NAVIGATION IMPROVEMENTS
VALDEZ, ALASKA

APPENDIX A
HYDRAULIC DESIGN

APRIL 2010

TABLE OF CONTENTS

1.0 INTRODUCTION 1

1.1 Draft Appendix Purpose 1
1.2 Project Purpose........ 1

2.0 CLIMATOLOGY, OCEANOGRAPHY, HYDROLOGY........ 1

2.1 Climatology 1
2.2 Oceanography 1
2.3 Water Levels 2
2.4 Currents 3
2.5 Ice Conditions 3
2.6 Wind Data 3
2.7 Littoral Drift........ 6

3.0 WAVE STUDIES 6

3.1 Normal Wave Conditions........ 6
3.2 Wave Exposure for the Harbor Alternatives........ 8
3.3 Extreme Waves for the Harbor Alternatives........ 8
3.4 Boat Wakes........ 10

4.0 EXISTING HARBOR 10

5.0 HARBOR DESIGN CRITERIA........ 11

5.1 Design Vessel and Design Fleet 11
5.2 Allowable Wave Height in the Entrance Channel........ 12
5.3 Allowable Wave Height in the Mooring Area........ 12
5.4 Entrance Channel, Maneuvering Channel and Mooring Basin Design.... 12
5.5 Entrance Channel Depth 13
5.6 Basin and Entrance Channel Depth 13
5.7 Basin Size 14
5.8 Breakwater Design........ 15

6.0 ALTERNATIVES CONSIDERED........ 17

6.1 General 17
6.2 Alternative Sites 19
6.3 Alternative Sites Considered in Detail 20

7.0 PLAN IMPLEMENTATION........ 38

7.1 Aids to Navigation 38
7.2 Operations and Maintenance 39
7.3 Detailed Quantity Estimates 40
7.4 Construction Schedule 40
7.5 References........ 41

TABLES

Table A-1 *Tidal Datums, Valdez, Alaska*.....2
Table A-2 *Wind Frequency in Percent as a Function of Direction and Speed Class*...4
Table A-3 *Maximum Wind Speed (m/s) for Selected Return Periods*.....6
Table A-4 *Fetch Lengths and Directions Used to Predict Wave Conditions*.....7
Table A-5 *Significant Wave Height and Period for Principal Directions and Given Wind Class*.....8
Table A-6 *Fetch Lengths and Directions Used to Predict Wave Conditions*.....9
Table A-7 *Maximum Significant Wave Height and Periods for Selected Fetches at Each Site*.....9
Table A-8 *320 Vessel Design Fleet*.....11
Table A-9 *Entrance Channel Optimization*.....14
Table A-10 *178 Vessel Design Fleet*.....14
Table A-11 *226 Vessel Design Fleet*.....15
Table A-12 *228 Vessel Design Fleet*.....15
Table A-13 *313 Vessel Design Fleet*.....15
Table A-14 *Armor Rock Size*.....25

FIGURES

Figure A-1 *Project Location Map*.....44
Figure A-2 *Effective Fetches West Site*.....45
Figure A-3 *Effective Fetches East Site*.....46
Figure A-4 *Existing Harbor*.....47
Figure A-5 *Commercial Tenders Length vs. Draft*.....48
Figure A-6 *Alternative Signs*.....49
Figure A-7 *Sites Considered in Detail*.....50
Figure A-8 *Alternate Harbor Site Plans*.....51
Figure A-9 *West Site Rubble-Mound 226-Boat Plan, Plan View*.....52
Figure A-10 *West Site Rubble-Mound 226-Boat Plan, Project Profiles*.....53
Figure A-11 *West Side Rubble-Mound 226-Boat Plan, Wave Diffraction Diagram*.....54
Figure A-12 *West Site Wave Barrier 313-Boat Plan, Plan View*.....55
Figure A-13 *West Side Wave Barrier 313-Boat Plan, Wave Diffraction Diagram*.....56
Figure A-14 *West Site Wave Barrier 228-Boat Plan, Plan View*.....57
Figure A-15 *West Side Wave Barrier 228-Boat Plan, Wave Diffraction Diagram*.....58
Figure A-16 *West Site Rubble-Mound Breakwater Sections*.....59
Figure A-17 *East Site Alternative Harbor Plans*.....60
Figure A-18 *East Site Rubble-Mound 178-Boat Plan, Plan View*.....61
Figure A-19 *East Site Rubble-Mound 178-Boat Plan, Harbor Profiles*.....62
Figure A-20 *East Side Rubble-Mound 178-Boat Plan, Cross Sections*.....63
Figure A-21 *East Side Rubble-Mound 178-Boat Plan, Wave Diffraction Diagram*.....64
Figure A-22 *East Site Rubble-Mound 320-Boat Plan, Plan View*.....65
Figure A-23 *East Side Rubble-Mound 320-Boat Plan, Cross Sections*.....66
Figure A-24 *East Side Rubble-Mound 320-Boat Plan, Wave Diffraction Diagram*.....67

ATTACHMENTS

A. Quantity Estimates

West Site Rubble-Mound 226-Boat Plan
West Site Wave Barrier 313-Boat Plan
West Site Wave Barrier 228-Boat Plan
East Site Rubble-Mound 178-Boat Plan
East Site Rubble-Mound 320-Boat Plan

B. 35% Structural Design Analysis, Partial Depth Vertical Wave Barrier for Valdez Alaska

C. Preliminary Conceptual Plans, Navigation Improvements, Valdez, Alaska

1.0 INTRODUCTION

1.1 Draft Appendix Purpose

This draft appendix describes the hydraulic design of select alternatives for navigation improvements at Valdez, Alaska. It provides the hydraulic background for determining the engineering feasibility in the major construction features including water levels, wind and wave analyses, engineering design criteria and structure design.

1.2 Project Purpose

The city of Valdez requested the Corps of Engineers conduct a feasibility study of navigation improvements in Valdez, Alaska in June 1999. Additional moorage space for commercial fishing boats has been identified as necessary to reduce vessel damage associated with rafting and to increase the overall efficiency of the fishing industry in Valdez.

2.0 CLIMATOLOGY, OCEANOGRAPHY, HYDROLOGY

2.1 Climatology

The City of Valdez and Port Valdez are located in the northeastern part of Prince William Sound in South Central Alaska. See Figure A-1. Port Valdez has a maritime climate characterized by cool summers and mild winters. The mean annual temperature is 3.5 °C (38.3 F); the average summer temperature—between May and September—is 10.3 °C (50.5 °F), while in the winter—between October and April—the mean temperature is –4.2 °C (24.5 °F).

Valdez receives just over 157.5 cm. (62 in.) of precipitation annually. About half of the precipitation occurs between August and November with much of that as snowfall. The maximum and minimum annual precipitation amounts were 193.5 and 101.6 cm. (76.2 and 40.0 in.) in 1955 and 1950, respectively.

Winds are generally directed from the north-northeast sector between October and March and from the southwest-west sector between May and August.

2.2 Oceanography

Port Valdez is the northern extension of Valdez Arm in the northeast corner of Prince William Sound. It is a deep fjord that is separated from Valdez Arm by a pair of sills at the Valdez Narrows. From the Narrows, it extends 18 kilometers eastward to its head (Colonell, 1980). The fjord is about 5 kilometers wide and has a mean depth of about 180 meters with an extreme depth of 247 meters. The depth is fairly uniform from the entrance to about three-quarters of its length and then gradually slopes upward to its eastern end. The City of Valdez is located in the northeast corner of Port Valdez.

Port Valdez is a “positive” estuary since rain and land runoff there is a net outflow of water from the fjord (Pritchard, 1952). This implies a seaward flow of fresher, surface waters on top of a landward flow of more marine waters. Muench and Nebert (1973) found this fresher, seaward flow in the top 15 meters during the summer. The mean ratio of freshwater input to tidal prism peaks at just over 0.1 during the higher runoff July-August period. During the winter, the fjord is considerably less stratified (more homogeneous) than during the higher runoff summer.

According to Colonell (1980), Port Valdez has a tidal prism of about 1.6 % of its total volume. The classic “positive” estuary circulation is periodically interrupted by large surges of marine water into the fjord near the surface with accompanying discharges at depths. Colonell suggests that these may be related to the passage of weather systems. Muench and Nebert have suggested that due to the inflow of marine water at depth and the outflow of brackish water on the surface, Port Valdez completely replenishes itself in about 40 days.

2.3 Water Levels

2.3.1 Tides

Tide datums at Valdez, referenced to mean lower low water (MLLW), are provided in Table A.1. The tidal datums shown below are based on the 1983-2001 tidal epoch.

Table A-1. ***Tidal Datums, Valdez, Alaska***

Tide	Elevation (m/ft)(MLLW)
Highest Observed Water Level	5.20 / 17.1
Mean Higher High Water	3.70 / 12.1
Mean High Water	3.42 / 11.2
Mean Tide	1.98 / 6.5
Mean Low Water	0.46 / 1.5
Mean Lower Low Water (datum)	0.00 / 0.0
Lowest Observed Water Level	-1.51 / -5.0

2.3.2 Storm Surges

Since Port Valdez is a deep fjord, it does not experience significant storm surges due to wind stresses. Storm surge can be shown to be inversely dependent on water depth. That is, for a given wind speed, storm surge is less in deep water than in shallow water. To reinforce this assertion, to the study team’s knowledge, no one has indicated that storm surge has been, or should be, of concern in Port Valdez. No historical record of storm surge activity is known to exist. There may be a surge elevation of a few centimeters on occasion due to atmospheric pressure differentials. We have included a value of about 0.15 meters to account for this possibility; however, there is no direct correlation that these pressure-induced surges would occur at the same time as large-wave generating winds.

2.4 Currents

Flows in the eastern portion of Port Valdez, from Jackson Point are slow and displayed little correlation to tides or winds (Colonell, op. cit.) measured currents at several locations in Port Valdez including those in the Narrows and in the vicinity of Jackson Point. In the Narrows, tidal velocities were routinely above 40 cm/s; near the eastern end, currents rarely exceeded 5 cm/s. Colonell suggested that some of the strongest current events, where speeds in the eastern end exceeded 5 cm/s, were due to disturbances that originated in Prince William Sound outside of Port Valdez. Jones (1992) modeled circulation in Port Valdez and similarly found velocities generally less than 5 cm/s. Surface floats and subsurface drogues (-3 meters) were deployed in the eastern end of Port Valdez in the vicinity of the Duck Flats (Jones, 1992). The surface floats consisted of plywood disks and traveled at velocities considerably faster than the conventionally accepted rates of 3-4 percent of the wind speed. The subsurface drogues traveled at speeds of only a few centimeters per second (maximum was 7.9 cm/s). It is believed that the rough skin of the surface floats and their tendency to float above the water's surface caused the high travel rates.

2.5 Ice Conditions

Port Valdez is an ice-free bay for the entire year.

2.6 Wind Data

Fifteen years (1985-1999) of meteorological records were acquired from NOAA's National Climatic Data Center (NCDC). The wind recorder was located no more than 100 meters from the water with little or no obstruction from the directions indicated in the following table (with the possible exception of northwest winds). It was on a tower which itself was on a hill probably 50 meters above water level. Due to its proximity to the water, no "over water" correction was used. According to the NCDC, the anemometer was located 16.5 m. (54 ft.) above ground level. While this makes the anemometer situated about 65 to 70 m. above sea level, it is the distance above ground level that is important in adjusting the values for elevation. The adjustment for elevation was conducted according to:

$$U_{10} = U_z \left(\frac{z}{10} \right)^{1/7}$$

where z represents the height of the anemometer above ground in meters and U_{10} and U_z are the wind speeds at a height of 10 and z meters, respectively.

The data consisted of hourly winds speed and direction entries that, according to NCDC, should represent 2-minute means for the period when the observation was made. Because these observations could represent any winds for the hour that they were a part, they are assumed to represent the hourly mean. They were processed into a frequency histogram as shown in Table A-2. Eight direction classes and seven speed classes were used to create the histogram.

Table A-2. ***Wind frequency in percent as a function of direction and speed class***

Speed (m/s)	Direction Sectors							
	N (000)	**NE (045)**	**E (090)**	**SE (135)**	**S (180)**	**SW (225)**	**W (270)**	**NW (315)**
0.5-2.6	4.6	7.1	8.1	4.0	3.3	7.7	4.9	4.9
2.6-5.1	1.6	6.8	9.3	2.3	0.7	7.8	6.3	2.8
5.1-7.7	0.3	2.6	3.7	0.7	0.1	0.9	2.8	0.4
7.7-10.3	0.2	1.0	1.2	0.5	0.1	0.1	0.5	0.1
10.3-12.9	0.3	0.3	0.4	0.3	0	0	0	0
12.9-15.4	0.1	0.1	0.1	0.1	0	0	0	0
15.4-18	0	0	0	0	0	0	0	0

Since the compass was divided into eight directional sectors, each sector consisted of 45 degree increments. For example, north, in fact, consists of the averages for the wind speeds between 337.5° and 22.5° and is designated as the direction 000°, northeast is the average for the speeds between 22.5° and 67.5° and is designated as the direction 045°, and so on for the other six sectors.

However, in fact, the data could not be simply analyzed to produce these averages. The data obtained for this analysis from NOAA's National Climatic Data Center had directions to the nearest 10 degrees. No straight-forward average would produce those sought. The compass was divided into two different sector groupings. The first consisted of the following eight sectors:

- North; 340°-30°
- Northeast; 30°-70°
- East; 70°-120°
- Southeast; 120°-160°
- South; 160°-210°
- Southwest: 210°-250°
- West; 250°-300°
- Northwest; 300°-340°

The second consist of the sectors:

- North; 340°-20°
- Northeast; 20°-70°
- East; 70°-110°
- Southeast; 110°-160°
- South; 160°-200°

- Southwest: 200°-250°
- West; 250°-290°
- Northwest; 290°-340°

Several files of data were analyzed simultaneously, each representing a single year. As each speed/direction duet was read, it was assigned one of the sector groups shown above in an alternating scheme. These values were used to construct histograms and to generate average speed values for the eight compass directions designed by north, northeast, east, etc. While this method could not achieve the desired averages exactly, it came close. Because of the somewhat complex method of analysis, the directions are shown only as a single (rather than ranges) alphanumeric and numeric value on the charts where appropriate.

If the wind averages are for a duration of less than one hour, they are often converted to their one-hour average equivalent. For the current wind data, this was not done since with equal probability the value chosen to record could have been the hourly maximum, hourly minimum, or any value in between. Therefore, it was assumed that these represented the one-hourly mean value. If this procedure caused an error, then that error should generate higher, not lower, wind speeds and, therefore, would have produced a more conservative result.

The 50-year extreme wave was calculated from these winds by first sorting the wind speeds that were indexed according to direction. By then observing the date and time of these sorted values, a set of the largest independent values was selected and analyzed to obtain their extreme values by fitting them to the Weibull distribution. That distribution can be expressed as:

$$P(w \le \hat{w}) = 1 - \exp(-\left(\frac{\hat{w} - B}{A}\right)^k)$$

where

w is the sorted wind speed
$\hat{w}$ is the particular wind speed value
A and B are coefficients determined from the data distribution
k is a shape parameter.

Mentioned above was the idea of “independent” value. This independence was achieved by selecting only one value (the maximum) from any single storm. In this way, a single large storm cannot overly influence an extreme value result. The data were fit to several Weibull distributions that were distinguished from each other on the basis of the “k” value. The k value that produced the minimum variance in fitting the data to the curve was assumed to be the correct value.

Five wind directions were originally analyzed in detail (the east, southeast, south, southwest and west directions). The other three directions came almost directly off the land and would not generate waves of any significance for these proposed project sites.

The extreme values for the southeast and west directions are shown in Table A-3.[1]

Table A-3. ***Maximum wind speed (m/s) for selected return periods (yrs)***					
Return Period	**Wind Direction Sector**				
	East (090º)	**Southeast (135º)**	**South (180º)**	**Southwest (225º)**	**West (270º)**
2	16.6	17.2	10.7	9.7	11.9
5	18.4	18.6	11.8	10.4	13.4
10	19.8	19.6	12.5	10.7	14.6
25	21.6	21.0	13.3	11.2	16.4
50	23.0	22.1	13.8	11.5	17.8

2.7 Littoral Drift

The shorelines in the vicinity of the proposed harbors consist of solid rock or relatively large rock debris. It is not easily transported as littoral material. There are not obvious indications that littoral transport is significant at all at either of the two proposed project sites. Lowe River probably contributes most of the sediments that enter Port Valdez, but it is well removed to the southeast from the potential sites with no direct pathways to connect it to the sites. The largest contribution to littoral drift would be the construction itself. Shoaling from this source is expected to be minor.

There has been no maintenance dredging in the existing harbor entrance channel since its initial construction in 1965. There is, however, a very minor amount of shoaling in the entrance channel at the project limits, which has not required maintenance dredging.

3.0 WAVE STUDIES

3.1 Normal Wave Conditions

Since the proposed harbor will be located along the north shore of Port Valdez, only winds from five sectors (east, southeast, south, southwest, and west) were considered for this analysis. There are no fetches associated with the other three sectors. We have assumed they represent hourly means.

The wave heights were calculated using the ACES v. 1.07 program. This program has the capability to consider a group of fetches for any particular wind direction. This ability recognizes the reality that winds are not uniform and their actual direction varies on either side of the principal direction. To represent this non-uniformity, we selected an angle increment and established fetch lengths for multiples of this increment on each side of the principal angle. For this case, we used 5º as the increment. The angle between the principal direction and a particular fetch is weighted so that the greater the difference

[1] See previous discussion on wind directions.

between a particular fetch and the principal direction, the less effect that fetch will have on the wave growth. An effective fetch can be calculated from this ensemble of fetches.

Table A-4 shows the fetches and their associated angles (relative to true north) that were used for each compass sector. This table is for information purposes only. The point in Port Valdez where these waves apply is in the general area of the two projects but not specific for either project alternative. The design waves that apply to each site will be presented later in this section.

Table A-4. ***Fetch Lengths (km) and Directions Used to Predict Wave Conditions. The principal directions have been highlighted with bold italics.***

Fetch No.	Compass Sectors									
	East		Southeast		South		Southwest		West	
	Dir	Len	Dir	Len	Dir	Len	Dir	Len	Dir	Len
1	***090***	***0***	110	5.6	155	5.0	200	4.4	245	12.0
2	095	3.9	115	5.9	160	5.0	205	4.4	250	15.2
3	100	4.4	120	6.3	165	5.0	210	5.2	255	20.4
4	105	5.0	125	6.1	170	5.0	215	5.7	260	20.4
5	110	5.6	130	6.1	175	4.8	220	6.1	265	19.6
6	115	5.9	***135***	***5.9***	***180***	***4.6***	***225***	***6.7***	***270***	***13.9***
7			140	5.9	185	4.6	230	7.2		
8			145	5.7	190	4.8	235	9.6		
9			150	5.2	195	4.6	240	11.3		
10			155	5.0	200	4.4	245	12.0		
11			160	5.0	205	4.6	250	15.2		

The temperature difference between air and water can also contribute to wave growth. When the water temperature is warmer than that of air, the resulting over-water winds may be more intense. In Valdez, we have assumed that east and southeast are more likely to be associated with winter conditions while south, southwest, and west winds are more often summer events. For the winter winds, we have assumed that the air can be 5ºC colder than the water temperature, and for summer winds the air can be 1ºC colder. These temperature differences can amplify the winter-wind waves and summer-wind waves by about 1.15 and 1.06, respectively (SPM, 1984, Figure 3-14).

Table A-5[2] presents the results of the significant wave heights and periods for the given directions and wind speed classes. These wave parameters included the temperature corrections indicated above.

[2] See previous discussion on wind directions.

Table A-5. ***Significant wave height (m) and period (sec) for principal directions and given wind classes.***

Speed (m/s)	Compass Sectors									
	East (090)		Southeast (135)		South (180)		Southwest (225)		West (270)	
	H_s	T	H_s	T	H_s	T	H_s	T	H_s	T
3.1-5.1	0.12	1.4	0.15	1.5	0.12	1.4	0.12	1.5	0.18	1.7
5.7-7.7	0.21	1.7	0.24	1.8	0.21	1.7	0.27	2.0	0.37	2.4
8.2-10.3	0.30	2.0	0.34	2.2	0.30	2.0	0.43	2.4	0.61	3.1
10.8-12.9	0.40	2.3	0.46	2.4	0.40	2.3	0.55	2.8	0.85	3.4
13.4-15.4	0.52	2.5	0.58	2.7	0.52	2.5	0.73	3.1	1.10	3.8
15.9-18	0.61	2.8	0.70	2.9	0.61	2.8	0.88	3.4	1.34	4.2

By overlaying Tables A-2 and A-5, it is possible to obtain the percent frequency distribution for wave heights and periods. These waves have not been adjusted for any transformations due to refraction, shoaling, or diffraction.

An independent analysis using the wave generation and transformation program STWAVE was conducted using short-term data collected by the U.S. Air Force. While the 50-year extreme winds from that analysis were considerable higher than the estimates from the longer-term record described in this report, the comparisons were reasonably close. STWAVE also predicted extreme heights in the 1- to 1.6-meter range.

3.2 Wave Exposure for the Harbor Alternatives

Each harbor alternative has somewhat different wave exposures (fetches). Since only those wave-generating conditions that will produce the largest design waves either on the structure and/or in the harbor entrance need be considered, not all directions have been used in the analysis. The important directions have been determined to be toward the west, southwest, and the southeast. The other directions (east and south) either have winds that are too low, fetches that are too short, or else they are oriented so that large waves could not arrive at the entrance; some directions meet more than one of these limitations.

Figures A-2 and A-3 show the fetches for the southeast, southwest, and west directions for the western and eastern harbor alternatives, respectively.

3.3 Extreme Waves for the Harbor Alternatives

The wave analysis presented above represents the operational wave conditions and does not account for the rare extreme wave event. The extreme wave conditions have been determined from the extreme winds presented in a previous section using the fetches that correspond to a particular harbor alternative and principal wind direction. For the actual harbor design, the important wave parameters are the design heights and periods that can develop at the harbor entrance and on the protecting breakwaters. Waves at the entrance can effect navigation into the harbor and ultimately can dictate inter-harbor wave conditions. Wave conditions on the breakwaters can dictate rock sizes and configurations and breakwater design heights. It appears that only the southeast, southwest, and west wind conditions need to be analyzed.

The fetches for these three directions at each site are provided in Table A-6.

Table A-6. ***Fetch lengths (km) and directions used to predict wave conditions. The principal angles have been highlighted with bold italics.***

Fetch No.	Compass Sectors											
	Western Alternative						**Eastern Alternative**					
	SE		**SW**		**WEST**		**SE**		**SW**		**West**	
	Dir	**Len**	**Dir**	**Len**	**Dir**	**Len**	**Dir**	**Len**	**Dir**	**Len**	**Dir**	**Len**
1	115	*5.85*	205	4.92	250	18.1	115	5.55	205	3.77	250	18.2
2	120	6.31	210	4.94	255	16.9	120	5.76	210	3.91	255	17.4
3	125	6.46	215	5.18	260	16.5	125	5.82	215	4.5	260	16.9
4	130	6.76	220	5.43	265	15.3	130	5.76	220	4.78	265	13.2
5	***135***	**6.58**	***225***	***6.11***	***270***	***11.5***	***135***	***5.70***	***225***	***5.03***	***270***	***12.3***
6	140	6.58	230	6.85	275	7.92	140	5.39	230	5.26	275	8.75
7	145	*6.22*	235	7.17	280	7.71	145	5.03	235	5.71	280	8.32
8	150	5.76	240	7.99			150	4.72	240	6.48	285	7.31
9	155	5.46	245	8.23			155	4.51	245	7.50		

The 50-year extreme winds, based on elevation-corrected winds, were presented in Table A-3. Those winds, along with the appropriated fetches, were used to determine the 50-year extreme wave heights and periods shown in Table A-7. A temperature difference of -1° C between the water and air was used for the southwest and west waves and a difference of -5° C was used for the southeast waves. These extremes represent the deep-water significant wave as yet unaffected by transformation processes, such as refraction, shoaling, or diffraction.

Table A-7. ***Maximum significant wave height (m) and periods (s) for selected fetches at each site***

Alternative	Southeast Direction (135)		Southwest Direction (225)		West Direction (270)	
	H_s	T	H_s	T	H_s	T
Eastern	1.0	3.5	0.5	2.5	1.4	4.3
Western	1.1	3.6	0.5	2.6	1.2	4.1

Clearly, the waves from the southwest are significantly lower than from the other two directions. They have been retained only because they approach the harbor entrances of both alternatives on a more direct path and, therefore, are less susceptible to reduction by refraction.

3.4 Boat Wakes

Since both of the proposed harbor locations are beyond most of Valdez's waterfront, the likelihood of large boats traversing the harbor entrance with enough speed to generate any significant boat wake is small. Boat wakes are complex and not fully understood. Tobiasson and Kollemeyer (1991) suggest that non-planing boats traveling at speeds of 8 knots or less, which includes most of the boats affecting the harbor, would produce wakes with periods of less than 2.2 seconds.

According to Tobiasson *et. al.,* such wakes should be below 0.3 meters high within about 120 meters of the source vessel. While it is possible that non-planing vessels may pass the harbor entrance at rates less than 8 knots or closer than 120 meters, or that small, powerful planing boats at high speeds could possibly generate waves higher than 0.3 meters in the entrance, it is very unlikely that these short period waves could enter the harbor basin without a significant reduction in height.

Boat wakes may be of greater concern to the west basin option due to the proximity of its entrance with the entrance of the existing boat basin. Boats entering the existing harbor will be of more concern than those leaving. Except for the smaller boats, most of the boats entering the existing basin will have cut power considerably by the time they reach the mouth of the west basin option. This alone will reduce the boat wake. In addition, as the boats turn into the existing entrance channel, their wakes will be diverging and the wave amplitude will drop off quickly. Also, they will be approaching from a direction that will not provide a direct path for their wakes into the west basin alternative. In fact, it appears clear that any boat waves would be reflected from at least one rubble mound slope before entering the inner basin. This combined with the height and period characteristics described above, would ensure that they would not cause any noticeable impact in the basin.

4.0 EXISTING HARBOR

The original Valdez small boat basin was constructed in "Old Valdez" in 1939 with breakwaters added in 1957. This project was completely destroyed by a tsunami during the March 1964 Alaska earthquake. The present harbor, authorized by the Rivers and Harbors Act, 19 August 1964, was completed in 1965 at the relocated Valdez town site, approximately 6.4 kilometers northwest of the original harbor. See Figure A-4.

The reconstructed Corps of Engineers' project consisted of a 4 hectare moorage basin, dredged to –3.7 meters Mean Lower Low Water (MLLW) and a 36.5 meter wide entrance channel, also dredged to –3.7 meters MLLW. The entrance channel is protected by two rock breakwaters, 190 and 209 meters in length. In 1985, the harbor mooring basin was expanded by local interests to about 8 hectares. The harbor has a capacity of 513 vessels and includes a two lane launch ramp, two shallow draft cargo docks with two mast and boom derricks and a 54.4 metric ton boat travel lift. Dry storage for approximately 100 boats is located adjacent to the harbor. The original project included floats B to E. The State of Alaska added floats A, F and G in 1978. The 1985 expansion added floats H to K. The floats include potable water and electricity. Additional harbor improvements consisting of a tour boat float and sheet pile bulkhead on the northwest side were completed in 1987.

Two seafood processors, Sea Hawk Seafoods and Peter Pan Seafoods, use the cargo docks for off loading fish from commercial fishing vessels. The U. S. Coast Guard also has a dock within the harbor at the entrance channel for mooring patrol craft.

The existing harbor has poor circulation, especially at the eastern end farthest from the entrance channel. An underground spring entering somewhere in the northern or northwestern part of the basin can cause considerable icing in the winter. The ice does not eliminate navigation for the largest vessels, but smaller vessels may need to have the ice broken up or follow a larger vessel out of the harbor. No maintenance dredging has been required in the entrance channel or basin and no maintenance of the breakwaters has been necessary since their construction in the late 1960’s. Wave action in the mooring area is limited to boat wakes, since the entrance channel protects the basin very well from wind generated waves in Port Valdez.

5.0 HARBOR DESIGN CRITERIA

5.1 Design Vessel and Design Fleet

The Alaska District Economics Section determined the design fleet for this study. The number and length class of vessels in the design fleet are given in Table A-8 below. Lengths, beams and drafts for the fleet were developed in conjunction with the harbormaster and various harbor users.

Table A-8, *320 Vessel Design Fleet*

Length Class (m)	**Number**
9	245
13	59
16	10
30	6

As defined by the above fleet, the largest vessels that are anticipated to regularly use the harbor are tenders. A list of tenders that used the harbor in year 2000 was used to determine the design vessel dimensions. Using the dimensions of these vessels, a “generic” design vessel was determined that would include all but one of these vessels as shown in Figure A-5, Commercial Tenders, Length vs. Draft. The design vessel is 30 meters (98 ft) long with a beam of 7.9 meters (26 ft) and a draft of 3.66 meters (12 ft).

5.2 Allowable Wave Height in the Entrance Channel

Breakwaters for the proposed alternatives were positioned so that the waves inside the harbor entrance channel would minimize to the maximum extent possible. Storm waves quickly dissipate to less than 0.3 meters before reaching the mooring area.

5.3 Allowable Wave Height in the Mooring Area

The maximum allowable wave height in the mooring area was limited to 0.3 meters. This criterion is outlined in EM 1110-2-1615, Hydraulic Design of Small Boat Harbors and the ASCE Planning and Design Guidelines for Small Craft Harbors.

Diffraction analyses diagrams from the Shore Protection Manual were used to determine the wave heights expected for each harbor alternative considered in this study.

5.4 Entrance Channel, Maneuvering Channel and Mooring Basin Design

The entrance channel width was determined using criteria in EM 1110-2-1615 and the ASCE Planning and Design Guidelines for Small Craft Harbors. Both references recommend minimum channel width of 5 beam widths for two-way traffic in entrance channels. Factors considered include vessel size, vessel maneuverability, traffic congestion, and effects of wind, waves and currents. The design vessel will have sufficient width to enter or leave the harbor in a one-way traffic mode. Smaller vessels, by far the more numerous to use the harbor, will have two-way capability. The design width for the entrance channel is 40 meters (8m x 5) (131 ft.).

Maneuvering channel and fairway widths were designed so there would be enough room for vessels to turn and dock. Width of fairways was determined using a factor of 1.5 times the length of the longest finger piers in that area of the basin. The 1.5 times the longest finger pier length factor is the minimum acceptable fairway width. Vessels extending beyond the finger pier length must be prohibited when specifying the minimum width fairway.

5.5 Entrance Channel Depth

The entrance channel was established based on the following criteria from the entrance channel depth optimization analysis and EM 1110-2-1615:

Design Tide Level	-0.33 m MLLW	(-1.1 ft) MLLW
Vessel Draft	3.66 m	(12.0 ft)
Wave Allowance (2/3 entrance wave height)	0.75 m	(2.4 ft)
Squat	0.15 m	(0.5 ft)
Safety Clearance (sand & gravel bottom)	0.61 m	(2.0 ft)
Entrance Channel Depth	-5.50 m MLLW	(-18.0 feet) MLLW

5.6 Basin and Entrance Channel Depth

The basin depth was established by eliminating the wave allowance and squat from the above criteria, leaving the vessel draft and safety clearance. The lowest predicted tide was selected as the design tide level. The minimum depth is then:

Design Tide Level (Lowest Predicted Tide)	-1.18 m MLLW	(-3.9 ft) MLLW
Vessel Draft	3.66 m	(12.0 ft)
Safety Clearance (sand & gravel bottom)	0.61 m	(2.0 ft)
Mooring Basin Depth	-5.45 m MLLW	(-17.9 ft) MLLW

The design depth was then rounded to –5.5 meters. The design vessel will have at least 0.61 meters clearance at the lowest predicted tide to prevent grounding of keel coolers, instruments and other outside hull features from damage. The basin was stepped up from the –5.5 meter depth to –4.0 and –2.7 depth to account for the shallower draft vessels that would be using the inner harbor basin. Note that a reduced safety clearance of 0.3 meters was used for the shallower draft 9 meter vessels.

A brief analysis was made to determine the percent of time the design vessel could transit the entrance channel while remaining outside the safety zone as defined by the above criteria related to entrance channel depths and the lowest predicted tide elevation. This analysis shows that the –5.5 meter channel depth provides for 97.9 % accessibility for the predicted tide and concurrent 50 year storm event.

Table A-9, Entrance Channel Optimization							
Channel Depth (m)	-4.60	-4.90	5.20	**-5.50**	-5.80	-6.10	-6.30
Safety Clearance (m)	0.61	0.61	0.61	**0.61**	0.61	0.61	0.61
Vessel Squat (m)	0.15	0.15	0.15	**0.15**	0.15	0.15	0.15
Pitch, Heave & Roll (m)	0.75	0.75	0.75	**0.75**	0.75	0.75	0.75
Design Vessel Draft (m)	3.66	3.66	3.66	**3.66**	3.66	3.66	3.66
Tide Level (m MLLW)	0.57	0.27	-0.03	**-0.33**	-0.63	-0.93	-1.23
Percent Accessibility	86.9	91.8	95.4	**97.9**	99.3	99.9	100.0

The minimum entrance channel depth is established as the maximum basin depth of –5.5 meters as determined above. Creating a sill by reducing the entrance channel depth to less than the basin depth is inappropriate for small harbors, because of the negative effects on the water exchange, circulation, water quality and potential sedimentation. Larger and/or deep draft harbors will benefit from an economically optimized entrance channel with little or no detrimental environmental effects. Channel depths deeper than the deepest basin depth may be appropriate if wave conditions and vessel navigation dictate.

5.7 Basin Size

The two sites considered in detail and further described in Section 6 of this draft appendix are physically or environmentally constrained. Several alternatives were developed that would reasonably accommodate varying numbers of vessels, considering these constraints. In addition to the 320 vessel fleet shown in Section 5.1, the following fleets were used.

Table A-10, *178 Vessel Design Fleet*	
Length Class (m)	**Number**
9	103
13	59
16	10
30	4

Table A-11, *226 Vessel Design Fleet*	
Length Class (m)	**Number**
7.5	112
9.5	36
12	42
16	36

Table A-12, *228 Vessel Design Fleet*	
Length Class (m)	**Number**
9	153
13	59
16	10
30	6

Table A-13, *313 Vessel Design Fleet*	
Length Class (m)	**Number**
9	214
13	66
16	27
30	6

Basin float layout and dimensions were adjusted to fit these fleets and costs were determined. Changing the size of the basin was accomplished by lengthening or shortening each basin alternative in the east/west direction to fit the fleet and then determining the cost associated with that harbor design.

5.8 Breakwater Design

Evaluation of the sites and various breakwater configurations resulted in the rubble mound breakwater being selected as the most appropriate for the shallow East Site and the wave barrier being selected for the deeper West Site. Several alternative breakwater types were considered; rubble mound, floating, composite berm with vertical wall and partial depth vertical wave barrier.

Rubble mound breakwaters are the most common type of breakwater and are often used in shallow water, usually less than 10 meters and are most effective against long period waves. They can withstand minor settlement and damage without catastrophic failure, require a minimum of long term maintenance and are often more cost effective from a life cycle perspective than other types of breakwaters in similar water depths. Historically in Alaska, rubble mound breakwaters have performed very well.

A floating breakwater was also considered. In general practice, floating breakwaters are not used in wave climates exceeding a 1.2 meter (4 ft.) wave height and 4 second periods. They work principally by both reflection and absorption of wave energy and must be relatively wide to be effective. The waves from the westerly direction at this site approach the upper limit of height and period. Anchoring the floating breakwater would also create a significant challenge with very shallow water on the north side and a steep drop off on the south side. Additionally, depending on the location of the breakwater, potential grounding could occur. Floating breakwaters are generally most cost effective in water depths of 10 to 20 meters. Floating breakwaters were found to be unsuitable and less economical than rubble for the Valdez sites.

A composite low rubble berm with vertical sheet pile wall, similar to those found in the Aurora and Harris Harbors in Juneau, was considered, but was found to have no significant advantage over the rubble mound alternative. The lower weight of the breakwater structure was the only advantage, but the cost of supplying and constructing both rock and sheet pile was determined to be more expensive than rock only for the initial construction. Maintenance of the sheet pile was a major disadvantage.

A partial depth vertical wave barrier was also considered. This consists of a wall penetrating down through a portion of the water column. There are, however, drawbacks to this type of wave protection. The wave barrier attenuates the wave almost entirely by reflection so wave conditions near the wave barrier will be more severe. There is a potential for a “mach-stem” type wave running along the seaward face during oblique wave exposure. The use of steel and reinforced concrete also introduces concerns for corrosion and maintenance. Curtain wall wave barriers are most often used for wave heights and periods similar to floating breakwaters, less than about 1.3 meters high and 4-second period. In the case of Valdez, in order to provide acceptable wave transmission in a similar water depth as the rubble breakwater, the barrier would need to extend to the seabed and would require toe protection. This would be similar to the composite breakwater described above. Reflection is probably the biggest concern. An entrance channel configuration similar that designed for the rubble mound breakwaters would create extremely undesirable reflection in and near the entrance. This would result from the combination of the incident wave and boat wakes reflecting off of the vertical surfaces. To minimize the entrance channel wave conditions the entrance would need to be sited in shallow water to allow rubble-mound structures to protect and define the entrance channel. The wave barrier would be used in deeper water where the rubble-mound structure would become unstable and cost prohibitive. Maintenance of a steel structure is also very expensive. Well-maintained anodes and coatings and/or thicker steel sections would be needed to even approach a 50-year design life. Concrete piling and panels could be used and could extend the service life. However, driving concrete piling may prove difficult with the known high potential for boulders or rock slabs. While steel structures have lasted longer than 25 years in the marine environment, a 25 year service life was used for economic and life cycle analysis. Steel or concrete curtain wall wave barriers are technically feasible, however, plan layout, installation and maintenance cost difficulties and uncertainties resulted in their being not selected.

The initial cost of a wave barrier type structure would be substantially higher than the cost of a rubble structure in minus one or two meters of water and would not be considered economically feasible unless it was placed in deeper water where it could also be used as a dock. Reflection and maintenance would still be points of concern.

After a careful examination of alternatives, a rubble mound breakwater alternative was chosen for the Valdez harbor. The breakwater design (rock sizes and layer thickness) is based on conservative wave (H_{10}) and accepted rock sizing criteria.

Life Cycle. Alaska harbors have typically been designed for a 50-year economic life. Previous harbor studies have analyzed the effect of project maintenance costs with an economic life less than the typical 50-year life. Assumptions and analyses attempted to determine the damage to the breakwater during the design 50-year frequency event and subsequent repair costs. The repair costs were included in an economic analysis to determine if it would be more cost effective to have a higher maintenance cost by designing for a lower frequency design storm vs. higher initial construction cost of the 50-year design storm. The results of these analyses indicated it was most cost-effective to design the breakwater to the 50-year design storm. The cost of producing and placing the armor stone designed for the reduced design storm events is not significantly different from the cost for the 50-year design storm event. This is especially true of the more remote harbor sites, but is also true for sites where quarries are closer to the harbor.

The initial construction cost for the curtain wall wave barrier is equal to or more than a rubble structure with the additional long term maintenance and replacement cost over the project life, which would result in a more costly project. In addition to the initial cost and high maintenance, the potential for wave reflection problems resulted in dropping the wave barrier from consideration. The use of rubble mound type breakwaters, which can withstand some settlement or minor damage, is the best and most common type of breakwater. A rubble mound breakwater is the recommended breakwater type for the Valdez Harbor Expansion project. The breakwater design (rock sizes and layer thicknesses) is based on conservative wave (H_{10}) and rock sizing criteria.

A brief review of breakwater maintenance projects with the Alaska District Construction Operations Division also indicates there has been very little maintenance on the District's breakwaters.

6.0 ALTERNATIVES CONSIDERED

6.1 General

Six alternative sites were considered for the development of additional harbor facilities in the Valdez Arm area. Most plans have been under discussion for many years and prior reports have removed them from further consideration. These are discussed briefly below. Three sites received serious consideration in this feasibility study: Harbor Cove, the East Site, and West Site. Harbor Cove dropped from consideration early in the study, because of the extreme environmental sensitivity of the cove, although it is favored by many in the community. Study efforts were directed to the East and West Site alternatives. These showed economic feasibility and more detailed studies were then made of the two sites.

Hydrographic surveys and geophysical studies were completed and the economics were reviewed in more detail. With this information, more detailed design analyses were made to locate the various project features within very restrictive physical and environmental constraints. The existing fill and Hotel Hill limit development to the north. The bathymetry drops off steeply to the south, limiting development in that direction. The SERVS dock, the existing harbor entrance channel and increasingly sensitive environmental constraints to the east limit east–west development. Specific details of each site are described and shown in the following sections.

The costs of these plans were compared to estimates of the economic benefits of the plans. Alternative plans were evaluated using established design criteria as found in the appropriate Corps of Engineer's Engineering Manuals (EM's) and the Shore Protection Manual (SPM).

6.2 Alternative Sites

6.2.1 General – The following sites were considered in earlier reports and have been rejected from consideration. See Figure A-6.

6.2.2 Mineral Creek – The 1995 Reconnaissance Report by Raytheon listed this site. However, this site is not available for consideration for the development of a harbor. The land is to be used by others for a different project. Additionally, there is a seismic risk and sedimentation from the adjacent Mineral Creek to consider. The development cost would include extending the access road and utilities as a project cost. From a community perspective, it is located away from the business and tourist center and existing harbor related support facilities.

6.2.3 Old Town Site – This site was the original waterfront area for the City of Valdez before the 1964 earthquake. The near shore area contains ruins of the old waterfront structures. The area is subject to submarine landslides and because of the seismic considerations is precluded from construction of permanent structures. A 1965 Seismic Task Force recommended no Federal funding be used for construction in this area due to the seismic risk. The only development that has been considered is a minimal launch ramp facility. However, the area is far from the present city of Valdez and would require additional harbor staff, plus it would not provide the needed additional moorage capacity.

6.2.4 Allison Point – This location is the furthest from the present city of Valdez and has no existing utilities or facilities. There are no available adjacent uplands. This area does have a sport and recreational fishery and is adjacent to a fish hatchery. A limited preliminary analysis indicated that the costs would be more than sites adjacent to the City, would negatively impact the fish hatchery and sport fishery, and is not supported by the community.

6.2.5 Expansion of the Existing Harbor – This option was briefly studied, but quickly revealed that there was no available area in which the harbor could expand. The existing harbor project also has very poor water quality and most expansion plans would only make the situation worse. Increasing the depth of the existing harbor would not solve the crowding, damage and delay problems. Reconfiguring the harbor float system also is not a viable solution, since the available space is well utilized in its present layout.

6.2.6 Harbor Cove – This site is located just east of the existing harbor separated only by Kennicott Avenue. It is naturally well protected from winds and waves and is the local favorite harbor expansion location. It is also used for recreation and marine education experiences. However, the site is the most environmentally sensitive and would have the greatest impact on fish and wildlife. Resource agencies consider it very closely related to the Duck Flats area, an area cited as an Aquatic Resource of National Importance. This designation precludes any development, unless there are no other alternatives. Other sites east and west of the existing SERVS Dock did indicate feasible project locations, therefore Harbor Cove was dropped from further consideration.

6.3 Alternative Sites Considered in Detail

6.3.1 General – The sites noted above were dropped from consideration in reconnaissance level or very early preliminary feasibility studies, leaving only the No Action and East or West of the existing SERVS Dock sites available for further consideration. See Figure A-7.

6.3.2 No Action – The No Action plan would leave the community with no additional harbor moorage space. Increased rafting, crowding and damage would continue and become worse. Delays to commercial enterprises would increase costs and reduce the quality of product.

6.3.3 West Site – The West Site is located between the existing harbor entrance channel and the SERVS Dock. It is constrained physically on all four sides. The bathymetry drops off steeply into Port Valdez to the south, the existing harbor entrance channel is to the west, the existing upland fill is to the north and the SERVS Dock is to the east. However, this site is the least environmentally sensitive of the two sites under detailed consideration. Access to the harbor is provided via South Harbor Drive to City owned property at the northwest portion of the basin. The area needed to accommodate the design fleet on the limited area of the shallow tidal bench required that a portion of the harbor be excavated out of the existing upland fill to the north to allow for complete harbor protection using rubble-mound breakwaters alone. Plans to accommodate the design fleet with minimal excavation the upland fill to the north were also prepared using a combination of rubble-mound breakwaters and partial depth vertical wave barriers.

Wave Heights. The west alternative presents a significant challenge to waves attempting to enter the harbor basin. The entrance channel configuration is restricted by the steep drop off of the off shore bathymetry, the existing entrance channel and existing upland fill/staging area. The extreme wave height that could develop in the west harbor entrance would either be the refracted wave approaching from the west or the unrefracted wave approaching from the southwest.

From the west, the unrefracted wave approaching the harbor entrance would be about 1.2 meters high with a period of 4.1 seconds. Assuming a control depth of 4.6 meters (15 feet), and a tide of 3.7 meters, the depth in the entrance would be 8.3 meters. The azimuth of the offshore normal (normal to the bottom contours) is 208°, and the wave direction is 270°. Therefore the refraction angle is 62° (270-208). From this, a refraction coefficient, k_R, of 0.95 and a shoaling coefficient, k_S, of 0.95 are found. This gives a refracted wave height of

$$H = 1.2 * 0.95 * 0.95 = 1.1 \text{ meters}$$

in the harbor entrance. Waves from west winds would have a difficult time attempting to negotiate the bend into the harbor and most of the waves would simply propagate across the entrance and dissipate their energy on the breakwater.

Though considerably smaller, the southwest waves would have a greater opportunity of entering the basin. Recall that the design wave height from this direction is just over 0.5 meters. From this direction (or from any direction), a wave entering the harbor would have to contend with both the western and eastern breakwaters flanking the entrance. Even from the southwest, which would produce the most direct wave, waves would be affected by the western breakwater and would allow only a partial wave to proceed on toward the eastern breakwater. If the wave attenuating effects of the western breakwater are disregarded and all the attenuation effects are assumed to come from the eastern breakwater, then an uncomplicated case for analysis is produced. However, it should be borne in mind that this is conservative as some attenuation undoubtedly results from the first breakwater encountered.

With regard to the diffraction diagrams, one can see from any such diagram in the Shore Protection Manual that if one looks directly along the line of travel of a wave (from the southwest for this alternative) toward the obstruction, the wave height beyond the obstruction never exceeds 60% of the original wave height along this line or further into the obstruction's lee. For a wave whose original height is 0.5 m., the maximum value that would be realized (using diffraction diagrams) would be 0.3 m. beyond the obstruction and inline with the waves travel direction. This is shown in A-12.

One can also use the *Combined Reflection and Diffraction by a Vertical Wedge* routine supplied as part of the ACES program package to determine wave heights in the lee of an obstacle. This will generate diffracted wave heights on a user-defined grid. An analysis using this routine demonstrated an even smaller diffracted wave height than is represented on figure A-12.

Shoaling. Shoaling has not been a problem at the existing harbor entrance or within the existing harbor. Any littoral drift material will tend to move into deeper waters off the breakwaters. Suspended sediments, most likely from the Lowe River, also have not been a problem in the existing harbor and, therefore, are unlikely to be a problem in the new harbor.

6.3.3.1 West Site Rubble-mound 226-Boat Plan – The West Site Rubble-mound 226-Boat plan is located partially on the existing tidal flats and excavated into the existing uplands fill area. The 3.5 hectare (ha) mooring basin can contain a fleet of 226 vessels from 7.5 m to 16 m in length. The entrance channel is on the west end of the basin adjacent to the entrance of the existing Valdez Small Boat Harbor. The entrance channel is 40 meters wide, sufficient for one way traffic of the design vessel and two way traffic for the smaller vessels. Vessels would enter to the north and turn from 90 to 180 degrees to enter the maneuvering channel to access the fairways and floating docks. A small breach protected by a short stub breakwater is located at the east end of the mooring basin to provide the required fish passage. Figures A-9, A-10, A-11 and A-16 show the West Site Rubble-Mound 226-Boat Plan, profiles, sections, and wave diffraction diagram.

Harbor Basin. The basin would be approximately 312 m by 110 m and dredged to depths varying from –5.5 m at the entrance to –4 m in the center and to –2.7 m at the east end as the length and draft of the vessels dictated.

Breakwaters. Two breakwaters will be constructed to protect the harbor. The main south breakwater is 332 meters long and runs west from the upland fill to the entrance channel. It will contain 11,108 cu m of primary armor rock, 7,289 cu m of secondary rock and 28,152 cu m of core material. The south breakwater curls to the north at the entrance channel to provide the wave protection overlap. The west breakwater is 53 meters long running south from the existing upland fill. It contains 4,400 cu m of primary armor rock, 5,016 cu m of secondary rock and 7,543 cu m of core rock. These quantities include the primary and secondary rock protecting the entrance channel slope excavated from the existing upland fill area. This entrance channel slope protection extends from the base of the west breakwater to a point directly north of the end of the south breakwater.

The 1V:1.5H harbor side slope follows down to the basin depth. A toe trench will be excavated adjacent to the breakwater at a 1V:3H slope down to the basin depth. Then the breakwater core and secondary rock will be placed in the trench at the 1V:1.5H slope as the breakwaters are fully constructed. This approach is used to maximize the basin width in the north-south direction. Once the breakwaters are constructed, basin dredging could proceed within the protected area, as is often the case.

A breach approximately 3 meters at the bottom is located at the point where the main south breakwater nears the existing upland fill on the east end of the basin. The breach is protected by a small stub breakwater containing 532 cu m of primary armor rock, 154 cu m of secondary rock and 220 cu m of core rock. The crest elevation is +5.0 meters and crest width is 2.4 meters.

The breakwater crest elevation is determined from a combination of the tides, storm surge, wave setup, and wave run-up. Since these variables may differ with orientation to the wind, the determined crest elevation may not be equal in all parts of the harbor. However, to simplify the planning and design, the crest elevation is usually given as a single value for the harbor. For this case, the design tide level is 3.7 meters (MHHW), and the largest of the unrefracted significant wave heights is the 50-year extreme height from the west—1.2 meters. As an added factor of safety, the $H_{1/10}$, rather than the significant wave height is used. The $H_{1/10}$ value is assumed to be 1.27 times the 50-year significant wave height (1.2 meters) or about 1.5 meters. This will be designated as the H'_o (unrefracted design wave) to determine the crest elevation. The storm surge is estimated as 0.15 meters. The water depth (from the bathymetric survey) at the toe of the breakwater (d_s) is –2.4 meters (MLLW).

The wave run-up, R, is estimated using guidance provided in the Shore Protection Manual suggesting that:

$$d_s / H'_o \approx 3.$$

Using Stoa's curve, assuming the rough slope factor is 0.6 and the scale factor is 1, get

$$H'_o/(gT^2) = 0.01$$

and

$$R/H'_o = 1.2$$

So the run-up is

$$R = 1.2 * 1.5 = 1.8 \text{ meters.}$$

The design crest elevation is then:

<u>Top Elevation = 3.7 + 0.15 + 1.8 = 5.7 meters.</u>

The existing entrance channel breakwaters are at elevation 5.8 meters (19 feet) MLLW. The surrounding fill elevations for the existing shore-side facilities are approximately 6 meters. Based on local observation, there has been no overtopping of either the existing breakwaters or the fill. Wave transmission through the breakwater is also insignificant. A crest elevation of 6.0 meters is a reasonable choice. Minor overtopping or more likely wind driven spray could occur during a design event combined with extreme high tides. This is expected to be a very rare event.

As a general rule, the breakwater crest width should be equal to three armor units if overtopping is not considered a significant concern. Minor overtopping is acceptable. The minimum width can be obtained from the formula

$$B = nk_\Delta (W/w_r)^{1/3}$$

Where

- B = crest width
- n = number of stones (3 minimum)
- k_Δ = layer coefficient (SPM Table 7-13, use 1.0)
- W = Mass of armor unit (kg)(1352 kg)
- W_r = mass density of armor unit (kg/m3)(2643 kg/m^3)

The minimum crest width is then 2.4 meters. As noted from the above run-up calculation and local observation of the existing breakwaters and fill protection, which are at or slightly below the breakwater design crest elevation, there will be no significant wave overtopping of the breakwaters. Extending the primary armor down to the –H elevation on the harbor side of the south breakwater is considered unnecessary. The entrance breakwaters will have primary armor on both faces. The main south breakwater will have primary armor on the seaward face and crest. The harbor side face will be the secondary rock. The design engineer may, if desired, extend the primary armor down the harbor side. One wave height, to approximately the +3 m MLLW elevation would be considered sufficient with appropriate modification to the secondary and core rock.

The layer thickness utilizes a similar formula.

$$r = n\, k_{\Delta}\, (W/w_r)^{1/3}$$

Where

r = the average layer thickness
n = the number of armor units thick

and the other symbols are the same as the crest width formula.

Using a minimum of 2 armor units thickness for the primary armor layer, the thickness is then 1.6 meters. The secondary layer is 0.75 meters.

Rock Gradations. Methods described in the SPM using Hudson's equation were used to calculate the stone sizes. Design waves from the west direction controlled the stone size for the breakwaters. The largest unrefracted and unshoaled significant wave of 1.4 meters, 1V:1.5H side slope, specific gravity of 2.58, and a stability coefficient of 1.9 were used. Rock sizes were calculated using the H_{10} wave (1.4 m * 1.27) and Kd value of 1.9.

The slope armor gradation was calculated using the modified Hudson equation for a graded armor stone often used for revetments experiencing small wave heights. A 0.3-meter wave was assumed for inside the basin. The W_{min} weight was reduced to provide for filter requirements when placed over in-situ materials. The need for a filter layer under the slope armor is unknown pending additional geotechnical analysis to be conducted in a later project phase.

Preliminary proposed armor rock sizes are shown in Table A- 12 below.

Table A-14, ***Armor Rock Size (kilograms)***			
Classification	**W_{MAX}**	**W_{AVE}**	**W_{MIN}**
Armor	1690	1352	1016
Secondary	844	135	91
Core	91	14	2.2
Slope Armor	21.5	6.0	0.23

Foundation Conditions and Slope Stability. Foundation conditions for the breakwaters are composed primarily of coarse-grained soils with cobbles from 15 to 24 cm. These are suitable for support of the breakwaters with any foundation settlement occurring only during construction.

The breakwaters on the south side of the harbor are located near the steep drop off into Port Valdez. In general, the seaward toe of the breakwaters was placed at about 30 meters shoreward from the –4 meter contour to minimize the potential for slope instability. A preliminary slope stability analysis was made using the Corps of Engineers UTEXAS4 slope stability model to check for unstable conditions. Evaluations were made using the extreme high tide and at MLLW. Additionally, a quick seismic analysis was made using a seismic coefficient of 0.3 for the extreme high tide condition. The minimum recommended factor of safety is 1.5. Using the best available information for estimating materials properties, a preliminary minimum factor of safety of 1.6 was calculated. Additional soils investigations to determine material's properties and reanalyzing the slope stability is recommended during final design.

Field data collection included sub-bottom reflection profiling and test pits. No drill holes have been made. The results of the subbottom profiles indicate that no bedrock will be encountered at the West Site seaward of the existing fill. It is unlikely that bedrock will be encountered in the excavation of the existing fill. Geotechnical studies for this project contain additional information regarding the subsurface and foundation conditions.

Circulation. The circulation in the proposed West and East Site harbors was estimated using the methods outlined in " EFFECTS OF PLANFORM GEOMETRY ON TIDAL FLUSHING AND MIXING IN MARINAS, (Nece, et. al.).

The parameters that most effect circulation and harbor flushing are 1) Tidal Prism Ratio (TPR), 2) Planform Aspect Ratio of the basin (AR), 3) Ratio of the basin area to the channel cross-section (A/a), and 4) the relative roundness of the basin.

The TPR is governed by both the local tide conditions and the basin depth required for the design fleet. A shallow basin has a larger TPR and, therefore, a greater exchange of ambient water. The West site alternative has stepped the bottom elevations with deeper draft vessels near the entrance and shallow draft vessels to the back. This minimizes the average depth and maximizes the TPR. The TPR for the West site basin is roughly 0.42 for the average tide, which is considered good.

The ratio of basin area to channel area (A/a) is governed strongly by the requirement for moorage and navigation. The size of the fleet and mooring density will determine the basin size (A) and the vessel draft, beam, wave conditions, and tides will determine the channel cross-section (a). A large A/a value is preferred to achieve the momentum necessary for driving circulation cells. This can be improved by reducing the channel width or depth, however this can restrict navigation. The A/a parameter can also be improved by increasing the basin area, which will add to the cost and may not be economically justified. The A/a for the West site is roughly 100.

The Aspect Ratio (AR) should normally be no greater than 3 to 1 and preferably less than 2 to 1; however, like the other design parameters, this is determined primarily by site specific constraints. These include deep water on the south boundary, the existing harbor entrance on the west boundary, the SERVs dock on the east boundary and upland development on the north boundary. The West site basin that provides the necessary capacity and fits within the physical constraints has an aspect ratio of roughly 2.8 to 1. This would be considered marginally acceptable.

Rounding basin corners has been found to eliminate local stagnation zones and generally increase the mixing within the harbor. The West Site basin could improve circulation near the corners slightly by using a longer radius on the inside corners

Adding a second entrance or breach may improve circulation in elongated channels, particularly in locations of low tidal range and strong longshore currents. However secondary channels also decrease the A/a ratio, which robs some of the available energy that drives both convective and turbulent diffusion. Based on available information it is uncertain what effect the breach in the west site breakwater will have on circulation.

Summary and Quantification of Circulation and Flushing. The tidal range in Valdez is 3.70 meters with a mean tide level of 1.98 meters. If we assume an average bottom depth of –4.2 meters the tidal prism ratio is roughly 0.42. The West site basin has a rectangular shape with an aspect ratio of roughly 2.8 to 1. The ratio of basin area to the cross-section of the entrance is roughly 100. Based on this information we can interpolate an exchange coefficient from figure 6 of the Nece report. The figure gives us a gross exchange coefficient of roughly 0.32 with an efficiency of about 77%. This is greater than the 0.30 value of exchange that is normally considered the lower acceptable limit. It is also recommended that at least 95% of the basin exceed an exchange value of 0.15 (Cardwell and Koons, 1981). These lower values are characteristic of stagnation zones that may occur where circulation is minimal.

Due to the entrance channel alignment, one might expect the flood tide to generate some back eddies coming off the head of the main south breakwater, which might carry some circulation and turbulent diffusion back into the basin. This may be good for mixing;

however, the increased turbulent diffusion could decrease the convective diffusion that drives the large circulation cells.

From the Nece report optimum aspect ratios should be less than 2 and no more than 3. The aspect ratio of 2.8 is near the upper limit. In addition the Nece report shows that exchange drops off quickly for relatively wide entrances where the flood is aligned with the basin. The channel for the West site (and also the East site discussed later in this draft appendix) is slightly different than those modeled by Nece, so some caution should be used when interpreting results.

Based on the analysis described in the Nece report the circulation and water quality will be adequate.

Construction Dredging. The dredging will include an off shore portion on the existing tidal flat and excavation from the existing upland fill. Geophysical data collected as part of this study indicates the tidal flats dredge material to be sands and gravels with an occasional large rock/boulder that should be capable of being dredged with common clamshell type equipment. No bedrock was identified in these recent studies. There is an east-west rock ridge that extends from the small islands to the east of Hotel Hill, through Hotel Hill and extends to a larger hill to the west of the existing harbor. The West Site alternative harbors are located further from potential bedrock or large rock slabs expected near Hotel Hill, therefore little, if any, difficult or bedrock dredging is expected. This is further described in the geotechnical studies.

The portion of the existing uplands to be dredged is composed of dredge material from development of the existing harbor. Some debris of unknown quantity, size and make up could be encountered within the existing fill area. The upper portion above the high tide line could be excavated “in the dry” with standard earthmoving equipment, rather than a barge mounted clamshell dredge. Depending on the contractors operations, the “in the dry” excavation could extend to low tide.

A total of approximately 258,000 cubic meters of dredging would be required for the entrance channel, maneuvering channel and basin. Dredging side slopes will be 1V:3H. All dredged material will be disposed of in an approved disposal site in Two Moon Bay as part of the mitigation plan.

6.3.3.2 West Site Wave Barrier 313-Boat Plan – The West Site Rubble-mound 313-Boat plan is located partially on the existing tidal flats and excavated into the existing uplands fill area. The 5.0 hectare (ha) mooring basin can contain a fleet of 313 vessels from 9 m to 30 m in length. The basin is protected by a combination of rubble-mound breakwaters and a vertical wave barrier. The shallow tide flat areas and entrance channel are protected using the rubble-mound structures and the deeper areas on the slope are protected using the wave barrier. The entrance channel is on the west end of the basin adjacent to the entrance of the existing Valdez Small Boat Harbor. The entrance channel is 40 meters wide, sufficient for one way traffic of the design vessel and two way traffic for the smaller vessels. Vessels would enter to the north and turn 135 to 180 degrees to enter the maneuvering channel to access the fairways and floating docks. A small breach protected by a short stub breakwater is located at the east end of the mooring basin to provide the required fish passage. Figures A-12, A-13 and A-16 show the West Site plan, profiles, sections, and wave diffraction diagram.

Harbor Basin. The basin would be approximately 300 m by 170 m and dredged to depths varying from –5.5 m at the entrance to –4 m in the center and to –2.7 m at the west end as the length and draft of the vessels dictated.

Breakwaters. Four sections of rubble-mound breakwater will be constructed to protect the harbor in conjunction with the wave barrier. The entrance channel west breakwater is 57 meters long and runs south along the entrance channel from the upland fill area. It will contain 2,710 cu m of primary armor rock, 1,630 cu m of secondary rock and 2,950 cu m of core material. The entrance channel east breakwater also runs south paralleling the entrance channel to end with a tie-in section for the wave barrier. The entrance channel east breakwater will contain 6,720 of primary armor rock, 3,700 cu m of secondary rock and 11,780 cu m of core material. The east main breakwater is 50 meters long running south from the breach between the breakwater and the existing upland fill. It contains 2,720 cu m of primary armor rock, 1,620 cu m of secondary rock and 2,520 cu m of core rock. The east stub breakwater is 25 meters long and runs south from the existing upland fill along the east side of the breach. It contains 670 cu m of primary armor rock, 520 cu m of secondary rock and 130 cu m of core rock. Breakwater quantities given here exclude the primary and secondary rock protecting the entrance channel slope excavated from the existing upland fill area and the slope protection on the north mooring basin slope.

The section design of the rubble-mound breakwater structures incorporated in this plan is identical to that described above for the West Site Rubble-mound 226-Boat plan in Table A-12.

Rock Gradations. The rock gradations designated for the rubble-mound breakwater structures incorporated in this plan are identical to those described above for the West Site Rubble-mound 226-Boat plan in Table A-12.

Wave Barrier. The partial depth vertical wave barrier will provide wave protection in areas too deep for the use of a rubble-mound structure. The wave barrier will run from the south end of the entrance channel east rubble-mound breakwater to the south end of the east rubble-mound breakwater. The length of the wave barrier will be 486 m. The maximum water depth for wave barrier piling placement will be -35 m MLLW.

The partial depth vertical wave barrier structure will essentially be a vertical reinforced concrete wall that is held in place above the bottom by steel piling. The top elevation of the wave barrier‘s concrete wall will be +6.5 m MLLW and the bottom of the wall will be at elevation -5.75 m MLLW or the bottom if shallower.

The wave barrier will be comprised of three main parts; the vertical pile, the batter pile, and the reinforced concrete wall panel. The vertical pile will be a 36 inch diameter steel pipe section with a 5/8 inch thickness. Vertical piles will be placed on 6 m centers along the length of the wall. The vertical piles will be placed in water depths of up to -35 m MLLW and shall be driven into the bottom sediments a minimum of 12 m. The top elevation of the vertical piles will be +6.5 m MLLW. Batter piles will be placed on the outside of the harbor for lateral support. The batter piles will also be 36 inch diameter steel pipe sections with a 5/8 inch thickness. Batter piles will be attached to the top of the vertical piles with restraining bands. The batter piles will be driven into the bottom sediments at a slope of 3V:1H such that the pile tips will have a minimum vertical depth of 12 m. The reinforced concrete wall panels will be placed within channels on the restraining bands that are attached to the vertical piles. Multiple panels will be placed on top of one another until the required height is obtained. The dimensions of the reinforced concrete panels are 5.02 m in length, 2.45 m in height, and 0.20 m in thickness. Concrete wall panel length is nominal. The actual length will depend on the as-built spacing of the vertical piles. The concrete wall panels cannot exceed 5.5 m in length.

For additional information concerning the detailed design of the partial depth vertical wave barrier refer to the structural design analysis attached to this appendix.

Foundation Conditions and Slope Stability. Foundation conditions for the breakwaters are composed primarily of coarse-grained soils with cobbles from 15 to 24 cm. These are suitable for support of the breakwaters with any foundation settlement occurring only during construction.

The breakwaters on the south side of the harbor are located near the steep drop off into Port Valdez. In general, the seaward toe of the breakwaters was placed shoreward from the –4 meter contour to minimize the potential for slope instability. A slope stability analysis was made using the Corps of Engineers UTEXAS4 slope stability model to check for unstable conditions. The UTEXAS4 slope analysis indicates stability of the existing ground slope is unaffected by the construction of the breakwater approximately 9.5 meters from the perceived slope break. The analysis found that the breakwaters were stable with minimum factors of safety of 1.5 or greater.

Field data collection included sub-bottom reflection profiling, test pits, and bore holes. The results of the subbottom profiles indicate that no bedrock will be encountered at the West Site seaward of the existing fill. It is unlikely that bedrock will be encountered in the excavation of the existing fill. Geotechnical studies for this project contain additional information regarding the subsurface and foundation conditions.

Circulation and Flushing. Tidal flushing parameters recommended by Nece do not apply to unconfined basins. The partial depth vertical wave barrier section does not the block tidal current from flowing under the wave barrier wall panels. The area under the wave barrier panels allows tidal flow into and out of the harbor basin. This harbor configuration, using the partial depth wave barrier, will likely have tidal flushing conditions better than those of a similarly sized rubble-mound breakwater, but slightly worse than those of the undisturbed site. The only significant difference is that surface conditions will be affected. Floating debris and surface contamination such as oil and other petroleum products will still be trapped in the harbor just as if the harbor was confined. Best management practices for harbor operations will alleviate the problems associated with floating debris and surface contaminates.

Construction Dredging. The dredging will take place on the existing tidal flat and excavation from the existing upland fill. Geophysical data collected as part of this study indicates the tidal flats dredge material to be sands and gravels with an occasional large rock/boulder that should be capable of being dredged with common clamshell type equipment. No bedrock was identified in these recent studies. There is an east-west rock ridge that extends from the small islands to the east of Hotel Hill, through Hotel Hill and extends to a larger hill to the west of the existing harbor. The West Site alternative harbors are located further from potential bedrock or large rock slabs expected near Hotel Hill, therefore little, if any, difficult or bedrock dredging is expected. This is further described in the geotechnical studies.

The portion of the existing uplands to be dredged is composed of dredge material from development of the existing harbor. Some debris of unknown quantity, size and make up could be encountered within the existing fill area. The upper portion above the high tide line could be excavated “in the dry” with standard earthmoving equipment, rather than a barge mounted clamshell dredge. Depending on the contractors operations, the “in the dry” excavation could extend to low tide.

A total of approximately 205,700 cubic meters of dredging would be required for the entrance channel, maneuvering channel and basin. Dredging side slopes will be 1V:3H with the exception of the dredging along the existing uplands these slopes will be dredged to a 1V:2H. All dredged material will be disposed of in an approved disposal site in Two Moon Bay as part of the mitigation plan.

6.3.3.3 West Site Wave Barrier 228-Boat Plan – The West Site Rubble-mound 228-Boat plan is located wholly on the existing tidal flats. No excavation into the existing upland fill area is proposed. Like the 313-boat wave barrier plan this plan also includes a combination of rubble-mound breakwaters in the shallow areas of the tide flats and partial penetration vertical wave barrier in the deeper areas of the slope to the south. The 2.5 hectare (ha) mooring basin will contain a fleet of 228 vessels from 9 m to 30 m in length. The entrance channel is on the east end of the basin adjacent to the SERVS dock. The entrance channel is 40 meters wide, sufficient for one way traffic of the design vessel and two way traffic for the smaller vessels. Vessels would enter to the northeast and turn from 135 to 180 degrees to enter the maneuvering channel to access the fairways and floating docks. Figures A-14, A-15, and A-16 show the West Site Wave Barrier plan, sections, and wave diffraction diagram.

Harbor Basin. The basin would be approximately 370 m by 115 m and dredged to depths varying from –5.5 m at the entrance to –4 m in the center and to –2.7 m at the west end as the length and draft of the vessels dictated.

Breakwaters. Three breakwaters will be constructed as part of the harbor protection. The west breakwater is 55 m long and runs south from the upland fill area to the tie-in with the wave barrier. It will contain 2,340 cu m of primary armor rock, 1,430 cu m of secondary rock and 2,110 cu m of core material. The entrance channel west breakwater extends 27 m southwest to the tie-in with the wave barrier. It will contain 3,080 cu m of primary armor rock, 1,750 cu m of secondary rock and 4,830 cu m of core material. The entrance channel east breakwater is 110 meters long running south from the existing upland fill. It contains 5,430 cu m of primary armor rock, 3,140 cu m of secondary rock and 6,000 cu m of core rock. The breakwater quantities stated exclude the primary and secondary armor rock protecting the basin and entrance channel slope excavated from the existing tide land area. The slope protection extends from the base of the west breakwater along the north edge of the basin and around the edge of the entrance channel to the toe of the entrance channel east breakwater.

The section design of the rubble-mound breakwater structures incorporated in this plan is identical to that described above for the West Site Rubble-mound 226-Boat Plan.

Rock Gradations.
The rock gradations designated for the rubble-mound breakwater structures incorporated in this plan are identical to those described above for the West Site Rubble-mound 226-Boat plan in Table A-12.

Wave Barrier. The partial depth vertical wave barrier will provide wave protection in areas too deep for the use of a rubble-mound structure. The wave barrier will run from the south end of the west rubble-mound breakwater to the south end of the entrance channel east rubble-mound breakwater. The length of the wave barrier will be 456 m. The maximum water depth for wave barrier piling placement will be -25 m MLLW.

The structural description of the partial depth vertical wave barrier incorporated in this plan is identical to that described above for the West Site Wave Barrier 313-Boat plan. For additional information concerning the detailed design of the partial depth vertical wave barrier refer to the structural design analysis attached to this appendix.

Foundation Conditions and Slope Stability. Foundation conditions for the breakwaters are composed primarily of coarse-grained soils with cobbles from 15 to 24 cm. These are suitable for support of the breakwaters with any foundation settlement occurring only during construction.

The breakwaters on the south side of the harbor are located near the steep drop off into Port Valdez. In general, the seaward toe of the breakwaters was placed shoreward from the –4 meter contour to minimize the potential for slope instability. A slope stability analysis was made using the Corps of Engineers UTEXAS4 slope stability model to check for unstable conditions. The UTEXAS4 slope analysis indicates stability of the existing ground slope is unaffected by the construction of the breakwater approximately

9.5 meters from the perceived slope break. The analysis found that the breakwaters were stable with minimum factors of safety of 1.5 or greater.

Field data collection included sub-bottom reflection profiling, test pits, and bore holes. The results of the subbottom profiles indicate that no bedrock will be encountered at the West Site seaward of the existing fill. It is unlikely that bedrock will be encountered in the excavation of the existing fill. Geotechnical studies for this project contain additional information regarding the subsurface and foundation conditions.

Circulation and Flushing. Tidal flushing and circulation parameters for this alternative will be very similar to those of mentioned above for the West Site Wave Barrier 313-Boat plan.

Construction Dredging. The dredging will only include removal of material existing tidal flat. No removal of material from the existing uplands is required. Geophysical data collected as part of this study indicates the tidal flats dredge material to be sands and gravels with an occasional large rock/boulder that should be capable of being dredged with common clamshell type equipment. No bedrock was identified in these recent studies. There is an east-west rock ridge that extends from the small islands to the east of Hotel Hill, through Hotel Hill and extends to a larger hill to the west of the existing harbor. The West Site alternative harbors are located further from potential bedrock or large rock slabs expected near Hotel Hill, therefore little, if any, difficult or bedrock dredging is expected. This is further described in the geotechnical studies.

A total of approximately 157,000 cubic meters of dredging would be required for the entrance channel, maneuvering channel and basin. Dredging side slopes will be 1V:3H with the exception of the north basin slope which will be 1V:2H. All dredged material will be disposed of in an approved disposal site in Two Moon Bay as part of the mitigation plan.

6.3.4 East Site – The East Site is located between the existing SERVS Dock and the eastern end of Hotel Hill on the existing tidal flats the south of Hotel Hill. See Figure A-7. The same criteria have been used to design the basins and breakwaters as those used for the West Site.

Hotel Hill to the north and the steep drop off into Port Valdez establish the north and south harbor feature limits. The east site alternatives also have to include the relocation of a primary fiber optic communications cable located to the east of the SERVS Dock. The only practical potential for harbor expansion is in the eastern direction, which is also limited by highly sensitive environmental conditions. The East Site is more environmentally sensitive due to the tidal flat habitat that improves in an easterly direction in addition to the increased potential impact to the more highly sensitive Harbor Cove and Duck Flats area.

Many harbor layouts were developed attempting to minimize the negative aspects of the East Site. Several early configurations are shown in Figure A-13. The fiber optic cable on the west end and increasing environmental sensitivity to the east caused considerable concern in arriving at an acceptable plan. The further east the harbor was placed, the more resistance received from the environmental community. Moving to the west increased costs for relocation of the fiber optic cable. In addition, the geophysical investigations revealed the potential presence of rock slabs closer to Hotel Hill that would result in more costly basin excavation. The plan also significantly reduces the size of the dredge material disposal area that would be eventually used for upland harbor access. The East Site harbor plans described below are the most acceptable plans that were developed.

The harbor is located as close as possible to the SERVS Dock, allowing navigation setback from the dock. Breaches were added at the east and west ends for juvenile fish migration from the Duck Flats area through the basin. A small stub breakwater protects the western breach.

The entrance to the harbor is located on the eastern end of the harbor away from potentially conflicting navigation at the SERVS Dock. Vessels would enter in a northeasterly direction turning 90 degrees to 180 degrees to enter the maneuvering channel and mooring floats in the inner harbor. The larger radius to the entrance channel should provide good entry/exit conditions. The entrance is 40 meters wide, the same as the West Site.

Wave in the Harbor Entrance. The extreme wave height that could develop in the harbor entrance for this alternative would either be the refracted wave approaching from the west or the unrefracted wave approaching from the southwest. The southeast wave would have to undergo refraction through approximately 90° before entering the harbor. This would severely reduce its height and is therefore not considered.

From the west, the unrefracted wave approaching the harbor entrance would be about 1.4 meters high with a period of 4.3 seconds. Assuming a control depth of 4.6 meters (15 feet), and a tide of 3.7 meters, the depth would be 8.3 meters. The azimuth of the offshore normal (normal to the bottom contours) is 198°, and the wave direction is 270°. Therefore, the refraction angle is 72° (270-198). From this, a refraction coefficient, k_R, 0.94, and a shoaling coefficient, k_S, of 0.86 are found. This gives a refracted wave height of

$$H = 1.4 * 0.94 * 0.86 = 1.1 \text{ meters}$$

in the harbor entrance.

Since the refracted wave from the west is as large or larger than the unrefracted wave from the southwest, it is the maximum wave that can develop in the entrance.

Wave in the Harbor Basin. The East Site also presents a significant challenge to waves attempting to enter the harbor basin from all directions. The bend in the harbor entrance leading into the basin is even more severe (approximately 135°) than for the West Site. Such a severe bend will easily reduce the wave height, through diffraction and refraction, from the west to less than 0.3 meters, but the smaller more direct wave approaching from the southwest will travel through the entrance channel until it is dissipated on the east breakwater. The discussion pertaining to analyzing waves entering the harbor presented for the West Site alternatives also applies to this alternative. The diffraction diagrams for the entrances are shown in Figures A-21 and A-24.

Shoaling. As discussed for the West Site, similar conditions will apply to the East Site. The east breakwater will divert any littoral drift to deep water before it enters the entrance channel. There may be a minor amount of shoaling from the west, but this is not expected to be significant. Suspended sediments have not been a concern in the existing harbor and are not expected to be a concern for the new harbor.

6.3.4 East Site Rubble-mound 178-Boat Plan – The East Site Rubble-mound 178-Boat Plan includes three rubble-mound breakwaters, dredged entrance channel, maneuvering channel and mooring basin, and a small upland disposal area. The upland disposal site has an area of 0.465 ha. (1.15 acre) and will be located at the northwest corner of the harbor. The 1.15 acres of staging area is insufficient and additional planning is required to identify appropriately sized staging areas. The 3.5 hectare (ha) basin will contain a fleet of 228 vessels from 9 m to 30 m in length. The entrance channel is on the east end of the harbor. The entrance channel is 40 meters wide, sufficient for one way traffic of the design vessel and two way traffic for the smaller vessels. Vessels would enter to the northeast and turn from 135 to 180 degrees to enter the maneuvering channel to access the fairways and floating docks. Figure A-18, A-19 and A-20 show the plan, profile and section views of the East Site Rubble-mound 178-Boat Plan.

Harbor Basin. The basin is approximately 120 meters by 285 meters with depths of –2.7 meters, -4 meters and –5.5 meters, similar to the West Site.

Breakwaters. Three breakwaters will be constructed to protect the harbor. The main south breakwater is 396 meters long and protects the south side of the harbor. The eastern most 80 meters of the south breakwater angle to the northeast forming the west side of the entrance channel. The east breakwater, approximately 183 meters long, curves in an arc from the northeast to northwest to form the eastern side of the entrance and harbor. The east breakwater stops short of Hotel Hill forming the eastern breach. Side slopes are 1V:1.5H. Both breakwaters will be constructed in 0.0 MLLW to –3 MLLW meters water depths. A small stub breakwater protects the western breach. It is 30 meters long with a crest elevation of +5 meters and constructed similar to the south and east breakwaters.

The 1V:1.5H harbor side slope for the breakwaters follows down to the basin depth. A toe trench will be excavated adjacent to the breakwater at a 1V:3H slope down to the basin depth. Then the breakwater core and secondary rock will be placed in the trench at the 1V:1.5H slope as the breakwaters are fully constructed. This approach is used to maximize the basin width in the north-south direction. Once the breakwaters are constructed, basin dredging could then be completed within a protected area, as is often the case.

Breakwater quantities for both breakwaters and stub (not including the disposal berm) are 24,442 cu m armor rock, 19,040 cu m secondary rock and 35,817 cu m core rock.

The disposal berm will be constructed using breakwater core material with slope armor protection provided on the basin side slopes. The disposal berm will need to be constructed similar to the breakwaters, i.e. a toe trench excavated and the core and slope armor protection built up to form the berm. Approximately 8,057 cu m of core stone and 881 cu m of slope protection material would be required.

For this alternative, the harbor is protected from the largest waves from the west by natural and man-made features. Therefore, the largest wave from the west (1.4 meter) is used as the design wave. The $H_{1/10}$ wave height and period for this wave are 1.8 meters (H_S = 1.4 for southeast waves) and 4.3 seconds. The design water depth at the toe is 4.0 meters.

Using a similar analysis that was used on the western option:

$$R/H'_o \text{ meters} = 0.8$$

So the run-up is

$$R = 0.8 * 1.8 = 1.5 \text{ meters}$$

Since these conditions are identical to those of the west site the design crest elevation is then:

$$\underline{\text{Top Elevation} = 3.7 + 0.15 + 1.5 = 5.4 \text{ meters.}}$$

Since the surrounding elevations of the existing entrance breakwaters and fill area are at 6 meters, experience dictates it is more practical to equate the crest elevation to that value.

Rock Gradations. The rock gradations are the same as those for the West Site as shown in Table A-12.

Foundation Conditions and Slope Stability. The foundation conditions are expected to be the same as those for the West Site, except as one approaches Hotel Hill from the south. The closer to Hotel Hill, the more likely large rock slabs may be encountered. These slabs could affect the dredging and subsequent inner harbor float pile driving. The slope stability is assumed to be equal to or better than that of the East Site Rubble-Mound 320-Boat Plan, which has a preliminary minimum factor of safety of 1.5, since this plan incorporates a large setback from the steep slope at he edge of the tideflats.

Circulation. The circulation in the proposed East Site basin was also estimated using the methods outlined in "EFFECTS OF PLANFORM GEOMETRY ON TIDAL FLUSHING AND MIXING IN MARINAS," (Nece, et. al.).

The circulation and flushing characteristics for the East Site should be slightly better than for the West Site. The calculated Tidal Prism Ratio (TPR) for the harbor is 0.57. The Aspect Ratio is 2.5 to 1. The estimated average exchange coefficient based on the Nece results will be 0.43. This harbor does have more tidal openings into the basin (entrance and two breaches) so the 'a' value will be more; however, because of the added tidelands the 'A' value will also increase. The resulting A/a for this harbor is 136. The corners of the harbor are also rounded to reduce the possibility of local stagnation zones. The entrance channel alignment is more conducive to driving the convective diffusion so circulation cells should be more pronounced. As with the West Site, there will be currents, but these should be small and have little effect on vessel navigation. So based on the recommended acceptable values for the four important circulation values the East Site 178-Boat Plan circulation is also considered adequate.

Construction Dredging. All dredging for the East Site Rubble-Mound 168-Boat Plan will be in the tidal flat to the south of Hotel Hill and east of the SERVS Dock. The materials are expected to be similar to the West Site for the most part. As the dredging gets closer to Hotel Hill on the north, evidence was found in the geophysical data that larger rock slabs would be encountered in the dredge material. There was no evidence of bed rock within the zone to be dredged. The rock slabs may need to be broken up prior to being moved. There are several options the contractor could use to break up the slabs, for example, drill and blast, drill and split or break up using a large impact hammer (pile driver). Much will depend on the quantity and character of the slabs and the contractor's chosen method of operation. Once the material is broken up, since it is not bedrock, it will become part of the material to be dredged. Cost estimates assumed up to 10 percent of the total dredged material would require "extra effort" to remove.

A total of 175,648 cu m of dredging will be required for the entrance channel, maneuvering channel and mooring basin. Dredging side slopes will be 1V:3H. A small portion of the dredge material, approximately 16,500 cu m, will be disposed of in the upland disposal area at the northwest corner of the harbor. The remainder of the dredge material will be disposed of in the mitigation site at Two Moon Bay.

6.3.4 East Site Rubble-mound 320-Boat Plan – The East Site Rubble-mound 320-Boat Plan also includes three rubble-mound breakwaters, dredged entrance channel, maneuvering channel and mooring basin, and a small upland disposal area. The upland disposal site has an area of 1.87 ha. (4.62 acres) and will be located along much of the length of the north edge of the mooring basin. The 4.62 acres of staging area is still likely insufficient to handle the upland requirements of a 300 plus boat harbor so again additional planning will be required to identify appropriately sized staging areas. The 5.0 hectare (ha) mooring basin will contain a fleet of 320 vessels from 9 m to 30 m in length. The entrance channel will be on the east end of the harbor. The entrance channel will be 40 meters wide, which is sufficient for one way traffic of the design vessel and two way traffic for the smaller vessels. Vessels would enter to the northeast and turn from 135 to 180 degrees to enter the maneuvering channel to access the fairways and floating docks. Figure A-22, A-23 and A-24 show the plan and section views and diffraction diagram of the East Site Rubble-mound 320-Boat Plan.

Harbor Basin. The basin is approximately 130 meters by 435 meters with depths of –2.7 meters, -4 meters and –5.5 meters, similar to the West Site. The dredge area of this harbor will be slightly different from those of the other four alternatives. The dredge slopes will be separate from the breakwater footprint. No dredging under the toe berm of the breakwaters or the upland disposal area will occur. This reduces dredge volumes and breakwater rock quantities while slightly increasing the overall length and width of the harbor. The inner harbor slope will be dredged to a 1V:2H and covered with slope protection. The inner dredge slopes will remain unprotected and dredged at a 1V:3H slope.

Breakwaters. Three breakwaters will be constructed to protect the harbor. The main south breakwater is 473 meters long. The eastern most 70 meters of the south breakwater angles to the northeast forming the west side of the entrance channel. The east breakwater, approximately 240 meters long, curves in an arc from the northeast to northwest to form the eastern side of the entrance and harbor. The east breakwater stops short of Hotel Hill forming the eastern breach. Side slopes are 1V:1.5H. Both breakwaters will be constructed in 0.0 MLLW to –5 MLLW meters water depths. A small stub breakwater protects the western breach. It is 30 meters long with a crest elevation of +5 meters and constructed similar to the south and east breakwaters.

The section design of the rubble-mound breakwater structures incorporated in this plan is similar to that described previously for the West Site Rubble-mound 226-Boat. The changes are that the toe berm of the breakwater will be placed on the tide flat instead of the dredged basin. See Figure A-23 for the breakwater section of the East Site Rubble-Mound 320-Boat Plan.

Breakwater quantities for the south, east, and stub breakwaters, not including the disposal berm, are 37,300 cu m armor rock, 21,000 cu m secondary rock and 46,700 cu m core rock.

Rock Gradations. The rock gradations designated for the rubble-mound breakwater structures incorporated in this plan are identical to those described above for the West Site Rubble-mound 226-Boat plan in Table A-12.

Foundation Conditions and Slope Stability. The foundation conditions are expected to be the same as those for the West Site, except as one approaches Hotel Hill from the south. The closer to Hotel Hill, the more likely large rock slabs may be encountered. These slabs could affect the dredging and subsequent inner harbor float pile driving. A preliminary analysis of the slope stability was completed for the south breakwater section that is the closest to the steep slope at the south edge of the tide flats. The preliminary factor of safety for this worst case section is 1.5.

Summary and Quantification of Circulation and Flushing. As recommended by Nece the four parameters that most effect circulation and flushing are 1) Tidal Prism Ratio (TPR), 2) Planform Aspect Ratio of the basin (AR), 3) Ratio of the basin area to the channel cross-section (A/a), and 4) the relative roundness of the basin. The East Site Rubble-Mound 320-Boat harbor has a tidal prism volume of 283,000 cu m and a harbor volume of 529,000 cu m. This information results in a tidal prism ratio (TPR) for the harbor of 0.53 which is much higher than the recommended minimum value of 0.3. The harbor has a rectangular shape with an aspect ratio (AR) of roughly 2.9 to 1 which is on the upper end of the acceptable limit for aspect ratio. The ratio of basin area to the cross-section of the entrance (A/a), including fish breach areas is roughly 138 at mean tide level. Based on this information we can interpolate an exchange coefficient from figure 6 of the above referenced document. The figure gives us a gross exchange coefficient of roughly 0.42. This is greater than the 0.30 value of exchange that is normally considered the lower acceptable limit. It is also recommended that at least 95% of the basin exceed an exchange value of 0.15 (Cardwell and Koons, 1981). These lower values are characteristic of stagnation zones that may occur where circulation is minimal. The corners in the back of the elongated basin may create zones of stagnation that might have lower values of exchange but the corners have been rounded to minimize the low exchange values. From the results of the four important circulation parameters it is assumed that this harbor will have acceptable circulation and flushing.

Construction Dredging. All dredging for the East Site Rubble-Mound 320-Boat Plan will be in the tidal flat to the south of Hotel Hill and east of the SERVS Dock. The materials are expected to be similar to the West Site for the most part. As the dredging gets closer to Hotel Hill on the north, evidence was found in the geophysical data that larger rock slabs would be encountered in the dredge material. There was no evidence of bed rock within the zone to be dredged. The rock slabs may need to be broken up prior to being moved. There are several options the contractor could use to break up the slabs, for example, drill and blast, drill and split or break up using a large impact hammer (pile driver). Much will depend on the quantity and character of the slabs and the contractor’s chosen method of operation. Once the material is broken up, since it is not bedrock, it will become part of the material to be dredged. Cost estimates assumed up to 10 percent of the total dredged material would require “extra effort” to remove.

A total of 186,400 cu m of dredging will be required for the entrance channel, maneuvering channel and mooring basin. Dredging side slopes will be 1V:3H. A small portion of the dredge material, approximately 16,500 cu m, will be disposed of in the upland disposal area at the northwest corner of the harbor. The remainder of the dredge material will be disposed of in the mitigation site at Two Moon Bay.

7.0 PLAN IMPLEMENTATION

7.1 Aids to Navigation

Navigation marker bases will be constructed at the entrance channel ends of the breakwaters as part of the initial project. Navigation aids are typically installed and maintained by the U.S. Coast Guard upon completion of a project.

7.2 Operations and Maintenance

Operation and maintenance of the local service facilities would be accomplished by the City of Valdez. These include the mooring basin and float system and disposal berms. The Federal Government would be responsible for the breakwaters, entrance channel and maneuvering channel for the project. The Alaska District would make periodic site visits to inspect the breakwaters and accomplish hydrographic surveys of the harbor at approximately 5 year intervals. The inspections and surveys provide the information necessary to determine if maintenance of the breakwater or dredging of the basin, maneuvering channel or entrance channel is needed. Federal and local maintenance dredging would likely be combined to minimize costs by reducing the mobilization and demobilization cost, and maximizing the dredging quantity. The existing harbor and entrance channel have not required dredging since its completion over 50 years ago. Based on past experience with the existing harbor it is assumed that the harbor alternatives will not require maintenance dredging.

The breakwater was designed to be stable in storm conditions that could be expected in Port Valdez. Little, if any, loss of armor rock or other maintenance of the breakwater would be expected over the life of the project (50 years). Historically, breakwaters designed to the conservative criteria used for these new breakwaters for Valdez have experienced no deterioration requiring maintenance for approximately 35 years. However, a value of 2% of the armor stone has been assumed for evaluation of the alternatives to need replacement at 5-year intervals.

The wave barrier breakwater is still a relatively new type breakwater with just a handful of structure built worldwide. Well-built and properly protected steel and concrete structures can last well over 50 years even in a marine environment. However the lack of a long-term track record with wave barriers makes this type of prediction impossible. Besides the obvious inspection and maintenance required, a wave barrier is subject to the cyclical loading of the waves. This fatigue limits the life of steel, even when the loading is small. The fatigue analysis has not been completed, but until that is done, a conservative 25-year design life is justified. This outlook may be improved by ensuring through the design that base and connection materials are sufficiently non-stressed during the typical wave loading. However, in order to get even the 25-year life, an inspection and maintenance program must be faithfully executed. It is recommended that the local sponsor complete a bi-annual inspection of the entire wave barrier. This inspection will necessitate periodic replacement of sacrificial anodes, and galvanizing repairs. It is likely that some of the panels may experience some form of deterioration or damage over the life of the structure. While spare panels can be fabricated as part of the construction cost, provided there is warehouse space available, the equipment (barge and crane) and divers (release and re-secure the under water panels) necessary to replace them will be make this a fairly expensive item. It is assumed that wave barrier inspection, replacement of sacrificial anodes, galvanizing repairs, and panel replacement will be required an annual maintenance budget of 2% of the initial construction cost.

Shoaling has not been a problem at the existing harbor entrance or within the existing harbor. Any littoral drift material will tend to move into deeper waters off the breakwaters. Suspended sediments most likely from the Lowe River, also have not been a problem in the existing harbor and, therefore, are unlikely to be a problem in the new harbor.

7.3 Detailed Quantity Estimates

Quantity estimates for each alternative are provided in the Tables attached at the end of this draft Hydraulics Appendix. Quantities were estimated from detail drawings using AutoCAD software and were checked and verified by hand calculated quantities.

7.4 Construction Schedule

The major harbor construction items from the alternatives previously described include the wave barriers, rubble-mound breakwaters, dredging and disposal areas. The sequence of construction will depend on the components that make up the final plan but several construction sequencing requirements will dictate the construction schedule. The wave barrier used in two of the three west site alternatives will require that the rubble-mound breakwater sections in the tie-in section of the two breakwater type to be constructed after the completion of the wave barrier sections. For all the rubble-mound breakwaters except those in the East Site Rubble-mound 320-Boat Plan the breakwater toe trenches must be dredged first, followed by construction of the breakwaters. Basin dredging would then proceed as the breakwaters are constructed or wait until they are fully completed. The East Site Rubble-mount 320-Boat Plan will have no such restriction on the breakwater construction since no toe berm was used in the design. Dredging is limited to October 1st through March 31st. The construction time is estimated to be 24-36 months depending on the alternative selected for construction. Construction restrictions would be detailed in the development of plans and specifications.

7.5 References

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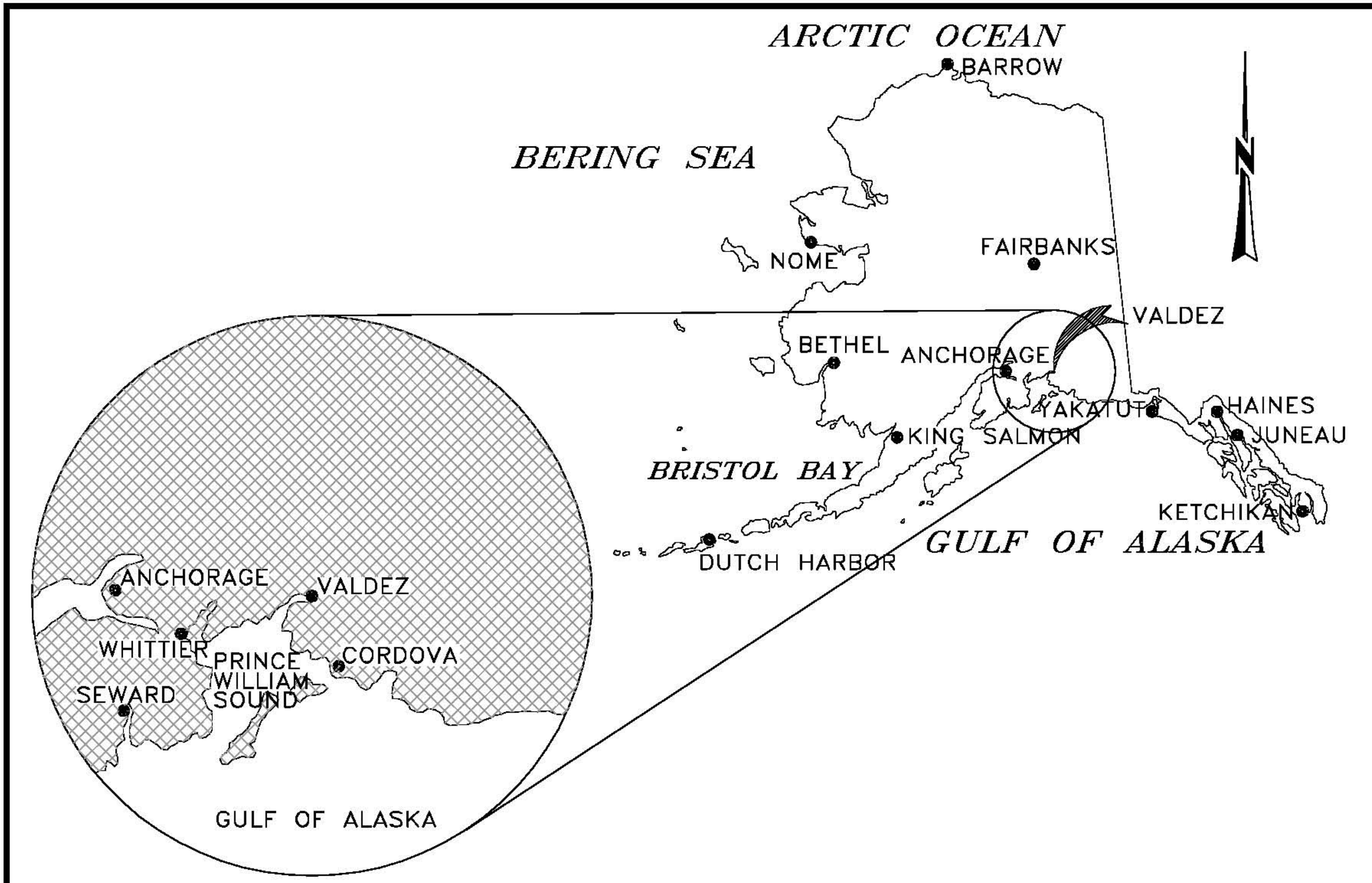
FIGURES



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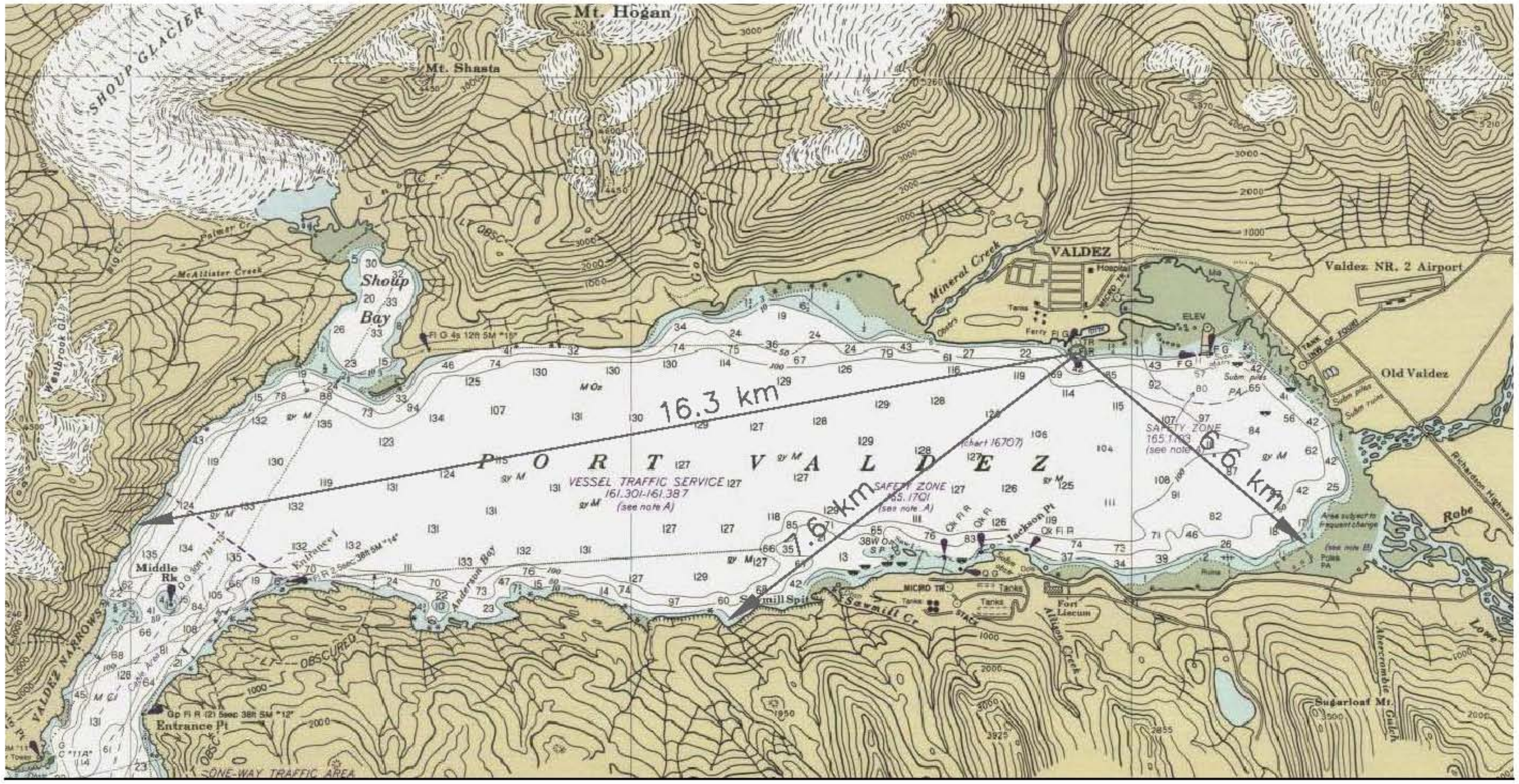
PROJECT LOCATION MAP

NAVIGATION IMPROVEMENTS, VALDEZ, ALASKA

A–1



KILOMETERS

1 0 1 2 3 4 5

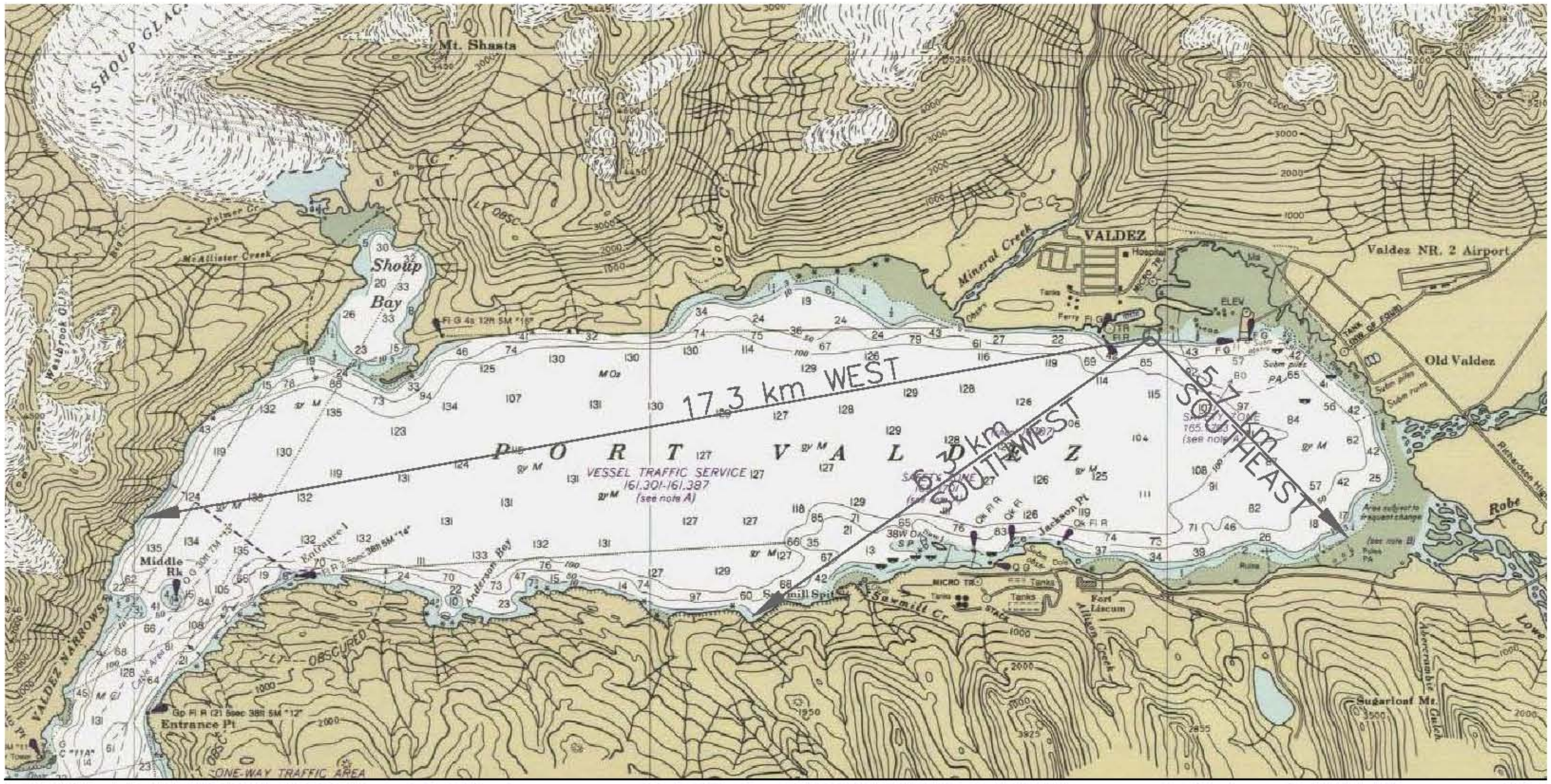
1 Nautical Mile = 1.852 Kilometers

EFFECTIVE FETCHES
WEST SITE

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

FIGURE A-2



KILOMETERS

1 0 1 2 3 4 5

1 Nautical Mile = 1.852 Kilometers

N

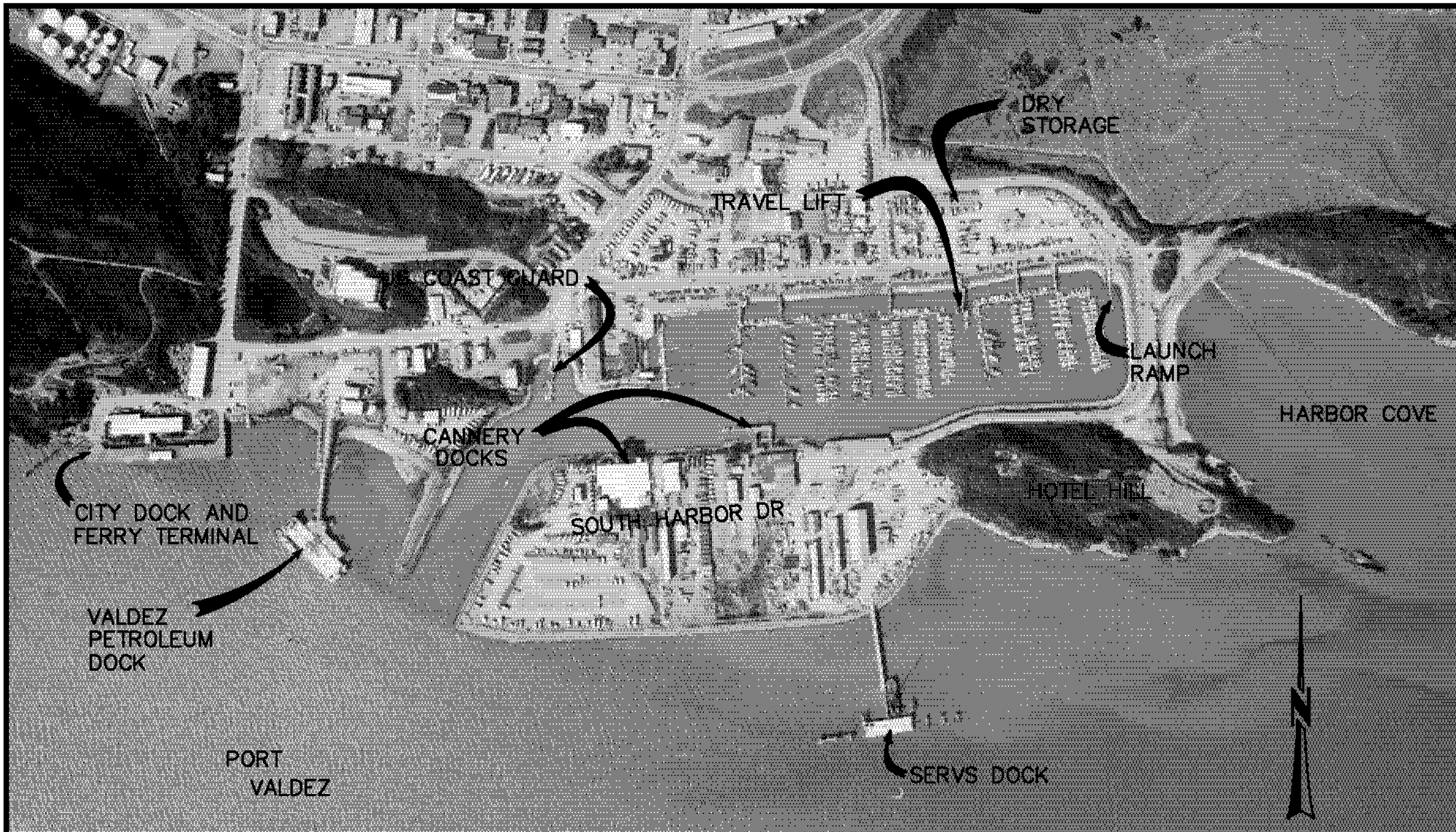
EFFECTIVE FETCHES
EAST SITE

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

FIGURE A-3

Tryck Nyman Hayes, Inc.



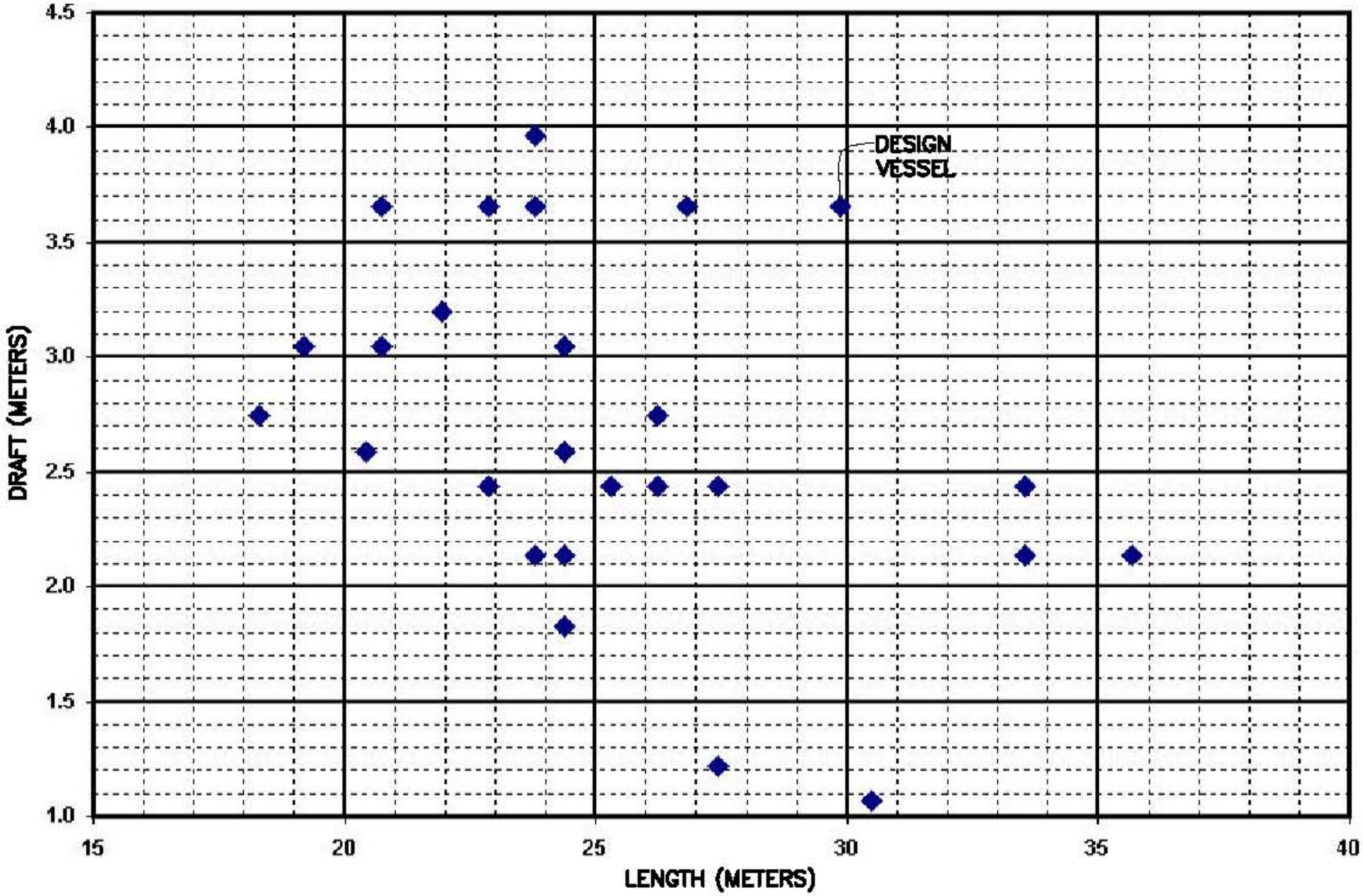
NOT TO SCALE

EXISTING HARBOR

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

FIGURE A-4

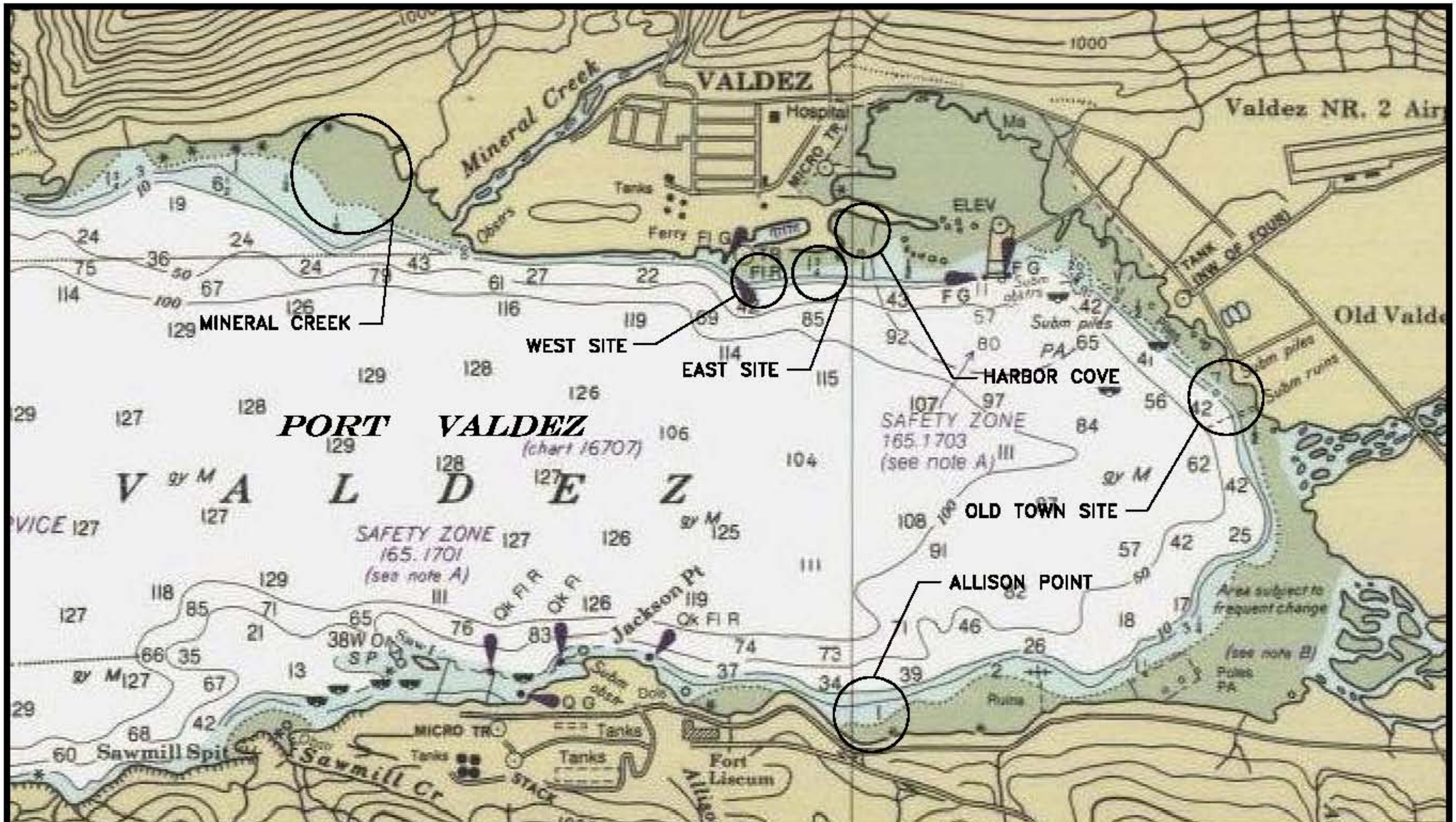


COMMERCIAL TENDERS
LENGTH v.s. DRAFT

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

FIGURE A-5



ALTERNATIVE SITES

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

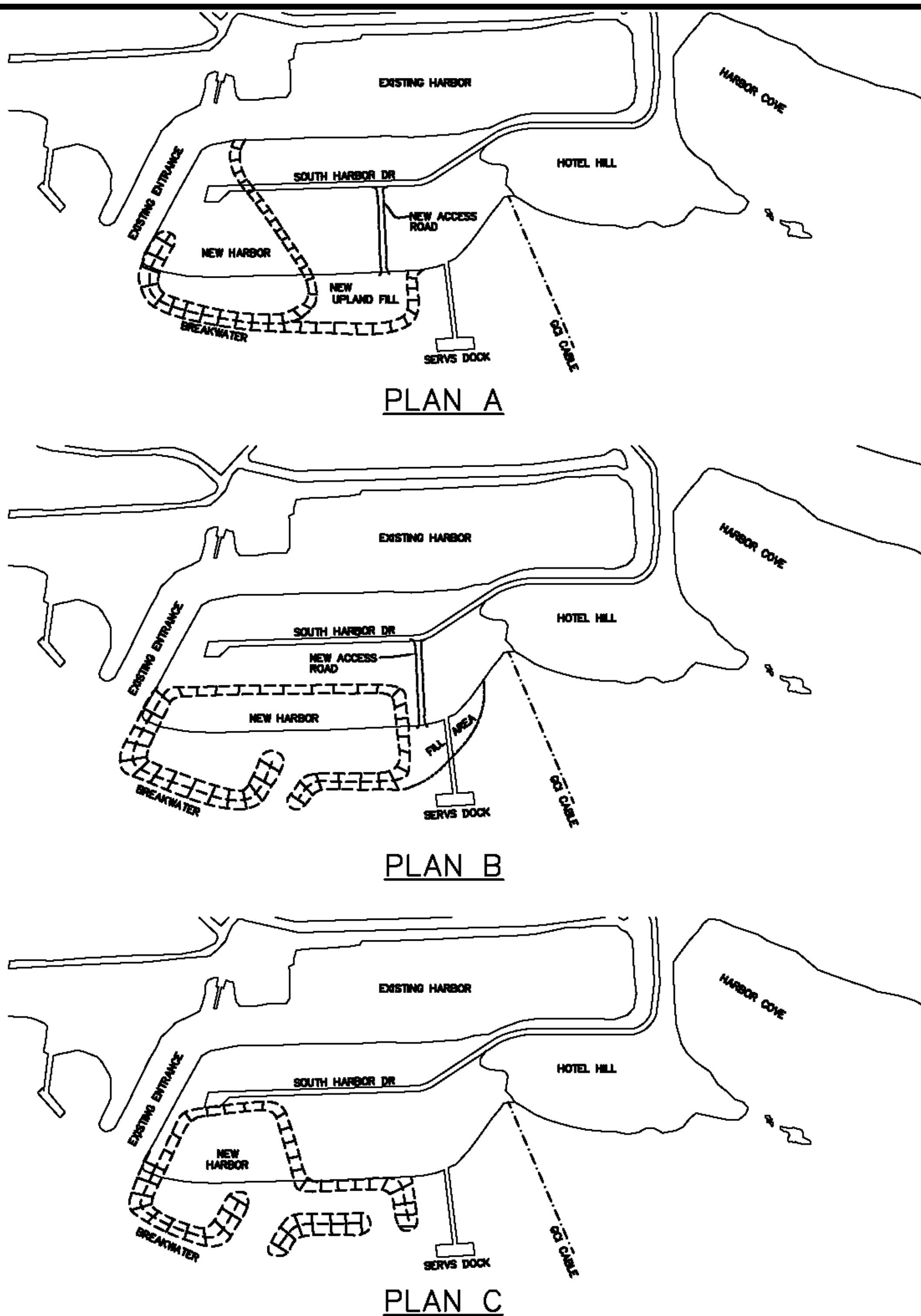
FIGURE A-6

SITES CONSIDERED IN DETAIL

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

FIGURE A–7

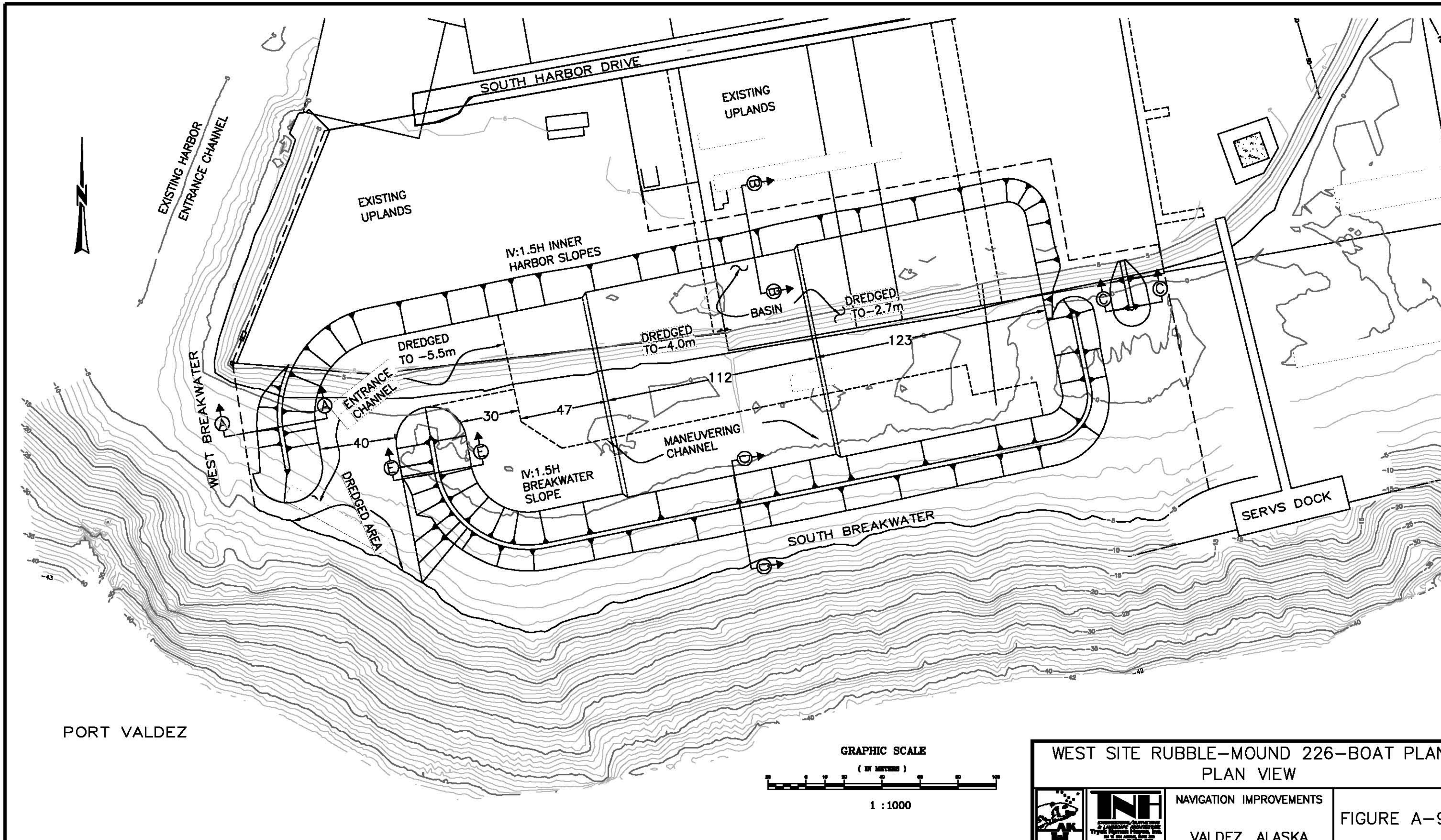


WEST SITE ALTERNATE
HARBOR PLANS

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

FIGURE A–8

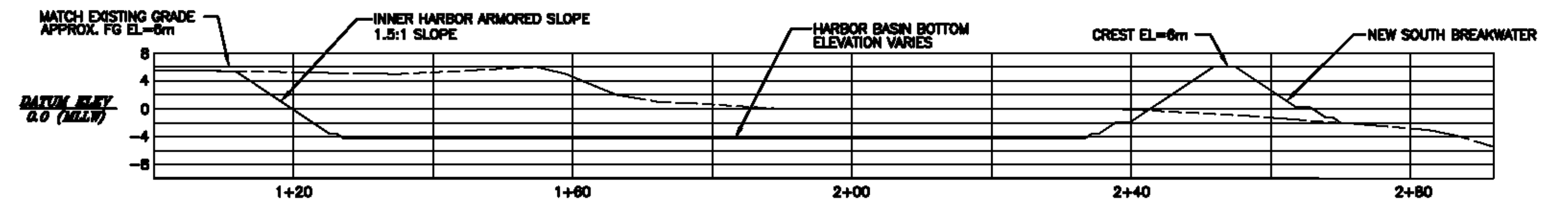


WEST SITE RUBBLE-MOUND 226-BOAT PLAN
PLAN VIEW

NAVIGATION IMPROVEMENTS

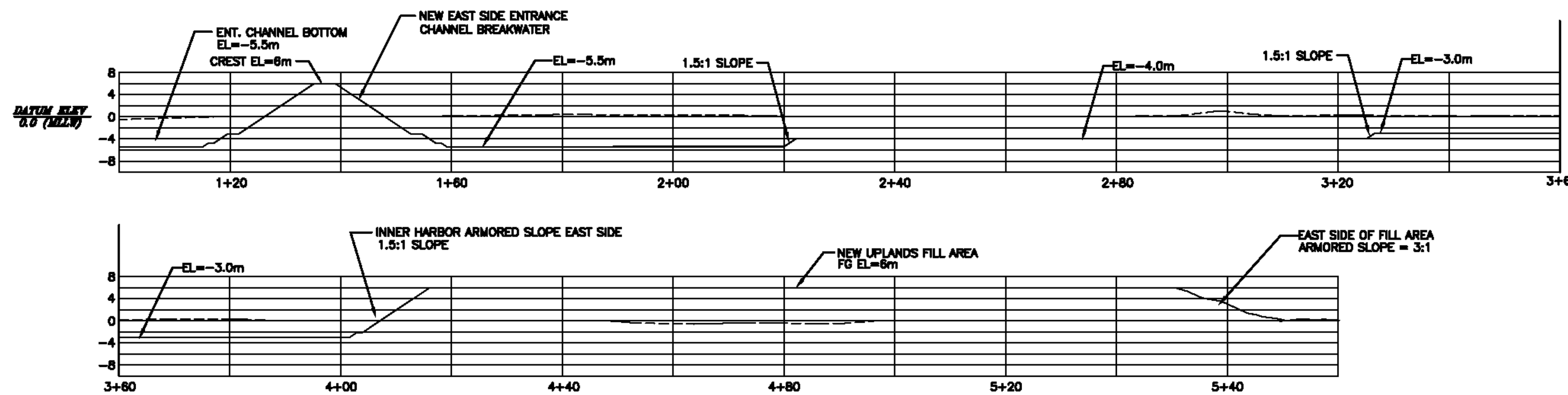
VALDEZ, ALASKA

FIGURE A-9



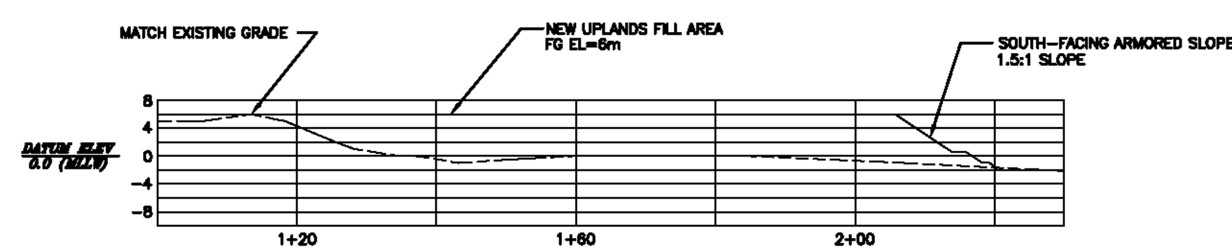
SECTION A

NORTH/SOUTH PROJECT SECTION EG & FG



SECTION B

EAST/WEST PROJECT SECTION FG & EG



SECTION C

NORTH/SOUTH PROJECT SECTION THROUGH UPLAND FILL AREA

LEGEND:

EXISTING GROUND - - - - - - - -

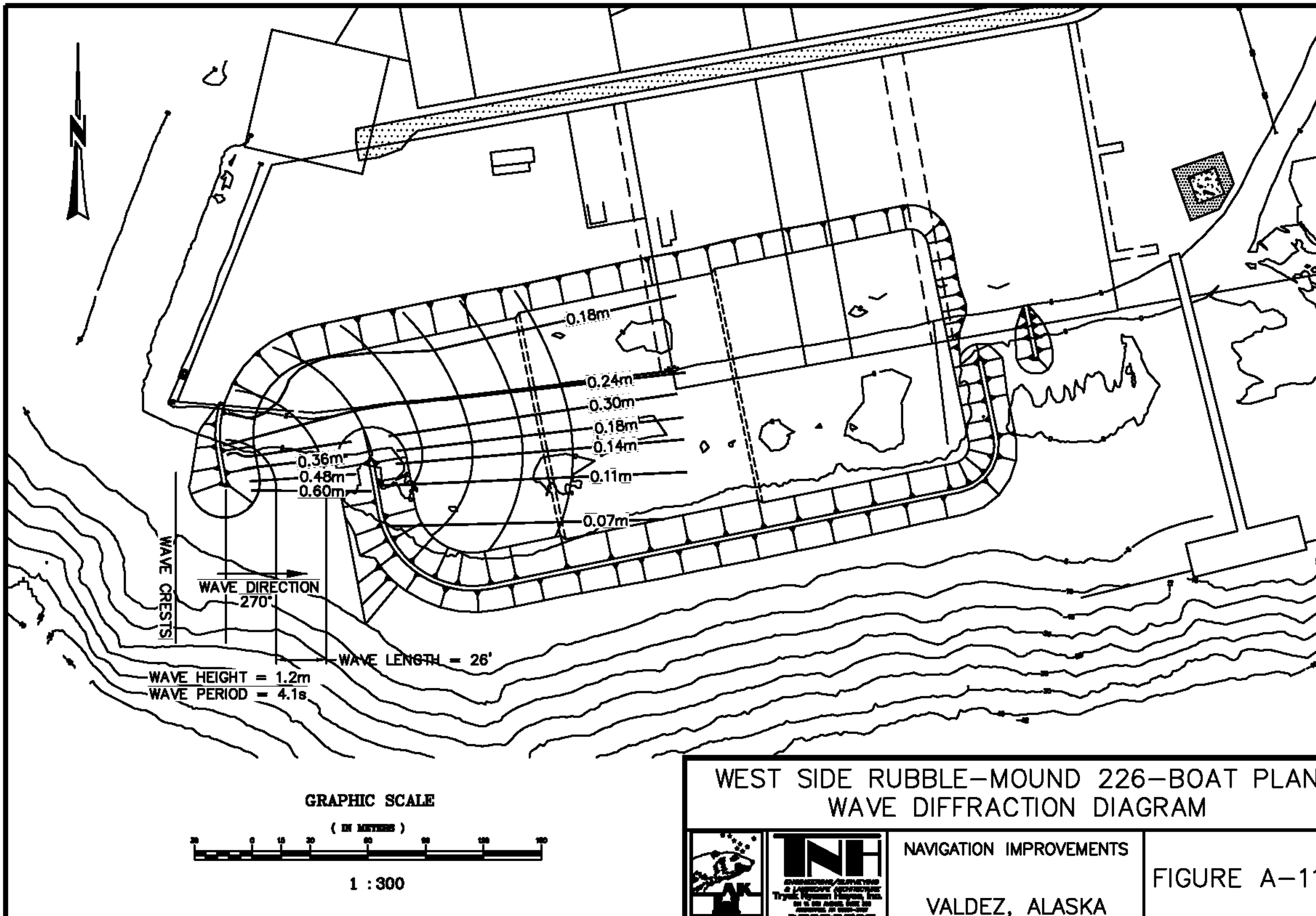
FINISH GRADE ————————

GRAPHIC SCALE

(IN METERS)

1 : 500

WEST SITE RUBBLE-MOUND 226-BOAT PLAN PROJECT PROFILES		
NAVIGATION IMPROVEMENTS		
VALDEZ, ALASKA	FIGURE A-10	

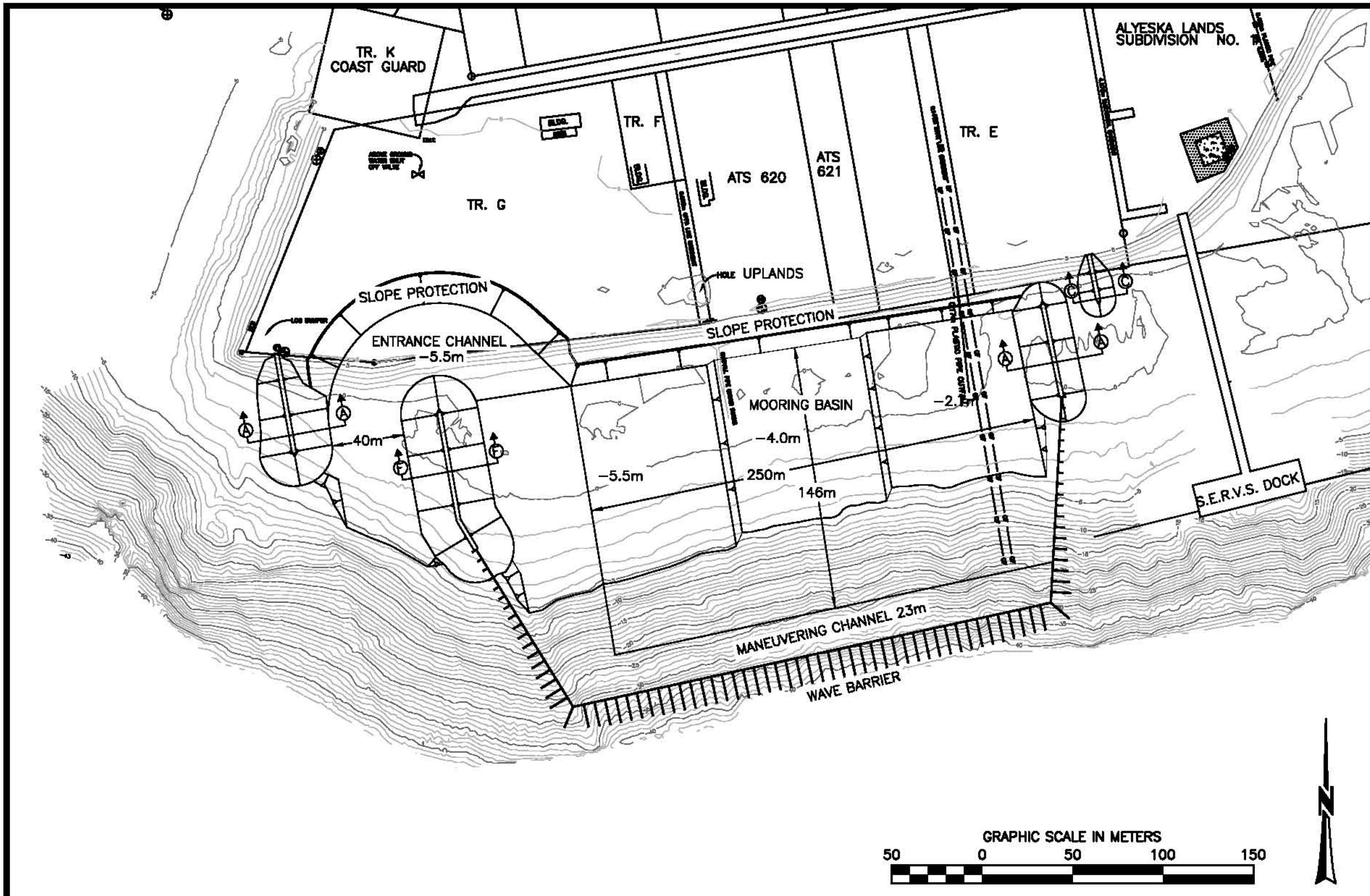


WEST SIDE RUBBLE-MOUND 226-BOAT PLAN
WAVE DIFFRACTION DIAGRAM

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

FIGURE A-11



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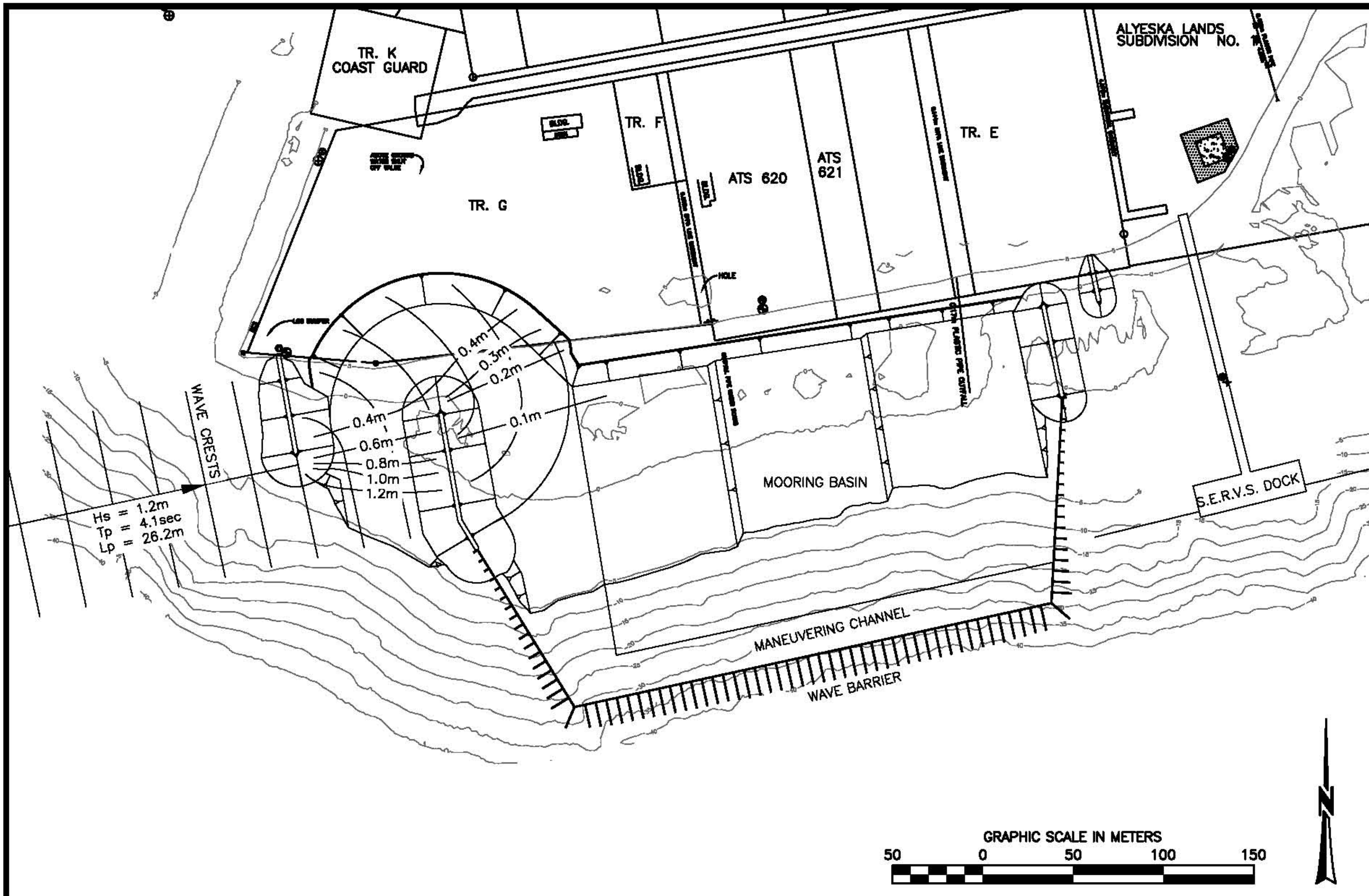
WEST SITE WAVE BARRIER 313–BOAT PLAN
PLAN VIEW
NAVIGATION IMPROVEMENTS, VALDEZ, ALASKA

A–12



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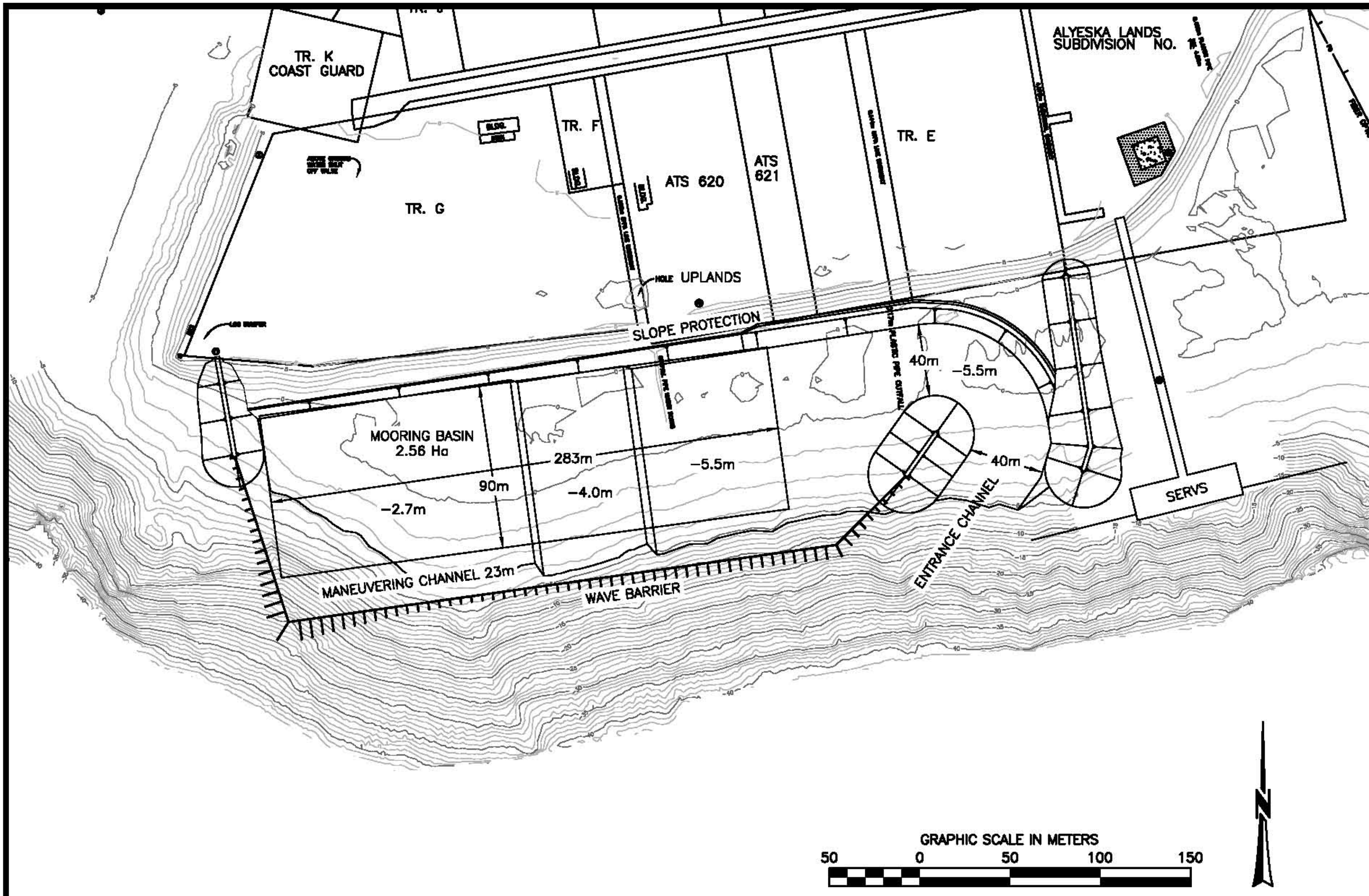
WEST SITE WAVE BARRIER 313-BOAT PLAN
WAVE DIFFRACTION DIAGRAM
NAVIGATION IMPROVEMENTS, VALDEZ, ALASKA

A-13



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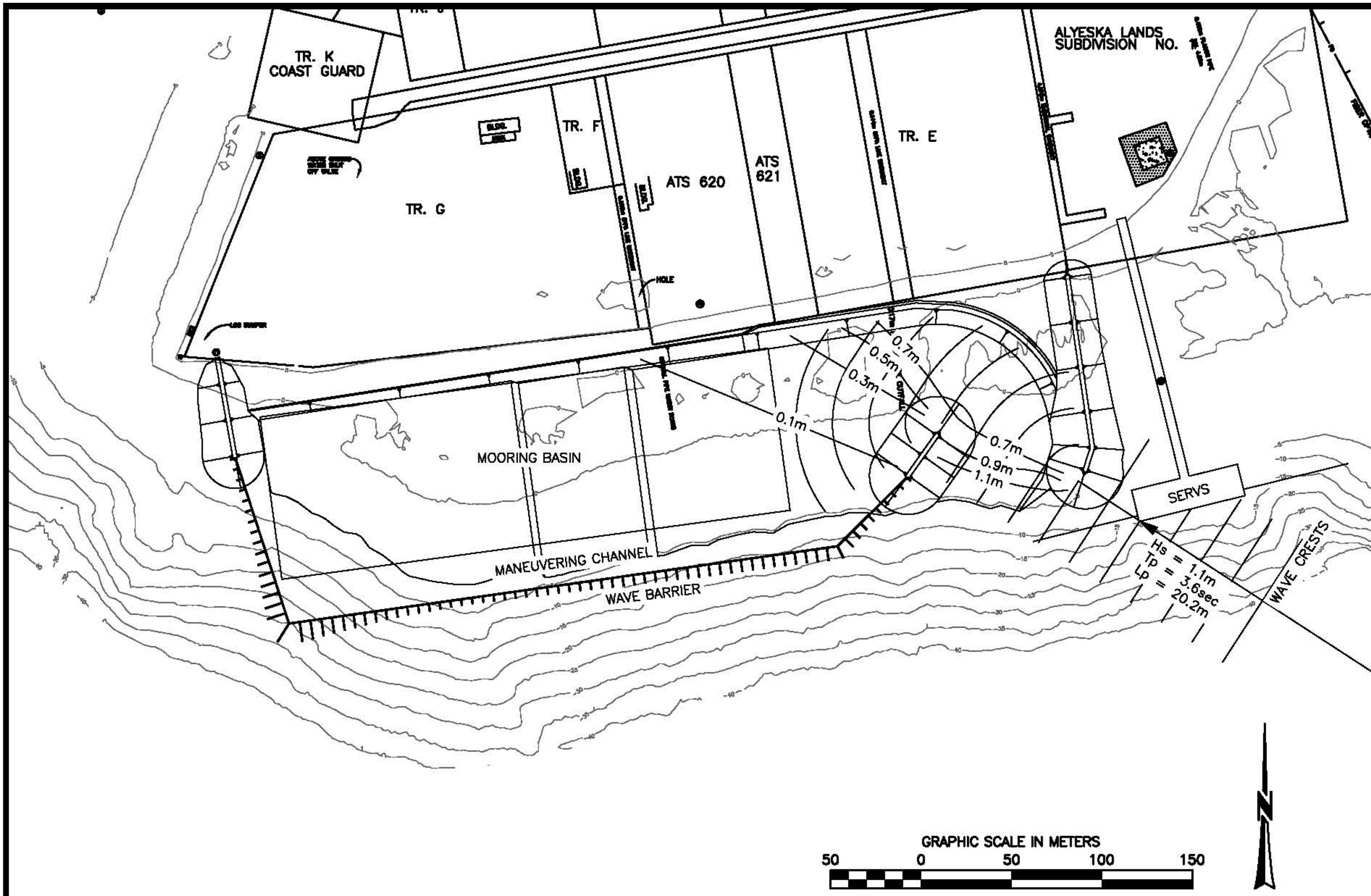
WEST SITE WAVE BARRIER 228-BOAT PLAN
PLAN VIEW
NAVIGATION IMPROVEMENTS, VALDEZ, ALASKA

A-14



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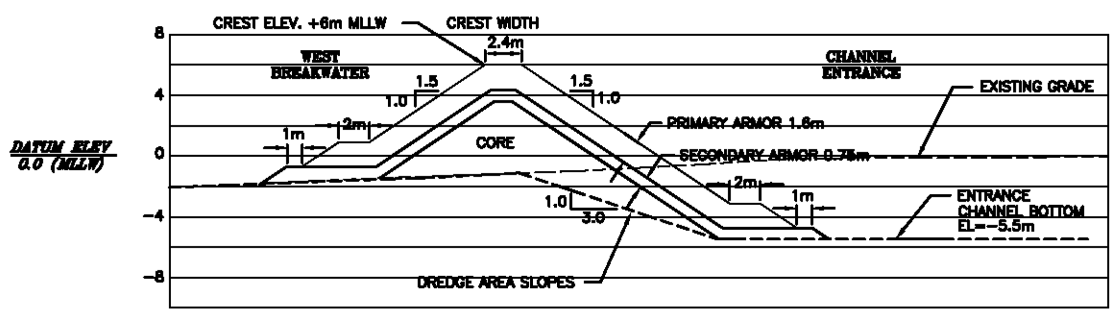
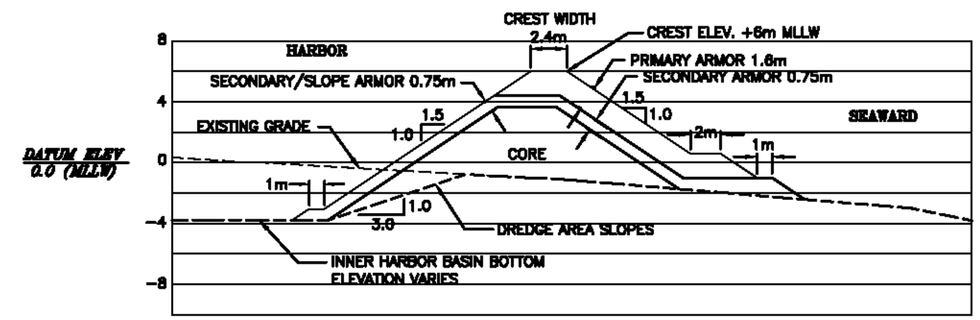
WEST SITE WAVE BARRIER 228-BOAT PLAN
WAVE DIFFRACTION DIAGRAM
NAVIGATION IMPROVEMENTS, VALDEZ, ALASKA

A-15



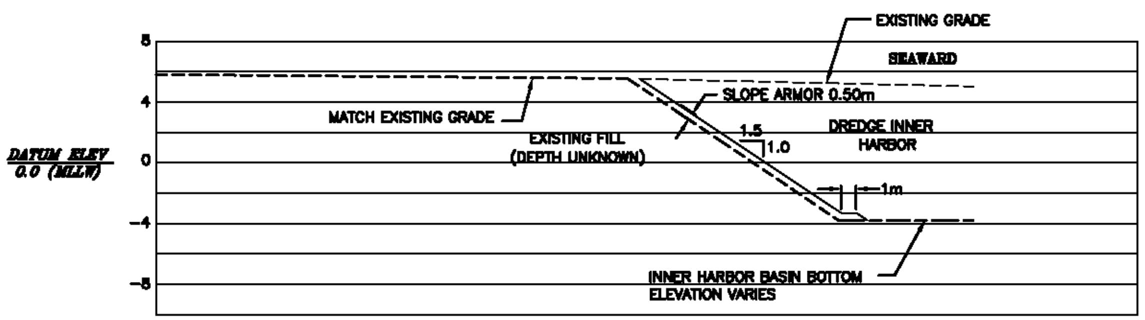
SECTION A

WEST ENTRANCE CHANNEL BREAKWATER SECTION



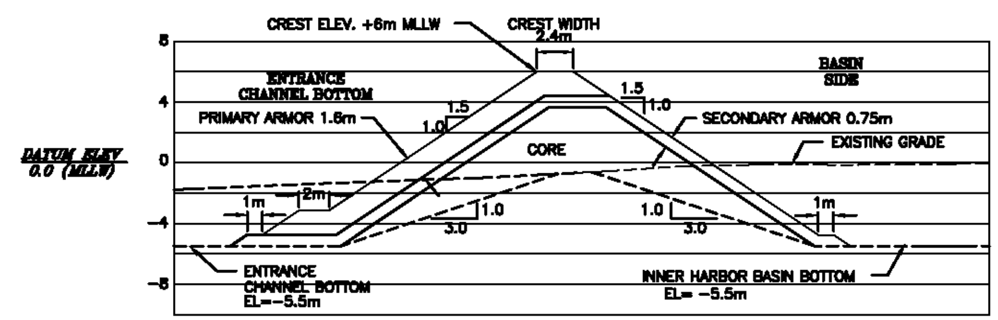
SECTION D

SOUTH BREAKWATER SECTION



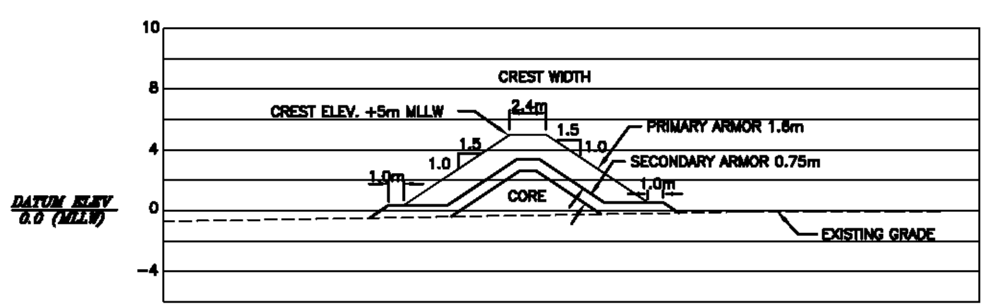
SECTION B

NORTH SIDE INNER HARBOR ARMORED SLOPE



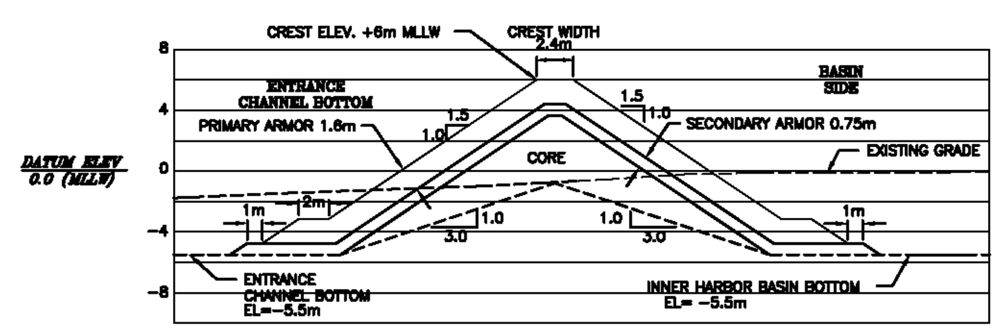
SECTION E

SOUTH ENTRANCE CHANNEL BREAKWATER SECTION



SECTION C

STUB BREAKWATER SECTION



SECTION F

SOUTH ENTRANCE CHANNEL BREAKWATER SECTION

LEGEND:

EXISTING GROUND - - - - - - - -

FINISH GRADE ————

GRAPHIC SCALE

(IN METERS)

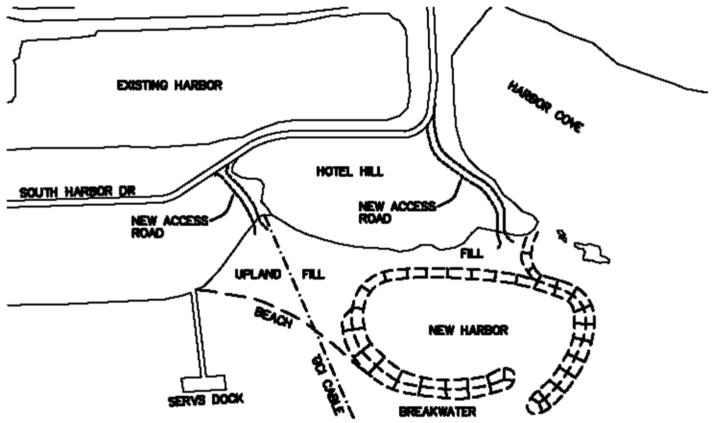
1 : 250

WEST SITE RUBBLE-MOUND BREAKWATER SECTIONS

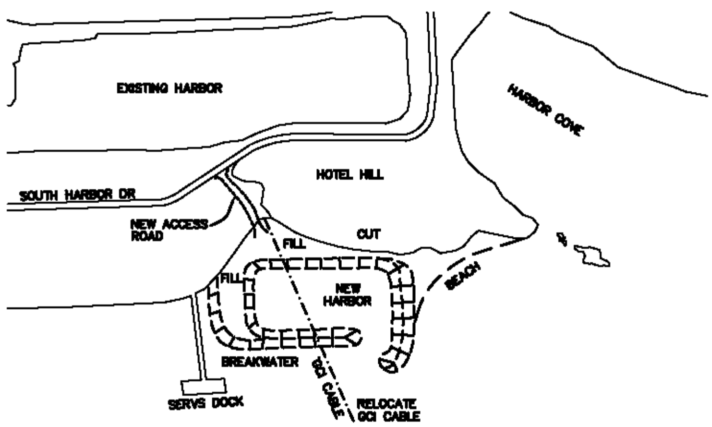
NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

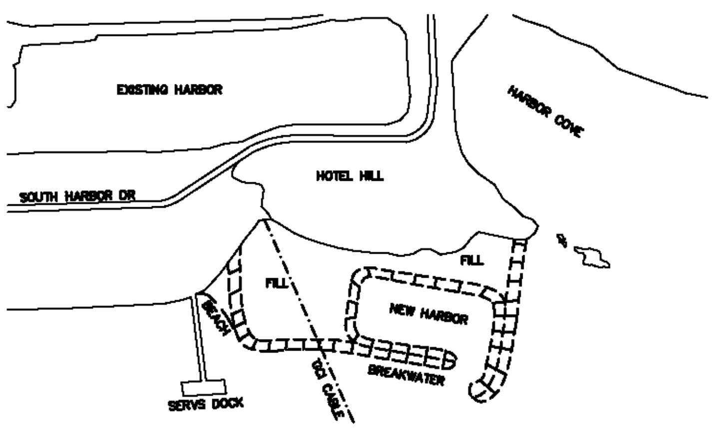
FIGURE A-16



PLAN A



PLAN B



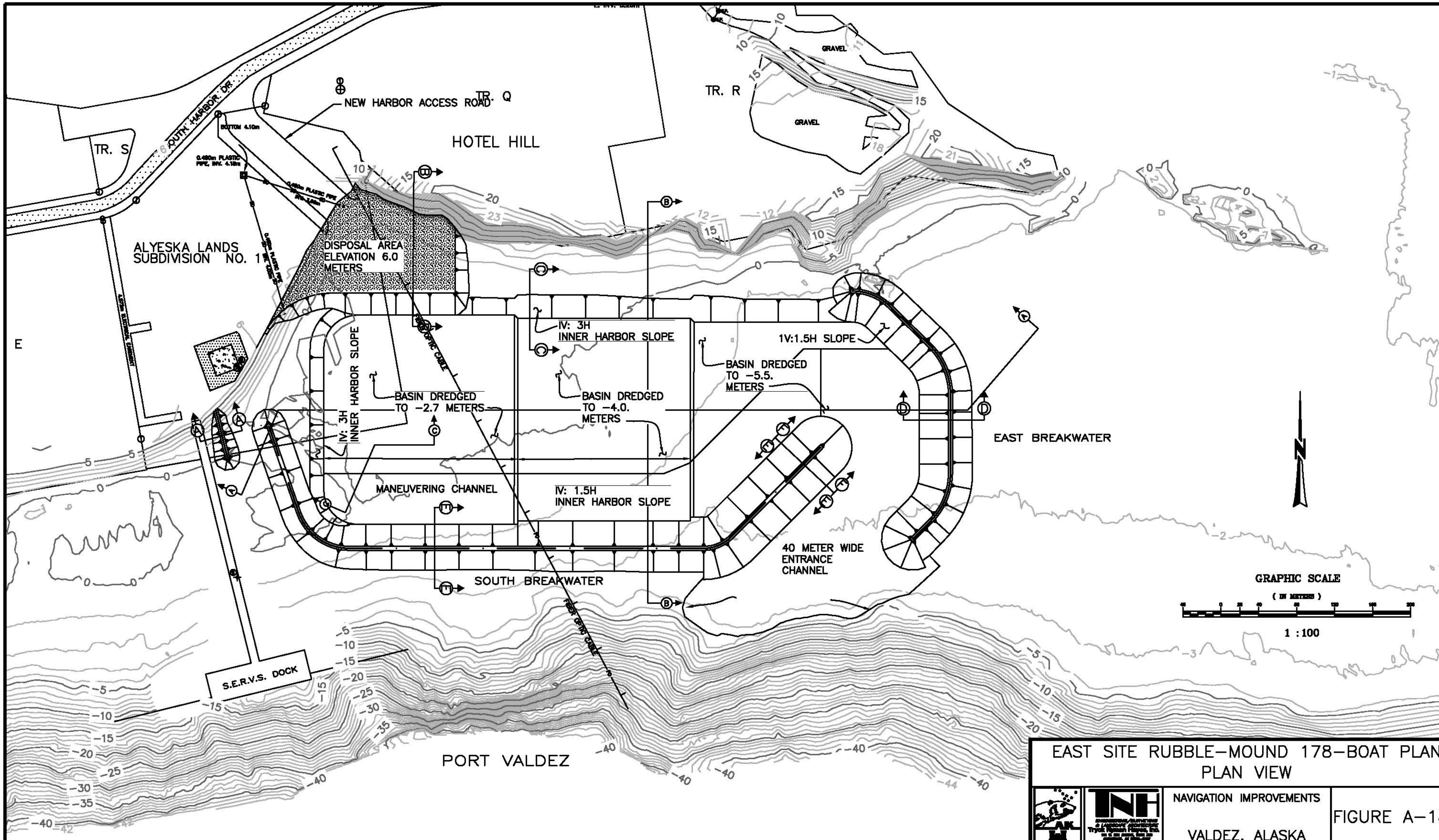
PLAN C

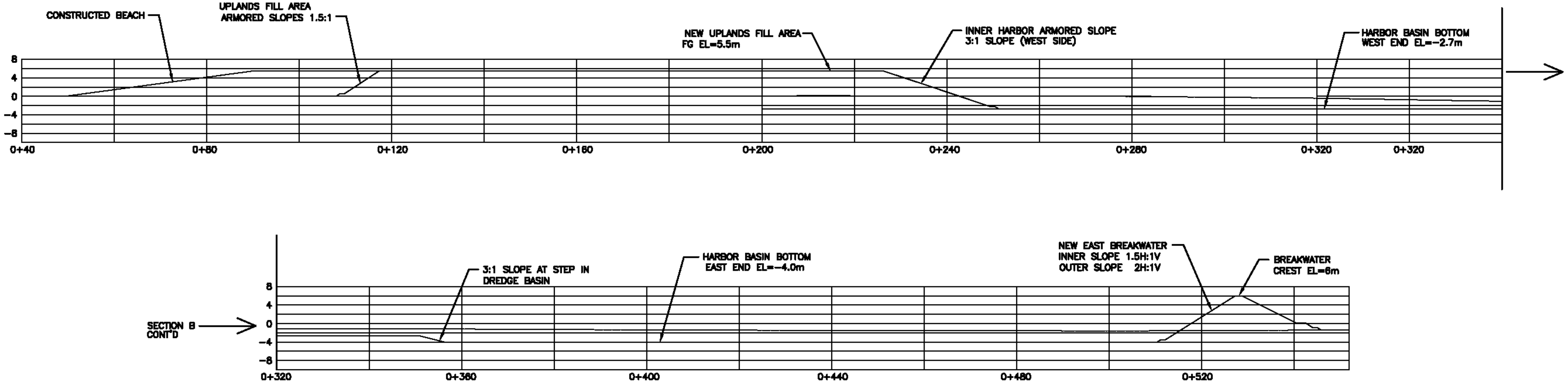
EAST SITE ALTERNATIVE
HARBOR PLANS

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

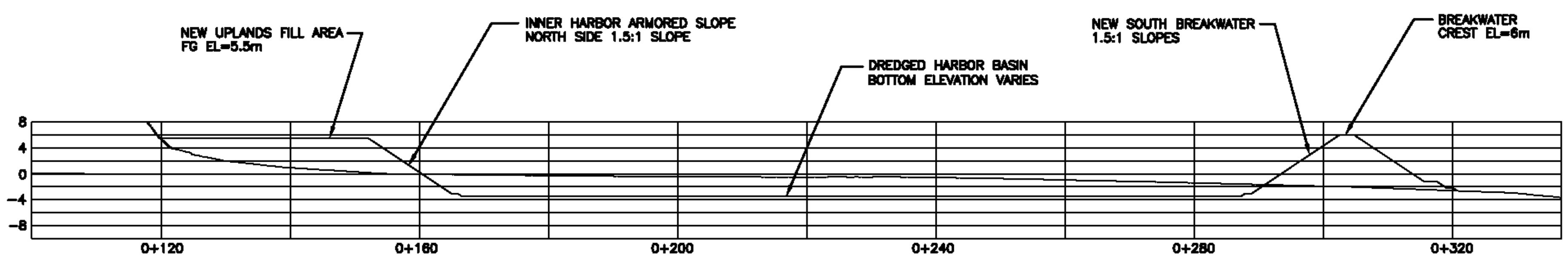
FIGURE A–17


NEW HARBOR ACCESS ROAD
TR. Q
TR. R
TR. S
HOTEL HILL
GRAVEL
ALYESKA LANDS SUBDIVISION NO. 1
DISPOSAL AREA ELEVATION 6.0 METERS
IV: 3H INNER HARBOR SLOPE
1V:1.5H SLOPE
BASIN DREDGED TO -2.7 METERS
BASIN DREDGED TO -4.0. METERS
BASIN DREDGED TO -5.5. METERS
MANEUVERING CHANNEL
IV: 1.5H INNER HARBOR SLOPE
EAST BREAKWATER
SOUTH BREAKWATER
40 METER WIDE ENTRANCE CHANNEL
S.E.R.V.S. DOCK
PORT VALDEZ
GRAPHIC SCALE
1 : 100
EAST SITE RUBBLE-MOUND 178-BOAT PLAN
PLAN VIEW
NAVIGATION IMPROVEMENTS
VALDEZ, ALASKA
FIGURE A-18



SECTION A

EAST/WEST PROJECT SECTION EG & FG



LEGEND

ORIGINAL GRADE

FINISH GRADE

SECTION B

NORTH/SOUTH PROJECT SECTION EG & FG

GRAPHIC SCALE

(IN METERS)

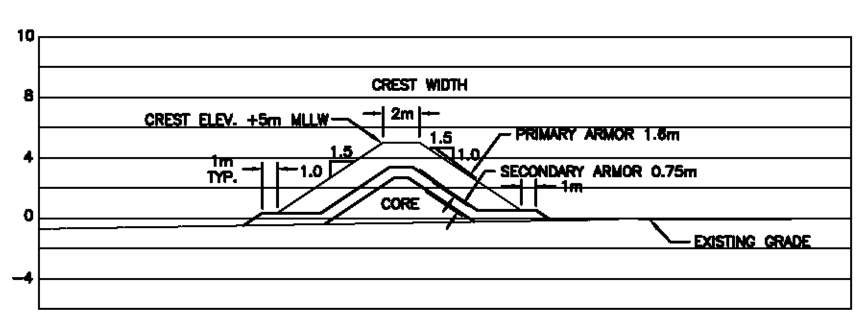
1 : 500

EAST SITE RUBBLE-MOUND 178-BOAT PLAN
HARBOR PROFILES

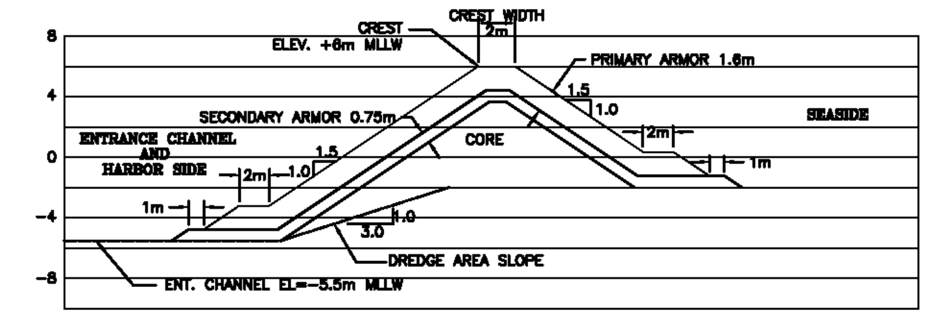
NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

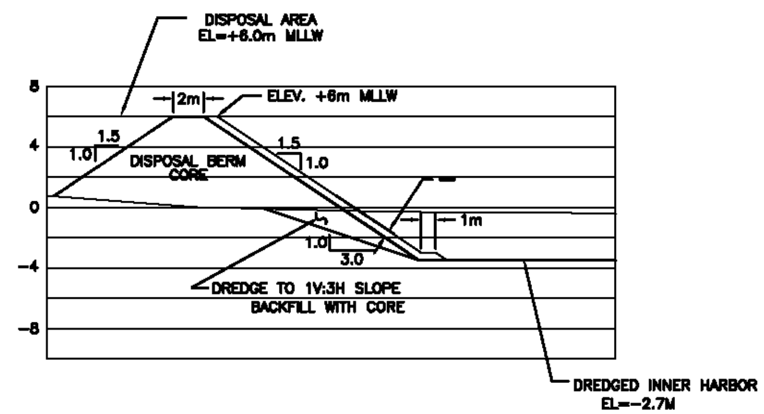
FIGURE A-19



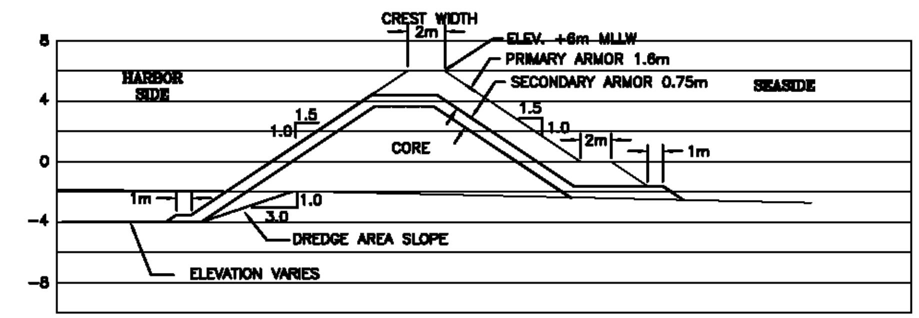
SECTION A
STUB BREAKWATER SECTION



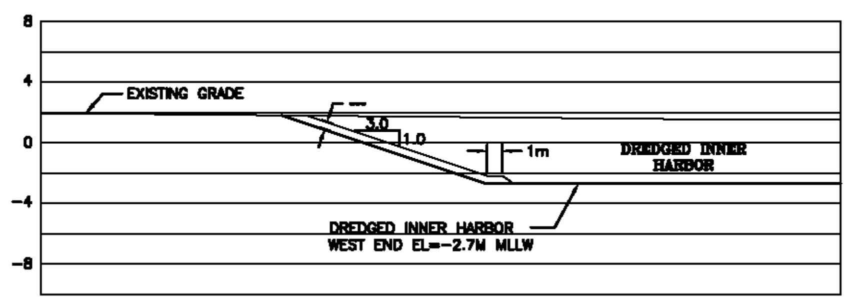
SECTION D
EAST SIDE ENTRANCE CHANNEL BREAKWATER



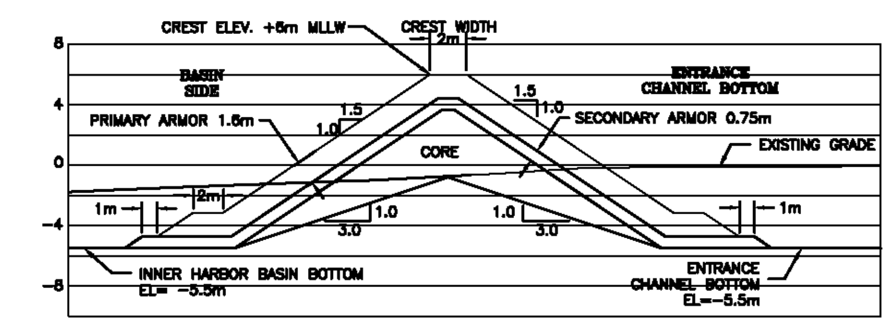
SECTION B
NORTH SIDE INNER SLOPE



SECTION E
SOUTH SIDE BREAKWATER



SECTION C
NORTH & WEST SIDE INNER HARBOR SLOPE



SECTION F
SOUTH ENTRANCE CHANNEL BREAKWATER SECTION

LEGEND

ORIGINAL GRADE

FINISH GRADE

GRAPHIC SCALE

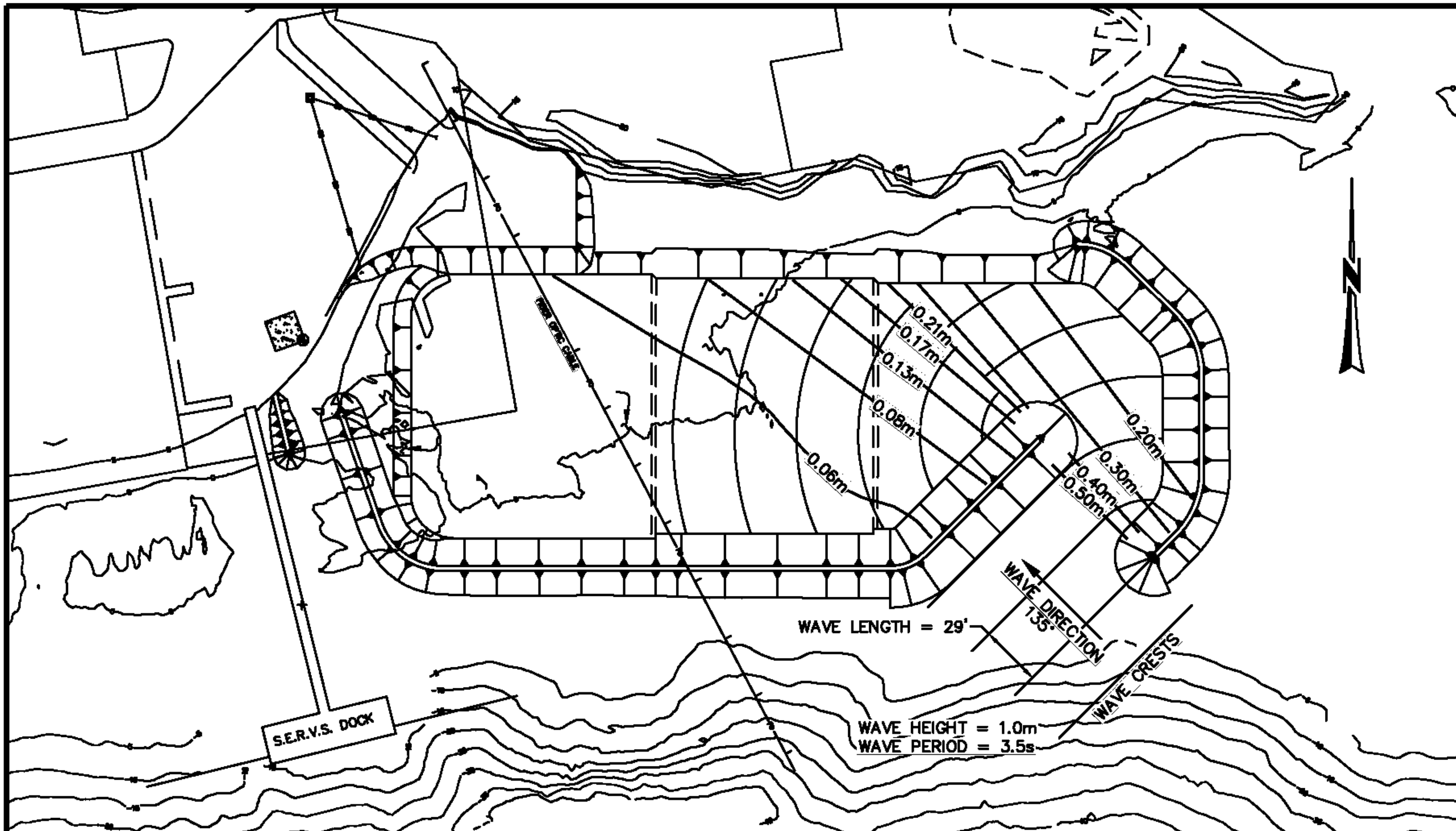
(IN METERS)

1 : 250

NOTES:

1. SECONDARY ARMOR AND SLOPE ARMOR ARE THE SAME SIZE QUARRY ROCK.

EAST SITE RUBBLE-MOUND 178-BOAT PLAN CROSS SECTIONS		
	NAVIGATION IMPROVEMENTS VALDEZ, ALASKA	FIGURE A-20



GRAPHIC SCALE

(IN METERS)

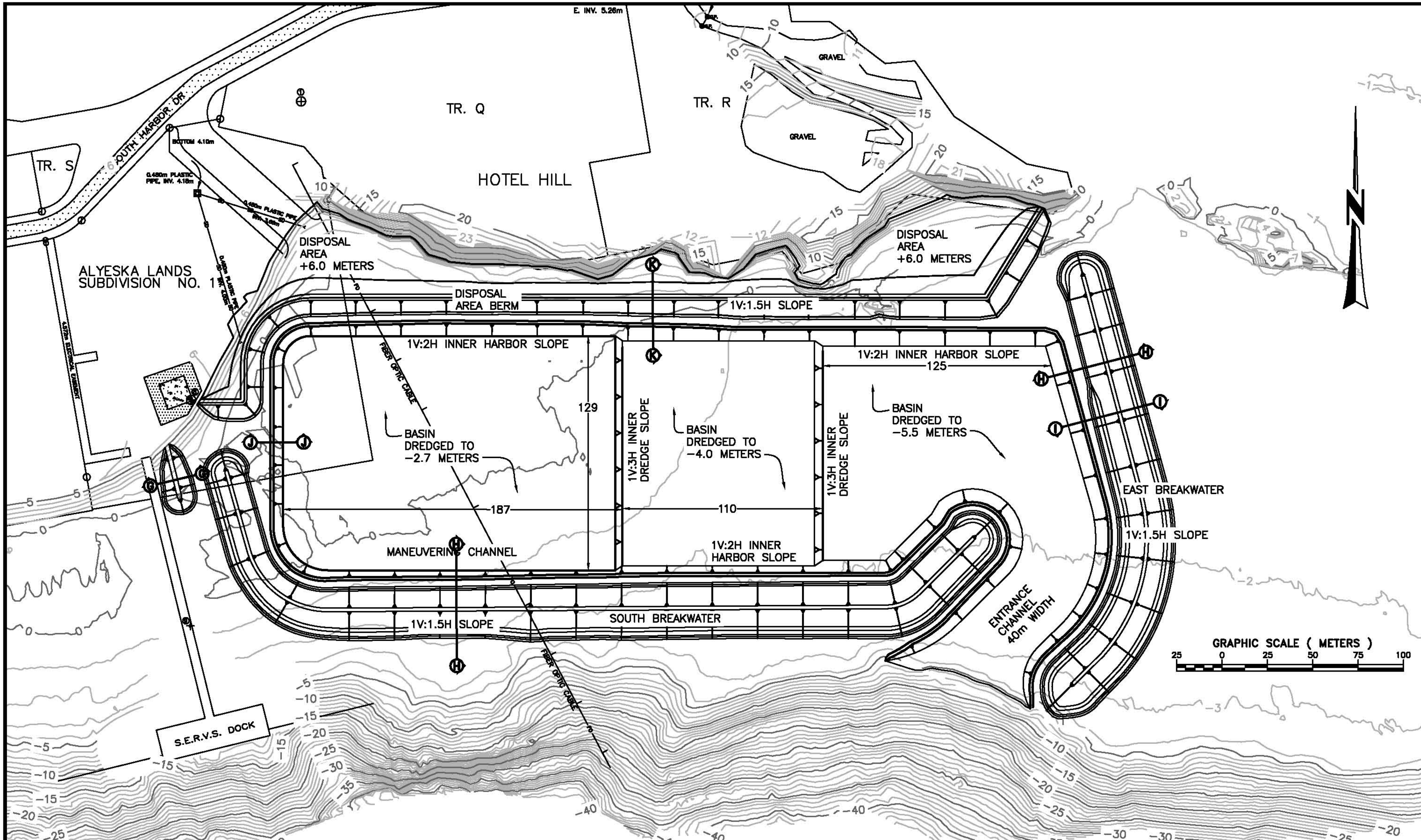
1 : 300

EAST SITE RUBBLE–MOUND 178–BOAT PLAN
WAVE DIFFRACTION DIAGRAM

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

FIGURE A–21



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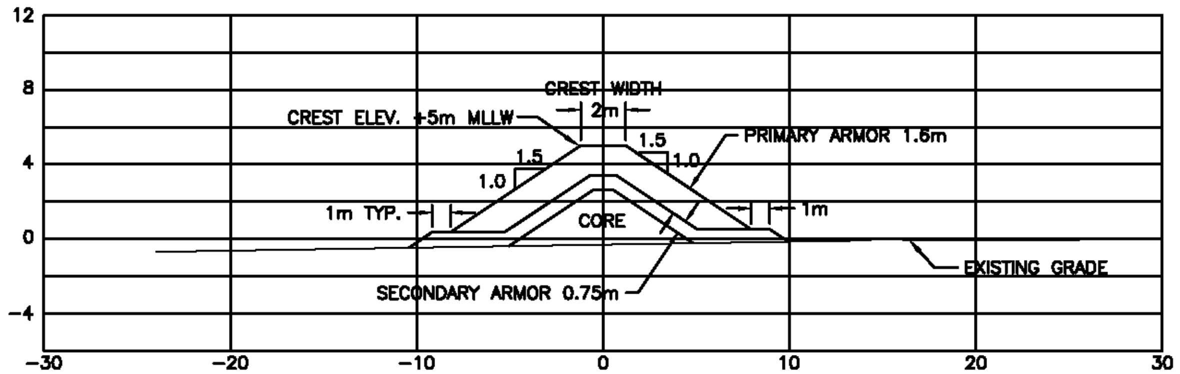
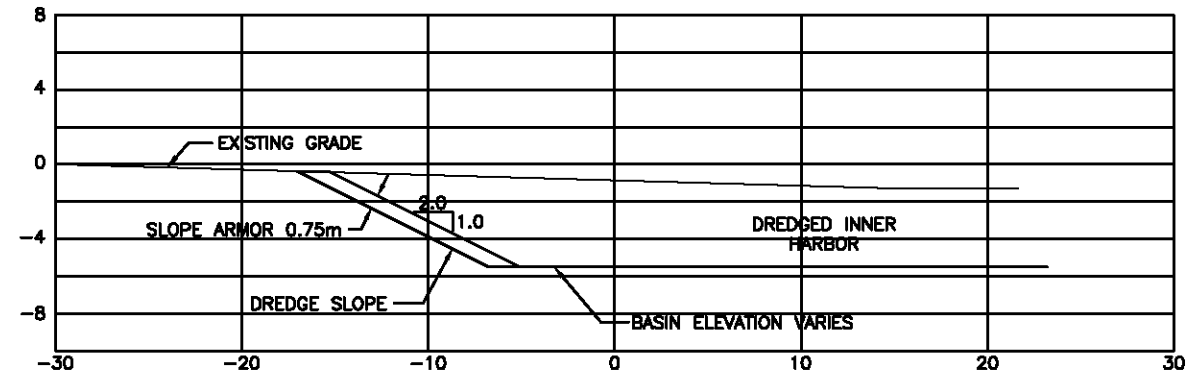
EAST SITE RUBBLE−MOUND 320−BOAT PLAN
PLAN VIEW
NAVIGATION IMPROVEMENTS, VALDEZ, ALASKA

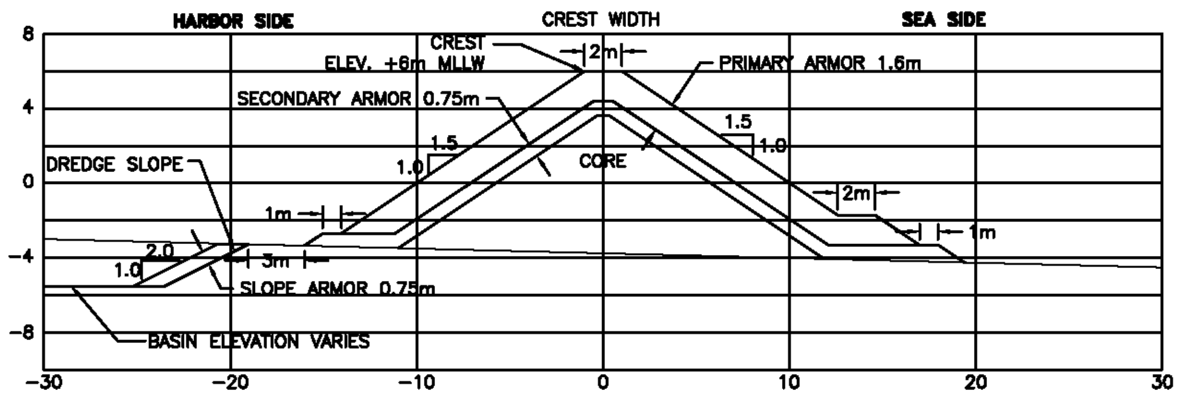
A−22



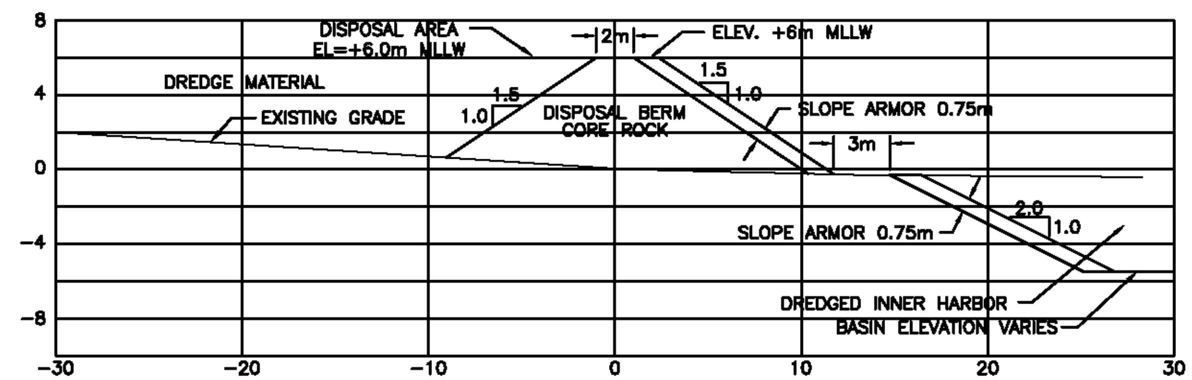
SECTION G
STUB BREAKWATER SECTION



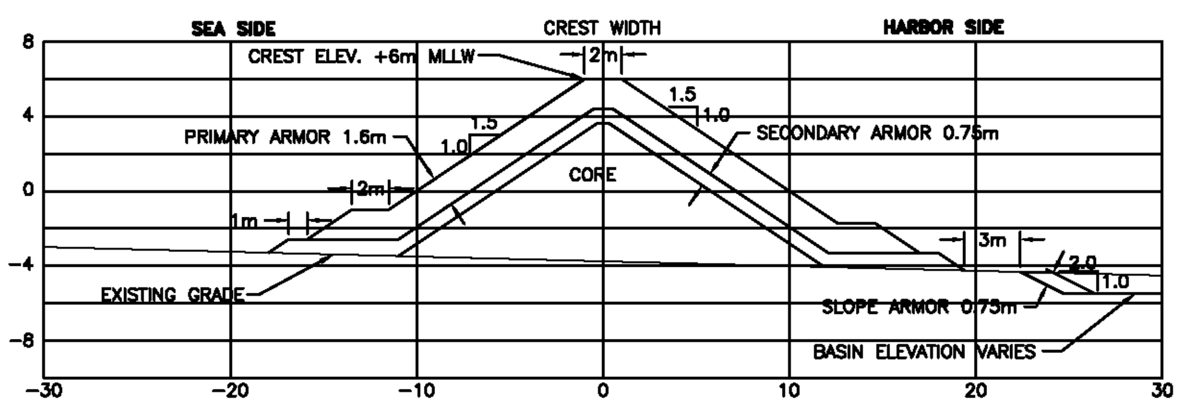
SECTION J
BASIN SLOPE PROTECTION



SECTION H
SOUTH AND EAST BREAKWATER TRUNK SECTION



SECTION K
DISPOSAL AREA SECTION



SECTION I
SOUTH AND EAST BREAKWATER HEAD SECTION

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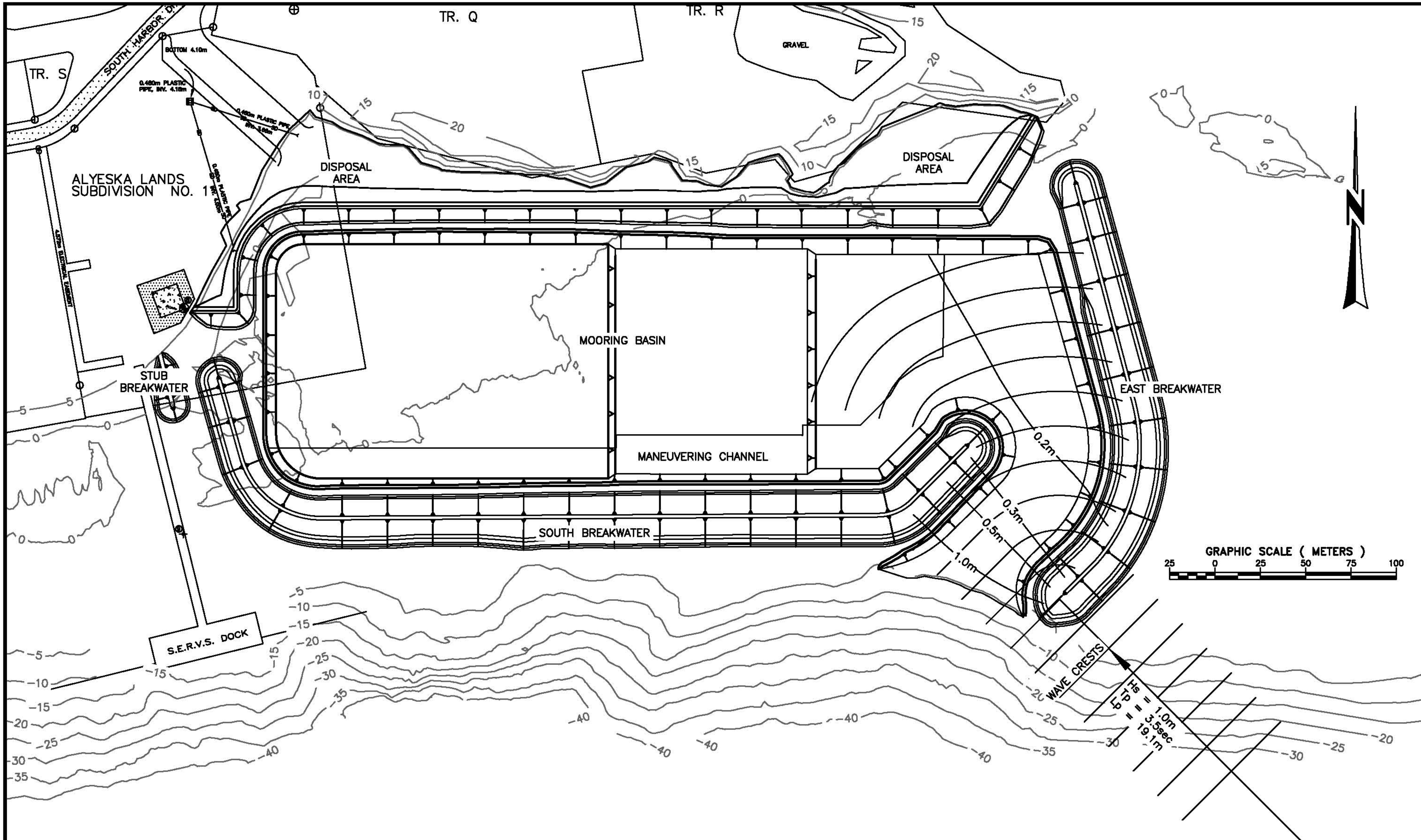
EAST SITE RUBBLE–MOUND 320–BOAT PLAN
CROSS SECTIONS
NAVIGATION IMPROVEMENTS, VALDEZ, ALASKA

A–23



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EAST SITE RUBBLE–MOUND 320–BOAT PLAN
WAVE DIFFRACTION DIAGRAM
VALDEZ HARBOR EXPANSION

A–24

ATTACHMENT A
Quantity Estimates

West Site Rubble-Mound 226-Boat Plan
Detailed Quantity Estimate

Item	Quantity	Unit
Breakwaters		
West Breakwater		
Armor rock placement	2,916	m^3
Secondary rock placement	1,137	m^3
Core rock placement	2,836	m^3
South Breakwater		
Armor rock placement	11,108	m^3
Secondary rock placement	7,289	m^3
Core rock placement	28,152	m^3
Breach Stub Breakwater		
Armor rock placement	532	m^3
Secondary rock placement	154	m^3
Core rock placement	220	m^3
Hydrographic surveys	3	ea
Navigation aid foundation	2	ea
Entrance & Maneuvering Channel Dredging		
Entrance channel, Dry Excavation	26,769	m^3
Wet Excavation	28,039	m^3
Maneuvering channel, Dry Excavation		m^3
Wet Excavation	43,322	m^3
Mooring Basin		
Dredging, Dry Excavation	89,964	m^3
Wet Excavation	69,576	m^3
Slope protection	5,706	m^3
Local Harbor Facilities		
Design/construct floats & Utilities	1	LS
Relocate Outfalls (2)	850	m
Dredge Material Disposal		
Offshore disposal	257,670	m^3

West Site Wave Barrier 313-Boat Plan
Detailed Quantity Estimate

Item	Quantity	Unit
Breakwaters		
Entrance Channel West Breakwater	57	m
Armor rock placement	2,710	m^3
Secondary rock placement	1,630	m^3
Core rock placement	2,950	m^3
Entrance Channel East Breakwater	77	m
Armor rock placement	6,720	m^3
Secondary rock placement	3,700	m^3
Core rock placement	11,780	m^3
East Main Breakwater	50	m
Armor rock placement	2,720	m^3
Secondary rock placement	1,620	m^3
Core rock placement	2,520	m^3
East Stub Breakwater	25	m
Armor rock placement	670	m^3
Secondary rock placement	520	m^3
Core rock placement	130	m^3
Hydrographic surveys	3	ea
Navigation aid foundation	2	ea
Wave Barrier	486	m
Vertical Piling	82	ea
Total length	3400	m
Batter Piling	82	ea
Total length	3900	m
Reinforced Concrete Wall Panels	380	ea
Entrance & Maneuvering Channel Dredging		
Dredging	119,500	m^3
Slope Protection	1,530	m^3
Mooring Basin		
Dredging	86,200	m^3
Slope Protection	1,170	m^3
Local Harbor Facilities		
Design/construct floats & Utilities	1	LS
Relocate Outfalls (2)	850	m
Dredge Material Disposal		
Offshore disposal	205,700	m^3

West Site Wave Barrier 228-Boat Plan
Detailed Quantity Estimate

Item	Quantity	Unit
Breakwaters		
West Breakwater	55	m
Armor rock placement	2,340	m^3
Secondary rock placement	1,430	m^3
Core rock placement	2,110	m^3
Entrance Channel West Breakwater	27	m
Armor rock placement	3,080	m^3
Secondary rock placement	1,750	m^3
Core rock placement	4,830	m^3
Entrance Channel East Breakwater	108	m
Armor rock placement	5,430	m^3
Secondary rock placement	3,140	m^3
Core rock placement	6,000	m^3
Hydrographic surveys	3	ea
Navigation aid foundation	2	ea
Wave Barrier		
Vertical Piling	77	ea
Total length	2150	m
Batter Piling	77	ea
Total length	2400	m
Reinforced Concrete Wall Panels	352	ea
Entrance & Maneuvering Channel Dredging		
Dredging	71,100	m^3
Slope Protection	5,570	m^3
Mooring Basin		
Dredging	85,900	m^3
Slope Protection	1,110	m^3
Local Harbor Facilities		
Design/construct floats & Utilities	1	LS
Relocate Outfalls (2)	850	m
Dredge Material Disposal		
Offshore disposal	157,000	m^3

East Site Rubble-Mound 178-Boat Plan
Detailed Quantity Estimate

Item	Quantity	Unit
Breakwaters		
East Breakwater		
Armor rock placement	11,012	m^3
Secondary rock placement	6,501	m^3
Core rock placement	11,016	m^3
South Breakwater		
Armor rock placement	12,876	m^3
Secondary rock placement	12,121	m^3
Core rock placement	24,650	m^3
Breach Stub Breakwater		
Armor rock placement	554	m^3
Secondary rock placement	418	m^3
Core rock placement	151	m^3
Hydrographic surveys	3	ea
Navigation aid foundation	2	ea
Entrance & Maneuvering Channel Dredging		
Entrance channel excavation	45,156	m^3
Maneuvering channel excavation	34,406	m^3
Slope Protection	100	m^3
Mooring Basin		
Dredging	96,086	m^3
Slope protection	1,875	m^3
Local Harbor Facilities		
Design/construct floats & Utilities	1	LS
Access road	120	m
Relocate/bury fiber optic cable	1	LS
Dredge Material Disposal		
Upland disposal berm	160	m
Slope protection	881	m^3
Core rock placement	8,057	m^3
Upland disposal	16,504	m^3
Offshore disposal	159,144	m^3

East Site Rubble-Mound 320-Boat Plan
Detailed Quantity Estimate

Item	Quantity	Unit
Breakwaters		
East Breakwater		
Armor rock placement	12,180	m^3
Secondary rock placement	6,980	m^3
Core rock placement	11,590	m^3
South Breakwater		
Armor rock placement	18,370	m^3
Secondary rock placement	10,160	m^3
Core rock placement	25,750	m^3
Breach Stub Breakwater		
Armor rock placement	650	m^3
Secondary rock placement	520	m^3
Core rock placement	230	m^3
Hydrographic surveys	3	ea
Navigation aid foundation	2	ea
Entrance & Maneuvering Channel Dredging	2.0	hec
Dredging	65,630	m^3
Slope protection	3,870	m^3
Mooring Basin	3.5	hec
Dredging	120,780	m^3
Slope protection	2,650	m^3
Local Harbor Facilities		
Design/construct floats & Utilities	1	LS
Access road	120	m
Relocate/bury fiber optic cable	1	LS
Dredge Material Disposal		
Upland disposal berm	532	m
Slope protection	4,420	m^3
Core rock placement	30,670	m^3
Upland disposal	41,610	m^3
Offshore disposal	144,800	m^3

ATTACHMENT B
35% Structural Design Analysis
Partial Depth Vertical Wave Barrier for Valdez Alaska

35%
Structural
Design Analysis

Partial Depth Vertical
Wave Barrier
for
Valdez Harbor

by
US Army Corps of Engineers,
Alaska District

January 2005

Partial Depth Vertical Wave Barrier for Valdez Harbor

35% Structural Design Analysis

Scope

Valdez, Alaska intends to expand their small boat harbor. Unfortunately, the sea floor drops off quickly (approximately 2.32H:1V) outside of the existing harbor, limiting the size of conventional rubble mound solutions. The idea of using a partial depth vertical wave barrier (see Figure 1 for schematic representation) was proposed, and we were tasked to determine the deep-water limits of such a wave barrier. The answer was assumed to be a structural question, and indeed the deeper we go the larger the structure will be, however the answer to the question really lies in the economics of the harbor. Deeper water will require more expensive materials and construction techniques. We have analyzed a general deep-water case, and provided a specific solution at a given depth. This costs of which can now be estimated, and the harbor layout considered.

Wave Climate

The primary purpose of the wave barrier is to keep the rough seas out of the harbor, and this will drive the geometry of the barrier. To ensure that the wave barrier is always visible to boat traffic, and to avoid the potentially difficult analysis associated with over-topping, we will set the top of the wall to preclude over-topping during our design event. The bottom of the wall will be set to limit the transmission of wave energy to an acceptable level. We will start by considering the wave climate inside and outside the wave barrier, as well as the tidal ranges.

Waves inside the harbor

Some waves will enter the harbor via the entrance channel, through the wave barrier, and from boat wake. Design criteria[1] require that the protective structure limit the waves inside the harbor to 1 foot (.3048m). The waves that are transmitted through the wave barrier will be in-phase (or slightly delayed) with from the attacking waves outside the harbor[2], and the loads due to those can be considered along with the attacking waves.

The size of the waves through the entrance channel will be limited to the 1-foot criteria, and will be largest during a major event. However without a completed entrance design, it isn't possible to consider how they will interact with the wave barrier. It is likely that they will be 180 out of phase with the attacking waves, and therefore increase the wave loads. This will be mitigated by the fact that the wave barrier will enter shallower water to meet the rubble mound entrance channel. If the design of the wave barrier doesn't change from deep water to shallower water, we will most likely be over-designed, despite not considering the entrance channel wave effect. This analysis should be evaluated as the design is refined.

The size and period of the boat wake has not been analyzed, but it is reasonable to assume that during a major event, the boats will be moored, and therefore there will be no wake. Any wake generated at other times will be much less significant than the design event.

Waves outside the harbor

In the Hydraulic Design for Valdez[3] a wave study was done to predict 50-year extreme wave heights. Results for the western alternative are summarized below.

Direction	H_s (meters)	T (sec)	L(m)
Southeast (135)	1.1	3.6	20.23
West (270)	1.2	4.1	26.24

Table 1
Summary of 50-Year Wave Events

These directions represent the overlay of a 5-degree fetch bin, with a 45-degree wind bin. Therefore I have assumed that the waves described can occur within 22.5 degrees of the stated direction. Waves outside that range will be considered oblique to the structure, and reduced appropriately. Non-perpendicular wave forces decay sinusoidally. When the wave train is perpendicular to the wave barrier the force is proportional to H_s. When the wave train is parallel to the wave barrier the force is proportional to $H_s/2$. This is not a decrease in actual wave height, but rather a force decrease due to point pressure reduction[4] from the oblique attack. Another reduction for oblique waves is called peak-delay force reduction, and this accounts for the fact that an oblique wave does not strike the entire barrier at the same time, rather the wave front moves along the wall. This effect is ignored, since the inter-pile spacing is much less than a wavelength, we will consider the full effect of the wave on the entire tributary area of the wall for a given pile. The general layout, and the reductions for oblique waves are summarized in the sketch in Figure 2.

According to Kriebel[2], there may be a sinusoidal reduction in transmission for oblique waves as well, but it is not documented. For this reason, I will assume no reduction in transmission for oblique waves.

The wave height above is H_s, the significant wave height, also known as H_{33}, or the average of the highest 33% of the waves. Therefore, 33% of the waves are higher. The H_{10}, H_1, and $H_{1/250}$ represent average wave heights for the highest 10%, 1% and .25% respectively. Assuming a Raleigh distribution this can be translated to an increase in wave height of 127%, 167% and 180%. I understand that this is based on a random open ocean, and that in a fetch limited port like Valdez, the maximum wave heights will not be that far from H_s, however to conservative, we have chosen to use the H_1 wave. This maybe revisited later.

Tidal Ranges

The tidal ranges for Valdez were obtained from the NOAA web site[5] and are summarized below.

Highest observed water level (11/11/81)	5.198m
Mean Higher High Water (MHHW)	3.702m
Mean High Water (MHW)	3.416m
Mean Sea Level (MSL)	1.979m
Mean Tide Level (MTL)	1.938m
Mean Low Water (MLW)	0.460m
Mean Lowest Low Water (MLLW)	0.000m
Lowest Observed Water Level (01/01/91)	-1.513m

Table 2
Summary of Tidal Ranges

It is unlikely that we will have an extreme low water (ELW) tidal event at the same time as our design storm (50yr event), however we should assume that the design storm has a duration through an entire tide cycle and therefore there is a 50% chance of the SWL will be above MHHW and below MLLW during the event. I have assumed the EHW and ELW events occurred as a result of large waves and extreme SWL. I have set the extreme SWL by subtracting half the design wave height from the ELW and EHW events. The design wave as discussed above will be a H_1 or 1% wave, which is 1.67 times the significant wave height, H_s. The largest significant wave height is 1.2m, so half of H_1 is 1.0m. Therefore the highest SWL we will consider is 4.198m above MLLW, and the lowest SWL we will consider is 0.513m below MLLW.

Barrier Geometry

Transmitted wave

The primary variables that determine how much energy (or the size of the transmitted wave) gets past the wave barrier are the depth of the barrier below the still water level (SWL), the depth of the ocean at the wave barrier, and the height and length of the attacking waves. Weigel, in 1960, originally presented a formula based on theory with some experimental validation to determine the depth of the barrier below the SWL necessary given a maximum transmission coefficient. The results of this formula are presented in chart form in the Navy Design Manual 26.2, Coastal Protection in Figure 92, page 26.2-140. Kriebel, in 1995, revisited this theory, and applied a modification to account for the reflected energy of the incident wave. This theory provided a better fit to experimental data, including Weigel's. This paper is in Appendix B, and provides formulas for both theories. The experimental data does not go to relative depths as deep as our project, although correspondence with Kriebel[2] suggests that there is no reason to expect any unusual behavior in very deep water.

The following table computes the depth from MLLW for a wave barrier using both theories. The depth is computed for various depths of water, and the H_1 wave from both the West and Southeast directions. Since we are concerned with the lower limit of the wall for all tide levels, we did some preliminary calculations to determine that the lowext tide level would always control, so we used a SWL of –0.513 from MLLW.

	Kriebel		Weigel	
Mudline	West	Southeast	West	Southeast
-5m	-4.37m	-4.12m	<-5m	<-5m
-7.5m	-5.90m	-4.87m	-7.03m	-6.52m
-10m	-6.38m	-4.65m	-8.72m	-6.90m
-12.5m	-6.14m	-4.47m	-9.24m	-6.54m
-15m	-5.91m	-4.41m	-8.90m	-6.37m
-20m	-5.75m	-4.38m	-8.46m	-6.30m
-25m	-5.73m	-4.38m	-8.39m	-6.30m
-30m	-5.73m	-4.38m	-8.38m	-6.30m

Table 3
Wave Barrier penetration from MLLW (SWL=-0.513m)

The calculations for this table were done in a MathCAD worksheet that is in Appendix C.

Both theories reach an asymptotic limit at our project depth, however the depth of the barrier will need to increase as we go into shallower water.

Note that the wave period/length seems to be the controlling parameter. Even if we reduce the wave height H_s to 1.03 for the west wave due to an oblique attack angle, the penetration is only reduced by about 0.35 meters, so we will not concern ourselves with oblique angles for determining barrier depth.

Currently the barrier is set at –5.75m everywhere since this is the maximum for the deep water condition. However this only applies to a small portion of the West leg of the barrier. The main leg of the waver barrier is only attacked by the SE wave, and should be reduced to –4.5m, furthermore, the West leg will need to go deeper than –5.75m in shallower water. In summary, we have over predicted our required penetration on the South and East legs, and under predicted it on the West leg. This will be refined in subsequent design.

The deeper penetration will increase the forces, however the shallower water results in a stouter structure, so we should still be conservative.

Run up discussion,

To set the top of the barrier, we assume complete reflection, and no clapotis rise, therefore the maximum water level at the wall will be the extreme SWL + H_D (4.198m + 2m = 6.198). Since our analysis of the interior wave climate assumes that there is no overtopping, we must set the top of the wall above this elevation. We will round off to +6.5m (21.33ft) above MLLW.

Wave forces

Breaking waves

Kreibel[2] notes that there is the possibility of deep water breaking waves. He suggests that if 2*pi*Hs/Lo (Hs = significant wave height, Lo = deep water wave length) exceeds 0.3 then some breaking may occur. For our western waves:

$$2*pi*Hs/Lo = 2*3.14*1.2m/26.24m = 0.29 < 0.3$$

however for our South-eastern waves,

$$2*pi*Hs/Lo = 2*3.14*1.1m/20.23m = 0.34 > 0.3$$

Therefore there is some concern for breaking waves. However the Shore Protection Manual provides a limiting wave steepness of $H_o/L_o <= 1/7$ which we are within. Therefore we have chosen not to consider breaking waves on the deep-water structure at this stage.

It should also be noted that PND[6] reported that "Reflected waves tended to cause some wave breaking in front of the wave barrier" at the Seattle Central Water Front project. This occurred during a storm with 3 to 4 foot waves at about a 4 second period. The reflected wave is going to be twice the height as the incident wave, and the formulas above would predict they might break, however they are headed away from the wall, and of no concern.

In subsequent design stages, we will need to consider the potential for breaking waves on the shallower parts of the structure.

Oblique waves

From the wave attack angles and the preliminary site plan, we can see the waves attack nearly head on. We have applied the obliquing factors to obtain a slight reduction in the wave forces.

Dynamic loads

Sainflou

Sainflou, circa 1920, provided the first solution for non-breaking regular (airy) wave forces on a vertical wall. The Navy Design Manual provides a solution for a partial penetration vertical wall, which reduces this force by a linear factor, based on percentage of penetration to depth of water. A MathCAD worksheet was developed to solve the Sainflou method. A typical solution on our deep wall is provided in Appendix D, for comparison with the Kriebel method below.

Miche-Rundgren

This method utilizes charts, not formulas, and is presented in the SPM and the Navy Design Manual 26.2. For our deep-water condition, the charts do not work, as we are approaching an asymptotic limit. We did conclude however that for our case the Miche-Rundgren curves are the same as the Sainflou curves, so we aborted any further attempts at using this method.

Goda

Goda provided an improved method for obtaining wave forces on vertical walls. His theory is based on random waves. A MathCAD worksheet was developed to solve the Goda method. A typical solution is provided in Appendix E, for comparison with the Kriebel method below.

Kriebel, et al

The Kriebel method is discussed in the CEM, and the rigorous solution involves complicated eigen-value solutions. The CEM has developed a simplified solution involving a best fit curve that is valid for 0.4 < w/d < 0.7 and 0.14 < d/L < 0.5, where w is the wave barrier penetration from SWL, and d is the depth from SWL, and L is the local wavelength. A MathCAD worksheet was developed for solving this formula and populating the tables below. The MathCAD worksheet for a typical solution is presented in Appendix F.

The following is a summary of the H_1 wave from Southeast with barrier penetration of 5.75m below MLLW.

	SWL=-0.513m		SWL=1.979m		SWL=4.198m	
Depth	F (kN/m)	p (kPa)	F (kN/m)	p (kPa)	F (kN/m)	p (kPa)
-10m	39.75	11.25	46.31	9.69	49.83	8.43
-15m	36.05	10.20	42.02	8.79	45.53	7.73
-20m	35.19	9.96	40.48	8.47	43.77	7.43
-25m	35.19	9.96	39.93	8.35	42.95	7.29
-30m	35.49	10.04	39.79	8.33	42.59	7.23
-35m	35.90	10.16	39.86	8.34	42.45	7.21
-40m	36.35	10.29	40.03	8.38	42.46	7.21
-45m	36.81	10.42	40.26	8.42	42.54	7.22

Table 4
Wave Forces for Southeast Wave

The following is a summary of the H_1 wave from West with barrier penetration of 5.75m below MLLW.

	SWL=-0.513m		SWL=1.979m		SWL=4.198m	
Depth	F (kN/m)	p (kPa)	F (kN/m)	p (kPa)	F (kN/m)	p (kPa)
-10m	43.54	12.54	52.98	11.22	57.90	9.93
-15m	39.47	11.36	47.51	10.07	52.34	8.98
-20m	38.43	11.06	45.47	9.63	49.93	8.57
-25m	38.44	11.06	44.73	9.48	48.82	8.38
-30m	38.83	11.18	44.55	9.44	48.32	8.29
-35m	39.37	11.33	44.64	9.46	48.14	8.26
-40m	39.97	11.51	44.87	9.51	48.15	8.26
-45m	40.58	11.68	45.17	9.57	48.25	8.28

Table 5
Wave Forces for West Wave

The following table summarizes the maximum depth to mudline (from MLLW in meters) for which the CEM approximation to Kriebel method would be applicable, assuming a wave barrier penetration of 5.75m below MLLW.

SWL	W Wave		SE Wave	
	Min	Max	Min	Max
-0.513m	8.0	13.5	8	10.5
0.000m	8.3	13.1	8.2	10.1
1.000m	8.7	12.1	8.6	9.1
1.979m	9.1	11.1	9.1	8.1
3.000m	9.5	10.1	~9.5	~7.1
4.000m	9.9	9.1	~10.0	~6.1
4.198m	10.0	9.0	~10.1	~6.0

Table 6

Valid depths for CEM approximation

The following table varies the SWL through the tidal range for an H_1 wave with barrier penetration at –5.75m and mud line at –30m (MLLW)

SWL	West Wave		Southeast Wave	
	F (kN/m)	p (kPa)	F (kN/m)	p (kPa)
-0.513m	38.83	11.18	35.49	10.04
0.000m	40.18	10.77	36.51	9.63
1.000m	42.53	10.05	38.29	8.92
1.979m	44.55	9.44	39.79	8.33
3.000m	46.41	8.87	41.17	7.78
4.000m	48.02	8.38	42.37	7.32
4.198m	48.32	8.29	42.59	7.23

Table 7

Wave force for various tidal levels with mud-line at –30m (MLLW)

The following table varies the SWL through the tidal range for an H_1 wave with barrier penetration at –5.75m and mud line at –10m (MLLW)

SWL	West Wave		Southeast Wave	
	F (kN/m)	p (kPa)	F (kN/m)	p (kPa)
-0.513m	43.54	12.54	39.75	11.25
0.000m	45.96	12.32	41.47	10.94
1.000m	49.91	11.80	44.21	10.31
1.979m	52.98	11.22	46.31	9.69
3.000m	55.53	10.62	48.05	9.08
4.000m	57.55	10.04	49.41	8.53
4.198m	57.90	9.93	49.65	8.43

Table 8

Wave force for various tidal levels with mud-line at –10m (MLLW)

In the following table we combine an H_1 wave with barrier penetration at –5.75m and mud line at –10m (MLLW) with a maximum wind force (calculated later in the DA), to see if there is a worst case tidal level

SWL	Exp'd H	Wind F (kN/m)	W Wave +Wind		SE Wave + Wind	
			F (kN/m)	F (kN/m)	F (kN/m)	F (kN/m)
-0.513m	7.013m	10.85	43.54	54.39	39.75	50.60
0.000m	6.500m	10.06	45.96	56.02	41.47	51.53
1.000m	5.500m	8.51	49.91	58.42	44.21	52.72
1.979m	4.521m	6.99	52.98	59.97	46.31	53.30
3.000m	3.500m	5.41	55.53	60.94	48.05	53.46
4.000m	2.500m	3.87	57.55	61.42	49.41	53.28
4.198m	2.302m	3.56	57.90	61.46	49.65	53.21

Table 9
Wave and wind forces for various tidal levels.

The wind force is 1.547kPa times the exposed height of wall. This gives a force per plan meter of wall, which is added to the total wave force. The total shows that the force is always largest when the SWL is the highest (that is not quite true for the lower wave forces from the SE, but there is only a difference of 0.5%). This total force does not take into consideration the effects from the distribution of the force. At the lower SWL, the higher pressures are further from the reaction point, which introduces a longer lever arm. At this point it is unclear if the lower force with the longer lever arm introduces more moment than the larger force closer to the reaction point, therefore both the high, low and an average SWLs will be analyzed in RISA.

Gilman-Kriebel Design Procedure

Gilman and Kriebel prepared a paper titled "Partial depth pile supported wave barriers: A design procedure" for Coastal Structures '99. In this paper, a design procedure is outrline. A typical solution is provided in Appendix G, for comparison with the Kriebel method above.

Comparison of methods.

The Gilman-Kriebel method appears to be a simplified/graphical solution to the similar to the formulaic solution in the CEM, and both methods provide more realistic force predictions than the other methods. The CEM method was used since it was easier to implement in MathCAD. Not that since the CEM method was not valid for the deeper water conditions, the higher forces at the shallower depths (were the method was valid) were used in the RISA analysis.

Hydrostatic Loads

The prevailing train of though seems to be that since the SWL on both sides of the wall is the same that there is no net hydrostatic force on the wave barrier. That is the difference in water height comparing an incident wave crest to a transmitted wave crest is not considered a hydrostatic load, but part of the dynamic load of the wave.

Forces on Piles

For the portion of the vertical piles connected to the wall, I have considered the piles to be part of the wall. For the portion of the vertical pile below the wall, I have ignored any additional wave forces. This maybe acceptable, since the purpose of the wall is to limit the wave energy passing under the wall, and therefore there should be very little wave force on the lower portions of the pile. I have also ignored all wave forces on the batter pile. This should be remedied in subsequent portions of the design.

Seismic loads

The seismic analysis only concerns the super-structure. According to the Geotechnical evaluation, the soils in the area are potentially liquefiable. There is no economical way to mitigate this risk. The installation of the wave barrier will neither improve nor worsen the stability of the subsurface.

The seismic analysis of the superstructure is a typical lateral force analysis, and is based on maintaining serviceability of the structure during a 2% (50yr event). This means that the super-structure will be designed to be fully functional after a major seismic event. The analysis is based on a building/structure in the air. No attempt is made to model the force increase/reduction due to changes in inertia or dampening due to the water.

The seismic loads were calculated in the RISA Analysis by providing a .5g factor to the mass of the structure. As anticipated, this was not a controlling load case.

We will assume that a seismic event will not happen during a major storm, therefore the major wind and wave forces will not be present.

Wind Loads

The wind pressure is calculated in appendix H, and is conservatively 1.55kPa (32.3psf). Strong winds mean large waves, so we will need to consider the maximum wind loads with the maximum wave loads. However, the total wind force on the wall is greatest when the SWL level is as low as possible, and the wave forces are largest when the SWL is as high as possible. Both conditions will need to be analyzed.

Tidal/Current Drag

Just as the moving air (wind) puts a force on the wall, so does the moving water. The drag on an immersed body is given in <u>Fluid Mechanics</u>[7] as $\rho C_d U^2/2$ times the area. C_d is the coefficient of drag, which for a rectangular plate approaches 2.0 as the ratio of the length to the height approaches infinity (see page 259 of Streeter and Wylie). U is the fluid velocity, and ρ is the fluid density (1025kg/m^3). The TNH report identifies the current in this area of Valdez bay as generally under 5cm/s, but there are recorded values as high as 7.9cm/s. Using the fastest value, the drag pressure is 2*1025*0.079*0.079/2 = 6.4N/m^2 (= 0.134psf). This force is insignificant compared to the wave and wind forces, and will therefore be ignored. The drag coefficient is valid only for Reynolds numbers greater than 10, and for smaller Reynolds numbers, C_d gets much larger. The Reynolds number is LU/(.01cm^2/s). If the velocity, U, is less than 7.9cm/s, then R = 790L/cm. Where L is the characteristic dimension. For R to be less than 10, L must be less than 0.013cm. Since the wave barrier is larger than that, we can safely use 2.0 for the drag coefficient.

Dead load

This will consist of the weight of the piles, the concrete wall, and all connection hardware. The weight of the piles is calculated by RISA, based on the pile size selected. For the analysis, the wall is assumed to be 12" thick, normal weight reinforced concrete, applied as a distributed load along the vertical pile. The connection hardware will be two steel slots connected to the vertical pile with bolted band clamps (see the design drawings). The specifics of this connection detail are not designed, and may change before the final design. As a conservative approach to the dead load for the hardware, we can replace it with the weight of the missing concrete wall. That is, we will assume the weight of the concrete wall goes from pile centerline to pile centerline. To test this assumption, we note that for a 24" in diameter pile, we will approximate the steel connection hardware with 1 cubic foot of concrete (145lbs) per foot of pile. This provides for a substantial amount of steel. If the pile size gets larger, this approximation will continue to get more conservative.

Corrosion control

We will provide some form of cathodic protection to be designed later. This will probably include a protective coating on the piles in the splash zone and above, as well as sacrificial anodes underwater. Due to the potential for damage, an extremely hostile environment, or irregular maintenance there may be periods were the cathodic protection system is not performing as designed. A brief survey of the web for corrosion rates for steel in a marine environment indicated that there are numerous factors that make prediction of steel loss nearly impossible. Nevertheless, it seemed that .1mm/year on the inside, and .4mm/year on the outside may be reasonable numbers for unprotected steel. At these rates, if we analyze the piles $1/8^{th}$ thinner than the specified thickness, we will be allowing for about 6 years of unprotected life. It should be noted that corrosion rates may be much higher, and if the structure is not adequately protected and monitored it may fail long before its programmed life.

Fatigue analysis

Not provided at the stage

Combined Forces

The load case combinations are typically 1.2 * Dead + 1.6 * Live. Live was either wind + wave or seismic. The seismic load was also considered with a –1.2 factor on the dead load, which is essentially a negative g factor, or a weightlessness of the structure due to vertical acceleration. The wind and live load combinations are shown in figure 3.

Member Geometry

In developing the geometry, we original looked at the possibility of a single cantilever pile. This solution would be the most economical as it would require only half of the pile driving. However, it quickly became obvious that the pile diameters were becoming prohibitively large even in relatively shallow water. Next we considered a vertical pile to support the wall, and a batter pile to brace the vertical pile. A simple truss analysis was all that was necessary provided we considered the pile to pile connection and mud to pile connection as pinned (free to rotate). This resulted in very large moments in the vertical pile, and relatively small axial loads in the batter pile. In order to more realistically model the pile to pile connection and the pile to mud interface, we switched to the RISA modeling software, which allowed us to consider the pile fixed against rotation at some depth below the mud line, and also to treat the pile to pile connection as a rigid moment carrying connection. This allowed much better load sharing among the piles, bringing the peak stresses down. If the batter pile can be connected at the vertical wall at a point close to the vertical center of the load, then all the lateral force will be carried in axial loading, instead of shear in the vertical. For our analysis of preliminary configurations, this did indeed provide for the lowest combined stresses in the members (i.e. most efficient use of steel). Nevertheless, the provided two construction hardships, 1) this connection would be so low that the contractor would either have to work around the tide schedule, or work underwater. 2) The vertical pile would be in the way of driving the batter pile, which would either require an eccentric connection (introducing torque into the vertical), or it would require the contractor to full penetration field weld to attach the top portion of the vertical. This weld would be at location of large stresses, and would therefore be very large. For these two reasons, we elected to move the connection point as high as possible. The result is higher steel costs, due to a longer batter pile, and potentially larger member size (due to the larger loads), however this should be more than offset by the decrease labor for construction.

Rotational fixity at the base: horizontal loads on a vertical pile will cause it to deflect and rotate at the mud-line. This can be modeled as a pinned (hinged base) at the mud-line. This is generally conservative, as it requires the moment at the base of the pile to be zero. This ignores the fact that the pile has a large moment carrying capacity, and if the pile is driven deep enough the soil will act as a clamp, preventing the pile from rotating below a certain depth. If we can determine this depth, then we can call that the base of the structure, and model that as a rotationally fixed connection. In analyzing preliminary models, we found that despite the longer lengths of the piles, a fixed base was a much more efficient way to carry load in the pile frame. The geotechnical engineers suggested that a rule of thumb indicates that at 5 feet (or 1.6m) below the mud-line (assuming at least 9m of total embedment) the pile can be considered fixed against rotation. A more rigorous method of finding the depth of fixity was attempted in the worksheets in appendix I. We finally settled on using a depth of 2.8m below the mud-line, however there is some uncertainty that this is correct, and further analysis is definitely required.

Depth: We increased the depth until we reached 35 meters below MLLW. The very long lengths of these piles, batter piles are greater than 65m (215ft), and the deep water pile driving let us to assume that we may be reaching or even exceeding an economic limit. If the 35% cost estimate warrants it, then deeper solutions can be explored in subsequent stages of the design. It should be noted that any deeper will require a new hydrographic survey, as the current design is right on the edge of the existing survey.

Batter angle: Having the batter pile kicked way out (more flat, less steep) results in a more efficient frame, which (provided the pile does not get too long/slender) will result in a smaller pile size. However having the batter pile steeper has advantages as well. The pile is shorter (less steel), and pile driving is more efficient. It is not recommended that you try to drive piles that are flatter than 1 horizontal to 2 vertical. We were originally aiming for a 3H:12V angle, and preliminary analysis indicated this would work for our model. However, after all the final tweaking of load cases and geometry, we were about 8% overstressed in one load case. When we increased the batter angle to 4H:12V (1H:3V) the model was only 1% over stressed in that load case.

Member sizes: From a survey of manufacturers on the web, it appeared there were several manufacturers that could provide steel pipe piles in large diameters. I have included the Skyline Steel LCC cut sheets and contact in Appendix J. Pipes in the larger diameters are going to be spiral welded, and can typically be ordered in lengths up to 100ft (30.5m) and in grades up to 50ksi (345mPA) yield stress. For the larger diameters, piles are usually available in diameters that are even increments of 6" and thicknesses up to ¾". Due to the potential for corrosion losses, and the larger diameters being considered, we only evaluated the maximum diameters available. This was 5/8" for diameters less than 42" and ¾" for diameters greater than 36". The steel section properties that were used in the RISA model were reduced to the minimum dimensions allowable by ASTM tolerances, and for the corrosion losses.

RISA Model and Results: The model as discussed above was evaluated in the RISA 3D finite element package. Input parameters, and output is available in appendix K. The highest stresses were combined stresses due to compression and bending in the vertical pile with an onshore wave/wind at about mid-tide-level. This resulted in a combined stress of about 101% of capacity, which is well within our level of certainty on the loads. It should be noted that in several cases, we have made conservative assumptions. For example, 1) Since the wave barrier will follow a straight line, and not the contours nearly all the piles will be in less than 35m of water. 2) The full weight of a 12" not 8" concrete wall was considered, no reduction for buoyancy was provided. 3) The wave forces are for a wall in 10m of water, even though theory predicts that the forces should get lower for deeper water. 4) All walls were designed with the west wave, which is conservative for all but the western portion of the wave barrier. The point of fixity remains a open question, and this may increase of decrease the stresses in the member. If stresses go up substantially, we can probably accommodate that by bringing the deeper portions into slightly shallower water. This will decrease the load, at a small reduction in harbor size.

Concrete panel design

See appendix L for a MathCAD worksheet calculating the required steel area. Note that the wall is reduced to 8" (20cm) from the original 12" estimated in the deadload. This will reduce the stress on the vertical compression member.

We need to consider how the concrete panels and their connections will resist a seismic load longitudinal to the wave barrier (i.e. the concrete panels will act as shear walls).

Connection details

Calculation for the connection details are not complete at this time, see the drawings for a potential connection detail. Note that we need to leave capacity in the pile to resist the punching stresses to the connection point loads.

Foundation design

Ultimate tension and compression capacities for open ended vertical and batter pipe piles was tabulated in tables 1 (page 13) and 2 (page 14) of the COE Geotechnical report[8]. A copy of the tables is provided in Appendix M. A safety factor of 0.5 (50%) was applied to the ultimate load to derive a maximum allowable design load. From the RISA analysis, the maximum tension and compression in the vertical pile was 478kN (tension) and 2159kN (compression). The maximum in the batter pile was 1305kN (tension) and 1503kN (compression). The RISA values already have a 1.6 or 1.2 load factor on them as well, so the safety factor is now over 2.5. By interpolating the tables in the geotechnical report, we come up with the required embedment depth below the mudline:

Condition	Design Load (kN)	Ultimate Load (kN)	Required Embedment
Vertical Pile in Tension	478	956	< 9m
Vertical Pile in Compression	2159	4318	<9m
Batter Pile in Tension	1305	2610	12.1m
Batter Pile in Compression	1503	3006	<9m

We will specify a minimum embedment of 12.2 meters for the batter piles, and 9m (29.5ft) for the vertical piles.

As I completed the 35% design, I realized that there is a significant problem developing the tensile capacity. The problem is that in order to drive the piles that deep, they will may encounter a compression resistance greater than the compression capacity of the pile. Depending on the assumptions regarding the effective length (k) factor, the location of followers or other lateral restraints, and the ability of the pile driver to keep from loading the pile in bending while driving, it appears that the vertical pile is ok, but any unexpected resistance, or problems in the assumptions could lead to a failure in driving. The batter pile appears to be overloaded. This can be remediate by either bringing the wave barrier into shallower water, putting the batters on the inside, or upsizing the batters.

Since the problem is in construction not the final designed configuration, it may be possible that there is some solution the contractor can implement, such as providing intermediate lateral bracing, vibration driving, or other techniques.

Constructability

The primary considerations for constructability were size and length of the piles. A survey of marine pile driving firms indicated that hammers and equipment exist to drive piles larger than 6 feet in diameter, and techniques developed for off-shore platforms could be used to drive piles in very deep water. Hopefully these expensive techniques will not be required. It is sufficient for the structural design to know that a solution is possible. It is assumed that the deep water will require pre-welding shorter sections of pile together before lowering it into the water. This may require two cranes to handle the very long pipe.

Another constructability consideration was to minimize underwater work. This drove us to raise the vertical to batter pile connection point to the top of the wall, instead of the middle of the wall where it is most efficient. We have also designed a connection “frame” that can be largely bolted to the vertical piles, which provides a ledger and guides for the wall, so that wall panels can be conveniently lowered into place. Additional underwater bolting will be required to secure the wall panels from moving around in the guides. The connection “frame” also provides a saddle for guiding the batter pile while it is being driven. The saddle is adjustable so that it can provide a solid, no-shim, connection between piles.

Since the concrete panels will rest on ledgers and be held in place between guides, they are dependent on the vertical to vertical spacing of the piles. If the piles are too close, the concrete panel will not fit, and if the piles are too far apart, the connection will be too weak. Since it is unlikely that the vertical piles can be placed with sufficient accuracy, it is recommended that the concrete panels be made to order once the as-built spacing of the vertical piles is established.

Piles:

Wood – Wood piles are not feasible due to the depth of water. We anticipate that the piles will be over 100 feet long.

Concrete – delivering concrete piles of sufficient length to the project site will be very difficult, and it will be necessary to adjust the

Steel – This the only practical pile material due to the availability of large diameters, long lengths, and the ability to weld extension on, or cut off additional stick up.

Walls:

Concrete walls were chosen due to their durability and the convenience of fabricating them offset, and simply lowering them into place. It may be possible to design a timber or steel wall. These options may be lighter and cheaper than the concrete solution provided in this design.

Connections:

Steel was the only alternative. See the details in the drawings. The connection pieces can be completely fabricated offsite, and then welded/bolted into place. All underwater work will be bolting. The only field welding required will be to make the top moment connection fully rigid after it is adjusted for the field arrangement of the vertical and batter piles.

References

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1 EM 1110-2-1615, “Hydraulic Design of Small Boat Harbors”

2 Kriebel, Dr. David L, personal email, dated November 6, 2004, see Appendix A

3 Prepared April 2003 by Tryck Nyman Hayes, Inc.

4 page VI-5-150 of the CEM.

5 NOAA, http:/co-ops.nos.noaa.gov/benchmarks/9454240.html on November 3, 2004

6 PN&D 95S100 Site visit Report to the Seattle Central Water Front Project, December 14, 1995

7 8th edition by Streeter and Wylie, page 258

8 Geotechnical Investigation for Navigation Improvements , Valdez Alaska (CWIS 010600), US Army Corps of Engineers, Alaska District, Soils and Geology Section, November 2004.

Figure 1
Wave Barrier Elevation Schematic

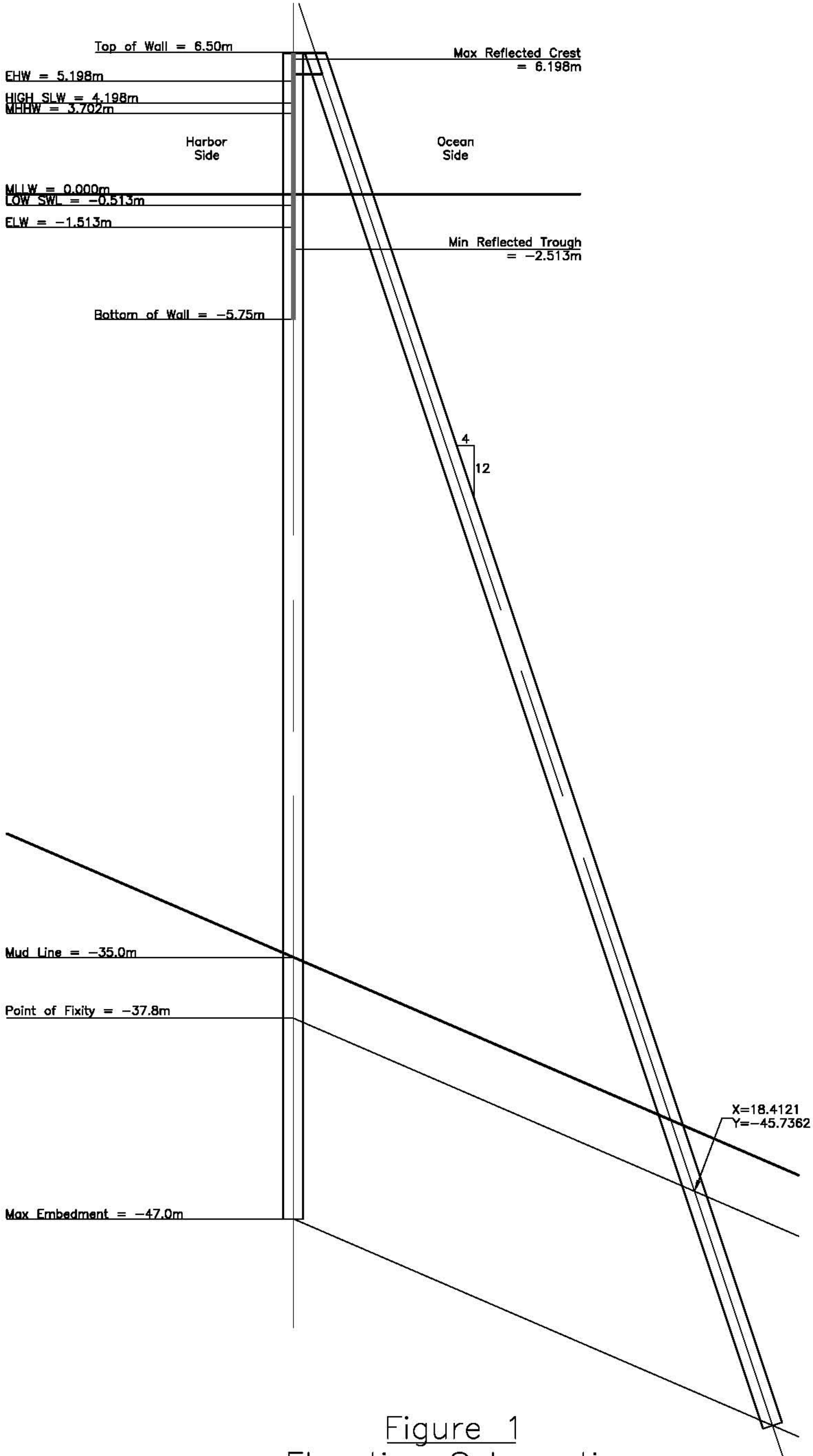


Figure 1
Elevation Schematic

Figure 2
Site Plan and Wave Attack Angles

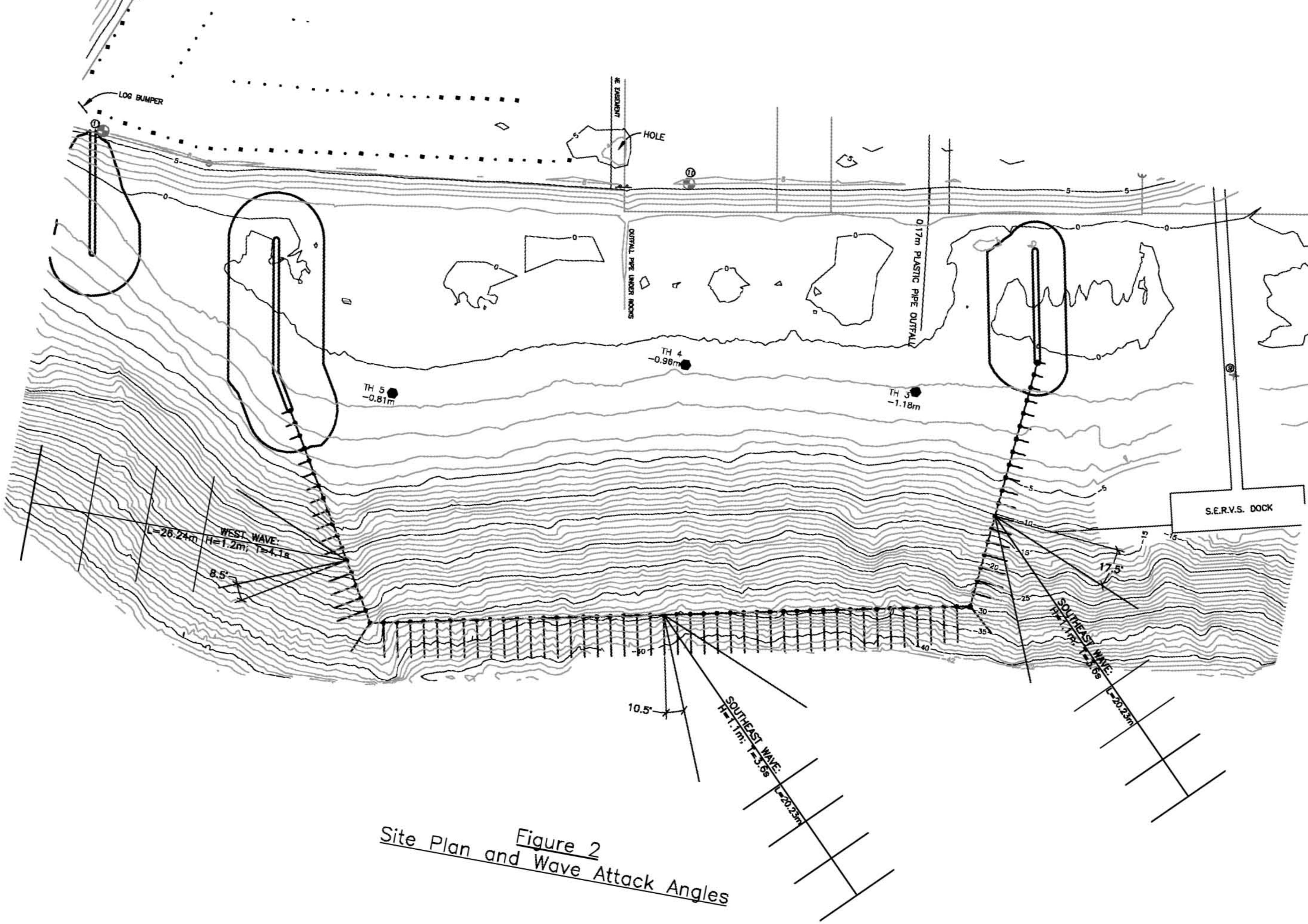
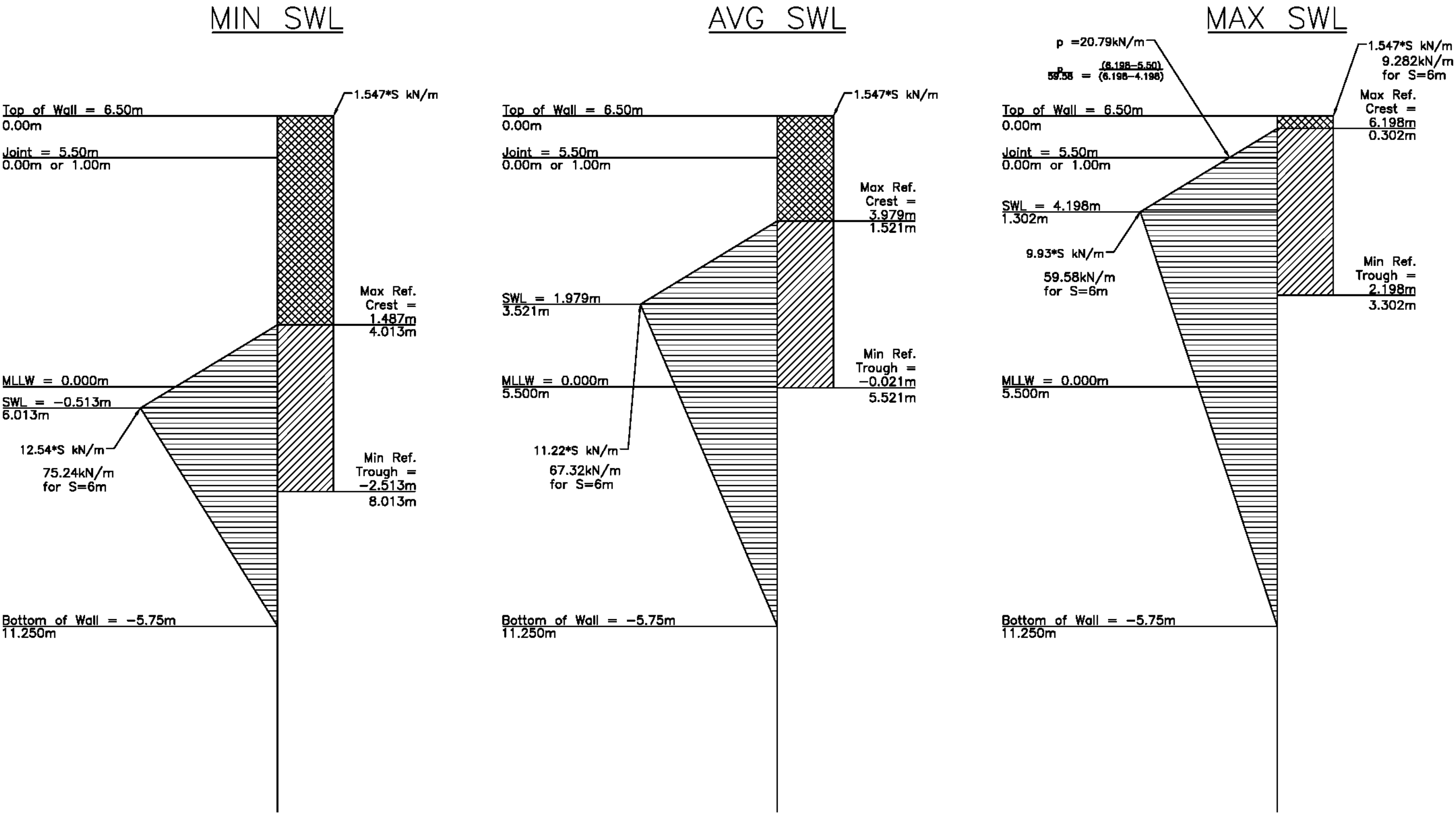


Figure 2
Site Plan and Wave Attack Angles

Figure 3
Combined Wave and Wind Forces



LEGEND

- Shoreward Wind Force with Shoreward Wave Force
- Shoreward Wind Force with Seaward Wave Force
- Shoreward or Seaward Wave Force

ABBREVIATIONS

SWL	Still Water Level
MLLW	Mean Lower Low Water
Ref.	Reflected
S	Pile to Pile Spacing in meters
m	meters
kN	Kilo-Newtons

NOTES

1. Number below the lines is the distance along the member from the joint above.
2. Numbers above the lines are referenced to MLLW = 0.0m
3. Wave pressures are for the West wave only (largest pressures).
4. Wind pressure per ASCE, assumed to be normal to wall, wind force extends from the top of the wall to the top of the water (crest or trough), not the SWL.
5. Pressures are not shown to scale., and wave forces will act in both directions.

Figure 3, Combined Wave and Wind Forces

ATTACHMENT C
Preliminary Conceptual Plans
Navigation Improvements, Valdez, Alaska

US Army Corps
of Engineers
Alaska District

NAVIGATION IMPROVEMENTS

VALDEZ, ALASKA

SCHEDULE OF DRAWINGS		
SHEET	REF #	TITLE
		GENERAL
1	G1.1	COVER SHEET
2	G2.1	LOCATION AND VICINITY MAP
		CIVIL: PLANS
3	W1.01	HARBOR PLAN
		CIVIL: SECTIONS
4	W2.01	CROSS SECTIONS
		STRUCTURAL: GENERAL
5	S0.01	NOTES AND ABBREVIATIONS
		STRUCTURAL: PLANS
6	S1.01	WAVE BARRIER PLAN
		STRUCTURAL: ELEVATIONS
7	S2.01	WAVE BARRIER
		STRUCTURAL: DETAILS
8	S3.01	PILE TO WALL CONNECTION
9	S3.02	PILE TO PILE CONNECTION
10	S3.03	WALL PANELS

PRELIM, 25JAN05

VAL002

CWIS 010600

PRELIMINARY CONCEPTUAL PLANS

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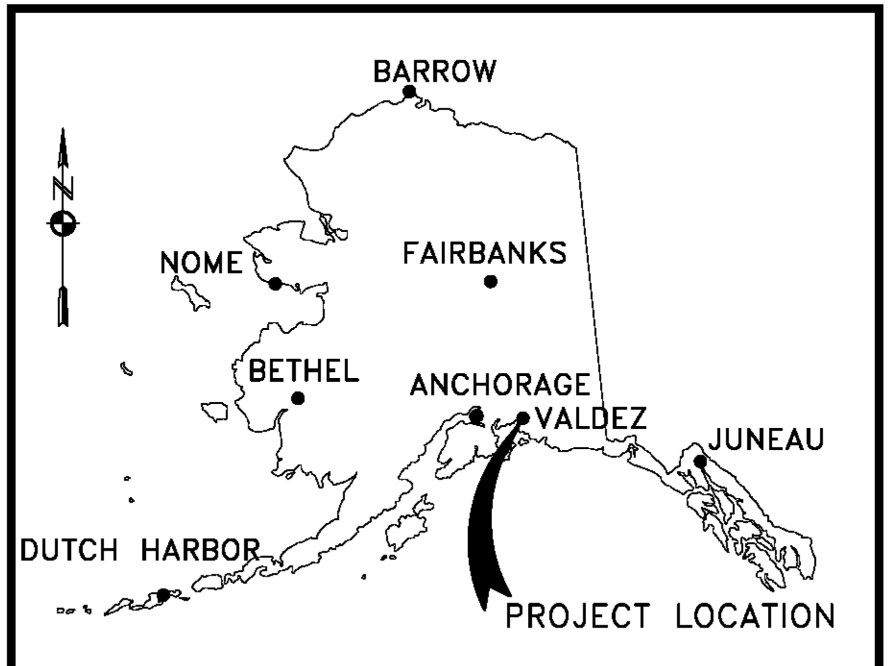
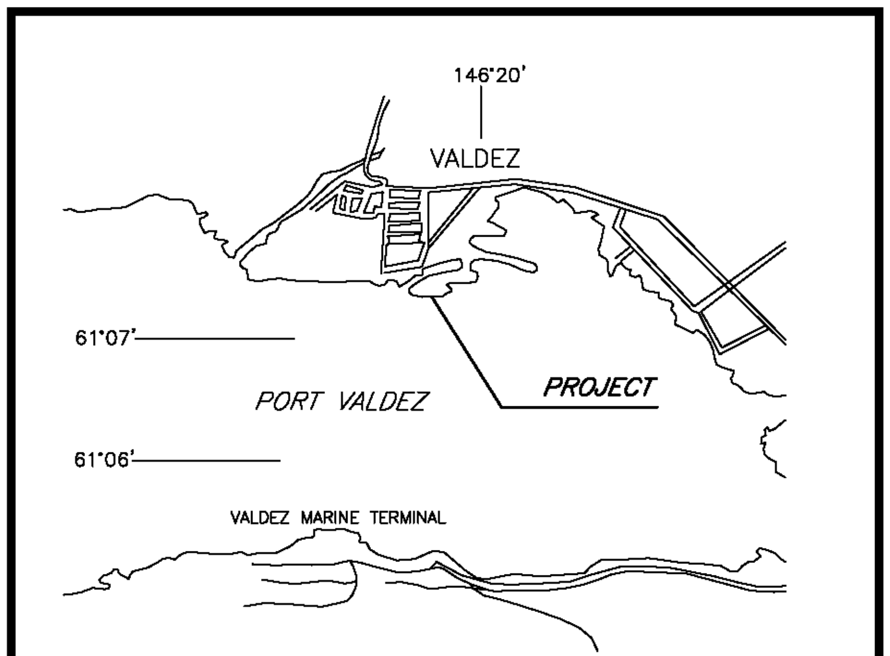


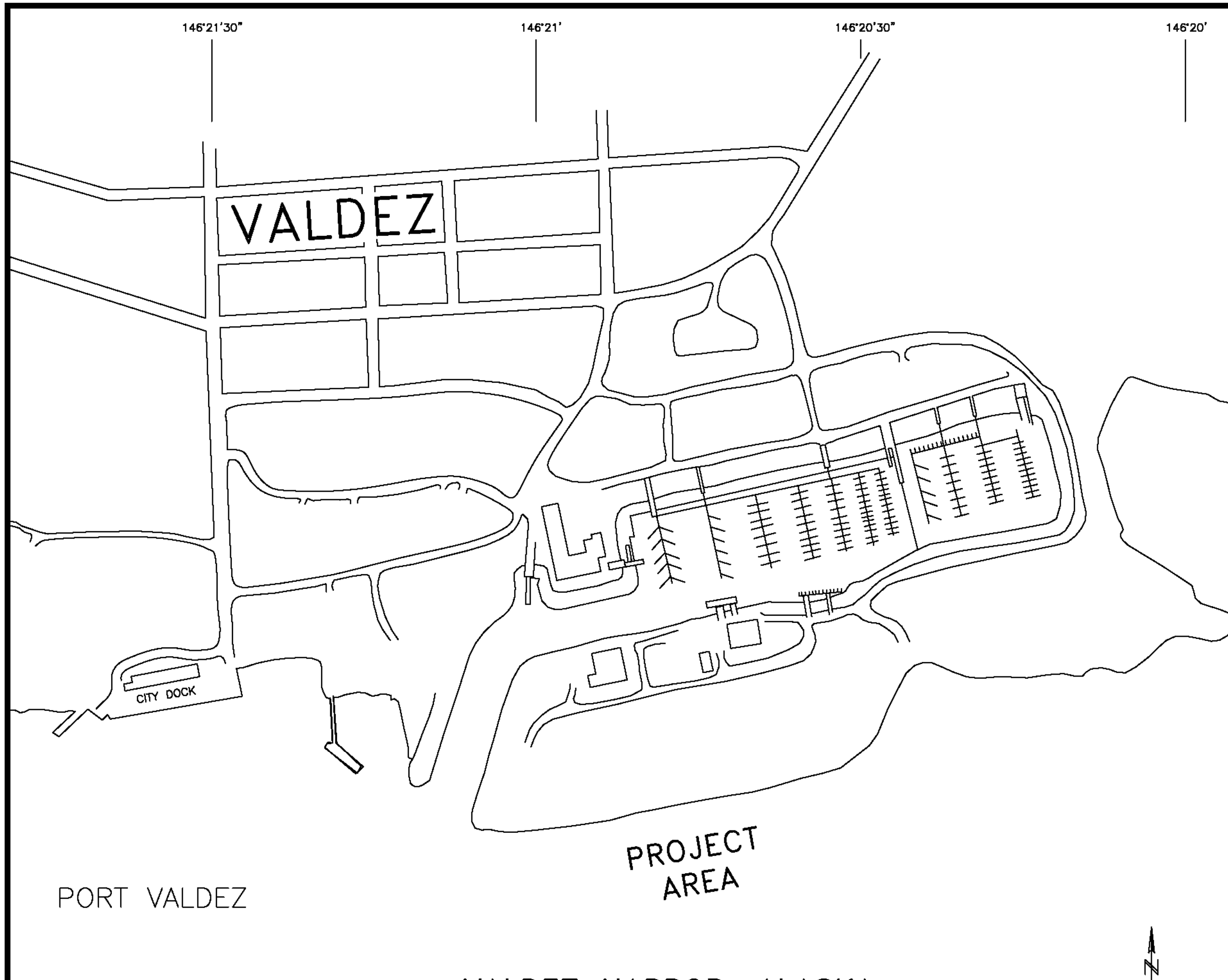
LOCATION MAP



VICINITY MAP

1 .5 0 1 2

SCALE IN MILES



VALDEZ HARBOR, ALASKA

400 0 400 800

SCALE IN FEET

150 0 150 300

SCALE IN METERS

NOT FOR CONSTRUCTION

PRELIM

US ARMY CORPS OF ENGINEERS
ALASKA DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

Date: 25JAN05
Dwg Scale: AS NOTED
Plot Scale: 1:2
File: VAL002-02-G2-1
Drawing #: unknown

Reviewed: K. Eisses
Submitted: D. Hardy

PRELIMINARY CONCEPTUAL PLANS
CWIS 010600 VAL002

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
HYDRAULICS & HYDROLOGY
GENERAL
LOCATION AND VICINITY MAP

Reference number:
G2.1
Sheet 2 of 10

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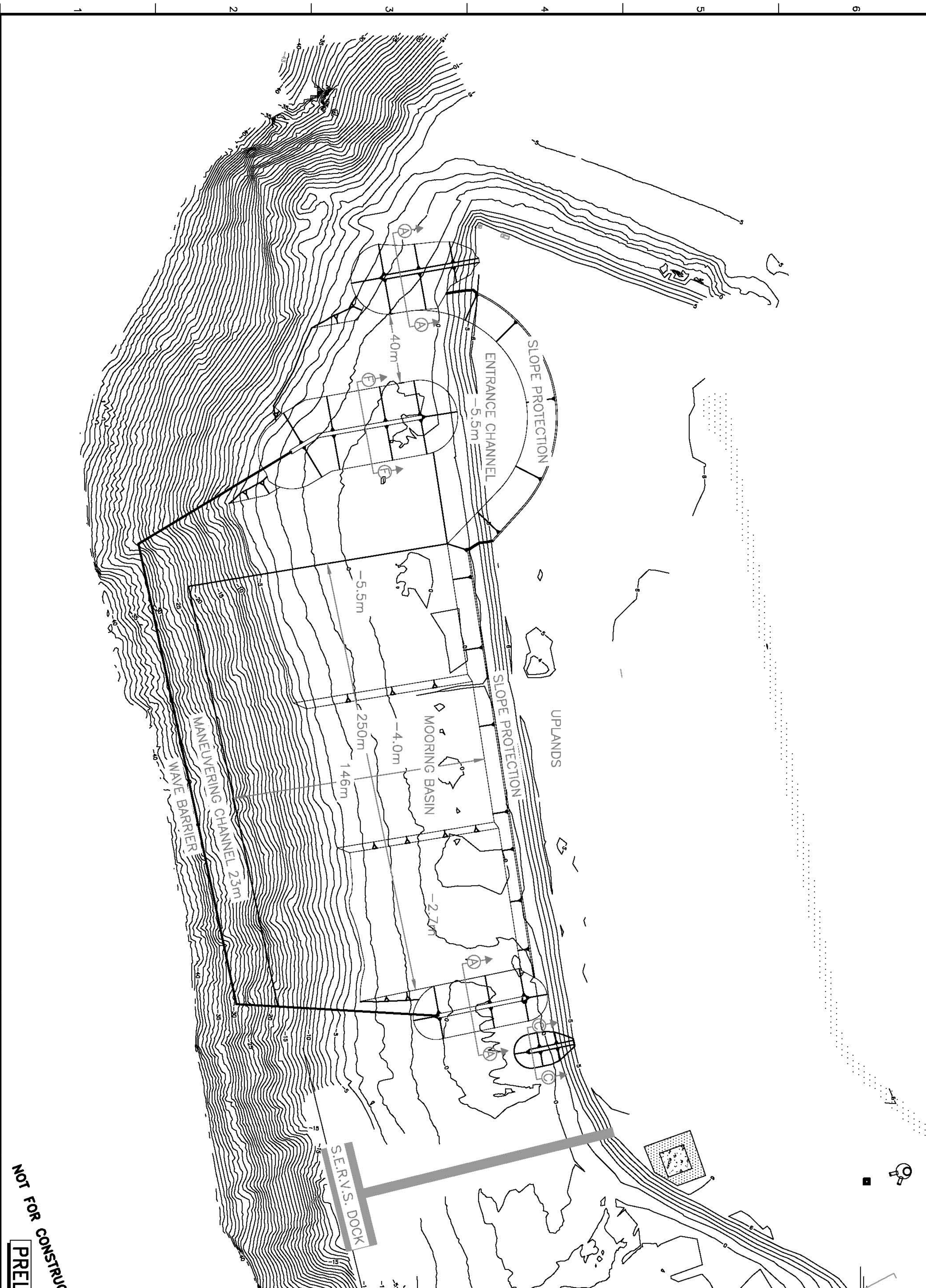

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Reference number: W1.01

Sheet 3 of 10

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
HYDRAULICS & HYDROLOGY
PLANS
HARBOR PLAN

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

Designed: MDP
Drawn: MDP
Reviewed:
Chief, EN-CW-HH Section
Submitted:
Chief, CEPOA-EN-CW Branch

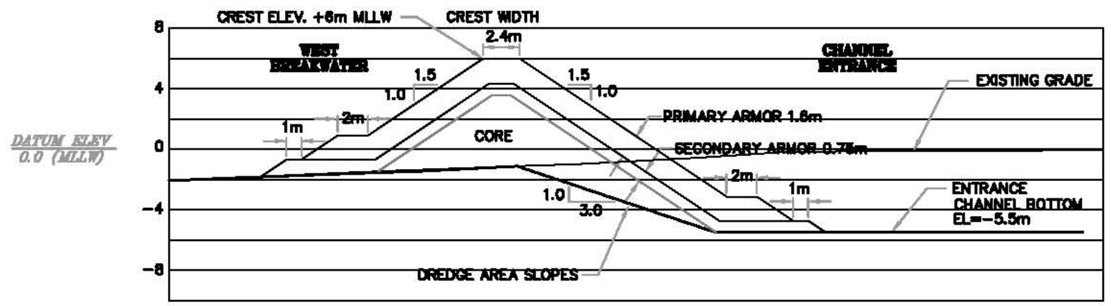
Date: 25JAN05
Dwg Scale: AS NOTED
Plot Scale: 1:2
File: VAL002-03-W1-01
Drawing #: unknown

PRELIMINARY CONCEPTUAL PLANS
CWIS 010600 VAL002

Sym.	Action	Description	Date	Appvd

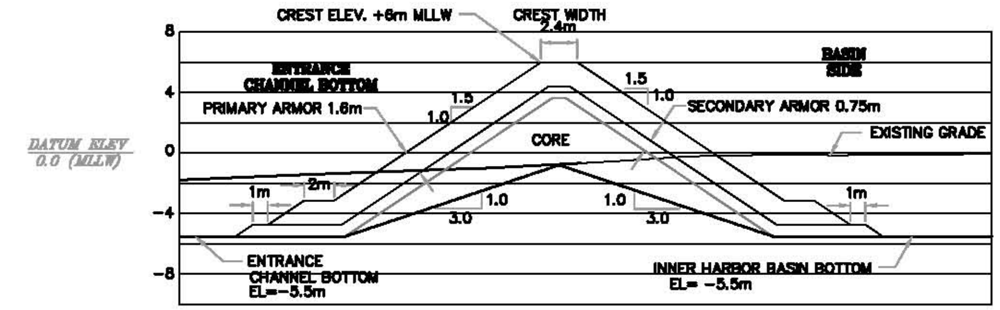
CONTRACT NO.
CONTRACTOR
CITY STATE
Recommended: PRIME CONTRACTOR
Approved: RESIDENT ENGINEER
Date:

US ARMY CORPS OF ENGINEERS ALASKA DISTRICT



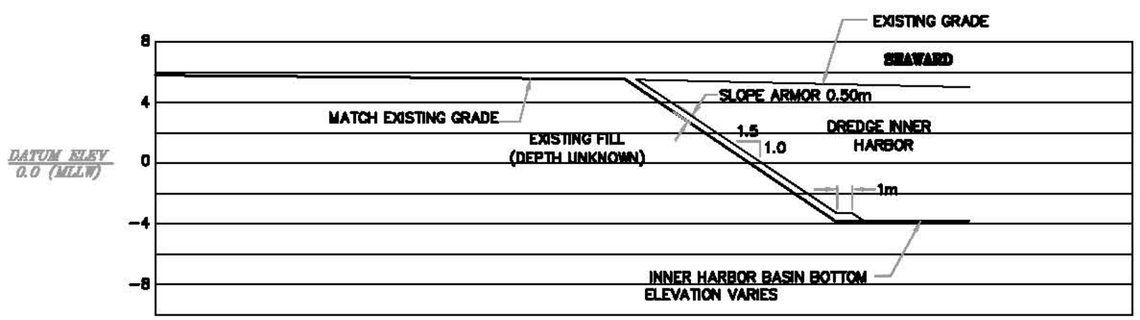
SECTION A

WEST ENTRANCE CHANNEL BREAKWATER SECTION



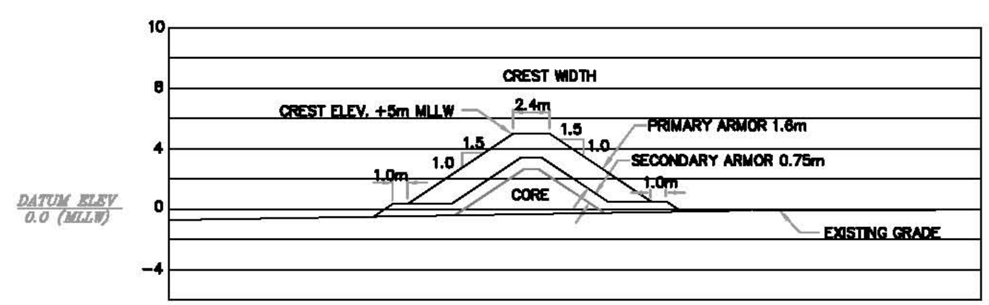
SECTION F

SOUTH ENTRANCE CHANNEL BREAKWATER SECTION



SECTION B

NORTH SIDE INNER HARBOR ARMORED SLOPE



SECTION C

STUB BREAKWATER SECTION

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US ARMY CORPS OF ENGINEERS ALASKA DISTRICT

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS ANCHORAGE, ALASKA

Designed: MDP
Drawn: MDP
Date: 25JAN05
Dwg Scale: AS NOTED
Plot Scale: 1:2
File: VAL002-04-W2-01
Drawing #: unknown

PRELIMINARY CONCEPTUAL PLANS VAL002
CWIS 010600

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
HYDRAULICS & HYDROLOGY
SECTIONS
CROSS SECTIONS

NOT FOR CONSTRUCTION

PRELIM

Reference number: W2.01

Sheet 4 of 10

NOTES

1) SEE THE DESIGN ANALYSIS FOR DESIGN CRITERIA
2) ALL UNITS ARE IN MILLIMETERS UNLESS NOTED OTHERWISE
3) ALL CONCRETE SHALL HAVE A MINIMUM 28 DAY COMPRESSIVE STRENGTH OF 27.5 MPa
4) ALL REINFORCING BARS SHALL CONFORM TO ASTM A615M GRADE 415, AND SHALL BE GALVANIZED PER _____.
5) ALL BOLTS SHALL CONFORM TO ASTM A325M
6) ALL WELDING SHALL BE IN ACCORDANCE WITH THE LATEST EDITION OF AWS D1.1, BY CERTIFIED WELDERS USING ELECTRODES CONFORMING TO CLASS E480. ALL WELDS SHALL BE 5mm MINIMUM, UNLESS NOTED OTHERWISE.
7) ALL STEEL SHALL BE GRADE 50 AND GALVANIZED

ABBREVIATIONS

°	DEGREES
'	MINUTES
"	SECONDS OR INCHES
#	NUMBER
ø	DIAMETER
CONC.	CONCRETE
E	EAST
E.F.	EACH FACE
L	STEEL ANGLE
MAX.	MAXIMUM
MIN.	MINIMUM
MLLW	MEAN LOWER LOW WATER
m	METERS
mm	MILLIMETERS
MPa	MEGAPASCALS
N	NORTH
O.C.	ON CENTER
PL	STEEL PLATE
REBAR	CONCRETE REINFORCING BAR
TYP.	TYPICAL

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PRELIM

US ARMY CORPS OF ENGINEERS
ALASKA DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

Designed: RES
Drawn: RES
Reviewed:
Chief, EN-CW-HH Section
Submitted:
Chief, CEPOA-EN-CW Branch

Date: 25JAN05
Dwg Scale: AS NOTED
Plot Scale: 1:2
File: VAL002-05-S0-01
Drawing #: unknown

PRELIMINARY CONCEPTUAL PLANS
CWIS 010600 VAL002

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
STRUCTURAL
GENERAL
NOTES AND ABBREVIATIONS

Reference number:
S0.01

Sheet 5 of 10

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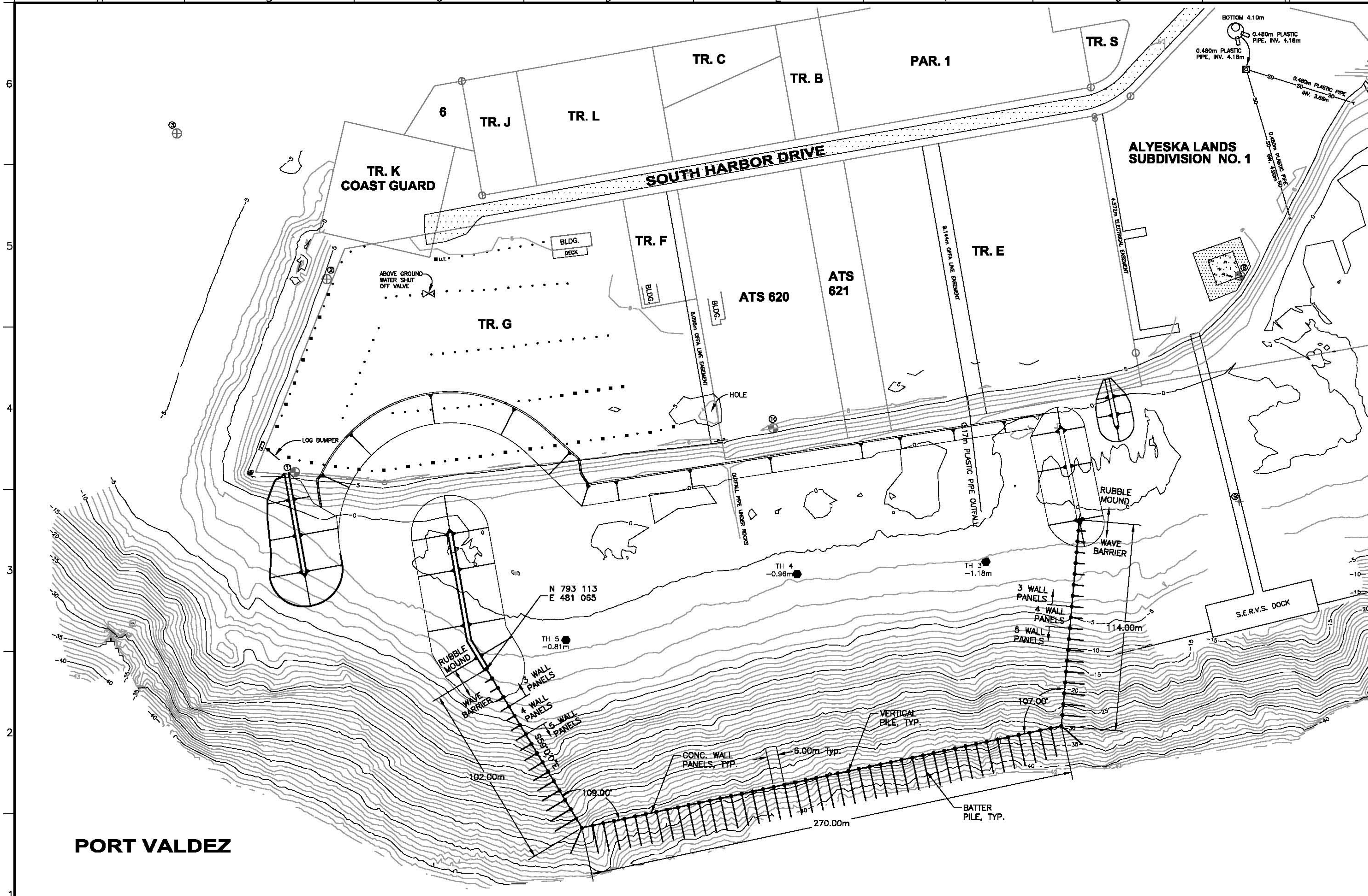


WAVE BARRIER PLAN

SCALE: 1:1000

NOT FOR CONSTRUCTION

PRELIM

US ARMY CORPS OF ENGINEERS
ALASKA DISTRICT

CONTRACT NO. ____
CONTRACTOR ____
CITY ____ STATE ____

Recommended:	Approved:	Date:
PRIME CONTRACTOR	RESIDENT ENGINEER	

Sym	Action	Description	Date	Apprvl

Designed: RES | Drawn: RES | Date: 25JAN05 | Dwg Scale: AS NOTED | Plot Scale: 1:2 | File: VAL002-06-S1-01 | Drawing #: unknown

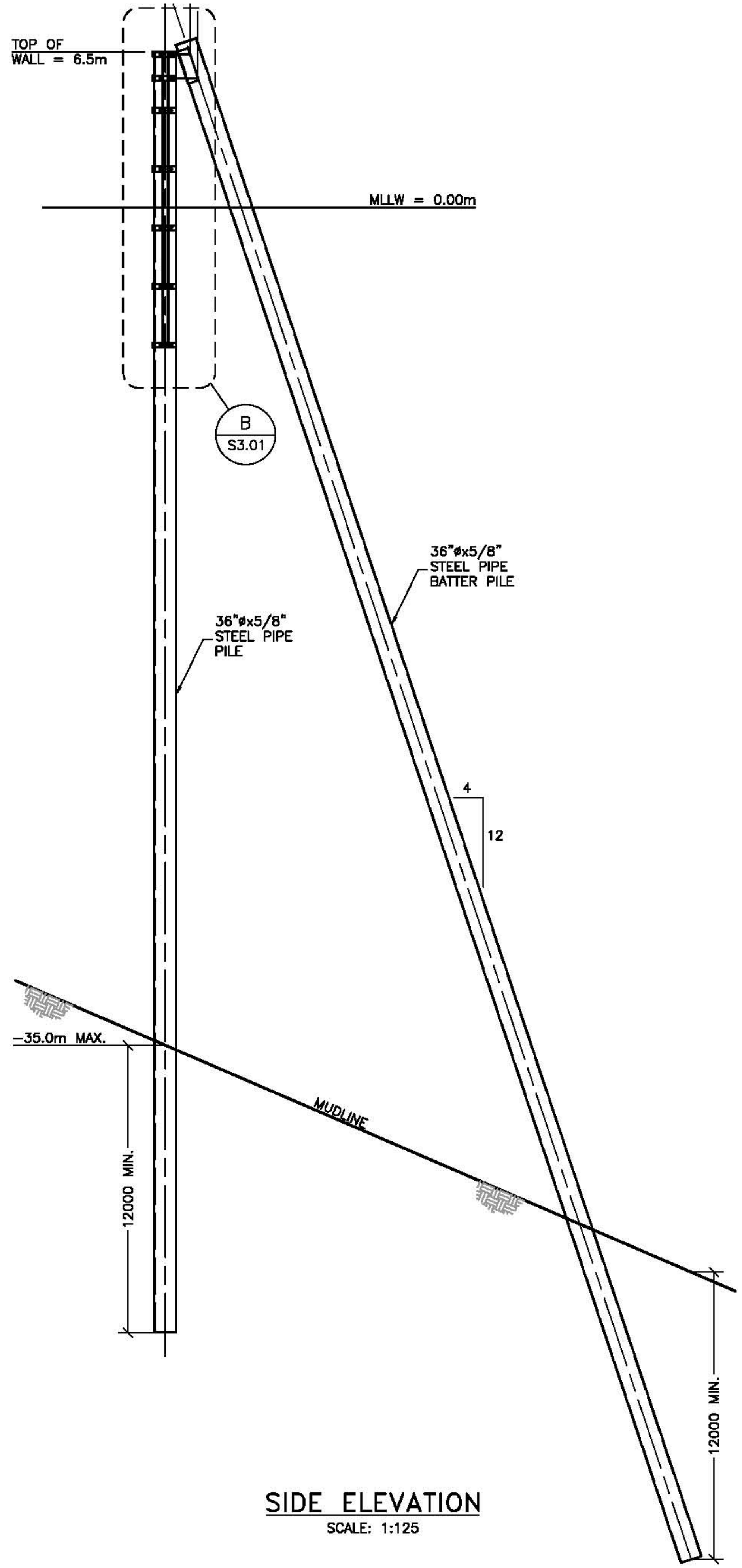
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

PRELIMINARY CONCEPTUAL PLANS VAL002
CWIS 010600

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
STRUCTURAL
PLANS
WAVE BARRIER PLAN

Reference number:
S1.01

Sheet 6 of 10



SIDE ELEVATION

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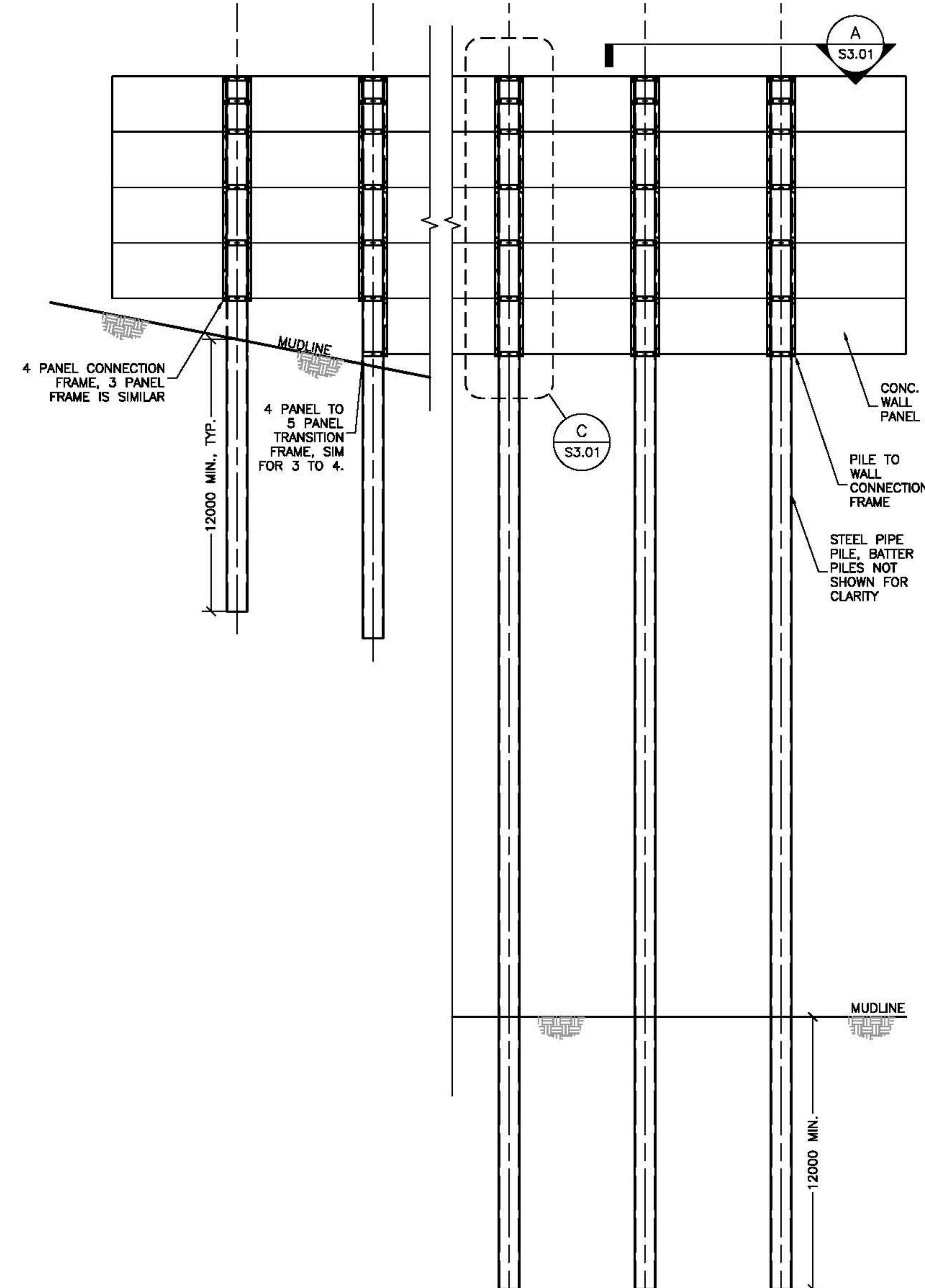

FRONT ELEVATION

SCALE: 1:125

NOTES

1) MUDLINE DEPTH AND SLOPE VARIES, SEE PLANS.

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PRELIM

US ARMY CORPS OF ENGINEERS
ALASKA DISTRICT

CONTRACT NO.
CONTRACTOR
CITY
STATE
Recommended:
PRIME CONTRACTOR
Approved:
RESIDENT ENGINEER
Date:

Sym.	Action	Description	Date	Apprvd

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

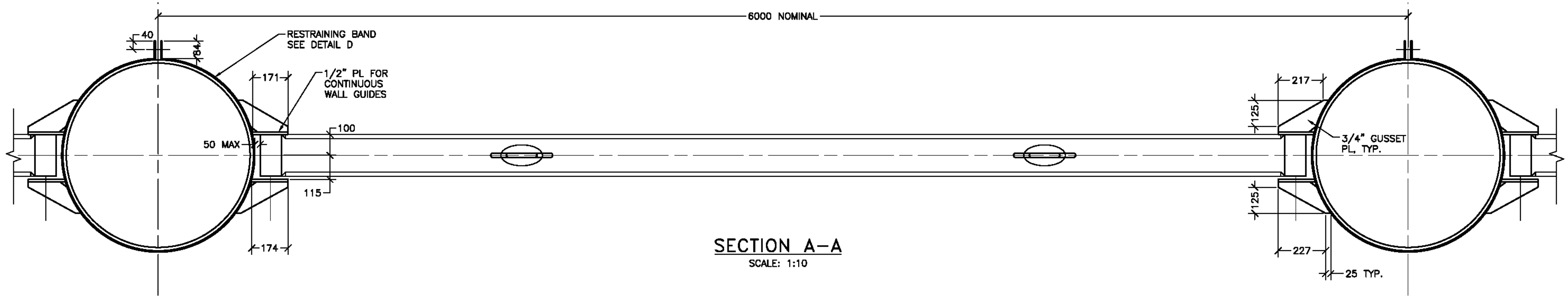
Designed: RES
Drawn: RES
Reviewed:
Chief, EN-CW-HH Section
Submitted:
Chief, CEPOA-EN-CW Branch

Date: 25JAN05
Dwg Scale: AS NOTED
Plot Scale: 1:2
File: VAL002-07-S2-01
Drawing #: unknown

PRELIMINARY CONCEPTUAL PLANS
CWIS 010600 VAL002

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
STRUCTURAL
ELEVATIONS
WAVE BARRIER

Reference number:
S2.01
Sheet 7 of 10



SECTION A–A
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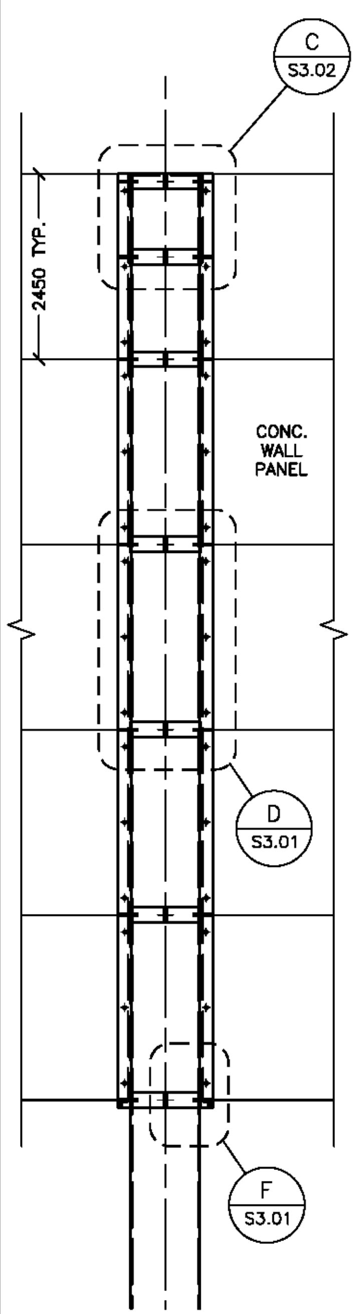
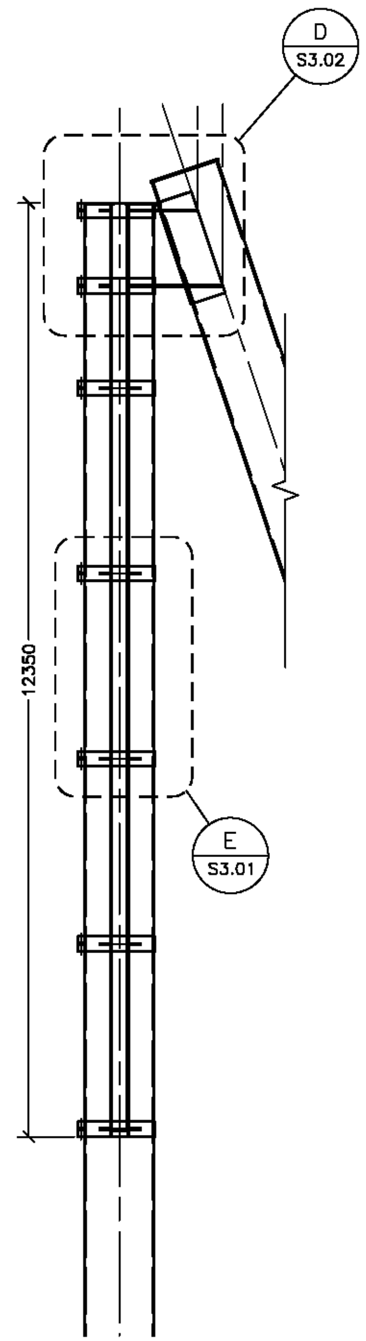
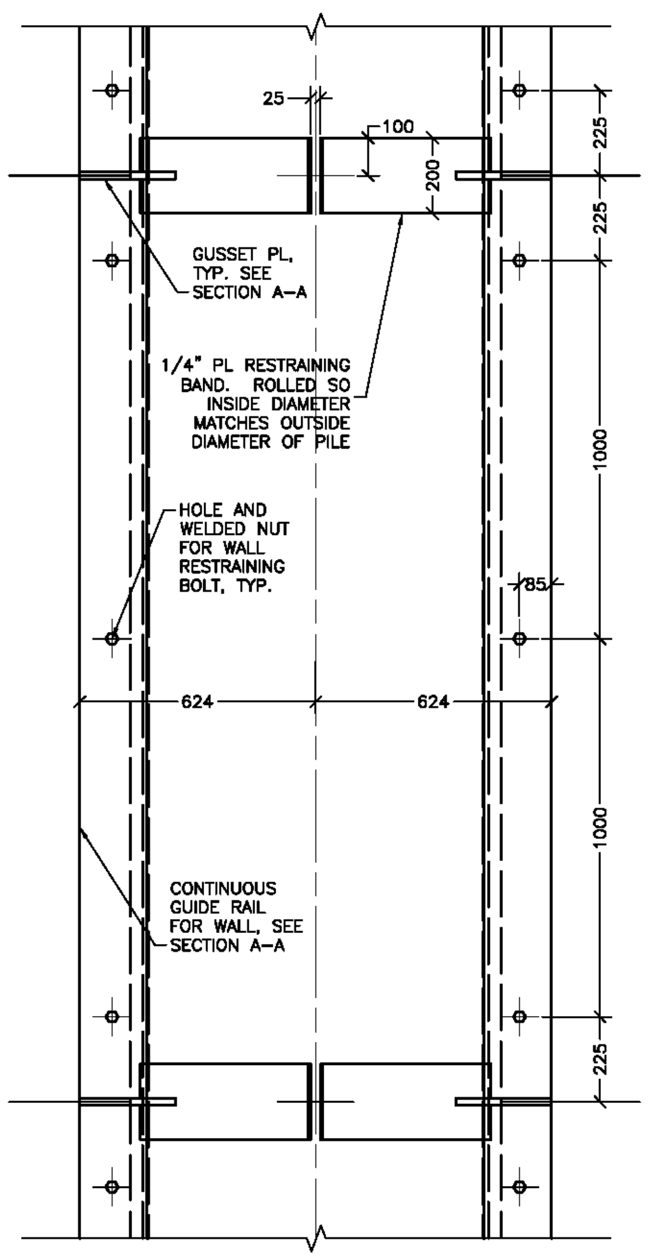


DETAIL B
SCALE: 1:50



DETAIL C
SCALE: 1:50



DETAIL D
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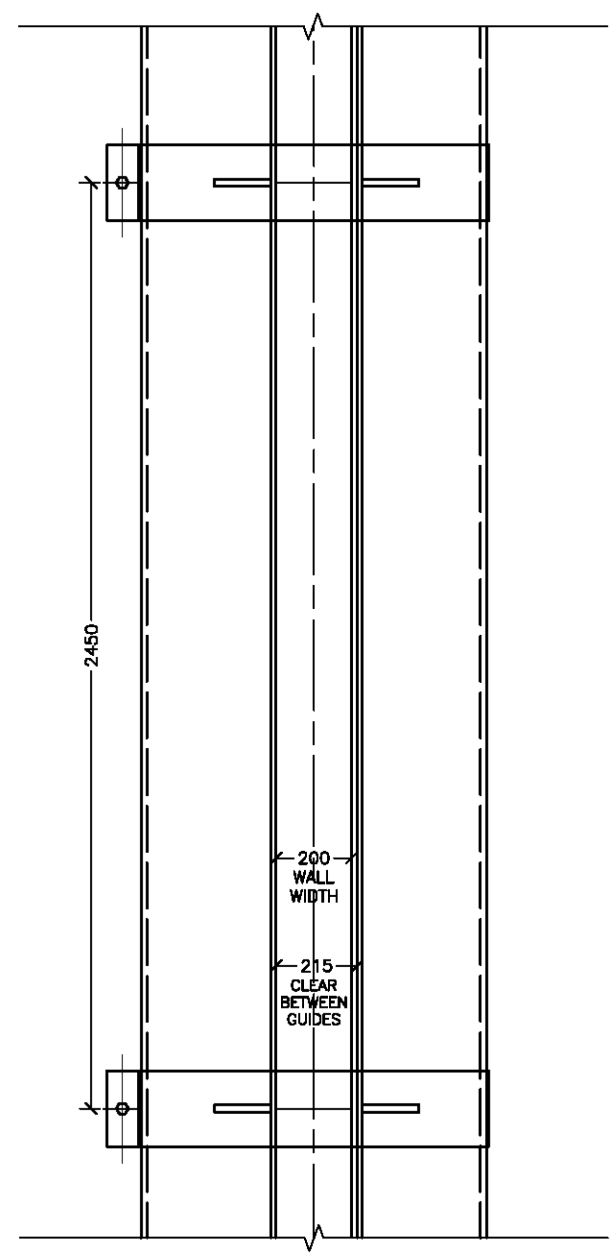
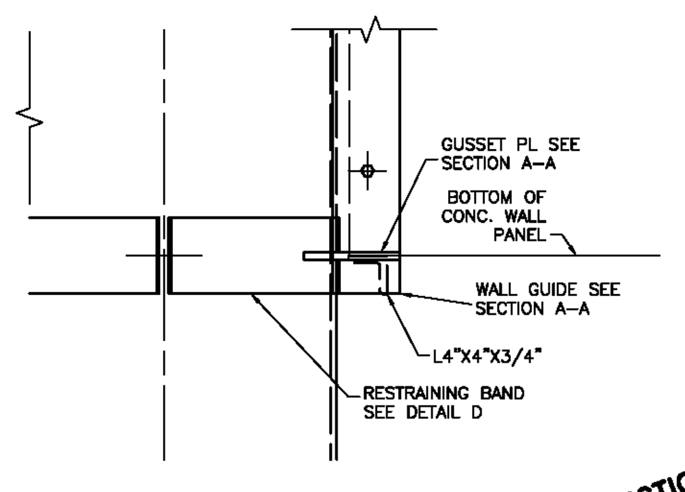


DETAIL E
SCALE: 1:10

NOTES

1) IF THE BAND IS TOO BIG, AND SHIMMING IS REQUIRED, A FULL LENGTH/HEIGHT SHIM IS REQUIRED AROUND THE ENTIRE PILE TO AVOID STRESS CONCENTRATIONS.
2) PROVIDE WEDGING TO HOLD THE BAND OPEN LARGE ENOUGH TO SLIP OVER THE PILE, OR AT THE CONTRACTORS OPTION, PROVIDE A SPLIT BAND, WITH TIGHTENING BOLTS ON BOTH SIDES.
3) PANELS CAN HAVE NO MORE THAN A 50MM GAP BETWEEN THEM AND EITHER PILE. SUGGEST THAT CONTRACTOR DOES NOT FABRICATE THE CONCRETE PANELS, UNTIL AS-BUILT GAPS BETWEEN PILES CAN BE MEASURED.
4) TIGHTEN THE RESTRAINING BAND BOLTS TO X.XX FOOT POUNDS
5) TIGHTEN THE PANEL RESTAINING BOLTS TO X.XX FOOT POUNDS. DO NOT OVER TIGHTEN, OR CONCRETE PANELS MAY CRACK. CONTRACTOR SHALL REPLACE ANY CRACKED CONCRETE PANELS.
6) BOLTS SHALL BE 3/4"ø AND PROVIDED WITH JAM NUTS TO PREVENT BACKING OUT OR LOOSENING.
7) THE CONNECTION FRAME WHICH COMPRISES EVERYTHING BUT THE PIPE PILES AND THE WALL PANELS SHALL BE ENTIRELY SHOP WELDED, AND THEN GALVENIZED BEFORE SHIPPING TO THE JOB SITE.
8) THE CONNECTION FRAME IS DESIGNED TO SLIP DOWN THE TOP OF THE VERTICAL PILE AFTER IT IS DRIVEN TO DEPTH AND CUT OFF TO THE CORRECT HEIGHT. THE CAP PLATE WILL BEAR ON THE TOP OF THE PILE. THE RESTRAINING BANDS WILL BE TIGHTENED TO SECURE THE ASSEMBLY IN PLACE. THE BATTER PILE WILL BE WELDED TO THE SADDLE AFTER BEING DRIVEN TO DEPTH. WALL PANELS WILL BE LOWERED INTO THE SLOTS TO REST ON THE LEDGER ANGLE OR THE PANEL BELOW.



DETAIL F
SCALE: 1:10

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PRELIM

US ARMY CORPS OF ENGINEERS
ALASKA DISTRICT

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS ANCHORAGE, ALASKA

Designed: RES
Drawn: RES
Date: 25JAN05
Dwg Scale: AS NOTED
Plot Scale: 1:2
File: VAL002-08-S3-01
Drawing #: unknown

PRELIMINARY CONCEPTUAL PLANS
CWIS 010600
VAL002

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
STRUCTURAL
DETAILS
PILE TO WALL CONNECTION

Reference number:
S3.01
Sheet 8 of 10

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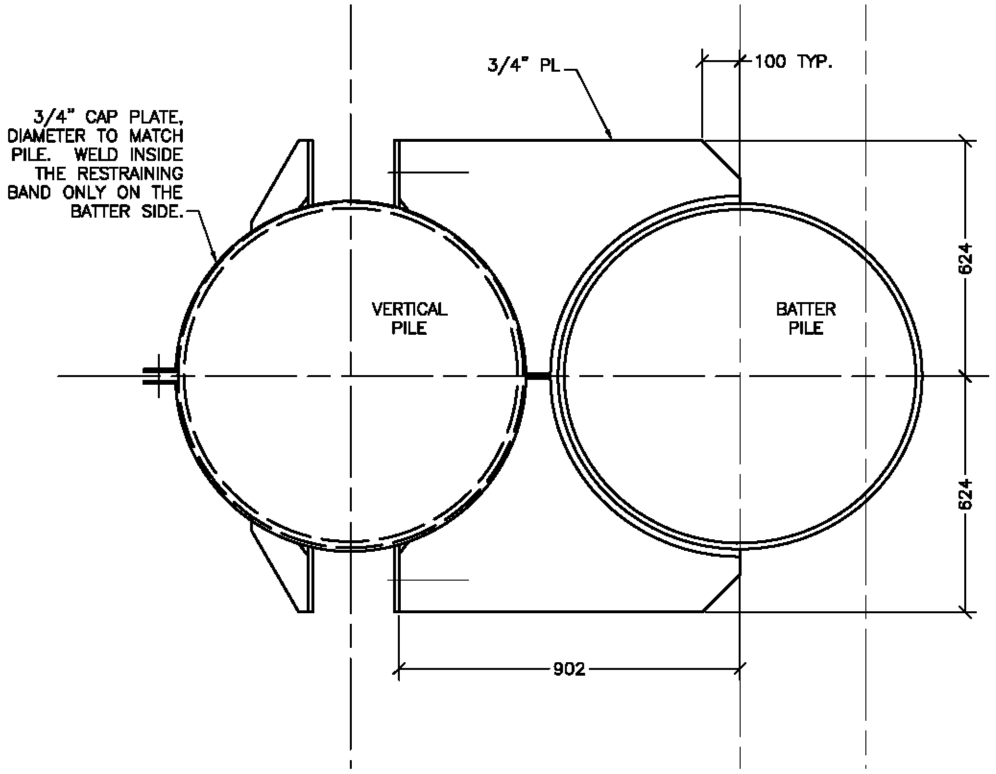
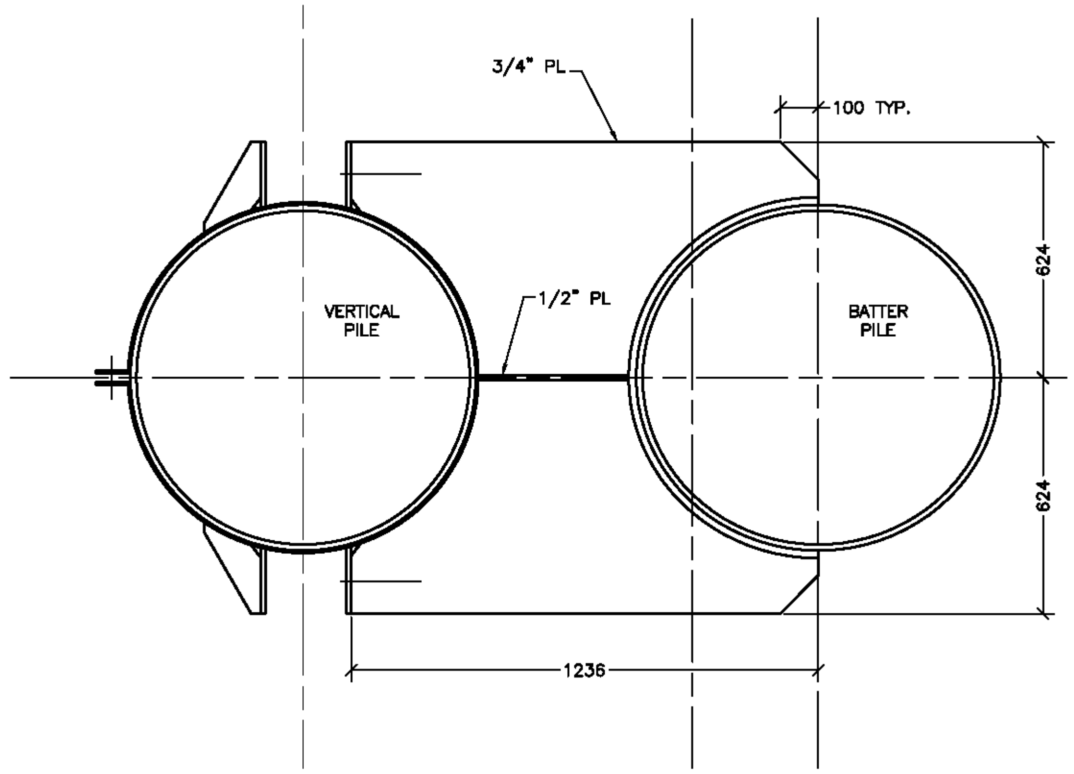
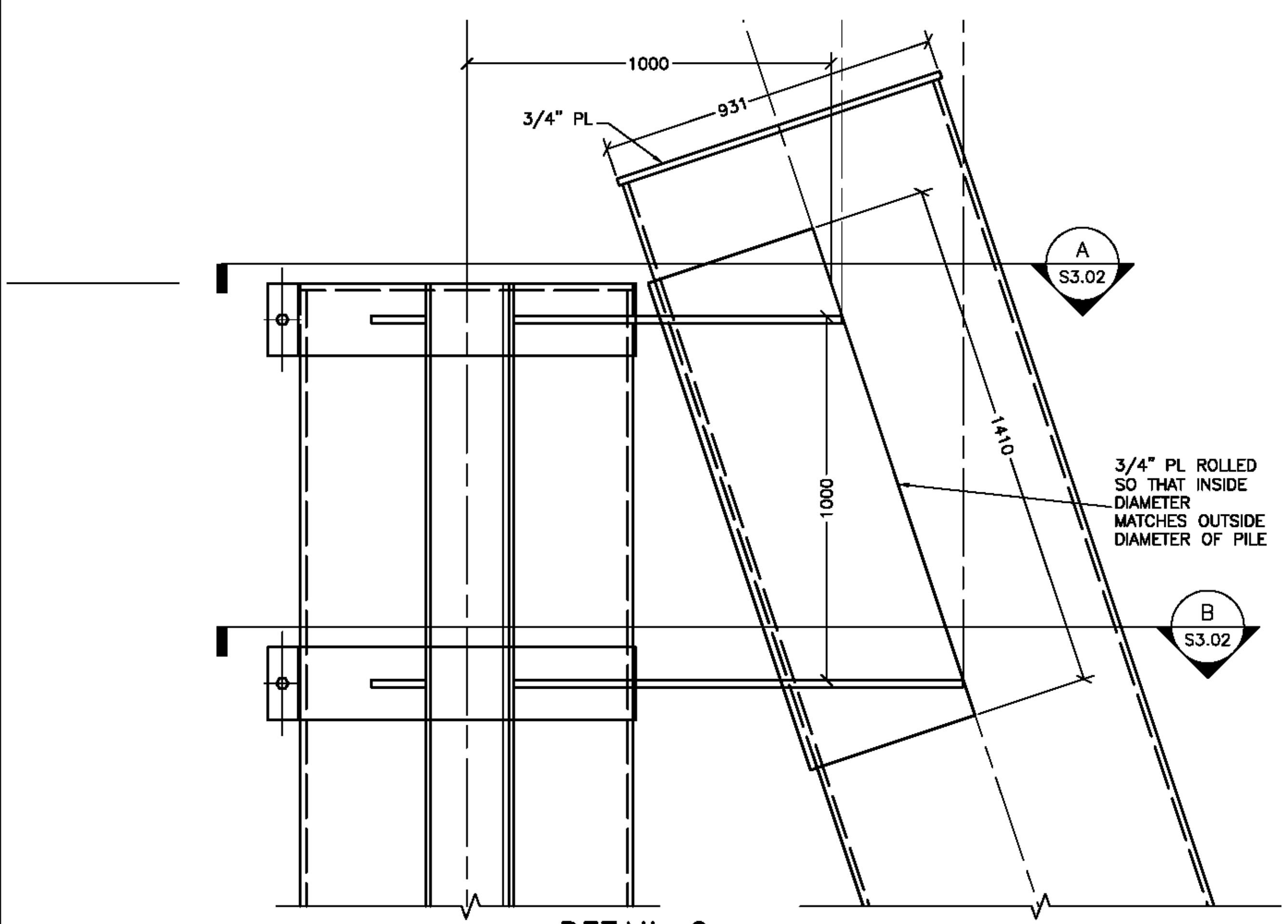


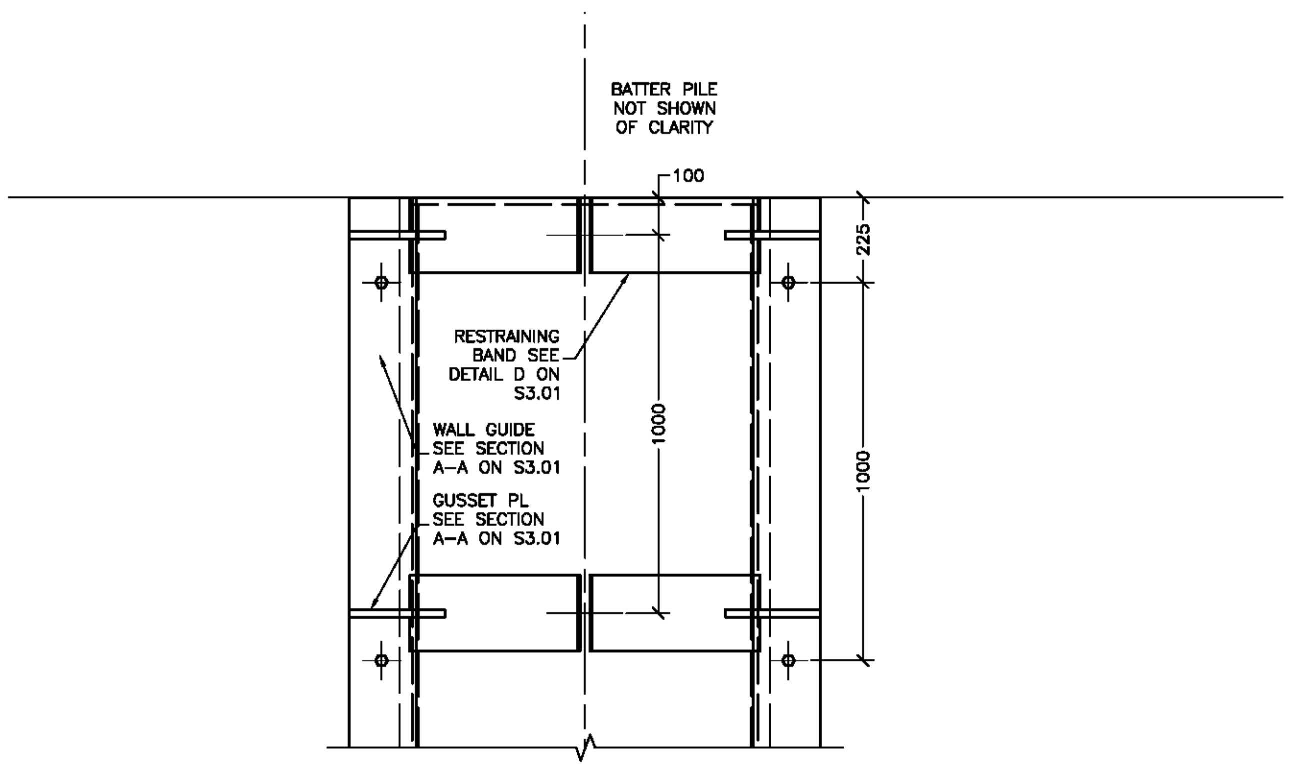
SECTION A–A
SCALE: 1:10



SECTION B–B
SCALE: 1:10



DETAIL C
SCALE: 1:10



DETAIL D
SCALE: 1:10

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PRELIM

US ARMY CORPS OF ENGINEERS
ALASKA DISTRICT

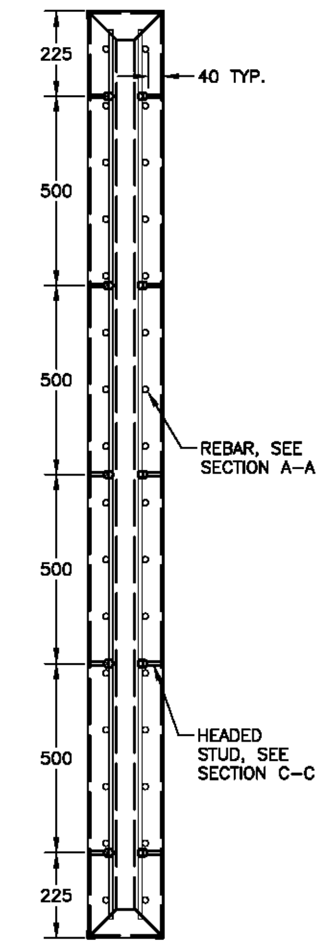
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

Designed: RES
Drawn: RES
Date: 25JAN05
Dwg Scale: AS NOTED
Plot Scale: 1:2
File: VAL002-09-S3-02
Drawing #: unknown

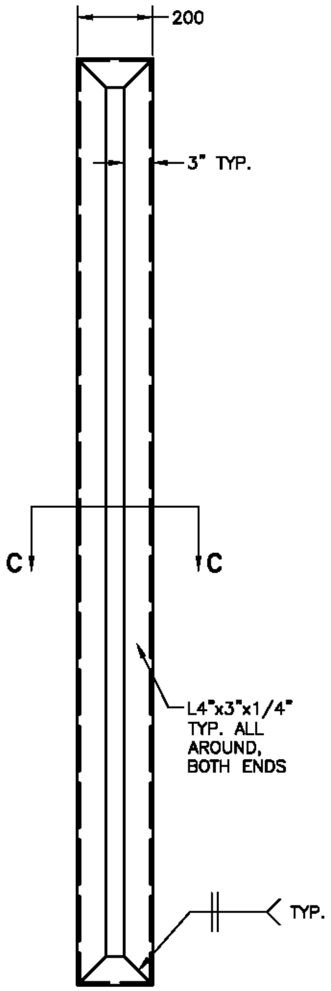
PRELIMINARY CONCEPTUAL PLANS VAL002
CWIS 010600

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
STRUCTURAL
DETAILS
PILE TO PILE CONNECTION

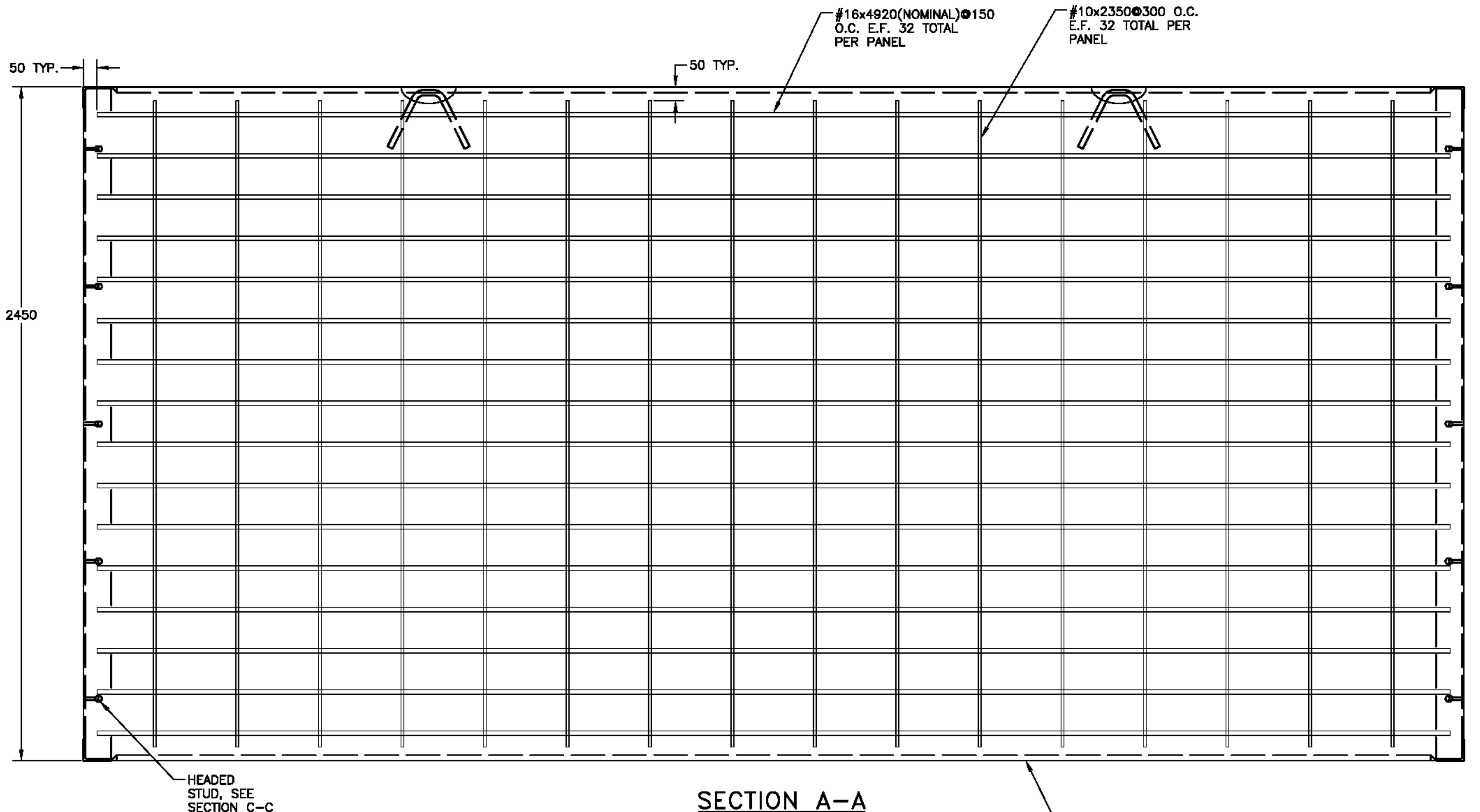
Reference number:
S3.02
Sheet 9 of 10



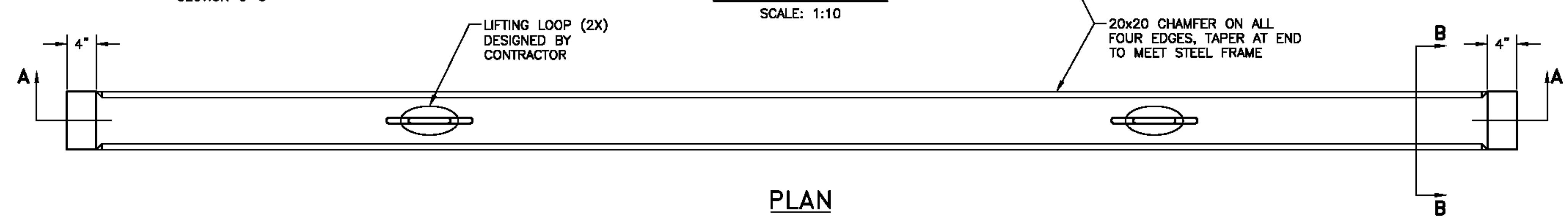
SECTION B–B
SCALE: 1:10



ELEVATION
SCALE: 1:10



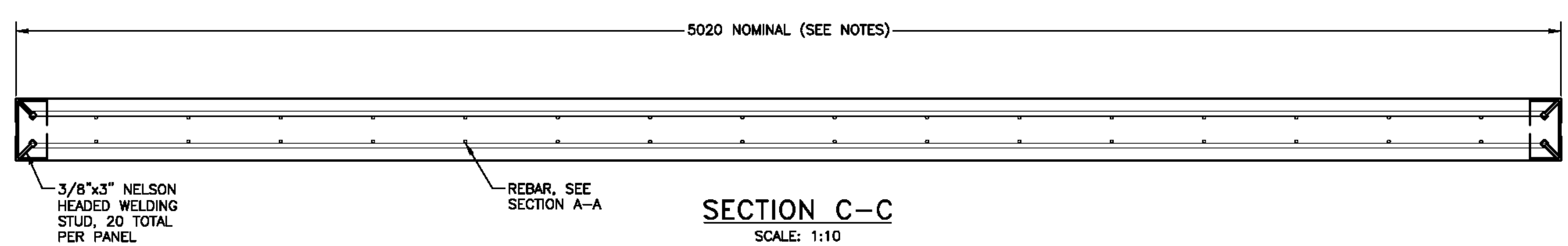
SECTION A–A
SCALE: 1:10



PLAN
SCALE: 1:10

NOTES

1. ALL REBAR SHALL BE GALVANIZED, SEE SPECS
2. END FRAMES SHALL BE GALVANIZED AFTER ALL WELDING IS COMPLETED
3. ALL DIMENSIONS ARE MILLIMETERS EXCEPT WHERE INCHES ARE DESIGNATED WITH ".
4. LIFTING LOOPS TO BE DESIGNED BY THE CONTRACTOR. ENSURE THAT THEY WILL NOT CONFLICT WHEN PANELS ARE STACKED
5. PANEL LENGTH IS NOMINAL. ACTUAL PANEL LENGTH WILL DEPEND ON THE AS-BUILT SPACING OF THE PILES. PANELS CANNOT EXCEED 5.5 METERS.



SECTION C–C
SCALE: 1:10

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PRELIM

US ARMY CORPS OF ENGINEERS
ALASKA DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

Date: 25JAN05
Dwg Scale: AS NOTED
Plot Scale: 1:2
File: VAL002-10-S3-03
Drawing #: unknown

PRELIMINARY CONCEPTUAL PLANS VAL002
CWIS 010600

VALDEZ, ALASKA
NAVIGATION IMPROVEMENTS
STRUCTURAL
DETAILS
WALL PANELS

Reference number:
S3.03
Sheet 10 of 10

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