boat harbor would remain operational during construction. Project specifications would detail time restrictions for the contractor to conduct certain activities during specified time periods.

10.6.2 Point Spencer Port Schedule

The scope of the Point Spencer Port would not require a long production time to generate the rock and material necessary to construct the project. The channel and basin would be built first followed by installation of the caisson and causeway. The upland facilities could be built at any time during the project and could be constructed earlier in the schedule to provide equipment staging areas for the contractor to build the project. The caisson would be a long lead item and would likely be constructed concurrently with the dredging and moved to the site as the dredging nears completion. All construction on site would be limited to the open water window for Port Clarence, which is assumed to currently be 15 June through 15 October. The construction time for this alternative is expected to take a minimum of 2 to 3 years.

10.6.3 Cape Riley Harbor Schedule

The likely construction sequence for the Cape Riley alternative would have the breakwater and dredging occurring first, followed by the dock and staging area, with the access road construction occurring last. Due to potential environment restrictions, the breakwater would likely be required to be built prior to entrance channel and basin dredging. Dredging of the entrance channel and turning basin would be required before the caisson dock can be floated into position. All water-side construction on site would be limited to the open water window for Port Clarence, which is assumed to currently be 15 June through 15 October.

The access road could be constructed independent of the port related features. Access to the Nome-Teller Highway would allow the road to be built with local materials. Construction of this alternative is expected to take 2 to 3 years.

11. OPERATIONS AND MAINTENANCE

11.1 Port of Nome Expansion

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. The Alaska District, U.S. Army Corps of Engineers 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 Nome alternatives, a quantity of 3,600 cubic yards of “A1” stone and 600 cubic yards of “A5” stone would be required for replacement on the causeway extension at year 25. For Alternative 1C, a quantity of 400 cubic yards of “A5” stone and 100 cubic yards of “A6” stone would be required for replacement on the re-aligned main breakwater head section at year 25.

Maintenance dredging would be conducted on an estimated 10-year cycle. The outer channel and maneuvering area would require dredging of approximately 36,000 cubic yards. The expanded inner maneuvering area would require dredging of approximately 12,000 cubic yards. 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 outer channel and maneuvering area material would be disposed of in the offshore disposal area east of the port. For the expanded inner maneuvering area, the material would likely be placed on the beach east of the main breakwater as is the current dredged material from the navigation improvements project. Hydraulic cutter head dredging equipment with pipe-line discharge would likely be used for maintenance dredging. Dredged material characteristics should be similar to the current material dredged from the existing navigation channel and sediment trap: sand.

The concrete caisson dock structure(s) would require maintenance on an estimated 20-year cycle. Repairs would include patching damaged concrete surfaces with epoxy grout and grout injection for internal areas. The modified diaphragm steel sheet pile docks would require replacing anodes on an estimated 15-year cycle. For the mooring dolphins, the anodes would be replaced on an estimated 15-year cycle.

11.2 Point Spencer

Operations and Maintenance activities would be similar to those at Nome. The non-Federal sponsor, who would own and operate the project once constructed, has not been identified at this time. Maintenance dredging is arbitrarily assumed to occur on a 5-year cycle with an estimated 100,000 cubic yards of dredging required per cycle. Study of sedimentation rates at the site would be needed to produce better estimates of O&M dredging requirements for the site. Maintenance of the armor rock is expected to require placement of 330 cubic yards of A rock 25 years after construction is completed.

Anodes for the dolphins are expected to require replacement every 15 years. The exposed concrete surfaces of the caissons are expected to require regular surface and crack repairs on a 20-year cycle.

11.3 Cape Riley

Operations and Maintenance responsibilities would be shared by both the Federal Government and the non-Federal sponsor. The non-Federal sponsor, who would own and operate the project once constructed, has not been identified at this time. General navigation features including the breakwater, entrance channel, and turning basin would be maintained by the Federal Government. The non-Federal sponsor would be responsible for maintenance of the dock, dredging the moorage adjacent to the dock, staging area, and the Nome-Teller Highway access road.

Federal maintenance of the armor rock is expected to require replacement of 960 cubic yards of armor rock 25 years after construction is completed. Federal maintenance dredging is assumed to occur on a 20-year cycle with an estimated 7,500 cubic yards of dredging required per cycle. Study of sedimentation rates at the site would be needed to produce better estimates of O&M dredging requirements for the site.

12. RECOMMENDED FURTHER DESIGN STUDIES

The following are items that warrant further study in the preconstruction engineering design (PED) phase of the project:

a. Detailed analysis of winds and the wave climate at the three sites, particularly for the Nome and Point Spencer alternatives. This could involve deployment of instrumentation for multiple years to collect data on wave heights and periods during storm events, additional numerical and physical modeling, wave refraction /diffraction/reflection analyses, and a wave resonance analysis for the Point Spencer and Cape Riley sites.

b. Geotechnical investigation and analysis of subsurface materials at all three sites 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.

c. Detailed ice engineering analysis including ice-sheet and ice ridge-up modeling and armor rock stability studies.

d. Detailed design of local service facilities including the proposed caisson dock, sheet pile docks, fender systems, mooring dolphins and bollards, utilities, access roads, uplands staging and laydown areas, fuel storage, sewage treatment, water supply and treatment, solid waste, and crew facilities..

e. Ship simulation studies to confirm entrance channel width and maneuvering area turning requirements for the design vessel.

13. REFERENCES

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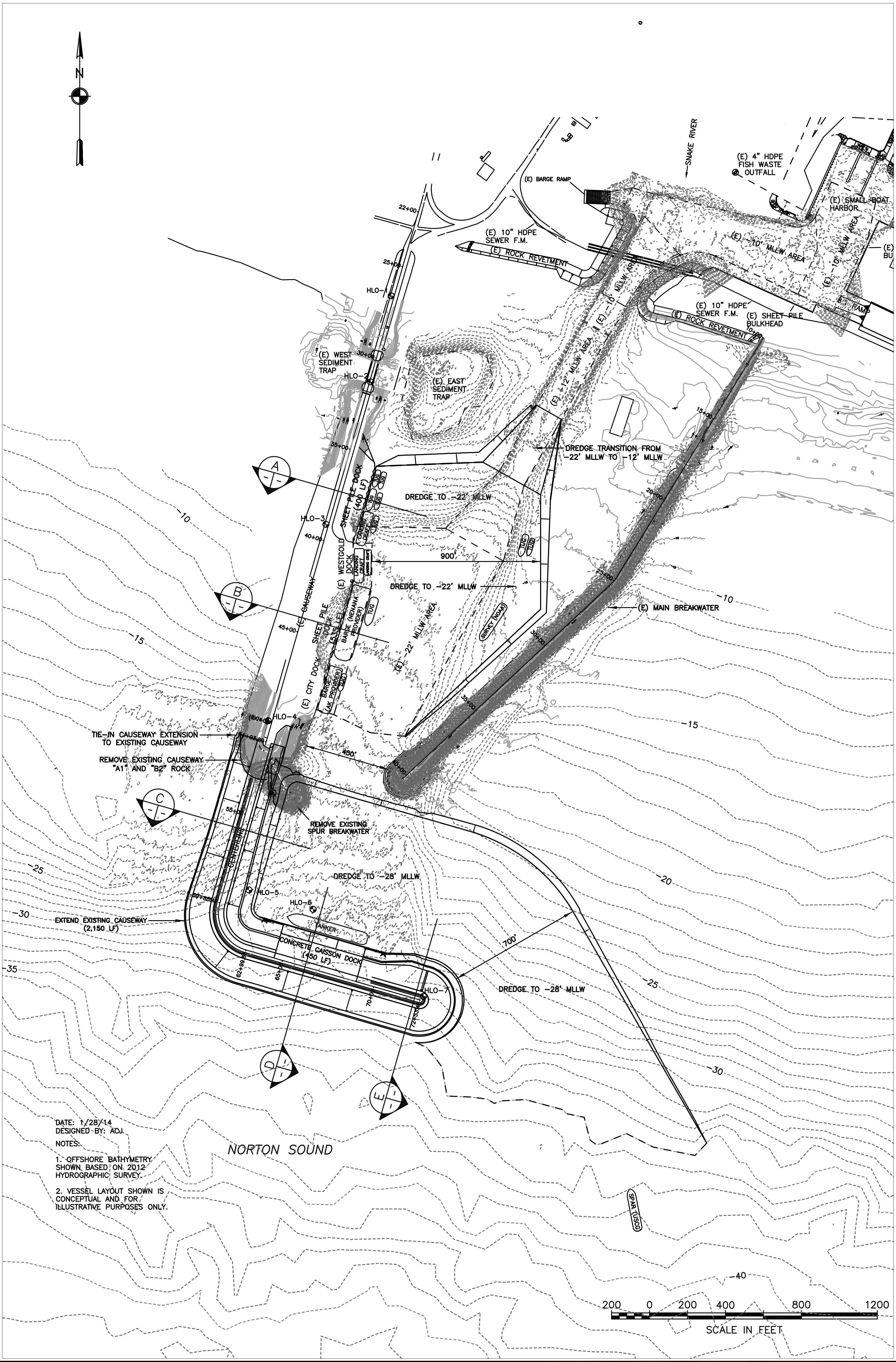
14. ATTACHMENTS (FIGURES)



DATE: 1/28/14
DESIGNED BY: ACJ

NOTES:

1. OFFSHORE BATHYMETRY SHOWN BASED ON 2012 HYDROGRAPHIC SURVEY.

2. VESSEL LAYOUT SHOWN IS CONCEPTUAL AND FOR ILLUSTRATIVE PURPOSES ONLY.

ALASKA DISTRICT
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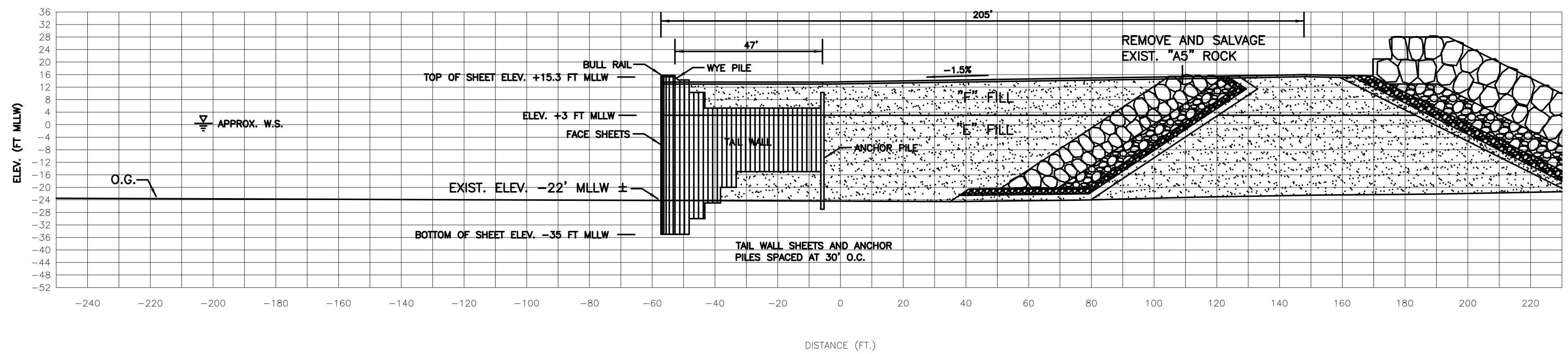
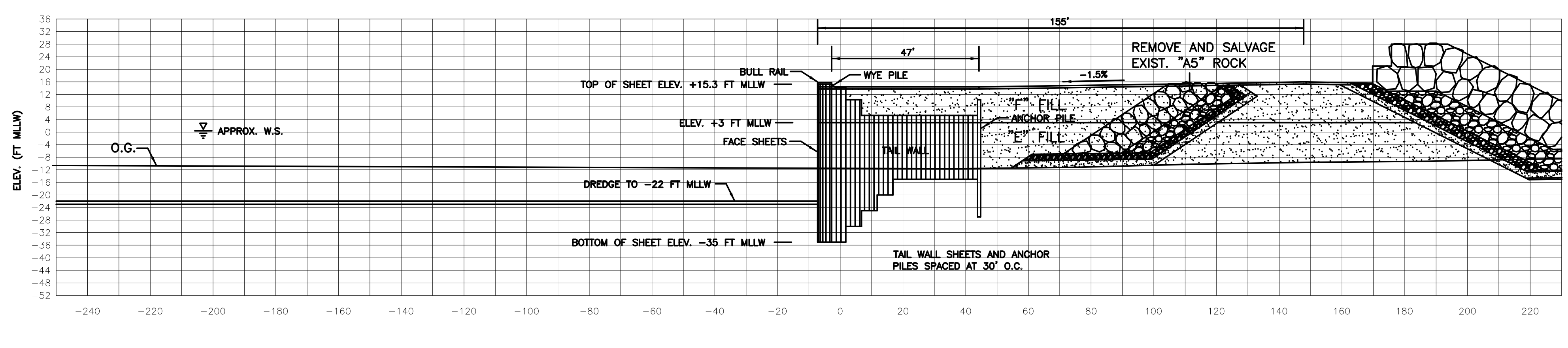
PORT OF NOME ALTERNATIVE 1A
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

1A-1



B
TYPICAL SECTION – MID DOCK
STA. 41+80 TO 47+15



A
TYPICAL SECTION – NORTH DOCK
STA. 35+25± TO 39+68

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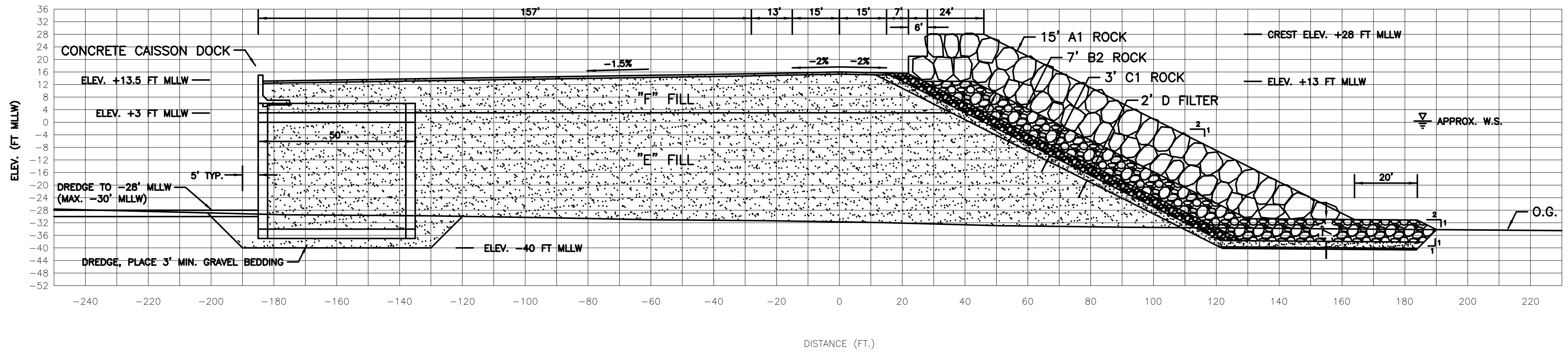
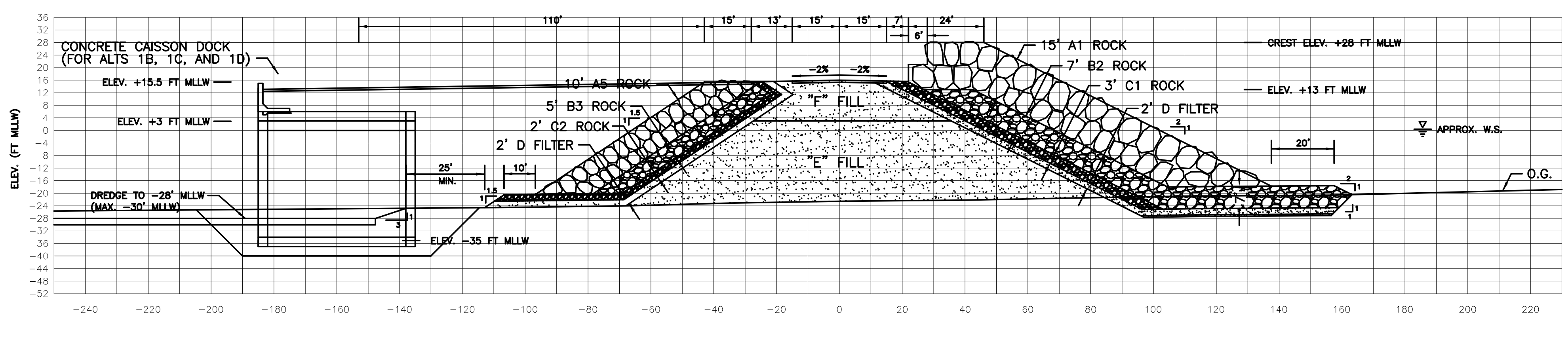
PORT OF NOME ALTERNATIVE 1A
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

1A–2



TYPICAL SECTION – CAUSEWAY EXTENSION
STA. 63+78.46 TO 69+78.46



TYPICAL SECTION – CAUSEWAY EXTENSION
STA. 51+03.20 TO 63+78.46
CONCRETE CAISSON DOCK SHOWN
APPLIES FOR FOR ALTS 1B, 1C, AND 1D

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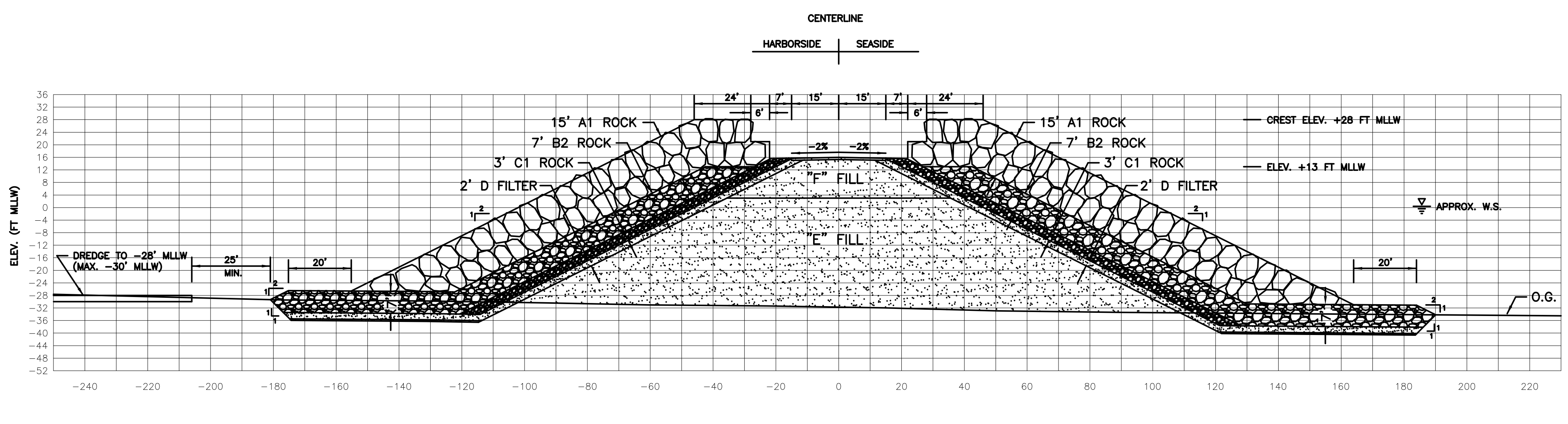
PORT OF NOME ALTERNATIVE 1A
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY



E

TYPICAL SECTION – CAUSEWAY EXTENSION HEAD
STA. 71+53.46 TO 72+53.46

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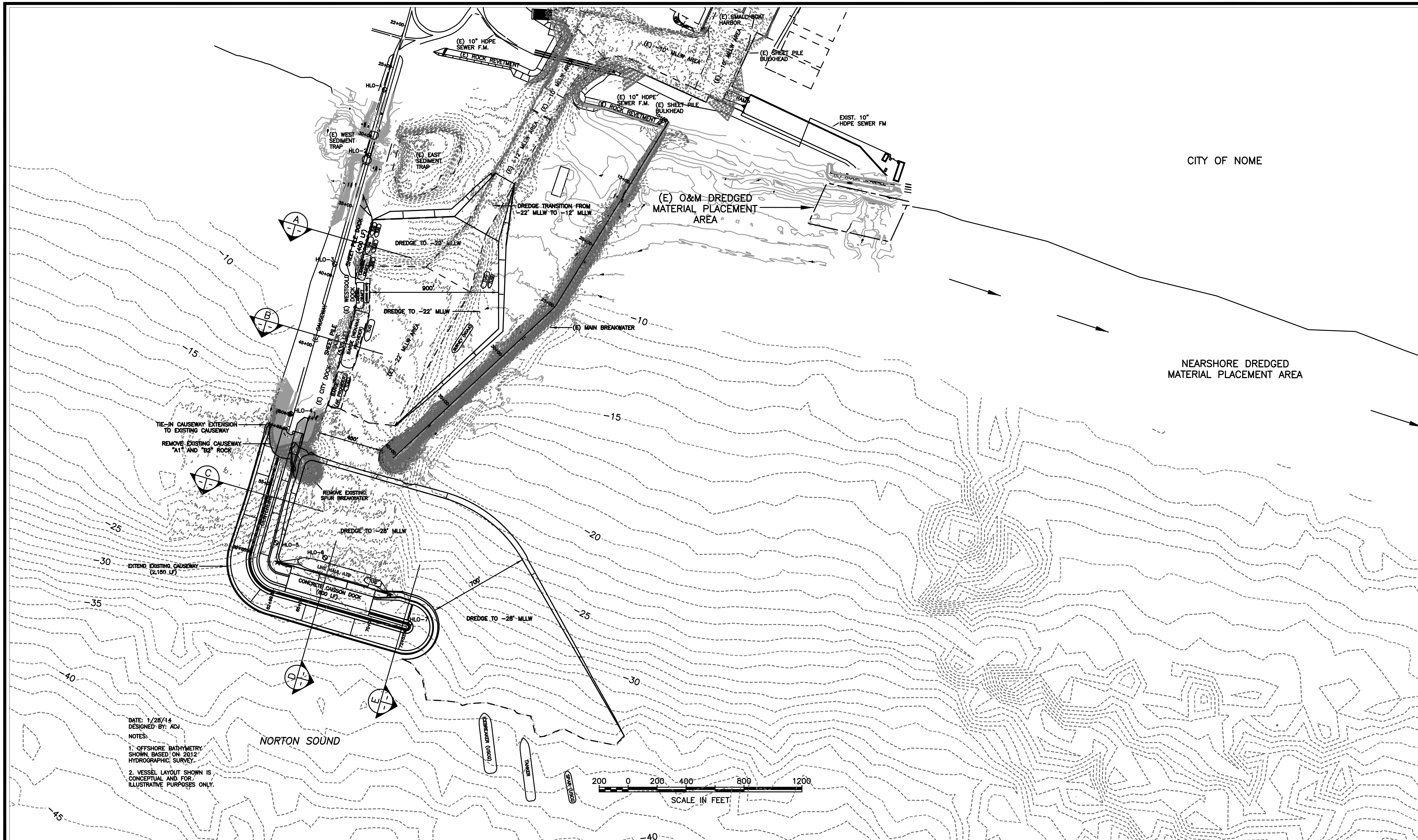
PORT OF NOME ALTERNATIVE 1A
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

1A–4



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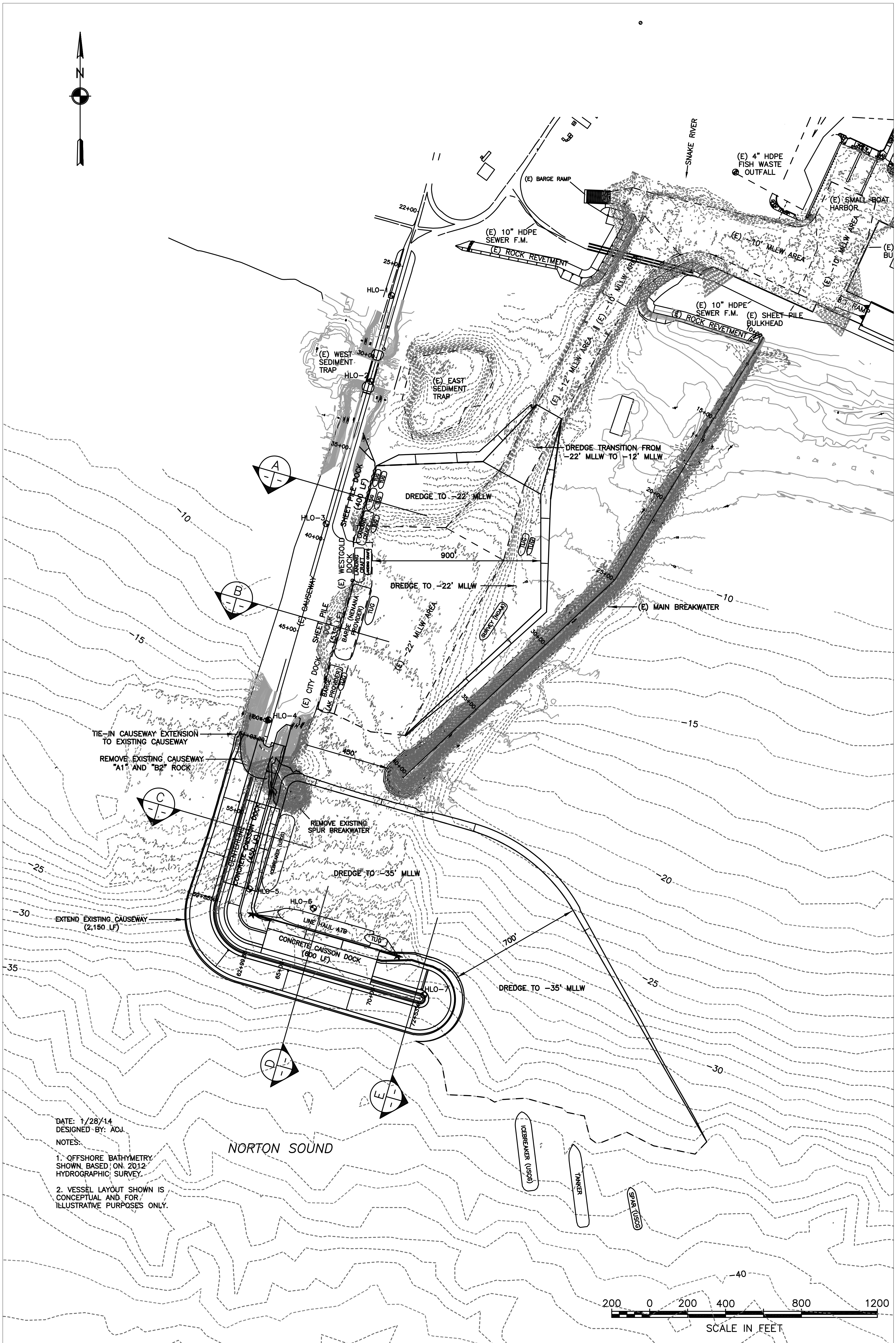
PORT OF NOME ALTERNATIVE 1A AND DREDGED MATERIAL DISPOSAL AREAS
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

1A–5



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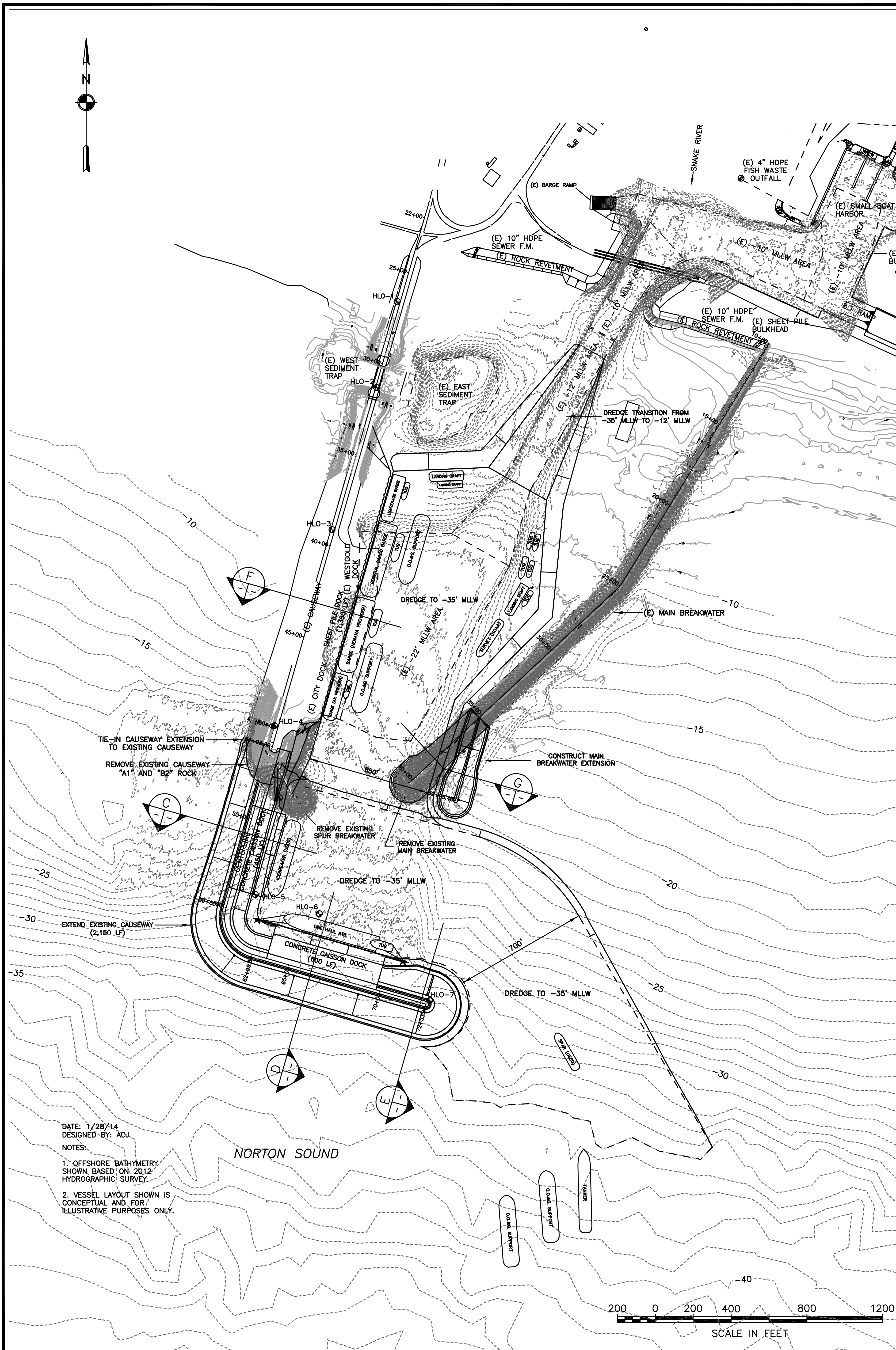
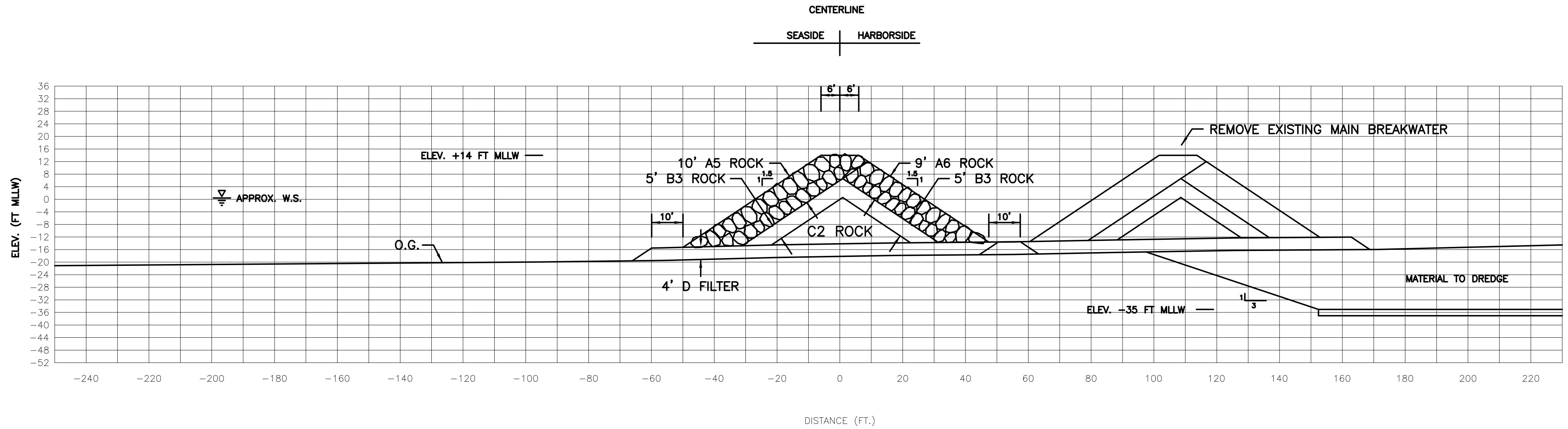
PORT OF NOME ALTERNATIVE 1B
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

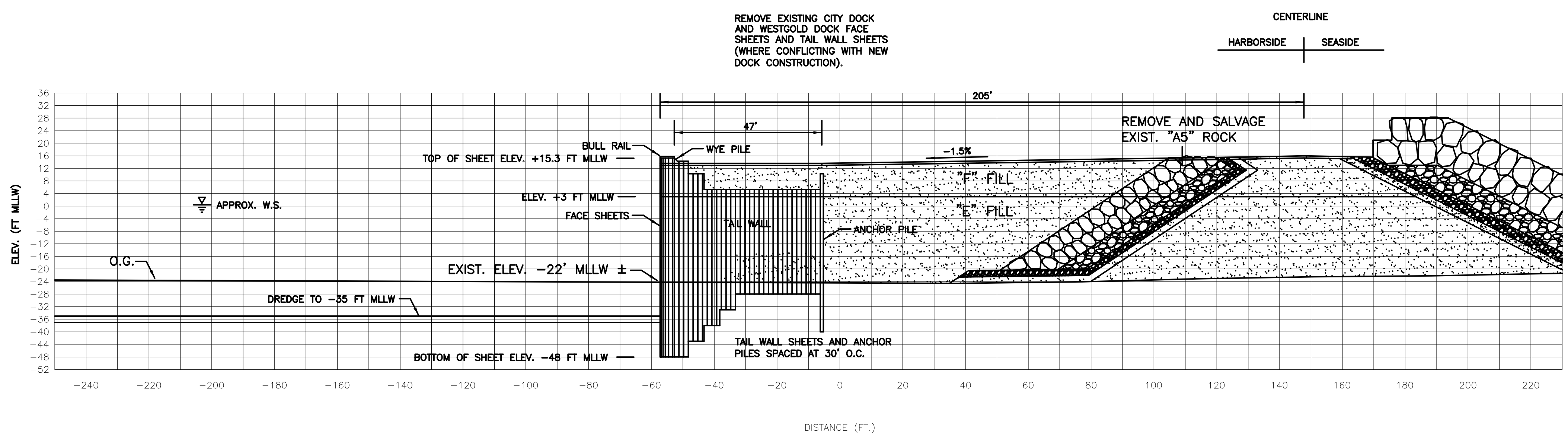
1B–1



ALASKA DISTRICT U.S. ARMY CORPS OF ENGINEERS	PORT OF NOME ALTERNATIVE 1C ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY	1C-1



TYPICAL SECTION – MAIN BREAKWATER TRUNK
STA. 35+18.20 TO 37+75.46



TYPICAL SECTION – NEW DOCK
STA. 35+25± TO 47+15

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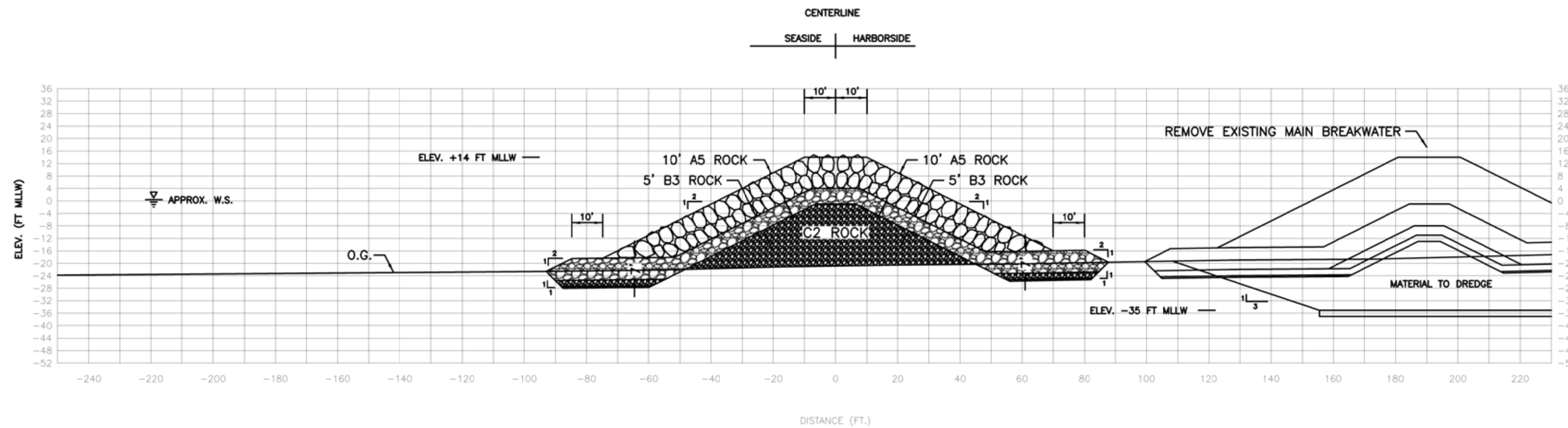
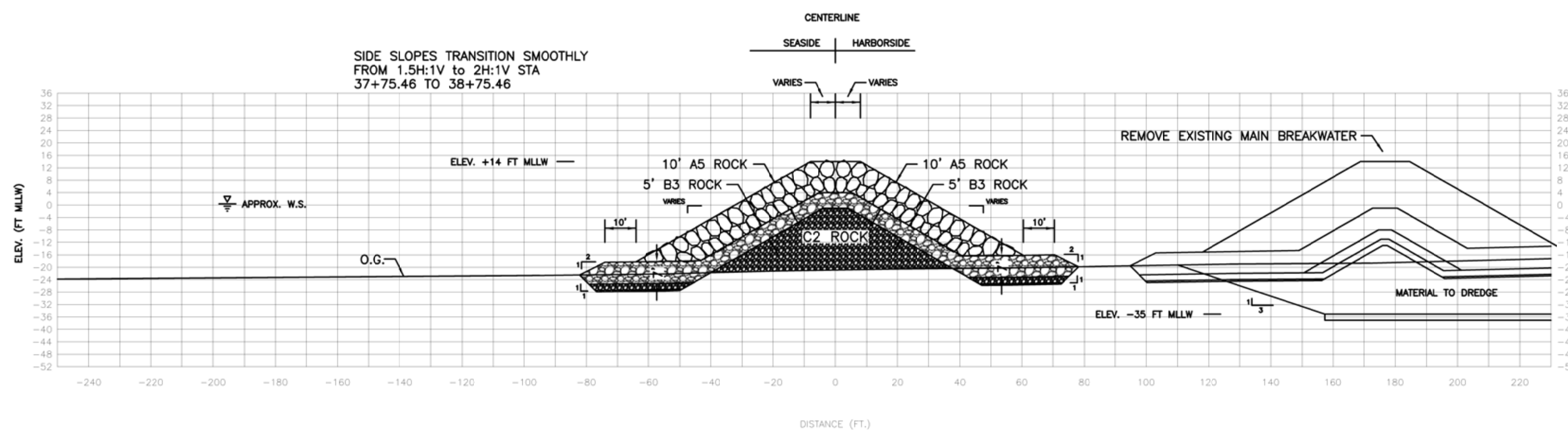
CORPS OF ENGINEERS

PORT OF NOME ALTERNATIVE 1C
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

1C–2



TYPICAL SECTION – MAIN BREAKWATER HEAD
STA. 38+75.46 TO 39+75.46



TYPICAL SECTION – MAIN BREAKWATER TRANSITION
STA. 37+75.46 TO 38+75.46

ALASKA DISTRICT
U.S. ARMY
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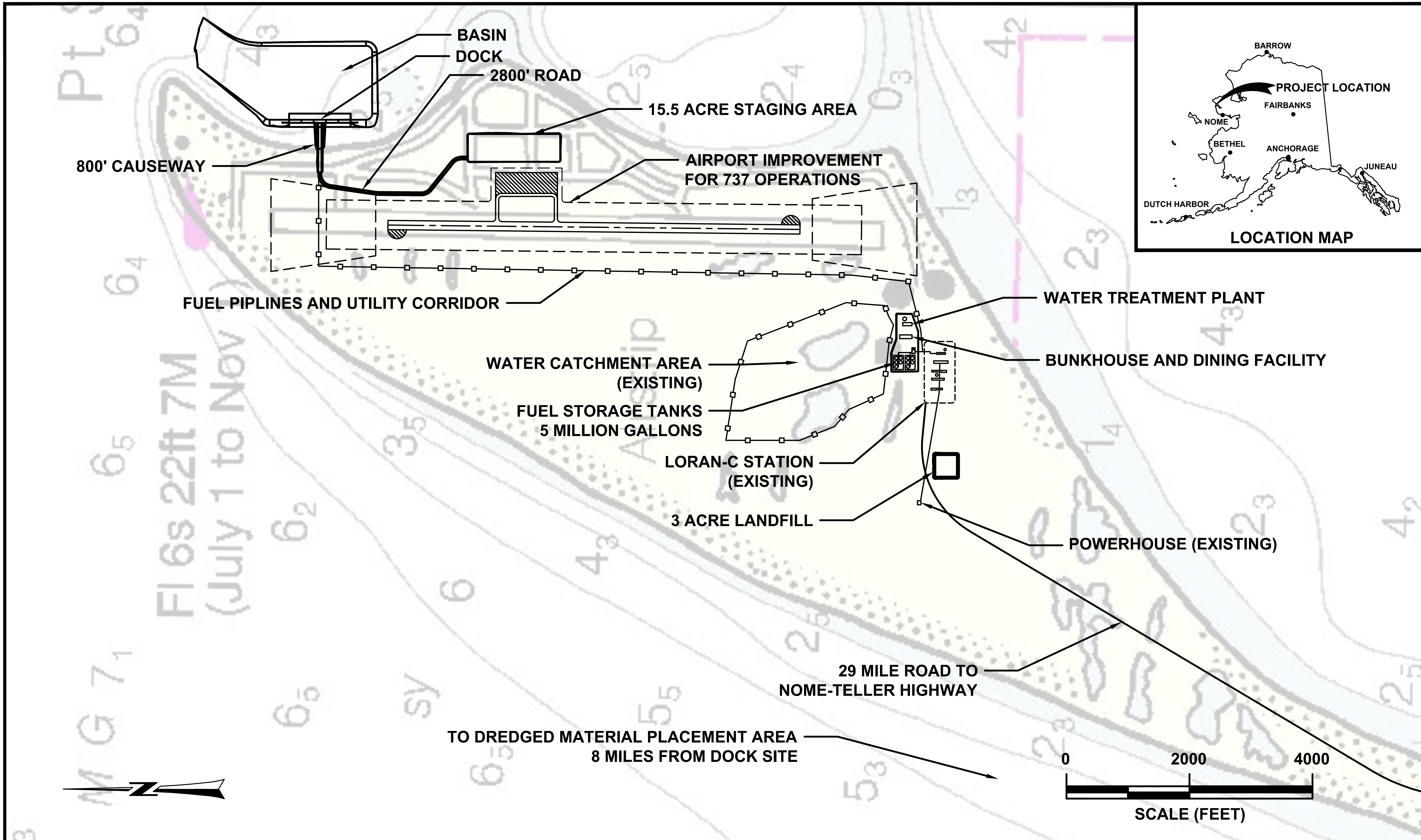
PORT OF NOME ALTERNATIVE 1C
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

1C–3



ALASKA DISTRICT
U.S. ARMY
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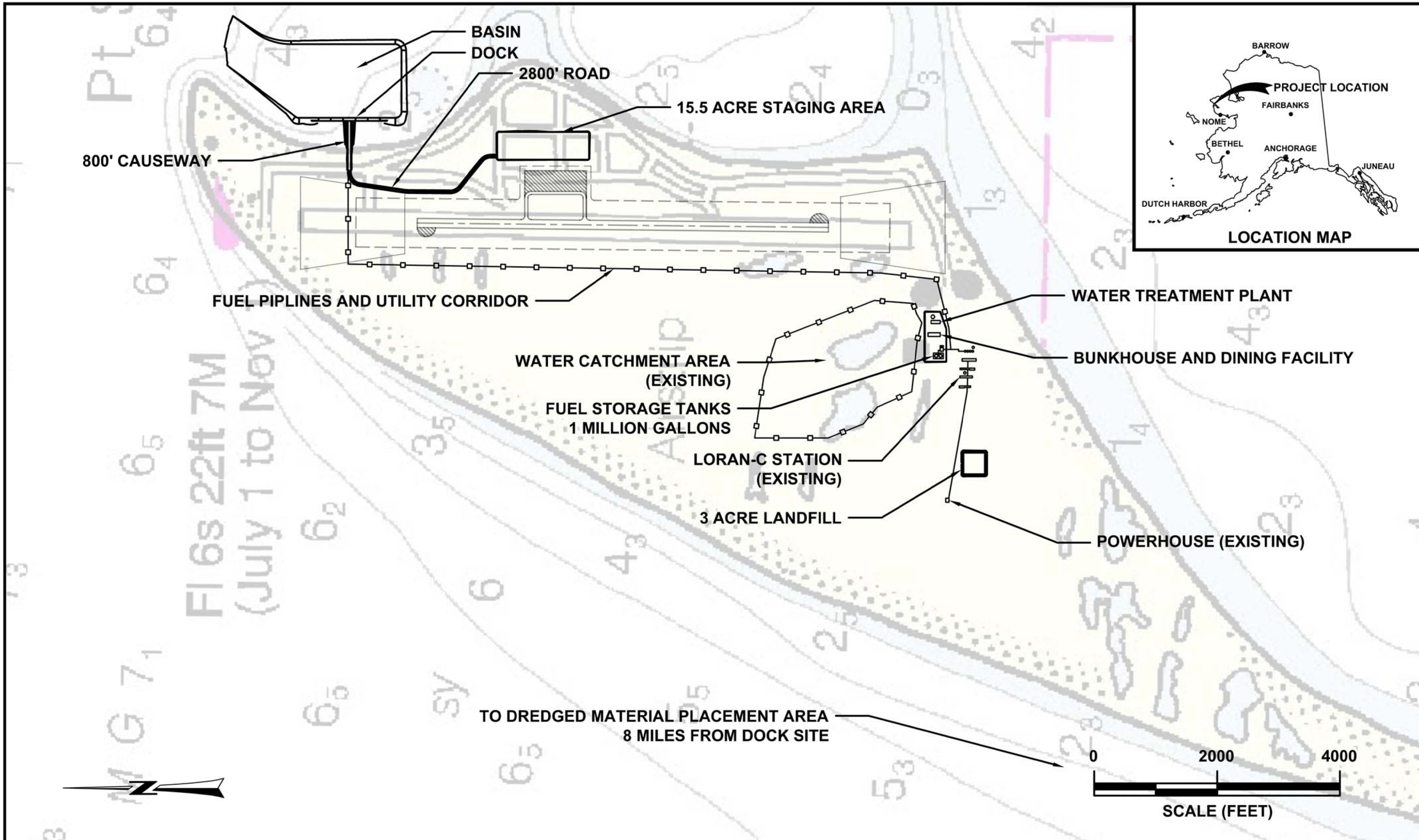
POINT SPENCER HARBOR MASTER PLAN ALTERNATIVES 2a, 2b AND 2c
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

S-1



ALASKA DISTRICT
U.S. ARMY
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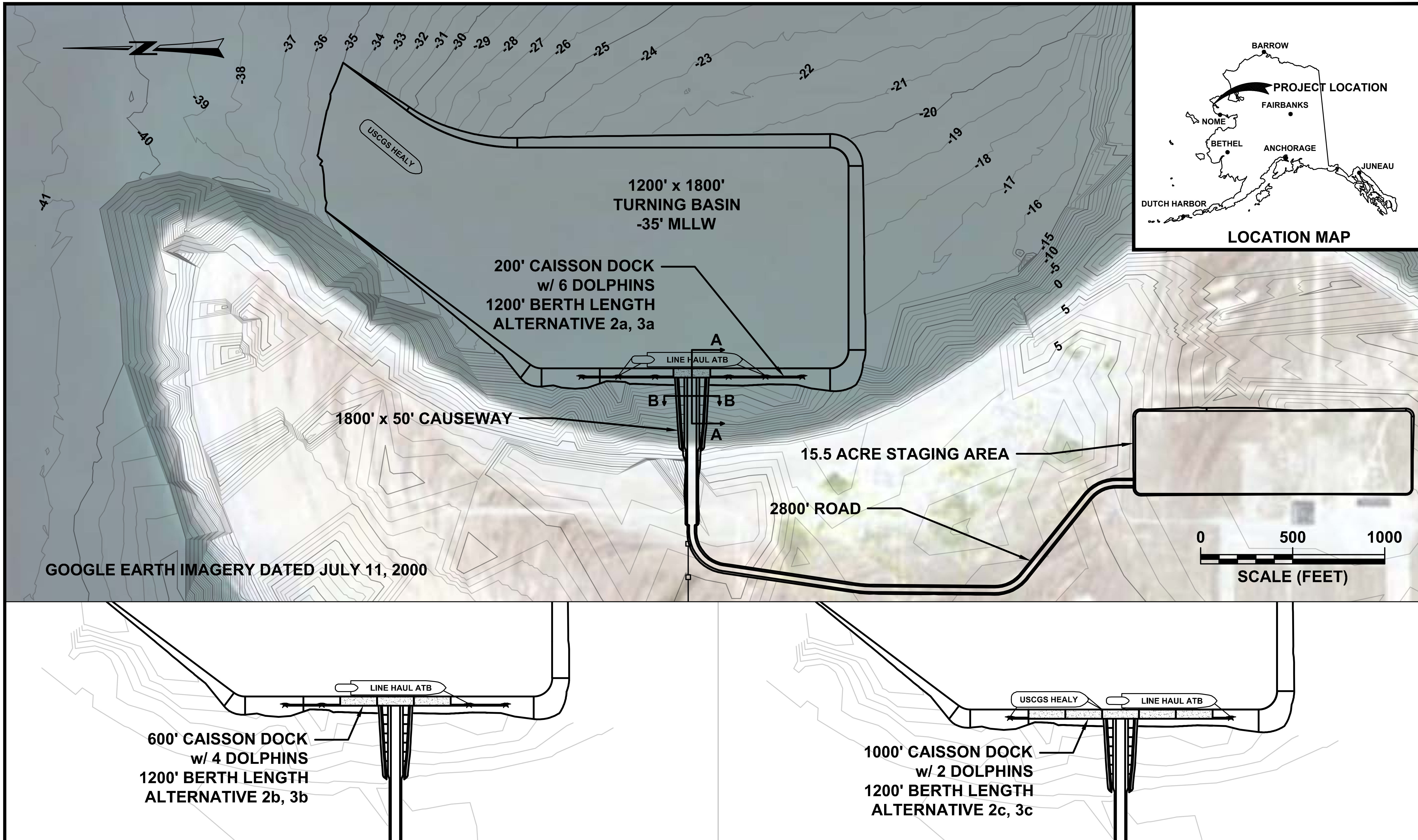
POINT SPENCER DOCK MASTER PLAN ALTERNATIVE 3a, 3b, AND 3c

ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

S-2



ALASKA DISTRICT
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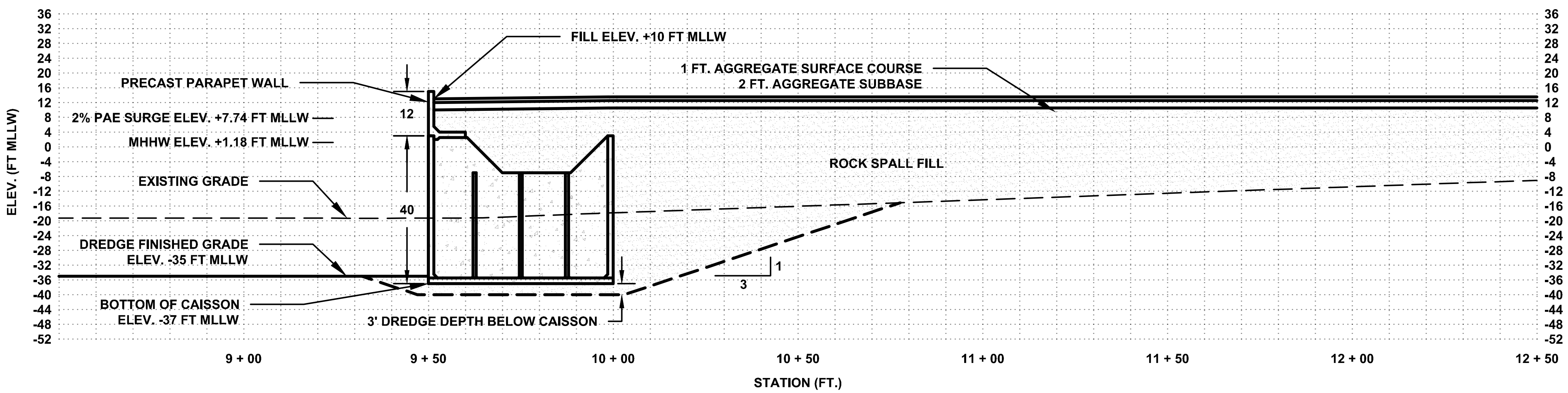
POINT SPENCER BASIN AND DOCK PLAN

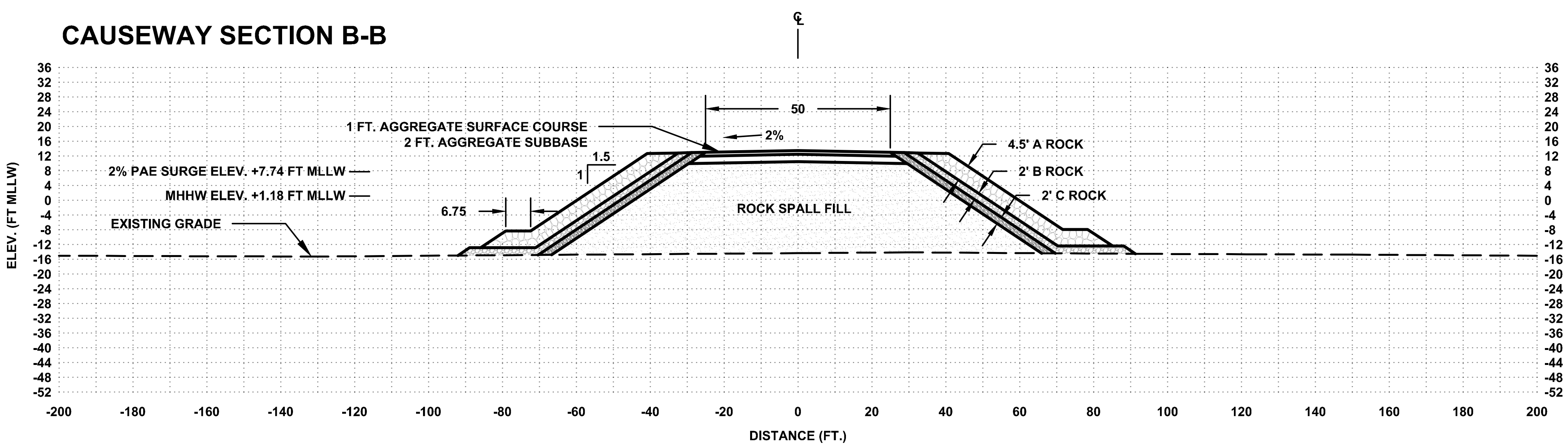
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

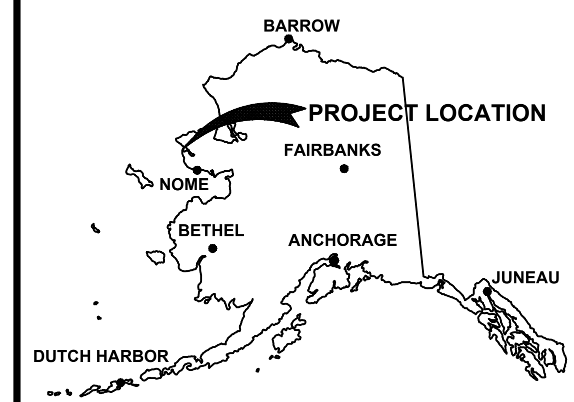
S-

DEEP DRAFT DOCK SECTION A-A



CAUSEWAY SECTION B-B





LOCATION MAP

NOTES:

DEFINITIONS & ACRONYMS:

PAE - PERCENT ANNUAL EXCEEDANCE

CAUSEWAY GRADATIONS:

A ROCK

WEIGHT (LB)	PERCENT SMALLER (%)
2,250	100
1,800	50
1,350	0

B ROCK

WEIGHT (LB)	PERCENT SMALLER (%)
234	100
180	50
126	0

C ROCK

WEIGHT (LB)	PERCENT SMALLER (%)
14	100
9	50
5	0

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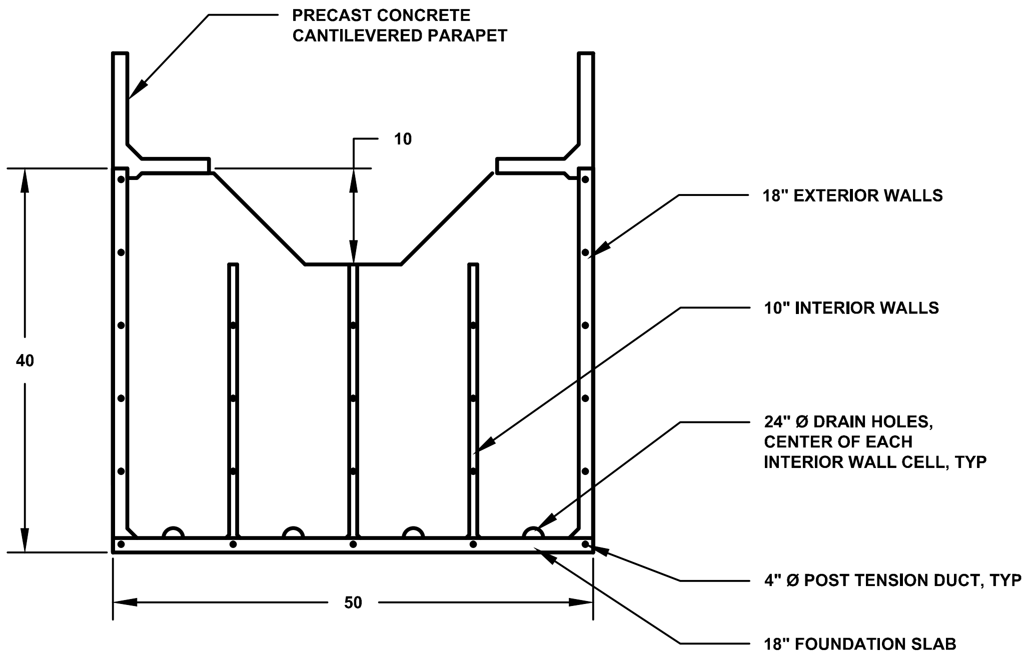
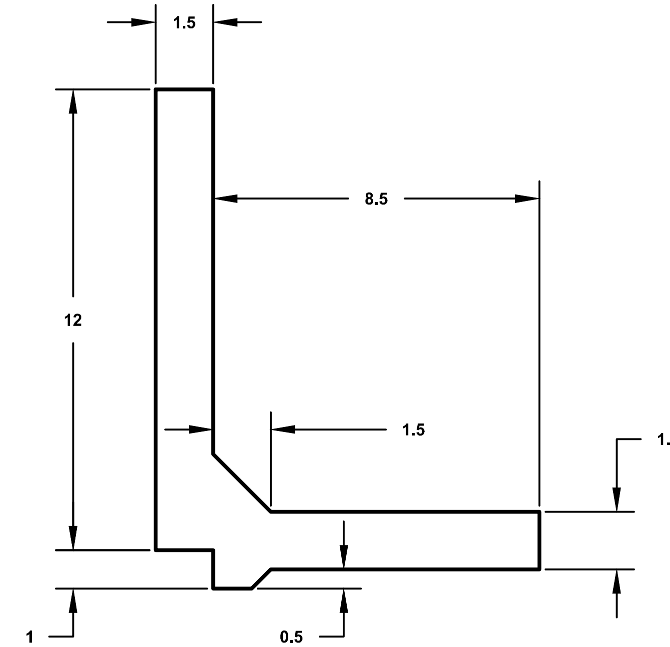
POINT SPENCER DOCK AND CAUSEWAY SECTIONS

ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

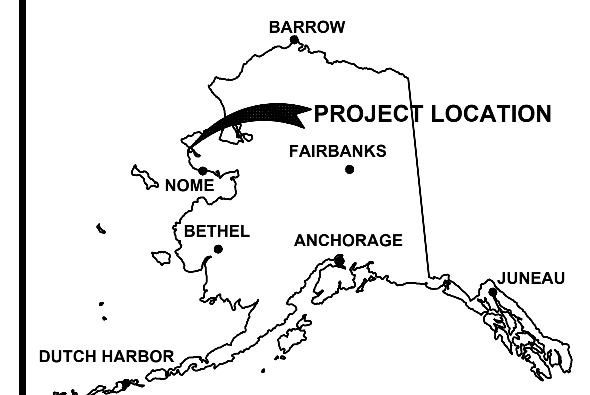
S-



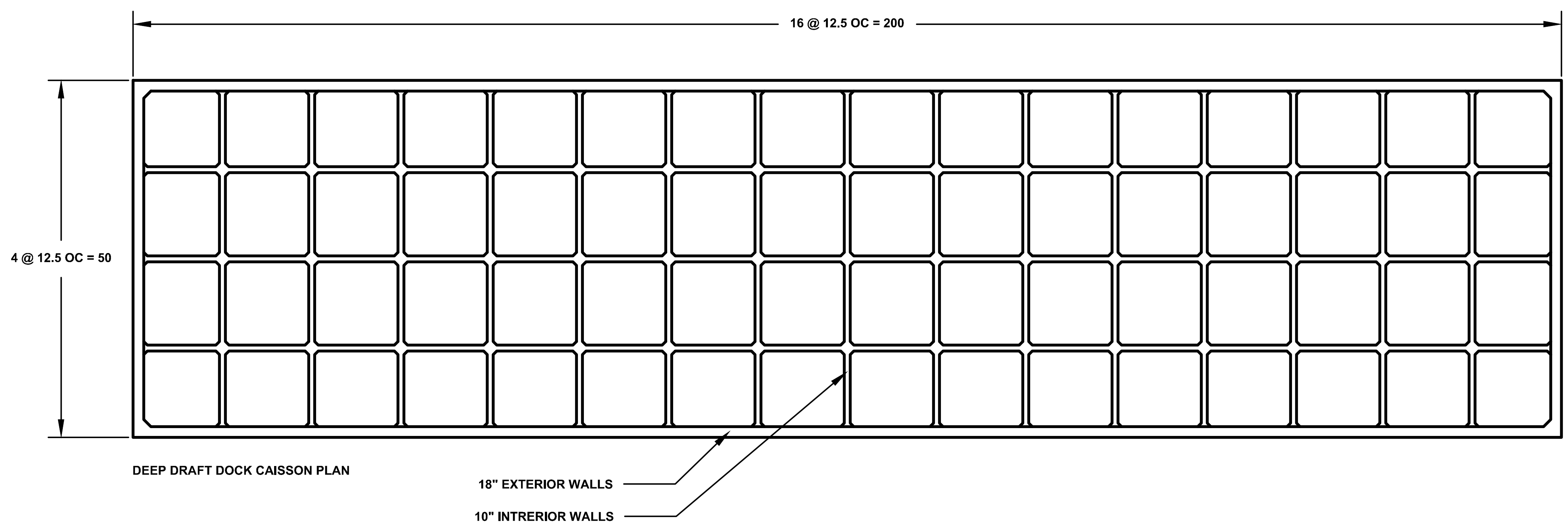
DEEP DRAFT DOCK CAISSON SECTION



PRECAST CANTILEVERED PARAPET



LOCATION MAP



DEEP DRAFT DOCK CAISSON PLAN

NOTES:

CONCRETE
f'c = 8000 psi

DEFORMED BARS
GRADE 60, GALVANIZED
EXTERIOR WALLS & KEEL SLAB - 1 MATS No.14 @ 18" OC EACH WAY
INTERIOR WALLS - 1 MAT No. 11 @ 18" OC EACH WAY

POST TENSION STRANDS
(19) 5/8"Ø STRANDS @ EACH DUCT
3 DUCTS EACH INTERIOR WALL
5 DUCTS EACH EXTERIOR WALL
6 DUCTS EACH KEEL SLAB 19 5/8"Ø STRANDS @ EACH DUCT

TRANSPORT - TOW TO SITE. ATTACH 4" THICK CONCRETE DECK SLAB (NOT SHOWN) OVER CAISSON FOR TRANSPORT, REMOVE PRIOR TO PLACEMENT . DECK SLABS TO REMAIN ON SITE.

PARAPET WALL
TONGUE AND GROOVE CONNECTION ALONG VERTICAL WALL EDGES.
FILL CAISSONS TO TOP OF INTERIOR WALLS
MINIMUM COMPACTION OF FILL 100% OF LABORATORY MAXIMUM DENSITY TO ONE FOOT BELOW THE WALL.
PREPARE BEDDING SURFACE OF PARAPET WALL TO A SURFACE TOLERANCE OF +/- 1 INCH OVER A 20 FOOT LENGTH.

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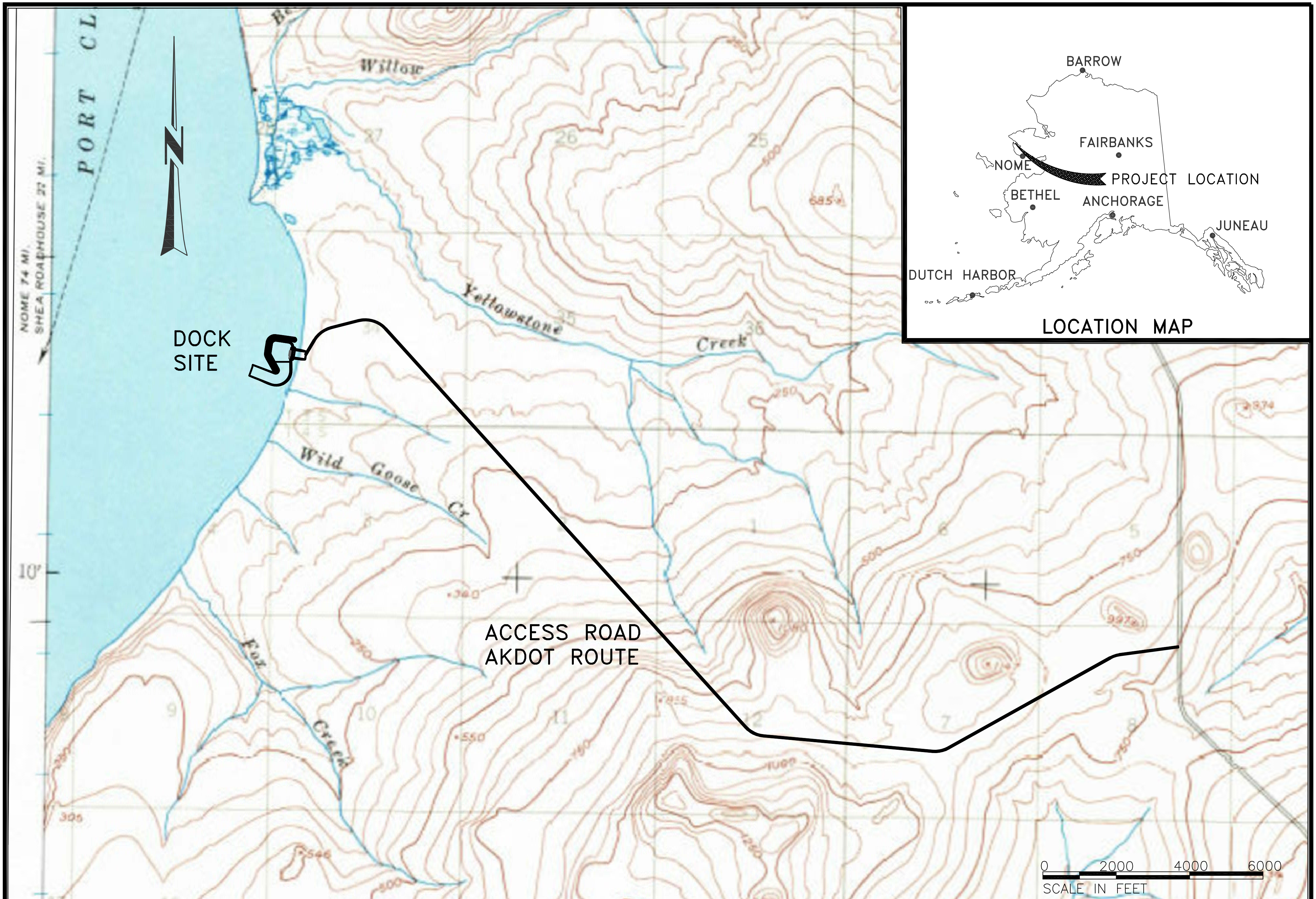
POINT SPENCER DOCK CONCRETE CAISSON AND PARAPET WALLS
ALASKA DEEP DRAFT ARCTIC PORT FEASIBILITY STUDY

S-



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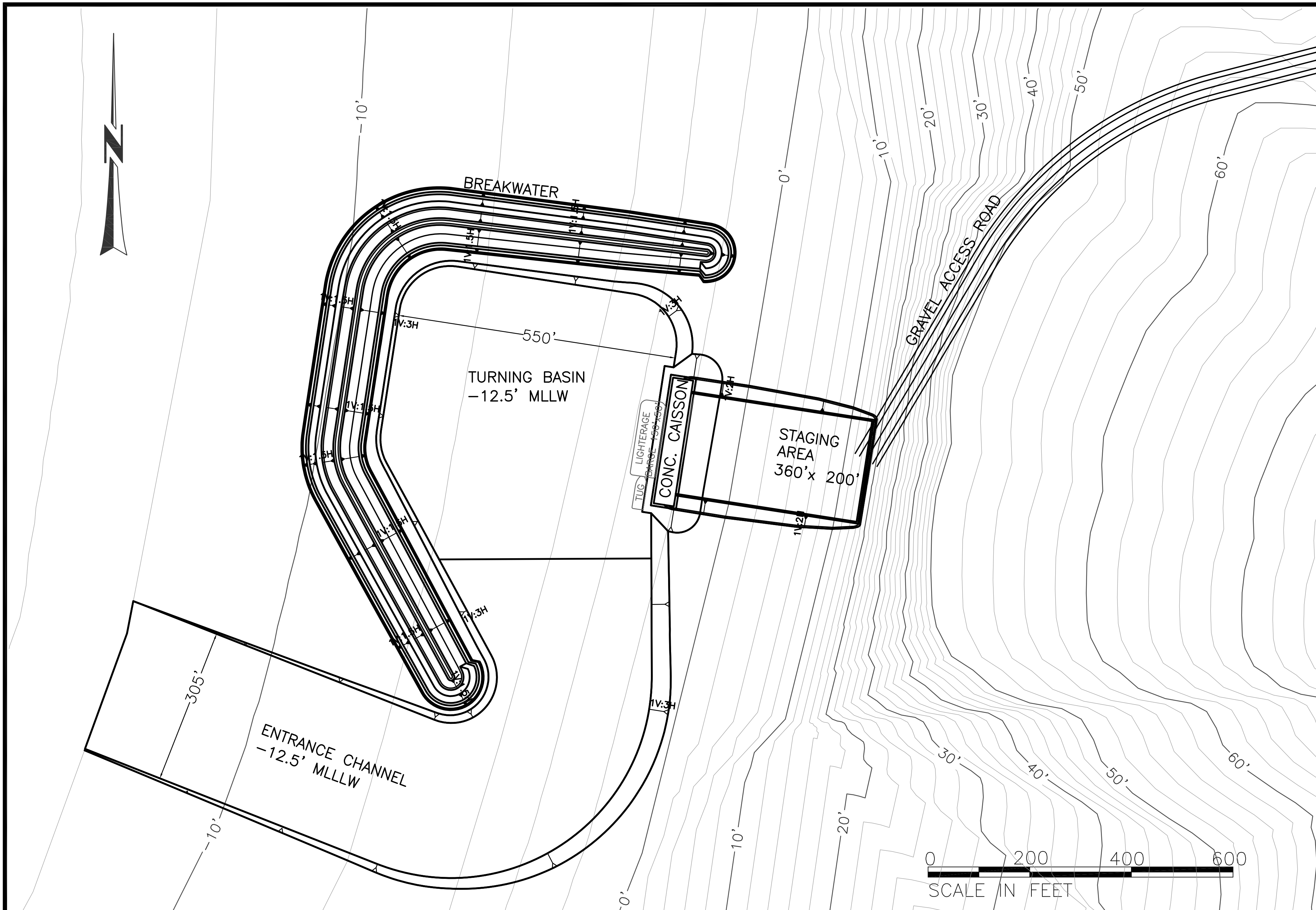
CAPE RILEY SHALLOW-DRAFT MINERAL EXTRACTION DOCK
ARCTIC DEEP DRAFT FEASIBILITY STUDY

R-1



ALASKA DISTRICT
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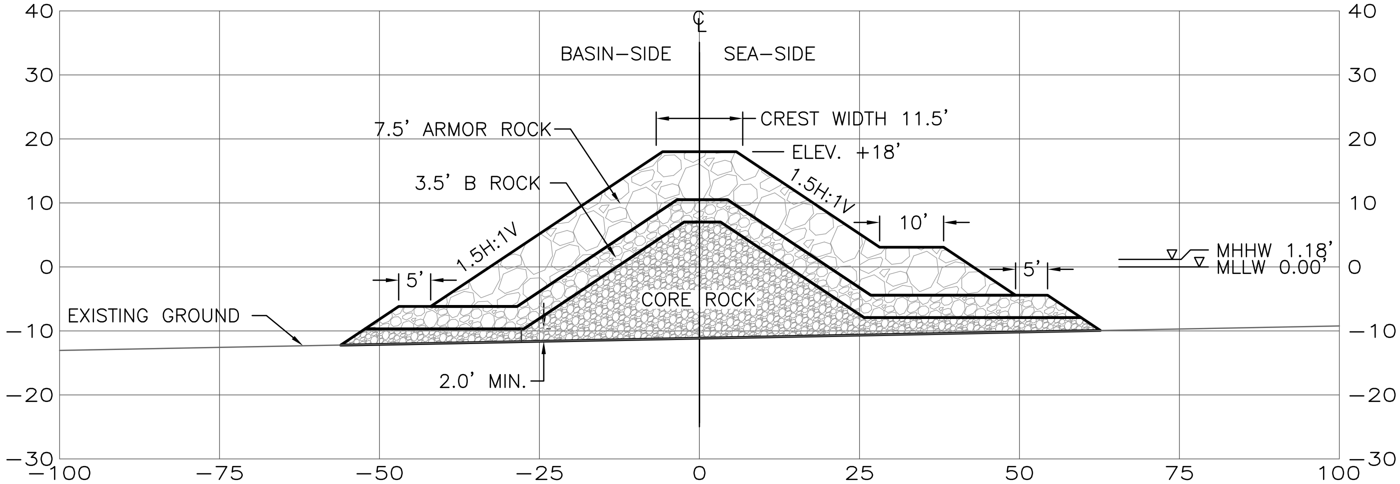
CAPE RILEY BARGE DOCK LAYOUT

ARCTIC DEEP DRAFT FEASIBILITY STUDY

R-2



TYPICAL BREAKWATER SECTION

1. SECTION VERTICAL UNITS ARE FEET MEAN LOWER LOW WATER (MLLW).
2. SECTION HORIZONTAL UNITS ARE REFERENCE PERPENDICULAR TO THE BREAKWATER CENTERLINE.
3. BREAKWATER ARMOR ROCK, B ROCK AND CORE ROCK SLOPES ARE 1.5H:1V UNLESS NOTED OTHERWISE.

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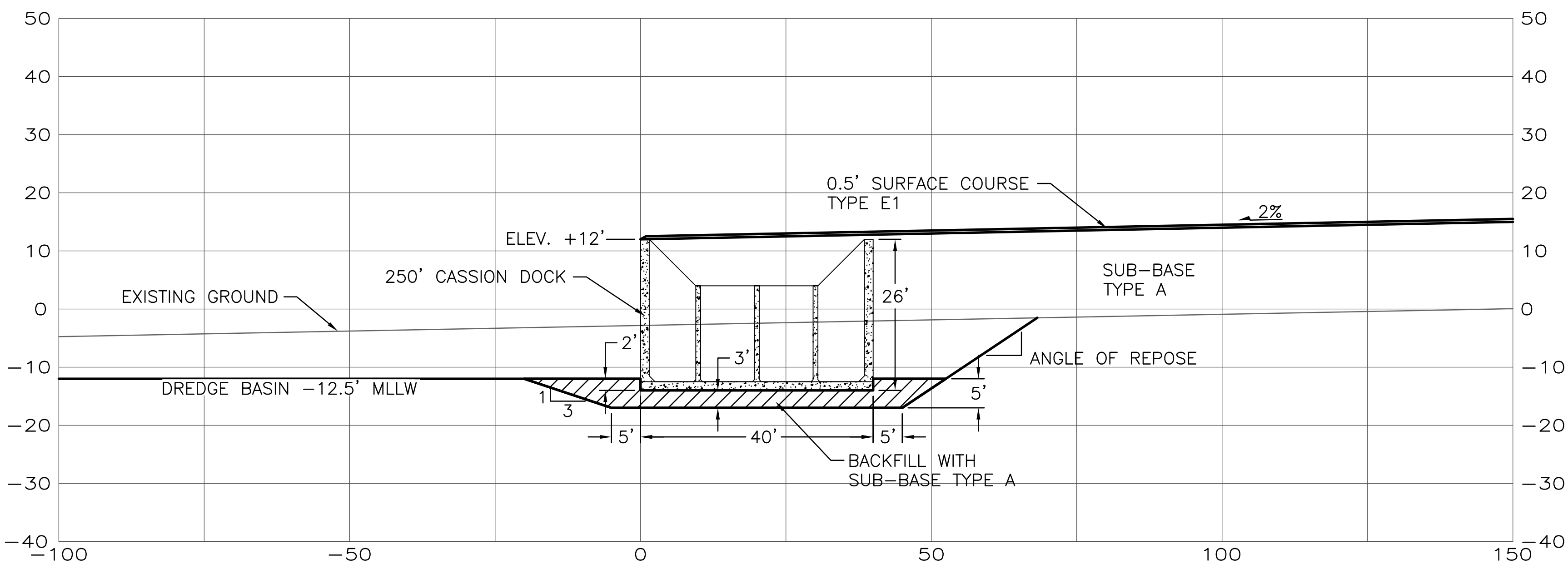
CAPE RILEY BREAKWATER SECTION

ARCTIC DEEP DRAFT FEASIBILITY STUDY

R–3



TYPICAL DOCK SECTION

1. SECTION VERTICAL UNITS ARE FEET MEAN LOWER LOW WATER (MLLW).
2. SECTION HORIZONTAL UNITS ARE REFERENCE PERPENDICULAR TO THE DOCK FACE.

ALASKA DISTRICT
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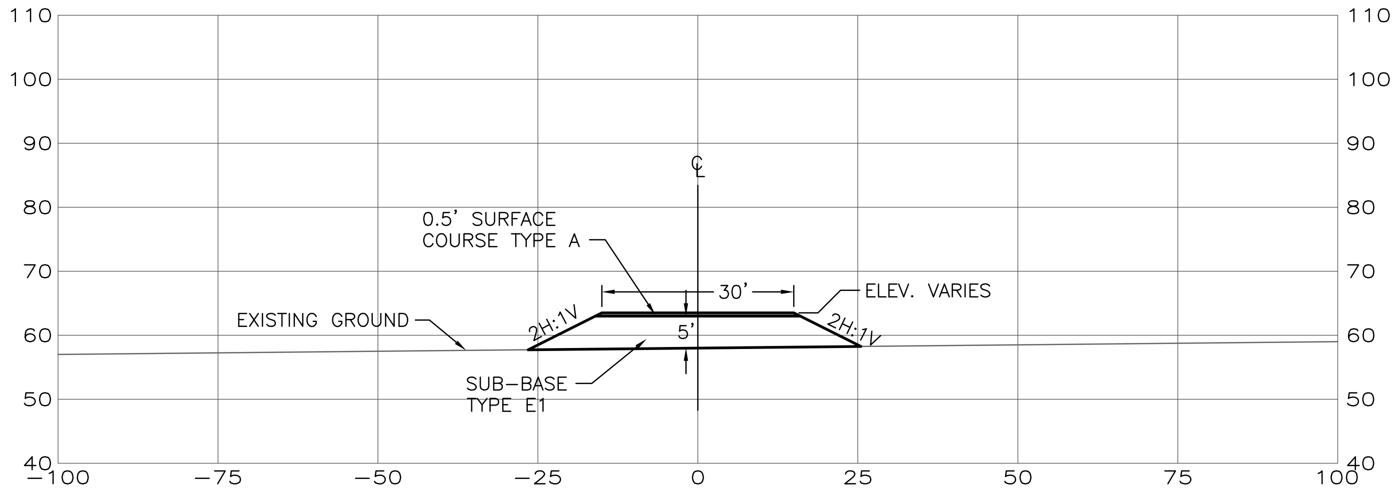
CORPS OF ENGINEERS

CAPE RILEY DOCK SECTION

ARCTIC DEEP DRAFT FEASIBILITY STUDY

R-4



TYPICAL ROAD SECTION

1. SECTION VERTICAL UNITS ARE FEET MEAN LOWER LOW WATER (MLLW).
2. SECTION HORIZONTAL UNITS ARE REFERENCE PERPENDICULAR TO THE NORTH AND SOUTH BREAKWATER ALIGNMENTS.

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CAPE RILEY ROAD SECTION

ARCTIC DEEP DRAFT FEASIBILITY STUDY

R-5