EXECUTIVE SUMMARY

This Basis of Design establishes the criteria necessary for design of the Wave Barrier structure for the South Portage Cove Harbor Expansion Project.

The design depicted on the drawings has been developed in conjunction with the Haines Borough Assembly.

Design calculations have been performed which demonstrate the adequacy of the design. A complete version of these calculations is found in the following sections.

Design calculations, drawings, and specifications were prepared under the direction of a registered civil Engineer licensed in Alaska.

SOUTH PORTAGE COVE HARBOR EXPANSION

1. GENERAL OVERVIEW

Portage Cove Harbor is the only protected harbor servicing Haines, Alaska. User growth over the years has made congestion an issue. Navigation improvements within the harbor need consideration for larger commercial-size vessels due to the Harbor's constrained size and shallow basin depth. Protection of the harbor from excessive wave action needs to be addressed as Portage Cove is exposed to broad, open marine waters of Lynn Canal that can funnel storm-generated waves into the harbor entrance with detrimental effects to harbor infrastructure and moored vessels.

The Haines Borough is planning to implement a phased approach for improvements to the harbor with the following primary objectives:

- Expansion in moorage capacity with improved navigation for larger vessels
- Improved protection from excessive wave action
- Enlarging the upland boat launch parking areas and waterfront accessibility.

One of the principle objectives for the harbor expansion is to provide wave protection at the harbor entrance. A partially penetrating steel wave barrier was selected to meet this task.

1.1 Partial Penetrating Wave Barrier

The partial penetrating wave barrier consists of PS31 steel sheet piles welded to 24" diameter steel pipe barrier piles and supported by 30" diameter bearing piles. The steel sheet piles are suspended above the sea bottom absorbing most of the incoming wave energy while allowing for water circulation, movement of bottom-dwelling sea life, and reduction in applied forces to the wall.

2. REFERENCES

2.1 Reports/Studies/Background Drawings

- Geotechnical Report. (March 2015). *Haines Borough South Portage Cove Harbor Expansion.* PND Engineers, Inc.
- Gilman and Kriebel. (1999). *Partial Depth Pile Supported Wave Barriers: A Design Procedure.* PND Engineers, Inc.
- Harbor Protection Alternatives. (August 21, 2013). *Haines Borough South Portage Cove Harbor Expansion.* PND Engineers, Inc.

2.2 Applicable Codes and Standards

- International Code Council "International Building Code 2012 (IBC-12)
- American Society of Civil Engineers "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10)
- American Institute of Steel Construction (AISC), "Specification for Structural

Steel Building" (AISC 360-10).

- American Society for Testing and Materials (ASTM) Standards, current edition.
- American Welding Society (AWS), "D1.1 Structural Welding Code – Steel, D1.1-10, 2010".

3. SITE LAYOUT

The curved horizontal layout of the wave barrier is based on an established work point Northing and Easting and a radius to the centerline of barrier piles and vertical bearing piles.

Vertical datum is established with a monument at the end of the existing rock breakwater.

4. SITE GEOTECHNICAL CONDITIONS

4.1 Bathymetry

The bathymetry for the project was taken from Haines Harbor Condition Survey performed in 2008 and 2011 by the United States Arm Corp of Engineers Alaska District (USACE).

4.2 Design Mudline

Based on the provided bathymetry, a mudline elevation of minus 30-ft MLLW is assumed for the wave barrier design mudline elevation with minor variations along its length. The North end of the wave barrier will intersect with an existing rock breakwater.

4.3 L-Pile Parameters

L-Pile design parameters are provided in PND's Geotechnical Report (Geotechnical Report, March 2015). Table 4-1 describes the soil profile parameters assumed in L-Pile for the Wave Barrier analysis (Geotechnical Report, March 2015).

Table 4-1. Soil Profile for Design of Wave Barrier Piles

Soil Layer	Soil Type	Approx. Depth Below Mudline (feet)	Effective Unit Weight (pcf)	Internal Angle of Friction ϕ' (degrees)	Undrained Shear Strength s_u (psf)	*p-y* Curve Model	*p-y* Modulus (static) K [1] (pci)	Strain Factor ε_{50} [1]
1	Loose Sand	0 to 2	--	--	--	--	--	--
2	Loose to Med. Dense Sand	2 to 10	56	30 top 34 bottom	--	Sand (Reese)	75	--
3	Soft to Very Soft Clay	10 to 35 south 10 to 50 north [2]	56	--	3.75 d +250 [3]	Soft Clay (Matlock)	--	0.020
4	Dense to Very Dense Sand	> 35 south > 50 north [2]	61	36	--	Sand (Reese)	125	--

Notes:

[1] Alternatively, use LPile program defaults for K and/or ε_{50}.

[2] Depth to bottom of clay layer varies along the wave barrier alignment. Based on boreholes B-14, B-10, and BH-4, the clay layer is approximately 25 feet thick and 40 feet thick at the south

and north ends of the breakwater alignment respectively (linearly interpolate linearly along alignment to estimate clay thickness).

[3] c′ = 3.75 × d + 250 psf; where d = depth from top of clay layer in feet.

By inspection, it was determined that using the parameters under the far North conditions (i.e. clay layer ~40-ft) would provide a conservative design assumption for the barrier pile and bearing pile design.

4.4 Pile Soil Interaction – Axial Capacity Charts

4.4.1 *Barrier Piles*

Barrier piles will be 24”ϕx0.500”t steel pipe piles with no pile tip accessory. Axial capacity charts for the barrier piles are provided in Appendix B of the Geotechnical Report (Geotechnical Report, March 2015).

4.4.2 *Bearing Piles*

Bearing piles will be 30”ϕx0.750”t piles equipped with a SPIN FIN® pile tip. Axial capacity charts for the bearing piles are provided in the design calculations. An average uncorrected SPT N-value of N=45 and a total unit soil unit weight of 125 pcf is used for design as recommended in section 4.1.2.2.2 of the Geotechnical Report (Geotechnical Report, March 2015).

5. MET-OCEAN

Met-ocean data per Met-Ocean Analysis by PND Engineers, Inc. (Harbor Protection Alternatives, August 21, 2013)

5.1 Tide and Water Levels

The water elevations listed in Table 5-1 are from NOAA tide information for Juneau and Haines.

Table 5-1. Tide and Vertical Datum

	Haines EL. (feet, MLLW)	Juneau EL. (feet, MLLW)
Highest Observed Water Level (11/2/1948)	-	24.4
Mean Higher High Water (MHHW)	16.8	16.3
Mean High Water (MHW)	15.8	15.3
Mean Tide Level (MTL)	-	8.5
Mean Low Water (MLW)	-	1.6
Mean Lower Low Water (MLLW)	0.0	0.0
Lowest Observed Water Level (1/1/1991)	-	-5.4
Extreme Low Water (NOAA chart 17317)	-6.0	-

5.2 Wave and Wind Speed Design Criteria

Table 5-2 and Table 5-3 establish the design criteria used for the design of the wave barrier structure.

Table 5-2. Design Operational Criteria (2-Yr Return Period)

Wind Direction	Water Elevation (feet, MLLW)	Wind Speed (knots)	Wave		Max. Wave Transmission, H_T (feet)
			Hs (feet)	Tp (s)	
Northeast (050º)	MHHW: 17.0 MLLW: 0.0	31	2.6	2.5	1.0
East (090º)			2.1	2.2	
Southeast (120º)			2.5	2.4	

Table 5-3. Design Environmental Criteria (50-Yr Return Period)

Wind Direction	Water Elevation (feet, MLLW)	Wind Speed (knots)	Wave		Max. Wave Transmission, H_T (feet)
			H_s (feet)	T_p (s)	
Northeast (050º)	MHHW: 17.0 MLLW: 0.0	68	6.5	4.3	2.0
East (090º)			6.9	4.4	
Southeast (120º)			6.3	4.3	

6. WAVE BARRIER DESIGN ELEVATIONS

6.1 Top of Wall

The top of wall elevation is set at +25-ft MLLW. Following PND's design procedure (Gilman and Kriebel, 1999), the top of wall elevation was established to withstand a 50-yr event at MHHW water elevation.

6.2 Bottom of Wall

The bottom of wall elevation is set at -19-ft MLLW. Following PND's design procedure (Gilman and Kriebel, 1999), it was determined that the minimum bottom wall elevation (draft) needed to be established at -17-ft MLLW in order to meet the wave transmission criteria for a 50-yr event at MLLW water elevation.

In addition to the criteria established by PND's design procedure (Gilman and Kriebel, 1999), consideration was given to include the effects of glacial rebound/uplift and sea level change. The Haines region is experiencing glacial rebound/uplift at the approximate rate of 0.9" per year or 3.75-ft in 50 years (Geotechnical Report, March 2015). At the same time, it is anticipated that the Haines region will experience a sea level rise of approximately 1.25-ft in 50 years (Harbor Protection Alternatives, August 21, 2013). The net difference between the glacial rebound/uplift and sea level change equals 2.5-ft.

To keep the sheet pile lengths at an economical length, an additional 2-ft of sheet pile length was added the wave barrier wall bottom elevation. (i.e. -17-ft minus 2-ft = -19-ft MLLW).

7. DESIGN LOAD CASES

7.1 Dead Load – "D"

The dead load shall consist of the weight of all materials of construction.

7.2 Wave Load – "WV"

Wave loads on the wave barrier are determined according to PND's design procedure (Gilman and Kriebel, 1999).

Resultant critical wave load due to 50-yr event at a MHHW: WL_{50} = 10.4 kips per ft at elevation +7.4-ft.

7.3 Wind Load – "W"

N/A - Wind loads on the wave barrier will not exceed the design wave load criteria. As such, wind loads are ignored for design simplicity.

7.4 Seismic Load – "E"

N/A - Seismic loads on the wave barrier will not exceed the design wave load criteria. As such, wind loads are ignored for design simplicity.

7.5 Fatigue – "F"

Fatigue is considered for critical elements on the wave barrier structure. The stress range conservatively assumed for fatigue considers the wave forces due to a 2-yr storm event which occurs 3% of the time over a 50-yr life cycle or approximately 11 days per calendar year. Based on the period of the 2-yr storm wave, the number of cycles is calculated at 1.89×10^7 cycles.

7.6 Load Combinations

Table 7-1. Load Combinations using Allowable Stress Design

Load Combination	D	WV	F
1	1.00	1.00	-
2	0.60	1.00	-
3	-	-	1.00

Table 7-2. Load Combinations using Load and Resistance Factor Design

Load Combination	D	WV	F
1	1.20	1.60	-
2	0.90	1.60	-
3	-	-	1.00