APPENDIX A
HYDRAULIC DESIGN
NAVIGATION IMPROVEMENTS
AKUTAN, ALASKA

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INTRODUCTION

1.1. Purpose

This appendix describes the engineering and technical aspects of the proposed harbor and navigation improvements for the head of the bay site in Akutan, Alaska. It includes an examination of several harbor alternatives, sections on existing climatology, the expected wave climate, and design criteria. Also included is an examination of the major construction features including breakwaters, entrance channels, dredging, and operations and maintenance. This appendix provides the background technical data for determining the Federal interest in the project.

Design criteria for this project were developed from published standards and methods as outlined in "*Shore Protection Manual*" (SPM), (USACE, 1984), "*Design of Breakwaters and Jetties,*" EM 1110-2-2904, (USACE 1986), and "*Hydraulic Design of Deep-Draft Navigation Projects,*" EM 1110-2-1613 (USACE 1994).

Other useful channel design criteria are found in:

"*Planning and Design Guidelines for Small Craft Harbors,*" (ASCE, 1994) "*Approach Channels A Guide for Design,*" (PIANC 1997)

1.2. Project Background/Purpose and Need

One of the largest shore based fish processing facilities in the United States (Trident Seafoods) is located in Akutan. Project location and site maps are provided in figure 1. This facility has been in operation since the late 1970s. The primary fleet that supplies the plant consists of commercial fishing vessels working in the Bering Sea. These vessels participate in the crab, pollock, Pacific cod, and halibut commercial fisheries. Most of these vessels are in the 85 to 210 foot length class.

In addition to the Trident plant activity, there are a number of small fishing vessels that are used by the residents of the Native Village of Akutan. Currently, the majority of these vessels are in the 16 to 24 foot length range. The Native village residents have the opportunity to participate in the Bering Sea fisheries under the Individual Fishing Quota (IFQ) and Community Development Quota (CDQ) programs.

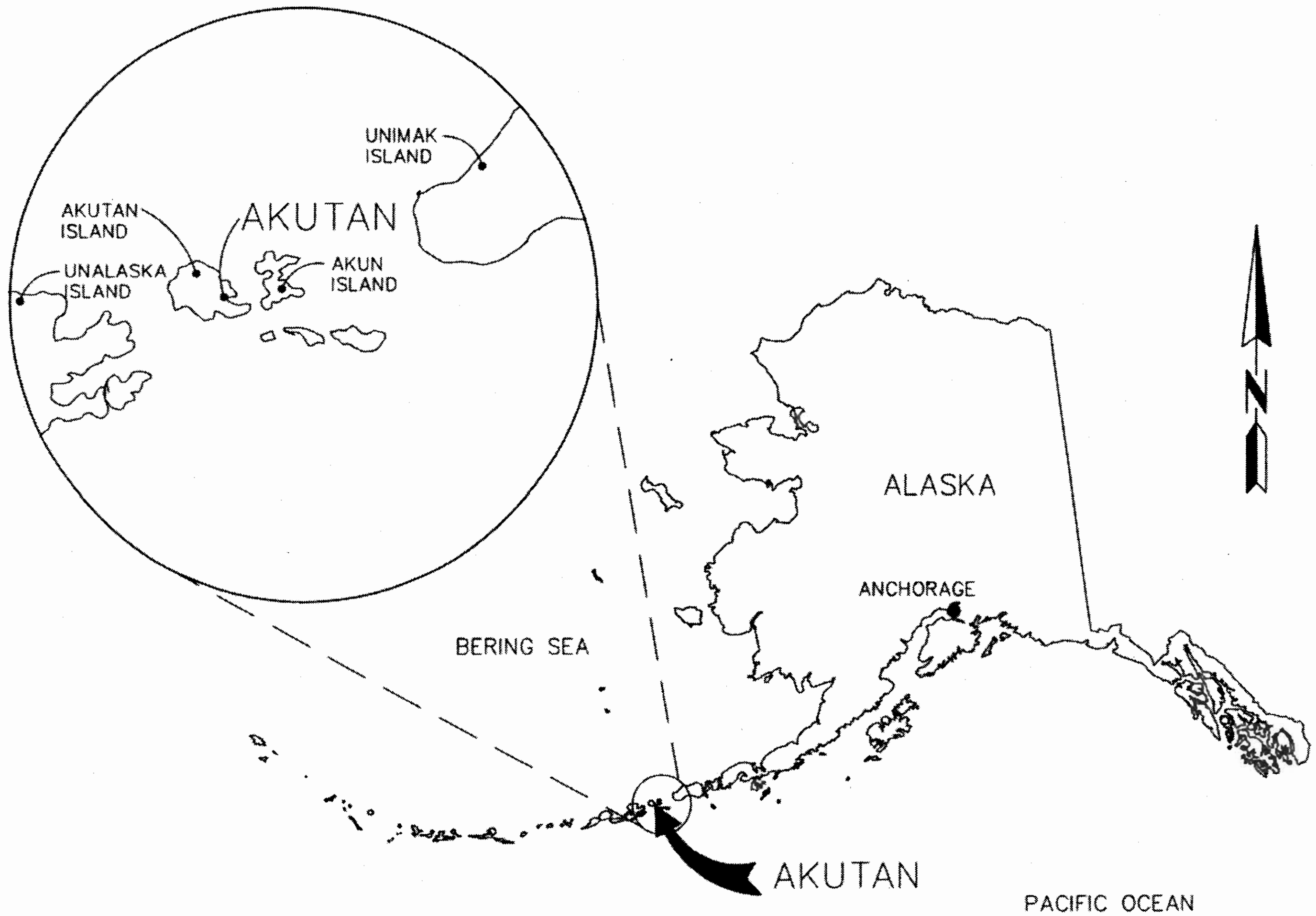
Since the early 1980s, the community of Akutan has been pursuing various means to construct a boat harbor to serve these vessels. Currently, the local fleet finds temporary transient moorage along a somewhat unprotected sheet pile wall adjacent to the Trident plant or elsewhere in the bay. When fishing season is over, many of the larger fishing vessels return to home ports, some as far away as Seattle. Smaller local vessels are pulled out of the water when not in use.

The purpose of this project is to provide a safe and efficient harbor. The harbor must be sized so that it will efficiently serve the existing fleet. The design must provide an economically sound facility with regard to both initial and long-term maintenance costs. In addition, the project must minimize any possible negative environmental impacts.

1.3. Use of English Units

Measurements used in this appendix are English rather than metric units. English units are used because much of the historical data and past studies were recorded in these units, much of the previous survey work was done in English units, and the survey control was based on Alaska State plane coordinates, which are in feet. Conversion of this previous information (especially the contours from the survey) would be laborious.

Therefore, the more cost-effective approach for the current phase of the project was to continue using English units. This allowed for the more seamless use of the past work.



LOCATION MAP

HARBOR FEASIBLIITY STUDY

AKUTAN ALASKA

FIGURE 1

2.0 CLIMATOLOGY AND HYDROLOGY

2.1. Climatology

No long term climatological data exists for Akutan. The National Weather Service and National Oceanic and Atmospheric Administration (NWS/NOAA) maintained an automated recording station for precipitation, snowfall, and temperature for Akutan from January 1986 through February 1990. Additional climatic information is taken from the 1988, *NOAA Climactic Atlas, Volume 2, Bering Sea area* (NOAA/CA, V2, 1988); and the *Aleutians East Borough Wave Study—Akutan Alaska, 1995/1996*, prepared by Peratrovich, Nottingham and Drage, Inc. (PN&D). The limited amount of data available and data gaps experienced during the NOAA/NWS data-logging period make developing a comprehensive report of climatological conditions at Akutan difficult. Climate data from nearby Unalaska, which approximates conditions at Akutan and has a longer period of record, will be used as it applies.

2.1.1. Precipitation

Akutan generally experiences mild winters and cool summers characteristic of a northern maritime climate. Cloud cover accompanied by precipitation, usually in the form of drizzle or light rain, is common. According to available NOAA data, the average number of days per year experiencing 0.01 inches or more of precipitation is 277. Days with over 1.0 inch of precipitation are relatively rare, numbering approximately 14 days annually. The mean annual precipitation for Akutan during the NOAA data-logging period was 79 inches. Table 1 provides a summary of annual precipitation at Akutan.

Table 1. Summary of Annual Precipitation at Akutan, 1986–89

	Mean (in)	High (in)	Year	Low (in)	Year	1 Day Max (in)	Date	Avg. # Days >= 0.01"	Avg. # Days >= 0.1"	Avg. # Days >= 0.5"	Avg. # Days >= 1.0"
January	7.35	9.44	1987	4.28	1986	1.76	1/15/1987	25	16	5	1
February	5.98	9.31	1988	3.16	1986	1.20	2/17/1988	22	13	5	1
March	5.09	8.81	1987	3.06	1986	1.30	3/20/1987	23	13	3	1
April	4.93	5.79	1987	4.07	1986	0.90	4/4/1986	22	16	3	0
May	4.14	5.46	1986	2.81	1987	0.98	5/2/1986	18	7	5	0
June	5.33	6.38	1986	4.20	1988	1.50	6/10/1986	21	12	4	1
July	4.77	6.16	1987	3.76	1986	1.10	7/22/1986	19	10	3	1
August	5.50	6.91	1988	4.38	1987	1.70	8/8/1987	20	11	3	1
September	7.36	8.28	1988	6.42	1986	2.00	9/18/1988	23	14	5	2
October	11.26	13.38	1988	10.08	1987	2.04	10/27/1988	28	21	8	3
November	7.34	10.96	1988	5.34	1987	2.25	11/1/1988	28	18	5	1
December	8.90	13.19	1986	4.23	1987	2.03	12/29/1988	28	20	5	3
Annual	79.01	89.32	1988	72.39	1986	2.25	11/1/1988	277	171	53	14
[1]Winter	34.65	44.24	1988	30.79	1986	2.25	11/1/1988	126	79	23	6
[2]Spring	9.07	9.53	1986	8.60	1987	0.98	5/2/1986	40	23	7	0
[3]Summer	15.60	15.96	1987	15.34	1986	1.70	8/8/1987	60	34	10	3
[4]Fall	18.62	21.66	1988	16.73	1986	2.04	10/27/1988	51	35	13	5

Note: Due to the limited amount of available data, some values were derived from 1, 2, or 3 years of data. Some of the data is derived from months with multiple daily data gaps.

[1] November through March

[2] April, May

[3] June through August

[4] September, October

Source: Western Regional Climate Center, NOAA

2.1.2. Snowfall

Monthly snowfall for the Akutan area is shown in table 2. January has the highest mean monthly snowfall of 13.9 inches. The mean annual snowfall is 19.5 inches with a maximum accumulation of 11 inches. Note that snowfall data from the limited period of record may not be representative.

Table 2. Summary of Annual Snowfall at Akutan 1986–89

	Mean (in)	High (in)	Year	Max. Accum. (in)
January	13.90	21.40	1986	10
February	1.25	1.90	1987	5
March	0.63	1.10	1986	11
April	2.60	4.50	1986	5
May	0.00	0.00	-	0
June	0.00	0.00	-	0
July	0.00	0.00	-	0
August	0.00	0.00	-	0
September	0.00	0.00	-	0
October	0.00	0.00	-	0
November	0.80	0.80	1987	5
December	1.50	2.90	1988	8
Annual	19.55	27.70	1986	11
[1]Winter	18.08	10.70	1988	11
[2]Spring	2.60	4.50	1986	5
[3]Summer	0.00	0.00	-	0
[4]Fall	0.00	0.00	-	0

Note: Due to the limited amount of available data, some values were derived from 1, 2, or 3 years of data. Some of the data is derived from months with multiple daily data gaps.

[1] November through March

[2] April, May

[3] June through August

[4] September, October

Source: Western Regional Climate Center, NOAA

2.1.3. Temperature

Temperatures at Akutan are typical of islands in the Aleutian chain with mild winter temperatures and cooler summer temperatures. Average annual temperature is 40.9 °F. The average winter temperature is 34.7 °F. Average summer temperatures reach 49.8 °F. The maximum temperature recorded during NOAA's 4-year monitoring period was 72 °F. The minimum temperature recorded was 8 °F. A summary of annual temperatures at Akutan is supplied in table 3. The PN&D report gives a higher summer average temperature of 55 °F and a winter average temperature of 35 °F.

Table 3. Summary of Annual Air Temperatures at Akutan 1986–89

	Averages			Daily Extremes				Monthly/Yearly Extremes				Max Temp		Min Temp	
	High (F)	Low (F)	Mean (F)	High	Date	Low	Date	Highest Mean	Year	Lowest Mean	Year	Avg. # Days >= 90 F	Avg. # Days <= 32 F	Avg. # Days <= 32 F	Avg. # Days <= 0 F
Jan	36.8	29.7	33.3	46.0	1/4/87	17.0	1/9/87	34.9	1987	30.5	1986	0.0	6.0	19.7	0.0
Feb	37.1	29.8	33.4	46.0	2/13/86	15.0	2/20/88	33.1	1988	32.8	1986	0.0	4.7	16.0	0.0
Mar	38.5	29.9	34.2	57.0	3/12/89	8.0	3/10/88	37.2	1987	31.0	1988	0.0	4.5	17.5	0.0
Apr	40.8	31.9	36.3	49.0	4/14/86	19.0	4/18/86	37.4	1987	35.2	1986	0.0	1.5	14.5	0.0
May	45.7	36.5	41.1	56.0	5/10/86	25.0	5/5/87	41.2	1987	41.1	1986	0.0	0.0	1.5	0.0
June	49.9	42.8	46.4	60.0	6/29/88	38.0	6/11/86	47.0	1987	45.4	1986	0.0	0.0	0.0	0.0
July	54.6	47.3	50.9	66.0	7/20/86	43.0	7/22/87	51.1	1986	50.7	1987	0.0	0.0	0.0	0.0
Aug	56.9	47.1	52.0	72.0	8/14/88	35.0	8/31/88	52.6	1987	51.5	1986	0.0	0.0	0.0	0.0
Sep	53.0	43.6	48.3	64.0	9/16/88	32.0	9/2/88	50.4	1986	47.1	1987	0.0	0.0	1.0	0.0
Oct	47.5	41.5	44.5	57.0	10/13/87	33.0	10/25/87	45.0	1986	44.1	1987	0.0	0.0	0.0	0.0
Nov	41.0	34.4	37.7	52.0	11/13/86	16.0	11/29/86	40.4	1986	36.4	1987	0.0	1.8	11.3	0.0
Dec	39.1	29.9	34.5	45.0	12/2/86	12.0	12/8/87	37.8	1986	30.8	1988	0.0	3.0	16.0	0.0
Ann	44.9	37.0	40.9	72.0	8/14/88	8.0	3/13/86	41.4	1987	40.3	1988	0.0	21.4	97.5	0.0
[1]Win	38.9	30.5	34.7	57.0	3/12/89	8.0	3/13/86	35.4	1987	33.4	1988	0.0	19.9	80.5	0.0
[2]Spr	42.6	34.0	38.3	56.0	5/10/86	19.0	4/18/86	39.3	1987	37.6	1988	0.0	1.5	16.0	0.0
[3]Sum	53.8	45.7	49.8	72.0	8/14/88	35.0	8/31/88	50.1	1987	49.3	1986	0.0	0.0	0.0	0.0
[4]Fall	50.3	42.6	46.4	64.0	9/16/88	32.0	9/2/88	47.7	1986	45.6	1987	0.0	0.0	1.0	0.0

Note: Due to the limited amount of available data, some values were derived from 1, 2, or 3 years of data. Some of the data is derived from months with multiple daily data gaps. The 1989 data set is missing several months of data.

[1] November through March

[2] April, May

[3] June through August

[4] September, October

Source: Western Regional Climate Center, NOAA

2.1.4. Unalaska Data

NOAA has archived climatic data since 1949 from Unalaska, and because of its close proximity to Akutan, some climatic data elements for Unalaska may be useful in estimating conditions at Akutan. The annual average precipitation for Unalaska is listed in NOAA/CA, V2, 1988 as 60.5 inches. Total annual snowfall is listed as 72.2 inches with a maximum accumulation of 25 inches. The mean annual maximum temperature is given as 45.3 °F and the mean annual minimum temperature is 35.9 °F. The maximum-recorded temperature was 80.1 °F and the minimum-recorded temperature was 1.9 °F. Prevailing winds are from the southeast with an average speed of 9.6 knots (11.0 mph). Highest wind speeds are from the east with speeds of 82 knots (94.4 mph).

Differences between Unalaska climatic data and available data from Akutan (particularly for snowfall) may be partially explained by the lack of long-term climatic data for the Akutan area. Personal interviews with Akutan residents have yielded some anecdotal information for precipitation and snowfall for the current year (1999/2000) and recent history (past two decades). According to Akutan residents interviewed, 1999 and early 2000 have had much higher than normal snowfalls (one estimate was over 100 inches) and similar weather patterns occurred in the early 1980s and 1990s. NOAA does not supply climatic data for the periods during which "more extreme" weather conditions are reported to have occurred, indicating that the actual annual snowfall and precipitation values may be greater than those shown in tables 1 and 2.

When Akutan and Unalaska snowfall data are compared for the years 1986 and 1987 (table 4), Unalaska shows an average annual snowfall of 45 inches compared to Akutan's average snowfall of 19.5 inches for the same time period. This indicates that Unalaska may receive more snowfall on average than Akutan.

Table 4. Comparison of Unalaska and Akutan Snowfall Data 1986–1987

	Unalaska		Akutan	
	Average Snowfall (in)	Average Accumulation (in)	Average Snowfall (in)	Average Accumulation (in)
1986	48.3	21	27.7	9
1987	41.7	19	11.4	11
Annual Average (in)	45.0	20	19.55	10
NOAA/CA Listed Average Annual Snowfall (in)	72.2	-	-	-

Source: National Data Center, NOAA

2.1.5. Wind

No long-term wind record data for Akutan Harbor exists. Neither the National Climatic Data Center nor the University of Alaska Environmental Research Institute were able to locate any archived wind data. During 1992, a wind gage was installed at the Trident Fish Processing plant approximately one half mile west of the community. This wind data collection effort was in support of the circulation study done to evaluate mixing efficiency from a submerged discharge. This short record appears to be the most representative local data available.

Because of the topography of the bay, wind directions seem to align with the long axis (east and west) of the bay. On the north and south sides, the terrain directly adjacent to the bay rapidly ascends to about 1,000 feet or more. This severely restricts cross-bay winds except near the bay mouth. Even if cross-bay winds do exist, they would not be effective in generating any appreciable waves because of the very limited fetch in the cross-bay direction.

The monthly mean wind speeds for two NOAA wave buoys, one in the Gulf of Alaska (No. 46003) and the other in the Bering Sea (No. 46035), were available on the World Wide Net. The distances from Akutan to No. 46003 and No. 46035 were about 400 and 500 miles, respectively. Due to their lack of proximity to Akutan and to their non-directional format, it was decided they were not particularly useful to this study.

The nearest long-term wind record was collected at Unalaska Airport. The anemometer there is situated to maximize its use by airplane traffic. As such, it is not well suited for use at Akutan Harbor.

The Climatic Atlas (Brower, et al, 1988) provides wind speed information for 5° latitude by 5° longitude rectangular grids based on ship observations. These were presented as a series of monthly wind roses for each grid. However, instead of sorting on wind speed class, only the mean monthly wind speed for each heading was provided. Table 5 provides this information for the grid that includes Akutan Harbor.

Data was collected from the Trident Fish Processing Plant on the north side of Akutan Harbor for a year and is summarized in compass rose plots for four annual quarters. These quarters were calendar based without regard to season, hence, the periods were for January through March, April through June, etc. These plots were modified to display only eight major compass directions and then combined into a single compass rose. This is presented as figure 2.

The data used to create figure 2 show a definite bi-modal direction pattern from the northwest and the southeast. Such a pattern would be expected given the strongly linear shape of Akutan Harbor and the relatively high elevations that border its north and south shoreline. However, the major wind directions are not aligned with the long east-west axis of the bay. It is possible that 1992 was not a representative year in terms of wind direction. However, it is also possible that the anemometer was placed in such a way that the measurements were biased, perhaps by the local orientation of the coast or due to an obstructing facility. It is also possible that the records were incorrectly recorded or analyzed.

Table 5. Mean Speed (kts) and Frequency (%) for Winds From Given Directions for 5° by 5° Rectangular Grid That Contains Akutan Harbor

Month		N	NE	E	SE	S	SW	W	NW
January	Mean	19	21	22	20	19	19	19	19
	Frequency	13	13	13	13	11	11	15	11
February	Mean	21	20	20	21	19	19	20	20
	Frequency	13	13	15	15	11	11	11	11
March	Mean	18	18	20	19	18	20	19	18
	Frequency	11	11	11	11	11	13	17.5	13
April	Mean	17	17	17	20	19	19	19	19
	Frequency	8.5	6	6	7.5	13	19	20	19
May	Mean	15	14	15	17	15	17	17	16
	Frequency	10	8	8	9	9	18	20	19
June	Mean	12	12	14	15	14	15	14	13
	Frequency	11	7	9	12	13	15	19	15
July	Mean	11	10	12	14	14	15	14	13
	Frequency	5	4	8	10	15	20	21	15
August	Mean	13	13	14	15	15	16	15	14
	Frequency	6	6	8	10	15	20	21	13
September	Mean	16	15	16	16	16	16	17	18
	Frequency	10	8	8	8	10	16	21	19
October	Mean	19	17	19	19	18	19	19	20
	Frequency	12	8	8	8	8	21	21	21
November	Mean	20	19	20	20	20	20	21	21
	Frequency	13	10	10	10	12	15	21	18
December	Mean	19	19	22	21	20	20	21	20
	Frequency	11	11	11	10	11	14	16	16

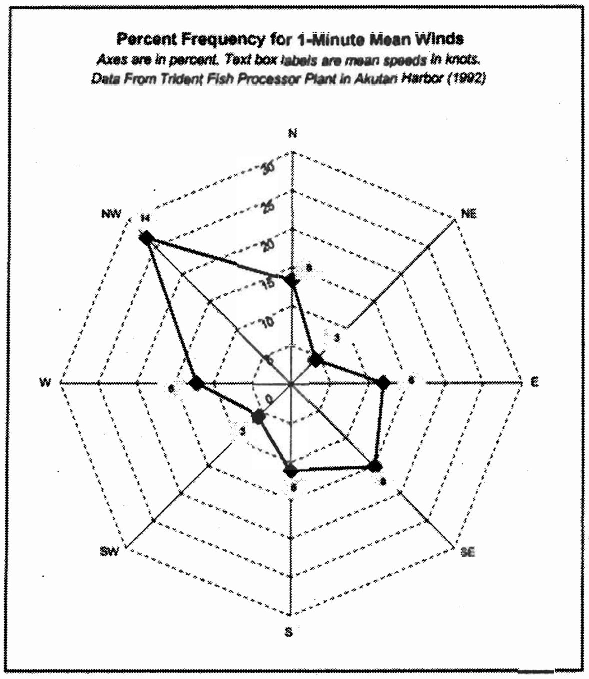


Figure 2. Akutan Harbor Wind Rose

(Trident Processing Plant. Data collected by Jones & Stokes 1993).

This one year of data is far less than is needed to base extreme wind conditions. For extreme wind estimates, the Climatic Atlas (op. cit.) was used. Although the atlas did not have any data for Akutan, it presented results for Cold Bay to the north and Nikolski to the south. Akutan Harbor is roughly equidistant between these two recording stations. Based on extremes at these stations, the extreme 1-minute winds for Akutan are as shown in table 6.

Table 6. Extreme 1-Minute Winds at Akutan Harbor

Probability	Return Period (yr)	Wind Speed (kt)
0.05	20	64
0.02	50	73
0.01	100	79

2.2. Hydrology

2.2.1. Tides

Tides prediction in Akutan is based on a primary National Ocean Service (NOS) station in Unalaska. This information is based on a 15-year period of record from January 1, 1960 to December 31, 1975. It comes from the 1960 to 1978 tidal epoch and is considered "preliminary" data by the NOS. The following tidal statistics apply to Unalaska:

Extreme high water	6.62 ft
MHHW	3.73 ft
MHW	3.46 ft
MTL	2.21 ft
MLW	0.97 ft
MLLW	0.00 ft
Extreme low water	–2.64 ft

The NOS has the following tidal corrections published for Akutan, based on the Unalaska station:

Time Difference		Height Difference	
High	Low	High	Low
–0.17	–0.17	1.08	1.10

Note the time difference values are additive and that the height difference correction is a multiplier.

Based on these published corrections the following tidal information is extrapolated for Akutan:

Extreme high water	7.15 ft
MHHW	4.03 ft
MHW	3.74 ft
MTL	2.41 ft
MLW	1.07 ft
MLLW	0.00 ft
Extreme low water	–2.90 ft

There was some water level information collected in Akutan during 1934 and 1935. This data does not meet NOS criteria for tidal datums. Also, in 1995 PN&D installed and monitored some wave and tide equipment in Akutan Harbor primarily for a wave study. This data has a relatively short period of record (one year) and does not conflict with the above extrapolated tidal data.

2.2.2. Storm Surge

Akutan Harbor is a relatively deep bay; therefore, it is highly unlikely that appreciable storm surges can be generated. To calculate the complete storm surge requires a numerical solution of the horizontal momentum equations. If the equation is simplified so that the wind stresses on the water's surface and at the bottom are used to balance the hydrostatic pressure gradient terms, a form as suggested by Dean and Dalrymple (1984) can be used to approximate the surge. In that reference, one example was presented for a constant depth situation and another for a linearly sloping bottom. These two can be combined to make a surge estimate for Akutan. First consider a region over the majority of the bay where the depth is nearly uniform. This region extends for about 3 miles from the mouth of the bay to near the head of Akutan Harbor; the depth is assumed to be 150 feet. By vertically integrating the simplified momentum equations and combining the ratio of the bottom friction to the surface wind stress into a single coefficient, n, the following expression is produced:

$$n = 1 - \frac{\tau_b}{\tau_s}$$

Where τ is the shear stress and the subscripts b and s refer to the bottom and surface of the water, respectively. The Shore Protection Manual suggests that n varies between 1.15 and 1.3. The more conservative (higher surge prediction) value of 1.3 will be used here. The formula for the surge over that 3-mile zone becomes:

$$\eta = \sqrt{h_o^2 + \frac{2n\tau_s x}{\rho g}}$$

Where η is the surge, h_o is the uniform depth, g is the coefficient of gravity, ρ is the density of sea water, and x is the distance from the mouth of the bay. For the following parameters:

h_0 = 150 feet

n = 1.3

τ_s = $3.1 * 10^{-6} W^2$ (W is the wind speed in fps)

ρg ≈ 65 pounds/ft^3

x = 18,200 feet (3 nautical miles)

The surge is about 0.18 feet.

The second region to consider is the sloping beach from deep water to the shoreline, which can be approximated by a uniform slope. The expression for the surge in this zone cannot be found explicitly, but must be expressed as the implicit relationship:

$$x = l\left[\left(1 - \frac{h+\eta}{h_o}\right) - A\ln\left(\frac{\frac{h+\eta}{h_o} - A}{1-A}\right)\right]$$

h is the water depth at some distance x, l is the length of the sloping bottom region and A is given as:

$$A = \frac{n\tau_s l}{\rho g h_o^2}$$

For the following parameters:

X = l (1,400 feet)

H = 0: h_o = 150 feet

A = 9×10^{-5}

The surge is given as 0.05 feet. Combining this with the constant depth zone, the total surge is just over 0.2 feet.

The portion of the surge that results directly from atmospheric pressure differentials is usually an order of magnitude less than that generated by shear stresses. It is not considered except where a region might be under the influence of a tropical storm with extremely high horizontal pressure differential and, therefore, will not be considered further in this case.

From the above discussion, it can be seen that storm surge is not a significant concern in Akutan Harbor. A conservative value of 0.2 feet can be used for design purposes.

2.2.3. Wave Setup

The approach to calculating wave setup uses the concept of radiation stress or excess momentum stress in the surf zone. This approach can be simplified and is a function of the breaking wave height. This has been plotted as a function of the wave characteristics and the beach slope in the SPM using Weggel's 1972 description of the breaking wave criterion. Further, the setup as a function of the breaking wave height and the beach slope also has been plotted in the SPM.

The wave setup can be determined using these tools combined with the wave conditions determined from the STWave model, and the maximum winds for the 20-, 50-, and 100-year return periods (presented in a later section), the wave setups can be determined and are shown in table 7.

Table 7. Extreme Wave Setups for the Head of Akutan Harbor

Probability	Return Period (yr)	Wave Setup (ft)
0.05	20	0.31
0.02	50	0.38
0.01	100	0.41

2.2.4. Design High Water Level

Based on the above discussions, a design high still water level can be taken to be the sum of the tide, wave setup, and storm surge. Using the extreme, 50 year values outlined above, this equates to an elevation of 7.73 feet above MLLW. It is important to note that this number does not take into account the height of the wave or any run up that may occur.

2.2.5. Currents

There is no current data available for Akutan Harbor in the NOAA *Tide Current Tables*. With a mean diurnal tidal range of approximately 3 feet, and with the semi-enclosed shape of the bay, it is highly unlikely that there will be significant current at the head of the bay site.

Akutan Harbor is in communication with both the Bering Sea and the Gulf of Alaska. North of the Aleutians, the Bering Sea current is eastward; south of the chain, the Alaska current flows to the west. In the passes that connect the Bering Sea and the Gulf of Alaska, tidal currents dominate the flows. At higher longitudes, part of the Alaska current flows into and merges with the Bering Sea system. It is probable that the circulation in Akutan Harbor depends very little on these regional current systems.

Jones and Stokes (1992) modeled the bay circulation and found that the currents were predominately driven by local winds. Currents on the order of 15 cm/s or about 0.3 knots were noted in the model, and these were somewhat confirmed by current measurements.

Akutan Harbor appears to have a classic 2-layer current system with flow in the direction of the wind on the surface and a countercurrent (opposed to the wind) near the bottom. It is not clear how these countercurrents are distributed horizontally or vertically. Hence, winds may drag surface water in the same direction near the center of the bay, or along one or both sides, or they may occur only at depth. Countercurrents are required to satisfy continuity.

The freshwater quantity entering the bay is limited and has little effect on bay currents or circulation. Stratification, which might be enhanced by freshwater inflows, would primarily effect how the countercurrents are distributed.

2.2.6. Fresh Water Input

There are two streams that traverse the valley at the head of the bay, one on the north side of the valley and one on the south side. These streams are near the toes of the steep slopes that define the edges of the valley. The stream on the north side of the head of the bay is larger than the one on the south side. This stream is classified by the State of Alaska Department of Fish and Game as an anadromous fish stream and is reported to support pink and coho salmon, as well as Dolly Varden. The stream on the south side of the bay is reported to support pink salmon. Both of these streams have an associated alluvial fan of deposited sediment at their mouths.

Measurements of the flows of these streams have been recorded during several previous studies. In June of 1983, Jones and Stokes estimated the flow in the north side stream at 27 cfs. This appears to be a peak value. In April of 1992, this same company reported a much lower "base flow" of 2.0 cfs for this same north side stream. Winter-like conditions with snow on the ground could have contributed to this low number. In August of 1982, Peratrovich and Nottingham, Inc. recorded a flow of 3.9 cfs in the south side stream and 10.9 cfs in the north side stream. These readings were reported to have been taken after several days of no significant rainfall. Measurements taken at different locations along the streams resulted in different flows, pointing to a high ground water infiltration and influence in the flows.

Also, there is a seasonal drainage near the middle of the beach at the head of the bay. This appears to be an outlet to the wetland impounded behind the berm. There is a small alluvial fan associated with this drainage that is visible in air photographs.

2.2.7. Icing in the Proposed Harbor

Information used for developing an estimate of icing conditions in the proposed Akutan Harbor came from the National Oceanic and Atmospheric Administration (NOAA), the National Oceanic Data Center (NODC), the National Weather Service (NWS), and anecdotal information from local sources.

Local sources interviewed had spent many years in the Akutan and eastern Aleutian area and provided accounts from the past 20 years. Residents from Akutan, Unalaska, King Cove, and Sand Point were interviewed. Most of the individuals interviewed held land-based maritime related positions: either serving as Harbor Master, or employed in the harbor of their community.

Interviews with harbor employees at Unalaska, King Cove, and Sand Point revealed that these harbors all have similar icing conditions. These harbors experience occasional icing, however, the icing only occurs during the coldest winter days and usually consists of a thin slushy layer that does not interfere with boat maneuvering operations. A harbor employee from Sand Point indicated that on one occasion during the last 10 years the ice in the Sand Point Harbor became thick enough to walk on, however, large boat maneuverability was not hampered. All contacts indicated that their respective harbors remained ice-free during most of the winter months.

Air temperatures in the Akutan area remain relatively mild during the winter months with about 100 days per year experiencing minimum air temperatures below 32 degrees Fahrenheit (F). The average January temperature is above the freezing point at 33.3 °F. Cold snaps are usually short lived with no air temperatures below 0 °F recorded in recent history. The coldest seawater temperatures occur in December and January and hover slightly above 32 °F. These climatic and seawater temperature conditions do not favor the development of substantial sea ice.

Long-time Akutan residents who hunt and fish year-round at the west end (near the head of the bay) of Akutan Harbor state that icing does occur near the proposed harbor site. The ice that forms is thin and slushy, easily broken up by wave activity, and does not impede navigation in the area, even for smaller boats such as skiffs.

There are other factors that may add to the potential for ice formation inside the harbor. The harbor is expected to experience some freshwater in-flow from ground water seepage and surface water flow. Freshwater, being less dense than seawater will tend to remain on the surface and, therefore, exposed to ambient air temperatures. Adding to the potential for freezing is the possibility of relatively limited circulation in the harbor due to minimal tidal currents that occur in Akutan Harbor. Both of these factors are difficult to quantify.

Keeping in mind that the west end of Akutan Harbor in its present undeveloped state is experiencing the same fresh water in-flow and tidal currents as would be noted following development of the harbor basin, it can be expected that icing conditions should remain static outside of the dredged harbor basin. Icing conditions inside the harbor basin may be exacerbated by localized higher concentrations of freshwater and retarded circulation inside the basin. It should be noted that the harbors in Unalaska, King Cove, and Sand Point all experience a large amount of freshwater in-flow while remaining relatively ice free.

Based on the above stated information, it is anticipated that the proposed Akutan Harbor will experience some icing during the coldest winter months (November through February). For the most part, icing will consist of a thin slushy layer that will be easily broken up by wave action and should pose no hazard to navigation for both large and small vessels. Occasionally (up to 2 times per year) the harbor may experience heavier icing that may impede smaller vessels. Heavier icing events are expected to be of short duration (1 to 2 weeks per event) and should not prevent larger vessels from maneuvering in the harbor.

3.0 WAVE CLIMATE

3.1. Local Wave Generation

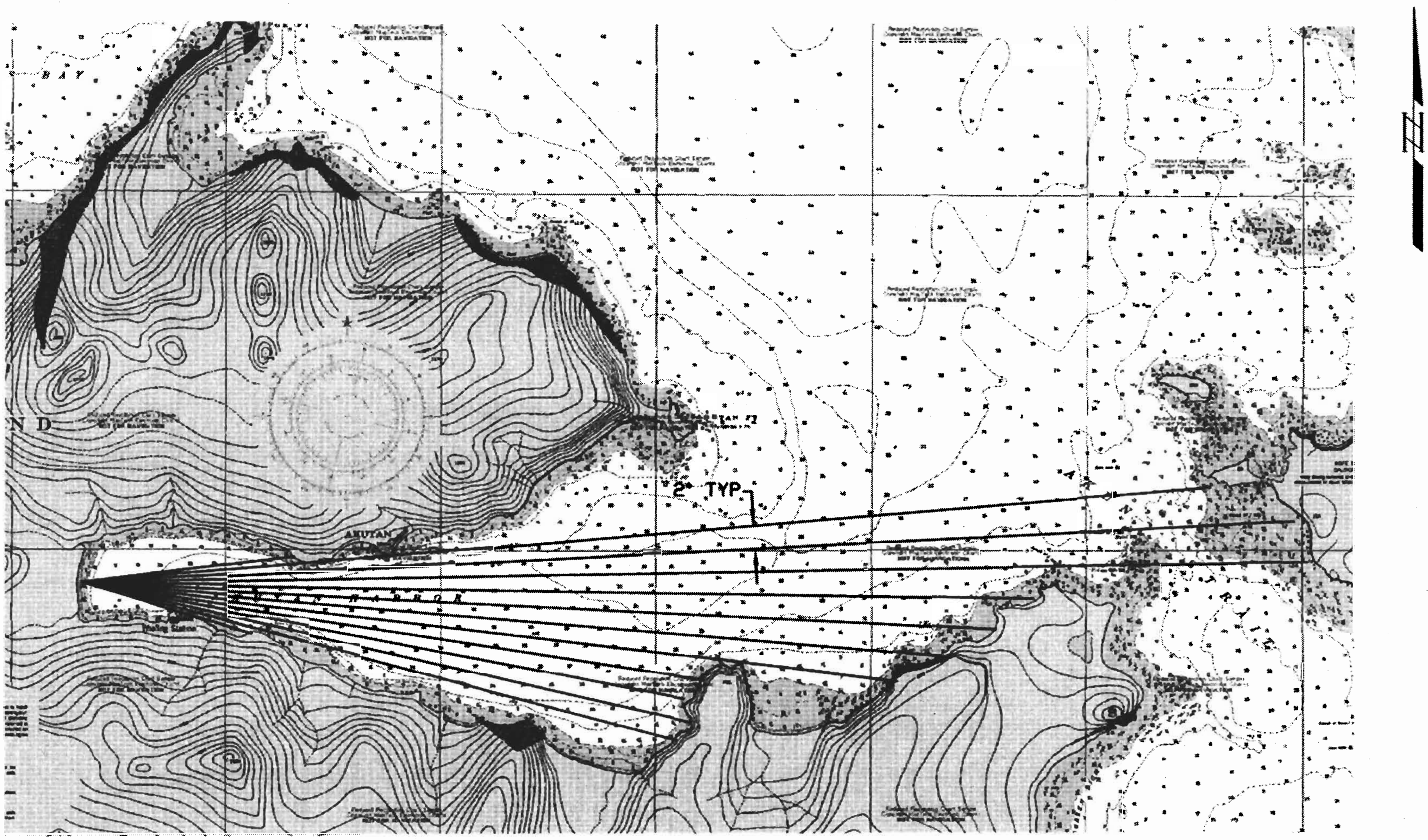
Local winds of any significance must conform to the east-west axis of the bay. Waves, with the potential to impact the project, could be generated only from the east. The fetches would be too limited from any other direction. From the east, fetches could begin far outside the bay and attain lengths of nearly 7 miles. Locally generated waves have been estimated using the restricted fetch method found in the ACES routines and by wave modeling using the steady-state spectral wave model STWave. With the latter, it was determined that the maximum wave-producing wind direction was one directed toward slightly (5 degrees) north of west.

3.1.1. Analysis by ACES Method.

For this method, a series of nine fetches were established radiating eastward from the general location of the potential boat harbor at the head of the bay (figure 3). There is a separation angle of 2 degrees between these radials. The length of the radials varied between 1.42 miles for the northernmost radial to 6.4 miles for the second radial. The average fetch was found to be 5.2 miles. The wind is assumed to be from 095 degrees, which was found from the STWave analysis (table 9) to produce the largest wave. The three extreme wind conditions presented in the wind analysis section were used in the model. The length of time to attain equilibrium wave conditions for the average fetch for this situation and for the three wind conditions presented above is just over an hour. Therefore, the 1-minute winds were transformed to 1-hour winds by dividing by 1.24 (according to SPM). The winds given in table 6 were transformed into hourly values of 64, 59, and 52 knots respectively, for the 100-, 50-, and 20-year return values. For these winds, the duration to attain equilibrium conditions was again checked, and was still found to be close to 1 hour; therefore, no additional adjustments were made. The following wave conditions resulted.

Table 8. Extreme Waves at the Head of Akutan Harbor (ACES)

Probability	Return Period (yrs)	Wave	
		Ht (ft)	Period (sec)
0.05	20	5.4	4.4
0.02	50	6.4	4.8
0.01	100	7.1	5.0



GRAPHIC SCALE

2,500 0 2,500 5,000

(IN FEET)
1" = 5,000'

WAVE FETCHS
ACES METHOD

HARBOR FEASIBLIITY STUDY

AKUTAN ALASKA

FIGURE 3

3.1.2. Analysis by STWave Method.

Locally generated waves at the head of the bay were also estimated using STWave. The grid consisted of 164 by 56 grid points positioned 200 feet apart (figure 4). Those results are shown in table 9.

Table 9. Extreme Waves at Head of Akutan Harbor (STWave)

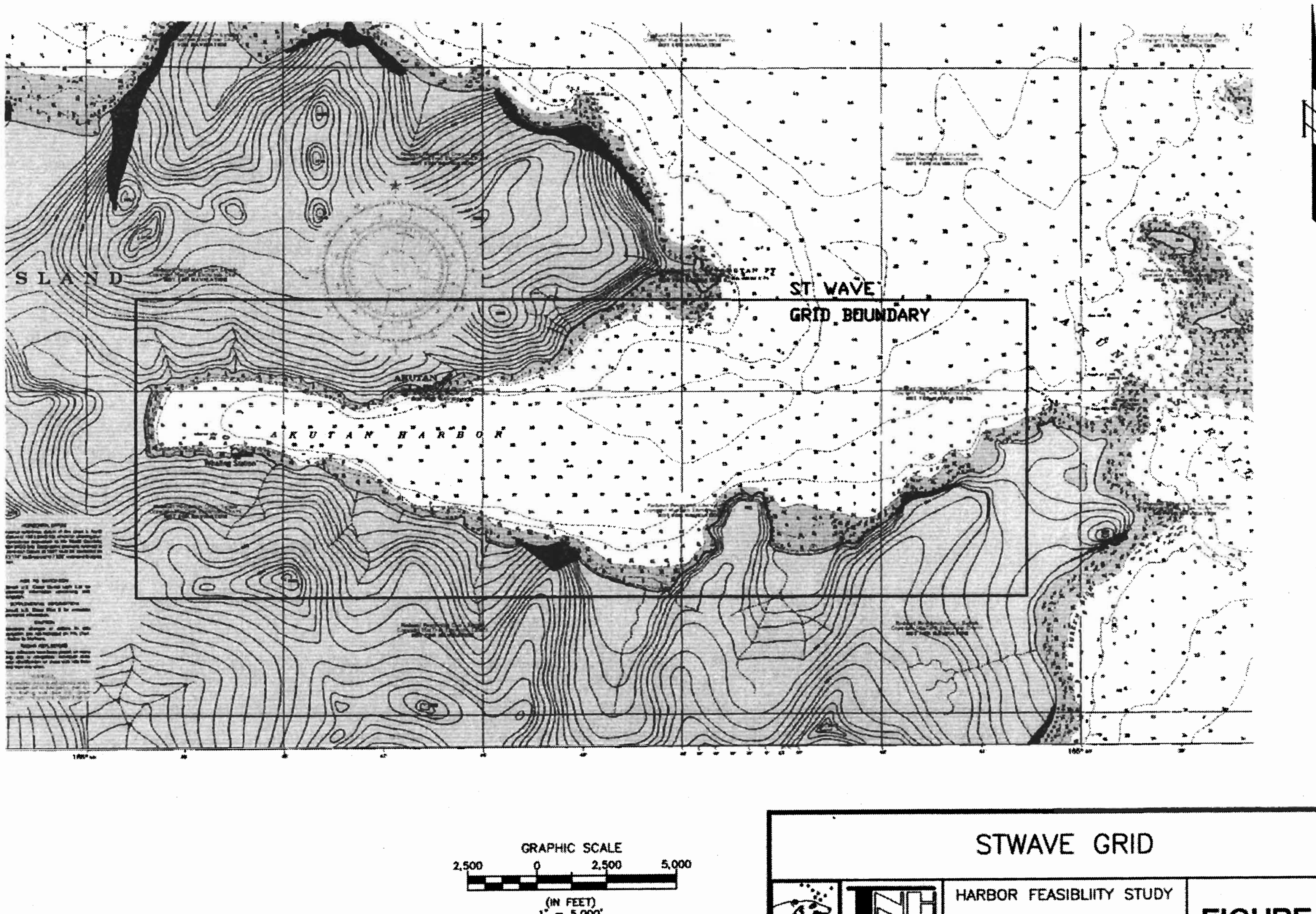
Probability	Return Period (yrs)	Wave	
		Ht (ft)	Period (sec)
0.05	20	2.7	4.4
0.02	50	3.1	4.7
0.01	100	3.3	5.0

Although the periods are nearly identical to the ACES estimates, the wave heights show a marked decrease using STWave. The wave height values tended to increase steadily toward the head of the bay. At a distance of just less than 1 mile from the end, the wave heights began to diminish because of narrowing at the head end. The wave energy is being refracted shoreward along each side, and remaining energy is being redistributed over the wave crest. This effect is not considered in the ACES method.

3.2. Waves of Non-local Origin

Large waves with periods in excess of 8 seconds occur routinely outside the confines of Akutan Harbor. The mouth of the bay is in direct communication with the Bering Sea to the north and the Gulf of Alaska to the south. However, to the east it is protected from direct contact to the Gulf and the Bering Sea by Akun Island. It seems possible that through a combination of wave refraction, diffraction, or reflection some of this wave energy could enter Akutan Harbor.

Waves approaching from the Gulf of Alaska through Akun Strait would encounter shoals and reefs on the western end of this passage that would severely redirect the energy of long-period waves and reduce their total energy through refraction, breaking, and bottom friction. Redirection due to the shoals would severely reduce wave height by refraction. This area should function as a relatively effective filter for long waves and it is doubtful that significant long period wave energy would enter the bay from this direction.



STWAVE GRID

HARBOR FEASIBLIITY STUDY

AKUTAN ALASKA

FIGURE 4

The passage between Akun and Akutan Islands to the north is exposed to the Bering Sea. It is wide and has deep water throughout. The angular change required by a wave to enter the bay from refraction alone would be well over 90 degrees. Except for isolated situations, angular changes of more than 90 degrees will not permit the propagation of any significant energy. Even situations that might allow larger angular changes at relatively minor loss of energy can be shown to occupy only a small horizontal extent, that is they occupy only a very shortened crest length. If such a crest section is permitted to propagate, diffraction and additional refraction will severely reduce its energy per crest length values. By this reasoning, refraction alone can be eliminated as a mechanism that directs energy into the inner bay. It might appear that a wave approaching from the Bering Sea might be diffracted as it passes Akutan Point (on the northeast corner of the bay). However, the water tends to be relatively shallow south of the point (directly in the lee for a wave from the Bering Sea). Also the bay side of the point is adjacent to a pocket beach. This situation is certain to combine wave refraction to any diffraction that is occurring. Both of these will reduce the bay-directed wave energy per foot of wave crest.

Waves, approaching from the northeast, could be diffracted by Akun Point with some of the energy being redirected into the bay. The angle of this approach would be between 45 and 60 degrees relative to the long axis of the bay. The typical wave diffraction diagrams (figures 2-30 or 2-31 in the Shore Protection Manual) point to diffraction coefficients of less than 0.1. Therefore, it would seem unlikely that appreciable amounts of wave energy could reach the head of the bay by this method and it would seem that diffraction also could be eliminated as a serious source of wave energy at the head of the bay.

Another possible mode of propagating Bering Sea wave energy reasonably far into the bay is by reflection. Evidence exists that waves (possible 2 to 3 feet high with a period of 12 to 14 seconds) do break onto the beach fronting the village of Akutan (VCR tape supplied by Harvey Smith of the State of Alaska Department of Transportation and Public Facilities). Regional winds during this event were northerly, out of the Bering Sea.

The waves would first reflect off the southern shoreline. Then, following a series of successive reflections or by a combination of reflections and refraction, it might be possible for energy to be directed toward the inner bay. This might seem possible, particularly for long-period waves that tend to reflect more efficiently.

Consider a 12-second period wave impinging on a 1 on 10 slope (this slope seems to be about the norm for the southern shoreline). Seelig and Ahrens (1981) developed a relationship between the reflection coefficient (χ) and the surf similarity parameter (ξ) that allows considering the effectiveness of reflection given certain slope and wave conditions. The surf similarity parameter is given in terms of the incident wave height (H_i), the deep-water wavelength (L_0), and the shoreline slope (cot2) as:

$$\xi = \frac{1.0}{\cot\theta\sqrt{H_i / L_0}}$$

For an incident height of 10 feet, ξ is about 0.9 that yields a reflection coefficient of just less than 0.1 on a natural beach and just over that for a plane (smooth) slope. Even a 14-second period wave with the same incident wave height impinging on a steeper 1 on 5 slope has a reflection coefficient of no more than 0.4.

Since several reflections would probably be necessary to propagate this wave energy into the bay, this would not be particularly effective if more than one reflection were required. Only a single reflection is necessary to be directed toward the community.

These reflected waves would tend to develop crests that are essentially parallel to the long axis of the bay and would probably become even more parallel with each successive reflection. Therefore, only diffraction (the crest-parallel propagation of energy) could generate any energy at the head of the bay from these reflected waves. The original reflective surface on the south shore near the mouth of the bay was, at most, only a few hundred feet long. Hence, the reflected wave, in effect, becomes a short-crested wave. To reach the head of the bay through diffraction would require the wave energy from this short-crested wave to travel along its crest a distance of several thousand feet. It is clear that the resulting wave height, if discernable at that distance, would be very small. Therefore, it is unlikely that significant long period wave energy will be reflected to the head of the bay site.

3.3. Wave Summary and Design Wave

It is recommended that the 50-year wave forecasted by the STWave model be adapted as the "design wave" for the project. For purposes of design of structural items it is recommended that the Rayleigh wave height distribution average of the highest 10 percent of all waves be used (H_{10}). The 10 percent wave (H_{10}) is defined in the SPM as the significant wave height (H_S) multiplied by a factor of 1.27. The period remains the same. The design waves are summarized in table 10.

Table 10. 50 Year Return Period Waves

Wave	Ht (ft)	Period (Sec)
H_S	3.1	4.7
H_{10}	3.94	4.7

4.0 SEDIMENTATION ANALYSIS

No site specific sedimentation studies have been performed for the head of the bay. Therefore, sedimentary processes there must be inferred from the beach and nearshore morphology and from the principles of wave propagation and transformation in the coastal zone. It is probably safe to conclude that unless waves are from the east they do not generate any significant transport potential.

4.1. Existing Environment

Field studies conducted in 1998 have confirmed air photo observations that show sediments to be relatively limited and coarse except for at the head of the bay and at the occasional site along the north and south coasts of Akutan Harbor. A large accumulation of relatively fine beach sediments is present only at the bay's head. This accumulation forms a relatively long and narrow pocket beach at the head of the bay that is about 2,500 feet long and 50 to 75 feet wide. This is similar to a typical pocket beach that is formed from sediments becoming trapped between two headlands; however, it differs in that it has two streams that border it on each end. The headlands in this case are the north and south shoreline.

A soils exploration report prepared by Peratrovich and Nottingham, Inc. (now PN&D) in 1982 in support of a then proposed barge landing, described the beach at the head of the bay as a "storm barrier beach." This report describes the typical formation of backwater lakes behind these storm barrier beaches. It describes the beach deposit material as medium to coarse-grained sand and fine gravel with pebbles ranging up to a half inch in diameter. It reported that the thickness of the deposit could be expected to be 90 to 200 feet.

The beach has a somewhat classic shape that contains a berm. This berm has a crest elevation of approximately +10 feet MLLW. Behind the berm there is generally a drop of about 2 feet to elevation +8 feet. Then the terrain gradually slopes upward to the west into the valley. The slope of the beach is approximately 15 to 20 percent.

4.2. Longshore Transport

It is probable that the beach at the head of the bay was formed by the deposit of materials transported up the north and south shores of the harbor from the east to the west. This points to a general along-shore transport down the bay toward the beach at the head.

Undoubtedly, sediment moves along the beach (north and south) in response to relatively small changes in the wind direction. Therefore, it takes only a minor change in the direction of an east wind to switch the dominant shoreline supplying the sediments to the head from the south to the north shoreline and visa versa. There is no obvious indication which shoreline contributes most of the sediment.

It is worth noting that neither stream mouth migrates any appreciable distance along the shoreline. This probably indicates that the north and south longshore transports are nearly equal. As sediment is produced from erosion on the adjacent north and south shorelines, it is transported and deposited at the headland beach and in the nearshore. This probably results in the gradual seaward migration of the stillwater-line and the increased elevations of the backshore. Simultaneously, a small amount of beach sediment is blown further onshore by the wind.

4.3. Effects of a Harbor

Two questions that need to be addressed are:

- How would a harbor and its associated inlet be affected by this beach system?
- How would the system, in turn, respond to the harbor and inlet?

The answer to these questions will probably depend on the extent to which the facilities are built seaward. Ultimately, the tendency will be for the beach to move seaward as it absorbs the sediments transported to it as it has been doing. Any harbor inlet that will be encroached on by sediments being transported alongshore will be subject to shoaling on its harbor end (flood-tide shoals) and on its seaward end (ebb-tide shoals). Clearly, this tendency at Akutan Harbor would be severely reduced by the minimal amount of sediments available for transport and the fact that winds and presumably waves will be approaching the beach nearly head on. Such wave angles tend not to generate much longshore transport. The tendency to build up deposits in the inlet would be further reduced by constructing jetties on one or both sides of the inlet. As sediment begins filling the corners formed where the jetties connect to the beach, sediment fillets will develop and expand. However, these fillets could quite easily be excavated and the sediment mechanically transported back to the beach. This would greatly reduce the amount of sediment that could enter the inlet.

A portion of the sediment that forms the beach is transported from inland sources by the two streams. This material also contributes to the small alluvial fans fronting both streams. There are no particular depositional features that would suggest a strong interaction between the streams and the beach, such as creation of long-shore bars at their mouths. Therefore, it is unlikely that isolating them from the beach sediments through the construction of the harbor facilities would change them appreciably.

Since it appears that only a small amount of sediment participates in the transport processes at the head of the bay, the amount of maintenance dredging will be minimal. Alluvial fans have developed offshore of each stream, and it is suspected that these fans are probably more a result of long-shore sediments being swept offshore by the stream than sediment being transported down the valley by the stream. A third, but considerably smaller (perhaps ephemeral) stream also bisects the northern half of the beach. It appears that this stream may flow only during periods of high runoff, and it has also created a small alluvial fan offshore. It is likely that maintenance dredging would not be required at intervals shorter than 10 years.

5.0 HARBOR DESIGN CRITERIA

As stated in Section 1.0, design criteria for this project were developed from published standards and methods as outlined in "*Shore Protection Manual*" (SPM), (USACE, 1984), "*Design of Breakwaters and Jetties,*" EM 1110-2-2904, (USACE 1986), and "*Hydraulic Design of Deep-Draft Navigation Projects,*" EM 1110-2-1613 (USACE 1994).

Other useful channel design criteria are found in "*Planning and Design Guidelines for Small Craft Harbors,*" (ASCE, 1994) "*Approach Channels A Guide for Design,*" (PIANC 1997).

5.1. Design Vessel and Design Fleet

The typical vessel using Akutan Harbor is a larger sized Bering Sea commercial fishing vessel consisting of trawlers and catcher processors. These vessels range in size from about 80 feet length overall (LOA) to more than 160 feet LOA. Beams range from about 24 to more than 40 feet. Drafts range from about 8 to 16 feet.

The local Village of Akutan fleet consists mainly of vessels under 40 feet in length. Currently, there are about 20 of these smaller vessels used by the locals.

The Akutan Harbor fleet is summarized below.

Table 11. Akutan Harbor Fleet

Length (ft)	0–24	24–32	32–110	110–140	140–160	160–180	Total
Village	10	10	0	0	0	0	20
Trident	0	0	8	22	23	7	60
Total	10	10	8	22	23	7	80

The design vessel is a Bering Sea trawler type vessel. Although there are larger vessels that may use the harbor, such as catcher processors, the design vessel is thought to represent the upper end (in terms of size) of a Bering Sea commercial fishing vessel that might reasonably be expected to use the harbor. Dimensions are summarized in table 12.

Table 12. Design Vessel

LOA (ft)	160
Beam (ft)	35
Draft (ft)	14

5.2. Design Basin Area per Vessel

The amount of published data related to how many vessels of various sizes can be accommodated per acre in a moorage basin is somewhat sparse. One source is "*Marinas and Small Craft Harbors*" by Tobiasson and Kollmeyer (1991). This text has a table that outlines typical boats per acre for vessels up to 60 feet in length. Harvey Smith, State of Alaska Department of Transportation Coastal Engineer, has also developed some criteria for vessels per acre for vessels up to 180 feet in length.

The number of vessels per acre is largely dependent of the moorage arraignment. The moorage arrangement is dependent on fairway width, general float layout, stall or parallel moorage, and whether rafting is allowed.

The design vessels per acre values used for this project are summarized in table 13. The values were compiled and adjusted from the above two referenced sources. They should be used for planning purposes only.

Table 13. Design Vessels Per Acre

Vessel Length (ft)	1.75 LOA Fairway, Stall Moorage, No Rafting Allowed	1.75 LOA Fairway, Parallel Moorage, No Rafting Allowed	1.75 LOA Fairway, Parallel Moorage, Rafting Allowed
20	81		
24	57		
30	43		
40	26		
50	18		
60	13		
70	7.5	9.5	11.75
80	6.5	8.25	10
90	5.5	6.9	8.3
100	5	5.35	5.7
120	3	3.3	5.1
140	2	2.75	4.25
160	2	2.2	3.4
180	2	2	2.5

5.3. Allowable Wave Heights

In general, the disturbance in the mooring basin should not exceed a 1-foot height for a 50-year event. In addition, the final mooring basin design should meet the standards outlined in table 14.

Note that the standards above include a vessel orientation parameter (head or beam seas) and a maximum horizontal motion criterion. These standards were supplied by the State of Alaska Department of Transportation and Public Facilities and are based upon a Canadian study of acceptable wave climates in harbors commissioned by the Canadian Department of Fisheries "*Study to Determine Acceptable Wave Climate in Small Craft Harbours,*" (Northwest Hydraulic Consultants Ltd., 1980). The horizontal motion and vessel orientation criteria are important parameters not accounted for in the maximum one-foot wave height requirement.

Table 14. Wave Criteria for Mooring Basin

Recurrence, Orientation and Period (T in seconds) For Wave Heights (H1/3):	Wave Height (ft)
1 year interval, Beam Sea, T>6	0.50
1 year interval, Beam Sea, 2<T<6	0.50
1 year interval, Beam Sea, T<2	1.00
50 year interval, Beam Sea, T>6	0.75
50 year interval, Beam Sea, 2<T<6	0.75
50 year interval, Beam Sea, T<2	1.00
1 year interval, Head Sea, T>6	1.00
1 year interval, Head Sea, 2<T<6	1.00
1 year interval, Head Sea, T<2	1.00
50 year interval, Head Sea, T>6	2.00
50 year interval, Head Sea, 2<T<6	2.00
50 year interval, Head Sea, T<2	2.00
For Horizontal Motion (ft):	Horizontal Motion (ft)
1 year interval, Beam Sea, T>6	1.00
50 year interval, Beam Sea, T>6	2.00
1 year interval, Head Sea, T>6	2.00
50 year interval, Head Sea, T>6	4.00

5.4. Entrance Channel

The entrance channel to the small boat harbor has four primary design parameters width, depth, length, and alignment. The location for the entrance channel is roughly the same for all the inland harbor alternatives and was chosen for ease of navigation and environmental reasons. During initial study, the entrance channel was aligned with a natural offshore channel near the south side of the head of the bay. This location was thought to be advantageous because of the possibility that shorter breakwaters and jetties could be used due to the wave refraction effect of the offshore channel. Subsequent preliminary environmental studies indicated that this area is frequented by Eiders. For this reason, the entrance channel was moved to the north side of the head of the bay.

During review of the initial draft of this study, concerns were raised about the circulation and flushing within the inland basin. This led to further study and revisions to the entrance channel as well as to the general shape of the basin in an effort to improve circulation and flushing. These improvements are shown on the "Reconfigured 12 acre basin" figures in this document. The revisions include maintaining a rather narrow entrance channel from the bay into the basin in an attempt to maintain the momentum of the tidal prism and hence to increase circulation.

Initially the inland harbor alternatives were depicted with a somewhat "V" shaped entrance channel that opened up into the harbor. The reason for this was so that an adequate turning radius could be maintained for access into the east side of the mooring basin and to floats located along the west side of the basin. To improve circulation, this was later reconfigured into a more traditional parallel bank entrance channel. This required that the access into the float area be along the west side of the basin with floats located along the east side. This resulted in a slightly larger maneuvering channel and slightly larger over all basin area.

5.4.1. Width

Primary factors involved in the design width of the entrance channel include the traffic pattern (one or two lanes), the design ship beam and length, environmental factors such as wind waves and current, and the channel cross section shape. Channel width is defined as the toe-to-toe width measured from the bottom or toe of the side slopes.

Channel width elements consist of a maneuvering lane, bank clearance, and ship clearance if 2-way traffic is anticipated. The width of each of these elements is dependent on the beam of the design ship, the controllability of the vessel, and on the alignment of the channel.

Based on the criteria set forth in EM 1110-2-1613 and EM1110-2-1615, an entrance channel width of 100 feet would meet the criteria for 1-way traffic of a 35-foot wide (beam) design vessel in a straight channel section.

A 100-foot channel width is on the narrower end of the spectrum of "industry standard" entrance channel widths. However, there are several factors that support the use of this width. First, the fleet using the harbor will be less then 200 vessels. This means that traffic into and out of the harbor will be light. Next, the tides at Akutan are quite small and visibility around the entrance channel should be fairly good. This is especially true for the larger commercial fishing vessels that are expected to make up the majority of the fleet. The wheelhouses of these vessels will generally be high enough to see any approaching vessel traffic over the crest of the jetties. Also, the relatively narrow entrance will increase the protection from the ambient wave climate. Finally, the relatively narrow width will maximize flushing of the harbor by increasing the momentum of the tidal current in the basin. Based on the above discussion, a minimum entrance channel width of 100 feet is recommended for this project.

The initial design called for a 100-foot toe-to-toe width at the beginning of the entrance channel (outer entrance) widening to approximately +300 feet at the entrance into the harbor basin. The widening the channel was configured to aid in navigation and maneuvering. Water quality concerns were raised during agency review of the draft report. These concerns led to a numerical model study of the basin and a redesign of the entrance channel in an effort to increase tidally generated momentum in the entrance channel and to thereby increase flushing and improve water quality. The "reconfigured 12 acre basin" design includes a relatively narrow (100' wide) uniform width entrance channel. Navigation and maneuvering are not impacted because the turning basin is not next to the entrance channel in the reconfigured design.

5.4.2. Depth

Primary factors involved in entrance channel depth include the at rest design vessel draft, tide height, vessel squat, vessel heave pitch and roll due to wave action, and a safety margin based on bottom type. For this project the following parameters were used:

a. Design vessel draft: 14.0 feet

b. Tide height (extreme low): 2.9 feet

c. Squat: 0.4 feet

   A ship in motion will cause a lowering of the water surface because of the change in velocity about the vessel. The amount of lowering will be dependent on the speed of the vessel and the characteristics of the channel. For smaller recreational vessels, squat is normally estimated at 0.5 feet. For larger vessels, squat is normally calculated using the procedures outlined in EM 1110-2-1615. This involves determining the "blockage ratio of the submerged cross section to channel cross section," and determining the Froude number for the vessel in the channel. These values are then applied to a table to determine squat.

For this project, the submerged cross-section of the design vessel was found by multiplying the beam times the draft times the midsection coefficient. The midsection coefficient was assumed to be 0.9 (a typical value). This results in a design vessel submerged cross section area of 441 square feet. The channel cross sectional area was found to be 4,000 square feet based on an assumed depth of 20 feet and a width of 200 feet. A ratio of vessel to channel cross section was found to be 0.11.

The Froude number is dependent on the vessel speed and channel depth. Assuming a channel depth of 20 feet and a vessel speed of 8.5 feet per second (about 5 knots), the Froude number was found to be 0.335.

The above discussion leads to the following calculated squat of approximately 2 percent of the channel depth or about:

d. Wave motion (½ 50 year significant wave): 1.5 feet

e. Safety clearance (soft bottom): 2.0 feet

Total Calculated Entrance Channel Depth: 20.8 feet

The above total is predicated on the extreme significant wave event coinciding with the extreme low tide event. The probability of these two events occurring simultaneously is very low. An optimization study was performed and it was found to be cost effective to provide an entrance channel depth of –18 feet MLLW. This was found to be a reasonable value and is recommended for the entrance channel bottom elevation.

5.4.3. Length

The length of the entrance channel will have an effect on the inner harbor wave climate. Generally, longer channels provide more wave attenuation due to refraction and turbulence along the armored channel slopes.

For this project, another consideration is the effect the basin could have on the existing beach. An entrance channel length of approximately 400 feet was chosen for the inland basin option. This allows for the majority of the existing beach to remain unaffected by the harbor basin and provides for an approximate 175 foot separation distance between the harbor and the beach. This channel length also allows a relatively straight approach to the harbor.

5.4.4. Alignment

The entrance channel alignment was chosen because it provided the most direct access possible to the harbor while maintaining an acceptable inner harbor wave climate. As a rule of thumb, the turning radius for the channel should be no less than 2.5 times the length of the design vessel or 400 feet.

The jetties protecting the entrance channel are angled slightly from perpendicular to the beach face. The opening of the channel essentially points directly into Akutan Harbor aiding navigation as no significant turning will be required to enter the channel. However, it also means that approaching waves will be aligned with the entrance channel increasing the wave climate just inside the entrance channel mouth. The entrance channel is designed to decrease wave energy sufficiently prior to entrance into the mooring basin.

5.4.5. Inner Harbor Wave Climate

As stated in the previous section, waves moving westerly inside Akutan Harbor will approach relatively unimpeded into the entrance channel. In order to achieve the desired inner harbor wave climate, the entrance channel must be designed to bleed off wave energy as the wave moves down the channel prior to entering the inner basin. Refraction, shoaling, and turbulence will dissipate a portion of the wave energy as it travels down the entrance channel.

As the wave enters the harbor a portion of the wave is diffracted around the corner of the south breakwater into the harbor basin. Diffraction is the spreading of wave energy from areas of high energy to areas of less energy often behind or around obstructions. An often-used example of diffraction is the reduced wave energy on the lee side of an island. The waves impacting on the windward side expend a portion of their energy effectively leaving a gap in the wave. In this example, at some distance from the island's lee shore, waves again reform and build as wave energy moves along the crests to fill the void. If there were no reforming of waves (diffraction), there would be perfectly calm wave shadow areas extending indefinitely behind islands.

The wave climate for the "reconfigured 12 acre basin" was modeled using REFDIF software. The results of this modeling are included in an appendix to this document and are summarized in figure 5.

As can be seen from figure 5, most of the mooring basin area falls in an area where waves of less than 0.6 feet are expected. There are a few areas where the waves will be 0.8 feet or less and there are no areas where waves of 1.0 feet or larger are predicted during a 50-year event.

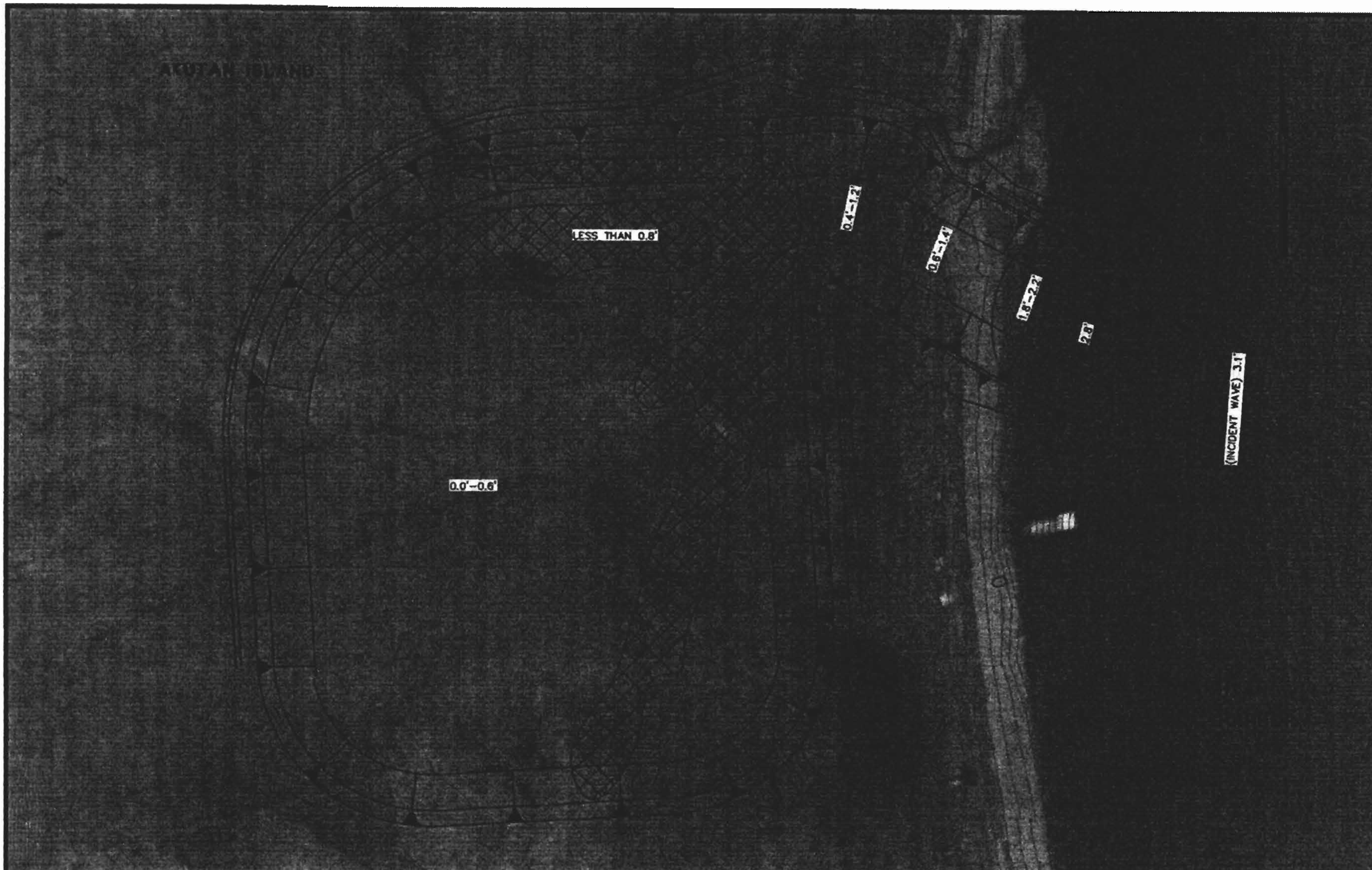
5.5. Circulation and Flushing

Circulation and flushing in the proposed harbor is a design consideration. The Akutan area has relatively low tidal range. As outlined previously, MHHW is about 4 feet. This means there is a relatively small tidal driving force for circulation and exchange in the proposed harbor.

5.5.1. Design Aspects to Improve Circulation

The ratio of length-to-width (aspect ratio) for the basins considered in this report varies from about 1.3 to 1.6. Aspect ratios of 0.5 to 2.0 are generally recommended for adequate harbor circulation and flushing.

The inland basins in this report were configured with large radius corners, about 200 feet. This was done to enhance circulation. The entrance channel was designed with a 100-foot wide section at the mouth. This was done to provide wave protection and to increase the momentum of the tidal exchange and to improve circulation.



NOTES
INCIDENT WAVE HEIGHT = 3.1'
4.7 SECOND WAVE PERIOD
50 YEAR RETURN PERIOD
WAVE CLIMATE BASED ON MODELING BY
"REFDIF" (TRADEMARK) WAVE ANALYSIS SOFTWARE

GRAPHIC SCALE
100 0 50 100 200
(IN FEET)
SCALE: 1" = 200'

INNER HARBOR WAVE CLIMATE

NAVIGATION IMPROVEMENTS

AKUTAN ALASKA

FIGURE 5

5.5.2. Circulation Modeling

Circulation in the 12 acre inland basin (described later) was modeled using a three dimensional numerical model as part of this study work. A report entitled "Circulation Modeling in Akutan Harbor and the Potential Impacts by and to the Proposed Small Boat Harbor" is included in a separate appendix to this report. Circulation in the reconfigured 12 acre inland basin was modeled using a three dimensional numerical model. A report entitled "Additional wave and water quality analyses for the potential boat basin at Akutan harbor, Alaska" is include in a separate appendix to this report.

5.6. Basin Depths

The basin design depths were established by considering water levels, wave activity, vessel motions, and safety factors. For this project, due to the wide range in vessel sizes, it will mean a stepped harbor basin.

Vessels from 20 to 120 feet will be moored in an area that has a minimum depth of –14 feet MLLW. Vessels from 120 to 150 feet will have a minimum depth of –16 feet. Vessels over 150 feet will have a minimum depth of –18 feet MLLW. These depth ranges are shown in table 15.

Table 15. Basin Depths

Harbor Area	Depth (ft)
Entrance Channel	18
Vessels > 150 ft	18
Vessels 120–150 ft	16
Vessels 20–120 ft	14

5.7. Wind Protection

As discussed in the previous section on climatology, the local winds are heavily influenced by the local topography. As such, the principal wind directions are east and west. To reduce wind-induced motion as well as to reduce wind loads, the inner harbor floats should be configured to align the long axis of the moored vessels parallel to the wind.

5.8. Geotechnical Stability

Earthquakes are not normally given a great deal of consideration in federally funded USACE projects such as this one. This is due to the generally low statistical probability of a significant event occurring in the fifty-year design life of the structures. Akutan has several significant features that merit seismic consideration, even for a fifty-year design life.

Akutan is located in a very active seismic zone. It is very close to the convergence of the North American and Pacific lithospheric plates in a region known as the Aleutian "megathrust" fault. The State of Alaska Department of Transportation (ADOT) has estimated that an earthquake with a peak acceleration of 0.35 g will have a 90% probability of not being exceeded in 50 years. This level of acceleration represents a large earthquake. Seismic mapping, within 25 miles of the site, has pointed to one recorded earthquake in excess of magnitude 7.1 and two recorded earthquakes of magnitude 6.1 to 7.0.

As stated above, Akutan is located almost directly above the Alaska Aleutian megathrust. According to the "*Probabilistic Seismic Hazard Maps of Alaska,*" (USGS Open File Report 99-36, Wesson et al, 1999):

> *Clearly the majority of the seismicity in the region is associated with the Alaska-Aleutian megathrust fault extending eastward along the Aleutian arc into south-central Alaska. The northwestward – moving Pacific plate is subducted along this megathrust beneath the North American Plate giving rise to the Aleutian trench and islands.*
>
> *The Alaska-Aleutian megathrust has been responsible for several of the largest earthquakes known in instrumental seismology, including the 1964 Prince William Sound (Mw 9.2) and the 1957 (Mw 9.1) earthquakes.*

This seismic activity raises concerns related to slope stability and liquefaction. Seismic conditions in the Akutan area are discussed in more detail in the Geotechnical Section (Appendix C).

5.8.1. Liquefaction

The existing soils at the head of the bay have been found to be only moderately prone to liquefaction. This is due to the existing medium dense, well-graded, coarse, sandy material that is fairly permeable.

In "*Seismic Guidelines for Ports*" by Stuart Werner, (ASCE, March 1998), the following discussion of liquefaction appears:

> *By far the most widespread source of earthquake-induced damage to port and harbor facilities has been liquefaction of the loose, saturated, sandy soils that often prevail at ports. Liquefaction has occurred at ports, even under only moderate levels of ground shaking. It leads to a reduction in stiffness and a loss of shear strength of the liquefied soils which, in turn, induces ground failures and soil settlement as well as increased lateral pressures on retaining walls and a loss of passive resistance against walls and anchors.*
>
> *The liquefaction of a loose, saturated granular soil occurs when the cyclic shear stress/strains passing through the soil deposit induce a progressive increase in the pore water pressure in excess of hydrostatic. In loose to medium dense sands pore pressures can be generated which are equal in magnitude to the confining stress. At this state, no effective stress exists between the sand grains and a complete loss of shear strength is temporarily experienced.*

There are a number of techniques available for mitigating liquefaction hazards. These include:

- Increasing the density of loose soils.
- Providing for higher permeability.
- Confining susceptible soils with a heavy layer of granular material.

- Using less steep slopes where possible.

The density of the soils can be increased by various means including vibrocompaction. Vibrocompaction can be one of the more cost effective soil densification methods. It is technically possible to densify the soils for an increase of about N > 10 (where N = the standard penetration value, a measure of density). An increase such as this can take a loose soil condition and make it medium dense. This can be done to depths exceeding 60 feet. For Akutan vibrocompaction could be achieved for a cost of about $250,000 per acre with a minimum of about 5 acres.

Permeability can be increased through the use of granular fill material and drains. One technique is to place granular fill in vertical "stone columns" that are designed to relieve excess pore water pressure. These columns are placed on a specified grid designed to ensure that the pore pressure never reaches the liquefaction limit.

Confining susceptible soils with a heavy layer of granular or rock fill can result in soil pressures that remain above the liquefaction pore pressure during an earthquake.

Using less steep slopes can lessen the chance of lateral spreading or other slope failure mechanisms.

5.8.2. Seismic Design Criteria

For this project, slopes and structural stability were analyzed using a seismic coefficient of 0.15 g. Note that this is less than the peak value of 0.35 g outlined by ADOT for the Akutan area. Peak seismic acceleration values have been found to poorly represent actual seismic design accelerations as they represent maximum accelerations of generally short duration during a seismic event. Designing to an extreme value of 0.35 g would lead to a prohibitively costly basin. The design seismic coefficient is generally taken to be between ⅓ to ½ of the peak value.

It is likely that during a major earthquake some damage will occur to the harbor slopes. The level of damage to expect during a design size seismic event is difficult to surmise. Many factors contribute to potential damage during an earthquake, such as magnitude, duration, and direction of ground movement. An approximation of damage during various seismic events is shown in the Geotechnical Section provided as Appendix C.

Slope failures would likely begin at the toe of the slope, where the lateral earth pressure is greatest with respect to the confining pressure. Therefore, it is important to carry the riprap down to the reinforced toe structure. It is estimated that under most anticipated seismic events, the majority of slopes would remain intact and slope repair would be limited to dredging of some sloughed materials and patchwork repair of the armored slope. Complete slope failure is possible if the area is subjected to a very large or long duration earthquake.

5.8.3. Breakwater Slopes

Slopes of 2 horizontal to 1 vertical were used during preliminary design of breakwaters and jetties. This slope transitions to a 3:1 inner harbor slope approximately 150 feet into the entrance channel. Slopes of 3:1 for the breakwater were investigated and proved to offer more stability during seismic events. However, armor stone and other breakwater construction materials are an important factor in overall project cost. Therefore, 2:1 slopes were deemed the more economical alternative.

5.8.4. Inner Harbor Slopes

The inner harbor slopes will be set at 3 horizontal to 1 vertical below the water line and at 2 horizontal to 1 vertical above the water line in the harbor basin. These criteria will apply to any slope in the water table area. The slopes will be covered with two layers consisting of 18 inches of 6 to 12 inch diameter armor stone (riprap) placed over 12 inches of filter rock. The rock layers will be underlain with a geotextile filter fabric. Figures 6 shows a cross section of the typical inner harbor slope.

5.8.5. Upland Fill Areas

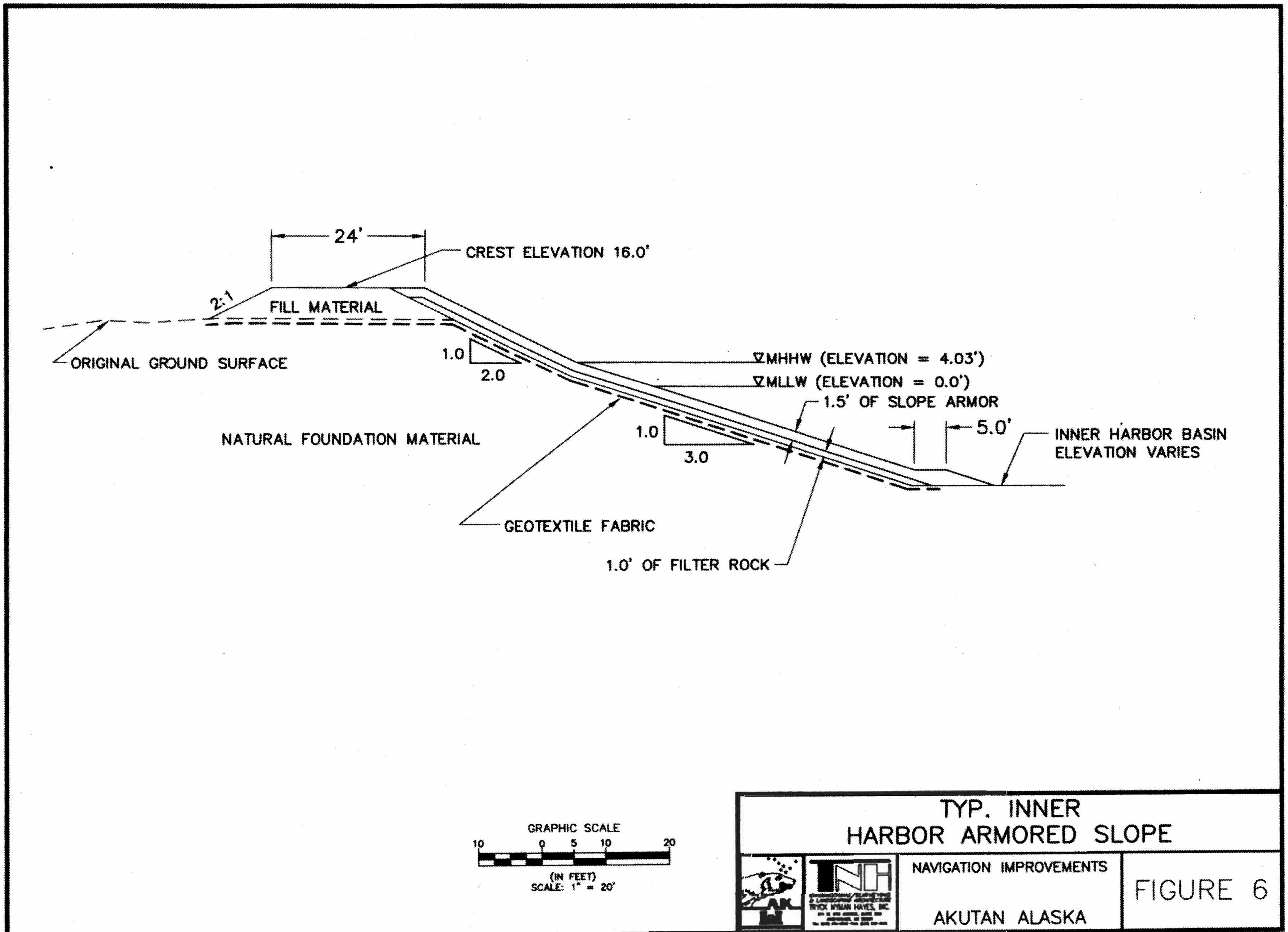
Upland fill areas will be constructed using harbor dredge materials. Slopes of 2 horizontal to 1 vertical above the water table and outside of the basin may be used. Dredged soils will have to be well drained prior to placing. A drainage basin should be prepared during construction to temporarily hold dredged materials and allow them to drain. The drainage basin runoff should be diverted back into the harbor basin. Once drained, dredged materials can be spread out on the uplands areas.

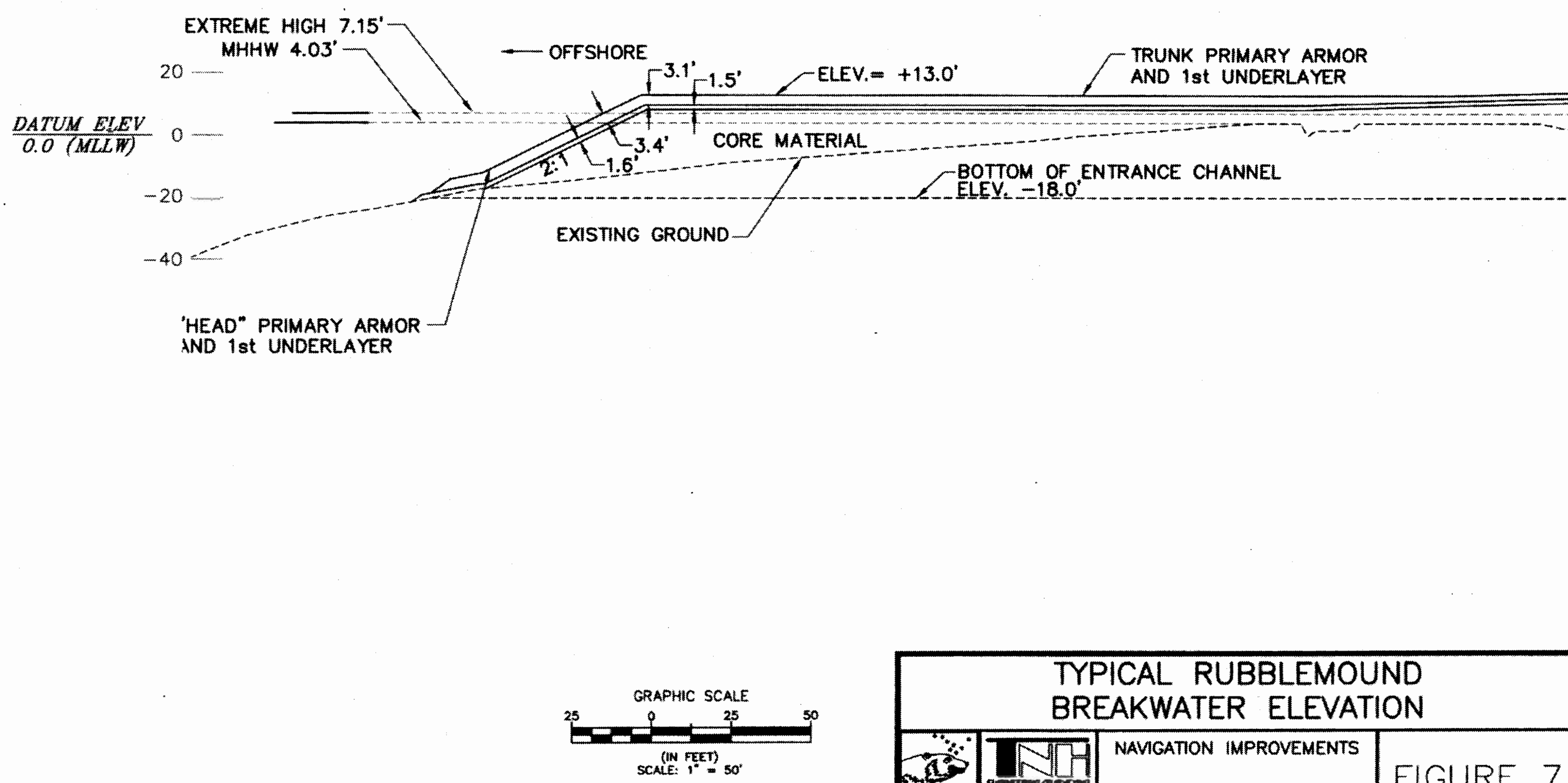
5.8.6. Upland Buildings

Buildings will likely be placed on fill. All buildings should be placed on engineered foundations. These foundations may include piles or compacted base materials.

5.9. Moorage Configuration

The moorage configuration considered for the harbor basin is a rafting type parallel moorage arrangement for the larger vessels. Large vessels are allowed to raft two deep alongside main floats. There would be no individual stall floats for vessels over 40 feet in length. Vessels under 40 feet in length will be berthed in stalls. The rafting parallel float arrangement for larger vessels will allow for more vessels per acre in the harbor. For the larger vessels, the main floats including the marginal float should be a minimum of 10 feet wide. The inner harbor float system should be constructed along the east side of the basin with the turning basin and maneuvering channel located along the west side of the basin.


24'
CREST ELEVATION 16.0'
2:1
FILL MATERIAL
ORIGINAL GROUND SURFACE
1.0
2.0
▽MHHW (ELEVATION = 4.03')
▽MLLW (ELEVATION = 0.0')
1.5' OF SLOPE ARMOR
1.0
3.0
5.0'
INNER HARBOR BASIN
ELEVATION VARIES
NATURAL FOUNDATION MATERIAL
GEOTEXTILE FABRIC
1.0' OF FILTER ROCK
GRAPHIC SCALE
10 0 5 10 20
(IN FEET)
SCALE: 1" = 20'
TYP. INNER
HARBOR ARMORED SLOPE
NAVIGATION IMPROVEMENTS
AKUTAN ALASKA
FIGURE 6

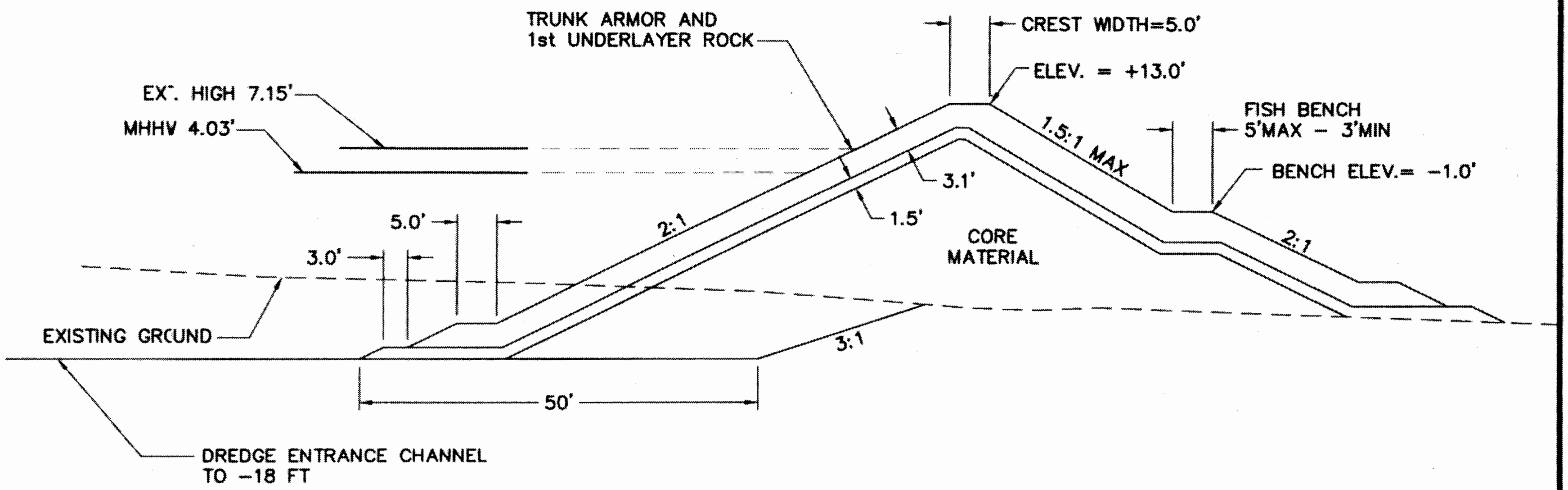


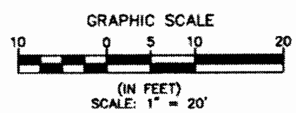
TYPICAL RUBBLEMOUND BREAKWATER ELEVATION

NAVIGATION IMPROVEMENTS

AKUTAN ALASKA

FIGURE 7





TYP. RUBBLE MOUND
BREAKWATER SECTION

NAVIGATION IMPROVEMENTS

AKUTAN ALASKA

FIGURE 8

5.10. Rubblemound Breakwaters/Jetties

All the alternatives advanced for further consideration incorporate various rubblemound breakwater components. These rubblemound breakwaters/jetty structures define and protect the entrance channel.

Methods described in the SPM and EM 1110-2-2904 were used to develop values for breakwater primary armor stone weights and dimensions, breakwater crest heights and widths, primary armor stone layer and under layer thickness, and under layer and core material sizes.

5.10.1. Rubblemound Foundation

Two rubblemound foundation design options were explored during the study. The first design places the rubblemound structure directly on existing soils. The second design utilizes an excavated buttress filled with shot rock (core material) as a foundation for the rubblemound structure. The main controlling factors in these designs are the seismic and geotechnical conditions at the site, constructability, and cost.

Rubblemound Structure on Existing Soils. The first design option places the rubblemound structure on top of existing soils. Existing soils may be vibrocompacted to decrease the tendency toward liquefaction and increase stability during seismic events. The channel is dredged an additional 50 feet wider under the breakwater toe so that more core material is placed under the breakwater armor to increase overall stability. Dredge slopes then turn upward at 3:1 to meet the existing ground surface. The advantages and disadvantages of this design are as follows:

Advantages:

- Overall ease of construction.
- Substantial cost savings in dredging and in core materials.

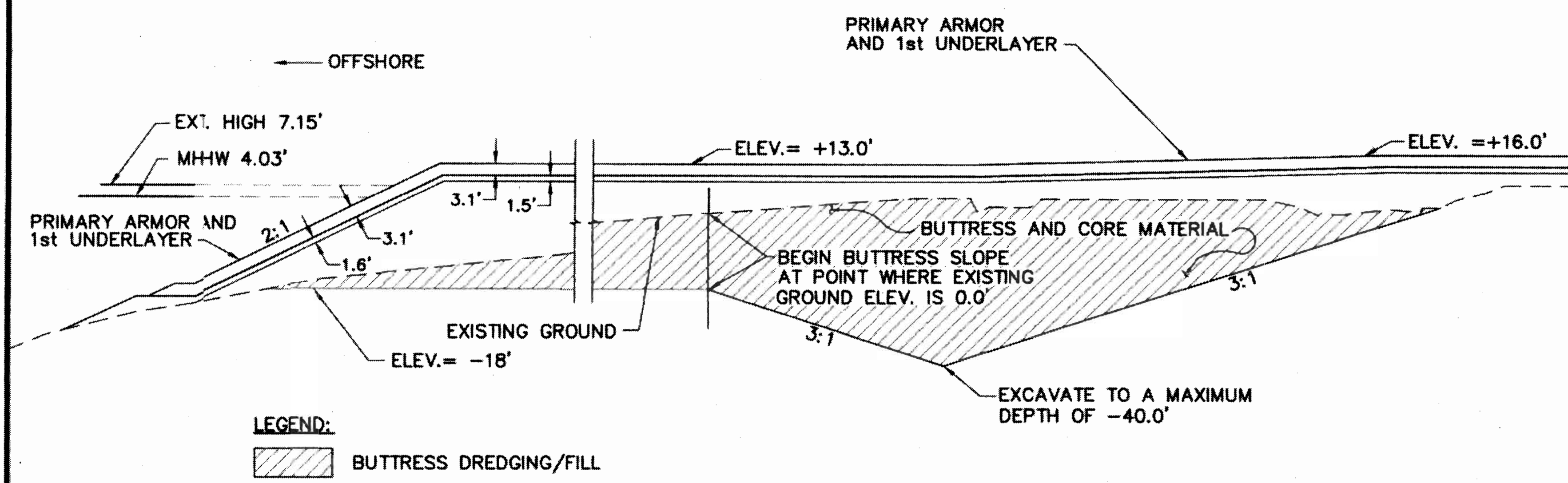
Disadvantages:

- This design is less stable during a seismic event. The seismic factor of safety for this design during the design seismic event is 0.8 (see Appendix C).

Rubblemound Structure on Excavated Buttress. The second design option calls for the construction of a buttress under portions of both breakwaters. The buttress, as defined for this project, is an excavation under the rubblemound structure that is filled with shot rock. This forms a kind of shear key into the substratum upon which the structures are built. Approximately 50,000 cubic yards of total additional dredging would be required to construct buttresses under both breakwater sections. An equal amount of shot rock/core material would be needed to fill the buttresses. The shot rock material is not susceptible to liquefaction and provides for increased foundation stability. In addition, the weight of this fill adds to the confining stress under the buttress, furthering the resistance to liquefaction. The advantages and disadvantages of this design are as follows:

Advantages:

- Seismically, the design is more stable. This design has a seismic factor of safety of 1.1.



TYP. LONGITUDINAL SECTION THROUGH BREAKWATER AND BUTTRESS

GRAPHIC SCALE
20 0 10 20 40
(IN FEET)
SCALE: 1" = 40'

RUBBLE MOUND W/BUTTRESS

NAVIGATION IMPROVEMENTS

AKUTAN ALASKA

FIGURE 9

Disadvantages:

- The design is geometrically complex and difficult to present graphically.
- The design is difficult to construct. This design calls for a large area to be over-excavation under the breakwaters with 3:1 side-slopes. The entire buttress would have to be excavated underwater down to an elevation of –40 feet MLLW.
- The cost of excavation, removal, draining, and storage of extra dredge materials, as well as the cost for extra potentially imported core material to fill the buttress drives up total project costs. A preliminary cost estimate shows the cost for buttress dredging and fill material alone is about $2 million.

Chosen Study Design Option. The design option pursued in detail in this study is option one (without the buttress). The reason is that despite a level of risk in constructing the breakwater with a seismic factor of safety of less than 1.0, the overall cost savings and ease of construction make this option the one most likely to be funded and constructed.

5.10.2. Stone Weight and Size and Layer Thickness:

Primary armor stone weight (W) was calculated using the Hudson Formula.

The Hudson formula is given as

$$W_a = \frac{\gamma_a H^3}{K_D (S_a - 1)^3 \cot \alpha}$$

Where:

W_a = weight of an individual armor stone

γ_a = unit weight of the armor unit

H = design wave height

K_D =stability coefficient

S_a =specific gravity of armor unit relative to the water $(S_a = \gamma_a / \gamma_w)$

; =angle of the structure in degrees from horizontal

Design parameters adopted for Akutan breakwater/jetty calculations include:

- A unit weight for armor stone of 165 lb/ft^3 (γ_a).
- The design wave (H_{10}) equaling 3.94 feet was used as H.
- The stone is assumed to be randomly placed rough quarry stone.
- A portion of the jetty trunk section will be in the breaking wave zone. Therefore K_D = 2.0 for the trunk section.
- The head of the jetty will use K_D = 1.6.
- A side slope of 2 horizontal to 1 vertical.

An average stone layer porosity (P) of 37% (based on randomly placed, rough quarry stone with a primary layer thickness of two armor units; i.e., n = 2) used to calculate layer thickness.

As stated above, portions of the breakwater/jetty are in the breaking wave zone. Therefore, a K_D value of 2.0 (as recommended by the SPM) is used in Hudson's formula for calculations pertaining to the trunk portion of the breakwater. The calculated primary armor stone weight for conditions present at Akutan is 640 pounds with an average stone thickness of 1.95 feet.

The head of the jetty requires special consideration due to the more dynamic wave environment. A K_D value of 1.6 was recommended by the SPM for breaking-wave conditions at the structural head. Using this K_D value, the calculated weight for the primary rock on the head of the structure was found to be 800 pounds. The average stone thickness was found to be 2.0 feet in diameter. This rock is relatively close in size to the trunk portion rock, and is in fact captured by the allowable range of rock sizes for the trunk. Therefore, for simplicity, a separate size of rock is not called out for the head of the breakwater. Special care should be taken to select larger stones within the recommended size range during construction for the head section. Table 16 provides stone weights and diameters, as well as layer thickness. Table 17 provides gradation ranges of stone weights that apply to the project.

Table 16. Stone Weights, Sizes, and Layer Thickness

Layer	Stone Weight Range (lb.)			Average Stone	Layer
	Minimum	Calculated wt.	Maximum	Diameter	Thickness (ft.)
	Breakwater Armor Rock				
Primary Armor (W) Layer	480	640	800	1.95 ft.	3.1
	Underlayer, Core, Buttress, And Filter Bed				
1st Underlayer (W/10)	45	64	83	10.8 in.	1.5
Core (W/200)	0.96	3.2	5.4	4.5 in.	Varies
Buttress (W/200)	0.96	3.2	5.4	4.5 in.	Varies
Filter	$D_{15\ (filter)}$ [5 x $D_{85\ (foundation)}$			N/A	3.0

Table 17. Stone Gradation

Rock Size	Rock Size Gradation Limits (%)
W	125 to 75
W/10	130 to 70
Core/Buttress	170 to 30
Filter	170 to 30

As shown above, the acceptable gradation for primary armor stone is considered to be ∀25% by weight. Therefore, the structure trunk primary stone may range from 480 lbs to 800 lbs. Other stone weights follow the same range methodology.

5.10.3. Crest Elevation and Overtopping:

Crest height was set to minimize overtopping of the structure by extreme wave events. A small amount of overtopping can be acceptable if it does not cause damage by waves or other effects behind the structure. The crest elevation is dependent upon several factors including; high tide, storm surge, wave setup, and wave runup.

The high tide, storm surge, and wave setup values were previously discussed in relation to the design still high water level. To recap, extreme high tide was found to be 7.15 feet, storm surge was found to be 0.2 feet, and set up was found to be 0.38 feet. These combine to form the design still high water level of 7.73 feet.

Runup is dependent on several factors including wave characteristics, design water level, structure depth, structure slope, and the roughness of the material. Wave runup calculations were performed using methods outlined in the SPM. Calculations were based on the following parameters:

- A deep water wave height (H_o) of 3.94 feet.
- 2 to 1 slopes.
- Randomly placed quarry stone with a roughness correction of 0.55.
- Depth at the toe of the structure (extreme high water) 27.73 feet.

Calculations were performed using several tables in the SPM. The results of the calculations show a wave runup of 4.92 feet from the still water level. Adding the runup value of 4.92 feet to the design high water level of 7.73 feet yields 12.65 feet. This value has been rounded to a practical value of 13.0 feet.

5.10.4. Crest Width

Crest width calculations were determined using methods from the SPM. Crest width is found by applying a formula, which takes into account overlap and meshing of the individual armor units. The SPM states that the minimum crest width should be based on the width of 3 armor stones. With nesting and overlap, the result is a crest width somewhat less than the sum of the individual armor unit widths.

$$\beta = 3k_{\Delta}\left[\frac{W_a}{\gamma_a}\right]^{1/3}$$

Where:

β = The crest width in feet.

k_{Δ}= The layer thickness coefficient, dimensionless.

W_a = Weight of individual armor units, pounds.

γ_a = Unit weight of armor, pounds per cubic feet.

For this project the layer thickness coefficient was 1.0 as outlined in the SPM for randomly placed rough armor stone. The weight of the armor unit was taken to be 640 pounds, and the unit weight of the armor rock was 165 pounds per cubic feet. These values lead to a calculated crest width of 4.7 feet. This value has been rounded up to the reasonable minimum crest width value of 5.0 feet.

Another rule of thumb is that the breakwater crest should be wide enough to allow access for construction equipment if possible. This access does not have to be at the final elevation of 13 feet, but must be set for a reasonable water level condition. Given a 2 to 1 side slope, there would be a useable base approximately 17 feet wide at a construction elevation of +10 feet. This elevation would be well above the high water level for all but the most extreme events.

6.0 ALTERNATIVES CONSIDERED

6.1. Introduction

A wide range of sites and alternatives were considered for navigation improvements in Akutan Harbor. These included many alternatives examined in previous study efforts.

6.2. No Action

Under a no action scenario, there would continue to be no permanent moorage facilities in Akutan. Larger vessels would travel to other areas or ports for long-term moorage. Small vessels would continue to be pulled out of the water when not in use.

6.3. Alternative Sites

Akutan Harbor contains a number of sites that could be potential harbor locations. A site selection drawing for the various sites in Akutan Harbor is presented in figure 10. This drawing basically subdivides and delineates the entire bay into various regions for discussion.

In previous studies, several areas of the bay have been summarily dismissed as potential harbor locations due to high ambient wave climates, steeply sloping upland and/or offshore terrain, difficult access issues, sensitive environmental areas, or other concerns. What follows is a brief narrative description of the site locations. Table 18 outlines the advantages and disadvantages of each of the site locations in Akutan Harbor.

6.3.1. Akutan Point

Akutan Point is located about 1.85 miles east of the community of Akutan. The site contains a small cove just to the southwest of and in the lee of the point. The uplands are steeply sloping and various birds use local sea cliffs for roosting and nesting. The area is used for the placement of subsistence set nets by the locals. A small pocket beach and offshore deposit exists south of the cove. This results in relatively shallow waters in the cove area.

6.3.2. North Shore Area 1

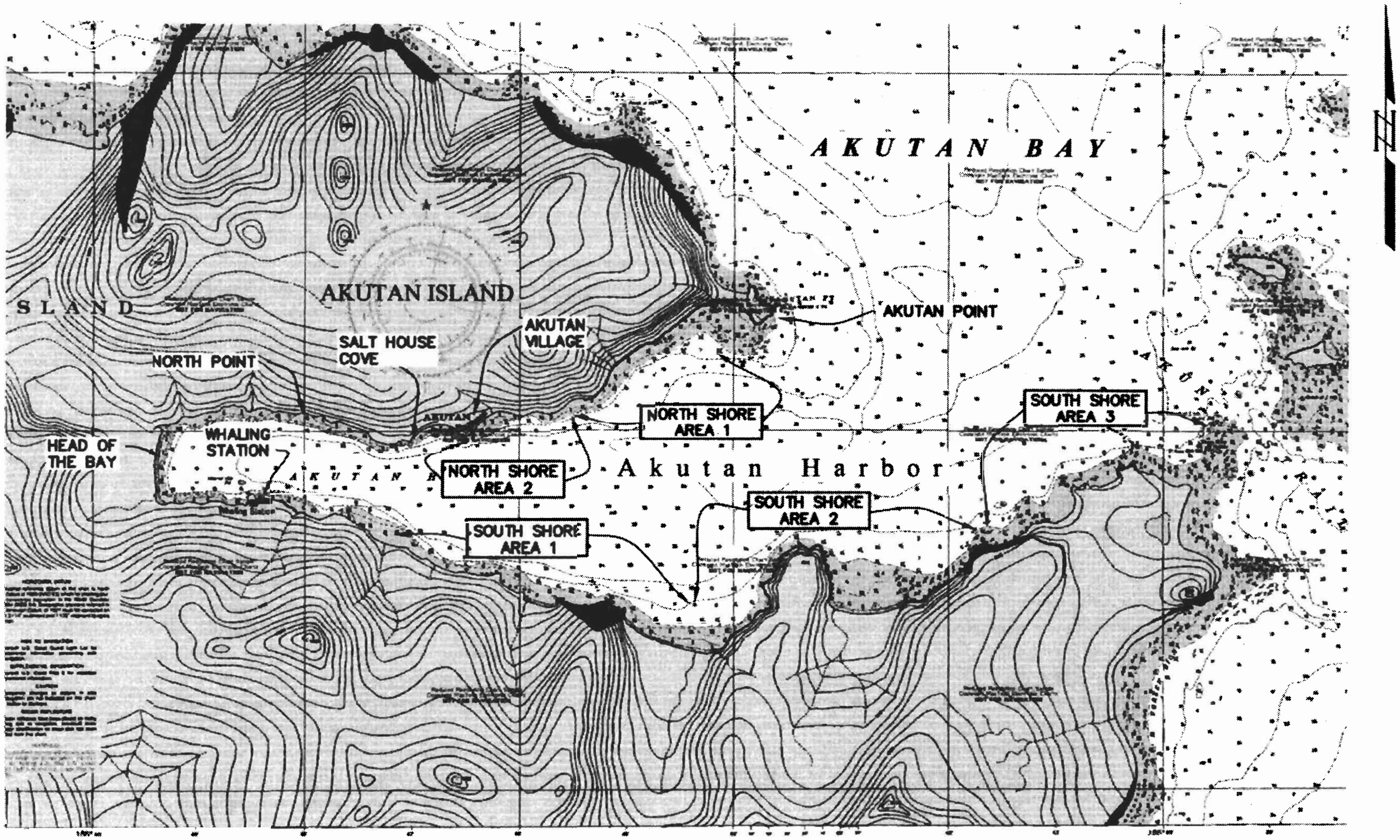
North Shore Area 1 is located about 1.4 miles east of the community of Akutan. The site is bordered by steeply sloping bluffs on the upland side. A relatively shallow bench with depths of about 25 feet extends offshore for approximately 400 feet. From there the bottom drops of rapidly to depths of 60 feet or more.

6.3.3. North Shore Area 2

North Shore Area 2 is located about ½ mile east of the community of Akutan. The site is bordered by steeply sloping upland terrain and relatively deep water (90 feet deep approximately 400 feet offshore).

6.3.4. Salthouse Cove

Salthouse Cove essentially separates the native village of Akutan from the Trident Seafood facility. The Trident facility is located on the western shore of the cove and the native village



GRAPHIC SCALE

2,500 0 2,500 5,000

(IN FEET)
1" = 5,000'

AKUTAN HARBOR
SITE SELECTION

HARBOR FEASIBLIITY STUDY

AKUTAN ALASKA

TRYCK NYMAN HAYES, INC

FIGURE 10

is located of the eastern shore. The eastern corner of Salthouse Cove contains the City dock and seaplane landing facility. This area currently receives regularly scheduled seaplane, state ferry, fuel barges, and other services. A church and gymnasium owned by the Trident facility overlooks the center of the cove.

6.3.5. North Point

The North Point site is located just west of the Trident plant. The site is bordered by steeply sloping upland terrain and relatively deep water (80 feet deep approximately 500 feet offshore). Four submerged HDPE pipelines carry water to the Trident Plant from a dam located high on a hillside. The east end of the site is bordered by a sheet pile wall related to the Trident plant.

6.3.6. Head of the Bay

The head of the bay is characterized by a gently sloping sand beach. It is located about 1.75 miles west of the Trident plant. The beach contains an elevated and vegetated sand berm that separates it from the mostly flat lowland behind it. The uplands extend up into a broad U shaped valley. The valley is defined by steeply sloping uplands with two creeks at the margins. Both these creeks support runs of fish.

6.3.7. Whaling Station

The Whaling Station was constructed in 1912 by the Pacific Whaling Company and was operated until 1942. It is located in the southwest corner of the bay. The land is now privately owned and is used for gear storage by fishermen and the Trident plant. The site has been previously classified as contaminated by the Federal Government and was cleaned up as part of a FUDS program TERC contract.

6.3.8. South Shore Area 1

South Shore Area 1 extends from the area just east of the Whaling Station to a point near the mouth of Akutan Harbor for a distance of about 2 miles. It is characterized by steeply sloping on shore terrain and relatively deep offshore bathymetry. There is a large landslide area near the east end of this section. South Shore Area 1 receives a lot of wave energy from Akutan Bay to the northeast. There is little developable uplands and poor access to the site.

6.3.9. South Shore Area 2

South Shore Area 2 includes the area just west of a small peninsula near the mouth of Akutan Harbor. It is located about 2-1/2 miles from the Whaling Station. The site is characterized by a slight cove like feature that results in an offshore bench. South Shore Area 2 receives a lot of wave energy from Akutan Bay to the northeast. There are some developable uplands, but poor access to the site.

6.3.10. South Shore Area 3

South Shore Area 3 includes the area just east of a small peninsula near the mouth of Akutan Harbor. This area is outside the Akutan Harbor area. The site is characterized by a slight pocket beach resulting in an offshore bench. South Shore Area 3 is exposed to the full fetch and resultant wave energy from outside of Akutan Harbor to the north and east. There are some developable uplands but poor access to the site.

Table 18. Potential Harbor Sites

Location	Advantages	Disadvantages
Akutan Point	Relatively sheltered from north and west storms. Relatively shallow water could accommodate cost effective rubblemound breakwater structure.	Excessive distance from community and Trident facility. Access will require motor vessels for access to and from the community. Exposed to some long period waves from Bering Sea. Facility will be difficult to operate and to maintain due to distance from community. Environmentally sensitive area due to bird habitat, kelp beds, and other marine habitat. May be exposed to large ocean swells from the southerly direction or exposed to reflected waves. Not a local preference.
North Shore Area 1	Relatively sheltered from north and west storms.	Excessive distance from community and Trident facility. Access will require motor vessels for access to and from the community. Exposed to some long period waves from the Bering Sea. Facility will be difficult to operate and to maintain due to distance from community. May be exposed to large ocean swells from the southerly direction or exposed to reflected waves. Relatively deep water offshore, which limits the type of construction and effects cost. Not a local preference.
North Shore Area 2	Relatively sheltered from north and west storms. Relatively close to existing Trident facility and to the community of Akutan.	Exposed to some long period waves from the Bering Sea. May be exposed to large ocean swells from the southerly direction or exposed to reflected waves. Relatively deep water offshore, which limits the type of construction and effects cost. Not a local preference.
Salthouse Cove	Relatively close to existing Trident facility and to the community of Akutan. Relatively good natural wave protection.	Site too close to the existing community, ferry dock and seaplane ramp. Harbor would dominate the bay and adversely impact the quality of life in the community. Exposed to some long period waves from the Bering Sea. Limited potential for upland development. Not locally preferred.
North Point	Relatively close to existing Trident facility and to the community of Akutan. Relatively good natural wave protection. Locally preferred site by Trident and the community of Akutan.	Deep water offshore, which limits the type of construction and effects cost. Limited area for upland development. Access to site from community will have to be through the Trident plant lands. Shallow bedrock near shoreline and in uplands will likely result in rock excavation as part of construction.
Head of the Bay	Good upland area. Shallow water will support efficient and cost effective construction methods. Good natural protection from north and	Uplands contain two fish bearing streams and some wetlands. Relatively long distance from both the community

Location	Advantages	Disadvantages
	south directions.	and from the Trident plant.
	Relatively good natural wave protection.	
	Upland is owned by Akutan Corporation, which may stimulate local economic development.	
	Locally preferred option.	
Whaling Station	Naturally protected from southeast and west directions.	Long distance from Trident plant and community of Akutan
	Relatively good natural wave protection.	Possible contaminated soils requiring further remediation and cleanup.
	Land area classified for industrial use after TERC cleanup is complete.	Relatively deep water offshore, which limits the type of construction and affects cost.
	Some uplands development possible.	Not locally preferred.
	Historical industrial use could mean less challenging environmental concerns.	
South Shore Area 1	Little identified environmental concerns.	Unacceptable wave climate.
	Probable good water quality and mixing.	Limited area for upland development.
		Long distance from Trident plant and community of Akutan.
		Relatively deep water offshore, which limits the type of construction and affects cost.
		Not locally preferred.
South Shore Area 2	Little identified environmental concerns.	Unacceptable wave climate.
	Some possible upland area.	Long distance from Trident plant and community of Akutan.
	Relatively shallow water will support efficient, cost effective construction methods.	Not locally preferred.
South Shore Area 3	Little identified environmental concerns.	Unacceptable wave climate.
	Some possible upland area.	Long distance from Trident plant and community of Akutan.
	Relatively shallow water will support efficient, cost effective construction methods.	Not locally preferred.

6.4. Project Development

A July 1998 preliminary site assessment report ("*Akutan Harbor Feasibility Study, Phase 1, Preliminary Site Assessment Report,*" July 13, 1998) examined five harbor site alternatives as Phase I of this study. These sites were North Point (west of the Trident facility), Akutan Point, Salthouse Cove, head of Akutan Harbor, and the Old Whaling Station. The study recommended that the North Point site and the head of the bay site be considered for further feasibility studies.

In October 1998, after the Phase I report was issued, a field exploration program was undertaken that included upland and bathymetric survey, geotechnical explorations and environmental sampling. This effort focused on the North Point site, which appeared then to be the most economically viable and least environmentally sensitive alternative at the time. A lower priority data collection effort advanced at the head of the bay site.

Subsequent study revealed that the North Point site had limitations because of a steeply sloping onshore and offshore terrain. This constrained the conceptual harbor to a long thin rectangular shape with little uplands. With these constraints, only about a 9-acre basin could be considered for that site. Economic analysis led to the conclusion that the North Point basin was not large enough to accommodate enough vessels to generate sufficient benefits to justify the project. The focus of the project then shifted to the head of the bay site.

Initial examinations of the head of the bay site were focused on three alternatives:

- An offshore harbor.
- An onshore/offshore harbor.
- A dredged inland harbor.

Conceptual designs were advanced for these three alternatives. During conceptual design, a potential problem with earthquake-induced liquefaction was uncovered. This problem was due to the saturated sands present at the head of the bay. Geotechnical engineers suggested several slope stabilization techniques that could be used to deal with this potential problem. These techniques, such as an excavated buttress under the breakwaters, generally increased estimated project costs.

After examining the three design alternatives, conceptual cost estimates and economic evaluation pointed to the dredged inland basin as being the most economically feasible alternative. The design team then advanced several versions of the inland basin. These versions include a 12-acre basin, a 15-acre basin, and a 20-acre basin. By varying the size of the basin, different portions of the overall fleet could be serviced. Also, different overall costs and benefits could be compared.

As the study moved forward, it became apparent that insufficient geotechnical and survey data had been collected at the head of the bay. The reason for this is that initially the head of the bay was not seen as the likely final project location and so data collection there was limited. The soils data collected up to that point consisted of two offshore borings near the head of the bay, but no borings in the upland area where the dredged basin was to go. In addition, survey data extended only a few hundred feet inland and was not sufficient to produce an accurate topographic map of the project site. Offshore bathymetry data was lacking as well.

In order to make an assessment of slope stability and obtain accurate dredge material quantities more geotechnical and survey data would be needed. In August 2000, the U.S. Army Engineer Research and Development Center, Waterways Experiment Station (ERDC-WES) conducted a hydrogeology and wetlands delineation study at the head of the bay in support of the project. This work included some GPS based uplands survey, which was subsequently made available to the study team.

During the ERDC-WES fieldwork, several abandoned 55-gallon drums were discovered near the beach berm. The presence of these drums raised concerns related to possible contaminated soils.

Two reports were produced summarizing the results of these ERDC-WES investigations. These include:

"*Delineation of Wetlands on the Proposed Site of the Akutan Harbor Project, Akutan Island, Alaska,*" by James Wakeley, ERDC May 2001.

"*Hydrogeology of Proposed Harbor Site at Head of Akutan Bay, Akutan Island, Alaska,*" by Joseph Dunbar, Maureen Corcoran, and William Murphy ERDC-WES, July 2001.

Subsequent to the site visit by ERDC-WES, another geotechnical field investigation program was mobilized to the site. This work was done in March 2001. The purpose of this work was to advance geotechnical borings in the upland area of the proposed new basin, and to perform an environmental site investigation.

These site investigations are summarized in the following reports:

"*Geotechnical Report, Akutan Small Boat Harbor, Akutan Alaska,*" by Shannon & Wilson, June 2001 (included in the Appendix)

"*Draft Environmental Site Investigation Report, Proposed Harbor Location, Akutan Alaska,*" by Shannon & Wilson, July 2001 (included in the Appendix)

In September 2002 the draft feasibility reports were released. These reports underwent a thorough agency review and a number of comments and concerns were advanced. Items effecting the design included concerns over circulation and as a result water quality in the new basin, and the overall footprint associated with the stockpile area. In response to comments the study team completed a numerical circulation model of the basin, redesigned the entrance channel and a portion of the harbor perimeter to improve circulation, and reconfigured the stockpile area to minimize impacts. The results of this work are shown in the "reconfigured 12 acre basin" drawing in this report.

6.5. Harbor Alternatives Considered in Detail

As stated previously, three primary alternatives have been advanced for study at the head of the bay including:

- An offshore harbor.
- An onshore/offshore harbor.
- A dredged inland harbor.

These three alternatives constitute the primary alternatives examined in this study.

6.5.1. Offshore Harbor

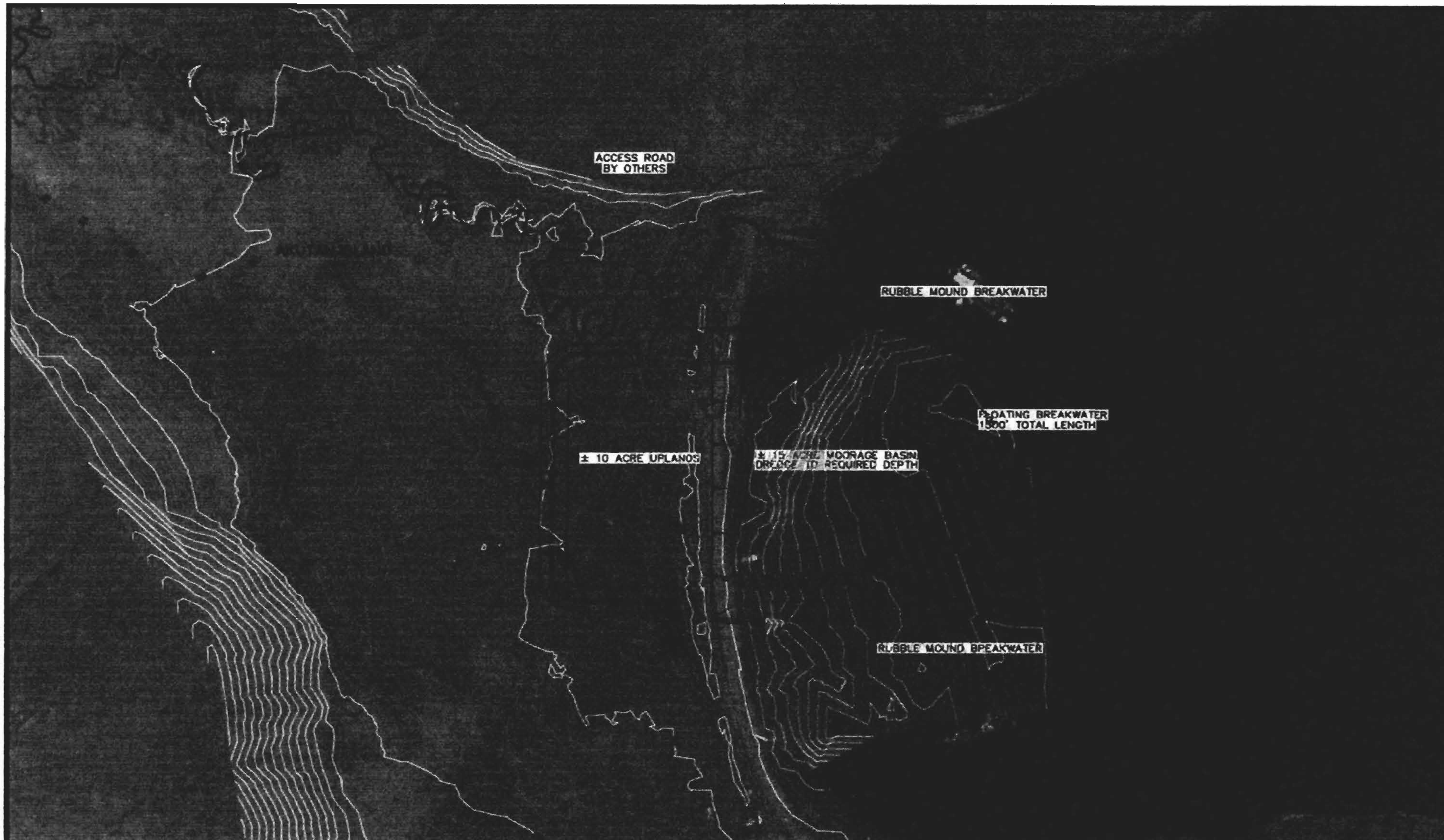
An offshore harbor concept was advanced through the use of a floating breakwater. In this alternative, a floating breakwater, approximately 2,000 feet long, would be anchored near the head of the bay to provide protected moorage. In this alternative, most of the moorage area of the harbor would be offshore with some portion of the existing shoreline area developed for related upland facilities and access. The offshore harbor concept is shown in figure 11.

Floating breakwaters work principally by reflection. They are required to be a significant portion of the incident wavelength in width to be effective. The closer to a width of 50 percent of the incident wavelength, the better the performance will be. Generally, floating breakwaters are used in limited fetch areas that are subjected to waves of less than a 4 second period and a wave height of 4 feet or less. This type of wave climate is generally found in relatively short fetches. The period of the design wave for this project is 4.7 seconds. The height of the design wave (H_{10}) is 3.9 feet. The deep-water wavelength associated with a 4.7 second period is about 113 feet. This points to a floating wave barrier with a width approaching 50 feet.

Conceptually, a barge-like structure with a width of approximately 40 feet could work. A number of these could be linked together and anchored in relatively deep water to form an offshore wave barrier. There are a number of disadvantages associated with this type of structure. Maintenance and inspection could be more frequent and involved than with other structures. This is primarily due to the mooring chain and fixtures that would require frequent periodic inspection.

Another consideration is cost. A steel structure 1,500 to 2,000 feet long, 40 feet wide, and 15 feet deep would cost about $16 million for the fabricated structure alone. There would still be some dredging required and a short breakwater section may have to be constructed. Add to this the costs of towing, moorage chain, anchors, and installation, and the costs for a floating wave barrier alone could exceed $20 million. Another consideration is the risk that a portion of the floating breakwater may come loose from its anchorage due to a broken mooring chain or failed anchor. Because of the remoteness of Akutan Harbor, emergency repairs would be difficult and costly.

Based on the above discussion, a floating breakwater while technically possible is not practically feasible for this project.



GRAPHIC SCALE
250 0 125 250 500
(IN FEET)
SCALE: 1" = 500'

OFFSHORE HARBOR

HARBOR FEASIBLIITY STUDY

AKUTAN ALASKA

TRYCK NYMAN HAYES, INC.

FIGURE 11

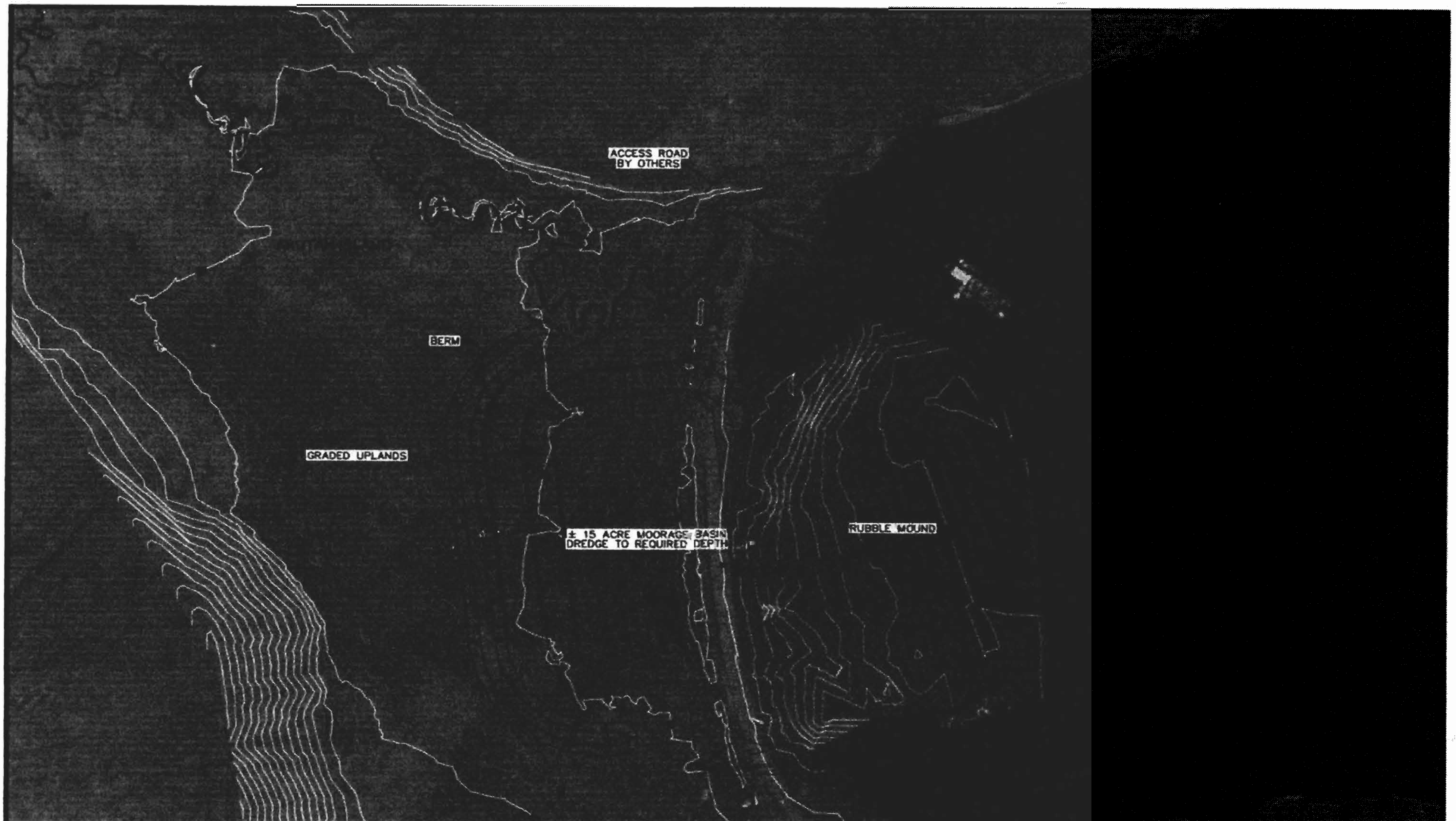
6.5.2. Offshore/Onshore Harbor

A concept was advanced for a harbor basin dredged partially inland. Two alternative methods were examined for the offshore breakwater portion of this concept: a rubblemound structure, and a curtain-wall wave barrier.

Offshore/Onshore Harbor; Rubblemound. The rubblemound version of the onshore/offshore harbor would include a rubblemound as the offshore breakwater component of the basin. The rubblemound would be placed in approximately 25 feet of water and would be approximately 1,100 feet long. This is near the maximum economic practical depth normally associated with this type of structure. (At depths over about 25 to 30 feet the costs of rubblemound breakwaters increase dramatically.) The centerline of this breakwater would be about 100 to 150 feet offshore from the existing beach. Figure 12 shows the conceptual design of the rubblemound option.

As previously discussed, breakwater construction materials are a main component contributing to the total project cost. This alternative greatly increases the amount of armor rock, secondary rock, and core material needed for the project. In addition, the added breakwater length may necessitate the need for a buttress foundation. A conceptual cost of over $4 million was calculated for the buttress alone for this alternative. The costs of this alternative outweigh any anticipated per acre benefits.

Offshore/Onshore Harbor; Curtain-wall Wave Barrier. The curtain-wall wave barrier version of the onshore/offshore harbor would include a curtain-wall wave barrier as the offshore component of the basin. The curtain-wall wave barrier would be placed in about 60 feet of water and would be about 1000 feet long. The wave barrier would be placed about 350 feet offshore from the existing beach. It would be a pile-supported structure consisting of wave barrier panels that extend a distance below the water level but not necessarily all the way to the bottom. There would be a section of rubblemound jetty about 450 feet long that traverses the breaking wave zone and connects the wave barrier to the beach on one side. Figure 13 shows a conceptual design for the offshore/onshore wave barrier option.



GRAPHIC SCALE
250 0 125 250 500
(IN FEET)
SCALE: 1" = 500'

OFFSHORE/ONSHORE RUBBLE MOUND

HARBOR FEASIBLIITY STUDY

AKUTAN ALASKA

FIGURE 12

GRAPHIC SCALE
250 0 125 250 500
(IN FEET)
SCALE: 1" = 500'

OFFSHORE/ONSHORE WAVE BARRIER

HARBOR FEASIBLIITY STUDY

AKUTAN ALASKA

FIGURE 13

TRYCK NYMAN HAYES, INC

Curtain-wall wave barriers are similar to floating breakwaters in that they are ideally suited to shorter period, small amplitude waves. They work best in wave periods less then 4 seconds and in wave heights less then 4 feet. Again, the period of the design wave for this project is 4.7 seconds. The height of the design wave (H_{10}) is 3.94 feet.

Using Wiegle's method for power transmission under a wall, a barrier depth of about 30 feet below MLLW would provide an inner harbor wave height of less than one foot at extreme low tides. The panel would be required to extend above MHHW to account for runup. An estimated elevation of +12 feet above MHHW would be required to minimize over topping. At 1000 feet long, the wave barrier would include about 42,000 square feet of panels.

The pile-supported structure could work well in the liquefaction prone soils. The piles could simply be driven deep enough to remain unaffected by any loss of support by the upper layer of soil.

The costs associated with the wave barrier could be about $150 per square foot of barrier panel. This leads to an estimated cost of about $6.3 million for the wave barrier alone. This combined with the rubblemound jetty sections and other structures and features make the cost of this alternative outweigh any anticipated per acre benefits.

6.5.3. Inland Harbor

Initial cost estimates indicate that a dredged inland basin is the most economic alternative. The inland harbor is also among the most environmentally acceptable alternatives. Various layouts and sizes were examined. The same design criteria outlined below was applied to all inland alternatives.

Examining the various basin sizes resulted in an optimization study for the size of the dredged basin at the head of the bay site. The primary factors that entered into the optimization were the following:

- The size of the fleet that could be serviced.
- The benefits that could be generated by the fleet.
- The construction costs.
- The environmental impacts and associated costs.

Orientation. Two primary basin orientations were examined; the long axis of the basin aligned east/west, and the long axis aligned north/south. Orientating the long axis of the harbor basin east/west and centering it in the valley provides for the advantage of maximizing separation distance from the two streams. The disadvantages of this orientation include:

- Alignment of the entrance channel with the long axis of the harbor allowing for more direct communication with the offshore wave environment,
- orienting the basin so that it runs northward into the valley may cause drainage and/or drainage diversion problems due to natural surface drainages concentrated in this area,
- upland areas would likely have to be split north and south of the basin due to the back of the basin butting up against steep inland terrain, and

- the more space-efficient moorage arrangement forces vessels to be moored broadside to the wind under a rafting type moorage arrangement.
- Due to these disadvantages, basins with orientations that aligned the long axis of the harbor north/south were chosen for the alternatives advanced in this study.

Inland Basin Design Alternatives. The inland basin design alternatives that were advanced in this study essentially utilized the same rectangular basin shape with the long axis oriented north/south. The main difference between the alternatives is the size of the basin. The three basin sizes examined were a 12-acre basin, a 15-acre basin, and a 20-acre basin. All the alternatives share the same entrance channel configuration and depth (–18 feet below MLLW). Each of the alternatives will have three primary basin bottom depths of –18 feet, –16 feet, and –14 feet below MLLW to accommodate various vessel sizes. The ratio of length-to-width (aspect ratio) for the basins varies from 1.3 to 1.6. Aspect ratios of 0.5 to 2.0 are recommended for adequate harbor circulation and flushing.

As mentioned previously, the original basin design was modified in response to comments received on the draft report to include features to improve circulation and to minimize the footprint associated with the stockpile area. The modifications include changes to the entrance channel, rounding of the perimeter and steeper inner harbor slopes above the waterline.

Dredging Material Stockpile Area. All of the inland basin alternatives advanced in this study generate a considerable amount of dredge materials. Several local projects may be able to make good use of this as fill. These projects include a currently planned access road and a potential airfield/runway. The potential reuse of the material is dependent on the type of material that exists at the site. Geotechnical data collected at the site indicates that the dredged material would consist mostly of coarse to fine grained sands. This implies that, once drained, the dredged material would be suitable for use in construction of the upland areas and as a sub-base material for an access road or airstrip.

The stockpile areas are located to minimize the environmental impact to natural upland areas and allow access to dredged materials so that they can be used on this and other project sites easily. It should be noted that the size of the stockpile for all the alternatives is significant. Increasing the size of the stockpile may have a direct environmental impact on existing upland areas at the site, and therefore, an impact on associated project environmental and permitting costs. Concept stockpile heights aboveground range from about 25 to 50 feet at the tallest points, this equates to substantial loading on existing subsurface soils. Due to the existing soil being well draining sands, effects should be limited to localized immediate settlement. Estimated stockpile areas and volumes for each of the inland alternatives are shown in table 19.

Table 19. Estimated Stockpile Areas and Quantities

Basin	Stockpile Area (Acres)	Stockpile Quantities (Cubic Yards)
12 Acre Inland	36	850,000
15 Acre Inland	38	990,000
20 Acre Inland	39	1,175,000
Reconfigured 12 Acre	28.5	843,000

Environmental Concerns. Environmental agencies have expressed concerns on the size and location of the stockpile associated with the dredged inland basin. Several suggestions have been proposed to minimize the effect of these areas and the basins on the environment including:

- Place the stockpiles on areas intended for future developed uplands. Typically 40% of the total harbor developed area is dedicated to useable uplands.
- Confine the harbor basin and stockpile area to the southern two thirds of the head of the bay existing uplands. This is the area south of a small drainage that runs through the north valley and effectively separates the northern and southern portions of the uplands.

If the above-mentioned drainage must be affected by construction, its channel must be approximately reconstructed in plan form and cross section to the north of the construction site. It is expected that each of the alternatives will require that the drainage be moved.

All of the inland alternatives attempt to mitigate these environmental concerns.

Alternative 1–12 Acre Dredged Basin. The 12-acre dredged basin alternative is the smallest of the inland alternatives. Approximately 36 acres of uplands can be created with the associated dredge materials. The fleet associated with this harbor is shown in table 20.

Table 20. 12-acre Basin Fleet

Vessel Length (ft)	Number
0–24	10
24–32	10
32–90	0
90–110	8
110–120	15
120–155	13
155–180	2

The 12-acre basin will be dredged to varying depths to accommodate different vessel sizes. These depths, and their associated dredge areas, are outlined in table 21.

Table 21. 12-acre Basin Depths and Areas

Basin Depth (ft)	Acres
–14	2.6
–16	4.0
–18	5.4

As this is the smallest basin alternative, the 12-acre basin has the least environmental impact on the upland area at the head of the bay. The total basin toe-to-toe dredge area, including the entrance channel is 15.3 acres. 79 percent of the total dredge area is dedicated to the mooring basin. The associated concept stockpile covers approximately 36 acres and has a constant crest elevation of +35 feet above MLLW. This equates to a stockpile height above ground of approximately 25 feet at its tallest point. It is likely that some redirection will be required of the small drainage that separates the north and south portions of the uplands. A plan view of the 12-acre alternative is shown in figure 14.

Alternative 2–15 Acre Dredged Basin. The 15-acre dredged basin alternative is the mid-size of inland alternatives. Approximately 38 acres of uplands can be created with the associated dredge materials. The fleet associated with this basin is outlined in table 22.

Table 22. 15-acre Basin Fleet

Vessel Length (ft)	Number
0–24	10
24–32	10
32–90	0
90–110	8
110–120	20
120–155	18
155–180	2

The 15-acre basin will be dredged to varying depths to accommodate different vessel sizes. These depths, and their associated dredge areas, are outlined in table 23.

Table 23. 15-acre Basin Depths and Acres

Basin Depth (ft)	Acres
–14	4.0
–16	5.4
–18	5.6

The total toe-to-toe dredge area is 17.6 acres. 86 percent of the total dredge area is dedicated to the mooring basin. (Note that the percentage of area devoted to the mooring basin increases as the total project size increases. This is due to the fact that the larger mooring areas in the 15-acre and 20-acre alternatives allow for sufficient maneuvering space. The 12-acre alternative includes slightly more additional area to allow extra maneuvering room). As expected, the 15-acre basin has a larger stockpile area than the 12-acre basin and less remaining upland areas to place the stockpile on. The concept stockpile footprint area is 38 acres with a crest elevation increased by 5 feet to +40 feet above MLLW. This equates to a stockpile height aboveground of approximately 30 feet at its tallest point. The 15-acre alternative is shown in figure 15.

Alternative 3–20 Acre Dredged Basin. The 20-acre dredged basin alternative is the largest of the inland alternatives examined. Approximately 39 acres of uplands are covered with the associated dredge materials. The fleet associated with this basin is outlined in table 24.

Table 24. 20-acre Basin Fleet

Vessel Length (ft)	Number
0–24	10
24–32	10
32–90	0
90–110	8
110–120	22
120–155	23
155–180	7

The basin will have three primary depths to accommodate various vessel sizes. These are outlined in table 25.

Table 25. 20-acre Basin Depths and Acres

Basin Depth (ft)	Acres
–14	6.0
–16	6.7
–18	7.3

The total toe-to-toe dredge area of the 20-acre basin is 21.8 acres. This equates to about 92 percent of the total dredged area being devoted to mooring basin. In the case of the 20-acre basin, the basin combined with the stockpile area is large enough, both in volume and plan area, to begin to dominate the topography at the head of the bay and up into the north valley. Under this alternative, the basin and stockpile area will use a significant portion of the available uplands. The stockpile storage area is reduced significantly by the size of the basin. This makes reduced upland storage areas contain larger quantities of dredge material. The result is a stockpile that must now encroach into areas further up the north valley. The footprint area of this concept stockpile is 39 acres and the constant crest elevation is +50 feet above MLLW. This equates to a maximum stockpile height aboveground of 40 feet at its tallest point. The 20-acre alternative has the advantage of being able to service the entire design fleet. However, the size of the project footprint may incur additional environmental and permitting costs. The 20-acre alternative is shown in figure 16.

Reconfigured 12 Acre Dredged Basin. Approximately 28.5 acres of uplands are covered with the associated dredge materials. The fleet associated with this basin is outlined in table 26.

Table 26. Reconfigured 12-acre Basin Fleet

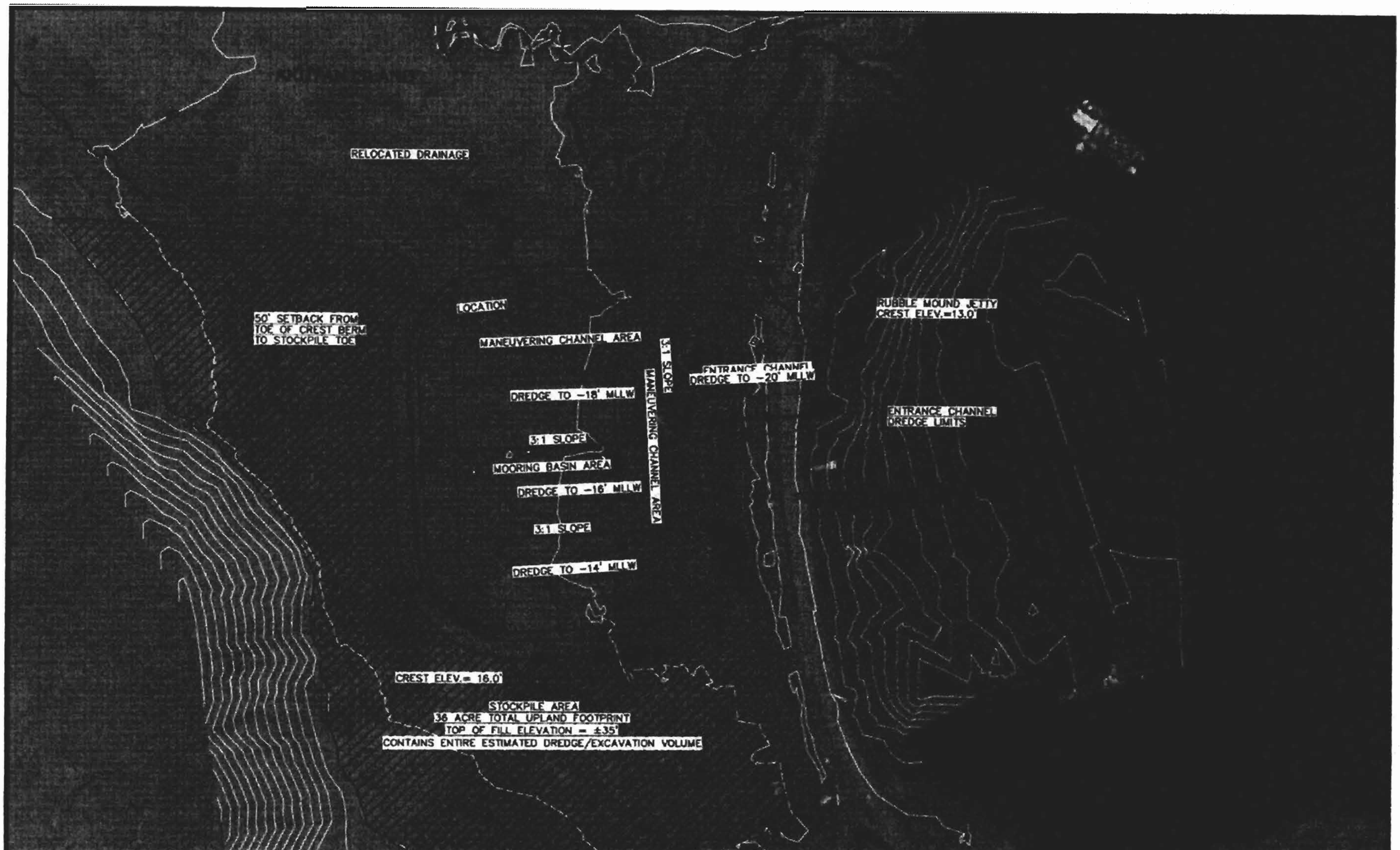
Vessel Length (ft)	Number
0–24	10
24–32	10
32–90	0
90–110	8
110–120	15
120–155	13
155–180	2

The basin will have three primary depths to accommodate various vessel sizes. These are outlined in table 27.

Table 27. Reconfigured 12-acre Basin Depths and Acres

Basin Depth (ft)	Acres
–14	2.6
–16	5.4
–18	2.4

The total toe-to-toe dredge area (mooring basin and entrance channel) of the reconfigured 12-acre basin is 16.2 acres. The footprint area of the stockpile is 28.5 acres and the maximum crest elevation is +50 feet above MLLW. This equates to a maximum stockpile height aboveground of 40 feet at its tallest point. The reconfigured 12-acre alternative is shown in figure 17.



NOTES

TOTAL DREDGE VOLUME = 850,000 CY
BASIN AREA = 12.7 ACRES
ENTRANCE CHANNEL AREA = 2.6 ACRES
(AREA MEASUREMENTS AT TOE OF EXCAVATION)
TOTAL HARBOR PROJECT AREA = 15.3 ACRES

GRAPHIC SCALE

200 0 100 200 400

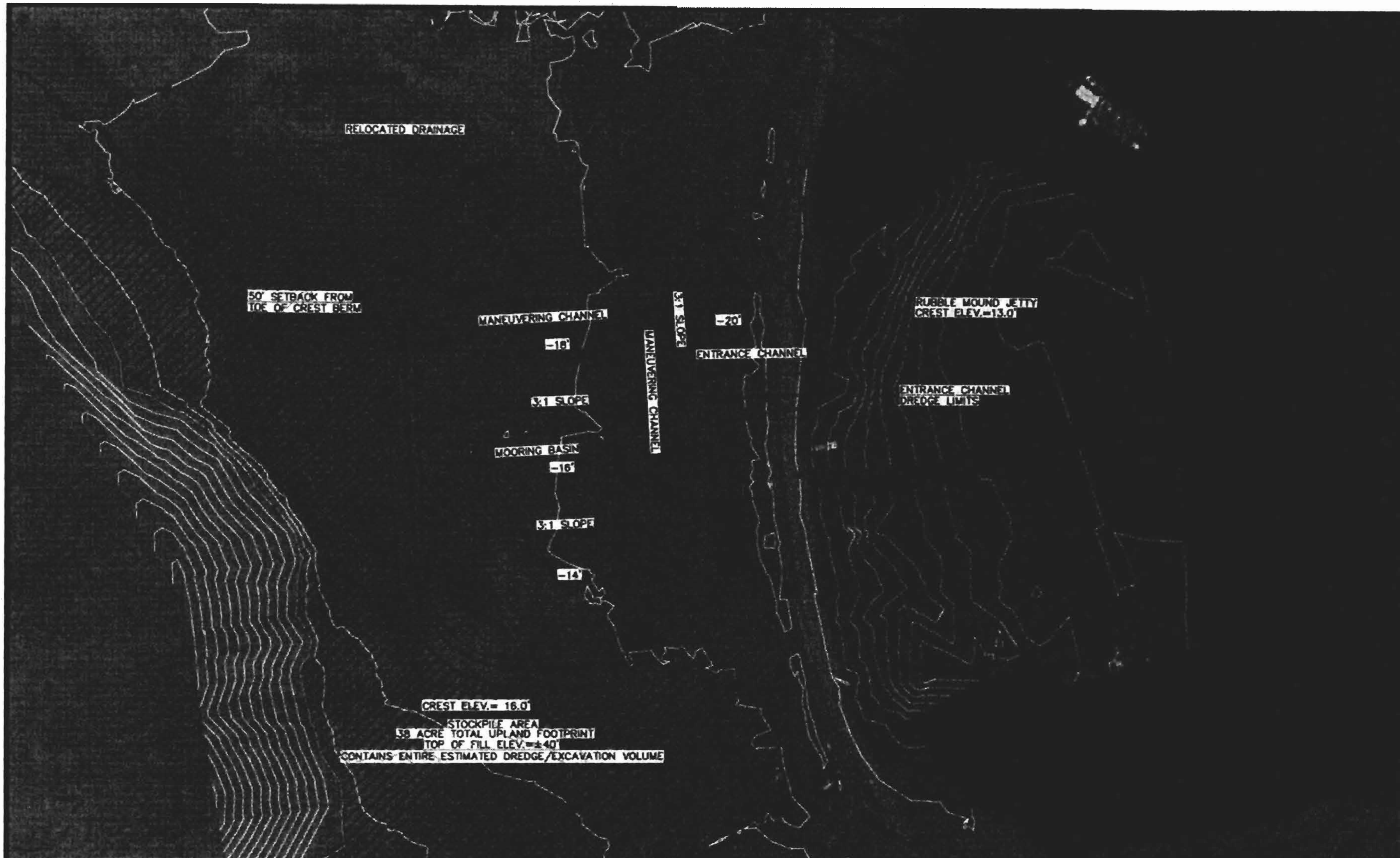
(IN FEET)
SCALE: 1" = 400'

12 ACRE ALTERNATIVE

HARBOR FEASIBILITY STUDY

AKUTAN ALASKA

FIGURE 14



NOTES
TOTAL DREDGE VOLUME = 990,000 CY
BASIN AREA = 15 ACRES
ENTRANCE CHANNEL AREA = 2.6 ACRES
(AREA MEASUREMENTS AT TOE OF EXCAVATION)
TOTAL HARBOR PROJECT AREA = 17.6 ACRES

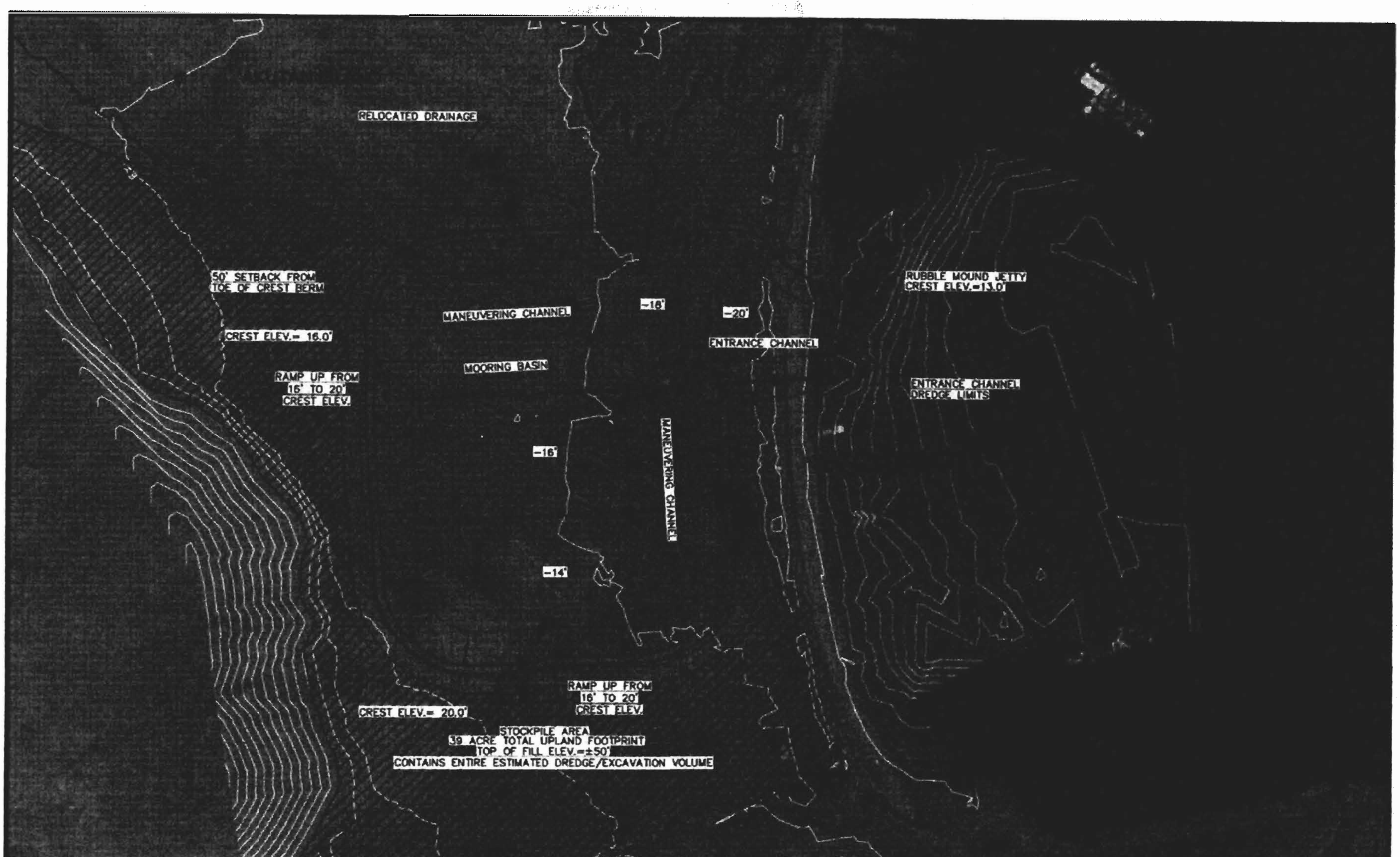
GRAPHIC SCALE
200 0 100 200 400
(IN FEET)
SCALE: 1" = 400'

15 ACRE ALTERNATIVE

HARBOR FEASIBILITY STUDY

AKUTAN ALASKA

FIGURE 15



NOTES

TOTAL DREDGE VOLUME = 1,175,000 CY
BASIN AREA = 19.2 ACRES
ENTRANCE CHANNEL AREA = 2.6 ACRES
(AREA MEASUREMENTS AT TOE OF EXCAVATION)
TOTAL HARBOR PROJECT AREA = 21.8 ACRES

GRAPHIC SCALE
200 0 100 200 400
(IN FEET)
SCALE: 1" = 400'

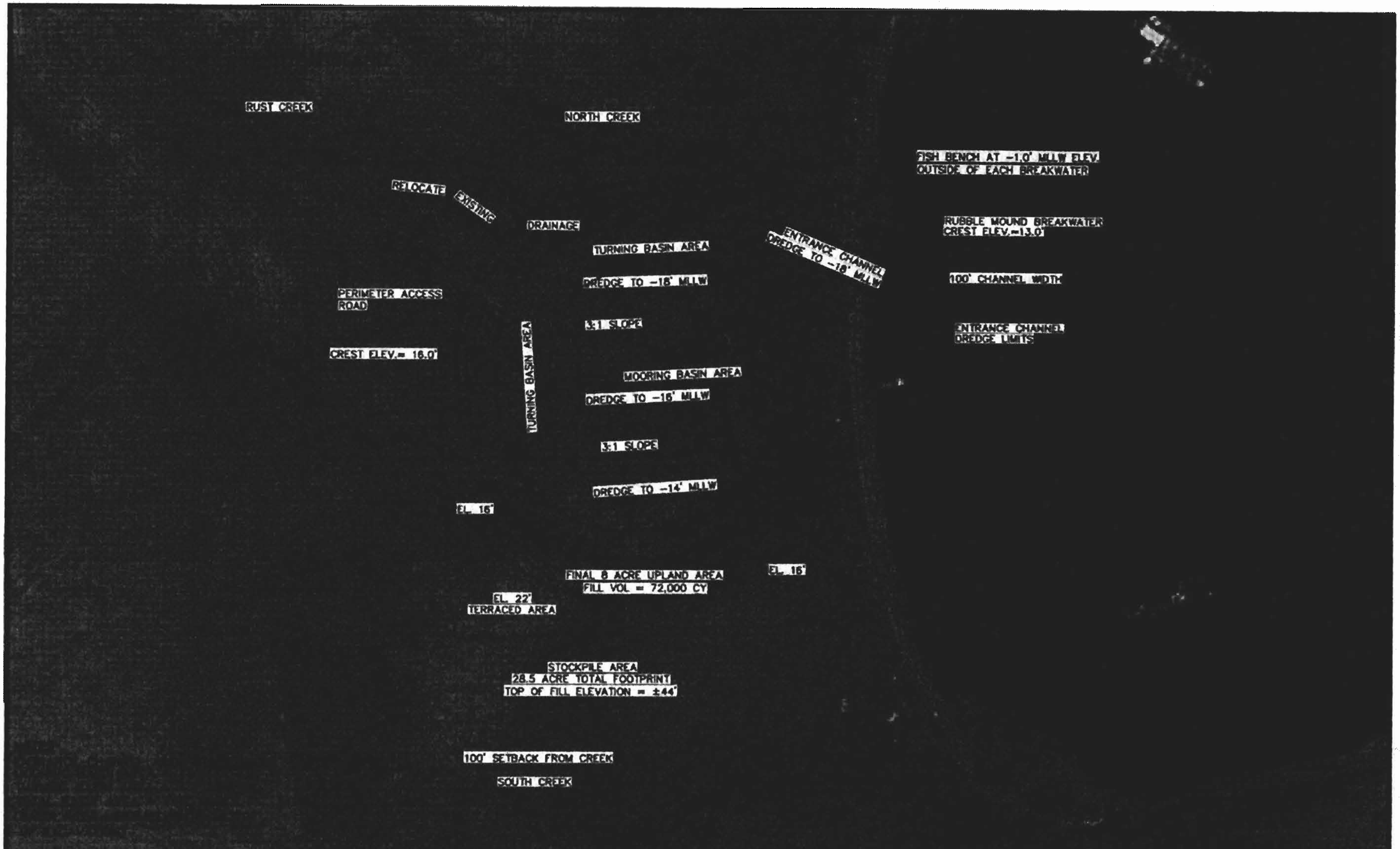
20 ACRE ALTERNATIVE

HARBOR FEASIBILITY STUDY

AKUTAN ALASKA

TRYCK NYMAN HAYES, INC

FIGURE 16



NOTES

TOTAL HARBOR BASIN PROJECT AREA = 16.2 ACRES (TO TOP OF SLOPE)
TOTAL HARBOR USEABLE UPLANDS AREA = 8.0 ACRES (DOES NOT INCLUDE ROADS AND SLOPES)
TOTAL USABLE HARBOR PROJECT AREA = 28.7 ACRES (INCLUDES PERIMETER ROAD, UPLANDS AND SLOPES)
TOTAL STOCKPILE AREA = 28.5 ACRES (INCLUDES 8 ACRES FOR FUTURE USABLE UPLANDS)
TOTAL HARBOR PROJECT AREA =57.2 ACRES (INCLUDES STOCKPILE FOOTPRINT)
TOTAL DREDGE VOLUME = 843,000 CY
VOLUME REQUIRED FOR USABLE UPLAND FILL = 72,000 CY
TOTAL STOCKPILE VOLUME = 771,000 CY

RECONFIGURED
12 ACRE ALTERNATIVE

TRYCK NYMAN HAYES, INC.

NAVIGATION IMPROVEMENTS

AKUTAN ALASKA

FIGURE 17

7.0 CONSTRUCTION METHODS AND SEQUENCING

7.1. Construction Methods

The choice between dredging by suction, clamshell dredging, or by excavation using a dragline, cat, or large hoe is dependent upon the type of materials being excavated, the water table elevation, and whether the material is to be reused as fill.

If the material is clean sand, as is suggested by the borings, the suction dredging method would be most suitable. This type of dredging has been used successfully on projects that involve the efficient moving of large volumes of materials. A large suction dredge can move uniform small grained material very efficiently.

Excavation can be an attractive method of material removal if the material is drained and reused as local fill. Excavation and onshore handling generally results in less mixing of the water and soil compared to suction dredging making it easier to stockpile and drain the dredged material. The relatively high water table dictates that only a small portion of the overall material can be efficiently excavated in this manner.

It is anticipated that the initial site preparation and excavation will be carried out most efficiently using cats and backhoes. Once the water table is reached, suction dredging will be the most efficient means.

7.2. Construction Sequence

The following general sequence of harbor construction is anticipated:

1. Establish silt fences around local streams. Redirect drainages as required. Establish project limits on the uplands.
2. Work would begin in the inner harbor basin. Blade off the top two or three feet of the vegetative mat of material into the upland stockpile area.
3. Create a stockpile drainage containment berm. The containment may include temporary sub drains that are directed into the harbor basin.
4. Excavate down to the water table using cats and backhoes. Push the material into the upper section of the stockpile area. Saturated material should be drained in the containment area.
5. Once the water table is reached, begin suction dredging of the inner harbor basin. Note that the entrance channel would remain plugged. Pump the material into the bermed stockpile containment area to drain. As the material is drained, push it into the upper sections of the stockpile area.
6. Excavate the basin slopes to grade and lay down the geotextile fabric. Place the slope filter rock and armor.
7. Once the main basin has been dredged, excavate the entrance channel to open the harbor basin to the bay. This work should begin on the basin side to minimize sedimentation getting into Akutan Harbor.
8. Construct breakwater jetties.

9. Construct inner harbor features, such as floats systems, etc. Install aids to navigation.
10. Prepare uplands for intended use.

7.3. Operation and Maintenance

Operations and maintenance of a USACE navigation improvement harbor, such as the proposed basin at the head of the bay requires a division of responsibilities between the local sponsor and the Federal Government. Typically, the operation of the harbor basin along with maintenance of the floats and utilities would be the responsibility of the local sponsor (the City of Akutan).

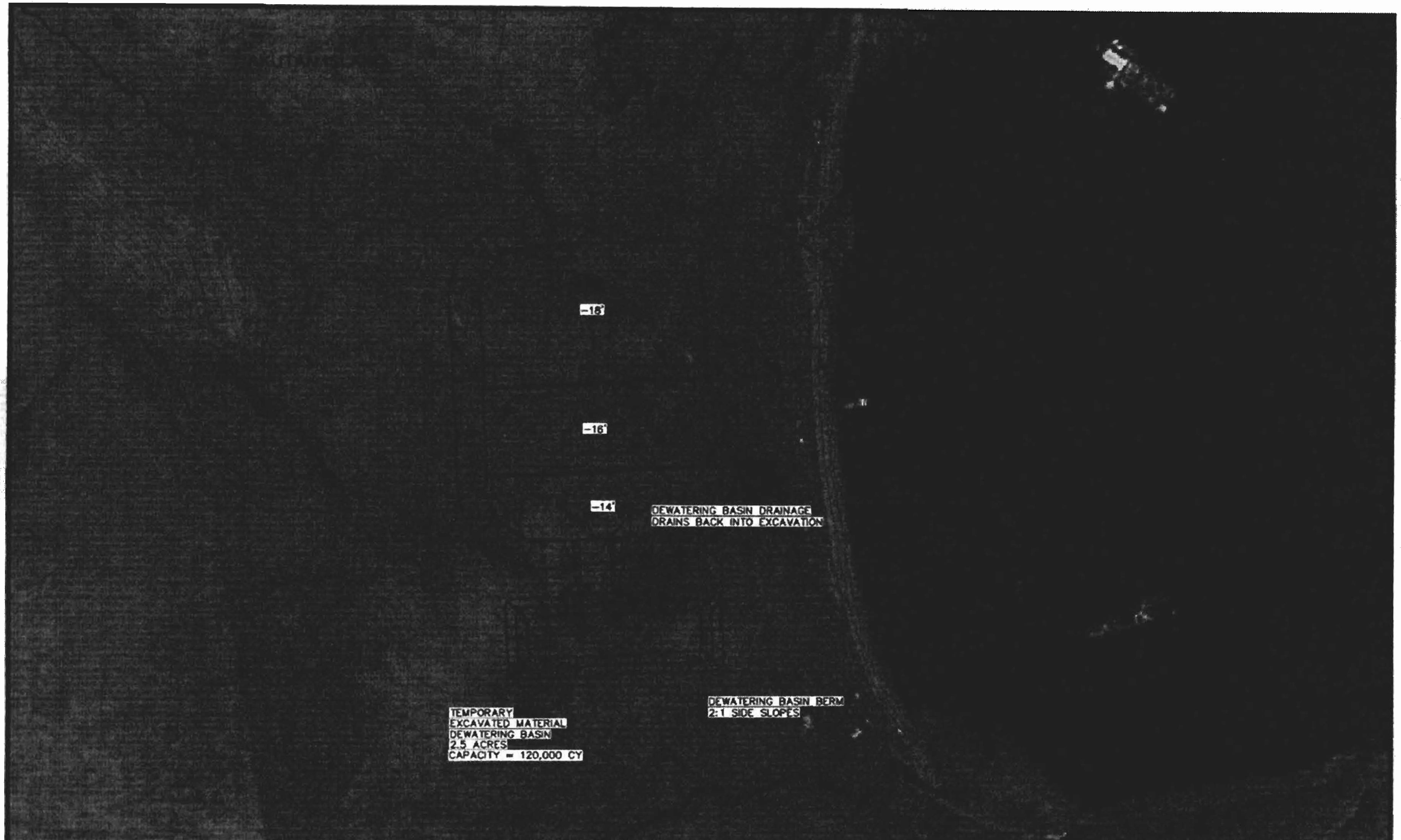
The Federal Government is typically responsible for the maintenance of the breakwaters, entrance channel, and maneuvering basin. This responsibility may entail the periodic hydrographic survey of these areas. Maintenance dredging and repair of the breakwaters may be required periodically. The wave climate at the head of the bay is fairly benign. It is unlikely that the breakwater jetties will be damaged by wave action. It is anticipated that there will be very little sediment transport across the entrance channel. Therefore, it is likely that any type of significant maintenance will be associated with damage due to an earthquake.

For planning purposes the following federal maintenance requirements may apply:

Hydrographic survey every 5 years.

Maintenance dredging (associated with an earthquake) of the entrance channel and maneuvering basin every 25 years. For concept planning purposes, this dredging is assumed to involve a total of 5% of the wetted volume of the harbor basin.

Replacement of 5% of the armor stone on the breakwater jetties every 25 years (again associated with an earthquake).



2:1 SIDE SLOPES

GRAPHIC SCALE

200 0 100 200 400

(IN FEET)
SCALE: 1" = 400'

DRAINAGE BASIN

NAVIGATION IMPROVEMENTS

AKUTAN ALASKA

FIGURE 18

7.4. Aids to Navigation

It is anticipated that navigation signs and lights will be required on the end of each of the breakwater jetties. A red light (port side when leaving the harbor) will be required on the north jetty and a green light (starboard when leaving the harbor) will be required on the south jetty. Suitable solar powered units are available commercially. Signage may include harbor master call frequencies and identification, as well as no wake zone/harbor speed limit.

The USCG should be notified and consulted with prior to final design and construction.

7.5. Construction Schedule

The construction schedule depends on several factors including the timing of the release of the plan set and on the equipment and techniques used by the contractor. It is possible to work year round at the head of the bay in Akutan. However, overall work efficiency will be reduced in the winter months. In addition, barging in equipment and materials can be more difficult in the winter.

A preliminary estimate of the duration of the major project elements is presented below:

Item	Duration
Bidding and contracting	2 months
Submittals, materials procurement, and shop drawings	3 months
Mobilization	1 month
Basin excavation and dredging	4 months
Breakwater construction	3 months
Inner harbor floats and utilities	3 months
Winter shut down	6 months
Environmental window shutdowns	2 months
Total	**24 months**

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