APPENDIX A – HYDRAULIC DESIGN
NAVIGATION IMPROVEMENTS
HAINES, ALASKA

CONTENTS

TABLES

FIGURES

1.0 INTRODUCTION

1.1 Appendix Purpose

This hydraulic design appendix describes the technical aspects of the Haines Navigation Improvements project. It provides the background for determining the Federal interest in the major construction features including breakwaters, dredging, and operation and maintenance.

1.2 Project Purpose

The city of Haines requested the Corps of Engineers to conduct a feasibility study of navigation improvements. Additional demand for vessel moorage was identified as a critical issue facing the community. The following objectives were identified to accomplish navigation improvements at Haines prior to initiating the engineering analysis:

a. Prevent overcrowding in the existing harbor by providing a larger, safer, and more efficient moorage area for the fleet.

b. Provide additional moorage for commercial fishing vessels that have been on the waiting list for mooring space for many years.

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

2.0 CLIMATOLOGY, METEOROLOGY, HYDROLOGY

2.1 Climatology

Haines is on the Chilkat Peninsula, approximately 145 water-kilometers northwest of Juneau and 800 air-kilometers southwest of Anchorage. Portage Cove and the contiguous marine waters are at latitude 59°14' N and longitude 135°26' W as shown on figure A-1. The cove opens to the Chilkoot Inlet toward the east. The Chilkat River lies to the west of the city of Haines. The area has a maritime climate primarily influenced by strong low-pressure centers generated in the Gulf of Alaska and eastern Pacific Ocean. Cool summers, mild winters, and year-round rainfall characterize the climate. Snow falls primarily between November and April and the average annual snowfall is 335 centimeters (cm). Rains may occur any time of the year, and annual average precipitation per year is 154 cm. The wettest months occur in the fall with October and November having the highest monthly and record rainfall. Fog is generally uncommon but can occur under certain conditions in both summer and winter. Normal winter temperature ranges from –5.0 °C to +2.2 °C, while summer temperatures range from +7.8 °C to +18.9 °C. Temperatures can reach record lows of –26.7 °C and record highs of +32.0 °C.

2.2 Wind Data

Predominant winds at Haines are generally caused by low-pressure systems that track in an easterly direction across the eastern Pacific Ocean and Gulf of Alaska. Strong winds occur throughout the year; however, wind patterns have a strong seasonal component. Summer winds are generally from the south-southeast and are lighter. Winter winds are predominantly from the north and are generally stronger. Historical wind speed and direction data are summarized in the wind roses shown in figures A-2 through A-14. The Haines area as with most of southeast Alaska is known for intense storms that occur from the south-southeast direction. According to local residents, the severe and damage-causing storms usually occur in the fall and come from the south-southeast direction. High winds and waves have caused problems in the existing boat harbor at Portage Cove under such conditions. Local residents have estimated wind speeds to be a sustained 65 to 80 kilometers per hour (km/hr) during major storms. Gusts of up to 175 km/hr have been observed.

A wind data summary was prepared by the National Weather Service for a twenty- two year period of record taken between 1925 and 1947 and was analyzed by the Corps of Engineers in 1974 for a harbor improvements study. The data was non-directional, and was analyzed for wind speed only. A frequency table was developed which gave wind velocity ranges versus percentage of occurrence (COE 1974). An estimate of the 50- year wind speed used for design purposes was 83 km/hr. The principal wind direction considered was the south-southeast.

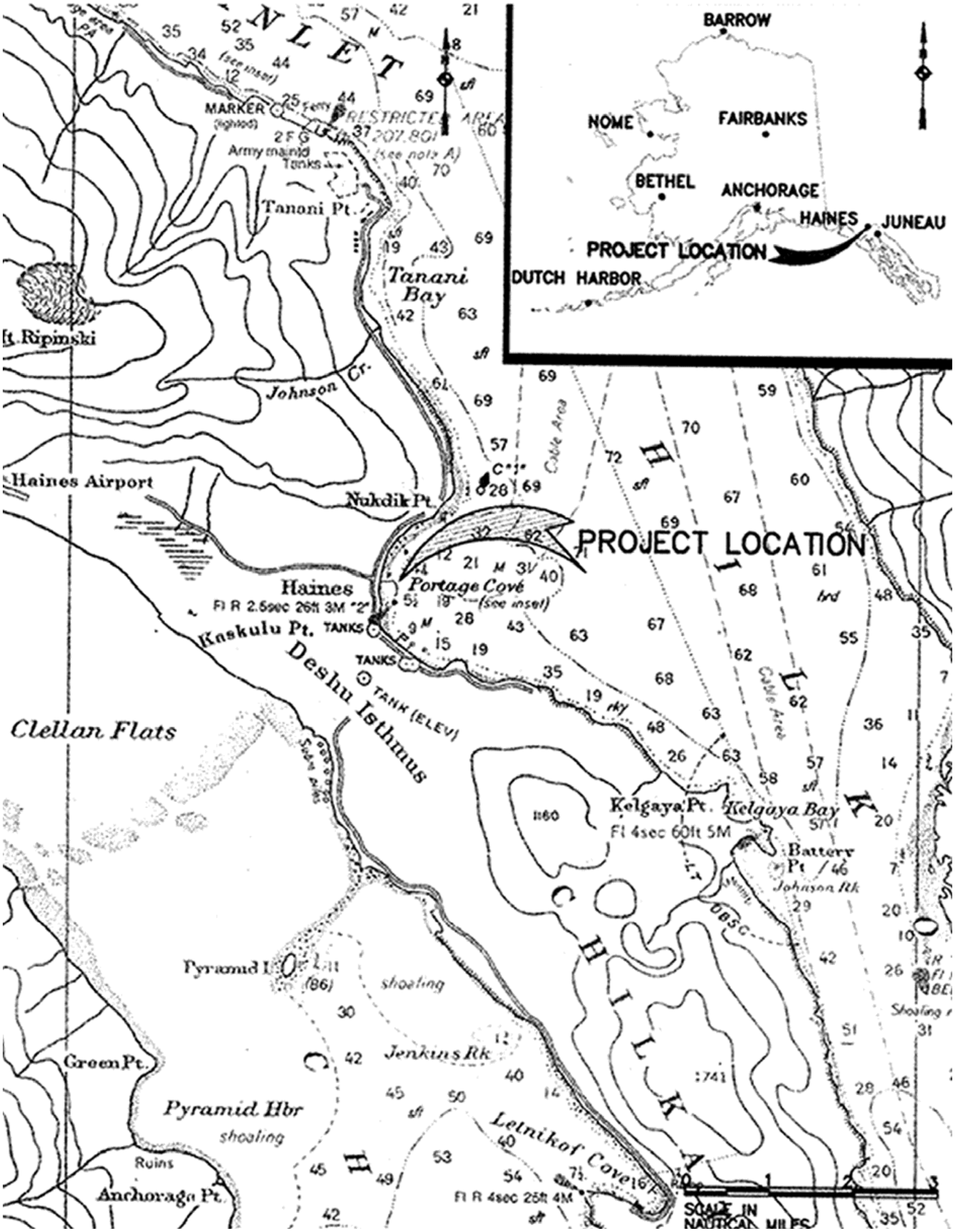


Figure A-1. Location/Vicinity Map

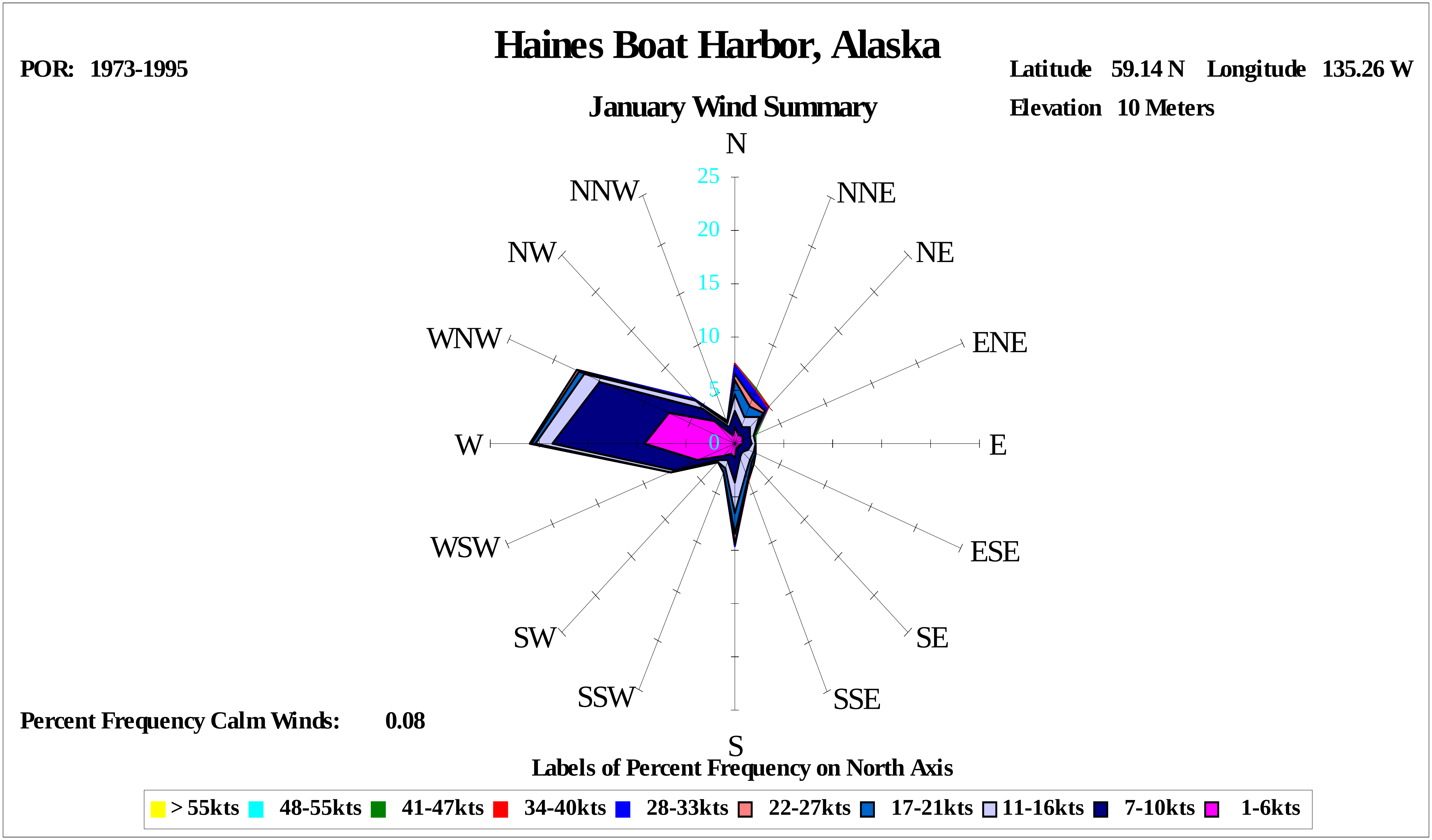


Figure A-2. Wind Rose

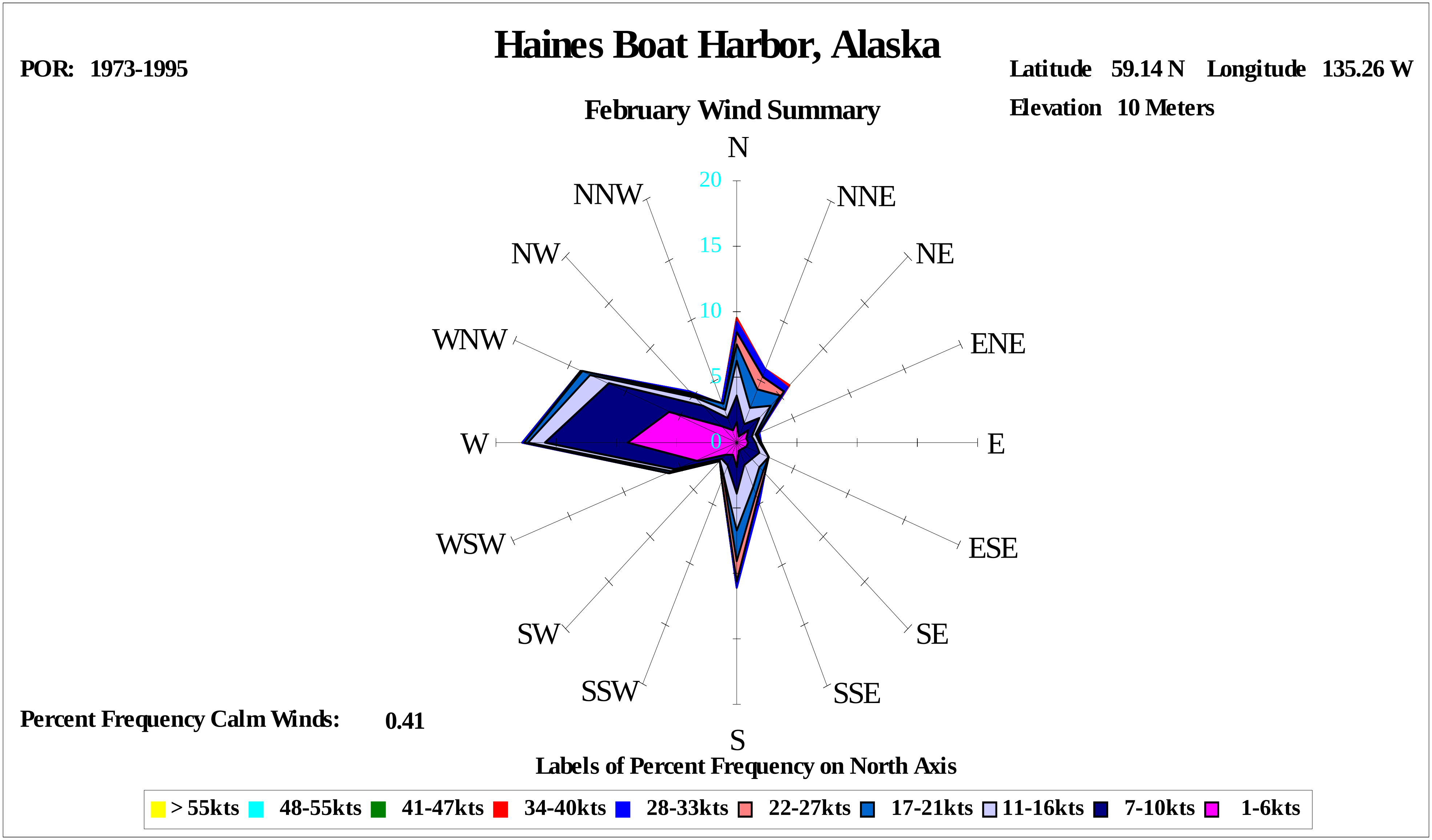


Figure A-3. Wind Rose

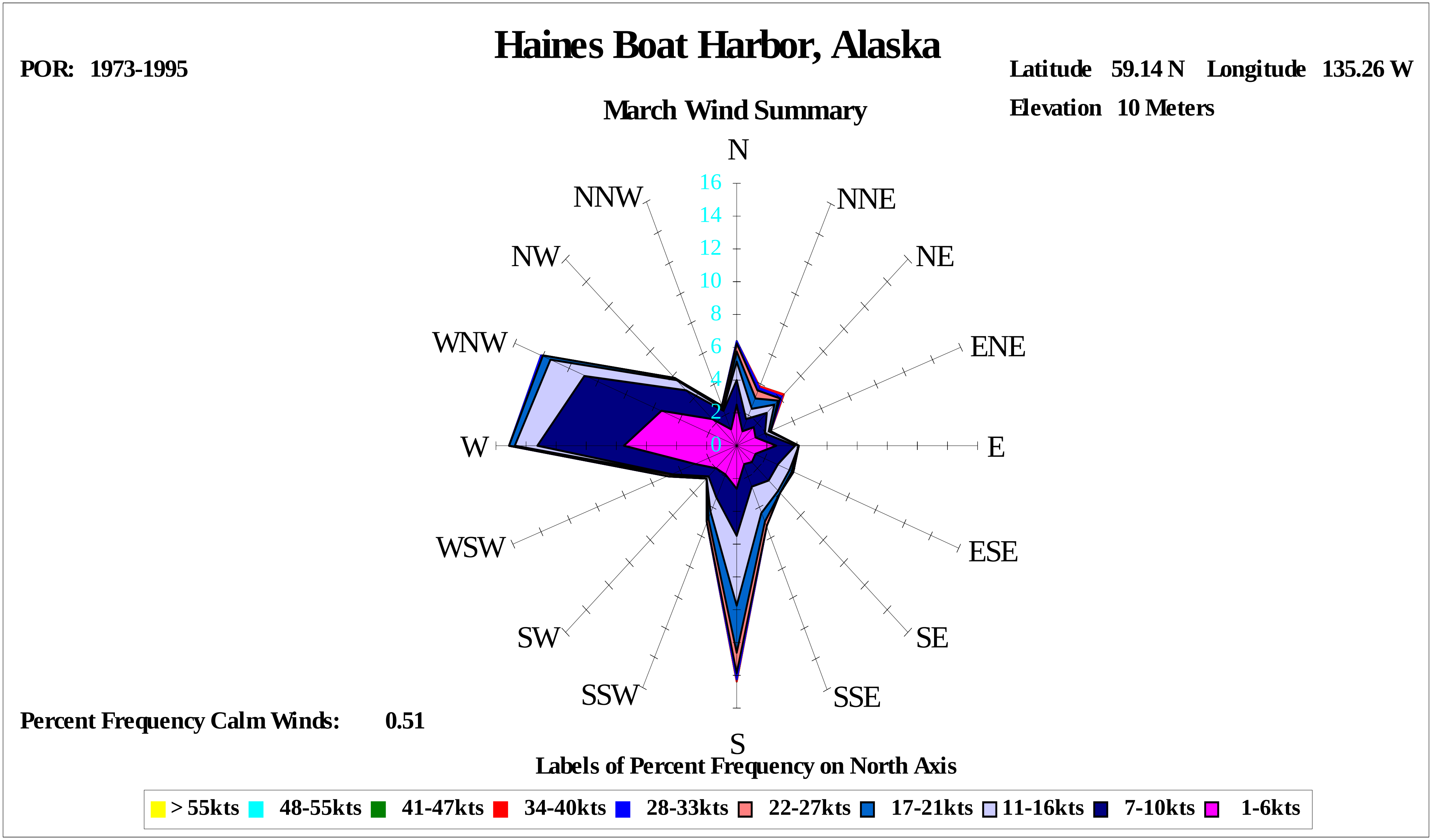


Figure A-4. Wind Rose

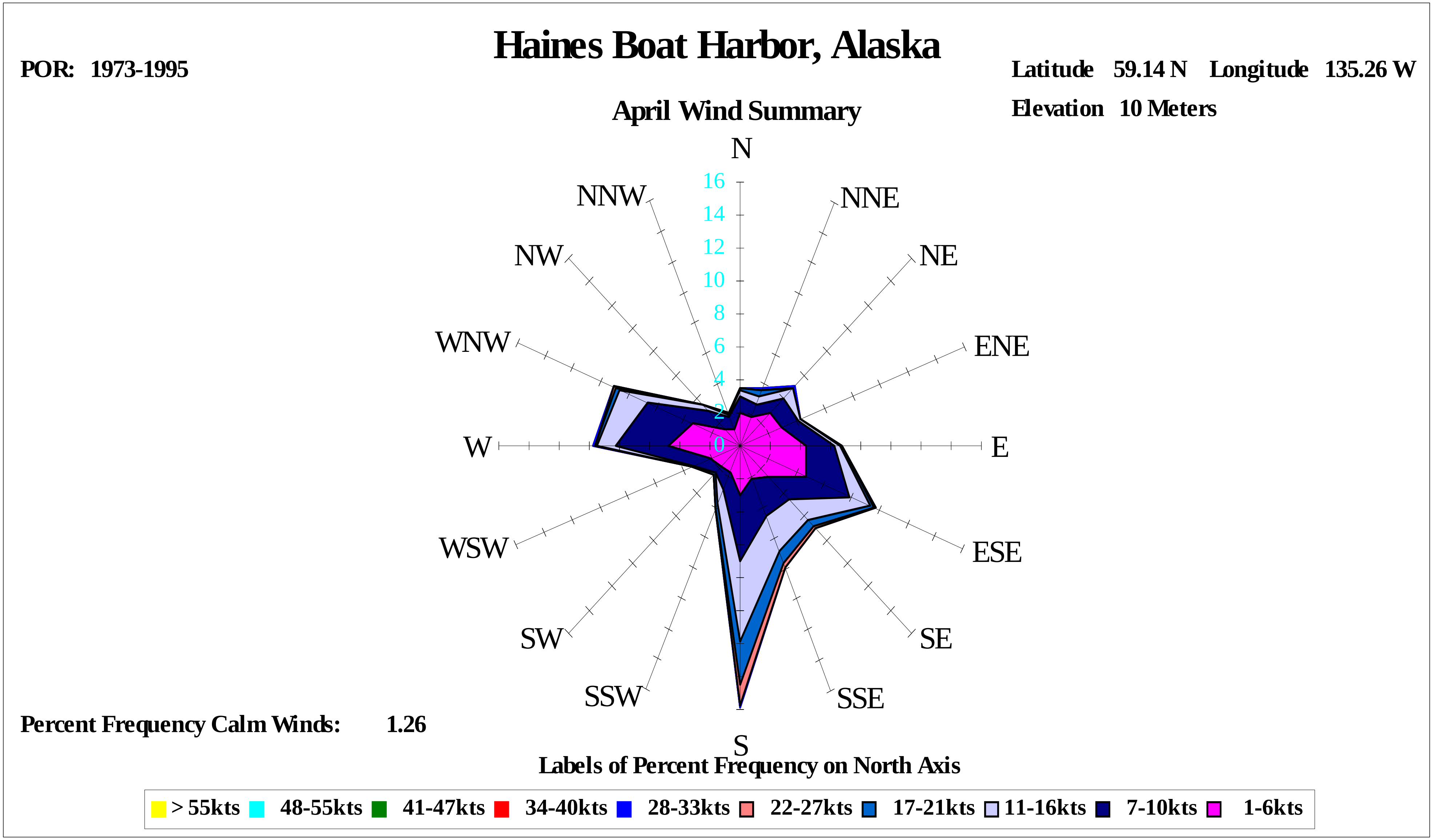


Figure A-5. Wind Rose

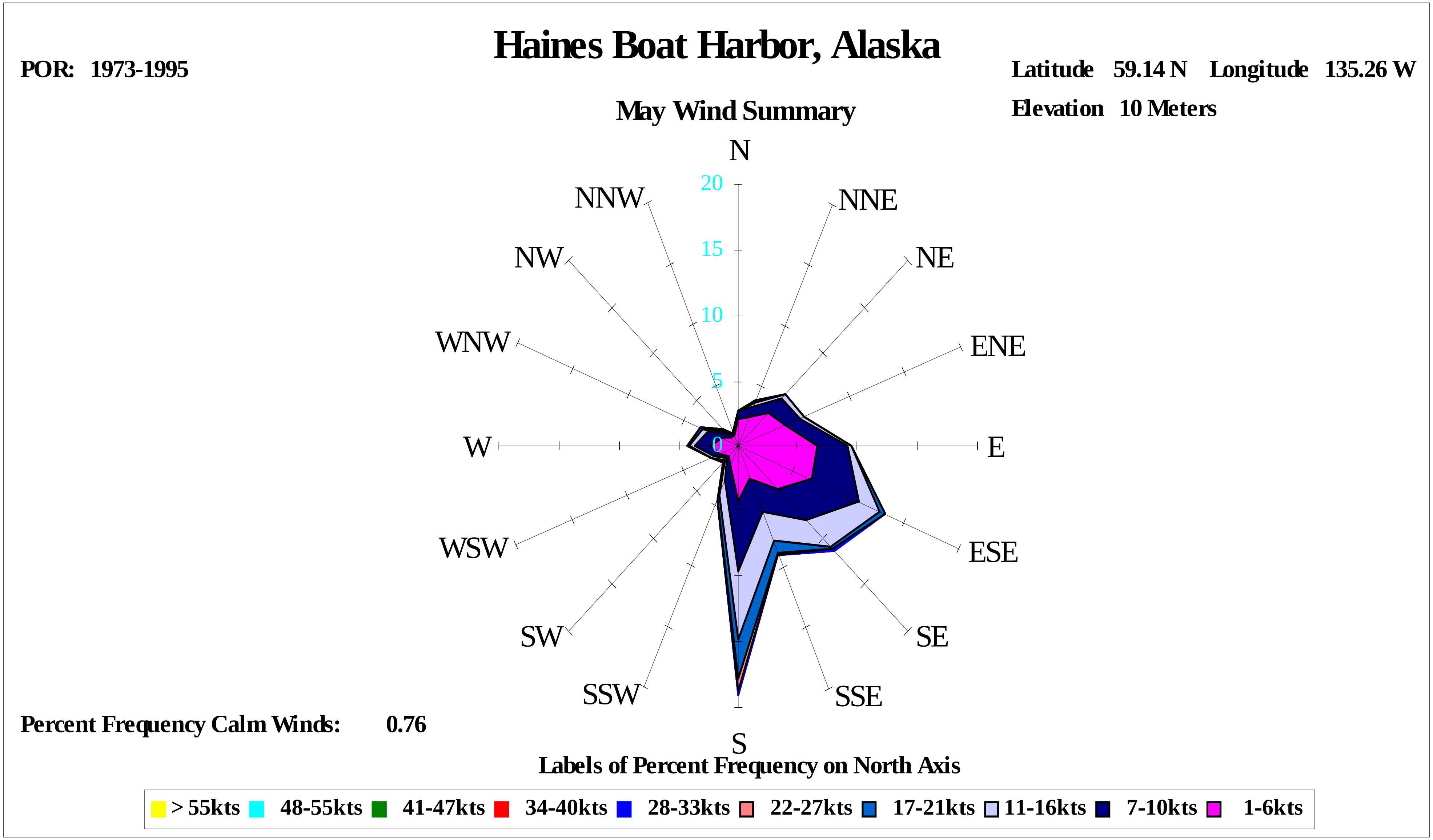


Figure A-6. Wind Rose

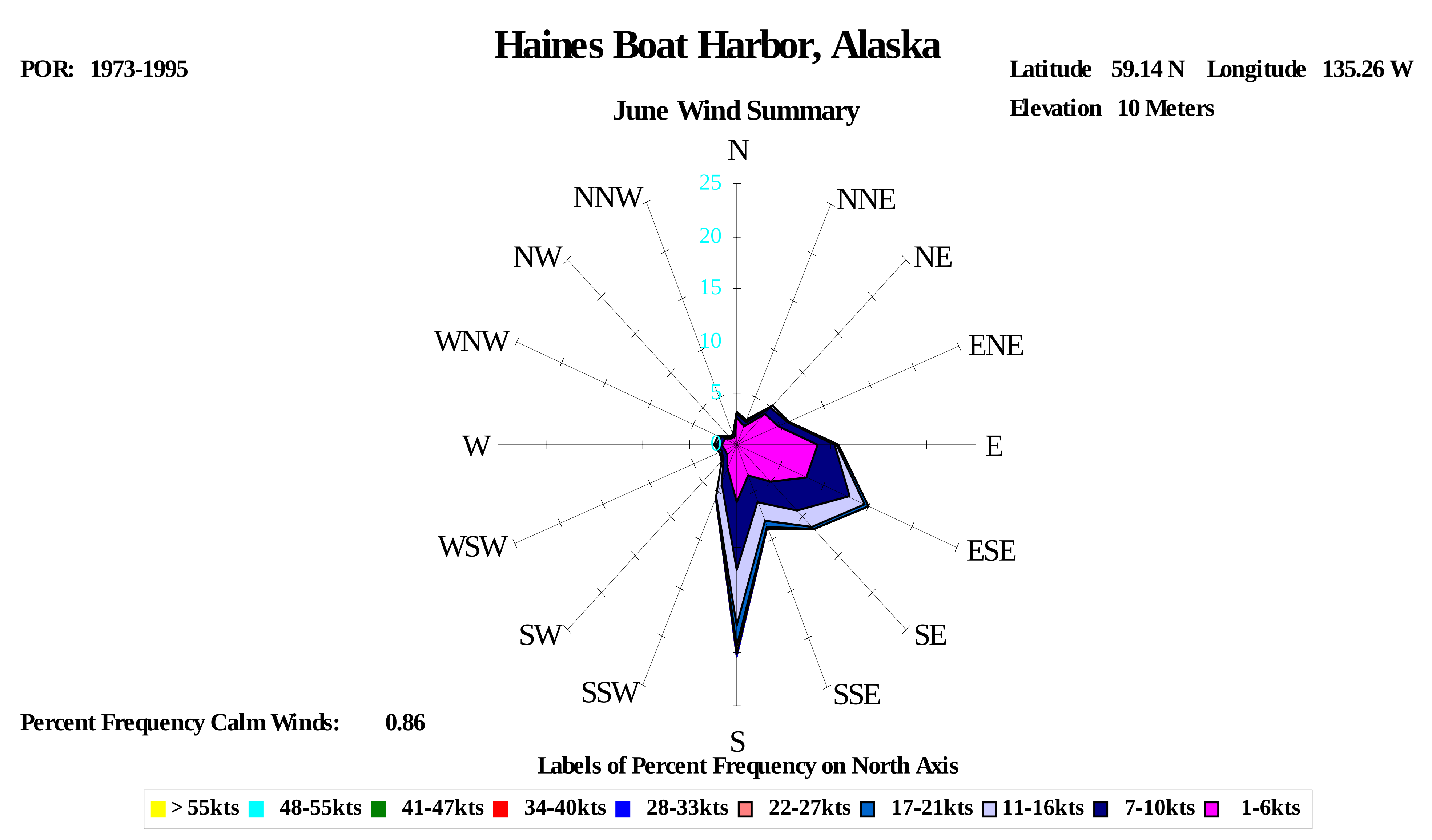


Figure A-7. Wind Rose

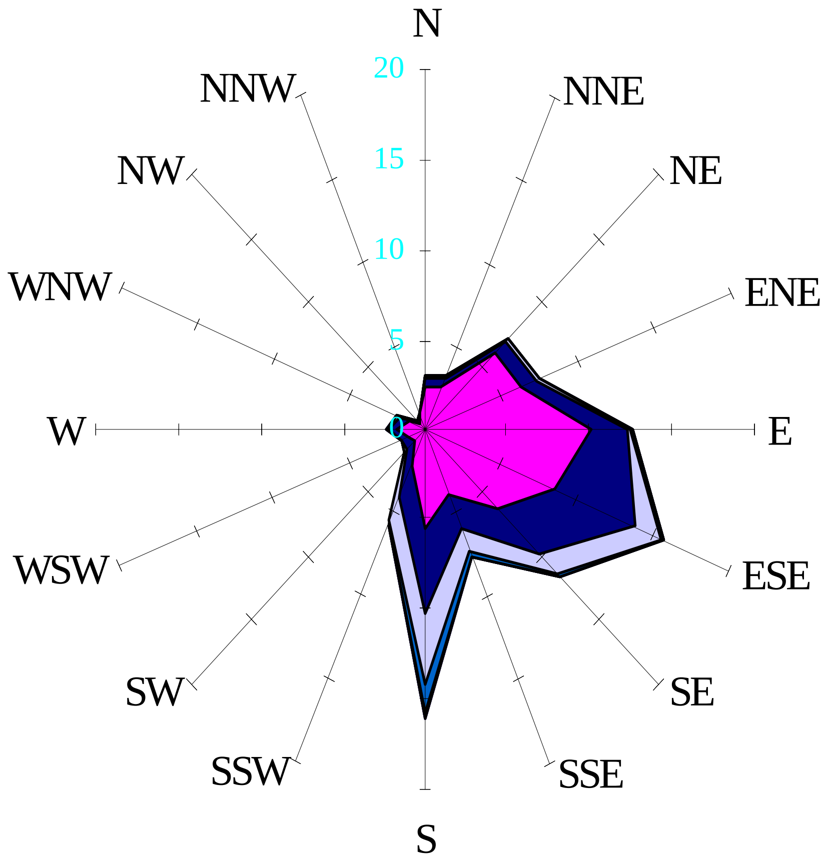
Haines Boat Harbor, Alaska

POR: 1973-1995

Latitude 59.14 N Longitude 135.26 W

Elevation 10 Meters

July Wind Summary



Percent Frequency Calm Winds: 0.64

Labels of Percent Frequency on North Axis

> 55kts 48-55kts 41-47kts 34-40kts 28-33kts 22-27kts 17-21kts 11-16kts 7-10kts 1-6kts

Figure A-8. Wind Rose

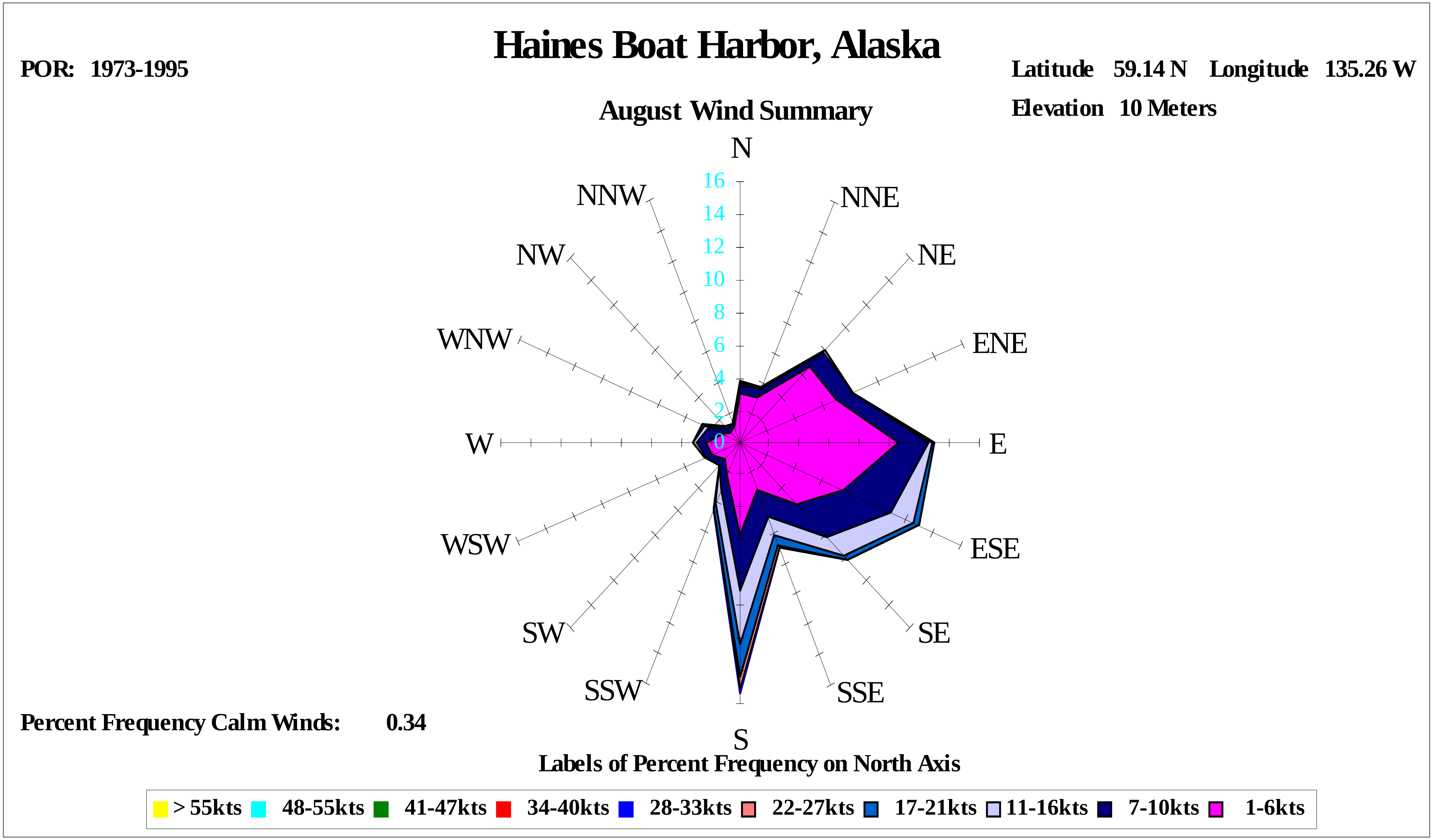


Figure A-9. Wind Rose

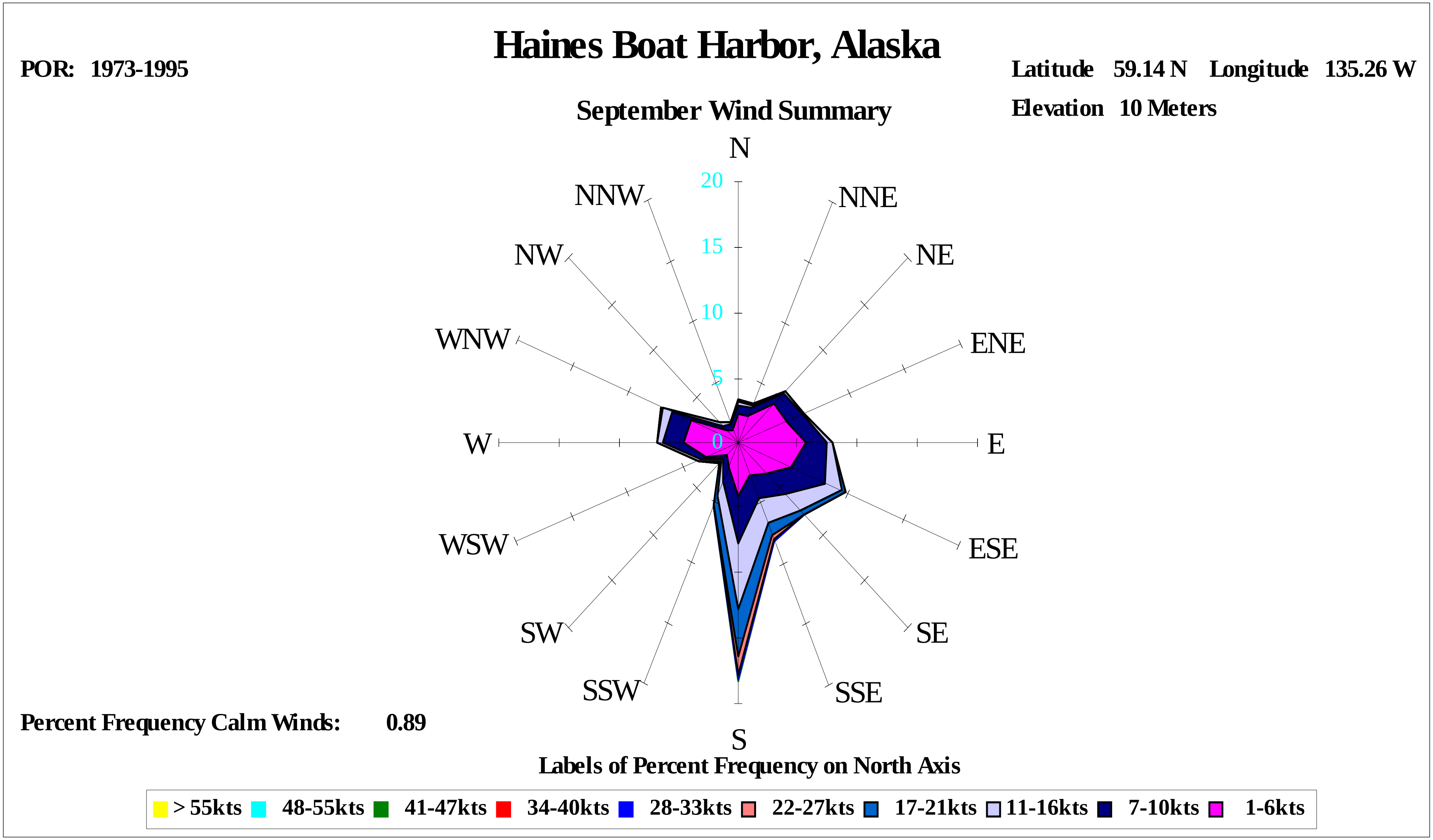


Figure A-10. Wind Rose

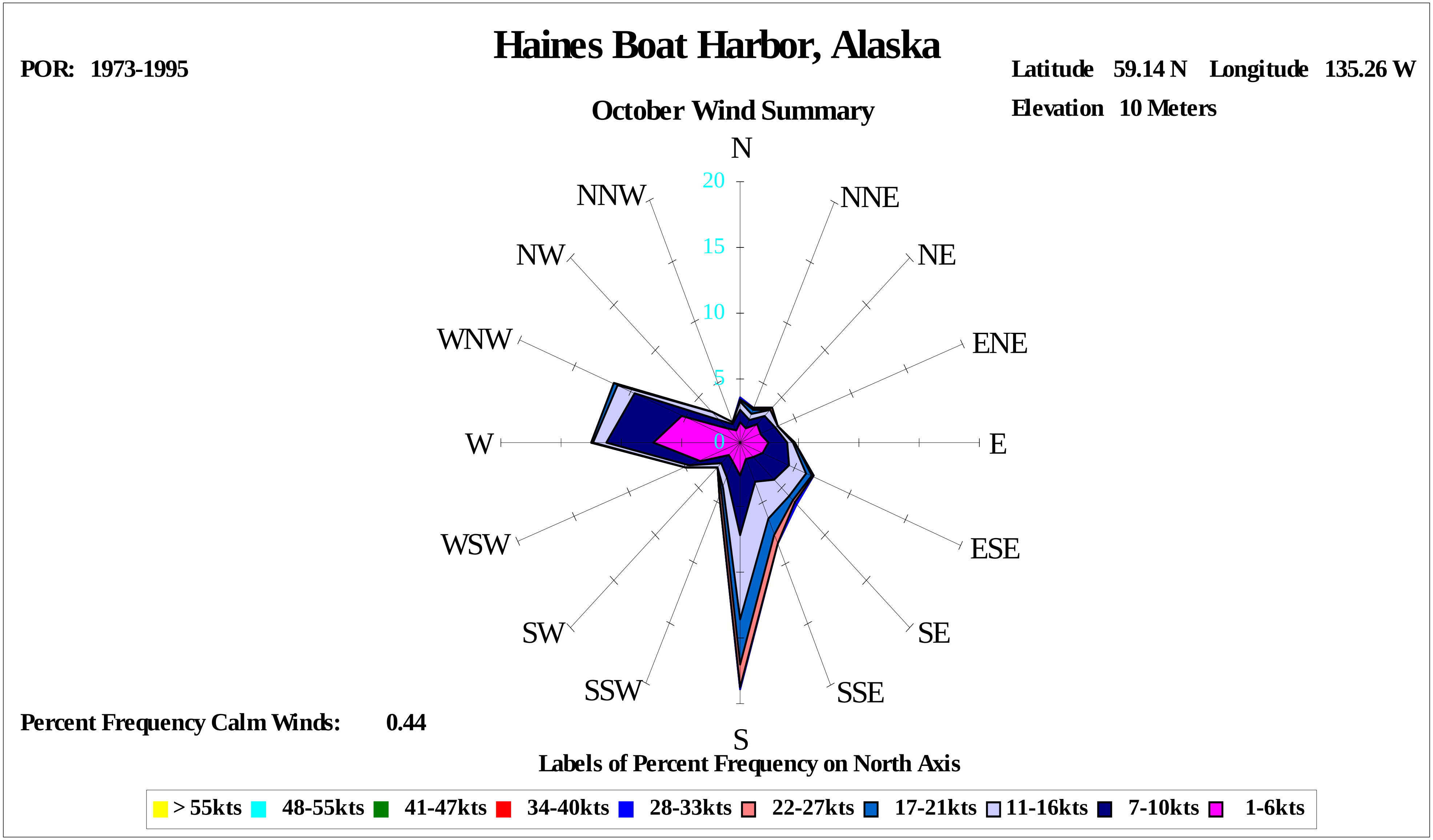


Figure A-11. Wind Rose

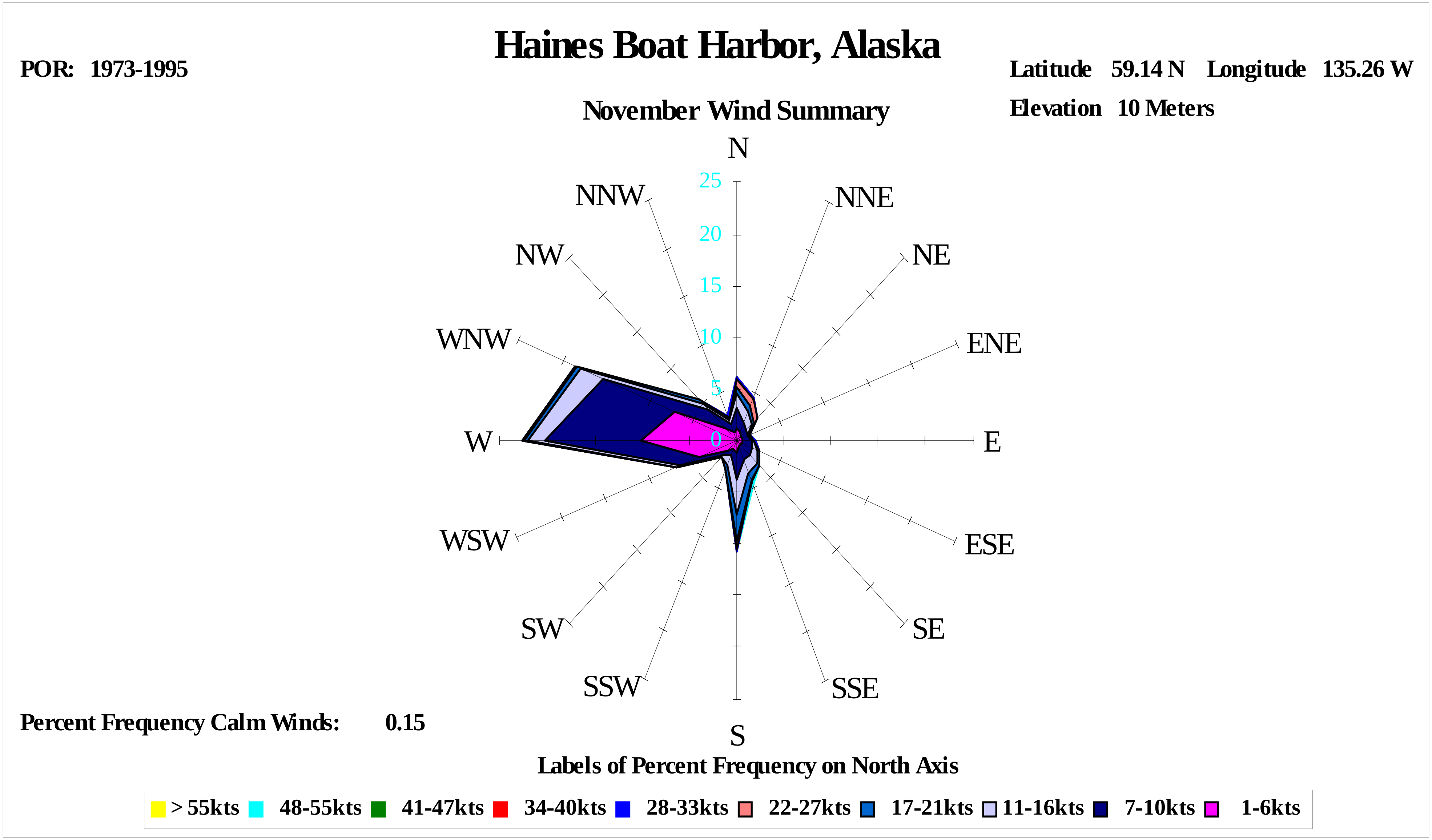


Figure A-12. Wind Rose

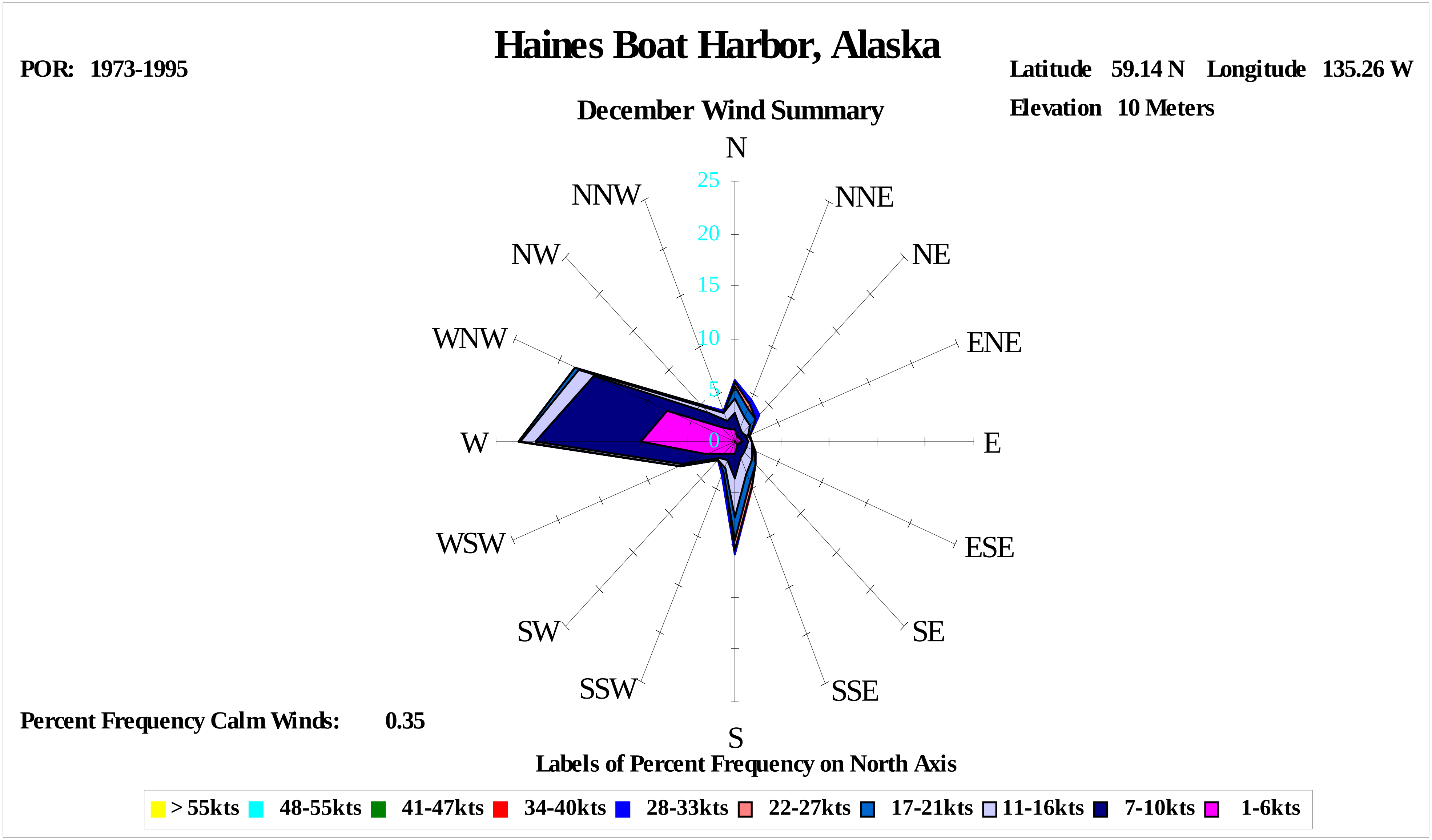


Figure A-13. Wind Rose

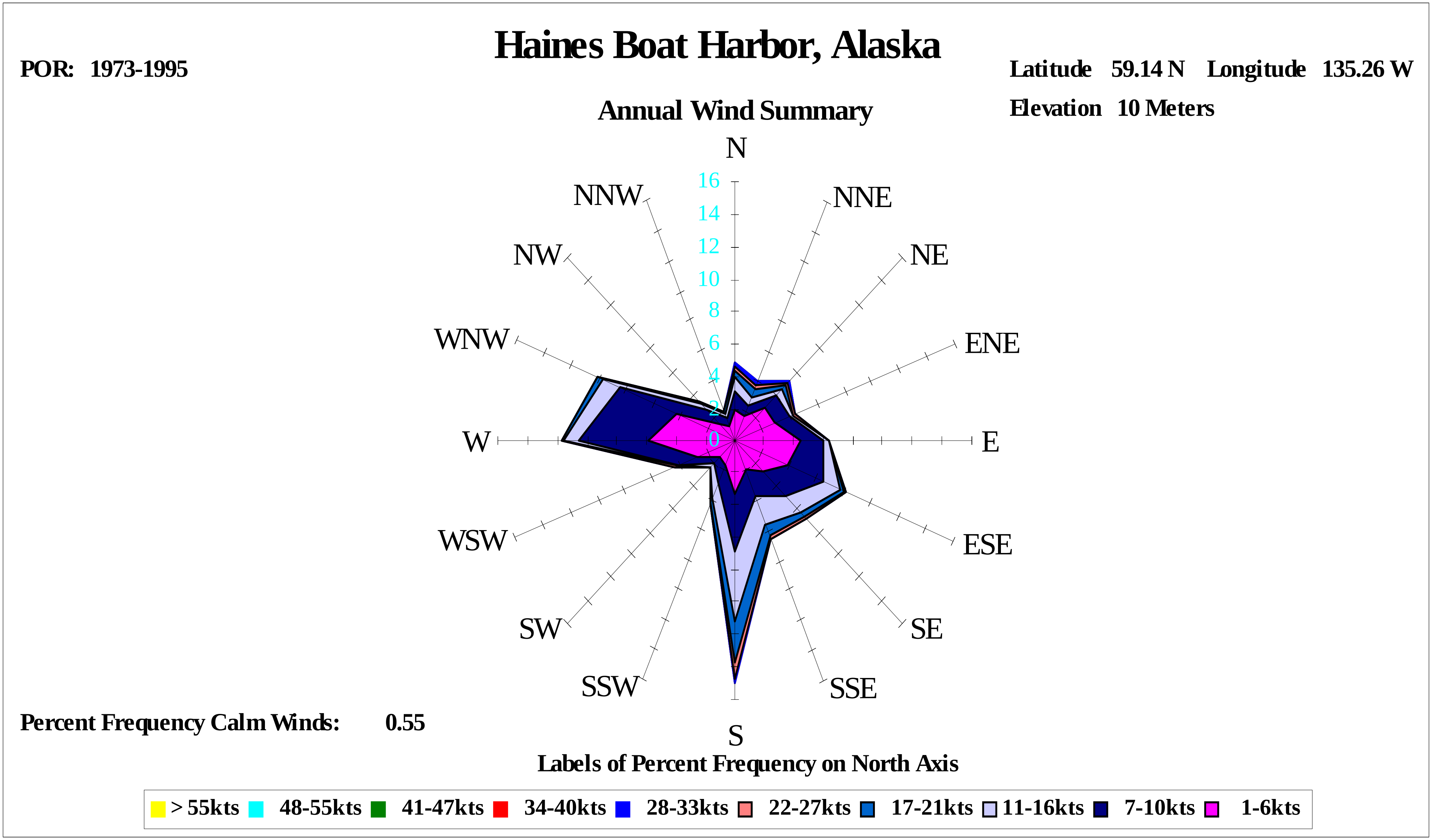


Figure A-14. Wind Rose

Since then, wind data recorded by a National Weather Service anemometer at the Haines Harbor was obtained from the Air Force Combat Climatology Center (AFCCC) for a period from 1973 to 1995. AFCCC provided an extreme value analysis that gave peak winds for various return periods along with the design-calculated risk. The data was converted to 1-hour duration winds using methods described in the Shore Protection Manual. Results of this analysis are shown in table A-1.

Table A-1. Extreme wind analysis results for Haines, Alaska

(1-hour-duration winds in km/hr)

Wind direction	Return period (years)							
	1.1	1.25	2	5	10	20	50	100
South 165°-195°	44.8	47.1	52.4	59.3	63.4	67.4	72.1	75.6
Southeast 120°-150°	38.7	38.9	41.9	55.2	68.9	84.5	105.6	122.5
East 075°-105°	26.1	26.5	30.9	46.7	61.5	76.9	98.0	113.8
North 345°-015°	43.0	45.8	51.3	57.4	60.6	63.4	66.9	68.9
All directions	53.6	54.8	64.3	88.4	109.0	130.3	159.9	182.5

Source: Air Force Combat Climatology Center, period of record (1973-1995).

2.3 Tides

The mean tide range at Haines is 4.33 meters and the diurnal range is 8.02 meters. The tides are generally diurnal with two highs and two lows occurring daily. Tide levels, referenced to mean lower low water (MLLW), are shown in table A-2. Extreme high water levels result from the combination of astronomic tides and rises in local water levels due to atmospheric and wave conditions. Water surface elevations have been recorded as high as +6.9 meters and as low as –1.83 meters at Haines under combinations of extreme high or low pressure systems and tides.

Table A-2. Tide elevations, Haines, Alaska

Level	Elevation (m MLLW))
Highest Tide (predicted)	+6.49
Mean Higher High Water (MHHW)	+5.12
Mean High Water	+4.82
Mean Low Water	+0.49
Mean Lower Low Water (MLLW)	0.00
Lowest Tide (predicted)	-1.52

Source: NOAA National Ocean Service.

Currents. The regional currents in Portage Cove and Chilkat Inlet are driven primarily by tides and only partially by wind. Discharge from the Chilkat River also affects currents in Chilkat Inlet near the mouth of the river during high flows. In general, current velocities average 5.1 to 25.7 to centimeters per second (cm/sec) along the western shores of Portage Cove and eastern shores of Chilkat Inlet. The wind driven component of the currents in the project vicinity is variable and depends on wind velocity.

A maximum flood current velocity of 25.7 cm/sec and a maximum ebb current velocity of 41.1 cm/sec are predicted in *Tides & Currents* 1997 for the Haines area.

2.4 Storm Surge

Storm surges are increases in water surface elevation caused by a combination of relatively low atmospheric pressure and wind-driven transport of seawater over relatively shallow and large unobstructed waters. Storm-induced surges can produce short-term increases in water levels to an elevation considerably above mean water levels. Storm surge at Haines has not been studied in depth; however, indications are that the area does not experience significant storm surges. Rugged terrain onshore and steep offshore bathymetry are conditions that preclude high storm surges. Highest surges are likely to be on the order of 0.3 to 0.75 meters in addition to wave set-up and tides during extreme low-pressure events. Typically, storm surges at Haines would be expected to be less than 0.5 meter. As table A-2 shows, tides at Haines are the major factor in the fluctuations in water surface elevations. The wind-driven transport of seawater is the second most important factor, followed by wave set-up.

2.5 River and Creeks in Project Vicinity

Several small creeks drain off of the eastern slope of Mt. Ripinski into Chilkoot Inlet north of Portage Cove. These are relatively small contributors of sediments to the waters in the area due to low flows throughout most of the year. At the northern limit of Chilkat Inlet, the Chilkat River converges with tidewater. Large sediment loads are indicated by the alluvial fan that forms the river's delta. Turbid water indicates the glacial origin of the river. However, due to deep water immediately south of the alluvial fan, relatively little accumulation of sediments along the shoreline farther south has occurred. At the Letnikof Cove site, no creeks drain directly into the immediate area. At the Portage Cove site, a small drainage swale south of the existing harbor discharges from a culvert under the road. The sediment load entering the existing harbor from this swale is relatively minor and it is deposited south of the entrance channel.

2.6 Soil Conditions

General information about the soil conditions at the Portage Cove site indicates relatively deep bedrock offshore. Soils are generally firm lean clay with sands, gravels, and cobbles. Large boulders are present on the surface, but generally do not extend beneath the surface. There are sand and gravel deposits on the beach along the immediate shoreline at the site. No exposed bedrock is evident at the site. The offshore materials at the site were characterized by a geotechnical investigation (see Geotechnical Appendix). Subsurface materials appear to be mostly clay with some sand and gravel. Also, some hard clay (diamicton) was encountered at a relatively shallow depth south of the existing harbor.

Soil conditions at the Letnikof Cove site were not investigated in detail. General observations indicate they would likely be mostly sands and gravels. Shallow bedrock is not indicated in the area. The beach at the site is composed of gravels and loose angular rock ranging from 2 to 10 cm.

2.7 Littoral Drift

Net littoral drift appears to be from north to south for Portage Cove. Predominant currents to the south in the vicinity of the existing harbor indicate that net sediment transport is to the south. At the existing harbor site, the north breakwater stub intercepts littoral material and forms a small pocket beach on its up-drift side. This traps sediments on the north side of the harbor before they can enter the harbor through the breach between the breakwaters. Total annual volume of sediment deposition is small, however, since the build-up of material since construction of the breakwater more than 20 years ago has been minimal. Sediments are also deposited along the shoreline south of the existing harbor at the cruise ship dock. A large pocket beach has resulted from this deposition along the southern boundary of Portage Cove.

It is estimated that approximately 125 cubic meters per year (m^3/yr) of littoral material are transported along the shoreline adjacent to the existing harbor. This is based on an estimated volume of material north of the north breakwater using aerial photography and survey data. Since completion of the previous harbor expansion in 1976, sand and silt has slowly accumulated at this location.

The large sediment load in the Chilkat River is the primary source of sediments in the Chilkat Inlet. However, much of this material is deposited in extreme depths of water offshore in the vicinity of the Letnikof Cove site. Indications are that the net littoral transport is from north to south. Beaches along the eastern shoreline are composed of gravels and cobbles with the exception of the pocket beach at the southern limit of Letnikof Cove. Sediments are deposited on this beach to the south of the existing float system. Sediment deposition on shoreline and at the harbor itself is minimal.

2.8 Ice Conditions

2.8.1 Portage Cove

Sea ice is absent in Portage Cove during the summer and winter months. In general, the waters of Southeast Alaska’s Inside Passage are ice free year round. Some local icing conditions along the shoreline can occur during extreme cold temperatures where fresh water enters Portage Cove at the creeks’ mouths. Strong low-pressure systems associated with storms in winter generally bring warmer temperatures that prevent the formation of significant quantities of ice. Some ice has been reported in the existing harbor area from local minor freshwater sources but it is relatively short lived. Ice can form in protected bodies of water, such as harbors, if freshwater enters the harbor and wind and wave action do not disperse it.

2.8.2 Letnikof Cove

Letnikof Cove can experience significant icing during northerly winds and under certain conditions in the Chilkat River. Extreme cold conditions during the winter months have caused severe icing problems at the existing float system. Ice destroyed several floats during the winter of 1998. Extensive repairs were required to restore the harbor to service. At present, the harbor at Letnikof Cove is only used during the summer months. It is shut down in the winter mainly due to the icing conditions but also due to its long distance from the downtown Haines area.

3.0 WAVE ANALYSIS

3.1 Wave Climate

The wave climate in the Haines area varies with orientation to Chilkoot Inlet to the east of Portage Cove and Chilkat Inlet to the west. No open ocean swells or long period waves, however, reach the area. The northern half of the Portage Cove shoreline is directly exposed to the east-southeasterly fetch from Chilkoot Inlet and experiences moderately high waves under storm conditions. Such waves are generally in the 1.5- to 1.9-meter range with periods of 4 to 5 seconds. During northerly winds, the existing harbor and surrounding shoreline is exposed to waves of up to 2.3 meters high with periods of up to 5 seconds wrapping into Portage Cove from Taiya and Chilkoot Inlets. Also, waves may be generated from easterly winds coming across Chilkoot Inlet toward the existing harbor.

Long-time Haines residents have observed extreme wave conditions in the area and report that no long-period swells reach the existing harbor site. Residents have reported that southeasterly winds of approximately 65 to 80 km/hr maximum sustained produces waves up to 2.1 meters high at the breakwater. In 1998 a major storm produced winds to 110 km/hr sustained with gusts to 175 km/hr. Wind data records indicate that strong north winds measured at the existing harbor during the winter months are westerly. Due to topographic effects of the Chilkat River valley, winds are channeled toward the east through town. Waves coming down Taiya Inlet and refracting into the Portage Cove harbor area are reported to be 2.1 to 2.4 meters high. The existing breakwater has overtopped under these conditions combined with extreme high tides.

The cruise ship dock at Portage Cove is fully serviceable during its season of operation without wave protection. However, its use is generally confined to the summer tourist season when predominant winds are from the southeast. Some natural protection from southeasterly waves is provided by its orientation to the shoreline.

Also, mooring cruise ships with the bow oriented into the predominant wave direction and the size of the ships themselves allow dock usage during the summer months.

Letnikof Cove is somewhat exposed to northerly waves, although due to the shallow bathymetry of the Chilkat River delta, wave heights of less than 1 meter and periods of less than 3 seconds impact the existing harbor there. Strong northerly winds are common during the winter months and do cause rough conditions in Letnikof Cove. However, available fetch north of the existing harbor is limited by the river and shoals in the lee of Pyramid Island and the spit that partially connects it to shore. The river mouth is generally ice covered and discharge is reduced during the winter months. These conditions impede wave generation. The existing harbor is also naturally protected from southerly waves coming up Chilkat Inlet.

The harbormaster reported that extreme winds in the area cause severe conditions at the existing harbor site. While northerly winds do not generate significantly large waves, winds and icing conditions cause serious problems at the harbor. Winds of up to 165 km/hr have been observed. These winds are intensified due to the topographic effects of the Chilkat River valley.

3.2 Fetches

The shoreline of Portage Cove at the existing harbor is oriented generally to the east. The longest fetch for the existing harbor is in the east-southeasterly direction at 6.32 km long at an azimuth of 105°. Shorter fetches from the east and northeast are 5.57 km at an azimuth of 90° and 5.58 km at an azimuth of 45°, respectively. Portage Cove is indirectly exposed to the long fetches up Taiya Inlet and down Chilkoot Inlet. These fetches are oriented in a north-south direction and do not generate waves that directly effect the site. However, waves do refract into the project site from these directions when winds are out of the north or south.

Letnikof Cove is along a shoreline that is oriented in a northwest-southeast direction. The longest fetch for this site is in the northwesterly direction at 4.17 km at an azimuth of 315°. This fetch assumes that the shoal area connecting the shoreline with Pyramid Island is very shallow at high tide. Therefore, insignificant wave energy can be generated from the Chilkat River Delta northwest of the shoal.

Table A-3 provides a summary of the fetch distances for both sites. Effective fetches were determined using 9 radials at 3-degree increments centered on the predominant wind direction. The radial lengths were arithmetically averaged in the calculations for fetch length. Figures A-15 and A-16 show layout of the fetch radials in the direction of the wind for the various sites.

Table A-3. Fetches for two Haines harbor sites*

Direction	Fetch distance (km)
Portage Cove site	
Southeast (135°)	2.53
East Southeast (112.5°)	6.32
East (90°)	5.57
Northeast (45°)	5.58
South (180°)	0.74
East Northeast (67.5°)	4.92
Letnikof Cove site	
Northwest (315°)	4.17

*Fetches were calculated according to methods specified in the 1984 *Shore Protection Manual* (9 radials at 3° increments). Portage Cove site Northeast (45°) and east-southeast (112.5°) reflect local fetches and do not include effects from down Taiya Inlet and up Chilkoot Inlet.

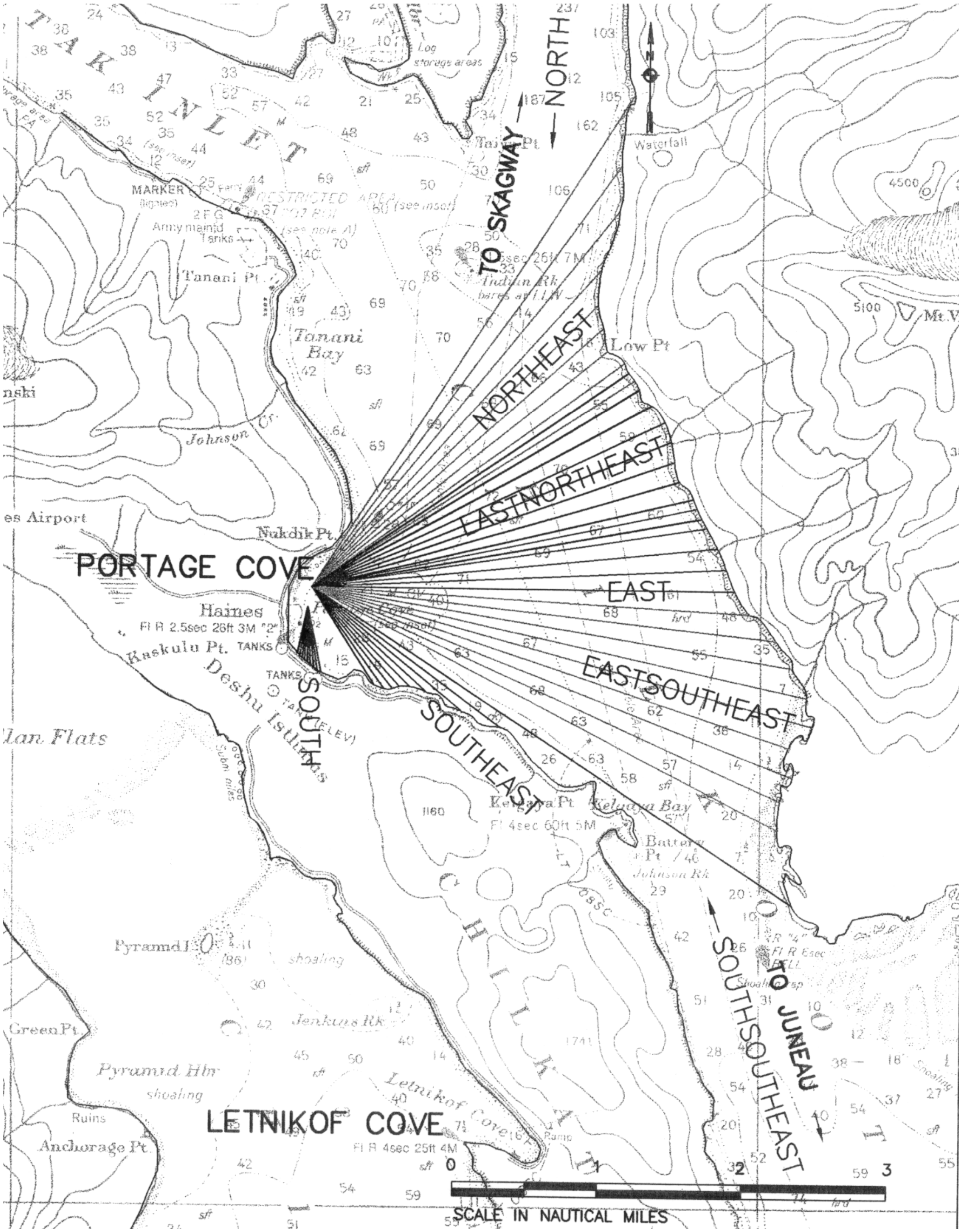


Figure A-15. Portage Cove, local fetch

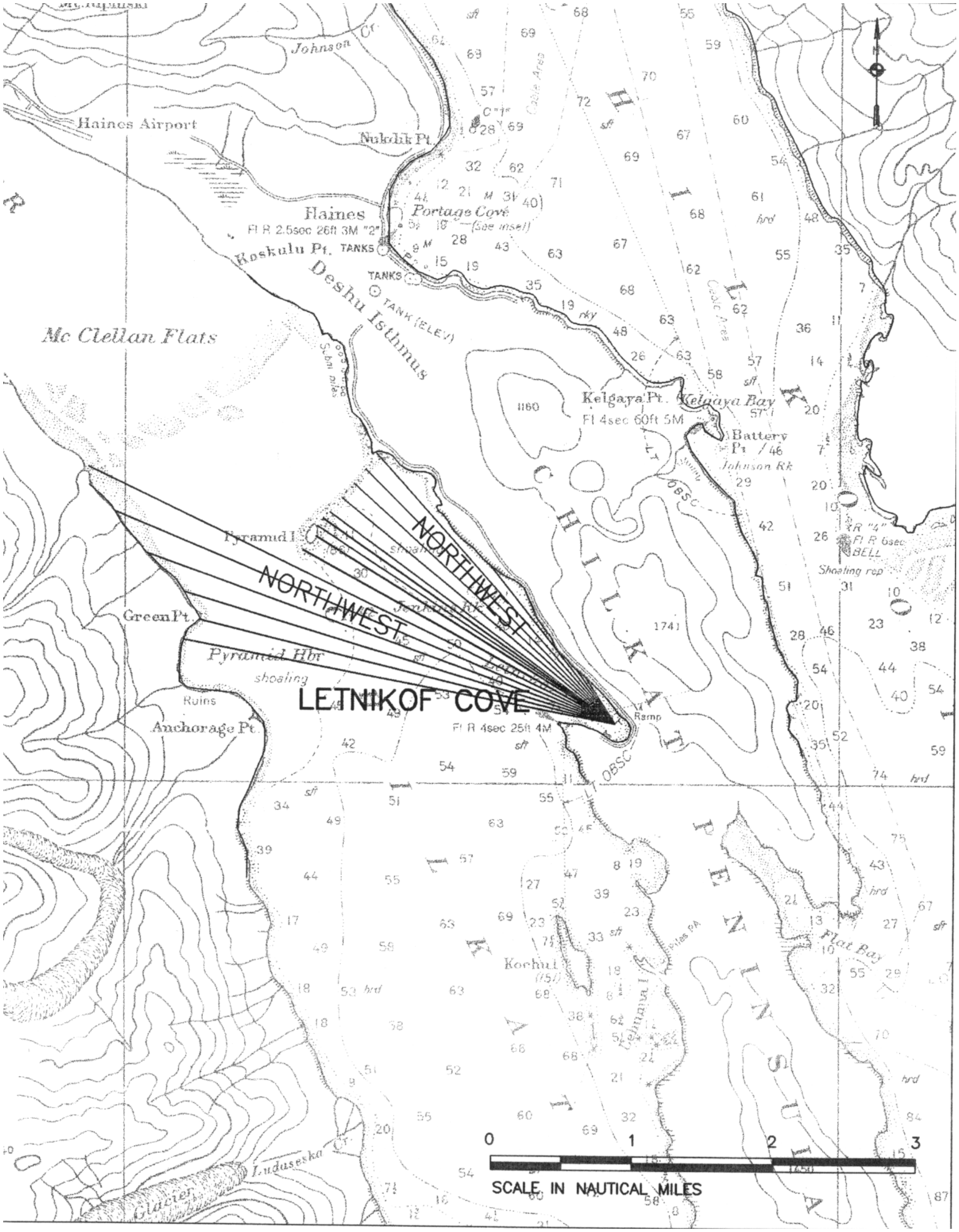


Figure A-16. Letnikof Cove, local fetch

3.3 Wave Prediction

Predicted wave heights for the project area were calculated using the 50-year design windspeeds presented in table A-1. Methods described in the 1984 *Shore Protection Manual* (SPM) and the STWAVE numerical model were used to predict wave heights. The design waves for the various sites were determined from the results of the two different prediction methods. The STWAVE results were used to take into account refraction, diffraction, and shoaling internally based on bathymetry for comparison with the results from the SPM method.

The wind data used to model wave growth was transformed to more accurately reflect the boundary layer above the water surface. Windspeed depends on elevation, roughness of the surface over which the wind is blowing, and temperature gradients. The wind data obtained from the AFCCC was converted to 1-hour duration winds and was taken at a height of 10 meters above land. The wind data was corrected to account for air-sea temperature differences and overland versus overwater differences.

The SPM equations predict wave heights based on fetch distances and windspeeds. The fetch distance and windspeed are used to determine whether the wave condition is limited by the fetch length or by the duration of the wind. STWAVE is a spectral wave energy propagation model that includes refraction, diffraction, and shoaling, but does not include reflection. Shoreline and bathymetric conditions were defined by inputting water depths and the locations of land into the STWAVE model at a specified grid spacing. Depths were obtained from NOAA charts showing the bathymetry of the area. Grids were established for the north direction down Taiya Inlet and for the south direction up Chilkoot Inlet. The model was run for the Portage Cove site using the 50-year wind speed for the north and south directions. Runs were performed with water levels at Mean Lower Low Water (MLLW) and Mean Higher High Water (MHHW).

Results of the wave analysis are shown in table A-4. The wave heights calculated represent the significant wave height, H_s, which is the average of the highest one-third of all waves generated. The design waves shown in table A-4 are for a design still- water-level (SWL) of +5.12 meters MLLW. The design waves selected appear to correlate well with what longtime residents have observed during extreme storm events in the Haines area.

Wave heights calculated for the Letnikof Cove site vary depending on depth, distance offshore, and the effects from the shoals at Chilkat River Delta to the north of the site. Complex wave refraction and shoaling may occur due to the shallow shelf and shoals north of this site. However, direct exposure to northwest waves just southeast of the shoal was evaluated using the SPM methods.

The STWAVE model was run for the Portage Cove site with long narrow fetches down Taiya Inlet from Skagway and up Chilkoot Inlet from the Chilkat Islands as contributing to wave generation. These runs were used to test the effect of incoming waves propagating around the southern and northern tips of Portage Cove (Taiya Point and Battery Point, respectively) and into the proposed site. Model results showed that waves do propagate into the proposed site under the worst conditions with winds from the north or south. Wave heights and periods predicted by the model were compared with local observations for the existing harbor under extreme storm conditions. The model predicted significant wave heights of 2.3 meters and 2.1 meters, respectively, for these two directions. Such wave heights correlate closely with local observations during the highest winds from the north and south. This analysis indicates that the long, narrow fetches to the north and south of the project site do contribute to the overall wave climate significantly. Therefore, a design wave height of 2.3 meters with a period of 5 seconds from the east-northeast was selected for the Portage Cove site.

Locally generated vessel wakes were also considered for design of wave protection structures. Due to the relatively large vessel traffic adjacent to the existing harbor at the Portage Cove site, a wake of 1 meter with a period of 3 seconds was selected for design purposes for structures to provide wave protection from the east. Large cruise ships generally reduce their speeds considerably when approaching the dock adjacent to the existing harbor for mooring. This 1-meter wave represents a worst-case scenario and would impact the proposed Portage Cove harbor expansion from the east and would be considered to impact the site perpendicularly. Since this wave is less severe than the 50-year wind generated wave impacting the site, it is not used for sizing the stone for rubblemound breakwaters. It was considered, however, in the wave diffraction analysis for the entrance channel and mooring basin of the proposed harbor improvements.

Table A-4. Wave Analysis Results

	SPM		STWAVE		Selected Design	wave
Direction	H_s (m)	T (s)	H_s (m)	T (s)	H_s (m)	T (s)
Portage Cove site						
Southeast (135°)	1.31	3.15			1.31	3.15
East-Southeast (112.5°)	2.07	4.27	2.10	4.30	2.10	4.30
East (90°)	1.77	3.97			1.77	3.97
Northeast (45°)	1.11	3.40			1.11	3.40
South (180°)	0.62	2.00			0.62	2.00
East-Northeast (67.5°)	1.04	3.26	2.30	5.00	2.30	5.00
Letnikof Cove site						
Northwest (315°)	0.96	3.09			0.96	3.09

Legend: H_s = Significant Wave Height (meters); T = Wave Period (seconds)

4.0 EXISTING HARBOR FACILITIES

4.1 General Description and Background

The existing harbor facilities at Haines are shown in figures 1 and 2 of the main report. The territory (later the State) of Alaska and the Alaska Public Works Agency constructed the original small boat harbor at Haines in 1958. The U.S. Army Corps of Engineers constructed the expanded harbor at Portage Cove in 1976. The project consisted of demolishing the seaward leg of the original breakwater and constructing a new longer breakwater farther offshore. Additional dredging was performed to provide an expanded mooring area and entrance channel. The mooring facilities constructed in subsequent years were put in with local funds provided by the state.

The State of Alaska also constructed protected mooring facilities in Letnikof Cove in the late-1960’s. A floating breakwater configuration was selected as the design for this site since the wave climate is relatively mild. Severe wind and ice damage to the float system has occurred several times in the past at the Letnikof Cove site.

4.2 Portage Cove Harbor

Portage Cove Harbor is just east of the city of Haines on the eastern shoreline of the Deshu Isthmus of the Chilkat Peninsula. It has limited space for vessels in the existing moorage area and cannot adequately accommodate vessels larger than 18.3 meters in length. The float system is outdated and undersized and the mooring basin is severely overcrowded for the number and size of vessels that moor there. The float system’s stalls are also poorly oriented (in an east-west direction) causing vessels entering or leaving the harbor to have severe difficulty during high winds. The existing basin area, dredged to a depth of –3.7 and –4.3 meters MLLW, has approximately 2.2 hectares (ha) available for mooring. Limited maneuvering and turning areas are also characteristic of the existing harbor.

A dredged entrance channel to a depth of –4.6 meters MLLW accommodates access to the harbor around the southern tip of the existing breakwater. The southern limit of the harbor is somewhat open to wave action from the southeast. This exposure has caused damage to vessels and the float system and has also created hazardous navigation conditions inside the harbor during storms.

A dock is inside the harbor to support the fishing fleet and transient vessel traffic. Temporary moorage for offloading fish products can occur at these facilities. Sufficient space for permanent mooring facilities is not available since high vessel traffic is common throughout the area.

4.3 Letnikof Cove

The moorage facilities at Letnikof Cove consist of a pontoon-supported floating breakwater and long finger floats in which vessels moor in parallel. There are currently two main floats, each 46 meters in length, which can accommodate a small portion of the local fishing fleet. Previously, two additional floats, each 30 meters in length, also were used at this facility. A severe storm in the mid-1990’s destroyed these floats and they have not been replaced. The float system was originally constructed by the State of Alaska during the late 1960’s. A launch ramp and small parking area are also at the site.

High winds and severe icing conditions have historically impacted the facilities at Letnikof Cove. Commercial and recreational vessels do use the harbor during the summer fishing season due to its proximity fishing grounds. Generally, however, the floats are not used during the winter months due to the extreme wind and ice conditions and the long distance from town.

Water depths are typically in the 10- to 15-meter range. Such depths can accommodate the larger commercial fishing vessels that frequent the area. Vessel moorage is limited to rafting along the floats and floating breakwater.

A cannery and private dock are located along the western shoreline of Letnikof Cove. The dock and haul-out are primarily used for transferring fish and loading and offloading fish products. Facilities for mooring vessels are limited and no permanent slips are available.

5.0 HARBOR DESIGN CRITERIA

5.1 Design Vessel and Fleet

The economic analysis for this study generated the vessel demand for the proposed project. The fleet considered for the various alternatives is described in Appendix B, Economic Analysis. Lengths, beams, and drafts for these vessels were developed in conjunction with the harbormaster. Proposed harbor plans were laid out to accommodate the identified fleet. The design vessel (the largest vessel in the fleet) for the north harbor is 36.6 meters long with a beam of 9.8 meters and a draft of 3.0 meters. It is anticipated that vessels in the larger range of the fleet will use the proposed north entrance channel and mooring basin. The design vessel for the south harbor is 15.2 meters long with a beam of 5.3 meters and a draft of 2.1 meters.

5.2 Wave Height Criteria for the Entrance Channel

Breakwaters for the proposed alternatives were positioned to reduce wave heights in the harbor entrance(s). However, due to the orientation of the entrance channel(s) into the predominant wave direction(s), wave energy would still propagate into the entrance channel areas. Reduction of wave heights to a maximum height of 0.75 meters at the inside limit of the entrance channel should be achieved with the proposed layouts. Progressively smaller wave heights down to 0.29 meters and less were allowed into the channel. Such wave heights would not impact vessels entering and leaving the harbor.

5.3 Wave Height Criteria for the Mooring Area

The Alaska Department of Transportation and Public Facilities (ADOTPF) provided the wave height criteria for the mooring basin. The criteria shown in table A-5 summarize the wave heights and horizontal motion considered for the mooring basin design. Such criteria closely follow “Planning and Design Guidelines for Small Craft Harbors” (ASCE, 1994). Values under the “moderate” category were used as the basis for determining the necessary breakwater configuration to reduce wave heights in the proposed mooring basin. It is anticipated that the mooring facilities to be located nearest the entrance channel to the expanded harbor would be oriented faced into the residual wave coming into the harbor. Therefore, vessels moored at these locations would experience “head seas” with respect to the inner harbor wave energy.

Maximum allowable wave heights of 0.29 meter in the mooring areas were used for a 50-year incident design wave event. This criterion parallels that outlined in EM 1110-2-1615, "Hydraulic Design of Small Boat Harbors," which represents many years of experience in harbor design. This criterion is appropriate to capture the economic benefits for the fleet by adequately minimizing damages.

Since no long-period ocean swell reaches the project site (i.e. wave periods greater than 6 seconds), criteria for horizontal motion were not applicable. Little to no horizontal motion is expected in the proposed expanded harbor and none from long- period swell is observed in the existing harbor.

Table A-5. Wave Criteria For Mooring Basin

Recurrence, Orientation, and Period	Good	Excellent	Moderate
For wave heights (H1/3):	(m)	(m)	(m)
1 year interval, Beam Sea,T>6	0.15	0.12	0.19
1 year interval, Beam Sea,2<T<6	0.15	0.12	0.19
1 year interval, Beam Sea,T<2	0.30	0.23	0.38
50 year interval, Beam Sea,T>6	0.23	0.17	0.29
50 year interval, Beam Sea,2<T<6	0.23	0.17	0.29
50 year interval, Beam Sea,T<2	0.30	0.23	0.38
1 year interval, Head Sea,T>6	0.30	0.23	0.38
1 year interval, Head Sea,2<T<6	0.30	0.23	0.38
1 year interval, Head Sea,T<2	0.30	0.23	0.38
50 year interval, Head Sea,T>6	0.61	0.46	0.76
50 year interval, Head Sea,2<T<6	0.61	0.46	0.76
50 year interval, Head Sea,T<2	0.61	0.46	0.76
For horizontal motion (ft):			
1 year interval, Beam Sea,T>6	0.30	0.23	0.38
50 year interval, Beam Sea,T>6	0.61	0.46	0.76
1 year interval, Head Sea,T>6	0.61	0.46	0.76
50 year interval, Head Sea,T>6	1.22	0.91	1.52

Note: "head sea" only applies to vessels that are aligned within + or - 15 degrees of the wave direction.

Diffraction diagrams from the SPM were used to calculate wave heights expected in the harbor alternatives. The 0.29-meter wave height criterion was used for the alternatives considered.

5.4 Entrance Channel, Maneuvering Area, and Mooring Basin Design

The new entrance channel width was determined using criteria given in EM 1110-2-1615 “Hydraulic Design of Small Boat Harbors” (USACE 1984), in "Planning and Design Guidelines for Small Craft Harbors" (ASCE 1994), and in the State of California's "Layout and Design Guidelines for Small Craft Berthing Facilities" (1980). For a two-way channel with 0.25 to 0.75 m/s currents, the width should be 180 percent of the beam of the design vessel, plus an additional 80 percent for traffic clearance and 60 percent for breakwater clearance. For the proposed entrance channel, a minimum bottom width of 36.0 meters in straight sections and 39.6 meters in turning sections would allow adequate maneuverability and clearance on each side of the breakwaters.

The maneuvering areas and the fairway widths were designed so that there would be adequate room for vessels to turn and dock. Width for turning was determined using a factor of 1.75 times the length of the largest vessel using the finger piers in that area of the basin. These criteria are also given in ASCE 1994.

5.5 Depths

The north entrance channel depth was established based on the following criteria:

Entrance channel-	
Vessel draft	-3.0 m
Pitch, roll, and heave, based on 1/2 of the wave height in the channel	-0.50m
Squat	-0.15 m
Access (tide)	-1.25 m MLLW
Safety clearance (based on sand/gravel bottom)	-0.61 m
TOTAL	-5.50 m MLLW

The depth for the new maneuvering area was calculated as follows:

Maneuvering area-	
Vessel draft	-3.0 m
Pitch, roll, and heave	-0.0 m
Squat	-0.05 m
Access (tide)	-1.25 m MLLW
Safety clearance (based on sand/gravel bottom)	-0.61 m
TOTAL	-4.91 m MLLW
	(use –4.90 m MLLW)

The depth for the new mooring area (for vessels 15.2 to 36.6 meters in length was calculated as follows:

Mooring area--	
Vessel draft	-3.0 m
Pitch, roll, and heave	-0.0 m
Squat	-0.00 m
Access (tide)	-1.25 m MLLW
Safety clearance (based on sand/gravel bottom)	-0.61 m
TOTAL	-4.86 m MLLW
	(use –4.90 m MLLW)

The depth for the new mooring area (for vessels less than 15.2 m in length) was calculated as follows:

Mooring area--	
Vessel draft	-2.4 m
Pitch, roll, and heave	-0.0 m
Squat	-0.0 m
Access (tide)	-1.25 m MLLW
Safety clearance (based on sand/gravel bottom)	-0.61 m
TOTAL	-4.26 m MLLW
	(use –4.30 m MLLW)

The south entrance channel (into the existing harbor) depth was established based on the following criteria:

Entrance channel-	
Vessel draft	-2.1 m
Pitch, roll, and heave, based on 1/2 of the	
wave height in the channel	-0.50 m
Squat	-0.15 m
Access (tide)	-1.25 m MLLW
Safety clearance (based on sand/gravel bottom)	-0.61 m
TOTAL	-4.61 m MLLW
	(use –4.60 m MLLW)

The entrance channel to the existing harbor is dredged to a depth –4.6 meters MLLW. Therefore, the new south entrance channel would be the same depth as the existing channel.

The natural depths offshore of the existing shoreline varied down to –7.0 meters MLLW at the proposed harbor entrances. Commercial fishing vessels may enter the proposed north harbor basin loaded due to weather or other extenuating circumstances. Loaded drafts were used to calculate required depths for the entrance channels. Mooring basin depths were determined assuming that vessels would be loaded with their product before mooring in the harbor.

5.6 Entrance Channel Depth Optimization

Optimization of the entrance channel depths was based on the information shown in tables A-6 and A-7, which list a range of channel depths and the percentage of time the channel would be accessible. A north entrance channel depth of –5.50 meters MLLW was determined to be the optimum channel depth based on percentage of time accessible, costs for construction, and economic benefits. Similarly, a south entrance channel depth of –4.6 m MLLW was determined to be optimum. Such a depth represents that of the existing entrance channel.

Table A-6. North Entrance Channel Optimization

	Entrance Channel Depth (m MLLW)						
	-5.78	-5.50	-5.20	-4.90	-4.60	-4.30	-4.00
Tide (m MLLW)	-1.52	-1.25	-0.95	-0.64	-0.34	-0.03	+0.28
Design Vessel Draft (m)	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Pitch, Roll & Heave (m)*	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Squat (m)	0.15	0.15	0.15	0.15	0.15	0.15	0.15
Safety Clearance (m)	0.61	0.61	0.61	0.61	0.61	0.61	0.61
% Time Accessible	100	99.9	99.5	98.5	96.9	94.5	91.5

*1/2 times wave height in entrance channel.

Table A-7. South Entrance Channel Optimization

	Entrance Channel Depth (m MLLW)					
	-4.88	-4.61	-4.31	-4.00	-3.70	-3.40
Tide (m MLLW)	-1.52	-1.25	-0.95	-0.64	-0.34	+0.28
Design Vessel Draft (m)	2.1	2.1	2.1	2.1	2.1	2.1
Pitch, Roll & Heave (m)*	0.50	0.50	0.50	0.50	0.50	0.50
Squat (m)	0.15	0.15	0.15	0.15	0.15	0.15
Safety Clearance (m)	0.61	0.61	0.61	0.61	0.61	0.61
% Time Accessible	100	99.9	99.5	98.5	96.9	94.5

*1/2 times wave height in entrance channel.

A south entrance channel depth of –4.6 meters MLLW was similarly shown to be optimum. This represents the depth of the existing harbor's entrance channel. Therefore, the new south entrance channel depth would replicate that of the existing.

5.7 Life-Cycle Breakwater Design

Rock for the breakwaters was sized using the 50-year design wave. This was determined to be the most cost-effective means of protection for any harbor alternative considered. The maximum armor rock size for a 25-year design is 604 kilograms (kg), 50-year design is 1,136 kg and for a 100-year design is 1,386 kg. 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 was minimal difference in cost between rock sized for a 25-year event versus a 50-year event. Rock would likely be trucked from a local quarry to the project location. The quarry within the project vicinity has the capacity to produce rock for either a 25-year event or a 50-year event. Using the 25-year design rock did not result in an overall cost savings due to the higher charge for large replacement costs. Replacement costs are high because the project location is relatively remote and mobilization costs are expensive. A 75- or 100-year design would reduce the chance of needed maintenance. A 50-year design provides the best balance between minimizing maintenance and keeping the rock cost reasonable. The loss of a small amount of armor stone over time would have little to no effect on the operation and use of the harbor; therefore, there was not sufficient justification for basing the design beyond the 50-year level.

5.8 Floating Breakwater and Wave Barrier Design Considerations

Floating breakwaters reduce wave action by reflecting the incident wave and by dissipating some of the wave energy through friction and turbulence. Wave barriers reduce waves more by reflection than by turbulence. Some of the incident wave energy passes through both floating breakwaters and wave barriers resulting in a transmitted wave. The height of the transmitted wave is calculated as follows:

$$H_t = C_t * H_i$$

where H_t = transmitted wave height

C_t = transmission coefficient

H_i = incident wave height

The transmission coefficient is greatly affected by the width of the floating breakwater compared with the wavelength of the incident wave, and the draft of the breakwater compared with the depth of water. Transmission coefficients for wave barriers are a function of the depth of the barrier, the depth of water, and the wavelength of the incident wave.

The transmitted wave is also affected by the angle at which the incident wave impacts the breakwater. The waves inside the harbor are a combination of the transmitted wave and the waves diffracted around the ends of the breakwater. This can be expressed by the following equation:

$$H = \sqrt{H_t^2 + H_d^2}$$

where H = the wave height inside the harbor

H_d = diffracted wave height

For this project, floating breakwater and wave barrier design concepts were considered. At the Portage Cove site, design wave heights and periods exceed the criteria for economically viable floating breakwater applications. Costs associated with very wide and deep draft floating structures preclude use of such designs. Poor performance of the existing transient float acting as a floating breakwater in the existing harbor has been well documented. Damage to vessels and severe impediments to safe navigation in the harbor have resulted. The wave barrier design concept also has limitations in adequately reducing wave energy to acceptable levels. High costs for construction due to deep offshore bathymetry and varying existing sub-bottom conditions are factors that render wave barrier designs inappropriate for this site.

The Letnikof Cove site would lend itself to the use of a floating breakwater design. Wave heights and periods are within the range where such designs are applicable. Water depths of up to 20 meters allow floating breakwaters to be more cost effective than conventional rubblemound structures. Previous studies have presented economically viable designs for floating breakwaters at this site. Wave barriers would also perform well in reducing wave heights to within acceptable criteria. Due to extreme water depths, however, costs associated with such designs would exceed those for floating breakwaters.

Severe ice and wind conditions in Letnikof Cove would require special design considerations for either floating breakwaters or wave barriers. The possibility of removing, either partially or fully, these structures during the winter months could be considered. However, this would not provide the needed protection to keep the proposed harbor open and capture the economic benefits identified on a year-round basis. The floating breakwater and wave barrier designs are therefore not considered further in this analysis.

5.9 Water Quality and Circulation

Water quality and circulation criteria were established to minimize environmental degradation associated with harbor improvements. The conventional method for estimating harbor basin flushing is to use an average exchange coefficient for one tidal cycle. Flushing coefficients can be approximated by the tidal prism ratio: the difference in basin volume at high tide and low tide divided by the basin volume at high tide. It has been determined that average spatial values greater than 0.30 will provide for acceptable harbor basin flushing. It is also recommended that no more than 5 percent of the basin have values less than 0.15. The areas of possible low tidal prism ratios would be in the corners of the basin and should therefore be checked to ensure they meet this minimum value.

Another criterion for water quality and circulation is the aspect ratio of the basin. This value is a measure of the length divided by the width of the basin. Generally, aspect ratios of greater than 0.3 and less than 3.0 are desirable. Such geometry will minimize possible zones of stagnation and short-circuiting of circulation cells within the basin.

5.10 Uplands

The State of Alaska Department of Transportation and Public Facilities (ADOT/PF) requires that harbors in Alaska have a minimum uplands to total harbor area ratio of 0.40. This criterion was used as the basis for the layout of the various alternatives considered. Upland uses would include vehicle parking, boat and trailer storage, harbormaster’s office, restrooms, and harbor support facilities.

Since existing uplands are severely limited at the existing Portage Cove and Letnikof Cove sites, creation of additional uplands or purchase of vacant land would be necessary to meet this criterion.

6.0 ALTERNATIVES CONSIDERED IN DETAIL

6.1 General

A wide range of alternatives and sites was considered for navigation improvements at Haines. A matrix of possible sites for consideration was developed in the initial phase of the study that included Letnikof Cove, Paradise Cove, Flat Bay, Lutak Inlet, and Portage Cove. This phase narrowed the site options to two: one at Letnikof Cove and one adjacent to the existing harbor at Portage Cove. Letnikof Cove does not lend itself to siting the proposed harbor expansion due to extreme depths of water, severe icing conditions during the winter months, and extremely high wind velocities from the Chilkoot River valley. The site selection process narrowed the scope of alternatives considered to Portage Cove.

The alternatives were evaluated using established design guidance given in the appropriate Corps of Engineers Engineering Manuals (EMs) and the SPM. Physical modeling of the alternatives was not included in the scope of this analysis.

After a thorough evaluation of the wave climate in Portage Cove, it was determined that rubblemound breakwaters for protection from the northeasterly to southerly wave exposures were most appropriate and cost-effective. Relatively shallow water depths lend themselves to economically constructed rubblemound breakwaters for the project.

Vessel traffic conditions, including cruise ship operations, were considered in the layout of proposed alternatives. Development of a new harbor at this site would not impact current operations at the existing cruise ship dock. Large vessels would continue to be able to maneuver and moor at both docks south of the existing harbor and coexist with the increased vessel usage in the area.

The site has limited uplands but these can be readily expanded by constructing fill to facilitate a functional harbor in this area. Creation of such uplands would require hauling and placing material produced from a quarried rock source. This site also represents the most practical site for harbor development due to its proximity to the core downtown area of Haines. Other sites farther down the road have the disadvantage of being located more than 8 km from town.

6.2 Portage Cove Site

The Portage Cove site is immediately adjacent to the existing harbor east of the town of Haines and has natural bottom elevations that range from +8 meters MLLW to –12 meters MLLW. Such depths in the area of the proposed harbor are suitable for cost effective rubblemound breakwater construction. The wave climate for the various directions of exposure is also suitable for cost effective rubblemound breakwater construction. The southern limit of the site is constrained by the existing cruise ship dock. The northern limit of the site is constrained by several large tide pools that are considered very productive marine habitat. A rubblemound breakwater structure would be required for wave protection from the various directions and would make use of the relatively shallow depths offshore. Many different harbor configurations were considered and optimized to find the most effective and least costly alternative at this site. Optimum locations for the breakwaters were determined so that the quantities of material were reasonable for the size of the basin being protected. The alternative plans at this site for a 50-year design life were laid out using breakwater alignments to protect the proposed entrance channel, maneuvering area, and mooring basin.

6.2.1 Alternative 1

This alternative, shown in figure A-17, incorporates the following rubblemound breakwaters: a 67-meter-long north spur breakwater, a 92-meter-long north breakwater, a 459-meter-long main breakwater, a 62.2-meter-long extension of the existing breakwater to the south, and a 49.9-meter-long south spur breakwater. The existing breakwater would be modified slightly by removing 46 meters of its length at its northern end, but the majority of its length would be unchanged. Two separate mooring basins would be created with this alternative. The 5.19-ha north basin could accommodate the larger range of vessels in the fleet with stalls oriented with the prevailing wind direction. The 2.25-ha south basin (existing) would remain unchanged in size and depth; however, additional wave protection would be provided and the existing float system would be removed and the replacement system reoriented. Smaller vessels in the fleet would use the south harbor basin. The north harbor entrance would be oriented with an approach around the end of the main breakwater and into the maneuvering area. This entrance channel configuration was preferred and recommended by the local sponsor. Marker pilings would be placed along the outside of the dredged channel limits to guide mariners into the harbor. The entrance channel into the south basin would be dredged and oriented similar to the existing south entrance channel.

North Harbor Basin. The north harbor basin would be step-dredged to depths of –4.9 meters and –4.3 meters MLLW. These depths are based on criteria given in Section 5 of this appendix. The deeper portion of the mooring basin would be located nearest the entrance channel. The shallower portion would be located farther into the harbor away from the entrance channel. The maneuvering area just inside the basin would be dredged to –4.9 meters MLLW. A total combined maneuvering and mooring basin area of approximately 5.19 ha would be available in the north basin for alternative 1.

South Harbor Basin. The south harbor basin would remain unchanged with respect to area and depth. Currently, the basin has depths of –3.7 meters and –4.3 meters MLLW. The deeper portion of the mooring basin would be located nearest the entrance channel. A total combined maneuvering and mooring basin area of approximately 2.25 ha would be available in the south basin for alternative 1.

Wave Heights. This alternative would meet the wave criteria established in Section 5 of this appendix along the floats inside both harbor basins. Breakwaters were positioned to reduce incident wave heights from the various directions of exposure to acceptable levels. The maximum wave heights in the mooring areas, based on the 50-year design incident wave, were calculated to be 0.29 meter and less. Progressively smaller wave heights would occur farther into the harbor mooring areas, as shown in the diffraction diagrams in figures A-18 and A-19. All directions of wave exposure were taken into account in determining the highest wave heights in the mooring area.

Circulation. Circulation in the harbor basins would be driven primarily by tidal action and by wind-driven surface water currents that contribute to mixing in the water column. Tides would drive circulation gyres in both basins. This alternative would incorporate basin geometries that would provide for adequate water circulation based on established criteria. The north and south basins would have tidal prism ratios of 0.53 and 0.55, respectively. The corners (15 percent of the basin's volume) of the north basin were checked as worst-case possible zones of stagnation. The northeast corner had the lowest value tidal prism ratio of 0.46.

The aspect ratios of the north and south basins were calculated to be 1.42 and 1.30, respectively. Good water quality and circulation are therefore expected in both harbor basins for alternative 1.

Shoaling. Shoaling of both entrance channels would not be expected since there is little evidence of significant long-shore transport of sediments at the site. There are no significant sources of sediment such as major rivers or creeks in the area. A small fillet of sandy material is present along the north side of the existing stub breakwater indicating some accumulation of material from the north. The proposed north stub breakwater would likely see a similar accumulation of material, but it would not reach the basin or proposed entrance channel. Similarly, the existing entrance channel has not required maintenance dredging and is not expected to with this alternative.

Construction Dredging. Dredging quantities and the characteristics of the materials were estimated from the hydrographic survey performed in August 2000 and the geotechnical investigation done in September 2000 (Appendix C). The dredged material would consist of clay, sand, gravel, cobbles, and boulders to the project limits. A total of 205,100 m^3 of clay, 5,600 m^3 of harder clay (diamicton), and 2,500 m^3 of dredging of boulders would be required for alternative 1. Dredged materials, with the exception of the boulders, would be disposed of in a designated area approximately 1.2 km offshore and east from the harbor.

Dredging work inside the harbor could be accomplished with a large clamshell dredge since clay, sand, and gravel would be encountered. The boulders would likely be removed at low tide with an excavator or dozer. According to the September 2000 geotechnical investigation in Appendix C, there would be areas of dredging where hard clay material would be encountered near the existing harbor entrance channel. It is not anticipated that this material would require blasting; however, heavy equipment and extra effort would likely be necessary to remove this material. Dredging equipment and methods would be left as an option for the contractor.

Side slopes for the basin would be dredged to 1.5H:1V and would require rock slope protection. The entrance channels would be dredged with 3H:1V side slopes and would not require slope protection.

A small channel would be dredged to accommodate fish passage along the shoreward end of the south stub breakwater. This channel would be 5 meters wide by 51 meters long and be dredged to a depth of +1.75 meters MLLW (replicating the existing fish passage at the northern limit of the existing harbor). This would allow continuous uninterrupted migration of fish through the harbor system by not altering the existing condition with respect to elevation and width of passage.

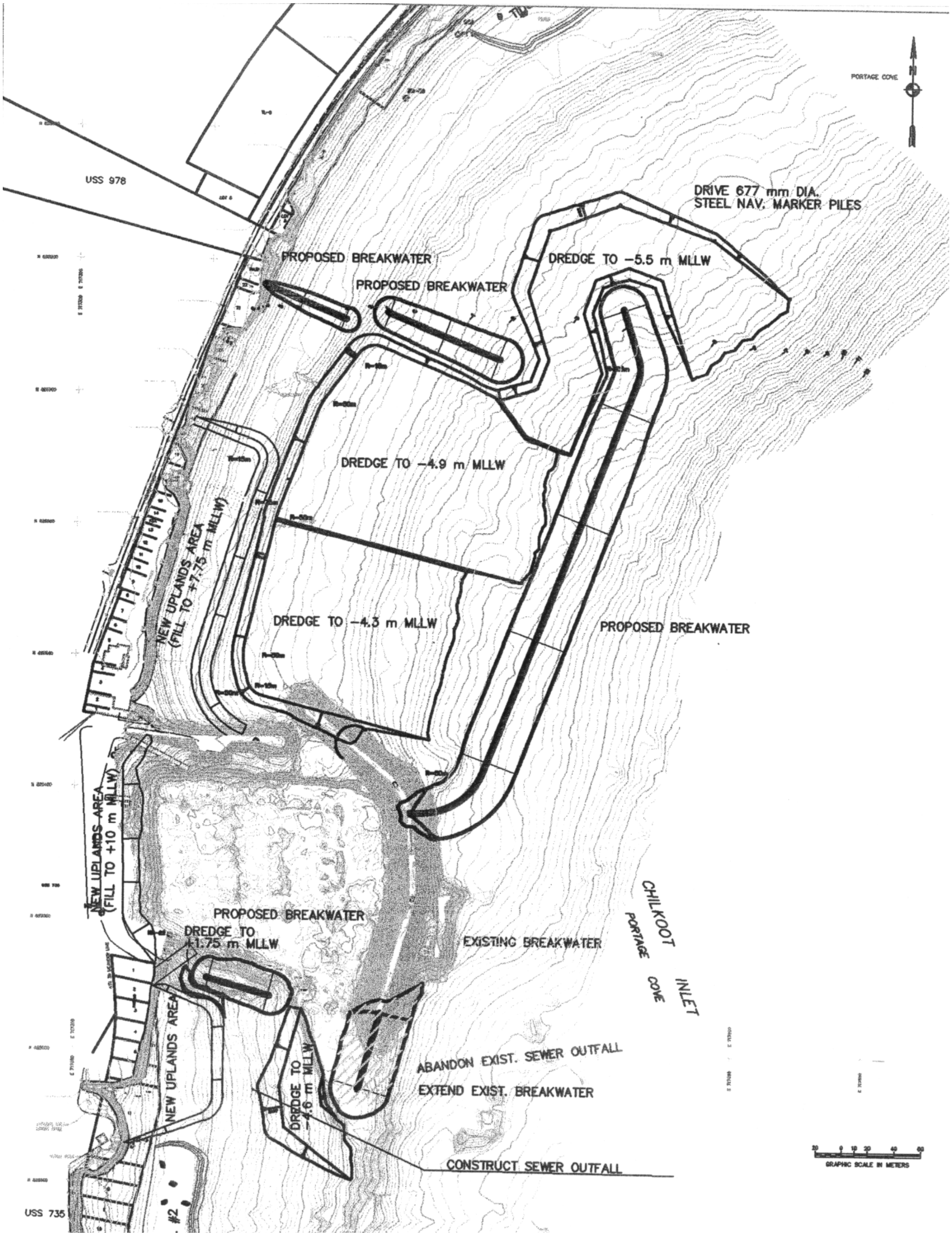


Figure A-17. Alternative 1

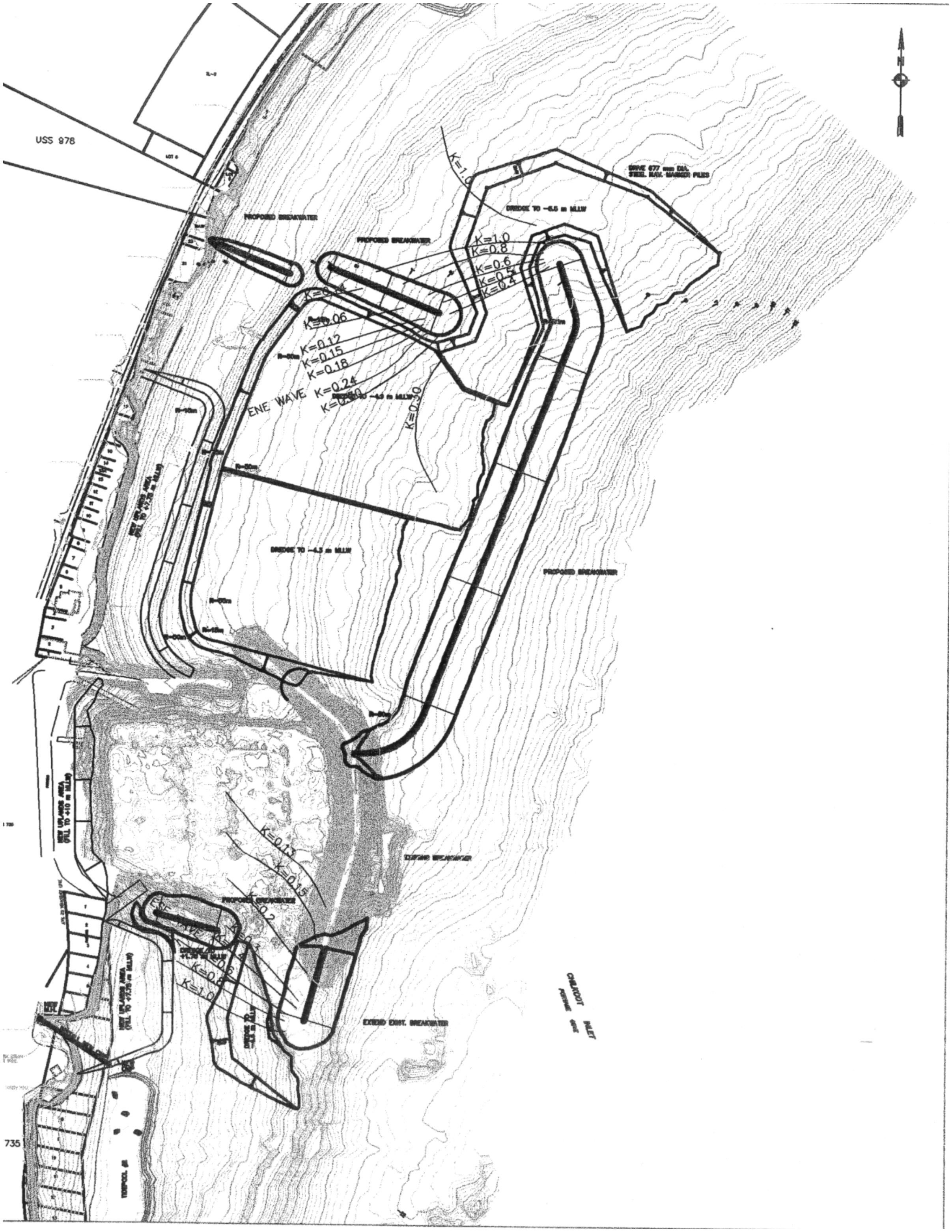


Figure A-18. Diffraction Diagram Alternative 1

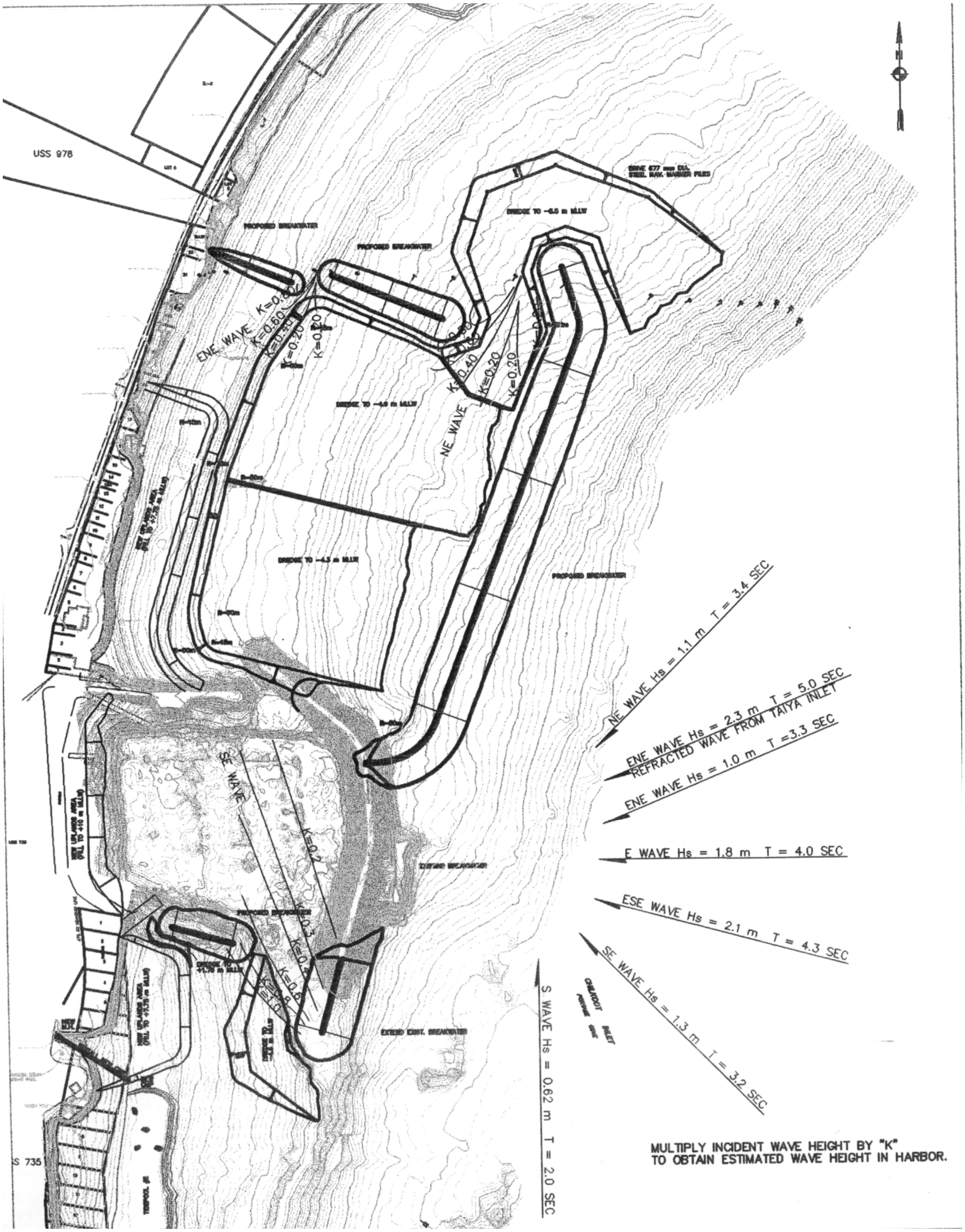


Figure A-19. Diffraction Diagram Alternative 1

Maintenance Dredging. Maintenance dredging is expected to be minimal. Dredging has not been required in the existing harbor since its previous expansion. Littoral transport of sediments generally appears to be from north to south. Some deposition is indicated on the north side of the existing breakwater. After construction, sediment is expected to be deposited in a similar manner north of the north stub breakwater. Maintenance dredging of the new harbor basin would be minimal during the project life. It would depend on the intensity of storm conditions and other factors over the years, but would be very infrequent if necessary at all.

Dredged Material Disposal. The dredged material would be disposed of in a deep-water area approximately 1.2 km east of the basin offshore from the existing harbor. A total of 210,700 m^3 of dredged material—mostly clay, sand, and gravel—would be deposited in the disposal area. The material could be excavated and transported a short distance to the disposal area efficiently.

Breakwaters. The positioning of the breakwaters would create entrance channel alignments allowing access from the east to the both basins. Maximum depths of water are –6.25 meters MLLW along the alignment of the breakwater. Foundation materials would be clay, sand, and gravel, which would serve as a suitable base for the rubblemound structures. The north stub and north breakwaters were separated by an 11.5-meter-wide gap for fish passage. The gap was sized to replicate the width and elevation of the existing fish passage at the existing harbor.

Rubblemound Breakwater Design. Methods described in the SPM using Hudson's equation were used to determine armor stone sizes for the rubblemound breakwaters. Stone size for the rubblemound breakwaters was determined using the significant wave heights presented in table A-4, along with sea-side side slopes of 1.5H:1V and harbor-side slope side slopes of 1.5H:1V and a K_d value of 4 for a non-breaking wave. A stone specific gravity of 2.89 was used in the calculations assuming that the local quarry in Haines would be the rock source. Armor stone ("A" rock) with a range of sizes from 1,136 kg maximum weight, 909 kg average weight, to 682 kg minimum weight would be used on the face of the breakwaters. Secondary stone ("B" rock) would range from 682 kg maximum weight, 91 kg average, to 68 kg minimum weight. Core material would range from 68 kg maximum, 9 kg average, to 0.5 kg minimum. Armor stone thickness would be 1.52 meters, and secondary stone thickness would be 0.76 meter. Cross sections for the rubblemound breakwaters are shown in figures A-20 and A-21. The recommended "A" stone size is considerably larger than armor stone on the existing breakwater. This is primarily due to a steeper side slope specified for the proposed breakwater.

The crest elevation of the breakwaters was determined by considering wave run-up, storm surge, and extreme high tides. Several methods were used to calculate wave run-up that resulted in an average value of 2.09 meters, including storm surge during design storm wave conditions. Using a still water level of +5.8 meters MLLW, a crest elevation of +7.89 meters MLLW was calculated. For simplicity, the existing breakwater crest elevation of +7.93 meters MLLW was selected for design. A crest width of 2.44 meters was selected based on the armor size and constructability considerations.

The "A" rock would extend to a 1.52-meter-wide toe configuration at elevation –4.25 meters MLLW on the seaside of the breakwaters. As natural depths vary toward shallower water, the toe elevation would vary as well. The harbor side "A" rock would extend to a minimum elevation of –1.52 meters MLLW.

A total of 46,600 m^3 of "A" rock, 29,900 m^3 of "B" rock, and 114,300 m^3 of "Core" rock would be required for construction of the breakwaters. Approximately 10,600 m^3 of rock from the existing breakwater would be removed and used as additional “Core” rock in the new breakwaters.

Uplands. Uplands for alternative 1 would be created by filling in tidelands along the shoreline in the new north harbor basin, in the existing basin, and south of the existing basin. Fill material could be derived from waste rock during quarry operations and hauled to the site for placement. A total uplands area of 3.06 ha would be created and available for use. Given the total harbor area of 10.5 ha, an uplands to harbor area ratio of 0.29 was calculated. This is significantly less than the required 0.40 ratio; however, an exemption to the established criterion was approved by the ADOT/PF and the local sponsor in this case due to the local opposition from several property owners along the waterfront to uplands fill. There will, however, be sufficient uplands area associated with alternative 1 to provide the needed facilities to support the harbor.

6.2.2 Alternative 2

Alternative 2 is very similar in configuration to alternative 1. The difference between the two is primarily the size of the basin. The breakwaters are slightly farther offshore in deeper water and extend farther to the north on the north side. This alternative, shown in figure A-22 incorporates the following rubblemound breakwaters: a 72.9-meter-long north spur breakwater, a 109.4-mete- long north breakwater, a 489.1-meter-long main breakwater, a 62.2-meter-long extension of the existing breakwater to the south, and a 49.9-meter-long south spur breakwater. The existing breakwater would be modified slightly by removing 46 meters of its length at its northern end, but the majority of its length would be unchanged. Two separate mooring basins would be created with this alternative. The 6.57-ha north basin could accommodate the larger range of vessels in the fleet with stalls oriented with the prevailing wind direction. The 2.25-ha south basin (existing) would remain unchanged in size and depth; however additional wave protection would be provided and the existing float system would be removed and reoriented. Smaller vessels in the fleet would use the south harbor basin. The north harbor entrance would be oriented with an approach around the end of the main breakwater and into the maneuvering area. This entrance channel configuration was again preferred by the local sponsor. Marker pilings would be placed along the outside of the dredged channel limits to guide mariners into the harbor. The entrance channel into the south basin would be dredged and oriented similar to the existing south entrance channel.

North Harbor Basin. The north harbor basin would be step-dredged to depths of –4.9 meters and –4.3 meters MLLW. These depths are based on criteria given in Section 5 of this appendix. The deeper portion of the mooring basin would be located nearest the entrance channel. The shallower portion would be located farther into the harbor away from the entrance channel. The maneuvering area just inside the basin would be dredged to –4.9 meters MLLW. A total combined maneuvering and mooring basin area of approximately 6.57 ha would be available in the north basin for alternative 2.

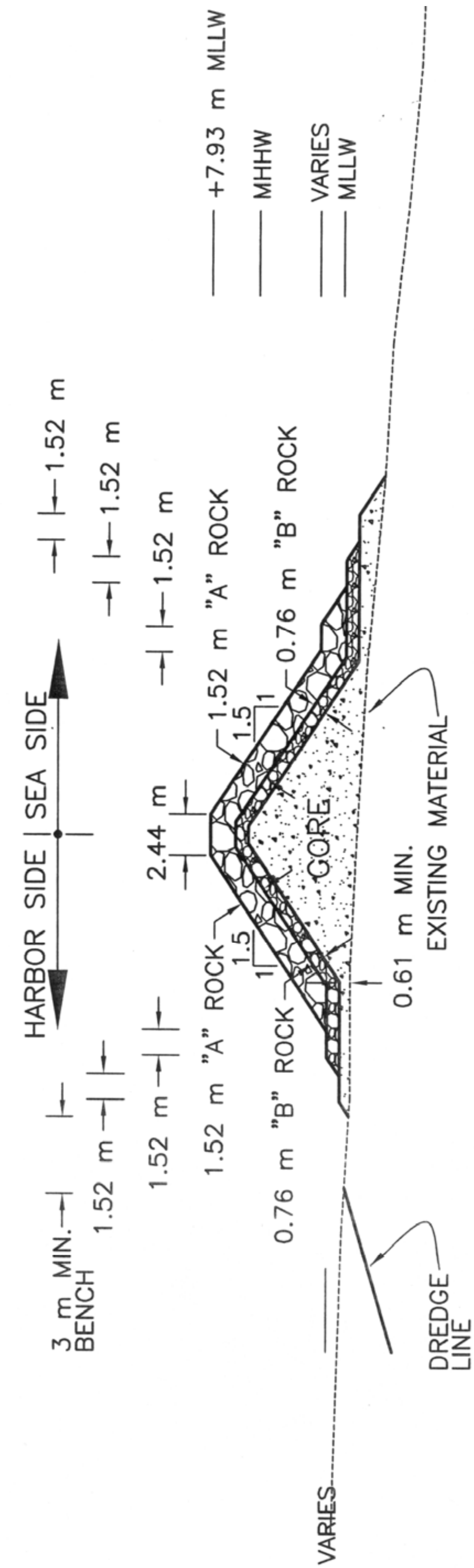


Figure A-20. Typical Breakwater Cross-Section 1

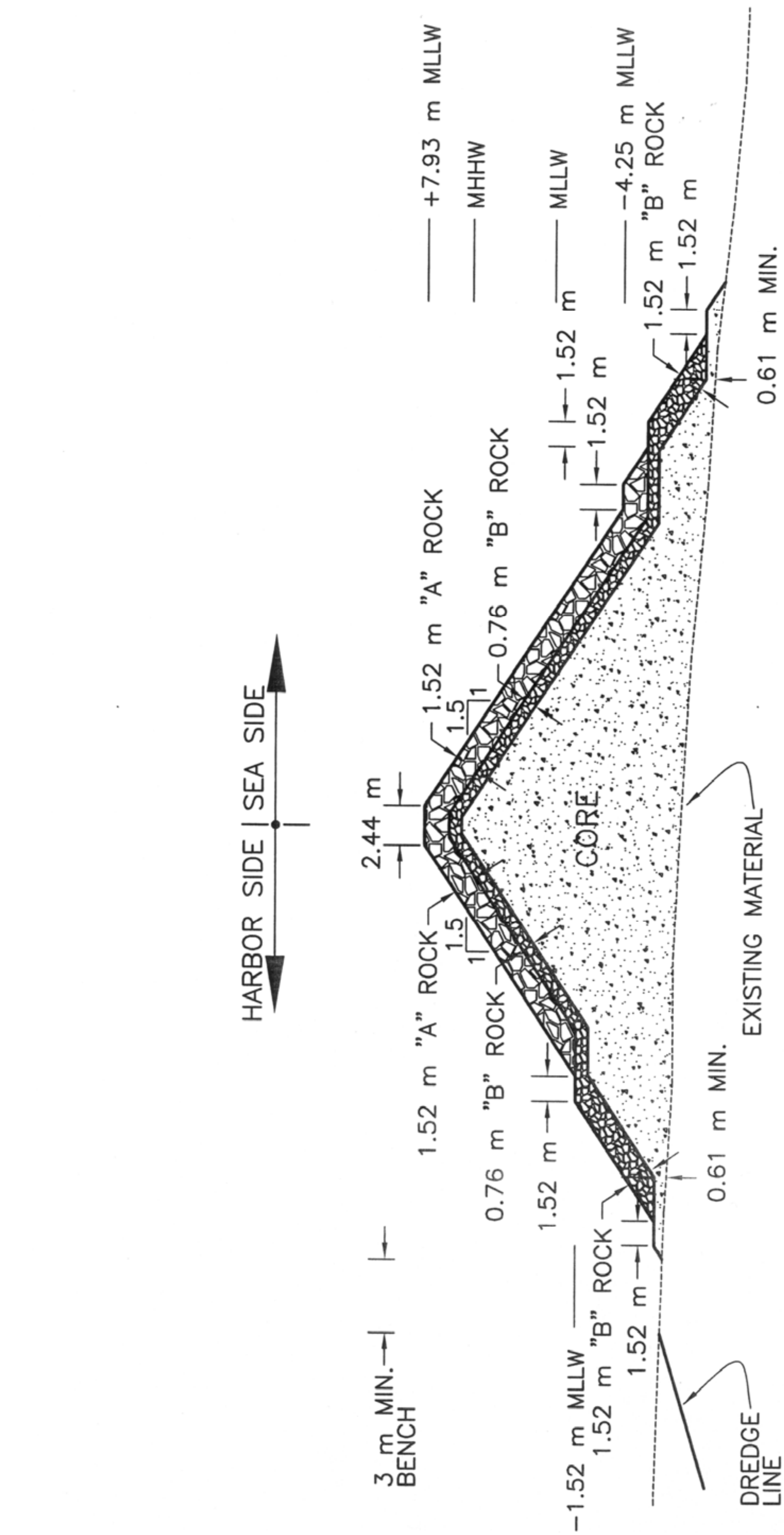


Figure A-21. Typical Breakwater Cross-Section 2

South Harbor Basin. The south harbor basin would remain unchanged with respect to area and depth. Currently, the basin has depths of –3.7 meters and –4.3 meters MLLW. The deeper portion of the mooring basin would be located nearest the entrance channel. A total combined maneuvering and mooring basin area of approximately 2.25 ha would be available in the south basin for alternative 2, the same as for alternative1.

Wave Heights. This alternative would meet the wave criteria established in Section 5 of this appendix along the floats and inside both harbor basins. Breakwaters were positioned to reduce incident wave heights from the various directions of exposure to acceptable levels. The maximum wave heights in the mooring areas, based on the 50-year design incident wave, were calculated to be 0.29 meter and less. Progressively smaller wave heights would occur farther into the harbor mooring areas, as shown in the diffraction diagrams in figures A-23 and A-24. All directions of wave exposure were taken into account in determining the highest wave heights in the mooring area.

Circulation. Circulation in the harbor basins would be driven primarily by tidal action and by wind-driven surface water currents that contribute to mixing in the water column. Tides would drive circulation gyres in both basins. This alternative would incorporate basin geometries that would provide for adequate water circulation based on established criteria. The north and south basins would have tidal prism ratios of 0.44 and 0.55, respectively. The corners (15 percent of the basin’s volume) of the north basin were checked as worst-case possible zones of stagnation. The northeast corner had the lowest value tidal prism ration of 0.46.

The aspect ratios of the north and south basins were calculated to be 1.46 and 1.30, respectively. Good water quality and circulation are therefore expected in both harbor basins for alternative 2.

Shoaling. Shoaling of both entrance channels would not be expected since there is little evidence of significant long-shore transport of sediments at the site. There are no significant sources of sediment such as major rivers or creeks in the area. A small fillet of sandy material is present along the north side of the existing stub breakwater indicating some accumulation of material from the north. The proposed north stub breakwater would likely see a similar accumulation of material, but it would not reach the basin or proposed entrance channel. Similarly, the existing entrance channel has not required maintenance dredging and is not expected to with this alternative.

Construction Dredging. Dredging quantities and characteristics of materials were estimated from the hydrographic survey performed in August 2000 and the geotechnical investigation done in September 2000 (Appendix C). The dredged material would consist of clay, sand, gravel, cobbles, and boulders to the project limits. A total of 223,700 m^3 of clay, 5,600 m3 f harder clay (diamicton), and

2,800 m^3 of dredging of boulders would be required for alternative 2. Dredged materials, with the exception of the boulders, would be disposed of in a designated area approximately 1.2 km offshore and east from the harbor.

Dredging work inside the harbor could be accomplished with a large clamshell dredge since clay, sand, and gravel would be encountered. The boulders would likely be removed at low tide with an excavator or dozer. According to the September 2000 geotechnical investigation in Appendix C, there would be areas of dredging where hard clay material would be encountered near the existing harbor entrance channel. It is not anticipated that this material would require blasting; however, heavy equipment and extra effort would likely be necessary to remove this material. Dredging equipment and methods would be left as an option for the contractor.

Side slopes for the basin would be dredged to 1.5H:1V and would require rock slope protection. The entrance channels would be dredged with 3H:1V side slopes and would not require slope protection.

A small channel would be dredged to accommodate fish passage along the shoreward end of the south stub breakwater. This channel would be 5 meters wide by 51 meters long and be dredged to a depth of +1.75 meters MLLW (replicating the existing fish passage at the northern limit of the existing harbor). This would allow continuous uninterrupted migration of fish through the harbor system by not altering the existing condition with respect to elevation and width of passage.

Maintenance Dredging. Maintenance dredging is expected to be minimal. Dredging has not been required in the existing harbor since its previous expansion. Littoral transport of sediments appears generally to be from north to south. Some deposition is indicated on the north side of the existing breakwater. After construction, sediment is expected to be deposited in a similar manner north of the north stub breakwater. Maintenance dredging of the new harbor basin would be minimal during the project life. It would depend on storm conditions and other factors over the years, but would be very infrequent if necessary at all.

Dredged Material Disposal. The dredged material would be disposed of in a deep-water area approximately 1.2 km east of the basin offshore from the existing harbor. A total of 229,300 m^3 of dredged material—mostly clay, sand, and gravel—would be deposited in the disposal area. The material could be excavated and transported a short distance to the disposal area efficiently.

Breakwaters. Positioning of the breakwaters would create entrance channel alignments allowing access from the east to the both basins. Maximum depths of water are –7.25 meters MLLW along the alignment of the breakwater. Foundation materials would be clay, sand, and gravel, which would serve as a suitable base for the rubblemound structures. The north stub and north breakwaters would be separated by an 11.5-meter-wide gap for fish passage. The gap was sized to replicate the width and elevation of the fish passage at the existing harbor.

Rubblemound Breakwater Design. Similar breakwater design methodology described for alternative 1 was used for alternative 2. This resulted in the same crest height, crest width, rock size and layer thickness, and toe configurations.

A total of 48,900 m^3 of “A” rock, 32,600 m^3 of “B” rock, and 135,000 m^3 of “Core” rock would be required for construction of the breakwaters. Approximately 10,600 m^3 of rock from the existing breakwater would be removed and used as additional “Core” rock in the new breakwaters.

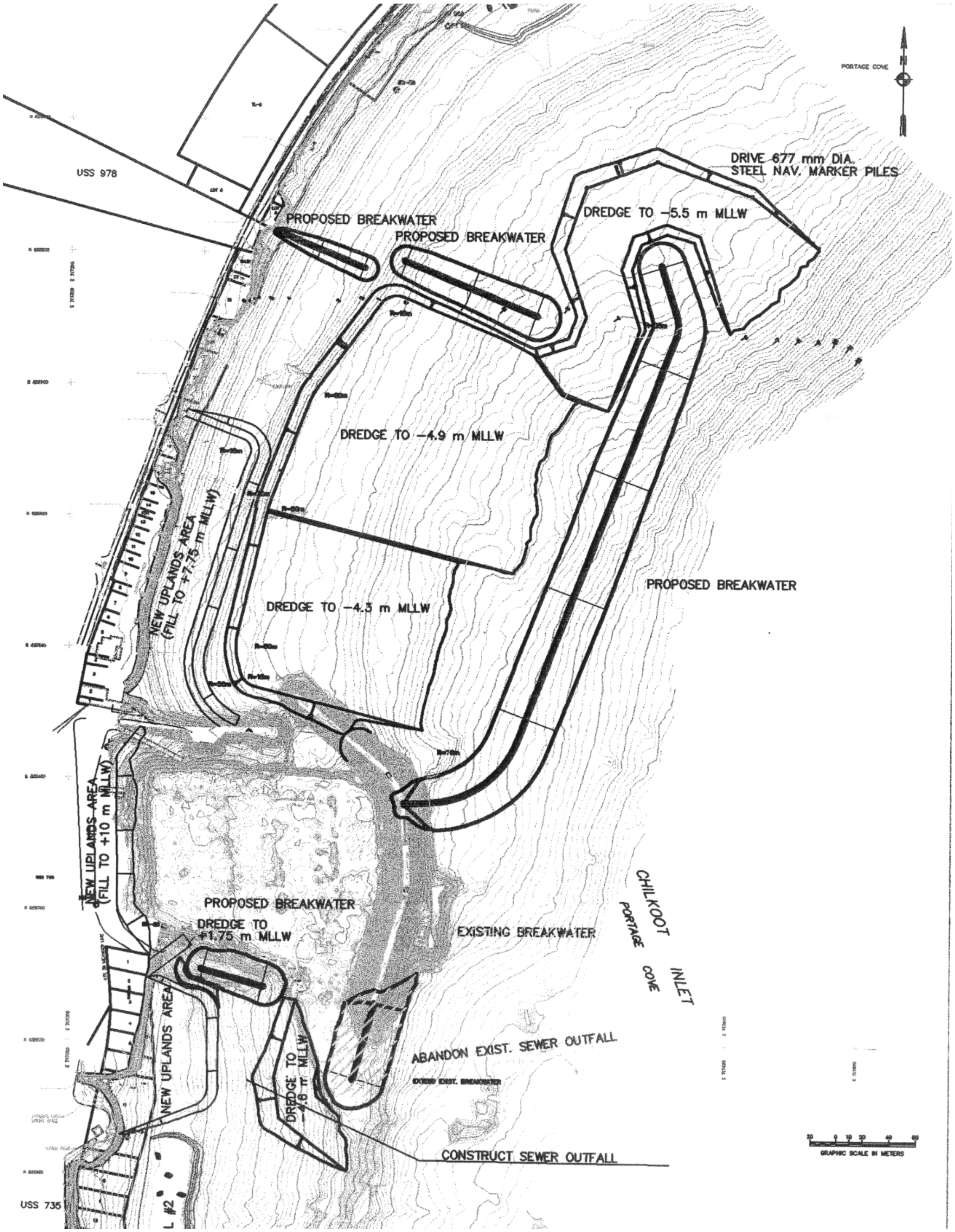


Figure A-22. Alternative 2

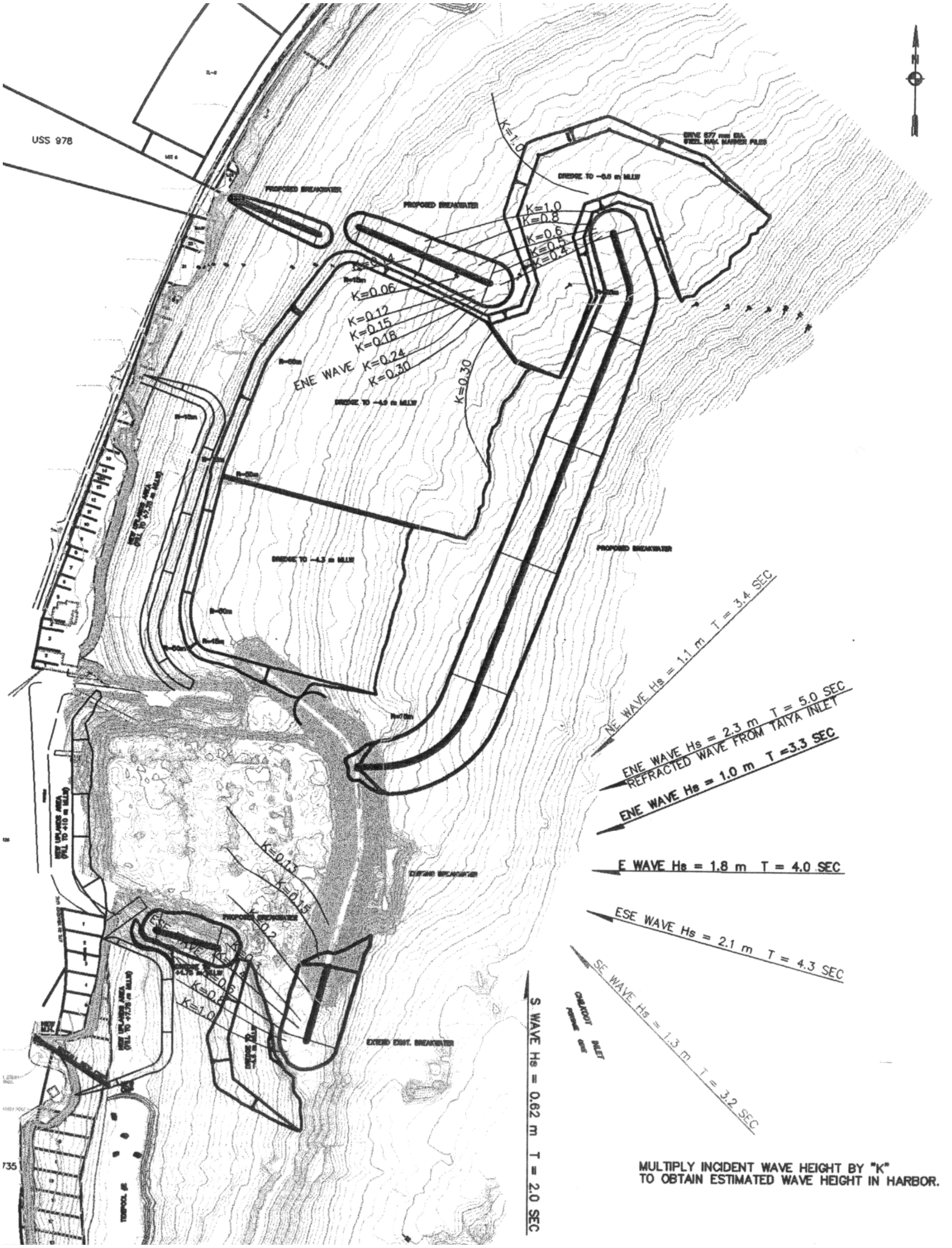


Figure A-23. Diffraction Diagram Alternative 2

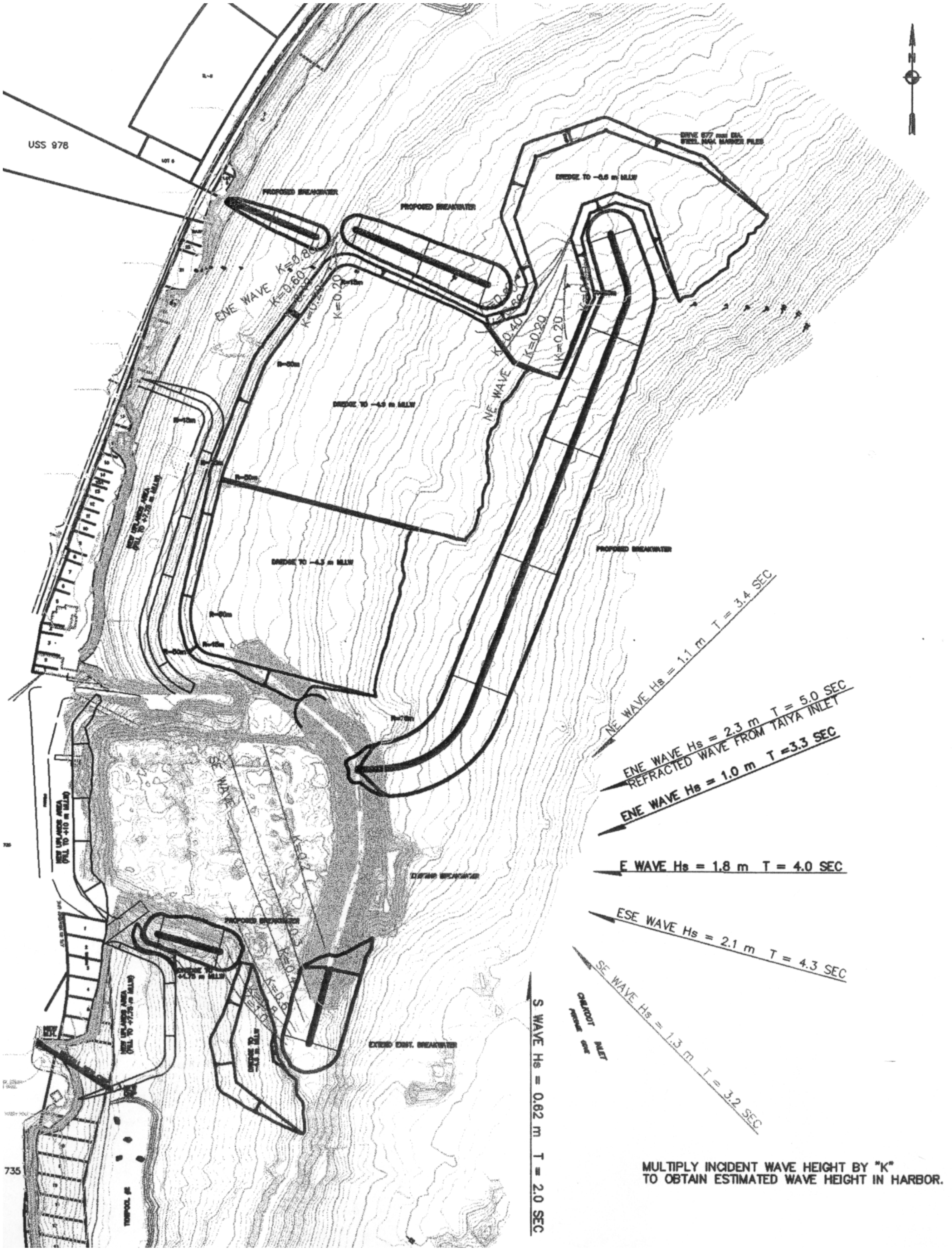


Figure A-24. Diffraction Diagram Alternative 2

Uplands. Uplands for alternative 2 would be created by filling in tidelands along the shoreline in the new north harbor basin, in the existing basin, and south of the existing basin. Fill material could be derived from waste rock during quarry operations and hauled to the site for placement. A total uplands area of 3.06 ha would be created and available for use. Given the total harbor area of 11.88 ha, an uplands to harbor area ratio of 0.26 was calculated. This is again significantly less than the required 0.40 ratio; however, an exemption to the established criterion was approved by the ADOT/PF and the local sponsor for this alternative also. There would, however, be sufficient uplands area associated with alternative 2 to provide the needed facilities to support the harbor.

6.2.3 Alternative 3

The layout for alternative 3 was provided by the local sponsor in coordination with the ADOT/PF (the local sponsor's technical advisor). This alternative was designed to maximize the available mooring area within the north basin and to allow future use of the main breakwater for access to a future dock outside the harbor. The main breakwater would be located farther offshore in deeper water and extend farther to the north on the north side than with the previous two alternatives. The north spur and first portion of the main breakwater would have a widened crest to accommodate vehicle access for a future dock to be located at the turn-around. This alternative, shown in figure A-25 incorporates the following rubblemound breakwaters: a 103- meter-long north spur breakwater, a 191-meter-long first portion of the main breakwater, a turnaround portion of the main breakwater with a radius of 18.5 meters, a 325.9-meter-long second portion of the main breakwater, a 51.2-meter-long extension of the existing breakwater to the south, and a 33.3-meter-long south spur breakwater. The existing breakwater would be unchanged except for the extension of the head to the south and the creation of a new fish passage channel near its northern angle point. A concrete floating breakwater would be constructed and placed along the western edge of the new north entrance channel. Two separate mooring basins would be created with this alternative. The 7.02-ha north basin could accommodate the larger range of vessels in the fleet with stalls oriented with the prevailing wind direction. The 2.25-ha south basin (existing) would remain unchanged in size and depth; however, additional wave protection would be provided and the existing float system would be removed and reoriented. Smaller vessels in the fleet would use the south harbor basin. The north harbor entrance would be oriented with an approach around the end of the main breakwater and into the maneuvering area. This entrance channel configuration represents the preference of the local sponsor for this alternative. The entrance channel into the south basin would be dredged and oriented similar to the existing south entrance channel.

North Harbor Basin. The north harbor basin would be step-dredged to depths of

–4.3 meters and –4.9 meters MLLW with the deeper portion of the basin located in the northern half. These depths are based on criteria given in Section 5 of this appendix. The shallower portion of the mooring basin would be located nearest the entrance channel. The maneuvering area just inside the basin would be left un-dredged since natural depths are sufficient for maneuvering. A total combined maneuvering and mooring basin area of approximately 7.02 ha would be available in the north basin for alternative 3.

South Harbor Basin. The south harbor basin would remain unchanged with respect to area and depth. Currently, the basin has depths of –3.7 meters and –4.3 meters MLLW. The deeper portion of the mooring basin would be located nearest the entrance channel. A total combined maneuvering and mooring basin area of approximately 2.25 ha would be available in the south basin for alternative 3, the same as for alternatives 1 and 2.

Wave Heights. Breakwaters were positioned to reduce incident wave heights from the various directions of exposure to acceptable levels. The maximum wave heights in the mooring areas, based on the 50-year design incident wave, were calculated. Wave heights acceptable to the local sponsor were determined. Progressively smaller wave heights would occur farther into the harbor mooring areas, as shown in the diffraction diagrams in figures A-26 and A-27. All directions of wave exposure were taken into account in determining the highest wave heights in the mooring area.

Circulation. Circulation in the harbor basins would be driven primarily by tidal action and by wind-driven surface water currents that contribute to mixing in the water column. Tides would drive circulation gyres in both basins. This alternative would incorporate basin geometries that would provide for adequate water circulation based on established criteria. The north and south basins would have tidal prism ratios of 0.49 and 0.55, respectively. The corners (15 percent of the basin’s volume) of the north basin were checked as worst-case possible zones of stagnation. The northeast corner had the lowest value tidal prism ration of 0.46.

The aspect ratios of the north and south basins were calculated to be 1.41 and 1.30, respectively. Good water quality and circulation are therefore expected in both harbor basins for alternative 3.

Shoaling. Shoaling of both entrance channels would not be expected since there is little evidence of significant long-shore transport of sediments at the site. There are no significant sources of sediment such as major rivers or creeks in the area. A small fillet of sandy material is present along the north side of the existing stub breakwater indicating some accumulation of material from the north. The proposed north stub breakwater would likely see a similar accumulation of material, but it would not reach the basin or proposed entrance channel. The north entrance channel would be located in deep water far offshore and would not be expected to experience shoaling. Similarly, the existing entrance channel has not required maintenance dredging and is not expected to with this alternative.

Construction Dredging. Dredging quantities and characteristics of materials were estimated from the hydrographic survey performed in August 2000 and the geotechnical investigation done in September 2000 (Appendix C). The dredged material would consist of clay, sand, gravel, cobbles, and boulders to the project limits. A total of 142,600 m^3 of clay, 3,300 m^3 of harder clay (diamicton), and

2,200 m^3 of dredging of boulders would be required for alternative 3. Dredged materials, with the exception of the boulders, would be disposed of in a designated area approximately 1.2 km offshore and east from the harbor.

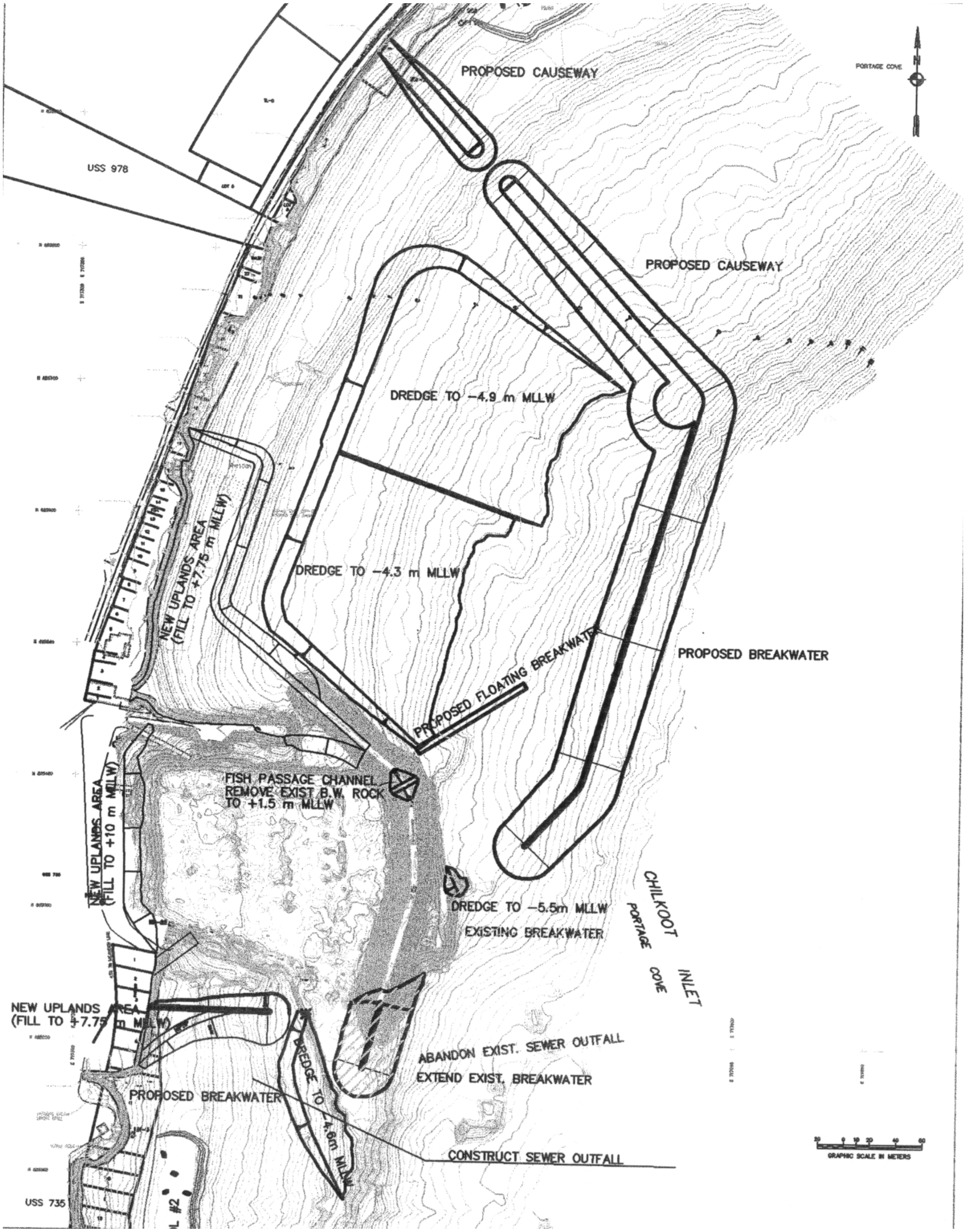


Figure A-25. Alternative 3

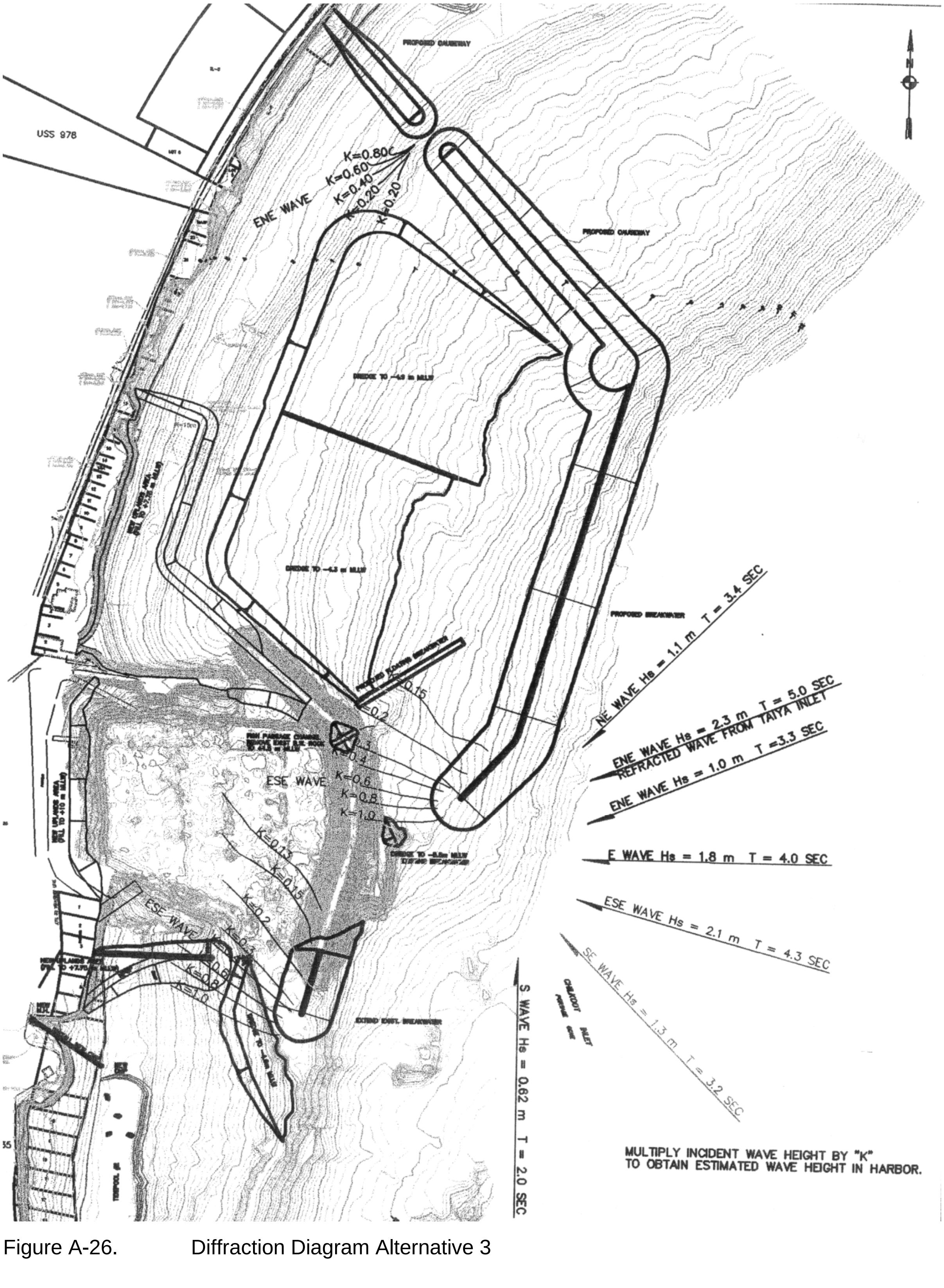


Figure A-26. Diffraction Diagram Alternative 3

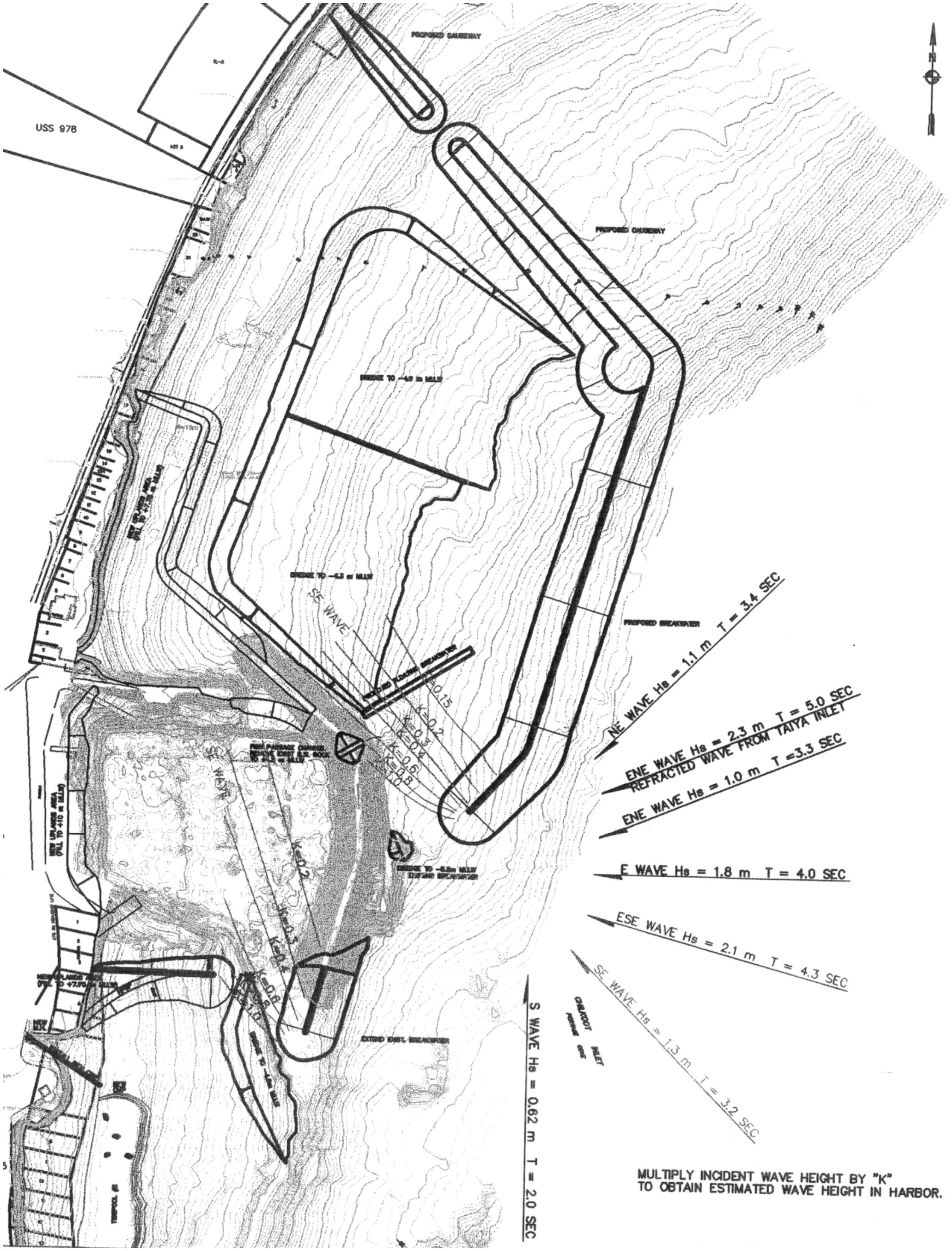


Figure A-27. Diffraction Diagram Alternative 3

Dredging work inside the harbor could be accomplished with a large clamshell dredge since clay, sand, and gravel would be encountered. The boulders would likely be removed at low tide with an excavator or dozer. According to the September 2000 geotechnical investigation in Appendix C, there would be areas of dredging where hard clay material would be encountered near the existing harbor entrance channel. It is not anticipated that this material would require blasting; however, heavy equipment and extra effort would likely be necessary to remove this material. Dredging equipment and methods would be left as an option for the contractor.

Side slopes for the basin would be dredged to 3H:1V and would not require rock slope protection. The entrance channels would be dredged with 3H:1V side slopes and also would not require slope protection.

A small channel would be excavated through the existing breakwater to accommodate fish passage from the north basin into the south basin and vice versa. This channel would be 4 meters wide by 22 meters long and be excavated to a depth of +1.5 meters MLLW. Side slopes would be 3H:1V on the inside and 1V:1.5H on the outside. This would allow migration of fish through the harbor system since this alternative would close off the existing fish passage with uplands fill.

Maintenance Dredging. Maintenance dredging is expected to be minimal. Dredging has not been required in the existing harbor since its previous expansion. Littoral transport of sediments appears generally to be from north to south. Some deposition is indicated on the north side of the existing breakwater. After construction, sediment is expected to be deposited in a similar manner north of the north stub breakwater. Maintenance dredging of the new harbor basin would be minimal during the project life. It would depend on storm conditions and other factors over the years but would be very infrequent if necessary at all.

Dredged Material Disposal. The dredged material would be disposed of in a deep-water area approximately 1.2 km east of the basin offshore from the existing harbor. A total of 146,200 m^3 of dredged material—mostly clay, sand, and gravel—would be deposited in the disposal area. The material could be excavated and transported a short distance to the disposal area efficiently.

Breakwaters. The positioning of the breakwaters would create entrance channel alignments allowing access from the east to the both basins. Maximum depths of water are –9.25 meters MLLW along the alignment of the main breakwater. Foundation materials would be clay, sand, and gravel, which would serve as a suitable base for the rubblemound structures. The north stub and first portion of the main breakwaters would be separated by a 4 meter-wide gap for fish passage. The elevation of the gap was set at the +0.80-meter MLLW contour.

Rubblemound Breakwater Design. Similar breakwater design methodology described for alternatives 1 and 2 was used for alternative 3. This resulted in the same crest height, rock size and layer thickness, and toe configurations for the seaside. The crest width for the north spur and first portion of the main breakwater for alternative 3 was increased to 13.8 meters. “A” rock would only extend up to the full crest height of +7.93 meters MLLW on the seaside. The crest itself would be “core” rock and presumably surfaced with sub-base and base course material in the future for vehicle access. The harbor side would have “B” rock only since no overtopping would be anticipated over the widened crest portions. The turn-around portion of the main breakwater would be widened further to a radius of 18.5 meters, with a similar cross-section to the north spur and first portion of the main breakwater. Figure A-21 shows a typical section of north spur and first portion of the main breakwater. The second portion of the main breakwater and south breakwater extensions and south spur breakwaters would use the same cross-section design as those for alternatives 1 and 2.

A total of 38,600 m^3 of “A” rock, 39,300 m^3 of “B” rock, and 225,300 m^3 of “Core” rock would be required for construction of the breakwaters. Approximately 2,600 m^3 of rock from the existing breakwater would be removed and used as additional “Core” rock in the new breakwaters.

Floating Breakwater Design. The floating breakwater design for alternative 3 was performed by the ADOT/PF. The structure would reduce residual wave heights to acceptable levels inside the harbor by attenuation. Based on wave height reduction criteria in the SPM, the floating breakwater dimensions required were calculated to be 4.88 meters wide and 2.00 meters high (0.6-meter freeboard and 1.4-meter draft). The length of the structure would be 95.72 meters to provide adequate wave protection and allow for use as a mooring float for larger vessels. A concrete box-type design was selected for the structure. It would be supported by steel pilings driven into the existing bottom. A typical section of the floating breakwater is shown in Figure A-29.

Uplands. Uplands for alternative 3 would be created by filling in tidelands along the shoreline in the new north harbor basin, in the existing basin, and south of the existing basin. The existing fish passage channel would be filled in as well. Fill material could be derived from waste rock during quarry operations and hauled to the site for placement. A total uplands area of 2.66 ha would be created and available for use. Given the total harbor area of 11.93 ha, an uplands to harbor area ratio of 0.22 was calculated. This again is significantly less than the required 0.40 ratio; however, an exemption to the established criterion was approved by the ADOT/PF and the local sponsor for this alternative also. There will, however, be sufficient uplands area associated with alternative 3 to provide the needed facilities to support the harbor.

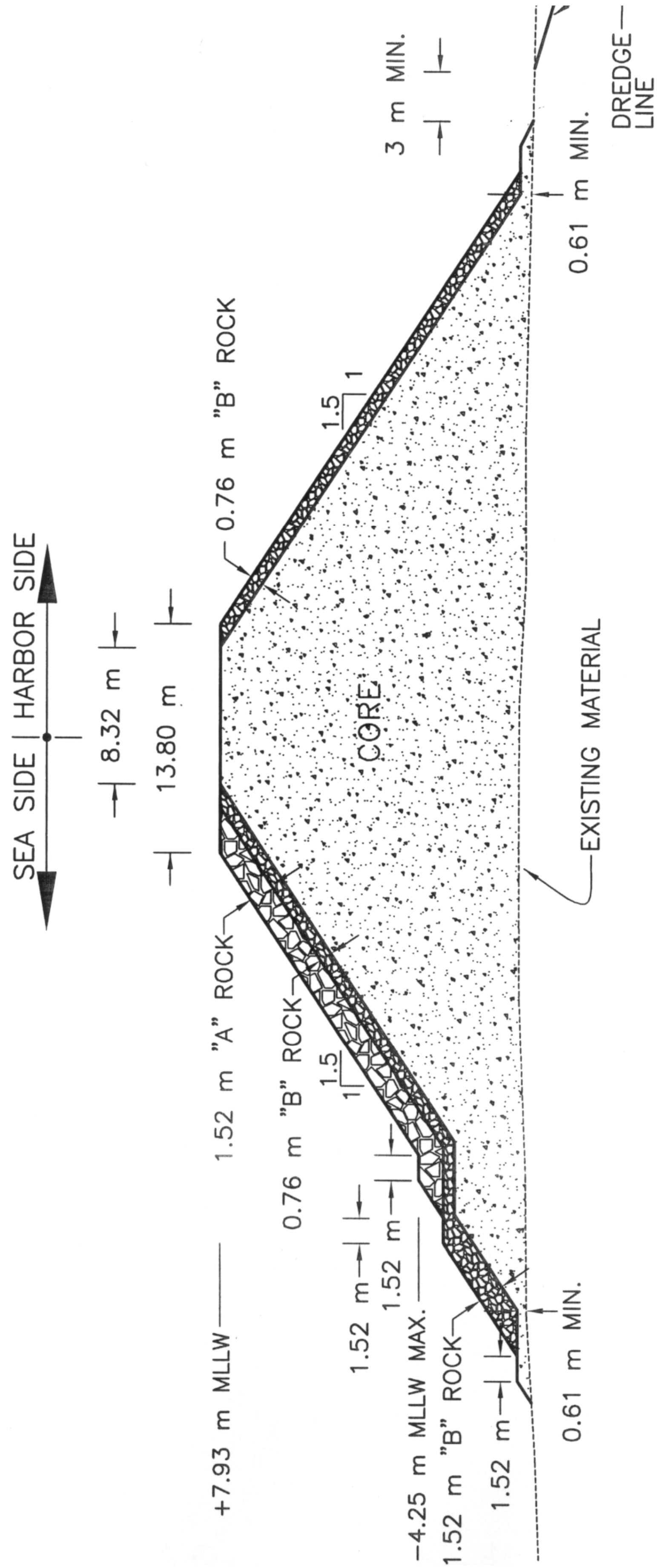


Figure A-28. Typical Breakwater Cross-Section (causeway)

6.2.4 Alternative 4

The layout for alternative 4 was also provided by the local sponsor in coordination with the ADOT/PF (the local sponsor's technical advisor). This alternative is very similar to alternative 3; however, it incorporated a smaller mooring basin. It would allow future use of the main breakwater for access to a future dock outside the harbor similar to alternative 3. The main breakwater, however, is located closer inshore and in shallower water. The north spur and first portion of the main breakwater would have a widened crest to accommodate vehicle access for a future dock to be located at the turn-around. This alternative, shown in figure A-30, incorporates the following rubblemound breakwaters: a 103-meter-long north spur breakwater, a 154-meter-long first portion of the main breakwater, a turnaround portion of the main breakwater with a radius of 18.5 meters, a 316-meter-long second portion of the main breakwater, a 46.7-meter-long stub breakwater attached to the existing breakwater, a 51.2 meter- long-extension of the existing breakwater to the south, and a 33.3-meter-long south spur breakwater. The existing breakwater would be unchanged except for the extension of the head to the south and the creation of a new fish passage channel near its northern angle point. Two separate mooring basins would be created with this alternative. The 6.60-ha north basin could accommodate the larger range of vessels in the fleet with stalls oriented with the prevailing wind direction. The 2.25-ha south basin (existing) would remain unchanged in size and depth; however, additional wave protection would be provided and the existing float system would be removed and reoriented. Smaller vessels in the fleet would use the south harbor basin. The north harbor entrance would be oriented with an approach around the end of the main breakwater and into the maneuvering area. This entrance channel configuration represents the preference of the local sponsor for this alternative. The entrance channel into the south basin would be dredged and oriented similar to the existing south entrance channel.

North Harbor Basin. The north harbor basin would be step-dredged to depths of –4.3 meters and –4.9 meters MLLW, with the deeper portion of the basin located in the northern half. These depths are based on criteria given in Section 5 of this appendix. The shallower portion of the mooring basin would be located nearest the entrance channel. The maneuvering area just inside the basin would be left un-dredged since natural depths are sufficient for maneuvering. A total combined maneuvering and mooring basin area of approximately 6.60 ha would be available in the north basin for alternative 4.

South Harbor Basin. The south harbor basin would remain unchanged with respect to area and depth. Currently, the basin has depths of –3.7 meters and –4.3 meters MLLW. The deeper portion of the mooring basin would be located nearest the entrance channel. A total combined maneuvering and mooring basin area of approximately 2.25 ha would be available in the south basin for alternative 4, the same as for alternatives 1, 2, and 3.

Wave Heights. Breakwaters were positioned to reduce incident wave heights from the various directions of exposure to acceptable levels. The maximum wave heights in the mooring areas, based on the 50-year design incident wave, were calculated. Wave heights acceptable to the local sponsor were determined. Progressively smaller wave heights would occur farther into the harbor mooring areas, as shown in the diffraction diagrams in figures A-31 and A-32. All directions of wave exposure were taken into account in determining the highest wave heights in the mooring area.

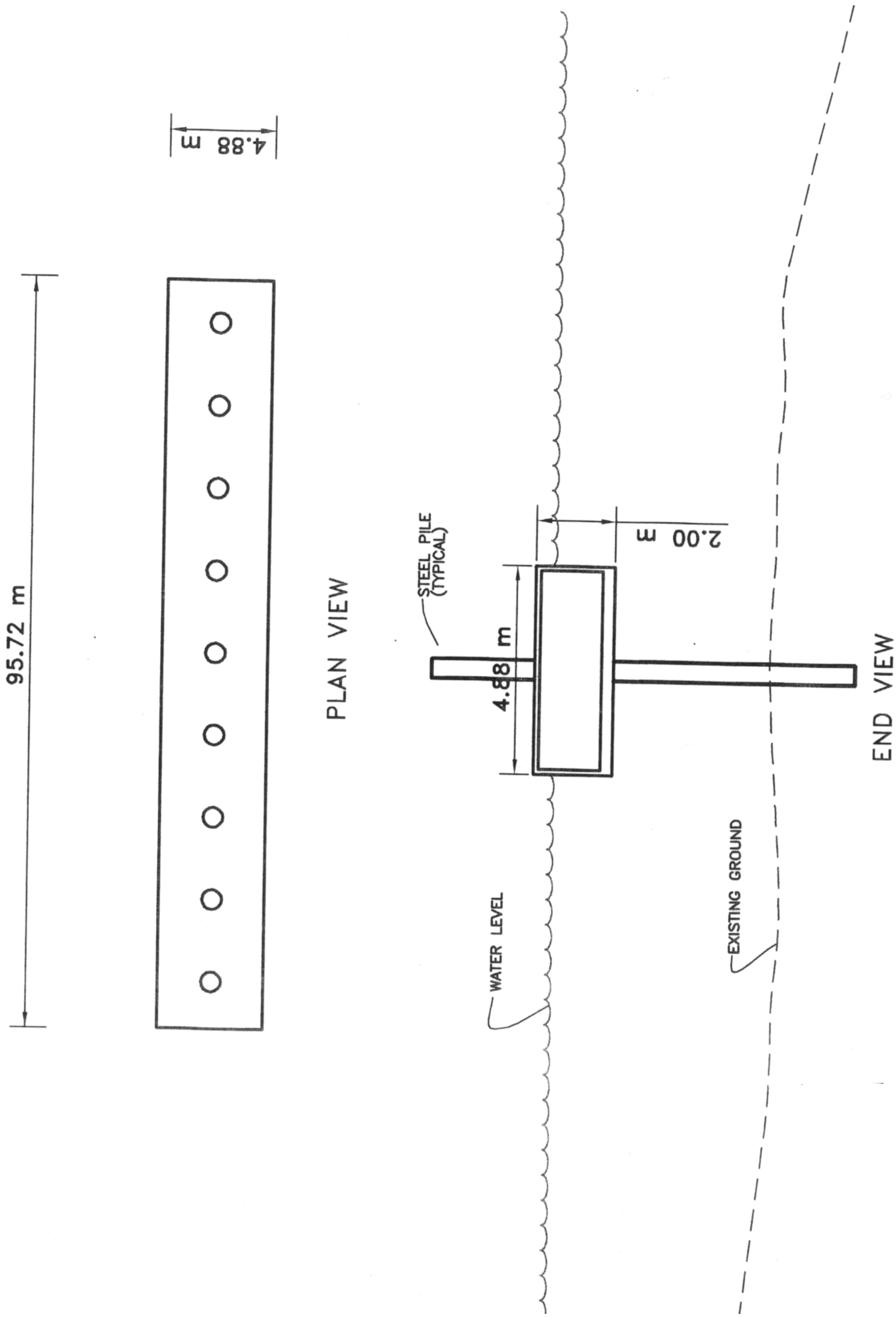


Figure A-29. Floating Breakwater Cross Section

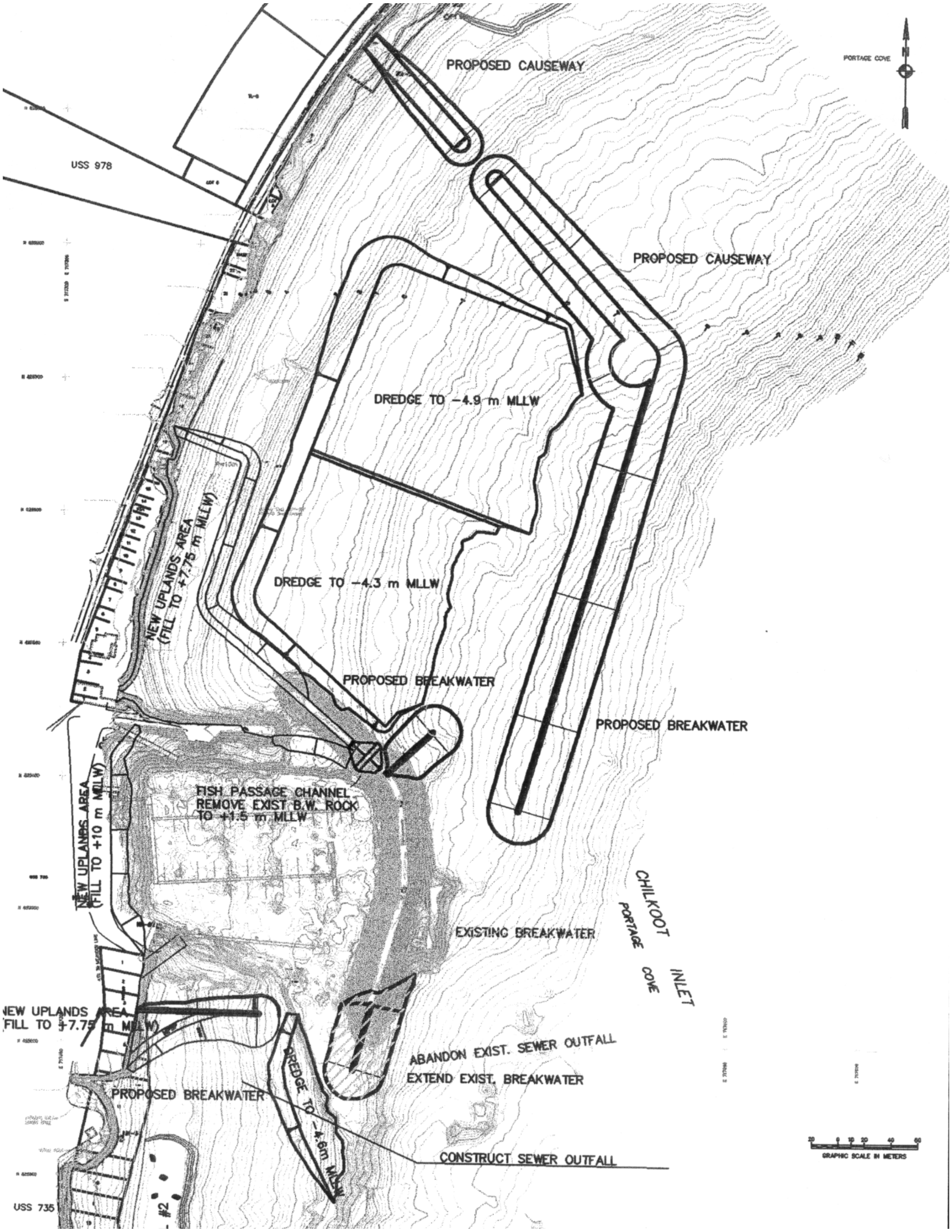


Figure A-30. Alternative 4

Circulation. Circulation in the harbor basins would be driven primarily by tidal action and by wind-driven surface water currents that contribute to mixing in the water column. Tides would drive circulation gyres in both basins. This alternative would incorporate basin geometries that would provide for adequate water circulation based on established criteria. The north and south basins would have tidal prism ratios of 0.53 and 0.55, respectively. The corners (15 percent of the basin's volume) of the north basin were checked as worst-case possible zones of stagnation. The northeast corner had the lowest value tidal prism ratio of 0.45.

The aspect ratios of the north and south basins were calculated to be 1.67 and 1.30, respectively. Good water quality and circulation are therefore expected in both harbor basins for alternative 4.

Shoaling. Shoaling of both entrance channels would not be expected since there is little evidence of significant long-shore transport of sediments at the site. There are no significant sources of sediment such as major rivers or creeks in the area. A small fillet of sandy material is present along the north side of the existing stub breakwater indicating some accumulation of material from the north. The proposed north stub breakwater would likely see a similar accumulation of material, but it would not reach the basin or proposed entrance channel. The north entrance channel would be in deep water far offshore and would not be expected to experience shoaling. Similarly, the existing entrance channel has not required maintenance dredging and would not be expected to with this alternative.

Construction Dredging. Dredging quantities and characteristics of materials were estimated from the hydrographic survey performed in August 2000 and the geotechnical investigation done in September 2000 (Appendix C). The dredged material would consist of clay, sand, gravel, cobbles, and boulders to the project limits. A total of 159,900 m^3 of clay, 3,300 m^3 of harder clay (diamicton), and 1,900 m^3 of dredging of boulders would be required for alternative 4. Dredged materials, with the exception of the boulders, would be disposed of in a designated area approximately 1.2 km offshore and east from the harbor.

Dredging work inside the harbor could be accomplished with a large clamshell dredge since clay, sand, and gravel would be encountered. The boulders would likely be removed at low tide with an excavator or dozer. According to the September 2000 geotechnical investigation in Appendix C, there would be areas of dredging where hard clay material would be encountered near the existing harbor entrance channel. It is not anticipated that this material would require blasting; however, heavy equipment and extra effort would likely be necessary to remove this material. Dredging equipment and methods would be left as an option for the contractor.

Side slopes for the basin would be dredged to 3H:1V and would not require rock slope protection. The entrance channel would be dredged with 3H:1V side slopes and would also not require slope protection.

A small channel would be excavated through the existing breakwater to accommodate fish passage from the north basin into the south basin and vice versa. This channel would be similar to that for alternative 3.

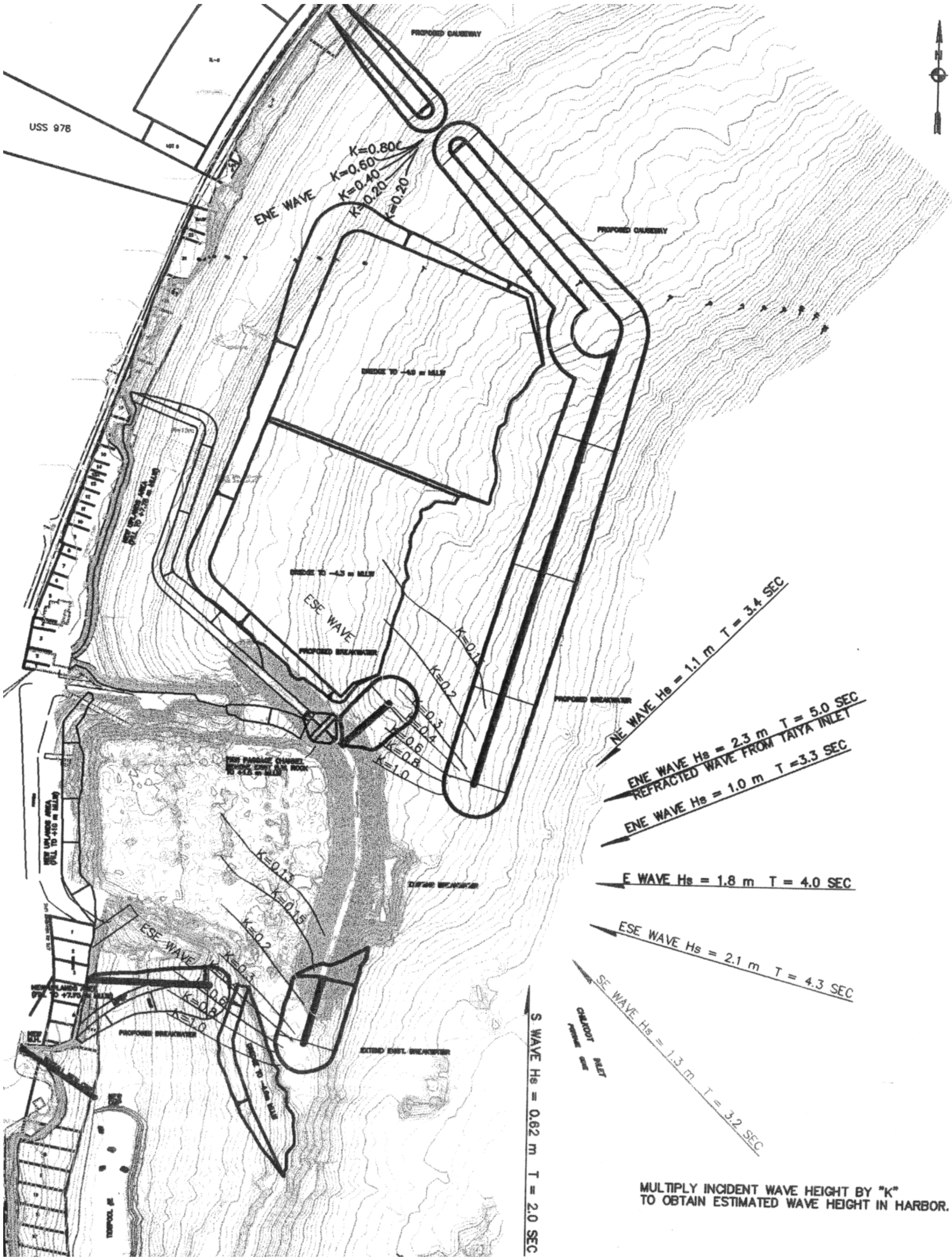


Figure A-31. Diffraction Diagram Alternative 4

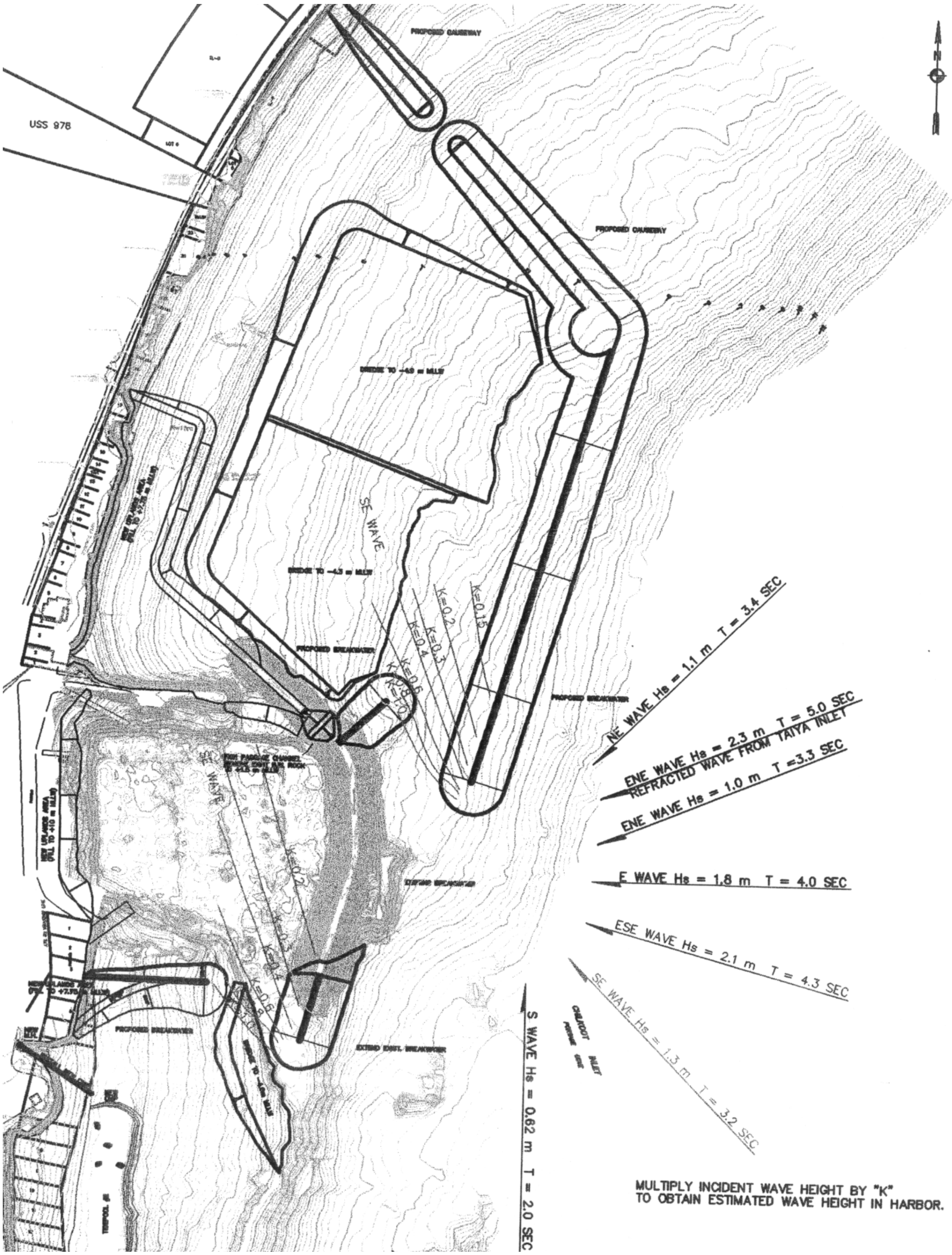


Figure A-32. Diffraction Diagram Alternative 4

Maintenance Dredging. Maintenance dredging is expected to be similar to that for alternative 3.

Dredged Material Disposal. The dredged material would be disposed of in a deep-water area approximately 1.2 km east of the basin offshore of the existing harbor. A total of 163,200 m^3 of dredged material—mostly clay, sand, and gravel—would be deposited in the disposal area. The material could be excavated and transported a short distance to the disposal area efficiently.

Breakwaters. The positioning of the breakwaters would create entrance channel alignments allowing access from the east to the both basins. Maximum depths of water are –7.75 meters MLLW along the alignment of the main breakwater. Foundation materials would be clay, sand, and gravel, which would serve as a suitable base for the rubblemound structures. The north stub and first portion of the main breakwaters were separated by a 4-meter-wide gap for fish passage. The elevation of the gap was set at the +0.80-meter MLLW contour.

Rubblemound Breakwater Design. Similar breakwater design methodology described for alternatives 1, 2, and 3 was used for alternative 4. This resulted in the same crest height, rock size and layer thickness, and toe configurations for the seaside. The crest width for the north spur and first portion of the main breakwater for alternative 4 was set to 13.8 meters, the same as that for alternative 3. “A” rock would only extend up to the full crest height of +7.93 meters MLLW on the seaside. The crest itself would be “core” rock and presumably surfaced with sub-base and base course material in the future for vehicle access. The harbor side would have “B” rock only since no overtopping would be anticipated over the widened crest portions. The turn-around portion of the main breakwater would be widened further to a radius of 18.5 meters with a similar cross-section to the north spur and first portion of the main breakwater. The first and second portions of the main breakwater and south breakwater extensions and south spur breakwaters would use the same cross-section design as those for alternatives 3.

A total of 38,500 m^3 of “A” rock, 39,100 m^3 of “B” rock, and 191,100 m^3 of “Core” rock would be required for construction of the breakwaters. Approximately 2,600 m^3 of rock from the existing breakwater would be removed and used as additional “Core” rock in the new breakwaters.

Uplands. Uplands for alternative 4 would be created by filling in tidelands along the shoreline in the new north harbor basin, in the existing basin, and south of the existing basin. The existing fish passage channel would be filled in as well. Fill material could be derived from waste rock during quarry operations and hauled to the site for placement. A total uplands area of 2.66 ha would be created and available for use. Given the total harbor area of 11.51 ha, an uplands to harbor area ratio of 0.23 was calculated. This again is significantly less than the required 0.40 ratio; however, an exemption to the established criterion was approved by the ADOT/PF and the local sponsor for this alternative also. There will, however, be sufficient uplands area associated with alternative 4 to provide the needed facilities to support the harbor.

7.0 PLAN IMPLEMENTATION

7.1 Aids To Navigation

As part of the construction of the project, navigation marker bases would be constructed at the heads of the breakwaters. Discussions with the U.S. Coast Guard have been conducted to assure that necessary marking of the new entrance channels was considered. New navigation lights would be incorporated into the head of the new breakwaters for any of the alternatives. The Coast Guard would install the navigation lights and signage after construction is completed.

For alternatives 1 and 2, navigation aid marker pilings would be driven at the angle points along the inside of the north entrance channel. These pilings would be signed and lighted in red. The Coast Guard would provide the signage and lighting. For alternative 3, the north end of the floating breakwater would be signed and lighted.

7.2 Operation and Maintenance Plan

Operation of the completed mooring basin portion of the project would be the city of Haines' responsibility. The Federal Government would be responsible for the breakwaters and the entrance channel portions of the project. The Alaska District, Corps of Engineers, would visit the site periodically to inspect the 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 basins. Maintenance requirements for the 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.

Minimal maintenance dredging is anticipated with any of the four alternatives. It is estimated that essentially no maintenance dredging would be necessary for the first 30 years after the project is constructed. However, over a period of 20 years following that, it is estimated that a total quantity of 1,400 m^3 of material (mostly sand and silt) may require dredging in the north mooring basin. This material would be near shore and would presumably be excavated and disposed of in an upland disposal site. The local sponsor would be responsible for such maintenance dredging. No maintenance dredging in the Federal entrance channels or maneuvering areas over the project life is anticipated.

The breakwaters were designed to be stable for the 50-year predicted wave conditions. Therefore, no significant loss of stone from the rubblemound structure 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 15 years. Since stone quality would be strictly specified in the contract, little to no armor stone degradation is anticipated.

Maintenance of the floating breakwater would be the responsibility of the local sponsor since it is intended to be used as a mooring float or dock for small cruise vessels. Condition of the concrete, flotation, connections, anchoring system, and cathodic protection would need to be evaluated, and maintenance requirements would be determined by the sponsor. It is estimated that approximately 5 percent of the connections and 2 percent of the concrete deck area would require repairs at 15-year intervals based on past performance of similar structures around the State.

7.3 Detailed Quantity Estimates

Detailed estimates of quantities for Federal dredging and breakwaters for all three alternatives were performed for this appendix. Dredging quantities were estimated for local portions of the project as well. Quantity estimates were based on hydrographic surveys performed in August 2000 by contract. The AutoCAD and Land Development software, as well as Excel spreadsheet quantity calculation programs were used to determine the quantities. The quantities were checked and verified to be within 10 percent by the ADOT/PF using independent methods.

7.4 Construction Schedule

Major construction items include the rubblemound breakwaters, floating breakwater, and dredging. The rubblemound breakwaters would likely be constructed first. Work on the dredging and disposal would then be completed. The floating breakwater for alternative 3 would likely be constructed in Tacoma, Washington, and barged to the site for assembly and positioning. The time needed to construct the project is estimated at 36 months. Construction scheduling would minimize conflict with the continued use of the existing harbor facilities in Portage Cove Harbor or in Letnikof Cove. Also, the cruise ship dock facilities in Portage Cove would remain operational during construction. Project specifications would detail time restrictions for the contractor to conduct certain activities.

The inner harbor facilities such as the float system, docks, upland facilities, etc. would be constructed after the Federal project was completed. Such facilities would be the responsibility of the local sponsor and would be constructed under separate contract.

7.5 Initial Dredged Material Disposal

For the four alternatives considered, all dredged material would be disposed of in the offshore disposal site discussed in Section 6 of this appendix. The site is located in approximately 55 meters of water, approximately 1.2 km east and offshore from the existing harbor in Portage Cove. A square area measuring 0.47 km by 0.47 km would be designated for disposal of the dredged material. The material would likely be transported to the site by barge or dump scow. Assuming that each dump is made at roughly the same location, a single mound of material would be created having dimensions of approximately 29 meters high off the existing bottom and a footprint area of 2.37 ha for alternative 2. Slightly smaller dimensions of the dredged material disposal mound would occur with alternatives 1, 3, and 4 since they would have lesser quantities of dredged material. Side slopes on the mound would be approximately 1V:2H to 1V:3H.

The large majority of dredged material according to the Geotechnical Appendix is lean clay. This material is highly cohesive and would likely be in the form of large clumps when dredged and transported to the disposal site. As it is dumped, it would likely fall to the bottom in the same form with minimal dissolution into the surrounding water column. A relatively small quantity of fines including silts and sands, however, would be suspended and transported by prevailing currents in the form of a plume. Calculations on this plume size indicate that its maximum extent would be approximately 750 meters to the south on an ebb tide and 465 meters to the north on the flood tide. These extents assumed that the material would be dumped during maximum tidal currents. If the material was dumped during days with lower tide ranges or at slack water, the extent of the plume would be considerably less. Some mixing could occur if wind velocities are high at the time of disposal; however, wind generated currents are relatively insignificant with depth in the water column.

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