APPENDIX A
HYDRAULIC DESIGN
EMERGENCY BANK STABIIZATION
DILLINGHAM, ALASKA

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APPENDIX A
HYDRAULIC DESIGN

1.0 INTRODUCTION

1.1. Appendix Purpose

This hydraulic design appendix provides background for design of the major construction features, including operation and maintenance, of the Dillingham Bank Stabilization project. This project consists of a revetment on the west side of the harbor, extending approximately 1,100 feet from the Bristol Fuels dock, a breakwater approximately 371 feet long extending east into the Nushagak River from the west side of the harbor, and a revetment on the city dock side of the harbor, extending 850 feet from the eastern terminus of the existing harbor sheet-pile bulkhead, and wrapping around the existing dredged material containment berm. This dredged material disposal site is no longer used.

1.2. Project Purpose

The city of Dillingham requested the Corps of Engineers to conduct a feasibility study for providing erosion protection in the vicinity of Scandinavian Beach and the city-dock side of the harbor near the Peter Pan Facility and City Park (figure A-0). This study was initiated to determine the most environmentally acceptable and cost effective means to stabilize the riverbank on both sides of the harbor entrance. Protecting this area from further erosion was identified as a critical issue facing the community. This erosion has affected Scandinavian Beach, effectively removing a spit of land that had historically provided protection for the harbor. With the deterioration of Scandinavian Beach, more storm waves have been entering the harbor causing damage to vessels and mooring structures. On the city dock side of the harbor, the east terminus of the sheet-pile protection has been outflanked, resulting in erosion behind the seawall and in the City Park. Continued bank erosion east of this protection has proceeded to the point where the seaward side of the berm containing the Peter Pan Dredged Material Disposal Site (containment berm) has been compromised. If unchecked, continued erosion will impact Dillingham's small boat harbor by loss of access to the south parking lot, south boat ramp, associated utilities, and eventually erosion will reach the harbor itself. To address these issues, the following objectives were identified to accomplish a bank stabilization project at Dillingham prior to initiating the engineering analysis:

a. Provide erosion protection for the project area, minimizing the damages experienced under the conditions that produce the design wave.
b. Replace storm protection for the harbor that was previously provided by Scandinavian Beach.

1.3. Project Location

The project site is in Dillingham, Alaska along the Nushagak River at the extreme northern end of Nushagak Bay in Northern Bristol Bay. The community is located at the confluence of the Wood and Nushagak rivers, approximately 325 miles southwest of

Anchorage (59°02'25'' N Latitude and 158°27'30'' W Longitude), and is in the Bristol Bay Recording District. The Dillingham project vicinity and location maps are shown in figure A-0.

2.0 CLIMATOLOGY, METEOROLOGY, HYDROLOGY

2.1 Climatology

Dillingham's climate is a product of the maritime conditions in Bristol Bay and continental conditions from inland areas. From May through October, the winds are predominantly from the south over the bay. This moist airflow results in a more moderate climate than is found farther inland. From November through April, the prevailing winds bring drier cooler air from the interior. This modifies what would usually be a warm maritime climate to a characteristically cold continental climate. Wind gusts of up to 60 to 70 miles per hour may occur between December and March. Average summer temperatures range from 35 °F to 63 °F; average winter temperatures range from 4 °F to 30 °F. Annual precipitation is 26 inches, with 82.9 inches of snowfall. Heavy fog is common in July and August. The Nushagak River is ice-free from June through November. Table A-1 is a summary of climatic data from 1951 through the present for the Dillingham Airport.

Table A-1. Climate Data Summary for Dillingham FAA Airport

Mean annual temperature	33.8° F
January mean temperature	16.1° F
July mean temperature	55° F
Record High Temperature	92° F (June 1953)
Record Low Temperature	-53° F (January 1989)
Mean annual precipitation	26.0 inches
Mean annual snowfall	82.9 inches

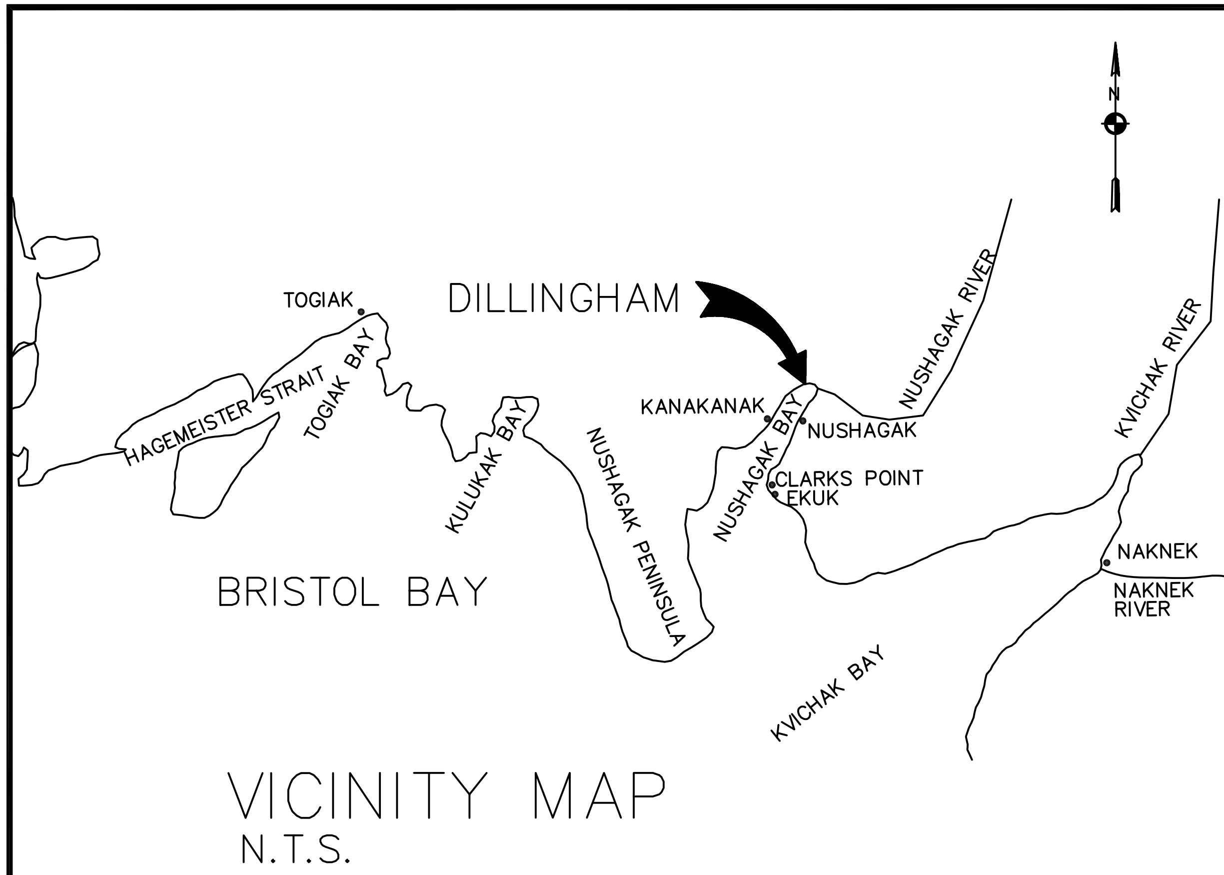
Source: Western Regional Climate Center

2.2 Wind Data

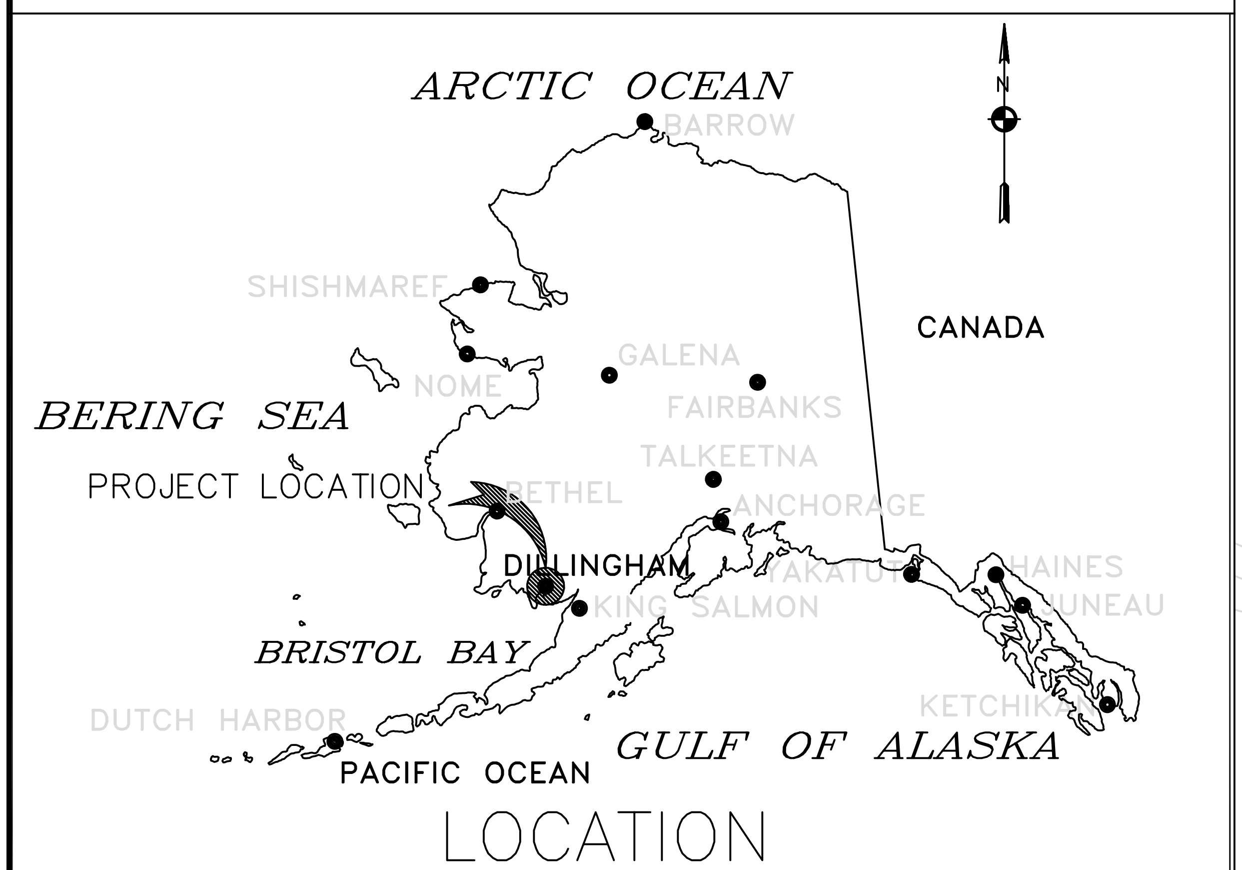
Wind data for Dillingham was obtained from the Air Force Combat Climatology Center in Ashville, North Carolina. Wind roses for the area indicate that winds are predominantly from the north to northeast from October to April and range from the west-southwest to northeast from May to September. See figure A-1 for annual winds. These values were converted to 1-hour average winds using the methods specified in the 1984 Shore Protection Manual (SPM). Table A-2 shows the wind data used in this study, summarized by direction and return period. Figure A-1 shows a wind rose for Dillingham.

Table A-2. Dillingham Wind Data Summary (knots)

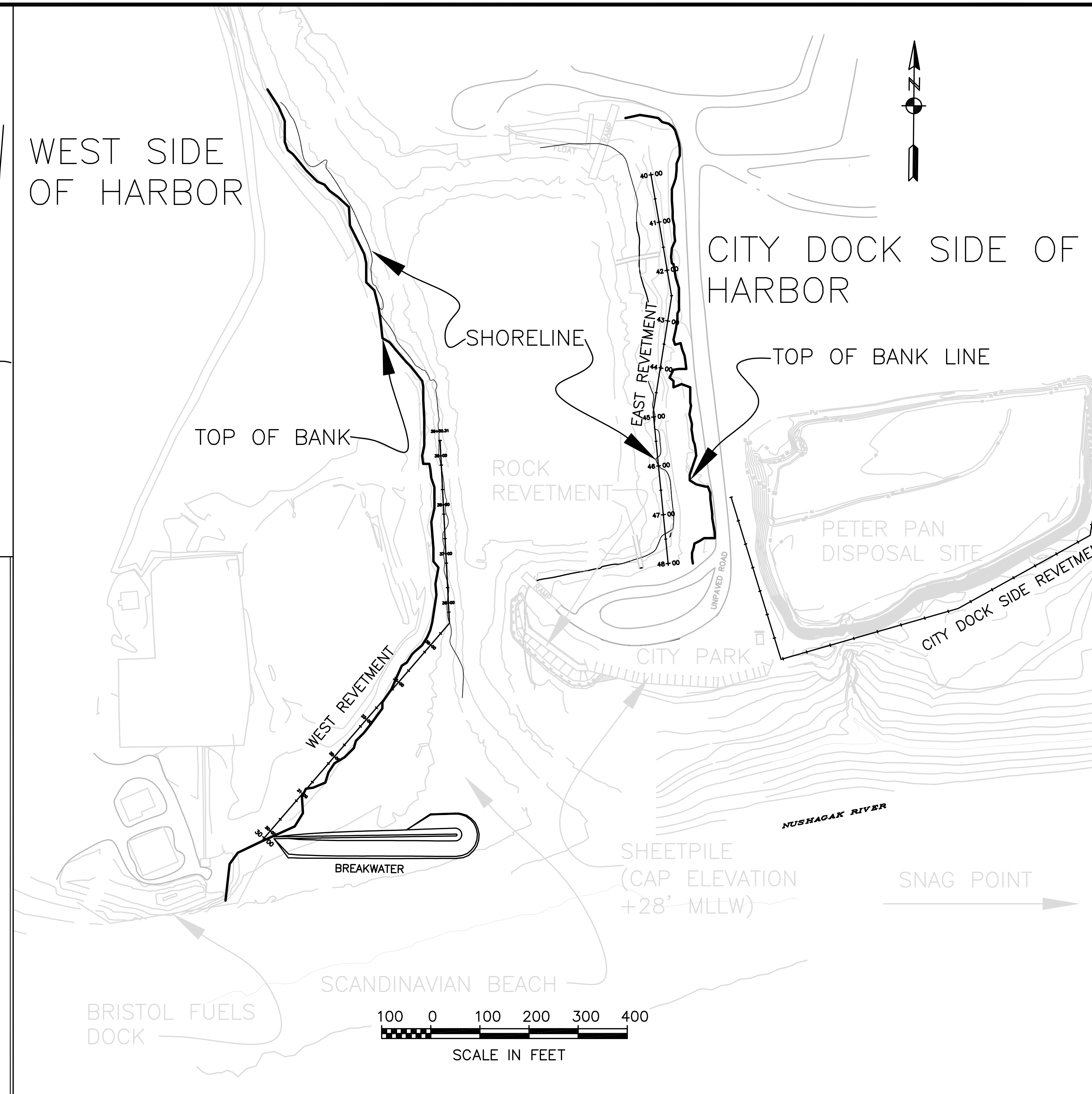
	Return Period				
Wind Direction	2	5	10	20	50
Southwest 190° - 220°	33.7	39.7	42.9	45.3	47.9
East 60° - 90°	29.8	33.4	34.8	36	36.9
Southeast 150° - 180°	32.8	39.1	42.9	46.4	50.5



VICINITY MAP
N.T.S.



LOCATION



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LOCATION AND VICINITY MAP
DILLINGHAM BANK STABILIZATION

A-0

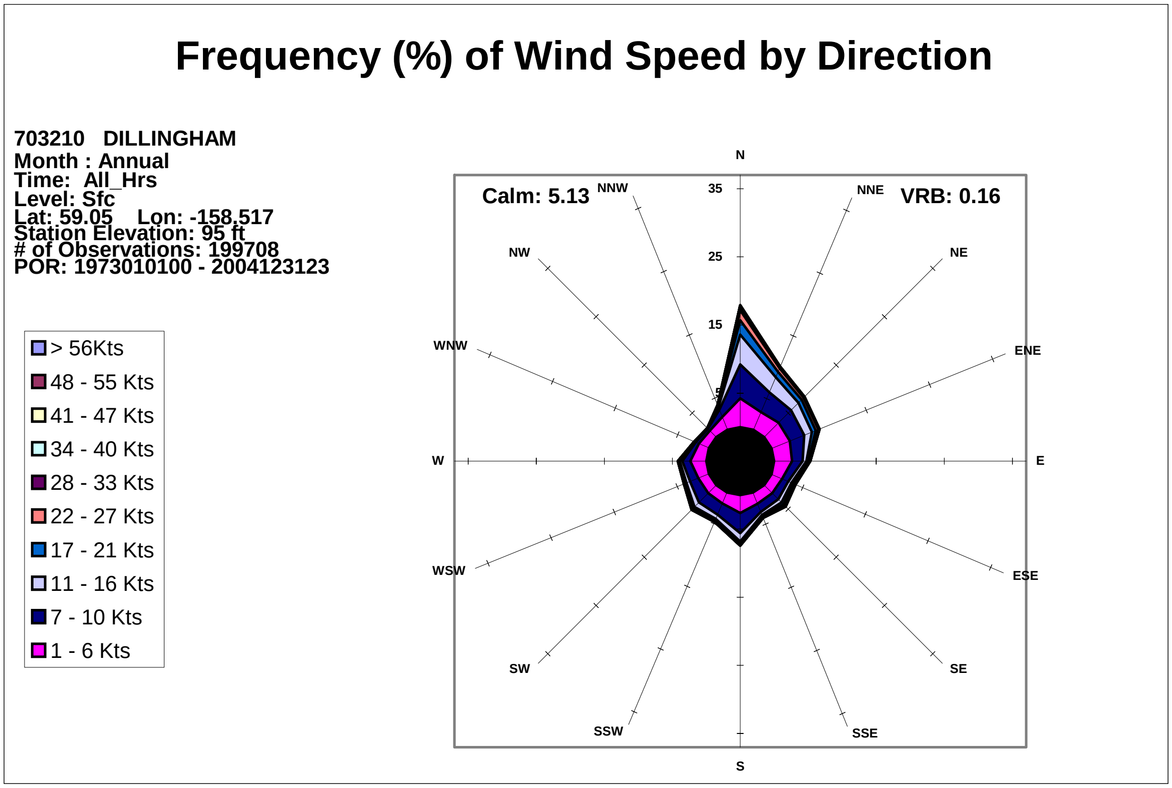


Figure A-1. Frequency of wind speed by direction.

2.3 Tides

Tide levels at Dillingham, referenced to mean lower low water (MLLW), are shown in table A-3. Extreme high tide levels result from a combination of astronomic tides and rises in local water levels due to atmospheric pressure. Tides in the project vicinity are semidiurnal in nature, with monthly spring and neap cycles. The average range of daily tide levels is approximately 16 feet. Spring tide ranges exceeding 24 feet occur several times per year (USACE 1997), with an extreme range of 27 feet (USACE 2002).

Table A-3. Tide Elevations at Dillingham

Tide Level Type	Levels Referred to MLLW (ft)
Extreme High Water	23
Mean Higher High Water (MHHW)	19.8
Mean High Water (MHW)	18
Mean Tide Level	10
Mean Low Water (MLW)	2.1
Mean Lower Low Water (MLLW)	0
Lowest Tide (Estimated)	-4.6

The above statistics were complied by NOAA for the Snag Point tidal station. Snag Point is a subordinate NOAA station, with corrections applied to the Nushagak Bay Clarks Point reference station in order to compute water levels. Figure A-2 shows tidal levels from Snag Point during a typical 1-month period.

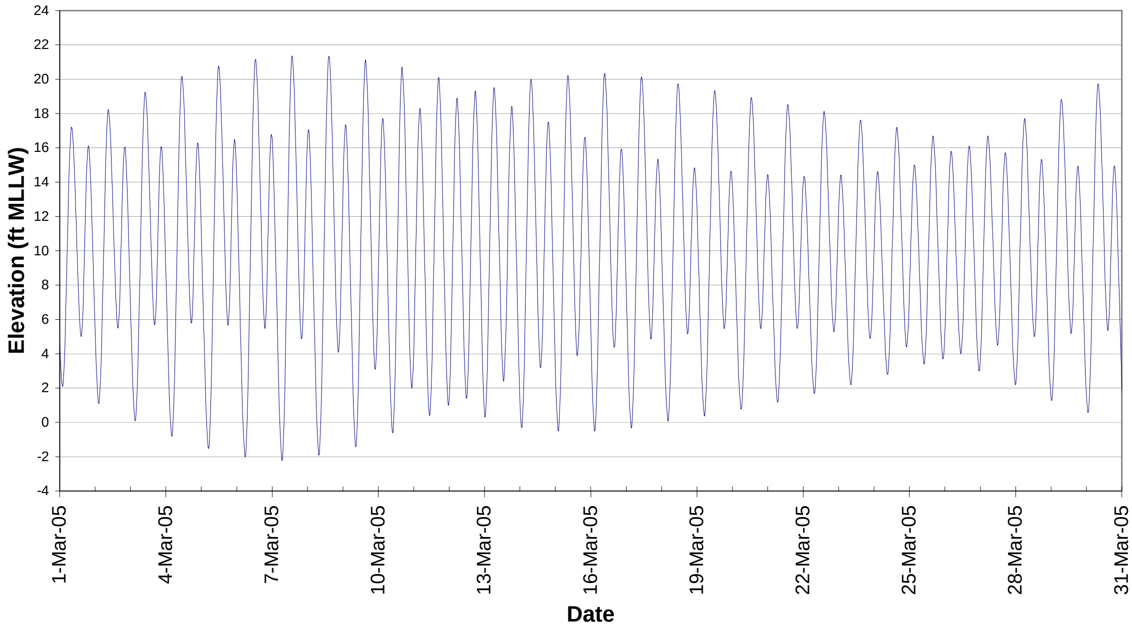
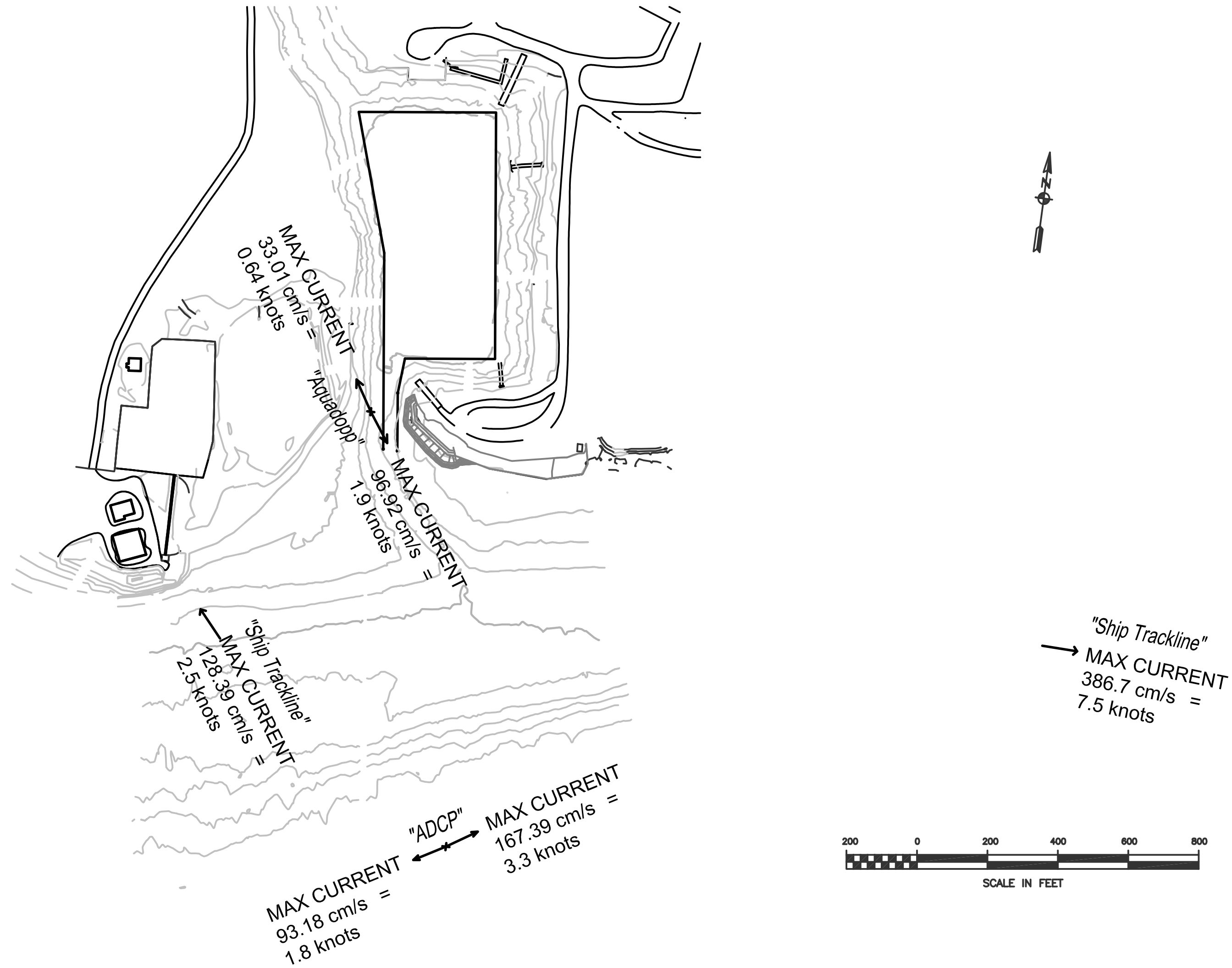


Figure A-2. Tide Levels at Snag Point, March 2005 (March, 2005)

2.4 Currents

Nushagak Bay currents are affected by the marine influences of the Bering Sea and fresh water effects from Scandinavian Creek, Squaw Creek, the Nushagak River, and the Wood River. The predominant direction of the current is east to northeast. Current measurements were taken August 9-17, 2001, by Evans-Hamilton, Inc. using a Bottom Mounted Acoustic Doppler Current Profile Meter (ACDP), an Aquadop Acoustic Doppler Profile Meter (Aquadop), and an ACDP mounted on the side on a ship that made tracklines inside and outside the small boat harbor (ship tracklines). Figure A-3 shows the measured currents maximum depth-averaged magnitude and direction. The ADCP and Aquadop recorded currents in two predominant directions. The maximum current recorded from all ship trackline data was an easterly velocity of 7.5 knots measured offshore in the vicinity of the city dock at flood tide. Current velocities within the project area ranged from 0.64 knot to 2.5 knots.



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MEASURED CURRENT DATA

DILLINGHAM BANK STABILIZATION

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2.5 Storm Surge

Storm surges are increases in water elevation caused by a combination of relatively low atmospheric pressure and wind-driven transport of seawater over relatively shallow and large unobstructed waters. Friction at the air-sea interface is increased when the air is colder than the water, which causes more wind-driven transport. Storm induced surges can produce short-term increases in water level, which rises to an elevation considerably above tidal levels. Dillingham has low-pressure events that may cause an increase in the water levels at the shoreline. However, the many obstructions presented by bends and sandbars over the fetch are expected to prevent storm surges greater than 6 feet.

2.6 Rivers and Creeks in the Project Vicinity 2.6

Scandinavian Creek is incorporated as part of a navigation channel through the harbor. Flow from the creek keeps the entrance channel and the western side of the harbor mostly free from sedimentation with discharges of approximately 5.3 cubic feet per second (ft^3/sec) in the summer months. Squaw Creek is approximately 1.5 miles west of the harbor. The Nushagak and Wood rivers converge upriver from Dillingham to flow into Nushagak Bay. The United States Geological Survey (USGS) gage at Ekwok on the Nushagak River has measured maximum mean daily stream flows of approximately 100,000 ft^3/sec for a period of record from 1979 to 1992. A USGS gage on the Wood River near Aleknagik shows maximum daily stream flows of approximately 20,000 ft^3/sec for a period of record from 1957 to 1970.

Suspended sediment concentrations in the Nushagak River range from 60 to 1700mg/l. The sediment from Dillingham Harbor deposited onto the Peter Pan Dredged Material Disposal Site is generally very fine silt and has been found acceptable for upland or aquatic disposal because of low concentrations of contaminants.

2.7 Ice Conditions

Nushagak Bay generally begins to freeze up around the first of November. Break-up usually begins sometime in May. Ice ride-up on the shore is common and should be expected on any rubblemound structure. Controlling ice forces on the harbor side were listed at 13 kips/ft over the upper elevations (USACE, 1988).

3.0 GEOLOGY AND SOILS

3.1 Geology

The project site is located on unconsolidated alluvial deposits formed by the Nushagak River delta. These deposits form steep bluffs that are common around Dillingham. The toe of the bluffs is subject to direct wave attack and impact and is subject to continual erosion. It is typical to find a thin layer of gravel (old beach strata) seeping water with iron solutions and fine silts at the toe of area bluffs (City of Dillingham, Oct 1994, citing USACE, 1981).

3.2 Soil Conditions

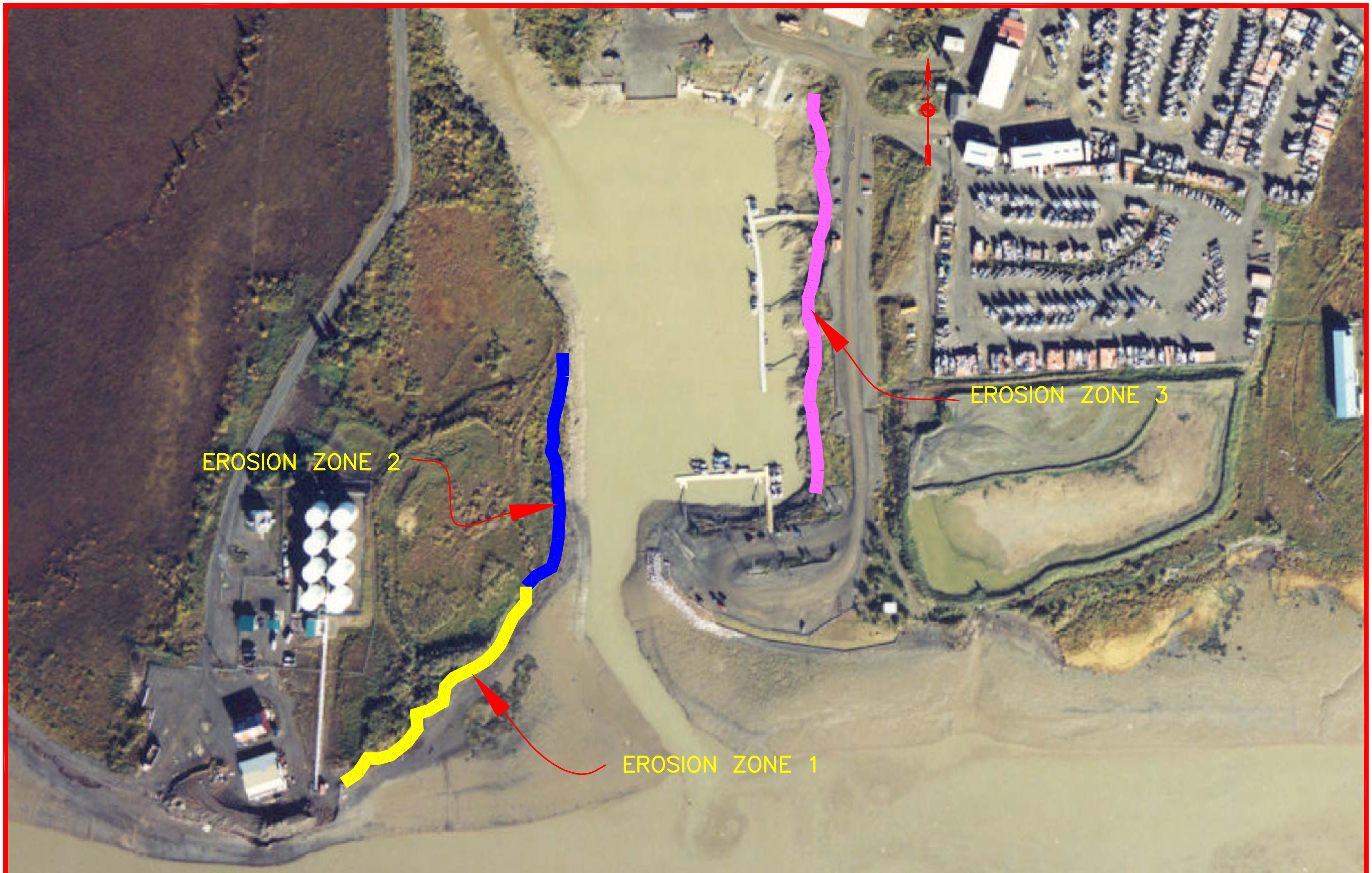
In a geotechnical investigation for the Dillingham Wastewater Lagoons, CH2MHill excavated seven test pits in October 1985 in the area where the Snag Point sheet-pile bulkhead is now. These pits showed the area to be silty-gravel to poorly graded sand with silt and gravel to 2 feet below the surface and lean clay to 8 feet below ground surface. Soil conditions in the project area are considered to be similar to the test pit. Clay balls are visible on the ground in the project vicinity and surrounding area. Soil in the project area has proven sufficient for foundation material for rock revetment. This has been visible with the existing rock revetment on the east side of the harbor entrance. Soil with D_{50} = 0.004 mm was assumed for filtering requirements for the revetment.

3.3 Erosion Rates – West Side of Harbor

Erosion on the west side of the harbor entrance (Scandinavian Beach) is predominantly caused by wave action in conjunction with high tides. Locals have reported up to 10 feet of unprotected bank being lost at one time during storms that occur at the same time as high tides in the area of Bristol Alliance's fuel dock. Erosion measurements of the Dillingham Harbor area are not available. In the absence of these actual measurements of shoreline change, aerial photographs were used to estimate the average erosion rate. To determine erosion rates for the Scandinavian Beach area (Zone 1) over time, aerial photos for the years 1972, 1980, 1988, 1992, and 2001 were compared, see figure A-5. An arbitrary baseline was first established roughly parallel to the bluff. Offset distances were then measured from the baseline to the top of bank. These offset distances were measured at 50-foot intervals along the entire top of bank. The offsets were then compared to determine the average erosion rate for the periods 1972-1980, 1980-1988, 1988-1992, and 1992-2001. The average annual erosion rates ranged from 17.14 to 3.11 feet per year. Both the west (Zone 2) and the east (Zone 3) sides directly adjacent to the harbor were also analyzed using this method to determine the amount of erosion taking place, see figure A-4. Conclusions drawn from these efforts were that erosion in these areas was minimal and therefore not included in this document.

Buildings and a dock owned by the Bristol Bay Packing Company Cannery once stood where the Bristol Fuel's dock is currently located. The cannery was dismantled in the late 1960s and the Ball Brothers then used the site in the 1980s during which they constructed a wooden bulkhead. The bulkhead prevented further erosion of that area until a storm destroyed it sometime around 1997 or 1998. Bristol Fuels then constructed a sheet-pile bulkhead. In the summer of 2004, Bristol Fuels completed construction of a sheet-pile dock in same location as the old sheet-pile and timber bulkhead

The upland area adjacent to the west side of the harbor has been used intermittently for dredged material disposal since the early 1970s. It is likely that some erosion has been due to dewatering of the dredged material disposal area. However, the material that has been placed for the berms has helped to further reduce erosion in other areas. The last year the area was used for dredged material disposal was in 1995. It is highly unlikely the area will be used for dredged material disposal activities again. The erosion in the area is likely to continue in the same manner as was seen from 1992 to 2001. Table A-4 shows a summary of the erosion rates over the years and the average erosion rate of 11 feet per year in the vicinity of Scandinavian Beach.



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EROSION ZONE DELINATION

DILLINGHAM BANK STABILIZATION

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EROSION RATES – 2001 AERIAL PHOTO
DILLINGHAM EMERGENCY EROSION PROTECTION

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Table A-4. Erosion Data

Comparison Years	Total Erosion (ft)	Erosion Per Year (ft/year)
1972 to 1980	137.14	17.14
1980 to 1988	24.86	3.11
1988 to 1992	62.15	15.54
1992 to 2001	62.01	7.75
Average Erosion Per Year		11

Source: Aerial Photography provided by Aeromap U.S.

3.4 Erosion – City Dock Side

The Dillingham Beach Erosion Study (USACE, 1972) documented an average erosion rate of four linear feet per year from 1948 through 1967 in the project site vicinity, with an intensified rate during the period from 1963 to 1967. Up to 8 feet of erosion was documented during a single storm on August 7, 1980 (USACE, 1981 cited in City of Dillingham, Oct 1994). USACE (2001) estimated 100 feet of bank loss between 1991 and 2001.

Historical erosion in the project area has been influenced by conditions that may not exist in the future without project condition. The two water outfalls from the dredged material disposal area, for example, have developed channels through the nearshore vegetative cover, which have allowed more energy to arrive in the nearshore area. The vegetative cover has then been stripped in those areas by subsequent wave activity. The initial rapid erosion in these zones reflects the removal of that cover and the exposure of a more erosion resistant clay shoreline. The removal of vegetation has left the disposal area containment berm and other areas vulnerable to erosion, and some failures at the contact zone of the berm to original ground are already evident.

The actual recession of the underlying clay layer is unknown but intuitively should be less than the measured recession of the vegetation line. The actual landward shift of contours in the clay zone is estimated to be only about 20 percent of that that has occurred in the vegetated area, or about 2 feet per year, as opposed to 11 feet per year that has been measured in the vegetated area. This lower rate of recession coincides with the lower rates witnessed in the study area where vegetative cover has not been impacted by dredging operations and where the underlying clay controls the recession rate. Unless some protection measures are implemented at the east end of the existing sheet-pile bulkhead, continued erosion in the area adjacent to and behind the wall could result in loss of structural integrity of the bulkhead in the future. Figure A-6 shows a typical schematic cross-section with the vegetation zone and clay underlayer.

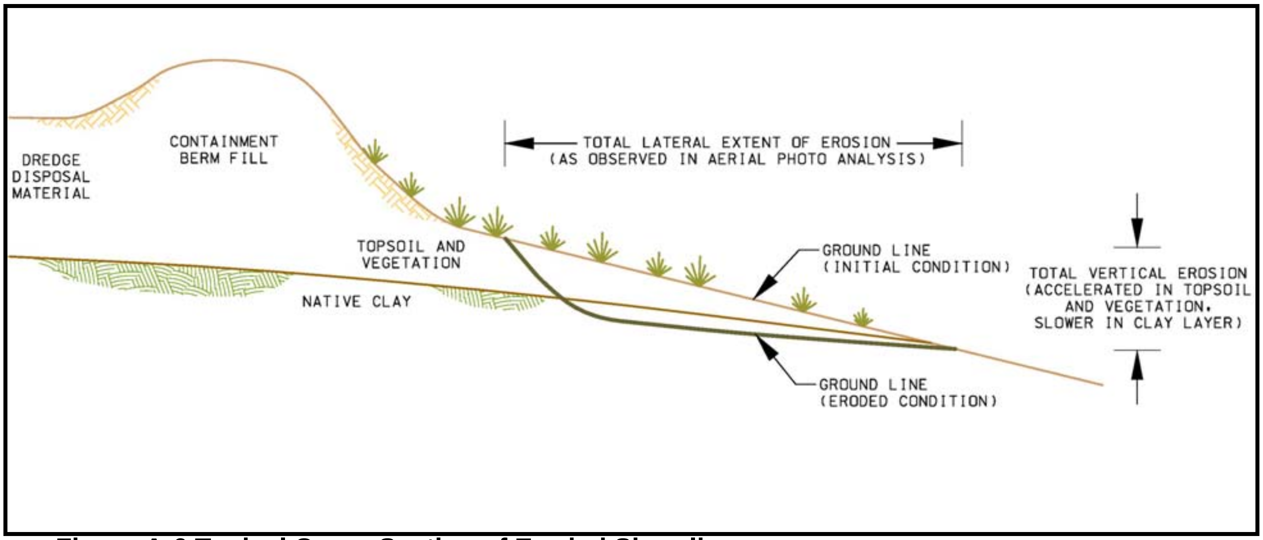


Figure A-6.Typical Cross-Section of Eroded Shoreline

Tetra Tech, as part of the current design, conducted a historical erosion analysis of the proposed project area. Five aerial photographs of the study area spanning the years from 1972 to 2001 were obtained and digitally adjusted to the NAD 83, Zone 6, State Plane coordinate system. The visible vegetation line in each aerial photograph was plotted and the area lost to erosion between each set of photographs was measured using GIS software.

Erosion rates within the project area were highly variable along the shoreline, with the general trend being more pronounced erosion toward the west and less pronounced erosion toward the east of the project area. As a result, the erosion analysis was separated into two erosion zones based upon common historical erosion rates and proximity to affected infrastructure. Zone 1 encompasses the western portion of the study area where historical erosion has been more pronounced (7.6 feet per year over the period 1972-2001). Zone 2 encompasses the eastern portion of the study area, which has experienced a slower erosion rate (2.4 feet per year over the same period).

The area lost to erosion between each set of historic aerial photographs was tabulated for both erosion zones. The measured areas were then divided by the time elapsed between photographs and the length along the shoreline to determine an average lateral erosion distance (perpendicular to the shoreline). Table A-5 summarizes the historical erosion rates in the project area by erosion zone.

Table A-5. Historical Erosion Rates

Zone 1: West (shoreline length: 1,000 feet)								
Year			**Area lost to erosion**			**Linear distance lost to erosion**		
(from)	(to)	(# years)	incremental (ac)	cumulative (ac)	average annual (ac)	incremental (ft)	cumulative (ft)	average annual (ft)
1972	1980	8	2.0	2.0	0.25	87	87	10.8
1980	1988	8	1.2	3.2	0.15	53	139	6.6
1988	1992	4	0.5	3.7	0.11	20	159	4.9
1992	2001	9	1.3	5.0	0.15	60	219	6.7
Total Historical Erosion Loss:								
1972	2001	29	5.0		0.17	219		7.6
Zone 2: East (Shoreline length: 750 feet)								
Year			**Area lost to erosion**			**Linear distance lost to erosion**		
(from)	(to)	(# years)	incremental (ac)	cumulative (ac)	average annual (ac)	incremental (ft)	cumulative (ft)	average annual (ft)
1972	1980	8	0.5	0.5	0.06	29	29	3.6
1980	1988	8	0.3	0.8	0.03	16	45	2.0
1988	1992	4	0.2	1.0	0.05	12	56	2.9
1992	2001	9	0.2	1.2	0.03	14	70	1.5
Total Historical Erosion Loss:								
1972	2001	29	1.2		0.04	70.3	70.3	2.4

The average historical rates in each zone were extrapolated to provide the initial basis of the expected erosion zone in the study area over the 50-year period of analysis. Adjustments to the bounds of the expected erosion area were based upon engineering judgment to account for expected without project conditions in the study area, such as consideration of the stoppage of use of the south dredged material containment area outfall, the relationship between recession rate of beach slopes, scour depths, and soil characteristics in the project area. Additionally, the erosion bounds were smoothed to depict a more general trend than the maximum rates at any particular point. Figure A-7 shows the historical and projected erosion areas in the vicinity of the disposal area relative to the 2001 aerial photograph.

As shown in figure A-7, erosion in Zone 2 was predominantly in the western and central portions of the zone. Only minimal erosion was witnessed at the eastern end of Zone 2, where Peter Pan Seafoods' dock facilities are located. The reduced level of erosion within Zone 2 is likely a result of the wave energy dissipation provided by the extensive set of pilings under the Peter Pan Seafoods' docks and the attenuation provided by the natural wetland to the west of those docks.

As a result of the analysis of historic erosion rates in the area, formulation and evaluation of protective features was focused within the two erosion zones (between the eastern terminus of the existing sheet-pile bulkhead at the harbor and the westernmost Peter Pan dock.

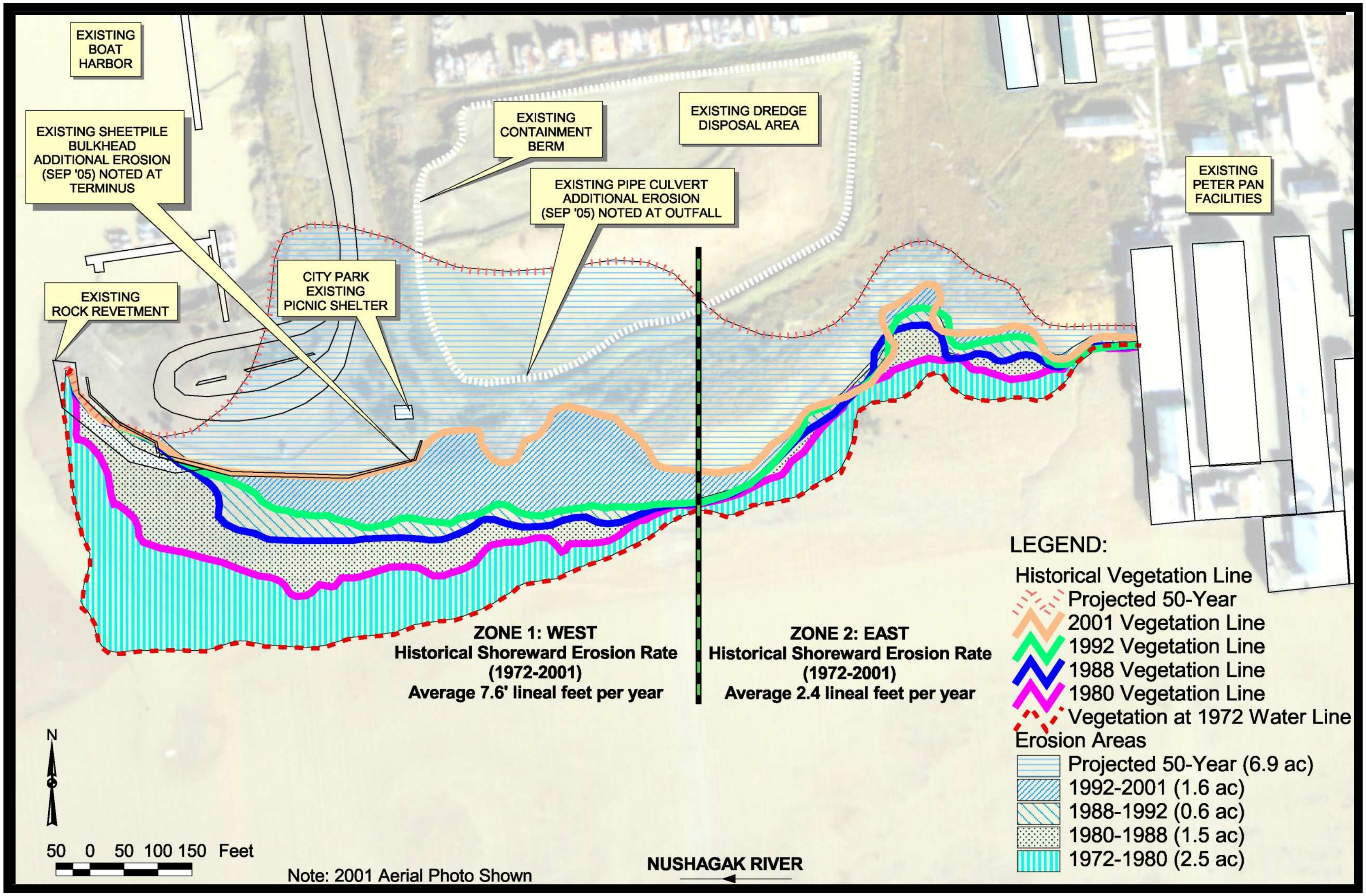


Figure A-7 Historical and Projected Erosion Rates in Project Vicinity

4.0 WAVE CLIMATE

The wave climate at Dillingham is generally moderate and is subject to short-period wind generated waves from the southwest to northeast. Waves coming from the southwest are predominant and are subject to diffraction, refraction, and shoaling as they pass through bends in the river.

4.1 Fetches

Fetches were calculated according to methods specified in the SPM, with 9 radials at 3° increments as shown in figure A-8. The longest fetch is from the southwest and is 25.7 miles. Table A-6 provides a summary of the fetch distances.

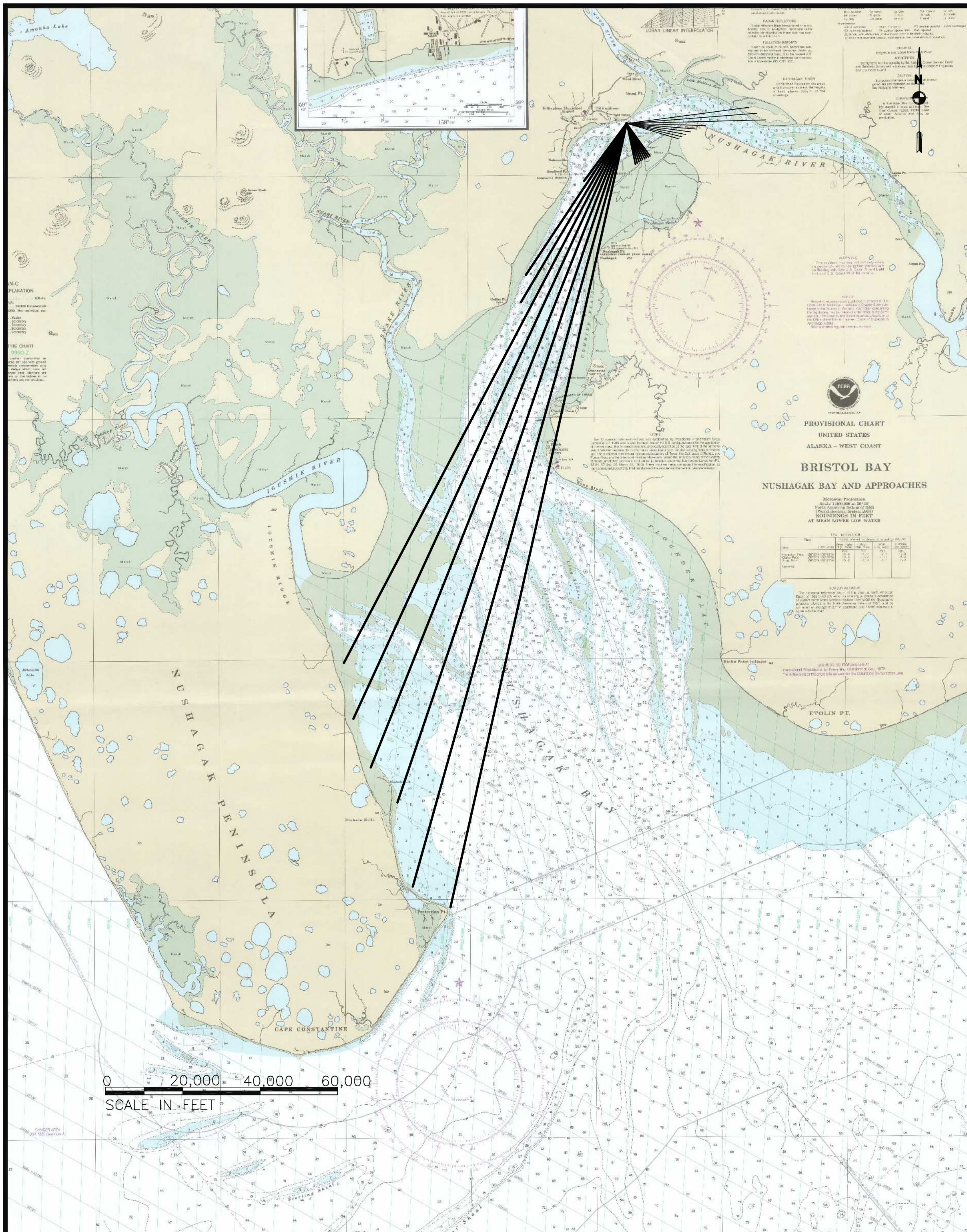
Table A-6. Fetches

Direction	Fetch Distance (miles)
Southwest	25.7
East	4.2
Southeast	1.9

4.2 Wave Prediction

Predicted wave heights for the project area were calculated using the 50-year design wind speeds in table A-2. Winds were then corrected for land versus water effects and air versus sea effects. Winds used in the SPM equations were converted to a wind stress. Methods described in the Engineering Manual 1414 (EM 1414): Water Levels and Wave Heights for Coastal Engineering Design, the SPM shallow water curves, the Coastal Engineering Manual (CEM), the Automated Coastal Engineering System (ACES) program, and the STWAVE numerical model were used to predict wave heights. The design wave was calculated as an average of the results of the different methods that applied to each situation. The EM 1414 equations and ACES program predict wave heights based on fetch distances and wind speeds. The fetch distance and wind speed are used to determine if the wave condition is limited by the fetch length or by the duration of the wind speed. STWAVE is a spectral wave energy propagation model that includes refraction, diffraction, and shoaling, but does not include reflection. Inputting water depths and land locations at grid spacings of 1,000 feet into the STWAVE model defined shoreline and bathymetric conditions. Depths were obtained from NOAA charts showing the bathymetry of the area.

The 50-year design storm wave was determined to be a 6.22-foot breaking wave from the southwest with a period of 5.0 seconds. The design wave for the southwest is the average of the results from the SPM shallow water curves, ACES, and STWAVE (8.5 feet, 5.23 feet, and 4.92 feet). The design waves for the southeast and east are the results of the EM 1414 equations, ACES, and STWAVE. For the east wave, the results from the three methods were 2.87 feet, 2.97 feet, and 2.3 feet, and for the southeast wave the results were 3.22 feet, 2.93 feet, and 4.26 feet, respectively. These results are shown in table A-7. The wave heights calculated represent the significant wave height, H_s, which is the average of the highest 1/3 of all waves.



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FETCHES

DILLINGHAM BANK STABILIZATION

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Table A-7. Wave Analysis Results.

	Design Wave	
Direction	H_s (ft)	T(s)
Southwest	6.22	5
East	2.71	3.4
Southeast	3.47	3.5

4.3 Allowable Wave Height in the Entrance Channel and Mooring Area

The rock and sheet-pile revetments in the proposed alternatives were positioned so that the waves inside the entrance channel are a maximum of 6.2 feet and reduce to 1.7 feet before entering the harbor and reaching the float nearest to the entrance channel during the 50-year storm event. Storm waves in the entrance channel are quickly reduced to approximately 1.0 feet before reaching the mooring area. Diffraction and reflection analyses were used to calculate wave heights expected in the alternatives for this study.

5.0 DESIGN ALTERNATIVES – WEST SIDE

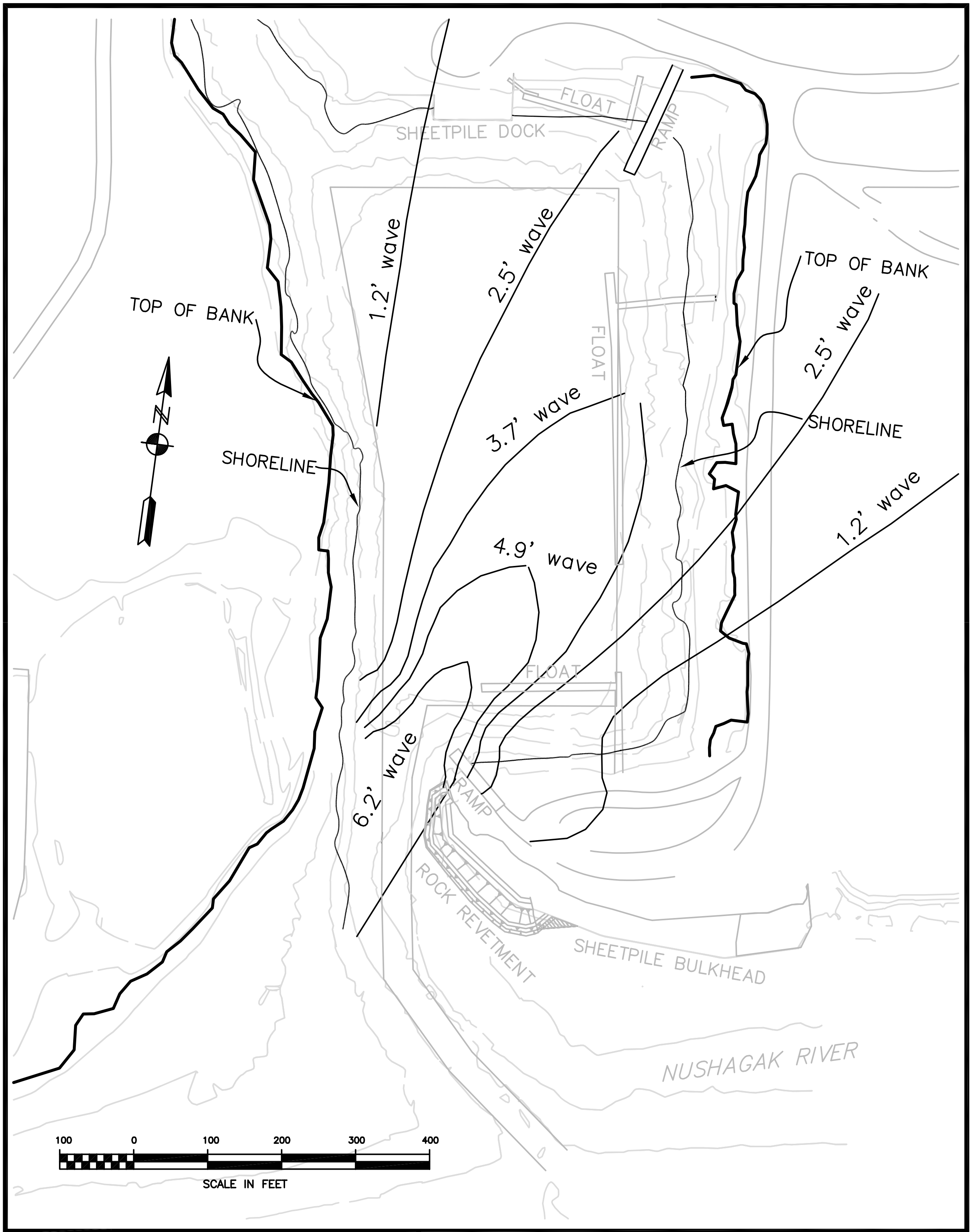
Seven alternatives, including the no-action alternative, were considered for stabilizing the bank and reducing wave heights in the harbor at Dillingham. These alternative plans were evaluated using established design guidance given in the appropriate Corps of Engineers EMs, the SPM, and the CEM. Three of these alternatives (W2,W3, and W4) were found to provide identical benefits; therefore, only the least cost of these, Alternative W2, was chosen for further analysis.

5.1 No-Action

Erosion of the riverbank would continue at approximately the same rate. Vessels and the existing facilities would continue to sustain damage during storms. A diffraction analysis of the current conditions at Dillingham shows that waves of up to 5 feet can be expected at the outer floats as shown in figure A-9. Locals contacted about the wave climate said they have seen 3-foot or higher waves in the harbor.

5.2 Alternative W1 - Rock Revetments

This plan consists of a rock revetment on both the west and east sides of the harbor. Both revetments would consist of a 3-layer system of core, secondary, and armor stone. The west revetment would start adjacent to the Bristol Fuel's dock and continue along an alignment consistent with a natural shoreline for approximately 1,100 feet as shown in figure A-10. No adverse impact on Bristol Fuels dock or the existing harbor is expected to occur from construction of this alternative. Erosion at the dock is not expected to increase, as the rock would tie into the existing sheet-pile dock. Velocities at the dock are not expected to change appreciably as the water level is relatively shallow at all but



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DIFFRACTION DIAGRAM – EXISTING CONDITIONS
DILLINGHAM BANK STABILIZATION

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the highest tides. The east revetment would extend the length of the east side of the harbor along an alignment consistent with the natural shoreline for approximately 800 feet. The top of each revetment would be vegetated for bird habitat by incorporating live willow stakes, sprigging of native grasses, and seeding.

Wave Attenuation

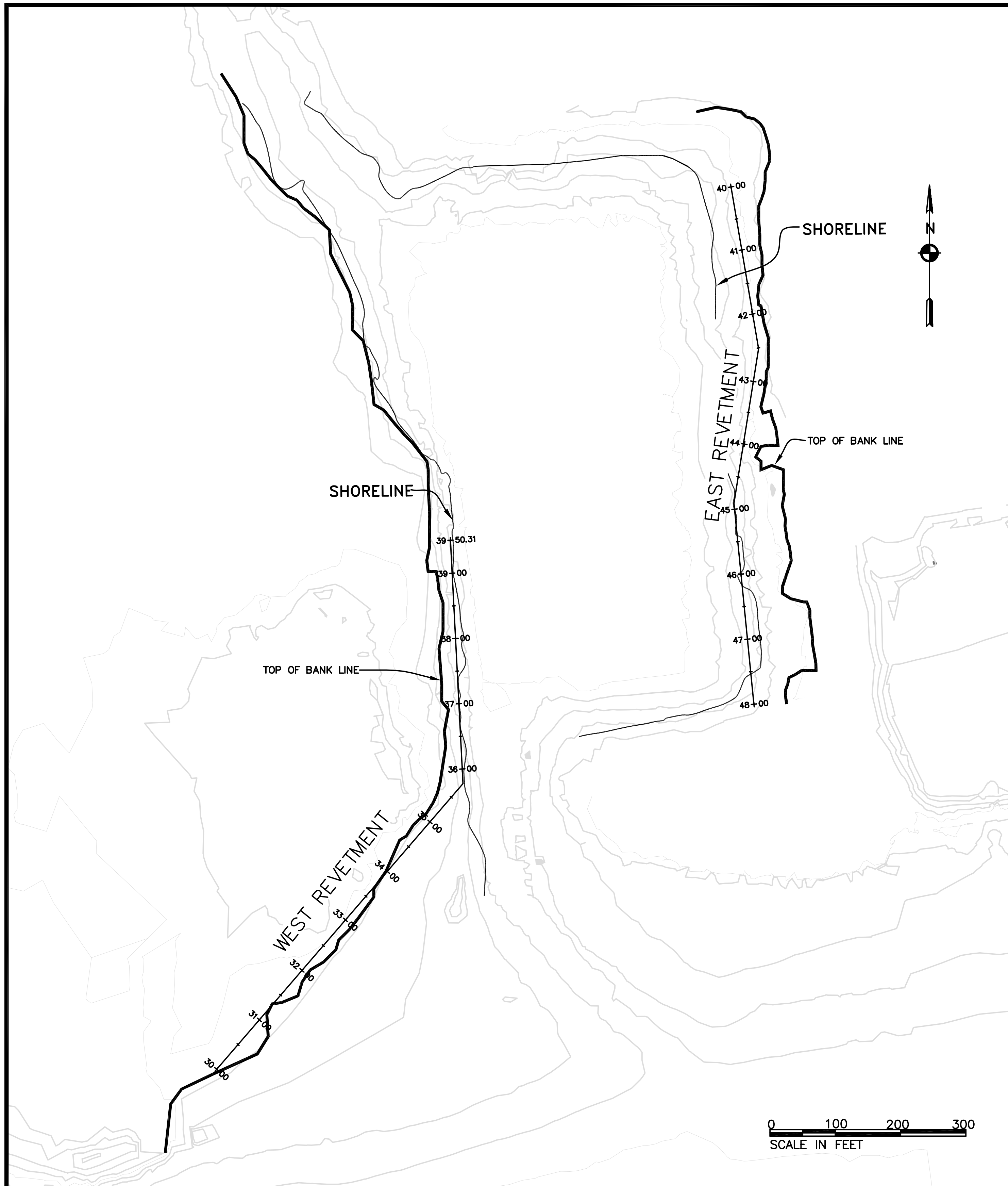
A diffraction analysis was done using methods described in the SPM. Results of the analysis show that the proposed alignment would not reduce the waves inside the harbor. Waves of up to 5 feet can be expected at the outer floats as is shown in figure A-11. Although this alternative will not reduce wave heights in the harbor, it would protect the shoreline from erosion caused by these waves.

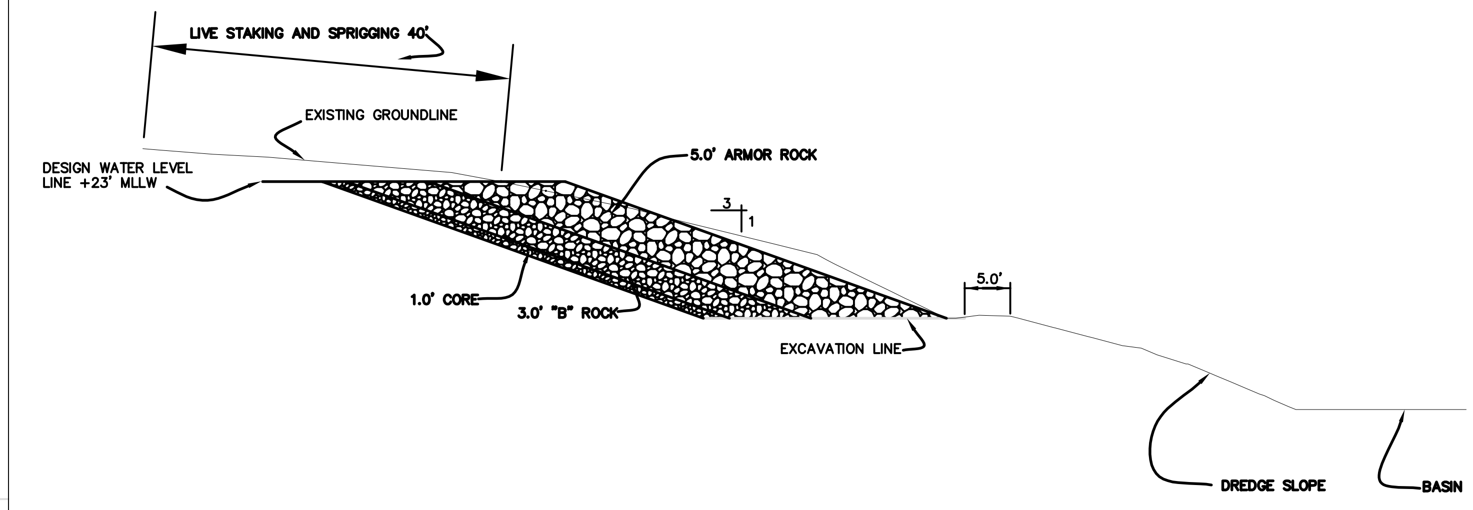
Rock Size

Methods described in the SPM using Hudson's equation were used to calculate stone sizes. The southwest wave height, 1v:3h side slopes, a specific gravity of 2.6, and a stability coefficient of 2.2 were used in the computation for rock size on the west revetment. For the west revetment, approximately 19,300 yd^3 of armor rock would be used that range in size from 1,914 to 1,148 pounds. Secondary rock size would be from 1,148 to 115 pounds and would require 11,700 yd^3. Four thousand yd^3 of core rock would be placed behind the secondary layer and would range in size from 115 to 11 pounds. Approximately 3,600 yd^3 of porous fill would be placed behind the revetment. The east revetment would have a different typical section than that of the west revetment due to the decreased wave height inside the harbor. The southeast wave height, 1v: 3h side slopes, a specific gravity of 2.6, and a stability coefficient of 2.2 were used in the computation for rock size for the east revetment. The east revetment would require 7,900 yd^3 of armor rock ranging in size from 330 to 200 pounds. Secondary rock size would be from 200 to 20 pounds and would require 5,300 yd^3. Twenty-six hundred yd^3 of core rock would be placed behind the secondary layer and would range in size from 20 pounds to 1 pound. Approximately 2,600 yd^3 of porous fill would be placed behind the revetment. A cross-section of both revetments is shown in figure A-10.

The rock revetments would have a top of +32 feet MLLW. The top elevation of the revetments was determined from 6 feet of wave run-up with a design water level of 26 feet. The design water level equates to the mean higher high water level plus 6 feet of storm surge.

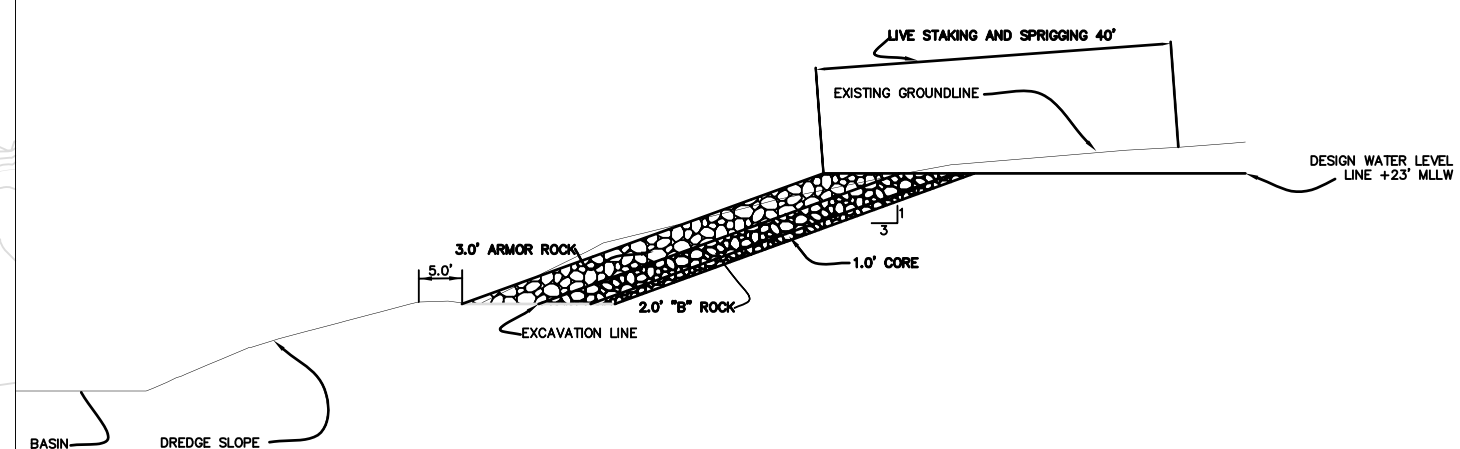
Maintenance of the rock revetments would be the responsibility of the City of Dillingham. The Corps of Engineers, Alaska District would conduct periodic site inspections to verify whether maintenance is warranted on the revetments. The rock revetments were designed to be stable for the 50-year wave condition. Therefore, it is not anticipated that there will be significant loss of stone on the structure during the life of the project. Typically, rock revetments have a 50-year design life, even though they may last much longer. Maintenance of a rock revetment may require replacement of 2 percent of the armor stone every 25 years.





WEST REVETMENT TYPICAL SECTION

N.T.S.



EAST REVETMENT TYPICAL SECTION

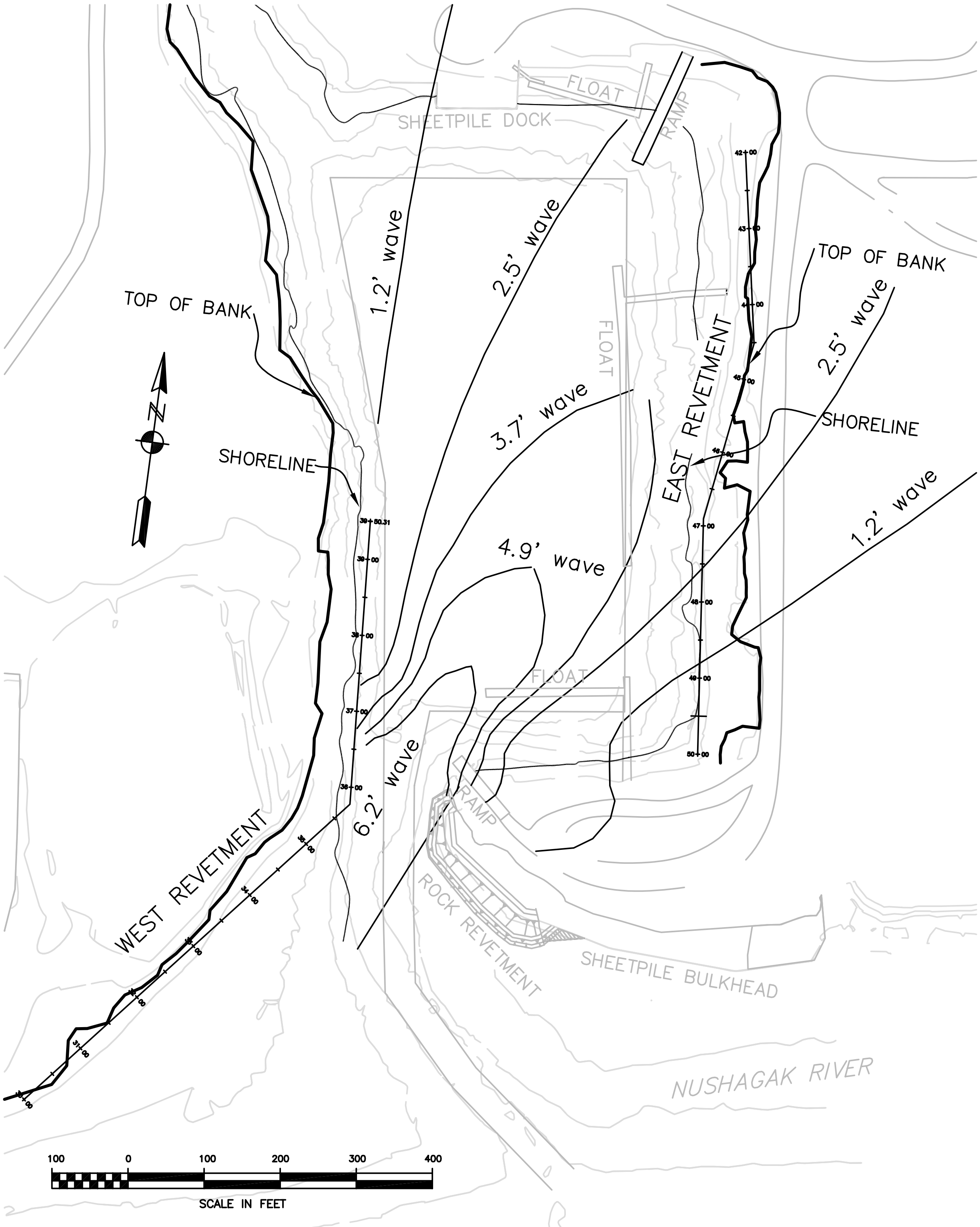
N.T.S.

ALASKA DISTRICT
CORPS OF ENGINEERS
CIVIL WORKS BRANCH

ALTERNATIVE W1 – ROCK REVETMENTS

DILLINGHAM BANK STABILIZATION

A–10



ALASKA DISTRICT CORPS OF ENGINEERS CIVIL WORKS BRANCH	DIFFRACTION DIAGRAM – ROCK REVETMENTS DILLINGHAM BANK STABILIZATION	A–11

5.3 Alternative W1A - Rock Revetments and Sheet-pile Bulkhead

This plan consists of approximately 725 feet of rock revetment and 375 feet of sheetpile on the west bank of the harbor and 800 feet of revetment on the east side of the harbor as shown in figure A-12. No adverse impact is expected downstream of Bristol Fuels dock for the same reasons as stated in the previous alternative.

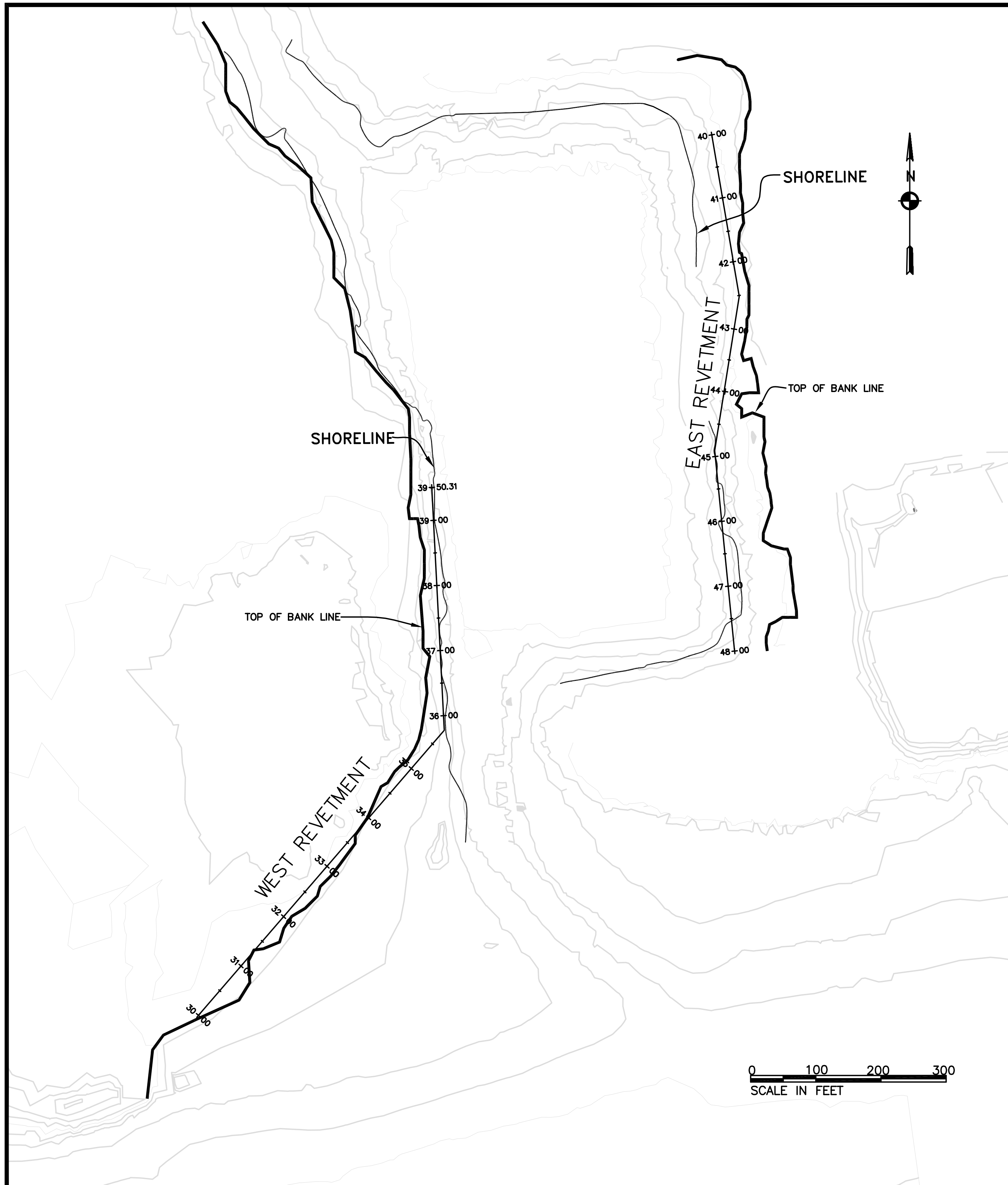
The top of both the west and east revetments and the sheet-pile would be vegetated for bird habitat with willow cuttings, sprigging of native grasses, and seeding.

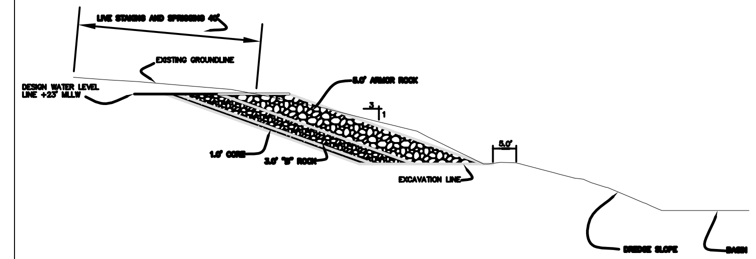
Wave Attenuation
A diffraction analysis was done using methods described in the SPM. Results of the analysis show that the proposed alignment would not reduce the waves inside the harbor. Waves of up to 5 feet can be expected at the outer floats as shown in figure A-13. This alternative would protect the shoreline from erosion caused by these waves.

Rock Size
Methods used for rock sizing are the same as for the previous alternative. For the west revetment, approximately 12,700 yd^3 of armor rock would be used that range in size from 1,914 to 1,148 pounds. Secondary rock size would be from 1,148 to 115 pounds and would require 7,700 yd^3. Twenty-seven hundred yd^3 of core rock would be placed behind the secondary layer and would range in size from 115 to 11 pounds; 2,700 yd^3 of porous fill would be placed behind the revetment. The east revetment would have a different typical section from that of the west revetment due to the reduced wave climate inside the harbor. The east revetment would require 7,900 yd^3 of armor rock ranging in size from 350 to 200 pounds. Secondary rock size would be from 200 to 20 pounds and would require 5,300 yd^3. Twenty-six hundred yd^3 of core rock would be placed behind the secondary layer and would range in size from 20 pounds to 1 pound; 2,600 yd^3 of porous fill would be placed behind the revetment. A cross-section of both revetments is shown in figure A-12.

Both the west and east revetments would have a top elevation of +32 feet MLLW. The top elevation of the revetments was determined from 6 feet of wave run-up with a design high water level of 26 feet. The design water level is mean higher high water plus 6 feet of storm surge.

Maintenance of the rock revetments would be the responsibility of the City of Dillingham. The Corps of Engineers, Alaska District would conduct periodic site inspections to verify whether maintenance is warranted on the revetments. The rock revetments were designed to be stable for the 50-year wave condition. It is not anticipated that there will be significant loss of stone on the structure during the life of the project. Typically, rock revetments have a 50-year design life, even though they may last much longer. Maintenance of a rock revetment may require replacement of 2 percent of the armor stone every 25 years.





WEST REVETMENT TYPICAL SECTION

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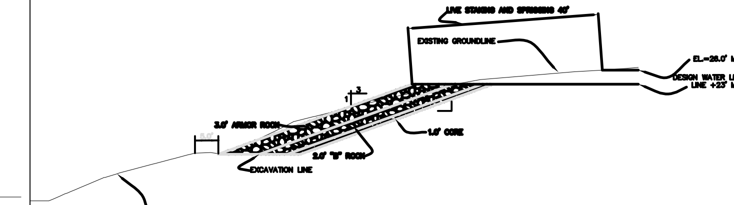

EAST REVETMENT TYPICAL SECTION

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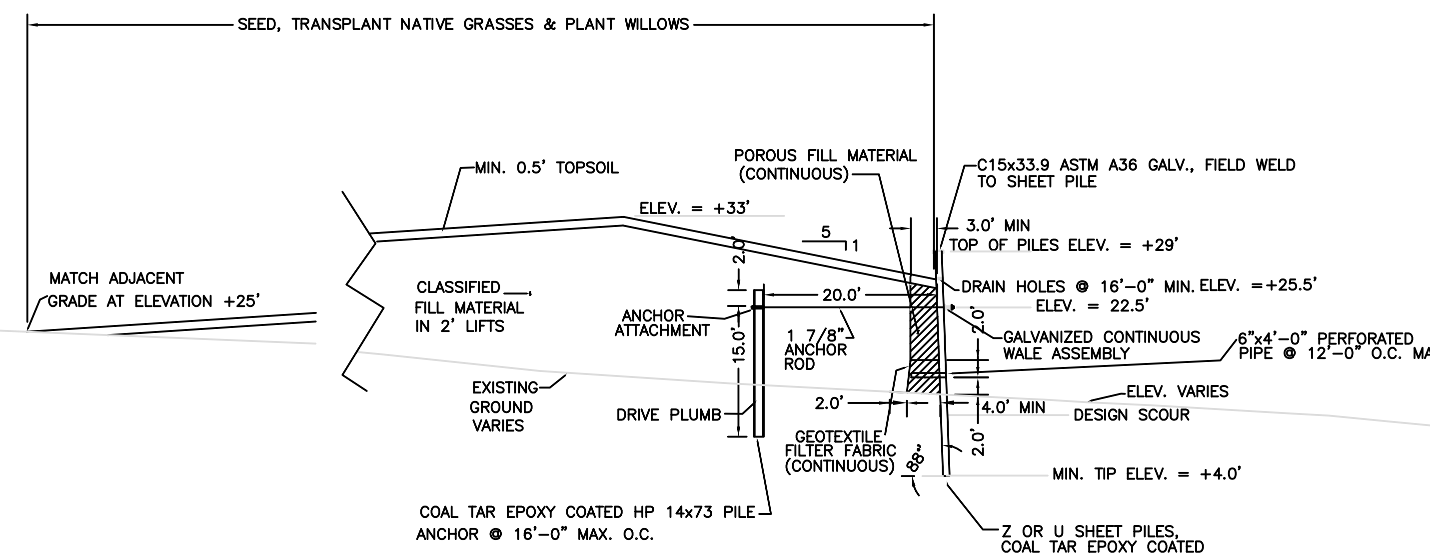


TYPICAL SHEETPILE SECTION

N.T.S.

ALASKA DISTRICT
CORPS OF ENGINEERS
CIVIL WORKS BRANCH

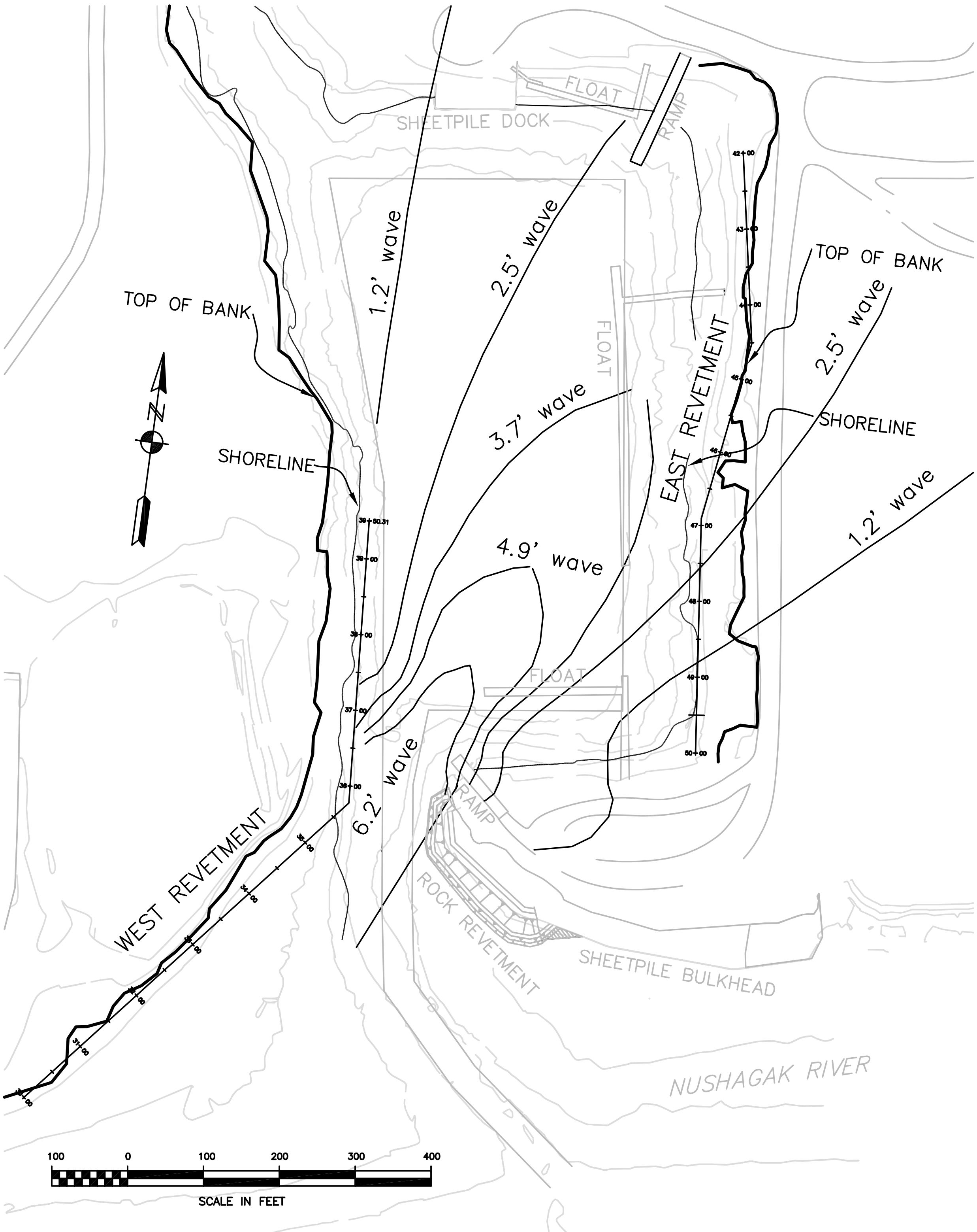
ALTERNATIVE W1A – ROCK REVETMENTS AND SHEETPILE BULKHEAD

DILLINGHAM BANK STABILIZATION

A–12

Steel Sheet-pile Bulkhead
The steel sheet-pile bulkhead would consist of coal tar epoxy coated Z or U piles approximately 45 feet in length with anchor rods extending back to anchor piles at 20-foot spacings. A galvanized wale assembly would be continuous along the face. Six-inch weepholes would be placed on a 12-foot spacing to drain water from behind the wall and minimize overburden pressures. Numerous zinc anodes would be installed on the sheet-pile sections and replaced periodically for cathodic protection. Approximately 1,900 yd^3 of fill (1,500 yd^3 of porous fill and 400 yd^3 of classified fill) would have to be placed behind the sheet-pile to prevent the drains from clogging with fine material. Classified fill would be specified as having less than 10 percent passing the #4 sieve. The top elevation of the sheet-pile would be +32 feet MLLW and the sheets would extend to a minimum elevation of +4 feet MLLW for a total exposed length of 28 feet. A cross-section of the bulkhead is shown on figure A-12.

The sheet-pile has a 30-year design life. It is assumed that the sheet-pile would need to be replaced at year 30 and that periodic maintenance would have to be performed on the structure at intervals of not more than 15 years. This replacement is expected to be accomplished by driving new sheet-pile in front of the old and possible replacement of the waler and anchor rods. The cathodic protection of the structure would need to be inspected annually.



ALASKA DISTRICT
CORPS OF ENGINEERS
CIVIL WORKS BRANCH

DIFFRACTION DIAGRAM – ROCK REVETMENTS AND SHEETPILE BULKHEAD
DILLINGHAM BANK STABILIZATION

A–13

5.4 Alternative W2 - Breakwater and West Revetment

This plan consists of a rubblemound breakwater and a rock revetment on the west side of the harbor. The revetment and the breakwater would consist of a 3-layer system of core, secondary, and armor stone. The revetment would start at the east end of the Bristol Fuel’s dock and continue along an alignment consistent with a natural shoreline for approximately 1,100 feet as shown in figure A-14. No adverse impact on the Bristol Fuel’s dock or the existing harbor is expected to occur from construction of this alternative for the same reasons as stated in the first alternative. The breakwater would be approximately 371 feet long and extend east into the Nushagak River from the west side of the harbor. Erosion inside of the harbor would be reduced due to the decreased wave climate. The top of the west revetment would be vegetated for bird habitat with willow cuttings, sprigging of native grasses, and seeding.

Wave Attenuation

A diffraction analysis was done using methods in the SPM. The rock revetment and breakwater are positioned so that the waves inside the entrance channel reduce to 1.7 feet before entering the harbor and reaching the float nearest to the entrance channel during the 50-year storm event. Storm waves from the southwest are quickly reduced before reaching the mooring area as shown in the diffraction diagram in figure A-15. The proposed alignment would reduce waves to approximately 1.7 feet at the outermost float in the mooring area. Wave heights would become progressively smaller farther into the harbor. Diffraction diagrams for the southeast and east wave are shown in figures A-16 and A-17, respectively.

Rock Size

Methods described in the SPM using Hudson’s equation were used to calculate the stone size for the revetment. The southeast wave height, 1v:3h side slopes, a specific gravity of 2.6, and a stability factor of 2.2 were used in the computation of stone size. The west revetment would require approximately 11,000 yd^3 of armor rock, ranging in size from 350 to 200 pounds. Secondary rock size would be between 200 and 20 pounds and would require 7,200 yd^3; 3,800 yd^3 of core rock would be placed behind the secondary layer and would range in size from 20 pounds to 1 pound; 3,600 yd^3 of porous fill would be placed behind the revetment. A cross-section of the revetment is shown in figure A-14

Methods described in the SPM using Hudson’s equation were also used to calculate the stone size for the breakwater. The southwest wave height, 1v: 1.5h side slopes, a specific gravity of 2.6, and a stability factor of 2.2 were used in the computation of stone size. The breakwater would require approximately 5,500 yd^3 of armor between 3,826 and 2,295 pounds. Secondary rock size would be from 2,295 to 230 pounds and would require 3,250 cubic yards. Forty-three hundred yd^3 of core rock from 230 to 21 pounds would also be required. A cross-section of the breakwater is shown in figure A-14.

The west revetment would have a top elevation of +32 feet MLLW. The top elevation of the revetment was determined from 6 feet of wave run-up with a design high water level of 26 feet. The design water level equates to mean higher high water plus 6 feet of storm surge. The breakwater would have a crest elevation of +32 feet MLLW. The breakwater is designed to be an overtopping structure.

It is anticipated that navigation marker lights would be installed at the seaward end of the breakwater.

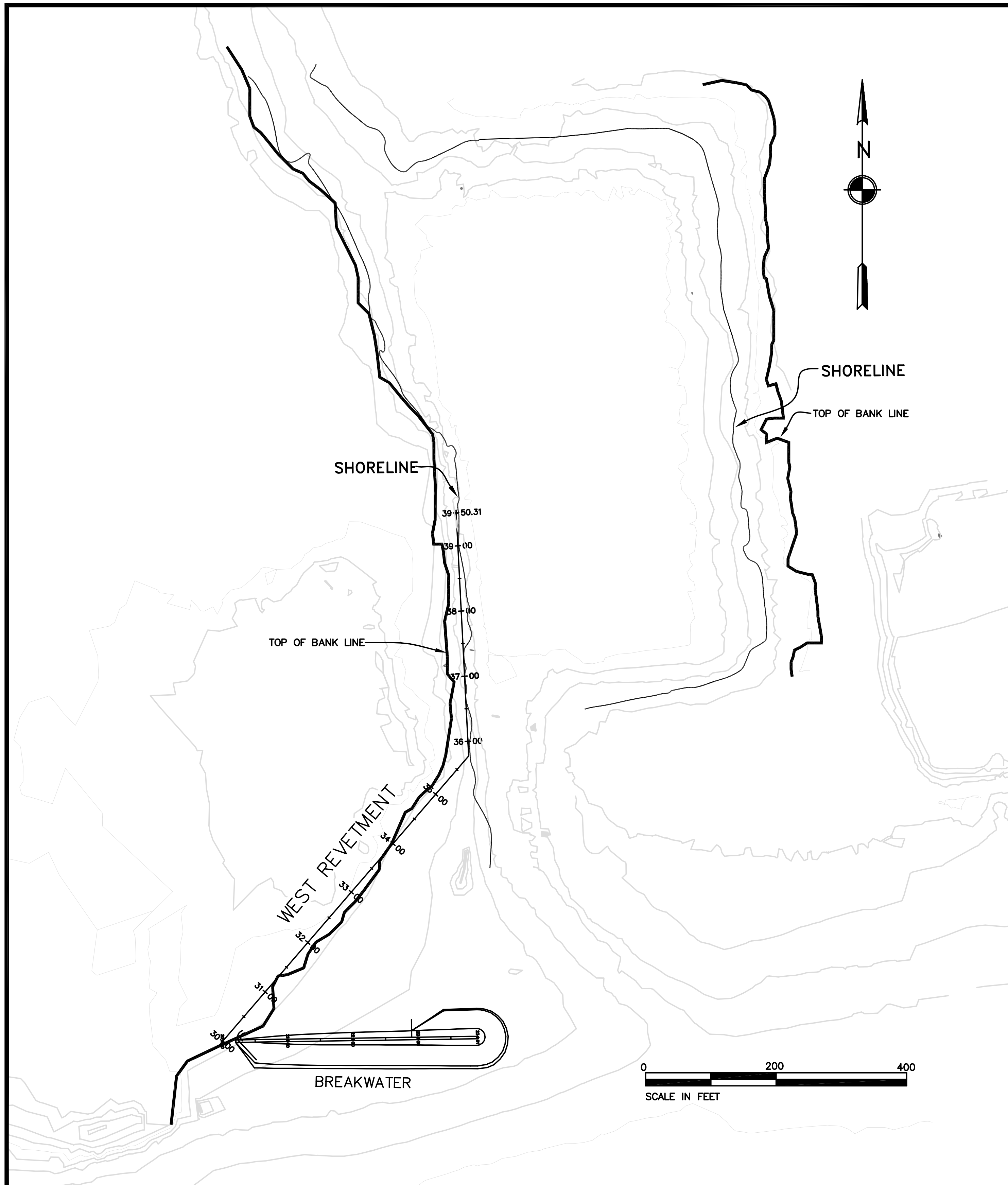
Maintenance of the rock revetment and breakwater would be the responsibility of the City of Dillingham. The Corps of Engineers, Alaska District would conduct periodic site inspections to verify whether maintenance is warranted on the revetment and breakwater. Both the revetment and breakwater were designed to be stable for the 50-year wave condition. Therefore, it is not anticipated that there will be significant loss of stone on the structures during the life of the project. Typically rock revetments and breakwaters have a 50-year design life, even though they may last much longer. Maintenance of a rock revetment and breakwater may require replacement of 2 percent of the armor stone every 25 years.

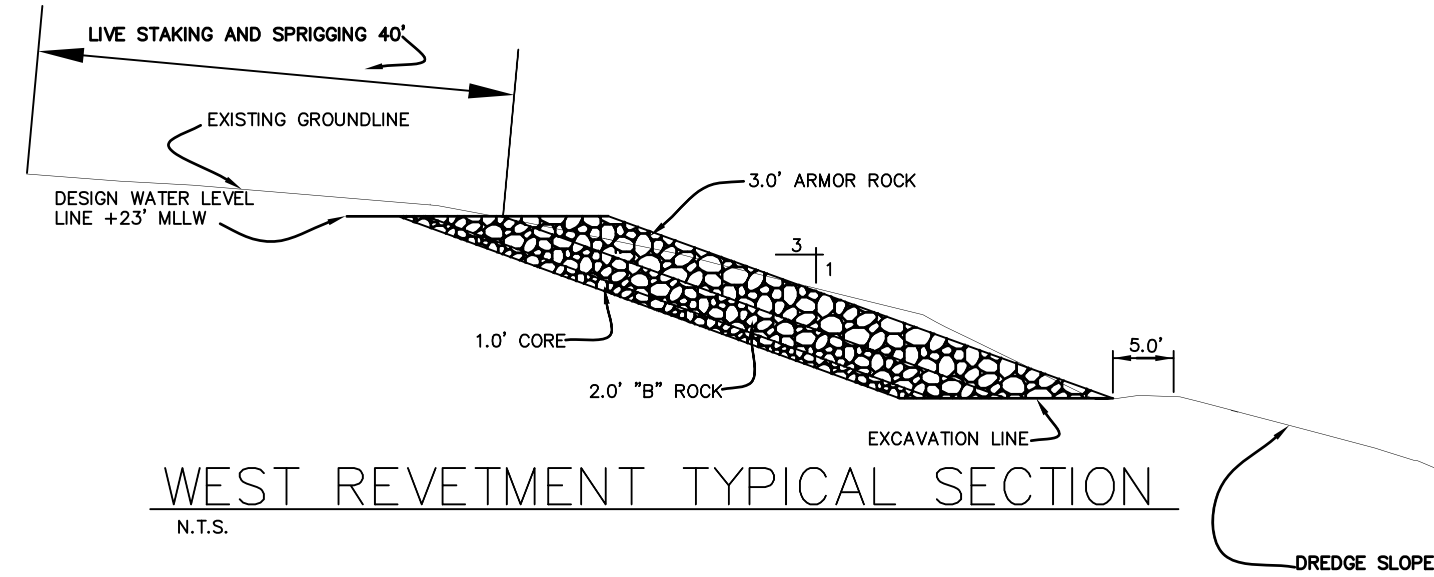
5.5 Alternative W5- Breakwater and West and East Revetments

This plan consists of a rubblemound breakwater and a rock revetment on both the west and east sides of the harbor. An east revetment was considered in this alternative to stabilize and refurbish the east bank of the harbor, which has experienced some minor erosion. The west revetment would consist of a 3-layer system of core, secondary, and armor stone. The revetment would start at the east end of the Bristol Fuel’s dock and continue along an alignment consistent with a natural shoreline for approximately 1,100 feet as shown in figure A-18. The east revetment would extend the length of the east side of the harbor along an alignment consistent with the natural shoreline (approximately 800 feet) and consist of a 2-layer system of secondary and core rock. While erosion of the east side of the harbor is not expected to continue with construction of a breakwater, this two-layer system of rock would repair this part of the shoreline that has already been impacted by erosion. No adverse impacts are anticipated with construction of the east revetment. No adverse impact on the Bristol Fuel’s dock or the existing harbor are expected to occur from construction of this alternative for the same reasons as stated in the alternative W1. The rubblemound breakwater would consist of a 3-layer system of core, secondary, and armor rock. The breakwater would be approximately 371 feet in length and extend east into the Nushagak River from the west side of the harbor. Erosion inside the harbor would be reduced due to the decreased wave climate.

Wave Attenuation

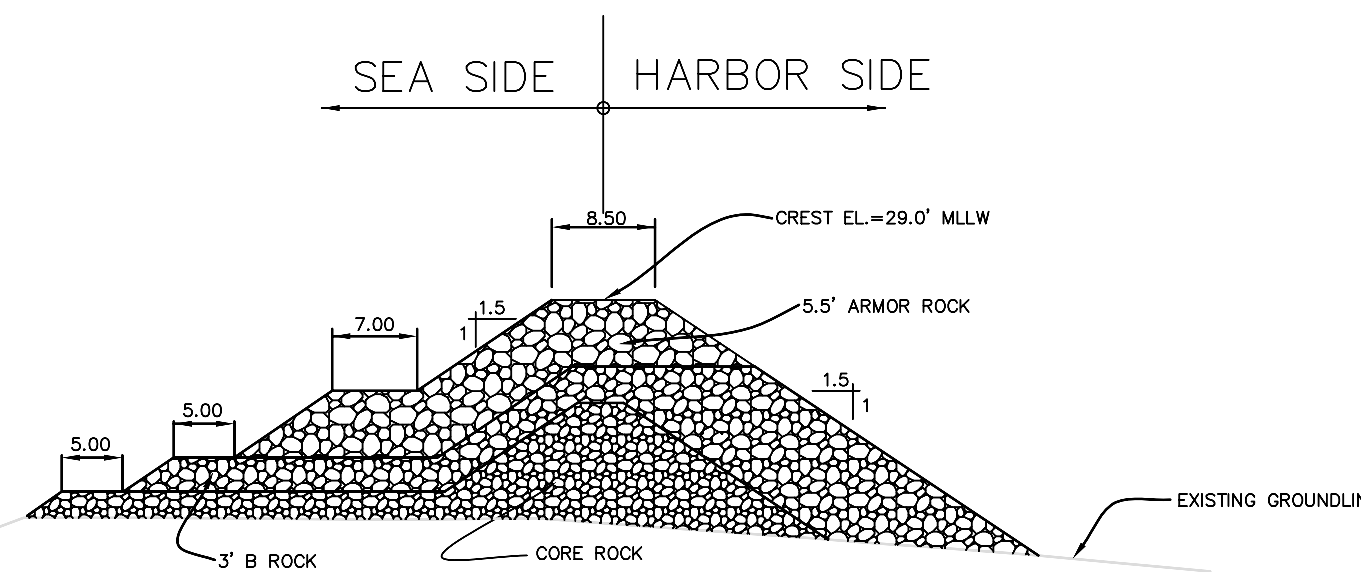
A diffraction analysis was done using methods in the SPM. Results of the analysis show that the proposed alignment would reduce waves to approximately 1.7 feet at the outermost float in the mooring area as shown in the diffraction diagram in figure A-19. Wave heights would become progressively smaller farther into the harbor.





WEST REVETMENT TYPICAL SECTION

N.T.S.



BREAKWATER TYPICAL SECTION

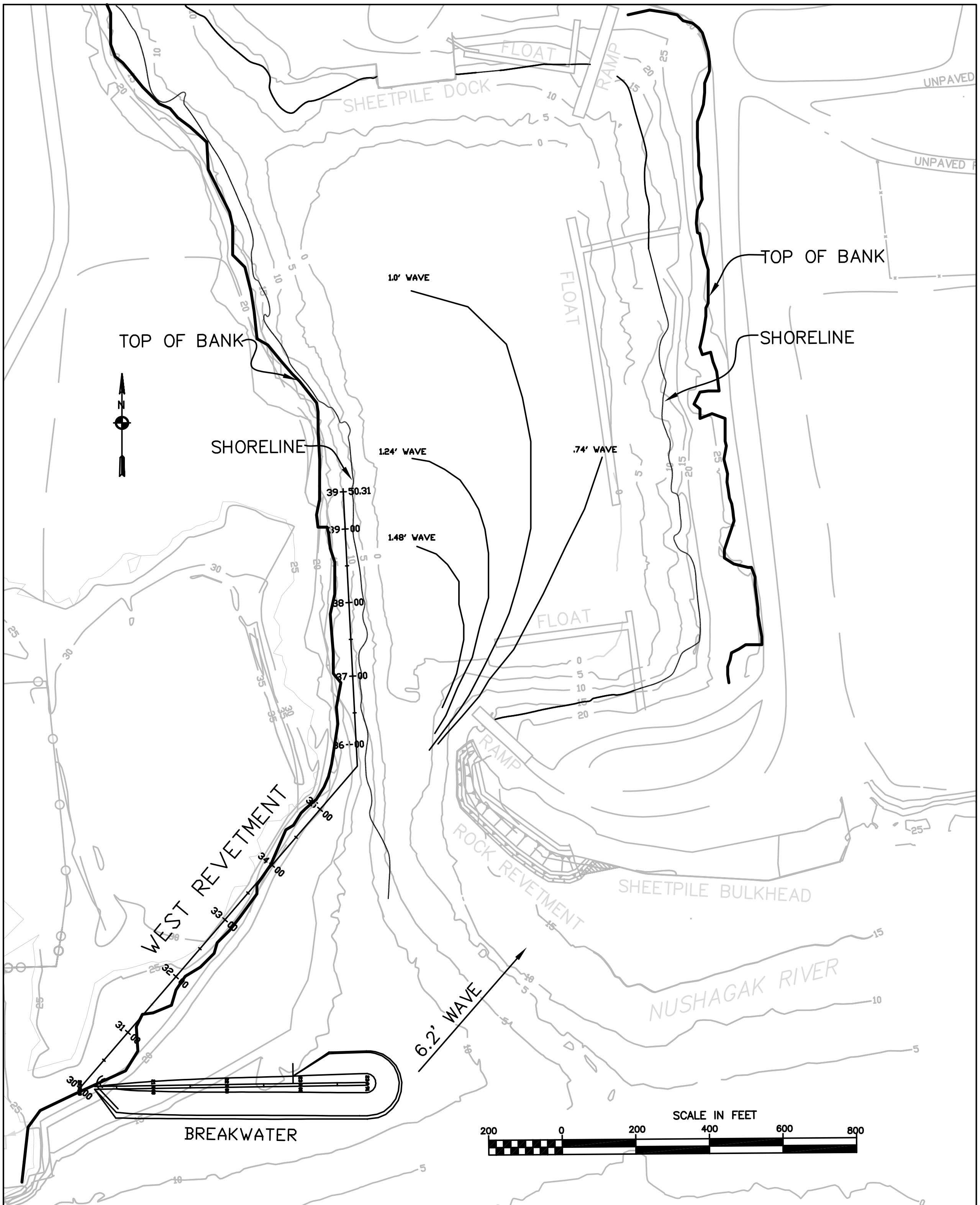
N.T.S.

ALASKA DISTRICT
CORPS OF ENGINEERS
CIVIL WORKS BRANCH

ALTERNATIVE W2 – BREAKWATER AND WEST ROCK REVETMENT

DILLINGHAM BANK STABILIZATION

A–14



ALASKA DISTRICT
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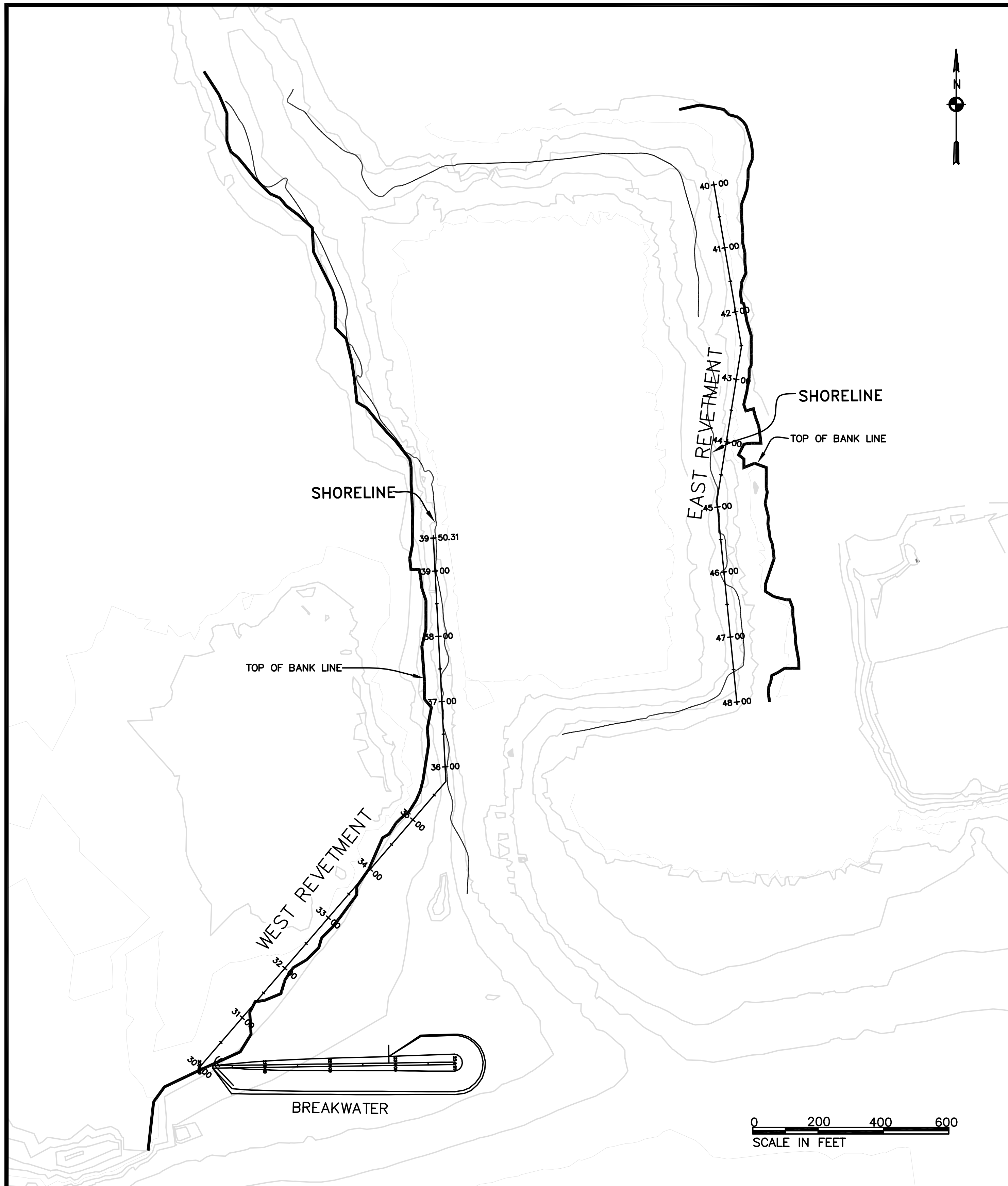
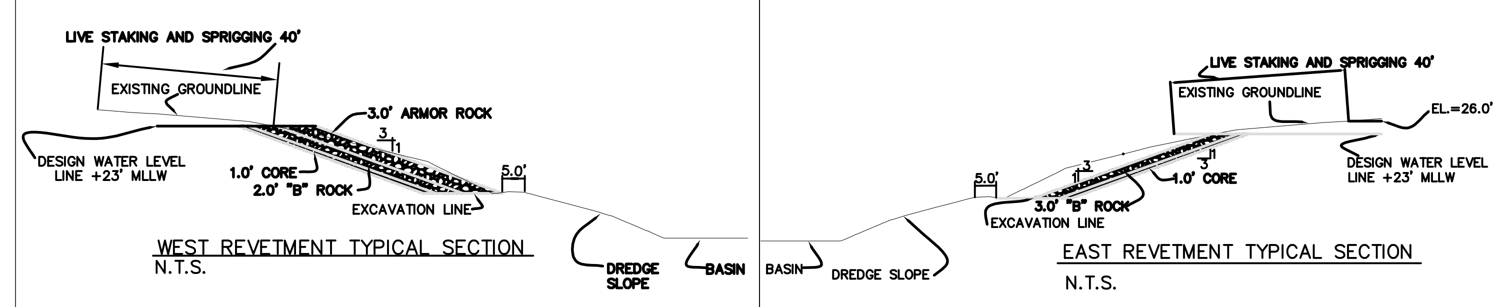
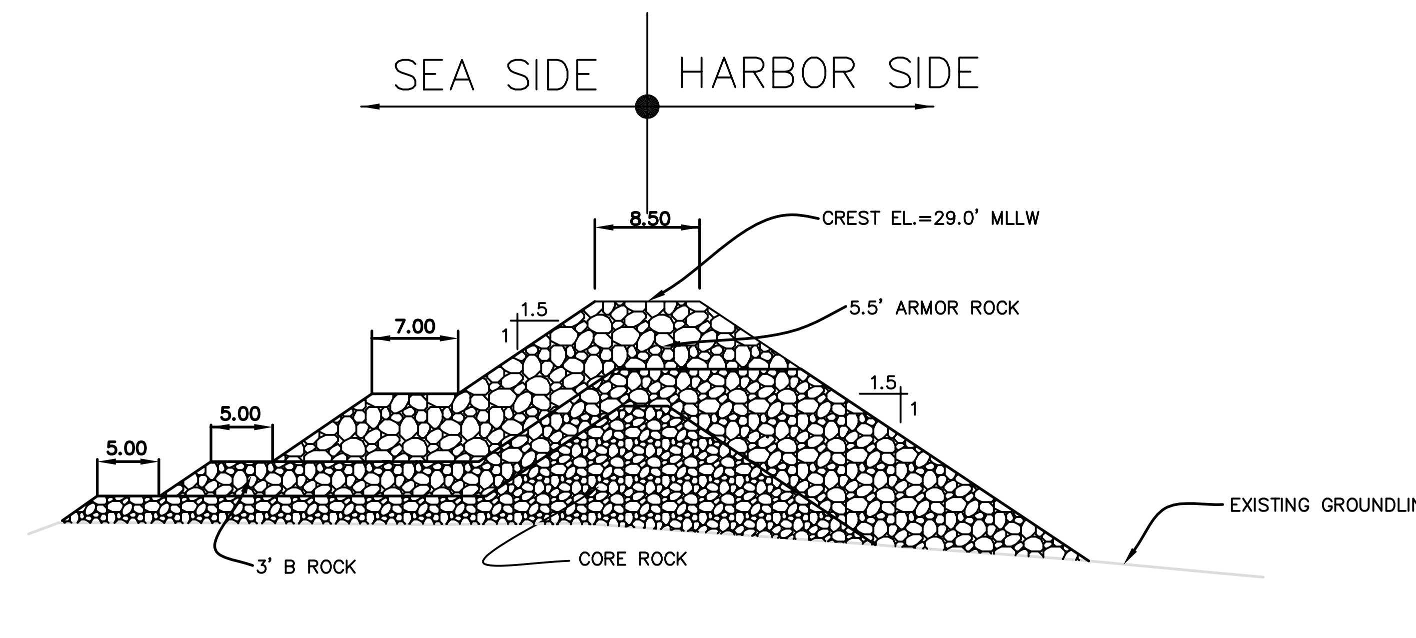
DIFFRACTION DIAGRAM – BREAKWATER AND ROCK REVETMENT

DILLINGHAM BANK STABILIZATION

A–15





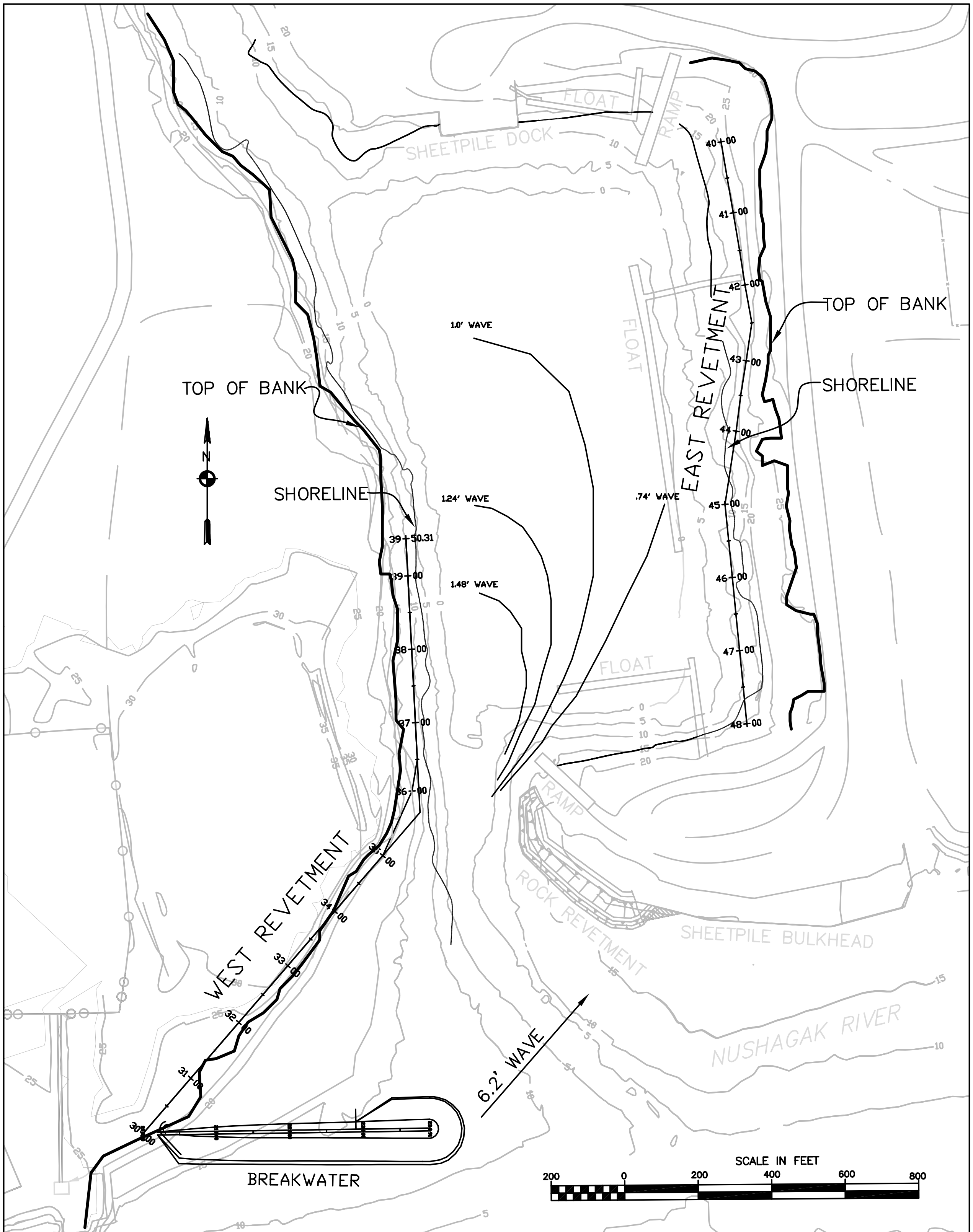


BREAKWATER TYPICAL SECTION
N.T.S.

ALASKA DISTRICT
CORPS OF ENGINEERS
CIVIL WORKS BRANCH

ALTERNATIVE W5 – BREAKWATER AND WEST AND EAST REVETMENTS
DILLINGHAM BANK STABILIZATION

A–16



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CORPS OF ENGINEERS
CIVIL WORKS BRANCH

DIFFRACTION DIAGRAM – BREAKWATER AND ROCK REVETMENTS

DILLINGHAM BANK STABILIZATION

A–17

Rock Size
The methods used to determine stone size for this alternative are the same as that of W2. The west revetment would require approximately 11,000 yd^3 of armor rock, ranging in size from 350 to 200 pounds. Secondary rock size would be between 200 and 20 pounds and would require 7,200 yd^3. Thirty-eight hundred yd^3 of core rock would be placed behind the secondary layer and would range in size from 20 pounds to 1 pound; 3,600 yd^3 of porous fill would be placed behind the revetment. For the east revetment, secondary rock size would be from 330 to 20 pounds and 5,300 yd^3 would be required. Twenty-five hundred yd^3 of core rock would be placed behind the secondary layer and would range in size from 20 pounds to 1 pound. Approximately 2,600 yd^3 of porous fill would be placed behind the revetment. There would be no armor rock on the east revetment in this alternative. A cross-section of the revetments is shown in figure A-18.

Methods described in the SPM using Hudson's equation were also used to calculate the stone size for the breakwater. The southwest wave height, 1v:1.5h side slopes, a specific gravity of 2.6, and a stability factor of 2.2 were used in the computation of stone size. The breakwater would require approximately 5,500 yd^3 of armor stone between 3,826 and 2,295 pounds. Secondary rock size would be from 2,295 to 230 pounds and would require 3,250 yd^3. Forty-three hundred yd^3 of core rock from 230 to 21 pounds would also be required. A cross-section of the breakwater is shown in figure A-18.

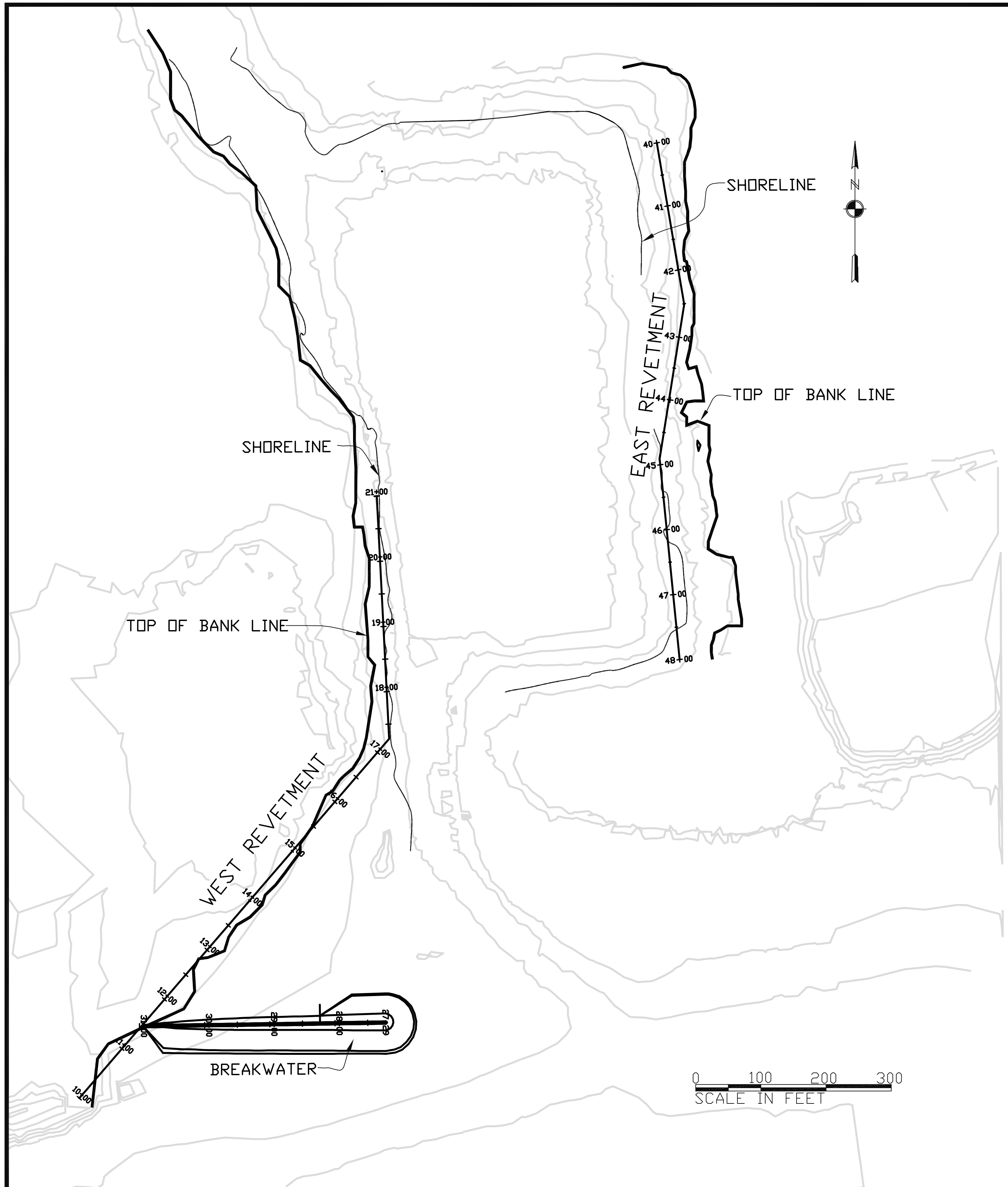
Both the west and east revetments would have a top elevation of +32 feet MLLW. The top elevation of the revetments was determined from 6 feet of wave run-up with a design high water level of 26 feet. The design water level equates to mean higher high water plus 6 feet of storm surge. The breakwater would also have a top elevation of +32 feet MLLW. The breakwater is designed to be an overtopping structure.

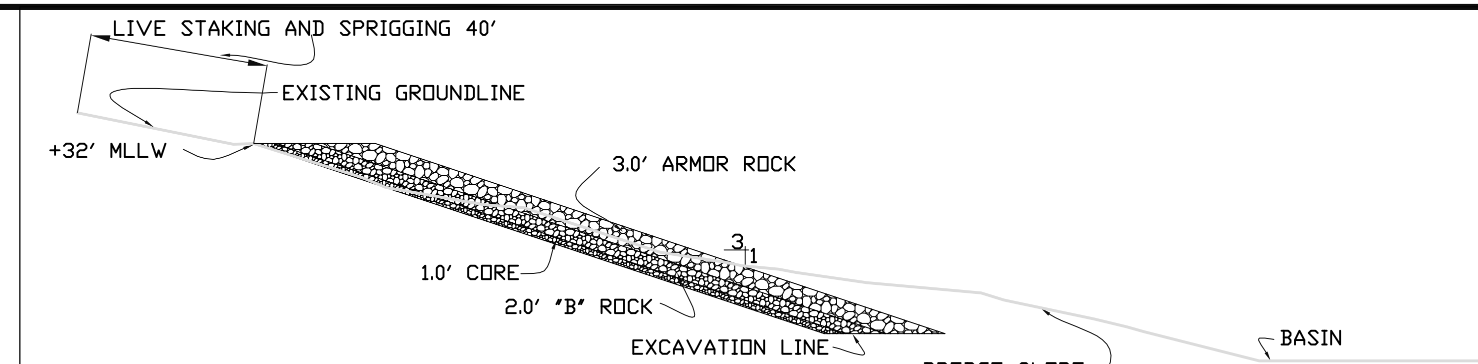
It is anticipated that navigation marker lights would be installed at the seaward end of the breakwater.

Maintenance of the rock revetments and breakwater would be the responsibility of the City of Dillingham. The Corps of Engineers, Alaska District would conduct periodic site inspections to verify whether maintenance is warranted on the revetments and breakwater. Both the rock revetments and breakwater were designed to be stable for the 50-year wave condition. Therefore, it is not anticipated that there would be significant loss of stone on the structure during the life of the project. Typically, rock revetments and breakwaters have a 50-year design life, even though they may last much longer. Maintenance of a rock revetment or breakwater may require replacement of 2 percent of the armor stone every 25 years.

6.0 DESIGN ALTERNATIVES – CITY DOCK SIDE OF HARBOR

Five alternatives, including the No-Action alternative were considered for stabilizing the city-dock side of the harbor at Dillingham. These alternative plans were evaluated using established design guidance given in the appropriate Corps of Engineers Engineering Manuals (EMs), the SPM, and the CEM.





WEST REVETMENT TYPICAL SECTION

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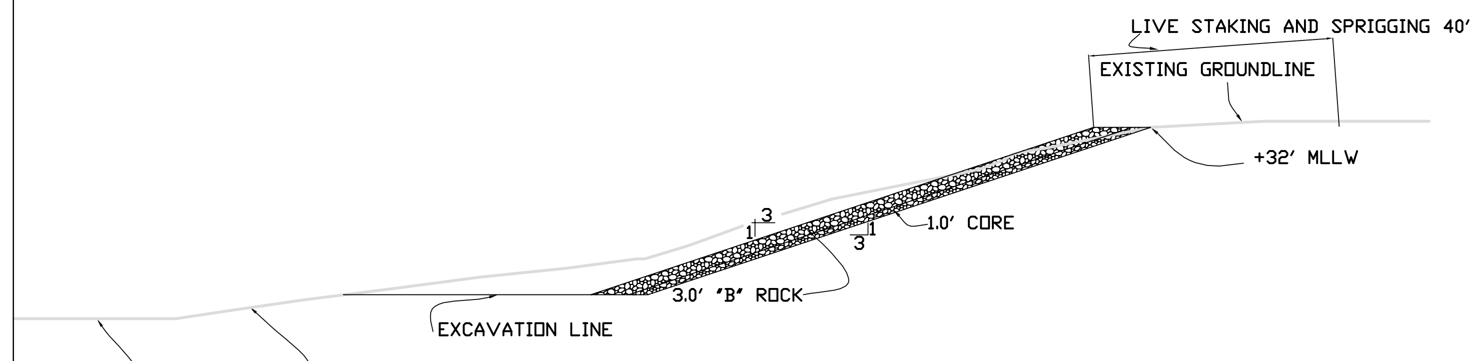

EAST REVETMENT TYPICAL SECTION

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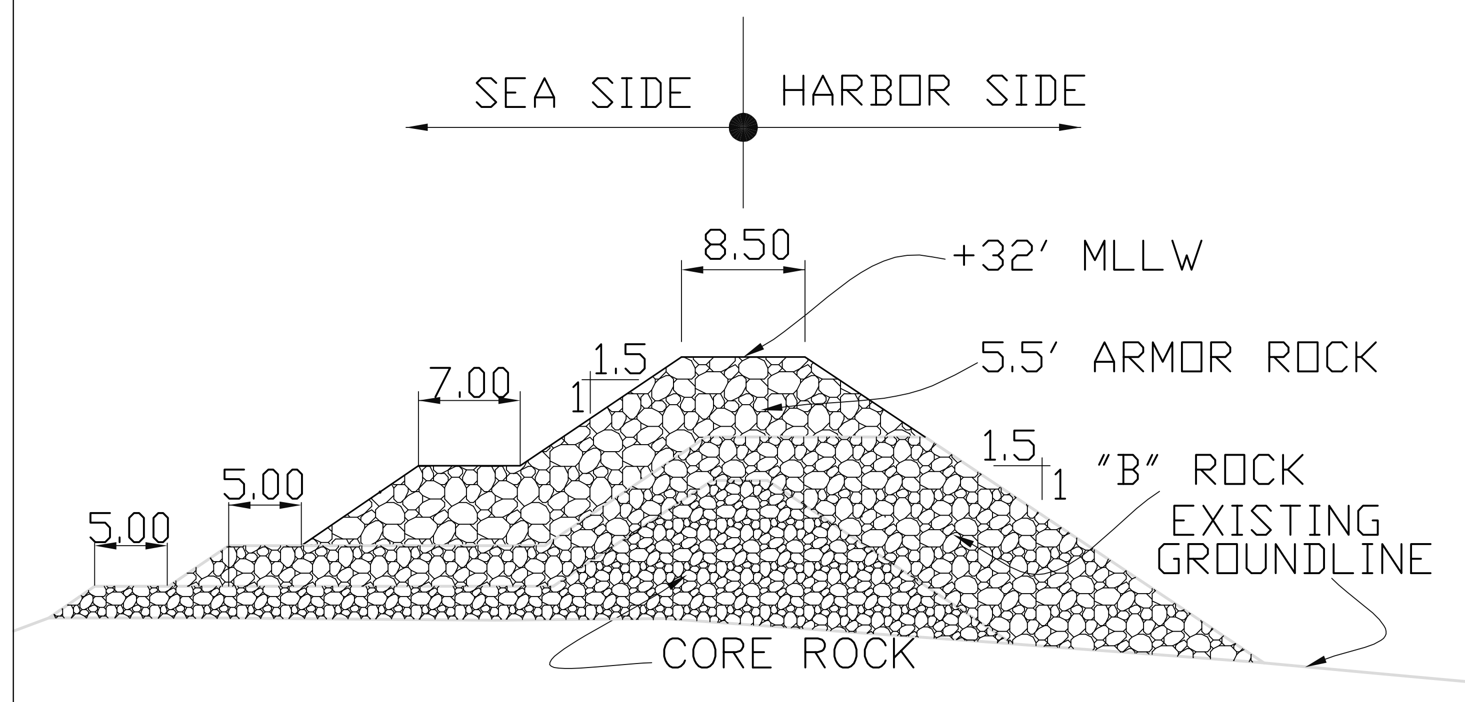


BREAKWATER TYPICAL SECTION

N.T.S.

ALASKA DISTRICT
CORPS OF ENGINEERS
CIVIL WORKS BRANCH

ALTERNATIVE W5 – BREAKWATER AND EAST AND WEST REVETMENTS

DILLINGHAM BANK STABILIZATION

A–18

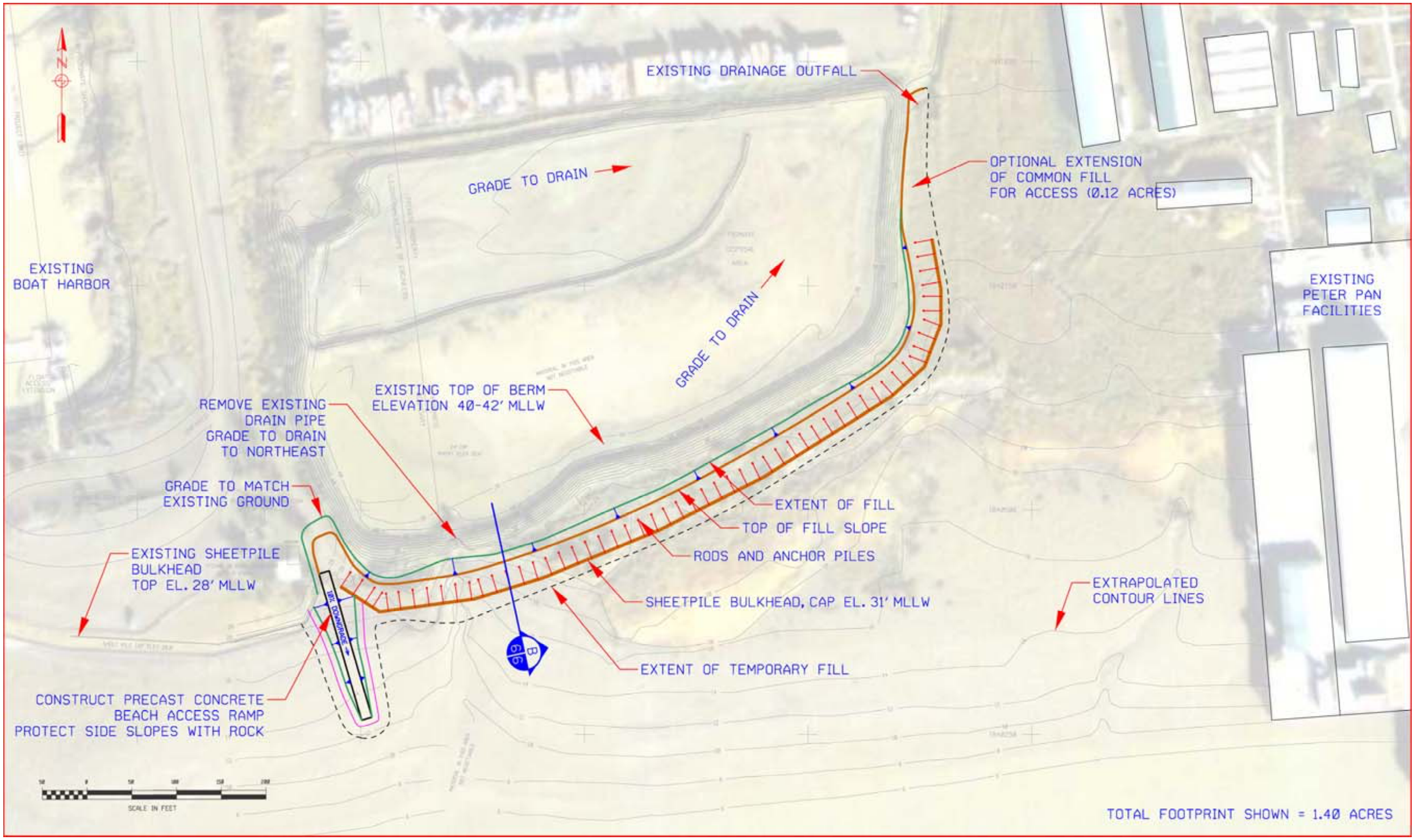


Figure 19, Alternative C2 – Sheetpile: Project Plan/Site Map

6.1 NO ACTION

The No-Action alternative assumes that the erosion continues at its current rate. Physical and financial impacts to existing facilities under this alternative are described in the accompanying economics report.

6.2 Alternative C1 – Rock Revetment

Alternative C1 includes a rock revetment that extends 850 feet from the eastern terminus of the existing harbor sheet-pile bulkhead, wrapping around the existing dredged material containment berm and extending approximately 100 feet landward and parallel to the shoreline where it keys into the east side of the existing containment berm. The dredged material disposal area behind the containment berm is no longer in use. The top elevation of the preliminary revetment design is set at +32 feet MLLW. This is based upon 6 feet of wave run-up the rubble slope with a design high water level of 26 feet, which equates to mean higher high water plus 6 feet of storm surge. The eastern terminus matches the existing disposal site embankment with free draining quarry waste material. A beach access ramp and disposal area drainage features are included in this alternative.

Total construction footprint for this alternative is 1.6 acres, including a construction-maintenance road extension for access to the project area from the southeast corner of the Peter Pan Facilities yard. Alternative C1 is shown in plan view in figure A-20.

The Corps of Engineers, Alaska District would perform periodic inspections to verify whether maintenance on the revetment is warranted. Rock surfaces should be inspected for ice damage and rock sizes should be checked to ensure that freeze-thaw action does not reduce the design gradation. The rock revetment is designed to be stable in extreme wave conditions; however, some maintenance activity is assumed throughout the project life. To account for maintenance activities resulting from the inspections, stone replacement is assumed at 2 percent of the installed armor layer every 25 years.

6.3 Alternative C2: Sheet-pile

Alternative C2 includes a sheet-pile bulkhead that extends along the same alignment as the revetment in Alternative C1. The preliminary bulkhead design has a capped top at elevation +32 feet MLLW. For the concept design, structural components of the sheet-pile system were consistent with those documented in USACE, September 2002. The eastern terminus of the bulkhead wraps around the southeast corner of the existing containment berm and extends an additional 100 feet landward. Along this eastern reach, the bulkhead transitions to rock revetment, which is keyed into the east side of the existing containment berm.

A drainage system is included with free-draining material placed against the bulkhead and 6-inch-diameter weepholes at maximum 12-foot spacing. Safety ladders are included at regular intervals, as required by City of Dillingham regulations. Fish net attachments are included at 100-foot spacing to accommodate local subsistence fishing. Corrosion protection (coal tar epoxy coating and galvanic anodes) is recommended for sheetpiles, HP-piles, and anchor rods.

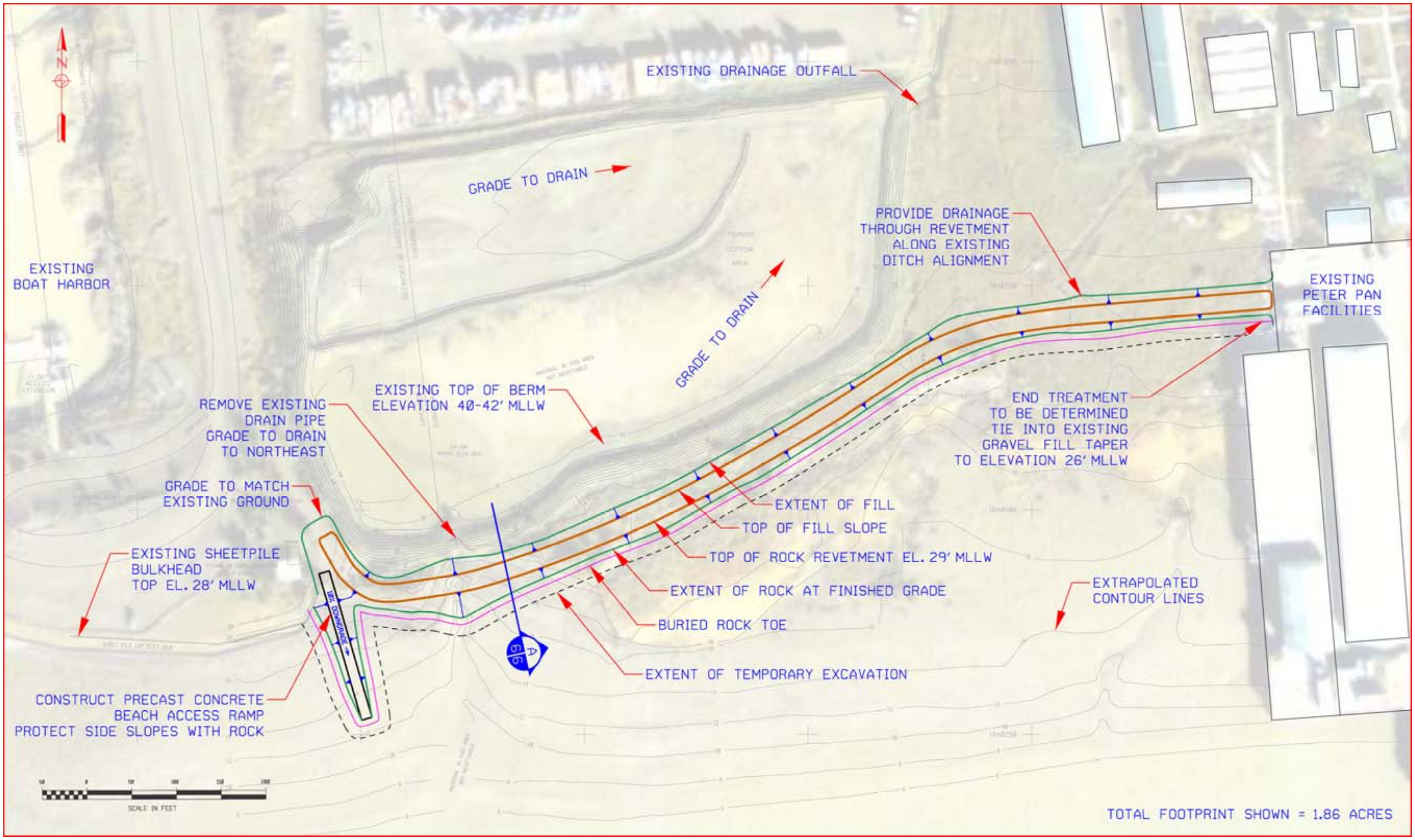


Figure 20, Alternative C3 – Revetment: Project Plan/Site Map

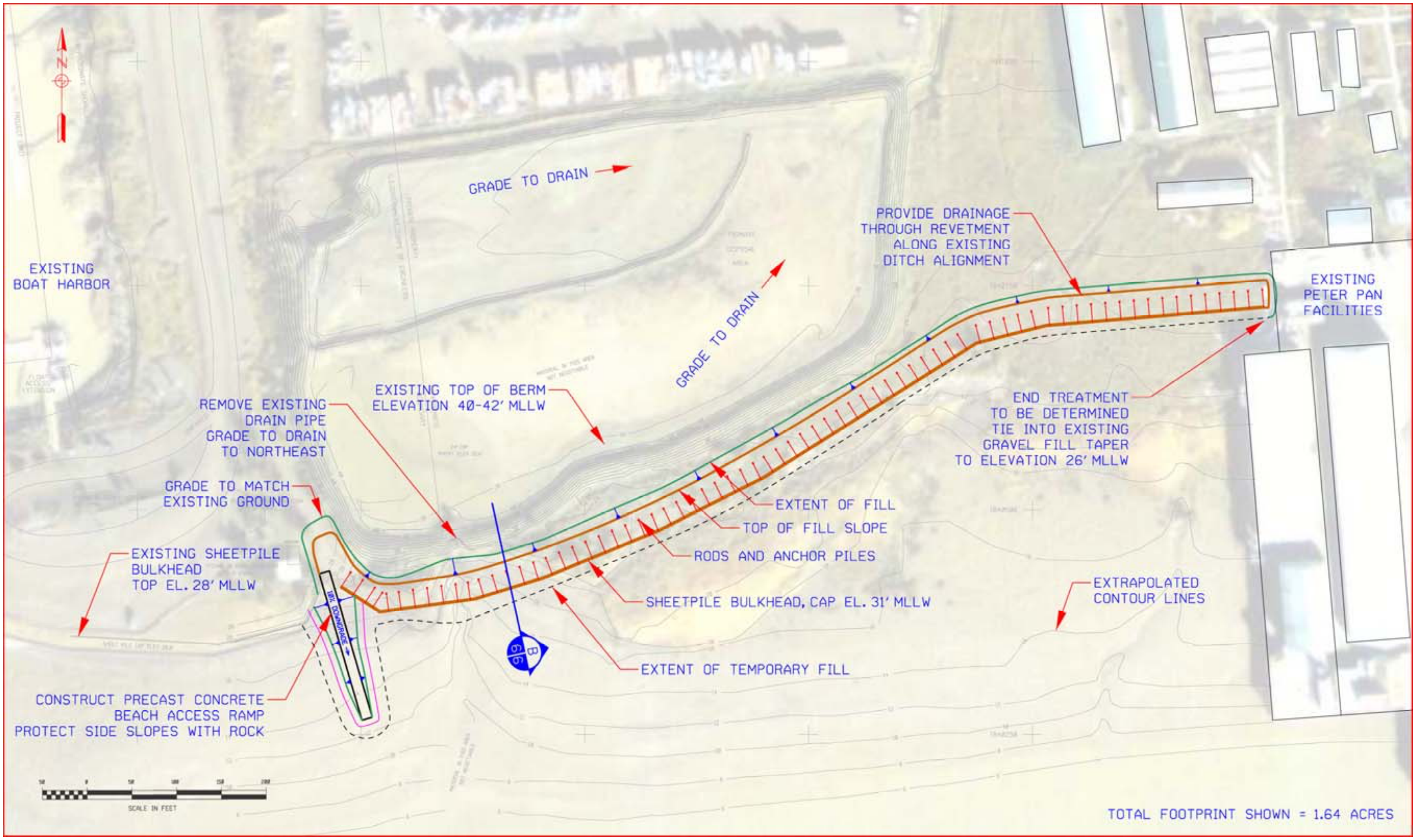


Figure 21, Alternative C4 – Sheetpile: Project Plan/Site Map

The total construction footprint for this alternative is 1.4 acres, including a construction-maintenance road extension for access to the project area from the southeast corner of the Peter Pan Facilities yard.

A beach access ramp and disposal area drainage features are included in this alternative. Alternative C2 is shown in plan view in Figure A-21.

The Alaska District will perform periodic inspections to verify whether maintenance on the revetment is warranted. Rock surfaces should be inspected for ice damage, and rock sizes should be checked to ensure that freeze-thaw action does not reduce the design gradation. The rock revetment is designed to be stable in extreme wave conditions; however, some maintenance activity is assumed throughout the project life. To account for maintenance activities resulting from the inspections, stone replacement is assumed at 2 percent of the installed armor layer 25 years into the project life.

The sheet-pile has a 30-year design life. It is assumed that the sheet-pile would need to be replaced at year 30 and that periodic maintenance would have to be performed on the structure at intervals of not more than 15 years. This replacement is expected to be accomplished by driving new sheet-pile in front of the old. The cathodic protection of the structure would need to be inspected annually.

6.4 Alternative C3: Rock Revetment

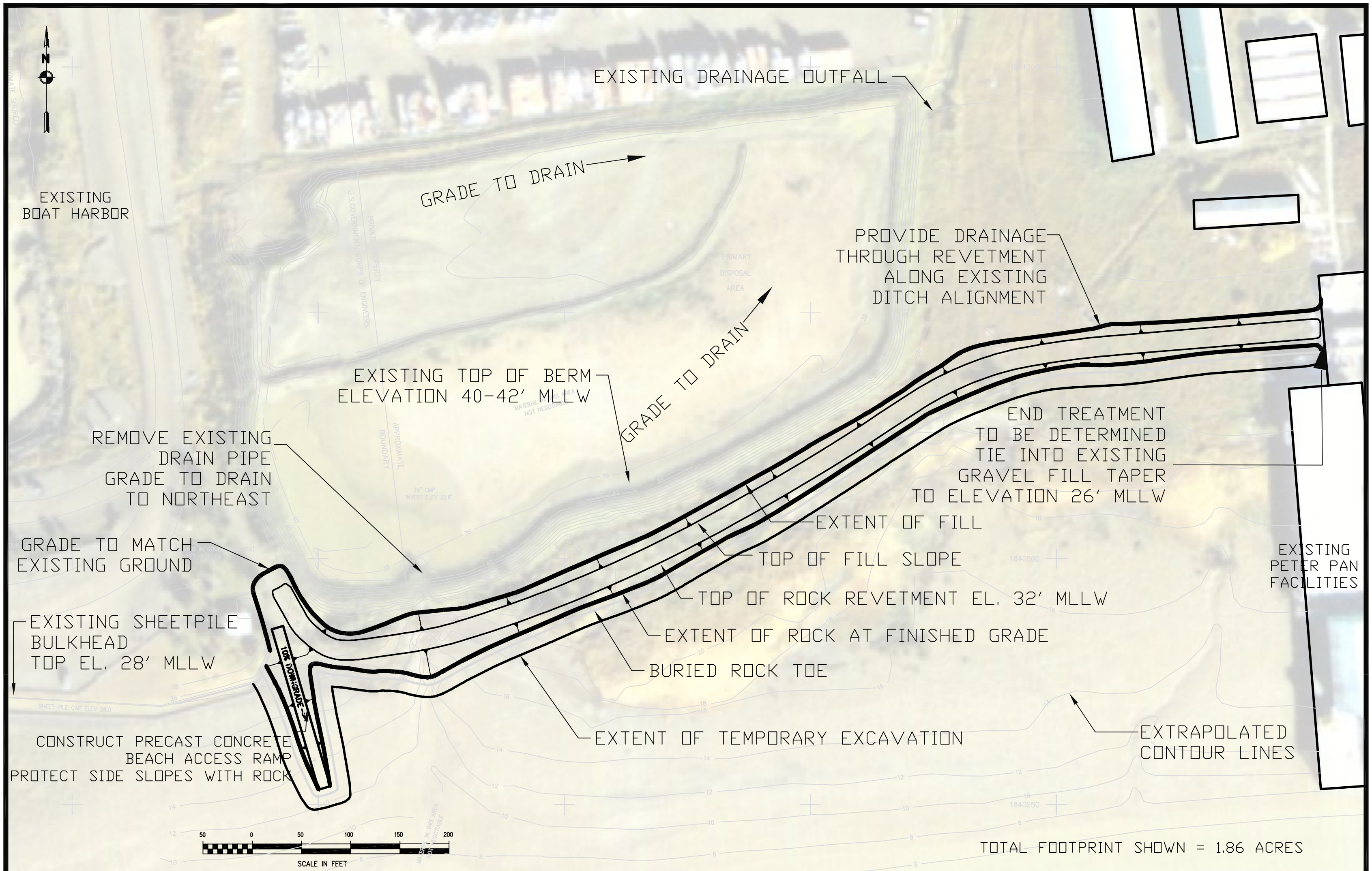
The configuration of Alternative C3 is the same as Alternative C1 but the alignment extends eastward to the westernmost dock of Peter Pan Seafoods. The extended alignment runs approximately 1,150 feet to the east from the eastern terminus of the existing harbor sheet-pile bulkhead. A beach access ramp and disposal area drainage features are included.

This alignment crosses an existing drainage channel between the dredged material disposal area and the Peter Pan docks, requiring a drainage culvert(s) through the proposed revetment and fill section in this location. The specific location and sizing of this culvert would require further evaluation if this alternative was selected for further consideration, requiring development of new hydrologic and topographic data.

The revetment would allow transfer of energy along its alignment. Excess energy would cause some disruption of the topography at the terminus unless dissipated. The end treatment would require further investigation if this alternative was considered further.

Total construction footprint for this alternative is 1.9 acres, including the access road extension described in Alternatives C1 and C2. Alternative C3 is shown in plan view in figure A-22.

The Alaska District would perform periodic inspections to verify whether maintenance on the revetment is warranted. Rock surfaces should be inspected for ice damage and rock sizes should be checked to ensure that freeze-thaw action does not reduce the design gradation. The rock revetment is designed to be stable in extreme wave conditions; however, some maintenance activity is assumed throughout the project life. To account for maintenance activities resulting from the inspections, stone replacement is assumed at 2 percent of the installed armor layer every 25 years.



ALASKA DISTRICT
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CIVIL WORKS BRANCH

ALTERNATIVE C3 – ALIGNMENT 2 WITH REVETMENT

DILLINGHAM BANK STABILIZATION

A–22

6.5 ALTERNATIVE C4: SHEETPILE

The composition of the bulkhead for Alternative C4 is the same as for Alternative C2, and the alignment and length is the same as Alternative C3. This alignment crosses an existing drainage channel between the Peter Pan dock and the dredged material disposal area. A drainage culvert would be required through the proposed bulkhead at this location. The beach access ramp and disposal area drainage features are included.

As with Alternative C3, the sheet-pile alternative will transfer energy along the alignment. When bank-hardening projects terminate, the excess energy causes some disruption of the topography unless dissipated. With revetment designs, natural vegetation may be adequate to dissipate much of the energy. Vertical walls, however, conserve and transfer energy much more effectively than do laid back revetment slopes. The terminating ends of the sheet-pile would require rock revetment protection and additional structural features for energy dissipation. The end treatment would require further investigation if this alternative is considered further.

The total construction footprint for this alternative is 1.6 acres, including the access road extension described in Alternatives C1, C2, and C3. Alternative C4 is shown in plan view in figure A-23.

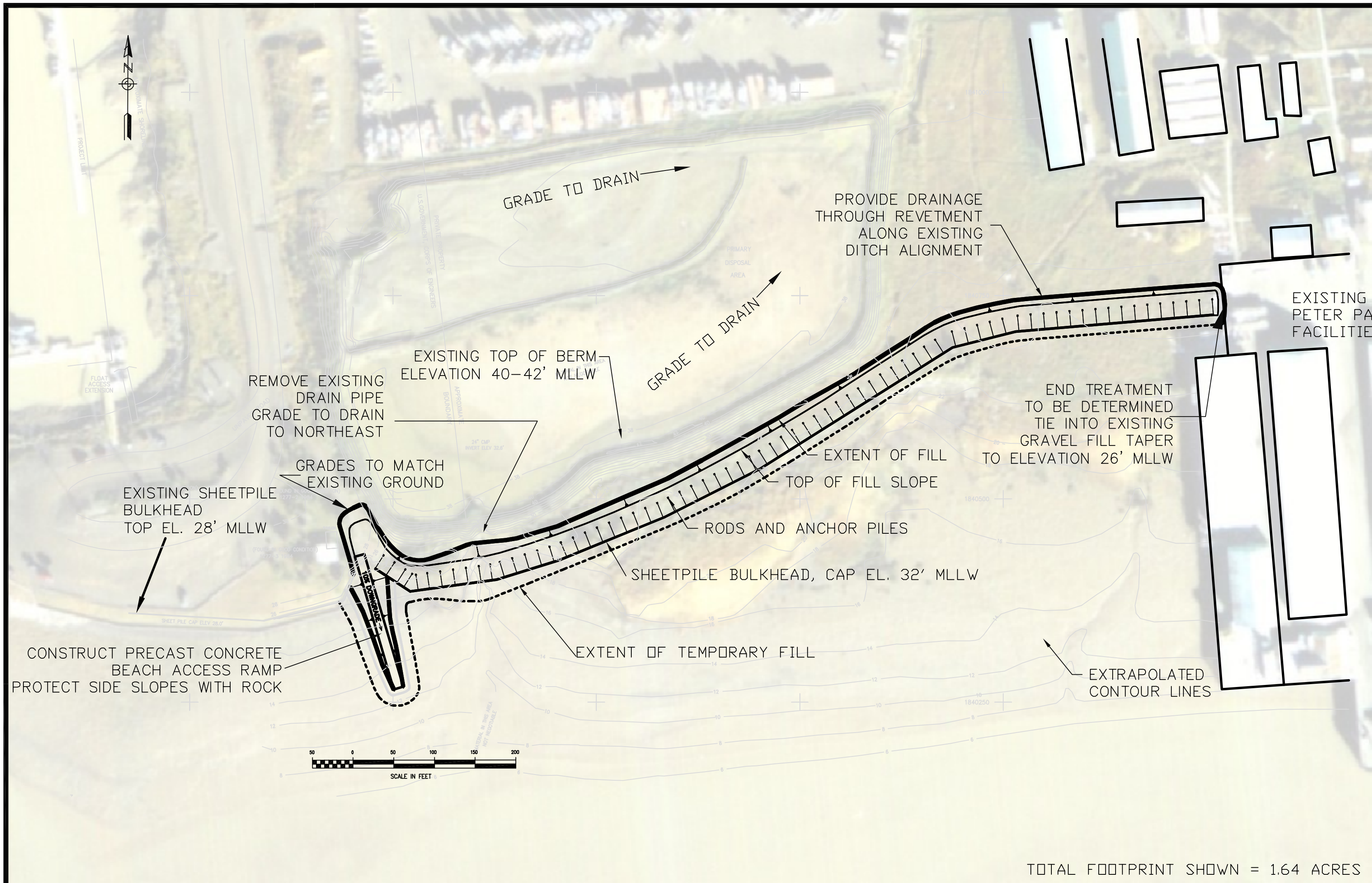
The U.S. Army Corps of Engineers, Alaska District will perform periodic inspections to verify whether maintenance on the revetment is warranted. Rock surfaces should be inspected for ice damage and rock sizes should be checked to ensure that freeze-thaw action does not reduce the design gradation. The rock revetment is designed to be stable in extreme wave conditions; however, some maintenance activity is assumed throughout the project life. To account for maintenance activities resulting from the inspections, stone replacement is assumed at 2 percent of the installed armor layer every 25 years.

7.0 SELECTED PLAN DRAFT DESIGN

The draft design is shown in Appendix A, figures A-14 and A-20. The revetments and breakwater in the selected plan follows alignments W2 and C1.

The recommended plan for the west side of the harbor consists of a rubblemound breakwater and a rock revetment. The revetment and the breakwater would both consist of a 3-layer system of core, secondary, and armor stone. The revetment would start at the eastern terminus of the Bristol Fuel's dock and continue along an alignment consistent with a natural shoreline for approximately 1,100 feet as shown in figure A-14. The top of the revetment would be vegetated for bird habitat with willow cuttings, sprigging of native grasses, and seeding. The breakwater would be approximately 371 feet and extend east into the Nushagak River from the west side of the harbor.

Methods described in the SPM using Hudson's equation were used to calculate the stone size for the revetment. The southeast wave height, 1v:3h side slopes, a specific gravity of 2.6, and a stability factor of 2.2 were used in the computation of stone size. The west revetment would require approximately 11,000 yd^3 of armor rock, ranging in size from 350 to 200 pounds. Secondary rock size would be between 200 and 20 pounds and would require 7,200 yd^3. Thirty-eight hundred yd^3 of core rock would be placed



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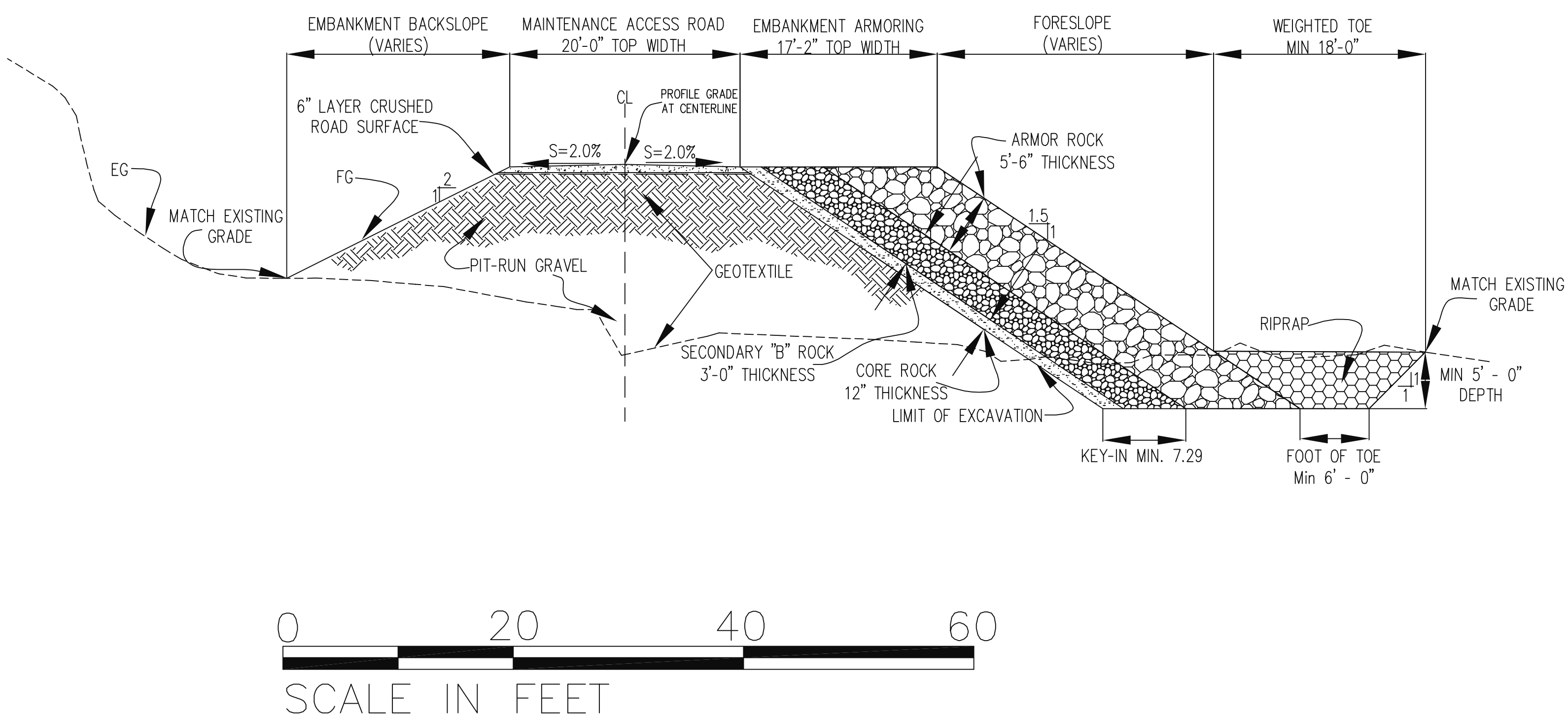
ALTERNATIVE C4 – ALIGNMENT 2 WITH SHEETPILE

DILLINGHAM BANK STABILIZATION

A-23



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CITY DOCK SIDE REVETMENT TYPICAL CROSS–SECTION

DILLINGHAM BANK STABILIZATION

A–24

behind the secondary layer and would range in size from 20 pounds to 1 pound. Approximately 3,600 yd^3 of porous fill would be placed behind the revetment. A cross-section of the revetment is shown in figure A-14.

Methods described in the SPM using Hudson's equation were also used to calculate the stone size for the breakwater. The southwest wave height, 1v: 1.5h side slopes, a specific gravity of 2.6, and a stability factor of 2.2 were used in the computation of stone size. The breakwater would require approximately 5,500 yd^3 of armor between 3,826 and 2,295 pounds. Secondary rock size will be from 2,295 to 230 pounds and would require 3,250 yd^3. Forty-three hundred yd^3 of core rock from 230 to 21 pounds would also be required. A cross-section of the breakwater is shown in figure A-14.

The west revetment would have a top elevation of +32 feet MLLW. The top elevation of the revetment was determined from 6 feet of wave run-up with a design high water level of 26 feet. The breakwater would have a crest elevation of +32 feet MLLW. The breakwater is designed to be an overtopping structure.

The western extent of the city dock side revetment is the existing USACE sheet-pile bulkhead constructed in 1999. The revetment extends approximately 850 feet adjacent to the existing dredged material disposal area containment berm. The eastern terminus of the revetment reaches a point approximately 100 feet landward of the primary alignment parallel to the shoreline. The depth of the armored toe in this area tapers up to meet existing ground at the eastern terminus.

The revetment on the city dock side of the harbor on the seaward side of the proposed alignment is to be placed at a 1v:1.5h slope. The armor layer is 5.5 feet thick, measured perpendicular to the face of the slope. The rock in the armor layer ranges in weight from 2,590 to 4,315 pounds; 5,600 yd^3 of armor rock would be needed for the revetment. The gradation should be as uniform as possible with a median stone weight of 3,450 pounds. The secondary rock layer measures 3 feet thick and contain 3,100 yd^3. The secondary rock ranges in weight from 260 to 2600 pounds, with a median stone weight of 350 pounds. The core layer measures 1 foot thick with a weight range of 12 to 260 pounds and a median stone weight of 17 pounds. It is anticipated that the material for the core layer would be selected from the quarry waste material; 1,100 yd^3 of core material would be required for the revetment. A cross-section of the revetment is shown in figure A-24.

The trench extends outward 6 feet laterally from the seaward face of the toe, and then slopes up to meet existing ground at an assumed 1v:1h slope. The actual slope may vary with the stability and angle of repose of the native material encountered during construction. A geotextile fabric would underlie the armor layers in the toe trench. The entire trench outside the revetted section would be backfilled with riprap gradation material. The riprap material would range in weight from 140 to 4,500 pounds with a median stone weight of 1,120 pounds. The toe trench backfill material is assumed to match the gradation of the beach access ramp surface layer. The porous fill material (free-draining, pit-run gravel) would extends along the alignment with a minimum 20-

foot top width. Approximately 9,300 yd^3 of porous fill would be needed. The top width is provided as a minimum operable width for construction and maintenance equipment.

The existing 24-inch CMP drainpipe located near the southwest corner of the disposal area is to be plugged as part of the selected plan.

The city dock side selected alternative includes a beach access ramp adjacent to the east end of the existing sheet-pile bulkhead. This access ramp would serve as a foundation for temporary dredged material slurry lines and potentially for serving as public access to the beach for local subsistence and recreational activities.

8.0 CONSTRUCTION SCHEDULE

The time needed for construction of this project is estimated at 15 months. Project specifications will detail time restrictions for the contractor to conduct certain activities during specified time periods.

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