APPENDIX M
FUTURE INDEPENDENT RESTORATION ACTIONS

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TABLE OF CONTENTS

1 INTRODUCTION M-1
1.1 Overview of Restoration Concepts M-1

2 INTERTIDAL HABITAT CREATION BY UPLAND FILL REMOVAL M-2
2.1 Restoration Concept M-2
2.2 Potential Cap Design Modifications M-3
2.3 Operations, Monitoring, and Maintenance M-4

3 JETTY REMOVAL M-5
3.1 Project Description M-5
3.2 Potential Cap Design Modifications M-5

4 REFERENCES M-6

List of Tables

Table M-1 Summary of Cleanup and Restoration Cap Designs M-4

List of Figures

Figure M-1 Existing Conditions
Figure M-2 HCCC South Shoreline Mitigation and Restoration Design
Figure M-3 HCCC South Shoreline Mitigation and Restoration Design Cross Sections
Figure M-4 North Shoreline Mitigation and Restoration Design and Intertidal Cap Armor Sizes

List of Attachments

Attachment M-1 Wave Modeling in Support of Restoration Design

1 INTRODUCTION

The main body and other appendices of this *Engineering Design Report* (EDR) describe the approach and criteria for the engineering design of sediment cleanup actions in Port Gamble Bay that will be implemented in accordance with the requirements of Consent Decree 13-2-02720-0 between the Washington State Department of Ecology (Ecology) and Pope Resources, LP/OPG Properties Group, LLC (PR/OPG). Possible nearshore/shoreline restoration options at the former Mill Site, coordinated with but separate from the cleanup project, are also being considered by PR/OPG, Hood Canal Coordinating Council (HCCC), Port Gamble S'Klallam Tribe (PGST), and other stakeholders. This appendix to the EDR describes how concurrent or future habitat restoration projects at the former Mill Site, if implemented, would affect the design and long-term monitoring and maintenance of the cleanup project.

1.1 Overview of Restoration Concepts

Several different restoration concepts have been contemplated for the former Mill Site. This appendix specifically addresses two of these broad concepts:

1. Intertidal habitat creation by excavating upland fill to a flatter shoreline grade than required by the cleanup
2. Partial or full jetty removal northwest of sediment management area 1 (SMA-1)

HCCC and PGST are evaluating intertidal habitat creation options in the southern and northern portions of the former Mill Site, respectively. PGST is also evaluating options for partial or full rock jetty removal adjacent to SMA-1. If implemented, these restoration actions would be permitted separate from the cleanup project, but would use protective designs to ensure that cleanup requirements are achieved, as described herein.

2 INTERTIDAL HABITAT CREATION BY UPLAND FILL REMOVAL

2.1 Restoration Concept

The overall objectives of upland fill removal to create intertidal habitat include the following:

- Increase the amount of intertidal and shallow subtidal habitat acreage
- Restore shorelines in the former mill area to more naturally sloped conditions supported by riparian vegetation to provide habitat for forage fish, shellfish, and juvenile salmonids
- Improve natural sediment transport processes

The current HCCC and PGST concepts include flattening the post-remediation shoreline slope of 3 horizontal to 1 vertical (3H:1V) as follows:

- Between top of bank and elevation +13 feet mean lower low water (MLLW): 8H:1V
- Between +13 and +5 feet MLLW: 10H:1V
- Between +5 and 0 feet MLLW: 15H:1V

Figure M-1 summarizes existing topography and bathymetry at the former Mill Site. Figure M-2 depicts a concept of how this option could be developed along the east-facing shoreline at the south end of the Mill Site uplands, in the area currently being considered for mitigation/restoration by HCCC. A similar approach could be used in the area currently being considered for restoration by the PGST. Figure M-3 presents representative cross sections illustrating the pre- and post-restoration conditions for this broad restoration concept, including a depiction of how the restoration overlays on the planned cleanup excavation (to be performed by PR/OPG) along the intertidal banks.

The conceptual designs presume that excavated material could be beneficially reused in upland areas at the former Mill Site. Details of the design evaluation for the beneficial reuse of materials in upland areas are discussed in the *Habitat Embankment Basis of Design* (Anchor QEA 2015). If implemented, additional details of beneficial reuse plans will be described in separate permitting documents.

2.2 Potential Cap Design Modifications

As discussed in more detail in Appendix B of the EDR, engineered caps within existing intertidal and subtidal areas were designed to ensure protectiveness. This section summarizes how the cleanup cap design would be modified if separate restoration actions, implemented either concurrent with or following the cleanup project, result in the creation of additional intertidal habitat in the former Mill Site area.

The cleanup cap design evaluated in the EDR consists of a mixture of sand, gravel, and cobble, similar to the existing sediment substrate present in the former wharf area and throughout much of the intertidal area. The cleanup design excavation, if implemented independent of restoration, will achieve final slopes of 3H:1V or flatter. Following intertidal cleanup excavation, material will be placed to create stable sloping caps. Intertidal caps will typically be composed of two layers—a sand and gravel filter material, overlain by gravel or larger-sized armor—to isolate underlying sediments and concurrently provide erosion protection primarily from wind/wave forces. The cleanup cap will consist of a minimum 6-inch thickness of sand and gravel filter material and a minimum 12-inch thickness of armor material. The cleanup design cap types (i.e., Cap Types 2 and 3), minimum thicknesses, and typical particle size (d_{50}) are summarized in Table M-1.

As discussed in Section 2.1, the current HCCC and PGST concepts include flattening the intertidal slope to 10H:1V or flatter. Because the restoration excavations will still be completed in historic upland fill containing concentrations of chemicals of concern that exceed sediment cleanup levels (Anchor QEA 2015), the restoration excavations will need to be capped to ensure the long-term protectiveness of the integrated cleanup/restoration action. Current habitat restoration concepts include placement of 2 feet of mixed sand and gravel overlain by 2 feet of habitat substrate material. Additional wave modeling was conducted by PGST (Attachment M-1) to support refined designs of the integrated cleanup/restoration actions, including the minimum cap thickness and typical particle size (d_{50}). The results of this engineering design evaluation are summarized in Table M-1.

Table M-1
Summary of Cleanup and Restoration Cap Designs

Action	Cap Description	Post-removal Slope	Cap Grain Size
Remediation without Restoration	• Minimum 12-inch to 18-inch thickness of armor material overlying • Minimum 6-inch thickness of sand and gravel filter material	3H:1V or flatter	Armor d_{50} particle size range: 2.5 inches (Cap Type 3), and 9 inches (Cap Type 2)
Integrated Restoration and Remediation	• Minimum 24 inches of habitat substrate material overlying • Minimum 24 inches of mixed sand and gravel isolation cap	10H:1V or flatter	Isolation cap layer d_{50} particle size: 2 inches

2.3 Operations, Monitoring, and Maintenance

The refined wave modeling summarized in Attachment M-1 revealed that where gravel habitat substrates are used to construct the intertidal habitat restoration surface, beach profile changes can be expected within the surf zone due to storm wave attack. During and following peak wave conditions, substrate would locally move up and down the slope due to wave energy; habitat substrate material could develop a profile with a trough depth of up to 16 to 18 inches from the post-construction surface grade. However, even under these peak wave conditions, the bottom of the trough would still have at least 30 inches of habitat substrate and cap material overlying contaminated sediments.

The *Operations, Maintenance, and Monitoring Plan* (OMMP; Appendix F of the EDR) describes monitoring and maintenance requirements for cleanup caps to ensure their continued protectiveness. Monitoring includes bathymetric surveys to verify that the cap surface profile maintains its long-term integrity, which is defined as a minimum of 18 inches of cap material overlying contaminated sediment. Details of the bathymetric monitoring and contingency actions are described in the OMMP.

3 JETTY REMOVAL

3.1 Project Description

The overall objective of partial or full jetty removal would be to improve sediment transport processes and circulation within Port Gamble Bay. Figure M-4 presents conceptual plan views of the partial and full jetty removal options that are currently being considered by PGST. Briefly, the existing rock would be removed until native subgrade is reached, and the rock would be beneficially reused in an upland or nearshore application as practicable.

3.2 Potential Cap Design Modifications

This section summarizes how the cleanup cap design would be modified if the existing rock jetty were to be removed in full or in part. The specifications of the cleanup design cap types (i.e., Cap Types 1, 2, or 3) as well as the minimum thicknesses and armor sizes are equivalent under the cleanup-only, partial jetty removal, or full jetty removal scenarios (minimum 12-inch to 18-inch thickness of armor material overlying a minimum 6-inch thickness of sand and gravel filter material). The existing jetty creates a “shadow” that reduces energy in areas of SMA-1, which allows the use of smaller cap armor substrate. Removal of the jetty will increase the wave energy along the shoreline, necessitating the use of larger armor stone over bigger areas to resist the increased wave energy. Thus, the extents of Cap Types 1, 2, and 3 vary under the different jetty removal scenarios, as summarized in Figure M-4.

4 REFERENCES

Anchor QEA, LLC, 2015. *Habitat Embankment Basis of Design, Port Gamble Bay*. Prepared for Pope Resources, LP/OPG Properties, LLC. March 2015.

FIGURES

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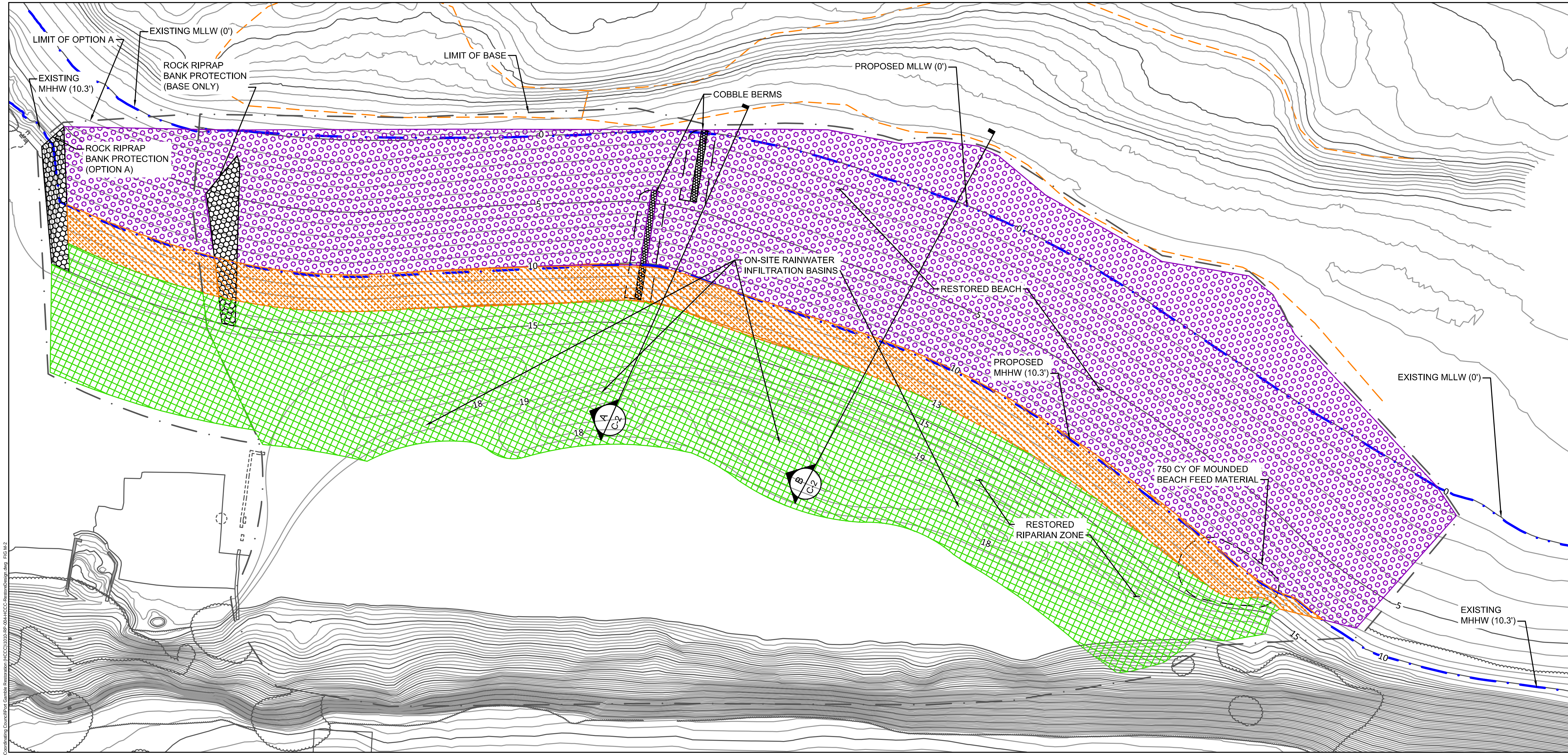
Figure M-1
Existing Conditions
Appendix M: Integrated Restoration and Cleanup Actions
Port Gamble Bay Cleanup



LEGEND:

PROJECT LIMITS	COBBLE BERM
CONTOUR IN FEET	NATIVE RIPARIAN PLANTING
EXISTING EELGRASS BOUNDARY	NATIVE BEACH GRASS PLANTING
ROCK RIPRAP	
BEACH SAND AND REMEDIAL CAP	
HABITAT SUBSTRATE AND REMEDIAL CAP	

NORTH

0 40
Scale in Feet

NOTES:
1. HORIZONTAL DATUM: WASHINGTON STATE PLANE NORTH ZONE, NAD 83 FEET.
2. VERTICAL DATUM: MEAN LOWER LOW WATER (MLLW)
3. SURVEY BY TRIAD AND ETRAC, DATED AUGUST 27, 2014

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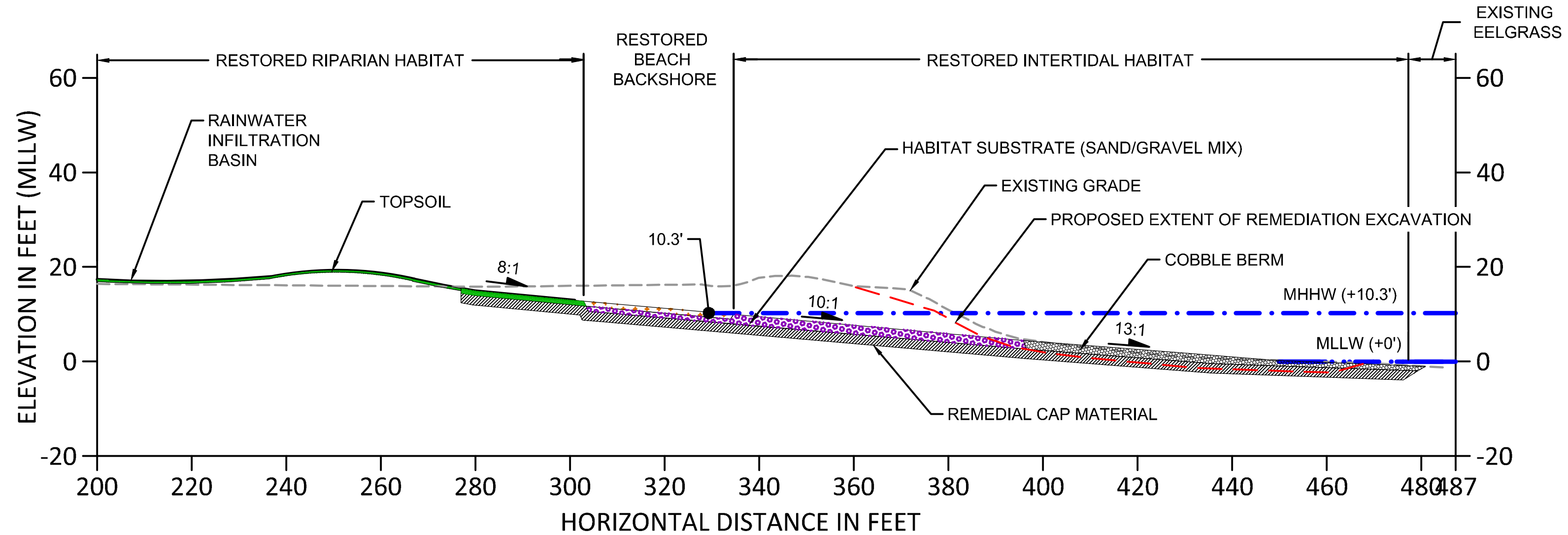
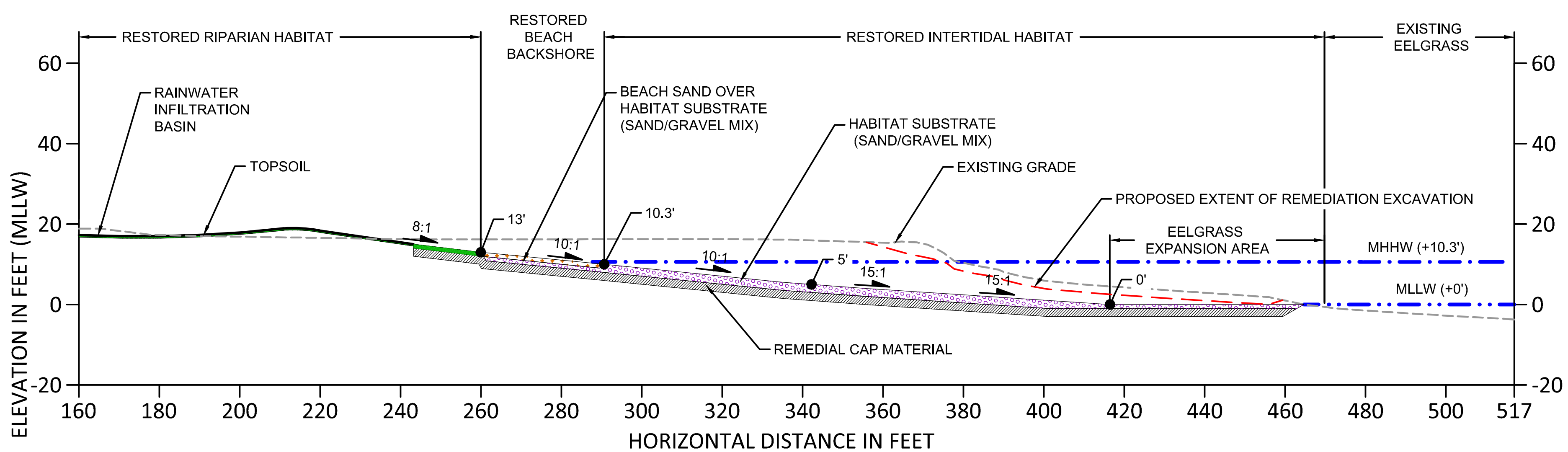
Figure M-2
HCCC South Shoreline Mitigation and Restoration Design
Appendix M: Integrated Restoration and Cleanup Actions
Port Gamble Bay Cleanup



A C-1 SECTION A
SCALE:1" = 20'



B C-1 SECTION B
SCALE:1" = 20'

Apr 21, 2015 5:08pm bsevertsen K:\Projects\1010-Hood Canal Coordinating Council\Port Gamble Restoration (HCCC)\1010-RP-004-HCCC-RestoreDesign.dwg FIG M-3

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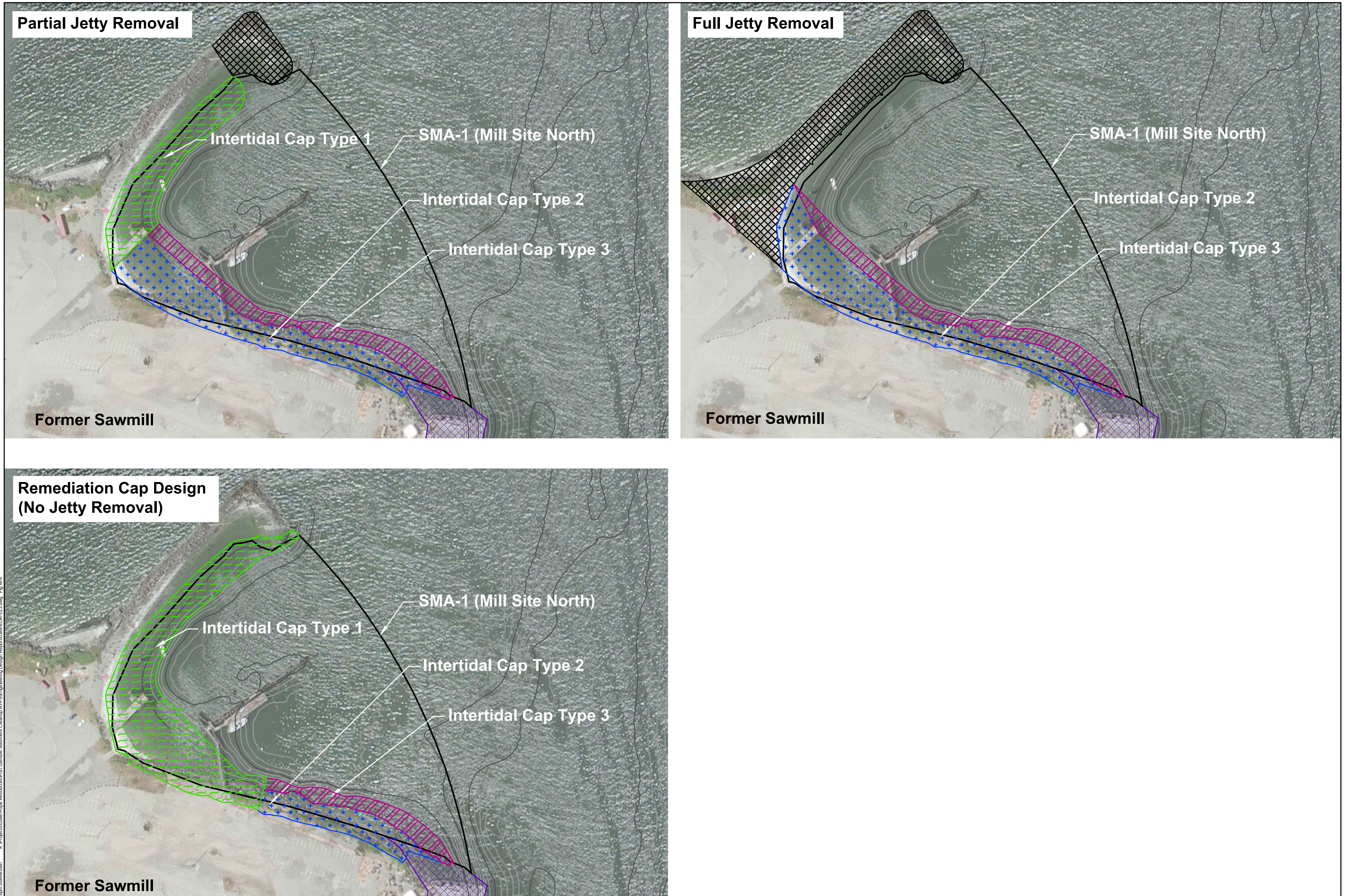
Figure M-3
HCCC South Shoreline Mitigation and Restoration Design Cross Sections
Appendix M: Integrated Restoration and Cleanup Actions
Port Gamble Bay Cleanup



LEGEND:

Sediment Management Area

Intertidal Cap Type 1
- Min. 6-inch sand and gravel
- Min. 12-inch armor

(Armor size d50 = 2.5 inches)

Intertidal Cap Type 2
- Min. 6-inch sand and gravel
- Min. 18-inch armor

(Armor size d50 = 9 inches)

Intertidal Cap Type 3
- Min. 6-inch sand and gravel
- Min. 12-inch armor

(Armor size d50 = 1.5 inches)

Piling Residuals Cap
(Armor size d50 = 12 inches)

Jetty Removal Area

0 200
Scale in Feet

AERIAL SOURCE: ESRI, 2010
BATHYMETRY: eTrac, dated August 27, 2014, and Ecology, dated March, 2015.
HORIZONTAL DATUM: Washington State Plane North, NAD83, U.S. Feet.
VERTICAL DATUM: Mean Lower Low Water (MLLW).

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Figure M-4
North Shoreline Mitigation and Restoration Design and Cap Armor Sizes
Appendix M: Integrated Restoration and Cleanup Actions
Port Gamble Bay Cleanup

ATTACHMENT M-1
WAVE MODELING IN SUPPORT OF RESTORATION DESIGN

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WAVE MODELING IN SUPPORT OF RESTORATION DESIGN (PART 1)
PORT GAMBLE MILL SITE RESTORATION

Prepared for
Port Gamble S'Klallam Tribe

Prepared by
Anchor QEA, LLC

April 2015

WAVE MODELING IN SUPPORT OF RESTORATION DESIGN (PART 1)

PORT GAMBLE MILL SITE RESTORATION

Prepared for

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April 2015

TABLE OF CONTENTS

1 INTRODUCTION 1

2 MODEL DEVELOPMENT 3

2.1 Wind Input 3

2.2 Model Grid Extents 4

2.3 Bathymetry and Topography 6

3 MODEL SCENARIOS 11

4 MODEL RESULTS 14

4.1 Comparison of Model Results with Wind-Wave Hindcast Calculations..................14

4.2 Overview of Predicted Nearshore Storm Wave Conditions..................20

4.2.1 Northwest and Northeast Shorelines 20

4.2.2 East Shoreline 20

4.2.3 South and Southeast Shoreline 21

4.3 Impacts on Nearshore Waves due to Breakwater Modification..................21

4.4 Impacts on Predicted Nearshore Waves due to Shoreline Cut-Back..................26

5 SEDIMENT TRANSPORT EVALUATION..................28

5.1 Net Littoral Drift 28

5.1.1 Reference Site Comparison..................29

5.1.2 Calculation of Littoral Drift Rate..................29

5.2 Beach Profile Changes due to Storm Waves..................31

6 NEXT STEPS 32

7 REFERENCES 33

List of Tables

Table 1 Estimated Extreme Wind Speeds for the Project Site.................. 4

Table 2 Tide Levels for NOAA Station 9445016 6

Table 3 Primary Wave Modeling Scenarios 11

Table 4 Additional Wave Modeling Scenarios 13

Table 5 Comparison of Modeled and Calculated Deep Water Wave Heights.................. 15

Table 6 Changes to Wave Heights due to Proposed Breakwater Modifications 25
Table 7 Select Wave Results for South Shoreline 27
Table 8 Median Wave Conditions.. 30

List of Figures

Figure 1 Project Site Vicinity Map 2
Figure 2 Full Extent of Model Grid 5
Figure 3 Full Extent of Model Bathymetry 7
Figure 4 Wave Model Grid at Northern Project Site – Existing Conditions..................... 8
Figure 5 Wave Model Grid at Southern Project Site – Existing Conditions..................... 9
Figure 6 Site Features 10
Figure 7 Run 2, Existing 2-year MSL; 335 Degree Wind 16
Figure 8 Run 4, Existing 2-year MSL; 150 Degree Wind 17
Figure 9 Run 6, Existing 20-year MSL; 335 Degree Wind 18
Figure 10 Run 8, Existing 20-year MSL; 150 Degree Wind 19
Figure 11 Run 5, Existing Breakwater, 20-year MSL; 0 Degree Wind 22
Figure 12 Run 17, Shortened Breakwater, 20-year MSL; 0 Degree Wind 23
Figure 13 Run 18, Breakwater Removed, 20-year MSL; 0 Degree Wind.......................... 24

List of Attachments

Attachment 1 South Shoreline Cut-back Figures

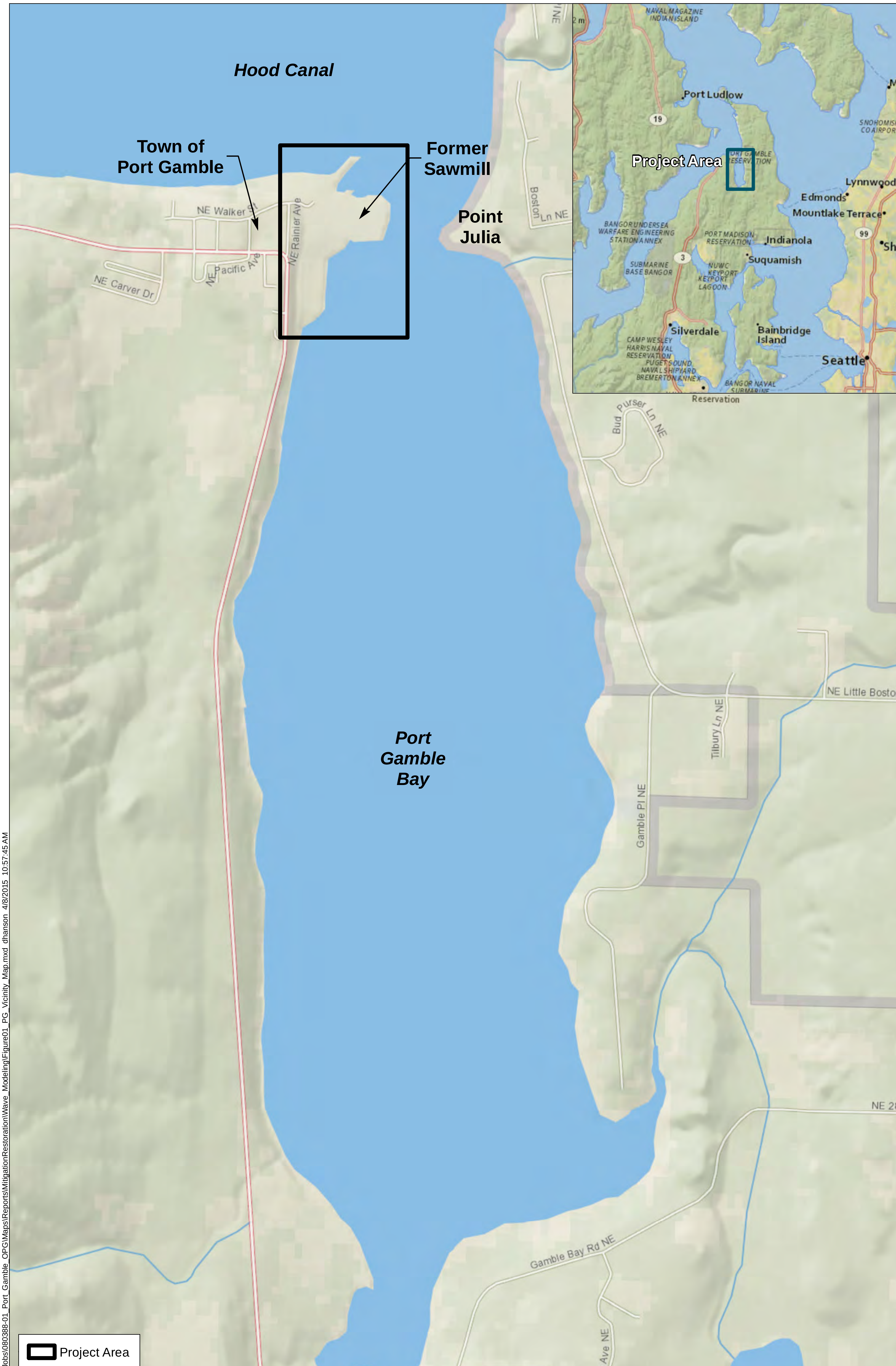
LIST OF ACRONYMS AND ABBREVIATIONS

CERC	Coastal Engineering Research Center
EDR	*Draft Engineering Design Report*
HCCC	Hood Canal Coordinating Council
H:V	horizontal to vertical
MHHW	mean higher high water
MLLW	mean lower low water
MSL	mean sea level
NOAA	National Oceanic and Atmospheric Administration
PGST	Port Gamble S'Klallam Tribe
PR/OPG	Pope Resources, LP/OPG Properties, LLC

1 INTRODUCTION

The Port Gamble S'Klallam Tribe (PGST) and Pope Resources, LP/OPG Properties, LLC (PR/OPG) are collaboratively evaluating potential restoration opportunities for the former Port Gamble mill site (see Figure 1). Anchor QEA is developing engineering designs for potential restoration alternatives that include fill removal along the northeast, east, southeast, and south shorelines, and partial breakwater removal at the north end of the former mill site. To support technical evaluations of these potential restoration design alternatives, including resultant modifications to sediment cleanup designs and considerations for a potential future PR/OPG dock in the northern mill site, Anchor QEA developed a detailed wave transformation model of the site and vicinity. This report provides a summary of the model development and initial summaries of model results.

The modeling work was conducted in two parts to efficiently complete the modeling and provide timely information to inform the ongoing design efforts. Part 1, which is the focus of this report, included model development, benchmark testing of the existing conditions model, evaluation of proposed breakwater changes on nearshore wave climate along the northeast shoreline, and evaluation of shoreline cut-back changes on nearshore wave conditions. Impacts of shoreline cut-back on model results were evaluated based on designs previously developed by Anchor QEA for cut-back of the south shoreline at the former mill site for the Hood Canal Coordinating Council (HCCC). Information gained through this evaluation is being used to inform the 30% restoration design for the northeast and east shorelines at the former mill site, which is currently in progress. Part 2 of Anchor QEA's modeling work will reflect the 30% design for the northeast and east shorelines at the former mill site, once it has been completed. Documentation of that effort (Part 2) will be provided as an addendum to this report.



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Feet
0 600 1,200 1,800 2,400

Figure 1
Project Site Vicinity Map
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

2 MODEL DEVELOPMENT

The Delft3D-WAVE model (utilizing the Simulating Waves Nearshore, or SWAN model) was used as the platform for developing the wave model for the site and vicinity. This model simulates depth-induced wave refraction and shoaling, depth- and steepness-induced wave breaking, diffraction, wave growth from wind input, and wave-wave interaction and white capping (Deltares 2014). The model was used to predict nearshore wave conditions based on extreme wind events (storms); evaluate potential changes in the nearshore wave climate based on shoreline changes; predict potential littoral transport rates at the site following restoration; and inform the restoration design and resultant modifications to sediment cleanup design and considerations for a potential future PR/OPG dock in the northern mill site.

2.1 Wind Input

The model simulates wave growth using an input wind-field that is applied across the entire model grid at a set speed and direction. Winds used in the model simulation represent extreme wind events with various return periods from 2 years to 20 years. These extreme wind speeds and directions were evaluated from hourly sustained wind data at Hansville and West Point, Washington, with the largest predicted wind speed between the two sites chosen for the analysis. Extreme wind speeds estimated for the project site are provided in Table 1. These wind speeds are slightly different than those reported in the *Draft Engineering Design Report*, *Port Gamble Bay Cleanup Project* (EDR) Appendix D, Coastal Engineering Evaluation and Propeller Wash Evaluation (Anchor QEA 2014). The changes were made in the predicted extreme wind speeds based on removal of outliers (bad data points) that were discovered in the data file. The wind-wave analysis the Final EDR for the cleanup project will reflect these revisions. Specific storm wind events used for this modeling effort are provided in Section 3 of this report.

Table 1
Estimated Extreme Wind Speeds for the Project Site

Direction (degrees)		Hansville, Washington		West Point, Washington	
Start	End	2-year Windspeed (miles per hour)	20-year Windspeed (miles per hour)	2-year Windspeed (miles per hour)	20-year Windspeed (miles per hour)
0	45	6	11	22	28
136	180	26	33	36	49
181	225	6	12	39	46
226	270	17	42	22	30
316	360	26	34	25	35

Note:
Greyed values represent those not used in the analysis; West Point wind data were primarily used, but due to its orientation and location, it results in artificially low westerly winds and was supplemented by Hansville wind data where appropriate.

2.2 Model Grid Extents

The model grid extends approximately 14 miles north, 10 miles southwest, and 3 miles east of the former mill site, and covers all of Port Gamble Bay. The model grid was developed to capture the full extent of fetch directions and distances that could impact wave development at the project shorelines. Grid cells at the project site are approximately 12 feet by 21 feet, and gradually become larger farther away from the project site, reaching sizes of up to 0.25 mile by 0.3 mile. Figure 2 shows the full extent of the model grid developed for the project.

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Feet
0 690 1,380

Figure 2
Full Extent of Model Grid
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

2.3 Bathymetry and Topography

Bathymetry and topography data were used to define elevations for each model grid cell. Data used for the model came from three sources:

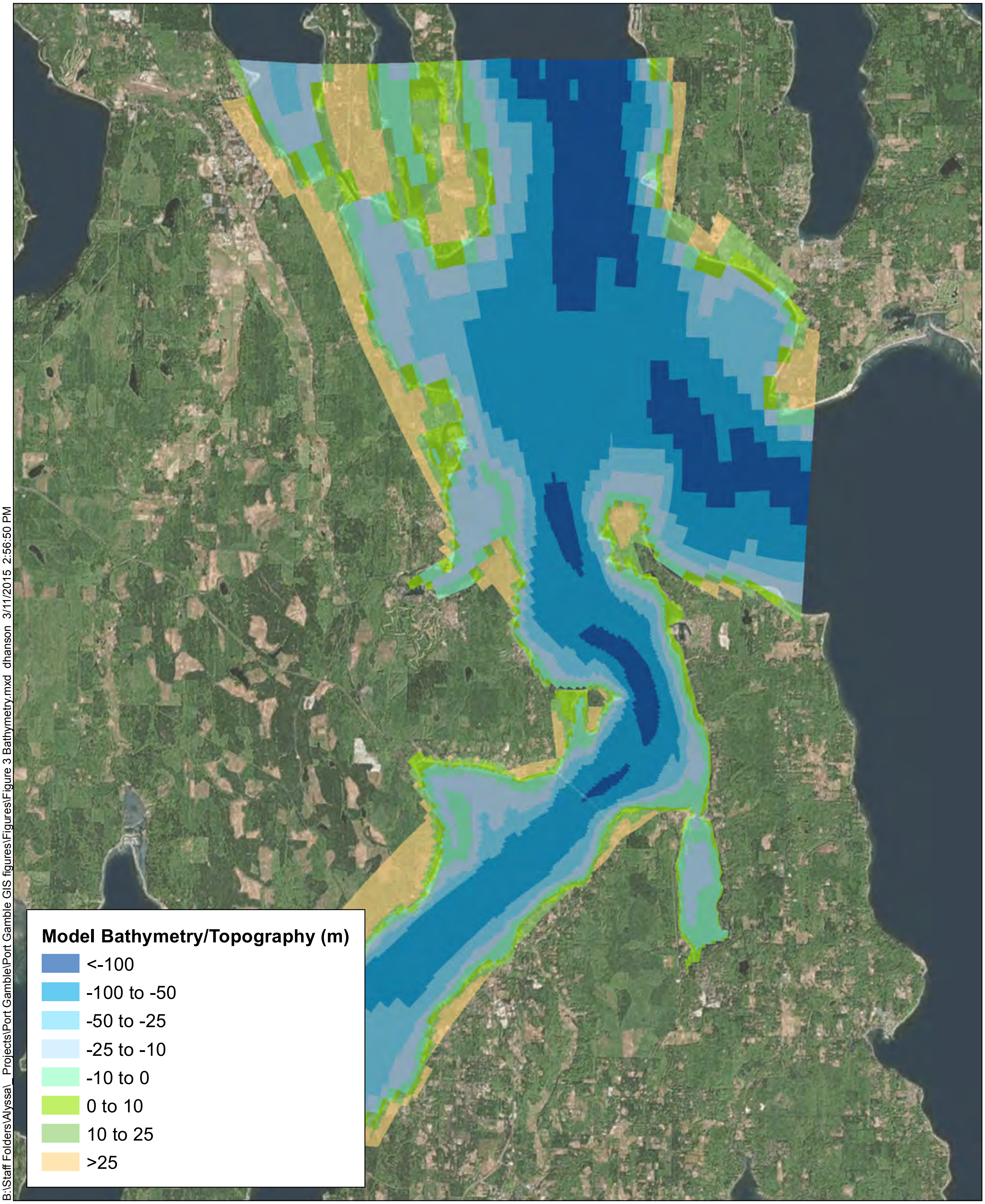
- Site bathymetry – conducted by E-trac on August 27, 2014
- Site topography – photogrammetric topographic survey conducted by Triad Associates in June and July 2012
- Far-field bathymetry and topography – Finlayson D.P. (2005), University of Washington

Figure 3 shows the bathymetry/topography for the full extent of the model grid, and Figures 4 and 5 show the same information for the model grid near the northern and southern portions of the former mill site, respectively. Figure 6 identifies site features, such as shoreline areas, that are discussed in this report.

Tidal information, as shown in Table 2, was obtained from National Oceanic and Atmospheric Administration (NOAA) Station 9445016 at Foulweather Bluff, located 5.75 miles north of Port Gamble.

Table 2
Tide Levels for NOAA Station 9445016

Tide Type	Tide Level (feet MLLW)
Mean Higher High Water	10.2
Mean High Water	9.3
Mean Tide Level	6.0
Mean Sea Level	6.0
Mean Low Water	2.8
Mean Lower Low Water	0.0



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Feet
0 7,200 14,400

Figure 3
Full Extent of Model Bathymetry
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

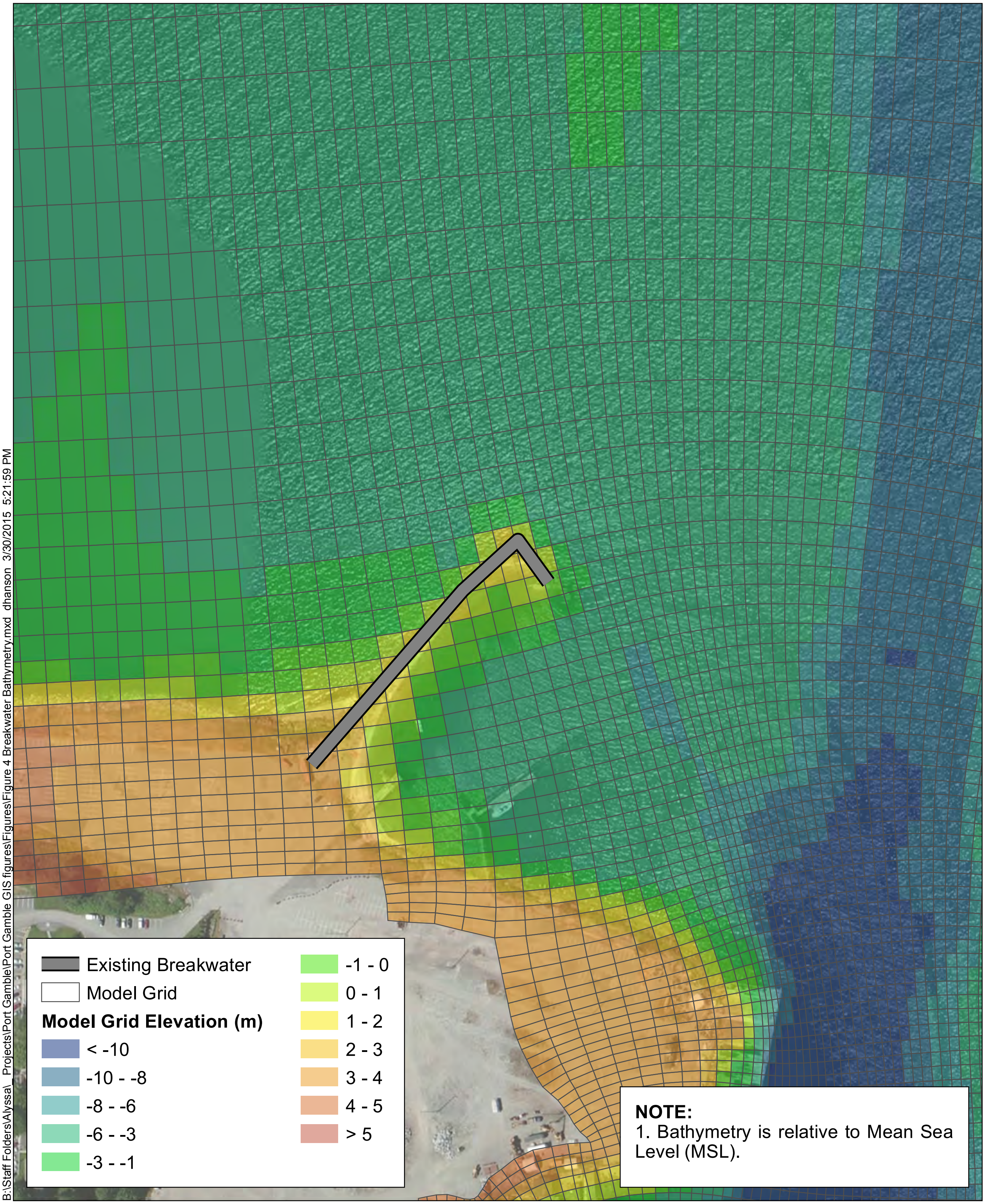

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Figure 4
Wave Model Grid at Northern Project Site – Existing Conditions
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

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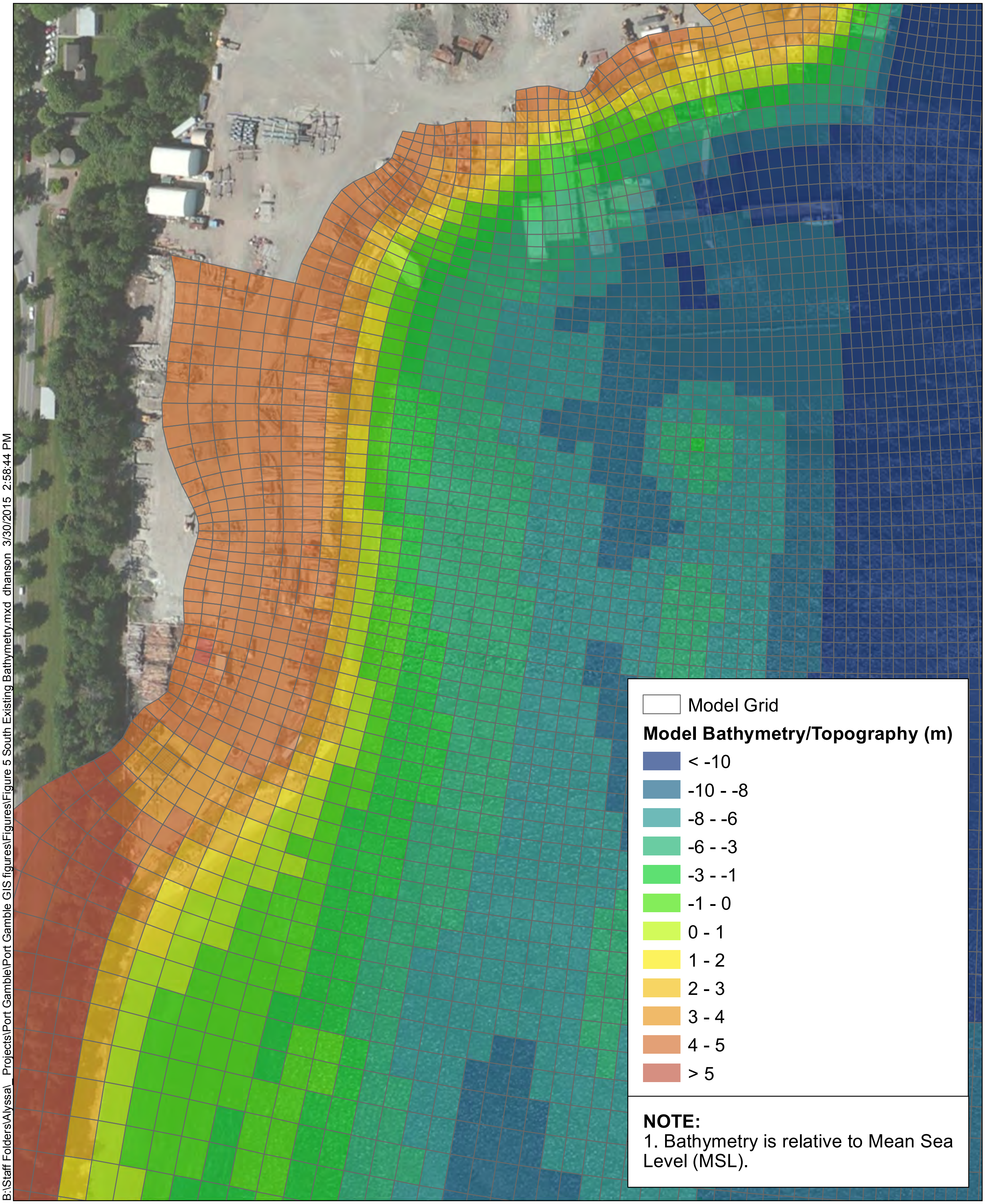
Feet
0 130 260



B:\Staff Folders\Alyssa_Projects\Port Gamble\Port Gamble GIS figures\Figures\Figure 5 South Existing Bathymetry.mxd dhanson 3/30/2015 2:58:44 PM

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Figure 5
Wave Model Grid at Southern Project Site – Existing Conditions
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

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Feet
0 175 350

Figure 6
Site Features
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

3 MODEL SCENARIOS

Model scenarios were developed to conduct benchmark testing of the wave model and to support evaluation of proposed breakwater modifications and shoreline cut-back along the former mill site shoreline. Breakwater modification scenarios included removal of the last 200 linear feet of the breakwater (shortened breakwater) and a full removal of the entire structure. The 2-year and 20-year return period wind events for various wind directions were used with different combinations of water levels to characterize nearshore wave conditions at the site. The existing conditions model grid was also modified to reflect full and partial removal of the northern breakwater and shoreline cut-backs along the south shoreline to evaluate potential impacts of these actions on nearshore waves. Results of the shoreline cut-back analysis were also used to evaluate proposed restoration design along the south shoreline cut-back area (Figure 6) and to inform in-progress design efforts for restoration along the northeast and east shorelines (north and east shoreline cut-back area shown in Figure 6). These primary modeling scenarios are summarized in Table 3.

Table 3
Primary Wave Modeling Scenarios

Run	Site Conditions (Grid Geometry)	Water Level	Return Period for Wind	Wind Direction (degrees from North)	Wind Speed (miles per hour)
1	Existing Conditions[1]	MSL	2-year	0	22
2				335	26
3				180	39
4				150	36
5			20-year	0	28
6				335	35
7				180	46
8				150	49
9		MLLW	20-year	0	28
10		MHHW	20-year	0	28
11		MLLW	20-year	150	49
12		MHHW	20-year	150	49
13	Shoreline Cut-Back of South Shoreline[2]	MSL	2-year	180	39
14				150	36
15			20-year	180	46
16				150	49

Run	Site Conditions (Grid Geometry)	Water Level	Return Period for Wind	Wind Direction (degrees from North)	Wind Speed (miles per hour)
17	Partial Removal of Breakwater	MSL		0	28
18	Full Removal of Breakwater	MSL		0	28

Notes:

1. Existing conditions represents current shoreline conditions, current configuration of the breakwater and removal of the wooden breakwater in Port Gamble Bay located in the southern portion of the former mill site.
2. Shoreline cut-back in the south was based on existing 30% design developed by Anchor QEA for HCCC

MLLW = mean lower low water
MSL = mean sea level
MHHW = mean higher high water

In addition to the primary modeling scenarios summarized in Table 3, additional modeling was also conducted to address specific sensitivity analyses. These additional modeling scenarios are summarized in Table 4. Runs 19 through 21 were completed to demonstrate that the wave model grid was configured far enough north to effectively capture the full wind fetch or possible swell waves from the Strait of Juan de Fuca. Runs 22 through 27 were completed to evaluate impacts on wave conditions from a 2-year storm event resulting from partial and full removal of the existing breakwater along the north shoreline. The 20-year storm events were simulated in in Runs 5, 17, and 18 (see Table 3).

Table 4
Additional Wave Modeling Scenarios

Run	Modeled Condition	Water Level	Direction (degrees from North)	Speed (miles per hour)
19	Increased Fetch Distance to the North of the Site[1]	MHHW	345 degrees (11 miles fetch)	35
20		MHHW	0 degrees (8 miles fetch)	28
21	Wave Swell Component added to Northern Model Boundary[2]	MHHW	345 degrees	Wave input[2]
22	2-year Return Period Wind; Existing Breakwater	MSL	0	22
23	2-year Return Period Wind; Partial Removal of Breakwater		0	22
24	2-year Return Period Wind; Full Breakwater Removed		0	22
25	2-year Return Period Wind; Existing Breakwater		335	26
26	2-year Return Period Wind; Partial Removal of Breakwater		335	26
27	2-year Return Period Wind; Full Breakwater Removed		335	26

Notes:
1. North fetch wave added to northern boundary
2. Swell wave was added to northern boundary; based on National Data Buoy Center, Station # 46088- resulting in a 3.75-meter wave with a 10-second period.

MSL = mean sea level
MHHW = mean higher high water

4 MODEL RESULTS

Wave heights and directions were modeled for the entire model grid (Figure 2) for the scenarios outlined in Tables 3 and 4. Model results were examined in detail closer to the site, within the extents shown in Figures 4 and 5. These results were used to evaluate the following:

1. Comparison of model-predicted deep water wave heights with analytical calculations (Section 4.1)
2. Existing nearshore wave conditions along the project shorelines (Section 4.2)
3. Changes to nearshore wave conditions due to proposed modifications of the breakwater (Section 4.3)
4. Changes to nearshore wave conditions due to shoreline cut-back (Section 4.4).

4.1 Comparison of Model Results with Wind-Wave Hindcast Calculations

The model used wind speed and direction as input to estimate wave conditions in deep water offshore of the site (Table 1). Direct wave measurements are not available for the project vicinity; therefore, model predictions were compared to calculated wave heights in order to evaluate model performance.

Analytical calculations of wave heights and wave periods were completed using wind-wave-hindcast equations as outlined in the U.S. Army Corps of Engineers *Coastal Engineering Manual* (USACE 2002). These calculations assume a fully-developed sea state is achieved and use wind speed, wind direction, fetch distance, and average water depth over the fetch distance to estimate wave conditions in deep water. A comparison was done for four different wind conditions: 2-year wind speed from 335 degrees and 150 degrees, and 20-year wind speed from the same directions. These represent model scenarios ("runs") 2, 4, 6, and 8 as shown in Table 3. Results of the comparison are shown in Table 5.

Table 5
Comparison of Modeled and Calculated Deep Water Wave Heights

Run # (Figure)	Direction (degrees)	Storm Event	Wind Speed (miles per hour)	Hindcast Wave Height (feet)	Model Predicted Wave Height (feet)
2 (Figure 7)	335	2-year	26	3.1	3.5
4 (Figure 8)	150	2-year	36	1.9	2.0
6 (Figure 9)	335	20-year	35	4.2	3.6
8 (Figure 10)	150	20-year	49	2.8	2.6

As shown in Table 5, the model estimates of deep water wave conditions are aligned with analytical calculations using wind-wave hindcast methods. Comparisons of wave heights for winds from the north are not as favorable as winds from the south. This is because the fetch distance and shape of the waterbody to the north of the site are more complex than to the south, which the model takes into account but the analytical calculations do not. Based on results provided in Table 5, the model is producing reasonable estimates of wave conditions offshore at the site using wind conditions as input.

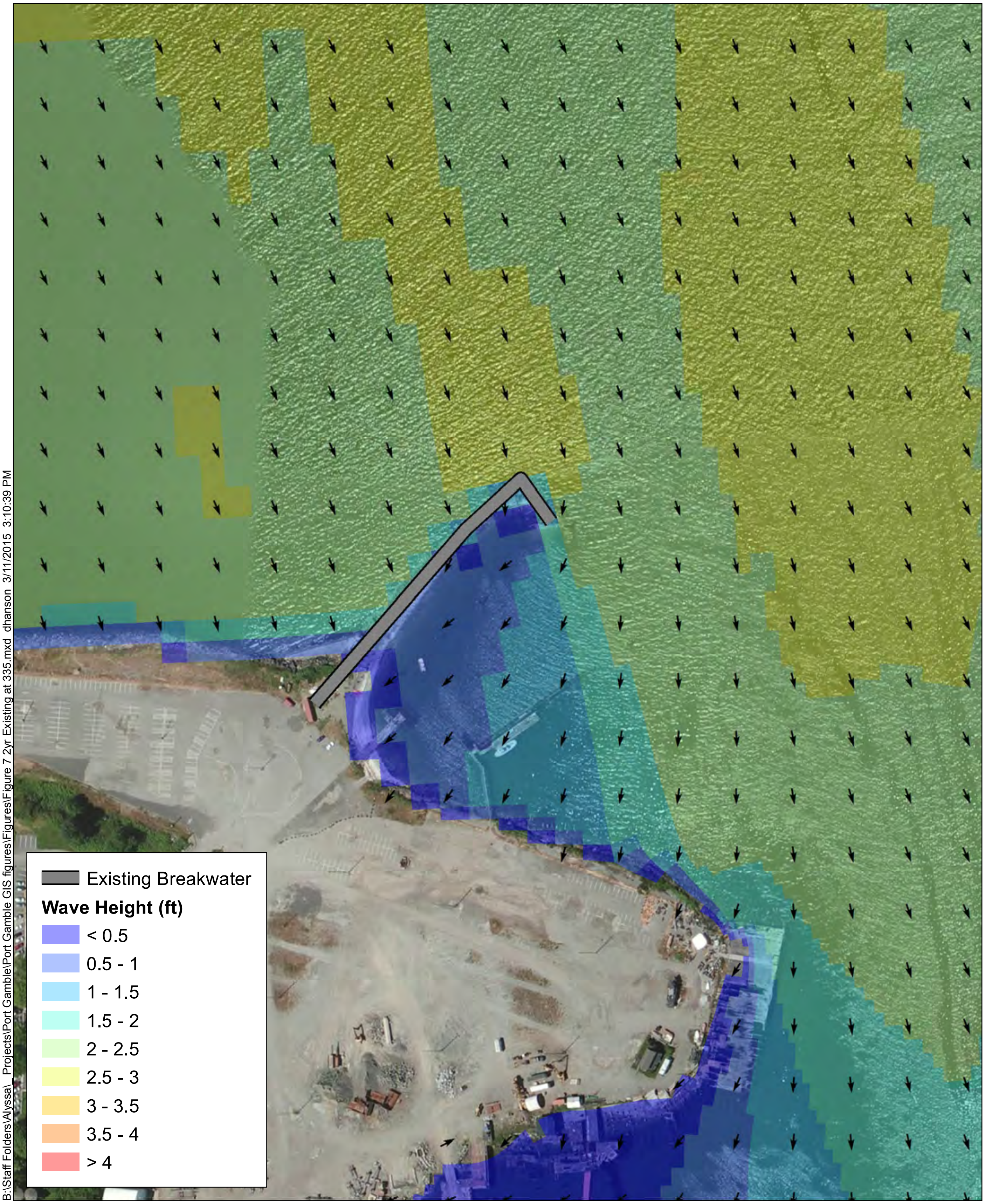

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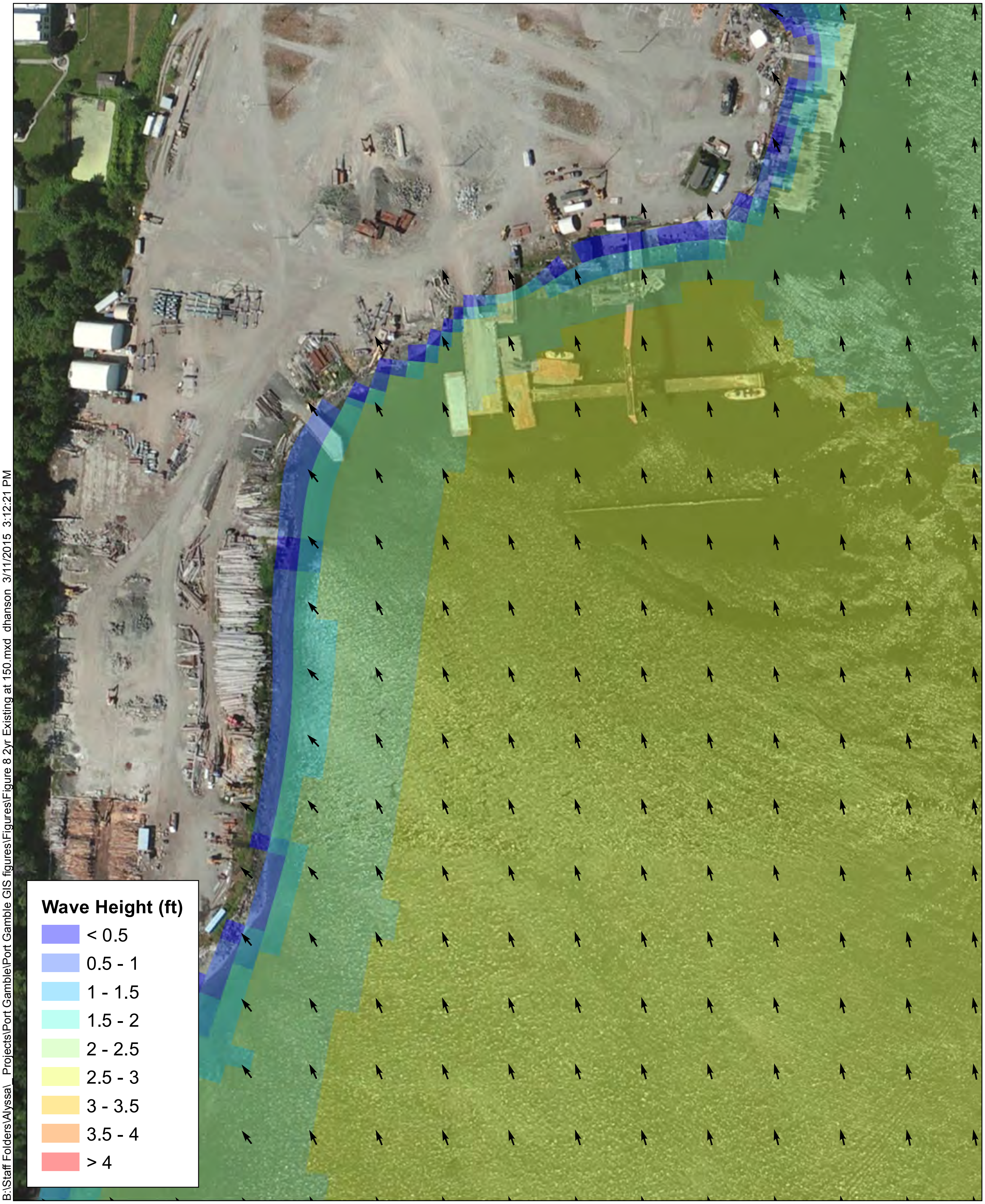
Figure 7
Run 2-Existing 2-year MSL; 335 degree wind
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration



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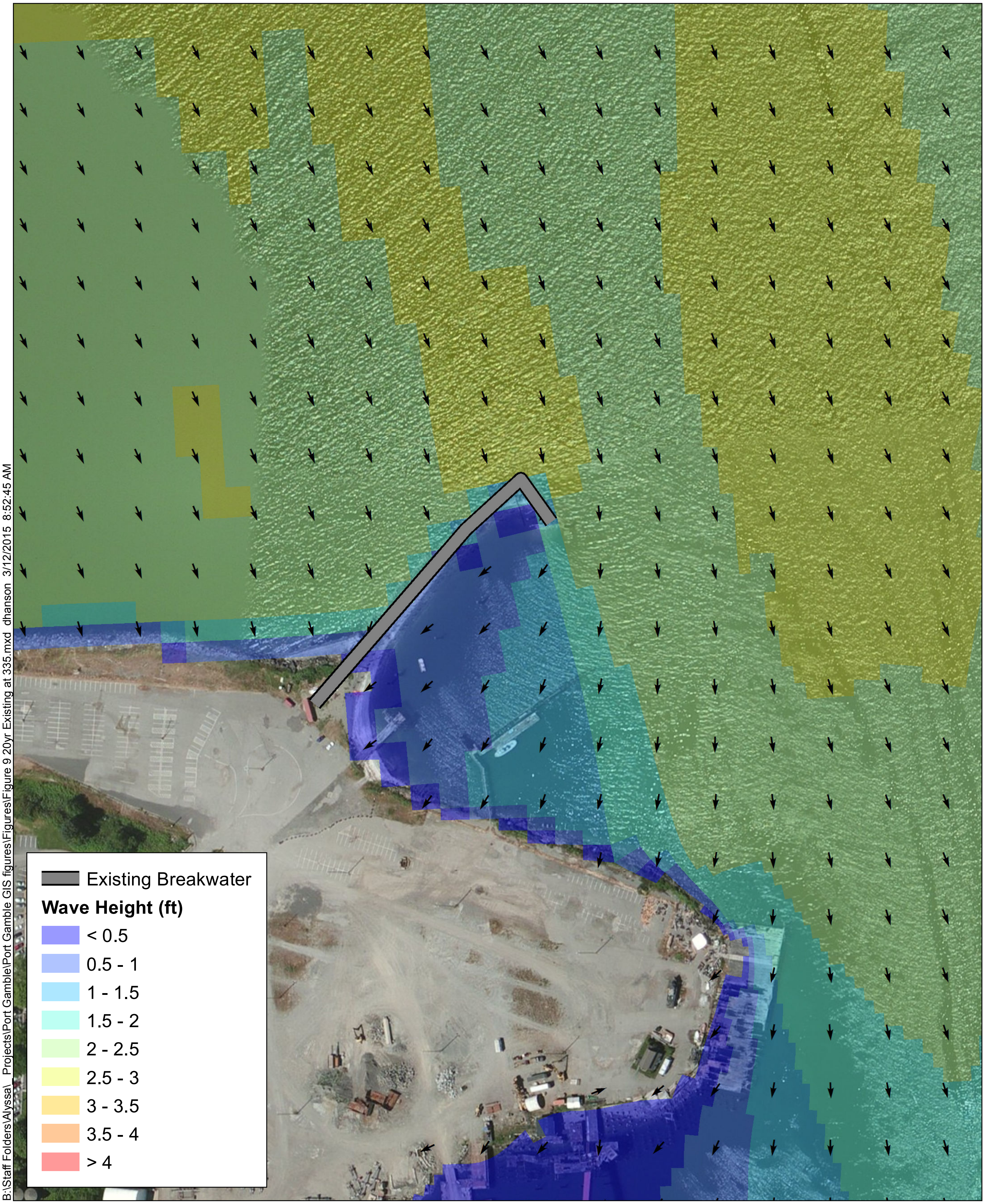
Feet
0 100 200

Figure 8
Run 4-Existing 2-year MSL; 150 degree wind
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration



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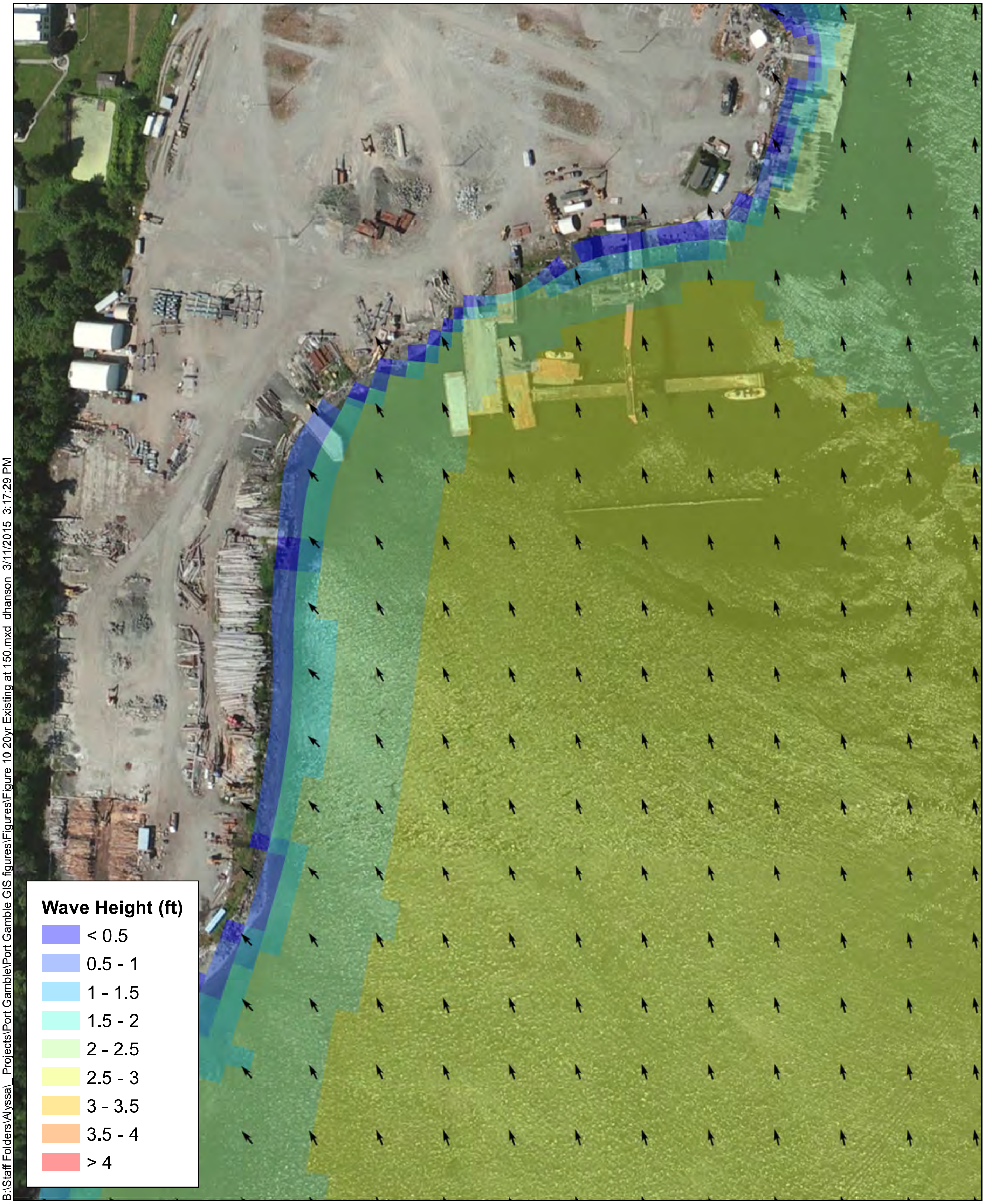
Feet
0 130 260

Figure 9
Run 6-Existing 20-year MSL; 335 degree wind
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration



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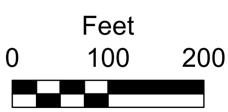

Figure 10
Run 8-Existing 20-year MSL; 150 degree wind
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

4.2 Overview of Predicted Nearshore Storm Wave Conditions

Deep water wave conditions predicted by the model (offshore of the site) range from 2 to 3 feet for the 2-year return period wind events and 3 to 4 feet for the 20-year return period wind events. These deep water waves transform as they move towards shore through processes of dispersion, refraction, diffraction, and shoaling. The wave model was used to predict how deep water waves change as they move towards shore, and to provide predictions of nearshore wave conditions along each of the project shorelines (see Figure 6). Figures 7 and 8 show predicted nearshore waves for 2-year wind events from the northwest and southeast, respectively. Figures 9 and 10 show predicted nearshore waves for the same directions for the 20-year wind event. Sections 4.2.1 through 4.2.3 provide a brief narrative of predicted wave conditions along the project shorelines.

4.2.1 Northwest and Northeast Shorelines

The northwest shoreline is impacted by waves between 2 and 2.5 feet due to waves from the northwest (for the 2- and 20-year events). The northeast shoreline is protected from waves from the northwest by the existing breakwater. Waves from the north impact the entire north shoreline, except for a small stretch of the northeast shoreline directly adjacent to the breakwater. The 2-year wave conditions result in waves along the north shoreline of approximately 1.0 to 1.5 feet, while the 20-year wave conditions result in waves along the north shoreline of approximately 1.5 to 2.0 feet.

4.2.2 East Shoreline

The east shoreline has very little fetch resulting in direct wave attack; however, oblique waves can impact the shoreline from the north and southeast. From the north, the 2-year waves are less than 0.5 foot, and the 20-year waves are approximately 1.0 foot. From the southeast, the waves more directly impact the east shoreline. The 2-year southeasterly waves are approximately 1.8 feet along the east shoreline, and the 20-year southeasterly waves are approximately 2.2 feet.

4.2.3 South and Southeast Shoreline

Along the south and southeast shorelines, the greatest impact is from southeasterly waves. Due to the shallow slope of the shoreline, the waves break offshore and therefore reach the site at lower heights (see Section 4.3). The offshore wave height along the south shoreline for the 2-year southeasterly waves is approximately 2.0 feet, and the 20-year wave height is approximately 2.5 feet.

4.3 Impacts on Nearshore Waves due to Breakwater Modification

Nine model scenarios were used to evaluate impacts to nearshore waves along the north-east shoreline due to proposed modifications to the breakwater. Runs 10, 17, and 18 (Table 3) were used to evaluate nearshore waves from the 20-year wind from the northwest direction (335 degrees) for existing conditions, shortened breakwater, and full removal, respectively. Runs 22 to 27 were used to evaluate changes to nearshore waves for 2-year wind conditions from the northwest (335 degrees) and north (0 degrees) directions.

Figures 11 through 13 provide model results for the 20-year wind simulations from the northwest. These figures also show three discrete locations where wave heights were extracted from the model for all nine model scenarios; this information is summarized in Table 6. Location 1 is near the shoreline directly adjacent to the breakwater; location 2 is alongside the existing dock and within the influence of the breakwater; location 3 is in an area not expected to be significantly affected by the breakwater. The full breakwater, shortened breakwater, and fully removed breakwater conditions were compared to calculate the percent increase in wave height that would result from various breakwater/jetty removal options. Table 6 provides this comparison for all nine model scenarios.

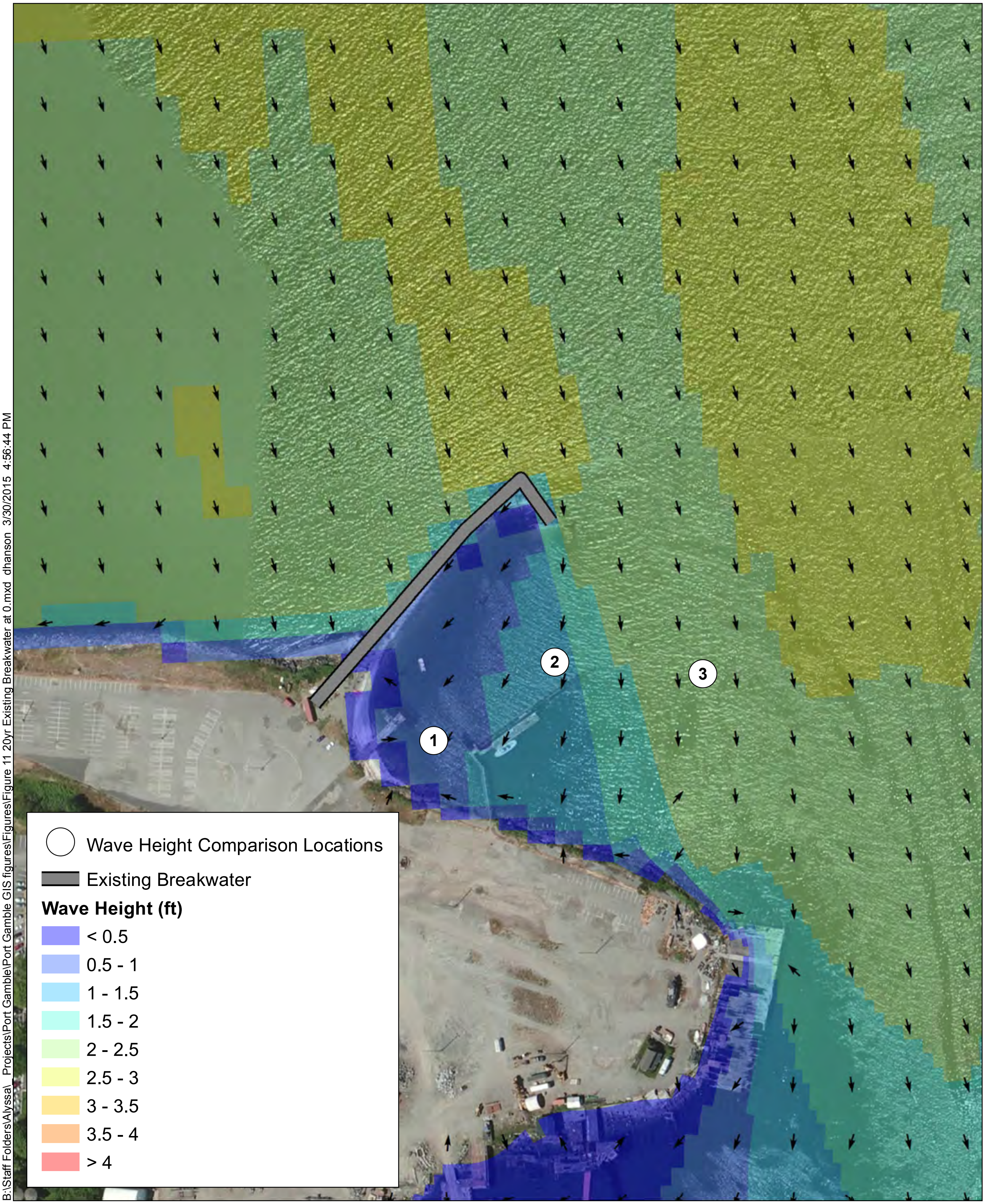

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Figure 11
Run 5-Existing Breakwater 20-year MSL; 0 degree wind
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

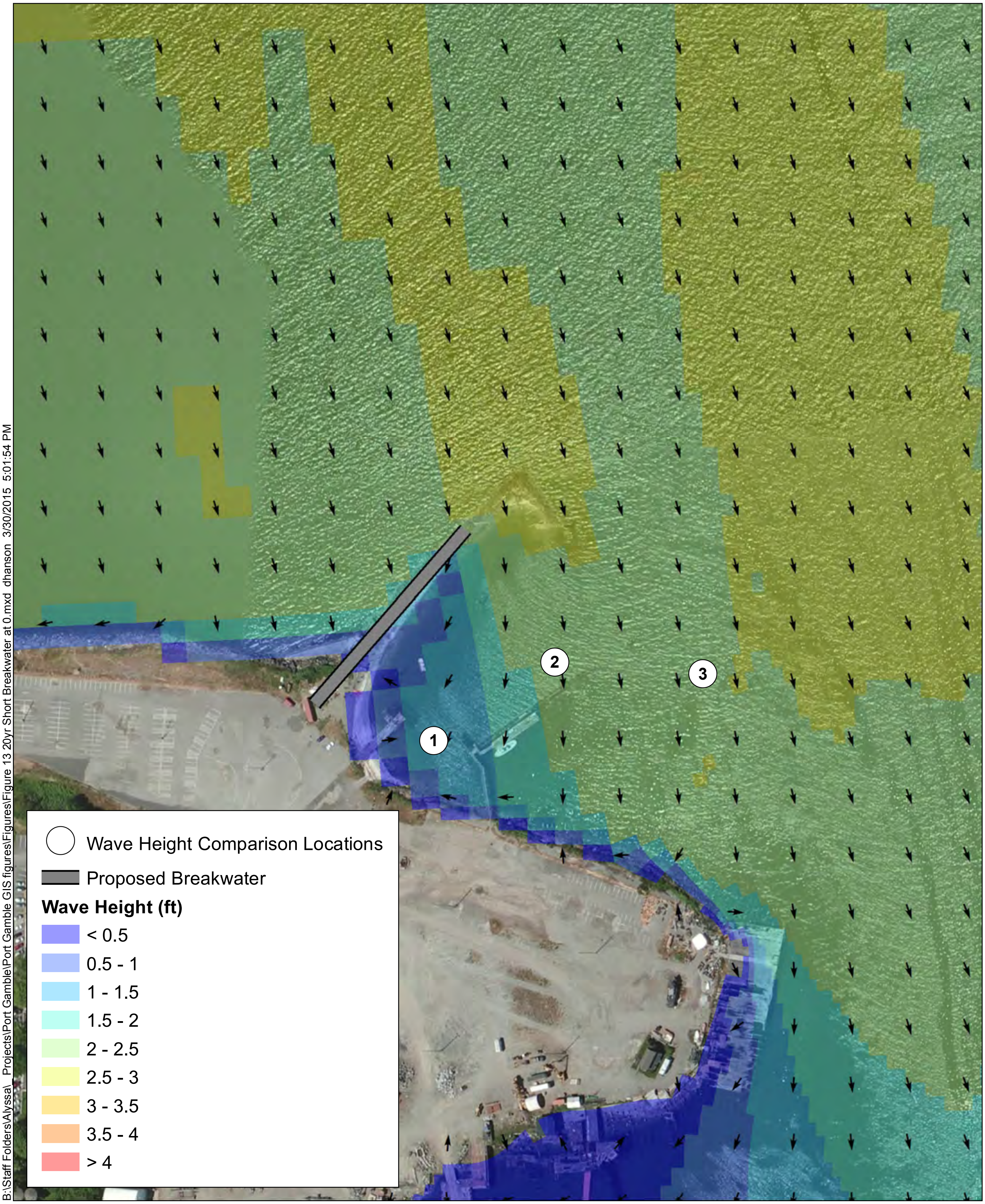

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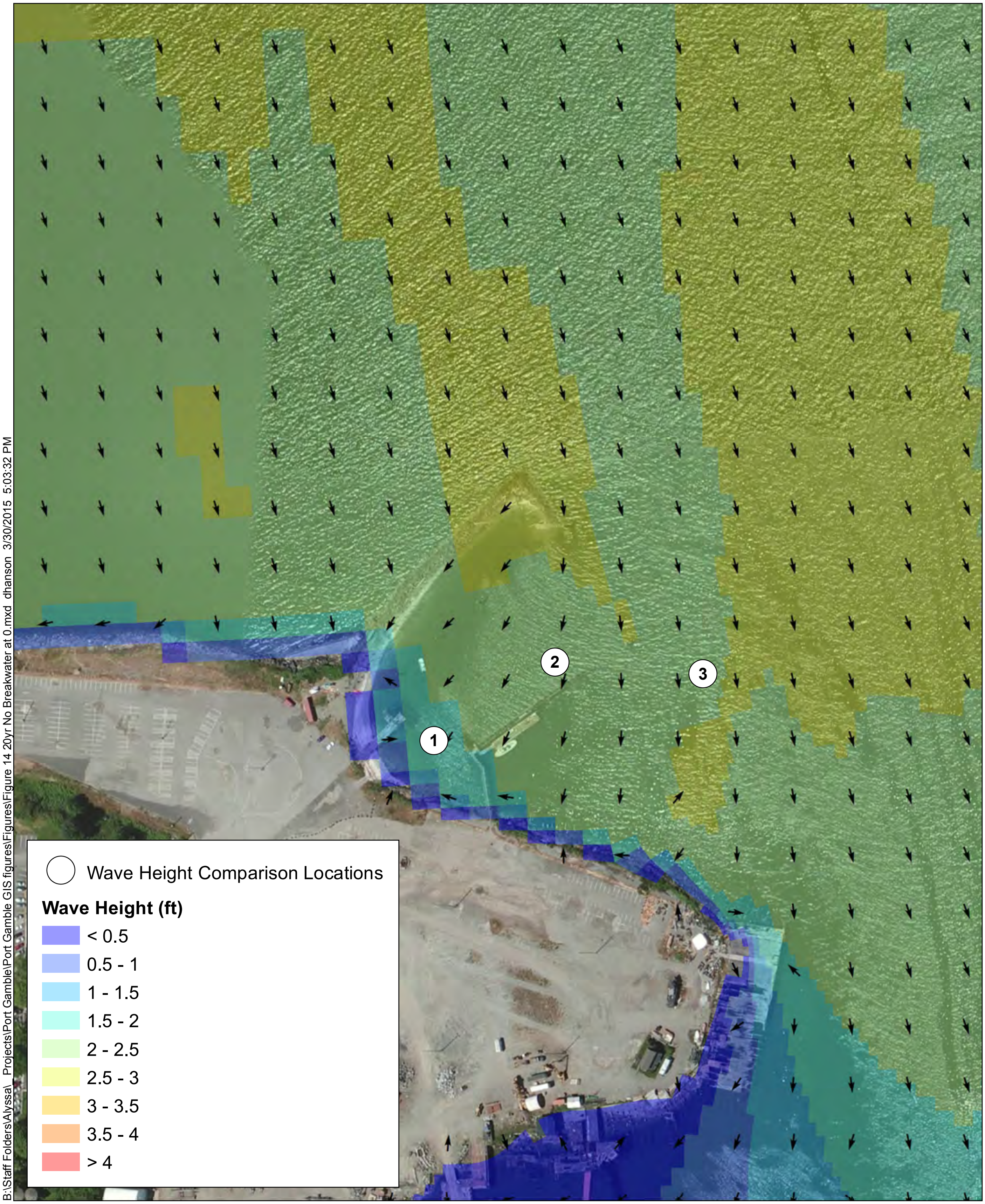
Figure 12
Run 17- Shortened Breakwater 20-year MSL;
0 degree wind
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration



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Feet
0 130 260

Figure 13
Run 18- Breakwater Removed 20-year MSL;
0 degree wind
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

Table 6
Changes to Wave Heights due to Proposed Breakwater Modifications

Breakwater Configuration (Run #)	Location 1	Location 2	Location 3
Wind Condition: 2-year N Wind (0 degrees, 22 mph, MSL)			
Existing (Run 22)	0.5	0.6	1.6
Shortened (Run 23)	0.5	0.8	1.6
Removed (Run 24)	1.2	1.8	1.6
% Increase for Shortened	0%	33%	0%
% Increase for Removed	140%	200%	0%
Wind Condition: 2-year NW Wind (335 degrees, 26 mph, MSL)			
Existing (Run 25)	0.5	0.8	2
Shortened (Run 26)	0.6	1	2.1
Removed (Run 27)	1.4	2.2	2.2
% Increase for Shortened	20%	25%	5%
% Increase for Removed	180%	175%	10%
Wind Condition: 20-year N Wind (0 degrees, 28 mph, MSL)			
Existing (Run 10)	0.5	1.4	2.2
Shortened (Run 17)	1	2	2.4
Removed (Run 18)	1.6	2.3	2.4
% Increase for Shortened	100%	43%	9%
% Increase for Removed	220%	64%	9%

Shortening the breakwater by 200 feet would increase wave heights from north winds by 20% to 100% at location 1 for the 2-year and 20-year wind speeds, respectively. Winds from the northwest would not be changed at location 1 due to shortening of the breakwater. Location 2, which is alongside the exiting dock, shows an increase in wave heights due to shortening of the breakwater, from approximately 25% to 40% based on model runs examined. Changes to wave height at location 3 would be small: less than 5% for the 2-year events and less than 10% for the 20-year event. Shortening the breakwater can increase wave heights at the dock (location 2) by a few tenths of a foot for the 2-year event and by more than 0.5 foot for the 20-year event. Impacts to the use of the dock would be significant for larger storm events, but may be relatively unchanged during average or milder storm conditions with a shorter breakwater.

Full breakwater removal would result in more than 100% increases in wave heights at locations 1 and 2 for all events, except for the 20-year event from the north where waves would increase by approximately 60%. Several model runs showed increases around 200% at those locations. Location 3 showed the same minor increase to wave heights exhibited by the shortened breakwater alternative. Removing the breakwater could increase wave heights at the dock for the 2-year event (location 2) between a few tenths of a foot due to winds from the north to almost 1.5 feet due to winds from the northwest. The 20-year event from the north shows wave height increases of 1 foot at Location 2. Impacts to the use of the dock would be significant for all storm events, with greater impacts during storms from the northwest where the existing breakwater provides the most protection to the dock.

4.4 Impacts on Predicted Nearshore Waves due to Shoreline Cut-Back

Proposed restoration actions at the project site along the south, southeast, east, and northeast shorelines include cutting back the shoreline on mild slopes (e.g., 10 horizontal to 1 vertical [H:V] to 15H:1V) and adding mixed sand and gravel sediments to the beach. There was concern that wave heights and directions nearshore would be significantly changed due to proposed cut-backs along the shoreline. It was anticipated that cut-backs to the shoreline that did not significantly change the shoreline orientation would not result in significant changes to predicted nearshore wave heights or directions. However, to validate that assertion, a comparison was made of wave heights and directions for the south shoreline based on existing conditions and a scenario where the shoreline was cut back approximately 60 feet from its current location.

Figure A1 (in Attachment 1) shows the area where the shoreline was cut back in the model, as well as two shore-normal transects where wave height information was extracted from the model for comparison. Four wind conditions were modeled using the model grid with the cut-back shoreline: 2- and 20-year wind conditions in the south (180 degrees) and southeast (150 degrees) directions (Runs 13 through 16 in Table 3). Figures A2 and A3 in Attachment 1 illustrate model results along the south shoreline for cut-back conditions for the 2-year and 20-year southeast wind conditions[1].

[1] Figures 8 and 10 in this report provide model results for existing conditions for the same wind events.

Table 7 shows the comparison of wave heights and directions for the 20-year simulations for south and southeast winds for the existing and cut-back shoreline. Wave heights and directions were extracted from each of the simulations at set water depths (5, 10, and 20 feet) along the transect lines shown in Figure A1. These comparisons are also shown graphically in Attachment 1. Figures A4 and A5 show wave height comparisons for Transects 1 and 2, respectively, and Figures A6 and A7 show wave direction comparisons for Transects 1 and 2, respectively.

Table 7 and Figures A4 through A7 illustrate that proposed cut-backs to the shoreline (without significant changes to the shoreline orientation) do not alter the wave heights or direction within the precision of the model. Therefore, predicted nearshore wave conditions based on existing shoreline conditions can be used to estimate littoral drift rates, and address other sediment transport questions, for proposed flatter shorelines around the project site. However, proposed designs for shoreline cut-back should be modeled directly at 30% to ensure that the proposed changes to the shoreline do not impact other areas in the vicinity of the site. This will be completed during Part 2 of the modeling effort for this site for the north and east shoreline cut-back area (see Figure 6).

Table 7
Select Wave Results for South Shoreline

Transect*	5-foot Depth		10-foot Depth		20-foot Depth	
	Wave Height (feet)	Direction (degrees)	Wave Height (feet)	Direction (degrees)	Wave Height (feet)	Direction (degrees)
Run 8: 20-year Southeast Wind (150 degrees, 49 mph, MSL) Existing						
1	1.9	140	2.3	148	2.6	156
2	1.6	126	1.8	137	2.3	150
Run 14: 20-year-Southeast Wind (150 degrees, 49 mph, MSL) Proposed						
1	1.9	139	2.4	150	2.6	157
2	1.5	125	1.8	136	2.2	148

Note:
* See Figure A1 for transect locations

5 SEDIMENT TRANSPORT EVALUATION

Model results were used to evaluate sediment transport issues and concerns related to design of proposed restoration actions at the project site. Specific evaluations include:

- Estimates of net littoral drift rates on the south and northwest and northeast shorelines (assuming no breakwater)
- Estimates of beach profile change (vertical) due to storm events

5.1 Net Littoral Drift

The net littoral drift rate is the annual net volume of sediment that moves along the shore in a designated direction (net drift direction). Littoral drift (also called longshore transport) occurs in the surf zone and swash zones for gravel or mixed sand/gravel beaches, and is caused by breaking waves. The magnitude of littoral drift is proportional to the wave height, wave period, beach slope, sediment size and gradation, and angle of approach of the waves in relation to the shoreline orientation[2]. It is also dependent on sediment supply in the system.

Port Gamble Bay shorelines receive potential sediment from unstable slopes and six streams that empty into the bay. Based on Washington State Department of Ecology's Coastal Atlas (https://fortress.wa.gov/ecy/coastalatlas/Default.aspx) there is approximately 0.6 mile of intermediate slopes (steep grade or weaker material) immediately to the south of the project, which could result in erosion to nourish the south shoreline of the project. Of the six streams that enter the Port Gamble Bay, three streams are along the west shoreline and sediment loads could result in direct nourishment for the south shoreline of the project. For the north shoreline, there is approximately 1.0 mile of eroding bluffs along the shoreline just updrift of the project shoreline that can supply sediment to the littoral zone.

Littoral drift rates are site-specific, and are not trivial to predict without site-specific wave and survey (shoreline location) data. No direct wave measurements are available for the site; therefore, two methods were used to estimate the net littoral drift rate along the project shorelines: comparison with reference sites (see Section 5.1.1), and an analytical calculation based on physical understanding of littoral drift rate (see Section 5.1.2).

[2] Waves that approach the beach head-on (perpendicular to shoreline) do not produce any littoral drift. Littoral drift rate increases as the angle of approach of the waves away from perpendicular increases.

5.1.1 Reference Site Comparison

Reference sites, where estimates of net littoral drift rates are available, were used to estimate possible rates of littoral drift for the Port Gamble site.

The first reference site used for this project was the Mount Baker Terminal Restoration Site (Mukilteo, Washington). This site was selected because it has a north-facing shoreline with similar effective wind directions and fetch distances as the north shoreline of the Port Gamble site. This site was nourished with a sand/gravel mix in 2006 and has been monitored annually for more than 7 years. By comparing shore-normal transects taken yearly, an estimated annual net volume loss (or gain) can be calculated. For the Mount Baker Terminal Restoration Site, the erosion rate was estimated to be 150 to 300 cubic yards per year.

Four additional reference site estimates were gathered from The Puget Sound Nearshore Partnership's technical report *The Geomorphology of Puget Sound Beaches* (Finlayson 2006). This report provides estimates of net littoral transport rates for 17 sites throughout Puget Sound, including the north shoreline of the Port Gamble project. The north shoreline of the project area (referred to as the "Pope and Talbot Mill Breakwater (Kitsap County)" site in the report) is estimated to have a net littoral drift rate of 100 cubic yards per year (to the east). There are three other sites that have similar characteristics to shorelines at Port Gamble, based on wave directions, fetch, and littoral cell length. Zittel's Marina, similar to the north shoreline at Port Gamble, has an estimated net littoral drift rate of 130 cubic yards per year. South Foss Tug Jetty and North Foss Tug Jetty, similar to the south and southeast shorelines at Port Gamble, have estimated net littoral drift rates between 100 and 130 cubic yards per year.

5.1.2 Calculation of Littoral Drift Rate

For the analytical calculation of net littoral drift rate, the Coastal Engineering Research Center (CERC) formula (USACE 1984) was used. For these calculations, median wave conditions were used to represent typical day-to-day conditions. Table 8 outlines the wave climate used. An empirical value (K) is required for the CERC formula based on field and laboratory research, and should be calibrated from site-specific field data if available. Since appropriate field data are not available, values for K were estimated from literature and calibrated values developed by Anchor QEA for a similar location.

Two K values were used to estimate net littoral drift rates: K of 0.053 based on literature (van Wellen et al. 2000) and a K value of 0.03 based on a modeled site (Seahurst Park; Anchor QEA 2012). For the south shoreline, the resulting estimated transport is 300 to 550 cubic yards per year and for the north shoreline (assuming no breakwater) the resulting estimated transport is from 80 to 130 cubic yards per year. Calculated transport for the south shoreline is large to the prevalence and size of median wave conditions from the south-southeast (151 to 180 degrees).

Table 8
Median Wave Conditions

Direction (degrees)	Wind Speed (miles per hour)	Depth (feet)	Fetch (miles)	Hs (feet)	Tp (seconds)	% time
South Shoreline						
61 to 90	5	20	0.2	0.06	0.5	1.74%
91 to 120	3	10	0.3	0.04	0.45	1.44%
121 to 150	8	5	0.6	0.16	0.83	9.77%
151 to 180	13	8	2.2	0.51	1.5	25.98%
North Shoreline (assuming no breakwater)						
271 to 300	3	50	2.5	0.09	0.7	0.72%
301 to 330	5	40	2.4	0.18	0.99	1.31%
331 to 360	9	60	5	0.51	1.65	9.19%
0 to 30	8	5	1.2	0.22	1.01	22.05%

Notes:
Hs is the significant wave height
Tp is the peak period

Based on comparison with reference sites, the net littoral transport rate for the south shoreline is most likely at or below the low end of the range estimated using the CERC formula (300 cubic yards per year). The net littoral rate for the northern shorelines is likely between 80 and 130 cubic yards per year.

Littoral drift is expected to vary along the project shoreline dependent on shoreline orientation and predominant wave direction. Drift rates can also vary over the short term depending on the variability in the average wave climate. Based on review of predominant wave/wind directions and shoreline orientation at the site, the largest drift rates are expected to be along the south and northwest shorelines (see Figure 6). Littoral drift rates are expected to be smaller, but measureable, along the northeast shoreline (beyond the influence of the breakwater to the east) and the southeast shoreline. Areas that are expected retain or accrete sediment would be the east shoreline and the area where the south and southeast shorelines meet.

5.2 Beach Profile Changes due to Storm Waves

The beach profile (transect perpendicular to the shoreline) will exhibit changes within the surf zone due to storm wave attack. These changes are due to movement of mixed sand and gravel materials by breaking waves up or down the slope. Typical profile changes for gravel or mixed sand/gravel beaches include the development of a trough at the still water level and mounding up of material landward of the still water level. Changes to the beach profile are highly dynamic, and should be expected to change often in response to wave conditions at the site.

One concern to the restoration effort is the layer thickness for placed materials. Beach profile changes during the storm event could thin this layer of material due to the creation of the trough feature. Empirical equations were developed by Jentsje W. van der Meer (1988) that can be used to estimate the depth of the trough (and other features) that could form as a result of large storm waves impacting the beach. These calculations were completed using a range of D50 gravel sizes based on the 20-year storm events (see Table 3). For all sediment sizes in the gravel range, over a range of initial beach slope, the depth of the trough was 16 to 18 inches from existing grade (a flat slope). Proposed restoration actions, or intertidal capping actions, should take into account the potential for beach profile changes when determining effective thickness of placed materials within the surf zone.

6 NEXT STEPS

The wave model developed as part of this project, and information provided in this report, will be used to inform 30% design of proposed restoration actions along the south, southeast, east, and northeast shorelines of the project site. Additional modeling of the 30% design for the northeast and east shorelines will be conducted (once the shoreline regrading concept is finalized) to verify that there are no adverse impacts to shorelines at Point Julia and to evaluate the potential need for a sediment retention structure to be included in the restoration design.

7 REFERENCES

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ATTACHMENT 1
SOUTH SHORELINE CUT-BACK FIGURES

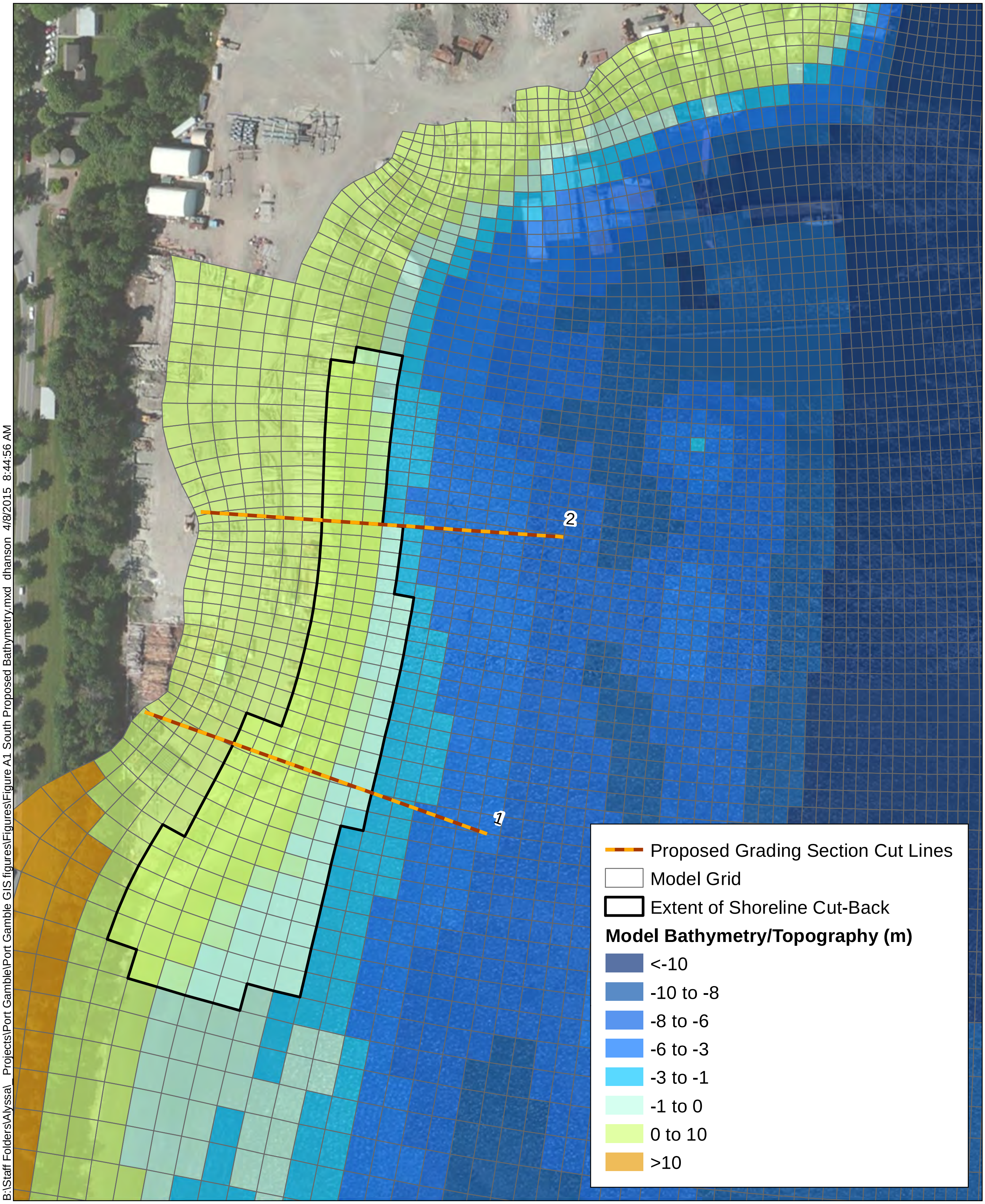

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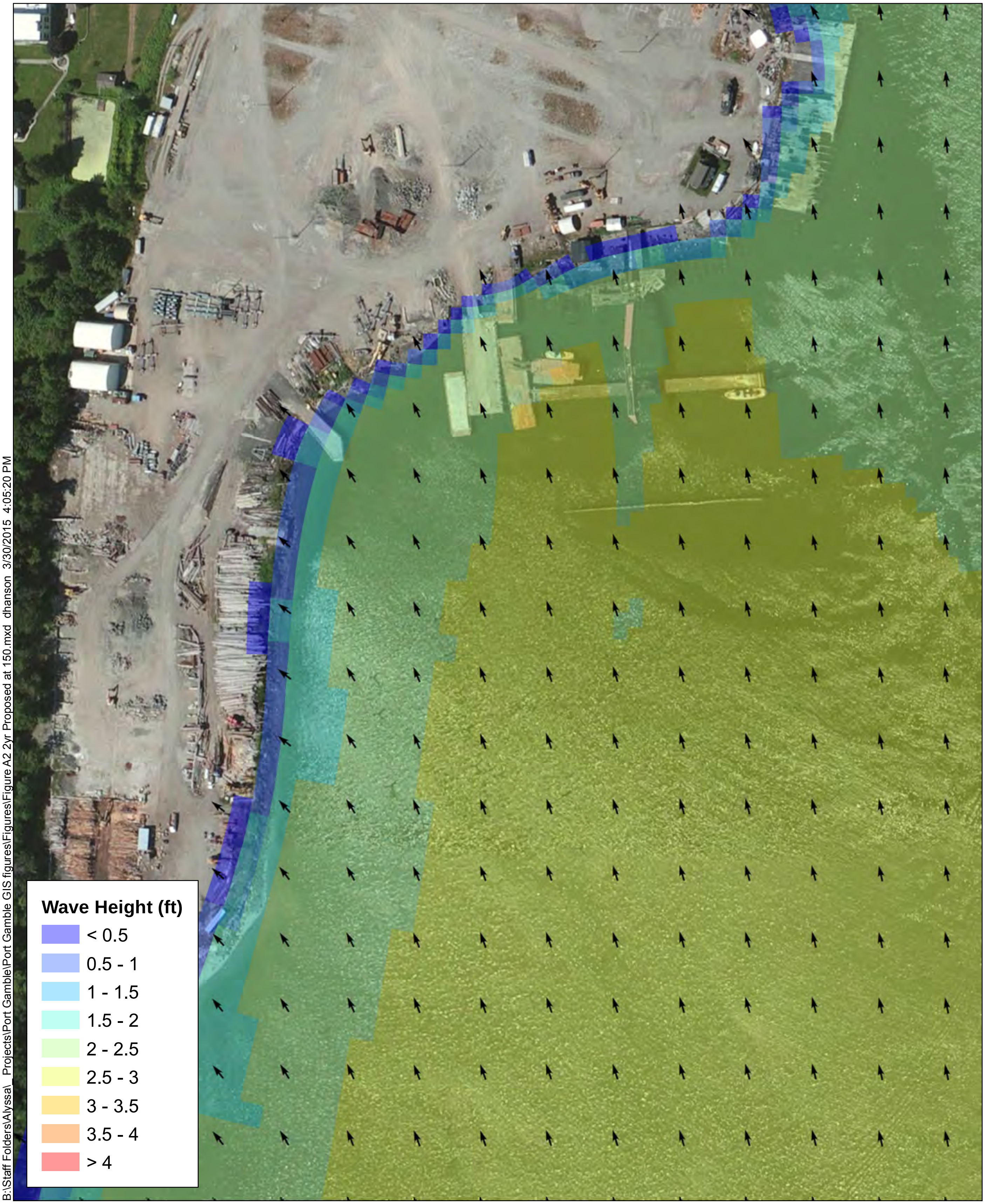
Figure A1
Benchmark Model Testing,
Model Grid for South Shoreline Cut-Back
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

ANCHOR QEA

Feet
0 110 220



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Feet

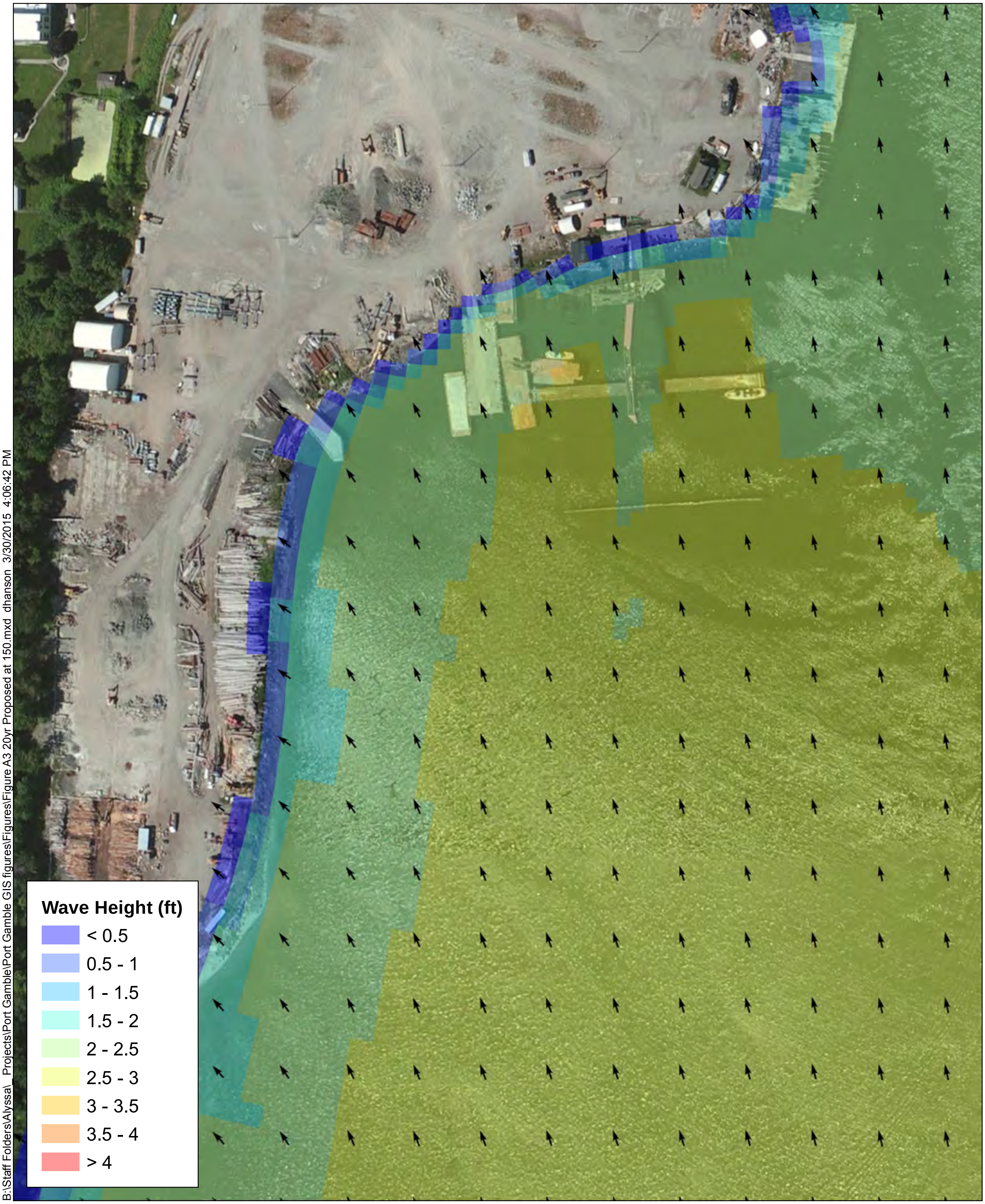
0 100 200

Figure A2

Run 14-Proposed 2-year MSL; 150 degree wind

Wave Modeling in Support of Restoration Design

Port Gamble Mill Site Restoration



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Feet
0 100 200

Figure A3
Run 16-Proposed 20-year MSL; 150 degree wind
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

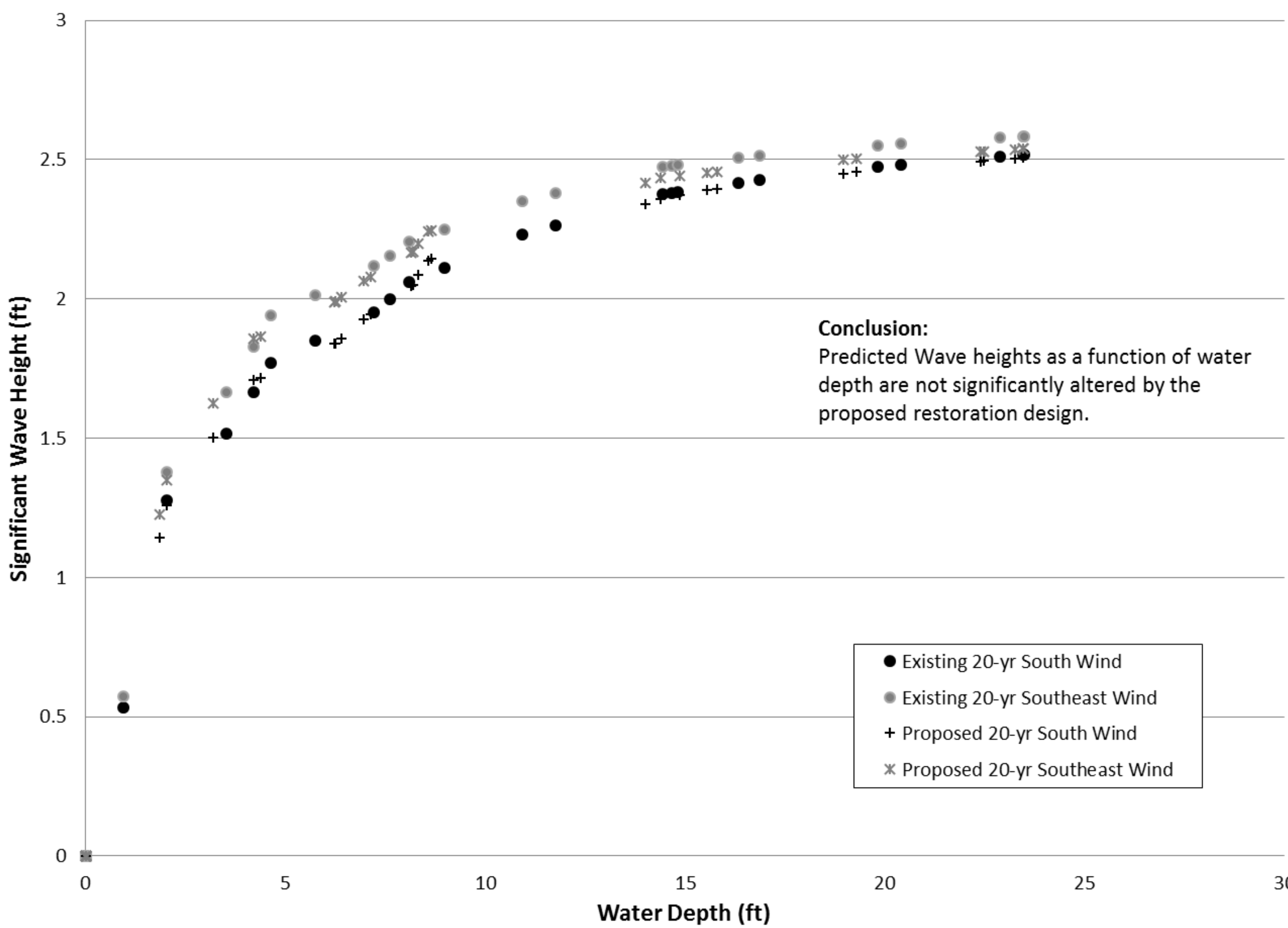

Figure A4
Cross Section 1, Wave Heights for Existing and Proposed South Shoreline Conditions
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

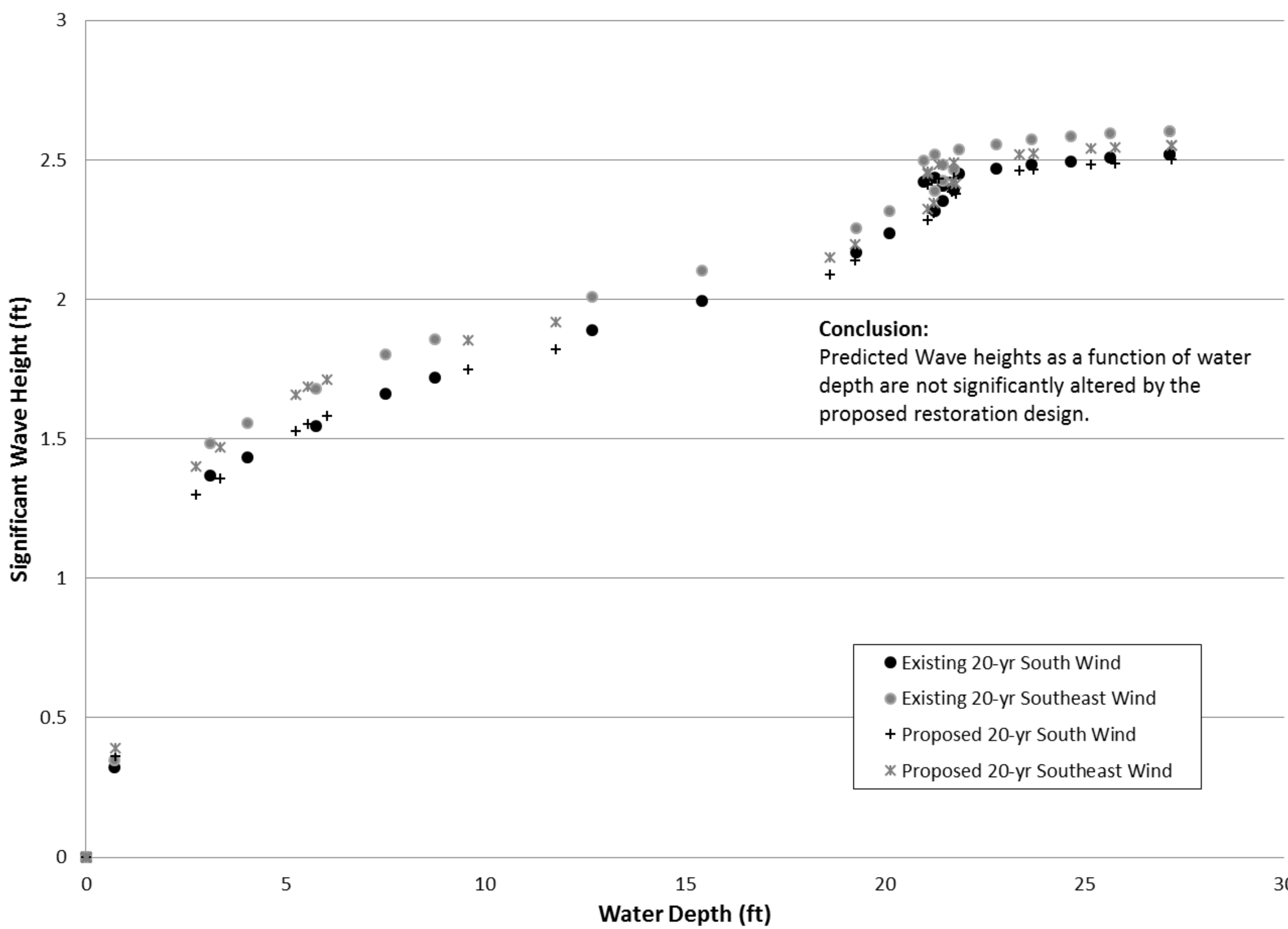

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Figure A5
Cross Section 2, Wave Heights for Existing and Proposed South Shoreline Conditions
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration

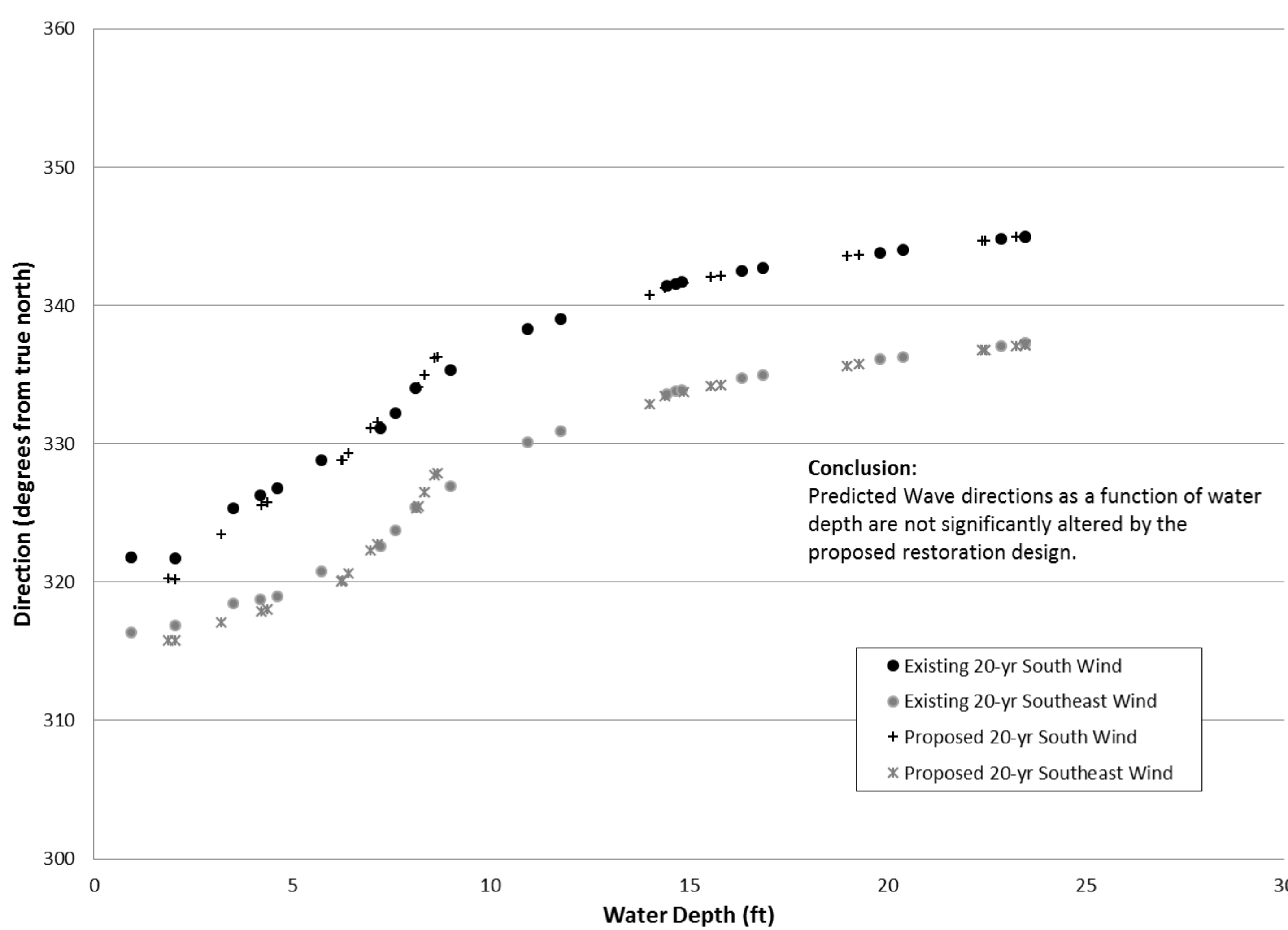

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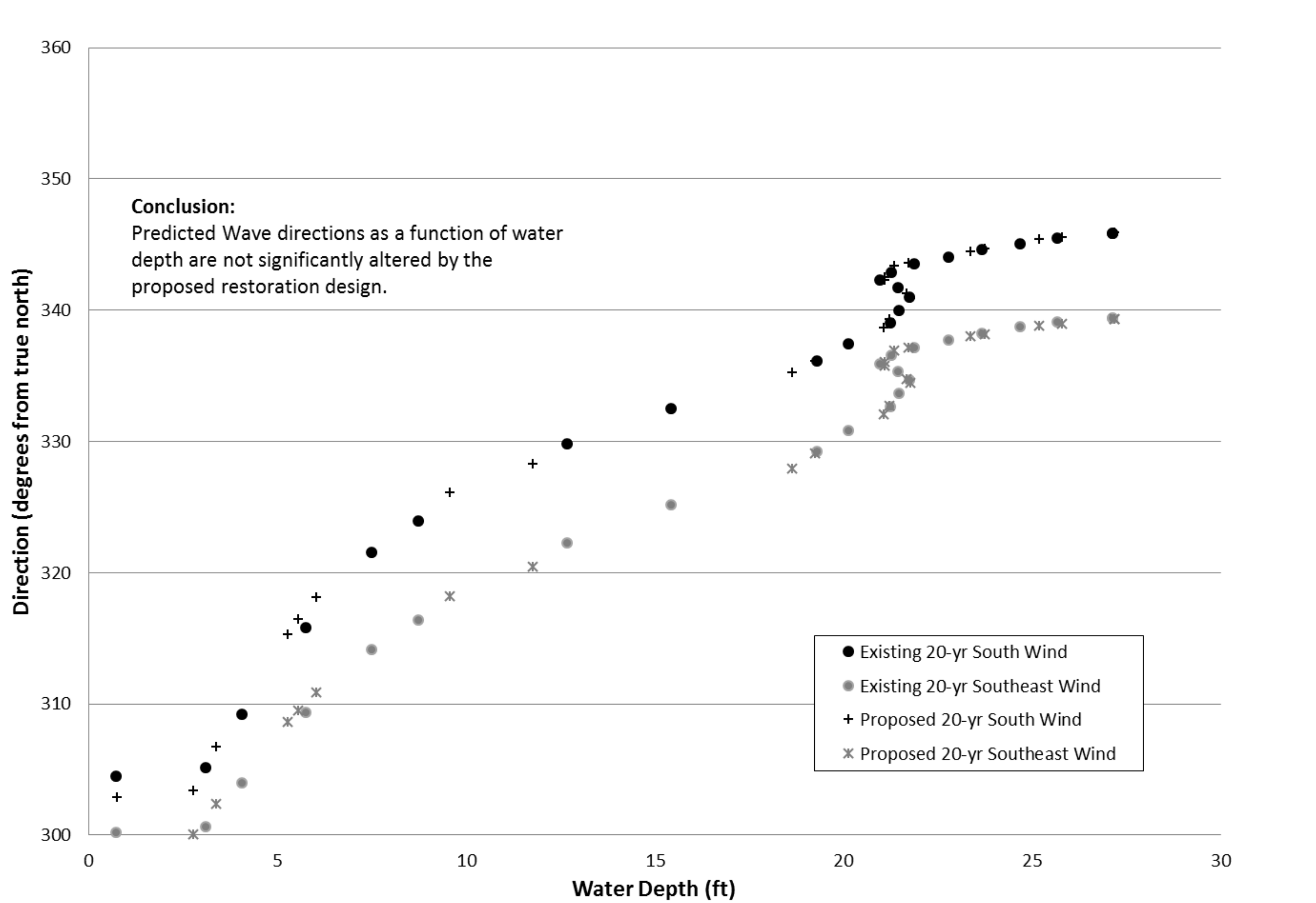
Figure A6
Cross Section 1, Wave Directions for Existing and Proposed South Shoreline Conditions
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration



I:\Projects\Port Gamble\2013-2015 Cleanup Design\EDR\Appendix D Coastal and Propwash Evaluations\Restoration Model Report\Figure A7.docx

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Figure A7
Cross Section 2, Wave Directions for Existing and Proposed South Shoreline Conditions
Wave Modeling in Support of Restoration Design
Port Gamble Mill Site Restoration