APPENDIX C

HYDRAULIC DESIGN

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1.0 INTRODUCTION

1.1 Appendix Purpose

This appendix describes the hydraulic design of the Craig Navigation Improvement Project. It provides the background for determining the Federal interest in the major construction features including breakwater construction dredging, and operation and maintenance.

1.2 Project Purpose

The purpose of this study is to identify a design to provide safe and efficient moorage for the design fleet identified in this study. Improvements were screened to ensure the navigation improvement measures were evaluated in detail for the National Economic Development (NED) and locally preferred plan.

1.3 Description of Project Area

Craig is located on a small Island off the west coast of Prince of Wales Island (Figure 1). It is 56 air miles northwest of Ketchikan, 750 miles north of Seattle, and 220 miles south of Juneau.

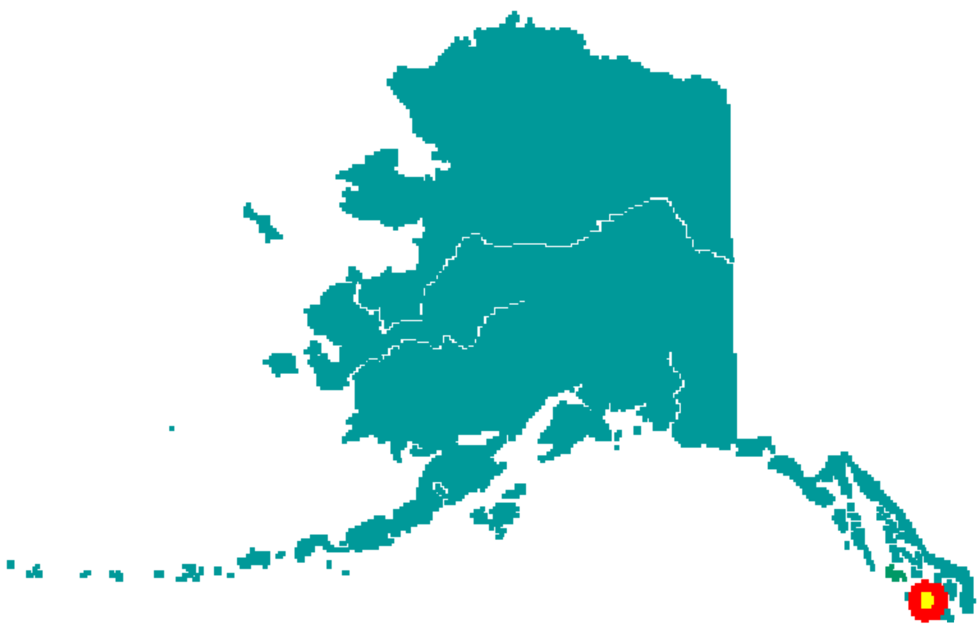

Figure 1 State of Alaska location map with location of Craig.

2.0 CLIMATOLOGY, METEOROLOGY, HYDROLOGY

2.1 Temperature and Precipitation

Craig (Figure 2) is dominated by a cool, moist, maritime climate. Summer temperatures range from 49-63° F. Winter temperatures range from 32 to 42° F. Average annual precipitation is 120 inches, and average annual snowfall is 40 inches (**Table** 1).

Figure 2 Craig's location on Prince of Wales Island

Table 1 Monthly Climate Summary CRAIG, ALASKA (502227)

Period of Record : 9/ 2/1949 to 9/30/2012

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	39.4	41.4	43.1	49.3	55.0	60.1	62.5	63.5	59.2	51.8	44.6	41.7	51.0
Average Min. Temperature (F)	29.6	31.4	31.9	36.2	41.6	47.5	51.2	51.4	48.3	42.0	35.7	33.0	40.0
Average Total Precipitation (in.)	8.24	8.40	8.07	7.41	5.38	3.05	4.13	6.02	10.17	13.06	12.29	10.80	97.04
Average Total SnowFall (in.)	5.1	6.3	5.8	0.4	0.0	0.0	0.0	0.0	0.0	0.0	1.9	3.0	22.5
Average Snow Depth (in.)	1	0	0	0	0	0	0	0	0	0	0	0	0

2.2 Ice Conditions

Craig is ice free year round.

2.3 Tides

Craig is in an area of semi-diurnal tides with two high waters and two low waters each lunar day. The tidal parameters in Table 2 were determined using National Oceanic and Atmospheric Administration published data. The tide data is based on observations made during the months May through June 2007. There was no reported highest observed water level and no lowest observed water level.

Table 2 Tidal Parameters – Craig

Parameter	Elevation (ft)
Highest Astronomical Tide	12.59
Mean Higher High Water (MHHW)	10.17
Mean Sea Level (MSL) *	5.34
Mean Tide Level (MTL) **	5.35
Mean Lower Low Water (MLLW)	0.00
Lowest Astronomical Tide	-2.95

***MSL** The arithmetic mean of hourly heights observed over the National Tidal Datum Epoch. Shorter series are specified in the name; e.g. monthly mean sea level and yearly mean sea level.
****MTL** The arithmetic mean of mean high water and mean low water.

2.4 Currents

Current data was collected by NOAA off Fish Egg Island from 26 April 2009 through 7 June 2009 (Figure 3). The data collected during that time period indicates that currents can reach up to 1.26 knots (Figure 4). Average current velocities associated with approximate depths are shown in Table 3. No data is available for current velocities in the fall when storms in the Gulf of Alaska are more common.

Over the 28 year period from 1986 to 2014, the highest predicted flood current was 1.5 knots, and the highest predicted ebb current was 1.9 knots, using the Tides and Currents for Windows program.

Figure 3 Location of current meter

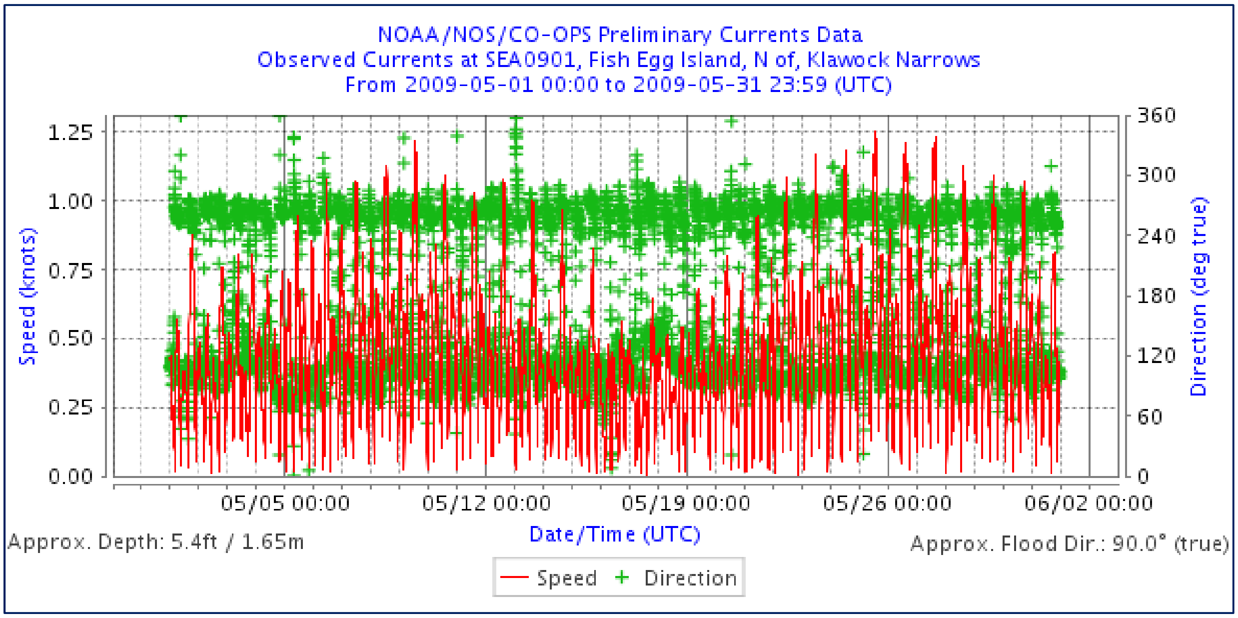

Figure 4. Sample data from May 2009

Table 3 Average Current Velocity

Fish Egg Island, N of, Klawock Narrows (SEA0901) Deployed (UTC): 2009-04-26 18:11:00 to 2009-06-07 17:07:00						
Approximate Depth	5.4 ft	15.3 ft	25.1 ft	34.9 ft	44.8 ft	54.6 ft
Average Velocity [knots]	0.44	0.40	0.38	0.35	0.33	0.30

2.5 Water Level

The effect of an increase in water level needs to be evaluated when designing a navigation project. Water level increase is typically a result of wave set up, storm surge, and tide. Relative sea level rise is a longer term increase in water level and its effects on a project is an additional factor that needs to be considered in a breakwater design.

Wave Setup
Wave setup is the water level rise at the coast caused by breaking waves. The breakwaters evaluated for this project extend beyond the area of breaking waves so wave set up was not considered in the calculations for the Craig Navigation Improvement project.

Storm Surge
Storm surge is an increase in water elevation caused by a combination of relatively low atmospheric pressure and wind driven transport of seawater over relatively shallow and large unobstructed waters. Friction at the air-sea interface is increased when the air is colder than the water, which causes more wind-driven transport. Storm induced surge can produce short term increases in water level, which can rise to an elevation considerably above tidal levels. Craig experiences low pressure events that could contribute to storm surge, but the water is too deep to stack up and cause a significant surge. A rise in the water elevation due to surge has not been a problem reported at Craig, so no storm surge was used in the calculations for the project.

Tide
The mean higher high tide of 10.17 feet was used for the high water elevation.

Sea Level Rise
The Corps of Engineers requires that planning studies and engineering designs over the project life cycle, for both existing and proposed projects consider alternatives that are formulated and evaluated for the entire range of possible future rates of sea-level change (SLC), represented by three scenarios of “low,” “intermediate,” and “high” sea-level change. The SLC “low” rate is the historic SLC. The “intermediate” and “high” rates are computed using the following:

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

NRC Equations

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

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

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

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

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

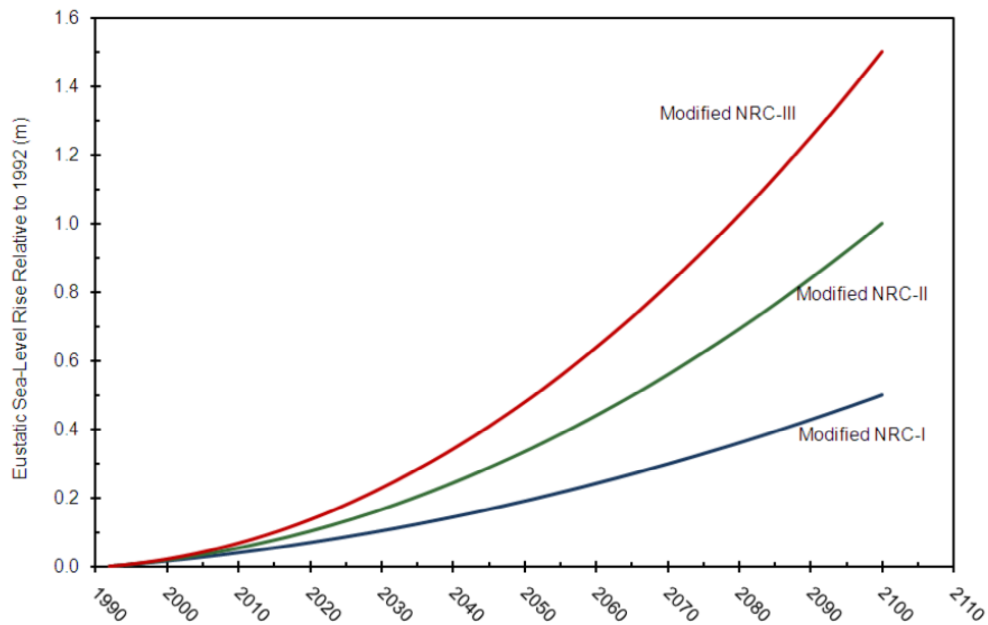

Figure 5 Scenarios for GMSL Rise (based on updates to NRC 1987 equation).

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

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

where t_1 is the time between the project's construction date and 1992 and t_2 is the time between a future date at which one wants an estimate for sea-level change and 1992 (or $t_2 = t_1$ + number of years after construction) . For the three scenarios proposed by the NRC, b is equal to 2.71E-5 for Curve 1, 7.00E-5 for Curve 2, and 1.13E-4 for curve 3.

This sea level rise was then added to a measured sea level trend for the Craig area. There is no sea level trend data for Craig or the Prince of Wales Island area. Guidance in Appendix C of Engineering Circular (EC) 1165-2-212 recommends that the next closest long term gage be used. NOAA has sea level trends published for Ketchikan, Alaska, which is the closest station to Craig. The sea level trend for Ketchikan is -0.007 inches/year. This value was used to obtain the values from the NRCS equation (Table 4). A plot of the values is shown in Figure 6

Table 4 Sea Level Rise Prediction for a 50 Year Project Life.

Sea Level Change	Low	Intermediate	High
	-.04 feet	0.43 feet	1.93 feet

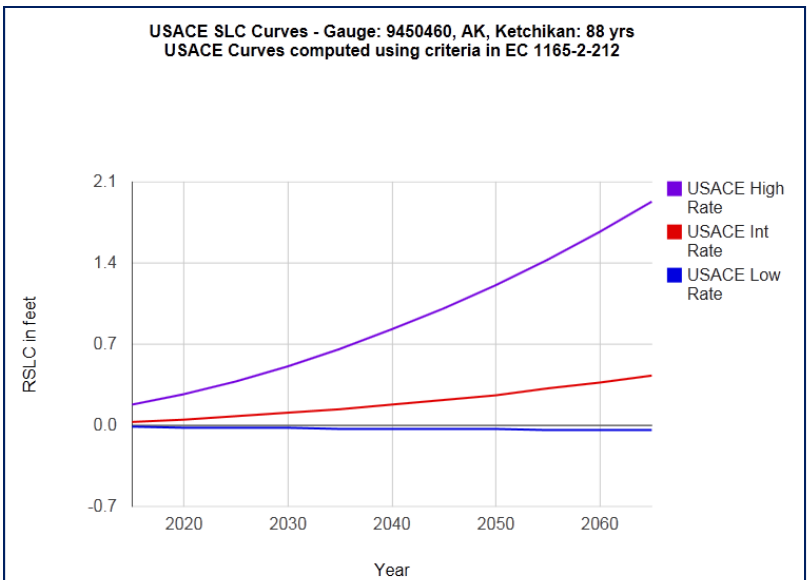

Figure 6 Plot of Sea Level Rise curves

For this study the low level of sea level change was used for calculations. For an assumed construction start in 2015 and a fifty year project life, a project at Craig could see sea level rise as little as -0.04 feet or much as 1.93 feet (Table 4). The design can be adapted to increase the breakwater height in the unlikely event that the High Level of Sea Level Change noted in Table 4 occurs. The proposed design can be modified by adding armor stone or a parapet wall to the breakwater crest to prevent overtopping during storm events.

2.6 Wind

The wind speeds presented in Table 5 were developed by Air Force Combat Climatology Center using historical wind speeds from the Five Finger Coastal-Marine Automated Network (C-MAN) at the Five Finger lighthouse (Figure 7). The Five Fingers data was used since it represented an unobstructed wind from the north. Wind speeds from the Klawok airport were available, but the airport appears to be sheltered from wind from the dominant fetch direction. Instead, north wind from the unobstructed C-MAN site was used. A wind generated southern wave would be minor at the site, so only north winds were evaluated for wave growth. According to the local residents, a southern swell from the Gulf of Alaska passes between Fish Egg Island and the proposed harbor site. The swell was considered for design purposes.

Figure 7 Location of C-MAN station used for wind data

Table 5 Wind Speed Extremal Analysis and Calculated Risk

One-Hour Sustained Wind (Knots)
EXTREME VALUE ANALYSIS
Five Finger AK Buoy - NORTH WIND
55.27 N 133.63 W
PERIOD OF RECORD: 1985-2013

QUANTILES	0.1	0.2	0.5	0.8	0.9	0.95	0.98	0.99	0.999	0.9999
RETURN PERIOD (YRS)	1.1	1.25	2	5	10	20	50	100	1000	10000
VARIATE 1 Hour Sustained Winds (Knots)	37.0	37.6	41.2	50.3	58.0	66.0	77.0	85.4	114.0	143.1

NOTE: The return period is the average elapsed time between occurrences of an event with a certain magnitude or greater.

2.7 Rivers and Creeks in the Project Vicinity

There are no rivers are creeks in the area of the proposed harbor.

2.8 Littoral Drift

Sediment transport has not been reported to be an issue in the area of the proposed harbor and visually does not appear to be an issue. The shore by the proposed harbor area is composed of gravel and does not show signs of movement. The area was previously used as a cannery and had a stable shoreline. Additionally, an existing rubble mound protected harbor south of the proposed harbor at Craig has not experienced infilling since its construction in 1982.

3.0 WAVE ANALYSIS

3.1 Wave Climate

The wave climate at Craig is generally moderate and is subject to short period wind generated waves from the northeast. Local residents have reported that these waves can reach a height of six feet. Long period swell from the Gulf of Alaska reaches the area from the southwest. Swell heights of up to two feet have been reported by the local residents.

3.2 Fetches

The coastline near Craig is oriented generally north east to south west. Fetches were calculated using the average length of nine radial lines at 3 degree spacing, extending from the harbor area to the shoreline. The radial lines used to determine the fetch are shown in Figure 8

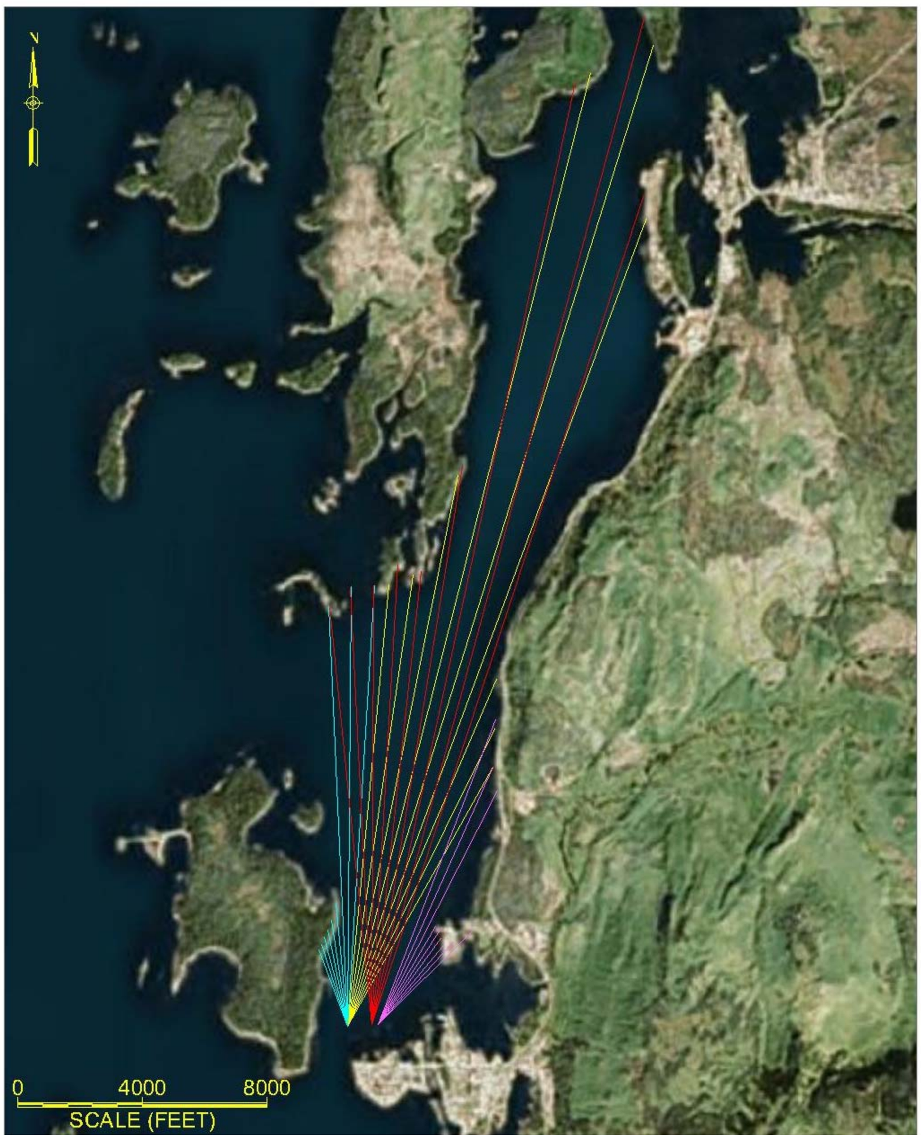

Figure 8 Fetches used in design

3.3 Wave Prediction

Methods described in the Coastal Engineering Manual (CEM), and the Automated Coastal Engineering System (ACES) program, were used to predict wave heights. The design wave was calculated as an average of the results of the two methods. The CEM equations and ACES program predict wave heights based on fetch distances and wind speeds. The fetch distance and wind speed were also used to determine if the wave condition is limited by the fetch length or by the duration of the wind.

The 72.6 year return interval wind was used to determine the design storm wave corresponding to a 50 year design life with a 50% probability of being equaled or exceeded (
Figure 9). The design wave from the northeast is 3.3 feet with a period of 3.0 seconds. The design wave from the north-northeast is 6.6 feet with a period of 4.3 seconds. The design wave from the northwest is 3.3 feet with a period of 2.5 seconds. The wave heights calculated represent the significant wave height, H_s which is the average height of the one-third highest waves of a given group. The design waves are non breaking in depths greater than 8 feet. The design wave correlates well with what long-time residents have seen during extreme storm events from the north east at Craig. The residents also reported a two foot swell that comes from the south between Fish Egg Island and Craig. The 6.6 foot wave will be used as a design wave for the breakwater design.

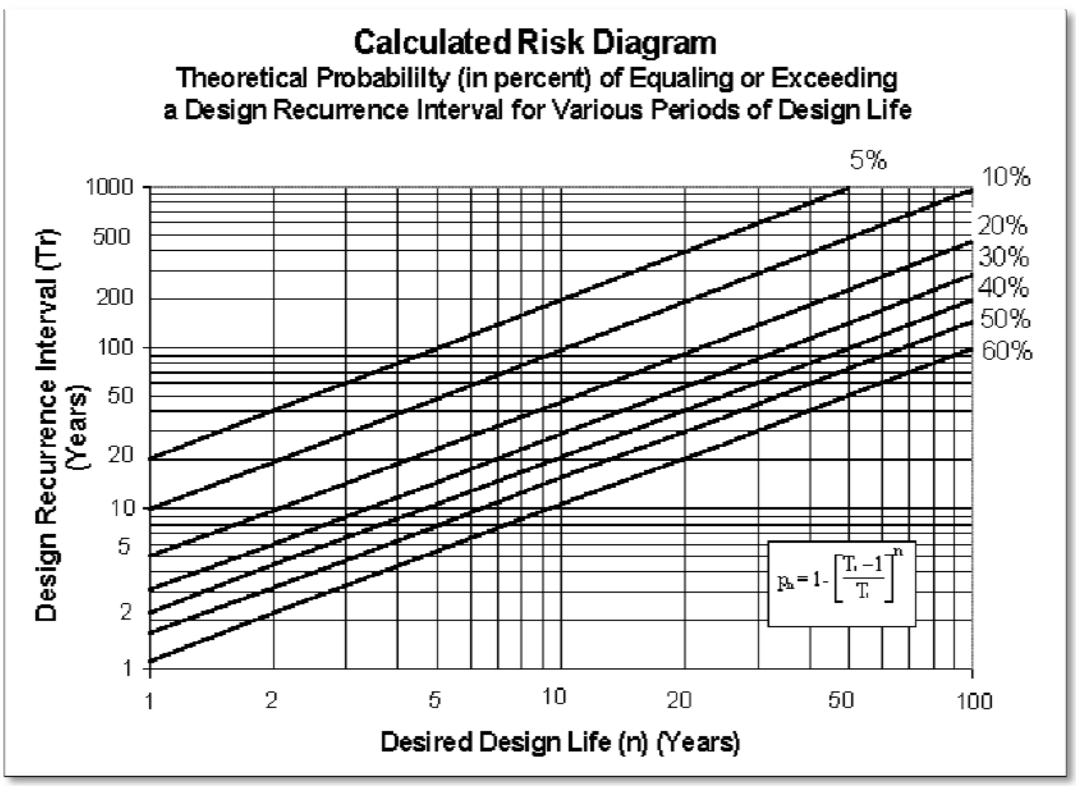

Figure 9 Calculated Risk Diagram

4.0 DESIGN CRITERIA

4.1 Design Vessel and Fleet

The economic analysis generated the vessel demand for this study. The characteristics of the fleet proposed to occupy the various alternatives are shown in Table 6. Proposed harbor plans were laid out to accommodate the identified vessels. The design vessel is 60 feet long with a beam of 18 feet.

Table 6 Fleet Characteristics

Vessel Length [ft]	Design Beam [ft]	Design Draft [ft]
20	10	2.5
28	10	3.5
36	14	5
46	16	5.5
60	18	7

4.2 Entrance Channel and Maneuvering Area

The entrance channel width was determined by criteria given in EM-1110-2-1613. For a two way ship channel with currents between 0.5 to1.5 knots, the width should be 6 times the beam of the design ship. This would be 108 feet. The harbor is open on the eastern side for all of the alternatives, which provides adequate clearance for all boats to exit and return using the two way traffic design criteria.

The maneuvering areas and the fairway widths were designed so that there would be enough room for vessels to turn and dock. Width for turning was determined using a minimum of 1.5 times the length of the largest vessel using the finger piers in that area of the basin.

4.3 Entrance Channel Depth

The entrance depth was checked against the criteria listed in Table 7. Vessels were assumed to be unloaded when entering the harbor, so unloaded drafts were used to calculate the required depths for the entrance and mooring basin depth requirements. The lowest astronomical tide is -2.95 feet MLLW. When this is added to the total required depth noted in Table 7 results in a depth of -12.95 feet MLLW which is usable 100% of the time. The existing bathymetry in the entrance area and maneuvering channel ranges from -20 feet to -45 feet. This provides adequate depth needed for the entrance 100% of the time without the need for dredging.

Table 7 Entrance Channel Criteria

Vessel Draft [ft]	7.0
Pitch, Roll, Heave [ft]	0.5
Squat [ft]	0.5
Tide Allowance [ft]	
Safety Clearance	2.0
Total depth required [ft]	10

5.0 NAVIGATION IMPROVEMENT OPTIONS

Options considered for vessel protection during launching and landing include:
- Floating Breakwater
- Rubblemound breakwater

Floating Breakwater
A floating breakwater consists of a floating structure that can provide wave protection for short period waves with heights up to 4 feet. A floating breakwater is anchored with chain or piles. Because the design wave at Craig is greater than 4 feet, a floating breakwater was dropped from further consideration.

Rubble mound Breakwater
The use of a rubble mound breakwater to provide wave protection is a proven concept. Rubble mound breakwaters have been successfully used in southeast Alaska. Because rubble mound breakwaters have a proven history in similar environments, the decision was made to pursue a rubble mound breakwater option.

6.0 DESIGN PARAMETERS

6.1 Armor Stone

Using Hudson's equation for a wave of 6.6 feet from the north northeast and a K_d of 4 results in an average armor stone size of 2,012 pounds. Typical breakwater cross sections are shown in Figure 10.

6.2 Crest Height

The crest height was set at 18 feet using ACES and equation VI-5-13 in the Coastal Engineering Manual to determine run-up. The mean higher high water level of 10.17 feet was used as the still water level. Storm surge was not included in the calculations since storm surge in not typically an issue at Craig. The crest width was set at 7.0 feet based on armor stone size.

6.3 Water Quality and Circulation

The circulation in the small harbors was evaluated against recommendations outlined in *Planning and Design Guidelines for Small Craft Harbors,* (ASCE Task Committee on Marinas 2020).

The tidal prism ratio (TPR) is the volume of water entering the basin during the flood tide compared with the total basin volume at high tide. For good flushing the TPR needs to be at least 0.25 and preferably 0.35.

The aspect ratio is a measure of the length divided by the width of the basin. The aspect ratio should normally be close to unity for peak flushing efficiency. The maximum aspect ratio for basin should be 1:4. Such geometry will minimize possible zones of stagnation and short-circuiting of circulation cells within the basin.

The area ratio (AR) is the ratio of the basin area to channel cross sectional area (A/a). The size of the fleet and mooring density determines the basin size (A) and the vessel draft, beam, wave conditions, and tides determine the channel cross-section (a). A large A/a value (greater than 200) is preferred. The entire east side of the harbor is open for each of the harbor configurations at Craig and as a result there is no entrance channel, so this parameter is not appropriate to use for circulation evaluation.

6.4 Dredge Material

Dredging will not be required for the alternatives considered.

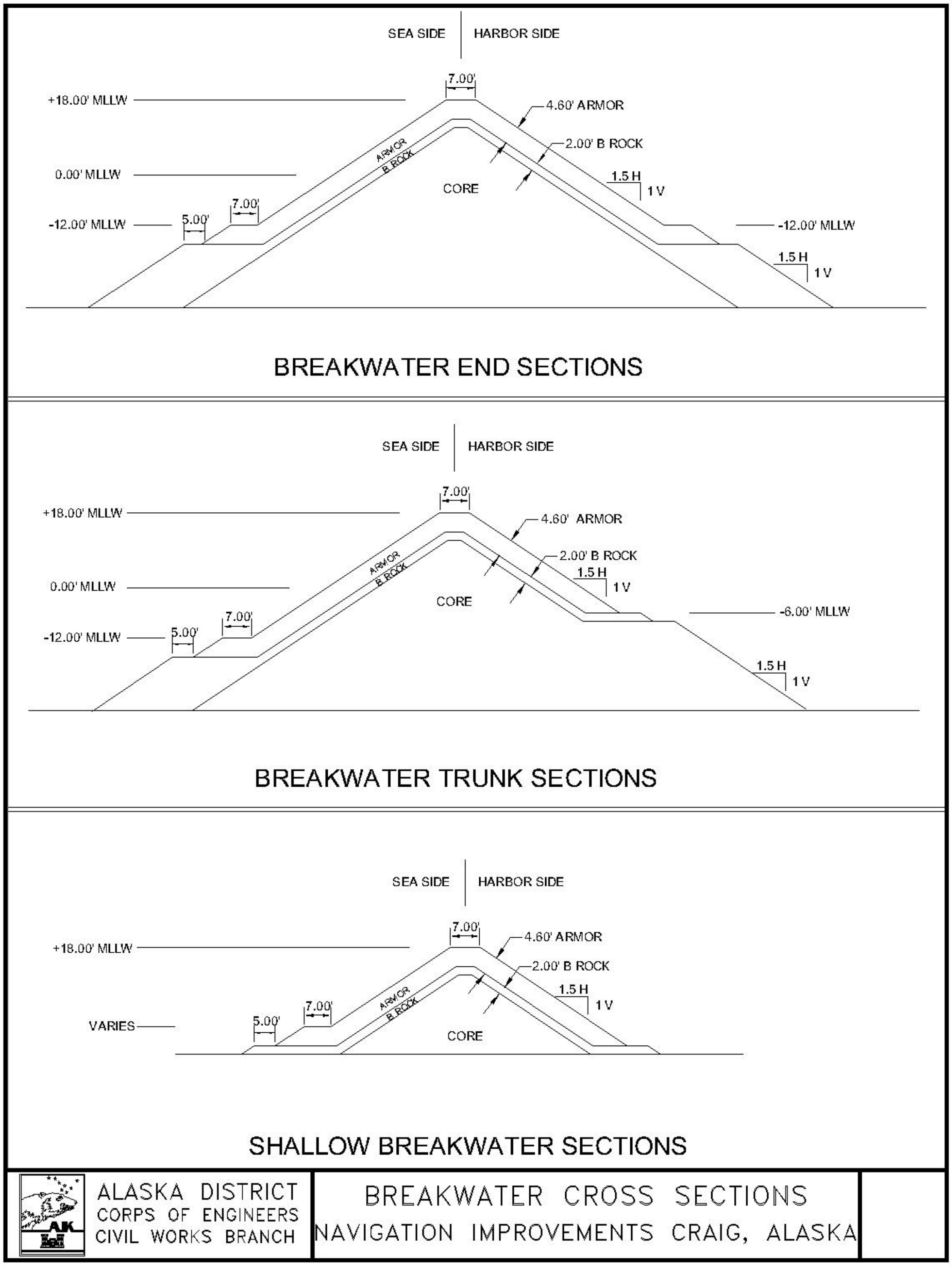

Figure 10 Typical cross sections

7.0 ALTERNATIVES CONSIDERED IN DETAIL

The site for the navigation improvements at Craig was selected during the charrette process at the beginning of the project. Working with the chosen site, several alternatives were considered for navigation improvements. Six plans were evaluated along with a no action alternative.

7.1 No Action

This alternative would leave the community without an additional harbor. Vessels will continue to sustain time lost and damages as they would continue to raft up in the existing harbors. Damages associated with the rafting would continue to occur.

7.7 Alternative 1 – Smaller Basin with Fish Passage

This plan consists of one 1,462 foot, and one 318 foot rubble mound breakwater that would provide an 8 acre basin for 105 boats ranging in from 20 feet to 120 feet. This plan would provide shelter from north storm waves, south swell, and wakes from boats travelling between Fish Egg Island and Craig. This alternative would not impact the area where float planes currently land and take off (Figure 11).

Breakwaters. Stone size and crest elevation are described in Section 6.0 DESIGN PARAMETERS. The breakwater would require approximately 181,000 cubic yards of core rock, 37,600 cubic yards of B rock, and 31,400 cubic yards of armor stone. Typical breakwater cross sections are shown in Figure 10

Shoaling. No shoaling in the entrance is anticipated due to the material type observed on shore and the lack of shoaling experienced by the other harbors at Craig.

Wave Reduction. Diffraction analysis was used to determine the wave height expected for this alternative (Figure 12). The maximum wave height in the proposed basin was calculated to be one foot or less in the mooring area. All directions of wave exposure were taken into account, and the largest wave heights in the basin were generated from the incident wave from the northeast direction.

Circulation. The TPR for alternative 1 is 0.3 which is considered good. The aspect ratio for alternative 1 is 1:2.2, which is below the maximum recommendation of 1:4.

Maintenance. It is not anticipated that there will be a significant loss of stone from the structure over the life of the project. It is estimated that approximately 1,570 cubic yards of armor stone will need to be replaced every 20 years.

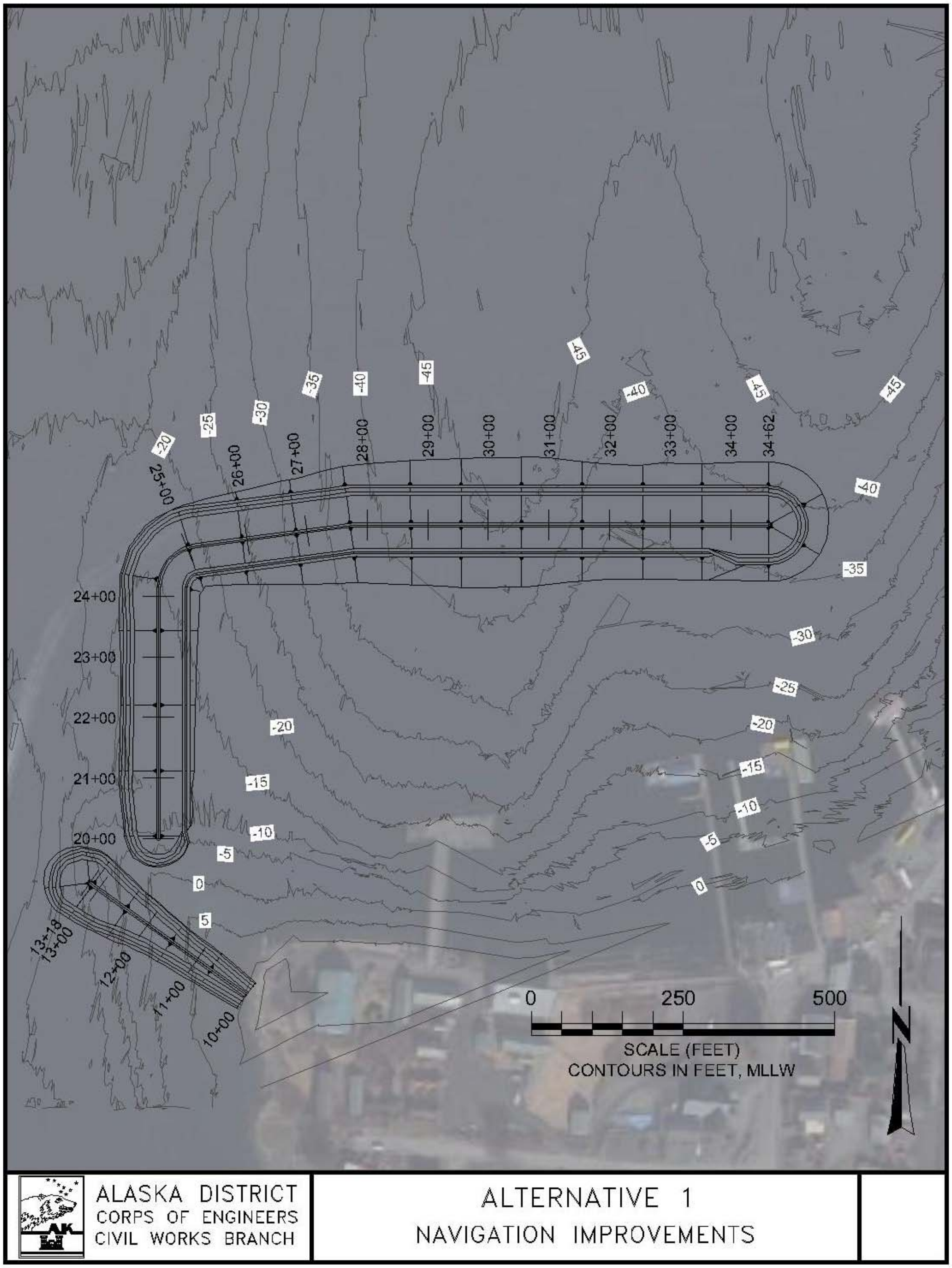

Figure 11 Plan view of Alternative 1

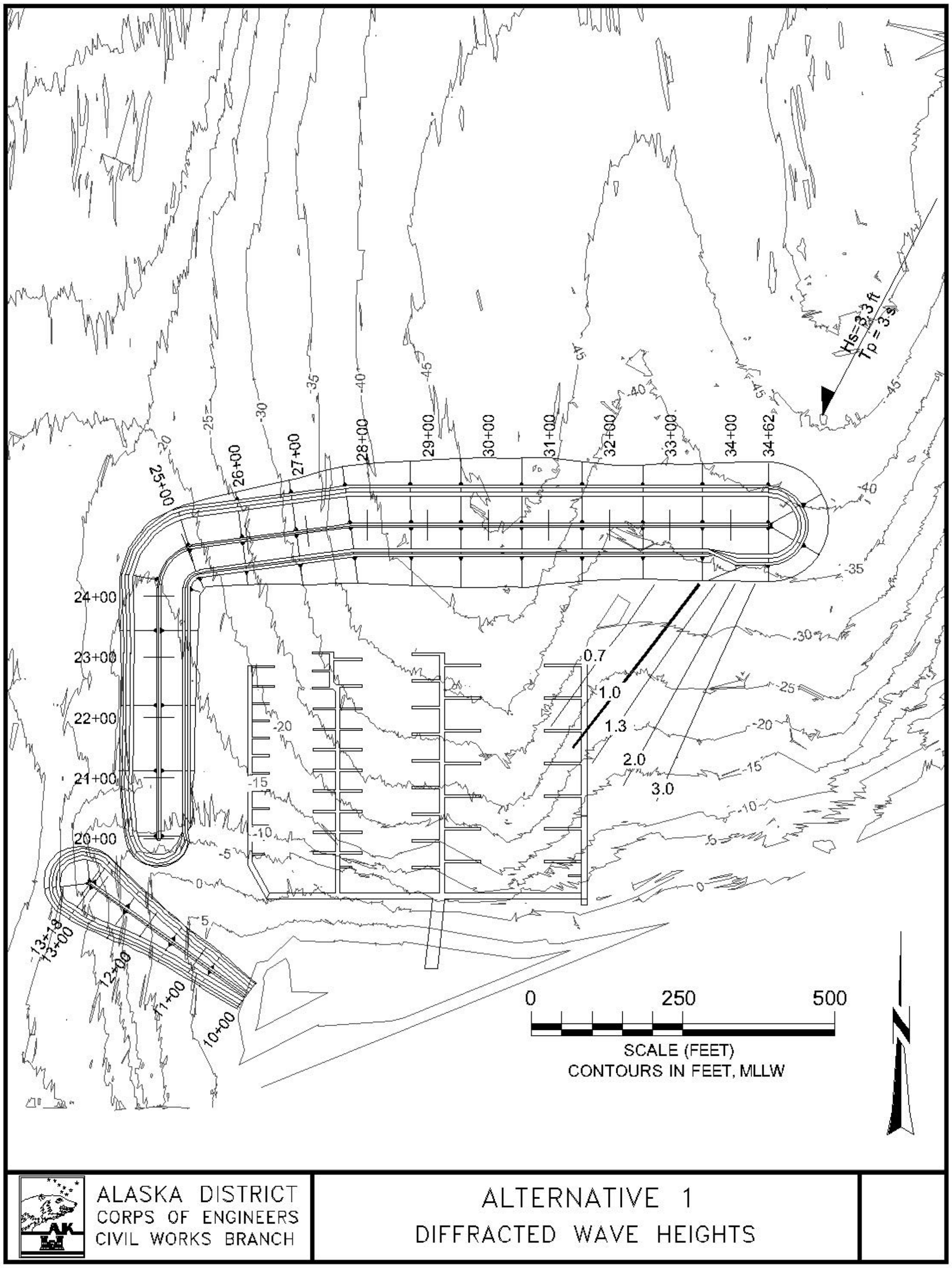

Figure 12 Diffracted wave heights for Alternative 1. Note that harbor floats are shown for illustrative purposes only. Float construction is responsibility of local sponsor.

7.2 Alternative 2 – Small Basin

This plan consists of one 650 foot and one 850 foot rubble mound breakwater that would provide a 10 acre basin for 145 boats ranging in from 20 feet to 120 feet. This alternative would provide shelter from north storm waves and wakes from boats travelling between Fish Egg Island and Craig. This alternative would not provide adequate protection from the two foot swell noted by the local residents. This alternative allows wave heights in the basin to exceed one foot during extreme events. The east float would need to be over built to withstand waves greater than one foot. Because of the lack of harbor protection, this alternative was dropped from further consideration (Figure 13).

Breakwaters. Stone size and crest elevation are described in Section 6.0 DESIGN PARAMETERS. The two breakwaters would require approximately 156,500 cubic yards of core rock, 34,500 cubic yards of B rock, and 27,000 cubic yards of armor stone.

Shoaling. No shoaling in the entrance is anticipated due to the material type observed on shore and the lack of shoaling experienced by the other harbors at Craig.

Wave Reduction. All directions of wave exposure were taken into account, and the largest wave heights in the basin were generated from the incident wave from the north –northeast direction. Diffraction analysis was used to determine the wave height expected for this alternative (Figure 14). The maximum wave height in the proposed basin would be greater than one foot.

Circulation. The TPR for this alternative is 0.3 which is considered good. The aspect ratio is 1:1, which is below the maximum recommendation of 1:4.

Maintenance. It is not anticipated that there will be a significant loss of stone from the structure over the life of the project. It is estimated that approximately 1,350 cubic yards of armor stone will need to be replaced every 20 years.

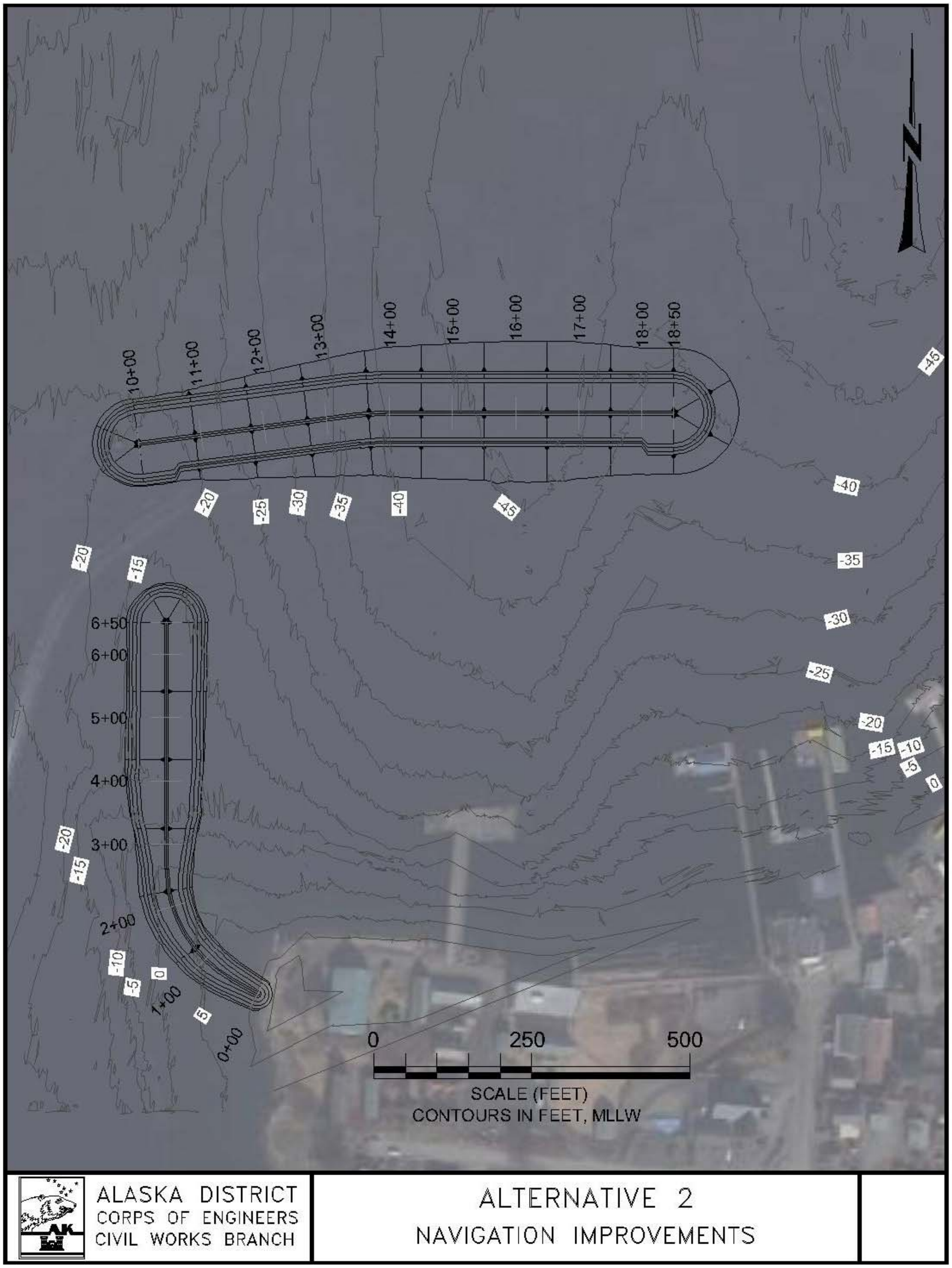

Figure 13 Plan view of Alternative 2

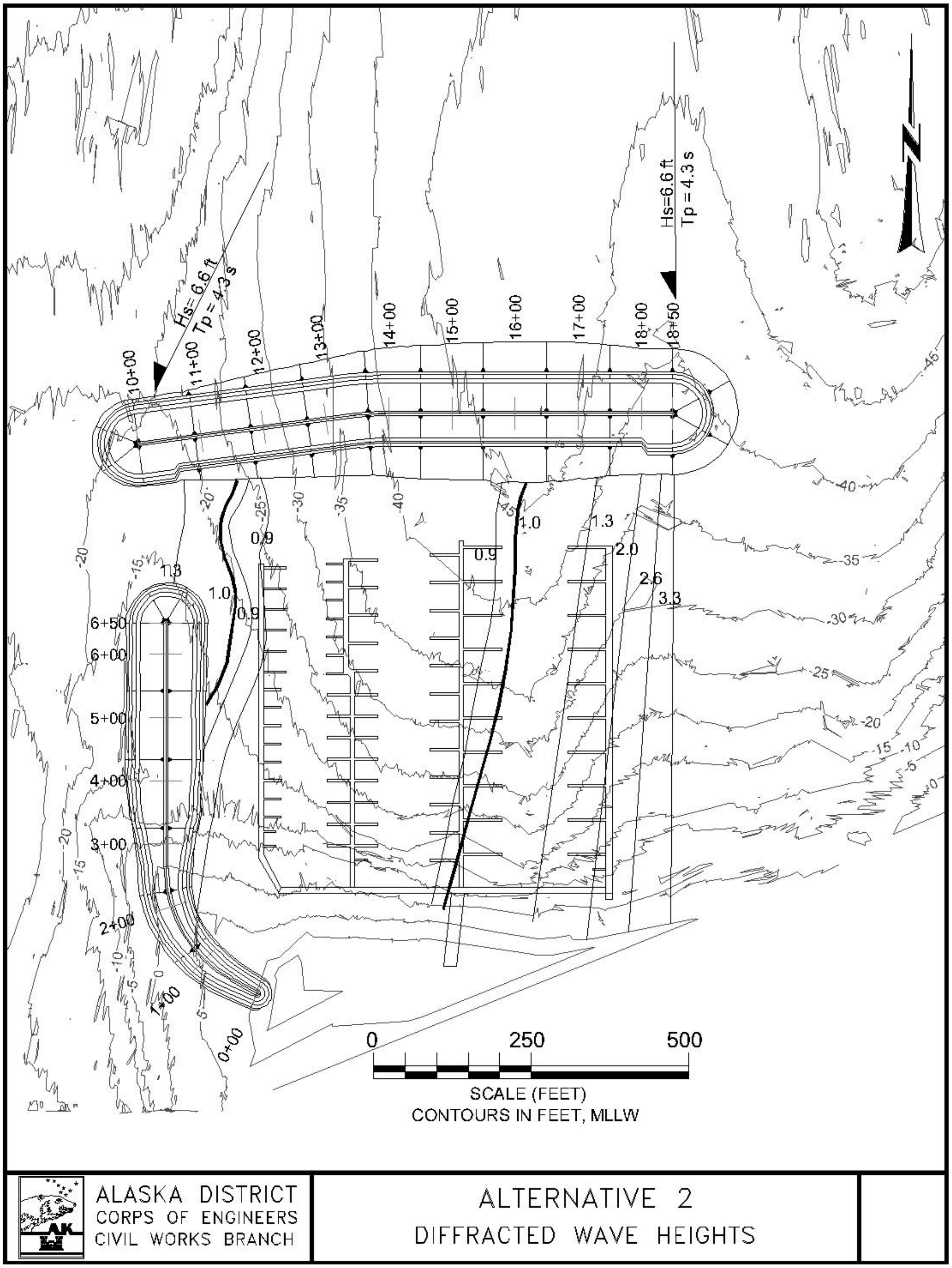

Figure 14 Diffracted wave heights for Alternative 2. Note that harbor floats are shown for illustrative purposes only. Float construction is responsibility of local sponsor.

7.5 Alternative 2a – Small Basin with Two Entrances

This plan consists of one 956 foot, and one 957 foot rubble mound breakwater that would provide a 10 acre basin for 145 boats ranging in from 20 feet to 120 feet. This alternative would provide shelter from north storm waves, south swell, and wakes from boats travelling between Fish Egg Island and Craig. This alternative would not impact the area where float planes currently land and take off (Figure 15).

Breakwaters. Stone size and crest elevation are described in Section 6.0 DESIGN PARAMETERS. The two breakwaters would require approximately 220,000 cubic yards of core rock, 47,500 cubic yards of B rock, and 35,500 cubic yards of armor stone. Typical breakwater cross sections are shown in Figure 10

Shoaling. No shoaling in the entrance is anticipated due to the material type observed on shore and the lack of shoaling experienced by the other harbors at Craig.

Wave Reduction. Diffraction analysis was used to determine the wave height expected for this alternative (Figure 16 and Figure 17). All directions of wave exposure were taken into account, and the largest wave heights in the basin were generated from the incident wave from the north –northeast direction. Diffraction analysis was used to determine the wave height expected for this alternative (Figure 23). The maximum wave height in the proposed basin was would be greater than one foot.

Circulation. The TPR for alternative 2a is 0.3 which is considered good. The aspect ratio for alternative 2a is 1:1.5, which is below the maximum recommendation of 1:4.

Maintenance. It is not anticipated that there will be a significant loss of stone from the structure over the life of the project. It is estimated that approximately 1,775 cubic yards of armor stone will need to be replaced every 20 years.

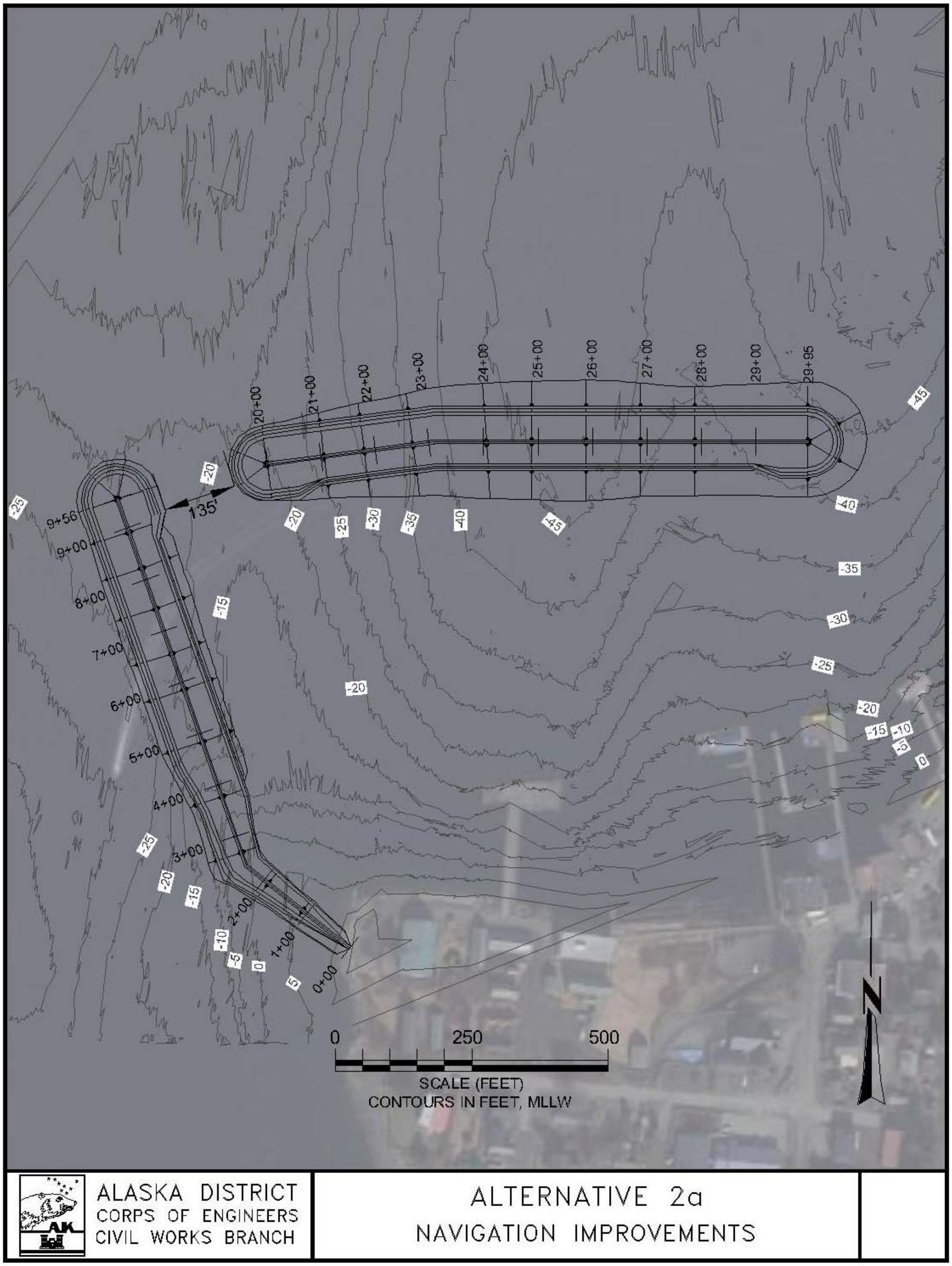

Figure 15 Plan view of Alternative 2a

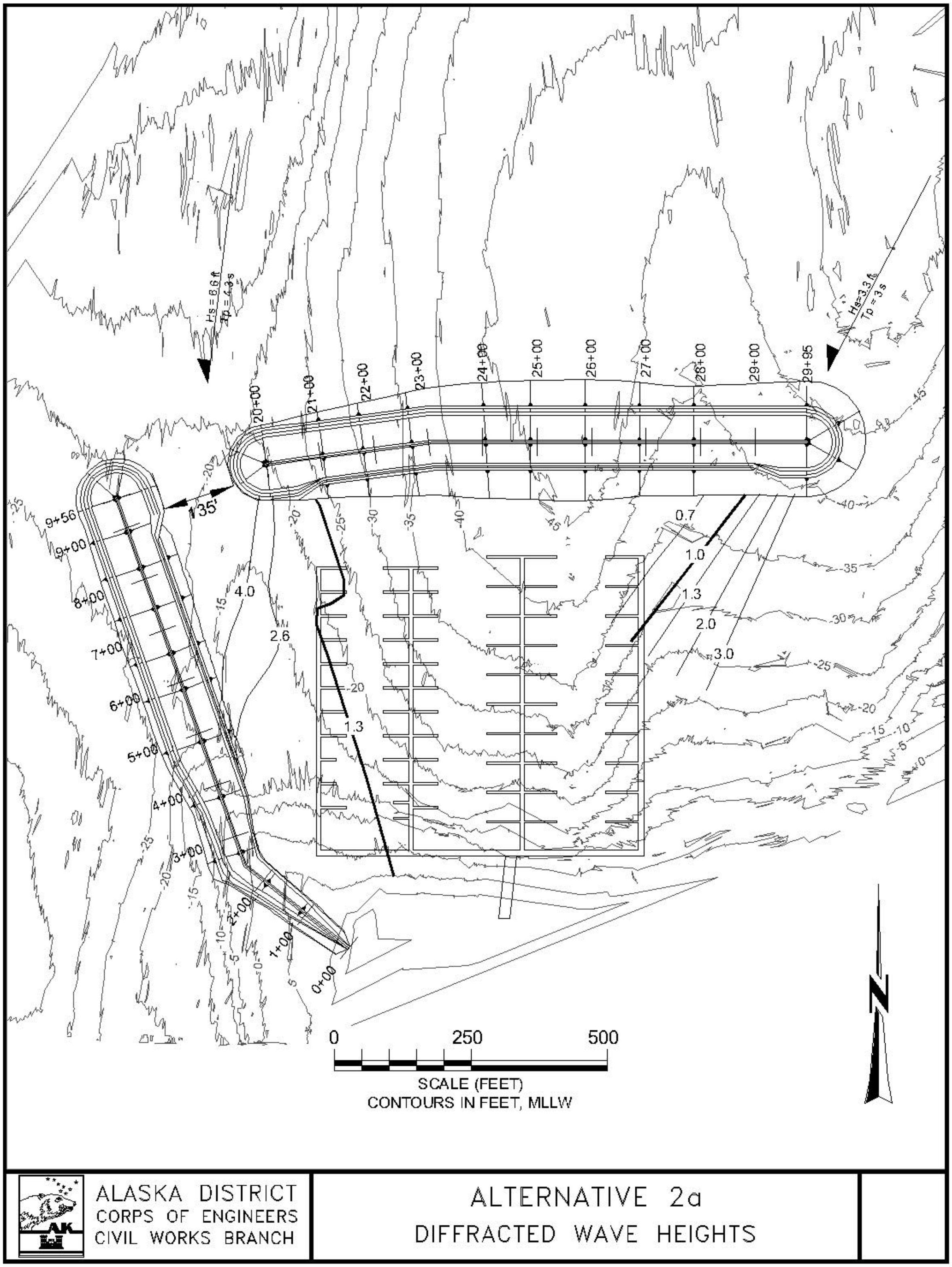

Figure 16 Diffracted wave heights for Alternative 2a. Note that harbor floats are shown for illustrative purposes only. Float construction is responsibility of local sponsor.

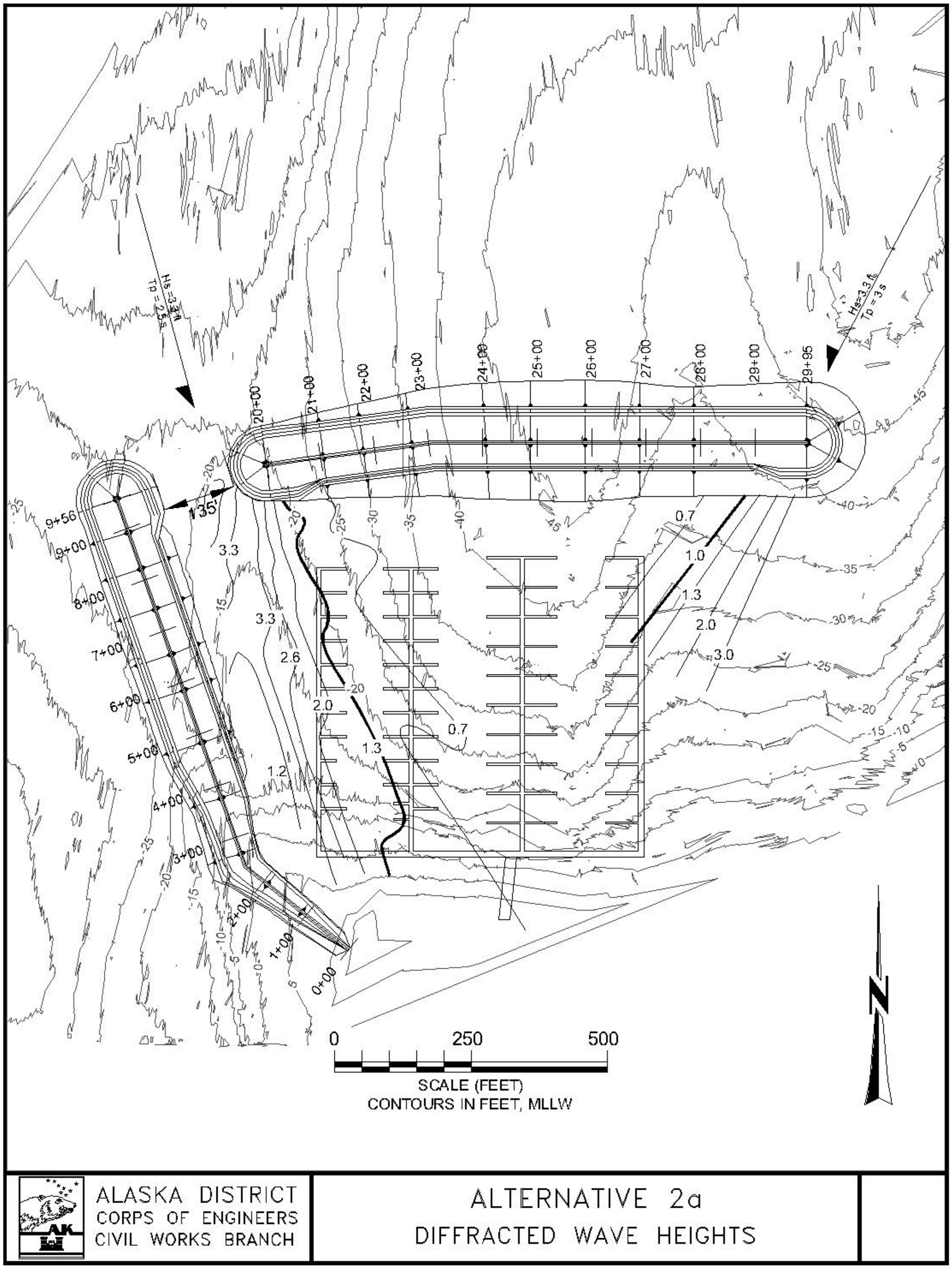

Figure 17 Diffracted wave heights for Alternative 2a. Note that harbor floats are shown for illustrative purposes only. Float construction is responsibility of local sponsor.

7.6 Alternative 2b – Small Basin with Fish Passage

This plan consists of one 1,606 foot, and one 318 foot rubble mound breakwater that would provide a 10 acre basin for 145 boats ranging in from 20 feet to 120 feet. This plan would provide shelter from north storm waves, south swell, and wakes from boats travelling between Fish Egg Island and Craig. This alternative would not impact the area where float planes currently land and take off (Figure 18).

Breakwaters. Stone size and crest elevation are described in Section 6.0 DESIGN PARAMETERS. The breakwater would require approximately 205,500 cubic yards of core rock, 43,000 cubic yards of B rock, and 31,500 cubic yards of armor stone. Typical breakwater cross sections are shown in Figure 10

Shoaling. No shoaling in the entrance is anticipated due to the material type observed on shore and the lack of shoaling experienced by the other harbors at Craig.

Wave Reduction. Diffraction analysis was used to determine the wave height expected for this alternative (Figure 19). The maximum wave height in the proposed basin was calculated to be one foot or less in the mooring area. All directions of wave exposure were taken into account, and the largest wave heights in the basin were generated from the incident wave from the northeast direction.

Circulation. The TPR for alternative 2b is 0.3 which is considered good. The aspect ratio for alternative 2b is 1:15, which is below the maximum recommendation of 1:4.

Maintenance. It is not anticipated that there will be a significant loss of stone from the structure over the life of the project. It is estimated that approximately 1,575 cubic yards of armor stone will need to be replaced every 20 years.

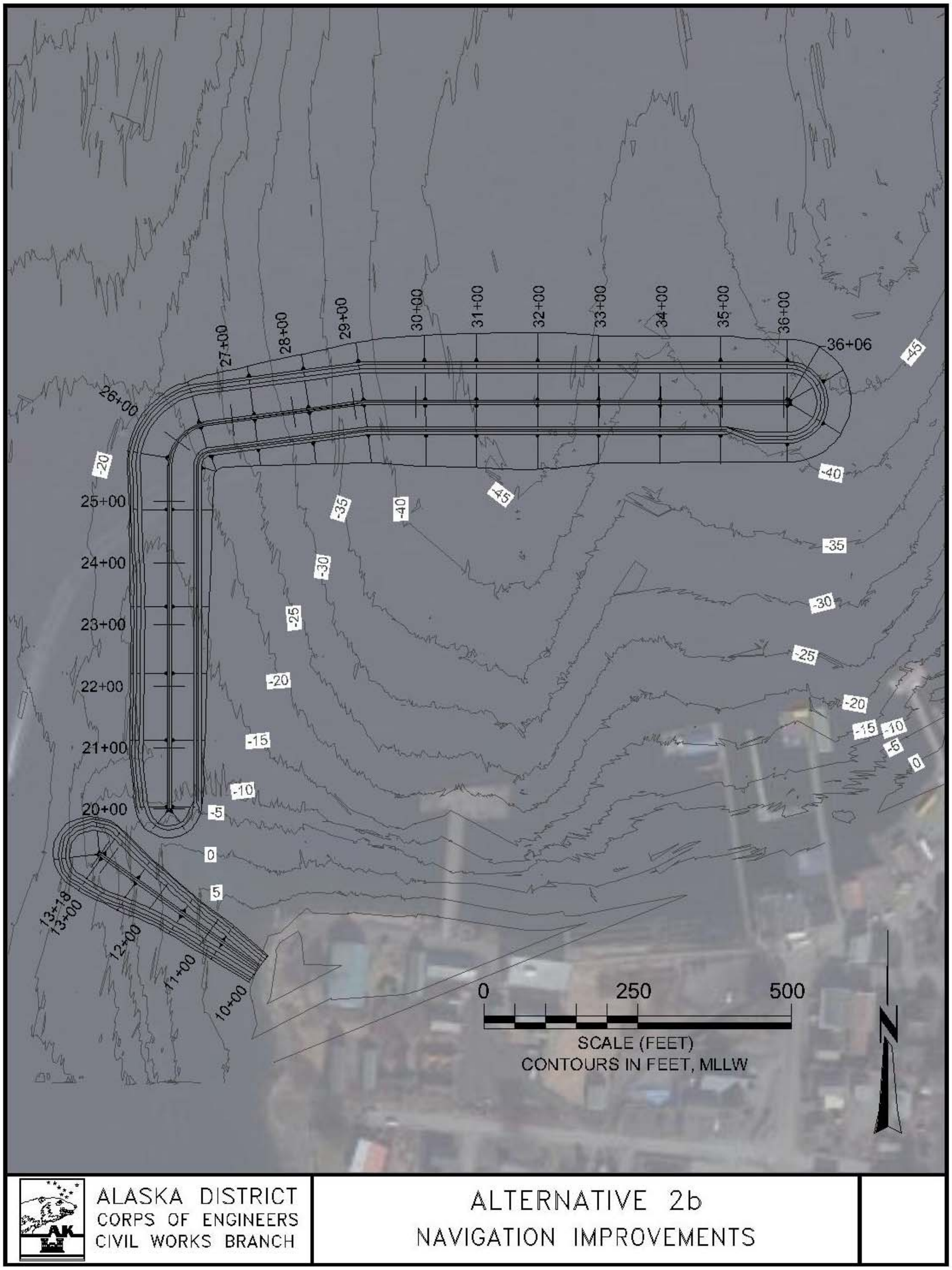

Figure 18 Plan view of Alternative 2b

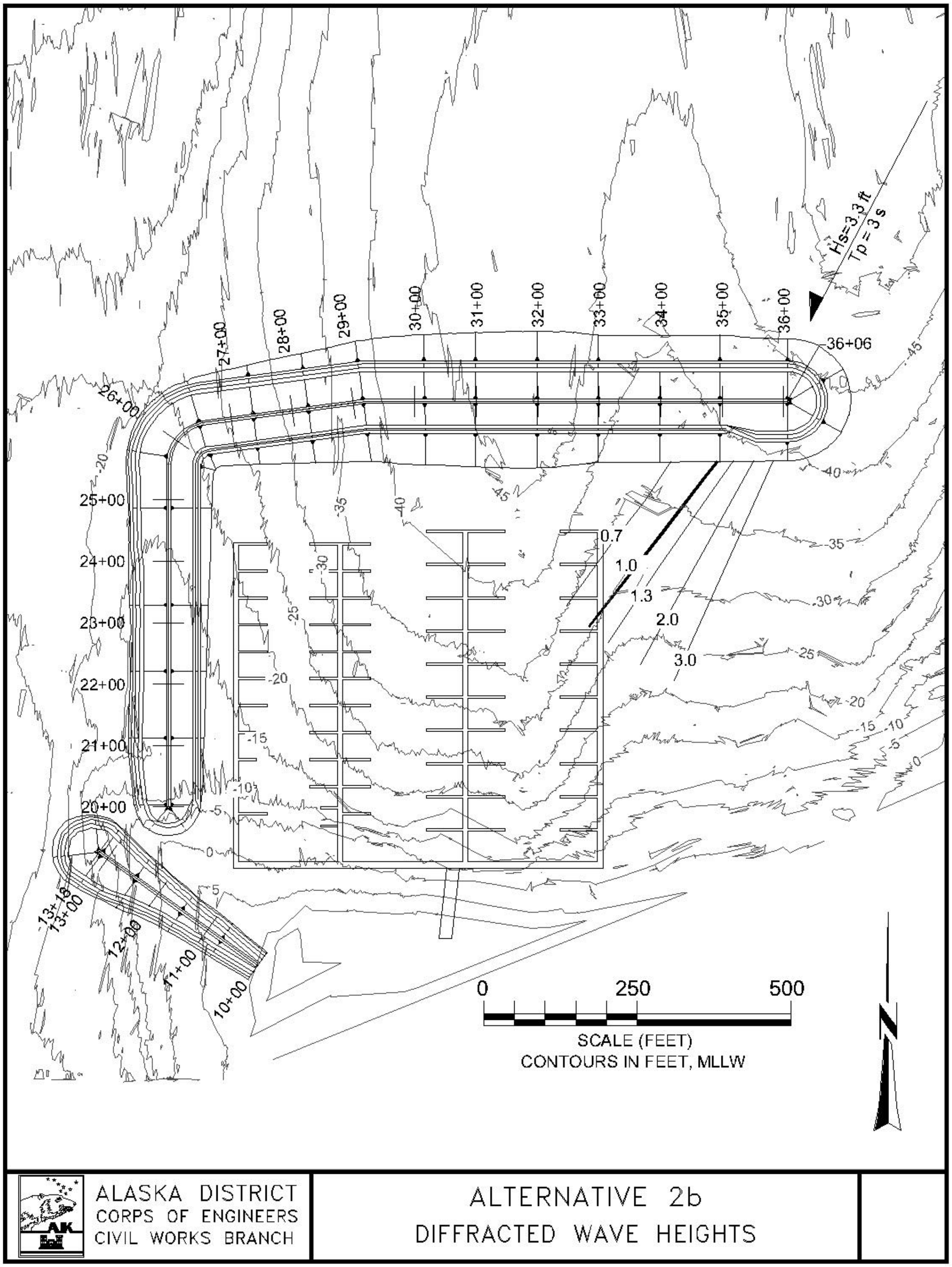

Figure 19 Diffracted wave heights for Alternative 2b. Note that harbor floats are shown for illustrative purposes only. Float construction is responsibility of local sponsor.

7.3 Alternative 3 – Medium Size Basin

This plan consists of one 650 foot and one 1,450 foot rubble mound breakwater that would provide a 25 acre basin for 303 boats ranging in from 20 feet to 120 feet. This alternative would provide shelter from north storm waves, but would not provide adequate protection from the two foot swell noted by the local residents. This alternative allows wave heights in the basin to exceed one foot during extreme events. The outer east floats would need to be over built to withstand waves greater than one foot. This alternative would also impact the area where float planes currently land and take off. Because of the lack of harbor protection and the impact on float plane traffic, this alternative was dropped from further consideration (Figure 20).

Breakwaters. Stone size and crest elevation are described in Section 6.0 DESIGN PARAMETERS. The two breakwaters would require approximately 310,500 cubic yards of core rock, 55,000 cubic yards of B rock, and 36,500 cubic yards of armor stone. Typical breakwater cross sections are shown in Figure 10

Shoaling. No shoaling in the entrance is anticipated due to the material type observed on shore and the lack of shoaling experienced by the other harbors at Craig.

Wave Reduction. All directions of wave exposure were taken into account, and the largest wave heights in the basin were generated from the incident wave from the north –northeast direction. Diffraction analysis was used to determine the wave height expected for this alternative (Figure 21). The maximum wave height in the proposed basin would be greater than one foot.

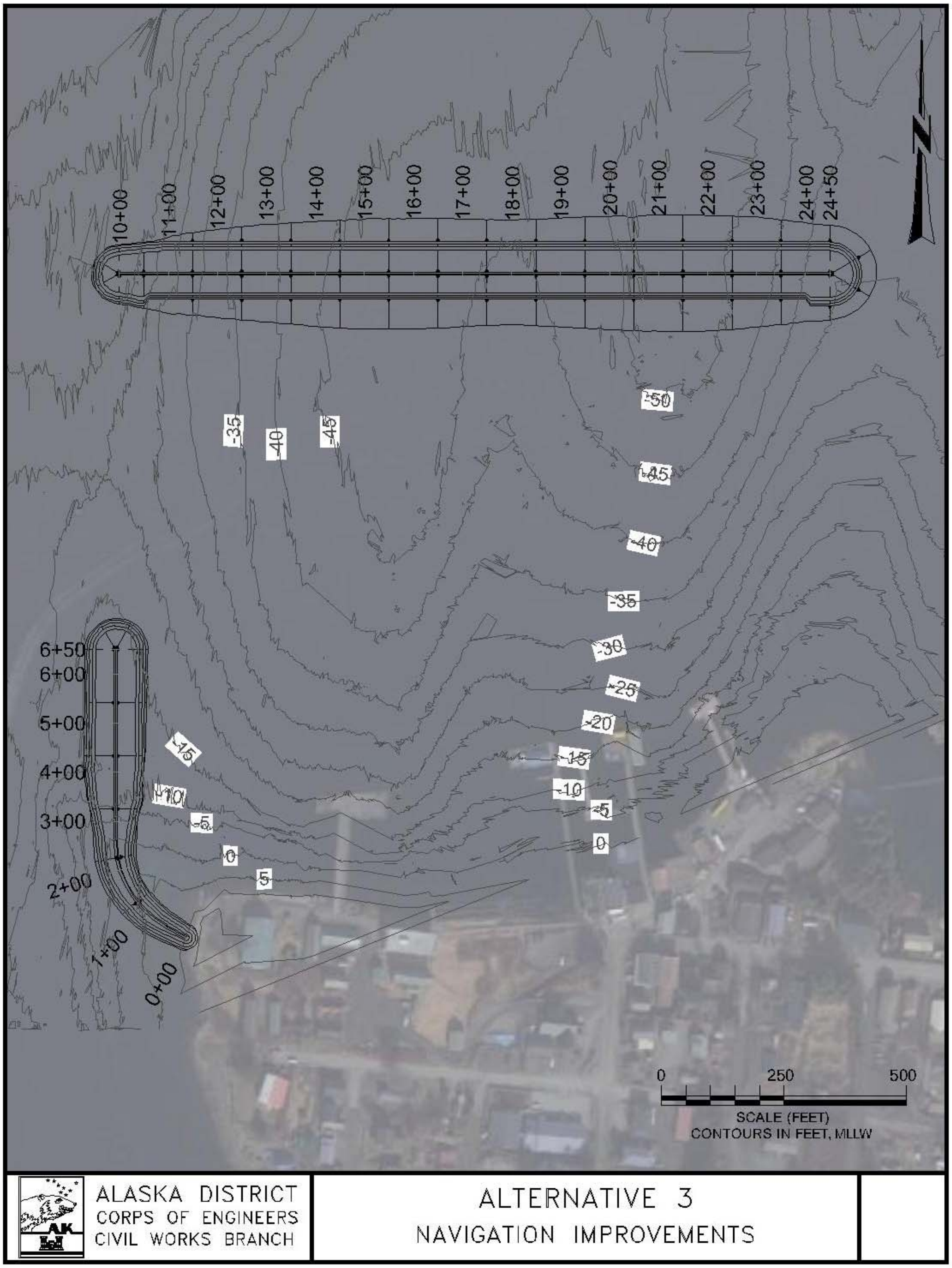

Figure 20 Plan view of Alternative 3

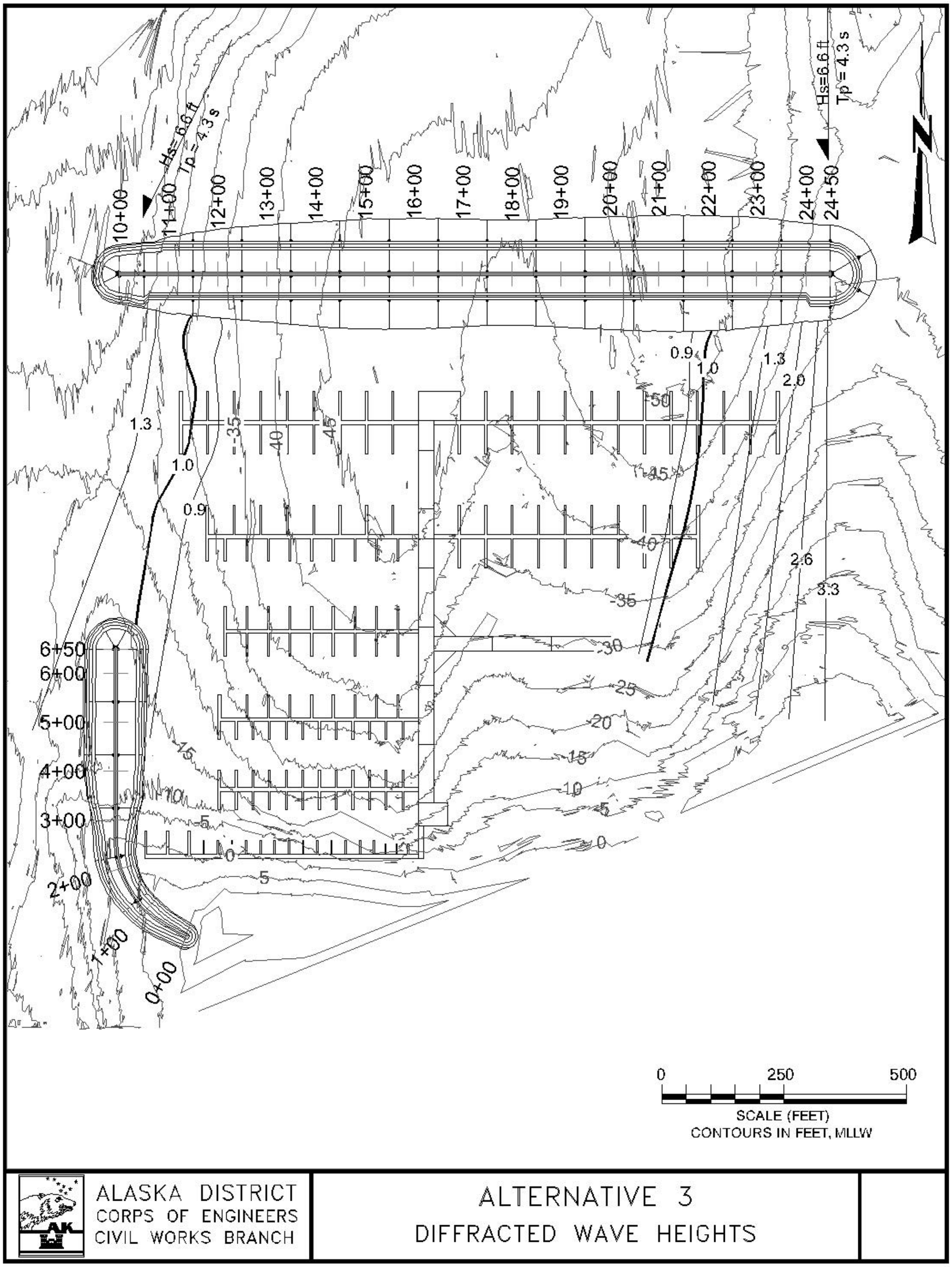

Figure 21 Diffracted wave heights for Alternative 3. Note that harbor floats are shown for illustrative purposes only. Float construction is responsibility of local sponsor.

7.4 Alternative 4 – Large Basin

This plan consists of one 650 foot and one 1,600 foot rubble mound breakwater that would provide a 42 acre basin for 530 boats ranging in from 20 feet to 120 feet. This alternative would provide shelter from north storm waves, but would not provide adequate protection from the two foot swell noted by the local residents. This alternative allows wave heights in the basin to exceed one foot during extreme events. The outer east floats would need to be over built to withstand waves greater than one foot. This alternative would also impact the area where float planes currently land and take off. Because of the lack of harbor protection and the impact on float plane traffic, this alternative was dropped from further consideration (Figure 22).

Breakwaters. Stone size and crest elevation are described in Section 6.0 DESIGN PARAMETERS. The two breakwaters would require approximately 313,500 cubic yards of core rock, 55,000 cubic yards of B rock, and 36,500 cubic yards of armor stone. Typical breakwater cross sections are shown in Figure 10

Shoaling. No shoaling in the entrance is anticipated due to the material type observed on shore and the lack of shoaling experienced by the other harbors at Craig.

Wave Reduction. All directions of wave exposure were taken into account, and the largest wave heights in the basin were generated from the incident wave from the north –northeast direction. Diffraction analysis was used to determine the wave height expected for this alternative (Figure 23). The maximum wave height in the proposed basin was would be greater than one foot.

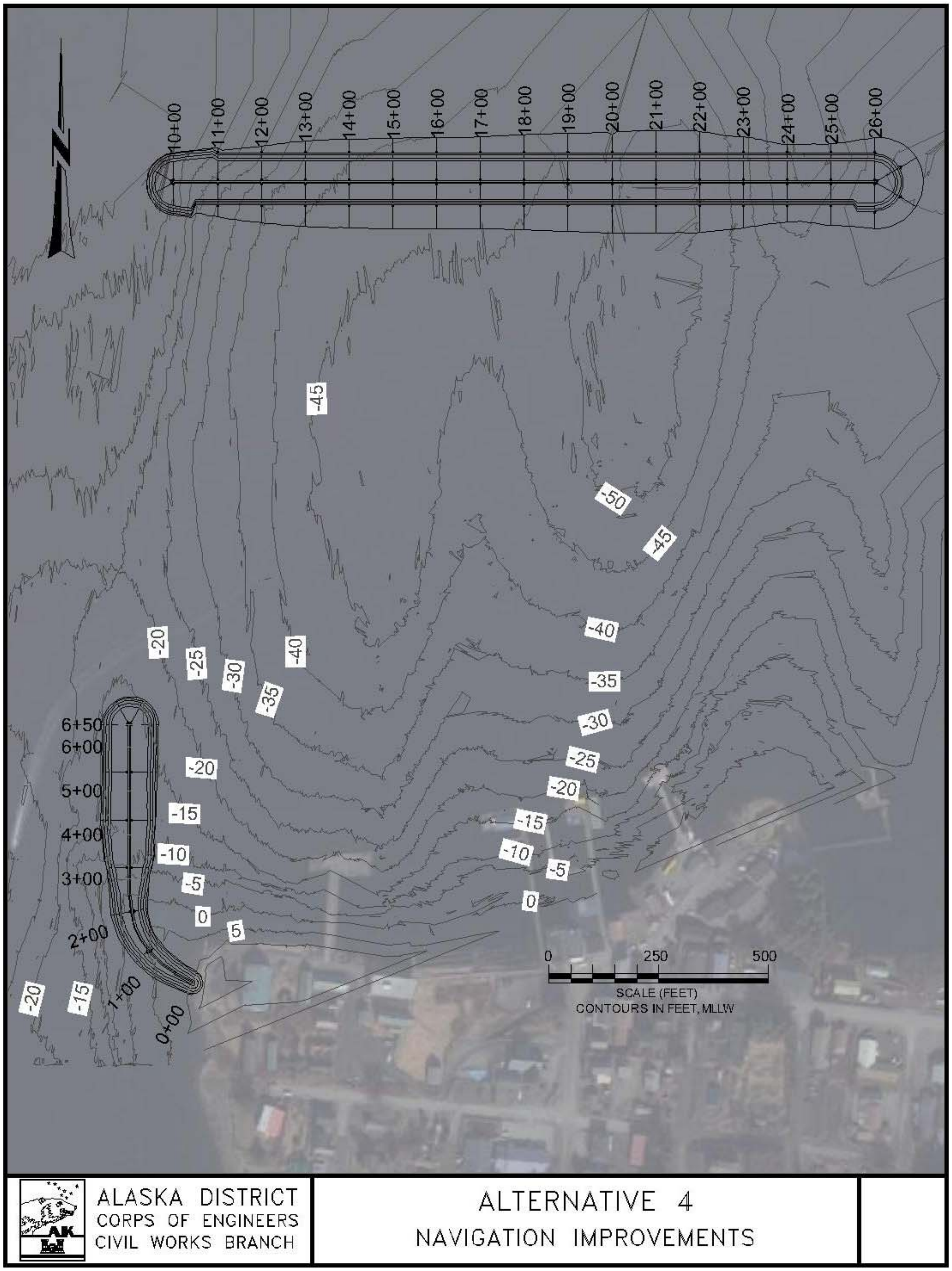

Figure 22 Plan view of Alternative 4

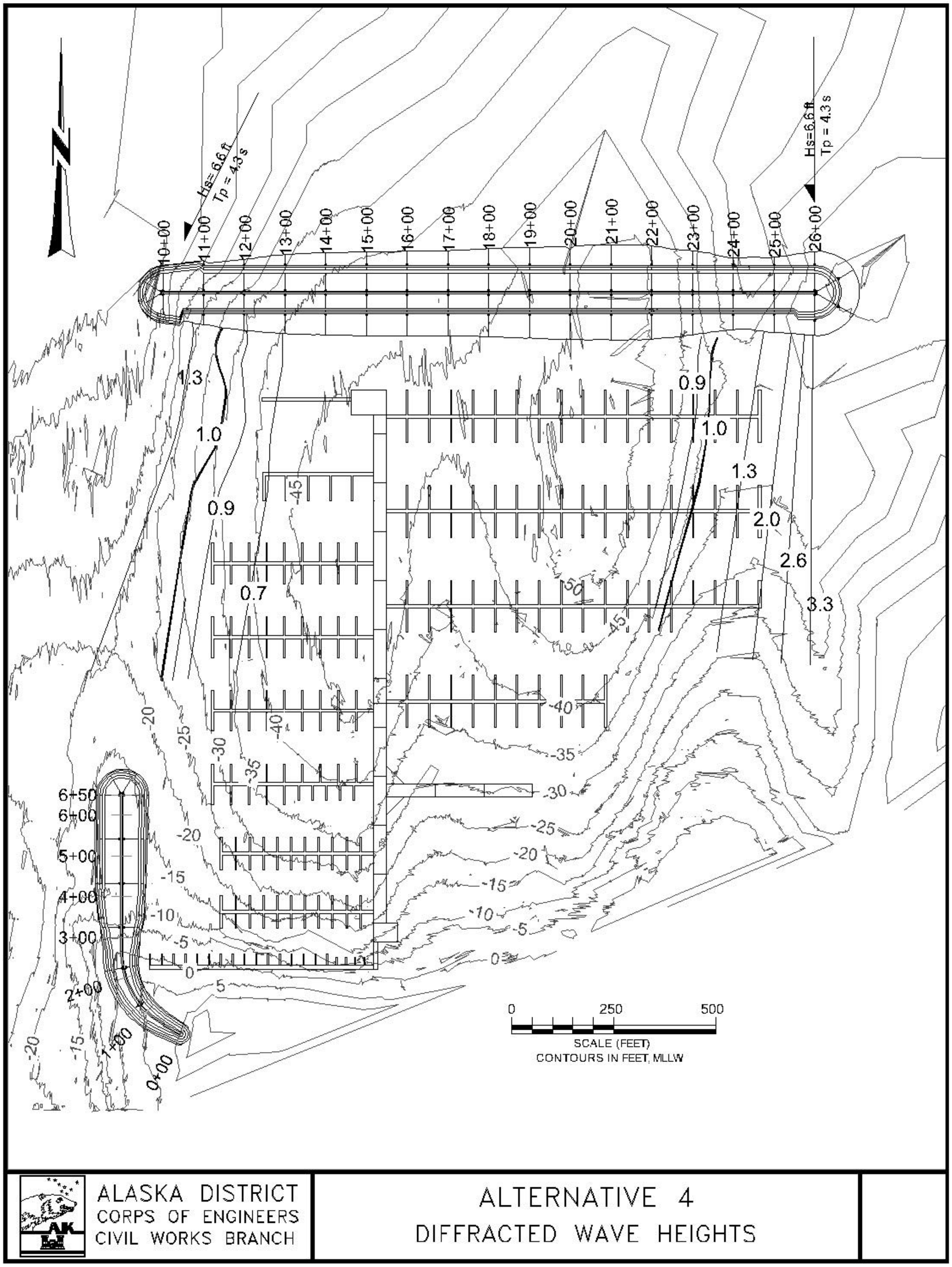

Figure 23 Diffracted wave heights for Alternative 4. Note that harbor floats are shown for illustrative purposes only. Float construction is responsibility of local sponsor.

8.0 NAVIGATION AIDS

The Coast Guard will require a fixed navigation aid for the breakwater. During development of plans and specifications the Coast Guard will be contacted to determine the navigation aid requirements.

9.0 CONSTRUCTION CONSIDERATIONS

The breakwater construction is anticipated to take two years to complete. It is expected that the stone for the breakwater will come from Craig. Construction can occur throughout the year with the exception that no in-water work will be performed between 15 March and 15 June in order to avoid the peak herring spawn and juvenile salmon out-migration periods as well as the period when humpback whales and other marine mammals are most likely to be present in the project area. In order to attract a number of bidders, it is recommended that the project be advertised early in the year to maximize the number of contractors to bid on this project.

11.0 REFERENCES

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