P | N | D

ENGINEERS, INC.

March 15, 2016 PND No. 102029.12

Julie L. Anderson, P.E.
Operations Branch Chief
Engineering and Construction Operations Division
Alaska District
US Army Corps of Engineers
P.O. Box 6898
JBER, Alaska 99506-0898

Subject: Portage Cove Harbor Expansion Project – Section 408 Permit

Dear Ms. Anderson:

This letter is in response to your January 22, 2016 letter requesting additional information concerning the Section 408 permit for the Portage Cove Harbor Expansion Project. PND and the Haines Borough have investigated the concerns outlined in your letter and believe the information below and enclosed should be sufficient to allow the USACE to proceed with review of the Section 408 permit application. A response to each item in your January 22, 2016 letter is presented below. Included for ease of reference is the original comment in italics:

1. Potential wave transmission through the gap between the wave barrier and crest of the existing breakwater will cause larger waves inside the harbor than designed for. On Sheet 5.04 in the 95% design review submittal, there should not be a gap between the wave barrier and the crest of the existing breakwater.

A design alternative that closes the gap with an extended wave barrier is presented in the enclosed drawings. PND agrees that a design without a gap will reduce the transmitted wave energy. However, it is questionable whether the added cost of approximately $290,000 (excluding armor rock) has sufficient value. Wave numerical modeling and diffraction diagram calculations (enclosed) indicate that the transmitted wave energy is acceptable, based on discussions with the harbormaster and local officials. In addition to the reduced construction cost, the benefits of the gap include improved circulation and fish passage, and reduced armor rock displacement forces near the wave barrier. The Haines Borough would like to discuss this issue further with the USACE and the Alaska DOT&PF before deciding whether to include this added length of wave barrier in the design.

2. There is concern that the armor size on the existing breakwater is not large enough to withstand the reflected wave force from a wave barrier installed through the head of the rubble-mound breakwater and that the armor rock would be pulled off of the breakwater. There is conflicting information on the size of armor rock and the design of the existing breakwater (Ref. 1.1 & 1.2). Based on photos and the design drawings, the armor rock on the existing breakwater appear to be in the 600 lb. range. A field site visit should be performed by the project designers to determine the size of the existing armor rock on the head of the breakwater.

The size and gradation of the existing armor rock was measured during a recent site visit by PND. The enclosed memo describes the existing conditions at the tip of breakwater. The mean armor rock size is approximately 450 pounds, although the gradation is wide with the measured armor rocks ranging in size from 8875 pounds to 31 pounds. 82% of the armor rocks are less than 1000 pounds. The armor layer appears sound, with no obvious armor rock displacement. Smaller size underlayer or core rock was not visible through the voids in the armor rock, indicating the outer layer is likely at least 2 armor rocks thick.

3. A flume study is recommended to determine the exact rock size necessary to ensure stability of the existing breakwater once the wave barrier is installed through the breakwater. If no flume study is done, at a minimum the armor rock size should be increased to 2500 lbs if 80% of the armor rock at the end of the existing breakwater is less than 1000 lbs and increased to 3500 lbs if 80% of the armor rock is larger than 1000 lbs on the existing breakwater. The replaced section of breakwater should have two layers of armor rock backed by two "B" layers in the W10 range.

The proposed wave barrier extension and rubble mound breakwater armor rock design is shown on the attached drawings. The design includes a median armor rock size of W50= 2,500 pounds. The underlayer rock size is 250 pounds.

PND agrees that a flume study (hydraulic model) would provide the best information on the size of armor rock required to armor the tip of a breakwater that includes a vertical wave barrier. However, a flume study is not planned due to cost and schedule considerations.

4. Please submit cross section drawings for review that show the redesigned two layers of armor rock at 2500 lbs or 3500 lbs (based on site investigation), two layers of "B" rock, and core that will fit within the current breakwater neat lines. It must also detail the transition from the existing armor rock configuration to the new armor stone size, starting at a minimum of 25 feet from the wave barrier on the Portage Cove side of the breakwater.

The attached drawings include plan, cross-section and details of the proposed design. The typical section shows a conventional 3 layer slope protection design, with armor rock, underlayer rock ("B" rock), and core rock.

5. The project designers should evaluate potentially adverse impacts the wave barrier wall construction could have on stability of the breakwater in view of the sensitive foundation soil conditions present. The design should include the following details and analyses.

Attached is a technical memo presenting the slope stability analysis.

5.1 Provide a detailed construction sequence for the planned wave barrier wall connection to the existing breakwater, involving excavation within the breakwater, installation of piles and barrier wall elements, and reconstruction of the breakwater nose.

Attached is a drawing showing the recommended construction sequence. The existing armor rock and underlayer rock will be removed within the limits indicated and replaced with larger rock. The new rock will fit within the existing breakwater neat lines. The total volume, and weight, of rock at the existing breakwater will not change significantly.

5.2 Provide a construction monitoring plan that would track and document continued stability of the breakwater and achievement of the required breakwater elevations and grades.

Construction monitoring will include repeated surveys of excavation and rock placement during construction. The survey standards, required submittals, and other details are described in notes included in the attached drawings.

5.3 Perform a slope stability analysis to evaluate post-construction stability of the breakwater, considering pile driving and reconstruction of the breakwater nose. Post construction condition of the breakwater should be at least a factor of safety of 1.3 against slope failure as determined in accordance with Corps of Engineers criteria.

As discussed in the enclosed memo, the added wave barrier will improve the stability of the existing rock breakwater. Post-construction the tip of the rock breakwater will have a factor of safety greater than 1.3. Some distance from the new wave barrier, the existing rock breakwater factor of safety will remain unchanged post-construction.

5.4 This office assumes that reconstruction of the breakwater will retain the existing elevation and cross-section configuration, placing no additional load on the underlying very soft clay foundation. If that is not the case, the slope stability analysis should reflect the expected increased loading from breakwater reconstruction and any localized weakening of the underlying clay stratum.

The proposed design for reconstructing the end of the breakwater retains the existing elevation and cross section configuration and no increased foundation soil loading will occur.

Although not part of the scope of this review, the Alaska District is hopeful that a wave analysis has been performed to assess the potential adverse impacts of reflected and diffracted waves from the wave barrier on the Port Chilkoot Dock.

There is no adverse wave impact at Chilkoot Dock or the entrance channel. PND performed this analysis previously, and the results were verified for the 95% design wave barrier alignment using the numerical model MIKE 21-BW as part of the work for this task. The results are included in the attached memo.

After you have a chance to review please let us know your further thoughts, concerns and recommendations. Feel free to call me if you have any questions at 907.586.2093, or by email at dsomerville@pndengineers.com.

Sincerely,
PND Engineers, Inc. | Juneau Office

Dick Somerville, P.E.
Vice President

Enclosures:

i. Technical memo presenting the wave numerical model analysis and diffraction diagram analysis (18 pages)
ii. Technical memo describing the existing rubble mound breakwater armor rock condition based on a site visit February 12, 2016 (7 pages)
iii. Design drawings illustrating the wave barrier extension, armor rock placement on existing breakwater and construction monitoring program (8 each, 11x17 sheets)
iv. Drawing illustrating suggested construction sequence (1 each, 11x17 sheets)
v. Drawing illustrating proposed Navigational Channel (1 each, 11x17 sheets)
vi. Technical memo presenting the slope stability analysis (4 pages)
vii. Construction cost estimate for added length of wave barrier at the tip of the existing rock breakwater (1 page)

cc: Shawn Bell, Haines Harbormaster
Brad Ryan, Haines Borough Manager
Randy Vigil, USACE Alaska District Juneau Regulatory Branch

P | N | D
ENGINEERS, INC.

MEMORANDUM

To: Dick Somerville

Date: March 4, 2016
Project No: 102029.12

From: Ajay Sampath and Nels Sultan

Subject: Portage Cove Harbor Expansion – Section 408 Permit Wave Analysis

This memo summarizes PND's analysis of waves for the Portage Cove Harbor Expansion Project, to address questions that are part of the Section 408 Permit application. PND applied the wave numerical model MIKE 21-BW and diffraction diagrams in the Shore Protection Manual. The key questions addressed by the analysis are the following:

i. Is wave transmission through a gap between the planned vertical wave barrier and existing rock breakwater acceptable? or should the gap be closed?
ii. How are wave heights and armor rock stability affected by the presence of a wave barrier near the tip of the existing rock breakwater?
iii. Are wave heights at the tip of the rock breakwater higher or lower if a gap is present? and is the armor rock at the tip more or less stable if a gap is present?
iv. Are wave conditions near the entrance acceptable considering wave reflections from the vertical wave barrier and the proximity to the cruise ship dock to the south?

Two wave barrier alternatives were tested, one with a gap and one without. The first alternative includes the gap and is shown in Figure 1. Alternative 2, with the gap closed is shown in the partial plan in Figure 2, the area at the tip of the existing rock breakwater. The wave barrier alternatives are based on the 95% design drawings and both include the 33 feet length of wave barrier in Additive Alternate C. Model runs were also performed for existing conditions, with the rock breakwater only.

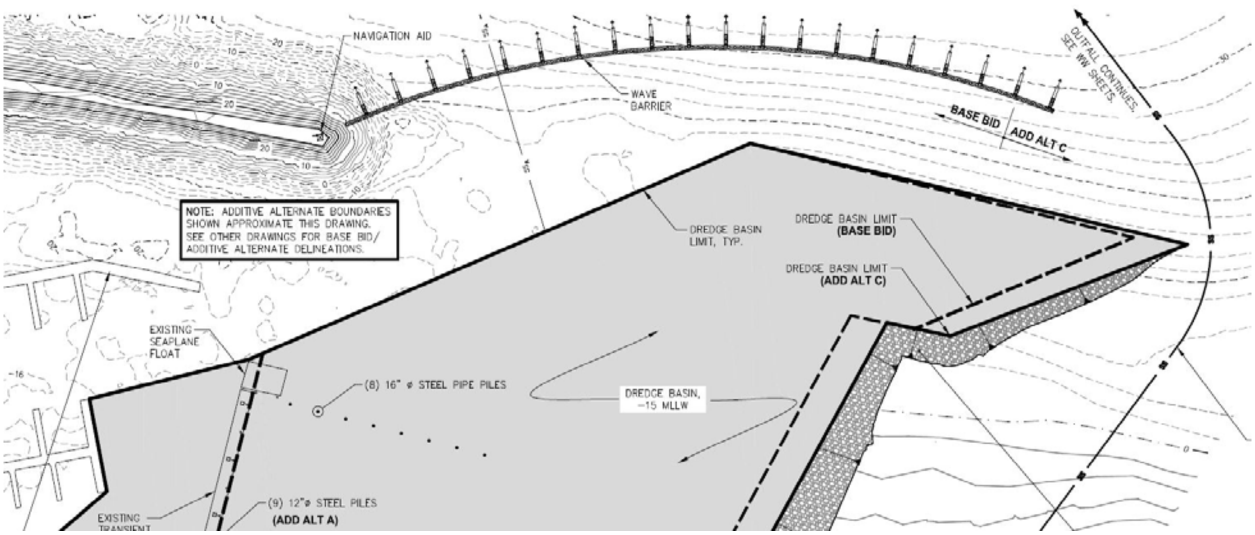


Figure 1. Alternative 1 - PCHE Wave Barrier – 95% Design with Gap (Length 633 feet)

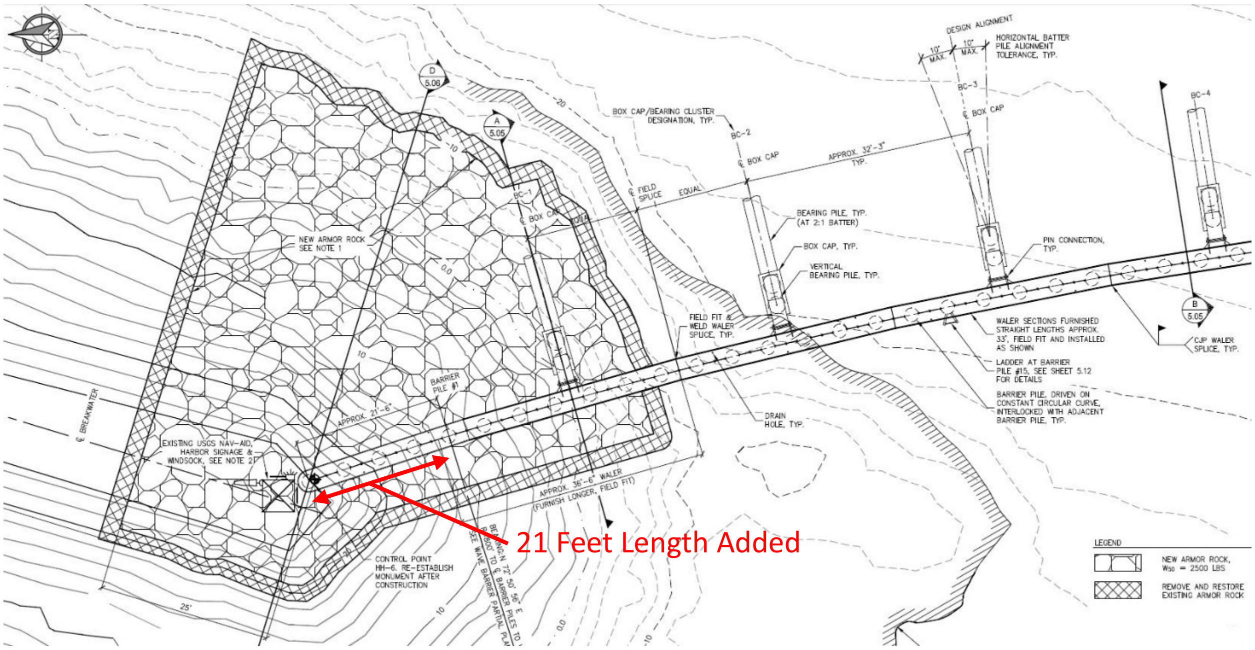


Figure 2. Alternative 2 Partial Plan - PCHE Wave Barrier without Gap (Length 654 feet)

MET-OCEAN DESIGN CRITERIA

Previous met-ocean analysis and numerical modeling of the wave barrier alternatives can be found in the technical memo "Portage Cove Wave Barrier Analysis "(PND, 2014) and the report "Harbor Protection Alternatives" (PND, 2013).

The environmental design criteria from the previous reports by PND are summarized in the tables below for ease of reference. The water elevations listed in Table 1 are from NOAA tide data and tide prediction software. Haines is in a region experiencing a relatively large rate of glacial rebound/uplift. As a result, relative sea level is falling and this should be considered in determining design water levels and dredging depths. In Skagway, the relative sea level is falling at a rate of 5.6 feet per 100 years. However, in Juneau the relative sea level is falling at a rate of 4.2 feet per 100 years. A reasonable assumption for Haines is that the local sea level will fall at a rate in between, approximately 5.0 feet per 100 years. Assuming a project life of 50 years, it may be reasonable to design for water levels 2.5 feet lower than those listed in Table 1.

Table 1. Water Levels and Vertical Datum

	Skagway (feet, MLLW)	Haines (feet, MLLW)	Juneau (feet, MLLW)
Highest Observed Water Level	26.5 (10/22/1945)	-	24.8 (11/2/1948)
Highest Astronomic Tide	21.0	21.1	20.6
Mean Higher High Water (MHHW)	16.7	16.8	16.3
Mean High Water (MHW)	15.7	15.8	15.3
Mean Tide Level (MTL)	8.7	-	8.5
Mean Low Water (MLW)	1.6	-	1.6
Mean Lower Low Water (MLLW)	0.0	0.0	0.0
Lowest Astronomic Tide	-5.1	-4.8	-4.8
Lowest Observed Water Level	-6.5 (12/14/2008)		-5.9 (12/14/2008)
Extreme Low Water (NOAA chart 17317)		-6.0	-

Table 2. Portage Cove – Design Operational Criteria (2-Year Return Period)

Direction	Water Elevation (feet, MLLW)	Wind Speed (knots)	Wave	
			Significant Height (feet)	Peak Period (sec)
Northeast (050°)	+17	31	2.6	2.5
East (090°)		31	2.1	2.2
Southeast (120°)		31	2.5	2.4

Table 3. Portage Cove – Design Operational Criteria (50-Year Return Period)

Direction	Water Elevation (feet, MLLW)	Wind Speed (knots)	Wave	
			Significant Height (feet)	Peak Period (sec)
Northeast (050°)	+20	68	6.5	4.3
East (090°)		68	6.9	4.4
Southeast (120°)		68	6.3	4.3

WAVE DIFFRACTION CALCULATIONS

Wave transmission through a gap between the wave barrier and existing rock breakwater was analyzed using diffraction diagrams from the Shore Protection Manual (USACE, 1984). Diffraction was also analyzed with the wave numerical model MIKE 21-BW, discussed in the following section.

Two conditions were analyzed, a 50 year return period wave (Hs=6.9 feet, Tp=4.4 seconds) and a 2 year return period wave (Hs=2.6 feet, Tp=2.5 seconds). A high water level of +21 feet, MLLW and waves perpendicular to the gap were assumed, as conservative assumptions. At a water level of +21 feet, MLLW, the gap width at the stillwater line would be 16 feet. The diffraction diagram that is the best fit to the 2 year incident waves and gap width is Figure 2-43 from the Shore Protection Manual. This diagram was scaled and overlaid on the harbor plan. The results for the 2 year return period incident wave height are shown in Figure 3. The results for the 50 year incident wave are shown in Figure 4. For the 50 year wave the transmitted wave height results are extrapolated from diffraction diagrams for larger gap widths, included in Appendix A. The line corresponding to the limit of acceptable wave height for good wave conditions is shown on each figure. The hatched area is the portion of the harbor that exceeds the acceptable wave height. The assumed criteria for good wave conditions are listed in Table 4.

The diffraction diagram gap width is 17 feet for the 2 year return period wave, approximately the same as the 16 feet width in the 95% design. The area inside the harbor where waves exceed 1 feet is relatively small, approximately 70 feet long and 50 feet wide (Figure 3). For the 50 year wave (Figure 4) the area inside the harbor where waves exceed 2 feet is also relatively small, approximately 100 feet long and 100 feet wide.

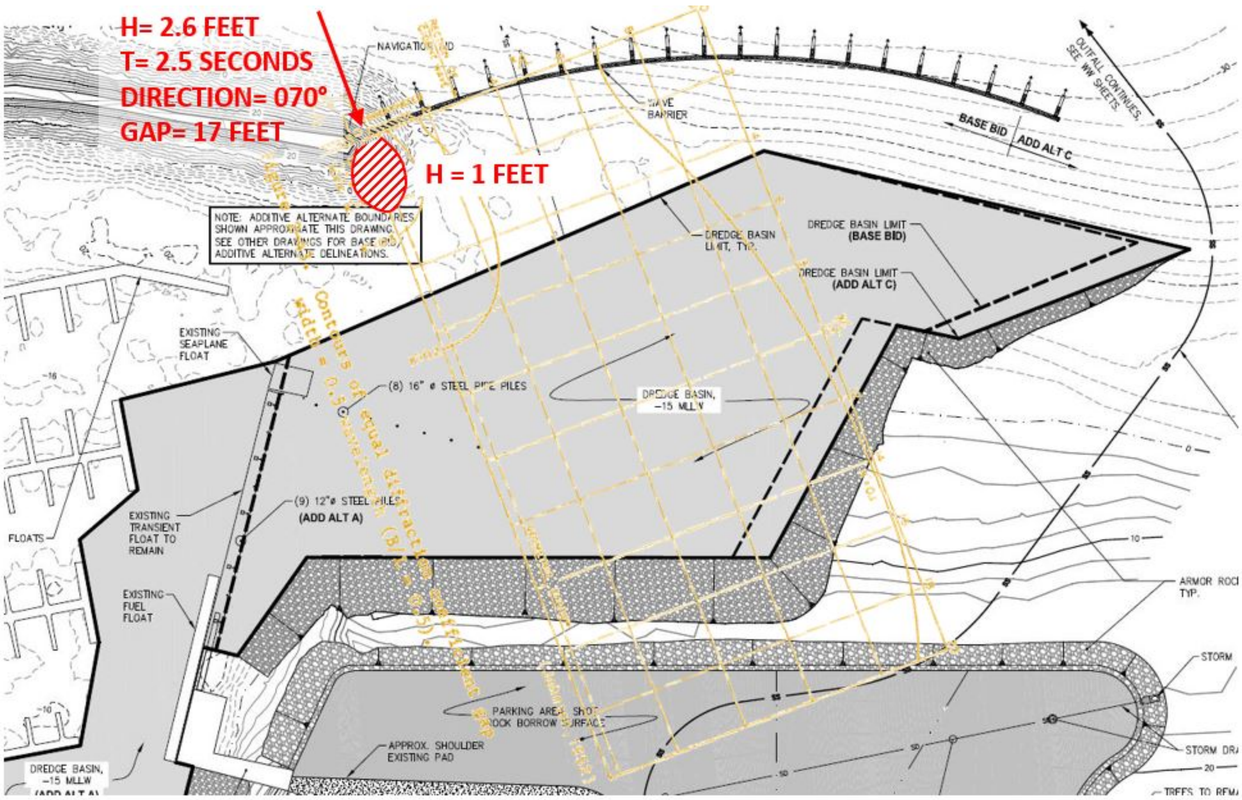


Figure 3. Wave Transmission at Gap - Diffraction Diagram Analysis – 2 Year Return Period Wave

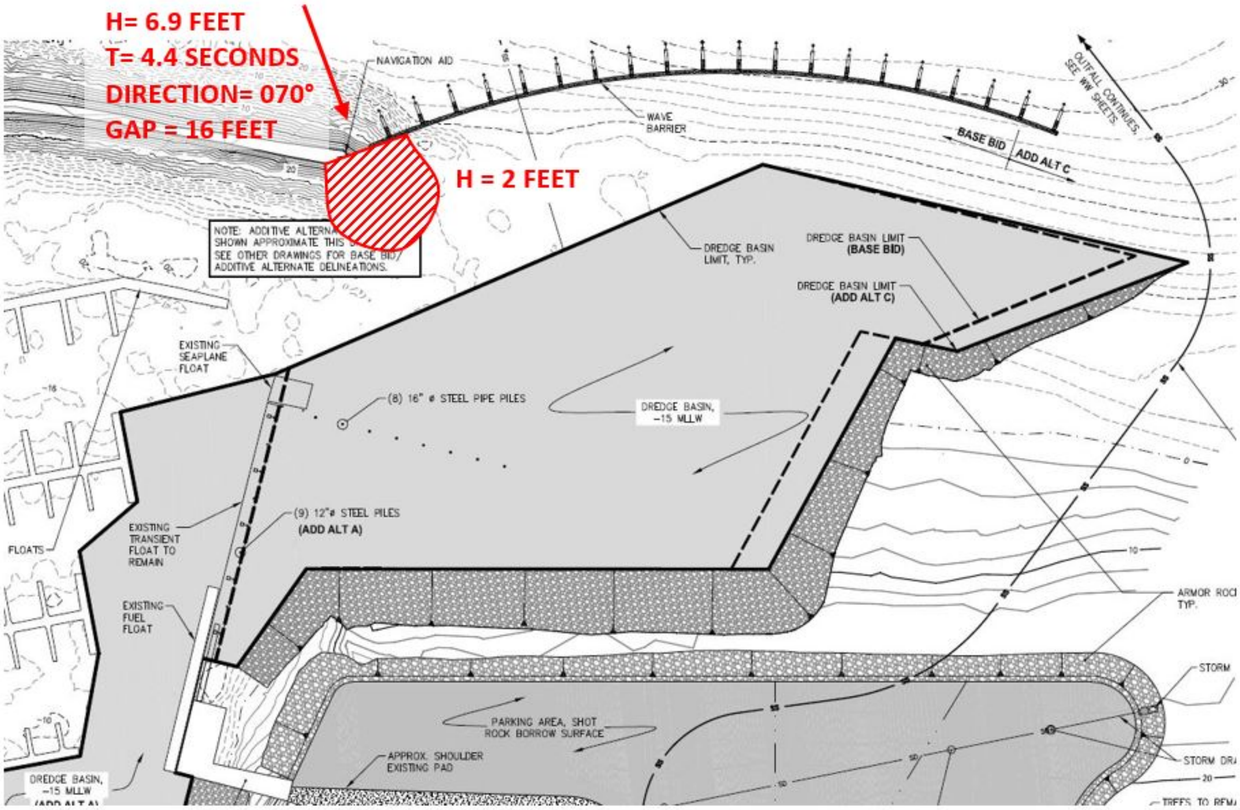


Figure 4. Wave Transmission at Gap - Diffraction Diagram Analysis – 50 Year Return Period Wave

Table 4. Criteria for "Good" Wave Conditions inside a Small Boat Harbor

Design Wave		Return Period		
Direction	**Peak Period**	**50 Year**	**1 Year**	**1 Week**
Head Seas	< 2 seconds	not applicable	<1 feet wave height	<1 feet wave height
	2 to 6 seconds	<2 feet wave height	<1 feet wave height	<0.5 feet wave height
	>6 seconds	<2 feet wave height	<1 feet wave height	<0.5 feet wave height
Beam Seas	< 2 seconds	not applicable	<1 feet wave height	<1 feet wave height
	2 to 6 seconds	<0.75 feet wave height	<0.5 feet wave height	<0.25 feet wave height
	>6 seconds	<0.75 feet wave height	<0.5 feet wave height	<0.25 feet wave height

[1] Reference: Small Craft Harbor Criteria, Canada Department of Fisheries and Oceans, Small Craft Harbors Branch.
[2] For "excellent" wave climate multiply by 0.75, for "moderate" wave climate multiply by 1.25.
[3] "Head seas are waves that approach from the bow or stern of the boat. "Beam seas" approach from the side.

WAVE NUMERICAL MODEL

The MIKE 21 Boussinesq wave (BW) module was used to analyze wave penetration inside the harbor. MIKE 21 BW is a state-of-the-art numerical (computer) model for analyzing wave disturbance in ports, harbors, and coastal areas. The model is capable of reproducing the combined effects of wave phenomena relating to wave penetration, including shoaling, diffraction, wave breaking, and bottom friction.

The met-ocean conditions tested are summarized in Table 5. The model runs varied the wave heights, and direction. The wave period is sensitive to the grid resolution in the BW model, a smaller wave period will require a highly refined grid and increase computation time. A constant 4.5 second period was used to maintain a uniform grid for all the simulations to keep the computation times reasonable and avoid blow-up (instability) in the model. The wave conditions tested are 2, 5, 10 and 50 year return period events. The water level for all model runs was assumed +21 feet, MLLW.

Table 5. MIKE 21 Wave Input - Summary

No.	Direction	Return Period (years)	Wave	
			Significant Height (feet)	Peak Period (sec)
1	Northeast (050°)	2	3.2	4.5
2		5	4.1	4.5
3		10	4.9	4.5
4		50	6.5	4.5
5	East (090°)	2	2.6	4.5
6		5	3.4	4.5
7		10	4.1	4.5
8		50	6.6	4.5
9	Southeast (115°)	2	3.1	4.5
10		5	3.9	4.5
11		10	4.7	4.5
12		50	6.9	4.5

Notes: Water Elevation +21 feet, MLLW for all runs

Model Set-up

The computational domain, shown in Figure 5, is a rectangle with waves generated at the eastern boundary. Waves were generated at a wave generation line approximately 2,600 feet from the wave barrier. The grid spacing is 2 m x 2 m in both the x and y direction and a time step of 0.1 s was used. A minimum water depth of 4.5 m was applied to eliminate wave run-up and breaking at the shoreline and reduce model run time. The shoreline was modeled as fully absorbing and the wave barrier was assigned a reflection coefficient of 0.9. A wave barrier reflection coefficient of 0.9 was selected to reduce the buildup of unrealistic wave energy in the model domain. The numerical model can be considered a "digital wave basin" with output similar to those in a hydraulic model.

The bathymetry used for the numerical model was obtained from a hydrographic survey completed by PND and David Evans and Associates in 2013. The model bathymetry inside the harbor includes the proposed dredging depths, resulting in deeper water depths than existing conditions, which allows greater wave heights in and near the harbor than the existing depths.

A JONSWAP type spectral input was applied at the model boundary. Waves from the southeast, east and northeast were tested in the model. The model simulations were run for 40 minutes. A summary of the model set-up is in Table 6.

Figure 5. MIKE 21-BW Numerical Model Domain

Table 6. MIKE 21 Numerical Model Input Conditions

Input Conditions	Remarks
Bathymetry	Minimum water depth = 4.5 m, reflects proposed dredge depth inside the harbor
Grid Spacing	2 m x 2 m to simulate peak period of 4.5 s and to reduce simulation times
Structures	Rock breakwater and shoreline modeled as absorbing boundaries to reduce wave energy build-up inside the model domain. Planned vertical wave barrier assigned a reflection coefficient of 0.9.
Wave Input	JONSWAP spectrum with Tp=4.5 seconds applied at the boundary.
Wave Direction	Southeast, East and Northeast
Simulation Time	40 minutes to allow for adequate wave energy to build inside the harbor

Results

Wave height outputs were obtained along two lines, shown in Figure 6. The distances in feet along each line are shown. The results are discussed and comparison plots of the significant wave height along each line for different test runs are included in subsequent sections below.

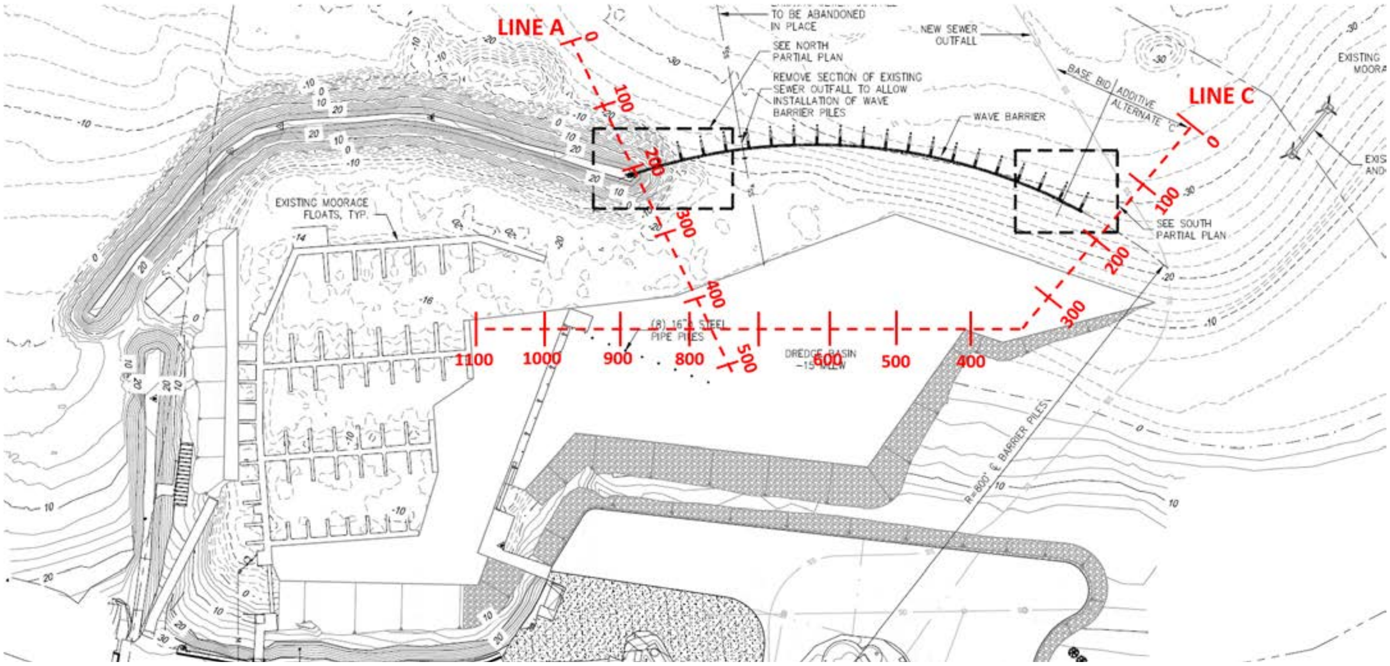


Figure 6. Wave Height Output Locations

The significant wave height statistics were averaged at 1 minute intervals during the simulations. The wave height at the end of the 40-minute simulation (cumulative statistics) was used for comparing the two wave barrier alternatives. The assumed design criteria for wave conditions inside a small boat harbor are listed in Table 4.

Wave transmission through the gap

Wave heights along "Line A" shown in Figure 6 are compared from Figure 7 through Figure 10 for waves from the east and northeast. The figures compare the wave heights for a 2-year and 50-year return period input condition for the wave barrier alternatives with and without the gap. It is evident from the figures that some wave energy is propagated through the gap between the wave barrier and rock breakwater as expected. The area inside the harbor where the wave heights are unacceptable have also been analyzed using diffraction diagrams in the previous section (Figure 3 and Figure 4) and are comparable to the numerical model results as indicated in Figure 7 and Figure 8. Wave heights meet the required design criteria in the harbor with the exception of a relatively small area near the gap. The waves are acceptable where floats and harbor facilities are planned to be located. Screenshots from the animations comparing the sea surface elevation for the two alternatives are shown in Figure 11. There is no difference visually in wave heights in the harbor for the two alternatives modeled (with a gap, and without a gap). The animations files are available on request as avi files.

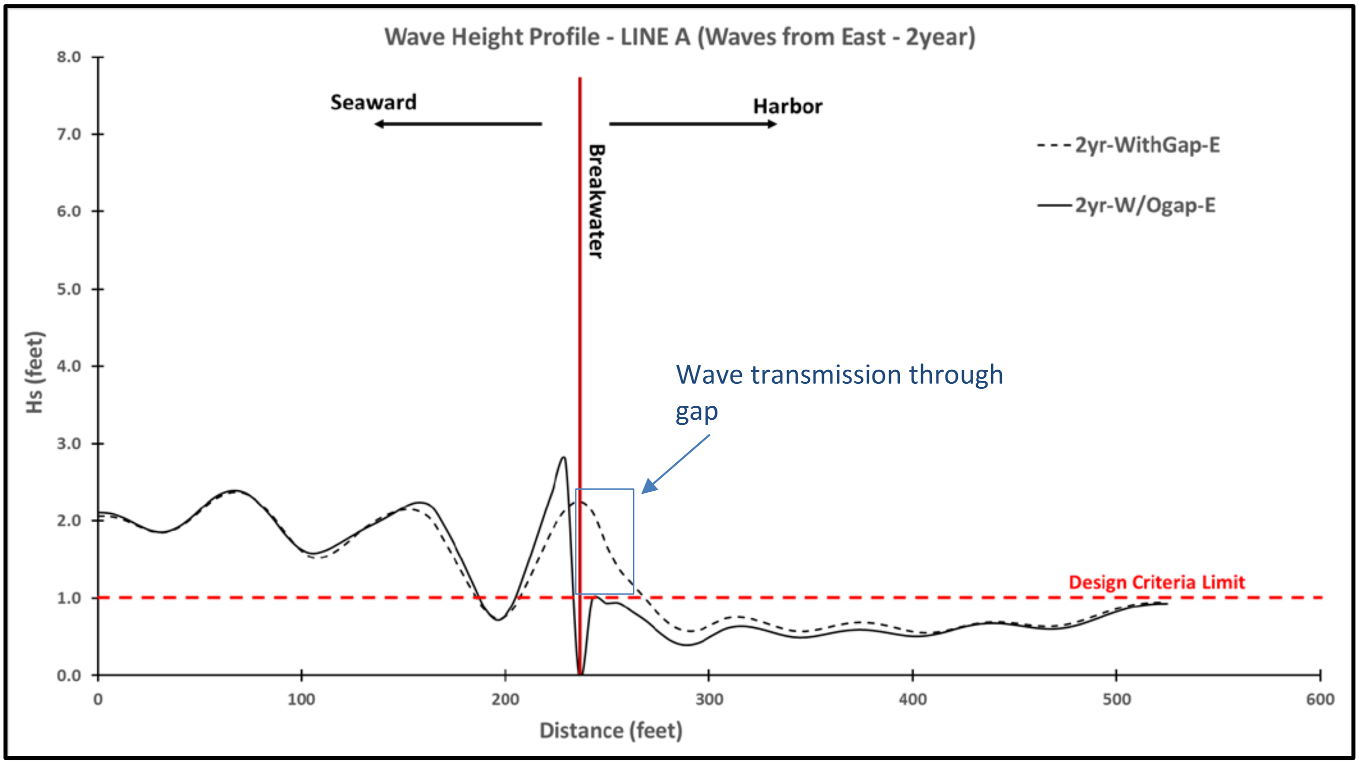


Figure 7. Wave Height Comparison – LINE A - Waves from East – 2 year Return Period (Hs = 2.6 ft, Tp = 4.5s)

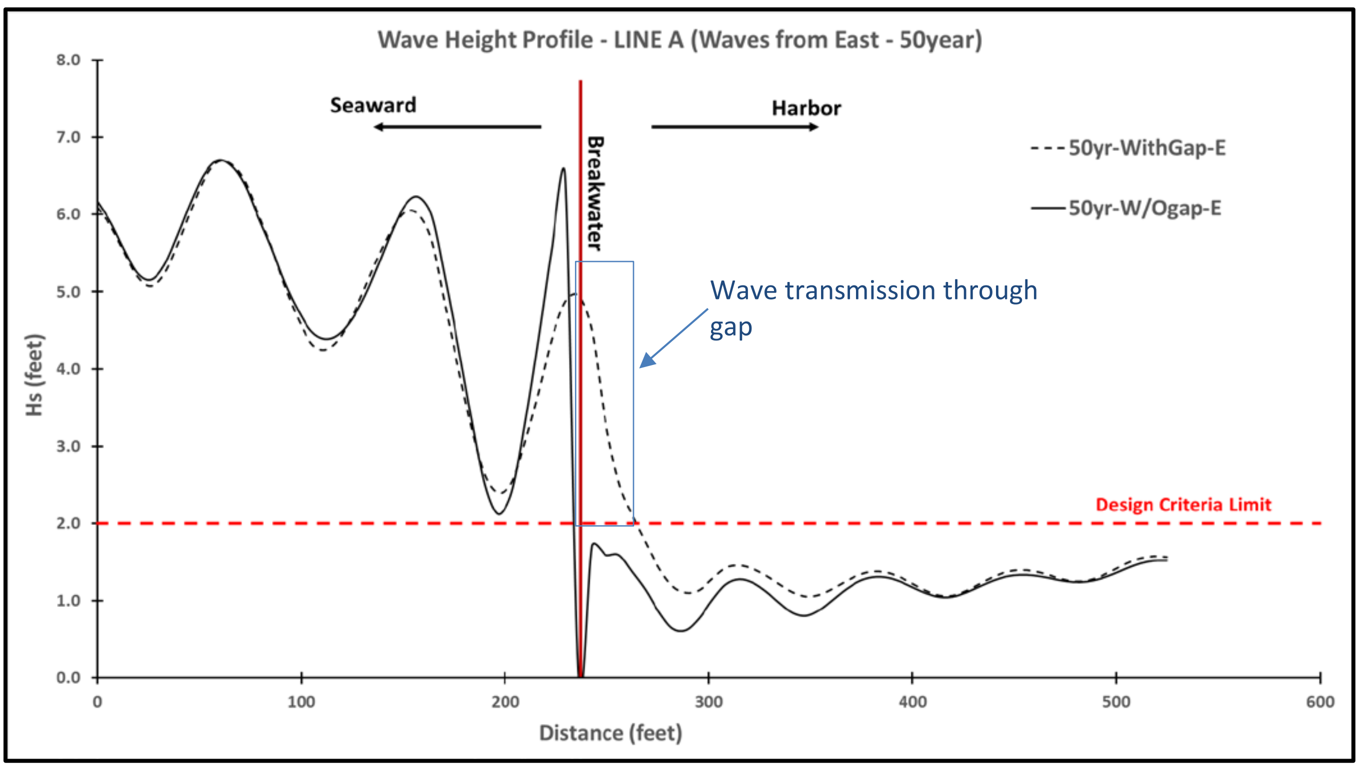


Figure 8. Wave Height Comparison – LINE A - Waves from East – 50 year Return Period (Hs = 6.6 ft, Tp = 4.5s)

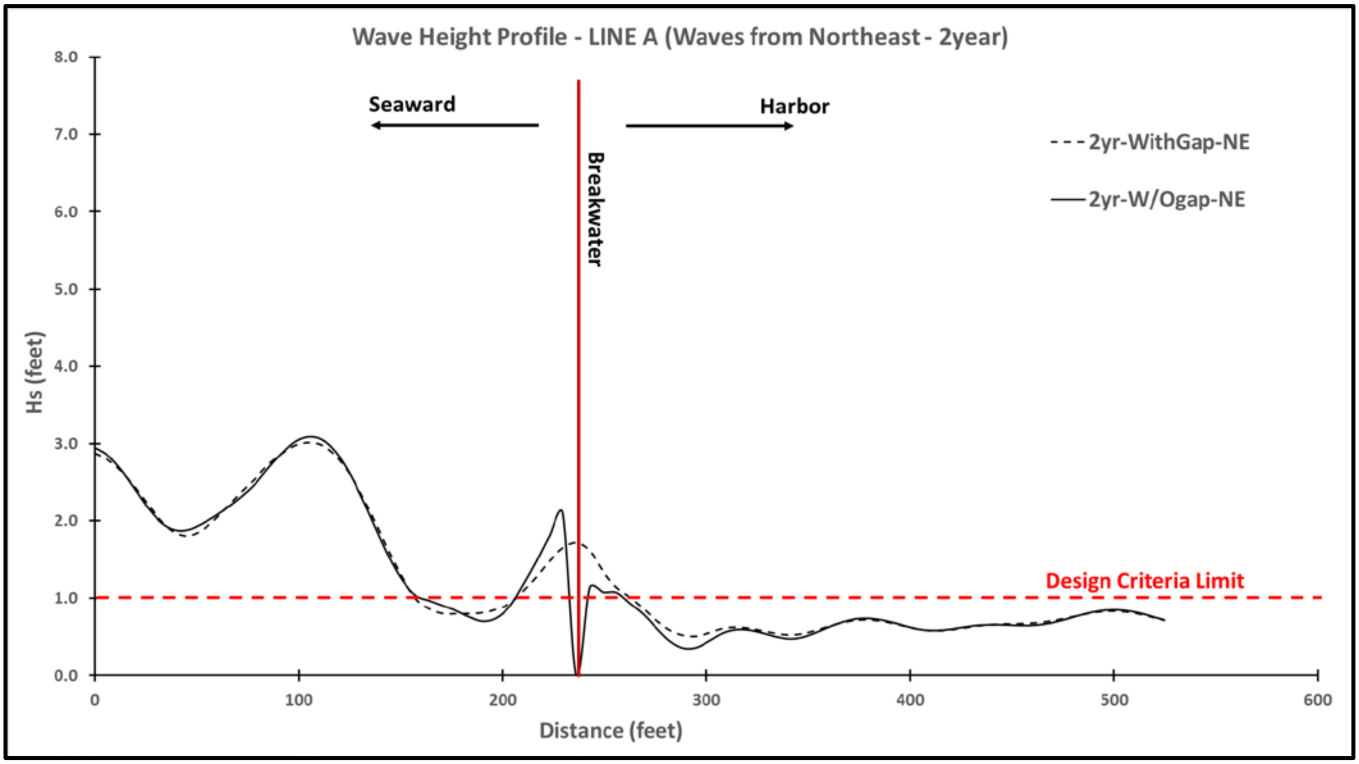


Figure 9. Wave Height Comparison – LINE A - Waves from Northeast – 2 year Return Period (Hs = 3.2 ft, Tp = 4.5s)

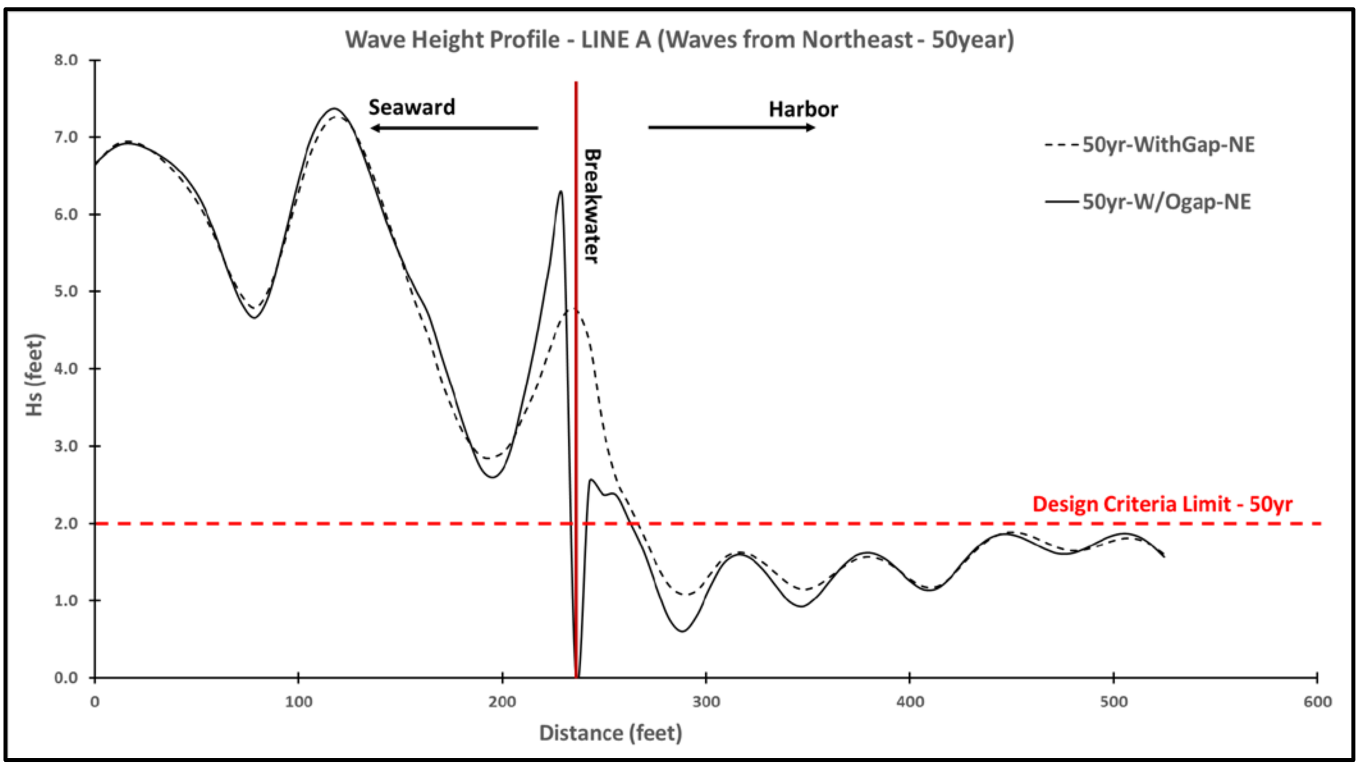


Figure 10. Wave Height Comparison – LINE A - Waves from Northeast – 50 year Return Period (Hs = 6.5 feet, Tp = 4.5 seconds)

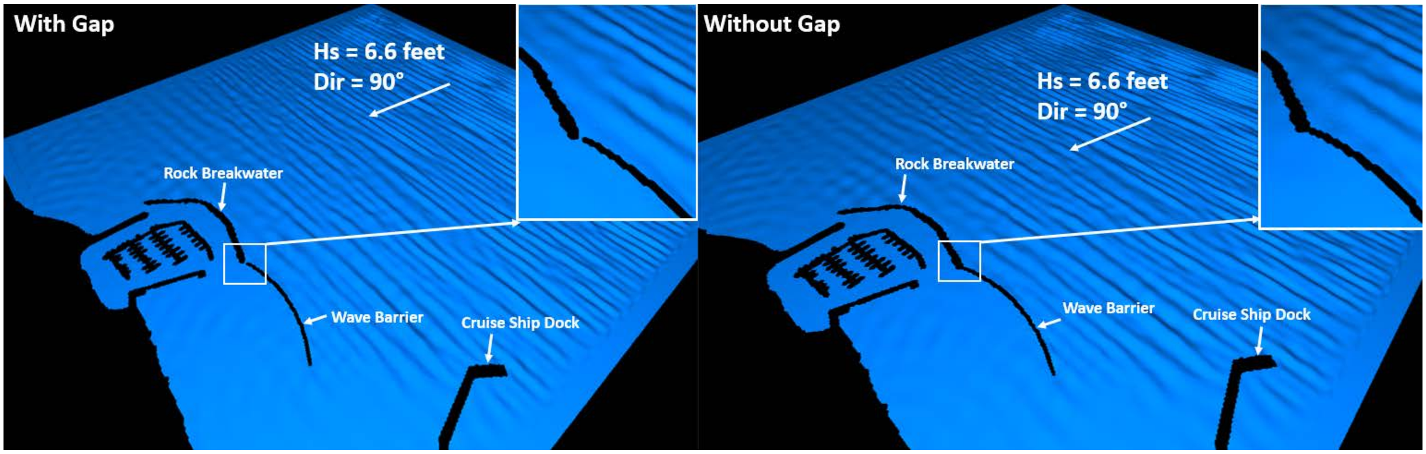


Figure 11. Model Animation Screenshots – Water Surface for Waves from east (Azimuth 090°)

Wave conditions near the entrance

Wave reflection from the wave barrier and the rock breakwater were also analyzed using the numerical model. The primary purpose of the analysis was to determine if reflected waves will be an issue for the vessels coming into the harbor during rough weather conditions. The numerical simulations were run for the 2-year, 5-year, 10-year and 50-year design input conditions, waves from the east. Data was extracted along the two lines shown in Figure 12. Figure 13 through Figure 16 compare the wave height near the rock breakwater and wave barrier. Figure 17 is a screenshot form the animation showing the same for a 50-year return period. The results of the analysis show larger significant wave heights within about 100 feet from the wave barrier. Larger wave heights can also be seen near the rock breakwater due to reflected waves. The increased wave heights due to reflection are larger for the vertical wave barrier.

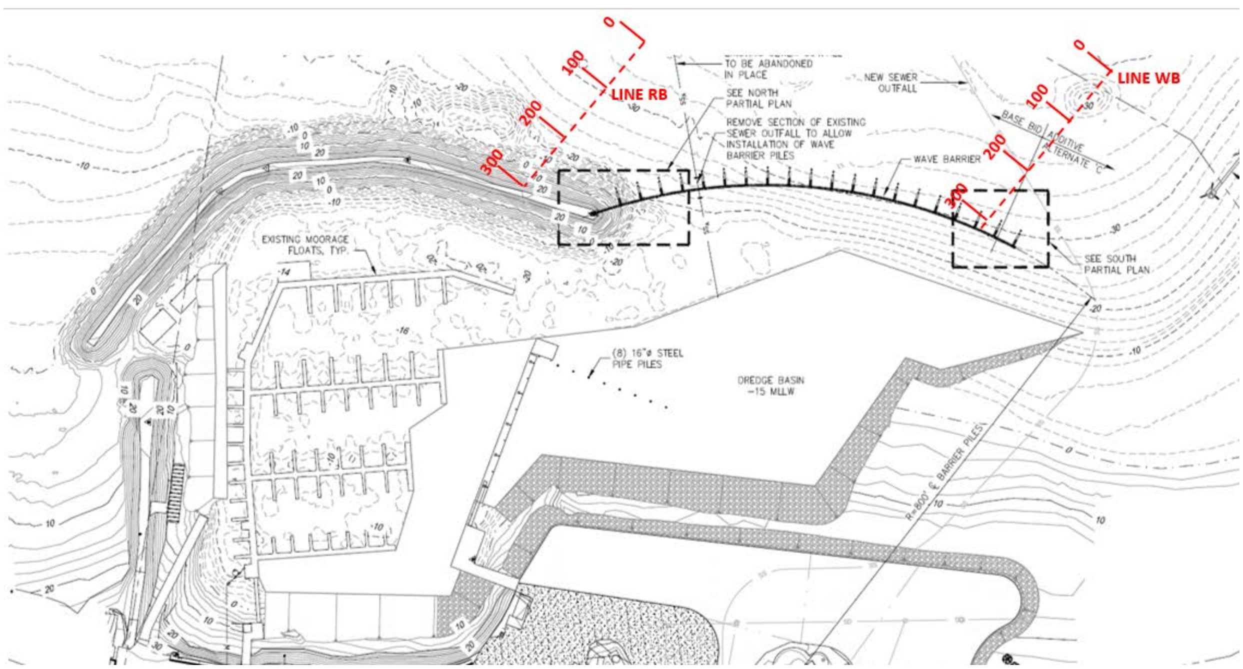


Figure 12. Wave Reflection Analysis – Output Locations

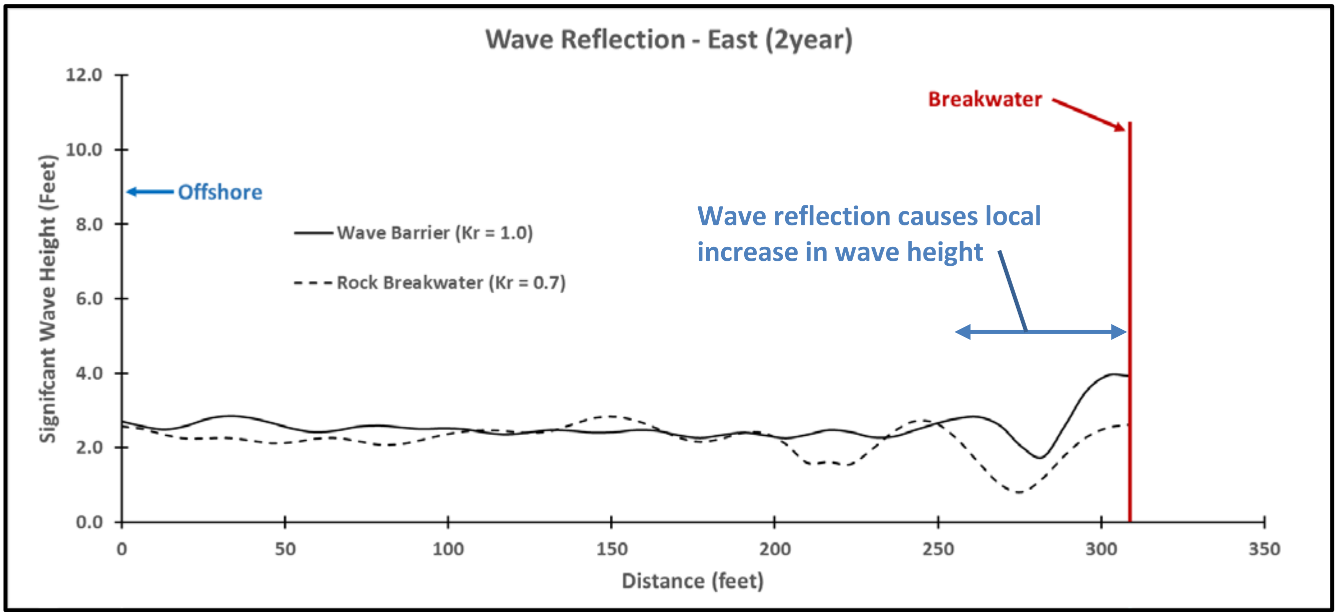


Figure 13. Wave Height Comparison near Tip of Wave Barrier and Rock Breakwater – Waves from East (2-year Return Period)

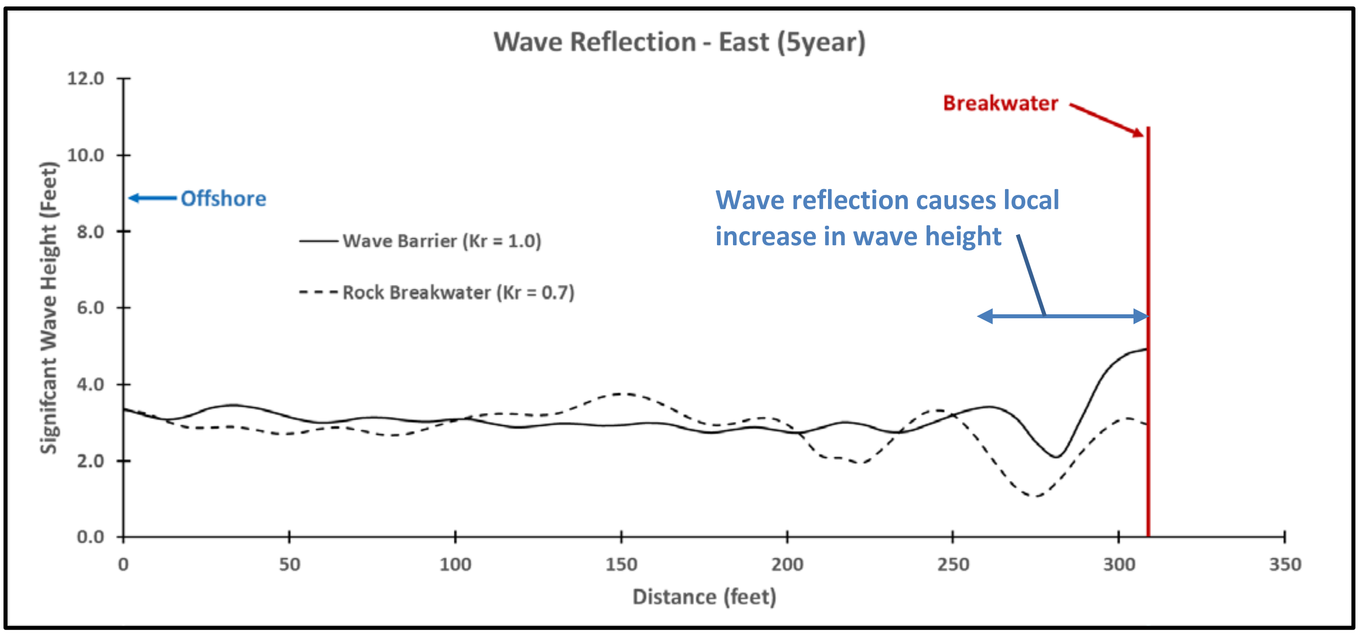


Figure 14. Wave Height Comparison near Tip of Wave Barrier and Rock Breakwater – Waves from East (5-year Return Period)

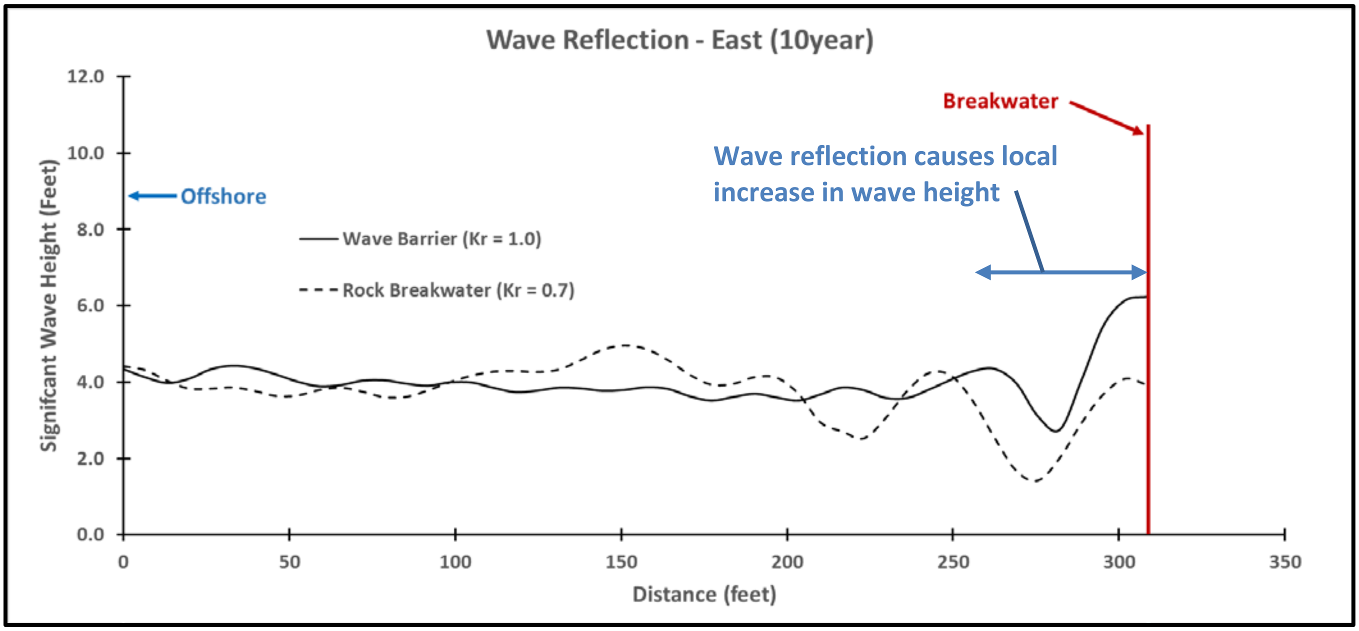


Figure 15. Wave Height Comparison near Tip of Wave Barrier and Rock Breakwater – Waves from East (5-year Return Period)

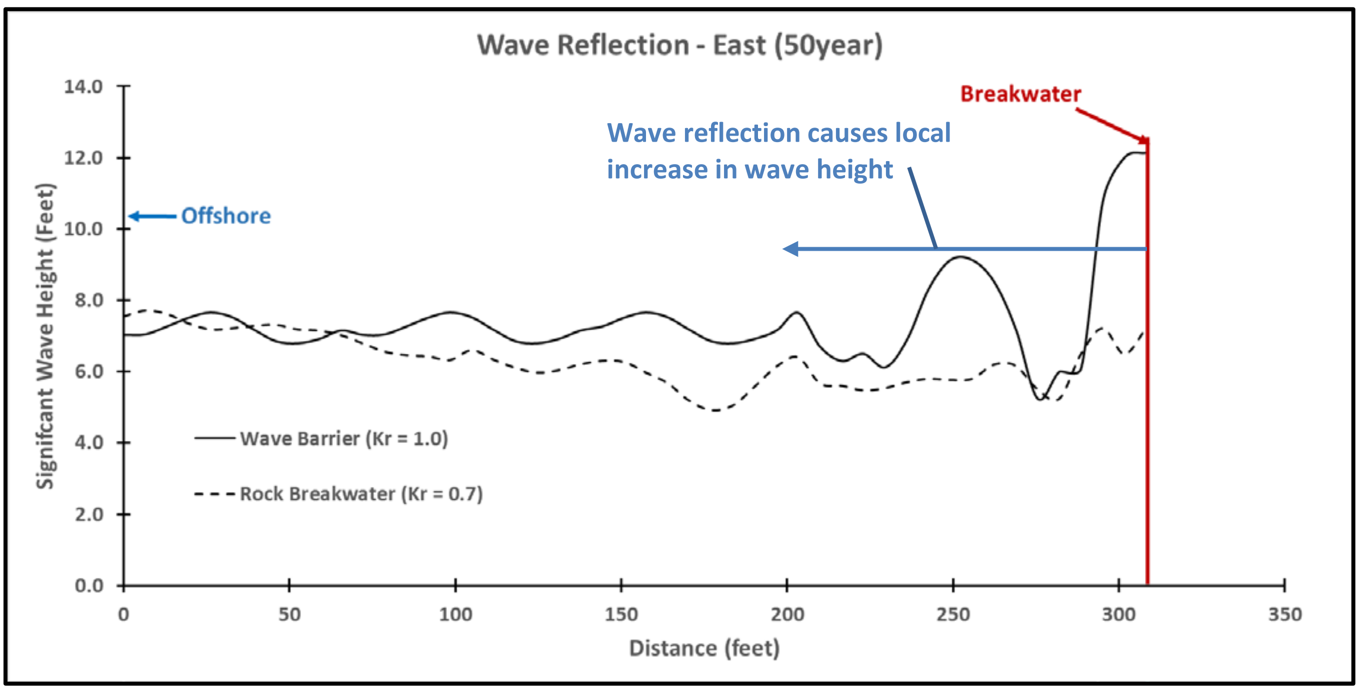


Figure 16. Wave Height Comparison near Tip of Wave Barrier and Rock Breakwater – Waves from East (50-year Return Period)

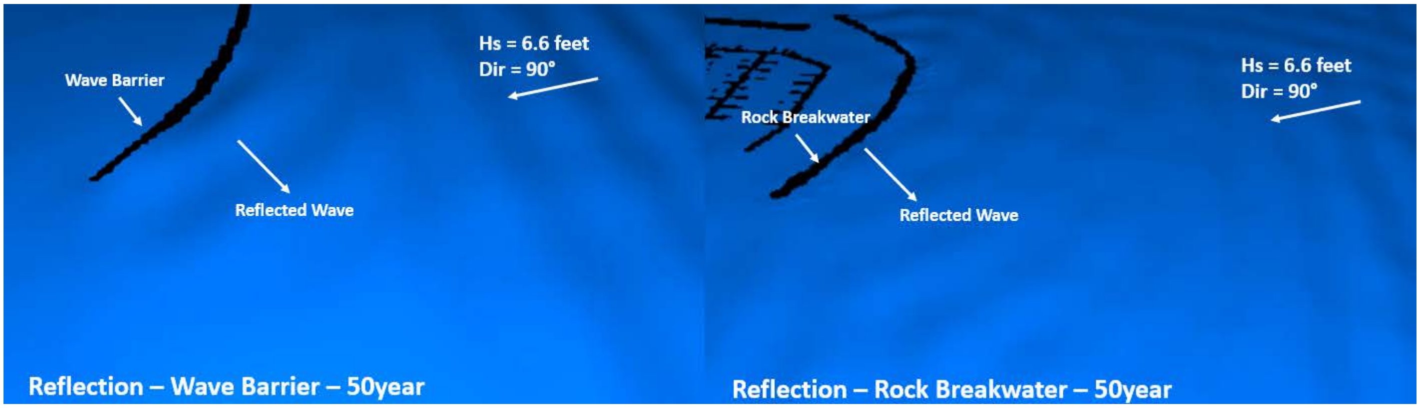


Figure 17. Animation Screenshot - Wave Height Comparison near Tip of Wave Barrier and Rock Breakwater – Waves from East (50-year Return Period)

Wave Height Reduction as Waves Enter Harbor from South

Wave heights along "Line C" shown in Figure 6 are compared from Figure 18 through Figure 21 for waves from the east and southeast. The figures compare the wave heights for a 2-year and 50-year return period input condition for the wave barrier alternatives with and without the gap. The wave heights are within the design criteria limits inside the harbor for the design wave input conditions from the east and the southeast. Figure 22 is from animations of the two model runs, showing the sea surface elevation for waves from the southeast. There is no visual difference in waves inside the marina for the two alternatives (with and without a gap).

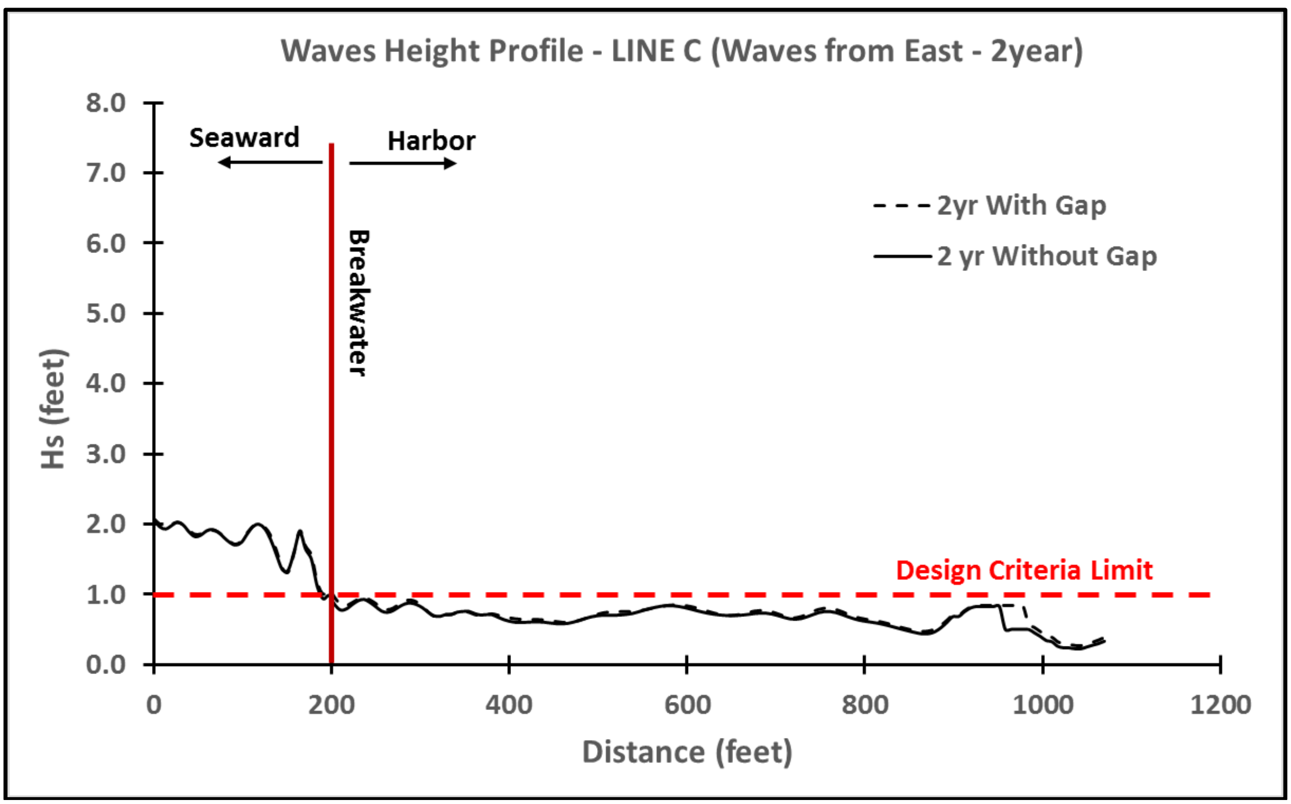


Figure 18. Wave Height Comparison – LINE C - Waves from the East (2-year)

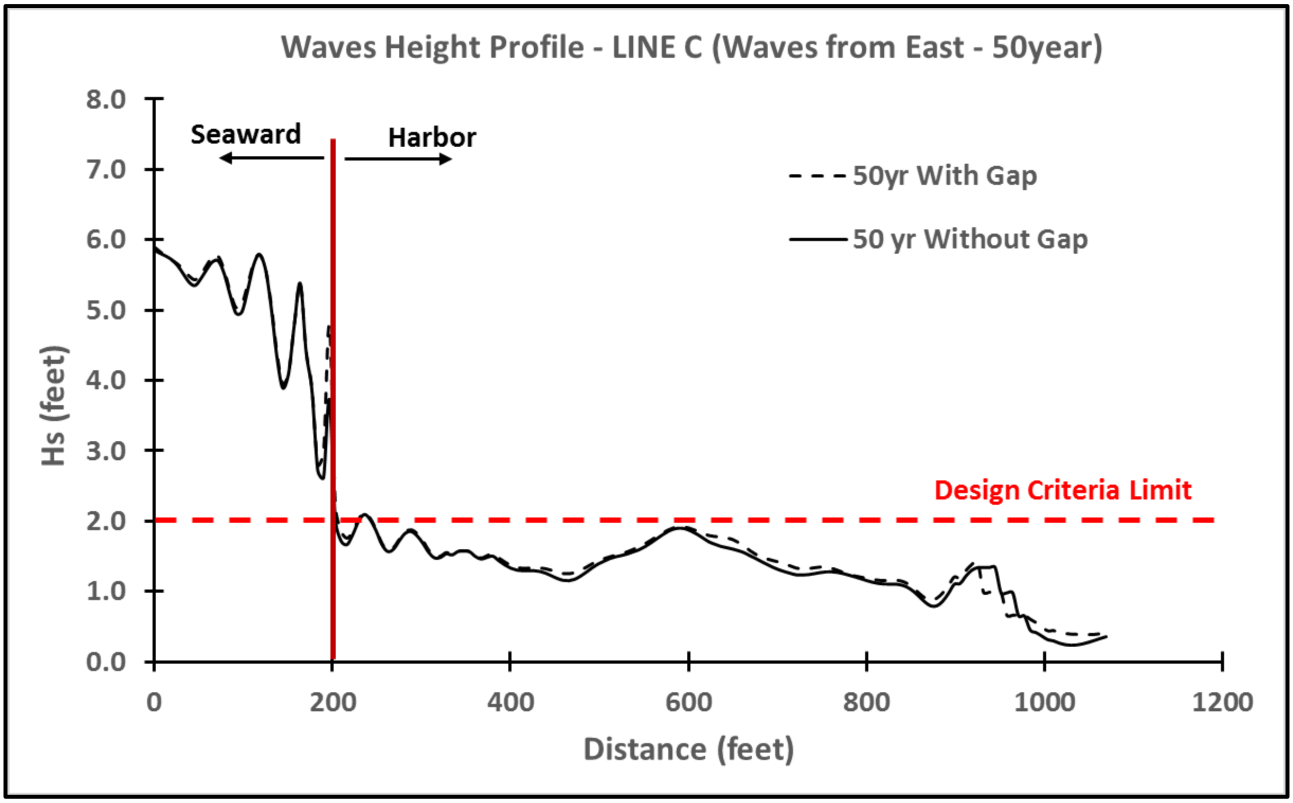


Figure 19. Wave Height Comparison – LINE C - Waves from the East (50-year)

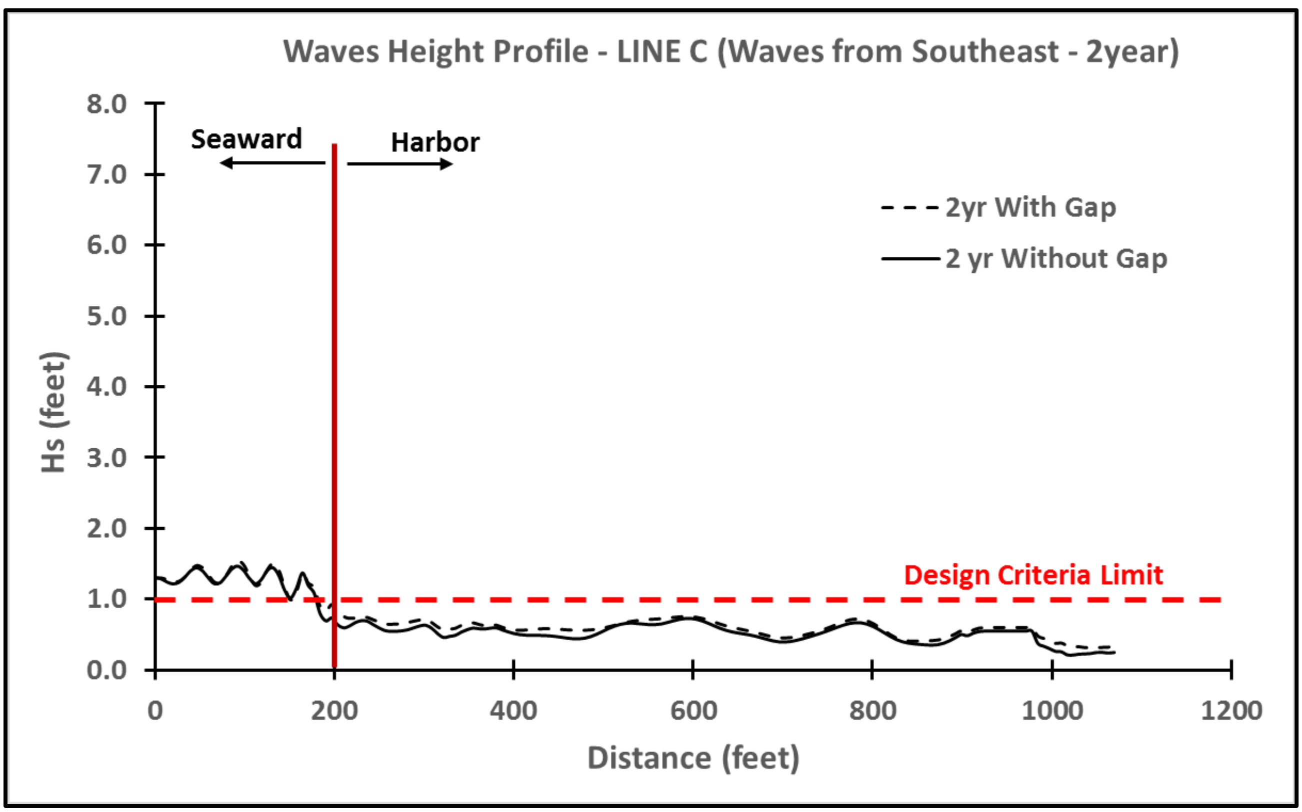


Figure 20. Wave Height Comparison – LINE C - Waves from the Southeast (2-year)

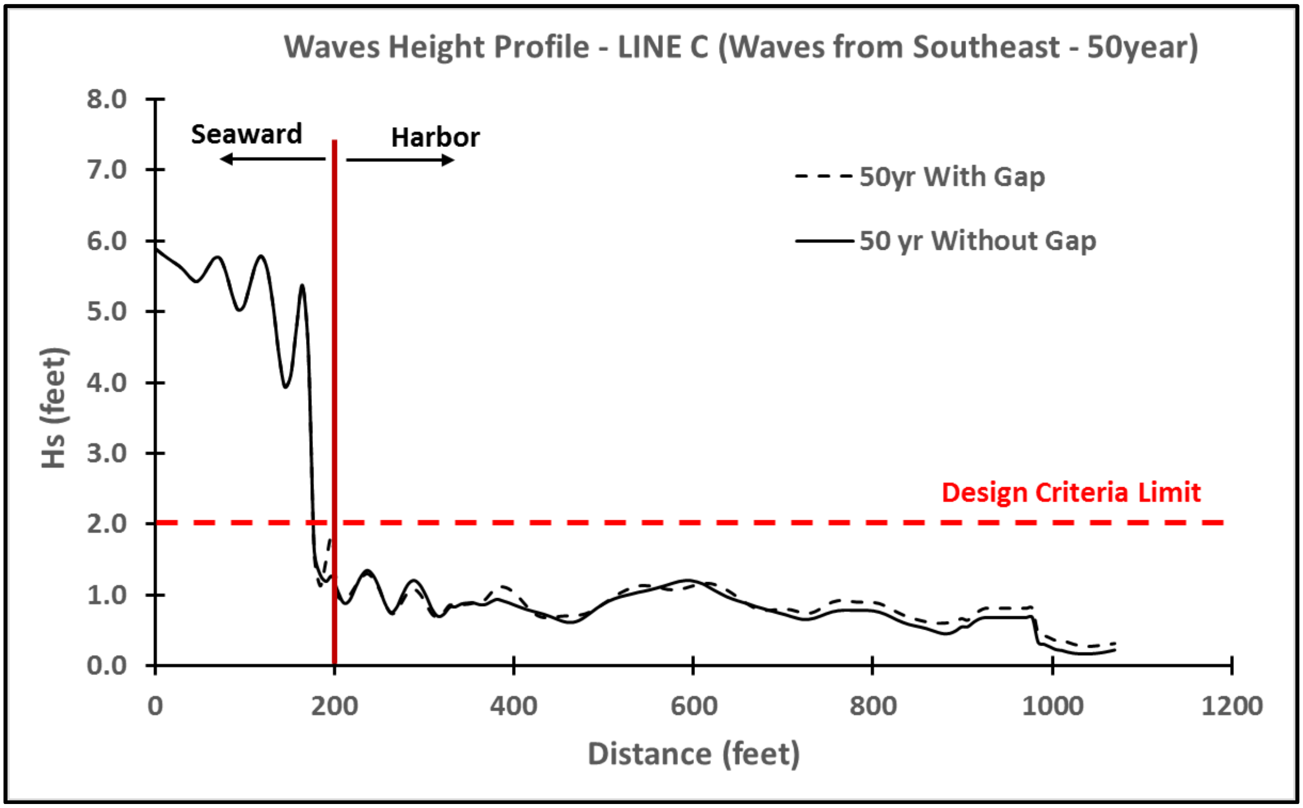


Figure 21. Wave Height Comparison – LINE C - Waves from the Southeast (50-year)

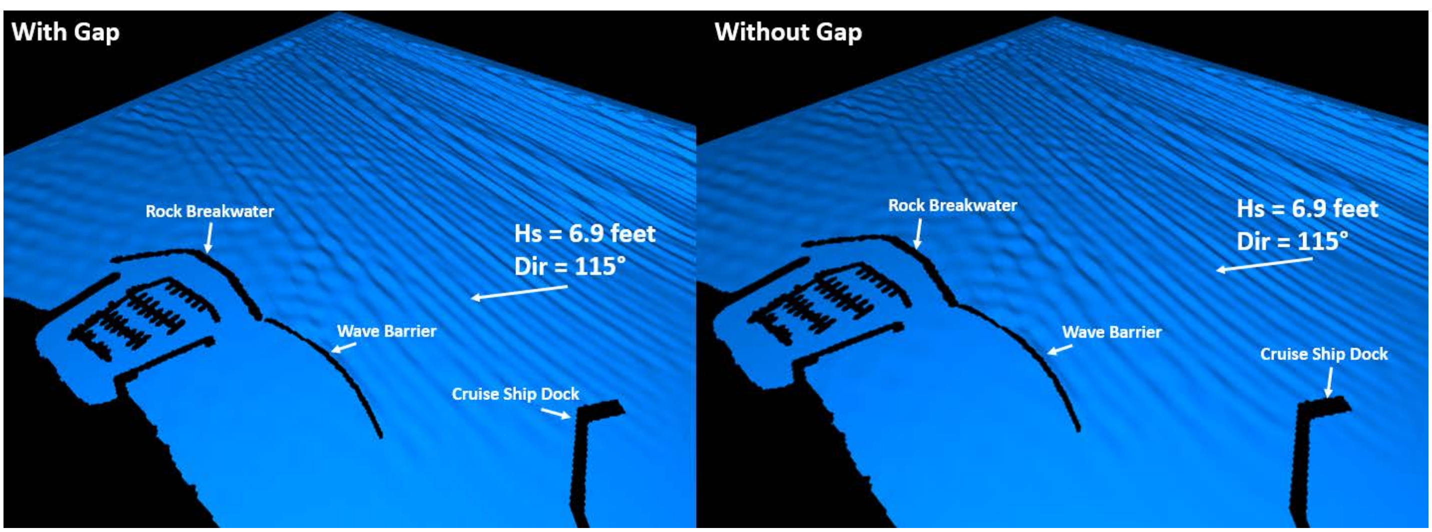


Figure 22. Model Animation Screenshots – Water Surface for Waves from southeast (Azimuth 115°)

CONCLUSIONS

The results of the analysis show minimal wave transmission through the gap between the planned wave barrier and existing rock breakwater. Closing the gap reduces the wave transmission to zero, as expected, although some wave energy enters the marina from the southern entrance. With a 17 feet wide gap (at design high water) the area inside the harbor with high waves that exceed allowable is on the order of 100 feet x 100 feet for a 50 year wave, and 50 x 50 feet for a 2 year wave. These dimensions are from the diffraction diagram calculations. The numerical model analysis shows similar results to the diffraction diagram and results from the numerical wave model. In all other areas within the harbor the wave heights are within the design criteria limits.

Wave reflection from the planned wave barrier was analyzed using the numerical wave model. The results show an increase in significant wave height near the wave barrier, (a 1.5 feet increase for the 50 year return period wave). The increase is caused by superposition of incident and reflected waves in front of the wave barrier, as expected. The magnitude of the increase varies depending on the incident wave angle and distance from the wave barrier. However, the large wave heights are localized and the effects of wave reflection are relatively small at a distance further than approximately 100 feet from the wave barrier. The wave conditions at the cruise ship dock will not be affected by the presence of a wave barrier.

At the tip of the rock breakwater, closing the gap between the wave barrier and rock breakwater results in higher waves outside the harbor than if the gap is closed, due to increased reflected wave energy, a 1.5-foot increase in wave height This increase in wave height due to reflection will likely not affect the stability of the planned new armor rock at the breakwater tip.

REFERENCES

Canada Department of Fisheries and Oceans (1980). "Study to Determine Acceptable Wave Climate in Small Craft Harbours", Small Craft Harbours Branch, Report No. 1581.

DHI (2012). MIKE 21 Boussinesq Wave Module Scientific Documentation.

PND (2013). Harbor Protection Analysis Report for South Portage Cove Harbor Expansion Project.

PND (2014). Portage Cove Wave Barrier Analysis.

National Oceanographic and Atmospheric Administration (2016). "Water Level Data", Co-ops Online Tide Data, web address: http://co-ops.nos.noaa.gov/data_res.html

US Army Corps of Engineers (1984). "Shore Protection Manual", 4th edition. Coastal Engineering Research Center, Waterways Experiment Station, Vicksburg, Mississippi.

APPENDIX A – DIFFRACTION DIAGRAMS

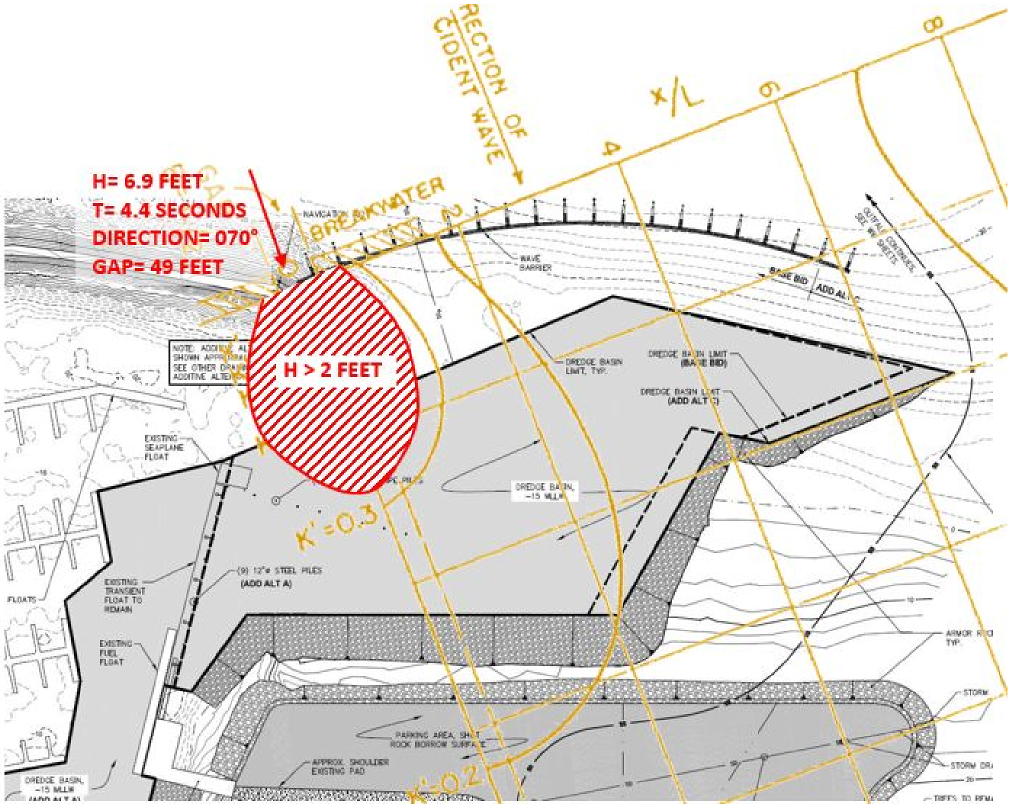


Figure A1. Diffraction Diagram – 50 Year Wave with Gap Width = 49 feet (B=L/2)

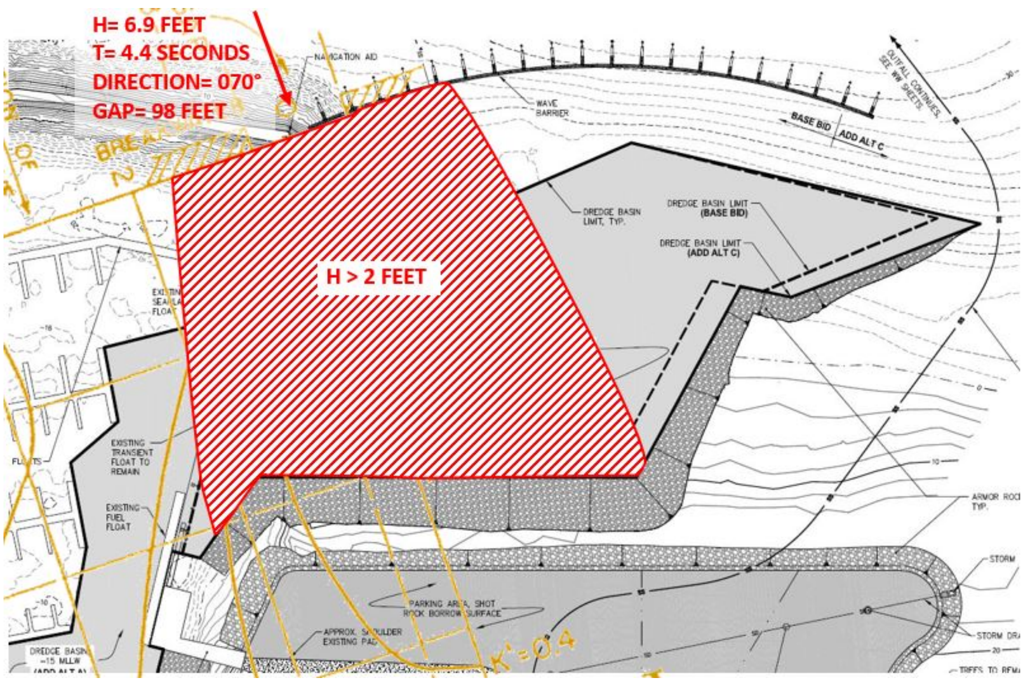


Figure A2. Diffraction Diagram – 50 Year Wave with Gap Width = 98 feet (B=L)

MEMORANDUM

To:	Dick Somerville, Principal in Charge PND Engineers, Inc.	Date: February 23, 2016 Project No: 102029.12
Cc:	Nels Sultan, Senior Engineer PND Engineers, Inc.	
From:	Sean Sjostedt, Senior Engineer PND Engineers, Inc.	
Subject:	South Portage Cove Harbor Expansion – Existing Rubble-Mound Breakwater Armor Rock Evaluation	

1.0 Introduction and Project Description

The Haines Borough is planning to expand South Portage Cove Harbor located in Haines, Alaska. The existing harbor is protected by an armor rock rubble-mound breakwater. The current harbor expansion design calls for a new partially-penetrating, pile-supported wave barrier which will tie in to the southern nose of the existing breakwater.

The United States Army Corps of Engineers, who designed the existing breakwater, have requested that PND evaluate the condition of the existing breakwater and potential adverse effects that the proposed wave barrier could incur on it; namely, increased wave energy resulting from wave refraction off of the proposed wave barrier. This memorandum will summarize a site visit and findings of the armor rock evaluation performed by PND to help satisfy this request.

2.0 Site Visit and Armor Rock Evaluation Procedures

PND visited the site on February 12, 2016 to examine, collect samples and perform a gradation of the armor rock at the southern nose of the existing breakwater. Haines Borough Ports and Harbors staff assisted PND with the evaluation. The site visit was conducted during the morning low tide (approximately a -0.4' MLLW) to maximize the quantity of armor rock accessible for measurement. The majority of the armor rock appears to be hard, competent, subangular to angular greywacke. The armor layer structure appears sound, with no obvious armor rock displacement. Voids in the armor rock layer were not large enough to make observations of underlayer or core rock.

To obtain a representative sample of stones for the armor rock gradation three straight "sample lines" were painted on the surface of the breakwater, from the navigational marker at the crest to the water level. The lines were oriented as follows: one along the alignment of the proposed wave barrier where it intersects the existing breakwater, one perpendicular to the longitudinal axis of the existing breakwater, and one approximately bisecting the two. Every visible rock crossed by the line was measured. The rock was measured along three axes to obtain an approximate volume. A total of (91) stones were measured and recorded.

A sample of the armor rock was retained and delivered to R&M Engineering in Juneau for the purpose of determining the bulk saturated surface dry specific gravity. The sample consisted of two stones, approximately football-sized, taken from an upper and lower elevation on the nose of the breakwater. This specific gravity was then used to estimate the weight of each stone measured.

3.0 Armor Rock Gradation

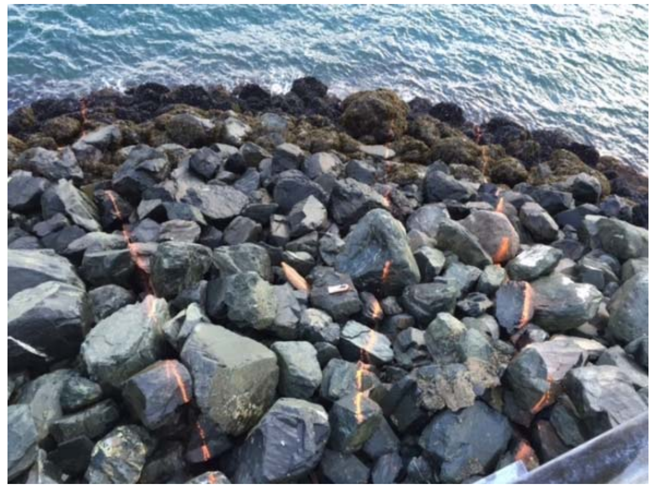
The volumes of the stones measured and the specific gravity obtained from laboratory testing were utilized to develop a gradation of the armor rock at the southern nose of the existing breakwater. Results are summarized in the table below.

	% Smaller	Rock Size (lbs)
W_{Max}	100	8,870
W_{75}	75	700
W_{50}	50	450
W_{25}	25	200
W_{Min}	0	31
1000 lbs	82	-

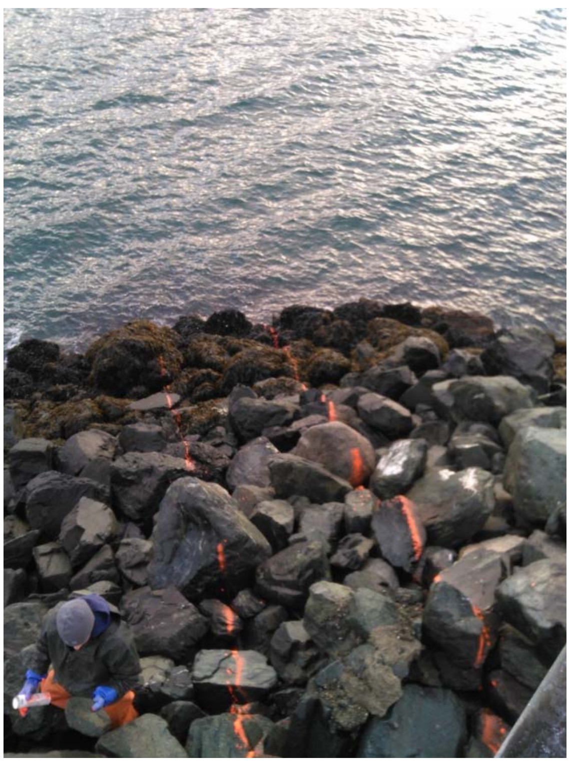
Note: Stone weights estimated based on field measurements and a bulk saturated surface dry specific gravity of 2.87 as determined from laboratory testing

Photographs

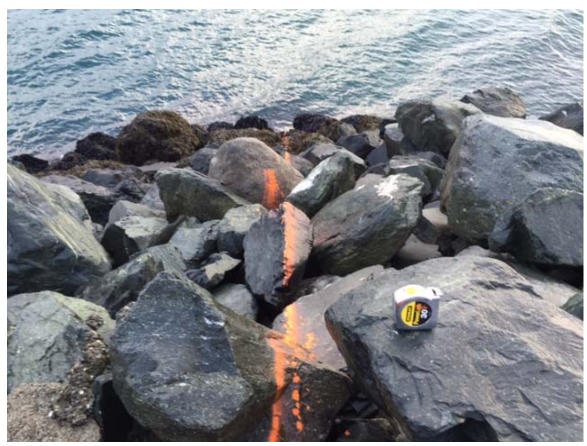
Fragment of armor rock retained for laboratory testing



Sample lines painted on southern nose of existing breakwater



PND personnel measuring existing armor rock



Example of existing armor rock

Example of existing armor rock

Example of existing armor rock

Example of existing armor rock

Example of existing armor rock

South Portage Cove Harbor Expansion
PND FILE NO: 102029; D

Specific Gravity	2.87
Total Stone Count	91

ARMOR ROCK GRADATION

Prepared By: PND Engineers, Inc. on February 12, 2015

Sample #	X"	Y"	Z"	CF	POUNDS
1	33	5	18	1.7	308
2	47	30	23	18.8	3,361
3	14	14	11	1.2	223
4	31	16	12	3.4	617
5	52	18	13	7.0	1,261
6	19	25	15	4.1	738
7	21	20	11	2.7	479
8	16	26	10	2.4	431
9	23	20	24	6.4	1,144
10	21	22	19	5.1	910
11	28	28	16	7.3	1,300
12	16	20	24	4.4	796
13	10	10	11	0.6	114
14	11	14	5	0.4	80
15	19	23	14	3.5	634
16	10	7	6	0.2	44
17	23	11	13	1.9	341
18	15	16	19	2.6	473
19	18	25	11	2.9	513
20	30	17	10	3.0	529
21	12	9	6	0.4	67
22	31	14	13	3.3	585
23	8	20	9	0.8	149
24	19	30	19	6.3	1,122
25	23	16	14	3.0	534
26	6	8	12	0.3	60
27	20	30	25	8.7	1,555
28	22	21	9	2.4	431
29	22	43	22	12.0	2,157
30	8	16	6	0.4	80
31	12	30	16	3.3	597
32	15	10	12	1.0	187
33	20	22	9	2.3	410
34	18	19	8	1.6	284
35	21	15	11	2.0	359
36	21	30	16	5.8	1,045
37	17	26	17	4.3	779
38	17	25	11	2.7	485
39	14	19	19	2.9	524
40	12	6	6	0.3	45
41	19	8	14	1.2	221
42	15	20	6	1.0	187
43	40	16	25	9.3	1,658
44	22	16	17	3.5	620
45	23	18	11	2.6	472
46	12	6	9	0.4	67
47	34	17	15	5.0	899
48	24	17	14	3.3	592

Sample #	X"	Y"	Z"	CF	POUNDS
49	23	10	9	1.2	215
50	39	19	20	8.6	1,536
51	21	12	12	1.8	313
52	15	24	15	3.1	560
53	22	13	11	1.8	326
54	26	11	10	1.7	296
55	17	22	13	2.8	504
56	36	25	19	9.9	1,772
57	10	17	9	0.9	159
58	18	7	11	0.8	144
59	10	6	5	0.2	31
60	8	21	13	1.3	226
61	8	9	5	0.2	37
62	26	21	11	3.5	622
63	16	27	12	3.0	537
64	11	10	9	0.6	103
65	12	13	15	1.4	243
66	16	27	12	3.0	537

Data Range
308
3,361
223
617
1,261
738
479
431
1,144
910
1,300
796
114
80
634
44
341
473
513
529
67
585
149
1,122
534
60
1,555
431
2,157
80
597
187
410
284
359
1,045
779
485
524
45
221
187
1,658
620
472
67
899
592
215
1,536
313
560
326
296
504
1,772
159
144
31
226
37
622
537
103
243
537
162
562

<1000	<700	<450	<200	AVE
82%	75%	48%	26%	690

MIN	31
MAX	8,865

67	12	13	10	0.9	162	687
68	29	17	11	3.1	562	496
69	34	13	15	3.8	687	830
70	19	12	21	2.8	496	8,865
71	22	26	14	4.6	830	358
72	48	33	54	49.5	8,865	48
73	24	12	12	2.0	358	620
74	11	7	6	0.3	48	279
75	20	23	13	3.5	620	41
76	14	24	8	1.6	279	65
77	8	7	7	0.2	41	50
78	7	10	9	0.4	65	1,259
79	8	10	6	0.3	50	3,681
80	23	24	22	7.0	1,259	1,949
81	30	37	32	20.6	3,681	124
82	33	19	30	10.9	1,949	538
83	8	15	10	0.7	124	929
84	19	21	13	3.0	538	1,596
85	28	20	16	5.2	929	151
86	28	25	22	8.9	1,596	311
87	8	14	13	0.8	151	37
88	21	13	11	1.7	311	326
89	12	6	5	0.2	37	216
90	21	10	15	1.8	326	0
91	16	13	10	1.2	216	0
92				0.0	0	0
93				0.0	0	0
94				0.0	0	0
95				0.0	0	0
96				0.0	0	0
97				0.0	0	0
98				0.0	0	0
99				0.0	0	
100				0.0	0	

R&M ENGINEERING
Juneau
has joined the
PDC INC. ENGINEERS
family

February 17, 2016

Mr. Sean Sjostedt
PND Engineers, Inc.
9360 Glacier Highway, Suite 100
Juneau, AK 99801

Sent Via Email

Re: South Portage Cove Harbor Expansion (PND # 102029.12)
Armor Rock
Laboratory Test Results
PDC Project No. 16054JN

Mr. Sjostedt,

On February 12, 2016, R&M Engineering (R&M) received one, 5-gallon bucket containing armor rock. The material source is unknown. The sample was collected from rock obtained at an existing breakwater. The sample was taken by Sean Sjostedt (PND Engineers, Inc.). The following table comprises a summary of the test results performed by R&M:

Lemon Creek Sand			
Requested Test	**Test Method**	**Comments**	**Test Results**
Specific Gravities & Absorption, CA	ASTM C-127	*See Note Below	**G_{sb} = 2.87** **G_{sb} SSD = 2.87** **G_{sa} = 2.88** **Absorption = 0.2 %**

*Note: The rock sample R&M Engineering received on 2/12/16 did not satisfy minimum sample mass requirements as described in ASTM C-127, Section 7.3. PND Engineers, Inc., understanding that results may not be representative of actual material, requested the rock sample to be tested as-is.

If you have any questions regarding the test procedure or the results, please do not hesitate to call.

Sincerely,

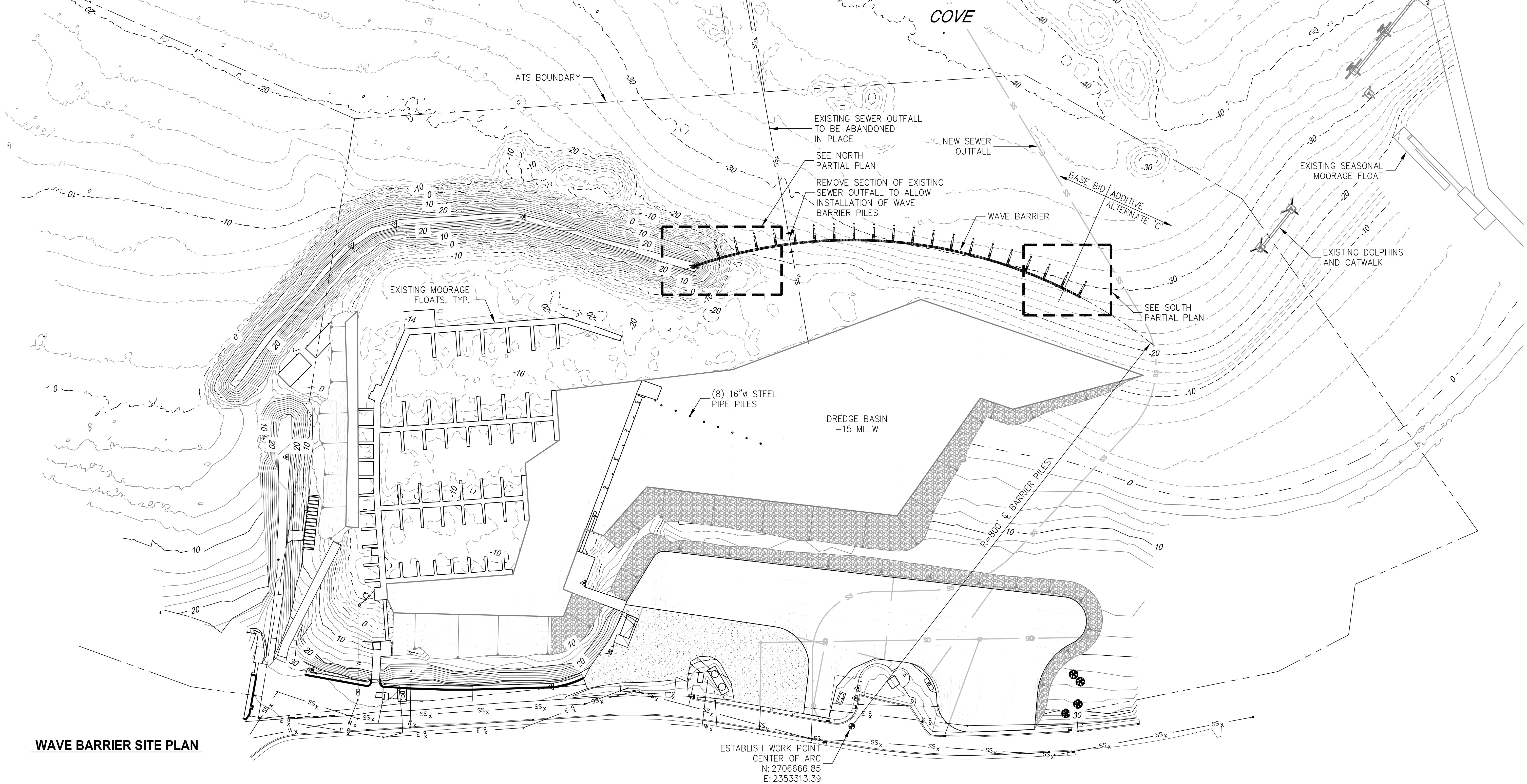
R&M ENGINEERING

William Steele
NICET Level I – Soils

I:\2016\16054JN\160218, Specific Gravity Report.docx

NOTES:

1. VERTICAL DATUM IS MEAN LOWER LOW WATER (MLLW = 0 FEET).
2. ENCOUNTERING COBBLE AND BOULDER-SIZE MATERIAL SHOULD BE ANTICIPATED WITHIN ANY OF THE REPRESENTED SOIL HORIZONS.
3. MAINTAIN VESSEL ACCESS TO EXISTING MOORAGE FLOATS AT ALL TIMES.



WAVE BARRIER SITE PLAN

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

HAINES BOROUGH ALASKA

PND ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS

SCALE: SCALE IN FEET 0 40 80 160 FT.

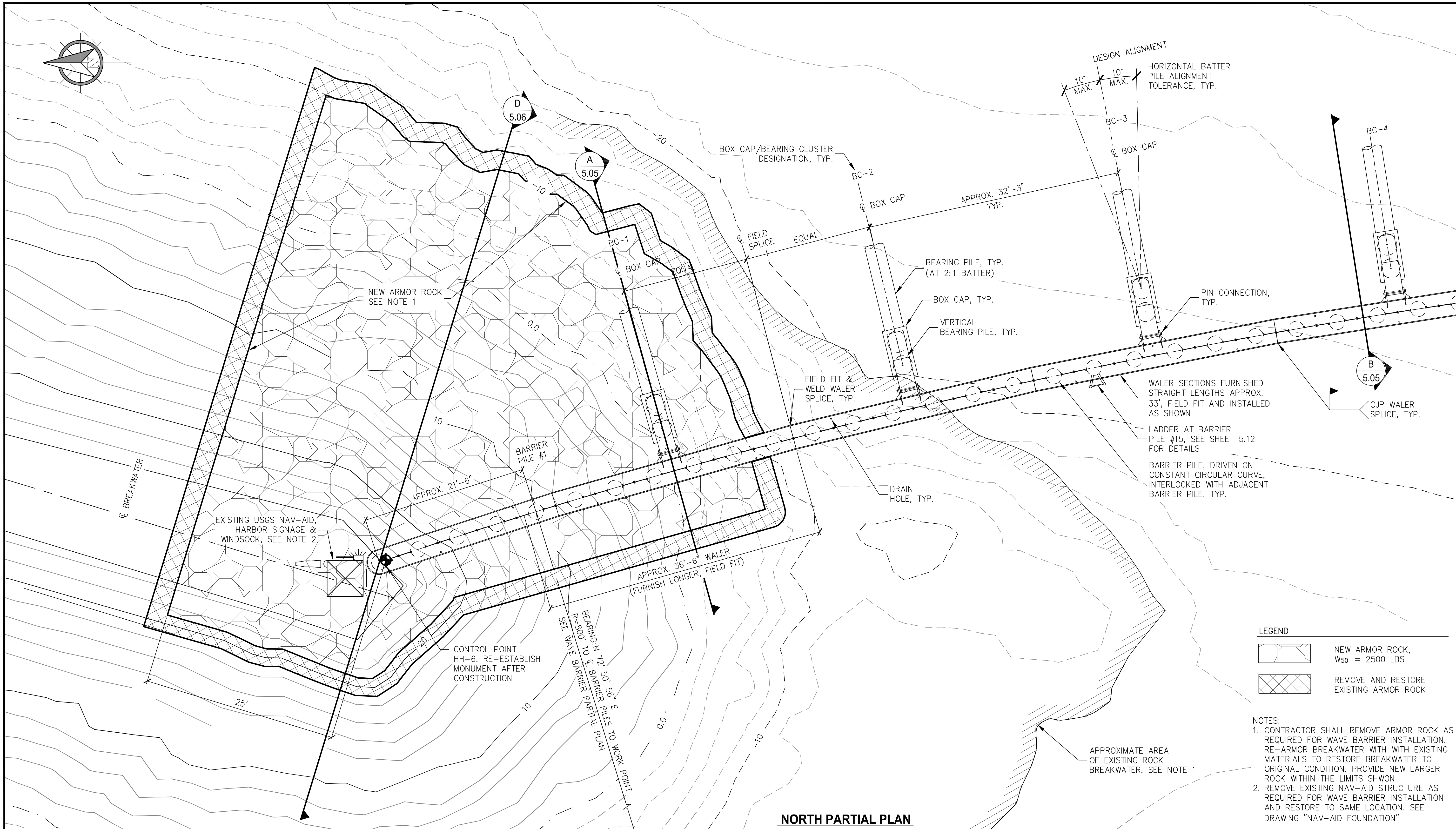
DATE: 2/10/16

HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION

SHEET TITLE: WAVE BARRIER SITE PLAN

PN&D PROJECT NO.: 102029.10

5.01
SHEET
OF



NORTH PARTIAL PLAN

LEGEND

- NEW ARMOR ROCK, W_{50} = 2500 LBS
- REMOVE AND RESTORE EXISTING ARMOR ROCK

NOTES:

1. CONTRACTOR SHALL REMOVE ARMOR ROCK AS REQUIRED FOR WAVE BARRIER INSTALLATION. RE-ARMOR BREAKWATER WITH WITH EXISTING MATERIALS TO RESTORE BREAKWATER TO ORIGINAL CONDITION. PROVIDE NEW LARGER ROCK WITHIN THE LIMITS SHOWN.
2. REMOVE EXISTING NAV-AID STRUCTURE AS REQUIRED FOR WAVE BARRIER INSTALLATION AND RESTORE TO SAME LOCATION. SEE DRAWING "NAV-AID FOUNDATION"

HAINES BOROUGH ALASKA

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

P N D ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS
SCALE: SCALE IN FEET 0 6 12 FT.
DATE: 2/10/16

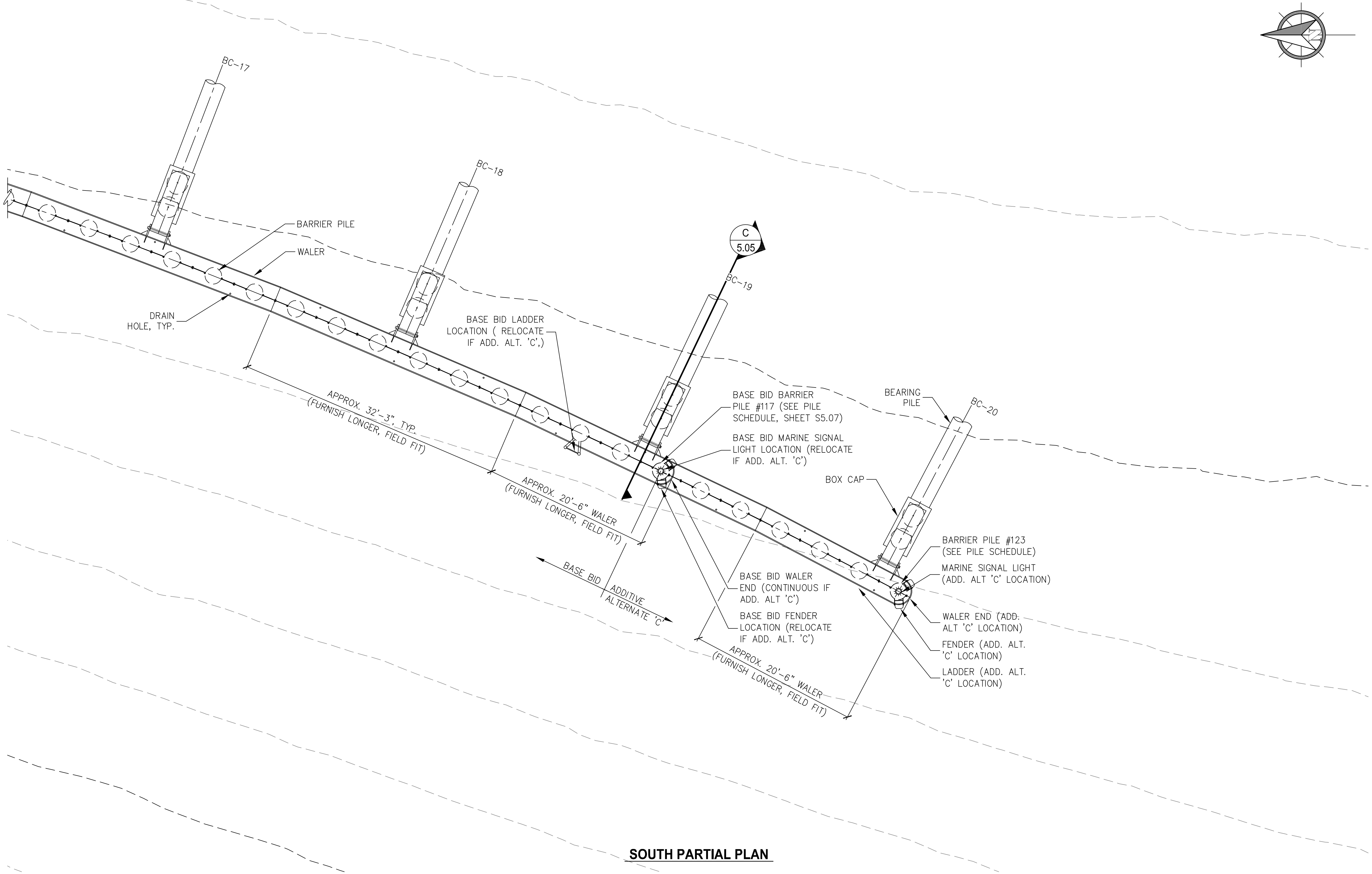
HAINES BOROUGH PORTAGE COVE HARBOR EXPANSION

SHEET TITLE: **WAVE BARRIER NORTH PARTIAL PLAN**

PN&D PROJECT NO.:102029.10

5.02

SHEET OF



SOUTH PARTIAL PLAN

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

PND ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

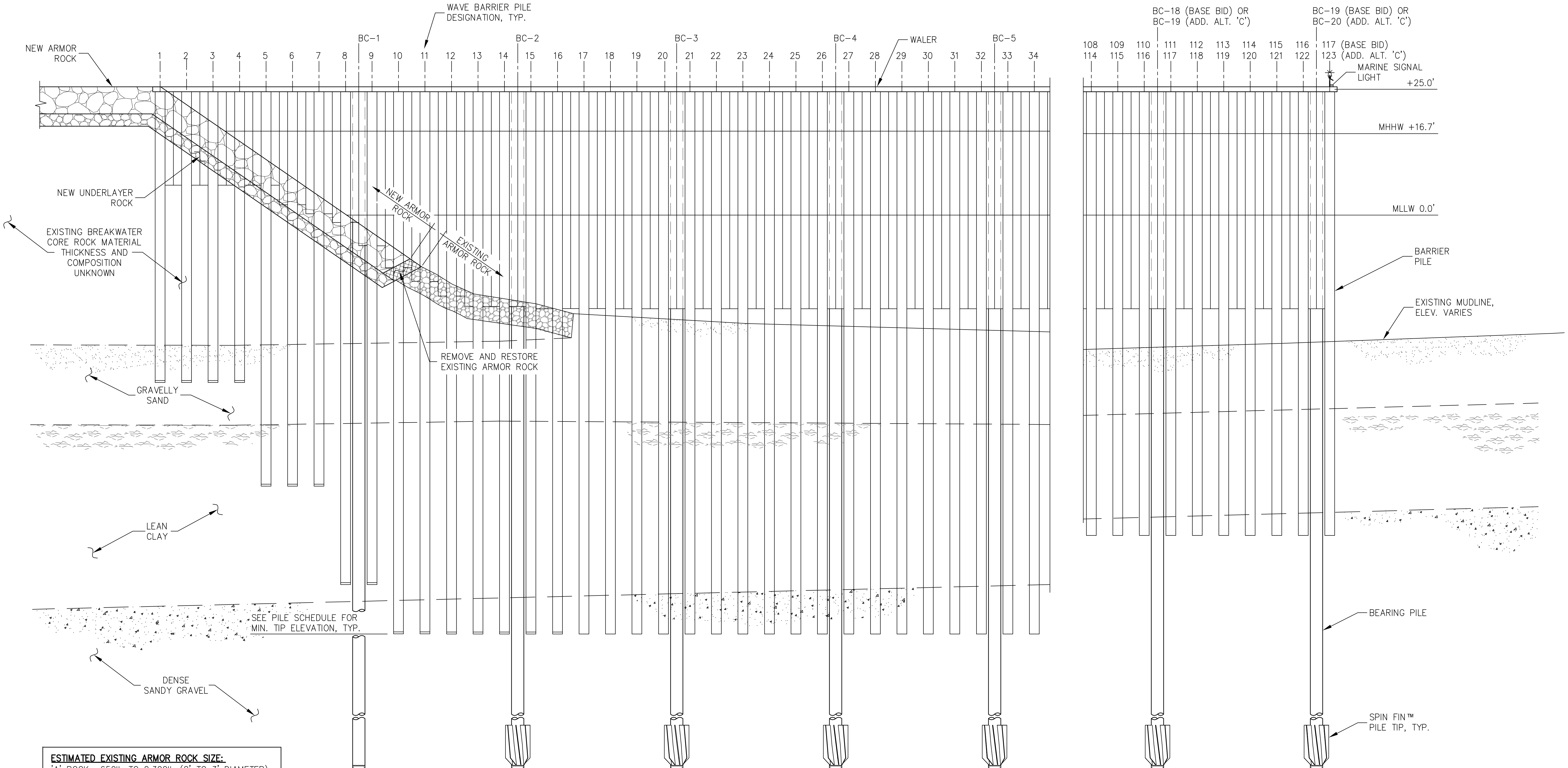
DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS
SCALE: SCALE IN FEET 0 6 12 FT.
DATE: 2/10/16

HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION

SHEET TITLE: WAVE BARRIER SOUTH PARTIAL PLAN

PN&D PROJECT NO.: 102029.10

5.03
SHEET OF



ESTIMATED EXISTING ARMOR ROCK SIZE:
'A' ROCK 650lb TO 2,300lb (2' TO 3' DIAMETER)
'B' ROCK 65lb TO 650lb (1' TO 2' DIAMETER)

NOTE:
1. SOIL INFORMATION SHOWN IS APPROXIMATE AND FOR GENERAL ILLUSTRATION PURPOSES ONLY.SEE GEOTECHNICAL REPORT FOR SPECIFIC SOILS INFORMATION.
2. SEE PILE SCHEDULE FOR DETAILS NOT SHOWN AND INSTALLATION CRITERIA.

PARTIAL ELEVATION
NOTE: LADDERS NOT SHOWN FOR CLARITY

HAINES BOROUGH ALASKA

SPIN FIN™ PILE IS A REGISTERED TRADEMARK OF PND ENGINEERS, INC.

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

P N D ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS
SCALE: SCALE IN FEET 0 10 20 FT.
DATE: 2/10/16

HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION

SHEET TITLE: PARTIAL ELEVATION

PN&D PROJECT NO.:102029.10

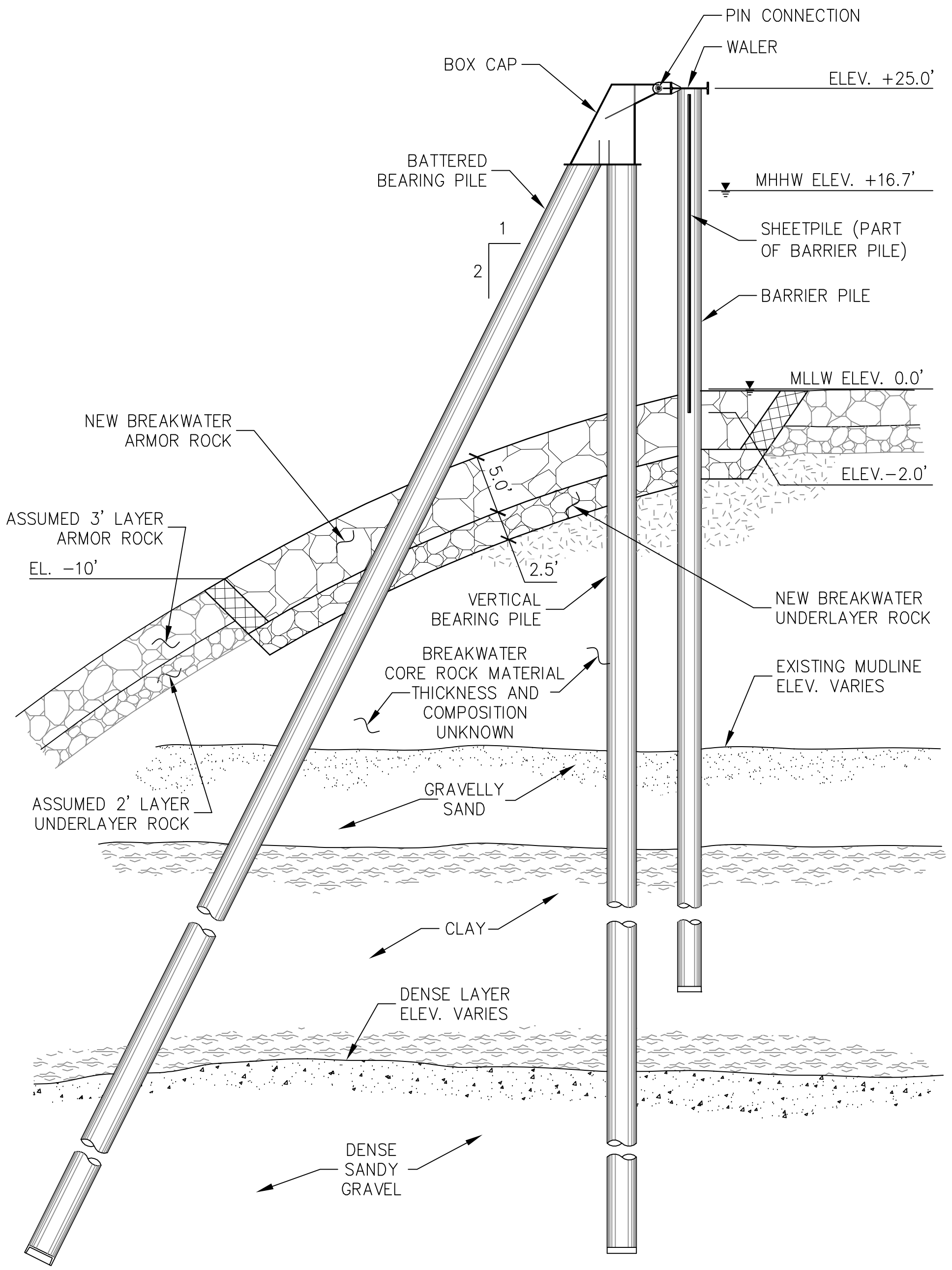
5.04
SHEET
OF

NEW ARMOR ROCK SIZE:

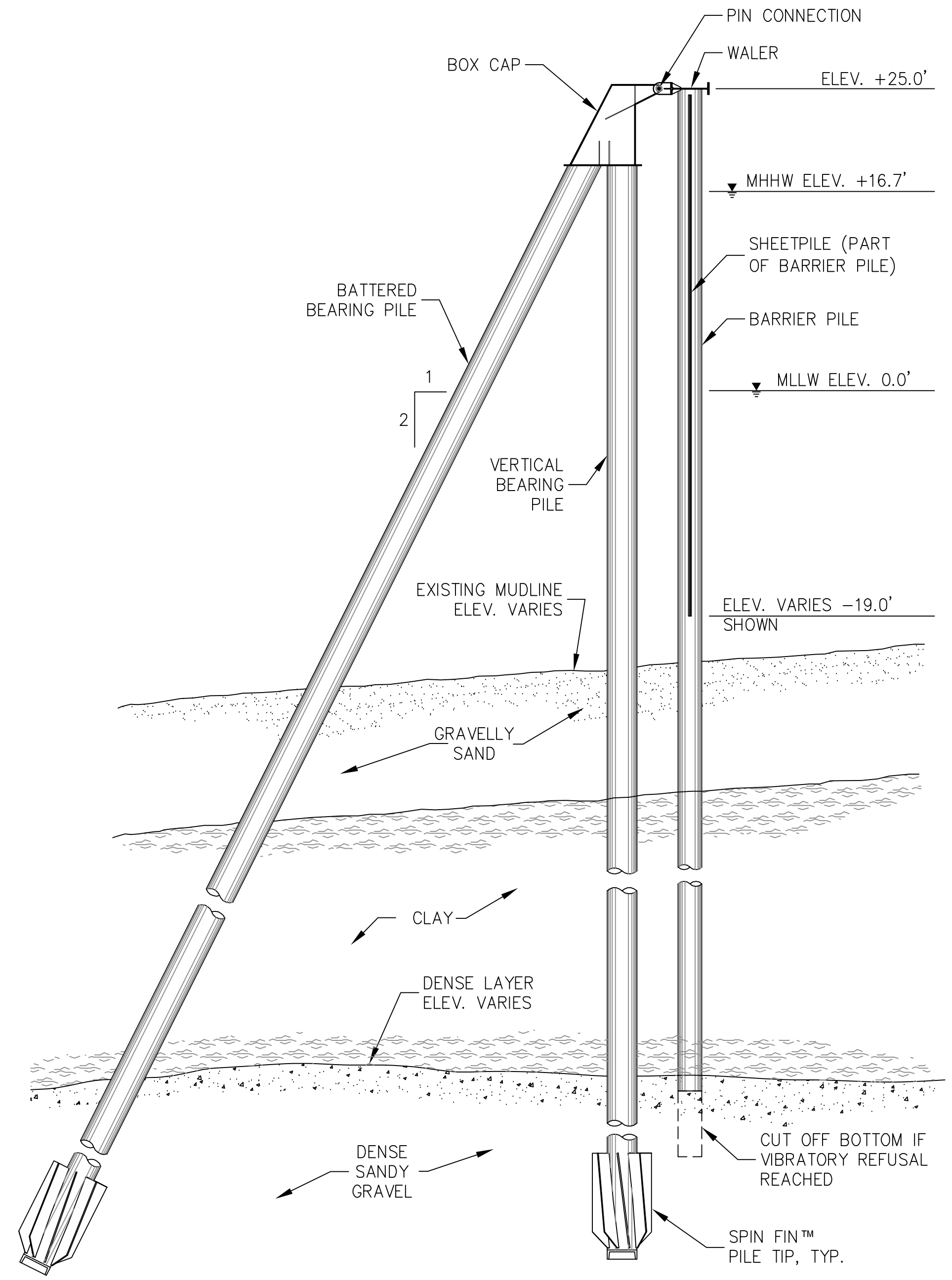
NEW ARMOR ROCK
W_{50} = 2500 LBS

NEW UNDERLAYER ROCK
W_{50} = 250 LBS

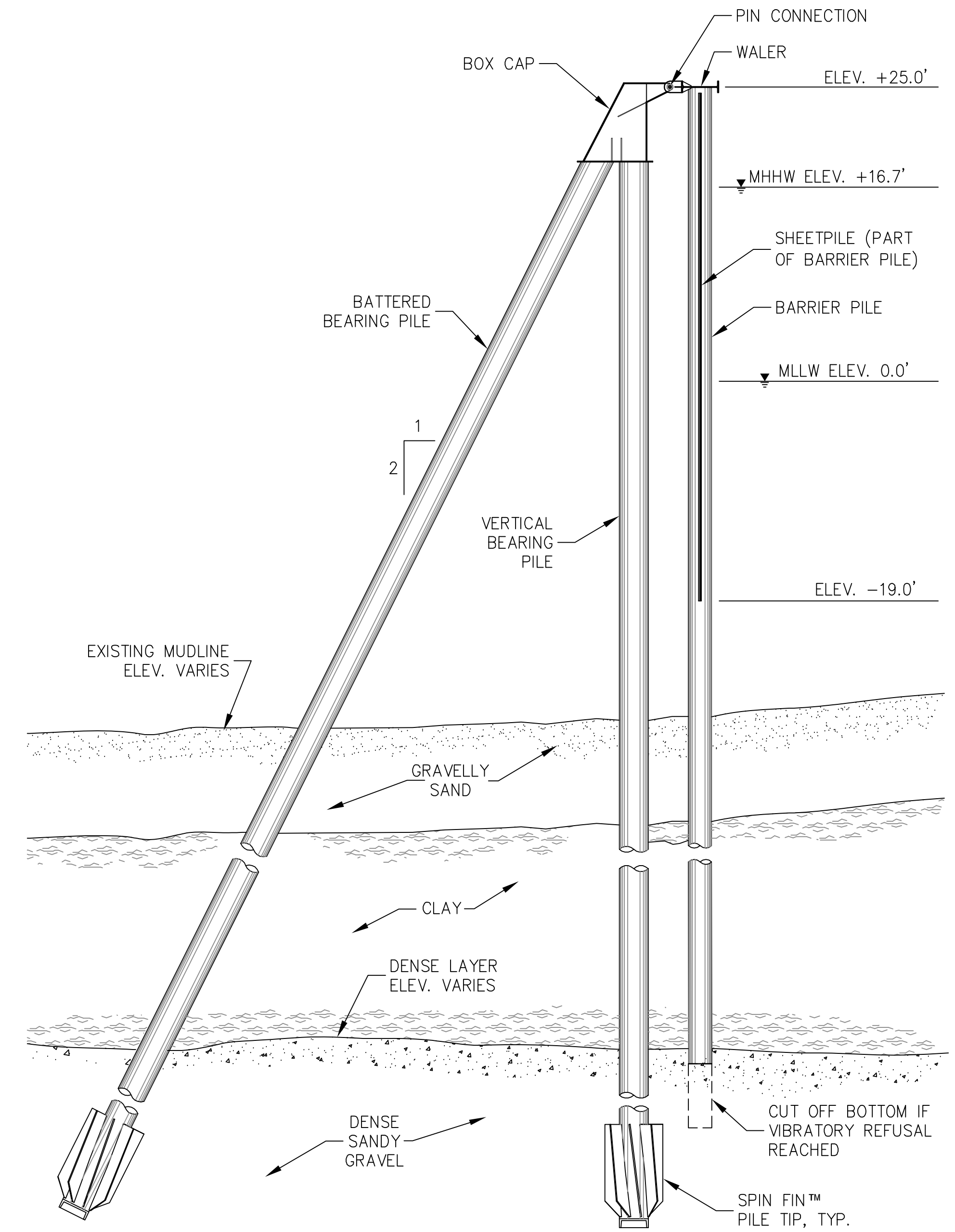
REMOVE AND RESTORE
EXISTING ARMOR ROCK



A
5.02
TYPICAL SECTION



B
5.02
TYPICAL SECTION



C
5.03
TYPICAL SECTION

SPIN FIN™ PILE IS A
REGISTERED TRADEMARK
OF PND ENGINEERS, INC.

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

P N D
ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS

SCALE: SCALE IN FEET 0 8 16 FT.

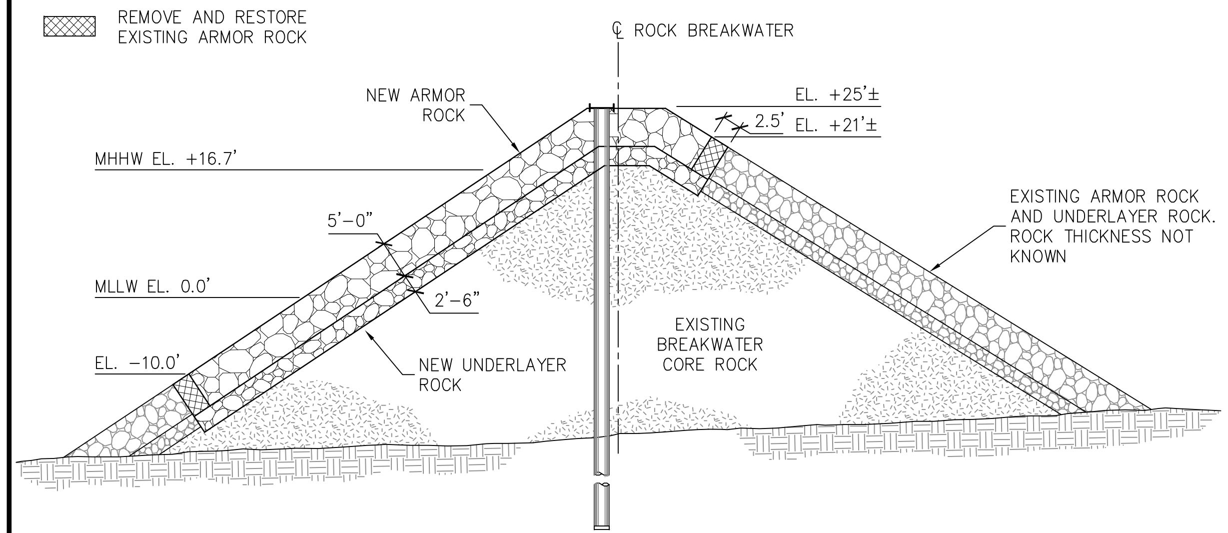
DATE: 2/10/16

HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION

SHEET TITLE:
TYPICAL SECTIONS

PN&D PROJECT NO.: 102029.10

5.05
SHEET
OF



D
5.02
TYPICAL SECTION

GENERAL NOTES – ROCK BREAKWATER

REFERENCE

ASTM C 127–88 (R93)	Specific Gravity and Absorption of Coarse Aggregate
ASTM D 422 (1963; R 1998)	Particle–Size Analysis of Soils
ASTM D 2487–00	Classification of Soils for Engineering Purposes (Unified Soil Classification System)
CRD–C 148–69 (USACE)	Method of Testing Rock for Expansive Breakdown on Soaking in Ethylene Glycol

ABBREVIATIONS

ASTM	American Society for Testing and Materials
BSSD	Bulk Specific Gravity (Saturated–Surface Dry)
CRD	Concrete Research Division (US Army Corps of Engineers)
CF	Cubic Feet
CFR	Code of Federal Regulations
DIA	Diameter
E	Easting
EL	Elevation
ELEV	Elevation
FT	Feet
LBS	Pounds
MIN	Minimum
N	Northing
PSI	Pound per Square Inch
STA	Station
USACE	US Army Corps of Engineers

PERMITS

Contractor shall comply with the requirements of all permits. A copy of the permits are included in an appendix to the contract documents.

SUBMITTALS

Five (5) copies of each submittal shall be delivered to the Owner.
a. Work plan
b. Quarry location and development plan
c. Rock quality test results and reports
d. Rock transportation and handling plan
e. Cross sections and quantity computations
f. Daily Report of Operations
g. Catalog cuts for navigation aids

WORK PLAN

A work plan shall be submitted, including;
a. Sequence of operations
b. Schedule and hours of operation
c. Personnel
d. Supervisory organization and reporting chain up to and including the owner of the Contractor
e. Procedures and equipment to be used
f. Name and/or quantity of equipment and its capacity

DAILY REPORT OF OPERATIONS

On a daily basis, a Daily Report of operations shall be prepared and submitted to the Owner. The following information will be included: date, period covered by report, personnel on site, equipment used, area dredged or rock placed. The report shall include the results of all inspections, surveys and monitoring activities and shall be signed by the Contractor's on–site superintendent.

ROCK SOURCE AND QUARRY DEVELOPMENT PLAN

There is no designated rock source for this project.

The Contractor shall obtain rock which meets all requirements specified herein. The Contractor shall identify its proposed rock source. The Contractor shall comply with all Federal, State and local laws and regulations pertaining to surface mining, safety, and protection of the environment. The Contractor shall be responsible for obtaining all permits and/or easements for the rock source.

Development of a new or existing off–site quarry for furnishing rock required by this contract may require review by Local, State and Federal agencies. The Contractor is responsible for investigating and obtaining all necessary reviews and permits from Local, State and Federal agencies.

Quarry sites shall be evaluated per ASTM D4992–07 Standard Practice for Evaluation of Rock to be used for erosion control.

EXISTING BREAKWATER
Rock removed from the existing breakwater may be reused if it meets the quality and size specifications for Armor Rock Type 1 or Underlayer Rock Type 2.

Removed rock shall become the property of the Haines Borough and stockpiled on land at a location designated by the Owner.

Rock shall be rough, angular, dense, sound and durable. Rock shall be fine grained, free from faults, fissures, seams, laminations, planes of weakness, or bands of minerals or deleterious materials that would result in breakage during or after placement in the breakwater. Rock shall be free of expansive or other materials which would cause accelerated deterioration by exposure to project conditions.

Rock shall be from a source pre–approved by the Owner. The greatest dimension of each rock shall be no greater than 3 times the least dimension.

All rock will be accepted or rejected at the quarry site based on test results and visual geologic examinations by the Owner. Test results shall be furnished to the Owner 30 days prior to any transport of rock. Rock shall be tested as specified below.

No further laboratory testing of rock will be necessary if results meet the requirements specified, and a continuous visual geologic examination of the rock by the Owner indicates no change in rock type or quality for rock passing the laboratory tests. Rock exhibiting significant changes in type or quality will be rejected unless additional testing shows that the rock meets the specified requirements. Rock shall meet the following test requirements for quality.

A test result that indicates the rock does not meet a standard for quality will not necessarily mean that the rock source is rejected. Rock will be accepted or rejected by the Owner based on evaluation of all test results and visual geologic examination of the rock at the quarry.

Test	Results
Specific Gravity, BSSD ASTM C 127–88 (R93)	Not less than 2.65 (Stone density not less than 165 lbs/cf)
Absorption, ASTM C 127–88 (R93)	Not greater than 6%
Freeze–Thaw (300 cycles) NPD Test Method	Not greater than 10% loss by weight
Wetting–Drying (300 cycles) NPD Lab Method by weight	Not greater than 15% loss

Test	Results
L.A. Abrasion ASTM C535	less than 20% loss in 500 revs
Soundness–Magnesium Sulfite ASTM C88	less than 5% loss

Testing shall be the responsibility of the Contractor and shall be performed by an independent commercial test laboratory approved by the Owner. The Contractor shall furnish certified, complete copies of all test results to the Owner. Previous tests for an existing quarry dated within 12 months of contract are acceptable.

ARMOR ROCK – TYPE 1 ROCK

Armor Rock shall meet the gradation in the following table.

Armor Rock Gradation (percent by weight)		
	% Smaller	Rock Size (pounds)
Wmax	100	3250
W_{50}	25–50	2500
Wmin	0	1750

UNDERLAYER ROCK – TYPE 2 ROCK

Underlayer Rock shall meet the gradation in the following table.

Underlayer Rock Gradation (percent by weight)		
	% Smaller	Rock Size (pounds)
Wmax	100	500
W_{50}	25–50	250
Wmin	0	125

PRODUCTION TESTING

The Contractor shall perform the following minimum gradation tests.

Samples shall be taken at the source of the materials, and at subsequent points during transport or placement directed. No failing tests shall count toward meeting the minimum number of representative tests. Tests shall be evenly spaced throughout production. Tests shall be by actual weighing. Results shall be provided to the Owner within 24 hours, or sooner if requested.

Tests shall consist of determining the total weight of all the rocks and the individual weight of each rock in the sample respectively. Percent smaller by weight shall be determined by dividing the total weight of the sample into the sum of the total weight of the rocks smaller than the specified rock weight.

a. Armor Rock: Test at least 2 representative samples. Each sample shall be approximately 50 cubic yards in volume.
b. Underlayer Rock: Test at least 2 representative samples. Each sample shall be approximately 25 cubic yards in volume.

The Contractor shall display at least one typical rock for each type of rock specified at the quarry loading area and at the project site. The weight shall be clearly marked on each rock, and shall be located within easy sight of rock handling equipment at the quarry loading area and project site, to ensure proper sizing and grading of the specified material. Each armor rock shall be weighed prior to transport from the quarry.

FREEZE–THAW TESTING METHOD

The test sample shall consist of approximately 50 pounds of pieces passing the 2 inch sieve and retained on the 1–1/2 inch sieve, will be prepared by jaw crushing or hand chipping. All sharp edges will be chipped off and only pieces approximately cubical in shape will be used. The original dry weight of pieces selected for the freeze–thaw test will be computed by determining moisture content of room dry rock from representative surplus or undersized pieces.

$$\text{Dry weight of pieces selected for freeze–thaw} = \frac{\text{Weight Room Dry}}{1 \div MC/100}$$

(MC = moisture content in %)

Specimens shall be immersed in water for 24 hours prior to start of test. Sample will then be placed in a pan approximately 15 inches x 9–1/2 inches x 2–1/4 inches and the pan filled from 1/4 inch to 1/2 inch depth of water. Sample in pan will be subjected to freezing and thawing in freeze–thaw apparatus described in CRD–C 20–94, "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing" (ASTM C 666–92). Freezing and thawing will be at the rate of 12 cycles per day, each cycle consisting of approximately one hour at 0 + 2°F and one hour at 40 + 2°F. At the end of the test, the samples will be washed, dried, sieved over the 1–1/2 inch sieve and weighed. Tests shall consist of 100 cycles unless other wise specified. The percent loss will be computed based on the original dry weight. Observations of appearance of each piece with comment as to apparent soundness, cracking, etc., will be reported. Photographs of the sample at the end of the test or during the test will be made when significant cracking, flaking, crumbling, or disintegration has taken place.

WETTING AND DRYING

The test sample shall be approximately 250 pounds of 2 inch to 1–1/2inch sized particles prepared as specified above for freezing and thawing tests. The test sample shall be oven–dried and weighed, then soaked for 24 hours prior to starting test. Testing will consist of soaking for 3 hours in tap water at approximately 60°F, and drying for 3 hours with an infrared heat lamp so that the surface temperature of rocks will reach 165°F. Upon completion of the test, samples will be oven–dried, screened over 1–1/2 inch sieve and weighed. Percent loss will be based on original dry weight. Significant changes in appearance such as cracking, splitting, etc., will be noted.

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

PND ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS
SCALE: SCALE IN FEET 0 8 16 FT.
DATE: 2/10/16

HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION

SHEET TITLE: ROCK BREAKWATER & NOTES
PN&D PROJECT NO.:102029.10
5.06
SHEET
OF

GENERAL NOTES – ROCK BREAKWATER (cont.)

ROCK PLACEMENT
Place rock to the lines and grades indicated on the drawings. The finished slope shall form a uniform and regular surface not steeper than the slopes indicated on the drawings.

All armor rock shall be stable, keyed and interlocked with neighboring rocks. Armor rock placement shall be without overhanging, or "floater" rocks and without voids underneath a layer of rock. Armor rock shall be seated on the underlayer rock to prevent slipping, rocking or displacement under wave action or the weight of overlying rock. All armor rock shall be placed individually and in a manner to avoid displacing underlying materials or placing undue impact force on underlying material. Armor rock shall not be dropped.

Armor rock above water line at time of placement additionally have Selective Placement as follows: place rock side by side with staggered vertical joints. All rocks shall be interlocked and keyed into adjacent rocks. The longitudinal axis of each outer rock shall be normal to the axis of the breakwater and slope downward toward the center of the armor rock berm. The rocks shall be placed with maximum interlocking of rocks and maximum points of contact with adjacent rocks. Re–handling of individual rocks after initial placement may be required to achieve the above requirements.

Armor rock below water line shall have Random Placement. Attached to the contract documents are photographs which illustrate the difference between "selective placement" and "random placement".

Equipment proposed for use shall be capable of placing the armor rock near its final position before release and capable of moving and manipulating the armor rock if necessary to its final position. Dragline buckets and skips shall not be used for placement of armor rock. Placement shall begin at the bottom of the slope and proceed up the slope placing rock to the full armor thickness in one operation. Casting or dropping of rock from a height greater than one (1) foot or moving by drifting or manipulating down the slope shall not be permitted.

Underlayer rock may be placed by bottom–dump barge, clamshell or other methods to meet the lines and grades shown on the plans.

Reference: (US Army Corps of Engineers, Coastal Engineering Manual, Part VI –Chapter 4, Materials and Construction Aspects, page VI–21, revised June 1, 2006)

VERIFICATION TEST SECTION
The Contractor shall construct an initial 25 feet by 25 feet area of rock breakwater which upon acceptance shall become a model for further rock placement and shall become a part of the finished structure. The purpose of the verification test section is to verify the rock size, layer thickness and rock placement. The Owner may direct changes in the work based on the results of the verification test section.

SURVEYS
Control of all rock placement and dredging shall be by neat line surveys.

All surveys shall be performed by an independent licensed surveyor at the Contractor's sole expense. The surveyor shall be normally engaged in the business of hydrographic surveying. The independent surveyor, their equipment and methods, shall be pre–approved by the Owner.

The contractor shall install a water level gauge at the project site so that the equipment operator and hydrographic surveyors can observe the water level at all times. The water level gauge shall be installed under the supervision of a licensed surveyor.

The Owner shall be notified a minimum of 5 days prior to any surveys. The Owner may be present during all surveys and may accompany the surveyor on board the survey vessel.

Surveys shall be daily or as required to control the work and to determine neat line rock placement and dredging limits. The following surveys shall be required at a minimum:

a. Pre–Construction Survey: A pre–construction survey shall be performed prior to initial removal of any rock or excavation of any materials.
b. Interim Condition Surveys: Interim condition surveys shall be conducted before covering the excavated/dredged area, core rock and underlayer rock, and at intervals of no more than 3 days. Cross–sections of the interim condition surveys shall be plotted every one week in which required survey data has been collected and shall be provided to the Owner.
c. Post–Construction Survey: A post–construction survey shall be conducted immediately following completion of the breakwater.

Cross–sections at 20 feet stations along the breakwater centerline shall be prepared before and after rock placement. Soundings shall be taken along each station, at a minimum of 20 feet intervals, and perpendicular to the centerline of the breakwater. Soundings shall extend a minimum of 50 feet beyond the toe on each side, and shall capture all break points.

Cross–sections shall be plotted at a scale of 1"=10" (1 inch equals 10 feet) both horizontally and vertically and shall show the existing ground, all excavated material, all placed rock, and the correct breakwater design template for each 25 feet station, together on the same axis. Elevations shall be displayed and plotted to the nearest 0.1 foot. Each section shall be identified and labeled with the excavation and fill calculated areas. Surveys shall include the location, date and time (hours and minutes) and water elevation for each sounding line. Data and notes shall include bar checks and time of readings.

All survey submissions shall include a hard (paper) copy and an electronic copy of the survey data, plotted cross–sections and calculations. The electronic data shall include all point files, breaklines, digital terrain models, triangular irregular network (TIN) and other digital data used to complete the survey and quantity calculations. The contractor shall submit sounding sheets (plan view of all soundings), plotted cross–sections, field notes and quantity calculations within 5 days of the completion of a survey.

Deficiencies identified by the surveys shall be corrected before continuing with excavations, dredging or placement of rock.

Quantities shall be calculated using the average–end–areas method and using original ground and design template neat lines and delivered to the Owner with the survey cross–sections. Cross–sections and quantity calculations shall be performed by the independent surveyor or registered engineer. Surveys may need to be repeated at the Contractor's sole expense until the placed rock, excavation and dredging is within the limits and tolerances indicated on the drawings.

Prior to the start of work, the Contractor's surveyors shall meet with the Owner to review survey procedures, methods and equipment to be used for the Contractor's surveys. Surveys shall conform to the minimum technical performance standards described in US Army Corps of Engineers Manuals EM–1110–1–1005 "Topographic Surveying" and EM 1110–2–1003 "Hydrographic Surveying". Surveys shall conform to the following maximum allowable tolerances:

a. Land Surveying: plus or minus 0.02 feet horizontal, and plus or minus 0.1 feet vertical.
b. Hydrographic Surveying: plus or minus 0.50 feet horizontal, and plus or minus 0.2 feet vertical.

Survey tolerances shall not accumulate.

BREAKWATER FOUNDATION – POTENTIAL SETTLEMENT
A geotechnical investigation has identified a soft clay layer under the existing breakwater and planned Wave Barrier. Settlement is not expected to occur during construction.

The contractor shall monitor settlement during construction using settlement plates or other approved methods. Contractor shall increase the volume of the breakwater core and/or underlayer rock as needed to account for settlement. The post–construction surveys must demonstrate that the as–built breakwaters meet the crest elevations and side slopes indicated on the drawings.

MEASUREMENT BY WEIGHT
Rocks shall be weighed, by barge or truck following the technical standards described in US Army Corps of Engineers Manuals EM 1110–2–2302 "Construction with Large Stone – Appendix C: Measurement for Payment" and as further described herein.

The method of measurement for determining the weight of rock delivered by truck shall be certified weigh bills provided by the supplier. Weigh bills and the scales used for weighing of trucks and materials contained therein shall have approval by the Owner or representative prior to notice to proceed is issued. Contractor shall submit a copy of the truck scale's certification of accuracy from the local weights and measures regulating agency. Contractor shall also submit weight bills, including certification of exact weight and time of weighing for each load of rocks delivered.

The method of measurement for determining the weight of rock delivered by barge shall be displacement of the barge, based on certified barge gage marks. Barge gage marks and certification shall have approval by the Owner or representative prior to notice to proceed is issued. Contractor shall submit a copy of the barge gauging table prepared by an accredited agent satisfactory to Owner. Contractor shall also submit weight bills, including certification of exact weight and time of weighing for each load of rocks delivered.

DESTINATION AND DELIVERIES
The Rock shall be delivered to the destination indicated on the bid schedule with all transportation. The supplier shall stockpile the Rock in accordance with its gradation. The contractor shall be responsible for all personnel and equipment to load, unload and stockpile the Rock. The exact location of the Rock stockpiles shall be as directed by the Owner

MISPLACED MATERIALS
Should the Contractor, during the execution of the work, lose, dump, throw overboard, sink or misplace any dredge material, dredge, barge, machinery, appliance, or other materials, the Contractor shall promptly recover and remove the same.

Figure VI-4-3. Selective placement (USACE, COASTAL ENGINEERING MANUAL)

Figure VI-4-2. Random placement (USACE, COASTAL ENGINEERING MANUAL)

HAINES BOROUGH ALASKA

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

P N D ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS
SCALE: SCALE IN FEET 0 8 16 FT.
DATE: 2/10/16

HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION

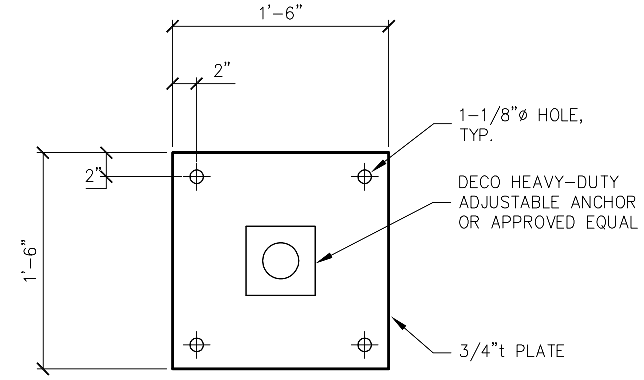
SHEET TITLE:
ROCK BREAKWATER & NOTES

PN&D PROJECT NO.: 102029.10

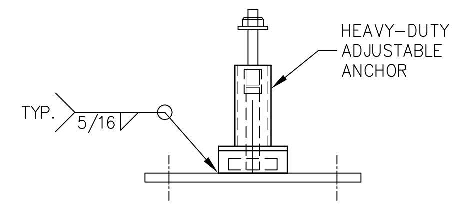
5.07
SHEET
OF

EXISTING NAV-AID STRUCTURE

EXISTING NAV-AID FOUNDATION



PLAN

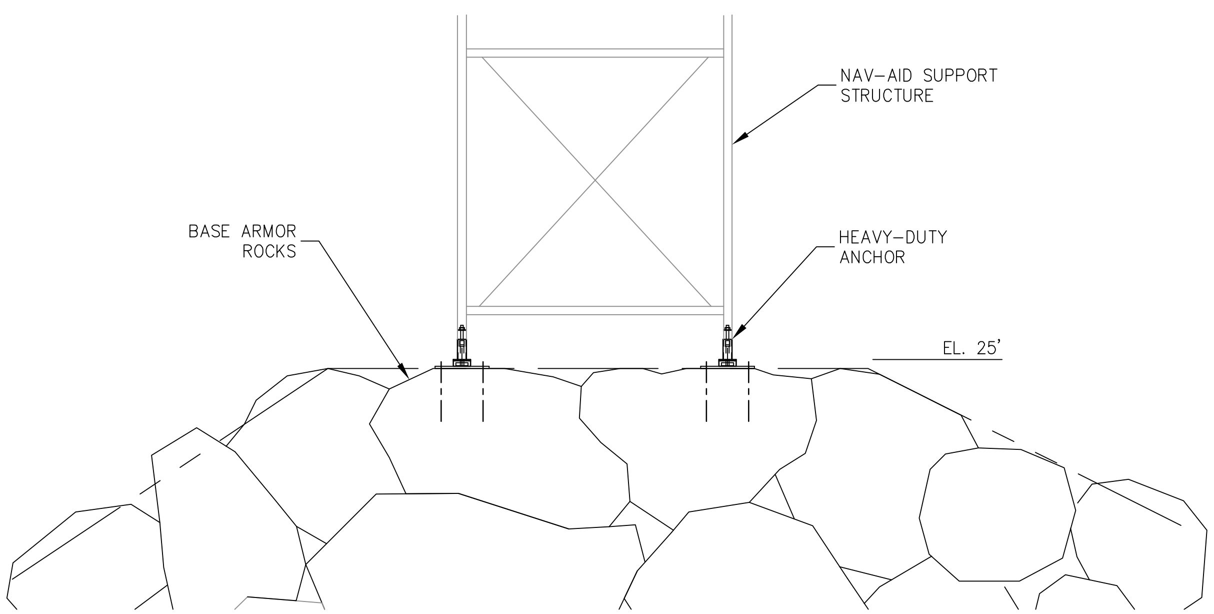


SECTION

BASE PLATE

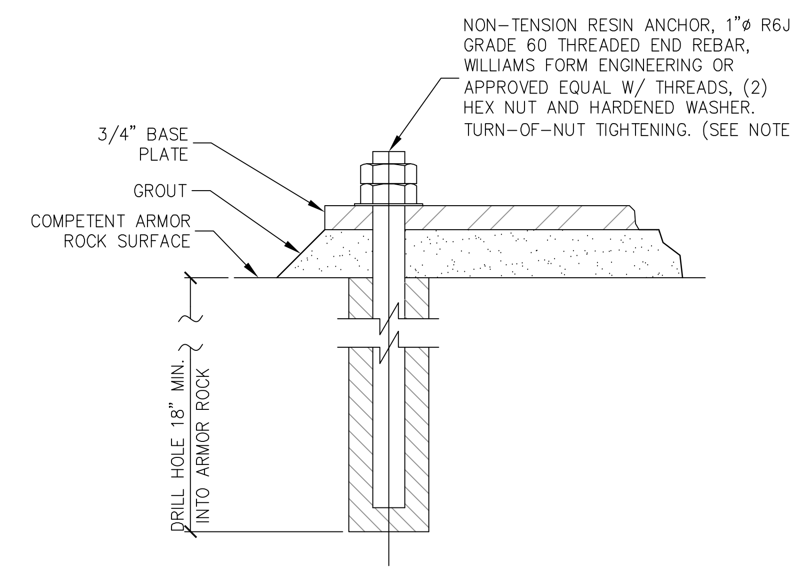
NOTES:

1) NAVIGATION AID BASE PLATES (4 EACH) SHALL BE PLACED ON ONE ARMOR ROCKS WHICH ARE SPECIALLY PLACED WITH A FLAT SIDE UP ORIENTATION. ARMOR ROCKS ON ALL SIDES SHALL BE PLACED TO PROVIDE MAXIMUM INTERLOCK AND STABILITY BETWEEN THE BASE AND THE ARMOR ROCK. NAVIGATION AID BASE PLATE SHALL BE HORIZONTAL, LEVEL AND SHALL BE ROCK BOLTED AND GROUTED INTO FLAT SIDE UP ARMOR ROCK.
2) RESIN ANCHORS SHALL BE EPOXY TYPE RESIN CARTRIDGES PER MANUFACTURER'S RECOMMENDATIONS. USE RESIN THE FULL LENGTH OF THE DRILL HOLE, CLEAN HOLE CAREFULLY BEFORE INSERTING RESIN. INSTALL IN STRICT COMPLIANCE WITH MANUFACTURER'S RECOMMENDATIONS.
3) BASE ARMOR ROCK SHALL BE THE LARGEST ROCK THAT CAN BE QUARRIED AND PLACED, WITH MINIMUM NOMINAL DIMENSIONS 4'x4'x3'.
4) ALL METALS AND HARDWARE SHALL BE HOT DIP GALVANIZED PER ASTM A123 OR A153 AS APPROPRIATE.
5) BOLTS SHALL BE ASTM A325. STEEL PLATE SHALL BE A MINIMUM ASTM A36.
6) GROUT SHALL BE PLACED PER MANUFACTURER'S RECOMMENDATIONS. GROUT SHALL BE NON-CORROSIVE, NON-METALLIC, CEMENT BASED GROUT MEETING ASTM C-1107, GRADE C, MEET THE REQUIREMENTS OF ASTM 520, AND DEVELOP A 28 DAY COMPRESSIVE STRENGTH OF 9,000 PSI.
7) SIZE OF HEAVY DUTY ADJUSTABLE ANCHOR SHALL MATCH BOLT SIZE AND THREAD OF EXISTING ANCHOR.
8) REMOVE EXISTING NAV-AID STRUCTURE, CLEAN, REPLACE EXISTING THREADED ROD WITH NEW AND RESTORE ON NEW FOUNDATION AT SAME LOCATION AND ORIENTATION, SOLAR PANEL FACING SOUTH.
9) APPLY ANTI-SEIZE COMPOUND TO THREADED ROD.



NAVIGATION AID ATTACHMENT

NOT TO SCALE



ROCK BOLT ANCHOR

NOT TO SCALE

R E V I S I O N S

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

BASE PLATE

PND ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS

SCALE: SCALE IN FEET 0 8 16 FT.

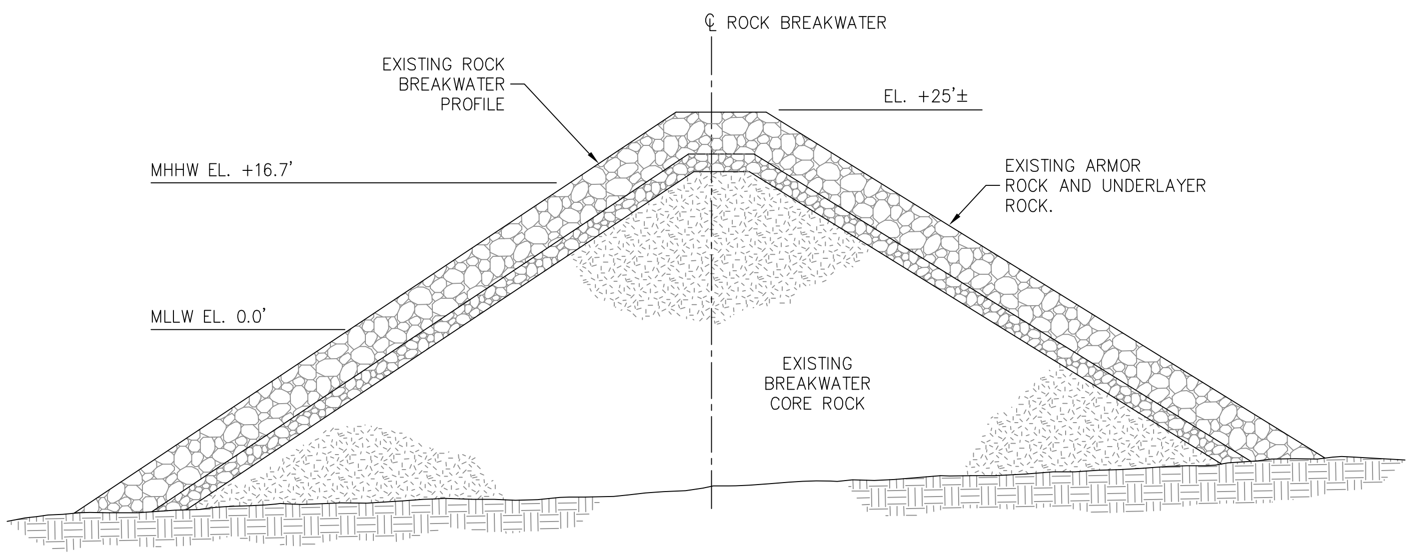
DATE: 2/10/16

HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION

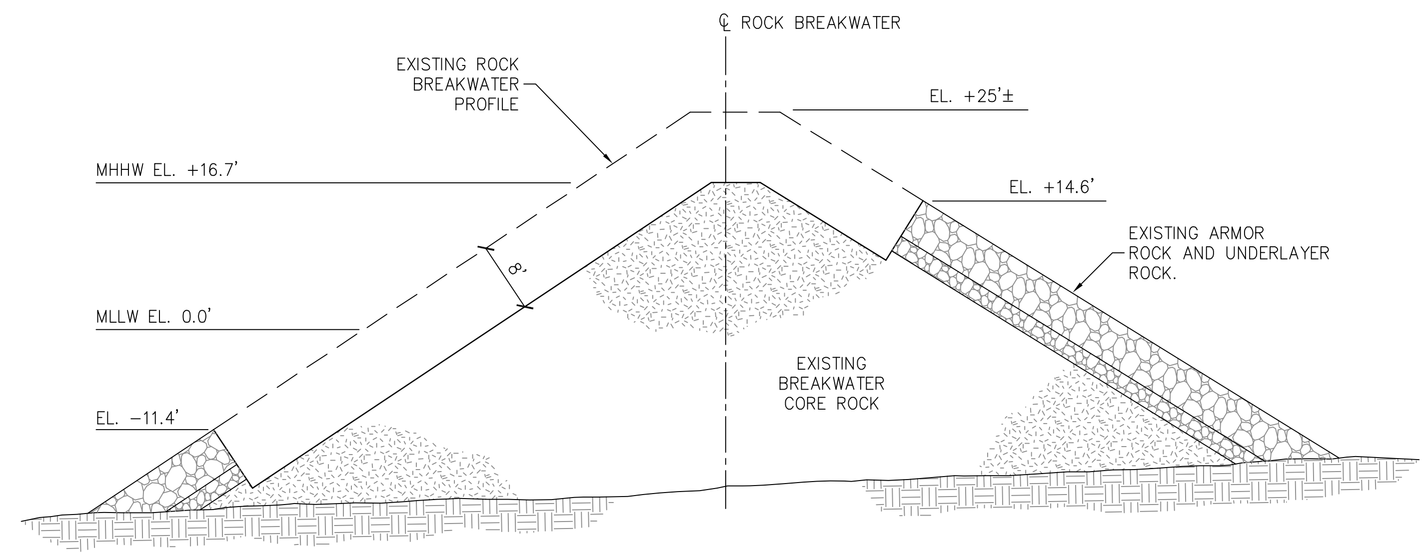
SHEET TITLE: NAVIGATION STRUCTURE DETAILS

PN&D PROJECT NO.:102029.10

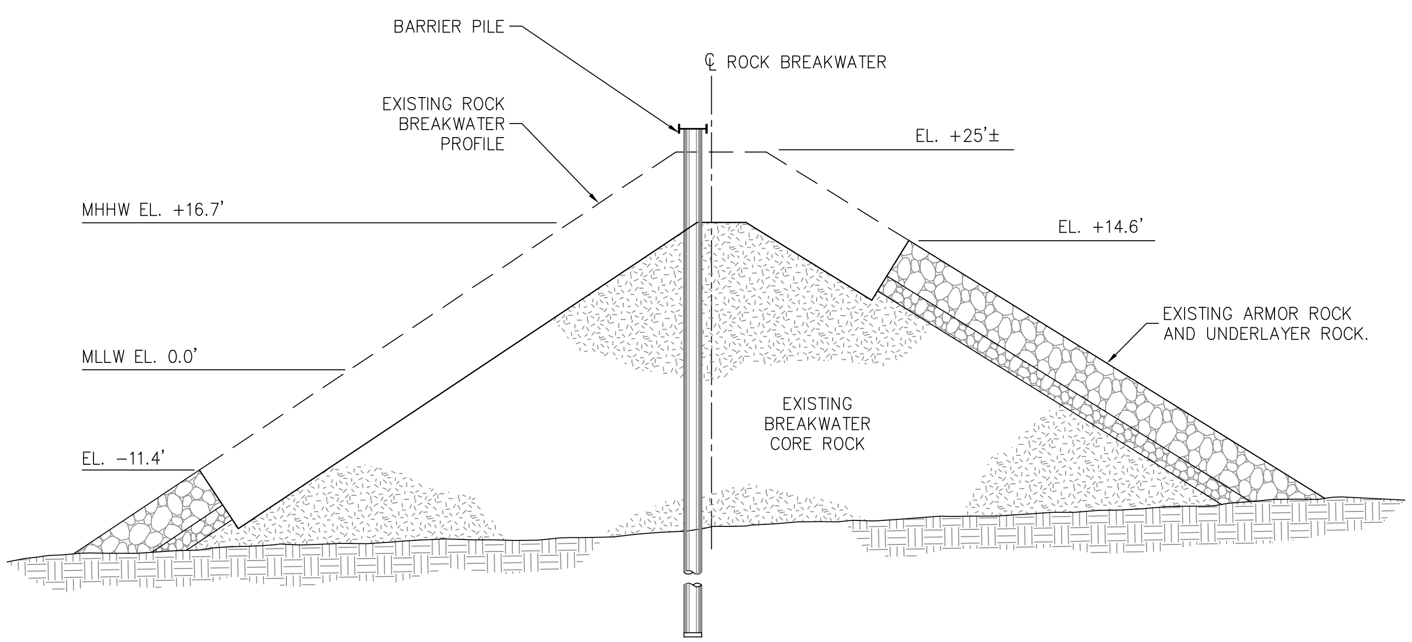
5.08
SHEET
OF



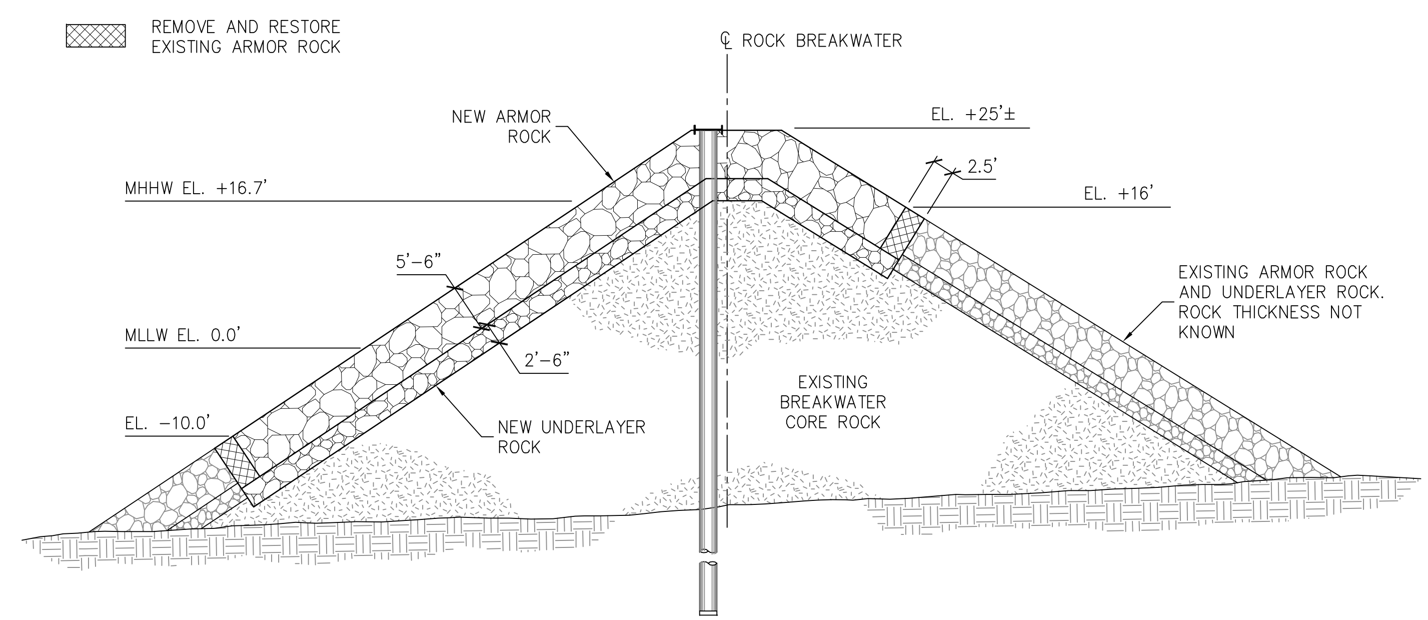
1 **EXISTING ROCK BREAKWATER**
NOT TO SCALE



2 **REMOVE EXISTING ROCK**
NOT TO SCALE



3 **INSTALL WAVE BARRIER**
NOT TO SCALE



4 **PLACE NEW ROCK**
NOT TO SCALE

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

P N D
ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

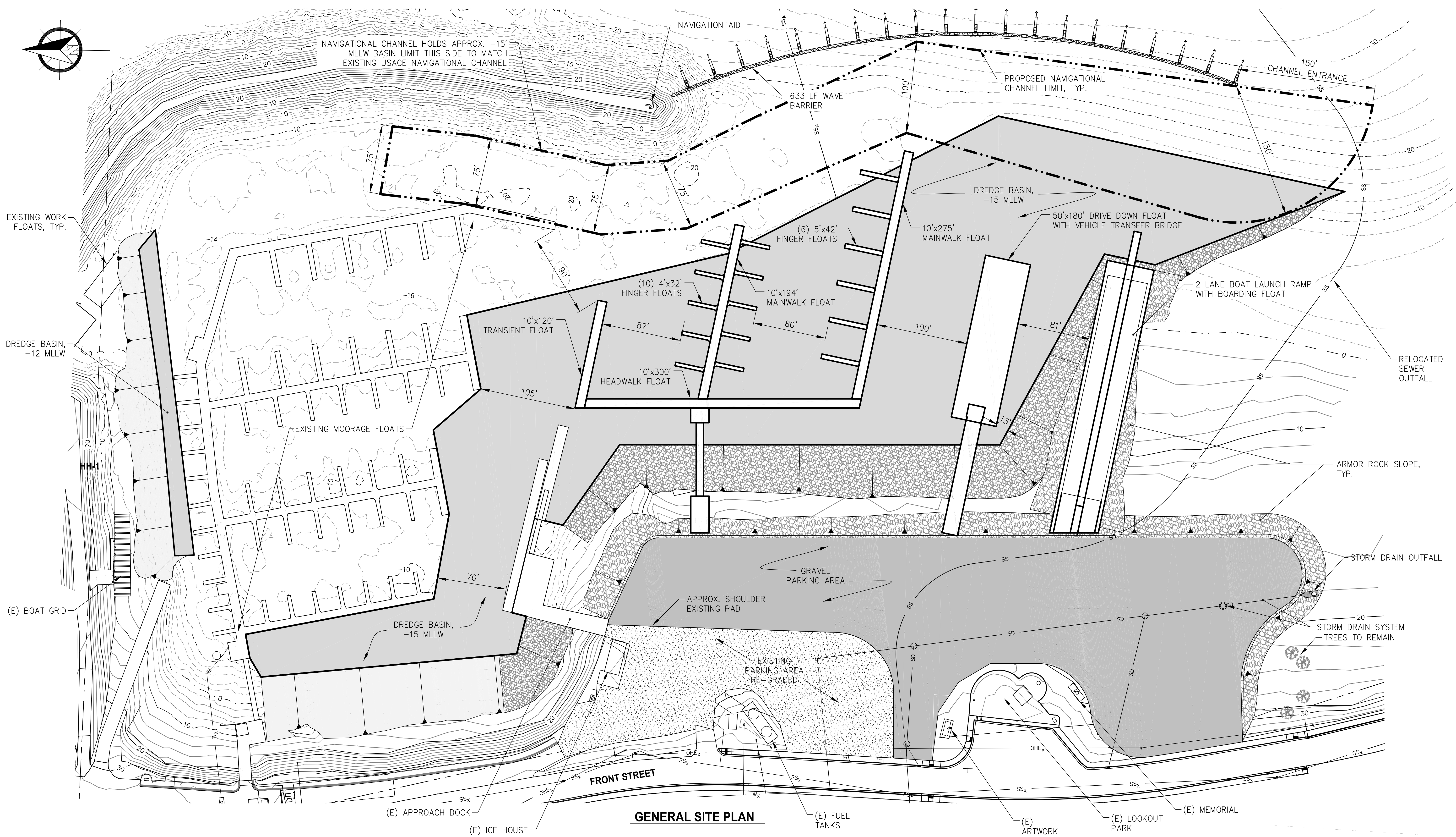
DESIGN: JDO CHECKED: CRS
DRAWN: DRH APPROVED: CRS
SCALE: SCALE IN FEET 0 8 16 FT.
DATE: 2/10/16

**HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION**

SHEET TITLE: **SUGGESTED CONSTRUCTION SEQUENCE**

PN&D PROJECT NO.: 102029.10

SHEET 1 OF 1



GENERAL SITE PLAN

HAINES BOROUGH ALASKA

REVISIONS

REV.	DATE	DESCRIPTION	DWN.	CKD.	APP.

PND ENGINEERS, INC.
9360 Glacier Highway, Ste. 100
Juneau, Alaska 99801
Phone: 907-586-2093
Fax: 907-586-2099
www.pndengineers.com

DESIGN: TCB CHECKED: CRS
DRAWN: PJD APPROVED: CRS

SCALE: SCALE IN FEET 0 50 100 FT.

DATE: 03/15/16

**HAINES BOROUGH
PORTAGE COVE
HARBOR EXPANSION**

SHEET TITLE: **NAVIGATIONAL CHANNEL**

PND PROJECT NO.: 102029

1.00

SHEET 1 OF 1

PND ENGINEERS, INC.

MEMORANDUM

To: Dick Somerville, P.E., Principal

Date: Feb 29, 2016
Project No: 102029

From: Steven Halcomb, P.E., G.E., Senior Geotechnical Engineer

Subject: Haines Harbor Existing Rubble Mound Breakwater Post-Construction Slope Stability

Introduction

The Haines Borough Portage Cove Harbor Expansion current design consists of a partial penetrating wave barrier that will be installed as an extension of the existing Corp of Engineers (COE) rubble mound breakwater. In support of the design, PND has performed a static end-of-construction (EOC) slope stability analysis of the existing breakwater. This memo summarizes the results of that analysis.

Loading and Geometry

The proposed partial penetrating wave barrier will require piles to be installed into the existing nose of the breakwater. This will require removal of the existing armor rock and underlayer rock. The piles will then be installed to the required depths. On completion of the pile installation, new armor and underlayer rock will then be placed to return the breakwater to the approximated existing conditions.

The existing breakwater geometry was modeled with subsurface conditions interpreted based on the geotechnical report for the project by PND dated March 2015. The presence of the new piles will prevent several potential failure planes with respect to the existing nose of the breakwater with the critical failure plane occurring approximately perpendicular to the breakwater therefore this cross section was modeled.

Geotechnical Soil Parameters

The general lithology of the site consists of three layers of variable thicknesses:

- Surficial poorly graded sand
- Cohesive sediment of lean clay
- Alternating stratum of matrix-supported sediments and cohesionless poorly graded sand and gravel

A summary of the effective and total strength soil properties are found in Table 1.

Table 1: Soil Properties

Layer	Unit Weight pcf	c' psf	ϕ' degrees	s_u psf
Sand	125	0	34	0
Gravel	125	0	36	0
Breakwater	145	0	46	0
Lean Clay	120	0	32	SHANSEP

The soil properties for the lean clay were modeled considering three zones:

- Zone 1 - directly beneath the existing breakwater

- Zone 2 - areas beneath the side slopes of the breakwater
- Zone 3 - the clay outside the breakwater.

The unloading of a portion of the breakwater will cause the lean clay, currently in a consolidated state beneath the breakwater, to become lightly overconsolidated. An overconsolidated ratio (OCR) profile was estimated for the two zones beneath the breakwater based on elastic theory and the undrained shear strength was modeled following the Stress History and Normalized Soil Engineering Properties (SHANSEP). The SHANSEP relationship was developed by GeoEngineers as found in their memo in Appendix G of the Geotechnical Report (PND, 2015).

The three predominant failure modes in clay, compression, direct simple shear, and extension, were modeled in accordance with the SHANSEP method by altering the "S" coefficient for each shear mode. PND assumed the appropriate "S" for each failure mode based on conservative assumptions from literature relationships as no specific testing for each failure mode was performed. A summary of the SHANSEP parameters are found in Table 2.

Table 2: SHANSEP Parameters

Zone	Failure Mode	S	m
1	Compression (TXC)	0.15	0.80
2	Direct Simple Shear (DSS)	0.12	0.80
3	Extension (TXE)	0.075	0.80

Slope Stability Analysis

The EOC slope stability analysis was performed using the commercially available program SLIDE v. 7.0 produced by Rocscience (www.rocscience.com/). SLIDE is a 2-dimensional limit equilibrium slope stability program with new soil models that include the SHANSEP relationship. The existing breakwater that is not to be removed was considered in the computations as providing effective stress to the SHANSEP relationship as well as in-situ soil layers. The portion of the removed/replaced portion of the breakwater was considered to be only contributory to the driving forces of the stability analysis. Both directions, towards and away from Haines Harbor, were considered in the analysis.

Failure planes are generated by selecting a method (Bishops, Janbu, Spencer, Morgenstern-Price, Ordinary Method of Slices, etc.) and discretizing the failure plane into a series of slices in which the forces on each slide are computed. The ratio of forces driving slope movement with forces resisting slope movement are presented as the resulting factor of safety (FS) which is the same for each slice and in turn, for the entire failure plane. Trial failure surfaces are then sorted to identify the surface with the lowest FS, or the "critical" failure surface.

The critical failure plane was determined considering circular and non-circular failure planes using Spencer's method. The Morgenstern-Price method using a half sine interslice function was also computed as a comparison to Spencer to ensure results were consistent with other methods though results from other methods are not presented here. The critical failure plane was determined to be a non-circular, path search failure plane. Optimization was performed and the resulting failure plane was reviewed, found to be reasonable, and therefore the "critical" failure plane is concluded to be an optimized, non-circular failure plane.

Results and Conclusion

PND determined the EOC FS to be 1.34 and the results are presented in the attached Figure. The results and conclusion of the analysis are that a required FS of 1.3 is achieved therefore the existing breakwater is deemed stable for the temporary condition of installation of the piles and reconstruction of the armor rock.

References:

Haines Borough Portage Cove Expansion 95% Plan Set dated 2/10/16.

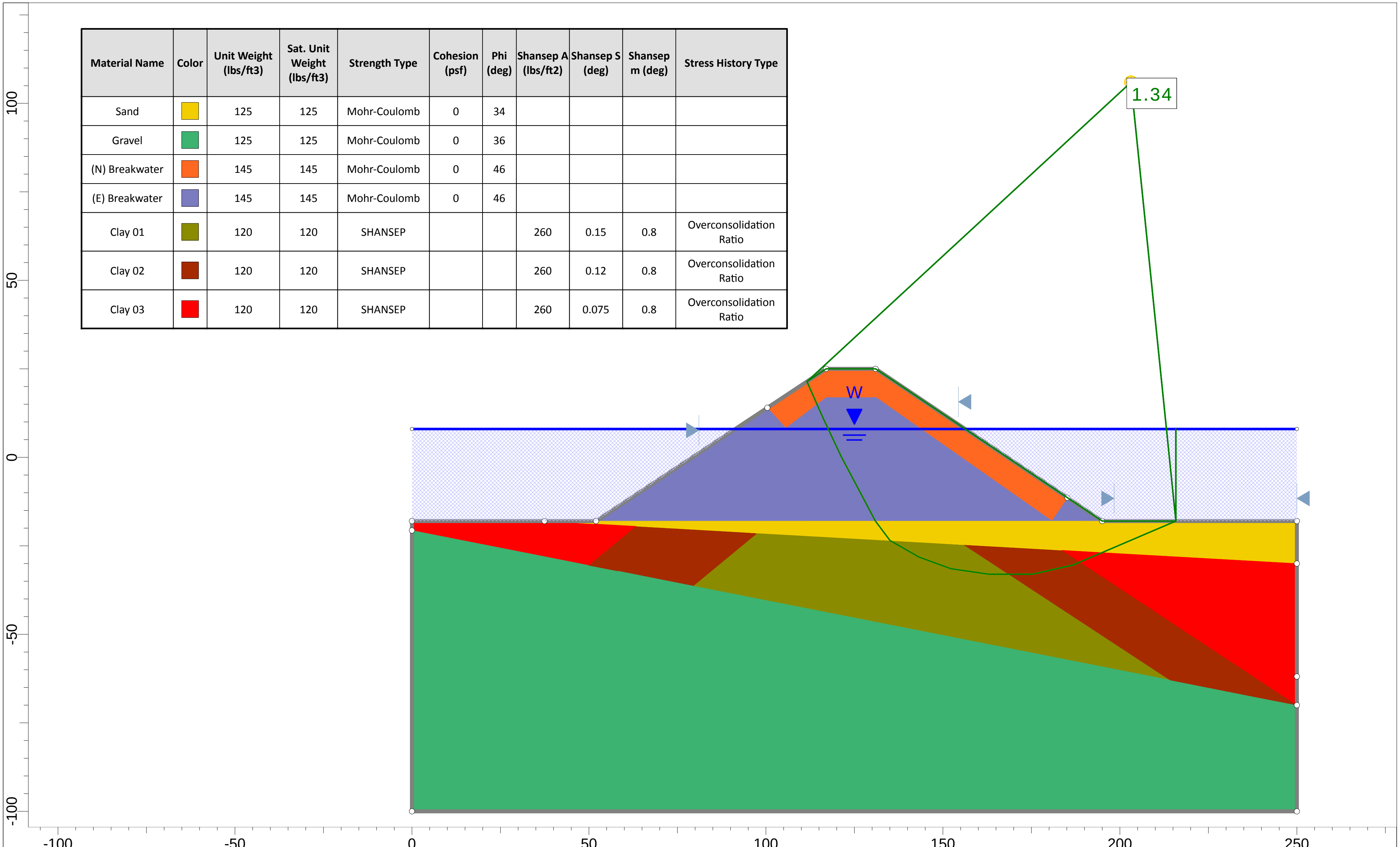
South Portage Cove Harbor Expansion Geotechnical Engineering Report, March 2015. Prepared for the Haines Borough by PND Engineers, Inc.

Rocscience, Inc. (2016). www.rocscience.com/

ATTACHMENTS:

- *SLIDE Output*

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Shansep A (lbs/ft2)	Shansep S (deg)	Shansep m (deg)	Stress History Type
Sand		125	125	Mohr-Coulomb	0	34				
Gravel		125	125	Mohr-Coulomb	0	36				
(N) Breakwater		145	145	Mohr-Coulomb	0	46				
(E) Breakwater		145	145	Mohr-Coulomb	0	46				
Clay 01		120	120	SHANSEP			260	0.15	0.8	Overconsolidation Ratio
Clay 02		120	120	SHANSEP			260	0.12	0.8	Overconsolidation Ratio
Clay 03		120	120	SHANSEP			260	0.075	0.8	Overconsolidation Ratio



PND ENGINEERS, INC.

SLIDEINTERPRET 7.011

Project	SPCHE Rubble Mound Stability Analysis
Analysis Description	End of Construction
Drawn By	SH
Company	PND Engineers, Inc.
Date	2/22/2016, 12:11:25 PM
File Name	Existing Breakwater EOC.slim

Portage Cove Harbor - Wave Barrier Extension Cost Estimate

Prepared by PND, March 15, 2016

Item	Item Description	Units	Quantity	Unit Cost	Amount
1505.1	Mobilization	LS	All Req'd	$ 21,663	$ 21,663
2896.2	Furnish & Install Wave Barrier Pile, 24 Inch Dia. X 0.500 Inch Thick w/ Sheetpile Wings	EA	5	$ 34,575	$ 172,875
2901.1	Furnish & Install Barrier Waler	LF	25	$ 550	$ 13,750
2702.01	Surveying, Re-Establish Monument	LS	All Req'd	$ 15,000	$ 15,000
2901.1	Remove & Reinstall Navigation Aid Structure	LS	All Req'd	$ 15,000	$ 15,000
	Estimated Construction Price				$ 238,288
	Contingency (10%)				$ 23,829
	Design (5%)				$ 11,914
	Contract Administration & Construction Inspection (7%)				$ 16,680
	Total Recommended Project Budget				$ 290,711